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Stormwater Detention Pond Analysis

prepared by

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16. Abstract

A detention pond is a small impoundment of water used for flood protection. Several methods are available to design detention ponds. The objective of this report is to investigate the design of detention pond facilities and provide Nebraska Department of Roads (NDOR) with a consistent, methodical procedure for evaluating and designing detention pond facilities. This report is composed of an introduction to the history of stormwater management and the policies that evolved as related to detention pond design. It also reviews literature of the existing detention pond design methods to determine the design storm, runoff volume, detention storage capacity and each of the methods respective limitations. A simultaneous solution for the storage volume and outlet facility design is reviewed. A design example of an existing detention pond is presented for analysis and evaluation of existing methods. A recommendation and conclusion of a design method are provided for future use.

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STORMWATER DETENTION POND ANALYSIS

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Chapter 1: Introduction

1.1 Overview

It is widely recognized that land development, especially in urban areas, is responsible for significant changes in runoff characteristics. The removal of trees and vegetation prevents interception and often decreases the permeability of the surface soil layer. Changes in topography reduce the potential for storage of rainfall in the soil matrix. In urban areas, increased impervious cover also diminishes the possibility for infiltration and soil storage of rainwater. Any change of the natural storage, through means of interception, depression, or soil storage produces changes in the runoff characteristics.

Runoff characteristics that may be affected are the total volume and the peak of the surface (or direct) storm runoff. Both of these characteristics increase in magnitude with the elimination of natural storage. In addition, the timing of runoff is altered, specifically by a decrease in both the time to peak and the time of concentration.

Modifications of these runoff characteristics result in a physical change to the landscape. When runoff velocities increase, the risk of surface rill and gully erosion intensifies. Higher stream velocities may also increase the rate of bedload movement. Land development is often associated with adjustments of drainage patterns and channel characteristics. For example, channels may be cleared of vegetation and straightened, with some also being lined with concrete or riprap. Channel alteration may result in less channel storage and roughness, both of which can increase flow velocities and the potential for flooding at locations downstream from the developing area.

Ultimately, changes in the runoff characteristics of an area affect the inhabitants of that community. Recognizing this, stormwater management policies have been introduced with the intent of mitigating the negative hydrologic impacts of lost natural storage. Stormwater management policies have been adopted to limit peak flow rates from developed areas to that which occurred prior to development. In addition to specifying the conditions under which stormwater management methods must be used, these policies are designed to maintain runoff characteristics after development to those that existed prior to development. In analytical terms this means that the flood frequency curve for the post-development conditions coincides with the curve for the pre-development conditions. Furthermore, policy statements usually specify one or two exceedence frequencies (i.e. return periods) at which the post-development peak rate must not exceed the pre-development peak rate for the same exceedence frequency. Such policies usually use return periods of 2,10, 25, 50, or 100 years as the target points of the frequency curve. Ideally, stormwater management policies specify a particular design method to be used in design. Although data does not exist that suggests that any one method is best, the identification of a single method as part of a stormwater management policy will ensure design consistency.

Throughout the history of stormwater management, a variety of methods have been developed to regulate excess precipitation. Frequently, this is accomplished by creating manmade storage. Among the methods for creating manmade storage, the basin is one of the most popular solutions. In particular, the detention pond type basin will be discussed here.

A number of procedures have been proposed for use in designing stormwater detention facilities. Design requires the simultaneous sizing of both the storage volume characteristics and the riser/outlet characteristics. Some stormwater management methods can only be used to estimate the volume of storage that would be required to satisfy the intent of the stormwater management policies; will refer to such methods as *preliminary sizing methods*. Other methods are used to determine the characteristics of the outlet facility. Ultimately, the final design should incorporate a method that simultaneously estimates the volume of storage and the characteristics of the outlet facility; will refer to these types of methods as *the design methods*. The simultaneous solution of the volume of storage and the outlet facility characteristics is important because there are a wide array of feasible solutions for any one site and set of design conditions. Independently determining the volume of storage and the characteristics of the outlet facility can introduce inconsistencies and lead to an ineffective design.

1.2 Objectives

The objective of this report is to review and assess existing procedures for sizing detention ponds and to recommend a consistent, methodical procedure for evaluating and designing detention pond facilities.

1.3 Report Outline

This report is composed of four chapters. Chapter one is an introduction to the history of stormwater management and the policies that evolved as related to detention pond design. Chapter two is a summary of the literature review of the existing detention pond design methods to determine the design storm, runoff volume, detention storage capacity and each of the methods respective limitations. In this chapter, a simultaneous solution for the storage volume and outlet facility design will be reviewed. Chapter three follows a design example of an existing detention pond for analysis and evaluation of existing methods. Finally, in Chapter four is a conclusion and recommendation of a design method for future use.

Chapter 2: Literature Review

2.1 The Design Storm

The general characteristics of storms include volume, duration, frequency, and intensity. Some design problems, like sewer design, only require the total volume of rainfall for a specified duration and frequency. In contrast, for many hydrologic design problems it is necessary to show the variation of the rainfall volume with time (McCuen, 1989). For input of a storm into a design method, this variation might be expressed as a hyetograph as opposed to a total volume for the storm.

Hyetographs are made up of key characteristics that describe the storm. These characteristics are the peak, the time to peak, and the distribution, as well as the volume, duration, and frequency. When developing a design storm for any region, empirical analyses of measured rainfall records are made to determine the most likely arrangement of the ordinates of the hyetograph. For example, a storm event might have an early peak (i.e., front-loaded), a late peak (i.e., rear loaded), or possibly peak in the center of the storm (i.e., center loaded), while others may have more than one peak. The empirical analysis of measured rainfall hyetographs at a location will determine the shape that is most likely to occur and aid in developing the design storm.

2.1.1 The SCS 24-Hour Storm Distributions

One method to determine the design storm is to use the storm distributions prepared by the Soil Conservation Service (SCS). SCS developed four dimensionless rainfall distributions using the Weather Bureau's Rainfall Frequency Atlases (National Weather Service, 1961). The Atlases contain data for areas less than 400 mi^2, for durations up to 24 hours, and for frequencies from 1 to 100 years. Data analysis indicated four major regions, and the resulting rainfall distributions were labeled Type I, IA, II, and III. The geographic locations of where each type of design storm is applicable are shown in Figure 2.1. The corresponding rainfall distributions are shown in Figure 2.2. Note that Nebraska is located in the region named Type II.

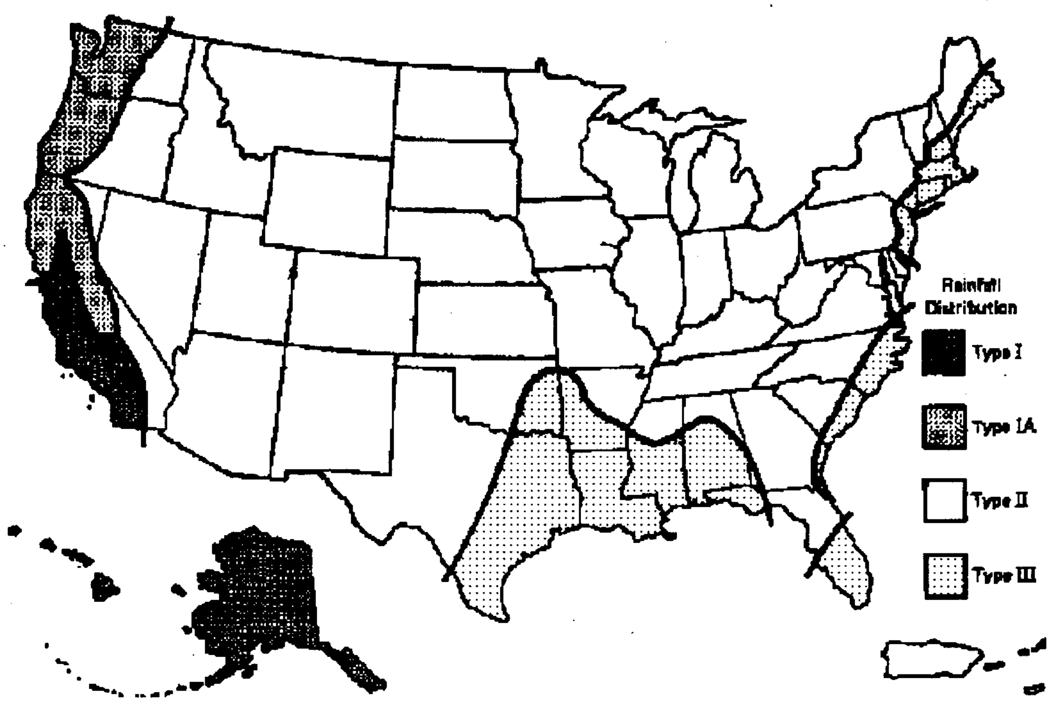


Figure 2.1 SCS Rainfall Distribution Types (Soil Conservation Service, 1986).

These distributions are based on the generalized rainfall volume-duration frequency relationships as documented in technical publications of the Weather Bureau. Rainfall depths for durations of 30 minutes to 24 hours were obtained from the volume-duration-frequency information in these publications and were utilized to derive the storm distributions. Incremental rainfall depths were calculated every 6 minutes.

For example, the maximum 6-minute depth was subtracted from the maximum 12-minute depth, and this 12-minute depth was subtracted from the maximum 18-minute depth, and so on for 24 hours. Then using durations of 6 minutes to 24 hours, the distributions were generated by arranging the incremental depths such that the rainfall depth corresponding to a particular duration and frequency is represented by a continuous sequence of the 6-minute incremental depths.

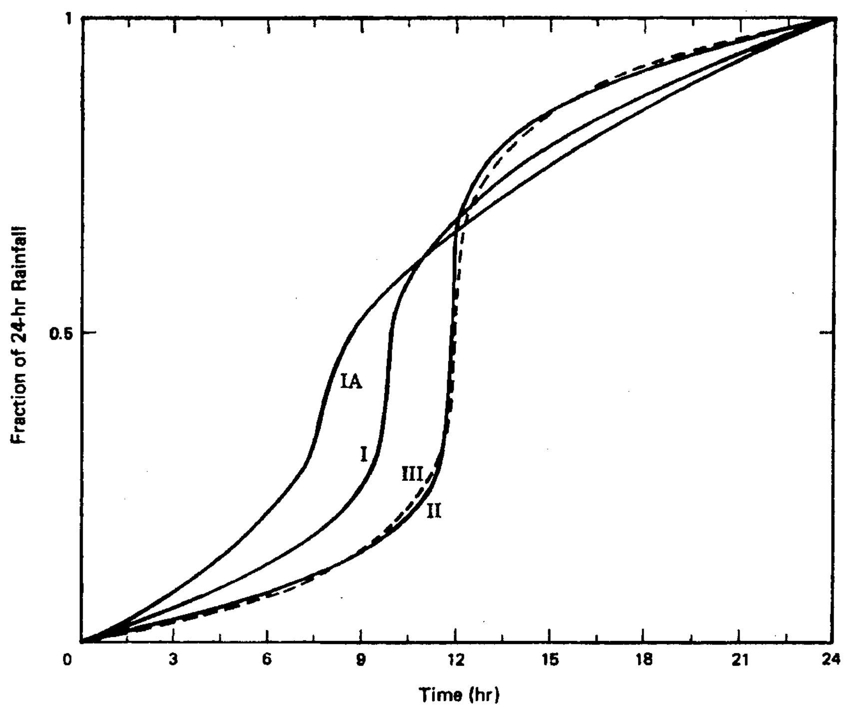


Figure 2.2 SCS 24-hour Rainfall Distributions (Soil Conservation Service, 1986)

From the analysis of measured storm events, the position of the peak was found to be dependent on the rainfall distribution type. For regions with Types I and IA storms, the peak intensity occurs in the first half of the storm. For example, the peak intensity of a Type 1 storm takes place at a storm time of about 9 hours. While for the regions with Types II and III storms, the peak occurs at the center of the storm. Therefore, for Types

II and III storm events, the greatest 6-minute depth is assumed to occur at about the middle of the 24-hour period. The second largest 6-minute incremental depth is assumed to occur in the 6 minutes following the maximum intensity and the third largest in the 6-minute interval preceding the maximum intensity. The fourth largest incremental depth occurs after the second largest, while the fifth largest incremental depth happens before the third. This pattern continues with each incremental rainfall. Thus the smaller increments fall at the beginning and end of the 24-hour storm. The final result is that the maximum 6-minute depth is contained within the maximum 1-hour depth, the maximum 1-hour depth is contained within the maximum 6-hour depth, and so on. Because all of the critical storm depths are contained within the storm distributions, the distributions are appropriate for designs on both small and large watersheds.

After the design storm is determined, a runoff hydrograph must be developed. There are numerous techniques to generate such a unit hydrograph (UH). Among those are 1) using a gamma distribution, 2) Snyder UH, 3) Rational Method, 4) SCS unit hydrograph, 5) Santa Barbara Urban Hydrograph, and 6) US Geological Survey (USGS) Unit Hydrograph.

2.2 Synthesized Unit Hydrographs

Frequently during the design process, it is necessary to have rainfall and runoff data in order to derive a unit hydrograph for the watershed. This information is not always available, especially if the location is not gaged. When this occurs, a synthesized unit hydrograph must be created. Unit hydrographs that are used at ungaged locations are often referred to as regionalized or synthetic unit hydrographs. The regional or synthetic unit hydrographs are a result of the analysis of measured storm data at a gaged location and are then applied to ungaged locations. Such extrapolation can potentially lead to inaccurate designs. Therefore, for design purposes, it is desirable to use a unit hydrograph that was developed from a watershed in a region that has similar watershed characteristics and meteorological conditions. To use the synthetic unit hydrograph on watersheds that are dissimilar in nature, the unit hydrograph should be made dimensionless.

Since a universal procedure for calibrating synthetic unit hydrographs has not been developed, a number of methods have been proposed and used. One method, time-area diagrams, has been used in a number of models as the basis for representing the unit hydrograph. These diagrams are based on the cumulative proportions of watershed that supposedly contribute runoff to the watershed outlet as the storm time increases (McCuen, 1989). Probability density functions, such as the exponential and gamma functions, are another method. And as previously discussed, dimensionless unit hydrographs are also available for use on ungaged watersheds.

The unit hydrograph (UH) is used to develop the design runoff hydrograph at the watershed outlet by convolution. Convolution is based on the theory of linear superpositioning and consists of an iterative process of multiplication, translation with time, and addition. The end result is a means of deriving a design runoff hydrograph from a UH with a base time of D and a series of rainfall excesses that each last for a duration, d. The first interval of rainfall excess of duration d is multiplied by the ordinates of the UH, the UH is then translated into a time length of D, and the next interval of rainfall excess is multiplied by the UH. After the UH has been translated for all intervals of rainfall excess of duration d, the results of the multiplication are summed. This summation is the direct runoff hydrograph (DRH) for the entire storm.

2.2.1 Definition and Assumptions of the Unit Hydrograph

The unit hydrograph is the unit pulse response function of a linear hydrologic system. First proposed by Sherman (1932), the unit hydrograph (originally named unit-graph) of a watershed is defined as a direct runoff hydrograph resulting from 1 in. (1 cm) of excess rainfall generated uniformly over the drainage area at a constant rate for an effective duration. Sherman intended the word "unit" to denote a unit of time, but since that time it has often been interpreted as a unit depth of excess rainfall. Sherman classified runoff into surface runoff and groundwater runoff and defined the unit hydrograph to determine surface runoff only.

The unit hydrograph is a simple linear model that can be used to derive the hydrograph resulting from any amount of excess rainfall. The following basic assumptions are inherent in this model (Chow, et al., 1988):

1. The excess rainfall has a constant intensity within the effective duration.
2. The excess rainfall is uniformly distributed throughout the whole drainage area.
3. The base time of the direct runoff hydrograph (the duration of direct runoff) resulting from an excess rainfall of given duration is constant.
4. The ordinates of all direct runoff hydrographs of a common base time are directly proportional to the total amount of direct runoff represented by each hydrograph.
5. For a given watershed, the hydrograph resulting from a given excess rainfall reflects the unchanging characteristics of the watershed.

The above assumptions can only be satisfied under ideal conditions. However, when the hydrologic data to be used are carefully selected so that they come close to meeting the above assumptions, the results obtained by the unit hydrograph model are generally acceptable for practical purposes (Heerdegen, 1974). Although the model was originally devised for large watersheds, it has been found applicable to small watersheds from less than about 1 acre to 10 mi^2 (about 0.5 hectares to 25 km^2) (Heerdegen, 1974). Some cases do not support the use of the model because one or more of the assumptions are not met. For such reasons, when generating runoff originating from snow or ice the unit hydrograph model is not applicable.

In order to comply with the above assumptions and properly use the unit hydrograph model, certain guidelines have been established. First, the duration of the excess rainfall that is selected for analysis should be of short duration, since these will most likely produce an intense and nearly constant excess rainfall rate. This is advantageous because ultimately it will yield a well-defined single-peaked hydrograph with a short time base. Concerning the second assumption, the unit hydrograph may become inapplicable when the drainage area is too large to be covered by a nearly uniform distribution of rainfall. In such cases, the area has to be divided and each portion of the watershed must be analyzed for the storm covering that portion of the watershed. Generally, the base time of the direct runoff hydrograph is unknown. It is known however, that the base time is dependent on the method of baseflow separation. If the direct runoff is a result of surface runoff only, the base time is usually short. Conversely, if the direct runoff includes both surface and subsurface runoff, the base time is long. Another guideline to developing a unit hydrograph is that the principles of superposition and proportionality are applicable. Actual hydrologic data are not exactly linear. When applying superposition and proportionality to actual data, the resulting hydrograph is only an approximation. This approximation is acceptable in most cases. Finally, the principle of time invariance must be observed. Since the unit hydrograph is considered unique for a given watershed and invariable with respect to time, unit hydrographs are only applicable when channel conditions remain unchanged and watersheds do not have appreciable storage. Situations where this assumption would be violated are when the drainage area contains many reservoirs, or when the flood overflows into the flood plain, thereby producing considerable storage.

2.2.2 Gamma Distribution as a Synthetic UH

When unit hydrographs are derived from measured data, the ordinates must be smoothed to achieve a reasonable shape. To provide for a systematically varying shape, it may be preferable to use a probability function and the moments of the measured data unit hydrograph to fit the parameters of the probability function. Aron and White (1982) provide a method of estimating the parameters of a gamma probability distribution using the peak discharge and time to peak of a computed unit hydrograph. The gamma distribution has the form

$$f(t;a,b) = \frac{t^a e^{-t/b}}{b^{a+1}\Gamma(a+1)} \qquad \text{Equation 2.2.2.1}$$

in which t is the time, a and b are called the shape and scale parameters, respectively; and $\Gamma(a)$ is the gamma function with argument a. The parameters a and b are obtained from the computed peak discharge q_p and the time to peak t_p of the unit hydrograph using the following steps:

1. Compute

$$f_a = \frac{q_p t_p}{A} \qquad \text{Equation 2.2.2.2}$$

where q_p has units of ft^3/sec, t_p has units of hours, and A is the drainage area in acres. The dimensional imbalance of this equation is accounted for in the factors of equation 2.2.2.3.

2. Find the value of a from the following:

$$a = 0.045 + 0.5 f_a + 5.6 f_a^2 + 0.3 f_a^3 \qquad \text{Equation 2.2.2.3}$$

3. Compute the parameter b from

$$b = \frac{t_p}{a} \qquad \text{Equation 2.2.2.4}$$

Given the values of the parameters a and b, the ordinates of the gamma distribution unit hydrograph can be computed from the following:

$$q(f) = q_p f^a e^{a(1-f)} \qquad \text{Equation 2.2.2.5}$$

in which f is any fraction of t_p from 0 to $5t_p$. Figure 2.3 shows the Gamma DUH. This is the same unit hydrograph calculated later in Chapter 3.

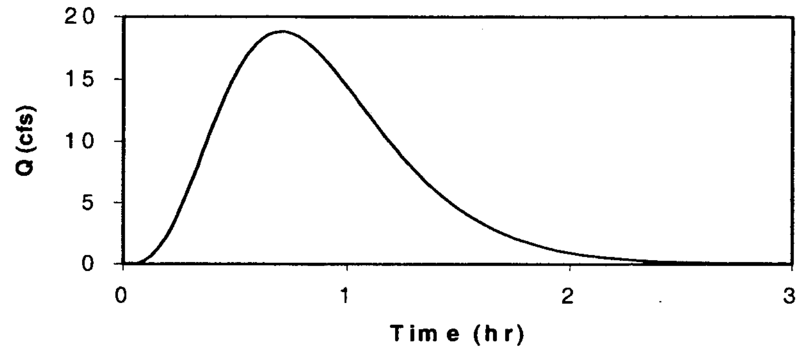


Figure 2.3 Gamma Distribution Unit Hydrograph

2.2.3 Snyder's Synthetic UH

In a study of watersheds located mainly in the Appalachian Highlands of the United States and varying in size from about 10 to 10,000 mi^2 (30 to 30,000 km^2), (Snyder, 1958) found synthetic relationships for some of the characteristics of a standard unit hydrograph (Figure 2.4a). From the relationships, in modified form (Figure 2.4b), five parameters of a unit hydrograph for a given excess rainfall duration may be calculated. The first of these parameters is the peak discharge per unit area of the watershed, q_{pR}. The basin lag, t_{pR} is the time difference between the centroid of the excess rainfall hyetograph and the unit hydrograph peak. Another parameter is the base time, t_b. Finally, the width of the unit hydrograph at 50 and 75 percent of the peak discharge, W in unit time, must be determined. Knowing these parameters, the unit hydrograph can be drawn. The physical representation of the variables on a unit hydrograph is illustrated in Figure 2.4.

Snyder defined a standard unit hydrograph as one whose rainfall duration t_r is related to the basin lag t_p by

$$t_p = 5.5 t_r$$ Equation 2.2.3.1

For a standard unit hydrograph the following relationships hold:

1. The basin lag is

$$t_p = C_1 C_t (L L_c)^{0.3}$$ Equation 2.2.3.2

where t_p is in hours and L is the length of the main stream in miles (kilometers) from the outlet to the upstream divide. The distance in miles (kilometers) from the outlet to a point on the stream nearest the centroid of the watershed area is L_c. $C_1 = 1.0$ (0.75 in SI units), and C_t is a coefficient representing slope of the basin which varies from 1.8 to 2.2 for a distance in miles (1.4 to 1.7 for a distance in kilometers). The variable C_t may be estimated as 0.6/√S, where S is the basin slope (Gupta, 1989).

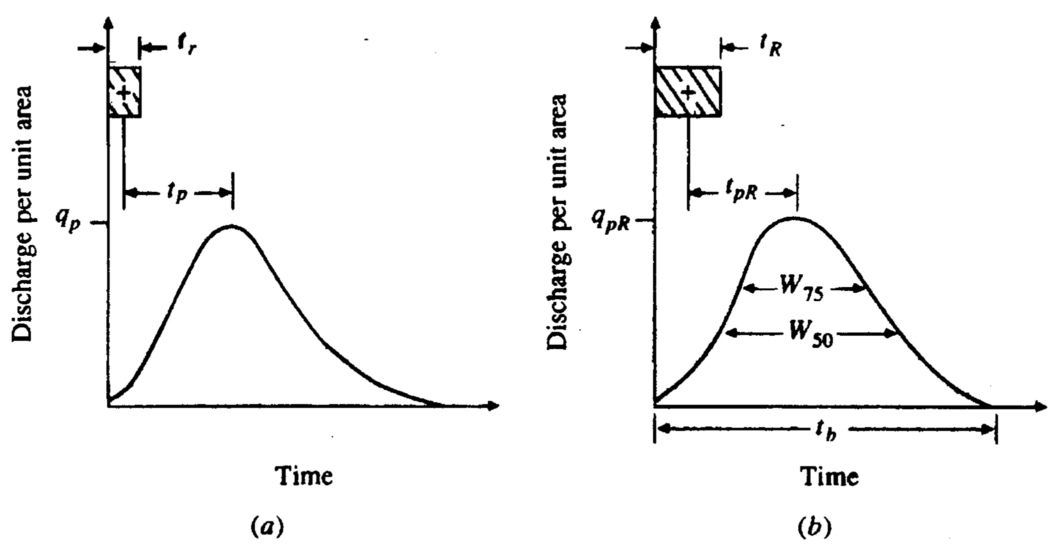


Figure 2.4 Snyder Unit Hydrograph (Snyder, 1958)

2. The peak discharge per unit drainage area in $m^3/s.km^2$ (cfs/mi^2) of the standard unit hydrograph is

$$q_p = \frac{C_2 C_p}{t_p}$$ Equation 2.2.3.3

with,

$$Q_p = q_p A$$

where A is the watershed area in mi^2

C_2 = 640 (2.75 in SI Units) and C_p is a coefficient indicating the storage capacity; varies from 360 to 440 for U.S. units (0.15 to 0.19 S.I. Units).

To compute C_t and C_p for a gaged watershed, the values of L and L_c are measured from the basin map. Since the watershed is gaged, the unit hydrograph of the watershed can be obtained, and the values of the effective duration t_D in hours, basin lag t_{pR} in hours, and peak discharge per unit drainage area, q_{pR} in $m^3/s\text{-}km^2\text{-}cm$ ($cfs/mi^2\text{-}in$) can be determined. If $t_{pR} = 5.5t_D$, then $t_D = t_r$, $q_{pR} = q_p$, and C_t and C_p are computed by Equations 2.2.3.2 and 2.2.3.3. If t_{pR} is quite different from $5.5t_D$, the standard basin lag is

$$t_{pR} = t_p + 0.25(t_r - t_D)$$ Equation 2.2.3.4

and Equations 2.2.3.1 and 2.2.3.4 are solved simultaneously for t_r and t_p. The values of C_t and C_p are then computed from Equations 2.2.3.2 and 2.2.3.3 with $q_{pR} = q_p$ and $t_{pR} = t_p$.

When an ungaged watershed appears to be similar in characteristics to a gaged watershed, the coefficients C_t and C_p for the gaged watershed can be used in the above equations to derive the required synthetic unit hydrograph for the ungaged watershed.

3. The relationship between q_p and the peak discharge per unit drainage area q_{pR} of the required unit hydrograph is

$$q_{pR} = q_p \frac{t_p}{t_{pR}}$$

or Equation 2.2.3.5

$$Q_{pR} = Q_p \frac{t_p}{t_{pR}}$$

4. The base time T of the unit hydrograph can be determined using:

$$T = 3 + \frac{t_p}{8} \qquad \text{Equation 2.2.3.6}$$

where T is in days.

5. The width, in hours, of a unit hydrograph at a discharge equal to a certain percent of the peak discharge q_{pR} is given by

$$W = C_w \left(\frac{A}{Q_{pR}} \right)^{1.08} \qquad \text{Equation 2.2.3.7}$$

where C_w = 440 (1.22 in SI Units) for the 75-percent width and 770 (2.14 in SI Units) for the 50-percent width. Usually, one-third of this width is distributed before the unit hydrograph peak time and two-thirds after the peak.

A further advancement of Snyder's method has been the regionalization of unit hydrograph parameters. Espey, et al., 1977, developed a set of generalized equations for the construction of 10-minute unit hydrographs using a study of 41 watersheds ranging in size from 0.014 to 15 mi^2, and in impervious percentage from 2 to 100 percent. Of the 41 watersheds, 16 are located in Texas, 9 in North Carolina, 6 in Kentucky, 4 in Indiana, 2 in each Colorado and Mississippi, and 1 in each Tennessee and Pennsylvania. The equations are as follows:

$$T_p = 3.1L^{0.23}S^{-0.25}I^{-0.18}\Phi^{1.57}$$ Equation 2.2.3.8

$$Q_p = 31.62\times10^3 A^{0.96}T_p^{-1.07}$$ Equation 2.2.3.9

$$T_B = 125.89\times10^3 AQ_p^{-0.95}$$ Equation 2.2.3.10

$$W_{50} = 16.22\times10^3 A^{0.93}Q_p^{-0.92}$$ Equation 2.2.3.11

$$W_{75} = 3.24\times10^3 A^{0.79}Q_p^{-0.78}$$ Equation 2.2.3.12

where

$L =$ The total distance (in feet) along the main channel from the point being considered to the upstream watershed boundary.

$S =$ The main channel slope (in feet per foot), defined by H/0.8L, where H is the difference in elevation between A and B. A is the point on the channel bottom at a distance of 0.2L downstream from the upstream watershed boundary. B is a point on the channel bottom at the downstream point being considered.

$I =$ The impervious area within the watershed (in percent) is assumed equal to 5 percent for an undeveloped watershed.

$\Phi =$ The dimensionless watershed conveyance factor, which is a function of percent impervious and roughness.

$A =$ The watershed drainage area in (square miles).

$T_p =$ The time of rise to the peak of the unit hydrograph from the beginning of runoff (in minutes).

$Q_p =$ The peak flow of the unit hydrograph (in cfs/in).

$T_B =$ The time base of the unit hydrograph (in minutes).

$W_{50} =$ The width of the hydrograph at 50 percent of Q_p (in minutes).

$W_{75} =$ The width of the hydrograph at 75 percent of Q_p (in minutes).

2.2.4 Hydrograph Assumptions of the Rational Method

The rational method is commonly used for estimating peak discharges. The development of the rational method was based on several assumptions. The first of these assumptions is that the rainfall intensity, *i*, is constant over the storm duration. Secondly, the rainfall is uniformly distributed over the watershed. Another assumption is that the maximum rate of runoff will occur when runoff is being contributed to the outlet from the entire watershed. In addition, the peak rate of runoff equals some fraction of the rainfall intensity. Finally, the watershed system is linear. The hydrograph that originates from the rational method encompasses these assumptions.

The rational formula is as follows (Chow, et al., 1988):

$$q_p = CiA \qquad \text{Equation 2.2.4.1}$$

where q_p is peak discharge and C is the runoff (rational) coefficient. A is the drainage area and i is the rainfall intensity.

One of several possible methodologies can be employed to formulate the hydrograph. The easiest solution would be as follows:

1. Estimate the peak discharge of the runoff hydrograph using Equation 2.2.4.1.
2. Assume that the runoff hydrograph is an isosceles triangle with a time to peak equal to t_c and a time base of $2t_c$.

This method would produce a hydrograph with 50 percent of the volume under the rising limb of the hydrograph and a total volume of $CiAt_c$. The assumption that the shape of the hydrograph would be that of an isosceles triangle would probably be reasonable for most design problems on small urban watersheds. This method is advantageous because it is easy to develop and is usually sufficient for design purposes in small, highly urbanized watersheds. It is important to note that the generation of the unit hydrograph using the rational method is subject to the same assumptions and limitations that govern the rational equation.

A unit hydrograph is inherent in the rational method. Since the volume of the unit hydrograph must equal 1 in., the ordinates of the UH can be determined by multiplying each ordinate of the direct runoff hydrograph of the rational method by the conversion factor K:

$$K = \frac{1}{Cit_c} \qquad \text{Equation 2.2.4.2}$$

where i and t_c are in in./hr and hours, respectively. Thus the peak discharge of the unit hydrograph will be Kq_p.

Since there are methods available that represent the response of a watershed better than an isosceles triangle shaped hydrograph, the rational method is not a preferred method for a final design. In addition, the run-off volume will not generally be equal to $CiAt_c$, so detention pond volumes will also be incorrect.

The following section describes and discusses a method that is commonly used in Nebraska, called the modified rational method.

2.2.4.1 The Modified Rational Method

This method is limited to small areas like rooftops, parking lots, or other stream areas with tributary basins less than 20 acres (Omaha Stormwater Manual, 1988). There are three major steps to design a detention pond using this method: constructing a set of inflow hydrographs, constructing the outflow hydrograph, and computing the storage volume.

The set of inflow-hydrographs is developed using the following steps:

- A set of design storm durations, t, has to be selected, starting with the time of concentration, *tp*. For example, if *tp* is 6 minutes, the set of storm durations can be: 6, 10, 15, 20, 30, 40, 50, and 60 minutes.
- These storm durations are used to obtain the rainfall intensities, *i*, from the IDF curves.
- Using the rational formula, $Q = CiA$, the peak inflow, Q, for each storm duration can be obtained.
- The first inflow hydrograph is a triangle with *2*tp* as the base and Q, which corresponds to *tp*, as the peak flow.
- Each of the other hydrographs will be a trapezoid with the storm duration as the base and the corresponding Q as the peak flow. The peak flow for these hydrographs starts at t_p and continues for a time equal to $t - 2t_p$. See figure 2.5.

For the outflow hydrograph, an outflow discharge peak value has to be established. The manual did not describe a specific method of how this value can be established. The outflow hydrograph starts from zero and goes up to the intersection of the falling limb of the triangular inflow hydrograph and the peak outflow discharge. The discharge remains constant after that point.

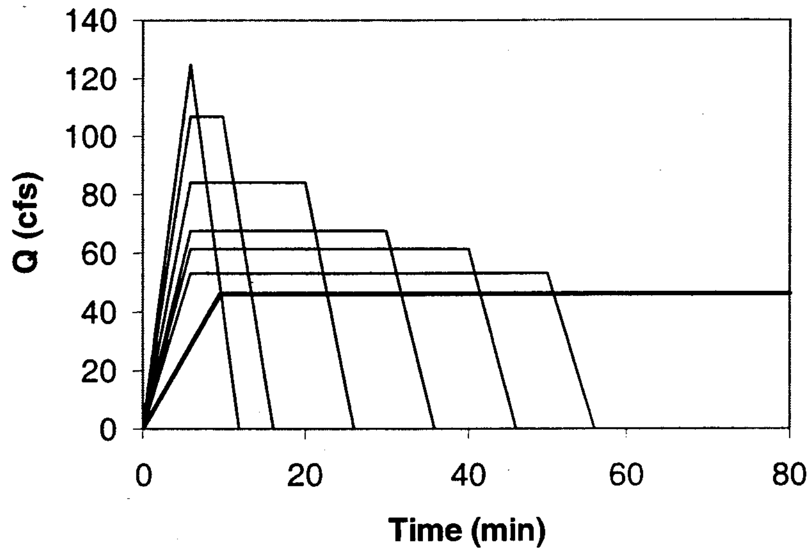


Figure 2.5 The Modified Rational Method Hydrographs

The storage volume is computed by subtracting the area under the outflow hydrograph that is bounded by the inflow hydrograph from the total area under the inflow hydrograph. By carrying this calculation for each inflow hydrograph, a set of storage volumes is obtained. The required storage volume is the highest amount among that set.

Contrasting the routing procedures of working with a single design storm for a specific watershed, this method varies the storm duration and generates a set of inflow hydrographs. As it will be demonstrated in chapter 3, longer storm durations will require more storage volumes. It is clear that this will not be true for this method because the inflow and outflow hydrographs are not related.

All the hydrographs generated in the manual are for durations less one hour. It is not uncommon to have a rainfall of 6 hours duration in Omaha. If the rainfall duration for a watershed with 8-minute time of concentration is 6 hours, the time base will be 6 hours and the peak flow will be constant for 5 hours and 44 minutes long. Also, the assumption of having a constant rainfall intensity will be violated by having long storm durations, such as this 6-hour storm.

The procedure to calculate the required storage is missing two very important points. First, the method is not solving simultaneously for both the storage volume and outlet size. This means a 2-ft or 4-ft pipe outlet can be used for the same design. Second, the outflow hydrograph is the same for all the inflow hydrographs. By applying the theory of continuity equation ($\Delta S/\Delta t = \Delta I - \Delta Q$), the outflow hydrograph is related to the type of the outlet, inflow hydrograph, and the stage-storage curves. Since the design solution is independent of the stage-storage curve and the type of the outlet, one design can virtually fit anywhere in Omaha, if the area and coefficient C are the same. This method cannot be used as a final design procedure.

2.2.5 The Soil Conservation Service (SCS) Unit Hydrograph

A method developed by SCS for constructing synthetic unit hydrographs is based on a dimensionless unit hydrograph (Soil Conservation Services, 1972). This dimensionless unit hydrograph is a result of an analysis of a large number of natural unit hydrographs from a wide range in size and geographic locations. The method requires only determination of the time to peak and the peak discharge as follows:

$$t_p = D/2 + t_l \qquad \text{Equation 2.2.5.1}$$

where t_p = the time to peak (hour).

D = duration of rainfall (hour).

t_l = the lag time from the centroid of the rainfall to the peak discharge (hour).

The peak flow, Q_p for the hydrograph is calculated by approximating the unit hydrograph as a triangular shape with base of $(8/3)t_p$ and unit area as follows:

$$Q_p = (484*A)/t_p \qquad \text{Equation 2.2.5.2}$$

The time base of $(8/3)t_p$ is based on empirical values for typical rural experimental watersheds. This variable should be reduced when steep slopes are encountered, causing an expected increase in the peak flow, and increased for flat conditions to cause the peak flow to decrease. The resulting coefficient in Equation 2.2.5.2 ranges from nearly 600 for steep mountainous surroundings to 300 for flat swampy conditions (Soil Conservation Services, 1972).

The relationship between t_l and the size of the watershed can be used to estimate the lag time. For example, typical equations from two different geographic regions are:

$t_l = 1.44A^{0.6}$ Texas

$t_l = 0.54A^{0.6}$ Ohio

The average lag is $0.6t_c$, where t_c is the time of concentration. SCS defines the time of concentration to be either the time for runoff to travel from the furthermost point in the watershed to the watershed outlet (called the upland method) or the time from the end of excess rain to the inflection of the unit hydrograph. For the first case:

$$t_c = 1.7t_p - D \qquad \text{Equation 2.2.5.3}$$

The dimensionless unit hydrograph (DUH) has a point of inflection at approximately $1.7t_p$. If the lag time of $0.6t_c$ is assumed, Equations 2.2.5.1 and 2.2.5.3 give:

$D = 0.2t_p$ Equation 2.2.5.4

$D = 1.33t_c$ Equation 2.2.5.5

A small variation in D is permissible, but it should not exceed $0.25t_p$ or $0.17t_c$. By defining a value of t_l, a synthetic UH of chosen duration, D, is obtainable. The ordinates of the DUH are shown in Figure 2.5

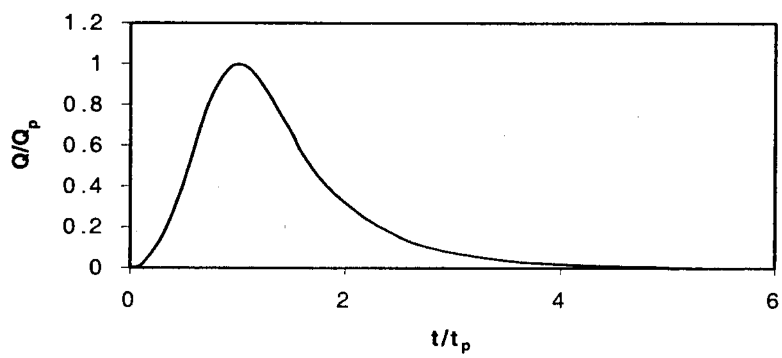


Figure 2.6 SCS Dimensionless Unit Hydrograph

2.2.6 The Santa Barbara Urban Runoff Hydrographs

The Santa Barbara Urban Hydrograph (SBUH) method is similar to the SCS method (Stubchaer, 1975). This method utilizes the curve number (CN) as well as the SCS equations for computing soil absorption and precipitation excess. As explained previously, the SCS method converts incremental runoff depths for a given basin and design storm hydrographs of equal time base according to the basin time of concentration and adds them to form the final runoff hydrograph. In contrast, the Santa Barbara

Hydrograph method converts the incremental runoff depths into instantaneous hydrographs, which are then routed through an imaginary reservoir with a time delay equal to the basin time of concentration. The advantage to the SBUH approach is that it directly computes a runoff hydrograph without going through an intermediate process, like the unit hydrograph that the SCS method requires.

The SBUH consists of two steps to create the final runoff hydrograph:

1. Compute the instantaneous hydrograph.

   The instantaneous hydrograph, $I(t)$ at each time interval, dt, is computed as follows:

   $$I(t) = 60.5 \times R(t) \times A/dt$$

   where: $R(t)$ = runoff depth at time interval dt

   A = basin area in acres

   dt = time interval in minutes

2. Compute the runoff hydrograph.

   $$Q(t+1) = Q(t) + w[I(t) + I(t+1) - 2Q(t)]$$

   where: $Q(t)$ = runoff hydrograph ordinate at time t

   $I(t)$ = instantaneous hydrograph from Step 1

   and $w = dt/(2Tc + dt)$

   where: Tc = basin time of concentration

   dt = time interval in minutes

It is important to note that this method was developed for use in the northwestern part of the United States. The SBUH method may not be applicable in regions with different topography and weather conditions, unless it is calibrated by actual data from that region.

2.2.7 The USGS Unit Hydrograph

Under the direction of the USGS (Stricker and Sauer, 1982) developed the dimensionless unit hydrograph (DUH) for urban basins using theoretical techniques and data collected from 80 basins having a drainage area less than 20 mi^2. Using actual stream data for both urban and rural streams in Georgia (Inman, 1987) confirmed the validity of the theoretical DUH fashioned by Stricker and Sauer. Other researchers have since developed similar DUHs for numerous other states (Sauer, 1989). Excluding some slow-runoff areas with relatively flat topography, the USGS DUH seems to apply with an acceptable degree of accuracy. The USGS DUH has three essential components: the peak discharge for which a hydrograph is desired, the basin lag time, and the DUH ordinates. The procedure employed by Stricker and Sauer to construct their DUH is as follows:

1. Compute an UH and lag time for three to five storms for each of the 80 gauging stations. All unit hydrographs should be for the same time interval (duration) at each station. The lag time is computed to be the time at the centroid of the UH minus one-half the time of the computation interval (duration).
2. Eliminate the hydrographs with inconsistent shapes and compute additional unit hydrographs if needed.
3. Compute an average UH for each station by aligning the peaks and averaging each ordinate of discharge for the final selection of unit hydrographs. The correct timing of the average UH is obtained by averaging the time of the center of mass

of the individual unit hydrographs and plotting the average center of mass at this average time. The time of the center of mass of the discharge hydrograph is calculated by adding one-half the UH computation interval (duration) to the lag time of the hydrograph.

4. Transform the average unit hydrographs computed on step 3 to hydrographs having one-fourth, one -third, one-half, and three-fourth lag time. These durations must be to the nearest multiple of the original duration (computation interval). These transformed unit hydrographs will have durations of two times, three times, four times, and six times the duration of the original UH. The transformation of a short-duration UH to a long-duration UH (for instance, a 5-minute to a 20-minute duration) can be accomplished using the following equations:

$$TUHD(t) = 1/2(TUH(t)+TUH(t-1)) \quad \text{for } D/\Delta t=2$$

$$TUHD(t) = 1/3(TUH(t)+TUH(t-1)+TUH(t-2)) \quad \text{for } D/\Delta t=3$$

$$TUHD(t) = 1/4(TUH(t)+TUH(t-1)+TUH(t-2)+TUH(t-3)) \quad \text{for } D/\Delta t=4$$

$$TUHD(t) = 1/n(TUH(t)+TUH(t-1) \ldots\ldots TUH(t-n+1)) \quad \text{for } D/\Delta t= n$$

where,

Δt = computation interval (the original UH has a duration equal to Δt)

D = design duration of the UH (this must be a multiple of Δt)

TUHD(t) = ordinate of the desired UH at time t

TUH(t), TUH(t-1), etc. = ordinate of the original UH at time t, t-1, t-2, etc.

5. Reduce the one-fourth, one-third, one-half, and three-fourth lag-time hydrographs to dimensionless terms by dividing the time by the lag time and the discharge by peak discharge.

6. For hydrologic regions 1, 2, and 3 as defined by Price and the Atlanta urban area as reported by Inman, 1987, compute an average dimensionless hydrograph by using the dimensionless hydrographs at the stations within that area or region. The hydrographs were computed by aligning the peaks and averaging each ordinate of the discharge ratio (Q/Qp).

The above steps were performed on stations provided by the USGS that had previously had hydrographs created from earlier studies. A total of 355 unit hydrographs from 80 stations were used to develop the one-fourth, one-third, one-half, and three-fourth lag-time dimensionless hydrographs. Next, a statistical analysis was executed to select the best-fitting design duration. This was done by comparing the width of the hydrographs as estimated (or computed) from the one-fourth, one-third, one-half, and three-fourth lag-time dimensionless hydrographs from each region or area with the observed hydrograph widths from their respective region or area. The results of that analysis concluded that the one-half lag-time was the best fit of width at 50 percent and 75 percent of peak flow.

Finally, another statistical analysis was performed to test the accuracy of the dimensionless hydrograph technique. For this analysis, the simulated hydrograph widths at 50 and 75 percent of peak flow, derived from simulated hydrographs using the statewide one-half lag-time dimensionless hydrograph, were compared with the 355 observed hydrographs. The standard error for estimating the width at 50 percent of peak flow and 75 percent of the peak flow was ±31.8 and ±35.9 percent, respectively. This standard error is based on mean square difference between observed and simulated widths. Based on verification and bias testing, the USGS dimensionless hydrograph can be used for flood-hydrograph simulation for ungaged basins up to 500 mi^2. For the ordinates of the Dimensionless Unit Hydrograph, see Table 2.1.

Table 2.1 The Ordinates of the USGS DUH (Inman, 1987)

Time Ratio (t/T_L)	Discharge Ratio (Q/Q_p)	Time Ratio (t/T_L)	Discharge Ratio (Q/Q_p)
0.25	0.12	1.35	0.62
0.30	0.16	1.40	0.56
0.35	0.21	1.45	0.51
0.40	0.26	1.50	0.47
0.45	0.33	1.55	0.43
0.50	0.40	1.60	0.39
0.55	0.49	1.65	0.36
0.60	0.58	1.70	0.33
0.65	0.67	1.75	0.30
0.70	0.76	1.80	0.28
0.75	0.84	1.85	0.26
0.80	0.90	1.90	0.24
0.85	0.95	1.95	0.22
0.90	0.98	2.00	0.20
0.95	1.00	2.05	0.19
1.00	0.99	2.10	0.17
1.05	0.96	2.15	0.16
1.10	0.92	2.20	0.15
1.15	0.86	2.25	0.14
1.20	0.80	2.30	0.13
1.25	0.74	2.35	0.12
1.30	0.68	2.40	0.11

2.3 Detention Pond Preliminary Design Procedures

In this section, six detention pond sizing methods are introduced for preliminary design. These methods can be used to estimate the required volume of detention storage for a first design trial. The complete design process will require trial and error to obtain the final design. The preliminary sizing methods are different from the final design methods in two ways. First, the preliminary methods only require peak discharge estimates, as opposed to requiring flood hydrographs. Thus, routing hydrographs through the detention basin is not necessary when using these methods. Second, since routing is not required, a stage-storage-discharge relationship is not required.

To design the basin for stormwater control, the stormwater must be routed through the basin. Six models for detention pond preliminary sizing are as follows:

1. Baker's formula (1979):

   $$S_f/V_f = 1-(Q_p/I_p)$$

   Inflow and outflow hydrographs are assumed to be triangular.

   Sf = required flood storage.

   Vf = the flood volume.

   Sf/Vf = flood-storage ratio.

   Qp = peak outflow.

   Ip = peak inflow.

   Qp/Ip = peak-discharge ratio.

2. Abt and Grigg formula (1978):

   $$S_f/V_f = (1-(Q_p/I_p))^2$$

   This is based on a triangular inflow hydrograph and a trapezoidal outflow hydrograph, with the rising limbs coincident up to the maximum release rate.

3. Wycoft and Singh formula (1976):

   $$S_f/V_f = 1.291*(1-(Q_p/I_p))^{0.753}/(t_b/T)^{0.411}$$

On the inflow hydrograph, t_b and T are the time base and time to peak. Located on the falling limb is t_b for which $I/I_p = 0.05$.

This method is based on fitting a regression equation to the results of 50 numerical simulations. For the analysis, 10 hydrographs with different characteristics were routed through 5 hypothetical reservoirs with outlets of different sizes. Wycoft and Singh, 1976, did not investigate the effect of the type of outlet on the storage requirement.

4. TR 55 (U.S. Soil Conservation Services, 1986):

   This is the most widely used method for sizing detention reservoirs. It represents two curves that relate the flood-storage and peak-discharge ratios for different geographic regions. These curves were fitted to the results of flood routing simulations with inflow hydrographs generated for hypothetical storms by means of the TR-20 flood hydrograph model of the SCS.

5. Akan, 1989, graphical procedure:

   Akan's procedure yields the approximate size of the outlet. The required information to use this procedure are the peak inflow, the time to peak on the inflow hydrograph, the peak outflow, the stage storage relationship expressed as a simple power function, and the type of the outlet (orifice or rectangular weir). Once the outlet has been sized, the required flood storage can be determined from the stage-storage and stage-discharge functions. Akan's graphical relationships are based on the SCS dimensionless unit hydrograph.

6. Bruce M. McEnroe formula, 1992:

   This method is based on a gamma function hydrograph, where the stage-storage relationship is assumed to be a simple power function. The procedure is dependent on the shape of the outlet.

   For an orifice or pipe outlet:

   $$Sf/Vf = 0.97 - 1.42*(Qp/Ip) + 0.82*(Qp/Ip)^2 - 0.34*(Qp/Ip)^3$$

   For a spillway, weir, or perforated riser outlet:

   $$Sf/Vf = .98 - 1.17*(Qp/Ip) + 0.77*(Qp/Ip)^2 - 0.46*(Qp/Ip)^3$$

2.4 Design Procedures from Different States

As mentioned before there are many methods available to design a detention pond. Here are some of the methods that are used in DOTs as listed in their design manuals:

- The Florida Department of Transportation (DOT), 1997 recommends using the SCS Method or the Rational Method.
- The Washington DOT, 1997, is using the Santa Barbara Urban Hydrograph Method.
- The Oklahoma DOT, 1992, recommends the SCS Method.
- The Missouri DOT, 1998, recommends the use of the SCS TR-55 Method for small watersheds and the USGS Method for large watersheds (> 200 acres).
- The Montana DOT Manual, 1995, named three unit hydrographs:
  - The Wyoming Unit Hydrograph.
  - The Montana Unit Hydrograph.
  - The SCS Unit Hydrograph.

The FHWA Urban Drainage Manual, 1996, cited the Snyder Unit Hydrograph, the SCS Unit Hydrograph, and the USGS Urban Hydrograph.

2.5 The Routing Equation

The final step in simultaneously solving for the storage capacity and the outlet facility characteristics is to route the runoff through the reservoir. Reservoir routing is accomplished by repeatedly solving the continuity equation:

$$I - O = \Delta S / t \qquad \text{Equation 2.5.1}$$

where I and O are the average inflow and outflow rates for the time period, t. The change in storage is ΔS during the time period. A more convenient form of the above equations is yielded by assuming that the average flow rates for the time period, t, is equal to the average of the flows at the beginning and end of the time period:

$$\frac{I_1 + I_2}{2} - \frac{O_1 + O_2}{2} = \frac{S_2 - S_1}{t} \qquad \text{Equation 2.5.2}$$

For this assumption to be true, the hydrograph must effectively be a straight line between I_1 and I_2 and a routing period, t, must be selected which does not violate this concept. A routing period between one-fourth and one-half of the time of concentration will normally be acceptable, but the shape of the inflow hydrograph must be considered when selecting the routing period.

When using Equation 2.5.2, I_1, I_2, O_1 and S_1 are either known values or are assumed to be zero. O_2 and S_2 must he determined. The equation can be rearranged to yield:

$$I_1 + I_2 + \left(\frac{2S_1}{t} - O_1\right) = \left(\frac{2S_2}{t} + O_2\right) \qquad \text{Equation 2.5.3}$$

From the stage-storage and stage-discharge curves for the proposed detention facility, a $\frac{2S}{t} + O$ versus O curve can be constructed. After a value for $\frac{2S}{t} + O$ is computed from Equation 2.5.3, the value of O_2 can be obtained directly from this curve. The computation is then repeated for succeeding routing periods.

The techniques presented to obtain the volume of runoff and the volume of detention storage provide a direct solution to reservoir routing. The design of a detention basin, however, requires an iterative trial and error approach. A detention basin must be sized and an outlet structure selected before the stage-discharge, stage-storage, and $\frac{2S}{t} + O$ vs. O curves can be prepared. The design storm must then be routed through the tentative detention facility to determine its ability to produce an acceptable discharge rate. If the peak discharge rate is too high or the detention volume is excessive, the design must be modified and the routing calculations repeated for the new design.

Chapter 3: Evaluation of Existing Pond

3.1 General

In this chapter, the main steps to design a detention pond will be discussed with the aid of an example. The example is a detention pond that already exists and is located in Omaha, between the I-80 and 102nd Street. The Nebraska Department of Roads (NDOR) performed the design of the existing pond.

The three major steps to design a detention pond are:

- Obtain the design storm and rainfall excess
- Obtain a hydrograph
- Route the hydrograph through the pond outlet

The preliminary sizing methods, however, do not follow these steps.

3.2 Basic Information:

In order to design a detention pond, the following information is required:

Stage-Storage Relationship: The stage-storage relationship is a curve that displays the storage capacity of the pond at a corresponding elevation. Figure 3.1 is the stage-storage curve for the Omaha site.

Drainage Basin Information: This is available from the topographic map of the site. The watershed shall be delineated to obtain this information. The area of the specific basin is 17.4 acres (with 2.65 acres as impervious and 14.72 acres as pervious area). The main channel length, L, is 1300 ft. L_c, the length along the main channel measured from the outlet to a point on the main channel that would be the centroid of the basin, is 500 ft. The slope of the watershed, S, is 0.5 percent.

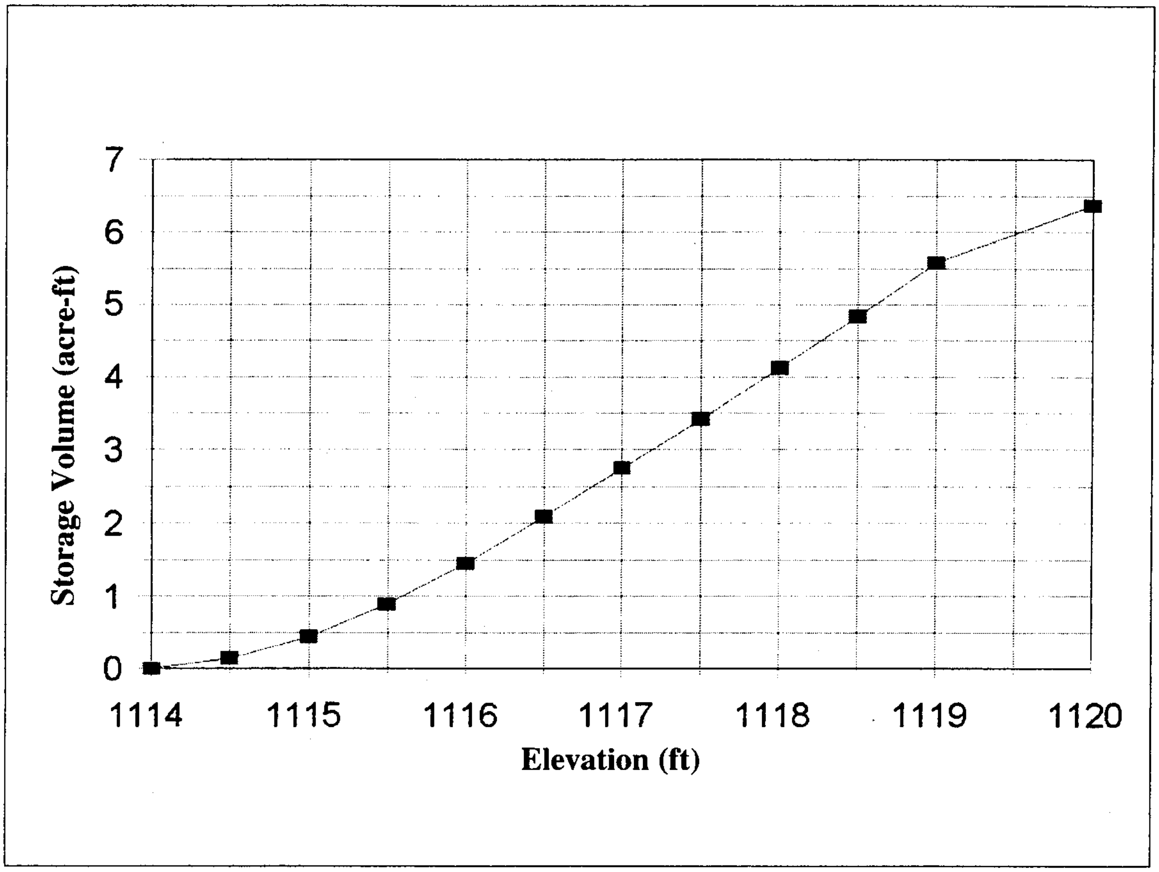


Figure 3.1 Stage Storage Curve

The Rating Curve: The outlet of this particular pond is a 3-ft culvert. In this example, the rating curve, otherwise known as the performance curve, was calculated using the HY-8 computer program (See Appendix II). Figure 3.2 shows the performance curve for the specific pond.

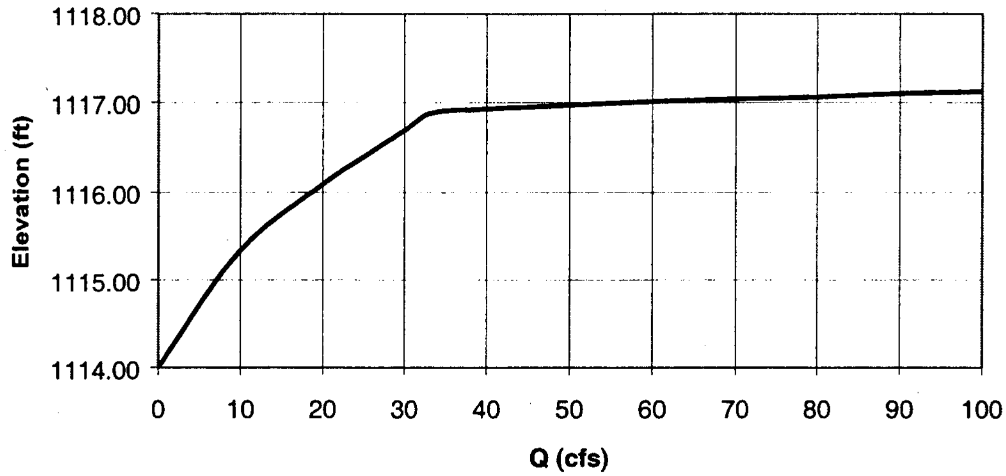


Figure 3.2 The Outlet Performance Curve

3.3 The Design Storm and Rainfall Excess

The design storms for different durations and return periods, specific to this site, are obtained from the Rainfall Frequency Atlas of the National Weather Service. Table 3.1 shows the two different storms that will be analyzed in this example. These design storms will then be used to create the IDF curves as seen in Fig 3.3.

Table 3.1 The Design Storms

Duration	Frequency			
	50-Year		100-Year	
(hr)	Depth (in)	Intensity (in/hr)	Depth (in)	Intensity (in/hr)
0.5	2.54	5.080	2.83	5.660
1.0	3.23	3.230	3.64	3.640
2.0	3.75	1.875	4.11	2.055
3.0	3.95	1.317	4.50	1.500
6.0	4.36	0.727	5.19	0.865
12.0	5.39	0.449	6.00	0.500
24.0	6.00	0.250	6.73	0.280

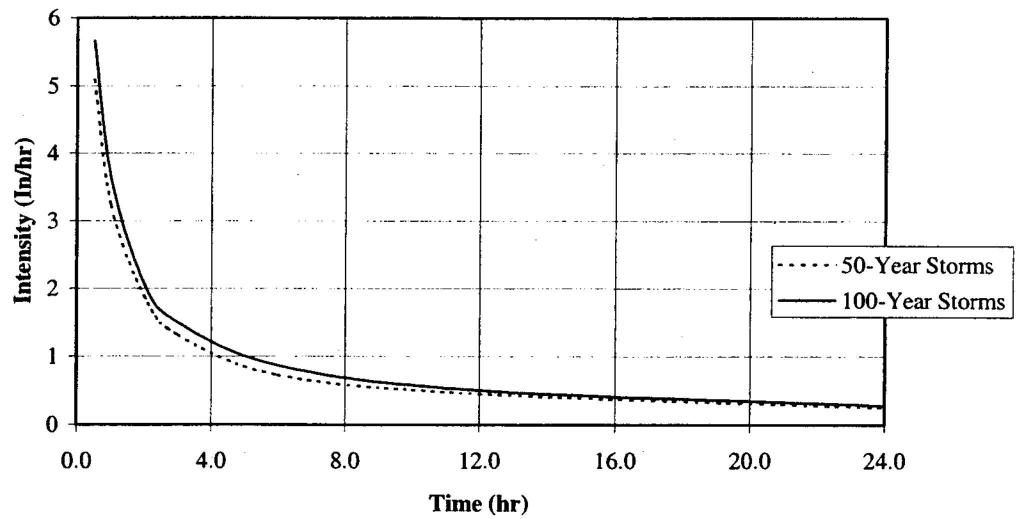


Figure 3.3 Intensity-Duration-Frequency Curves

The rainfall excess is calculated using the SCS equation (Gupta, 1989):

$$Q = (P-0.2S)^2/(P+0.8S) \qquad \text{Equation 3.3.1}$$

and $$S = (1000/CN) - 10 \qquad \text{Equation 3.3.2}$$

where, Q = accumulated runoff over the drainage area (in.)

P = accumulated rainfall depth (in.)

S = potential maximum retention of water by the soil (in.)

The Curve Number (CN) has been estimated as 82 (98 for the impervious and 79 for the pervious area) for the post-development, and 74 for the pre-development. Therefore, for post-development conditions and a 50-year storm of 24-hour rainfall excess,

$$S = (1000/82)-10 = 2.2$$

$$Q = (6-0.2(2.2))^2/(6+0.8(2.2)) = 4 \text{ inches}$$

Finally, the designer will need to determine the ratio of the pre-development to post-development maximum discharge. This is achieved by constructing the pre-development and post-development runoff hydrographs. For simplicity, in this example, the allowable ratio of the pre- to post-development maximum discharge is assumed to be 60 percent.

3.4 Preliminary Sizing Procedures

These methods will be illustrated by using the 50-year recurrence interval, 24-hour duration of rainfall excess of 4 in. The total volume will be 4 in. x 17.4 acre, about 252,648 ft^3. The resulting capacity of the detention pond, S_f calculated by each sizing method is shown below, also see Appendix III.

Preliminary Sizing Method	**Storage Volume, S_f (acre-ft)**
Baker Formula	2.3
Abt and Grigg formula	0.9
TR 55 graph	1.4
Bruce M. McEnroe formula	2.0

The storage volume estimated using the TR 55 method is read from a graph that relates the flood-storage and peak-discharge ratios for different geographic regions. The detention pond under investigation is in a Type II storm distribution region.

The Wycoft and Singh formula and the Akan graphical procedure were discarded as preliminary sizing methods. In order to use the Wycoft and Singh method, an inflow hydrograph would need to be obtained in order to calculate a time base and time to peak. Further work could be done to create an inflow hydrograph, but for purposes of preliminary sizing, the effort is not justified. In the case of the Akan method, the stage-discharge relationship, used to calculate the storage capacity, is not applicable to the culvert type of orifice at the detention pond under investigation.

3.5 The Unit Hydrographs (UH)

For the purposes of this example, only four of the six methods for deriving a unit hydrograph will be employed. Those four methods are the SCS UH, USGS UH, Snyder UH and the Gamma Distribution UH, see Appendix V. The Rational Method is a method that was developed to determine a maximum peak discharge and not necessarily reflect the change in runoff with time. Because of that, the shape of the resulting hydrograph is triangular and is not the most accurate representation of the system. As discussed previously, the Santa Barbara Urban Hydrograph was developed for use in a geographic region much different than that of Nebraska. Since there is not data available to calibrate the region under investigation to this method, it will not be used to generate a unit hydrograph.

To compute a unit hydrograph using the USGS method, the peak runoff and lag time were used as calculated by the SCS method. Likewise, for the Gamma Distribution, there are no procedures allocated to determine the peak discharge or time to peak. Again, the values for peak runoff and time to peak were generated using the SCS method. Figure 3.4 shows the resulting unit hydrographs.

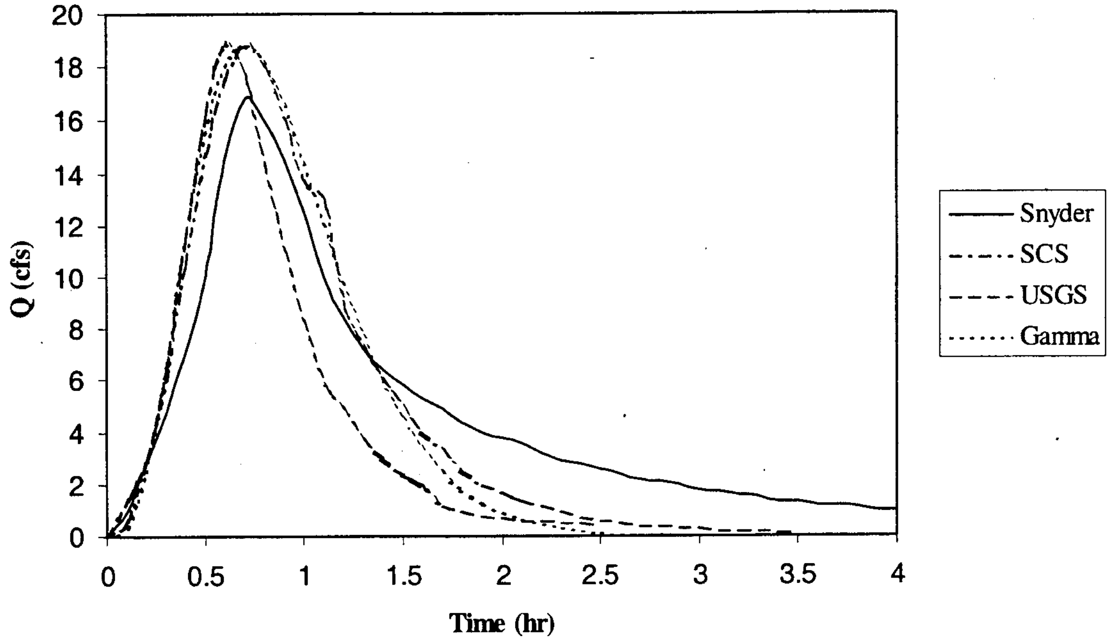


Figure 3.4 The Unit Hydrographs

3.6 Runoff Hydrographs

For each of the four methods, for which a unit hydrograph was created, the process of convolution is employed to develop a series of runoff hydrographs. The ordinates of the unit hydrograph are multiplied by the successive values of the incremental runoff; each lagged by a time step. The total of the units results in the direct runoff hydrograph. This procedure is used for each of the unit hydrograph methods. From each method, 12 runoff hydrographs will be generated, using 6 rainfall durations and 2 return periods for a total of 48 runoff hydrographs (See Appendix VI).

Tables 3.2a and 3.2b show the peak runoff values and time to peak for each runoff hydrograph. Although all the methods are similar in their calculations, the Snyder method results in the lowest estimation of the peak runoff. One explanation for the lower values is the possible underestimation of the parameters C_t and C_p. Calibrating these two parameters is subject to the availability of data for the watershed, or a watershed with similar characteristics. In the case of the Gamma hydrographs, it is important to note that this method does not have a specific procedure to calculate the peak runoff and time to peak. Since some values calculated using the SCS method were used in the Gamma analysis, it is expected that the Gamma hydrograph would result in similar hydrographs. This is also the situation for the hydrographs obtained using the USGS method.

Table 3.2a Peak Values from 50-year Runoff Hydrographs

Duration	SCS		USGS		Snyder		Gamma	
of Rainfall (hr)	Q_p (cfs)	t_p (hr)	Q_p (cfs)	t_p (hr)	Q_p (cfs)	t_p (hr)	Q_p (cfs)	t_p (hr)
1	26.0	1.1	24.0	1.0	21.5	1.1	26.6	1.1
2	29.8	1.2	27.6	1.0	25.6	1.2	30.5	1.2
3	30.8	1.2	28.5	1.1	26.7	1.3	31.4	1.3
6	34.6	1.6	31.8	1.5	29.8	1.7	35.2	1.6
12	38.1	3.1	34.8	3.0	33.1	3.1	38.8	3.1
24	41.1	6.4	37.3	6.3	36.0	6.4	41.8	6.4

Table 3.2b Peak Values from 100-year Runoff Hydrographs

Duration	SCS		USGS		Snyder		Gamma	
of Rainfall (hr)	Q_p (cfs)	t_p (hr)	Q_p (cfs)	t_p (hr)	Q_p (cfs)	t_p (hr)	Q_p (cfs)	t_p (hr)
1	31.1	1.0	29.1	0.9	25.6	1.1	31.7	1.0
2	35.1	1.1	32.7	1.0	30.0	1.2	35.8	1.1
3	37.8	1.2	35.1	1.1	32.5	1.3	38.5	1.2
6	41.7	1.6	38.3	1.5	36.1	1.6	42.4	1.6
12	45.1	3.1	41.2	3.0	39.3	3.1	45.8	3.1
24	48.0	6.4	43.7	6.3	42.1	6.4	48.7	6.4

The last step to determining the detention pond storage capacity is to route the hydrograph through the detention pond. Although all hydrographs can be routed through the detention pond, for the purposes of this design, only the SCS hydrographs will be routed. As stated before, the other three methods are dependent on further calibration or calculation of variables. The SCS method provides a procedure to calculate all of the parameters needed and has proven itself to be an easy to follow, reliable method.

3.7 The Routing Equation

The SCS unit hydrograph method has been selected, according to the results and the discussion in the previous section, to be routed through the detention pond as described in Section 2.4.

The first step is to establish a relationship between Q and (2S/dt + Q) using the stage-storage curve and the performance curve. This relationship is shown in Figure 3.5 below.

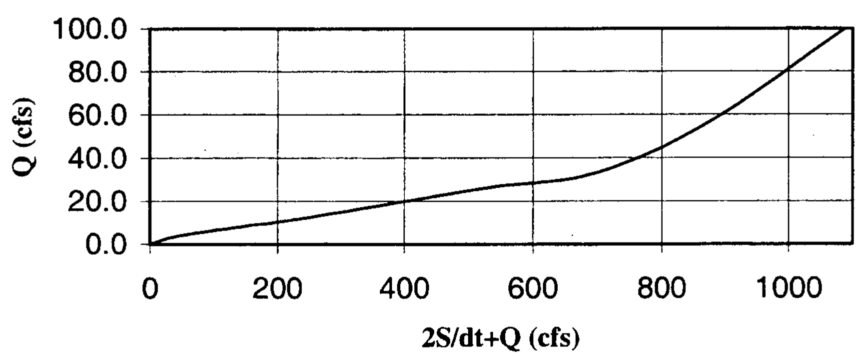


Fig. 3.5 (2S/dt+Q) Vs. Q

The inflow and outflow hydrographs are created simultaneously from the unit hydrograph. This is done for each storm duration and both the 50-year and the 100-year return period, see Appendices VII and IX. The peak values are of interest and are summarized in Table 3.3a and 3.3b, below

Table 3.3a 50-year Hydrograph Routing Results

Duration (hr)	I_{max} (cfs)	Q_{max} (cfs)	Q_{max}/I_{max} %	Storage (acre-ft)
1	26.0	13.7	53	1.0
2	29.8	16.3	55	1.3
3	30.8	16.8	55	1.3
6	34.6	18.8	54	1.5
12	38.1	20.8	55	1.7
24	41.1	22.6	55	1.8

Table 3.3b 100-year Hydrograph Routing Results

Duration (hr)	I_{max} (cfs)	Q_{max} (cfs)	Q_{max}/I_{max} %	Storage (acre-ft)
1	31.1	16.2	52	1.2
2	35.1	18.9	54	1.5
3	37.8	20.5	54	1.6
6	41.7	22.6	54	1.8
12	45.1	24.6	55	2.0
24	48.0	26.4	55	2.1

The following trends can be observed from the inflow/outflow hydrographs. First, for the longer storm durations, more storage is required. Also, a detention cell that is designed for a 50-year frequency will not hold a 100-year storm of the same duration. At this point it is also important to note that the resulting ratios between the peak inflow and peak outflow ranges from 52 percent to 55 percent, for all durations and frequencies. This range is less than the 60 percent ratio that was required for the maximum pre-development to post-development discharge at the beginning of this analysis. If the range had been larger than 60 percent, it would be necessary to reiterate the process by modifying the outlet and/or increase the amount of available storage. If the range had been significantly below the target ratio, design efficiency would not have been accomplished.

Chapter 4: Conclusions

The volume of detention needed to produce a specific reduction in peak discharge for a design flood depends on the type of outlet structure on the reservoir and the inflow hydrograph. The final design is determined by simultaneously solving for the storage capacity and outlet characteristics.

The design procedure is divided into two stages, the preliminary sizing and the hydrograph routing. The design of the detention pond is a trial and error process, starting with one of the methods discussed in Section 2.3. The size of the detention pond determined from any preliminary sizing method should not be used as the final design.

The modified rational method discussed in Section 2.2.4.1 is not recommended to use as a final design procedure. The steps to calculate the required storage are missing two very important points. First, the method is not solving simultaneously for both the storage volume and outlet size. This means a 2-ft or 4-ft pipe outlet can be used for the same design. Second, the outflow hydrograph is the same for all the inflow hydrographs. By applying the theory of continuity equation ($\Delta S/\Delta t = \Delta I - \Delta Q$), the outflow hydrograph is related to the type of the outlet, inflow hydrograph, and the stage-storage curves. Since the design solution is independent of the stage-storage curve and the type of the outlet, one design can virtually fit anywhere in Omaha, if the area and coefficient C are the same. Refer to section 2.2.4.1 for more details.

Although there are numerous methods available to construct a unit hydrograph, for the purposes of this analysis, the SCS method proved to be the most effective. The SCS method is a consistent, reliable technique whose parameters are well defined within the methodology. There are also numerous methods to estimate a storage capacity for use in preliminary design. Since the SCS method is frequently used to establish a runoff hydrograph, it is strongly suggested to use the TR-55 storage volume estimation method as developed by the U.S. Soil Conservation Services, in order to achieve consistency.

Twelve SCS hydrographs were routed in order to evaluate the effects of using different storm durations with different frequencies. As expected, it is the 24-hour duration that will result in the largest storage volume, see Table 3.3. However, the difference between the 24-hour rainfall and 6-hour rainfall is approximately 15 percent for the 50-year and 100-year return periods, and about 5 percent between the 24-hour and 12-hour for both return periods. There is no apparent benefit to use a 12-hour rainfall instead of a 24-hour rainfall. Furthermore, if a detention pond was designed using a 1-hour storm, the storage capacity would be approximately 75 percent less than what is required for a 24-hour storm of the same frequency. Likewise, if the pond was designed for a 3-hour storm, the storage capacity would be approximately 35 percent less than what would be required for a 24-hour storm of the same frequency. The required storage capacity for a storm with a 100-year frequency ranges from approximately 15 to 20 percent more than the storage required for a storm of the same duration and a 50-year return period. To decide which storm duration and frequency to use in design will ultimately depend on the level of risk that the designer is willing to accept.

It is important to note that the unit hydrograph is not the hydrograph used to route through the detention pond. The unit hydrograph must be utilized with the excess rainfall to develop a runoff hydrograph by convolution. The area under a runoff hydrograph is approximately equal to the volume of the generated rainfall excess.

The detention pond under investigation was monitored to compare its performance with the design hydrographs to select optimum storm duration. There was no significant rainfall during the period of observation. It would be beneficial to have gauges at that location and other locations for a period long enough to collect data from different rainfall events. Further research in this area is recommended.

Appendix I: List of Symbols

a = shape parameter
b = scale parameter
q_p = peak discharge (f_t^3/sec)
t_p = time to peak (hours)
t_r = rainfall duration (hours)
L = length of the main stream (mile)
L_c = distance from the outlet to a point in the stream (mile)
C_1 = 0.75
C_t = coefficient from gaged watersheds in the same region
C_2 = 2.75
C_p = coefficient from gaged watersheds in the same region
C_3 = 5.56
C_w = 1.22 for 75 percent width, in Snyder
C_w = 2.14 for 50 percent width, in Snyder
L = the total distance along the main channel from the point being considered to the upstream watershed boundary (ft)
S = the main channel slope (feet per foot)
I = the impervious area within the watershed assumed equal to 5 percent for an undeveloped watershed (percent)

Φ = the dimensionless watershed conveyance factor.
A = the watershed drainage area (square miles)
T_p = the time of rise to the peak of the unit hydrograph from the beginning of runoff (minutes)
Q_p = the peak flow of the unit hydrograph (cfs/in)
T_B = the time base of the unit hydrograph (minutes)
W_{50} = the width of the hydrograph at 50 percent of Qp (minutes) (Snyder)
W_{75} = the width of at 75 percent of Q_p (minutes) (Snyder)
i = rainfall intensity (in/hr)
t_c = time of concentration (hours)
d = duration of rainfall (hour)
t_l = the lag time from centriod of rainfall to peak discharge (hour)
$R_{(t)}$ = runoff depth at time interval dt (ft)
d_t = time interval (minute)
$Q_{(t)}$ = runoff hydrograph ordinate at time t (cfs)
$I_{(t)}$ = instantaneous hydrograph
T_c = basin time of concentration (minute)
d_t = time interval (minutes)
Δ_t = computation interval
D = design duration of the unit Hydrograph
$TUHD_{(T)}$ = ordinate of the desired unit Hydrograph at time t
$TUH_{(t)}$,$TUH_{(T-1)}$ etc. = ordinate of the original unit hydrograph at time t,t-1,t-2,etc
Sf = required flood storage
Vf = the flood volume
Sf/Vf = flood storage ratio
Q_p = peak outflow
I_p = peak inflow
Q_p/I_p = peak discharge ratio

Appendix II: HY8 Results For Outlet performance Curve

CURRENT DATE: 03-08-2002 FILE DATE: 03-08-2002
CURRENT TIME: 15:15:06 FILE NAME: OMAHA

FHWA CULVERT ANALYSIS
HY-8, VERSION 6.1

C U L V NO.	SITE DATA: INLET ELEV. (ft)	OUTLET ELEV. (ft)	CULVERT LENGTH (ft)	CULVERT SHAPE, MATERIAL, INLET: BARRELS SHAPE MATERIAL	SPAN (ft)	RISE (ft)	MANNING n	INLET TYPE
1	1114.00	1112.75	125.01	1 RCP	3.00	3.00	.012	CONVENTIONAL
2								
3								
4								
5								
6								

SUMMARY OF CULVERT FLOWS (cfs) FILE: OMAHA DATE: 03-08-2002

ELEV (ft)	TOTAL	1	2	3	4	5	6	ROADWAY	ITR
1114.00	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.00	1
1115.34	10.0	10.0	0.0	0.0	0.0	0.0	0.0	0.00	1
1116.09	20.0	20.0	0.0	0.0	0.0	0.0	0.0	0.00	1
1116.69	30.0	30.0	0.0	0.0	0.0	0.0	0.0	0.00	1
1116.93	40.0	34.0	0.0	0.0	0.0	0.0	0.0	5.57	20
1116.97	50.0	34.8	0.0	0.0	0.0	0.0	0.0	14.82	8
1117.01	60.0	35.4	0.0	0.0	0.0	0.0	0.0	24.25	7
1117.04	70.0	35.9	0.0	0.0	0.0	0.0	0.0	33.65	6
1117.06	75.0	36.2	0.0	0.0	0.0	0.0	0.0	38.10	4
1117.10	90.0	36.8	0.0	0.0	0.0	0.0	0.0	52.37	5
1117.12	100.0	37.3	0.0	0.0	0.0	0.0	0.0	62.24	5
1116.88	33.2	33.2	0.0	0.0	0.0	0.0	0.0	OVERTOPPING	

SUMMARY OF ITERATIVE SOLUTION ERRORS FILE: OMAHA DATE: 03-08-2002

HEAD ELEV (ft)	HEAD ERROR (ft)	TOTAL FLOW (cfs)	FLOW ERROR (cfs)	% FLOW ERROR
1114.00	0.000	0.00	0.00	0.00
1115.34	0.000	10.00	0.00	0.00
1116.09	0.000	20.00	0.00	0.00
1116.69	0.000	30.00	0.00	0.00
1116.93	-0.001	40.00	0.38	0.95
1116.97	-0.001	50.00	0.38	0.76
1117.01	-0.001	60.00	0.35	0.58
1117.04	-0.001	70.00	0.43	0.61
1117.06	-0.002	75.00	0.75	1.00
1117.10	-0.002	90.00	0.81	0.90
1117.12	-0.001	100.00	0.51	0.51

<1> TOLERANCE (ft) = 0.010 <2> TOLERANCE (%) = 1.000

CURRENT DATE: 03-08-2002
CURRENT TIME: 15:15:06

FILE DATE: 03-08-2002
FILE NAME: OMAHA

PERFORMANCE CURVE FOR CULVERT 1 - 1(3.00 (ft) BY 3.00 (ft)) RCP

DIS-CHARGE FLOW (cfs)	HEAD-WATER ELEV. (ft)	INLET CONTROL DEPTH (ft)	OUTLET CONTROL DEPTH (ft)	FULL FLOW HDS5 <F4>	NORMAL DEPTH (ft)	CRIT. DEPTH (ft)	OUTLET DEPTH (ft)	TW DEPTH (ft)	OUTLET VEL. (fps)	TW VEL. (fps)
0.00	1114.00	0.00	0.00	0-NF	0.00	0.00	0.00	0.00	0.00	0.00
10.00	1115.34	1.34	-0.17	6-FFt	0.74	0.99	1.01	1.01	4.76	1.10
20.00	1116.09	2.09	0.48	6-FFt	1.07	1.43	1.45	1.45	5.91	1.33
30.00	1116.69	2.69	1.16	6-FFt	1.34	1.77	1.78	1.78	6.88	1.49
34.05	1116.92	2.92	1.61	6-FFt	1.45	1.89	2.05	2.05	6.63	1.61
34.80	1116.97	2.97	1.88	6-FFt	1.46	1.91	2.28	2.28	6.05	1.71
35.40	1117.00	3.00	2.12	6-FFt	1.48	1.93	2.49	2.49	5.66	1.79
35.92	1117.03	3.03	2.33	6-FFt	1.49	1.94	2.68	2.68	5.40	1.87
36.15	1117.05	3.05	2.43	6-FFt	1.50	1.95	2.76	2.76	5.33	1.90
36.82	1117.09	3.09	2.71	4-FFt	1.51	1.97	1.51	3.01	10.29	1.99
37.25	1117.12	3.12	2.89	4-FFt	1.53	1.98	1.53	3.16	10.32	2.05

El. inlet face invert 1114.00 ft El. outlet invert 1112.75 ft
El. inlet throat invert 0.00 ft El. inlet crest 0.00 ft

***** SITE DATA ***** CULVERT INVERT **************

INLET STATION	0.00 ft
INLET ELEVATION	1114.00 ft
OUTLET STATION	125.00 ft
OUTLET ELEVATION	1112.75 ft
NUMBER OF BARRELS	1
SLOPE (V/H)	0.0100
CULVERT LENGTH ALONG SLOPE	125.01 ft

***** CULVERT DATA SUMMARY ************************

BARREL SHAPE	CIRCULAR
BARREL DIAMETER	3.00 ft
BARREL MATERIAL	CONCRETE
BARREL MANNING'S n	0.012
INLET TYPE	CONVENTIONAL
INLET EDGE AND WALL	SQUARE EDGE WITH HEADWALL
INLET DEPRESSION	NONE

CURRENT DATE: 03-08-2002 FILE DATE: 03-08-2002
CURRENT TIME: 15:15:06 FILE NAME: OMAHA

TAILWATER

******* REGULAR CHANNEL CROSS SECTION ****************

BOTTOM WIDTH	6.00 ft
SIDE SLOPE H/V (X:1)	3.0
CHANNEL SLOPE V/H (ft/ft)	0.001
MANNING'S n (.01-0.1)	0.035
CHANNEL INVERT ELEVATION	1112.75 ft
CULVERT NO.1 OUTLET INVERT ELEVATION	1112.75 ft

******* UNIFORM FLOW RATING CURVE FOR DOWNSTREAM CHANNEL

FLOW (cfs)	W.S.E. (ft)	FROUDE NUMBER	DEPTH (ft)	VEL. (f/s)	SHEAR (psf)
0.00	1112.75	0.000	0.00	0.00	0.00
10.00	1113.76	0.192	1.01	1.10	0.06
20.00	1114.20	0.195	1.45	1.33	0.09
30.00	1114.53	0.197	1.78	1.49	0.11
40.00	1114.80	0.198	2.05	1.61	0.13
50.00	1115.03	0.199	2.28	1.71	0.14
60.00	1115.24	0.200	2.49	1.79	0.16
70.00	1115.43	0.201	2.68	1.87	0.17
75.00	1115.51	0.201	2.76	1.90	0.17
90.00	1115.76	0.202	3.01	1.99	0.19
100.00	1115.91	0.203	3.16	2.05	0.20

ROADWAY OVERTOPPING DATA

ROADWAY SURFACE	GRAVEL
EMBANKMENT TOP WIDTH	75.00 ft
CREST LENGTH	200.00 ft
OVERTOPPING CREST ELEVATION	1116.88 ft

Appendix III: Preliminary Sizing Methods

Baker's formula:

$$\frac{S_f}{V_f} = 1 - \frac{Q_p}{I_p}$$

where, $\frac{Q_p}{I_p} = 0.6$ and $V_f = 252{,}648\ ft^3$

$$S_f = (1 - 0.6)252{,}648\ ft^3$$

$$S_f = 101{,}059\ ft^3 \left(\frac{1\ acre}{43560\ ft^2} \right) = 2.3\ acre - ft$$

Abt and Grigg Formula:

$$\frac{S_f}{V_f} = \left[1 - \frac{Q_p}{I_p} \right]^2$$

where, $\frac{Q_p}{I_p} = 0.6$ and $V_f = 252{,}648\ ft^3$

$$S_f = (1 - 0.6)^2 (252{,}648\ ft^3)$$

$$S_f = 40424\ ft^3 \left(\frac{1\ acre}{43560\ ft^2} \right) = 0.9\ acre - ft$$

Bruce M. McEnroe formula:

$$\frac{S_f}{V_f} = \left[0.97 - 1.42 \left(\frac{Q_p}{I_p} \right) + 0.82 \left(\frac{Q_p}{I_p} \right)^2 - .34 \left(\frac{Q_p}{I_p} \right)^3 \right]$$

where, $\frac{Q_p}{I_p} = 0.6$ and $V_f = 252{,}648\ ft^3$

$$S_f = 85{,}840 ft^3$$

$$S_f = 85{,}840\ ft^3 \left(\frac{1\ acre}{43560\ ft^2} \right) = 2.0\ acre - ft$$

Appendix IV: Generation of Design Storms

Generation of 50-year, 1-hour storm:

Column (2):

$$i = \frac{a}{D+b} \quad \text{for } D \leq 2\ hr$$

$$i = \frac{4.46}{D+0.38} \quad \text{for } D \leq 2\ hr$$

$$i = cD^{d} \quad \text{for } D > 2\ hr$$

$$i = 3.10D^{-0.79} \quad \text{for } D > 2\ hr$$

Column (7):

$$Q = \frac{(P-0.2S)^2}{P+0.8S} \quad \text{if } P \leq 0.2S,\ Q = 0$$

$$Q = \frac{(P-0.2(2.2))^2}{P+0.8(2.2)} = \frac{(P-0.44)^2}{P+1.76} \quad \text{if } P \leq 0.44,\ Q = 0$$

(1) Duration (hr)	(2) Intensity (in/hr)	(3) Cumulative Depth (1)x(2) (in)	(4) Incremental Depth (in)	(5) Precipitation[a] (in)	(6) Cumulative Precipitation, P (in)	(7) Cumulative Runoff, Q (in)	(8) Incremental Runoff (in)
0.10	9.31	0.93	0.93	0.11	0.11	0.000	0.000
0.20	7.70	1.54	0.61	0.16	0.27	0.000	0.000
0.30	6.57	1.97	0.43	0.25	0.52	0.003	0.003
0.40	5.72	2.29	0.32	0.43	0.95	0.096	0.093
0.50	5.07	2.54	0.25	0.93	1.88	0.570	0.474
0.60	4.56	2.73	0.20	0.61	2.49	0.988	0.419
0.70	4.13	2.89	0.16	0.32	2.81	1.228	0.240
0.80	3.78	3.03	0.13	0.20	3.01	1.381	0.153
0.90	3.49	3.14	0.11	*0.13*	3.14	1.486	0.105
1.00	3.23	3.23	0.10	0.10	3.23	1.563	0.077

[a]Using alternating block method

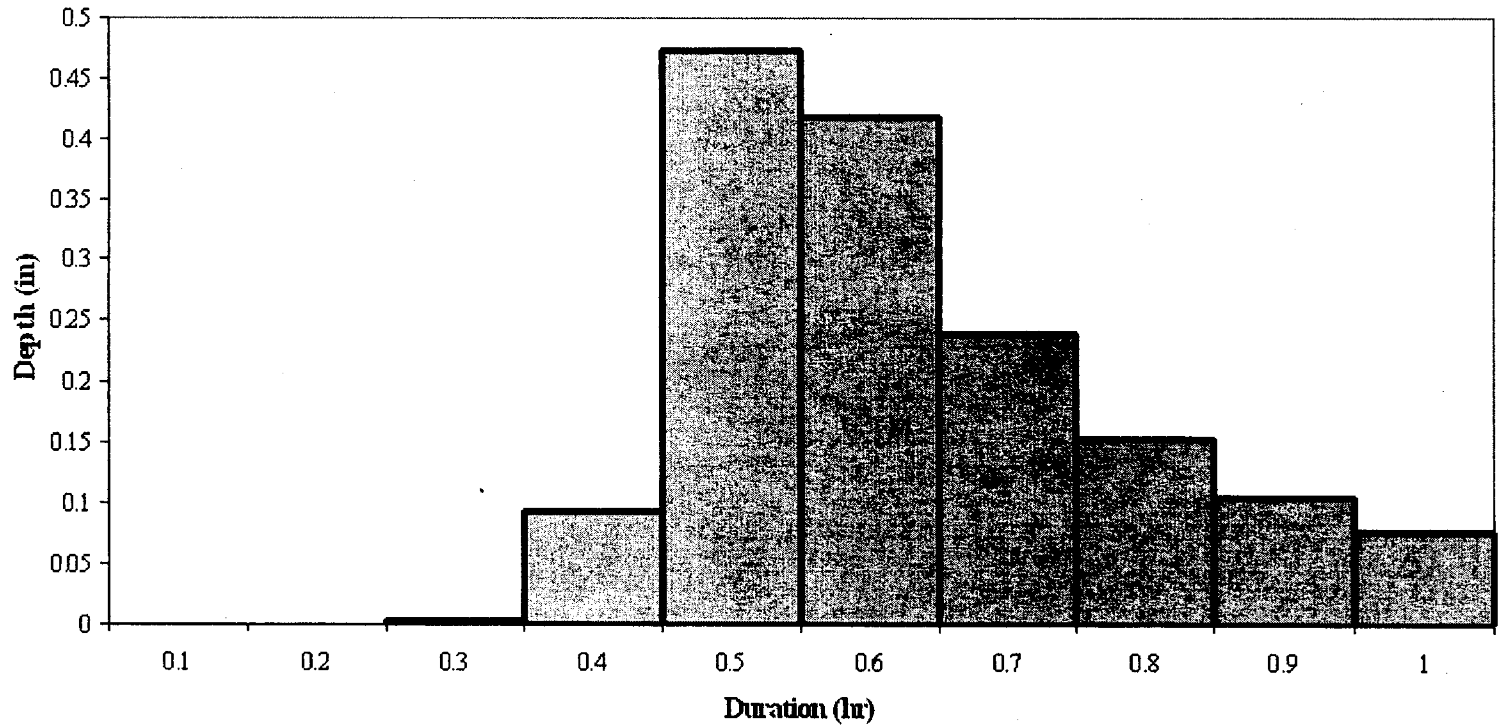


Figure 4.1 50-yr 1-hr Rainfall Excess

Appendix V: Unit Hydrographs

SCS Unit Hydrograph:

t/t_p	Q/Q_p	t (hr)	Q (cfs)
0	0	0	0
0.1	0.02	0.07	0.3
0.2	0.08	0.14	1.4
0.3	0.16	0.21	3.0
0.4	0.28	0.28	5.3
0.5	0.43	0.35	8.1
0.6	0.60	0.42	11.3
0.7	0.77	0.49	14.5
0.8	0.89	0.56	16.7
0.9	0.97	0.63	18.3
1.0	1.00	0.70	18.8
1.1	0.98	0.77	18.4
1.2	0.92	0.84	17.3
1.3	0.84	0.91	15.8
1.4	0.75	0.98	14.1
1.5	0.66	1.05	12.4
1.6	0.56	1.12	10.5
1.8	0.42	1.26	7.9
2.0	0.32	1.40	6.0
2.2	0.24	1.54	4.5
2.4	0.18	1.68	3.4
2.6	0.13	1.82	2.5
2.8	0.10	1.96	1.8
3.0	0.08	2.10	1.4
3.5	0.04	2.45	0.7
4.0	0.02	2.80	0.3
4.5	0.01	3.15	0.2
5.0	0.00	3.50	0.1

$$Q_p = \frac{484A}{t_p}$$

$$t_p = \frac{D}{2} + t_l$$

$$t_l = C_t (L\,L_c)^{0.3}$$

Assume $C_t = 2.0$

$L = 1300\ ft = 0.246\ mi$

$L_c = 500\ ft = 0.095\ mi$

$D = 6\ \text{min} = 0.1\ hr$

$A = 17.42\ acre = 0.0272\ mi^2$

$$t_l = 2.0(0.246(0.095))^{0.3} = 0.65\ hr$$

$$t_p = \frac{0.1}{2} + 0.65 = 0.7\ hr$$

$$Q_p = \frac{484(0.0272)}{0.7} = 18.81\ cfs$$

USGS Unit Hydrograph:

Use Q_p and t_l as calculated in the SCS method.

t/t_l	Q/Q_p	t (hr)	Q (cfs)
0.00	0.00	0.00	0.0
0.25	0.12	0.16	2.3
0.30	0.16	0.20	3.0
0.35	0.21	0.23	4.0
0.40	0.26	0.26	4.9
0.45	0.33	0.29	6.2
0.50	0.40	0.33	7.5
0.55	0.49	0.36	9.2
0.60	0.58	0.39	10.9
0.65	0.67	0.42	12.6
0.70	0.76	0.46	14.3
0.75	0.84	0.49	15.8
0.80	0.90	0.52	16.9
0.85	0.95	0.55	17.9
0.90	0.98	0.59	18.4
0.95	1.00	0.62	18.8
1.00	0.99	0.65	18.6
1.05	0.96	0.68	18.1
1.10	0.92	0.72	17.3
1.15	0.86	0.75	16.2
1.20	0.80	0.78	15.1
1.25	0.74	0.81	13.9
1.30	0.68	0.85	12.8
1.35	0.62	0.88	11.7
1.40	0.56	0.91	10.5
1.45	0.51	0.94	9.6
1.50	0.47	0.98	8.8
1.55	0.43	1.01	8.1
1.60	0.39	1.04	7.3
1.65	0.36	1.07	6.8
1.70	0.33	1.11	6.2
1.75	0.30	1.14	5.6
1.80	0.28	1.17	5.3
1.85	0.26	1.20	4.9
1.90	0.24	1.24	4.5
1.95	0.22	1.27	4.1
2.00	0.20	1.30	3.8
2.05	0.19	1.33	3.6
2.10	0.17	1.37	3.2
2.15	0.16	1.40	3.0
2.20	0.15	1.43	2.8
2.25	0.14	1.46	2.6
2.30	0.13	1.50	2.5
2.35	0.12	1.53	2.3
2.40	0.11	1.56	2.1

Snyder Unit Hydrograph:

$$Q_p = \frac{C_p A}{t_p} cfs$$

$$t_p = C_t (L\, L_c)^{0.3}\ hr$$

$$T = 3 + \frac{t_p}{8}\ hr$$

$$t_D = \frac{t_p}{5.5}\ hr$$

$$t_{pR} = t_p + 0.25(t_r - t_D)\, hr$$

$$Q_{pR} = Q_p \frac{t_p}{t_{pR}}\ hr$$

Assume $C_p = 400$

$L = 1300\ ft = 0.246\ mi$

$L_c = 500\ ft = 0.095\ mi$

$t_r = 6\ \text{min} = 0.1\ hr$

$A = 17.42\ acre = 0.0272\ mi^2$

$$t_p = 2.0(0.246(0.095))^{0.3} = 0.65\ hrs$$

$$Q_p = \frac{400(0.0272)}{0.65} = 16.79\ cfs$$

$$T = 3 + \frac{0.65}{8} = 3.08\ days \quad \approx \quad 74\ hrs$$

$$t_D = \frac{0.65}{5.5} = .12\ hrs$$

$$t_{pR} = 0.65 + 0.25(0.1 - 0.12) \approx 0.65\ hrs$$

$$Q_{pR} = Q_p \frac{0.65}{0.65} = Q_p = 16.79\ cfs$$

$$W_{50} = \frac{770 A^{1.08}}{Q_{pR}^{\ 1.08}} = \frac{770(0.0272)^{1.08}}{16.79^{1.08}} = 0.75\ hr$$

$$W_{75} = \frac{440 A^{1.08}}{Q_{pR}^{\ 1.08}} = \frac{440(0.0272)^{1.08}}{16.79^{1.08}} = 0.43\ hr$$

Gamma Distribution Unit Hydrograph:

Use Q_p and t_p as calculated in SCS Unit Hydrograph method.

For Gamma Distribution Unit Hydrograph, $Q_p = q_p$.

t (hr)	f	$q(f)$ (cfs)
0.00	0.00	0.0
0.10	0.14	0.3
0.20	0.29	2.5
0.30	0.43	6.7
0.40	0.57	11.5
0.50	0.71	15.6
0.60	0.86	18.0
0.70	1.00	18.8
0.80	1.14	18.2
0.90	1.29	16.5
1.00	1.43	14.4
1.10	1.57	12.0
1.20	1.71	9.8
1.30	1.86	7.7
1.40	2.00	6.0
1.50	2.14	4.5
1.60	2.29	3.4
1.70	2.43	2.5
1.80	2.57	1.8
1.90	2.71	1.3
2.00	2.86	0.9
2.10	3.00	0.6
2.20	3.14	0.5
2.30	3.29	0.3
2.40	3.43	0.2
2.50	3.57	0.1
2.60	3.71	0.1
2.70	3.86	0.1
2.80	4.00	0.0
2.90	4.14	0.0
3.00	4.29	0.0
3.10	4.43	0.0
3.20	4.57	0.0
3.30	4.71	0.0
3.40	4.86	0.0
3.50	5.00	0.0

$$q(f) = q_p f^a e^{a(1-f)}$$

where f is any fraction of t_p from 0 to $5t_p$.

$$a = 0.045 + 0.5f_a + 5.6f_a^2 + 0.3f_a^3$$

$$f_a = \frac{q_p t_p}{A}$$

$$A = 17.42\ acres$$

$$f_a = \frac{18.81(0.7)}{17.42} = 0.76$$

$$a = 0.045 + 0.5(0.76) + 5.6(0.76)^2 + 0.3(0.76)^3 = 3.75$$

$$q(f) = 18.81\left(f^{3.75}\right)e^{3.75(1-f)}$$

Appendix VI: Generation of Runoff Hydrographs

		Direct Runoff Hydrograph Ordinates								
Time (hr)	SCS UH (cfs)	Unit 1 0.003xUH	Unit 2 0.093xUH	Unit 3 0.474xUH	Unit 4 0.419xUH	Unit 5 0.240xUH	Unit 6 0.153xUH	Unit 7 0.105xUH	Unit 8 0.077xUH	Total (cfs)
0.0	0.0	0.000								0.0
0.1	0.6	0.002	0.000							0.0
0.2	3.0	0.008	0.056	0.000						0.1
0.3	6.0	0.016	0.279	0.284	0.000					0.6
0.4	10.5	0.029	0.557	1.422	0.251	0.000				2.3
0.5	14.7	0.040	0.975	2.844	1.256	0.144	0.000			5.3
0.6	17.5	0.048	1.365	4.977	2.512	0.720	0.092	0.000		9.7
0.7	18.8	0.051	1.625	6.968	4.397	1.439	0.458	0.063	0.000	15.0
0.8	18.0	0.049	1.746	8.295	6.155	2.519	0.916	0.316	0.046	20.0
0.9	16.0	0.044	1.671	8.911	7.328	3.526	1.604	0.632	0.231	24.0
1.0	13.7	0.037	1.486	8.532	7.872	4.198	2.245	1.106	0.461	25.9
1.1	13.0	0.036	1.272	7.584	7.537	4.510	2.673	1.548	0.807	26.0
1.2	9.0	0.025	1.207	6.494	6.700	4.318	2.872	1.843	1.130	24.6
1.3	7.3	0.020	0.836	6.162	5.737	3.838	2.749	1.980	1.345	22.7
1.4	6.0	0.016	0.678	4.266	5.443	3.286	2.444	1.896	1.445	19.5
1.5	5.0	0.014	0.557	3.460	3.769	3.118	2.093	1.685	1.384	16.1
1.6	4.0	0.011	0.464	2.844	3.057	2.159	1.986	1.443	1.230	13.2
1.7	3.3	0.009	0.371	2.370	2.512	1.751	1.375	1.369	1.053	10.8
1.8	2.5	0.007	0.306	1.896	2.094	1.439	1.115	0.948	0.999	8.8
1.9	2.0	0.005	0.232	1.564	1.675	1.199	0.916	0.769	0.692	7.1
2.0	1.7	0.005	0.186	1.185	1.382	0.960	0.764	0.632	0.561	5.7
2.1	1.4	0.004	0.158	0.948	1.047	0.792	0.611	0.527	0.461	4.6
2.2	1.2	0.003	0.130	0.806	0.837	0.600	0.504	0.421	0.384	3.7
2.3	1.0	0.003	0.111	0.664	0.712	0.480	0.382	0.348	0.307	3.0
2.4	0.7	0.002	0.093	0.569	0.586	0.408	0.305	0.263	0.254	2.5
2.5	0.6	0.002	0.065	0.474	0.502	0.336	0.260	0.211	0.192	2.0
2.6	0.5	0.001	0.056	0.332	0.419	0.288	0.214	0.179	0.154	1.6
2.7	0.4	0.001	0.046	0.284	0.293	0.240	0.183	0.147	0.131	1.3
2.8	0.4	0.001	0.037	0.237	0.251	0.168	0.153	0.126	0.108	1.1
2.9	0.3	0.001	0.037	0.190	0.209	0.144	0.107	0.105	0.092	0.9
3.0	0.3	0.001	0.028	0.190	0.167	0.120	0.092	0.074	0.077	0.8
3.1	0.2	0.001	0.028	0.142	0.167	0.096	0.076	0.063	0.054	0.6
3.2	0.2	0.001	0.019	0.142	0.126	0.096	0.061	0.053	0.046	0.5
3.3	0.2	0.001	0.019	0.095	0.126	0.072	0.061	0.042	0.038	0.5
3.4	0.1	0.000	0.019	0.095	0.084	0.072	0.046	0.042	0.031	0.4
3.5	0.1	0.000	0.009	0.095	0.084	0.048	0.046	0.032	0.031	0.3
3.6			0.009	0.047	0.084	0.048	0.031	0.032	0.023	0.3
3.7				0.047	0.042	0.048	0.031	0.021	0.023	0.2
3.8					0.042	0.024	0.031	0.021	0.015	0.1
3.9						0.024	0.015	0.021	0.015	0.1
4.0							0.015	0.011	0.015	0.0
4.1								0.011	0.008	0.0
4.2									0.008	0.0

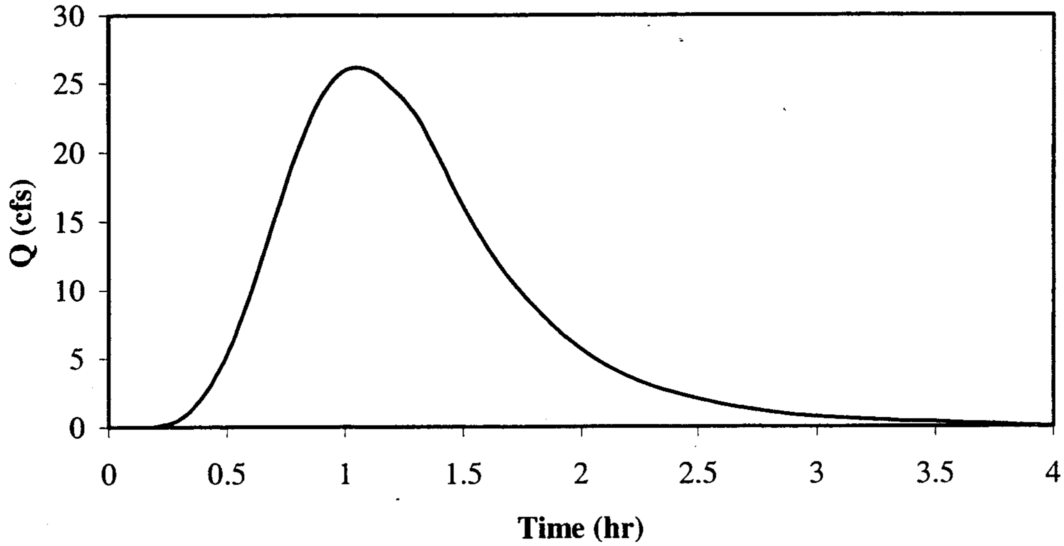
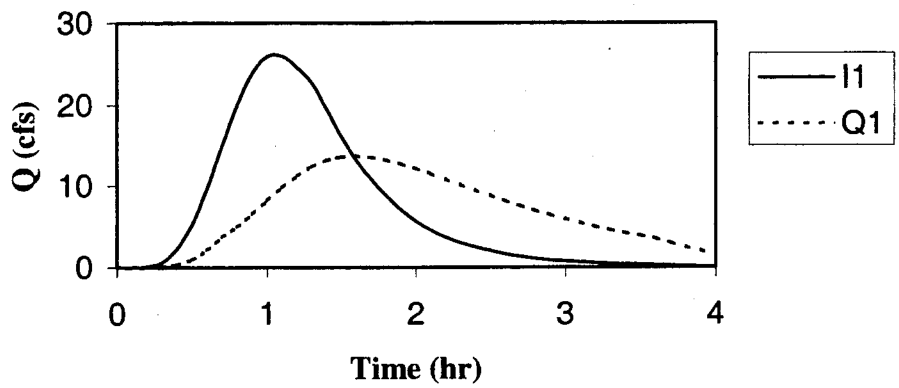


Fig A6.1 SCS Runoff Hydrograph for 50-yr 1-hr Storm

Appendix VII: Routing of Runoff Hydrographs

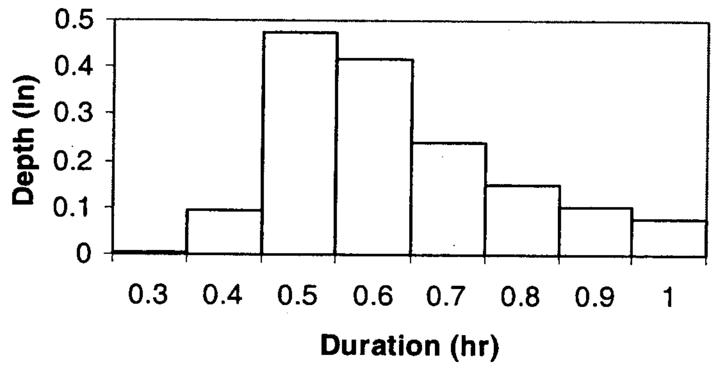
Step 1. Inflow values are retrieved from the direct runoff hydrograph.
Step 2. From the values in columns 5 and 6, calculate $(2S_1/\Delta t - Q_1)$, where Δt=0.1 hr.
Step 3. $(2S_2/\Delta t + Q_2)$ is the sum of columns 4 and 7.
Step 4. Q_2 is read from Figure 3.15 corresponding to the value of column 8.
Step 5. S_2 is obtained by solving $(2S_2/\Delta t + Q_2)$.
Step 6. Q_2 becomes the next Q_1, S_2 becomes the next S_1 and the process repeats itself, starting with Step 1.

(1) Time (hr)	(2) I_1 (cfs)	(3) Inflow I_2 (cfs)	(4) $I_1 + I_2$ (cfs)	(5) Storage S_1 (ft^3)	(6) Q_1 (cfs)	(7) $2S_1/\Delta t$ - Q_1 (cfs)	(8) $2S_2/\Delta t + Q_2$ (cfs)	(9) Outflow Q_2 (cfs)	(10) S_2 (ft^3)
0.0	0.00								
0.1	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.27
0.2	0.06	0.00	0.07	0.27	0.00	0.00	0.07	0.01	10.95
0.3	0.58	0.06	0.64	10.95	0.01	0.05	0.70	0.06	114.25
0.4	2.26	0.58	2.84	114.25	0.06	0.57	3.41	0.31	558.09
0.5	5.26	2.26	7.52	558.09	0.31	2.79	10.31	0.94	1687.40
0.6	9.71	5.26	14.97	1687.40	0.94	8.44	23.41	2.12	3831.94
0.7	15.00	9.71	24.71	3831.94	2.12	19.17	43.88	3.74	7224.46
0.8	20.04	15.00	35.04	7224.46	3.74	36.39	71.44	5.00	11958.29
0.9	23.95	20.04	43.99	11958.29	5.00	61.43	105.42	6.55	17797.09
1.0	25.94	23.95	49.88	17797.09	6.55	92.32	142.21	8.23	24116.09
1.1	25.97	25.94	51.90	24116.09	8.23	125.75	177.65	9.84	30205.84
1.2	24.59	25.97	50.55	30205.84	9.84	157.97	208.52	11.25	35508.28
1.3	22.67	24.59	47.25	35508.28	11.25	186.02	233.27	12.38	39760.23
1.4	19.47	22.67	42.14	39760.23	12.38	208.51	250.65	13.17	42746.06
1.5	16.08	19.47	35.55	42746.06	13.17	224.31	259.86	13.59	44327.89
1.6	13.19	16.08	29.27	44327.89	13.59	232.67	261.95	13.69	44686.36
1.7	10.81	13.19	24.00	44686.36	13.69	234.57	258.57	13.53	44107.06
1.8	8.80	10.81	19.61	44107.06	13.53	231.51	251.12	13.19	42826.63
1.9	7.05	8.80	15.86	42826.63	13.19	224.73	240.59	12.71	41017.45
2.0	5.67	7.05	12.73	41017.45	12.71	215.16	227.89	12.13	38835.41
2.1	4.55	5.67	10.22	38835.41	12.13	203.62	213.84	11.49	36421.81
2.2	3.69	4.55	8.23	36421.81	11.49	190.85	199.08	10.82	33886.90
2.3	3.01	3.69	6.69	33886.90	10.82	177.44	184.13	10.14	31318.50
2.4	2.48	3.01	5.49	31318.50	10.14	163.85	169.34	9.47	28777.18
2.5	2.04	2.48	4.52	28777.18	9.47	150.41	154.93	8.81	26301.89
2.6	1.64	2.04	3.68	26301.89	8.81	137.31	141.00	8.17	23908.39
2.7	1.33	1.64	2.97	23908.39	8.17	124.65	127.62	7.56	21610.33
2.8	1.08	1.33	2.41	21610.33	7.56	112.49	114.90	6.98	19425.49
2.9	0.89	1.08	1.97	19425.49	6.98	100.94	102.90	6.44	17364.15
3.0	0.75	0.89	1.63	17364.15	6.44	90.03	91.67	5.92	15433.57
3.1	0.63	0.75	1.38	15433.57	5.92	79.82	81.19	5.45	13634.75
3.2	0.54	0.63	1.17	13634.75	5.45	70.30	71.47	5.00	11964.73
3.3	0.45	0.54	1.00	11964.73	5.00	61.47	62.46	4.59	10417.09
3.4	0.39	0.45	0.84	10417.09	4.59	53.28	54.12	4.21	8984.01
3.5	0.34	0.39	0.73	8984.01	4.21	45.70	46.43	3.86	7662.92
3.6	0.27	0.34	0.62	7662.92	3.86	38.71	39.33	3.54	6442.65
3.7	0.21	0.27	0.49	6442.65	3.54	32.26	32.74	2.97	5358.92
3.8	0.13	0.21	0.34	5358.92	2.97	26.80	27.15	2.46	4443.24
3.9	0.08	0.13	0.21	4443.24	2.46	22.22	22.43	2.03	3671.37
4.0	0.04	0.08	0.12	3671.37	2.03	18.36	18.48	1.68	3024.51
4.1	0.02	0.04	0.06	3024.51	1.68	15.13	15.19	1.38	2485.59
4.2	0.01	0.02	0.03	2485.59	1.38	12.43	12.46	1.13	2038.94

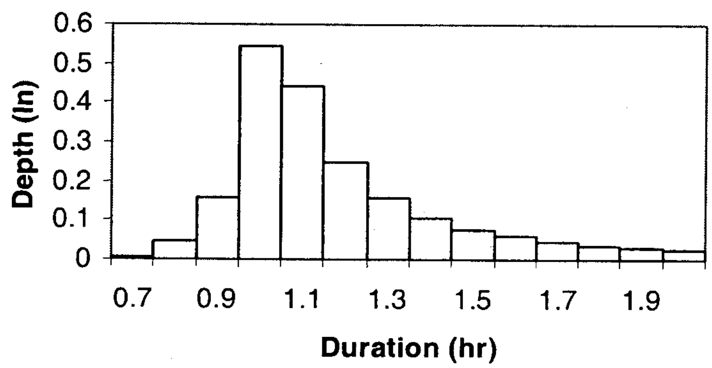


50-yr, 1-hr Inflow Outflow Hydrograph

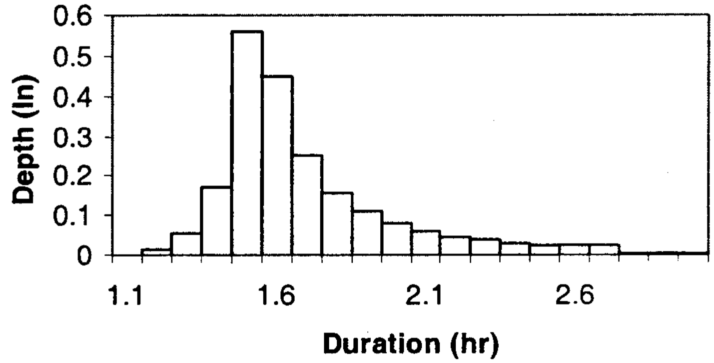
Appendix VIII: Rainfall Excess Heytographs



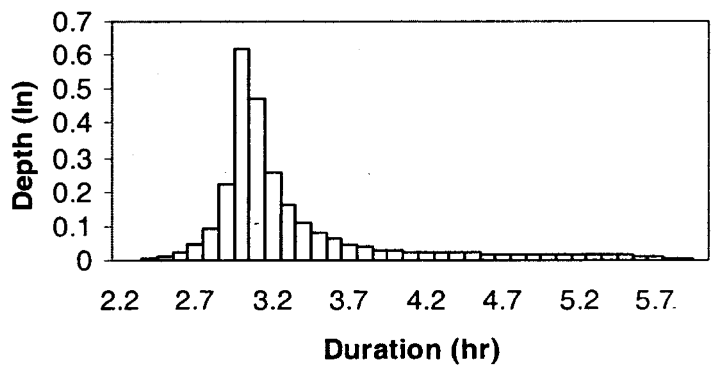
50-Year 1-hr Rainfall Excess



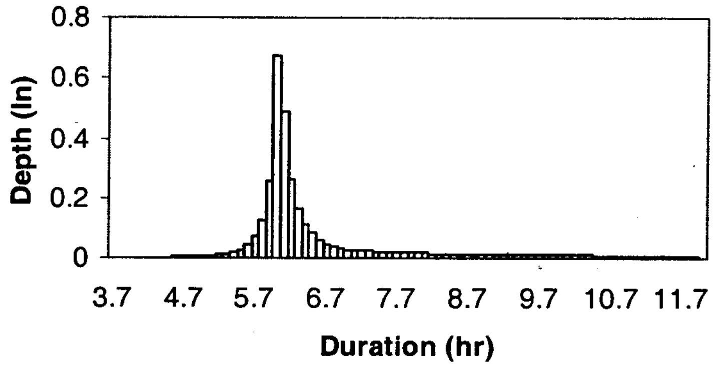
50-Year 2-hr Rainfall Excess



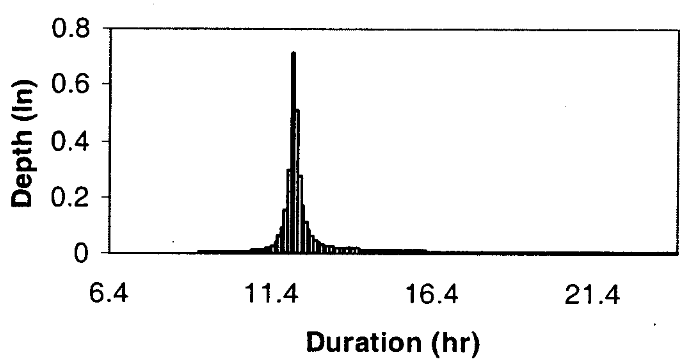
50-Year 3-hr Rainfall Excess



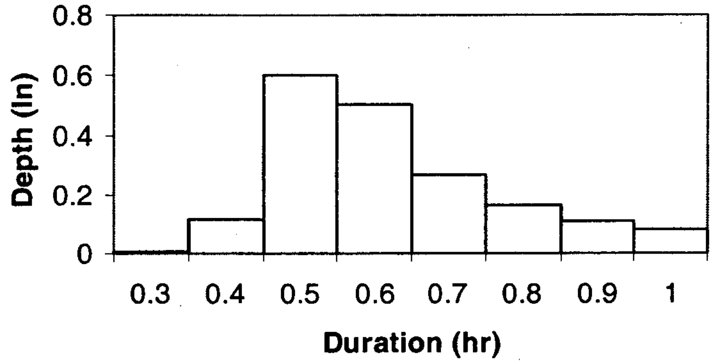
50-Year 6-hr Rainfall Excess



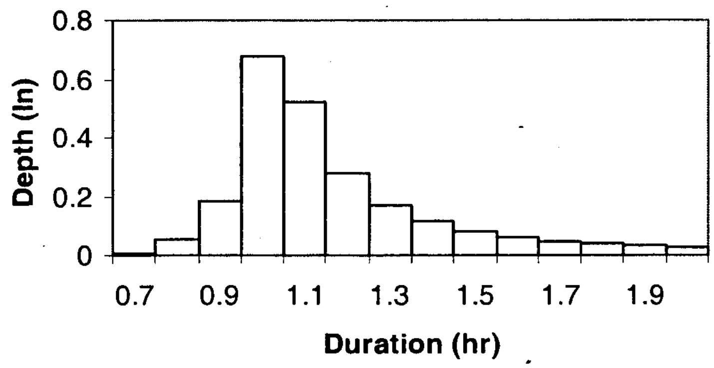
50-Year 12-hr Rainfall Excess



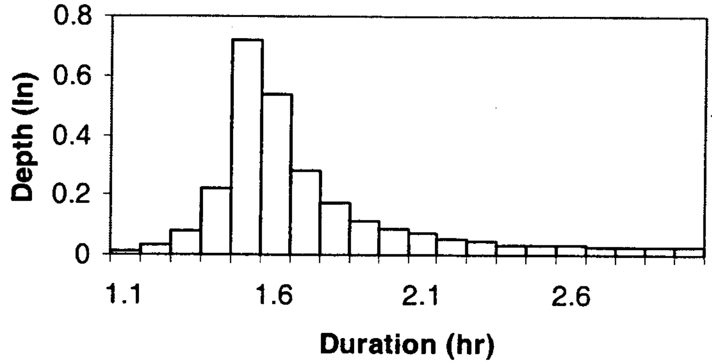
50-Year 24-hr Rainfall Excess



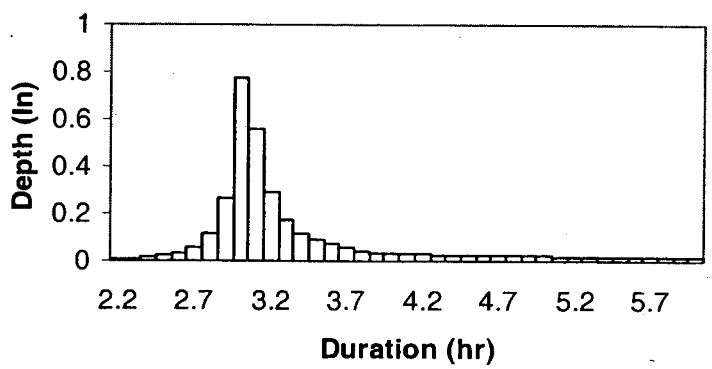
100-Year 1-hr Rainfall Excess



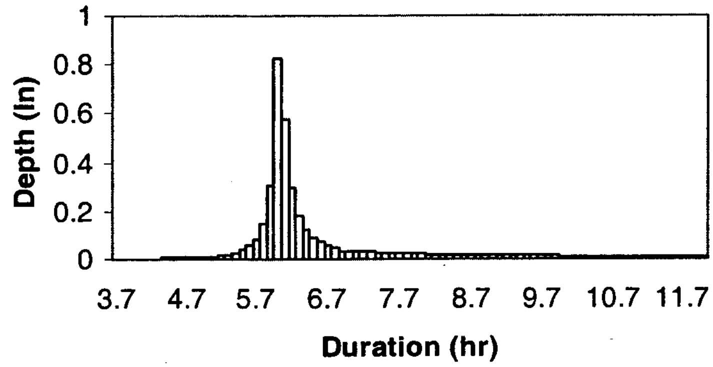
100-Year 2-hr Rainfall Excess



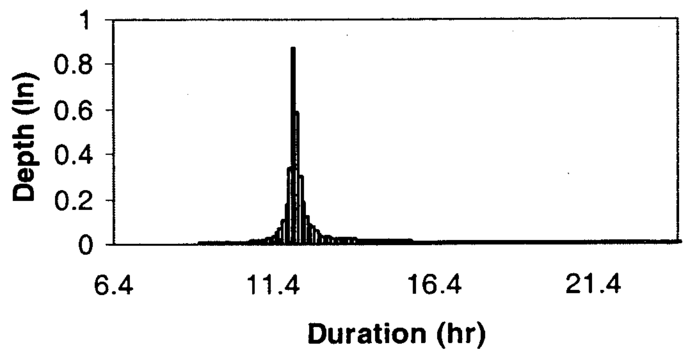
100-Year 3-hr Rainfall Excess



100-Year 6-hr Rainfall Excess

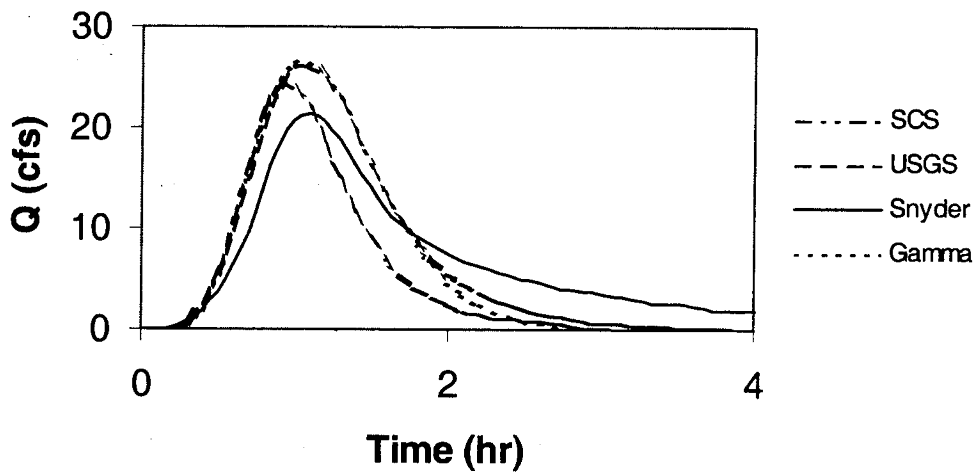


100-Year 12-hr Rainfall Excess

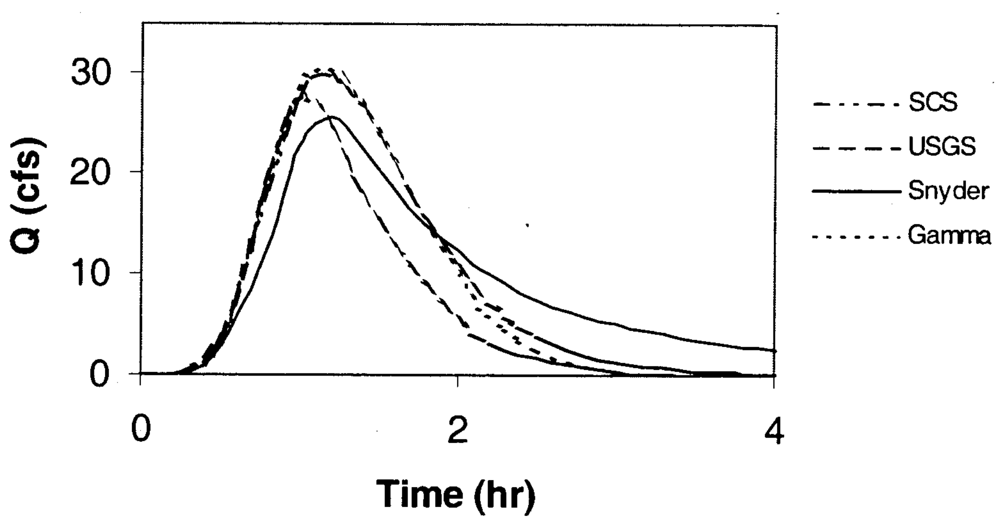


100-Year 24-hr Rainfall Excess

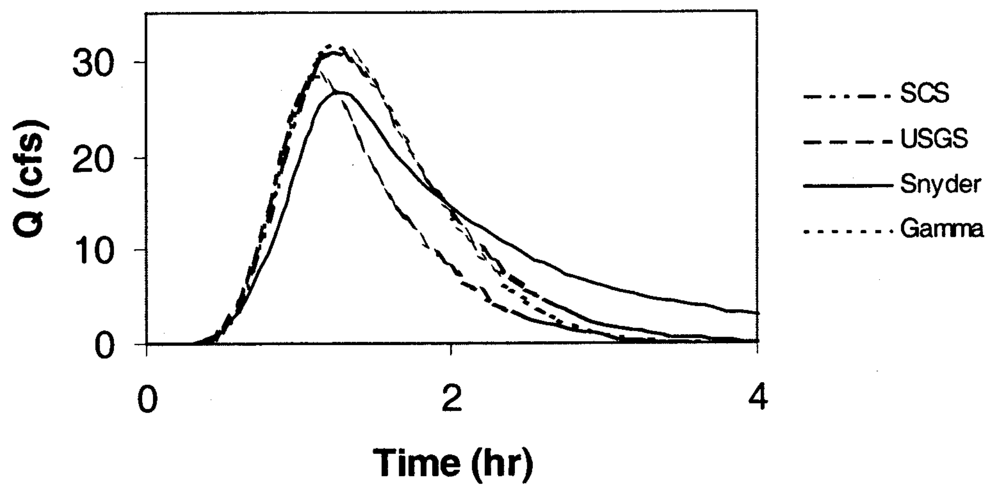
Appendix IX: Runoff Hydrographs



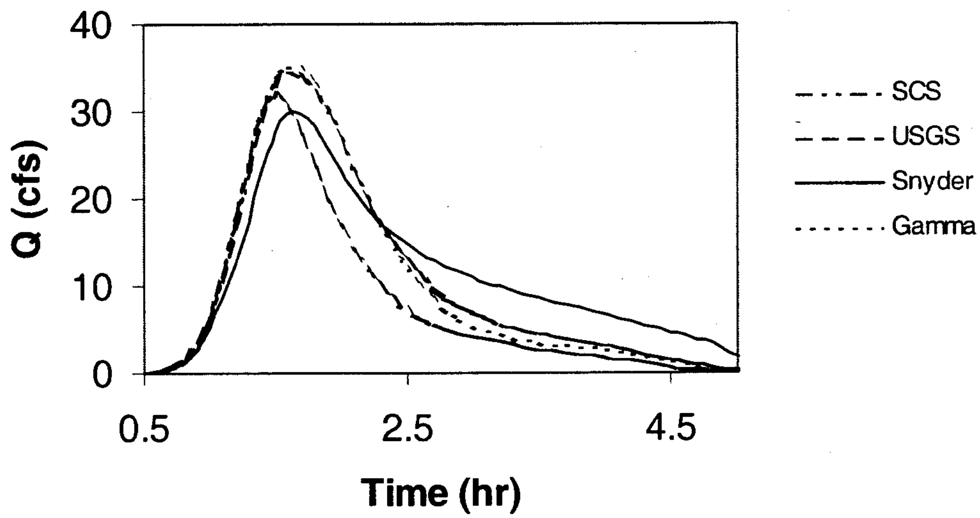
50-yr 1-hr Runoff Hydorgraphs



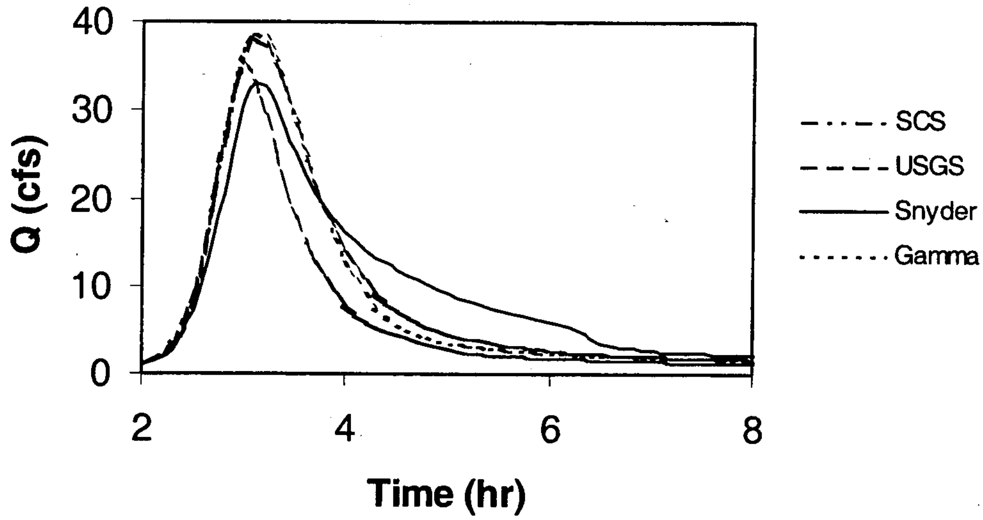
50-yr 2-hr Runoff Hydrographs



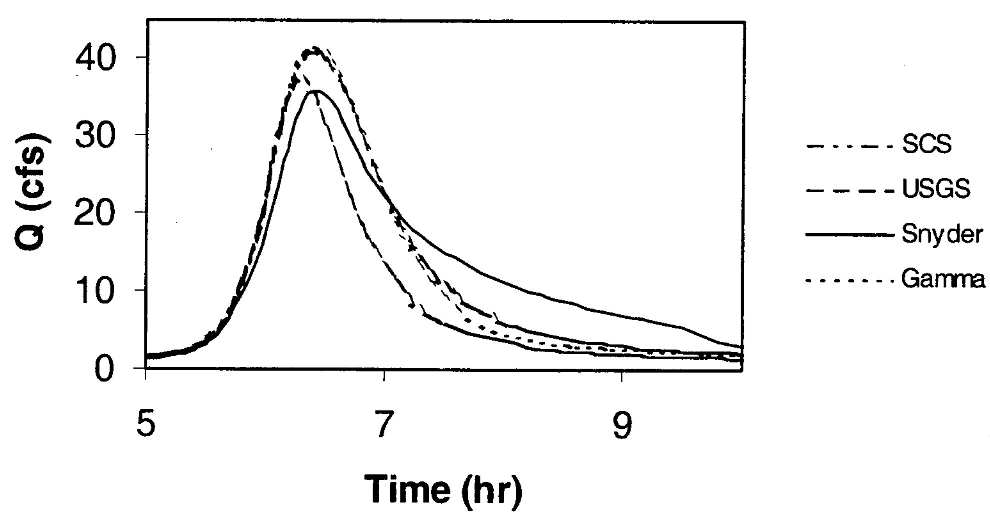
50-yr 3-hr Runoff Hydrographs



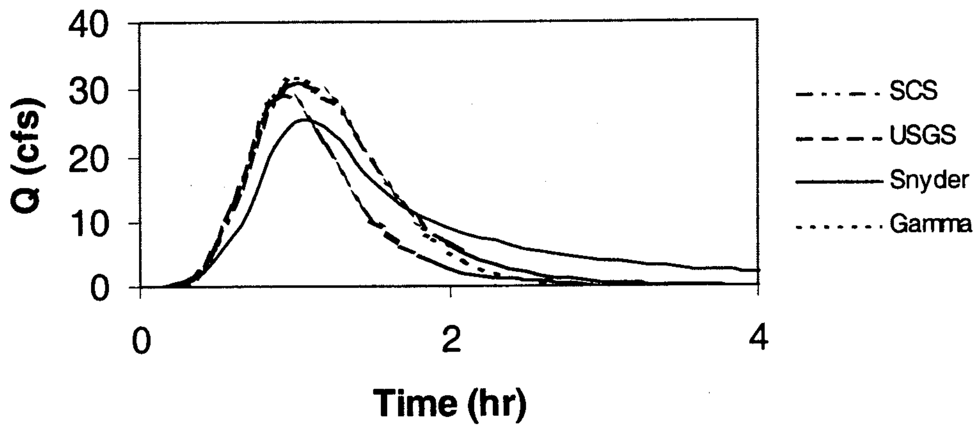
50-yr 6-hr Runoff Hydrographs



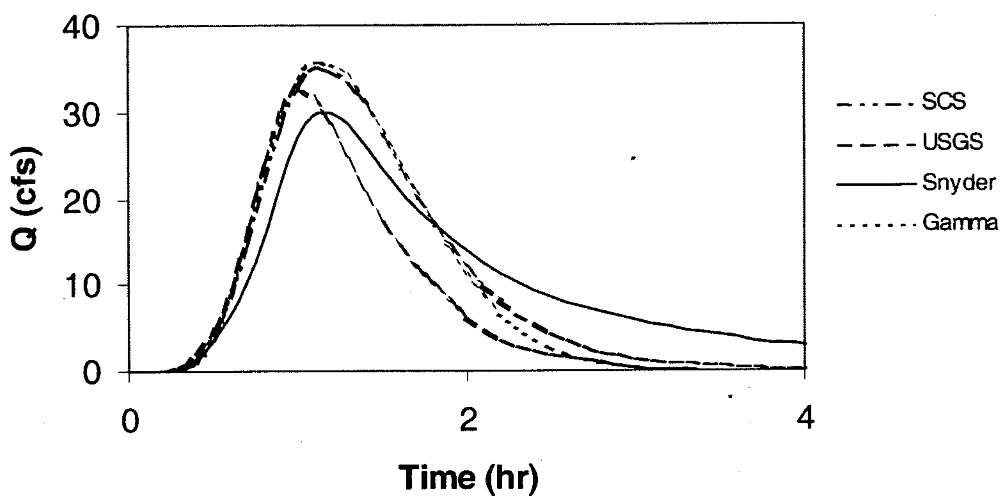
50-yr 12-hr Hydrographs



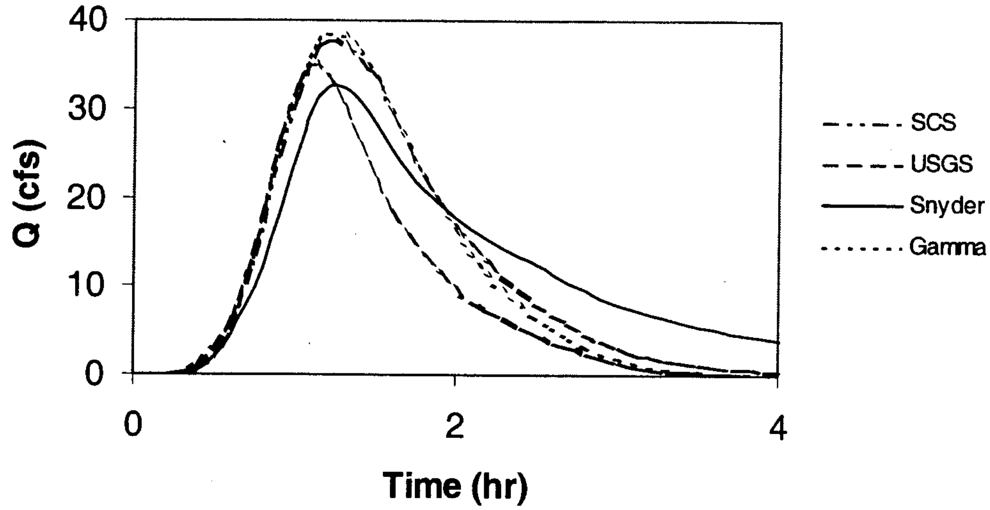
50-yr 24-hr Hydrographs



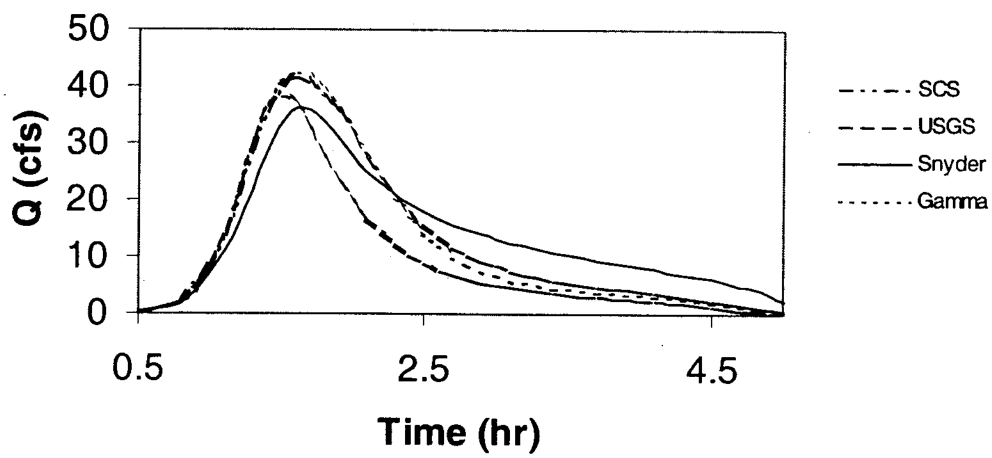
100-yr 1-hr Runoff Hydorgraphs



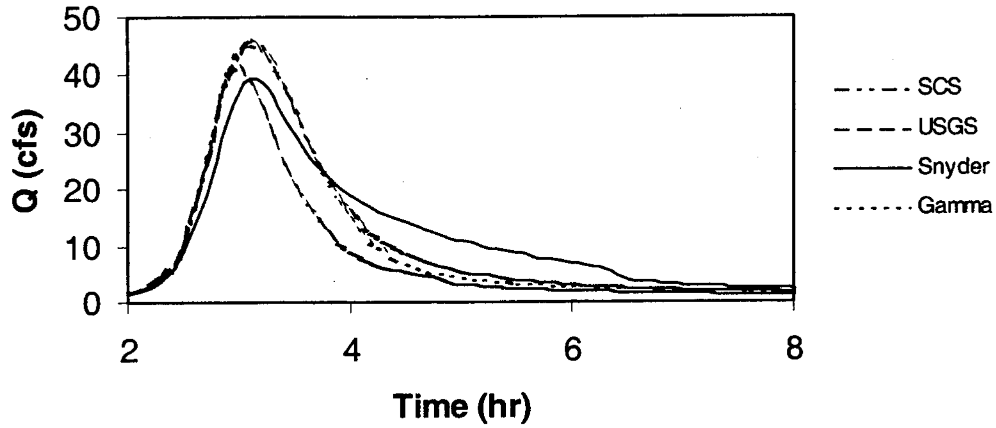
100-yr 2-hr Runoff Hydrographs



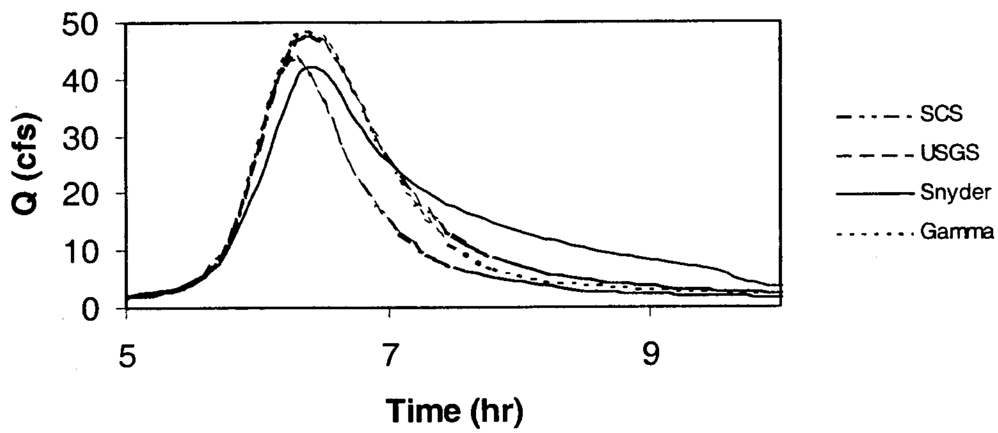
100-yr 3-hr Runoff Hydrographs



100-yr 6-hr Runoff Hydrographs

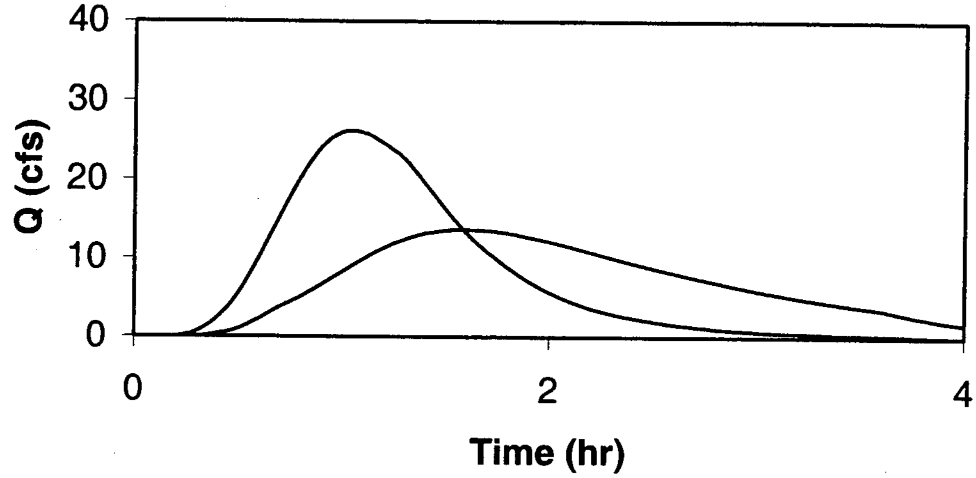


100-yr 12-hr Hydrographs

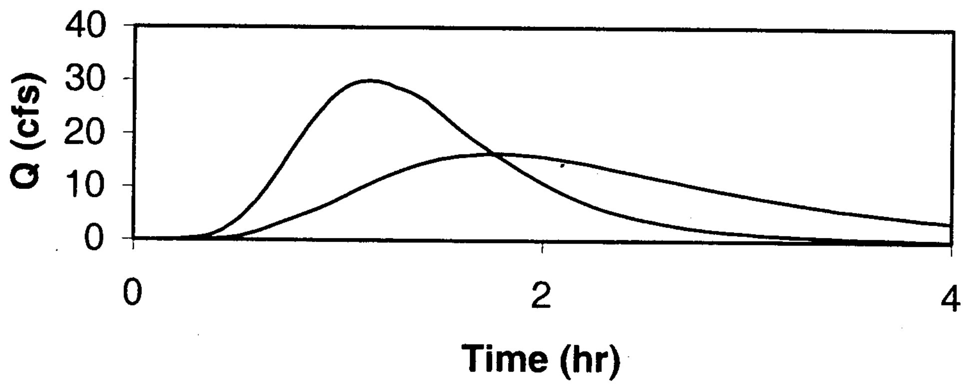


100-yr 24-hr Hydrographs

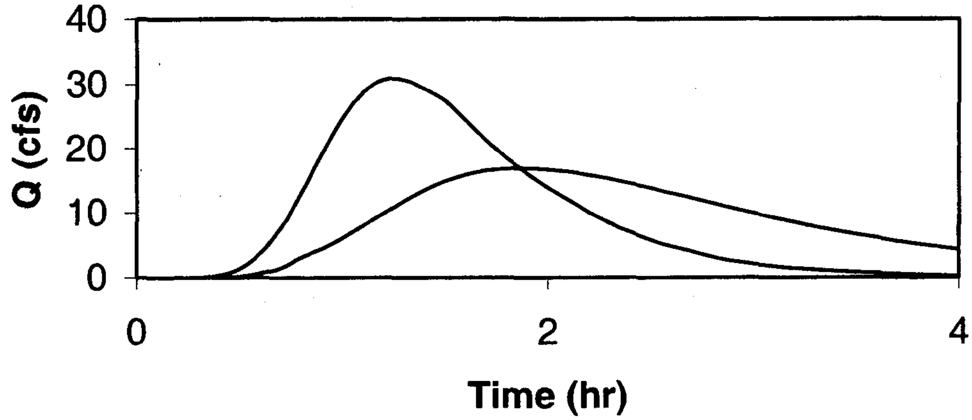
Appendix X: Inflow-Outflow Hydrographs



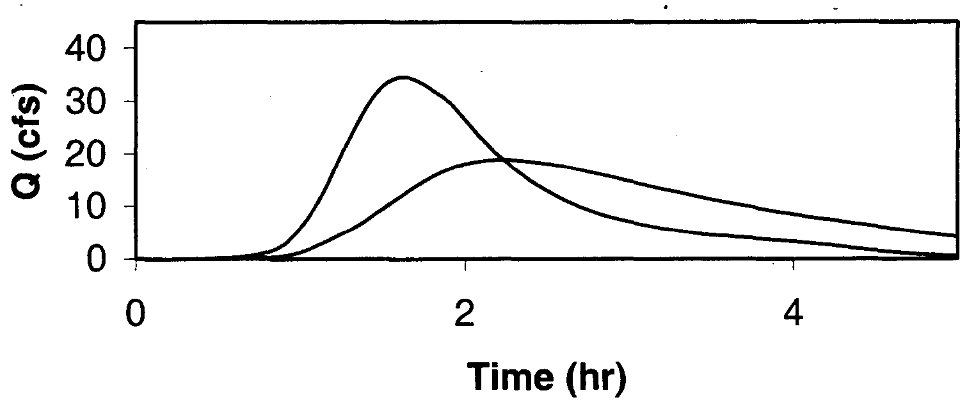
50-yr 1-hr Inflow-Outflow



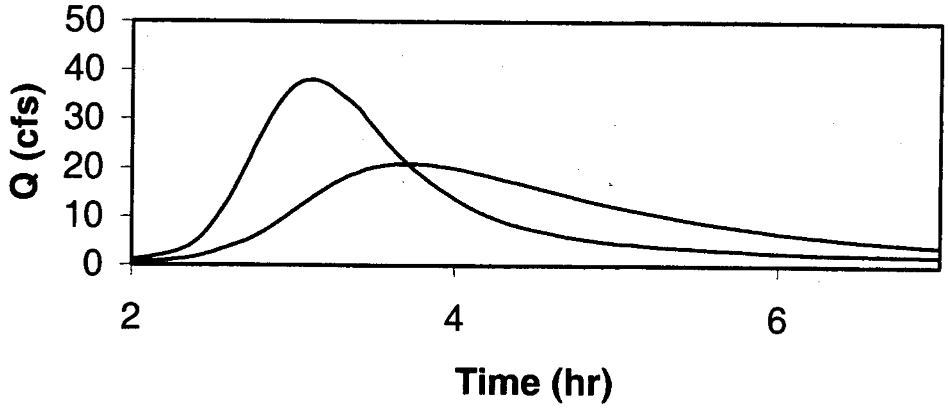
50-yr 2-hr Inflow-Outflow



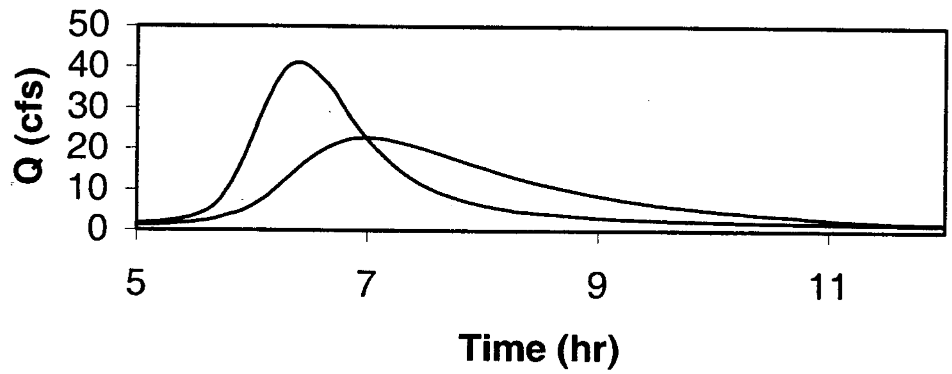
50-yr 3-hr Inflow Outflow



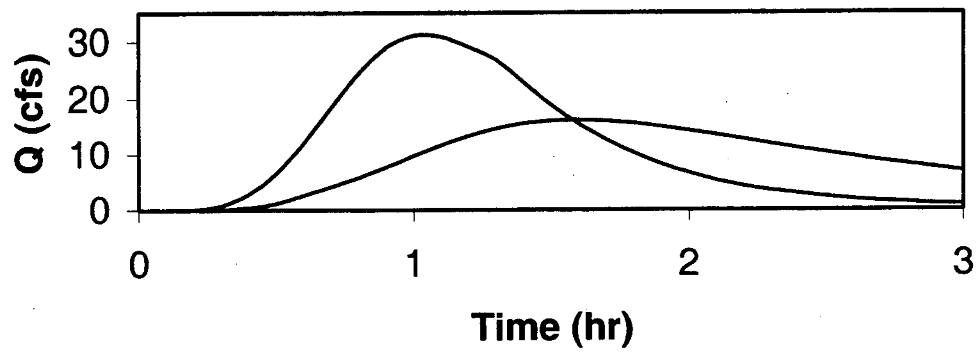
50-yr 6-hr Inflow-Outflow



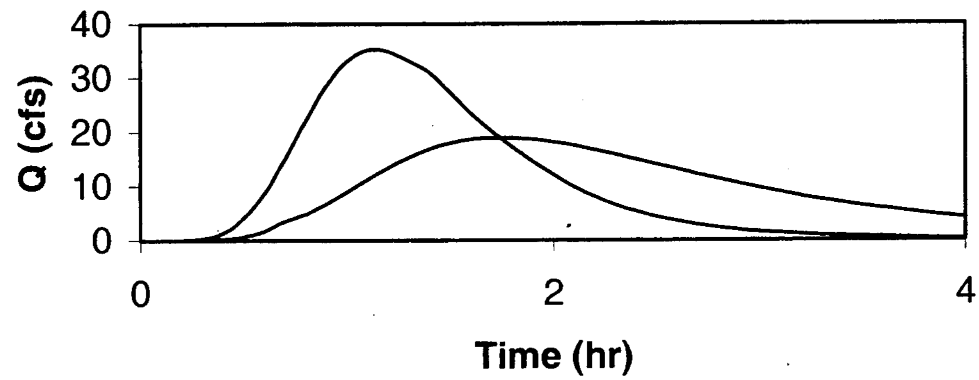
50-yr 12-hr Inflow Outflow



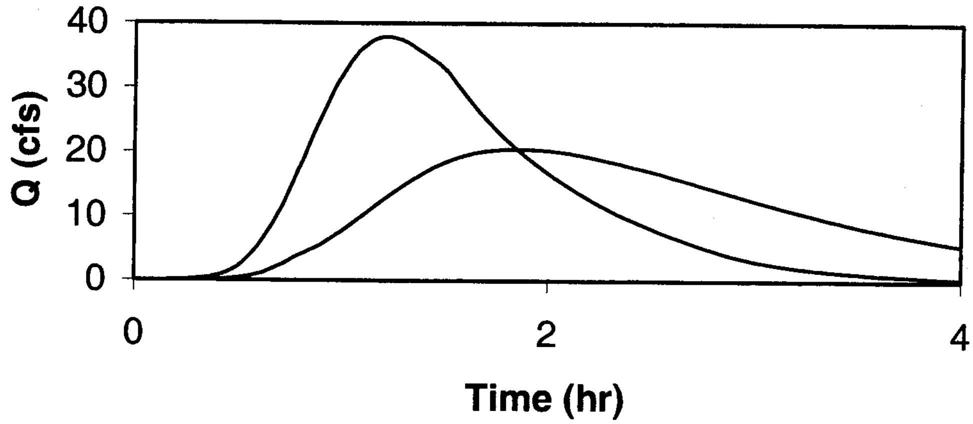
50-yr 24-hr Inflow-Outflow



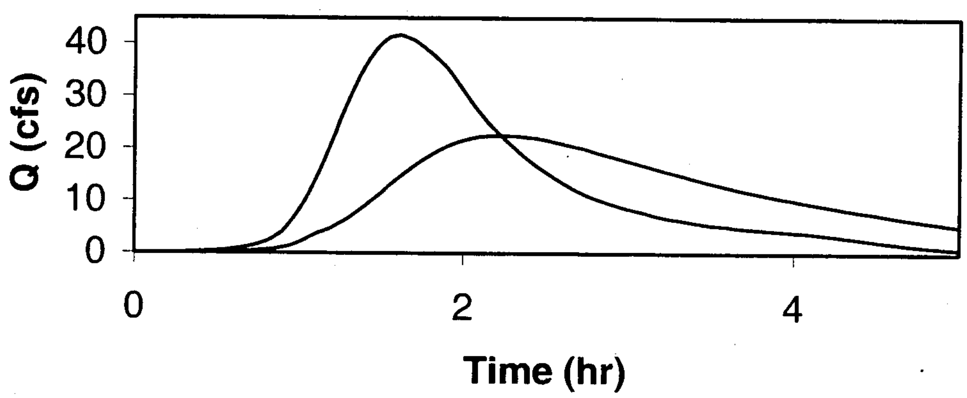
100-yr 1-hr Inflow-Outflow



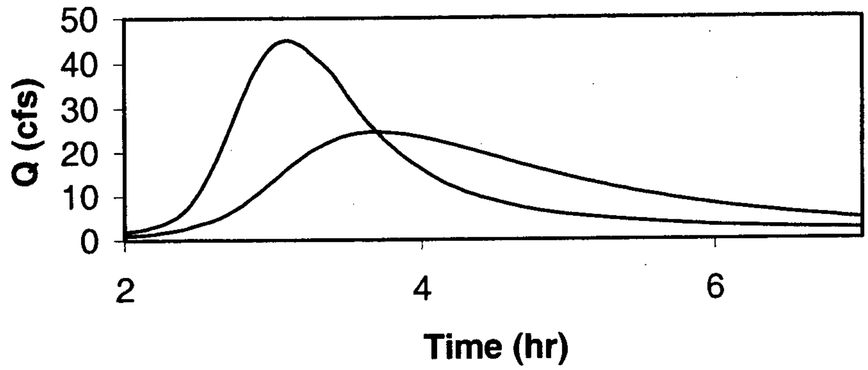
100-yr 2-hr Inflow-Outflow



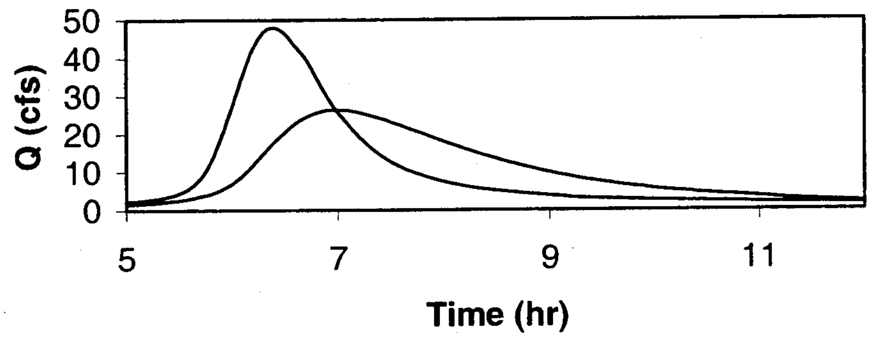
100-yr 3-hr Inflow Outflow



100-yr 6-hr Inflow-Outflow



100-yr 12-hr Inflow Outflow



100-yr 24-hr Inflow-Outflow

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