Virginia Stormwater Management Handbook

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Virginia Department of Conservation and Recreation
Division of Soil and Water Conservation

CHAPTER 4

HYDROLOGIC METHODS

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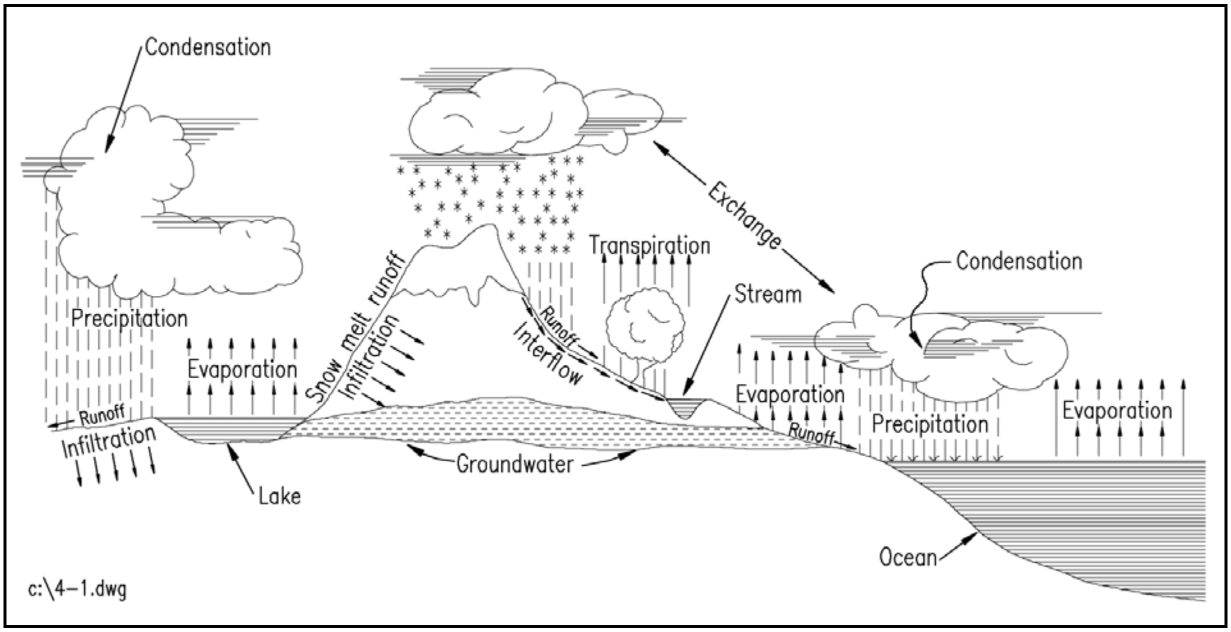
HYDROLOGIC METHODS

4-1 INTRODUCTION

Hydrology is the study of the properties, distribution, and effects of water on the earth's surface, and in the soils, underlying rocks, and atmosphere. The ***hydrologic cycle*** is the closed loop through which water travels as it moves from one phase, or surface, to another.

FIGURE 4 - 1
The Hydrologic Cycle



Source: Federal Highway Administration HEC No. 19

The hydrologic cycle is complex, and to simulate just a small portion of it, such as the relationship between precipitation and surface runoff, can be an inexact science. Many variables and dynamic relationships must be accounted for and, in most cases, reduced to basic assumptions. However, these simplifications and assumptions make it possible to develop solutions to the flooding, erosion, and water quality impacts associated with changes in land cover and hydrologic characteristics.

Proposed engineering solutions typically involve identifying a storm frequency as a benchmark for controlling these impacts. The 2-year, 10-year, and 100-year frequency storms have traditionally been used for hydrologic modeling, followed by an engineered solution designed to offset increased peak flow rates. The hydraulic calculations inherent in this process are dependent upon the designer's ability to predict the amount of rainfall and its intensity. Recognizing that the frequency of a specific rainfall depth or duration is developed from a statistical analysis of historical rainfall data, the designer cannot presume to accurately predict the characteristics of a future storm event.

One could argue that the assumptions in this simulation process undermine the regulatory requirement of mitigating the adverse impacts of development on the hydrologic cycle. However, it is because of these same assumptions and uncertainties that strict adherence to an acceptable methodology is justified. Ongoing efforts to collect and translate data will help to improve the current methodology so that it evolves to more closely simulate the natural hydrologic cycle.

The purpose of this chapter is to provide guidance for preparing acceptable calculations for various elements of the hydrologic and hydraulic analysis of a watershed.

4-2 PRECIPITATION

Precipitation is a random event that cannot be predicted based on historical data. However, any given precipitation event has several distinct and independent characteristics which can be quantified as follows:

Duration - The length of time over which precipitation occurs (hours).
Depth - The amount of precipitation occurring throughout the storm duration (inches).
Frequency - The recurrence interval of events having the same duration <u>and</u> volume.
Intensity - The depth divided by the duration (inches per hour).

A specified amount of rainfall may occur from many different combinations of intensities and durations, as shown in **Table 4-1**. Note that the peak intensity of runoff associated with each combination will vary widely. Also, storm events with the same intensity may have significantly different volumes and durations if the specified storm frequency (2-year, 10-year, 100-year) is different, as shown in **Table 4-2**. It, therefore, becomes critical for any regulatory criteria to specify the volume (or intensity) **and** the duration for a specified frequency design storm. Although specifying one combination of volume and duration may limit the analysis, with regard to what is considered to be the critical variable for any given watershed (erosion, flooding, water quality, etc.), it does establish a baseline from which to work. (This analysis supports the SCS 24-hour design storm since an entire range of storm intensities is incorporated into the rainfall distribution.) Localities may choose to establish criteria based on specific watershed and receiving channel conditions, which will dictate the appropriate design storm. (Refer to **Channel Capacity/Channel Design** in **Chapter 5**, and MS-19 in the Virginia Erosion & Sediment Control Regulations.)

4-2.1 Frequency

The frequency of a specified design storm can be expressed either in terms of ***exceedence probability*** or ***return period***.

Exceedance Probability is the probability that an event having a specified volume and duration will be exceeded in one time period, which is most often assumed to be one year.

Return Period is the average length of time between events having the same volume and duration.

If a storm of a specified duration and volume has a 1 percent chance of occurring in any given year, then it has an exceedence probability of .01 and a return period of 100 years. The return period concept is often misunderstood in that it implies that a 100-year flood will occur only once in a 100-year period. This will not always hold true because storm events cannot be predicted deterministically. Because storm events are random, the exceedence probability indicates that there is a finite probability (.01 for this example) that the 100-year storm may occur in any given year or consecutive years, regardless of the historic occurrence of that storm event.

TABLE 4 - 1
Variations of Duration and Intensity for a Given Volume

Duration (*hr.*)	**Intensity (*in./hr.*)**	**Volume (in.)**
0.5	3.0	1.5
1.0	1.5	1.5
1.5	1.0	1.5
6.0	0.25	1.5

TABLE 4 - 2
Variations of Volume, Duration and Return Frequency for a Given Intensity

Duration (*hr.*)	**Volume (*in.*)**	**Intensity (*in./hr.*)**	**Frequency (*yr.*)**
1.0	1.5	1.5	2
2.0	3.0	1.5	10
3.0	4.5	1.5	100

4-2.2 Intensity-Duration-Frequency Curves

To establish the importance of the relationship between average intensity, duration, and frequency, the U.S. Weather Bureau compiled Intensity-Duration-Frequency (I-D-F) curves based on historic rainfall data for most localities across the country. The rational method uses the I-D-F curves directly, while SCS methods generalize the rainfall data taken from the I-D-F curves and create rainfall distributions for various regions of the country. Selected I-D-F curves for regions of Virginia are provided in the Appendix at the end of this chapter.

There is an ongoing debate concerning which combinations of storm durations and intensities are appropriate to use in a hydrologic analysis for a typical urban development. Working within the limitations of the methodology as described later in this section, small drainage areas (1 to 20 acres) in an urban setting can be accurately modeled using either SCS or rational methods. The belief that the short, very intense storm generates the greatest need for stormwater management often leads designers to use the rational method for stormwater management design, since this method is based on short duration storms. However, the SCS 24-hour storm is also appropriate for short duration storms since it includes short storm intensities within the 24-hour distribution.

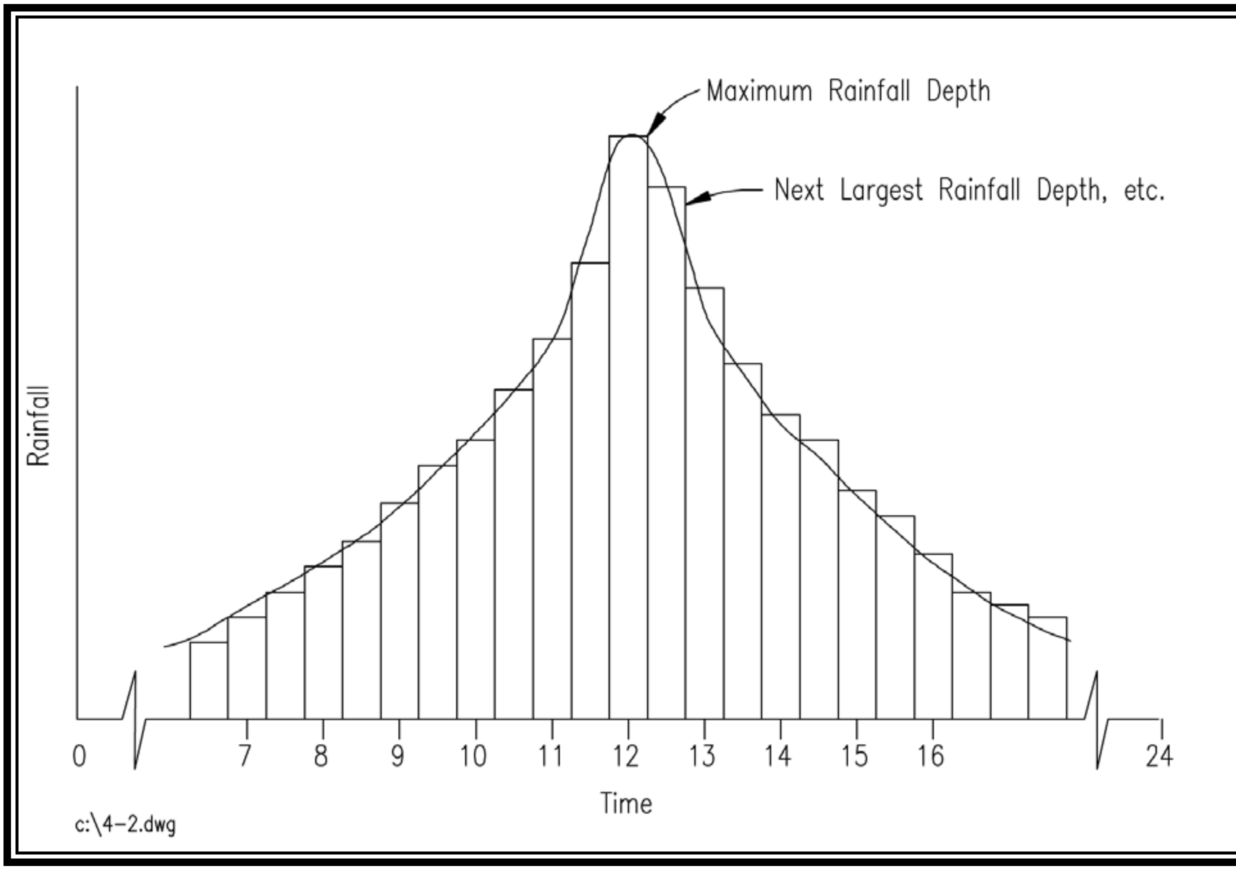
4-2.3 SCS 24-Hour Storm Distribution

The SCS 24-hour storm distribution curve was derived from the National Weather Bureau's Rainfall Frequency Atlases of compiled data for areas less than 400 square miles, for durations up to 24 hours, and for frequencies from 1 to 100 years. Data analysis resulted in four regional distributions: *TYPE I* and *IA* for use in Hawaii, Alaska, and the coastal side of the Sierra Nevada and Cascade Mountains in California, Washington, and Oregon; *TYPE II* distribution for most of the remainder of the United States; and *TYPE III* for the Gulf of Mexico and Atlantic coastal areas. The *TYPE III* distribution represents the potential impact of tropical storms which can produce large 24-hour rainfall amounts. Most of the Commonwealth of Virginia falls under the *TYPE II* distribution, while Virginia Beach is classified as *TYPE III.*

For a more detailed description of the development of dimensionless rainfall distributions, refer to the USDA Soil Conservation Service's National Engineering Handbook, Section 4 (SCS NEH).

The SCS 24-hour storm distributions are based on the generalized rainfall depth-duration-frequency relationships collected for rainfall events lasting from 30 minutes up to 24 hours. Working in 30-minute increments, the rainfall depths are arranged with the maximum rainfall depth assumed to occur in the middle of the 24-hour period. The next largest 30-minute incremental depth occurs just after the maximum depth; the third largest rainfall depth occurs just prior to the maximum depth, etcetera. This continues with each decreasing 30-minute incremental depth until the smaller increments fall at the beginning and end of the 24-hour rainfall (see **Figure 4-2**).

FIGURE 4 - 2
Typical 24-Hour Rainfall Distribution



It is important to note that this process includes all of the critical storm intensities within the 24-hour distributions. The SCS 24-hour storm distributions are, therefore, appropriate for rainfall and runoff modeling for small and large watersheds for the entire range of rainfall depths.

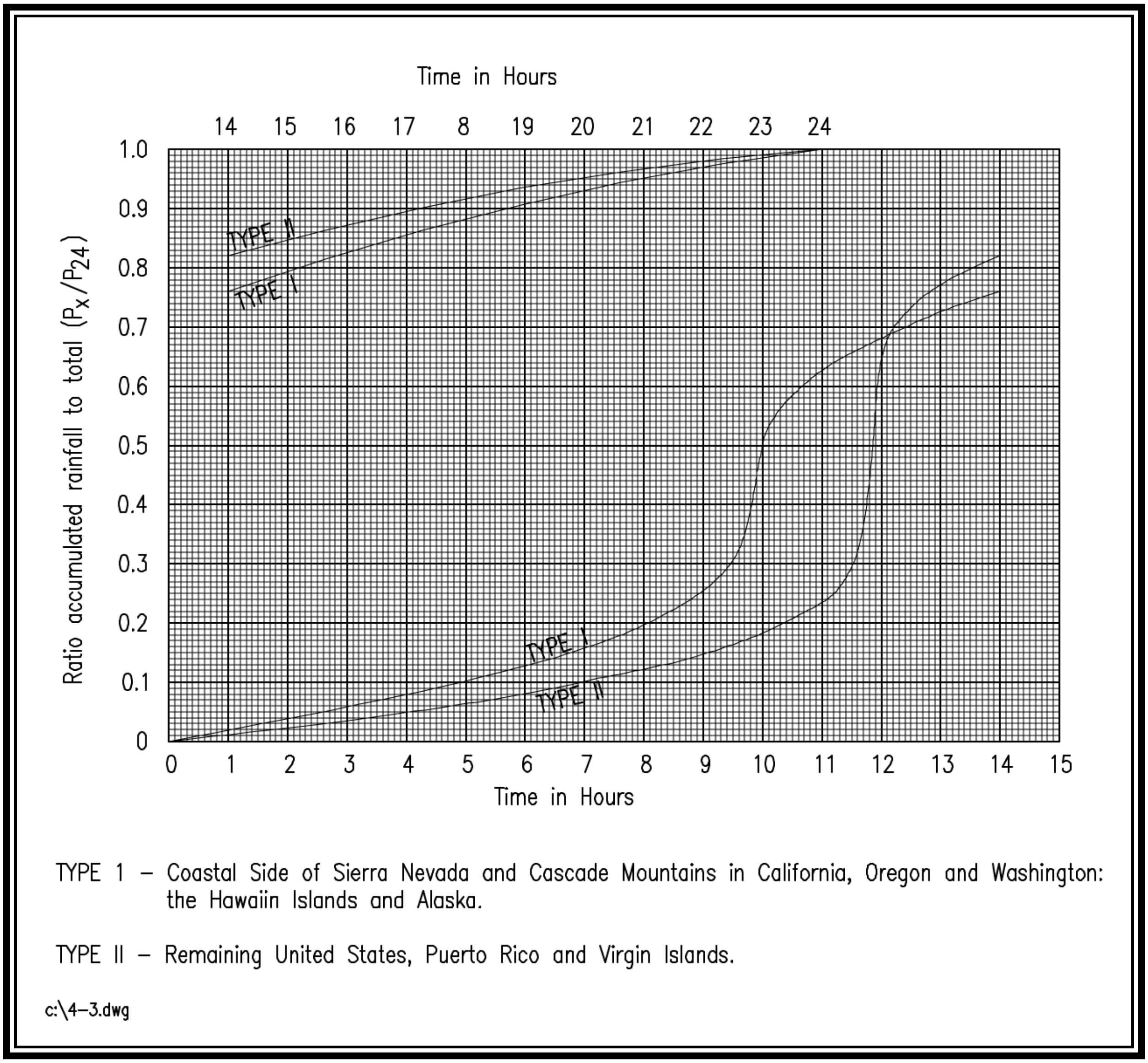
One of the stated disadvantages of using the SCS TR-55 method for hydrologic modeling is its restriction to the use of the 24-hour storm. The following discussion, taken directly from Appendix B of the TR-55 manual (U.S. Department of Agriculture, 1986) addresses this limitation:

> *"To avoid the use of a different set of rainfall intensities for each drainage area's size, a set of synthetic rainfall distributions having "nested" rainfall intensities was developed. The set "maximizes" the rainfall intensities by incorporating selected short-duration intensities within those needed for larger durations at the same probability level.*
>
> *For the size of the drainage areas for which SCS usually provides assistance, a storm period of 24 hours was chosen for the synthetic rainfall distributions. The 24-hour storm, while longer than that needed to determine peaks for these drainage areas, is appropriate for determining runoff volumes. Therefore, a single storm duration and associated synthetic rainfall distribution can be used to*

represent not only the peak discharges but also the runoff volumes for a range of drainage area sizes."

Figure 4-3 shows the SCS 24-hour rainfall distribution, which is a graph of the fraction of total rainfall at any given time, *t*. Note that the peak intensity for the *TYPE II* distribution occurs between time *t = 11.5 hours* and *t = 12.5 hours*.

FIGURE 4 - 3
SCS 24-Hour Rainfall Distribution



Source: USDA SCS

4-2.4 Synthetic Storms

The alternative to a given rainfall "distribution" is to input a custom design storm into the model. This can be compiled from data gathered from a single rainfall event in a particular area, or a synthetic storm created to test the response characteristics of a watershed under specific rainfall conditions. Note, however, that a single historic design storm of known frequency is inadequate for the design of flood control structures, drainage systems, etc. The preferred procedure for such design work is to synthesize data from the longest possible grouping of rainfall data and derive a frequency relationship as described with the I-D-F curves.

4-2.5 Single Event vs. Continuous Simulation Computer Models

The fundamental requirement of a stormwater management plan is a quantitative analysis of the watershed hydrology, hydraulics, and water quality, with consideration for associated facility costs. Computers have greatly reduced the time required to complete such an analysis. Computers have also greatly simplified the statistical analysis of compiled rainfall data.

In general, there are two main categories of hydrologic computer models: *single-event computer models* and *continuous-simulation models*.

Single-event computer models require a minimum of one design-storm hyetograph as input. A ***hyetograph*** is a graph of *rainfall intensity* on the vertical axis versus *time* on the horizontal axis, as shown in **Figure 4-4**. A hyetograph shows the volume of precipitation at any given time as the area beneath the curve, and the time-variation of the intensity.

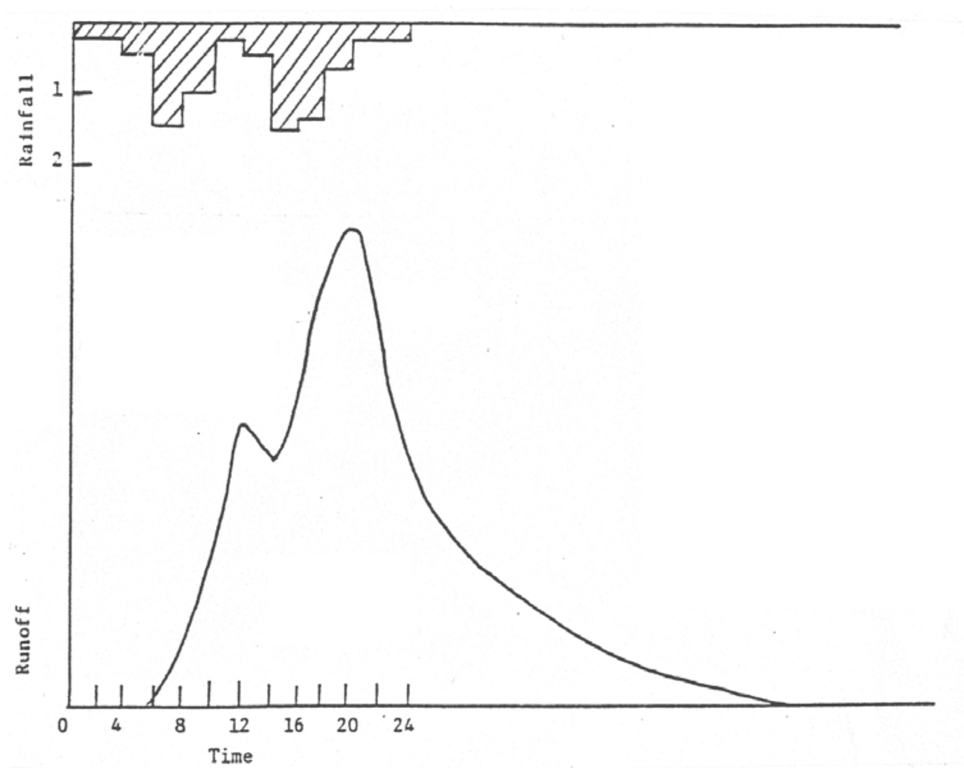
The hyetograph can be a *synthetic hyetograph* or an *historic storm hyetograph*. When a frequency or recurrence interval is specified for the input hyetograph, it is assumed that the resulting output runoff has the same recurrence interval. (This is one of the general assumptions which is made for most single-event models.)

Continuous simulation models, on the other hand, incorporate the entire meteorologic record of a watershed as their input, which may consist of decades of precipitation data. The data is processed by the computer model, producing a continuous runoff hydrograph. The continuous hydrograph output can be analyzed using basic statistical analysis techniques to provide discharge-frequency relationships, volume-frequency relationships, flow-duration relationships, etc. The extent to which the output hydrograph may be analyzed is dependent upon the input data available. The principal advantage of the continuous simulation model is that it eliminates the need to choose a design storm, instead providing long-term response data for a watershed which can then be statistically analyzed for the desired frequency storm.

Computer advances have greatly reduced the analysis time and related expenses associated with continuous models. It can be expected that future models, which combine some features of continuous modeling with the ease of single-event modeling, will offer quick and more accurate analysis procedures.

The hydrologic methods discussed in this handbook are limited to single-event methodologies, based on historic data. Further information regarding the derivation of the I-D-F curves and the SCS 24-hour rainfall distribution can be found in NEH, Section 4 - Hydrology.

FIGURE 4 - 4
Rainfall Hyetograph and Associated Runoff Hydrograph



4-3 RUNOFF HYDROGRAPHS

A ***runoff hydrograph*** is a graphical plot of the runoff or discharge from a watershed with respect to time. Runoff occurring in a watershed flows downstream in various patterns which are influenced by many factors, such as the amount and distribution of the rainfall, rate of snowmelt, stream channel hydraulics, infiltration capacity of the watershed, and others, that are difficult to define. No two flood hydrographs are alike.

Empirical relationships, however, have been developed from which complex hydrographs can be derived. The critical element of the analysis, as with any hydrologic analysis, is the accurate description of the watershed's rainfall-runoff relationship, flow paths, and flow times. From this data, runoff hydrographs can be generated.

This section provides a brief description of some of the types of hydrographs used for modeling watersheds.

Natural hydrographs obtained directly from the flow records of a gauged stream.

Synthetic hydrographs obtained by using watershed parameters and storm characteristics to simulate a natural hydrograph.

Unit hydrographs which are natural or synthetic hydrographs adjusted to represent one inch of direct runoff.

Dimensionless unit hydrographs which are made to represent many unit hydrographs by using the time to peak and the peak rates as basic units and plotting the hydrographs in ratios of these units.

4-3.1 Natural Hydrographs

Extensive watershed gauge data is required to develop a ***natural hydrograph***. Frequently, the data must be interpolated between points in order to provide a complete hydrograph. Stream gauge data is very useful for calibrating models or synthetic hydrographs. However, the lack of such data often eliminates the option of using a natural hydrograph.

4-3.2 Synthetic Hydrographs

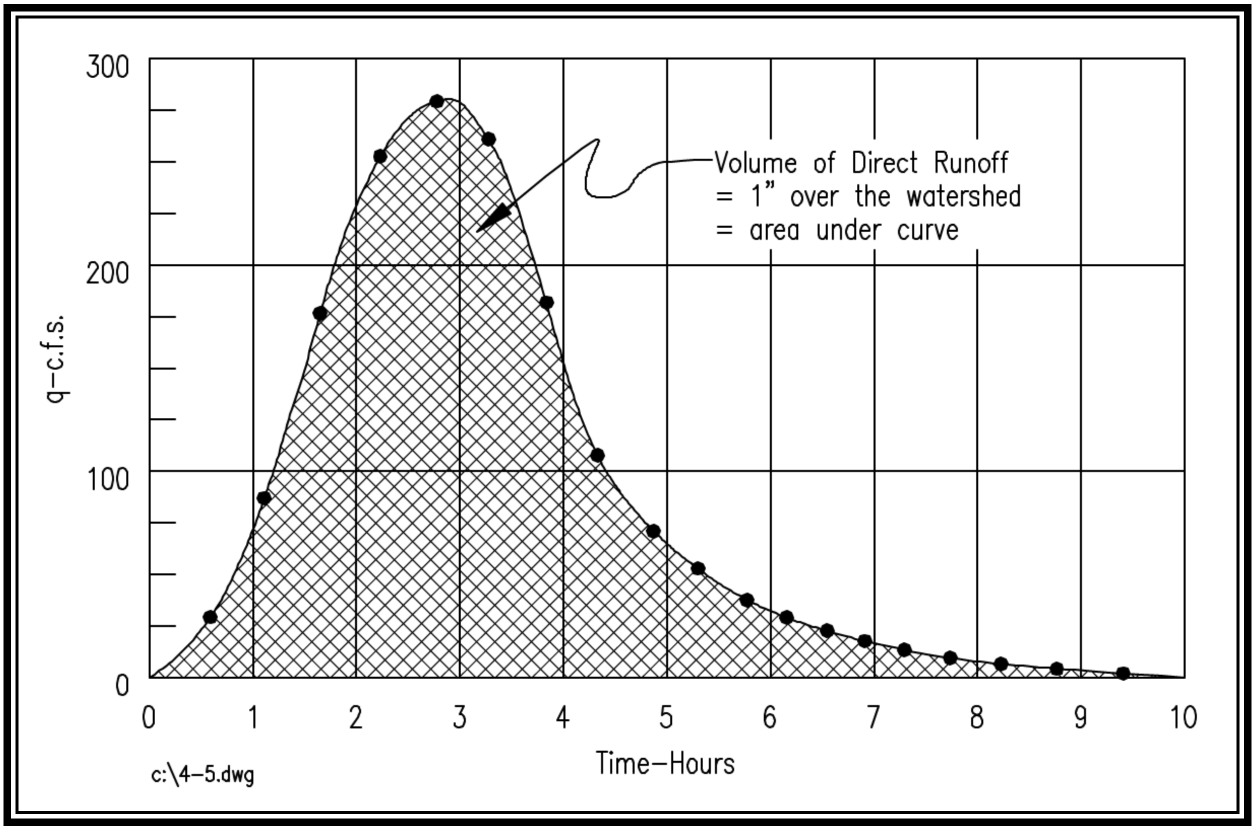
A ***synthetic hydrograph*** is a hydrograph which is generated from the synthesis of data from a large number of watersheds. The basis of a synthetic hydrograph is the establishment of a relationship between the physical geometry of the watersheds and resulting hydrographs. The most commonly used synthetic hydrograph for modeling and design is the ***unit hydrograph***. The following section briefly describes synthetic unit hydrograph methods.

4-3.3 Synthetic Unit Hydrographs

The ***unit hydrograph*** is the hydrograph that results from 1 inch of precipitation excess generated uniformly over the watershed at a uniform rate during a specified time period.

The shape and characteristics of the runoff hydrograph for a given watershed are determined by the specific characteristics of the storm and the physical characteristics of the watershed. Since the physical characteristics of a watershed (shape, slope, ground cover, etc.) are constant, one might expect considerable similarity in the shape of hydrographs from storms of similar rainfall characteristics. This is the essence of the unit hydrograph. The unit hydrograph is a typical hydrograph for a watershed where the runoff volume under the hydrograph is adjusted to equal 1 inch of equivalent depth over the watershed, as shown in **Figure 4-5.**

FIGURE 4 - 5
Typical Synthetic Unit Hydrograph



As mentioned, the unit hydrograph shape is also determined by the storm characteristics, such as rainfall duration, time-intensity patterns, area distribution of rainfall, and depth of rainfall. The following assumptions are made regarding the rainfall-runoff relationship when using a unit hydrograph:

1. *The runoff is from precipitation excess, the difference between precipitation and losses.*

2. *The volume of runoff is **1 inch**, which is equal to the precipitation excess.*

3. *The precipitation excess is applied at a constant/uniform rate.*

4. *The excess is applied with uniform spatial distribution.*

5. *The intensity of rainfall excess is constant over the duration.*

Many of these same assumptions are made when using almost any single-event hydrologic model. These assumptions, however, do not hold true for all storms. Therefore, one can expect variations in the ordinates of the unit hydrograph for different storms.

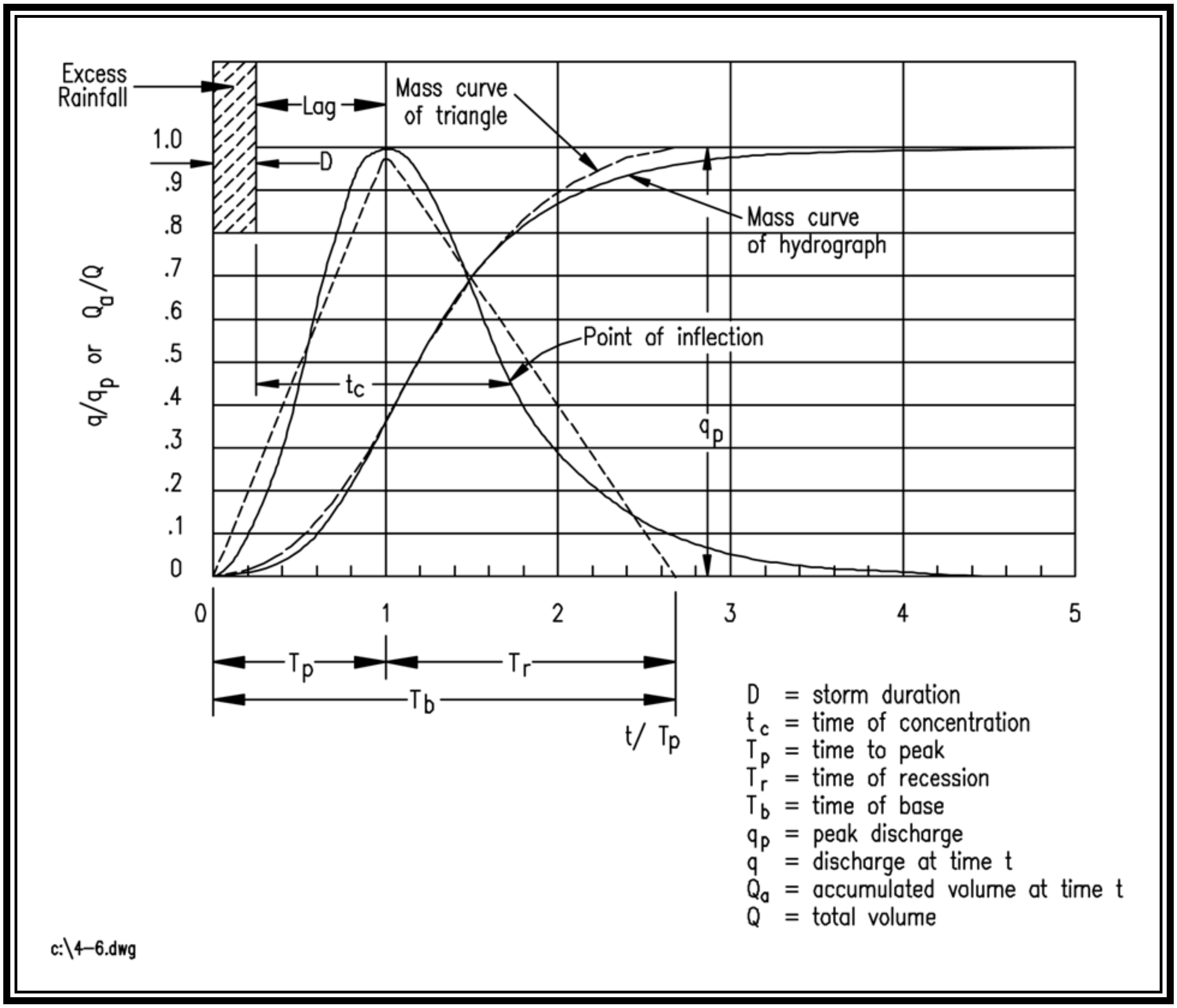
The unit hydrograph does not represent either the total runoff volume or the design hydrograph. The unit hydrograph is simply used to translate the time distribution of precipitation excess into a runoff hydrograph. In other words, the unit hydrograph provides the shape for the actual runoff hydrograph. The physical characteristics of the watershed and the amount of precipitation excess, as determined by the storm event and the rainfall-runoff relationship, will translate the unit hydrograph into the actual runoff hydrograph. The peak discharge and the time to peak are considered to be the defining parameters of the physical characteristics of the watershed. The unit hydrograph is translated into an actual runoff hydrograph through a process called convolution, which takes into account the ***peak*** and ***time to peak***. The convolution process is an exercise in multiplication, translation with time, and addition.

A unit hydrograph can be based on the analysis of a single watershed and can be used specifically for that watershed. This is often the case when conducting flood studies for river basins. Rainfall-runoff and streamflow data compiled within the watershed are analyzed and a unit hydrograph is generated to better predict the response characteristics to various storm events. Generally, however, basic streamflow and runoff data are not available to create a unit hydrograph for most development projects. Therefore, techniques have been developed that allow for the generation of synthetic unit hydrographs.

4-3.4 SCS Dimensionless Unit Hydrograph

The method developed by the Soil Conservation Service (SCS) for constructing synthetic unit hydrographs is based on the ***dimensionless unit hydrograph***. This dimensionless graph is the result of an analysis of a large number of natural unit hydrographs from a wide range of watersheds varying in size and geographic locations. This approach is based on using the watershed peak discharge and time to peak discharge to relate the watershed characteristics to the dimensionless hydrograph features. SCS methodologies provide various empirical equations, as discussed in this chapter, to solve for the ***peak*** and ***time to peak*** for a given watershed. Various equations are then used to define critical points on the hydrograph and thus define the runoff hydrograph. **Figure 4-6** shows the SCS Dimensionless Unit Hydrograph. The critical points are the ***time to peak***, represented by the watershed ***lag time***, and the ***point of inflection***, represented by the ***time of concentration***. The *lag time* of a watershed is the time from the center of mass of excess rainfall to the time to peak of a unit hydrograph. The average relationship of lag, L, to time of concentration, t_c, is $L = 0.6\ t_c$. The reader is encouraged to read Chapters 15 and 16 of the National Engineering Handbook, Section 4; Hydrology, for more information on unit hydrographs.

FIGURE 4 - 6
Dimensionless Curvilinear Unit Hydrograph and Equivalent Triangular Hydrograph



Source: NEH-4, Chapter 16

4-4 RUNOFF and PEAK DISCHARGE

The practice of estimating runoff as a fixed percentage of rainfall has been used in the design of storm drainage systems for over 100 years. Despite its simplification of the complex rainfall - runoff processes, it is still the most commonly used method for urban drainage calculations. It can be accurate when drainage areas are subdivided into homogenious units, and when the designer has enough data and experience to use the appropriate factors..

For watersheds or drainage areas comprised primarily of pervious cover such as open space, woods, lawns, or agricultural land uses, the rainfall/runoff analysis becomes much more complex. Soil conditions and types of vegetation are two of the variables that play a larger role in determining the amount of rainfall which becomes runoff. In addition, other types of flow have a larger effect on stream flow (and measured hydrograph) when the watershed is less urbanized. These are:

1. **Surface runoff** occurs only when the rainfall rate is greater than the infiltration rate and the total volume of rainfall exceeds the interception, infiltration, and surface detention capacity of the watershed. The runoff flows on the land surface collecting in the stream network.

2. **Subsurface flow** occurs when infiltrated rainfall meets an underground zone of low transmission and travels above the zone to the soil surface to appear as a seep or spring.

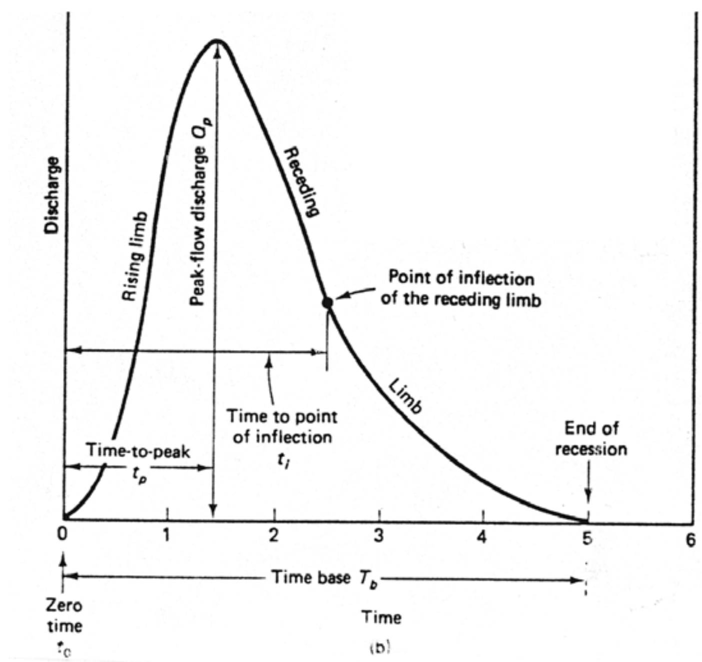
3. **Base flow** occurs when there is a fairly steady flow into a stream channel from natural storage. The flow comes from lakes or swamps, or from an aquifer replenished by infiltrated rainfall or surface runoff.

In watershed hydrology, it is customary to deal separately with base flow and to combine all other types of flow into direct runoff. Depending upon the requirements of the study, the designer can calculate the ***peak flow rate***, in *cfs* (cubic feet per second), of the direct runoff from the watershed, or determine the ***runoff hydrograph*** for the direct runoff from the watershed. A ***hydrograph*** is a plot of *discharge* or *runoff*, on the vertical axis, versus *time*, on the horizontal axis, as shown in **Figure 4-7**. A hydrograph shows the volume of runoff as the area beneath the curve, and the time-variation of the discharge rate.

If the purpose of a hydrologic study is to measure the impact of various developments on the drainage network within a watershed or to design flood control structures, then a hydrograph is needed. If the purpose of a study is to design a roadway culvert or other simple drainage improvement, then only the peak rate of flow is needed. Therefore, the purpose of a given study will dictate the methodology which should be used. **Procedures such as the Rational Method and TR-55 Graphical Peak Discharge Method *do not* generate a runoff hydrograph. The TR-55 Tabular Method and the Modified Rational Method *do* generate runoff hydrographs**.

This section will present some of the different methods for calculating runoff from a watershed. Designers should be familiar with all of them since they require different types of input and generate different types of results.

FIGURE 4 - 7
Runoff Hydrograph



The methods covered here are: The Rational Method, Modified Rational Method, and SCS Methods' TR-55, Urban Hydrology for Small Watersheds (USDA 1986): Graphical Peak Discharge and Tabular Hydrograph Methods. Many computer programs are available which develop these methodologies, utilizing the rainfall-runoff relationship described previously. Many of these programs also "route" the runoff hydrograph through a stormwater management facility, calculating the peak rate of discharge and a discharge hydrograph.

Examples provided in Chapter 6 utilize SCS TR-20 "Project Formulation, Hydrology (USDA 1982). Other readily available computer programs also utilize SCS Methods. Additional examples utilizing many different computer programs which offer a variety of hydrologic methods will be provided through DCR as ongoing guidance. The accuracy of the computer model is based upon the accuracy of the input which is typically generated through the Rational or SCS methodologies covered here. The designer should be familiar with all of the methods covered here since any one may be appropriate for the specific site on watershed being modeled.

All the methods presented here make assumptions and have limitations on the accuracy. Simply put, however, when these methods are used correctly, they will all provide a reasonable estimate of the peak rate of runoff from a drainage area or watershed.

It should be noted that for small storm events (< 2" rainfall) TR-55 tends to underestimate the runoff, while it has been shown to be fairly accurate for larger storm events (Pitt, 1994). Similarly, the Rational formula has been found to be fairly accurate on smaller homogeneous watersheds, while tending to lose accuracy in the larger more complex watersheds. The following discussion provides further explanation of these methods, including assumptions, limitations, and information needed for the analysis.

4-4.1 The Rational Method

The Rational Method was introduced in 1880 for determining peak discharges from drainage areas. It is frequently criticized for its simplistic approach, but this same simplicity has made the Rational Method one of the most widely used techniques today.

The Rational Formula estimates the peak rate of runoff at any location in a drainage area as a function of the *runoff coefficient*, *mean rainfall intensity*, and *drainage area*. The ***Rational Formula*** is expressed as follows:

$$Q = C\,I\,A$$

Equation 4-1
Rational Formula

where:
Q = *maximum rate of runoff, cfs*
C = *dimensionless runoff coefficient, dependent upon land use*
I = *design rainfall intensity, in inches per hour, for a duration equal to the time of concentration of the watershed*
A = *drainage area, in acres*

4-4.1.1 Assumptions

The Rational Method is based on the following assumptions:

1) *Under steady rainfall intensity, the maximum discharge will occur at the watershed outlet at the time when the entire area above the outlet is contributing runoff.*

This "time" is commonly known as the ***time of concentration***, t_c, and is defined as the time required for runoff to travel from the most hydrologically distant point in the watershed to the outlet.

The assumption of steady rainfall dictates that even during longer events, when factors such as increasing soil saturation are ignored, the ***maximum discharge*** occurs when the entire watershed is contributing to the peak flow, at time $t = t_c$.

Furthermore, this assumption limits the size of the drainage area that can be analyzed using the rational method. In large watersheds, the time of concentration may be so long that constant rainfall intensities may not occur for long periods. Also, shorter, more intense bursts of rainfall that occur over portions of the watershed may produce large peak flows.

2) *The time of concentration is equal to the minimum duration of peak rainfall.*

The time of concentration reflects the minimum time required for the entire watershed to contribute to the peak discharge as stated above. The rational method assumes that the discharge does not increase as a result of soil saturation, decreased conveyance time, etc. (refer to **Figure 4-8**). Therefore, the time of concentration is not necessarily intended to be a measure of the actual storm duration, but simply the critical time period used to determine the average rainfall intensity from the Intensity-Duration-Frequency curves.

3) *The frequency or return period of the computed peak discharge is the same as the frequency or return period of rainfall intensity (design storm) for the given time of concentration.*

Frequencies of peak discharges depend not only on the frequency of rainfall intensity, but also the response characteristics of the watershed. For small and mostly impervious areas, rainfall frequency is the dominant factor since response characteristics are relatively constant. However, for larger watersheds, the response characteristics will have a much greater impact on the frequency of the peak discharge due to drainage structures, restrictions within the watershed, and initial rainfall losses from interception and depression storage.

4) The fraction of rainfall that becomes runoff is independent of rainfall intensity or volume.

This assumption is reasonable for impervious areas, such as streets, rooftops, and parking lots. For pervious areas, the fraction of rainfall that becomes runoff varies with rainfall intensity and the accumulated volume of rainfall. As the soil becomes saturated, the fraction of rainfall that becomes runoff will increase. This fraction is represented by the dimensionless runoff coefficient, C. Therefore, the accuracy of the rational method is dependent on the careful selection of a coefficient that is appropriate for the storm, soil, and land use conditions. Selection of appropriate C values will be discussed later in this chapter.

It is easy to see why the rational method becomes more accurate as the percentage of impervious cover in the drainage area approaches 100 percent.

5) *The peak rate of runoff is sufficient information for the design of stormwater detention and retention facilities.*

4-4.1.2 Limitations

Because of the assumptions discussed above, the rational method should only be used when the following criteria are met:

1) *The given watershed has a time of concentration, t_c, less than 20 minutes;*

2) *The drainage area is less than 20 acres.*

For larger watersheds, attenuation of peak flows through the drainage network begins to be a factor in determining peak discharge. While there are ways to adjust runoff coefficients (CN factors) to account for the attenuation, or routing effects, it is better to use a hydrograph method or computer simulation for these more complex situations.

Similarly, the presence of bridges, culverts, or storm sewers may act as restrictions which ultimately impact the peak rate of discharge from the watershed. The peak discharge upstream of the restriction can be calculated using a simple calculation procedure, such as the Rational Method, however a detailed storage routing procedure which considers the storage volume above the restriction should be used to accurately determine the discharge downstream of the restriction.

4-4.1.3 Design Parameters

The following is a brief summary of the design parameters used in the rational method:

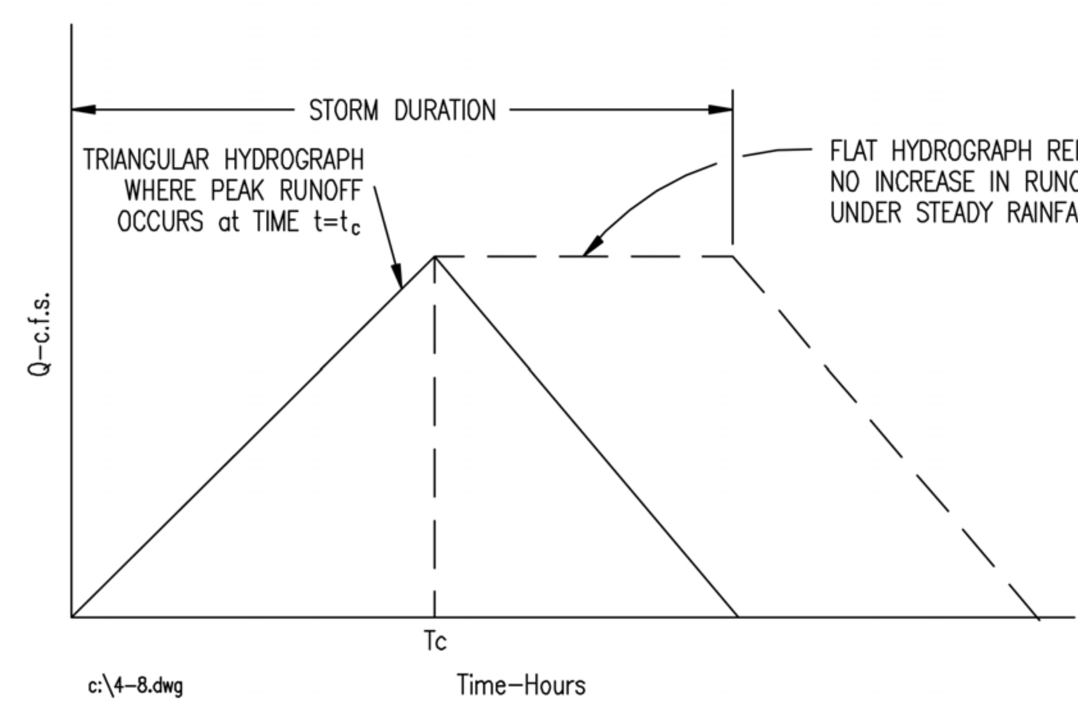
1) Time of concentration, t_c

The most consistent source of error in the use of the rational method is the oversimplification of the time of concentration calculation procedure. Since the origin of the rational method is rooted in the design of culverts and conveyance systems, the main components of the time of concentration are *inlet time* (or overland flow) and *pipe or channel flow time*. The *inlet or overland flow time* is defined as the time required for runoff to flow overland from the furthest point in the drainage area over the surface to the inlet or culvert. The *pipe or channel flow time* is defined as the time required for the runoff to flow through the conveyance system to the design point. In addition, when an inlet time of less than 5 minutes is encountered, the time is rounded up to 5 minutes, which is then used to determine the rainfall intensity, *I*, for that inlet.

Variations in the time of concentration can impact the calculated peak discharge. When the procedure for calculating the time of concentration is oversimplified, as mentioned above, the accuracy of the Rational Method is greatly compromised. To prevent this oversimplification, it is recommended that a more rigorous procedure for determining the time of concentration be used, such as those outlined in **Section 4-4.3.2** of this manual, Chapter 5 of the Virginia Erosion and Sediment Control Handbook (VESCH), 1992 edition, Chapter 15, Section 4 of SCS National Engineering Handbook, or the Virginia Department of Transportation (VDOT) drainage manual.

There are many procedures for estimating the time of concentration. Some were developed with a specific type or size watershed in mind, while others were based on studies of a specific watershed. The selection of any given procedure should include a comparison of the hydrologic and hydraulic characteristics used in the formation of the procedure, versus the characteristics of the watershed

FIGURE 4 - 8
Rational Method Runoff Hydrograph



under study. The designer should be aware that if two or more methods of determining time of concentration are applied to a given watershed, there will likely be a wide range in results. The SCS method is recommended because it provides a means of estimating overland sheet flow time and shallow concentrated flow time as a function of readily available parameters such as land slope and land surface conditions. Regardless of which method is used, the result should be reasonable when compared to an average flow time over the total length of the watershed.

2) Rainfall Intensity, I

The rainfall intensity, *I*, is the average rainfall rate, in inches per hour, for a storm duration equal to the time of concentration for a selected return period (i.e., 1-year, 2-year, 10-year, 25-year, etc.). Once a particular return period has been selected, and the time of concentration has been determined for the drainage area, the rainfall intensity can be read from the appropriate rainfall Intensity-

Duration-Frequency (I-D-F) curve for the geographic area in which the drainage area is located. These charts were developed from data furnished by the National Weather Service for regions of Virginia, and are provided in the Appendix at the end of this chapter.

3) Runoff Coefficient, C

The runoff coefficients for different land uses within a watershed are used to generate a single, weighted coefficient that will represent the relationship between rainfall and runoff for that watershed. Recommended values can be found in **Table 4-3**. In an attempt to make the rational method more accurate, efforts have been made to adjust the runoff coefficients to represent the integrated effects of drainage basin parameters: *land use*, *soil type*, and *average land slope*. **Table 4-3** provides recommended coefficients based on urban land use only, while **Table 4-5** gives recommended coefficients for various land uses based on soil type and land slope parameters.

A good understanding of these parameters is essential in choosing an appropriate coefficient. As the slope of a drainage basin increases, runoff velocities increase for both sheet flow and shallow concentrated flow. As the velocity of runoff increases, the ability of the surface soil to absorb the runoff decreases. This decrease in infiltration results in an increase in runoff. In this case, the designer should select a higher runoff coefficient to reflect the increase due to slope.

Soil properties influence the relationship between runoff and rainfall even further since soils have differing rates of infiltration. Historically, the Rational Method was used primarily for the design of storm sewers and culverts in urbanizing areas; soil characteristics were not considered, especially when the watershed was largely impervious. In such cases, a conservative design simply meant a larger pipe and less headwater. For stormwater management purposes, however, the existing condition (prior to development, usually with large amounts of pervious surfaces) often dictates the allowable post-development release rate, and therefore, must be accurately modeled.

Soil properties can change throughout the construction process due to compaction, cut, and fill operations. If these changes are not reflected in the runoff coefficient, the accuracy of the model will decrease. Some localities arbitrarily require an adjustment in the runoff coefficient for pervious surfaces due to the effects of construction on soil infiltration capacities. This is discussed in more detail in **Section 4-4.3** of this handbook. Such an adjustment is not possible using the Rational Method since soil conditions are not considered. However, **Table 4-5** attempts to provide a graduated scale which correlates the rational method runoff coefficient with soil and land condition characteristics.

4) Adjustment for Infrequent Storms

The Rational Method has undergone further adjustment to account for infrequent, higher intensity storms. This adjustment is in the form of a frequency factor, C_f, which accounts for the reduced impact of infiltration and other effects on the amount of runoff during larger storms. With the adjustment, the Rational Formula is expressed as follows:

$$Q = C\,C_f\,I\,A$$

Equation 4-2
Rational Formula Frequency Factor

The C_f values are listed in **Table 4-4**. The product of $C_f \times C$ should not exceed 1.0.

TABLE 4 - 3
Rational Equation Runoff Coefficients

Land use	"*C*" Value
Business, industrial and commercial	0.90
Apartments	0.75
Schools	0.60
Residential - lots of 10,000 *sq. ft.*	0.50
- lots of 12,000 *sq. ft.*	0.45
- lots of 17,000 *sq. ft.*	0.45
- lots of ½ acre or more	0.40
Parks, cemeteries and unimproved areas	0.34
Paved and roof areas	0.90
Cultivated areas	0.60
Pasture	0.45
Forest	0.30
Steep grass slopes (2:1)	0.70
Shoulder and ditch areas	0.50
Lawns	0.20

Source:VDOT

TABLE 4 - 4
Rational Equation Frequency Factors

C_f	Storm Return Frequency
1.0	10 *yr.* or less
1.1	25 *yr.*
1.2	50 *yr.*
1.25	100 *yr.*

Source: VDOT

TABLE 4 - 5a

Rational Equation Coefficients for SCS Hydrologic Soil Groups (A, B, C, D)

Urban Land Uses

STORM FREQUENCIES OF LESS THAN 25 YEARS													
Land Use	Hydrologic Condition	HYDROLOGIC SOIL GROUP/SLOPE											
		A			B			C			D		
		0-2%	2-6%	6%+	0-2%	2-6%	6%+	0-2%	2-6%	6%+	0-2%	2-6%	6%+
Paved Areas and Impervious Surfaces		0.90	0.90	0.90	0.90	0.90	0.90	0.90	0.90	0.90	0.90	0.90	0.90
Open Space, Lawns, etc.	Good	0.08	0.12	0.15	0.11	0.16	0.21	0.14	0.19	0.24	0.20	0.24	0.28
Industrial		0.67	0.68	0.68	0.68	0.68	0.69	0.68	0.69	0.69	0.69	0.69	0.70
Commercial		0.71	0.71	0.72	0.71	0.72	0.72	0.72	0.72	0.72	0.72	0.72	0.72
Residential Lot Size 1/8 Acre		0.25	0.28	0.31	0.27	0.30	0.35	0.30	0.33	0.38	0.33	0.36	0.42
Lot Size 1/4 Acre		0.22	0.26	0.29	0.24	0.29	0.33	0.27	0.31	0.36	0.30	0.34	0.40
Lot Size 1/3 Acre		0.19	0.23	0.26	0.22	0.26	0.30	0.25	0.29	0.34	0.28	0.32	0.39
Lot Size ½ Acre		0.16	0.20	0.24	0.19	0.23	0.28	0.22	0.27	0.32	0.26	0.30	0.37
Lot Size 1.0 Acre		0.14	0.19	0.22	0.17	0.21	0.26	0.20	0.25	0.31	0.24	0.29	0.35

Source: Maryland State Highway Administration

TABLE 4 - 5b
Rational Equation Coefficients for SCS Hydrologic Soil Groups (A, B, C, D)
Rural Land Uses

STORM FREQUENCIES OF LESS THAN 25 YEARS														
Land Use	Treatment / Practice	Hydrologic Condition	HYDROLOGIC SOIL GROUP/SLOPE											
			A			B			C			D		
			0-2%	2-6%	6%+	0-2%	2-6%	6%+	0-2%	2-6%	6%+	0-2%	2-6%	6%+
Pasture or Range		Good	0.07	0.09	0.10	0.18	0.20	0.22	0.27	0.29	0.31	0.32	0.34	0.35
	Contoured	Good	0.03	0.04	0.06	0.11	0.12	0.14	0.24	0.26	0.28	0.31	0.33	0.34
Meadow			0.06	0.08	0.10	0.10	0.14	0.19	0.12	0.17	0.22	0.15	0.20	0.25
Wooded		Good	0.05	0.07	0.08	0.08	0.11	0.15	0.10	0.13	0.17	0.12	0.15	0.21

Source: Maryland State Highway Administration

TABLE 4 - 5c

Rational Equation Coefficients for SCS Hydrologic Soil Groups (A, B, C, D)

Agricultural Land Uses

STORM FREQUENCIES OF LESS THAN 25 YEARS														
Land Use	Treatment/ Practice	Hydrologic Condition	HYDROLOGIC SOIL GROUP/SLOPE											
			A			B			C			D		
			0-2%	2-6%	6%+	0-2%	2-6%	6%+	0-2%	2-6%	6%+	0-2%	2-6%	6%+
Fallow	Straight Row		0.41	0.48	0.53	0.60	0.66	0.71	0.72	0.78	0.82	0.84	0.88	0.91
Row Crops	Straight Row	Good	0.24	0.30	0.35	0.43	0.48	0.52	0.61	0.65	0.68	0.73	0.76	0.78
	Contoured	Good	0.21	0.26	0.30	0.41	0.45	0.49	0.55	0.59	0.63	0.63	0.66	0.68
	Contoured and Terraced	Good	0.20	0.24	0.27	0.31	0.35	0.39	0.45	0.48	0.51	0.55	0.58	0.60
Small Grain	Straight Row	Good	0.23	0.26	0.29	0.42	0.45	0.48	0.57	0.60	0.62	0.71	0.73	0.75

Source: Maryland State Highway Administration

TABLE 4 - 5d

Rational Equation Coefficients for SCS Hydrologic Soil Groups (A, B, C, D)

Agricultural Land Uses

STORM FREQUENCIES OF LESS THAN 25 YEARS														
Land Use	Treatment/ Practice	Hydrologic Condition	HYDROLOGIC SOIL GROUP/SLOPE											
			A			B			C			D		
			0-2%	2-6%	6%+	0-2%	2-6%	6%+	0-2%	2-6%	6%+	0-2%	2-6%	6%+
Small Grain	Contoured	Good	0.17	0.22	0.27	0.33	0.38	0.42	0.54	0.58	0.61	0.62	0.65	0.67
	Contoured and Terraced	Good	0.16	0.20	0.24	0.31	0.35	0.38	0.45	0.48	0.50	0.55	0.58	0.60
Closed-seeded Legumes or Rotation Meadow	Straight Row	Good	0.15	0.19	0.23	0.31	0.35	0.38	0.55	0.58	0.60	0.63	0.65	0.66
	Contoured	Good	0.14	0.18	0.21	0.30	0.34	0.37	0.45	0.48	0.51	0.58	0.60	0.61
	Contoured and Terraced	Good	0.07	0.10	0.13	0.28	0.32	0.35	0.44	0.47	0.49	0.52	0.54	0.56

Source: Maryland State Highway Administration

4-4.2 Modified Rational Method

The modified rational method is a variation of the rational method, developed mainly for the sizing of detention facilities in urban areas. The modified rational method is applied similarly to the rational method except that it utilizes a fixed rainfall duration. The selected rainfall duration depends on the requirements of the user. For example, the designer might perform an iterative calculation to determine the rainfall duration which produces the maximum storage volume requirement when sizing a detention basin. This procedure will be discussed later in **Chapter 5**, **Hydraulic Calculations**.

4-4.2.1 Assumptions

The modified rational method is based on the following assumptions:

1. *All of the assumptions used with the rational method apply. The most significant difference is that the time of concentration for the modified rational method is equal to the rainfall intensity averaging period rather than the actual storm duration.*

This assumption means that any rainfall, or any runoff generated by the rainfall, that occurs before or after the ***rainfall averaging period*** is unaccounted for. **Thus, when used as a basin sizing procedure, the modified rational method may seriously underestimate the required storage volume**. (Walesh, 1989)

2) *The runoff hydrograph for a watershed can be approximated as triangular or trapezoidal in shape.*

This assumption implies a linear relationship between ***peak discharge*** and ***time*** for any and all watersheds.

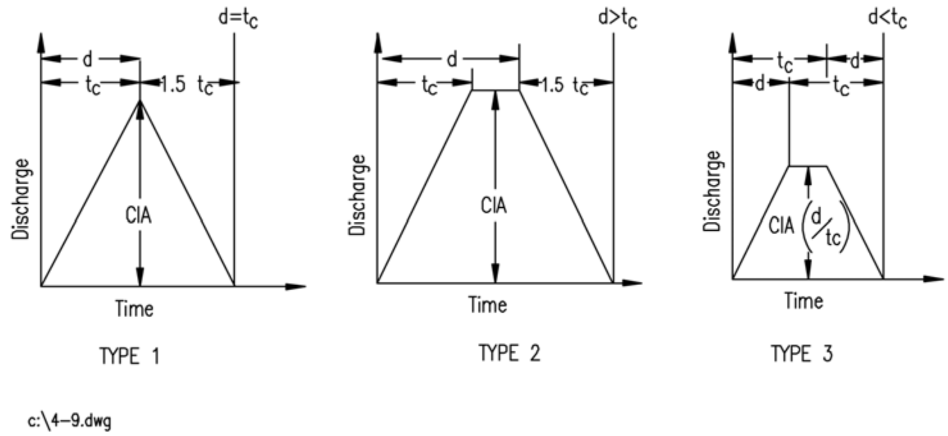
4-4.2.2 Limitations

All of the limitations listed for the rational method apply to the modified rational method. The key difference is the assumed shape of the resulting runoff hydrograph. The rational method produces a triangular shaped hydrograph, while the modified rational method can generate triangular or trapezoidal hydrographs for a given watershed, as shown in **Figure 4-9**.

4-4.2.3 Design Parameters

The equation $Q = C\,I\,A$ (the rational equation) is used to calculate the peak discharge for all three hydrographs shown in **Figure 4-9**. Notice that the only difference between the rational method and the modified rational method is the incorporation of the *storm duration*, d, into the modified rational method to generate a *volume* of runoff in addition to the peak discharge.

FIGURE 4 - 9
Modified Rational Method Runoff Hydrographs



Type 1 - Storm duration, *d*, is equal to the time of concentration, t_c.
Type 2 - Storm duration, *d*, is greater than the time of concentration, t_c.
Type 3 - Storm duration, *d*, is less than the time of concentration, t_c.

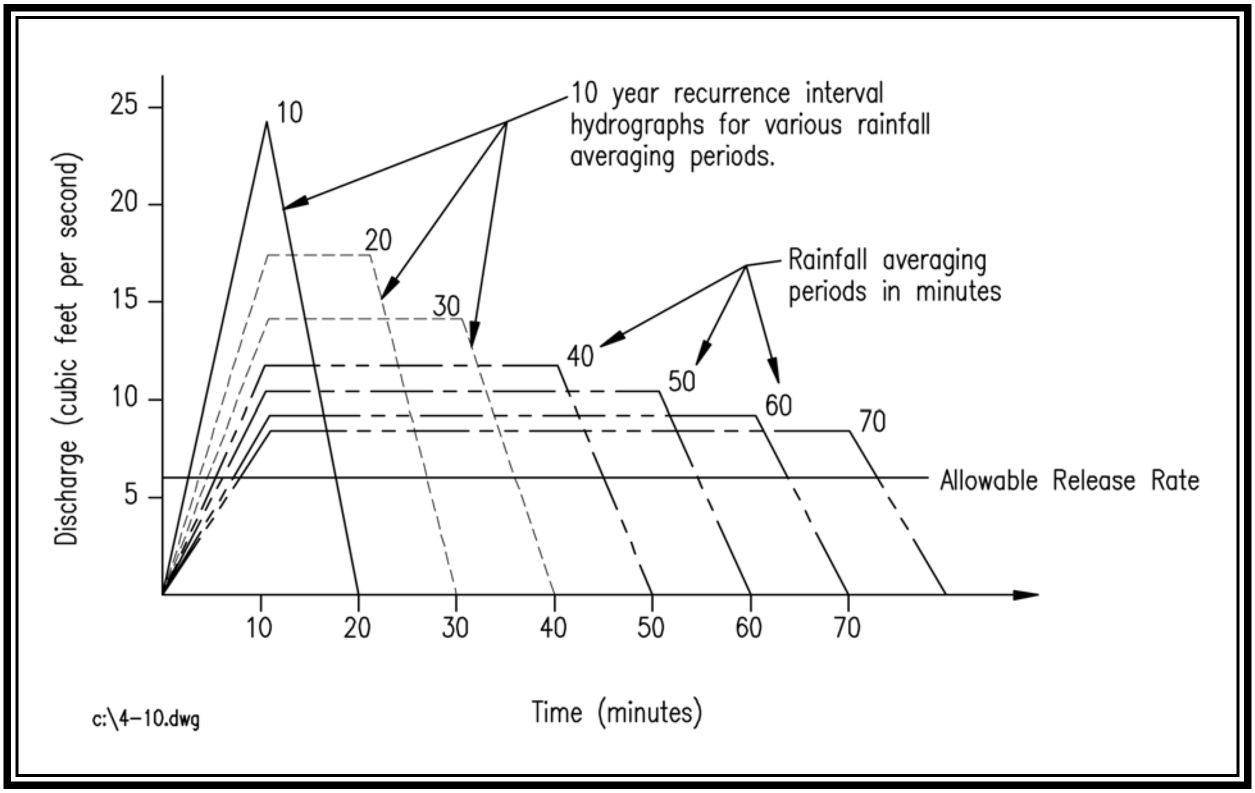
Source: Urban Surface Water Management, Walesh, Stuart G.

The rational method generates the peak discharge that occurs when the entire watershed is contributing to the peak (at a time $t = t_c$) and ignores the effects of a storm which lasts longer than time *t*. The modified rational method, however, considers storms with a longer duration than the watershed t_c, which may have a smaller or larger peak rate of discharge, but will produce a greater volume of runoff (area under the hydrograph) associated with the longer duration of rainfall. **Figure 4-10** shows a family of hydrographs representing storms of different durations. The storm duration which generates the greatest volume of runoff may not necessarily produce the greatest peak rate of discharge.

Note that the duration of the receding limb of the hydrograph is set to equal the time of concentration, t_c , or 1.5 times t_c . The direct solution, which will be discussed in **Chapter 5**, uses $1.5t_c$ as the receding limb. This is justified since it is more representative of actual storm and runoff dynamics. (It is also more similar to the SCS unit hydrograph where the receding limb extends longer than the rising limb.) **Using 1.5 times t_c in the direct solution methodology provides for a more conservative design and will be used in this manual**.

The modified rational method allows the designer to analyze several different storm durations to determine the one that requires the greatest storage volume with respect to the allowable release rate. This storm duration is referred to as the ***critical storm duration*** and is used as a basin sizing tool. The technique is discussed in more detail in **Chapter 5** of this handbook.

FIGURE 4 - 10
Modified Rational Method Family of Runoff Hydrographs



4-4.3 SCS Methods - TR-55 Estimating Runoff

The U.S. Soil Conservation Service published Technical Release Number 55 (TR-55), 2nd edition, in June of 1986, entitled Urban Hydrology for Small Watersheds. The techniques outlined in TR-55 require the same basic data as the rational method: drainage area, time of concentration, land use and rainfall. The SCS approach, however, is more sophisticated in that it allows the designer to manipulate the time distribution of the rainfall, the initial rainfall losses to interception and depression storage, and the moisture condition of the soils prior to the storm.

The procedures developed by SCS are based on a dimensionless rainfall distribution curve for a 24-hour storm, as described in **Section 4-2.3**.

TR-55 presents two general methods for estimating peak discharges from urban watersheds: the ***graphical method*** and the ***tabular method***. The ***graphical method*** is limited to watersheds whose runoff characteristics are fairly uniform and whose soils, land use, and ground cover can be represented by a single Runoff Curve Number (*CN*). The graphical method provides a peak discharge only and is not applicable for situations where a hydrograph is required.

The ***tabular method*** is a more complete approach and can be used to develop a hydrograph at any point in a watershed. For large areas it may be necessary to divide the area into sub-watersheds to account for major land use changes, analyze specific study points within sub-watersheds, or locate stormwater drainage facilities and assess their effects on peak flows. The tabular method can generate a hydrograph for each sub-watershed for the same storm event. The hydrographs can then be ***routed*** through the watershed and combined to produce a partial composite hydrograph at the selected study point. The tabular method is particularly useful in evaluating the effects of an altered land use in a specific area within a given watershed.

Prior to using either the graphical or tabular methods, the designer must determine the volume of runoff resulting from a given depth of precipitation and the time of concentration, t_c , for the watershed being analyzed. The methods for determining these values will be discussed briefly in this section. However, the reader is strongly encouraged to obtain a copy of the TR-55 manual from the Soil Conservation Service to gain more insight into the procedures and limitations.

The SCS ***Runoff Curve Number (CN) Method*** is used to estimate runoff. This method is described in detail in the SCS National Engineering Handbook, Section 4 (SCS 1985). The runoff equation (found in TR-55 and discussed later in this section) provides a relationship between runoff and rainfall as a function of the *CN*. The *CN* is a measure of the land's ability to infiltrate or otherwise detain rainfall, with the excess becoming runoff. The *CN* is a function of the land cover (woods, pasture, agricultural use, percent impervious, etc.), hydrologic condition, and soils.

4-4.3.1 Limitations

1. *TR-55 has simplified the relationship between rainfall and runoff by reducing all of the initial losses before runoff begins, or initial abstraction, to the term I_a, and approximating the soil and cover conditions using the variable S, potential maximum retention. Both of these terms, I_a and S, are functions of the runoff curve number.*

Runoff curve numbers describe average conditions that are useful for design purposes. If the purpose of the hydrologic study is to model a historical storm event, average conditions may not be appropriate.

2. *The designer should understand the assumption reflected in the initial abstraction term, I_a. I_a represents interception, initial infiltration, surface depression storage, evapotranspiration, and other watershed factors and is generalized as a function of the runoff curve number based on data from agricultural watersheds.*

This can be especially important in an urban application because the combination of impervious area with pervious area can imply a significant initial loss that may not take place. On the other hand, the combination of impervious and pervious area can underestimate initial losses if the urban area has significant surface depression storage. (To use a relationship other than the one established in TR-55, the designer must redevelop the runoff equation by using the original rainfall-runoff data to establish new curve number relationships for each cover and hydrologic soil group. This would represent a large data collection and analysis effort.)

3. *Runoff from snowmelt or frozen ground cannot be estimated using these procedures.*

4. *The runoff curve number method is less accurate when the runoff is less than 0.5 inch. As a check, use another procedure to determine runoff.*

5. *The SCS runoff procedures apply only to surface runoff and do not consider subsurface flow or high groundwater.*

6. *Manning's kinematic solution (**Chapter 4-4.3.3.E**) should not be used to calculate the time of concentration for sheet flow longer than 300 feet. This limitation will affect the time of concentration calculations. Note that many jurisdictions consider 150 feet to be the maximum length of sheet flow before shallow concentrated flow develops.*

7. *The minimum t_c used in TR-55 is 0.1 hour.*

4-4.3.2 Information Needed

Generally a good understanding of the physical characteristics of the watershed is needed to solve the runoff equation and determine the time of concentration. Some features, such as topography and channel geometry can be obtained from topographic maps such as the USGS *1″ = 2000′* quadrangle maps. Various sources of information may be accurate enough for a watershed study, however, the accuracy of the study will be directly related to the accuracy and level of detail of the base information. Ideally, a site investigation and field survey should be conducted to verify specific features such as channel geometry and material, culvert sizes, drainage divides, ground cover, etc. Depending on the size and scope of the study, however, a site investigation may not be economically feasible.

The data needed to solve the runoff equation and determine the watershed time of concentration, t_c , and travel time, T_t , is listed below. These items are discussed in more detail in **Section 4-4.3.3**.

1. *Soil information (to determine the hydrologic soil group).*

2. *Ground cover type (impervious, woods, grass, etc.).*

3. *Treatment (cultivated or agricultural land).*

4. *Hydrologic condition (for design purposes, the hydrologic condition should be considered "GOOD" for the pre-developed condition).*

5. *Urban impervious area modifications (connected, unconnected, etc.).*

6. *Topography – detailed enough to accurately identify drainage divides, t_c and T_t flow paths and channel geometry, and surface condition (roughness coefficient).*

4-4.3.3 Design Parameters

A. Soils

In hydrograph applications, runoff is often referred to as ***rainfall excess*** or ***effective rainfall***, and is defined as the amount of rainfall that exceeds the land's capability to infiltrate or otherwise retain the rainwater. The soil type or classification, the land use and land treatment, and the hydrologic condition of the cover are the watershed factors that will have the most significant impact on estimating the volume of rainfall excess, or runoff.

HYDROLOGIC SOIL GROUP CLASSIFICATION

SCS has developed a soil classification system that consists of four groups, identified as ***A***, ***B***, ***C***, and ***D***. Soils are classified into one of these categories based upon their ***minimum infiltration rate***. By using information obtained from local SCS offices, soil and water conservation district offices, or from SCS Soil Surveys (published for many counties across the country), the soils in a given area can be identified. Preliminary soil identification is especially useful for watershed analysis and planning in general. When preparing a stormwater management plan for a specific site, it is recommended that soil borings be taken to verify the hydrologic soil classification. Virginia soils and their respective ***Hydrologic Soil Group*** (***HSG***) classifications are provided in the Appendix at the end of this Chapter, as well as VESCH, 1992 edition. TR-55 contains similar information for soils across the United States.

Soil characteristics associated with each Hydrologic Soil Group are generally described as follows:

Group A: Soils with low runoff potential due to high infiltration rates, even when thoroughly wetted. These soils consist primarily of deep, well to excessively drained sands and gravels with high water transmission rates (**0.30 in./hr.**). Group **A** soils include **sand, loamy sand,** or **sandy loam**.

Group B: Soils with moderately low runoff potential due to moderate infiltration rates when thoroughly wetted. These soils consist primarily of moderately deep to deep, and moderately well to well-drained soils. Group **B** soils have moderate water transmission rates (**0.15-0.30 in./hr.**) and include **silt loam or loam**.

Group C: Soils with moderately high runoff potential due to slow infiltration rates when thoroughly wetted. These soils typically have a layer near the surface that impedes the downward movement of water or soils. Group **C** soils have low water transmission rates (**0.05-0.15 in./hr.**) and include **sandy clay loam**.

Group D: Soils with high runoff potential due to very slow infiltration rates. These soils consist primarily of clays with high swelling potential, soils with permanently high water tables, soils with a claypan or clay layer at or near the surface, and shallow soils over nearly impervious parent material. Group **D** soils have very low water transmission rates (**0-0.05 in./hr.**) and include **clay loam**, **silty clay loam**, **sandy clay**, **silty clay**, or **clay**.

Any disturbance of a soil profile can significantly alter the soil's infiltration characteristics. With urbanization, the hydrologic soil group for a given area can change due to soil mixing, introduction of fill material from other areas, removal of material during mass grading operations, or compaction from construction equipment. A layer of topsoil may typically be saved and replaced after the earthwork is completed, but the native underlying soils have been dramatically altered. **Therefore, any disturbed soil should be classified by its physical characteristics** as given above for each soil group.

Some jurisdictions require all site developments to be analyzed using an ***HSG*** classification that is one category below the actual pre-developed ***HSG***. For example, a site with a pre-developed ***HSG*** classification of ***B***, as determined from the soil survey, would be analyzed in its developed state using an ***HSG*** classification of ***C***.

B. Hydrologic Condition

Hydrologic condition represents the effects of ***cover type*** and ***treatment*** on infiltration and runoff. It is generally estimated from the density of plant and residue cover across the drainage area. ***Good hydrologic condition*** indicates that the cover has a low runoff potential, while ***poor hydrologic condition*** indicates that the cover has a high runoff potential. ***Hydrologic condition*** is used in describing non-urbanized lands such as woods, meadow, brush, agricultural land, and open spaces associated with urbanized areas, such as lawns, parks, golf courses, and cemeteries. ***Treatment*** is a cover type modifier to describe the management of cultivated agricultural lands. **Table 4-6(a,b)** provides an excerpt from Table 2-2 in TR-55 which shows the treatment and hydrologic condition for various land uses.

When a watershed is being analyzed to determine the impact of proposed development, Virginia's stormwater management regulations require the designer to consider all existing or undeveloped land to be in *hydrologically good condition*. This results in lower existing condition peak runoff rates which, in turn, results in greater post-development peak control. In most cases, undeveloped land is in good hydrologic condition unless it has been altered in some way. Since the goal of most stormwater programs is to reduce the peak flows from developed or altered areas to their pre-developed or pre-altered rates, this is a reasonable approach. In addition this approach eliminates any inconsistencies in judging the condition of undeveloped land or open space.

C. Runoff Curve Number (CN) Determination

The soil group classification, cover type and the hydrologic condition are used to determine the runoff curve number, ***CN***. The *CN* indicates the runoff potential of an area when the ground is not frozen. **Table 4-6(a,b)**, excerpted from TR-55,provides the *RCNs* for various land use types and soil groups. (A more complete table can be found in TR-55.)

Several factors should be considered when choosing an *CN* for a given land use. First, the designer should realize that the curve numbers in **Table 4-6** and TR-55 are for the ***average antecedent runoff*** or ***moisture condition***, ***ARC***. The *ARC* is the index of runoff potential before a storm event and can have a major impact on the relationship between rainfall and runoff for a watershed. Average *ARC* implies that the soils are neither very wet nor very dry when the design storm begins. Average *ARC* runoff curve numbers can be converted to dry or wet values, however the average antecedent runoff condition is recommended for design purposes.

A decision to use "wet" or "dry" antecedent runoff conditions should be based on thorough field work, such as carefully monitored rain gauge data.

It is also important to consider the list of assumptions made in developing the runoff curve numbers as provided in **Table 4-6** and in TR-55. Some of these assumptions are outlined below.

TABLE 4 - 6a

Runoff Curve Numbers for Urban Areas [1]

Adapted from TR-55 Table 2-2a. --Runoff Curve Numbers for Urban Areas *

Cover Description		Curve Numbers for Hydrologic Soil Group:			
Cover Type and Hydrologic Condition	**Average percent impervious area** [2]	**A**	**B**	**C**	**D**
Fully developed urban areas (vegetation established) :					
Open space (lawns, parks, golf courses, cemeteries, etc.)[2]:					
Good condition (grass cover > 75%)		39	61	74	80
Impervious areas:					
Paved parking lots, roofs, driveways, etc. (excluding right-of-way)		98	98	98	98
Streets and roads:					
Paved; curbs and storm sewers (excluding right-of-way)		98	98	98	98
Paved; open ditches (including right-of-way)		83	89	92	93
Gravel (including right-of-way)		76	85	89	91
Dirt (including right-of-way)		72	82	87	89
Urban districts:					
Commercial and business	85	89	92	94	95
Industrial	72	81	88	91	93
Residential districts by average lot size:					
1/8 acre or less (town houses)	65	77	85	90	92
1/4 acre	38	61	75	83	87
1/3 acre	30	57	72	81	86
½ acre	25	54	70	80	85
1 acre	20	51	68	79	84
2 acres	12	46	65	77	82
Developing urban areas:					
Newly graded areas (pervious areas only, no vegetation)[2]		77	86	91	94
Idle lands (CN's are determined using cover types similar to those in TR-55 Table 2-2c).					

**Average runoff condition and* $I_a = 0.2S$

[1]*Refer to TR-55 for additional cover types and general assumptions and limitations.*

[2]*For specific footnotes, see TR-55* ***Table 2-2a.***

TABLE 4 - 6b
Runoff Curve Numbers for Agricultural Areas[1]

Adapted from TR-55 Table 2-2b-- Runoff Curve Numbers for Other Agricultural Lands[*]					
Cover Description		**Curve Numbers for Hydrologic Soil Group:**			
Cover Type	**Hydrologic Condition**	**A**	**B**	**C**	**D**
Pasture, grassland, or range - continuous forage for grazing [2].	Good	39	61	74	80
Meadow - continuous grass, protected from grazing and generally mowed for hay . . .	----	30	58	71	78
Brush - brush-weed-grass mixture with brush the major element[2]	Good	[2]30	48	65	73
Woods - grass combination (orchard or tree farm)[2]	Good	32	58	72	79
Woods [2]	Good	[2]30	55	70	77
Farmsteads - buildings, lanes, driveways, and surrounding lots.	----	59	74	82	86

**Average runoff condition and $I_a = 0.2S$*

[1]*Refer to TR-55 for additional cover types and general assumptions and limitations.*
[2]*For specific footnotes, see TR-55* ***Table 2-2b****.*

RCN Determination Assumptions (TR-55):

1. *The urban curve numbers, for such land uses as residential, commercial, and industrial, are computed with the percentage of impervious area as shown. A composite curve number should be re-computed using the actual percentage of imperviousness if it differs from the value shown.*

2. *The impervious areas are directly connected to the drainage system.*

3. *Impervious areas have a runoff curve number of 98.*

4. *Pervious areas are considered equivalent to open space in good hydrologic condition.*

These assumptions, as well as others, are footnoted in TR-55, Table 2-2. **TR-55 provides a graphical solution for modification of the given *RCNs* if any of these assumptions do not hold true**.

The designer should become familiar with the definition of ***connected*** versus ***unconnected*** impervious areas along with the graphical solutions and the impact that their use can have on the resulting *RCN*. After some experience in using this section of TR-55, the designer will be able to make field evaluations of the various criteria used in the determination of the *RCN* for a given site. In addition, the designer will need to determine if the watershed contains sufficient diversity in land use to justify dividing the watershed into several sub-watersheds. If a watershed or drainage area cannot be adequately described by one weighted curve number, then the designer must divide the watershed into sub-areas and analyze each one individually, generate individual hydrographs, and add those hydrographs together to determine the composite peak discharge for the entire watershed.

Figure 4-11 shows the decision making process for analyzing a drainage area. The flow chart can be used to select the appropriate tables or figures in TR-55 from which to then choose the runoff curve numbers. Worksheet 2 in TR-55 is then used to compute the weighted curve number for the area or sub-area.

D. The Runoff Equation

The SCS runoff equation is used to solve for runoff as a function of the initial abstraction, I_a, and the potential maximum retention, S, of a watershed, both of which are functions of the *RCN*. This equation attempts to quantify all the losses before runoff begins, including infiltration, evaporation, depression storage, and water intercepted by vegetation.

TR-55 provides a graphical solution for the runoff equation. The graphical solution is found in Chapter 2 of TR-55: Estimating Runoff. Both the equation and graphical solution solve for the depth of runoff that can be expected from a watershed or sub-watershed, of a specified *RCN*, for any given frequency storm. Additional information can be found in the USDA-SCS National Engineering Handbook, Section 4.

These procedures, by providing the basic relationship between rainfall and runoff, are the basis for any hydrological study based on SCS methodology. Therefore, it is essential that the designer conduct a thorough site visit and consider all the site features and characteristics, such as soil types and hydrologic condition, when analyzing a watershed or drainage area.

E. Time of Concentration and Travel Time

The time of concentration, t_c, is the length of time required for a drop of water to travel from the most hydraulically distant point in the watershed or sub-watershed to the point of analysis. The travel time, T_t, is the time it takes that same drop of water to travel from the study point at the bottom of the sub-watershed to the study point at the bottom of the whole watershed. The travel time, T_t, is descriptive of the sub-watershed by providing its location relative to the study point of the entire watershed.

Similar to the rational method, the time of concentration, t_c, plays an important role in developing the peak discharge for a watershed. **Urbanization usually decreases the t_c, which results in an increase in peak discharge**. For this reason, to accurately model the watershed, the designer must be aware of any conditions which may act to decrease the flow time, such as channelization and channel improvements. On the other hand, the designer must also be aware of the conditions within the watershed which may actually lengthen the flow time, such as surface ponding above undersized conveyance systems and culverts.

1. Heterogeneous Watersheds

A heterogeneous watershed is one that has two or more hydrologically-defined drainage areas of differing land uses, hydrologic conditions, times of concentration, or other runoff characteristics, contributing to the study point. Quite often, development will turn a homogeneous watershed into a heterogeneous one. **Example 1** from **Chapter 6** provides an example of such a case.

Example 1 presents a heterogeneous watershed (in the developed condition) that generates a majority of its runoff from a portion of the watershed that does not contain the most hydrologically distant flow path. Therefore, the development has a very minor impact on the time of concentration. Since the longest t_c flow path is not representative of the peak flows from the area that contributes the majority of the total peak discharge to the study point, an alternate flow path should be selected that accurately reflects the timing and volume of the peak flow. **Figure 4-12** shows a schematic of the **Example 1** watershed, pre- and post-developed conditions. Note the location of the pre-developed t_c flow path and the minimal impact that development has on it.

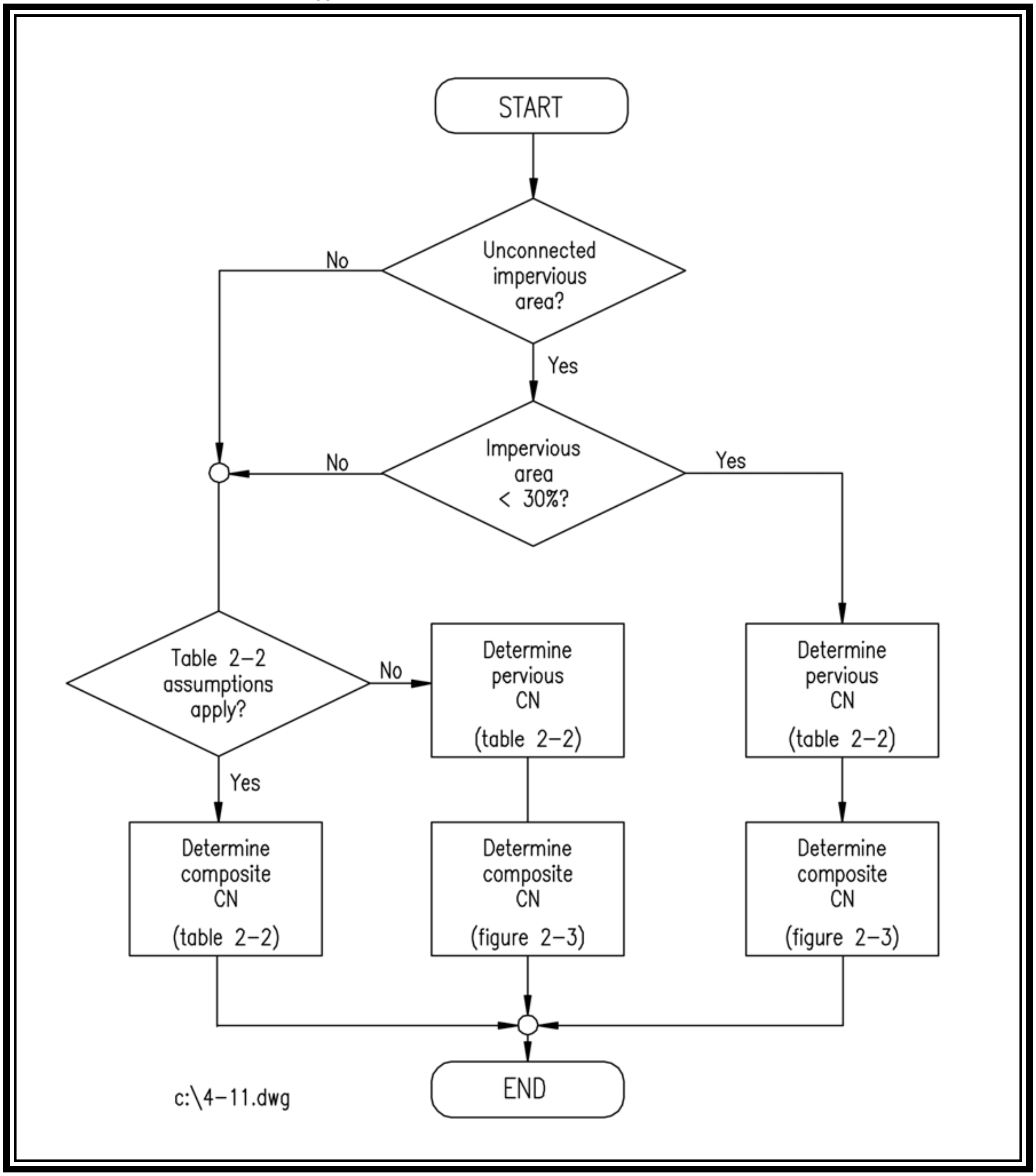
Table 4-7 gives a summary of the **post-developed** hydrologic data for **Example 1**, with the t_c flow path computed three different ways:

1. The entire watershed considered as one homogeneous watershed with the t_c flow path representing the most hydraulically distant point.

2. The entire watershed considered as one homogeneous watershed with the t_c flow path adjusted to reflect the flow from the developed area.

3. The watershed divided into two sub-watersheds and their peak flow hydrographs added together at the study point.

FIGURE 4 - 11
Runoff Curve Number Selection Flow Chart



Source: SCS TR-55 - Urban Hydrology for Small Watersheds

TABLE 4 - 7
Hydrologic Summary - Example 1

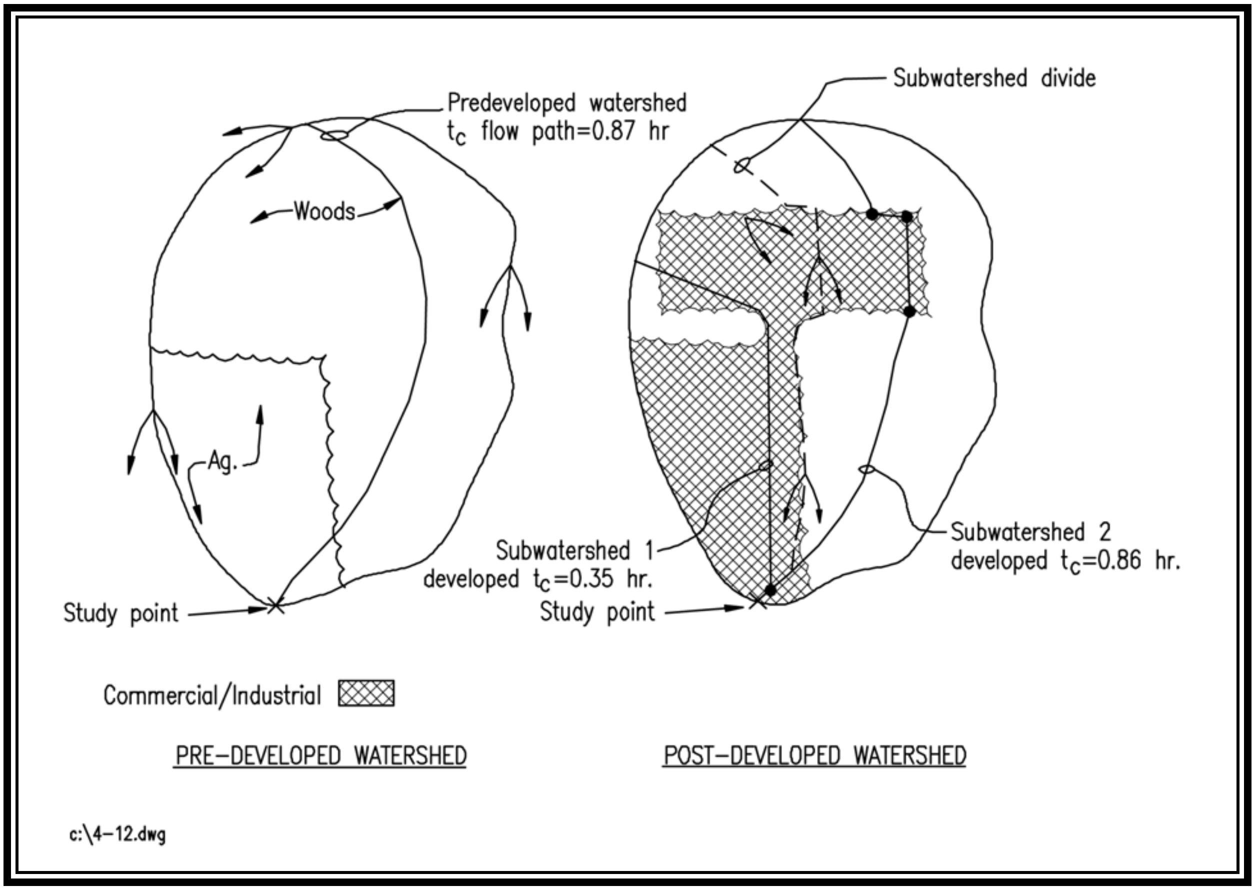
WATERSHED CONDITION	*RCN*	t_c *(hr.)*	Q_2 *(cfs)*
1. Homogeneous Watershed[1] Post-Developed	75	0.86	18.3
2. Homogeneous Watershed[1] Post-Developed (Adjusted T_c Path)	75	0.35	29.9
3. Sub-Watershed 1[2] Sub-Watershed 2 COMBINED:	84 67	0.35 0.86	21.9 <u>7.2</u> 26.2

Notes: 1. Conditions 1 and 2 were computed using the <u>TR-55</u> graphical peak discharge method.
2. Condition 3 was computed, and hydrographs added together, using the <u>TR-55</u> Tabular hydrograph method.

Refer to ***Chapter 6, Example 1*** *for completed worksheets.*

Note that the combined peak discharge (Condition #1, using the <u>TR-55</u> tabular method) for sub-watersheds 1 and 2 is smaller than the sum of their individual peaks (Condition #3). This occurs because their peak flows do not coincide simultaneously at the study point, per the t_c determination. **Example 1** illustrates the impact of the t_c flow path selection on a given study point for **any** watershed being examined for the effects of development. The second (or third) method in **Table 4-7** is the most representative of the impacts to the watershed for this particular example. **The flow path should be carefully selected to accurately reflect the development within a watershed and the resulting peak discharge.** See **Section 4-4.5** for details on the <u>TR-55</u> tabular method, and **Chapter 6** for **Example 1** <u>TR-55</u> worksheets.

FIGURE 4 - 12
Pre- and Post-Developed Watersheds - Example 1



2. Flow Segments

The time of concentration is the sum of the time increments for each flow segment present in the t_c flow path, such as ***overland*** or ***sheet flow***, ***shallow concentrated flow***, and ***channel flow***. These flow types are influenced by surface roughness, channel shape, flow patterns, and slope, and are discussed below:

a. ***Overland (sheet) flow*** is shallow flow over plane surfaces. For the purposes of determining time of concentration, overland flow usually exists in the upper reaches of the hydraulic flow path.

TR-55 utilizes Manning's kinematic solution to compute t_c for overland sheet flow. The roughness coefficient is the primary culprit in the misapplication of the kinematic t_c equation. Care should be taken to accurately identify the surface conditions for overland flow. **Table 4-9(a)** in this handbook and Table 3-1 in TR-55 provide selected coefficients for various surface conditions. Refer to TR-55 and Examples in **Chapter 6** for the use of Mannings Kinematic Equation.

NOTE: *Sheet flow can influence the peak discharge of small watersheds dramatically because the ratio of flow length to flow velocity is usually very high.* ***Surface roughness, soil types, and slope will dictate the distance before sheet flow transitions into shallow concentrated flow****.* *TR-55 stipulates that the maximum length of sheet flow is 300 feet. Many hydrologists and geologists will argue that, based on the definition of sheet flow, that 100 to 150 feet is the maximum distance before the combination of quantity and velocity create shallow concentrated flow. In an urban application (usually a relatively small drainage area), the flow time associated with 300 feet of sheet flow will result in a disproportionately large segment of the total time of concentration for the watershed. This will result in a very slow overall* t_c *, and may not be representative of the drainage area as a whole.* ***As stated previously, the designer must be sure that the flow path chosen is not only representative of the drainage area, but also is the flow path for the significant portion of the total peak discharge****.*

Notice that in **Example 1**, the majority of the pre- and post-developed condition t_c flow time for the watershed is overland sheet flow. **Table 4-8** computes the t_c for **Example 1**, (total $t_c = 0.87$ *hrs*., overland sheet flow = *0.75 hrs*.) **Table 4-8** also shows the sensitivity of the peak discharge to adjustments in the Manning's roughness coefficient, *n*, for overland sheet flow. Note that the manipulation of the roughness coefficient can have a significant impact on the computed discharge. This illustrates the need for accurate watershed condition data. An on-site investigation should be completed to determine actual land cover conditions, or the designer should state that assumed values are being used. **Careful selection of the surface roughness coefficient is essential to calculate an accurate t_c and peak discharge**.

TABLE 4 - 8

T_c and Peak Discharge Sensitivity to Overland Sheet Flow Roughness Coefficients

Description	Manning's 'n'	Overland Sheet Flow Time* *(hrs.)*	Pre-Developed Total Time of Concentration, t_c *(hrs.)*	2-Yr. Pre-Developed Peak Discharge** *(cfs)*
Woods - Light Underbrush	.40	0.75	0.87	8.5
Rangeland-Natural	.13	0.31	0.43	15.0
Woods - Dense Underbrush	.80	1.31	1.43	6.0

* *overland flow time calculated using Manning's kinematic solution (TR-55)*
** *peak discharge computed using Example 6.1 hydrology*

b. ***Shallow Concentrated Flow*** usually begins where overland flow converges to form small rills or gullies. Shallow concentrated flow can exist in small manmade drainage ditches (paved and unpaved) and in curb and gutters.

TR-55 provides a graphical solution for shallow concentrated flow. The input information needed to solve for this flow segment is the land slope and the surface condition (paved or unpaved).

c. ***Channel flow*** occurs where flow converges in gullies, ditches or swales, and natural or manmade water conveyances (including storm drainage pipes). Channel flow is assumed to exist in perennial streams or wherever there is a well-defined channel cross-section.

The Manning Equation is used for open channel flow and pipe flow, and usually assumes full flow or bank-full velocity. Manning coefficients can be found in **Table 4-9(b-d)** for open channel flow (natural and man-made channels) and closed channel flow. Coefficients can also be obtained from standard textbooks such as Open Channel Hydraulics or Handbook of Hydraulics.

TABLE 4 - 9a

Roughness Coefficient 'n' for the Manning Equation - Sheet Flow

Surface Description	**'*n*' Value**
Smooth Surfaces (Concrete, Asphalt, Gravel, or Bare Soil	0.011
Fallow (No Residue)	0.05
Cultivated Soils:	
Residue Cover < 20%	0.06
Residue Cover > 20%	0.17
Grass:	
Short Grass Prairie	0.15
Dense Grasses[2]	0.24
Bermuda grass	0.41
Range (Natural)	0.13
Woods:[3]	
Light Underbrush	0.40
Dense Underbrush	0.80

[1] The 'n' values are composite of information compiled by Engman (1986).
[2] Includes species such as weeping lovegrass, bluegrass, buffalo grass, blue grama grass, and native grass mixtures.
[3] When selecting n, consider cover to a height of about 0.1 ft. This is the only part of the plant cover that will obstruct sheet flow.

From 210-VI-TR-55, Second Edition, June 1986

TABLE 4 - 9b
Roughness Coefficient 'n' for the Manning Equation - Pipe Flow

Material	'*n*' Value Range	
	From	To
Coated Cast-iron	0.010	0.014
Uncoated Cast-iron	0.011	0.015
Vitrified Sewer Pipe	0.010	0.017
Concrete Pipe	0.010	0.017
Common Clay Drainage Tile	0.011	0.017
Corrugated Metal (2 2/3 x ½)	0.023	0.026
Corrugated Metal (3 x 1 and 6 x 1)	0.026	0.029
Corrugated Metal (6 x 2 Structural Plate)	0.030	0.033

Source: Handbook of Hydraulics, Sixth Edition, Brater & King

TABLE 4 - 9c

Roughness Coefficient 'n' for the Manning Equation - Constructed Channels

Lining Material	**'*n*' Value Range**	
	From	**To**
Concrete Lined	0.012	0.016
Cement Rubble	0.017	0.025
Earth, Straight and Uniform	0.017	0.022
Rock Cuts, Smooth and Uniform	0.025	0.033
Rock Cuts, Jagged and Irregular	0.035	0.045
Winding, Sluggish Canals	0.022	0.027
Dredged Earth Channels	0.025	0.030
Canals with Rough Stony Beds, Weeds on Earth Banks	0.025	0.035
Earth Bottom, Rubble Sides	0.028	0.033
Small Grass Channels:		
Long Grass - 13"	0.042	
Short Grass - 3"	0.034	

Adapted from Handbook of Hydraulics, Sixth Edition, Brater & King

TABLE 4 - 9d

Roughness Coefficient 'n' for the Manning Equation - Natural Stream Channels

Channel Lining	**'*n*' Value Range**	
	From	**To**
1. Clean, Straight Bank, Full Stage, No Rifts or Deep Pools	0.025	0.030
2. Same as #1, But Some Weeds and Stones	0.030	0.035
3. Winding, Some Pools and Shoals, Clean	0.033	0.040
4. Same as #3, Lower Stages, More Ineffective Slope and Sections	0.040	0.050
5. Same as #3, Some Weeds and Stones	0.035	0.045
6. Same as #4, Stony Sections	0.045	0.055
7. Sluggish River Reaches, Rather Weedy with Very Deep Pools	0.050	0.070
8. Very Weedy Reaches	0.075	0.125

Adapted from Handbook of Hydraulics, Sixth Edition, Brater & King

4-4.4 TR-55 Graphical Peak Discharge Method

The ***graphical peak discharge method*** was developed from hydrograph analyses using TR-20, Computer Program for Project Formulation-Hydrology (SCS, 1983). The graphical method develops the peak discharge in cubic feet per second(cfs) for a given watershed.

4-4.4.1 Limitations

There are several limitations that the designer should be aware of before using the TR-55 graphical method:

1. *The watershed being studied must be hydrologically homogeneous, i.e., the land use, soils, and cover are distributed uniformly throughout the watershed and can be described by one curve number.*

2. *The watershed may have only one main stream or flow path. If more than one is present they must have nearly equal t_c's so that the entire watershed is represented by one t_c .*

3. *The analysis of the watershed cannot be part of a larger watershed study which would require adding hydrographs since the graphical method does not generate a hydrograph.*

4. *For the same reason, the graphical method should not be used if a runoff hydrograph is to be routed through a control structure.*

5. *When the initial abstraction - rainfall ratio (I_a / P) falls outside the range of the Unit Peak Discharge curves (0.1 to 0.5), the limiting value of the curve must be used.*

The reader is encouraged to review the TR-55 Manual to become familiar with these and other limitations associated with the graphical method.

The graphical method can be used as a planning tool to determine the impact of development or land use changes within a watershed, or to anticipate or predict the need for stormwater management facilities or conveyance improvements. Sometimes, the graphical method can be used in conjunction with the TR-55 short-cut method for estimating the storage volume required for post-developed peak discharge control. This short-cut method is found in Chapter 6 of TR-55 and is discussed in **Chapter 5** of this handbook. However, it should be noted that a more sophisticated computer model such as TR-20 or HEC-1, or even TR-55 Tabular Hydrograph Method, should be used for analyzing complex, urbanizing watersheds.

4-4.4.2 Information Needed

The following represents a brief list of the parameters needed to compute the peak discharge of a watershed using the TR-55 Graphical Peak Discharge Method. For a detailed explanation of the terms listed, refer to **Section 4-4.4.3** in this handbook.

1. *The drainage area, in square miles*
2. t_c *, in hours*
3. *Weighted runoff curve number, CN*
4. *Rainfall amount, P, for specified design storm, in inches*
5. *Total runoff, Q, in inches (see runoff equation, TR-55)*
6. *Initial abstraction,* I_a *, for each subarea*
7. *Ratio of* I_a / P *for each subarea*
8. *Rainfall distribution (Type I, IA, II or III)*

4-4.4.3 Design Parameters

The TR-55 ***peak discharge equation*** is:

$$q_p = q_u A_m Q F_p$$

Equation 4-3
TR-55 Peak Discharge Equation

where:

q_p = *peak discharge, cfs*
q_u = *unit peak discharge, cfs/mi²/in (csm/in)*
A_m = *drainage area, mi²*
Q = *runoff, in inches, and*
F_p = *pond and swamp adjustment factor*

All the required information has been determined earlier except for the unit peak discharge, q_u , and the pond and swamp adjustment factor, F_p.

The unit peak discharge, q_u , is a function of the initial abstraction, I_a , precipitation, P, and the time of concentration, t_c , and can be determined from the Unit Peak Discharge Curves in TR-55. The unit peak discharge is expressed in *cubic feet per second per square mile per inch of runoff.*

Initial abstraction, as indicated previously, is a measure of all the losses that occur before runoff begins, including infiltration, evaporation, depression storage, and water intercepted by vegetation, and can be calculated from empirical equations or Table 4-1 in TR-55.

The pond and swamp adjustment factor is an adjustment in the peak discharge to account for pond and swamp areas if they are spread throughout the watershed and are not considered in the t_c computation. Refer to TR-55 for more information on pond and swamp adjustment factors.

The unit peak discharge, q_u , is obtained by using t_c and the I_a /P ratio with Exhibit 4-I, 4-IA, 4-II, or 4-III (depending on the rainfall distribution type) in TR-55. As limitation number 5 above indicates, the ratio of I_a /P must fall between 0.1 and 0.5. The designer must use the limiting value on the curves when the computed value is not within this range. The unit peak discharge is determined from these curves and entered into the above equation to calculate the peak discharge.

4-4.5 TR-55 Tabular Hydrograph Method

The ***tabular hydrograph method*** can be used to analyze large heterogeneous watersheds. The tabular method can develop partial composite flood hydrographs at any point in a watershed by dividing the watershed into homogeneous subareas. The method is especially applicable for estimating the effects of land use change in a portion of a watershed.

The tabular hydrograph method provides a tool to efficiently analyze several sub-watersheds to verify the combined impact at a downstream study point. It is especially useful to verify the timing of peak discharges. Sometimes, the use of detention in a lower sub-watershed may actually increase the combined peak discharge at the study point. This procedure allows a quick check to verify the timing of the peak flows and to decide if a more detailed study is necessary.

4-4.5.1 Limitations

The following represents some of the basic limitations that the designer should be aware of before using the TR-55 tabular method:

1. *The travel time, T_t , must be less than 3 hours (largest T_t in TR-55 , Exhibit 5).*

2. *The time of concentration, t_c , must be less than 2 hours (largest t_c in TR-55 , Exhibit 5).*

3. *The acreage of the individual sub-watersheds should not differ by a factor of 5 or more.*

When these limitations cannot be met, the designer should use the TR-20 computer program or other available computer models which will provide more accurate and detailed results.

The reader is encouraged to review the TR-55 manual to become familiar with these and other limitations associated with the tabular method.

4-4.5.2 Information Needed

The following represents a brief list of the parameters needed to compute the peak discharge of a watershed using the TR-55 Tabular method. For a detailed explanation of the terms listed, refer to **Section 4-4.4.3** in this handbook.

1. *Subdivision of the watershed into areas that are relatively homogeneous.*
2. *The drainage area of each subarea, in square miles.*
3. *Time of concentration,* t_c *, for each subarea in hours.*
4. *Travel time,* T_t *, for each routing reach, in hours.*
5. *Weighted runoff curve number, RCN, for each subarea.*
6. *Rainfall amount, P, in inches, for each specified design storm.*
7. *Total runoff, Q, in inches (see runoff equation, TR-55) for each subarea.*
8. *Initial abstraction,* I_a *, for each subarea.*
9. *Ratio of* I_a */P for each subarea.*
10. *Rainfall distribution (I, IA, II or III)*

4-4.5.3 Design Parameters

The use of the tabular method requires that the designer determine the travel time through the entire watershed. As stated previously, the entire watershed is divided into smaller sub-watersheds that must be related to one another and to the whole watershed with respect to *time.* The result is that the time of peak discharge is known for any one sub-watershed relative to any other sub-watershed or for the entire watershed.

> ***Travel time***, T_t , represents the time for flow to travel from the study point at the bottom of a sub-watershed to the bottom of the entire watershed. This information must be compiled for each sub-watershed.

The data for up to 10 sub-watersheds can be compiled on one TR-55 worksheet. (TR-55 Worksheets 5a and 5b.)

To obtain the peak discharge using the *graphical method*, the unit peak discharge is read off of a curve. However, the *tabular method* provides this information in the form of a table of values, found in TR-55, Exhibit 5. These tables are arranged by rainfall type (I, IA, II, and III), I_a /P, t_c , and T_t . In most cases, the actual values for these variables, other than the rainfall type, will be different from the values shown in the table. Therefore, a system of rounding these values has been established in the TR-55 manual. The I_a/P term is simply rounded to the nearest table value. The t_c and T_t values are rounded together in a procedure that is outlined on pages 5-2 and 5-3 of the TR-55 manual. The accuracy of the computed peak discharge and time of peak discharge is highly dependent on the proper use of these procedures.

The following equation, along with the information compiled on TR-55 Worksheet 5b, is then used to determine the flow at any time:

$$q = q_t A_m Q$$

Equation 4-4
Tabular Hydrograph Peak Discharge Equation

where:
- q = *hydrograph coordinate in cfs, at hydrograph time t;*
- q_t = *tabular hydrograph unit discharge at hydrograph time t from TR-55 Exhibit 5, csm/in;*
- A_m = *drainage area of individual subarea, mi^2; and*
- Q = *runoff in.*

The product A_mQ is multiplied by each table value in the appropriate unit hydrograph in TR-55 Exhibit 5, (each sub-watershed may use a different unit hydrograph) to generate the actual hydrograph for the subwatershed. This hydrograph is tabulated on TR-55 worksheet 5b and then added together with the hydrographs from the other sub-watersheds, being careful to use the same time increment for each subwatershed. The result is a composite hydrograph at the bottom of the worksheet for the entire watershed. Refer to Example 1 in Chapter 6 for a completed analysis using the TR-55 tabular hydrograph method.

The preceding discussion on the Tabular Method is taken from TR-55 and is NOT complete. The designer should obtain a copy of TR-55 and learn the procedures and limitations as outlined in that document. Examples and worksheets are provided in TR-55 that lead the reader through the procedures for each chapter.

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CHAPTER 4

APPENDIX

APPENDIX 4A

Hydrologic Soil Groups in Virginia

The majority of soils found in Virginia along with their corresponding Hydrologic Soil Group designation are listed on the following pages. **All stormwater BMP designs that require specific soil conditions to be present should be based on an actual soils analysis.**

Soil Name	Hydgrp	Soil Name	Hydgrp	Soil Name	Hydgrp
APPOMATTOX	B	AQUENTS	D	AQUULTS	D
ARAPAHOE	B/D	ARCOLA	C	ARGENT	D
ASBURN*	C	ASHE	B	ASHLAR	B
ASSATEAGUE	A	ATKINS	D	ATLEE	C
AUGUSTA	C	AURA	B	AUSTINVILLE	B
AXIS	D	AYCOCK	B	BACKBAY	D
BADIN	B	BAILE	D	BAILEGAP	B
BAMA	B	BAYBORO	D	BEACHES	D
BECKHAM	B	BELHAVEN	D	BELTSVILLE	C
BELVOIR	C	BERKS	C	BERMUDIAN	B
BERTIE	B	BIBB	D	BILTMORE	A
BIRDSBORO	B	BLADEN	D	BLAIRTON	C
BLAND	C	BLEAKHILL	C	BLUEMONT*	B
BOHICKET	D	BOJAC	B	BOLLING	C
BOLTON	B	BONNEAU	A	BOOKWOOD	B
BOTETOURT	C	BOURNE	C	BOWMANSVILLE	B/D
BRADDOCK	B	BRADLEY	C	BRANDYWINE	C
BRECKNOCK	B	BREMO	C	BRENTSVILLE	C
BROADWAY	B	BROCKROAD	C	BRUSHY	B
BUCHANAN	C	BUCKHALL	B	BUCKS	B
BUCKTON	B	BUFFSTAT	B	BUGLEY	C/D
BUNCOMBE	A	BURKETOWN	C	BURROWSVILLE	C
CALVERTON	C	CALVIN	C	CAMOCCA	A/D
CANEYVILLE	C	CARBO	C	CARDIFF	B
CAROLINE	C	CARRVALE	D	CARTECAY	C
CATASKA	D	CATHARPIN	C	CATLETT	C/D
CATOCTIN	C	CATPOINT	A	CAVERNS	B
CECIL	B	CHAGRIN	B	CHAPANOKE	C

Soil Name	Hydgrp	Soil Name	Hydgrp	Soil Name	Hydgrp
CHASTAIN	D	CHATUGE	D	CHAVIES	B
CHENNEBY	C	CHESTER	B	CHEWACLA	C
CHICKAHOMINY	D	CHILHOWIE	C	CHINCOTEAGUE	D
CHIPLEY	C	CHISWELL	D	CHRISTIAN	C
CID	C	CLAPHAM*	C	CLEARBROOK	D
CLIFTON	C	CLUBCAF	D	CLYMER	D
COASTAL BEACH	D	CODORUS	C	COLFAX	C
COLLEEN	C	COLVARD	B	COMBS	B
COMUS	B	CONETOE	A	CONGAREE	B
COOSAW	B	COROLLA	D	CORYDON	D
COTACO	C	COURSEY	C	COWEE	B
COXVILLE	D	CRAIGSVILLE	B	CRAVEN	C
CREEDMOOR	C	CROTON	D	CULLEN	C
CULPEPER	C	DALEVILLE	D	DANDRIDGE	D
DAVIDSON	B	DAWHOO VARIANT	B/D	DECATUR	B
DEKALB	C	DELANCO	C	DELOSS	B/D
DERROC	B	DILLARD	C	DOGUE	C
DOROVAN	D	DOTHAN	B	DRAGSTON	C
DRALL	B	DRYPOND	D	DUCKSTON	A/D
DUFFIELD	B	DULLES	D	DUMFRIES	B
DUNBAR	D	DUNNING	D	DUPLIN	C
DURHAM	B	DYKE	B	EBBING	C
EDGEHILL	C	EDNEDYTOWN	B	EDNEYVILLE	B
EDOM	C	ELBERT	D	ELIOAK	C
ELSINBORO	B	EMPORIA	C	ENDCAV	C
ELIOK	C	ELKTON	C/D	ELLIBER	A

Soil Name	Hydgrp	Soil Name	Hydgrp	Soil Name	Hydgrp
ENON	C	ENOTT	C	ERNEST	C
EUBANKS	B	EULONIA	C	EUNOLA	C
EVANSHAM	D	EVARD	B	EVERGREEN	B
EXUM	C	FACEVILLE	B	FAIRFAX	B
FALLSINGTON	B/D	FAUGUIER	C	FAYWOOD	C
FEATHERSTONE	D	FISHERMAN	D	FLATWOODS	C
FLETCHER	B	FLUVANNA	C	FLUVAQUENTS	D
FORESTDALE	D	FORK	C	FRANKSTOWN	B
FREDERICK	B	FRENCH	C	FRIPP	A
GAILA	B	GAINESBORO	C	GALESTOWN	A
GEORGEVILLE	B	GILPIN	C	GLADEHILL	B
GLENELG	B	GLENVILLE	C	GLENWOOD	B
GOLDSBORO	B	GOLDSTON	C	GOLDVEIN	C
GORESVILLE*	B	GREENLEE	B	GRIMSLEY	B
GRITNEY	C	GROSECLOSE	C	GROVER	B
GUERNSEY	C	GULLION	C	GUNSTOCK	C
GUYAN	C	GWINNETT VARIENT	B	HAGERSTOWN	C
HALEWOOD	B	HARTLETON	B	HATBORO	D
HAWKSBILL	B	HAYESVILLE	B	HAYMARKET	D
HAYTER	B	HAYWOOD	B	HAZEL	C
HAZEL CHANNERY	C	HAZELTON	B	HELENA	C
HERNDON	B	HIWASSEE	B	HOADLY	C
HOBUCKEN	D	HOGELAND*	C	HOLLYWOOD	D
HUNTINGTON	B	HYATTSVILLE	B	HYDE	B/D
HYDRAQUENTS	B	INGLEDOVE	B	IREDELL	C/D

Soil Name	Hydgrp	Soil Name	Hydgrp	Soil Name	Hydgrp
IRONGATE	B	IUKA	C	IZAGORA	C
JACKLAND	D	JEDBURG	C	JEFFERSON	B
JOHNS	C	JOHNSTON	D	JUNALUSKA	B
KALMIA	B	KELLY	D	KEMPSVILLE	B
KENANSVILLE	A	KENANSVILLE VARIANT	C	KEYPORT	C
KINKORA	D	KINSTON	B/D	KLEJ	B
KLINESVILLE	C/D	KONNAROCK	C	LAIDIG	C
LAKEHURST VARIANT	A	LAKELAND	A	LANEXA	D
LANSDALE	B	LAROQUE	B	LAWNES	D
LEAF	D	LEAKSVILLE	D	LECK KILL	B
LEEDSVILLE*	B	LEETONIA	C	LEGORE	B
LEHEW	C	LENOIR	D	LEON	B/D
LEVY	D	LEW	B	LEWISBERRY	B
LIBRARY	D	LIGNUM	C	LILY	B
LINDSIDE	C	LITTLEJOE	B	LITZ	C
LLOYD	C	LOBDELL	B	LODI	B
LOUISA	B	LOUISBURG	B	LOWELL	C
LUCKETTS	B	LUCY	A	LUGNUM	C
LUMBEE	B/D	LUNT	C	LYNCHBURG	C
MACOVE	B	MADISON	B	MAGOTHA	D
MANASSAS	B	MANOR	B	MANTACHIE	C
MANTEO	C/D	MARBIE	C	MARGO	B
MARLBORO	B	MARR	B	MARUMSCO	C
MASADA	C	MASSANETTA	B	MASSANUTTEN	B
MATAPEAKE	B	MATNELFLAT	B	MATTAN	D

Soil Name	Hydgrp	Soil Name	Hydgrp	Soil Name	Hydgrp
MATTAPEX	C	MATTAPONI	C	MAURERTOWN	D
MAYODAN	B	MCGARY	C	MCQUEEN	C
MEADOWS	D	MEADOWVILLE	B	MECKLENBURG	C
MEGGETT	D	MELFA	D	MELVIN	D
MILLROCK	A	MINNIEVILLE	C	MIXED ALLUVIUM	D
MOLENA	A	MONACAN	C	MONGLE	C
MONONGAHELA	C	MONTALTO	C	MONTRESSOR*	B
MONTROSS	C	MOOMAW	C	MORRISONVILLE*	B
MORVEN	B	MOUNT LUCAS	C	MT WEATHER*	B
MUCKALEE	D	MUNDEN	B	MURRILL	B
MYATT	D	MYATT VARIANT	D	MYERSVILLE	B
NAHUNTA	C	NANSEMOND	C	NASON	B
NAWNEY	D	NEABSCO	C	NESTORIA	C/D
NEVARC	C	NEWARK	C	NEWBERN	C
NEWFLAT	D	NEWHAN	A	NEWMARC	C
NICHOLOSON	C	NIMMO	D	NIXA	C
NOLICHUCKY	B	NOLIN	B	NOMERVILLE	B
NORFOLK	B	OAKHILL	B	OAKLET	C
OATLANDS	B	OCCOQUAN	B	OCHLOCKONEE	B
OKEETEE	D	OPEQUON	C	ORANGE	D
ORANGEBURG	B	ORENDA	B	ORISKANY	B
OSIER	A/D	OTHELLO	C/D	PACOLET	B
PACTOLUS	A	PAGEBROOK	D	PAMLICO	D
PAMUNKEY	B	PAMUNKEY VARIANT	A	PANORAMA	B
PARKER	B	PARTLOW	D	PASQUOTANK	B/D
PEAKS	C	PEAWICK	D	PENN	C/D

Soil Name	Hydgrp	Soil Name	Hydgrp	Soil Name	Hydgrp
PHILO	B	PHILOMOMT*	B	PINEYWOODS	D
PINKSTON	B	PISGAH	C	POCATY	D
POCOMOKE	B/D	POINDEXTER	B	POLAWANA	A/D
POOLER VARIANT	D	POPE	B	POPLIMENTO	C
PORTERS	B	PORTSMOUTH	B/D	POUNCEY	D
PUNGO	D	PURCELLVILLE	B	PURDY	D
RABUM	B	RAINS	B/D	RAMSEY	D
RAPIDAN	B	RAPPHANNOCK	D	RARITAN	C
RAYNE	B	READINGTON	C	REAVILLE	C
REMLIK	A	RIGLEY	B	RION	B
RIVERVIEW	B	ROANOKE	D	ROHRERSVILLE	D
ROSS	B	ROWLAND	C	RUMFORD	B
RUSHTOWN	A	RUSTON	B	SAFELL	B
SASSAFRAS	B	SASSAFRAS	B	SAUNOOK	B
SAVANNAH	C	SCATTERSVILLE*	C	SCHAFFENAKER	A
SEABROOK	C	SEDGEFIELD	C	SEKIL	B
SENECA	B	SEQUOIA	C	SHELOCTA	B
SHENVAL	B	SHERANDO	B	SHEVA	C
SHOTTOWER	B	SINDION	B	SKETERVILLE	C
SLABTOWN	B	SLAGLE	C	SLICKENS	B
SNICKERSVILLE	B	SPEEDWELL	B	SPESSARD	A
SPIVEY	B	SPOSTSYLVANIA	C	SPRIGGS	C
SPRINGWOOD	B	STANTON	D	STARR	C
STATE	B	STEINSBURG	C	STONEVILLE	B
STUART	C	STUMPTOWN	B	SUCHES	B
SUDLEY	B	SUEQUEHANNA	D	SUFFOLK	B
SUSDLEY	B	SUSQUEHANNA	D	SWAMP	D

Soil Name	Hydgrp	Soil Name	Hydgrp	Soil Name	Hydgrp
SWEETAPPLE	B	SWIMLEY	C	SYCOLINE	D
SYLCO	C	SYLVATUS	D	TALLADEGA	C
TALLAPOOSA	C	TARBORO	A	TATE	B
TATUM	B	TETOTUM	C	THUNDER	B
THURMONT	B	TIDAL MARSH	D	TIMBERVILLE	B
TIOGA	B	TOCCOA	B	TODDSTAV	D
TOMOTLEY	B/D	TOMS	C	TORHUNTA	C
TOTIER	C	TOXAWAY	B/D	TRAPPIST	C
TREGO	B	TRENHOLM	D	TUCKAHOE	B
TUMBLING	B	TURBEVILLE	C	TUSQUITEE	B
TYGART	C	UCHEE	A	UDIFLUVENTS	B
UNISON	B	VANCE	C	VARINA	C
VAUCLUSE	C	VERTREES	B	WADESBORO	B
WAHEE	D	WAKULLA	A	WALLEN	B
WARMINSTER	C	WATAUGA	B	WATEREE	B
WATT	D	WAXPOOL	D	WEAVER	C
WEAVERTON*	C	WEBBTOWN	C	WEDOWEE	B
WEEKSVILLE	B/D	WEHADKEE	D	WEIKERT	C/D
WESTMORELAND	B	WESTON	D	WESTPHALIA	B
WEVERTON	B	WHEELING	B	WHITE STONE	D
WHITEFORD	B	WICKHAM	B	WILKES	C
WOLFGAP	B	WOODINGTON	B/D	WORSHAM	D
WRIGHTSBORO	C	WRYICK	B	WURNO	C
WYRICK	B	YADKIN	C/D	YEMASSEE	C
YEOPIM	B	YORK	C	ZEPP	B
ZION	C	ZOAR	C		

APPENDIX 4B

24-hour Rainfall Data for Virginia

24 HOUR RAINFALL DEPTHS

	YEAR						
COUNTY	1	2	5	10	25	50	100
Accomack	3.0	3.7	4.9	6.0	6.8	7.5	8.5
Albemarle	3.3	4.0	5.0	6.0	7.0	8.0	8.5
Alleghany	2.5	3.0	4.0	5.0	5.5	6.0	7.0
Amelia	3.0	3.5	4.5	5.5	6.0	7.0	7.5
Amherst	3.3	4.0	5.0	6.0	7.0	8.0	8.5
Appomattox	3.0	4.0	4.7	5.8	6.2	7.0	8.0
Augusta	3.0	4.0	4.5	5.5	6.5	7.2	8.0
Bath	2.5	3.0	4.0	5.0	5.5	6.0	7.0
Bedford	3.3	4.0	5.0	5.8	6.8	7.5	8.2
Bland	2.4	2.9	3.9	4.6	5.0	5.8	6.0
Botetourt	3.0	3.5	4.5	5.0	6.0	7.0	7.8
Brunswick	3.0	3.5	4.6	5.6	6.2	7.0	8.0
Buchanan	2.4	2.9	3.7	4.3	4.8	5.5	6.2
Buckingham	3.0	3.5	4.7	5.8	6.3	7.0	8.0
Campbell	3.0	3.7	4.7	5.8	6.3	7.0	7.9
Caroline	2.7	3.5	4.5	5.5	6.0	6.8	7.7
Carroll	2.8	3.2	4.0	4.9	5.2	6.0	6.8
Charles City	3.0	3.5	4.5	5.5	6.2	7.0	7.9
Charlotte	3.0	3.5	4.5	5.5	6.0	7.0	7.7
Chesapeake	3.2	3.8	5.1	6.0	7.0	8.0	8.9
Chesterfield	3.0	3.9	4.5	5.5	6.0	7.0	7.6
Clarke	2.7	3.1	4.5	5.0	6.0	7.0	7.6

COUNTY	YEAR 1	2	5	10	25	50	100
Craig	2.5	3.0	4.0	4.7	5.5	6.0	6.5
Culpeper	3.0	3.6	4.7	5.5	6.5	7.5	8.0
Cumberland	3.0	3.5	4.7	5.8	6.3	7.0	8.0
Dickenson	2.4	2.9	3.7	4.3	4.8	5.5	6.2
Dinwiddie	2.9	3.5	4.6	5.6	6.2	7.0	8.0
Essex	3.0	3.2	4.5	5.5	6.0	6.9	7.8
Fairfax	2.7	3.2	4.5	5.2	6.0	7.0	7.7
Fauquier	2.9	3.5	4.5	5.4	6.5	7.2	7.7
Floyd	3.0	3.3	4.3	5.0	5.5	6.2	7.0
Fluvanna	3.0	3.5	4.7	5.7	6.5	7.0	8.0
Franklin	3.3	3.7	4.7	5.7	6.0	7.0	8.0
Frederick	2.5	3.0	4.0	4.9	5.8	6.5	6.0
Giles	2.4	2.9	3.9	4.7	5.0	5.9	6.0
Gloucester	3.0	3.5	4.7	5.9	6.8	7.4	8.0
Goochland	3.0	3.5	4.7	5.7	6.5	7.0	8.0
Grayson	2.8	3.2	4.0	4.9	5.2	6.0	6.8
Greene	3.3	4.0	5.0	6.0	7.0	8.0	9.0
Greensville	3.0	3.5	4.7	5.6	6.5	7.2	8.0
Halifax	3.0	3.5	4.5	5.5	6.0	7.0	7.5
Hanover	2.8	3.3	4.5	5.5	6.0	6.9	7.6
Henrico	2.8	3.3	4.5	5.5	6.0	7.0	7.8
Henry	3.0	3.5	4.6	5.2	6.0	6.5	7.5
Highland	2.8	3.0	4.0	4.9	5.5	6.0	6.8

YEAR

COUNTY	1	2	5	10	25	50	100
Isle of Wight	2.9	3.7	5.0	5.8	6.6	7.5	8.4
James City	2.8	3.5	4.7	5.8	6.4	7.2	8.0
King and Queen	2.8	3.4	4.5	5.7	6.2	7.0	7.9
King George	2.8	3.2	4.5	5.5	6.0	7.0	7.5
King William	2.8	3.4	4.5	5.7	6.2	7.0	7.9
Lancaster	2.8	3.5	4.7	5.7	6.5	7.2	8.0
Lee	2.7	3.0	3.7	4.5	5.0	5.6	6.0
Loudoun	3.0	3.3	4.5	5.2	6.0	6.9	7.5
Louisa	2.9	3.5	4.7	5.5	6.0	7.0	8.0
Lunenburg	2.9	3.5	4.5	5.5	6.0	7.0	7.5
Madison	3.3	4.0	5.0	6.0	7.0	8.0	9.0
Mathews	3.0	3.6	4.8	5.8	6.6	7.2	8.1
Mecklenburg	2.9	3.5	4.5	5.5	6.0	7.0	7.8
Middlesex	3.0	3.5	4.7	5.7	6.5	7.0	8.0
Montgomery	2.5	3.0	4.0	5.0	5.5	6.0	7.0
Nelson	3.3	4.0	5.0	6.0	7.0	8.0	8.5
New Kent	2.8	3.5	4.5	5.6	6.2	7.0	7.9
Northampton	3.1	3.7	5.0	6.0	6.8	7.6	8.6
Northumberland	2.8	3.5	4.7	5.7	6.5	7.2	8.0
Nottoway	3.0	3.5	4.5	5.5	6.0	7.0	7.9
Orange	3.2	3.5	4.7	5.5	6.5	7.5	8.0
Page	2.5	3.2	4.7	5.5	7.0	7.5	8.5
Patrick	2.8	3.5	4.5	5.0	5.8	6.2	7.3

	YEAR							
COUNTY	1	2	5	10	25	50	100	
Pittsylvania	2.8	3.5	4.5	5.2	6.2	6.7	7.5	
Powhatan	3.0	3.5	4.5	5.5	6.0	7.0	7.5	
Prince Edward	3.0	3.5	4.5	5.5	6.0	7.0	7.8	
Prince George		3.0	3.5	4.7	5.7	6.2	7.0	8.0
Prince William	3.0	3.5	4.S	5.3	6.0	7.0	7.8	
Pulaski	2.5	3.0	4.0	4.8	5.0	6.0	6.5	
Rappahannock	3.0	4.0	4.7	5.7	7.0	8.0	8.5	
Richmond	3.0	3.5	4.5	5.7	6.2	7.0	7.9	
Roanoke	3.0	3.5	4.5	5.0	6.0	6.7	7.5	
Rockbridge	3.0	3.5	4.5	5.5	6.2	7.0	8.0	
Rockingham	3.0	3.5	4.5	5.0	6.0	7.0	8.0	
Russell	2.5	3.0	3.8	4.4	5.0	5.5	6.0	
Scott	2.6	3.0	3.7	4.5	5.0	5.5	6.0	
Shenandoah	2.5	3.0	4.0	5.0	6.0	6.5	7.0	
Smyth	2.6	2.9	3.8	4.5	5.0	5.6	6.0	
Southampton	2.8	3.4	4.8	5.7	6.5	7.2	8.0	
Spotsylvania	3.1	3.5	4.5	5.5	6.0	7.0	7.5	
Stafford	2.9	3.5	4.5	5.5	6.0	7.0	7.5	
Suffolk	3.2	3.7	5.0	6.0	6.7	7.7	8.5	
Surry	2.8	3.4	4.8	5.7	6.5	7.2	8.0	
Sussex	2.8	3.4	4.8	5.7	6.5	7.2	8.0	
Tazewell	2.5	2.9	3.8	4.4	5.0	5.5	6.0	
Virginia Beach	3.0	3.8	5.0	6.0	7.0	8.0	9.0	

YEAR

COUNTY	1	2	5	10	25	50	100
Warren	2.8	3.5	4.5	5.1	6.5	7.0	8.0
Washington	2.6	3.0	3.8	4.5	5.0	5.6	6.0
Westmoreland	2.8	3.5	4.5	5.6	6.1	7.0	7.9
Wise	2.5	2.9	3.8	4.5	5.0	5.5	6.0
Wythe	2.6	2.9	3.8	4.6	5.0	5.8	6.0
York	3.0	3.7	4.8	6.0	6.6	7.4	8.2

APPENDIX 4C

Tabular Listing of Runoff Depths for Curve Numbers

CURVE 55

RUNOFF FOR INCHES OF RAINFALL

Inches \ Tenths	0.0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9
0										
1								0.00	0.00	0.01
2	0.02	0.02	0.04	0.05	0.07	0.08	0.10	0.12	0.15	0.17
3	0.19	0.22	0.25	0.28	0.31	0.35	0.38	0.42	0.45	0.49
4	0.53	0.57	0.61	0.65	0.70	0.74	0.79	0.83	0.88	0.93
5	0.98	1.03	1.08	1.13	1.19	1.24	1.29	1.35	1.40	1.46
6	1.52	1.58	1.63	1.69	1.75	1.81	1.87	1.94	2.00	2.06
7	2.12	2.19	2.25	2.32	2.38	2.45	2.51	2.58	2.65	2.72
8	2.78	2.85	2.92	2.99	3.06	3.13	3.20	3.27	3.34	3.42
9	3.49	3.56	3.63	3.71	3.78	3.85	3.93	4.00	4.08	4.15
10	4.23	4.30	4.38	4.46	4.53	4.61	4.69	4.76	4.84	4.92
11	5.00	5.08	5.15	5.23	5.31	5.39	5.47	5.55	5.63	5.71
12	5.79	5.87	5.95	6.03	6.12	6.20	6.28	6.36	6.44	6.53
13	6.61	6.69	6.77	6.86	6.94	7.02	7.11	7.19	7.27	7.36
14	7.44	7.53	7.61	7.69	7.78	7.86	7.95	8.03	8.12	8.20
15	8.29	8.38	8.46	8.55	8.63	8.72	8.81	8.89	8.98	9.07
16	9.15	9.24	9.33	9.41	9.50	9.59	9.68	9.76	9.85	9.94
17	10.03	10.11	10.20	10.29	10.38	10.47	10.56	10.64	10.73	10.82
18	10.91	11.00	11.09	11.18	11.27	11.36	11.45	11.53	11.62	11.71
19	11.80	11.89	11.98	12.07	12.16	12.25	12.34	12.43	12.52	12.61
20	12.70	12.80	12.89	12.98	13.07	13.16	13.25	13.34	13.43	13.52

NOTE: Runoff value determined by equation $Q = \dfrac{(P-0.2\,S)^2}{P+0.8\,S}$

REFERENCE: National Engineering Handbook, Section 4, HYDROLOGY

RAINFALL

CURVE

55

RUNOFF FOR INCHES OF RAINFALL

Inches \ Tenths	0.0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9
21	13.61	13.70	13.80	13.89	13.98	14.07	14.16	14.25	14.34	14.44
22	14.53	14.62	14.71	14.80	14.90	14.99	15.08	15.17	15.26	15.36
23	15.45	15.54	15.63	15.73	15.82	15.91	16.00	16.10	16.19	16.28
24	16.37	16.47	16.56	16.65	16.75	16.84	16.93	17.03	17.12	17.21
25	17.31	17.40	17.49	17.59	17.68	17.77	17.87	17.96	18.05	18.15
26	18.24	18.33	18.43	18.52	18.62	18.71	18.80	18.90	18.99	19.08
27	19.18	19.27	19.37	19.46	19.56	19.65	19.74	19.84	19.93	20.03
28	20.12	20.22	20.31	20.40	20.50	20.59	20.69	20.78	20.88	20.97
29	21.07	21.16	21.26	21.35	21.45	21.54	21.64	21.73	21.83	21.92
30	22.01	22.11	22.21	22.30	22.40	22.49	22.59	22.68	22.78	22.87
31	22.97	23.06	23.16	23.25	23.35	23.44	23.54	23.63	23.73	23.82
32	23.92	24.02	24.11	24.21	24.30	24.40	24.49	24.59	24.68	24.78
33	24.88	24.97	25.07	25.16	25.26	25.35	25.45	25.55	25.64	25.74
34	25.83	25.93	26.03	26.12	26.22	26.31	26.41	26.51	26.60	26.70
35	26.79	26.89	26.99	27.08	27.18	27.28	27.37	27.47	27.56	27.66
36	27.76	27.85	27.95	28.05	28.14	28.24	28.33	28.43	28.53	28.62
37	28.72	28.82	28.91	29.01	29.11	29.20	29.30	29.40	29.49	29.59
38	29.69	29.78	29.88	29.98	30.07	30.17	30.27	30.36	30.46	30.56
39	30.65	30.75	30.85	30.94	31.04	31.14	31.23	31.33	31.43	31.52
40	31.62	31.72	31.82	31.91	32.01	32.11	32.20	32.30	32.40	32.49

NOTE: Runoff value determined by equation $Q = \frac{(P-0.2\,S)^2}{P+0.8\,S}$

REFERENCE: National Engineering Handbook, Section 4, HYDROLOGY

RAINFALL-RUNOFF DEPTHS FOR SELECTED RUNOFF CURVE NUMBERS

CURVE 95

Inches \ Tenths	0.0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9
0	0.00	0.00	0.01	0.05	0.11	0.17	0.24	0.32	0.40	0.48
1	0.56	0.65	0.74	0.83	0.92	1.01	1.11	1.20	1.30	1.39
2	1.48	1.58	1.68	1.78	1.87	1.97	2.07	2.16	2.26	2.36
3	2.44	2.54	2.64	2.74	2.84	2.93	3.03	3.13	3.23	3.32
4	3.42	3.52	3.62	3.72	3.82	3.92	4.02	4.12	4.21	4.31
5	4.41	4.51	4.61	4.71	4.81	4.91	5.01	5.11	5.21	5.30
6	5.40	5.50	5.60	5.70	5.80	5.90	6.00	6.10	6.20	6.30
7	6.40	6.50	6.60	6.70	6.80	6.90	7.00	7.10	7.19	7.29
8	7.39	7.49	7.59	7.69	7.79	7.89	7.99	8.09	8.19	8.29
9	8.39	8.49	8.59	8.69	8.79	8.89	8.99	9.09	9.19	9.29
10	9.39	9.49	9.59	9.69	9.79	9.89	9.99	10.09	10.19	10.29
11	10.39	10.49	10.59	10.69	10.79	10.89	10.99	11.09	11.19	11.29
12	11.39	11.49	11.59	11.69	11.79	11.89	11.99	12.09	12.19	12.29

CURVE 96

Inches \ Tenths	0.0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9
0	0.00	0.01	0.04	0.08	0.14	0.22	0.29	0.37	0.46	0.55
1	0.64	0.73	0.82	0.91	1.01	1.10	1.20	1.29	1.39	1.48
2	1.58	1.68	1.77	1.87	1.97	2.07	2.17	2.26	2.36	2.46
3	2.56	2.66	2.76	2.86	2.95	3.05	3.15	3.25	3.35	3.45
4	3.55	3.65	3.75	3.85	3.95	4.05	4.14	4.24	4.34	4.44
5	4.54	4.64	4.74	4.84	4.94	5.04	5.14	5.24	5.34	5.44
6	5.54	5.64	5.74	5.84	5.93	6.03	6.13	6.23	6.33	6.43
7	6.53	6.63	6.73	6.83	6.93	7.03	7.13	7.23	7.33	7.43
8	7.53	7.63	7.73	7.83	7.93	8.03	8.13	8.23	8.33	8.43
9	8.53	8.63	8.73	8.83	8.93	9.03	9.13	9.23	9.33	9.43
10	9.53	9.63	9.73	9.83	9.93	10.03	10.13	10.23	10.32	10.42
11	10.52	10.62	10.72	10.82	10.92	11.02	11.12	11.22	11.32	11.42
12	11.52	11.62	11.72	11.82	11.92	12.02	12.12	12.22	12.32	12.42

CURVE 97

Inches \ Tenths	0.0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9
0	0.00	0.00	0.04	0.11	0.18	0.26	0.35	0.44	0.53	0.62
1	0.71	0.81	0.90	1.00	1.10	1.19	1.29	1.39	1.49	1.58
2	1.68	1.78	1.88	1.98	2.08	2.17	2.27	2.37	2.47	2.57
3	2.67	2.77	2.87	2.97	3.07	3.16	3.26	3.36	3.46	3.56
4	3.66	3.76	3.86	3.96	4.06	4.16	4.26	4.36	4.46	4.56
5	4.66	4.76	4.86	4.96	5.06	5.16	5.26	5.36	5.46	5.55
6	5.65	5.75	5.85	5.95	6.05	6.15	6.25	6.35	6.45	6.55
7	6.65	6.75	6.85	6.95	7.05	7.15	7.25	7.35	7.45	7.55
8	7.65	7.75	7.85	7.95	8.05	8.15	8.25	8.35	8.45	8.55
9	8.65	8.75	8.85	8.95	9.05	9.15	9.25	9.35	9.45	9.55
10	9.65	9.75	9.85	9.95	10.05	10.15	10.25	10.35	10.45	10.55
11	10.65	10.75	10.85	10.95	11.05	11.15	11.25	11.35	11.45	11.55
12	11.65	11.75	11.85	11.95	12.05	12.15	12.25	12.35	12.45	12.55

Exhibit 2-7A

RAINFALL-RUNOFF DEPTHS FOR SELECTED RUNOFF CURVE NUMBERS

CURVE 92

Inches \ Tenths	0.0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9
0	0.0	0.00	0.00	0.02	0.05	0.09	0.14	0.20	0.26	0.33
1	0.41	0.48	0.56	0.64	0.72	0.80	0.89	0.97	1.06	1.15
2	1.24	1.33	1.42	1.51	1.60	1.69	1.78	1.88	1.98	2.07
3	2.16	2.26	2.35	2.45	2.54	2.64	2.74	2.83	2.93	3.02
4	3.12	3.22	3.31	3.41	3.51	3.60	3.70	3.80	3.90	4.00
5	4.09	4.19	4.29	4.39	4.48	4.58	4.68	4.78	4.88	4.97
6	5.07	5.17	5.27	5.37	5.47	5.56	5.66	5.76	5.86	5.96
7	6.06	6.15	6.25	6.35	6.45	6.55	6.65	6.75	6.85	6.95
8	7.05	7.15	7.25	7.34	7.44	7.54	7.64	7.74	7.84	7.94
9	8.04	8.14	8.24	8.33	8.43	8.53	8.63	8.73	8.83	8.93
10	9.03	9.13	9.23	9.33	9.43	9.53	9.63	9.73	9.83	9.93
11	10.03	10.13	10.23	10.33	10.42	10.52	10.62	10.72	10.82	10.92
12	11.02	11.12	11.22	11.32	11.42	11.52	11.62	11.72	11.82	11.92

CURVE 93

Inches \ Tenths	0.0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9
0	0.00	0.00	0.00	0.03	0.06	0.11	0.17	0.23	0.30	0.38
1	0.45	0.53	0.61	0.70	0.78	0.87	0.96	1.04	1.13	1.22
2	1.32	1.41	1.50	1.59	1.69	1.78	1.88	1.97	2.06	2.16
3	2:26	2.35	2.45	2.54	2.64	2.74	2.83	2.93	3.03	3.12
4	3.22	3.32	3.42	3.51	3.61	3.71	3.81	3.91	4.00	4.10
5	4.20	4.30	4.40	4.50	4.59	4.69	4.79	4.89	4.99	5.09
6	5.18	5.28	5.38	5.48	5.58	5.68	5.78	5.88	5.97	6.07
7	6.17	6.27	6.37	6.47	6.57	6.67	6.77	6.87	6.97	7.07
8	7.17	7.26	7.36	7.46	7.56	7.66	7.76	7.86	7.96	8.06
9	8.16	8.26	8.36	8.46	8.56	8.66	8.75	8.85	8.95	9.05
10	9.15	9.25	9.35	9.45	9.55	9.65	9.75	9.85	9.95	10.05
11	10.15	10.25	10.35	10.45	10.55	10.65	10.75	10.85	10.95	11.05
12	11.14	11.24	11.34	11.44	11.54	11.64	11.74	11.84	11.94	12.04

CURVE 94

Inches \ Tenths	0.0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9
0	0.00	0.00	0.01	0.04	0.08	0.14	0.20	0.27	0.34	0.42
1	0.50	0.58	0.67	0.76	0.84	0.93	1.02	1.11	1.21	1.30
2	1.39	1.49	1.58	1.68	1.77	1.87	1.97	2.06	2.15	2.24
3	2.35	2.44	2.54	2.64	2.73	2.83	2.93	3.03	3.13	3.22
4	3.32	3.42	3.52	3.62	3.72	3.81	3.91	4.01	4.11	4.21
5	4.30	4.40	4.50	4.60	4.70	4.80	4.90	5.00	5.10	5.19
6	5.29	5.39	5.49	5.59	5.69	5.79	5.89	5.99	6.09	6.18
7	6.28	6.38	6.48	6.58	6.68	6.78	6.88	6.98	7.08	7.18
8	7.27	7.37	7.47	7.57	7.67	7.77	7.87	7.97	8.07	8.17
9	8.27	8.37	8.47	8.57	8.67	8.77	8.87	8.97	9.07	9.17
10	9.27	9.37	9.47	9.57	9.67	9.77	9.87	9.97	10.07	10.17
11	10.27	10.37	10.47	10.57	10.66	10.76	10.86	10.96	11.06	11.16
12	11.26	11.36	11.46	11.56	11.66	11.76	11.86	11.96	12.06	12.16

Exhibit 2-7A

RAINFALL-RUNOFF DEPTHS FOR SELECTED RUNOFF CURVE NUMBERS

CURVE 89

Inches \ Tenths	0.0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9
0	0.00	0.00	0.00	0.00	0.02	0.04	0.08	0.12	0.17	0.22
1	0.28	0.35	0.41	0.48	0.55	0.63	0.71	0.78	0.86	0.94
2	1.03	1.11	1.19	1.28	1.37	1.46	1.54	1.63	1.72	1.81
3	1.90	1.99	2.08	2.17	2.26	2.36	2.45	2.54	2.64	2.73
4	2.82	2.92	3.01	3.11	3.20	3.30	3.39	3.49	3.58	3.68
5	3.77	3.87	3.96	4.06	4.16	4.25	4.35	4.45	4.54	4.64
6	4.74	4.83	4.93	5.02	5.12	5.22	5.32	5.42	5.51	5.61
7	5.71	5.80	5.90	6.00	6.10	6.20	6.30	6.39	6.49	6.59
8	6.69	6.79	6.88	6.98	7.08	7.18	7.28	7.38	7.47	7.57
9	7.67	7.77	7.87	7.97	8.06	8.16	8.26	8.36	8.46	8.56
10	8.66	8.76	8.86	8.95	9.05	9.15	9.25	9.35	9.45	9.55
11	9.65	9.75	9.84	9.94	10.04	10.14	10.24	10.34	10.44	10.54
12	10.64	10.73	10.83	10.93	11.03	11.13	11.23	11.33	11.43	11.53

CURVE 90

Inches \ Tenths	0.0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9
0	0.00	0.00	0.00	0.01	0.03	0.06	0.10	0.15	0.20	0.26
1	0.32	0.39	0.46	0.53	0.61	0.69	0.77	0.85	0.93	1.01
2	1.10	1.18	1.27	1.35	1.44	1.53	1.62	1.71	1.80	1.89
3	1.99	2.08	2.17	2.26	2.36	2.45	2.54	2.64	2.73	2.83
4	2.92	3.02	3.11	3.20	3.30	3.40	3.49	3.59	3.69	3.78
5	3.88	3.97	4.07	4.17	4.26	4.36	4.46	4.56	4.65	4.75
6	4.85	4.95	5.04	5.14	5.24	5.34	5.44	5.54	5.63	5.73
7	5.83	5.92	6.02	6.12	6.21	6.31	6.41	6.51	6.61	6.71
8	6.81	6.91	7.01	7.11	7.20	7.30	7.40	7.50	7.60	7.70
9	7.79	7.89	7.99	8.09	8.19	8.29	8.39	8.49	8.58	8.68
10	8.78	8.88	8.98	9.08	9.18	9.28	9.38	9.48	9.57	9.67
11	9.77	9.87	9.97	10.07	10.17	10.27	11.37	10.47	10.57	10.67
12	10.77	10.86	10.96	11.06	11.16	11.26	11.36	11.46	11.56	11.66

CURVE 91

Inches \ Tenths	0.0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9
0	0.00	0.00	0.00	0.01	0.03	0.07	0.12	0.17	0.23	0.29
1	0.36	0.43	0.50	0.58	0.66	0.74	0.82	0.91	0.99	1.08
2	1.17	1.25	1.34	1.43	1.52	1.61	1.70	1.80	1.89	1.98
3	2.07	2.16	2.26	2.35	2.44	2.54	2.63	2.73	2.83	2.92
4	3.02	3.11	3.21	3.30	3.40	3.50	3.59	3.69	3.79	3.89
5	3.99	4.08	4.17	4.27	4.37	4.47	4.56	4.66	4.76	4.86
6	4.96	5.05	5.15	5.25	5.34	5.44	5.54	5.64	5.74	5.84
7	5.94	6.04	6.14	6.24	6.34	6.44	6.53	6.63	6.73	6.83
8	6.93	7.03	7.13	7.23	7.33	7.43	7.52	7.62	7.72	7.82
9	7.92	8.02	8.12	8.22	8.31	8.41	8.51	8.61	8.71	8.81
10	8.91	9.01	9.11	9.21	9.31	9.41	9.51	9.61	9.71	9.80
11	9.90	10.00	10.10	10.20	10.30	10.40	10.50	10.60	10.70	10.80
12	10.89	10.99	11.09	11.19	11.29	11.39	11.49	11.59	11.69	11.79

Exhibit 2-7A

RAINFALL-RUNOFF DEPTHS FOR SELECTED RUNOFF CURVE NUMBERS

CURVE 86

Inches \ Tenths	0.0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9
0	0.00	0.00	0.00	0.00	0.00	0.02	0.04	0.07	0.11	0.15
1	0.20	0.25	0.30	0.36	0.42	0.49	0.56	0.63	0.70	0.77
2	0.85	0.92	1.00	1.08	1.16	1.24	1.33	1.41	1.49	1.57
3	1.66	1.74	1.83	1.92	2.00	2.09	2.18	2.27	2.36	2.45
4	2.54	2.63	2.72	2.81	2.91	3.00	3.09	3.18	3.28	3.37
5	3.46	3.56	3.65	3.74	3.83	3.93	4.02	4.12	4.21	4.31
6	4.40	4.50	4.59	4.69	4.78	4.88	4.98	5.07	5.17	5.27
7	5.36	5.46	5.55	5.65	5.74	5.84	5.94	6.03	6.13	6.22
8	6.32	6.42	6.52	6.61	6.71	6.81	6.91	7.01	7.10	7.20
9	7.30	7.40	7.50	7.60	7.70	7.79	7.88	7.98	8.08	8.18
10	8.27	8.37	8.47	8.57	8.67	8.76	8.86	8.96	9.06	9.16
11	9.25	9.35	9.45	9.55	9.65	9.75	9.85	9.95	10.05	10.14
12	10.24	10.34	10.44	10.54	10.63	10.73	10.83	10.93	11.03	11.13

CURVE 87

Inches \ Tenths	0.0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9
0	0.00	0.00	0.00	0.00	0.00	0.02	0.05	0.08	0.13	0.17
1	0.22	0.28	0.34	0.40	0.47	0.54	0.61	0.69	0.75	0.83
2	0.91	0.98	1.06	1.15	1.23	1.31	1.39	1.47	1.56	1.65
3	1.74	1.83	1.92	2.01	2.09	2.18	2.27	2.36	2.45	2.55
4	2.64	2.73	2.82	2.91	3.01	3.10	3.19	3.28	3.38	3.47
5	3.57	3.67	3.76	3.85	3.95	4.04	4.13	4.23	4.32	4.42
6	4.51	4.61	4.71	4.80	4.90	5.00	5.09	5.19	5.29	5.38
7	5.48	5.58	5.67	5.77	5.87	5.97	6.06	6.16	6.25	6.35
8	6.45	6.55	6.64	6.74	5.84	6.94	7.03	7.13	7.23	7.33
9	7.43	7.52	7.62	7.72	7.81	7.91	8.01	8.11	8.21	8.31
10	8.41	8.51	8.61	8.70	8.80	8.90	9.00	9.10	9.20	9.30
11	9.39	9.49	9.59	9.69	9.78	9.88	9.98	10.08	10.18	10.28
12	10.38	10.47	10.57	10.67	10.77	10.87	10.97	11.07	11.17	11.27

CURVE 88

Inches \ Tenths	0.0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9
0	0.00	0.00	0.00	0.00	0.01	0.03	0.06	0.10	0.15	0.20
1	0.25	0.31	0.38	0.44	0.51	0.58	0.65	0.73	0.81	0.88
2	0.96	1.04	1.12	1.21	1.30	1.38	1.47	1.55	1.64	1.73
3	1.82	1.90	1.99	2.08	2.17	2.26	2.35	2.44	2.54	2.63
4	2.72	2.82	2.91	3.00	3.10	3.19	3.29	3.38	3.47	3.57
5	3.66	3.76	3.85	3.95	4.05	4.14	4.23	4.33	4.42	4.52
6	4.62	4.72	4.81	4.91	5.01	5.10	5.20	5.29	5.39	5.49
7	5.58	5.68	5.78	5.88	5.97	6.07	6.17	6.27	6.36	6.46
8	6.56	6.66	6.75	6.85	6.95	7.05	7.15	7.24	7.34	7.44
9	7.54	7.64	7.73	7.83	7.93	8.03	8.13	8.23	8.33	8.43
10	8.53	8.63	8.73	8.83	8.93	9.03	9.12	9.22	9.32	9.42
11	9.51	9.61	9.71	9.81	9.91	10.01	10.11	10.20	10.30	10.40
12	10.50	10.39	10.69	10.79	10.89	10.99	11.09	11.19	11.29	11.39

Exhibit 2-7A

RAINFALL-RUNOFF DEPTHS FOR SELECTED RUNOFF CURVE NUMBERS

CURVE 83

Inches \ Tenths	0.0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9
0	0.00	0.00	0.00	0.00	0.00	0.00	0.02	0.04	0.06	0.09
1	0.13	0.17	0.22	0.27	0.32	0.38	0.44	0.50	0.56	0.63
2	0.70	0.76	0.83	0.91	0.98	1.06	1.13	1.21	1.29	1.37
3	1.45	1.53	1.61	1.69	1.77	1.86	1.94	2.03	2.11	2.20
4	2.29	2.37	2.46	2.55	2.64	2.73	2.82	2.91	3.00	3.08
5	3.17	3.26	3.35	3.45	3.54	3.63	3.72	3.81	3.90	4.00
6	4.09	4.18	4.28	4.37	4.46	4.55	4.65	4.74	4.84	4.93
7	5.02	5.12	5.21	5.31	5.40	5.50	5.60	5.69	5.78	5.88
8	5.98	6.07	6.17	6.26	6.36	6.45	6.55	6.65	6.74	6.84
9	6.93	7.03	7.13	7.22	7.32	7.42	7.51	7.61	7.71	7.80
10	7.90	8.00	8.09	8.19	8.29	8.39	8.48	8.58	8.68	8.77
11	8.87	8.97	9.07	9.16	9.26	9.36	9.46	9.56	9.65	9.75
12	9.85	9.94	10.04	10.14	10.24	10.34	10.44	10.53	10.63	10.73

CURVE 84

Inches \ Tenths	0.0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9
0	0.00	0.00	0.00	0.00	0.00	0.01	0.02	0.05	0.08	0.11
1	0.15	0.20	0.25	0.30	0.35	0.41	0.48	0.54	0.61	0.68
2	0.74	0.82	0.89	0.97	1.04	1.12	1.20	1.28	1.36	1.44
3	1.52	1.60	1.68	1.77	1.85	1.94	2.03	2.11	2.20	2.29
4	2.37	2.46	2.55	2.64	2.73	2.82	2.91	3.00	3.09	3.18
5	3.27	3.37	3.46	3.55	3.64	3.73	3.82	3.92	4.01	4.11
6	4.20	4.29	4.39	4.48	4.58	4.67	4.76	4.86	4.95	5.05
7	5.14	5.24	5.33	5.43	5.52	5.62	5.71	5.81	5.91	6.00
8	6.10	6.20	6.30	6.39	6.48	6.58	6.68	6.77	6.87	6.97
9	7.06	7.16	7.26	7.35	7.45	7.55	7.65	7.74	7.84	7.94
10	8.03	8.13	8.23	8.33	8.42	8.52	8.61	8.71	8.81	8.91
11	9.01	9.10	9.20	9.30	9.40	9.50	9.60	9.69	9.79	9.89
12	9.99	10.09	10.19	10.28	10.38	10.48	10.57	10.67	10.77	10.87

CURVE 85

Inches \ Tenths	0.0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9
0	0.00	0.00	0.00	0.00	0.00	0.01	0.03	0.06	0.09	0.13
1	0.18	0.22	0.28	0.33	0.39	0.45	0.52	0.59	0.65	0.73
2	0.80	0.87	0.95	1.02	1.10	1.18	1.26	1.34	1.42	1.51
3	1.59	1.68	1.76	1.85	1.93	2.02	2.11	2.20	2.28	2.37
4	2.46	2.55	2.64	2.73	2.82	2.91	3.00	3.09	3.19	3.28
5	3.37	3.47	3.56	3.65	3.74	3.84	3.93	4.03	4.12	4.21
6	4.31	4.40	4.50	4.59	4.69	4.78	4.87	4.97	5.06	5.16
7	5.26	5.35	5.45	5.55	5.64	5.74	5.84	5.93	6.03	6.12
8	6.22	6.32	6.41	6.50	6.60	6.70	6.80	6.90	6.99	7.09
9	7.19	7.28	7.38	7.48	7.57	7.67	7.77	7.87	7.97	8.06
10	8.16	8.26	8.35	8.45	8.55	8.65	8.75	8.84	8.94	9.04
11	9.14	9.24	9.33	9.43	9.53	9.63	9.73	9.82	9.92	10.02
12	10.12	10.22	10.32	10.42	10.51	10.61	10.71	10.81	10.91	11.01

Exhibit 2-7A

RAINFALL-RUNOFF DEPTHS FOR SELECTED RUNOFF CURVE NUMBERS

CURVE 80

Inches \ Tenths	0.0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9
0	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.01	0.03	0.05
1	0.08	0.11	0.15	0.19	0.24	0.29	0.34	0.39	0.44	0.50
2	0.56	0.62	0.68	0.75	0.82	0.89	0.96	1.03	1.10	1.17
3	1.25	1.33	1.40	1.48	1.56	1.64	1.72	1.80	1.88	1.96
4	2.04	2.12	2.20	2.29	2.38	2.46	2.55	2.63	2.72	2.81
5	2.89	2.98	3.07	3.16	3.25	3.34	3.43	3.52	3.61	3.69
6	3.78	3.87	3.96	4.05	4.14	4.23	4.32	4.42	4.51	4.60
7	4.69	4.79	4.88	4.97	5.06	5.16	5.25	5.34	5.44	5.53
8	5.62	5.72	5.81	5.91	6.00	6.09	6.19	6.28	6.38	6.47
9	6.57	6.66	6.76	6.85	6.95	7.04	7.14	7.23	7.33	7.43
10	7.52	7.62	7.71	7.81	7.90	8.00	8.10	8.19	8.29	8.38
11	8.48	8.58	8.67	8.77	8.87	8.97	9.06	9.16	9.26	9.35
12	9.45	9.55	9.65	9.75	9.84	9.94	10.04	10.14	10.24	10.33

CURVE 81

Inches \ Tenths	0.0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9
0	0.00	0.00	0.00	0.00	0.00	0.00	0.01	0.02	0.04	0.07
1	0.10	0.13	0.17	0.21	0.26	0.31	0.36	0.42	0.48	0.54
2	0.60	0.66	0.73	0.80	0.87	0.94	1.01	1.09	1.16	1.23
3	1.31	1.39	1.47	1.55	1.63	1.71	1.79	1.87	1.95	2.03
4	2.12	[illegible]	2.29	2.37	[illegible]	2.55	2.63	2.72	2.81	2.89
5	2.98	3.07	3.16	3.25	3.34	3.43	3.52	3.61	3.70	[illegible]
6	3.88	3.97	4.06	4.16	4.25	4.34	4.43	4.52	4.61	4.71
7	[illegible]	[illegible]	4.99	5.08	5.17	5.27	5.36	5.46	5.55	5.64
8	[illegible]	[illegible]	5.93	6.02	6.11	6.20	6.29	6.39	6.49	6.59
9	6.69	6.79	6.88	6.97	7.06	7.15	7.25	7.35	7.45	7.55
10	7.64	7.74	7.84	7.93	8.03	8.12	8.21	8.31	8.41	8.51
11	[illegible]	[illegible]	8.79	8.88	8.98	9.08	9.18	9.28	9.38	9.48
12	9.58	9.68	9.77	9.86	9.95	10.05	10.15	10.25	10.35	10.45

CURVE 82

Inches \ Tenths	0.0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9
0	0.00	0.00	0.00	0.00	0.00	0.00	0.01	0.03	0.05	0.08
1	0.11	0.15	0.19	0.24	0.29	0.34	0.40	0.46	0.52	0.58
2	0.65	0.72	0.78	0.85	0.92	0.99	1.06	1.14	1.22	1.30
3	1.38	1.46	1.53	1.61	1.69	1.77	1.86	1.94	2.02	2.11
4	[illegible]	[illegible]	[illegible]	2.46	2.55	2.64	2.73	2.81	2.90	2.99
5	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	3.71	3.80	[illegible]
6	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	4.63	4.73	[illegible]
7	4.92	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	5.48	[illegible]	5.67	[illegible]
8	[illegible]	[illegible]	6.05	6.14	6.24	6.33	6.43	6.53	6.62	6.71
9	6.81	6.91	7.01	7.11	7.21	7.30	7.40	7.49	7.59	7.68
10	7.77	[illegible]	7.97	8.07	8.17	8.26	8.36	8.46	8.55	8.64
11	8.74	8.84	8.94	9.04	9.14	9.23	9.33	9.42	9.52	9.61
12	9.71	9.81	9.90	10.00	10.10	10.20	10.29	10.39	10.49	10.59

Exhibit 2-7A

RAINFALL-RUNOFF DEPTHS FOR SELECTED RUNOFF CURVE NUMBERS

CURVE 77

Inches \ Tenths	0.0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9
0	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.01	0.03
1	0.05	0.07	0.10	0.14	0.18	0.22	0.26	0.30	0.34	0.39
2	0.45	0.50	0.56	0.62	0.68	0.74	0.80	0.86	0.93	1.00
2yr 3	1.07	1.14	1.21	1.28	1.35	1.43	1.50	1.57	1.65	1.73
5yr 4	1.81	1.89	1.97	2.05	2.13	2.21	2.29	2.37	2.45	2.53
10yr 5	2.62	2.70	2.79	2.87	2.96	3.04	3.13	3.22	3.30	3.39
50yr 6	3.48	3.56	3.65	3.74	3.83	3.92	4.00	4.09	4.18	4.27
100yr 7	4.36	4.45	4.54	4.63	4.72	4.81	4.90	5.00	5.09	5.18
8	5.27	5.36	5.45	5.55	5.64	5.73	5.82	5.92	6.01	6.10
9	6.19	6.29	6.38	6.47	6.57	6.66	6.76	6.85	6.94	7.04
10	7.13	7.23	7.32	7.42	7.51	7.60	7.70	7.79	7.89	7.98
11	8.08	8.18	8.27	8.37	8.46	8.55	8.65	8.75	8.84	8.94
12	9.03	9.13	9.23	9.32	9.42	9.51	9.61	9.71	9.81	9.90

CURVE 78

Inches \ Tenths	0.0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9
0	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.01	0.02	0.03
1	0.06	0.09	0.12	0.15	0.19	0.23	0.27	0.32	0.37	0.42
2	0.48	0.54	0.60	0.66	0.72	0.79	0.86	0.93	0.99	1.06
3	1.13	1.20	1.27	1.35	1.43	1.50	1.58	1.65	1.73	1.81
4	1.89	1.97	2.05	2.13	2.22	2.30	2.38	2.46	2.54	2.63
5	2.72	2.81	2.89	2.98	3.06	3.15	3.24	3.32	3.41	3.50
6	3.59	3.67	3.76	3.85	3.94	4.03	4.12	4.22	4.31	4.39
7	4.48	4.58	4.67	4.76	4.85	4.94	5.03	5.12	5.22	5.31
8	5.40	5.49	5.58	5.67	5.77	5.86	5.95	6.05	6.14	6.24
9	6.33	6.43	6.52	6.61	6.71	6.80	6.90	6.99	7.09	7.18
10	7.27	7.37	7.46	7.56	7.65	7.75	7.84	7.94	8.04	8.13
11	8.23	8.33	8.42	8.52	8.61	8.71	8.80	8.90	8.99	9.09
12	9.19	9.28	9.37	9.47	9.56	9.66	9.76	9.86	9.95	10.05

CURVE 79

Inches \ Tenths	0.0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9
0	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.01	0.03	0.05
1	0.07	0.10	0.13	0.17	0.21	0.26	0.31	0.36	0.41	0.46
2	0.52	0.58	0.64	0.70	0.77	0.84	0.91	0.98	1.05	1.12
3	1.19	1.26	1.34	1.42	1.49	1.56	1.64	1.72	1.80	1.88
4	1.96	2.04	2.13	2.21	2.29	2.38	2.46	2.55	2.63	2.72
5	2.80	2.89	2.98	3.07	3.15	3.24	3.32	3.41	3.50	3.59
6	3.68	3.77	3.86	3.95	4.04	4.13	4.22	4.31	4.40	4.49
7	4.58	4.68	4.77	4.86	4.95	5.04	5.13	5.23	5.32	5.42
8	5.51	5.60	5.68	5.78	5.88	5.98	6.07	6.16	6.26	6.35
9	6.45	6.54	6.63	6.73	6.82	6.92	7.01	7.11	7.20	7.30
10	7.39	7.49	7.58	7.68	7.77	7.87	7.97	8.06	8.16	8.25
11	8.35	8.44	8.54	8.64	8.74	8.83	8.92	9.02	9.12	9.22
12	9.31	9.40	9.50	9.60	9.70	9.79	9.89	9.98	10.08	10.18

Exhibit 2-7A

RAINFALL-RUNOFF DEPTHS FOR SELECTED RUNOFF CURVE NUMBERS

CURVE 74

Inches \ Tenths	0.0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9
0									0	0.01
1	0.02	0.04	0.06	0.09	0.12	0.15	0.18	0.21	0.27	0.31
2	0.35	0.40	0.45	0.50	0.55	0.61	0.67	0.73	0.79	0.85
2yr 3	0.91	0.97	1.04	1.11	1.18	1.25	1.32	1.38	1.45	1.52
5yr 4	1.60	1.68	1.75	1.83	1.90	1.98	2.06	2.13	2.21	2.29
10yr 5	2.37	2.45	2.53	2.61	2.69	2.78	2.86	2.94	3.02	3.10
50yr 6	3.19	3.28	3.36	3.45	3.53	3.62	3.70	3.79	3.88	3.96
100yr. 7	4.05	4.14	4.23	4.31	4.39	4.48	4.57	4.66	4.75	4.84
8	4.93	5.02	5.11	5.20	5.29	5.38	5.47	5.56	5.65	5.75
9	5.84	5.93	6.02	6.11	6.20	6.29	6.38	6.47	6.56	6.65
10	6.75	6.84	6.94	7.03	7.12	7.21	7.30	7.40	7.50	7.60
11	7.69	7.79	7.88	7.98	8.07	8.16	8.25	8.35	8.44	8.54
12	8.63	8.73	8.82	8.91	9.00	9.10	9.20	9.30	9.40	9.49

CURVE 75

Inches \ Tenths	0.0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9
0	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.01
1	0.03	0.05	0.07	0.10	0.13	0.16	0.20	0.24	0.28	0.33
2	0.38	0.43	0.48	0.53	0.59	0.65	0.71	0.77	0.83	0.89
3	0.95	1.02	1.09	1.16	1.23	1.30	1.37	1.44	1.52	1.59
4	1.67	1.74	1.82	1.89	1.97	2.04	2.12	2.20	2.28	2.36
5	2.44	2.52	2.61	2.69	2.77	2.85	2.94	3.02	3.11	3.19
6	3.27	3.36	3.45	3.53	3.62	3.71	3.79	3.88	3.97	4.06
7	4.15	4.24	4.32	4.40	4.49	4.58	4.67	4.76	4.86	4.95
8	5.04	5.13	5.22	5.31	5.40	5.49	5.58	5.67	5.76	5.85
9	5.94	6.04	6.13	6.22	6.32	6.41	6.50	6.59	6.68	6.78
10	6.87	6.96	7.05	7.15	7.24	7.34	7.43	7.53	7.63	7.73
11	7.82	7.91	8.00	8.10	8.19	8.29	8.38	8.47	8.56	8.66
12	8.76	[illegible]	8.95	9.04	9.14	9.23	9.32	9.42	9.51	9.61

CURVE 76

Inches \ Tenths	0.0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9
0	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.01	0.02
1	0.04	[illegible]	0.09	0.11	0.15	0.19	0.23	0.27	0.31	0.36
2	0.41	0.46	0.51	[illegible]	0.63	0.69	0.75	0.81	0.88	0.95
3	1.01	1.08	1.15	1.22	1.29	1.36	1.43	1.51	1.58	1.66
4	1.74	1.81	1.89	1.97	2.05	2.13	2.21	2.29	2.37	2.46
5	2.54	2.62	2.70	2.78	2.87	2.95	3.04	3.12	3.21	3.29
6	3.38	3.47	3.55	3.64	3.73	3.81	3.90	3.99	4.08	4.17
7	4.26	4.35	4.44	4.53	4.62	4.71	4.80	4.89	4.98	5.07
8	5.16	5.25	5.34	5.43	5.52	5.61	5.70	5.80	5.89	5.98
9	6.07	6.17	6.26	6.35	6.45	6.54	6.63	6.73	6.82	6.91
10	7.01	7.10	7.19	7.28	7.37	7.47	7.57	7.66	7.75	7.85
11	7.94	8.04	8.14	8.23	8.33	8.42	8.51	8.61	8.71	8.80
12	8.90	8.99	9.08	9.18	9.28	9.37	9.47	9.56	9.66	9.76

Exhibit 2-7A

RAINFALL-RUNOFF DEPTHS FOR SELECTED RUNOFF CURVE NUMBERS

CURVE 71

Inches \ Tenths	0.0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9
0								0.00	0.00	0.00
1	0.01	0.02	0.03	0.05	0.07	0.10	0.13	0.16	0.19	0.22
2	0.26	0.30	0.34	0.39	0.44	0.49	0.54	0.59	0.64	0.70
3	0.76	0.82	0.88	0.94	1.00	1.06	1.13	1.19	1.26	1.33
4	1.39	1.46	1.53	1.60	1.67	1.74	1.81	1.89	1.97	2.04
5	2.11	2.19	2.27	2.34	2.42	2.50	2.58	2.66	2.74	2.82
6	2.90	2.98	3.06	3.14	3.22	3.30	3.38	3.47	3.55	3.63
7	3.72	3.80	3.88	3.97	4.06	4.14	4.23	4.32	4.40	4.49
8	4.58	4.67	4.75	4.82	4.93	5.02	5.10	5.19	5.28	5.37
9	5.46	5.55	5.64	5.73	5.82	5.90	5.99	6.08	6.17	6.27
10	6.36	6.45	6.54	6.63	6.72	6.81	6.90	6.99	7.08	7.18
11	7.27	7.36	7.45	7.54	7.63	7.73	7.82	7.91	8.01	8.10
12	8.19	8.28	8.38	8.47	8.56	8.65	8.75	8.84	8.94	9.03

CURVE 72

Inches \ Tenths	0.0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9
1	0.01	0.02	0.04	0.06	0.08	0.11	0.14	0.17	0.21	0.25
2	0.29	0.33	0.38	0.43	0.48	0.53	0.58	0.63	0.69	0.75
3	0.81	0.87	0.93	0.99	1.05	1.12	1.19	1.26	1.32	1.39
4	1.46	1.53	1.60	1.68	1.75	1.82	1.89	1.97	2.05	2.12
5	2.19	2.27	2.35	2.43	2.51	2.59	2.67	2.75	2.83	2.91
6	2.99	3.08	3.16	3.24	3.32	3.41	3.49	3.57	3.66	3.75
7	3.83	3.91	4.00	4.08	4.17	4.26	4.35	4.44	4.52	4.61
8	4.69	4.78	4.87	4.96	5.05	5.14	5.23	5.31	5.40	5.49
9	5.58	5.67	5.76	5.85	5.94	6.03	6.13	6.22	6.31	6.40
10	6.49	6.58	6.67	6.76	6.85	6.95	7.04	7.13	7.22	7.31
11	7.40	7.50	7.59	7.69	7.78	7.87	7.96	8.05	8.15	8.25
12	8.34	8.43	8.52	8.62	8.71	8.81	8.90	8.99	9.09	9.18

CURVE 73

Inches \ Tenths	0.0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9
0	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.01
1	0.02	0.03	0.05	0.07	0.10	0.13	0.16	0.20	0.24	0.28
2	0.32	0.37	0.41	0.46	0.52	0.57	0.62	0.68	0.74	0.80
3	0.86	0.92	0.98	1.04	1.11	1.18	1.25	1.32	1.39	1.46
4	1.53	1.60	1.67	1.75	1.82	1.90	1.97	2.04	2.12	2.20
5	2.28	2.36	2.44	2.52	2.60	2.68	2.76	2.84	2.92	3.00
6	3.09	3.17	3.25	3.34	3.43	3.51	3.60	3.68	3.76	3.85
7	3.94	4.02	4.11	4.20	4.29	4.37	4.46	4.55	4.64	4.73
8	4.81	4.90	4.99	5.08	5.17	5.26	5.35	5.44	5.53	5.62
9	5.71	5.80	5.89	5.98	6.07	6.16	6.25	6.35	6.44	6.53
10	6.62	6.71	6.81	6.90	6.99	7.08	7.17	7.27	7.36	7.45
11	7.54	7.64	7.73	7.82	7.92	8.01	8.10	8.20	8.29	8.38
12	8.48	8.57	8.67	8.76	8.86	8.95	9.05	9.14	9.23	9.33

Exhibit 2-7A

RAINFALL-RUNOFF DEPTHS FOR SELECTED RUNOFF CURVE NUMBERS

CURVE 68

	Inches \ Tenths	0.0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9
	0										
	1	0.00	0.01	0.01	0.02	0.04	0.06	0.08	0.11	0.13	0.16
	2	0.19	0.23	0.26	0.30	0.34	0.38	0.43	0.48	0.53	0.58
2	3	0.63	0.68	0.73	0.78	0.84	0.90	0.96	1.02	1.08	1.14
5	4	1.20	1.26	1.33	1.40	1.47	1.53	1.60	1.67	1.74	1.81
10	5	1.88	1.95	2.02	2.09	2.16	2.24	2.32	2.39	2.47	2.54
50	6	2.62	2.70	2.78	2.85	2.93	3.01	3.09	3.17	3.25	3.33
100	7	3.41	3.49	3.57	3.65	3.73	3.82	3.90	3.99	4.07	4.15
	8	4.23	4.32	4.40	4.49	4.57	4.66	4.74	4.83	4.91	5.00
	9	5.09	5.17	5.26	5.35	5.43	5.52	5.61	5.70	5.78	5.87
	10	5.96	6.05	6.14	6.23	6.32	6.40	6.49	6.58	6.67	6.76
	11	6.85	6.94	7.03	7.12	7.21	7.30	7.39	7.48	7.57	7.66
	12	7.76	7.84	7.94	8.03	8.12	8.21	8.31	8.40	8.49	8.58

CURVE 69

Inches \ Tenths	0.0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9
1	0.00	0.01	0.02	0.03	0.05	0.07	0.09	0.12	0.15	0.18
2	0.22	0.25	0.29	0.33	0.38	0.42	0.47	0.52	0.57	0.62
3	0.67	0.72	0.78	0.84	0.90	0.96	1.02	1.08	1.14	1.20
4	1.27	1.33	1.40	1.47	1.53	1.60	1.67	1.74	1.81	1.88
5	1.96	2.03	2.10	2.18	2.25	2.33	2.40	2.48	2.56	2.63
6	2.71	2.79	2.87	2.95	3.03	3.11	3.19	3.27	3.35	3.43
7	3.51	3.60	3.68	3.76	3.84	3.93	4.01	4.10	4.18	4.26
8	4.35	4.44	4.52	4.61	4.69	4.78	4.86	4.95	5.04	5.12
9	5.21	5.30	5.39	5.48	5.56	5.65	5.74	5.82	5.91	6.00
10	6.09	6.18	6.27	6.36	6.45	6.54	6.63	6.72	6.81	6.90
11	6.99	7.08	7.17	7.27	7.36	7.45	7.54	7.63	7.72	7.81
12	7.90	7.99	8.09	8.18	8.27	8.37	8.46	8.55	8.64	8.73

CURVE 70

Inches \ Tenths	0.0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9
1	0.00	0.01	0.02	0.04	0.06	0.08	0.11	0.14	0.17	0.20
2	0.24	0.28	0.32	0.36	0.40	0.45	0.50	0.55	0.60	0.65
3	0.71	0.77	0.83	0.89	0.95	1.01	1.07	1.13	1.19	1.25
4	1.33	1.40	1.46	1.53	1.60	1.67	1.74	1.81	1.88	1.96
5	2.04	2.11	2.19	2.26	2.33	2.41	2.49	2.57	2.64	2.72
6	2.80	2.88	2.96	3.04	3.13	3.21	3.29	3.37	3.45	3.53
7	3.61	3.70	3.79	3.87	3.95	4.04	4.12	4.20	4.28	4.37
8	4.46	4.55	4.64	4.72	4.81	4.90	4.98	5.07	5.16	5.25
9	5.33	5.42	5.51	5.60	5.69	5.78	5.87	5.96	6.05	6.14
10	6.23	6.32	6.41	6.50	6.59	6.68	6.77	6.86	6.95	7.04
11	7.13	7.22	7.31	7.40	7.49	7.58	7.68	7.77	7.86	7.96
12	8.05	8.14	8.23	8.33	8.42	8.51	8.60	8.70	8.79	8.88

Exhibit 2-7A

RAINFALL-RUNOFF DEPTHS FOR SELECTED RUNOFF CURVE NUMBERS

CURVE 65

Inches \ Tenths	0.0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9
0										
1	0.00	0.00	0.00	0.01	0.02	0.03	0.04	0.06	0.08	0.10
2	0.13	0.16	0.19	0.23	0.26	0.30	0.33	0.37	0.42	0.46
3	0.50	0.55	0.60	0.65	0.70	0.75	0.80	0.86	0.91	0.97
4	1.03	1.09	1.15	1.21	1.27	1.33	1.39	1.45	1.52	1.58
5	1.65	1.72	1.78	1.85	1.92	1.99	2.06	2.13	2.20	2.28
6	2.35	2.42	2.50	2.57	2.64	2.72	2.80	2.87	2.94	3.02
7	3.10	3.18	3.25	3.33	3.41	3.49	3.57	3.64	3.73	3.81
8	3.89	3.97	4.05	4.13	4.22	4.30	4.38	4.46	4.54	4.62
9	4.71	4.80	4.88	4.96	5.05	5.13	5.22	5.30	5.39	5.47
10	5.56	5.65	5.73	5.82	5.90	5.99	6.08	6.17	6.26	6.34
11	6.43	6.52	6.60	6.69	6.78	6.87	6.96	7.05	7.14	7.23
12	7.31	7.40	7.49	7.58	7.67	7.76	7.85	7.94	8.03	8.12

CURVE 66

Inches \ Tenths	0.0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9
1	0.00	0.00	0.01	0.01	0.02	0.04	0.06	0.08	0.10	0.12
2	0.15	0.18	0.21	0.25	0.29	0.33	0.37	0.41	0.45	0.50
3	0.55	0.60	0.65	0.60	0.75	0.80	0.86	0.91	0.97	1.03
4	1.09	1.15	1.21	1.27	1.33	1.39	1.46	1.53	1.59	1.66
5	1.73	1.80	1.87	1.94	2.01	2.08	2.15	2.22	2.29	2.36
6	2.44	2.51	2.59	2.67	2.74	2.82	2.89	2.97	3.05	3.12
7	3.20	3.28	3.36	3.44	3.52	3.60	3.68	3.76	3.84	3.93
8	4.01	4.09	4.17	4.26	4.34	4.43	4.51	4.59	4.67	4.76
9	4.84	4.93	5.01	5.10	5.18	5.27	5.35	5.43	5.50	5.61
10	5.70	5.78	5.87	5.96	6.05	6.13	6.22	6.31	6.40	6.49
11	6.57	6.66	6.75	6.84	6.93	7.02	7.11	7.20	7.29	7.38
12	7.46	7.55	7.64	7.73	7.82	7.92	8.01	8.10	8.19	8.28

CURVE 67

Inches \ Tenths	0.0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9
1	0.00	0.00	0.01	0.02	0.03	0.05	0.07	0.09	0.12	0.15
2	0.18	0.21	0.24	0.28	0.32	0.36	0.40	0.44	0.49	0.54
3	0.59	0.64	0.69	0.74	0.79	0.85	0.91	0.97	1.03	1.09
4	1.15	1.21	1.27	1.34	1.40	1.47	1.53	1.60	1.67	1.74
5	1.81	1.88	1.95	2.02	2.09	2.16	2.23	2.31	2.39	2.46
6	2.54	2.61	2.69	2.76	2.84	2.92	3.00	3.08	3.15	3.23
7	3.31	3.39	3.47	3.55	3.64	3.72	3.80	3.88	3.96	4.04
8	4.13	4.21	4.29	4.38	4.46	4.55	4.63	4.71	4.80	4.89
9	4.97	5.06	5.14	5.23	5.31	5.40	5.49	5.58	5.66	5.75
10	5.84	5.92	6.01	6.10	6.19	6.28	6.36	6.45	6.54	6.63
11	6.72	6.81	6.90	6.99	7.08	7.17	7.26	7.35	7.44	7.53
12	7.62	7.71	7.80	7.89	7.98	8.07	8.16	8.25	8.35	8.44

Exhibit 2-7A

RAINFALL-RUNOFF DEPTHS FOR SELECTED RUNOFF CURVE NUMBERS

CURVE 62

Inches \ Tenths	0.0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9
0										
1						0.01	0.02	0.03	0.05	0.07
2	0.09	0.11	0.13	0.16	0.19	0.22	0.25	0.28	0.32	0.36
3	0.40	0.44	0.48	0.52	0.56	0.61	0.66	0.71	0.76	0.81
4	0.86	0.91	0.96	1.02	1.08	1.14	1.20	1.26	1.32	1.38
5	1.44	1.50	1.56	1.62	1.68	1.74	1.81	1.88	1.95	2.02
6	2.09	2.16	2.23	2.30	2.37	2.44	2.51	2.58	2.65	2.72
7	2.80	2.87	2.94	3.02	3.09	3.17	3.24	3.32	3.40	3.48
8	3.55	3.63	3.71	3.79	3.86	3.94	4.02	4.10	4.18	4.26
9	4.34	4.42	4.50	4.59	4.67	4.75	4.83	4.91	5.00	5.08
10	5.16	5.25	5.33	5.41	5.50	5.58	5.66	5.75	5.83	5.92
11	6.00	6.09	6.17	6.26	6.34	6.43	6.52	6.60	6.69	6.77
12	6.86	6.95	7.04	7.13	7.21	7.30	7.39	7.48	7.56	7.65

CURVE 63

Inches \ Tenths	0.0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9
1						0.02	0.03	0.04	0.06	0.08
2	0.10	0.12	0.15	0.18	0.21	0.25	0.28	0.32	0.35	0.39
3	0.43	0.47	0.52	0.57	0.61	0.66	0.71	0.76	0.81	0.86
4	0.92	0.98	1.03	1.09	1.15	1.21	1.26	1.32	1.38	1.44
5	1.51	1.58	1.64	1.70	1.76	1.83	1.90	1.97	2.04	2.11
6	2.18	2.25	2.32	2.39	2.47	2.54	2.61	2.68	2.76	2.83
7	2.91	2.98	3.06	3.13	3.21	3.28	3.36	3.44	3.52	3.59
8	3.67	3.75	3.83	3.91	3.99	4.07	4.15	4.23	4.31	4.39
9	4.48	4.56	4.64	4.72	4.80	4.89	4.97	5.05	5.14	5.22
10	5.30	5.39	5.47	5.56	5.64	5.73	5.81	5.90	5.98	6.07
11	6.15	6.24	6.33	6.41	6.50	6.59	6.68	6.76	6.84	6.93
12	7.02	7.11	7.20	7.29	7.38	7.47	7.55	7.64	7.73	7.82

CURVE 64

Inches \ Tenths	0.0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9
1	0.00	0.00	0.00	0.00	0.01	0.02	0.03	0.05	0.07	0.09
2	0.11	0.14	0.17	0.20	0.23	0.26	0.30	0.34	0.38	0.42
3	0.47	0.51	0.56	0.60	0.65	0.70	0.75	0.80	0.85	0.91
4	0.97	1.03	1.09	1.15	1.21	1.26	1.32	1.38	1.45	1.51
5	1.58	1.64	1.71	1.77	1.84	1.91	1.98	2.05	2.12	2.19
6	2.26	2.33	2.40	2.48	2.55	2.62	2.70	2.77	2.85	2.92
7	3.00	3.07	3.15	3.23	3.30	3.38	3.46	3.54	3.62	3.69
8	3.77	3.85	3.93	4.01	4.09	4.18	4.26	4.34	4.42	4.50
9	4.59	4.67	4.75	4.84	4.92	5.00	5.09	5.17	5.26	5.34
10	5.43	5.51	5.59	5.68	5.76	5.85	5.94	6.02	6.11	6.20
11	6.28	6.37	6.46	6.55	6.64	6.72	6.81	6.90	6.99	7.07
12	7.16	7.25	7.34	7.43	7.52	7.61	7.70	7.78	7.87	7.96

Exhibit 2-7A

RAINFALL-RUNOFF DEPTHS FOR SELECTED RUNOFF CURVE NUMBERS

These tables may be used in lieu of Exhibit 2-7. They have been prepared to eliminate the need for interpolation which is often necessary with the use of Exhibit 2-7.

Inches of runoff associated with inches of precipitation for various curve numbers as shown in these tables are based on the runoff equation at bottom of page 2-5.

If inches of runoff are needed for precipitation amounts greater than 12", refer to Technical Release No. 16.

CURVE 60

	Inches \ Tenths	0.0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9
	0										
	1	0.00	0.00	0.00	0.00	0.00	0.00	0.01	0.02	0.03	0.04
	2	0.06	0.08	0.10	0.12	0.14	0.17	0.20	0.23	0.27	0.30
2yr.	3	0.33	0.37	0.41	0.45	0.49	0.53	0.57	0.62	0.67	0.72
5yr.	4	0.76	0.81	0.86	0.91	0.97	1.03	1.08	1.13	1.19	1.25
10yr.	5	1.30	1.36	1.42	1.48	1.54	1.61	1.67	1.73	1.80	1.86
50yr.	6	1.92	1.99	2.06	2.12	2.19	2.26	2.33	2.40	2.47	2.54
100yr.	7	2.61	2.68	2.75	2.82	2.89	2.97	3.04	3.11	3.18	3.26
	8	3.34	3.41	3.49	3.56	3.64	3.72	3.79	3.87	3.95	4.03
	9	4.10	4.18	4.26	4.34	4.42	4.50	4.58	4.66	4.74	4.82
	10	4.90	4.98	5.07	5.15	5.23	5.31	5.39	5.48	5.56	5.64
	11	5.73	5.81	5.89	5.98	6.06	6.14	6.22	6.31	6.40	6.48
	12	6.57	6.65	6.73	6.82	6.91	6.99	7.08	7.17	7.26	7.34

CURVE 61

Inches \ Tenths	0.0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9
1						0.01	0.02	0.03	0.04	0.05
2	0.07	0.09	0.11	0.14	0.17	0.20	0.23	0.26	0.29	0.32
3	0.36	0.40	0.44	0.48	0.52	0.57	0.62	0.67	0.71	0.76
4	0.81	0.86	0.91	0.96	1.02	1.08	1.13	1.19	1.25	1.31
5	1.37	1.43	1.49	1.55	1.61	1.68	1.74	1.81	1.87	1.94
6	2.01	2.07	2.14	2.21	2.28	2.35	2.42	2.49	2.56	2.63
7	2.70	2.77	2.84	2.91	2.98	3.06	3.14	3.22	3.29	3.37
8	3.44	3.52	3.60	3.67	3.75	3.83	3.91	3.99	4.07	4.14
9	4.22	4.30	4.38	4.46	4.54	4.62	4.71	4.79	4.87	4.95
10	5.03	5.11	5.20	5.28	5.36	5.44	5.53	5.61	5.70	5.78
11	5.87	5.95	6.03	6.12	6.20	6.29	6.38	6.46	6.55	6.63
12	6.72	6.80	6.89	6.98	7.06	7.15	7.24	7.33	7.41	7.50

Exhibit 2-7A

CURVE 60

RUNOFF FOR INCHES OF RAINFALL

Inches \ Tenths	0.0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9
21	14.69	14.78	14.87	14.97	15.06	15.15	15.25	15.34	15.44	15.53
22	15.62	15.72	15.81	15.91	16.00	16.09	16.19	16.28	16.38	16.47
23	16.57	16.66	16.76	16.85	16.94	17.04	17.13	17.23	17.32	17.42
24	17.51	17.61	17.70	17.80	17.89	17.99	18.08	18.18	18.27	18.37
25	18.46	18.56	18.65	18.75	18.84	18.94	19.03	19.13	19.22	19.32
26	19.42	19.51	19.61	19.70	19.80	19.89	19.99	20.08	20.18	20.28
27	20.37	20.47	20.56	20.66	20.76	20.85	20.95	21.04	21.14	21.23
28	21.33	21.43	21.52	21.62	21.71	21.81	21.91	22.00	22.10	22.20
29	22.29	22.39	22.48	22.58	22.68	22.77	22.87	22.97	23.06	23.16
30	23.26	23.35	23.45	23.54	23.64	23.74	23.83	23.93	24.03	24.12
31	24.22	24.32	24.41	24.51	24.61	24.70	24.80	24.90	24.99	25.09
32	25.19	25.28	25.38	25.48	25.57	25.67	25.77	25.87	25.96	26.06
33	26.16	26.25	26.35	26.45	26.54	26.64	26.74	26.84	26.93	27.03
34	27.13	27.22	27.32	27.42	27.52	27.61	27.71	27.81	27.90	28.00
35	28.10	28.20	28.29	28.39	28.49	28.59	28.68	28.78	28.88	28.97
36	29.07	29.17	29.27	29.36	29.46	29.56	29.66	29.75	29.85	29.95
37	30.05	30.14	30.24	30.34	30.44	30.53	30.63	30.73	30.83	30.92
38	31.02	31.12	31.22	31.32	31.41	31.51	31.61	31.71	31.80	31.90
39	32.00	32.10	32.19	32.29	32.39	32.49	32.59	32.68	32.78	32.88
40	32.98	33.08	33.17	33.27	33.37	33.47	33.56	33.66	33.76	33.86

NOTE: Runoff value determined by equation $Q = \frac{(P-0.2\,S)^2}{P+0.8\,S}$

REFERENCE: National Engineering Handbook, Section 4, HYDROLOGY

CURVE 60

RUNOFF FOR INCHES OF RAINFALL

Inches \ Tenths	0.0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9
0										
1					0.00	0.00	0.01	0.02	0.03	0.04
2	0.06	0.08	0.10	0.12	0.15	0.17	0.20	0.23	0.26	0.30
3	0.33	0.37	0.41	0.45	0.49	0.53	0.57	0.62	0.67	0.71
4	0.76	0.81	0.86	0.91	0.97	1.02	1.07	1.13	1.19	1.24
5	1.30	1.36	1.42	1.48	1.54	1.60	1.66	1.73	1.79	1.86
6	1.92	1.99	2.05	2.12	2.19	2.25	2.32	2.39	2.46	2.53
7	2.60	2.67	2.74	2.82	2.89	2.96	3.04	3.11	3.18	3.26
8	3.33	3.41	3.48	3.56	3.63	3.71	3.79	3.87	3.94	4.02
9	4.10	4.18	4.26	4.34	4.42	4.49	4.57	4.65	4.74	4.82
10	4.90	4.98	5.06	5.14	5.22	5.31	5.39	5.47	5.55	5.64
11	5.72	5.80	5.89	5.97	6.05	6.14	6.22	6.31	6.39	6.48
12	6.56	6.65	6.73	6.82	6.90	6.99	7.08	7.16	7.25	7.34
13	7.42	7.51	7.60	7.68	7.77	7.86	7.95	8.03	8.12	8.21
14	8.30	8.38	8.47	8.56	8.65	8.74	8.83	8.92	9.01	9.09
15	9.18	9.27	9.36	9.45	9.54	9.63	9.72	9.81	9.90	9.99
16	10.08	10.17	10.26	10.35	10.44	10.53	10.62	10.71	10.81	10.90
17	10.99	11.08	11.17	11.26	11.35	11.44	11.54	11.63	11.72	11.81
18	11.90	11.99	12.09	12.18	12.27	12.36	12.45	12.55	12.64	12.73
19	12.82	12.92	13.01	13.10	13.19	13.29	13.38	13.47	13.57	13.66
20	13.75	13.85	13.94	14.03	14.12	14.22	14.31	14.40	14.50	14.59

NOTE: Runoff value determined by equation $Q = \frac{(P-0.2S)^2}{P+0.8S}$

REFERENCE: National Engineering Handbook, Section 4, HYDROLOGY

CURVE 59

RUNOFF FOR INCHES OF RAINFALL

Inches \ Tenths	0.0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9
21	14.48	14.57	14.67	14.76	14.85	14.95	15.04	15.13	15.23	15.32
22	15.41	15.51	15.60	15.69	15.79	15.88	15.98	16.07	16.16	16.26
23	16.35	16.45	16.54	16.63	16.73	16.82	16.92	17.01	17.11	17.20
24	17.29	17.39	17.48	17.58	17.67	17.77	17.86	17.96	18.05	18.15
25	18.24	18.34	18.43	18.53	18.62	18.72	18.81	18.91	19.00	19.10
26	19.19	19.29	19.38	19.48	19.57	19.67	19.76	19.86	19.95	20.05
27	20.14	20.24	20.33	20.43	20.53	20.62	20.72	20.81	20.91	21.00
28	21.10	21.19	21.29	21.39	21.48	21.58	21.67	21.77	21.87	21.96
29	22.06	22.15	22.25	22.35	22.44	22.54	22.63	22.73	22.83	22.92
30	23.02	23.11	23.21	23.31	23.40	23.50	23.60	23.69	23.79	23.88
31	23.98	24.08	24.17	24.27	24.37	24.46	24.56	24.66	24.75	24.85
32	24.95	25.04	25.14	25.24	25.33	25.43	25.53	25.62	25.72	25.82
33	25.91	26.01	26.11	26.20	26.30	26.40	26.49	26.59	26.69	26.78
34	26.88	26.98	27.07	27.17	27.27	27.37	27.46	27.56	27.66	27.75
35	27.85	27.95	28.05	28.14	28.24	28.34	28.43	28.53	28.63	28.73
36	28.82	28.92	29.02	29.11	29.21	29.31	29.41	29.50	29.60	29.70
37	29.79	29.89	29.99	30.09	30.18	30.28	30.38	30.48	30.57	30.67
38	30.77	30.87	30.96	31.06	31.16	31.26	31.35	31.45	31.55	31.65
39	31.74	31.84	31.94	32.04	32.13	32.23	32.33	32.43	32.52	32.62
40	32.72	32.82	32.92	33.01	33.11	33.21	33.31	33.40	33.50	33.60

NOTE: Runoff value determined by equation $Q = \frac{(P-0.2\,S)^2}{P+0.8\,S}$

REFERENCE: National Engineering Handbook, Section 4, HYDROLOGY

CURVE 59

RUNOFF FOR INCHES OF RAINFALL

Inches \ Tenths	0.0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9
0										
1					0.00	0.00	0.01	0.01	0.02	0.03
2	0.05	0.07	0.08	0.11	0.13	0.15	0.18	0.21	0.24	0.27
3	0.30	0.34	0.37	0.41	0.45	0.49	0.53	0.58	0.62	0.67
4	0.71	0.76	0.81	0.86	0.91	0.96	1.01	1.07	1.12	1.18
5	1.23	1.29	1.35	1.41	1.47	1.53	1.59	1.65	1.71	1.77
6	1.84	1.90	1.97	2.03	2.10	2.17	2.23	2.30	2.37	2.44
7	2.51	2.58	2.65	2.72	2.79	2.86	2.93	3.00	3.08	3.15
8	3.22	3.30	3.37	3.45	3.52	3.60	3.67	3.75	3.82	3.90
9	3.98	4.05	4.13	4.21	4.29	4.37	4.45	4.53	4.60	4.68
10	4.76	4.84	4.92	5.01	5.09	5.17	5.25	5.33	5.41	5.49
11	5.58	5.66	5.74	5.82	5.91	5.99	6.07	6.16	6.24	6.33
12	6.41	6.50	6.58	6.66	6.75	6.83	6.92	7.01	7.09	7.18
13	7.26	7.35	7.43	7.52	7.61	7.69	7.78	7.87	7.95	8.04
14	8.13	8.22	8.30	8.39	8.48	8.57	8.66	8.74	8.83	8.92
15	9.01	9.10	9.19	9.28	9.36	9.45	9.54	9.63	9.72	9.81
16	9.90	9.99	10.08	10.17	10.26	10.35	10.44	10.53	10.62	10.71
17	10.80	10.89	10.98	11.07	11.16	11.25	11.35	11.44	11.53	11.62
18	11.71	11.80	11.89	11.98	12.08	12.17	12.26	12.35	12.44	12.53
19	12.63	12.72	12.81	12.90	13.00	13.09	13.18	13.27	13.36	13.46
20	13.55	13.64	13.74	13.83	13.92	14.01	14.11	14.20	14.29	14.39

NOTE: Runoff value determined by equation $Q = \frac{(P-0.2\,S)^2}{P+0.8\,S}$

REFERENCE: National Engineering Handbook, Section 4, HYDROLOGY

CURVE 58

RUNOFF FOR INCHES OF RAINFALL

Inches \ Tenths	0.0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9
21	14.27	14.36	14.45	14.55	14.64	14.73	14.83	14.92	15.01	15.10
22	15.20	15.29	15.38	15.48	15.57	15.66	15.76	15.85	15.95	16.04
23	16.13	16.23	16.32	16.41	16.51	16.60	16.70	16.79	16.88	16.98
24	17.07	17.17	17.26	17.35	17.45	17.54	17.64	17.73	17.83	17.92
25	18.01	18.11	18.20	18.30	18.39	18.49	18.58	18.68	18.77	18.8
26	18.96	19.06	19.15	19.25	19.34	19.44	19.53	19.63	19.72	19.8
27	19.91	20.01	20.10	20.20	20.29	20.39	20.48	20.58	20.67	20.7
28	20.86	20.96	21.05	21.15	21.25	21.34	21.44	21.53	21.63	21.7
29	21.82	21.91	22.01	22.11	22.20	22.30	22.39	22.49	22.58	22.6
30	22.78	22.87	22.97	23.06	23.16	23.26	23.35	23.45	23.54	23.6
31	23.74	23.83	23.93	24.03	24.12	24.22	24.31	24.41	24.51	24.6
32	24.70	24.80	24.89	24.99	25.08	25.18	25.28	25.37	25.47	25.5
33	25.66	25.76	25.86	25.95	26.05	26.15	26.24	26.34	26.44	26.5
34	26.63	26.73	26.82	26.92	27.02	27.11	27.21	27.31	27.40	27.5
35	27.60	27.69	27.79	27.89	27.98	28.08	28.18	28.28	28.37	28.
36	28.57	28.66	28.76	28.86	28.95	29.05	29.15	29.25	29.34	29.
37	29.54	29.63	29.73	29.83	29.93	30.02	30.12	30.22	30.31	30.
38	30.51	30.61	30.70	30.80	30.90	31.00	31.09	31.19	31.29	31.
39	31.48	31.58	31.68	31.77	31.87	31.97	32.07	32.16	32.26	32.
40	32.46	32.55	32.65	32.75	32.85	32.94	33.04	33.14	33.24	33.

NOTE: Runoff value determined by equation $Q = \frac{(P-0.2\,S)^2}{P+0.8\,S}$

REFERENCE: National Engineering Handbook, Section 4, HYDROLOGY

CURVE 58

RUNOFF FOR INCHES OF RAINFALL

Inches \ Tenths	0.0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9
0										
1						0.00	0.00	0.01	0.02	0.03
2	0.04	0.05	0.07	0.09	0.11	0.13	0.16	0.18	0.21	0.24
3	0.27	0.31	0.34	0.38	0.41	0.45	0.49	0.53	0.58	0.62
4	0.67	0.71	0.76	0.81	0.86	0.91	0.96	1.01	1.06	1.11
5	1.17	1.22	1.28	1.34	1.40	1.45	1.51	1.57	1.63	1.70
6	1.76	1.82	1.88	1.95	2.01	2.08	2.14	2.21	2.27	2.34
7	2.41	2.48	2.55	2.62	2.69	2.76	2.83	2.90	2.97	3.04
8	3.11	3.19	3.26	3.33	3.41	3.48	3.55	3.63	3.70	3.78
9	3.86	3.93	4.01	4.09	4.16	4.24	4.32	4.40	4.47	4.55
10	4.63	4.71	4.79	4.87	4.95	5.03	5.11	5.19	5.27	5.35
11	5.43	5.52	5.60	5.68	5.76	5.84	5.93	6.01	6.09	6.17
12	6.26	6.34	6.43	6.51	6.59	6.68	6.76	6.85	6.93	7.02
13	7.10	7.19	7.27	7.36	7.44	7.53	7.62	7.70	7.79	7.87
14	7.96	8.05	8.13	8.22	8.31	8.40	8.48	8.57	8.66	8.75
15	8.83	8.92	9.01	9.10	9.19	9.27	9.36	9.45	9.54	9.63
16	9.72	9.81	9.90	9.98	10.07	10.16	10.25	10.34	10.43	10.52
17	10.61	10.70	10.79	10.88	10.97	11.06	11.15	11.24	11.33	11.42
18	11.52	11.61	11.70	11.79	11.88	11.97	12.06	12.15	12.24	12.33
19	12.43	12.52	12.61	12.70	12.79	12.88	12.98	13.07	13.16	13.25
20	13.34	13.44	13.53	13.62	13.71	13.81	13.90	13.99	14.08	14.18

NOTE: Runoff value determined by equation $Q = \frac{(P-0.2\,S)^2}{P+0.8\,S}$

REFERENCE: National Engineering Handbook, Section 4, HYDROLOGY

CURVE 57

RUNOFF FOR INCHES OF RAINFALL

Inches \ Tenths	0.0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9
21	14.06	14.15	14.24	14.33	14.42	14.52	14.61	14.70	14.79	14.89
22	14.98	15.07	15.17	15.26	15.35	15.44	15.54	15.63	15.72	15.82
23	15.91	16.00	16.10	16.19	16.28	16.38	16.47	16.56	16.66	16.75
24	16.85	16.94	17.03	17.13	17.22	17.31	17.41	17.50	17.60	17.69
25	17.78	17.88	17.97	18.07	18.16	18.25	18.35	18.44	18.54	18.63
26	18.73	18.82	18.92	19.01	19.10	19.20	19.29	19.39	19.48	19.58
27	19.67	19.77	19.86	19.96	20.05	20.15	20.24	20.34	20.43	20.53
28	20.62	20.72	20.81	20.91	21.00	21.10	21.19	21.29	21.38	21.48
29	21.57	21.67	21.77	21.86	21.96	22.05	22.15	22.24	22.34	22.43
30	22.53	22.63	22.72	22.82	22.91	23.01	23.10	23.20	23.30	23.39
31	23.49	23.58	23.68	23.77	23.87	23.97	24.06	24.16	24.25	24.35
32	24.45	24.54	24.64	24.74	24.83	24.93	25.02	25.12	25.22	25.31
33	25.41	25.50	25.60	25.70	25.79	25.89	25.99	26.08	26.18	26.28
34	26.37	26.47	26.57	26.66	26.76	26.85	26.95	27.05	27.14	27.24
35	27.34	27.43	27.53	27.63	27.72	27.82	27.92	28.01	28.11	28.21
36	28.30	28.40	28.50	28.59	28.69	28.79	28.89	28.98	29.08	29.18
37	29.27	29.37	29.47	29.56	29.66	29.76	29.85	29.95	30.05	30.1[illegible]
38	30.24	30.34	30.44	30.53	30.63	30.73	30.83	30.92	31.02	31.1[illegible]
39	31.21	31.31	31.41	31.51	31.60	31.70	31.80	31.90	31.99	32.0
40	32.19	32.28	32.38	32.48	32.58	32.67	32.77	32.87	32.97	33.0

NOTE: Runoff value determined by equation $Q = \frac{(P-0.2\,S)^2}{P+0.8\,S}$

REFERENCE: National Engineering Handbook, Section 4, HYDROLOGY

CURVE 57

RUNOFF FOR INCHES OF RAINFALL

Inches \ Tenths	0.0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9
0										
1							0.00	0.00	0.01	0.02
2	0.03	0.04	0.06	0.08	0.09	0.12	0.14	0.16	0.19	0.22
3	0.25	0.28	0.31	0.34	0.38	0.42	0.45	0.49	0.53	0.58
4	0.62	0.66	0.71	0.75	0.80	0.85	0.90	0.95	1.00	1.05
5	1.11	1.16	1.21	1.27	1.33	1.38	1.44	1.50	1.56	1.62
6	1.68	1.74	1.80	1.86	1.93	1.99	2.05	2.12	2.18	2.25
7	2.31	2.38	2.45	2.52	2.58	2.65	2.72	2.79	2.86	2.93
8	3.00	3.07	3.15	3.22	3.29	3.36	3.44	3.51	3.59	3.66
9	3.73	3.81	3.88	3.96	4.04	4.11	4.19	4.27	4.34	4.42
10	4.50	4.58	4.65	4.73	4.81	4.89	4.97	5.05	5.13	5.21
11	5.29	5.37	5.45	5.53	5.61	5.69	5.78	5.86	5.94	6.02
12	6.10	6.19	6.27	6.35	6.44	6.52	6.60	6.69	6.77	6.85
13	6.94	7.02	7.11	7.19	7.28	7.36	7.45	7.53	7.62	7.70
14	7.79	7.88	7.96	8.05	8.13	8.22	8.31	8.39	8.48	8.57
15	8.66	8.74	8.83	8.92	9.00	9.09	9.18	9.27	9.36	9.44
16	9.53	9.62	9.71	9.80	9.89	9.98	10.06	10.15	10.24	10.33
17	10.42	10.51	10.60	10.69	10.78	10.87	10.96	11.05	11.14	11.23
18	11.32	11.41	11.50	11.59	11.68	11.77	11.86	11.95	12.04	12.13
19	12.22	12.31	12.41	12.50	12.59	12.68	12.77	12.86	12.95	13.04
20	13.14	13.23	13.32	13.41	13.50	13.59	13.69	13.78	13.87	13.96

NOTE: Runoff value determined by equation $Q = \frac{(P-0.2\,S)^2}{P+0.8\,S}$

REFERENCE: National Engineering Handbook, Section 4, HYDROLOGY

CURVE
56

RUNOFF FOR INCHES OF RAINFALL

Inches \ Tenths	0.0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9
21	13.83	13.92	14.02	14.11	14.20	14.29	14.38	14.48	14.57	14.66
22	14.75	14.84	14.94	15.03	15.12	15.21	15.31	15.40	15.49	15.58
23	15.68	15.77	15.86	15.96	16.05	16.14	16.24	16.33	16.42	16.51
24	16.61	16.70	16.79	16.89	16.98	17.07	17.17	17.26	17.36	17.45
25	17.54	17.64	17.73	17.82	17.92	18.01	18.11	18.20	18.29	18.39
26	18.48	18.58	18.67	18.76	18.86	18.95	19.05	19.14	19.24	19.33
27	19.42	19.52	19.61	19.71	19.80	19.90	19.99	20.09	20.18	20.28
28	20.37	20.46	20.56	20.65	20.75	20.84	20.94	21.03	21.13	21.22
29	21.32	21.41	21.51	21.60	21.70	21.79	21.89	21.98	22.08	22.18
30	22.27	22.37	22.46	22.56	22.65	22.75	22.84	22.94	23.03	23.13
31	23.22	23.32	23.42	23.51	23.61	23.70	23.80	23.89	23.99	24.09
32	24.18	24.28	24.37	24.47	24.56	24.66	24.76	24.85	24.95	25.04
33	25.14	25.24	25.33	25.43	25.52	25.62	25.72	25.81	25.91	26.01
34	26.10	26.20	26.29	26.39	26.49	26.58	26.68	26.78	26.87	26.97
35	27.06	27.16	27.26	27.35	27.45	27.55	27.64	27.74	27.84	27.93
36	28.03	28.13	28.22	28.32	28.42	28.51	28.61	28.71	28.80	28.90
37	29.00	29.09	29.19	29.29	29.38	29.48	29.58	29.67	29.77	29.87
38	29.96	30.06	30.16	30.25	30.35	30.45	30.54	30.64	30.74	30.84
39	30.93	31.03	31.13	31.22	31.32	31.42	31.51	31.61	31.71	31.81
40	31.90	32.00	32.10	32.19	32.29	32.39	32.49	32.58	32.68	32.78

NOTE: Runoff value determined by equation $Q = \frac{(P-0.2\,S)^2}{P+0.8\,S}$

REFERENCE: National Engineering Handbook, Section 4, HYDROLOGY

CURVE 56

RUNOFF FOR INCHES OF RAINFALL

Inches \ Tenths	0.0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9
0										
1							0.00	0.00	0.01	0.01
2	0.02	0.03	0.05	0.06	0.08	0.10	0.12	0.14	0.17	0.19
3	0.22	0.25	0.28	0.31	0.34	0.38	0.42	0.45	0.49	0.53
4	0.57	0.62	0.66	0.70	0.75	0.79	0.84	0.89	0.94	0.99
5	1.04	1.09	1.15	1.20	1.25	1.31	1.36	1.42	1.48	1.54
6	1.60	1.66	1.72	1.78	1.84	1.90	1.96	2.02	2.09	2.15
7	2.22	2.28	2.35	2.41	2.48	2.55	2.62	2.68	2.75	2.82
8	2.89	2.96	3.03	3.10	3.17	3.25	3.32	3.39	3.46	3.54
9	3.61	3.68	3.76	3.83	3.91	3.98	4.06	4.13	4.21	4.28
10	4.36	4.44	4.51	4.59	4.67	4.75	4.83	4.90	4.98	5.06
11	5.14	5.22	5.30	5.38	5.46	5.54	5.62	5.70	5.78	5.86
12	5.95	6.03	6.11	6.19	6.27	6.36	6.44	6.52	6.60	6.69
13	6.77	6.85	6.94	7.02	7.11	7.19	7.27	7.36	7.44	7.53
14	7.61	7.70	7.78	7.87	7.95	8.04	8.13	8.21	8.30	8.38
15	8.47	8.56	8.64	8.73	8.82	8.90	8.99	9.08	9.16	9.25
16	9.34	9.43	9.52	9.60	9.69	9.78	9.87	9.96	10.04	10.13
17	10.22	10.31	10.40	10.49	10.58	10.67	10.75	10.84	10.93	11.02
18	11.11	11.20	11.29	11.38	11.47	11.56	11.65	11.74	11.83	11.92
19	12.01	12.10	12.19	12.28	12.37	12.46	12.55	12.65	12.74	12.83
20	12.92	13.01	13.10	13.19	13.28	13.37	13.47	13.56	13.65	13.74

NOTE: Runoff value determined by equation $Q = \frac{(P-0.2\,S)^2}{P+0.8\,S}$

REFERENCE: National Engineering Handbook, Section 4, HYDROLOGY

RAINFALL-RUNOFF DEPTHS FOR SELECTED RUNOFF CURVE NUMBERS

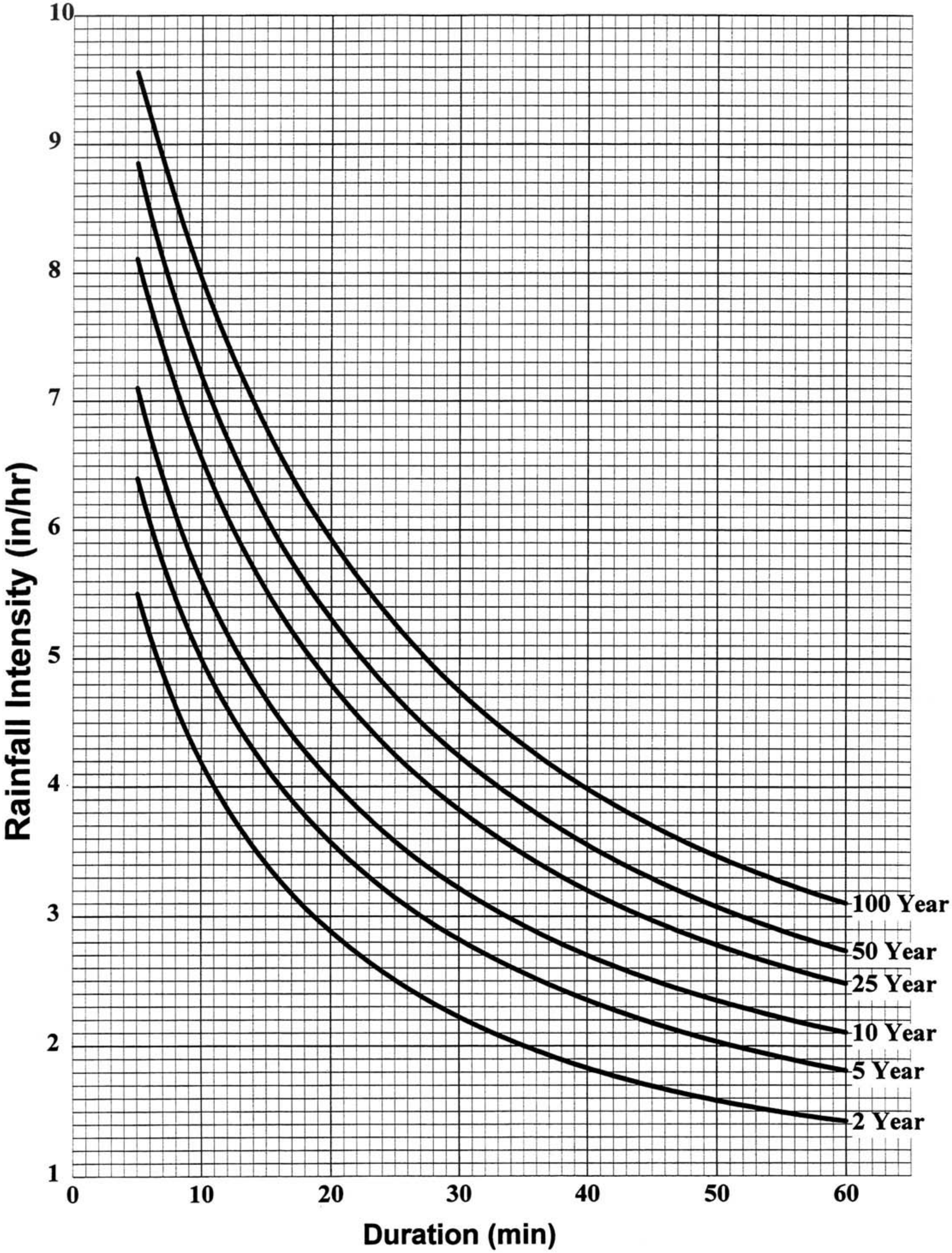
CURVE 98

Inches \ Tenths	0.0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9
0	0.00	0.01	0.07	0.15	0.23	0.32	0.41	0.51	0.60	0.69
1	0.79	0.89	0.99	1.09	1.19	1.28	1.38	1.48	1.58	1.68
2	1.78	1.88	1.98	2.08	2.18	2.27	2.37	2.47	2.57	2.67
3	2.77	2.87	2.97	3.07	3.17	3.27	3.37	3.47	3.57	3.67
4	3.77	3.87	3.97	4.07	4.17	4.27	4.37	4.47	4.57	4.67
5	4.77	4.87	4.97	5.07	5.17	5.27	5.37	5.47	5.57	5.67
6	5.77	5.87	5.97	6.07	6.17	6.27	6.37	6.47	6.57	6.67
7	6.77	6.87	6.98	7.07	7.17	7.27	7.37	7.47	7.57	7.67
8	7.76	7.86	7.96	8.06	8.16	8.26	8.36	8.46	8.56	8.66
9	8.76	8.86	8.96	9.06	9.16	9.26	9.36	9.46	9.56	9.66
10	9.76	9.86	9.96	10.06	10.16	10.26	10.36	10.46	10.56	10.66
11	10.76	10.86	10.96	11.06	11.16	11.26	11.36	11.46	11.56	11.66
12	11.76	11.86	11.96	12.06	12.16	12.26	12.36	12.46	12.56	12.66

Exhibit 2-7A

APPENDIX 4D

I - D - F Curves for Virginia


Rainfall Intensity (in/hr)
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Duration (min)
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50
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100 Year
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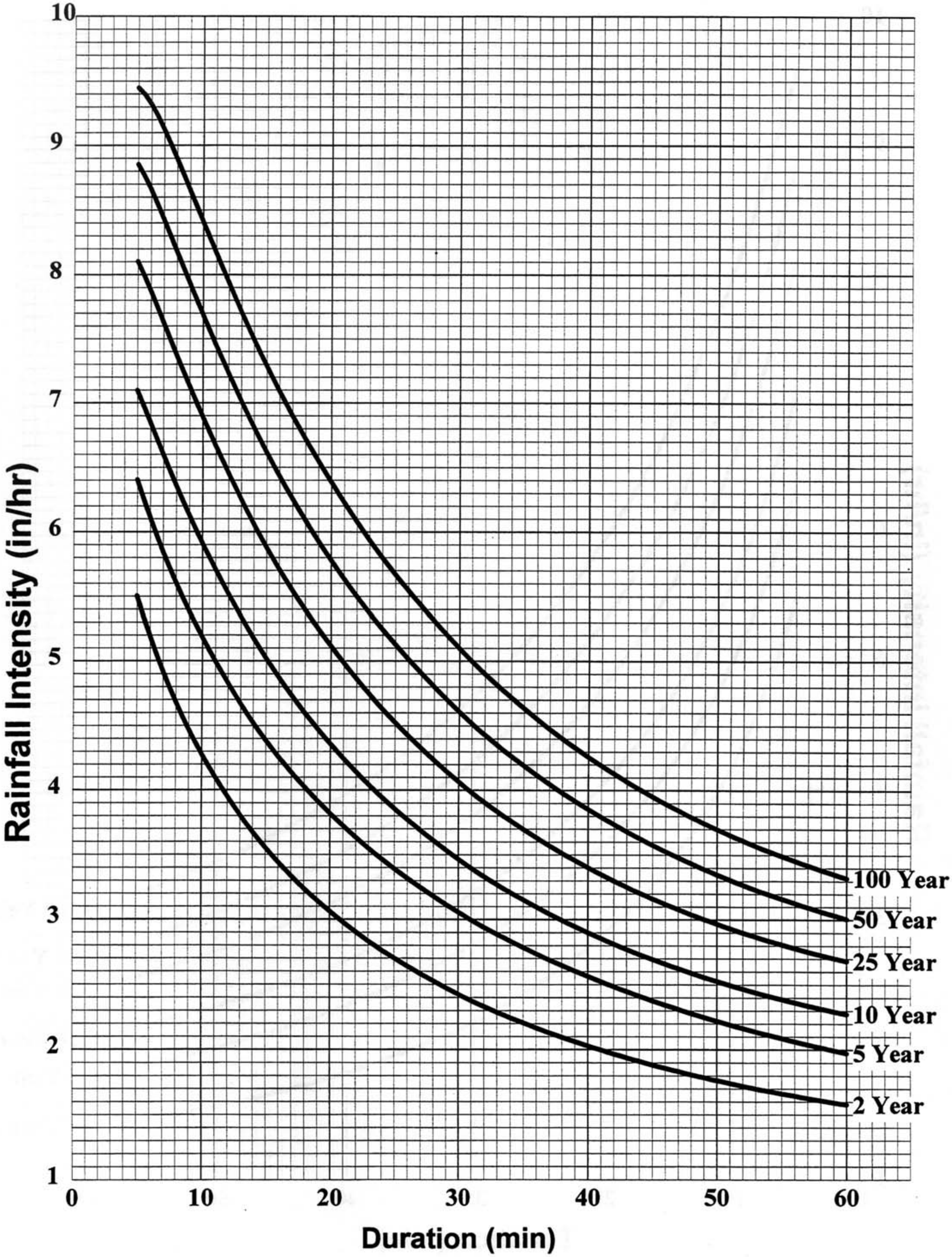
2 Year



Rainfall Intensity (in/hr)
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100 Year
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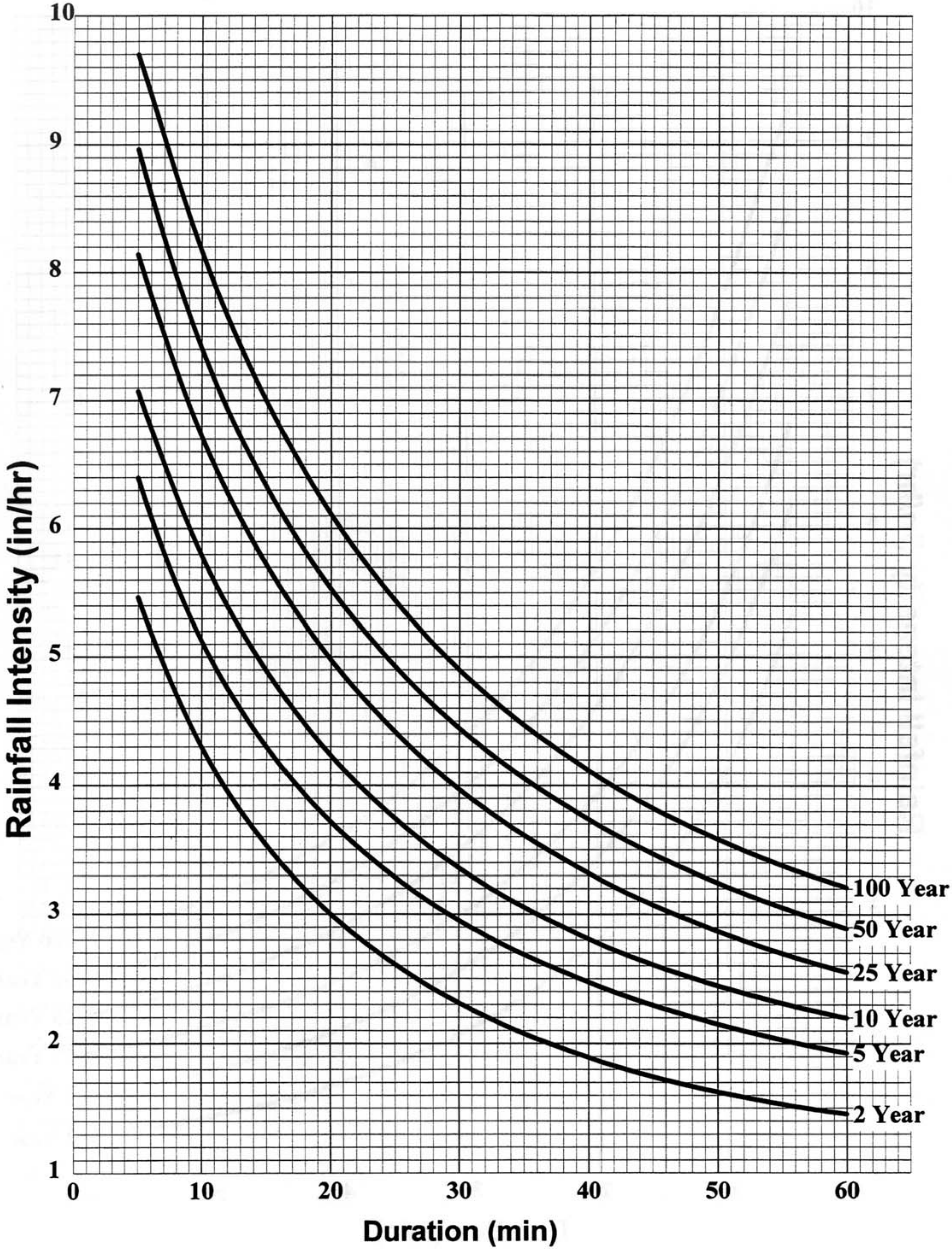
Rainfall Intensity (in/hr)
Duration (min)
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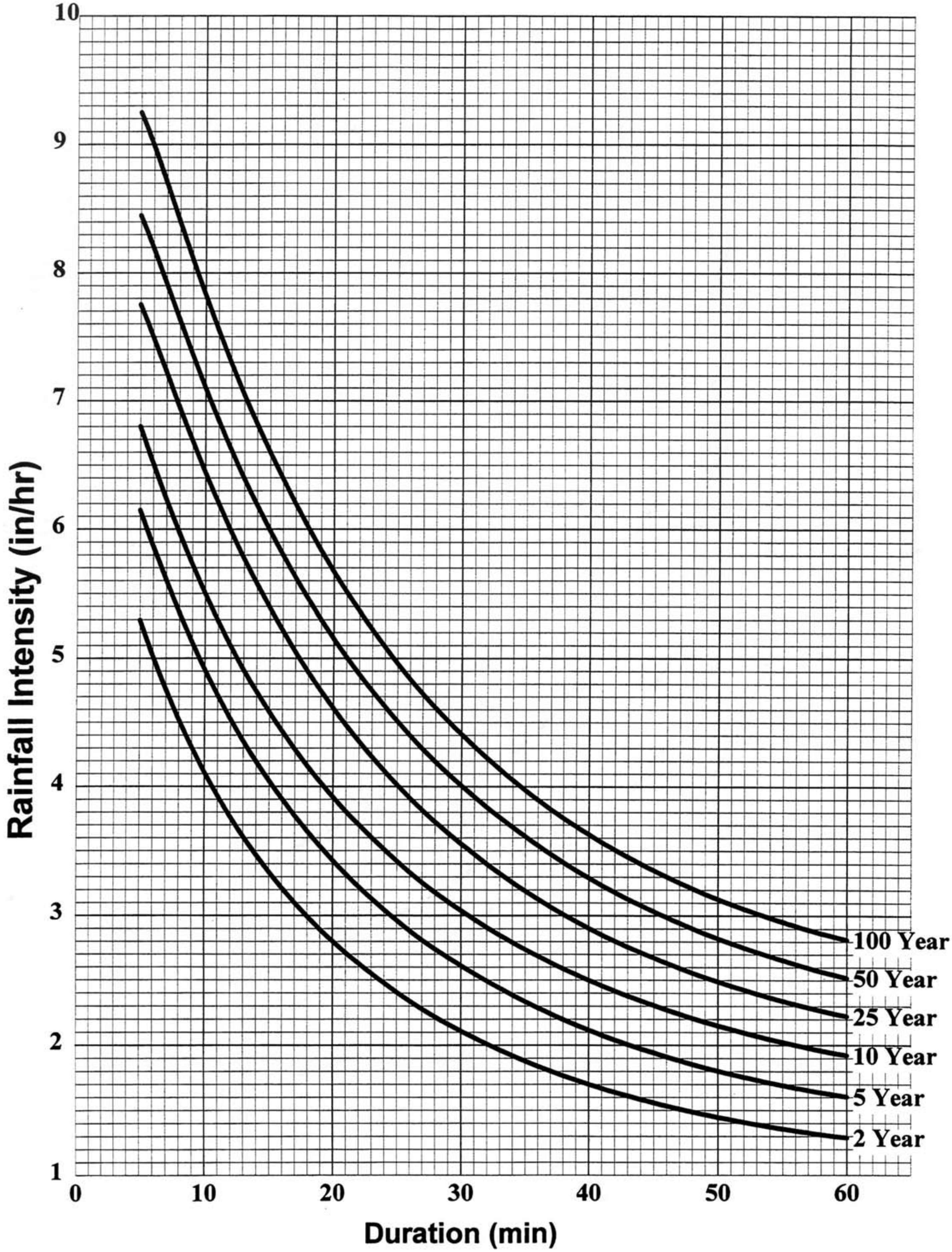
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Rainfall Intensity (in/hr)
Duration (min)
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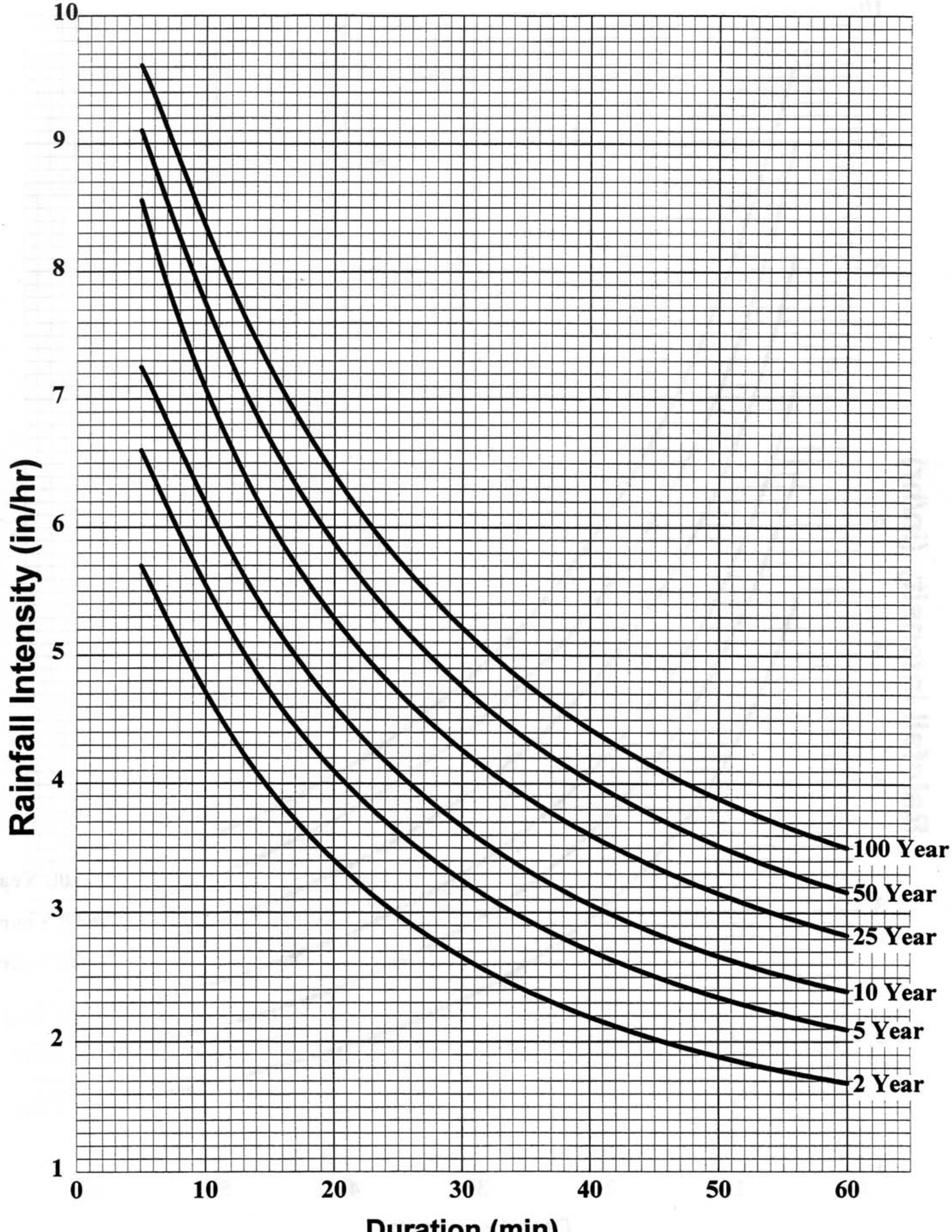

Rainfall Intensity (in/hr)
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Duration (min)
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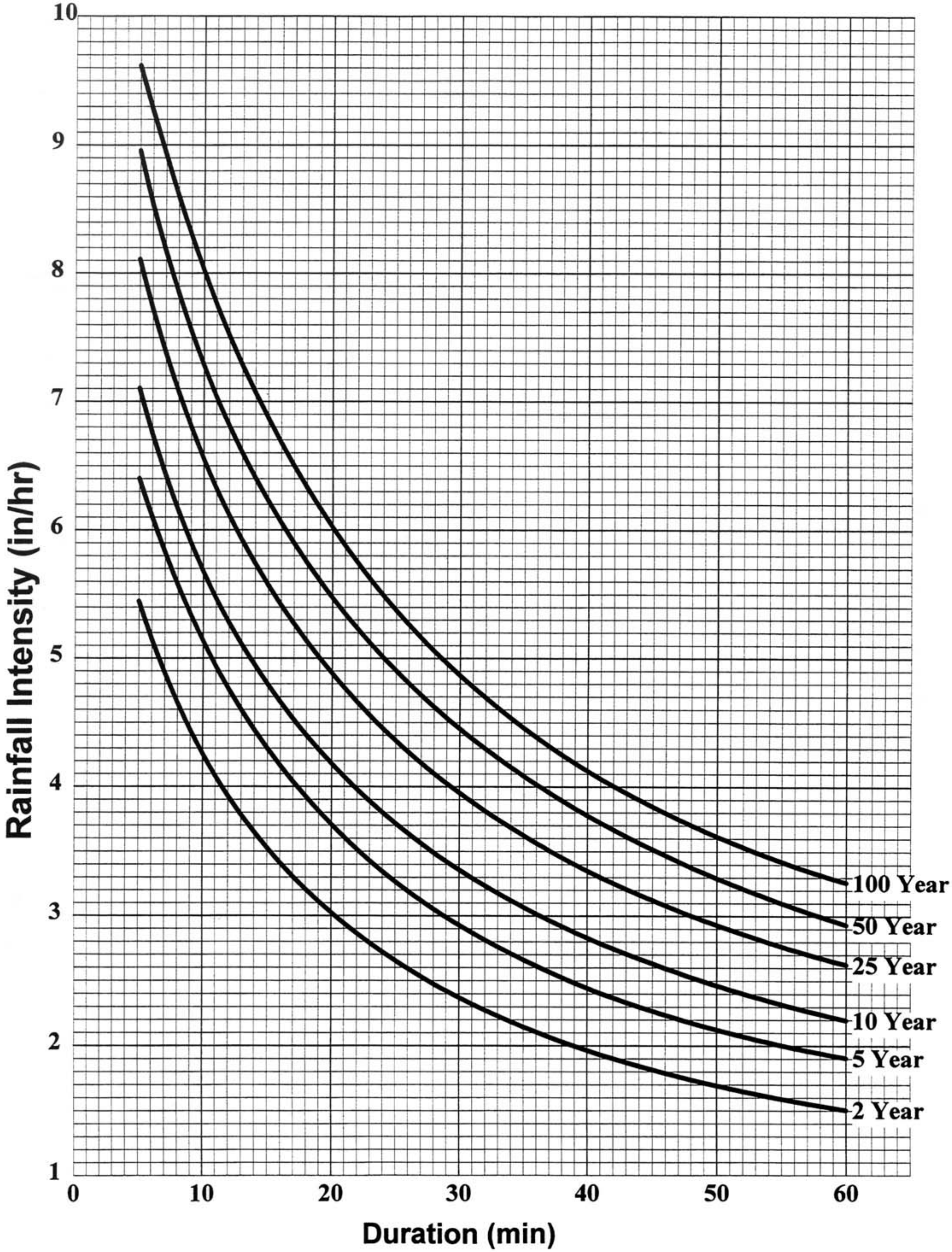

Rainfall Intensity (in/hr)
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Duration (min)
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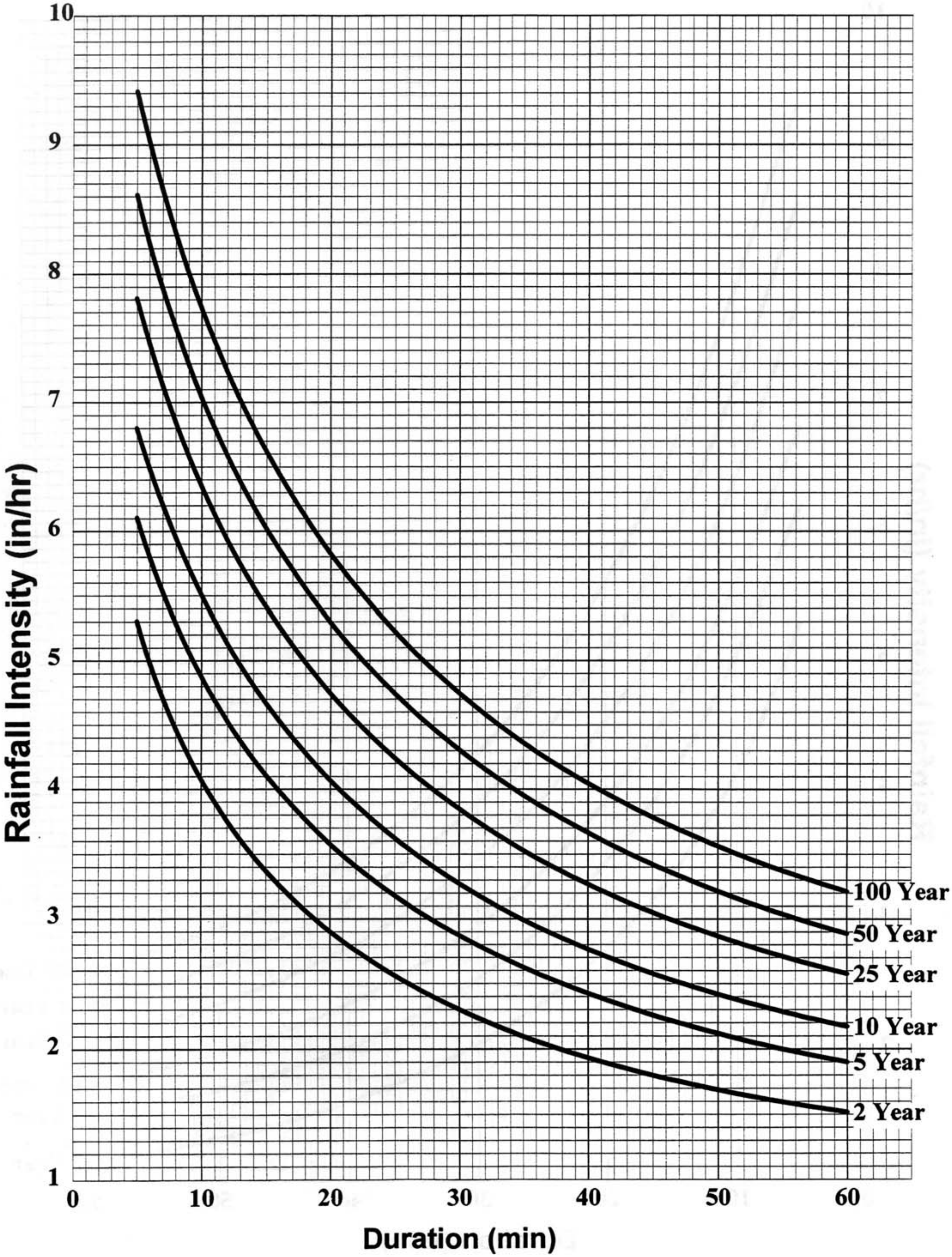
Rainfall Intensity (in/hr)
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Duration (min)
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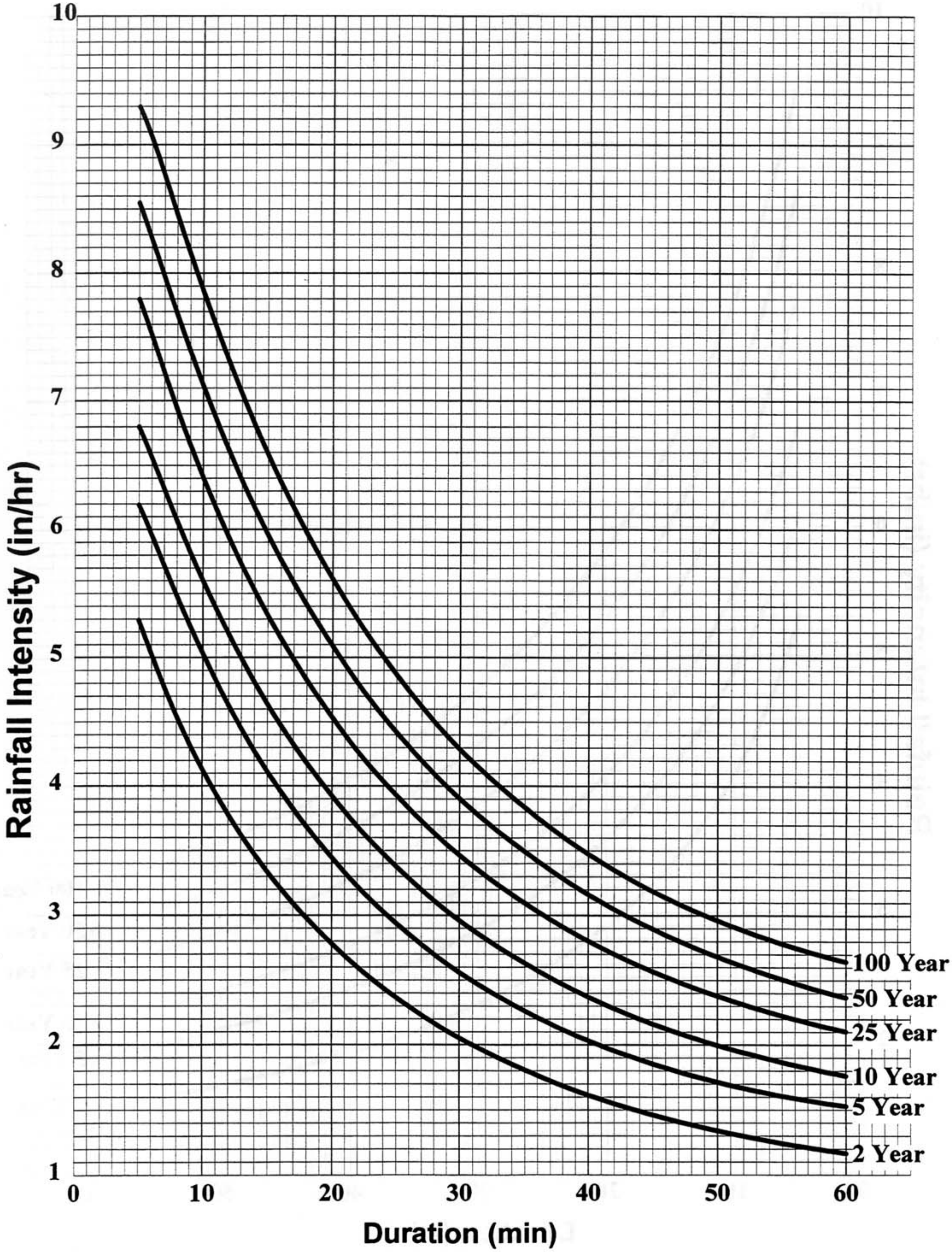
Rainfall Intensity (in/hr)
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Duration (min)
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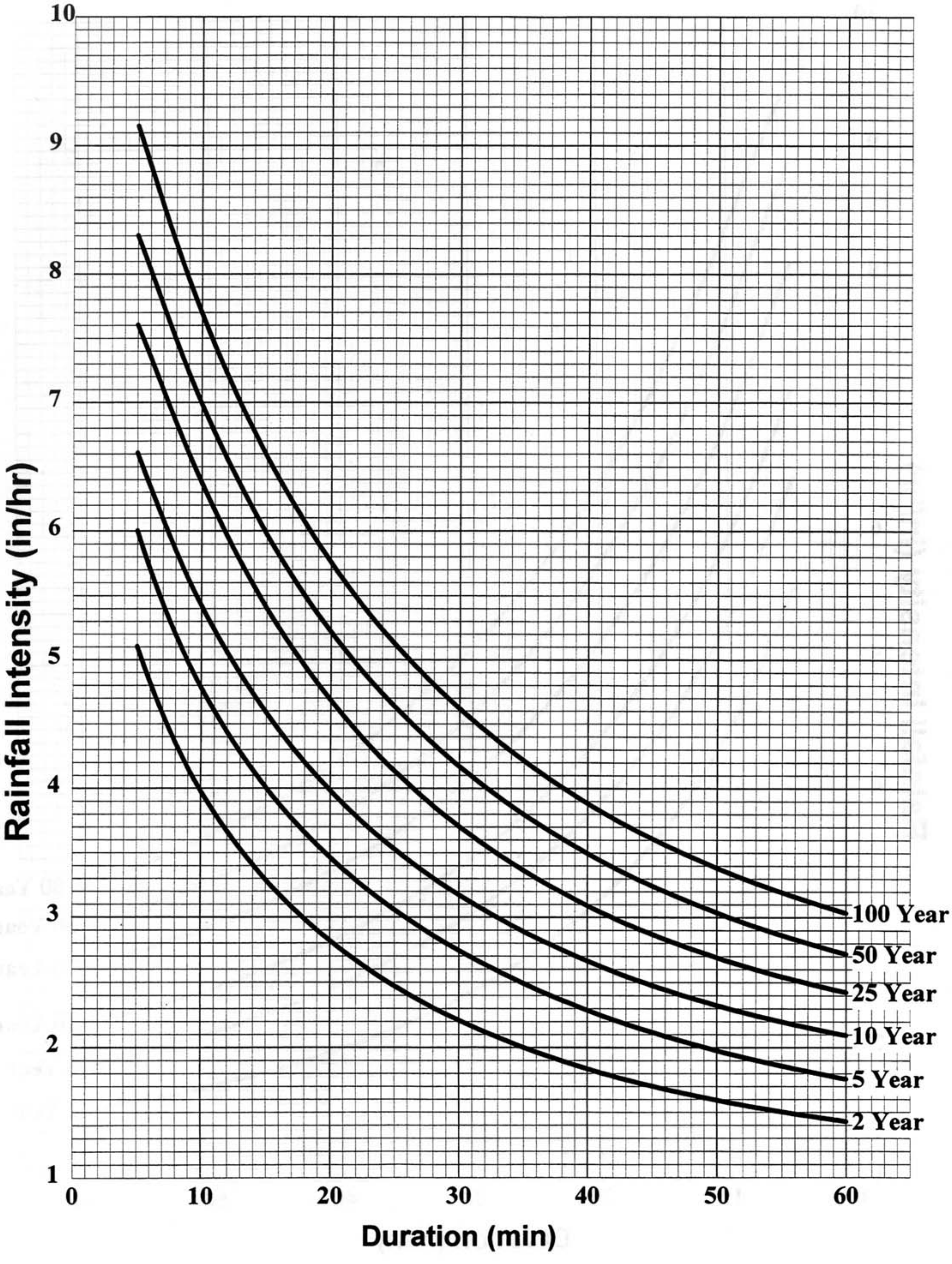
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Rainfall Intensity (in/hr)
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Duration (min)
100 Year
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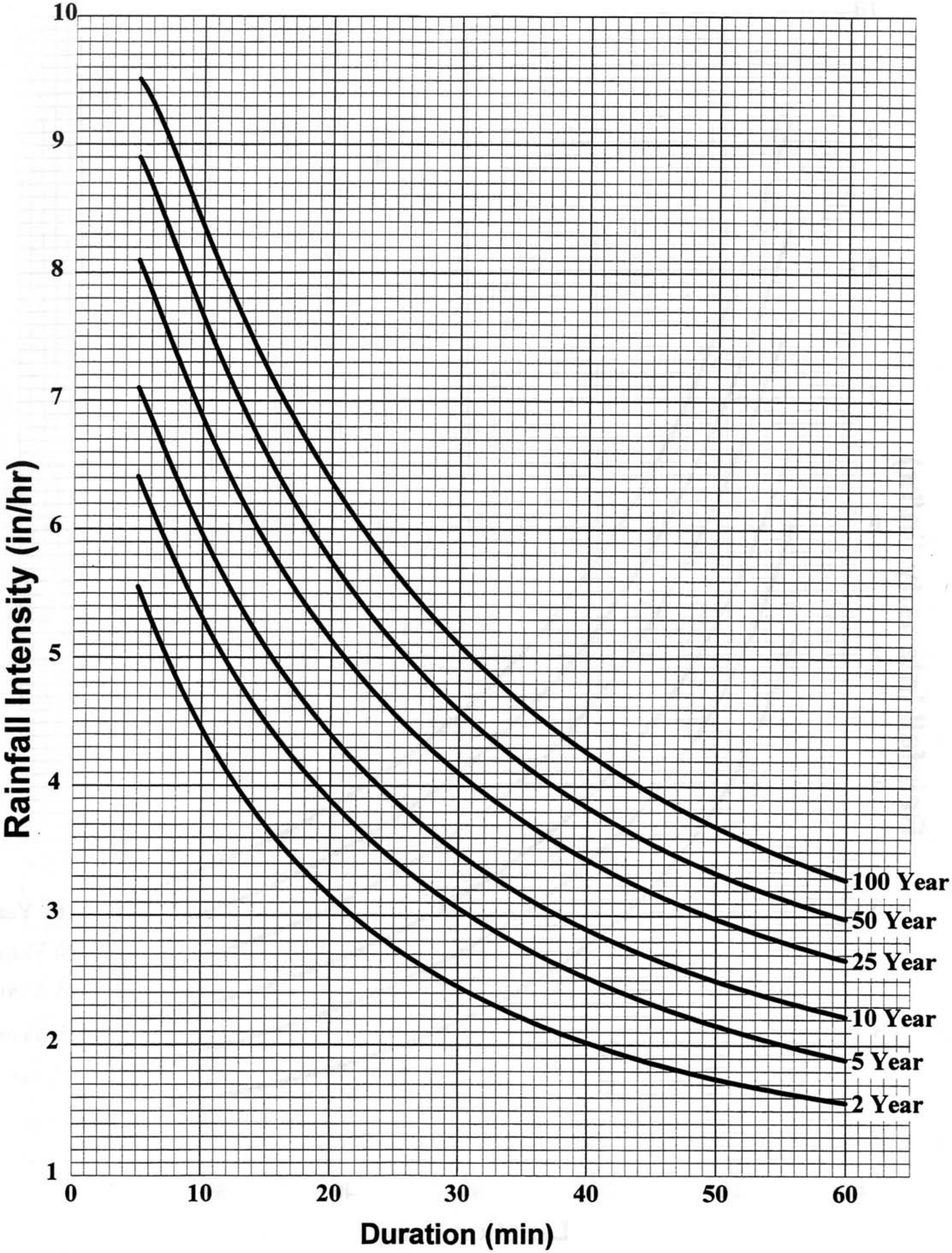
Rainfall Intensity (in/hr)
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Duration (min)
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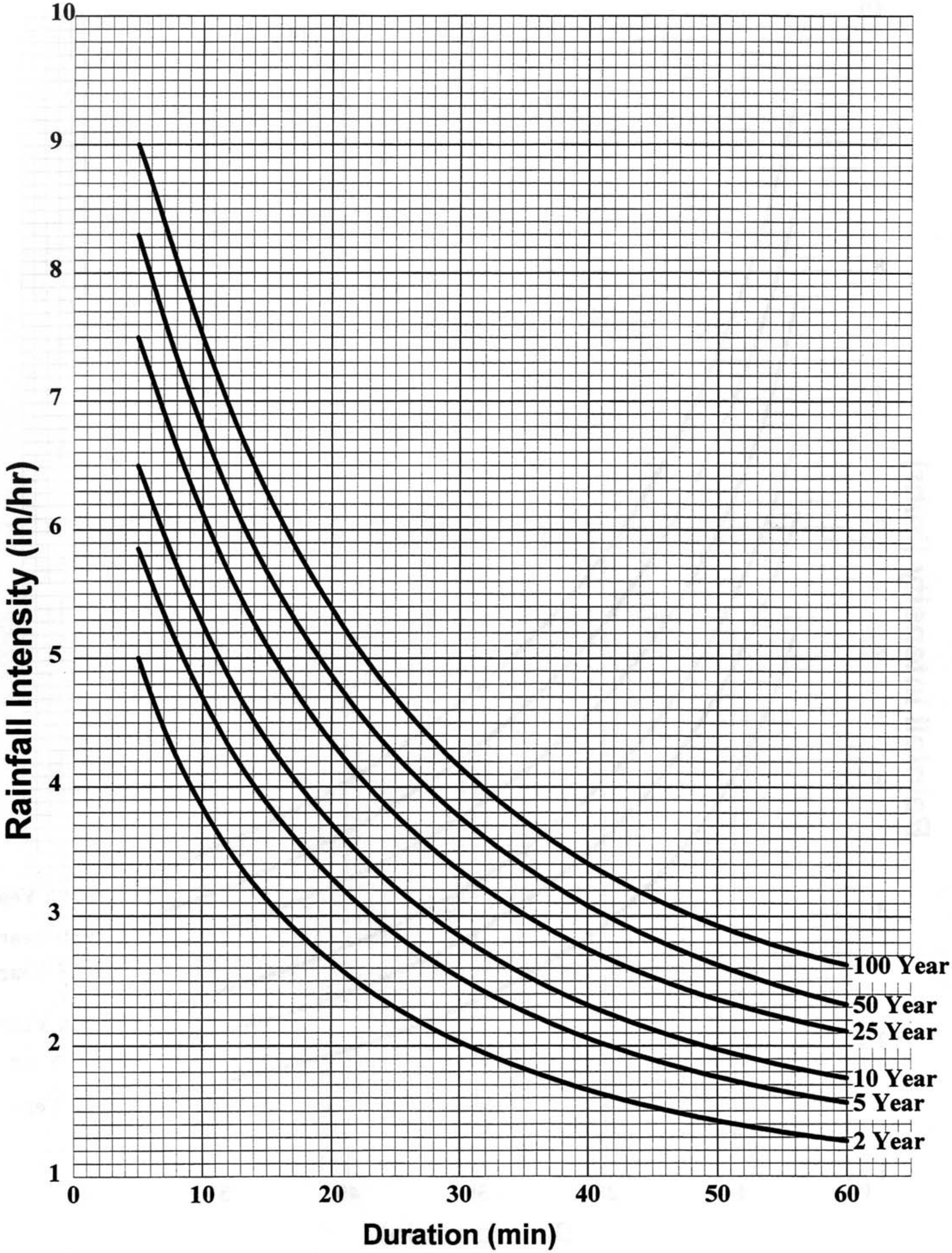
Rainfall Intensity (in/hr)
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Duration (min)
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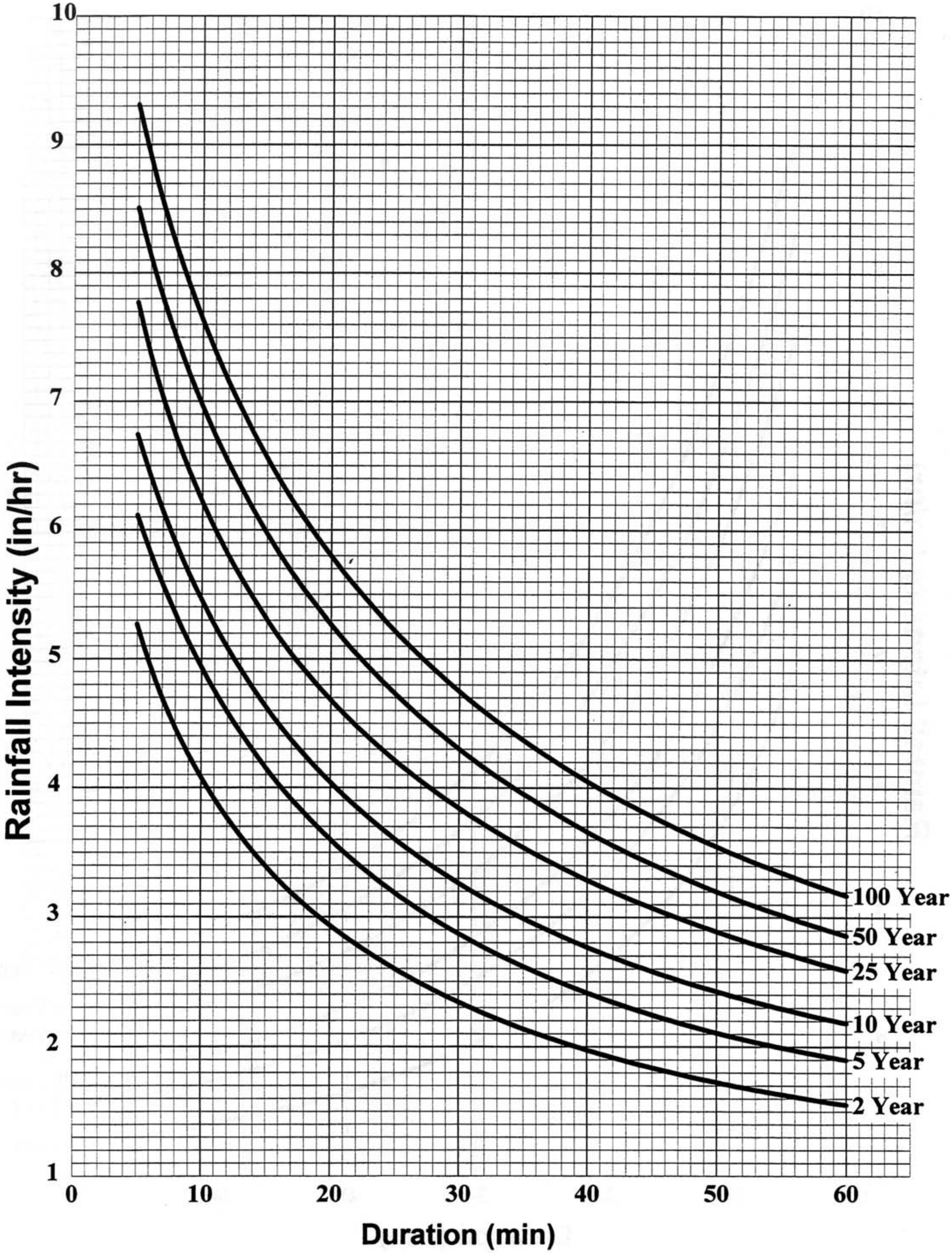
Rainfall Intensity (in/hr)
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Duration (min)
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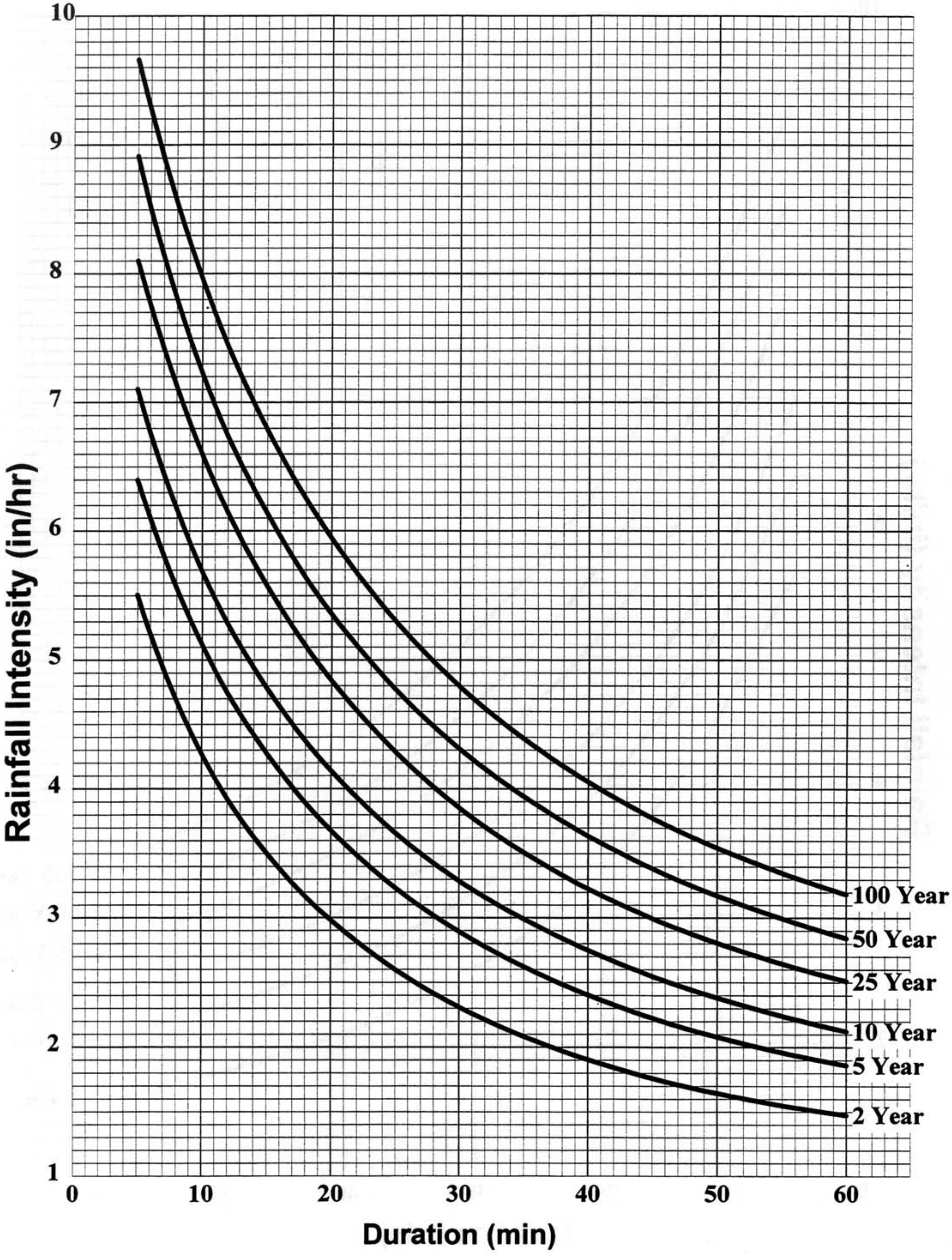
Rainfall Intensity (in/hr)
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Duration (min)
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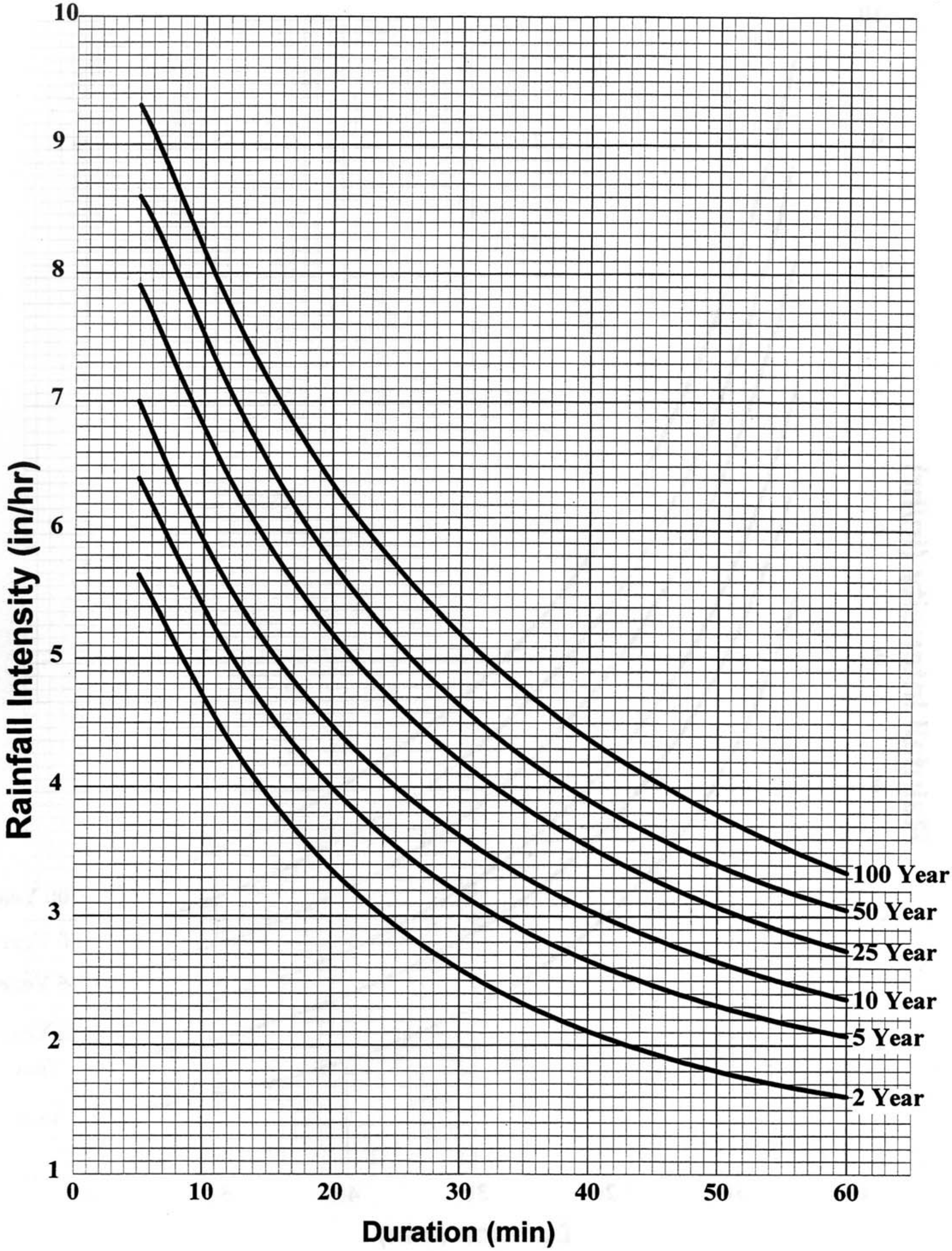
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Rainfall Intensity (in/hr)
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Duration (min)
100 Year
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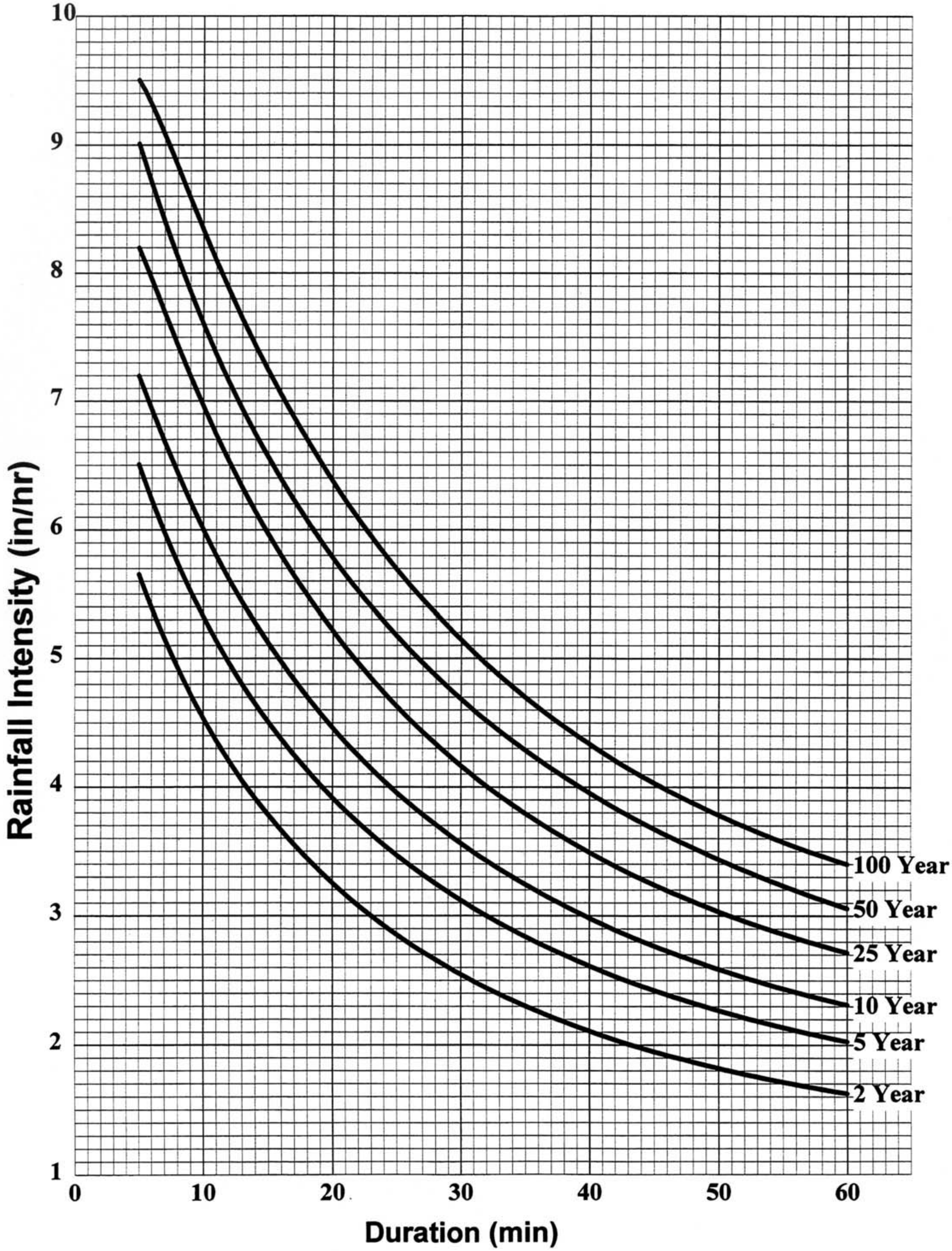
Rainfall Intensity (in/hr)
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Duration (min)
100 Year
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25 Year
10 Year
5 Year
2 Year

CHAPTER 5

ENGINEERING
CALCULATIONS

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LIST OF ILLUSTRATIONS (con't.)

LIST OF ILLUSTRATIONS (con't.)

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ENGINEERING CALCULATIONS

5-1 INTRODUCTION

This section provides guidelines for performing various engineering calculations associated with the design of stormwater management facilities such as extended-detention and retention basins and multi-stage outlet structures. The prerequisite information for using these calculations is the determination of the hydrologic characteristics of the contributing watershed in the form of the peak discharge (in *cfs*), or a runoff hydrograph, depending on the hydrologic and hydraulic routing methods used. (Refer to **Section 4-4** in **Chapter 4** for hydrologic methods.)

5-2 GENERAL INFORMATION: DETENTION, EXTENDED-DETENTION AND RETENTION BASIN DESIGN CALCULATIONS

Based on Virginia's Stormwater Management Regulations, a stormwater management basin may be designed to *control water quantity* (for flood control and channel erosion control) and to *enhance* (or treat) *water quality*. The type of basin selected (extended-detention, retention, infiltration, etc.) and the relationship between its design components *(design inflow, storage volume* and *outflow)* will dictate the size of the basin and serve as the basis for its hydraulic design. Some design component parameters such as *design storm return frequency*, *allowable discharge rates*, etc., may be specified by the local regulatory authority, based upon the specific needs of certain watersheds or stream channels within that locality. Occasionally, as in stream channel erosion control, it may be up to the engineer to document and analyze the specific needs of the downstream channel and establish the design parameters.

The ***design inflow*** is either the *peak flow* or the *runoff hydrograph* from the developed watershed. This *inflow* becomes the input data for the basin sizing calculations, often called ***routings***. Various routing methods are available. Note that the format of the hydrologic input data will usually be dictated by whatever routing method is chosen. (The methods discussed in this handbook require the use of a *peak discharge* or an actual *runoff hydrograph*.) Generally, larger and more complex projects will require a detailed analysis, which includes a runoff hydrograph. Preliminary studies and small projects may be designed using simpler, shortcut techniques that only require a peak discharge. For all projects, the designer must document the hydrologic conditions to support the inflow portion of the hydraulic relationship.

Achieving adequate ***storage volume*** within a basin can usually be accomplished by manipulation of the site grades and strategic placement of the permanent features such as buildings and parking lots. Sometimes, the location of a stormwater facility will be dictated by the site topography and available outfall location. (Refer to **Chapter 3** for basin planning considerations and design criteria.) Storage volume calculations will be discussed in detail later in this chapter.

5-3 ALLOWABLE RELEASE RATES

The allowable release rates for a stormwater facility are dependent on the proposed function(s) of that facility, such as ***flood control***, ***channel erosion control***, or ***water quality enhancement***. For example, a basin used for *water quality enhancement* is designed to detain the ***water quality volume*** and slowly release it over a specified amount of time. This water quality volume is the ***first flush*** of runoff, which is considered to contain the largest concentration of pollutants (Schueler 1987). (Refer to **Section 5-6** for water quality volume calculations.) In contrast, a basin used for *flood* or *channel erosion control* is designed to detain and release runoff from a given storm event at a *predetermined maximum release rate*. This *release rate* may vary from one watershed to another based on predeveloped conditions.

Localities, through stormwater management and erosion control ordinances, have traditionally set the allowable release rates for given frequency storm events to equal the watershed's pre-developed rates. This technique has become a convenient and consistent mechanism to establish the design parameters for a stormwater management facility, particularly as it relates to flood control or stream channel erosion control.

Chapter 4 discusses the impact of development on the hydrologic cycle and the difficulty in re-establishing the pre-developed runoff characteristics. Although it is popular to set a stormwater basin's allowable release rate to the watershed's pre-developed rate, this technique rarely duplicates existing conditions, particularly as it relates to storm frequencies and duration.

In Virginia, the allowable release rate for controlling stream channel erosion or flooding may be established by ordinance using the state's minimum criteria, or by analyzing specific downstream topographic, geographic or geologic conditions to select alternate criteria. **The engineer should be aware of what the local requirements are <u>before</u> designing**.

The design examples and calculations in this handbook use the state minimum requirements for illustrative purposes. **Example 1,** which considers a single homogeneous watershed, is summarized here to show the allowable release rates calculated for the basin. These release rates, as required by the state stormwater regulations, are the pre-developed runoff rates for the 2- and 10-year design storms. **Table 5-1** provides a summary of the hydrologic analysis for **Example 1**. (The complete solution to **Example 1** is provided in **Chapter 6**.)

TABLE 5 - 1
Hydrologic Summary**, **Example 1, SCS Methods

TR-55 GRAPHICAL PEAK DISCHARGE					
CONDITION	**DA**	**RCN**	t_c	Q_2	Q_{10}
PRE-DEV	25 *ac.*	64	0.87 *hr.*	8.5 *cfs**	26.8 *cfs**
POST-DEV	25 *ac.*	75	0.35 *hr.*	29.9 *cfs*	70.6 *cfs*
TR-20 COMPUTER RUN					
PRE-DEV	25 *ac.*	64	0.87 *hr.*	8.0 *cfs**	25.5 *cfs**
POST-DEV	25 *ac.*	75	0.35 *hr.*	25.9 *cfs*	61.1 *cfs*

***Allowable release rate**

5-4 STORAGE VOLUME REQUIREMENT ESTIMATES

Stormwater management facilities are designed using a trial and error process. The designer does many iterative routings to select a minimum facility size with the proper outlet controls. Each iterative routing requires that the facility size (*stage-storage relationship*) and the outlet configuration (*stage-discharge relationship*) be evaluated for performance against the watershed requirements. A graphical evaluation of the ***inflow hydrograph*** versus an approximation of the ***outflow rating curve*** provides the designer with an estimate of the required ***storage volume***. Starting with this **assumed** required volume, the number of iterations is reduced.

The ***graphical hydrograph analysis*** requires that the evaluation of the watershed's hydrology produce a runoff hydrograph for the appropriate design storms. The state stormwater management regulations allow the use of SCS methods or the modified rational method (critical storm duration approach) for analysis. Many techniques are available to generate the resulting runoff hydrographs based on these methods. It is the designer's responsibility to be familiar with the limitations and assumptions of the methods as they apply to generating hydrographs (refer to **Chapter 4, Hydrologic Methods**).

Graphical procedures can be time consuming, especially when dealing with multiple storms, and are therefore not practical when designing a detention facility for a small site development. Shortcut procedures have been developed to allow the engineer to approximate the storage volume requirements. Such methods include **TR-55: Storage Volume for Detention Basins**, **Section 5-4.2**, and **Critical Storm Duration-Modified Rational Method-Direct Solution**, **Section 5-4.4**, which can be used as planning tools. Final design should be refined using a more accurate hydrograph routing procedure. Sometimes, these shortcut methods may be used for final design, but they must be used with caution since they only approximate the required storage volume (refer to the assumptions and limitations for each method).

It should be noted that the TR-55 tabular hydrograph method does not produce a full hydrograph. The tabular method generates only the portion of the hydrograph that contains the peak discharge and some of the time steps just before and just after the peak. The missing values must be extrapolated, thus potentially reducing the accuracy of the hydrograph analysis. It is recommended that if SCS methods are to be used, a full hydrograph be generated using one of the available computer programs. The accuracy of the analysis can only be as accurate as the hydrograph used.

5-4.1 Graphical Hydrograph Analysis - SCS Methods

The following procedure represents a graphical hydrograph analysis that results in the approximation of the required storage volume for a proposed stormwater management basin. **Example 1** is presented here to illustrate this technique. See **Table 5-1** for a summary of the hydrology. The TR-20 computer-generated inflow hydrograph is used for this example. The allowable discharge from the proposed basin has been established by ordinance (based on pre-developed watershed discharge).

Information Needed:

The pre- and post-developed hydrology, which includes the pre-developed peak rate of runoff ***(allowable release rate)*** and the post-developed runoff hydrograph (***inflow hydrograph***) is required for hydrograph analysis (see **Table 5-1**).

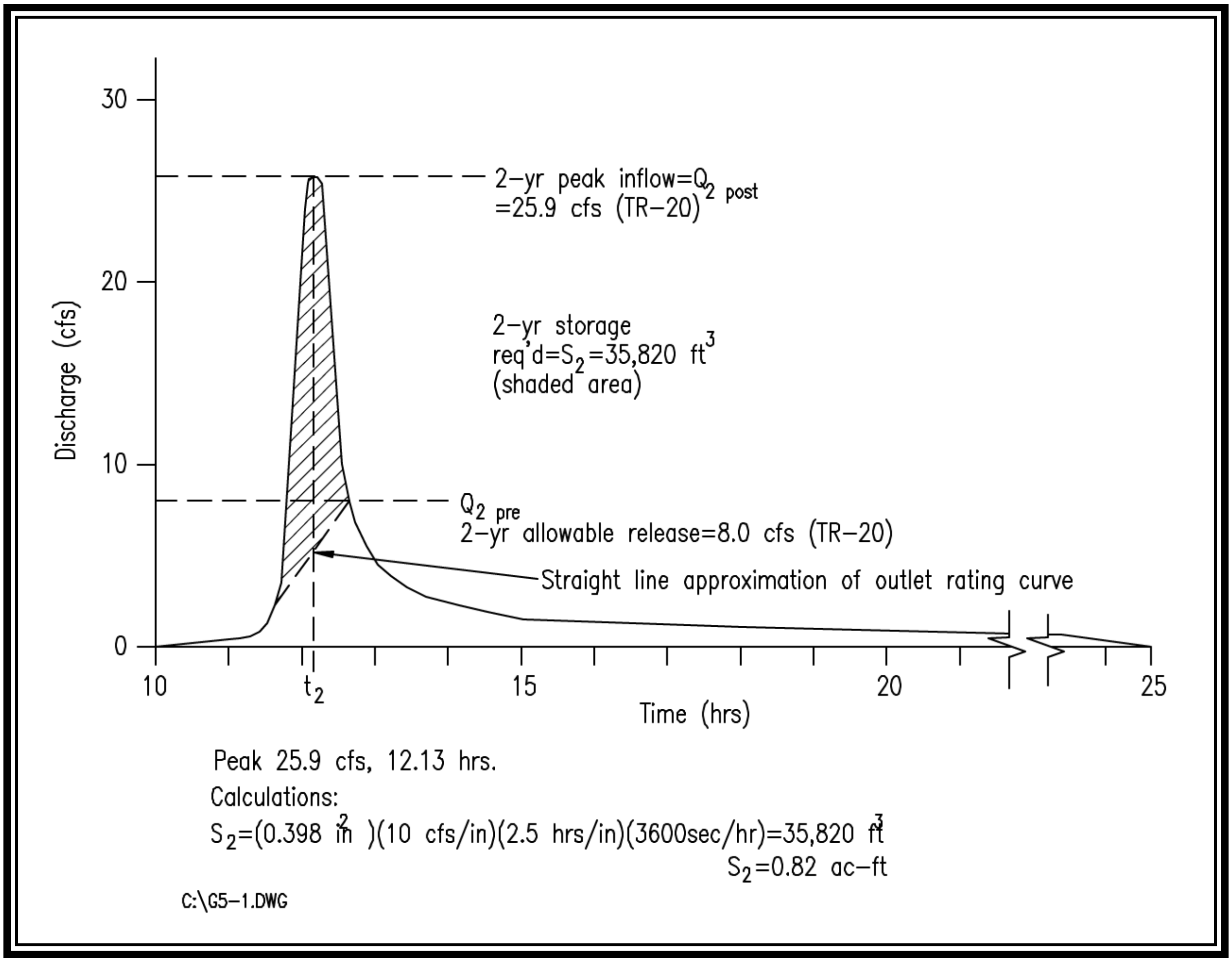
Procedure

(Refer to **Figure 5-1** for the 2-year developed inflow hydrograph and **Figure 5-2** for the 10-year developed inflow hydrograph):

1. Commencing with the plot of the 2-year developed inflow hydrograph (Discharge vs. Time), the 2-year allowable release rate, $Q_2 = 8\ cfs$, is plotted as a horizontal line starting at time $t = 0$ and continuing to the point where it intersects the falling limb of the hydrograph.

2. A diagonal line is then drawn from the beginning of the inflow hydrograph to the intersection point described above. This line represents the ***hypothetical rating curve*** of the control structure and approximates the rising limb of the outflow hydrograph for the 2-year storm.

FIGURE 5 - 1
SCS Runoff Hydrograph, Example 1, 2-Year Post-Developed



3. The ***storage volume*** is then approximated by calculating the area under the inflow hydrograph, less the area under the rising limb of the outflow hydrograph. This is shown as the shaded area in **Figure 5-1**. The storage volume required for the 2-year storm, S_2, can be approximated by measuring the shaded area with a planimeter.

The vertical scale of a hydrograph is in cubic feet per second (*cfs*) and the horizontal scale is in hours (hrs). Therefore, the area, as measured in square inches (in^2), is multiplied by scale conversion factors of *cfs* per inch, hours per inch, and 3600 seconds per hour, to yield an area in cubic feet (ft^3). The conversion is as follows:

$$
\begin{aligned}
S_2 &= (0.398\ in^2)(10\ cfs/in.)(2.5\ hrs./in.)(3{,}600\ sec./hr.) \\
&= 35{,}820\ ft^3 \\
&= 0.82\ ac.ft.
\end{aligned}
$$

4. On a plot of the 10-year inflow hydrograph, the 10-year allowable release rate, Q_{10} , is plotted as a horizontal line extending from time zero to the point where it intersects the falling limb of the hydrograph.

5. By trial and error, the time t_2 , at which the S_2 volume occurs while maintaining the 2-year release, is determined by planimeter. This is represented by the shaded area to the left of t_2 on **Figure 5-2**. From the intersection point of t_2 and the 2-year allowable release rate, Q_2, a line is drawn to connect to the intersection point of the 10-year allowable release rate and the falling limb of the hydrograph. This intersection point is t_{10} , and the connecting line is a straight line approximation of the *outlet rating curve*.

6. The area under the inflow hydrograph from time t_2 to time t_{10} , less the area under the rising limb of the hypothetical rating curve, represents the additional volume (shaded area to the right of t_2 on **Figure 5-2**) needed to meet the 10-year storm storage requirements.

7. The total storage volume, S_{10} , required, can be computed by adding this additional storage volume to S_2 . This is represented by the total shaded area under the hydrograph.

$$\begin{aligned} S_{10} &= (0.89\ in^2)(10\ cfs/in.)(2.5\ hrs./in.)(3{,}600\ sec./hr.) \\ &= 80{,}100\ ft^3 \\ &= 1.84\ ac.ft. \end{aligned}$$

These steps may be repeated if storage of the 100-year storm, or any other design frequency storm, is required by ordinance or downstream conditions.

In summary, the total volume of storage required is the area <u>under</u> the runoff hydrograph curve and <u>above</u> the basin outflow curve. It should be noted that the outflow rating curve is approximated as a straight line. The actual shape of the outflow rating curve will depend on the type of outlet device used. **Figure 5-3** shows the typical shapes of outlet rating curves for orifice and weir outlet structures. The straight line approximation is reasonable for an orifice outlet structure. However, this approximation will likely **underestimate** the storage volume required when a weir outlet structure is used. Depending on the complexity of the design and the need for an exact engineered solution, the use of a more rigorous sizing technique, such as a storage indication routing, may be necessary.

FIGURE 5 - 2
SCS Runoff Hydrograph, Example 1, 10-Year Post-Developed

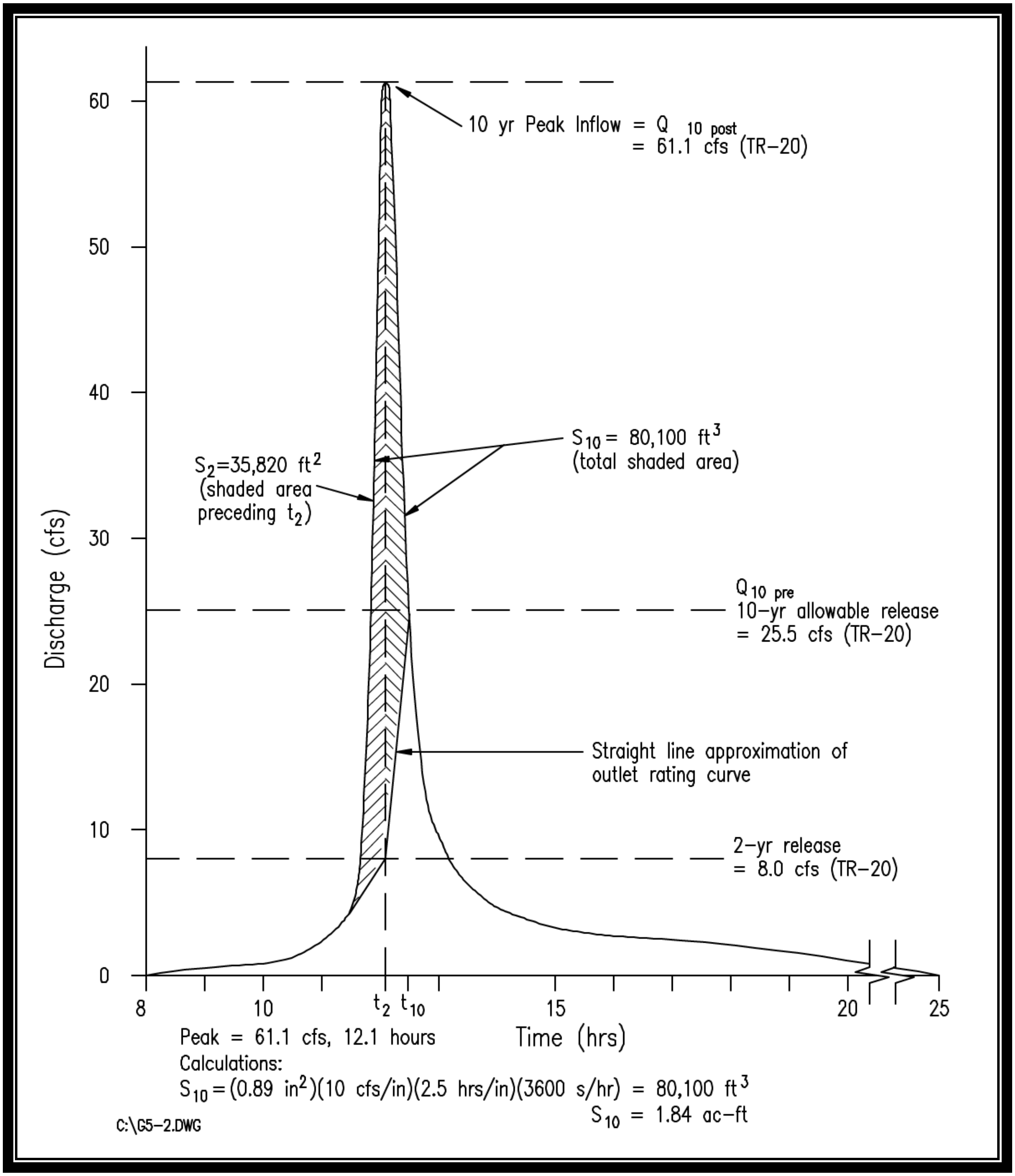
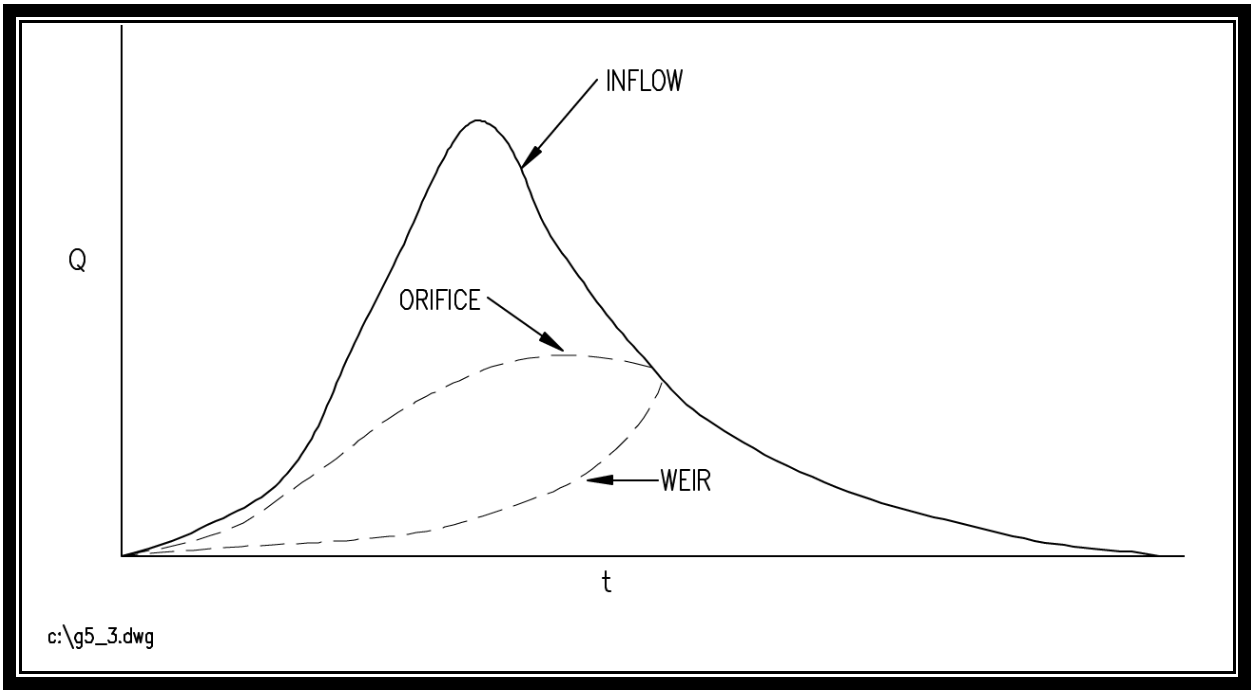


FIGURE 5 - 3

Typical Outlet Rating Curves for Orifice and Weir Outlet Devices

5-4.2 TR-55: Storage Volume for Detention Basins (Shortcut Method)



The TR-55 Storage Volume for Detention Basins, or TR-55 shortcut procedure, provides similar results to the graphical analysis described in **Section 5-4.1**. This method is based on average storage and routing effects for many structures. TR-55 can be used for single-stage or multi-stage outflow devices. The only constraints are that 1) *each stage requires a design storm and a computation of the storage required for it*, and 2) *the discharge of the upper stage(s) includes the discharge of the lower stage(s)*. Refer to TR-55 for more detailed discussions and limitations.

Information Needed:

To calculate the required storage volume using TR-55, the pre- and post-developed hydrology per SCS methods is needed (refer to **Chapter 4**). This includes the watershed's *pre-developed peak rate of discharge,* or *allowable release rate*, Q_o, the watershed's *post-developed peak rate of discharge*, or *inflow*, Q_i, for the appropriate design storms, and the watershed's *post-developed runoff*, Q, in inches. (Note that this method does **not** require a hydrograph.)

Once the above parameters are known, the TR-55 Manual can be used to approximate the storage volume required for each design storm. The following procedure summarizes the TR-55 shortcut method using the 25-acre watershed presented in **Example 1**.

Procedure:

1. Determine the peak developed inflow, Q_i, and the allowable release rate, Q_o, from the hydrology for the appropriate design storm. The 2-year storm flow rates from Example 1 (TR-55 Graphical peak discharge) are used here:

$$Q_{o_2} = 8.5\ cfs\ ; \quad Q_{i_2} = 29.9\ cfs$$

   Using the ratio of the allowable release rate, Q_o, to the peak developed inflow, Q_i, or Q_o/Q_i, for the appropriate design storm, use **Figure 5-4** (or Figure 6-1 in TR-55) to obtain the ratio of storage volume, V_s, to runoff volume, V_r, or V_s/V_r.

   From **Example 1**:

$$Q_{o_2}/Q_{i_2} = 8.5 / 29.9 = 0.28$$

   From **Figure 5-4** or TR-55 Figure 6.1:

$$V_{s_2}\ /\ V_{r_2}\ ;39$$

2. Determine the runoff volume, V_r, in *ac.ft.*, from the TR-55 worksheets for the appropriate design storm.

$$V_r = Q\ A_m\ 53.33$$

where:

	Q	=	*runoff, in inches, from TR-55 Worksheet 2*
	A_m	=	*drainage area, in square miles*
	53.33	=	*conversion factor to acre-feet*

From **Example 1**:

$$Q_2 = 1.30\ in.$$
$$A_m = 25\ ac. / 640\ ac./mi^2 = 0.039\ mi^2$$

$$V_{r_2} = 1.30(.039)\ 53.33$$
$$= 2.70\ ac.ft.$$

3. Multiply the V_s/V_r ratio from Step 1 by the runoff volume, V_r, from Step 2, to determine the volume of storage required, V_s, in acre-feet.

$$\left(\frac{V_s}{V_r}\right)V_r = V_s$$

From **Example 1**:

$$(.39)(2.70\ ac.ft.) = 1.05\ ac.ft.$$

4. Repeat these steps for each additional design storm as required to determine the approximate storage requirements. The 10-year storm storage requirements from **Example 1** are presented here:

    a. $Q_o = 26.8\ cfs$
    $Q_i = 70.6\ cfs$

    $Q_o/Q_i = 26.8/70.6 = 0.38$; From **Figure 5-4** or TR-55 Figure 6-1: $V_s/V_r = .33$

    b. $V_r = QA_m\, 53.33 = 2.85\ in.(.039\ sq.mi.)(53.33) = 5.93\ ac.ft.$

    c. $V_s = (V_s/V_r)V_r = (.33)\ 5.93\ ac.ft. = 1.96\ ac.ft.$

This volume represents the total storage required for the 2-year storm and the 10-year storm.

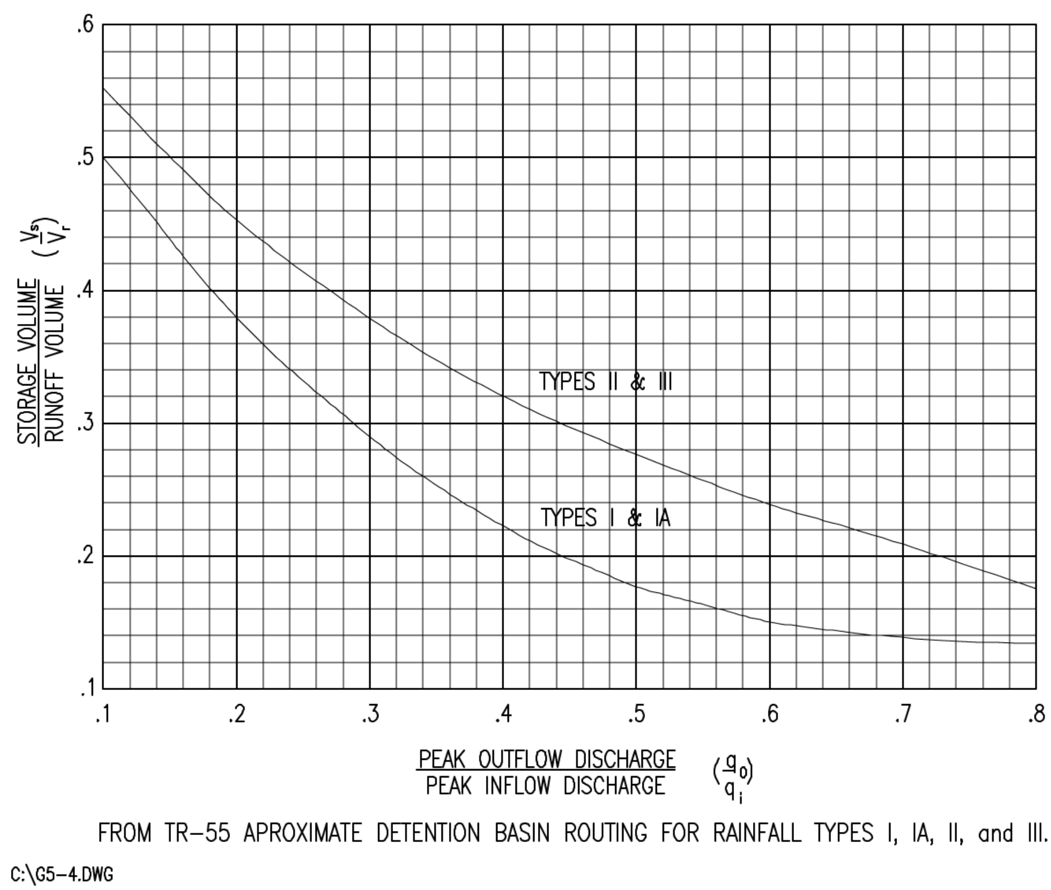
5. NOTE: The volume from **#4** above may need to be increased if additional storage is required for water quality purposes or channel erosion control. Refer to **Section 5-6** or **Section 5-10**, respectively.

The design procedure presented above should be used with TR-55 Worksheet 6a, as shown in **Example 1** of **Chapter 6**. The worksheet includes an area to plot the *stage-storage curve*, from which actual elevations corresponding to the required storage volumes can be derived. **Table 5-2** provides a summary of the required storage volumes using the graphical SCS hydrograph analysis and the TR-55 shortcut method.

TABLE 5 - 2
Storage Volume Requirements, Example 1

Method	2-*yr*. Storage Required	10-*yr*. Storage Required
Graphical Hydrograph Analysis	0.82 *ac.ft.*	1.84 *ac.ft.*
TR-55 Shortcut Method	1.05 *ac.ft.*	1.96 *ac.ft.*

FIGURE 5 - 4
Approximate Detention Basin Routing for Rainfall Types I, IA, II and III



Source: SCS TR-55 Urban Hydrology for Small Watersheds: Figure 6-1

5-4.3 Graphical Hydrograph Analysis, Modified Rational Method - Critical Storm Duration

The Modified Rational Method uses the ***critical storm duration*** to calculate the ***maximum storage volume*** for a detention facility. This *critical storm duration* is the storm duration that generates the greatest volume of runoff and, therefore, requires the most storage. In contrast, the Rational Method produces a triangular runoff hydrograph that gives the peak inflow at time = t_c and falls to zero flow at time = $2.5t_c$. In theory, this hydrograph represents a storm whose duration equals the time of concentration, t_c, resulting in the greatest peak discharge for the given return frequency storm. The volume of runoff, however, is of greater consequence in sizing a detention facility. A storm whose duration is longer than the t_c may not produce as large a peak rate of runoff, but it may generate a greater **volume** of runoff. By using the Modified Rational Method, the designer can evaluate several different storm durations to verify which one requires the greatest volume of storage with respect to the allowable release rate. It is this *maximum storage volume* that the basin must be designed to detain.

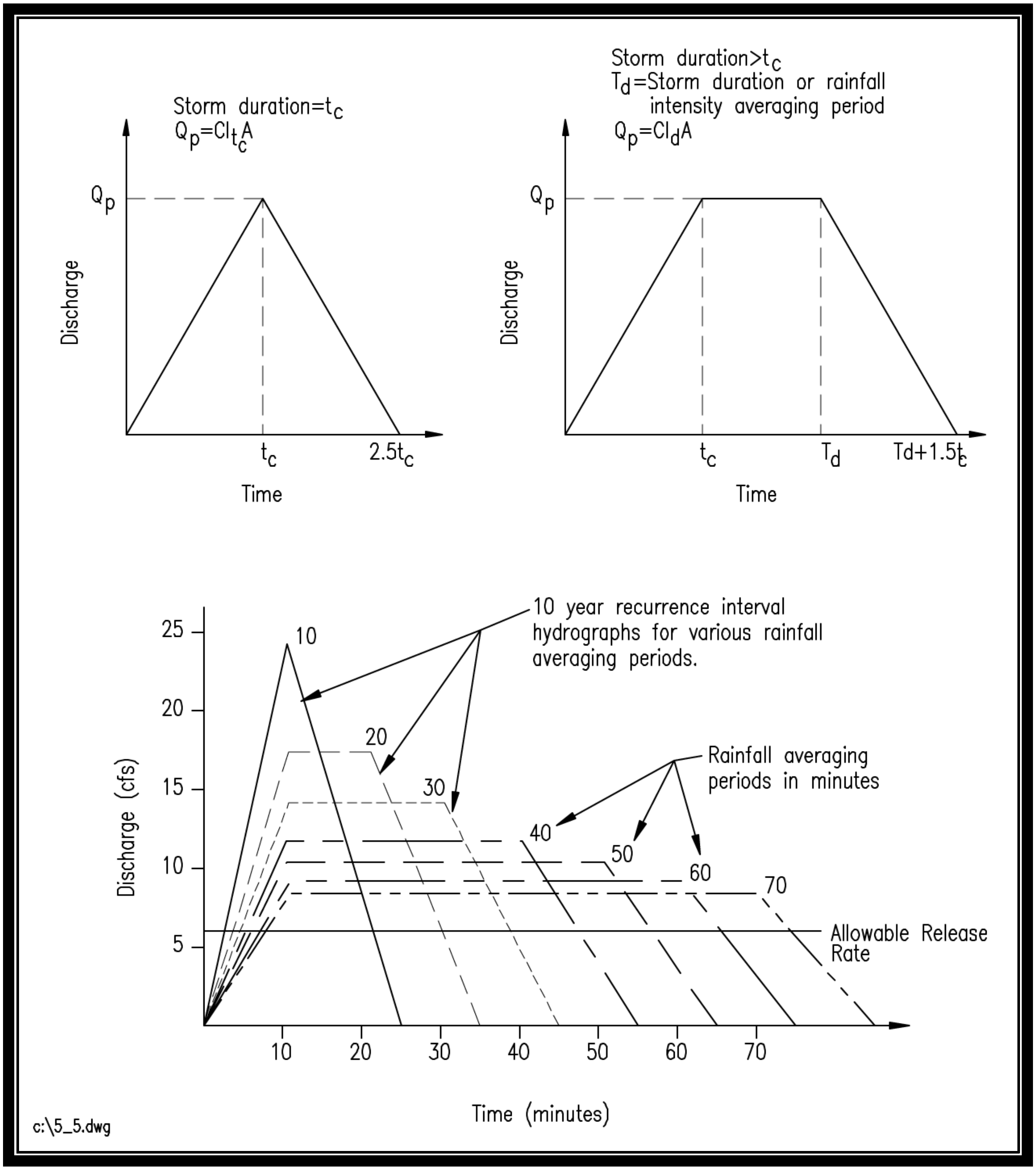
The first step in determining the critical storm duration is to use the post-developed time of concentration, t_c, to generate a post-developed runoff hydrograph. Rainfall intensity averaging periods, T_d, representing time periods incrementally longer than the t_c, are then used to generate a "family" of runoff hydrographs for the same drainage area. These hydrographs will be trapezoidal with the peak discharges, Q_i, based upon the intensity, I, of the averaging period, T_d. **Figure 5-5** shows the construction of a typical triangular and trapezoidal hydrograph using the modified rational method, and a family of trapezoidal hydrographs representing storms of different durations.

Note that the duration of the receding limb of the trapezoidal hydrograph, in **Figure 5-5**, is set to equal 1.5 times the time of concentration, t_c. Also, the total hydrograph duration is $2.5t_c$ versus $2t_c$ as discussed in **Chapter 4**. This longer duration is considered more representative of actual storm and runoff dynamics. It is also more analogous to the SCS unit hydrograph where the receding limb extends longer than the rising limb.

The Modified Rational Method assumes that the rainfall intensity averaging period is equal to the actual storm duration. This means that the rainfall and runoff that occur before and after the rainfall averaging period are not accounted for. **Therefore, the Modified Rational Method may underestimate the required storage volume for any given storm event**.

The rainfall intensity averaging periods are chosen arbitrarily. However, the designer should select periods for which the corresponding intensity-duration-frequency (I-D-F) curves are available, i.e., 10 *min*., 20 *min*., 30 *min*., etc. The shortest period selected should be the time of concentration, t_c. A straight line starting at $Q = 0$ and $t = 0$ and intercepting the inflow hydrograph on the receding limb at the allowable release rate, Q_o, represents the outflow rating curve. The time averaging period hydrograph that represents the greatest storage volume required is the one with the largest area between the inflow hydrograph and outflow rating curve. This determination is made by a graphical analysis of the hydrographs.

FIGURE 5 - 5
Modified Rational Method Hydrographs



The following procedure represents a graphical analysis very similar to the one described in **Section 5-4.1**. **Example 1** from **Chapter 6** will be used again. Note that **the rational and modified rational methods should normally be used in homogeneous drainage areas of less than 20 acres, with a t_c of less than 20 minutes**. Although the watershed in **Example 1** has a drainage area of 25 acres and a t_c of greater than 20 minutes, it will be used here for illustrative purposes. Note that the pre- and post-developed peak discharges are much greater than those calculated using the SCS method applied to the same watershed. This difference may be the result of the large acreage and t_c values.

A summary of the hydrology is found in **Table 5-3**. Note that the t_c calculations were performed using the more rigorous SCS TR-55 method.

TABLE 5 - 3
Hydrologic Summary, Example 1, Rational Method

Rational Method					
CONDITION	**D.A.**	***C***	**T_c**	**Q_2**	**Q_{10}**
Pre-developed	*25 ac.*	*.38*	*.87 hr* *52 min.*	*17 cfs*	*24 cfs*
Post-developed	*25 ac.*	*.59*	*.35 hr.* *21 min.*	*49 cfs*	*65 cfs*

Information Needed:

The Modified Rational Method-Critical Storm Duration Approach is very similar to SCS methods since it requires pre- and post-developed hydrology in the form of a pre-developed peak rate of runoff (***allowable release rate)*** and a post-developed runoff hydrograph (***inflow hydrograph***), as developed using the Rational Method.

Procedure: (Refer to **Figures 5-6** and **5-7**.)

1. Plot the 2-year developed condition inflow hydrograph (triangular) based on the developed condition, t_c.

2. Plot a family of hydrographs, with the time averaging period, T_d, of each hydrograph increasing incrementally from 21 minutes (developed condition t_c) to 60 minutes, as shown in **Figure 5-6**. Note that the first hydrograph is a Type 1 Modified Rational Method triangular hydrograph, as shown in **Figure 4-7** in **Chapter 4,** where the storm duration, *d*, or T_d, is equal

to the time of concentration, t_c . The remaining hydrographs are trapezoidal, or Type 2 hydrographs. The peak discharge for each hydrograph is calculated using the rational equation, $Q = CIA$, where the intensity, I, from the I-D-F curve is determined using the rainfall intensity averaging period as the storm duration.

3. Superimpose the outflow rating curve on each inflow hydrograph. The area between the two curves then represents the storage volume required, as shown in **Figure 5-6**. Similar cautions, as described in the SCS Methods, **Section 5-4.1**, regarding the straight line approximation of the outlet discharge curve apply here as well. The actual shape of the outflow curve depends on the type of outlet device.

4. Compute and tabulate the required storage volume for each of the selected rainfall durations or time averaging periods, T_d, using the procedures described in **Section 5-4.1**.

   The storm duration that requires the maximum storage is the *critical storm* and is used for the sizing of the basin. (A storm duration equal to the t_c produces the largest storage volume required for the 2-year storm presented here.)

5. Repeat Steps 1 through 4 above for the analysis of the 10-year storage requirements. (**Figure 5-7 represents this procedure repeated for the 10-year design storm**.)

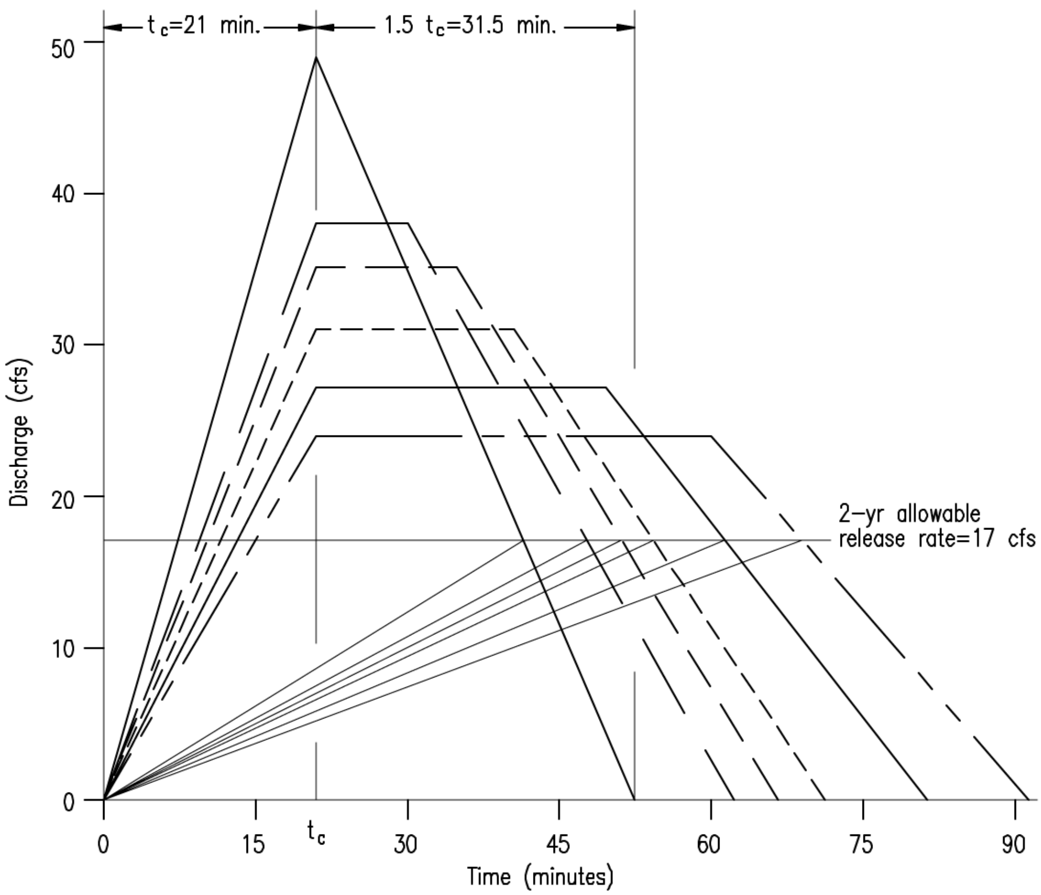
Conveyance systems should still be designed using the Rational Method, as opposed to the Modified Rational Method, to ensure their design for the peak rate of runoff.

TABLE 5 - 4
Storage Volume Requirements - Example 1

Method	2-*yr*. Storage Required	10-*yr*. Storage Required
Graphical Hydrograph Analysis	*0.82 ac.ft.*	*1.84 ac.ft.*
TR-55 Shortcut Method	*1.05 ac.ft.*	*1.96 ac.ft.*
Modified Rational Method - Critical Storm Duration	*1.16 ac.ft.* T_d = *21 min.*	*1.56 ac.ft.* T_d = *40 min.*

FIGURE 5 - 6

Modified Rational Method Runoff Hydrograph, Example 1, 2-Year Post-Developed Condition



Conversion – in^2 to ft^3= 10 cfs/in x 0.25 hrs/in x 3600 sec/hr=9000 ft^3/in^2

Rainfall Avg. Period (T)	Intensity I(in/hr)	Q Peak (cfs)	Area (in^2)	Storage Volume (ft^3)
21 min	3.3	49	5.62	50,580*
30 min	2.6	38	5.51	49,590
35 min	2.4	35	5.53	49,770
40 min	2.1	31	5.29	47,610
50 min	1.8	27	5.25	47,250
60 min	1.6	24	5.24	47,160

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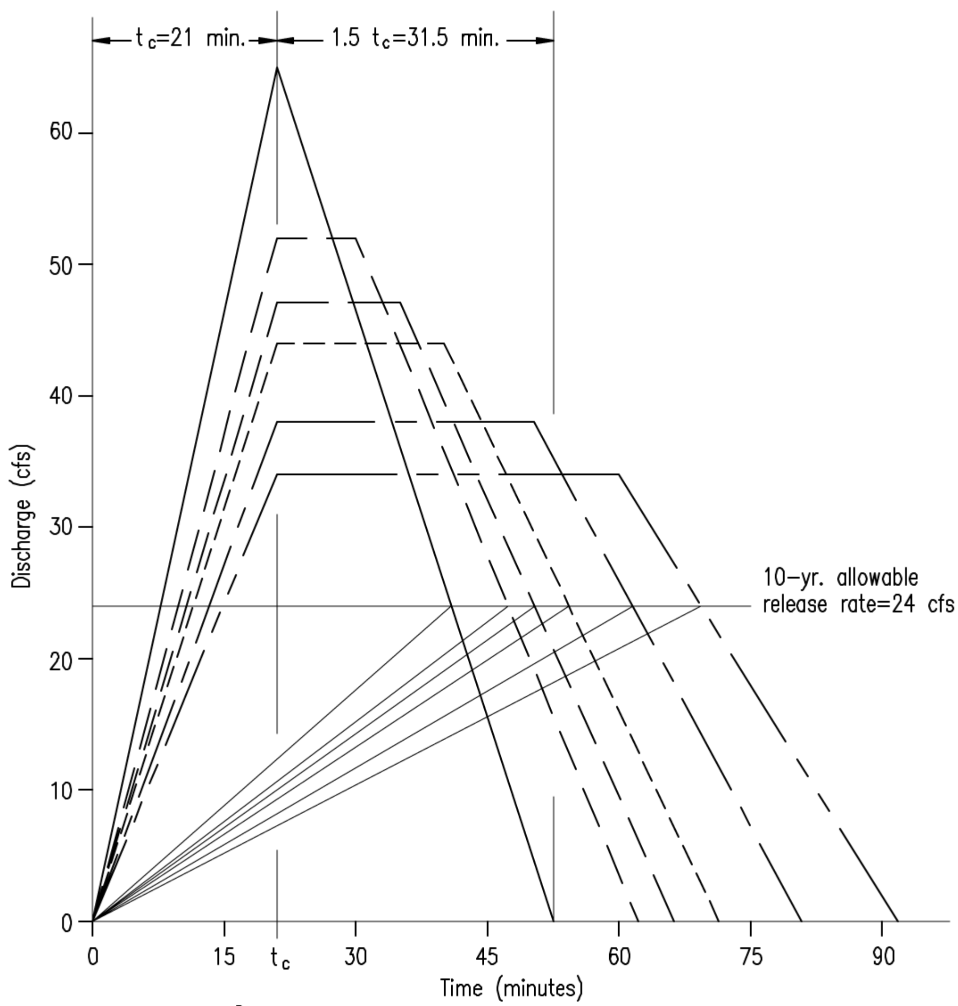
FIGURE 5 - 7
Modified Rational Method Runoff Hydrograph, Example 1,
10-Year Post-Developed Condition



Conversion – in^2 to ft^3= 9000 ft^3/in^2

Rainfall Avg. Period (T)	Intensity I(in/hr)	Q Peak (cfs)	Area (in^2)	Storage Volume (ft^3)
21 min	4.4	65	7.07	63,630
30 min	3.5	52	7.33	65,970
35 min	3.2	47	7.31	65,790
40 min	3.0	44	7.53	67,770*
50 min	2.6	38	7.45	67,050
60 min	2.3	34	7.48	67,320

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5-4.4 Modified Rational Method, Critical Storm Duration - Direct Solution

A direct solution to the Modified Rational Method, Critical Storm Duration has been developed to eliminate the time intensive, iterative process of generating multiple hydrographs. This direct solution takes into account the storm duration and allows the designer to solve for the time at which the storage volume curve has a slope equal to zero, which corresponds to maximum storage. The basic derivation of this method is provided below, followed by the procedure as applied to **Example 1**.

Storage Volume

The runoff hydrograph developed with the Modified Rational Method, Critical Storm Duration will be either triangular or trapezoidal in shape. The outflow hydrograph of the basin is approximated by a straight line starting at *0 cfs* at the time *t=0* and intercepting the receding leg of the runoff hydrograph at the allowable discharge, q_o.

> ***The straight line representation of the outflow hydrograph is a conservative approximation of the shape of the outflow hydrograph for an orifice control release structure. This method should be used with caution when designing a weir control release structure.***

The required storage volume is represented by the area between the inflow hydrograph and the outflow hydrograph in **Figure 5-8**. This area can be approximated using the following equation:

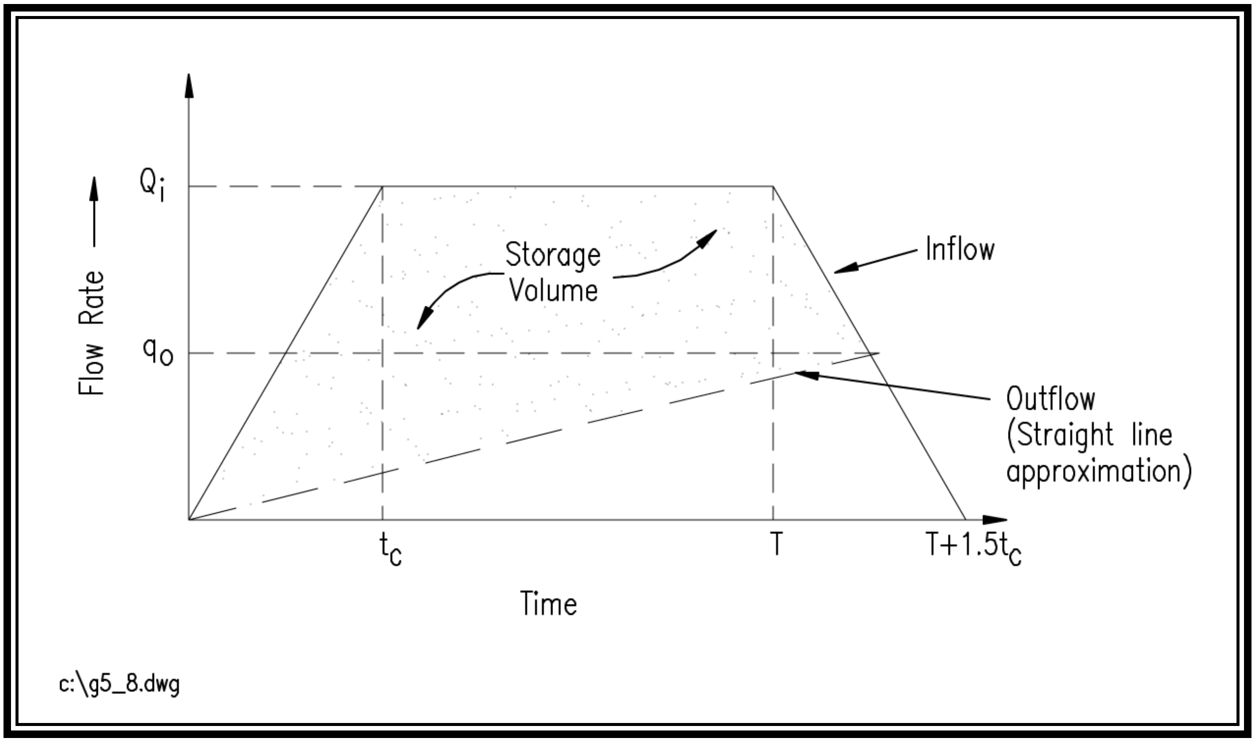
$$V = \left[Q_i T_d + \frac{Q_i t_c}{4} - \frac{q_o T_d}{2} - \frac{3 q_o t_c}{4}\right] 60$$

Equation 5-1
Trapezoidal Hydrograph Storage Volume Equation

Where:
V = *required storage volume,* ft^3
Q_i = *inflow peak discharge, cfs, for the critical storm duration,* T_d
t_c = *post-developed time of concentration, min.*
q_o = *allowable peak outflow, cfs*
T_d = *critical storm duration, min.*

The allowable peak outflow is established by ordinance or downstream conditions. The ***critical storm duration***, T_d, is an unknown and must be determined to solve for the intensity, *I*, and to ultimately calculate the peak inflow, Q_i. Therefore, a relationship between rainfall intensity, *I*, and the critical storm duration, T_d, must be established.

FIGURE 5 - 8
Trapezoidal Hydrograph Storage Volume Estimate



Rainfall Intensity

The rainfall intensity as taken from the I-D-F curves is dependent on the time of concentration, t_c, of a given watershed. Setting the storm duration, T_d, equal to the time of concentration, t_c, will provide the maximum peak discharge. As stated previously, however, it does not necessarily generate the maximum ***volume*** of discharge. Since this maximum volume of runoff is of interest, and the storm duration is unknown, the rainfall intensity, *I*, must be represented as a function of *time*, *frequency*, and *location*. The relationship is expressed as follows:

$$I = \frac{a}{b+T_d}$$

Equation 5-2
Modified Rational Method Intensity, (*I*), Equation

where: I = *rainfall intensity, in./hr.*
T_d = *rainfall duration or rainfall intensity averaging period, min.*
a & b = *rainfall constants developed for storms of various recurrence intervals and various geographic locations, as shown in **Table 5-5***

TABLE 5 - 5
Rainfall Constants for Virginia*

Duration - 5 minutes to 2 hours			
Station	**Rainfall Frequency**	**Constants**	
Wytheville	2	117.7	19.1
	5	168.6	23.8
	10	197.8	25.2
Lynchburg	2	118.8	17.2
	5	158.9	20.6
	10	189.8	22.6
Richmond	2	130.3	18.5
	5	166.9	20.9
	10	189.2	22.1
Norfolk	2	126.3	17.2
	5	173.8	22.7
	10	201.0	23.9
Cape Henry	2	143.2	21.0
	5	173.9	22.7
	10	203.9	24.8

The above constants are based on linear regression analyses of the frequency intensity-duration curves contained in the VDOT Drainage Manual. (Adapted from DCR Course "C" Training Notebook.)

*****For a more comprehensive list, see Appendix 5A***.

The rainfall constants, ***a*** and ***b***, were developed from linear regression analyses of the I-D-F curves and can be generated for any area where such curves are available. The limitations associated with the I-D-F curves, such as duration, return frequency, etc., will also limit development of the constants. **Table 5-5** provides rainfall constants for various regions in Virginia. Substituting **Equation 5-2** into the rational equation results in the following:

$$Q = C\left(\frac{a}{b+T_d}\right)A$$

Equation 5-3
Rearranged Rational Equation

where: Q = *peak rate of discharge, cfs*
a & b = *rainfall constants developed for storms of various recurrence intervals and various geographic locations, as shown in* ***Table 5-5***
T_d = *critical storm duration, min.*
C = *runoff coefficient*
A = *drainage area, acres*

Substituting this relationship for Q_i , **Equation 5-1** then becomes:

$$V = \left[\left[C \left(\frac{a}{b+T_d} \right) A \right] T_d + \frac{\left[C \left(\frac{a}{b+T_d} \right) A \right] t_c}{4} - \frac{q_o T_d}{2} - \frac{3 q_o t_c}{4} \right] 60$$

Equation 5-4
Substitute *Equation 5-3* into *Equation 5-1*

Maximum Storage Volume

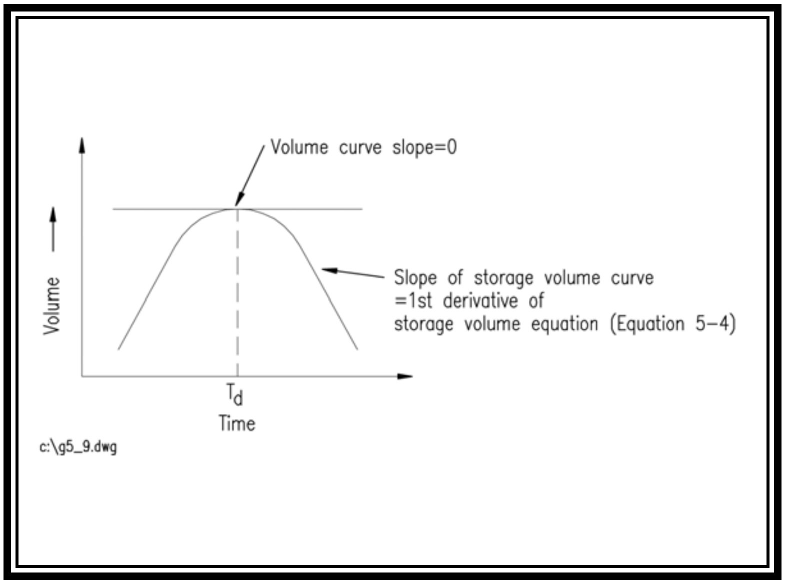
The first derivative of this storage volume equation, **Equation 5-4,** with respect to time is an equation that represents the slope of the storage volume curve plotted versus time. When this equation is set to equal zero, and solved for T_d, it represents the time, T_d, at which the slope of the storage volume curve is zero, or at a maximum, as shown in **Figure 5-9**. **Equation 5-5** represents the first derivative of the storage volume equation with respect to time and can be solved for T_d.

$$T_d = \sqrt{\frac{2CAa(b - t_c/4)}{q_o}} - b$$

Equation 5-5
Critical Storm Duration, T_d

where: T_d = *critical storm duration, min.*
C = *runoff coefficient*
A = *drainage area, acres*
a & b = *rainfall constants developed for storms of various recurrence intervals and various geographic locations, as shown in* ***Table 5-5***
t_c = *time of concentration, min.*
q_o = *allowable peak outflow, cfs*

FIGURE 5 - 9
Storage Volume vs. Time Curve



Equation 5-5 is solved for T_d. Then, T_d is substituted into **Equation 5-3** to solve for Q_i, and Q_i is substituted into **Equation 5-1** to solve for the required storage volume. Once the storage volume is known, the outlet structure and the stormwater facility can be sized. This method provides a direct solution to the graphical analysis of the family of hydrographs described in **Section 5-4.3** and is quicker to use. The following procedure illustrates this method using **Example 1**:

<u>Information Needed:</u>

The Modified Rational Method-Direct Solution is similar to the previous methods since it requires determination of the pre- and post-developed hydrology, as described in **Section 4-3.1**, resulting in a pre-developed peak rate of runoff (***allowable release rate***) and a post-developed ***runoff hydrograph***. **Table 5-3** provides a summary of the hydrology from **Example 1**. The rainfall constants ***a*** and ***b*** for the watershed are determined from **Table 5-5**.

Procedure:

1. Determine the 2-year critical storm duration by solving **Equation 5-5**:

$$T_{d_2} = \sqrt{\frac{2CA\,a\,(b - t_c/4)}{q_{o_2}}} - b$$

Where, from **Example 1**:

T_{d_2} = *2-year critical storm duration, min.*
C = *developed condition runoff coefficient = .59*
A = *drainage area = 25.0 acres*
t_c = *post-developed time of concentration = 21 min.*
q_{o_2} = *allowable peak outflow = 17 cfs (pre-developed peak rate of discharge)*
a_2 = *2-year rainfall constant = 130.3*
b_2 = *2-year rainfall constant = 18.5*

$$T_{d_2} = \sqrt{\frac{2(.59)(25.0)(130.3)(18.5 - 21/4)}{17}} - 18.5$$

$$= \sqrt{2995.9} - 18.5$$

$$T_{d_2} = 36.2 \text{ min.}$$

2. Solve for the 2-year critical storm duration intensity, I_2, using **Equation 5-2** and the 2-year critical storm duration T_{d_2}:

$$I_2 = \frac{a}{b + T_{d_2}}$$

where: T_{d_2} = *critical storm duration = 36.2 min.*
a = *2-year rainfall constant = 130.3*
b = *2-year rainfall constant = 18.5*

$$I_2 = \frac{130.3}{18.5 + 36.2} = 2.38 \text{ in./hr.}$$

3. Determine the 2-year peak inflow, Q_{i_2}, using the **Rational Equation** and the critical storm duration intensity I_2:

$$Q_{i_2} = CI_2A$$

where: Q_{i_2} = *2-year peak inflow, cfs*
C = *developed condition runoff coefficient = .59*
I_2 = *critical storm intensity = 2.38 in./hr.*
A = *drainage area = 25 acres*

$$Q_{i_2} = (0.59)(2.38)(25)$$

$$Q_{i_2} = 35.1 \text{ cfs}$$

4. Determine the 2-year required storage volume for the 2-year critical storm duration, T_{d_2}, using **Equation 5-1**:

$$V_2 = \left[Q_{i_2}T_{d_2} + \frac{Q_{i_2}t_c}{4} - \frac{q_{o_2}T_{d_2}}{2} - \frac{3q_{o_2}t_c}{4}\right] 60$$

where: V_2 = *2-year required storage, ft*^{3}
Q_{i_2} = *2-year peak inflow for critical storm = 35.1 cfs*
C = *developed runoff coefficient = .59*
A = *area = 25.0 acres*
T_{d_2} = *critical storm duration = 36.2 min.*
t_c = *developed condition time of concentration = 21 min.*
q_{o_2} = *2-year allowable peak outflow = 17 cfs*

$$V_2 = \left|(35.1)(36.2) + \left(\frac{(35.1)(21)}{4}\right) - \left(\frac{(17)(36.2)}{2}\right) - \frac{3(17)(21)}{4}\right| 60$$

$$V_2 = 52{,}764 \text{ ft}^3 = 1.21 \text{ ac.ft.}$$

Repeat Steps 2 through 4 for the 10-year storm, as follows:

5. Determine the 10-year critical storm duration $T_{d_{10}}$, using **Equation 5-5** as follows:

$$T_{d_{10}} = \sqrt{\frac{2(.59)(25.0)(189.2)(22.1-21/4)}{24}} - 22.1$$

$$T_{d_{10}} = \sqrt{3918.6} - 22.1$$

$$T_{d_{10}} = 40.5 \ min.$$

Where:
$T_{d_{10}}$ = *10-year critical storm duration, min.*
C = *developed condition runoff coefficient = .59*
A = *drainage area = 25 acres*
t_c = *post-developed time of concentration = 21 min.*
$q_{o_{10}}$ = *24 cfs*
a_{10} = *189.2*
b_{10} = *22.1*

6. Solve for the 10-year critical storm duration intensity, I_{10}, using **Equation 5-2**, and the 10-year critical storm duration, $T_{d_{10}}$.

$$I_{10} = \frac{a}{b+T_{d_{10}}}$$

$$I_{10} = \frac{189.2}{22.1 + 40.5} = 3.02 \ in./hr.$$

7. Determine the 10-year peak inflow, $Q_{i_{10}}$, using the **Rational Equation** and the critical storm duration intensity I_{10}:

$$Q_{i_{10}} = C \, I_{10} \, A$$

Where:
$Q_{i_{10}}$ = *10-year peak inflow*
C = *developed condition runoff coefficient = .59*
I_{10} = *critical storm intensity = 3.02 in./hr.*
A = *drainage area = 25.0 ac.*

$$Q_{i_{10}} = (.59)(3.02)(25.0)$$

$$Q_{i_{10}} = 44.5\ cfs$$

8. Determine the required 10-year storage volume for the 10-year critical storm duration, $T_{d_{10}}$, using **Equation 5-1**:

$$V_{10} = \left[Q_{i_{10}} T_{d_{10}} + \frac{Q_{i_{10}} t_c}{4} - \frac{q_{o_{10}} T_{d_{10}}}{2} - \frac{3 q_{o_{10}} t_c}{4} \right] 60$$

Where:
V_{10} = *required storage, ft*^{3}
$Q_{i_{10}}$ = *44.5 cfs*
C = *.59*
A = *25 ac.*
$T_{d_{10}}$ = *40.5 min.*
t_c = *21 min.*
$q_{o_{10}}$ = *24 cfs*

$$V_{10} = \left[(44.5)(40.5) + \frac{(44.5)(21)}{4} - \frac{(24)(40.5)}{2} - \frac{3(24)(21)}{4} \right] 60$$

$$V_{10} = 70{,}308\ ft^3 = 1.61\ ac.ft.$$

V_2 and V_{10} represent the total storage volume required for the 2-year and 10-year storms, respectively. **Table 5-6** provides a summary of the four different sizing procedures used in this chapter, as applied to **Example 1**. The engineer should choose one of these methods based on the complexity and size of the watershed and the chosen hydrologic method. Using the stage-storage curve, a multi-stage riser structure can then be designed to control the appropriate storms and, if required, the water quality volume.

TABLE 5 - 6
Summary of Results: Storage Volume Requirement Estimates, Example 1

Method	2-*yr*. Storage Required	10-*yr*. Storage Required
Graphical Hydrograph Analysis	*0.82 ac.ft.*	*1.84 ac.ft.*
TR-55 Shortcut Method	*1.05 ac.ft.*	*1.96 ac.ft.*
Modified Rational Method - Critical Storm Duration - Graphical Solution	*1.16 ac.ft.*	*1.56 ac.ft.*
Modified Rational Method - Critical Storm Duration - Direct Solution	*1.21 ac.ft.* T_d = *36.2 min.*	*1.61 ac.ft.* T_d = *40.5 min.*

5-5 STAGE-STORAGE CURVE

By using one of the above methods for determining the storage volume requirements, the engineer now has sufficient information to place and grade the proposed stormwater facility. Remember, **this is a preliminary sizing which needs to be refined during the actual design**. By trial and error, the approximate required volume can be achieved by designing the basin to fit the site geometry and topography. The storage volume can be computed by planimetering the contours and creating a ***stage-storage curve***.

5-5.1 Storage Volume Calculations

For retention/detention basins with vertical sides, such as tanks and vaults, the storage volume is simply the bottom surface area times the height. For basins with graded (2H:1V, 3H:1V, etc.) side slopes or an irregular shape, the stored volume can be computed by the following procedure. **Figure 5-10** represents the stage-storage computation worksheet completed for **Example 1**. A blank worksheet can be found at the end of this chapter (see **Figure 5-27**). (Note that other methods for computing basin volumes are available, such as the Conic Method for Reservoir Volumes, but they are not presented here.)

Procedure:

1. Planimeter or otherwise compute the area enclosed by each contour and enter the measured value into Columns 1 and 2 of **Figure 5-10**. The invert of the lowest control orifice represents zero storage. This will correspond to the bottom of the facility for extended-detention or detention facilities, or the permanent pool elevation for retention basins.

2. Convert the planimetered area (often in square inches) to units of square feet in Column 3 of **Figure 5-10**.

3. Calculate the average area between each contour.

   The average area between two contours is computed by adding the area planimetered for the first elevation, column 3, to the area planimetered for the second elevation, also Column 3, and then dividing their sum by 2. This average is then written in Column 4 of **Figure 5-10**.

   From **Figure 5-10**:

   Average area, elevation *81-82:* $\dfrac{0 + 1800}{2} = 900\ ft^2.$

   Average area, elevation *82-84*: $\dfrac{1800 + 3240}{2} = 2{,}520\ ft^2.$

   Average area, elevation *84-86:* $\dfrac{3240 + 5175}{2} = 4{,}207\ ft^2.$

   This procedure is repeated to calculate the <u>average</u> area found between any two consecutive contours.

4. Calculate the *volume* between each contour by multiplying the average area from step 3 (Column 4) by the contour interval and placing this product in Column 6. From **Figure 5-10**:

   Contour interval between *81* and $82 = 1\ ft.\ x\ 900\ ft^2 = 900\ ft^3$
   Contour interval between *82* and $84 = 2\ ft.\ x\ 2{,}520\ ft^2 = 5{,}040\ ft^3$

   This procedure is repeated for each measured contour interval.

FIGURE 5 - 10
Stage-Storage Computation Worksheet, Example 1

PROJECT: **EXAMPLE 1** SHEET ____ OF ____

COUNTY: ____________ COMPUTED BY: ____________ DATE: ________

DESCRIPTION: ____

ATTACH COPY OF TOPO: SCALE - 1" = **30** *ft.*

1	2	3	4	5	6	7	8
ELEV.	AREA *(in²)*	AREA *(ft²)*	AVG. AREA *(ft²)*	INTERVAL	VOL. *(ft³)*	TOTAL VOLUME *(ft³)*	TOTAL VOLUME *(ac.ft.)*
81	0	0				0	0
			900	1	900		
82	2.0	1800				900	.02
			2520	2	5040		
84	3.6	3240				5940	.14
			4207	2	8414		
86	5.75	5175				14354	.33
			7614	2	15228		
88	11.17	10053				29582	.68
			12991	2	25982		
90	17.7	15930				55564	1.28
			20700	2	41400		
92	28.3	25470				96964	2.23
			31102	1	31102		
93	40.8	36734				128066	2.94
			38105	1	38105		
94	43.9	39476				166171	3.81

5. Sum the volume for each contour interval in Column 7. Using **Figure 5-10,** this is simply the sum of the volumes computed in the previous step:

   Contour *81*, $Volume = 0$

   Contour $82, Volume = 0 + 900 = 900\ ft^3$

   Contour $84, Volume = 900 + 5{,}040 = 5{,}940\ ft^3$

   Contour *86*, $Volume = 5{,}940 + 8{,}414 = 14{,}354\ ft^3$

   Column 8 allows for the volume to be tabulated in units of acre-feet: $ft^3 \div 43{,}560\ ft^2/ac.$

   This procedure is then repeated for each measured contour interval.

6. Plot the stage-storage curve with *stage* on the y-axis versus *storage* on the x-axis. **Figure 5-11** represents the stage-storage curve for **Example 1** in units of feet (stage) versus acre-feet (storage).

The stage-storage curve allows the designer to estimate the *design high water elevation* for each of the design storms if the required storage volume has been determined. This allows for a preliminary design of the riser orifice sizes and their configuration.

5-6 WATER QUALITY AND CHANNEL EROSION CONTROL VOLUME CALCULATIONS

Virginia's Stormwater Management Regulations require that the first flush of runoff, or the water quality volume, be treated to enhance water quality. The ***water quality volume*** (V_{wq}) is the first 0.5 inches of runoff from the impervious area of development. The water quality volume must be treated
using one or a combination of BMP's depending on the total size of the contributing watershed, amount of impervious area, and site conditions. (Refer to **Chapters 2** and **3** for **BMP Selection Criteria** and **BMP Minimum Standards and Specifications**, respectively.)

The water quality volume is calculated as follows:

$$V_{wq}\ (ft^3) = Impervious\ area\ (ft^2)\ x\ (\tfrac{1}{2}\ in.)\ /\ (12\ in./ft.)$$

$$V_{wq}\ (ac.ft.)\ = V_{wq}\ (ft^3)\ /\ 43{,}560\ ft^2/ac.$$

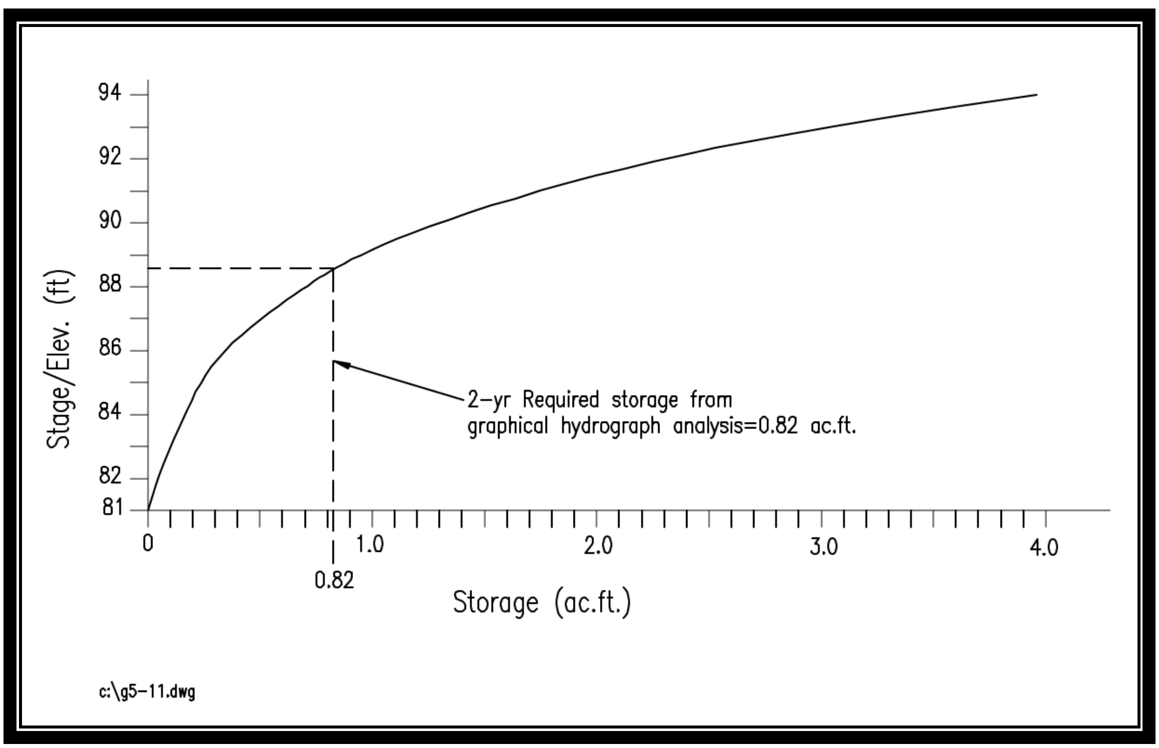
The water quality volume for a wet BMP may be dependent on the specific design criteria for that BMP based on the watershed's imperviousness or the desired pollutant removal efficiency (using performance-based or technology-based criteria, respectively). The design criteria for each of the

BMPs, including extended-detention and retention basins, infiltration devices, constructed wetlands, marshes, etc., are presented in **Chapter 3**. This discussion is focused on the calculations associated with the control of the water quality volume in extended-detention and retention basins.

Virginia's Stormwater Management Regulations allow for the control of downstream channel erosion by detaining a specified volume of runoff for a period of time. Specifically, 24-hour extended detention of the runoff from the 1-year frequency storm is proposed as an alternate criteria to the 2-year peak rate reduction specified in MS-19 of the Virginia Erosion and Sediment Control Regulations, and the channel erosion component of the Virginia Stormwater Management Regulations. The channel erosion control volume (V_{ce}) is calculated by first determining the depth of runoff (in inches) based on the fraction of rainfall to runoff (runoff curve number) and then multiplying the runoff depth by the drainage area to be controlled. This procedure will be discussed in **5-6.3**.

FIGURE 5 - 11

Stage vs. Storage Curve, Example 1



5-6.1 Retention Basins - Water Quality Volume

The permanent pool feature of a retention basin allows for settling of particulate pollutants, such as sediment and other pollutants that attach adsorb to these particulates. Therefore, it is essential that the volume of the pool be both large enough and properly configured to prevent ***short-circuiting***. (Short-circuiting results when runoff enters the pool and exits without sufficient time for the settling process to occur.)

The permanent pool, or "dead" storage volume, of a retention facility is a function of the water quality volume. For example, a permanent pool sized to contain four times the water quality volume provides greater pollutant removal capacity than a permanent pool sized to contain two times the water quality volume. **Chapter 3** provides the pollutant removal efficiencies for various permanent pool sizes and criteria for permanent pool geometry.

Example 1 analyzes a 25-acre watershed. The water quality volume and permanent pool volume calculations for a retention basin serving this watershed are as follows:

Procedure

1. Calculate the water quality volume, V_{wq} , for the given watershed.

   From **Example 1**, the commercial/industrial development disturbs *11.9* acres, with *9.28* acres (*404,236 ft*^{2}*.*) of impervious cover after development.

$$V_{wq} = 404{,}236\ ft^2 \times \tfrac{1}{2}\ in. / 12\ in./ft.$$
$$= 16{,}843\ ft^3$$
$$= 16{,}843\ ft^3/43{,}560\ ft^2/ac.$$
$$V_{wq} = 0.38\ ac.ft.$$

2. Size the permanent pool based on the desired *pollutant removal efficiency* or the drainage area *impervious cover*.

   The pool volume will be sized based upon the desired pollutant removal efficiency. Referring to **Table 3.06-1**, the permanent pool must be sized for 4 x V_{wq} for a pollutant removal efficiency of 65%.

$$\text{Permanent Pool Volume} = V_{wq} \times 4.0$$

$$\text{Permanent Pool Volume} = 0.38\ ac.ft. \times 4.0 = 1.52\ ac.ft.$$

5-6.2 Extended-Detention Basins - Water Quality Volume and Orifice Design

A ***water quality extended-detention basin*** treats the water quality volume by detaining it and releasing it over a specified amount of time. In theory, extended-detention of the water quality volume will allow the particulate pollutants to settle out of the *first flush* of runoff, functioning similarly to a permanent pool. Virginia's Stormwater Management Regulations pertaining to water quality specify a 30-hour draw down time for the water quality volume. This is a *brim draw down* time, beginning at the time of peak storage of the water quality volume. Brim-draw down time means the time required for the entire calculated volume to drain out of the basin. This assumes that the brim volume is present in the basin prior to any discharge. In reality, however, water is flowing out of the basin prior to the full or brim volume being reached. Therefore, the extended detention orifice can be sized using either of the following methods:

1. Use the *maximum hydraulic head* associated with the water quality volume (V_{wq}) and calculate the orifice size needed to achieve the required draw down time, and route the the water quality volume through the basin to verify the actual storage volume used and the drawdown time.

2. Approximate the orifice size using the *average hydraulic head* associated with the water quality volume (V_{wq}) and the required draw down time.

The two methods for calculating the required size of the extended detention orifice allow for a quick and conservative design (Method 2 above) and a similarly quick estimation with a routing to verify the performance of the design (Method 1).

Method 1, which uses the *maximum hydraulic head* and maximum discharge in the calculation, results in a slightly larger orifice than the same procedure using the *average hydraulic head* (Method 2). The routing allows the designer to verify the performance of the calculated orifice size. As a result of the routing effect however, the actual basin storage volume used to achieve the draw down time will be less than the computed brim draw down volume. It should be noted that the routing of the extended detention of the runoff from storms larger than the water quality storm (such as the 1-year frequency storm for channel erosion control) will result in proportionately larger reduction in the actual storage volume needed to achieve the required extended detention. (Refer to **Section 5-6.3** for the extended detention design procedures for channel erosion protection.)

The procedure used to size an extended detention orifice includes the first steps of the design of a multi-stage riser for a basin controlling water quality and/or channel erosion, and peak discharge. These steps are repeated for sizing the 2-year and 10-year release openings. Other design storms may be used as required by ordinance or downstream conditions.

Method 1: Water quality orifice design using maximum hydraulic head and routing of the water quality volume.

A water quality extended-detention basin sized for two times the water quality volume will be used here to illustrate the sizing procedure for an extended-detention orifice.

Procedure:

1. Calculate the water quality volume, V_{wq}, required for treatment.

   From **Example 1**:

   V_{wq} = 404,236 ft^2 x ½ in/ 12 in/ft = 16,843 ft^3

   V_{wq} = 16,843 ft^3/ 43,560 ft^2/ac = 0.38 ac.ft.

   For extended-detention basins, 2 x V_{wq} = 2(0.38 ac.ft.) = 0.76 ac.ft.= 33,106 ft^3.

2. Determine the maximum hydraulic head, h_{max}, corresponding to the required water quality volume.

   From the **Example 1** stage vs. storage curve (**Figure 5-11**):

   0.76 *ac.ft.* occurs at elevation 88 *ft*. (approximate). Therefore, $h_{max} = 88 - 81 = 7.0$ *ft.*

3. Determine the maximum discharge, Q_{max}, resulting from the 30-hour drawdown requirement.

   The *maximum discharge* is calculated by dividing the required volume, in ft^3, by the required time, in seconds, to find the average discharge, and then multiplying by 2, to determine the maximum discharge.

   From **Example 1**:

$$Q_{avg} = \frac{33{,}106\ ft^3}{(30\,hr.)(3{,}600\,sec./hr.)} = 0.30\ \ cfs$$

$$Q_{max} = 2 \text{ x } 0.30\ cfs = 0.60\ cfs$$

4. Determine the required orifice diameter by rearranging the **Orifice Equation, Equation 5-6** to solve for the orifice area, in ft^2, and then diameter, in *ft.*

   Insert the values for Q_{max} and h_{max} into the **Rearranged Orifice Equation, Equation 5-7** to solve for the orifice area, and then solve for the orifice diameter.

$$Q = Ca\sqrt{2gh}$$

Equation 5-6
Orifice Equation

$$a = \frac{Q}{C\sqrt{2gh}}$$

Equation 5-7
Rearranged Orifice Equation

where: Q = *discharge, cfs*
C = *dimensionless coefficient = 0.6*
a = *area of the orifice, ft*^{2}
g = *gravitational acceleration, 32.2 ft/sec*^{2}
h = *head, ft.*

From **Example 1**:

For orifice area:

$$a = \frac{0.6}{0.6\sqrt{(2)(32.2)(7.0)}}$$

For orifice diameter:

$$a = 0.047\ ft^2 = \pi r^2 = \pi d^2/4$$

$$d = \sqrt{\frac{4a}{\pi}} = \sqrt{\frac{4(0.047\ ft^2)}{\pi}}$$

d = orifice diameter = 0.245 ft = 2.94"

Use a 3-inch diameter water quality orifice.

Routing the water quality volume (V_{wq}) of 0.76 *ac.ft.,* occurring at elevation 88 feet, through a 3-inch water quality orifice will allow the designer to verify the draw down time, as well as the maximum elevation of 88 feet.

<u>Route the water quality volume</u>.

This calculation will give the engineer the *inflow-storage-outflow relationship* in order to verify the actual storage volume needed for the extended detention of the water quality volume. The routing procedure takes into account the discharge that occurs before maximum or *brim* storage of the water quality volume, as opposed to the brim drawdown described in Method 2. The routing procedure is simply a more accurate analysis of the storage volume used while water is flowing into and out of the basin. Therefore, the actual volume of the basin used will be less than the volume as defined by the regulation. This procedure will come in handy if the site to be developed is tight and the area needed for the stormwater basin must be “squeezed” as much as possible.

The routing effect of water entering and discharging from the basin simultaneously will also result in the actual drawdown time being less than the calculated 30 hours. Judgement should be used to determine whether the orifice size should be reduced to achieve the required 30 hours or if the actual time achieved will provide adequate pollutant removal.

NOTE: The designer will notice a significant reduction in the actual storage volume used when routing the extended detention of the runoff from the 1-year frequency storm (channel erosion control). Please refer to **Chapter 5-6.3** and **Chapter 5-11** for the appropriate design procedures when extended detention is provided for channel erosion control.

Routing the water quality volume depends on the ability to work backwards from the design runoff volume of 0.5 inches to find the rainfall amount. Using SCS methods, the rainfall needed to generate 0.5 inches of runoff from an impervious surface (*RCN=98*) is 0.7 inches. The SCS design storm is the Type II, 24-hour storm. Therefore, the *water quality storm* using SCS methods is defined as the SCS Type II, 24-hour storm, with a rainfall depth = 0.7 inches.

The rational method does not provide a design storm derived from a specified rainfall depth. Its rainfall depth depends on the storm duration (watershed t_c) and the storm return frequency. **Since the water quality storm varies with runoff amount, not the design storm return frequency, an input runoff hydrograph representing the water quality volume cannot be generated using rational method parameters**. Therefore Method 1, routing of the water quality volume, must use SCS methods. See **Chapter 4** for details on SCS methods.

Continuing with **Example 1**, the procedure is as follows:

Procedure (contd.)**:**

5. Calculate a stage-discharge relationship using the **Orifice Equation, Equation 5-6** and the orifice size determined in Step 4.

From **Example 1**, using the 3-inch diameter orifice, the calculation is as follows:

$$Q = Ca\sqrt{2gh}$$

Orifice Equation 5-6

$$Q = 0.6(.047)\sqrt{(2)(32.2)(h)}$$

$$Q = 0.22\sqrt{h}$$

where: h = *water surface elevation minus the orifice's centerline elevation*, in ft.*

Note: If the orifice size is small relative to the anticipated head, h, values of h may be defined as the water surface elevation minus the invert of the orifice elevation.

7. Complete a stage-discharge table for the range of elevations in the basin, as shown in **Table 5-7**:

TABLE 5 - 7
Stage-Discharge Table: Water Quality Orifice Design

Elevation	*h* (*ft*)	*Q* (*cfs*)
81	0	0
82	1	0.2
83	2	0.3
84	3	0.4
85	4	0.4
86	5	0.5
87	6	0.5
88	7	0.6

8. Determine the time of concentration as defined in **Chapter 4** for the impervious area.

From **Example 1**, the developed time of concentration, $t_c = 0.46$ *hours*. The impervious area time of concentration, $t_{c_{imp}}$ = 0.09 hours, or 5.4 minutes.

9. Using $t_{c_{imp}}$, the stage-discharge relationship, the stage-storage relationship, and the impervious acreage (*RCN = 98*), route the water quality storm through the basin. The water quality storm for this calculation is the SCS Type 2, 24-hour storm, rainfall depth = 0.7 inches. (Note that the rainfall depth is established as the amount of rainfall required to generate 0.5 inches of runoff from the impervious area.)

The water quality volume may be routed using a variety of computer programs such as TR-20, HEC-1, or other storage indication routing programs. Alternatively, it can be routed by hand using the storage indication routing procedure outlined in **Section 5-9** of this chapter.

10. Evaluate the discharge hydrograph to verify that the drawdown time from maximum storage to zero discharge is at least 30 hours. (Note that the maximum storage corresponds to the maximum rate of discharge on the discharge hydrograph.)

    The routing of the water quality volume using TR-20 results in a maximum storage elevation is 85.69 ft. versus the approximated 88.0 ft. The brim drawdown time is 17.5 hours (peak discharge occurs at 12.5 hours and .01 discharge occurs at 30 hours). For this example, the orifice size may be reduced to provide a more reasonable drawdown time and another routing performed to find the new water quality volume elevation.

METHOD 2: Water quality orifice design using *average hydraulic head* and *average discharge*.

The procedure described in Method 2 is presented in the next section. For the previous example, Method 2 results in a 2.5 inch orifice (versus a 3.0 inch orifice), and the design extended detention water surface elevation is set at 88 ft.(versus 85.69ft.). (It should be noted that trial two of Method 1 as noted above may result in a design water surface elevation closer to 88 ft.) If the basin is to control additional storms, such as the 2-year and/or 10-year storms, the additional storage volume would be "stacked" just above the water quality volume. The invert for the 2-year control, for example, would be set at 88.1 *ft.*

5-6.3 Extended-Detention Basins - Channel Erosion Control Volume and Orifice Design

Extended detention of a specified volume of stormwater runoff can also be incorporated into a basin design to protect downstream channels from erosion. Virginia's Stormwater Management Regulations recommend 24-hour extended detention of the runoff from the 1-year frequency storm as an alternative to the 2-year peak rate reduction required by MS-19 of the Virginia Erosion and Sediment Control Regulations. A full discussion of this channel erosion criteria will be presented in a future Technical Bulletin, along with practical guidance from DCR on the effective implementation of the criteria. The discussion presented here is for the design of a channel erosion control extended-detention orifice.

The design of a channel erosion control extended-detention orifice is similar to the design of the water quality orifice in that two methods can be employed:

1. Use the *maximum hydraulic head* associated with the specified channel erosion control (V_{ce}) storage volume and calculate the orifice size needed to achieve the required draw down time and route the 1-year storm through the basin to verify the storage volume and the draw down time, or

2. Approximate the orifice size using the *average hydraulic head* associated with the channel erosion control volume (V_{ce}) and draw down time.

The routing procedure takes into account the discharge that occurs before maximum or *brim* storage of the channel erosion control volume (V_{ce}). The routing procedure simply provides a more accurate accounting of the storage volume used while water is flowing into and out of the basin, and results in less storage volume being used than the calculated brim storage volume associated with the maximum hydraulic head. The actual storage volume needed for extended detention of the runoff generated by the 1-year frequency storm will be approximately 60 percent of the calculated volume (V_{ce}) of runoff for curve numbers between 75 and 95 and time of concentration between 0.1 and 1 hour.

The following procedure illustrates the design of the extended-detention orifice for channel erosion control. Refer to **Chapter 6** for **Example 6.2** which includes the design of an extended-detention orifice for channel erosion control, Method 1, within the design of a multi-stage riser.

Method 2:

Procedure

1. Calculate the channel erosion control volume, V_{ce}.

   Determine the rainfall amount (inches) of the 1-year frequency storm for the local area where the project is located (**Appendix 4B**). With the rainfall amount and the runoff curve number (RCN), determine the corresponding runoff depth using the runoff Equation (**Chapter 4: Hydrologic Methods - SCS** TR-55) or the Rainfall - Runoff Depth Charts (**Appendix 4C**).

   From **Example 2**:

   1-year rainfall = 2.7 inches, RCN = 75; using **Appendix 4C**, the 1-year frequency depth of runoff = 0.8 inches, therefore:

   $$V_{ce} = 25\ ac. \times 0.8\ in. \times 1'/12'' = 1.66\ ac.ft.$$

   To account for the routing effect, reduce the channel erosion control volume:

   $$V_{ce} = (0.6)(1.66\ ac.ft.) = 1.0\ ac.ft. = 43{,}560\ ft.^3$$

2. Determine the *average hydraulic head*, h_{avg}, corresponding to the required channel erosion control volume.

   From **Example 2** - Stage - Storage Curve: 1.0 *ac.ft.* occurs at elevation 89.0 *ft.* Therefore,

   $$h_{avg} = (89 - 81) / 2 = 4.0\ ft.$$

3. Determine the *average discharge*, Q_{avg}, resulting from the 24-hour draw down requirement.

The average discharge is calculated by dividing the required volume, in ft^3, by the required time, in seconds, to find the average discharge.

From **Example 2**:

$$Q_{avg} = \frac{43{,}560\ ft^3}{(24\,hr.)\,(3{,}600\,sec./hr.)} = 0.5\ cfs$$

4. Determine the required orifice diameter by rearranging the **Orifice Equation, Equation 5-6** to solve for the orifice area, in ft^2, and then diameter, in *ft.*

Insert the values for Q_{avg} and h_{avg} into the **Rearranged Orifice Equation, Equation 5-7** to solve for the orifice area, and then solve for the orifice diameter.

$$Q = Ca\sqrt{2gh}$$

Equation 5-6
Orifice Equation

$$a = \frac{Q}{C\sqrt{2gh}}$$

Equation 5-7
Rearranged Orifice Equation

where: Q = *discharge, cfs*
C = *dimensionless coefficient = 0.6*
a = *area of the orifice,* ft^2
g = *gravitational acceleration, 32.2* ft/sec^2
h = *head, ft.*

From **Example 2**:

For orifice area:

$$a = \frac{0.5}{0.6\sqrt{(2)\,(32.2)\,(4.0)}}$$

$$a = 0.052\ ft^2 = \pi r^2 = \pi d^2/4$$

For orifice diameter:

$$d = \sqrt{\frac{4a}{\pi}} = \sqrt{\frac{4(0.052\ ft^2)}{\pi}}$$

d = orifice diameter = 0.257 ft = 3.09 inches

Use 3.0-inch diameter channel erosion extended detention orifice

The use of Method 1, utilizing the maximum hydraulic head and a routing of the 1-year storm is illustrated in **Chapter 6: Example 6.2**. Methd 1 results in a 3.7" diameter orifice and a routed water surface elevation of 88.69 ft. Additional storms to may be "stacked" just above this volume if additional controls are desired.

5-7 MULTI-STAGE RISER DESIGN

A principal spillway system that controls the rate of discharge from a stormwater facility will often use a multi-stage riser for the drop inlet structure.

A multi-stage riser is a structure that incorporates separate openings or devices at different elevations to control the rate of discharge from a stormwater basin during multiple design storms. Permanent multi-stage risers are typically constructed of concrete to help increase their life expectancy; they can be precast or cast-in-place. The geometry of risers will vary from basin to basin. The engineer can be creative to provide the most economical and hydraulically efficient riser design possible. **Figure 3-02.1** in **Chapter 3** provides some examples of multi-stage riser structures.

In a stormwater management basin design, the multi-stage riser is of utmost importance since it controls the design water surface elevations. In designing the multi-stage riser, many iterative routings are usually required to arrive at a minimum structure size and storage volume that provides proper control. Each iterative routing requires that the facility's size (*stage-storage* curve) and outlet shape (*stage-discharge table* or *rating curve*) be designed and tested for performance. Prior to final design, it is helpful to approximate the required storage volume and outlet shape using one of the "shortcut" methods, as described in **Section 5-4**. In doing this, the number of iterations may be reduced. The following procedures outline methods for approximating and then completing the design of a riser structure. (These design procedures are illustrated in the examples found in **Chapter 6**.)

<u>Information needed</u>:

1. The hydrology for the watershed or drainage area to be controlled, calculated by using one of the methods outlined in **Chapter 4**, and

2. The allowable release rates for the facility, as established by ordinance or downstream conditions.

The design procedure provided here will incorporate the traditional 2-year and 10-year design storms and the pre-developed hydrology will establish the allowable discharge rates of the developed watershed. It should be noted that any design storm, 1-year, 5-year, etc., can be substituted into this design procedure, as required.

Procedure:

<u>STEP 1</u> Determine Water Quality or Extended Detention Requirements

Calculate the water quality volume and decide what method (extended-detention or retention) will be used to treat it, and/or calculate the channel erosion control volume for extended-detention, if required. (Virginia's Stormwater Management Regulations state that the **water quality volume is equal to the first 0.5 inch of runoff multiplied by the total impervious area of the land development project, and that the channel erosion control volume for extended detention is the runoff generated by the site during the 1-year frequency storm**.)

a. **Water Quality Extended-Detention Basin**: The water quality volume must be detained and released over 30-hours. The established pollutant removal efficiency is based on a 30-hour drawdown.

b. **Water Quality Retention Basin**: The volume of the permanent pool is established by the site impervious cover or the desired pollutant removal efficiency.

c. **Channel Erosion Control Extended-Detention Basin**: The channel erosion control volume must be detained and released over 24 hours.

Refer to **Chapter 3** for minimum BMP design standards and details.

<u>STEP 2</u> Compute Allowable Release Rates

Compute the pre- and post-developed hydrology for the watershed. Sometimes, the pre-developed hydrology will establish the allowable release rate from the basin. Other times, the release rate will be established by downstream conditions. In either case, the post-developed hydrology will provide the peak inflow into the basin, as a peak rate (*cfs*) or a runoff hydrograph. Refer to **Section 5-3, Allowable Release Rates**.

<u>STEP 3</u> Estimate the Required Storage Volume

Estimate the storage volume required using one of the "shortcut" volume estimate methods described in **Section 5-4**. The information required includes the developed condition peak rate of runoff, or runoff hydrograph, and the allowable release rates for each of the appropriate design storms.

STEP 4 **Grade the Basin; Create Stage-Storage Curve**

After considering the site geometry and topography, select a location for the proposed stormwater management basin. By trial and error, size the basin such that it will hold the approximate required storage volume. Ensure that the storage volume is measured from the lowest stage outlet. (Note: the storage volume can be computed by planimetering the contours and creating a stage-storage relationship as described in **Section 5-5**.) Remember that this is a preliminary sizing which needs to be fine-tuned during the final design.

STEP 5a **Design Water Quality Orifice (Extended-Detention)**

The procedure for sizing the water quality orifice for an extended-detention basin is covered in **Section 5-6.2** of this chapter. Using either Method 1 or Method 2, the designer establishes the size of the water quality or stream channel erosion control orifice and the design maximum water surface elevation.

The lowest stage outlet of an extended-detention basin is the invert of the extended-detention (or water quality) orifice, which corresponds to zero storage. **Section 5-6.2** provides a detailed discussion for sizing the water quality orifice and **Chapter 6** gives examples of the calculation procedure.

STEP 5b **Set Permanent Pool Volume (Retention)**

In a retention pond, the permanent pool volume, from **STEP 1**, establishes the lowest stage outlet for the riser structure (not including a pond drain, if provided). The permanent pool elevation, therefore, corresponds to "0" storage for the design of the "dry" storage volume stacked on top of the permanent pool.

STEP 6 **Size 2-Year Control Orifice**

(The 2-year storm is used here to show the design procedure. Other design storms or release requirements can be substituted into the procedure.)

Knowing the 2-year storm storage requirement, from design **STEP 3**, and the water quality volume, from design **STEP 1**, the engineer can do a preliminary design for the 2-year release opening in the multi-stage riser. To complete the design, some iterations may be required to meet the allowable release rate performance criteria. This procedure is very similar to the water quality orifice sizing calculations:

1. Approximate the 2-year maximum head, $h_{2_{max}}$.

Establish the approximate elevation of the 2-year maximum water surface elevation using the stage-storage curve and the preliminary sizing calculations. Subtract the water quality volume elevation from the approximate 2-year maximum water surface elevation to find the 2-year maximum head, $h_{2_{max}}$. If there are no water quality requirements, use the elevation of the basin bottom or invert.

2. Determine the maximum allowable 2-year discharge rate, $Q_{2_{allowable}}$, from **STEP 2**.

3. Calculate the size of the 2-year control release orifice using the **Rearranged Orifice Equation, Equation 5-7** and solve for the area, *a*, in ft^2.

The engineer may choose to use any one of a variety of orifice shapes or geometries. Regardless of the selection, the orifice will initially act as a weir until the top of the orifice is submerged. Therefore, the discharges for the first stages of flow are calculated using the weir equation:

$$Q_w = C_w L h^{1.5}$$

Equation 5-8
Weir Equation

where:
Q_w = *weir flow discharge, cfs*
C_w = *dimensionless weir flow coefficient, typically equal to 3.1 for sharp crested weirs. Refer to **Table 5-8**.*
L = *length of weir crest, ft.*
h = *head, ft., measured from the water surface elevation to the crest of the weir*

Flow through the rectangular opening will transition from *weir flow* to *orifice flow* once the water surface has risen above the top of the opening. This orifice flow is expressed by the orifice equation. The area, *a*, of a rectangular orifice is written as $a = L \times H$,

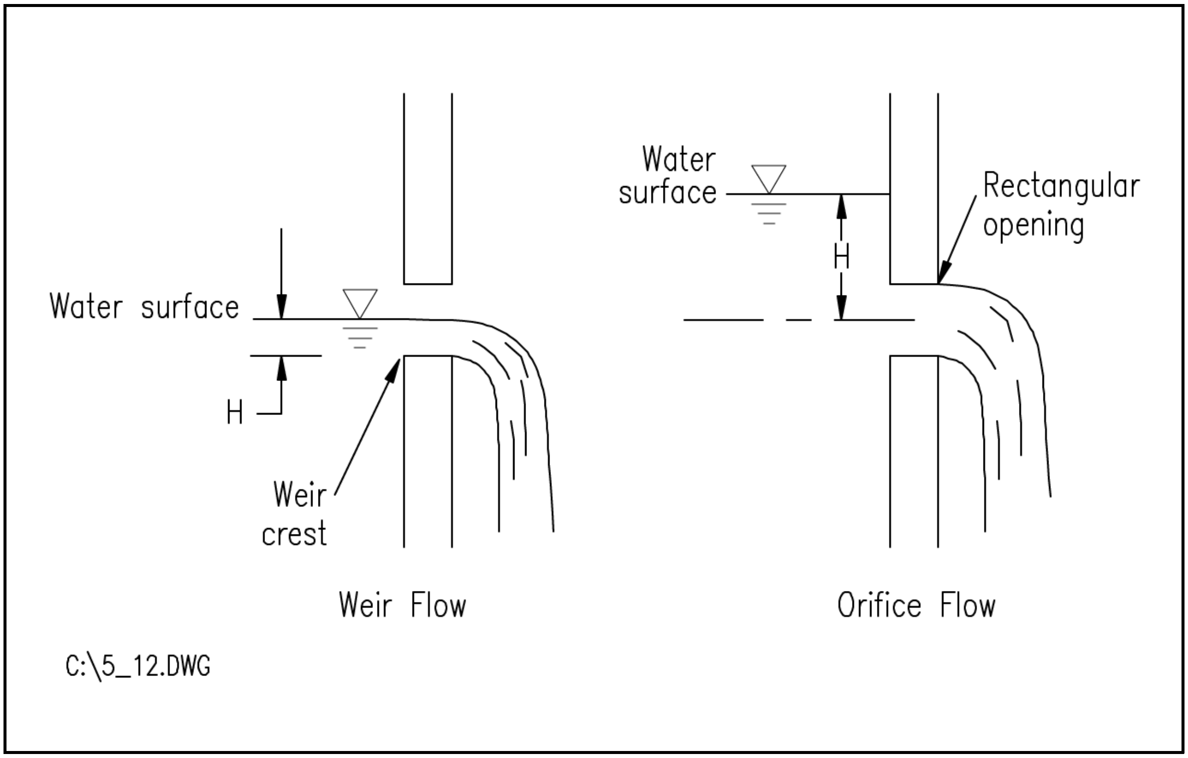
where:
L = *length of opening, ft.*
H = *height of opening, ft.*

Figure 5-12 shows a rectangular orifice acting as a weir at the lower stages and as an orifice after the water surface rises to height *H*, the height of the opening.

4. Develop the stage-storage-discharge relationship for the 2-year storm.

Calculate the discharge using the orifice equation and, if a rectangular opening is used, the weir equation as needed for each elevation specified on the stage-storage curve. Record the discharge on a Stage-Storage-Discharge Worksheet. **Figure 5-13** shows a completed Stage-Storage-Discharge Worksheet for **Example 2**. A blank worksheet is provided in **Appendix 5D**.

FIGURE 5 - 12
Weir and Orifice Flow



<u>STEP 7</u> **Check Performance of 2-Year Opening**

(Note: This step may not be necessary if the design is to be completed using one of the shortcut routing procedures where the water surface elevations are established by the required storage volume and not by an actual routing.)

1. Check the performance of the 2-year control opening by a) *reservoir routing the 2-year storm through the basin using an acceptable reservoir routing computer program* or by b) *doing the long hand calculations outlined in **Section 5-9** of this chapter*. **Verify that the 2-year release rate is less than or equal to the allowable release rate**. If not, reduce the size of the opening or provide additional storage and repeat **STEP 6**.

This procedure presents just one of many riser configurations. The engineer may choose to use any type of opening geometry for controlling the design storms and, with experience, may come to recognize the most efficient way to configure the riser. Note that if a weir is chosen for the 2-year storm control, the procedures outlined here for the 10-year storm may be used by substituting with the appropriate values for the 2-year storm. Refer to **Figure 3-02.1** for several different riser shapes.

STEP 8 **Size 10-Year Control Opening**

The design of the 10-year storm control opening is similar to the procedure used in sizing the 2-year control opening:

1. From the routing results, identify the exact 2-year water surface elevation.

2. Set the invert of the 10-year control just above the 2-year design water surface elevation and determine the corresponding storage volume from the stage-storage curve. Add this elevation, storage, and 2-year discharge to the stage-storage-discharge worksheet, **Figure 5-13**.

> ***The 10-year control invert may be set at a small distance, such as 0.1 feet minimum, above the 2-year maximum water surface elevation. If the 2-year orifice is also to be used for the 10-year control, the head is measured from the maximum water surface elevation to the centerline of the 2-year orifice. See Figure 5-14.***

3. Establish the approximate 10-year maximum water surface elevation using the stage-storage curve and the preliminary sizing calculations. Subtract the invert elevation of the 10-year control (from Step 2 above) from the approximate 10-year maximum water surface elevation to find the 10-year maximum head, $h_{10_{max}}$.

4. Determine the maximum allowable 10-year discharge rate, $Q_{10_{allowable}}$, from **STEP 2**.

5. Calculate the required size of the 10-year release opening. The engineer may choose between a circular and rectangular orifice, or a weir. If a weir is chosen, the weir flow equation can be rearranged to solve for L as follows.

$$Q_W = C_W L\, h^{1.5}$$

Equation 5-8
Weir Equation

$$L = Q_{10_{allowable}} / C_W\, h^{1.5}$$

Equation 5-9
Rearranged Weir Equation

Where:
L = *length of weir required, ft.*
C_W = *dimensionless weir flow coefficient, see* ***Table 5-8***
$Q_{10_{allowable}}$ = *10-year allowable riser weir discharge, cfs*
h = *hydraulic head; water surface elevation minus the weir crest elevation*

6. Develop the stage-storage-discharge relationship for the 10-year storm. Calculate the discharge for each elevation specified on the stage-storage curve, and record the discharge on a Stage-Storage-Discharge Worksheet, as shown in **Figure 5-13**.

Any weir length lost to the trash rack or debris catcher must be accounted for. See Chapter 3 for Trash Rack Specifications and example riser configurations.

TABLE 5 - 8
Weir Flow Coefficients

WEIR FLOW COEFFICIENTS, C			
Measured head, *h*, (*ft.*)	**Breadth of weir crest (*ft.*)**		
	0.50	**0.75**	**1.00**
0.2	2.80	2.75	2.69
0.4	2.92	2.80	2.72
0.6	3.08	2.89	2.75
0.8	3.30	3.04	2.85
1.0	3.32	3.14	2.98
1.2	3.32	3.20	3.08
1.4	3.32	3.26	3.20
1.6	3.32	3.29	3.28
1.8	3.32	3.32	3.31
2.0	3.32	3.32	3.30
3.0	3.32	3.32	3.32
4.0	3.32	3.32	3.32
5.0	3.32	3.32	3.32

Source: Kings Handbook of Hydraulics

<u>STEP 9</u> Check Performance of 10-Year Opening

(Note: This step may not be necessary if the design is to be completed using one of the short-cut routing procedures where the water surface elevations are established by the required storage volume and not by an actual routing.)

Check the performance of the 10-year control opening by a) *reservoir routing the 2-year and 10-year storms through the basin using an acceptable reservoir routing computer program* (see **Appendix**) or by b) *doing the long hand calculations outlined in **Section 5-9***. **Verify that the 10-year release rate is less than or equal to the allowable release rate**. If not, reduce the size of the opening and/or provide additional storage and repeat **STEP 8**.

<u>STEP 10</u> Perform Hydraulic Analysis

At this point, several iterations may be required to calibrate and optimize the hydraulics of the riser and the riser and barrel system. Drop inlet spillways should be designed so that full flow is established in the outlet conduit and riser at the lowest head over the riser crest as is practical. Also, the structure should operate without excessive surging, noise, vibration, or vortex action at any stage. This requires the riser to have a larger cross-sectional area than the outlet conduit.

As the water passes over the rim of the riser, the riser acts as a weir (**Figure 5-15a**); this discharge is described as ***riser weir flow control***. However, when the water surface reaches a certain height over the rim of the riser, the riser will begin to act as a submerged orifice (**Figure 5-15b**); such discharge is called ***riser orifice flow control***. The engineer must compute the elevation at which this transition from riser weir flow control to riser orifice flow control takes place. (This transition usually occurs during high hydraulic head conditions, such as between the 10-*yr.* and 100-*yr.* design high water elevations.)

Note in **Figure 5-15a & b** that the riser crest controls the flow, not the barrel. Thus, either condition can be described as ***riser flow control***. **Figure 5-15c & d** illustrates ***barrel flow control***. Barrel flow control occurs when the barrel controls the flow at the upstream entrance to the barrel (*barrel inlet flow control*, **Figure 5-15c**), or along the barrel length (*barrel pipe flow control*, **Figure 5-15d**).

Barrel flow control conditions illustrated in **Figure 5-15c & d** are desirable because they reduce or even eliminate cavitation forces, or surging and vibration (as described above), in the riser and barrel system. Cavitation forces in the riser and barrel system can greatly reduce the design flow capacity of the system. Cavitation forces may also cause vibrations that can damage the riser (especially corrugated metal risers) and the connection between the riser and barrel. This connection may crack and lose its watertight seal. Additionally, if a concrete riser is excessively tall with a minimum amount of the riser secured in the embankment, the cavitation forces may cause the riser to rock on its foundation, risking possible structural failure.

The surging, vibrations, and other cavitation forces result when the riser is restricting flow to the barrel such that the riser is flowing full and the barrel is <u>not</u> flowing full. This condition occurs when the flow through the riser structure transitions from *riser weir flow control* to *riser orifice flow control* <u>before</u> the barrel controls. Therefore, the barrel and riser system should be designed so that as the storm continues and the hydraulic head on the riser increases, **the barrel controls the flow <u>before</u> the riser transitions from riser weir flow control to riser orifice flow control.** This can be accomplished by checking the flow rates for the riser weir, riser orifice, and barrel inlet and outlet flow control at each stage of discharge. **The lowest discharge for any given stage will be the controlling flow**.

The following procedures are for designing and checking riser and barrel system hydraulics.

a. Riser Flow Control

During the design of the control orifices and riser weir, the geometry of the riser is established. Subsequently, the riser must be checked to determine at what stage it transitions from *riser weir* to *riser orifice* flow control. The riser weir controls the flow initially, and then as the water rises, the top of the riser acts as a submerged horizontal orifice. Thus, the flow transitions from riser weir flow control to riser orifice flow control as the water in the basin rises. The flow capacity of the riser weir is determined using the **Weir Equation**, **Equation 5-8**, and the flow capacity of the riser orifice is determined using the **Orifice Equation**, **Equation 5-6**, for each elevation. **The smaller of the two flows for any given elevation is the controlling flow**.

1. Calculate the flow, in *cfs,* over the riser weir using the standard **Weir Equation**, **Equation 5-8**, for each elevation specified on the Stage-Storage-Discharge Worksheet, **Figure 5-13**. Record the flows on the worksheet.

The ***weir length***, *L*, is the circumference or length of the riser structure, measured at the crest, less any support posts or trash rack. The ***head*** is measured from the water surface elevation to the crest of the riser structure (refer to **Figure 5-14**).

2. Calculate the flow, in *cfs,* through the riser structure using the standard **Orifice Equation**, **Equation 5-6,** for each elevation specified on the Stage-Storage-Discharge Worksheet, **Figure 5-13**. Record the flows on the worksheet.

The ***Orifice flow area**, a,* is measured from the inside dimensions of the riser structure. The ***head*** is measured from the water surface elevation to the elevation of the orifice centerline, or, since the orifice is horizontal, to the elevation of the riser crest.

3. Compare the riser weir flow discharges to the riser orifice flow discharges. **The smaller of the two discharges is the controlling flow for any given stage**.

b. Barrel Flow Control

Two types of barrel flow exist: 1) ***barrel flow with inlet control***, as shown in **Figure 5-15c**, and 2) ***barrel flow with outlet, or pipe flow control***, as shown in **Figure 5-15d**. For both types, different factors and formulas are used to compute the hydraulic capacity of the barrel. During barrel inlet flow control, the diameter of the barrel, amount of head acting on the barrel, and the barrel entrance shape play a part in controlling the flow. For barrel outlet, or pipe flow, control, consideration is given to the length, slope, and roughness of the barrel, and the elevation of the tailwater, if any, in the outlet channel.

1. Barrel Inlet Flow Control

Barrel inlet flow control means that the capacity of the barrel is controlled at the barrel entrance by the depth of headwater and the barrel entrance, which is acting as a submerged orifice. The flow through the barrel entrance can be calculated using the **Orifice Equation, Equation 5-6,** or by simply using the Pipe Flow Nomograph shown in **Figure 5-16.** This nomograph provides stage-discharge relationships for concrete culverts of various sizes. [Additional nomographs for other pipe materials and geometrics are available; refer to the U.S. Bureau of Public Roads (BPR) Hydraulic Engineering Circular (H.E.C.) 5.] The ***headwater***, or depth of ponding, is the vertical distance measured from the water surface elevation to the invert at the entrance to the barrel. Refer to **Figure 5-16** for ratios of *headwater* to *pipe diameter*, or *HW/D*. This nomograph, based on the orifice equation, provides flow rates for three possible hydraulic entrance shapes, as shown in **Figure 5-17**. During barrel inlet flow control, neither the barrel's length nor its outlet conditions are factors in determining the barrel's capacity. Note that when the *HW/D* design values exceed the chart values, the designer may use the orifice equation (**Equation 5-6**) to solve for the flow rate.

The inlet control nomographs are not truly representative of barrel inlet flow. These nomographs should be used carefully and with the understanding that they were developed to predict flow through highway culverts operating under inlet control. However, depending on the size relationship between the riser and outlet conduit, the inlet control nomograph may provide a

The following procedure outlines the steps to calculate the discharge during ***barrel inlet flow*** control conditions:

1. Determine the *entrance condition* of the barrel (see **Figure 5-17**).

2. Determine the *headwater to pipe diameter ratio* (*HW/D*) for each elevation specified on the stage-storage-discharge worksheet. *Headwater* is measured from the water surface elevation to the upstream invert of the barrel (see **Figures 5-14 and 5-18**).

3. Determine the *discharge*, *Q,* in *cfs,* using the inlet control nomograph for circular concrete pipe presented in **Figure 5-16** (or the BPR H.E.C. 5 pipe flow nomographs for other pipe materials), or the **Orifice Equation**, **Equation 5-6** (for *HW/D* values which exceed the range of the nomographs) for each elevation specified on the Stage-Storage-Discharge Worksheet. Enter the values on the worksheet.

2. Barrel Outlet Flow Control

Barrels flowing under outlet or pipe flow control experience full flow for all or part of the barrel length, as shown in **Figure 5-15d**.

The general pipe flow equation is derived by using the Bernouli and Continuity Principles and is simplified to:

$$Q = a\sqrt{\frac{2gh}{1 + K_m + K_p L}}$$

Equation 5 - 10
Pipe Flow Control Equation

Where:
Q = *discharge, cfs*
a = *flow area of the barrel, ft*^{2}
g = *acceleration due to gravity, ft./sec*^{2}
h = *elevation head differential, ft., see* ***Figure 5-18***
K_m = *coefficient of minor losses:* $K_e + K_b$
K_e = *entrance loss coefficient, see* ***Table 5-9***
K_b = *bend loss coefficient, typically = 0.5 for riser and barrel system*
K_p = *coefficient of pipe friction, see* ***Table 5-10***
l = *length of the barrel, ft.*

This equation is derived and further explained in the SCS's Engineering Field Manual, Chapter 3.

The following procedure outlines the steps to check for ***barrel outlet control***:

1. Determine the discharge for each elevation specified in the stage-storage-discharge table using the general **Pipe Flow Equation**, **Equation 5-10**.

2. Record the discharge on the stage-storage-discharge worksheet, **Figure 5-13**.

3. Compare the barrel inlet flow control discharges with the barrel outlet flow control discharges. **The smaller of the two discharges is the controlling flow for any given stage**.

STEP 11 Size 100-Year Release Opening or Emergency Spillway

It is recommended that all stormwater impoundment structures have a vegetated emergency spillway, if possible. This provides a degree of safety to prevent overtopping of the embankment if the principal spillway should become clogged, or otherwise inoperative. **If an emergency spillway is not practical due to site constraints, the 100-year storm must be routed through the riser and barrel system**.

100-Year Release Opening

The design procedure for sizing the 100-year release opening is the same as that of the 10-year design, except that the 100-year storm values are used instead of the 10-year values.

Emergency Spillway

Refer to **Minimum Standard 3.03, Vegetated Emergency Spillway** in **Chapter 3** for location and design requirements of an emergency spillway and to **Section 5-8** in this chapter for the design procedure. An emergency spillway is a broad crested weir. It can act as a control structure by restricting the release of flow, or it can be used to safely pass the 100-year storm flow with a minimum of storage. The impact of the 100-year storm on the required storage is lessened by using an emergency spillway due to the spillway's ability to pass significant volumes of flow with little head. If an emergency spillway is not used, additional storage may be needed since the riser and barrel will usually pass only a small portion of the 100-year inflow. This remains true unless the riser and barrel are sized for the 100-year storm, in which case they will be oversized for the 2- and 10-year storms.

The following procedure can be used to design an emergency spillway that will safely pass, or control, the rate of discharge from the 100-year storm.

1. Identify the 10-year maximum water surface elevation based on the routing from **STEP 9**. This elevation will be used to establish the elevation of the 100-year release structure.

2. Determine the storage volume that corresponds to the 100-year control elevation from the stage-storage curve. Add this elevation, storage, and appropriate storm discharges to the Stage-Storage-Discharge Worksheet.

3. Set the invert of the emergency spillway at the 10-year high water elevation.

4. Determine the 100-year developed inflow from the hydrology.

A distance of 0.1 feet, minimum, is recommended between the 10-year high water mark and the invert of the emergency spillway.

5. Using the design procedure provided in **Chapter 5-8**, determine the required bottom width of the spillway, the length of the spillway level section, and the depth of flow through the spillway that adequately passes the 100-year storm within the available free board. The minimum free board required is 1 foot from the 100-year water surface elevation to the settled top of embankment.

6. Develop the stage-storage-discharge relationship for the 100-year storm. Calculate the

discharge for each elevation specified on the stage-storage curve and record the discharge on the Stage-Storage-Discharge Worksheet, **Figure 5-13**. If a release rate is specified, then the TR-55 shortcut method can be used to calculate the approximate storage volume requirement. If a fixed storage volume is available, the same shortcut method can be used to decide what the discharge must be to ensure that the available storage is not exceeded. Refer to TR-55.

STEP 12 Calculate Total Discharge and Check Performance of 100-Year Control Opening

1. Calculate total discharge.

   The stage-storage-discharge table is now complete and the total discharge from the riser and barrel system and emergency spillway can be determined. The designer should verify that the barrel flow controls before the riser transitions from riser weir flow control to riser orifice flow control.

   The combined flows from the water quality orifice, the 2-year opening, the 10-year opening, and the riser will, at some point, exceed the capacity of the barrel. At this water surface elevation and discharge, the system transitions from riser flow control to barrel flow control. The total discharge for each elevation is simply the sum of the flows through the control orifices of the riser, or the controlling flow through the barrel and riser, whichever is **less**.

   In **Chapter 6,** the examples contain completed Stage-Storage-Discharge Worksheets. Notice that the flows that do not control are crossed out. The controlling flows are then summed in the total flow column to provide the total stage-storage-discharge relationship of the basin.

2. Check the performance of the 100-year control by a) *reservoir routing the 2-year, 10-year, and 100-year storms through the basin using an acceptable reservoir routing computer program* or by b) *doing the long hand calculations outlined in* ***Section 5-9***. **Verify that the design storm release rates are less than or equal to the allowable release rates, and that the 100-year design high water is:**

   a. at least 2 *ft.* lower than the settled top of embankment elevation if an emergency spillway is NOT used, or

   b. at least 1 *ft.* lower than the settled top of embankment if an emergency spillway is used.

Also, the designer should verify that the release rates for each design storm are not too low, which would result in more storage being provided than is required.

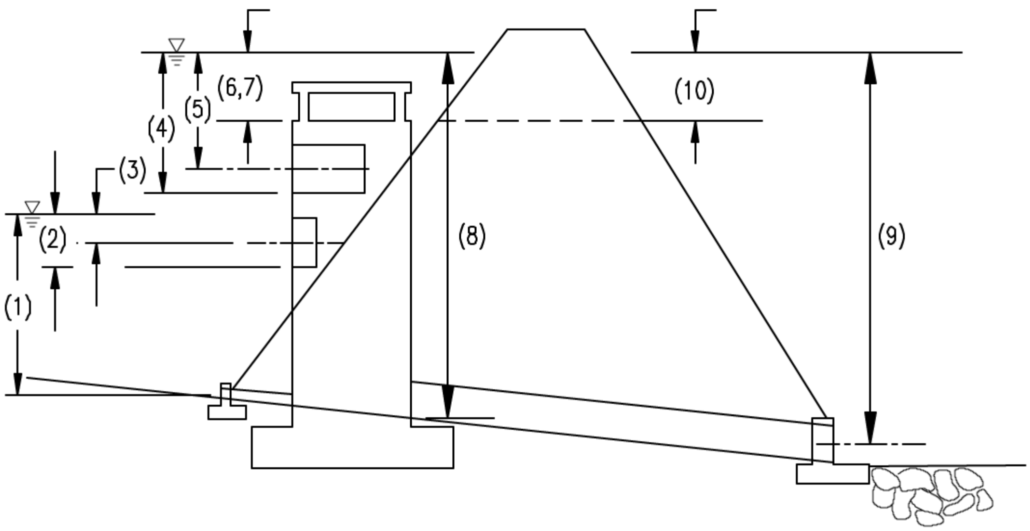
FIGURE 5 - 13
Stage - Storage - Discharge Worksheet, Example 1

ELEV (MSL)	STORAGE (ac.ft.)	1-yr. ext. det. ~~WATER QUALITY~~ ORIFICE (1)		TRIAL 2 10-YEAR CONTROL: WEIR (4)		TRIAL 2 10-YEAR CONTROL: ORIFICE (5)		RISER STRUCTURE: WEIR (6)		RISER STRUCTURE: ORIFICE (7)		BARREL: INLET (8)		BARREL: OUTLET (9)		EMERGENCY SPILLWAY (10)		TOTAL Q (cfs)
		h	Q	h	Q	h	Q	h	Q	h	Q	HW/D	Q	h	Q	h	Q	
81	0	0	0															0
82	.02	1	0.3															0.3
84	.14	3	0.6															0.6
86	.33	5	0.7															0.7
88	.68	7	0.9															0.9
88.8	.90	7.8	0.98	0	0				0/0	0	0							1.0
89	.95	8	1.0	0.2	0.4				0/0.4	0.2	~~34.4~~	5.5	~~24~~	8.3	~~22.0~~			1.4
90	1.28	9	1.1	1.2	5.8				0/5.8	1.2	~~84.4~~	6.2	~~26~~	9.3	~~23.3~~			6.9
91	1.75	10	1.1	2.2	14.3				0/14.3	2.2	~~114.3~~	6.8	~~27~~	10.3	~~24.5~~			15.4
92	2.23	11	~~1.1~~	3.2	~~25.1~~				0/25.1	3.2	~~139.8~~	7.5	~~28~~	11.3	25.7			25.7
92.2	2.40	11.2	~~1.1~~	3.4	~~27.5~~			0	0/27.5	3.4	~~142.0~~	7.6	~~29~~	11.5	25.9	0	0	25.9
93	2.94	12.0	~~1.1~~	4.2	~~37.8~~			0.8	17.0/54.8	4.2	~~157.9~~	8.1	~~30~~	12.3	26.8	0.8	54	80.8
94	3.81	13.0	~~1.2~~	5.2	~~52.0~~			1.8	51.5/109.5	5.2	~~175.7~~	8.8	~~31~~	13.3	27.8	1.8	214	241.8

(6) Q for elevations 88.8 to 92.2 ft. represents 10-yr. weir flow (4)
Q for elevations 92.2 to 94 ft represents 10-yr weir flow plus the top of riser weir flow: $Q = 23.8 (h)^{1.5}$ where h = wse - 92.2 ft.

(7) $Q = 77.03 (h)^{1/2}$ where h = wse - 88.8 ft
(8) $Q = 8.5 (h)^{1/2}$ where h = wse - 80.75 ft
(9) $Q = 7.64 (h)^{1/2}$ where h = wse - 80.75 ft
(10) Figure 5-23: Design Data for Earth Spillways.

FIGURE 5 - 14
Typical Hydraulic Head Values - Multi-Stage Riser



1. WQ Orifice: H measured from water surface elevation (WSE) to centerline of pipe, or orifice. (orifice flow)
2. 2–year Control: Weir flow – H measured from WSE to invert of 2–year control weir.
3. 2–year Control: Orifice flow – H measured from WSE to centerline of opening (submerged).
4. 10–year Control: Weir flow – H measured from WSE to crest of opening.
5. 10–year Control: Orifice flow – H measured from WSE to centerline of opening.
6. Riser Structure: Weir flow – H measured from WSE to crest of riser top (if open).
7. Riser Structure: Orifice flow – H measured from WSE to crest of riser top, acting as horizontal orifice.
8. Barrel flow: Inlet control – H measured from WSE to upstream invert of outlet barrel.
9. Barrel flow: Outlet control – H measured from WSE to centerline of outlet of barrel or tailwater – whichever is higher.
10. Emergency Spillway: H measured from WSE to crest of Emergency Spillway.

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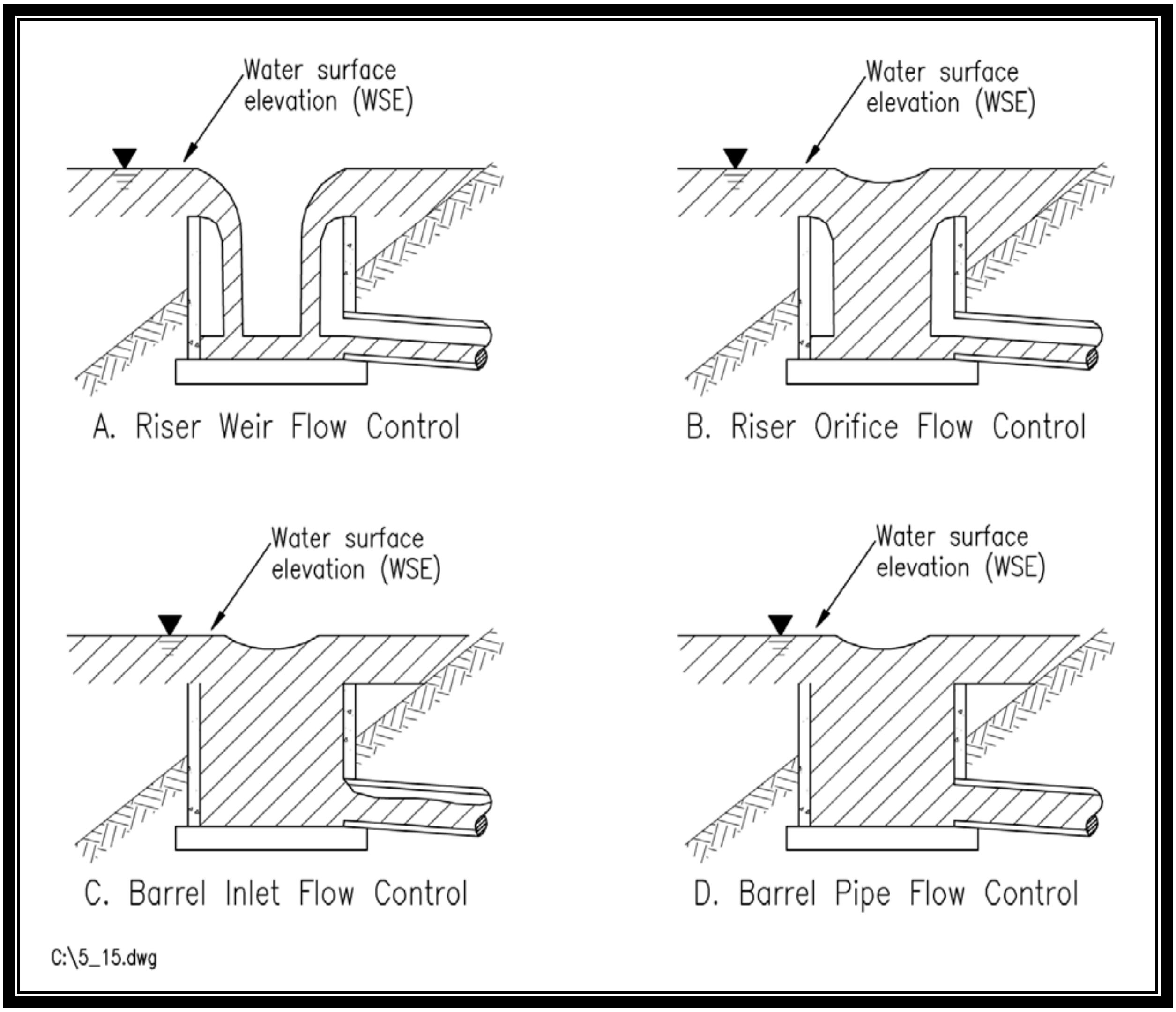
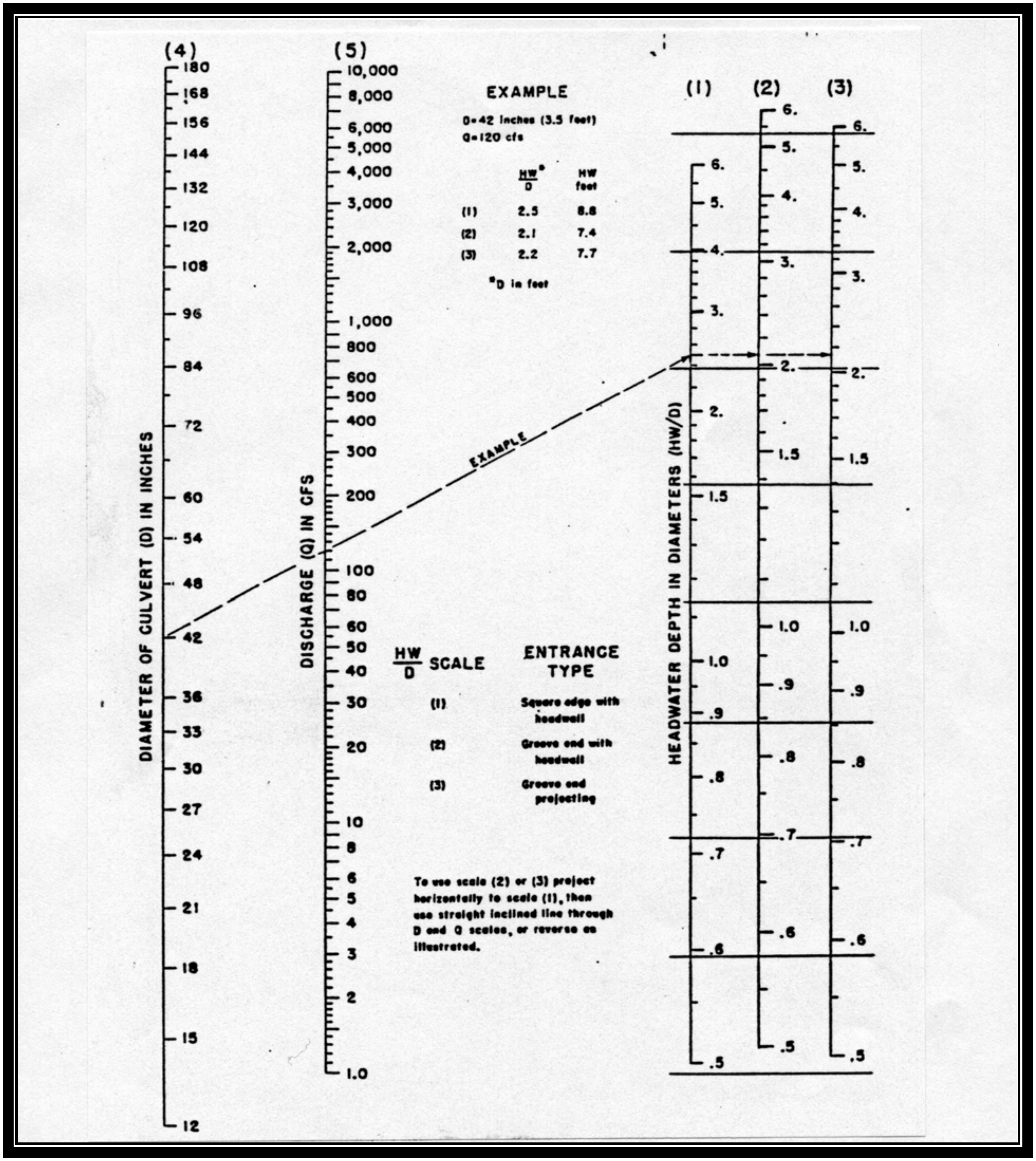
FIGURE 5 - 15 a, b, c, & d
Riser Flow Diagrams



Source: SCS Engineering Field Manual - Chapter 6

FIGURE 5 - 16
Headwater Depth for Concrete Pipe Culverts With Inlet Control



Source: Bureau of Public Roads

FIGURE 5 - 17
Headwater Depth Entrance Conditions

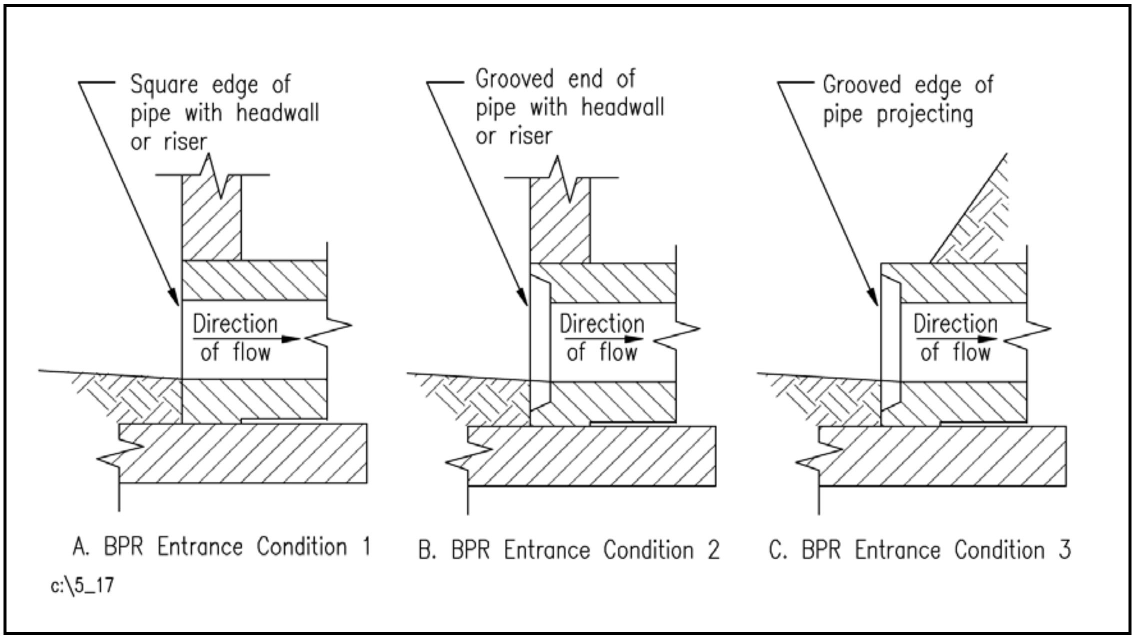


FIGURE 5 - 18
Hydraulic Head Values - Barrel Flow

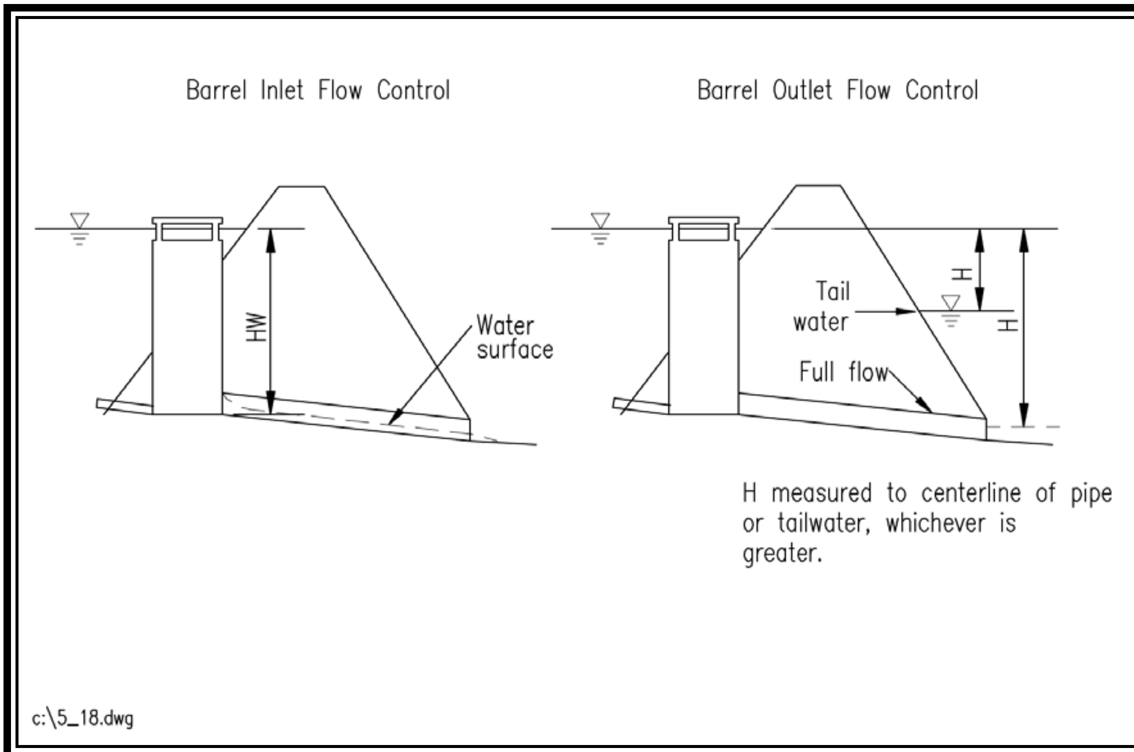


TABLE 5 - 9
Pipe Entrance Loss Coefficients - K_e

Type of Structure and Design of Entrance	**Coefficient K_e**
Pipe, Concrete	
Projecting from fill, socket end (groove end)	0.2
Projecting from fill, square cut end	0.5
Headwall or headwall and wingwalls	
Socket end of pipe (groove end)	0.2
Square end	0.5
Rounded (radius = 1/12D)	0.2
Mitered to conform to fill slope	0.7
*End-section conforming to fill slope	0.5
Pipe, or Pipe-Arch, Corrugated Metal	
Projecting from fill (no headwall)	0.9
Headwall or headwall and wingwalls	
Square end	0.5
Mitered to conform to fill slope	0.7
*End-section conforming to fill slope	0.5

**Note: "End-section conforming to fill slope" made of either metal or concrete, is the section commonly available from manufacturers. Based on limited hydraulic tests, it appears to be equivalent in operation to a headwall in either inlet or outlet control.*

Source - Federal Highway Administration, Bureau of Public Roads

TABLE 5 - 10
Head Loss Coefficients, K_p, for Circular and Square Conduits Flowing Full

HEAD LOSS COEFFICIENT, K_p, FOR CIRCULAR PIPE FLOWING FULL $K_p = \frac{5087\, n^2}{d_i^{4/3}}$

Pipe diam. inches	Flow area sq. ft.	MANNING'S COEFFICIENT OF ROUGHNESS "n"															
		0.010	0.011	0.012	0.013	0.014	0.015	0.016	0.017	0.018	0.019	0.020	0.021	0.022	0.023	0.024	0.025
6	0.196	0.0467	0.0565	0.0672	0.0789	0.0914	0.1050	0.1194	0.1348	0.151	0.168	0.187	0.206	0.226	0.247	0.269	0.292
8	0.349	.0318	.0385	.0458	.0537	.0623	.0715	.0814	.0919	.1030	.1148	.1272	.140	.154	.168	.183	.199
10	0.545	.0236	.0286	.0340	.0399	.0463	.0531	.0604	.0682	.0765	.0852	.0944	.1041	.1143	.1249	.136	.148
12	0.785	.0185	.0224	.0267	.0313	.0363	.0417	.0474	.0535	.0600	.0668	.0741	.0817	.0896	.0980	.1067	.1157
14	1.069	.0151	.0182	.0217	.0255	.0295	.0339	.0386	.0436	.0488	.0544	.0603	.0665	.0730	.0798	.0868	.0942
15	1.23	.0138	.0166	.0198	.0232	.0270	.0309	.0352	.0397	.0446	.0496	.0550	.0606	.0666	.0727	.0792	.0859
16	1.40	.0126	.0153	.0182	.0213	.0247	.0284	.0323	.0365	.0409	.0455	.0505	.0556	.0611	.0667	.0727	.0789
18	1.77	.01078	.0130	.0155	.0182	.0211	.0243	.0276	.0312	.0349	.0389	.0431	.0476	.0522	.0570	.0621	.0674
21	2.41	.00878	.01062	.0126	.0148	.0172	.0198	.0225	.0254	.0284	.0317	.0351	.0387	.0425	.0464	.0506	.0549
24	3.14	.00735	.00889	.01058	.0124	.0144	.0165	.0188	.0212	.0238	.0265	.0294	.0324	.0356	.0389	.0423	.0459
27	3.98	.00628	.00760	.00904	.01061	.0123	.0141	.0161	.0181	.0203	.0227	.0251	.0277	.0304	.0332	.0362	.0393
30	4.91	.00546	.00660	.00786	.00922	.01070	.0228	.0140	.0158	.0177	.0197	.0218	.0241	.0264	.0289	.0314	.0341
36	7.07	.00428	.00518	.00616	.00723	.00839	.00963	.01096	.0124	.0139	.0154	.0171	.0189	.0207	.0226	.0246	.0267
42	9.62	.00348	.00422	.00502	.00589	.00683	.00784	.00892	.01007	.01129	.0126	.0139	.0154	.0169	.0184	.0201	.0218
48	12.57	.00292	.00353	.00420	.00493	.00572	.00656	.00747	.00843	.00945	.01053	.01166	.0129	.0141	.0154	.0168	.0182
54	15.90	.00249	.00302	.00359	.00421	.00488	.00561	.00638	.00720	.00808	.00900	.00997	.01099	.0121	.0132	.0144	.0156
60	19.63	.00217	.00262	.00312	.00366	.00426	.00487	.00554	.00626	.00702	.00782	.00866	.00955	.01048	.0115	.0125	.0135

HEAD LOSS COEFFICIENT, K_c, FOR SQUARE CONDUIT FLOWING FULL $K_c = \frac{29.16\, n^2}{r^{4/3}}$

Conduit Size feet	Flow area sq. ft.	MANNING'S COEFFICIENT OF ROUGHNESS "n"				
		0.012	0.013	0.014	0.015	0.016
2×2	4.00	0.01058	0.01242	0.01440	0.01653	0.01880
2½×2½	6.25	0.00786	0.00922	0.01070	0.01228	0.01397
3×3	9.00	.00616	.00723	.00839	.00963	.01096
3½×3½	12.25	.00502	.00589	.00683	.00784	.00892
4×4	16.00	.00420	.00493	.00572	.00656	.00746
4½×4½	20.25	.00359	.00421	.00488	.00561	.00638
5×5	25.00	.00312	.00366	.00425	.00487	.00554
5½×5½	30.25	.00275	.00322	.00374	.00429	.00488
6×6	36.00	.00245	.00287	.00333	.00382	.00435
6½×6½	42.25	.00220	.00258	.00299	.00343	.00391
7×7	49.00	.00199	.00234	.00271	.00311	.00354
7½×7½	56.25	.00182	.00213	.00247	.00284	.00323
8×8	64.00	.00167	.00196	.00227	.00260	.00296
8½×8½	72.25	.00154	.00180	.00209	.00240	.00273
9×9	81.00	.00142	.00167	.00194	.00223	.00253
9½×9½	90.25	.00133	.00156	.00180	.00207	.00236
10×10	100.00	.00124	.00145	.00168	.00193	.00220

$$H_1 = (K_p \text{ or } K_c)\, L\, \frac{v^2}{2g}$$

Nomenclature:

- a = Cross-sectional area of flow in sq. ft.
- d_i = Inside diameter of pipe in inches.
- g = Acceleration of gravity = 32.2 ft. per sec.
- H_1 = Loss of head in feet due to friction in length L.
- K_c = Head loss coefficient for square conduit flowing full.
- K_p = Head loss coefficient for circular pipe flowing full.
- L = Length of conduit in feet.
- n = Manning's coefficient of roughness.
- Q = Discharge or capacity in cu. ft. per sec.
- r = Hydraulic radius in feet.
- v = Mean velocity in ft. per sec.

Example 1: Compute the head loss in 300 ft. of 24 in. diam. concrete pipe flowing full and discharging 30 c.f.s. Assume $n = 0.015$

$v = \frac{Q}{a} = \frac{30}{3.14} = 9.55$ f.p.s.; $\frac{v^2}{2g} = \frac{(9.55)^2}{64.4} = 1.42$ ft.

$H_1 = K_p L \frac{v^2}{2g} = 0.0165 \times 300 \times 1.42 = 7.03$ ft.

Example 2: Compute the discharge of a 250 ft., 3×3 square conduit flowing full if the loss of head is determined to be 2.25 ft. Assume $n = 0.014$.

$H_1 = K_c L \frac{v^2}{2g}$; $\frac{v^2}{2g} = \frac{H_1}{K_c L} = \frac{2.25}{0.00839 \times 250} = 1.073$ ft.

$v = \sqrt{64.4 \times 1.073} = 8.31$; $Q = 9 \times 8.31 = 74.8$ c f s.

Head loss coefficients for circular and square conduits flowing full (Ref. NEH Section 5, ES-42)

STEP 13 Design Outlet Protection

With the total discharge known for the full range of design storms, adequate outlet protection can now be designed. Protection is necessary to prevent scouring at the outlet and to help reduce the potential for downstream erosion by reducing the velocity and energy of the concentrated discharge. The most common form of outlet protection is a riprap-lined apron, constructed at zero grade for a specified distance, as determined by the outlet flow rate and tailwater elevation. The design procedure follows:

> ***Note that this procedure is for riprap outlet protection at the downstream end of an embankment conduit. It DOES NOT apply to continuous rock linings of channels or streams. Refer to Figure 5 -19***.

1. Determine the tailwater depth, for the appropriate design storm, immediately below the discharge pipe.

Typically, the discharge pipe from a stormwater management facility is sized to carry the allowable discharge from the 10-year frequency design storm. Manning's equation can be used to find the water surface elevation in the receiving channel for the 10-year storm, which represents the ***tailwater elevation***. If the tailwater depth is less than half the outlet pipe diameter, it is called a ***minimum tailwater condition***. If the tailwater depth is greater than half the outlet pipe diameter, it is called a ***maximum tailwater condition***. Stormwater basins that discharge onto flat areas with no defined channel may be assumed to have a *minimum tailwater condition.*

> ***Outflows from stormwater management facilities must be discharged to an adequate channel. Basins discharging onto a flat area with no defined channel will usually require a channel to be provided which can convey the design flows.***

2. Determine the required riprap size, D_{50}, and apron length, L_a.

Enter the appropriate figure, either **Figure 5-20: Minimum Tailwater Condition**, or **Figure 5-21: Maximum Tailwater Condition**, with the design discharge of the pipe spillway to read the required apron length, L_s. (The apron length should not be less than 10 feet.)

3. Determine the required riprap apron width, W.

When the pipe discharges directly into a well-defined channel, the apron shall extend across the channel bottom and up the channel banks to an elevation 1 foot above the maximum tailwater depth or the top of bank, whichever is less.

If the pipe discharges onto a flat area with no defined channel, the width of the apron shall be determined as follows:

a. The upstream end of the apron, next to the pipe, shall be 3 times wider than the diameter of the outlet pipe.

b. For a ***minimum tailwater condition***, the width of the apron's downstream end shall equal the pipe diameter plus the length of the apron.

c. For a ***maximum tailwater condition***, the width of the apron's downstream end shall equal the pipe diameter plus 0.4 times the length of the apron.

Using the same figure as in Step 2, above, determine the D_{50} riprap size and select the appropriate class of riprap, as shown in **Table 5-11**. Values falling between the table values should be rounded up to the next class size.

4. Determine the required depth of the rip rap blanket.

   The depth of the rip rap blanket is approximated as: 2.25 x D_{50}

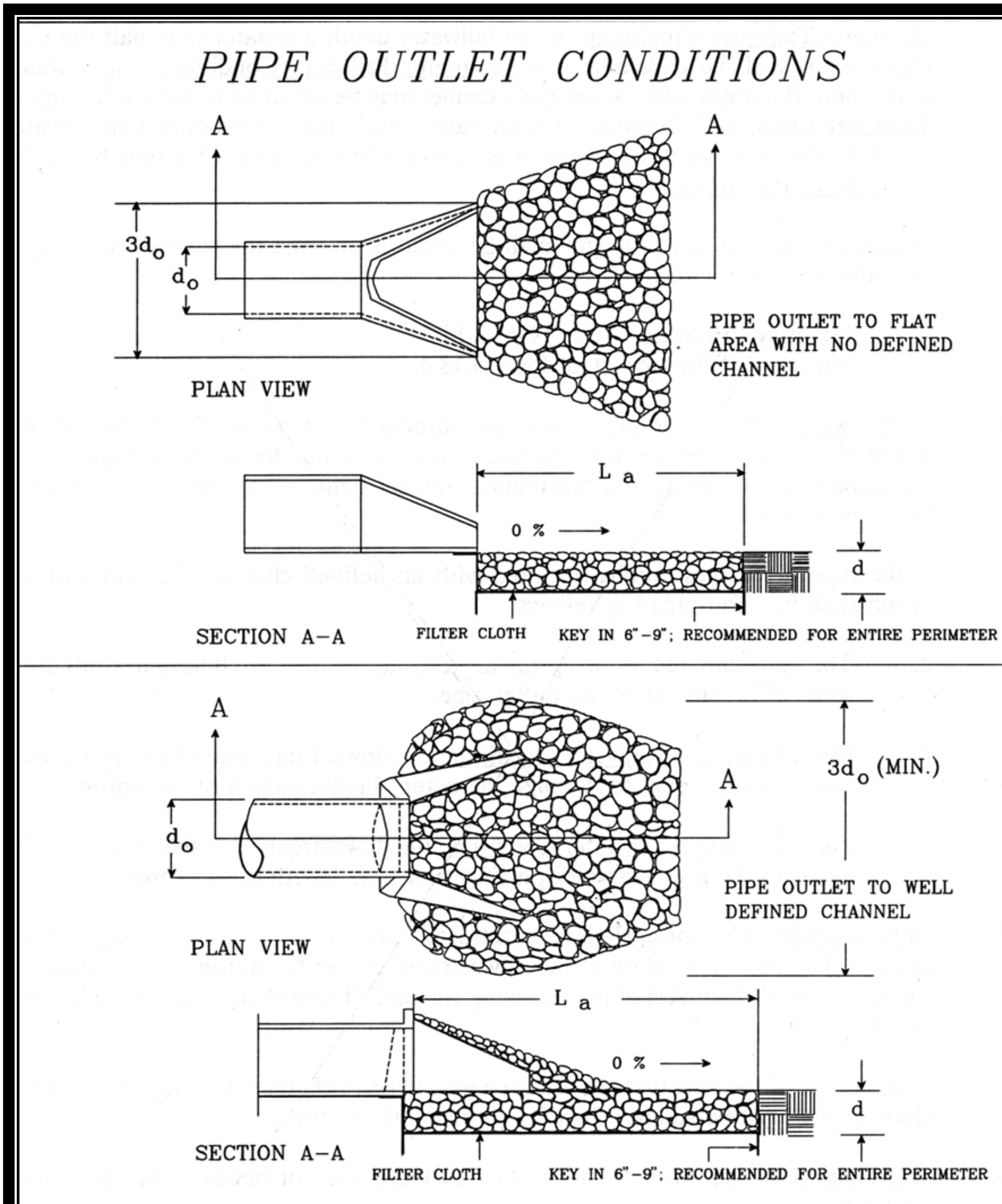
Additional design considerations and specifications can be found in **Minimum Standard 3.02, Principal Spillway** and Std. and Spec. 3.18 and 3.19 of the Virginia Erosion and Sediment Control Handbook, 1992 edition.

TABLE 5 - 11
Graded Riprap Design Values

Riprap Class	D_{15} Weight *(lbs.)*	Mean D_{15} Spherical Diameter *(ft.)*	Mean D_{50} Spherical Diameter *(ft.)*
Class AI	25	0.7	0.9
Class I	50	0.8	1.1
Class II	150	1.3	1.6
Class III	500	1.9	2.2
Type I	1,500	2.6	2.8
Type I	6,000	4.0	4.5

Source: VDOT Drainage Manual

FIGURE 5 - 19
Outlet Protection Detail



NOTES:
1. APRON LINING MAY BE RIPRAP, GROUTED RIPRAP, GABION BASKET, OR CONCRETE.
2. La IS THE LENGTH OF THE RIPRAP APRON AS CALCULATED USING FIG 5-20 AND FIG 5-21.
3. d = 1.5 TIMES THE MAXIMUM STONE DIAMETER, BUT NOT LESS THAN 6 INCHES.

FIGURE 5 - 20
Minimum Tailwater Condition

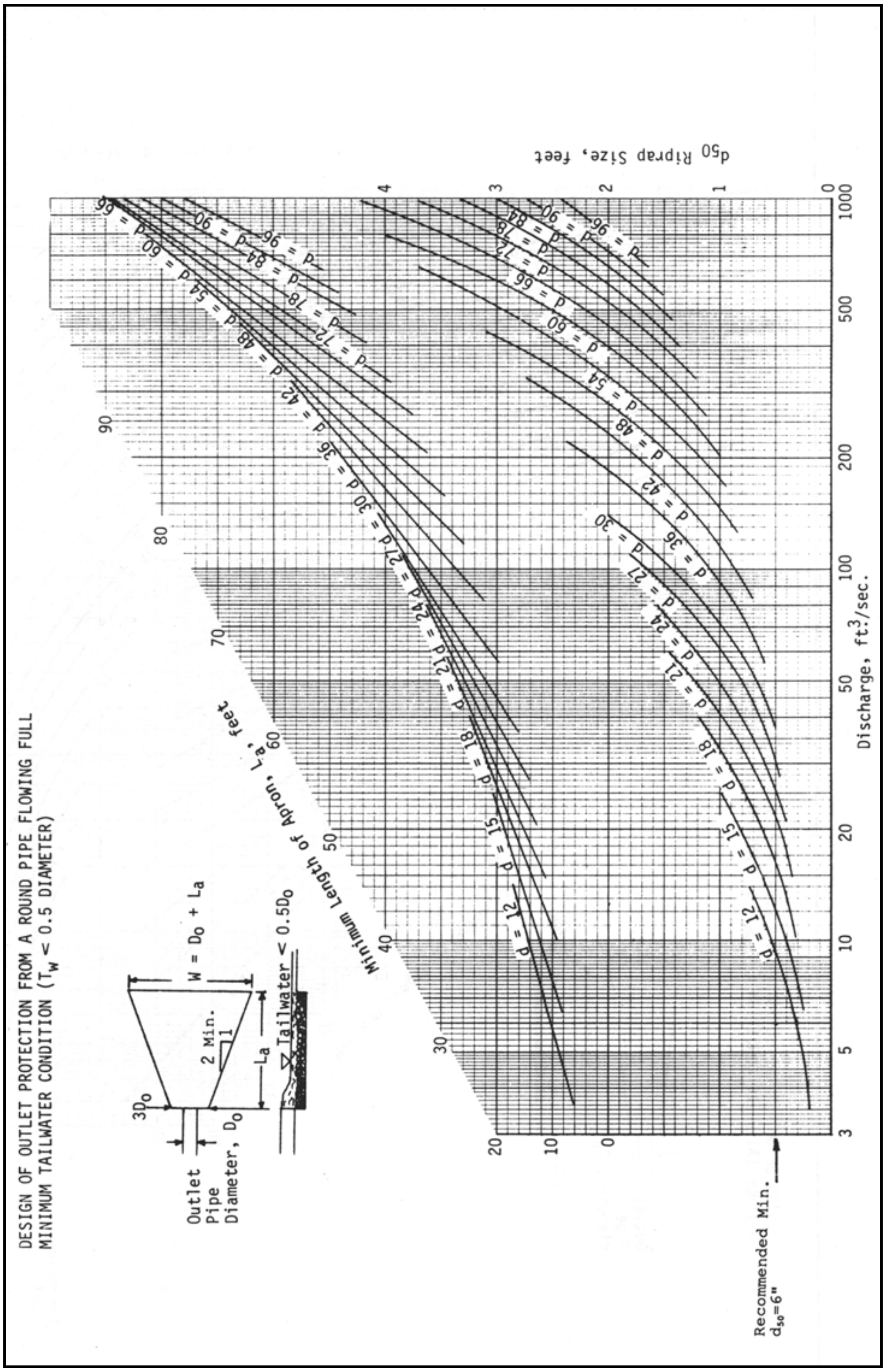
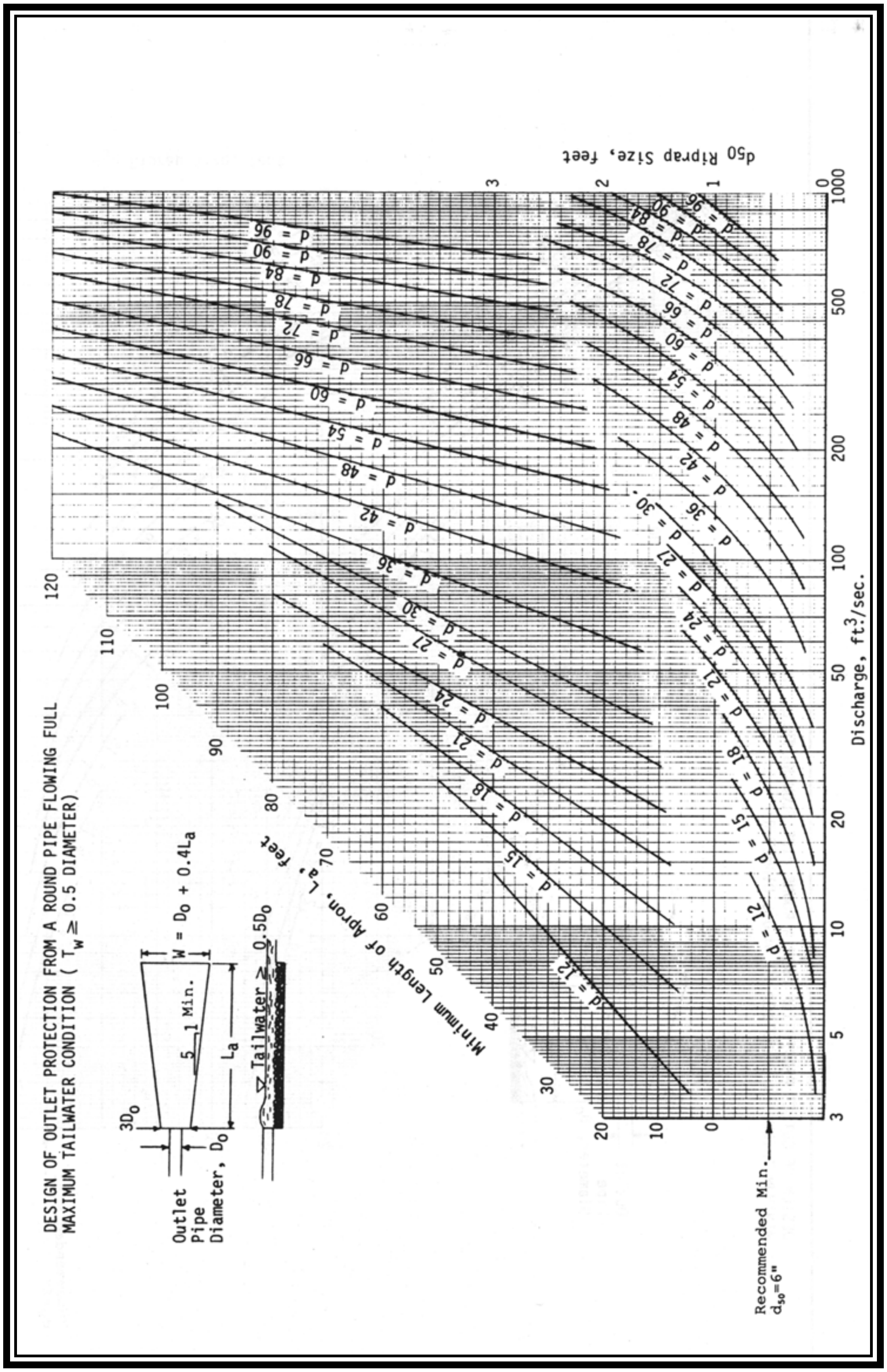


FIGURE 5 - 21
Maximum Tailwater Condition



STEP 14 **Perform Buoyancy Calculation**

The design of a multi-stage riser structure must include a buoyancy analysis for the riser and footing. When the ground is saturated and ponded runoff is at an elevation higher than the footing of the riser structure, the riser structure acts like a vessel. During this time, the riser is subject to uplifting, buoyant forces that are relative in strength to the volume of water displaced. **Flotation will occur when the weight of the structure is less than or equal to the buoyant force exerted by the water**. Flotation forces on the riser can lead to failure of the connection between the riser structure and barrel, and any other rigid connections. Eventually, this can also lead to the failure of the embankment.

A buoyancy calculation is the summation of all forces acting on the riser. The upward force is the weight of the water, or 62.4 lb/ft^3. The downward force includes the weight of the riser structure, any components, such as trash racks, and the weight of the soil above the footing. Note that conventional reinforced concrete weighs about 150 lb/ft^3 and the unit weight of soil is approximately 120 lb/ft^3 . The weight of components such as trash racks, anti-vortex devices, hoods, etc. is very specific to each structure and, depending upon the design, may or may not be significant in comparison to the other forces. If an extended base footing is used below the ground surface to support the control structure, then the weight of the soil above the footing may also be a significant force.

The outlet pipe is excluded from the buoyancy analysis for the control structure. However, the barrel should be analyzed separately to insure that it is not subject to flotation. The method used to attach the control structure to the outlet pipe is considered to have no bearing on the potential for these components to float.

The following procedure compares the upward force (buoyant force) to the downward force (structure weight). To maintain adequate stability, **the downward force should be a minimum of 1.25 times the upward force**.

1. Determine the buoyant force.

The buoyant force is the total volume of the riser structure and base, using outside dimensions (i.e., *the total volume displacement of the riser structure*) multiplied by the unit weight of water (62.4 lb/ft^3).

2. Determine the downward or resisting force.

The downward force is the total volume of the riser walls below the crest, including any top slab, footing, etc., less the openings for any pipe connections, multiplied by the unit weight of reinforced concrete (150 lb/ft^3). Additional downward forces from any components may also be added, including the weight of the soil above the extended footing.

3. Decide if the downward force is greater than the buoyant force by a factor of 1.25 or more.

If the downward force is not greater than the buoyant force by a factor of 1.25 or more, then additional weight must be added to the structure. This can be done by sinking the riser footing deeper into the ground and adding concrete to the base. Note that this will also increase the buoyant force, but since the unit weight of concrete is more than twice that of water, the net result will be an increase in the downward force. The downward and buoyant forces should be adjusted accordingly, and step 3 repeated.

STEP 15 Provide Seepage Control

Seepage control should be provided for the pipe through the embankment. The two most common devices for controlling seepage are 1) *filter and drainage diaphragms* and 2) *anti-seep collars*. The use of these devices is discussed in detail in **Minimum Standard 3.02, Principal Spillway**. Note that filter and drainage diaphragms are preferred over anti-seep collars for controlling seepage along pipe conduits.

a. Filter & Drainage Diaphragms

The design of filter and drainage diaphragms depends on the foundation and embankment soils and is outside the scope of this manual. When filter and drainage diaphragms are warranted, their design and construction should be supervised by a registered professional engineer. Design criteria and construction procedures for filter and drainage diaphragms can be found in the following references:

- USDA SCS TR-60
- USDA SCS Soil Mechanics Note No. 1: Guide for Determining the Gradation of Sand and Gravel Filters*
- USDA SCS Soil Mechanics Note No. 3: Soil Mechanics Consideration for Embankment Drains*
- U.S. Department of the Interior ACER Technical Memorandum No. 9: Guidelines for Controlling Seepage Along Conduits Through Embankments

* *These publications include design procedures and examples and are provided in* ***Appendix 5B***.

b. Anti-Seep Collars

The Bureau of Reclamation, the U.S. Army Corps of Engineers and the Soil Conservation Service no longer recommend the use of anti-seep collars. In 1987, the Bureau of Reclamation issued Technical Memorandum No. 9 that states:

> *"When a conduit is selected for a waterway through an earth or rockfill embankment, cutoff [anti-seep]collars will not be selected as the seepage control measure."*

Alternative measures to anti-seep collars include *graded filters* (or *filter diaphragms*) and *drainage blankets*. These devices are not only less complicated and more cost-effective to construct than cutoff collars, but also allow for easier placement of the embankment fill. Despite this information, anti-seep collars may be appropriate for certain situations. A design procedure is provided below. Criteria for the use and placement of anti-seep collars are presented in **Minimum Standard 3.02, Principal Spillway**.

1. Determine the length of the barrel within the saturated zone using the following equation:

$$L_s = Y\,(Z + 4)\left[1 + \frac{S}{0.25 - S}\right]$$

Equation 5 - 11
Barrel Length in Saturated Zone

Where:
- L_s = *length of the barrel in the saturated zone, ft.*
- Y = *the depth of water at the principal spillway crest (10-year frequency storm water surface elevation), ft.*
- Z = *slope of the upstream face of the embankment, in Z ft. horizontal to 1 ft. vertical (Z ft. H:1V).*
- S = *slope of the barrel, in feet per foot.*

The length of pipe within the saturated zone can also be determined graphically on a *scale profile* of the embankment and barrel. The saturated zone of the embankment can be approximated as follows: starting at a point where the 10-year storm water surface elevation intersects the embankment slope, extend a line at a *4H:1V* slope downward until it intersects the barrel. The area under this line represents the ***theoretical zone of saturation*** (refer to **Figure 5-22**).

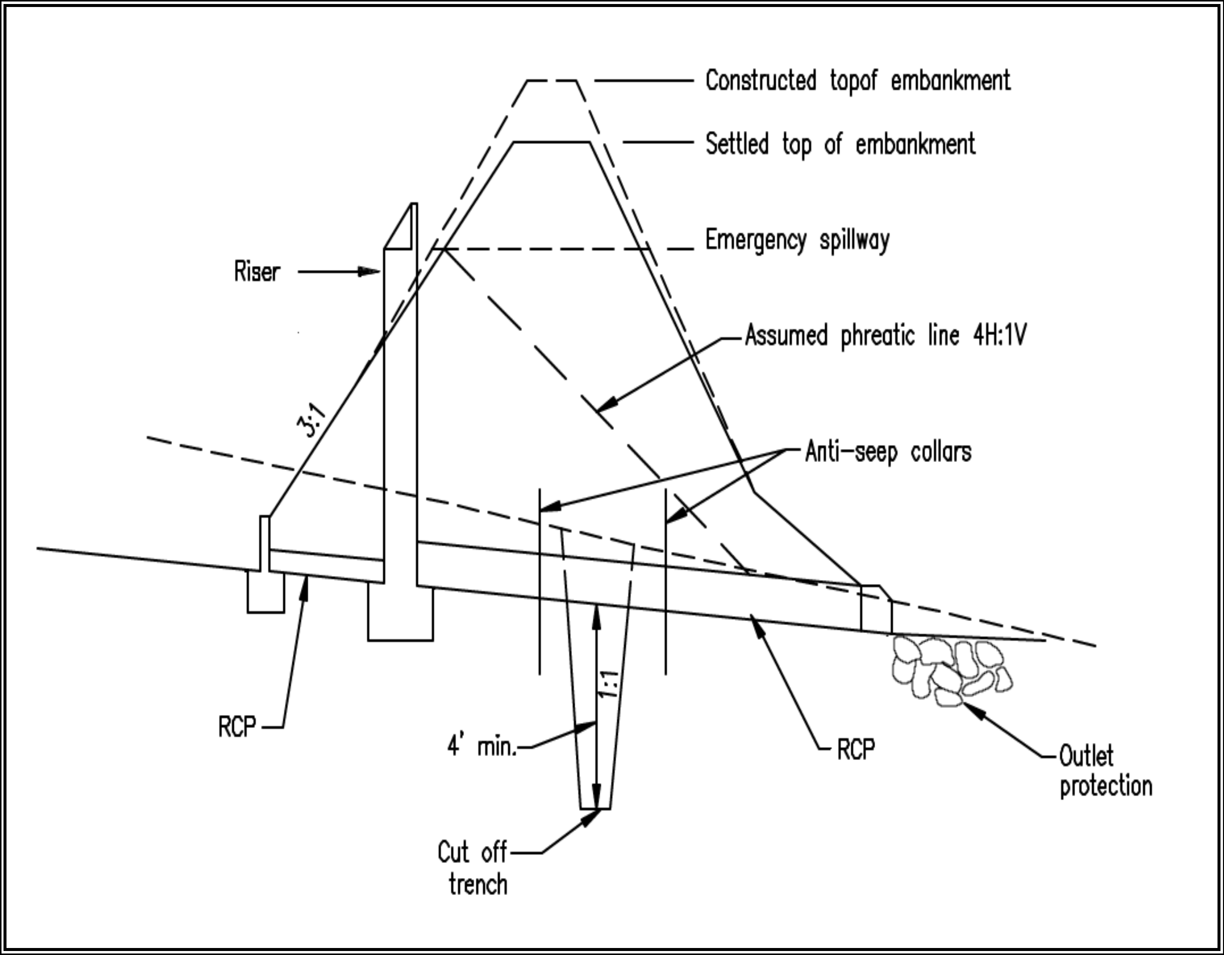
2. Determine the length required by multiplying 15% times the seepage length: $0.15\,L_s$. The increase in seepage length represents the total collar projection. This can be provided for by one or multiple collars.

3. Choose a collar size that is at least 4 feet larger than the barrel diameter (2 feet above and 2 feet below the barrel). For example, a 7-feet square collar would be selected for a 36-inch diameter barrel.

4. Determine the collar projection by subtracting the pipe diameter from the collar size.

5. Determine the number of collars required. The number of collars is found by dividing the seepage length increase, found in Step 2, by the collar projection from Step 4. To reduce the number of collars required, the collar size can be increased. Alternatively, the collar size can be decreased by providing more collars.

FIGURE 5 - 22
Phreatic Line Graphical Determination



SUMMARY
MULTI-STAGE RISER DESIGN PROCEDURE

STEP 1: Determine Water Quality Volume Requirements
- a. Extended-Detention
- b. Retention

STEP 2: Compute Allowable Release Rates

STEP 3: Estimate the Required Storage Volume

STEP 4: Grade the Basin; Create Stage-Storage Curve

STEP 5a: Design Water Quality Orifice (Extended-Detention)

STEP 5b: Set Permanent Pool Volume (Retention)

STEP 6: Size 2-Year Control Orifice

STEP 7: Check Performance of 2-Year Opening

STEP 8: Size 10-Year Control Opening

STEP 9: Check Performance of 10-Year Opening

STEP 10: Perform Hydraulic Analysis
- a. Riser Flow Control
- b. Barrel Flow Control
    1. Barrel Inlet Flow Control
    2. Barrel Outlet Flow Control

STEP 11: Size 100-Year Release Opening or Emergency Spillway

STEP 12: Calculate Total Discharge and Check Performance of 100-Year Control Opening

STEP 13: Design Outlet Protection

STEP 14: Perform Buoyancy Calculation

STEP 15: Provide Seepage Control

5-8 EMERGENCY SPILLWAY DESIGN

A vegetated emergency spillway is designed to convey a predetermined design flood volume without excessive velocities and without overtopping the embankment.

Two design methods are presented here. The first (Procedure 1) is a conservative design procedure which is also found in The Virginia Erosion & Sediment Control Handbook, 1992 edition, Std. & Spec. 3.14. This procedure is typically acceptable for stormwater management basins. The second method (Procedure 2) utilizes the roughness, or retardance, and durability of the vegetation and soils within the vegetated spillway. This second design is appropriate for larger or regional stormwater facilities where construction inspection and permanent maintenance are more readily enforced. These larger facilities typically control relatively large watersheds and are located such that the stability of the emergency spillway is essential to safeguard downstream features.

The following design procedures establish a stage-discharge relationship (H_p versus Q) for a vegetated emergency spillway serving a stormwater management basin (refer to **Figure 5-23**).

The information required for these designs includes the determination of the hydrology for the watershed draining to the basin. Any of the methods, as outlined in **Chapter 4**, may be used. The design should include calculations for the ***allowable release rate*** from the basin if the spillway is to be used to control a design frequency storm. Otherwise, the ***design peak flow rate*** should be calculated based on the spillway design flood, or downstream conditions.

(In general, a vegetated emergency spillway should not be used as an outlet for any storm less than the 100-year frequency storm, unless it is armored with a non-erodible material. The designer must consider the depth of the riprap blanket when riprap is used to armor the spillway. As noted previously, Class I riprap would require a blanket thickness or stone depth of 30" which may add considerable height to the embankment.)

The design maximum water surface elevations for the emergency spillway should be at least 1 foot lower than the settled top of the embankment. Refer to Minimum Standard 3.03, Vegetated Emergency Spillways.

Procedure 1:

1. Determine the ***design peak rate of inflow*** from the spillway design flood into the basin using the developed condition hydrology **or** determine the ***allowable design peak release rate, Q,*** from the basin based on downstream conditions or watershed requirements.

2. Estimate the maximum water surface elevation and calculate the maximum flow through the

riser and barrel system at this elevation (refer to the stage-storage-discharge table). Subtract this flow volume from the ***design peak rate of inflow*** to determine the desired maximum spillway design discharge.

3. Determine the crest elevation of the emergency spillway. This is usually a small increment (0.1 feet) above the design high water elevation of the next smaller storm, typically the 10-year frequency storm.

4. Enter **Table 5-12** with the maximum *Hp* value (maximum design water surface elevation from Step 2, less the crest elevation of the emergency spillway), and read across for the desired maximum spillway design discharge (from Step 2 above). Read the design bottom width of the emergency spillway (in feet) at the top of the table, and verify the minimum exit slope (s) and length (x), **or**;

   If a maximum bottom width (b) is known due to grading or topographic constraints, enter **Table 5-12** at the top with the desired bottom width and read down to find the desired discharge, Q, and then read across to the left to determine the required flow depth, *Hp*.

5. Add the appropriate *Hp* and discharge Q values to the stage-storage-discharge table.

Example Procedure 1:

Given: Q = *250 cfs* (determined from post-developed condition hydrology)
s_o = *4%* (slope of exit channel)
L = *50 ft.*(length of level section)

Find: Width of spillway, b, velocity, v, and depth of water above the spillway crest, H_p.

Solution: Complete Steps 1 through 5 of design **Procedure 1** for vegetated emergency spillways by using the given information as follows:

1. Peak rate of inflow: given Q = *250 cfs.*

2. The flow through the riser and barrel at the estimated maximum water surface elevation is calculated to be *163 cfs*. The desired maximum spillway design discharge is *250 cfs - 163 cfs = 87 cfs*, at a H_p value of *1.3 ft.*

3. Emergency spillway excavated into undisturbed material. The slope of the exit channel and length and elevation of level section: given, s_o = *4%, L = 50 ft.*, elevation = 100.0' (given).

4. Enter **Table 5-12** with the desired H_p value of *1.3 ft.* And read across to *86 cfs*, and read up to a bottom width of *24 ft*. at the top of the table. The minimum exit channel slope is *2.7%* which

is less than the *4%* provided, and the length of exit channel is required to be *63 ft*. The velocity within the exit channel is *4.7 ft/s* at an exit channel slope of *2.7%*. Since the provided exit channel slope is *4.0%*, erosive velocities may warrant special treatment of the exit channel.

5. Add the elevation corresponding to *1.3 ft*. above the crest of the emergency spillway to the Stage-Storage-Discharge Worksheet.

Procedure 2:

1. Determine the ***design peak rate of inflow*** from the spillway design flood into the basin, using the developed condition hydrology, **or** determine the ***allowable design peak release rate** , Q,* from the basin based on downstream conditions or watershed requirements.

2. Estimate the maximum water surface elevation and calculate the associated flow through the riser and barrel system for this elevation. Subtract this flow value from the ***design peak rate of inflow*** to determine the desired maximum spillway design discharge.

3. Position the emergency spillway on the basin grading plan at an embankment abutment.

4. Determine the slope, s_o , of the proposed exit channel, and the length, *L*, and elevation of the proposed level section from the basin grading plan.

5. Classify the natural soils around the spillway as ***erosion resistant*** or ***easily erodible*** soils.

6. Determine the type and height of vegetative cover to be used to stabilize the spillway.

7. Determine the permissible velocity, *v*, from **Table 3-03.1**, based on the vegetative cover, soil classification, and the slope of the exit channel, s_o .

8. Determine the retardance classification of the spillway based on the type and height of vegetative cover from **Table 3-03.2**.

9. Determine the unit discharge of the spillway, *q*, in *cfs/ft*, from **Table 5-13(a-d)** for the selected retardance, the maximum permissible velocity, *v*, and the slope of the exit channel, s_o .

10. Determine the required bottom width of the spillway, in *ft.*, by dividing the allowable or design discharge, *Q* , by the spillway unit discharge, *q*:

$$\frac{Q(cfs)}{q(cfs/ft)} \; ' \; ft.$$

11. Determine the depth of flow, H_p , upstream of the control section based on the length of the

level section, L, from **Table 5-13(a-d)**.

12. Enter the stage-discharge information into the stage-storage-discharge table.

The following examples use **Tables 3-03.1, 3-03.2** and **5-13** to find the capacity of a vegetated emergency spillway.

Example Procedure 2:

Given: $Q = 250\ cfs$ (determined from post-developed condition hydrology)
$s_o = 4\%$ (slope of exit channel)
$L = 50\ ft.$(length of level section)
Erosion resistant soils
Sod forming grass-legume mixture cover, 6 to 10-inch height

Find: Permissible velocity, v, width of spillway, b, depth of water above the spillway crest, H_p.

Solution: Complete Steps 1 through 12 of design **Procedure 2** for vegetated emergency spillways by using the given information as follows:

1. Peak rate of inflow: given $Q = 250\ cfs$.

2. The flow through the riser and barrel at the estimated maximum water surface elevation is calculated to be *163 cfs*. The desired maximum spillway design discharge is $250\ cfs - 163\ cfs = 87\ cfs$.

3. Emergency spillway excavated into undisturbed material.

4. Slope of exit channel, and length and elevation of level section: given, $s_o = 4\%$, $L = 50\ ft.$, elevation = 100.0 feet (given).

5. Soil classification: given, erosion resistant soils.

6. Vegetative cover: given, sod-forming grass-legume mixture.

7. Permissible velocity $v = 5\ ft/s$ from **Table 3-03.1** for sod-forming grass-legume mixtures, erosion resistant soils, and exit channel slope $s_o = 4\%$.

8. Retardance classification, C, from **Table 3.03.2** for sod-forming grass-legume mixtures, *expected height = 6 to 10 inches*.

9. The unit discharge of the spillway $q = 3\ cfs/ft$ from **Table 5-13c** for Retardance C, maximum permissible velocity $v = 5\ ft/s$, and exit channel slope $s_o = 4\%$.

10. The required bottom width $b = Q \ ' \ q = 87 \ cfs/ \ 3 \ \ cfs/ft = 29 \ ft$.

11. The depth of flow, $H_{p,}$ from **Table 5-13c** for Retardance C ; enter at $q = 3 \ cfs/ft$, find $H_p = 1.4 \ ft.$ for level section $L = 50 \ ft.$

12. The stage-discharge relationship: at stage elevation 1.4 feet above the spillway crest (*101.4'*), the discharge is 87 *cfs*.

Example Procedure 2:

Given: $Q = 175 \ cfs$ (determined from post-developed hydrology)
$s_o = 8 \ \%$ (slope of exit channel)
$L = 25 \ ft$. (length of level section)
Easily erodible soil
Bahiagrass, good stand, 11 to 24 inches expected

Find: Permissible velocity, v, width of spillway, b, depth of water above the spillway crest, H_p.
Analyze the spillway for stability during the vegetation establishment period, and capacity once adequate vegetation is achieved.

Solution: Complete Steps 1 through 12 of the design **Procedure 2** for vegetated emergency spillways by using the given information as follows:

1. $Q = 175 \ cfs.$

2. The flow through the riser and barrel at the estimated maximum water surface elevation is calculated to be *75 cfs*. The desired maximum spillway design discharge is $175 \ cfs - 75 \ cfs = 100 \ cfs$.

3. Emergency spillway in undisturbed ground.

4. $s_o = 8 \ \%; \quad L = 25 \ ft.$, elevation = *418.0 feet* (given)

5. Easily erodible soils.

6. Bahiagrass, good stand, 11 to 24 inches expected.

7. Permissable velocity, $v = 5 \ ft/s,$ from **Table 3-03.1**.

8. a) Retardance used for **stability** during the establishment period - good stand of vegetation 2 to 6 inches; Retardance D**.**

   b) Retardance used for **capacity** - good stand of vegetation 11 to 24 inches; Retardance B**.**

9. Unit discharge $q = 2\ cfs/ft$ for stability. From **Table 5-13d** for Retardance D, permissable velocity, $v = 5\ ft/s.$, and $s_o = 8\%$

10. Bottom width $b = Q/\ q = 100\ cfs/\ 2\ cfs/\ ft = 50\ ft.$ (stability)

11. The depth of flow, H_p, for capacity. From **Table 5-13b** for Retardance B, enter at $q = 2\ cfs/ft$, find $H_p = 1.4\ ft.$ for $L = 25\ ft$.

12. The stage-discharge relationship: at stage (elevation) 1.4 *ft.* above the spillway crest (*419.4'*), the discharge, Q, is 100 *cfs*.

TABLE 5-12
Design Data for Earth Spillways

STAGE (Hp) IN FEET	SPILLWAY VARIABLES	BOTTOM WIDTH (b) IN FEET																
		8	10	12	14	16	18	20	22	24	26	28	30	32	34	36	38	40
0.5	Q	6	7	8	10	11	13	14	15	17	18	20	21	22	24	25	27	28
	V	2.7	2.7	2.7	2.7	2.7	2.7	2.7	2.7	2.7	2.7	2.7	2.7	2.7	2.7	2.7	2.7	2.7
	S	3.9	3.9	3.9	3.9	3.8	3.8	3.8	3.8	3.8	3.8	3.8	3.8	3.8	3.8	3.8	3.8	3.8
	X	32	33	33	33	33	33	33	33	33	33	33	33	33	33	33	33	33
0.6	Q	8	10	12	14	16	18	20	22	24	26	28	30	32	34	35	37	39
	V	3.0	3.0	3.0	3.0	3.0	3.0	3.0	3.0	3.0	3.0	3.0	3.0	3.0	3.0	3.0	3.0	3.0
	S	3.7	3.7	3.7	3.7	3.6	3.7	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6
	X	36	36	36	36	36	36	37	37	37	37	37	37	37	37	37	37	37
0.7	Q	11	13	16	18	20	23	25	28	30	33	35	38	41	43	44	46	48
	V	3.2	3.2	3.3	3.3	3.3	3.3	3.3	3.3	3.3	3.3	3.3	3.3	3.3	3.3	3.3	3.3	3.3
	S	3.5	3.5	3.4	3.4	3.4	3.4	3.4	3.4	3.4	3.4	3.4	3.4	3.4	3.4	3.4	3.4	3.4
	X	39	40	40	40	41	41	41	41	41	41	41	41	41	41	41	41	41
0.8	Q	13	16	19	22	26	29	32	35	38	42	45	46	48	51	54	57	60
	V	3.5	3.5	3.5	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6
	S	3.3	3.3	3.3	3.2	3.2	3.2	3.2	3.2	3.2	3.2	3.2	3.2	3.2	3.2	3.2	3.2	3.2
	X	44	44	44	44	45	45	45	45	45	45	45	45	45	45	45	45	45
0.9	Q	17	20	24	28	32	35	39	43	47	51	53	57	60	64	68	71	75
	V	3.7	3.8	3.8	3.8	3.8	3.8	3.8	3.8	3.8	3.8	3.8	3.8	3.8	3.8	3.8	3.8	3.8
	S	3.2	3.1	3.1	3.1	3.1	3.1	3.1	3.1	3.1	3.1	3.1	3.1	3.1	3.1	3.1	3.1	3.1
	X	47	47	48	48	48	48	48	48	48	48	49	49	49	49	49	49	49
1.0	Q	20	24	29	33	38	42	47	51	56	61	63	68	72	77	81	86	90
	V	4.0	4.0	4.0	4.0	4.0	4.0	4.0	4.0	4.0	4.0	4.0	4.0	4.0	4.0	4.0	4.0	4.0
	S	3.1	3.0	3.0	3.0	3.0	3.0	3.0	3.0	3.0	3.0	3.0	3.0	3.0	3.0	3.0	3.0	3.0
	X	51	51	51	51	52	52	52	52	52	52	52	52	52	52	52	52	52
1.1	Q	23	28	34	39	44	49	54	60	65	70	74	79	84	89	95	100	105
	V	4.2	4.2	4.2	4.3	4.3	4.3	4.3	4.3	4.3	4.3	4.3	4.3	4.3	4.3	4.3	4.3	4.3
	S	2.9	2.9	2.9	2.9	2.9	2.9	2.9	2.9	2.9	2.8	2.8	2.8	2.8	2.8	2.8	2.8	2.8
	X	55	55	55	55	55	55	55	56	56	56	56	56	56	56	56	56	56
1.2	Q	28	33	40	45	51	58	64	69	76	80	86	92	98	104	110	116	122
	V	4.4	4.4	4.4	4.4	4.4	4.5	4.5	4.5	4.5	4.5	4.5	4.5	4.5	4.5	4.5	4.5	4.5
	S	2.9	2.9	2.8	2.8	2.8	2.8	2.8	2.8	2.8	2.8	2.8	2.8	2.8	2.8	2.8	2.8	2.8
	X	58	58	59	59	59	59	59	59	60	60	60	60	60	60	60	60	60
1.3	Q	32	38	46	53	58	65	73	80	86	91	99	106	112	119	125	133	140
	V	4.5	4.6	4.6	4.6	4.6	4.6	4.7	4.7	4.7	4.7	4.7	4.7	4.7	4.7	4.7	4.7	4.7
	S	2.8	2.8	2.8	2.7	2.7	2.7	2.7	2.7	2.7	2.7	2.7	2.7	2.7	2.7	2.7	2.7	2.7
	X	62	62	62	63	63	63	63	63	63	63	63	64	64	64	64	64	64
1.4	Q	37	44	51	59	66	74	82	90	96	103	111	119	127	134	142	150	158
	V	4.7	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.9	4.9	4.9	4.9	4.9	4.9	4.9	4.9
	S	2.8	2.7	2.7	2.7	2.7	2.7	2.7	2.6	2.6	2.6	2.6	2.6	2.6	2.6	2.6	2.6	2.6
	X	65	66	66	66	66	67	67	67	67	67	67	68	68	68	68	68	69
1.5	Q	41	50	58	66	75	85	92	101	108	116	125	133	142	150	160	169	178
	V	4.8	4.9	4.9	5.0	5.0	5.0	5.0	5.0	5.0	5.0	5.0	5.0	5.0	5.0	5.1	5.1	5.1
	S	2.7	2.7	2.6	2.6	2.6	2.6	2.6	2.6	2.6	2.6	2.6	2.6	2.6	2.6	2.5	2.5	2.5
	X	69	69	70	70	71	71	71	71	71	71	71	72	72	72	72	72	72
1.6	Q	46	56	65	75	84	94	104	112	122	132	142	149	158	168	178	187	197
	V	5.0	5.1	5.1	5.1	5.1	5.2	5.2	5.2	5.2	5.2	5.2	5.2	5.2	5.2	5.2	5.2	5.2
	S	2.6	2.6	2.6	2.6	2.5	2.5	2.5	2.5	2.5	2.5	2.5	2.5	2.5	2.5	2.5	2.5	2.5
	X	72	74	74	75	75	76	76	76	76	76	76	76	76	76	76	76	76
1.7	Q	52	62	72	83	94	105	115	126	135	145	156	167	175	187	196	206	217
	V	5.2	5.2	5.2	5.3	5.3	5.3	5.3	5.4	5.4	5.4	5.4	5.4	5.4	5.4	5.4	5.4	5.4
	S	2.6	2.6	2.5	2.5	2.5	2.5	2.5	2.5	2.5	2.5	2.5	2.5	2.5	2.5	2.5	2.5	2.5
	X	76	78	79	80	80	80	80	80	80	80	80	80	80	80	80	80	80
1.8	Q	58	69	81	93	104	116	127	138	150	160	171	182	194	204	214	226	233
	V	5.3	5.4	5.4	5.5	5.5	5.5	5.5	5.5	5.5	5.5	5.5	5.6	5.6	5.6	5.6	5.6	5.6
	S	2.5	2.5	2.5	2.5	2.4	2.4	2.4	2.4	2.4	2.4	2.4	2.4	2.4	2.4	2.4	2.4	2.4
	X	80	82	83	84	84	84	84	84	84	84	84	84	84	84	84	84	84
1.9	Q	64	76	88	102	114	127	140	152	164	175	188	201	213	225	235	248	260
	V	5.5	5.5	5.5	5.6	5.6	5.6	5.7	5.7	5.7	5.7	5.7	5.7	5.7	5.7	5.7	5.7	5.7
	S	2.5	2.5	2.5	2.4	2.4	2.4	2.4	2.4	2.4	2.4	2.4	2.4	2.4	2.4	2.4	2.4	2.4
	X	84	85	86	87	88	88	88	88	88	88	88	88	88	88	88	88	88
2.0	Q	71	83	97	111	125	138	153	164	178	193	204	218	232	245	256	269	283
	V	5.6	5.7	5.7	5.7	5.8	5.8	5.8	5.8	5.8	5.8	5.8	5.9	5.9	5.9	5.9	5.9	5.9
	S	2.5	2.4	2.4	2.4	2.4	2.4	2.4	2.4	2.3	2.3	2.3	2.3	2.3	2.3	2.3	2.3	2.3
	X	88	90	91	91	91	91	92	92	92	92	92	92	92	92	92	92	92
2.1	Q	77	91	107	122	135	149	162	177	192	207	220	234	250	267	276	291	305
	V	5.7	5.8	5.9	5.9	5.9	5.9	5.9	6.0	6.0	6.0	6.0	6.0	6.0	6.0	6.0	6.0	6.0
	S	2.4	2.4	2.4	2.4	2.4	2.3	2.3	2.3	2.3	2.3	2.3	2.3	2.3	2.3	2.3	2.3	2.3
	X	92	93	95	95	95	95	95	95	95	96	96	96	96	96	96	96	96
2.2	Q	84	100	116	131	146	163	177	194	210	224	238	253	269	288	301	314	330
	V	5.9	5.9	6.0	6.0	6.0	6.1	6.1	6.1	6.1	6.1	6.1	6.1	6.1	6.2	6.2	6.2	6.2
	S	2.4	2.4	2.4	2.3	2.3	2.3	2.3	2.3	2.3	2.3	2.3	2.3	2.3	2.3	2.3	2.3	2.3
	X	96	98	99	99	99	99	99	100	100	100	100	100	100	100	100	100	100
2.3	Q	90	108	124	140	158	175	193	208	226	243	258	275	292	306	323	341	354
	V	6.0	6.1	6.1	6.1	6.2	6.2	6.2	6.2	6.3	6.3	6.3	6.3	6.3	6.3	6.3	6.3	6.3
	S	2.4	2.4	2.3	2.3	2.3	2.3	2.3	2.3	2.2	2.2	2.2	2.2	2.2	2.2	2.2	2.2	2.2
	X	100	102	102	103	103	103	104	104	104	105	105	105	105	105	105	105	105
2.4	Q	99	116	136	152	170	189	206	224	241	260	275	294	312	327	346	364	378
	V	6.1	6.2	6.2	6.3	6.3	6.3	6.4	6.4	6.4	6.4	6.4	6.4	6.4	6.4	6.4	6.4	6.4
	S	2.3	2.3	2.3	2.3	2.3	2.2	2.2	2.2	2.2	2.2	2.2	2.2	2.2	2.2	2.2	2.2	2.2
	X	105	105	106	107	107	108	108	108	108	109	109	109	109	109	109	109	109

Source: USDA - SCS

TABLE 5 - 13a

H_p and Slope Range for Discharge, Velocity and Crest Length - Retardance A

Max. Velocity, *v* (*ft/s*)	**Unit Discharge, *q* (*cfs/ft*)**	**Depth of Water Above Spillway Crest, H_p (*ft.*) — Length of Level Section, *L* (*ft.*)**				**Slope Range, s_o %**	
		25	**50**	**100**	**200**	**Min.**	**Max.**
3	3	2.3	2.5	2.7	3.1	1	11
4	4	2.3	2.5	2.8	3.1	1	12
4	5	2.5	2.6	2.9	3.2	1	7
5	6	2.6	2.7	3.0	3.3	1	9
6	7	2.7	2.8	3.1	3.5	1	12
7	10	3.0	3.2	3.4	3.8	1	9
8	12.5	3.3	3.5	3.7	4.1	1	10

Source: SCS Engineering Field Manual

TABLE 5 - 13b

H_p and Slope Range for Discharge, Velocity, and Crest Length - Retardance B

Max. Velocity, *v* (*ft/s*)	Unit Discharge, *q* (*cfs/ft*)	Depth of Water Above Spillway Crest, H_p (*ft.*) / Length of Level Section, *L* (*ft.*)				Slope Range, s_o %	
		25	50	100	200	Min.	Max.
2	1	1.2	1.4	1.5	1.8	1	12
2	1.25	1.3	1.4	1.6	1.9	1	7
3	1.5	1.3	1.5	1.7	1.9	1	12
3	2	1.4	1.5	1.7	1.9	1	8
4	3	1.6	1.7	1.9	2.2	1	9
5	4	1.8	1.9	2.1	2.4	1	8
6	5	1.9	2.1	2.3	2.5	1	10
7	6	2.1	2.2	2.4	2.7	1	11
8	7	2.2	2.4	2.6	2.9	1	12

Source: SCS Engineering Field Manual

TABLE 5 - 13c

H_p and Slope Range for Discharge, Velocity, and Crest Length - Retardance C

Max. Velocity, *v* (*ft/s*)	Unit Discharge, *q* (*cfs/ft*)	Depth of WaterAbove Spillway Crest, H_p (*ft.*) / Length of Level Section, *L* (*ft.*)				Slope Range, s_o %	
		25	50	100	200	Min.	Max.
2	0.5	0.7	0.8	0.9	1.1	1	6
2	1	0.9	1.0	1.2	1.3	1	3
3	1.25	0.9	1.0	1.2	1.3	1	6
4	1.5	1.0	1.1	1.2	1.4	1	12
4	2	1.1	1.2	1.4	1.6	1	7
5	3	1.3	1.4	1.6	1.8	1	6
6	4	1.5	1.6	1.8	2.0	1	12
8	5	1.7	1.8	2.0	2.2	1	12
9	6	1.8	2.0	2.1	2.4	1	12
9	7	2.0	2.1	2.3	2.5	1	10
10	7.5	2.1	2.2	2.4	2.6		12

Source: SCS Engineering Field Manual

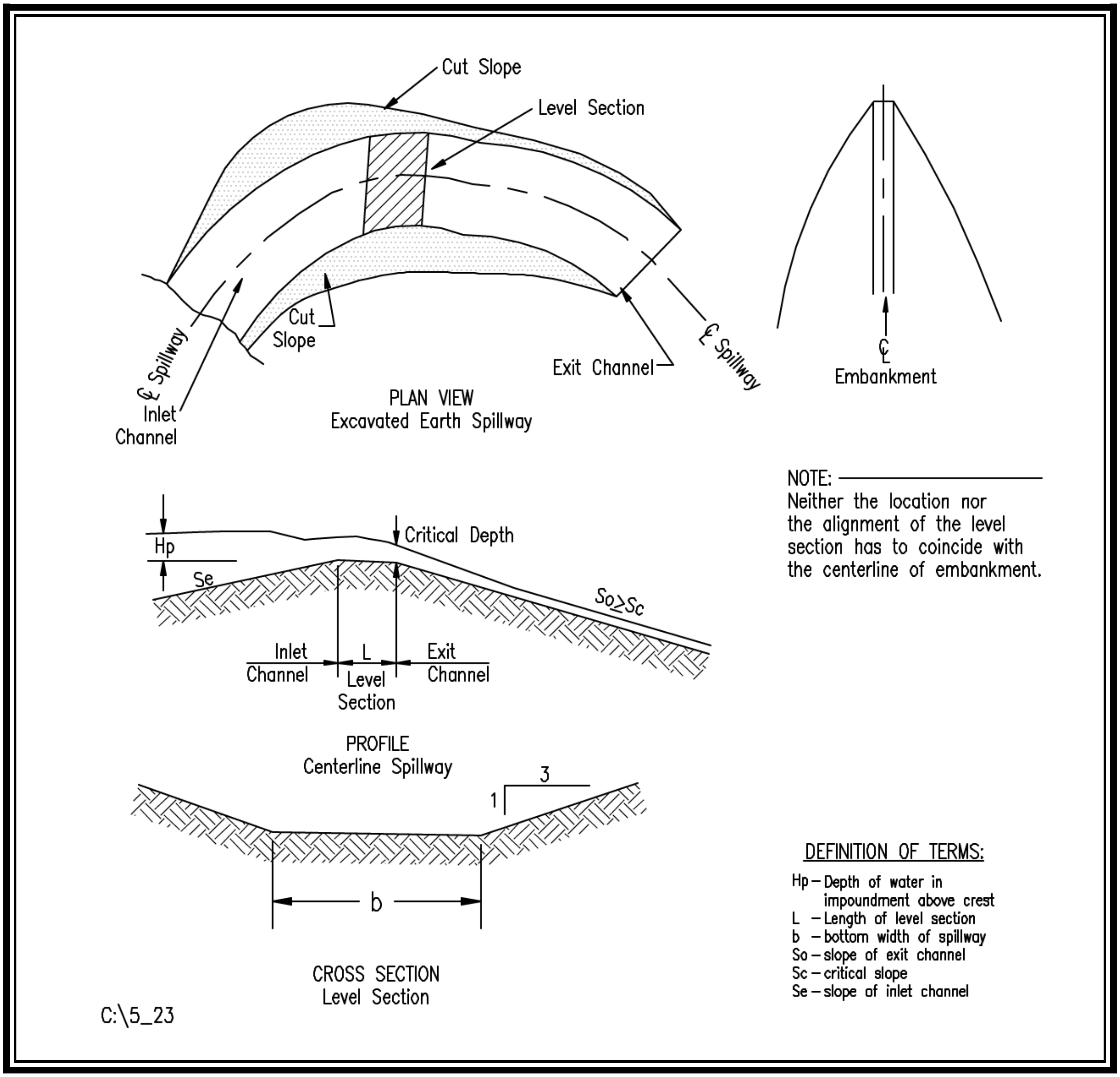
TABLE 5 - 13d

H_p and Slope Range for Discharge, Velocity, and Crest Length, Retardance D

Max. Velocity, v (ft/s)	Unit Discharge, q (cfs/ft)	Depth of Water Above Spillway Crest, H_p (ft.) / Length of Level Section, L (ft.)				Slope Range, s_o %	
		25	50	100	200	Min.	Max.
2	0.5	0.6	0.7	0.8	0.9	1	6
3	1	.8	.9	1.0	1.1	1	6
3	1.25	.8	.9	1.0	1.2	1	4
4	1.5	.8	.9	1.0	1.2	1	10
4	2	1.0	1.1	1.3	1.4	1	4
5	1.5	.9	1.0	1.2	1.3	1	12
5	2	1.0	1.2	1.3	1.4	1	9
5	3	1.2	1.3	1.5	1.7	1	4
6	2.5	1.1	1.2	1.4	1.5	1	11
6	3	1.2	1.3	1.5	1.7	1	7
7	3	1.2	1.3	1.5	1.7	1	12
7	4	1.4	1.5	1.7	1.9	1	7
8	4	1.4	1.5	1.7	1.9	1	12
8	5	1.6	1.7	1.9	2.0	1	8
10	6	1.8	1.9	2.0	2.2	1	12

Source: SCS Engineering Field Manual

FIGURE 5 - 23
Vegetated Emergency Spillways: Typical Plan and Section



5-9 HYDROGRAPH ROUTING

This section presents the methodology for routing a runoff hydrograph through an existing or proposed stormwater basin. The "level pool" or storage indication routing technique is one of the simplest and most commonly used methods, and is based on the continuity equation:

$$I \quad - \quad O \quad = \quad ds / dt$$

Inflow - Outflow = Change in Storage over time

The goal of the routing process is to create an outflow hydrograph that is the result of the combined effects of the outlet device and the available storage. This will allow the designer to evaluate the performance of the outlet device or the basin storage volume, or both. When multiple iterations are required to create the most efficient basin shape, the routing procedure can be time consuming and cumbersome, especially when done by hand using the methods presented in this section. It should be noted that several computer programs are available to help complete the routing procedure.

A step-by-step procedure for routing a runoff hydrograph through a stormwater basin is given below. Note that the first four steps are part of the multi-stage riser design of the previous section. Due to the complexity of this procedure, **Example 1** from **Chapter 6** will be used. Note that the water quality volume is **not** considered and only one design storm will be routed, the 2-year storm. Other design frequency storms can be easily analyzed with the same procedure. Blank worksheets for this procedure are provided in **Appendix 5D**.

Procedure:

1. Generate a post-developed condition inflow hydrograph. The runoff hydrograph for the 2-year frequency storm, post-developed condition from **Example 1**, as calculated by the SCS TR-20 computer program and shown in **Figure 5-1**, will be used for the inflow hydrograph. (Refer to **Chapter 6** for details on the hydrology from **Example 1**. Refer to **Chapter 4** for information on the hydrologic methods used.)

2. Develop the stage-storage relationship for the proposed basin. The hydrologic calculations and the hydrograph analysis for **Example 1**, in **Section 5-3** and **Section 5-4.1**, revealed that the storage volume required to reduce the 2-year, post-developed peak discharge back to the pre-developed rate was 35,820 ft^3. Therefore, a preliminary grading plan should have a stormwater basin with this required storage volume, as a minimum, to control the 2-year frequency storm. The stage-storage relationship of the proposed stormwater facility can be generated by following the procedures outlined in **Section 5-5**. **Figure 5-10** shows the completed Storage Volume Calculations Worksheet, and **Figure 5-11** shows the stage vs. storage curve.

3. Size the outlet device for the design frequency storm and generate the stage-discharge relationship. An outlet device or structure must be selected to define the stage-discharge relationship. This procedure is covered in **Section 5-7, STEP 6** of the multi-stage riser design. Using the procedure within **STEP 6** from **Section 5-7** and **Example 1**, the procedure is as follows (from **STEP 6, Section 5-7**):

   1. Approximate the 2-year maximum head, $h_{2_{max}}$.

   Enter the stage-storage curve, **Figure 5-11**, with the 2-year required storage: 35,820 ft^3 and read the corresponding elevation: 88.5 *ft*. Then, $h_{2_{max}}$ *= 88.5 ft. - 81.0 ft. (bottom of basin) = 7.5 ft*. Note that this is an approximation because it ignores the centerline of the orifice as the point from which the head is measured. The head values can be adjusted when the orifice size is selected.

   2. Determine the maximum allowable 2-year discharge rate, $Q_{2_{allowable}}$.

   From the pre-developed hydrologic analysis, the 2-year allowable discharge from the basin was found to be 8.0 *cfs*. (This assumes that watershed conditions or local ordinance limit the developed rate of runoff to be # the pre-developed rate.)

   3. Calculate the size of the 2-year controlled release orifice.

   Solve for the area, *a*, in ft^2 by inserting the allowable discharge *Q = 8.0 cfs* and $h_{2_{max}}$ *= 7.5 ft.* into the **Rearranged Orifice Equation**, **Equation 5-7**. This results in an orifice diameter of 10 inches.

$$a = \frac{Q}{C\sqrt{2gh}}$$

Equation 5-7
Rearranged Orifice Equation

Where:
- a = *required orifice area,* ft^2
- Q = *maximum allowable discharge = 8.0 cfs*
- C = *orifice coefficient = 0.6*
- g = *gravitational acceleration = 32.2 ft/sec*
- h = *maximum 2-year hydraulic head,* $h_{2_{max}}$ *= 7.5 ft.*

$$a = \frac{8.0}{0.6\sqrt{(2)(32)(7.5)}}$$

$$a = 0.61\ ft^2$$

For orifice diameter:

$$a = 0.61\ ft^2 = \pi \left(\frac{d}{2}\right)^2$$

$$d = 0.88\ ft. = 10.6\ inches$$

Use a 10-inch diameter orifice.

4. Develop the stage-storage-discharge relationship for the 2-year storm.

   Substituting the 10-inch orifice size into the **Orifice Equation**, **Equation 5-6**, and solving for the discharge, *Q*, at various stages provides the information needed to plot the stage vs. discharge curve and complete the Stage-Storage-Discharge Worksheet.

$$Q = C_o a\sqrt{2gh}$$

Equation 5-6
Orifice Equation

Where: $a = a_{10''} = 0.545\ ft^2$

$$Q = (0.6)(0.545)\sqrt{(2)(32.2)(h)}$$

$$Q_2 = 2.62\ (h)^{0.5}$$

Where: h = *water surface elev. - (81.0 + 0.83/2)*
h = *water surface elev. - 81.4*

Note that the *h* is measured to the centerline of the 10-inch orifice.

Figure 5-24 shows the result of the calculations: the stage vs. discharge curve and table.

Continuing with the Hydrograph Routing Procedure:

5. Develop the relationship *2S/Δt vs. O* and plot *2S/Δt vs. O.*

The plot of the curve *2S/Δt vs. 0* is derived from the continuity equation. The continuity equation is rewritten as:

$$\frac{I_n + I_{n+1}}{2} - \frac{O_n + O_{n+1}}{2} = \frac{S_{n+1} - S_n}{\Delta t}$$

Equation 5-12
Continuity Equation

where: I_n & I_{n+1} = *inflow at time n=1 and time n=2*
O_n & O_{n+1} = *outflow at time n=1 and time n=2*
S_n & S_{n+1} = *storage at time n=1 and time n=2*
Δt = *time interval (n=2 - n=1)*

This equation describes the *change in storage over time* as the difference between the average inflow and outflow at that given time. Multiplying both sides of the equation by 2 and rearranging allows the equation to be re-written as:

$$I_n + I_{n+1} + \left(\frac{2S_n}{\Delta t} - O_n\right) = \frac{2S_{n+1}}{\Delta t} + O_{n+1}$$

Equation 5-13
Rearranged Continuity Equation

The terms on the left-hand side of the equation are known from the inflow hydrograph and from the storage and outflow values of the previous time interval. The unknowns on the right hand side, O_{n+1} and S_{n+1} , can be solved interactively from the previously determined stage vs. storage curve, **Figure 5-11**, and stage vs. discharge curve, **Figure 5-24**.

First, however, the relationship between *2S/Δt + O* and *O* must be developed. This relationship can best be developed by using the stage vs. storage and stage vs. discharge curves to fill out the worksheet shown in **Figure 5-25**, as follows:

a) Columns 1, 2, and 3 are completed using the stage vs. discharge curve.

b) Columns 4 and 5 are completed using the stage vs. storage curve.

c) Column 6 is completed by determining the time step increment used in the inflow hydrograph. (For Example 1, $\Delta t = 1\ hr. = 3{,}600\ sec.$) Δt is in seconds to create units of cubic feet per second (*cfs*) for the $2S/\Delta t$ calculation.

d) Column 7 is completed by adding Columns 3 and 6. The completed table is presented in **Figure 5-26**, and **Example 1** in **Chapter 6**, along with the plotted values from Column 3, *O* or outflow, and Column 7, $2S/\Delta t + O$.

6. Route the inflow hydrograph through the basin and 10-inch diameter orifice. The routing procedure is accomplished by use of another worksheet, **Figure 5-27**, Hydrograph Routing Worksheet. Note that as the work is completed for each value of *n,* it becomes necessary to jump to the next row for a value. The table is completed by the following steps:

   a. Complete Column 2 and Column 3 for each time *n*. These values are taken from the inflow hydrograph. The inflow hydrograph is provided in tabular form in **Figure 5-29**. This information is either taken from the plot of the inflow hydrograph or read directly from the tabular version of the inflow hydrograph (TR-20, TR-55, etc.).

   b. Complete Column 4 for each time *n* by adding two successive inflow values from Column 3. Therefore, $Column\ 4_n = Column\ 3_n + Column\ 3_{n+1}$.

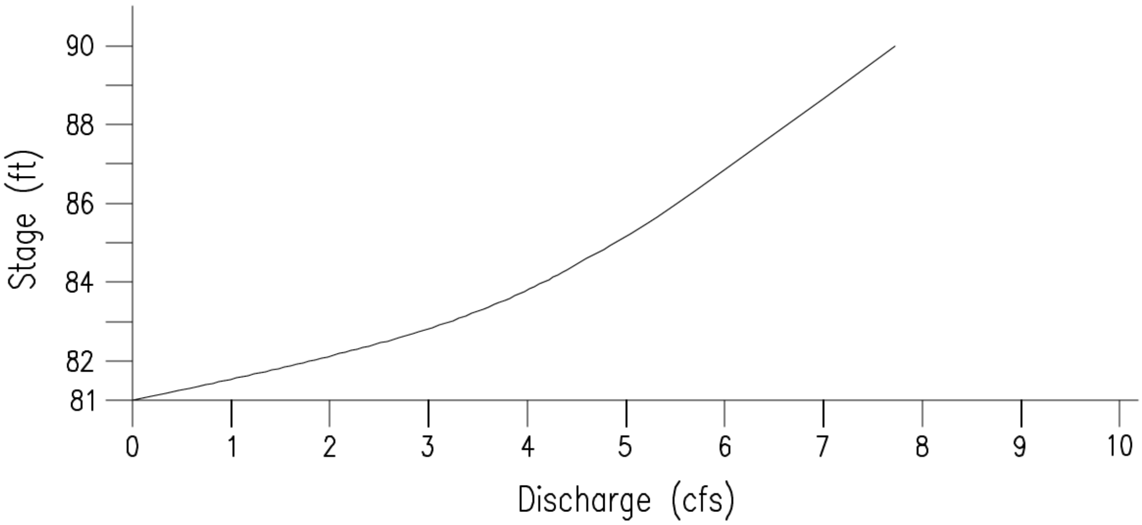
   c. Compute the values in Column 6 by adding Columns 4 and 5 from the previous time step. Note that for $n = 0$, Columns 5, 6, and 7 are given a value of zero before starting the table. Therefore, $Column\ 6_{n=2} = Column\ 4_{n=1} + Column\ 5_{n=1}$. (Note that this works down the table and not straight across.)

   d. Column 7 is read from the $2S/\Delta t + O$ *vs.* O curve by entering the curve with the value from Column 6 to obtain the outflow, *O*.

   e. Now backtrack to fill Column 5 by subtracting twice the value of Column 7 (from step d) from the value in Column 6. $Column\ 5_n = Column\ 6_n - 2(Column\ 7_n)$.

   f. Repeat steps c through e until the discharge (*O*, Column 7) reaches zero.

FIGURE 5 - 24
Stage vs. Discharge Curve, Example 1



Stage	(h)	(Q)
81.4	0	0
82	0.6	2.0
84	2.6	4.2
86	4.6	5.6
88	6.6	6.7
90	8.6	7.7

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FIGURE 5 - 25
Storage Indication Hydrograph Routing (2S/Δt + O) vs. O Worksheet

1	2	3	4	5	6	7
elevation (ft)	***stage (ft)***	***outflow (cfs)***	***storage (cf)***	***2S (cf)***	***2S/Δt (cfs)***	***2S/Δt + O (cfs)***
from plan	$elev_n$ - $elev_o$	*based on outflow device & stage*	*based on stage*	*2 × Col 4*	*Col 5 /Δt of hydrograph*	*Col 3 + Col 6*

FIGURE 5 - 26
Storage Indication Hydrograph Routing (2S/Δt + O) vs. O Worksheet, Example 1, Curve & Table

1	2	3	4	5	6	7
Elev	Stage	Outflow (0 cfs)	Storage (S cf)	2S (cf)	2S/Δt (cfs)	2S/Δt + 0
81	0	0	0	0	0	0
8	0.6	2.0	900	1800	0.50	2.5
84	2.6	4.2	5940	11,880	3.3	7.5
86	4.6	5.6	14,354	28,708	7.97	13.6
88	6.6	6.7	29,582	59,164	16.4	23.1
90	8.6	7.7	55,564	111,128	30.9	38.6

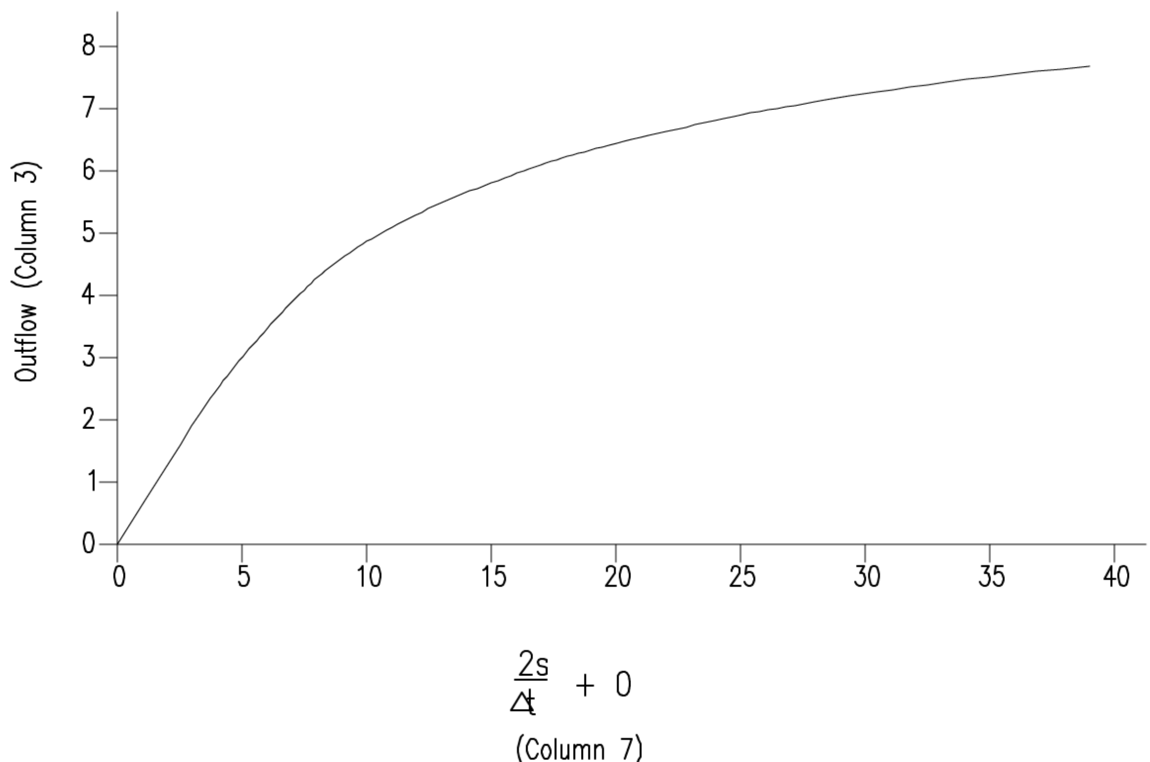

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FIGURE 5 - 27
Storage Indication Hydrograph Routing Worksheet

1	2	3	4	5	6	7
n	*Time (min)*	I_n (*cfs*)	$I_n + I_{n+1}$ (*cfs*)	$2S_n/\Delta t - O_n$ (*cfs*)	$2S_{n+1}/\Delta t + O_{n+1}$ (*cfs*)	O_{n+1} (*cfs*)
	from hydrograph		*Col* 3_n +*Col* 3_{n+1}	*Col* 6_n - 2(*Col* 7_n)	*Col* 4_{n-1} + *Col* 5_{n-1}	*from chart; use Col* 6_n
0				0	0	0

The above steps are repeated here for the first four time steps in **Example 1** and displayed in the completed Hydrograph Routing Worksheet, **Figure 5-28**.

1. Columns 2 and 3 are completed for each time step using the inflow hydrograph.

2. Column 4 is completed as follows:

   Column 4_n = *Column* 3_n + *Column* 3_{n+1}
   for n = 1: Column $4_{n=1}$ = *Column* $3_{n=1}$ + *Column* $3_{n=2}$
   Column $4_{n=1} = 0 + 0.32 = 0.32$
   for n = 2: Column $4_{n=2} = 0.3 + 23.9 = 24.2$
   for n = 3: Column $4_{n=3} = 23.9 + 4.6 = 28.5$
   for n = 4: Column $4_{n=4} = 4.6 + 2.4 = 7.0$
   etc.

n = 1

3. *Column* $6_{n=1} = 0$. $n = 1$ is at time 0. The first time step has a value of zero.

4. *Column* $7_{n=1} = 0$. Entering the $2S/\Delta t$ *vs.* O curve with a value of zero gives $O = 0$ cfs. (The discharge is always zero at time $t=0$ unless a base flow exists.)

5. *Column* $5_{n=1}$ = *Column* $6_{n=1}$ - 2 (*Column* $7_{n=1}$) *Column* $5_{n=1} = 0 - 0 = 0$.

n = 2

3. *Column* $6_{n=2}$ *= Column* $4_{n=1}$ *+ Column* $5_{n=1}$*.*
   Column $6_{n=2}$ *= 0.3 + 0 = 0.3.*

4. *Column* $7_{n=2}$ *= 0.3.* Enter the $2S/\Delta t + O$ *vs.* O curve with $2S/\Delta t + O = 0.3$ (from Column 6) and read $O = 0.3$.

5. *Column* $5_{n=2}$ *= Column* $6_{n=2}$ *- 2(Column* $7_{n=2}$*).*
   Column $5_{n=2}$ *= 0.3 - 2(0.3) = -0.3 = 0.* (A negative outflow is unacceptable.)

n = 3

3. *Column* $6_{n=3}$ *= 24.2 + 0 = 24.2.*

4. *Column* $7_{n=3}$ *= 6.8.* Enter $2S/\Delta t + O$ *vs.* O curve with 24.2, read $O = 6.8$.

5. *Column* $5_{n=3}$ *= 24.2 - 2(6.8) = 10.6.*

n = 4

3. *Column* $6_{n=4}$ *= 28.5 + 10.6 = 39.1*

4. *Column* $7_{n=4}$ *= 7.7.* Enter $2S/\Delta t = O$ *vs.* O curve with 39.1, read $O = 7.7$.

5. *Column* $5_{n=4}$ *= 39.1 - 2(7.7) =23.7.*

n = 5, etc.

This process is continued until the discharge (O, Column 7) equals “0”. The values in Column 7 can then be plotted to show the *outflow rating curve*, or *discharge hydrograph*, as shown in **Figure 5-29**. **The designer should verify that the maximum discharge from the basin is less than the allowable release**. If the maximum discharge is greater than or much less than the allowable discharge, the designer should try a different outlet size or basin shape.

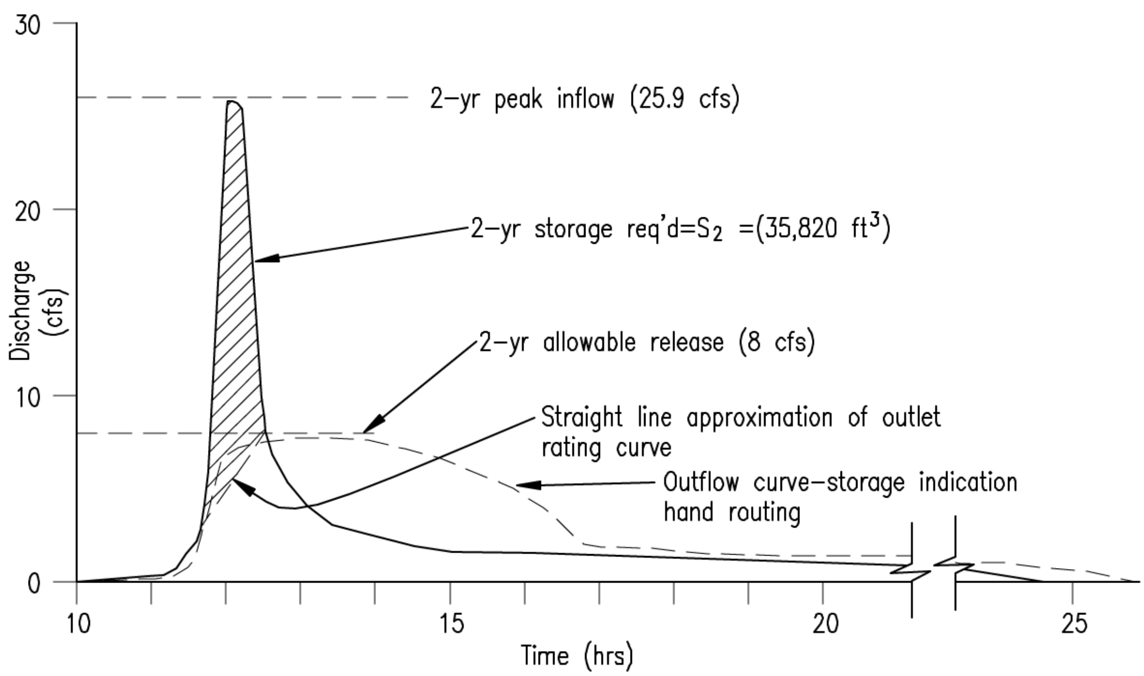
FIGURE 5 - 28
Storage Indication Hydrograph Routing Worksheet, Example 1

1	2	3	4	5	6	7
n	*Time (min)*	I_n *(cfs)*	$I_n + I_{n+1}$ *(cfs)*	$2S_n/\Delta t - O_n$ *(cfs)*	$2S_{n+1}/\Delta t + O_{n+1}$ *(cfs)*	O_{n+1} *(cfs)*
	from hydrograph		*Col* 3_n + *Col* 3_{n+1}	*Col* 6_n - 2(*Col* 7_n)	*Col* 4_{n-1} + *Col* 5_{n-1}	*from chart; use Col* 6_n
1	0	0	0.32	0	0	0
2	60	0.32	24.2	0 (-0.3)	0.3	0.3
3	120	23.9	28.5	10.6	24.2	6.8
4	180	4.6	7.0	23.7	39.1	7.7
5	240	2.4	4.0	15.7	30.7	7.5
6	300	1.6	3.0	6.9	19.7	6.4
7	360	1.4	2.6	0.3	9.9	4.8
8	420	1.2	2.3	0 (-0.7)	2.9	1.8
9	480	1.1	2.1	0 (-0.7)	2.3	1.5
10	540	1.0	1.9	0 (-0.7)	2.1	1.4
11	600	0.9	1.6	0 (-0.7)	1.9	1.3
12	660	0.7	1.4	0 (-0.6)	1.6	1.1
13	720	0.7	1.4	0 (-0.6)	1.4	1.0
14	780	0.7	1.3	0 (-0.6)	1.4	1.0
15	840	0.6	0.6	0 (-0.5)	1.3	0.9
16	900	0	0	0 (-0.4)	0.6	0.5
17	960	0	0		0	0

FIGURE 5 - 29
Inflow and Discharge Hydrographs, Example 1

hr.	Qi	Qo	hr.	Qi	Qo	hr.	Qi	Qo
10	0	0	16	1.4	4.8	22	0.7	1.0
11	0.3	0.3	17	1.2	1.8	23	0.7	1.0
12	23.9	6.8	18	1.1	1.5	24	0.6	0.9
13	4.6	7.7	19	1.0	1.4	25	0.0	0.5
14	2.4	7.5	20	0.9	1.3	26	0	0
15	1.6	6.4	21	0.7	1.1			

Inflow and Outflow Hydrograph Values
Qi taken from TR-20 Hydrology Computer Run
Qo taken from Routing Worksheet – Fig. 5-28



Peak 25.9 cfs, 12.13 hrs

Calculations=
$(0.40\ in^2)(10\ cfs/in)(2.5\ hrs/in)(3600\ sec/hr)=35{,}820\ ft^3$
$=0.82\ ac\text{-}ft$

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5-10 WATER QUALITY CALCULATION PROCEDURES

This section presents procedures for complying with the water quality criterion outlined in the stormwater management regulations. The water quality criterion represent a consolidation of the requirements of three state agencies charged with the responsibility of monitoring and improving the water resources of the Commonwealth: The Department of Conservation and Recreation (DCR), the Department of Environmental Quality (DEQ), and the Chesapeake Bay Local Assistance Department (CBLAD). The specific responsibilities of these agencies are presented in **Chapter 1**.

The stormwater management water quality regulations require compliance by either a **performance-based water quality criteria** or a **technology-based water quality criteria**. The performance-based water quality criteria requires the designer to implement a Best Management Practice (BMP) or combination of BMPs which effectively remove the anticipated increase in pollutant load from a development site. This approach requires the designer to calculate the pollutant load to be removed, implement a BMP strategy, and then calculate the performance of that strategy, based on the effectiveness or pollutant removal efficiency of the selected BMP(s).

The technology-based water quality criteria simply states that for land uses of given amounts of impervious cover, measured in percent, there are best available technologies with which to remove the anticipated pollutant load increase.

These two criterion are considered to be equivalent when implemented as described in this handbook. A more detailed discussion of these water quality criterion and the selection of water quality BMPs is presented in **Chapter 2**.

5-10.1 Performance-Based Water Quality Criteria

This procedure is for determining compliance with the performance-based water quality criteria of the Commonwealth's stormwater management regulations. The **Performance-based water quality criteria** is defined as follows:

For land development, the calculated post-development nonpoint source pollutant runoff load shall be compared to the calculated pre-development load based upon the average land cover condition <u>or</u> the existing site condition. A BMP(s) shall be located, designed, and maintained to achieve the target pollutant removal efficiencies specified in **Table 5-14** and to effectively reduce the pollutant load to the required level based upon the four applicable land development situations for which the performance criteria apply. (Refer to **STEP 3** for a discussion of the development situations.)

The "nonpoint source pollutant runoff load" or "pollutant discharge" is defined as the average amount of a particular pollutant(s) measured in pounds per year, delivered in a diffuse manner by stormwater runoff. The calculation procedure described herein uses the contaminant **phosphorous** for the purposes of calculating pollutant discharge in order to determine compliance with the performance-based water quality criteria. **However, other pollutants may be targeted if**

determined to be more appropriate for the intended land use. Refer to **Chapter 2** for a discussion of urban nonpoint source pollution.

The accepted calculation procedure for the determining the pre- and post-developed pollutant loads from development sites is referred to as the Simple Method. A more detailed discussion and derivation of the Simple Method can be found in *Appendix A* of Controlling Urban Runoff: A Practical Manual for Planning and Designing Urban BMPs, published by the Metropolitan Washington Council of Governments. The simple method uses impervious cover as the key variable in calculating the levels of pollutant export. (It should be noted that other more data intensive methods for calculating pollutant loads are available. DCR will evaluate the option of utilizing these methods in the future.

Equation 5-14 presents the Simple Method General Pollutant Load Equation.

$$L = P \times P_j \times [0.05 + (0.009 \times I)] \times C \times A \times 2.72 \div 12$$

Equation 5-14
Simple Method Pollutant Load (L)

where:
- L = *relative total phosphorous load (pounds per year)*
- P = *average annual rainfall depth (inches), assumed to be 43 inches for Virginia**
- P_j = *unitless correction factor for storm with no runoff = 0.9*
- I = *percent impervious cover (percent expressed in whole numbers)*
- C = *flow-weighted mean pollutant concentration = 0.26 milligrams per liter*
- A = *applicable area (acres)*

Note: 12 and 2.72 are conversion factors

* - The annual rainfall depth may vary across the commonwealth based on locally collected rainfall data. The designer should verify actual rainfall values which may be required in the local jurisdiction. Also note that the use of the same value in the pre- and post-developed computations allows for the cancellation of this and other values as discussed below.

The purpose of this calculation is to provide a comparison between the pre- and post-development pollutant loads. Therefore, in an effort to simplify **Equation 5-14**, any value which will not change with the development of land, such as rainfall (P) and the flow weighted mean pollutant concentration (C), and any constants, such as the correction factor(P_j) and conversion factors, can be multiplied through. Thus **Equation 5-14** simplifies to:

$$L = [0.05 + (0.009 \times I)] \times A \times 2.28$$

Equation 5-15
Simple Method Pollutant Load (L), Simplified

where: L = *relative total phosphorous load (pounds per year)*
I = *percent impervious cover (percent expressed in whole numbers)*
A = *applicable area (acres)*

The Performance-based criteria requires that a pre- and post-developed condition pollutant load be calculated in order to determine the relative increase. A consistent, calculated pre-developed annual load (L_{pre}), or base annual load, with which to compare the calculated post-developed annual load (L_{post}) is therefore required. The Chesapeake Bay Local Assistance Department has determined a base line annual load of phosphorous for Tidewater Virginia and has established a corresponding baseline impervious value, or average land cover condition ($I_{watershed}$), of 16%. A locality may choose to adopt this value as the pre-developed default for the entire locality. **Or** the locality may choose to calculate a watershed or locality-wide pre-developed annual load and corresponding impervious value, and designate a watershed-specific or locality-specific average land cover condition.

Localities have the following options when determining average land cover conditions:

Option 1: A locality may designate specific watersheds within its jurisdiction and calculate the average land cover condition ($I_{watershed}$) and associated average total phosphorous loading for those watersheds (**Table 5-15** presents representative land uses and associated percent impervious cover and phosphorous export values); or

Option 2: A locality may assume the Chesapeake Bay default value for total phosphorous loading of 0.45 pounds/acre/year (F_{VA}) and an equivalent impervious cover ($I_{watershed}$) of 16 percent for its entire jurisdiction.

The calculation of watershed-specific average total phosphorous loadings must be based upon the following:

1. existing land use data at time of local program adoption,
2. watershed size, and
3. determination of equivalent values of impervious cover for non-urban land uses which contribute nonpoint source pollution, such as agriculture, silviculture, etc.

Some localities may begin with *Option 2* while they gather the necessary data for *Option 1*. The average land cover condition, once established for a locality (or watershed), **should not change**, and the designer simply uses that value as the existing condition baseline value for the specific watershed or locality in which the project is located.

5-10.2 Performance-Based Water Quality Calculation Procedure

The following steps represent the performance-based water quality calculation procedure:

<u>STEP 1</u> **Determine the applicable area (A) and the post-developed impervious cover (I_{post}).**

<u>STEP 2</u> **Determine the existing impervious cover ($I_{existing}$) <u>or</u> use the average land cover condition ($I_{watershed}$)**

<u>STEP 3</u> **Determine the appropriate development situation.**

<u>STEP 4</u> **Determine the relative pre-development pollutant load (L_{pre}).**

<u>STEP 5</u> **Determine the relative post-development pollutant load (L_{post}).**

<u>STEP 6</u> **Determine the relative pollutant removal requirement (RR).**

<u>STEP 7</u> **Identify best management practice (BMP) options for the site.**

The following discussion presents each step of the calculation procedure:

<u>STEP 1</u> **Determine the applicable area (A) and the post-developed impervious cover (I_{post}).**

<u>Applicable Area</u>

The applicable area (A) is the parcel of land being developed. For large developments such as subdivisions, shopping centers, or office / institutional campus style developments, use of the entire parcel or development areas can result in unreasonable water quality requirements. In these cases, the designation of a *planning area* may be more appropriate. A planning area is a designated portion of the parcel of land, measured in acres, on which the development project is located. The planning area may be established by drainage areas or development areas. A designated planning area can be helpful when analyzing developments where the density of impervious cover, construction phasing, or other factors vary across the total site and create distinctly separate areas of analysis. (The concept and advantages of planning areas are discussed further in **Chapter 2**.)

The use of planning areas must be preceeded by the development of a master plan to ensure that the entire development is accounted for, as well as document the consistent application of the designated planning areas (land can not be included in more than one planning area).

Post-development Impervious Cover (I_{post})

The designer must determine the amount of post-development impervious cover (I_{post}), in percent, within the applicable area. The zoning classifications or proposed density of a site will allow the designer to estimate impervious cover. It is important that the roadways, sidewalks, and other public or common ground improvements are included in the overall total impervious cover calculations when calculating the average lot size and the associated impervious cover. Compliance and final engineering calculations, however, should be based on impervious cover shown on the final site or subdivision plan. A locality may set minimum acceptable impervious percentages for particular land uses, and may also require a determination of the actual proposed impervious cover and **use the higher value**. Representative land use categories and associated average impervious cover values are shown in **Table 5-15**.

STEP 2 Determine the existing impervious cover ($I_{existing}$) or use the average land cover condition ($I_{watershed}$) as determined by the locality.

Existing Impervious Cover ($I_{existing}$):

The existing impervious cover ($I_{existing}$) is the percentage of the site that is occupied by impervious cover prior to the development of the proposed project. For new construction there is typically no existing impervious cover and therefore the average land cover condition or the watershed-specific value is used. Two of the four development situations presented in this standard, however, are based on the presence of existing site features or previous development and use the existing impervious cover as the basis for determining the pre-development total phosphorous load (L_{pre}).

Average Land Cover Condition ($I_{watershed}$):

A locality must establish the base pollutant load for specific watersheds or for the locality as a whole based on all of the land uses within the established boundary and, in turn, must determine the corresponding average land cover condition ($I_{watershed}$) measured in percent impervious cover. The average land cover condition, therefore, will be a watershed- or locality-specific value, or the Chesapeake Bay default value of 16%. The average land cover condition, once established for a locality (or watershed), should not change, and the designer simply uses that value as the pre-developed or existing average land cover condition for the specific watershed or locality in which the project is located.

STEP 3 Determine the appropriate development situation.

The performance-based criteria is applied through the use of four development situations. The application of each of these situations uses the same development characteristic (impervious cover) to determine the post-development pollutant load (L_{post}). However, the pre-development pollutant load (L_{pre}) is determined using either the average land cover condition ($I_{watershed}$) or the existing impervious cover ($I_{existing}$), depending on the development situation. The situations are as follows:

Situation 1: Land development where the existing percent impervious cover ($I_{existing}$) is less than or equal to the average land cover condition ($I_{watershed}$) and the proposed improvements will create a total percent impervious cover (I_{post}) which is less than the average land cover condition ($I_{watershed}$).

Requirement: No reduction in the after development pollutant discharge (L_{post}) is required.

Situation 2: Land development where the existing percent impervious cover ($I_{existing}$) is less than or equal to the average land cover condition ($I_{watershed}$) and the proposed improvements will create a total percent impervious cover (I_{post}) which is greater than the average land cover condition ($I_{watershed}$).

Requirement: The pollutant discharge after development (L_{post}) shall not exceed the existing pollutant discharge based on the average land cover condition ($L_{pre(watershed)}$).

Situation 3: Land development where the existing percent impervious cover ($I_{existing}$) is greater than the average land cover condition ($I_{watershed}$).

Requirement: The pollutant discharge after development (L_{post}) shall not exceed 1) the pollutant discharge based on existing conditions ($L_{pre(existing)}$) less 10%; or 2) the pollutant discharge based on the average land cover condition ($L_{pre(watershed)}$), whichever is greater.

Situation 4: Land development where the existing percent impervious cover ($I_{existing}$) is served by an existing stormwater management BMP(s) that addresses water quality.

Requirement: The pollutant discharge after development (L_{post}) shall not exceed the existing pollutant discharge based on the existing percent impervious cover while served by the existing BMP ($L_{pre(existingBMP)}$). The existing BMP shall be shown to have been designed and constructed in accordance with proper design standards and specifications, and to be in proper functioning condition.

If the proposed development meets the criteria for development Situation 1, than the low density development is considered to be the BMP and no pollutant removal is required. The calculation procedure for Situation 1 stops here. Development Situations 2 through 4 proceed to STEP 4.

STEP 4 **Determine the relative pre-development pollutant load (L_{pre}).**

The pre-developed pollutant load is based on either the average land cover condition ($L_{pre(watershed)}$): Situation 2; or the existing site conditions ($L_{pre(existing)}$): Situation 3; **or** the existing site conditions while being served by a water quality BMP ($L_{pre(existingBMP)}$): Situation 4.

The simplified version of the Simple Method Pollutant Load Equation (**Equation 5-15**) is modified by inserting the specific values of I ($I_{watershed}$ or $I_{existing}$) to calculate the relative pre-development total phosphorous load for the different development situations (2 through 4). The Simple Method Pollutant Load Equation is applied to the development situations as follows:

Situation 2:

The treatment requirement for Situation 2 states that the pollutant discharge after development (L_{post}) shall not exceed the existing pollutant discharge based on the average land cover condition ($L_{pre(watershed)}$). Therefore, the Simple Method Pollutant Load Equation is slightly modified to calculate the relative pre-development pollutant load ($L_{pre(watershed)}$) as follows:

$$L_{pre(watershed)} = [0.05 + (0.009 \times I_{watershed})] \times A \times 2.28$$

Equation 5-16
Pollutant Load Based on Average Land Cover Conditions ($L_{pre(watershed)}$)

where:
$L_{pre(watershed)}$ = *relative pre-development total phosphorous load (pounds per year)*
$I_{watershed}$ = *average land cover condition for specific watershed or locality* ***or*** *the Chesapeake Bay default value of 16% (percent expressed in whole numbers)*
A = *applicable area (acres)*

Situation 3:

The treatment requirement for Situation 3 states that the pollutant discharge after development (L_{post}) shall not exceed the greater of: 1) the pollutant discharge based on existing conditions ($L_{pre(existing)}$) less 10%; or 2) the pollutant discharge based on the average land cover condition ($L_{pre(watershed)}$).

The pre-development pollutant discharge must be calculated twice in order to determine compliance with this requirement: first based on the existing impervious cover ($I_{existing}$) to calculate the pre-development load ($L_{pre(existing)}$) (**Equation 5-17**); and again based on the average land cover condition ($I_{watershed}$) to calculate the pre-development load ($L_{pre(watershed)}$) (**Equation 5-16**). The Simple Method Pollutant Load Equation is used as follows:

$$L_{pre(existing)} = [0.05 + (0.009 \times I_{existing})] \times A \times 2.28$$

Equation 5-17
Pollutant Load Based on Existing Site Conditions ($L_{pre(existing)}$)

where: $L_{pre(existing)}$ = *relative pre-development total phosphorous load (pounds per year)*
$I_{existing}$ = *existing site impervious cover (percent expressed in whole numbers)*
A = *applicable area (acres)*

The existing pollutant discharge based on the average land cover condition ($L_{pre(watershed)}$) is calculated the same as was done in **STEP 2** using **Equation 5-16**. The comparison of $L_{pre(existing)}$ less 10% and $L_{pre(watershed)}$ is made in **STEP 5** of this procedure.

Situation 4:

The requirement for Situation 4 states that the pollutant discharge after development (L_{post}) shall not exceed the existing pollutant discharge based on the existing percent impervious cover while served by the existing BMP(s) ($L_{pre(existingBMP)}$). The existing BMP(s) shall be shown to have been designed and constructed in accordance with proper design standards and specifications, and to be in proper functioning condition.

This requirement assumes that either all or a portion of the pollutant load generated by the existing impervious cover on a development is being reduced by one or more BMPs designed and constructed for that purpose. It becomes the responsibility of the designer or applicant to demonstrate that the facility was designed and constructed in accordance with the proper design standards and specifications, and is in proper functioning condition in order to justify the pollutant removal efficiency attributed to that particular BMP. Acceptable pollutant removal efficiency values attributed to some of the more commonly used BMPs for which there is adequate performance data are presented in **Table 5-14**. **Chapter 3** provides the design and maintenance requirements for these BMPs.

It should be noted that there may be more than one existing BMP. The drainage area to each BMP must be evaluated independantly. All areas being evaluated should be clearly documented on an existing conditon drainage area map.

The pre-developed total phosphorous load based on existing site conditions ($L_{pre(existing)}$) is calculated using **Equation 5-17**. The designer must then determine how much of the existing impervious cover is captured by the existing BMP(s), and the relative pollutant load removed. The Simple Method Pollutant Load Equation is therefore applied independently to each BMP drainage area of the site to determine the relative pollutant load of the area draining to the existing BMP(s) (**Equation 5-18**) and then the efficiency of each BMP is applied to the respective load to determine the load removed (**Equation 5-19**) as follows:

$$L_{pre(BMP)} = [0.05 + (0.009 \times I_{pre(BMP)})] \times A_{existBMP} \times 2.28$$

Equation 5-18
Pollutant Load to Existing BMP ($L_{pre(BMP)}$)

where:
- $L_{pre(BMP)}$ = *relative pre-development total phosphorous load entering existing BMP (pounds per year)*
- $I_{pre(BMP)}$ = *existing impervious cover to existing BMP (percent expressed in whole numbers)*
- $A_{existBMP}$ = *drainage area to existing BMP (acres)*

The relative pollutant load removed by the existing BMP ($L_{removed(existingBMP)}$) is determined as follows:

$$L_{removed(existingBMP)} = L_{pre(BMP)} \times EFF_{existBMP}$$

Equation 5-19
Pollutant Load Removed by Existing BMP ($L_{removed(existingBMP)}$)

where:
- $L_{removed(existingBMP)}$ = *relative pre-development total phosphorous load removed by existing BMP (pounds per year)*
- $L_{pre(BMP)}$ = *relative pre-development total phosphorous load entering existing BMP, **Equation 5-18** (pounds per year)*
- $EFF_{existBMP}$ = *documented pollutant removal efficiency of existing BMP (expressed in decimal form)*

Equations 5-18 and **5-19** are thus applied independantly to each existing BMP on the site.

The relative pre-development pollutant load from the site can now be calculated using **Equation 5-20** as follows:

$$L_{pre(existingBMP)} = L_{pre(existing)} \; ! \; (L_{removed(existingBMP1)} + L_{removed(existingBMP2)} + L_{removed(existingBMP3)})$$

Equation 5-20
Pollutant Load Based on Existing BMP Removal Efficiency ($L_{pre(existingBMP)}$)

where:
- $L_{pre(existingBMP)}$ = *relative pre-development total phosphorous load while being served by an existing BMP (pounds per year)*
- $L_{pre(existing)}$ = *relative pre-development total phosphorous load based on existing site conditions, **Equation 5-17** (pounds per year)*
- $EFF_{existBMP}$ = *documented pollutant removal efficiency of existing BMP (expressed in decimal form)*
- $L_{removed(existingBMP)}$ = *relative pre-development total phosphorous load removed by existing BMP, **Equation 5-19** (pounds per year)*

<u>STEP 5</u> **Determine the relative post-development pollutant load (L_{post}).**

The post-development pollutant load (L_{post}) is calculated based on the proposed impervious cover for each development situation. The Simple Method Pollutant Load Equation based on the proposed post-development impervious cover (I_{post}) is used as follows:

$$L_{post} = [0.05 + (0.009 \times I_{post})] \times A \times 2.28$$

Equation 5-21
Pollutant Load Based on Post-Development Site Conditions (L_{post})

where:
- L_{post} = *relative post-development total phosphorous load (pounds per year)*
- I_{post} = *post-development impervious cover (percent expressed in whole numbers)*
- A = *applicable area (acres)*

<u>STEP 6</u> **Determine the relative pollutant removal requirement (RR).**

The pollutant removal requirement (RR) is defined as the relative amount of the keystone pollutant (in pounds per year) which must be removed by a BMP. The development situations discussed in **<u>STEP 3</u>** present the different removal or treatment requirements for each situation. There is no treatment requirement for Situation 1 due to the low density of development (proposed impervious cover less than the average land cover condition). The requirements for Situations 2, 3, and 4 are as follows:

Situation 2: $RR = L_{post} \; ! \; L_{pre(watershed)}$

Situation 3: $RR = L_{post} \; ! \; (0.9 \times L_{pre(existing)})$; **<u>or</u>**
$RR = L_{post} \; ! \; L_{pre(watershed)}$, which ever value of RR is less.

Situation 4: $RR = L_{post} \; ! \; L_{pre(existingBMP)}$

If the calculated RR value is less than or equal to zero, no BMPs are required. If the RR value greater than zero, continue on with <u>STEP 7</u>.

<u>STEP 7</u> **Identify best management practice (BMP) options for the site.**

The selection criteria for choosing an appropriate BMP for any given development site is often dictated by the physical characteristics of the site, such as soil types, topography, and drainage area. In addition, the pollutant removal requirement (RR) for the site may dictate that a BMP with a high removal efficiency (EFF_{BMP}) be used, while the physical characteristics of the site may dictate that a combination of strategically located BMPs be used. Specific siting and design criterion, as well as the accepted pollutant removal efficiencies for generally acceptable BMPs, are discussed in **Chapter 3: BMP Minimum Standards**.

The first step in determining which BMP may satisfy the pollutant removal requirement is to determine the necessary BMP pollutant removal efficiency. When the entire development is to be served by one BMP, this can be calculated using the following equation:

$$EFF = (RR \div L_{post}) \times 100$$

Equation 5-22
Required Pollutant Removal Efficiency (EFF)

where:
- EFF = *required pollutant removal efficiency*
- RR = *pollutant removal requirement (pounds per year)*
- L_{post} = *relative post-development total phosphorous load, Equation 5-21 (pounds per year)*

If more than one BMP will be used on the site, the removal requirement (RR) and post-development total load (L_{post}) must be calculated for each area using **Equation 5-22**. The designer can then use the required pollutant removal efficiency (RR) value to make a preliminary BMP(s) selection from **Table 5-15**. This is a preliminary selection since the specific siting and design criteria for the selected BMP must now be satisfied. Refer to **Chapter 3** for more information.

Once the BMP is selected and sited the designer must verify that the BMP(s) satisfies the removal requirement (RR) for the development. This is done by applying the pollutant removal efficiency (EFF_{BMP}) of the selected BMP to the post-developed pollutant load **entering the BMP as sited** (L_{BMP}). If the entire site drains to the proposed BMP, then the post-development pollutant load entering the BMP (L_{BMP}) is that which was calculated in **STEP 5** ($L_{post} = L_{BMP}$). In many cases, however, the topographic constraints of the site, or siting constraints of the specific BMP chosen, may result in some impervious areas not draining to the proposed BMP. Therefore, the Simple Method General Pollutant Load Equation must be applied to the actual drainage area of the BMP(s) as follows:

$$L_{BMP} = [0.05 + (0.009 \times I_{BMP})] \times A_{propBMP} \times 2.28$$

Equation 5-23
Pollutant Load Entering Proposed BMP (L_{BMP})

where:
- L_{BMP} = *relative post-development total phosphorous load entering proposed BMP(pounds per year)*
- I_{BMP} = *post-development percent impervious cover to proposed BMP (percent expressed in whole numbers)*
- $A_{propBMP}$ = *drainage area to proposed BMP (acres)*

The load removed by the BMP is then calculated as follows:

$$L_{removed} = Eff_{BMP} \times L_{BMP}$$

Equation 5-24
Pollutant Load Removed by Proposed BMP ($L_{removed}$)

where: $L_{removed}$ = *post-development total phosphorous load removed by proposed BMP (pounds per year)*
Eff_{BMP} = *pollutant removal efficiency of BMP (expressed in decimal form)*
L_{BMP} = *relative post-development total phosphorous load entering proposed BMP,* ***Equation 5-23*** *(pounds per year)*

The calculation in this step is performed for each BMP and the various $L_{removed}$ values for the existing and proposed BMPs are summed for the total pollutant load removal as follows:

$$L_{removed/total} = L_{removed/BMP1} + L_{removed/BMP2} + L_{removed/BMP3} + \ldots + L_{removed(existingBMP1)} + L_{removed(existingBMP2)} + L_{removed(existingBMP3)}$$

Equation 5-25
Total Pollutant Load Removed by Proposed BMPs ($L_{removed/total}$)

where: $L_{removed/total}$ = ***total*** *pollutant load removed by proposed BMPs (pounds per year)*
$L_{removed/BMP1}$ = *pollutant load removed by proposed BMP No. 1,* ***Equation 5-24***
$L_{removed/BMP2}$ = *pollutant load removed by proposed BMP No. 2,* ***Equation 5-24***
$L_{removed/BMP3}$ = *pollutant load removed by proposed BMP No. 3,* ***Equation 5-24***
$L_{removed(existingBMP)}$ = *pollutant load removed by existing BMP No. 1,* ***Equation 5-19***
$L_{removed(existingBMP)}$ = *pollutant load removed by existing BMP No. 2,* ***Equation 5-19***
$L_{removed(existingBMP)}$ = *pollutant load removed by existing BMP No. 3,* ***Equation 5-19***

The BMP or combination of BMPs is determined to be adequate if the total pollutant load removed ($L_{removed/total}$) is greater than or equal to the removal requirement (RR) calculated in **STEP 6**: $L_{removed/total}$ $ RR

If the total load removed is less than the removal requirement (RR) than an alternate BMP or combination of BMPs must be selected. It may be possible to simply increase the drainage area to the BMP(s) (if the entire site does not already drain to the BMP) in order to increase the overall pollutant removal from the site. Another option may be to reduce the impervious cover of the development in order to lower the removal requirement. The designer may also investigate the opportunities to capture off-site impervious area drainage in the proposed BMP to compensate for on-site areas which cannot be captured. In all cases the designer should contact the local program authority to determine if options are available in the local program as a result of a watershed or regional BMP plan.

Table 5-14
Water Quality BMP Pollutant Removal Efficiencies

Water Quality BMP*	Target Pollutant Removal Efficiency	Percent Impervious Cover
Vegetated filter strip Grassed swale	10% 15%	16-21%
Constructed wetlands Extended detention (2 x WQ Vol) Retention basin I (3 x WQ Vol)	30% 35% 40%	22 -37%
Bioretention basin Bioretention filter Extended detention-enhanced Retention basin II (4 x WQ Vol) Infiltration (1 x WQ Vol)	50% 50% 50% 50% 50%	38 -66%
Sand filter Infiltration (2 x WQ Vol) Retention basin III (4 x WQ Vol with aquatic bench)	65% 65% 65%	67 -100%

* Innovative or alternative BMPs not included in this table may be allowed at the discretion of the local program administrator, the plan approving authority, or the Department

Table 5-15
Simple Method General Pollutant Load Equation Solved for Incremental Impervious Cover Values (Urban Land Uses)

Representative Land Uses	Average Impervious Cover	Annual Pollutant Load (lb/ac/yr)
	0	0.11
2-5 Acre Residential	5	0.22
	10	0.32
	15	0.42
1 Acre Residential	20	0.52
½ Acre Residential	25	0.63
1/3 Acre Residential	30	0.73
1/4 Acre Residential	35	0.83
	40	0.94
1/8 Acre Residential	45	1.04
	50	1.14
Townhouses/ Garden Apartments	55	1.24
	60	1.35
	65	1.45
Light Industrial	70	1.55
	75	1.65
Heavy Industrial/ Commercial	80	1.76
	85	1.86
	90	1.96
	95	2.06
Pavement	100	2.17

Note: The average impervious cover values may be used for estimating or planning purposes when considering the representative land use as shown. When possible, final design calculations should be based on actual percent impervious cover as measured from the site plan.

Table 5-15 (Cont.)
Simple Method General Pollutant Load Equation Solved for
Incremental Impervious Cover Values
(Non-Urban Land Uses)
(in pounds/acre/year)

Land Use	Silt Loam Soils	Loam Soils	Sandy Loam Soils
Conventional Tillage Cropland	3.71	2.42	0.83
Conservation Tillage Cropland	2.32	1.52	0.52
Pasture Land	0.91	0.59	0.20
Forest Land	0.19	0.12	0.04

CHAPTER 5

APPENDIX

APPENDIX 5A

a b Constants for Virginia

a b Constants for Virginia

COUNTY	#	2 YEAR a	2 YEAR b	10 YEAR a	10 YEAR b	100 YEAR a	100 YEAR b
ARLINGTON	00	119.34	17.86	178.78	20.66	267.54	22.32
ACCOMACK	01	107.75	14.69	175.90	20.64	277.44	24.82
ALBEMARLE	02	106.02	15.51	161.60	18.73	244.82	20.81
ALLEGHENY	03	95.47	13.98	145.89	17.27	220.94	19.29
AMELIA	04	112.68	15.11	173.16	18.81	266.77	22.13
AMHERST	05	106.72	15.39	162.75	18.83	245.52	21.02
APPOMATTOX	06	109.11	15.39	167.44	19.12	254.03	21.61
AUGUSTA	07	84.21	10.44	135.74	14.54	210.02	16.99
BEDFORD	09	114.59	17.21	171.51	20.47	258.17	22.80
BLAND	10	105.33	16.56	162.75	20.41	247.84	22.87
BOTETOURT	11	110.32	16.95	164.94	20.01	247.92	22.16
BRUNSWICK	12	126.74	17.27	190.73	21.52	287.02	24.46
BUCHANAN	13	87.14	13.22	128.51	15.15	189.98	16.22
BUCKINGHAM	14	109.95	15.41	168.28	19.11	254.59	21.47
CAMPBELL	15	110.26	15.76.	167.27	19.18	252.65	21.56
CAROLINE	16	121.21	17.33	182.56	20.88	275.65	23.30
CARROLL	17	119.79	18.65	188.13	23.81	288.94	27.06
CHARLES CITY	18	124.23	17.14	186.52	21.05	281.04	23.85
CHARLOTTE	19	109.87	14.71	171.75	19.25	265.18	22.56
CHESTERFIELD	20	124.66	17.55	186.15	21.03	277.94	23.26
CLARKE	21	94.13	12.88	141.03	15.39	210.66	16.85
CRAIG	22	106.67	16.54	166.19	20.94	251.27	22.95
CULPEPER	23	111.90	16.25	169.78	19.51	255.26	21.52
CUMBERLAND	24	111.34	15.29	172.73	19.29	271.55	24.02

COUNTY	#	a	b	a	b	a	b
DICKENSON	25	87.03	13.10	128.09	14.82	190.08	15.98
DINWIDDIE	26	125.08	17.29	189.77	21.51	284.68	24.02
ESSEX	28	119.70	16.76	180.50	20.18	271.79	22.58
FAIRFAX	29	117.06	17.34	178.32	20.49	269.23	22.40
FAUQUIER	30	116.55	17.52	172.47	20.02	255.06	21.38
FLOYD	31	121.22	19.16	185.59	23.38	281.91	26.26
FREDERICK	34	93.79	13.15	141.02	15.77	211.40	17.42
GILES	35	106.14	16.72	165.04	20.80	252.79	23.46
GLOUCESTER	36	119.62	16.09	182.54	20.40	276.43	23.35
GOOCHLAND	37	114.42	15.95	177.24	19.93	269.07	22.27
GRAYSON	38	119.29	18.94	176.02	22.06	262.24	24.25
GREEN	39	105.71	15.10	159.92	18.20	241.18	20.34
GREENSVILLE	40	129.97	17.80	194.08	22.01	291.37	24.83
HALIFAX	41	111.92	15.14	173.81	19.52	267.09	22.70
HANOVER	42	122.80	17.29	185.01	20.91	278.40	23.40
HENRICO	43	123.51	17.35	185.51	21.13	277.61	23.44
HENRY	44	116.19	17.33	177.84	21.34	270.32	24.01
HIGHLAND	45	90.13	12.61	134.38	15.02	199.74	16.50
ISLE OF WIGHT	46	125.69	17.02	190.34	21.71	287.14	24.73
JAMES CITY	47	121.86	16.58	185.06	20.81	279.14	23.67
KING GEORGE	48	120.31	17.28	181.05	20.50	273.29	22.83
KING & QUEEN	49	113.84	15.29	179.09	19.95	275.98	23.15
KING WILLIAM	50	114.92	15.58	180.36	20.13	277.03	23.26
LANCASTER	51	109.80	14.49	170.27	18.72	259.78	21.41
LEE	52	93.78	14.40	143.28	17.58	215.10	19.22
LOUDOUN	53	104.05	14.91	157.67	17.71	237.83	19.65

COUNTY	#	a	b	a	b	a	b
LOUISA	54	112.63	15.89	174.35	19.72	265.20	22.11
LUNENBERG	55	122.01	16.82	184.70	20.80	278.38	23.48
MADISON	56	106.87	15.33	161.43	18.49	242.78	20.62
MATHEWS	57	118.61	15.83	180.56	20.17	274.12	23.29
MECKLENBERG	58	121.77	16.55	184.54	20.74	278.33	23.48
MIDDLESEX	59	110.72	14.57	172.76	19.15	264.49	22.13
MONTGOMERY	60	118.78	19.21	176.95	22.39	262.93	24.17
NELSON	62	103.46	14.52	160.23	18.36	245.04	20.89
NEW KENT	63	121.03	16.58	183.93	20.72	277.89	23.51
NORFOLK	64	124.88	17.02	190.64	22.14	288.73	25.60
NORTHAMPTON	65	111.07	14.78	173.72	19.63	267.48	23.04
NORTHUMBERLAND	66	111.20	14.99	171.55	19.00	260.59	21.63
NOTTOWAY	67	122.38	17.06	183.97	20.87	275.78	23.19
ORANGE	68	116.77	16.63	178.14	20.19	270.55	22.72
PAGE	69	84.19	10.29	135.43	14.29	209.57	16.86
PATRICK	70	123.68	19.26	189.08	23.60	284.78	26.12
PITTSYLVANIA	71	112.30	16.02	173.58	20.27	263.51	22.98
POWHATAN	72	114.14	15.64	175.93	19.65	266.86	22.15
PRINCE EDWARD	73	111.01	15.06	172.73	19.29	264.28	22.20
PRINCE GEORGE	74	126.22	17.46	188.62	21.39	283.12	24.09
VIRGINIA BEACH	75	129.20	17.84	196.25	22.74	294.74	26.33
PRINCE WILLIAM	76	116.04	17.08	176.18	20.19	266.75	22.36
PULASKI	77	117.44	18.71	182.33	23.39	279.39	26.49
RAPPAHANNOCK	78	104.86	15.05	159.40	18.34	239.30	20.19
RICHMOND	79	117.41	16.23	177.35	19.85	267.20	22.24
ROANOKE	80	117.53	18.79	174.97	21.80	261.95	23.81

COUNTY	#	a	b	a	b	a	b
ROCKBRIDGE	81	84.23	10.46	143.41	15.89	229.43	19.56
ROCKINGHAM	82	83.83	10.55	128.80	13.37	195.24	15.29
RUSSELL	83	92.64	14.17	143.00	17.32	216.40	19.36
SCOTT	84	92.64	14.17	143.00	17.32	216.40	19.35
SMYTH	86	106.19	16.57	169.30	21.37	262.49	24.57
SOUTHAMPTON	87	129.91	17.77	195.84	22.34	294.40	25.43
SPOTSYLVANIA	88	117.31	16.86	179.21	20.48	269.84	22.55
STAFFORD	89	118.72	17.34	179.62	20.64	270.74	22.79
SURRY	90	124.79	16.97	188.62	21.39	283.36	24.16
SUSSEX	91	130.37	18.03	193.23	21.91	287.99	24.56
TAZEWELL	92	91.25	13.56	141.61	17.04	217.59	19.48
WARREN	93	89.03	11.53	137.69	14.73	210.46	16.87
WASHINGTON	95	106.65	16.86	162.19	20.02	244.60	21.98
WESTMORELAND	96	114.40	15.76	174.96	19.47	266.16	22.12
WISE	97	89.83	13.49	132.05	15.44	194.10	16.35
WYTHE	98	116.78	18.83	174.91	22.13	261.68	24.25
YORK	99	122.93	16.72	186.78	21.22	282.80	24.39

		2 YEAR		10 YEAR		100 YEAR	
CITIES	#'s	a	b	a	b	a	b
RICHMOND	127/43	122.47	17.10	185.51	21.13	278.85	23.60
HAMPTON	114/27	123.93	16.94	186.78	21.22	283.18	24.56
LYNCHBURG	118/15	107.39	15.15	166.87	19.37	255.02	22.08
SUFFOLK	133/61	129.97	17.80	196.63	22.61	298.69	26.35
NEWPORT NEWS	121/94	126.11	17.37	189.27	21.62	285.24	24.71

Source: Virginia Department of Transportation

APPENDIX 5B

Filter and Drainage Diaphragm Design

- **USDA-SCS Soil Mechanics Note No. 1: Guide for Determining the Gradation of Sand and Gravel Filters (AVAILABLE UPON REQUEST)**

- **USDA-SCS Soil Mechanics Note No. 3: Soil Mechanics Considerations for Embankment Drains (AVAILABLE UPON REQUEST)**

APPENDIX 5C

Water Balance Analysis

Water Balance Analysis

The water balance analysis helps determine if a drainage area is large enough to support a permanent pool during normal conditions. The maximum draw down due to evaporation and infiltration is checked against the anticipated inflows during that same period. The anticipated drawdown during an extended period of no appreciable rainfall is checked as well. This will also help establish a planting zone for vegetation which can tolerate the dry conditions of a periodic draw down of the permanent pool.

The water balance is defined as the change in volume of the permenant pool resulting from the potential total inflow less the potential total outflow.

change volume = inflows ! *outflows*

where: *inflows* = *runoff*, *baseflow*, and *rainfall*.
outflows = *infiltration*, *surface overflow*, *evaporation*, and *evapotranspiration*.

This procedure will assume no inflow from baseflow, and because only the permanent pool volume is being evaluated, no losses for surface overflows. In addition, infiltration should be addressed by a geotechnical report. A clay liner should be specified if the analysis of the existing soils indicates excessive infiltration. In many cases, the permeability of clayey soils will be reduced to minimal levels due to the clogging of the soil pores by the fines which eventually settle out of the water column. This may be considered in the water balance equation by assuming the permeability of a clay liner: 1×10^{-6} cm/s (3.94×10^{-7} in/sec.) per specifications. Therefore, the change in storage = runoff ! evaporation ! infiltration.

Example

Given:

Drainage Area:	*85 ac.* (Average 65% impervious cover)
SCS RCN:	*72*
Precipitation P (2-year storm):	*3.1* inches
Runoff, Q:	*1.1* inches
Permanent Pool Volume:	*0.65 x 85 ac. = 55 ac.* impervious cover
	WQ volume = *(0.5in.) (55 ac.) (12in./ft.) = 2.29 ac.ft.*
	Retention Basin II (*4* x WQ vol.) = *4 x 2.29 = 9.16 ac.ft.*
Permanent Pool Surface area:	*2.4 ac.*
Infiltration (clay liner per specs.):	1×10^{-6} *cm/s* (3.94×10^{-7} *in/sec.*)

Find:

a) Draw down during highest period of evaporation.
b) Draw down during extended period of no appreciable rainfall.

Solution:

a) Draw down during highest period of evaporation: July

Inflow = Monthly Runoff = *P x E*

Where *P* = precipitation
E = efficiency of runoff (assumed to be ratio of SCS runoff depth to rainfall depth for 2 year storm)
= 1.1 in./ 3.1" = 0.35

	April	May	June	July	Aug.	Sept.
Precip. (in)	*2.96*	*3.84*	*3.62*	***5.03***	*4.40*	*3.34*
Evap. (in)	*2.28*	*3.89*	*5.31*	***6.23***	*5.64*	*3.92*

(From **Table 5C-1** and **5C-2**)

Inflow: Runoff = *5.03 in. × 0.35 = 1.76 in. = 1.76 in. × 85 ac. ' 12 in./ft. = 12.5 ac.ft.*

Outflow: Evaporation = *2.4 ac. × 6.23 in. ' 12 in./ft. = 1.24 ac.ft.*

Infiltration (w/ liner)= *2.4 ac. × (3.94×10^{G7} in./sec.) (3600 sec./hr.) (24 hr./day) (31 days) ' (12 in./ft.) = 0.21 ac. ft.*

Water balance (w/ liner) = *(inflow) ! (outflow) = (12.5 ac.ft.) ! (1.24 + 0.21) ac.ft. = + 11.05 ac.ft.*

Infiltration (w/o liner); assume infiltration rate of .02 in./hr. (clay/silty clay) =
2.4 ac. × .02 in./hr. × (24 hr./day) (31 days) ' 12in./ft. = 2.97 ac.ft.

Water balance (w/o liner) = *(12.5 ac.ft.) ! (2.97 + 0.21) ac.ft. = + 9.32 ac.ft.*

b) Drawdown during period of no appreciable rainfall. Assume 45 day period during July and August with no rainfall.

Inflow: runoff *= 0"*

Outflow: Evaporation = Avg. evaporation (July-Aug.) = *6.23 in. + 5.64 in.* ' *2 = 5.93 in.*
Avg. daily evaporation = *5.93 in.* ' *31 days =* *<u>0.191 in./day</u>*
Evaporation for 45 days = *45 days × 0.191 in./day = 8.61 in.*
Total evaporation = *2.4 ac. × 8.61 in.* ' *12 in./ft. =* *<u>1.7 ac.ft.</u>*

Infiltration (w/ liner): *2.4 ac. × (3.94 x* 10^{-7} *in./sec.) (3600 sec./hr.) (24 hr./day) (45 days)* ' *12 in./ft. =* *<u>0.30 ac.ft.</u>*

<u>Water balance</u> (w/ liner): *(0) ! (1.7 + 0.30) ac.ft. =* *<u>! 2.0 ac.ft.</u>*

Specify drawdown tolerant plants in areas corresponding to a depth of 2.0 ac.ft. (use stage storage curve).

Infiltration (w/o liner): *2.4 ac.× (.02 in./hr.) (24 hr./day) (45 day)* ' *12 in./ft.*
= *<u>4.32 ac.ft.</u>*

<u>Water balance</u> (w/o liner): *(0) ! (1.7 + 4.32) ac.ft. =* *<u>! 6.02 ac.ft</u>*.

This basin (with out a liner) will experience a significant draw down during drought conditions. Over time, the rate of infiltration may decrease due to the clogging of the soil pores. However, the aquatic and wetland plants may not survive the potential drought conditions and subsequent draw down during the first few years, and eventually give way to invasive species.

Note: A permanent pool volume of *9.16 ac.ft. = 1.29* watershed inches. A rainfall event yielding 1.29" or more of runoff will fill the pool volume.

Table 5C-1
Monthly Precipitation Normals (Inches)

Station	April	May	June	July	August	Sept.
Charlottesville	3.34	4.88	3.74	4.75	4.71	4.10
Danville	3.24	3.85	3.65	4.42	3.80	3.39
Farmville	3.03	4.05	3.41	4.34	3.99	3.18
Fredericksburg	3.05	3.85	3.35	3.65	3.61	3.49
Hot Springs	3.43	4.15	3.36	4.49	3.70	3.39
Lynchburg	3.09	3.91	3.45	4.16	3.59	3.24
Norfolk	3.06	3.81	3.82	5.06	4.81	3.90
Page County	3.84	4.77	4.41	4.50	4.34	4.81
Pennington Gap	4.25	4.83	4.09	4.77	3.76	3.67
Richmond	2.98	3.84	3.62	5.03	4.40	3.34
Roanoke	3.25	3.98	3.19	3.91	4.15	3.50
Staunton	2.82	3.60	2.95	3.49	3.67	3.46
Wash. National Airport	2.31	3.66	3.38	3.80	3.91	3.31
Williamsburg	3.01	4.52	4.03	4.96	4.72	4.25
Winchester	3.08	3.74	3.87	3.89	3.46	3.11
Wytheville	3.09	3.95	3.03	4.20	3.44	3.09

Source: Department of Environmental Services, Virginia State Climatology Office, Charlottesville, Virginia

Table 5C-2
Potential Evapotranspiration (Inches) *

Station	April	May	June	July	August	Sept.
Charlottesville	2.24	3.84	5.16	6.04	5.45	3.87
Danville	2.35	3.96	5.31	6.23	5.69	3.91
Farmville	2.34	3.81	5.13	6.00	5.41	3.71
Fredericksburg	2.11	3.80	5.23	6.11	5.46	3.83
Hot Springs	1.94	3.41	4.50	5.14	4.69	3.33
Lynchburg	2.21	3.72	4.99	5.85	5.31	3.70
Norfolk	2.20	3.80	5.37	6.34	5.79	4.14
Page County	1.68	3.06	4.09	4.71	4.26	3.05
Pennington Gap	2.14	3.59	4.72	5.45	4.97	3.60
Richmond	2.28	3.89	5.31	6.23	5.64	3.92
Roanoke	2.20	3.75	4.99	5.85	5.30	3.67
Staunton	2.00	3.52	4.77	5.52	4.95	3.47
Wash. National Airport	2.13	3.87	5.50	6.51	5.84	4.06
Williamsburg	2.27	3.86	5.23	6.14	5.61	3.97
Winchester	2.07	3.68	4.99	5.82	5.26	3.67
Wytheville	2.01	3.43	4.46	5.17	4.71	3.39

Source:Department of Environmental Services, Virginia State Climatology Office, Charlottesville, Virginia

***** Calculated using the Thornthwaite method

APPENDIX 5D

Worksheets

C **Stage Storage Worksheet**

C **Stage-Storage-Discharge Worksheet**

C **Storage Indication Hydrograph Routing Worksheet; *$2S/\Delta t + O$ Vs. O***

C **Storage Indication Hydrograph Routing Worksheet**

C **Performance-Based Water Quality Calculations Worksheet 1**

C **Performance-Based Water Quality Calculations Worksheet 2: Situation 2**

C **Performance-Based Water Quality Calculations Worksheet 3: Situation 3**

C **Performance-Based Water Quality Calculations Worksheet 4: Situation 4**

Stage-Storage Worksheet

PROJECT: ____ SHEET _____ OF _____

COUNTY: ___________________ COMPUTED BY:______________ DATE:________

DESCRIPTION: ____

ATTACH COPY OF TOPO: SCALE - 1" = ______ *ft*.

1	2	3	4	5	6	7	8
ELEV.	AREA *(in²)*	AREA *(ft²)*	AVG. AREA *(ft²)*	INTERVAL	VOL. *(ft³)*	TOTAL VOLUME	
						(ft³)	*(ac.ft.)*

Stage - Storage - Discharge Worksheet

ELEV *(MSL)*	STORAGE *(ac.ft.)*	WATER QUALITY ORIFICE		2-YEAR CONTROL				10-YEAR CONTROL				RISER STRUCTURE				BARREL				EMERGENCY SPILLWAY		TOTAL *Q* *(cfs)*
		(1)		WEIR (2)		ORIFICE (3)		WEIR (4)		ORIFICE (5)		WEIR (6)		ORIFICE (7)		INLET (8)		OUTLET (9)		(10)		
		h	*Q*	*h*	*Q*	*h*	*Q*	*h*	*Q*	*h*	*Q*	*h*	*Q*	*h*	*Q*	*HW/D*	*Q*	*h*	*Q*	*h*	*Q*	

Storage Indication Hydrograph Routing Worksheet
2S/Δt + O Vs. O

1	**2**	**3**	**4**	**5**	**6**	**7**
elevation (ft)	*stage (ft)*	*outflow (cfs)*	*storage (cf)*	*2S (cf)*	*2S/Δt (cfs)*	*2S/Δt + O (cfs)*
from plan	$elev_n$ - $elev_o$	*based on outflow device & stage*	*based on stage*	*2 × Col 4*	*Col 5 /Δt of hydrograph*	*Col 3 + Col 6*

Storage Indication Hydrograph Routing Worksheet

1	2	3	4	5	6	7
n	*Time (min)*	I_n *(cfs)*	$I_n + I_{n+1}$ *(cfs)*	$2S_n /\Delta t - O_n$ *(cfs)*	$2S_{n+1} /\Delta t + O_{n+1}$ *(cfs)*	O_{n+1} *(cfs)*
	from hydrograph		*Col* 3_n + *Col* 3_{n+1}	*Col* 6_n - 2(*Col* 7_n)	*Col* 4_{n-1} + *Col* 5_{n-1}	*from chart; use Col* 6_n

Worksheet 1
Page 1 of 3

STEP 1 **Determine the applicable area (A) and the post-developed impervious cover (I_{post}).**

Applicable area (A)* = ________ acres

Post-development impervious cover:

structures =__________acres

parking lot =__________acres

roadway =__________acres

other:

______________=__________acres

______________=__________acres

Total = __________acres

I_{post} = (total post-development impervious cover ÷ A) × 100 = ________%

* The area subject to the criteria may vary from locality to locality. Therefore, consult the locality for proper determination of this value.

STEP 2 **Determine the average land cover condition ($I_{watershed}$) or the existing impervious cover ($I_{existing}$).**

Average land cover condition ($I_{watershed}$):
If the locality has determined land cover conditions for individual watersheds within its jurisdiction, use the watershed specific value determined by the locality as $I_{watershed}$.

$I_{watershed}$ = ________%

Otherwise, use the Chesapeake Bay default value:

$I_{watershed}$ = 16%

Worksheet 1

Page 2 of 3

Existing impervious cover ($I_{existing}$):

Determine the existing impervious cover of the development site if present.

Existing impervious cover:

structures =__________acres

parking lot =__________acres

roadway =__________acres

other:

______________=__________acres

______________=__________acres

Total = __________acres

$I_{existing}$ = (total existing impervious cover ÷ A*) × 100 = _______%

* The area should be the same as used in STEP 1.

STEP 3 **Determine the appropriate development situation.**

The site information determined in STEP 1 and STEP 2 provide enough information to determine the appropriate development situation under which the performance criteria will apply. Check (U) the appropriate development situation as follows:

_____ **Situation 1**: This consists of land development where the existing percent impervious cover ($I_{existing}$) is less than or equal to the average land cover condition ($I_{watershed}$) and the proposed improvements will create a total percent impervious cover (I_{post}) which is less than or equal to the average land cover condition ($I_{watershed}$).

$\mathbf{I_{post}}$ ________% # $\mathbf{I_{watershed}}$ ________%

Worksheet 1

Page 3 of 3

_____ **Situation 2**: This consists of land development where the existing percent impervious cover ($I_{existing}$) is less than or equal to the average land cover condition ($I_{watershed}$) and the proposed improvements will create a total percent impervious cover (I_{post}) which is greater than the average land cover condition ($I_{watershed}$).

$I_{existing}$ ________% # **$I_{watershed}$** ________%; **and**

I_{post} ________% > **$I_{watershed}$** ________%

_____ **Situation 3**: This consists of land development where the existing percent impervious cover ($I_{existing}$) is greater than the average land cover condition ($I_{watershed}$).

$I_{existing}$ ________% > **$I_{watershed}$** ________%

_____ **Situation 4**: This consists of land development where the existing percent impervious cover ($I_{existing}$) is served by an existing stormwater management BMP(s) that addresses water quality.

If the proposed development meets the criteria for development Situation 1, than the low density development is considered to be the BMP and no pollutant removal is required. The calculation procedure for Situation 1 stops here. If the proposed development meets the criteria for development Situations 2, 3, or 4, then proceed to STEP 4 on the appropriate worksheet.

Worksheet 2 : Situation 2

Page 1 of 4

Summary of Situation 2 criteria: from calculation procedure **STEP 1** thru **STEP 3**, Worksheet 1:

Applicable area (A)* = ________ acres

$\mathbf{I_{post}}$ = (total post-development impervious cover ÷ A) × 100 = ________%

$\mathbf{I_{watershed}}$ = ________% **or** $I_{watershed}$ = 16%

$\mathbf{I_{existing}}$ = (total existing impervious cover ÷ A*) × 100 = ________%

$\mathbf{I_{existing}}$ ________% # $\mathbf{I_{watershed}}$ ________%; **and**

$\mathbf{I_{post}}$ ________% > $\mathbf{I_{watershed}}$ ________%

STEP 4 **Determine the relative pre-development pollutant load (L_{pre}).**

$\mathbf{L_{pre(watershed)}}$ = [0.05 + (0.009 × $I_{watershed}$)] × A × 2.28 (**Equation 5-16**)

where: $L_{pre(watershed)}$ = relative pre-development total phosphorous load (pounds per year)
$I_{watershed}$ = average land cover condition for specific watershed or locality **or** the Chesapeake Bay default value of 16% (percent expressed in whole numbers)
A = applicable area (acres)

$\mathbf{L_{pre(watershed)}}$ = [0.05 + (0.009 × ________)] × ________ × 2.28

= __________ pounds per year

Worksheet 2 : Situation 2

Page 2 of 4

STEP 5 **Determine the relative post-development pollutant load (L_{post}).**

$$L_{post} = [0.05 + (0.009 \times I_{post})] \times A \times 2.28 \quad \textbf{(Equation 5-21)}$$

where:
- L_{post} = relative post-development total phosphorous load (pounds per year)
- I_{post} = post-development percent impervious cover (percent expressed in whole numbers)
- A = applicable area (acres)

L_{post} = [0.05 + (0.009 × ________)] × ________ × 2.28

= __________ pounds per year

STEP 6 **Determine the relative pollutant removal requirement (RR).**

RR = L_{post} ! $L_{pre(watershed)}$

RR = ________ ! ________

= __________ pounds per year

STEP 7 **Identify best management practice (BMP) for the site.**

1. Determine the required pollutant removal efficiency for the site:

$$EFF = (RR \div L_{post}) \times 100 \quad \textbf{(Equation 5-22)}$$

where:
- EFF = required pollutant removal efficiency (percent expressed in whole numbers)
- RR = pollutant removal requirement (pounds per year)
- L_{post} = relative post-development total phosphorous load (pounds per year)

EFF = (________ ÷ ________) × 100

= __________%

Worksheet 2 : Situation 2
Page 3 of 4

2. Select BMP(s) from **Table 5-15** and locate on the site:

BMP 1:____

BMP 2:____

BMP 3:____

3. Determine the pollutant load entering the proposed BMP(s):

$\mathbf{L_{BMP}} = [0.05 + (0.009 \times I_{BMP})] \times A \times 2.28$ **(Equation 5-23)**

where: L_{BMP} = relative post-development total phosphorous load entering proposed BMP (pounds per year)
I_{BMP} = post-development percent impervious cover of BMP drainage area (percent expressed in whole numbers)
A = drainage area of proposed BMP (acres)

$\mathbf{L_{BMP1}} = [0.05 + (0.009 \times$ ________$)] \times$ _______ $\times 2.28$

= __________ pounds per year

$\mathbf{L_{BMP2}} = [0.05 + (0.009 \times$ ________$)] \times$ ________ $\times 2.28$

= __________ pounds per year

$\mathbf{L_{BMP3}} = [0.05 + (0.009 \times$ ________$)] \times$ ________ $\times 2.28$

= __________ pounds per year

Worksheet 2 : Situation 2

Page 4 of 4

4. Calculate the pollutant load removed by the proposed BMP(s):

$\mathbf{L_{removed}} = Eff_{BMP} \times L_{BMP}$ **(Equation 5-24)**

where:
$L_{removed}$ = Post-development pollutant load removed by proposed BMP (pounds per year)
Eff_{BMP} = pollutant removal efficiency of BMP (expressed in decimal form)
L_{BMP} = relative post-development total phosphorous load entering proposed BMP (pounds per year)

$\mathbf{L_{removed/BMP1}}$ = ________ × ________ = __________ pounds per year

$\mathbf{L_{removed/BMP2}}$ = ________ × ________ = __________ pounds per year

$\mathbf{L_{removed/BMP3}}$ = ________ × ________ = __________ pounds per year

5. Calculate the total pollutant load removed by the BMP(s):

$\mathbf{L_{removed/total}} = L_{removed/BMP1} + L_{removed/BMP2} + L_{removed/BMP3} + \ldots$ **(Equation 5-25)**

where:
$L_{removed/total}$ = **total** pollutant load removed by proposed BMPs
$L_{removed/BMP1}$ = pollutant load removed by proposed BMP No. 1
$L_{removed/BMP2}$ = pollutant load removed by proposed BMP No. 2
$L_{removed/BMP3}$ = pollutant load removed by proposed BMP No. 3

$\mathbf{L_{removed/total}}$ = __________ + __________ + __________ + . . .

= __________ pounds per year

6. Verify compliance:

$\mathbf{L_{removed/total}}$ $ **RR**

__________ $ __________

Worksheet 3 : Situation 3

Page 1 of 5

Summary of Situation 3 criteria: from calculation procedure **STEP 1** thru **STEP 3**, Worksheet 1:

Applicable area (A)* = ________ acres

$\mathbf{I_{post}}$ = (total post-development impervious cover ÷ A) × 100 = ________%

$\mathbf{I_{watershed}}$ =________% **or** $I_{watershed}$ = 16%

$\mathbf{I_{existing}}$ = (total existing impervious cover ÷ A*) × 100 = ________%

$\mathbf{I_{existing}}$ ________% > $\mathbf{I_{watershed}}$ ________%

STEP 4 **Determine the relative pre-development pollutant load (L_{pre}).**

1. Pre-development pollutant load based on the existing impervious cover:

$\mathbf{L_{pre(existing)}}$ = [0.05 + (0.009 × $I_{existing}$)] × A × 2.28 **(Equation 5-17)**

where:

$L_{pre(existing)}$ = relative pre-development total phosphorous load (pounds per year)
$I_{existing}$ = existing site impervious cover (percent expressed in whole numbers)
A = applicable area (acres)

$\mathbf{L_{pre(existing)}}$ = [0.05 + (0.009 × ________)] × ________ × 2.28

= __________ pounds per year

Worksheet 3 : Situation 3

Page 2 of 5

2. Pre-development pollutant load based on the average land cover condition:

$$\mathbf{L_{pre(watershed)}} = [0.05 + (0.009 \times I_{watershed})] \times A \times 2.28 \quad (\textbf{Equation 5-16})$$

where:
$L_{pre(watershed)}$ = relative pre-development total phosphorous load (pounds per year)
$I_{watershed}$ = average land cover condition for specific watershed or locality **or** the Chesapeake Bay default value of 16% (percent expressed in whole numbers)
A = applicable area (acres)

$$\mathbf{L_{pre(watershed)}} = [0.05 + (0.009 \times \underline{\hspace{2cm}})] \times \underline{\hspace{2cm}} \times 2.28$$

$$= \underline{\hspace{2.5cm}} \text{ pounds per year}$$

STEP 5 **Determine the relative post-development pollutant load (L_{post}).**

$$\mathbf{L_{post}} = [0.05 + (0.009 \times I_{post})] \times A \times 2.28 \quad (\textbf{Equation 5-21})$$

where:
L_{post} = relative post-development total phosphorous load (pounds per year)
I_{post} = post-development percent impervious cover (percent expressed in whole numbers)
A = applicable area (acres)

$$\mathbf{L_{post}} = [0.05 + (0.009 \times \underline{\hspace{1.5cm}})] \times \underline{\hspace{1.5cm}} \times 2.28$$

$$= \underline{\hspace{2.5cm}} \text{ pounds per year}$$

STEP 6 **Determine the relative pollutant removal requirement (RR).**

$$\mathbf{RR} = L_{post} \; ! \; (0.9 \times L_{pre(existing)})$$

$$= \underline{\hspace{2cm}} \; ! \; (0.9 \times \underline{\hspace{2cm}}) = \underline{\hspace{2.5cm}} \text{ pounds per year}$$

or

$$\mathbf{RR} = L_{post} \; ! \; L_{pre(watershed)}$$

$$= \underline{\hspace{2cm}} \; ! \; \underline{\hspace{2cm}} = \underline{\hspace{2.5cm}} \text{ pounds per year}$$

Worksheet 3 : Situation 3
Page 3 of 5

The pollutant removal requirement (RR) for Situation 3 is the lesser of the two values calculated above:

RR = __________ pounds per year

STEP 7 **Identify best management practice (BMP) for the site.**

1. Determine the required pollutant removal efficiency for the site:

$$\mathbf{EFF} = (RR \div L_{post}) \times 100 \qquad \textbf{(Equation 5-22)}$$

where: EFF = required pollutant removal efficiency (percent expressed in whole numbers)
RR = pollutant removal requirement (pounds per year)
L_{post} = relative post-development total phosphorous load (pounds per year)

EFF = (_______ ÷ _______) × 100

= __________%

2. Select BMP(s) from **Table 5-15** and locate on the site:

BMP 1:____

BMP 2:____

BMP 3:____

Worksheet 3 : Situation 3

Page 4 of 5

3. Determine the pollutant load entering the proposed BMP(s):

$\mathbf{L_{BMP}} = [0.05 + (0.009 \times I_{BMP})] \times A \times 2.28$ **(Equation 5-23)**

where: L_{BMP} = relative post-development total phosphorous load entering proposed BMP (pounds per year)
I_{BMP} = post-development percent impervious cover of BMP drainage area (percent expressed in whole numbers)
A = drainage area of proposed BMP (acres)

$\mathbf{L_{BMP1}} = [0.05 + (0.009 \times$ ________)] × _______ × 2.28

= __________ pounds per year

$\mathbf{L_{BMP2}} = [0.05 + (0.009 \times$ ________)] × ________ × 2.28

= __________ pounds per year

$\mathbf{L_{BMP3}} = [0.05 + (0.009 \times$ ________)] × ________ × 2.28

= __________ pounds per year

4. Calculate the pollutant load removed by the proposed BMP(s):

$\mathbf{L_{removed}} = Eff_{BMP} \times L_{BMP}$ **(Equation 5-24)**

where: $L_{removed}$ = Post-development pollutant load removed by proposed BMP (pounds per year)
Eff_{BMP} = pollutant removal efficiency of BMP (expressed in decimal form)
L_{BMP} = relative post-development total phosphorous load entering proposed BMP (pounds per year)

$\mathbf{L_{removed/BMP1}}$ = ________ × ________ = __________ pounds per year

$\mathbf{L_{removed/BMP2}}$ = ________ × ________ = __________ pounds per year

$\mathbf{L_{removed/BMP3}}$ = ________ × ________ = __________ pounds per year

Worksheet 3 : Situation 3

Page 5 of 5

5. Calculate the total pollutant load removed by the BMP(s):

$$\mathbf{L_{removed/total}} = L_{removed/BMP1} + L_{removed/BMP2} + L_{removed/BMP3} + \ldots \quad \textbf{(Equation 5-25)}$$

where: $L_{removed/total}$ = **total** pollutant load removed by proposed BMPs
$L_{removed/BMP1}$ = pollutant load removed by proposed BMP No. 1
$L_{removed/BMP2}$ = pollutant load removed by proposed BMP No. 2
$L_{removed/BMP3}$ = pollutant load removed by proposed BMP No. 3

$\mathbf{L_{removed/total}}$ = __________ + __________ + __________ + . . .

= __________ pounds per year

6. Verify compliance:

$\mathbf{L_{removed/total}} \ \$ \ \mathbf{RR}$

$\_\_\_\_\_\_\_\_\_\_ \ \$ \ \_\_\_\_\_\_\_\_\_\_$

Worksheet 4 : Situation 4

Page 1 of 6

Summary of Situation 3 criteria: from calculation procedure **STEP 1** thru **STEP 3**, Worksheet 1:

Applicable area (A) = ________ acres

$\mathbf{I_{post}}$ = (total post-development impervious cover ÷ A) × 100 = ________%

$\mathbf{I_{watershed}}$ = ________% **or** $I_{watershed}$ = 16%

$\mathbf{I_{existing}}$ = (total existing impervious cover ÷ A*) × 100 = ________%

$\mathbf{I_{existing}}$ ________% > $\mathbf{I_{watershed}}$ ________%

Summary of existing BMP:

Existing BMP drainage area ($\mathbf{A_{existBMP}}$) = ________ acres

$\mathbf{I_{pre(BMP)}}$ = (total pre-development impervious cover ÷ $A_{existBMP}$) × 100 = ________%

$\mathbf{EFF_{existBMP}}$ = documented pollutant removal efficiency of existing BMP (expressed in decimal form)

STEP 4 **Determine the relative pre-development pollutant load ($\mathbf{L_{pre}}$).**

1. Calculate pre-development pollutant load based on the existing impervious cover:

$\mathbf{L_{pre(existing)}} = [0.05 + (0.009 \times I_{existing})] \times A \times 2.28$ **(Equation 5-17)**

where:
$L_{pre(existing)}$ = relative pre-development total phosphorous load (pounds per year)
$I_{existing}$ = existing site impervious cover (percent expressed in whole numbers)
A = applicable area (acres)

$\mathbf{L_{pre(existing)}}$ = [0.05 + (0.009 × ________)] × ________ × 2.28

= __________ pounds per year

Worksheet 4 : Situation 4
Page 2 of 6

2. Calculate pre-development pollutant load to existing BMP:

$\mathbf{L_{pre(BMP)}} = [0.05 + (0.009 \times I_{pre(BMP)})] \times A_{existBMP} \times 2.28$ **(Equation 5-18)**

where:
$L_{pre(BMP)}$ = relative pre-development total phosphorous load to existing BMP (pounds per year)
$I_{pre(BMP)}$ = existing impervious cover to existing BMP (percent expressed in whole numbers)
$A_{existBMP}$ = drainage area of existing BMP (acres)

$L_{pre(BMP)} = [0.05 + (0.009 \times$ ________$)] \times$ ________ $\times 2.28$

= __________ pounds per year

3. Calculate pre-development pollutant load removed by existing BMP:

$\mathbf{L_{removed(existingBMP)}} = L_{pre(BMP)} \times EFF_{existBMP}$ **(Equation 5-19)**

where: $L_{removed(existingBMP)}$ = relative pre-development total phosphorous load removed by existing BMP (pounds per year)
$L_{pre(BMP)}$ = relative pre-development total phosphorous load entering existing BMP, **Equation 5-18** (pounds per year)
$EFF_{existBMP}$ = documented pollutant removal efficiency of existing BMP (expressed in decimal form)

$\mathbf{L_{removed(existingBMP)}}$ = _________ × ________

= __________ pounds per year

Steps 2 and 3 are repeated for each existing BMP on the site.

Worksheet 4: Situation 4

Page 3 of 6

4. Calculate the pre-development pollutant load while being served by existing BMP(S):

$$\mathbf{L_{pre(existingBMP)}} = L_{pre(existing)} \; ! \; (L_{removed(existingBMP1)} + L_{removed(existingBMP2)} + L_{removed(existingBMP3)})$$

Equation 5-20

where:

$L_{pre(existingBMP)}$ = relative pre-development total phosphorous load while being served by an existing BMP (pounds per year)

$L_{pre(existing)}$ = relative pre-development total phosphorous load based on existing site conditions, **Equation 5-17** (pounds per year)

$EFF_{existBMP}$ = documented pollutant removal efficiency of existing BMP (expressed in decimal form)

$L_{removed(existingBMP)}$ = relative pre-development total phosphorous load removed by existing BMP, **Equation 5-19** (pounds per year)

$\mathbf{L_{pre(existingBMP)}}$ = _________ ! (_________ + _________ + _________)

= ____________ pounds per year

STEP 5 **Determine the relative post-development pollutant load (L_{post}).**

$\mathbf{L_{post}}$ = $[0.05 + (0.009 \times I_{post})] \times A \times 2.28$ (**Equation 5-21**)

where:

L_{post} = relative post-development total phosphorous load (pounds per year)

I_{post} = post-development percent impervious cover (percent expressed in whole numbers)

A = applicable area (acres)

$\mathbf{L_{post}}$ = [0.05 + (0.009 × ______)] × ______ × 2.28

= __________ pounds per year

STEP 6 **Determine the relative pollutant removal requirement (RR).**

RR = L_{post} ! $L_{pre(existingBMP)}$

= ________ ! ________

RR = __________ pounds per year

Worksheet 4 : Situation 4

2. Pre-development pollutant load based on the average land cover condition:

$\mathbf{L_{pre(watershed)}} = [0.05 + (0.009 \times I_{watershed})] \times A \times 2.28$ (**Equation 5-16**)

where: $L_{pre(watershed)}$ = relative pre-development total phosphorous load (pounds per year)
$I_{watershed}$ = average land cover condition for specific watershed or locality **or** the Chesapeake Bay default value of 16% (percent expressed in whole numbers)
A = applicable area (acres)

$\mathbf{L_{pre(watershed)}} = [0.05 + (0.009 \times$ ________)] × ________ × 2.28

= __________ pounds per year

STEP 7 **Identify best management practice (BMP) for the site.**

1. Determine the required pollutant removal efficiency for the site:

EFF = $(RR \div L_{post}) \times 100$ **(Equation 5-22)**

where: EFF = required pollutant removal efficiency (percent expressed in whole numbers)
RR = pollutant removal requirement (pounds per year)
L_{post} = relative post-development total phosphorous load (pounds per year)

EFF = (________ ÷ ________) × 100

= __________%

2. Select BMP(s) from **Table 5-15** and locate on the site:

BMP 1:____

BMP 2:____

BMP 3: ____

Worksheet 4 : Situation 4

Page 5 of 6

3. Determine the pollutant load entering the proposed BMP(s):

$\mathbf{L_{BMP}} = [0.05 + (0.009 \times I_{BMP})] \times A \times 2.28$ **(Equation 5-23)**

where:
L_{BMP} = relative post-development total phosphorous load entering proposed BMP (pounds per year)
I_{BMP} = post-development percent impervious cover of BMP drainage area (percent expressed in whole numbers)
A = drainage area of proposed BMP (acres)

$\mathbf{L_{BMP1}} = [0.05 + (0.009 \times$ ________)] × ________ × 2.28

= __________ pounds per year

$\mathbf{L_{BMP2}} = [0.05 + (0.009 \times$ ________)] × ________ × 2.28

= __________ pounds per year

$\mathbf{L_{BMP3}} = [0.05 + (0.009 \times$ ________)] × ________ × 2.28

= __________ pounds per year

4. Calculate the pollutant load removed by the proposed BMP(s):

$\mathbf{L_{removed}} = Eff_{BMP} \times L_{BMP}$ **(Equation 5-24)**

where:
$L_{removed}$ = Post-development pollutant load removed by proposed BMP (pounds per year)
Eff_{BMP} = pollutant removal efficiency of BMP (expressed in decimal form)
L_{BMP} = relative post-development total phosphorous load entering proposed BMP (pounds per year)

$\mathbf{L_{removed/BMP1}}$ = ________ × ________ = __________ pounds per year

$\mathbf{L_{removed/BMP2}}$ = ________ × ________ = __________ pounds per year

$\mathbf{L_{removed/BMP3}}$ = ________ × ________ = __________ pounds per year

Worksheet 4 : Situation 4

Page 6 of 6

5. Calculate the total pollutant load removed by the existing and proposed BMP(s):

$$\mathbf{L_{removed/total}} = L_{removed/BMP1} + L_{removed/BMP2} + L_{removed/BMP3} +$$

$$+ \; L_{removed(existingBMP1)} + L_{removed(existingBMP2)} + L_{removed(existingBMP3)} \quad \textbf{(Equation 5-25)}$$

where: $L_{removed/total}$ = **total** pollutant load removed by proposed BMPs
$L_{removed/BMP1}$ = pollutant load removed by proposed BMP No. 1, **Equation 5-24**
$L_{removed/BMP2}$ = pollutant load removed by proposed BMP No. 2, **Equation 5-24**
$L_{removed/BMP3}$ = pollutant load removed by proposed BMP No. 3, **Equation 5-24**
$L_{removed(existingBMP)}$ = pollutant load removed by existing BMP No. 1, **Equation 5-19**
$L_{removed(existingBMP)}$ = pollutant load removed by existing BMP No. 2, **Equation 5-19**
$L_{removed(existingBMP)}$ = pollutant load removed by existing BMP No. 3, **Equation 5-19**

$\mathbf{L_{removed/total}}$ = __________ + __________ + __________ + . . .

= __________ pounds per year

6. Verify compliance:

$\mathbf{L_{removed/total}}$ $ **RR**

_________ $ _________

CHAPTER 6

EXAMPLE PROBLEMS

Example 6.1

HYDROLOGY

ILLUSTRATIONS

EXAMPLE 6.1

INTRODUCTION

Example 6.1 (referred to as **Example 1** in **Chapter 5** develops the hydrology of a 25-acre watershed drained by two distinct channels. A study point is established at the confluence of these channels. The proposed development disturbs 11.9 acres.

Several design elements are illustrated in this problem. **Example 6.1A** uses SCS TR-55 methodology for determining the peak discharge from the watershed. It also shows the impact of the time of concentration, t_c , on the peak discharge.

Example 6.1B uses the Rational Method* for determining the peak discharge from the same watershed described in **6.1A**.

Example 6.1C uses the SCS Tabular Method and divides the watershed into sub-watersheds. The watershed analyzed in this example is simple, but it serves to illustrate the conditions where a development may be large enough or diverse enough to warrant the use of sub-watersheds.

Example 6.1D uses the Rational Method to determine the peak discharge from the sub-watersheds described in **6.1C**.

* *Note that the 25-acre drainage area exceeds the Rational Method's recommended limit of 20 acres, but the method is still used in this example for comparison purposes.*

EXAMPLE 6.1A

Example 6.1A uses SCS TR-55 for the hydrologic analysis of the 25-acre watershed, which is considered homogeneous. The critical design decision is the selection of the post-developed time of concentration, t_c, flow path. Typically, the pre-developed condition t_c flow path is the path from the most hydrologically distant point. The post-developed condition flow path, however, should represent the peak discharge. Note that if the watershed has more than one flow path, the longest one may not be the most representative of the watershed's peak. Therefore, engineering judgement may be required to select the appropriate path. This example highlights the effect that the t_c flow path can have on the peak discharge.

Given:

A 25-acre watershed consisting of woods and agricultural lands. Two channels drain the 25 acres to the study point. The study point is at the confluence of these two channels. The proposed development disturbs 11.9 acres. Refer to **Figure 6-1** for a schematic drawing of the pre- and post-developed condition watershed.

Find:

The pre- and post-developed peak discharges from the watershed using SCS methods. The pre-developed t_c flow path should be the flow path from the most hydrologically distant point to the watershed study point. The selected post-developed t_c flow path should be the path that is most representative of the proposed development and the associated increase in peak discharge.

Solution:

Pre- and post-developed peak discharges for the watershed, as shown in **Figure 6-1**, can be calculated using the SCS TR-55 Graphical Peak Discharge Method or the Tabular Peak Discharge Method. For this example, the watershed is considered homogeneous, so the Graphical Peak Discharge Method will be used. The effect of the selected t_c flow path on the peak discharge is summarized in **Table 6-1**. TR-55 worksheets are included at the end of this example.

FIGURE 6.1 - 1a,b
Example 1 - 25-Acre Watershed Pre- and Post-developed Condition

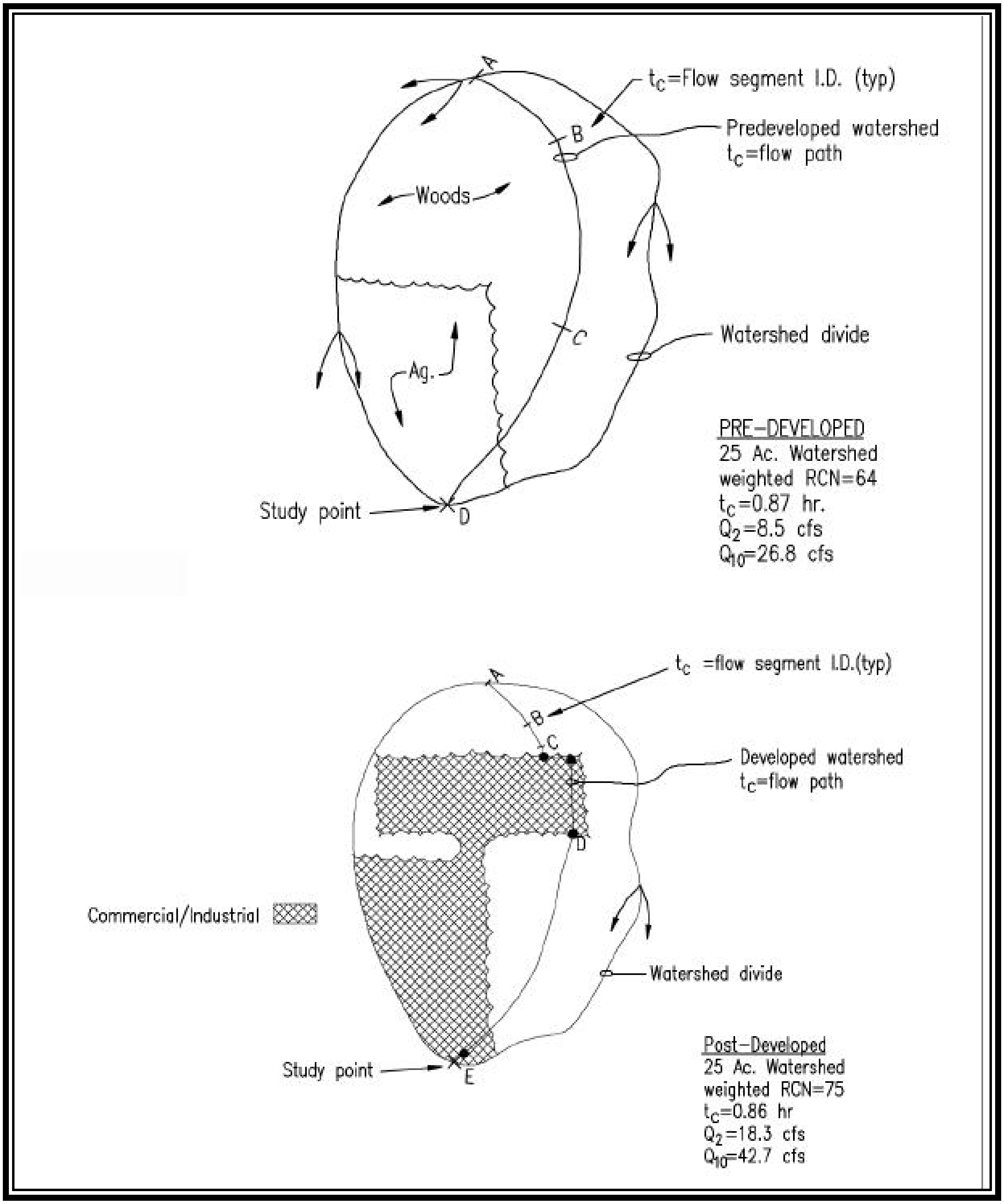


FIGURE 6.1-1c

Example 1 - 25-Acre Watershed Post-developed Condition - Most Representative t_c Flow Path

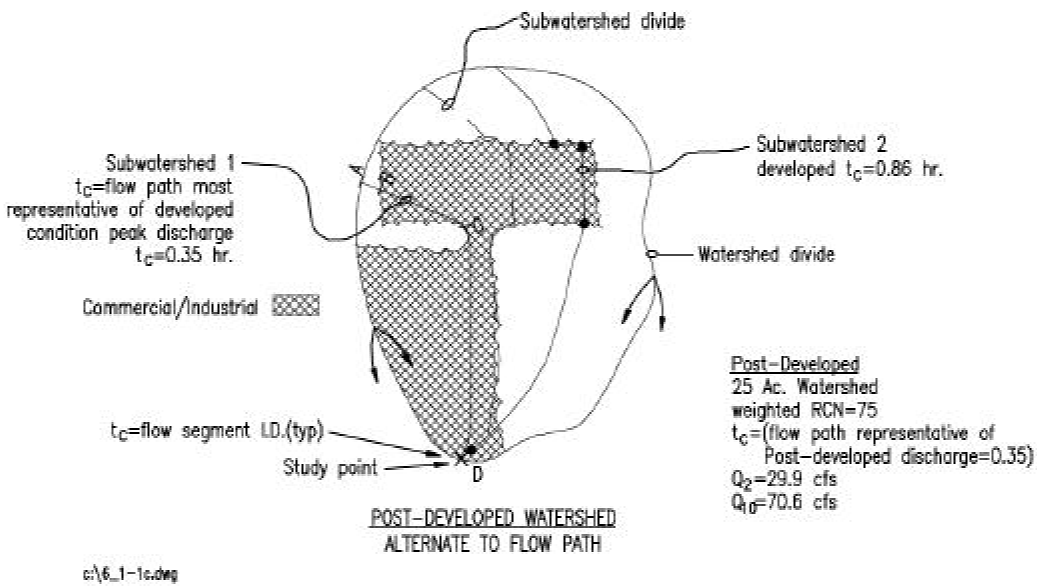


TABLE 6.1 - 1
Hydrologic Summary - Full Watershed, Example 6.1A
TR-55 Graphical Peak Discharge Method

Condition	**Area *(ac.)***	**RCN**	**t_c *(hrs.)***	**Q_2 *(cfs)***	**Q_{10} *(cfs)***	**Remarks**
Pre-developed	25	64	0.87	8.5	26.8	Longest t_c path (a)
Post-developed	25	75	0.86	18.3	42.7	Longest t_c path (b)
Post-developed	25	75	0.35	29.9	70.6	Most representative t_c path (c)

By using the flow path that best represents the developed area, a significant increase in the design peak discharge occurs. To prove that this higher discharge is more accurate, the watershed can be divided into two sub-watersheds that are analyzed independently using the Tabular Peak Discharge Method (which allows for analysis of heterogeneous sub-watersheds). The discharge hydrographs from each sub-watershed are then added at the watershed study point. See **Example 1.6C** for this solution.

Worksheet 2: Runoff curve number and runoff

Project EXAMPLE 6.1 A By TSB Date 5-96

Location ______ Checked ____ Date ______

Circle one: Present Developed ENTIRE 25acre watershed

1. Runoff curve number (CN)

Soil name and hydrologic group (appendix A)	Cover description (cover type, treatment, and hydrologic condition; percent impervious; unconnected/connected impervious area ratio)	CN 1/			Area ☐acres ☐mi² ☐%	Product of CN x area
		Table 2-2	Fig. 2-3	Fig. 2-4		
B	Woods - good condition	55			12.75	701.25
C	Woods- good condition	70			6.0	420.0
B	Agriculture- SR + CR good Condition	75			6.25	468.75
				Totals =	25.0	1590.0

1/ Use only one CN source per line.

CN (weighted) = $\frac{\text{total product}}{\text{total area}} = \frac{1590}{25} = 63.6$; Use CN = $\boxed{64}$

2. Runoff

		Storm #1	Storm #2	Storm #3
Frequency	yr	2	10	
Rainfall, P (24-hour)	in	3.5	5.5	
Runoff, Q .... (Use P and CN with table 2-1, fig. 2-1, or eqs. 2-3 and 2-4.)	in	0.70	1.91	

Worksheet 3: Time of concentration (T_c) or travel time (T_t)

Project EXAMPLE 6.1A By TSB Date 5-96

Location ______ Checked ______ Date ______

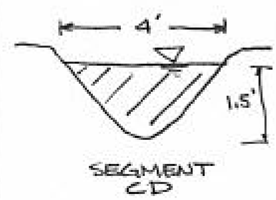
Circle one: Present Developed ENTIRE 25 acre watershed

Circle one: T_c T_t through subarea - longest flow path

NOTES: Space for as many as two segments per flow type can be used for each worksheet.

Include a map, schematic, or description of flow segments.

Sheet flow (Applicable to T_c only)	Segment ID	AB			
1. Surface description (table 3-1)		woods - light			
2. Manning's roughness coeff., n (table 3-1) ..		0.40			
3. Flow length, L (total L $\leq$ 300 ft)	ft	300			
4. Two-yr 24-hr rainfall, P_2	in	3.5			
5. Land slope, s	ft/ft	.025			
6. $T_t = \frac{0.007\ (nL)^{0.8}}{P_2^{0.5}\ s^{0.4}}$ Compute T_t	hr	0.75	+		= 0.75
Shallow concentrated flow	Segment ID	BC			
7. Surface description (paved or unpaved)		Unpaved			
8. Flow length, L	ft	600			
9. Watercourse slope, s	ft/ft	.045			
10. Average velocity, V (figure 3-1)	ft/s	3.4			
11. $T_t = \frac{L}{3600\ V}$ Compute T_t	hr	0.05	+		= 0.05
Channel flow	Segment ID	CD			
12. Cross sectional flow area, a	ft^2	3.0			
13. Wetted perimeter, p_w	ft	5.0			
14. Hydraulic radius, $r = \frac{a}{p_w}$ Compute r	ft	0.6			
15. Channel slope, s	ft/ft	.045			
16. Manning's roughness coeff., n		.05			
17. $V = \frac{1.49\ r^{2/3}\ s^{1/2}}{n}$ Compute V	ft/s	4.7			
18. Flow length, L	ft	1200			
19. $T_t = \frac{L}{3600\ V}$ Compute T_t	hr	0.07	+		= 0.07
20. Watershed or subarea T_c or T_t (add T_t in steps 6, 11, and 19)	hr				0.87



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Worksheet 4: Graphical Peak Discharge method

Project EXAMPLE 6.1 A By TSB Date 5-96

Location ______ Checked ____ Date ______

Circle one: Present Developed ENTIRE 25 acre watershed

1. Data:

Drainage area .... A_m = .039 mi^2 (acres/640)

Runoff curve number .... CN = 64 (From worksheet 2)

Time of concentration .. T_c = 0.87 hr (From worksheet 3)

Rainfall distribution type = II (I, IA, II, III)

Pond and swamp areas spread throughout watershed ...... = — percent of A_m (____ acres or mi^2 covered)

		Storm #1	Storm #2	Storm #3
2. Frequency	yr	2	10	
3. Rainfall, P (24-hour)	in	3.5	5.5	
4. Initial abstraction, I_a .... (Use CN with table 4-1.)	in	1.125	1.125	
5. Compute I_a/P		0.32	0.20	
6. Unit peak discharge, q_u .... (Use T_c and I_a/P with exhibit 4-____)	csm/in	310	360	
7. Runoff, Q .... (From worksheet 2).	in	0.70	1.91	
8. Pond and swamp adjustment factor, F_p .... (Use percent pond and swamp area with table 4-2. Factor is 1.0 for zero percent pond and swamp area.)		—	—	
9. Peak discharge, q_p .... (Where $q_p = q_u A_m Q F_p$)	cfs	8.5	26.8	

Worksheet 2: Runoff curve number and runoff

Project EXAMPLE 6.1A By TSB Date 5-96

Location ______ Checked ____ Date ____

Circle one: Present Developed ENTIRE 25 acre watershed

1. Runoff curve number (CN)

Soil name and hydrologic group (appendix A)	Cover description (cover type, treatment, and hydrologic condition; percent impervious; unconnected/connected impervious area ratio)	CN 1/ Table 2-2	CN 1/ Fig. 2-3	CN 1/ Fig. 2-4	Area ☐acres ☐mi^2 ☐%	Product of CN x area
B	Woods - Good Condition	55			7.1	390.5
C	Woods - Good Condition	70			6.0	420.0
B	Commercial / Industrial 78% IMP.	90			11.9	1071.0
				Totals =	25.0	1881.5

1/ Use only one CN source per line.

CN (weighted) = $\frac{\text{total product}}{\text{total area}} = \frac{1881.5}{25.0} = 75.3$; Use CN = 75

2. Runoff

		Storm #1	Storm #2	Storm #3
Frequency	yr	2	10	
Rainfall, P (24-hour)	in	3.5	5.5	
Runoff, Q (Use P and CN with table 2-1, fig. 2-1, or eqs. 2-3 and 2-4.)	in	1.30	2.85	

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Worksheet 3: Time of concentration (T_c) or travel time (T_t)

Project EXAMPLE 6.1 A By JSB Date 5-96

Location ______ Checked ____ Date ______

Circle one: Present Developed

Circle one: T_c T_t through subarea

ENTIRE 25 acre watershed - longest flow path

NOTES: Space for as many as two segments per flow type can be used for each worksheet.

Include a map, schematic, or description of flow segments.

Sheet flow (Applicable to T_c only)		Segment ID	AB		
1. Surface description (table 3-1)			Woods light		
2. Manning's roughness coeff., n (table 3-1)			0.40		
3. Flow length, L (total L $\le$ 300 ft)	ft		300		
4. Two-yr 24-hr rainfall, P_2	in		3.5		
5. Land slope, s	ft/ft		.025		
6. $T_t = \frac{0.007\ (nL)^{0.8}}{P_2^{0.5}\ s^{0.4}}$ Compute T_t	hr		0.75	+	= 0.75

Shallow concentrated flow		Segment ID	BC		
7. Surface description (paved or unpaved)			unpaved		
8. Flow length, L	ft		100'		
9. Watercourse slope, s	ft/ft		.045		
10. Average velocity, V (figure 3-1)	ft/s		3.4		
11. $T_t = \frac{L}{3600\ V}$ Compute T_t	hr		0.01	+	= 0.01

Channel flow		Segment ID	CD	DE	
12. Cross sectional flow area, a	ft^2		PIPE	3.0	
13. Wetted perimeter, p_w	ft		FLOW	5.0	
14. Hydraulic radius, $r = \frac{a}{p_w}$ Compute r	ft			0.6	
15. Channel slope, s	ft/ft			.045	
16. Manning's roughness coeff., n				.05	
17. $V = \frac{1.49\ r^{2/3}\ s^{1/2}}{n}$ Compute V	ft/s		AVG. 5.3	4.7	
18. Flow length, L	ft		500	1200	
19. $T_t = \frac{L}{3600\ V}$ Compute T_t	hr		0.03	+ 0.07	= 0.10
20. Watershed or subarea T_c or T_t (add T_t in steps 6, 11, and 19)	hr				0.86

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Worksheet 4: Graphical Peak Discharge method

Project EXAMPLE 6.1 A By TSB Date 5-96

Location ______ Checked ____ Date ______

Circle one: Present **Developed** ENTIRE 25 acre watershed (longest flow path)

1. Data:

   Drainage area .... A_m = .039 mi^2 (acres/640)

   Runoff curve number .... CN = 75 (From worksheet 2)

   Time of concentration .. T_c = 0.86 hr (From worksheet 3)

   Rainfall distribution type = II (I, IA, II, III)

   Pond and swamp areas spread throughout watershed ...... = — percent of A_m (____ acres or mi^2 covered)

		Storm #1	Storm #2	Storm #3
2. Frequency	yr	2	10	
3. Rainfall, P (24-hour)	in	3.5	5.5	
4. Initial abstraction, I_a .... (Use CN with table 4-1.)	in	0.667	0.667	
5. Compute I_a/P		0.19	0.12	
6. Unit peak discharge, q_u .... (Use T_c and I_a/P with exhibit 4-____)	csm/in	360	385	
7. Runoff, Q .... (From worksheet 2).	in	1.30	2.85	
8. Pond and swamp adjustment factor, F_p .... (Use percent pond and swamp area with table 4-2. Factor is 1.0 for zero percent pond and swamp area.)		—	—	
9. Peak discharge, q_p .... (Where $q_p = q_u A_m Q F_p$)	cfs	18.3	42.7	

Worksheet 3: Time of concentration (T_c) or travel time (T_t)

Project EXAMPLE 6.1 A By TSB Date 5-96

Location ______ Checked ______ Date ______

Circle one: Present **Developed**

Circle one: T_c T_t through subarea

ENTIRE 25 acre watershed flow path representative of peak.

NOTES: Space for as many as two segments per flow type can be used for each worksheet.

Include a map, schematic, or description of flow segments.

Sheet flow (Applicable to T_c only)

	Unit	Segment 1	Segment 2	Total
Segment ID		AB		
1. Surface description (table 3-1)		woods light		
2. Manning's roughness coeff., n (table 3-1)		0.40		
3. Flow length, L (total L ≤ 300 ft)	ft	100		
4. Two-yr 24-hr rainfall, P_2	in	3.5		
5. Land slope, s	ft/ft	.035		
6. $T_t = \frac{0.007\ (nL)^{0.8}}{P_2^{0.5}\ s^{0.4}}$ Compute T_t	hr	0.27	+	= 0.27

Shallow concentrated flow

	Unit	Segment 1	Segment 2	Total
Segment ID		BC		
7. Surface description (paved or unpaved)		Paved		
8. Flow length, L	ft	460		
9. Watercourse slope, s	ft/ft	0.03		
10. Average velocity, V (figure 3-1)	ft/s	3.5		
11. $T_t = \frac{L}{3600\ V}$ Compute T_t	hr	0.04	+	= 0.04

Channel flow

	Unit	Segment 1	Segment 2	Total
Segment ID		CD		
12. Cross sectional flow area, a	ft^2	Pipe		
13. Wetted perimeter, p_w	ft	Flow		
14. Hydraulic radius, $r = \frac{a}{p_w}$ Compute r	ft			
15. Channel slope, s	ft/ft			
16. Manning's roughness coeff., n				
17. $V = \frac{1.49\ r^{2/3}\ s^{1/2}}{n}$ Compute V	ft/s	Avg. 4.7		
18. Flow length, L	ft	700		
19. $T_t = \frac{L}{3600\ V}$ Compute T_t	hr	0.04	+	= 0.04
20. Watershed or subarea T_c or T_t (add T_t in steps 6, 11, and 19)	hr			0.35

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Worksheet 4: Graphical Peak Discharge method

Project EXAMPLE 6.1 A By ____ Date ________

Location ________________ Checked ____ Date ________

Circle one: Present Developed ENTIRE 25 acre watershed using most representative t_c flow path.

1. Data:

Drainage area .... A_m = .039 mi^2 (acres/640)

Runoff curve number .... CN = 75 (From worksheet 2)

* Time of concentration .. T_c = 0.35 hr (From worksheet 3) *

Rainfall distribution type = II (I, IA, II, III)

Pond and swamp areas spread throughout watershed ...... = — percent of A_m (____ acres or mi^2 covered)

		Storm #1	Storm #2	Storm #3
2. Frequency	yr	2	10	
3. Rainfall, P (24-hour)	in	3.5	5.5	
4. Initial abstraction, I_a .... (Use CN with table 4-1.)	in	0.667	0.667	
5. Compute I_a/P		0.19	0.12	
6. Unit peak discharge, q_u .... (Use T_c and I_a/P with exhibit 4-____)	csm/in	590	635	
7. Runoff, Q .... (From worksheet 2).	in	1.30	2.85	
8. Pond and swamp adjustment factor, F_p .... (Use percent pond and swamp area with table 4-2. Factor is 1.0 for zero percent pond and swamp area.)				
9. Peak discharge, q_p .... (Where $q_p = q_u A_m Q F_p$)	cfs	29.9	70.6	

EXAMPLE 6.1B

The Rational Method can be applied to any given watershed to find the peak discharge at a desired study point. As discussed in **Chapter 4**, the Rational Method is most accurate for watersheds having drainage areas of 20 acres or less and t_c flow paths of 20 minutes or less. The watershed in this example will be analyzed using the Rational Method, although it exceeds these limits, to provide a comparison to the SCS Methods of **Part A**. The decision to use a particular method is based on the watershed conditions (size, type of land cover, etc.) and the desired output (the peak rate of runoff, or a runoff hydrograph). For further discussion of the various methods, see **Chapter 4**.

Given:

The watershed in **Part A** of this example, as presented in **Figure 6.1-1a,c**.

Find:

The pre- and post-developed peak discharge using the Rational Method and the post-developed time of concentration flow path as described in **Part A** of this example.

Solution:

The rational method is applied to the 25-acre watershed as follows:

Pre-developed weighted runoff coefficient C:

Land Use	**Area *A* (*ac.*)**	**Runoff Coefficient *C***	***C x A***
Agriculture	6.25	0.6	3.75
Woods	18.75	0.3	5.63
TOTAL	25.0		9.38

$$\text{Weighted runoff coefficient } C = \frac{9.38}{25.0} \text{ ' } 0.38$$

Post-developed weighted runoff coefficient C:

Land Use	Area *A* (*ac.*)	Runoff Coefficient *C*	*C x A*
Industrial/ Commercial	11.9	0.9	10.71
Woods	13.1	0.3	3.93
TOTAL	25.0		14.64

$$\text{Weighted runoff coefficient } C = \frac{14.64}{25.0} \approx 0.59$$

TABLE 6.1 - 2
Hydrologic Summary - Full Watershed, Example 6.1B
Rational Method Peak Discharge, Q=CIA

Condition	Area *A* (*ac.*)	Runoff Coefficient *C*	t_c* (*min.*)	*I*** (*in/hr*)	Q_2*** (*cfs*)	Q_{10}*** (*cfs*)
Pre-developed	25.0	0.38	52 .87 *hr.*	I_2 = 1.8 I_{10} = 2.5	17	24
Post-developed	25.0	0.59	21 .35 *hr.*	I_2 = 3.3 I_{10} = 4.4	49	65

* *t_c based on SCS methods, Part A*
** *Intensity from Richmond area IDF curve*
*** *Rational Method equation: Q=CIA*

Comparing these results with those using SCS methods for the same watershed [**Table 6.1-1**, Post Developed t_c Path (b)], the Rational Method gives a significant increase in the 2-year pre- and post-developed peak discharges but gives similar discharges for the 10-year storm.

EXAMPLE 6.1C

Example 6.1C divides the given 25-acre watershed into two sub-watersheds. The TR-55 Tabular Method is used to generate their hydrographs, which are then added at the study point.

Given:

The proposed development disturbs 11.9 acres. Ten acres drain through subwatershed 1 and 1.9 acres drain through subwatershed 2.

Find:

The peak discharge from the watershed by adding the runoff hydrographs from the two sub-watersheds at the study point.

Solution:

The 25-acre watershed is divided into Sub-watersheds 1 and 2 based on pre- and post-developed land uses and drainage divides, as shown in **Figure 6.1-2**. To obtain the peak discharge from the total watershed, hydrographs must be generated for each sub-watershed and then added at the study point. The SCS Tabular Method (or any other hydrologic computer program that generates a runoff hydrograph) should be used.

The results, using the SCS TR-55 Tabular Hydrograph Method, are summarized in **Table 6.1-3**. Completed TR-55 worksheets are included at the end of this example.

Referring to the TR-55 Tabular Method Worksheets, note that the peak discharge obtained from adding the two hydrographs is less than the sum of their individual peaks. This is due to the timing effect of the peak flow through the watershed. Sometimes, the peak discharge will decrease with development because of the decreased flow time. Also note that the peak discharge hydrograph for the developed area travels through the study point before discharge from the other subwatershed(s). For additional examples and discussion on the Tabular Method, refer to TR-55.

TABLE 6.1 - 3
Hydrologic Summary - Sub-watersheds, Example 6.1C
SCS TR-55 Tabular Hydrograph Analysis

Sub-watershed 1					
Condition	**Area (*ac.*)**	**RCN**	t_c **(*hrs.*)**	Q_2 **(*cfs*)**	Q_{10} **(*cfs*)**
Pre-developed	9.5	68	.74	4.7	14.4
Post-developed	12.0	84	.35	21.9	42.0
Sub-watershed 2					
Condition	**Area (*acres*)**	***RCN***	t_c **(*hrs.*)**	Q_2 **(*cfs*)**	Q_{10} **(*cfs*)**
Pre-developed	15.5	61	.87	4.8	13.9
Post-developed	13.0	67	.86	5.9	18.2
Sub-watershed 1 and 2 Composite Hydrograph					
Condition	-			Q_2 **(*cfs*)**	Q_{10} **(*cfs*)**
Pre-developed	-			9.5	27.8
Post-developed	-			24.2	53.4

FIGURE 6.1-2a
Example 6.1C Sub-watersheds 1 & 2 Pre-developed Condition

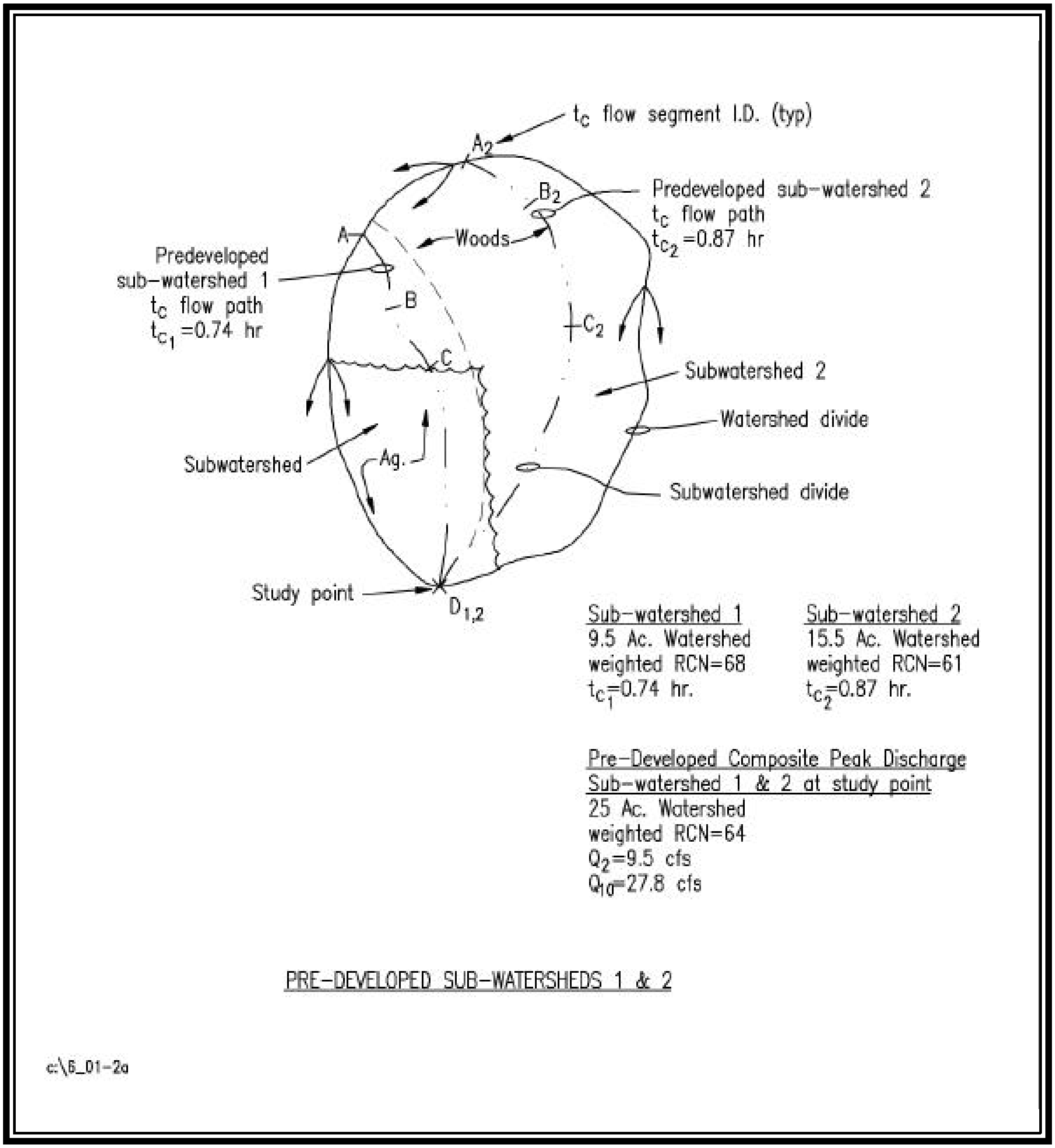
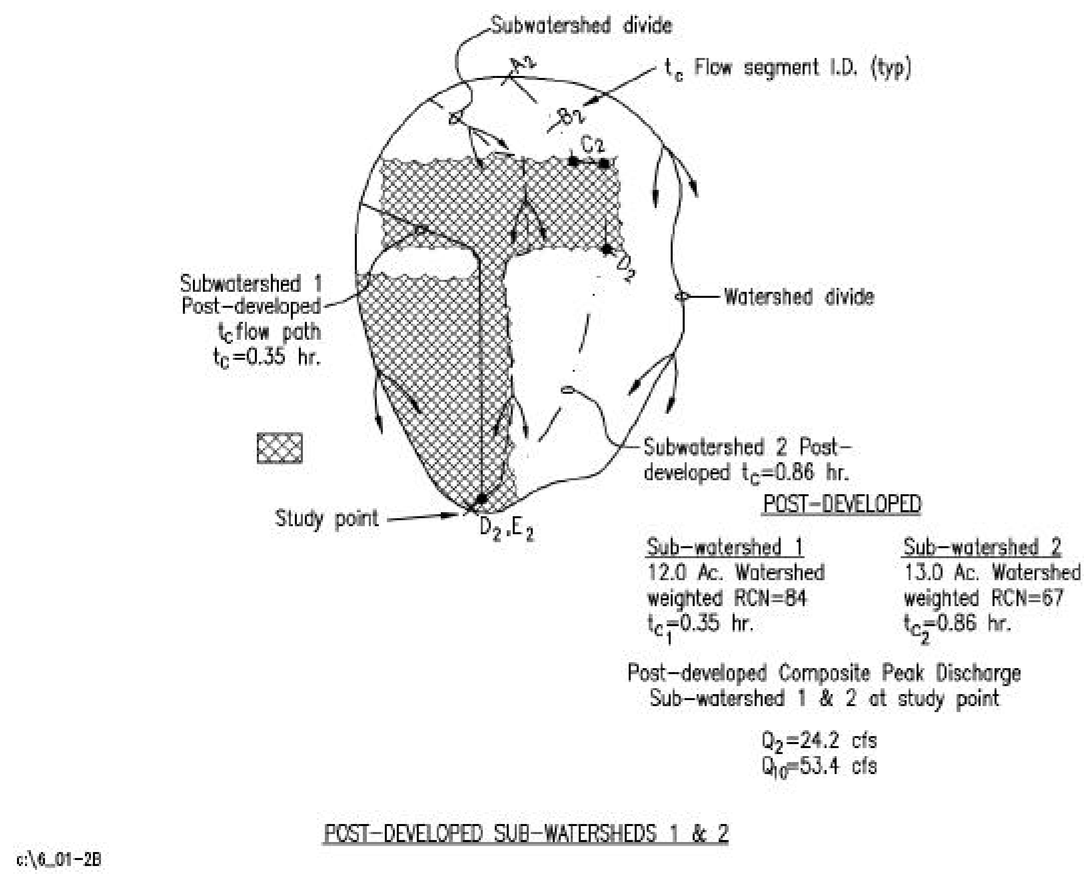


FIGURE 6.1-2b
Example 6.1C Sub-watersheds 1 & 2 Post-developed Condition



Worksheet 2: Runoff curve number and runoff

Project EXAMPLE 6.1 C By TSB Date 5-96

Location ______ Checked ____ Date ____

Circle one: Present Developed Sub-watershed 1

1. Runoff curve number (CN)

Soil name and hydrologic group (appendix A)	Cover description (cover type, treatment, and hydrologic condition; percent impervious; unconnected/connected impervious area ratio)	CN 1/ Table 2-2	Fig. 2-3	Fig. 2-4	Area ☐acres ☐mi² ☐%	Product of CN x area
B	Woods - good Condition	55			3.25	178.75
B	Agriculture - SR + CR good Condition	75			6.25	468.75
				Totals =	9.5	647.5

1/ Use only one CN source per line.

CN (weighted) = $\frac{\text{total product}}{\text{total area}} = \frac{647.5}{9.5} =$ 68.1; Use CN = $\boxed{68}$

2. Runoff

		Storm #1	Storm #2	Storm #3
Frequency	yr	2	10	
Rainfall, P (24-hour)	in	3.5	5.5	
Runoff, Q (Use P and CN with table 2-1, fig. 2-1, or eqs. 2-3 and 2-4.)	in	0.90	2.24	

D-2 (210-VI-TR-55, Second Ed., June 1986)

Worksheet 2: Runoff curve number and runoff

Project EXAMPLE 6.1C By TSB Date 6-96

Location ______ Checked ______ Date ______

Circle one: Present Developed Sub-watershed 2

1. Runoff curve number (CN)

Soil name and hydrologic group (appendix A)	Cover description (cover type, treatment, and hydrologic condition; percent impervious; unconnected/connected impervious area ratio)	CN 1/ Table 2-2	CN 1/ Fig. 2-3	CN 1/ Fig. 2-4	Area ☐ acres ☐ mi² ☐ %	Product of CN x area
B	Woods-good condition	55			9.5	522.5
C	Woods-good condition	70			6.0	420.0
				Totals =	15.5	942.5

1/ Use only one CN source per line.

CN (weighted) = $\frac{\text{total product}}{\text{total area}} = \frac{942.5}{15.5} = 60.8$; Use CN = $\boxed{61}$

2. Runoff

		Storm #1	Storm #2	Storm #3
Frequency	yr	2	10	
Rainfall, P (24-hour)	in	3.5	5.5	
Runoff, Q .... (Use P and CN with table 2-1, fig. 2-1, or eqs. 2-3 and 2-4.)	in	0.57	1.68	

D-2 (210-VI-TR-55, Second Ed., June 1986)

Worksheet 3: Time of concentration (T_c) or travel time (T_t)

Project EXAMPLE 6.1C By TSB Date 6-96

Location ______ Checked ______ Date ______

Circle one: Present Developed Sub-watershed 1

Circle one: T_c T_t through subarea

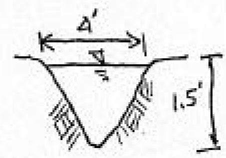
NOTES: Space for as many as two segments per flow type can be used for each worksheet.

Include a map, schematic, or description of flow segments.

Sheet flow (Applicable to T_c only)	Unit	Segment ID: AB			
1. Surface description (table 3-1)		Woods light			
2. Manning's roughness coeff., n (table 3-1)		0.40			
3. Flow length, L (total L ≤ 300 ft)	ft	300			
4. Two-yr 24-hr rainfall, P_2	in	3.5			
5. Land slope, s	ft/ft	.035			
6. $T_t = \frac{0.007\,(nL)^{0.8}}{P_2^{0.5}\,s^{0.4}}$ Compute T_t	hr	0.66	+	=	0.66

Shallow concentrated flow	Unit	Segment ID: BC			
7. Surface description (paved or unpaved)		unpaved			
8. Flow length, L	ft	260			
9. Watercourse slope, s	ft/ft	.035			
10. Average velocity, V (figure 3-1)	ft/s	3.0			
11. $T_t = \frac{L}{3600\,V}$ Compute T_t	hr	0.04	+	=	0.04

Channel flow	Unit	Segment ID: CD			
12. Cross sectional flow area, a	ft^2	3.0			
13. Wetted perimeter, p_w	ft	5.0			
14. Hydraulic radius, $r = \frac{a}{p_w}$ Compute r	ft	0.6			
15. Channel slope, s	ft/ft	.045			
16. Manning's roughness coeff., n		.05			
17. $V = \frac{1.49\,r^{2/3}\,s^{1/2}}{n}$ Compute V	ft/s	4.7			
18. Flow length, L	ft	700			
19. $T_t = \frac{L}{3600\,V}$ Compute T_t	hr	0.04	+	=	0.04
20. Watershed or subarea T_c or T_t (add T_t in steps 6, 11, and 19)	hr				0.74



Worksheet 3: Time of concentration (T_c) or travel time (T_t)

Project EXAMPLE 6.1 C By TSB Date 6-96

Location ________ Checked ____ Date ______

Circle one: Present Developed Sub-watershed 2

Circle one: T_c T_t through subarea

NOTES: Space for as many as two segments per flow type can be used for each worksheet.

Include a map, schematic, or description of flow segments.

Sheet flow (Applicable to T_c only)

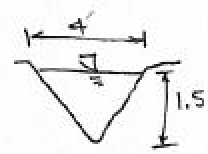
	Unit	Segment ID: AB			
1. Surface description (table 3-1)		woods light			
2. Manning's roughness coeff., n (table 3-1)		0.40			
3. Flow length, L (total L ≤ 300 ft)	ft	300			
4. Two-yr 24-hr rainfall, P_2	in	3.5			
5. Land slope, s	ft/ft	.025			
6. $T_t = \frac{0.007\ (nL)^{0.8}}{P_2^{0.5}\ s^{0.4}}$ Compute T_t	hr	0.75	+	=	0.75

Shallow concentrated flow

	Unit	Segment ID: BC			
7. Surface description (paved or unpaved)		Unpaved			
8. Flow length, L	ft	600			
9. Watercourse slope, s	ft/ft	.045			
10. Average velocity, V (figure 3-1)	ft/s	3.4			
11. $T_t = \frac{L}{3600\ V}$ Compute T_t	hr	.05	+	=	0.05

Channel flow

	Unit	Segment ID: CD			
12. Cross sectional flow area, a	ft^2	3.0			
13. Wetted perimeter, p_w	ft	5.0			
14. Hydraulic radius, $r = \frac{a}{p_w}$ Compute r	ft	0.6			
15. Channel slope, s	ft/ft	.045			
16. Manning's roughness coeff., n		.05			
17. $V = \frac{1.49\ r^{2/3}\ s^{1/2}}{n}$ Compute V	ft/s	4.7			
18. Flow length, L	ft	1200			
19. $T_t = \frac{L}{3600\ V}$ Compute T_t	hr	.07	+	=	0.07
20. Watershed or subarea T_c or T_t (add T_t in steps 6, 11, and 19)	hr				0.87



Worksheet 5a: Basic watershed data

Project EXAMPLE 6.1C Location ______ By TSB Date 6-96

Circle one: Present Developed Sub-watersheds 1 & 2 Frequency (yr) 2, 10 Checked ______ Date ______

Subarea name	Drainage area A_m (mi^2)	Time of concentration T_c (hr)	Travel time through subarea T_t (hr)	Downstream subarea names	Travel time summation to outlet ΣT_t (hr)	24-hr Rainfall P (in)	Runoff curve number CN	Runoff Q (in)	A_mQ (mi^2-in)	Initial abstraction I_a (in)	I_a/P	
1	.015	0.74				3.5	68	0.90	.0135	.941	.27	2-yr
2	.024	0.87				3.5	61	0.57	.0137	1.279	.36	
1	.015	0.74				5.5	68	2.24	.034	.941	.17	10-yr
2	.024	0.87				5.5	61	1.68	.040	1.279	.23	

From worksheet 3 (Time of concentration, Travel time through subarea) — From worksheet 2 (Runoff curve number, Runoff) — From table 5-1 (Initial abstraction)

Worksheet 5b: Tabular hydrograph discharge summary

Project EXAMPLE 6.1 C Location ______ By TSB Date 6-96

Circle one: Present Developed Sub-watersheds 1 & 2 Frequency (yr) 2, 10 Checked ______ Date ______

Subarea name	Basic watershed data used 1/ Sub-area T_c (hr)	ΣT_t to outlet (hr)	I_a/P	A_mQ (mi^2-in)	Select and enter hydrograph times in hours from exhibit 5-II 2/ 12.0	12.1	12.2	12.3	12.4	12.5	12.6	12.7	12.8	13.0	13.2	13.4
					Discharges at selected hydrograph times 3/ (cfs)											
1	.75		.30	.0135	0	0.1	0.4	1.2	2.4	3.6	4.4	4.7	4.4	3.3	2.4	1.9
2	.75		.30	.0137	0	0.1	0.4	1.2	2.4	3.6	4.5	4.8	4.5	3.4	2.5	1.9
2-yr	Composite		hydrograph		0	0.2	0.8	2.4	4.8	7.2	8.9	9.5	8.9	6.7	4.9	3.8
1	.75		.10	.034	1.6	2.3	3.9	6.6	10.0	12.9	14.4	13.9	12.5	8.6	5.8	4.2
2	.75		.30	.040	0	0.2	1.2	3.4	7.0	10.6	13.0	13.9	13.1	9.8	7.2	5.5
10-yr	Composite		hydrograph		1.6	2.5	5.1	10.0	17.0	23.5	27.4	27.8	25.6	18.4	13.0	9.7
Composite hydrograph at outlet																

1/ Worksheet 5a. Rounded as needed for use with exhibit 5.
2/ Enter rainfall distribution type used.
3/ Hydrograph discharge for selected times is A_mQ multiplied by tabular discharge from appropriate exhibit 5.

T_c, T_t Rounding

AREA 1		✓ 1	2	3	AREA 2	✓ 1	2	3
t_c	.74	.75	.5	.75	.87	.75	.75	1.0
t_t	0	0	0	0	0	0	0	0
SUM	.74	.75	.5	.75	.87	.75	.75	1.0

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Worksheet 2: Runoff curve number and runoff

Project EXAMPLE 6.1C By TSB Date 6-96

Location ______ Checked ____ Date ______

Circle one: Present **Developed** Sub-watershed 1

1. Runoff curve number (CN)

Soil name and hydrologic group (appendix A)	Cover description (cover type, treatment, and hydrologic condition; percent impervious; unconnected/connected impervious area ratio)	CN 1/ Table 2-2	CN 1/ Fig. 2-3	CN 1/ Fig. 2-4	Area ☐acres ☐mi² ☐%	Product of CN x area
B	Woods - good condition	55			2.0	110.0
B	Commercial / Industrial 78% Imp.	90			10.0	900.0
				Totals =	12.0	1010.0

1/ Use only one CN source per line.

CN (weighted) = $\frac{\text{total product}}{\text{total area}} = \frac{1010}{12.0} = 84.16$; Use CN = $\boxed{84}$

2. Runoff

		Storm #1	Storm #2	Storm #3
Frequency	yr	2	10	
Rainfall, P (24-hour)	in	3.5	5.5	
Runoff, Q (Use P and CN with table 2-1, fig. 2-1, or eqs. 2-3 and 2-4.)	in	1.94	3.73	

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Worksheet 3: Time of concentration (T_c) or travel time (T_t)

Project EXAMPLE 6.1 C By TSB Date 6-96

Location ______ Checked ____ Date ____

Circle one: Present **Developed** Sub-watershed 1

Circle one: T_c T_t through subarea

NOTES: Space for as many as two segments per flow type can be used for each worksheet.

Include a map, schematic, or description of flow segments.

Sheet flow (Applicable to T_c only)		Segment ID	AB			
1.	Surface description (table 3-1)		woods light			
2.	Manning's roughness coeff., n (table 3-1)		.40			
3.	Flow length, L (total L ≤ 300 ft)	ft	100			
4.	Two-yr 24-hr rainfall, P_2	in	3.5			
5.	Land slope, s	ft/ft	.035			
6.	$T_t = \frac{0.007\ (nL)^{0.8}}{P_2^{0.5}\ s^{0.4}}$ Compute T_t	hr	0.27	+		= 0.27
Shallow concentrated flow		Segment ID	BC			
7.	Surface description (paved or unpaved)		unpaved			
8.	Flow length, L	ft	460			
9.	Watercourse slope, s	ft/ft	.035			
10.	Average velocity, V (figure 3-1)	ft/s	3.0			
11.	$T_t = \frac{L}{3600\ V}$ Compute T_t	hr	0.04	+		= 0.04
Channel flow		Segment ID	CP			
12.	Cross sectional flow area, a	ft^2	3.0			
13.	Wetted perimeter, p_w	ft	5.0			
14.	Hydraulic radius, $r = \frac{a}{p_w}$ Compute r	ft	0.6			
15.	Channel slope, s	ft/ft	.04			
16.	Manning's roughness coeff., n		.05			
17.	$V = \frac{1.49\ r^{2/3}\ s^{1/2}}{n}$ Compute V	ft/s	4.7			
18.	Flow length, L	ft	700			
19.	$T_t = \frac{L}{3600\ V}$ Compute T_t	hr	0.04	+		= 0.04
20.	Watershed or subarea T_c or T_t (add T_t in steps 6, 11, and 19)	hr				0.35

Worksheet 2: Runoff curve number and runoff

Project EXAMPLE 6.1C By TSB Date 6-96

Location ______ Checked ______ Date ______

Circle one: Present Developed Sub-watershed 2

1. Runoff curve number (CN)

Soil name and hydrologic group (appendix A)	Cover description (cover type, treatment, and hydrologic condition; percent impervious; unconnected/connected impervious area ratio)	CN 1/ Table 2-2	CN 1/ Fig. 2-3	CN 1/ Fig. 2-4	Area ☐acres ☐mi² ☐%	Product of CN x area
B	Woods - good condition	55			5.1	280.5
C	woods-good condition	70			6.0	420.0
B	Commercial/Industrial 78% Imp.	90			1.9	171.0
				Totals =	13.0	871.5

1/ Use only one CN source per line.

CN (weighted) = $\frac{\text{total product}}{\text{total area}} = \frac{871.5}{13.0} = 67.04$; Use CN = $\boxed{67}$

2. Runoff

		Storm #1	Storm #2	Storm #3
Frequency	yr	2	10	
Rainfall, P (24-hour)	in	3.5	5.5	
Runoff, Q .... (Use P and CN with table 2-1, fig. 2-1, or eqs. 2-3 and 2-4.)	in	0.85	2.16	

D-2 (210-VI-TR-55, Second Ed., June 1986)

Worksheet 3: Time of concentration (T_c) or travel time (T_t)

Project EXAMPLE 6.1 C By TSB Date 6-96

Location ______ Checked ______ Date ______

Circle one: Present Developed Sub-watershed 2 -

Circle one: T_c T_t through subarea

NOTES: Space for as many as two segments per flow type can be used for each worksheet.

Include a map, schematic, or description of flow segments.

Sheet flow (Applicable to T_c only)	Unit	Segment 1		Segment 2		Total
Segment ID		AB				
1. Surface description (table 3-1)		Woods light				
2. Manning's roughness coeff., n (table 3-1)		.40				
3. Flow length, L (total L $\leq$ 300 ft)	ft	300				
4. Two-yr 24-hr rainfall, P_2	in	3.5				
5. Land slope, s	ft/ft	.025				
6. $T_t = \frac{0.007 (nL)^{0.8}}{P_2^{0.5} s^{0.4}}$ Compute T_t	hr	0.75	+		=	0.75

Shallow concentrated flow	Unit	Segment 1		Segment 2		Total
Segment ID		BC				
7. Surface description (paved or unpaved)		Unpaved				
8. Flow length, L	ft	100'				
9. Watercourse slope, s	ft/ft	.045				
10. Average velocity, V (figure 3-1)	ft/s	3.4				
11. $T_t = \frac{L}{3600\ V}$ Compute T_t	hr	0.01	+		=	0.01

Channel flow	Unit	Segment 1		Segment 2		Total
Segment ID		CD		DE		
12. Cross sectional flow area, a	ft^2	Pipe		3.0		
13. Wetted perimeter, p_w	ft	Flow		5.0		
14. Hydraulic radius, $r = \frac{a}{p_w}$ Compute r	ft			0.6		
15. Channel slope, s	ft/ft			.04		
16. Manning's roughness coeff., n				.05		
17. $V = \frac{1.49\ r^{2/3}\ s^{1/2}}{n}$ Compute V	ft/s	Avg. 5.3		4.7		
18. Flow length, L	ft	500		1200		
19. $T_t = \frac{L}{3600\ V}$ Compute T_t	hr	0.03	+	0.07	=	0.10
20. Watershed or subarea T_c or T_t (add T_t in steps 6, 11, and 19)	hr					0.86

(210-VI-TR-55, Second Ed., June 1986) D-3

Worksheet 5a: Basic watershed data

Project EXAMPLE 6.1C Location ______ By TSB Date 6-96

Circle one: Present Developed Sub-watersheds 1&2 Frequency (yr) 2,10 Checked ______ Date ______

Subarea name	Drainage area A_m (mi^2)	Time of concen-tration T_c (hr)	Travel time through subarea T_t (hr)	Downstream subarea names	Travel time summation to outlet ΣT_t (hr)	24-hr Rain-fall P (in)	Runoff curve number CN	Run-off Q (in)	A_mQ (mi^2-in)	Initial abstrac-tion I_a (in)	I_a/P	
1	.019	0.35				3.5	84	1.94	.037	.381	0.11	2-yr
2	.020	0.86				3.5	67	0.85	.017	.985	0.28	
1	.019	0.35				5.5	84	3.73	.071	.381	.07	10-yr
2	.020	0.86				5.5	67	2.16	.043	.985	.18	

From worksheet 3 (Time of concentration, Travel time through subarea) From worksheet 2 (Runoff curve number, Run-off) From table 5-1 (Initial abstraction)

Worksheet 5b: Tabular hydrograph discharge summary

Project EXAMPLE 6.1C Location ______ By TSB Date 6-96

Circle one: Present **Developed** Sub-watersheds 1 & 2 Frequency (yr) 2, 10 Checked ______ Date ______

Subarea name	Basic watershed data used 1/				Select and enter hydrograph times in hours from exhibit 5-II 2/											
	Sub-area T_c (hr)	ΣT_t to outlet (hr)	I_a/P	A_mQ (mi^2-in)	12.0	12.1	12.2	12.3	12.4	12.5	12.6	12.7	12.8	13.0	13.2	13.4
					Discharges at selected hydrograph times 3/ (cfs)											
1	.40		.10	.037	5.2	10.0	17.3	21.9	21.2	15.9	11.0	8.0	6.0	3.8	2.8	2.3
2	.75		.30	.017	0	0.1	0.5	1.5	3.0	4.5	5.5	5.9	5.6	4.2	3.1	2.3
2-yr	Composite	hydrograph			5.2	10.1	17.8	23.4	24.2	20.4	16.5	13.9	11.6	8.0	5.9	4.6
1	.40		.10	.071	10.0	19.2	33.2	42.0	40.8	30.6	21.2	15.3	11.6	7.4	5.5	4.5
2	.75		.10	.043	2.0	2.9	4.9	8.3	12.6	16.3	18.2	17.6	15.9	10.8	7.4	5.3
10-yr	Composite	hydrograph			12.0	22.1	38.1	50.3	53.4	46.9	39.4	32.9	27.5	18.2	12.9	9.8
Composite hydrograph at outlet																

1/ Worksheet 5a. Rounded as needed for use with exhibit 5.
2/ Enter rainfall distribution type used.
3/ Hydrograph discharge for selected times is A_mQ multiplied by tabular discharge from appropriate exhibit 5.

t_c, t_t Rounding

	AREA1	1 ✓	2	3	AREA2	1	2	3
T_c	.35	.4	.3	.4	.86	.75	.75	1.0
T_t	0	0	0	0	0	0	0	0
SUM	.35	.4	.3	.4	.86	.75	.75	1.0

EXAMPLE 6.1D

Using the Rational Method to analyze the full 25-acre watershed (**Example 6.1B**) yielded much higher peak discharges for the 2-year design storm then when using the Tabular Method in **Part C**. The analysis of Sub-watershed 1 using the Rational Method will provide a better opportunity to compare the two methods, since Sub-watershed 1 is less than the recommended maximum of 20 acres and the t_c is close to the recommended 20 minute upper limit.

Given:

Sub-watershed 1 as shown in **Figure 6.1-2** and described in **Part A**.

Find:

The peak discharge from Sub-watershed 1 using the Rational Method. Compare these results with those from **Part C**.

Solution:

The Rational Method is applied to Sub-watershed 1 as follows:

Sub-watershed 1:

Pre-developed weighted runoff coefficient C:

Land Use	**Area *A* (*ac.*)**	**Runoff Coefficient *C***	***C x A***
Agriculture	6.25	0.6	3.75
Woods	3.25	0.25	0.81
TOTAL	9.5		4.56

$$\text{Weighted runoff coefficient } C = \frac{4.56}{9.5} \approx 0.48$$

Post-developed weighted runoff coefficient C:

Land Use	Area *A* (*ac.)*	Runoff Coefficient *C*	*C x A*
Commercial/ Industrial	10.0	0.9	9.0
Woods	2.0	0.25	0.5
TOTAL	12.0		9.5

$$\text{Weighted runoff coefficient} = \frac{9.5}{12.0} \approx 0.79$$

TABLE 6.1 - 4
Hydrologic Summary - Sub-watershed 1, Example 6.1D
Rational Method Peak Discharge, Q = CIA

Condition	Area *A* (*ac.*)	Runoff Coefficient *C*	t_c* (*min.*)	*I*** (*in/hr*)	Q_2 (*cfs*)	Q_{10} (*cfs*)
Pre-developed	9.5	0.48	43.2	I_2 = 2.0; I_{10} = 2.8	9.1	12.8
Post-developed	12.0	0.79	21	I_2 = 3.3; I_{10} = 4.4	31.3	41.7

* t_c *based on SCS methods,* ***Part A***
** *Intensity from Richmond area I-D-F Curve,* ***Appendix 4D***

The results show a significant increase in both the pre- and post-developed 2-year storm peak discharges, and very similar pre- and post-developed 10-year storm peak discharges. This may be attributed to any one of several factors, however, there seems to be a consistent trend that the Rational Method over estimates the 2-year storm discharge when compared to the SCS methods.

Example 6.2

HYDRAULICS

EXAMPLE 6.2
ILLUSTRATIONS

#	FIGURES	PAGE

EXAMPLE 6.2

INTRODUCTION:

Example 6.2 will use the same hydrology for the 25-acre watershed that was presented in **Example 6.1** and illustrated in **Chapter 5**. A stormwater facility will be sized and designed to accept the runoff from the full 25-acre watershed. Note that it is usually most efficient to control stormwater quality and quantity within the subwatershed where the majority of the development occurs.

GIVEN:

The hydrology of the 25-acre watershed represented in **Example 6.1**, Part A, using the most representative flow path of the proposed development to determine the time of concentration. The summary of the hydrology (developed using the TR-20 computer program) is as follows:

Table 6.2-1
Hydrologic Summary*, *Example 6.1, SCS Methods

TR-20 COMPUTER RUN					
Condition	**DA**	**RCN**	t_c	Q_{10}	Q_{100}
PRE-DEV	25 *ac.*	64	0.87 *hr.*	25.5 *cfs**	52.7 *cfs*
POST-DEV	25 *ac.*	75	0.35 *hr.*	61.1 *cfs*	108.9 *cfs*

***** 10-year Allowable release rate***

The TR-20 hydrologic analysis input file and output summary are provided on the following pages.

FIND:

Design a stormwater management facility which provides 24 hour extended detention of the 1-year frequency design storm for channel erosion control, attenuation of the post-developed 10-year frequency design storm released at the pre-developed rate for flood control, and safe passage of the 100-year frequency design storm through a vegetated emergency spillway.

Note: There is no water quality enhancement required for this example. Also, extended detention of the runoff from the 1-year frequency storm is a calculation based on the *volume* of runoff rather than the *rate* of runoff. Therefore the hydrologic summary includes only the 10- and 100-year frequency storm peak rates of runoff.

SOLUTION:

Use the design procedures found in **Chapter 5-7** to design the basin and multi-stage riser structure. This is to include the design of the 1-year storm extended detention orifice in lieu of a 2-year control orifice. The design procedures for channel erosion extended detention are found in **Chapter 5-6.3**.

STEP 1 No water quality requirements for this facility.

STEP 2 Allowable release rates from the TR-20 computer run: $Q_{10\ allowable}$ = 25.5 cfs

STEP 3 The required storage volume for extended detention of the 1-year storm (V_{ce}):

1-year frequency design storm rainfall = 2.7" (**Appendix 4B**); 1-year frequency design storm runoff = 0.8" (**Appendix 4C**).
Runoff volume (25 ac.) (0.8") (1'/12") = V_{ce} = 1.66 ac.ft.

Note: The routing affect on the extended detention of the 1-year storm results in the actual use of approximately 60% of the design storage allocated in the basin (**Chapter 5-6.3**). Therefore, V_{ce} = (1.66 ac. ft.) (0.6) = 1.0 ac. ft. = 43,560 ft^3.

Required storage volume for 10-year flood control (V_{10}):

1. From TR-55: Storage Volume for Detention Basin **(Chapter 5-4.2)**:

$$Q_{o_{10}}/Q_{i_{10}} = 25.5 / 61.1 = 0.42$$

From **Figure 5-4**: $V_{s_{10}} / V_{r_{10}} = 0.31$

2. Runoff volume: $V_{r_{10}} = Q_{10} A_m 53.33$ = (2.85in.)(0.039 mi^2)(53.33) = 5.93 ac.ft.

3. Storage volume required: $$V_{s_{10}} = \left(\frac{V_{s_{10}}}{V_{r_{10}}}\right) V_{r_{10}} = (0.31)\ 5.93\ ac.ft. = \underline{1.84\ ac.ft.}$$

Note: Approximately 10% should be added to the required storage to account for the extended detention of the l-year storm within the 10-year design pool: (1.84 ac.ft.) (1.10) = 2.0 ac.ft.

STEP 4 The development of the stage-storage worksheet and curve, **Figure 6.2-1**, was presented in **Chapter 5-5.1**.

<u>STEP 5</u> The extended detention orifice (1-year storm) is designed using the procedure outlined in **Chapter 5-6.3** as follows:

1. V_{ce} = 1.0 ac.ft.
2. Elevation for 1-year h_{max} = 89.0 (1.0 ac.ft.). h_{max} = 89 - 81 = <u>8.0 ft.</u>.
3. $$Q_{avg} = \frac{43{,}560\ ft^3}{(24\,hr.)(3{,}600\,\text{sec.}/hr.)} = 0.5\ cfs$$

   $Q_{max} = 2 \times Q_{avg}\ cfs = 2 \times 0.5 cfs = \underline{1.0\ cfs}$

4. The required orifice area, a, in ft^2 is:

$$a = \frac{Q}{C\sqrt{2gh_{max}}}$$

Equation 5-7
Rearranged Orifice equation

$$a = \frac{1.0}{0.6\sqrt{(2)(32.2)(8.0)}} = 0.073\ ft^2 = \pi r^2 = \pi d^2/4$$

$$d = \sqrt{\frac{4a}{\pi}} = \sqrt{\frac{4(0.073\ ft^2)}{\pi}} = .304 ft. = \underline{3.7\text{ in.}}$$

5. Route the 1-year storm to establish the 1-year design water surface elevation (wse) by completing steps 6 and 7.
6. The stage-discharge relationship is as follows:

$$Q = Ca\sqrt{2gh}$$

Equation 5-6
Orifice equation

$$Q = 0.6(.073)\sqrt{(2)(32.2)(h)} = \underline{0.33\,(h)^{1/2}}$$

where h = wse − 81.0 ft.

FIGURE 6.2-1
Stage-Storage Worksheet

PROJECT: Example 6.2 SHEET ____ OF ____

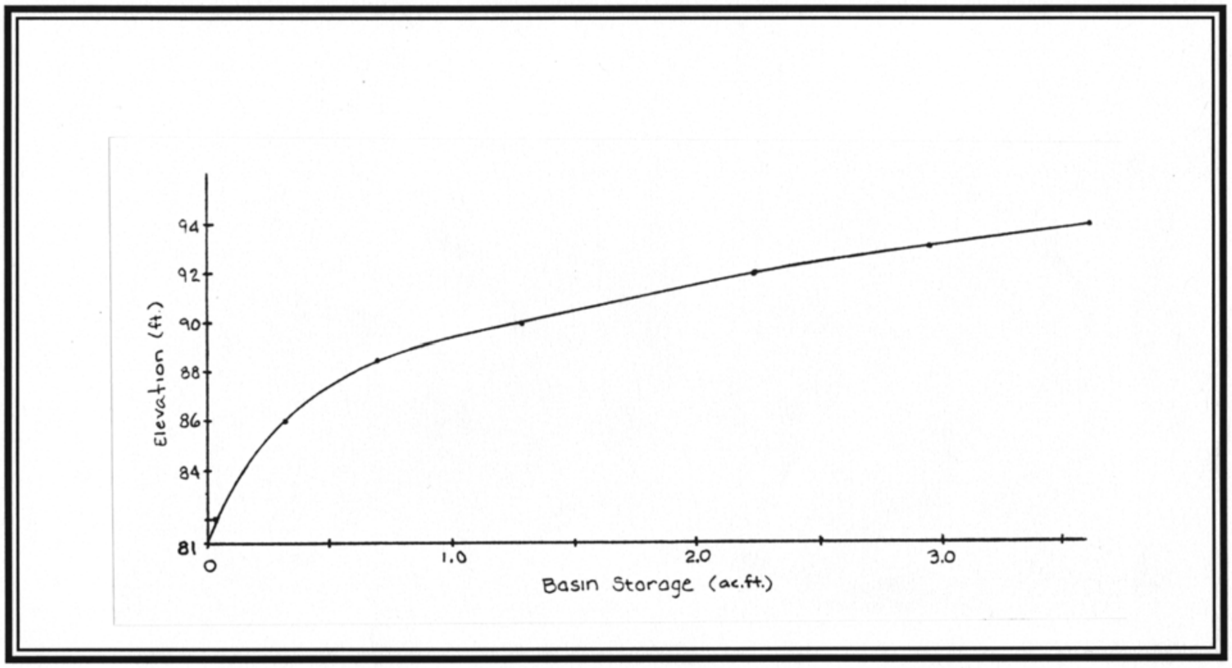
COUNTY: ____________ COMPUTED BY: ____________ DATE: ________

DESCRIPTION: 1-yr. extended detention, 10-yr. flood control, 100-yr. E.S.

ATTACH COPY OF TOPO: SCALE - 1" = ______ *ft.*

1	2	3	4	5	6	7	8
ELEV.	AREA *(in²)*	AREA *(ft²)*	AVG. AREA *(ft²)*	INTERVAL	VOL. *(ft³)*	TOTAL VOLUME *(ft³)*	TOTAL VOLUME *(ac.ft.)*
81	0	0				0	0
			900	1	900		
82	2.0	1800				900	0.02
			2520	2	5040		
84	3.6	3240				5940	0.14
			4207	2	8414		
86	5.7	5175				14354	0.33
			7614	2	15228		
88	11.2	10053				29582	0.68
			12991	2	25982		
90	17.7	15930				55564	1.28
			20700	2	41400		
92	28.3	25470				96964	2.23
			31102	1	31102		
93	40.8	36734				128066	2.94
			38105	1	38105		
94	43.9	39476				166171	3.81

FIGURE 6.2-1 contd.
Stage - Storage Curve



7. Complete a stage-storage-discharge table for the anticipated range of elevations using the stage-discharge relationship established in item 6 above. The completed extended detention portion of the stage-storage-discharge worksheet is presented in **Figure 6.2-2**.

In order to establish the 1-year extended detention water surface elevation, the designer may use the approximate value of 89.0 ft. established by the required storage volume calculation and the stage-storage curve, or an exact value may be determined by routing the 1-year storm through the basin. The TR-20 input file and tabular hydrograph output for the routing of the 1-year storm are provided, **Figure 6.2-3**. The routing results in a maximum extended detention water surface elevation of 88.69', a peak discharge of 0.97 cfs, and a brim drawdown time of 23.5 hrs. (39 hrs. - 15.5 hrs).

FIGURE 6.2-2
Extended Detention Stage-Storage-Discharge Worksheet

ELEV (MSL)	STORAGE (ac.ft.)	1-yr. ext.det. ~~WATER QUALITY~~ ORIFICE (1)	
		h	Q
81	0	0	0
82	0.02	1	0.3
84	0.14	3	0.6
86	0.33	5	0.7
88	0.68	7	0.9
89	0.95	8	1.0
90	1.28	9	1.1
91	1.75	10	1.1

$Q = 0.33(h)^{1/2}$ where h = w.s.e. ! 81.0ft.

STEP 6 / **STEP 7** 2-year storm control is not required since the channel erosion component of this design is covered by the extended detention of the 1-year storm.

FIGURE 6.2-3

TR-20 Input and Tabular Output - 1-yr. Extended Detention

```
******************80-80 LIST OF INPUT DATA FOR TR-20 HYDROLOGY******************

JOB TR-20                     FULLPRINT          SUMMARY
TITLE 001 STORMWATER HANDBOOK EXAMPLE PROBLEM 1: EX1ROUTA.DAT
TITLE     EXTENDED DETENTION 1 YR STORM
 3 STRUCT      01
 8                          81.0       0.0         0.0
 8                          82.0       0.3         0.02
 8                          84.0       0.6         0.14
 8                          86.0       0.7         0.33
 8                          88.0       0.9         0.68
 8                          89.0       1.0         0.95
 8                          90.0       1.1         1.28
 9 ENDTBL
 6 RUNOFF 1 001       1  .039       75.         0.35          1 1   1   1
 6 RESVOR 2     01 1  2  81.0                                 1 1   1   1
   ENDATA
 7 INCREM 6              .25
 7 COMPUT 7 001      01 0.0         2.7         1.0           2 2  01  01
   ENDCMP 1
   ENDJOB 2                                 1-yr storm rainfall

********************************END OF 80-80 LIST********************************

EXECUTIVE CONTROL OPERATION INCREM     MAIN TIME INCREMENT =   .25 HOURS                              RECORD ID

EXECUTIVE CONTROL OPERATION COMPUT     FROM XSECTION   1  TO STRUCTURE  1                             RECORD ID
          STARTING TIME =    .00    RAIN DEPTH =  2.70    RAIN DURATION=  1.00    RAIN TABLE NO.= 2    ANT. MOIST. COND= 2
          ALTERNATE NO.= 1          STORM NO.= 1     MAIN TIME INCREMENT =   .25 HOURS

OPERATION RESVOR    STRUCTURE  1
          INPUT HYDROGRAPH= 1     OUTPUT HYDROGRAPH= 2
          SURFACE ELEVATION=   81.00

*** WARNING-NO PEAK FOUND, MAXIMUM DISCHARGE =          .97 CFS.

          PEAK TIME(HRS)             PEAK DISCHARGE(CFS)          PEAK ELEVATION(FEET)
            15.50                           .97                          88.69

TIME(HRS)          FIRST HYDROGRAPH POINT =    .00 HOURS      TIME INCREMENT =  .25 HOURS      DRAINAGE AREA =     .04 SQ.MI.
  10.00   DISCHG       .00      .00      .00      .00      .00      .00      .02      .25      .61      .75
  12.50   DISCHG       .86      .91      .92      .94      .95      .95      .96      .96      .97      .97
  15.00   DISCHG       .97      .97      .97      .97      .97      .97      .97      .97      .97      .96
  17.50   DISCHG       .96      .96      .96      .96      .96      .95      .95      .95      .95      .94
  20.00   DISCHG       .94      .94      .93      .93      .93      .92      .92      .92      .91      .91
  22.50   DISCHG       .91      .90      .90      .90      .89      .89      .88      .88      .87      .86
  25.00   DISCHG       .85      .84      .83      .82      .81      .80      .79      .78      .77      .76
  27.50   DISCHG       .75      .74      .74      .73      .72      .71      .70      .69      .69      .68
  30.00   DISCHG       .67      .66      .66      .65      .64      .64      .63      .62      .62      .61
  32.50   DISCHG       .60      .58      .55      .52      .50      .47      .45      .43      .40      .38
  35.00   DISCHG       .37      .35      .33      .31      .28      .21      .15      .11      .08      .06
  37.50   DISCHG       .04      .03      .02      .02      .01      .01      .01   39 hrs.

 RUNOFF VOLUME ABOVE BASEFLOW =   .77 WATERSHED INCHES,     19.30 CFS-HRS,     1.60 ACRE-FEET;   BASEFLOW =    .00 CFS
```

<u>**STEP 8**</u> (Trial 1) The 10-year flood control opening is designed using the procedures outlined in **Chapter 5-7** as follows:

1. 1-year extended detention water surface elevation is 88.69ft.
2. Set 10-year control elevation at 88.8ft.
3. Approximate storage volume required for 10-year storm control is 2.0 ac.ft. from <u>**STEP 3**</u> above. From the stage storage curve h_{10max} = 91.5ft. ! 88.8ft. = 2.7ft.
4. The maximum allowable discharge, $Q_{10allowable}$ = 25.5 cfs.
5. A weir is chosen to control the 10-year release rate:

$$L = Q_{10allowable} / C_w h^{1.5}$$

Equation 5-9
Rearranged weir equation

$$L = 25.5 \text{ cfs} / (3.3)\ (2.7\text{ft.})^{1.5}$$
L = 1.74ft. For Trial 1, use a <u>1ft. - 8in. (1.7ft.) weir</u>

Note: Since the maximum head of 2.7 ft. is used, an average value of 3.3 for the weir coefficient (C_w) is used. See **Table 5-8**: Weir Flow Coefficients.

6. The stage discharge relationship is as follows:

$$Q_w = C_w\ L(h)^{1.5}$$

Equation 5-8
Weir flow equation

$$= 3.3\ (1.7\text{ft.})\ (h)^{1.5}$$
$Q_w = 5.6\ (h)^{1.5}$ where h = wse ! 88.8ft.

7. Complete a stage-storage-discharge table for the anticipated range of elevations using the stage-discharge relationship established in item 6 above. The completed extended detention and 10-year control (TRIAL 1) portion of the stage-storage-discharge worksheet is presented in **Figure 6.2-4**. Note the addition of the elevation 88.8ft. representing the crest of the 10-year weir.

<u>**STEP 9**</u> The TR-20 input file and output summary table for the routing of the 10-year storm through the basin (Trial 1) are provided (**Figure 6.2-5**). The routing results in a 10-year maximum water surface elevation of 91.77ft., and a peak discharge of <u>29.00 cfs. > 25.5 cfs</u>.

Try smaller weir and repeat from <u>**STEP 8**</u>, #6.

FIGURE 6.2-4
Extended Detention and 10-year Flood Control (TRIAL 1) Stage-Storage-Discharge Worksheet

ELEV (MSL)	STORAGE (ac.ft.)	1-yr. ext. det. ~~WATER QUALITY~~ ORIFICE		TRIAL 1 10-YEAR CONTROL				TOTAL Q (cfs)
		(1)		WEIR (4)		ORIFICE (5)		
		h	Q	h	Q	h	Q	
81	0	0	0					0
82	.02	1	0.3					0.3
84	.14	3	0.6					0.6
86	.33	5	0.7					0.7
88	.68	7	0.9					0.9
88.8	.90	7.8	0.98	0	0			.98
89	.95	8	1.0	0.2	0.5			1.5
90	1.28	9	1.1	1.2	7.4			8.5
91	1.75	10	1.1	2.2	18.3			19.4
92	2.23	11	1.1	3.2	32.0			33.1

(1) $Q = 0.33(h)^{1.5}$ where h = w.s.e. ! 81.0ft.
(4) $Q_w = 5.6\ (h)^{1.5}$ where h = wse ! 88.8ft.

FIGURE 6.2-5

TR-20 Input and Output Summary - 1-yr. Extended Detention and 10-yr. Flood Control (TRIAL 1)

```
******************80-80 LIST OF INPUT DATA FOR TR-20 HYDROLOGY******************

JOB TR-20                  FULLPRINT          SUMMARY
TITLE 001 STORMWATER HANDBOOK EXAMPLE PROBLEM 1: EX1ROUTB.DAT
TITLE     EXTENDED DETENTION 1 YR STORM, 10 YR FLOOD CONTROL: TRIAL 1
 3 STRUCT     01
 8                      81.0       0.0        0.0
 8                      82.0       0.3        0.02
 8                      84.0       0.6        0.14
 8                      86.0       0.7        0.33
 8                      88.0       0.9        0.68
 8                      88.8       0.98       0.90
 8                      89.0       1.5        0.95
 8                      90.0       8.5        1.28
 8                      91.0       19.4       1.75
 8                      92.0       33.1       2.23
 9 ENDTBL
 6 RUNOFF 1 001       1  .039      75.        0.35        1 1   1   1
 6 RESVOR 2    01 1   2  81.0                             1 1   1   1
   ENDATA
 7 INCREM 6            .25
 7 COMPUT 7 001      01 0.0        2.7        1.0         2 2  01  01   1-yr storm
   ENDCMP 1
 7 COMPUT 7 001      01 0.0        5.5        1.0         2 2  01  02   10-yr storm
   ENDCMP 1
   ENDJOB 2

********************************END OF 80-80 LIST********************************
```

SUMMARY TABLE 1 - SELECTED RESULTS OF STANDARD AND EXECUTIVE CONTROL INSTRUCTIONS IN THE ORDER PERFORMED
(A STAR(*) AFTER THE PEAK DISCHARGE TIME AND RATE (CFS) VALUES INDICATES A FLAT TOP HYDROGRAPH
A QUESTION MARK(?) INDICATES A HYDROGRAPH WITH PEAK AS LAST POINT.)

SECTION/ STRUCTURE ID	STANDARD CONTROL OPERATION	DRAINAGE AREA (SQ MI)	RAIN TABLE #	ANTEC MOIST COND	MAIN TIME INCREM (HR)	PRECIPITATION BEGIN (HR)	PRECIPITATION AMOUNT (IN)	PRECIPITATION DURATION (HR)	RUNOFF AMOUNT (IN)	PEAK DISCHARGE ELEVATION (FT)	PEAK DISCHARGE TIME (HR)	PEAK DISCHARGE RATE (CFS)	PEAK DISCHARGE RATE (CSM)
ALTERNATE 1 STORM 1													
XSECTION 1	RUNOFF	.04	2	2	.25	.0	2.70	24.00	.76	---	12.17	14.68	376.4
STRUCTURE 1	RESVOR	.04	2	2	.25	.0	2.70	24.00	.77	88.68	15.50?	.97?	24.8
ALTERNATE 1 STORM 2													
XSECTION 1	RUNOFF	.04	2	2	.25	.0	5.50	24.00	2.83	---	12.10	61.12	1567.1
STRUCTURE 1	RESVOR	.04	2	2	.25	.0	5.50	24.00	2.87	91.77	12.49	29.99	769.0

STEP 8 6. (Trail 2) Try smaller weir : 1ft. - 4in. (1.33ft.). The stage-discharge relationship is
as follows:

$Q_w = C_w L h^{1.5}$ **Equation 5-8**
Weir equation

$= (3.3)(1.33\text{ft.})(h)^{1.5}$
$Q_w = 4.39 (h)^{1.5}$ where h = wse - 88.8ft.

7. Complete a stage-storage-discharge table for the anticipated range of elevations using the stage-discharge relationship established in item 6 above. The completed extended detention and 10-year control (TRIAL 2) portion of the stage-storage-discharge worksheet is presented in **Figure 6.2-6**.

STEP 9 The TR-20 input file and output summary table for the routing of the 10-year storm through the basin (TRIAL 2) are provided (**Figure 6.2-7**). The routing results in a maximum water surface elevation of 92.03ft. and a peak discharge of 26.82 cfs. The peak discharge is slightly greater than the allowable, further reduction may be achieved through the design of the barrel.

FIGURE 6.2-6

Extended Detention and 10-year Flood Control (TRIAL 2) Stage-Storage-Discharge Worksheet

ELEV (MSL)	STORAGE (ac.ft.)	1-yr. ext. det. ~~WATER QUALITY~~ ORIFICE		TRIAL 2 10-YEAR CONTROL				TOTAL Q (cfs)
		(1)		WEIR (4)		ORIFICE (5)		
		h	Q	h	Q	h	Q	
81	0	0	0					0.0
82	.02	1	0.3					0.3
84	.14	3	0.6					0.6
86	.33	5	0.7					0.7
88	.68	7	0.9					0.9
88.8	.90	7.8	0.98	0	0			.98
89	.95	8	1.0	0.2	0.4			1.4
90	1.28	9	1.1	1.2	5.8			6.9
91	1.75	10	1.1	2.2	14.3			15.4
92	2.23	11	1.1	3.2	25.1			26.2
93	2.94	12	1.1	4.2	37.8			38.9

(1) $Q = 0.33(h)^{1.5}$ where h = wse ! 81.0ft.

(4) $Q_w = 4.39(h)^{1.5}$ where h = wse ! 88.8ft

FIGURE 6.2-7

TR-20 Input and Output Summary - 1-yr. Extended Detention and 10-yr. Flood Control (TRIAL 2)

```
*******************80-80 LIST OF INPUT DATA FOR TR-20 HYDROLOGY******************

JOB TR-20                    FULLPRINT          SUMMARY
TITLE 001 STORMWATER HANDBOOK EXAMPLE PROBLEM 1: EX1ROUTB.DAT
TITLE     EXTENDED DETENTION 1 YR STORM, 10 YR FLOOD CONTROL: TRIAL 2
 3 STRUCT       01
 8                        81.0       0.0        0.0
 8                        82.0       0.3        0.02
 8                        84.0       0.6        0.14
 8                        86.0       0.7        0.33
 8                        88.0       0.9        0.68
 8                        88.8       0.98       0.90
 8                        89.0       1.4        0.95
 8                        90.0       6.9        1.28
 8                        91.0      15.4        1.75
 8                        92.0      26.2        2.23
 8                        93.0      48.9        2.94
 9 ENDTBL
 6 RUNOFF 1 001        1  .039      75.        0.35          1 1   1   1
 6 RESVOR 2     01 1   2  81.0                                1 1   1   1
   ENDATA
 7 INCREM 6             .25
 7 COMPUT 7 001       01 0.0        2.7        1.0           2 2  01  01   1-yr storm
   ENDCMP 1
 7 COMPUT 7 001       01 0.0        5.5        1.0           2 2  01  02   10-yr storm
   ENDCMP 1
   ENDJOB 2

********************************END OF 80-80 LIST*********************************
```

SUMMARY TABLE 1 - SELECTED RESULTS OF STANDARD AND EXECUTIVE CONTROL INSTRUCTIONS IN THE ORDER PERFORMED
(A STAR(*) AFTER THE PEAK DISCHARGE TIME AND RATE (CFS) VALUES INDICATES A FLAT TOP HYDROGRAPH
A QUESTION MARK(?) INDICATES A HYDROGRAPH WITH PEAK AS LAST POINT.)

SECTION/ STRUCTURE ID	STANDARD CONTROL OPERATION	DRAINAGE AREA (SQ MI)	RAIN TABLE #	ANTEC MOIST COND	MAIN TIME INCREM (HR)	PRECIPITATION BEGIN (HR)	PRECIPITATION AMOUNT (IN)	PRECIPITATION DURATION (HR)	RUNOFF AMOUNT (IN)	PEAK DISCHARGE ELEVATION (FT)	PEAK DISCHARGE TIME (HR)	PEAK DISCHARGE RATE (CFS)	PEAK DISCHARGE RATE (CSM)
ALTERNATE 1 STORM 1													
XSECTION 1	RUNOFF	.04	2	2	.25	.0	2.70	24.00	.76	---	12.17	14.68	376.4
STRUCTURE 1	RESVOR	.04	2	2	.25	.0	2.70	24.00	.77	88.68	15.50?	.97?	24.8
ALTERNATE 1 STORM 2													
XSECTION 1	RUNOFF	.04	2	2	.25	.0	5.50	24.00	2.83	---	12.10	61.12	1567.1
STRUCTURE 1	RESVOR	.04	2	2	.25	.0	5.50	24.00	2.86	92.03	12.53	26.82	687.6

STEP 10 The barrel should be sized to control the flow before the riser structure transitions from riser weir flow control to riser orifice flow control. Therefore determine the geometry of the riser and the elevation at which the transition occurs. Also, if possible, a reduction in the 10-year discharge would be desirable. This is only possible if an emergency spillway is provided, since a barrel which controls the 10-year flow will be too small to efficiently pass the 100-year flow without a significant increase in storage volume.

a. Riser flow control: try a 4ft. × 4ft. (inside dimension) square box riser with 6" wall thickness. Set top of riser at elevation 92.2ft.

WEIR FLOW - Total weir length: 3ft. sides (2) = 6ft., 1.67ft. front; weir length = 7.67ft., elevation 92.2ft. (riser top weir); weir length = 1.33ft., elevation 88.8ft. (10-year weir)

$$Q_w = C_w L h^{1.5}$$

Equation 5-8
Weir equation

$Q_w = 3.1\,(7.67\text{ft.})\,(h)^{1.5}$

$Q_w = 23.8\,(h)^{1.5}$ Where h = wse - 92.2ft.

Note: The flow measured from elevation 92.2ft. represents the flow over the top of the riser (weir length 7.67ft.). The flow over the 10-year weir (elevation 88.8ft., length 1.33ft.) is added in the Stage-Storage-Discharge table (**Figure 6.2-8**) to provide a total riser weir flow (Refer to **Figure 6.2-10**). This value will then be compared to the riser orifice flow capacity calculated below. C_w values for low head conditions are averaged at 3.1. See **Table 5-8**: Weir Flow Coefficients

ORIFICE FLOW - Riser structure inside dimensions - 4ft. × 4ft., total riser orifice area = 16 ft^2 at elevation 88.8ft.

$$Q = Ca\sqrt{2gh}$$

Equation 5-6
Orifice Equation

$$Q = 0.6\,(16ft^2)\sqrt{(2)\,(32.2)\,(h)}$$

$Q = 77.03\,(h)^{1/2}$ where h = wse − 88.8ft.

Add the riser weir flow and orifice flow values to the stage-storage-discharge table, **Figure 6.2-8**.

This analysis shows that the riser does not transition from weir flow to orifice flow within the range of water surface elevations. Therefore, the barrel does not have to control the flow. However, we want to slightly restrict the 10-year discharge in order to reduce it to within 5% of the 10-year allowable release rate.

b. Barrel flow control: upstream invert: 80.75ft., downstream invert: 79.95ft., length = 80ft.; s = 1.0%

Start with elevation 92ft., determine the HW/D value to be: (92ft. − 80.75ft.) / D = 11.25 / D. To provide the most economical pipe size, or as in this case, to restrict the flow at approximately 26 cfs in (during the 10-year storm) in order to achieve better 10-year control, set HW/D = 26 cfs and by trial and error, using Federal Highway Administration (FHA) culvert nomograph: **Figure 5-16** (Headwater Depths for Concrete Pipe Culverts with Inlet Control), entrance condition 1, determine that an 18 in. pipe comes closest to the desired flow: HW/D = 11.25 / 1.5 = 7.5 Since the upper limit of the nomograph is HW / D = 6, use 6, and read Q = 25 cfs. <u>Try 18 in. RCP Barrel</u>.

1. <u>Inlet Control</u> Using the above referenced FHA culvert chart **(Figure 5-16)** establish the stage - discharge relationship for an 18 in. concrete pipe barrel, (HW = wse − 80.75), and add these values to the stage-storage-discharge table, **Figure 6.2-8**.

Where the upper limit of the nomograph HW/D values are exceeded, use the orifice equation to approximate the inlet control flow values:

$$Q = Ca\sqrt{2gh}$$

Equation 5-6
Orifice Equation

$$Q = 0.6\,(1.77ft^2)\sqrt{(2)\,(32.2)\,(h)}$$

$Q = 8.5\ (h)^{1/2}$ where h = wse − 80.75ft.

2. <u>Outlet Control</u> Use **Equation 5-10** to establish the stage-discharge relationship for the 18" concrete barrel as follows:

$$Q = a\sqrt{\frac{2gh}{1+K_m+K_pL}}$$

Equation 5-10
Pipe flow control equation

Where: $a = 1.77\ ft^2$
$h = wse - (79.95 + D/2) = wse - 80.7$
Km = Ke + Kb (from **Figure 5-9**; square end pipe: Ke =
0.5

= 0.5 + 0.5 = 1.0
Kp = .0182 (from **Table 5.10**, 18" pipe, ‘n’ = .013
L = 80

$$Q = 1.77ft^2\sqrt{\frac{2(32.2)(h)}{1+1+(.0182)(80ft.)}}$$

$Q = 7.64\ (h)^{½}$ where h = wse − 80.75

Add the barrel inlet and outlet control flow values to the stage-storage-discharge table, **Figure 6.2-8**.

These flow values indicate the barrel is in outlet control for the entire range of expected water surface elevations. (The inlet control values are struck out to indicate that the outlet flow condition controls the discharge). The stage -storage-discharge table indicates that the barrel controls the discharge at elevation 93ft. and above.

The performance of the 10-year control and barrel hydraulics (Trial 2) can now be checked by routing the 10-year storm, or the designer may choose to size the emergency spillway first.

STEP 11 The emergency spillway is designed using the procedure outlined in **Chapter 5-8** as follows:

1. 10-year design water surface elevation is 92.03ft. Set the invert of the emergency spillway at 92.2ft.
2. Q_{100} = 109 cfs − [10-year weir release (27 cfs)] = 82 cfs
3. Since this is a relatively small facility with a low potential downstream hazard in the case of an embankment failure, the alternate design using **Figure 5-23**: Design Data for Earth Spillways, is used for the design. We want the maximum stage to be 93ft. which allows for approximately 0.8ft. (use 1.0ft.) of flow through spillway. From **Figure 5-23**: h_p = 1.0ft. for a design flow of 81 cfs, read b = 36ft., v_{max} = 4.0ft/s, and s_{min} = 3.0%.

Add these stage-discharge values to the stage-storage-discharge table, **Figure 6.2-8**.

<u>STEP 12</u> The TR-20 input file and full output file for the routing of the 1-, 10-, and 100-year storms through the basin are provided.

<u>STEP 13</u> Outlet protection is designed using the design procedure presented in **Chapter 5-7** (and STD & SPEC 3.18 in the <u>VESCH</u>) as follows:

1. Outlet discharges into a channel with a minimum tailwater condition (T_w < 0.5 barrel diameter).
2. Using **Figure 5-20**, Q_{10} = 25.8 cfs and d = 18in., read D_{50} = 0.8ft. and L_a = 22ft. Use Class AI riprap.
3. Riprap apron width is to conform to the existing channel geometry to the top of bank.
4. The depth of the riprap blanket is 2.25 x 0.9 (Class AI) = 2 ft.

<u>STEP 14</u> Riser buoyancy calculation (**Chapter 5-7)** is as follows:

1. Determine buoyant force:
   height: 92.2ft. ! 80.75ft. = 11.45ft. × (4ft.× 4ft.) = 183 ft^3;
   base: 6ft. × 6ft. × 1ft. = 36 ft^3;
   total: 219 ft^3 × 62.4 lb/ft^3 = <u>13, 678 lb8</u>

2. Downward/resisting force:
   volume of riser walls = (11.45ft. × 4ft. × 0.5ft. × 2) + (11.45ft. × 3ft. × 0.5ft. × 2) = 80.5 ft^3;
   volume of extended back wall = (1ft. × 4ft. × 0.5ft.) = 2 ft^3;
   volume of base = (6ft. × 6ft. × 1ft.) = 36 ft^3;
   volume of 10-year weir cutout = (3.4ft. × 1.33ft. × 0.5ft.) = !2.3 ft^3;
   volume of 18in. barrel and 10in. ext detention pipe cutouts = (2.41ft^2 + 0.78 ft^2) × 0.5ft. = !1.6 ft^3;
   total downward force = volume of concrete × unit weight (150lb/ft^3) : (80.5 ft^3 + 2 ft^3 + 36 ft^3 ! 2.3 ft^3 ! 1.6 ft^3) × (150lb/ft^3) = <u>17190 lb 9</u>

3. Safety factor: 13, 678 lb8 x 1.25 = <u>17,0988 < 17190 lb 9</u> **Riser OK**

FIGURE 6.2-8
Stage-Storage-Discharge Table

ELEV (MSL)	STORAGE (ac.ft.)	1-yr. ext. det. ~~WATER QUALITY~~ ORIFICE (1)		TRIAL 2 10-YEAR CONTROL WEIR (4)		10-YEAR CONTROL ORIFICE (5)		RISER STRUCTURE WEIR (6)		RISER STRUCTURE ORIFICE (7)		BARREL INLET (8)		BARREL OUTLET (9)		EMERGENCY SPILLWAY (10)		TOTAL Q (cfs)
		h	Q	h	Q	h	Q	h	Q	h	Q	HW/D	Q	h	Q	h	Q	
81	0	0	0															0
82	.02	1	0.3															0.3
84	.14	3	0.6															0.6
86	.33	5	0.7															0.7
88	.68	7	0.9															0.9
88.8	.90	7.8	0.98	0	0				0/0	0	0							1.0
89	.95	8	1.0	0.2	0.4				0/0.4	0.2	~~34.4~~	5.5	~~24~~	8.3	~~22.0~~			1.4
90	1.28	9	1.1	1.2	5.8				0/5.8	1.2	~~84.4~~	6.2	~~26~~	9.3	~~23.3~~			6.9
91	1.75	10	1.1	2.2	14.3				0/14.3	2.2	~~114.3~~	6.8	~~27~~	10.3	~~24.5~~			15.4
92	2.23	11	~~1.1~~	3.2	~~25.1~~				0/25.1	3.2	~~139.8~~	7.5	~~28~~	11.3	25.7			25.7
92.2	2.40	11.2	~~1.1~~	3.4	~~27.5~~			0	0/27.5	3.4	~~142.0~~	7.6	~~29~~	11.5	25.9	0	0	25.9
93	2.94	12.0	~~1.1~~	4.2	~~37.8~~			0.8	17.0/54.8	4.2	~~157.9~~	8.1	~~30~~	12.3	26.8	0.8	54	80.8
94	3.81	13.0	~~1.2~~	5.2	~~52.0~~			1.8	57.5/109.5	5.2	~~175.7~~	8.8	~~31~~	13.3	27.8	1.8	214	241.8

(6) Q for elevations 88.8 to 92.2 ft. represents 10-yr. weir flow (4)
Q for elevations 92.2 to 94 ft represents 10-yr weir flow plus the top of riser weir flow: $Q = 23.8\,(h)^{1.5}$ where h = wse - 92.2 ft.

(7) $Q = 77.03\,(h)^{1/2}$ where h = wse - 88.8 ft

(8) $Q = 8.5\,(h)^{1/2}$ where h = wse - 80.75 ft

(9) $Q = 7.64\,(h)^{1/2}$ where h = wse - 80.75 ft

(10) Figure 5-23: Design Data for Earth Spillways.

FIGURE 6.2-9
TR-20 Input and Output Summary - 1-yr. Extended Detention, 10-yr. Flood Control, 100-yr. Emergency Spillway

```
*******************80-80 LIST OF INPUT DATA FOR TR-20 HYDROLOGY*******************

JOB TR-20                   FULLPRINT        SUMMARY
TITLE 001 STORMWATER HANDBOOK EXAMPLE PROBLEM 1: EX1ROUTC.DAT
TITLE    EXTENDED DETENTION 1 YR STORM, 10 YR FLOOD CONTROL, 100 YR SAFETY
 3 STRUCT      01
 8                       81.0      0.0       0.0
 8                       82.0      0.3       0.02
 8                       84.0      0.6       0.14
 8                       86.0      0.7       0.33
 8                       88.0      0.9       0.68
 8                       88.8      0.98      0.90
 8                       89.0      1.4       0.95
 8                       90.0      6.9       1.28
 8                       91.0     15.4       1.75
 8                       92.0     25.7       2.23
 8                       92.2     25.9       2.45
 8                       93.0     80.8       2.94
 8                       94.0    241.8       3.81
 9 ENDTBL
 6 RUNOFF 1 001      1  .039      75.        0.35       1 1  1  1
 6 RESVOR 2    01 1  2  81.0                            1 1  1  1
   ENDATA
 7 INCREM 6             .25
 7 COMPUT 7 001     01 0.0        2.7        1.0        2 2 01 01   1-yr storm
   ENDCMP 1
 7 COMPUT 7 001     01 0.0        5.5        1.0        2 2 01 02   10-yr storm
   ENDCMP 1
 7 COMPUT 7 001     01 0.0        8.0        1.0        2 2 01 03   100-yr storm
   ENDCMP 1
   ENDJOB 2
```

SUMMARY TABLE 1 - SELECTED RESULTS OF STANDARD AND EXECUTIVE CONTROL INSTRUCTIONS IN THE ORDER PERFORMED
(A STAR(*) AFTER THE PEAK DISCHARGE TIME AND RATE (CFS) VALUES INDICATES A FLAT TOP HYDROGRAPH
A QUESTION MARK(?) INDICATES A HYDROGRAPH WITH PEAK AS LAST POINT.)

SECTION/ STRUCTURE ID	STANDARD CONTROL OPERATION	DRAINAGE AREA (SQ MI)	RAIN TABLE #	ANTEC MOIST COND	MAIN TIME INCREM (HR)	PRECIPITATION BEGIN (HR)	PRECIPITATION AMOUNT (IN)	PRECIPITATION DURATION (HR)	RUNOFF AMOUNT (IN)	PEAK DISCHARGE ELEVATION (FT)	PEAK DISCHARGE TIME (HR)	PEAK DISCHARGE RATE (CFS)	PEAK DISCHARGE RATE (CSM)
ALTERNATE 1 STORM 1													
XSECTION 1	RUNOFF	.04	2	2	.25	.0	2.70	24.00	.76	---	12.17	14.68	376.4
STRUCTURE 1	RESVOR	.04	2	2	.25	.0	2.70	24.00	.77	88.68	15.50?	.97?	24.8
ALTERNATE 1 STORM 2													
XSECTION 1	RUNOFF	.04	2	2	.25	.0	5.50	24.00	2.83	---	12.10	61.12	1567.1
STRUCTURE 1	RESVOR	.04	2	2	.25	.0	5.50	24.00	2.87	92.13	12.54	25.83	662.3
ALTERNATE 1 STORM 3													
XSECTION 1	RUNOFF	.04	2	2	.25	.0	8.00	24.00	4.99	---	12.09	108.87	2791.7
STRUCTURE 1	RESVOR	.04	2	2	.25	.0	8.00	24.00	5.10	93.06	12.30	90.54	2321.6

STEP 15 Anti-seep collars, rather than a drainage blanket, will be used on this facility since it is a dry facility. Anti-seep collars are designed using the procedure outlined in **Chapter 5-7** as follows:

1. Length of barrel within saturated zone:

$$L_s = Y\,(Z + 4)\ \hat{}\,1 + \ \hat{}\ \frac{S}{0.25 - S}$$

Equation 5-11
Barrel Length in Saturated Zone

where: Y = 92.13ft. ! 81.0ft. = 11.2ft.
Z = 3H : 1V; = 3
S = 1.0%

$$L_s = 11.2(\ 3 + 4\ )\ \hat{}\,1 + \hat{}\ \frac{.01}{0.25 - .01}$$

L_s = 81ft.

OR

Extend a line at 4H:IV from the 10-year water surface elevation at the upstream face of the embankment downward until it intersects the barrel. The resulting point on the barrel measures approximately 81ft. From the low flow headwall.

2. (L_s) (0.15) = (81ft.) (0.15) = 12.1ft.
3. For an 18in. (1.5ft.) diameter barrel: 4ft. + 1.5ft. = 5.5ft.
4. Projection = 4ft.
5. Number of collars = 12.1 / 4 = 3 collars. Only two collars are desired: use a 6ft. projection 6ft. + 1.5ft. = 7.5ft. collars. 12.1 / 6 = 2. <u>Use 2 - 7.5ft. collars</u>

FIGURE 6.2-10
Riser Weir and Trash Rack - Perspective

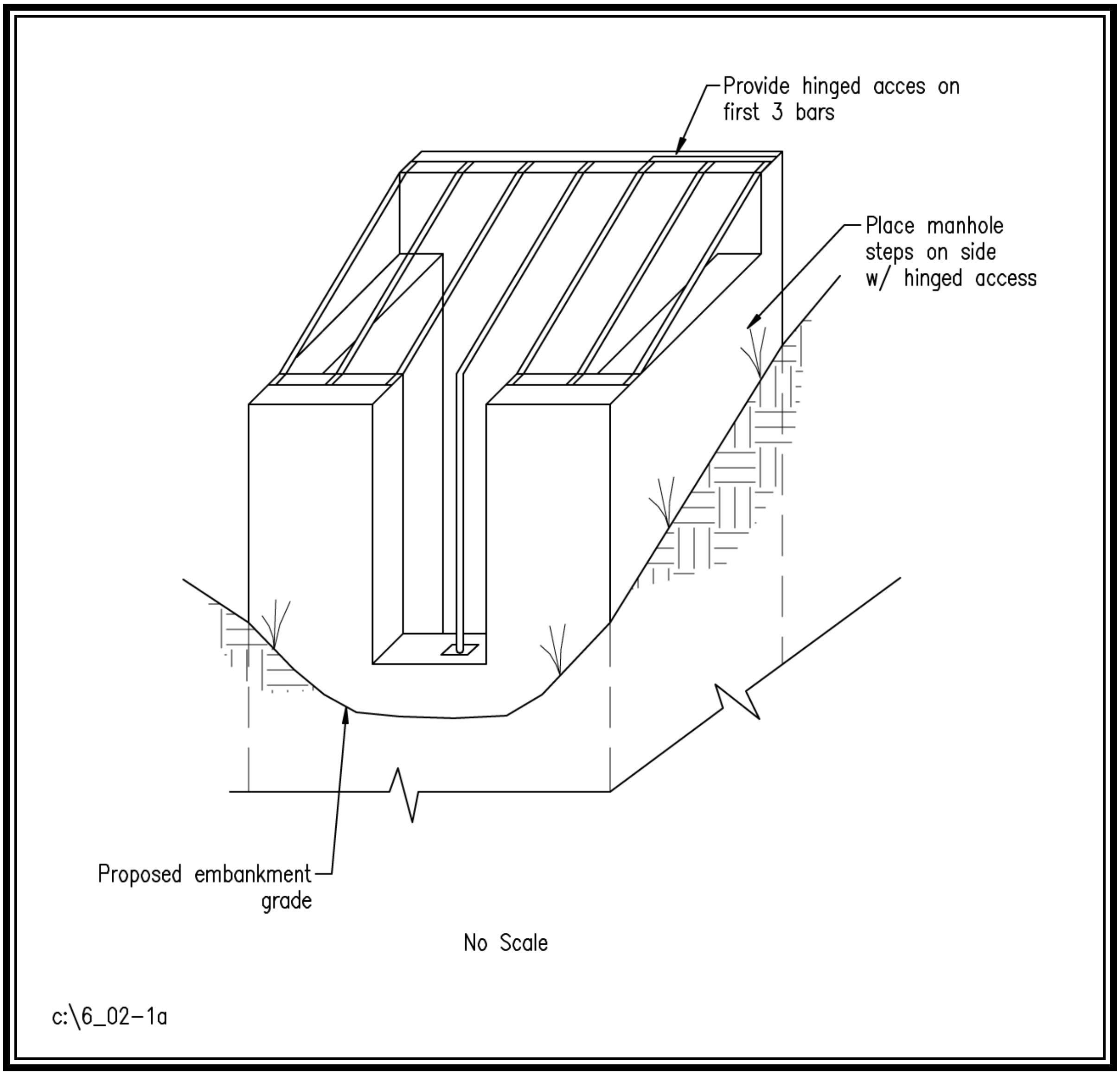


FIGURE 6.2-11
Riser Detail - Section

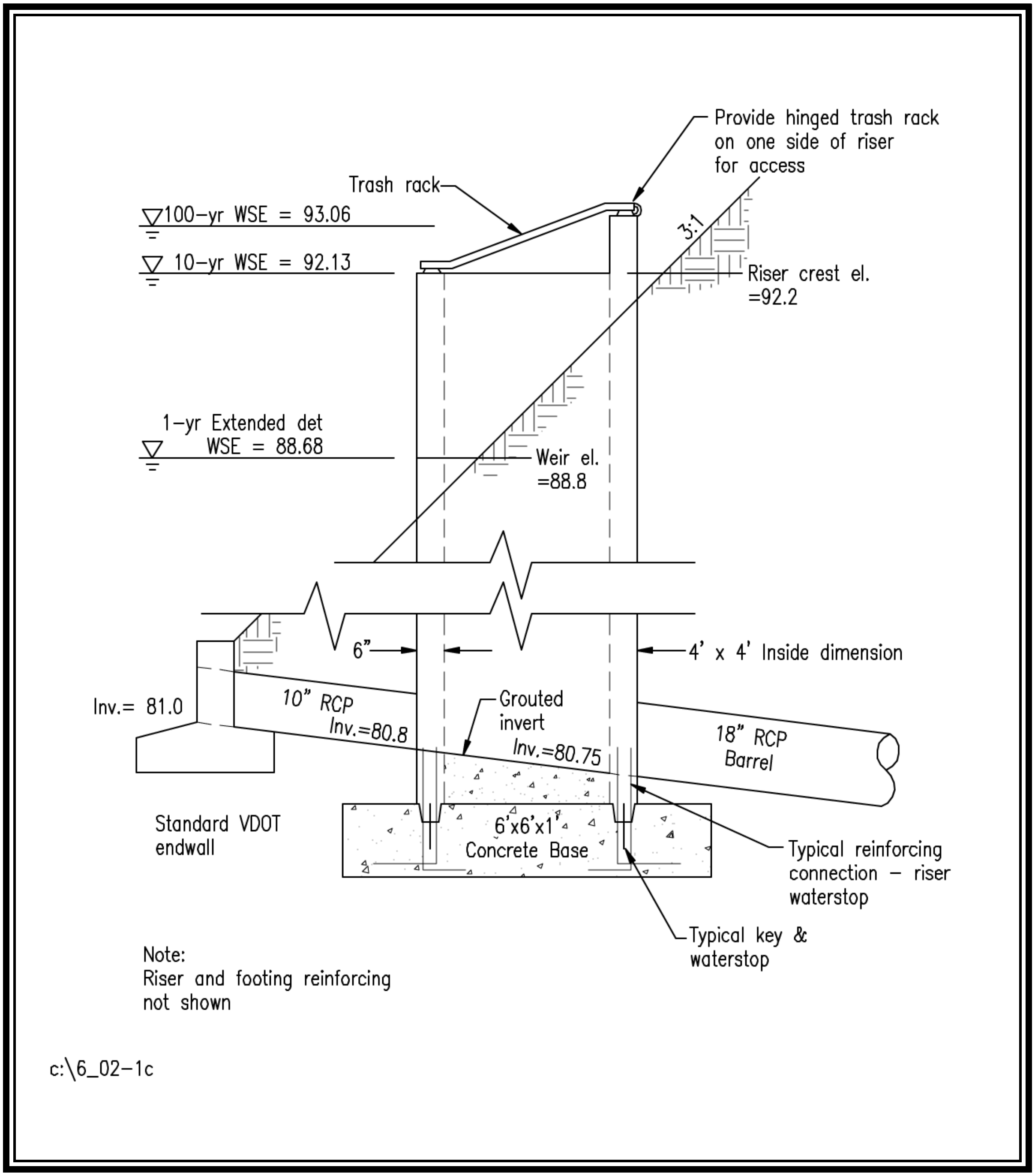
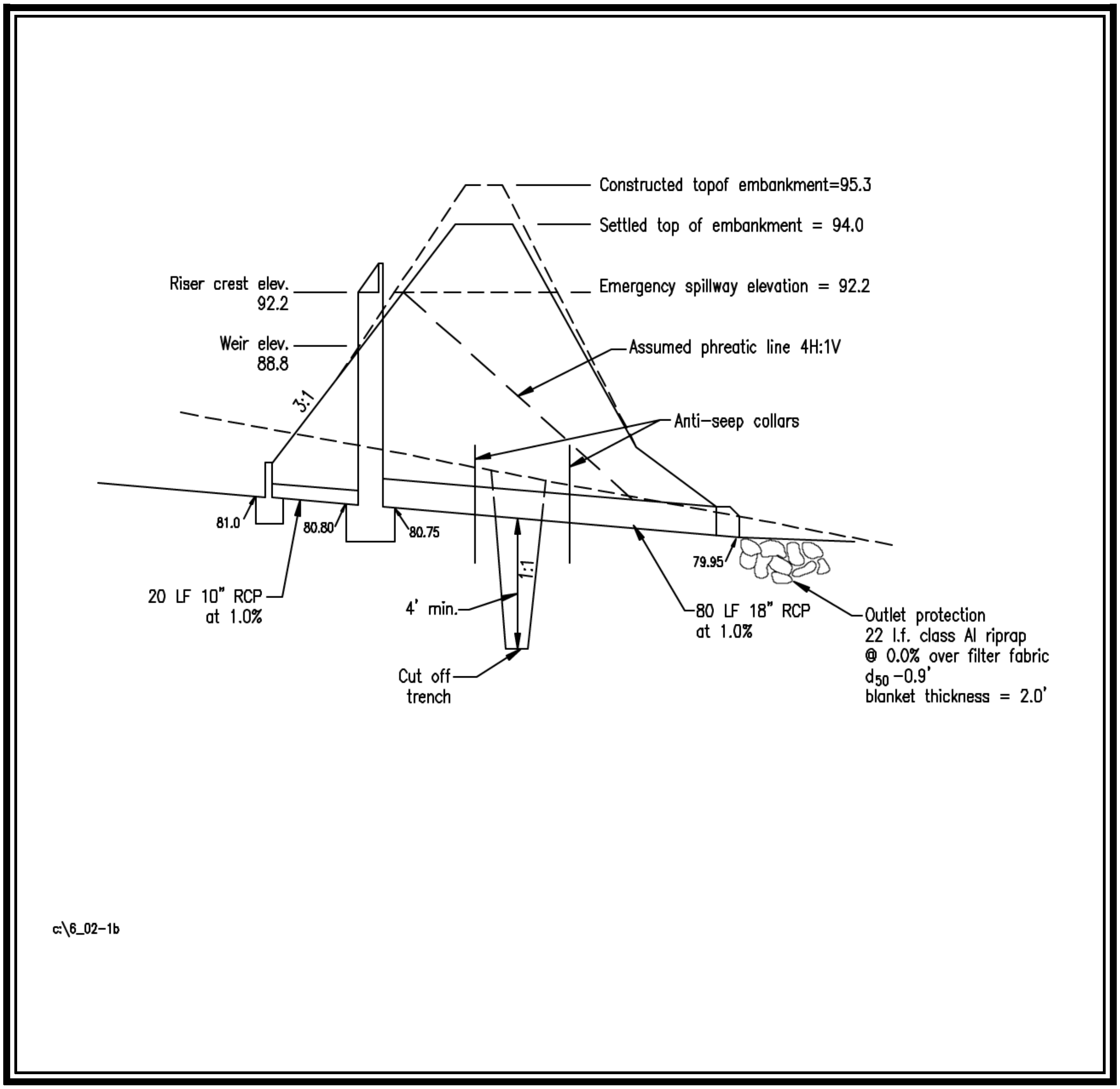


FIGURE 6.2-12
Principal Spillway - Profile



Example 6.3

BMP SELECTION

EXAMPLE 6.3

INTRODUCTION:

A 90.2 acre site is to be developed into an office park. Perimeter portions of the site are composed of steep slopes adjacent to two small tributaries which bound the site. Local zoning and Resource Protection Ordinances restrict the development to the upper, relatively flat center of the property. (This restriction is does not necessarily limit the development since the engineering costs of developing on the steep slopes, as well as minimizing the environmental impacts, would be prohibitive.) The site consists of soils with a moderately high runoff potential (Hydrologic Soil Group C) with no existing impervious cover.

The proposed buildings, parking lots, and other infrastructure are located on the site such that the developed condition is drained by three outfalls. These outfalls drain into the adjacent stream channels at the naturally occurring drainage paths from the site. The local stormwater management (SWM) program requires that all three components of stormwater management, water quality, stream channel erosion, and flooding, be investigated. This example will illustrate the application of these components, including a discussion of the *Performance-based* and *Technology-based* water quality criteria.

GIVEN:

The pre- and post-developed hydrology for the site is presented in **Table 6-3.1**. The drainage areas are measured to the proposed study points which have been located at the downstream limit of the potential BMP locations.

FIND:

Evaluate the BMP options and select the combination which best serve the proposed development.

SOLUTION:

The implementation of stormwater BMPs should be considered during the initial stages of the site design. An evaluation of the stormwater requirements prior to site design will allow the engineer to identify potential BMP locations and provide the most efficient alignment of the drainage infrastructure so as to enhance the use of natural drainage ways to convey stormwater runoff. (The internal drainage systems must satisfy the Erosion and Sediment Control, MS-19 criteria for channel adequacy.) Once the preliminary site design is completed, the engineer can calculate the water quality volume, pollutant loadings, peak rates of discharge, and channel capacities in order to finalize the BMP strategy for the site.

The most efficient method of evaluating the BMP requirements for a development site is one based on a hierarchy of potential impacts to the site design. This method would start with an evaluation of the flood component requirements since it is potentially the most land intensive component with regard to storage volume. In some cases the stream channel erosion component may require the largest storage volume. In either case, the analysis of the downstream conditions will determine the level of detention required to comply with either of these components. Water quality requirements, on the other hand, may be addressed with smaller BMPs or incorporated into the design of the detention structures required by the flooding or stream channel erosion components.

Table 6.3-1
Hydrologic Summary
TR-55 Graphical Peak Discharge Method

	Condition	Area (acres)	% Imp Cover	RCN	t_c (hrs.)	Q_2 (cfs)	Q_{10} (cfs)
SITE	Pre-dev	90.2		74			
	Post-dev	90.2	17	78			
DA - 1	Pre-dev	9.79		74	0.36	14	26
	Post-dev	16.16	36	83	0.34	37	59
DA - 2	Pre-dev	26.61		75	0.41	38	68
	Post-dev	26.39	35	83	0.40	56	89
DA - 3	Pre-dev	2.65		65	0.58	2.0	4.0
	Post-dev	2.65	19	71	0.58	3.0	5.0

Flooding

An analysis of the downstream receiving system must be performed in order to evaluate the flood way conveyance capacity of the of the two tributaries adjacent to the site as well as the downstream channel. The Virginia SWM Regulations (4VAC3-20-85) require that downstream properties be protected form damages from localized flooding due to increases in volume, velocity, and peak flow rate of stormwater runoff by detaining the 10-year post-developed peak rate of runoff and releasing it at the pre-developed rate. There is also a provision which allows an alternate criteria based upon geographic, land use, topographic, geologic factors or other downstream conveyance factors as appropriate. In this case, the local government has a Flodplain Management Ordinance in place which has restricted development within the flood way of the tributaries and the downstream channel. The analysis of the flood way reveals that there is sufficient capacity to convey the ultimate development condition runoff within the flood way, and that there is no existing development (or structures) within the flood way for the entire downstream reach to the confluence with the river.

Flood control (10-year storm) is not required in this case.

Stream Channel Erosion

The Virginia SWM Regulations (4VAC3-20-81) require that properties and receiving waterways downstream of any land development project be protected from erosion and damage due to increases in volume, velocity, and peak flow rate of stormwater runoff. A rigorous analysis of the downstream channel is required in order to verify the adequacy for conveying the post-developed runoff. The following items were completed for each channel in order to adequately verify the analysis:

1. **Channel geometry** - A minimum of three surveyed cross-sections were taken at a minimum spacing of 50' along the channel length downstream of the discharge point.

2. **Channel lining** - A sample of the channel lining was collected and analyzed to determine the composition relative to the permissible velocities found in Table 5-22 of the Virginia Erosion and Sediment Control Handbook.

3. **Channel slope** - Relative elevations were taken along the channel length at the channel cross sections in order to determine the average longitudinal slope of the channel.

4. **Channel Inspection** - The channel was physically inspected by walking the length to verify that there are no significant changes or obstructions such as undersized culverts or other “improved” restrictions which may restrict the flow and cause it to jump the banks or increase in velocity to an erosive level.

The channel analysis indicated that the post-developed condition runoff would cause an erosive condition in specific sections of the channel where the flow area narrows considerably. In addition, the physical inspection verified that several of these narrow areas as well as several bends are already experiencing some erosion under existing runoff conditions. Some of the options considered include:

1. **Channel improvements** - Channel improvements are is ruled out due to poor access conditions to the channel. Significant clearing would be required to not only gain access but also to maneuver construction equipment adjacent to the channels. Some possibilities do

exist for hand placed bioengineering stabilization of the eroded portions of the channel.

2. **Alternative site design** - Several alternate site configurations were evaluated in an effort to reduce the impervious cover, disconnect the impervious cover from the drainage system (disconnecting impervious cover include discharging roof down spouts into dry wells or into sheet flow conditions over pervious areas, placing grass or landscaped buffer strips between impervious surfaces and the improved drainage structures and conveyances), and create small pockets in which to detain runoff in an effort to increase the hydrologic flow time and decrease the peak rate of runoff from the site. While these efforts did result in some significant reductions in post-developed runoff, the resulting peak rate was still determined to be too high for the stream channels to convey in a non-erosive manner.

3. **Combination of channel improvements, site design, and detention** - The hydrologic analysis of the post-developed condition with the various alternative site designs and 24 hour extended detention of the runoff from the 1-year frequency storm yielded a peak rate of runoff significantly less than the runoff from the pre-developed condition 2-year storm. Further analysis indicated that the statistical occurrence frequency of the post-developed condition peak runoff equivalent to the pre-developed 2-year peak runoff occurs less than once in 5 years. This means that, according to the statistical analysis, it would take a five year frequency storm event to generate the pre-developed 2-year peak rate of runoff leaving the site.

Alternative 3 was selected in order to attempt to stabilize the natural channels adjacent to the site. Extended detention basins designed to detain the runoff from the 1-year 24 hour storm will be placed in drainage areas 1 (DA-1) and 2 (DA-2).

<u>Water Quality</u>

The designer must select either the *Performance-based* or *Technology-based* water quality criteria. The technology-based criteria considers the drainage area size and impervious cover draining to a BMP to establish the best available technology (BMP) for the drainage area or site being evaluated. The performance-based criteria uses the percent impervious cover of the site to calculate a total <u>site</u> pre- and post-developed pollutant load. The engineer then implements a BMP strategy which satisfies the <u>total</u> site pollutant reduction requirement.

Since the Performance-based water quality criteria allows for overall site compliance, it is not always necessary to place a water quality BMP in each drainage area on the site. DCR recommends, however, that the engineer evaluate the potential pollutant loading based on the amount of impervious cover and the concentration of that cover. In other words, if the impervious cover is concentrated, or located such that an improved drainage system is collecting the runoff, then the implementation of a water quality BMP(s) should be implemented for that area. This is the preferred solution, rather than placing a BMP in one drainage area, satisfying the performance-based total site pollutant removal requirement for the site, and then ignoring the other drainage areas and their associated impacts on downstream water quality. The key to the successful implementation of this recommendation is a menu of available, cost effective, and low maintenance BMPs. The menu of BMPs found in **Table 1** of the SWM Regulations, as well as the several new and innovative BMPs currently available, provide several low cost options. This is one of the strategies of Low Impact Development: relatively small and maintenance free BMPs to control small portions of the development area in landscaped settings in addition to development strategies which effectively reduce the impact of the impervious area on the runoff from the site. (Refer to the references found at the end of **Chapter 2** for more information on Low Impact Development.)

This example illustrates a development scenario in which the percent impervious cover determination for the performance-based criteria plays a significant role in the BMP strategy. The percent impervious cover for the *property* is low (15.73 acres of impervious cover on a 90.2 acre site: 17% impervious cover). The percent impervious cover for the *individual drainage areas* to the potential BMP locations, however, is much higher (a total of 45.2 acres at 35% impervious cover). As discussed in **Chapter 2**, one of the effects of using impervious cover as a regulatory water quality yardstick is to encourage the minimization of impervious cover and the preservation of green space and environmentally sensitive areas. In this example, the <u>total</u> impervious cover is limited to 17% of the <u>site</u>.

When using the performance-based criteria the derived benefit of such a low percent impervious cover is a minimal pollutant removal requirement. This minimal removal requirement illustrates the discussion above: a BMP placed in one of the drainage areas will most likely satisfy the removal requirement for the site, and the remaining drainage areas could be left uncontrolled for water quality. It stands to reason, however, that the highly concentrated impervious cover found within the other drainage areas will have a significant impact on the water quality of the adjacent streams. The use of the entire site to determine the percent impervious cover does not accurately reflect the changes to the land use. **Therefore, when using the performance-based criteria for large development parcels, DCR recommends that the percent impervious cover be calculated by using the drainage areas or an established *planning area*. In this example, a planning area may be established which consists of the portion of the site which is able to be developed.**

A planning area is defined as a designated portion of the parcel on which the land development project is located. Planning areas shall be established by delineation on a master plan. Once established, planning areas shall be applied consistently for all future projects.

The water quality requirements in terms of pollutant load removal are calculated using the **Performance-Based Water Quality Calculations: Worksheet 2** provided in **Appendix 5D**, and summarized below:

TRIAL 1: Entire site.

STEP 4: Equation 5-16

$L_{pre(watershed)} = [0.05 + (0.009 \times \underline{16\%})] \times \underline{90.2\text{ ac.}} \times 2.28 = \underline{39.9}$ pounds per year

STEP 5: Equation 5-21

$L_{post} = [0.05 + (0.009 \times \underline{17.4\%})] \times \underline{90.2\text{ ac.}} \times 2.28 = \underline{42.5}$ pounds per year

STEP 6:

RR = $\underline{42.5}$! $\underline{39.9}$ = $\underline{2.6}$ pounds per year

STEP 7:

EFF = $(\underline{2.6} \div \underline{42.5}) \times 100 = \underline{6.1\ \%}$

If a BMP could serve the entire site, then a removal efficiency of 6.1% would be required. Since this can not be done, a minimum of 2.6 pounds of phosphorus must be removed from any one or combination of the drainage areas of the developed portion of the site.

TRIAL 2: Planning area consisting of the developable area of the site: 45.2 acres

STEP 4: Equation 5-16

$L_{pre(watershed)} = [0.05 + (0.009 \times \underline{16\%})] \times \underline{45.2\text{ ac.}} \times 2.28 = \underline{20.0}$ pounds per year

STEP 5: Equation 5-21

$L_{post} = [0.05 + (0.009 \times \underline{35.0\%})] \times \underline{45.2\text{ ac.}} \times 2.28 = \underline{37.6}$ pounds per year

STEP 6:

RR = $\underline{37.6}$! $\underline{20.0}$ = $\underline{17.6}$ pounds per year

STEP 7:

(1.) **EFF** = $(\underline{17.6} \div \underline{37.6}) \times 100 = \underline{46.8\ \%}$

When considering the whole site (90.2 acres at 17% impervious cover), the pollutant removal requirement is 6% of the post-developed load (or 2.6 lbs of phosphorus) as calculated in Trial 1. When just the drainage areas to the BMP locations are considered (45.2 acres at 35% impervious cover), the pollutant removal requirement is 46.8% of the post-developed load (or 17.6 lbs of phosphorus).

Continuing with **TRIAL 2, STEP 7:**

(2.) Select BMP(s) from Table 5-15:

BMP 1: DA 1 - 16.16 ac.: Extended Detention (2×WQV) - 35% eff.
BMP 2: DA 2 - 26.39 ac.: Retention Basin III (4×WQV) - 65% eff.
BMP 3: DA 3 - 2.65 ac.: Bioretention Basin - 50% eff.

(3.) Determine pollutant load entering BMPs:

$\mathbf{L}_{\mathbf{BMP1}}$ = [0.05 + (0.009 × 36%)] × 16.16 ac. × 2.28 = 13.8 pounds per year
$\mathbf{L}_{\mathbf{BMP2}}$ = [0.05 + (0.009 × 35%)] × 26.39 ac. × 2.28 = 22.0 pounds per year
$\mathbf{L}_{\mathbf{BMP3}}$ = [0.05 + (0.009 × 19%)] × 2.65 ac. × 2.28 = 1.3 pounds per year

(4.) Calculate the pollutant load removed by BMPs:

$\mathbf{L}_{\mathbf{removed/BMP1}}$ = .35 × 13.8 = 4.83 pounds per year
$\mathbf{L}_{\mathbf{removed/BMP2}}$ = .65 × 22.0 = 14.3 pounds per year
$\mathbf{L}_{\mathbf{removed/BMP3}}$ = .50 × 1.3 = 0.65 pounds per year

(5.) Calculate the total pollutant load removed by the BMPs:

$\mathbf{L}_{\mathbf{removed/total}}$ = 4.83 + 14.3 + 0.65 = 19.78 pounds per year

Several other combination of BMPs will satisfy the removal requirements of TRIAL 2 of this example. Since both drainage areas 1 and 2 require stream channel erosion protection, and an aesthetic retention pond was desirable as a focal point of the office setting, the combination presented above was selected. The dry storage above the permanent pool, as well as the storage above the water quality extended detention volume, are both to be designed to provide extended detention of the runoff from the 1-year 24 hour storm for stream channel erosion control.

GLOSSARY

Glossary of Stormwater Management Terms and Acronyms

AASHTO - American Association of State and Highway Transportation Officials

Adsorption - The process by which a solute is attracted to a solid surface. Adsorption is the process utilized in stormwater management BMPs to enhance the removal of soluble pollutants.

Anti-seep collar - A device constructed around a pipe or other conduit and placed into a dam, levee, or dike for the purpose of reducing seepage losses and piping failures along the conduit it surrounds.

Anti-vortex device - A device placed at the entrance to a pipe conduit structure to help prevent swirling action and cavitation from reducing the flow capacity of the conduit system.

Aquatic bench - A 10- to 15-foot wide bench around the inside perimeter of a permanent pool that ranges in depth from zero to 12 inches. Vegetated with emergent plants, the bench augments pollutant removal, provides habitats, protects the shoreline from the effects of water level fluctuations, and enhances safety.

Aquifer - A porous, water bearing geologic formation generally restricted to materials capable of yielding an appreciable supply of water.

As-built (drawing) - Drawing or certification of conditions as they were actually constructed.

Atmospheric Deposition - The process by which atmospheric pollutants reach the land surface either as dry deposition or as dissolved or particulate matter contained in precipitation.

Average land cover condition - The percentage of impervious cover considered to generate an equivalent amount of phosphorus as the total combined land uses within the Chesapeake Bay watershed at the time of the Chesapeake Bay Preservation Act adoption, assumed to be 16%. Note that a locality may opt to calculate actual watershed specific values for the average land cover condition based upon 4VAC 3-20-101.

Baffle - Guides, grids, grating or similar devices placed in a pond to deflect or regulate flow and create a longer flow path from the inlet to the outlet structure.

Bankfull flow - Condition where flow fills a stream channel to the top of bank and at a point where the water begins to overflow onto a floodplain.

Barrel - Closed conduit used to convey water under or through an embankment, part of the principal spillway.

Base flow - Discharge of water independent of surface runoff conditions, usually a function of groundwater levels.

Basin - A facility designed to impound stormwater runoff.

Best Management Practice (BMP) - Structural or nonstructural practice which is designed to minimize the impacts of changes in land use on surface and groundwater systems. Structural BMP refers to basins or facilities engineered for the purpose of reducing the pollutant load in stormwater runoff, such as Bioretention, constructed stormwater wetlands, etc. Nonstructural BMP refers to land use or development practices which are determined to be effective in minimizing the impact on receiving stream systems, such as preservation of open space and stream buffers, disconnection of impervious surfaces, etc.

Biochemical Oxygen Demand (BOD) - An indirect measure of the concentration of biologically degradable material present in organic wastes. It usually reflects the amount of oxygen consumed in five days by biological processes breaking down organic waste.

Biological Processes - A pollutant removal pathway in which microbes break down organic pollutants and transform nutrients.

Bioretention basin - Water quality BMP engineered to filter the water quality volume through an engineered planting bed, consisting of a vegetated surface layer (vegetation, mulch, ground cover), planting soil, and sand bed (optional), and into the in-situ material. Also called rain gardens.

Bioretention filter - A bioretention basin with the addition of a sand layer and collector pipe system beneath the planting bed.

CBLAD - Chesapeake Bay Local Assistance Department (Virginia state agency).

COE - United States Army Corps of Engineers

Catch Basin - An inlet chamber usually built at the curb line of a street or low area, for collection of surface runoff and admission into a sewer or subdrain. These structures commonly have a sediment sump at its base, below the sewer or subdrain discharge elevation, designed to retain solids below the point of overflow.

Channel - A natural or manmade waterway.

Channel stabilization - The introduction of natural or manmade materials placed within a channel so as to prevent or minimize the erosion of the channel bed and/or banks.

Check dam - Small dam constructed in a channel for the purpose of decreasing the flow velocity, minimize channel scour, and promote deposition of sediment. Check dams are a component of grassed swale BMPs.

Chemical Oxygen Demand (COD) - A measure of the oxygen required to oxidize all compounds, both organic and inorganic, in water.

Chute - A high velocity, open channel for conveying water to a lower level without erosion.

Compaction - The process by which soil grains are rearranged so as to decrease void space and bring them in closer contact with one another, thereby reducing the permeability and increasing the soils unit weight, and shear and bearing strength.

Conduit - Any channel intended for the conveyance of water, whether open or closed.

Constructed stormwater wetlands - Areas intentionally designed and created to emulate the water quality improvement function of wetlands for the primary purpose of removing pollutants from stormwater.

Contour - A line representing a specific elevation on the land surface or a map.

Cradle - A structure usually of concrete shaped to fit around the bottom and sides of a conduit to support the conduit, increase its strength and, in dams, to fill all voids between the underside of the conduit and soil.

Crest - The top of a dam, dike, spillway or weir, frequently restricted to the overflow portion.

Curve number (CN) - A numerical representation of a given area's hydrologic soil group, plant cover, impervious cover, interception and surface storage derived in accordance with Natural Resource Conservation Service methods. This number is used to convert rainfall depth into runoff volume. Sometimes referred to as Runoff Curve Number.

Cut - A reference to an area or material that has been excavated in the process of a grading operation.

Dam - A barrier constructed for the purpose of confining or impounding water.

DCR - Virginia Department of Conservation and Recreation.

DEQ - Virginia Department of Environmental Quality.

Design Storm - A selected rainfall heyetograph of specified amount, intensity, duration and frequency that is used as a basis for design.

Detention - The temporary impoundment or holding of stormwater runoff.

Detention Basin - A stormwater management facility which temporarily impounds runoff and discharges it through a hydraulic outlet structure to a downstream conveyance system. While a certain amount of outflow may also occur via infiltration through the surrounding soil, such amounts are negligible when compared to the outlet structure discharge rates and, therefore, are not considered in the facility's design. Since an extended detention basin impounds runoff only temporarily, it is normally dry during nonrainfall periods. See MS 3.08.

Dike - An embankment, usually linear, to confine or direct water.

Discharge - Flow of water across the land surface or within the confines of a natural or manmade channel, or stream.

Dissolved Oxygen - A form of oxygen found in water that is essential to the life of aquatic species.

Disturbed area - An area in which the natural vegetative soil cover or existing surface treatment has been removed or altered and, therefore, is susceptible to erosion.

Diversion - A channel or dike constructed to direct water to areas where it can be used, treated, or disposed of safely.

Drainage basin - An area of land that contributes stormwater runoff to a designated point. Also called a drainage area or, on a larger scale, a watershed.

Drop structure - A manmade device constructed to transition water to a lower elevation.

Duration - The length of time over which precipitation occurs.

EPA - The United States Environmental Protection Agency.

Embankment - A man-made deposit of soil, rock or other material used to form an impoundment.

Emergency Spillway - A channel, usually an open channel constructed adjacent to an embankment, which conveys flows in excess of the design capacity of the principal spillway.

Energy dissipator - A device used to reduce the velocity or turbulence of flowing water.

Erosion - The wearing away of the land surface by running water, wind, ice or other geological agents.

> *Accelerated erosion* - erosion in excess of what is presumed or estimated to be naturally occurring levels and which is a direct result of human activities.
>
> *Gully erosion* - erosion process whereby water accumulates in narrow channels and removes the soil to depths ranging from a few inches to 1 or 2 feet to as much as 75 to 100 feet.
>
> *Rill erosion* - erosion process in which numerous small channels only several inches deep are formed.
>
> *Sheet erosion* - spattering of small soil particles caused by the impact of raindrops on wet soils. The loosened and spattered particles may subsequently be removed by surface runoff.

Eutrophication - The process of over-enrichment of water bodies by nutrients often typified by the presence of algal blooms.

Extended detention basin - A stormwater management facility which temporarily impounds runoff and discharges it through a hydraulic outlet structure over a specified period of time to a downstream conveyance system for the purpose of water quality enhancement or stream channel erosion control. While a certain amount of outflow may also occur via infiltration through the surrounding soil, such amounts are negligible when compared to the outlet structure discharge rates and, therefore, are not considered in the facility's design. Since an extended detention basin impounds runoff only temporarily, it is normally dry during nonrainfall periods.

Extended detention basin-enhanced - An extended detention basin modified to increase pollutant removal by providing a shallow marsh in the lower stage of the basin.

Exfiltration - The downward movement of runoff through the bottom of a stormwater facility and into the soil.

Fill - A reference to an area or material that has been placed by mechanical equipment in the process of a grading operation.

Filter bed - The section of a constructed filtration device that houses the filtering media.

Filter Strip - An area of vegetation, usually adjacent to a developed area, constructed to remove sediment, organic matter, and other pollutants from runoff in the form of sheet flow.

First flush - The first portion of runoff, usually defined as a depth in inches, considered to containing the highest pollutant concentration resulting from a rainfall event.

Flooding - When the volume or rate flow exceeds the capacity of the natural or manmade conveyance system and overflows onto adjacent lands, causing or threatening damage.

Floodplain - For a given flood event, that area of land adjoining a continuous water course which has been covered temporarily by water.

Flow splitter - An engineered hydraulic structure designed to divert a portion of storm flow to a BMP located out of the primary channel, or to direct stormwater to a parallel pipe system, or to bypass a portion of baseflow around a BMP.

Forebay - Storage space, commonly referred to as a sediment forebay, located near a stormwater BMP inlet that serves to trap incoming coarse sediments before they accumulate in the main treatment area.

Freeboard - Vertical distance between the surface elevation of the design high water and the top of a dam, levee, or diversion ridge.

French drain - A type of drain consisting of an excavated trench filled with pervious material such as coarse sand, gravel or crushed stone, through whose voids water percolates and exfiltrates into the soil.

Frequency (design storm frequency) - The recurrence interval of storm events having the same duration and volume. The frequency of a specified design storm can be expressed either in terms of exceedence probability or return period.

> *Exceedence probability* - The probability that an event having a specified volume and duration will be exceeded in one time period, usually assumed to be one year. If a storm has a one percent chance of occurring in any given year, than it has an exceedence probability of 0.01.
> *Return period* - The average length of time between events having the same volume and duration. If a storm has a one percent chance of occurring in any given year, than it has a return period of 100 years.

GIS - Geographic Information System. A method of overlaying spatial land and land use data of different kinds. The data are referenced to a set of geographical coordinates and encoded in a computer software system. GIS is used by many localities to map utilities and sewer lines and to delineate zoning areas.

Gabion - A flexible woven wire basket composed of rectangular cells filled with large cobbles or riprap. Gabions may be assembled into many types of structures such as revetments, retaining walls, channel liners, drop structures, diversions, check dams, and groins.

Grade - The slope of a specific surface of interest such as a road, channel bed or bank, top of embankment, bottom of excavation, or natural ground. Grade is commonly measured in percent (unit of measurement per one hundred units) or a ratio of horizontal to vertical distance.

Grassed swale - An earthen conveyance system which is broad and shallow with check dams and vegetated with erosion resistant and flood tolerant grasses, engineered to remove pollutants from stormwater runoff by filtration through grass and infiltration into the soil.

Green Alleys - A network of bioretention basins, infiltration trenches or bioretention filters that provide both redundant water quality management and stormwater conveyance.

HEC-1 - Hydraulic Engineering Circular - 1; a rainfall-runoff event simulation computer model sponsored by the U.S. Corps of Engineers.

Head - The height of water above any plane or object of reference; also used to express the energy, either kinetic or potential, measured in feet, possessed by each unit weight of a liquid.

Hydraulics - The physical science and technology of the static and dynamic behavior of fluids.

Hydric soil - A soil that is saturated , flooded, or ponded long enough during the growing season to develop anaerobic conditions in the upper part.

Hydrodynamic structure - An engineered flow through structure which uses gravitational settling to separate sediments and oils from stormwater runoff.

Hydrograph - A plot showing the rate of discharge, depth or velocity of flow versus time for a given point on a stream or drainage system.

Hydrologic cycle - A continuous process by which water is cycled from the oceans to the atmosphere to the land and back to the oceans.

Hydrologic Soil Group (HSG) - SCS classification system of soils based on the permeability and infiltration rates of the soils. '*A*' type soils are primarily sandy in nature with a high permeability while '*D*' type soils are primarily clayey in nature with a low permeability.

Hydrology - Science dealing with the distribution and movement of water.

Hyetograph - A graph of the time distribution of rainfall over a watershed.

Impervious cover - A surface composed of any material that significantly impedes or prevents natural infiltration of water into soil. Impervious surfaces include, but are not limited to, roofs, buildings, streets, parking areas, and any concrete, asphalt, or compacted gravel surface.

Impoundment - An artificial collection or storage of water, as a reservoir, pit, dugout, sump, etc.

Industrial Stormwater Permit - NPDES permit issued to a commercial industry for regulating the pollutant levels associated with industrial stormwater discharges. The permit may specify on-site pollution control strategies.

Infiltration facility - A stormwater management facility which temporarily impounds runoff and discharges it via infiltration through the surrounding soil. While an infiltration facility may also be equipped with an outlet structure to discharge impounded runoff, such discharge is normally reserved for overflow and other emergency conditions. Since an infiltration facility impounds runoff only temporarily, it is normally dry during nonrainfall periods. Infiltration basin, infiltration trench, infiltration dry well, and porous pavement are considered infiltration facilities.

Initial abstraction - The maximum amount of rainfall that can be absorbed under specific conditions without producing runoff. Also called initial losses.

Intensity - The depth of rainfall divided by duration.

Invert - The lowest flow line elevation in any component of a conveyance system, including storm sewers, channels, weirs, etc.

Karst topography - Regions that are characterized by formations underlain by carbonate rock and typified by the presence of limestone caverns and sinkholes.

Kjeldahl Nitrogen (TKN) - A measure of the ammonia and organic nitrogen present in a water sample.

Lag time - The interval between the center of mass of the storm precipitation and the peak flow of the resultant runoff.

Land development - A manmade change to, or construction on, the land surface that changes its runoff characteristics. Certain types of land development are exempted from stormwater management requirements as provided in the Stormwater Management Act, § 10.1-603.8 B of the Code of Virginia.

Landscaping - The placement of vegetation in and around stormwater management BMP's.

Linear development project - A land development project that is linear in nature such as , but not limited to, (I) the construction of electric and telephone utility lines, and natural gas pipelines; (ii) construction of tracks, rights-of-way, bridges, communication facilities and other related structures of a railroad company; and (iii) highway construction projects.

Locality - A county, city, or town.

Low Impact Development (LID) - Hydrologically functional site design with pollution prevention measures to reduce impacts and compensate for development impacts on hydrology and water quality.

Manning's formula - Equation used to predict the velocity of water flow in an open channel or pipeline.

Marsh - A wet area, periodically inundated with standing or slow moving water, that has grassy or herbaceous vegetation and often little peat accumulation; the water may be salt, brackish or fresh.

Micropool - A smaller permanent pool which is incorporated into the design of larger stormwater ponds to avoid resuspension of particles, provide varying depth zones, and minimize impacts to adjacent natural features.

Modified Rational Method - A variation of the rational method used to calculate the critical storage volume whereby the storm duration can vary and does not necessarily equal the time of concentration.

Mulch - Any material such as straw, sawdust, leaves, plastic film, loose soil, wood chips, etc. that is spread or formed upon the surface of the soil to protect the soil and/or plant roots from the effects of raindrops, soil crusting, freezing, evaporation, etc.

Municipal Stormwater Permit - NPDES permit issued to municipalities to regulate discharges from municipal separate storm sewers for compliance with EPA regulations and specify stormwater control strategies.

National Pollutant Discharge Elimination System (NPDES) - The national program for issuing, modifying, monitoring and enforcing permits under Sections 307, 402, 318 and 405 of the Clean Water Act.

Nonpoint source pollution - Contaminants such as sediment, nitrogen and phosphorous, hydrocarbons, heavy metals, and toxins whose sources cannot be pinpointed but rather are washed from the land surface in a diffuse manner by stormwater runoff.

Normal depth - Depth of flow in an open conduit during uniform flow for the given conditions.

Off-line - Stormwater management system designed to manage a portion of the stormwater which has been diverted from a stream or storm drain. A flow splitter is typically used to divert the desired portion of the flow.

On-line - Stormwater management system designed to manage stormwater in its original stream or drainage channel.

Outfall - Place where effluent is discharged into receiving waters.

Peak discharge - The maximum rate of flow at associated with a given rainfall event or channel.

Percolation rate - The velocity at which water moves through saturated, granular material.

pH - An expression of the intensity of the basic or acidic condition of a liquid. Natural waters usually have a pH range between 6.5 and 8.5.

Phosphorus - An element found in fertilizers and sediment runoff which can contribute to the eutrophication of water bodies. It is the keystone pollutant in determining pollutant removal efficiencies for various BMP's as defined by the Virginia Stormwater Management Regulations.

Planning area - A designated portion of the parcel on which a land development project is located. Planning areas must be established by delineation on a master plan. Once established, planning areas must be applied consistently for all future projects.

Point Source - The discernible, confined and discrete conveyance, including but not limited to, any pipe, ditch, channel, tunnel, conduit, well, container, concentrated animal feeding operation, landfill leachate collection system from which pollutants may be discharged. This term does not include return flows from irrigated agriculture or agricultural storm water runoff.

Porosity - The ratio of pore or open space volume to total solids volume.

Post-development - Refers to conditions that reasonably may be expected or anticipated to exist after completion of the land development activity on a specific site or tract of land.

Pre-development - Refers to the conditions that exist at the time that plans for the land development of a tract of land are approved by the plan approval authority. Where phased development or plan approval occurs (preliminary grading, roads and utilities, etc.), the existing conditions at the time prior to the first item being approved or permitted establishes the pre-development conditions.

Pretreatment - The techniques employed in a stormwater management plan to provide storage or filtering to help trap coarse materials before they enter the stormwater BMP. Pretreatment is required on some BMPs to help avoid costly maintenance.

Principal spillway - The primary spillway or conduit for the discharge of water from an impoundment facility; generally constructed of permanent material and designed to regulate the rate of discharge.

Rational method - Means of computing peak storm drainage flow rates based on average percent imperviousness of the site, mean rainfall intensity, and drainage area.

Recharge - Replenishment of groundwater reservoirs by infiltration and transmission of water through permeable soils.

Redevelopment - Any construction, alteration, or improvement on existing development.

Retention - Permanent storage of stormwater.

Retention basin - A stormwater management facility which includes a permanent impoundment, or normal pool of water, for the purpose of enhancing water quality and, therefore, is normally wet, even during nonrainfall periods. Storm runoff inflows may be temporarily stored above this permanent impoundment for the purpose of reducing flooding, or stream channel erosion.

Rip-rap - Broken rock, cobbles or boulders placed on earth surfaces such as the face of a dam or the bank of a stream for the protection against erosive forces such as flow velocity and waves.

Riser - A vertical structure which extends from the bottom of an impoundment facility and houses the control devices (weirs/orifices) to achieve the desired rates of discharge for specific designs.

Roughness coefficient - A factor in velocity and discharge formulas representing the effect of channel roughness on energy losses in flowing water. Manning's '*n*' is a commonly used roughness coefficient.

Routing - A method of measuring the inflow and outflow from an impoundment structure while considering the change in storage volume over time.

Runoff - The portion of precipitation, snow melt or irrigation water that runs off the land into surface waters.

Runoff coefficient - The fraction of total rainfall that appears as runoff. Represented as *C* in the rational method formula.

SCS - Soil Conservation Service (now called Natural Resource Conservation Service, NRCS), a branch of the U.S. Department of Agriculture.

Safety bench - A flat area above the permanent pool and surrounding a stormwater pond designed to provide a separation to adjacent slopes.

Sand filter - A contained bed of sand which acts to filter the first flush of runoff. The runoff is then collected beneath the sand bed and conveyed to an adequate discharge point or infiltrated into the in-situ soils.

Sediment - Material, both mineral and organic, that is in suspension, is being transported, or has been moved from its site of origin by water or wind. Sediment piles up in reservoirs, rivers and harbors, destroying wildlife habitat and clouding water so that sunlight cannot reach aquatic plants.

Sediment Forebay - A settling basin or plunge pool constructed at the incoming discharge points of a stormwater facility.

Sedimentation (or settling) - A pollutant removal method to treat stormwater runoff in which gravity is utilized to remove particulate pollutants. Pollutants are removed from the stormwater as sediment settles or falls out of the water column. An example of a BMP utilizing sedimentation is a detention basin.

Shallow marsh - A zone within a stormwater extended detention basin that exists from the surface of the normal pool to a depth of six to 18 inches, and has a large surface area and, therefore, requires a reliable source of baseflow, groundwater supply, or a sizeable drainage area, to maintain the desired water surface elevations to support emergent vegetation.

Silviculture - A branch of forestry dealing with the development and care of forests.

Site - The parcel of land being developed, or a designated planning area in which a land development project is located.

Soil science - Science dealing with soils as a natural resource on the surface of the earth including soil formation, classification, mapping; physical, chemical, biological, and fertility properties of soils per se; and these properties in relation to the use and management of soils.

Soil test - Chemical analysis of soil to determine the need for fertilizers or amendments for species of plant being grown.

Soil texture - Relative proportion of the physical components of any given soil. For instance, clay is defined as soil having >40% clay, <45% sand and <40% silt.

Stage - Water surface elevation above any chosen datum.

State Project - Any land development project which is undertaken by any state agency, board, commission, authority or any branch of state government, including state supported institutions of higher learning.

Storm Sewer - A system of pipes, separate from sanitary sewers, that only carries runoff from buildings and land surfaces.

Stormwater Filtering (or filtration) - A pollutant removal method to treat stormwater runoff in which stormwater is passed through a filter media such as sand, peat, grass, compost, or other materials to strain or filter pollutants out of the stormwater.

Stormwater Hot spot - An area where the land use or activities are considered to generate runoff with concentrations of pollutants in excess of those typically found in stormwater.

Stormwater management facility - A device that controls stormwater runoff and changes the characteristics of that runoff including, but not limited to, the quantity and quality, the period of release or the velocity of flow.

Stormwater management plan - A document containing material for describing how existing runoff characteristics will be affected by a land development project and methods for complying with the requirements of the local program or this chapter.

Stream buffers - The zones of variable width which are located along both sides of a stream and are designed to provide a protective natural area along a stream corridor.

Surcharge - Flow condition occurring in closed conduits when the hydraulic grade line is above the crown of the sewer. This condition usually results localized flooding or stormwater flowing out the top of inlet structures and manholes.

SWMM (Storm Water Management Model) - Rainfall-runoff event simulation model sponsored by the U.S. Environmental Protection Agency.

Technical Release No. 20 (TR-20) - Project Formulation - Hydrology. SCS watershed hydrology computer model that is used to compute runoff volumes and route storm events through stream valleys and/or impoundments.

Technical Release No. 55 (TR-55) - Urban Hydrology for Small Watersheds. SCS watershed hydrology computation model that is used to calculate runoff volumes and provide a simplified routing for storm events through stream valleys and/or ponds.

Time of concentration - The time required for water to flow from the hydrologic most distant point (in time of flow) of the drainage area to the point of analysis (outlet). This time will vary, generally depending on the slope and character of the surfaces.

Total Suspended Solids (TSS) - The total amount of particulate matter which is suspended in the water column.

Trash rack - A structural device used to prevent debris from entering a spillway or other hydraulic structure.

Travel time - The time required for water to flow from the outlet of a drainage sub-basin to the outlet of the entire drainage basin being analyzed. Travel time is normally concentrated flow through an open or closed channel.

Turbidity - Cloudiness of a liquid, caused by suspended solids; a measure of the suspended solids in a liquid.

Ultimate condition - Full watershed build-out based on existing zoning.

Ultra-urban - Densely developed urban areas in which little pervious surface exists.

Urban runoff - Stormwater from city streets and adjacent domestic or commercial properties that carries nonpoint source pollutants of various kinds into the sewer systems and receiving waters.

VDOT - The Virginia Department of Transportation.

VESCH - The Virginia Erosion and Sediment Control Handbook, latest edition.

Water quality standards - State-adopted and EPA-approved ambient standards for water bodies. The standards prescribe the use of the water body and establish the water quality criteria that must be met to protect designated uses.

Water quality volume - The volume equal to the first ½ inch of runoff multiplied by the impervious surface of the land development project as defined by the Virginia Stormwater Management Regulations.

Water surface profile - Longitudinal profile assumed by the surface of a stream flowing in an open channel; hydraulic grade line.

Water table - Upper surface of the free groundwater in a zone of saturation.

Watershed - A defined land area drained by a river, stream, or drainage way, or system of connecting rivers, streams, or drainage ways such that all surface water within the area flows through a single outlet.

Weir - A wall or plate placed in an open channel to regulate or measure the flow of water.

Wet weather flow - Combination of dry weather flows and stormwater runoff.

Wetted perimeter - The length of the wetted surface of a natural or manmade channel.