

## **ABSTRACT**

**Title of Thesis:** MODELING WATER QUANTITY AND WATER QUALITY  
WITH THE SWMM CONTINUOUS STREAMFLOW MODEL  
UNDER NON-STATIONARY LAND-USE CONDITION USING  
GIS.

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GIS data widely available today can be used to better estimate watershed parameters for the SWMM model. An interface was developed to create SWMM input files from spatial data. The interface delineates watersheds and allows update of land-use parameters.

SWMM performs continuous simulation but it assumes a time-invariant land use. A “hot-start” technique was developed that uses end values from one year’s simulation to initialize state variables for the next year. This technique allows for dynamically changing the land use model parameters to reflect changes in time.

Based on the simulation results, three regression models were developed to adjust constant land-use model results to account for land use changes in peak discharges, baseflow, and total phosphorus loads. These adjustments use imperviousness as an index of land use changes. The regression equations adjust streamflow and water quality results from a constant land use SWMM simulation to conditions with time-varying land use.

MODELING WATER QUANTITY AND WATER QUALITY  
WITH THE SWMM CONTINUOUS STREAMFLOW MODEL  
UNDER NON-STATIONARY LAND-USE CONDITION USING GIS

by

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## **Chapter 1**

### **Introduction**

#### **1.1. Motivation**

There has been an increasing interest for the last couple of decades to develop estimates of discharge and water quality as two important tasks facing water resources engineers. An increasing amount of hydrologic data has been collected characterizing the response to precipitation of different watershed in the United States. These data (precipitation, discharge, and pollutant loads) can be used to better understand how the watershed response is affected by alteration of the landscape, particularly urbanization.

Water resource engineers have increased the understanding of the physical and chemical processes involved in the flows of water to streams. Mathematical models that describe these processes more realistically have been continuously developed and improved. Faster and larger computers are able to process the formulations of the new models and to manipulate the large amount of data recorded.

Some of these models divide the watershed into impervious and pervious sections, each one with its own set of parameters that characterizes the unique response of that watershed. Traditionally, these models are calibrated by adjusting these parameters until simulated responses (e.g., streamflow) match observations. This approach assumes that these parameters do not change with time. This is an assumption that is not valid in the

face of urbanizing watersheds like the ones that can be found in the suburbs of Washington, D.C., in the twentieth, and now twenty-first, century.

Models for simulation of urban runoff hydrographs such as the Stormwater Management Model (SWMM), Illinois Urban Drainage Area Simulator (ILLUDAS), and STORM among others have been widely used for several years. SWMM is one of the most common hydrologic models used in the simulation of watersheds in U.S. and Canada. Using SWMM the watershed must be divided into smaller areas with assumed homogeneous characteristics (e.g., land use, slope, soil type) and results are combined together according to the drainage structure of the watershed.

The manual preparation of input data can take anywhere from a few weeks for small watersheds to months for large and/or highly urbanized areas. Computers can be used to process input data and to automate the preparation of input files. This allows more time to be dedicated to calibration, verification, and sensitivity analyses.

Another part of the hydrologic modeling is related to the calibration. Calibration is done by adjusting some parameters so that the model discharges are close enough to observed data. Certain measures, such as goodness-of-fit (GOF), are tested until the model is considered to reproduce accurately the watershed response. The model is then executed for longer periods of time to test its ability to model the response for different periods not used in calibration.

There are several measures that quantify goodness-of-fit. One of the most common measures assumes that events are independent which does not apply when modeling hydrology on a continuous basis.

## **1.2. Scope of the project**

The goal of this study is to use SWMM, the Stormwater Management Model, as a tool for modeling both water quantity and quality in four watersheds over a period of decades. SWMM is a mathematical model developed originally for the EPA by Metcalf and Eddy (1972). It was developed to model surface runoff in urbanized areas, but it can be used in undeveloped areas and in watersheds with a mixed land use. In this thesis, a GIS-based interface will be created to process data and reduce the time for generating the input file needed to run SWMM so that more time can be dedicated to improving the estimation of the parameters used in the input file, as well as analyzing the influence of uncertainties of input parameter estimation on the model estimation of discharges.

Another goal of this study is to analyze the effect of errors in discharges on nutrient loading that can be introduced when assuming constant land use (either past, present or future conditions) when land use is actually changing over time. This goal will be achieved by comparing five different scenarios: constant land use (1979 and 1988 conditions), continuous simulation with annual stops to update land use dependent variables (referred to as state variables) with constant land use (1979 and 1988 conditions), and dynamic annually changing land use. Differences between these five scenarios will be analyzed for water quantity and quality.

Estimates of streamflow and water quality will be made not just at the outlet of the watershed but at numerous internal points within the watershed corresponding to available field observations. Because urbanization has been significant over the period of time being modeled, a major part of this study will be concerned with characterizing land

use change and translating this change into spatially and temporally varying parameters needed within the SWMM model.

The tools and the techniques developed in this study will be useful for performing similar analyses in other watersheds. Such tools would also be useful for making predictions of the impact of future land use changes, climate changes, or effectiveness of some best management practices (BMPs).

### **1.3. Approach**

Four watersheds were selected in the area of the Washington, D.C., suburbs. Data collected are in different formats. Some are hard copies, while other data are in digital format. Additionally, high-resolution GIS data (topography-DEMs, land use, soil type) were also available from different sources for the same watersheds. By combining these data together it will be possible to facilitate the process of generating the input files required by SWMM.

Annual input files will be generated by using a GIS interface and information from the completion of previous year's simulation and new precipitation and loading information corresponding to the present year, to model the five different scenarios mentioned above. Each one of the new input files considers the annual change in land use along the whole watershed. The GIS interface will call and execute the SWMM model with each one of these input files and will summarize modeled output data.

Two types of data series will be compared: discharge and nutrient pollutographs for each of the scenarios under analysis. It is expected that the results of the analysis will show an improvement with respect to the constant land use in the synthetic generation of



hydrographs and pollutographs obtained by making land use vary annually with the simulation of constant land use. Final information is available in the text format and can be manipulated with other software to compare results and generate custom reports.

This research contributes new methods to the hydrologic modeling community by defining a method to model dynamic annually changing land use using a GIS interface developed for SWMM. This research also provides methodologies to measure how and when the failure to account for changing land use may lead to large errors in flow and pollutant estimates.

#### **1.4. Overview**

Chapter Two of this thesis provides background information from the literature to help put this work in its appropriate context. Chapter Three describes the watersheds being studied and the tools and approach that were developed to characterize these watersheds. Chapter Four summarizes the results obtained for each sub-watershed and each simulation. Chapter Five presents conclusions and recommendations for future studies.

## Chapter 2

### Literature review

#### 2.1. Watershed modeling

Hydrologic modeling can be approached from two different perspectives: Deterministic and Stochastic models (Linsley et al., 1982). The first type of these models "attempt to describe the actual physical processes of the hydrologic cycle so as to simulate actual hydrologic events such as the transformation of a series of rainfall inputs to resulting streamflow hydrographs" (Linsley et al., 1982). Stochastic models involve the statistical relation of independent variables without consideration of the physical or mathematical descriptive relation among these variables. Bernoulli defined stochastic models as "the art of estimating as best as one can, the probability of things" (Bernoulli, 1713). Stochastic models must be used when estimating values of the parameters based on other characteristics (e.g., estimate porosity based on soil type). Both types of models give hydrologists useful tools with which to study complicated watershed systems and their streamflow behavior.

According to Chapra (1997), modeling, and especially water-quality modeling, could be oversimplified. In modeling, the mathematical aspects are just one component of the process. There are other aspects that are as important as the mathematical formulation and should be given as much as attention. Chapra (1997) suggests that even before writing the first equation, the model process begins with a problem specification phase.

The water quality engineer must make a clear definition of the objectives at the beginning of the process. Once the objectives have been defined, the engineer can face the model selection, which can be either to develop a new one or to select an existing one. In the specific case of water-quality, there are so many different phenomena that one model may not be suitable to achieve all of the objectives. Therefore, a preliminary evaluation is necessary to identify data deficiencies and theoretical gaps. This step can also be helpful in the identification of model sensitivities and thus model strengths and weaknesses for the specific problem.

Once the objectives have been defined, the model has been selected, and that data have been obtained to conduct the simulation, then the model can be calibrated and verified. Calibration is a process of varying model parameters to obtain optimal agreement between model simulations and observed data. Calibration must be oriented to the objectives of the modeling effort. If the model is intended to simulate point source pollution, it should be calibrated in a different way than if it is intended to model continual non-point source pollution. The calibrated model is used to simulate other time series different than those used in the calibration process in order to verify the model. Model verification confirms that the calibrated model is able to model other periods than the one used in the calibration process and is referred to as model confirmation and robustness (Chapra, 1997).

### **2.1.1. The Stanford Watershed Model and related models**

Since ancient times there has been interest in predicting the type and amount of precipitation but it has been just in the last half of the twentieth century that the

deterministic modeling has matured. This is due to the ability to solve complex mathematical formulations with the use of computers. Taking advantage of this improvement, Crawford and Linsley (1962) developed one of the most complete programs for watershed simulation known as the Stanford watershed model (Wanielista, 1978).

The Stanford watershed model was designed to work for single event simulation and it generated hydrographs and pollutographs for individual storm events but it was not a resident memory program. A few years later, in 1966, Crawford and Lindsey developed the first version of the Stanford watershed model (Wanielista, 1978) that could be saved in the computer memory and parameters and inputs could be changed for any particular simulation.

Improved models were developed based on the Stanford watershed model like the one developed by Johanson et al., (1980), that was designed to simulate time steps of 15 minutes; the Dorsch QQS model (Geiger and Dorsch, 1980) that used a time step of 5 minutes; and several other models like the Long-Term Hydrologic Impact Assessment (L-THIA) model (Engel, 2002). Some other models for continuous simulation were developed like STORM developed by Water Resources Engineers Inc. for the US Army Corps of Engineers (Roesner et al., 1966) for the City of San Francisco. One of the main characteristics of STORM was that it was based on one-hour time steps coupled with simplified runoff and pollutant estimation procedures. STORM has been extensively used for planning studies (Roesner et al., 1966) and urban runoff evaluation (Heaney et al., 1977).

### **2.1.2. Stormwater Management Model - SWMM**

Many of the models developed after the Stanford water management model were more complex or specifically designed for particular situations. The EPA Storm Water Management Model, SWMM, developed in 1969-1971, was one these models. SWMM has been updated over the years and is one of the most commonly applied urban stormwater models. Technical details of the SWMM model can be found in the original documentation by Metcalf and Eddy et al., (1971a, 1971b, 1971c, 1971d), Huber et al. (1975), and Heaney et al. (1975). More recent documentation can be found in Huber and Dickinson (1992).

The first SWMM model was developed as a very simple model and throughout its updates has maintained its initial simplicity as much as possible. Private companies have released versions that give options of input and output in a friendlier environment and present results using the latest state-of-the-art graphics, but the program core itself has remained basically unaltered. SWMM is written in FORTRAN with different subroutines that are called from a main program. This structure allows the user to adapt the model to particular situations by selecting only those processes that are relevant to the problem.

## **2.2. Stormwater management principles**

Stormwater management (SWM) is a discipline that has a practical application in different activities like: agricultural drainage, forest water management, urban runoff, and lake level management among others. The hydrologic cycle is the system under analyses in each one of these areas and is the basis for stormwater management engineering.

Understanding how water is transported, altered and retained in each watershed must be well understood in order to simulate the total amount of water and the difference with the water budget either in quantity (floods or droughts) or quality (pollution, erosion, or sedimentation). (Wanielista, 1978).

Stormwater management engineering analyses can be divided into the following categories for a better understanding: meteorology, watershed and hydrological data, hydrograph flow routing, water quality response to non-point sources, stormwater management alternatives and receiving water quality assessment (Wanielista, 1978). A brief description of each one follows:

- *Meteorology, Watershed and Hydrological Data.* Ground cover, topography and meteorology of an area determine the resulting hydrology. In general, more runoff can be the result of more paved areas and steeper slopes. But larger precipitation, higher temperatures, and the more solar energy can also be the cause of higher runoff because at these conditions the vegetation can be dry out and there could be a lower surface water detention.
- *Sub-watersheds.* Each area under analysis is divided in different sub-watersheds with homogeneous conditions (slopes, land use, channel shapes, etc.). Hydrographs and pollutographs are generated for each one of these sub-watersheds based on input and data describing each sub-watershed.
- *Hydrograph routing.* Hydrographs obtained from each sub-watershed are routed individually through a network of channels and combined at nodes taking into consideration the time of travel in the channels and they added as they flow to one point known as the outlet by a process generally called as flow routing.

- *Pollution.* One of the goals of SWM is to simulate the water quality response to non-point sources of pollution (NPSP). Pollution as a consequence of NPSP has become a greater concern in recent years.
- *Pollution control.* Remediation alternatives can be proposed once the level of pollution and discharge has been identified. SWM alternatives can be simulated and the improvement in the water quality and quantity can be assessed. Economic goals can also be analyzed to reach a compromise between effectiveness and cost.
- *Receiving water quality assessment.* Outlet hydrographs and pollutographs can be included as input for other analyses that can be done in a later stage.

These activities or goals are described more extensively in the SWMM documentation (Huber et al., 1992) as it has been designed in blocks or operating units. Each block processes one of the steps from above stormwater management and their outputs are coordinated by the executive block. Main SWMM blocks are: Rain, Runoff, Transport, Extended transport (EXTRAN), Storage and treatment, and Statistics. There are other blocks that assist the manipulation of data like the Combine block. These blocks will be described in more detail in Chapter Three.

### **2.3. Water quality modeling**

Increasingly non-point source pollution (NPSP) is getting more attention from governmental entities and researchers. It can be defined as pollution whose origin cannot be established to be one particular site or activity, but it comes from different common activities and is spread over a significant area.

The Chesapeake Bay is one of the most studied bodies of water in the world (Lenwood et al., 1991). Specialized monitoring programs have been developed since the 1970's, and extended data results have been accumulated showing not only all kinds of pollutants in the Bay but also all existing natural resources. This research has generated concern for the Bay in the scientific community and from the general public. As a result the environmental goal to restore the Bay has gained popularity. Since the late 1980's the Bay has shown environmental recovery (Lenwood et al., 1991).

Many environmental indicators continue being measured to track the Bay's environmental health. Indicators are factors that can affect an ecosystem, for example concentrations of nitrogen, phosphorous or toxics, and the amount of nutrients. Currently there are about 90 indicators used by the Chesapeake Bay Program. All of these, have contributed to the development of The Chesapeake Bay Program, and have been used to communicate the progress of the restoration efforts and the health of the Bay to the public.

In the U.S. there are regulations for both types of runoff pollution, point and non-point source pollution. Point source pollution is addressed by the National Pollution Discharge Elimination System (Lenwood et al., 1991) and non-point source pollution are addressed by states, territories and tribes programs under the Clean Water Act (CWA) and in some sites of U.S. NPSP have to be referred also to the Coastal Zone Act Reauthorization Amendments (CZARA).

Non-point source pollution threatens to erase much of the progress achieved by the Clean Water Act in restoring the nation's water resources, The most promising and



controversial tool the CWA offers to address this growing problem is contained in the total maximum daily load (TMDL) provisions of Section 303(d). (Birkeland, 2001).

#### **2.4. Previous studies in land use change**

Engineers have addressed concern about the changes in hydrology due to changes in land use since earlier times. Analyses of increasing peak discharges of about two to five times higher in developed areas compared to pre-developed areas were noticed (Dunne and Leopold, 1978; Anderson, 1973) along with a decrease in the time needed for runoff to reach the outlet, known as time of concentration, by as much as 50% (Dunne and Leopold, 1978).

Some other researches have analyzed the changes in the hydraulics of the streams due to land use and land cover changes. Some of the results of these studies showed an increase in widening as a primary adjustment to the increased storm flows (Robinson, 1993; Jackson et al., 1976; Hammer, 1972).

In general, most of the models, simple and complex, use land use and land cover (LULC) data or some other data derived from them to generate runoff and pollutant/nutrient loads. Nevertheless it has been found that not all LULC data lead to the same solution. Variations using different land uses lead to differences between 8 and 14% regarding runoff and between 13 to 40% regarding total suspended solids (TSS). This is due to two main reasons: different resolutions and different approximations to land use (Burian et al., 2002). There is another reason not considered in this study why LULC could be different, the date at which the land cover was determined. Land use in an area can change on a short time scales and these changes can accrue to large differences over

time. Several assumptions must be made in order to differentiate the temporal change of land use and even more to simulate the future land use based on the trend of the land development in certain areas (Van Rompaey, 2002).

From several papers analyzed it was found that though the interest is to compare different land uses there has been little work done for a continuous change in land use simulation (Lohani et al., 2002; Tollan, 2002). Instead, other methods of comparison have been used including the analyses of different areas with similar conditions but different land uses (Kondolf et al., 2002) and the analysis of different land uses for defined watersheds at some predefined points in time ("before/after" analysis) for which land use information is available (Lohani et al., 2002). In this study we have incorporated an annual variation of land use with a continuous simulation of water quantity and water quality.

The Soil Conservation Service (SCS) developed an empirical method to estimate the total volume of runoff as a consequence of changes in land use cover. This method was developed to assist in the design of stormwater management facilities, and is based on the classification of several parameters describing the direct runoff and soil hydrologic types for different areas across the U.S. The water balance used in this method gives the runoff as a function of the total precipitation (P), the potential maximum storage retention (S), and the initial abstraction ( $I_a$ ) as shown in equation 2.1. (Bras, 1990).

$$R = \frac{(P - I_a)^2}{P - I_a - S} \quad (2.1)$$

where:

$R =$  Total volume of runoff in inch of water.

$P =$  Total precipitation in inch of rain.

$S =$  Maximum storage retention.

$I_a =$  Initial abstraction in inch of water.

The initial abstraction is commonly taken as  $I_a = 0.2S$ . The potential maximum storage retention is a function of the hydrologic soil type (from low runoff potential through high runoff potential), and the antecedent moisture condition both represented by the runoff curve number (CN) (Bras, 1990), as follows:

$$S = \frac{1000}{CN} - 10 \quad (2.2)$$

where:

$S =$  Potential maximum surface retention in inch of water.

$CN =$  Dimensionless runoff curve number.

Different values of runoff curve numbers are given for each land use cover type. The lower the CN value, the lower the runoff. Curve numbers are taken from tables depending on the hydrologic soil type, antecedent moisture conditions, and locations in the U.S. Hydrologic soil types vary in general from A through D, being A the soil with lowest runoff potential and D the highest. (Bras, 1990).

The SCS runoff method is widely used in the U.S. to estimate the effects that changes in land use have on direct runoff. The changes in land uses are represented by changes in CN; in general, a large CN value represents a more developed land use.

The USGS has developed sets of regional regression equations that provide another alternative for analyzing the effect of urbanization. These equations are published for every state in the U.S. and they allow estimation of peak flow frequency and

magnitude at ungaged sites by using topographic, physiographic, and climate characteristics. These sets of equations; are based on a statistical relationship of the flood characteristics to the physical and climatic characteristics of watersheds for a certain group of rural streamgaging (Perl et al., 2002). Fpr example, the USGS rural regression equations for the State of Maryland are presented in Dillow (1996). These equations were developed from 219 rural gauging stations in the states of Maryland, Delaware, Pennsylvania, Virginia, and West Virginia. A computer program titled National Flood Frequency Program was developed to provide regression equation estimates of flood-peak discharges for unregulated rural and urban watersheds (Ries III et al., 2002).

The above regression equations are generally developed from rural watersheds; therefore, adjustments must be made to reflect the effects of urbanization. Some of the parameters used in the adjustment are percent of imperviousness and a variable known as the Basin Development Factor (BDF). The BDF is an index that varies from 0 to 12, where 0 means a rural watershed and 12 a fully developed urban drainage system based on channel improvement, channel lining, storm drains, and curb-and-gutter street drainage (Sauer et al., 1983).

The National Engineering Handbook, section 4 (NEH-4) (Soil Conservation Service, 1985), outlines several general procedures and establishes how the impact of development changes the hydrology of the watersheds. The evaluation is defined as a detailed investigation of the present (no project) and future (with project) conditions of a watershed in order to determine whether given objectives will be met (Soil Conservation

Service, 1985). There is a work plan (See Figure 2.1) that has been established as the official document for carrying out, maintaining, and operating the certain project.

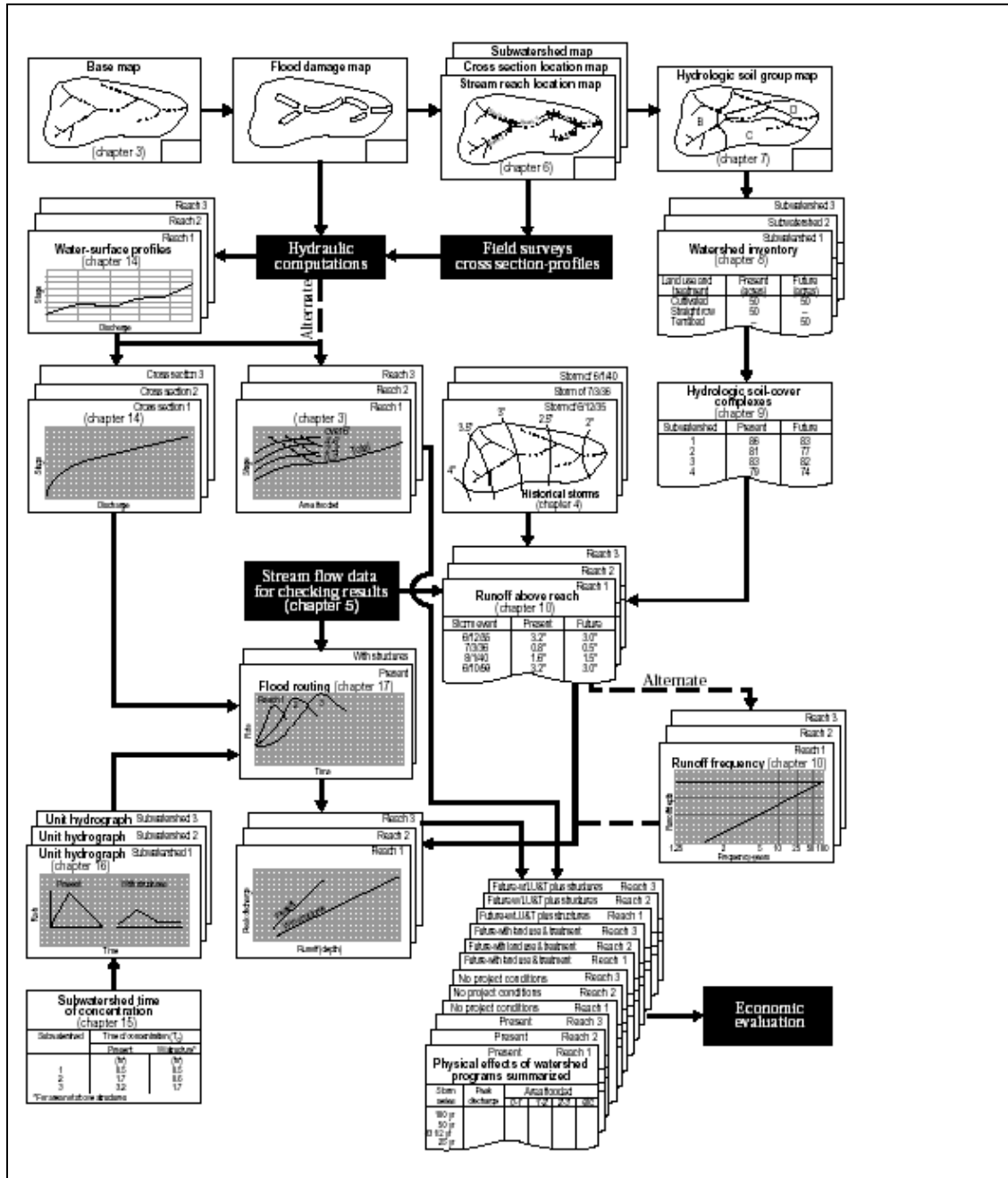


Figure 2.1. General process for the hydrology of watershed project evaluation with streamflow and rainfall data available (Soil Conservation Service, 1985).

This procedure outlined in the NEH-4 has been used to analyze the effect of land use changes from the existing conditions of a watershed. The Maryland Department of Environment (MDE), has created the Stormwater management design manual (CWR and MDE, 2000). This manual compiles the experience gained from over 14 years of field studies and analyses to set guidelines for the management of stormwater runoff to reduce channel erosion, pollution, sedimentation, and local flooding impacts on the water and land resources of Maryland. The main objective is to preserve as closely as possible the predevelopment hydrologic conditions (peak, time to peak, time of concentration) and pollutant discharge conditions (total pollutant loads and total sediment load) (CWR and MDE, 2000).

Studies made for impact analyses are done on the basis of present and future conditions. Future conditions are understood as the present conditions including the project under analysis as if it has been constructed and the best management practices developed for attenuating the impact in the disturbed area. Flow peaks and pollutant concentrations must not differ very much from the present conditions. (CDM, 1996).

The Maryland Chesapeake Bay Water Quality Monitoring Program consists of a database of information collected in an almost monthly schedule from 22 stations located in the Maryland portion of the Chesapeake Bay. There are two types of data collected: physical parameters and water quality parameters. These data are used to control the growth of harmful algae blooms (MD DNR, 2001).

There have been several studies in the tributaries of the Chesapeake Bay, some done on the Patuxent River and Choptank River. Fisher, whose interest is to simulate the temporal and spatial variation of pollutant in the Choptank River used a modified

generalized watershed loading functions (GWLF), (Haith et al., 1992). Calibration of the model reproduced the concentrations of total nitrogen, total phosphorus with accumulated errors through an eleven-year series below 1%. The results of simulation using GWLF model for estimation nitrogen and phosphorus for fluxes of water showed accuracy between 10-50% at annual time scale. (Fisher et al., 1998).

Several studies have been performed in the area of Montgomery County, Maryland, most of them have been done on spatially distributed assumption linked to temporal variations in land use with the use of GIS. This approach has been used to simulate both the temporal and spatial changes of peak discharge within a watershed (Moglen and Beighley, 2002).

## **2.5. Goodness-of-fit methods**

Three different goodness-of-fit methods were considered in this study: (1) deviation volume coefficient ( $D_v$ ), (2) cumulative distribution function of discharges (CDF), and (3) Nash-Sutcliffe index ( $R^2$ ). A description of each one of these methods, follows.

### **2.5.1. Deviation volume coefficient**

ASCE (1993) define three indices that may be used when comparing modeled discharges with observed discharges based on the fact that the common assumption of statistical independence among data is violated in daily flows. One of the indices is the deviation volume coefficient ( $D_v$ ) defined as the fraction by which the modeled or

simulated total amount of discharge over or under estimates the total amount of observed discharges:

$$D_v = \frac{\sum V_o - \sum V_m}{\sum V_o} \quad (2.3)$$

where:

$D_v$  = Deviation volume coefficient.

$V_m$  = Total volume of discharges modeled in a time period in ft<sup>3</sup>/s.

$V_o$  = Total volume of discharges observed in the same time period in ft<sup>3</sup>/s.

$\sum$  = Indicates a summation over time periods simulated (days or years, for example).

The closer this index is to zero the better the simulation of the total volume of discharges for the period under analysis (ASCE, 1993). A value of  $D_v$  close to zero means that the total volume of simulated discharges is similar to the total volume of observed discharges. If  $D_v$  is positive, the model has underestimated total discharge; if negative, the model has overestimated the total discharge. This index is used as one of the goodness-of-fit measures in this study.

### 2.5.2. Cumulative distribution function of discharges

A cumulative distribution function (CDF) of discharges is calculated by defining the percentile to be the percent of flows not exceeding a specified discharge. For example, a percentile of 0.70 equal to 50 ft<sup>3</sup>/s means that seventy percent of the discharges in the period included in the analysis are less or equal to 50 ft<sup>3</sup>/s. Curves



reflecting different discharges indices are plotted for each one of the series under analysis, and the resulting “S” shapes are graphically compared to the observed discharge series to assess if the observed and simulated discharge distributions were similarly distributed (Figure 2.2). A Chi square test is done after the graph analysis to prove the hypothesis that both CDF series had the same distribution.

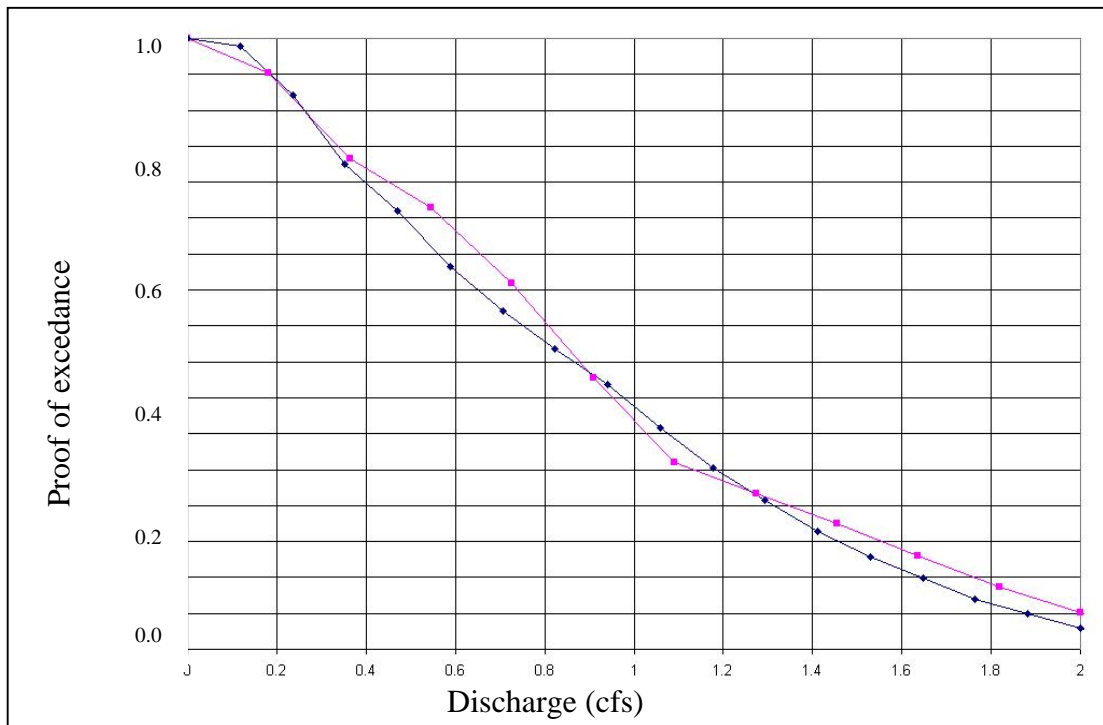


Figure 2.2. Cumulative distribution function of discharges for simulated and observed discharges. The gray line represents observed discharges and the dark line represents the simulated discharges.

### 2.5.3. Nash-Sutcliffe index

The Nash-Sutcliffe index is a measure of the improvement in using a simulated discharge as opposed to use the average of the measured discharges. As described in equation 2.4, the Nash-Sutcliffe index measures the square of the difference between the measured daily discharge and the computed or simulated daily discharge and the square

of the difference between the observed measured daily discharge and the average measured discharge. A value of one means a perfect fit, and a value of zero means that using the average of observed discharges has the same error as using the simulated discharges. The Nash-Sutcliffe index can have negative values but these are rather meaningless in the context of interpretation (ASCE, 1993).

$$R^2 = 1 - \frac{\sum_{i=1}^n (Q_i - Q_i')^2}{\sum_{i=1}^n (Q_i - Q)^2} \quad (2.4)$$

where:

$R^2$  = Nash-Sutcliffe index.

$Q_i$  = Observed daily discharge in ft<sup>3</sup>/s.

$Q_i'$  = Simulated daily discharge in ft<sup>3</sup>/s.

$Q$  = Observed average daily discharge in ft<sup>3</sup>/s.

$n$  = Number of days of simulation.

## 2.6. SWMM model

SWMM is a comprehensive conceptual computer model for simulation of the runoff quantity and quality in urbanized systems with combined sewer systems. SWMM simulates real storm events on the basis of rainfall and meteorological inputs and system characterization. SWMM rainfall data (single event or continuous long-term) and makes a step by step account of snowmelt, infiltration losses, evaporation, surface detention, overland flow, channel flow and pollutants washed into inlets (nodes) giving hydrographs and pollutographs at each node.

SWMM models direct runoff from drainage areas based on the physiographic characteristics (area, slope, shape, roughness, detention storage). SWMM can model groundwater flow by using infiltration volumes subtracted from direct runoff. Two methods to account for infiltration are proposed: Horton and Green-Ampt. Different mathematical formulations can be used to model groundwater flow. Groundwater flow can be modeled in two areas, one an intermediate zone and the other a lower model. Water quantity interchange between these two zones is done by water balances that account for plant uptake, groundwater flow in and from the channel network and deep percolation, among others.

There is also flexibility in the amount of detailed information that can be output from SWMM. Hydrographs can include discharges and water table elevation for each time step for all the nodes or for selected ones. Pollutographs are given as a total load per time step and can be obtained for all or some of the nodes.

The SWMM model is a complex model. It is recommended to read some of the manuals (Metcalf and Eddy et al., 1971a, 1971b, 1971c, 1971d) that are available in order to understand the mathematical formulations included in the model. As this is not a goal of the research to describe SWMM modeling process, but it is necessary to understand how the SWMM model has been designed, a description of the model architecture follows.

SWMM simulations are the result of the sequential execution of parts of the program referred to as operational blocks. Some of these blocks are: Rain, Runoff, Transport, Combine, and Statistics (Figure 2.3). Each one executes a part of the hydrologic cycle, combines blocks together to increase the capacity of SWMM to

simulate large models, or provides statistical analyses on some of the variables. All of the blocks can be combined to give a variety of options for the user to model a watershed and to obtain different types of output depending on the goals of the study.

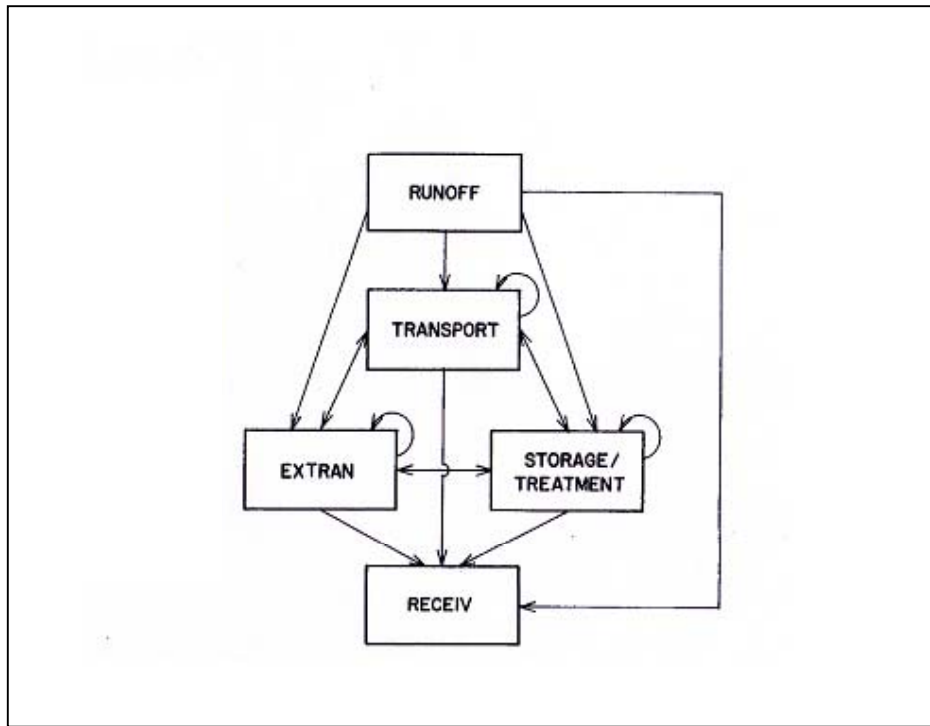


Figure 2.3. SWMM operational block interrelations. (Huber et al., 1992)

The RUNOFF block is the first one to be executed, it generates the input files required by all the other blocks. The RUNOFF block first reduces water input by evaporation, infiltration and finally by surface depression storage, in that order.

## 2.7. Conclusions

Hydrologic modeling has improved in the last decades mostly from advances in computer capabilities. Several computer models have been developed since Crawford and

Lindsey (1972) developed the first Stanford watershed model. SWMM (Huber et al., 1992) is a comprehensive model that is versatile and easily adapted to each watershed analysis.

There is a high interest in the analysis of the effects of development in the environment. Planners need assistance in developing more realistic modeling tools that can be used now with today's technology.

Changes in land use and the consequent effects on non-point source pollution have been a central factor in the planning and conversion of forested areas into agricultural or urbanized (e.g., residential, commercial, institutional) areas. Nevertheless the studies of disturbance have been done on a series of constant land use analyses without analyzing the transition from one land use distribution to another.

In the next chapter it will be explained in more detail the way SWMM was designed, through the development of a case study example. A contrast of the effects of simulating water quantity and water quality using both static and dynamic characterizations of land use will be presented.

## **Chapter 3**

### **Methodology**

#### **3.1. Introduction**

The focus of this research is to study the effects of urbanization on water quantity and water quality. Both of these phenomena are affected by changing land use through time. This chapter will describe the procedure that was followed to: (1) develop a water quantity model to adjust a constant land use model results to time-variable land use by modifying peaks and baseflow; (2) develop a water quality model to relate pollutant concentration to changes in land use.

#### **3.2. Data requirements**

The Northwest Branch of the Anacostia River watershed was selected for analysis in this study using SWMM model. This watershed (USGS gage 01650500) was subdivided into three sub-watersheds, each with different degrees of development. Two of them were subject to a low increase in percent of development (one almost unchanged through the modeling period) and the other was subject to a high level of development change. These actual conditions were reflected in the model as well. The entire watershed was also included in the analysis providing the perspective of an area with development rates between the extremes observed in the individual sub-watersheds.

### 3.2.1. Precipitation data

The most important input into a hydrologic model is precipitation. In SWMM, precipitation is the only source of water into the system. SWMM is designed to utilize different precipitation formats. The type of format most commonly used and recommended by the software developers and users is TD3240 from National Oceanic and Atmospheric Administration (NOAA) - National Climatic Data Center (NCDC) (Huber et al., 1992). TD3240 is a one-hour time interval record and is available for many rain gages across the United States.

Several rain gages close to the study area were analyzed based on the geographical location as a first consideration. Then the length and continuity of the records were used as the second criterion to select a good source of data. While the Washington metropolitan area has many National Climatic Data Center (NCDC) precipitation stations, there are a limited number of gages offering a long, continuous period of record. From the initial group of 36 gauges, only four offer relatively complete records of 30 years or more. Beltsville has the longest and most complete series of data based on the time period selected for this model and also the online availability of data in electronic format. Even still, several day gaps in the record still exist making it difficult to assemble a continuous period of record from any one gauge. Figure 3.1 shows the location of the 36 precipitation gauges initially considered in this study and the one used in this study is shown in bold. The hourly precipitation data were obtained from NCDC station 180700 located in Beltsville (National Climatic Data Center, 2002). Also considered were data from station 181125 (Brighton Dam). Data were available only at a

daily time step at Brighton Dam so trying to unify time steps between precipitation and streamflow was not as successful as will be described in the next chapter.

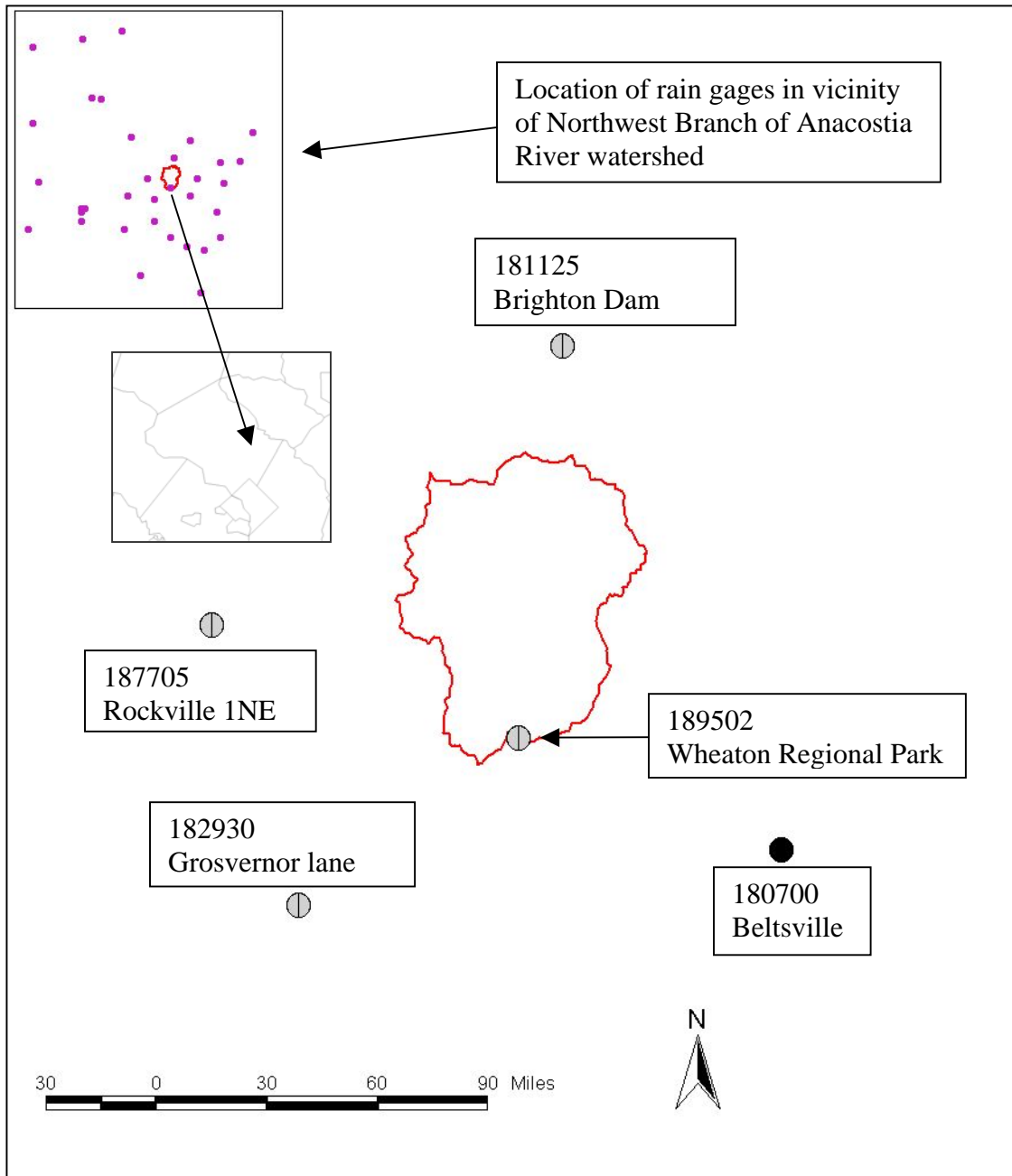


Figure 3.1. Rain Gage locations. Precipitation data from rain gage 180700 at Beltsville, (shown as a dark bold point) was used in the SWMM model.



### **3.2.2. Streamflow**

United States Geological Survey (USGS) streamflow data offer a useful database of streamflow in Maryland. Maryland, a state with a relatively high gage density, 0.00826 gages/km<sup>2</sup> (0.0214 gages/mi<sup>2</sup>), has had in operation at one time or another a total of 217 gages, with 87 currently active (Moglen and Beighley, 2002). USGS daily stream gage 01650500, NW Branch Anacostia River near Colesville, MD (U.S. Geological Service, 2001) was selected, as it was located on the Northwest Branch of the Anacostia River including the area of our study. This gage is located at latitude 39°03'56.4", longitude 77°01'45.6" NAD83, datum of gage 264.85 feet above sea level. The drainage area at this gage is 21.1 square miles. The period of record of the gage is from 1923 to current, with a missing period from 1983 to 1997.

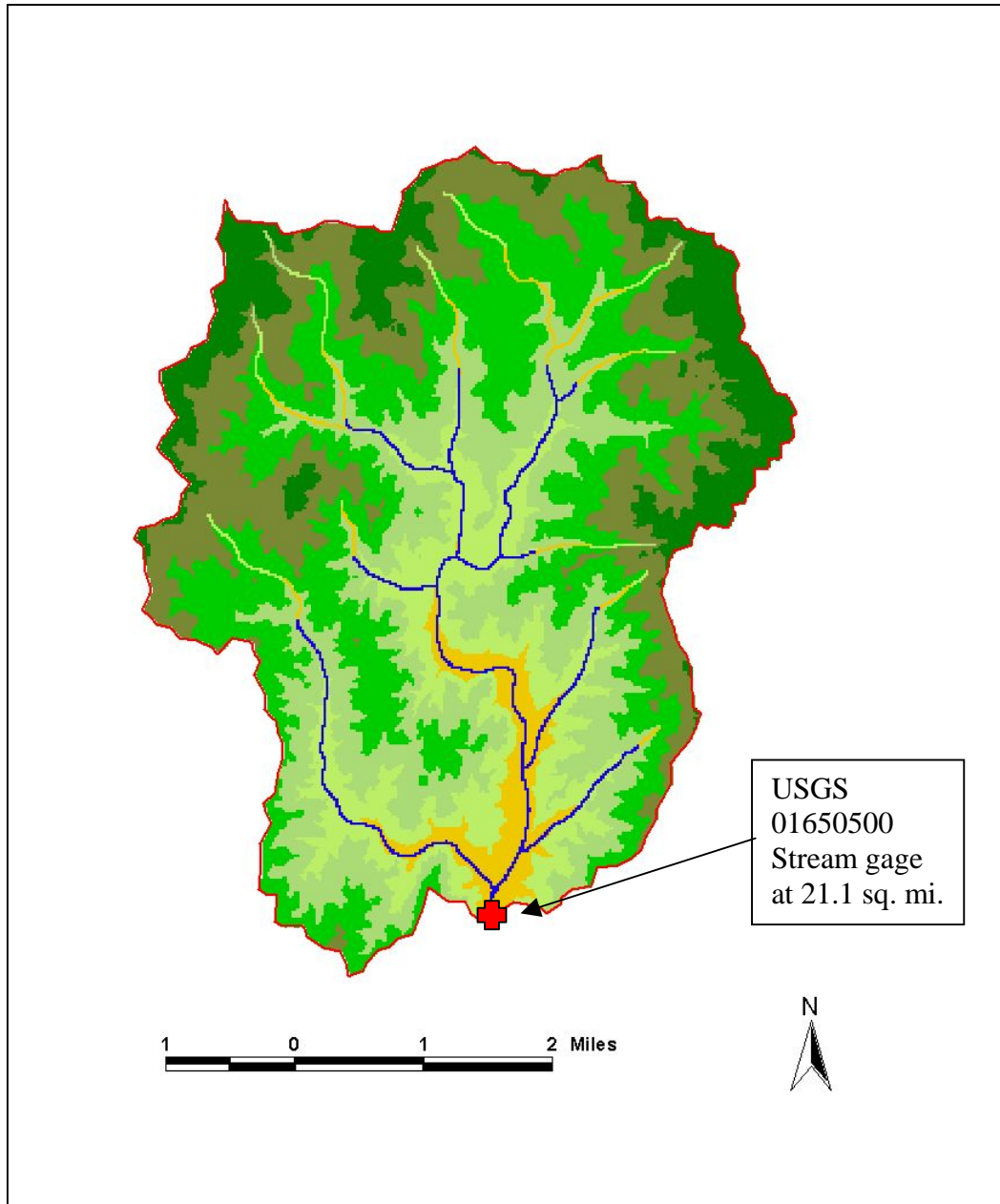


Figure 3.2. Watershed topography and stream gage location for USGS gage 01650500 Northwest Branch Anacostia River near Colesville, MD.

### **3.2.3. Land use changes**

Urbanization is the change of land use from forested or agricultural uses to residential/commercial uses. Urbanization is generally linked to an increase in the imperviousness. A common measure of urbanization is the overall increase in urban land uses or impervious area. Using GIS and more detailed land use data as provided by the GIS linked tax-map data from the MD property view dataset provided by the Maryland Department of Planning enables the determination of the spatial distribution of imperviousness at an annual time step (Moglen and Beighley, 2002). A series of digital land use representations were developed using the methods of Moglen and Beighley for each watershed for a period of years from 1948 through 2000 (Figure 3.3).

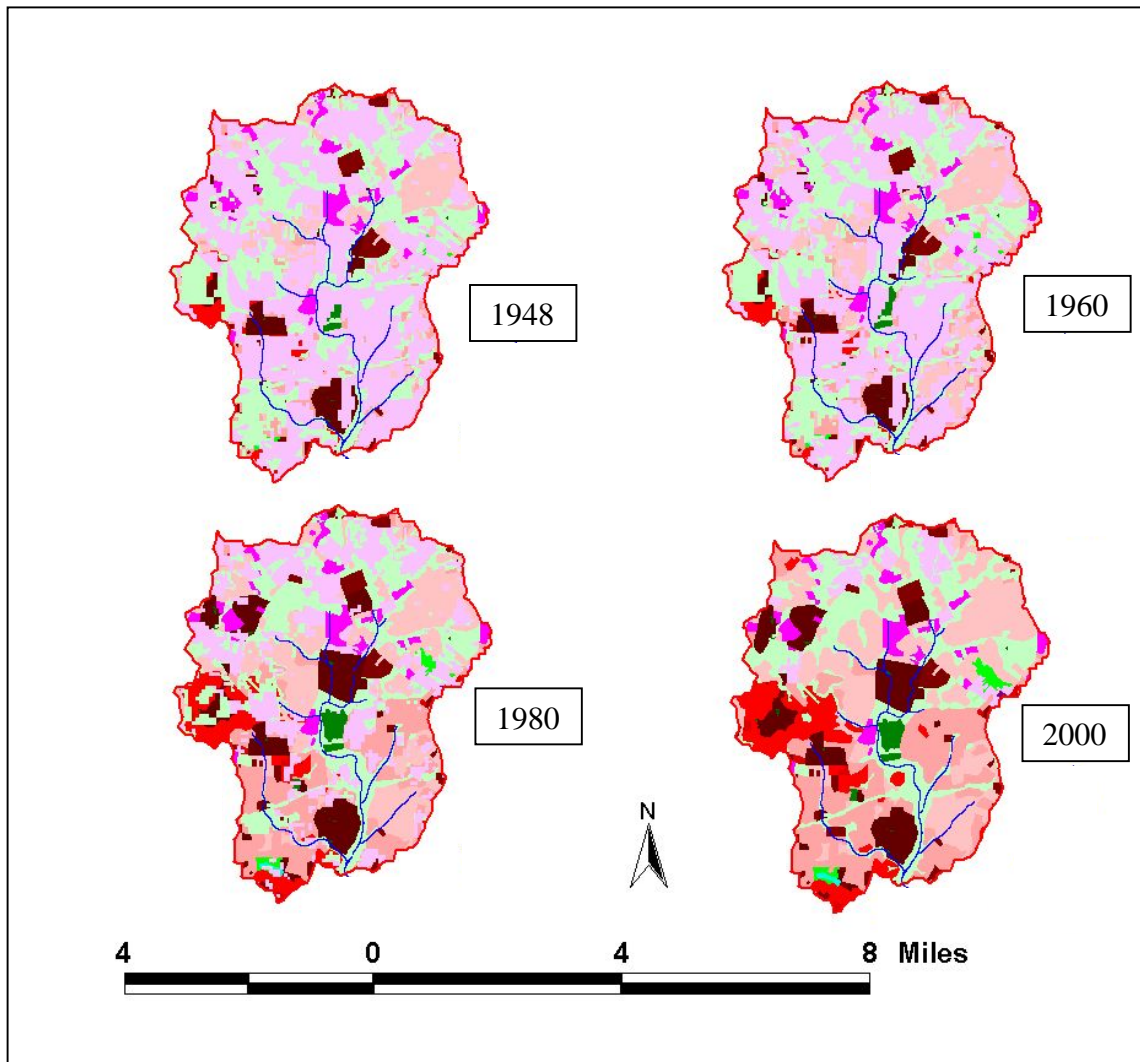


Figure 3.3. Change in land use distribution in Northwest Branch of the Anacostia River near Colesville from 1948 through 2000. Dark areas represent urbanized land use (e.g., highly residential and commercial) light colored areas represent forested and agricultural land uses.

The Northwest Branch of the Anacostia River land use change analysis showed that there was a different land use development ratio in three sub-watersheds (Figure 3.4). Taking advantage of the method used in subdividing the sub-watersheds for the SWMM model, data for these three sub-watersheds and for the entire Northwest Branch of the Anacostia River watershed were analyzed and compared throughout this study. Figure 3.4 shows sub-watersheds 9, 10, 11 and 1029. These sub-watershed identification

numbers are generated by the GIS interface. The Northwest Branch of the Anacostia River watershed is identified by 1 and it is shown enclosed by the bold boundary in Figure 3.4. The Northwest Branch of Anacostia River watershed identified as 1 includes sub-watersheds 9, 10, 11 and 1029. Sub-watershed 11 was not used in this study as a separate sub-watershed discharges for this sub-watershed include the effect of land use from sub-watersheds 9, 10 and 1029.

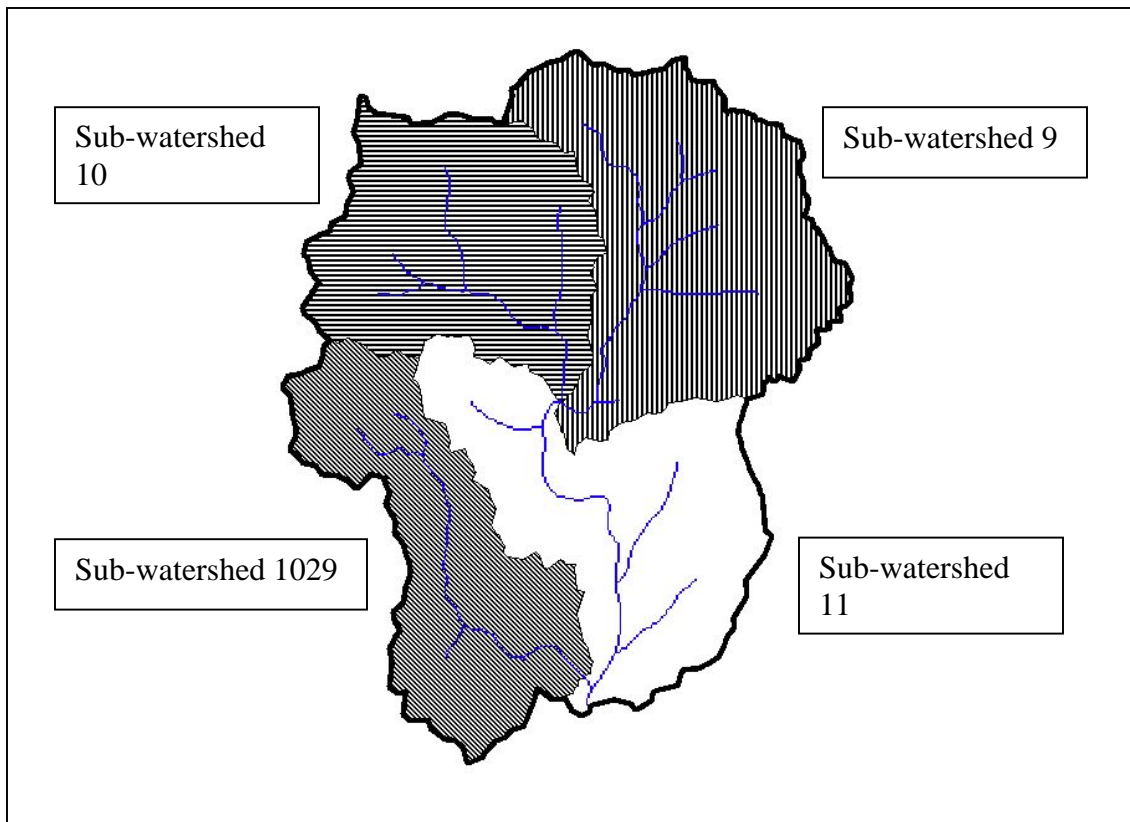


Figure 3.4. Major sub-watershed classification for the Northwest Branch of Anacostia River study. Sub-watersheds 9, 10, and 1029 drain to independent outlets. Sub-watershed 1 is defined as the entire Northwest Branch of Anacostia River (e.g., covers sub-watersheds 9, 10, 11, and 1029).

#### **3.2.4. Soils data**

The United States Department of Agriculture, National Resources Conservation Service (USDA NRCS) has digitized soils maps for different areas in Maryland. The Northwest Branch of the Anacostia River is located in Montgomery County, which is one of the Maryland counties where high-resolution soil information can be obtained in digital format. The NRCS provides information about the soil type, hydrologic soil condition and other soil characteristics required by SWMM. This information is contained in the Soil Survey Geographic (NRCS NCDC National SSURGO Data, 2001) (Figure 3.5), which has been digitized and reviewed for approximately half of the counties in Maryland.

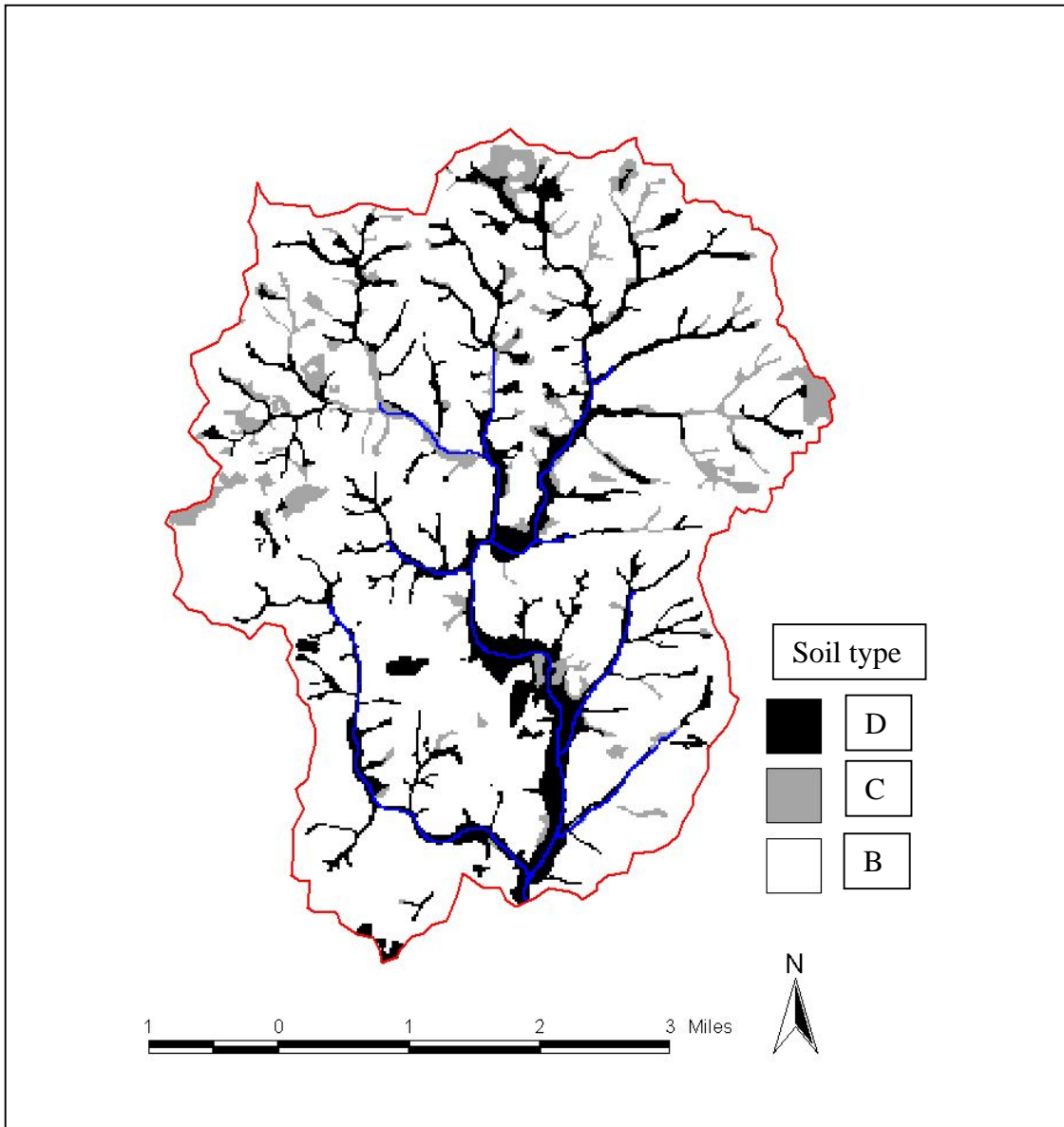


Figure 3.5. Hydrologic soil types for Northwest Branch of the Anacostia River.

Other parameters were averaged based on the soil type using relational tables developed by Dingman (1977). Soils were classified to be sandy, loamy or clay and values for groundwater flow estimation parameters (e.g., hydraulic conductivity, porosity, field capacity) were based on these classifications (Figure 3.6).

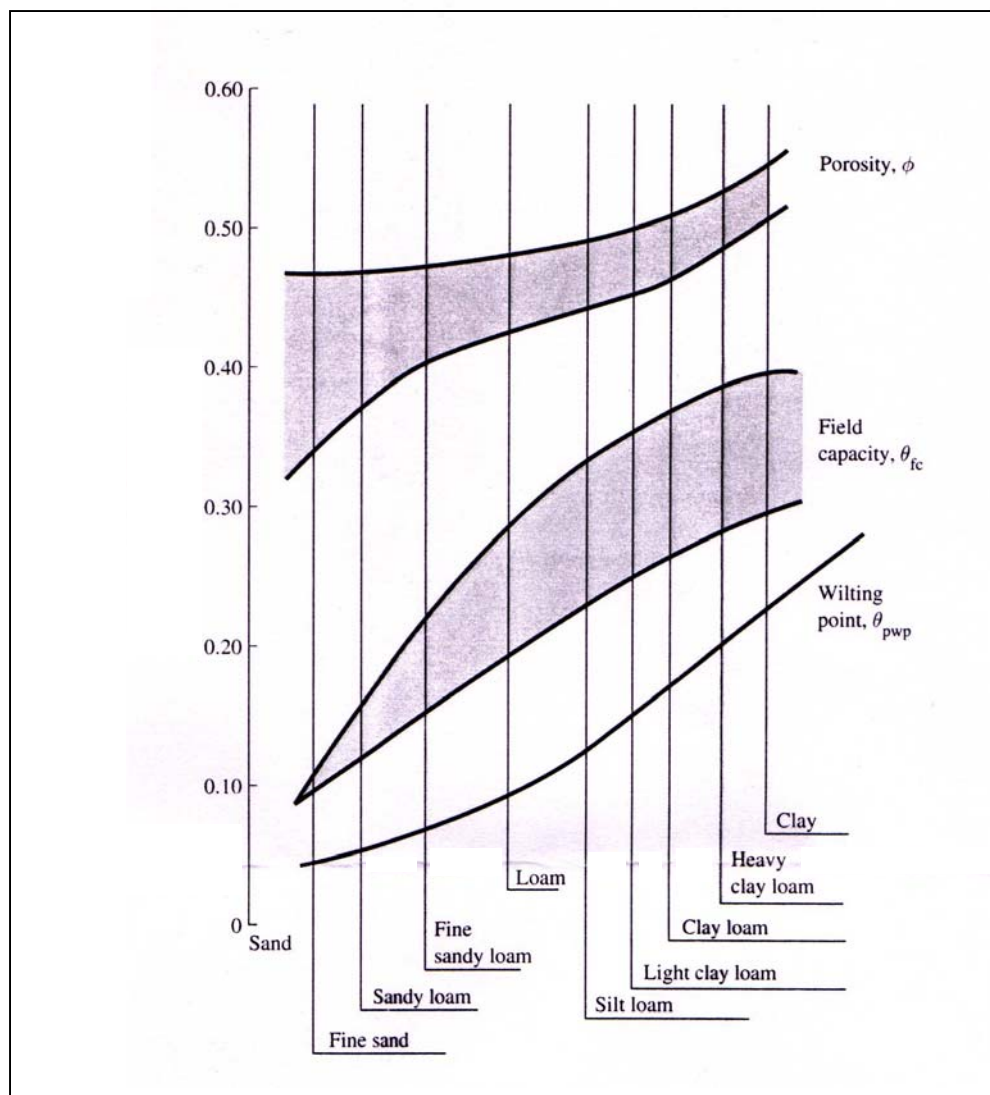


Figure 3.6. Groundwater flow parameters as a function of soil texture (Dingman, 1977).

### 3.3. Sub-watershed classification

The SWMM manual allows the watershed to be modeled in smaller areas, which are referred to as catchments, in this study we have referred to them as sub-watersheds. We selected these sub-watersheds so the entire watershed would be divided in three areas with different development trends. These sub-watersheds were identified by the number of the outlet to which they were draining. Table 3.1 shows some characteristics of these sub-watersheds and for the whole watershed of Northwest Branch of Anacostia River.



Table 3.1. Characteristics of the Northwest Branch of Anacostia River sub-watersheds as shown in Figure 3.4.

Sub-watershed ID	Area (sq. miles)	Imperviousness	
		1979	1988
1029	4.49	26.2%	32.8%
10	4.91	7.0%	10.7%
9	5.75	10.1%	11.1%
1	21.2	14.4%	18.7%

Four scenarios of land use change through the period under analysis were represented in the imperviousness data. The watersheds can be categorized in terms of current level of development and rate of development. Sub-watersheds 9 and 10 have a low level of imperviousness, although watershed 10 experiences a higher development rate over the study period. Sub-watershed 1029 is highly developed and continues at a high development rate over the study period. The overall watershed (1) is moderately developed with a moderate development rate.

### 3.4. SWMM interface

In order to take advantage of the digital information, a GIS interface was created to process and generate SWMM input files reflecting several different scenarios of land use which allow the calibration of the SWMM model and the generation of data to define the proposed models of changing land use. The SWMM model with annually changing land use scenarios will use hot-start simulations (a term which will be defined later on in this section). Hot-starts initialize state variables in each simulation with values taken from the end state of the previous simulation. Other tools were developed to extract data from SWMM output files into text and information readable by a spreadsheet application.

### **3.4.1. Hot-start simulation**

In this study, hot-start simulation is defined as a process by which some variables are initialized using previous simulation results. The values of the internal modeling variables represent the final state that the model reached at the end of the simulation period. Some of these values are written into the output file for each simulation period. Hot-start takes the available values and initializes the corresponding variable initial values at the start of the next simulation period. These variables are defined as state variables.

The procedure described in the previous paragraph enables the model to reduce the time period in which the model initializes the state variables (model spin-up) by giving them the final value corresponding to the previous simulation end. In a continuous simulation the values of the state variables are continuously updated within the model; in the hot-start method most of these values are updated outside the model simulation and plugged into the input file as initial values for the state variables for the next simulation period.

In order to update the groundwater conditions, the hot-start interface reads the water surface elevation and moisture content from the SWMM output files. The interface then initializes the corresponding state variables in the input file for each one of the 55 sub-watersheds (Figures 3.7 and 3.8).

```

*****
* Subsurface Summary for Stage, Soil Moisture, *
* and Groundwater Flow *
*****
* Flow from Subcatchment # to Channel/Pipe # *
*****
302 to 2

```

Mo/Da/Year	Stage Hr:Min	Soil Feet	Flow Moisture	cfs
-----	-----	----	-----	-----
10/ 1/1979	1 0	479.484	0.200	0.000
10/ 1/1979	1 15	479.484	0.202	0.000
10/ 1/1979	1 30	479.484	0.205	0.000
.				
9/30/1980	21 0	479.572	0.130	0.001
9/30/1980	22 0	479.572	0.130	0.001
9/30/1980	23 0	479.572	0.130	0.001
<b>10/ 1/1980</b>	<b>0 0</b>	<b>479.571</b>	<b>0.130</b>	0.001
Flow wt'd means.....		480.544	0.222	0.009
Flow wt'd std-devs...		0.000	0.182	0.005
Maximum value.....		480.938	0.470	0.013
Minimum value.....		479.452	0.124	0.000
Total loads.....	2.721E+05			
Cubic-feet				

Figure 3.7. Detail of one of the SWMM simulation output file for sub-watershed 302 year 1979. Values of stage (water surface elevation) in feet and soil moisture in inches were taken at 10/1/80 at 0 hours (shown in bold) and set as input data for the same sub-watershed for the period starting on 10/1/80.

```

.
.
H1 1 302 2 11123 19.2 5 0.02113 0.04 0.30 0.05 0.10 0.21 0.19 0.00017 0 0 0
*====*
*   BELEV*   : Elevation of bottom of water table aquifer, ft [m].
*   GRELEV*  : Elevation of ground surface, ft [m].
*   STG*    : Elevation of initial water table stage, ft [m].
*   BC*      : Elevation of channel bottom or threshold stage
*             for groundwater flow, ft [m].
*   TW*      : Channel water influence parameter
.
.
* GROUNDWATER DATA
* NMSUB NGWGW ISFPF ISFGF BELEV GRELEV STG  BC  TW
H2 302      2      1      0      479   480.935 479.571 479 479.484
*====*
=*   Input Groundwater Flow Coefficients And Exponents from
*       (Equations X-24 and X-25) on line H3.
*====*
*   H3 Line   :
*   A1*       : Groundwater flow coefficient, in/hr-ft^B1 [mm/hr-m^B1].
*   B1*       : Groundwater flow exponent, dimensionless.
*   A2*       : Coefficient for channel water influence,
*             in/hr-ft^B2 [mm/hr-m^B2].
*   B2*       : Exponent for channel water influence, dimensionless.
*   A3*       : Coefficient for the cross product between groundwater
*             flow and channel water, in/hr-ft^2 [mm/hr-m^2].
*   POR*      : Porosity expressed as a fraction.
*   WP*       : Wilting point expressed as a fraction.
*   FC*       : Field capacity expressed as a fraction.
*   HKSAT*    : Saturated hydraulic conductivity, in./hr [mm/hr].
*   TH1*   : Initial upper zone moisture expressed as a fraction.
*====*
*   A1  B1  A2  B2  A3  POR  WP  FC  HKSAT  TH1
H3 1.0E-4 2.7 0 1 1.0E-4 0.47 0.13 0.28925 5.0 0.130

```

Figure 3.8. Schematic of runoff input file with initial variables for water elevation and moisture content in bold. Value of water stage is equal to (479.571 ft. minus 479 ft. that is the actual elevation of the bottom of the channel as all elevations were selected to be referenced to the bottom of the channel).

### 3.4.2. SWMM scenarios

Three scenarios were analyzed: (1) a single 10-year continuous simulation years 1979 through 1988 with constant land use, (2) 10-year simulations using the hot start approach to initialize the model for each year (constant land use model with hot-start every year), (3) annually changing land use scenario with hot-start every year (Table 3.2). Constant land use scenarios assumed that land use corresponding to each year remained unchanged through the period 1979 through 1988. Each one of these models are described below.

Table 3.2. Different simulation scenarios in SWMM used in this study.

Scenario	Land Use	Hot-start
1	Annually constant	No
2	Annually constant	Yes
3	Annually varying	Yes

The first scenario was used to calibrate the SWMM model. This is the way the SWMM model is applied at the present: obtain SWMM model results for a certain period based on a certain land use distribution and calibrate parameters (surface detention, Manning's n, watershed width, etc.) to reflect observed discharges (peaks and baseflow). The results from the first scenario were used to compare the hot-start simulation procedure, as the results from second scenario should be very close to the results of the first scenario if the hypothesis that the hot-start model does not alter the results of the SWMM model. The results from the third scenario were used to analyze the impact of the varying land use in water quantity and water quality by comparing them with the results from the second scenario.

The third scenario was used so that both effects would be assessed: (1) hot-start simulation and (2) annually changing land use. Results from the third scenario were used to develop simplified adjustment methods for peak flow discharges, baseflow discharges, and pollutant loadings.

### **3.5. Imperviousness**

One of the most effective ways of quantifying development is through the change in imperviousness; therefore, the different scenarios of land use were represented by their corresponding imperviousness fractions. These imperviousness fractions were assigned based on the values utilized in the runoff curve numbers for urban areas (Soil Conservation Service, 1986) and the land use distribution for each year (Moglen and Beighley, 2002). Figure 3.9 shows the values of imperviousness determined for the three sub-watersheds and the overall value for the Northwest Branch of the Anacostia River for the years 1979 through 1988.

### **3.6. Calibration**

Statistical methods can be used to achieve two different goals in hydrology, description and inference. This study is model prediction oriented; therefore, goodness-of-fit measures presented here are based on the linear relationship expressed by:

$$Y_i = a*Y_e + b \tag{3.1}$$

where:

$Y_i$  = Discharge field measurement.

$Y_e$  = SWMM model simulated discharge in ft<sup>3</sup>/s .

The goal is to have “a” as close to 1 and “b” as close to zero as possible using rational parameter values within the SWMM model. Meeting this goal means that the SWMM model simulated discharge are a good estimate of the discharges measured in the field.

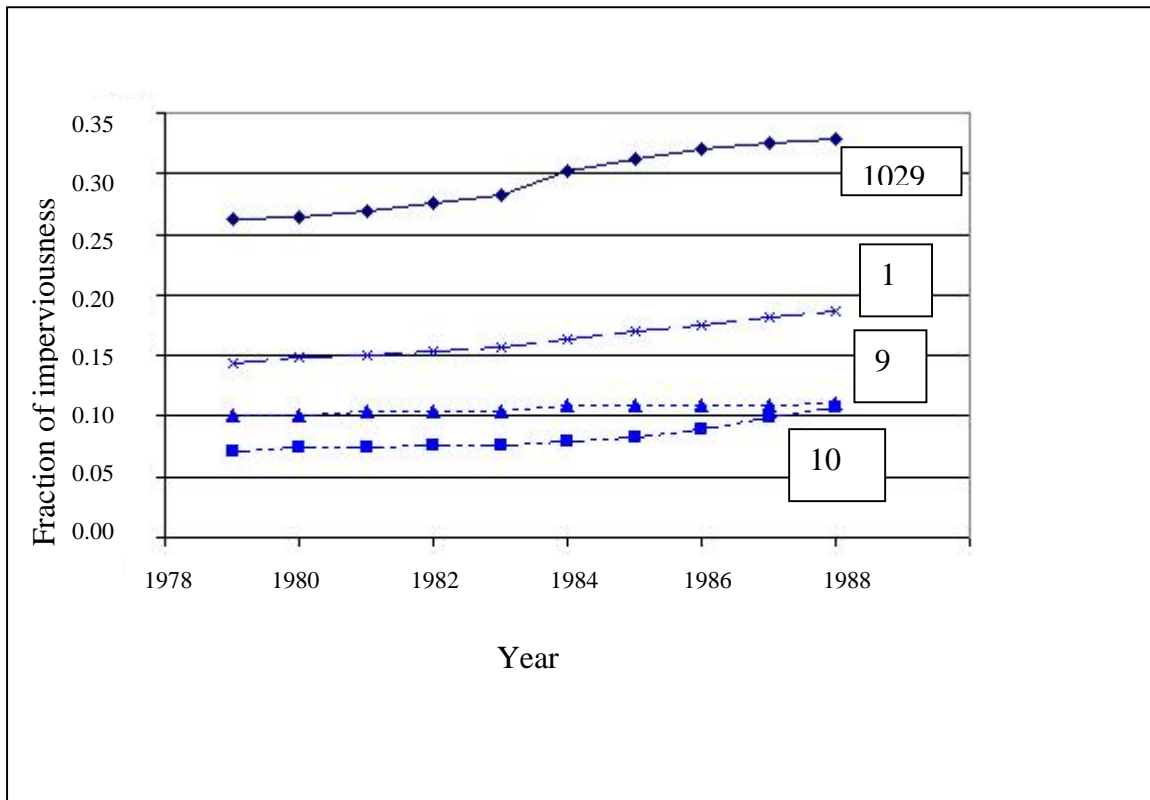


Figure 3.9. Variation of imperviousness for each sub-watershed (sub-watersheds are identified as 1, 9, 10, and 1029) of Northwest Branch of Anacostia River from 1979 to 1988.

As a first approach a time series analysis of the daily discharges normalized by the drainage area of each sub-watershed was performed. The differences in these normalized discharges were large in some cases because peaks and baseflow can differ

greatly. Therefore, two different analyses were performed: one for peak discharges and the other one for baseflows.

In order to be able to compare results from different sub-watersheds and different storms, two types of unit discharges are defined in this study: one for peak discharges and one for baseflow. These two types of quantities attempt to represent normalized discharges from the four sub-watersheds so they are comparable to each other. The unit peak discharges were defined for the largest storms as the peak discharge divided by the drainage area of each watershed and by the total precipitation of the storm that generated this discharge as shown in equation 3.2.

$$Q_{unit} = \frac{Q}{A * I} \quad (3.2)$$

where:

$$Q_{unit} = \text{Unit discharge per area per inch of rain in } \frac{\text{ft}^3}{\text{s} * \text{sq.mi.} * \text{inch of rain}}$$

$$Q = \text{Daily discharge in ft}^3/\text{s}.$$

$$A = \text{Area in square miles.}$$

$$I = \text{Total precipitation in inch of rain for each storm event.}$$

In a similar way, the unit discharge for baseflow was defined as the discharge divided by the drainage area. Precipitation was not considered in the baseflow analysis because there is not a direct relation between baseflow and precipitation.

$$q_{unit} = \frac{Q}{A} \quad (3.3)$$

where:



$q_{unit}$  = Baseflow unit discharge per area in  $\frac{ft^3}{s * sq.mi.}$ .

$Q$  = Daily discharge in  $ft^3/s$ .

$A$  = Area in square miles.

Daily flows depend on the seasonal factors. Winter season will have higher precipitation than summer season in some areas. Therefore the timing of the storms and the seasonal effect cannot be neglected. Considering that discharges depend on the time of the year and the previous day storm events, independence of discharges is not a valid assumption in this study. Therefore, it was necessary to consider goodness of fit methods that take into consideration these effects.

The timing of simulated hydrograph peaks may not always coincide with observed hydrograph peaks. This can be due to rainfall that occurs near the midnight hour and is, hence, potentially distributed across two days. Further, the different time steps between the rainfall and the observed hydrograph data can lead to discrepancies in the timing of the simulated hydrograph peaks (Figure 3.10). In this figure the white bars represent daily averages of simulated discharges with precipitation time step of 15 minutes. Inside the circled area it can be seen that the peak for this particular storm occurred on the day 13 from the start of the selected period. Dashed bars show the simulated discharges using a precipitation time step of 6 hours. It is shown inside the circled area that the peak storm for this particular case occurs one day before, on day 12 when using a 6 hour rainfall time step. Peak discharge has been shifted because a change in the precipitation time step. This was found to be mostly the case when storm peaks

occurred late at night, but considering the differences in discharges (peak and baseflow) the errors introduced by this shift in time can be large.

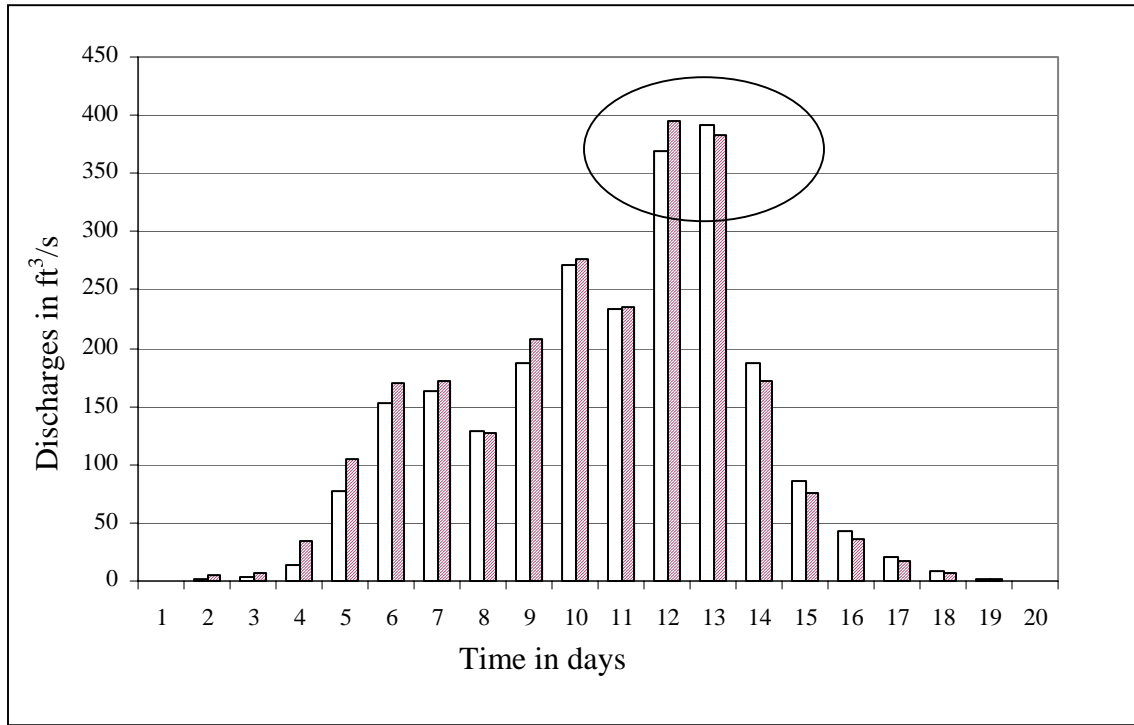


Figure 3.10. Shift of peak discharges from one day to another due to time step in precipitation data. White bars correspond to discharges simulated using a 15 minute time interval precipitation, while dashed bars correspond to discharges simulated using a 6 hour time interval precipitation. The shift of peaks from one day to the next day is shown inside the circled area.

Three measures were used in this research: (1) the deviation volume coefficient (ASCE, 1993), (2) the cumulative distribution function of discharges, and (3) the Nash-Sutcliffe index (ASCE, 1993). These three measurements, which were described in Section 2.5 (Goodness-of-fit methods), were used to analyze the goodness-of-fit between the modeled series of discharges and the observed discharges.

### **3.7. Simulation scenarios**

The study of the effect of changes in land use on water quantity and water quality was carried out through the use of the SWMM model (Huber et al., 1992). The SWMM model was used to simulate three different scenarios for three separate sub-watersheds within the Northwest Branch of the Anacostia river watershed. SWMM results included estimates of hourly discharges and nitrate pollutant concentration. The three scenarios were described earlier in Section 3.3.2 and Table 3.2.

The first scenario simulation was performed following the commonly used calibration techniques suggested by the SWMM users list online discussion group, and the SWMM model was calibrated to the observed discharges (details of this calibration will be presented in Chapter Four).

The second scenario was designed to determine the error introduced by the hot-start method compared to the continuous simulation method. Errors in  $D_v$  and CDF (as defined in Section 2.5) were small enough so it was accepted that hot-start simulations were a good approximation than continuous simulation (again, details will be presented in Chapter Four).

The third scenario was used to actually develop the simplified adjustment methods that will be shown in the next chapter for adjusting discharges modeled from a constant land use simulation to reflect a changing land use without running hot-start scenarios, reducing the time, effort, and cost of performing the more complicated Scenario 3 procedure whenever land use change is present.

Figure 3.11 shows the impervious fractions used in each of the simulation scenarios. Each one of these impervious fractions corresponds to the year of the land use

being used in the scenario. The constant land use scenario uses the same impervious fraction throughout the entire simulation period. The dynamic annually changing land use scenario uses an annually varying impervious fraction.

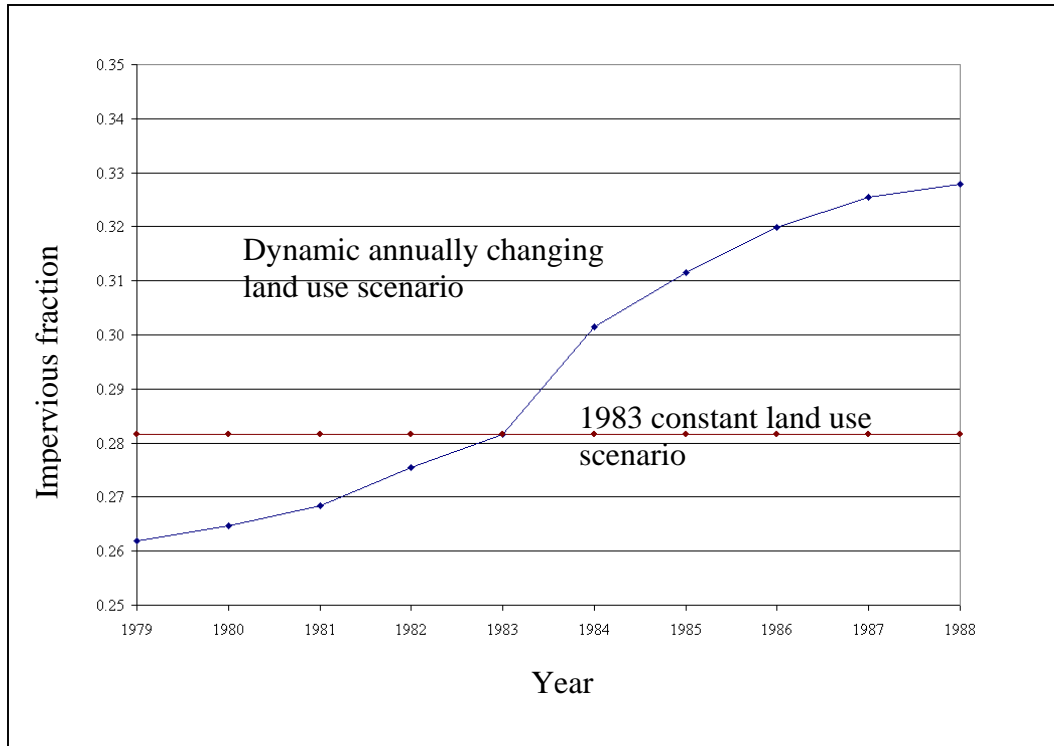


Figure 3.11. Impervious fractions used in the different SWMM scenarios. In this figure the 1983 constant land use impervious fraction is compared to the dynamic annually changing land use impervious fraction. Note that both scenarios have the same value for the year 1983. This is true for the other years 1979 through 1988.

### 3.7.1. Water quantity

In a similar way, the approach used to develop a simplified model for adjusting SWMM water quantity estimates considered two different flow conditions: (1) peak discharge and (2) base flow. Although the approaches are similar there are some fundamental differences when defining discharge variables as described earlier in the definition of equation 3.1 and 3.2.

### **3.7.1.1. Direct runoff discharges**

Many studies have shown that direct runoff peaks increase with higher values of imperviousness (McCuen, 1998). In order to develop a relationship for adjusting SWMM model estimates, the unit peak discharges for the constant hot-start land use scenario were plotted against the ratio of imperviousness corresponding to the land use for each year for some of the highest unit peak discharges as previously defined. Results for the constant land use scenarios are shown in Figure 3.12. From this graph it can be seen that, as a general rule, unit peak discharges are higher for greater imperviousness. Some storms were found to have an inverse relationship between the unit peak discharge and the imperviousness. After a further analysis it was found that these inverse relationships occurred in storms that followed a smaller storm one or two days before. They are referred to as composite storms and are omitted from this analysis.

Linear regressions were performed on each one of the selected storm peak discharges for the constant hot-start land use scenarios resulting from each one of the selected storms. Imperviousness was set as the independent variable and the unit peak discharge as the dependent variable.

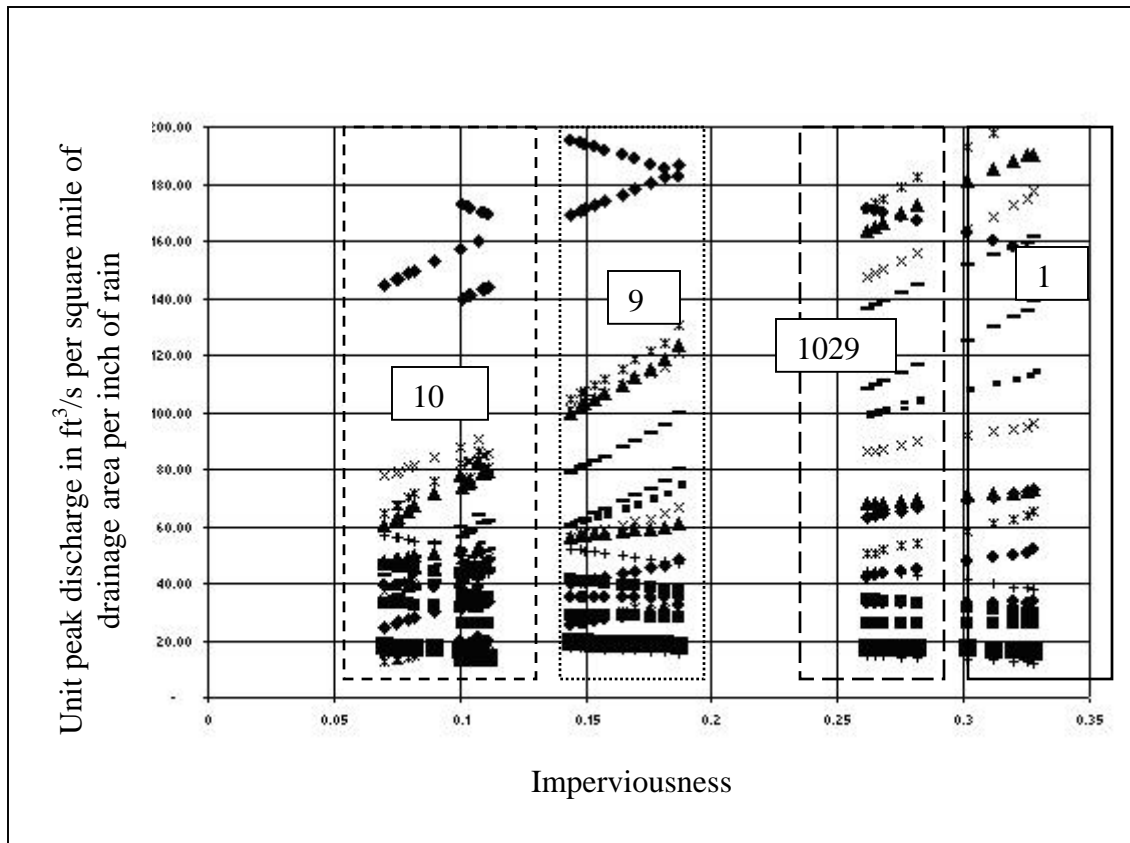


Figure 3.12. Northwest branch of Anacostia River main watershed and sub-watershed hot restart constant land use discharges vs. imperviousness for 24 peaks during the period 01/01/79 through 12/31/1988. (Each different plotting symbol corresponds to a different storm). Discharges from each major sub-watershed (1, 9, 10, and 1029 from Figure 3.4) can be identified by the different imperviousness fractions. Dotted box around the discharges are from sub-watershed 9, short dashed line box are from sub-watershed 10, long dashed line box are from sub-watershed 1029 and continuous line box are from sub-watershed 1.

The values of the slopes of the relationships in Figure 3.12 for each individual storm were plotted against the unit peak discharges in units of  $\text{ft}^3/\text{s}$  per square mile per inch of rain, which were obtained from the annually changing land use scenario, as shown in Figure 3.13. A linear regression was performed for each watershed and each storm relating each one with the corresponding imperviousness. Results will be discussed in greater detail in Chapter Four. From this graph it can be seen that the change in unit

peak discharge is more sensitive to changes in imperviousness when watersheds have a higher percent of development.

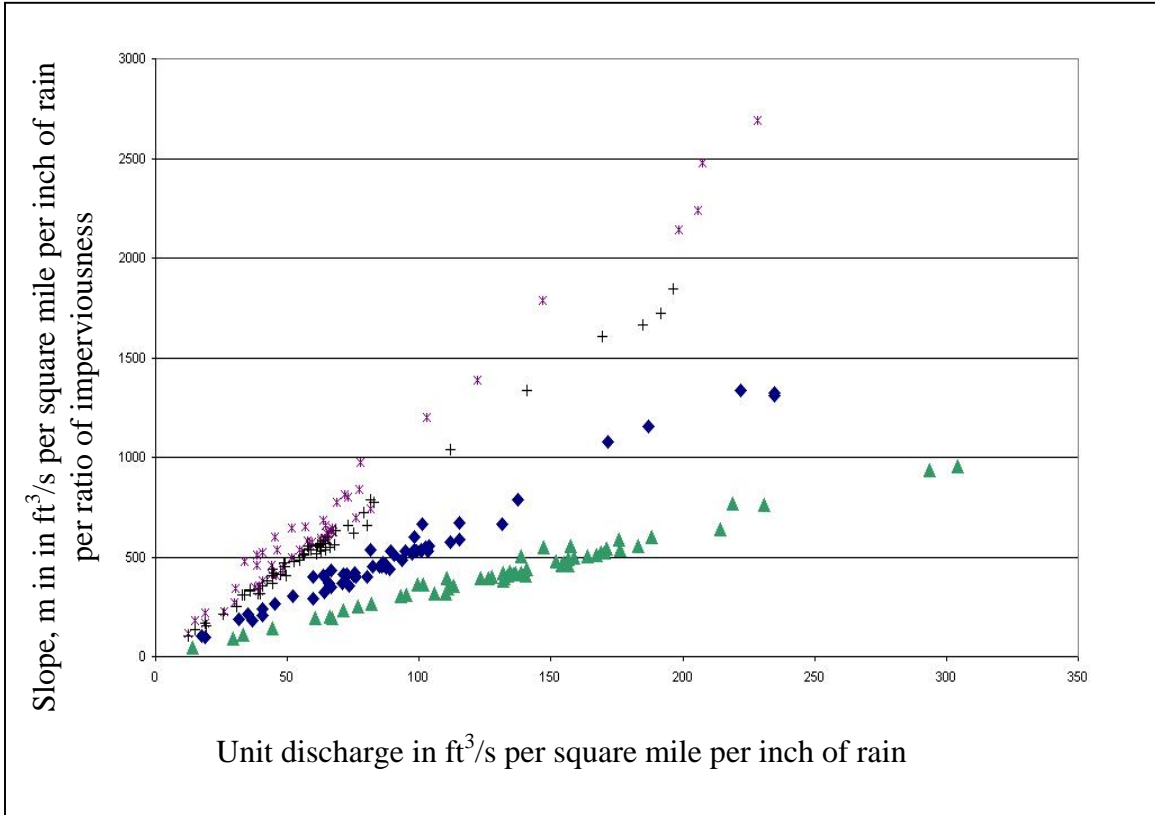


Figure 3.13. Constant land use unit discharge slope coefficient vs. dynamically changing land use unit discharge for Northwest Branch of Anacostia River. “x” are for most developed watershed, triangles are for least developed watershed.

### 3.7.1.2. Baseflow

Unit discharges relevant to the development of a simplified model for baseflows were derived using equation 3.2. Unit discharges were calculated for each hot-start constant land use scenario and changing land use scenario. Then the ratios of unit peak discharges from hot-start constant land use and the annually varying land use discharges were calculated. These ratios were plotted against the ratios of imperviousness for each year of constant land use to the imperviousness for the year corresponding to the

discharge. Results are shown in Figure 3.14. In this figure the ratio,  $R_{Qu}$  is defined as follows:

$$R_{Qu} = \frac{Q_{uc}}{Q_{uv}} \quad (3.6)$$

where:

$R_{Qu}$  = Discharge ratio between  $Q_{uc}$  and  $Q_{uv}$ .

$Q_{uc}$  = Discharges resulting from the simulation with constant land use (Scenario 2).

$Q_{uv}$  = Discharges resulting from the annually varying land use (Scenario 3).

The ratio  $R_{imp}$  is defined as follows:

$$R_{imp} = \frac{imp_c}{imp_v} \quad (3.7)$$

where:

$R_{imp}$  = Ratio between imperviousness dimensionless.

$imp_c$  = Imperviousness corresponding to the year of the land use from the constant land simulation scenario in acres of impervious land use per total acres.

$imp_v$  = Imperviousness for each year of each discharge in acres of impervious land use per total acres.

Four extreme value (envelope) lines were estimated as linear regression equations from the discharge ratios and the imperviousness ratios such that all points are included



inside these boundary lines. These envelope boundary lines cross at (1,1). This means that discharges resulting from annually varying land use were closer to the discharges from each constant land use simulation for the year of the land use distribution used in the constant land use simulation, because the imperviousness, that is the only changing parameter, is the same for that certain year. As an example, annually varying land use discharges for year 1979 are close to constant 1979 land use discharges for the year 1979 because imperviousness is the same for both scenarios in that year. Simulated values from these two curves were adjusted later on by a factor depending on the imperviousness of each sub-watershed, which is related to the families of curves shown in Figure 3.14. Results will be discussed in Chapter Four.

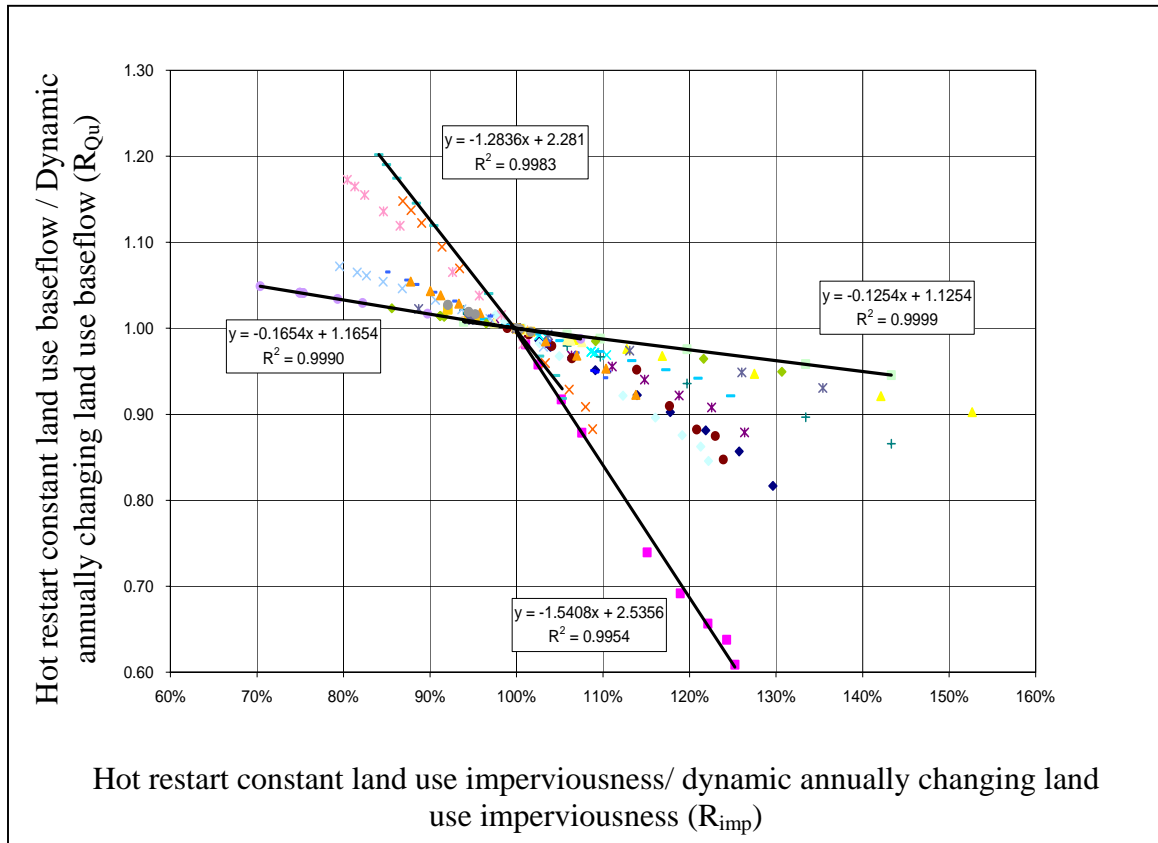


Figure 3.14. Ratio of baseflow from dynamically changing land use model and baseflow from hot-start constant land use versus the ratio of the imperviousness of the dynamically changing land use cover and the imperviousness of the hot-start land use cover.

### 3.7.2. Water quality

The pollutant simulation method selected for this research was based on event mean concentrations (EMC). This method uses an exponential build up of pollutant with time, and assumes linear wash off by rain into the streams. This method was used to simulate pollutant concentrations for four pollutants (nitrate, phosphorus, total nitrogen, total suspended solids) being modeled. Therefore, conclusions arising from the analysis of one of the pollutants (e.g., total phosphorus) can be applied to any other pollutant for which the EMC build up and wash off method is appropriate.

SWMM produces a detailed output with pollutant loads for each of the selected time steps over the entire period of modeling. Daily time step for years between 1979 and 1983 for each sub-watershed were used in this study.

The land use distribution for each year was used to define the different scenarios. Therefore, the above values were obtained for each land use, and consequently for each level of imperviousness. An average pollutant load per unit area was obtained by dividing pollutant loads from each sub-watershed by the corresponding drainage area in square miles. In this manner it was possible to compare loads between the different sub-watersheds. Changes in average pollutant load were plotted against the change in imperviousness for each different year and land use scenario as shown in Figure 3.15 and these data were fitted by a regression equation.

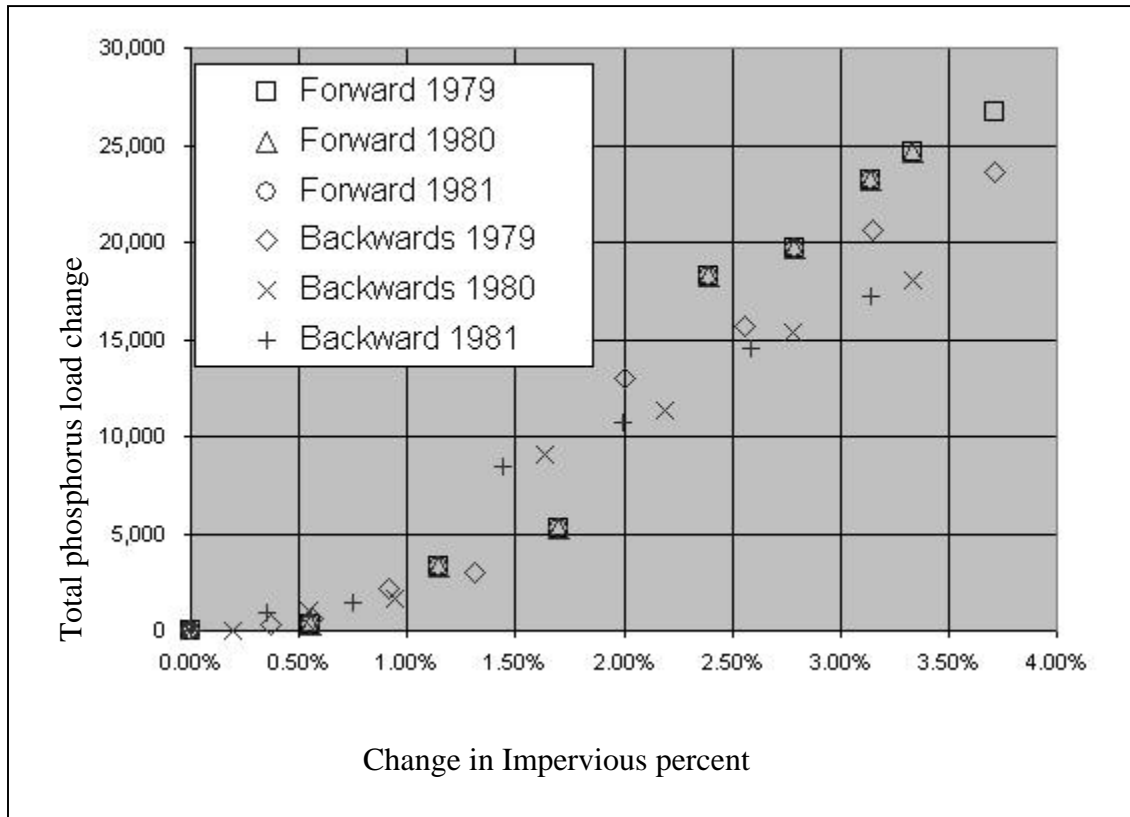


Figure 3.15. Total phosphorus (TP) load change vs. change in imperviousness for the Northwest Branch of the Anacostia River. Forward data refer to comparisons from 1979 with 1980 and 1981, and backwards refer to comparisons from 1981 to 1980 and 1979 pollutant load simulations.

### 3.8. Summary

Four sub-watersheds were modeled using the SWMM model to simulate three different scenarios: (1) different land use distributions were simulated continuously keeping land use constant for each year of the modeled period (the SWMM model was calibrated using these results), (2) several years were modeled with a constant land use hot-start method in order to verify that hot-start improves the simulations from constant land use model results, and (3) discharges for the same period of time using the annually varying land use scenario with hot-start method. This scenario allows for the comparison

to the observed conditions value each year and the development of proposed models that adjust discharges from a constant land use scenario to an annually varying land use scenario.

Three model outputs were analyzed: (1) peak discharges (2) base flow discharges and (3) pollutant concentration in each sub-watershed. An adjustment method was developed for peak discharges, baseflow, and pollutant load that accounts for land use changes without using the hot-start method. The procedures developed to adjust water quantity and water quality give a better estimation of these variables by taking into consideration the temporal changes in the land use distribution. The strength of this methodology is that results from the simple model can be modified to reflect the effects of annually varying land use.

The focus of this research is to analyze the effect of land use change on water quantity and water quality. With this in mind three models were developed from SWMM generated time series of discharge and pollutant loads. In the next chapter the results obtained, problems encountered, and the assumptions that were made to overcome such situations will be described. Each model related one variable (either runoff peak discharge, base flow discharge, or pollutant loading) with the change in imperviousness either as a percent or as a ratio. Models will be discussed in more detail in Chapter Four stating assumptions that were made and restrictions that were presented in this chapter.

## **Chapter 4**

### **Results**

#### **4.1. Introduction**

In the previous chapter the methodology followed in this study was presented. In this chapter an analysis of the calibration, goodness-of-fit, model results, and some of the limitations that are associated with these steps will be presented. Most of these limitations follow from the assumptions that were made when modeling and calibrating the different SWMM models. Others are due to the parameter estimates when modeling the watersheds like permeability, surface roughness, and water depression storage.

Two models were developed in this thesis work: one to adjust water quantity and the other to estimate pollutant load from constant land use SWMM model results to annually changing land use. These models can estimate discharges and pollutant loading resulting from annually changing land use using constant land use modeled time series generated from SWMM and then adjust them according to the changes in imperviousness resulting from changes in land use.

Chapter Four is divided in four sections as follows: Analysis of uncertainties, Calibration process, Results of simulations, and the Application of the models. The first section deals with the errors introduced in the model results because of either input data or data used in the calibration process. The second section is related to the results from each of the calibration methods. The third section presents the two models developed

explaining the restrictions that apply. The last section presents the conclusions of this chapter.

## **4.2. Analysis of uncertainties**

Models include several uncertainties from the input data as well as from the mathematical representations of the processes involved. Hydrologic and hydraulic models require several input data and they involve representations of various phenomena: infiltration, evaporation, saturation, runoff, etc. Therefore hydrologic modeling is subject to a wide variety of uncertainties.

There are many sources of uncertainties, at least one for each source data, for each model of a hydrologic and hydraulic process as well as for each parameter involved. Three main sources of uncertainties have been identified: recorded data, theoretical data and the modeling process.

### **4.2.1. Recorded data**

#### **4.2.1.1. Precipitation**

In this study, rainfall is the only input of water into the system. No snow or recharge from groundwater is included in our models. Although there was a good precipitation record, some of the data were incomplete.

One of the tests that can be used to check data consistency between rain gages is the double-mass analysis. A double-mass plot essentially makes a graphical comparison between the records from two different stations. One station is intended to be a standard

station that has data believed to be representative. (Bras, 1990). In this study, four rain gages with longer periods located close to the area of study were selected.

The rain gage with the longest record and least missing data was selected to be representative for our study and compared to the average of the other three. Considering four rain gages provides the ability to test the data from the selected rain gage to be consistent and representative for the area. Nevertheless, the double-mass plot showed that data for all stations had some periods of missing data and therefore it was not possible to estimate the missing data during these periods.

The location of the rain gage is a major source of uncertainty. Ideally the rain gage should be located as close as possible to the area under study in order to be representative of the rainfall inside the watershed. If the rain gage is not representative of the real precipitation over the study area, the SWMM model results will not reflect properly the observed streamflow data. Based on the double-mass plot and the proximity of the rain gage to the Northwest Branch of Anacostia River watershed, the closest among the rain gages with one hour time interval (TD3240 NWS format) was selected to be used in this research.

Although the location close to the area of study is important, the spatial variation of rainfall inside the area of study has a strong influence in the model results. There are several methods that can be used to account for the rainfall spatial variation (Thiessen polygons, inverse distance square method among others), but they required continuous periods of data from rain gages that are spatially distributed. In this study, only one rain gage was considered as input into SWMM model, therefore no spatial variation was considered. The rain gage was selected based on a double-mass curve analysis that



showed that data from all pre-selected raingages were similar. Spatial variation is more significant for larger watersheds than it is for smaller ones. The assumption of uniform precipitation induces larger errors in larger watersheds than in smaller watersheds.

Double mass analysis helps in the general comparison between raingage data, a closer analysis on a daily basis was done for some of the peak storms recorded of two of the most complete data series of the rain gages located closer to the study area (181125 and 180700) (Figure 4.1). It was observed that even though these gages are located in the same area and that the double-mass analysis showed consistency between them, there are still different records for the same day. Figure 4.1 shows the cases of July 11 and 25, 1985. This difference in the record of precipitation of these two rain gages shows that although it was concluded that the data from the selected raingage was assumed to be representative for this area, certain peak storms may differ in time or space.

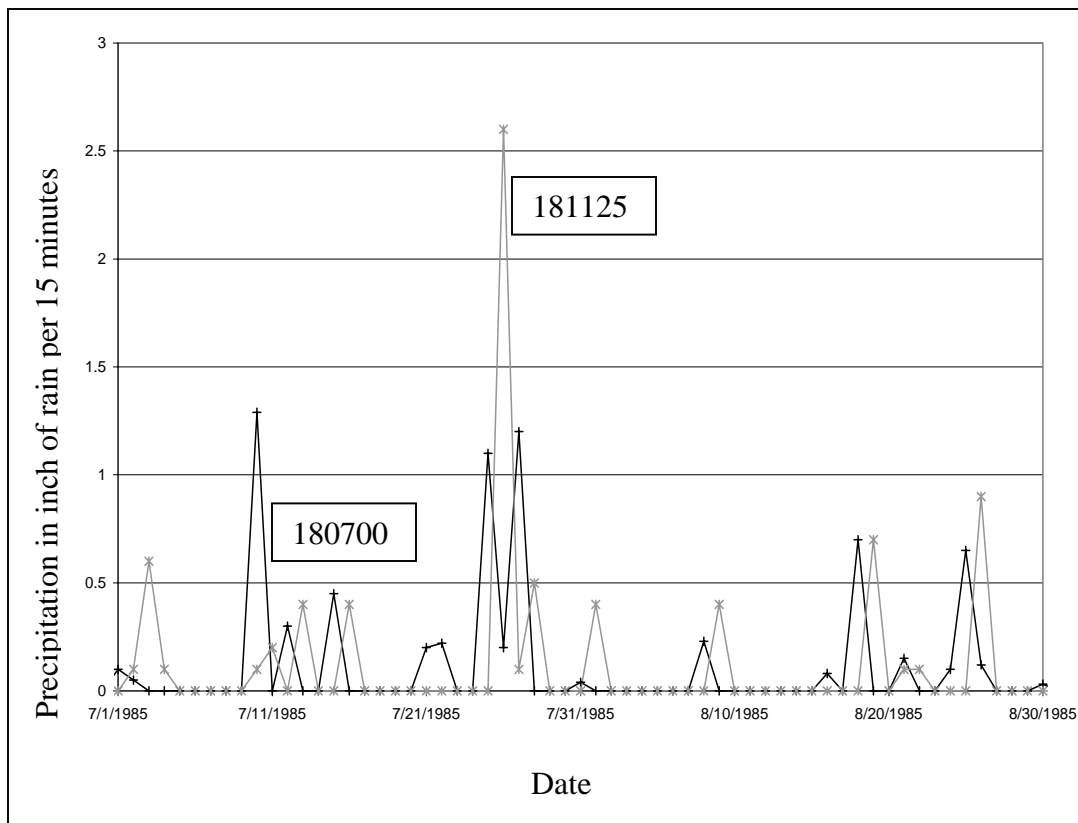


Figure 4.1. Rain gauges 181125 and 180700 data comparison. This figure illustrates typical differences in some of the storms registered. Station 180700 is nearer to northwest Branch of Anacostia River than rain gage 181125. Dark lines represent precipitation from rain gage 180700 and grey lines represent the precipitation from rain gage 181125.

The SWMM model divides the process in time steps, the smaller the time step the smaller the error introduced in the modeling process by minimizing the finite difference error. Discharge peaks from SWMM model were shifted from one day to the next when the time step of the precipitation data series was increased (Figure 3.11).

In order to analyze this shifting in peaks, synthetic precipitation data with time steps varying from 15 minutes to 12 hours based on original precipitation data from USGS 180700 rain gage were used. The SWMM model has been set up to generate daily

average discharges. Results indicated that the peak discharge was shifted from one day to the next day when a time step of 6 hours or more was used (Figure 3.11).

#### **4.2.1.2. Discharges**

Similar to precipitation, the discharge time series are published on time intervals. This study used discharges on a daily time step. In general, discharges are measured by either of two methods: (1) by referencing the water surface elevation readings to a rating curve or, (2) by directly measuring the cross-sectional flow area and mean velocity of the stream (Ponce, 1989). The development of a good rating curve is crucial to have an accurate represent the relation between water stage and discharge. Water elevation is measured by special devices (scales, wire gages, recording gages, pressure-actuated recorder, etc.), discharge measures require a measure of the channel characteristics (cross section, Manning's  $n$ , slope) (Ponce, 1989). Errors can be introduced in the water elevation reading method and also in the calibration curve.

There could also be a big difference between the instantaneous peak discharges and the average daily discharges (Figure 4.2). In this study daily average discharge recorded data from USGS was compared with model results. In order to make these two time series comparable, the runoff discharges were modeled from an hourly time step rainfall wit but estimated a daily average by changing the time step for the routing process.

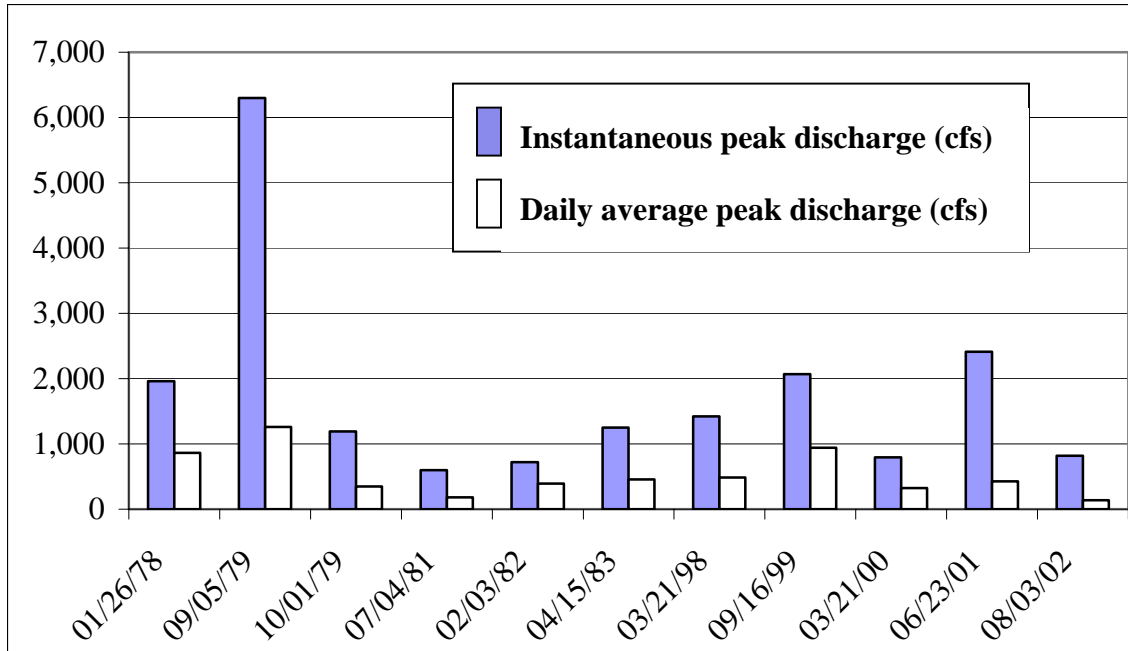


Figure 4.2. Difference between instantaneous peak discharge and daily average discharge for the Northwest Branch of Anacostia River, station 01650500 for selected dates within the modeled period. Data taken from the USGS water resources website (U.S. Geological Survey, 2000).

#### 4.2.1.3. Land use distribution in space and time

Most hydrologic models, including SWMM, assume a constant land use through the modeled period. The method used to estimate the variable annual changing land use was developed by Moglen and Beighley (2002) to generate a quantitative GIS representation of the changes in land use for each year. The improvements realized in discharge estimation when variable land use conditions are considered will be shown in the calibration section later on this chapter.

#### 4.2.1.4. Soil classification

Based on the soil classification included in the USDA soil survey (USDA, 1986) the different types of soils inside the watershed were classified in the categories shown in

Table 4.1. These categories were selected in order to match the ones from Dingman (1997) based on the soil description given in the soil study. Based on these soil types, the values of the parameters used to model groundwater flow (Ksat, porosity, wilting point) were estimated as detailed in Table 4.1. These estimates were made using the average value for each soil type taken from Figure 3.5. The values of these parameters were averaged based on the area inside each one of the sub-watersheds into which the watershed was subdivided. According to the SWMM manual (Huber et al., 1992), groundwater subroutines were added to the model but there are several limitations based on the uncertainties of the estimation of the parameters involved in groundwater models. In this study we selected Horton as the infiltration method. Horton is an empirical method developed based on the following behavior: under a given soil type and moisture content, there is an initial infiltration rate, as water infiltrates this rate decreases until it reaches an ultimate infiltration capacity (Bras, 1990). The other method for estimating infiltration available in SWMM is Green-Ampt, which is also an empirical method developed from Darcy-type water flux that includes a suction factor in the infiltration equation (Bras, 1990). The Horton method was selected based on the availability of data to set up the model. The SWMM manual suggests that both of these methods include considerable uncertainty about the estimations of parameters and the mathematical description of the flow processes involved. As a practical matter, a method needed to be selected and the Horton method was considered preferable.

Table 4.1. Groundwater flow model parameter estimation from Dingman (1997) and the soil classification given in the NCRS soil study for Montgomery County, Maryland (Natural Resources Conservation Service NRCS, National SSURGO Data, 2001).

Soil type	SSURGO soil code	Wilting Point volume fraction	Field Capacity volume fraction	Porosity volume fraction	Ksat in in/hour
Silty loam	16B, 16C, 16D, 19A, 19B, 1B, 1C, 21A, 21B, 21C, 21D, 21E, 22A, 22B, 23A, 25B, 25C, 26B, 26C, 27B, 27C, 29B, 2A, 2B, 2C, 35C, 36A, 37B, 41A, 41B, 4B, 4C, 54A, 57B, 57C, 57D, 59A, 59B, 5A, 5B, 65B, 6A, 9B, 9C, 43A, 45A, 46A, 47A, 48A, 50A, 51A, 53A, 109D, 109E, 116C, 116D, 116E, 24C, 24D, 18C, 18E	0.13	0.2893	0.47	0.00017
Gravelly Loam	61B, 61C, 61D, 61E	0.05	0.118	0.413	0.0176
Loam	17B, 17C, 58B, 58C	0.105	0.258	0.458	0.00069
Loamy Sand	55C	0.08	0.21075	0.444	0.00347
Outcrop Complex	21F				
Sandy Loam	20A, 20B, 20C	0.0625	0.1605	0.429	0.0156
Silty Clay Loam	28A	0.1475	0.306	0.48	0.00017

#### 4.2.2. Theoretical data

##### 4.2.2.1. Groundwater flow

As previously mentioned, the groundwater flow modeling involves the estimation of several parameters (Ksat, wilting point, porosity, and field capacity). The values of these parameters are estimated from the USDA soil survey and the soil spatial extent

estimated from the GIS layers. In this process several uncertainties were introduced as described below.

Dingman indicates a range of values for each soil type. Without any other additional information the average of the maximum and minimum values was assigned to each soil type. As a first approach, these average values were considered valid due to the uncertainties found in the SWMM literature on the groundwater flow model (Huber et al., 1992).

On the other hand, the soil information from the SSURGO survey is based on discrete information collected at specific sites. These data are assumed representative of surrounding areas. Although soil data are intended to be an accurate representation of the unsaturated zone characteristics, these data may not reflect exactly what is actually present. Further, the same soils in different areas may have different degrees of compactness and percentages of fines and gravels. Soil types may vary vertically and it is assumed that they are homogeneous in all three dimensions.

In the same way, each sub-watershed is given a weighted average value for each parameter based on the area of each soil type inside that sub-watershed assuming horizontal homogeneity. Soils are not homogeneous even within a small area. Groundwater models assume homogeneity of soils without which, theories of groundflow would be of little practical value.

Currently, SSURGO soil survey data are among the best soil representations available. Despite of the uncertainties involved in soil data, we took advantage of the GIS technology to utilize, as best as possible, data from the SSURGO survey to characterize the unique parameters for each individual sub-watershed.

#### 4.2.2.2. Pollutant load

In this study the pollutant loading depends on the land use distribution. Only non-point source pollution is estimated which is a characteristic of residentially developed watersheds. Non-point sources are estimated from the event mean concentrations (EMC) average values. EMC values were taken for the Washington D.C. and suburban area based on previous studies (Schueler, 1987) as shown in Table 4.2. These values are estimated from several measured pollutant concentrations and statistically related to different types of land use.

Table 4.2. EMC values used in this study. Values based on NURP, Controlling Runoff: a practical manual for planning and designing urban BMP's in Washington D.C. The values used in this study were taken from various sources (Wanielista, 1978; NURP, 1983) and adjusted with the values from local studies (Schueler, 1987).

Pollutant	Land Use Category				
	Commercial	Forested	Residential	Agriculture	Other
TSS (mg/l)	69.000	70.000	101.000	85.000	85.000
TKN (mg/l)	1.179	0.965	1.900	1.100	1.500
TP (mg/l)	0.201	0.121	0.383	1.200	0.250
Nitrates (mg/l)	0.572	0.543	0.736	0.750	65.000

#### 4.2.3. Modeling processes

Several processes are involved in the modeling of hydrologic cycle as well as in the physical representation of the watershed. One of the most important values to be estimated is the drainage area. Watershed delineation was done based on a DEM of 30 m. cell size and the drainage area is estimated based on the number cells draining to each outlet. In larger watersheds this approximation is subject to smaller errors than in smaller watersheds.



Watersheds were divided into sub-watersheds in order to have a homogeneous land use distribution within the sub-watershed and to match the blue lines from the USGS maps. Drainage areas to the sub-watersheds vary from almost 16,000 sq. ft. to over 923,000 sq. ft. (Table 4.3) and a cell size is 10,000 sq. ft. Therefore, an error of 100 cells in the estimation of the drainage areas in this study could represent an uncertainty of 0.02% in the larger watershed and 3.60% in the smaller one.

Table 4.3 shows some of the hydrologic estimates for each sub-watershed estimated from grids. The total drainage area in square miles is the total area draining to the outlet of each watershed, while the area of the sub-watershed is the difference between the drainage area and the drainage area of the upstream sub-watershed. Maximum and minimum elevations are the maximum and minimum ground elevations in the sub-watershed. The slope is the slope of the longest flow path (distance from the hydrologic most distant point to the outlet in a watershed) and the main stream length is the length of the main channel. The downstream sub-watershed is the sub-watershed identification number to which each sub-watershed drains.

Table 4.3. Northwest Branch of Anacostia River basic physical characteristics for each sub-watershed included in this study.

Cat. ID	Total Drain. Area (sq. mi.)	Area sub water sheds (sq. mi.)	Max. Elev. (ft)	Min. Elev. (ft)	Slope (%)	Channel length (ft)	Down stream sub watershed ID
1	1.10	1.10	654	400	2.385	10650	102
2	1.39	0.05	649	385	3.075	1266	1011
3	3.46	0.67	654	338	1.810	4787	1014
4	1.09	1.09	629	348	2.597	10818	105
5	1.09	1.09	623	395	2.196	10381	100
6	1.06	0.06	668	394	3.401	949	104
7	3.23	0.01	677	353	2.201	300	109
8	1.59	0.24	681	458	2.050	2283	1013
9	4.97	0.03	681	351	2.042	724	1012
10	4.90	0.29	654	328	1.531	3390	1015
11	6.27	0.15	681	328	1.486	1873	1018
12	11.29	0.09	681	328	1.414	824	1017
13	1.26	0.24	611	299	2.914	2555	1023
14	13.69	1.51	681	299	1.093	9111	1024
15	15.37	0.26	681	295	0.982	2924	1026
16	0.91	0.91	615	299	3.112	10154	1025
17	16.54	0.09	681	295	0.939	1249	1030
18	4.47	0.55	613	295	1.216	3745	1029
19	21.18	0.03	681	295	0.899	824	1032
23	1.35	1.35	681	480	2.339	8580	8
24	1.29	0.01	613	369	2.505	300	1022
25	1.02	1.02	611	315	3.632	8150	13
100	1.24	0.15	623	376	2.067	1566	106
101	0.55	0.55	626	491	1.903	7094	6
102	1.28	0.19	654	370	2.241	2024	3
103	0.45	0.45	668	394	4.189	6528	6
104	1.17	0.11	668	376	2.882	2072	106
105	1.14	0.05	629	336	2.441	1183	10
106	2.58	0.17	668	356	2.158	2490	7
107	0.64	0.64	677	357	3.035	10557	7
108	0.55	0.55	649	392	3.512	7318	2
109	3.34	0.11	677	351	2.117	683	9
1010	0.79	0.79	641	488	2.167	7060	2
1011	1.51	0.12	649	370	2.845	1224	3
1012	5.71	0.75	681	331	1.600	5711	11

Table 4.3. Northwest Branch of Anacostia River basic physical characteristics for each sub-watershed included in this study. (Continued)

Cat. ID	Total Drain. Area (sq. mi.)	Area sub water sheds (sq. mi.)	Max. Elev. (ft)	Min. Elev. (ft)	Slope (%)	Channel length (ft)	Down stream sub watershed ID
1013	1.60	0.01	681	450	2.048	400	9
1014	3.47	0.01	654	334	1.787	441	10
1015	4.91	0.01	654	328	1.500	442	12
1016	0.41	0.41	641	331	3.551	8738	11
1017	11.31	0.03	681	323	1.385	883	14
1018	6.29	0.02	681	328	1.463	383	12
1019	0.56	0.56	613	378	2.904	8091	24
1020	0.86	0.86	623	323	3.418	8777	14
1021	0.72	0.72	613	468	1.536	9440	24
1022	2.47	1.19	613	333	1.493	9018	1028
1023	1.30	0.04	611	296	2.667	1107	15
1024	13.81	0.12	681	296	1.058	1442	15
1025	1.04	0.14	615	295	2.904	866	17
1026	15.41	0.03	681	295	0.968	541	17
1027	0.54	0.54	564	333	3.261	7084	1028
1028	3.56	0.55	613	311	1.348	3639	18
1029	4.49	0.01	613	295	1.182	766	19
1030	16.66	0.11	681	295	0.916	1007	19
1031	0.36	0.36	564	401	2.675	6094	18
1032	21.19	0.01	681	295	0.897	100	N/A

The drainage network was estimated from the DEM using standard GIS methods. Stream data such as slope, channel length, and cross section area were derived from this drainage network. The channel length is estimated by adding the distances between the cell centers of the cells along the main channel flow path.

The pollutant load is modeled as a temporal average load depending on the amount of two separate processes. In the first process the pollutant load increases with time based on the land use percentage and built up parameters type and the built up and wash off areas. These models the amount of pollutant that is produced and is stored until it is washed off by rainfall into streams. The second process models how the pollutant

that has been built up is taken to streams by storms. Both processes are modeled by power equations based on the time since the last storm.

#### **4.2.3.1. SWMM hydraulic and hydrologic approach**

The SWMM model can accomplish different levels of complexity depending on the number of processes included in the simulation. These processes can be water quantity processes like: surface runoff, infiltration, groundwater flow, and routing. There are also different methods to simulate pollutant load depending on the available data. The SWMM model has included different methods for estimating pollutant load. The simplest method uses the event mean concentrations (EMC) and more sophisticated methods use complex simulations that include build up and wash off processes, and pollutant build up reduction. Some of the most important parameters used in this study will be described as follows like: sub-watershed geometry description, infiltration, groundwater flow, and pollutant load.

##### **4.2.3.1.1. Watershed shape description**

Self drainage areas are defined in this study as the areas that are draining to each point (internal outlet) but not to any other outlet (Figure 4.3). This information is used in the estimation of sub-watershed width, rain precipitation and pollutant loading.

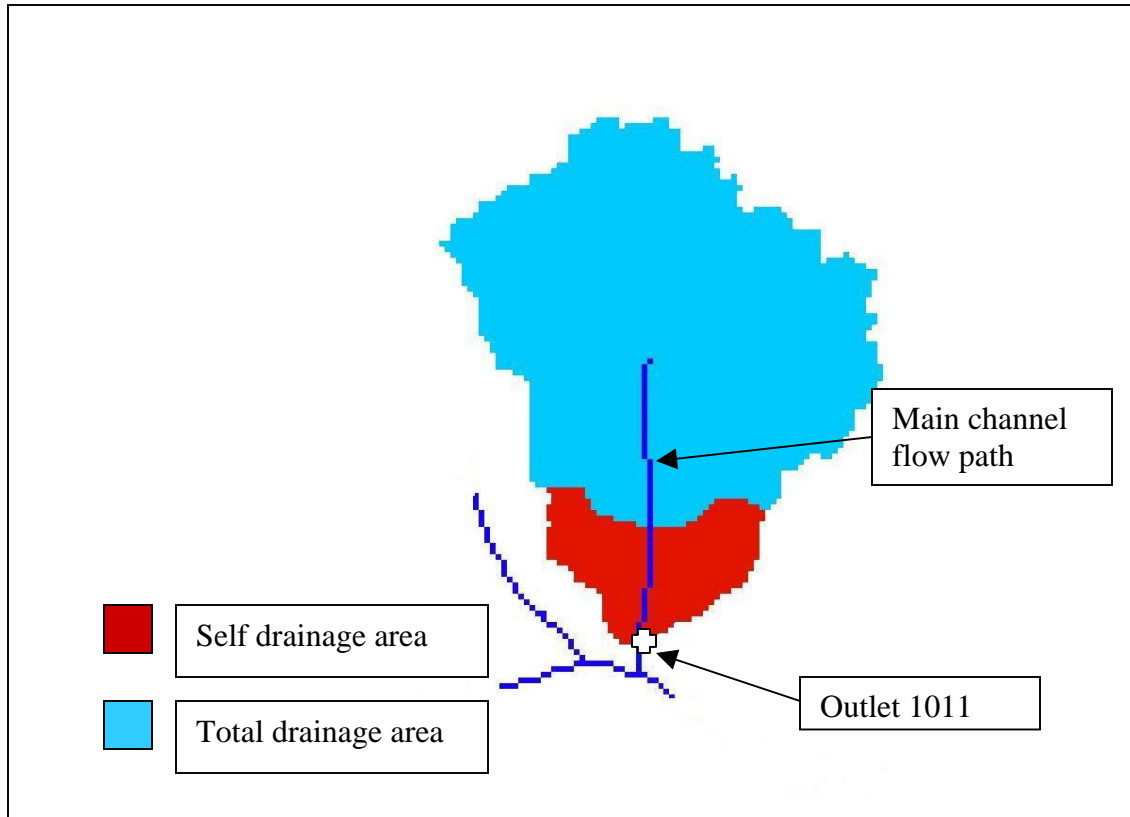


Figure 4.3. Total area and self drainage area for sub-watershed 1011 draining to outlet 1011. The self drainage area is shown in black.

The sub-watershed width is an estimate of the average distance that water has to travel before reaching a channel (Huber et al., 1992). This parameter is important in the modeling of flood peaks. There are two approaches to estimating the sub-watershed width: one is make it equal to two times the main channel flow path length and the other one is based on a skew factor (Figure 4.4).

The main channel divides each sub-watershed in two and the skew coefficient is defined as a weighted average of the difference of each area with relation to the sub-watershed area:

$$S_k = \frac{A_1 - A_2}{A_{tot}} \quad (4.1)$$

where:

$S_k =$  Skew coefficient as dimensionless fraction.

$A_1 =$  Greater of the two areas split by the main channel flow path in area units.

$A_2 =$  Smaller of the two areas split by the main channel flow path in area units.

$A_{tot} =$  Total area of the sub-watershed in area units.

The sub-watershed width is defined as:

$$W = (2 - SL_k) * L \quad (4.2)$$

where:

$W =$  Sub-watershed width in feet.

$S_k =$  Skewed coefficient as dimensionless fraction.

$L =$  Channel length in feet.

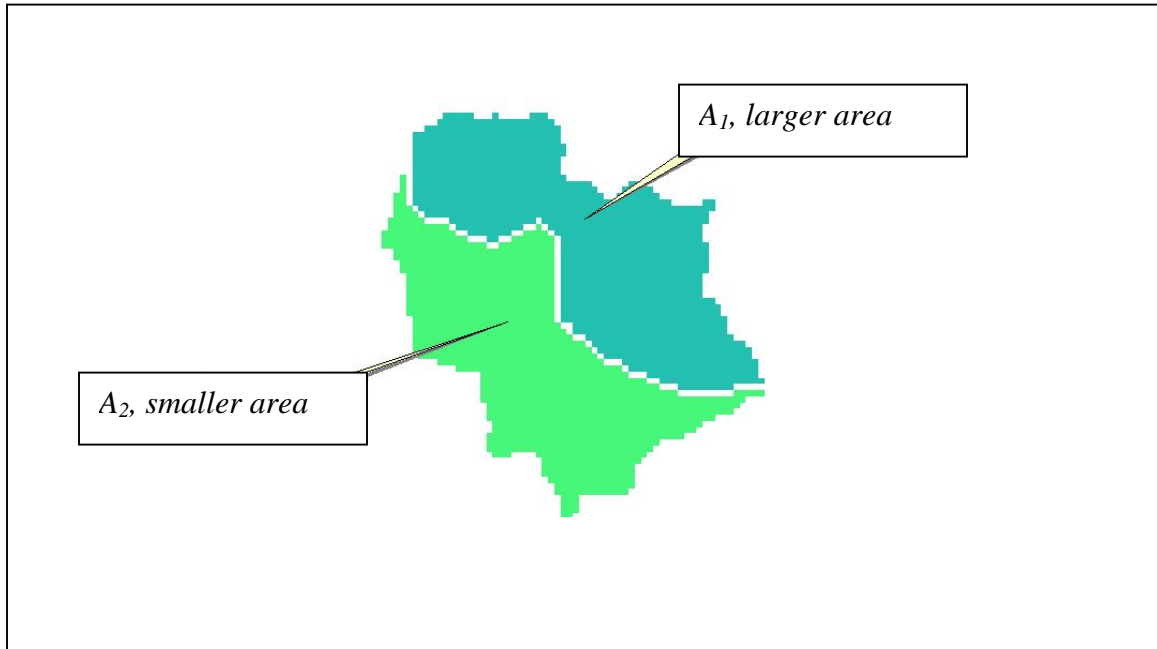


Figure 4.4. Sub-watershed 1011 showing how sub-watersheds were split in two to estimate the skew coefficient ( $S_k$ ) used in the width estimation.

#### 4.2.3.1.2. Infiltration parameters

The Horton infiltration equation (Equation 4.3) is modeled with: initial infiltration,  $f_o$  (0.21 in/hr), minimum infiltration,  $f_c$  (0.19 in/hr) and exponential coefficient,  $k$  (0.00015 1/sec). Initial estimation of the values for these parameters before calibrating the SWMM model, were also obtained from GIS data and will be explained under that section as part of soil characteristics.

$$f(t) = f_c + (f_o - f_c) * \exp(-kt) \quad (4.3)$$

where:

$f(t)$  = Infiltration rate at a time  $t$  (in/hr).

- $f_c =$  Minimum infiltration rate (in/hr).
- $f_o =$  Maximum or initial infiltration rate (in/hr).
- $k =$  Decay coefficient (1/sec).
- $t =$  Time (sec).

#### 4.2.3.1.3. Channel flow conditions

Channel geometry and channel related elevation parameters are shown in Figure 4.5, the bottom of the channel is assumed to be the elevation of the channel obtained directly from GIS data. Other values are derived from assumed cross-section geometry. According to Dunne and Leopold (1978), channel width and depth for the eastern US can be approximated by:

$$d = 1.57 * A^{0.28} \quad (4.4)$$

where:

$d =$  Bankfull channel depth in feet.

$A =$  Drainage area in square miles.

In the same way bankfull (maximum) water width can be related to drainage area in this region by:

$$w = 15.5 * A^{0.36} \quad (4.5)$$

where:

$w =$  Bankfull water width channel in feet.

$A =$  Drainage area in square miles.



By solving for area in one of these equations and replacing in the other it can be shown that water depth and channel width are related by a power equation with an exponent of 1.286. The resulting relationship between width and depth is:

$$w = 15.5 * \left( \frac{d}{1.57} \right)^{\frac{0.36}{0.28}} \quad (4.6)$$

$$w = C * d^{1.286}$$

where:

$w =$  Channel width with a depth  $d$  in feet.

$d =$  Channel depth  $d$  in feet.

$C =$  Constant in units of  $\frac{1}{ft^{1.286}}$ .

The ground elevation is assumed to be the bottom elevation plus the bankfull depth. The initial condition of water table elevation is assumed to be the next integer value below the channel bottom elevation. The water table elevation is an initial assumption and will be updated in each simulation. Figure 4.5 shows each of these channel geometry values.

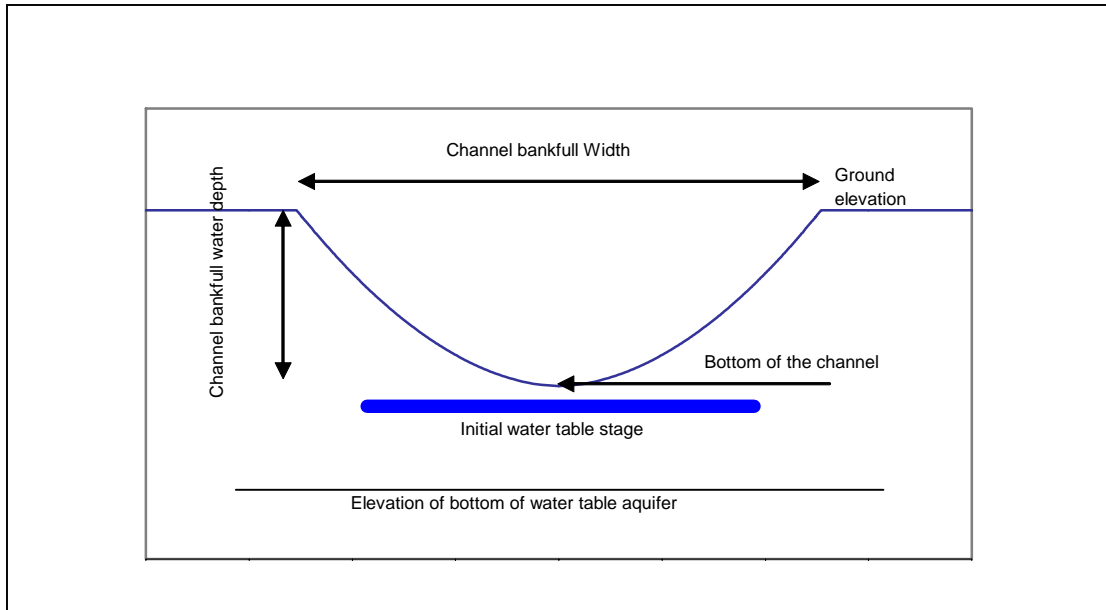


Figure 4.5. Description of channel geometry and its relation to water elevations.

#### 4.2.3.1.4. Groundwater flow

SWMM has three models, which can be combined together and are intended to give a description of several possible alternatives of groundwater. The three models are: water flow from channel into ground, water flow from ground into channel and water flow that depends on the difference between the water table elevation and the water surface elevation on the channel. Equation 4.7 describes all the three water flows.

$$GWFLW = A1 * (DI - BC)^{B1} - TWFLW + A3 * DI * TW \quad (4.7)$$

where:

$TWFLW$  = Flow from the receiving water into the groundwater in  $ft^3/s$  defined as:

$GWFLW$  = Groundwater flow into the receiving water in  $ft^3/s$ .

$DI$  = Groundwater level in feet.

$BC$  = Channel bottom elevation in feet.

$TW$  = Water level in the receiving water in feet.

$A1$ ,  $B1$ , and  $A3$  are coefficients.

and  $TWFLW$  is defined as follows:

$$TWFLW = A2 * ( TW - BC )^{B2} \quad (4.8)$$

where  $A2$  and  $B2$  are coefficients.

For uniform channel flow and horizontal groundwater flow modeled by the Dupuit-Forcheimer approximation, (Figure 4.6) the relationship between water table elevation and flow into the channel is:

$$K * (h_1^2 - h_2^2) = L^2 * f \quad (4.9)$$

where:

$K =$  Hydraulic conductivity in inches per hour.

$h_1 =$  Water table elevation at a distance “far” from the channel in feet.

$h_2 =$  Water table elevation before the channel in feet.

$L =$  Distance between  $h_1$  and  $h_2$  in feet.

$f =$  Infiltration rate inches per hour.

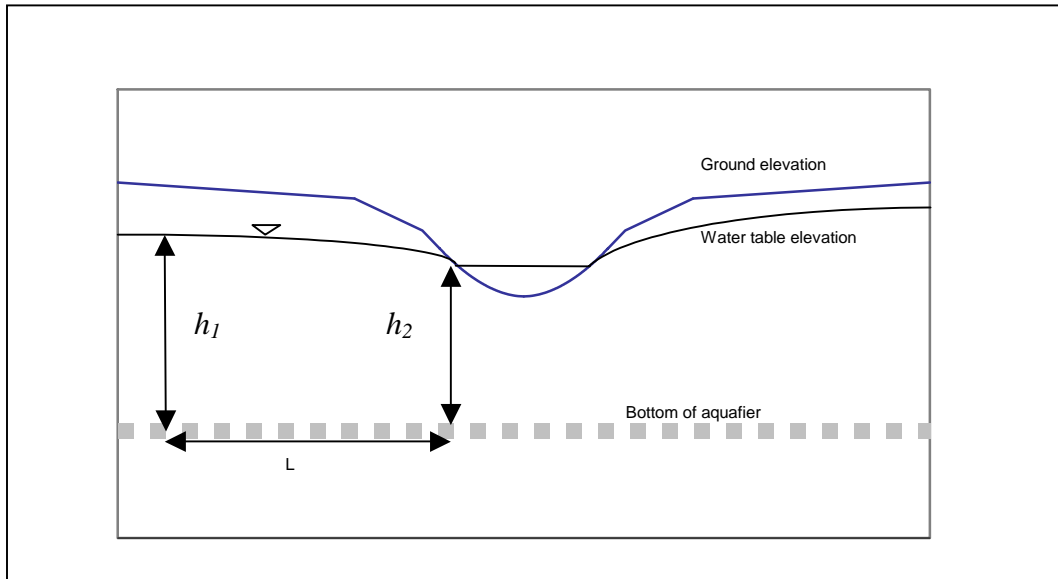


Figure 4.6. Groundwater flow Dupuit-Forcheimer approximation.

by definition  $D_1$  is the mean water table elevation.

$$D_1 = (h_1 + h_2) \quad (4.10)$$

by arranging and solving for  $h_1$  and replacing in equation 4.10 we obtain:

$$(D_1^2 - D_1 * h_2) * 4 * K / L^2 = f \quad (4.11)$$

Comparing this equation to 4.9 it is seen that:  $A1 = A3 = 4 * K / L^2$ ,  $A2 = 0$ ,  $B1 = 2$  and  $B2$  can be any positive number. The following values have been defined as below:

$$A1 = 0.0045$$

$$B1 = 3$$

$$A2 = 0$$

$$B2 = 1$$

$$A3 = 0.0045$$

#### **4.2.3.1.5. Unsaturated zone**

Some other factors such as the relation between hydraulic conductivity and moisture content, average slope of tension and soil moisture content, the fraction of evapotranspiration from the upper zone, the coefficient for unquantified losses and maximum depths over which significant lower zone evapotranspiration occurs. These parameters are very hard to obtain and more difficult to define in a continuous modeling process where spatially averaged data must be used, therefore they were used in the calibration process of the groundwater flow of each model. Initial values were obtained from SWMM user's manual.

Computed values derived from GIS data which are detailed in the next section are: porosity (0.41317), wilting point (0.11428) and field capacity (0.25428). All values are in fractions and are therefore dimensionless. Finally there are saturated hydraulic conductivity (1.0 inch per hour) and initial moisture content data (0.2 as a volume fraction).

#### **4.2.3.1.6. Water quality modeling**

Water quality is modeled in runoff for almost all the blocks in SWMM, except for some specific alternatives in transport and storage/treatment modules or blocks. The SWMM runoff block has different methods for estimating water quality among others. The selection of the method depends mostly on the detail and amount of information available. All methods presented in the runoff block estimate the pollutant load and

transport (build up and wash off). The pollutant or nutrient load for different land uses are estimated by the total load in each sub-watershed based on the average load.

The method selected in this research is a rating curve approach that relates pollutant load to runoff by using the event mean concentration (EMC). There were studies performed in the Washington area that provide EMC values for different land uses (Schueler, 1987). EMC's are estimated for different areas from several measures taken in the field. The concentration of each pollutant is related to the dominant land use in the sampling area. The amount of pollutant for each sub-watershed is calculated in this study as follows:

$$POFF = RCOEF * WFLOW^{WASHPO} \quad (4.12)$$

where:

*POFF* = Constituent load wash off at time t in mg/l.

*RCOEF* = Coefficient with correct unit conversion.

*WFLOW* = Sub-watershed runoff in ft<sup>3</sup>/s.

*WASHPO* = Exponent.

Setting *WASHPO* to 1 and making *RCOEF* equal to EMC defines a linear relation between pollutant load and runoff.

### 4.3. Calibration

In this study, three different SWMM models were set up and three different calibration exercises were performed. The three SWMM models, described in Chapter Three, were: (1) continuous model with constant land use for several years of land use

distributions, (2) Hot-start holding constant the land use distributions similar to the model (1), and (3) Hot-start with annually changing land use for the same period as for models (1), and (2).

Three different calibrations processes were done. The first one was done using model (1) results with different land use distributions for the years (1979 to 1982) and the observed data. The second one was done between the continuous land use time series and the hot restart model with constant land use distribution. The last calibration was between the hot run restart constant land use distribution and the hot restart annually changing land use distribution.

Three goals were sought in these calibrations. A brief description of each goal is discussed here. The first goal was to calibrate the SWMM model. This goal was achieved by adjusting the constant land use model parameters following the standard procedures suggested by SWMM manual and modelers. The changes were made in the groundwater parameters and by adjusting initial estimates of the geometry, slope, manning's n and depression storages.

The second goal was to confirm that the hot-start simulations were equivalent to the continuous simulations with constant land use (model 1). This was done by comparing the result from model (1) to the results of model (2) and confirming that there was no significant difference between the two of them.

The third goal was to support the hypothesis that annually changing land use more closely reproduced the observed data than the constant land use. For consistency we used the results of model (2) and not the results from model (1) because the annually changing

land use and the hot-start model with constant land use both used the same (hot-start) approach.

#### **4.3.1. Constant land use model results vs. observed discharges**

The first calibration was done taking into consideration three characteristics describing a hydrograph: the peak, the recession limb, and base flow. As a first step, a graphical analysis was made for selected storms. Calibration parameters were adjusted subjectively until good agreement was obtained. The goal of this process was to calibrate the SWMM model flows by estimating the surface runoff, infiltration, groundwater flow, and physical description of watershed geometry parameters. Once these parameters were estimated, they were held constant for all the other models so the changes in land use would be the only ones to influence the modeled discharge results.

Three indices of agreement were calculated: (1) total volume deviation, (2) the Nash-Sutcliffe index which measures the correlation between two time data series, and (3) the cumulative discharge distribution that measures the number and magnitude of discharges.

##### **4.3.1.1. Total volume deviation**

The total volume deviation index calculated from equation 3.4, showed a good agreement between the modeled results and the observed data with only small difference between the sum of the modeled discharges and the observed data for the modeled period. Results are shown in Table 4.4. It is expected that as urban development increases peak flows are increased and base flows are reduced, however the overall discharge



remained almost unchanged (Table 4.4). In this table it can be seen that deviations are very close to zero, indicating a good estimate of the total volume of discharges (ASCE, 1993).

Table 4.4. Northwest Branch of Anacostia River volume deviation index for extreme land use scenarios for 1979.

Land Use	Total water volume in ft <sup>3</sup> *day/s	Deviation volume
Observed	203,563	
1979	186,254	8.4%
1988	187,291	8.0%

#### 4.3.1.2. Nash-Sutcliffe index

The Nash-Sutcliffe index measures the goodness-of-fit between the modeled discharges and the observed discharges. In this study the Nash-Sutcliffe index values were close to zero and sometimes negative. According to the definition of this index these values indicate poor agreement (ASCE, 1993). We tested the sensitivity of this index to plus or minus a one day lag, because it was found previously how sensitive SWMM results could be to time step duration selected for rainfall and model output.

We took a series of discharges for a given period. Five possibilities were considered, three with larger discharges (3 and 9 ft<sup>3</sup>/s more than observed, and 10% more discharge) and two with different offset days (one day before and one day after). Data are shown in Table 4.5. It was found that Nash-Sutcliffe index was negative (indicating poor agreement) for the offset discharges but a Nash-Sutcliffe index value close to 1 for the larger discharges. Based on the sensitivity of the SWMM model to switching peaks from day to day depends on the time step it was expected that we might obtain more negative

indices than indices close to 1. In this study it was considered that it was more relevant to model discharges in volume and frequency than in time, therefore the analysis was focused on the frequency curve analysis.

The daily-observed discharges shown in Table 4.5 correspond to the month of July 1979. The daily observed discharges plus 3 and 9 ft<sup>3</sup>/s (arbitrary selected values to test the Nash-Sutcliffe index sensitivity to the discharge series) are shown in the two following columns labeled as “Observ plus 3 ft<sup>3</sup>/s” and “Observ plus 9 ft<sup>3</sup>/s”. The daily observed discharges shifted one day before and after are shown under the columns “One day shifted ahead” (e.g., observed discharge of July 15, 1979 of 15 ft<sup>3</sup>/s is shifted to July 16, 1979) and “One day shifted behind” (e.g., observed discharge of July 15, 1979 of 15 ft<sup>3</sup>/s is shifted to July 14, 1979). The daily observed discharges increased in 10% are shown in the last column “Obsev plus 10% more”.

Nash-Sutcliffe index is calculated for each one of these discharge series by comparing them to the observed discharges. Nash-Sutcliffe index values thus calculated are shown at the end of the Table 4.5 under the respective column.

Nash-Sutcliffe indices for the discharges with larger values (3 and 9 ft<sup>3</sup>/s, and 10% more) are equal to 1.0, this would mean a good correlation between augmented discharges and observed discharges. While the Nash-Sutcliffe indices for the shifted discharges (one day before and ahead) show a poor correlation (0.22826). In this study it was considered more important to account the amount and frequency of the discharges rather than the timing, therefore Nash-Sutcliffe index was not found to be a goodness-of-fit criteria for the SWMM model calibration process.

Table 4.5. Discharges used to analyze Nash-Sutcliffe index sensitivity to peaks occurring on different days compared to different volumes of discharge.

Date	Observed	Observ plus 3 cfs	Observ plus 9 cfs	One day shifted ahead	One day shifted behind	Observ plus 10% more
7/1/1979	45	48	54	11	25	51.75
7/2/1979	25	28	34	45	12	28.75
7/3/1979	12	15	21	25	14	13.8
7/4/1979	14	17	23	12	14	16.1
7/5/1979	14	17	23	14	11	16.1
7/6/1979	11	14	20	14	9.7	12.65
7/7/1979	9.7	12.7	18.7	11	9.3	11.155
7/8/1979	9.3	12.3	18.3	9.7	8.9	10.695
7/9/1979	8.9	11.9	17.9	9.3	9	10.235
7/10/1979	9	12	18	8.9	9.2	10.35
7/11/1979	9.2	12.2	18.2	9	9.2	10.58
7/12/1979	9.2	12.2	18.2	9.2	32	10.58
7/13/1979	32	35	41	9.2	23	36.8
7/14/1979	23	26	32	32	15	26.45
7/15/1979	15	18	24	23	11	17.25
7/16/1979	11	14	20	15	10	12.65
7/17/1979	10	13	19	11	9.2	11.5
7/18/1979	9.2	12.2	18.2	10	9.2	10.58
7/19/1979	9.2	12.2	18.2	9.2	14	10.58
7/20/1979	14	17	23	9.2	16	16.1
7/21/1979	16	19	25	14	12	18.4
7/22/1979	12	15	21	16	9.6	13.8
7/23/1979	9.6	12.6	18.6	12	9.2	11.04
7/24/1979	9.2	12.2	18.2	9.6	8.7	10.58
7/25/1979	8.7	11.7	17.7	9.2	9.6	10.005
7/26/1979	9.6	12.6	18.6	8.7	7.8	11.04
7/27/1979	7.8	10.8	16.8	9.6	25	8.97
7/28/1979	25	28	34	7.8	14	28.75
7/29/1979	14	17	23	25	26	16.1
7/30/1979	26	29	35	14	11	29.9

Table 4.5. Discharges used to analyze Nash-Sutcliffe index sensitivity to peaks occurring on different days compared to different volumes of discharge (Continued).

Date	Observed	Observ plus 3 cfs	Observ plus 9 cfs	One day shifted ahead	One day shifted behind	Observ plus 10% more
7/31/1979	11	14	20	26	45	12.65
Nash-Sutcliffe Index		0.87	(0.188266)	(0.543487)	(0.543487)	0.908380
Minimum Square Correl.		1.00000	1.00000	0.22826	0.22826	1.00000

#### 4.3.1.3. Cumulative discharge distribution

The cumulative discharge distribution measures the percent of discharges that exceed a given discharge value. It was used in this study as a calibration index and as method of comparison to measure the effects of changing the land use in each one of the models (Figure 4.7). It was observed that although the number of peaks was the same for each modeled time series, the magnitude of the discharges were higher for the more developed land use scenario than for less developed land user scenario. In the same way, it was found that the low flows corresponding to base flow conditions, were lower for the more developed land use scenario than the low flows modeled for the less developed land use scenario. A Chi square test was done after the graph analysis to prove the hypothesis that both CDF series had the same distribution.

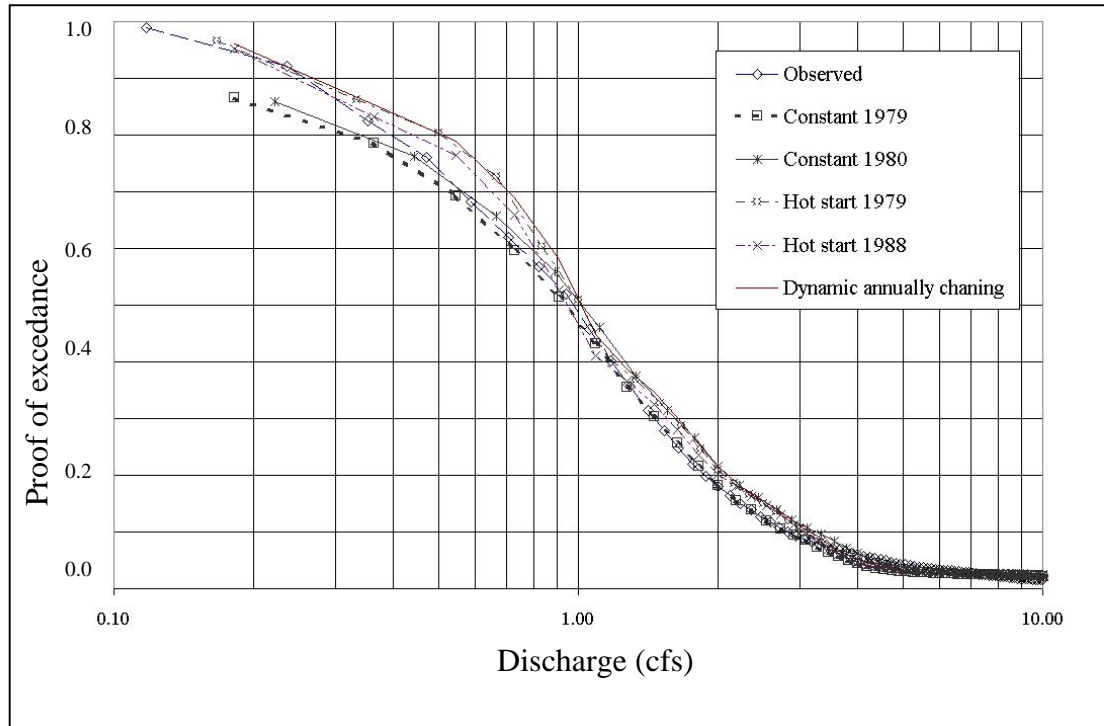


Figure 4.7. Cumulative distribution function of discharges for different simulations and the observed distribution.

It was also observed that the cumulative discharge distribution derived from the observed discharges was between the more developed land use cumulative distribution and the less developed land use cumulative discharge distribution. This means that the peaks resulting from a less developed land use distribution are smaller than the peaks resulting from a more developed land use distribution. In the same way, the baseflows are higher in the less developed land use distribution model than in the more developed land use distribution. Therefore the model results are consistent with previous studies (Dunne and Leopold, 1978; Anderson, 1973).

#### 4.3.1.4. Conclusions

Based on the conclusion that the model peaks are sensitive to the time of occurrence of the storm because of the time step of rain fall data, we wanted to develop a

method that, in conjunction with the two indices that measure the goodness-of-fit, could measure the number of discharges and their corresponding volume for both a modeled discharge series and a reference discharge series in a defined time period. It was decided to use the cumulative discharge function as the third calibration measure.

When peaks flows are large they have a stronger influence in the goodness-of-fit between the two discharge series than the baseflows. This means that more careful attention should be given to peak classification than to base flows. Several methods for separating peak discharges from base flows were considered: finding the inflection point on the recession limb, estimating the number of days of peak flow occurrence as a function of the drainage area. All methods showed similar results, indicating that the study watershed return to baseflow conditions between one and two days after the peak flow occurrence. In order to use an objective approach for separating peaks and baseflow, peak discharges were defined as those events where the modeled result from a more developed land use scenario were larger than the corresponding ones resulting from less developed land use scenarios.

#### **4.3.2. SWMM calibration**

As mentioned at the beginning of the calibration section the results from the constant land use model were used to calibrate SWMM model, therefore we will briefly describe how this calibration was done. It is not the intention of this study to make an illustration on how to calibrate SWMM models. The SWMM model was calibrated by: first matching peaks, then recession rates, and finally base flows. In order to match peaks, we altered parameters describing the shape of the watershed and the infiltration ratios.

The recession limb was calibrated by changing Manning's  $n$ , depression storages and infiltration and percolation parameters. Baseflows were calibrated by changing the groundwater flow parameters.

#### **4.3.2.1. Peaks**

Two basic criteria were considered when calibrating peaks, their distribution in time and the magnitude of each one. In previous sections it was discussed that the time distribution of the peaks depended on the storm, as rainfall is the only water source considered in this study, and the rainfall time step. We did not have control over these two parameters; therefore we concentrated the calibration on matching the discharge volume.

We selected year 1979 as the calibration period for the SWMM model. Peaks within this period were adjusted by changing the roughness coefficient and changes in depression storage for pervious and impervious areas.

#### **4.3.2.2. Recession limb**

The recession limb was found to be very steep because of the initial elevation assumed for groundwater flow. The recession slope was adjusted by changing the groundwater stage elevation. Increasing this parameter caused less flow to enter into the channels and increased the moisture content in the ground, which reduced the rate of infiltration.

#### **4.3.2.3. Baseflow**

The modeled baseflow was lower than observed one. Occasionally baseflow dropped to zero one or two days after a storm. Baseflows were raised by reducing the deep percolation factor and by increasing the water recharge from the groundwater into the channels.

#### **4.3.3. Hot-start constant land use compared to constant land use model results**

It was expected that both results from these model executions would be similar, though some differences were expected due to the fact that not all state variables could be retrieved at the end of each year. The SWMM model output option was selected to generate the most detailed output in order to be able to update as much as state variables as possible for each sub-watershed. To the extent that these state variables were unavailable the hot restart vs. the constant land use simulations differ slightly.

Three indices of agreement were analyzed for these two time series and the observed data. We found that deviation volume was close to zero between the two-modeled discharges meaning that both had similar total volume discharges, see Table 4.4. The Nash-Sutcliffe index did not show a meaningful result, values were negative as shown in Table 4.5. Due to the sensitivity of the Nash-Sutcliffe index to off set discharges it was not further considered in this study. Finally the cumulative discharge distributions were very similar as shown in Figure 4.7.

A chi square test was done on the cumulative distribution function (CDF) (Roscoe, 1969) for both scenarios, constant land use and hot-start with constant land use, with 199 degrees of freedom to test the hypothesis that the constant land use had the same



cumulative distribution function as the hot-start constant land use. The values of the Chi square test were: 79.38 and 81.52 for each scenario respectively. The hypothesis that both of the scenarios, constant land use and hot-start with constant land use, was not rejected. It was concluded that the hot-start method does not introduce a large additional error to the model and that it could be used to simulate model the annually changing land use scenario. Once the SWMM model had been calibrated and the hot-start process was tested we developed an annually changing land use model as described in the next section.

#### **4.3.4. Annually changing land use compared to hot-start constant land use**

A comparison between the annually changing land use model discharges and the hot-start model with constant land use provides insight into the effects of changing land use on simulated streamflow. The logic of our approach was to treat the hot restart with constant land use simulations as representative of typical engineering practice use of the SWMM model. The simple constant land use model results were not used for comparison because of the few unknown state variables that could not be transferred to the hot restart simulations.

Two indices were considered at this time, the deviation volume and the cumulative discharge distribution when making the comparison. The Nash-Sutcliffe index was not included at this time due to the poor results caused by timing problems with the peak flows. We relied on the total volume and the cumulative discharge distribution to make comparisons.

#### 4.4. Simplified water models

Two types of model results were examined: (1) water quantity (discharge), and (2) water quality (total phosphate pollutant concentration). The goal is to correlate changes in the predicted time series (discharge or pollutant concentration) to the level imperviousness. Imperviousness was chosen as an indicator of development. Once that calibration was done for the Northwest Branch of the Anacostia River it was assumed to accurately represent the other three sub-watersheds described in section 4.4. No other observed data was available for comparison.

##### 4.4.1. Peak discharges

Discharges were first separated into peaks and baseflow. It was expected that the peaks had some relation with the rainfall intensity of the storm that caused that peak following the rational method equation (Bras, 1990) where:

$$Q = CiA \tag{4.13}$$

where

$Q$  = Peak discharge in  $\text{ft}^3/\text{s}$ .

$i$  = Rainfall intensity in inch of rain per hour.

$A$  = Drainage area in acres.

$C$  = Nondimensional coefficient generally varying between 0.5 and 0.8.

Daily peaks were defined as those daily discharges where both preceding and following discharges were smaller. These peaks were divided by drainage area of each

sub-watershed in square miles to get a unit peak discharge per square mile for comparability among the watersheds.

As a first approach we selected 25 major precipitation events within the simulated period from January 1, 1979 through December 31, 1988. A precipitation event has been defined as the time period in which any precipitation occurred with less than a day of zero precipitation separating time periods of observed rainfall.

One of these events (September 2, 1979) was classified as an outlier and it was not considered in this analysis. It was found that this event corresponded to a large storm resulting from a hurricane. The other 24 precipitation events are summarized in Table 4.6.

It was assumed that after a day from the peak of a given storm baseflow regime would be prevailing over direct runoff regime. The total area of the Northwest Branch of Anacostia River is 21.2 square miles. According to Bras (1990), one of the empirical methods commonly used for hydrograph separation defines the number of days from the time of peak to the beginning of baseflow equal to the drainage area in square miles to the 0.2 power. Applying this empirical method it was found that the time from the peak to the baseflow for the Northwest Branch of Anacostia River is less than 2 days. Some other methods were tested and it was found that the length of periods from the peak discharge to the baseflow regime was approximately one day. Therefore, the assumption that the rain events with more than one dry day apart can be considered as separate events was considered valid bearing in mind that this assumption will affect only the way peak flows are classified in the model development.

Table 4.6. Twenty-four major storms affecting Northwest Branch of Anacostia River from 01/01/1979 through 12/31/1988.

Precip. Event	Starting time	Ending time	Max P inches of rain in 15 min.	Min P inches of rain in 15 min	Total P in inches of rain per storm	Rain intensity in inches per hour	Estimated return period in years
1	11/30/1986 0:00	11/30/1986 23:00	0.26	0.22	6.2	0.26	100
2	9/5/1979 8:00	9/5/1979 22:00	1.1	0.1	5	0.33	10
3	8/28/1979 0:00	8/28/1979 7:00	0.69	0.67	5.5	0.69	10
4	11/5/1981 23:00	11/6/1981 0:00	0.5	0.1	0.6	0.30	1
5	9/15/1981 16:00	9/16/1981 4:00	1	0.1	2.8	0.22	2
6	6/6/1983 17:00	6/6/1983 22:00	0.7	0.1	2.3	0.38	2
7	9/11/1987 23:00	9/12/1987 5:00	0.3	0.1	0.6	0.09	1
8	10/2/1979 22:00	10/3/1979 5:00	0.2	0.1	0.9	0.11	1
9	9/26/1985 20:00	9/27/1985 10:00	0.7	0.1	4.2	0.28	5
10	5/17/1988 18:00	5/19/1988 2:00	0.5	0.1	1.9	0.06	1
11	6/18/1983 7:00	6/21/1983 14:00	0.9	0.1	4.7	0.06	10
12	3/7/1983 21:00	3/8/1983 18:00	0.2	0.1	1.1	0.05	1
13	5/5/1988 6:00	5/10/1988 20:00	0.3	0.1	2.1	0.02	1
14	2/22/1987 20:00	2/23/1987 8:00	0.3	0.1	0.8	0.06	1
15	5/3/1984 14:00	5/3/1984 23:00	0.4	0.1	1.3	0.13	1
16	10/22/1983 21:00	10/24/1983 0:00	0.2	0.1	2.5	0.09	1
17	10/9/1979 18:00	10/10/1979 11:00	0.2	0.1	1.6	0.09	1
18	4/4/1984 9:00	4/5/1984 16:00	0.2	0.1	1.1	0.03	1
19	3/28/1984 3:00	3/29/1984 11:00	0.4	0.1	2.4	0.07	1

Table 4.6. Twenty-four major storms affecting Northwest Branch of Anacostia River from 01/01/1979 through 12/31/1988 (continued).

Precip. Event	Starting time	Ending time	Max P inches of rain in 15 min.	Min P inches of rain in 15 min	Total P in inches of rain per storm	Rain intensity in inches per hour	Estimated return period in years
20	9/11/1987 23:00	9/13/1987 9:00	1.1	0.1	3.1	0.09	2
21	3/22/1982 0:00	3/22/1982 2:00	0.8	0.8	2.4	0.80	1
22	10/16/1984 12:00	10/16/1984 12:00	0.1	0.1	0.1	0.10	1
23	3/15/1987 19:00	3/16/1987 5:00	0.1	0.1	0.2	0.02	1
24	8/28/1987 1:00	8/28/1987 12:00	0.3	0.1	0.4	0.03	1

The peak discharges resulting from the hot restart constant land use scenarios were divided by the drainage area of each sub-watershed and the peak rainfall intensity of the associated storm. These unit peak discharges were graphed against the imperviousness corresponding to the land use cover distribution for each sub-watershed during the year in which the peak occurred. Results were shown in Figure 3.12.

In Figure 3.12, it can be seen that discharges from each sub-watershed tend to group into sets of linear trends. It can also be seen that there is continuity in the linear trend from sub-watersheds with low development to sub-watersheds with higher development for the same storm. Also, in most of the cases, an increase in the imperviousness corresponds to an increase in the unit peak discharge. Two new points of investigation were determined from this analysis: (1) Identify the causes for the negative slopes, meaning a decrease in discharge when imperviousness increased and (2) find a way to estimate the slope for each storm based on known storm and watershed characteristics.

#### 4.4.1.1. Composite storms

A precipitation event is defined in this study as a period of time where there is some precipitation with dry periods less than one day between each one. One value of such definition is to determine antecedent precipitation events that may have had some influence in the peak discharge under analysis. It was hypothesized that some conditions like antecedent moisture conditions or the influence of past storm hydrographs may alter the hydrograph in a way such that discharges classified as peaks were not the result of merely the effects of a single storm. Several such composite storms are shown in Figure 4.8.

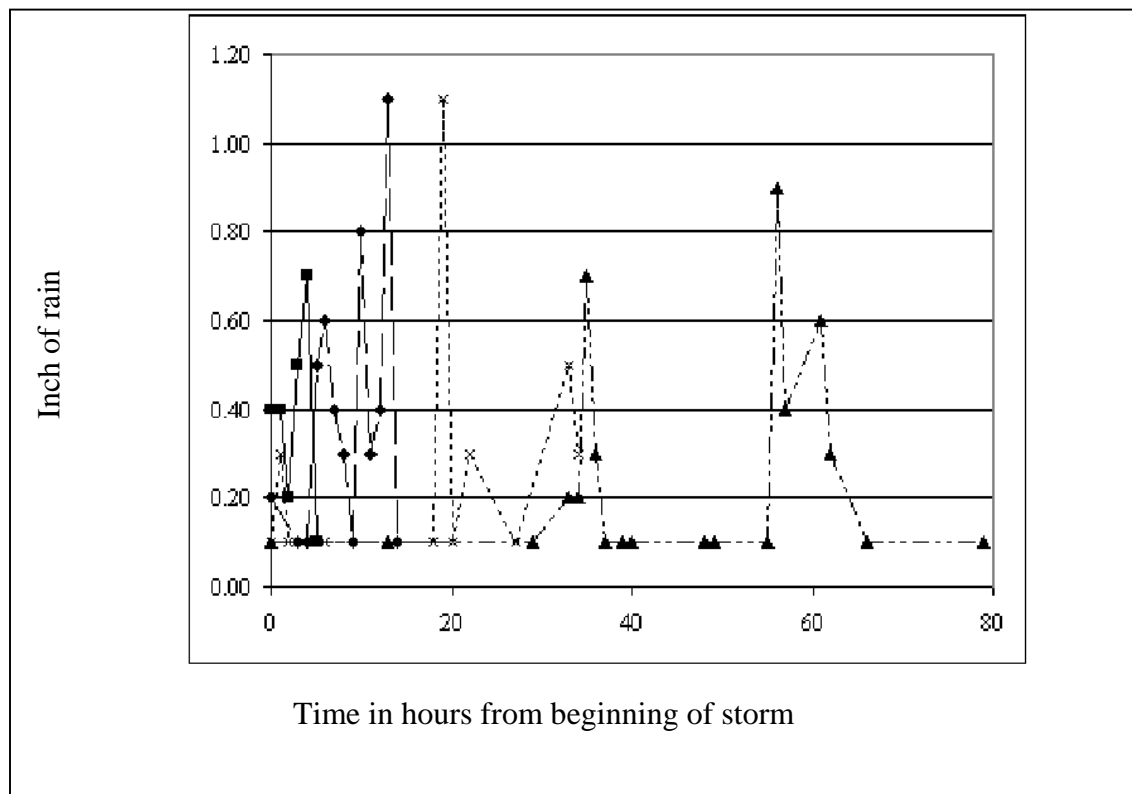


Figure 4.8. Four composite events showing previous storm events for raingage 180700.

The storm that caused the peak of 09/06/79 (shown as the triangles and semi-dashed line in Figure 4.8) was selected to analyze the effect of composite storms in the peak discharge modeling. A SWMM simulation was performed for a unit storm over one hour period. The unit storm was added several times with different timing until the composite storm was reproduced. The hydrograph resulting from the unit storm hyetograph was added in the same sequence as the unit storm to produce a synthetic composite hydrograph. Comparisons between these two composite hydrographs (synthetic and modeled) indicated similarities.

Unit hydrographs were obtained for the land use distributions of 1979 and 1988. Each individual one showed that the peak was smaller for 1979 than for 1988. Imperviousness in 1979 is 14.4% while it is 18.7% for 1988 confirming the hypothesis that peaks are higher for higher imperviousness. Nevertheless, the peaks after convoluting the unit hydrographs for the composite storms showed an adverse behavior. Figure 3.12.

Having proven that an inverse relation between peaks and imperviousness is not a result of the model but of some other nature, we left this as subject for further research and chose to focus on simple storms rather than composite storms.

#### **4.4.1.2. Final model**

Linear regressions were done for each storm with a positive relationship between peak discharges and imperviousness. Slopes for the previous mentioned relationship, which we will refer to as “m”, as described in equation 4.14 were plotted vs. the

dynamically changing land use discharge per drainage area per rain intensity as shown in Figure 3.13.

$$Q_{phcl} = m * imp \quad (4.14)$$

where:

$$Q_{phcl} = \text{Hot-start constant land use discharge per drainage area per rain intensity in } \frac{ft^3 / s}{sq.mi.*inch \text{ of rain}} .$$

$imp$  = Imperviousness for the year in which the peak discharge occurred in impervious area per total drainage area.

$m$  = Linear regression coefficient in  $ft^3/s$  per square mile per inch of rain per imperviousness.

Slope coefficients were compared to the dynamically changing land use discharges per drainage area per rain intensity as shown in Figure 3.13. From this figure it can be seen that “m” increases as the dynamically changing land use discharge increases. It can also be seen that there seems to be a linear relationship between m and dynamic changing land use unit discharge. Finally it can be seen that the value of m is also greater for more developed watersheds than for less developed watersheds.

The equation describing this relationship can be stated as follows:

$$Q_{dyn} = k * m \quad (4.15)$$

where:



$Q_{dyn}$  = Dynamically changing land use peak discharge per drainage area per rain intensity in  $\frac{ft^3 / s}{sq.mi.* inch\ of\ rain}$ .

$m$  = Slope from equation 4.15 in inverse of imperviousness.

$k$  = Regression equation constant for each sub-watershed in  $ft^3/s$  per square mile per inch of rain times the imperviousness.

Values of  $k$  are shown and  $R^2$  values are shown in Table 4.7.

Table 4.7.  $k$  and  $R^2$  for dynamically changing land use unit discharge vs.  $m$ .

Sub-watershed	$k$	$R^2$	$1/k$
1029	10.951	0.9794	0.09
1	9.1044	0.9937	0.11
9	5.6375	0.9745	0.18
10	3.1618	0.9782	0.32

It was found that  $k$  values can be estimated from equation 4.16 resulting from a linear regression between  $k$  and imperviousness for each sub-watershed.

$$k = 0.9753 * imp^{1.05} \tag{4.16}$$

where:

$k$  = Regression equation constant for each sub-watershed in  $ft^3/s$  per square mile per inch of rain times imperviousness.

$imp$  = Imperviousness.

By substituting  $m$  from equation 4.16 and 4.14 into equation 4.15 we obtain a relationship between dynamically changing land use unit discharge and constant land use unit discharge as follows:

$$Q_{pdlu} = 1.025 * Q_{phcl} * \frac{(imp_{pdlu})^{1.05}}{imp_{hcl}} \quad (4.17)$$

where

$Q_{pdlu}$  = Peak discharge per area in square miles per inch of rain for dynamically changing land use model results in  $\frac{ft^3 / s}{sq.mi.*inch\ of\ rain}$ .

$Q_{phcl}$  = Peak discharge per area in square miles per inch of rain for hot-start constant land use model results  $\frac{ft^3 / s}{sq.mi.*inch\ of\ rain}$ .

$imp_{pdlu}$  = Imperviousness for year of the storm in impervious area per total area.

$imp_{hcl}$  = Imperviousness corresponding to the land use cover used in the hot-start constant land use model in impervious area per total area.

Equation 4.17 means that the peak discharges for Northwest Branch of Anacostia River modeled by SWMM are proportional to the ratio of imperviousness under dynamic vs. constant land use change conditions. Peak discharges can be adjusted to represent a land use condition if they are multiplied by the appropriate ratio of imperviousness. Having found the adjustment factor for peaks we proceed to work on baseflow to complete the adjustment of a modeled discharge series.

#### 4.4.2. Baseflow

A similar procedure as the one followed in the peak model development was used in the baseflow model development. The ratio between baseflow resulting from the

dynamically changing land use model and the hot-start constant land use model was graphed against the ratio of the imperviousness of the dynamically changing model and the imperviousness of the land use cover used in the hot-start model. Results are shown in Figure 3.14.

It can be seen from Figure 3.14 that there are several linear trends but all cross close to the point (1,1). This means that the baseflows from both models (dynamically changing and hot-start) are similar as for events occurring in the year of the land use cover used in the hot-start model (e.g., when both imperviousness fractions are the same).

There is a negative relationship between both of the ratios previously described. Considering a certain land use cover, e.g., 1982, the ratio of imperviousness from 1979 until 1981 and the imperviousness of 1982 is less than one, because land use is less developed for previous years, but for years 1983 and over this ratio is greater than one.

Considering the same year under analysis, baseflows for years previous to 1982 are higher than those from 1982 therefore the ratio of dynamically changing land use and hot-start constant land use are greater than one. On the contrary, baseflow for years 1983 and over are smaller, therefore the ratio of baseflows resulting from the dynamically changing land use model and hot-start constant land use model are less than one. In conclusion, Figure 4.7 supports the hypothesis that the hot-start model simulated discharges are a better prediction of the baseflow discharges than the constant land use model simulated discharges. Linear regressions were performed for all the land use covers and the envelop boundary lines were selected to be a first approximation in the modeling of baseflow as described below. The results are shown in equations 4.17 through 4.20.

$$\frac{q_{dyn}}{q_{hcl}} = -0.1654 * \frac{imp_{dyn}}{imp_{hcl}} + 1.1654 \quad (4.18)$$

$$\frac{q_{dyn}}{q_{hcl}} = -1.2836 * \frac{imp_{dyn}}{imp_{hcl}} + 2.281 \quad (4.19)$$

$$\frac{q_{dyn}}{q_{hcl}} = -0.1254 * \frac{imp_{dyn}}{imp_{hcl}} + 1.1254 \quad (4.20)$$

$$\frac{q_{dyn}}{q_{hcl}} = -1.5408 * \frac{imp_{dyn}}{imp_{hcl}} + 2.5381 \quad (4.21)$$

where:

$q_{dyn}$  = Dynamically changing land use baseflow discharge in ft<sup>3</sup>/s per square mile.

$q_{hcl}$  = Hot-start constant land use baseflow in ft<sup>3</sup>/s per square mile.

$imp_{dyn}$  = Imperviousness from dynamic annually changing land use in pervious area per total area.

$imp_{hcl}$  = Imperviousness from constant land use in pervious area per total area.

Two envelope equations were defined arbitrarily in order simplify the model and considering the similarity of the boundaries. Equations 4.20 and 4.21 include the other two envelop equations and were the ones selected to develop the baseflow model. The following procedure was used to adjust final baseflow estimates. Extreme values for the baseflows were calculated using the two equations. Considering that the most accurate baseflow estimation must lie between these two boundaries, the following equations must be fulfilled:

$$q = q_{\min} * N + q_{\max} *(l-N) \quad (4.22)$$

where:

$q$  = Simplified model approximation of baseflow in ft<sup>3</sup>/s per square mile.

$q_{\min}$  = Lower baseflow boundary in ft<sup>3</sup>/s per square mile.

$q_{\max}$  = Upper baseflow boundary in ft<sup>3</sup>/s per square mile.

$N$  = Dimensionless proportionality factor.

Several baseflow events were modeled and N value was found for each one. A linear regression was performed between N and imperviousness for each year of the corresponding baseflow in order to estimate N. The proportional factor, N, is defined by equation 4.23 as follows:

$$N = 1.2405 - 3.42 * imp_{hcl} \quad (4.23)$$

where:

$N$  = Dimensionless proportional factor as defined in equations 4.22 and 4.23.

$imp_{hcl}$  = Fraction of imperviousness for hot restart model in pervious area per total area.

#### 4.4.3. Water quality

The goals of the water quality modeling were to develop a relationship between the change in pollutant concentration caused by a change in land use distribution and to be able to predict the total pollutant load without using the dynamic annually changing land use SWMM model. SWMM offers different methods for estimating pollutant loads that can be used depending on the amount of available information. A method based on the

event mean concentration (EMC) that is the average load for total discharge in a storm. The same method of pollutant modeling was used for all other pollutants, therefore the conclusions derived for one of them would be applicable to all. Total phosphorus will be used to develop this method. Pollutant loads were first modeled by using the hot-start continuous land use. The difference between each pollutant load for a given year for each land use condition was compared to the difference in imperviousness for land use in different years. Differences for previous and subsequent years were plotted against the changes in imperviousness (Figure 3.15).

Pollutants accumulate on the land surface during dry periods and are washed off into the streams by rainfall events; therefore, pollutant loads are lower in dry years than in the wet years. The SWMM model used in this research was set up to model pollutant loads resulting from storms and not from baseflows; therefore, in order to adjust the pollutant loads from one year to another, it is necessary to take into consideration the total volume of peak discharges. The total volumes of peak discharges were obtained from the adjusted water quantity model and are shown in Table 4.8 for years 1979 through 1988.

Table 4.8. Simulated total peak discharge volumes and total phosphorus pollutant load for Northwest Branch of Anacostia River watershed for years 1979 through 1988 resulting from the SWMM model.

<b>Year</b>	<b>Fraction of imperviousness</b>	<b>Total volume of peak discharges in ft<sup>3</sup>*day/s</b>	<b>Total phosphorus load from dynamic annually changing land use model in (mg/l)*(ft<sup>3</sup>*day/s)</b>
1979	0.1442	223.69	36,890
1980	0.1479	24.89	37,169
1981	0.1498	191.75	37,439
1982	0.1533	88.66	38,025
1983	0.1573	121.59	38,246
1984	0.1643	132.51	40,852
1985	0.1698	57.38	41,193
1986	0.1757	281.63	42,171
1987	0.1813	280.58	42,439

The SWMM model total phosphorus load results from the dynamic annually changing land use were plotted against the total peak discharge volumes as shown in Figure 4.9. A linear regression with zero intercept was performed on this series of data as shown in equation 4.24. The correlation coefficient for this linear regression is  $R^2 = 0.9731$ .

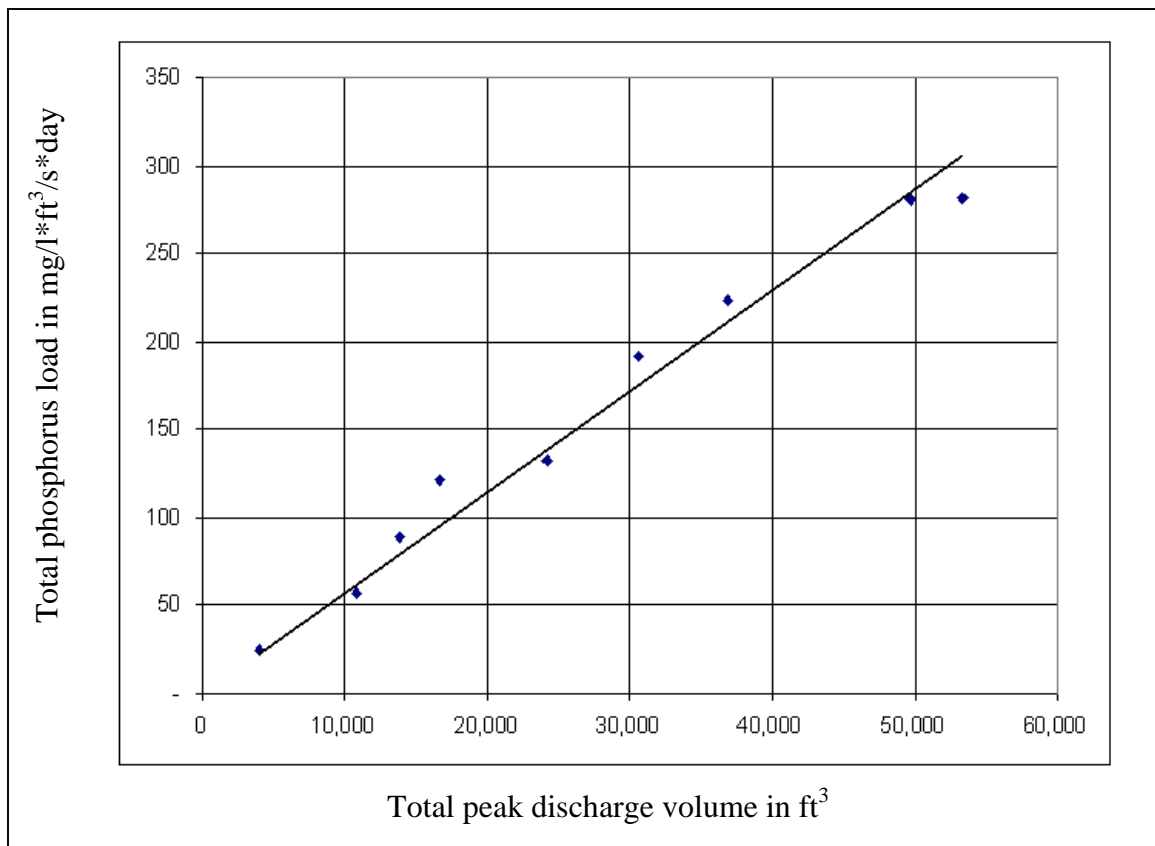


Figure 4.9. Total pollutant loads compared to total peak discharges estimated from SWMM model for the Northwest Branch of Anacostia River for year 1979 through 1988.

$$TL = 173.96 * V_{peak} \tag{4.24}$$

where:

$TL =$  Total load of pollutant in (mg/l)\*(ft³\*day/s).

$V_{peak} =$  Total volume of peak discharges in ft³.

#### 4.4.3.1. Simplified water quality model

Results supported the hypothesis of a direct relationship between peak flows and imperviousness, as well as an inverse relationship between baseflows and imperviousness. Considering that the imperviousness is an index of the amount of development in a watershed, the hypothesis of higher peak flows and lower baseflows



correspond to a more developed watershed when compared to less developed watersheds. This is consistent within data reported in the literature (CWR and MDE, 2000; Schueler, 1987; Klein, 1979; Barringer et al., 1994; Paul and Meyer, 2001).

Total phosphorus annual loads increase as imperviousness increases (Figure 4.9). It can be seen that the change in total loads is more dispersed for larger changes in imperviousness fraction than for small changes in imperviousness. Also it seems that the ratio of the change in total pollutant load is smaller for smaller changes in the imperviousness than for larger changes in imperviousness. This may be due to different land use changes (e.g., some watersheds have more residential development while others have more residential and others more commercial or industrial development) that lead to similar changes in imperviousness but with different EMC values.

The increase in total load is indirectly related to the fraction of imperviousness. Imperviousness is representative of development, and one of the consequences of development is an increase in pollutant load. Nevertheless, the pollutant load depends on the type of land use rather than just the amount of imperviousness. It was out of the scope of this research to determine a relationship between EMC and ratio of imperviousness, but it was found that there is a linear correlation between the total annual load and the total volume of peak discharges. Table 4.9 shows the imperviousness and the EMC values for total phosphorus for different land uses. The EMC is plotted against the imperviousness as shown in Figure 4.10. It can be seen that residential and commercial development have similar imperviousness ratios, but the total phosphorus EMC is higher for commercial than for residential development. If the changes in land use were from forested to predominantly urbanized land use (e.g., from agriculture to residential only),

then Figure 3.15 will give a better approximation of the total phosphorus load than if the land use changes were from several undeveloped land uses to several urbanized land uses (e.g., from agriculture or forested to residential, commercial, or industrial).

Table 4.9. Total phosphorus EMC values in mg/l and the percent of imperviousness per land use classification. Percent of imperviousness are taken from TR-55 (Soil Conservation Service, 1986) and EMC values are taken from NURP study. (NURP, 1983).

<b>Land use code</b>	<b>Land use description</b>	<b>Percentage of imperviousness</b>	<b>EMC TP mg/l</b>
11	Low density residential	25%	0.383
12	Medium density residential	30%	0.383
13	High density residential	65%	0.383
14	Commercial	82%	0.201
15	Industrial	70%	0.201
16	Institutional	50%	0.250
17	Extractive	11%	0.250
18	Open urban land	11%	0.250
21	Cropland/Pasture	0%	1.200
22	Orchards	0%	1.200
41	Deciduous forest	0%	0.121
42	Evergreen forest	0%	0.121
43	Mixed forest	0%	0.121
70	Barren Land	50%	0.250
72	Bare ground	100%	0.250
73	Sandy Areas	50%	0.250
191	Large lot forest	15%	1.200
192	Large lot agriculture	15%	0.121
241	Feeding operations	10%	1.200
242	Other agricultural land	10%	1.200

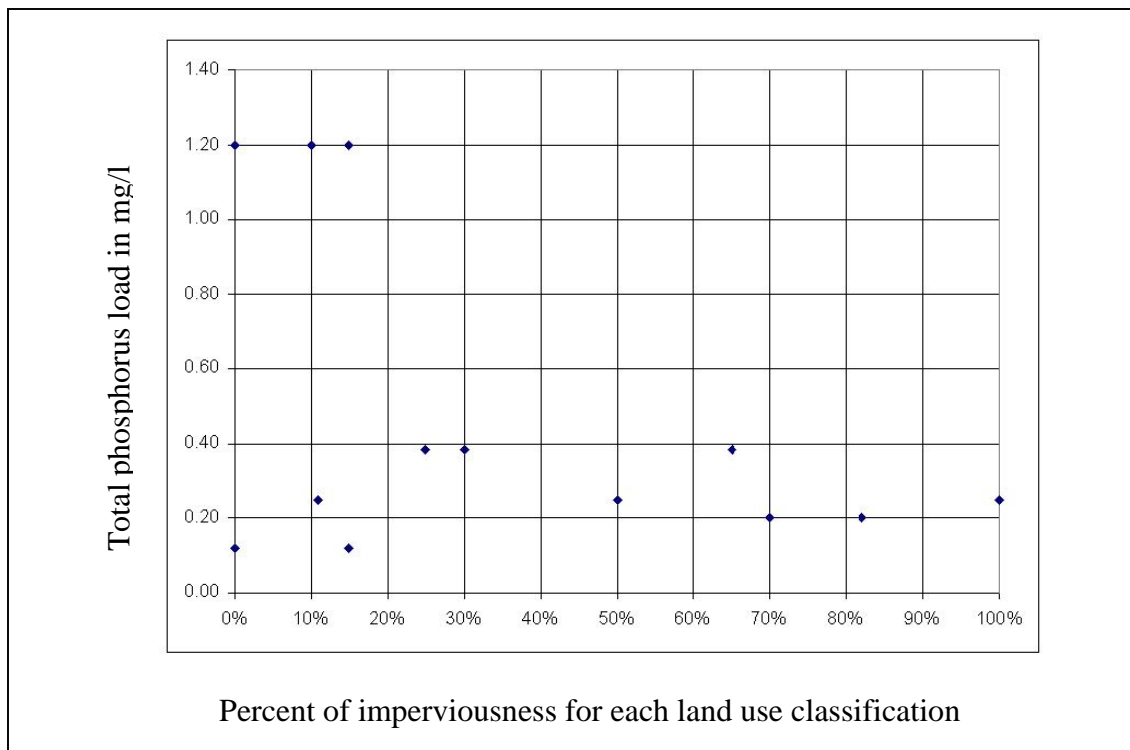


Figure 4.10. Total Phosphorus EMC values in mg/l as a function of imperviousness corresponding to each land use classification.

#### 4.5. Model application

In this section we will outline the steps to follow in order to adjust a constant land use SWMM simulation time series to reflect the effects of changing land use through time. These steps take the constant land use SWMM model results and an annual imperviousness time series to create an adjusted discharge and pollution time series.

##### 4.5.1. Water quantity and model results

###### 4.5.1.1. Base discharge time series to be adjusted

The first step is to generate the time series to be adjusted. This time series is the result of a calibrated SWMM model following the traditional SWMM modeling processes based on constant land use. It is also necessary to create a time series of the imperviousness based on additional information such as land use maps or census data.

#### **4.5.1.2. Separation of peaks and baseflows**

The peak and baseflow discharges must be separated. There are several methods that could be used. In this study a second series of discharges based on a more developed different land use distribution was generated and compared to the hot-start constant land use discharge series. In this study, and for the simplified adjustment method, peak discharges are defined as those that are higher for the corresponding more developed land use. Baseflows are defined as those discharges that are lower for the more developed land use scenario and that are lower than a threshold value that was selected arbitrary based on an analysis of the discharges. This threshold value eliminates considering some small storms that are more dominated by baseflow than by surface runoff process to be modeled as peaks.

A sample storm recorded in November 1984 was selected and is shown in Table 4.10 with: the SWMM model results from constant land use distribution (1979 and 1988), the dynamic annually land use changing model, and the adjusted discharge model results from the procedure being described. From Table 4.10 it can be seen that all discharges for which the 1988 modeled value is larger than the 1979 modeled value were classified as peaks (understanding that 1979 represents the constant land use modeling for 1979 and 1988 represents the discharges corresponding to 1988 constant land use model results for sub-watershed 1 of the Northwest Branch of the Anacostia River).

The discharge of  $4.27\text{ft}^3/\text{s}$  following the peak discharge of  $68.77\text{ft}^3/\text{s}$  was classified and modeled as a peak discharge although it occurs on the recession limb. In

this particular case this discharge is more influenced by the large peak discharge than by the baseflow regime.

Table 4.10. Simulated discharges for sub-watershed 1 of the Northwest Branch of the Anacostia River, period 11/01/84 through 11/10/84.

Date	Dynamic annually changing land use Imp = 0.1643	Hot restart with 1979 land use Imp = 0.1442	Hot restart with 1988 land use Imp = 0.1869	Adjusted discharges
11/1/1984	0.08	0.09	0.08	0.08
11/2/1984	0.07	0.07	0.06	0.07
11/3/1984	0.07	0.08	0.07	0.07
11/4/1984	0.07	0.07	0.06	0.07
11/5/1984	68.77	61.13	79.62	65.25
11/6/1984	4.27	3.94	4.71	4.21
11/7/1984	0.77	0.80	0.70	0.75
11/8/1984	0.21	0.22	0.19	0.21
11/9/1984	0.10	0.11	0.09	0.10
11/10/1984	0.15	0.16	0.13	0.14

The adjustment process applied is described as follows. The discussion is divided into peak flows and baseflow by applying equations 4.17, 4.18, 4.19, and 4.23.

#### 4.5.1.3. Peak adjustment

Equation 4.25 is obtained by replacing the values of imperviousness in equation 4.17 ( $imp_{pdlu} = 0.1643$  and  $imp_{hcl} = 0.1442$ ). Applying this equation to 1979 peak discharges of  $61.13\text{ft}^3/\text{s}$  and  $3.94\text{ft}^3/\text{s}$  taken from Table 4.10, the simplified adjusted peaks of  $65.25\text{ft}^3/\text{s}$  and  $4.21\text{ft}^3/\text{s}$  are obtained. Computations for the  $65.25\text{ft}^3/\text{s}$  are shown below:

$$Q_{pdlu} = 61.13 * \frac{(0.1643)^{1.05}}{0.9753 * (0.1442)} \quad (4.25)$$

$$Q_{pdlu} = 65.25 \text{ ft}^3 / \text{s} / \text{acre} / \text{inch of rain}$$

where:

$Q_{pdlu}$  = Peak discharge per area in square miles per inch of rain for simplified method model results in  $\frac{\text{ft}^3 / \text{s}}{\text{sq.mi.} * \text{inch of rain}}$ .

$Q_{phcl}$  = Peak discharge per area in square miles per inch of rain for hot-start constant land use model results in  $\frac{\text{ft}^3 / \text{s}}{\text{sq.mi.} * \text{inch of rain}}$ .

Values are summarized in Table 4.10. Comparing the adjusted discharges with their corresponding 1979 land use distribution peak discharges, it can be seen that the adjusted peaks are 6.73% larger than the 1979 peak discharges. This shows that the effect of a larger imperviousness is causing a larger peak discharge as expected. The constant land use 1979 major peak discharge underestimates the corresponding dynamic annually changing discharge by 11.1% while the adjusted discharge underestimates the corresponding dynamic annually changing discharge by 5.1%.

Comparing the major adjusted peak discharge of 65.25ft<sup>3</sup>/s with the corresponding peak discharge of 1988 land use distribution, the adjusted peak discharge is approximately 82% of the 1988 peak discharge. The adjusted peak is an intermediate value between the peak discharges from both extreme constant land use conditions, 1979 and 1988 (61.13 and 79.62 ft<sup>3</sup>/s, respectively). Comparing both of them to the dynamic

annually changing land use peak discharge, the 1988 peak discharge overestimation is 15.8% while the adjusted discharge underestimation is just 5.1% which is lower than the 1988 constant land use model peak discharge. The absolute difference is reduced from 15.8% to 5.1%. There is an improvement in the individual predictions of the peak discharges by adjusting the peak discharges when comparing them with the constant land use peak discharges.

The total volumes of peak discharges for 1979 through 1988 were 1788.8 ft<sup>3</sup>\*day/s, 1907.9 ft<sup>3</sup>\*day/s, 1956.4 ft<sup>3</sup>\*day/s, and 2037.5 ft<sup>3</sup>\*day/s for model results from 1979 constant land use, dynamic annually changing land use, adjusted peak discharges, and 1988 constant land use respectively. From these values, it can be seen that the adjusted peak method described in this research reduced the error of estimation of the peak discharges from an underestimation percentage of 6.2% for constant 1979 land use model results and an overestimation percentage of 6.8% for constant 1988 land use model results to an over estimate of 2.5% when comparing the total volume of peak discharges from 1979 model and adjusted discharges to represent the dynamic annually changing land use from the 1979 discharges as described above (Table 4.11).

Table 4.11. Summary of water quantity total volumes in ft<sup>3</sup>\*day/s for the period 1979 through 1988 resulting from four different scenarios (Constant land use 1979, constant land use 1988, dynamic annually changing land use, and simplified water quantity model), and error in percent when compared to the dynamic annually changing land use discharges. Simplified water quantity model is a better estimation for dynamic annually changing land use model than 1979 and 1988 constant land use model estimations.

	<b>1979 constant land use</b>	<b>Dynamic annually changing land use</b>	<b>Simplified water quantity model</b>	<b>1988 constant land use</b>
<b>Peak discharges in ft<sup>3</sup>*day/s</b>	1788.8	1907.9	1956.4	2037.5
<b>Peak discharge error in percent</b>	-6.2	N/A	+2.5	+6.8
<b>Baseflow in ft<sup>3</sup>*day/s</b>	798.1	743.9	733.7	647.8
<b>Baseflow error in percent</b>	+7.3	N/A	-1.4%	-13.0%

#### 4.5.1.4. Baseflow adjustment

Two baseflow discharges resulting from either equations 4.18 and 4.19 or 4.20 and 4.21 are computed. The set of equations (4.18/4.19, 4.20/4.21) are selected if the adjustment is done from a less developed land use scenario to a more developed scenario or from a more developed scenario to a less developed scenario, respectively.

The following are the computational procedures using data from Table 4.10. Computation for November 10, 1984, will be shown, but similar adjustments were applied to all baseflow values. The discharges from 1979 will be adjusted following the simplified method in order to account for a more developed land use distribution, 1984; therefore, equations 4.18 and 4.19 will be used. Equations 4.26 and 4.27 were developed by replacing impervious fractions from Table 4.10 in equations 4.18 and 4.19 respectively, as follows:



$$\frac{q_{dyn}}{q_{hcl}} = -0.1654 * \frac{0.1643}{0.1442} + 1.1654 \quad (4.26)$$

$$q_{dyn} = 0.9769 * q_{hcl}$$

In a similar way from equation 4.19 and by replacing imperviousness values it can be found that:

$$\frac{q_{dyn}}{q_{hcl}} = -1.2836 * \frac{0.1643}{0.1442} + 2.281 \quad (4.27)$$

$$q_{dyn} = 0.8185 * q_{hcl}$$

The adjustment factor,  $N$ , from equation 4.24 is calculated by replacing imperviousness values from Table 4.10 as follows:

$$\begin{aligned} N &= 1.2405 - 3.42 * imp_{hcl} \\ N &= 1.2405 - 3.42 * 0.1442 \\ N &= 0.7473 \end{aligned} \quad (4.30)$$

From equations 4.27, 4.29, 4.22, and 4.30 and from values of discharges from Table 4.10 the final adjusted baseflow is calculated as follows:

$$q_{min} = 0.9769 * 0.16 = 0.1563 \quad (4.31)$$

$$q_{max} = 0.8185 * 0.13 = 0.1064 \quad (4.32)$$

$$\begin{aligned}
q_{adj} &= q_{min} * N + q_{max} * (1-N) \\
q_{adj} &= 0.1563 * 0.7473 + 0.1064 * (1-0.7473) \\
q_{adj} &= 0.1437
\end{aligned}
\tag{4.33}$$

where:

$q_{min}$  = Minimum baseflow boundary estimated from equation 4.28 in ft<sup>3</sup>/s.

$q_{max}$  = Maximum baseflow boundary estimated from equation 4.29 in ft<sup>3</sup>/s.

$q_{adj}$  = Final adjusted baseflow in ft<sup>3</sup>/s.

From Table 4.10 it can be seen that the baseflow discharges of November 10, 1984, are as follows: 0.16ft<sup>3</sup>/s for the 1979 constant land use, 0.15ft<sup>3</sup>/s for the dynamic annually changing land use, 0.14 ft<sup>3</sup>/s for the adjusted baseflow discharge, and 0.13 ft<sup>3</sup>/s for the 1988 land use. The adjusted baseflow is a better estimation of the dynamic annually changing land use than either the baseflow from 1979 or 1988, constant land use scenarios.

The total volumes of the baseflow discharges were analyzed for the period starting on 1/1/79 and ending on 1/1/1988. The total volume of baseflow discharges were 798.1 ft<sup>3</sup>\*day/s, 743.9 ft<sup>3</sup>\*day/s, 733.7 ft<sup>3</sup>\*day/sft<sup>3</sup>, and 647.8 ft<sup>3</sup>\*day/s, for model results from 1979 constant land use, dynamic annually changing land use, adjusted baseflows and 1988 constant land use respectively. From these values, it can be seen that the adjusted baseflow method improved the overestimate of baseflows from 7.29% (1979 constant land use model) and underestimate of 13% to an underestimation of 1.4% (Table 4.11).

## **4.5.2. Water quality**

### **4.5.2.1. Pollutograph time series**

Two different approaches can be taken to adjust the pollutant loads: (1) adjust the total load for a given year assuming a different land use distribution, and (2) estimate the total load of a pollutant, phosphorus in this research, from the peak discharge volume. The first method can be used to analyze changes in pollutant load by eliminating the effects of precipitation. The second method can be used to estimate the loads resulting from a given land use distribution and precipitation.

The first method requires two values as input. The first value is the total load from the land use that is being adjusted and the second value is the change in imperviousness. Using Figure 3.15, it is possible to enter the graph with the difference in imperviousness and obtain the additional load to be added or subtracted from the estimated value.

The annual total phosphorus loads for the Northwest Branch of Anacostia River are shown in Table 4.12 along with the imperviousness fraction and the additional load to be added to the 1979 annual total phosphorus load to adjust it to each one of the different land uses (e.g., 1980 through 1988). It can be seen that the model overestimates the loads for larger changes in imperviousness, as would be expected.

Table 4.12. Simulated total annual phosphorus loads for Northwest Branch of Anacostia River.

Year	Imperviousness fraction	Difference in imperviousness	Change in total pollutant load loads in pounds	Values from dynamic annually changing land use loads in pounds	Adjusted total loads in pounds
1979	0.1442		0	198,976	198,976
1980	0.1479	0.0037	1,348	200,481	200,325
1981	0.1498	0.0057	4,045	201,938	203,022
1982	0.1533	0.0092	10,788	205,098	209,764
1983	0.1573	0.0136	26,969	206,290	225,945
1984	0.1643	0.0200	53,938	220,347	252,914
1985	0.1698	0.0256	80,907	222,186	279,883
1986	0.1757	0.0315	102,482	227,461	301,458
1987	0.18123	0.0371	113,269	228,906	312,246
1988	0.18521	0.0398	148,330	233,776	347,296

For the second method the first step in estimating the total load of phosphorus is to create a peak discharge time series. This time series can be obtained and adjusted by following the peak adjustment procedure previously described in the water quantity section (4.9.1.3.). Pollutographs can be derived by applying equation 4.24 to the peak discharges. In this example we will generate a synthetic annual total load series for phosphorus for the same period shown in the water quantity adjustment section. Table 4.13 shows the peak discharge time series after being adjusted.

The values of the synthetic total phosphorus load are obtained by applying equation 4.34 as follows:

$$\begin{aligned}
 TL &= 173.96 * V_{peak} \\
 TL &= 173.96 * 68.77 \\
 TL &= 11,963 \text{ mg / l} * \text{ft}^3 / \text{s} * \text{day} \\
 TL &= 64,526 \text{ lb}
 \end{aligned}
 \tag{4.34}$$

where:

$TL =$  Total phosphorus load in  $(\text{mg/l}) * (\text{ft}^3 * \text{day} / \text{sft}^3)$ .

$V_{peak} =$  Total volume of peak discharges in  $\text{ft}^3$ .

Table 4.13. Dynamic changing land use total phosphorus load compared to the total phosphorus load from equation 4.34 for the Northwest Branch of Anacostia for November 5, 1984 storm.

<b>Date</b>	<b>Adjusted peak discharge</b>	<b>Total phosphorus load from equation 4.24 in pounds</b>	<b>Modeled total phosphorus load in pounds</b>
11/05/84	68.77	64,526	73,355
11/06/84	4.27	4,008	4,428

#### **4.6. Conclusions**

In this research we have developed a model that can adjust SWMM modeled discharges (peak flows and baseflows) resulting from a constant land use based on imperviousness. A time series of imperviousness represents the changing land use distribution over a period time. In the same way, a model to adjust pollutant concentration was developed based on the difference between imperviousness.

Discharge model adjustment is performed for peak discharges and baseflow discharges. Peak discharges are adjusted by multiplying the constant land use SWMM model discharges by a function of the ratio of the imperviousness fractions from each land use, imperviousness from the year that discharges are being modeled to and the imperviousness from the year of the SWMM model simulated discharges.

In a similar way, baseflows are adjusted by a two-step procedure. First two baseflow extreme values are estimated by applying the two linear boundary equations to the values of imperviousness. Then either of these extreme values is adjusted by applying

the interpolation adjustment ratio. This ratio is a function of the imperviousness of the land use distribution from the year that is being modeled.

We also developed a model for pollutant concentration adjustment that is based on adding or subtracting load concentrations estimated from the difference in imperviousness between the constant land use imperviousness ratio and the land use to which adjustment is desired.

The SWMM model is widely used in the United States and world wide to estimate water quantity and water quality. The SWMM model simulates the effects on hydrology and pollutant loading in a watershed resulting from a constant land use distribution. In this research we have developed a procedure to model variable land use by reading as many as state variables as possible from the end of a simulation and transferring them to the input file for subsequent simulation. Although some of the state variables could not be updated (because these values were unknown or could not be initialized) these variables had little influence on model results.

Based on the dynamic changing land use model results, we developed a simpler method for adjusting a constant land use model results to reflect the effects of changing land use. This adjustment method requires two constant land use model simulations, which can be used to identify peak discharges and baseflows, and to generate a dynamic changing land use adjusted discharge series without the need to execute the dynamic annually changing SWMM model at all.

## Chapter 5

### Conclusions and Recommendations

#### 5.1. Conclusions

In this research we have developed a GIS interface to create the SWMM input file from GIS data. This interface also reads SWMM output files and updates state variables to be used in further SWMM simulations in order to create a dynamic changing land use simulation. The Arc View interface processes GIS data and relational tables to obtain average values that characterize the watershed. SWMM requires large amounts of input for each sub-watershed, which can be time consuming to put together manually. SWMM output files for continuous simulations are large text files that are cumbersome to glean results from. Therefore the interface was designed to read and process the output file to summarize the time series results for water quantity and water quality. The Arc View interface also played an important role in the calibration process where changes were made to the relational tables and the interface updated the input file and executed the SWMM model.

Watershed analysis has lately been oriented towards a better understanding of human impacts on water quantity and quality. In this study human impacts are quantified by the imperviousness resulting from changes in the land use. Therefore, watershed analysis includes not only the peak events but also takes into account baseflow and intermediate events.

Constant land use models provide conditions at different development stages without considering the dynamic changes in land use; continuous simulation provides a continuous monitoring of the changes in a watershed. Continuous modeling can be used as a predictive tool to model planning development alternatives.

Another goal achieved in this work is the use of the GIS interface to generate input files to simulate a dynamically changing land use continuous SWMM model by annually updating as many state variables as possible with the values contained in the SWMM output file. This task was considerable given the number of sub-watersheds (about 50) and the fact that 10 years of annual updates were required in the simulation. This goal was achieved by reading the state variables, last value for each one of the sub-watersheds from the SWMM output file and updating the initial value for each state variable in the input file for the following year simulation. Some variables (water remaining on the channels, for example) could not be updated because they are not printed in SWMM output file.

Probably the most significant contribution of this was the creation of a model to adjust the SWMM derived constant land use water quantity and quality results to account for dynamically changing land use. Adjusted discharges were closer to the observed flows without running the more complicated dynamic SWMM model. Discharge adjustments are made for each peak discharge and baseflow independently taking into consideration the different response of each type of discharge to land use.

This adjustment process reduces the time and memory required to generate flows from dynamically changing land use. The processing time required by the SWMM model when simulating continuous modeling depends on the time step of the precipitation input



data, the model time step for the Runoff and the Transport blocks, the length of the modeling period, and the number of sub-watersheds and interconnections among them.

In this research each simulation took between 30 minutes to one hour using a Pentium 3, 1.2 GHz, with 312K RAM while it took over three hours in a Pentium II processor. The need of powerful computers is a must in continuous simulation of SWMM.

With the advance of digital data collection and processing these are new resources available for civil engineers to make modeling based decisions. These new resources make hydrologic modeling faster and more accessible for comparison of different scenarios. Mathematical calculations based on areal averages of values based on land use, soil type, or any other classification of an area can be performed faster using GIS data representing the area under study.

## **5.2. Recommendations**

There is a need for better measures of goodness-of-fit between observed and simulated streamflow time series. Large errors in current measures such as the Nash-Sutcliffe index can unrealistically reflect the true goodness-of-fit of a simulation. Daily peak discharges that are offset in time introduce large errors in the model. Because of the scale of watersheds studied in this research, at a daily time step a peak discharge is generally followed by a baseflow discharge in the next day. Therefore the difference between the observed and the offset simulated peak is larger than if it was compared with a smaller simulated peak. For each day when a modeled peak discharge is offset there is another day with an observed baseflow that will be compared to a modeled discharge

corresponding to the offset peak. New streamflow goodness-of-fit measures are needed that would somehow quantify differences between observed and predicted discharges in a way that does not penalize such differences as severely as the Nash-Sutcliffe index and other such measures while still rewarding good agreement at the time scale of individual values.

## **Appendix A**

### **SWMM input files**

An example of the SWMM input files are shown in this addendum. Three different input files were used as input files: one for Runoff block, another for Transport block and the last one to combine the output from Runoff to Transport blocks. A basic knowledge of the SWMM block description is recommended when reading this addendum.

Northwest Branch of Anacostia River watershed was subdivided in 55 sub-watersheds draining to equal number of outlets. Outlets were identified with two different series of data: (1) sequential numbers from 1 through 25 corresponding to points with a particular interest, and (2) numbers starting at 100 where 10 represents a code for stream junction points and 0 is a sequential number for each stream junction (e.g., 1029 represents the stream junction outlet number 29).

Sub-watersheds are identified by adding 30 before the outlet identification code. Therefore, sub-watershed 30101 is draining to outlet 101, that is the stream junction 1. The GIS interface delineates the stream network from digital elevation models (DEM) by filling sinks, calculating flow direction and flow accumulation.

Channel network is identified by adding 20 before the outlet code identification number, understanding that streams have one downstream outlet only. GIS interface creates the connectivity (connections between outlets and channel identification codes), estimates the length of each channel and the cross section geometry parameters.

Drainage areas draining independently to each outlet (refer to self drainage areas in this study) are estimated and intersected with soil type grids and land use grids to estimate the sub-watershed characteristics. This data is included in H1 through H4 lines in the runoff input file.

In a similar way the transport input file is populated from the connectivity data produced by the GIS interface. Lengths, channel bank full width and depth, and channel slope are estimated from the total area draining to each outlet, that is larger or equal to the self drainage areas.

These input files are created for each land use cover, each model scenario (constant land use, hot-start, and dynamic annually changing land use). Sample for the Northwest Branch of Anacostia River continuous modeling follows.

## A.1. SWMM model runoff input file for continuous constant land use model

```

SW 1 0 9
MM 8 1 2 3 10 11 12 13 14
@ 1 'c:\thesis\rain\td3240\180700.dnt'
@ 9 'c:\thesis\northwestbranch\runoff\n01979.dnt'
$RUNOFF
*****
  TITLE LINES (A)
*****
A1 'Run of Runoff block'
A1 'NorthwestBranch watershed'
*
*****
  RUN AND PRINT CONTROL (B)
*****
* METRIC ISNOW  NRGAG  INFILM  KWALTY  IVAP  NHR  NMN  NDAY  MONTH  IYRSTR
IVCHAN
* Type of Snowmelt # of Type Quality Evap Start Start Start Start Start Channel
* Units Modeled Hyetos of Inf. Modeled Data Hour Min Day Month Year Evap
* ---- - - - - - - - - - - - - - - - - - - - - - - - - - - - - - - - - - - -
B1 0 0 1 0 1 2 00 1 30 6 1979
=====
*
* IPRN1 IPRN2 IPRN3 IRPNGW
* Print Plot Output Gndwater
* Control Control Control Messages
* ---- - - - - - - - - - - - - - - - - - - - - - - - - - - - - - - - - - -
B2 1 1 0 0
=====
*
* WET WETDRY DRY LUNIT LONG
* Wet Wet/Dry Dry Time Model
* Time Time Time Step Run
* Step Step Step Units Length
* ---- - - - - - - - - - - - - - - - - - - - - - - - - - - - - - - - - - -
B3 900 3600 21600 4 830630
*****
* EVAPORATION DATA (F)
*****
* VAP(1) VAP(2) VAP(3) VAP(4) VAP(5) VAP(6) VAP(7) VAP(8) VAP(9) VAP(10) VAP(11) VAP(12)
* Jan Feb Mar Apr May Jun Jul Aug Sep Oct Nov Dec
* ---- - - - - - - - - - - - - - - - - - - - - - - - - - - - - - - - - - -
F1 1.0 2.0 2.0 2.0 3.0 4.0 5.0 5.0 4.0 3.0 2.0 1.0
*
*****
* CHANNEL DATA (G)
*****
* NOTE: In this model, RUNOFF channels represent the following:
* (S) shallow concentrated flow
* (G) gutters
* (XX) small pipe systems
*
* NAMEG NGTO NP GWIDTH GLEN G3 GS1 GS2 G6 DFULL GDEPTH
* Channel Drain Type of Diam/ Channel Invert L Side R Side Manning Full Start
* Name Node Channel Width Length Slope Slope Slope "n" Depth Depth
* ---- - - - - - - - - - - - - - - - - - - - - - - - - - - - - - - - - - -
* MODELED UNDER TRANSPORT
*****
* SUBCATCHMENT DATA (H)
*****

```

*	JK	NAMEW	NGTO	WIDTH	WAREA	IMPERV	WSLOPE	IMPER_N	PERV_N	WSTR1	WSTR2
*	INFIL1	INFIL2	INFIL3	RMAXINF							
H1	1	301	1	19257	704	4	0.02385	0.015	0.2	0.04	0.1
	0.6	0.05	0.000115								
H2	301	1	1	0	0	20	5	5	5		
H3	4.50E-05	2.6	0	1	0	0.46	0.15	0.3	5.02	0.301	
H4	10	15	0.35	2.00E-03	14						
H1	1	302	2	2097	32	2	0.03075	0.015	0.2	0.04	0.1
	0.6	0.05	0.000115								
H2	302	2	1	0	0	20	5	5	5		
H3	4.50E-05	2.6	0	1	0	0.46	0.15	0.3	5.02	0.301	
H4	10	15	0.35	2.00E-03	14						
H1	1	303	3	9515	428.8	15	0.0181	0.015	0.2	0.04	0.1
	0.6	0.05	0.000115								
H2	303	3	1	0	0	20	5	5	5		
H3	4.50E-05	2.6	0	1	0	0.46	0.15	0.3	5.02	0.301	
H4	10	15	0.35	2.00E-03	14						
H1	1	304	4	20927	697.6	5	0.02597	0.015	0.2	0.04	0.1
	0.6	0.05	0.000115								
H2	304	4	1	0	0	20	5	5	5		
H3	4.50E-05	2.6	0	1	0	0.46	0.15	0.3	5.02	0.301	
H4	10	15	0.35	2.00E-03	14						
H1	1	305	5	17656	697.6	7	0.02196	0.015	0.2	0.04	0.1
	0.6	0.05	0.000115								
H2	305	5	1	0	0	20	5	5	5		
H3	4.50E-05	2.6	0	1	0	0.46	0.15	0.3	5.02	0.301	
H4	10	15	0.35	2.00E-03	14						
H1	1	306	6	1675	38.4	0	0.03401	0.015	0.2	0.04	0.1
	0.6	0.05	0.000115								
H2	306	6	1	0	0	20	5	5	5		
H3	4.50E-05	2.6	0	1	0	0.46	0.15	0.3	5.02	0.301	
H4	10	15	0.35	2.00E-03	14						
H1	1	307	7	534	6.4	0	0.02201	0.015	0.2	0.04	0.1
	0.6	0.05	0.000115								
H2	307	7	1	0	0	20	5	5	5		
H3	4.50E-05	2.6	0	1	0	0.46	0.15	0.3	5.02	0.301	
H4	10	15	0.35	2.00E-03	14						
H1	1	308	8	4437	153.6	13	0.0205	0.015	0.2	0.04	0.1
	0.6	0.05	0.000115								
H2	308	8	1	0	0	20	5	5	5		
H3	4.50E-05	2.6	0	1	0	0.46	0.15	0.3	5.02	0.301	
H4	10	15	0.35	2.00E-03	14						
H1	1	309	9	1156	19.2	8	0.02042	0.015	0.2	0.04	0.1
	0.6	0.05	0.000115								
H2	309	9	1	0	0	20	5	5	5		
H3	4.50E-05	2.6	0	1	0	0.46	0.15	0.3	5.02	0.301	
H4	10	15	0.35	2.00E-03	14						
H1	1	3010	10	6277	185.6	11	0.01531	0.015	0.2	0.04	0.1
	0.6	0.05	0.000115								
H2	3010	10	1	0	0	20	5	5	5		
H3	4.50E-05	2.6	0	1	0	0.46	0.15	0.3	5.02	0.301	
H4	10	15	0.35	2.00E-03	14						
H1	1	3011	11	2616	96	2	0.01486	0.015	0.2	0.04	0.1
	0.6	0.05	0.000115								
H2	3011	11	1	0	0	20	5	5	5		

H3	4.50E-05	2.6	0	1	0	0.46	0.15	0.3	5.02	0.301		
H4	10	15	0.35	2.00E-03	14							
H1	1	3012	12	893	57.6	13	0.01414	0.015	0.2	0.04	0.1	
	0.6	0.05	0.000115									
H2	3012	12	1	0	0	20	5	5	5			
H3	4.50E-05	2.6	0	1	0	0.46	0.15	0.3	5.02	0.301		
H4	10	15	0.35	2.00E-03	14							
H1	1	3013	13	3372	153.6	16	0.02914	0.015	0.2	0.04	0.1	
	0.6	0.05	0.000115									
H2	3013	13	1	0	0	20	5	5	5			
H3	4.50E-05	2.6	0	1	0	0.46	0.15	0.3	5.02	0.301		
H4	10	15	0.35	2.00E-03	14							
H1	1	3014	14	17269	966.4	10	0.01093	0.015	0.2	0.04	0.1	
	0.6	0.05	0.000115									
H2	3014	14	1	0	0	20	5	5	5			
H3	4.50E-05	2.6	0	1	0	0.46	0.15	0.3	5.02	0.301		
H4	10	15	0.35	2.00E-03	14							
H1	1	3015	15	5265	166.4	12	0.00982	0.015	0.2	0.04	0.1	
	0.6	0.05	0.000115									
H2	3015	15	1	0	0	20	5	5	5			
H3	4.50E-05	2.6	0	1	0	0.46	0.15	0.3	5.02	0.301		
H4	10	15	0.35	2.00E-03	14							
H1	1	3016	16	16097	582.4	23	0.03112	0.015	0.2	0.04	0.1	
	0.6	0.05	0.000115									
H2	3016	16	1	0	0	20	5	5	5			
H3	4.50E-05	2.6	0	1	0	0.46	0.15	0.3	5.02	0.301		
H4	10	15	0.35	2.00E-03	14							
H1	1	3017	17	2341	57.6	4	0.00939	0.015	0.2	0.04	0.1	
	0.6	0.05	0.000115									
H2	3017	17	1	0	0	20	5	5	5			
H3	4.50E-05	2.6	0	1	0	0.46	0.15	0.3	5.02	0.301		
H4	10	15	0.35	2.00E-03	14							
H1	1	3018	18	5663	352	17	0.01216	0.015	0.2	0.04	0.1	
	0.6	0.05	0.000115									
H2	3018	18	1	0	0	20	5	5	5			
H3	4.50E-05	2.6	0	1	0	0.46	0.15	0.3	5.02	0.301		
H4	10	15	0.35	2.00E-03	14							
H1	1	3019	19	1232	19.2	7	0.00899	0.015	0.2	0.04	0.1	
	0.6	0.05	0.000115									
H2	3019	19	1	0	0	20	5	5	5			
H3	4.50E-05	2.6	0	1	0	0.46	0.15	0.3	5.02	0.301		
H4	10	15	0.35	2.00E-03	14							
H1	1	3023	23	15407	864	10	0.02339	0.015	0.2	0.04	0.1	
	0.6	0.05	0.000115									
H2	3023	23	1	0	0	20	5	5	5			
H3	4.50E-05	2.6	0	1	0	0.46	0.15	0.3	5.02	0.301		
H4	10	15	0.35	2.00E-03	14							
H1	1	3024	24	496	6.4	21	0.02505	0.015	0.2	0.04	0.1	
	0.6	0.05	0.000115									
H2	3024	24	1	0	0	20	5	5	5			
H3	4.50E-05	2.6	0	1	0	0.46	0.15	0.3	5.02	0.301		
H4	10	15	0.35	2.00E-03	14							
H1	1	3025	25	13836	652.8	22	0.03632	0.015	0.2	0.04	0.1	
	0.6	0.05	0.000115									
H2	3025	25	1	0	0	20	5	5	5			
H3	4.50E-05	2.6	0	1	0	0.46	0.15	0.3	5.02	0.301		
H4	10	15	0.35	2.00E-03	14							
H1	1	30100	100	2225	96	25	0.02067	0.015	0.2	0.04	0.1	
	0.6	0.05	0.000115									
H2	30100	100	1	0	0	20	5	5	5			
H3	4.50E-05	2.6	0	1	0	0.46	0.15	0.3	5.02	0.301		
H4	10	15	0.35	2.00E-03	14							

H1	1	30101	101	11822	352	10	0.01903	0.015	0.2	0.04	0.1
	0.6	0.05	0.000115								
H2	30101	101	1	0	0	20	5	5	5		
H3	4.50E-05	2.6	0	1	0	0.46	0.15	0.3	5.02	0.301	
H4	10	15	0.35	2.00E-03	14						
H1	1	30102	102	3239	121.6	2	0.02241	0.015	0.2	0.04	0.1
	0.6	0.05	0.000115								
H2	30102	102	1	0	0	20	5	5	5		
H3	4.50E-05	2.6	0	1	0	0.46	0.15	0.3	5.02	0.301	
H4	10	15	0.35	2.00E-03	14						
H1	1	30103	103	12017	288	9	0.04189	0.015	0.2	0.04	0.1
	0.6	0.05	0.000115								
H2	30103	103	1	0	0	20	5	5	5		
H3	4.50E-05	2.6	0	1	0	0.46	0.15	0.3	5.02	0.301	
H4	10	15	0.35	2.00E-03	14						
H1	1	30104	104	3147	70.4	6	0.02882	0.015	0.2	0.04	0.1
	0.6	0.05	0.000115								
H2	30104	104	1	0	0	20	5	5	5		
H3	4.50E-05	2.6	0	1	0	0.46	0.15	0.3	5.02	0.301	
H4	10	15	0.35	2.00E-03	14						
H1	1	30105	105	2251	32	12	0.02441	0.015	0.2	0.04	0.1
	0.6	0.05	0.000115								
H2	30105	105	1	0	0	20	5	5	5		
H3	4.50E-05	2.6	0	1	0	0.46	0.15	0.3	5.02	0.301	
H4	10	15	0.35	2.00E-03	14						
H1	1	30106	106	3789	108.8	7	0.02158	0.015	0.2	0.04	0.1
	0.6	0.05	0.000115								
H2	30106	106	1	0	0	20	5	5	5		
H3	4.50E-05	2.6	0	1	0	0.46	0.15	0.3	5.02	0.301	
H4	10	15	0.35	2.00E-03	14						
H1	1	30107	107	20794	409.6	19	0.03035	0.015	0.2	0.04	0.1
	0.6	0.05	0.000115								
H2	30107	107	1	0	0	20	5	5	5		
H3	4.50E-05	2.6	0	1	0	0.46	0.15	0.3	5.02	0.301	
H4	10	15	0.35	2.00E-03	14						
H1	1	30108	108	12080	352	8	0.03512	0.015	0.2	0.04	0.1
	0.6	0.05	0.000115								
H2	30108	108	1	0	0	20	5	5	5		
H3	4.50E-05	2.6	0	1	0	0.46	0.15	0.3	5.02	0.301	
H4	10	15	0.35	2.00E-03	14						
H1	1	30109	109	733	70.4	12	0.02117	0.015	0.2	0.04	0.1
	0.6	0.05	0.000115								
H2	30109	109	1	0	0	20	5	5	5		
H3	4.50E-05	2.6	0	1	0	0.46	0.15	0.3	5.02	0.301	
H4	10	15	0.35	2.00E-03	14						
H1	1	301010	1010	10089	505.6	6	0.02167	0.015	0.2	0.04	0.1
	0.6	0.05	0.000115								
H2	301010	1010	1	0	0	20	5	5	5		
H3	4.50E-05	2.6	0	1	0	0.46	0.15	0.3	5.02	0.301	
H4	10	15	0.35	2.00E-03	14						
H1	1	301011	1011	1519	76.8	9	0.02845	0.015	0.2	0.04	0.1
	0.6	0.05	0.000115								
H2	301011	1011	1	0	0	20	5	5	5		
H3	4.50E-05	2.6	0	1	0	0.46	0.15	0.3	5.02	0.301	
H4	10	15	0.35	2.00E-03	14						
H1	1	301012	1012	9293	480	18	0.016	0.015	0.2	0.04	0.1
	0.6	0.05	0.000115								
H2	301012	1012	1	0	0	20	5	5	5		
H3	4.50E-05	2.6	0	1	0	0.46	0.15	0.3	5.02	0.301	
H4	10	15	0.35	2.00E-03	14						
H1	1	301013	1013	655	6.4	4	0.02048	0.015	0.2	0.04	0.1
	0.6	0.05	0.000115								



H2	301013	1013	1	0	0	20	5	5	5		
H3	4.50E-05	2.6	0	1	0	0.46	0.15	0.3	5.02	0.301	
H4	10	15	0.35	2.00E-03	14						
H1	1	301014	1014	863	6.4	11	0.01787	0.015	0.2	0.04	0.1
	0.6	0.05	0.000115								
H2	301014	1014	1	0	0	20	5	5	5		
H3	4.50E-05	2.6	0	1	0	0.46	0.15	0.3	5.02	0.301	
H4	10	15	0.35	2.00E-03	14						
H1	1	301015	1015	769	6.4	0	0.015	0.015	0.2	0.04	0.1
	0.6	0.05	0.000115								
H2	301015	1015	1	0	0	20	5	5	5		
H3	4.50E-05	2.6	0	1	0	0.46	0.15	0.3	5.02	0.301	
H4	10	15	0.35	2.00E-03	14						
H1	1	301016	1016	16389	262.4	8	0.03551	0.015	0.2	0.04	0.1
	0.6	0.05	0.000115								
H2	301016	1016	1	0	0	20	5	5	5		
H3	4.50E-05	2.6	0	1	0	0.46	0.15	0.3	5.02	0.301	
H4	10	15	0.35	2.00E-03	14						
H1	1	301017	1017	1479	19.2	0	0.01385	0.015	0.2	0.04	0.1
	0.6	0.05	0.000115								
H2	301017	1017	1	0	0	20	5	5	5		
H3	4.50E-05	2.6	0	1	0	0.46	0.15	0.3	5.02	0.301	
H4	10	15	0.35	2.00E-03	14						
H1	1	301018	1018	543	12.8	4	0.01463	0.015	0.2	0.04	0.1
	0.6	0.05	0.000115								
H2	301018	1018	1	0	0	20	5	5	5		
H3	4.50E-05	2.6	0	1	0	0.46	0.15	0.3	5.02	0.301	
H4	10	15	0.35	2.00E-03	14						
H1	1	301019	1019	12650	358.4	28	0.02904	0.015	0.2	0.04	0.1
	0.6	0.05	0.000115								
H2	301019	1019	1	0	0	20	5	5	5		
H3	4.50E-05	2.6	0	1	0	0.46	0.15	0.3	5.02	0.301	
H4	10	15	0.35	2.00E-03	14						
H1	1	301020	1020	17477	550.4	12	0.03418	0.015	0.2	0.04	0.1
	0.6	0.05	0.000115								
H2	301020	1020	1	0	0	20	5	5	5		
H3	4.50E-05	2.6	0	1	0	0.46	0.15	0.3	5.02	0.301	
H4	10	15	0.35	2.00E-03	14						
H1	1	301021	1021	17503	460.8	36	0.01536	0.015	0.2	0.04	0.1
	0.6	0.05	0.000115								
H2	301021	1021	1	0	0	20	5	5	5		
H3	4.50E-05	2.6	0	1	0	0.46	0.15	0.3	5.02	0.301	
H4	10	15	0.35	2.00E-03	14						
H1	1	301022	1022	17270	761.6	22	0.01493	0.015	0.2	0.04	0.1
	0.6	0.05	0.000115								
H2	301022	1022	1	0	0	20	5	5	5		
H3	4.50E-05	2.6	0	1	0	0.46	0.15	0.3	5.02	0.301	
H4	10	15	0.35	2.00E-03	14						
H1	1	301023	1023	1294	25.6	10	0.02667	0.015	0.2	0.04	0.1
	0.6	0.05	0.000115								
H2	301023	1023	1	0	0	20	5	5	5		
H3	4.50E-05	2.6	0	1	0	0.46	0.15	0.3	5.02	0.301	
H4	10	15	0.35	2.00E-03	14						
H1	1	301024	1024	1794	76.8	8	0.01058	0.015	0.2	0.04	0.1
	0.6	0.05	0.000115								
H2	301024	1024	1	0	0	20	5	5	5		
H3	4.50E-05	2.6	0	1	0	0.46	0.15	0.3	5.02	0.301	
H4	10	15	0.35	2.00E-03	14						
H1	1	301025	1025	900	89.6	20	0.02904	0.015	0.2	0.04	0.1
	0.6	0.05	0.000115								
H2	301025	1025	1	0	0	20	5	5	5		

H3	4.50E-05	2.6	0	1	0	0.46	0.15	0.3	5.02	0.301		
H4	10	15	0.35	2.00E-03	14							
H1	1	301026	1026	716	19.2	8	0.00968	0.015	0.2	0.04	0.1	
	0.6	0.05	0.000115									
H2	301026	1026	1	0	0	20	5	5	5			
H3	4.50E-05	2.6	0	1	0	0.46	0.15	0.3	5.02	0.301		
H4	10	15	0.35	2.00E-03	14							
H1	1	301027	1027	12475	345.6	28	0.03261	0.015	0.2	0.04	0.1	
	0.6	0.05	0.000115									
H2	301027	1027	1	0	0	20	5	5	5			
H3	4.50E-05	2.6	0	1	0	0.46	0.15	0.3	5.02	0.301		
H4	10	15	0.35	2.00E-03	14							
H1	1	301028	1028	5706	352	18	0.01348	0.015	0.2	0.04	0.1	
	0.6	0.05	0.000115									
H2	301028	1028	1	0	0	20	5	5	5			
H3	4.50E-05	2.6	0	1	0	0.46	0.15	0.3	5.02	0.301		
H4	10	15	0.35	2.00E-03	14							
H1	1	301029	1029	1355	6.4	11	0.01182	0.015	0.2	0.04	0.1	
	0.6	0.05	0.000115									
H2	301029	1029	1	0	0	20	5	5	5			
H3	4.50E-05	2.6	0	1	0	0.46	0.15	0.3	5.02	0.301		
H4	10	15	0.35	2.00E-03	14							
H1	1	301030	1030	1241	70.4	11	0.00916	0.015	0.2	0.04	0.1	
	0.6	0.05	0.000115									
H2	301030	1030	1	0	0	20	5	5	5			
H3	4.50E-05	2.6	0	1	0	0.46	0.15	0.3	5.02	0.301		
H4	10	15	0.35	2.00E-03	14							
H1	1	301031	1031	11987	230.4	42	0.02675	0.015	0.2	0.04	0.1	
	0.6	0.05	0.000115									
H2	301031	1031	1	0	0	20	5	5	5			
H3	4.50E-05	2.6	0	1	0	0.46	0.15	0.3	5.02	0.301		
H4	10	15	0.35	2.00E-03	14							
H1	1	301032	1032	105	6.4	14	0.00897	0.015	0.2	0.04	0.1	
	0.6	0.05	0.000115									
H2	301032	1032	1	0	0	20	5	5	5			
H3	4.50E-05	2.6	0	1	0	0.46	0.15	0.3	5.02	0.301		
H4	10	15	0.35	2.00E-03	14							

JJ 5

J1 2 5 0 0 1 2 1 0 0 0 0

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Land Use Name	METHOD	JACGUT	DDLIM	DDPOW	DDFACT	CLFREQ	ASSWP	DSLCL
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J2 'resid'		0	0	0	0	0	0	0
J2 'comme'		0	0	0	0	0	0	0
J2 'agric'		0	0	0	0	0	0	0
J2 'fores'		0	0	0	0	0	0	0
J2 'other'		0	0	0	0	0	0	0

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* Nutrient	Units	NDIM	KALC	KWASH	KACGUT	LINKUP	QFACT1	QFACT2	QFACT3	QFACT4
* QFACT5	WASHPO	RCOEF	CBFACT	COINCRN	REF					

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Land Use resid

J3 'Nitrates'	'mg/L'	0	4	1	0	0	0	0	0	0
		0	1	20.8288	0	0	0			

\* Land Use comme

J3 'Nitrates'	'mg/L'	0	4	1	0	0	0	0	0	0
		0	1	16.1876	0	0	0			

\* Land Use agric

J3 'Nitrates'	'mg/L'	0	4	1	0	0	0	0	0	0
		0	1	21.225 0	0	0				

\* Land Use fores

J3	'Nitrates'	'mg/L'	0	4	1	0	0	0	0	0	0
	0	1	15.3669		0	0	0				
*	Land Use other										
J3	'Nitrates'	'mg/L'	0	4	1	0	0	0	0	0	0
	0	1	1839.5	0	0						
*	Land Use resid										
J3	'TP'	'mg/L'	0	4	1	0	0	0	0	0	0
	0	1	10.8389		0	0	0				
*	Land Use comme										
J3	'TP'	'mg/L'	0	4	1	0	0	0	0	0	0
	0	1	5.6883	0	0						
*	Land Use agric										
J3	'TP'	'mg/L'	0	4	1	0	0	0	0	0	0
	0	1	33.96	0	0						
*	Land Use fores										
J3	'TP'	'mg/L'	0	4	1	0	0	0	0	0	0
	0	1	3.4243	0	0						
*	Land Use other										
J3	'TP'	'mg/L'	0	4	1	0	0	0	0	0	0
	0	1	7.075	0	0						

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Subcat							
L.U.	BASINS	GQLEN	PSHED1	PSHED2	PSHED3	PSHED4	
L1	301	1	0	0	/		
L2	0.327273	0	0.2	0.463636	0.00909091		
L1	302	1	0	0	/		
L2	0.2	0	0	0.8	0		
L1	303	1	0	0	/		
L2	0.742424	0	0	0.242424	0.0151515		
L1	304	1	0	0	/		
L2	0.309091	0	0.318182	0.3	0.0727273		
L1	305	1	0	0	/		
L2	0.165138	0.00917431	0.376147	0.40367	0.0458716		
L1	306	1	0	0	/		
L2	0	0	0.166667	0.833333	0		
L1	307	1	0	0	/		
L2	0	0	0	1	0		
L1	308	1	0	0	/		
L2	0.478261	0	0.0434783	0.391304	0.0869565		
L1	309	1	0	0	/		
L2	0.5	0	0	0.5	0		
L1	3010	1	0	0	/		
L2	0.0689655	0	0.0689655	0.0344828	0.827586		
L1	3011	1	0	0	/		
L2	0.133333	0	0	0.8	0.0666667		
L1	3012	1	0	0	/		
L2	0.444444	0	0	0.555556	0		
L1	3013	1	0	0	/		
L2	0.666667	0	0	0.333333	0		
L1	3014	1	0	0	/		
L2	0.529801	0	0.0397351	0.403974	0.0264901		
L1	3015	1	0	0	/		
L2	0.296296	0	0	0.259259	0.444444		
L1	3016	1	0	0	/		
L2	0.888889	0	0.0111111	0.0555556	0.0444444		
L1	3017	1	0	0	/		
L2	0.222222	0	0	0.444444	0.333333		
L1	3018	1	0	0	/		
L2	0.407407	0	0	0.0740741	0.518519		
L1	3019	1	0	0	/		
L2	0.25	0	0	0.75	0		

L1	3023	1	0	0	/
L2	0.281481	0.0148148	0.118519	0.362963	0.222222
L1	3024	1	0	0	/
L2	1	0/			
L1	3025	1	0	0	/
L2	0.852941	0.00980392	0	0.0882353	0.0490196
L1	30100	1	0	0	/
L2	0	0	0	0.4	0.6
L1	30101	1	0	0	/
L2	0.444444	0	0.185185	0.259259	0.111111
L1	30102	1	0	0	/
L2	0.368421	0	0	0.631579	0
L1	30103	1	0	0	/
L2	0.555556	0	0	0.422222	0.0222222
L1	30104	1	0	0	/
L2	0	0	0.416667	0.583333	0
L1	30105	1	0	0	/
L2	0.25	0	0.25	0	0.5
L1	30106	1	0	0	/
L2	0.111111	0	0.0555556	0.388889	0.444444
L1	30107	1	0	0	/
L2	0.0625	0	0.03125	0.703125	0.203125
L1	30108	1	0	0	/
L2	0.2	0	0.236364	0.436364	0.127273
L1	30109	1	0	0	/
L2	0.5	0	0.166667	0.333333	0
L1	301010	1	0	0	/
L2	0.253165	0	0.177215	0.405063	0.164557
L1	301011	1	0	0	/
L2	0.666667	0	0	0.333333	0
L1	301012	1	0	0	/
L2	0.22973	0	0.135135	0.162162	0.472973
L1	301013	1	0	0	/
L2	0	0	0	1	0
L1	301014	1	0	0	/
L2	0	0	0	0	1
L1	301015	1	0	0	/
L2	0	0	0	1	0
L1	301016	1	0	0	/
L2	0.55	0	0.125	0.325	0
L1	301017	1	0	0	/
L2	0	0	0	1	0
L1	301018	1	0	0	/
L2	0	0	0	0.5	0.5
L1	301019	1	0	0	/
L2	0.618182	0.0363636	0	0.109091	0.236364
L1	301020	1	0	0	/
L2	0.837209	0	0.0348837	0.127907	0
L1	301021	1	0	0	/
L2	0.763889	0.0277778	0	0.0277778	0.180556
L1	301022	1	0	0	/
L2	0.596639	0.0420168	0.0168067	0.092437	0.252101
L1	301023	1	0	0	/
L2	0.5	0	0	0.5	0
L1	301024	1	0	0	/
L2	0.333333	0	0	0.5	0.166667
L1	301025	1	0	0	/
L2	0.857143	0	0	0.0714286	0.0714286
L1	301026	1	0	0	/
L2	0	0	0	0.333333	0.666667
L1	301027	1	0	0	/
L2	0.773585	0	0	0.0943396	0.132075

L1	301028	1	0	0	/
L2	0.696429	0	0	0.232143	0.0714286
L1	301029	1	0	0	/
L2	0	0	0	1	0
L1	301030	1	0	0	/
L2	0.416667	0	0	0.333333	0.25
L1	301031	1	0	0	/
L2	0.742857	0.0571429	0	0	0.2
L1	301032	1	0	0	/
L2	0	0	0	1	0

\*

\*\*\*\*\*

\* CHANNEL AND INLET PRINT CONTROL (M)

\*\*\*\*\*

\* NPRNT INTERV

\* Number Print

\* Printed Control

\* -----

M1 0 1

\*

\* NOTE: If NPRNT = 0 on line M1 skip groups M2 and M3.

\* NDET STARTP1 STOPPR1

\* # Print Start Stop

\* Periods Date Date

\* -----

\*M2 1 0 0

\*

\* IPRNT(1)

\* Print

\* Name

\* -----

\*M3

\*\*\*\*\*

\* PROGRAM END

\*\*\*\*\*

\$ENDPROGRAM

## A.2. SWMM model transport input file for continuous constant land use model

```

SW 1 8 9
MM 7 1 2 3 10 11 12 13
@ 8 'c:\thesis\northwestbranch\runoff\n01979.dnt'
@ 9 'c:\thesis\northwestbranch\runoff\n01979tr.dnt'
$TRANSPORT
*****
* Title Lines
*****
A1 'Run of Runoff block'
A1 'NorthwestBranch watershed - Transport block'
* ISLOPE ITRAP IFLIP INFLEW
*****
B0 0 0 0 0
*****
NDT NINPUT NNYN NNPE NOUTS NPRINT NPOLL NITER IDATEZ METRIC INTPRT
*****
B1 1462 0 55 55 55 0 2 4 19790630 0 0
*****
DT EPSIL DWDAYS TZERO GNU TRIBA
*****
B2 86400 0.0001 4 0 0.00001 21.2
*****
NCNTRL NINFIL NFILTH NDESN
*****
B3 0 0 0 0
*****
FLOW ROUTING FOR NEW SHAPES
* NKLASS KPRINT
*****
C1 0 0
*****
* SEWER ELEMENT DATA
*
*****
NOE NUE(1) NUE(2) NUE(3) NTYPE DIST GEOM1 SLOPE ROUGH GEOM2 BARREL
GEOM3 KGEOM
*****
E1 1 0 0 0 19 0 0 0 0 0 0
0
E1 202 108 1010 0 15 1266 1.72165 3.075 0.025 17.4509 1
1.53846
E1 2 202 0 0 19 0 0 0 0 0 0
0
E1 203 102 1011 0 15 4787 2.2225 1.81 0.025 24.2325 1
1.53846
E1 3 203 0 0 19 0 0 0 0 0 0
0
E1 4 0 0 0 19 0 0 0 0 0 0
0
E1 5 0 0 0 19 0 0 0 0 0 0
0
E1 206 101 103 0 15 949 1.59583 3.401 0.025 15.8286 1
1.53846
E1 6 206 0 0 19 0 0 0 0 0 0
0
E1 207 106 107 0 15 300 2.1801 2.201 0.025 23.6398 1
1.53846
E1 7 207 0 0 19 0 0 0 0 0 0
0

```

E1	208	23	0	0	15	2283	1.78768	2.05	0.025	18.3162	1
	1.53846										
E1	8	208	0	0	19	0	0	0	0	0	0
	0										
E1	209	109	1013	0	15	724	2.45968	2.042	0.025	27.607	1
	1.53846										
E1	9	209	0	0	19	0	0	0	0	0	0
	0										
E1	2010	105	1014	0	15	3390	2.44993	1.531	0.025	27.4664	1
	1.53846										
E1	10	2010	0	0	19	0	0	0	0	0	0
	0										
E1	2011	1012	1016	0	15	1873	2.62503	1.486	0.025	30.0157	1
	1.53846										
E1	11	2011	0	0	19	0	0	0	0	0	0
	0										
E1	2012	1015	1018	0	15	824	3.09495	1.414	0.025	37.0938	1
	1.53846										
E1	12	2012	0	0	19	0	0	0	0	0	0
	0										
E1	2013	25	0	0	15	2555	1.67496	2.914	0.025	16.8448	1
	1.53846										
E1	13	2013	0	0	19	0	0	0	0	0	0
	0										
E1	2014	1017	1020	0	15	9111	3.26657	1.093	0.025	39.7591	1
	1.53846										
E1	14	2014	0	0	19	0	0	0	0	0	0
	0										
E1	2015	1023	1024	0	15	2924	3.37418	0.982	0.025	41.4509	1
	1.53846										
E1	15	2015	0	0	19	0	0	0	0	0	0
	0										
E1	16	0	0	0	19	0	0	0	0	0	0
	0										
E1	2017	1025	1026	0	15	1249	3.44421	0.939	0.025	42.5603	1
	1.53846										
E1	17	2017	0	0	19	0	0	0	0	0	0
	0										
E1	2018	1028	1031	0	15	3745	2.38773	1.216	0.025	26.5731	1
	1.53846										
E1	18	2018	0	0	19	0	0	0	0	0	0
	0										
E1	2019	1029	1030	0	15	824	3.69113	0.899	0.025	46.5227	1
	1.53846										
E1	19	2019	0	0	19	0	0	0	0	0	0
	0										
E1	23	0	0	0	19	0	0	0	0	0	0
	0										
E1	2024	1019	1021	0	15	300	1.68603	2.505	0.025	16.9881	1
	1.53846										
E1	24	2024	0	0	19	0	0	0	0	0	0
	0										
E1	25	0	0	0	19	0	0	0	0	0	0
	0										
E1	20100	5	0	0	15	1566	1.66747	2.067	0.025	16.748	1
	1.53846										
E1	100	20100	0	0	19	0	0	0	0	0	0
	0										
E1	101	0	0	0	19	0	0	0	0	0	0
	0										
E1	20102	1	0	0	15	2024	1.68236	2.241	0.025	16.9405	1
	1.53846										

E1	102	20102	0	0	19	0	0	0	0	0	0	0
	0											
E1	103	0	0	0	19	0	0	0	0	0	0	0
	0											
E1	20104	6	0	0	15	2072	1.64056	2.882	0.025	16.4013	1	
	1.53846											
E1	104	20104	0	0	19	0	0	0	0	0	0	0
	0											
E1	20105	4	0	0	15	1183	1.62867	2.441	0.025	16.2487	1	
	1.53846											
E1	105	20105	0	0	19	0	0	0	0	0	0	0
	0											
E1	20106	100	104	0	15	2490	2.04717	2.158	0.025	21.8029	1	
	1.53846											
E1	106	20106	0	0	19	0	0	0	0	0	0	0
	0											
E1	107	0	0	0	19	0	0	0	0	0	0	0
	0											
E1	108	0	0	0	19	0	0	0	0	0	0	0
	0											
E1	20109	7	0	0	15	683	2.20064	2.117	0.025	23.9266	1	
	1.53846											
E1	109	20109	0	0	19	0	0	0	0	0	0	0
	0											
E1	1010	0	0	0	19	0	0	0	0	0	0	0
	0											
E1	201011	2	0	0	15	1224	1.76203	2.845	0.025	17.9789	1	
	1.53846											
E1	1011	201011	0	0	19	0	0	0	0	0	0	0
	0											
E1	201012	9	0	0	15	5711	2.55716	1.6	0.025	29.0216	1	
	1.53846											
E1	1012	201012	0	0	19	0	0	0	0	0	0	0
	0											
E1	201013	8	0	0	15	400	1.79083	2.048	0.025	18.3576	1	
	1.53846											
E1	1013	201013	0	0	19	0	0	0	0	0	0	0
	0											
E1	201014	3	0	0	15	441	2.22429	1.787	0.025	24.2577	1	
	1.53846											
E1	1014	201014	0	0	19	0	0	0	0	0	0	0
	0											
E1	201015	10	0	0	15	442	2.45133	1.5	0.025	27.4866	1	
	1.53846											
E1	1015	201015	0	0	19	0	0	0	0	0	0	0
	0											
E1	1016	0	0	0	19	0	0	0	0	0	0	0
	0											
E1	201017	12	0	0	15	883	3.09649	1.385	0.025	37.1175	1	
	1.53846											
E1	1017	201017	0	0	19	0	0	0	0	0	0	0
	0											
E1	201018	11	0	0	15	383	2.62737	1.463	0.025	30.0501	1	
	1.53846											
E1	1018	201018	0	0	19	0	0	0	0	0	0	0
	0											
E1	1019	0	0	0	19	0	0	0	0	0	0	0
	0											
E1	1020	0	0	0	19	0	0	0	0	0	0	0
	0											
E1	1021	0	0	0	19	0	0	0	0	0	0	0
	0											



E1	201022	24	0	0	15	9018	2.02234	1.493	0.025	21.4636	1
	1.53846										
E1	1022	201022	0	0	19	0	0	0	0	0	0
	0										
E1	201023	13	0	0	15	1107	1.68968	2.667	0.025	17.0354	1
	1.53846										
E1	1023	201023	0	0	19	0	0	0	0	0	0
	0										
E1	201024	14	0	0	15	1442	3.27457	1.058	0.025	39.8842	1
	1.53846										
E1	1024	201024	0	0	19	0	0	0	0	0	0
	0										
E1	201025	16	0	0	15	866	1.58734	2.904	0.025	15.7204	1
	1.53846										
E1	1025	201025	0	0	19	0	0	0	0	0	0
	0										
E1	201026	15	0	0	15	541	3.37664	0.968	0.025	41.4897	1
	1.53846										
E1	1026	201026	0	0	19	0	0	0	0	0	0
	0										
E1	1027	0	0	0	19	0	0	0	0	0	0
	0										
E1	201028	1022	1027	0	15	3639	2.2403	1.348	0.025	24.4824	1
	1.53846										
E1	1028	201028	0	0	19	0	0	0	0	0	0
	0										
E1	201029	18	0	0	15	766	2.39072	1.182	0.025	26.6159	1
	1.53846										
E1	1029	201029	0	0	19	0	0	0	0	0	0
	0										
E1	201030	17	0	0	15	1007	3.45119	0.916	0.025	42.6712	1
	1.53846										
E1	1030	201030	0	0	19	0	0	0	0	0	0
	0										
E1	1031	0	0	0	19	0	0	0	0	0	0
	0										
E1	201032	19	0	0	15	100	3.69161	0.897	0.025	46.5306	1
	1.53846										
E1	1032	201032	0	0	19	0	0	0	0	0	0
	0										

\*\*\*\*\*

Water Quality

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KPOL PNAME PUNIT NDIM DECAF SPG PSIZE2 PGR(2) PSIZE3 PGR(3)  
PSIZE4 PGR(4) PSIZE5 PGR(5) PSDWF

\*\*\*\*\*

F1	1	'Nitrates'	'MG/L'	0	0.1654	1	0	0	0	0	0
	0	0	0	0	0						
F1	2	'TP'	'MG/L'	0	0.1311	1	0	0	0	0	0
	0	0	0	0	0						

\*\*\*\*\*

\* Print Control

\*\*\*\*\*

\* JN(X)...

\*\*\*\*\*

\*H1 1 2 3 4 5 6 7 8 9 10 11 12 13 14 15 16 17 18 19 23 24 25 100 101 102 103 104 105 106 107 108 109 1010 1011  
1012 1013 1014 1015 1016 1017 1018 1019 1020 1021 1022 1023 1024 1025 1026 1027 1028 1029 1030 1031 1032

\*\*\*\*\*

\* NYN(X)...

\*\*\*\*\*

J1 1 2 3 4 5 6 7 8 9 10 11 12 13 14 15 16 17 18 19 23 24 25 100 101 102 103 104 105 106 107 108 109 1010 1011  
1012 1013 1014 1015 1016 1017 1018 1019 1020 1021 1022 1023 1024 1025 1026 1027 1028 1029 1030 1031 1032



### A.3. SWMM model combined input file for continuous constant land use model

```
SW 1 8 9
MM 7 1 2 3 11 12 13 14
@ 8 'c:\thesis\northwestbranch\runoff\n01979tr.dnt'
@ 9 'c:\thesis\northwestbranch\runoff\n01979trco.txt '
$ANUM
$COMBIN
A1 4
*****
* TITLES
*****
*B1 ' - Combine Block'
B1 "
*****
* NODEOT NPOLL
*****
B2 0 0
*****
* NUMX NUMR
*****
C1 0 0
$ENDPROGRAM
```

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