DISSERTATION

DESIGN AND IMPLEMENTATION OF HYDROLOGIC UNIT WATERSHEDS FOR RAINFALL-RUNOFF MODELING IN URBAN AREAS

Submitted by
Iván Rivas Acosta
Department of Civil and Environmental Engineering

In partial fulfillment of the requirements
For the Degree of Doctor of Philosophy
Colorado State University
Fort Collins, Colorado
Fall 2009

COLORADO STATE UNIVERSITY

November 6, 2009

WE HEREBY RECOMMEND THAT THE DISSERTATION PREPARED UNDER OUR SUPERVISION BY IVAN RIVAS ACOSTA ENTITLED "DESIGN AND IMPLEMENTATION OF HYDROLOGIC UNIT WATERSHEDS FOR RAINFALL-RUNOFF MODELING IN URBAN AREAS" BE ACCEPTED AS FULFILLING IN PART REQUIREMENTS FOR THE DEGREE OF DOCTOR OF PHILOSOPHY.

Committee on Graduate work

Darrell G. Fontane

Neil S. Grigg

Evan Vlachos

Advisor **Larry A. Roesner**

Department Head Luis A. Garcia

ABSTRACT OF DISSERTATION

DESIGN AND IMPLEMENTATION OF HYDROLOGIC UNIT WATERSHEDS FOR RAINFALL-RUNOFF MODELING IN URBAN AREAS

The calibration of complex hydrology and hydraulics of rainfall-runoff models represents one of the most challenging problems in water resources engineering. Unlike undeveloped watersheds, but specifically urban basins with surface drainage. From the available models, SWMM (Storm Water Management Model) was used as the modeling engine since it was developed for urban watersheds.

Calibration procedure used a Multi-Criteria Decision Analysis (MCDA) approach that minimized the RMSE (Root Mean Square Error) between the flow duration curves of the modeled and the observed runoff. The flow duration curve was divided in High and Low Flows using the 1-Yr storm to split the curve, since there is a change in flow regime at this point. Pareto optimal front surfaces were obtained.

Two case studies in North Carolina (Pigeon and SW Prong basins) were used to illustrate a proposed methodology for calibration. The methodology simplified the drainage network and irregular sub-catchments shapes were converted to regular shapes using a Kinematic Wave (KW) cascading plane approach.

The KW cascading plane approach showed to be effective to convert irregular sub-basins shapes to rectangular features. A discretization analysis was performed where a set of hydrologic experiments using different levels of discretization were used and a threshold discretization value in urban hydrology was investigated. Needed GIS data was extracted through a toolbox. MCDA methodology and numerical simulations showed that Horton's decay coefficient (K, 1/h) and drying time (Tw, days) needed to have different values for the High and Low Flow portions of the flow duration curve to improve performance. Longer drying times were required to improve estimation of High Flows than Low Flows because the soils would take more time to recover their initial infiltration capacity.

The Representative Element Area (REA) concept was explored in SWMM and it was found that sub-catchment sizes of 3% of the total basin size were appropriate. This magnitude represents the suggested level of discretization in urban watersheds since the improvement in performance became asymptotic either to 1.00 (Pearson's Moment Correlation Coefficient - PMCC, Nash-Sutcliff Coefficient - NSC and Index of Agreement - IOA) or to zero (RMSE) and therefore, it is not significant to improve the spatial resolution. Coarser resolution levels underestimated peak flow rates and total runoff volumes. Research results are summarized in a proposed protocol to discretisize urban watersheds.

Iván Rivas Acosta
Department of Civil and Environmental Engineering
Colorado State University
Fort Collins, CO 80523
Fall 2009

ACKWNOLEDGMENTS

I am deeply grateful for the support and guidance provided by Dr. Larry A. Roesner. Many thanks to my graduate committee members, Professors Neil S. Grigg, Darrell G. Fontane and Evan Vlachos. Additional support, including computer equipment, was provided by the Harold H. Short Foundation, the Department of Civil Engineering, and Engineering Network Services at Colorado State University.

I am very thankful to the National Council of Science and Technology (CONACyT-México) for their economic support during graduate studies. Thanks to the Mexican Institute of Water Technology (IMTA) for their assistance during the last few months, especially to Dr. Francisco J. Aparicio Mijares and MSc. Alberto Güitron de los Reyes.

I thank my parents, Amalia and Abel, for their unconditional love and assistance throughout my life. Special thanks to personnel of Bulloch Brothers Engineering, Inc. (Mesquite, NV) and PBS&J (Beltsville, MD), for the professional experience gained as a water resources engineer in the United States.

Many thanks to all members and coaches of the CSU Triathlon Team for sharing their time in the roads through the amazingly beautiful Rocky Mountains, athletic track, streets, swimming pool and unforgettable national championship races. Finishing a doctoral program has been in many aspects similar to completing a triathlon race, a real test of tenacity, mental and physical endurance. Thank you to all my ex-roommates in Fort Collins from four continents.

TABLE OF CONTENTS

LIST OF TABLES

LIST OF FIGURES

1.0 INTRODUCTION

1.1 Background for this study

The construction of rainfall-runoff models represents one of the most complex and challenging problems in water resources engineering. This is particularly true for urban watersheds in which, the entire watershed is discretized in smaller sub-basins and additional systems need to be modeled such as the drainage network. Rainfall-runoff models have, of course, been under a continuous state of evolution (Hundecha, 2004).

The simulation of rainfall-runoff relationships has been a prime focus of hydrological research for several decades and has resulted in an abundance of models having been proposed (Kokkonen and Jakeman, 2001), but accurate and reliable modeling of stormwater runoff and associated phenomena has been in the past and continues today to be a challenge (Urbonas, 2007).

Basin discretization is the first process to be done when a rainfall-runoff model is applied. However, there is no a clear procedure to perform this task. Neither is it fully obvious to what extent these models can provide reliable simulations over a wide range of spatial scales (Moretti and Montanari, 2008). Many hydrologic models are available, varying in nature, complexity, and purpose (Shoemaker et al., 1997).

Once the model has been developed, traditional methods of estimating model parameters are based on calibrating the model against observed catchment responses, such as runoff from the watershed.

The ongoing expansion of urbanized areas has placed increasing emphasis on related water management problems such as flooding and pollution control. Booth (1991) identified that hydrologic changes including increased impervious area, soil compaction, and increased drainage efficiency generally lead to increased direct runoff, decreased groundwater recharge, and increased flooding, among other problems. To understand these changes, hydrologic models, especially simple rainfall-runoff models, are widely used in understanding and quantifying the impacts of land-use changes, and to provide information that can be used in land-use decision making.

Population densities and the size of these areas have led to considering the detailed behavior of water drainage systems at various scales (Rodriguez et al., 2003). In effect, continued land development and land-use changes within cities and at the urban fringe present considerable challenges for environmental management.

While extensive distributed hydrologic modeling research has been done, most of it has been done in non-urban watersheds. It seems that urban hydrology models have been left out in this topic and there is little guidance in how to sub-divide an urban watershed for modeling purposes.

Despite all the advances in understanding the watershed response under a rainfall event, the relationship between rainfall and runoff is very complex and still not completely understood; this is especially in true in urban environments where drainage does not follow natural paths given the artificial cover imposed by human settlements.

1.2 Objectives of research

The main objective of this research was to develop a hydrologic procedure to discretisize highly urbanized watersheds. The proposed methodology was tested in two urban basins in North Carolina. Modeling experiments showed what level of spatial scale is needed. Unlike most of the calibration methodologies, which are based upon a single or multi-storm, a unique feature of this research is to calibrate the model using a Multi-Criteria Decision Analysis (MCDA) applied to the flow-duration curve. This idea was originated given the fact that traditional calibration does not achieve a successful flow prediction for the entire diversity of storm sizes and water resources applications might have a quite different return period. Indeed, water resources applications cover a wide spectrum of return periods, ranging from floodplain delineation (500 or 100 Yr), hydraulic design of structures (100 or 50 Yr), bankfull determination (2.5 Yr) and urban stream ecology (0.5 Yr), to mention just a few examples.

A review of a number of distributed hydrologic models already developed revealed one of the most important issues is what level of watershed discretization is required to adequately represent runoff from the watershed. The level would vary depending on the particular algorithm used to translate rainfall into runoff. Several models were examined preliminarily and for this study USEPA-SWMM5 (EPA, 2008) was selected for investigation because it is perhaps the most widely urban drainage simulation model in the world.

1.3 Research approach and HUW

In order to create a hydrologic distribution model that addresses the two crucial issues of urban hydrology, discretization and flow duration curve, it is necessary in

SWMM5 to 1) subdivide the watershed into smaller units which we will define as the Hydrologic Unit Watershed (HUW), and 2) use a calibration procedure that is based on the total flow-duration curve rather than individual storms.

A HUW is a sub-unit in the entire watershed in which the routing channel length is estimated through optimization as a function of the sub-watershed size; then, the optimized routing channel is used as a parameter for the conversion from irregular to regular sub-catchments using a Kinematic Wave (KW) procedure. The latter method features two important advantages: the only parameters needed in each sub-catchment are the sub-catchment area and its corresponding routing channel length; and the KW conversion sets any irregular sub-catchment form into a rectangular shape to use directly in SWMM. The proposed procedure simplifies the drainage network. The HUW concept will be explained in detail in Chapter 3.

In this dissertation, the effects of subdivision level (sub-basin scale) on runoff simulation were investigated as well. An attempt was made to find a proper sub-basin scale for applying the USEPA Storm Water Management Model (SWMM) to large urban watersheds. SWMM (Huber and Dickinson, 1988) is a dynamic rainfall-runoff simulation model, used for single-event or long-term (continuous) simulation of runoff quantity and quality from primarily urban areas. It is one of the most successful models produced by the U.S. Environmental Protection Agency (EPA) for the water environment (Rossman et al, 2004).

Urbonas (2007) pointed out that for distributed rainfall-runoff models such as SWMM, research and studies are needed to develop guidance in how discretisize a study catchment properly, at least from the end user's perspective. Urbonas (2007) specifies that an issue not solved yet, is how to reduce an irregular shaped catchment with an array of street and development patterns into regular shapes called for in models such as SWMM to get consistent and accurate results.

Another goal was to investigate a proper discretization level in urban hydrology and its use with SWMM, since it was developed specially for urban watersheds. In effect, the SWMM is a dynamic rainfall-runoff simulation model used for single event or long-term (continuous) simulation of runoff quantity and quality from primarily urban areas (EPA, 2008). In addition a MCDA (Multi-Criteria Decision Analysis) methodology was used for model calibration.

At the conclusion of the modeling experiments and calibration methodology, the findings were presented in the ASCE World Environmental and Water Resources Congress celebrated in Kansas City on May 2009.

1.4 Main contributions

Develop a MCDA formulation for calibration based on the observed flow-duration curve. Compare the model's performance indexes for the five different levels of discretization. Provide a practical approach to building large scale rainfall-runoff models in urban hydrology. Develop a GIS toolbox to extract all necessary information for the SWMM5 hydrologic models. Based on a hydrologic systems approach investigate the existence of a Representative Elementary Area (REA) in urban hydrology. The REA represents a hydrologic concept in which the flow volumes for each subcatchment are ranked and filtered according to their size (Wood et al., 1988). The REA is obtained as the scale where $|dq/da|$ becomes small, with q being the hydrograph peak or volume and a the subcatchment area.

The REA concept is explained in detail in Chapter 5.

Using GIS 9.3, describe the analysis needed to delineate a Hydrologic Unit Watershed (HUW) by preserving its physiographic heterogeneity of a drainage basin. Apply the suggested concepts to two highly urbanized basins in North Carolina using Geographic Information Systems (GIS) and SWMM5 as the hydrologic model.

The modeling results are summarized in a protocol to disaggregate urban watershed. This methodology is presented in Chapter 5.

1.5 Significance of the findings

The proposed modeling approach using Hydrologic Unit Watersheds (HUW) represents an innovative concept in urban hydrology. Since with its use, there is no need to model the complete drainage network in the sub-basins. The Kinematic Wave procedure allowed the conversion of irregular sub-watersheds to rectangular shapes as required by SWMM.

The proposed MCDA for calibration is also an original suggestion, because hydrologic parameters for high and low flows are in conflict during traditional calibration. However, the proposed approach permitted separating both objectives by dividing the flow duration curve in two sectors.

Finally, the disaggregation analysis tested the proposed methodology considering different spatial resolutions. The hydrologic experiments permitted to identify a threshold zone in which modeling results become accurate; therefore, the level of resolution where the HUW concept is reliable.

1.6 Organization of dissertation

The thesis is organized in seven chapters. Chapter 2.0 provides an overview of current state of practice in terms of rainfall-runoff modeling and issues needed to solve. Chapter 3.0 addresses the necessary data for modeling and a description of the case studies. Chapter 4.0 describes the proposed Multi-Criteria Decision Analysis to optimize the models and their results to the case studies. Chapter 5.0 presents a discretization analysis of the case studies and compares the simulation results between model outputs. Guidance to disaggregate urban watersheds in SWMM is given. Chapter 6.0 includes a summary of findings, conclusions and recommendations for future research. Finally, Chapter 7.0 presents the references used during the development of this work.

2.0 LITERATURE REVIEW

2.1 Urban drainage models

A model is a representation of a system in some form other than the system itself (Shannon, 1975). The system in this case is the urban watershed. Mathematical models use algebraic relationships to simulate the behavior of the system. Such mathematical models might be as simple as one equation or be composed by hundreds of expressions. Simulation is the process of conducting experiments with a model for the purpose of understanding the system or evaluating strategies for the operation of the system (Shannon, 1975).

Even the most refined models are incorrect to some degree in their representation of the urban watershed and the rainfall-runoff process. Therefore, the model user must always interpret the model results with a clear knowledge of model limitations and assumptions. Also the model user must understand the scientific, engineering and mathematical concepts employed by the model. Any model may extract information from a data base, but cannot overcome data inadequacies, for example, precipitation data. Rainfall records drive the urban hydrologic models. Unfortunately, such records are a major source of uncertainty and bias in many modeling studies. Quite often this happens because too few rain gages have been established to properly capture the spatial variability of precipitation events. Nevertheless, there is no shortage to avoid such potential problems.

Review of past and recent literature revealed extensive advancement in research regarding rainfall-runoff modeling. Some models are better suited for some applications than another ones. For example, if a specific project contains neither the time not money to provide detailed data, a sophisticated model would be unsatisfactory. In a similar way, in case of design work, a model designed to provide rough estimates will be of little use.

This review covered the major areas of research related to the dissertation proposal. Extensive work has been done in developing computer models to simulate the rainfall-runoff phenomena. Some examples of these models are identified below. Watershed models abound in the hydrological literature (Singh, 1989) and state-of-the-art of watershed modeling is reasonably advanced. A brief description of the most common models is given next.

1. Rational Method

Empirical and simplest method to compute storm runoff peak, limited in application to a maximum drainage area of 40 acres. The Rational Method computes peak discharge from an area based on rainfall intensity and a runoff coefficient. Not useful for long term continuous simulation.

As a method of urban hydrology, the rational method falls short in several ways. First, the method does not produce a hydrograph, only a single flow rate. Second, the rational method does not account for changing (time dependent) conditions such as soil condition or rainfall intensity. Finally, results are not very accurate for large areas.

2. MODRAT

MODRAT is a modified rational method computer program developed by the Los Angeles County Department of Public Works (LACDPW, 2006). It was built to enhance the rational method given their shortcomings.

The watershed may be undeveloped, partially developed, or completely developed. The model will compute runoff rates for a 50-year, 25-year, or 10-year frequency design storm. Two main modifications were done to the rational method, *rainfall intensity* is considered as a variable dependent on rainfall frequency, storm time, and time of concentration. Such variation is represented by a temporal distribution curve (rainfall mass curve). The *runoff coefficient* varies with soil type, rainfall intensity, and imperviousness. The above modifications to the rational method allowed for the computation of storm hydrographs for any size watershed. These hydrographs are routed and combined.

3. HEC family programs (US Army Corps of Engineers, 2007)

a) HEC-1. Written in Fortran, it is still used in some US counties because existing old models already developed. Input data is entered in as a text file. HEC-1 is the most commonly-used lumped parameter model available, designed to simulate surface runoff from a single precipitation event.

b) HEC-HMS. The program features a completely integrated work environment including a database, data entry utilities, computation engine, and results reporting tools. A graphical user interface allows the seamless movement between the different parts of the program. The model includes a powerful constrained optimization algorithm when calibration is performed.

c) HEC-GeoHMS. It uses ArcView and Spatial Analyst to develop a number of hydrologic modeling inputs and analyzing digital terrain information, HEC-GeoHMS transforms the drainage paths and watershed boundaries into a hydrologic data structure that represents the watershed response to precipitation.

4. Storm Water Management Model, SWMM (EPA, 2008)

SWMM is a dynamic rainfall-runoff simulation model used for single event or long-term (continuous) simulation of runoff quantity and quality from primarily urban areas. The runoff component of SWMM operates on a collection of subcatchment areas that receive precipitation and generate runoff and pollutant loads. With no doubt, SWMM is the paramount urban stormwater management model and widely used worldwide, therefore, SWMM was the model of choice used in this research work.

5. STORM (Storage Treatment Overflow Runoff Model)

The STORM model was developed by Water Resources Engineers, Inc., in 1977 under a contract with the U.S. Army Corps of Engineers' Hydrologic Engineering Center (USACE-HEC, 1973). In spite of its age, STORM remains a viable and commonly used model. STORM was designed to model urban watersheds and is capable of calculating loads and concentrations of water quality parameters, such as suspended and settleable solids, biochemical oxygen demand, total nitrogen, orthophosphate, and total coliform. STORM is also capable of calculating land surface erosion. STORM is used to aid in sizing of storage and treatment facilities to control the quantity and quality of stormwater runoff and land surface erosion. A continuous simulation model, STORM requires hourly precipitation data.

6. CUHP (Urban Drainage and Flood Control District, 2008)

The Colorado Urban Hydrograph Procedure (CUHP) generates a synthetic unit hydrograph for each sub-catchment, based upon basic equations to define its shape (time to peak and peak runoff). Developed in a excel spreadsheet with seven workbooks.

7. SCS (US Department of Agriculture Soil Conservation Service)

The Soil Conservation Service (SCS) Method was originally developed for small ungaged streams. Should not be used if the basin is not reasonably homogenous, has several main branches, has large storage reservoirs, has a weighted CN less than 40, or has a concentration time (T_c) less than 0.1 hours, or greater than 10 hours. Basic equation relates total and excess precipitation, initial storage and infiltration loses. Allocates runoff curve numbers for land uses and hydrologic soil groups (Mays, 2001).

8. QUALHYMO (QALity HYdrologic MOdel)

This model was developed at British Columbia, Canada, Water Balance Model (2008). Allows users to simulate four situations that integrate the site with the watershed and the stream (site surface alteration, site controls with baseflow discharge, detention pond storage and stream erosion).

9. TR-20

Developed by the USDA Soil Conservation Service (SCS, 1992). Model output consists of peaks and flood hydrographs. Surface runoff is computed by using the dimensionless unit hydrograph, drainage areas, times of concentration, and SCS runoff curve numbers. Routing is established to convey hydrographs. There is no provision for recovery of initial abstraction or infiltration during periods of no rainfall within an event.

10. TR-55

Also developed by the USDA Soil Conservation Service (SCS, 1986). Model particularly applied to small watersheds in urban areas. TR-55 was designed primarily as a set of manual worksheets. TR-55 utilizes the SCS runoff equation to predict the peak rate of runoff as well as the total volume. The model was developed as a simplified method to compute storm runoff in small, urbanized watersheds.

11. HSPF (Hydrologic Simulation Program - FORTRAN)

Originally developed in the early 1960's as the Stanford Watershed Model, ten years later, water-quality processes were added by the Research Lab of EPA in Athens, Georgia (Bicknell et al., 1993). In the 1980's, preprocessing and post-processing software, algorithm enhancements were developed jointly by the USGS and EPA. The HSPF simulates hydrologic and water quality processes. HSPF is generally used to perform a watershed-based analysis of the effects of land use, reservoir operations, point and nonpoint source treatment alternatives. It is accepted by the EPA as a tool for the development of TMDLs (Total Maximum Daily Loads) in the United States.

12. GSSHA (Gridded Surface Subsurface Hydrologic Analysis)

Developed by the US Army Corps of Engineers Engineering Research and Development Center (Downer et al., 2006). The GSSHA model is a significant reformulation and enhancement of the CASCade 2 Dimensional SEDiment (CASC2D-SED) model. The CASC2D-SED runoff model began with a two-dimensional overland flow routing algorithm developed and written in APL at Colorado State University (Rojas et al., 2003).

As it can been seen, extensive work has been done and new models have been created lately. Furthermore, existing ones are in a continuous evolution. Some of them have included optimization algorithms (HEC-HMS, 2007). Select the proper model requires a good engineering judgment, but it is suggested to choose the simplest model capable of meeting the study goals.

In addition, GIS technologies have been successfully coupled with the models to extract terrain data (HEC-GeoHMS, 2007). In general, Geographic Information Systems (GIS) have been applied successfully for modeling purposes in many water resources areas, a few examples are:

a) Automated floodplain mapping.
b) Terrain modeling using Light Detection and Ranging (LiDAR).
c) Hydraulic modeling in river systems.
d) Development of real-time flood warning systems.

GIS has been particularly helpful for hydrologic modeling, given the spatial features of the needed data: ground slope, areas, land uses coverage and so on. For example, hydrologic modeling using digital terrain coverages has allowed automated watershed delineation. From all the available models, SWMM was chosen as the model engine, since it was developed specifically for urban areas, its world wide acceptance and long-term modeling capacity.

2.2 Distributed parameter hydrologic models

Distributed parameter hydrologic models sub-divide an entire watershed into smaller units to represent heterogeneity within the watershed. Distributed parameter hydrological models are being increasingly used in investigations of spatial scale and catchment heterogeneity as well as general rainfall-runoff applications. Sivapalan and

Kalma (1995) recognized that spatial and temporal scales generally lead to predictive uncertainty in distributed hydrological modeling.

Heterogeneity in urban watersheds presents a great complexity and so far has virtually challenged detailed description and/or measurement. Other branches of science which deal with similar heterogeneity issues adopt a 'continuum approach' and ignore the local (microscale) heterogeneity for modeling purposes, replacing the real system by a fictitious system at the macroscale. For example, in groundwater hydrology, the detailed patterns of pore structure at the microscopic level are replaced by a continuous field of porosity at the macroscopic level (Blöschl et al., 1995). In hydrologic modeling, a critical problem in the application of these models is the choice of a proper *element size*, which must able to represent the heterogeneity of the catchment response.

Through time, hydrological basin models have become more distributed and physically based to reflect heterogeneous and complex basin structures and the various interactive processes controlling the basin response (Flügel, 1995).

In Robertson, Australia (between November 30 and December 2, 1993) was held a three-day workshop on Scale Issues in Hydrological and Environmental Modeling. The objective of this workshop was to discuss recent progress in this area, and to develop appropriate research strategies for linking model parameterization across a range of spatial and temporal scales. The presented papers reflected the thinking in scale-related research at that moment.

Wood et al. (1988) was the first to introduce the idea of a *Representative Elementary Area* (REA) in the context of hydrologic modeling on the catchment scale. REA is a spatial scale over which the process representations can remain simple and at which distributed catchment behavior can be represented without the apparently indefinable complexity of heterogeneity. The case study was the Coweeta River (Area=17 km^2), an experimental basin in North Carolina, with synthetic realizations for rainfall and soils. Wood used a 30 m Digital Elevation Model (DEM). The hydrologic response of this catchment was modeled by a modified version of TOPMODEL, modeling both infiltration excess and saturation excess runoff and incorporating the spatial variability of soils, topography, and rainfall. The effect of scale was analyzed by first dividing the catchment into smaller subcatchments and determining the average water fluxes for each subcatchment. Modeling results showed the stabilization of the mean areal response ocurred when the average area is equal to about 1.0 km^2.

Blöschl et al. (1995) used the same Coweeta basin mentioned previously with a 30 m GIS raster resolution. The resulting element network contained 6206 elements with an average size of 50 by 50 m, where model parameters were assumed to be constant within each element. The flow volumes for each subcatchment were ranked and filtered according to subcatchment size in an identical manner to that of Wood et al. (1988). The REA is described as the scale where $|dq/da|$ is small, with q being the hydrograph peak or volume and a the subcatchment area. Blöschl et al. (1995) found that this occurs when the square root of the sub-catchments Area is around 600 to 1,200 m. However, the author concluded that there is no evidence for one universal size of an REA or one universal 'optimum element size' in the context of distributed rainfall-runoff modeling.

Arabi et al. (2006) used the Soil and Water Assessment Tool (SWAT) model to calibrate and validate for streamflow, sediment and nutrient yields at the outlet of the Dreisbach (623 ha) and Smith Fry (730 ha) watersheds in Maumee River Basin, Indiana.

The author was seeking an appropriate level of watershed subdivision to evaluate BMP's. The conclusion was to use 4% of the total watershed area. These two watersheds were mostly undeveloped and relative small.

In 2003, the hydrologic community worldwide noticed that scale issues of distributed hydrological modeling needed further study, to address these research issues, a symposium entitled "Weather Radar Information and Distributed Hydrological Modelling" was held during the International Union of Geodesy and Geophysics (IUGG), the XXIII General Assembly of the at Sapporo, Japan, from 30 June to 11 July, 2003. A total of 42 papers were presented and one special section was entitled "Assessment of Performance in Distributed Hydrological Modeling". The symposium was organized by the International Commission on Surface Water (ICSW) of the International Association of Hydrological Sciences (IAHS). Researchers agreed that a suitable model resolution and required forcing resolution might be a function of catchment scale, but the relationship between was still unknown.

Roshan et al. (2006) applied a distributed macro-scale hydrological model (MaScOD) using a 10-minute spatial resolution to the Huaihe River basin in China. The objective was to simulate the discharge in three basins: Bengbu (132,350 km^2), Wangjiaba (29,844 km^2) and Suiping (2,093 km^2). The author used different IC-ratio values, where IC-ratio is defined as the input resolution divided by the total catchment area. In this way, a high IC-ratio corresponds to a finer resolution and a low IC-ratio means a coarser resolution. Modeling results showed that results are satisfactory while the IC-ratio remains above 1:10. Also, model performance was found to level off above an IC-ratio 1:20. Roshan et al. (2006) presented a qualitative relationship between model performance and scale performance.

If the cost of modeling is included (dashed line) and also a performance cost is associated (continuous line) and both costs are added, an optimum performance range for IC-ratio might be obtained as shown in Figure (2.1).

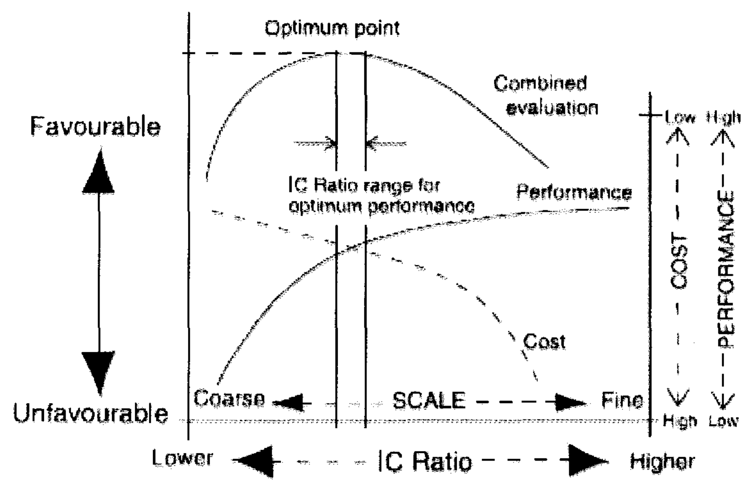


Figure 2.1 Trade-off between model performance and model cost with respect to scale in terms of the IC-ratio (Roshen, 2006).

Bathurst (1986) suggested to divide the watershed into elements no larger than 1% of the total area. This was the main conclusion from his study on the Wye watershed (10.55 km^2) using the SHE (Système Hydrologique Européen) model, to ensure that each grid element was more or less homogeneous.

2.3 Discretization issues in urban hydrology

Essentially all urban runoff models are lumped models, in other words, the physical characteristics of the watersheds are assumed to be spatially constant. This is in contrast to more sophisticated models that directly account for the physical heterogeneities in the watershed (distributed models). A lumped model can be made to perform somewhat like a distributed model if the watershed is divided into several sub-watersheds. In this case, each watershed is given spatially constant characteristics. The spatial detail of a watershed can be increased by dividing it into more subwatersheds. However, additional detail increases complexity that perhaps is not needed.

An appropriate level of discretization or detail depends to the modeling objectives. If rapid urban development is predicted for a particular watershed, it would be convenient to use several sub-watersheds and then, update the model once urbanization process occurs. In the context of urban hydrology, it is not clear what level of discretization is necessary. Once a disaggregation level is selected, another issue to be solved is how to represent the complex system of storm drainage components in the rainfall-runoff model.

The appropriate level of discretization is also related to how the model responds to different levels of discretization.

A finer resolution input data is preferred for its better description of spatial variability. However, it may be an impractical effort to include every detail of input field in catchment-scale modeling, especially in the case of urban watersheds, where complete drainage network information might be difficult to find or impractical to model.

In general, the minimum amount of input data, the minimum number of parameters and the minimum computational load which produce reasonable simulation results, make the model easier to apply and more effective (Roshan et al., 2006). The challenge is to find a scale above which spatial variability can be neglected, nevertheless, with average characteristics of a given area providing sufficient information for proper modeling of basin runoff in urban areas.

Currently a critical problem in urban hydrologic models is the proper choice of sub-watersheds, which must able to represent the heterogeneity of the urban basin response. The proper choice of an adequate resolution level not only affects classical rainfall-runoff models, since also hydrologic models are developed also to predict other environmental parameters, such as sediment and nutrient yield prediction.

Previous research on long-term rainfall modeling has not incorporated the effect of sub-watershed resolution. This particular topic was investigated in this research work. A sensitivity analysis was performed considering different discretization scenarios and results were compared through measurement of performance described with in Section 2.5. An optimal watershed sub-division level was identified trough long-term numerical simulations. This was done by using the SWMM model in the context of two urban watersheds in North Carolina (Section 3.2).

2.4 Imperviousness in urban hydrology

Imperviousness is probably the most important indicator of the impact of urbanization on storm water systems. Man-made impervious cover has long been known to significantly affect the hydrologic response of a watershed. Imperviousness is the most critical indicator for analyzing urbanization impacts on the water environment (Schueler, 1994; Arnold and Gibbons 1996; Joint Task Force of the Water Environment Federation and the ASCE, 1998).

Urban development can have a major impact on the local hydrology and water environment, since higher levels of impervious surfaces result in a higher volume of runoff with higher peak discharge, shorter travel time, and significant pollutant loadings.

In urban hydrology, it is well known that imperviousness levels reduces the time to peak and increases the peak flow. Huang et al. (2008) studied the effect of growing watershed imperviousness in the Wu-Tu watershed in Taiwan (Area=204 Km^2). The author analyzed 50 rainfall-runoff events from 1966 to 1984; during that period, the time to peak of flood hydrographs for various storms was reduced approximately from 11 hours to 6 hours, while peak flow increased almost five times, from 127 m^3/s (4,500 cfs) to 629 m^3/s (22,000 cfs).

For modeling purposes, two types of impervious areas have been identified. The first type, Effective Impervious Area (EIA), comprises those impervious surfaces that are hydraulically connected to the channel drainage system. Streets with curb and gutter and paved parking lots that drain onto streets are examples of effective impervious surfaces. This area is also known as *Directed Connected Impervious Area* (DCIA). The second type, non-effective impervious area, comprises those impervious surfaces that drain to pervious ground such as roof that drains onto a lawn. The sum of both is known as *Total Impervious Area* (TIA). Alley and Veenhuis (1983) developed the following empirical relationship between TIA and DCIA from a highly urbanized portion of Denver, using 14 basins. The coefficient of determination (r^2) for the regression equation (2.1) was 0.98 and the standard error of estimate 7.5%. The obtained equation was:

$$DCIA = 0.15\ (TIA)^{1.41} \qquad \dots (2.1)$$

Alley (1983) suggested to calibrate DCIA using the smaller storms for which runoff is largely from the effective impervious area of the watershed and to calibrate infiltration parameters using the larger storms.

Direct connectivity to the drainage system is an important attribute of urban imperviousness. Lee and Heaney (2003) performed a 52-year simulation in a 5.95 ha drainage basin in South Florida. The author found that about 70% of the runoff events are contributed by DCIA runoff only. While DCIA represents about 44% for the site, it contributes about 72% of the total runoff during the simulated period. In addition, Lee and Heaney (2003) presented a detailed analysis of urban imperviousness using GIS and field investigations on a 5.81 ha residential area in Boulder, Colorado. For this study area, the total impervious area was 35.9% and the DCIA was 13.0%. The author found that curb and gutter drainage is the major source of DCIA. In effect, transportation-related imperviousness was 64% of the TIA and 97.2% of the DCIA. Similar to the study in South Florida, DCIA was the primary contributing area for smaller storms.

2.5 Measurements of performance

Measurements of performance allow an assessment of the reliability of the modeling results and therefore, the predictive power of hydrological models. The agreement between the observed and simulated volume and peak flow may be expressed in terms of a bias or departure. *Bias* indicates systematic over or under prediction. *Departure* serves as a measure of the prediction accuracy (Vieux, 2004). The following discussion addresses the measurements of performance that will be used in this dissertation.

i) Root Mean Square Error (RMSE). This measure takes the distance vertically for all the given points (the error) and square the value. The squaring is done so negative values do not cancel positive values. Then all values are added and divided by the number of points. Finally, the square root is taken to have the same original units. Hence, the RMSE is the vertical distance, on average, between the modeled and the observed flows.

$$RMSE = \sqrt{\frac{1}{N}\sum_{i=1}^{N}(O_i - S_i)^2} \qquad \dots (2.2)$$

ii) Pearson's Moment Correlation Coefficients (PMCC). Pearson's correlation reflects the degree of linear relationship between two variables. It ranges from +1 to -1. A correlation of +1 means that there is a perfect positive linear relationship between variables. A correlation of -1 means that there is a perfect negative linear relationship between variables. A correlation of 0 means there is no linear relationship between the two variables. The statistic is defined as the sum of the products of the standard scores of the two measures divided by the degrees of freedom.

$$PMC = \frac{\sum_{i=1}^{N}(O_i - \bar{O})\cdot(S_i \bar{S})}{\sqrt{\left[\sum_{i=1}^{N}(O_i - \bar{O})^2\right]}\sqrt{\left[\sum_{i=1}^{N}(S_i - \bar{S})^2\right]}} \qquad \dots (2.3)$$

iii) Nash-Sutcliff Coefficient, NSC (Nash and Sutcliffe, 1970). This calibration performance equation was suggested by ASCE Task Committee on Definition of Criteria for Evaluation of Watershed Models of the Watershed Management Committee (1993).

$$NSI = 1 - \frac{\sum_{i=1}^{N}(O_i - S_i)^2}{\sum_{i=1}^{N}(O_i - \bar{O})^2} \qquad \dots (2.4)$$

iv) Index of Agreement, IOA (Willmott, 1981). The IOA is a standardized RMSE, it can vary from 0 (total disagreement) to 1 (total agreement).

$$IOA = 1 - \frac{\sum_{i=1}^{N}(O_i - S_i)^2}{\sum_{i=1}^{N}(|S_i - \overline{O}| + |O_i - \overline{O}|)^2} \quad \ldots (2.5)$$

Where in equations (2.2) through (2.5):

O_i	Observed value at the i^{th} time
S_i	Simulated value at the i^{th} time
N	Total number of observations
$\overline{O}$ and $\overline{S}$	Mean values of O_i and S_i

In summary, the best condition is that PMCC, NSC, IOA yield a value of unity and RMSE is zero.

2.6 Importance of major and minor floods

Major and minor hydrologic events are both important in urban hydrology. Particular applications require an accurate prediction of major events for floodplain delineation or design of hydraulic structures, associated with high return periods. The Federal Emergency Management Agency defines the floodway based upon a 100-yr event (FEMA, 2008). Culvert design under federal highways must have enough hydraulic capacity to pass a 50-yr storm (USDOT, 2005). However, other purposes need a more correct estimation of smaller events. Leopold et al. (1964) hypothesized that a channel adjusts its downstream hydraulic geometry in response to the bank full flow, which is the annual peak flood with a return period of about 2.3 years. Hence, peak floods are viewed as one of the main geomorphic agents which shape drainage networks and landscapes. Pomeroy (2007) found that qualitative EPT (macroinvertebrate metric Ephemeroptera, Plecoptera and Trichoptera) richness metric values are linearly related to the 0.5-year return interval storm. Roesner et al. (1991) studied the urban runoff in six U.S. cities with different climate conditions and Roesner found that most of the runoff is generated by small storms (4-month storm) and generally produces less than 0.5 inches of runoff.

2.7 Flow duration curve applications

The issue of whether a modeling study should focus on a single design event or on a long continuous series of events deserves some discussion. In general, continuous simulation is better suited for planning purposes and preliminary assessments, whereas single-event simulation is better suited for the analysis of design conditions.

In general, a model calibrated for a small event performs poorly for a large event and viceversa. Therefore, in this research, modeling was done on a long term basis where all events sizes were considered by using the flow duration curve. The flow duration curve is a plot that shows the percentage of time that flow in a stream is likely to equal or exceed some specified value of interest. For example, it can be used to show the percentage of time river flow can be expected to exceed a design flow of some specified value or to show the discharge of the stream that occurs or is exceeded some percent of the time.

For most water resources applications, a mean daily discharge is used. However, any time unit could be used. The basic time unit used in preparing a flow-duration curve will greatly affect its appearance. When the mean flow over a long period is used (such as mean monthly flow), the resulting curve will be flatter due to averaging of short-term peaks with intervening smaller flows during a month. Extreme values are averaged out more and more, as the time period gets larger (e.g., for a flow duration curve based on annual flows at a long-record station).

A flow duration curve is a plot of discharge vs. percent of time that a particular discharge was equaled or exceeded. The area under the flow duration curve (with arithmetic scales) gives the average daily flow, and the median daily flow is the 50% value. A flow duration curve characterizes the ability of the basin to provide flows of various magnitudes. Information concerning the relative amount of time that flows past a site are likely to equal or exceed a specified value of interest is extremely useful for the design of structures on a stream. For example, a structure can be designed to perform well within some range of flows, such as flows that occur between 20 and 80% of the time (or some other selected interval).

The shape of a flow-duration curve in its upper and lower regions is particularly significant in evaluating the stream and basin characteristics. The shape of the curve in the high-flow region indicates the type of flood regime the basin is likely to have, whereas, the shape of the low-flow region characterizes the ability of the basin to sustain low flows during dry seasons.

A very steep curve (high flows for short periods) would be expected for rain-caused floods on small watersheds. Snowmelt floods, which last for several days, or regulation of floods with reservoir storage, will generally result in a much flatter curve near the upper limit. In the low-flow region, an intermittent stream would exhibit periods of no flow, whereas, a very flat curve indicates that moderate flows are sustained throughout the year due to natural or artificial streamflow regulation, or due to a large groundwater capacity which sustains the base flow to the stream.

3.0 CASE STUDIES AND MODELING APPROACH

3.1 Methodology

As discussed in Chapter 1, one of the objectives of this research was to develop a hydrologic procedure to discretisize highly urbanized watersheds. Calibration was done using a Multi-Criteria Decision Analysis (MCDA) applied to the flow-duration curve. In addition, the effects of subdivision level (sub-basin scale) on runoff simulation were investigated. The proposed methodology and the discretization analysis were tested in two urban basins in North Carolina. From the several hydrologic models described in Chapter 2, the USEPA Storm Water Management Model (SWMM) was chosen because it was developed primarily for urban areas and it is been successfully applied worldwide.

3.2 Case studies

The proposed methodology was tested on two urban watersheds. Both basins are located in the Wake County, North Carolina as shown in Figure (3.1).

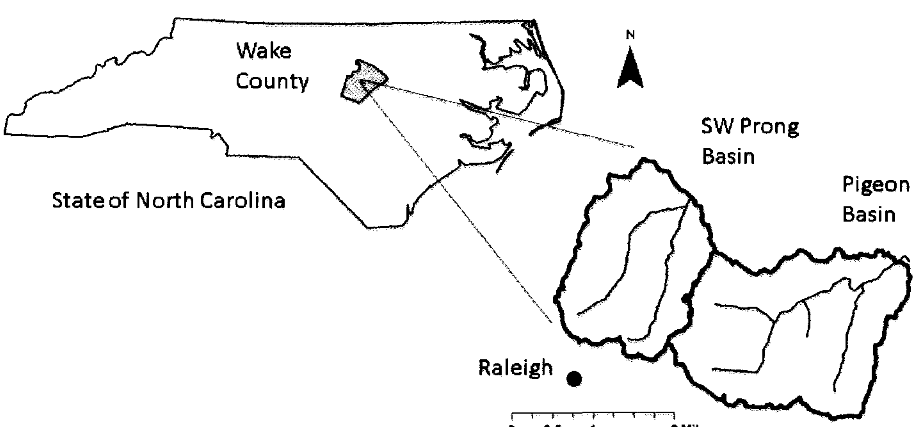


Figure 3.1 Location of the basins

The land use and main properties of these watersheds are shown in Tables (3.1) and (3.2), respectively. Pigeon presents a higher urbanization level than SW Prong, the much higher TIA value in Pigeon is because it has highly developed and commercial areas. Both basins are located in the Raleigh Metropolitan Area. These watersheds served as examples of the methodology described herein.

Table 3.1 Watersheds land use (Year 2000)

Watershed Name	Landuse Breakdown (%)				
	Developed High Intensity	Developed Medium Intensity	Developed Low Intensity	Developed Open Space	Forest or Rural
Pigeon	6	20	28	45	1
SW Prong	1	3	16	75	5

Table 3.2 Watersheds properties (Year 2000)

Watershed Name	Area (mi^2)	TIA[1] (%)	Population density (hab/mi^2)	Housing density (homes/mi^2)	UII [2]
Pigeon	4.45	30.35	3,204	239	100.00
SW Prong	3.02	11.45	3,093	198	90.80

[1] *TIA, Total Impervious Area*

[2] *UII, Urban Infrastructure Index* is a multi-metric parameter that represents the degree of development in the watershed. The UII includes census, socioeconomic, infrastructure, land use and land cover metrics that correlated with population density (McMahon and Cuffney, 2000). However, in this case, UII represents a specific index developed for a set of 30 watersheds in North Carolina, ranging from 0 to 100 (Giddings et al., 2007).

3.3 HUW modeling principles

The entire urban watersheds were divided in Hydrologic Unit Watersheds (HUW). A HUW represents a sub-catchment in the watershed in which width is obtained by optimization (Section 3.3.1). Next sections describe how to obtain them. Initially, their spatial distribution is made from a Digital Elevation Map (DEM) using ArcHydro (2003).

Unlike undeveloped watersheds, urban watersheds present an additional component to be modeled, the drainage network. However, even if there is an existing GIS shapefile with all links and their corresponding geometric features, modeling the complete network is not feasible in large watersheds because modeling the entire system require extensive field work just to obtain the data. Then, this task would represent a big engineering effort to model every single storm drainage structure (i.e., gutter flow towards a drop inlet). Therefore, realistic simplifications should be made to save computer time.

For the cases studies to be modeled, drainage network data was available. The City of Raleigh (2001) located important drainage structures (inlets, pipes and open channels) using Global Positing System (GPS). For Pigeon basin, the database was composed by more than 5,000 link elements. However, the procedure described in the next section was applied to obtain an equivalent drainage network.

3.3.1 Internal routing

Since drainage network was not modeled, a simplified and equivalent routing channel was obtained to represent the entire drainage network in each sub-watershed. Some attempts have been made to obtain a comparable drainage network. Brink (2004) suggested creating a routing channel with the objective of accounting for in-system storage and attenuation that would occur within a given subarea by developing empirical relationships for length and width of this routing channel as a function of the sub-catchment area (A):

$$L = \sqrt{A} \quad \text{... (3.1a)}$$

$$W = \frac{\sqrt{A}}{2} \quad \text{... (3.1b)}$$

Where:

L = Length of routing channel

W = Width of routing channel

For the case studies, since the only point in the watersheds with recorded flows is the outlet, there are no observed flows for any of the sub-watersheds; therefore, a synthetic hydrograph method was used to develop a proper size for the routing channel length in each sub-watershed.

Espey et al. (1977) developed a set of generalized equations for the construction of unit hydrographs using a study of 41 watersheds ranging in size from 0.014 to 15 mi^2, and impervious percentage from 2 to 100 percent, nine of the watersheds were located in North Carolina. Espey et al. (1997) found the following relationships to estimate the Time to Peak (T_p, minutes) and Peak Flow Rate (Q_p, cfs/in) as follows:

$$T_p = \frac{3.1L^{0.23}\Phi^{1.57}}{S^{0.25}I^{0.18}} \quad \text{... (3.2)}$$

$$Q_p = \frac{31.62x10^3 A^{0.96}}{T_p{}^{1.07}} \quad \text{... (3.3)}$$

Where:

L: Length of the routing channel (ft)

Φ: Dimensionless watershed conveyance factor, which is a function of the channel roughness and watershed imperviousness (Figure 3.2)

S: Main channel slope (feet/foot)

I: Imperviousness level (%)

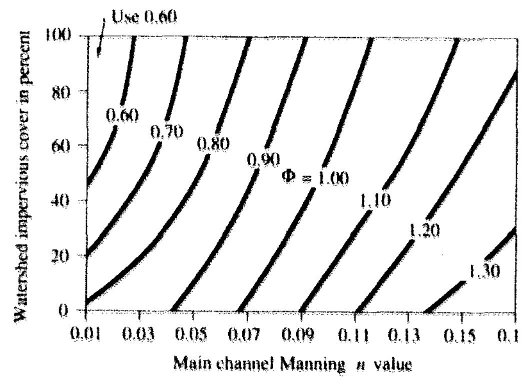


Figure 3.2 Watershed conveyance factor

Design storms are developed using long term runoff simulation and typically are used to design storm sewers, detention ponds and other flood control facilities. Nevertheless, Urbonas (1979) pointed out that is possible to develop design storms that reasonably duplicate the peak flows from small urban basins at various recurrence intervals.

For this study, a 24-h precipitation depth with a Return Interval of 1 Year was estimated to be 2.87 inches (NOAA, 2008) at Raleigh State University. Then, using a SCS Type II storm distribution, peak discharges and excess rainfall depths were computed in SWMM5 for eight experimental sub-watersheds ranging from 5 to 140 acres. Next, using the proposed methodology by Espey et al. (1997) for these sub-watersheds, the length of the main channel (L) was optimized to minimize the RMSE (Root Mean Square Error, Equation 2.2) between the peak runoff of both methods

(SWMM5 and the Unit Hydrograph). For this effect, an optimization algorithm was implemented. The relationships that minimized the RMSE were found to be: $L=A^{0.5967}$ in Pigeon and in $L=A^{0.5436}$ in SW Prong.

The lower exponent in SW Prong is due to its lower development level than Pigeon, in other words, in a given sub-catchment the flow paths to reach the outlet are shorter. The latter results are congruent with the relationship provided by Brink (2004). In both watersheds, typical channel slopes were used based on the DEM and DCIA imperviousness levels were applied. The imperviousness correction factors to obtain DCIA are shown in Appendix A. Table (3.3) shows the parameters used.

Table 3.3 Experimental sub-catchments data

Parameter	Pigeon basin	SW basin
Channel slope (S)	0.018	0.024
Manning's roughness (n)	0.035	0.035
Imperviousness level (DCIA, %)	13.57	5.36
Conveyance factor (Φ)	0.85	0.87

Figure (3.3) shows a comparison between flows. The RMSE in Pigeon was 1.72 cfs and 1.31 cfs in SW Prong.

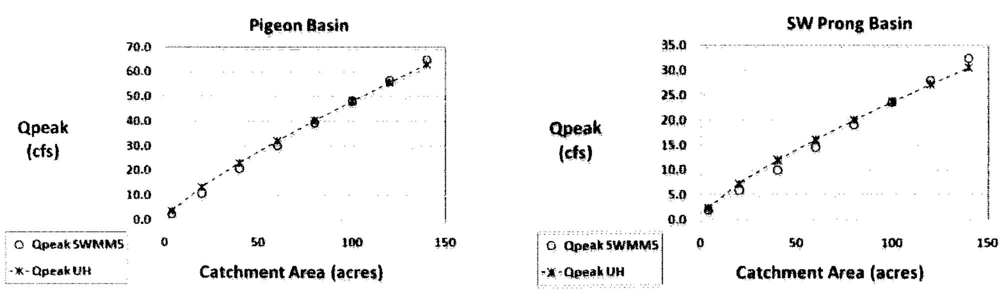


Figure 3.3 Peak flow rates in Pigeon and SW Prong basins

Results agreed with the Hack's law (Hack, 1957), an empirical relationship between the length of streams and the area of their basins. Such law relates the length of the longest stream in a basin (L) with the area of the basin (A):

$$L \propto A^{h} \qquad \ldots (3.4)$$

Many other researchers have corroborated Hack's original study and, although the exponent in the power law may slightly vary from region to region, it is generally accepted to be slightly less than 0.6 in most basins (Rigon et al., 1996). Exponent h decreases for larger basins (>8,000 mi^2 or 20,720 km^2).

3.3.2 Conversion of irregular watersheds

SWMM model requires the width be specified for every sub-watershed; the model assumes all sub-watersheds having a rectangular shape. However, the sub-watersheds defined with ArcHydro have irregular shapes and a conversion was needed.

As recommended (EPA, 2008), an irregular urban catchment can be converted to its equivalent rectangular shape. As illustrated in Figure (3.4), the uniform rainfall distribution is applied to the rectangular watershed that has a central channel collecting the overland flows from both sloping planes.

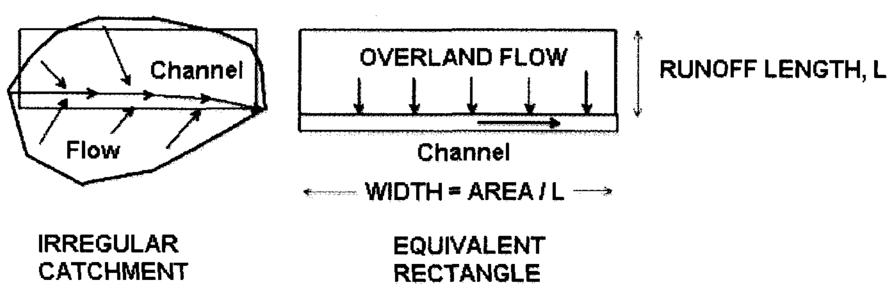


Figure 3.4 Conversion of Irregular Catchment into Equivalent Rectangular Shape (From Guo, 2005)

The SWMM5 user's manual suggests that the width parameter can be used to account for internal routing and attenuation, enabling delineation for larger subareas with less detail needed in defining the conveyance network. Huber and Dickinson (1988) suggested combining many sub-catchments into a single lumped or equivalent and calibrate the sub-catchment width. Reducing the width increases the flow length and storage within a subarea, resulting in an effective way to attenuate the runoff hydrograph without modeling in-system storage and pipe networks (Brink et al., 1996), for this reason, the width parameter is often the primary parameter adjusted to obtain desired peak flow rates and hydrograph shapes. Since travel time is larger, this results in more time for infiltration to occur and therefore, infiltration volume might be overestimated (Brink and Broek, 1988).

However, it is easily observed that a reduction in the width increases overland flow length and overland flow travel time; therefore, there is more time for infiltration to occur. This results in an effective attenuation of the runoff hydrograph without modeling in-system storage and pipe networks (Brink and Broek, 1988).

Brink and Broek (1988) also pointed out that a solution to this problem would be to avoid large sub-areas and developed a more detailed conveyance system network. This idea may or may not be feasible, depending whether the required information is available, project time and budget.

Another approach could be to estimate the runoff length (R_o), that is, the actual distance that flow typically could be expected to travel before reaching a directly connected impervious surface or a natural channel in an urban environment (usually from 100 to 300 feet). After traveling this distance, flow will either reach an impervious surface or becomes shallow concentrated flow. Then, compute the sub-catchment Width (W) as:

$$W = A / R_o \qquad \text{... (3.5)}$$

Where A is the total area of the sub-catchment. Rivas and Roesner (2007) used the latter procedure successfully in large urban watersheds. With this approach, for each sub-watershed, three to five runoff lengths need to be measured in typical lot sizes (from the back of the lot until the street center line) and then, the arithmetic average is computed. Estimating the runoff length and then computing the width yield more accurate results than taking a direct measurement of the width. This assumption is valid as long the homogeneity of the watershed remains constant; non-homogeneous watersheds require further discretization until homogeneous sub-watersheds result. However, this task requires the estimation of runoff lengths through aerial images in sub-catchments. Even when the last procedure is realistic, it might be time consuming and infeasible in large watersheds.

For example, a schematic in Figure (3.5) shows three sub-watersheds with three different runoff lengths (L1, L2 and L3). The runoff in each sub-watershed drains to the junctions (J1, J2 and J3) and then, through the drainage network to the outlet.

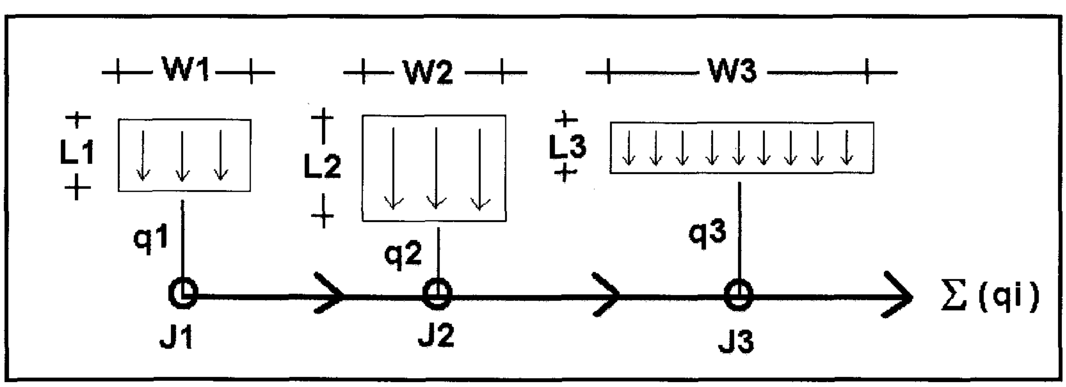


Figure 3.5 Width determination

A Kinematic Wave (KW) cascading plane is specified by the plane's area, width and slope (Guo and Urbonas, 2008). The current state-of-practice recommends that the KW plane width be twice the length of the central channel in a symmetric watershed or equal to the length of the side channel along the watershed boundary (Huber and Dickinson, 1998).

Guo and Urbonas (2008) developed a methodology to convert an irregular watershed to its equivalent rectangular watershed, where the *continuity* and *energy principles* were interpreted to preserve the watershed area and vertical fall over the receiving waterway's length. Figure (3.6) shows how a natural watershed with irregular shape may be converted to a rectangular watershed. The longitudinal slope (S_o) is defined by the vertical fall along the receiving waterway for the natural watershed. The KW plane slope (S_w) is virtual and only used in computation. After numerous tests Guo and Urbonas (2008) confirmed that the watershed and KW shape factors provide a consistent and stable basis for watershed geometric conversion.

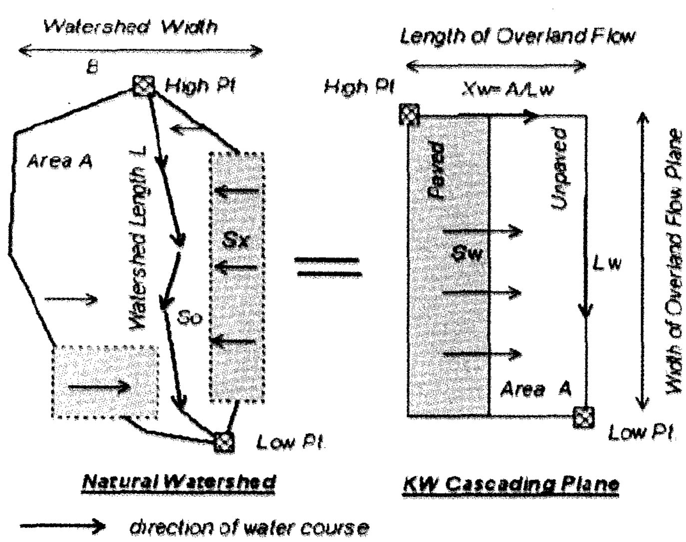


Figure 3.6 Natural Watershed and KW Plane, from Guo and Urbonas (2008).

The set of equations to perform the conversion are presented (Guo and Urbonas, 2008):

1. Natural watershed shape factor (X):

$$X = \frac{A}{L^2} \cong \frac{B}{L} \leq K_s \qquad \text{... (3.6)}$$

K_s is the upper limit of shape factor. The Colorado Urban Hydrograph Procedure (CUHP) suggests K=4 (UDFCD, 2005) to avoid sub-areas too wide in shape.

2. KW shape factor (Y)

$$Y = \frac{L_W}{L} = (1.50 - Z)\left[\frac{2}{1-2K_S}X^2 - \frac{4K_S}{1-2K_S}X\right] \qquad \text{... (3.7)}$$

In which Z= A_m/A, (area skewness coefficient between 0.5 and 1.0). A_m = dominating area that is the larger one between the two subareas separated by the collector channel. For a symmetric watershed, Z=0.5. For a side channel along the watershed boundary, Z=1. From visual inspection all sub-watersheds in Pigeon and SW Prong Basins were identified to have a central channel.

3. Finally, the potential energy along the water course is preserved by:

$$\frac{S_0}{S_W} = \frac{X}{Y} + Y; X \leq K_s \qquad \text{... (3.8)}$$

In summary, in a HUW (Hydrologic Unit Watershed), the routing channel length (L) was estimated through an optimization algorithm as a function of the sub-watershed size (A) and then, the routing channel length was used as a parameter for the conversion from irregular to regular sub-catchments using the procedure proposed by Guo and Urbonas (2008). An advantage of the latter method is that the only parameters needed in each sub-catchment are the sub-catchment area and its routing channel length. This proposed procedure showed to be effective and simple to implement and results were still accurate. The necessary adjustments for the hydrologic parameters (infiltration and correction factors K_1 and K_2 for imperviousness raster maps) from the standard literature values were found to be minimum.

Rainfall in each sub-catchment becomes overland surface runoff after the soil is saturated and then, it was routed through the main channels. However, runoff from the directly connected impervious area (DCIA) occurs rapidly and comprises the bulk of the runoff from urbanized areas (Lee and Heaney, 2002).

3.4 Main channels

Arc Hydro (2003) is a GIS extension to obtain drainage patterns in catchments from the Digital Elevation Model (DEM). Raster analysis was performed to generate data on flow direction, flow accumulation, stream definition, stream segmentation, and watershed delineation. These data are then used to develop a vector representation of catchments and drainage lines from selected points. It was found that the stream patterns defined with ArcHydro (2003) follow the drainage network.

Natural streams are an integral part of the drainage system. Preliminary site investigations were already performed to identify conveyance features of the main streams. Field work was done during November 2005 to obtain the cross sections in main channels using level and rod methods as described by Harrelson et al. (1994). Streams cross sections were taken at approximately 2,500-foot intervals along main channels (Pomeroy, 2007).

Channel Manning's roughness was estimated during the field work by following the guidelines provided by Chow (1959) and Arcement and Schneider (1989). GPS coordinates of the cross sections were mapped with a unit eTrex Legend (Garmin, 2008), see Figure (3.7).

Figure 3.7 Field work in main channels

3.5 Hydrologic parameters

Table (3.4) shows the needed watershed model input data, divided by type.

Table 3.4 Watershed model input data

WATERSHED MODEL INPUT DATA		
Sub-catchment Characteristics	**Channel Conveyance**	**Calibration Data**
Drainage area	Cross sections	Rainfall depth
Width	Bottom slope	Streamflow records
Overland ground slope	Length	
Overland flow slope	Roughness	
Soil infiltration rates		
DCIA coefficients		
Depresion storage		
Overland flow roughness		
Groundwater parameters		

Each HUW (Hydrologic Unit Watershed) was defined by GIS/Optimization analysis; the following data is necessary to describe a HUW:

1. Digital Elevation Model (DEM)

DEM was obtained through The USGS National Map Seamless Server (2008). The website provides The National Elevation Dataset (NED) with a 1/9 Arc Second resolution (approximately 3 meters). One Arc Second is the $1/3600^{th}$ of a degree (1 sec) of latitude or longitude. The length of arc subtended is approximately 30 meters. ArcHydro (2003) processes the DEM raster to discretisize the watershed, that is, the watersheds are discretized into sub-areas using the ArcHydro extension of ArcGIS; impervious areas and slopes were estimated for each sub-area.

Overland slope is obtained through the DEM, using the command *Slope* from Spatial Analysis, a GIS extension. Overland slope is calculated for the whole watershed and then extracted to each of its sub-catchments using the *Clip* tool in GIS.

2. Soils data and infiltration

The Piedmont ecoregion of North Carolina is characterized by smooth rolling hills. The region is a transition topographically between the Appalachian Mountains to the West and the coastal plains to the East.

The regions's lithology is composed of eroded metamorphic rocks and ultisol soils. Soils belonging to the ultisol type have a low amount of organic matter and a low fertility due to their acidic well-drained character (Hudson, 2002). Initial estimates of a watershed's soil infiltration were obtained through Soil Conservation Service (SCS) maps. These maps classify soils into four Hydrologic Soil Groups (A, B, C, and D) based on textures and runoff potentials.

It is possible to link these Hydrologic Soil Groups with initial (f_o) and ultimate (f_c) Horton infiltration values and obtain a weighted initial estimation of watershed infiltrations. Horton Method was used to model infiltration:

$$f_p(t) = f_C + \frac{f_0 - f_C}{e^{Kt}} \qquad \ldots (3.9)$$

Where: f_c: Minimum (final) infiltration rate, ([L]/[T])

f_o: Maximum (initial) infiltration rate, ([L]/[T])

K: Decay coefficient, (1/[T])

t: Time, ([T])

The decay coefficient (K) determines the time elapsed in which the soil becomes saturated and it reaches its final infiltration rate (f_c). Typical values of infiltration decay coefficients are shown in Table (3.5).

Table 3.5 K and f_c relationship

Decay coefficient (1/h)	% decline of infiltration capacity toward limiting value f_c after 1 hour
2	76
3	95
4	98
5	99

Horton infiltration rates are reported below by NRCS (Natural Resources Conservation Service) divided by Hydraulic Soil Groups (Table 3.6).

Table 3.6 Soil textures and hydrologic soil groups

Hydraulic Soil Group	f_c (in/hr)			f_o (in/hr)
	High	Low	Average	
A	0.45	0.30	0.38	7.50
B	0.30	0.15	0.23	4.50
C	0.15	0.05	0.10	3.00
C/D	0.10	0.03	0.06	2.25
D	0.05	0.00	0.03	1.50

In terms of the hydrologic soil group (HSG) classification, all soils in these regions are classified as either group B or C. Group B soils have infiltration rates of 0.30 to 0.15 in/h, a moderately low runoff potential and a moderate rate of water transmission. Group C have infiltration rates of 0.15 to 0.05 in/h, a moderately high runoff potential, a low infiltration rate, and are good to well drained soils (Patra, 2000).

Infiltration rates were estimated based on ArcGIS shapefiles, these features were converted to 10-m raster maps and a weighted average method was used to determine the infiltration rates in each basin.

In SWMM5 another parameter of infiltration is the drying time (T_w, days). T_w represents the time it takes of the soil to recover its initial infiltration capacity (return to the initial condition). In other words, it is the time it takes for the soil to dry out. The decay coefficient (K) and drying time (T_w) are especially important for long continuous simulations, since a successive set of storms are applied. For this reason, K and T_w were included in the MCDA (Multi Criteria Decision Analysis) described with more detail in Chapter 4. For a region, those values may have similar values. However, might differ slightly from one watershed to another one inside of the same region. These differences are due to special local conditions, such as the degree of urbanization and ground slope.

3. Imperviousness map

Impervious surface maps come in a GIS raster format; which allows an assessment of the percentage of impervious cover in a watershed. Since imperviousness changes over time, the most recent maps are preferred. In any case, they should be corrected to take into account only the Directly Connected Impervious Area (DCIA) through Equation (2.1), this is especially necessary in urban areas. DCIA is important to quantify accurately in modeling because it affects not only the large and medium discharge runoff events but also the frequent 2-year or more frequent events that have been shown to produce 90% of the total runoff (Roesner et al., 2001).

If the gross impervious area is taken to be zero (TIA=0), the original land surface is assumed and any kind of human development is ignored. Raster GIS imperviousness maps were obtained with a resolution of 30 x 30 m. To obtain the overall imperviousness of a particular sub-watershed, a weighted average from the individual values of each raster is obtained, that is:

$$Imperviousness = \frac{\sum_{i=1}^{n}(Value_i \cdot Count_i)}{\sum_{i=1}^{n}(Value_i)} \quad \dots (3.10)$$

4. Additional parameters

Depression Storage (Initial Storage)

The depression storage represents all losses before runoff begins, and includes water retained in surface depressions and water taken up by vegetation interception. In urban watersheds, any rainfall less than about 0.05 inches will not produce runoff due to depression storage.

Manning's roughness

Watershed models have three types of roughness parameters:

1) Roughness of the pervious ground surface. Typical values range from 0.26 (dense grass) to 0.40 (light underbrush).

2) Roughness of the impervious ground surface. For example 0.015 for smooth asphalt.

3) Roughness of the channels and/or conduit links.

Channel roughness changes globally from 0.025 to 0.150, the range of natural channel roughness for rural watersheds from clean, straight reaches on plains to very weedy reaches with heavy underbrush (Chow, 1988). Increases in channel roughness slow the discharge hydrograph, spread out its base and increase lag time at the outlet. Ground-surface and channel/conduit roughness affects runoff flow velocities and rate of discharge (Nix, 1994). Channel roughness was determined from field-estimated values and conduit roughness from conduit materials.

Aerial images

Even though aerial images do not provide a direct input to the watershed models, they do help in the visual recognition of features in the watershed, such as ponds and commercial/residential/industrial zones. Aerial images may be downloaded through the USGS website (http://seamless.usgs.gov/). Aerial images are especially useful to estimate runoff lengths (Rivas, 2007). Figure (3.8) shows an aerial image in Pigeon, where main drainage streams are shown.

Figure 3.8 Aerial image in Pigeon basin

Figure (3.9) shows an aerial image in SW Prong and also main streams are shown.

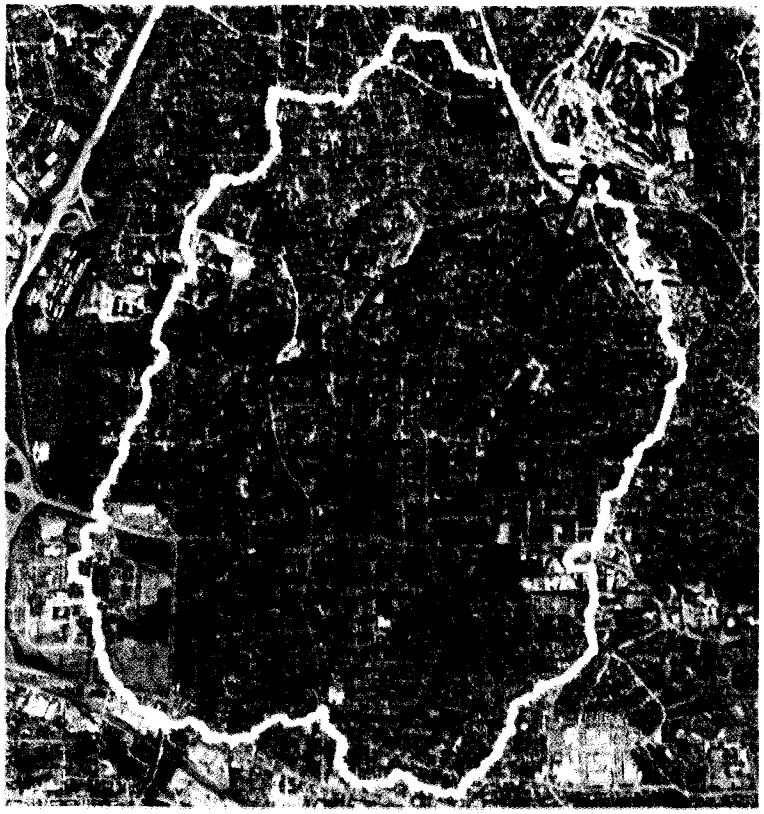


Figure 3.9 Aerial image in SW Prong basin

3.6 Groundwater simulation

Interflow is the residual groundwater flow that occurs after each storm event. The interflow is not as deep as the baseflow of a watershed and it feeds into stream channels at a slower rate than the flow of surface runoff produced by the same event. In both basins the baseflow was estimated on a monthly basis and removed from the USGS monitored record (Figure 3.10). Then, groundwater component in SWMM5 was added to meet the medium and small events discharges due to the presence of interflow in the USGS discharge record.

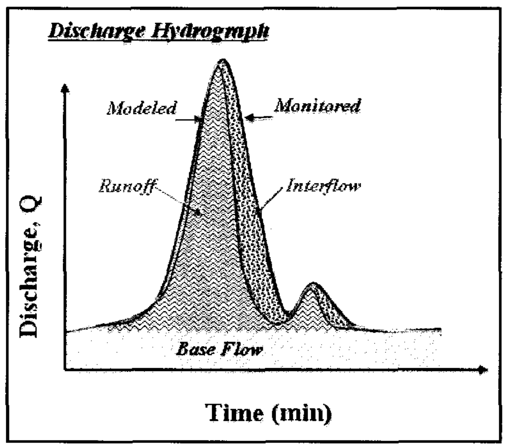


Figure 3.10 Runoff, base flow, and interflow volumes

While baseflow can be estimated on a monthly basis and subtracted from the discharge record, interflow, which occurs on a storm-by-storm basis, cannot be easily be removed from the record and must be simulated to improve modeling results.

A single aquifer was assumed in both watersheds and the following assumptions were made:

i) The ground surface elevation was assumed as the average of all junctions in the drainage network.

ii) The initial groundwater elevation (at the beginning of simulation) was assumed at the receiving node's elevation.

iii) The aquifer bottom elevation was assumed to be 34 ft below the receiving nodes in both watersheds. This is an impervious surface containing the aquifer and represents a bedrock layer.

Groundwater was modeled in SWMM5 using a single aquifer for each watershed. The aquifer accepts surface water infiltration and diverts it to the receiving node (Figure 3.11). The receiving node is the last node of the watershed.

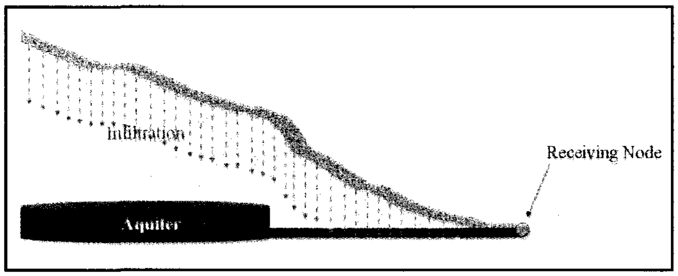


Figure 3.11 Conceptual Groundwater Setup

The groundwater equation used in SWMM5 relates the ground surface elevation of the watershed, the elevation of the groundwater table and a threshold groundwater elevation (Rossman, 2005):

$$Q_{gw} = A_1(H_{gw} - E)^{B_1} - A_2(H_{sw} - E)^{B_2} + A_3\, H_{gw}H_{sw} \qquad \text{... (3.11)}$$

Where:

Q_{gw} : Groundwater flow into or out of the channel (cfs/acre)

A_1 : Groundwater flow coefficient

H_{gw} : Groundwater table elevation (feet)

B_1 : Groundwater flow exponent

A_2 : Surface water flow coefficient

B_2 : Surface water flow exponent

H_{sw} : Elevation of the surface water at the receiving node (feet)

A_3 : Surface-groundwater interaction coefficient

Literature review showed that the coefficient A3 has been assigned a value of zero always (Dent et al., 2004). Thus, the third component was neglected. Figure (3.12) shows the physical meaning of variables in Equation (3.11):

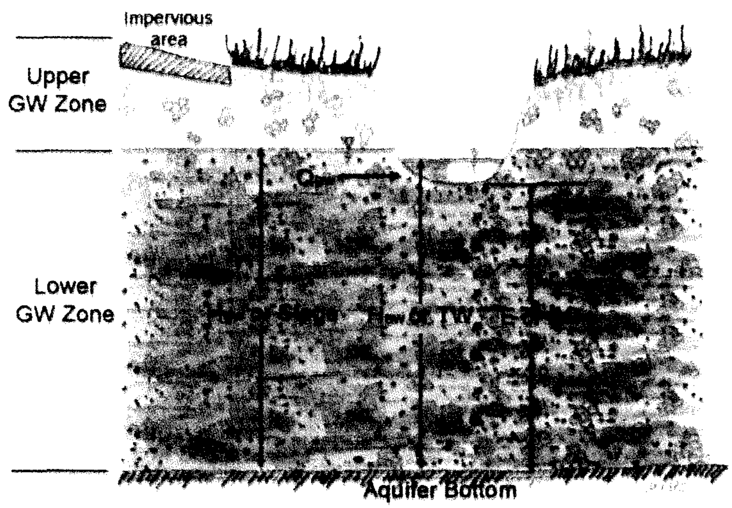


Figure 3.12 Groundwater Equation Parameters (From Romero-Davis, 2008)

As mentioned earlier, initial groundwater elevation was set equal to the receiving's node elevation. Then, during the modeling, the possible scenarios are described next:

a) First term: Groundwater component

$H_{gw} > E \rightarrow$ Groundwater flows from the aquifer into the channel.

$H_{gw} < E \rightarrow$ In theory, groundwater flows from the channel into the aquifer, however, SWMM5 is not capable of adding water to the aquifer under this condition (Huber and Dickinson, 2000). Thus, the first term vanish under this condition.

b) Second term: Surface water component

$H_{sw} > E \rightarrow$ Flow from the channel to the aquifer is present (percolation)

$H_{sw} < E \rightarrow$ Flow into the channel from the aquifer is not affected.

According to the analysis of 534 wells in Orange and Wake counties in North Carolina, the depth of the water table below the ground surface ranged between 88 to 0 ft with an average of 26.6 ft (Cunningham and Daniel, 2001). According to the USGS, groundwater in the Piedmont region is likely to flow near the surface as interflow. Because of this, groundwater component was added to the models to account this phenomenon and to properly model the low flows.

Research of the hydrogeology of the North Carolina Piedmont region revealed both basins are within the Piedmont and Blue Ridge Principle aquifer, a crystalline-rock aquifer of the North Carolina region. This aquifer is composed of nearly impermeable bedrock partially covered by glacial deposits of sand and gravel called regolith; the water yielded from this aquifer is due mainly to secondary porosity and permeability created by the fractures in the bedrock (Romero-Davis, 2008).

The regolith layer of this aquifer averages between 32.8 to 65.6 ft in thickness and may be as much as 328 ft thick in some regions (Romero-Davis, 2008). The porosities for regolith are 20 to 30% (Pettyjohn, 1987). The hydraulic conductivities of regolith vary over a large range from 0.20 in/h to 13.4 in/h with an average of 2.21 in/h (Heath, 1994).

3.7 Rainfall data (NCDC, 2008) and climate

Precipitation data constitutes the main input to the model. It comes in a spatial distribution of precipitation over time. Rainfall data should be as close as possible to the watershed being modeled. In some regions, spatial variability plays an important role.

However, the use of radar-generated rainfall data overestimates runoff in some cases (Urbonas, 2007). USGS rain gage (Site No. 0208732885) located at Marsh Creek, New Hope, NC provided 15 min rainfall data. Refer to Table (3.7) for a complete description.

Table 3.7 Rain gage data

Data	Value
ID	317079
In service	31 May 1954 to Present
Elevation	121.9 m (400') above s/l
Lat/Lon	35°48'N / 78°42'W
County	Wake

Rainfall is well distributed throughout the year as a whole. July and August have the greatest amount of rainfall, and October and November the least. There are times in Spring and Summer when soil moisture is scarce. Most summer rain is produced by thunderstorms, which may occasionally be accompanied by strong winds, intense rains, and hail. The Raleigh-Durham area is far enough from the coast so that the effects of coastal storms are reduced.

While snow and sleet usually occur each year, excessive accumulations of snow are rare. The weather station is featured as an Automated Surface Observation System (ASOS), it forms part of the National Weather Service (NWS) network (NCDC, 2008). Table (3.8) shows the monthly average and total annual precipitation of the rain gage. During the simulation period, the annual total precipitation during 2002 and 2003, were 42.4 inches and 45.1 inches respectively.

Table 3.8 Average monthly precipitation

Month	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Annual
P (in)	2.92	2.65	4.27	3.10	3.05	3.73	4.83	5.26	4.46	3.47	3.02	2.26	43.02

In terms of climate, the Raleigh-Durham area is located in the zone of transition between the Coastal Plain and the Piedmont Plateau. The surrounding terrain is rolling, with an average elevation of around 400 feet, the range over a 10-mile radius is roughly between 200 and 550 feet. Being centrally located between the mountains on the west and the coast on the south and east, the Raleigh-Durham area enjoys a favorable climate. The mountains form a partial barrier to cold air masses moving eastward from the interior of the nation. As a result, there are few days in the heart of the winter season when the temperature falls below 20 degrees.

Tropical air is present over the eastern and central sections of North Carolina during much of the summer season, bringing warm temperatures and rather high humidity to the Raleigh-Durham area. Afternoon temperatures reach 90 degrees or higher on about one-fourth of the days in the middle of summer, but reach 100 degrees less than once per year. Even in the hottest weather, early morning temperatures almost always drop into the lower 70s (NCDC, 2008).

3.8 Ecoregions

Ecoregions have been designated by the U.S. Environmental Protection Agency (EPA) and the Commission for Environmental Cooperation (CEC). An ecoregion may be identified by similarities in geology, physiography, vegetation, climate, soils, land use, wildlife distributions, and hydrology (CEC, 1997). The CEC was established in 1994 by the member states of Canada, Mexico, and the United States to address regional environmental concerns under the North American Agreement on Environmental Cooperation (NAAEC), the environmental side accord to the North American Free Trade Agreement (NAFTA).

There are four levels in the ecosystem hierarchy:

i) Level I divides North America into 15 broad ecoregions appropriate for analysis at a global or intercontinental scale.

ii) Level II divides the Level I into 52 smaller ecoregions, this level is useful for national and subcontinental scales.

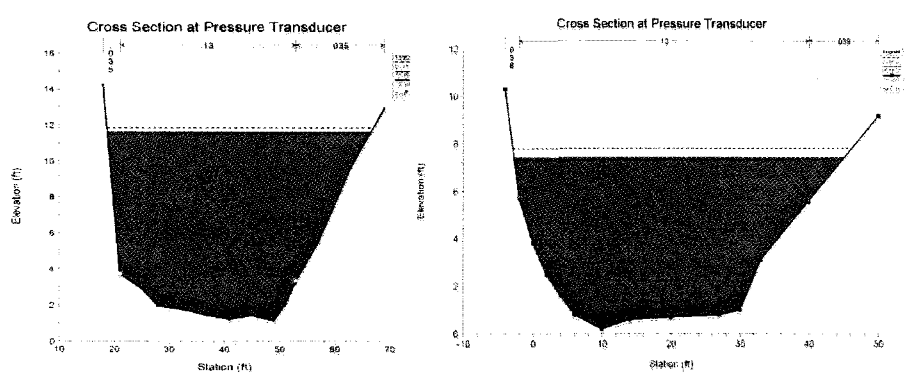
iii) Level III represents a further subdivision, with 194 ecoregions to describe North America, of which 104 apply to the continental United States; this level is appropriate for regional analysis and decision-making.

iv) Work to define Level IV ecoregions, a scale that provides useful information for local analysis, is currently underway or complete for most of the US.

Both watersheds are located in the same Level III EPA Piedmont ecoregion (USEPA, 2007).

3.9 Runoff data (USGS, 2008)

The stream flow gages were located at the watershed outlets of both basins. Figure (3.13) shows the natural cross section where the data was recorded.



Pigeon **SW Prong**

Figure 3.13 HEC-RAS cross sections at watershed outlets

Stream depth data was collected by the USGS. Water level data was collected at 15-minute increments for the monitoring period. The water level data were converted to discharge values by USGS staff using stage-discharge curves established using HEC-RAS models (Pomeroy, 2007). Streamflow records identified in Table (3.9) provided data for calibration.

Table 3.9 General data for stream flow gages

Name	Pigeon Basin	SW Prong
USGS Site Number:	0208732610	02087304
Geographic location (NAD83)	Latitude 35°48'25" (N) Longitude 78°36'50" (W)	Latitude 35°49'04" (N) Longitude 78°39'35" (W)
Hydrologic Unit:	03020201	
Drainage area (mi^2):	4.45	3.02
Range of 15-min data	July 4, 2002 October 28, 2003	July 4, 2002 July 28, 2003
Number of years	1.32	1.07
Datum of gage, NGVD29 (Feet above sea level)	200	240

3.10 Model calibration

Traditionally, stormwater model calibration is performed on six or more individual storms of varying size (Akan and Houghtalen, 2003); it is considered complete for the single storm event when the peak discharge and volume of runoff are accurately reflected by the model.

In general, calibration can be done at a number of temporal scales (Pomeroy, 2007):

i) Single event: one storm.

ii) Multiple events: several storms.

iii) Continuous simulation: long records (months to several years).

Calibration to a single storm is not appropriate for continuous simulation modeling because adjustments of variables to match runoff from one event may over or under adjust variables and inhibit matching of other events. Pomeroy (2007) pointed out that it is important that rainfall-runoff models to be able to accurately simulate the full spectrum of flows to evaluate biologic integrity in streams. Therefore, the models were calibrated across the full spectrum of flows during the 2002-2003 [temporal scale ii)] period of level-flow monitoring by the USGS personnel, where model outputs and observed flows were compared.

Figure (3.14) shows an overview of the calibration processes.

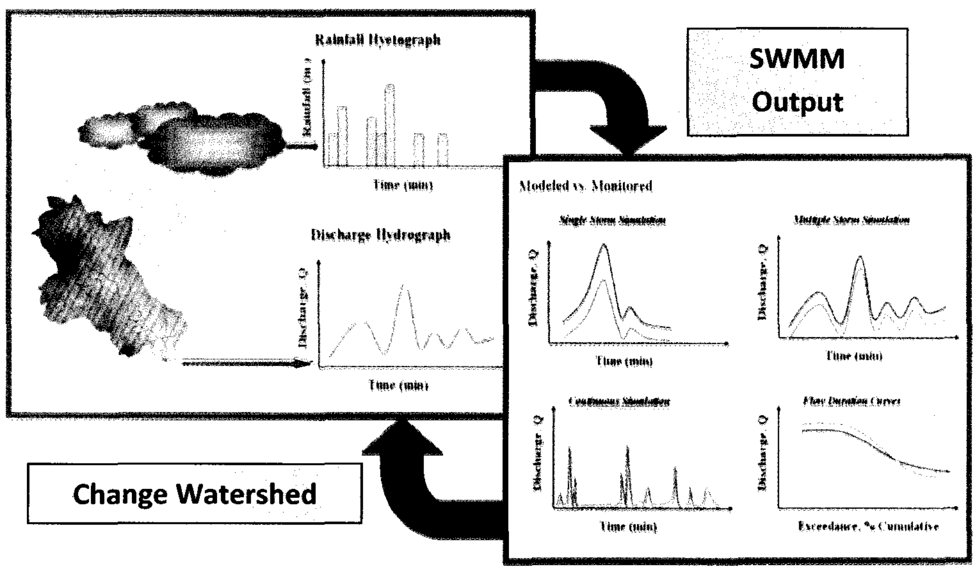


Figure 3.14 Overview of the calibration processes and temporal scales (Romero-Davis, 2008)

Large events for flow duration are those equal to or less frequent than 0.2% of time exceeded. Medium events are those falling between 0.2 to 10% of time exceeded and small events are those less than 10% of time exceeded. Small storms are generated mainly by DCIA, however, peak floods also play an important role in urban watershed modeling. Hence, a unique feature of this dissertation is to consider all types of events in the calibration procedure.

The flow duration curve is a graph of all the discharges during a continuous record, and their cumulative exceedences, or the percent of time each discharge occurs during the period of record. These curves were developed from the partial duration series of peak flows. This approach, which is in contrast to the examination of the annual maximum series, was used because it allowed for the analysis of high frequency, low runoff producing storms.

Calibration of the partial duration series requires definition of an inter-event time, and the minimum threshold to define an event. A six-hour inter-event time was specified to separate the flow data into individual events based on recommendations by Heineman (2007). A sensitivity analysis showed threshold values of 3.0 cfs for Pigeon and SW Prong basins (Pomeroy, 2007). Final event threshold event parameters were selected to identify the greatest number of events during the period of USGS flow monitoring.

The percentage of time for each flow magnitude is equaled or exceeded can be computed by arranging the flow rates in order of descending magnitude (Mays, 2001). The return period (T_r) is computed for each event using the Cunnane Method:

$$T_r = \frac{\mathrm{n+1-2\cdot a}}{m-a} \qquad \ldots (3.12)$$

Where,

n = length of record, years

a = plotting position, usually taken as 0.40

m = rank, 1 for the highest

The return interval was computed to Exceedences per year (E) using $1/T_r$. Then, this percentage of time of exceedance is plotted against the flow magnitude. Let Q(A) be a random variable denoting the annual peak floods from a watershed of drainage area A. Then, the p*th* quantile $Q_p(A)$ is defined as:

$$P\left(Q(A) > Q_p(A)\right) = p \qquad \text{... (3.13)}$$

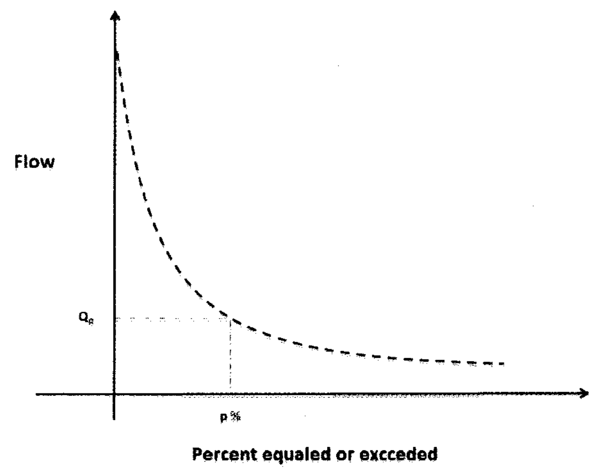


Figure 3.15 Flow duration curve

3.11 Advantages of continuous calibration

Continuous calibration involves calibrating to a long duration of multiple events ranging from months to years. The main advantage of continuous calibration is that it makes maximum use of available data over a variable spectrum of hydrologic-hydraulic events (Walsh et al., 1989).

Continuous calibration eliminates the need to select specific storms with various antecedent conditions because all or a large portion of the events of the calibration record are being simulated. Continuous calibration eliminates the time required to select discrete events to calibrate and ensures that a wide range of conditions are assessed in a shorter period of calibration time.

Continuous simulation allows modeling the complex interactions between the precipitation patterns. Return periods for storms can be defined on the basis of the simulated record; critical events chosen for study may be substituted for synthetic design storms.

Finally, SWMM simulation errors were verified after simulations. Mass continuity simulation errors for runoff and flow routing represent the percent difference between initial storage + total inflow and final storage + total outflow for the entire drainage system. All simulation errors were verified to be less than 1%. The most common reasons for an excessive continuity error are computational time steps that are too long or conduits that are too short.

4.0 MODEL OPTIMIZATION

4.1 Multi-Criteria Decision Analysis (MCDA) approach

Model variables need to be adjusted to generate a modeled behavior that best matches the monitored behavior of a site over a variety of runoff events. In this case, the following process was used to adjust SWMM5 parameters (such as infiltration parameters) to obtain modeled results that match or approximate USGS discharge records.

Model calibration using a multiobjective approach, can be performed on the basis of multi-variable measurements (i.e. stage and flow), multi-site measurements (i.e. several flow measurements in a basin), and/or multi-response modes (i.e. peak versus low flows), as described by Madsen and Jacobsen (2001).

The solution in a multi-objective analysis will not be a single unique set of parameters, but will consist of a *Pareto front of solutions*. Choosing a single point along this front is commonly referred to as *Pareto optimal* or *non-dominant solution*. Goal programming has most commonly been applied to linear problems, but it can be used for nonlinear and dynamic problems (Ko et al., 1992).

As explained in Chapter 3, instead of using single storms for calibration and validation, the idea is to use the flow-duration curve. Since calibration parameters are in conflict to estimate peak and low flows, the proposed approach for calibration involves a trade-off between storm sizes. In this sense a MCDA methodology was used.

Roesner et al. (1991) examinated six U.S. cities in areas with different climatic conditions. By using long term rainfall records, Roesner et al. (1991) found that most rainfall occurs during small storms. For most cities, this represented about 90% of the runoff. In addition, Nehrke and Roesner (2004) noticed that in most cases, the flow controls for BMP's targets low frequency events, storms equal to or larger than the 1-Year event ($Q_{1\text{-}Yr}$). Since at this value there is a change in flow regime (Nehrke and Roesner, 2004), the criteria to divide the curve was the flow corresponding to the $Q_{1\text{-}Yr}$.

Long term simulations (20 Yr) were performed in SWMM5 to estimate the $Q_{1\text{-}Yr}$. The corresponding rain gage was the USGS Site No: 0208732885 at Marsh Creek near New Hope, NC. The simulation period was from 1986 to 2005. The rain gage had an annual average rainfall depth of 43.02 in during that period. Simulation results are shown in Table (4.1).

Table 4.1 $Q_{1\text{-}Yr}$ in both basins

Basin	$Q_{1\text{-}Yr}$ (cfs)
Pigeon	645.4
SW Prong	270.7

The flow-duration curve was divided in two sectors: High and Low Flows where the threshold value to split the curve in two parts was the $Q_{1\text{-}Yr}$. The optimization algorithm attempted to minimize the Root Mean Square Error (RMSE, Equation 2.2) between the simulated flows (S_i) and the observed flows (O_i) in both sectors of the flow duration curve.

$$min \sum_{i=1}^{n} \begin{cases} RMSE \text{ High Flows} \quad For \quad S_i > Q_{1-Yr} \\ RMSE \text{ Low Flows} \quad For \quad S_i \leq Q_{1-Yr} \end{cases} \quad \ldots (4.1)$$

Model calibration followed the Ordered Physics-Based Parameter Adjustment (OPPA) method after Vieux and Moreda (2003). Measurement and modeling introduce errors and there is no objective way a unique model solution can be obtained (i.e. there is no "best" solution or a non-dominated solution).

Even in the case where a significant amount of data exists, modeled estimates will not produce one set of calibration parameters that are best for all conditions (Dent et al., 2004). This situation is especially true when high and low flow estimates are compared (Gupta et al., 1988).

Finding the best parameters to represent the watershed was done by *constrained optimization.* That is, setting constraints on the search by establishing a range of feasible parameters. Values outside of this range are not acceptable. Table (4.2) shows all needed input data divided by type.

Table 4.2 Watershed model input data

WATERSHED MODEL INPUT DATA		
Sub-catchment Characteristics	**Channel Conveyance**	**Calibration Data**
Drainage area	Cross sections	Rainfall depth
Width	Bottom slope	Streamflow records
Overland ground slope	Length	
Overland flow slope	Roughness	
Soil Infiltration rates		
DCIA coefficients		
Depresion storage		
Overland flow roughness		
Groundwater parameters		

During the optimization process, hard constraints cannot be violated and their values are limited by the use of traditional range of values for: Manning's roughness, depression storage and infiltration parameters.

Also variables may be classified in four groups as showed in Table (4.3).

Table 4.3 Type of variables

Variable Type	Features	Examples
i) State	Fixed value	Watershed area River cross sections DEM Conduit sizes Rainfall data
ii) Driven	Values obtained through equations	Runoff coefficient
iii) Derived	Obtained from state variables or field data	Ground slope Infiltration parameters
iv) Calibration	Require adjustment	DCIA coefficients Width, Tw, K

4.2 Mathematical formulation

During a traditional mono-criterion problem (minimization), the objective is to find a minimum global optimal (X*):

Find:

$$X^* = \{X_1, X_2, \dots, X_n\} \qquad \dots (4.2)$$

Such that:

$$f(X^*) \leq f(X) \ \forall \ X \in A \qquad \dots (4.2a)$$

However, in a Multi-criteria approach, the objective function to be minimized is represented by an optimization vector:

$$f(X) = [f_1(X), f_2(X), f_3(X), \dots, f_k(X)] \qquad \dots (4.3)$$

Considering a Multi-Criteria Optimization Approach model having the p-dimensional parameter vector (θ) represented as:

$$\theta = (\theta_1, \theta_2, \theta_3, \dots, \theta_n) \qquad \dots (4.4)$$

Which is to be calibrated using time series observations:

$$O_j(t_j),\ t_j = ta_j, \dots, tb_j, \qquad j = 1, 2, \dots, k \qquad \dots (4.5)$$

Which are collected from times ta_j through tb_j on k different response variables. The different responses represent the different model outputs (i.e., runoff). In this problem, two sets of system variables are identified: Simulated runoff (S_i) and Observed runoff (O_i) recorded by a USGS streamflow gage. To measure the distance between (S_i) and (O_i) a separate criteria $f_i(\theta)$ was defined. In this sense, it is a common practice to use a measure of residual variance, thus, the Root Mean Square Error (RMSE) was used (Equation 2.2).

That being said, the multi-criteria model calibration problem can then be formally stated as the optimization problem:

$$\text{Minimize } F(\theta)=\{f_1(\theta), f_2(\theta),\ldots,f_k(\theta)\} \qquad \ldots (4.6)$$

Where the goal is to find the values for (θ) within the feasible set (Q) that simultaneously minimizes all of the k criteria. The two criteria response functions are $RMSE_{\text{High Flows}}$ and $RMSE_{\text{Low Flows}}$ that characterize f_1 and f_2.

During continuous simulation, infiltration capacity is regenerated during dry periods. Technically, the drying time (T_w) is a hypothetical projected time at which f_p (infiltration rate) and f_c (final infiltration) become equal (Equation 3.9).

In this sense, the drying time (T_w, days) and Horton's decay coefficient (K, 1/h) were chosen as decision variables because these two parameters represent the highest sensitivity (Pomeroy, 2007) and there is no specific guidance in the literature for values to use. In addition, since the methodology was focused on continuous simulation, both parameters are important parameters to establish. These two variables represent the dimensional parameter vector (θ).

Nevertheless, the multiobjective minimization problem does not have a unique solution. In other words, it is not possible to find a single point θ at which all the criteria have their minima. It is common to have a set of solutions with the property that moving from one solution to another result in the improvement of one criterion while causing deterioration in another. The sub-catchment widths obtained with the kinematic wave conversion described in Chapter 3 were used to build the rainfall-runoff models. Additional adjustments were done to improve their performance with the MCDA methodology proposed herein. The values of all hydrologic/hydraulic parameters and model options are shown in the Appendix A.

4.3 Optimization results for Pigeon basin

Next Figure 4.1 shows the parameter space for the two selected variables and the point that minimizes the RMSE along the entire FDC. It was found that Tw=4.50 days and K=3.13 1/h minimized the difference between modeled and observed flows.

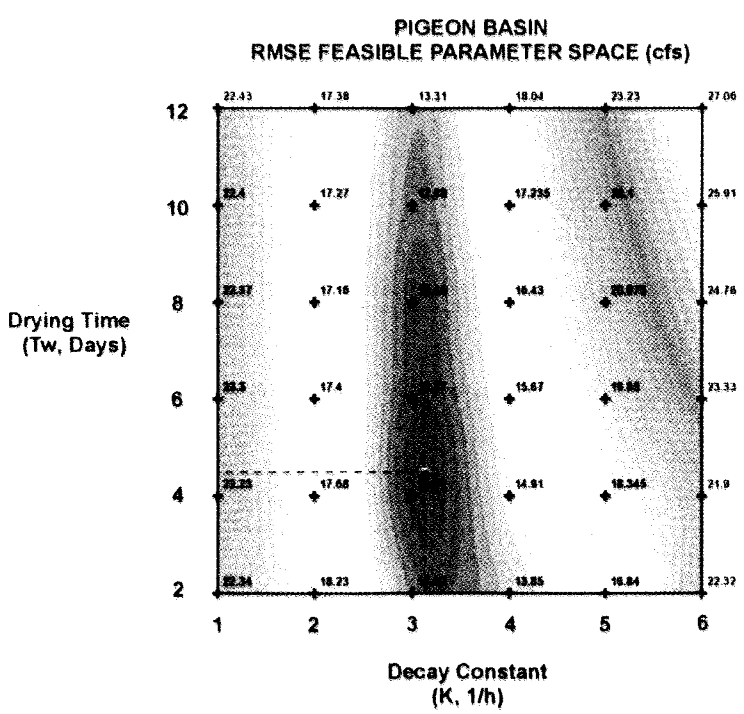


Figure 4.1 Feasible parameter space and RMSE (cfs) in Pigeon (<u>Total FDC</u>)

The correspnding FDC of the model output and observed flows is shown in Figure (4.2).

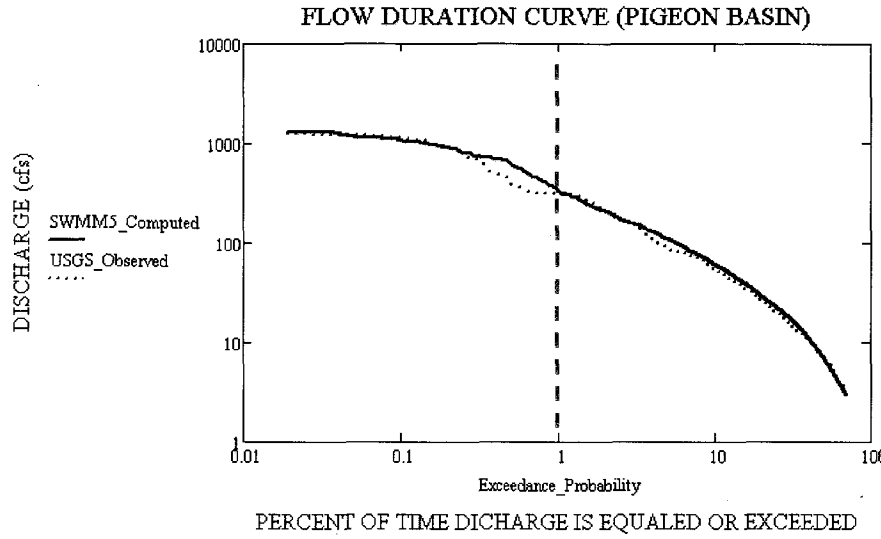


Figure 4.2 Flow Duration Curve (Pigeon basin)

Next, the effect of K and Tw along the Flow Duration Curve was further investigated. For the High Flows, K=3.87 1/h and Tw=8.33 days were found to minimize the RMSE in this portion of the curve as it is shown in Figure (4.3):

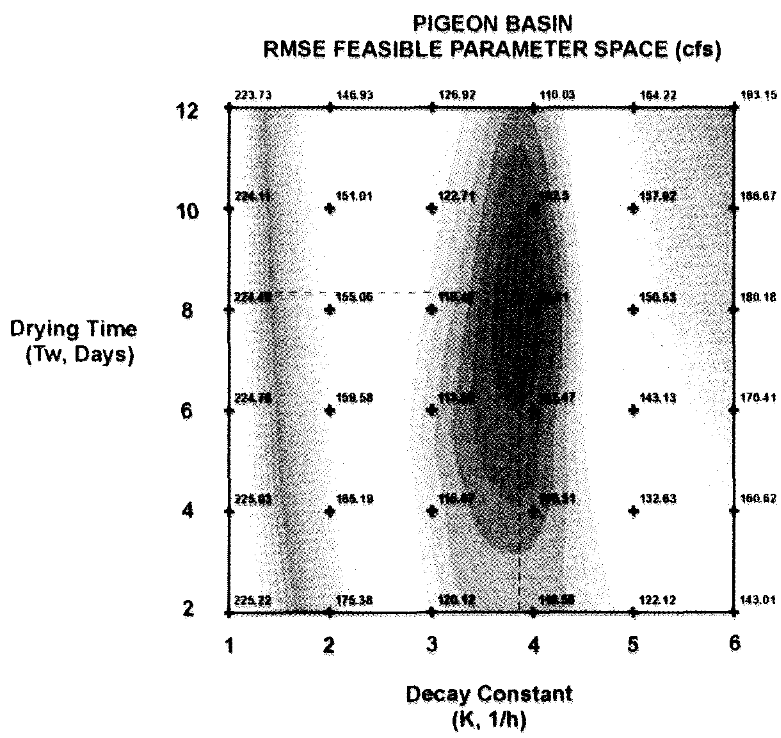


Figure 4.3 Feasible parameter space and RMSE (cfs) in Pigeon (<u>High Flows</u>)

However, for the Low Flows portion of the FDC, the optimized parameters were K=3.08 1/h and Tw=4.49 days (Figure 4.4).

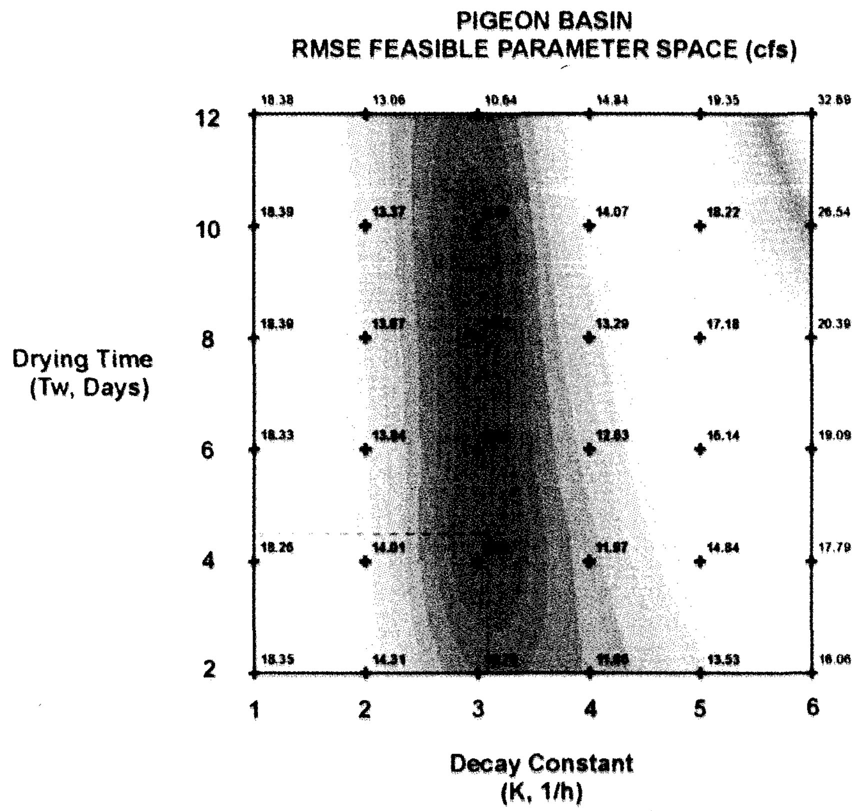


Figure 4.4 Feasible parameter space and RMSE (cfs) in Pigeon (<u>Low Flows</u>)

Next table shows a sumary of the results for Pigeon.

Table 4.4 Optimized parameters in Pigeon and RMSE (cfs)

Portion of the FDC	Tw (days)	K (1/h)	RMSE (cfs)
Entire	4.50	3.13	11.69
High Flows	8.33	3.87	96.93
Low Flows	4.49	3.08	9.47

From Table (4.4) it is obeserved that a proper modeling of high flows requires the use of longer drying times (Tw) and larger decay coefficients (K). Not much difference was observed between the entire FDC and the low flows, but one main conclusion is to have shorter drying times (Tw) and slighty smaller decay coefficients (K).

Next, the *pareto optimal front* was obtained. In this sense, the points that minimized the High/Low flow portions represent the tails of the *pareto optimal front*. This is shown schematically on Figure (4.5) where point A represents a better solution than point B in terms of minimizing objective f2. However, point B would be a better solution than point A for minizing objective f1. The red line connecting these points is called the pareto optimal front and the rest of the points are known as dominated solutions (for example, point C).

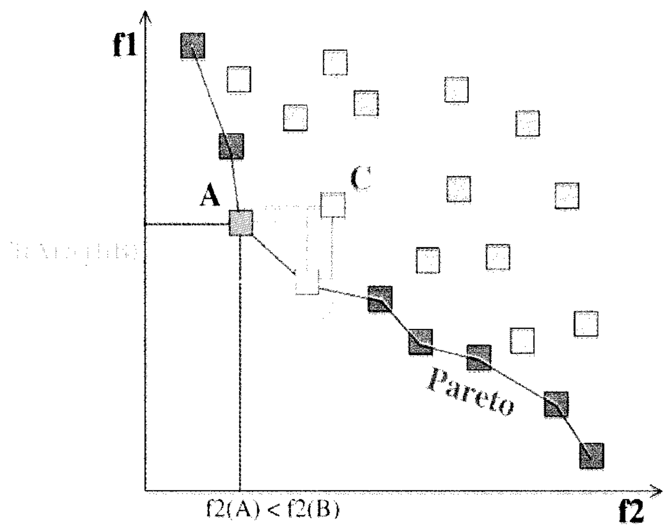


Figure 4.5 Scematic of the pareto optimal front criterion space

Using a population size of 1600 solutions, Figure (4.6) shows the criterion space for the FDC. The black dots are the pareto optimal front and represents the set of non-dominated solutions. This set stands for the best compromise between the two considered objectives. RMSE Low Flows criterion is minimized at the extreme point on the left (9.47 cfs) and RMSE High Flows criterion is minimized at the lowest point (99.93 cfs).

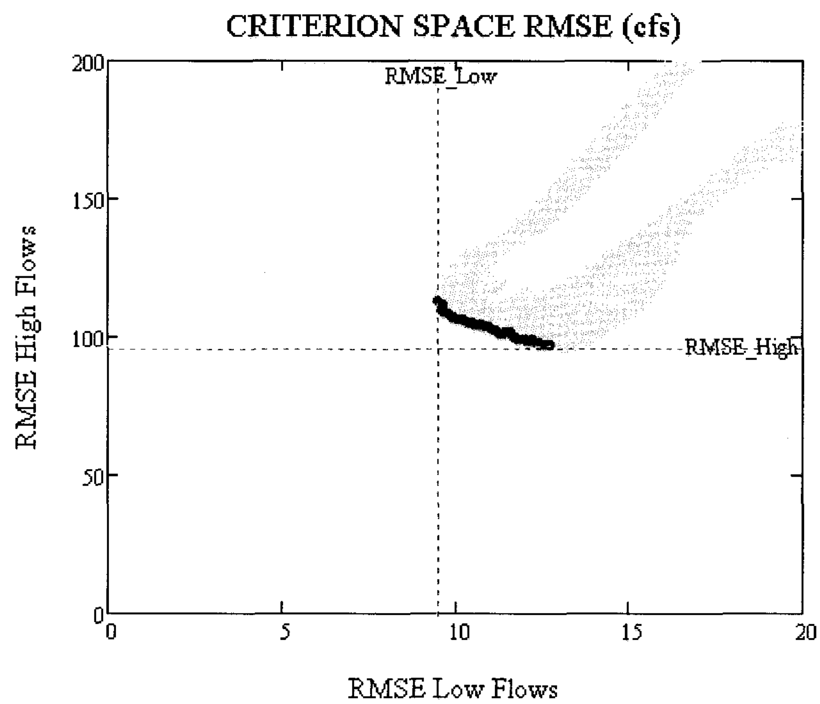


Figure 4.6 Pareto optimal front in Pigeon (cfs)

The pareto optimal front of solutions (set P) is called the trade-off set, non-inferior set, non-dominated set, efficient set or simply Pareto set. Let α to be any solution along the Pareto and δ any solution non-contained in the set P, then every point α is superior to every point δ because:

$$RMSE_{\text{High Flows}}(\alpha) < RMSE_{\text{High Flows}}(\delta) \qquad \ldots (4.7)$$

And:

$$RMSE_{\text{Low Flows}}(\alpha) < RMSE_{\text{Low Flows}}(\delta) \qquad \ldots (4.8)$$

4.4 Optimization results for SW Prong

The MCDA was performed in SW basin, the second case study. Despite of having a lower level of development, similar results were found. The optimal points were K=2.97 1/h and Tw=5.69 days as shown in Figure (4.7). As shown in Figure (4.8), the modeled and observed flows are presented in the flow-duration curve.

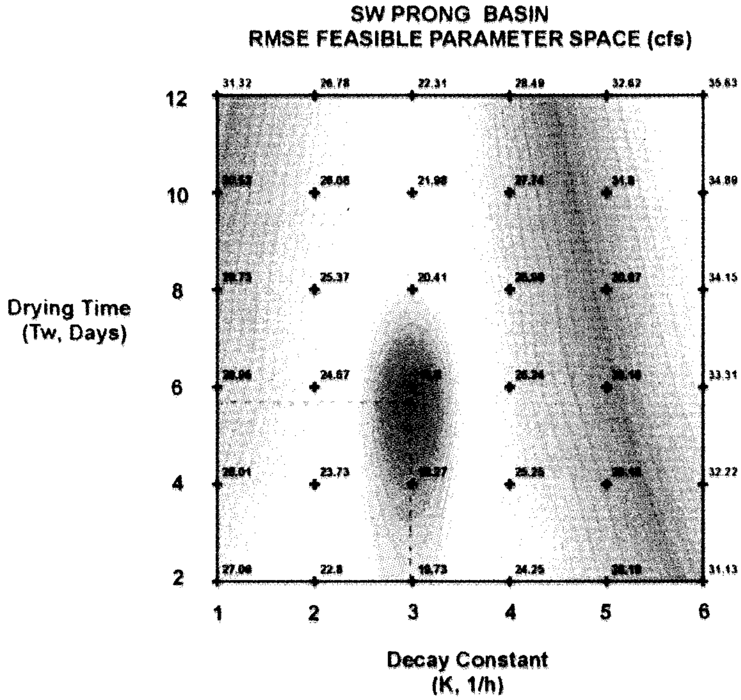


Figure 4.7 Feasible parameter space and RMSE (cfs) in SW Prong (Total FDC)

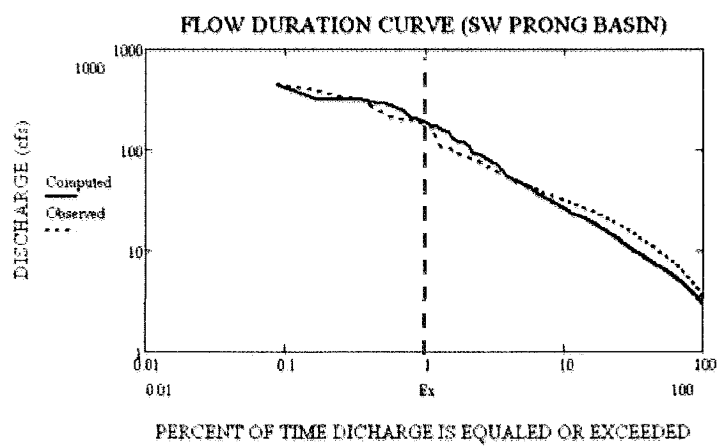


Figure 4.8 Flow Duration Curve (SW Prong basin)

For the High Flows, K=3.94 1/h and Tw=8.35 days were found to minimize the RMSE in this portion of the curve as it is shown in Figure (4.9). For the Low Flows, K=2.97 1/h and Tw=3.84 days minimized the RMSE in this portion of the curve as it is shown in Figure (4.10):

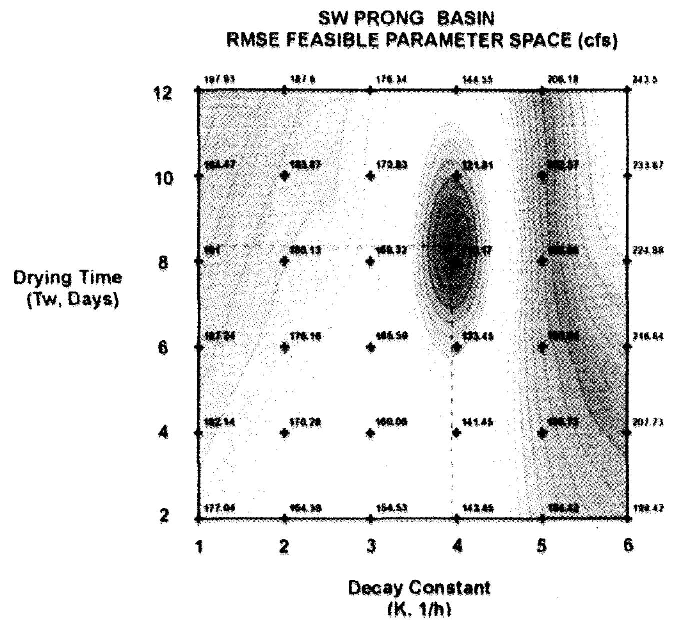


Figure 4.9 Feasible parameter space and RMSE (cfs) in SW Prong (<u>High Flows</u>)

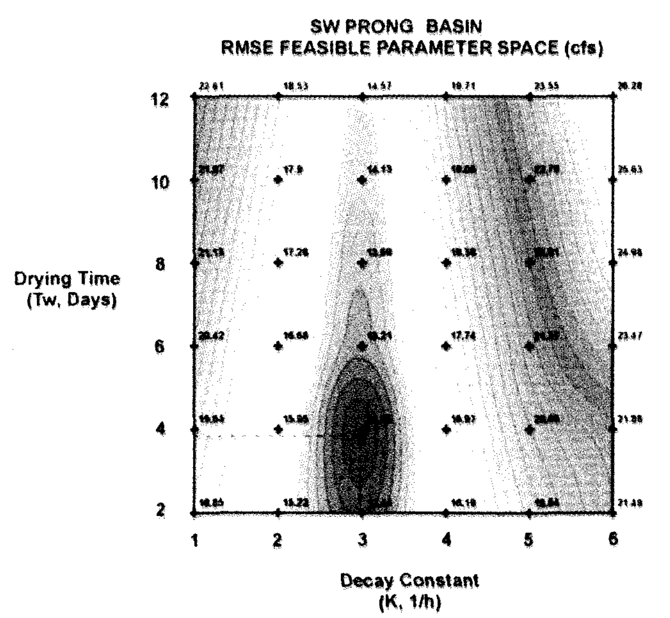


Figure 4.10 Feasible parameter space and RMSE (cfs) in SW Prong (<u>Low Flows</u>)

All results are summarized in Table (4.5):

Table 4.5 Optimized parameters in SW Prong and RMSE (cfs)

Portion of the FDC	Tw (days)	K (1/h)	RMSE (cfs)
Entire	5.69	2.97	16.71
High Flows	8.35	3.94	108.92
Low Flows	3.84	2.97	11.64

Results in SW Prong were consistent with Pigeon. Longer drying times (Tw) and larger decay coefficients (K) were required to minimize the RMSE for the high flows. To conclude the analysis in SW Prong, by using a population size of 1600 solutions the criterion space was obtained and it is shown in Figure (4.11). The black dots are the pareto optimal front and represents the set of non-dominated solutions.

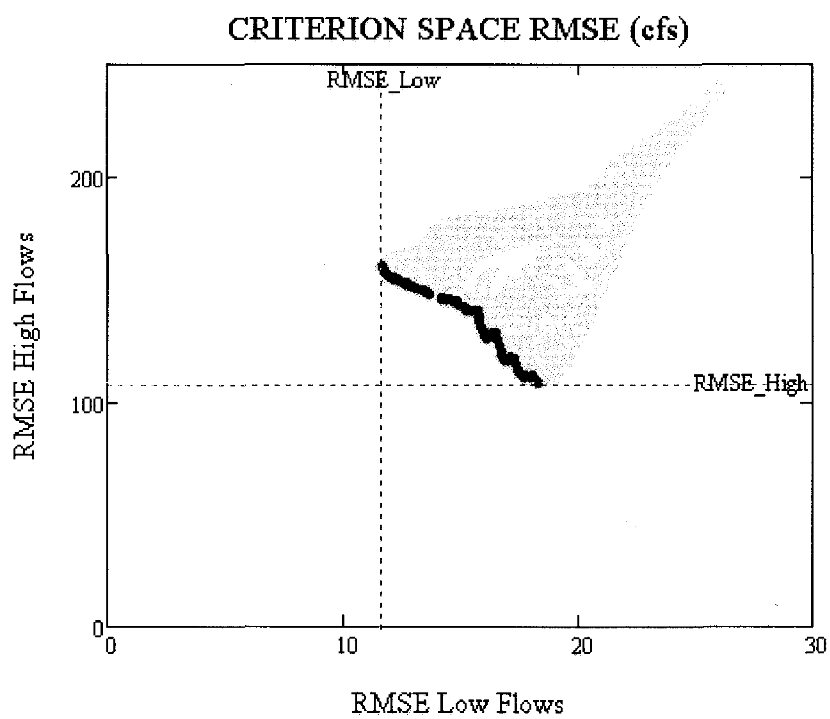


Figure 4.11 Criterion space and Pareto Optimal Front in SW Prong (cfs).

Different applications would require paying more attention to a given sector of the flow duration curve. For example, for floodplain modeling, the High Flow portion is more important. However, for stream ecology applications, when more importance is given to estimate small storms, the Low Flow portion would need more accuracy.

5.0 MODEL DISCRETIZATION

5.1 Distributed modeling issues

The second part of this dissertation involved a discretization analysis of the case studies. Discretization scenarios were obtained with ArcHydro (Maidment, 2002). A GIS toolbox was developed to obtain the needed GIS to build the SWMM5 models. Several long-term rainfall-runoff experiments were performed in each watershed, where a different level of discretization was modeled in each scenario. Then, from the *long term* hydrologic simulations with various watershed configurations, corresponding measurement of performance were computed and compared.

Selecting a higher resolution in distributed hydrological modeling implies a subsequent set of activities: data acquisition, defining the model parameter values, building the model, simulation, calibration and maintenance; the costs of all of these tasks are increased when selecting a higher resolution. On the other hand, selecting a lower resolution greatly reduces the workload, but there is the risk of losing the advantage of the distributed modeling approach, leading to poor results due to lack of consideration of important spatial features. Clearly, there is a tradeoff between both approaches and a balance should exist.

This research work attempted to analyze the spatial heterogeneity and catchment scale in urban hydrology. One of the main questions to be solved is to investigate an appropriate scale in which the hydrologic response varies slowly with increasing sub catchment size. In other words, the objective is to determine a scale above which the spatial variability can be neglected, so at this scale, the sub-catchments still provide sufficient information for reliable modeling of basin runoff. SWMM5 (EPA, 2008) was used as the main modeling tool, since it was developed especially for urban hydrology.

As it was shown in the literature review, a number of attempts have been made to find a proper level of discretization in rainfall-runoff models. Some important contributions have been made, such as the REA concept (Wood, 1988) and IC-ratios (Roshan, 2006). Many valuable ideas have been proposed to attempt to solve scale problems issues, although such difficult and serious scale problems are yet far from any form of solution (Ao et al., 2003)

Nevertheless, not much research has been done in terms of large urban watersheds (larger than 3 mi^2). Most of case studies were done in non-urban areas and the proposed concepts regarding scaling have not been applied in urban hydrology, especially for SWMM applications.

5.2 Modeling experiment set-up

Several models were built in each basin, ranging from a high resolution to the lowest possible resolution (a single watershed). Then, measurements of performance (PMCC, NSC, IOA and RMSE) were computed and compared to identify the existence of a threshold value in which modeling results are acceptable and a finer discretization is no longer needed in practical applications. The initial or base scenario was chosen to have an average sub-basin size of 43 acres in Pigeon and 55 acres in SW Prong. This size represents a reasonable value in which the homogeneity is kept. From this initial level of discretization, successive scenarios representing a coarser resolution with larger average sizes were built, until the last scenario, where the total watershed had no sub-divisions, that is, the entire watershed was lumped as a single unit.

This analysis allowed the determination of a scale-related threshold zone to discretisize urban basins; such threshold zone can be identified where the measurements of performance (Section 2.4) become asymptotic. The measurements of performance are shown in Figure (5.1); ideal conditions are when PMCC, NSC and IOA present a value of 1.00, RMSE has the best condition when is equal to zero. Also through the simulation analysis, the effects of sub-basin scale on peak flowrate and total runoff volume were investigated.

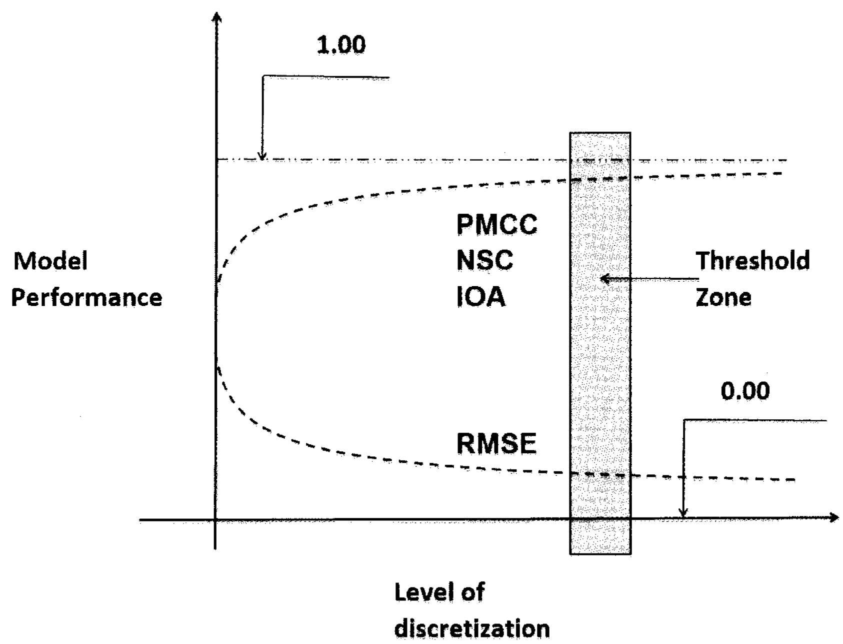


Figure 5.1 Threshold zone to be identified

5.3 Digital Elevation Model (DEM) preparation

This step is critical to set a proper DEM format. For instance, the correct computation of the slope in GIS requires adapting first the raw DEM provided by the USGS website. The data downloaded directly from the website, do not have a projection and the coordinate system is geographic (i.e., latitude and longitude). Once the data is received it needs to be re-projected into UTM zones (NED, 2008) or other plane coordinate system. In other words, the raw DEM's have XY units in decimal degree units and the Z unit is meters.

In order to have a correct computation of the slope, the elevation data must be in the same units of measure as the grid's coordinates. Once the grid is projected, the grid needs to be rescaled using the raster calculator to have the same units as the projected grid. For example, since 1 foot = 0.3048 meter, to change meters to feet, an expression like this should be used using the raster calculator: [DEM] * (1/0.3048).

Then, slope can be calculated directly from the projected and rescaled raster. Another case could be when the Z unit is in feet and the XY units are in meters, in this situation, a factor of 0.3048 must be applied to the raster to convert the Z unit from feet to meters. Then, having all units in meters, slope calculation can be performed as well.

For the case studies, the raw DEM from the USGS website were projected to the following projection: *NAD_1983_StatePlane_North_Carolina_FIPS_3200_Feet*. The rasters now have a XY cellsize of 8.2721 feet. Then, the projected DEM (still with Elevation in meters) was converted to feet using the raster calculator with a factor of (1/0.3048) and the slope was computed in degrees.

Traditional steps in ArcHydro (Maidment, 2002) were followed. The following procedure was adopted to construct basin models in both basins. The first step was preparing a Digital Elevation Model (DEM) (Stefan, 1996). Since downloaded DEMs include pits or ponds that should be removed before being used in hydrological modeling (Ashe, 2003). These are cells where water would accumulate when drainage patterns are being extracted. Pits are a sign of errors in the DEM arising from interpolation. These pits were removed by an algorithm known as sink filling in ArcHydro (Maidment, 2002).

After filling the DEM sinks, a flow direction map was computed by calculating the steepest slope and by encoding into each cell the eight possible flow directions towards the surrounding cells. Flow direction is then used to generate the flow accumulation map. The flow accumulation, generated by addressing each cell of the DEM, counts how many upstream cells contribute to flow through the given cell. Flow direction and accumulation maps are then used to delineate the stream network. The stream network can be divided into segments, which will determine the outlets of the sub-basins.

The last step is the basin delineation process, which depends on the generated flow direction and accumulation map. Furthermore, it also depends on a user-specified number known as threshold (Djokic et al., 1997). This threshold determines the minimum number of pixels within each delineated sub-basin. A value of 5,000 was chosen to delineate the sub-basins in both case studies. The area required to create a stream is computed as:

$$A_{stream} = Threshold\ (Grid\ size)^2 \qquad \dots (5.1)$$

Resulting ArcHydro sub-basins were successively aggregated in larger sizes until obtain a single unit for the entire watershed (Scenario 1 in both basins). Scenario 1 represents a model with a single unit, in other words, without any discretization.

Figure (5.2) shows the discretization scenarios in Pigeon.

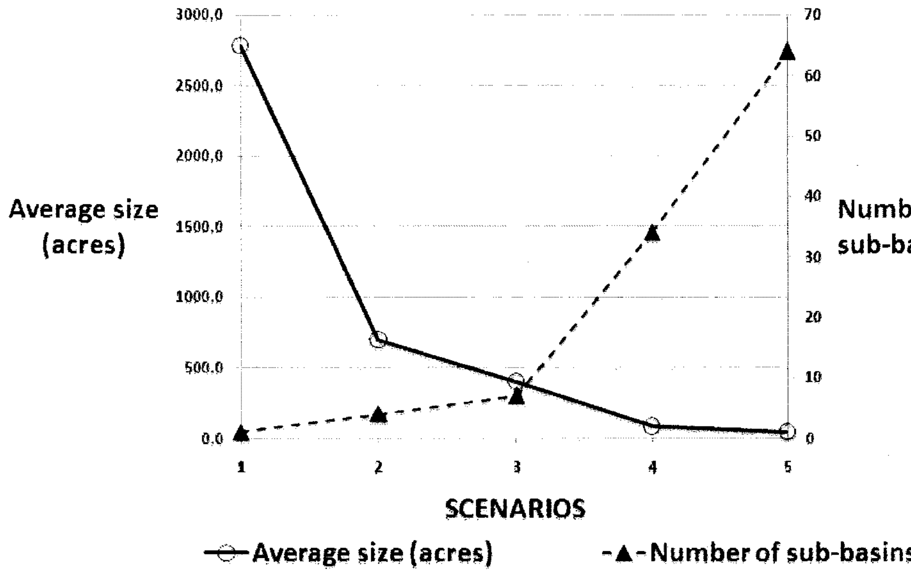


Figure 5.2. Discretization scenarios for Pigeon Basin

Similarly Figure (5.3) shows the discretization scenarios in SW Prong.

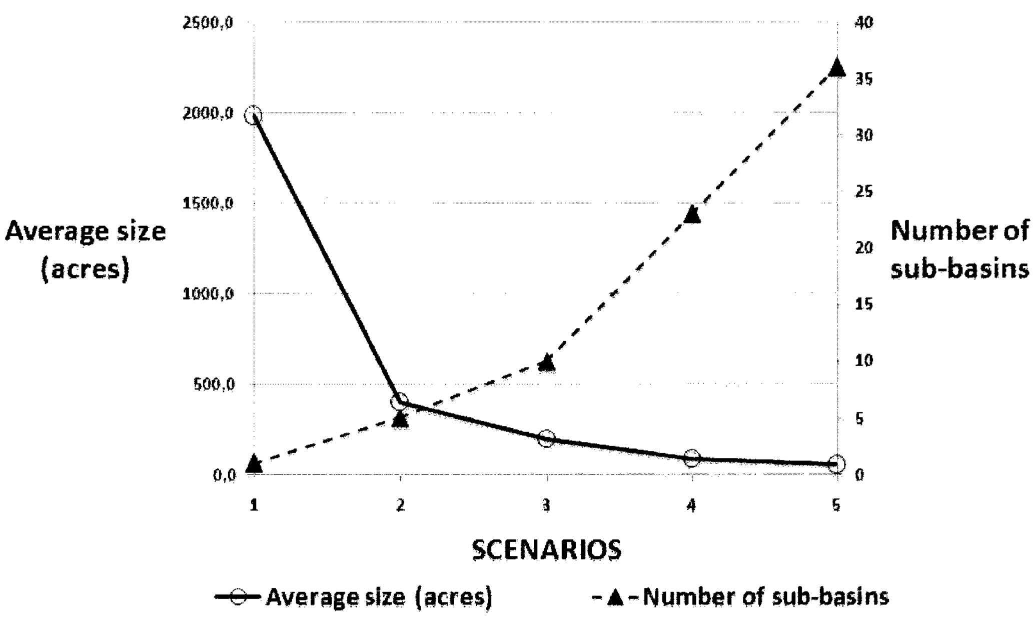


Figure 5.3 Discretization scenarios for SW Basin

Figure (5.4) shows the discretization scenarios maps for Pigeon

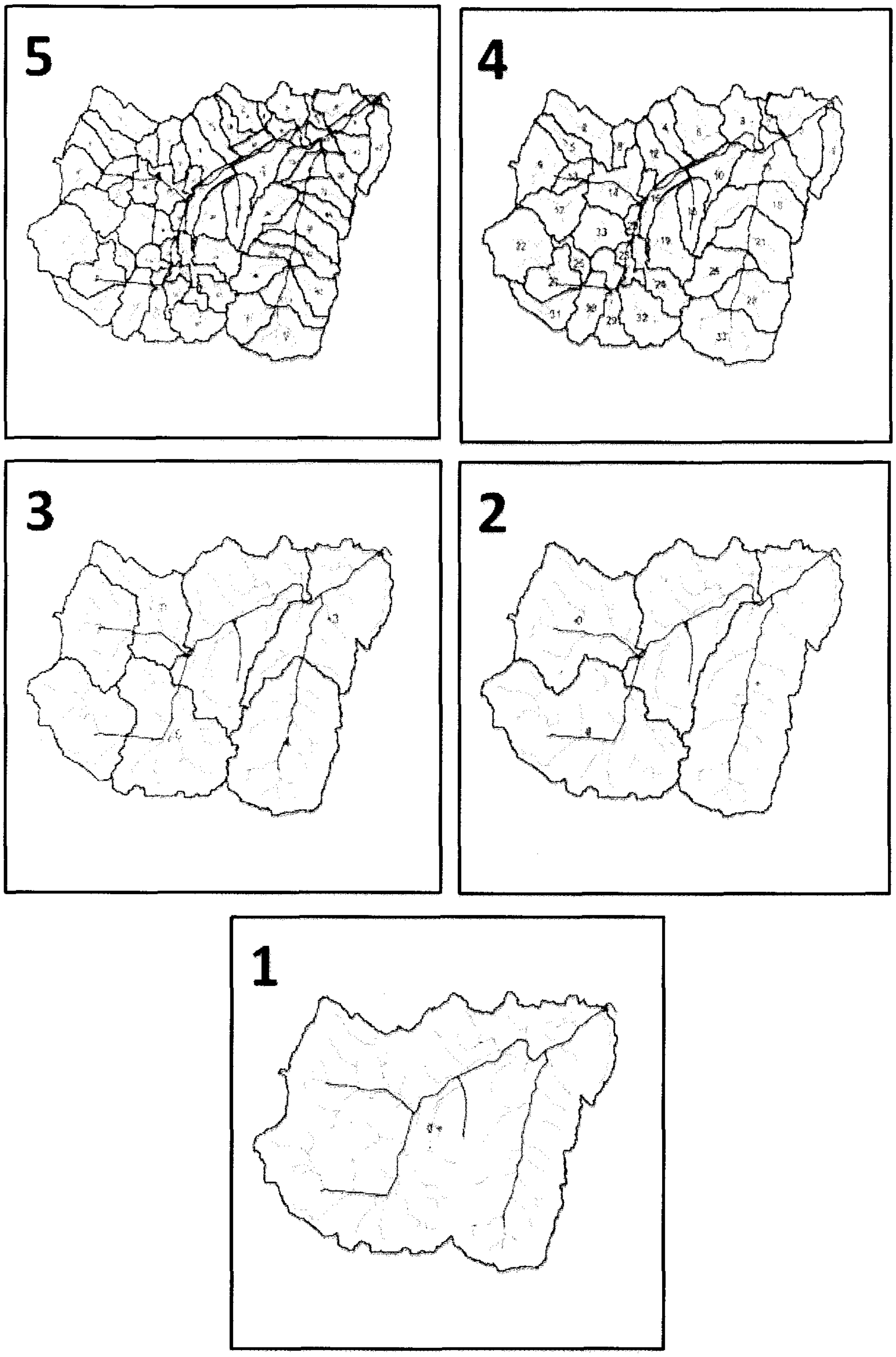


Figure 5.4 Discretization scenarios maps for Pigeon Basin

Figure (5.5) shows the discretization scenarios maps for SW Prong.

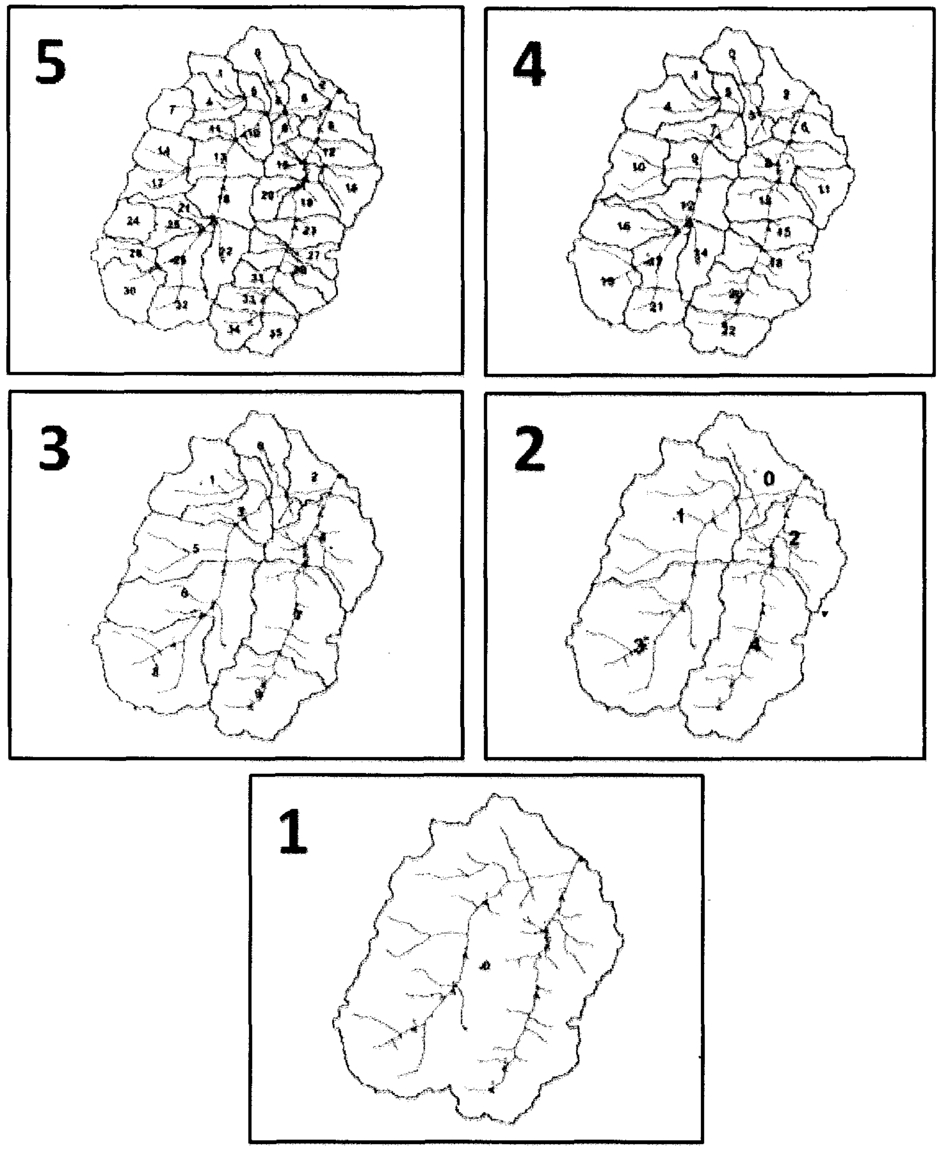


Figure 5.5 Discretization scenarios maps for SW Prong Basin

5.4 Peak flow rates estimation

Long term simulations were performed based over the range of dates shown in Table (3.9). In terms of peak flow rates, coarser resolution showed larger errors and an under estimation of peak flows in both basins, as it is shown in Table (5.1) and Figures (5.6) and (5.7). In other words, the smaller the sub-basins, the better the results.

Table 5.1 Peak flowrates modeling results

Basin	Scenario	RMSE (cfs)	Qmax (cfs)	Qmax Error (%)
Pigeon	5	11.69	1221.55	2.70
	4	14.25	1036.83	17.42
	3	19.67	918.75	26.82
	2	21.16	853.10	32.05
	1	28.68	739.64	41.09
SW Prong	5	16.71	395.49	11.32
	4	17.55	380.68	14.65
	3	19.30	354.79	20.45
	2	22.20	350.06	21.51
	1	26.63	330.17	25.97

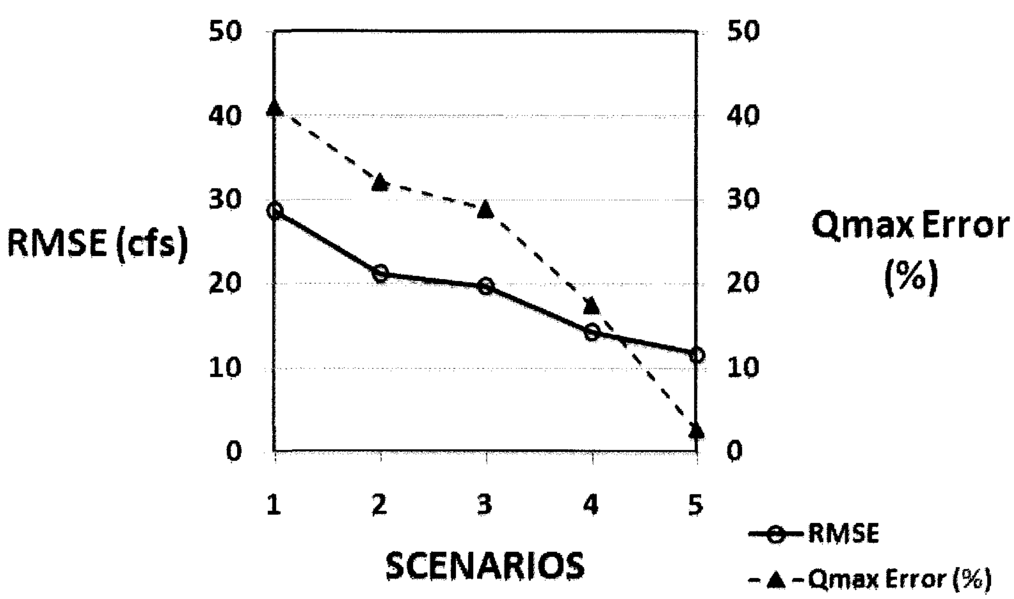


Figure 5.6 RMSE and Q_{max} Errors in Pigeon

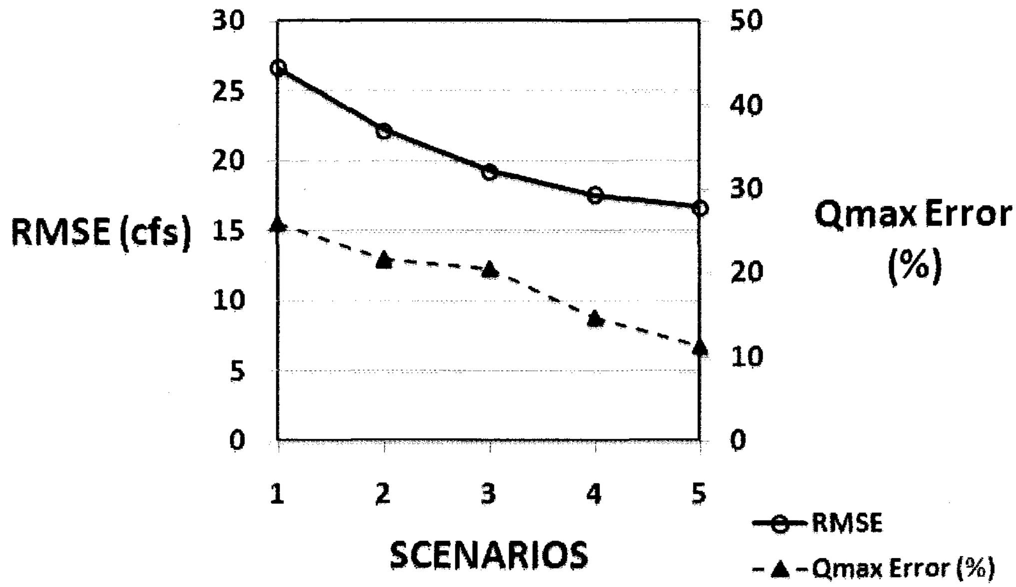


Figure 5.7 RMSE and Q_{max} Errors in SW Prong

Scenarios 1 showed a reasonable estimation of peak flow rate in both basins, considering estimation of peak flow rate was not solely intended during calibration. However, the runoff peak rate is the most important hydrologic variable for drainage system design and flooding analysis. Notice that Pigeon had larger errors for coarser scenarios than SW Prong, this could be explained because its higher level of urban development.

5.5 Representative Element Area (REA)

The REA represents a spatial case over which the hydrologic processes can remain simple in terms of distributed catchment behavior (Blöschl et al., 1995). Therefore, this concept was further investigated to find out an appropriate scale level in urban hydrology. Theoretically this element size is able to represent the complex heterogeneity in the basin and this relates directly to an ideal element size for distributed catchment modeling.

Wood et al. (1988) determined the runoff volume from 148 sub-catchments. These runoff volumes were ranked on the basis of sub-catchment size, irrespective of their relative position in the basin. The average of a 15 element filter, moving in steps of five, was plotted versus area. These plots were then used to determine the REA, defined as the area where the curve in flattened out. In other words, the REA is described as the scale where $|dq/da|$ becomes small, with q being the peak volume and a the sub-catchment area.

Based on the concept proposed by Wood (1988), the REA was estimated using the optimized Scenario 5 in both basins. Figure (5.8) shows the effect of REA averaging.

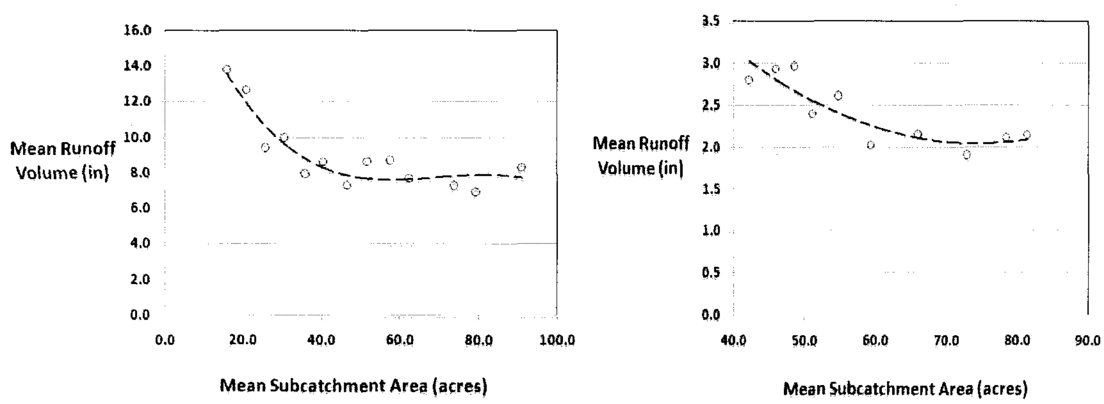


Figure 5.8 REA in Pigeon (left) and SW Prong (right)

From the results in Pigeon, it is clear to see than an average area sub-catchment of 50 acres might be appropriate (2% of the total basin size). In SW Prong, despite the lower runoff volumes, the runoff showed to be stabilized after a sub-catchment area of 60 acres (3% of the total size). We conclude that an average sub-basin size of 3% of the total basin may be an appropriate threshold scale in the context of urban hydrology. Arabi et al. (2006) using the Soil and Water Assessment Model (SWAT) found appropriate to use sub-catchment sizes of approximately 4% of total basin area. Arabi et al. (2006) used two mostly undeveloped basins: Dreisbach and Smith Fry, both basin sizes of about 2.5 square miles.

5.6 Measurements of performance

The indexes described in Chapter 2 were computed: the Pearson's Moment Correlation Coefficient (PMCC, Equation 2.3), Nash-Sutcliff Coefficient (NSC, Equation 2.4) and Index of Agreement (IOA, Equation 2.5). The model with a higher resolution level was taken as base model (Scenario 5) to compare the other four scenarios. In both basins, the IOA showed higher values than the PMCC and NSC as it shown in Figures (5.9) and (5.10).

In Pigeon, the metrics showed a poor performance for Scenarios 3, 2 and 1; as it was expected (Figure 5.9).

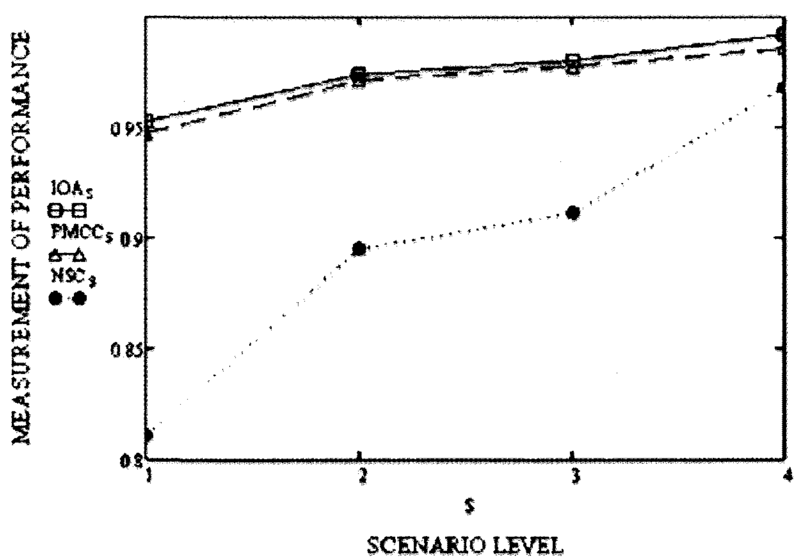


Figure 5.9 Measurement of performance in Pigeon

However, surprisingly Scenarios 3 and 2 showed reasonable values in SW Prong. (Figure 5.10). This behavior means that the imperviousness level might be a factor to estimate the REA. This idea should be further investigated, but it is outside the scope of this research.

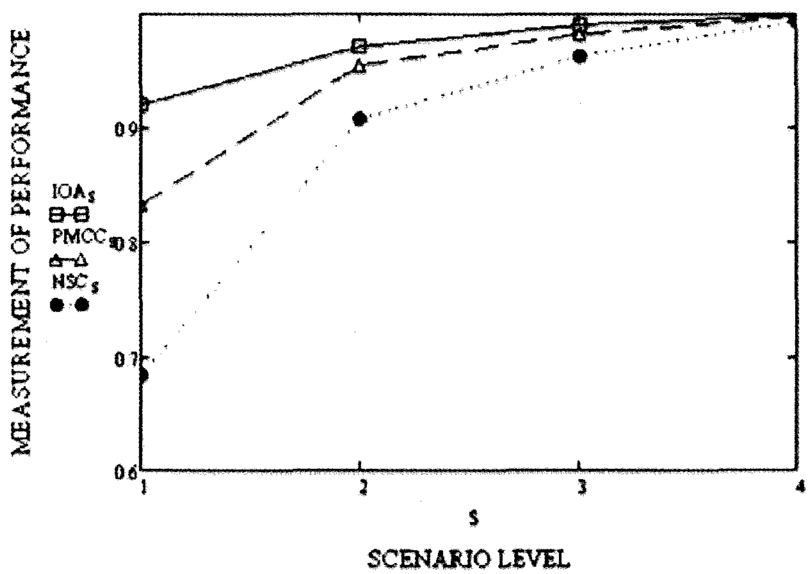


Figure 5.10 Measurement of performance in SW Prong

The effect of discretization levels is compared in Figures (5.11) and (5.12) on next page, where it is shown a deviation of simulated discharge between Scenario 5 and Scenarios 4 through 1; a 45 degree line would mean a perfect match. Notice that using a coarse level can result in a very different runoff to that obtained using a fine resolution model. This is especially certain for peak flowrates. Therefore, resolution issues would need more careful consideration to estimate high flows than low flows.

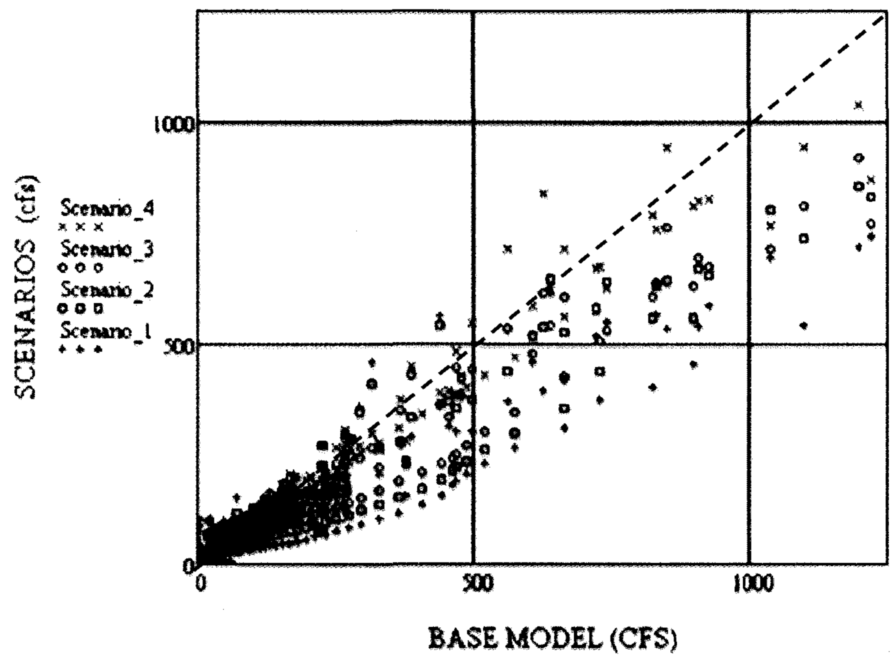


Figure 5.11 Deviation of simulated discharge in Pigeon

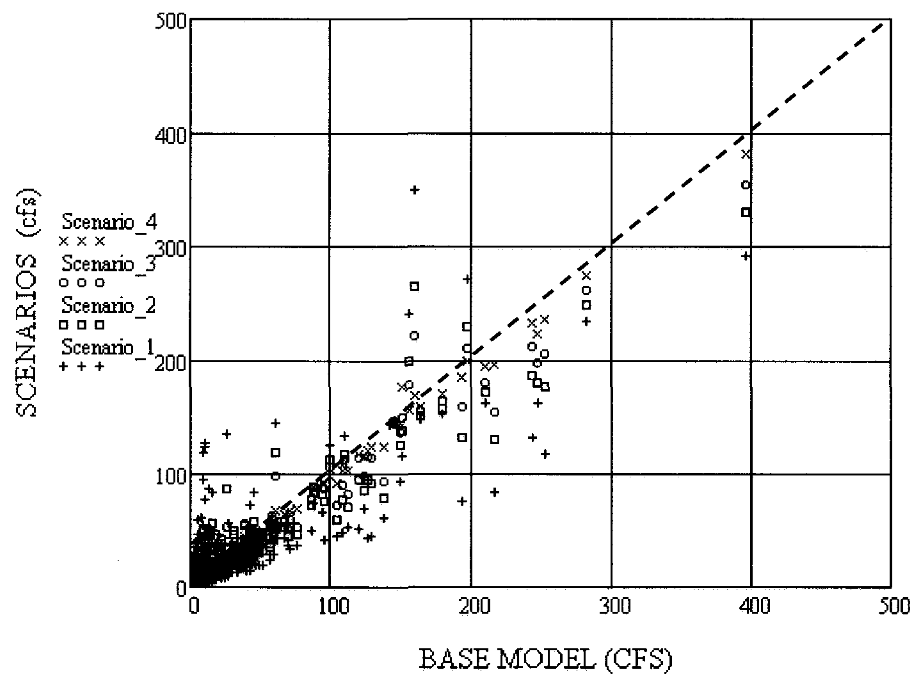


Figure 5.12 Deviation of simulated discharge in SW Prong

5.7 Effect on accumulated discharge volumes

Finally the effect on accumulated discharge volumes was investigated for the different scenarios. The runoff volume represents the volume of rainfall excess generated from the watershed area and it is the most important hydrologic variable for design and water quality protection. Apart from the peak flow discharge another very important streamflow characteristic of a stream river is its runoff volume. Some issues in water management and engineering hydrology require determining the peak discharge and the total runoff volume.

Long-term simulation results of the simplified (coarse) discretization are compared with those of the detailed (fine) discretization. Total runoff volumes were computed using the dates shown in Table (3.9). Table (5.2) shows the accumulated for the two study basins. Results were consistent with the initial analysis performed on peak flowrates, where coarser resolution under estimated runoff volumes.

Table 5.2 Cumulative discharge volumes

Scenario	Volume (Mgal)	
	Pigeon	SW Prong
5	970.49	222.57
4	939.85	221.73
3	888.50	199.44
2	860.38	161.72
1	529.20	111.51

Pigeon has a higher sensitivity in runoff response due to its higher imperviousness level than SW Prong. In fact, there was less difference in runoff volumes in SW Prong among Scenarios. Similar to the measurement of performance computed previously, a drastic difference in performance it was observed between Scenarios 2 and 1 in Pigeon; but SW Prong showed a more uniform behavior between scenarios.

One topic to be further investigated is how the Directed Connected Impervious Area (DCIA) reduces surface volumes and flow rate peaks. Even when BMP's techniques may not eliminate completely the effects of urbanization, their effectiveness could be evaluated through long term simulations.

5.8 Guidance for discretization of urban watersheds

The level of discretization used for a particular urban watershed establishes the cost of setting up and running a storm water management model. This research work attempted to investigate the needed level of discretization to effectively model an urban area and how different discretization scenarios affect the model results. Based upon the simulation results of the different levels of discretization, a methodology to discretisize urban watersheds was developed.

Urban drainage systems do not flow completely the undeveloped drainage, that is, the natural pathways previous to the urbanization. Nonetheless, generated runoff eventually reaches natural streams; for this reason, a Digital Elevation Model (DEM) constitutes very useful data.

In most cases the only point with recorded flows is the outlet and there are no observed flows for any interior point in the watershed, furthermore, having this data is not realistic. Thus, a synthetic hydrologic procedure could be used to size the routing channel length in each sub-watershed. In this sense, the corresponding channel routing length was optimized through an optimization algorithm (Section 3.3.1). The proposed methodology is intended for urban watersheds ranging in size from 1 to 10 mi^2. Smaller or larger sizes might require additional recommendations, but the mentioned range covers most of the applications in urban hydrology. A step by step protocol is given next.

Step 1: Identify watershed outlet and boundary.

Usually the watershed outlet is defined where streamflow records are available. Such data allows the model calibration. Once this point is defined, ArcHydro (2003) delineates the watershed boundary in a straightforward process in ArcGIS.

Step 2: Define subwatersheds shapes.

ArcHydro (2003) is used to define the sub-watersheds shapes by defining a threshold value to start a stream (Equation 5.1); this is a user-specified number that determines the minimum number of pixels within each delineated sub-basin (Djokic et al., 1997). The performed sensitivity analysis (Section 5.6) showed a performance stabilization when the average sub-watershed size reached about 3% of the total basin area. For this reason is recommendable to keep an average sub-watershed size no greater than 3% of the total basin area. If this condition is not met, it is suggested to reduce or increase the initial threshold value and repeat the process, refer to Equation (5.1). The initial step to define a HUW is once the sub-watershed size is satisfactory.

Step 3: Define channel routing length.

Since modeling the entire drainage network in each sub-watershed is not feasible, a simplified and equivalent routing channel should be obtained to represent the entire drainage network in each sub-watershed. This routing channel simulates the real drainage network storage and runoff attenuation.

A specific Hack's law (Hack, 1957) is developed for the urban watershed. This relationship relates the length of the routing channel (L) as a function of the sub-watershed area (A):

$$L=A^{h} \quad \ldots (5.2)$$

The exponent h in Equation (5.2) is obtained through optimization; a characteristic coefficient for h is 0.60. A typical large storm size is selected (1-Yr, 24-h is recommended) and applied to six or ten experimental sub-watersheds in SWMM, ranging in size from the minimum to the maximum sub-watershed sizes found previously in ArcHydro. Then, the peak flow rate is estimated for each sub-watershed.

Next, using the proposed Unit Hydrographs by Espey (1977), the peak flowrate is estimated as a function of the length of the routing channel (Equations 3.2 and 3.3). Finally, the exponent h is obtained through an optimization algorithm that minimized the RMSE (Root Mean Square Error, Equation 2.2) between the peak runoff of both methods (SWMM and Unit Hydrograph). The detailed procedure is given in the Section 3.3.1.

Step 4: Perform a Kinematic Wave (KW) conversion.

Since SWMM requires the use of rectangular shapes for the subwatersheds; using the optimized channel routing length, the corresponding sub-watershed width is obtained using the KW procedure proposed by Guo and Urbonas (2008). In that methodology the continuity and energy principles were interpreted to preserve the sub-watershed area and vertical fall over the receiving waterway's length. A KW cascading plane is specified by the plane's area, width and slope. This step finalizes the construction of a HUW.

Step 5: Build the SWMM model.

The SWMM schematic network comprises sub-catchments, junctions (nodes) and link conduits, represented by surveyed cross sections in natural streams (Section 3.4). The discretized watershed is idealized as a series of HUW's connected together by links. SWMM network is built by connecting successively HUW outlets to natural streams through nodes and each link element transmits flow from node to node, where the last junction is modeled as an outfall with free discharge. The proposed guidance is presented in Figure (5.13).

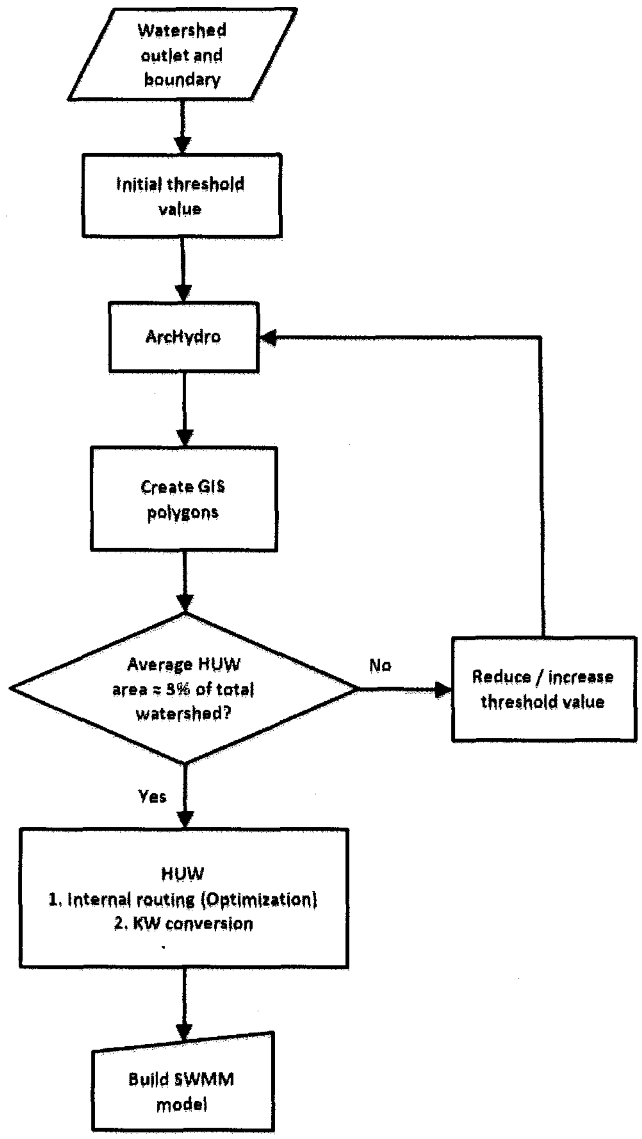


Figure 5.13 Discretization flow chart in urban hydrology

The proposed methodology constitutes a helpful tool to disaggregate urban watersheds in urban hydrology. The HUW's facilitate to model the complex urban drainage system in an urban watershed, with less detail needed in defining the conveyance network.

The found threshold value in which simulations results did not show improvement in performance is an important finding for future studies. Since the slight improvement in modeling results by using a higher resolution data is usually discouraged. The suggested spatial scale resolution provides a reasonable equilibrium of accuracy, cost, time and complexity. Above such threshold level the spatial variability of an urban watershed can be neglected, however, it provides sufficient information for accurate modeling of basin runoff. The KW conversion showed to effective and relatively simple to implement.

The methodology showed in Figure (5.13) would help engineers to construct rainfall-runoff models with confidence in terms of the spatial resolution. The proposed procedure is recommended for other urban watersheds to avoid uncertainties derived from the disaggregation level.

6.0 SUMMARY, CONCLUSIONS AND RECOMMENDATIONS

6.1 Summary

This research project met its two main objectives. The first one was to use a Multi-Criteria Approach for calibration and finding the existence of a threshold zone for discretization in urban hydrology. The second one was to perform a disaggregation analysis of the case studies. Preliminary criterion for selecting the scale in urban basins is given, but additional considerations should be addressed.

Two case studies in North Carolina (Pigeon and SW Prong basins) were used to illustrate the proposed methodology for calibration. Irregular sub-catchments shapes obtained with ArcHydro (Maidment, 2002) were converted to regular shapes using a kinematic wave cascading plane approach proposed by Guo and Urbonas (2008).

Calibration procedure used a MCDA approach that minimized the RMSE (Root Mean Square Error) between the flow duration curves of the modeled and the observed runoff. In this sense, the flow duration curve was divided in High and Low Flows using the 1-Yr storm to split the curve, since Nehrke and Roesner (2004) found a change in flow regime in several urban areas at this point of the curve. A long term simulation (20 Years) was performed on both basins to find the 1-Yr storm.

A Multi-Criteria Optimization methodology was implemented in both sectors of the curve, using the Horton's decay coefficient (K, 1/h) and drying time (Tw, days) as parameters. These parameters were chosen since their value is not clearly defined in the framework of urban hydrology and because they are significant variables for long term simulations. Total Impervious Area (TIA) is also an important variable to calibrate in urban hydrology; however, it was not included in the optimization algorithm since imperviousness raster maps were available in both basins and because Equation (2.1) already provides an approach to convert TIA into DCIA (Direct Connected Impervious Area).

Pareto optimal front surface were obtained for case studies using K and Tw as parameters. Significant difference was found in terms of Tw in both basins, an accurate estimation of the High Flow portion of the flow duration curve required larger values of Tw than the Low Flow portion. However, in terms of K, a smaller difference between both sectors of the flow duration curve was observed (Tables 4.4 and 4.5).

The second part of this research involved a discretization analysis. A set of hydrologic experiments was performed in each watershed, where different levels of discretization were used in each scenario. Discretization scenarios were obtained with ArcHydro (Maidment, 2002). Needed GIS data was extracted through a toolbox that was developed and SWMM5 simulations were performed with the watershed configurations. The main objective was to investigate a threshold value in urban hydrology. This value represents the needed level of discretization in urban watersheds after which the improvement in performance becomes asymptotic either to 1.00 (Pearson's Moment Correlation Coefficient - PMCC, Nash-Sutcliff Coefficient - NSC and Index of

Agreement - IOA) or to zero (Root Mean Square Error - RMSE) and thus, is not significant to improve the spatial resolution. Model performance depends upon the data resolution and finer resolution would yield better model results (Table 5.1), nevertheless, very high resolution discretization is not a reasonable or practical solution in response to this. In this sense, the Representative Element Area (REA) concept was explored using SWMM5 and it was found that sub catchment sizes of 3% of the total basin size were appropriate. Coarser resolution levels underestimated peak flow rates and total runoff volumes.

6.2 Conclusions

The practice of urban stormwater hydrology is not an exact science. While the hydrologic processes are well-understood, the necessary equations and boundary conditions required to solve them are often quite complex. However, this research work constitutes a step forward since some guidelines were found; however, the presented suggestions are not the last word in urban hydrology.

SWMM5 requirement to have rectangular sub-catchment shapes can be effectively met by a kinematic cascade plane conversion to convert irregular natural sub-catchments. Further understanding was gained for long term simulations. MCDA methodology and numerical simulations showed that Horton's decay coefficient (K, 1/h) and drying time (Tw, days) need to have different values for the High and Low Flow portions of the flow duration curve to improve performance. Longer drying times were required to improve estimation of High Flows than Low Flows because the soils would take more time to recover their initial infiltration capacity.

A 3% of total basin size is suggested to disaggregate watersheds since Representative Element Area (REA) values of 50 acres and 60 acres in Pigeon and SW Prong respectively were found. However, this result is by no means a universal concept to apply in all models.

6.3 Recommendations

This work arose some issues that should be further researched. Differences between runoff outputs using different DEM resolutions were not addressed; however, as DEM rasters become available with higher resolution, it may be worth it to investigate the effects of the runoff response with different resolution grids.

In terms of the MCDA approach for calibration, it would be worth it to investigate the variations of the drying time (Tw, days) and the Horton's decay coefficient (K, 1/h) along the pareto optimal front.

It is suggested the REA concept to be applied to other highly urbanized basins to confirm the values found here. The REA size found here was appropriate for the case studies, but its value may be different for another range of basin sizes, for example smaller than 1 mi^2 or larger than 10 mi^2. Thus, caution must be taken when using the presented results to other basins with different range sizes.

7.0 REFERENCES

[1] Ao T. et al. (2003). Effects of sub-basin scale on runoff simulation in distributed hydrological model: BTOPMC. Weather Radar Information and Distributed Hydrological Modelling (Proceedings of symposium HS03 held during IUGG2003 at Sapporo, July 2003). IAHS Publ. No. 282, 2003.

[2] Akan, A.O. and Houghtalen, R,J, (2003). Urban Hydrology, Hydraulics and Stormwater Quality: Engineering, Applications and Computer Modeling. John Wiley & Sons, Inc.

[3] American Society of Civil Engineers (1988). *MOP No. 87 on Urban Runoff Quality Management.*

[4] ASCE Task Committee on Definition of Criteria for Evaluation of Watershed Models of the Watershed Management Committee, Irrigation and Drainage Division (1993). Criteria for evaluation of watershed models. J. Irrigation and Drainage Engineering, Vol. 199, No. 3.

[5] Arabi, M., R.S. Govindaraju, M.M. Hantush, and B.A. Engel. (2006). Role of watershed subdivision on modeling the effectiveness of best management practices with SWAT. J. Amer. Water Res. Assoc. 42(2): 513-528.

[6] Arcement, G.J. and Schneider, V.R. (1989). Guide for selecting Manning's roughness coefficients for natural channels and floodplains. U.S. Geological Survey.

[7] Arc Hydro Tools (2003). *Tutorial, Version 1.1 Beta 2*. ESRI 380 New York St., Redlands, CA ASCE Task Committee on Definition of Criteria for Evaluation of Watershed Models. (1993). Criteria for evaluation of watershed models. Journal of Irrigation and Drainage Engineering, ASCE, 119 (3), 429-442.

[8] Arnold, Jr., C. L., and Gibbons, C. J. (1996). Impervious surface coverage: The emergence of a key environmental indicator. J. Am. Plan. Assoc., 62-2, 243.

[9] Ashe, R. (2003). Investigating Runoff Source Area for an Irrigation Drain System using ArcHydro tools. Proceedings of the Twenty-Third Annual ESRI User Conference, San Diego.

[10] Bathurst, J.C. (1986). Physically based distributed modeling of upland catchment using the Systeme Hydrologique Europeen. Journal of Hydrology, 87, 79–123.

[11] Bicknell, B.R., J. C. Imhoff, J. L. Kittle, A. S. Donigian, and R.C. Johanson. (1993). Hydrological Simulation Program - FORTRAN (HSPF): Users Manual for Release. EPA-600/R-93/174, U.S. EPA, Athens, GA, 30605.

[12] Blöschl, G., et al. (1995). On the representative elementary area (REA) concept and its utility for distributed rainfall-runoff modeling. Centre for Resource and Environmental Studies, Australian National University, Canberra City, Australia. Hydrological Processes, v. 9(3-4) p. 313-330.

[13] Booth, D. (1991). Urbanization and the natural drainage system impacts, solutions and prognoses. Northwest Environmental Journal, 7, 93-118.

[14] Brink, P. (2004). Wayne County Rouge Program Office (RPO). Memorandum. March 26, 2004.

[15] Brink, P., Regnemorter, L., and TenBroek, M. (1996). Evaluating the City of Detroit's Combined Sewer Overflow Storage/treatment Facilities through Continuous Modeling. Proc. Urban Wet Weather Pollution: Controlling Sewer Overflows and Stormwater Runoff, Specialty Conference, Quebec City, PQ, Canada, Water Environ. Fed., 2-61.

[16] Brink, P., Broek, T. (1988). Advances in Modeling the Management of Stormwater Impacts, Vol. 6. Chapter 17, Characteristic Width and Infiltration for Continuous SWMM. Edited William James. Pub. CHI, Guelph, Ontario, 504 pp.

[17] Commission for Environmental Cooperation, CEC (1997). Ecological Regions of North America, Toward a Common Perspective. Montreal, Canada.

[18] Chow, V.T. (1959). Open Channel Hydraulics. McGraw-Hill, New York, NY, 680 p.

[19] Chow, V.T. (1988). Open Channel Hydraulics, Chapter 5: Development of Uniform Flow and Its Formulas. McGraw-Hill, Boston.

[20] City of Raleigh (2001). *Central Engineering Department*. Stormwater Division. http://www.raleighnc.gov (Accessed on October 20, 2008).

[21] CUHP (2008). Colorado Urban Hydrograph Procedure Excel-Based Computer Program. Urban Drainage and Flood Control District. 2480 West 26th Avenue. Suite 156-B. Denver, Colorado 80211

[22] Dent, S., Hanna, R.B., Wright, L. (2004). Automated Calibration using Optimization Techniques with SWMM Runoff. Innovative Modeling of Urban

Water Systems, Monograph 12, W. James, Ed., Computational Hydraulics International, CHI. Guelph, Ontario, Canada.

[23] Djokic, D., Ye, Z., Miller, A. (1997). Efficient Watershed Delineation Using ArcView and Spatial Analyst. Proceedings of the 17th Annual ESRI User Conference, San Diego, CA

[24] Downer, C.W., et al. (2006). Gridded Surface/Subsurface Hydrologic Analysis (GSSHA) Model: A Model for Simulating Diverse Streamflow Producing Processes, p. 131-159, in Watershed Models, V.P. Singh, and D. Frevert, eds., Taylor and Francis Group, CRC Press, 637 pp.

[25] Environmental Protection Agency, EPA (2008) http://www.epa.gov/ednnrmrl/models/swmm/index.htm (Accessed on November 04, 2008).

[26] Espey, W. H., Jr, Altman, D. G. and Graves, C. B., Jr. (1977). Nomograph for 10 minutes unit hydrographs for urban wartersheds. Tech. Memo. 32, ASCE, New York, USA.

[27] Federal Emergency Management Agency, FEMA (2008). National Flood Insurance Program, NFIP. http://www.fema.gov/about/programs/nfip/index.shtm (Accessed on November 04, 2008).

[28] Flügel, Wolfgang-Albert (1995). Delineating hydrological response units by geographical information system analyses for regional hydrological modelling using PRMS/MMS in the drainage basin of the River Bröl, Germany. Hydrological Processes, vol. 9, issue 3-4, pp. 423-436

[29] Garmin (2008). eTrex Legend ®. http://www8.garmin.com/aboutGarmin/ (Accessed on November 05, 2008).

[30] Giddings, E.M.P., and Steuer, J.J. (2007). Hydrologic Variability in Urban Streams of the North Carolina Piedmont.

[31] Guo, James C.Y. (2005). Kinematic Wave Unit Hydrograph for Storm Water Prediction, ASCE J. of Irrigation and Drainage Engineering, Vol. 132, No 4, July/August. Pp 410-417.

[32] Guo, James C.Y. and Urbonas, B. (2008). Conversion of Natural Watershed to Kinematic Wave Cascading Plane, accepted to ASCE J. of Hydrologic Engineering, (revised in May 2008).

[33] Gupta, H.V., Sorooshian, S., and Yapo, P.O. (1988). Toward improved calibration of hydrologic models: Multiple and non-commensurable measures of information. Water Resources Research, 34(4), 751-763.

[34] Hack, J.T. (1957). Studies on longitudinal profiles in Maryland and Virginia. U.S. Geological Survey Professional Paper, 294B, 45-92.

[35] Harrelson, C.C., Rawlins, C.L., Potyondy, J.P. (1994). Stream channel reference sites: an illustrated guide to field technique. General Technical Report RM-245. Fort Collins, CO. US Department of Agriculture, Forest Service, Rocky Mountain Forest and Range Experimental Station. 61 p.

[36] Heath, R.C. (1994). Groundwater Recharge in North Carolina. Prepared for the Groundwater Section Division of Environmental Management in North Carolina. Department of Environment. Health and Natural Resources. Raleigh, North Carolina.

[37] Heineman, M. (2007). NETStorm Help. Accessed on December 17, 2008. http://www.dynsystem.com/NetSTORM/

[38] Huang, H., Cheng, S., Wen, J., Lee, J. (2008). Effect of growing watershed imperviousness on hydrograph parameters and peak discharge. Hydrological Processes, 22, 2075-2085.

[39] Huber, W. C. and Dickinson, R. E. (1988). Storm Water Management Model (SWMM) User's Manual, version 4.0. United States Environmental Protection Agency, Athens, Georgia. 595 pp.

[40] Hudson, J.C. (2002). Across This Land: A Regional Geography of the United States and Canada. The John Hopkins University Press: Baltimore, Maryland.

[41] Hundecha Y. (2004). Regionalization of Parameters of a Conceptual Rainfall-Runoff Model. Institut für Wasserbau der Universität Stuttgart. Stuttgart Germany, November 2005.

[42] Joint Task Force of the Water Environment Federation and the ASCE. (1998). Urban runoff quality management. WEF Manual of Practice. No. 23; ASCE Manual and Reports on Engineering Practice. Alexandria, VA. and Reston, VA.

[43] Joong Gwang Lee and James P. Heaney (2003). Estimation of Urban Imperviousness and its Impacts on Storm Water Systems. Journal of Water Resources Planning and Management, Vol. 129, No. 5.

[44] Ko, Seok-Ku; Fontane, Darrell G.; Labadie, John W. (1992). Multiobjective Optimization of Reservoir Systems Operation. Journal of the American Water Resources Association, vol. 28, Issue 1, p.111-127.

[45] Kokkonen, T., Jakeman, A. (2001). A comparison of metric and conceptual approaches in rainfall-runoff modeling and its implications. Laboratory of Water Resources, Helsinki University of Technology, Helsinki, Finland. Centre for

Resource and Environmental Studies, Australian National University, Canberra, ACT, Australia. Water Resources Research, Vol. 37, No. 9, Pages 2345–2352.

[46] LACDPW (2006). Los Angeles County Department of Public Works. Hydrology Manual. Water Resources Division. 900 South Fremont Avenue. Alhambra, CA.

[47] Lee, J. G., and Heaney, J. P. (2002). Directly connected impervious areas as major sources of urban stormwater quality problems, evidence from South Florida. Proceedings, 7th Biennial Conference on Stormwater Research and Water Quality Management, Southwest Florida Water Management District.

[48] Leopold, L.B., et al. (1964). Fluvial Process in Geomorphology. W.H. Freeman and Co., San Francisco, CA. 522pp.

[49] Madsen, H. and Jacobsen, T. (2001). Automatic Calibration of the MIKE 11/NAM rainfall-runoff model. Nordic Hydrological Conference, Uppsala, Sweden, NHP-Report No. 46, Vol. 1, 276-283.

[50] Maidment, D. (2002). Arc Hydro: GIS for Water Resources. ESRI Press.

[51] Mays, L. (2001). Water Resources Engineering. Arizona State University. Tempe, AZ. John Wiley & Sons, Inc.

[52] McMahon, G. and Cuffney, T.F. (2000). Quantifying Urban Intensity in Drainage Basins for Assessing Stream Ecological Conditions. Journal of the American Water Resources Association (JAWRA) 36:1247-1261.

[53] Moretti, G., Montanari, A. (2008). Inferring the flood frequency distribution for an ungauged basin using a spatially distributed rainfall-runoff model. Journal Hydrology and Earth System Sciences.

[54] Nash, J.E., Sutcliffe, J.V. (1970). River flow forecasting through conceptual models. Part 1. A discussion of principles. Journal of Hydrology 10, 282–290.

[55] National Climatic Data Center, NCDC (2008). Accessed on December 15, 2008. http://www.ncdc.noaa.gov/oa/climate/climateinventories.html

[56] NED (2008). National Elevation Data set.
http://ned.usgs.gov/Ned/faq.asp
Accessed on November 19, 2008.

[57] Nehrke, S.M. and L.A. Roesner. (2004). Effects of Design Practice for Flood Control and BMPs on the Flow Frequency Curve. Journal of Water Resources Planning and Mangement, ASCE. March/April 2004, pp 131-139.

[58] NERC (1975). Natural Environment Research Council. Flood Studies Report, 1975. Whitefriars Press., United Kingdom.

[59] Nix, S. (1994). Urban Stormwater Modeling and Simulation. CRC Press, Inc., Boca Raton, FL, USA.

[60] Patra, K.C. (2000). Hydrology and Water Resources Engineering. CRC Press: New York.

[61] Pettyjohn, W.A. (1987). Classification of Groundwater Regions. Protection of Public Water Supplies from Groundwater Contamination. William Andrew, Inc.

[62] Pomeroy, Ch.A. (2007). "Evaluating the Impacts of Urbanization and Stormwater Management Practices on Stream Response." PhD Thesis. Department of Civil and Environmental Engineering. Colorado State University. Fort Collins, CO.

[63] Rigon, R., Rodriguez-Iturbe, I., Maritan, A., Giacometti, A., Tarboton, D.G., Rinaldo, A. (1996). On Hack's law. Water Resources Research, Vol. 32, No. 11, Pages 3367-3374.

[64] Rivas, I., Roesner, L. (2007). Spatial Discretization of Large Urban Watersheds for Rainfall-Runoff Modeling. World Environmental and Water Resources Congress, Tampa, Florida. May 15-19, 2007.

[65] Rodriguez, F., et al. (2003). Surface runoff in urban catchments: morphological identification of unit hydrographs from urban databanks. Laboratoire Central des Ponts et Chausse´es, Nantes, France. Journal of Hydrology. Vol. 283, p 146–168

[66] Roesner, L.A., et al. (1991). Hydrology of Urban Runoff Quality Management. Proc. 18th National Conf Water Resources Planning and Management. Symposium Urban Water Resources. American Society of Civil Engineers. New York, NY.

[67] Roesner, L.A., Burgess, E.H., Aldrich, J.A. (1991). The hydrology of urban runoff quality management. 18^{th} National Conference Water Resources Planning and Management and Urban Water Resources. Edited by Jerry L. Anderson. Session S-03 Hydrologic Modeling for Urban Stormwater Management. American Society of Civil Engineers, New York, N.Y.

[68] Rojas, R., et al. (2003). CASC2D-SED v 1.0. Reference Manual. A 2-Dimensional Rainfall-Runoff and Sediment Model. Colorado State University. July 2003.

[69] Romero-Davis, J. (2008) Guidance for Rural Watershed Calibration with EPASWMM. Colorado State University. Department of Civil and Environmental Engineering. Master's Thesis.

[70] Roshan S., Yasuto T., and Kaoru T. (2006). Input data resolution analysis for distributed hydrological modeling. Journal of Hydrology, 319, 36-50.

[71] Rossman, L.A. (2005). Stormwater Managament Model User's Manual Version 5.0; United States Environmental Protection Agency; National Risk Management Research Laboratory. Office of Research and Development Report EPA/600/R-05-040; Cincinnati, Ohio.

[72] Rossman, L.A., Dickinson, R., Schade, T., Chan, C., Burguess, E.H., Sullivan, D., Lai, F. (2004). SWMM5 - The Next Generation of EPA's Storm Water Management Model. Innovative Modeling of Urban Water Systems, Monograph 12, W. James, Ed., Computational Hydraulics International, CHI. Guelph, Ontario, Canada.

[73] Schueler, T. R. (1994). The importance of imperviousness. Watershed Protection Technology, 1-3, 100–111.

[74] Shannon, R.E. (1975). Systems Simulation: The Art and Science, Prentice-Hall, Englewood Cliffs, NJ, 1975.

[75] Shoemaker L., et al. (1997). Compendium of tools for watershed assessment and TMDL development U.S. Environmental Protection Agency. Washington, DC.

[76] Shresthaa, et al. (2006). Input data resolution analysis for distributed hydrological modeling. Department of Urban and Environmental Engineering, Graduate School of Engineering. Kyoto University, Kyoto, Japan. Journal of Hydrology, 319, 36–50.

[77] Shrestha, R., Tachikawa, Y., Takara, K . (2005). Input data resolution analysis for distributed hydrological modeling. Journal of Hydrology. Volume 319, Issues 1-4, 15 March 2006, Pages 36-50.

[78] Singh, V.P. (1989). Hydrologic Systems. Vol. 2, Watershed Modeling. Prentice Hall, Inc., Englewood Cliffs, New Jersey.

[79] Sivapalan M. and Kalma, J.D. (1995). Scale problems in hydrology: the contributions of the Robertson workshop in: Scale Issues in Hydrological Modelling (ed. By J.D. Kalma and M. Sivapalan), 1-8 John Wiley & Sons Ltd, UK.

[80] Soil Conservation Service (1986). Urban Hydrology for Small Watersheds, U.S. Dept. of Agriculture, Soil Conservation Service, Engineering Division, Technical Release 55.

[81] Soil Conservation Service (1992). TR-20. Computer Program for Project Formulation Hydrology. The Hydrology Unit and the Technology Development Support.

[82] Stefan, W.K. (1996). Using DEMs and GIS to define input variables for Hydrological and Geomorphological analysis. Application of Geographic Information systems in Hydrology and Water Resource Management: International Association of Hydrological Sciences, Series of Proceedings and Reports, 235, 183-190.

[83] Storm Water Management Model, SWMM. (2008). *Version 5.0.013.* http://www.epa.gov/ednnrmrl/models/swmm/index.htm (Accessed on October 29, 2008)

[84] UDFCD (2005). "User Manual, Colorado Urban Hydrograph Procedure - CUHP2005". Urban Drainage and Flood Control District. http://www.udfcd.org/downloads/down_software.htm

[85] Urbonas, B. (1979). Reliability of Design Storms in Modelling. International Symposium on Urban Storm Runoff. University of Kentucky, Lexington, KY, July 23-26.

[86] Urbonas, B. (2007). Stormwater Runoff Modeling; Is it as Accurate as We Think?. Engineering Conferences International Conference on Urban Runoff Modeling: Intelligent Modeling to Improve Stormwater Management, Humboldt State University. Arcata, CA.

[87] USACE (2007). The Hydrologic Modeling System HEC-HMS User's Manual. http://www.hec.usace.army.mil
Accessed on October 20, 2008).

[88] USACE-HEC (1977). U.S. Army Corps of Engineers, Hydrologic Engineering Center. Storage, Treatment, Overflow, Runoff Model, STORM, Generalized Computer Program. 723-58-L77520. Davis, CA.

[89] U.S. Department of Transportation (2005). Federal Highway Administration. Hydraulic Design of Highway Culverts. Publication No. FHWA-NHI-01-020. Hydraulic Design Series Number 5

[90] United States Geological Survey, USGS (2008). Accessed on December 16, 2008. http://waterdata.usgs.gov/nwis/

[91] USGS (2008). National Map Seamless Server. Accessed on October 18, 2008. http://seamless.usgs.gov/index.php,

[92] Vieux, B.E. (2004). Distributed Hydrologic Modeling Using GIS. Water Science and Technology Library, Volume 48. Kluwer Academic Publishers. Printed in the Netherlands.

[93] Vieux, B.E., and Moreda, F.G. (2003). Ordered Physics-Based Parameter Adjustment of a Distributed Model. Chapter 20 in Calibration of Watersheds Models, Edited by Q. Duan, S. Sorooshian, H.V. Gupta, A.N. Rousseau, R. Turcotte, Water Science and Application Series, 6, American Geophysical Union, ISBN 0-87590-355-X pp. 267-281.

[94] Vieux, B.E., Vieux, J.E., Chen, Ch., Howard, K.W. (2003). Operational Deployment of a Physics-Based distributed Rainfall-Runoff Model for Flood Forecasting in Taiwan. Weather Radar Information and Distributed Hydrological Modelling, Proceedings of Symposium HS03 held during IUGG2003 at Sapporo, Japan. IAHS Publ. No. 282.

[95] Walsh, C.J., et al. (1988). The Urban Stream Syndrome: Current Knowledge and the Search for a Cure. J.N. American Benthology Society. 24(3): 706-723.

[96] Water Balance Model (2008). Accessed on August 21, 2009 http://www.waterbalance.ca/

[97] William M. Alley, Jack E. Veenhuis (1983). Effective Impervious Area in Urban Runoff Modeling. Journal of Hydraulic Engineering, Vol. 109, No. 2, pp. 313-319.

[98] Willmott, C.J. (1981). On the validation of models. Physical Geography 2, 184–194.

[99] Wood, E.F., et al. (1988). Effects of spatial variability and scale with implications to hydrologic modeling. Journal of Hydrology, 102, 29–47.

APPENDIX A: HYDROLOGIC / HYDRAULIC PARAMETERS

COMMON DATA

```
GW DATA COEFFICENTS         A1=0.015      B1=1
                            A2=0.01       B2=1
Aquifer: Sand Clay (Kh=0.02 in/h)

SIMULATION OPTIONS
Time step = 15 secs
Dynamic Wave
       Keep inertial terms (Full Saint Venant Equation).
       Supercritical flow defined by Slope and Fr
       Force Main Equation: Hazen Williams
       Variable Time Step (Adjustment Factor = 75%)
       No Conduit Lengthening
```

PIGEON

```
Dates: 07/04/2002 00:00
       10/28/2003 00:00
Length of record:   n=1.3178 Years

IMPERVIOUSNESS CORRECTION FACTORS: K1=0.13, K2=1.35

HORTON INFILTRATION PARAMETERS
       Fo=4.23 in/h
       Fc=0.70 in/h
       Kw=Variable
       Tw=Variable

DETENTION STORAGE
       DS (Imperv)=0.07
       DS (Perv)=0.25

MANNINGS ROUGHNESS
       N-Imperv=0.012
       N-Perv=0.30
       N-Natural Channels=0.043
```

SW PRONG

```
Dates: 07/04/2002 00:00
       07/28/2003 00:00
Time step = 15 secs
Length of record:   n=1.0658 Years

IMPERVIOUSNESS CORRECTION FACTORS: K1=0.15, K2=1.41

HORTON INFILTRATION PARAMETERS
       Fo=4.15 in/h
       Fc=0.65 in/h
       Kw=Variable
       Tw=Variable

DETENTION STORAGE
       DS (Imperv)=0.08
       DS (Perv)=0.26

MANNINGS ROUGHNESS
       N-Imperv=0.012
       N-Perv=0.30
       N-Natural Channels=0.041
```