7 HYDROLOGIC AND HYDRAULIC MODEL

The New Jersey Department of Agriculture (NJDA) conducted hydrologic and hydraulic watershed modeling as part of the RSWMP. The NJDA developed a technical report which is included by reference and is available for public review. This section provides an overview of the modeling study and summary of the results. The study and modeling effort by NJDA also provided input to much of the watershed recommendations in Book 2.

Understanding of the hydrologic cycling of water through the watershed is an essential element in developing a regional stormwater management plan. Evaluation of pollutant sources and transport, the nature and extend of flooding and the impact of land use changes depend on an accurate representation of the watershed hydrology. Watershed hydrology is typically expressed in a “model” which seeks to represent the real world processes numerically. The model output provides the numeric and location data needed by decision makers and stakeholders. For example, emergency management personnel wish to know when (time) a stream will reach its peak flow rate, and how high (elevation) the peak elevation will be, how much flow (quantity) will occur at the peak and where (spatial location) will the peak arrive. Engineers need to know how much water (cubic feet or gallons per time interval) would be expect to flow through a culvert or over a spillway given a certain amount of rainfall. Hydrologic and hydraulic models attempt to quantify the stream system to provide necessary data.

NJDA has selected two primary modeling environments to depict stream flow and stormwater runoff quantity. Both models originate from the United States Army Corps of Engineers (ACOE) Hydrologic Engineering Center (HEC) and are in the public domain. These models are well known in the engineering community and have a high degree of reliability as well as flexibility for modeling various and complex scenarios.

For stream flow modeling, the River Analysis System (HEC-RAS or RAS) was used. This model uses physical field measurements of stream and floodplain cross sections to estimate flow values (rate, velocity, energy, water surface elevation) from one section to another based on the laws of conservation of energy. The model is calibrated by adjusting parameters (channel roughness for example) until model estimates provide an acceptable match to those measured in the field. For this model, the calibration parameters were high water elevations. NJDA used physical stream flow velocity measurements made at various depths along with water surface elevation records from recording gages to develop a series of depth-flow relationships later used in runoff modeling.

For watershed runoff modeling, the ACOE Hydrologic Modeling System (HMS) model was used to represent the land areas of the watershed. The basic form of this model is the numerical representation of a parcel of land, precipitation and start and stop time for computation. Each of the subareas of the Wreck Pond Brook watershed is represented as separate but connected areas within the model. These subareas are connected by stream reaches, impoundments or junctions. Model inputs consist of subwatershed size, runoff coefficients, time parameters and stream flow hydrograph information. In addition, lakes and ponds are represented in the model by describing the storage and discharge relationships of the impoundments as stormwater runoff is “routed” through them to the next downstream sub area.

In total, NJDA has developed eight HEC-RAS models – one for each subarea gage station and approximately twelve runoff models – a calibration and verification model for sub areas and certain combinations of subareas. The result of this modeling effort is a numerical depiction of the watershed in terms of land area, runoff parameters, time parameters, impoundment hydraulics and stream reach hydraulics which can be used to analyze future build-out, zoning changes, stream erosion, flooding and numerous other land use/watershed planning issues.

7.1 Model Data Inputs

7.1.1 Stream Gage Measurements and Channel Cross-Sections

In order to develop stream flow hydrographs for use in watershed model calibration, continuously recording water depth loggers were installed at the outlet point of each subwatershed as discussed in Section 3.3. The gage locations are shown on Figure 10. The loggers were set to take measurements at approximately 15 minute intervals. This interval was selected based on results interval trials and provided the best mix of detail while avoiding excessive file size.

A “staff gage” was also installed at each station which gave an instantaneous reading of water surface depth or elevation. USGS-type steel staff gages, incremented in 0.02 foot intervals were installed approximately on the same cross section as the recording gages. Staff gages were used as a means of “double checking” the values recorded by the loggers as well as correlating stream velocity measurements to transducer recorded data.

All instrumentation placement was surveyed by Najarian Associates using a combination of high-precision GPS and traditional surveying equipment so that stream water surface level could be converted to USGS elevation, NADA 83 feet. Najarian also surveyed stream channel cross-sections, bank to bank for use in the development of rating curves, discussed below.

7.1.2 Stream Flow Velocity and Rating Curves

In order to convert simple depth logger data to volumetric flow rates, rating curves were needed at each sub-watershed station. Rating curves are equations to calculate flow from stream depth or elevation. The rating curve is used to calculate flow from the data logger depth data.

Rating curves were developed by a combination of directly relating depth to stream flow measurements and by hydraulic modeling using the United States Army Corps of Engineers (USACOE) Hydraulic Engineering Center – River Assessment System (HEC-RAS) model. The model was used to compute flow rates that were beyond the ability of the investigators to measure directly in cases were flood flows were so great that direct measurement posed a safety hazard.

For direct measurements, water depth and velocity measurements were taken in the field and used with stream cross-sections to develop rating curves. Flow versus depth was plotted and the best-fit equation calculated.

Once a particular form of equation was selected that appeared to fit the data, the equation was then used to reproduce the computed rating curve flow data points and compared to the flows as measured in the field. In this way, the equations of best-fit curves can be evaluated as to their acceptability for reproducing flow values at both the low and high end of the range of stream flow depths recorded by the gage. In some cases, such as at station W5 (Bailey's Corner Road), the dramatic change in flow regime from in-channel to overbank or floodplain could not be satisfactorily modeled by a single equation. Although a best-fit line had the appearance of faithfully representing the data points graphically, the range of flows (three to several hundred cfs) included a standard deviation that was often in excess of the lower flow values themselves. In order to overcome the accuracy problem, separate equations were used for in-channel and over bank flow conditions.

Rating curves were based on several field measurements of flow. However, field measurements were limited by time and scope. The small size of the sub-watershed mean that relatively small changes in water depth have a substantial impact on flow. As discussed later herein, additional flow measurements, particularly on Hannabrand Brook, may provide better understanding of flow.

In order to utilize the HEC-RAS model, multiple stream cross sections are needed for the model to compute and balance energy losses from one section to the next. Investigators used direct field measurements of stream flow to calibrate the HEC-RAS model for in-channel flows, but used traditional "trial and error" methods to calibrate larger, out-of-bank flows. In order to do this, surveyed cross section data, combined with Geographic Information Systems Digital Elevation Model (DEM) data and stream flow logger data was used.

NJDA, with the assistance of Monmouth County Office of GIS (MCOOGIS), developed a procedure to combine DEM data to define the floodplain portion of the stream cross section, with more highly detailed survey data of the stream channel itself to produce a hybrid cross section model used in HEC-RAS. Using a combination of add-on software packages from Environmental Systems Research Institute (ESRI, Inc) and USACOE, MCOOGIS was able to create three dimensional sections or "slices" through the floodplain and through the channel for use in HEC-RAS modeling. This method avoids extensive surveying and associated costs, but the accuracy may be limited by the DEM resolution. In this case, the available resolution of about 1 foot was adequate since much of the hydraulic and hydrologic response in the watershed is governed by small nuances in topography.

7.2 Watershed Hydrology and Modeling

In order to develop a comprehensive understanding of the hydrologic system of the Wreck Pond Watershed, the overall drainage area was sub-divided into three sub-systems for analysis. These systems are:

- Hannabrand Brook stem (Stations W5 and W2),
- Wreck Pond Brook main stem (Stations W6, W9, W7, W1 and W3),
- Black Creek (Station W8).

Within each system, there are several subwatersheds, defined by a gaging station at the downstream end. Figure 10 and Appendix A provide detailed descriptions of the monitoring locations and subwatershed areas.

Each of these subbasins was evaluated independently to simplify model development, calibration and verification. The Black Creek subwatershed is distinctly different in that the drainage outlet does not coincide with the outlets from the Hannabrand and Wreck Pond Brook. These two subareas discharge quite close to each other just below Old Mill Road in Wall Township such that hydrographs from each stem can be added together to get a complete storm hydrograph for the entire watershed.

7.2.1 Model Inputs

The NRCS method utilizes a runoff coefficient to represent the effects of soil type and land use cover complex on the generation of stormwater. The runoff curve number method is well documented in literature and widely used by consultants for designing stormwater control systems. In simplified terms, a curve number (CN) is chosen from a table of values published by NRCS using combinations of soil type and land use cover. For this investigation, GIS data for land use and soils provided a CN for each polygon which were then aggregated into a weighted CN for each subdrainage area. Curve numbers by subdrainage area are tabulated in Table 24:

In order to use NRCS procedures, a timing factor must be computed which is used to apportion runoff volume over time, creating a “hydrograph”. Thus, lag time was a key calibration parameter. Appendix D provides detail on calculation of this parameter.

Table 24: Curve Numbers		
Subwatershed	**Station#**	**CN**
WPB-Waterford Glen	W1	68.7
Hannabrand Brk - Old Mill Culvert	W2	70
WPB-Old Mill Culvert	W3	69.5
HB-Bailey's Corner Rd	W5	62
WPB-Martins Rd	W6	61
WPB-Glendola Rd	W7	66
Black Creek	W8	67
WPB-Hurley's Pond Dam	W9	65

A specific unit hydrograph was developed by the firm of Dewberry-Davis (Dewberry) under contract with NJDA. The hydrograph was formatted to be used in the modeling to convert runoff volume depth to a runoff hydrograph through use of a Peak Rate Factor (PRF). The use of local unit hydrograph provides a better estimate of actual hydrograph shape than a general hydrograph. For example, the watershed unit hydrograph uses a PRF of 230, while the standardized PRF used in the NRCS methodology is 484. The relatively flat topography in this coastal plain watershed is better represented by the lower PRF. The DELMARVA Coastal Plain unit hydrograph uses a PRF of 280, which is comparable the Dewberry results. The best results were obtained using the Dewberry Unit Hydrograph.

Precipitation data were collected as noted in Section 3.1, from the Wall Township RISE Station. When data were not available from that station, the NJ Mesonet weather data were used.

7.2.2 Peak Flow Attenuation and Other Flow Factors

The Wreck Pond Watershed contains numerous small ponds and several large lakes all of which affect storm runoff by damping peak flows. Further attenuation is provided by the riverine buffers along the main stem streams which, when inundated, act as basins themselves, storing, trapping and releasing stormwater as it moves downstream.

Impoundment Modeling: A combination of field survey and GIS measurements were used to develop reservoir rating tables when data were not otherwise available. Existing reports were available for Old Mill Pond and Hurley Pond (Monmouth County Engineering Department and Hatch-Mott McDonald for Monmouth County, respectively) which provided rating table data used for reservoir routing. Rating table data were developed for Albert's Pond, Osborne Pond, Mc Dowel Pond (18th Ave), Fairway Mews detention basins and the Spring Lake Golf Course impoundments.

Reach Routing. Investigators found that while impoundment routing was necessary for model development, impoundments alone could not account for all hydrograph peak attenuation or time lag. Therefore, several stream channel "reaches" (as defined by HMS) were modeled using the Muskingum routing method. .

Diversions: In some cases, peak flows and volumes were found to be lower downstream than upstream. This was particularly evident for some storm events within the Hannabrand stream (areas W5 and W2). Investigators conducted a detailed stream survey of the lower reaches of the Hannabrand Brook to determine if the data were 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 allowed flows to be diverted out of the channel. Investigators concluded that these characteristics occur randomly in time and location thus yielding variations in stream flow events.

A further examination of stream flow data during selected modeling events indicate that for this subwatershed, base flow conditions at the upper and lower gage stations were being recorded accurately indicating gages were functioning correctly. Diversion was incorporated into a calibration event for the Hannabrand Brook subwatershed. Computed results compared favorably with observed hydrographs. As noted above, however, additional rating curve data may also be needed to improve flow analysis for Hannabrand Brook.

Base Flow: Constant base flow values were set at levels determined from stream gage records which showed constant flow values before each storm event. Using this method, computed hydrographs compared very well with observed hydrographs at the receding hydrograph limb, confirming the use of the "constant monthly" method.

7.2.3 Storm Event Descriptions

Precipitation and stream gage data were examined to find appropriate storms to model. The following storms were selected for the various sub-models.

March 28, 2005. Precipitation data was procured from the New Jersey Mesonet weather station network gage located in Sea Girt as the Wall Township RISE network gage was not online at the time. 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 may 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.

June 24, 2006. The storm event of June 24 was characterized by heavy rains totaling about 2 inches for the 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 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.

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 is somewhat smaller than desirable however the gage provided a sufficient distribution pattern and the stream gage response was adequate.

April 12, 2007. This event totaled about 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 should be saturated and base flow in streams should be elevated.

7.3 Model Calibration and Verification

The HEC-RAS model produces a computed hydrograph and calculates runoff depth. The shape and peak flow of the hydrograph along with the computed runoff depth are compared with the actual flow hydrograph and runoff depth. The model parameters are adjusted so that the computed and actual hydrographs and runoff depth match. While peak flow can be affected by changes in curve number, lag time, unit hydrograph, drainage area and even computation interval used by HMS, runoff depth has only one primary variable (curve number).

Once the model parameters produced results that reasonably match the actual hydrograph in shape and peak and the runoff depth, the model is considered to be calibrated. The model is run again with a different storm and if this produces the expected result, the model is considered to be verified. If the model parameters are correct, inputting a real storm event of any type should approximate the gauged hydrograph of that event. In reality, physical conditions of the watershed are not the same for each event – antecedent soil moisture, distribution of the precipitation, physical changes in the watershed (blockages, debris, debris removal, plant growth etc.). Therefore, calibration and verification event parameters are presented as a range of values relative to computed values.

The sub-basin models were successfully calibrated and verified for two storms each. The June 2006 storm was used for both the Hannabrand and Wreck Pond Brook sub-models. Figure 14 shows the results of that calibration run.

7.4 Hydrologic Model Limitations and Results

Any modeling effort is a representation of actual natural conditions. Model limitations are typical of any modeling effort.

7.4.1 Limitations

NJDA has attempted to include as much relevant information as possible in the RAS, HMS and other computer models to obtain reasonable accuracy within given limitations of time and data. To this extent, the following limitations are identified both in model development and use.

(1) Models are planning tools, and not intended for design. Watershed conditions change, and may change rapidly. These models examine the Wreck Pond Watershed during a specific period of time and model output reflects the conditions existing at that time. The models may not be used under future conditions for the design of stormwater management structures without undergoing detailed examination and updating of model parameters. Specific numerical output should not be used to set or establish regulatory limitations such as discharge or effluent limits but as a general planning tool.

(2) Model output is an approximation of real world conditions and results should be evaluated in context with other data, such as high water marks, photographs, measurements made by other parties etc.

(3) Watershed models (HMS) are not intended to depict individual site conditions. The scope and scale of the HMS models are too larger to generally model individual land-use changes.

(4) Pond routing details were developed from GIS measurements and were not surveyed. Routing output therefore is approximate and should be verified by actual surveys of weir elevations, measurements of surface areas and elevations etc.

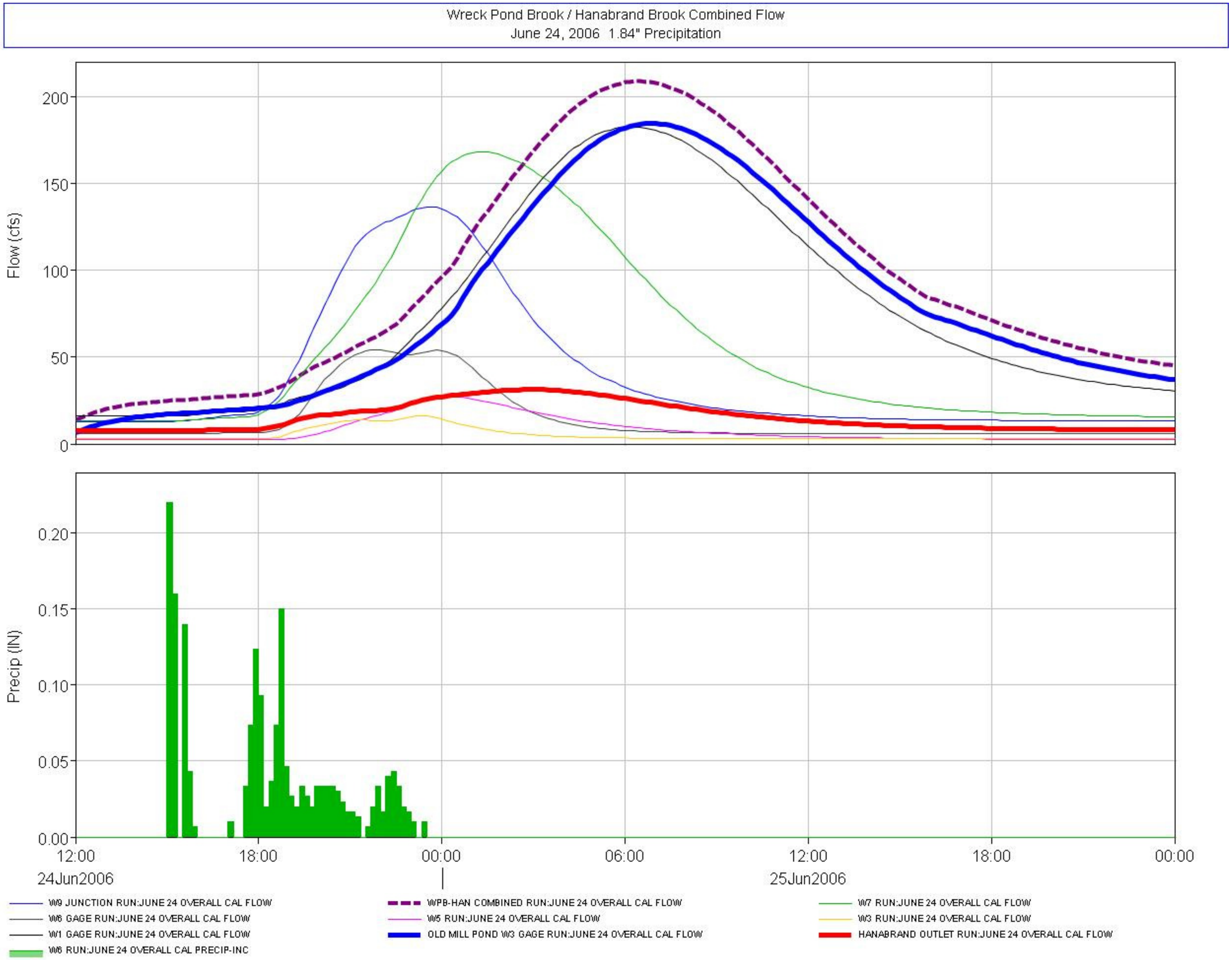


Figure 14: Wreck Pond Brook Watershed Study
NJDA Model Calibration Result – June 2006 Storm Event

(5) Rating curves for streams at gage station locations should be updated with more detailed field surveys of the channel for future hydrograph development. In addition, additional flow measurements during a variety of conditions, particularly in the Hannabrand Brook watershed, would increase the accuracy of the model. Due to the sandy/gravel soils, the cross sections were observed to have changed due to bedload movement during the study. Therefore, accuracy of the cross sections for future analysis cannot be assured.

7.4.2 Model Results

NJDA successfully developed a hydrologic/hydraulic model of the Wreck Pond Brook watershed to Old Mill Road using the USACE HEC RAS and HMS models. The following conclusions were obtained from the modeling. In addition, the results of NJDA data collection, field surveys and modeling were used to provide information for other sections of this Plan and to develop many of the recommendations in Book 2.

(1) Stream channel modeling with HEC RAS can be cost-effectively performed using a process which combines a physical survey of the channel cross section (bank to bank) with flood plain sections derived from high resolution digital terrain models (DTM) to form a composite cross section.

(2) Stream flow (HEC RAS) models indicate that overbank conditions along stream corridors contribute significantly to flood peak mitigation by storing excess runoff and slowly releasing it. Storage volume is highly dependent on seasonal variations of vegetation cover in the floodplain. The golf courses also provide significant flood storage.

(3) Hydrograph shape from gaged data reflects a fairly quick rise and fall of stream elevation for non-flood events. . This reflects the porous nature of the soils in the upper watershed (sands and gravels), which do not store incident rainfall for very long. Consequently, streams experience a fairly rapid discharge to baseflow during and immediately after the rain event.

(4) Stream floodplains appear to provide attenuation of larger storm events more so than do the many large man-made impoundments. Consequently, man-made impoundments tend to affect hydrographs of small (water quality sized) storm evens. Model output for the larger lakes, such as Hurley's Pond, Osborne's Pond and Albert's Pond show relatively little peak flow attenuation for larger events. This suggests that the major reservoirs in the watershed would require outlet modification and possibly dredging in order to increase their effectiveness with attenuation of larger storm events. Additional data, including outlet flow data, would be required to evaluate the need for and anticipated results of such modifications.

(5) The Lower Monmouth Dimensionless Unit Hydrograph was shown to be necessary for accurate reproduction of measured peak flows. The DELMARVA dimensionless hydrograph also represented gaged data fairly well. The use of the Standard (484 peak rate factor) hydrograph resulted in computed peak flows that were significantly in excess of measured gage data and could not be fit with modification of Curve Numbers or Lag time.

(6) Bankfull conditions in most streams in the watershed were achieved at rainfall depths less than 2 inches. This is less than or just equal to the NRCS one-year storm event of 2.9 inches in 24 hours for Monmouth County.

(7) Floodplain storage was randomly affected by debris jams across the channel resulting in bank overflow for small storm events. While random occurrences of debris jams made model consistency difficult at times, their occurrence prompted the recommendation contained below and elsewhere to examine select floodplain areas for possible use as regional detention facilities.

(8) The Spring Lake and Mews Golf Clubs serve as regional flood control and sediment traps for the upper Black Creek watershed due to extensive "water features" found on both sites. The combined storage at various peak basin elevations is approximately 10 acre-feet +/- and contributes to a reduction in peak discharge by about 35% at the Mews basins and a further reduction of about 10% at the Spring Lake Golf Club. Overall reduction is about 17% for small storms and about 3% for the statistical 100 year event. This indicates that their primary benefit to the Wreck Pond watershed is for the control of water quality events more than flood events. Model results also indicate that the primary source of runoff in the W8 subwatershed comes from the residential area to the north of the Mews golf and residential area. Discharge is piped via a 60" RCP under the Mews site and discharges directly into the upper reaches of the Spring Lake Golf Club water features. Time travel in the culvert and land use conditions in the upper portion of the subwatershed are the primary controllers of runoff peak and volume at the discharge point at the Rt. 71 culvert.

(14) The 100 year statistical storm event of 9.0 inches yields approximately 1900 cfs at peak discharge to Wreck Pond from the 13 square mile watershed. This peak value also may be impacted by the degree of flood plain storage available (for example, flows could be higher if not diverted upstream to the floodplain). At normal high tide elevation, a very rough estimate of the maximum discharge from the 7 foot concrete pipe is roughly 600 cfs and requires a depth in the pond (head on the pipe) of about 13 feet, effectively flooding much of the lower watershed if additional measures are not taken to empty the Pond during a storm event of that magnitude. It is essential to note that the exact conditions of the pipe lining and entrance conditions were unknown and can affect the discharge capacity of the culvert. Thus, additional flow analysis is needed at the outfall. The model does not account

for rising and falling of ocean tides, which will dynamically reduce or increase culvert capacity. Pond routing therefore assumed a static high tide condition occurring during the peak discharge into Wreck Pond.

This model will provide an ongoing tool for use in updating this model. The hydrologic model can also be extended to provide analysis of future conditions and other proposed watershed modifications.

8 WATERSHED SWMM MODEL

The County modeling efforts discussed in Section 7 provide a detailed analysis of the hydraulics and hydrology of the watershed. As part of the Borough of Spring Lake's EPA funded Wreck Pond Environmental Study, a watershed pollution budget model was developed to investigate the non-point sources of pollutants into Wreck Pond. Designed to present pollutant loading generation values for broader regions of the watershed, the model provided loadings for the upper and lower regions of the Wreck Pond Brook, the Hannabrand Brook basin, and areas in the vicinity of Wreck Pond.

The pollution budget model was expanded for the RSWMP to provide greater resolution on watershed pollutant generation. Data supplied by the MCOOGIS and NJDA allowed for enhanced model detail including analysis of the seven sub-watersheds above Old Mill Road and Black Creek. The results of this pollution budget model will complement the hydrologic and hydraulic modeling efforts of the NJDA. Additionally, the model provides flow and water quality analysis on the portion of the watershed downstream of Old Mill Road, where the NJDA model terminates, thus complementing NJDA's watershed model.

The watershed model was developed using the EPA's Stormwater Management Model (SWMM). SWMM is a comprehensive computer model designed to analyze runoff from urban watersheds. Specifically, PCSWMM was utilized, a version of SWMM developed by the Computational Hydraulics Institute, which incorporates the basic SWMM engine and includes additional interface options.

SWMM's Runoff and Transport Modules were utilized for this model. The Runoff Module uses basic rainfall and watershed data to generate the quantity and quality of stormwater flowing off the land. The Transport Module use stream channel data, along with the Runoff Module output, to route the stormwater through the watershed.

As discussed in Section 7, NJDA developed a hydrologic and hydraulic model of the upper watershed, west of Route 71. At the time the SWMM model was developed, the hydrologic model was not completed. The NJDA 's extensive in-field water level data located at each of the eight (8) sub-basins within the watershed above Old Mill Road were used to calibrate the hydraulic portion of the SWMM model. For the Wreck Pond direct watershed, watershed characteristics were used to calculate flow, along with data collected by NA for the Borough of Spring Lake's Wreck Pond Environmental Study. Because flows generated within the model drive the transport of land use generated pollutant loads, calibration of a pollution budget model to flow data is imperative.

8.1 Model Input and Flow Calibration

The GIS data for the watershed, available from Monmouth County and the NJDEP, provided much of the necessary data for model development. The data included watershed features such as sub-watershed boundaries, streams, lakes, topography, land use, and soils. Other required inputs were evaporation data, pollutant build-up wash-off rates and rainfall hyetographs. Pollutant build-up and wash-off data were taken from scientific literature and similar studies performed within the State.

Available calibration data for this model included the water level collected by the NJDA and water quality data collected by Najarian Associates (NA). Storm event water quality storm sampling was collected by NA for two storm events at Wreck Pond Brook at Old Mill (W3), Wreck Pond Brook at Glendola Road (W7) and Hannabrand Brook at Old Mill (W2). Details and results of the water quality monitoring at these stations is provided in Section 10.

The watershed model was set-up using the Runoff and Transport Modules of the SWMM program. Within the Runoff Module, each sub-watershed was input as an individual drainage basin. Flow quantities and land-use generated pollutants were then transferred to the Transport Module, where flows and water quality constituents for each sub-watershed were routed through specific stream sections. Ponds were modeled as internal storage elements within the Transport Module using available data or assumptions when data were not available.

8.1.1 Tributary Watersheds

Flows were calculated from water level data using the rating curves developed by NJDA with input from NA as discussed in Sections 3.3 and 7.1.2. Review of the flow data generated some unexpected results. Soils and land use characteristics of Wreck Pond's sub-watersheds are generally similar. Thus, it was expected that the storm-generated flow per unit area for each sub-watershed would be within a well-defined range. Further, in most cases, flow increases in a stream in the downstream direction unless the flow is interrupted by a discharge point or an impoundment. However, particularly on Hannabrand Brook, storm-generated flow as calculated from the rating curves decreased in the downstream direction. Further, significant flow volume was lost downstream on this Brook and flow per unit area was lower at the downstream station, W2 than at other stations in the study area.

To investigate this further, flow per unit area was computed for the sub-watersheds in the study as determined from the water depth data and the rating curves for the storm event of June 27, 2005. The calculated average daily flow was divided by sub-watershed area to provide an average daily flow per unit area. These unit flows were compared with the mean daily flow per unit area for that date from the USGS gaging station for the Jumping Brook in Neptune. The Jumping Brook watershed unit flow for this storm was in the range of the study sub-watershed value, although slightly lower than most. However, the flow per unit area for Hannabrand Brook at Old Mill Road (W2) and Wreck Pond Brook at Old Mill Road (W3) were significantly out of range. The Hannabrand Brook at the downstream station average daily flows was lower than expected. While the unit flow at upstream station on Hannabrand Brook, W5, was within 20-25% of the Jumping Brook flow, the flow at W2 was only 40-50% of the Jumping Brook unit flow.

On Wreck Pond Brook, the upstream Glendola Road station was within 10-20% of the Jumping Brook unit flows. However, downstream at Old Mill Road, flows were 40 to 60% higher.

The combination of higher than expected flows for Wreck Pond Brook and lower than expected flows for Hannabrand Brook at the Old Mill Road stations, suggests that some of the Hannabrand Brook flows may be diverted to Wreck Pond Brook upstream of the conjunction of these streams. As can be seen on Figure 10, these two stations are in close proximity. Thus, a further analysis was conducted that combined the flows for both streams at Old Mill Road. Using the total watershed area for both streams, the calculated unit flow is lower than for WPB alone, but it is still 15-26% higher than the unit flow for the USGS station.

Thus, the observed water depth data as converted to flow at Hannabrand Brook at Old Mill Road and Wreck Pond at Old Mill Road were not suitable for model calibration purposes for the hydraulic portion of the SWMM model. As noted, the NJDA model had not been completed when this model was developed. The NJDA modeling work later speculated that lower flows downstream on the Hannabrand Brook at W2 were due, in part, to the presence of debris dams along the stream that forced channel flows into the floodplains at flows lower than those that would typically cause flooding. In addition, NJDA noted that shifting bed load at some stations may have caused errors in the depth gage reading. These conditions are not permanent or consistent conditions within the streams. Thus, actual flows at W2 may be higher than predicted using the rating curves.

Since the SWMM model is primarily used for evaluation of generation and transport of water quality constituents herein, development of the model without consideration of redirected flow into the flood plain provides a more conservative estimate of pollutant generation within the Hannabrand Brook system. As flow in Wreck Pond Brook is still larger, the pollutant generation analysis will provide accurate estimates of the relative contributions of that watershed as well.

The flows at further upstream stations, Bailey's Corner Road (W5) and Waterford Glen (W7), respectively, were within range and used in the flow calibration process. The flows were then propagated downstream using the model. The modeled flows at Old Mill Road were compared to the data for W2 and W3 to ensure the flows were within an expected range. However, detailed flow calibration was not carried at those stations. Additional data may be needed to completely analyze the flow regime.

The flows herein compare to the NJDA modeling effort, with some exceptions noted above. As discussed in Section 7, the flow calibration process by NJDA focused primarily on storm events, while the SWMM modeling evaluated both storm and non-storm flows. The primary purpose of the SWMM model is to evaluate stormwater generated pollutant loading, which occurs at a range of flow events, while the NJDA was particularly interested in storm events related to flooding.

Further, the data loggers were not always operational, limiting available calibration periods. NJDA modeled the Hannabrand and Wreck Pond Brook systems separately and thus was able to use different storms to calibrate and verify their model for each basin. This SWMM model was developed as one model for Wreck Pond and Hannabrand Brooks. Thus, data loggers for both systems had to be operational, limiting the available flow calibration period. For these reasons, the process and results between the two models differ.

Further details of the flow calibration process, including storms selected, are provided in Appendix E. The calibration plots provided in Appendix E provide results for the summer of 2005. The flow results were validated using two other storms.

The Black Creek watershed flow was calibrated to the County station at Route 71. This subwatershed had to be calibrated separately. The data provided from the County data logger for this station (W8) was only available for 2006 while reliable data were not available at all the other stations in 2006. In addition, calibration issues arose at this station as flows peaked very quickly and connected impervious had to be increased to account for these flows. The flow per unit area appeared within range, although other information from the watershed suggests that the golf course acts to detain flow.

Flows here may have been somewhat overestimated by the direct impact of runoff from Route 71. No water quality data were available for this station.

8.1.2 Wreck Pond Direct Watershed

The eastern end of the watershed is considered the Wreck Pond direct sub-watershed. This includes the reach of Wreck Pond Brook from the station at Old Mill Road to the Pond, as well as the areas that drain directly to the Pond through stormwater outfalls. The sub-watershed was divided into three sections: Spring Lake and its contributing sub-catchments, major sub-catchments draining into Black Creek and major sub-catchments draining into the Wreck Pond Brook. Specifics regarding drainage areas, slopes and land use were input into the new model segment of the Wreck Pond SWMM Model. Flow data were not available on this portion of the stream, limiting the calibration.

8.2 Water Quality Calibration

Once the hydraulic and hydrologic portion of the model was complete, water quality calibration was conducted. The model was calibrated with the storm event water quality monitoring data collected by NA for the Borough study.

Runoff and Transport Modules of the SWMM program simulated the generation, transport and the fate of all contaminants of concern. Within the Runoff Module, pollutant build up on the watershed surface and are then washed off during storm events. The pollutant buildup occurs on a land use specific basis, i.e. pollutant generation varies by land use types. During a storm event, these pollutants are washed off the watershed surface and are then routed through the watershed streams and ponds via the Transport Module. The transported pollutants provide the calibration concentrations and loads.

The water quality calibration stations were Upper Wreck Pond Brook (at Glendola Road, W7); Lower Wreck Pond Brook (at Old Mill Road W3) and Hannabrand Brook (at Old Mill Road W2). Model parameters for the water quality calibration included land use, area, flow and pollutant buildup and washoff rates. As noted, land use was based on the Monmouth County GIS land use layer for 2006 land use and the pollutant rates were taken from scientific literature and available State studies. For the Wreck Pond direct subwatershed, wash-off coefficients developed for the outfall pipe sampling modeling of the Borough's Wreck Pond Environmental Study were used to develop area weighted averages of wash-off coefficients.

The water quality portion of the model was calibrated for total suspended solids (TSS), fecal coliform (FC), total nitrogen (TN) and total phosphorous (TP). The SWMM model generates pollutographs of water quality constituents (graphs of concentration vs. time) and estimate of total loads for the modeled period. The water quality model was calibrated for the October 17-18, 2006 storm event for which data was collected as discussed in Section 10 of this report. The calibration process compared the simulated constituent concentrations over time to the observed concentrations and the calibration parameters were adjusted until a good fit was obtained. The other storm event (September 2006) that was monitored did not provide sufficient data for validation although the available data was compared to the calibration data for consistency. Thus, the water quality component was calibrated but not verified. Further information is provided in Appendix E.

The model was generally able to match the water quality data to an acceptable level (Appendix E). However, the model over-predicted concentrations of certain parameters for lower Wreck Pond Brook (W3). Based on an assessment of the October 2006 storm sampling data, there is a significant mass loss of suspended solids and other pollutants on Wreck Pond Brook between the Glendola Road (W7) and Old Mill (W3) stations. This result is likely due to in-stream ponds located between these two stations. These ponds act to retain the flows along Wreck Pond Brook, allowing suspended sediments and associated chemical constituents to settle. This effect is also apparent in a review of the data from the September 2006 storm sampling. The peak TSS concentrations during the portion of the storm sampled for Wreck Pond Brook were 23 mg/l at Glendola Road (W7) and 2.3 mg/l at Old Mill Road (W3). Review of the annual monitoring data indicates that the base flow concentrations do not show the same pattern, suggesting this occurs at higher flows.

The model's Transport Module was not designed to simulate such extensive in-pond processes. Thus, a post-processing analysis was conducted. The observed water quality field data were coupled with the simulated SWMM flows to determine the "actual" in-stream load for the calibration storm. This provides an adjustment factor to account for settling within the watershed ponds. This comparison shows that Wreck Pond Brook loses about 40% of the TP load and 35% of its total TSS load between the Glendola Road and Old Mill Road stations.

The model is calibrated for Wreck Pond Brook and Hannabrand Brook up to Old Mill Road. Water quality data was not available for the calibration of the Black Creek sub-watershed. In addition, literature values were lacking for generation of pollutants from golf courses, which make up much of this watershed. The model was run for this sub-watershed using the calibrated model parameters. Due to the lack of sub-watershed specific data, these results must be considered a rough estimate.

The Wreck Pond-direct drainage sub-watershed consists of the reach of Wreck Pond Brook from Old Mill Road to the Pond and the watershed area directly adjacent to the Pond. No in-stream flow or water quality data were available for this sub-watershed, although the outfall pipe sampling provided flow and water quality data. Therefore, this sub-watershed is partially calibrated.

8.3 Model Predictive Runs

The watershed model was run under three scenarios for existing watershed conditions: the NJDEP 2-Year storm, a Dry Year (2001) and a Wet Year (1996). Loads generated from the land surface were calculated for all nine sub-watersheds. Calibration was done for transported loads at three stations as previously noted: W7, W3 and W2. Transported loads were also calculated for the Black Creek watershed (W8) and for Wreck Pond as a whole.

The pollution generation rate is on a per acre basis. Thus, the total load from a watershed depends on both the pollutant loading rate, the rainfall conditions, and the watershed area. As expected, the Upper Wreck Pond Brook watershed produces the largest loads for all pollutants due to its larger size. Figures 15 and 16 provide the watershed loads for TSS and total nitrogen for each model scenario for the sub-watersheds.

Watershed (runoff) loadings are conveyed downstream by the Transport Module and calculated for five sub-watersheds. Results of these model runs are depicted in Table 25 and Figures 15 and 16. The presented loadings are produced after the flows have been routed through streams and ponds, and therefore give the cumulative resultant loads at the exit point of each sub-watershed. For WPB-OM, the post-model processing (discussed above) was used to calculate the final transported loads.

These results are considered estimates as the model was not validated and the pollutant generation factors are those from the literature. The relative estimates provide valuable information about the generation of pollutants from the land surface and transport through the watershed. In particular, the result for the Black Creek and Wreck Pond direct sub-catchment are not calibrated to water quality data. The results for this

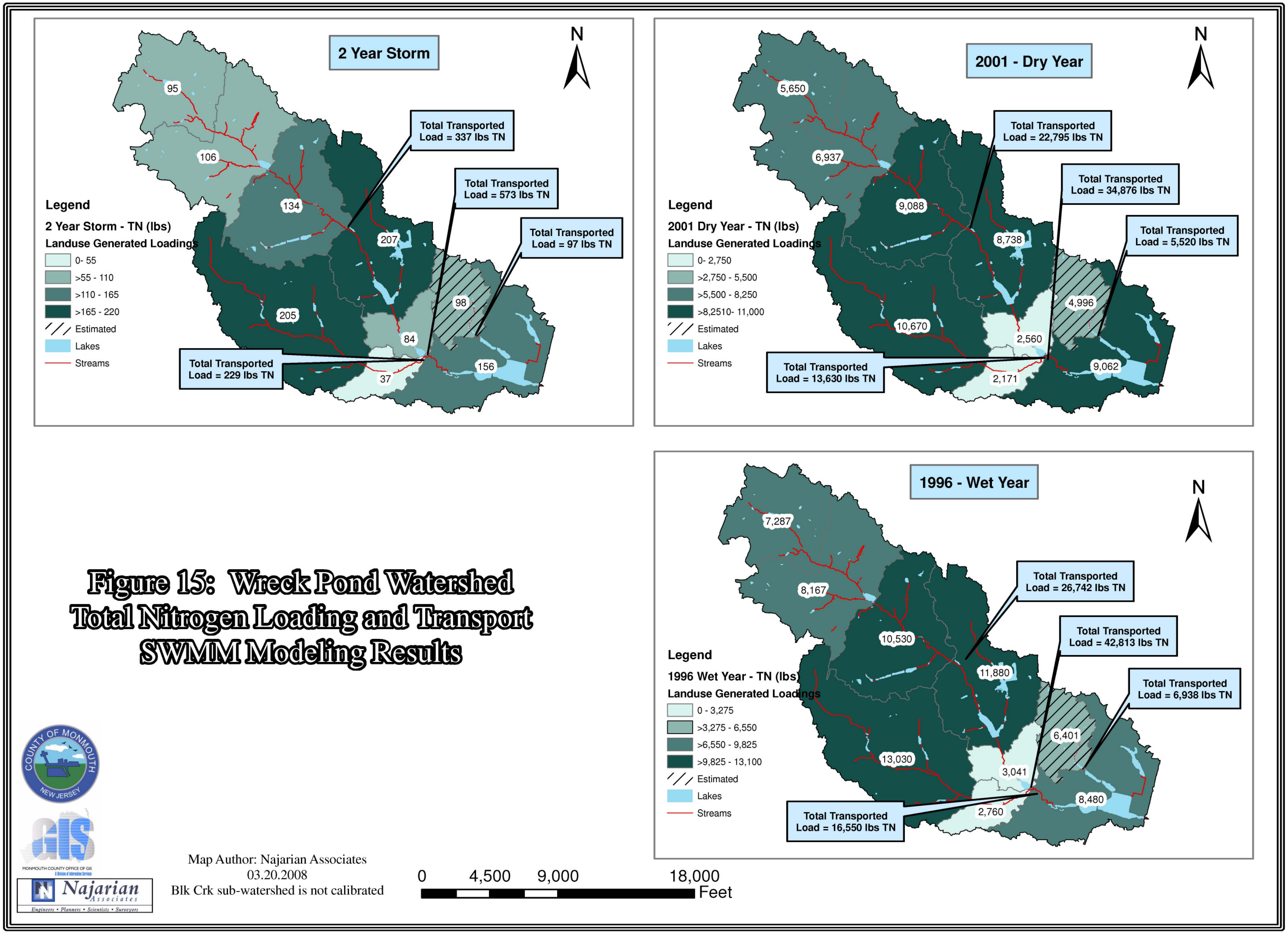
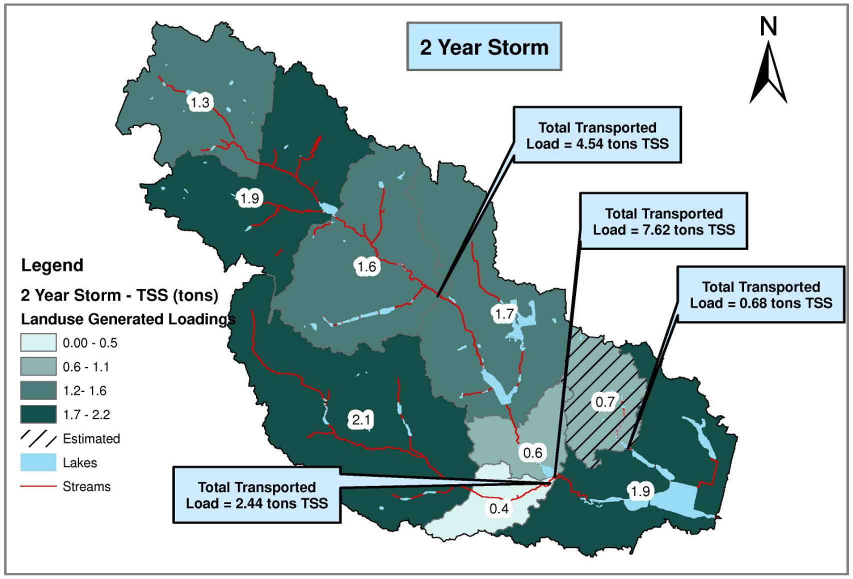


Figure 15: Wreck Pond Watershed Total Nitrogen Loading and Transport SWMM Modeling Results



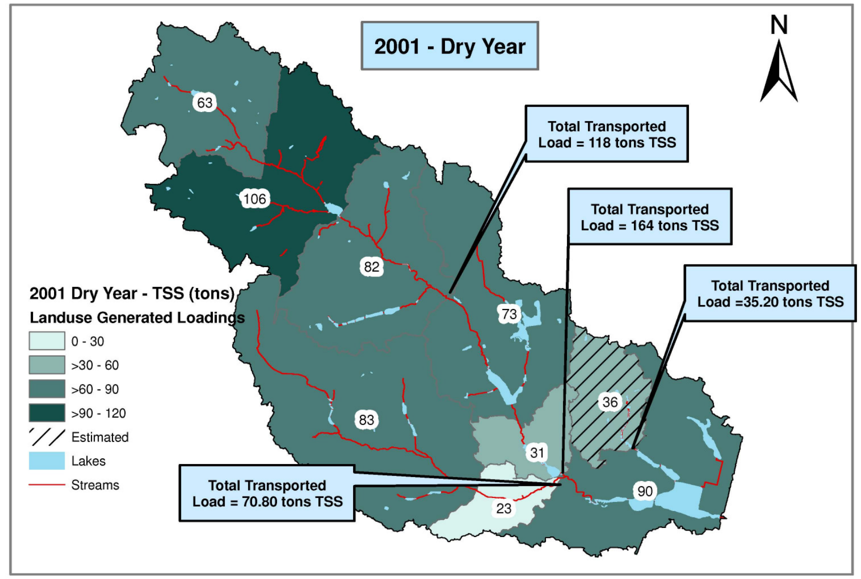
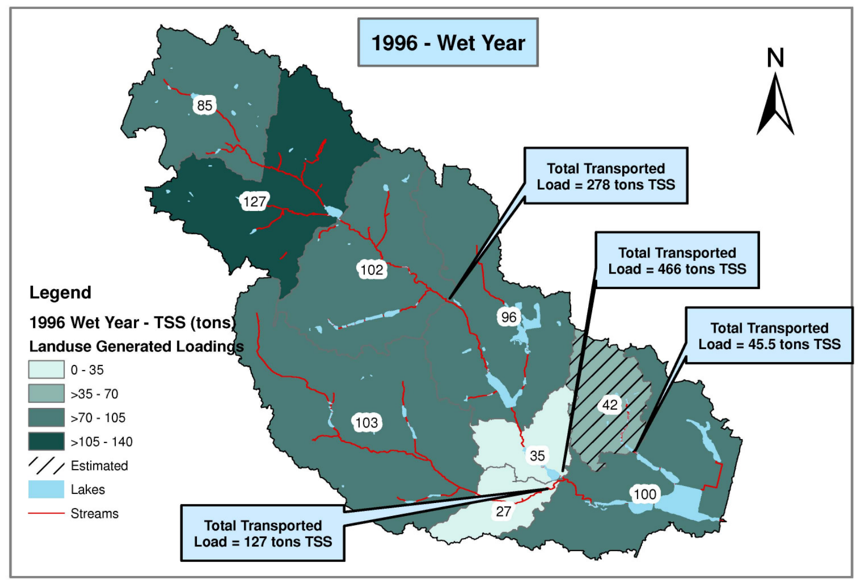


Figure 16: Wreck Pond Brook Watershed Total Suspended Solids Loading and Transport SWMM Modeling Results



Map Author: Najarian Associates
03.20.2008
Blk Crk not calibrated

0 4,350 8,700 17,400 Feet

Table 25: Model Results - Overall Pond Loadings

	FLOW (ft^3*10^6)		**TN (lb)**	**TN (%)**	**TP (lb)**	**TP (%)**	**TSS (tons)**	**TSS (%)**	**FC (billions)**	**FC (%)**
2-Year Storm										
*Upper Wreck Pond Brk**	*22.7*		*337*		*29*		*5*		*31,100*	
Wreck Pond Brook	33.7	57%	573	54%	18	30%	3	48%	38,800	43%
Hannabrand Brook	12.71	22%	229	22%	19	31%	2	32%	19,900	22%
Black Creek	3.52	6%	97	9%	8	13%	1	16%	14,900	17%
Wreck Pond Direct	8.97	15%	162	15%	15	26%	0.2	3%	15,900	18%
TOTAL	**58.9**		**1,061**		**60**		**6**		**89,500**	
1996 Wet Year										
*Upper Wreck Pond Brk**	*371.5*		*26,742*		*1,922*		*278*		*1,330,000*	
Wreck Pond Brook	529.7	55%	42,813	55%	1,236	34%	163	47%	969,000	41%
Hannabrand Brook	199.4	21%	16,550	21%	1,065	29%	127	37%	611,000	26%
Black Creek	64.0	7%	6,938	9%	529	14%	46	13%	512,000	22%
Wreck Pond Direct	177.2	18%	11076	14%	840	23%	9	3%	285000	12%
TOTAL	**970.3**		**77,377**		**3,670**		**345**		**2,377,000**	
2001 Dry Year										
*Upper Wreck Pond Brk**	*177.3*		*22,795*		*1,559*		*118.*		*480,000*	
Wreck Pond Brook	254.2	47%	34,876	55%	1,001	34%	58	35%	279,000	35%
Hannabrand Brook	116.10	21%	13,631	21%	789	26%	71	42%	236,000	29%
Black Creek	29.79	5%	5,520	9%	491	16%	35	21%	217,000	27%
Wreck Pond Direct	145.8	47%	9,559	15%	705	24%	4	2%	73,300	9%
TOTAL	**545.9**		**63,586**		**2,986**		**168**		**805,300**	

*due to settling in downstream ponds, the percentage contribution can not be properly assessed sub-catchment are hatched to illustrate this on the figures. Further, the Wreck Pond Direct results are not calibrated to a stream station. Figure 17 illustrates the relative loadings.

Wreck Pond Brook produces the highest relative flows under all scenarios due to its larger watershed area. Wreck Pond Brook produces 55-57% of the flows during the 2-year and wet-year simulations, with Hannabrand Brook providing just over 20% of the flow. During the dry year simulation, the proportion of flow from Wreck Pond Brook drops to just under 50%, while the Wreck Pond direct relative flow increases. As noted above, there is some question about under-prediction of Hannabrand Brook flows and whether there is some Hannabrand flows cross into Wreck Pond Brook upstream of Old Mill Road.

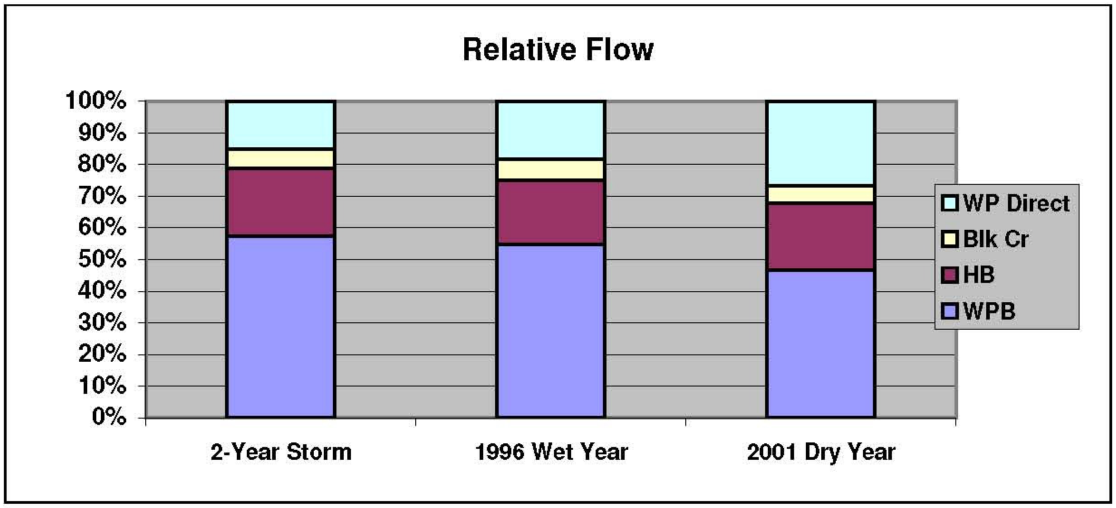
As expected, the largest watershed, Wreck Pond Brook provides most of the constituent loadings. For total N, the relative contributions mimic the flow regime. However, as shown in Figure 17, for TSS the contribution of Wreck Pond Brook is about 30% lower than expected from flow. This is due to the reduction in suspended solids seen on the lower part of the Brook. The Upper Wreck Pond Brook Watershed contributes significant loads of TSS. This watershed generates 32% of the flow in the dry-year simulation and 38% of the flow in the wet year and about 52% of the TSS load for both. Thus, without the loss of sediment in the lower Wreck Pond Brook sub-watershed, loading of TSS to Wreck Pond likely would be significantly larger.

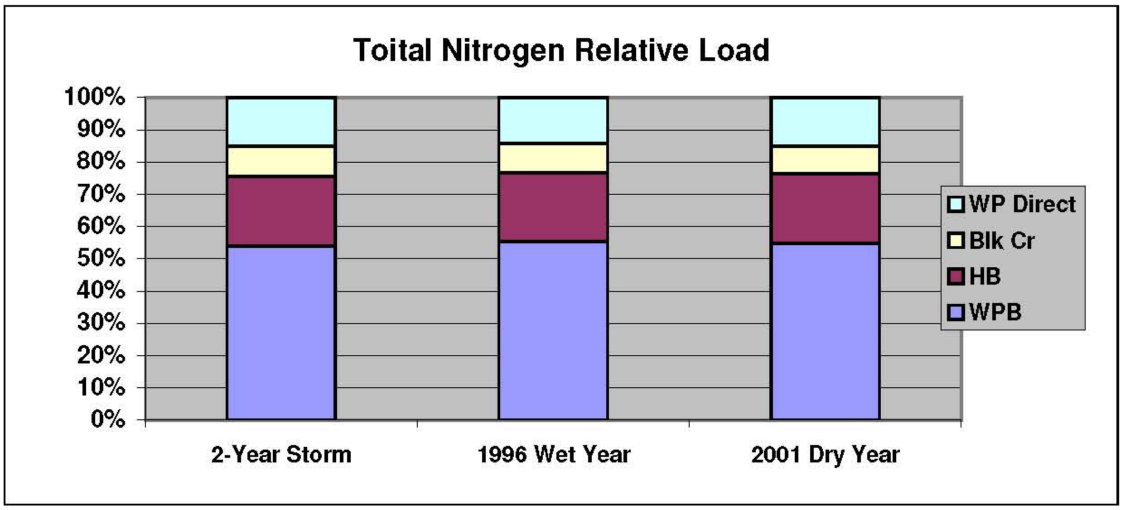
The relative Total Phosphorus (TP) loadings for Wreck Pond Brook are about 20% less than the flow contribution. In this case, the increased loads come from the more developed sub-basins in the lower watershed. Fecal coliform loadings are similar

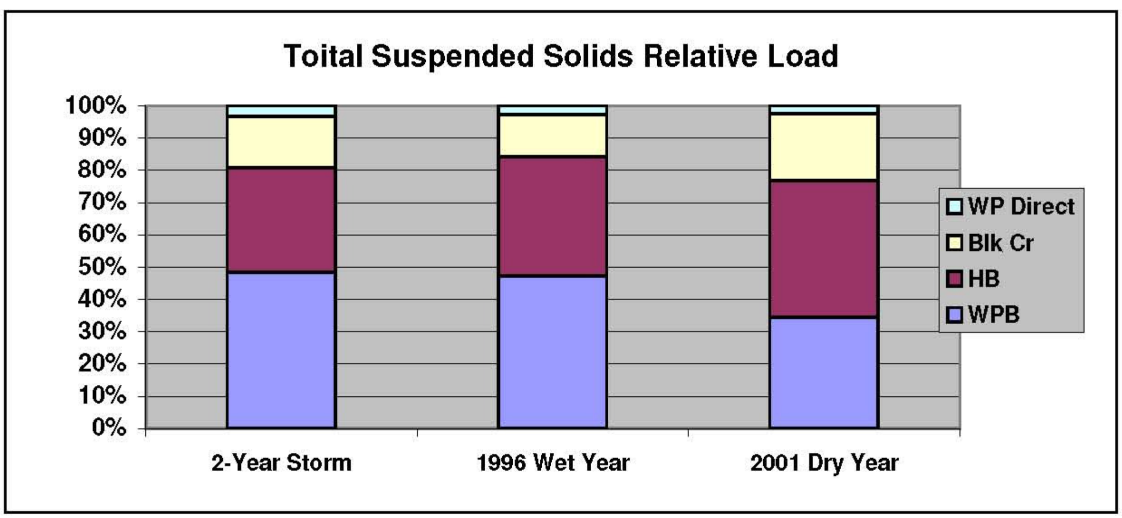
The comparison of the wet year versus the dry year show that the relative contributions of the watershed components are generally consistent. For flow, the Wreck Pond Brook watershed provides less in the dry year while Wreck Pond Direct provides more. For TSS, Hannabrand Brook provides a higher percentage of the Pond load during a dry year. Lower TSS loadings occur in the Wreck Pond direct watershed, due primarily to the extensive impervious area. However, it should be noted that sediment contributions associated with tidal inflows were not simulated by the model or reflected in these results.

In addition to looking at overall loading, loading rates were normalized by sub-watershed area to allow direct comparison of relative loadings. The normalized analysis provides a preliminary estimate of relative contribution of pollutants. These preliminary results suggest that the Black Creek, the Wreck Pond Direct and certain portions of the Wreck Pond Brook sub-watersheds provide the largest contributions of pollutant loadings on a per-acre basis for certain parameters. However, as noted above, water quality data were not available for the Black Creek sub-watershed and this is not calibrated. The NJDA modeling,

Figure 17: Relative Loading Analysis SWMM Model Results







completed after the SWMM model, has demonstrated that the Spring Lake Golf Course acts to detain stormwater flows and likely associated water quality constituents. Thus, actual water quality calibration data may show that the Black Creek watershed does not transport these loads to Wreck Pond.

8.4 Model Limitations and Conclusions

The SWMM model provides an estimate of watershed flows and pollutant loadings. The flow results were completed prior to the finalization of the NJDA hydrologic model. The limitations related to the NJDA model in Section 7, apply to this model as well. In particular, the depth gage data were inconsistent or unavailable in some cases and the rating curves were based on limited field flow measurements. Water level data may have been impacted by transient conditions, such as debris clogging the stream or shifting bed load. Additional flow and rating curve data, along with additional channel information, and the comparison of watershed flow to the USGS station at Jumping Brook, additional flow versus depth data would improve understanding and modeling of flow.

The SWMM model estimates watershed loadings of various water pollutants to Wreck Pond, demonstrating that existing land uses provide significant pollutant loads. This highlights the need to control watershed loadings to improve the quality of Wreck Pond. Although Wreck Pond Brook provides the largest contribution to the loads due in large measure to its larger flow, Hannabrand Brook provides higher unit loading rates for certain parameters.

The model also shows the important function of the Ponds on Wreck Pond Brook in controlling flow and water quality. This Brook showed a net loss of pollutant load for TSS and TP from the upper to the lower water quality station. If these Ponds cease to function to retain sediment and associated pollutants, loadings to the Pond will increase. The NJDA modeling showed a similar function for the Spring Lake golf course as a detention feature in the lower watershed.

Given the lack of control structures on Hannabrand Brook and the relatively high TSS concentrations seen here, more opportunities to improve water quality may be found in this sub-basin as reflected in the BMP suggestions in Book 2. The highly developed Wreck Pond direct sub-watershed generates roughly about 15-25% of the nutrient loadings and 9-18% of the bacteria loadings to the Pond. Implementation of stormwater BMPs in developed areas of the watershed, as discussed in Book 2, may reduce loadings of these parameters.

Future refinement of the SWMM model with additional water quality, pollutant generation, and flow data would provide additional understanding of watershed contributions of pollutants. The model demonstrates that the watershed is generating significant pollutant loads. Controlling existing watershed pollutant sources and stormwater flows will provide water quality benefits to Wreck Pond, other watershed ponds, and tributary streams.