ABSTRACT

SAUNDERS JR., WILLIAM LEWIS. Detention Basin Design to Mitigate Regional Peak Flow Impacts. (Under the direction of James Bowen.)

Volumes and rates of stormwater runoff increase due to urban-type-development. Detention basins provide a volume of storage lost through the construction of impervious surfaces, such as buildings, streets and parking lots. One method of stormwater management involves the construction of "on-site" detention facilities.

Runoff from on-site detention facilities, when these are placed at locations in the lower portion of a watershed, can combine with the regional flow and have their peak flow arrive downstream simultaneously with the regional peak flow; thereby, sometimes producing downstream peak flow rates greater than the developed watershed would have experienced without the detention basins. This phenomenon is known as the "regional effect."

In this research, simulated responses from hypothetical catchments using a wide range of hydrologic parameters were generated with a numerical model and used to study detention basin storage capacity or combined into watersheds to study the regional effect.

The regional effect analysis was conducted by placing hypothetical developments at different watershed locations and analyzing the regional impact due to uncontrolled and different levels of controlled release.

Based upon the simulation model results, it was found that regional impact levels increased due to uncontrolled release as the development location moved upstream from the base of the watershed.

In addition, it was found that as soil storage capacity increased a broader range of development locations produced a measurable impact, and the overall regional impact levels increased.

Analysis of the uncontrolled results indicates that regional impact can be mitigated if the detention basin elongated outfall peak-flow-crest comes and goes before the arrival of the primary watershed peak flow. If this is not possible, then the regional impact can be minimized by limiting the detention basin outflow to a value that is 50 to 80 percent of the pre-development peak flow. This ratio of detention pond peak flow to the predevelopment peak flow, referred to as the peak flow reduction factor (PFRF), was found to depend on the location of the detention basin within the watershed, but varied only slightly across the range of hydrologic and physical properties that were varied in the study.

Detention basin storage was estimated by routing the runoff from a hypothetical catchment through a detention basin limiting the peak outflow to the same limits of controlled release utilized in the earlier analysis. Each hypothetical catchment runoff was estimated using a unique combination of ground cover conditions and hydrologic parameter values. This analysis developed statistical relationships that estimated the percent of runoff used for basin storage. Using this analysis procedure, the percent of runoff volume required for basin storage was found to depend upon storm duration, soil type, post-development curve number (CN), and the ratio of detention pond peak flow to the predevelopment peak flow. Other hydrologic parameters that were varied in the analysis (watershed slope, storm return period, development size) did not affect the percent of runoff volume to be stored in the pond. For soil type A, the percent of runoff

volume required for basin storage did vary depending on storm return period and duration, but not on the other variables.

In conclusion, this dissertation provides comprehensive detention basin design guidelines to mitigate the regional effect due to on-site detention, and a screening-level method that quickly estimates basin storage using a limited number of hydrologic parameters, and the same prescribed peak flow reduction factors used in the regional effect study.

DETENTION BASIN DESIGN TO MITIGATE REGIONAL PEAK FLOW IMPACTS

by
WILLIAM LEWIS SAUNDERS JR.

A dissertation submitted to the Graduate Faculty of
North Carolina State University
in partial fulfillment of the
requirements for the Degree of Doctor of Philosophy

CIVIL ENGINEERING

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CHAPTER 1

INTRODUCTION

Volumes and rates of stormwater runoff increase due to urban-type development. Stormwater management is the means of compensating for adverse hydrologic effects due to urbanization. Stormwater management policy is the important link between stormwater management and design. An effective policy must provide design guidelines that will lead to stormwater management facilities that meet the intent of stormwater management.

Detention basins provide a volume of storage lost through the construction of impervious surfaces, such as buildings, streets and parking lots. Two generally accepted methods of stormwater management are to construct “regional” or “on-site” detention facilities.

Typically, regional detention facilities are strategically placed within the primary watershed to detain and release, at a controlled rate, a portion of the main channel flow; therefore, completely altering the primary watershed timing characteristics. As a result, the additional stormwater runoff is redistributed without adversely impacting the primary watershed peak flow.

An on-site facility is located on the development site and functions on the principal that if the peak outflow is limited to the pre-development level, than the downstream impacts will be controlled, or at least kept, to a minimum. Limiting the

maximum peak flow to a value that does not typically exceed the peak flow that existed before development results in an outflow hydrograph that has an elongated peak-flow crest.

As a result, runoff from on-site detention facilities placed at locations in the lower portion of a primary watershed can combine and arrive downstream simultaneously with the peak flow of the primary watershed, in some cases producing downstream peak-flow rates greater than the developed watershed would have experienced without the basins. Due to this phenomenon, known as the "regional effect," some municipalities do not require detention in the lower one-third of a primary watershed.

Early research recognized that the regional effect was a possible consequence of on-site detention. As a result, research focused on the sizing and placement of regional detention facilities that would alter the primary watershed timing and effectively redistribute the additional stormwater runoff due to urban-type development.

However, regional facilities proved unpopular because they were sometimes difficult to locate, and required municipal funding and maintenance. Without this maintenance, over time, detention basins could collect sediment, and in extreme cases fill with sediment, thereby becoming completely useless.

On the other hand, on-site facilities gained in popularity because their use was seen as a viable stormwater management policy, at least in the upper two-thirds of a watershed. Additionally, their use added the possibility of a measure of pollution control before stormwater entered the stream system.

Many metropolitan areas such as Charlotte-Mecklenburg County have chosen to manage urban runoff by the use of on-site detention facilities, but little research is

available to these cities that can be used to establish detention basin design guidelines for on-site detention in the lower portion of a watershed to mitigate the regional effect. Therefore, the objective of this study was to conduct a large scale hydrologic analysis of hypothetical watersheds in order to establish these guidelines.

Accordingly, the primary goals of this research were to study the causes of the regional effect, to establish detention basin design guidelines that would mitigate the regional effect due to on-site detention, and to develop a screening level method to estimate detention basin storage.

Each subject matter was investigated by analyzing a large set of unique hypothetical watersheds and detention basins. A numerical model was used to predict the watershed runoff and detention basin outflow from each hypothetical watershed. The response from each hypothetical catchment was then used to study detention basin storage requirements or routed and combined into larger watersheds to study the regional effect.

The first step in such an analysis was to establish a range of suitable hydrologic parameters. The objective of this phase of the research was to identify all the parameters that might influence runoff characteristics and establish the possible ranges for each of these parameters. To accomplish this it was necessary to establish a study location. For a variety of reasons, it was decided to conduct an analysis of the hydrologic parameters found in the Charlotte-Mecklenburg area. We consider the Charlotte-Mecklenburg area to be hydrologically similar to many other urbanized areas in the Southeastern and Eastern United States. Charlotte's warm, humid climate with abundant rainfall, its generally rolling topography, and its location abutting a relatively large river are in many

respects similar to that found in other urban areas such as Nashville, Raleigh, Atlanta, Washington, Hartford, and Philadelphia. With this in mind, an analysis was conducted to collect typical drainage patterns and hydrologic parameter values that represent the range of values seen in the Charlotte-Mecklenburg region.

Considering the vast number of cases needed to represent all the possible combinations of watershed, development, and rainfall parameters, a computer program was needed that would sequentially update the parameters for each case to be analyzed, perform the hydrologic analysis, and print the output. No suitable hydrologic computer package was found; therefore, it was decided to custom build a hydrologic computer program. This hydrologic program was constructed by utilizing generally accepted hydrologic methods for runoff and routing prediction. It was structured to automatically calculate the hydrologic response for each unique combination of watershed, development and rainfall parameters. Separate dedicated hydrologic programs were constructed to study detention basin storage capacity and the regional effect.

The regional effect investigation placed developments at different locations within a large number of unique hypothetical watersheds and analyzed the regional impact due to uncontrolled and different levels of controlled release. Detention basin outflow was limited to the pre-development peak flow and fractions of the peak flow level by specifying a peak flow reduction factor (PFRF) that was defined as the ratio of the detention pond peak flow to the predevelopment peak flow for that watershed.

This analysis found that the location of a development, its regional response, and the boundary where uncontrolled release should not be allowed can best be presented as a percent of the watershed time of concentration. Regional impact levels increased as the

development location moved upstream from the base of the watershed. Cases having a significant regional impact began for developments placed farther downstream and regional impact levels increased with increasing soil storage capacity.

This analysis also found that for cases when development was placed in the lower portion of a watershed, the regional impact was minimal if the elongated detention basin peak-flow crest ended before the arrival of the primary watershed peak flow. In cases when this was not possible, the regional effect magnitude could be limited by decreasing the detention basin outflow to a lesser value. This research topic is fully discussed in Chapter 2 and presented in a publishable format.

The analysis method, whereby the runoff hydrographs of many hypothetical watersheds was predicted with a numerical model, was also applied to develop a screening level method to estimate detention basin storage requirements. Detention basin storage was estimated by routing the runoff hydrographs from many hypothetical catchments through detention basins with peak outflows limited to the same range of peak flow reduction factors utilized in the earlier analysis. Each hypothetical catchment runoff hydrograph was estimated using a unique combination of ground cover conditions and hydrologic parameter values.

This analysis developed statistical models that estimated the percent of runoff used for basin storage (PRBS) using the PFRF and the post-development curve number (CN). Unique models were developed for each combination of soil group (D, C, and B) and design storm duration (6-, 12-, and 24-hr). However, the relationship that existed for D, C, and B soil groups did not exist for the A soil group; therefore, unique models were developed for the A soil group using each combination of design storm frequency (10-,

25-, 50-, and 100-yr) and design storm duration (6-, 12-, and 24-hr). This research topic is fully discussed in Chapter 3 and presented in a publishable format.

In conclusion, this dissertation provides detention basin design guidelines to mitigate the regional effect due to on-site detention, and a screening-level method that quickly estimates basin storage using the same prescribed peak flow reduction factors used in the regional effect study.

CHAPTER 2

Regional Impacts Due to On-Site Detention

William L. Saunders Jr.[1] and James D. Bowen[2]

INTRODUCTION

The philosophy of stormwater runoff management has undergone a marked change since the 1970's. At that time, the common and acceptable practice was to move the water downstream as fast as possible. Current practices detain a portion of the flow, thereby reducing downstream damages that unregulated flows could cause. Urban type development results in an increase in stormwater volumes and flow, therefore requiring urban stormwater management. Cities all over the United States, like Charlotte, North Carolina, have used the newer detention philosophy of stormwater management to reduce flood damages and to also meet requirements of the National Flood Insurance Program.

Stormwater management is the means of compensating for adverse hydrologic effects that result from urbanization. Urbanization results in the construction of many surfaces impervious to water absorption, such as, buildings, streets, and parking lots. In order to partially compensate for the loss of natural water storage, construction of "on-site" or "regional" detention facilities are generally accepted methods of stormwater management.

On-site detention basins are intended to hold the increased amount of stormwater due to urbanization and to slowly release it downstream. Ideally, these basins limit the peak discharge to the peak that existed before development. Unfortunately, because of the increased volume of direct runoff, this peak discharge rate must be maintained for a longer time period compared to that which existed prior to development. Stormwater basins designed this way have been found, in some circumstances, to be seriously deficient (McCuen, 1979a) as a means of stormwater management. McCuen (1979a) has shown that the timing change caused by a detention basin can result in increased downstream flooding. According to Smith and Bedient (1980) detention basins designed using a 10-year pre- and post-development storm will not necessarily limit downstream flow to the 10-year peak flow. McCuen (1974) was one of the first to suggest that a regional approach to stormwater detention can reduce peak rates of runoff, as opposed to the purely local on-site detention approach that may actually result in increased flooding.

Regional detention facilities are strategically placed within the primary watershed to detain and release, at a controlled rate, a portion of the main channel flow; therefore, completely altering the primary watershed timing characteristics. As a result, early research focused on the sizing and placement of regional detention facilities that would alter the primary watershed timing and effectively redistribute the additional stormwater runoff due to urban-type-development. Flores (1982) developed a method to study the placement and size of regional detention facilities. Mays (1982) and Bennett (1985) refined this model to be used by others to size and place regional detention facilities. Flores (1982) found that

there was no need for any regional storage in the lower 20% of a watershed and that regional detention should be placed in the upper 40% of a watershed.

However, regional facilities proved unpopular because they were sometimes difficult to locate, and required municipal funding and maintenance. Lindsey (1992) evaluated the maintenance of detention ponds and found that the problem encountered most often was excessive sediment and debris. Sediment can displace potential stormwater storage and alter the characteristics of a storm hydrograph.

On the other hand, on-site facilities gained in popularity because their use was seen as a viable stormwater management policy, at least in the upper two-thirds of a watershed (Leise, 1991). The on-site detention basin is the most widely used form of stormwater management (McCuen, 1979a; Hawley, 1981). Additionally, the use of on-site detention added the possibility of a measure of pollution control before stormwater entered the stream system.

While designing and monitoring the effectiveness of on-site detention basins is important, it is equally important to view how the detention facility and urbanization affect the runoff characteristics downstream of the facility. Each individual facility can be designed to limit the post-development peak outflow to that of pre-development; however, peak-flow limits will inevitably also produce changes in the outflow temporal response. For instance, since development increases the volume of direct runoff, detention basin outflow hydrographs will necessarily have an elongated peak-flow crest. This elongated outflow response will result in the possibility that the peak flow of the primary watershed hydrograph and the elongated detention basin outfall peak-flow crest

will arrive simultaneously downstream, thereby increasing the primary watershed peak flow. This has been referred to as the "regional" effect (McCuen, 1974; Bennett and Mays, 1985; McCuen, 1979b; and Ferguson and Deak, 1994).

Because of the regional effect, it is not appropriate to measure the effectiveness of detention basin solely by the peak flow at the outlet of the facility. To ensure that a stormwater management system is operating according to its intent, the effectiveness of individual basins must be viewed on a regional scale. Many municipalities, like Charlotte-Mecklenburg County, have chosen to manage urban runoff by the use of on-site detention facilities; however, little research is available to these cities that can be used to establish detention basin design guidelines to mitigate the regional effect. Therefore, the objective of this study is to conduct a large scale hydrologic analysis of hypothetical watersheds to establish these guidelines.

To study this topic, a computer model was developed that investigated uncontrolled and controlled urban-type development release by studying the regional response of a large set of unique hypothetical watersheds. The regional effect analysis was conducted by placing developments at different locations within a large range of unique hypothetical watersheds. Stormwater runoff from a development was released without control and from detention basins designed using ten different peak-flow limits, and then routed through the watersheds. Regional impacts were evaluated by comparing the post-development watershed peak flow to the corresponding pre-development value.

METHODS

A computer model was developed to analyze regional watershed impact of a development placed anywhere within a hypothetical watershed. To accomplish this,

urban-type developments with uncontrolled and controlled outflows were placed at different locations within many different hypothetical watersheds and the results analyzed. Typically, detention basins are designed to restrict the post-development peak flow to the peak flow that existed before development. Therefore, to determine if this design level or a more restrictive design level is appropriate, the detention basin outflow was set to, and analyzed for, a range of outflows from 0.1 to 1.0 of the pre-development peak flow. This peak flow reduction factor (PFRF), defined as the ratio of the detention pond peak outflow to the pre-development peak flow, was included to analyze the regional effects on a watershed due to a range of detention basin design outflows. Using the numerical model, hypothetical watersheds were simulated for many different sets of parameter values that might exist in a watershed in the Charlotte-Mecklenburg County, North Carolina area, and the results noted. The data were analyzed to determine the regional impacts due to the release from uncontrolled and controlled urbanization.

The general methodology used in this analysis was to estimate the hydrologic response from catchments within a watershed using a unique set of parameters for undeveloped conditions, and then route the catchment response downstream to generate the watershed hydrologic response. The controlled and uncontrolled development hydrologic response was generated and placed sequentially at different locations within the watershed to generate the watershed hydrologic response. Detention basin peak outflow limits were set to various levels between the pre-development peak flow (PFRF = 1.0) and 0.1 of the pre-development peak flow (PFRF = 0.1). The watershed responses were analyzed by comparing the post-development watershed peak flow to the pre-

development watershed peak flow to determine watershed impact due to development. A general overall methodology flow chart is shown in Figure 2.1.

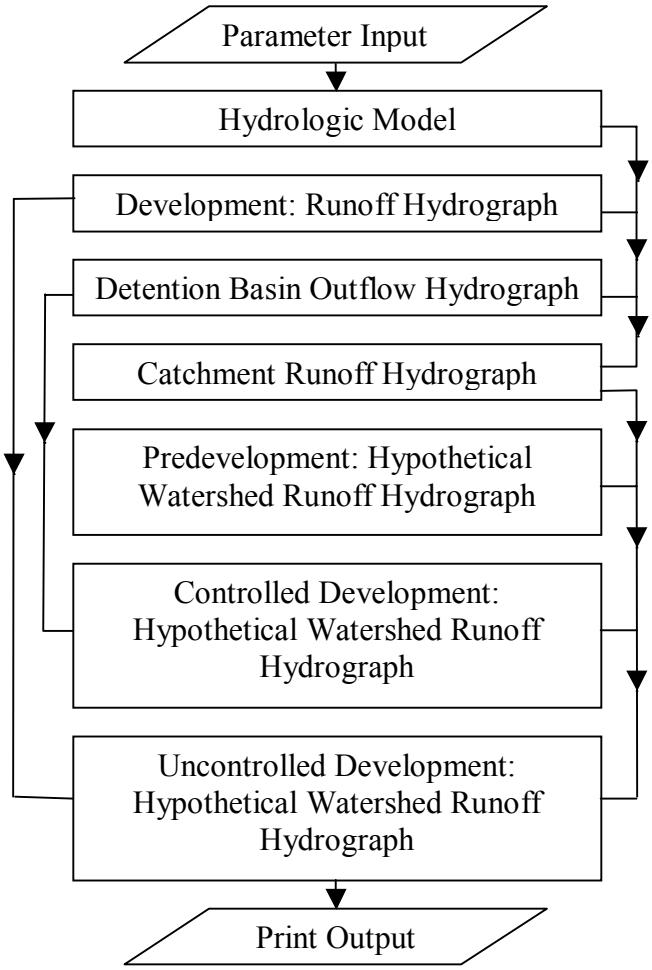

Figure 2.1 - General Design Methodology Flowchart

Some municipalities, like Charlotte-Mecklenburg, now require a minimum basin storage volume and drain-down time. Therefore, in this analysis, basins were designed to have a defined minimum storage and a 24 hour drain-down period. A simple, single mode (barrel) outlet structure did not allow the flexibility to control basin peak outflow

and the drain-down time independently. However, a multi-mode (riser/barrel) configuration allowed this flexibility, and was used in this analysis.

Detention basins were designed by estimating the barrel, riser, and drain-down orifice dimensions to limit the outflow to a predetermined value using a trial basin volume having a 1.8-m basin depth. Next, the post-development hydrograph was routed through the trial detention basin, with subsequent iterative resizing of the basin storage, until the basin depth equaled 1.8 m. We consider a basin depth of 1.8 m to be an average detention basin design depth. A detention basin methodology flow chart is shown in Figure 2.2. The parameters and the mathematical models used to develop the computer program are described below.

Hydrologic Parameter Estimation

Charlotte is the largest city in North Carolina and the 20th largest in the United States, with a population of approximately 650,000. Charlotte constitutes most of Mecklenburg County in the Carolina Piedmont. Gently rolling hills are typical to the Piedmont region, resulting in many relatively small watersheds. These watersheds drain into a large river, the Catawba, which runs along Mecklenburg County's western border. Charlotte is located in North America's humid subtropical climate zone resulting in Type II SCS rainfall distribution. Charlotte has largely clay type soils with a range of other type soils interspersed, resulting in B, C, and D SCS soil groups. We consider Charlotte-Mecklenburg area to be hydrologically similar to many other urbanized areas in the Southeastern and Eastern United States; therefore, an analysis was conducted to collect typical drainage patterns and hydrologic parameter values. Drainage patterns were used to develop the hypothetical watersheds and parameter ranges chosen to include those

found in the Charlotte-Mecklenburg area, but also include values thought to apply to other areas.

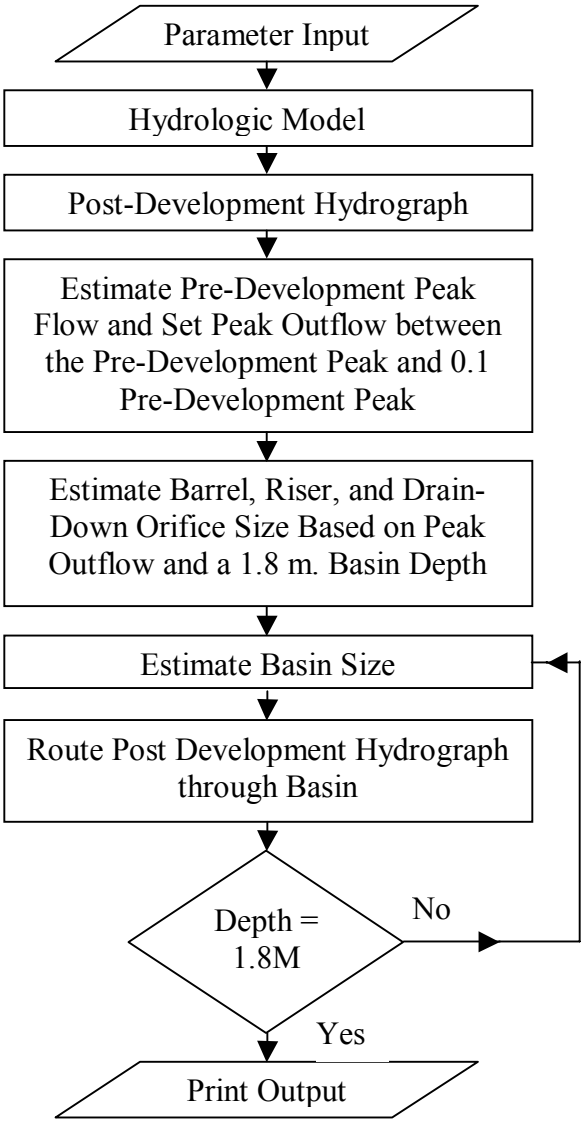

Figure 2.2 - Basin Design Methodology Flowchart

The Basins 3.0 computer program (USEPA, 2001) was used to delineate the Charlotte-Mecklenburg County region into watersheds. Watershed drainage patterns and typical values for shape, slope, soil type and size were collected.

For the Charlotte-Mecklenburg area, the relatively small creeks draining the municipality generally had one of two different drainage patterns. The most typical drainage pattern is one where a single main channel drains down the center of the watershed (Type I). The second drainage pattern is two channels running the watershed length in the shape of a V and connecting at the watershed base (Type V). Both types are studied in this analysis (Figure 2.3).

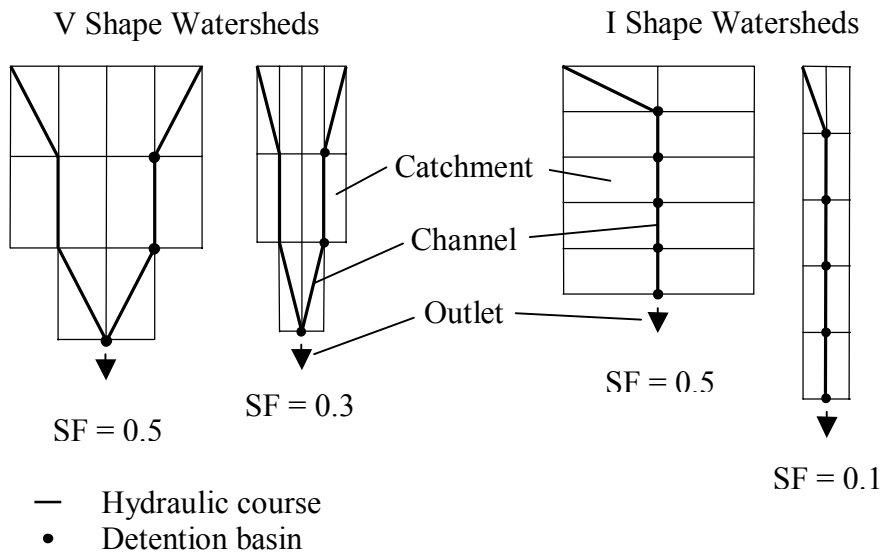

Figure 2.3 - Representation of the hypothetical watershed drainage patterns and maximum and minimum watershed shapes analyzed.

Because hydrologic response is affected by watershed shape, an analysis was conducted to find a range of watershed shapes found in the Charlotte-Mecklenburg area. Watershed shape was quantified by using a shape factor. The shape factor is defined as $SF = A/L^2$, where A is the watershed area in square kilometers and L is the hydraulic length of the watershed in kilometers. The hydraulic length is measured from the most

remote point to the watershed outlet. Nine of the most urbanized watersheds were analyzed for the Charlotte-Mecklenburg area. The analysis showed that for the "I" drainage pattern the average catchment shape factor is 0.20, with a range of values from 0.11 (elongated shape) to 0.33 (short and concentrated shape). For the "V" drainage pattern, the average catchment shape factor is 0.44, with a range of values from 0.35 (elongated shape) to 0.5 (short and concentrated shape). This study analyzed shape factors ranging from 0.1 to 0.5 and 0.3 to 0.5 for the "I" and "V" drainage pattern, respectively. The upper and lower shape limits for each drainage type analyzed are shown in Figure 2.3.

A range of values for watershed slope, and size were estimated to be 0.005 to 0.05 and 2,000 to 10,000 ha, respectively. The development size ranged from 0.1 to 2.5 percent of the watershed size. The 10yr-6hr, 10yr-12hr, and 10yr-24hr design storms, and B, C, and D soil groups were used in this analysis. Manning's roughness coefficient (n) and Muskingum's weighting factor were chosen to be 0.03 to 0.05 and 0.2 to 0.5, respectively.

The 10-year recurrence interval, 24-hour duration (10yr-24hr) rainfall for Charlotte, Mecklenburg County area is 12.39 cm. The precipitation value for the 10yr-24hr rainfall was interpolated from the precipitation data in the U. S. Weather Bureau (USWB, 1961) and the National Weather Service rainfall data (Frederick, 1977). The 24-hour rainfall hyetograph was calculated using the rainfall distribution found in the Charlotte, Mecklenburg, area (SCS Type II). The center 6 hour and 12 hour center sections from the SCS Type II rainfall distribution were used for the 10yr-6hr (8.76 cm)

and 10yr-12hr (10.44 cm) rainfall distribution, respectively. A summary of analysis parameter values and design variables is shown in Table 2.1.

Table 2.1- Summary of the parameters values and design variables.

(1)	Parameter (2)	Values (3)	# of Values Type I (4)	# of Values Type V (5)
1	# of Development Locations	Type I (5) Type V (3)	5	3
2	Watershed Shape Factor	Type I (0.1, 0.2, 0.3, 0.4, and 0.5) Type V (0.3, 0.4, and 0.5)	5	3
3	Watershed Size (ha)	2,000, 6,000, and 10,000	3	3
4	Watershed Slope (m/m)	0.005, 0.016, 0.028, 0.039, and 0.05	5	5
5	Development Size (%)	0.1, 0.7, 1.3, 1.9, and 2.5	5	5
6	Muskingum Coef.	0.2, 0.35, and 0.5	3	3
7	Manning's n	0.03, 0.04, and 0.05	3	3
8	SCS Soil Type	B, C, and D	3	3
9	Design Storms	10yr-6hr, 10yr-12hr, and 10yr-24hr	3	3
10	PFRF	0.1, 0.2, 0.3, 0.4, 0.5, 0.6, 0.7, 0.8, 0.9, and 1.0	10	10

Two different ground cover conditions were used in this analysis. "Fair-condition woods" was used for the undeveloped portion of the watershed and "Commercial" was used for the developed portion. CN values were chosen based on the ground cover conditions and SCS soil group. Commercial CN values range from 95 to 92 and from 79 to 60 for the fair-condition woods. A summary of CN values are shown in Table 2.2.

Table 2.2 - Summary of CN values chosen for each SCS soil group.

	Hydrologic soil group		
Cover Description	B	C	D
(1)	(2)	(3)	(4)
Fair condition, Woods	60	73	79
Commercial	92	94	95

Hypothetical Watershed Construction

Hypothetical watersheds were constructed to represent the range of possible watersheds found in the Charlotte-Mecklenburg area. Moreover, parameter ranges were sometimes enlarged in an effort to complete a more thorough analysis. Hypothetical watersheds were assembled to simulate type "I" and "V" drainage patterns. Both types of hypothetical watersheds were evenly divided into ten catchments. Watershed shape was established by the shape factor, and the catchment shape is determined by its dimensions as part of the hypothetical watershed. Developments were sequentially placed at five and three different locations in the I and V hypothetical watershed types, respectively. The outflow from the development was first released as uncontrolled outflow and then released after detention basin control. The outflow was released and routed through the watershed; then regional impacts analyzed for each outflow type. Figure 2.3 shows the hypothetical watershed structure with upper and lower shape limits for each drainage type.

Mathematical Watershed Model

The Soil Conservation Service (SCS) method as implemented in the HEC-1 Flood Hydrograph Package (Hydrologic Engineering Center, 1981) was used in this analysis for hydrograph formulation (Hoggan, 1996). The SCS method uses a dimensionless unit

hydrograph that is based on the analysis of measured data. To develop the SCS method, unit hydrographs were evaluated for a large number of actual watersheds and then made dimensionless by dividing all discharge ordinates by the peak discharge and the time ordinates by the time to peak. An average of these dimensionless unit hydrographs was then computed. HEC-1 implements the SCS method using a single parameter, TLAG, which equals the lag (hrs) between the center of mass of rainfall excess and the peak of the unit hydrograph. TLAG is assumed to be equal to 0.60 times the time of concentration. In this application, the unit hydrograph was interpolated for the specified computation interval, and then the computed peak flow was estimated from the dimensionless unit hydrograph. Runoff depth was estimated by the SCS curve-number method, where a curve-number estimate was obtained for the soil group and cover conditions of the watershed (Soil Conservation Service, 1975).

Time of concentration estimates were made using the techniques described in detail in TR-55 (Soil Conservation Service, 1975). TR-55 uses three different flow types: sheet flow, shallow concentrated flow, and open channel flow. Sheet flow is flow over flat surfaces usually occurring in the headwaters of a stream. Sheet flow usually becomes shallow concentrated flow, then open channel flow. Sheet flow used Manning's kinematic solution, shallow concentrated flow used equations given in Appendix F of TR-55, and open channel flow used Manning's equation to calculate travel time. The travel time for pipe sections in developments were calculated using an average velocity of 0.91 meters per second (Malcom, 1997). Time of concentration was determined by estimating travel time for each flow type and summing all individual travel times for a total travel time. Time of concentration for pre-development conditions was estimated by

dividing the total hydraulic travel distance into three different flow segments (sheet, shallow concentrated, open channel). Sheet flow travel time was estimated using the Manning's kinematic solution. The maximum flow distance to be used with this formula is 91.4 m. A flow distance of 76.2 m and a Manning's n of 0.4 (woods-light underbrush) were assumed to be typical. After a travel distance of 76.2 m, sheet flow is assumed to become shallow concentrated flow. The travel distance used for shallow concentrated flow was assumed to be another 76.2 m, making a total travel distance of 152.4 m. At this point the catchment area is about 1.2 hectares, and the remaining travel distance is assumed to be open channel flow. Open channel flow was calculated using the Manning's equation. Time of concentration for commercial development conditions was estimated by dividing the total travel hydraulic distance into four different segments. Flow segments were sheet, paved shallow concentrated, pipe, and open channel. Sheet flow travel time was estimated using the Manning's kinematic solution. Sheet flow used a Manning's n of 0.24 (dense grass) and a travel distance equal to a property line offset length of 15.2 m. From this point, paved shallow concentrated flow was assumed with a travel distance of 15.2 m. From this point, pipe flow was assumed. Pipe flow travel distance was assumed to be a maximum of 329 m. At this point, the catchment area is about 5.4 hectares, and the remaining travel distance is assumed open channel flow. Open channel flow was calculated using Manning's equation.

Estimation of the hydraulic radius for channels was necessary to evaluate the travel time through the channel portion of the watershed. Therefore, the following method was developed to estimate the channel hydraulic radius based on drainage basin size. Research has shown that channel cross-sectional area and width are highly

correlated with the size of the basin (drainage area). For instance, Doll (2002) collected and presented bankfull channel dimensions for Charlotte-Mecklenburg and other urbanized areas in North Carolina. Equations presented estimated channel cross-sectional area, width, and flow depth based on the size of the basin. Using these relationships and the geometrical relationship of a trapezoid, the following relationship was developed to estimate the hydraulic radius based on the size of the drainage basin.

$$R = 10.47 D_A^{0.3199} \tag{1}$$

Where R is the hydraulic radius (cm) and D_A is the drainage area (hectares). This relationship was used in this study to estimate hydraulic radius.

Mathematical Basin Model

Basin storage was estimated by the following power-curve that can easily estimate storage volume for a range of assumed shapes and sizes.

$$S = KsZ^b \tag{2}$$

Where S is the storage volume (m^3), Z is the basin stage (m) measured from the bottom of the reservoir, A_R is the reservoir surface area (m^2), Ks is the basin surface area (m^2) 0.3 m from the bottom of the reservoir, and b is a coefficient that determines the side slope of the reservoir. When b equals 1, the sides are vertical.

The detention basin outflow hydraulics utilized a typical riser/barrel configuration (Figure 2.4). The riser rim is a vertical pipe acting as a weir with length equal to its circumference. The horizontal barrel connected to the bottom of the riser acts independently as a culvert under inlet control. The flow over the riser was estimated by a basic weir equation and sized such that the flow did not transition through orifice control before the barrel acted to limit flow. The flow through the barrel and drain-down orifice

was estimated using an orifice flow equation with a basin depth of 1.8 meters. The drain-down orifice was sized to empty the storage in a 24-hour period.

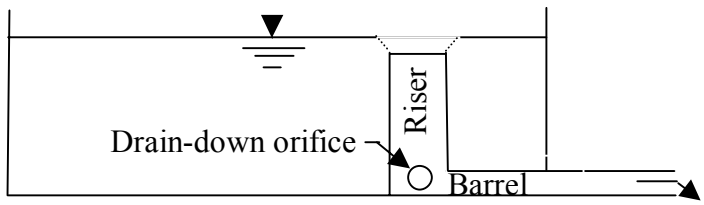

Figure 2.4 - Representation of the detention basin shape and the riser/barrel outflow configuration with drain-down orifice.

Basin storage was estimated and the post-development hydrograph was routed through the detention facility using the Storage-Indication Method (Malcom, H.R., 1989). If the routed basin depth did not match the 1.8-meter design depth, the basin storage was adjusted and the post-development hydrograph rerouted until the design depth and routed depth matched. This outflow hydrograph was placed sequentially at each of the locations and routed downstream.

The computer model was constructed in Microsoft® Excel® using the generally accepted hydrologic methods previously discussed. Within the spreadsheet, a different sheet was used for individual tasks. One sheet generated the hydrographs. Another sheet designed the detention facility and routed the uncontrolled hydrograph through the detention basin to generate the outflow response. A third sheet estimated time of concentration and handled parameter input and output. Additional macros were written to complete tasks not normally accomplished in a spreadsheet, such as channel routing, detention basin sizing, and data input and output.

The analysis was performed by identifying and grouping scenarios that either did or did not impact the regional peak flow. The group of interest was then analyzed by organizing, graphing and interpreting the data visually. Histograms were constructed for each parameter to identify the parameters that might impact the placement of developments in a watershed.

RESULTS

During the course of the analysis, 2,772,050 watershed responses were generated and analyzed. In excess of 540 hours of computer-time was necessary to generate watershed responses, and 2.2 gigabytes of data storage was used to store the results.

Drainage Type I was evaluated using six different parameters having a total of 24 different values (Table 2.1). The individual values for each parameter has been previously shown in Table 2.1, Column 3. This number of parameters and values resulted in 3,375 watersheds, each with a unique set of physical and routing parameters.

Drainage Type V was evaluated using six different parameters with 22 different values. The number of values for each parameter was shown in Table 2.1, Column 5, with the individual values shown in Column 4. This combination of parameters and values results in 2,025 watersheds, each with a unique set of physical and routing parameters. Each unique watershed was evaluated for the uncontrolled release and ten different PFRF's, resulting in 185,625 and 66,825 watershed responses for the Drainage Type I and V, respectively.

Each response was evaluated using three different design storms and three different soil groups, resulting in a total of 2,272,050 unique watershed responses (9

storm/soil type cases, 252,450 total watersheds). The numerical model was constructed using generally accepted hydrologic methods and checked for accuracy; however, considering the complexity of the hydrologic and scenario testing procedures, it was decided that a preliminary model verification test was warranted.

The numerical model was verified by comparing the calculated watershed response to the response using HEC-1 (Hydrologic Engineering Center, 1981). HEC-1 was chosen because it is one of the most widely recognized hydrologic models, and its capability to utilize similar numerical methods. Verifying every unique watershed response was unrealistic; therefore, the numerical model was verified using ten randomly selected watershed scenarios.

The verification process compared each of the ten cases for runoff hydrograph peak flow, volume, and volume before peak flow. In all ten cases, the calculated hydrographs were visually similar to those produced by HEC-1 when graphed and displayed as shown in Figure 2.5. Figure 2.5 shows the case with the largest peak-flow error. For every case examined, time to peak flow was identical to that calculated by HEC-1.

The total runoff volume varied between the numerical model and HEC-1 by an average of 0.36 percent, with a minimum of -0.93 percent and a maximum of 0.02 percent. The volume before the peak flow differed from the HEC-1 value by an average of 0.50 percent, with a minimum of 0.14 percent and a maximum of 0.80 percent. Peak flow differed by an average of 0.36 percent, with a minimum of -0.07 percent and a maximum of 0.78 percent. Based upon these results, it was decided that the numerical

model did produce hydrographs that were essentially identical to the HEC-1 results, and could be used to perform the subsequent scenario testing.

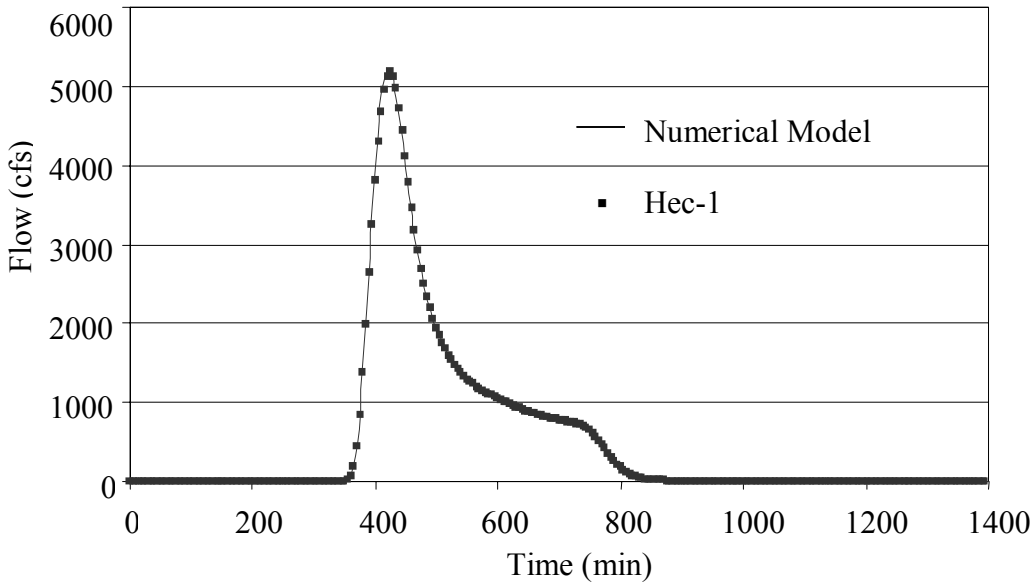

Figure 2.5 - Numerical model verification using Hec-1.

The numerical model generated pre- and post-development primary watershed hydrographs for every possible parameter combination. Hydrograph peak flows and necessary watershed timing data was saved for each simulated watershed. The output data were then analyzed to determine regional impact. A macro was written to segregate the simulated watersheds into those either having, or not having a measurable regional effect. The numerical model generated a primary watershed response for uncontrolled and controlled development released at different locations of a watershed. The "uncontrolled" analysis grouped simulated watersheds that resulted in an impact greater than that found with pre-development conditions. The "controlled" analysis grouped

simulated watersheds that resulted in an impact less than that found with pre-development conditions. Histograms were created that showed the impact level distribution for both "uncontrolled" and "controlled groups. Both "uncontrolled" and "controlled" groups resulted in a number of simulated watersheds showing a small level of change when compared to the pre-development level. Because there was no way to determine if these cases were true hydrologically relevant responses, or simply false positive responses, due to the intrinsic variations within the numerical model. A minimum response level was established, and cases below this minimum were removed from the analysis.

The numerical model was constructed by checking each hydrologic method for accuracy and verifying the overall response with HEC-1. Next the model was run using the three different design storms (10yr-6hr, 10yr-12hr, and 10yr-24hr) and three different SCS soil groups (B, C, and D); resulting in nine separate sets of scenario tests. Each set of scenarios was then analyzed to identify which cases produced a measurable regional effect.

Uncontrolled Release Results

The uncontrolled analysis placed urban-type development at different locations in the lower portion of a watershed and investigated the percent increase in regional flow due to the release of uncontrolled runoff. The time of concentration was identified as a watershed descriptor that could be used to differentiate the regional effect of the individual cases examined. The time of concentration is defined as the travel time that it takes a particle of water to reach the catchment discharge point (Wanielista et al., 1997). After analyzing thousands of different hypothetical watersheds, each with different time

of concentrations, it was found that all watershed responses could be presented as a percent of the time of concentration (PTOC), where the base of the watershed is zero PTOC and the origin of the watershed is 100 PTOC. The position of the development within the watershed was defined as the travel time from the point of entry into the channel system to the base of the watershed divided by the watershed time of concentration times 100.

The analysis began by examining developments with uncontrolled releases, selecting the cases where the post-development regional peak flow was greater than pre-development value. The magnitude of the regional effect was quantified by calculating the percentage increase in regional peak flow. Developments resulting in positive regional impact were plotted with the percent time of concentration (PTOC) on the x axis and the percent increase in regional peak flow on the y axis. The results seen for the 10yr-24hr design storm and SCS C soil group (Figure 2.6) were typical of those seen for all nine storm/soil type combinations.

The various regional effect levels resulting from uncontrolled runoff (Figure 2.6) is the result of modeling 22,950 developments, placed at different locations within hypothetical watersheds having a range of sizes, shapes, and other hydrologic parameters; where 7,390 (33.2 percent) of the developments resulted in a measurable impact.

The magnitude and range of development impact increased as the PTOC increased. Developments located from 0 to 15 PTOC resulted in no regional impact. The percent change in flow increased linearly from fifteen to thirty five PTOC; while the maximum impact remained under 5 percent change in regional peak flow (Figure 2.6).

From 35 to just over 60 PTOC the maximum level and range of impact increased exponentially.

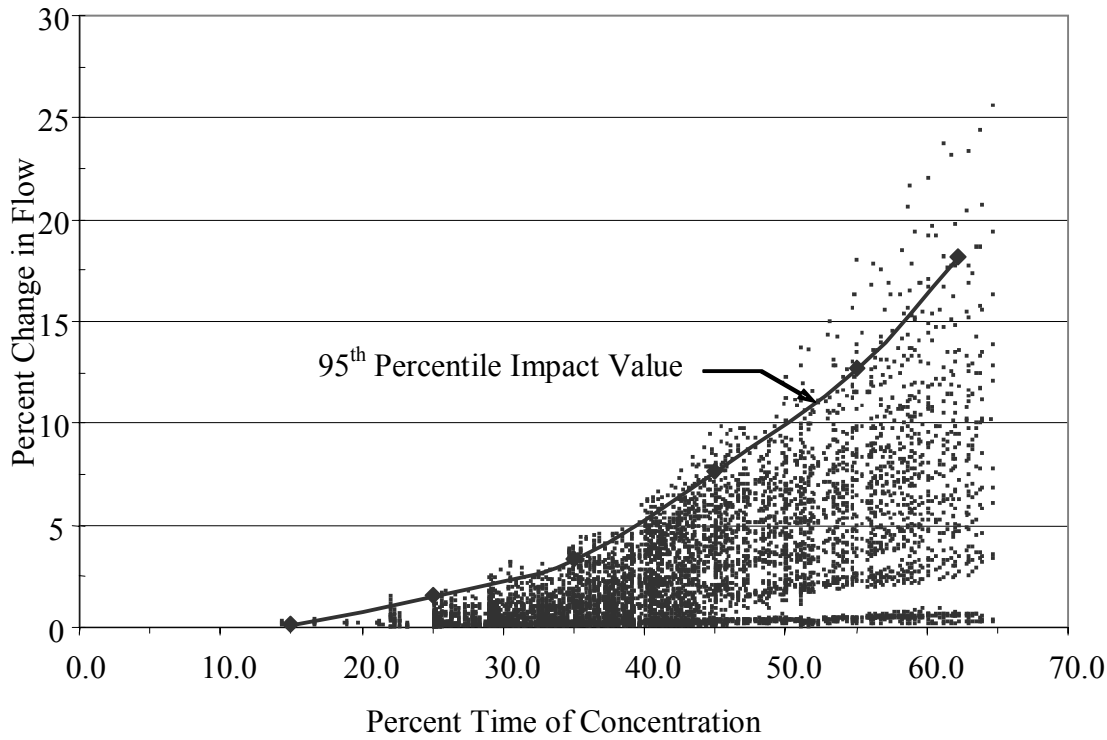

Figure 2.6 - Uncontrolled regional watershed response for the 10yr-24hr design storm and SCS C soil group. Developments resulting in positive regional impact were plotted as a percent time of concentration on the x axis and a percent increase in regional peak flow on the y axis. The 95th percentile impact value is the value where 95 percent of the values are at or below that level. The percent time of concentration is the travel time from the point of entry into the channel system to the base of the watershed divided by the watershed time of concentration times 100.

Muskingum "C", Channel "n", Shape Factor, watershed slope, percent development, and percent time of concentration were statistically evaluated to determine each parameters' importance in estimating the percent change in regional flow. The statistical analysis used the SPSS™ statistical package, version 10.0.7 (SPSS 1999).

Each parameter was analyzed using a "backward elimination" sequential search. Steps in this analysis were as follows:

1. A single regression equation was computed using all of the predictor variables of interest.
2. A t statistic was calculated to test for each variable.
3. Predictor variables were eliminated that were not statistically significant at the 95 percent level (two-tailed test, $\alpha = 0.025$)
4. Regression models were re-estimated using only the remaining predictor variables.
5. Return to step two and continue the process until all variables of interest and their contributions determined.

Percent time of concentration estimated the percent change in regional peak flow with an R^2 of 0.325; however, no single parameter or combinations of hydrologic parameters were found to significantly improve percent change in flow prediction.

Therefore, the percent change in flow response from each of the nine design storm, soil group combinations was summarized by a line of 95 percent level of impact. Each response was evaluated and, if appropriate, used to combine the nine unique combinations of design storms and SCS Soil Types.

It was found that individual soil groups responded similarly to different storm durations, therefore, the responses from the three different design storms (10yr-6hr, 10yr-12hr, and 10yr-24hr) were averaged and presented as one response for each soil group (Figure 2.7). The error bars represent the different level of response due to design storm duration, where the maximum level is due to the 24 hr duration design storm and the

minimum level is due to the 6 hr. This analysis summarized the response of 206,550 unique numerical model scenarios.

The regional impact increased as the PTOC increased for all three soil groups. The regional impact increased gradually from zero and 35 PTOC (Segment I) while having a maximum regional impact of less than five percent. The regional impact increased exponentially from 35 PTOC to about 60 PTOC (Segment II). The SCS hydrograph method defines the time to the hydrograph peak (T_p) as sixty percent of the time of concentration; therefore, the maximum impact should be expected at a PTOC level of about 60, where the travel time of the development discharge equals the time to peak of the regional watershed.

The response found in Figure 2.7 can be explained by using the drawing shown in Figure 2.8. This Figure displays three hydrographs illustrating the relationship between the uncontrolled development runoff and its impact on the regional hydrograph. When developments were located in the lower section of a watershed (Segment I), the level of regional impact, as noted by point 1, was relatively small and increased slowly as the development was placed farther upstream. Eventually, the regional impact increased exponentially as development location moved farther upstream (Segment II). The regional impact due to developments located within this section, as noted by point 2, were at a higher levels and within a part of the uncontrolled development hydrograph that changed quickly as development moved upstream and changed the relative timing of the development outflow and regional hydrographs (Figure 2.8).

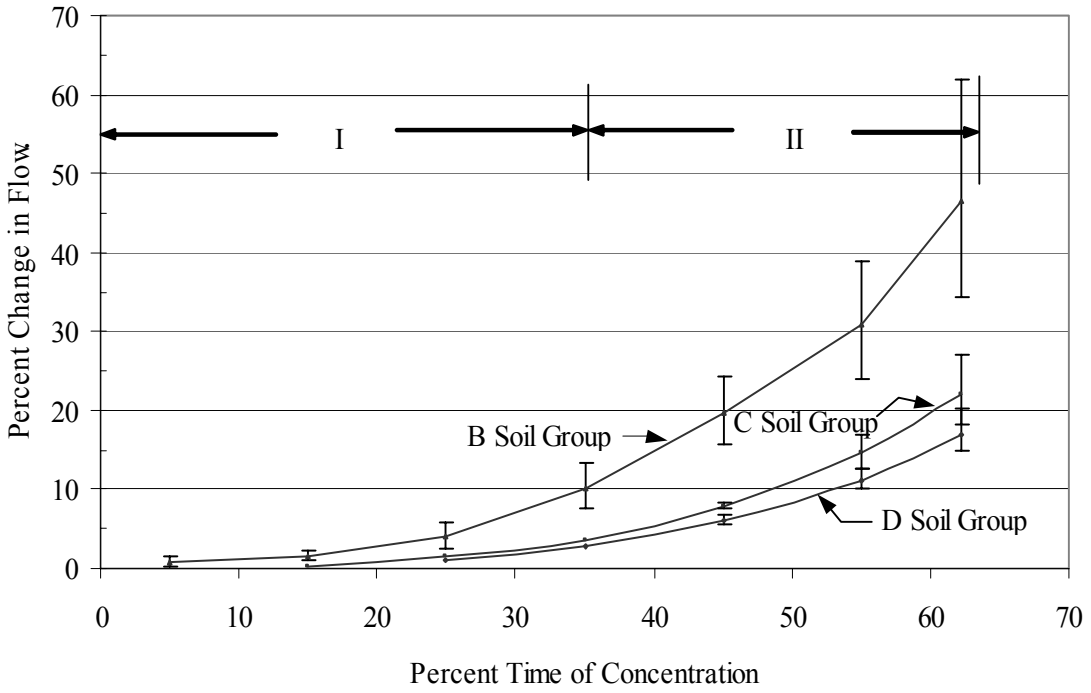

Figure 2.7 - Uncontrolled regional watershed response for B, C, and D soil groups. Each soil group response is the average of the 95th percentile impact response for the three design storms (10yr-6hr, 10yr-12hr, and 10yr-24hr) for each soil group. The error bars represent the different level of regional impact due to design storm duration. The percent time of concentration is the travel time from the point of entry into the channel system to the base of the watershed divided by the watershed time of concentration.

The downstream location where impact begins and the percent of regional impact scenarios change with soil groups. Regional impact for the B soil group begins between 0 to10 PTOC, with a total 81.4 percent of the developments scenarios resulting in a measurable regional impact. The regional impact for the C soil group begins between 10 to 20 PTOC, with 33.2 percent of the development scenarios resulting in a regional impact. Regional impact for the D soil group begins between 20 to 30 PTOC, with a total of 30.8 percent of the development scenarios resulting in regional impact. Thus it was

seen that the overall level of regional impact increased and the regional impact began with developments placed farther downstream as soil storage capacity increased.

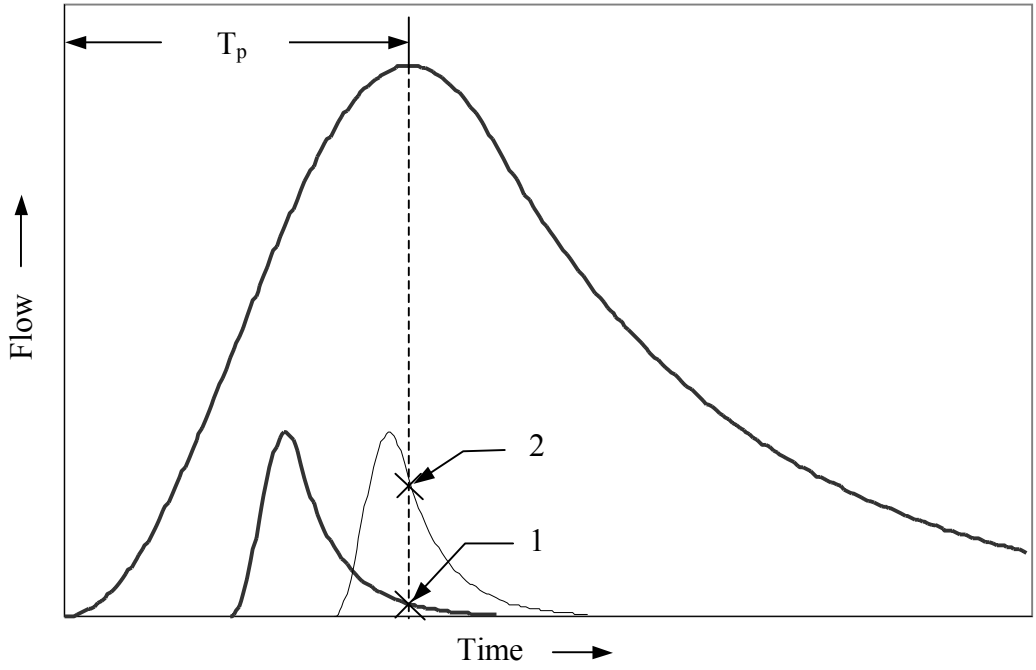

Figure 2.8 - Rendering illustrating the change in regional impact as development location moves upstream.

Curve number (CN) values used in this analysis for different ground cover conditions and soil groups were previously shown in Table 2.2. These curve number values indicate the variations in soil storage capacity considered in the analysis. Pre-development soil storage capacity increased significantly for D, C, and B soil groups, respectively. However, post-development storage capacity changed little for D, C, and B soil groups; which resulted in a varying level of development impact for the various soil group cases. Runoff volume increased as the difference between pre- and post-

development CN values increased. The difference between pre- and post-development CN value for D, C, and B soil groups is 16, 21, and 32, respectively. Therefore, the resulting C soil group regional impact was slightly greater than that for D soil group and the B soil group regional impact was much greater than that from C soil group.

In the SCS hydrograph generation method, runoff volume is calculated as the volume of rainfall remaining after storage capacity is reached. Thus, in this application, as storage capacity increased, runoff volume decreased; and as a result, with the same time of concentration, peak flow decreased. Consequently, as the difference between the pre- and post-development CN value increased, the development impact increased, and ultimately, the level of regional impact increased and regional impact locations began farther downstream as the soil storage capacity increased.

Controlled Release Results

The controlled analysis placed urban-type development at different locations in the lower portion of a watershed and estimated the maximum allowable detention basin outflow limit that did not increase regional peak flow.

Typically, detention basins are designed to restrict the maximum outflow rate to the peak rate that existed before development. In order to set a range of outflow rates, a detention basin peak flow reduction factor was established. This peak flow reduction factor (PFRF) resulted in a range of detention basin peak outflow rates from 0.1 to 1.0, divided into increments of 0.1, of the peak flow rate that existed before development. The pre-development peak flow multiplied by the PFRF was the design peak outflow level.

The post-development response from each catchment was routed through detention basins that were designed to limit the outflow rates to the prescribed value based on the pre-development peak flow and the selected PFRF. The basin outflow hydrograph was then routed through the watershed. Each post-development watershed hydrograph response was then compared to the corresponding pre-development values. Cases that successfully eliminated the regional impact were then identified for further analysis.

As found in the post-development uncontrolled response results, the controlled response can be presented as a percent of the time of concentration (PTOC). The PTOC was grouped into bins of ten PTOC, the successful no impact design scenarios were collected for each bin, and a distribution of the successful PFRF's was found.

The PFRF distribution was then analyzed and the PFRF 95th percentile value found for each PTOC bin, where 95 percent of the successful design scenarios were at or less than that particular PFRF value.

Nine separate model runs were completed; with each model run having a different combination of the three different design storms and the three different soil groups. After analyzing the 95th percentile PFRF response for each of the nine combinations, it was found that there was little variation in the PFRF response between the different design storms. However, the 95th percentile PFRF response changed for different soil groups and different combinations of soil groups; therefore, the results from the different soil groups cases were selectively combined.

Results shown in Figure 2.9 are the product of 2,065,500 different design scenarios, evaluated for a range of PFRF's, and placed at different locations within unique watersheds having a range of sizes, shapes, and other hydrologic parameters.

The error bars represent the variation in the 95th percentile PFRF due to the different design storms. The required PFRF needed to eliminate the regional impact was calculated as the average PFRF found for the three different design storms and soil groups. A critical location exists at a PTOC of 35 where the PFRF response changed. This critical location also exists in the uncontrolled response as previously shown in Figure 2.7. Therefore, the controlled response as shown in Figure 2.9 was divided into these two segments. Segment I was the range from zero to 35 PTOC, where the response of the B soil group and the C and D soil group were distinctly different. Segment II was the range from 35 to about 60 PTOC where the responses from the three soil groups were nearly identical.

The average 95th percentile required PFRF for C and D soil groups in Segment I was approximately one from zero to 25 PTOC and decreased to a required PFRF of 0.7 from 25 to 35 PTOC. The average 95th percentile required PFRF for B soil group in Segment I was approximately 0.5 from zero to 25 PTOC and increased to a required PFRF of 0.7 from 25 to 35 PTOC. The average 95th percentile required PFRF for all three soil groups in Segment II was dome shaped and varied from an average minimum of 0.7 to an average maximum of 0.8.

As discussed earlier, the basin storage capacity for pre-development conditions increased significantly for the D, C, and B soil groups, respectively; however, storage capacity for post-development conditions changed inconsequentially for the three soil

groups. Therefore, there were different levels of development impact for each soil group. Runoff volume and peak flow increased as the difference between pre- and post-development CN values increased. The difference between pre- and post-development CN values for D, C, and B soil groups was 16, 21, and 32, respectively. The difference between pre- and post-development CN values for the D and C soil groups was within five CN, resulting in the combining of the D and C soil groups and a separate response for the B soil group where the increased runoff volume was much greater.

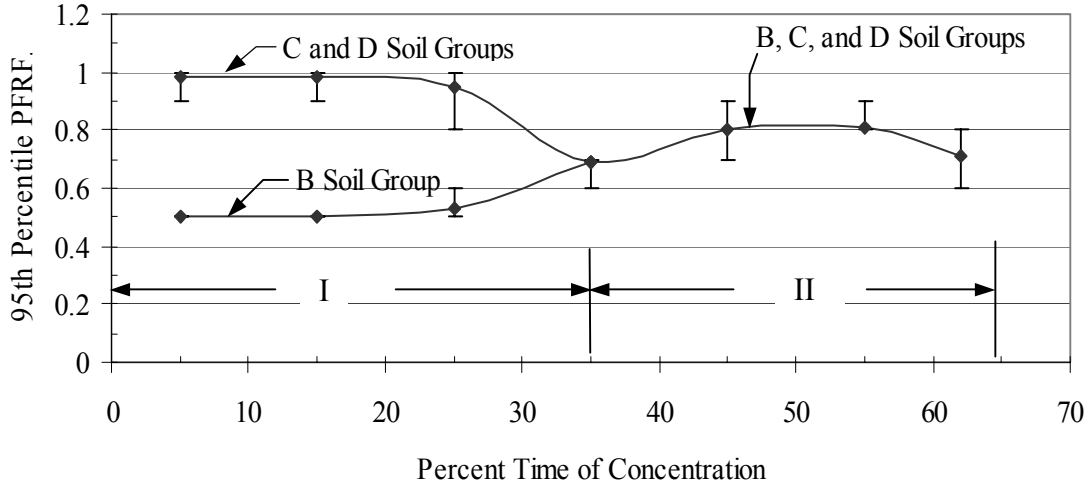

Figure 2.9 – Controlled development response showing two different PFRF relationships for different soil groups. The 95th percentile peak flow reduction factor (PFRF) design level to mitigate the regional effect for locations in the lower portion of a watershed for B, C, and D soil groups. The percent time of concentration is the travel time from the point of entry into the channel system to the base of the watershed divided by the watershed time of concentration times 100.

Detention basins are typically designed to limit the outflow which existed before development. Therefore, pre-development peak flow determines the detention basin outflow design limit, thus the pre-development outflow limit changed with soil group.

Pre-development CN for the B, C, and D soil groups are 60, 73, and 79, respectively (Table 2.2). As a result, runoff volume and peak flow increased for the B, C, and D soil groups, respectively. Therefore, due to development, the B soil group resulted in the most restrictive outflow design limit and the greatest addition runoff volume to control, and the D soil group resulted in the least restrictive outflow limit and the smallest amount of additional runoff volume to control.

The detention basin outflow structure used in this analysis was a typical riser-barrel configuration as previously shown in Figure 2.4. This outflow structure resulted in an outflow response having an elongated peak flow region followed by a 24 hour drain-down period. The length of the elongated peak flow region was determined by the basin peak outflow limit and the amount of runoff volume to be controlled. A detention basin design using a small outflow design limit and a large amount of runoff to control would result in a long period of outflow at the maximum outflow limit. A detention basin designed using a larger outflow design limit, and a smaller amount of runoff volume to control, would result in a shorter period of outflow at the maximum outflow limit.

Due to the more restrictive outflow limit and increased runoff volume found for the B soil group, the outflow from detention basins designed for B soil group have a longer period of maximum outflow than found for C and D soil groups. A rendering of this phenomenon is shown in Figure 2.10 that illustrates the different outflow response, using a PFRF of 1.0, for C and D soil groups and for the B soil group shown in Figure 2.9.

The pre-development response and its impact on the primary watershed hydrograph will determine the level of regional impact that should be maintained after

development. Each response was estimated using the same catchment where the only difference was soil group, while the pre-development peak flow limit using a maximum PFRF of 1.0 was established by soil group.

Segment I (0 to 25 PTOC) PFRF Results

Preexisting regional impact was small as shown in the "Pre-development" (upper) illustration of Figure 2.10. This drawing illustrates the preexisting regional impact due to a proposed development located within Segment I. The preexisting impact was small; therefore, the impact after development should be just as small to maintain the same level of regional impact.

The post-development impact is shown in the "Post-development" (lower) illustration of Figure 2.10. Two different basin outflow responses are shown; one is the response using the C and D soil groups and the other is the response using the B soil group. Soil group was the only design parameter difference between the two different basin responses. The response for the C and D soil groups showed that the maximum outflow had short peak-flow duration that would come and go relatively quickly; therefore, the post-development regional impact was less than the pre-development regional impact. However, the longer duration peak-flow response for the B soil group was the result of a more restrictive pre-development outflow design limit and larger runoff volume.

Since the response of the B soil group showed longer peak-flow duration, the post-development regional impact was greater than the pre-development regional impact. As a result, detention basin responses that have short peak-flow duration do not impact primary watershed peak flow, and detention basins responses that have long peak-flow

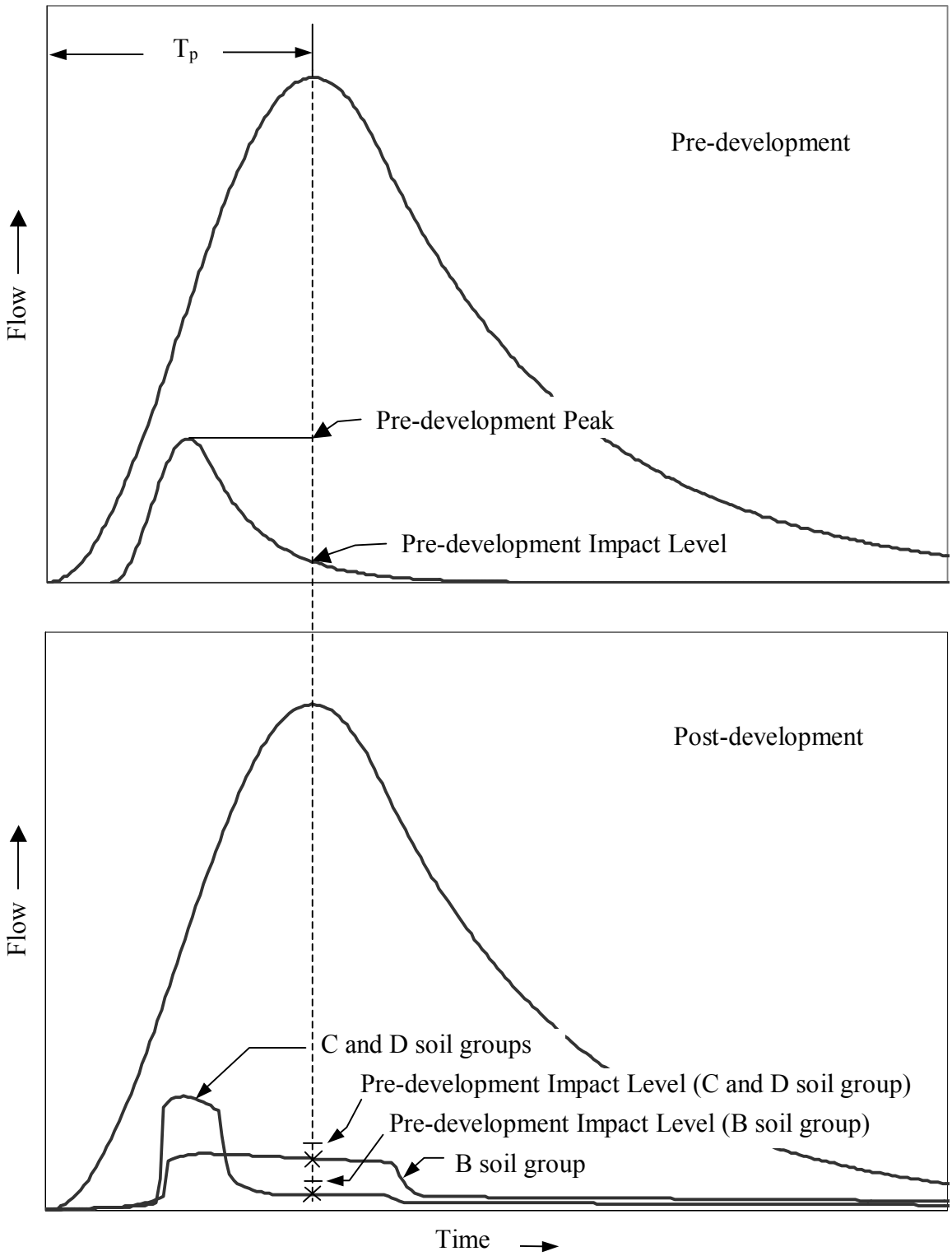

Figure 2.10 - Rendering illustrating regional post-development response that controls the PFRF reaction in Section I, Figure 2-9.

duration will increase regional watershed peak flow. Therefore, detention basins designed for C and D soil groups, resulting short peak flow durations can be designed using a 1.0 PFRF design level to minimize peak-flow duration and maintain pre-development peak outflow levels in the regional watershed. Detention basins designed for B soil group, having long peak flow duration, should be designed using a 0.5 PFRF design level, or lower, to maintain pre-development peak flow levels in the regional watershed.

Segment I (25 to 35 PTOC) PFRF Results

The separate responses found for the C and D soil groups and the B soil group at 25 PTOC begin to converge into one response. The three soil groups fully combine at 35 PTOC with a 0.7 PFRF. Beginning at 25 PTOC, the preexisting regional impact begins to slowly increase as development location moves upstream, thus collapsing the different post-development responses into one soil group response.

Segment II (35 to 60 PTOC) PFRF Results

The separate responses found earlier for the C and D soil groups and the B soil group have fully combined into one response at 35 PTOC. The PFRF begins to rise because the preexisting regional impact becomes greater as development moves farther upstream; this concept is illustrated in the "Pre-development" (upper) rendering of Figure 2.11. The two different soil group responses have combined into one response because the post-development response was greater than the pre-development impact level for all soil groups. This resulted in the same post-development impact levels as development continued to move upstream for all soil groups; this concept is illustrated in the "Post-development" (lower) rendering of Figure 2.11.

DISCUSSION AND CONCLUSIONS

Abt (1978) and Flores (1982) studied the impacts and control of the regional effects through the used of regional detention basins. Regional detention basins are placed within the main channel (in line) of a watershed. This earlier research showed that regional watersheds operate more efficiently when placed in the upper part of a watershed. Flores (1982) also found that regional detention facilities were needed only in the upper 40 percent of a watershed. The analysis conducted herein found that the necessary on-site detention storage needed to avoid a regional impact increased as development location moved upstream for C and D soil groups, and that the necessary detention storage decreased as development location move upstream for B soil group.

Regional detention basins placed within the main channel of a watershed completely alter watershed timing characteristics, and as a result, change the timing characteristics of the discharged runoff volume. However, on-site detention did not drastically alter the regional watershed timing thus as found in the analysis conducted herein, the additional runoff due to development using on-site detention should be redistributed either before the watershed time to peak (T_p), or at a lower peak flow level through the T_p to mitigate the regional effect.

Flores (1982) found that there was no need for regional detention in the lower 20 percent of a watershed. Curtis (1977) found that uncontrolled release in the lower portion of a watershed may have little or no regional effect and on-site detention ponds placed in the lower portion of a watershed can actually increase the primary watershed peak flow. As a result of these earlier studies, some municipalities have not required on-site

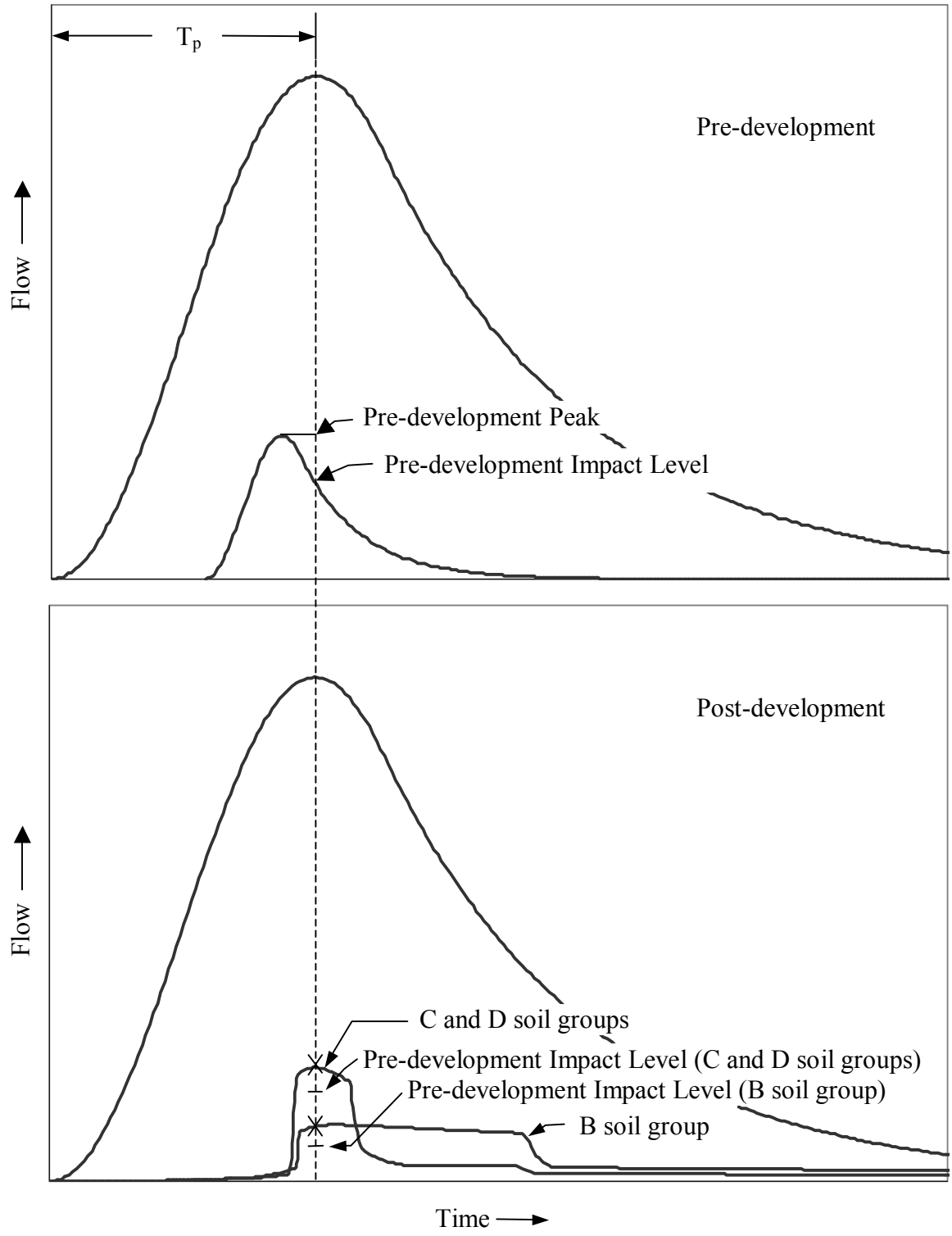

Figure 2.11 - Rendering illustrating regional post-development response that controls the PFRF reaction in Section II, Figure 2-9.

detention in the lower one-third of a primary watershed. The analysis conducted herein found that on-site detention was not necessary below 25 PTOC for the B soil group and below 35 PTOC for C, and D soil groups. If uncontrolled release is considered, this improved method, which considered soil type differences, should be used to delineate the portion of the watershed where uncontrolled release is allowed.

The level of regional impact due to uncontrolled release as the development location moves upstream is shown in Figure 2.9. The percent change in flow was less than five percent when the PTOC was from zero to 35 for C, and D soil groups, and from zero to 25 for the B soil group. The relationship was clearly established between PTOC and maximum regional impact for different soil groups. This relationship more accurately defines and delineates the region where uncontrolled release may be allowed.

This analysis examined regional watershed impact only and did not consider any local drainage impact due to additional uncontrolled runoff. Local drainage conditions should be considered before adopting any type of stormwater management program.

Some municipalities are currently changing basin design regulations in an attempt to mitigate the regional effect and settle out pollutants to satisfy NPDES, Phase II requirements. To meet this goal, municipalities will have to require on-site detention throughout the watershed.

Detention basins designed for pollution control require an extended detention time for pollutants to settle out; as a result, some municipalities, like Charlotte-Mecklenburg, are now requiring a minimum basin storage volume and longer drain down time.

Therefore, in this analysis, basins were designed to utilize a large inactive storage and a 24 hour drain down period. A simple, single mode (barrel) outlet structure did not

allow the flexibility to control basin peak outflow and the drain down time independently. However, a multi-mode (riser/barrel) configuration allowed this flexibility, and was used in this analysis. There are many variations of outlet structures regularly used in basin design. Outflow response can vary greatly with the wide variety of outlet structures and other basin design criteria. However, peak basin outflow will occur at the same time regardless of the basin design.

This analysis considered only the worst case development (commercial). In reality, there are many lesser impact levels from development and combination of soil groups. These differences result in different levels of basin storage, and therefore, different levels of peak flow duration that were not considered in this analysis. This analysis established that the duration of peak outflow was an important factor to mitigate regional impact in basin design, and peak flow duration can be estimated for every design.

Results show that, for developments placed in the lower portion of a watershed, there were new factors in basin design to consider that can mitigate the regional effect. Detention basins designed using the same peak flow that existed before development and a small amount of additional runoff will result in a short peak flow duration that will come and go before affecting primary watershed peak flow. However, the same basin designed with large additional runoff will result in a long peak-flow duration that will extend into a portion of the primary watershed hydrograph that will impact regional peak flow. Therefore, these basins should be designed using a lower peak outflow limit to mitigate regional impact. Figure 2.9 shows this required PFRF relationship between development location and soil group.

Different soil groups do not directly alter the 95th percentile required PFRF response to development location within a primary watershed. Extra runoff volume from the B soil group resulted in longer outflow peak flow durations. As a result, longer peak outflow durations resulted in a different PFRF response due to development location. Therefore, Figure 2.9 was redrawn and labeled to show that timing issues were responsible for the different 95th percentile PFRF responses and shown in Figure 2.12.

Clearly, PFRF is another important factor to consider in basin design to mitigate regional impact due to basin design. With the wide variety of basin designs, and using Figure 2.12, a general design method can be developed to mitigate the regional effect.

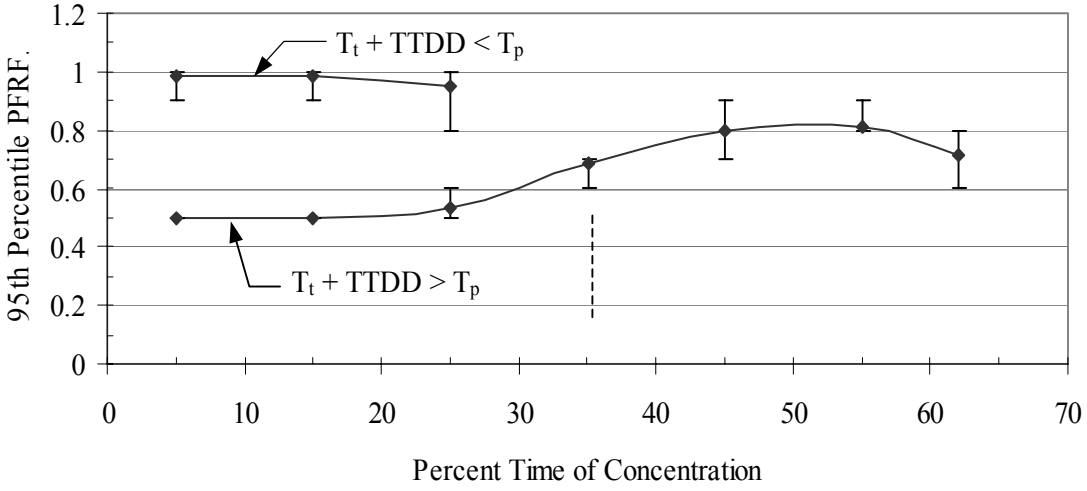

Figure 2.12 – Controlled development peak flow reduction factor (PFRF) responses for watershed timing characteristics. Controlled development peak flow reduction factor (PFRF) responses for the time of travel (T_t) plus detention basin peak flow duration (TTDD) was greater than time to peak (T_p) or T_t plus TTDD was less than T_p. The percent time of concentration was the travel time from the point of entry into the channel system to the base of the watershed divided by the watershed time of concentration.

This analysis found that the regional effects due to on-site detention can be mitigated by controlling the level and distribution of the extra runoff throughout the watershed. As previously discussed, there are four important design factors used to design detention basins to mitigate the regional effect: Development location within the watershed, basin peak flow duration, watershed time to peak, and peak flow reduction factor (PFRF). Development location was defined as the travel time (T_t) from where the basin outfall enters the channel system to the base of the primary watershed divided by the primary watershed time of concentration. The time to drain down (TTDD) was defined as the time that the detention basin outflow level was at or about the peak flow level to a point where basin outflow was well into its drain-down phase. Watershed T_p was defined by the SCS method as sixty percent of the time of concentration; however, T_p may be better established by watershed observations. An illustration of the watershed timing factors T_p, T_t, and TTDD are shown in Figure 2.13.

The following procedure was developed to control the level and distribution of the extra runoff throughout the watershed to mitigate the regional effect.

- When development location is from zero to 25 PTOC, design detention basins using a PFRF of one (pre-development peak flow), and then estimate TTDD.
    - If T_t plus TTDD is less than T_p, then basin peak outflow will have come and gone and well into its drain down phase before impacting primary watershed peak flow; therefore, the PFRF can be estimated using the upper PFRF relationship shown in Figure 2.12 (PFRF ≈ 1.0).

- If T_t plus TTDD is greater than T_p, then basin peak outflow and the primary watershed peak flow will arrive downstream simultaneously; therefore, the PFRF must be estimated by the lower PFRF relationship (PFRF ≈ 0.5).

- When development location is from 25 to 60 PTOC, the basin peak outflow and the primary watershed peak flow will arrive downstream simultaneously; therefore, PFRF must be estimated by the lower series.

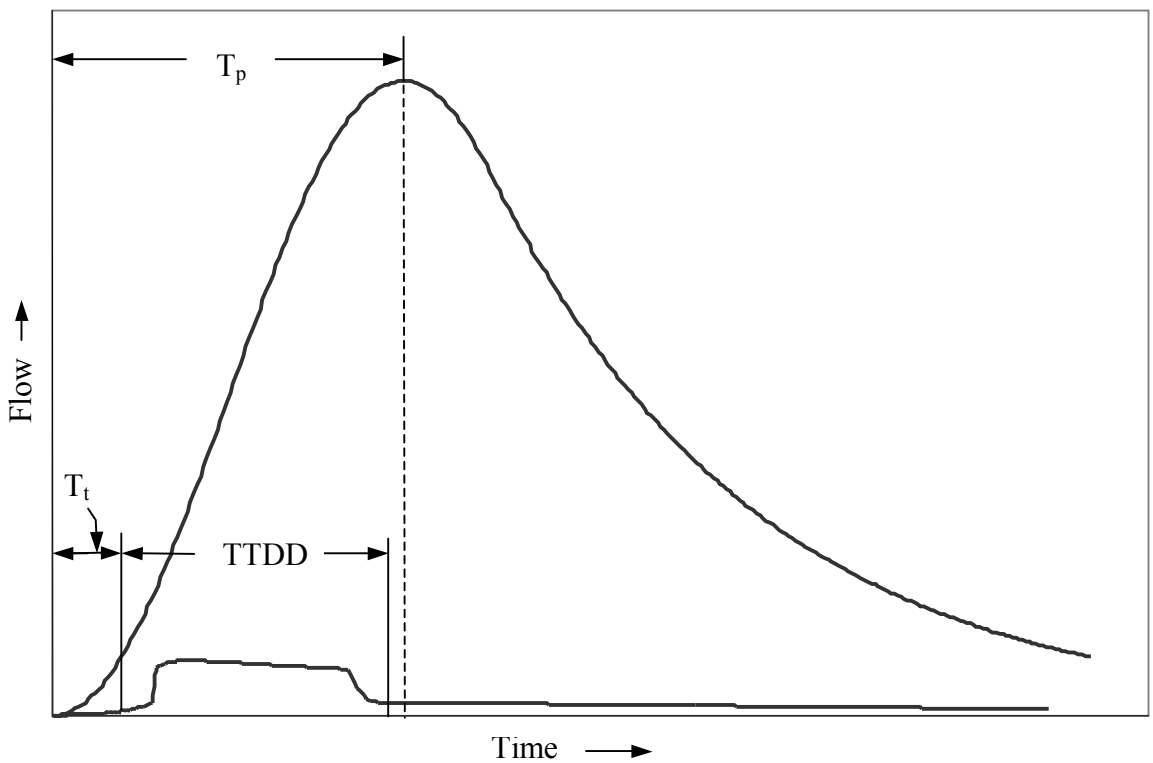

Figure 2.13 - Rendering illustrating the relationship between the time of travel (T_t), time to drain down (TTDD), and time to peak (T_p).

This analysis examined regional watershed impact only and did not consider any local drainage impact due to additional urban runoff controlled by using on-site

detention. Local drainage conditions should be considered before adopting any type of stormwater management program.

In conclusion, this hydrologic analysis has provided a better understanding of the regional effect, the reasons why on-site detention can exacerbate this effect, and detention basin design guidelines to mitigate the regional effect. In addition, it is believed, that the use of the peak flow reduction factor (PFRF) can provide rapid basin design peak outflow estimation for different development locations within a watershed to mitigate the regional effect.

CHAPTER 3

A Screening-Level Detention Basin Sizing Method to Reduce Downstream Impacts

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A Screening-Level Detention Basin Sizing Method to Reduce Downstream Impacts

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ABSTRACT

A screening-level detention basin storage method was developed that quickly estimates detention basin storage using a minimum amount of physical and hydrologic parameters with a range of detention pond outfall rates. The research used hypothetical catchments with ranges of parameters typical to the Charlotte-Mecklenburg County, North Carolina area. A hydrologic numerical analysis was conducted using generally accepted methods to size detention basins for thousands of different combinations of hydrologic parameters, physical parameters, and detention basin release rates. The results were then statistically analyzed to determine which parameters were important to perform a hydrologic analysis and estimate detention basin storage. It was determined that for the design storms of interest and the appropriate SCS soil group, post-development curve-number and a detention basin outfall factor can reasonably estimate the percent runoff volume used for detention basin storage. With this model, the designer can quickly determine the project's feasibility based on available space for above ground storage, or, if underground storage is necessary, its feasibility.

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A Screening-Level Detention Basin Sizing Method to Reduce Downstream Impacts

INTRODUCTION

Since the 1970s the philosophy of stormwater runoff management has changed from moving water as fast as possible downstream to detaining a portion of the flow, thereby reducing downstream damages that the unregulated flows could cause. Urban stormwater management is necessary since volumes and rates of stormwater runoff are increased due to urban-type development. Increased runoff may cause frequent flooding and severe channel erosion downstream. Throughout the United States, cities like Charlotte, North Carolina and other municipalities have learned that by requiring stormwater detention in their development regulations, flood damages can be reduced and National Flood Insurance Program requirements met. Stormwater management is the means of partially compensating for the adverse hydrologic effects that result from urbanization.

One generally accepted method of stormwater management is to construct on-site detention facilities. Detention basins provide storage volume lost through the construction of impervious surfaces, such as buildings, streets and parking lots. An increased volume of direct runoff is a significant effect of urbanization. The detention basin is intended to contain the increased volume and release the direct runoff gradually, thereby controlling or at least minimizing the downstream impacts due to urbanization. The peak discharge is most often limited to that which existed prior to development;

however, the increased volume of direct runoff is generally released at a rate that exceeds the rate that the water would be released from natural storage.

A number of researchers have investigated detention storage as a stormwater management option. The detention basin is the most widely used form of stormwater management (McCuen, 1979a; Hawley, 1981). Detention basins are typically designed to limit the peak flow rates after development to the peak flow rates that existed prior to urban development. This is most often the criterion used to assess the effectiveness of stormwater management facilities. Stormwater basins have been found, however to be seriously deficient (McCuen, 1979b) in some circumstances because they only strive to limit post-development peak flow rates to that peak flow that occurred before development. McCuen (1979a) has shown that the change in timing caused by a detention basin can result in increased downstream flooding. Smith and Bedient (1980) found that detention basins designed using a 10-year pre- and post-development storm will not necessarily limit downstream flow to the 10-year peak flow.

While the runoff characteristics at the outfall of a detention basin are important, it is also important to identify the effect of both urbanization and the detention facility on the runoff characteristics at points downstream from the detention facility. The runoff from detention facilities placed at different locations within a watershed can combine to produce downstream peak rates greater than pre-development peaks; sometimes, downstream peak rates can be even greater than the developed watershed would have experienced without the basins. While detention facilities can be designed to limit stormwater flows immediately downstream, their effect on the primary watershed downstream depends upon where they are placed. For instance, detention basin outflow

hydrographs have an elongated peak flow that will result in the possibility that the peak flow of the primary watershed hydrograph and the peak of the detention basin outfall could arrive simultaneously, thereby increasing the primary watershed peak flow. This has been referred to as the “regional” effect (McCuen, 1974; Bennett and Mays, 1985; McCuen, 1979b; and Ferguson and Deak, 1994).

The elongated detention basin outflow hydrograph and the time that it takes this hydrograph to reach a downstream location can change the peak flow at that location. The way the elongated detention basin outflow combines with the primary watershed's hydrograph determines the level and location of impact. McCuen (1979a) found for a study-watershed that the downstream impacts of detention basins exist for one mile downstream. Therefore, it is not enough to assure the effectiveness of detention basins by the peak flow at the outlet of the detention basin. In order to fulfill the purpose of stormwater management, the effectiveness of a detention basin must be measured on a regional basis.

A number of simple methods have previously been developed to size stormwater detention basins. The American Association of State Highway Transportation Officials (AASHTO) (1991) recommended a simplified method to determine preliminary estimates of storage volume using triangular-shaped inflow and outflow hydrographs. Abt and Grigg (1978) investigated a triangular inflow hydrograph and a trapezoidal outflow hydrograph to estimate the required storage volume. Another method developed by the Soil Conservation Service (SCS) (Soil Conservation Service, 1975) used a graphical method to estimate storage volume. Other simplified methods for sizing detention basin storage have been proposed in the literature (Akan, 1989; Bouthillier, 1978; Kessler and

Diskin, 1991; McEnroe, 1992; and Wycoff and Singh, 1976). Multicriterion methods were developed that size detention basins based on reducing downstream impacts. Akan and Antoun (1994) developed a method to size detention basins that controlled downstream erosion caused by post-development flows and Ordon (1974) developed a method to size detention basins such that combined sanitary system capacity is not exceeded. More complicated sizing methods using optimization have also been proposed (Bennett and Mays, 1985; Mays and Bedient, 1982; Nix and Heaney, 1988; and Taur et al., 1987).

The use of these simplified methods as design shortcuts should be used only as a preliminary design tool. Every detention basin design should be completed with a hydrologic analysis. To conduct a hydrologic analysis, certain physical and hydrologic parameters must be estimated. The estimation of these parameters can be a time consuming process; therefore, assessing the level of importance of each parameter would be a helpful design aid.

The objectives of this research are to determine the importance of each design parameter used in a hydrologic analysis, and using these findings, develop a screening-level method that includes a parameter that can be used to adjust for the regional effect to estimate detention basin storage volume. Because of the large number of simulations, a computer program was developed that designed detention basins for hypothetical catchments using all possible combinations of parameter values. Then, a sensitivity analysis was performed that statistically analyzed the results to determine the importance of each parameter. Based on this analysis, a statistical model was developed that will estimate detention-basin storage using a small number of parameters and includes a

parameter that will adjust for the regional effect. With this model, the designer can quickly estimate detention basin surface area, and depending on the available space for above ground storage can determine if underground storage will be necessary and feasible. In addition, this procedure will provide a baseline storage estimate for computer programs requiring an initial estimate.

METHODS

A computer model was developed to size detention basins for thousands of different combinations of hydrologic parameters, physical parameters, and detention basin release rates. The results were then statistically analyzed to determine which parameters were important to estimate detention basin storage. To accomplish this, a post-development runoff hydrograph was calculated and routed through a storage facility that had been designed to limit the peak flow to a particular fraction of the pre-development peak flow. This peak flow reduction factor (PFRF), which is included in the analysis to account for the regional effect of detention storage, is dimensionless and ranges from 1.0 to 0.1. The detention basins were sized using both above ground storage and underground storage. Sets of representative parameter values were chosen that included values typical to the Charlotte-Mecklenburg County, North Carolina area and used in this analysis. Detention basins were designed for each unique set of parameter values, and the storage noted. The data were analyzed and a statistical relationship established that estimates detention basin storage. The resulting statistical relationship uses a minimum number of parameters (e.g., soil groups, slope, basin shapes, development size and type) to estimate necessary storage. The mathematical models used to develop the computer program and the parameters used are described below.

Hydrograph Formulation

The hydrographs were generated using the SCS method (Hoggan, 1996) implemented in a spreadsheet calculation. The spreadsheet calculations were verified by comparing the results to hydrographs generated using the HEC-1 Flood Hydrograph Package (Hydrologic Engineering Center, 1981). The SCS method uses a dimensionless unit hydrograph that is based on the analysis of measured data. Precipitation used for the hydrograph development was computed by Dr. H. R. Malcom at North Carolina Stare University from data given in the U. S. Weather Bureau (USWB, 1961) and the National Weather Service rainfall data (Frederick, 1997) for the Charlotte-Mecklenburg County area (Table 3.1). The local precipitation values were interpolated from the precipitation data in these publications. The analysis in this research included the 10-, 25-, 50-, and 100-year frequencies and 6-, 12-, and 24-hour durations.

Table 3.1 - Intensity-duration-frequency precipitation data for Charlotte N.C.-Mecklenburg Co. (cm).

	Return Period (yr)			
Duration	10	25	50	100
(1)	(2)	(3)	(4)	(5)
5 min	1.50	1.73	1.88	2.06
15 min	3.20	3.68	4.04	4.42
1 hr	6.12	7.21	8.08	8.94
2 hr	6.81	8.03	8.97	9.90
3 hr	7.49	8.81	9.85	10.90
6 hr	9.25	10.87	12.12	13.36
12 hr	10.82	12.67	14.12	15.57
24 hr	12.39	14.50	16.15	17.78

Runoff Volume Estimation

Runoff depth was estimated by the SCS curve-number method, where a curve-number estimate was obtained for the soil group and cover conditions of the watershed (Soil Conservation Service, 1975). The following two equations estimate the depth of runoff.

$$S = 2.54\left(\frac{1000}{CN} - 10\right) \tag{1}$$

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

Where S is the ultimate soil storage (cm), P is the precipitation depth (cm) and Q^* is the runoff depth (cm), the asterisk was add to indicate depth and not discharge. Runoff volume was then calculated by multiplying runoff depth by watershed area. Watershed area for above ground storage was estimated using a range of values from two to eighty-one hectares. The application of underground storage is practically limited to smaller watersheds; therefore, watershed area ranged from 0.61 to 6.1 hectares for underground storage. A range of SCS curve-numbers (CN) values were chosen to correspond to the five zoning and one pre-development classification for the four SCS Hydrologic soil groups (A, B, C, and D) to be examined (Table 3.2). The zoning classifications were single family low density, single family medium density, single family high density, multi-family high density, and commercial/industrial. The pre-development classification was fair condition, woods. SCS curve-numbers ranged from a pre-development CN of 36 (A Soil, Fair Woods), to a post-development CN of 95 (D Soil, Commercial).

Table 3.2 - SCS Curve-numbers and distances to estimate sheet flow travel times used in analysis.

		Hydrologic soil group			
Cover Description (1)	Sheet Flow (m) (2)	A (3)	B (4)	C (5)	D (6)
Fair condition, Woods	76.2	36	60	73	79
Commercial	15.2	89	92	94	95
Apartment	15.2	77	85	90	92
0.10-ha Residential	31.8	61	75	83	87
0.20-ha Residential	45.0	54	70	80	85
0.40-ha Residential	63.6	51	68	79	84

Channel Hydraulic Radius Estimation

The estimation of the hydraulic radius for channels is necessary to evaluate the travel time through the channel portion of the watershed. Therefore, the following method was developed to estimate the channel hydraulic radius based on drainage basin size.

Previous research has shown that channel cross-sectional area and width are highly correlated with the size of the basin (drainage area). For instance, Dunne and Leopold (1978) collected and presented bankfull channel dimensions for the eastern United States. Equations are presented that estimate channel cross-sectional area, width, and flow depth based on the size of the basin. Using these relationships and the geometrical relationship of a trapezoid, Malcom (1989) developed the following relationship that estimates the hydraulic radius based on the size of the drainage basin.

$$R = 7.00 D_A^{0.313} \tag{3}$$

Where R is the hydraulic radius (cm), and D_A is the drainage area (hectare). This relationship was used in this study to estimate hydraulic radius.

Time of Concentration Estimation

The time of concentration for a watershed is the time for a particle of water to travel from the hydrologically most distant point in the watershed to a point of interest. The rate that the particle of water will travel through the catchment changes with catchment slope, among other things. Most of the commonly used methods to estimate time of concentration was not detailed enough reflect changes in time of concentration with different development types. Therefore, a method was chosen that could provide a detailed analysis of the time of concentration to determine if there was a connection between catchment slope (time of concentration) and basin storage. Time of concentration estimates were made using the techniques described in detail in TR-55 (Soil Conservation Service, 1975). TR-55 uses three different flow types: sheet flow, shallow concentrated flow, and open channel flow. Sheet flow is flow over flat surfaces usually occurring in the headwaters of a stream. Sheet flow eventually becomes shallow concentrated flow, then open channel flow. Sheet flow used Manning's kinematic solution, shallow concentrated flow used equations given in Appendix F of TR-55, and open channel flow used Manning's equation to calculate travel time. TR-55 used a minimum time of concentration of 0.1 hr. and this minimum time of concentration was also used in this analysis. Open channel cross section proportions were estimated using the method described earlier (Eq. 3). Pipe runoff velocity is usually engineered based on maintaining a minimum velocity fast enough to cleanse the pipe and a maximum velocity low enough to limit pipe wear. Based on experience, runoff velocity averages about 0.91

meters per second (Malcom, 1997). Time of concentration was determined by estimating travel time for each flow type and summing all individual travel times for a total travel time.

The watershed slope used to estimate time of concentration was collected by using the Basins 3.0 computer program (USEPA, 2001) to delineate the Charlotte-Mecklenburg County region into watersheds having a range of areas from 2,396 to 20,765 hectares. Typical values for Charlotte-Mecklenburg County ranged from a maximum slope of 0.05 m/m to a minimum slope of 0.005 m/m.

The guidelines presented in the TR-55 manual were also used to develop the distance estimations for sheet, shallow concentrated, pipe, and open channel flow. Because the shape of a development is dictated more by developmental factors than watershed shape, each catchment was assumed to be square. Total travel distance through a catchment was the length of the hypotenuse. Each flow type was assigned a hydraulic length based on development characteristics as described below.

Pre-development -Time of concentration for pre-development conditions was estimated by dividing the total hydraulic travel distance into three different flow segments (sheet, shallow concentrated, open channel). Sheet flow travel time was estimated using the Manning's kinematic solution. The maximum flow distance to be used with this formula is 91.4 m. A flow distance of 76.2 m and a Manning's n of 0.4 (woods-light underbrush) were assumed to be typical. From this point, sheet flow is assumed to become shallow concentrated flow. The travel distance used for shallow concentrated flow was assumed to be another 76.2 m, making a total travel distance of 152.4 m. At this point the catchment area is about 1.2 hectares, and the remaining travel

distance is assumed to be open channel flow. Open channel flow was calculated using the Manning's equation with a Manning's n of 0.03. Travel time for sheet, shallow concentrated, and open channel will decrease as catchment slope increases and the travel time for open channel flow will increase as the catchment size increases.

Development - Time of concentration for development conditions were estimated by dividing the total travel hydraulic distance into four different segments within the development. Flow segments were sheet, paved shallow concentrated, pipe, and open channel. Sheet flow travel times were estimated using the Manning's kinematic solution. The Manning's solution used a Manning's n of 0.24 (dense grass) and travel distances were estimated based on development type. Commercial and Apartment development travel distances were assumed equal to a typical property line offset (15.2 m). Residential development travel distance was assumed the length of one side of a square lot where the lot size is equal to the residential development size (Table 3.2). From this point, paved shallow concentrated flow was assumed until reaching the nearest culvert inlet (15.2 m). From this point, pipe flow was assumed. Pipe flow travel distance was assumed to be a maximum of 329 m. At this point the catchment area is about 5.4 hectares, and the remaining travel distance is assumed to be open channel flow. Open channel flow was calculated using the Manning's equation with a Manning's n of 0.03. Travel time for sheet, paved shallow, and open channel will decrease as catchment slope increases and the travel time for open channel flow will increase as the catchment size increases. Pipe velocity will remain constant.

Stage-Storage Function

Four different basin shapes were analyzed to determine whether storage volume changed with basin shape. Three above ground and one underground basin shape.

Above ground Storage - Above ground storage was estimated by the following power-curve that can easily estimate storage volume for a range of assumed shapes and sizes.

$$S = KsZ^{b} \tag{4}$$

Reservoir surface area can be estimated by substituting and solve for A_R:

$$A_R = \frac{dS}{dZ} = bKsZ^{(b-1)} \tag{5}$$

where S is the storage volume (m^3), Z is the basin stage (m) measured from the bottom of the reservoir, A_R is the reservoir surface area (m^2), Ks is the basin surface area (m^2) 0.3 m from the bottom of the reservoir, and b is a coefficient that determines the side slope of the reservoir. When b equals 1, the sides are vertical. The above ground detention basins were designed using three different basin shapes (Figure 3.1) ranging from vertical sides to bowl shape (b=1.01, 1.1, and 1.2).

Underground Storage - Underground storage was estimated using a table created within the computer program to estimate the water-area of a 2.44-m pipe at incremental depths. This table was interrogated to determine the water area at a specific depth, then multiplied by the pipe length to determine the storage volume. A 2.44 m pipe was used because it was determined to be the most efficient pipe size for underground storage (Roberts, 1995).

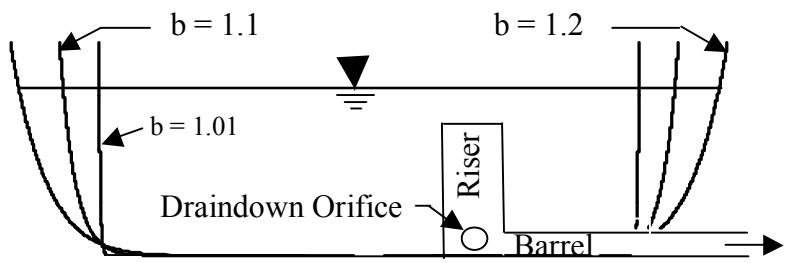

Figure 3.1 - Representation of the three different above ground detention basin shapes and the riser/barrel outflow configuration with drain-down orifice.

Detention Basin Discharge Hydraulics

The detention basin outflow hydraulics utilized a typical riser/barrel configuration (Figure 3.1). This simple riser/barrel configuration was chosen to detain a portion of the runoff, release it over time, and assure that all smaller design storms would be contained and released at a lower rate. Municipalities are requiring a portion of the runoff to be detained and released over time to meet NPDES, Phase II requirements. Optimizing the riser/barrel configuration with multistage release points would result in a more efficient design with a lower storage requirement; however, outlet design optimization was considered beyond the scope of this research. Of course, the designer using the preliminary storage estimate method described herein is free to consider outlet optimization during subsequent detailed design.

In the riser/barrel configuration, the riser rim is a vertical pipe acting as a weir with length equal to its circumference. The horizontal barrel connected to the bottom of the riser acts independently as a culvert under inlet control. The flow over the riser was estimated by a basic weir equation and sized such that the flow did not transition through

orifice control before the barrel acted to limit flow. The flow through the barrel and drain-down orifice was estimated using an orifice flow equation given below.

Riser Discharge - The riser discharge was estimated by the sharp-crested weir equation:

$$Q = C_w L H^{3/2} \tag{6}$$

where Q is the discharge (m^3/s), C_w is the weir coefficient (1.84 $m^{1/2}/s$), L is the length of weir measured along the crest (m), and H is the driving head measured vertically from the crest of the weir to the water surface (m).

Barrel Discharge - The barrel discharge was estimated by the orifice equation:

$$Q = C_d A \sqrt{2gh} \tag{7}$$

where Q is the barrel discharge (m^3/s), C_d is a discharge coefficient and is 0.6 (dimensionless), A is the cross-sectional area of flow at the orifice entrance (m^2), g is the gravitational acceleration (9.81 m/s^2), and h is the driving head (m) measured from the center of the orifice area to the water surface. Equation 7 is an inlet control equation and does not consider tailwater effects found in basins requiring outlet control design.

Drain-down Discharge - The drain-down orifice was sized to empty the storage in a 24-hour period. The barrel was sized to limit the peak flow of the pond as determined by the pre-development peak flow and the peak flow reduction factor (PFRF). As the PFRF becomes smaller, the size of the barrel orifice also becomes smaller. As a result, the barrel orifice can become more restrictive than the drain-down orifice. In this case, the drain-down orifice was set equal to the barrel orifice, and consequently, the drain-down period was greater than 24 hours.

Detention Pond Routing

The post-development hydrograph was routed through the detention facility using the Storage-Indication Method (Malcom, H.R., 1989). Using this procedure, the post-development hydrograph was routed through the detention facility using the stage-storage function and the previously presented hydraulic discharge relationships generated an outflow hydrograph.

The computer model was constructed in Microsoft® Excel® using the hydrologic methods previously discussed. Within the spreadsheet, a different sheet was used for individual tasks. A macro was written that manipulates the input and output values for each design and performed a trial and error procedure for each basin design that varied the basin area until water level equals the maximum water level. Each detention basin was designed twice. The first design set the riser height at 1.22 m with a maximum water level of 1.83 m and 2.44 m for above ground and underground storage, respectively. The next lower return period storm was routed through the design and the maximum water level noted. The maximum water level was found for every design and averaged. The average maximum water level was 1.5 m and 1.9 m for the above ground and underground designs, respectively. The riser height was then established at this height and the basin redesigned with maximum water levels previously established for above ground and underground storage. This resulted in basin designs in which all of the runoff from the lower return period storms was stored at or below the riser height, effectively containing and releasing all lower return period storms through the drain-down orifice. This design process allows for a conservative storage estimation where the design storm and all lower return period storms can be stored and released at an allowable rate.

Basin storage was estimated using the computer program for every unique parameter combination. After all the computer runs were complete, it was necessary to analyze the computer output to determine which parameters were important in prediction basin storage, then, using this information, develop a tool to easily estimate basin storage. Therefore, a statistical analysis was completed and a model developed.

Statistical Model

The output data from the program were organized and each parameter statistically evaluated to determine its importance in estimating basin storage. The parameters found to be statistically significant in estimating basin storage were used to develop multiple regression models to estimate detention basin storage. The statistical analysis used the SPSS™ statistical package, version 10.0.7 (SPSS 1999).

To find the "best" regression model, a "backward elimination" sequential search was used. The steps in this analysis were as follows:

1. A single regression equation was computed using all of the predictor variables of interest.
2. A partial F was calculated to test for each variable.
3. Predictor variables were eliminated for each partial F test that indicates no statistical significance.
4. Regression models were re-estimated using only the remaining predictor variables.
5. Return to step two and continue the process until all variables of interest and their contributions determined.

If the relationship between the predictor and independent variables were found nonlinear, data transformations were completed to improve linearity; then the previous five steps

were replicated. Final models were examined to ensure that regression residuals were normally distributed.

RESULTS

Regression models were developed from a statistical analysis of 22,000 unique detention basin designs. Calculations for above ground storage used six different catchment sizes, five watershed slopes, five development types, ten peak flow reduction factors, three different basin shapes, and four different storm frequencies, which gives 18,000 unique above ground detention pond designs. Underground calculations used four different catchment sizes, five watershed slopes, five development types, ten peak flow reduction factors, and four different storm frequencies, which totals 4,000 unique underground detention pond designs. A preliminary round of regression analysis sought to quantify a relationship between detention basin volume and one or more of the predictor variables (catchment size, shape, slope, storm return period and duration, SCS soil type, watershed land use as measured by the SCS curve-number, and peak flow reduction factor (PFRF)). During this analysis it was discovered that the number of predictor variables could be reduced by creating a new dependent variable, the percent of runoff necessary for basin storage (PRBS). Catchment size and slope, and detention basin shape, were found to be unimportant in predicting PRBS. Of the remaining parameters, post development curve-number (post-CN), and PFRF were found to be the only useful predictor variables. Storm return period and duration, and SCS soil group were found to be important to predicting PRBS, but not useful as regression variables. Based on these findings, regression models were developed to estimate the PRBS for every combination

of storm return period, storm duration, and SCS soil group (Tables 3 and 4). All regression models are of the form:

$$PRBS = E + F(PFRF)^X + G(PostCN) \tag{8}$$

where PRBS is the percent of runoff needed for detention basin storage, PFRF is the detention pond peak flow reduction factor, Post CN is the post-development CN, E is the regression constant, F is the PFRF regression coefficient, G is the post CN regression coefficient, and X is the PFRF regression exponent. PRBS increases as PostCN increases, and PRBS decreases with an increase in PFRF because the regression constant F is negative (Table 3.3). The relationship between PFRF and PRBS was found nonlinear; therefore, the exponent was added to PFRF to improve linearity. Design details of the outlet structure were not included because they are of secondary interest. The objective of the research is to design an outfall configuration that restricts the maximum outfall to a value of interest. Therefore, the maximum outfall rate of the design configuration is of primary interest and not the outfall structure details that restrict the outfall to that maximum outfall rate.

For soil types B, C, and D, storm return period was found not to be statistically significant in predicting PRBS. In these cases, pre-development (design basin outflow) and post development peak flows change with soil types and storm duration and return period but the ratio between the two flows remains constant for all storm frequencies (precipitation amount), thus this parameter need not be included in the regressions. For these soil types, sets of regression coefficients were developed for each combination of soil type and storm duration (Table 3.3). Soil group A has the largest storage capacity of the four soil groups; therefore, because of the large storage capacity of A soil groups,

lower return period pre-development storms were completely stored in the soil, resulting in no runoff and therefore no detention basin outfall. Consequently, the relationship that existed between the pre- and post-development hydrographs for B, C, and D SCS soil groups did not exist for A soils. For this soil type, therefore, regression model coefficients were determined for every combination of both storm return period and storm duration (Table 3.4).

Table 3.3 - Summary of PRBS regression results and coefficients for B, C, and D group soils and 10-, 25-, 50-, and 100-yr design frequencies. The appropriate coefficients are placed into the following equation to estimate the percent of runoff necessary for basin storage, $PRBS = E + F (PFRF)^X + G (PostCN)$.

Soil Group (1)	Duration (hr) (2)	Precip. (cm) (3)	E (4)	F (5)	G (6)	X (7)	R^2 (8)	Std. Error % (9)
D	6	9.25 – 13.36	33.70	-56.02	0.846	0.30	0.92	2.83
	12	10.82 – 15.57	23.16	-40.90	0.739	0.40	0.92	2.50
	24	12.40 – 17.78	20.07	-29.60	0.579	0.60	0.92	2.18
C	6	9.25 – 13.36	125.44	-136.51	0.770	0.10	0.91	2.95
	12	10.82 – 15.57	56.47	-66.57	0.694	0.20	0.92	2.59
	24	12.40 – 17.78	30.20	-36.24	0.575	0.40	0.92	2.25
B	6	9.25 – 13.36	1340.15	-1335.42	0.681	0.01	0.92	3.39
	12	10.82 – 15.57	1160.77	-1161.01	0.663	0.01	0.92	2.99
	24	12.40 – 17.78	167.80	-168.72	0.596	0.06	0.93	2.42

The relationship between PFRF and PRBS was found to be significantly nonlinear for some models. The amount of transformation is reflected in the value of the PFRF regression exponent (X). Based on the regression analysis, values of PFRF exponent X range from 0.01 to 1.0. The exponent X is closest to 1.0 (most linear) for A type soils and the shorter storm durations (Table 3.4). Smaller values for X (most non-linear) were found for the other soil types, with the lowest exponent (X=0.01) found for the B type soil and a storm duration of 12 hours (Table 3.3).

Table 3.4 - Summary of PRBS regression results for SCS A group soils. The appropriate coefficients are placed into the following equation to estimate the percent of runoff necessary for basin storage, PRBS = E + F (PFRF) ^{X} + G (PostCN).

Freq. (yr) (1)	Duration (hr) (2)	Precip. (cm) (3)	E (4)	F (5)	G (6)	X (7)	R^2 (8)	Std. Error (9)
10	6	9.25	--	--	--	--	--	--
	12	10.82	93.56	-4.66	0.096	1.00	0.80	0.97
	24	12.40	86.00	-11.13	0.209	1.00	0.82	2.04
25	6	10.87	93.08	-4.81	0.103	1.00	0.80	1.02
	12	12.67	86.02	-11.44	0.210	1.00	0.83	2.03
	24	14.50	71.66	-27.86	0.443	0.90	0.82	4.75
50	6	12.12	88.72	-8.57	0.167	1.00	0.83	1.58
	12	14.12	78.17	-19.45	0.327	1.00	0.82	3.40
	24	16.15	92.82	-67.17	0.550	0.30	0.89	4.54
100	6	13.36	86.37	-11.86	0.208	0.90	0.84	1.91
	12	15.57	75.01	-31.50	0.427	0.70	0.85	4.18
	24	17.78	295.00	-275.0	0.557	0.06	0.92	4.04

In every regression model, the percent of runoff necessary for basin storage (PRBS) was found to be inversely proportional to the peak flow reduction factor (PFRF), since all regression models had negative PFRF coefficients (Table 3.3 and 4). For B, C, and D soil types, the PFRF coefficient (F) was; found to have smaller negative values (closer to zero) for the longer duration storms (Table 3.3), while for the A soil type, longer duration storm storms had larger negative (farther from zero) F regression coefficients. On the hand, the post-development CN regression coefficient (G) was always positive; such that as the post-development CN increased, the percent of runoff needed for basin storage increased. The coefficient was always less than 1.0, and was generally higher for the B, C, and D soil types (Table 3.3) as compared with the A soil type (Table 3.4).

Storm return period (total precipitation) was unimportant in predicting PRBS for B, C, and D soil groups. This permitted a single model to be developed that combined the

10, 25, 50 and 100yr storms into one model for each design-storm duration (6-, 12-, and 24-hr) and soil group (Table 3.3).

The regression equations generally had adjusted R^2 values close to one and relatively small standard errors. For the B, C, and D soil types, the adjusted R^2 for the regression equations ranged from 0.81 to 0.91 (Table 3.3). For these soil types, the minimum value of the standard error of the estimate of the PRBS was 2.86 percent, while the maximum standard error was 3.59 percent

For the A type soils, the adjusted R^2 values, were slightly lower, with a range from 0.79 to 0.90. Not surprisingly, the standard errors of the estimate of PRBS were, in some cases, slightly higher, with a minimum value of 0.97 percent and a maximum of 4.81 percent (Table 3.4).

SAMPLE APPLICATIONS

The use of the screening-level models presented herein is best illustrated with a few sample applications, which are presented below.

Example 1 - A detention basin is to be sized for the Charlotte/Mecklenburg area to restrict the outfall to 80 percent of the pre-development flow (PFRF = 0.8) using a 10 yr-6 hr design storm. The development has a 2 ha area, C soil group, and a slope of 0.016. Post-development condition is 0.10-ha residential (CN=83). Detention ponds are to be designed for both above ground storage (shape factor b=1.2) and underground storage (2.44 m circular) facilities.

Example 2 - A detention basin is to be sized for a development located in the Charlotte/Mecklenburg area to restrict the outfall to 60 percent of the pre-development

flow (PFRF = 0.60) using a 25 yr-12 hr design storm. The development has a 6 ha area, B soil group, and slope of 0.039. Post-development condition is commercial (CN=92). Detention ponds are to be designed for both above ground storage (shape factor b=1.1) and underground storage (2.44 m circular) facilities.

Example 3 - A detention basin is to be sized to restrict the outfall to the pre-development flow (PFRF = 1.0) for a development in the Charlotte/Mecklenburg area using a 25 yr-12 hr design storm. The development has a 20 ha area, A soil group, and slope of 0.005. Post-development condition is 0.20 ha Residential (CN=54). Detention ponds are to be designed for both above ground storage (shape factor b= 1.1) and underground storage (2.44 m circular) facilities.

These three examples were chosen to represent a range of detention basin design values, such as design storm, PFRF, catchment area, soil group, catchment slope, and CN (Table 3.5). The solution to Example 1 is presented below in detail. The solution to Examples 2 and 3 use the same methods in Example 1; therefore, a detailed solution for Examples 2 and 3 are omitted.

Table 3.5 - Sample application parameters.

Case (1)	Design Storm (yr-hr) (2)	PFRF (3)	Basin Area (ha) (4)	Soil Group (5)	Slope (6)	CN (7)
1	10-6	0.8	2	C	0.016	83
2	25-12	0.6	6	B	0.039	92
3	25-12	1.0	20	A	0.005	54

The first step is to find the model coefficients for the proper soil group and design storm duration. From Table 3.3, the regression values for the appropriate soil group and storm duration (Soil group = C and duration = 6hr) are selected. The constant term (E = 125.44), coefficients (F = -136.51, G = 0.770) and exponent (x = 0.1) are substituted into Equation 8. For this case, the regression R^2 equals 0.91 and the standard error of the estimate is 2.95 percent. The resulting model is:

$$PRBS = 125.44 - 136.51(PFRF)^{0.1} + 0.770(PostCN) \tag{9}$$

The percent runoff volume required for basin storage is estimated by substituting the appropriate values of PFRF and PostCN into (8), as follows:

$$PRBS = 125.44 - 136.51(0.80)^{0.1} + 0.770(83) = 55.9\% \tag{10}$$

The percent of runoff volume required for basin storage is estimated to be 55.9 percent with a range of 53.0 percent to 58.9 percent using the standard error of the estimate (2.95 percent). The PRBS was checked against an analysis using the hydrologic methods described herein (HMDH) for above ground and underground storage, which gave values of 56.8 percent and 56.2 percent, respectively (Table 3.6).

For Example 2, the percent of runoff volume required for basin storage is estimated to be 66.7 percent with a range of 63.7 percent to 69.7 percent using the standard error of the estimate (2.99 percent). The PRBS was checked against an analysis using the (HMDH) methods for above ground and underground storage, which gave values of 65.5 percent and 64.2 percent, respectively (Table 3.6).

For Example 3, the percent of runoff volume required for basin storage is estimated to be 85.9 percent with a range of 83.9 percent to 88.0 percent using the standard error of the estimate (2.03 percent). The PRBS was checked against an analysis

using the (HMDH) methods for above ground and underground storage, which gave values of 85.5 percent and 86.7 percent, respectively (Table 3.6).

Table 3.6 - Model parameters, results, and (HMDH) methods comparisons.

	Model parameters					PRBS	HMDH	
Case (1)	E (2)	F (3)	G (4)	X (5)	R^2 (6)	Low/Est./Hig (7)	A.G. (8)	U.G (9)
1	125.44	-136.51	0.770	0.10	0.91	53.0/55.9/58.9	56.8	56.2
2	1160.77	-1161.00	0.663	0.01	0.92	63.7/66.7/69.7	65.5	64.2
3	86.02	-11.44	0.210	1.00	0.83	83.9/85.9/88.0	85.5	86.7

The estimated values of PRBS for the three Examples range from 55.9 to 85.9 percent and the (HMDH) methods results are slightly different for all three examples; however, the estimated values of PRBS follow the (HMDH) methods results and are within the range of regression model predictions in each case (Table 3.6).

To calculate a pond volume, runoff volume is multiplied by the percent of runoff necessary for basin storage (PRBS). Runoff volume is estimated by substituting the appropriate values of precipitation and curve number into Equations 1 and 2. For Example 1, the post-development CN of 83 is substituted into Equation 1 to estimate retention.

$$S = 2.54\left(\frac{1000}{83} - 10\right) = 5.20\ cm \tag{11}$$

The total precipitation for the Charlotte/Mecklenburg area for a 10 yr- 6hr event is 9.25 cm (Table 3.1). The retention calculated from Equation 11 and the rainfall depth of 9.25 cm are substituted into Equation 2.

$$Q^* = \frac{(9.25 - 0.2(5.20))^2}{9.25 + 0.8(5.20)} = 5.0\ cm \tag{12}$$

Runoff depth is 5.0 cm and runoff volume from the 2 ha catchment is 1,000 m^3. Therefore, the necessary storage volume is 559 m^3 (1000 m^3 x 0.559). Example 2 and 3 are solved in the same manner. For Example 2, runoff depth is 10.4 cm and runoff volume from the 6 ha catchment is 6240 m^3. Therefore, the necessary storage volume is 4162 m^3 (6240 m^3 x 0.667). For Example 3, runoff depth is 2.32 cm and runoff volume from the 20 ha catchment is 4640 m^3. Therefore, the necessary storage volume is 3986 m^3 (4640 m^3 x 0.859) (Table 3.7).

Table 3.7 - Detention basin sizing.

Case (1)	Q^* (cm) (2)	Runoff Vol. (m^3) (3)	Basin Storage (m^3) (4)	A_R (ha) (5)	Storage Length (m) (6)
1	5.0	1,000	559	0.04	120
2	10.4	6,240	4,162	0.25	891
3	2.3	4,640	3,986	0.24	854

Once the storage volume has been determined, basin surface area can be calculated. Equations 4 and 5 are utilized to estimate detention basin surface area. A typical value for the maximum surface elevation (1.83 m) is used with the given value for shape factor (1.2). The necessary storage volume, maximum surface elevation, and shape factor are then substituted into Equation 4, and the equation solved for the storage constant, Ks.

$$559 = Ks(1.83)^{1.2} \tag{13}$$

For Example 1, the storage constant Ks equals 270.7 m^2. This result is then substituted into Equation 5 and the resulting equation is solved for the above ground pond surface area, A_R.

$$A_R = 1.2(270.7)1.83^{(1.2-1)} = 367\,m^2 \tag{14}$$

The estimated surface area is 367 m^2 or 0.04 ha. The estimated length of underground circular storage with a diameter of 2.44 m (pipe cross-sectional area is 4.67 m^2) is 120 m. For Example 2, the estimated above ground surface area is 2502 m^2 or 0.25 ha. The estimated length of underground circular storage with a diameter of 2.44 m (pipe cross-sectional area is 4.67 m^2) is 891 m. For Example 3, the estimated above ground surface area is 2395 m^2 or 0.24 ha. The estimated length of underground circular storage with a diameter of 2.44 m (pipe cross-sectional area is 4.67 m^2) is 854 m (Table 3.7).

DISCUSSION AND CONCLUSIONS

The statistical analyses of the 264,000 hypothetical watersheds revealed that only a small subset of the parameters were important to detention basin sizing. It was also discovered that the relationships were somewhat different for watersheds having soils of group A as compared with the other three soil groups. Catchment size, detention basin shape, and design storm return period were unimportant in predicting the percentage of runoff necessary for basin storage (PRBS) for B, C, and D soil groups. While some of these parameters, such as catchment size and storm return period do affect runoff volume, they were found not to affect the fraction of this runoff that is needed to be detained in a pond. Therefore, the rainfall distribution is important and the storm return period is not

important in estimating PRBS. The Charlotte-Mecklenburg area and the vast majority of the United States have a Type II SCS rainfall distribution. The rainfall characteristics of the Charlotte-Mecklenburg area closely follow the Type II rainfall distribution. The SCS (TR-55) method to predict PRBS establishes a graphical relationship between PRBS and rainfall distribution. Examples 1 and 2 were recalculated using the (HMDH) method and rainfall data found in Raleigh, North Carolina (also Type II rainfall distribution). Example 3 was not analyzed because it used soil type A. The results of Example 1and 2 showed that the hydrologic analysis using the rainfall data found in the Raleigh area was within the standard error of the estimate for the model and was slightly outside the standard error of the estimate by an average of 1.25 percent, respectively. We expect that other areas having a Type II rainfall distribution would have similar results in terms of the percentage of storm runoff that needs to be captured in a detention pond. On the other hand, catchment slope, soil group, post-development CN, design storm duration, and PFRF were important in predicting the PRBS for B, C, and D soil groups. These parameters either have a direct effect on PRBS, as does the PFRF, or have an effect on the temporal distribution of runoff, and therefore affect the percentage of runoff that must be stored in a detention pond.

The statistical analysis of hypothetical watersheds found that detention basin shape was not important in estimating PRBS, but it is possible that the particular design process used may have played a role in this result. In this research, detention basins were designed where the runoff from the next-lower-return period design storm was stored below the top of the riser. Requiring this amount of inactive storage assures that all lower frequency design storms are completely captured by the lower portion of the basin, thus

the shape of this portion of the detention basin storage cannot influence the necessary pond volume**.** While variations in the shape of the basin storage above the maximum riser height and below the maximum water elevation were considered, they were found not to significantly affect the detention pond volume requirement. Judging from the relatively small scatter found in the relationships for PRBS, it is believed the restriction on basin design imposed in this analysis did not markedly affect the results.

Time of concentration was analyzed in detail to determine if there was a relationship between slope and development type and CN. Analysis of the hypothetical watersheds showed that catchment slope was significant in predicting PRBS; as slope increases, PRBS decreases. However, including catchment slope as a predictor variable added very little predictive value to the models when compared to the disadvantage of having another dependent variable. For example, the model for D type soils and 6-hr duration without slope had an R^2 and standard error of the estimate of 0.86 and 3.48 (Table 3.3), respectively. The R^2 and standard error of the estimate for the same model including slope was 0.89 and 3.09, respectively. Thus slope only slightly improved the power of the models; and therefore, was not included. Consequently, post-development CN was found to be the most important predictor of basin storage and not the development type or catchment slope. Therefore, care should be taken to correctly estimate CN for the particular design conditions. Possible modification of CN for different return periods and antecedent moisture conditions should be considered when estimating PRBS.

Storm design return period was significant in predicting PRBS for A group soils only. The PRBS changes with storm return period for A group soils because of the highly

permeable nature of these soils. The lower-return period pre-development hydrographs have essentially no runoff, thus detention storage is not an appropriate stormwater management method as no pond release would be allowed, while the relationship between the pre- and post-development hydrographs for higher return period storms approaches the same relationship found for the B, C, and D soil groups. The relationship between the pre- and post-development hydrographs will change depending upon the precipitation volume; therefore, making design storm return period important in estimating PRBS for A soil types. Because return period was important in estimating PRBS for A group soils, the applicability of the models developed for A group soils (Table 3.4) is limited. The use of these models should be limited to areas that use design storms having the same storm volume and duration as those used in the Charlotte-Mecklenburg area.

The statistical analysis found that storm duration was significant in predicting PRBS. As the design storm duration increases, the PRBS decreases; however, because the runoff volume increases at a greater rate than the PRBS decreases with design storm duration, the volume of basin storage increases with design storm duration. Consequently, detention basins designed with longer design storm durations will result in larger required storages. The analysis presented here was based upon the use of a balanced storm hydrograph. For the relatively long duration cases, balanced storms result in hydrographs that have a long period of low flow rates at the beginning and end of the hydrograph, with larger peak flow rates towards the center. The low flow portion at the beginning of the hydrograph is collected in the detention basin and released only through the drain-down orifice. As the design storm duration is increased, the volume of runoff in

these early flow periods increases, resulting in larger total basin storage for the longer design storm durations.

The pond design methodology presented here is proposed as an alternative to other "simple" methods for determining detention basin volume. It is interesting; therefore, to compare our results to other simplified methods now in use. The American Association of State Highway Transportation Officials (AASHTO) (1991) recommended a simplified method to determine preliminary estimates of storage volume using triangular-shaped inflow and outflow hydrographs. Another simple method (Abt and Grigg 1978) utilized the simplifying assumption that pond volume could be determined by assuming a triangular inflow hydrograph and a trapezoidal outflow hydrograph. A third method (SCS) used a graph method to estimate storage volumes. In comparing our results to these three methods, it was found that while our simplified method agrees well with the (HMDH) methods, these other three methods do not. For instance, the PRBS estimated for Example 1 using the (HMDH) methods was 56 percent, using the simplified method developed herein was 56 percent, and using the AASHTO, Abt and Grigg, and SCS methods were 70, 49, and 38 percent, respectively. The PRBS estimated for Example 2 using the (HMDH) methods was 65 percent, using the simplified method developed herein was 67 percent, and using the AASHTO, Abt and Grigg, and SCS methods were 89, 79, and 55 percent, respectively. The PRBS estimated for Example 3 using the (HMDH) methods was 85 percent, using the simplified method developed herein was 86 percent, and using the AASHTO and Abt and Grigg methods were 97 and 94 percent, respectively. The SCS method could not be used for this example because the peak outflow discharge to peak inflow discharge ratio exceeded the useful range of this

method. While these are only three cases of the 264,000-detention-basin-designs used to develop our models, they serve to demonstrate the relatively large variability in estimating PBRS using other simplified methods, and the consequent advantage of the simple method proposed here. The simplified method described herein is an improvement on the other simple methods because the other simple methods require pre- and post-development maximum flow rates to estimate PRBS while this method requires only post-development CN to estimate PRBS.

The model uses a parameter (PFRF) to adjust for the impacts of the "regional" effect. However, a method needs to be developed that will estimate a PFRF to adjust for the affects of the regional effect. Until this method is developed, the PFRF can be taken as one and the model will predict the PRBS with the post-development outfall limited to the maximum flow from an undeveloped wooded catchment.

This method was developed as a worse case scenario method; where pre-development condition was wooded and the outlet structure was a simple riser/barrel configuration. Therefore, the actual basin storage could be somewhat less depending on different pre-development conditions and the efficiency of the outlet structure. Nonetheless, the method presented here gives a useful preliminary and conservative storage estimate that can be refined in subsequent design work. The next step in this research could be to further develop the estimation procedure by reducing the number of detention basin designs necessary by eliminating variables found unimportant in this analysis, and including a range of pre-development conditions. This could result in a model where PRBS is estimated using both pre-development CN and post-development CN.

Detention pond designs should always be verified by routing a hydrograph through the storage facility. Part of the hydrologic analysis is to estimate each parameter based on its level of importance. From the statistical analysis, it was found that parameters that will result in a large impact on detention basin storage are precipitation, catchment area, post-development CN, and maximum detention basin outflow. An additional parameter, catchment slope was shown significant, but its impact in estimating detention basin storage was only marginal. Detention basin shape proved to be of little importance in estimating basin storage.

In conclusion, it is believed that the use of the models provided herein can give a rapid screening-level estimation of storage requirements and can be of use in comparing alternate designs for storm water management. In addition, the parameter evaluation conducted herein can provide a guide into parameter sensitivity used in a hydrologic analysis.

CHAPTER 4

CONCLUSIONS

This analysis simulated the placement of urban-type development at different locations in the lower portion of a watershed and investigated the regional impacts due to uncontrolled runoff and different levels of controlled release. Additionally, a screening-level method to estimate detention basin storage was developed.

Using a numerical simulation model, the uncontrolled analysis placed urban-type development at different locations in the lower portion of a watershed and investigated the percent increase in regional flow due to the release of uncontrolled runoff. Development location within a watershed can be defined for a wide range of watershed shapes and sizes by using the percent time of concentration. This analysis found that the regional impact increased as development location moved upstream and range of impact locations extended farther downstream as soil storage capacity increased.

Based on these findings, maximum regional impact for different development watershed locations should be evaluated by using the percent time of concentration and soil group. For example, the percent change in regional peak flow will remain under five percent for development locations from 0 to 35 percent time of concentration for D, and C soil groups, and the regional peak flow will remain under five percent for development locations from 0 to 25 percent time of concentration for B soil group (Figure 2.7).

Some municipalities assume negligible regional impact when uncontrolled runoff is allowed in the lower one-third of a watershed; however, this analysis found that using

the percent time of concentration and soil group was a more accurate method to delineate sections where uncontrolled release may be allowed.

The controlled analysis placed urban-type development at different locations in the lower portion of a watershed and estimated the maximum allowable detention basin outflow limit that did not increase regional peak flow.

This analysis found that the outflow from detention basins where the elongated peak-flow crest will come and go before the arrival of the regional watershed peak-flow may be designed at the pre-development peak flow limit (Figure 2.12 and 2.13).

However, if the elongated peak-outflow from detention basins and the regional peak-flow arrive downstream simultaneously, the detention basin outflow limit should be a fraction of the pre-development peak flow. Therefore, detention basin outflow should be limited to 0.5 to 0.8 of the pre-development peak-flow depending on the development location within the watershed, where the watershed location is defined as the percent time of concentration (Figure 2.12).

As discussed above, time to watershed peak (T_p) is used and ultimately controls the temporal distribution of detention basin outflow limits in the lower portion of a watershed. This analysis studied a wide range of watershed shape, sizes, and hydrologic parameters; however, this analysis should not be used where different watershed shapes or other conditions result in a T_p significantly different.

This analysis examined regional watershed impact only and did not consider any local drainage impact due to additional uncontrolled runoff or additional urban runoff controlled by using on-site detention. Local drainage conditions should be considered before adopting any type of stormwater management program.

This analysis also developed a screening-level method to estimate detention basin storage. Detention basin storage was investigated by simulating catchments using a wide range of hydrologic parameters.

This analysis developed statistical models to estimate the percent runoff used for detention basin storage using post-development CN and a peak flow reduction factor (PFRF). Different statistical models were developed for SCS soil groups B, C, and D and for 6, 12, and 24 hour design storm durations that quickly estimate the percent runoff used for detention basin storage (Table 3.3). These models are not controlled by design storm return periods, and as a result, can be used for a wide range of precipitation amounts. However, models developed for SCS soil group A (Table 3.4) were not as flexible and were developed for a specific design storm frequency and duration; therefore, these models should only be used for similar design storm frequency and durations.

This analysis investigated the regional peak-flow impacts due to urban-type development using different runoff release strategies. A wide range of simulated watersheds were constructed and analyzed. It is believed that this hydrologic analysis has provided;

- a better understanding of the regional effect and why on-site detention can exacerbate this effect,
- developed detention basin design guidelines to mitigate the regional effect, and
- developed a screening-level method to estimate detention basin storage.

Regional peak-flow impacts can be controlled by properly redistributing the additional runoff due to urban-type development; however, this redistribution results in a slightly altered watershed hydrograph shape. Sediment erosion may be affected by the additional runoff and altered hydrograph shape; therefore, the next logical step would be to use this same methodology and investigate regional sediment erosion impacts for uncontrolled and different levels of controlled release.

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Appendix

Appendix A – Excel® Model Description

The computer model was constructed using Microsoft® Excel® and generally accepted hydrologic methods. The spreadsheet used separate sheets and macros to perform individual tasks. Overall control of the analysis of many hypothetical watersheds was accomplished with the "Control" macro. The "Hydrograph" Sheet generated catchment hydrographs. The "Detention Pond Design" Sheet designed the detention basin and routed the uncontrolled development hydrograph through the detention basin to generate the detention basin outflow response. The "Data" Sheet estimated catchment time of concentration, watershed routing coefficients, watershed geometry and handled parameter input and output. Additional Macros were written to automatically complete tasks not normally accomplished in a spreadsheet, such as channel routing, detention basin sizing, data input and output.

To accomplish the analysis of many hypothetical watersheds, a data matrix was constructed in the "Data" Sheet, where each row of the matrix contained a unique set of hydrologic parameters describing the watershed to be evaluated. A Macro was written (the "Control" macro) to sequentially select a single row from the data matrix and update the hydrologic parameters in the "Data" Sheet. Next, all calculations were automatically updated in the "Data" Sheet to estimate time of concentration, watershed routing coefficients, and watershed geometry based upon these parameters. Separate Macros were written to analyze each watershed for uncontrolled and controlled release. Each of the sheets and macros used in the analysis are described in the following sections.

"Hydrograph" Sheet

The "Hydrograph" Sheet generated hydrographs using the SCS Unit Hydrograph Method. There were separate input data locations for time of concentration, CN, catchment area, and precipitation in five minute increments. Table A.1 shows input, output, data cell locations used in the hydrograph estimation.

The procedure to estimate the catchment hydrograph is shown in Figure A.1 and described below:

- Incremental precipitation is placed in row one.
- Accumulated precipitation is calculated from row one and placed in row 2.
- Accumulated runoff is calculated for each increment and placed in row 3.
- Incremental runoff is calculated from row 3 and placed in row 4.
- Incremental runoff is multiplied by the unit hydrograph and placed vertically below each segment, creating a matrix (each incremental hydrograph will begin vertically at its horizontal segment time).
- Each row of the matrix is added horizontally to estimate the aggregate runoff hydrograph.

"Detention Pond Design" Sheet

The "Detention Pond Design" Sheet imports a post-development hydrograph from the "Hydrograph" Sheet, sizes the outlet structure and basin, and then routes the post-development hydrograph through the detention basin. The riser/barrel was sized for a 6 foot depth using the orifice equation. The drain-down orifice was sized for a 6 foot depth and a 24 hour draindown period using Equation B.1. The first detention basin design was intentionally oversized, and then a macro, "DPDesign," used nested DoLoops to

systematically reduce the size of the basin until the maximum detention basin water level equaled 6 feet. Table A.2 shows the input, output, and intermediate calculation locations in the Sheet.

Table A.1 – Location of input, output, and data Cells in the "Hydrograph" Sheet.

Input	**Cells**
Time of Concentration (min)	C3
CN	C4
Catchment area (mi^2)	C5
Precipitation	I58:HP58 and I535:CB535
Precipitation Data	
10yr-6hr Design Storm	I48:HP48 and I525:CB525
10yr-12hr Design Storm	I49:HP49 and I526:CB526
10yr-24hr Design Strom	I50:HP50 and I527:CB527
Output	
Hydrograph	HQ65:HQ592

	Column	1	2	3	4	5
Row	Segment(min)	5	10	15	20	25
1	Incremental Precip. (in)	0.00732	0.00732	0.00732	0.00756	0.00773
2	Accumulated Precip. (in)	0.007	0.015	0.022	0.030	0.037
3	Accumulated Runoff (in)	0.000	0.000	0.000	0.000	0.000
4	Incremental Runoff (in)	0.000	0.000	0.000	0.000	0.000
5						
6						
7	Time (min)					
8	5	0.0				
9	10	0.0	0.0			
10	15	0.0	0.0	0.0		
11	20	0.0	0.0	0.0	0.0	
12	25	0.0	0.0	0.0	0.0	0.0
13	30	0.0	0.0	0.0	0.0	0.0

Figure A.1 – Copy of a portion of the "Hydrograph" Sheet showing the organization and procedure to calculate hydrograph.

Table A.2 - Location of input, output, and intermediate calculation Cells in the "Detention Pond Design" Sheet.

Input	Cells
Peak Flow Reduction Factor	B7
Post-development Hydrograph	C28:C3484
Output	
Outflow Hydrograph	F28:F3484
Intermediate Calculations	
$I_1+I_2+[(2S_1/\Delta t)-O_1]$	D28:D3484
$(2S_2/\Delta t)+0_2$	E28:E3484
Lookup Table	O28:V2028
Drain-down Orifice	J7
Barrel Orifice	J8

The Storage-Indication Method was used to route the post-development hydrograph through the detention basin. This method states that $I_1 + I_2 + [(2S_1/\Delta t)-O_1] = (2S_2/\Delta t) + 0_2$. The first portion of the equation was estimated from the input hydrograph and the latter portion unknown; therefore, a LOOKUP Table was constructed that established a range of $(2S_2/\Delta t) + 0_2$ values; estimating outflow values for each $(2S_2/\Delta t) + 0_2$ value. The LOOKUP Table was then interrogated and found the outflow value (O_2) where $I_1 + I_2 + [(2S_1/\Delta t)-O_1] = (2S_2/\Delta t) + 0_2$ for each time increment.

<u>"Data" Sheet</u>

The "Data" Sheet contains all hydrologic parameter input and data output. The "Data" Sheet distributes the hydrologic parameters to the appropriate sections, where individual blocks estimate:

- Catchment size and shape based on the size and shape of the watershed.
- Development and watershed time of concentration.
- Watershed routing parameters.

Table A.3 shows the input, output, and intermediate calculation locations in the "Data" Sheet.

Each complete model run is for a specific Design Storm and soil group, therefore, the Design Storm and CN values were updated manually. Macros managed hydrograph generation, combination and routing. There were several Macros written that complete specific tasks and are described below:

"Control" Macro

The "Control" Macro starts the watershed analysis process and sequentially updates the "Data" Sheet with a new set of hydrologic parameters, and then evaluates the hypothetical watersheds for Uncontrolled or Controlled runoff release using the "UncontrolledDevelopment-Watershed" or "DetPondWatershed" subroutine.

Table A.3 - Location of data, input, output, and intermediate calculation Cells in the "Data" Sheet.

Data	**Cells**
Hydrologic Data	A3651:J40775
Input	
Hydrologic Data Input Box	A3639:J3639
Pre-development CN	B54
Post-development CN	L3
Intermediate Calculations	
Development T_c	B2:X8
Watershed Geometry	A15:B22
Catchment T_c	A25:H34
Watershed Routing Parameters	A37:M48
Catchment Combination and Routing	Columns (AM – BS)
Output Watershed Hydrograph	BS5:BS3461
Output	
Results Output Box	L3639:DK3639
Results	L3651:DK3639

“UncontrolledDevelopmentWatershed” Macro

The “UncontrolledDevelopmentWatershed” macro used the “Data” and “Hydrograph” Sheets and initiates:

- Catchment and Development hydrograph generation
- Placement of developments with controlled runoff at the designated locations
- Routes the uncontrolled runoff downstream.

DetPondWatershed Macro

The “DetPondWatershed” macro used the “Data,” “Detention Pond Design,” and “Hydrograph” Sheets and initiates:

- Catchment and Development hydrograph generation
- Detention basin design
- The placement of developments with controlled runoff at the designated locations
- Watershed routing

Appendix B – Derivations

<u>Drawdown Time</u> – An equation was derived to size the drain-down orifice for a 24 hour drain-down period.

Flow over time = Reservoir volume reduction

$$Q\,dt = A_R\,dh$$

$$CA_o N\sqrt{2gh}\;dt = bK_S h^{(b-1)}\;dh$$

where

Q = flow through orifice,

A_R = surface area $= bK_S z^{(b-1)}$,

C = pipe coefficient.

N = number of orifices,

K_S = represents the approximate area (ft^2) one foot from the bottom of the reservoir,

b = a coefficient which varies the side slope of the reservoir,

h = storage depth, and

g = 32.2 ft/sec^2 = 417,312,000 ft/hr^2.

$$CN\left(\tfrac{1}{4}\pi\, d_0^2\right)\sqrt{2g}\;dt = bK_S h^{(b-1)} h^{-1/2}\;dh$$

$$22{,}690NCd_0^2\;dt = bK_S h^{\left(b-3/2\right)}\;dh$$

$$d_0^2\int_0^t dt = \frac{bK_S}{26{,}690NC}\int_0^h h^{\left(b-3/2\right)}\;dh$$

$$d_0^2\ \Delta t = \frac{bK_S}{22{,}690}\frac{h^{(b-\frac{1}{2})}}{(b-\frac{1}{2})}$$

$$d_0 = \left(\frac{bK_S}{\Delta t\, 22{,}690 CN}\frac{h^{(b-\frac{1}{2})}}{(b-\frac{1}{2})}\right)^{\frac{1}{2}} \times 12 \qquad \text{(B1)}$$

where

d_0 = orifice diameter (in).

Hydraulic Radius

The wetted perimeter can be estimated for a channel where the channel dimensions are estimated by using equations 18, 19, and 20 to estimate channel width (W), channel cross-sectional area (A), and depth of flow (y), respectively. A trapezoidal shape will be used to estimate channel dimensions. Figure A1 defines the variables.

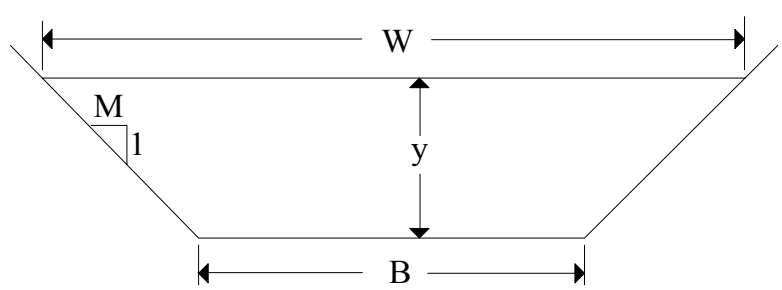

Figure B.1 – Variables used in trapezoidal applications.

Then, using the preceding estimates, the following equations are used to estimate channel side slope (M) (Malcom, 1989),

$$A = Wy - My^2 \qquad \text{(B2)}$$

the bottom width of the channel (B) can be calculated by the following equation,

$$B = W - 2My \tag{B3}$$

then, the wetted perimeter and hydraulic radius are estimated by the following equations.

$$P = B + 2y\sqrt{1 + M^2} \tag{B4}$$

$$R = \frac{A}{P} \tag{B5}$$

where

A = cross-sectional area (m^2),

W = top width of flow (m),

y = depth of flow (m),

M = side slope ratio (m horizontal/m vertical),

B = bottom width of the channel (m),

P = wetted perimeter (m), and

R = hydraulic radius (m).

Using the procedure above, a hydraulic radius is calculated for a range of drainage areas, then plotted on log scales and shown below in Figure B.2. Equation B6 was developed to predict the hydraulic radius relationship shown in Figure B.2.

$$R = 10.47 D_A^{0.3199} \tag{B6}$$

where

R = hydraulic radius (cm), and

D_A = drainage area (Hectare).

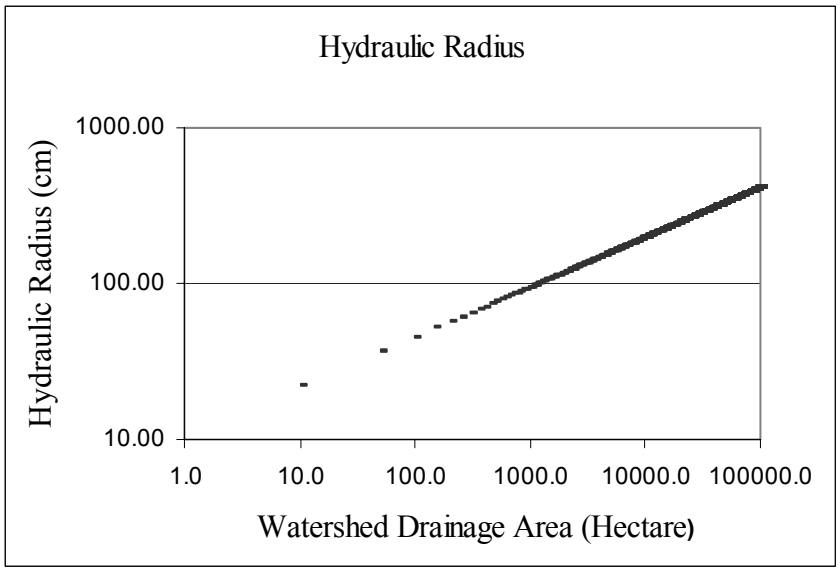

Figure B.2 – A graph plotting the watershed drainage area on the x axis and the hydraulic radius on the y axis.

Appendix C- Hydrologic Model

Hydrograph Formulation

The SCS (Soil Conservation Service) method will be used in this analysis for hydrograph formulation. The SCS method uses dimensionless unit hydrographs that are based on the analysis of measured data. Unit hydrographs were evaluated for a large number of actual watersheds and then made dimensionless by dividing all discharge ordinates by the peak discharge and the time ordinates by the time to peak. An average of these dimensionless unit hydrographs was then computed.

The HEC-1 SCS method consists of a single parameter, TLAG, which is equal to the lag (hrs) between the center of mass of rainfall excess and the peak of the unit hydrograph. The unit hydrograph is interpolated for the specified computation interval, then the computed peak flow is estimated from the dimensionless unit hydrograph (Hydrologic Engineering Center, 1981).

Runoff Volume Estimation

Runoff depth may be estimated by the Soil Conservation Service Curve Number Method, where a Curve Number estimate is obtained for the soil type and cover conditions of the watershed (Soil Conservation Service, 1975). The following two equations estimate the depth of runoff.

$$S = \frac{1000}{CN} - 10 \tag{C1}$$

$$Q^* = \frac{(P - 0.2S)^2}{P + 0.8S} \tag{C2}$$

where

S = ultimate soil storage (in),

P = precipitation depth (in), and

Q^* = runoff depth (in), the asterisk was add to signify depth and not discharge.

Volume is then calculated by multiplying runoff depth by watershed area.

Time of Concentration Estimation

Time of concentration estimates are made using the techniques described in TR-55 (Soil Conservation Service, 1975). TR-55 uses a three different flow types, sheet flow, shallow concentrated flow, and open channel flow. Time of concentration is determined by estimating travel time for each flow type, then adding all three for a total travel time. Sheet flow is estimated using the following equation.

$$T_t = \frac{0.007(nL)^{0.8}}{(P_2)^{0.5} s^{0.4}} \tag{C3}$$

where

T_t = travel time (hr),

N = Manning's roughness coefficient,

L = flow length (ft),

P_2 = 2-year, 24-hr rainfall (in), and

s = slope of hydraulic grade line (ft/ft).

Average velocities for estimating travel time for shallow concentrated flow:

$$Unpaved \quad V = 16.1345(s)^{0.5} \tag{C4}$$

$$Paved \quad V = 20.3282(s)^{0.5} \tag{C5}$$

where

V = average velocity (ft/s), and

s = slope of hydraulic grade line (ft/ft).

And, velocity within a channel is estimated by Manning's equation.

$$V = \frac{1.49\, r^{2/3} s^{1/2}}{n} \tag{C6}$$

where

V = average velocity (ft/s),

R = hydraulic radius (ft) and is equal to a/p_w,

a = cross sectional flow area (ft^2),

p_w = wetted perimeter (ft),

s = slope of the hydraulic grade line (ft/ft),

n = Manning's roughness coefficient for open channel flow.

<u>Stage-Storage Function</u>

Storage is estimated by a power-curve fit. It is estimated by the following equation:

$$S = KsZ^b \tag{C7}$$

where

S = storage volume (cu ft),

Z = stage (ft) referred to the bottom of the reservoir,

Ks = represents the approximate area (ft^2) one foot from the bottom of the reservoir, and

b = a coefficient which varies the side slope of the reservoir. When b equals 1, the sides are vertical.

Detention Pond Routing (Storage-Indication Method)

The continuity equation states that the rate of change of storage with respect to time is equal to the difference between inflow and outflow:

$$\frac{ds}{dt} = I - O \tag{C8}$$

Over a time increment:

$$\frac{\Delta S}{\Delta T} = I - O \tag{C9}$$

The incremental storage change can be estimated as:

$$\Delta S_{ij} = (\bar{I} - \bar{O})\Delta T_{ij} \tag{C10}$$

where

ΔS_{ij} = change in storage in the time increment i to j,

$\bar{I}$ = average inflow from time i to time j,

$\bar{O}$ = average outflow from tine i to time j, and

ΔT_{ij} = time increment.

Equation B10 is then algebraically manipulated to obtain the following:

$$I_i + I_j + \left[\frac{2S_i}{DT} - O_i\right] = \left[\frac{2S_j}{DT} + O_j\right] \tag{C11}$$

in which

I_i = inflow at the beginning of the interval,

I_j = inflow at the end of the interval,

S_i = storage at the beginning of the interval,

S_j = storage at the end of the interval,

O_i = outflow at the beginning of the interval,

O_j = outflow at the end of the interval, and

ΔT = time increment.

Detention Pond Discharge Hydraulics

The outflow structure uses a riser/barrel design. The basic weir or orifice equation will be used to estimate the flow through the riser. The controlling equation is the one that estimates the minimum flow. The basic weir equation is:

$$Q = C_w L H^{3/2} \tag{C12}$$

where

Q = discharge (cfs),

C_w = weir coefficient (dimensionless),

L = length of weir (ft), measured along the crest, and

H = driving head (ft), measured vertically from the crest of the weir to the water surface.

The orifice equation used to estimate the flow through the riser is:

$$Q = C_d A \sqrt{2gh} \tag{C13}$$

where

Q = discharge (cfs),

C_d = coefficient of discharge (dimensionless),

A = cross-section area of flow at the orifice entrance (sq ft),

g = acceleration of gravity (32.2 ft/sec^2), and

h = driving head (ft), measured from the water surface to the horizontal plane of the crest of the riser.

The barrel discharge is estimated by the orifice equation, which is:

$$Q = C_d A\sqrt{2gh} \tag{C14}$$

where

Q = discharge (cfs),

C_d = coefficient of discharge (dimensionless),

A = cross-section area of flow at the orifice entrance (sq. ft.),

g = acceleration of gravity (32.2 ft/sec^2), and

h = driving head (ft), measured from the center of the orifice area to the water surface.

Flood Routing Estimation

Hydrograph routing will be conducted by translating the hydrograph directly downstream or by using the Muskingum method. The Muskingum method for flood routing was developed for the Muskingum Conservancy District flood control study in the 1930's (Wanilista, 1990). This method is widely used and estimated using the following equation:

$$O_2 = c_0 I_2 + c_1 I_1 + c_2 O_1 \tag{C15}$$

in which

$$c_0 = \frac{-Kc + 0.5(\Delta t)}{K - Kc + 0.5(\Delta t)}$$

$$c_1 = \frac{Kc + 0.5(\Delta t)}{K - Kc + 0.5(\Delta t)}$$

$$c_2 = \frac{K - Kc - 0.5(\Delta t)}{K - Kc + 0.5(\Delta t)}$$

and

$$c_0 + c_1 + c_2 = 1.0$$

in which

c = weighting factor between 0 and 0.5, when c = 0.0 maximum attenuation is estimated, and when c = 0.5 the input hydrograph is not attenuated, and

K = storage time constant for the reach, normally estimated by the travel time within the reach.

Channel Dimension Estimation

Channel cross-sectional area and width are highly correlated with the size of the basin (drainage area). Bankfull dimensions for urban areas in North Carolina were collected and presented. (Doll, 2002). This data was then used to construct the following equations that estimate channel cross-sectional area, width, and flow depth. Then the channel hydraulic radius can be estimated. Channel width is estimated by:

$$W = 5.79 D_A^{0.32} \qquad \text{(C16)}$$

where

W = channel width at water surface (m), and

D_A = drainage area (sq. km.).

Channel cross-sectional area is estimated by:

$$A = 3.11 D_A^{0.64} \tag{C17}$$

where

A = cross-sectional area (sq. m.), and

D_A = drainage area (sq. km.).

Depth of flow can be estimated by:

$$y = 0.54 D_A^{0.32} \tag{C18}$$

where

y = flow depth (m), and

D_A = drainage area (sq. km.).

Then the hydraulic radius can be estimated by:

$$R = 10.47 D_A^{0.3199} \tag{C19}$$

where

R = hydraulic radius (cm), and

D_A = drainage area (Hectare).

This equation is derived in Appendix B (Eq. B6).