Spring 2010

The effectiveness of the Natural Resource Conservation Service (NRCS) and Huff rainfall distribution methods for use in detention basin design

Todd Wayne Dablemont

THE EFFECTIVENESS OF THE NATURAL RESOURCE CONSERVATION SERVICE (NRCS) AND HUFF RAINFALL DISTRIBUTION METHODS FOR USE IN DETENTION BASIN DESIGN

by

TODD WAYNE DABLEMONT

A THESIS

Presented to the Faculty of the Graduate School of the

MISSOURI UNIVERSITY OF SCIENCE AND TECHNOLOGY

In Partial Fulfillment of the Requirements of the Degree

MASTER OF SCIENCE IN CIVIL ENGINEERING

2010

Approved by

Charles D. Morris, Advisor
Cesar Mendoza
Jeffrey D. Cawlfield

ABSTRACT

This thesis focuses on the effectiveness of the NRCS and Huff rainfall distribution methods for use in detention basin design. This study required the use of HEC-HMS, hydrologic modeling software, in order to analyze the distribution methods. Three separate detention basins and their watersheds were modeled for this study. The watersheds were analyzed for both undeveloped and developed conditions. The parameters analyzed include detention basin inflow, detention basin outflow, watershed peak discharge, and detention basin storage capacity. The determination of detention basin effectiveness was based upon these parameters.

The NRCS distribution method is widely used; however, many who use it have little understanding of its effectiveness. The Huff distribution method differs in several ways from the NRCS distribution method including providing the user with an option to use different storm durations. This thesis aims to give insight into the effectiveness of the NRCS and Huff rainfall distribution methods for detention basin design.

TABLE OF CONTENTS

LIST OF FIGURES

LIST OF TABLES

1. INTRODUCTION

In order to design a detention basin, it is necessary to route a rainfall event through it in order to determine if the detention basin and outlet structure are functioning properly. This rainfall event, or design storm, must first be developed before it can be used in the design process. There are many methods for developing a rainfall event, also known as a hyetograph. Two of the most prominent methods are the NRCS and Huff rainfall distribution methods. These rainfall distribution methods can be used to develop a rainfall hyetograph when actual rainfall data is not available.

While both of these methods will develop a hyetograph that can be used for detention basin design, the storms they produce are very different. A rainfall event with a frequency of 100 years will vary in different aspects such as duration and intensity dependent upon which method is used. What is not known is how these differences affect the process of detention basin design. This thesis is aimed at looking at the NRCS and Huff rainfall distribution methods and how they affect detention basin design. These two distribution methods were chosen to compare because they are the two most commonly used methods today. Both methods have been established and accepted for many years. The NRCS method is commonly used by many cities, municipalities, and other agencies because it is simple to use and easy to standardize. However, many wonder if the desire for simplicity has sacrificed results. Does the NRCS distribution accurately represent a natural occurring storm? Do detention basins designed with this distribution perform the job they were intended for? The Huff distribution method is accepted by most as an accurate distribution method. It gives the user more options in the process of hyetograph development when compared to the NRCS method, but these additional options also make the design process more difficult and time consuming. Is the Huff method worth the additional work required, or can similar results be obtained by the NRCS method? This thesis seeks to answer these questions as they pertain to detention basin design.

2. URBANIZED WATERSHEDS

2.1. INCREASED STORMWATER FLOWS

As a watershed develops, many changes occur. Undeveloped areas are typically covered by grass, brush, and trees. This type of natural land cover allows a large amount of rainfall to infiltrate into the ground when a storm occurs. Undeveloped areas also tend to have many ponds and natural depressions that store water, keeping it from reaching the outlet of the watershed. The development of a watershed usually brings about an increase in impervious areas as well as a reduction in storage areas. Roads, parking lots, driveways, buildings, sidewalks, and other facilities increase the hydraulic efficiency of the land. These surfaces allow for little or no rainfall to be infiltrated into the soil. These smooth, impervious surfaces cause the majority of the rainfall to be quickly ushered to the watershed outlet. The reduction in infiltration means that a larger percentage of the total rainfall will be released from the watershed as runoff. This runoff will flow over the smooth concrete and asphalt surfaces that are typically found in developed areas and give much larger peak discharges than were previously found under undeveloped conditions. This runoff is then carried downstream by various means dependent upon the particular stormwater system present.

In areas with less development, runoff is often allowed to flow along the surface of the ground. This water will eventually collect in small ditches and channels and flow downstream until it is eventually emptied into a larger body of water such as a stream, river, or lake. In areas that are more urbanized, runoff typically flows into street curbs and gutters and is eventually deposited into nearby storm drains. This water is then transported downstream through pipes in the stormwater system at an increased rate increasing the peak flows downstream. Yet, no matter how the runoff is conveyed downstream, increased impervious areas due to development lead to larger discharges in both total volume and peak flows downstream.

Both the increase in total volume and peak flows can cause problems downstream. One issue that occurs is increased flooding. Areas downstream often cannot handle the increased flows causing frequent flooding. In urban areas this can mean the flooding of streets, parking lots, businesses, and even houses. Another problem is the increased erosion that occurs downstream in the streams. Stream channels are forced to carry much higher peak flows more frequently than were previously carried. In addition, high flows last much longer due to the increase in the total amount of runoff. These factors can lead to instability and increased erosion in the channel as it tries to adapt to the new conditions. Also, the increased erosion means that a larger amount of sediment will be carried downstream. This can have a dramatic effect on the quality of bodies of water downstream as this sediment is deposited into larger rivers, ponds, and lakes.

2.2. DECREASING STORMWATER PEAK RATES WITH DETENTION BASINS

Detention basins are often utilized in an attempt to mitigate some of the effects of urbanization on stormwater runoff. When used in developed areas, their main purpose is to control the increased runoff created by urbanization in order to lessen the effects downstream, such as increased flooding. While there are other methods to retard the increased flows such as infiltration basins and dry wells, detention basins are the most common structures used. Detention basins, or ponds, are designed to collect water and temporarily store it. This water is then released through an outlet structure at a lesser rate than it entered the basin. The peak rate from the outlet structure is typically less than the undeveloped peak rate for that watershed.

2.3. ANALYSIS OF THE DESIGN OF DETENTION BASINS

For the analysis discussed in this thesis, three separate watersheds and detention basins were analyzed. Design information, drawings, and calculations for these watersheds were obtained from the St. Louis Metropolitan Sewer District. Each watershed involved an area that had undergone some sort of development requiring the design and use of a detention basin in order to control the runoff leaving the development. The three watersheds will be referred to as Tuscany Hills, First National Bank, and Dietrich Forest.

3. RAINFALL DISTRIBUTIONS

3.1. HYETOGRAPHS

In order to analyze or design a detention basin, a runoff hydrograph for a given frequency must be routed through it. This is done to simulate the runoff that the basin will be required to detain. To determine the hydrograph that will be routed through a detention basin, it is first necessary to develop a rainfall hyetograph. A hyetograph is a distribution of rainfall over time. In the case of this analysis, a hyetograph was created to represent a particular frequency of storm and then routed through a watershed and detention basin by means of the hydraulic modeling software HEC-HMS. The details of how this was done are explained later in this thesis.

Before rainfall can be distributed over time, you must first determine the total amount of rainfall for the storm frequency that is being used. For this analysis, the total rainfall amount for all storms was determined using Bulletin 71 – Rainfall Frequency Atlas of the Midwest. Bulletin 71 was written by Floyd A. Huff and James R. Angel and published in 1992. The rainfall values contained therein were determined from an analysis of previous rainfall data. It is meant for use in the Midwest states of Minnesota, Iowa, Missouri, Illinois, Michigan, Indiana, Kentucky, Wisconsin, and Ohio. All of the detention basins analyzed in this thesis were located in Missouri Section 02 – The Northeast Prairie. Once the total amount of rainfall was determined for a particular event, it then needs to be distributed over time in order to develop the rainfall hyetograph.

3.2. DISTRIBUTION METHODS

There are several ways in which to develop a rainfall hyetograph. Two very commonly used rainfall distribution methods were chosen for this analysis. These two rainfall distribution methods are the Natural Resource Conservation Service (NRCS) Method and the Huff Distribution Method. Hyetographs were developed using both methods. The rainfall was then routed through each watershed in order to compare the effectiveness of the detention basin using these two rainfall distribution methods. It is important to note that both of these methods were developed to be used to temporally distribute the rainfall within a storm of a given duration. These methods seek to represent a naturally occurring storm

3.2.1. NRCS Distribution Method. The NRCS method was first published in 1975. At that time it was known as the SCS method, or Soil Conservation Service Method. It was published in the design manual *Urban Hydrology for Small Watersheds,* Technical Release 55, or TR-55. This manual was later revised in 1986. The NRCS distribution was developed using the Weather Bureau's Rainfall Frequency Atlases. Rainfall-frequency data from areas up to 400 square miles, durations up to 24 hours, and frequencies from one to 100 years, was used. Generalized volume-duration-frequency relationships from the Weather Bureau's technical publications were used to base the NRCS distribution on. When developing the distribution, rainfall depths were calculated using time increments of six minutes. The maximum six minute depth was found from the data and subtracted from the maximum twelve minute depth. The maximum twelve minute depth was subtracted from the maximum eighteen minute depth, and so on. The largest six minute value was placed in the middle of the 24 hour period, followed by the next largest, and so on until the smallest six minute intervals were placed at the beginning and end of the distribution. This means the greatest intensities were placed at the center of the storm, and the smallest intensities were placed at the very beginning and end.

The NRCS method mainly focuses on using a 24 hour duration to develop a rainfall hyetograph. This is a long duration for an urban rainfall event because urban watersheds typically consist of impervious areas such as parking lots, roofs, and roads which convey stormwater quickly and efficiently. This long duration is attempted to be compensated for by having a short period of intense rainfall in the middle of this distribution. In essence, there is a small, intense storm in the midst of the total 24 hour storm. The result is a long duration storm with a short period of intense rainfall that is intended to be used on large, small, urban, and rural watersheds. The long total duration accompanied with the short, intense period of rainfall is supposed to make the storm representative of both long and short duration storms. The NRCS distribution is a standardized distribution that, according to some studies, may not be appropriate for representing the statistical average value for storms. It is standardized in that the NRCS method has only one option of distribution. That distribution is 24 hours long, giving the user no choice of duration. If a storm is considered the statistical average value, it means that it is the typical storm for that frequency. It is especially important for the NRCS distribution to be the statistical average because there is no choice of duration. A naturally occurring two year storm can be of many different durations; therefore, a statistical average storm would be representative of all those different durations. This is not the case for the NRCS distribution. For instance, one study conducted in Denver in which rainfall data was collected and compared to the NRCS curves determined that the NRCS Type I and II Storms represented the worst case time distribution in order to form a severe storm. In other words, rather than producing a typical or average storm, they produce a severe case storm. For small urban watersheds, the highest peak discharges are often a result of short duration storms of very high intensity. Many times, when modeling urban watersheds, a storm of duration equal to the time of concentration is used. The time of concentration is the time it takes for water from the most hydraulically remote point on the watershed to reach the outlet. If a storm lasts as long as the time of concentration, that means that the entire watershed will be contributing to the runoff at the same time. When using the NRCS method, this is not an option because different durations cannot be chosen. The duration of the NRCS storms are set at 24 hours. The short period of very intense rainfall in the middle of the NRCS distribution does give large peak discharges, as can be seen as a result of the analysis in this thesis.

The NRCS method contains four different distributions. They are referred to as Type I, IA, II, and III. Which distribution is used is dependent upon the location of the watershed being analyzed within the United States. The Type I distribution is used for Alaska as well as parts of California. The Type IA distribution is used for much of the West Coast. The Type III distribution is used in portions of some southern states along the Gulf of Mexico as well as much of the East Coast. The Type II distribution covers the largest portion of the continental United States. The watersheds analyzed in this thesis are located in the region for the Type II distribution. This Type II distribution consists of very low intensities for the first half of the storm. Around the twelfth hour of the storm there is a period of very intense rainfall. At this point, over one third of the rainfall falls in a one hour period. In fact, approximately half of the total rainfall occurs between the eleventh and thirteenth hours. This period is then followed by another period of low intensity rainfall for the remainder of the duration. The four NRCS distributions and how they are distributed over time are shown in Table 3.1. They are also shown graphically in Figure 3.1.

The NRCS method is one of the most common, if not the most common, rainfall distribution method used. It is often viewed as an accepted, standard method because it has been so widely used for many years. One reason it has been so commonly used is due to its ease of use. Rainfall distributions such as the Huff distribution have many durations to choose from. The NRCS method only has the 24 hour duration, making it simpler to use due to the lack of choices. This, however, is also one area where it has been criticized. It has been questioned whether this long duration storm accurately represents a rainfall event that would occur naturally. The short period of high intensity rainfall in the middle of the storm attempts to simulate an event of short duration. This also has been questioned. In this case you have a period of rainfall that may be much shorter and of a higher intensity than an actual storm.

Table 3.1. NRCS Distributions (P/P_T)

Time (hours)	Type I	Type IA	Type II	Type III	Time (hours)	Type I	Type IA	Type II	Type III
0.5	0.008	0.010	0.0053	0.0050	12.5	0.706	0.683	0.7351	0.7020
1.0	0.017	0.020	0.0108	0.0100	13.0	0.728	0.701	0.7724	0.7500
1.5	0.026	0.035	0.0164	0.0150	13.5	0.748	0.719	0.7989	0.7835
2.0	0.035	0.050	0.0223	0.0200	14.0	0.766	0.736	0.8197	0.8110
2.5	0.045	0.067	0.0284	0.0252	14.5	0.783	0.753	0.8380	0.8341
3.0	0.055	0.082	0.0347	0.0308	15.0	0.799	0.769	0.8538	0.8542
3.5	0.065	0.098	0.0414	0.0367	15.5	0.815	0.785	0.8676	0.8716
4.0	0.076	0.116	0.0483	0.0430	16.0	0.830	0.800	0.8801	0.8860
4.5	0.087	0.135	0.0555	0.0497	16.5	0.844	0.815	0.8914	0.8984
5.0	0.099	0.156	0.0632	0.0568	17.0	0.857	0.830	0.9019	0.9095
5.5	0.112	0.180	0.0712	0.0642	17.5	0.870	0.844	0.9115	0.9194
6.0	0.126	0.206	0.0797	0.0720	18.0	0.882	0.858	0.9206	0.9280
6.5	0.140	0.237	0.0887	0.0806	18.5	0.893	0.871	0.9291	0.9358
7.0	0.156	0.268	0.0984	0.0905	19.0	0.905	0.884	0.9371	0.9432
7.5	0.174	0.310	0.1089	0.1016	19.5	0.916	0.896	0.9446	0.9503
8.0	0.194	0.425	0.1203	0.1140	20.0	0.926	0.908	0.9519	0.9570
8.5	0.219	0.480	0.1328	0.1284	20.5	0.936	0.920	0.9588	0.9634
9.0	0.254	0.520	0.1467	0.1458	21.0	0.946	0.932	0.9653	0.9694
9.5	0.303	0.550	0.1625	0.1659	21.5	0.956	0.944	0.9717	0.9752
10.0	0.515	0.577	0.1808	0.1890	22.0	0.965	0.956	0.9777	0.9808
10.5	0.583	0.601	0.2042	0.2165	22.5	0.974	0.967	0.9836	0.9860
11.0	0.624	0.624	0.2351	0.2500	23.0	0.983	0.978	0.9892	0.9909
11.5	0.655	0.645	0.2833	0.2980	23.5	0.992	0.989	0.9947	0.9956
12.0	0.682	0.664	0.6632	0.5000	24.0	1.000	1.000	1.0000	1.0000

The detention basins used for this study were previously designed using the NRCS method. As previously noted, this is commonly the method used in industry today. Due to the detention basins being designed by using this method, it was expected that they would function properly when modeled using the NRCS distribution.

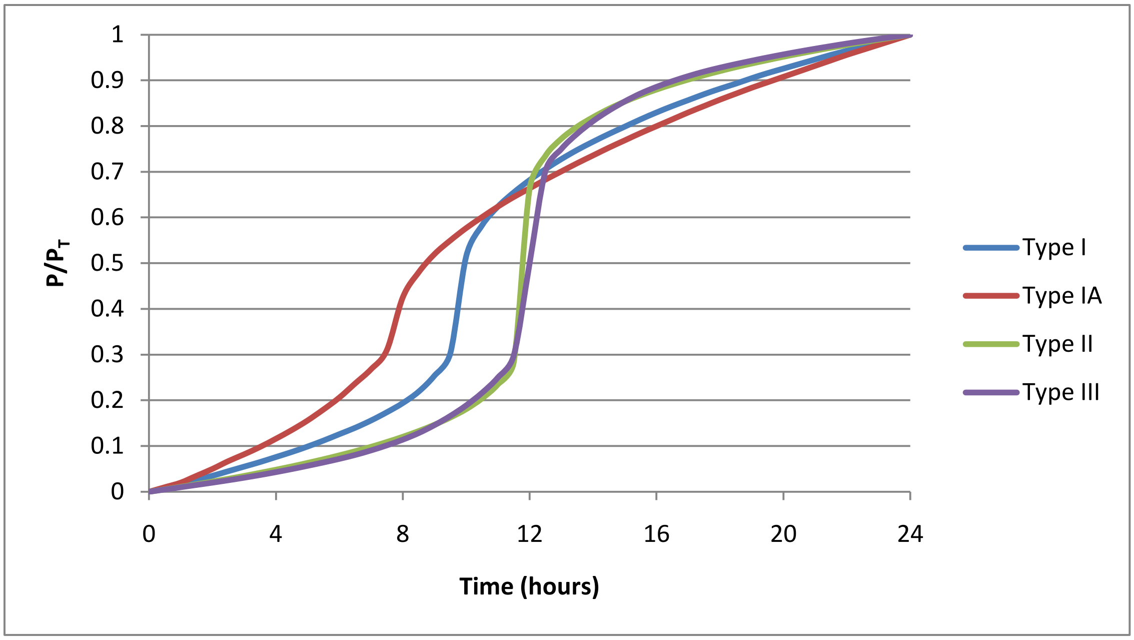

Figure 3.1. NRCS Distributions

3.2.2. Huff Distribution Method. The Huff method was developed by primarily analyzing data from 275 daily reporting stations from the National Weather Service Cooperative Network. The data was from the states of Illinois, Indiana, Iowa, Michigan, and Missouri as well as limited data from Minnesota, Wisconsin, Ohio, and Kentucky. These 275 stations had records exceeding 50 years. An additional 134 cooperative stations with shorter records were also used. A log-log graphical analysis referred to as the Huff-Angel method was used for the final derivation of the frequency relationships. As reported by Huff, many different statistical distributions were looked at before choosing this particular one. The method used is described in detail in *Bulletin 71 – Rainfall Frequency Atlas of the Midwest*.

To use the Huff distribution, the first thing that must be known is the area of the watershed. The Huff method uses three different sets of distributions dependent upon how large a watershed is. The three sets from largest to smallest are: 50 to 400 square miles, 10 to 50 square miles, and less than 10 square miles. The less than 10 square miles classification is also referred to as at a point. This is because Huff saw little or no difference for areas less than 10 square miles. The three detention basins studied were all relatively small, consisting of only a few acres. This means that the distributions at a point were used. For watersheds at a point, there are then four different distributions to choose from. These distributions are named the first, second, third, and fourth quartiles. The four quartiles and how they are distributed over time are shown in Table 3.2 as well as graphically in Figure 3.2.

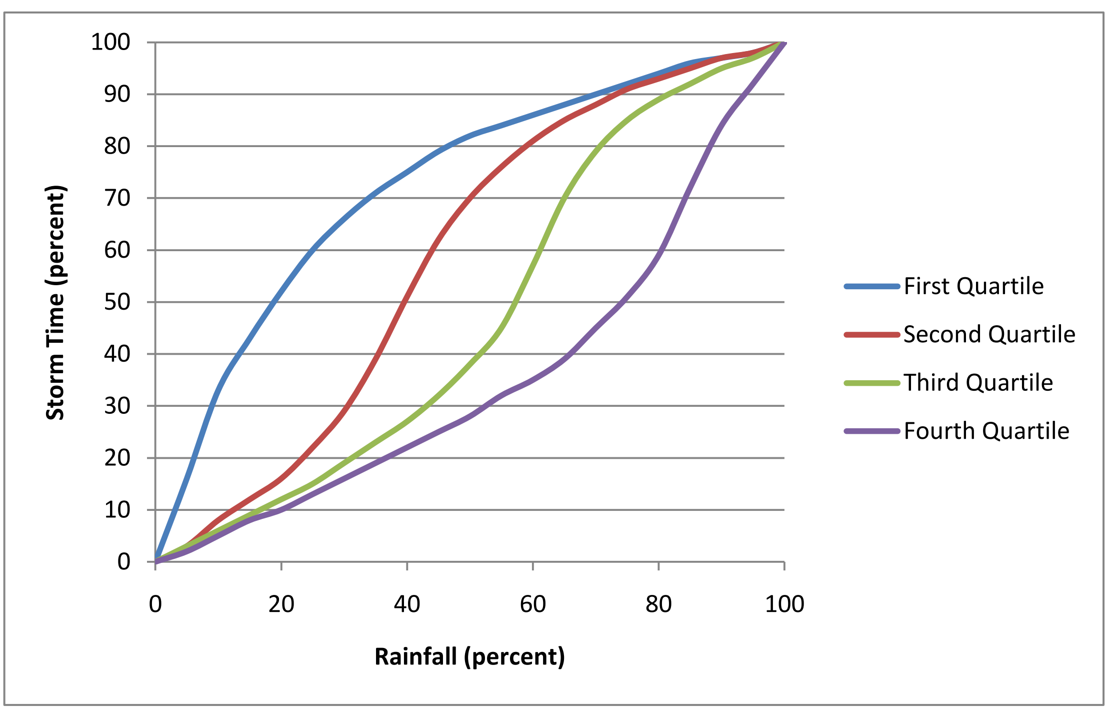

Figure 3.2. Huff Rainfall Distribution at a Point

Table 3.2. Huff Rainfall Distribution at a Point

Cumulative Storm Time (Percent)	Cumulative Storm Rainfall (percent)			
	First Quartile	Second Quartile	Third Quartile	Fourth Quartile
0	0	0	0	0
5	16	3	3	2
10	33	8	6	5
15	43	12	9	8
20	52	16	12	10
25	60	22	15	13
30	66	29	19	16
35	71	39	23	19
40	75	51	27	22
45	79	62	32	25
50	82	70	38	28
55	84	76	45	32
60	86	81	57	35
65	88	85	70	39
70	90	88	79	45
75	92	91	85	51
80	94	93	89	59
85	96	95	92	72
90	97	97	95	84
95	98	98	97	92
100	100	100	100	100

As can be seen, the distribution of rainfall over time varies between the four quartiles. The first quartile storm has its greatest intensity very early within the event. In contrast, the fourth quartile distribution has its greatest intensity very late within the storm. It can also be seen that the Huff method has a much more even distribution over time as compared to the NRCS method. As previously noted, the NRCS method has very low intensities except for a short period of high intensity rainfall. This can be seen in Figure 3.3 by the steep slope to the curves around the twelfth hour of the storm. The Huff distribution creates storms that have less variability in the intensity of the rainfall. While each quartile has a period of higher intensity rainfall in between periods of lower intensity rainfall, the difference between the intensities is much less pronounced. This is better shown in Figure 3.3 and Figure 3.4. These two figures show the rainfall hyetographs that result from one inch of rain. Figure 3.3 shows the NRCS Type II distribution. This figure shows the large peak in rainfall that occurs in the middle of the storm. This is in severe contrast to the low intensities that come before and after. Figure 3.4 shows the Huff second quartile distribution. While there is a definite peak in the rainfall, overall the rainfall intensity variation is much less than that of the NRCS Type II distribution. The peak intensity is also much lower than that observed with the NRCS method.

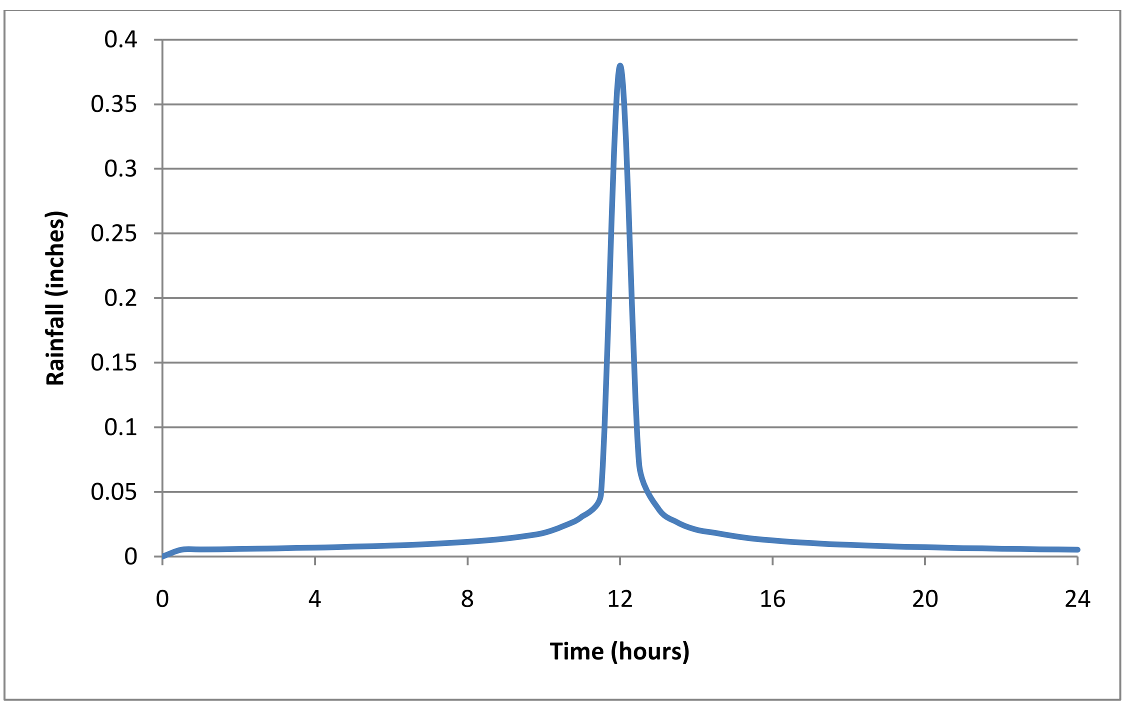

Figure 3.3. NRCS Type II One Inch Storm

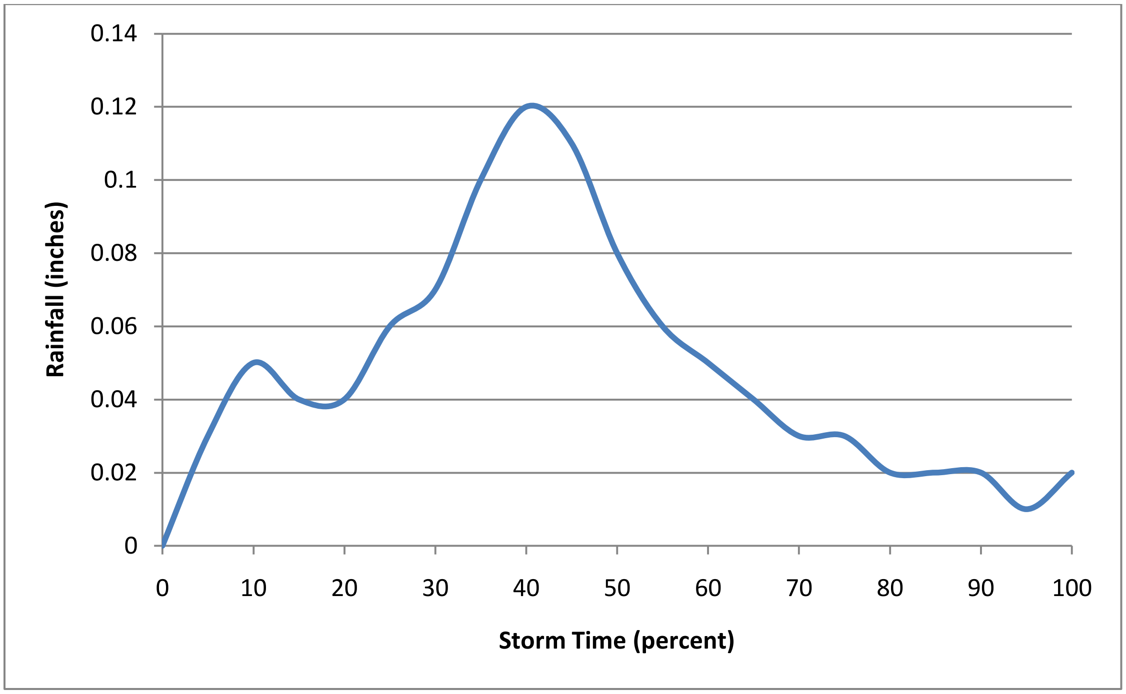

Figure 3.4. Huff Second Quartile One Inch Storm

The choice of which quartile to use is dependent upon the duration of the total storm being distributed in time. Storms with a short duration of six hours or less should be distributed using the first quartile. Storms greater than six hours up to 12 hours should be distributed using the second quartile. Huff discovered through observing the rainfall data that was collected that short duration storms tended to match up with first and second quartiles. Storms greater than 12 hours but no larger than 24 hours should use the third quartile, and storms greater than 24 hours should be distributed by using the fourth quartile.

3.2.3. Comparison. The NRCS distribution is an attempt at a standardized rainfall distribution for all durations. It is not statistically appropriate for all watersheds. One drawback to the NRCS method is that the user has no choice of duration. The NRCS distribution is set at a 24 hour duration. While this simplifies the its use given that there are fewer choices that have to be made with this distribution, it also limits the user's ability to model the critical duration for that watershed. The critical duration is typically defined as the storm which results in the highest peak discharge. The critical duration is different for every watershed based upon its hydrologic conditions. Many entities such as cities or municipalities prefer the NRCS distribution because it allows them to have a standardized storm that everyone must follow. Using the NRCS distribution gives them more control and a better understanding of the work that is done within their jurisdiction. This is why the NRCS distribution is so often used, and even required, in many places. The Huff distribution presents many more options that require more understanding of hydrology as far as duration is concerned, and therefore is more difficult to standardize.

A study conducted on temporal rainfall distributions for design took a look at the NRCS distribution as an option for their study (Thompson, Asquith, and Cleveland). They noted that the NRCS distribution was developed with recurrence intervals less than 100 years. Most of the data used to develop it was concentrated in the four to ten inch range for total rainfall. The type II distribution contains 45% of the total rainfall in a one hour period. The type III distribution is similar in that it contains 40% of the total rainfall in a one hour period. The rest of the rainfall is more evenly distributed over the other 23 hours. Since such a high percentage of the storm falls during a short time, this one hour period of high intensity rainfall tends to govern the design. In their analysis, their study used the probable maximum precipitation (PMP) as the total amount of rainfall for a given storm. The PMP tends to be very large in comparison to typical average rainfall amounts. Using the NRCS distribution with these large rainfall amounts resulted in a very large amount of rainfall falling during the high intensity one hour portion of the storm. They chose to exclude the type II and type III rainfall distributions from further investigation due to their conclusion that they over predict the rainfall rates during this period.

Figure 3.5 shows a comparison of the average intensity for each hour of a 100 year 24 hour storm for both the NRCS and Huff distributions. The total amount of rainfall for this storm was 7.21 inches. As you can see from the figure, the NRCS storm has very low intensities for the majority of the duration. Only in the very middle of the storm does the intensity increase; however, in the twelfth hour, the intensity takes a very large jump. In fact, the average intensity for every hour of the NRCS storm except for the twelfth hour is 0.18 inches per hour, with the highest intensity of 0.79 inches per hour coming in the thirteenth hour. The twelfth hour has an intensity of 3.09 inches per hour. This value is over sixteen times greater than the average intensity and over 3.9 times greater than the next highest intensity.

The Huff storm shown in Figure 3.5 has less variability in its intensities. It lacks the extreme contrast in intensities of the NRCS storm, and therefore better represents a natural occurring 24 hour storm. The peak for the Huff 24 hour storm is 0.76 inches per hour and occurs in the fifteenth hour. The maximum

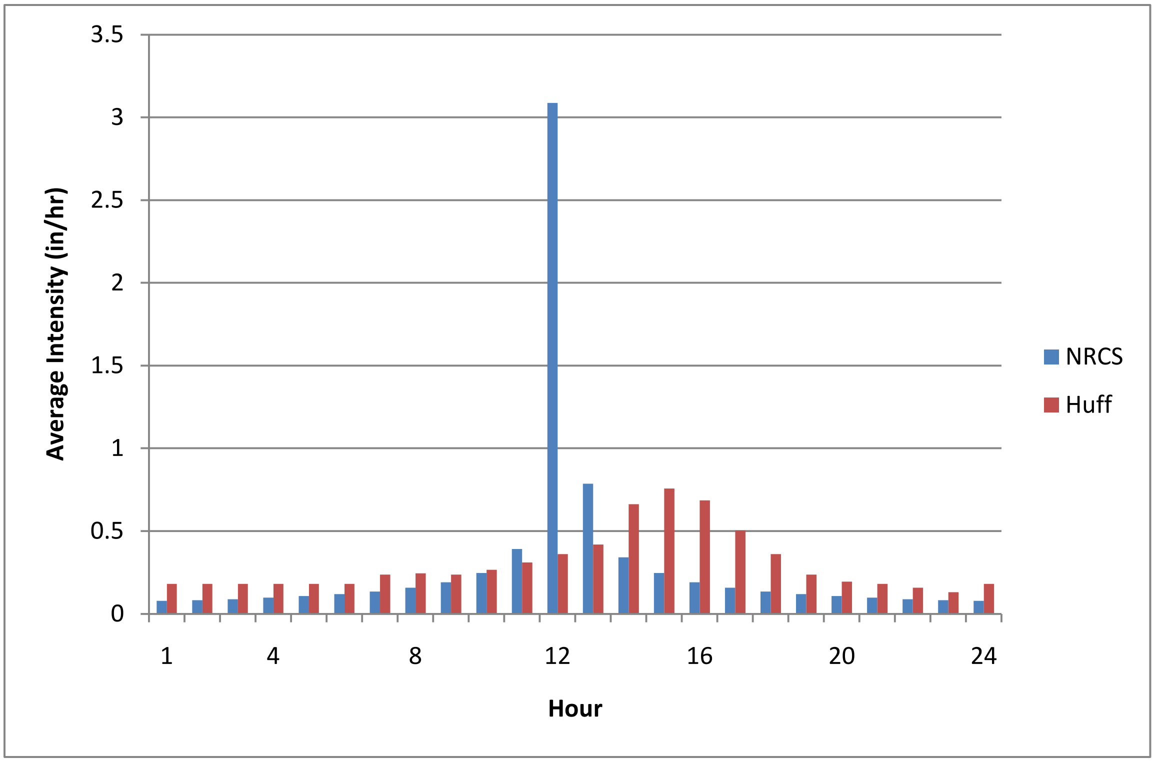

Figure 3.5. Average Hourly Intensities for a 100 Year 24 Hour Storm

intensity for the Huff storm is over for times less than the maximum intensity of the NRCS storm. While the maximum intensity is much smaller, for 21 of the 24 hours of the storm in Figure 3.5, the Huff distribution has a higher intensity. All of this shows how much more evenly distributed the Huff distribution is compared to the NRCS distribution.

4. DETENTION BASIN DESIGN

4.1. FLOW CHARACTERISTICS

As previously discussed, a detention basin's main purpose is to collect and store runoff for a period of time while releasing it at a controlled rate. This lessens the rate at which runoff leaves the watershed. This is done in order to reduce flooding downstream as well as lessen the damage that can be caused by increased runoff.

Unless there is permanent retention within a detention basin, the total amount of runoff does not change. The same amount of runoff is released, but it is released at a slower rate over a longer period of time in order to control the peak discharge. Figure 4.1 and Figure 4.2 give a graphical representation of the inflow and outflow of the Tuscany Hills detention basin for a storm with a frequency of 25 years. Tuscany Hills is one of the watersheds analyzed in this study. Figure 4.1 is the result of modeling a storm using the second quartile Huff distribution for a 12 hour storm. Figure 4.2 is from an NRCS Type II distribution which has a duration of 24 hours. The inflow is the runoff that came from the watershed and flowed into the detention basin. The outflow shows the water that was released from the outlet structure of the detention basin.

In both cases, the peak inflow rate is much higher than the peak outflow rate. This is because the detention basin is operating properly in as far as it is detaining the runoff and releasing it at a slower rate. The NRCS distribution

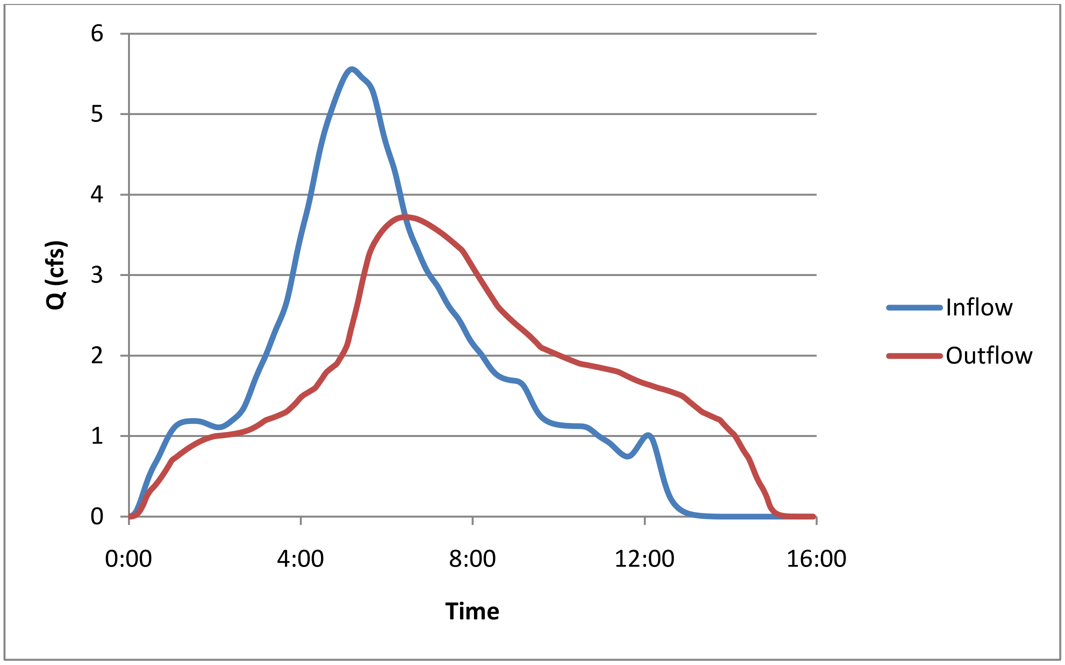

Figure 4.1. Huff Distribution – Flow Rate (Q) vs. Time for Tuscany Hills Detention Basin (25 year, 12 hour Storm)

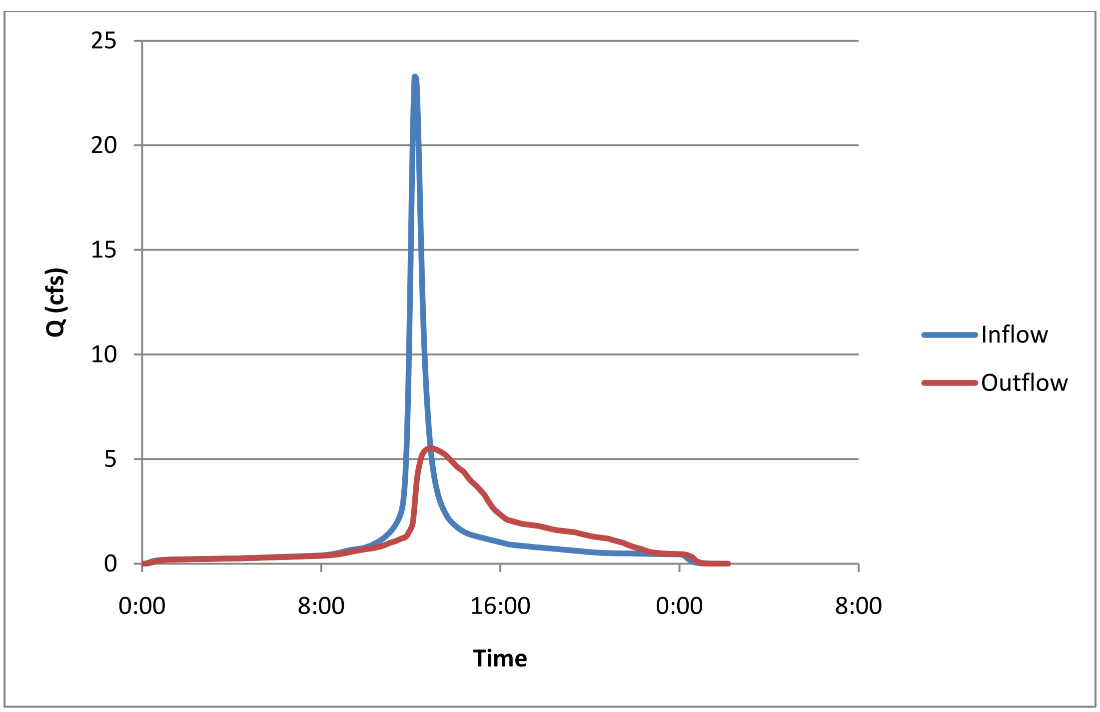

Figure 4.2. NRCS Distribution – Flow Rate (Q) vs. Time for Tuscany Hills Detention Basin (25 year, 24 hour Storm)

storm is reduced by a much larger amount, but it also had a much larger peak discharge to begin with.

4.2. REQUIREMENTS

Detention basins are often used in urban areas where an increase in impervious surfaces has resulted in an increase of the total amount of runoff as well as the peak discharge. Most ordinances pertaining to stormwater flows require the peak discharge leaving the watershed once it is developed to be equal to or less than the peak discharge found before it was developed. This is typically the main requirement when designing a detention basin.

5. WATERSHEDS ANALYZED

The watersheds and detention basins analyzed in this thesis were obtained from the St. Louis Metropolitan Sewer District (MSD). All of the watersheds are located in the St. Louis, Missouri area. MSD provided partial plans and design documents which were submitted to them by design engineers before development. At the time these developments were submitted, MSD required that detention basins were designed using the NRCS 24 hour distribution for 2 year and 100 year frequencies. These detention basins were designed based upon these requirements. Figure 5.1 shows the location of the watersheds relative to the St. Louis area. The following information describes the three watersheds, their individual characteristics, and the values used for their hydrologic models.

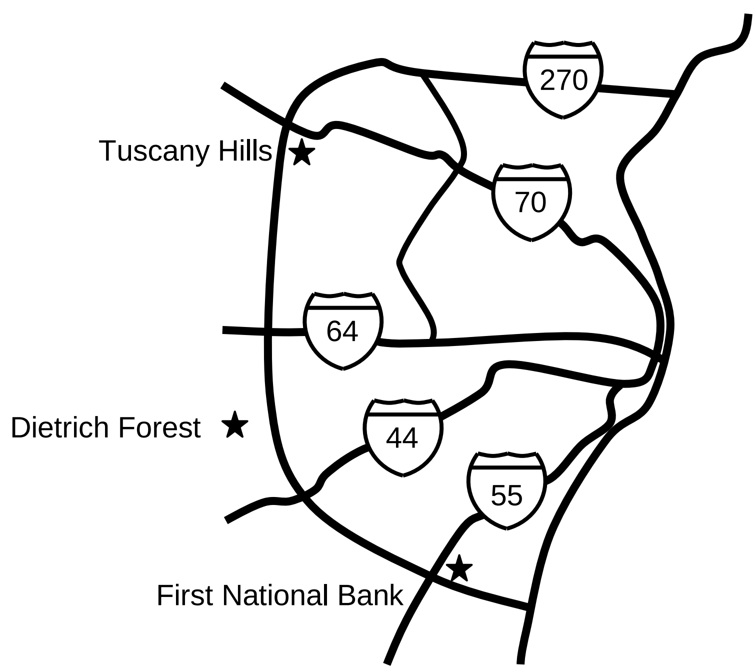

Figure 5.1. Watershed Location Map

5.1. TUSCANY HILLS

The Tuscany Hills watershed is located in the city of Bridgeton in St. Louis County, Missouri. This area was being developed into a residential neighborhood. A large part of this watershed consists of a street and cul-de-sac with twenty homes located on either side of the street. The following gives more detailed information on the undeveloped and developed conditions of the Tuscany Hills watershed.

5.1.1. Undeveloped Conditions. The Tuscany Hills development was found to have a total area of 8.77 acres. The undeveloped conditions land cover was described by a curve number of 67. This curve number yields an initial abstraction of 1.00 inch. Under these conditions, sheet flow and shallow concentrated flow were considered for calculating the basin's lag time. The final lag time was found to be equal to 20.0 minutes.

5.1.2. Developed Conditions. Under developed conditions, the Tuscany Hills watershed was divided into three separate subbasins. The largest of these subbasins, equaling 7.66 acres in size, drained directly into the detention basin. It had a curve number of 77. This was the result of a combination of grass cover and impervious surfaces with curve numbers of 61 and 98 respectively. The lag time of this subbasin was calculated to be 20.2 minutes. The remaining two subbasins, totaling 1.11 acres, drained to a point immediately downstream of the detention basin. The two smaller subbasins were found to be a mixture of grass and pavement. This combination resulted in a composite curve number of 80 for each of them. This curve number yielded an initial abstraction of 0.5 inches.

There was sheet flow, shallow concentrated flow, and channel flow present on these subbasins. The resulting lag time was found to be 3.96 minutes for the smaller of the two subbasins. This subbasin was 0.50 acres. The slightly larger subbasin of 0.61 acres had a lag time of 7.56 minutes. The Tuscany Hills detention basin had a maximum depth of 10 feet and a total volume of 82,330 cubic feet.

5.2. FIRST NATIONAL BANK

The First National Bank watershed is located on Union Road in south St. Louis County, Missouri. The development being designed here consisted of a bank, parking lot, and drive-thru facilities. The new detention basin must handle the increased flows from these areas. The undeveloped and developed conditions are as follows.

5.2.1. Undeveloped Conditions. The First National Bank watershed was the smallest of the three watersheds with an area of 1.45 acres. This development consisted of a single bank and parking lot. This differs from the other two watersheds which were both new subdivisions. The existing conditions were found to have a cover type of brush in good condition. This gives a curve number of 48 and an initial abstraction of 2.17 inches. The overland flow consisted entirely of sheet flow, and the lag time was calculated to be 13.7 minutes.

5.2.2. Developed Conditions. The developed conditions of the First National Bank watershed were split into 15 separate subbasins in order to create a detailed model. These subbasins were either made up of grass or impervious surfaces such as pavement or rooftops. The grass had a curve number of 61 while the impervious surfaces had a curve number of 98. The majority of the subbasins had a short lag time around one or two minutes. Two of the subbasins had longer lag times of 19 and 13 minutes due to the path the runoff had to take in order to reach the detention basin. All 15 of the watersheds flowed directly into the detention basin. The detention basin had a maximum depth of seven feet and a total volume of 19,621 cubic feet.

5.3. DIETRICH FOREST

The Dietrich Forest watershed is located just off of Dietrich Road in St. Louis County, Missouri. Similar to Tuscany Hills, the watershed consists of a residential development containing nineteen lots. New impervious surfaces being added here include roads, driveways, and houses. More detailed information is given for the undeveloped and developed conditions of the Dietrich Forest watershed in the following paragraphs.

5.3.1. Undeveloped Conditions. The Dietrich Forest Development was the largest of the three watersheds at a size of 10.83 acres. The soil was of the hydrologic soil group B, meaning that it had moderate infiltration rates ranging between 0.15 and 0.30 inches per hour. Typical soils in this group are loess or sandy loam. The existing conditions consisted of a residential district with an average lot size of two acres. This cover type along with the hydrologic soil group B gives a curve number of 65. This curve number yields an initial abstraction of 1.08 inches. Sheet flow and shallow concentrated flow were the two types of flow found on the watershed. No channel flow was present under these conditions. The lag time was found to be 14.8 minutes.

5.3.2. Developed Conditions. For the developed conditions of the Dietrich Forest watershed, the cover type was found to be a residential district with an average lot size of 0.5 acres. The hydrologic soil group is unaltered from the existing conditions giving a curve number of 70. This results in initial abstractions of 0.86 inches. The watershed was divided into four separate subbasins. These subbasins all had the same land cover; however, their lag times varied from 7.2 to 18.7 minutes. Two of these subbasins drained directly into the detention basin on the site. The other two drained to a point further downstream. The total area draining to the detention basin was 6.03 acres, leaving 4.80 acres in which the runoff exited the site without any detention. The detention basin had a maximum depth of 9.64 feet and a total volume of 46,206 cubic feet using the Conic Method for reservoir volumes.

6. HEC-HMS

6.1. BACKGROUND

In order to model the watersheds, the Hydrologic Engineering Center – Hydrologic Modeling System (HEC-HMS) was used. It was developed by an organization within the Institute for Water Resources known as the Hydrologic Engineering Center (HEC). HEC is the Center of Expertise for the United States Army Corps of Engineers. Specifically, HEC provides technical expertise in the areas of surface and groundwater hydrology, river hydraulics and sediment transport, hydrologic statistics and risk analysis, reservoir system analysis, planning analysis, real-time water control management, and several other related areas. HEC-HMS is provided free by HEC through their work with the United States Army Corps of Engineers. It is designed to simulate the rainfall-runoff process of watershed systems. Its ability to simulate both rural and urban watersheds makes it a useful tool for this project. Specifically, it allows the user to analyze both a watershed's pre-existing, undeveloped conditions as well as its final, fully developed state.

6.2. SETTING UP THE MODEL

6.2.1. Subbasins. Within HEC-HMS, each section of a watershed is entered into the program as a separate subbasin. For each subbasin in this study, its area was first calculated and then entered into HEC-HMS. Based off of the information available from the St. Louis Metropolitan Sewer District (MSD) for these watersheds, and in order to maintain consistency, the NRCS Curve Number loss method was used for all subbasins. When using the NRCS Curve Number method with HEC-HMS the user must input the initial abstractions, curve number, and percent of impervious area into the program for each subbasin. The curve number was already determined in the data obtained from MSD. A curve number is a single number value between zero and 100 developed by the NRCS. Standard curve numbers were originally developed by doing an analysis of gaged watersheds. The curve number depends upon conditions such as soil characteristics, land cover, and antecedent moisture. A watershed with no storage or abstractions of any kind would be characterized by a curve number of 100. Curve numbers for various watersheds can be determined using tables developed by the NRCS.

As can be seen below, after a watershed's curve number is determined, the storage value can be calculated, followed by the initial abstractions. Once the storage is calculated, the initial abstractions can be found by multiplying the storage by two tenths.

$$S = \frac{1000}{CN} - 10 \tag{1}$$

$$I_A = .2S \tag{2}$$

Where S = storage after runoff begins (inches)

CN = curve number (dimensionless)

I_A = initial abstractions (inches)

In some instances, the percent of impervious area was already given in the previous design calculations provided by MSD. In cases where the impervious area wasn't already provided, it was calculated using the design plans for each development. The curve number, initial abstractions, and percent of impervious area were all entered under the loss method for each subbasin.

Once the loss method data was input into the program, a transform method had to be chosen. The NRCS Unit Hydrograph method was used. This method required the input of a lag time for each subbasin. Lag time is defined as the time between the center of mass of the rainfall and the peak of the hydrograph. There are several empirical equations that have been developed for calculating a watershed's lag time. For subbasins where the time of concentration was already calculated, equation 3 shown below was used. For subbasins where the time of concentration was not available, the NRCS lag equation was used. This equation was designed for areas smaller than 2,000 acres and with a curve number between 50 and 95. This equation is shown below as equation 4.

$$t_L = 0.6t_C \tag{3}$$

$$t_L = \frac{l^{0.8}(1{,}000-9CN)^{0.7}}{1{,}900CN^{0.7}Y^{0.5}} \tag{4}$$

Where t_L = lag time (hours)

t_C = time of concentration (hours)

l = hydraulic length (feet)

CN = curve number

The time of concentration is the time it takes for rain that falls at the hydraulically most remote point of the watershed to reach the outlet. Once a rainfall event lasts for an amount of time equal to the time of concentration, the entire watershed will then be contributing to the runoff at the outlet. The hydraulic length is defined as the length from the most hydraulically remote point in the watershed to the outlet. For these models, there was assumed to be no baseflow present, therefore no baseflow method was chosen in HEC-HMS.

6.2.2. Reaches. The next step in modeling the watersheds in HEC-HMS was to input data for routing runoff through the channels and pipes present. This was done by adding reaches into the model. For the undeveloped conditions, no stormwater system was present to collect the runoff and carry it downstream. This meant that no pipes were present, and no reaches were needed for any of the watersheds. The undeveloped watersheds for all three models were fairly uniform in land cover and slope. This allowed each of them to be modeled as a single subbasin without any routing through reaches. After the watersheds were developed, a stormwater system was present to collect the runoff and carry it downstream. The pipes in the stormwater systems were entered as reaches into the models as needed. HEC-HMS has several options for routing flows through reaches. The kinematic wave routing method was chosen to convey the flows through the pipes. For this method, data had to be input for the length, slope, geometry, and roughness of each pipe. The length and slope of the pipes were obtained from the plans provided by MSD. The plans contained detailed information on the location, length, size, and elevation of the stormwater pipes.

All of the pipes present were circular with increasingly larger diameters as the runoff went downstream. A Manning's n value was chosen to represent the roughness of each pipe. For the concrete pipes, a Manning's n value of .012 was selected.

6.2.3. Detention. The last step in modeling the physical descriptions of the watersheds in HEC-HMS was to input the data for the detention basins. For this task, a table had to be input to show the relationship between the elevations in the detention basin versus the areas at those elevations. A table showing the relationship between the elevations versus discharge out of the basin also had to be input. This allows the program to calculate the storage and discharge from the basin as the model is run. This data was provided in the information obtained from MSD.

6.2.4. Rainfall Data. The next step to model the watersheds was to input the rainfall data. For each storm to be input into the program, a meteorological model had to be added. HEC-HMS allows the user to input precipitation data into the program through what they call a meteorological model. Within each meteorological model for a NRCS storm, several things had to be done. First, there was assumed to be no evapotranspiration or snowmelt. Next, the basins that were going to be included in that meteorological model were selected. For these models, the rainfall was considered uniform over the entire watershed, so all of the basins were chosen. Next the total amount of rainfall had to be input. This rainfall was determined from the Bulletin 71 – Rainfall Frequency Atlas of the Midwest. Bulletin 71 is discussed in greater detail elsewhere in this thesis.

The last item that had to be input was the rainfall distribution type. The NRCS storms are all 24 hour duration storms; however, how that rainfall is distributed over time is dependent upon where the watershed is located within the United States. Since these watersheds are located in the St. Louis, Missouri area, they are a Type II distribution.

For Huff distribution storms, it is necessary to input a larger amount of data into HEC-HMS. First, just like the NRCS storms, a meteorological model was created for each storm. As previously mentioned, there was assumed to be no evapotranspiration or snowmelt present. Due to NRCS storms all having a 24 hour duration and only four possible distribution types, these distributions are all built directly into HEC-HMS. This makes it simple to input the storm by inputting the total amount of rainfall and selecting the distribution as previously described. A Huff distribution storm has many more options when it comes to duration and distribution. Due to this, each storm must first be distributed over time and then manually input into the program. This data was input into the program by creating a rain gauge for each storm. A specific time step for each storm was input along with its duration. The amount of rainfall for each time step was individually input until all of the rainfall was entered. After the rain gauge data was input, the last step was to match each subbasin to the correct rain gauge. This had to be done for each individual meteorological model. Once this was done, the control specifications were set for the model.

6.2.5. Individual Runs. The last step necessary to run the models was to set up a run for each storm under both undeveloped and developed conditions.

Each run requires the selection of the proper basin, meteorological model, and control specifications. Once the runs were created, the models could then be analyzed in order to obtain the results.

7. RESULTS

7.1. METHOD

For this study, the NRCS and Huff distribution methods were compared in several different ways. The first comparison was in the history and nature of the distributions. Specifically, this was the comparison of characteristics such as how they were developed and how they are distributed over time. These characteristics have already been discussed. The next comparison was made by looking at how the same frequency storms compare when using the two distributions to develop them. After the storms themselves were compared, they were then routed through the three watersheds in the models to study how the different distributions affected the runoff. The effectiveness of the detention basins was analyzed in order to see how successful they were in reducing the peak flows coming from the watersheds. The existing and developed conditions were also compared to see if the developed peak runoff was effectively reduced to a level at or below the existing peak runoff. The size of the basin itself was then analyzed to see how well it was utilized.

7.2. DEVELOPED DISCHARGES

Table 7.1 shows the peak flows coming from each watershed for the 100, 25, 10, and 2 year storms. These are the peak flows from the entire watershed for the fully developed condition. This includes routing through the detention basin on each site.

Table 7.1. Peak Discharge for Developed Conditions

	Peak Discharge (cfs)		
	First National Bank	Tuscany Hills	Dietrich Forest
NRCS - 100 yr	1.0000	8.8786	26.5489
Huff - 100 yr, 24 hr	0.7000	4.6447	5.5580
Huff - 100 yr, 12 hr	0.8000	5.6125	6.9468
Huff - 100 yr, 6 hr	0.9000	6.0408	6.4592
Huff - 100 yr, 1 hr	0.9000	5.3480	7.4123
NRCS - 25 yr	0.9000	6.5365	15.3055
Huff - 25 yr, 24 hr	0.5927	3.2567	3.5917
Huff - 25 yr, 12 hr	0.7000	4.1797	3.4542
Huff - 25 yr, 6 hr	0.8000	4.5866	3.0459
Huff - 25 yr, 1 hr	0.8276	4.4301	4.1226
NRCS - 10 yr	0.8000	5.2100	10.8066
Huff - 10 yr, 24 hr	0.4963	2.2718	2.0608
Huff - 10 yr, 12 hr	0.6000	2.9699	2.1795
Huff - 10 yr, 6 hr	0.7261	3.6155	2.0920
Huff - 10 yr, 1 hr	0.8000	3.2768	2.5055
NRCS - 2 yr	0.7000	3.1200	4.3527
Huff - 2 yr, 24 hr	0.3000	1.5525	1.0819
Huff - 2 yr, 12 hr	0.4026	1.7693	1.0415
Huff - 2 yr, 6 hr	0.6000	1.9559	0.9043
Huff - 2 yr, 1 hr	0.7000	1.9587	0.7542

In general, the largest peak flows came from the Dietrich Forest watershed while the smallest flows came from First National Bank. This is the most obvious for the 100 year storm where the discharges for the NRCS distribution range from 26.5849 cfs for Dietrich Forest to 1.000 cfs for First National Bank. Tuscany Hills is in between at 8.8786 cfs. The peak discharges for the Huff distribution were much smaller than that of the NRCS distribution, but the Dietrich Forest watershed still had the largest discharge and First National Bank the smallest for the 100 year storm. The peak discharge was 7.4123 cfs for

Dietrich Forest, 6.0408 cfs for Tuscany Hills, and 0.9000 cfs for First National Bank. For each watershed and every frequency of storm, with the exception of the two and ten year storms for First National Bank, the NRCS distribution gave a peak discharge higher than that of the Huff distribution. For the two exceptions mentioned, the discharges were exactly the same for the two methods. This means that in no instance did the Huff distribution give a higher peak discharge than the NRCS distribution.

Table 7.2 shows the percent difference in peak discharge between the NRCS and Huff distributions for each frequency storm. The equation used to calculate the percent difference in peak flows is shown below.

$$\%\,Difference = \left(\frac{NRCS\ Peak\ Flow - Huff\ Peak\ Flow}{NRCS\ Peak\ Flow}\right) x\ 100 \qquad (5)$$

Table 7.2. Percent Difference in Peak Flows Between Distributions

	Percent Difference		
	First National Bank	**Tuscany Hills**	**Dietrich Forest**
100 yr	10.0%	32.0%	72.1%
25 yr	8.0%	29.8%	73.1%
10 yr	0.0%	30.6%	76.8%
2 yr	0.0%	51.1%	75.1%

Looking at Table 7.2, Dietrich Forest had the largest percent difference in peak flows between the NRCS and Huff distributions. For each storm, the NRCS peak discharge was over 70% larger than the Huff peak discharge. For Tuscany

Hills, the percent difference was about 30% for each frequency storm except for the two year storm, which had a difference of 51.1%. First National Bank had the lowest percent difference to go along with its low peak flows. For these three watersheds, the NRCS distribution gave consistently higher peak discharges. As the size of the watershed increased, the percent difference in the peak discharges between the methods went up as well. This seems to show that when a watershed is modeled using an NRCS distribution storm, it will result in a higher peak discharge coming from the watershed than if it were modeled using a Huff distribution storm. The larger the watershed is, the more this difference is magnified.

7.3. DETENTION BASIN – INFLOW VS. OUTFLOW

Table 7.3 shows the peak discharges for the inflow and outflow of each detention basin for various storms. This is strictly the peak runoff that flows directly into each detention basin for each storm as well as the corresponding peak flow coming out of the outflow structure. A comparison of the relationship between the inflow and outflow will show the effectiveness of the detention basin in reducing the peak discharge. If a detention basin is designed and working correctly, there should be a significant decrease in the outflow as compared to the inflow. For each watershed, the Inflow column shows what the discharge would be from that section of the watershed if there was no detention basin present. The Outflow column shows the discharge that comes from the watershed as a result of the detention basin being present. There is no permanent retention in any of the three detention basins meaning that the total amount of runoff is the same whether the detention basin is present or not. The detention basins' function is to control the runoff by reducing the peak flows coming from the watershed.

Table 7.4 summarizes Table 7.3 by giving the percent change in peak flows from inflow to outflow. For the Huff distribution storms in Table 7.4, the critical storm for each duration was used in order to simplify the results. The critical storm was chosen as the storm with the highest peak inflow for this analysis. That means that the critical storm is the one with the highest peak runoff coming from the watershed and flowing into the detention basin. Since the critical storm is the storm with the highest peak flow, it is the one that most greatly needs to be controlled by the detention basin. The percent changes in Table 7.4 were calculated by the following equation.

$$\%\ Change = \left(\frac{Inflow - Outflow}{Inflow}\right) x\ 100 \tag{6}$$

A quick look at the results shows that for every storm, regardless of which rainfall distribution was used, the detention basin was successful in reducing the peak discharge. In no case was the runoff allowed to flow freely without any detention occurring. This would occur if the outflow structure in a detention basin was too large allowing for a higher discharge than the flows entering the detention basin. The total amount of runoff for each watershed is the same since the NRCS curve number loss method was used with each run. For each

Table 7.3. Detention Basin Peak Inflow vs. Peak Outflow (cfs)

	First National Bank		Tuscany Hills		Dietrich Forest	
	Inflow	Outflow	Inflow	Outflow	Inflow	Outflow
NRCS - 100 yr	9.7118	1.0000	31.8191	6.8642	22.4376	10.3322
Huff - 100 yr, 24 hr	0.9217	0.7000	5.1549	4.0798	3.2015	3.0755
Huff - 100 yr, 12 hr	1.3760	0.8000	7.6306	4.9497	4.2153	3.8154
Huff - 100 yr, 6 hr	2.6639	0.9000	11.9415	5.3551	4.7514	3.6171
Huff - 100 yr, 1 hr	7.0596	0.9000	20.8392	5.3480	7.1798	2.9997
NRCS - 25 yr	7.2005	0.9000	23.2682	5.5114	13.5240	5.8831
Huff - 25 yr, 24 hr	0.6794	0.5927	3.8175	2.8875	2.1584	1.9687
Huff - 25 yr, 12 hr	1.0077	0.7000	5.5551	3.7199	2.7513	1.9467
Huff - 25 yr, 6 hr	2.0109	0.8000	8.4183	4.0976	2.8726	1.6398
Huff - 25 yr, 1 hr	5.4514	0.8276	14.8707	4.1198	3.8250	0.6591
NRCS - 10 yr	5.7531	0.8000	18.2720	4.5636	9.4467	3.0456
Huff - 10 yr, 24 hr	0.5391	0.4963	3.0261	1.9884	1.5749	1.0993
Huff - 10 yr, 12 hr	0.7986	0.6000	4.3540	2.6464	1.9354	1.0217
Huff - 10 yr, 6 hr	1.6674	0.7261	6.4799	3.2432	1.9115	0.6976
Huff - 10 yr, 1 hr	4.5173	0.8000	11.5520	3.2485	2.2622	0.5964
NRCS - 2 yr	3.5717	0.7000	10.6231	2.3656	3.7378	0.6000
Huff - 2 yr, 24 hr	0.3267	0.3000	1.7958	1.3787	0.7415	0.5126
Huff - 2 yr, 12 hr	0.4824	0.4026	2.5057	1.5645	0.7667	0.4879
Huff - 2 yr, 6 hr	1.1094	0.6000	3.7531	1.7616	0.6482	0.4393
Huff - 2 yr, 1 hr	3.0194	0.7000	6.7972	1.8076	0.5861	0.3237

Table 7.4. Percent Reduction in Peak Flows Due to Detention

	Percent Change		
	First National Bank	Tuscany Hills	Dietrich Forest
NRCS - 100 yr	89.7%	78.4%	54.0%
Huff - 100 yr	87.3%	74.3%	58.2%
NRCS - 25 yr	87.5%	76.3%	56.5%
Huff - 25 yr	84.8%	72.3%	82.8%
NRCS - 10 yr	86.1%	75.0%	67.8%
Huff - 10 yr	82.3%	71.9%	73.6%
NRCS - 2 yr	80.4%	77.7%	83.9%
Huff - 2 yr	76.8%	73.4%	30.9%

storm frequency, the NRCS distribution gave a higher peak inflow than any of the Huff distribution storms. This means that the bulk of the runoff came in a much shorter time for the NRCS distribution. This higher peak inflow yielded a higher peak outflow as well.

The percent reduction in peak flows for First National Bank and Tuscany Hills came out to be somewhat similar. For First National Bank, the percent reduction in peak flows for the NRCS distribution ranged from 89.7% for the 100 year storm to 80.4% for the two year storm. In each case, the percent reduction for the Huff distribution was approximately four percent lower than that of the NRCS distribution meaning that the Huff distribution storms were not reduced as much as the NRCS distribution storms on a percentage basis. The Huff distribution storms ranged from 87.3% to 76.8% reduction. For Tuscany Hills, the percent reduction in peak flows for the NRCS distribution ranged from 78.4% for the 100 year storm to 77.7% for the two year storm. Once again the percent reduction was approximately four percent lower for each frequency storm for the Huff distribution. Those values ranged from 74.3% to 73.4%.

The results for Dietrich Forest varied compared to those of First National Bank and Tuscany Hills. For the 100 year storm, the two distributions varied by 4.2%. In this case, the Huff distribution had the larger reduction at 58.2%. For the 25 year storm, the reduction for the NRCS distribution stayed very similar at 56.5%; however, the reduction for the Huff distribution jumped to 82.8%. This is a difference of 26.3% between the distributions. For the ten year storm, the percent reductions varied by 5.8% with the Huff distribution still being the highest at 73.6%. The distributions switched for the two year storm as the NRCS distribution had the highest percent reduction at 83.9%. The Huff distribution had a reduction of 30.9%.

For First National Bank and Tuscany Hills, the detention basin was more successful in reducing peak flows for the NRCS distribution storms. As previously stated, there was about a four percent greater reduction in peak discharges by the detention basin for the NRCS distribution storms as compared to that of the Huff distribution. For Dietrich Forest, the distribution with the highest reduction in peak flows varied based upon which frequency storm was being modeled.

7.4. EXISTING VS. DEVELOPED PEAK DISCHARGES

When an area is developed, it is typically required that the peak flows coming from the watershed after it is developed be less than or equal to the peak flows found for the same watershed before it is developed. This must be true for the entire development. Any increase in flows due to the addition of impervious areas, or any other change in the watershed, must be controlled by the detention basin. Sometimes not all of the watershed will drain to the detention basin that is placed on site. In these cases, the peak flows leaving the entire watershed must still be equal to or less than the peak flows that previously existed. Thus far, the peak discharges for the water flowing directly to the detention basin have been analyzed. In doing this it was found that the peak flow going to the detention basin was reduced in almost every case. It is now necessary to look at the entire watershed to see if the developed peak flows are smaller than or equal to the existing flows. The existing discharges are the flows found to be coming from the watershed before development.

Table 7.5 shows the existing and developed peak discharges for each watershed and distribution type for a 100, 25, 10, and 2 year storm. For a developed watershed to meet standard design requirements, the developed peak discharge must not be greater than the existing peak discharge. As can be seen from this table, this was not always found to be the case. Table 7.6 and Table 7.7 help to compare and summarize this information.

Table 7.5. Existing vs. Developed Peak Discharges (cfs)

	First National Bank		Tuscany Hills		Dietrich Forest	
	Existing	Developed	Existing	Developed	Existing	Developed
NRCS - 100 yr	1.5693	1.0000	23.8874	8.8786	34.9244	26.5849
Huff-100 yr, 24 hr	0.3119	0.7000	4.2857	4.6447	5.0707	5.5580
Huff-100 yr, 12 hr	0.3072	0.8000	5.5379	5.6125	6.4843	6.9468
Huff-100 yr, 6 hr	0.2567	0.9000	5.9277	6.0408	6.8521	6.4592
Huff-100 yr, 1 hr	0.2163	0.9000	7.7349	5.3480	9.3661	7.4123
NRCS - 25 yr	0.6563	0.9000	15.0147	6.5365	20.1124	15.3055
Huff-25 yr, 24 hr	0.1680	0.5927	2.8286	3.2567	3.3159	3.5917
Huff-25 yr, 12 hr	0.1378	0.7000	3.4833	4.1797	4.0232	3.4542
Huff-25 yr, 6 hr	0.1107	0.8000	3.4762	4.5866	3.9406	3.0459
Huff-25 yr, 1 hr	0.0471	0.8276	3.9574	4.4301	4.5848	4.1226
NRCS - 10 yr	0.2600	0.8000	10.1357	5.2100	13.3529	10.8066
Huff-10 yr, 24 hr	0.0939	0.4963	2.0436	2.2718	2.3649	2.0608
Huff-10 yr, 12 hr	0.0661	0.6000	2.3656	2.9699	2.6829	2.1795
Huff-10 yr, 6 hr	0.0482	0.7261	2.2165	3.6155	2.4461	2.0920
Huff-10 yr, 1 hr	0.0001	0.8000	2.2587	3.2768	2.4446	2.5055
NRCS - 2 yr	0.0096	0.7000	3.4492	3.1200	4.2287	4.3527
Huff-2 yr, 24 hr	0.0140	0.3000	0.8956	1.5525	0.9979	1.0819
Huff-2 yr, 12 hr	0.0090	0.4026	0.8278	1.7693	0.8813	1.0415
Huff-2 yr, 6 hr	0.0034	0.6000	0.6762	1.9559	0.7103	0.9043
Huff-2 yr, 1 hr	0.0000	0.7000	0.4745	1.9587	0.4561	0.7542

Table 7.6. Change in Peak Discharge From Existing Conditions

	Change in Discharge (cfs)		
	First National Bank	**Tuscany Hills**	**Dietrich Forest**
NRCS - 100 yr	-0.5693	-15.0088	-8.3395
Huff - 100 yr	0.6837	0.1131	-1.9538
NRCS - 25 yr	0.2437	-8.4782	-4.8069
Huff - 25 yr	0.7805	1.1104	-0.4622
NRCS - 10 yr	0.5400	-4.9257	-2.5463
Huff - 10 yr	0.7999	1.3990	0.0609
NRCS - 2 yr	0.6904	-0.3292	0.1240
Huff - 2 yr	0.7000	1.4842	0.0840

Table 7.7. Percent Change in Peak Discharge From Existing Conditions

	Percent Change		
	First National Bank	**Tuscany Hills**	**Dietrich Forest**
NRCS - 100 yr	-36.3%	-62.8%	-23.9%
Huff - 100 yr	316.1%	2.0%	-20.9%
NRCS - 25 yr	37.1%	-56.5%	-23.9%
Huff - 25 yr	1657.1%	31.9%	-10.1%
NRCS - 10 yr	207.7%	-48.6%	-19.1%
Huff - 10 yr	799900.0%	63.1%	2.5%
NRCS - 2 yr	7191.7%	-9.5%	2.9%
Huff - 2 yr	-	312.8%	8.4%

Table 7.6 shows the change in the peak discharge that occurs when the watershed was developed in cubic feet per second. A negative value means that the peak discharge for the developed condition is less than that for the existing condition. In other words, the detention basin was successful in reducing the peak flows from the watershed to the point that the developed peak discharge was less than that of the existing peak discharge. Table 7.7 is similar to Table

7.6 except that it shows the percent change in peak discharge rather than the actual change. Again, a negative value means that the peak discharge found after development is less than the peak discharge for the existing conditions. These tables only show the critical storm for each frequency of the Huff distribution. The storm which yielded the highest peak discharge under developed conditions was chosen as the critical storm. The equations used for these two tables are given below.

$$\Delta Q_P = Developed\ Q_P - Existing\ Q_P \tag{7}$$

$$\%\ Change = \frac{Developed\ \ Q_P - Existing\ \ Q_P}{Existing\ \ Q_P}\ x\ 100 \tag{8}$$

Where Q_P *= Peak Discharge (cfs)*

For First National Bank the only storm to show a decrease in the peak discharge was the NRCS 100 year storm. For every other case regardless of the distribution, the peak discharge increased when the watershed was developed. Looking at the percent change in peak discharges for First National Bank makes the increase in flow seem quite dramatic. A look at the actual change in cubic feet per second lessens this. While the percent changes are quite large, the flows themselves are very small. For example, the Huff 25 year storm shows that the developed conditions caused the peak discharge to increase by 1657.1%. While this is true, this only equates to a change of 0.7805 cfs. The percent change for the Huff two year storm is blank because the amount of flow was negligible for that storm under the existing conditions.

In the case of Tuscany Hills, the peak discharge decreased under developed conditions for each of the NRCS storms. The higher the peak discharge, the greater the decrease was found to be. This means that all of the storms modeled with the NRCS distribution would meet the peak flow reduction criteria. In contrast, the peak discharges for the Huff storms all increased under the developed conditions. That means that for Tuscany Hills, the development would meet the peak discharge requirement if modeled with the NRCS distribution, but it would not meet the requirements if it were modeled with the Huff distribution.

For Dietrich Forest, both the Huff and NRCS distributions for the 100 and 25 year storms yielded a lower peak discharge under developed conditions. For the ten year storm, only the NRCS distribution produced a lower peak discharge. The Huff distribution produced a slightly higher peak discharge for the developed condition. The peak discharge increased for both distributions when the two year storm was routed through the watershed. This means that this watershed meets the discharge requirement for large storms; however, it does not properly reduce peak flows for smaller flows. It should also be noted that the Huff distribution fell under non-compliance sooner than the NRCS distribution.

Another item to take note of is that for every frequency storm on every watershed the NRCS distribution was more successful in reducing the peak discharge than the Huff distribution. In cases where both distributions reduced the peak discharge, the NRCS peak flow was reduced by a larger total amount as well as by a larger percentage change. In many cases, the use of the NRCS distribution resulted in the reduction of peak flows when the watershed was developed while the Huff distribution produced an increase. In these cases, the choice of distributions is very important as it would determine whether or not a site is in compliance with the regulations. In these cases, the development is only in compliance when the NRCS distribution is used. When the Huff distribution is used, the detention basin fails to reduce the peak discharge to a level less than that found under the existing conditions. Lastly, in each case where the peak discharge increased under developed conditions for both distributions, the Huff method yielded the largest increase. Based off of this analysis, the NRCS method gives the largest peak flows overall; however, for the design of a detention basin to reduce peak flows, the Huff distribution produces the critical storms.

7.5. DETENTION BASIN ELEVATIONS

When studying the effectiveness of a detention basin, it is not only important to look at how well the basin performs the task of reducing peak flows; It is also important to take a look at how well the detention basin is utilized. For this study, this was done by looking at how much of the basin’s total volume or depth was used for various storms. For example, if a very small amount of freeboard, or none at all, is present above the high water level, the basin may be undersized for that particular storm. If this is the case, the basin would need to be made larger to safely and adequately store the runoff that it collects. This would ensure that the water is detained and ushered through the detention basin's outflow structure. If the detention basin is too small, the water may flow over the top of the sides of the detention basin resulting in little or no decrease, or even possibly an increase, in the peak discharge. This could also lead to flooding downstream. The water must pass through the detention basin's outflow structure for the detention basin to function as designed. If the high water level is very low in comparison to the top of the detention basin, the basin may be oversized. In this case, one option would be to make the basin smaller in plan view in order to decrease the overall area needed. Reducing the size of the detention basin in this way would free up more area on the site for other uses. The detention basin could also be made shallower in order to reduce its size. Either method for decreasing the size, or a combination of the two, would lessen the construction costs and therefore reduce the cost of the detention basin resulting in a more efficient and cost effective design.

Table 7.8 shows the maximum water surface elevation reached in each detention basin for the various storms. At the top of the table, the minimum and maximum elevations of each detention basin are given for reference. Table 7.9 shows the amount of freeboard present in each detention basin for each frequency storm. For the Huff distribution, only the critical storm for each frequency is shown. The storm with the highest water surface elevation for each given frequency was chosen to be the critical storm. This storm was chosen as critical because the detention basin will be rendered ineffective if the water is allowed to flow over its banks without being controlled. The equation used to calculate the freeboard is shown below. In this equation, the max elevation is the elevation of the top of the detention basin. The max water surface elevation is the highest elevation that the water surface reaches within the detention basin. All values are given in feet.

$$Freeboard = Max\ Elevation - Max\ Water\ Surface\ Elevation \tag{9}$$

Table 7.8. Detention Basin Maximum Water Surface Elevations

	Maximum Elevations (feet)		
	First National Bank	**Tuscany Hills**	**Dietrich Forest**
Min / Max Elevation	**585 / 592**	**560 / 570**	**561.34 / 571**
NRCS - 100 yr	590.358	567.896	567.706
Huff - 100 yr, 24 hr	587.867	564.260	565.191
Huff - 100 yr, 12 hr	588.767	565.150	565.369
Huff - 100 yr, 6 hr	589.547	565.633	565.320
Huff - 100 yr, 1 hr	589.788	565.622	565.173
NRCS - 25 yr	589.488	565.867	565.901
Huff - 25 yr, 24 hr	586.985	563.423	564.928
Huff - 25 yr, 12 hr	587.758	563.920	564.923
Huff - 25 yr, 6 hr	588.625	564.273	564.849
Huff - 25 yr, 1 hr	589.055	564.290	564.136
NRCS - 10 yr	588.918	564.745	565.184
Huff - 10 yr, 24 hr	586.393	562.833	564.562
Huff - 10 yr, 12 hr	587.118	563.320	564.519
Huff - 10 yr, 6 hr	588.052	563.576	564.328
Huff - 10 yr, 1 hr	588.514	563.578	563.322
NRCS - 2 yr	587.836	563.159	563.589
Huff - 2 yr, 24 hr	585.696	561.618	562.903
Huff - 2 yr, 12 hr	586.205	561.993	562.779
Huff - 2 yr, 6 hr	587.006	562.342	562.536
Huff - 2 yr, 1 hr	587.625	562.423	562.149

Table 7.9. Detention Basin Freeboard

	Freeboard (feet)		
	First National Bank	**Tuscany Hills**	**Dietrich Forest**
NRCS - 100 yr	1.642	2.104	3.294
Huff - 100 yr	2.212	4.367	5.631
NRCS - 25 yr	2.512	4.133	5.099
Huff - 25 yr	2.945	5.710	6.072
NRCS - 10 yr	3.082	5.255	5.816
Huff - 10 yr	3.486	6.422	6.438
NRCS - 2 yr	4.164	6.841	7.411
Huff - 2 yr	4.375	7.577	8.097

The detention basin for First National Bank has a maximum elevation of 592 feet. This means that if the water surface level goes higher than 592 feet, it will be allowed to spill out over the sides of the detention basin. The bottom elevation in the detention basin is 585 feet, giving a total depth of seven feet. Similarly, the depth of the Tuscany Hills detention basin is ten feet, while the Dietrich Forest detention basin has a depth of 13.66 feet. A look at the values in Table 8.8 shows that in no case did the maximum water surface elevation exceed the maximum elevation of the detention basin. It was also found that the maximum water surface elevation for the NRCS storms was higher than that of the Huff storms for each given frequency. A graphical representation of this is shown in Figure 7.1. It shows the total depth of each detention basin along with the maximum depths from the NRCS and Huff distribution 100 year storms. Only the 100 year depths are shown because these storms give the most runoff and the greatest depths within the basin.

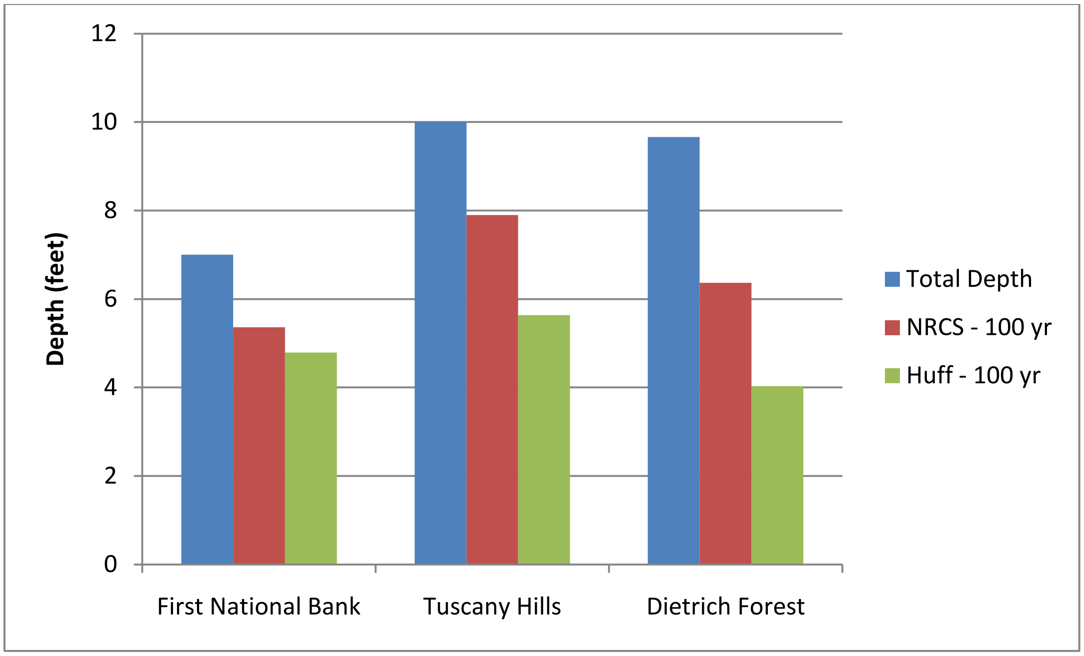

Figure 7.1. Detention Basin Maximum Depth

As previously stated, you can see from Figure 8.1 that none of the detention basins' storage capacity is exceeded by the 100 year storm regardless of which distribution was used. It can also be seen that in each case, the NRCS distribution resulted in a greater depth than that of the Huff distribution. This is not only true for the 100 year storm, but every other storm that was run as well. In each case the NRCS distribution resulted in a greater depth than that of the Huff distribution.

For the First National Bank detention basin, there was 1.642 feet of freeboard for the 100 year NRCS distribution storm. For the Huff distribution method, the same 100 year event yielded 2.212 feet of freeboard. This is an additional 0.57 feet of freeboard above that of the NRCS distribution. For Tuscany Hills, The 100 year NRCS storm yielded 2.104 feet of freeboard. The

100 year Huff distribution storm resulted in 4.367 feet of freeboard. In this case, the Huff distribution storm had an extra 2.263 feet above that of the NRCS distribution. In fact, the freeboard from the Huff distribution is over double that of the NRCS distribution. For Dietrich Forest, the 100 year NRCS distribution gave a freeboard of 3.294 feet. The 100 year Huff distribution yielded 5.631 feet of freeboard. In this case, the Huff distribution had 2.337 more feet of freeboard over the NRCS distribution.

Freeboard requirements vary depending upon where the detention basin is being constructed. Most requirements range from one to two feet. All of these detention basins have well over one foot of freeboard, and only the NRCS 100 year storm for First National Bank did not have two feet of freeboard. According to this, it appears that all of the detention basins have an adequate storage capacity to safely and effectively hold the runoff coming from each watershed.

8. CONCLUSIONS

8.1. OVERALL

After studying the NRCS and Huff rainfall distribution methods, it is clear that the two are not the same. Using one method instead of the other for detention basin design will certainly change the final detention design of the detention basin and its outlet structure. Since it has been determined that using the different methods will yield different results, the question then becomes, which method will produced the best functioning detention basin. The following is a summary of how the two distribution methods are different, how they performed in the modeling of the three watersheds found this study, and the final conclusions of the preferred method for detention basin design.

8.2. DISTRIBUTIONS

The storms themselves are distributed differently over time in that the NRCS method is spread out over 24 hours with a very intense period of rainfall in the middle. This is intended to simulate both a long duration and short duration storm within one distribution. The result is a storm that does not look similar to a naturally occurring storm of either duration. The lesser intensities for the majority of the storm are extremely small while the high intensities in the middle of the distribution are very large. The majority of a natural storm will have intensities somewhere in between these two extremes. In their attempt to develop a single distribution for all storms, it appears the developers have instead created a distribution that does not accurately represent either a long or short duration naturally occurring storm. The NRCS distribution did result in the highest peak discharges coming from each of the watersheds that were modeled. While some would say that this makes it the "conservative" method, it does support the idea that the NRCS distribution is more of a worst case scenario than the statistically average value. Before using the term conservative, one must understand what values are the most important. For detention basin design, we are most concerned with reducing the post development peak discharges to at or below the predevelopment levels. With that in mind, the most conservative, or critical, method would then be the one that requires the best design to meet this standard and comply with regulations. As discussed in the next section, based upon this analysis, the Huff method is the critical method.

The Huff method allows the user to model several different duration storms in order to determine the critical one. It is distributed differently over time as compared to the NRCS distribution. Which Huff distribution is used is based upon the duration of the storm. While there are periods of intense rainfall within each Huff distribution, the storms are more evenly distributed over time than the storms created using the NRCS distribution. The lower intensities are greater than the NRCS low intensities, and the peak intensities are much less than the peak intensities of the NRCS method. This resulted in lower peak discharges coming from all three watersheds than those of the NRCS storms.

8.3. ANALYSIS

All of the detention basins that were modeled, successfully collected, stored, and released the runoff that flowed to them. In this sense, they all functioned as you would expect a properly designed detention basin to function. This was true for every frequency storm regardless of which distribution method was used. After modeling these three watersheds, the NRCS peak discharges were reduced by a much larger amount in cubic feet per second than the Huff peak discharges when routed through the detention basins; however, this appears to be due to the fact that the peak discharges coming from the watersheds were already much higher for the NRCS method. The actual percent reduction in peak discharges tended to be similar for the two methods. These higher peak discharges coming from the watersheds as a result of the NRCS storms resulted in higher peak discharges coming from the detention basins when compared to the discharges from the Huff distribution storms. The next step after determining that the detention basins were successfully storing and reducing flows was to determine if the detention basins met the proper requirements for detention basin design.

The main requirement for detention basin design is whether or not the basin can reduce the developed peak discharge to a level at or below what the peak discharge was before the watershed was developed with the intent that this would be true for all naturally occurring storms of the design frequency in the future. For the three watersheds modeled, in approximately 67 percent of the cases the peak discharge for the NRCS storm was reduced to at or below the predevelopment levels. The only times this did not occur was when there were very low flows, such as those found at the First National Bank watershed. All of the detention basins that were modeled had previously been designed using the NRCS distribution method. Since the designs had been accepted by the St. Louis Metropolitan Sewer District, it was to be expected that the detention basins would function properly when modeled using the NRCS distribution. However, in contrast, when the same detention basins were modeled using the Huff distribution, the peak discharge was reduced to the predevelopment level 17 percent of the time, meaning that the rest of the cases did not meet the requirements. In fact, if you only look at the cases were the NRCS peak discharge was successfully reduced to predevelopment levels, only in 25 percent of those cases was the Huff peak discharge adequately reduced. This shows that these detention basins were successful in reducing the extremely high peak discharges produced by the NRCS method, but they were not successful in adequately reducing the peak discharges that were produced by the Huff distributed storms which are based on and are more similar to naturally occurring storms. If the detention basins had properly reduced the peak discharges for both methods, then one could conclude that the detention basins were meeting the regulations for each case. Since the Huff storms, which more accurately represent a naturally occurring storm, do not meet the peak discharge reduction regulations, this leads to the conclusion that detention basins designed and constructed with the NRCS method may not actually be performing the job they were intended for. While these detention basins do reduce the flows that come to them, they do not reduce the flows to the predevelopment levels that are required of them. Based on this analysis, the Huff Distribution Method is the critical distribution method for meeting predevelopment peak discharges and should therefore be the method of choice for detention basin design.

The results of this analysis also bring about a concern involving the many existing detention basins that have been designed using only the NRCS distribution method. It is likely that the areas downstream of the basins are experiencing higher flows than were existent before development. These higher peak flows have undoubtedly resulted in more frequent and severe flooding as well as increased erosion downstream. In order for these basins to successfully reduce peak discharges to levels at or below the predevelopment levels, modifications would need to made to their outlet structures. Additional analysis would be required in order to determine the proper modifications that had to be made. In order to prevent future problems, cities and municipalities requiring the NRCS method should consider revising their standards. In addition, engineers should use the Huff method instead of the NRCS method for detention basin design. Even in areas where the NRCS method is required, engineers should at the least check their design by using the Huff method before finalizing their plans.

One additional requirement for detention basin design is that a basin must have adequate storage capacity. This is to ensure that the runoff flowing to the basin is captured and held so that it can be released through the outflow structure. In every case, an adequate amount of freeboard was present to ensure that the runoff would not flow over the sides of the detention basin, which would render the basin ineffective. None of the basins were undersized according to the models, regardless of which distribution was used. When comparing the storage capacity needed for each of the two distribution methods, the Huff distribution produced a larger amount of freeboard than that of the NRCS method for each storm. This means that a detention basin designed with the NRCS method would have to be larger than a detention basin designed with the Huff method in order to ensure an adequate amount of freeboard. The high peak flows as a result of the NRCS distribution require the basin to store more water at one time. To account for this difference in storage capacity, a detention basin designed with the Huff Distribution Method should be given sufficient freeboard to ensure that the runoff is properly contained. While this is something to consider, the conclusion of this thesis remains that the Huff distribution method should be the preferred method for detention basin design.

BIBLIOGRAPHY

Akan, A. Osman. Open Channel Hydraulics. Burlington: Butterworth-Heinemann, 2006.

Akan, A. Osman and Robert J. Houghtalen. Urban Hydrology, Hydraulics, and Stormwater Quality: Engineering Applications and Computer Modeling. Hoboken: John Wiley and Sons, Inc., 2003.

Bedient, Philip B. and Wayne C. Huber. Hydrology and Floodplain Analysis. 3rd ed. Upper Saddler River: Prentice Hall, 2002.

Butler, Stanley S. Engineering Hydrology. Englewood Cliffs, NJ: Prentice Hall, 1957.

Chin, David A. Water-Resources Engineering. Upper Saddle River: Prentice Hall, 2000.

Chow, Ven Te, David R. Maidment, and Larry W. Mays. Applied Hydrology. New York: McGraw-Hill, Inc., 1988.

Crowe, Clayton T., Donald F. Elger, and John A. Roberson. Engineering Fluid Mechanics. 8th ed. Hoboken: John Wiley and Sons, Inc., 2005.

Gribbin, John E. Introduction to Hydraulics and Hydrology with Applications for Stormwater Management. 3rd ed. Clifton Park: Thomson Delmar Learning, 2007.

Guo, James C. Y. and Kelly Hargadin. “Conservative Design Rainfall Distribution.” *J. Hydrologic Engrg*. Volume 14, Issue 5 (May 2009): 528-530.

Gupta, Ram S. Hydrology and Hydraulic Systems. Second Edition. Prospect Heights: Waveland Press, 2001.

Huff, Floyd A. and James R. Angel. Rainfall Frequency Atlas of the Midwest. Midwest Climate Center, 1992.

Linsley, Ray K., Max A. Kohler, and Joseph L.H. Paulhus. Hydrology for Engineers. New York: McGraw-Hill, 1982.

Marke, Mark A. Hydraulic Design Manual. Design Division, 2009.

Mays, Larry W. Water Resources Engineering. 2005 Edition. Hoboken: John Wiley and Sons, Inc., 2005.

McCuen, Richard H. Hydrologic Analysis and Design. Third Edition. Upper Saddle River: Pearson Prentice Hall, 2005.

Singh, Vijay P. Elementary Hydrology. Englewood Cliffs, NJ: Prentice Hall, 1992.

Thompson, David B., William H. Asquith, and Theodore G. Cleveland. "Dimensionless Hyetographs and Distribution of Storm Depth: Findings and Recommendations," Project Summary Report, Texas Tech University, Austin, Texas.

"TR 55: SCS (NRCS) 24-hour Synthetic Rainfall Distributions," http://www.bossintl.com/tr55-rainfall-distributions.html, March, 2009.

Wurbs, Ralph A. and Wesley P. James. Water Resources Engineering. Upper Saddle River: Prentice Hall, 2002.

VITA

Todd Wayne Dablemont was born in Rolla, Missouri, on April 11, 1984. In May 2006 he received his Bachelor of Science degree in Civil Engineering from the University of Missouri-Rolla, now known as the Missouri University of Science and Technology. He completed his course work for his Masters of Science degree in Civil Engineering at the Missouri University of Science and Technology while performing duties as a Graduate Teaching Assistant and Graduate Research Assistant. As a student he was a member of Chi Epsilon Civil Engineering Honor Society, Phi Kappa Phi Honor Society, the Order of the Engineer, the Chancellor's Leadership Academy, and the Baptist Student Union. After completing his coursework, he began working for Morris & Munger Engineers LLC, a civil engineering consulting company specializing in water resources, located in Rolla, Missouri.