TECHNICAL MEMORANDUM

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Review of

PRE-DEVELOPMENT RUNOFF ANALYSIS METHODS

Volume I

By

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ABSTRACT

This is the first of a two volume report which reviews and provides decision-making information about available techniques for evaluation of pre-development runoff conditions. Volume I reviews most frequently used methods to estimate peak runoff rates, runoff volumes, and the time distribution of flow for a site to be developed or redeveloped. These methods are discussed with regard to their theoretical assumptions and limitations. In most cases, examples of a method's application are included as well.

Peak runoff estimation methods include: the Rational Method, several SCS methods, the SFWMD Sheetflow Procedure, and the Cypress Creek Formula. Those runoff volume estimation methods reviewed are: the SCS Curve Number method, the SFWMD Procedure, CREAMS, CREAMS-WT and infiltration estimations. Methods for describing the time distribution of runoff are discussed: several synthetic unit hydrograph methods are presented, along with discussions of the Santa Barbara Urban Hydrograph method, and the U.S. Army Corps of Engineer's HEC-1 flood hydrograph package. Both hydrologic and hydraulic routing theory is described. Those hydrologic routing methods discussed are: Modified Puls, Muskingum, and Convex. The assumptions and limitations of the kinematic, diffusion, and dynamic wave hydraulic routing approaches are briefly summarized, along with two common computer packages: the National Weather Service's Dynamic Wave Operational Model, and EPA's Stormwater Management Model.

Volume I of this review deals only with the theoretical assumptions and limitations of the above methods. Volume II, scheduled to be published in fiscal year

1990-99, will provide a detailed comparison of these same methods, and a more quantitative assessment of their applicability under the hydrologic conditions of south Florida.

KEY WORDS: DWOPER Model, flood routing, HEC-1, hydrographs, hydrologic analysis, hydrology, infiltration, methodology, runoff, Soil Conservation Service, storm water drainage design, storm water management, Storm Water Management Model, storm water runoff, surface water runoff, time of concentration, TR-20, TR-55, unit hydrographs

EXECUTIVE SUMMARY

The South Florida Water Management District's (SFWMD) surface water management regulatory program requires, for some basins, that post-development runoff rates remain equal to those prior to development. Post-development runoff rates are estimated using the surface water management system design. This, however, is not the case with pre-development runoff. The literature contains a multitude of techniques for estimating pre-development runoff rates and volumes. Each method has its own limitations and applicability. In many cases, one estimation method is more appropriate for a given situation than another.

This review is directed towards SFWMD permit reviewers, and is intended as an aid in decision-making. Herein, several pre-development runoff estimation methods are presented and discussed with regards to their theoretical assumptions and limitations and the process by which the method is applied. It is hoped that this review will provide a reasonably complete catalog of pre-development runoff estimation methods commonly used in south Florida. In addition, the reader should be able to make a better assessment of the application of a particular method in a given situation.

To summarize briefly, this review discusses methods to estimate several aspects of pre-development runoff:

1. *Time of Concentration*. Time of concentration is defined as the time required, during a storm, for the entire basin area to contribute to the outflow. It is an important parameter in several runoff estimation methods, and procedures for its estimation are widely varied. The

methods discussed in this review include: the SCS Upland Method; SCS Sheetflow method; SCS Method for Shallow Concentrated Flow; methods used for open channels; a hydrograph method; SCS Modified Curve Number method; and Akan's method.

2. *Runoff Peak.* The peak discharge rate which occurs during a storm is a necessary quantity for sizing storm sewers and other discharge structures. The most common peak estimation method is the Rational method, which is discussed herein. Other methods discussed are the SCS Chart and Graphical methods, SFWMD Sheetflow Procedure, and the Cypress Creek Formula.

3. *Runoff Volume.* The total amount of surface water which flows from a basin as the result of a storm is the runoff volume. This information is necessary to size detention/retention systems. Three methods for runoff volume estimation are discussed in this review: SCS Curve Number method, SFWMD Runoff Volume procedure, and methods using estimates of infiltration. Also presented are two simulation models for runoff volume: CREAMS and CREAMS - WT.

4. *Time Distribution of Runoff.* At times, it is useful to have a complete description of a basin's outflow as a result of a storm. This review divides discussion of methods which describe the time distribution of runoff into hydrograph and routing methods. Hydrograph methods discussed here include synthetic unit hydrographs, the Santa Barbara Urban Hydrograph, and the U.S. Army Corps of Engineer's HEC-1 flood

hydrograph package. These are used to estimate, over time, a watershed's outflow as the result of a storm.

5. *Flood Routing*. In some cases, especially for large basins, a flood routing analysis is necessary, in addition to runoff computations. Routing methods are used to predict the time distribution of runoff at one point given the time distribution at a point upstream. These are divided into two categories: hydrologic and hydraulic. The Modified Puls, Muskingum, and Convex hydrologic routing methods are discussed in this review. Some aspects of kinematic, diffusion, and dynamic wave hydraulic routing are discussed as well.

Most of these methods were developed under hydrologic conditions dissimilar to those found in south Florida. Most pre-development applications in south Florida are characterized by flat slopes, backwater conditions, and high water tables. As a consequence, many of these methods may not be directly applicable to situations found in South Florida. Any analysis method must be chosen carefully in accordance with the pertinent conditions of the area under consideration.

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1. INTRODUCTION

1.1. Subject

This review discusses a variety of runoff estimation methods commonly used in South Florida. Runoff analysis methods are those by which a watershed's[1] surface water outflow (runoff), as a consequence of rainfall events (storms), is estimated or described mathematically. This runoff information provides the basis for the planning, design, and construction of drainage facilities.

Many runoff analysis methods have been developed. Some of the methods are based on physical hydrologic and hydraulic principles. Others are empirical or semi-empirical. Although these methods are applied in South Florida, for the most part, they have been developed elsewhere. Inaccuracies often arise when these methods are applied to the unusual hydrologic conditions found in south Florida's watersheds. The typical South Florida watershed has a very flat land slope, highly permeable, sandy soils, high water tables, and wetlands or ponds scattered throughout the basin. They are not typical of the watersheds for which most methods were developed.

In the South Florida Water Management District's (SFWMD) surface water management regulatory program, the allowable discharge criteria for many basins requires that post-development discharge rates remain less than or equal to pre-development rates. Often the pre-development condition of a site and its surroundings is more appropriately analyzed by one method than another.

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[1]The terms "watershed", "basin", and "drainage area" are used interchangeably throughout this review. Refer to the Glossary for a definition.

Consequently, engineers and reviewers need to understand the underlying assumptions and limitations of runoff analyses in order to select and apply methods appropriate for a given case.

1.2. Purpose

This review is written for SFWMD permit reviewers, and is intended as a reference for decision-making. In any design situation, there are many analysis methodologies from which to choose. The final selection of a method will depend on any of the following:

1. *Does the method theoretically fit the circumstances?* Any methodology for estimating runoff is based on a set of assumptions. This would include theoretical assumptions used to derive the method, and empirical conditions used to test the method. In many cases, methods for estimating runoff have been developed for areas other than South Florida, and may provide unreasonable values for runoff peak, volume and timing.

2. *Is enough information available to apply the method?* Any method for estimation of runoff will require information about basin configuration. A particular method may be most appropriate from a theoretical point of view, but the information required by the method may not be available or too costly to acquire.

3. *Is the available information accurate enough to provide an accurate estimate of runoff?* The accuracy of any runoff estimation method depends upon the accuracy of the information the method requires. The design engineer must realize how important each method parameter is to the

final estimation. For example, if the estimation of basin area can only be estimated to with ±10 percent, the estimate of runoff peak may be in error by 10 percent, or the error may be more.

4. *Does the method require calibration and/or verification?* Many methods require parameters which are not directly measurable. These parameters must be estimated by using a past event which was measured. In many cases, previously measured events are not available.

This review is divided into two volumes. Volume I discusses several commonly used runoff estimation methods in with regard to points 1 and 2 above, that is the theoretical basis for a method and its data requirements. Volume I describes various methods for estimating runoff, and the assumptions and limitations upon which the methods are based. Volume I should provide a reasonably complete catalog of runoff analysis methods commonly used in south Florida. Furthermore, the reader will have enough information to appraise particular analyses with regard to theoretical assumptions and limitations of the method's use. Volume II addresses point 3 above for the methods discussed in Volume I, and assess their application to South Florida situations. Point 4 is not discussed in either Volume I or Volume II. The process by which a model is calibrated or verified is usually specific to the model being applied. This detail would be beyond the scope of this review, and therefore is not included.

This review is not a manual for runoff analysis, and is not a substitute for engineering judgement. Procedures for certain runoff analyses are merely outlined. Volume I limits discussion to <u>theoretical</u> limitations and data requirements involved in the application of a particular method. Volume I serves only as a catalog of various methods Volume II will address limitations for South Florida applications as much as possible. Volume II will provide a detailed comparison of these methods, discuss sensitivities to input parameters, and assess the applicability to south Florida conditions. Both Volume I and Volume II will be updated from time to time as more information becomes available.

1.3. Background

1.3.1. FACTORS AFFECTING RUNOFF

The primary source of runoff in south Florida is rainfall. However, not all the rainfall is converted to surface runoff. Collectively, portions of rainfall which are not destined to become surface water runoff are termed *abstractions,* or *runoff losses.* There are several factors which affect the relative magnitude of runoff abstractions, and hence, surface runoff. There are also factors which can affect the time required for runoff to reach the outlet. In this section, several terms are defined which describe these factors. These terms are used throughout the review.

Infiltration is the process by which rain water percolates through the soil surface. The amount of infiltration which occurs depends upon the soil type, moisture content, organic matter present, vegetation cover, depth to the groundwater table, and rainfall intensity. In many cases, infiltration is considered the largest, if not the only, surface runoff loss.

Interception is rainfall which is caught on vegetative or man-made surfaces, such as trees or flat roofs, before reaching the ground. During the first part of the storm, a large portion of the rain can be stored as interception. This water eventually returns to the atmosphere by evaporation or evapotranspiration at a later time. The magnitude of interception which occurs during a storm will vary depending on the vegetative or man-made surfaces available, and rainfall intensity.

Surface detention is a means, natural or man-made, by which runoff is delayed in reaching the basin outlet. The magnitude of surface detention will generally not affect the total volume of runoff, but can change its time distribution.

Surface retention, or *depression storage* is that portion of rainfall which collects in natural or man-made depressions or ponds, and remains within the basin after a runoff event has effectively concluded. Surface retention is considered a runoff loss.

Interflow is the portion of the water which infiltrates the soil surface and moves laterally through the upper layers of the soil until it re-emerges as surface water runoff. The amount of interflow is dependent upon the soil type and the geology of the watershed under consideration. Under pre-development conditions, especially for typical South Florida watersheds, interflow can have a significant effect on the timing and magnitude of surface water peak discharges. Typically, these effects are not considered in design runoff estimations.

Deep percolation is the process by which water which has infiltrated to the surface soil layer flows into lower soil layers and the groundwater. Rising groundwater can contribute to basin runoff. Additions to groundwater from deep percolation can cause water from the saturated groundwater system at some down gradient point to contribute to channel runoff. The amount of the groundwater contribution to the channel depends upon the level at which the water table intersects the channel.

Time of concentration is defined as the time required, during a storm, for the entire basin area to contribute to the surface water outflow. It is a useful basin characteristic which is used in several analyses. The time of concentration is affected by the watershed's surface conditions (such as vegetation types and land use), land slopes, soil types, and surface water management.

Baseflow can be considered to be the "constant" outflow from a basin, that is, the portion of surface water outflow which is (mostly) unaffected by rainfall. During intervals between storms many basins will have a surface water outflow. This flow usually is supplied by groundwater. During storms the groundwater table will rise above streambeds within the basin, and will recede at a much slower rate than streamflow attributed to the storm. Groundwater is then discharged into the stream, and leaves the basin as surface flow. This component of surface water flow can be very important in larger basins, but is usually negligible for small basins.

1.3.2. TYPES OF RUNOFF ANALYSES

Runoff analyses are used by the drainage engineer to provide planning and decision-making information for drainage system design. The analyses described in this review are mostly "event-based" methods, i.e., they describe the basin's response (runoff) to a single rainfall event (storm). Typically, the storm used is carefully chosen, in accordance with regulation, for example, to represent an event which has a certain frequency of recurrence.

Runoff analyses are used to estimate one or more aspects of the basin's flood response. Specifically discussed in this review are methods which estimate:

- peak discharge rate from the basin,
- total discharge volume, and
- the time distribution of runoff.

Peak discharge rate is the maximum flow rate which occurs during the runoff event. This information is necessary to determine the size of outflow structures, for instance. Total runoff volume is all the water which leaves the basin (via surface runoff) during the flood event. This information is used to size runoff detention/retention systems. The time distribution of runoff is a complete description of surface runoff over time. Since they produce more information, time distribution estimation methods require more information about the basin. Peak discharge and total runoff volume are immediately available from the time distribution information.

1.4. Overview

Although a large variety of runoff analyses exist, this review is limited to those methods which are commonly used in south Florida. Each method presented is discussed with regard to:

- theory and mathematical relationships,
- underlying assumptions,
- theoretical limitations on the method's use, and
- the process by which the method is applied.

In Volume I, methods are presented which estimate: time of concentration, peak discharge, total runoff volume, and the time distribution of runoff.

Time of concentration is an important factor in many of the analyses. Methods for estimating time of concentration are as varied as are those for runoff analysis. The following methods are discussed in Section 2 of this review:

- Soil Conservation Service (SCS) Upland method for overland flow;
- SCS method for sheetflow;
- SCS method for shallow, concentrated flow;
- Manning's formula applied to Channel Flow;
- Hydrograph methods;
- the SCS Modified Curve Number method; and
- Akan's method.

Methods for estimation of *peak discharge* are found in Section 3. They are:

- the Rational Method;
- the Soil Conservation Service Tabular and Graphical methods for Florida;
- the SFWMD's Sheetflow Procedure; and
- the Cypress Creek Formula

Runoff volume estimation methods are discussed in Section 4. Those included are:

- the Soil Conservation Service Curve Number Method;
- the SFWMD's Runoff Volume Procedure;
- CREAMS and CREAMS - WT; and
- infiltration estimation methods.

Methods for estimation of the *time distribution of runoff,* or hydrograph methods, are discussed in Section 5. Specific hydrograph methods included are:

- the SCS Dimensionless Curvilinear Unit Hydrograph;

- the SCS Dimensionless Triangular Unit Hydrograph;
- a General Dimensionless Curvilinear Unit Hydrograph;
- the Tracor Unit Hydrograph;
- the Santa Barbara Urban Hydrograph Method;
- the Easy Hydrograph Method;
- HEC-1 (Hydrologic Engineering Center) flood hydrograph package; and
- the SCS TR-20 project formulation program.

An important part of drainage system design, especially in larger basins, is flood routing. Routing is a process whereby the time distribution of surface water discharge at one point is estimated by a known time distribution of discharge at another point. A complete discussion of routing is not within the scope of this review, but some methods and their limitations are briefly discussed in Section 6. Routing methods are divided into two categories: hydrologic and hydraulic. The distinction is discussed in Section 6.1. Three hydrologic routing methods are discussed in Section 6.2:

- the Modified Puls Method;
- the Muskingum Method; and
- the Convex Method.

Hydraulic routing methods are briefly discussed in Section 6.3. An overview of the assumptions and limitations of

- kinematic wave;
- diffusion wave; and

- dynamic wave

hydraulic routing methods are presented there. Also, two hydraulic routing computer models, the Dynamic Wave Operational Model (DWOPER) and EPA's Storm Water Management Model (SWMM), are briefly described.

2. TIME OF CONCENTRATION

2.1. General

Time of concentration is defined time required, during a storm, for the entire basin to contribute to the surface water outflow. This definition is generally agreed upon, but its interpretation is varied. If one were to categorize these interpretations, they might be divided as follows:

- The time of concentration is represented as the time required for a *particle of water* to travel from the most hydraulically remote point in the watershed to the outflow point ("particle travel time").

- The time of concentration is represented by the time required for a *wave* to travel from the most hydraulically remote point in the watershed to the outflow point ("wave travel time").

- The time of concentration is represented by the time, on a discharge hydrograph, from the end of the excess rainfall to the inflection point on the falling limb ("hydrograph inflection", see Figure 5.3 for an example).

These are not equivalent, and hence, will provide significantly different estimates of time of concentration for a single set of conditions. For example, the *wave* travel time is usually significantly less than the *particle* travel time. Time of concentration is one of the most important parameters used in the Rational method (Section 3.1), the Hydrograph methods (Section 5), and other methods described in this review. These are dominated by the methods developed by the Soil Conservation Service

(SCS). The SCS tends to lump all of the above interpretations of time of concentration and call them equivalent. That is generally the approach taken in this review. The reader should be aware, though, that different interpretations of time of concentration will produce different results, and should choose a method to estimate time of concentration which produces the most conservative results. This will of course depend on how time of concentration is to be used.

Time of concentration (T_c) is usually calculated by summing the (particle or wave) travel times (T_t) within a given basin:

$$T_c = T_{t_1} + T_{t_2} + \dots + T_{t_n} \tag{2.1}$$

where the indices 1, 2, ..., n represent connected distinct flow paths upstream of the point under consideration. Travel time includes, for example, overland flow time, gutter flow time, sewer flow time and channel flow time. Usually within a given basin, there is more than one flow path upon which the time of concentration can be based. The time of concentration, for the basin outlet, is the longest of all travel times when more than one path is considered. Further illustration of this point is given in Example 3.1.

There are at least a dozen overland flow formulas in the literature for estimating time of concentration. This section discusses only a few. Table 2.1 lists these methods and which interpretation of time of concentration they are based upon. Most of these methods are empirical, and are limited to the site specific conditions under which they were developed.

TABLE 2.1. METHODS FOR ESTIMATING TIME OF CONCENTRATION DISCUSSED IN SECTION 2.

Section	Time of Concentration Estimation Method	Interpretation of Time of Concentration Used		
		Particle Travel Time	Wave Travel Time	Hydro-graph Inflection
2.2	SCS Upland Method	☒		
2.3	SCS Method for Sheetflow		☒	
2.4	SCS Method for Shallow Concentrated Flow	☒		
2.5	Methods for Open Channels	☒	☒	
2.6	Hydrograph Method			☒
2.7	SCS Modified Curve Number Method			☒
2.8	Akan's Method		☒	

2.2. SCS Upland Method

The SCS (Soil Conservation Service, USDA) Upland method is presented in the National Engineering Handbook, Section 4 (NEH-4, USDA-SCS, 1985). It is a general method which can be used for various land covers and topography. In the Upland method, and others, travel time is computed by dividing the total overland flow length by the average flow velocity:

$$T_t = \frac{L}{3600V} \tag{2.2}$$

where

L = overland flow length, in feet;

V = average flow velocity, in feet per second;

and travel time is computed in hours.

Estimation of average flow velocity is critical to the use of equation (2.2). In the Upland method, average flow velocity is determined from past observation. The SCS studied the average overland flow velocity for various land slopes and land covers. Figure 2.1 summarizes that information.

This method is termed the "Upland" method since it is meant to be applied to upland areas. Upland areas are those in which channel storage is not important, such as near drainage divides. The SCS recommends that the Upland method be applied only to areas less than 2000 acres (USDA-SCS, 1971). Use of Figure 2.1 is limited to the ground slope and cover information shown. Note that this does not include land slopes less than 0.5 percent, which limits application in South Florida. Use of the SCS Upland method is illustrated in Example 3.1.

2.3. SCS Method for Sheetflow

The SCS recommends that the following equation be used to calculate travel time for sheet flow of less than 300 feet (USDA-SCS, 1986):

$$T_t = 0.007 \frac{(nL)^{0.8}}{P_2^{0.5} S_o^{0.4}} \tag{2.3}$$

where

n = Manning's roughness coefficient for sheetflow ;

P_2 = 2-year, 24-hour design rainfall, in inches;

S_o = land slope, in feet per foot;

L = flow length, in feet;

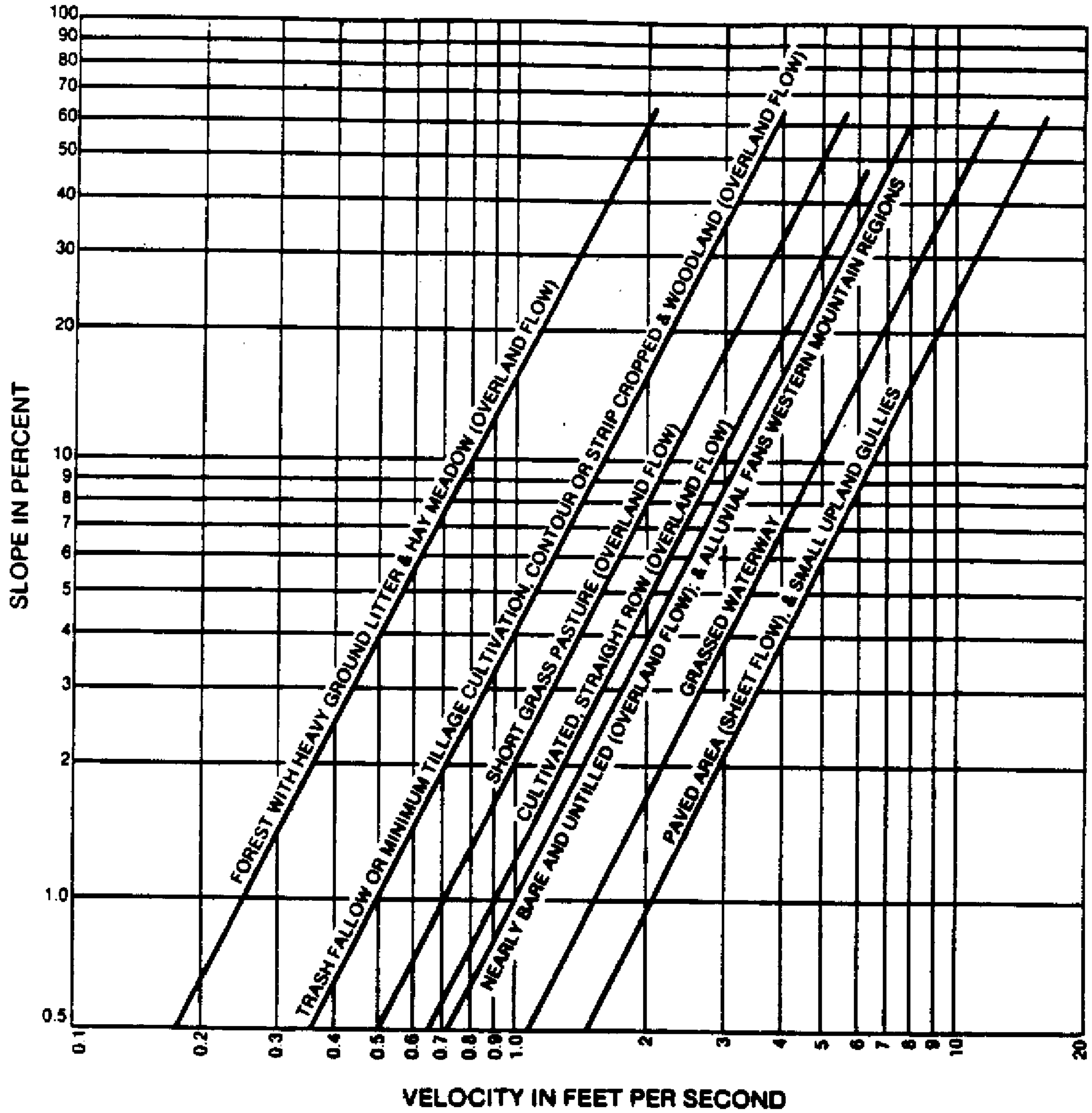


Figure 2.1. Average velocities for estimating travel time with the Upland Method. (reproduced from USDA-SCS, 1985)

TABLE 2.2. ROUGHNESS COEFFICIENTS (MANNING'S n) FOR SHEET FLOW. (Reproduced from USDA - SCS, 1986)

SURFACE DESCRIPTION	n
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[1]	0.24
Bermuda grass	0.41
Range (natural)	0.13
Woods[2]:	
Light underbrush	0.40
Dense underbrush	0.80

[1]Includes species such as weeping love grass, bluegrass, buffalo grass, blue grama grass, and native grass mixtures.

[2]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.

and travel time, T_t is computed in hours. Manning's roughness coefficient used in equation (2.3) should be specifically for sheetflow. Some example values are shown in Table 2.2. The reader should refer to Engman (1986) for a more complete set of values.

Equation (2.3) is a simplified form of Manning's kinematic solution developed by Overton and Meadows (1976). Assumptions used in the simplification were

1. *Flow is steady and uniform with a depth of about 0.1 feet.* The assumptions of uniform flow and an approximately uniform flow depth of 0.1 feet eliminate many areas where overland flow may be deeper. Manning's coefficients can change considerably as flow depth increases.

2. *Rainfall intensity is uniform over the basin.* The assumption of uniform rainfall intensity can be satisfied by considering small watersheds only. In most cases, the assumption that sheetflow conditions exist for only 300 feet places more limitation on this method's uses than does a uniform rainfall intensity assumption.

3. *The rainfall duration is 24 hours.* This method is limited to a 24-hour rainfall duration, which may be unacceptable in some cases. Some design cases may require a different duration.

4. *Infiltration has a minor effect on travel time.* Actual travel time can increase if there is a significant amount of infiltration or surface detention or retention in the basin. This method does not consider these effects, and its application in areas with high infiltration rates may be limited.

5. *Maximum flow length of 300 feet.* The SCS notes (USDA - SCS, 1986) that sheetflow will become shallow concentrated flow within a 300 foot flow length. Hence, equation (2.3) applies only to small areas. It may be incorporated as part of a larger basin, however, by the use of equation (2.1).

2.4. SCS Method for Shallow Concentrated Flow

Sheetflow usually becomes shallow concentrated flow after a maximum of 300 feet over a plane surface (USDA-SCS, 1986). The average velocity for the shallow concentrated flow can be determined from Figure 2.2. The figure is based upon Manning's equation:

$$V = \frac{1.49}{n} R^{\frac{2}{3}} S_o^{\frac{1}{2}} \tag{2.4}$$

where

- $V =$ the average velocity, in feet per second;
- $R =$ hydraulic radius, in feet;
- $S_0 =$ channel bottom slope, in feet per foot; and
- $n =$ Manning's roughness coefficient for open channel flow.

The coefficient 1.49 is usually assigned units of feet$^{1/3}$ per second to make Manning's equation dimensionally correct, (see also the discussion by Chow, 1959, pg. 98). Flow velocity obtained from equation (2.4) can be used with equations (2.2) and (2.1) to calculate time of concentration.

The hydraulic radius, R, and Manning's n are the most sensitive parameters in Manning's equation. The SCS made some very specific assumptions concerning R and n in order to create Figure 2.2. Specifically, a flow depth[1] of 0.4 feet for R and 0.05 for Manning's n were used for unpaved area (USDA-SCS, 1986, Appendix F). For paved areas, n was assumed to be 0.025, and R was assumed 0.2 feet. Some

[1] Hydraulic radius is the ratio of flow area to wetted perimeter. As the flow area becomes wider and more shallow the numeric value of hydraulic radius approaches that of the flow depth.

values for Manning's n are shown in Table 2.3. These are for general conditions and represent reference values only. Manning's n for South Florida conditions may be different, probably much higher, but insufficient field data exists to recommend a set of proper values. Values of n vary considerably for pre-development conditions, as can be seen in Table 2.3. The value of 0.05 used in Figure 2.2 is for fallow surfaces. Tillage can affect the direction of shallow concentrated flow; e.g., if tillage runs across the slope, flow may not always be directly down the watershed slope. Other conditions, such as vegetative cover, also play a significant role.

Use of Figure 2.2 is limited by the information shown and assumptions discussed. Equation (2.4) can be used where Figure 2.2 does not apply, but careful consideration must be given to the selection of values for R and n.

TABLE 2.3. EFFECTIVE ROUGHNESS COEFFICIENTS (MANNING'S n) FOR OVERLAND FLOW. (Reproduced from USACE - HEC, 1981)

SURFACE DESCRIPTION	n
Dense Growth*	0.4 -- 0.5
Pasture*	0.3 -- 0.4
Lawn*	0.2 -- 0.3
Bluegrass Sod**	0.2 -- 0.5
Short Grass Prairie**	0.1 -- 0.2
Sparse Vegetation**	0.05 -- 0.13
Bare Clay-Loam Soil(Eroded)**	0.01 -- 0.03

* from Crawford and Linsley (1966)
** from Woolhiser (1975)

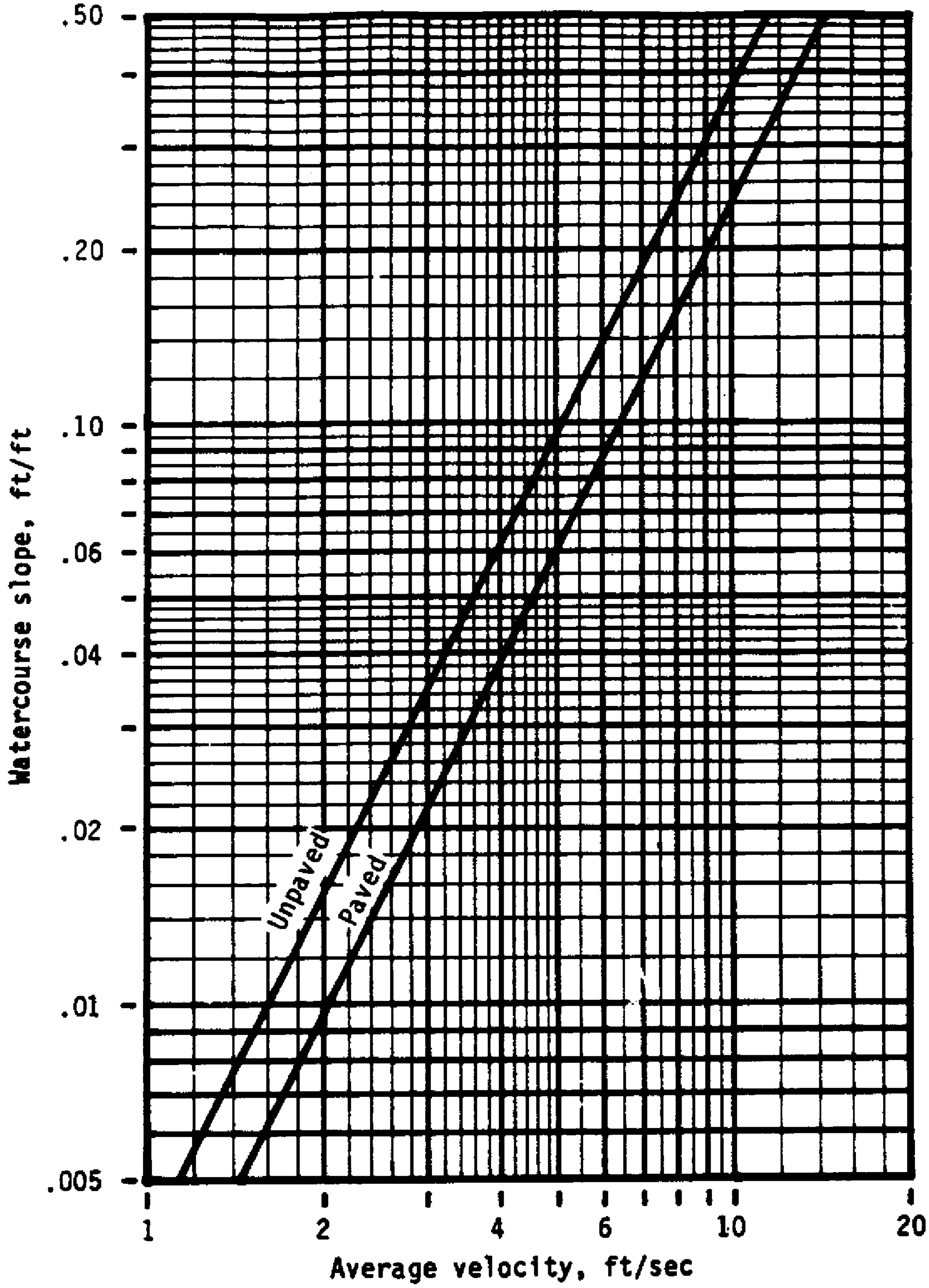


Figure 2.2. Average velocities for estimating travel time for shallow concentrated flow. (reproduced from USDA-SCS, 1986a)

2.5. Methods for Open Channels

Estimation of average flow velocity through open channels can be made using Manning's equation (equation (2.4)) or water surface profile information, if available. In most cases, a channel's average flow velocity is determined for the bank-full condition. Manning's coefficient for equation (2.4) should be specifically for open channel flow, and can be obtained from standard textbooks such as Chow (1959) or Linsley, et. al. (1982). After average velocity is computed, T_t for the channel segment can be estimated using equation (2.2), and T_c can be estimated using equation (2.1).

Use of Manning's equation requires channel section and slope data. At times, this is not available. The hydraulic radius, R, and Manning's n are the most sensitive parameters in Manning's equation. Care must be taken in selection of appropriate values.

Manning's equation provides an estimate of the particle flow time (as defined in Section 2.1). To estimate a wave travel time in an open channel, one might use the kinematic wave celerity as an estimate of wave velocity. Wave celerity is defined and explained by several authors: Chow (1959), Viessman, et. al. (1977), or Linsley, et. al. (1982), for example. The kinematic wave celerity is estimated by

$$C = V + \left[g \frac{A}{B} \right]^{\frac{1}{2}} \tag{2.5a}$$

where

C = wave celerity, or speed, feet per second;

V = flow velocity, perhaps estimated by equation (2.4), in feet per second;

g = acceleration of gravity (32.17 feet second^{-2});

A = channel cross-sectional area, feet^2; and

B = channel surface width, feet.

A wave travel time in the channel is calculated as

$$T_t = \frac{L}{C} \tag{2.5b}$$

where L is the channel reach length, in feet. Calculation of wave velocity requires the same information as the estimation of flow velocity by Manning's equation. Use of wave velocity will result in a shorter travel time.

2.6. Hydrograph Method

Time of concentration estimates can be made through hydrograph analysis, that is, if recorded events are available. Lag time, as used in other SCS methods (see Section 5.2.1), is the time from the center of mass of excess rainfall to the peak rate of runoff. Based on studies of many historical events for a range of watershed conditions, the following empirical relationship between lag (L_g) and time of concentration (T_c) was derived by the SCS:

$$L_g = 0.6T_c \tag{2.6}$$

This relationship is for average natural land conditions and for approximately uniform distribution of excess rainfall over the watershed. Time of concentration can be determined from equation (2.6) for watersheds where rainfall-runoff hydrographs are available. This requires a gaged outlet and precipitation data within the basin of interest.

The 0.6 coefficient is an average of a wide variety of conditions. It should be larger for watersheds having significant depression storage, and smaller for urbanized watersheds. Accordingly, watersheds containing a significant portion of wetlands or ponds or urbanization may not adhere to equation (2.6).

2.7. SCS Modified Curve Number Method

The SCS modified the Curve Number method to estimate the time of concentration for agricultural watersheds with conditions ranging from steep to flat slopes and from heavily forested to smooth land covers (USDA-SCS, 1971). The empirical equation for watershed lag is given as

$$L_g = \frac{L^{0.8}(S+1)^{0.7}}{1900 S_o^{0.5}} \tag{2.7}$$

where

L_g = lag time, in hours;

L = hydraulic length of the watershed, in feet;

S_0 = the average land slope, in feet per foot; and

S = a storage or surface detention factor, in inches;

where

$$S = \frac{1000}{CN'} - 10 \tag{2.8}$$

and

CN' = a retardant factor approximately equal to the runoff Curve Number (See Section 4.1).

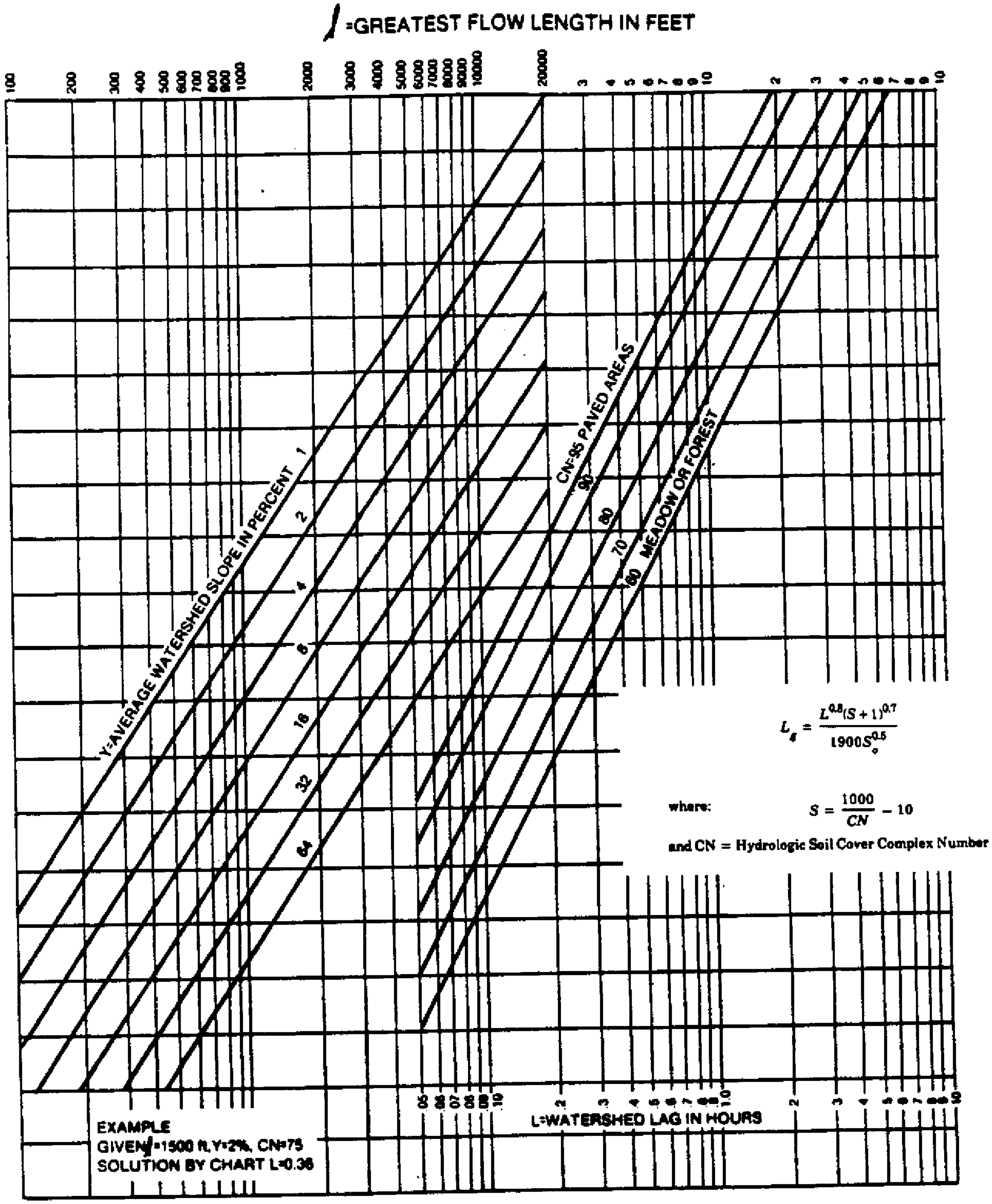


Figure 2.3. Estimating lag time by the Modified Curve Number method. (reproduced from USDA-SCS, 1971)

From the lag time produced in equation (2.7), time of concentration can be calculated using equation (2.6). A monograph solution for equation (2.7) is shown in Figure 2.3.

Equation (2.7) is valid for homogeneous watersheds under natural conditions up to 2000 acres. This approach is not applicable to urbanized watersheds and watersheds containing a significant percentage of wetlands or ponds, because of the constraints on equation (2.6). Use of the modified Curve Number method is illustrated in section 3.1.

2.8. Akan's Method

A mathematical model was developed by Akan (1983) to calculate the time of concentration for overland flow on a rectangular plane with a pervious surface. To simplify the method's use, the governing equations of the rainfall-infiltration-overland flow process are written in terms of various dimensionless parameters[1]. Time of concentration is determined from what is termed "relative" time of concentration, T_c',

$$T_c' = \frac{T_c}{T_e} \tag{2.9}$$

where

T_c = time of concentration for the given pervious surface; and

T_e = the time of concentration for the surface, had it been impervious.

T_e is termed the *equilibrium time*, and is calculated by

[1]In the discussion which follows, equations are presented without reference to units. This is because the equations are dimensionally correct and will apply with any *consistent* unit system.

$$T_e = \left(\frac{L}{i^{\frac{2}{3}} \alpha} \right)^{3/5} \tag{2.10}$$

where

L = flow length;

i = the rainfall intensity;

α = a friction coefficient derived from Manning's Equation;

and

$$\alpha = \frac{k_o S_o^{\frac{1}{2}}}{n} \tag{2.11}$$

where

k_0 = 1.49 $ft^{\frac{1}{3}}$/sec (if English units are used); and

n = Manning's roughness coefficient (Table 2.2 or 2.3).

In equation (2.9), T_c' is determined from dimensionless parameters K' and P'. These parameters are physically based and account for the subsurface soil properties and the antecedent moisture content, respectively. K' is given by

$$K' = \frac{K}{i} \tag{2.12}$$

where

K = the soil's hydraulic conductivity; and

i = the rainfall intensity.

P' is given by

$$P' = \frac{P_f \Phi (1 - S_i)}{i T_e} \tag{2.13}$$

where

P_f = Green-Ampt capillary pressure head for the soil;

Φ = soil porosity; and

S_i = antecedent degree of soil saturation.

Soil characteristics (K, ϕ, P_f) are best determined by measurement. However, some methods exist which determine various soil characteristics from texture data or Soil Survey (SCS) data (Akan recommends methods presented by Rawls and Brakensiek, 1983). The relationship between T_c', K' and P' is given in Figure 2.4.

Akan's method was derived from kinematic flow theory and uses the Green-Ampt infiltration relationships and Manning's equation. The reader should refer to Section 4.5 for a discussion of the Green-Ampt infiltration relationship, and to previous discussion regarding Manning's equation. In addition to the assumptions upon which those are based, Akan's derivation involved some further assumptions:

1. *The watershed is a rectangular plane of uniform slope, surface, and soil characteristics.* Application is limited to rectangular watersheds. While naturally rectangular watersheds are uncommon, many applications in South Florida deal with rectangular basins. The watershed in question should at least have an approximately constant runoff length throughout the basin. Uniform slope, surface conditions and soil characteristics can only be assumed for small basins.

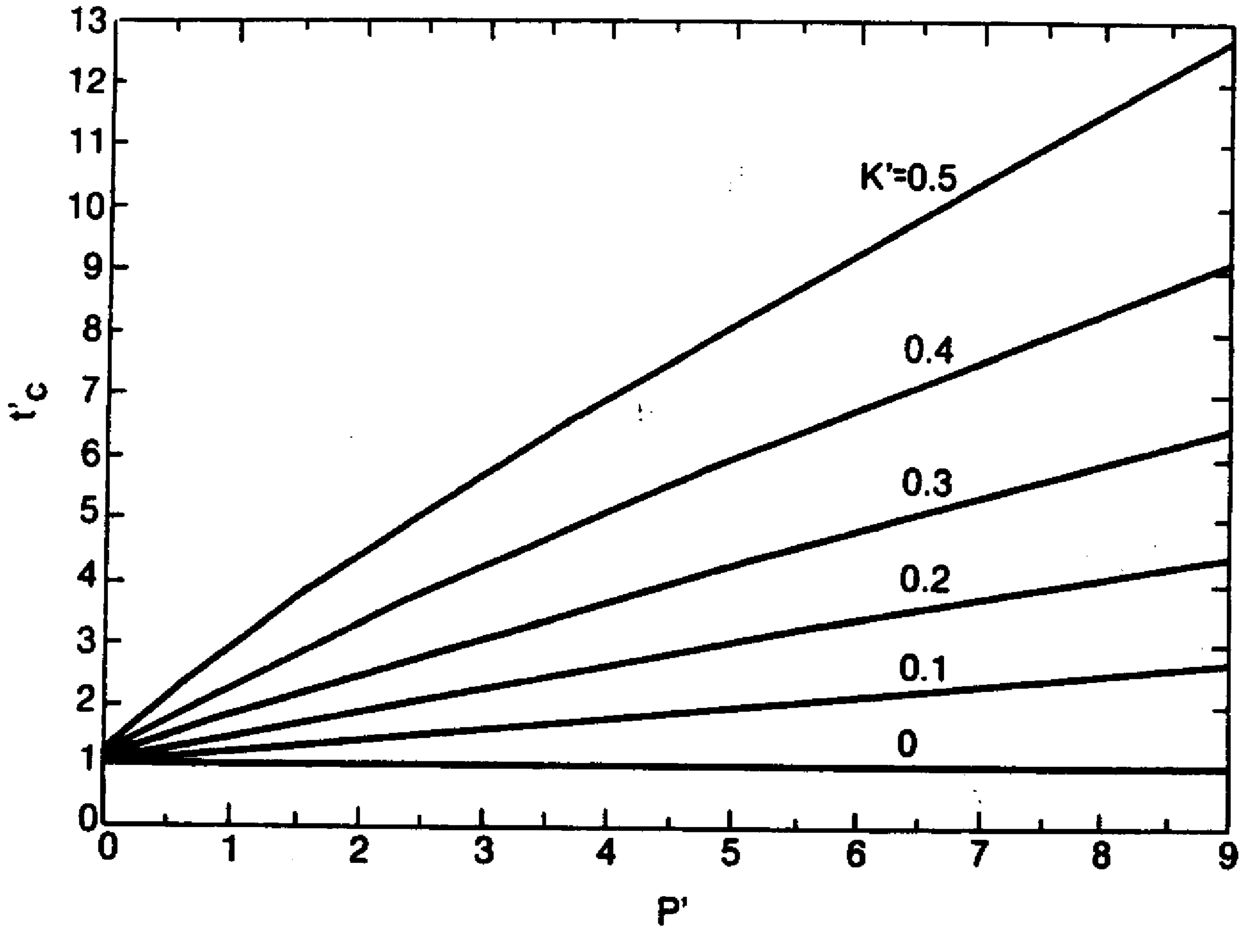


Figure 2.4. Relative time of concentration chart for Akan's Method. (from Akan, 1983)

2. *Rainfall is uniform over the basin and maintains a constant intensity up to the time of concentration.* Uniform depth and intensity of rainfall can be assumed when the basin is small and time of concentration short.

3. *Infiltration is the only mode of runoff loss.* Other runoff losses (i.e. water which does not become runoff) are ignored. This would include interception and surface detention, which, with natural surfaces, may be significant quantities. Consequently, Akan's method would not be readily applicable to areas with thick vegetated cover or where depression storage is high, since the Green-Ampt infiltration relationship does not consider them.

Example 2.1: Time of concentration estimation by Akan's Method. (from Akan, 1983). A rectangular plot has the following dimensions and characteristics

Length, L	5000 feet
Surface slope, S_o, ...	0.005 feet/foot
Manning's n	0.4
Soil conditions	
Porosity, ϕ	0.20
Saturated hydraulic conductivity, K, ...	0.5 inch/hr (1.151×10^{-5} fps)
Green-Ampt capillary pressure head, P_f .	12 inches (1.0 ft)
initial degree of saturation, S_i	0.70

This plot is subject to a storm with a constant rainfall intensity, i, of 2.5 inches per hour (5.787×10^{-5} fps). Determine the time of concentration for the basin.

Compute the friction coefficient, α, using equation (2.11):

$$\alpha = (1.49\ ft^{1/3}/sec)0.005^{1/2}/0.4$$
$$= 0.263\ ft^{1/3}/sec$$

Using equation (2.9), calculate the equilibrium time, T_e:

$$T_e = \{(5000\ ft)/[(0.00005787\ ft/sec)^{2/3}(0.263\ ft^{1/3}\ /sec)]\}^{3/5}$$
$$= 4600\ sec.$$

Dimensionless parameters for entry to Figure 2.4 are needed. Using equations (2.12) and (2.13), K' and P' are given as

$$K' = \frac{0.5\ in/hr}{2.5\ in/hr}$$

$$= 0.20$$

$$P' = \frac{(1.0\ ft)(0.20)(1 - 0.70)}{(0.00005787\ ft/sec)\ (4600\ sec)}$$

$$= 0.225$$

Consulting Figure 2.4, the dimensionless time of concentration, T_c', is determined to be 1.20. Equation (2.9) can be solved for time of concentration to yield

$$T_c = T'_c T_e$$
$$= 1.20\ (4600\ sec)$$
$$= 5520\ seconds$$
$$= \underline{92\ minutes\qquad Answer}$$

3. RUNOFF PEAK ESTIMATION

3.1 Rational Method

The Rational Method is one of the oldest[1], simplest, and most widely used (and often criticized) methods employed in the determination of peak discharges from a given watershed. This approach is frequently used to estimate peak runoff rates from small urban areas of variable size.

The Rational Method uses the equation

$$Q = CIA \tag{3.1}$$

where

Q = peak discharge in cfs;

C = a runoff coefficient;

I = a design rainfall intensity, inches per hour; and

A = contributing watershed area in acres.

Due to the measurement errors associated with A, I and C, the unit conversion factor of 1.008 cfs per acre-inch/hour, is neglected. The runoff coefficient is obtained empirically and represents the ratio of peak runoff rate to average rainfall rate over the watershed for a period equal to the time of concentration.

The Rational method very much oversimplifies a complex process. Several assumptions are made in order to make those simplifications. A list of the assumptions used in the application of this method follow:

---

1 Linsley and Franzini, 1979, credit the development of the Rational method to T. J. Mulvaney, 1851.

1. *The return frequency of the calculated peak discharge is the same as that of the chosen rainfall intensity.* This is a major assumption in the Rational method. It means, for example, if a rainfall intensity has a chance of recurring only once in ten years, the calculated peak discharge will have the same chance of recurrence. This relationship has been verified to some extent (see Schwab et al. 1981, pg. 72; and Viessman, et al. 1977, pg. 511) and seems quite logical, but may not always be the case. A watershed reacts to many different influences (antecedent moisture, for example), not just rainfall intensity.

2. *The rainfall is uniformly distributed over the drainage basin, and maintains a constant intensity during the storm.* The Rational Method is best suited to small well-defined drainage areas. In general, the Rational Method is recommended for application to drainage basins less than 200 acres in area, and is best suited for well-defined drainage basins (Burke, 1981). Many engineers have recommended that the application of this method be limited to watersheds less than 100 acres (E. F. Schulz, 1973).

3. *The storm duration associated with the peak discharge is equal to the time of concentration for the drainage basin.* (Time of concentration is defined and discussed in Section 2.) This assumption implies that after the time of concentration has elapsed, continued rainfall no longer has effect on the peak discharge. This ignores interflow, base flow, and groundwater recharge components, which take longer to appear in basin outflow. When small basins are considered this usually holds true.

4. *The runoff coefficient, C, is independent of the storm duration for a given watershed and is a constant value depending on soil cover type and quality.* Difficulty in the accurate selection of the runoff coefficient is the major limitation of the Rational method. Peak discharge estimates are no better than the estimate for C. For small urban areas, the runoff coefficient can be reasonably estimated from field investigations. For larger areas, the determination of the runoff coefficient is subject to a much greater error due to the variability of the drainage area characteristics.

   Estimation of rainfall losses for large watersheds due to evaporation, transpiration, infiltration, depression and channel storage are not included in C and will appreciably affect the estimation of the runoff peak rate. Hence, size of the drainage area is critical in application of this method. The size of the drainage area should be small enough to maintain a similar soil type, land use, land cover, and so on. For larger basins, a weighted coefficient or subbasin approach should be applied, or another method used.

   Assumption 4 is not always adhered to in practical use of the Rational method. The runoff coefficient is sometimes made a function of basin antecedent conditions, ground cover, soil type, and rainfall intensity (see Schwab et al. 1977, and Viessman et al. 1977).

Application. Choice of parameters in the Rational Method is sometimes arbitrary and leaves much to interpretation and preference. Consequently, variations in solutions from one designer to another may occur. To illustrate the use of the Rational method, a procedure, which is within the limitations already discussed, is presented:

1. Determine the contributing basin area, A (acres) by using USGS topographical maps, SFWMD or county drainage maps, maps developed from a survey of the area, or plans made specifically for the basin.

2. Determine the appropriate runoff coefficient value, C, from runoff coefficient tables. Tables 3.1, 3.2, and 3.3 contain some examples. Text books and other publications are available which present more detailed collections of coefficients.

   If the land is under a variety of uses, a composite C value may be determined by:

$$C = \frac{C_1A_1 + C_2A_2 + \dots + C_nA_n}{A_{total}} \tag{3.2}$$

   Where C_1, C_2, ..., C_n, are the runoff coefficients associated with component areas A_1, A_2, ..., A_n, and A_{total} is total area.

   The runoff coefficient represents integrated effects of infiltration, evaporation, retention, flow routing, and interception, all of which affect the time distribution and peak rate of runoff.

3. Determine the time of concentration for the watershed by using an appropriate method (Several methods are presented in section 2). Time of concentration is by far the most sensitive aspect of the Rational Method, and care must be taken in its estimation. Generally, calculating the time of concentration using a

TABLE 3.1. RURAL RUNOFF COEFFICIENTS FOR USE WITH THE RATIONAL METHOD. (from Schwab, et. al., 1971)

Vegetation and Topography	Open Sandy Loam	--------Soil Texture------- Clay Silt Loam	Tight Clay
Woodland			
Flat (0-5% slope)	0.10	0.30	0.40
Rolling (5-10% slope)	0.25	0.35	0.50
Hilly (10-30% slope)	0.30	0.50	0.60
Pasture			
Flat	0.10	0.30	0.40
Rolling	0.16	0.36	0.55
Hilly	0.22	0.42	0.60
Cultivated			
Flat	0.30	0.50	0.60
Rolling	0.40	0.60	0.70
Hilly	0.50	0.72	0.82

wave travel time (see Section 2.1) will provide more conservative results with the Rational Method.

4. Select a design frequency for the peak discharge. After the frequency has been selected, and the time of concentration has been determined, the rainfall intensity can be determined. The value of rainfall intensity, I, can be obtained from an intensity-duration-frequency diagram, obtainable from such sources as Technical Paper No. 40, U.S. Weather Bureau, or Technical Publication 81-3, SFWMD (MacVicar, 1981), with the storm duration equal to the time of concentration.

5. Use equation (3.1) to compute the peak runoff rate.

TABLE 3.2. RUNOFF COEFFICIENTS FOR URBAN AREAS FOR USE WITH THE RATIONAL METHOD Values are applicable for storms of 5-10 year frequencies. (ASCE,1976)

DESCRIPTION OF AREA	Runoff Coefficients
Business	
Downtown	0.70 -- 0.95
Neighborhood	0.50 --0.70
Residential	
Single-family	0.30 -- 0.50
Multi-units,detached	0.40 -- 0.60
Multi-units,attached	0.60 -- 0.75
Residential (suburban)	0.25 -- 0.40
Apartment	0.50 -- 0.70
Industrial	
Light	0.50 -- 0.80
Heavy	0.60 -- 0.90
Parks,cemeteries	0.10 -- 0.25
Playgrounds	0.20 -- 0.35
Railroad yard	0.20 -- 0.35
Unimproved	0.10 -- 0.30

6. If there is another basin downstream, the first time of concentration is added to the travel time in the channel found by Manning's equation or any of the methods presented in section 2. This is then compared to the inlet time of the second basin and the larger of two is used as the new time of concentration.

Example 3.1: Peak discharge by the Rational Method. (from Burke, 1981) Find the peak runoff rate from the following watershed, which is shown in Figure 3.1, during a 10-year storm event using the Rational Method. Subbasin #1 lies upstream of Subbasin #2. Runoff from subbasin #1 is collected at point "x" and conveyed,

TABLE 3.3. VALUES USED TO DETERMINE A COMPOSITE RUNOFF COEFFICIENT FOR AN URBAN AREA. Values are applicable for storms of 5-10 year frequencies. (ASCE, 1976)

CHARACTER OF SURFACE	Runoff Coefficients
Pavement	
Asphaltic and concrete	0.70 -- 0.95
Brick	0.70 - 0.85
Roofs	0.70 - 0.95
Lawns,sandy soil	
Flat,0-2 % slope	0.05 - 0.10
Average, 2-7 %	0.10 - 0.15
Steep,7 %	0.15 - 0.20
Lawns,Heavy soil	
Flat, 0-2 % slope	0.13 - 0.17
Average, 2-7 %	0.18 - 0.22
Steep , 7%	0.25 - 0.35
Water impoundment	1.00

through Subbasin #2, to point "y" in a canal, with a travel time of 15 minutes. Runoff from Subbasin #2 is collected at point "y". The information for each subbasin is given below:

Subbasin #1

Total area	120 acres
flat woodland	40 acres
flat pasture	80 acres
Travel Path length	3500 feet
Slope	0.01 ft/ft
Soil	group B

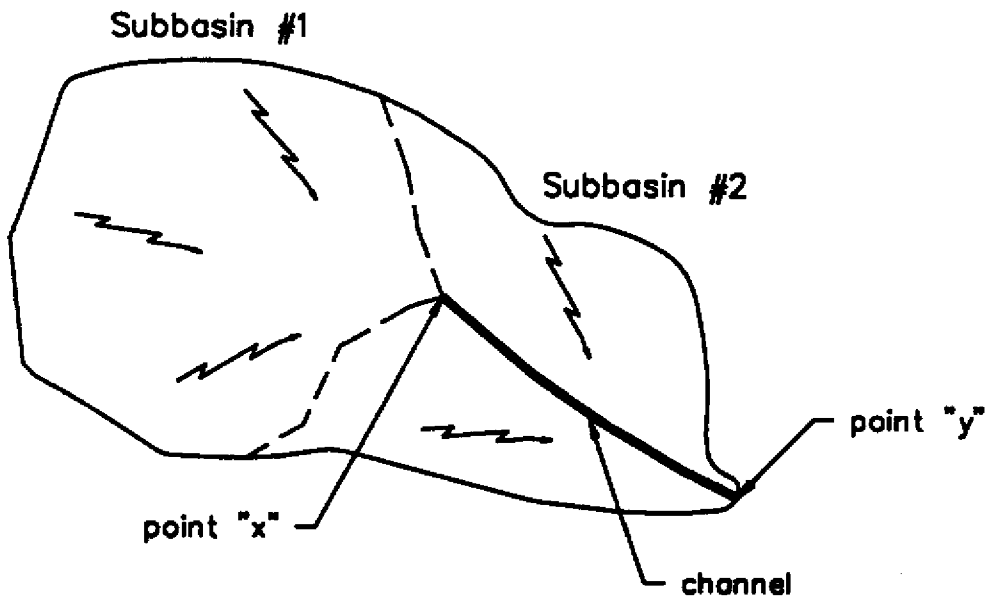


Figure 3.1. Layout of subbasins #1 and #2 considered in Example 3.1.

Subbasin #2

Total area	130 acres
roof area	20 acres
park	40 acres
pavement	20 acres
flat lawn	50 acres
Travel Path length	3000 feet
Slope	0.015 ft/ft
Soil Type	group B

From Tables 3.1, 3.2, and 3.3 some average C values are chosen:

flat woodland	0.10
flat pasture	0.10
roofs	0.85
park	0.17
pavement	0.82
flat lawn	0.08

Using equation (3.2), compute a composite value of the runoff coefficient, C, for each basin:

$$C_1 = [40(0.10) + 80(0.10)]/120$$
$$= 0.10$$
$$C_2 = [20(0.85) + 40(0.17) + 20(0.82) + 50(0.08)]/130$$
$$= 0.34$$

Time of concentration by the Upland Method. (section 2.2). From Figure 2.1, extract some average flow velocities:

flat woodland	0.5 fps
flat pasture	0.70
roofs, pavement	2.40
park, flat lawn	0.80

The average velocities can be combined, as with the runoff coefficients, to form an area-weighted average:

$$V_1 = [0.5(40) + 0.70(80)]/120$$

$$= 0.63 \text{ ft/sec}$$

$$V_2 = [2.4(40) + 0.80(90)]/130$$

$$= 1.29 \text{ ft/sec}$$

The time of concentration at point "x" need only consider the travel time in subbasin #1. Using equation (2.2),

$$T_{cx} = 3500/[3600(0.63)]$$

$$= 1.54 \text{ hours}$$

At point "y", however, time of concentration is either (1) the sum of subbasin #1 time of concentration and the canal travel time, or, (2) the time of concentration for subbasin #2, whichever is larger:

$$T_{cy} = T_{cx} + 15 \text{ min}$$

$= 1.54 + 0.25$ hrs

$=$ <u>1.79 hours   Answer</u>

or (using equation (2.2) again, for subbasin #2),

$$T_{cy} = 3000/[3600(1.29)]$$
$$= 0.65 \text{ hours}$$

The value of 1.79 hours is chosen for time of concentration, since it is larger.

<u>Time of concentration by the modified CN method</u> (section 2.7). From Tables 4.1, 4.2 and 4.3, choose proper CN's for the given soil group and land uses:

flat woodland	55
flat pasture	61
roofs	98
park	69
pavement	98
flat lawn	69

Compute weighted CN values, using equation (3.2), and S factors, with equation (2.7), for each basin:

$$CN_1 = [(40)55 + (80)61]/120$$
$$= 59$$

$$S_1 = (1000/59) - 10$$
$$= 6.95 \text{ inches}$$

$$CN_2 = [(20)98 + (40)69 + (20)98 + (50)69]/130$$
$$= 77.9$$

$$S_2 = (1000/77.9) - 10$$
$$= 2.84 \text{ inches}$$

From equation (2.6) or Figure 2.3, the lag time can be obtained, then the time of concentration is calculated, via equation (2.5):

$$L_{g_1} = 1.54 \text{ hours}$$
$$T_{cx} = 1.54/0.6$$
$$= 2.57 \text{ hours}$$

$$L_{g_2} = 0.67 \text{ hours}$$
$$T_{cy} = 0.67/0.6$$
$$= 1.12 \text{ hours}$$

As before, with the Upland method, the time of concentration at point "y" is a choice between that just calculated and

$$T_{cy} = T_{cx} + 15 \text{ min}$$
$$= 2.57 + 0.25 \text{ hrs}$$
$$= \underline{2.82 \text{ hours} \qquad \text{Answer}}$$

Choose the latter, since it is larger. Note the large differences in calculated T_c between the two methods.

The next step is to obtain the design rainfall intensity based on calculated times of concentration. From Technical Paper No.40 , U. S. Weather Bureau:

10-year, 1-hour rainfall at W. Palm Beach .... 3.6 inches

10-year, 2-hour rainfall at W. Palm Beach .... 4.5 inches

10-year, 3-hour rainfall at W. Palm Beach .... 5.1 inches

For the Upland method times of concentration, intensities are interpolated to be:

$$T_{cx} = 1.54 \text{ hours}, I_x = (4.09 \text{ inches})/(1.54 \text{ hours}) = 2.66 \text{ inches/hour},$$

$$T_{cx} = 1.79 \text{ hours}, I_y = (4.33 \text{ inches})/(1.79 \text{ hours}) = 2.42 \text{ inches/hour};$$

where: I_x = rainfall intensity, inches/hour, for Basin #1 and I_y = rainfall intensity, inches/hour, for Basin #2. Then, using equation (3.1) calculate peak discharge:

$$\begin{aligned} Q_x &= C_1 I_x A_1 \\ &= 0.1(2.66)(120) \\ &= \underline{31.92 \text{ cfs} \quad \text{Answer}} \end{aligned}$$

$$\begin{aligned} Q_y &= (C_1 A_1 + C_2 A_2) I_y \\ &= [0.10(120) + 0.34(130)]2.42 \\ &= \underline{136.0 \text{ cfs} \quad \text{Answer}} \end{aligned}$$

Then for the modified CN method times of concentration:

$$T_{cx} = 2.57 \text{ hours}, I_x = (4.84 \text{ inches})/(2.57 \text{ hours}) = 1.88 \text{ inches/hour},$$
$$T_{cy} = 2.82 \text{ hours}, I_y = (4.99 \text{ inches})/(2.82 \text{ hours}) = 1.77 \text{ inches/hour},$$

so that

$$\begin{aligned} Q_x &= C_1 I_x A_1 \\ &= 0.1(1.88)(120) \\ &= \underline{22.56 \text{ cfs} \quad \text{Answer}} \end{aligned}$$

$$\begin{aligned} Q_y &= (C_1 A_1 + C_2 A_2) I_y \\ &= [0.10(120) + 0.34(130)]1.77 \\ &= \underline{99.47 \text{ cfs} \quad \text{Answer}} \end{aligned}$$

The peak rates calculated in this example show the importance of choosing the proper time of concentration. The Rational method is very sensitive to the time of concentration value. In order to obtain justifiable results with the Rational method, the chosen value for time of concentration must be accurate.

3.2. SCS Graphical Method

The SCS Graphical Method is presented in TR-55 (USDA-SCS, 1986a), and is based on several generalized runs of the TR-20 project formulation program (USDA-SCS, 1983, see Section 5.8). The TR-20 program uses the Dimensionless Curvilinear unit hydrograph (see Section 5.2.1), and the Curve Number method (see Section 4.1).

Peak discharge is calculated by

$$Q = q_u A R_o F_p \tag{3.3}$$

where

Q = peak discharge, in cfs;

q_u = unit peak discharge in cfs per square mile per inch runoff (csm/in);

A = drainage area, in square miles;

R_0 = total runoff depth, inches;

F_p = a pond and swamp factor.

A design rainfall depth, P, is chosen, and used to calculate total runoff depth by the Curve Number (CN) method (Section 4.1). Initial abstraction, I_a is needed and also available from the CN method. Time of concentration is needed and can be calculated by any of the methods given in Section 2. With the values for I_a and T_c, peak unit discharge can be obtained from Figure 3.2. Figure 3.2 was produced using the Type III rainfall distribution, which is recommended for South Florida.

The pond and swamp factor, F_p, is used to account for effects of surface detention/retention. When a pond and swamp factor is selected, those ponds and swamps along the flow path used to calculate time of concentration should be ignored. Their effects on the peak flow should already be included in the time of concentration value. F_p is obtained from Table 3.4.

In this method, it is assumed that the watershed has uniform soil and cover characteristics, and channel storage and routing are not important. This would typically mean small basins. The design rainfall depth is limited to 24-hour duration. The method is limited to the information ranges in Figure 3.2 and Table 3.4, and for CN greater than 40. The constraints on the Dimensionless Curvilinear

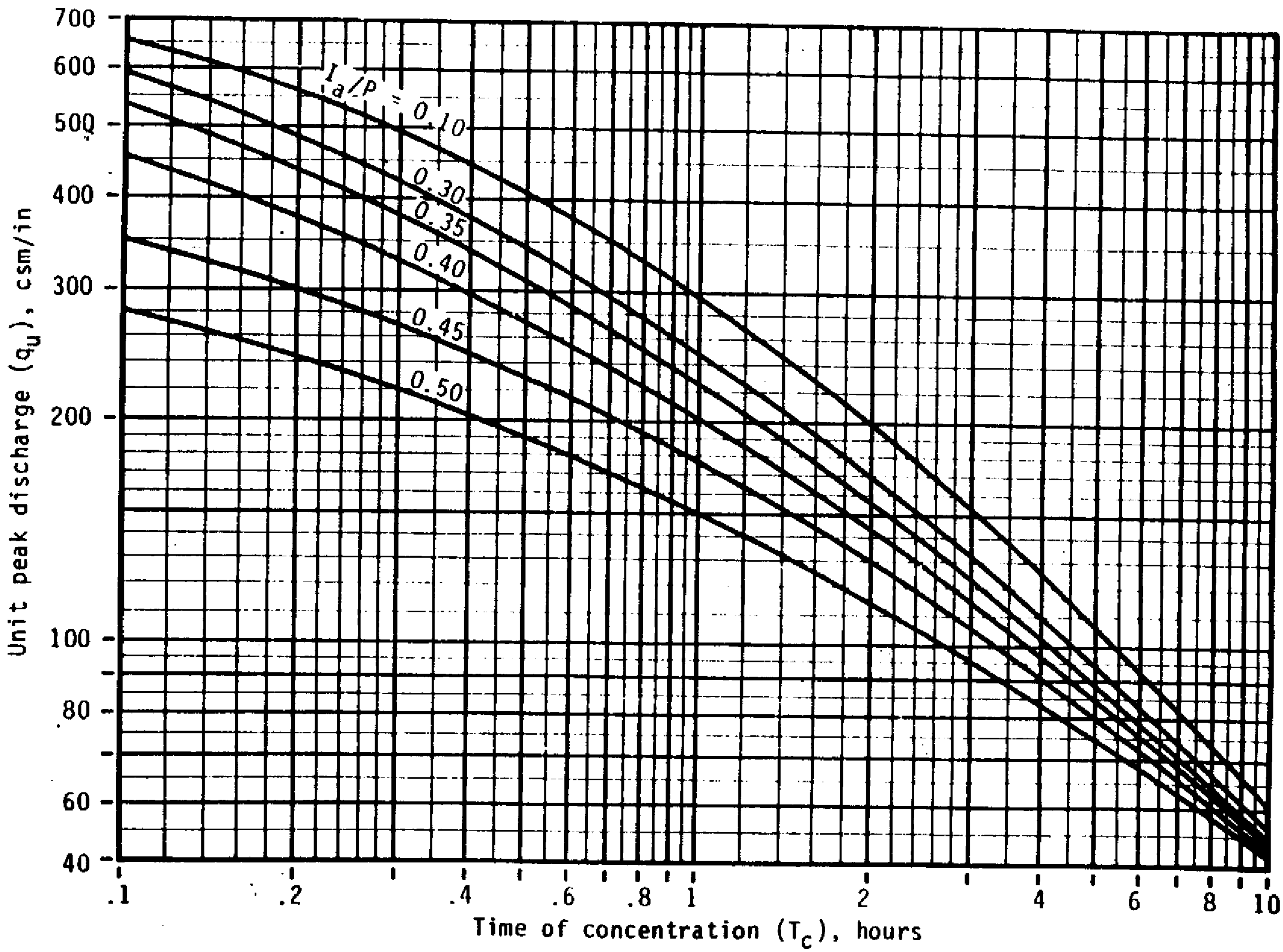


Figure 3.2. Unit peak discharge for the Updated SCS Graphical method. (reproduced from USDA-SCS, 1986a)

TABLE 3.4. POND AND SWAMP ADJUSTMENT FACTORS FOR USE WITH THE SCS GRAPHICAL METHOD. (USDA-SCS, 1986a)

Percentage pond/swamp area	F_p
0.0	1.00
0.2	0.97
1.0	0.87
3.0	0.75
5.0	0.72

unit hydrograph (Section 5.2.1) and the Curve Number method (Section 4.1) would apply here as well.

Figure 3.2 was produced using a Dimensionless unit hydrograph with a peak rate factor of 484 (see Section 5.2.1 for further discussion). The SCS (USDA-SCS, 1986b) recommends a peak rate factor of 284 for any basin with a slope of 0.5 percent or less, which would mean most South Florida applications. To compensate for this, the SCS (USDA-SCS, 1986b) recommends adjusting the values read from Figure 3.2 by multiplying by 0.59.

Example 3.2: Peak discharge estimation by the SCS Graphical method. Example 4.1 (Curve Number) presents a basin and calculates a runoff volume for a particular storm. Here, peak discharge will be estimated for those conditions. In Example 4.1, a design rainfall depth was chosen as

$$P = 10 \text{ inches}$$

Total runoff depth, R_0, and potential abstraction, S, are calculated as

$$R_0 = 7.25 \text{ inches}$$

$$S = 2.84 \text{ inches}$$

Time of concentration, T_c, was calculated, by the modified Curve Number method, for this basin in Example 3.1 (subbasin #2).

$$T_c = 0.65 \text{ hours}$$

Let us further assume that approximately 2% of the basin area in question is ponds or swamps. Using equation (4.2), the initial abstraction is calculated as

$$\begin{aligned} I_a &= 0.2S \\ &= 0.2(2.84 \text{ inches}) \\ &= 0.568 \text{ inches} \end{aligned}$$

which means

$$\begin{aligned} I_a/P &= (0.568 \text{ inches}) / (10 \text{ inches}) \\ &= 0.0568 \end{aligned}$$

Consulting Figure 3.2, with $T_c = 0.65$ hours and $I_a/P = 0.0568$ (rounded up to 0.10), a unit peak discharge is read:

$$q_u = 370 \text{ csm/in}$$

Since the basin slope is 1.5%, this should be corrected for a peak rate factor of 284, so that

$$q_u = (0.59)370 \text{ csm/in}$$
$$= 218.3 \text{ csm/in}$$

Interpolating from Table 3.4, $F_p = 0.81$. Substituting this into equation (3.3)

$$Q = (218{,}3 \text{ csm/in})(130 \text{ acres})(7.25 \text{ in})(0.81)(1 \text{ mile}^2)/(640 \text{ acres})$$
$$= \underline{260{,}4 \text{ cfs} \qquad\qquad \text{Answer}}$$

3.3. SCS Tabular Method

The SCS graphical method, presented in the previous section, is recommended for use only in relatively homogeneous basins. For a large, less uniform basin, the SCS recommends the use of the Tabular Hydrograph method. This method is presented in SCS Technical Release 55 (TR-55, USDA-SCS, 1986a), and is based on generalized runs of the TR-20 program (USDA-SCS, 1983, see Section 5.8).

The basin is delineated into relatively homogenous subareas, and a time of concentration and travel time is estimated for each one. Travel time is the time that the flow from the outlet of a subarea takes to reach the basin outlet. Based on the subarea time of concentration, travel time, and the ratio of initial abstraction, I_a, (see Section 4.1) and precipitation, a tabulated hydrograph is chosen for each subarea. The subarea's contribution to the basin outflow hydrograph is calculated by the equation

$$q = q_t A R_o \tag{3.4}$$

where

q = the subarea flow contributing to basin outflow, cfs;

q_t = unit subarea discharge from tabulated hydrographs, cfs per square mile per inch runoff (csm/in);

A = subarea drainage area, square miles; and

R_o = subarea runoff, inches.

A hydrograph for each subarea is computed in a like manner. These hydrographs are added together to form a composite hydrograph for the entire basin. The peak discharge is the maximum flow noted in the composite hydrograph. This procedure is briefly outlined in Example 3.3. The reader should refer to TR-55 for complete information on the method's use.

TR-55 presents four sets of tabulated hydrographs (Exhibit 5, USDA-SCS, 1986a), each for a different rainfall distribution. The SCS recommends that the type III rainfall distribution be used throughout Florida (USDA-SCS, 1986b). The SCS (USDA-SCS, 1986b) also recommends that a peak rate factor of 284 (see Section 5.2.1 for a definition) be used for all basins with an average slope of 0.5 percent. The tabulated hydrographs presented in TR-55 are for a peak rate factor of 484, and hence, not applicable for most South Florida conditions. Tables for a 284 peak rate factor are not available at present. Until such tables are published the SCS, the Tabular Method should not be used for most South Florida applications.

The tabulated hydrographs used in the Tabular Method were produced by several runs of the TR-20 program. A curve number of 75 was used for all of these runs, and rainfall was set such that approximately 3 inches of runoff occurred. If conditions differ significantly from these general parameters, the magnitude of estimated peak discharges is questionable (see McCuen, 1982, for more information). The SCS recommends that other methods be used under any of the following conditions:

- The time of concentration,T_c, is greater than 3 hours or the travel time, T_t, is greater than 2 hours.

- Subarea drainage areas differ by a factor of 5 or more.

- The entire outflow hydrograph, or an accurate time to peak is needed.

McCuen (1982) points out some further constraints on the application of the SCS Tabular Method:

- There should be very little variation of CN within subareas. Variation of CN between subareas is acceptable.

- Subareas should be less than 20 square miles.

- Runoff volumes should be greater than 1.5 inches.

The Tabular method is typically used for testing the effects of structural or other improvements planned for a basin; comparing the difference between before and after peak discharges. The method is reliable in these circumstances, but using the method for estimate the magnitude of peak discharges is questionable.

Example 3.3: Application of the SCS Tabular Hydrograph method to estimate peak discharge. (Adapted from McCuen, 1982) A watershed shown in Figure 3.3 is subject to a 24-hour rainfall of 7 inches. The watershed can be divided into 4 subareas as shown in Figure 3.3. A curve number, time of concentration, and travel time are estimated for each subarea as shown in Table 3.5a.

The excess rainfall is calculated according to the SCS curve number method (see Section 4.1). The results are shown in Table 3.5b. Table 3.6 shows the calculations to determine peak discharge. For each subarea, the time of concentration, time of travel, and I_a/P are used to select a tabulated hydrograph (q_t) values from Exhibit 5 of TR-55 (USDA-SCS, 1986). All of the I_a/P values were rounded to 0.10 for hydrograph selection.

The contribution of each subarea to basin outflow (q) is calculated by equation (3.4). For example, at 13.2 hours, the tabular hydrograph value for subarea 1 is

$$q_t = 164 \text{ cfs per square mile per inch of runoff (csm/in)}$$

From Table 3.5a, the area, A of subarea 1 is 0.40 square miles; and from Table 3.5b the runoff, R_o, from subarea 1 is 3.30 inches. Substituting these values into equation (3.4), the contribution to basin outflow from subarea 1 is found to be

$$\begin{aligned} q &= q_t A R_o \\ &= (164 \text{ csm/in})(0.40 \text{ miles}^2)(3.30 \text{ inches}) \\ &= 216 \text{ cfs} \end{aligned}$$

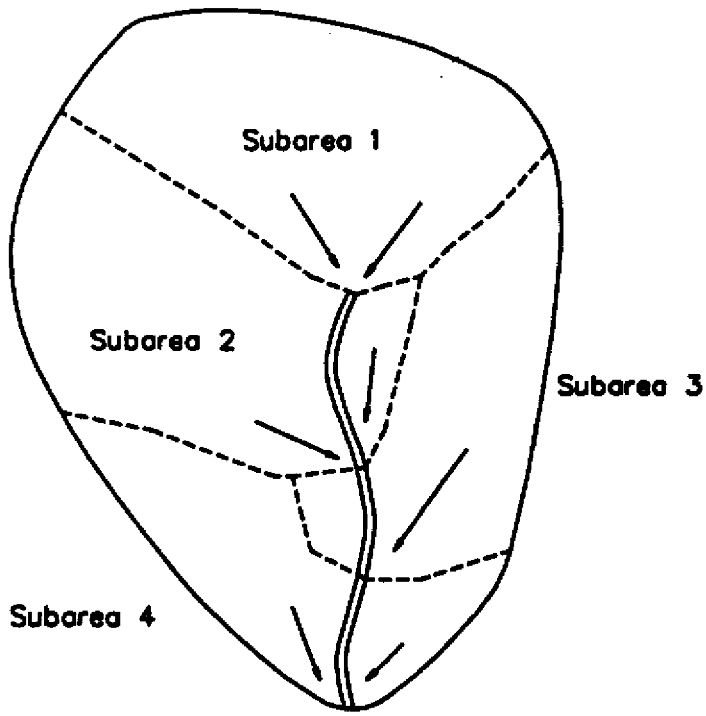


Figure 3.3. Configuration of basin considered in Example 3.3. (after McCuen, 1982)

TABLE 3.5a. CONFIGURATION OF BASIN CONSIDERED IN EXAMPLE 3.3 (data from McCuen, 1982)

Subarea	Drainage Area (miles^2)	Curve Number	Time of Concentration T_c (hours)	Travel Time T_t (hours)
1	0.40	67	2.00	0.00
2	0.25	71	1.50	0.75
3	0.20	75	1.00	0.25
4	0.30	81	1.25	1.00

TABLE 3.5b. EXCESS RAINFALL CALCULATIONS FOR SUBAREAS IN EXAMPLE 3.3.

Subarea	Curve Number	S = 1000/CN - 10 (inches)	Runoff R_o (inches)	I_a/P = 0.2S/P
1	67	4.93	3.30	0.09
2	71	4.08	3.73	0.12
3	75	3.33	4.15	0.10
4	81	2.35	4.80	0.07

TABLE 3.6. DETERMINATION OF PEAK FLOW BY THE SCS TABULAR HYDROGRAPH METHOD FOR EXAMPLE 3.3

Hydro-graph Time (hours)	Subarea 1 Contribution		Subarea 2 Contribution		Subarea 3 Contribution		Subarea 3 Contribution		Total Outflow (cfs)
	q_t (csm/inch)[1]	q (cfs)	q_t (csm/inch)	q (cfs)	q_t (csm/inch)	q (cfs)	q_t (csm/inch)	q (cfs)	
13.2	164	216	87	81	279	232	74	107	636
13.4	187	247	130	121	268	222	119	171	761
13.6	200	264	173	161	225	187	170	245	857
13.8	191	252	205	191	178	148	213	307	898
14.0	178	235	217	202	139	115	234	337	889
14.3	147	194	202	188	100	83	218	314	779
14.6	119	157	165	154	77	64	174	251	626

[1] csm/in = cfs per sqare mile per inch of runoff

Contributions from other subareas are calculated similarly, and the results are summed for each hydrograph time to form a composite hydrograph. The peak flow is the maximum value noted in the hydrograph. In this case, the peak discharge is 898 cfs.

3.4 SFWMD Sheetflow Procedure

The SFWMD Permit Information Manual, Volume IV (SFWMD, 1984a) presents a procedure for estimation of peak discharge rates. With this procedure, pre-development peak discharges can be estimated with the following information:

- an appropriate 24-hour rainfall amount (based on locality);
- average wet season water table depth prior to the design event;
- sheetflow flow length; and
- land slope.

A set of several curves are presented in the Manual (Figures C-8 through C-22), which are intended to include most possible situations. An example is shown in Figure 3.4.

To obtain a peak discharge using this procedure, one enters the appropriate figure, with the necessary information, and reads an areal peak discharge, i.e. peak discharge per unit area. This value is then multiplied by the area of the watershed under consideration. This value is then multiplied by a surface ponding adjustment factor which is obtained from Figure 3.5 (Figure C-23 in Volume IV, SFWMD, 1987).

The curves used in the Sheetflow procedure were produced using a computer model developed within the SFWMD. A detailed description of the program, called

"WSHS1", was originally presented in a memorandum report entitled, "A Procedure for The Estimation of Sheetflow Runoff in the South Florida Water Management District". Initial development of the model was presented in a SFWMD internal memorandum by R.W. Higgins (1979). Further documentation and some enhancements to the computer model (now called "PEAKQ") was presented by Cooper and Neidrauer (1989). The reader should refer to Cooper and Neidrauer for a detailed description of the computer model. The model solves Manning's overland flow equation and the continuity equation simultaneously. Infiltration losses are computed by Horton's equation. The figures are based on the following assumptions:

1. *Sheetflow Model Assumptions.* The simultaneous solution of Manning's and the continuity equation in the sheetflow computer model requires the assumption of a uniform rectangular plane. A uniform water depth is assumed over the plane. Rainfall will increase the water depth; infiltration will decrease it. During a time step, an exiting flow is determined by Manning's equation with the hydraulic radius equal to the drainable depth (see *4.*), and the slope equal to the ground surface slope. This solution will not account for inundated outfalls, which can be a problem with many South Florida applications.

2. *Manning's roughness coefficient is assumed to be 0.25 in all cases.* Manning's coefficients vary considerably, as can be seen in Table 2.1, depending on the surface cover and other conditions. The sheetflow procedure may overestimate peak flow in watersheds with heavy vegetative growth.

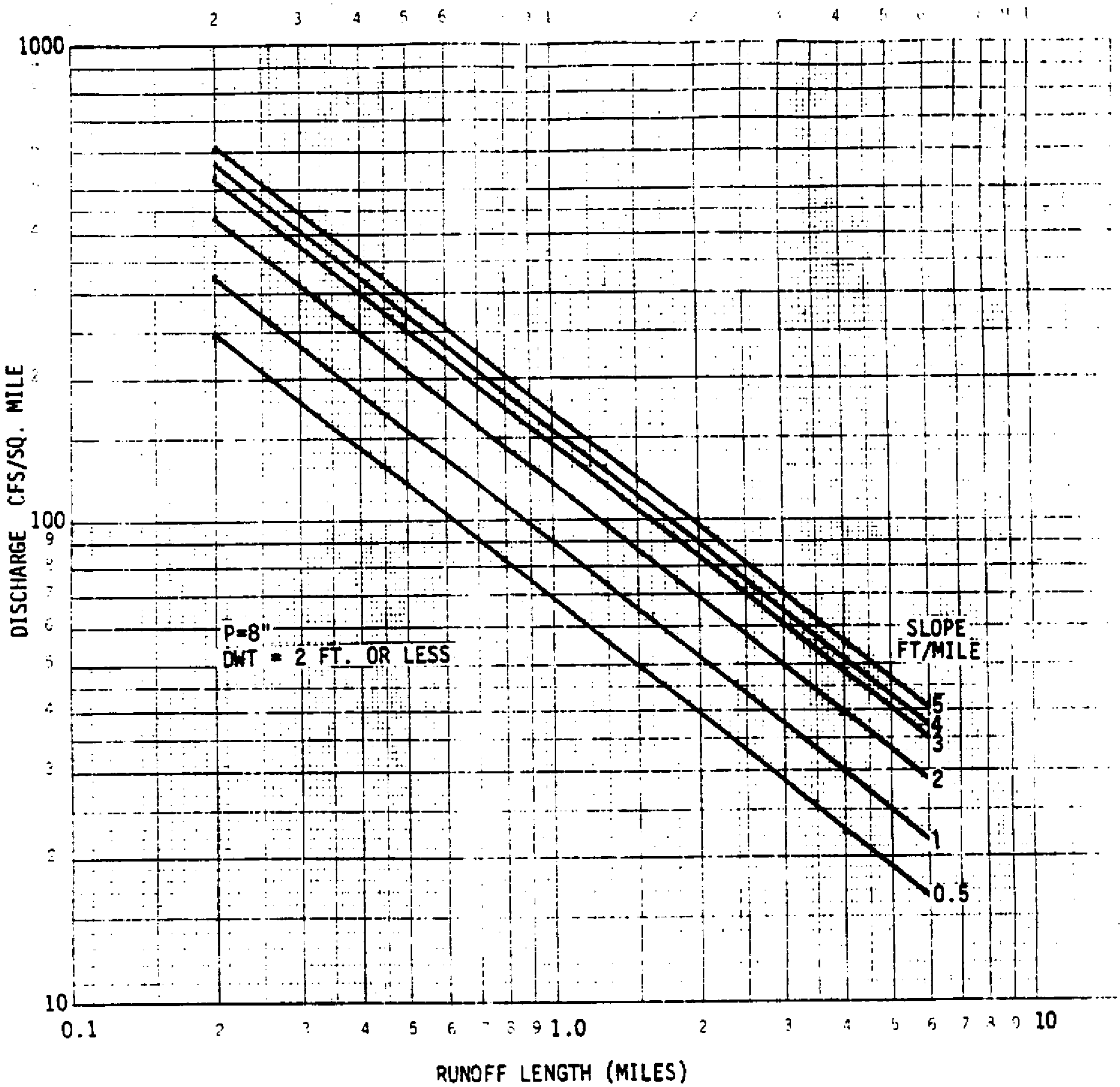


Figure 3.4. An example chart for use with the SFWMD Sheetflow procedure. (reproduced from SFWMD, 1984)

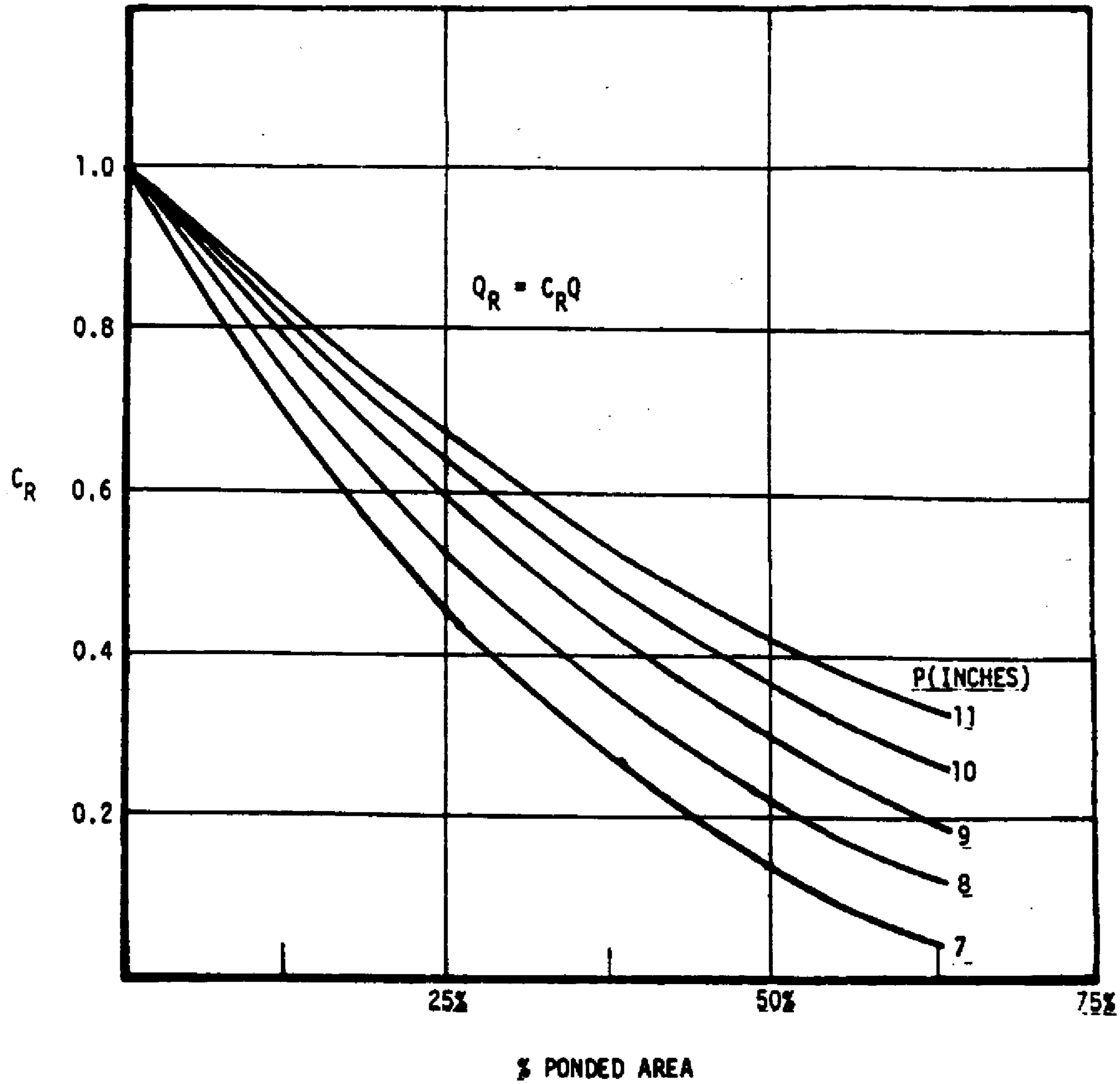


Figure 3.5. Curve for obtaining a surface ponding adjustment factor for the SFWMD Sheetflow procedure. (reproduced from SFWMD, 1984)

3. *Only sandy soils are included.* Furthermore, a final infiltration rate of 0.01 in/hr is assumed. Soils, and their infiltration rates, can vary considerably from case to case. Normally, final inflitration rates for sandy soils should be higher (see ASCE, 1960). The value of 0.01 inches per hour is used to reflect the effects of a high water table. Care must be taken to insure that the soils in the watershed of interest are comparable to these conditions.

4. *A constant detention depth of 2 inches is assumed.* In the SFWMD sheetflow program, runoff does not begin until the depth of water is greater than the detention depth. Depth of detention will vary from case to case. The curve shown in Figure 3.5 is meant to correct for detention depths higher than 2 inches.

In Section 2.3, the SCS sheet flow equation for estimation of travel time is discussed. The SCS states that sheetflow will exists for a flow length of 300 feet before becoming shallow concentrated flow (USDA-SCS, 1986). That limitation applies to the SFWMD Sheetflow procedure as well.

3.5 Cypress Creek Formula

There are a class of peak discharge estimation formulas of the form

$$Q = CA^x \tag{3.5}$$

where

Q = design peak discharge;

A = drainage area; and

$C, x =$ regression coefficients.

The coefficients C and x can depend not only on the locality but upon the design storm as well. Viessman et al. (1977, pg. 524) discuss a number of particular cases involving equation (3.6). Also, several C and x coefficients have been developed by the SCS for design purposes (USDA-SCS, 1973).

Stephens and Mills (1965) investigated the use of equation (3.5) to estimate maximum daily discharges from South Florida watersheds. Investigations were conducted in some typical flatwoods watersheds (near Taylor Creek and Vero Beach). Their results showed that reasonable estimates of maximum 24-hour discharges could be made with the "Cypress Creek Formula":

$$Q_{24} = CA^{5/6} \tag{3.6}$$

where

$Q_{24} =$ maximum 24-hour discharge, in cfs;

$A =$ drainage area, in square miles;

$C = 16.39 + 14.75R_o$;

$R_o =$ rainfall excess, in inches.

To estimate excess rainfall, Stephens and Mills estimated basin storage (e.g., soil moisture and depression storage) prior to a storm using an equation of the form

$$S_t = S_o K^t \tag{3.7}$$

where

$S_o =$ the initial basin storage, in inches;

$S_t =$ basin storage, in inches, t days after S_0 occurred; and

$K =$ a regression factor

$= 0.96$ for winter months, and

$= 0.94$ for the remainder of the year.

Rainfall excess was calculated by subtracting S_t, calculated with equation (3.7), from the storm rainfall depth. Rainfall excess estimation methods other than equation (3.7) may be equally applicable with equation (3.6) in design situations. Equation (3.8) is presented only to alert the reader to the conditions by which equation (3.6) was validated.

In addition to the calibration of equation (3.6), Stephens and Mills were able to relate maximum daily discharges to instantaneous peak discharge rates. Observed ratios of maximum daily discharge to instantaneous peak discharge are shown in Figure 3.6. Shown also are the 95 percent confidence limits for the ratios. The reader should note the wide range between the upper and lower 95 percent confidence limits. This range amounts to a probable error of ± 10 percent for the instantaneous peak estimate, at best. This probable error increases dramatically with watersheds of less than 20 square miles, which is a typical basin size.

The Cypress Creek formula is one of the very few runoff peak estimation methods specifically validated for South Florida. However, since the method is entirely empirical, it can only be applied to flatwoods watersheds which are similar to those investigated by Stephens and Mills. In addition, use of the Cypress Creek formula for estimating instantaneous peak discharges from small basins is questionable, due to the wide range of probable error shown in Figure 3.6.

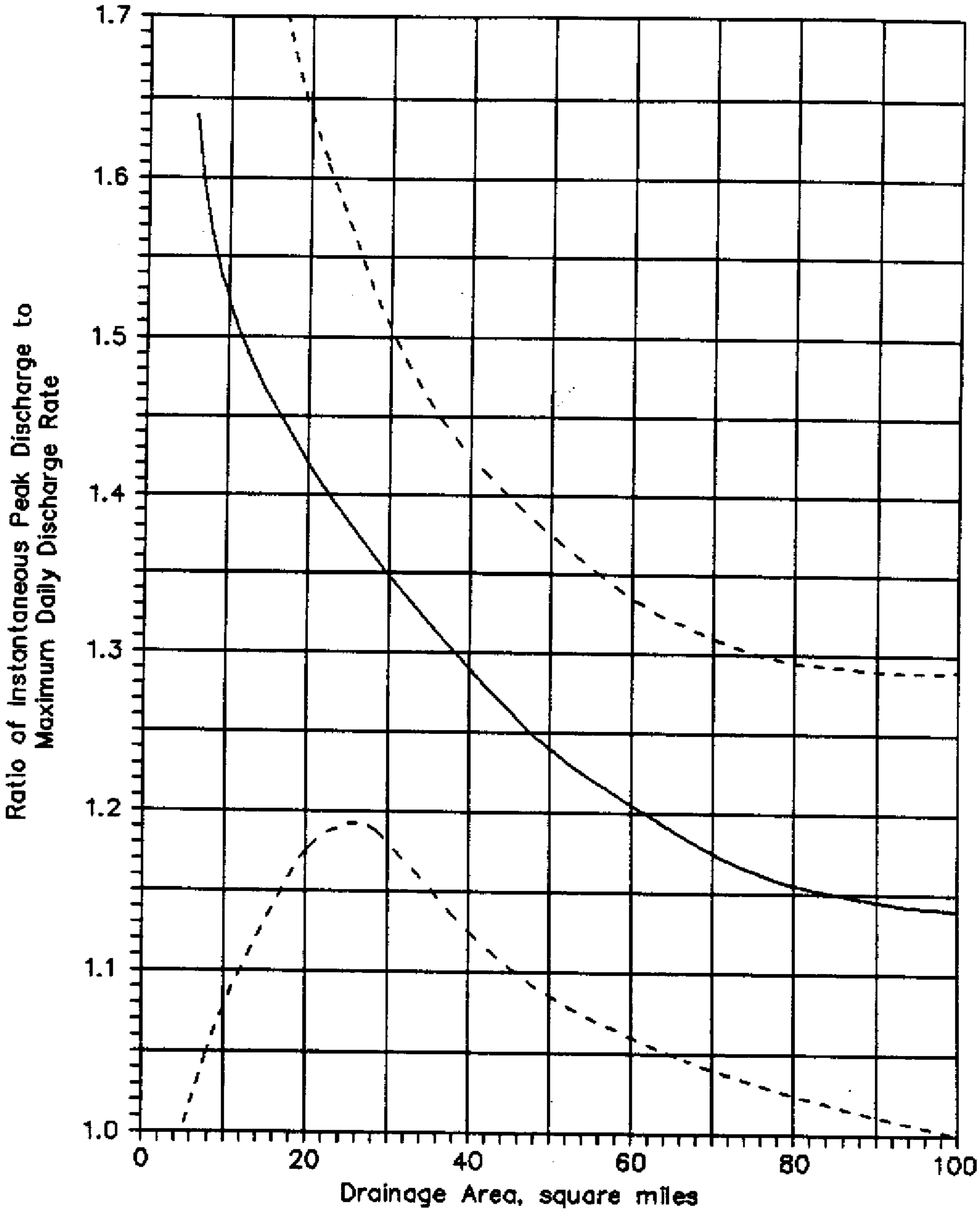


Figure 3.6. Ratios of instantaneous peak discharge to maximum daily discharge for the Cypress Creek Formula. The dashed lines represent the 95 percent confidence limits for the ratios. (reproduced from Stephens and Mills, 1965)

3.6 Time Distribution Methods

Any of the methods which are used to describe the time distribution of runoff (Sections 5 and 6) can be used to determine peak runoff rate. The SFWMD Sheetflow procedure is such a method. The program used to create the figures (example shown in Figure 3.4) is an overland flow routing model. Its use, however, has been mainly for estimation of peak runoff rates.

4. RUNOFF VOLUME ESTIMATION

4.1. SCS Curve Number Method

The Soil Conservation Service (SCS) Curve Number (CN) Method was developed to determine the quantity of runoff from a given amount of precipitation. It is described in detail in the National Engineering Handbook, Section 4 (USDA-SCS, 1985). The CN method uses basin soil and cover types, rainfall depth, and antecedent moisture condition to predict the runoff volume. This method has been recommended for both rural and urban watersheds.

The SCS runoff equation is given as

$$R_o = \frac{(P - 0.2S)^2}{P + 0.8S} \tag{4.1}$$

where

R_o = runoff depth, inches;
P = rainfall depth, inches; and
S = the basin's potential storage, inches.

The equation's solution is shown in Figure 4.1. The rainfall depth is the total depth which occurs during the period of interest, which is limited to 24 hours or less[1]. Potential storage represents maximum storage[2] in the basin, which is mainly

[1] During the development of the CN method, the duration of this rainfall was originally considered to be 24 hours. In practice, though, equation (4.1) has been used with rainfall durations shorter than 24 hours. NEH-4 (USDA - SCS, 1985) outlines a prodecure for the use of Equation (4.1) with rainfall durations longer than 24 hours. This procedure, however, uses a constant potential abstraction, S, and thereby ignores any possible changes in soil storage.

[2] The term "potential abstraction" or "potential runoff losses" is sometimes used in place of "potential storage".

infiltration, but can include surface detention, interception, and evaporation. It is related to the soil and cover conditions of a watershed. Initial storage is rainfall which is stored in the basin before runoff begins. This would include interception, infiltration and depression storage, for example. From empirical data for small agricultural watersheds, the SCS found that the initial storage could be approximated by

$$I_a = 0.2S \tag{4.2}$$

The SCS uses the curve number (CN) as an index of soil and land cover conditions and potential abstraction, and is given by the relation

$$CN = \frac{1000}{S + 10} \tag{4.3}$$

or, by rearranging,

$$S = \frac{1000}{CN} - 10 \tag{4.4}$$

Selection of a Curve Number depends upon the land use, type of soil, and Antecedent Moisture Condition (AMC). The soil types are classified into four hydrologic soil groups (A, B, C, and D) by the SCS. Appendix A defines and discusses these groups. Hydrologic soil group classifications for a specific soil can be obtained from county soil survey reports, which are published by the United Stated Department of Agriculture, Soil Conservation Service. Tables 4.1, 4.2, and 4.3 present example Curve Numbers (CN) for urban and agricultural land uses, respectively.

Assumptions involved, and limitations brought about by the assumptions, in the SCS CN method are as follows:

TABLE 4.1. CURVE NUMBERS FOR URBAN LAND USES[1].
(reproduced from USDA - SCS, 1986)

Cover description		Curve numbers for hydrologic soil group			
Cover type and hydrologic condition	Average % Impervious area[2]	A	B	C	D
Fully developed urban areas (vegetation established)					
Open space (lawns, parks, golf courses, cemeteries, etc.)[3]:					
Poor condition (grass cover < 50%		68	79	86	89
Fair condition (grass cover 50% to 75%)		49	69	79	84
Good condition (grass cover > 75%)		39	61	74	80
Impervious areas:					
Paved parking lots, roof, driveweays, 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
Western desert urban areas:					
Natural desert landscaping (pervious areas only)		63	77	85	88
Artificial desert landscaping (impervious weed barrier, desert shrub with 1- 2-inch sand or gravel mulch and basin borders)		96	96	96	96
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
1/2 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)		77	86	91	94
Idle lands (CN's are determined using cover types similar to those in Table 4.3).					

[1]Average runoff condition, AMC II, and $Ia = 0.2S$.
[2]The average percent impervious area shown was used to develop the composite CN's. Other assumptions are as follows: impervious areas are directly connected to the drainage system, impervious areas have a CN of 98, and pervious areas are considered equivalent to open space in good hydrologic condition.
[3]CN's shown are equivalent to those of pasture. Composite CN's may be computed for other combinations of open space cover type.

TABLE 4.2. CURVE NUMBERS FOR CULTIVATED AGRICULTURAL LAND USES[1].(reproduced from USDA - SCS, 1986)

Cover description			Curve numbers for hydrologic soil group			
Cover type	Treatment[2]	Hydrologic Condition[3]	A	B	C	D
Fallow	Bare soil	--	77	86	91	94
	Crop residue cover (CR)	Poor	76	85	90	93
		Good	74	83	88	90
Row Crops	Straight row (SR)	Poor	72	81	88	91
		Good	67	78	85	89
	SR + CR	Poor	71	80	87	90
		Good	64	75	82	85
	Contoured (C)	Poor	70	79	84	88
		Good	65	75	82	86
	C + CR	Poor	69	78	83	87
		Good	64	74	81	85
	Contoured & terraced (C & T)	Poor	66	74	80	82
		Good	72	71	78	81
	C & T + CR	Poor	65	73	79	81
		Good	61	70	77	80
Small grain	SR	Poor	65	76	84	88
		Good	63	75	83	87
	SR + CR	Poor	64	75	83	86
		Good	60	72	80	84
	C	Poor	63	74	82	85
		Good	61	73	81	84
	C + CR	Poor	62	73	81	84
		Good	60	72	80	83
	C & T	Poor	61	72	79	82
		Good	59	70	78	81
	C & T + CR	Poor	60	71	78	81
		Good	58	69	77	80
Close-seeded or broadcast legumes or rotation meadow	SR	Poor	66	77	85	89
		Good	58	72	81	85
	C	Poor	64	75	83	85
		Good	55	69	78	83
	C & T	Poor	63	73	80	83
		Good	51	67	76	80

[1]Average runoff condition, AMC II, and Ia = 0.2S.

[2]*Crop residue cover* applies only if residue is on at least 5% of the surface throughout the year.

[3]Hydrologic condition is based on combination of factors that affect infiltration and runoff, including (a) density and canopy of vegetative areas, (b) amount of year-round cover, (c) amount of grass or close-seeded legumes in rotations, (d) percent of residue cover on the land surface (good $\geq$ 20%), and (e) degree of surface roughness.

Poor: Factors impair infiltration and tend to increase runoff.

Good: Factors encourage average and better than average infiltration and tend to decrease runoff.

TABLE 4.3. CURVE NUMBERS FOR OTHER AGRICULTURAL LANDS[1]. (reproduced from USDA - SCS, 1986)

Cover description		Curve numbers for hydrologic soil group			
Cover type	Hydrologic Condition[3]	A	B	C	D
Pasture, grassland, or range--continuous forage for grazing.[2]	Poor	68	79	86	89
	Fair	49	69	79	84
	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.[3]	Poor	48	67	77	83
	Fair	35	56	70	77
	Good	30[4]	48	65	73
Woods--grass combination (orchard or tree farm).[5]	Poor	57	73	82	86
	Fair	43	65	76	82
	Good	32	58	72	79
Woods.[6]	Poor	45	66	77	83
	Fair	36	60	73	79
	Good	[4]30	55	70	77
Farmsteads--buildings, lanes, driveways, and surrounding lots.	--	59	74	82	86

1 Average runoff condition, AMC II, and Ia = 0.2S.

2 *Poor:* < 50% ground cover or heavily grazed with no mulch.
Fair: 50 to 75% ground cover and not heavily grazed.
Good: > 75% ground cover and lightly or only occasionally grazed.

3 *Poor:* < 50%ground cover.
Fair: 50 to 75% ground cover.
Good: > 75% ground cover.

4 Actual curve number is less than 30; use CN = 30 for runoff computations.

5 CN's shown were computed for areas with 50% woods and 50% grass (pasture) cover. Other combinations of conditoins may be computed from the CN's for woods and pasture.

6 *Poor:* Forest litter, small tress, and brush are destroyed by heavy grazing or regular burning.
Fair: Woods are grazed but not burned, and some forest litter covers the soil.
Good: Woods are protected from grazing, and litter and brush adequately cover the soil.

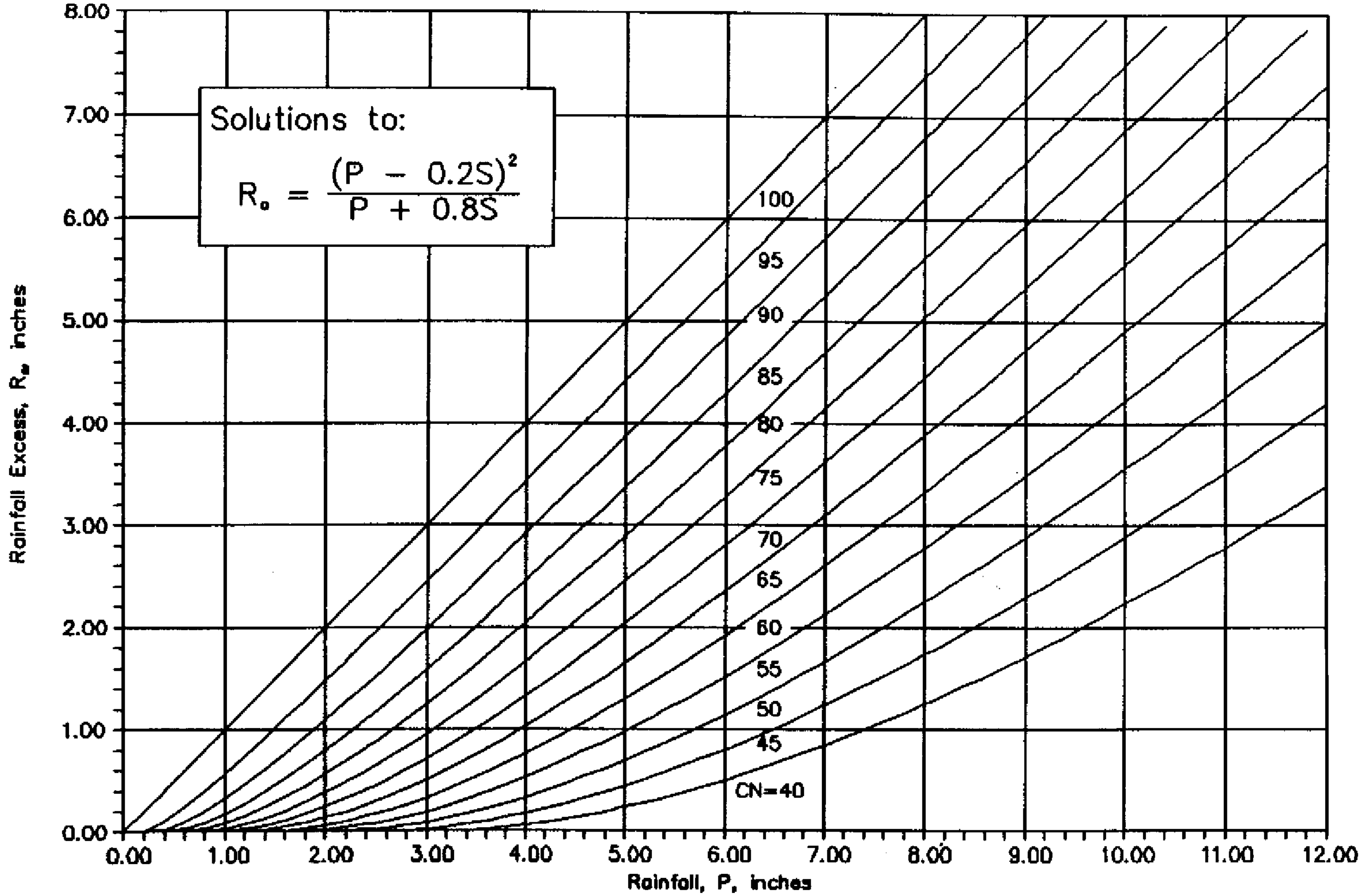


Figure 4.1. Solution of the SCS Curve Number runoff equation. (reproduced from USDA-SCS, 1986a)

1. *Any duration and intensity aspects of the rainfall depth are ignored.* The runoff equation is independent of time. Consequently, rainfall intensity or actual duration is ignored. The time distribution of rainfall can actually have a significant impact on the runoff volume. Consider two storms which occur in the same basin on two different days. Suppose the antecedent conditions and total rainfall depths were nearly identical for each. Furthermore, suppose that during the first storm, the entire depth of rainfall fell in the first three hours, and during the second, rainfall occurred uniformly during the entire day. The first storm probably (depending on the actual configuration of the basin) would have a higher runoff volume, since there would be little time for infiltration and other runoff losses to occur. Analysis by the CN method would produce the same runoff volume for each case.

2. *Rainfall depth is spatially uniform over the basin.* Assuming uniform rainfall depth over the basin limits application of the CN method to small basins.

3. *No runoff occurs unless the basin's I_a has been satisfied.* The assumption that the entire basin must satisfy its initial storage arises from assuming uniform basin characteristics and rainfall. It follows that the entire basin's initial storage will be satisfied by a certain amount of rainfall. This may be the case for small basins, but is seldom true for larger, less uniform basins. In one part of the basin, runoff may occur even with very small amounts of rainfall. For example, impervious areas within the basin, especially near the outlet, can cause runoff regardless of rainfall magnitude. The SCS recommends calculating impervious area runoff

separately (see Application below), when impervious area is directly connected to the basin outlet.

4. *Equation (4.2) is based on observations of agricultural basins of approximately 10 acres* (USDA-SCS, 1971). The coefficient, 0.2, may be different for individual watersheds, and those with differing land use. Developed watersheds will probably have an initial abstraction less than that given by equation (4.2). Watersheds having a large fraction of ponds or wetlands will have a larger initial abstraction.

5. *The ratio of runoff to total rainfall is assumed equal to the ratio of infiltration to potential maximum storage.* Equation (4.1) was developed on the basis of this assumption. When cumulative runoff is plotted against rainfall this relation generally holds true. It may not be valid for all situations, however. Equation (4.1) assumes that infiltrated water is lost and does not contribute to the basin outflow. If a shallow groundwater table exists within the basin, however, interflow and baseflow may contribute to the total runoff volume. These components are not considered in equation (4.1).

Further limitations of the CN method identified by the SCS are:

- If the calculated runoff is less than 0.5 inches, the CN method is less accurate. Other methods should be compared in this case.

- The SCS does not recommend use of this method when the composite CN for the basin is less than 40.

- Runoff volumes are calculated only for rainfall durations of 24 hours or less. If a particular storm is longer, runoff volumes are normally calculated for each day. (This procedure is discussed in NEH-4, USDA-SCS, 1985) The CN method does not account for variable total abstraction, S. This is particularly important when storms longer than 24-hours are considered. A basin's total abstraction will decrease following a rainfall event. Using normal CN methods, this change would be ignored for storms lasting several consecutive days, and thus, runoff volumes for the latter days of the storm can be underestimated.

Application. For a larger basin, with multiple land use or varying soils, the SCS presents two methods for computing a basin runoff: a weighted average CN, or a weighted average R_0. In the former, an overall basin CN can be computed as

$$CN_c = \frac{CN_1 A_1 + CN_2 A_2 + \ldots + CN_n A_n}{A_{total}} \tag{4.5a}$$

where CN_c is the composite CN for the basin, and CN_i is the CN for an individual basin area A_i. Figure 4.1 can be used to determine runoff depths based on equations (4.1) and (4.2). For the weighted R_0 method, a runoff is calculated for each portion of the basin and an overall runoff calculated as

$$R_{o_c} = \frac{R_{o_1} A_1 + R_{o_2} A_2 + \ldots + R_{o_n} A_n}{A_{total}} \tag{4.5b}$$

where R_{o_c} is the composite runoff, R_{o_i} is the runoff from individual portions of the basin, A_i, and A_{total} is the total basin area. The weighted R_0 method is preferred when there is a significant impervious area within the basin, and that area is *directly connected* to the outlet. Impervious area runoff, in this case, can be calculated independent of pervious area, and more realistic runoff values can be calculated. One must be sure, however, that the impervious area is directly connected to the outlet. If runoff from an impervious area is discharged over pervious ground (a swail for example) runoff losses will be increased.

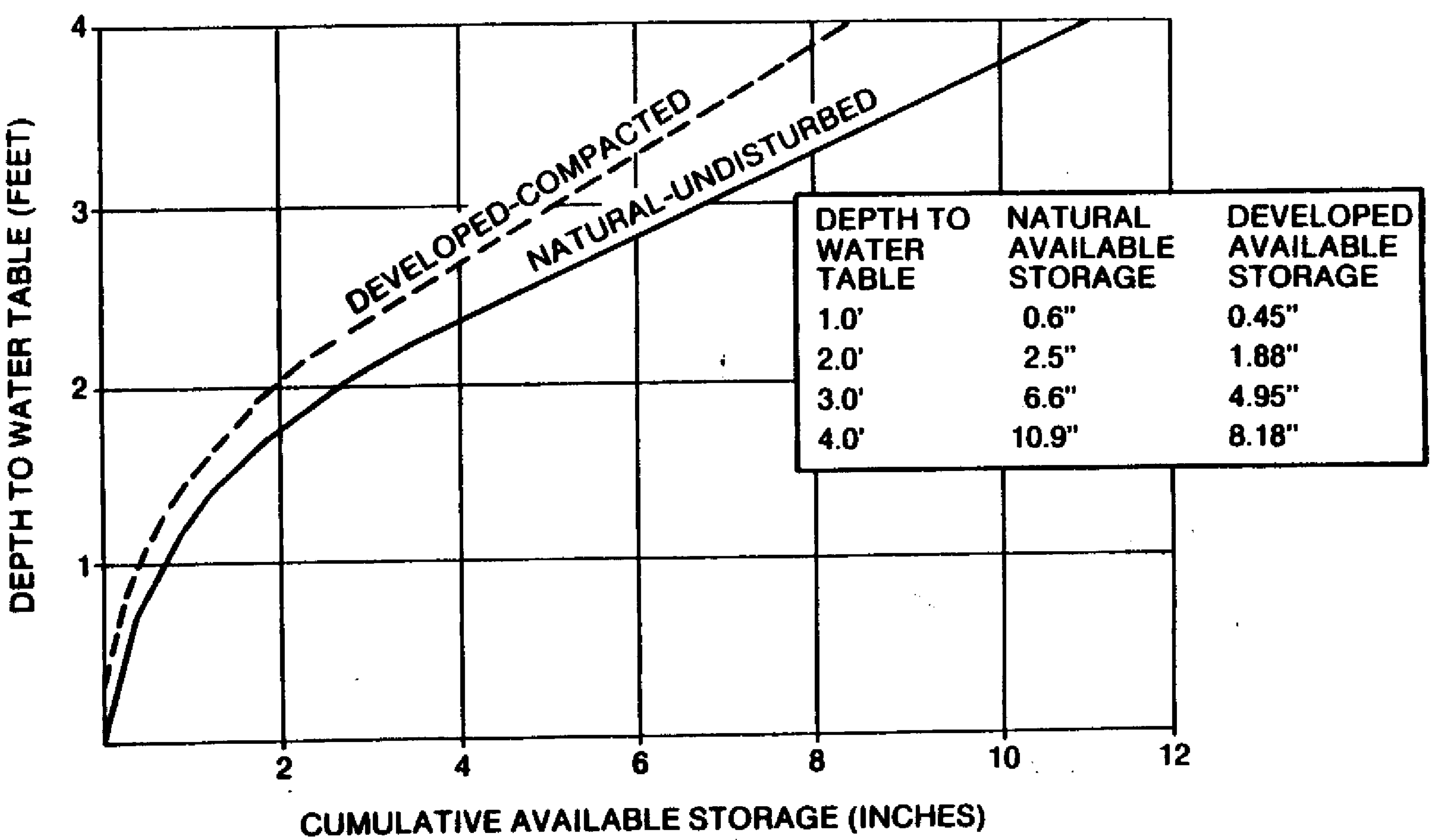
Runoff volume is related to the antecedent moisture conditions, e.g. wet antecedent moisture condition result in higher runoff volume. Clay or loamy soils expand upon wetting, thus reducing infiltration and producing more runoff. The SCS has partitioned clay and loamy soils as Antecedent Moisture Condition III (AMC III condition). With the sandy soils of Florida, this AMC classification was not considered a reliable indicator of watershed wetness, even though the soils may be wet enough to warrant classification as AMC-III. As a result, AMC-II (average condition) has been recommended by the SCS for use in Florida (USDA-SCS, 1980).

4.2. SFWMD Runoff Volume Procedure

In the SFWMD's Permit Information Manual, Volume IV, (SFWMD, 1987) a procedure is presented for the estimation of runoff volume. This approach is very similar to the Curve Number method discussed above, but one important alteration is made. Equation (4.1) is used to calculate runoff depth, but S, potential abstraction, is estimated by a different approach. In this procedure, the parameter S is given by

$$S = S_{DWT}(1 - IMP) \tag{4.6}$$

where S_{DWT} is overall watershed soil storage as a function of depth to water table; and IMP is the fraction of watershed area covered by impervious surface. S_{DWT} is obtained from Figure 4.2. The influence of development activity is represented by



DEPTH TO WATER TABLE	NATURAL AVAILABLE STORAGE	DEVELOPED AVAILABLE STORAGE
1.0'	0.6"	0.45"
2.0'	2.5"	1.88"
3.0'	6.6"	4.95"
4.0'	10.9"	8.18"

Figure 4.2. Curves for estimation of runoff volume using the SFWMD runoff volume procedure. (reproduced from SFWMD, 1984a)

two available soil storage curves: one for natural soil conditions, and another accounting for soil compaction resulting from development activities.

This method assumes that the only factor affecting the basin's potential storage is water table depth. This is in contrast with the CN method, where cover, soil type, land use, as well as water table depth play a role. This procedure does not allow for the assessment of impacts upon runoff due to crop cover, hydrologic conditions other than water table depth, soil types, or agricultural management practices. The SFWMD runoff volume procedure is typically applied to developed basins, most notably for 100-year flood levels.

Example 4.1: Runoff Volume estimation by the SCS Curve Number Method and SFWMD Procedure. A 24-hour design rainfall depth, P, of 10 inches falls on Subbasin #2 of Example 3.1. Determine the runoff volume. While using the SFWMD procedure, a water table depth of 3.0 feet will be assumed.

SCS Curve Number Method. In Example 3.1 the basin characteristics were given and a composite CN and S, was calculated:

$CN_2 \;=\; 77.9$

$S_2 \;=\; 2.84$ inches

Using equation (4.1), or consulting Figure 4.1, total runoff depth can be obtained as

$$R_o = \frac{\{10 - 0.2(2.84)\}^2}{10 + 0.8(2.84)} = 7.25\ inches \qquad \underline{\text{Answer}}$$

SFWMD Procedure. With a water table depth of 3.0 feet, and assuming uncompacted soils, S_{DWT} is obtained from Figure 4.2.

$$S_{DWT} = 7.0 \text{ inches}$$

Roofs and pavement make up the basin's impervious area. From the basin information given in Example 3.1, IMP is calculated as

$$\begin{aligned} IMP &= (20 + 20)/130 \\ &= 0.31 \end{aligned}$$

So, by equation (4.6), the potential storage estimate is

$$\begin{aligned} S &= (7.0 \text{ inches})(1 - 0.31) \\ &= 4.8 \text{ inches} \end{aligned}$$

The total runoff is then calculated by equation (4.1):

$$R_o = \frac{\{10 - 0.2(4.8)\}^2}{10 + 0.8(4.8)} = 5.9 \textit{ inches} \qquad \underline{\text{Answer}}$$

4.3. CREAMS

The CREAMS (Chemicals, Runoff, and Erosion from Agricultural Management Systems) model was developed by the Science and Education Administration of the U.S. Department of Agriculture (Knisel, 1980). CREAMS is a simulation model initially intended for evaluating long-term water quality and soil erosion impacts of various agricultural practices. Hydrology plays a key role in both water quality and erosion processes, so a hydrologic simulation model is included in CREAMS. This section presents the major hydrologic aspects of the model.

This review, except for this and the following section, deals only with event models, that is, models which predict runoff for only a few days following a single rainfall event. CREAMS, however, is a continuous simulation model. It predicts runoff continuously over time, as long as several decades, perhaps. From a hydrologic standpoint, this means that the model is able to predict watershed conditions prior to a rainfall event, particularly, soil water storage. For the specific prediction of runoff, CREAMS uses methods similar to those presented in this review. The important aspect of CREAMS and other continuous simulation models, however, is the ability to continuously predict hydrologic conditions prior to a rainfall event.

The hydrologic block of CREAMS predicts daily runoff volume, and peak runoff rate for small areas. The model has two modes of operation: (1) daily time step, and (2) hourly time step. Mode selection is dependent mainly on whether breakpoint (hourly) rainfall data is available. Model output, in both modes, is on a daily time step.

CREAMS maintains an accounting of soil water storage between and during rainfall events. This accounting is essential to the prediction of excess rainfall. The soil is divided into seven layers, and a daily mass balance is performed for each. Infiltration from the surface or upper layers is input to a layer; percolation to lower layers, or to groundwater, and evapotranspiration is output.

If the daily time step is used, runoff volume is calculated by a modified form of the SCS Curve Number (CN) method (Section 4.1). In this form, the storage parameter S (see equation (4.1)) is calculated from a weighted average of the seven layers of soil water storage. Peak discharge for the day is estimated using an empirical relation .

For the hourly time step, runoff volume is calculated by subtracting infiltration from the rainfall depth. Infiltration depths are estimated using the Green-Ampt infiltration model (see Section 4.5.1). Also in the hourly case, a peak discharge is estimated by using a kinematic wave overland flow model[1].

As a design tool for surface water management system design, CREAMS is limited for several reasons:

- *Basin area* - CREAMS is intended to be a "field-scale" model, which means very small basins. The hydrologic part of the model does not account for depression or channel storage within the watershed. This storage can be very important to runoff volume estimation in larger basins. Additionally, the soil layers are assumed to be the same over the entire basin. In larger basins, soil properties and depths can vary considerably. The model documentation (Knisel, 1980) presented data from several basins to which CREAMS was applied. The largest was approximately 100 acres. The coefficient of determination (r^2) for

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1This combination of Green-Ampt infiltration and a kinematic wave overland flow model is very similar to Akan's method (Section 2.8 and Akan, 1986) for calculation of time of concentration. A development of the governing equations is given in the CREAMS manual (USDA-SEA, 1980, pg. 16).

actual to modeled runoff was 0.15 for this basin. Heatwole (1986) applied CREAMS to a much larger area, with similar results.

- *Daily time step* - In most design cases, runoff volume information on a daily basis is unacceptable. However, CREAMS may be useful for analysis of a detention pond, for example, which must hold several months of runoff.

- *South Florida application* - The runoff volume estimation used by CREAMS does not perform well under typical South Florida conditions. Simulation of watersheds with sandy soils and flat slopes is typically not satisfactory. This is discussed further in the following section (CREAMS-WT).

CREAMS also requires a great deal more soil and weather information than a typical event model.

4.4. CREAMS-WT

Heatwole (1986), under contract with the SFWMD and USACE, adapted the CREAMS model (Section 4.3) to South Florida conditions, particularly the hydrologic simulation. Further adjustments were made to water quality aspects, but these are not discussed here. The adapted model is called CREAMS-WT, where "WT" stands for "water table".

The original CREAMS does not account for water table depth and its subsequent effects on runoff. Heatwole cites two reasons why the original CREAMS does not perform well with typical South Florida applications:

1. CREAMS will overestimate available soil storage, and thereby underestimate runoff, when the water table is near the surface; and

2. CREAMS will overestimate seepage to ground water, and again underestimate runoff, when sandy soils with high saturated conductivity are considered.

Heatwole modified the hydrologic simulation of CREAMS in order to account for these two problems.

Estimation of runoff by the SCS Curve number (CN) method remains in CREAMS-WT. The original soil moisture accounting of CREAMS, however, is altered. The soil storage factor, S (from equation (4.1)), is calculated in CREAMS-WT by

$$S = S_{max}\left[\frac{SM_{max} - SM}{SM_{max}}\right] \tag{4.7}$$

where

S_{max} = maximum value of S (from equation (4.4) with a CN for AMC I), in inches;

SM_{max} = maximum soil moisture for the profile, in inches; and

SM = current soil moisture in the profile, in inches.

as it is in the original CREAMS. These layers are collectively termed the "root zone"; soil below the root zone is termed the "lower zone".

The key aspect of the CREAMS-WT hydrologic simulation is the tracking of the ground water table in and out of the root zone. The depth to the ground water table is simulated by CREAMS-WT in two ways, depending on which zone the ground water table is in: the root zone or the lower zone.

With the water table in the root zone, a user supplied parameter, DSP, which governs the rate at which the water table falls. DSP represents the net groundwater outflow from the area considered. The ground water table will fall at this rate, provided there are no inputs to the root zone from infiltration.

If the water table is in the lower zone, its recession is estimated by an empirical curve:

$$D = T^k \tag{4.8}$$

where

D = the depth to the water table from the ground surface, in feet;

T = the number of rainless days following D = 0; and

k = a coefficient = 0.33

When the water table recedes below the root zone, D is assumed to be the depth of the root zone. For every rainless day which follows, T is incremented, and D recalculated. When seepage from the root zone occurs, D is increased according to the amount of seepage, and T recalculated.

As the ground water table recedes, in accordance with equation (4.8), storage becomes available between the bottom of the root zone and the water table. This storage in the lower zone has no effect on runoff volume calculations, except that it can limit seepage from the root zone. When the ground water table is in the root zone, the available soil moisture storage, SM_{max}, is reduced, which in turn, increases runoff volume.

CREAMS-WT, while more appropriate than the original CREAMS for South Florida, is still limited as a design tool. The CREAMS-WT model is designed for estimation of daily runoff volumes, over an extended period of time, in very small basins. Accurate daily runoff volumes cannot be expected from CREAMS-WT when it is applied to larger basins. This is for several reasons:

- Since CREAMS-WT is a rainfall driven model, it is very sensitive to rainfall data. In larger basins, a wider spatial variation in rainfall can be expected. This variation will lead to amplified errors in the CREAMS-WT results, since CREAMS-WT assumes a spatially uniform rainfall distribution.

- CREAMS-WT does not account for any surface storage within the basin. In larger basins, this can cause significant errors in CREAMS-WT results.

- CREAMS-WT does not account for overland flow or channel flow time within the basin. That is, CREAMS-WT assumes runoff leaves the basin on the same day the rainfall occurs. In larger basins, where there is a significant delay between rainfall and runoff, daily runoff volumes computed by CREAMS-WT are likely to be in error. Monthly totals, however, may be reasonable estimates.

- From a surface water management point of view, CREAMS-WT is very limited for design purposes. As is discussed for the original CREAMS, a daily time step, in most design cases, is unacceptable.

4.5 Inflitration Methods

A common method for estimation of runoff volume, and, to some extent, its time distribution, is to estimate infiltration and calculate other runoff losses separately or assume them negligible. A good deal of study has been devoted to infiltration phenomena. Complete treatises on the theory of infiltration are reviewed and presented by Hillel (1982) and Skaggs and Khaleel (1982). The theoretical basis of most infiltration analysis is Richard's equation (see Skaggs and Khaleel, 1982, pg. 126), which describes the problem completely. Richard's equation does not yield exact solutions and numerical solutions are typically not useful for design work.

For practical use, there are several approximate infiltration models, which are typically either empirical relationships or simplifications of the problem described by Richard's equation. This section discusses three approximate methods for calculation of infiltration:

- the Green-Ampt model;
- the Horton equation; and
- the Holtan equation.

In each of these methods, the soil's *infiltration capacity*[1], $f_p(t)$, or the

[1]Hillel (1982, pg. 212) uses the term *infiltrability* which may be a more clear term, but does not appear to be a standard yet.

maximum rate at which water can infiltrate the soil surface, is represented as a function of time. Figure 4.3 shows how f_p changes during a rainfall event and how runoff volume may be estimated for a steady rainfall intensity. The infiltration capacity of the soil is very high initially - much higher than the rainfall intensity, i. For a short time during the first part of the storm, all rainfall is infiltrated. As this water infiltrates, the soil's infiltration capacity is reduced. This continues until $t = t_p$, which is termed the *time to ponding*. After which, the rainfall intensity is larger than the infiltration capacity, so that water is "ponded" on the soil surface. Such ponded water, less any detention, is available for runoff. Eventually, the infiltration capacity will approach some minimum value, f_c.

4.5.1. GREEN-AMPT INFILTRATION MODEL

The Green-Ampt infiltration model was developed by Green and Ampt (1911). It is one of the more popular approximate infiltration models used in the U.S. Skaggs and Khaleel (1985, pg. 142), and Hillel (1982, pg. 217) present detailed derivations of the model. The model is incorporated into Akan's method for time of concentration (Section 2.8), the original CREAMS model (Section 4.3), HEC-1 (Section 5.6), and EPA's SWMM model (Section 6.3.3).

In the Green-Ampt model, infiltration is conceptualized as "slug flow" where the upper portion of the soil is filled (or nearly filled) by recent infiltration, and the lower soil is unaffected. This is shown in Figure 4.4. The border between these two layers is termed the wetting front. This front moves downward as more water infiltrates the soil. The rate at which the front moves determines the infiltration rate.

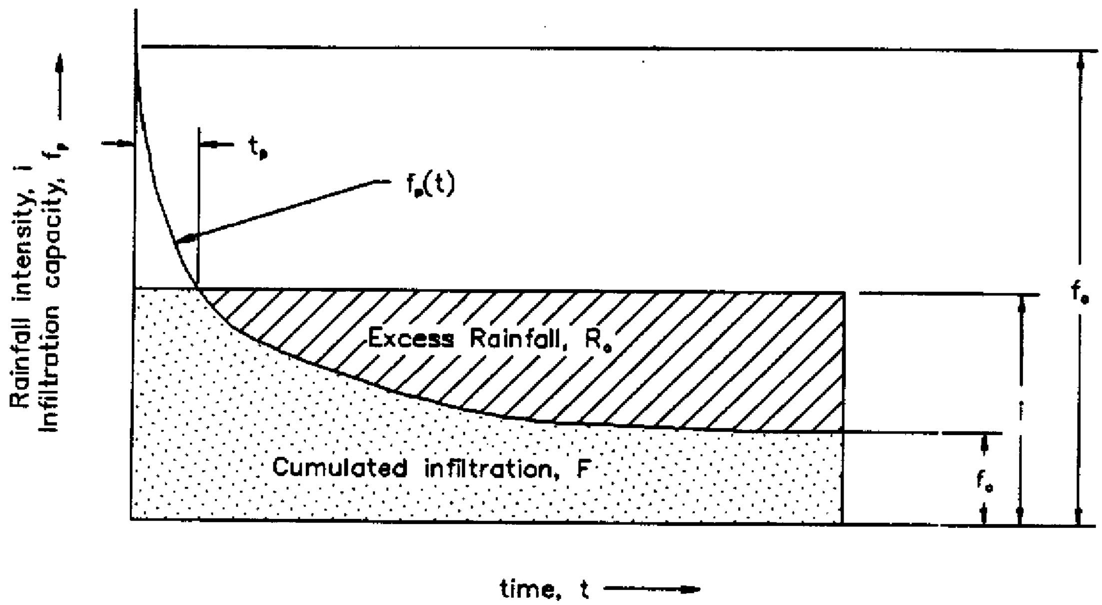

Figure 4.3. The typical change in filtration capacity over time, during a steady rainfall.

If Darcy's Law is applied to the situation shown Figure 4.4, the following results:

$$f_p(t) = K_s\left[\frac{H_o + P_f + L_F(t)}{L_F(t)}\right]$$

where

$f_p(t)$ = the infiltration capacity, as a function of time, t;

K_s = the hydraulic conductivity above the wetting front;

H_0 = the depth of ponding on the surface;

P_f = the soil water tension at the wetting front; and

$L_F(t)$ = the depth to the wetting front, as a function of time, t.

If H_0 is assumed negligible relative to L_F, and the total of previous infiltration, F, is

$$F_p(t) = (\theta_s - \theta_i)L_F(t) = ML_F(t)$$

where

M = initial soil water deficit (or fillable porosity);

θ_s = volumetric soil water content of the wet soil above the wetting front (also called effective or natural porosity) and

θ_i = initial volumetric soil water content,

the infiltration capacity can be derived as

$$f_p(t) = K_s + \frac{K_s M P_f}{F_p(t)} \tag{4.9}$$

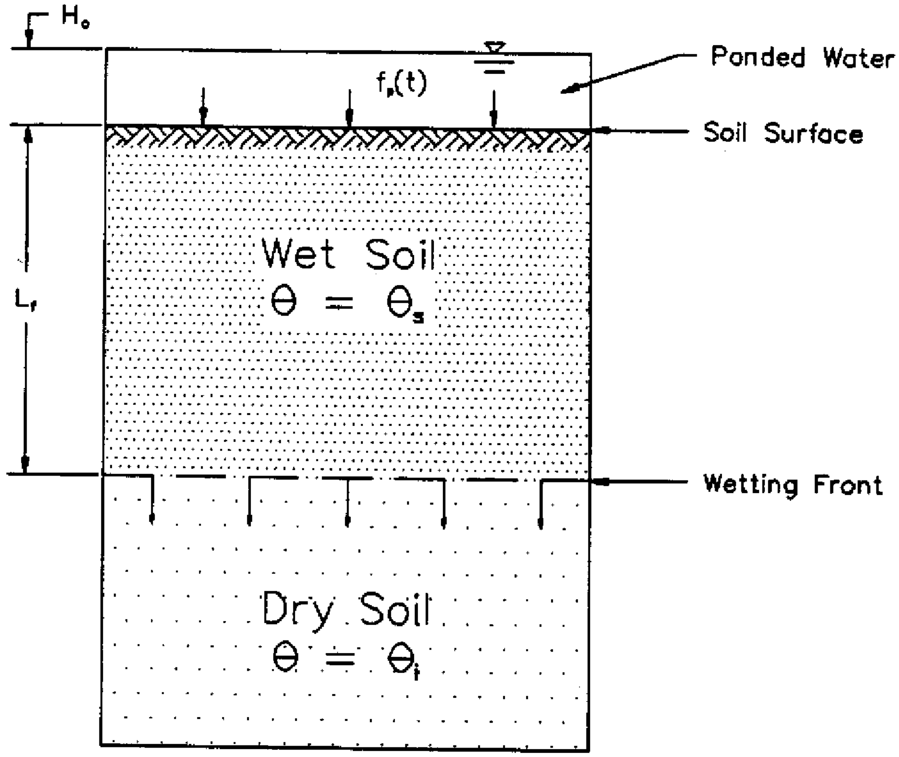

Figure 4.4. "Slug flow" conceptualization of infiltration used in the Green-Ampt infiltration model. (after Skaggs and Khaleel, 1982)

Equation (4.9) is dimensionally correct, so any consistent system of units is allowed[1]. For example, $f_p(t)$ and K_s could be in inches per hour and $F_p(t)$ and P_f could be in inches.

The parameters, K_s, θ_s, and P_f, are best determined by measurement, however, Skaggs and Khaleel (1982) outline some possible estimation procedures, as do Rawls and Brakensiek (1983). The hydraulic conductivity above the wetting front, K_s, should not be confused with saturated hydraulic conductivity, K. K_s is usually considerably lower than K because of air entrapment in the soil. Likewise, θ_s should not be confused with the soil porosity. θ_s will tend to be somewhat smaller than total porosity because of entrapped air.

If, in equation (4.9), $f_p(t) = dF_p(t)/dt$ is substituted and the equation integrated with time, with the condition that $F_p(t) = 0$ at $t = 0$, the equation

$$K_s t = F_p(t) - MP_f ln\left[1 + \frac{F_p(t)}{MP_f}\right] \tag{4.10}$$

results. The derivation of equation (4.9) is based on the following assumptions:

1. *The soil is homogeneous, deep and free of impeding layers.* This means that effects of hardpans and water tables are not directly accounted for in the Green-Ampt infiltration model. Direct application of the Green-Ampt model in South Florida is severely limited for this reason. Some research has shown that the Green-Ampt model may be extended to layered soils, crusted soils, and for other non-homogeneous

[1]This applies to the remainder of the section as well

conditions (for references see Skaggs and Khaleel, 1982, pg. 143). Some applications have been extended to account for a water table. One example is DRAINMOD (USDA-SCS, 1983), which is a model used by the SCS in South Florida for design and evaluation of subsurface drainage systems. The program is currently limited to humid regions. DRAINMOD estimates infiltration using the Green-Ampt model, but adjusts the parameters to account for the position of the water table[1].

2. *A sharp, well-defined wetting front.* This arises from the "slug-flow" concept used in the Green-Ampt model. Assuming a sharp wetting front is not entirely accurate; there should actually be a more gradual change in water content between the upper and lower soil layers. However, the sharp front seems to be a good approximation.

3. *The soil surface is always ponded and the depth of ponding is constant and small.* The derivation of equation (4.10) assumes a ponded surface, so that the infiltration rate is equal to the infiltration capacity at all times. This is not the case with rainfall infiltration, so equation (4.10) requires some modification before it is applicable to rainfall infiltration. Research[2] has shown that such modifications are acceptable since infiltration capacity, f_p, is accurately represented by a

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[1]At the time of this writing, the authors were not able to ascertain how DRAINMOD performed under South Florida conditions, or whether runoff information from the model had been compared to observed data.

[2]There are several discussions on this research. The reader is referred to Skaggs and Khaleel (1982, pg. 147), USDA-SCS (1983, pg. 2-7), and Reeves and Miller (1975).

function of total prior infiltration, F, regardless of the infiltration rate history involved.

A modification of equation (4.10), by Mein and Larson (1973), provides a good way to apply the Green-Ampt model to real rainfall events. This equation can be used to calculate rainfall infiltration after t_p:

$$K_s(t - t_p + t_p') = F_p(t) - MP_f ln\left[1 + \frac{F_p(t)}{MP_f}\right] \tag{4.11}$$

where

t = actual time during the rainfall event;

t_p = actual time to ponding during the event; and

t_p' = an estimate of the time to ponding assuming that the soil was initially ponded.

Equation (4.11) is a means of adjusting equation (4.10) for the limitations brought about by assumption 3 above. Water is very seldom ponded at the surface during the initial part of the storm, and any rainfall prior to ponding will infiltrate. The parameter t_p' is calculated by equation (4.10) using the actual infiltration prior to ponding, $F_p(t_p)$.

When applying the Green-Ampt model, the actual infiltration rate is calculated as

$$f(t) = i(t), \qquad for\ t \le t_p \tag{4.12a}$$

$$f(t) = f_p(t) = K_s + K_s\frac{MP_f}{F_p(t)}, \quad for\ t > t_p \tag{4.12b}$$

This simply states that prior to the time of ponding, all rainfall is infiltrated, and afterward the soil's infiltration capacity will limit infiltration. Time to ponding for breakpoint rainfall data can be estimated by an equation presented by Morel-Seytoux (1981):

$$t_p = t_{j-1} + \frac{1}{i_j}\left[\frac{MP_f}{i_j/K_s - 1} - \sum_{n=1}^{j-1} i_n(t_n - t_{n-1})\right] \tag{4.13}$$

where

- j = the number of the time interval in which t_p occurs;
- i_j = the rainfall rate during interval j;
- t_j = time at the end of interval j.

Equation (4.13) is applied to each successive rainfall interval until the t_p is less than the time at the end of the interval, t_j. After the time to ponding is calculated, equation (4.10) is used to calculate the infiltration for each successive rainfall interval. Once the interval in which ponding occurs is located, the infiltration prior to t_p is calculated by interpolating between the end points of the interval. An example of this process is presented in Example 4.2.

Heatwole (1986) commented that the Green-Ampt infiltration model, as presented above, (specifically, its use in the CREAMS model) does not accurately represent the infiltration process in South Florida flatwoods watersheds. This is for two reasons. First, the very high conductivity (used to estimate K_s) of most South Florida soils cause the Green-Ampt model to calculate high values for rates of deep seepage, or the limiting value of equation (4.9). In actuality, there is very little seepage from South Florida flatwoods watersheds, due to high groundwater tables, meaning the Green-Ampt model will considerably over estimate infiltration from a rainfall event. Second, the Green-Ampt model does not account for a ground water table and its influence on the infiltration process. Most South Florida flatwoods watersheds have water tables very close to the surface. Consequently, for South Florida applications, the Green-Ampt infiltration model should incorporate some representation of the groundwater table and its effects on infiltration.

Example 4.2. Estimation of runoff volume using the Green-Ampt Infiltration model. A basin is subject to the storm shown in Table 4.4. The soil in the basin is a fine sand, for which the following parameters were estimated:

$K_s =$ 0.90 inches/hour

$P_f =$ 2.75 inches

$\theta_s =$ 0.35

Assume the initial soil water content, θ_i, is 0.10, so that $M = 0.35 - 0.10 = 0.25$. In this example, infiltration is calculated for this situation using the Green-Ampt model. Rainfall excess is then calculated assuming no losses other than infiltration. The results are shown in Table 4.5 and in Figure 4.5.

Estimate of Time to Ponding. To estimate the time to ponding, equation (4.12) will be applied to each successive interval until the calculated t_p indicates that the time to ponding occurs within that period. To check period 1, the first try at time to ponding is calculated as

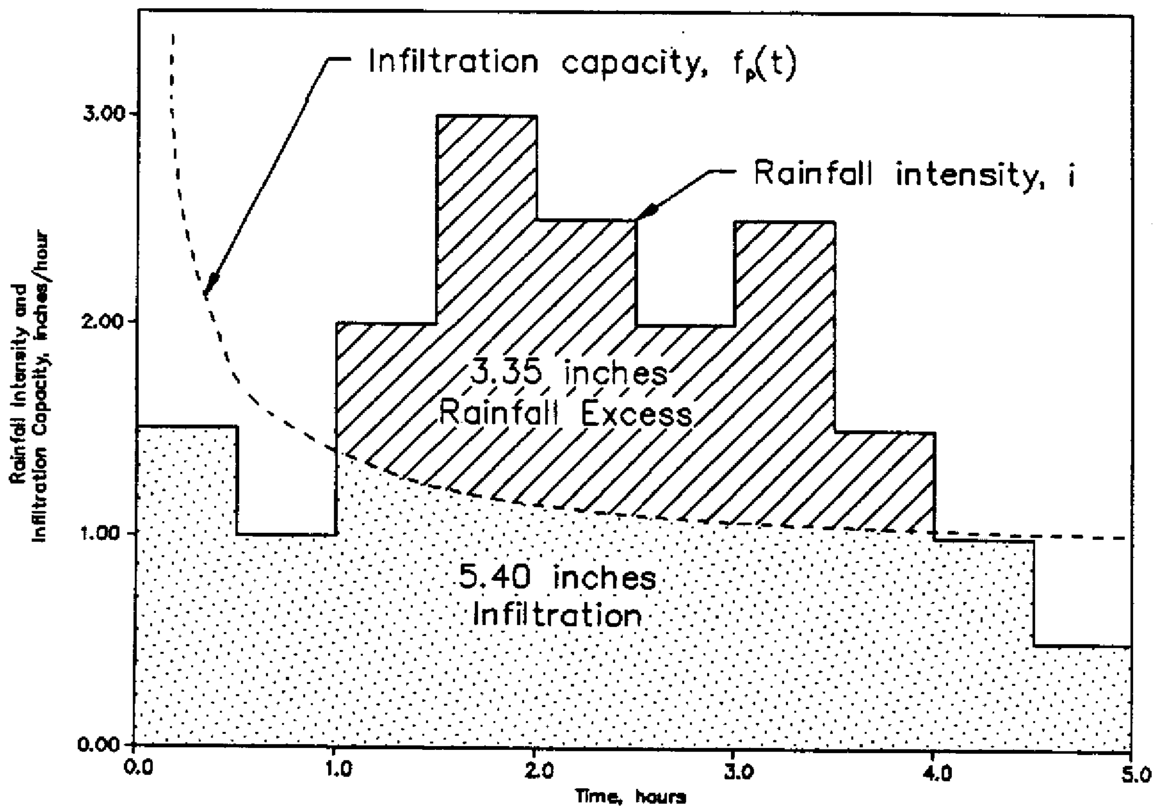

Figure 4.5. Results of runoff volume calculations using the Green-Ampt infiltration model in Example 4.2.

TABLE 4.4. RAINFALL DISTRIBUTION FOR EXAMPLE 4.2.

Period	Ending Time (hours)	Cumulative Rainfall (inches)	Period Rainfall (inches)	Rainfall Intensity (in/hr)
1	0.5	0.75	0.75	1.50
2	1.0	1.25	0.50	1.00
3	1.5	2.25	1.00	2.00
4	2.0	3.75	1.50	3.00
5	2.5	5.00	1.25	2.50
6	3.0	6.00	1.00	2.00
7	3.5	7.25	1.25	2.50
8	4.0	8.00	0.75	1.50
9	4.5	8.50	0.50	1.00
10	5.0	8.75	0.25	0.50

$$t_p = t_o + \frac{1}{i_1}\left[\frac{MP_1}{i_1/K_s - 1} - 0\right]$$

$$= 0.0\ hr + \frac{1}{1.5\ in/hr}\left[\frac{(.25)(2.75\ in)}{(1.5\ in/hr)/(0.9\ in/hr) - 1}\right]$$

$$= 0.69\ hr$$

which is longer than the first interval. Consequently, the time to ponding must occur in the second period or after. For period 2,

$$t_p = 0.5 + \frac{1}{1.00}\left[\frac{(0.25)(2.75)}{(1.00)/(0.90) - 1} - (0.5)(1.5)\right]$$

$$= 5.94\ hr$$

indicating that time to ponding does not occur in period 2 either. Then for period 3,

$$t_p = 1.0 + \frac{1}{2.00}\left[\frac{(0.25)(2.75)}{(2.00)/(0.90) - 1} - (0.5)(1.5 + 1.0)\right]$$

$= 0.66\ hr$

which indicates ponding occurred *before* period 3. This is a simple problem with the discretization of the rainfall. The infiltration capacity at the end of period 2 was more than the rainfall intensity, 1.00 inches/hour, but evidently less than the intensity of period 3, 2.00 inches/hour. So, ponding occurred right at the beginning of period 3, or $t_p = 1.0$ hours, as shown in Figure 4.5.

Calculation of Infiltration. The infiltration prior to ponding is the total rainfall prior to t_p:

$$F_p(1.0) = 0.75 \text{ inches} + 0.5 \text{ inches} = 1.25 \text{ inches.}$$

Had the soil been initially ponded the time to infiltrate $F_p(1.0)$, t_p', is calculated by solving equation (4.10) for t:

$$t_p' = \frac{1}{K_s}\left\{F_p(1.0) - MP_f ln\left[1 + \frac{F_p(1.0)}{MP_f}\right]\right\}$$

$$= \frac{1}{0.9\ in/hr}\left\{1.25\ in - (0.25)(2.75\ in)\ ln\left[1 + \frac{1.25\ in}{(0.25)(2.75\ in)}\right]\right\}$$

$$= 0.60\ hr$$

For each interval following ponding, the amount of infiltration is calculated by taking the difference in cumulative infiltration between the end and beginning of the period. For period 3, the potential infiltration is

$$F_3 = F_p(1.5 \text{ hours}) - F_p(1.0 \text{ hours})$$

TABLE 4.5. RESULTS OF RUNOFF VOLUME CALCULATIONS USING THE GREEN-AMPT INFILTRATION MODEL IN EXAMPLE 4.2.

Period	Time t (hours)	Period Rainfall (inches)	Period Rainfall (inches)	Cumulative Infilt. $F_p(t)$ (inches)	Period Runoff (inches)	Rainfall Intensity i (in/hr)	Ending Infilt. Rate $f_p(t)$ (in/hr)
1	0.5	0.50	0.75	0.75	0.00	1.50	1.725
2	1.0	0.75	0.50	1.25	0.00	1.00	1.395
3	1.5	1.00	0.65	1.90	0.35	2.00	1.226
4	2.0	1.50	0.59	2.49	0.91	3.00	1.148
5	2.5	1.25	0.56	3.05	0.69	2.50	1.103
6	3.0	1.00	0.55	3.60	0.45	2.00	1.072
7	3.5	1.25	0.53	4.13	0.72	2.50	1.050
8	4.0	0.75	0.52	4.65	0.23	1.50	1.033
9	4.5	0.50	0.50	5.15	0.00	1.00	1.020
10	5.0	0.25	0.25	5.40	0.00	0.50	1.015
Totals		8.75			3.35		

$F_p(1.5)$ is calculated by solving equation (4.11) for F_p at time $t = 1.5$ hours. This can be done graphically (see Morel-Seytoux, 1983), or by trial and error, to get

$$F_p(1.5) = 1.90 \text{ inches}$$

The infiltration for period 3 is then

$$F_3 = 1.90 \text{ inches} - 1.25 \text{ inches}$$
$$= 0.65 \text{ inches}$$

Similarly, $F_p(2.0)$ is found to be 2.49 inches, so that 2.49 - 1.90 = 0.59 inches is infiltrated during period 4. This process is repeated for each interval after ponding. As long as the rainfall intensity is greater than the infiltration capacity, infiltration proceeds at its potential rate.

4.5.2 HORTON INFILTRATION EQUATION

Horton (1939) developed an empirical three-parameter equation to describe infiltration capacity:

$$f_p(t) = f_c + (f_o - f_c)e^{-\beta t} \tag{4.14}$$

where

f_c = final constant infiltration rate;

f_o = initial (t = 0) infiltration rate;

β = a decay factor; and

$f_p(t)$ = infiltration capacity as a function of time t.

The parameters f_c is a constant which depends only on configuration of the soil and underlying layers. The parameters f_0 and β will depend primarily upon initial conditions in the soil. Equation (4.14) is merely a function which has approximately the right shape to represent infiltration capacity over time. There is no physical basis for the equation. Hence, the parameters must be measured, usually by infiltrometer. This can be a serious limitation in design work, since such measurements are costly and time consuming.

Note that in equation (4.14), the infiltration capacity depends only on time. This implies than the soil surface is assumed to be ponded, or the rainfall intensity, i, is always greater than $f_p(t)$. This does not make the Horton equation readily applicable to actual rainfall infiltration. The SFWMD Sheetflow procedure (Section 3.5) uses a modified Horton infiltration procedure to calculate excess rainfall. This procedure is outlined in Example 4.3.

Example 4.3. Estimation of runoff volume using the Horton infiltration equation. The Horton infiltration equation is used in the SFWMD Sheetflow program (see Section 3.5). In this example, the Sheetflow program's runoff volume estimation procedure will be derived and applied to an example situation.

If equation (4.14) is integrated over the time interval t = $t_2 - t_1$, the potential depth of infiltration during t can be represented as

$$F_{\Delta t} = \int_{t_1}^{t_2} f_p(x)\,dx = f_c(t_2 - t_1) + \frac{f_o - f_c}{\beta}(e^{-\beta t_1} - e^{-\beta t_2}) \tag{4.15}$$

For the interval from t = 0 to time t cumulative infiltration as a function of time, F(t), can be described as

$$F_c(t) = f_c t + \frac{f_o - f_c}{\beta}(1 - e^{-\beta t}) \tag{4.16}$$

if F(0) is assumed zero. Soil moisture storage over time, S(t), can be represented by

$$S(t) = S_o - [F(t) - f_c t] \tag{4.17}$$

where

S_o = the initial soil storage, inches;

S(t) = the soil storage at time, t; and

F(t) is given in inches, f_c in inches per hour, and t in hours. Combining equations (4.16) and (4.17),

$$S_o - S(t) = \frac{f_o - f_c}{\beta}(1 - e^{-\beta t}) \tag{4.18}$$

A useful representation of initial storage can be obtained by taking the limit of equation (4.18) as t goes to infinity, when the available storage is filled (S(t) = 0):

$$S_o = \frac{f_o - f_c}{\beta} \tag{4.19}$$

With this equation, the initial infiltration rate can be calculated based on the initial storage. Substituting equation (4.19) back into equation (4.18) results in

$$S(t) = \frac{f_o - f_c}{\beta} e^{-\beta t} = \frac{f_p(t) - f_c}{\beta}$$

or

$$f_p(t) = \beta S(t) + f_c \tag{4.20}$$

which relates the infiltration rate directly to the soil moisture storage at any time t.

The essential infiltration methodology in the SFWMD Sheetflow program is to apply equation (4.16) for each interval of discretized rainfall[1] to calculate a "potential" infiltration depth. If the rainfall during the interval is less than the potential infiltration, all rainfall is infiltrated. Otherwise, the potential infiltration limits the infiltration. The soil moisture storage at the end of the period is then calculated by equation (4.17), using the actual infiltration for the interval. The final infiltration rate for the period is then estimated by equation (4.20), with $t = \Delta t$, and used as the initial infiltration rate for the next interval.

A development of the Sheetflow program's infiltration methodology was initially presented at SFWMD by Higgins (1979); some additional documentation is

[1]In the sheetflow program itself, this procedure is followed only up to the time rainfall is greater than potential infiltration. The beginning of this interval is considered $t = 0$. Afterward, potential infiltration is calculated by equation (4.15) for each interval. The two methodologies can be shown to be equivalent.

presented by Cooper and Neidrauer (draft, 1988). The reader should refer to these two sources for more detail.

The SFWMD Sheetflow program assumes the following parameters for South Florida soils:

$f_0 =$ 3.1 inches/hour

$f_c =$ 0.01 inches/hour

Assume that a particular undeveloped basin has a depth to the water table of approximately 3 feet, and is subject to the storm given in Example 4.2 (Table 4.4). The results for each time step are shown in Table 4.6, and graphically in Figure 4.6. As a sample, calculations for the first three time steps follow.

From Figure 4.2, $S_0 = S(0) = 6.6$ inches. The decay parameter, β, can be estimated by equation (4.19), as

$$\beta = \frac{3.1 - 0.01\ \mathit{inches/hour}}{6.6\ \mathit{inches}} = 0.468\ \mathit{hour}^{-1}$$

At the beginning of period 1, soil storage is at its initial value, and the initial infiltration rate is f_0. The potential depth of infiltration for period 1 is given by equation (4.16), as

$$F_1 = (0.01\ \mathit{in/hr})(0.5\ \mathit{hr}) + \frac{(3.1 - 0.01\ \mathit{in/hr})}{0.468\ \mathit{hr}^{-1}}(1 - e^{-(0.468/\mathit{hr})(0.5\ \mathit{hr})})$$

$$= (0.005\ \mathit{in}) + (6.603\ \mathit{in})(0.2086)$$

$$= 1.38\ \mathit{inches}$$

which is considerably larger than the rainfall for the period (0.75 inches). This means that all of the rainfall for the period is infiltrated. So, the soil moisture at the end of period is calculated by equation (4.17) substituting actual infiltration, I, for the potential infiltration:

$$S_1 = S_o - (I_1 - f_c t_1) = (6.6 \text{ inches}) - [(0.75 \text{ inches}) - (0.01 \text{ inch/hr})(0.5 \text{ hr})]$$
$$= 5.85 \text{ inches}$$

and no runoff occurred. Since equation assumes that the potential infiltration is met at any time t, the initial infiltration rate for period 2 must be estimated from the infiltration which actually occurred. This is done by equation (4.20), so that the initial infiltration rate for period 2, f_2 is

$$f_2 = (0.468 \text{ hour}^{-1})(5.85 \text{ inches}) + 0.01 \text{ inches/hour} = 2.75 \text{ inches/hour}$$

The process for period 2 is the same, however, there is a different initial storage, and initial infiltration rate. Applying equation (4.16) again, to determine the potential infiltration depth:

$$F_2 = (0.005\ in) + \frac{(2.75 - 0.01\ in/hr)}{0.468\ hr^{-1}}(0.2086) = 1.23\ inches$$

This, again, is greater than the rainfall for the period, so all the rainfall, 0.50 inches, infiltrates. The resulting soil storage is

$$S_2 = (5.85 \text{ inches}) - [(0.50 \text{ inches}) - (0.01 \text{ inch/hr})(0.5 \text{ hr})] = 5.36 \text{ inches}$$

and the potential infiltration rate at the beginning of period 3 is

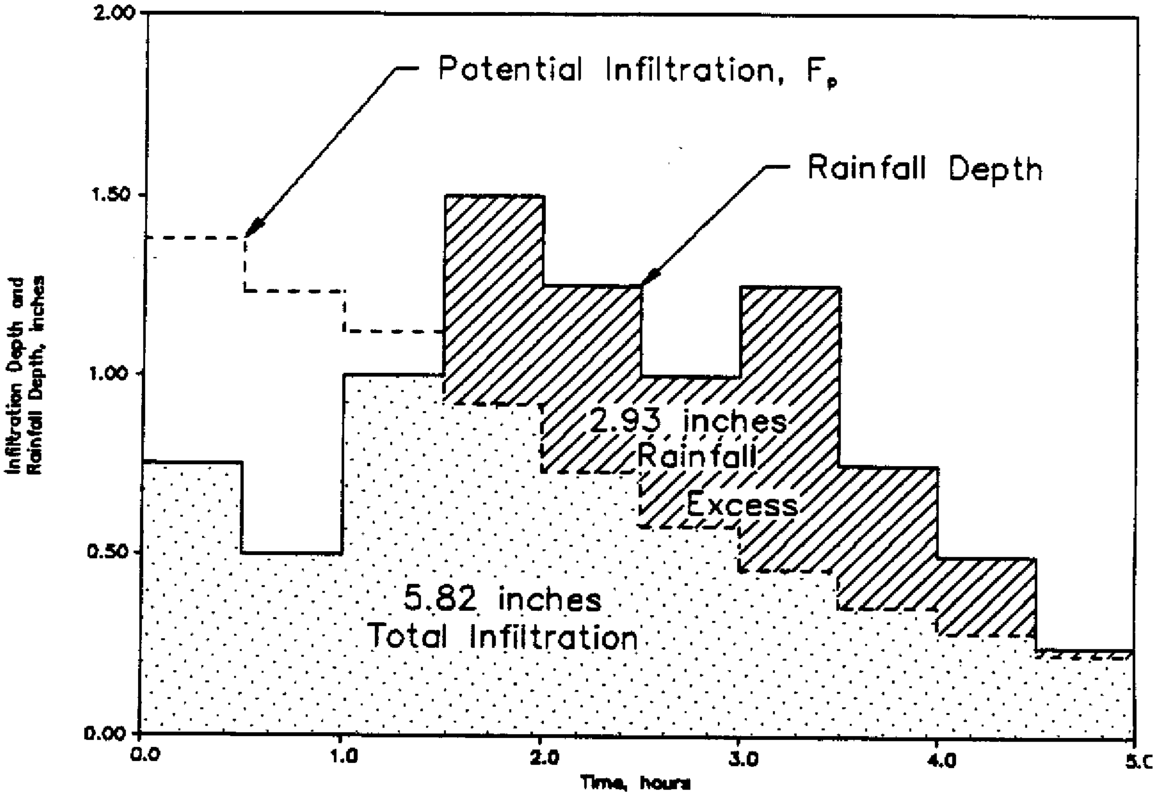

Figure 4.6. Results of runoff volume calculations using Horton's equation in Example 4.3.

TABLE 4.6. RESULTS OF RUNOFF VOLUME CALCULATIONS USING HORTON'S EQUATION IN EXAMPLE 4.3.

Period	Ending Time (hours	"rainfall (inches)	Initial Storage (inches)	Initial Infilt Rate (in/hr)	Pot. Infilt Depth (inches)	Actual Infilt Depth (inches)	Runoff Depth (inches)
1	0.5	0.75	6.60	3.10	1.38	0.75	0.00
2	1.0	0.50	5.85	2.75	1.23	0.50	0.00
3	1.5	1.00	5.36	2.52	1.12	1.00	0.00
4	2.0	1.50	4.36	2.05	0.92	0.92	0.58
5	2.5	1.25	3.45	1.63	0.73	0.73	0.52
6	3.0	1.00	2.73	1.29	0.58	0.58	0.42
7	3.5	1.25	2.16	1.02	0.46	0.46	0.79
8	4.0	0.75	1.71	0.81	0.36	0.36	0.39
9	4.5	0.50	1.35	0.64	0.29	0.29	0.21
10	5.0	0.25	1.07	0.51	0.23	0.23	0.02
Total		8.75				5.82	2.93

$$f_3 = (0.468\ \text{hour}^{-1})(5.36\ \text{inches}) + (0.01\ \text{inches/hour}) = 2.52\ \text{inches/hour}$$

Again, during period 2, no runoff occurred.

For period 3, the potential infiltration depth is

$$F_3 = (0.005\ in) + \frac{(2.52 - 0.01\ in/hr)}{0.468\ hr^{-1}}(0.2086) = 1.12\ inches$$

which is still greater than the rainfall. No runoff occurs in period 3. The initial storage for period 4 is

$$S_4 = (5.36\ \text{inches}) - [(1.00\ \text{inches}) - (0.01\ \text{in/hr})(0.5\ \text{hr})] = 4.36\ \text{inches},$$

and the initial infiltration rate for period 4 is

$f_4 = (0.468 \text{ hour}^{-1})(4.36 \text{ inches}) + (0.01 \text{ inch/hr}) = 2.05 \text{ inches/hour}$

4.5.3. HOLTAN INFILTRATION EQUATION

Holtan (1961) presented an approximate infiltration method which is used by the Agricultural Research Service (ARS, U.S. Department of Agriculture). A brief summary of the method's use is presented by Skaggs and Khaleel (1982). In the method, infiltration capacity is represented as

$$f_p = GaS^{1.4} + f_c \tag{4.21}$$

where

f_p = infiltration capacity, inches per hour;

G = a crop cover growth index, as a fraction of maturity;

a = an index of soil surface conditions, inches per hour per inch$^{1.4}$;

f_c = the final infiltration rate, inches per hour; and

S = available soil moisture storage, inches, which

$= (\theta_s - \theta_i)d$

where

θ_s = final volumetric soil water content, or effective porosity;

θ_i = initial volumetric soil water content; and

d = the depth of the soil layer, inches.

Equation (4.21) is an empirical equation based on data from a wide variety of watershed conditions. The soil surface index "a" represents the general ability of the soil to infiltrate water, which depends on the connected porosity of the soil and plant root density. Some example values of "a" are shown in Table 4.8. The final infiltration rate, f_c, is based on the SCS hydrologic soil group classifications (see Section 4.1 and Appendix A), as shown in Table 4.7.

Application of Holtan's equation, as outlined by Skaggs and Khaleel (1982), involves a budget of soil moisture storage. As water infiltrates, the available soil moisture decreases; as plants transpire, available soil moisture storage increases. Over a period of time, $\Delta t = t_2 - t_1$, the available soil moisture at t_2 is represented as

$$S_{t2} = S_{t1} - F_{\Delta t} + ET\Delta t$$

where

$F_{\Delta t}$ = the depth of infiltration during Δt, inches;

ET = the evapotranspiration rate during Δt; and

Δt is in hours.

The use of Holtan's equation in South Florida may be limited, due to the method's generality. The method is easy to apply and coefficients are readily

TABLE 4.7. EXAMPLE VALUES FOR FINAL INFILTRATION RATE, f_c, IN THE HOLTAN EQUATION(Reproduced from Sakaggs and Khaleel, 1982)

Hydrologic Soil Group	f_c inches/hour
A	0.45-0.30
B	0.30-0.15
C	0.15-0.05
D	0.05-0.00

TABLE 4.8. EXAMPLE VALUES FOR THE SOIL SURFACE CONDITION PARAMETER, a, IN THE HOLTAN INFILTRATION EQUATION. (Reproduced from Skaggs and Khaleel, 1982, pg. 142)

Land Use or Cover	Basal Rating*	
	Poor Condition	Good Condition
Fallow**	0.10	0.30
Row crops	0.10	0.20
Small grains	0.20	0.30
Hay (legumes)	0.20	0.40
Hay (sod)	0.40	0.60
Pasture (bunch grass)	0.20	0.40
Temporary pasture (sod)	0.20	0.60
Permanent pasture (sod)	0.80	1.00
Woods and forests	0.80	1.00

* Adjustment needed for "weeds" and "graz ing".

** For fallow land only, poor condition means "after row crop," and good condition means "after sod".

available. Equation (4.21), or a similar relationship, may be useful for South Florida conditions since the infiltration capacity is based strictly on the available soil moisture storage. However, some verification of existing coefficients or measurement of new coefficients would be necessary before Holtan's equation could become useful in South Florida.

5. TIME DISTRIBUTION OF RUNOFF - HYDROGRAPHS

This section presents various hydrograph methods for estimation of the time distribution of runoff. This material is presented in a slightly different manner than in previous sections. The main topic of the section is several synthetic unit hydrograph methods. These methods have a common theoretical basis, and, to some extent, rely on the same general assumptions. Furthermore, they are applied in a similar manner. This section is organized as follows:

- Section 5.1 discusses general unit hydrograph theory, and defines necessary terms and general hydrograph assumptions.

- Section 5.2 presents several synthetic unit hydrograph methods. Assumptions and limitations specific to the methods are presented there.

- Section 5.3 discusses the application of unit hydrographs, both real and synthetic.

- Sections 5.4 through 5.8 deal with other hydrograph methods. These use hydrograph methods in which composite hydrograph information is computed directly by a variety of methods. These are typically computer-based models.

5.1. Unit Hydrographs

In runoff analyses, a hydrograph is defined as a graph (or other representation) of watershed outflow over time. It is usually used to describe watershed response, in terms of discharge rate, to a particular rainfall or storm event. Figure 5.1(a) shows

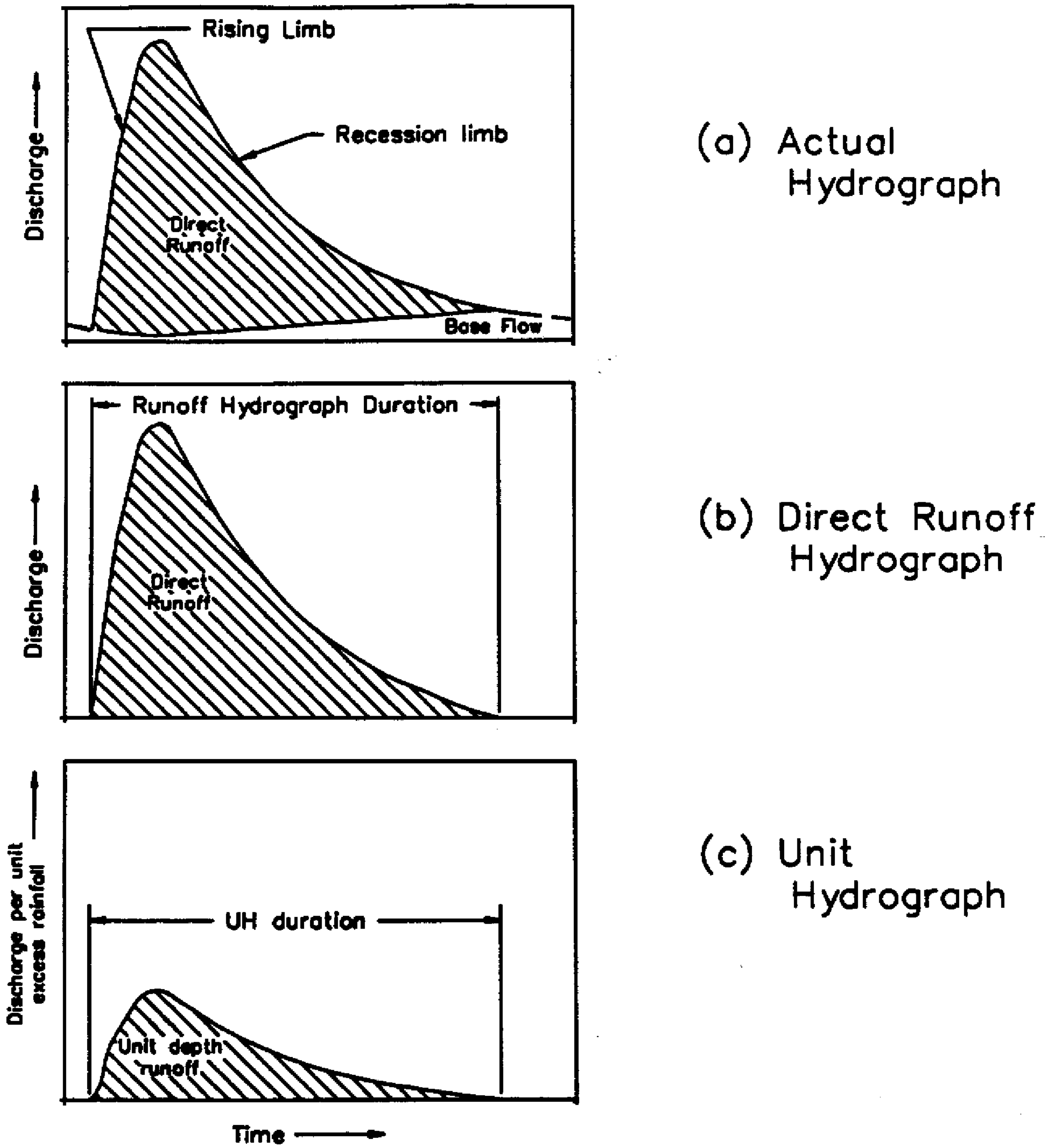

Figure 5.1. Development of a "real" unit hydrograph.

an example of a hydrograph with various parts identified. The area under the hydrograph curve represents the volume which has passed the outflow point. It can be broken into two parts: direct runoff and base flow. Direct runoff is the part of the outflow attributed to the storm, and is the main topic here. The remainder is termed base flow. This is the "constant" outflow of the basin, fed by groundwater. Base flow will not be discussed here, but further information can be found in Linsley, et. al. (1982).

The dimension and shape of a hydrograph depends on a multitude of basin and storm characteristics. Hydrograph shape will change depending on the precipitation pattern and basin shape, for example, as shown in Figure 5.2.

A *unit hydrograph* (UH) is defined by Huggins and Burney, (1982) as "the hydrograph of direct runoff resulting from one [unit] of excess rainfall falling uniformly over the basin at a constant rate during a specified period of time *(excess rainfall duration)*." It is meant to represent a characteristic basin response to a unit of rainfall input.

"Real" UH's are constructed from streamflow and precipitation data gathered within a particular watershed. In the process, a direct runoff hydrograph is produced by subtracting the base flow from the actual hydrograph (see Figure 5.1(b)). All points along the direct runoff hydrograph are divided by the depth of excess rainfall (direct runoff volume divided by the basin area). An example of the process is shown in Figure 5.1. Typically, several storms of a certain short duration are examined in this way, and the resulting UH's are averaged. Once it is determined, the UH can be used to predict basin response to storms of different duration and excess rainfall depth. This is explained further in Section 5.3.

The development and subsequent use of a UH is governed by several assumptions about storms and basin response to storm events:

1. *Excess rainfall is uniformly distributed over the basin area.* As can be seen in Figure 5.2, the spatial distribution of rainfall can have a dramatic effect on the resulting outflow hydrograph. Application of UH's is then limited to smaller areas, where a uniform distribution would be a more accurate assumption.

2. *During the rainfall duration chosen for the unit hydrograph, rainfall intensity is constant.* Although not usually the case, this assumption can be nearly true if a short excess rainfall duration is chosen.

3. *The runoff rate, at any time within the runoff hydrograph duration, is directly proportional to the total volume of excess rainfall.* This is generally true of most basins, provided the rainfall intensity is uniform. Simply put, it means that if the excess rainfall depth is doubled, the discharge rate will be doubled at all points along the resulting runoff hydrograph.

4. *For any volume of excess rainfall occurring within the specified duration, the resulting runoff hydrograph has the same duration.* This means that if 2 units of excess rainfall occur within the specified duration, the runoff hydrograph has the same duration as if only 1 unit had occurred.

5. *The unit hydrograph is invariant from storm to storm and during the storm to which it is applied.* The unit hydrograph lumps many different basin and storm characteristics together. The UH is the characteristic response of the basin,

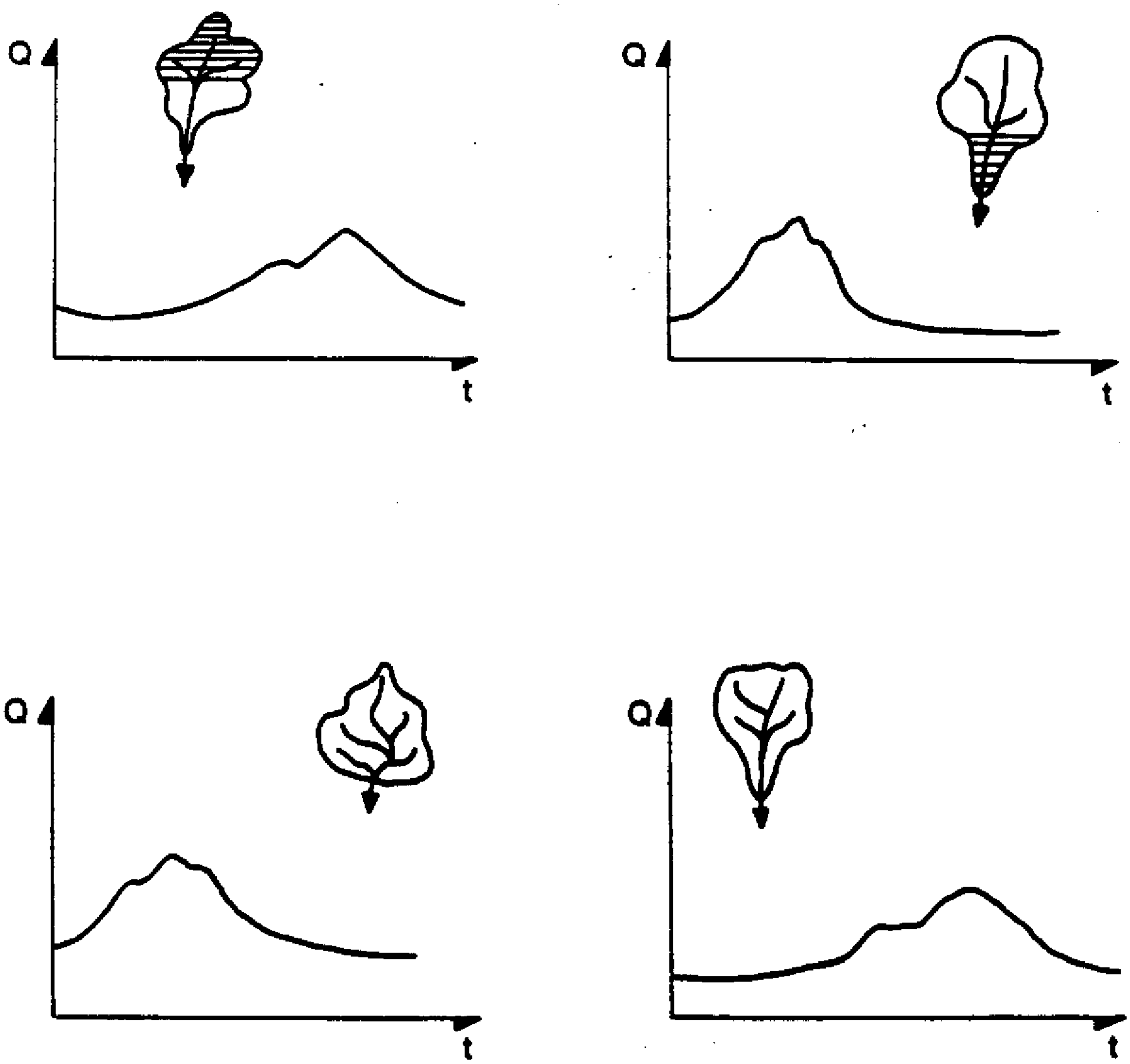

Figure 5.2. Changes in hydrograph shape brought about by different basin and rainfall configurations. The shaded areas represent the portion of the basin where rainfall occurred. (from Viessman, et. al., 1977)

and the basin is assumed to respond in a like manner, regardless of the immediate conditions. This very seldom reflects the real situation, since rainfall distributions, for example, can vary from storm to storm. Assuming an invariant UH is simply a practical consideration. Otherwise, an infinite number of unit hydrographs would be required to account for variable conditions before and during a storm.

Assumptions 3 and 4 are the basis for application of unit hydrographs. They allow superposition, or addition, of a series of UH's to form a composite hydrograph. Composite hydrographs are used to describe the runoff response to storms of varying intensity and different duration than that of the derived UH. This process is discussed in section 5.3.

5.2. Synthetic Unit Hydrographs

When watersheds are ungaged, "real" unit hydrographs cannot be produced. This is the case with many small watersheds. However, there are procedures by which a "synthetic" unit hydrograph can be produced for a basin. The synthetic UH is a generalized unit hydrograph utilizing adjustable parameters which enable it to be used for many different watersheds. They are applied in a manner similar to other UH's and rely on the assumptions given in Section 5.1.

There are several synthetic UH's available in the literature. The reader should refer to Linsley, et. al. (1982); Huggins and Burney (1982); or Viessman, et. al. (1977) for further information. This section will concentrate on methods developed by the Soil Conservation Service (SCS), and two methods used by the SFWMD.

5.2.1. SCS SYNTHETIC UNIT HYDROGRAPHS

The SCS has studied a large number of watersheds under various conditions and in different geographic locations. The unit runoff hydrographs from these locations were normalized and averaged. The resultant average was the Dimensionless Curvilinear unit hydrograph (DCUH). The term "dimensionless" means that each of the discharge rates along the UH have been divided by the peak discharge, and each of the times have been divided by the time to peak. "Nondimensionalizing" a hydrograph is a means by which a hydrograph can be normalized, i.e. UH's from different locations can be compared.

The second SCS synthetic UH considered here is the Dimensionless Triangular UH (DTUH). This is a simple approximation to the DCUH. The DTUH can be produced quickly and closely approximates the DCUH in some key ways: (1) the total volume under the dimensionless UH is the same, (2) the volume under the rising limb of the UH is the same, and (3) the peak discharge is the same.

5.2.1.1. SCS Dimensionless Curvilinear Unit Hydrograph (DCUH)

The derivation of the DCUH is described in National Engineering Handbook, section 4 (USDA-SCS 1985). The shape of the curvilinear unit hydrograph, and its cumulative mass curve, is shown in Figure 5.3 and Table 5.1 lists the coordinates of each.

Peak discharge is calculated by

$$q_p = 645.33K \frac{AR_o}{t_p} \tag{5.1}$$

where

TABLE 5.1. RATIOS FOR DIMENSIONLESS UNIT HYDROGRAPH AND MASS CURVE. (USDA-SCS, 1986)
This table is only valid for a peak rate factor of 484.

Time Ratio (t/t_p)	Discharge Ratio (q/q_p)	Mass Curve (R/R_o)*	Time Ratio (t/t_p)	Discharge Ratio (q/q_p)	Mass Curve (R/R_o)
0.0	0.000	0.000	1.7	0.460	0.790
0.1	0.030	0.001	1.8	0.390	0.822
0.2	0.100	0.006	1.9	0.330	0.849
0.3	0.190	0.012	2.0	0.280	0.871
0.4	0.310	0.035	2.2	0.207	0.908
0.5	0.470	0.065	2.4	0.147	0.934
0.6	0.660	0.107	2.6	0.107	0.953
0.7	0.820	0.163	2.8	0.077	0.967
0.8	0.930	0.228	3.0	0.055	0.977
0.9	0.990	0.300	3.2	0.040	0.984
1.0	1.000	0.375	3.4	0.029	0.989
1.1	0.990	0.450	3.6	0.021	0.993
1.2	0.930	0.522	3.8	0.015	0.995
1.3	0.860	0.589	4.0	0.011	0.997
1.4	0.780	0.650	4.5	0.005	0.999
1.5	0.680	0.700	5.0	0.000	1.000
1.6	0.560	0.751			

* R = accumulated runoff depth at the time ratio t/t_p

q_p = peak discharge, in cfs;

K = constant;

A = drainage area, in square miles;

R_o = total runoff depth, in inches; and

t_p = time to peak discharge, in hours.

R_o is calculated by the Curve Number method (section 4.1), or set equal to 1 to create a unit hydrograph. The coefficient 645.33 converts discharge units from square-mile-inch per hour to cubic feet per second. K represents the reciprocal of the dimensionless area under the DCUH curve. Usually, K and the conversion constant are lumped together, so that equation (5.1) can be written as

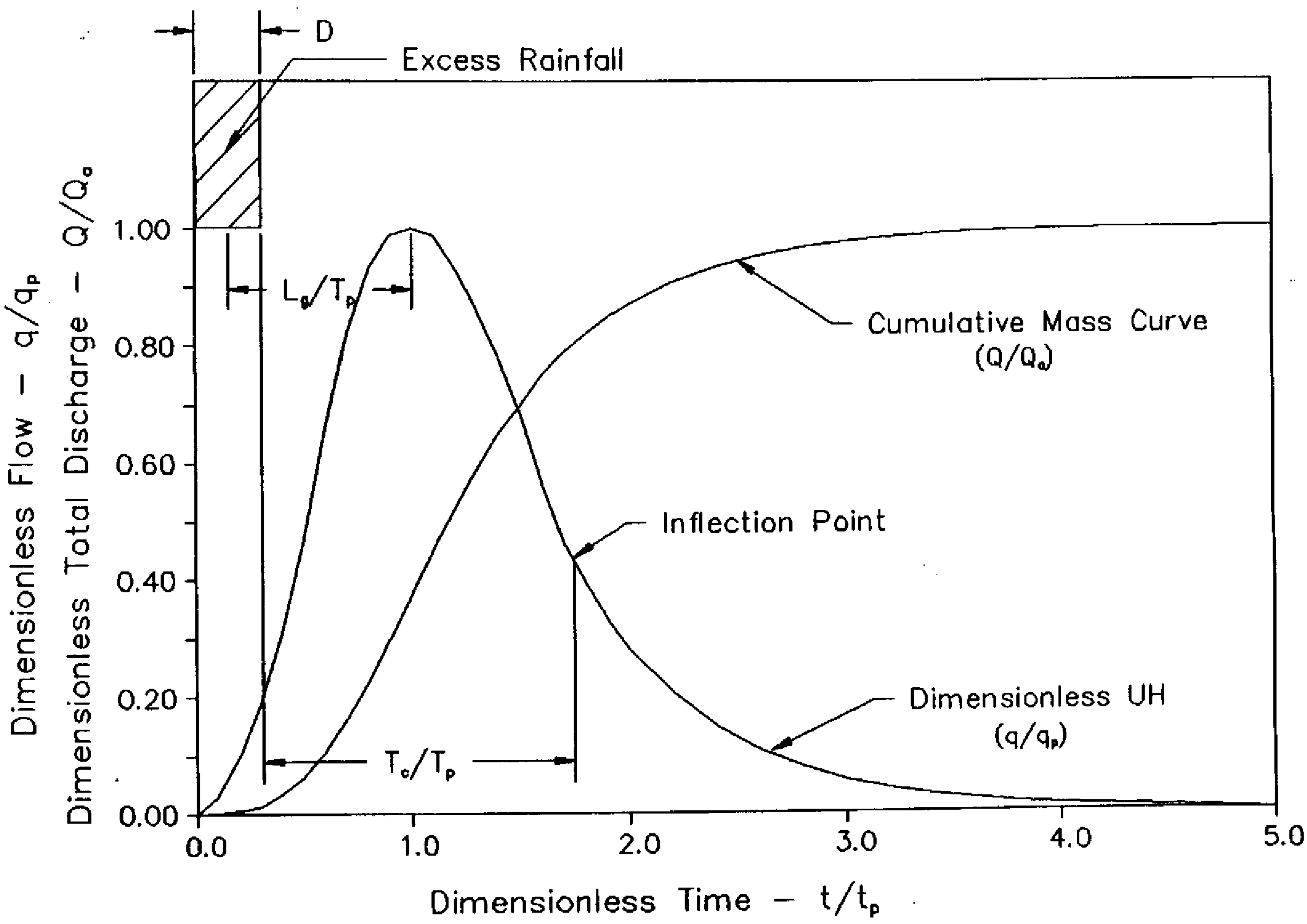

Figure 5.3. The SCS Dimensionless Curvilinear Unit Hydrograph. (USDA-SCS, 1985) The curve given is for a peak rate factor of 484 only.

$$q_p = \beta \frac{AR_o}{t_p} \tag{5.2}$$

where β is termed the *peak rate factor*. For the curve shown in Figure 5.3, K = 0.75 (dimensionless area = 1.33), so that β = 484.

From the relationships shown in Figure 5.3, time to peak, t_p, is calculated by

$$t_p = \frac{D}{2} + L_g \tag{5.3}$$

where

L_g = lag time, in hours; and

D = excess rainfall duration, in hours.

The SCS relates lag time to time of concentration, T_c, by

$$L_g = 0.6T_c \tag{5.4}$$

So, equation (5.3) can then be written as

$$t_p = \frac{D}{2} + 0.6T_c \tag{5.5}$$

where T_c is calculated by the methods presented in Section 2.

The curve in Figure 5.3 has an inflection point at about $t/t_p = 1.7$. From the definition of T_c in Figure 5.3, we see that

$$T_c + D = 1.7t_p \tag{5.6}$$

This relation constrains the selection of D for the UH. Combining equations (5.5) and (5.6) and solving for D, we find that

$$D = 0.2t_p \tag{5.7}$$

or,

$$D = 0.133T_c \tag{5.8}$$

These two relationships should be considered maximums for the chosen D. According to the SCS (USDA-SCS, 1985) some variation is allowed, but

$$D \leq 0.25t_p \tag{5.9}$$

defines the absolute maximum. In practice, D is usually chosen much smaller than that given by equation (5.7) or (5.8). Serious errors in composite hydrograph peak timing can occur if D is chosen too large. The reader should refer to NEH-4 (USDA-SCS, 1985) for a more complete discussion of these errors.

The DCUH was developed using data from a wide variety of natural watersheds. It is probably accurate for average natural conditions, but considerable error can be expected when it is applied to special cases, such as south Florida watersheds. For example, β has been known to vary from 600 in steep terrain to less than 300 in flat, swampy country (USDA-SCS, 1969, pg 16.7). The SCS recommends (USDA-SCS, 1986) that a peak rate factor of 284 be used for all cases in which the average land slope is less than 0.5 percent. Capece (1986) measured peak rate factors ranging from 75 to 100 for minor runoff events in several South Florida flatwoods watersheds. The reader must be aware that if a β other than 484 is chosen, the DCUH shown in Figure 5.3 and Table 5.1 is no longer valid.

5.2.1.2. SCS Dimensionless Triangular Unit Hydrograph (DTUH)

The DTUH is a very simple approximation of the DCUH. A straight line is drawn from the origin ($t/t_p = 0$, $q/q_p = 0$) to the peak ($t/t_p = 1$, $q/q_p = 1$), and a second straight line is drawn from the peak to the end of the recession ($t/t_p = 2.67$, $q/q_p = 0$).

Two key features of the DCUH remain in the DTUH. One, the total *dimensionless* area under the curve is the same, that is K = 0.75. Secondly, 37.5 percent of the total runoff (Q) occurs under the rising limb. Figure 5.4 shows the DTUH and its key dimensions. For comparison, the more realistic Dimensionless Curvilinear UH (DCUH) is shown as a dashed line.

Peak discharge, q_p, and time to peak, t_p, are calculated as with the DCUH, by equations (5.1) and (5.3), or (5.5), respectively. The end of the recession line, i.e. t_b, can be located by utilizing the constraints on the DTUH. Namely, the dimensionless area under the curve, 1/K, is the same as the DCUH. By simple geometry,

$$\frac{1}{K} = \frac{1}{2}\left(\frac{t_b}{t_p}\right) = 1.33$$

Solving for t_b,

$$t_b = 2.67 t_p \tag{5.10}$$

Also since $t_b = t_r + t_p$, we see immediately that

$$t_r = 1.67 t_p \tag{5.11}$$

The triangular unit hydrograph is produced very quickly with relatively small loss of runoff rate information. In the previous section, it was noted that the DCUH,

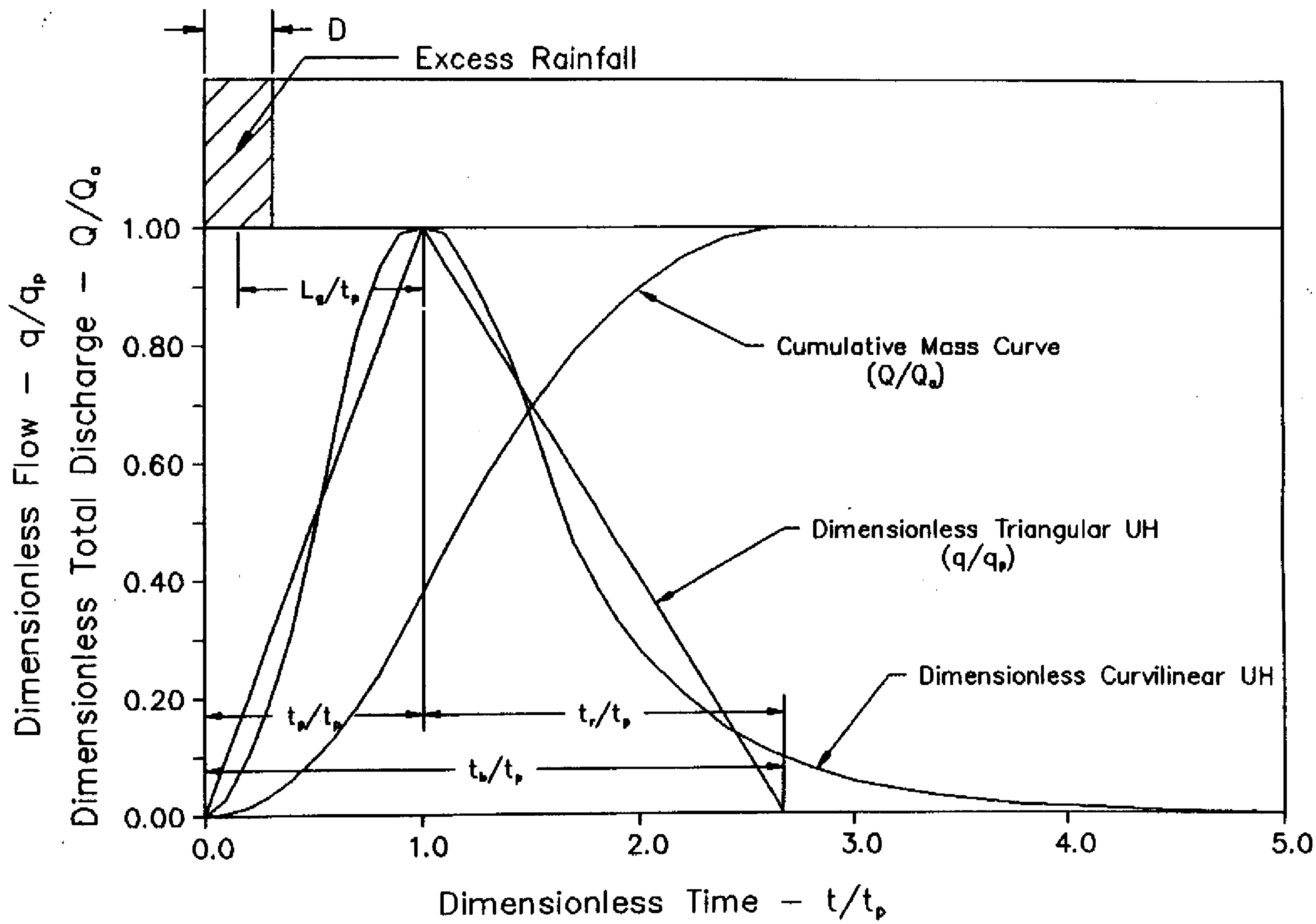

Figure 5.4. The SCS Triangular unit hydrograph, and its cumulative mass curve, shown in dimensionless terms.

as presented, is not applicable to most South Florida situations. The same applies with the DTUH. Equations (5.10) and (5.11) apply only with a peak rate factor of 484. The SCS recommends (USDA-SCS, 1986) that a peak rate factor of 284 be used for all cases in which the average land slope is less than 0.5 percent. If a β other than 484 is chosen, equations (5.10) and (5.11) are no longer valid. The DTUH, unlike the DCUH, can be easily derived for other β's, however. An example of Triangular Unit Hydrograph calculations is shown in Example 5.1.

Example 5.1. SCS Unit Hydrographs. Determine the triangular and dimensionless unit hydrograph for the following watershed:

Total area	120 acres
flat woodland	40 acres
flat pasture	80 acres
Travel Path length	3500 feet
Slope	0.01 ft/ft
Soil	Group B

Determine the time of concentration by the Modified CN method (Section 2.7), although any applicable method may be used. From Table 4.3, extract some CN values and use them to calculated a composite CN:

flatwoods	CN = 55
flat pasture	CN = 61

$$CN = [55(40) + 61(80)]/120 = 59$$

so,

$$S = 1000/59 - 10 = 6.95 \text{ inches}$$

From Figure 2.3, $L_g = 1.54$ hours, and

$$T_c = 1.54/0.6 = 2.57 \text{ hours}$$

Choose a duration of excess rainfall which is convenient yet in accordance with equation (5.8), and the condition that $D < 0.25t_p$:

$$\begin{aligned} D &= 0.133(2.57 \text{ hours}) \\ &= 0.33 \text{ hours} \end{aligned}$$

Time to peak discharge is calculated by equation (5.3):

$$\begin{aligned} t_p &= 0.33/2 + 1.54 \\ &= 1.705 \text{ hours} \end{aligned}$$

Compute the peak discharge by equation (5.2):

$$\begin{aligned} q_p &= 484(120 \text{ acres})(\text{square-mile}/640 \text{ acres})(1 \text{ inch})/(1.705 \text{ hours}) \\ &= 53.23 \text{ cfs} \end{aligned}$$

The hydrograph base time will be needed for the triangular hydrograph, so, by equation (5.10):

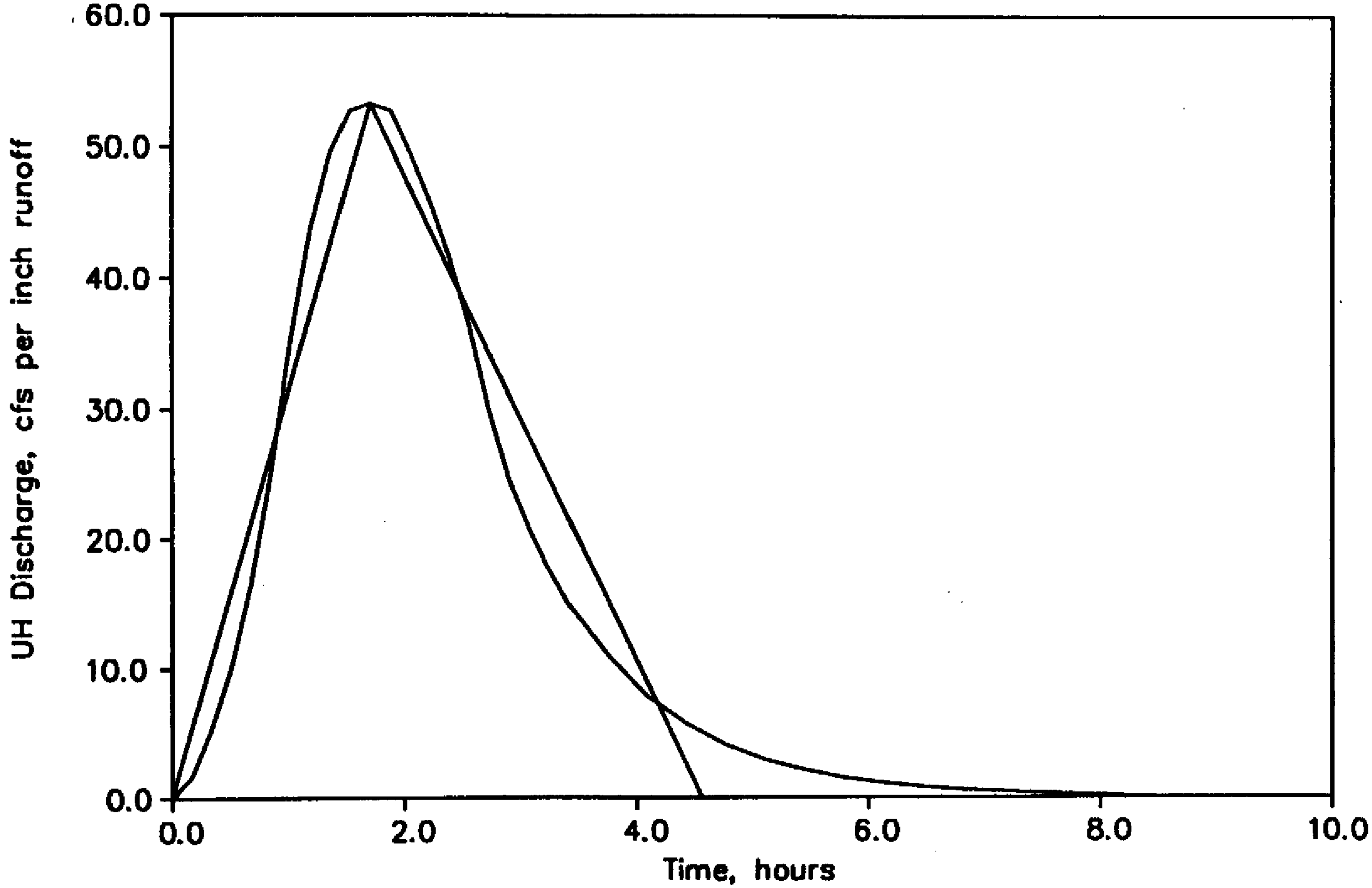

Figure 5.5. Curvilinear and Triangular UH's calculated in Example 5.1.

$$t_b = 2.67(1.705 \text{ hours})$$
$$= 4.57 \text{ hours}$$

The resulting triangular unit hydrograph is shown in Figure 5.5. Coordinate calculations for the curvilinear UH are completed in Table 5.2. The Curvilinear UH is also shown in Figure 5.5.

TABLE 5.2. COMPUTATION OF UH COORDINATES FOR EXAMPLE 5.1. (SCS) Values for columns (1) and (3) are taken from Table 5.1, for a peak rate factor of 484.

(1) t/t_p	(2) t (hrs) (1)*1.705	(3) q/q_p	(4) q (cfs) (3)*53.23	(1) t/t_p	(2) t (hrs) (1)*1.705	(3) q/q_p	(4) q (cfs) (3)*53.23
0	0	0	0	1.7	2.90	0.46	24.5
.1	0.17	0.03	1.6	1.8	3.07	0.39	20.8
.2	0.34	0.10	5.3	1.9	3.24	0.33	17.6
.3	0.51	0.19	10.1	2.0	3.41	0.28	14.9
.4	0.68	0.31	16.5	2.2	3.75	0.21	11.0
.5	0.85	0.47	25.0	2.4	4.09	0.15	7.8
.6	1.02	0.66	35.1	2.6	4.43	0.11	5.7
.7	1.19	0.82	43.6	2.8	4.77	0.08	4.1
.8	1.36	0.93	49.5	3.0	5.12	0.06	2.9
.9	1.53	0.99	52.7	3.2	5.46	0.04	2.1
1.0	1.70	1.00	53.2	3.4	5.80	0.03	1.5
1.1	1.88	0.99	52.7	3.6	6.14	0.02	1.1
1.2	2.05	0.93	49.5	3.8	6.48	0.015	0.8
1.3	2.22	0.86	45.8	4.0	6.82	0.011	0.6
1.4	2.39	0.78	41.5	4.5	7.67	0.005	0.3
1.5	2.56	0.68	36.2	5.0	8.50	0	0
1.6	2.73	0.56	29.8				

5.2.2. GENERAL DIMENSIONLESS CURVILINEAR UNIT HYDROGRAPH (GDCUH)

For most south Florida applications, the peak rate factor of equation (5.2), should be less than 484. The SCS recommends (USDA-SCS, 1980) that a peak rate factor of 284 be used for all cases in which the average land slope is less than 0.5 percent. Since the shape of the Dimensionless Curvilinear UH depends on the dimensionless area, and therefore the peak rate factor, the curve for a $\beta = 484$ (Figure 5.3) is not valid for any other β. Failure to use the correct dimensionless unit hydrograph will result in an incorrect computation of the runoff peak and duration.

C.J. Neidrauer, District Staff Water Resources engineer, has developed a simple equation to compute General Dimensionless Curvilinear Unit Hydrographs (GDCUH). This equation can be expressed as follows :

$$\frac{q}{q_p} = \left\{ \frac{t}{t_p} exp\left[1 - \frac{t}{t_p}\right]\right\}^c \tag{5.12}$$

where q is the discharge at time t. The peak discharge and time to peak are calculated by equations (5.1) and (5.3), respectively. The exponent, c, is determined from the following equation:

$$c = a_1\left(\frac{1}{x}\right) + a_2\left(\frac{1}{x}\right)^2 + a_3\left(\frac{1}{x}\right)^3 + a_4\left(\frac{1}{x}\right)^4 \tag{5.13}$$

where

$x = 645.33/\beta$

$= $ dimensionless area under the DUH $= 1/K$

$a_1 = 0.9684729$

$a_2 = 3.9895040$

$a_3 = 2.4688720$

$a_4 = -0.9946742$

Figure 5.6 shows the relationship of the peak rate factor to the parameter, c. Figure 5.7 shows graphs of equation (5.12) for several choices of β. The point of inflection on the GDCUH is calculated as

$$t_i = \left\{1 + \frac{1}{c^{0.5}}\right\} t_p \tag{5.14}$$

With time of concentration defined as with the DCUH, we see that

$$T_c + D = \left\{1 + \frac{1}{c^{0.5}}\right\} t_p \tag{5.15}$$

Constraints on the selection of D are a bit more ambiguous than with the SCS dimensionless UH. If relations such as equations (5.7) and (5.8) are derived for the GDCUH (using equation (5.14)), some probably unreasonable maximums for D are calculated. For instance, if a peak rate factor of 284 is used, a maximum for D is calculated at $1.13t_p$. Since there is no evidence to show otherwise, the SCS recommendation given by equation (5.9) probably holds, and should be followed. In most cases, however, D should be chosen considerably smaller than the SCS recommendations.

Although the GDCUH has not been tested as yet, it does produce a very close approximation of the SCS DCUH for β = 484. The SCS provides no means of producing a Curvilinear UH with a variable β. If the SCS recommendations are to be followed, the GDCUH may provide the best means.

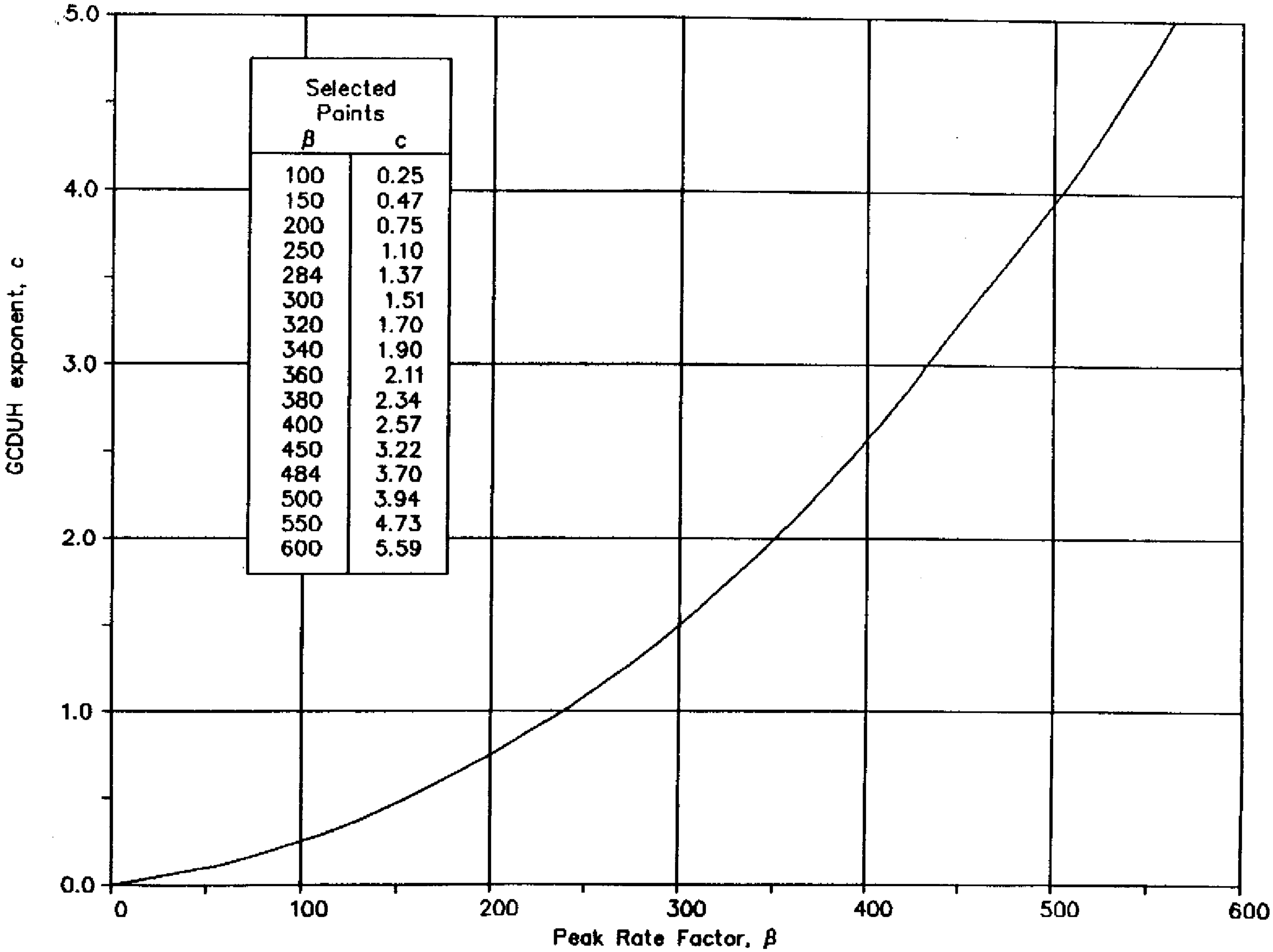

Figure 5.6. The relationship between peak rate factor, β, and the GCDUH exponent, c.

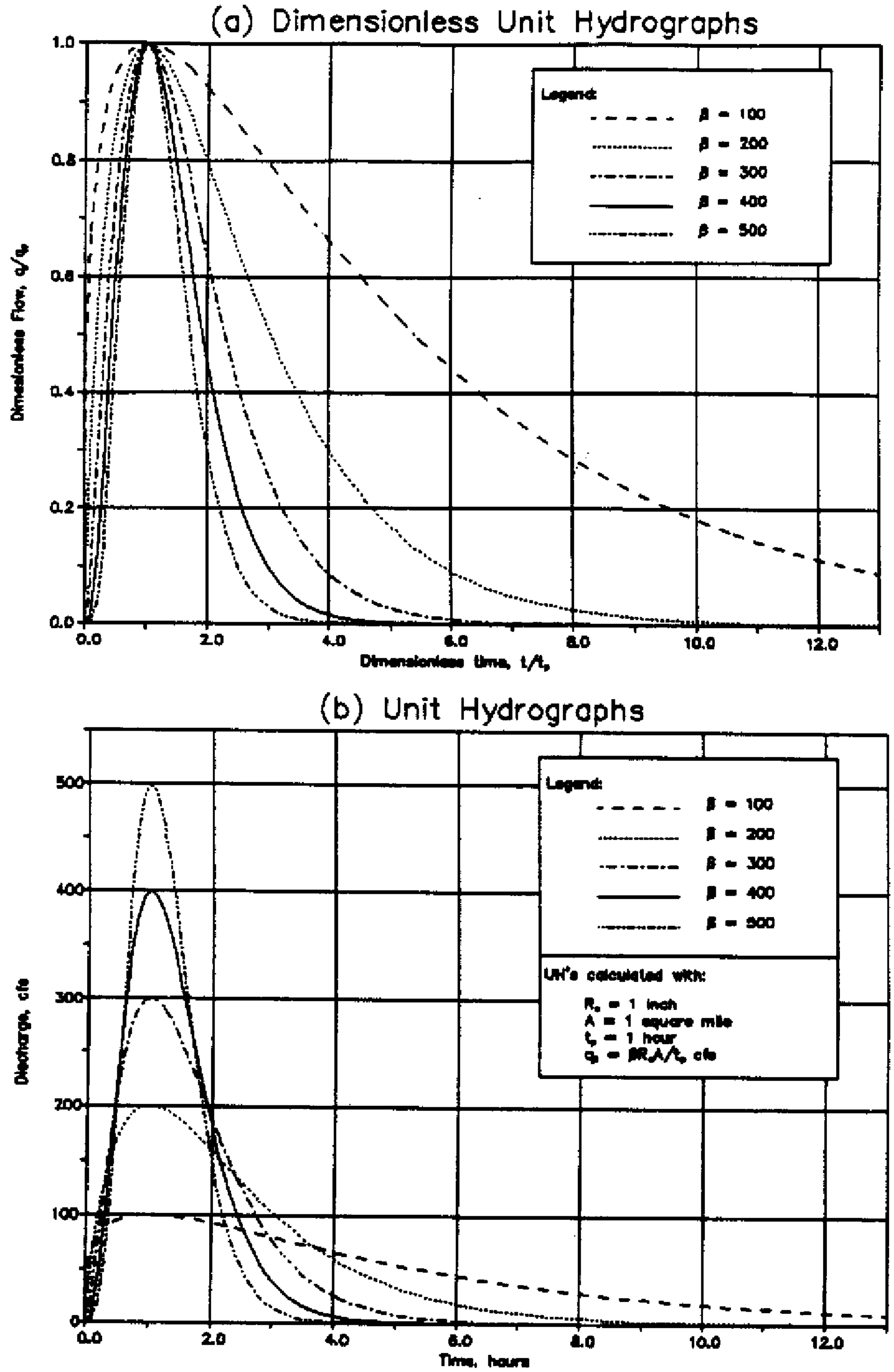

Figure 5.7. Graphs of (a) the General Dimensionless Curvilinear UH, and (b) their corresponding UH's for several choices of peak rate factor, β.

Example 5.2: The General Dimensionless Unit Hydrograph Based on Peak Rate Factor of 284. Assume the basin used in Example 5.1 has a slope of 0.001 ft/ft. Determine a unit hydrograph for that basin. According to SCS recommendations, with a slope of 0.001 ft/ft, a peak rate factor of 284 should be used.

Recalculate time of concentration based on the new slope. First, by the modified CN method (section 2.7):

CN = 59

S = 6.95 inches, as before

Using equation (2.6), lag time is

$$L_g = \frac{(3500\ ft)^{0.8}(6.95\ in + 1)^{0.7}}{1900(0.1\%)^{0.5}} = 4.9\ hours$$

then solving equation (5.5) for T_C,

T_C = 4.9/0.6 = 8.2 hours

Calculate time of concentration assuming shallow concentrated flow, and using equation (2.4):

$$V = \frac{1.49}{n} R^{\frac{2}{3}} S_o^{\frac{1}{2}}$$

From Table 2.2 n = 0.35, and assuming R = 0.4 ft

$$V = \frac{1.49}{0.35}(0.4)^{\frac{2}{3}}(0.001)^{\frac{1}{2}}$$

$= 0.073$ *feet per second*

so by equation (2.2),

$$T_c = 3500/[3600(0.073)] = 13.3 \text{ hours}$$

Choose T_c = 13.0 hours. Choose a excess rainfall duration within the limit of equation (5.8).

$$D = 0.5 \text{ hours will be used.}$$

With a peak rate factor of 284, c = 1.37, from Figure 5.7. The point of inflection is given by equation (5.14):

$$\frac{t_i}{t_p} = 1 + \left(\frac{1}{1.37^{0.5}}\right) = 1.85 \textit{ at inflection}$$

Time to peak can then be calculated by equation (5.15):

$$t_p = (T_c + D)/1.85$$
$$= 7.30 \text{ hours}$$

The chosen rainfall duration (increment) should be less than 0.25 t_p, and we see that it is ($0.5 < 0.25*7.30 = 1.83$). Finally, the peak discharge is calculated by equation (5.1):

$$q_p = (284)(120 \text{ acres})(\text{square-mile}/640 \text{ acres})(1 \text{ inch})/(7.30 \text{ hours})$$
$$= 7.29 \text{ cfs}$$

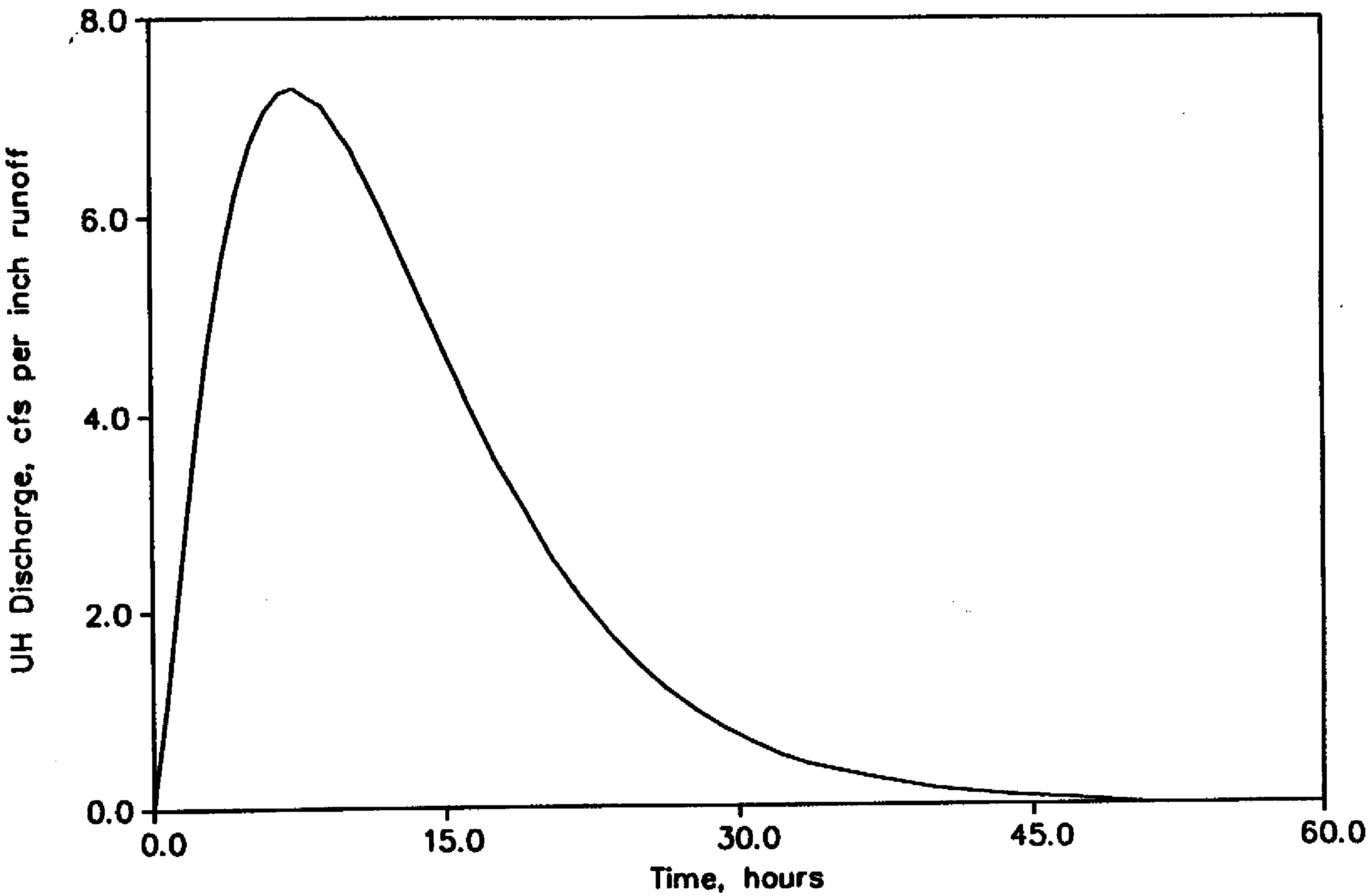

Figure 5.8. Unit hydrograph calculated in Example 5.2.

TABLE 5.3. COMPUTATION OF UH COORDINATES FOR EXAMPLE 5.2 (GDCUH). Column (3) was calculated with equation (5.12) for the given time ratios, and a peak rate factor of 284.

(1) t/t_p	(2) t (hrs) (1)*7.30	(3) q/q_p	(4) q (cfs) (3)*7.29	(1) t/t_p	(2) t (hrs) (1)*7.30	(3) q/q_p	(4) q (cfs) (3)*7.29
0	0	0	0	2.0	14.60	0.657	4.79
0.1	0.73	0.146	1.06	2.2	16.06	0.572	4.17
0.2	1.46	0.330	2.41	2.4	17.52	0.487	3.55
0.3	2.19	0.501	3.65	2.6	18.98	0.418	3.05
0.4	2.92	0.648	4.72	2.8	20.44	0.348	2.54
0.5	3.65	0.768	5.60	3.0	21.90	0.291	2.12
0.6	4.38	0.859	6.26	3.2	23.36	0.242	1.76
0.7	5.11	0.925	6.74	3.4	24.82	0.200	1.46
0.8	5.84	0.969	7.06	3.6	26.28	0.164	1.20
0.9	6.57	0.993	7.24	3.8	27.74	0.134	0.98
1.0	7.30	1.000	7.29	4.0	29.20	0.110	0.80
1.1	8.03	0.988	7.20	4.2	30.66	0.089	0.65
1.2	8.76	0.976	7.12	4.4	32.12	0.072	0.52
1.3	9.49	0.947	6.90	4.6	33.58	0.058	0.42
1.4	10.22	0.917	6.68	5.0	36.50	0.038	0.28
1.5	10.95	0.877	6.39	5.5	40.15	0.022	0.16
1.6	11.68	0.837	6.10	6.0	43.80	0.012	0.09
1.7	12.41	0.792	5.77	6.5	47.45	0.07	0.05
1.8	13.14	0.747	5.45	7.0	51.10	0.0	0
1.9	13.87	0.702	5.12				

Calculation of the coordinates for the resulting UH are completed in Table 5.3, and are graphed in Figure 5.8.

5.2.3. TRACOR SYNTHETIC UNIT HYDROGRAPH

Another synthetic UH which receives some use in South Florida was developed by Tracor, Incorporated of Austin, Texas under contract with the Office of Water Resources Research, U.S. Department of the Interior (Tracor, 1968). The Tracor UH is entirely empirical and was developed for watersheds throughout the U.S.

The shape of the Tracor UH is based on five parameters, which locate points along the UH. They are

1. t_p - the time to peak discharge, in minutes;
2. q_p - the peak discharge, in cfs per inch runoff;
3. t_{50}- the time, in minutes, between the point on the rising limb where discharge is 50 percent of q_p and the point on the falling limb where discharge is 50 percent of q_p;
4. t_{75}- the time, in minutes, between the point on the rising limb where discharge is 75 percent of q_p and the point on the falling limb where discharge is 75 percent of q_p; and
5. t_b - the base time, in minutes, which is the time from the start of runoff and the effective end of runoff.

These are shown graphically in Figure 5.9. The SFWMD has adjusted and adapted the Tracor UH procedure and parameter estimation equations for local use. The description presented here is limited to this modified Tracor procedure. The modified Tracor UH is only for a <u>30-minute excess rainfall duration</u>.

The time to peak for an "urbanized basin" is estimated by

$$t_p = 16.4\Phi L^{0.35} S^{-0.049} I^{-0.45} \tag{5.16}$$

where

L = length of the main channel, in feet;

S = the slope of the main channel, dimensionless;

I = percent of impervious surface area; and

Φ = an urbanization classification factor;

$$= \Phi_1 + \Phi_2 \tag{5.17}$$

where ϕ_1 and ϕ_2 describe the extensiveness of the storm sewer system and the condition of that storm sewer system within the basin, respectively. ϕ_1 and ϕ_2 are assigned values as shown in Table 5.4. Time to peak for a "rural basin" is estimated by an equation slightly different than equation (5.16), specifically,

$$t_p = 3.4L^{0.233}S^{-0.302} \tag{5.18}$$

where L and S have the same definitions as in equation (5.16).

The peak discharge, q_p, can be estimated by any appropriate method; some are discussed in Section 3. Typical application of the Tracor UH by the SFWMD has involved the use of the Cypress Creek Formula (see SFWMD, 1984 and Lin, 1988), which is discussed in Section 3.6. However, other peak discharge estimations could be used, provided they are valid for the basin under consideration.

The parameters t_{50} and t_{75} are estimated by

$$t_{50} = (2.91 \times 10^4)A^{0.959}q_p^{-0.983} \tag{5.19}$$

and

$$t_{75} = (1.15 \times 10^4)A^{0.857}q_p^{-0.915} \tag{5.20}$$

respectively, where A is the basin area in square miles and q_p is in cfs..

The original Tracor procedure calls for the calculation of a base time, t_b. A UH is then sketched through the known points (the start, peak and end), noting the constraints of t_{50} and t_{75}. The starting points of t_{50} and t_{75} (points A and B in Figure

TABLE 5.4a. VALUES ASSIGNED TO ϕ_1 FOR COMPUTATION OF THE TRACOR UH URBANIZATION FACTOR. (Reproduced from SFWMD, 1984b)

ϕ_1	Conditions
0.6	Extensive channel improvement and storm sewer system. Closed conduit channel system.
0.8	Some channel improvement and storm sewers. Mainly cleaning and enlargement of existing channel.
1.0	Natural channel conditions.

TABLE 5.4b. VALUES ASSIGNED TO ϕ_2 FOR COMPUTATION OF THE TRACOR UH URBANIZATION FACTOR. (Reproduced from SFWMD, 1984b)

ϕ_2	Conditions
0.0	No channel vegetation.
0.1	Light channel vegetation.
0.2	Moderate channel vegetation.
0.3	Heavy channel vegetation.

5.9) are moved laterally and the UH resketched until the area under the UH represents 1 inch of runoff. This is a tedious and time consuming process, and will not necessarily yield a proper UH.

The SFWMD's modification of the Tracor procedure assumes a linear rising limb. With this assumption, all coordinates of the UH up to the end of t_{50} can be plotted. The remainder of the UH is estimated by an exponential decay function whose decay coefficient is based on the remaining runoff volume necessary to yield 1 inch of runoff (see Figure 5.10). This procedure insures that the area under the UH

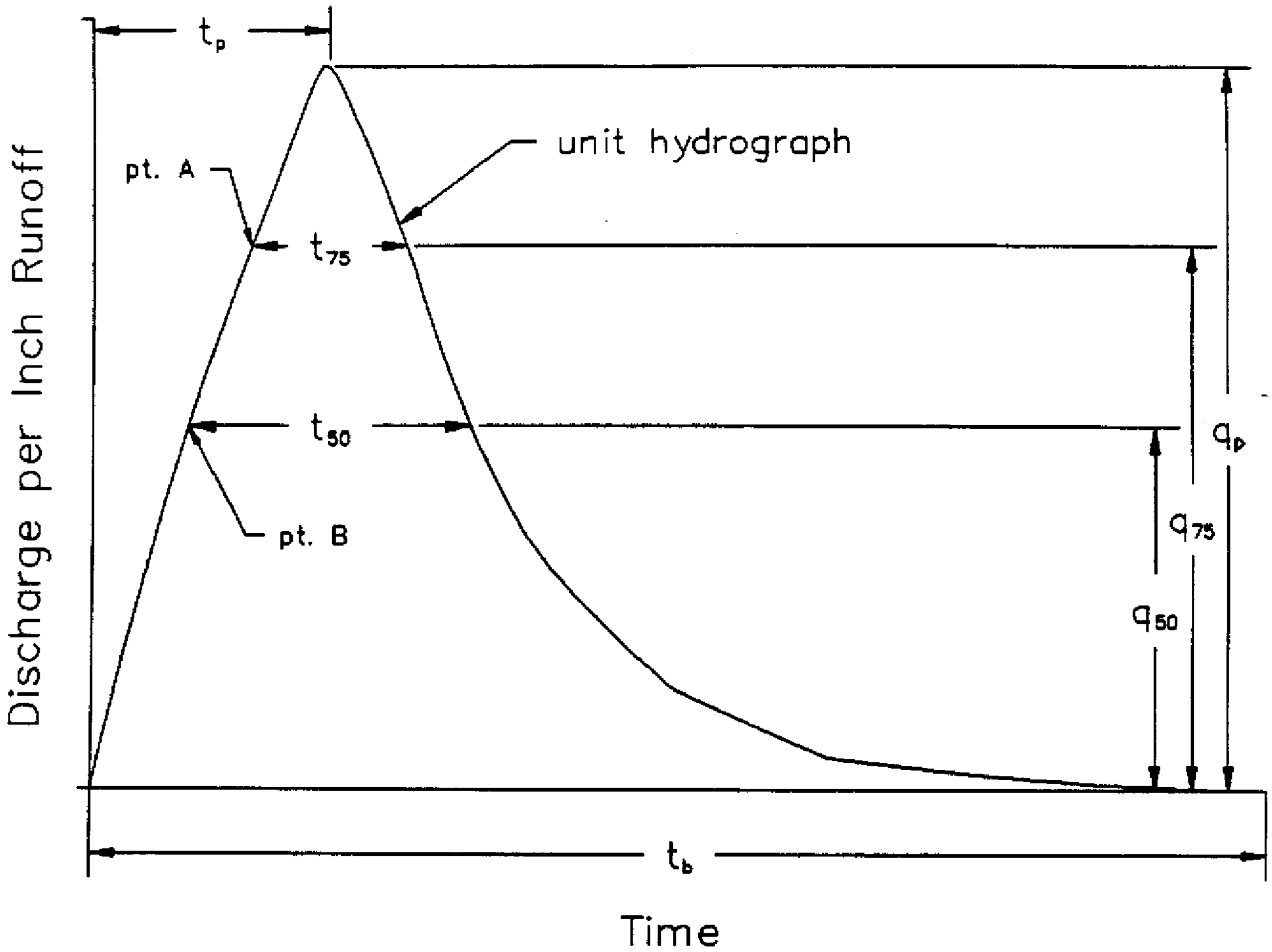

Figure 5.9. Parameters used in the original Tracor synthetic UH procedure. Points A and B are moved laterally to insure 1 inch of runoff under the curve.

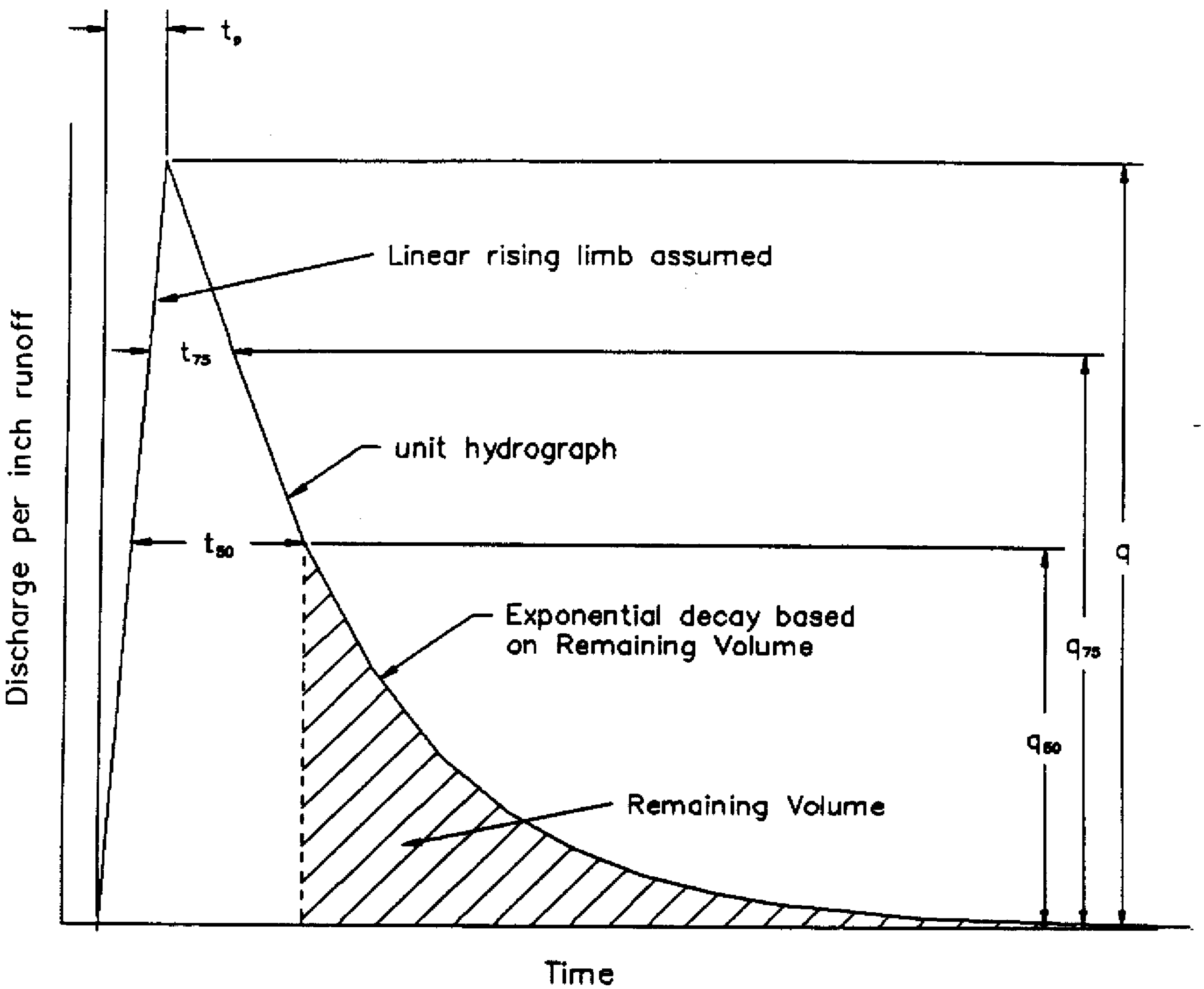

Figure 5.10. Use of parameters in SFWMD's modification of the Tracor procedure. The rising limb is assumed linear, and the falling limb is assumed to exponentially decay.

represents 1 inch of runoff over the basin, and helps to automate production of a UH.

The coefficients used in equations (5.16), (5.19), and (5.20) were derived from comparisons with only two subbasins located within eastern Palm Beach county. Hence, the method has not been extensively verified. The method is further limited by a 30 minute excess rainfall duration. The equations presented assume a 30 minute excess rainfall duration. This may be too large when very small basins are considered. The SCS (Section 5.2.1.1 and Section 5.2.1.2) limitations for selection of excess rainfall duration probably apply here as well.

5.3. Application of Unit Hydrographs

"Real" or synthetic unit hydrographs represent a characteristic response to an excess rainfall event of specific duration and unit depth. When a particular storm is under consideration, its duration and depth is usually different from that chosen for the development of the UH. In order to apply a unit hydrograph in this situation, the storm's excess rainfall distribution is divided into short time intervals. The lengths of which are equal to the excess rainfall duration used to develop the UH. The basin's response to each interval of excess rainfall is then calculated independently. The resulting "incremental hydrographs" are then summed to form a composite hydrograph of basin outflow.

Figure 5.11 shows this process graphically. In more specific terms, a composite hydrograph is produced by the following procedure:

1. Develop a unit hydrograph (real or synthetic) for the basin as described in Sections 5.1 and 5.2.

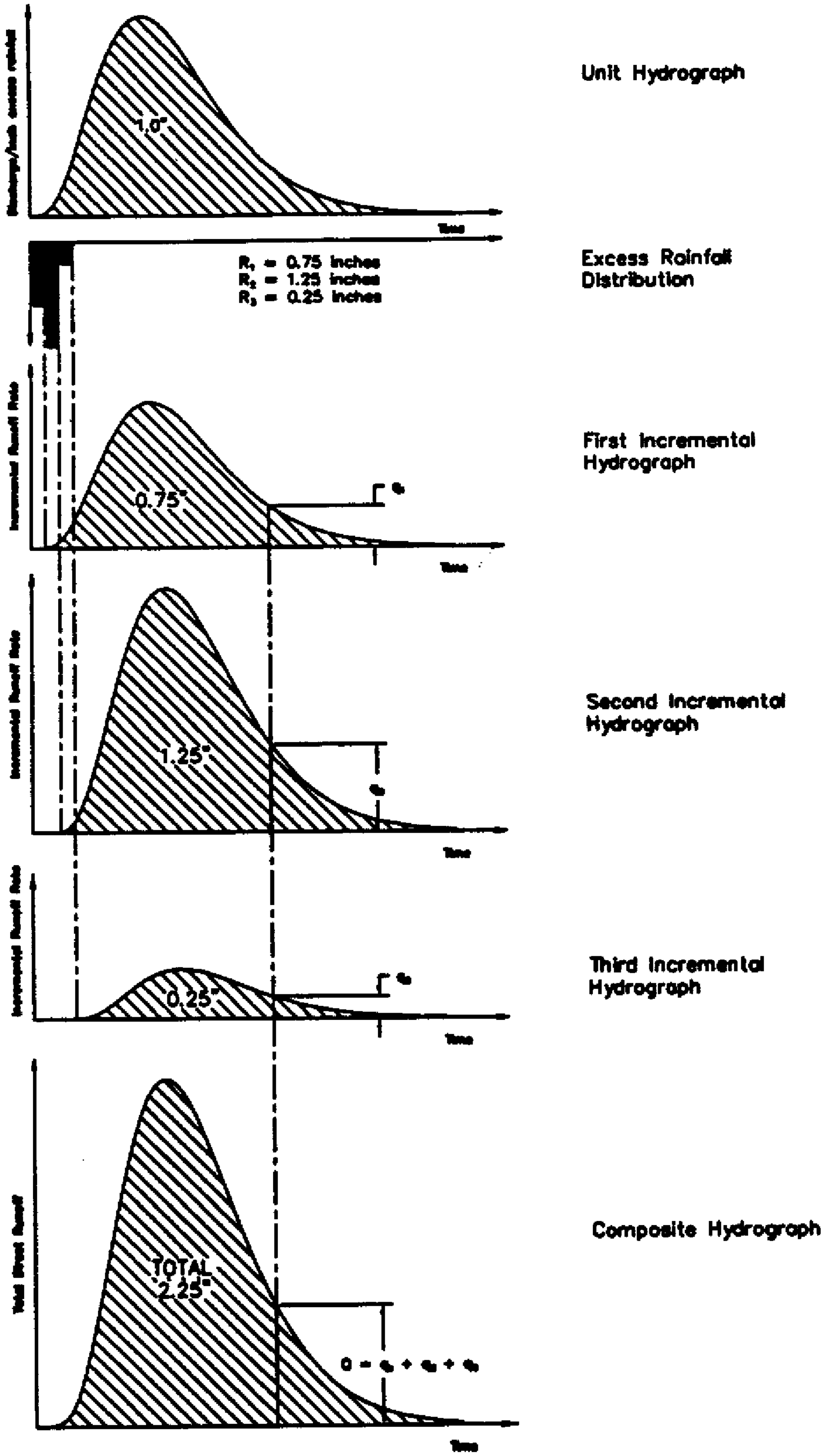

Figure 5.11. Graphical illustration of the process by which unit hydrographs are combined to produce a composite hydrograph.

2. Divide the actual storm rainfall distribution of interest into time intervals which are equal to the UH's excess rainfall duration, D, chosen in step 1.

3. For each time interval, compute direct runoff (effective rainfall) by subtracting the estimated rate of losses, estimated by a suitable method (e.g. Curve Number method, Section 4.1), from the total rainfall. The result for each interval is termed *incremental effective rainfall.*

4. Compute the watershed runoff hydrograph:

   a. Multiply each ordinate of the UH by the incremental effective rainfall for the first time period. The result is termed the incremental runoff hydrograph, and represents the basin response corresponding to the first interval of the storm only.
   b. Repeat step (a) for each of the time periods. Each resulting *incremental runoff hydrograph* is advanced by one time period, D.
   c. Sum the values of each incremental runoff hydrograph to produce the composite runoff hydrograph.
   d. Add base flow, if any, to the resultant flood hydrograph.

Example 5.3 illustrates this procedure.

Computation of a composite hydrograph is based, first, on the assumption that UH's are linearly superimposable, as previously discussed. Secondly, the UH is assumed to be invariant during the storm. This means, effects of changing depression and soil storage, for example, are ignored. Decreases in basin surface and soil storage can actually decrease the basin's lag, and vice versa. As available storage in the basin decreases during the storm, this effect on basin lag is not taken into account by the UH.

Example 5.3: Composite Hydrograph Calculations. This example presents a composite hydrograph computed for the basin described in Example 5.2. The basin is subject to a storm which has the time distribution shown in Table 5.5. The resultant composite hydrograph is quite lengthy, so only a portion is shown in Table 5.6.

To illustrate the process by which the composite hydrograph is computed, calculation of the composite discharge for time t = 3.0 hours follows. Note first, in Table 5.6, that the unit hydrograph from Example 5.2 has been recalculated at intervals of D, 0.5 hours . This is so that the hydrographs may be shown in table form. From Example 5.2,

$D =$ 0.5 hours

$t_p =$ 7.30 hours

$q_p =$ 7.29 cfs

TABLE 5.5. EXCESS RAINFALL DISTRIBUTION USED IN EXAMPLE 5.3.

Period	Time (hours)	Accumulated Excess Rainfall (inches)	Period Excess Rainfall (inches)
1	0.5	0.50	0.50
2	1.0	1.25	0.75
3	1.5	2.25	1.00
4	2.0	3.50	1.25
5	2.5	4.25	0.75
6	3.0	4.50	0.25

TABLE 5.6. CALCULATION OF THE COMPOSITE HYDROGRAPH FOR EXAMPLE 5.3. Note that not all values are included. The unit hydrograph of Example 5.2 was recalculated at 0.5 hour time steps.

Period:	1	2	3	4	5	6
Time, hours:	0.5	1.0	1.5	2.0	2.5	3.0
Period Excess Rainfall:	0.50	0.75	1.00	1.25	0.75	0.25

Time hours	UH[1] cfs/in	IRHG[2] cfs	IRHG cfs	IRHG cfs	IRHG cfs	IRHG cfs	IRHG cfs	Composite HG[3] cfs
0.0	0.00	0.00						0.00
0.5	0.66	0.33	0.00					0.33
1.0	1.56	0.78	0.50	0.00				1.28
1.5	2.48	1.24	1.17	0.66	0.00			3.07
2.0	3.34	1.67	1.86	1.56	0.83	0.00		5.92
2.5	4.13	2.07	2.51	2.48	1.95	0.50	0.00	9.50
3.0	4.81	2.41	3.10	3.34	3.10	1.17	0.17	13.28
3.5	5.43	2.72	3.61	4.13	4.18	1.86	0.39	16.88
4.0	5.94	2.97	4.07	4.81	5.16	2.51	0.62	20.14
4.5	6.35	3.18	4.46	5.43	6.01	3.10	0.84	23.01
5.0	6.68	3.34	4.76	5.94	6.79	3.61	1.03	25.47
6.0	7.11	3.56	5.26	6.88	7.94	4.46	1.36	29.18
7.0	7.28	3.64	5.42	7.11	8.66	5.01	1.59	31.43
8.0	7.25	3.63	5.47	7.28	9.04	5.33	1.73	32.48
9.0	7.06	3.53	5.38	7.25	9.11	5.46	1.81	32.54
10.0	6.76	3.38	5.19	7.06	8.96	5.44	1.82	31.85
15.0	4.61	2.31	3.62	5.06	6.61	4.13	1.44	23.17
20.0	2.67	1.34	2.13	3.01	3.99	2.53	0.89	13.88
25.0	1.42	0.71	1.14	1.62	2.16	1.38	0.49	7.51
30.0	0.71	0.36	0.58	0.82	1.10	0.71	0.25	3.82
35.0	0.34	0.17	0.28	0.40	0.54	0.35	0.13	1.86
40.0	0.16	0.08	0.19	0.19	0.25	0.17	0.06	0.87
50.0	0.03	0.02	0.03	0.04	0.05	0.04	0.01	0.19
60.0	0.01	0.01	0.01	0.01	0.01	0.01	0.00	0.05
61.0	0.01	0.01	0.01	0.01	0.01	0.01	0.00	0.05
61.5	0.01	0.01	0.01	0.01	0.01	0.01	0.00	0.05
62.0	0.00	0.00	0.00	0.01	0.01	0.01	0.00	0.04
62.5			0.00	0.01	0.01	0.01	0.00	0.03
63.0				0.00	0.01	0.01	0.00	0.02
63.5					0.01	0.01	0.00	0.02
64.0						0.01	0.00	0.01
64.5							0.00	0.00

[1]UH = unit hydrograph
[2]IRHG = incremental runoff hydrograph
[3]HG = hydrograph

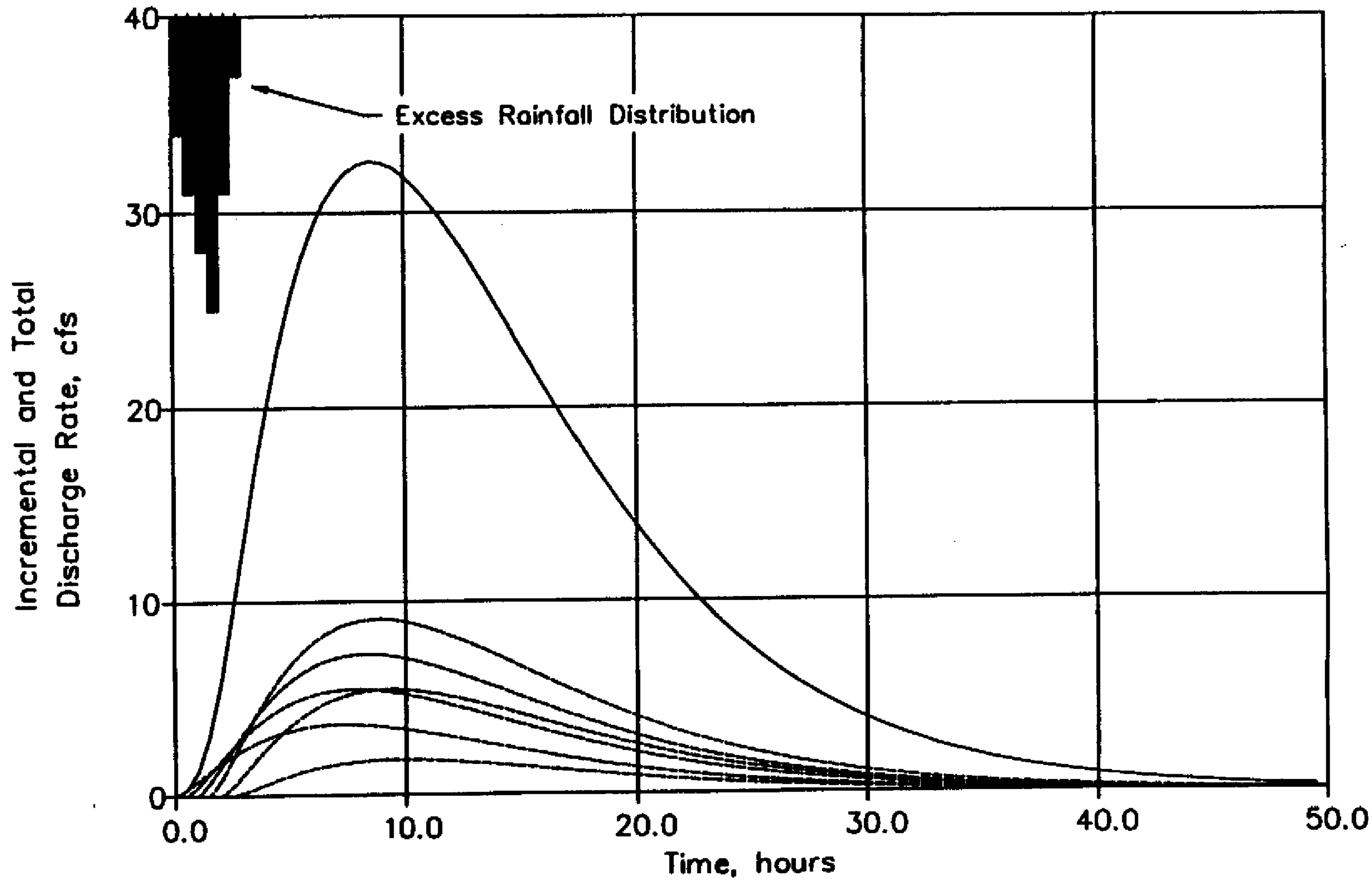

Figure 5.12. Composite hydrograph produced in Example 5.3. The individual incremental hydrographs are shown as dashed lines.

Using equation (5.12), at t = 3.0 hours,

$$UH_{3.0} = \frac{q}{q_p} \cdot q_p = \left\{ \frac{3.0}{7.30} exp\left[1 - \frac{3.0}{7.30}\right]\right\}^{1.37} \cdot (7.29\ cfs)$$

$$= 0.66(7.29\ cfs) = 4.81\ cfs\ per\ inch\ excess\ rainfall$$

Each period's incremental runoff hydrograph (IRHG) is calculated by multiplying the proper ordinate of the UH by the excess rainfall for the period. Each successive IRHG is delayed by the length of the period, D. For period 1 (from t=0 to t=0.5 hr), the excess rainfall was 0.5 inches, and the ordinate of the IRHG for t = 3.0 hr is given by

$$IRHG^1_{3.0} = (0.5\ in)UH_{3.0} = (0.5)(4.81) = 2.41\ cfs$$

Period 2's IRHG is delayed by 0.5 hours. The excess rainfall for period 2 is 0.75 inches. Consequently, the IRHG ordinate at t = 3.0 hours is given by

$$IRHG^2_{3.0} = (0.75\ in)UH_{2.5} = (0.75)(4.13) = 3.10\ cfs$$

So for the remaining periods:

$$IRHG^3_{3.0} = (1.00\ in)UH_{2.0} = (1.00)(3.34) = 3.34\ cfs$$

$$IRHG^4_{3.0} = (1.25\ in)UH_{1.5} = (1.25)(2.48) = 3.10\ cfs$$

$$IRHG^5_{3.0} = (0.75\ in)UH_{1.0} = (0.75)(1.56) = 1.17\ cfs$$

$$IRHG^6_{3.0} = (0.25)UH_{0.5} = (0.25)(0.66) = 0.17\ cfs$$

The ordinate for the total, or composite, hydrograph for t = 3.0 hours is the sum of each of the IRHG ordinates, or 13.28 cfs.

5.4. Santa Barbara Urban Hydrograph Method (SBUH)

The Santa Barbara Urban Hydrograph method was developed by James M. Stubchaer of the University of Kentucky (Stubchaer, 1975). It was originally developed for the Santa Barbara County Flood Control and Water Conservation District in California, from which its name is derived.

In the method, a design rainfall distribution is divided into short intervals, Δt hours long. The watershed is divided into pervious and hydraulically connected impervious portions. Rain falling on or flowing across pervious portions is subjected to infiltration losses, the magnitude of which is dependent upon the antecedent rainfall conditions. For each interval, Δt, a runoff depth is calculated as

$$R(t) = R_{perv} + R_{imp} \tag{5.21}$$

where

$R(t)$ = total runoff at time t, in inches;

R_{imp} = runoff from impervious basin area, in inches; and

R_{perv} = runoff from the pervious basin area, in inches.

R_{imp} is given by

$$R_{imp} = (IMP)P(t) \tag{5.22}$$

where

IMP = the fraction of impervious area; and

$P(t) =$ the total precipitation at time t, inches.

R_{perv} is given by

$$R_{perv} = (1 - IMP)\{P(t) - f\} \tag{5.23}$$

where

f = the depth of infiltration which occurs during the interval.

Originally, f was calculated by the Antecedent Precipitation Index (see Linsley, et.al. 1982, pp. 171-172), but may be calculated by other means.

The resulting rainfall excess, R(t), is converted to a rate of flow, I(t), by

$$I(t) = 1.008\frac{R(t)}{\Delta t}A \tag{5.24}$$

where

I(t) = runoff flow rate, in cfs;

A = drainage area, acres;

Δt = the length of the time interval, in hours; and

1.008 = a conversion from acre-inches per hour to cfs

I(t) is usually termed the "Instantaneous Hydrograph"[1].

[1]This is not an instantaneous *unit* hydrograph. An instantaneous unit hydrograph is a unit hydrograph (Section 5.1) whose chosen excess rainfall duration is infinitesimally small. Its application is not discussed here. The reader should refer to Linsley, et. al. (1982, p. 221) or Huggins and Burney (1982, p. 206) for further information.

To obtain the final basin hydrograph, the "instantaneous hydrograph" is routed through an imaginary linear reservoir. A linear reservoir is assumed to have a storage which is directly proportional to outflow. The routing is described by

$$Q(t) = Q(t-\Delta t) + K\{I(t-\Delta t) + I(t) - 2Q(t-\Delta t)\} \tag{5.25}$$

where

$Q(t)$ = outflow at time t, in cfs;

$Q(t\text{-}\Delta t)$ = outflow computed for the previous interval, in cfs; and

K = a dimensionless coefficient.

K[1] represents the reciprocal of the imaginary reservoir size, and has a value given by

$$K = \frac{\Delta t}{2t_c + \Delta t} \tag{5.26}$$

where t_c is the time of concentration, in hours, using the wave traveltime (see Section 2).

An important limitation of the SBUH method is that the peak discharge, as calculated by the SBUH, method cannot occur after precipitation ceases. In reality, for short duration storms over flat and large watersheds, the peak discharge can actually occur after the precipitation ceases.

5.5 Easy Hydrograph Method

The District uses a version of the Santa Barbara Urban Hydrograph Method for reviewing surface water management system permits. This version is called the

[1]Equation (5.25) is a special case of the Muskingum routing equation (see Section 6.2.2) where the parameter x is set equal to zero.

"Easy Hydrograph Method" (EHM) and was developed by Alan Hall (Hall, 1981). The EHM method utilizes the imaginary linear reservoir concept of the SBUH for determining basin outflow. It is different from the original SBUH in the estimation of rainfall excess.

As with the SBUH, the EHM design rainfall is segregated into periods of equal time, Δt hours. However, runoff for the interval is computed based on the SCS Curve Number Method (Section 4.1, specifically equation 4.1). The computation of the soil moisture storage, S in inches, can be determined from an appropriate SCS Curve Number, and equation (4.1), or the use of the District's soil moisture curves, presented in Figure 4.2 and equation 4.6.

The instantaneous runoff hydrograph is then obtained, as with the SBUH method, using equation (5.24). The final design runoff hydrograph is also calculated as in the SBUH method, by equation (5.25), utilizing the imaginary linear reservoir concept. The routing constant is set equal to the basin's time of concentration, as well.

5.6. HEC-1

The HEC-1 Flood Hydrograph package was developed by the Hydrologic Engineering Center (HEC) of the U.S. Army Corps of Engineers. The current version (HEC-USACE, 1981) provides a multitude of analysis capabilities: watershed simulation, dam safety analyses, precipitation depth-area relations, and flood damage analyses. Only the watershed simulation is of interest here.

HEC-1 is a modularized package containing a number of components, any of which may be employed for a particular case:

- *Stream network modeling:* The HEC-1 stream network model is the basic building block for the package. A schematic is shown in Figure 5.11.

- *Land surface runoff:* Calculation of excess rainfall and subsequent runoff can be calculated by a variety of methods. One method for calculation of excess rainfall is the SCS Curve Number method (Section 4.1). Runoff estimations can be made with the use of several synthetic unit hydrographs, or by overland flow kinematic wave routing.

- *River routing:* Flood routing methods which may be employed with HEC-1 include channel and overland flow by kinematic wave; and hydrologic routing methods such as Muskingum, Working R and D, level-pool, Average Lag, and Modified Puls. Some of these are discussed in Sections 6 of this review.

- *Reservoir, diversion, and pump components:* Procedures are available to account for these various basin inflows and outflows.

HEC-1 is obviously a very diverse and extensive package. A complete discussion regarding application and limitations for even a few cases would be lengthy and certainly not within the scope of this review. However, there are some fundamental assumptions involved with the use of HEC-1 which can and should be discussed:

1. *Parameters supplied to the model represent spatial averages within the subbasin.* Consequently, subbasin size should be small enough to allow application of averages.

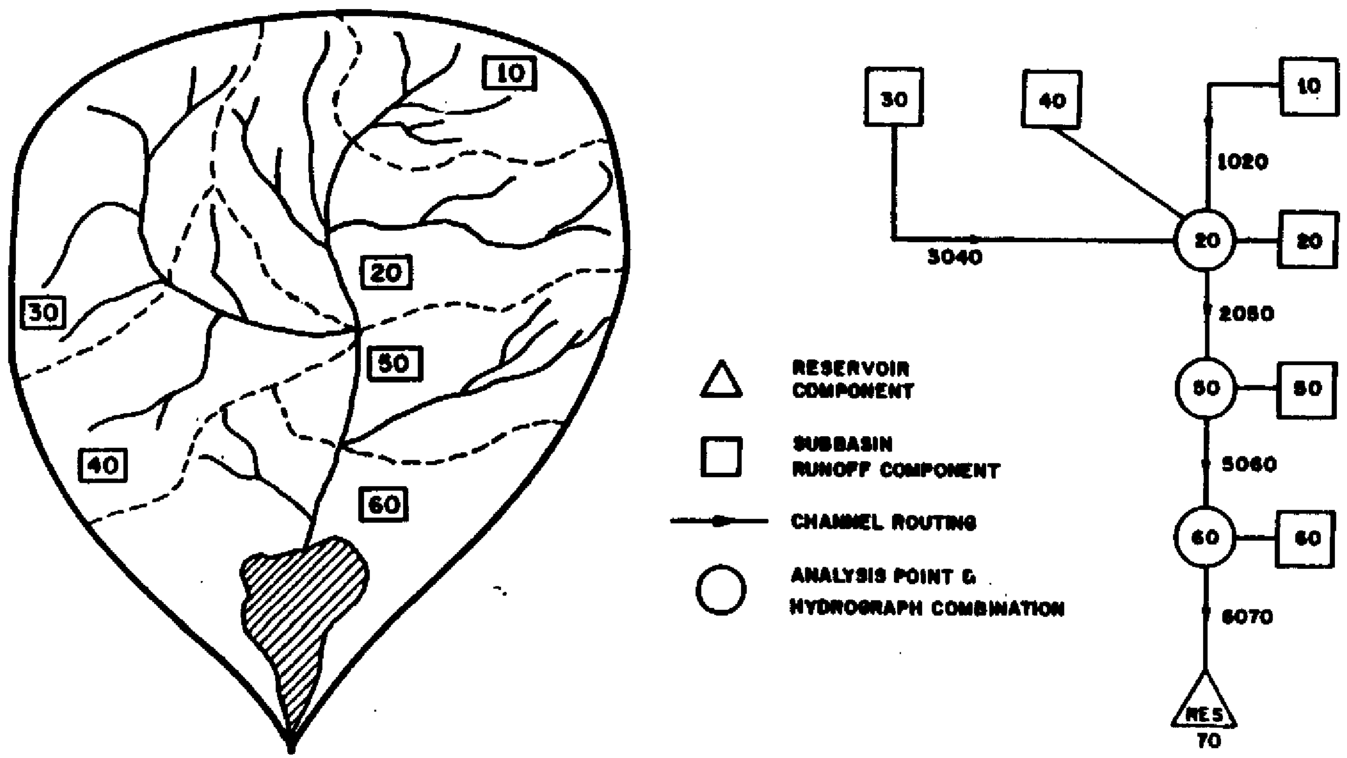

Figure 5.13. Schematic representation of the HEC-1 stream network model. (Reproduced from USACE-HEC, 1981)

2. *Simulations are limited to single storm events.* HEC-1 does not account for changes in basin soil storage over time. Consequently, long term simulations cannot be accurately performed.

3. *Use of kinematic wave based overland flow routing.* Kinematic wave routing does not account for backwater effects which can occur with very flat river bed slopes. See Section 6.3 for a further discussion. Other runoff estimation choices involve the use of synthetic unit hydrographs, some of which are discussed in Section 5.2.

4. *Streamflow routing is based on hydrologic routing methods.* This would include Muskingum, Modified Puls and others. Hydrologic routing techniques are not applicable to channels subject to backwater.

5. *Reservoir routing is based on the Modified Puls method.* The modified Puls method is not applicable to cases where automatic gates are controlling reservoir outflow.

Application of the HEC-1 program to South Florida is severely restricted due to these limitations.

5.7. TR-20

The TR-20 watershed model was developed by the Soil Conservation Service (SCS) and is described in Technical Release 20 (USDA-SCS, 1983). Most of the model components are based on SCS Hydrologic Methods, presented in NEH-4 (USDA-SCS, 1985), and many are discussed separately in this review. The TR-20 model is an event-based model, and is typically used for evaluation of impacts resulting from a proposed change in storm-water management, a diversion, or a change in land use.

When using the model, the basin under consideration is subdivided into relatively homogeneous subbasins. Any of the SCS design rainfall distributions (see USDA-SCS, 1986a) are available, or actual rainfall can be supplied (up to a total of 9 separate distributions). Rainfall excess is computed using the SCS Runoff Curve Number method (Section 4.1).

The unit hydrograph approach (see Section 5.3) is used to determine each subbasin outflow. The SCS Dimensionless Curvilinear unit hydrograph (DCUH, discussed in Section 5.2.1) is the default UH used. This UH has a peak rate factor of 484, which makes it unapplicable for most South Florida situations. The SCS has another dimensionless UH available for TR-20. The DELMARVA unit hydrograph has a peak rate factor of 284, which is recommended for any situation where the slope of the basin is 0.5 percent or less (USDA-SCS, 1986c). At the time of the writing, the authors were unable to find a description of this UH published by the SCS. If either of these UH's are not acceptable, the user of TR-20 is allowed to supply his own dimensionless UH. For instance, Example 5.2 describes a method for estimating a dimensionless UH for varying peak rate factors.

Subbasin outflow can be added to other subbasin outflows and routed to the basin outflow point. Along the routing path various structures and diversions can be considered. Routing computations are done by either the storage-indication method (see NEH-4, USDA-SCS, 1985, Chapter 17) for reservoirs or the Att-Kin method (see USDA-SCS, 1983, Appendix G) for channel reaches. The reservoir routing method requires an elevation-discharge-storage relation at each structure which defines a reservoir. The channel routing method requires discharge-area rating curve,at the end of each channel reach in the form of a table or

$$Q = xA^m$$

where

Q = discharge;

A = cross-sectional area; and

x, m = coefficients.

This curve may not change during the simulation, and looped curves are not allowed. Prior to 1983, the TR-20 model used the Convex Method (Section 6.2.3) for channel routing.

The TR-20 model is designed primarily for small watersheds where thunderstorms or other high intensity, short duration storms cause peak flows. The major limitations of the TR-20 model are discussed elsewhere in this review. The reader should refer to Section 4.1 for a discussion of the limitations of the runoff Curve Number method and Section 5.2 and 5.3 for a discussion of the SCS UH methodology. Some limitations more specific to the operation of TR-20 are as follows:

All hydrographs are limited to 300 points. This can be a problem with long duration storms. The hydrograph may not have the proper definition when long time step is used. Subbasin hydrographs may use a different time than the overall basin outflow.

Structure rating curves for reservoir routing ignore effects of changing tailwater stage. In most South Florida cases, flow through weirs and other flow control structures are effected by changing downstream stage. Structure rating curves in TR-20 are assumed constant, when in reality they can change as the tailwater stage changes.

Channel reach rating curves do not allow for changing downstream or upstream conditions, only the reach storage. TR-20 channel rating curves are assumed to vary only with the storage in the reach. In a real channel, the rating curve depends not only on the storage, but also on whether the stage is rising or falling (more specifically, the slope of the water surface in the reach). TR-20 does not allow looped rating curves, which would account for rising and falling stages.

6. FLOOD ROUTING

6.1. General

Hydrograph methods, discussed in Section 5, provide a means for predicting the time distribution of runoff at the single point in a watershed - the point for which the hydrograph was derived. However, operation of surface water management systems, or analysis of flood control problems require prediction of discharge rate behavior at several points within the system. The prediction or estimation of such behavior is accomplished with the use of *flood routing* techniques. These techniques are used to predict a hydrograph at one point in a watershed using a known hydrograph at another point.

6.1.1 TYPES OF FLOOD ROUTING

Routing is an analysis of unsteady flow. It is used to describe the translation of a flood wave, and associated changes in hydrograph shape, through a given waterway. There are two unknown quantities in such an analysis: flow, Q, and stage, y. These will vary with both time, t, and position, x, within the waterway. Routing techniques use mathematical relationships to determine Q and y as functions of t and x. Since there are two unknowns, it follows that two mathematical relationships are required to describe action of the flood wave in the waterway. Routing techniques are classified on the basis of the types of relationships chosen.

Routing techniques are, for the most part, based on the physics of fluid flow. Of particular concern here are two laws of physics:

- conservation of mass, or *continuity*, and
- conservation of momentum, or Newton's Second Law.

These two laws, and their application to fluid flow, are briefly discussed in Section 6.1.2.

In this review, routing methods are divided into two types:

- *hydrologic*, and
- *hydraulic*.

Hydrologic routing methods use the conservation of mass principle only. A second relationship is usually provided by making some assumptions about channel storage: For example, the Modified Puls method (Section 6.2.1) directly relates waterway storage to waterway outflow. These assumptions are not necessarily physically based, but often provides an adequate approximation of the physical situation. *Hydraulic* routing methods use both conservation of mass and conservation of momentum principles to analyze fluid flow. These two principles provide the two mathematical relationships required to determine the two unknowns, Q and y.

Section 6.2 discusses some specific hydrologic routing methods. Section 6.3 briefly discusses the use of hydraulic routing methods, and provides an overview of two hydraulic routing computer models.

6.1.2. PHYSICAL RELATIONSHIPS

Routing techniques are, for the most part, based on the physics of fluid flow. Of particular concern here are two physical laws:

- the conservation of mass, or *continuity*, and
- the conservation of momentum, or Newton's second law.

Before any discussion of fluid flow can be initiated the *control volume* concept must be introduced. Control volumes are analytical tools which help the engineer or scientist to clearly define a parcel of fluid for analysis. An imaginary boundary is drawn around a portion of fluid. Mathematical relations are developed to describe the reactions of this portion as it is acted upon by its surroundings. These mathematical relations do not include any description of the world outside the control volume, except forces and fluxes acting at its boundary. Using the control volumes depicted in Figure 6.1, the laws of conservation of mass and momentum are defined.

The conservation of mass law states that matter can be neither created, nor destroyed (aside from radioactive decay). *Continuity* is a special case of the conservation of mass principle for incompressible fluids, i.e., fluids with a constant density. If a fluid has a constant density, it follows that *volume* is conserved (i.e. neither created or destroyed). Specifically, then, continuity states that for any time period, Δt, the difference between the inflow to and outflow from a control volume must equal the change in storage within the volume:

$$I\Delta t - O\Delta t = \Delta S \tag{6.1}$$

where

I = rate of inflow;

O = rate of outflow; and

ΔS = change in storage volume.

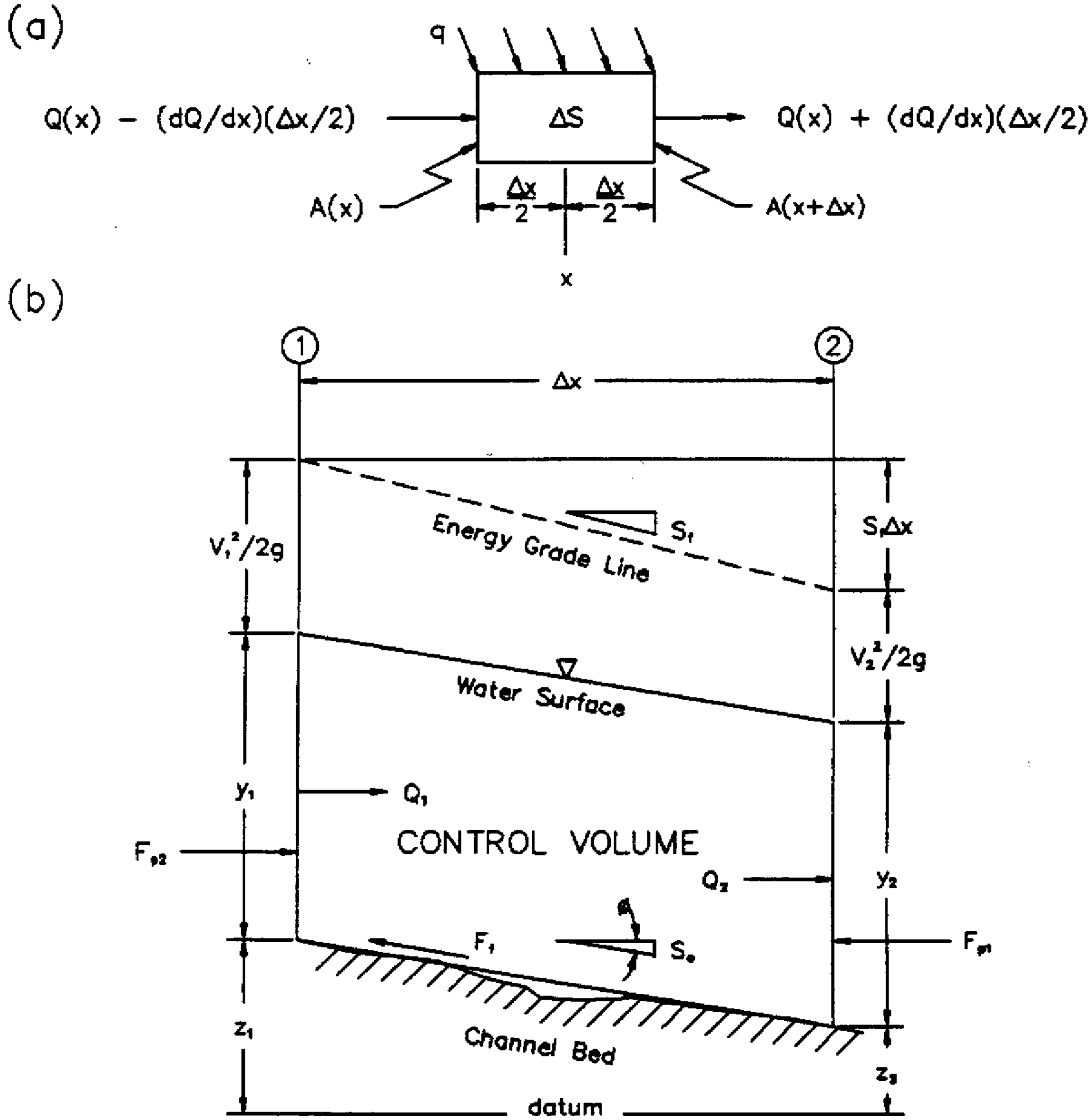

Figure 6.1. Control volumes to illustrate (a) the conservation of mass, and (b) conservation of momentum.

Continuity simply states the fact that fluid volume cannot be created nor destroyed within the control volume. For the control volume in Figure 6.1(a), we see that

$$I = Q - \frac{\partial Q}{\partial x}\frac{\Delta x}{z}\,dt + q\,\Delta x\,dt$$

$$O = Q + \frac{\partial Q}{\partial x}\frac{\Delta x}{z}\,dt$$

$$\Delta S = \frac{\partial A}{\partial t}\Delta x\,dt$$

Substituting these into equation (6.1), and simplifying, the *conservative* form of the continuity equation is derived:

$$q = \frac{\partial A}{\partial t} + \frac{\partial Q}{\partial x} \tag{6.2}$$

where

q = lateral inflow per unit length of the reach,

A = channel cross-sectional area;

Q = flow rate = AV

V = flow velocity;

and q, A, and Q are all functions of position x, and time, t. Equation (6.2) can also be expressed on a per unit width basis (i.e. a control volume 1 unit wide), which is dubbed the *nonconservative* form:

$$q = \frac{\partial A}{\partial t} + y\frac{\partial V}{\partial x} + V\frac{\partial y}{\partial x} \tag{6.3}$$

where y is the water surface stage.

The conservation of momentum law is derived from Newton's Second Law, which states that the sum of the external forces, ΣF_{ext}, acting on an object is equal to the product of the object's mass and its acceleration:

$$\Sigma F_{ext} = ma = \frac{dS_m}{dt}$$

but the product can be equated to the rate of change in the object's momentum with respect to time, dS_m/dt. When the conservation of momentum is applied to a fluid control volume, or reach, momentum flux in and out of the volume must be considered as well. Thus, for a fluid control volume, the conservation of momentum can be written as

$$\frac{dS_m}{dt} = \Sigma F_{ext} + (I_m - O_m) \tag{6.4}$$

where I_m and O_m are the rates of momentum inflow and outflow, respectively. The external force acting on the control volume is composed of the following:

F_p - pressure force. Pressure force is that which is caused by differences in hydrostatic pressures at either end of a reach associated with the change in water depth, y, over the reach.

F_g - gravity force. A fluid has mass that is acted upon by Earth's gravity. The magnitude of the gravity force in the direction of flow is dependent upon the volume of water in the reach, and its bottom slope, S_0.

F_f - friction forces. A moving fluid encounters resistance to flow from its surroundings, either from surrounding fluid, or the channel in which it flows, and possibly wind. This force is described by the slope of the energy grade line, S_f (see Figure 6.1(b)).

The remaining terms of equation (6.4) are sometimes called "inertial" forces.

If equation 6.4 is applied to a control volume such as that in Figure 6.1(b), and some simplifications made, the conservative form of the momentum equation is derived:

$$\underbrace{\underbrace{\frac{1}{A}\frac{\partial Q}{\partial t}}_{\text{local inertia term}} + \underbrace{\frac{1}{A}\frac{\partial(Q^2/A)}{\partial x}}_{\text{convective inertia term}}}_{\text{"inertial" forces}} + \underbrace{g\frac{\partial y}{\partial x}}_{\text{pressure forces}} - \underbrace{g(S_f - S_o)}_{\text{friction and gravity forces}} = 0 \tag{6.5}$$

where g is the acceleration of gravity. This equation is also used in the nonconservative (per unit width) form:

$$\frac{\partial V}{\partial t} + V\frac{\partial V}{\partial x} + g\frac{\partial y}{\partial x} - g(S_o - S_f) = 0 \tag{6.6}$$

Equations (6.2), (6.3), (6.5), and (6.6) are collectively called the *Saint Venant* equations - either the conservative form (equations (6.2) and (6.5)), or the nonconservative form (equation (6.3) and (6.6)). They deal specifically with open channel flow in a single dimension (x). Separate routing techniques are distinguished on the basis of which equations, and what part of those equations, are used to analyze flow. In general, hydraulic routing techniques solve equations (6.2) and (6.5), or (6.3) and (6.6), for flow, Q, and stage, y, as functions of time, t, and position, x. Many different techniques are used.

Typical application of the Saint Venant equations is based on the following assumptions:

1. *Flow is in a single direction.* This means that, within the confines of a reach, all quantities -- stage, discharge, and velocity -- vary longitudinally along the channel. Although these quantities can actually vary across the channel, they are computed as cross sectional averages for the channel.

2. *Water surfaces are horizontal in channel cross sections.* This assumption is used so that, again, only one dimension need be considered. If the water surface across a channel were not horizontal, other pressure forces, acting perpendicular to the flow, would be indicated. In order to properly analyze this situation, another equation, similar to equation (6.5), in another dimension, would be needed.

3. *Hydrostatic pressure dominates in the flow.* When flow is "gradually varied", the pressure on water at any depth is hydrostatic (pressure brought about by the weight of fluid above). When flow is rapidly varied, such as flow over a sharp crested weir, this hydrostatic pressure distribution no longer exists. The pressure term of equation (6.5) is based on a hydrostatic pressure distribution.[1]

---

1 For technical clarity, it must be stated that hydrostatic pressure is just one aspect of "gradually" versus "rapidly" varied flow calssification. For further information on the assumptions involved see Chow (1959), p. 217, and p. 357. The Saint Venant equations consider only *gradually* varied flow.

4. *The channel bottom slope is small.* This allows the gravity term of equation (6.5) to be conveniently related to the channel slope, S_o.

5. *The physical reach configuration is fixed.* This includes channel bottom slope and cross-sections, for example. If the channel scours or deposition occurs, channel cross section data becomes a function of time as well as position. Simplifications used to arrive at equation (6.5) consider channel cross sections constant with respect to time.

6. *The friction coefficient* (typically Manning's n) *for uniform flow is applicable to gradually varied flow.* The friction coefficient is used to calculate the energy line slope, S_f, based on calculated flows in the reach, Q. Most coefficients available are for uniform flow.

The reader should be aware that the Saint Venant approach is only one interpretation of fluid flow physics and others exist. This review presents some models which use the Saint Venant equations, particularly in Section 6.3. These models are not the only alternatives available to the design engineer.

6.2. Hydrologic Routing Methods

Hydrologic routing methods are based on various solutions of the continuity equation (equation (6.1)). Several methods are available in the literature (see Linsley, et. al., 1982; and Chow, 1964). This work will consider three methods which differ in their representation of reach storage:

- The *Modified Puls* (reservoir routing) method assumes that a unique and constant relation exists between outflow and reach storage.

- The *Muskingum* method relates reach storage to a linear combination of reach inflow and outflow.

- The *Convex* method assumes that reach storage has no effect on flood wave translation or attenuation.

6.2.1. MODIFIED PULS (RESERVOIR) ROUTING

The Modified Puls method is among the simplest of routing techniques. More complete descriptions of the method can be found in Linsley, et. al. (1982) or Chow (1964). The method is based on the storage equation, a form of the continuity equation, equation (6.1), which states that the average inflow to a reservoir during a routing period is equal to the average outflow during that period plus the change in reservoir storage. In algebraic terms,

$$\left[\frac{I_1 + I_2}{2}\right]\Delta t = \left[\frac{O_1 + O_2}{2}\right]\Delta t + (S_2 - S_1) \tag{6.7}$$

where

Δt = the routing period;

= $t_2 - t_1$

I_1 = rate of flow into the reservoir at time t_1;

O_1 = rate of flow out of the reservoir at time t_2 and

S_1 = reservoir storage volume at time t_1.

I_2, O_2, and S_2 have similar definitions at time t_2. Rearranging equation (6.7) we have:

$$\frac{S_2}{\Delta t} + \frac{O_2}{2} = \left\{\frac{I_1 + I_2}{2}\right\} + \left\{\frac{S_1}{\Delta t} - \frac{O_1}{2}\right\} \tag{6.8}$$

Initial conditions, at time t_1, are known, as is I_2, but S_2 and O_2, or the left hand side of equation (6.8), remain unknown. With two unknowns and one equation, another relation must be found. In the Modified Puls method, several assumptions are made to provide such a relation. The additional relationship is supplied by observing that typically both reservoir storage and outflow are some function of reservoir surface elevation. A function relating stage and outflow can be developed, and another relating stage and storage. Combining these two, a *routing curve* is produced which relates the unknown terms on the left hand side of equation (6.8) to reach outflow ($S/\Delta t + O/2$ versus O).

For each time step in the routing procedure, equation (6.8) is used to calculate $S_2/\Delta t + O_2/2$. The routing curve is used to obtain O_2. In preparation for the next routing period

$$\frac{S_2}{\Delta t} - \frac{O_2}{2} = \left\{\frac{S_2}{\Delta t} + \frac{O_2}{2}\right\} - O_2 \tag{6.9}$$

is calculated, and the process is repeated. Example 6.1 illustrates application of the modified Puls method.

The modified Puls method relies very much on the validity of its routing curve. In order to construct a routing curve, some assumptions must be made:

1. *Flow velocities and friction forces are negligible within the reach or reservoir.* It follows from this assumption that the reservoir water surface is level. This allows construction of a one to one relationship between reservoir stage and storage (i.e. for each stage there is one, and only one, storage). This usually applies in cases where the reservoir is large in comparison with the inflows and outflows. Nonlevel reservoir surfaces are more likely with small reservoirs which have large inflows (a channel reach is a good example). Other methods should be sought for analysis of small reservoirs.

2. *The routing curve is invariant throughout the analysis.* This particularly depends on the stage/outflow relationship. Reservoirs with uncontrolled spillways, and no tailwater problems, will have a consistent stage to outflow relation. However, when reservoir spillways have automatic gates, the Modified Puls method cannot be applied, since the routing curve would be changing within each routing period. There are ways to include changing gate controls, but only if those gates remain stationary during the routing period (see Linsley, et. al., 1982, pg. 273). Furthermore, since the stage/outflow relation depends only on reservoir stage, any tailwater conditions which exist at the spillway will induce errors.

Other aspects of the Modified Puls method can limit its applicability. Some observations include:

- Calculated outflow will begin at the same time as inflow begins, which means the flood wave passes instantaneously through the reservoir reach regardless of its length.

- Selection of the routing period, Δt, has a considerable effect on analysis results. There are no formal rules for selection of a proper routing period. Viessman, et. al., (1977) recommend that there be at least five known points on the rising limb of the inflow hydrograph. More points, however, will improve accuracy.

Although the Modified Puls method is best suited for routing through reservoirs, it has been applied to channel reaches as well (see Chow, 1964). The HEC-1 flood hydrograph package (Section 5.6) has an option for channel routing with the Modified Puls method.

Example 6.1: Routing by the Modified Puls Method. A particular reservoir is subject to a flood event. The reservoir surface area, A, is 39.66 acres. Within the normal range of surface stage, this area is constant. Consequently, reservoir storage can be calculated by

$$S = AH \tag{6.10}$$

where H is the reservoir stage in feet. The spillway discharge for the reservoir is given by:

$$O = 5H^{3/2} \tag{6.11}$$

where O is expressed in cfs. The inflow hydrograph is given in the second column of Table 6.2. Route this hydrograph through the given reservoir using the Modified Puls Method, and a routing period of 12 hours.

Sample Calculation for Routing Curve. The routing curve calculation for the given reservoir is shown in Table 6.1. The routing curve ordinate for H = 5 feet will be calculated to illustrate the process. The reservoir outflow is given by equation (6.11). At H = 5 feet,

$$O = (5\ feet^{3/2}/sec)(5\ feet)^{3/2}$$

$$= 56\ cfs \quad (as\ shown\ in\ Table\ 6.1)$$

Reservoir storage is given by equation (6.10):

$$S = (39.66\ acres)(5\ feet)(43{,}560\ sq.\ feet/acre)$$

$$= 8{,}637{,}900\ cubic\ feet$$

For convenience, the storage will be divided by the routing period, 12 hours.

$$\frac{S}{\Delta t} = \frac{8{,}637{,}900\ cubic\ feet}{12\ hours}\ \frac{hour}{3600\ sec}$$

$$= 200\ cfs \quad (as\ shown\ in\ Table\ 6.1)$$

Finally, the storage and outflow are combined to produce an ordinate of the routing curve:

$$\frac{S}{\Delta t} + \frac{O}{2} = 200\ cfs + (56\ cfs)/2$$

$$= 228\ cfs \quad (as\ shown\ in\ Table\ 6.1)$$

Sample Calculation of reservoir outflow. The results of the routing procedure are shown in Table 6.2. To illustrate the process, routed outflow during the 4th routing period will be calculated.

For routing period 4, the average inflow rate is

$$I_{avg} = \frac{(I_3 + I_4)}{2}$$

$$= (210\ cfs + 310\ cfs)/2$$

$$= 260\ cfs \quad (\textit{as shown in Table } 6.2)$$

$$\frac{S_3}{\Delta t} - \frac{O_3}{2} = \left\{\frac{S_3}{\Delta t} + \frac{O_3}{2}\right\} - O_3$$

$$= (245\ cfs) - (62\ cfs)$$

$$= 183\ cfs \quad (\textit{as shown in Table } 6.2)$$

Using equation (6.8),

$$\frac{S_4}{\Delta t} + \frac{O_4}{2} = (260\ cfs) + (183\ cfs)$$

$$= 443\ cfs \quad (\textit{as shown in Table } 6.2)$$

The rating curve in Table 6.1 is then consulted, and by interpolation

$$O_4 = 141 \text{ cfs} \qquad \text{(as shown in Table 6.2)}$$

6.2.2. MUSKINGUM METHOD

The Muskingum method was developed by G.T. McCarthy (1936). Discussions of the method can be found in Linsley, et. al. (1982), Chow (1964), and Viessman, et.

TABLE 6.1. CALCULATION OF ROUTING CURVE FOR EXAMPLE 6.1.

Reservoir Stage H, feet	Outflow O, cfs	Reservoir Storage $S/\Delta t$, cfs	$S/\Delta t + O/2$ cfs
0	0	0	0
1	5.0	40	42.5
2	14.2	80	87.1
3	26.0	120	133.0
4	40.0	160	180.0
5	56.0	200	228.0
6	74.0	240	277.0
8	113.1	320	376.6
10	157.0	400	478.5
12	207.8	480	583.9
15	290.0	600	745.0
18	381.8	720	910.9
20	445.0	800	1022.5

TABLE 6.2. RESERVOIR ROUTING RESULTS FOR EXAMPLE 6.1. Those items marked in bold print are the given initial information.

Routing Period (12 hours)	**Inflow I (cfs)**	$\frac{I_1 + I_2}{2}$ (cfs)	$\frac{S_1}{\Delta t} - \frac{O_1}{2}$ (cfs)	$\frac{S_2}{\Delta t} + \frac{O_2}{2}$ (cfs)	O_2 (cfs)
0	**30**				**0.0**
1	**50**	40.0	0	40.0	4.7
2	**100**	75.0	35.3	110.3	20.2
3	**210**	155.0	90.1	245.1	62.3
4	**310**	260.0	182.8	442.8	141.6
5	**350**	330.0	301.2	631.2	231.9
6	**300**	325.0	399.3	724.3	279.4
7	**220**	260.0	444.8	704.8	269.5
8	**150**	185.0	435.3	620.3	226.4
9	**95**	122.5	393.9	516.4	175.3
10	**60**	77.5	341.2	418.7	131.2
11	**50**	70.0	287.4	357.4	105.6
12	**42**	46.0	251.9	297.9	82.2
13	**35**	38.5	215.7	254.2	65.6
14	**30**	32.5	188.6	217.0	53.7
15	**30**	30.0	167.4	197.4	45.8

al. (1977). Like other hydrologic routing methods, the Muskingum method relies on the continuity, or storage, equation (equation (6.1)). However the Muskingum method relates reach storage, at any time, to a linear combination of inflow and outflow:

$$S = k\{xI + (1 - x)O\} = kO + kx(i - O) \tag{6.12}$$

Calculation of storage by this equation accounts for nonlevel reach surfaces, or wedge storage. Figure 6.2 depicts, graphically, calculation of storage using equation (6.12).

A change in storage during a routing period, $\Delta t = t_2 - t_1$, is then given by

$$\frac{S_2 - S_1}{\Delta t} = k\{x(I_2 - I_1) + (1 - x)(O_2 - O_1)\}$$

or solving for O_2:

$$O_2 = C_1 I_2 + C_2 I_1 + C_3 O_1 \tag{6.13}$$

where:

$$C_1 = -\frac{\{kx - 0.5\Delta t\}}{\{k - kx + 0.5\Delta t\}} \tag{6.14}$$

$$C_2 = \frac{\{kx + 0.5\Delta t\}}{\{k - kx + 0.5\Delta t\}} \tag{6.15}$$

$$C_3 = \frac{\{k - kx - 0.5\Delta t\}}{\{k - kx + 0.5\Delta t\}} \tag{6.16}$$

The parameter x is a weighting factor which describes the relative influence of the inflow, I, and the outflow, O. The value of x ranges from 0 to 0.5. The storage constant, k, has dimensions of time, and expresses the reach storage to discharge ratio. The numeric value of k is approximately equal to the travel time of the reach.

The values of k and x are obtained using the inflow and outflow hydrographs from a previous event. Figure 6.3 shows an inflow and outflow hydrograph for an example reach. Reach storage will be maximum when the outflow hydrograph intersects the inflow hydrograph. At this point (C in Figure 6.3), the change in storage with respect to time, dS/dt, equals zero. Differentiating equation (6.12) with respect to time, and setting dS/dt equal to zero, we find

$$x\left(\frac{dI}{dt}\right)_c = -(1-x)\left(\frac{dO}{dt}\right)_c \tag{6.17}$$

where x is the only unknown. The parameter k is determined by plotting values of $\{xI + (1\text{-}x)O\}$ (from equation (6.12)) against incremental reach storage, S_i (see Example 6.2 for further discussion of incremental storage calculations). This should yield a nearly straight line; the slope of the line is k.

The more typical process for estimating k and x begins with a guess of x. Plots of S versus $\{xI + (1\text{-}x)O\}$ are made for several guesses of x. For most choices of x, the resulting curve will be looped (see Figure 6.5). The value of x which produces the most linear graph of S versus $\{xI + (1\text{-}x)O\}$ is chosen, and the slope of this graph is k. The Muskingum routing equation (equation (6.13)) is relatively insensitive to x, so x is usually only estimated to the nearest tenth. Selecting x based on equation (6.17) is better suited to computer applications. Trial and error guesses at the value of x, is

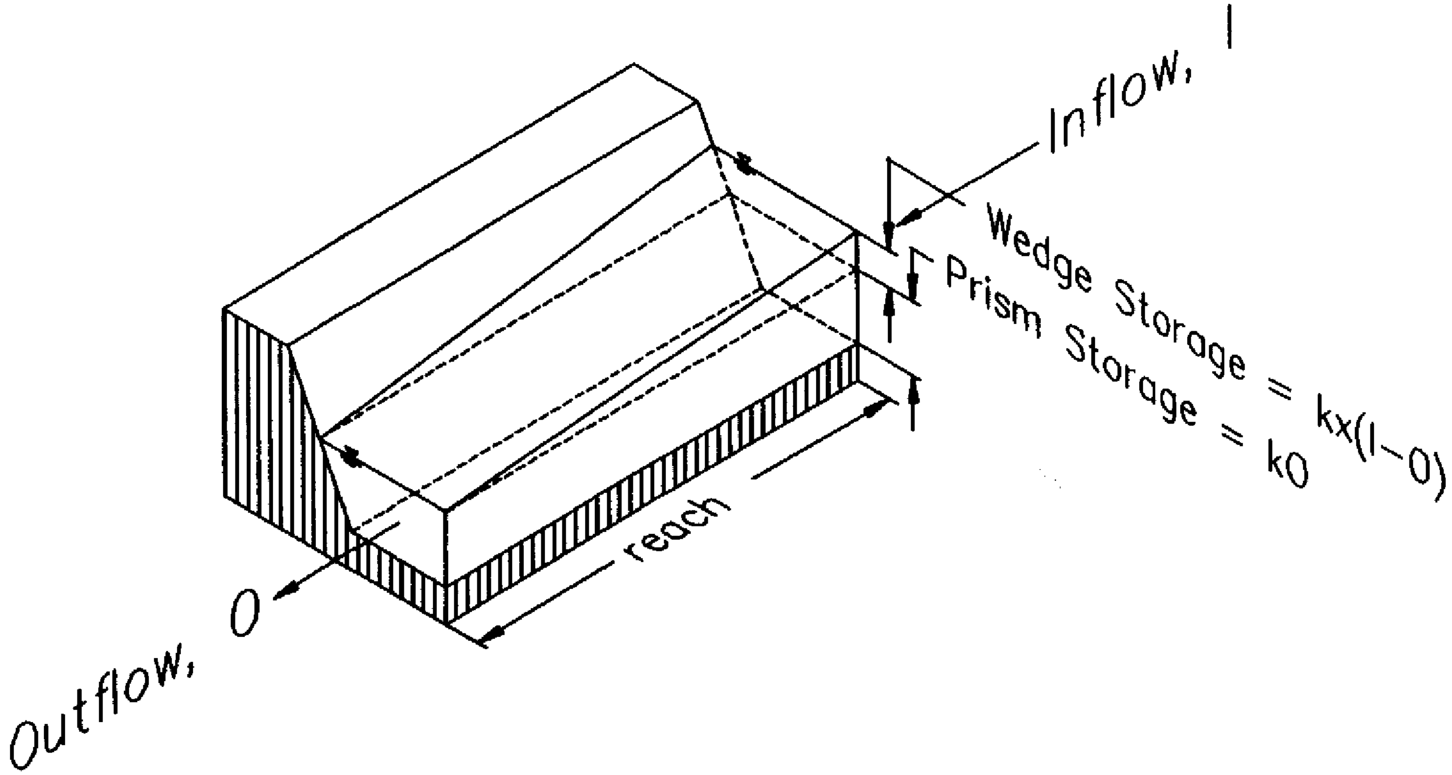

Figure 6.2. Cutaway drawing of a channel reach depicting the calculation of channel storage by the Muskingum method.

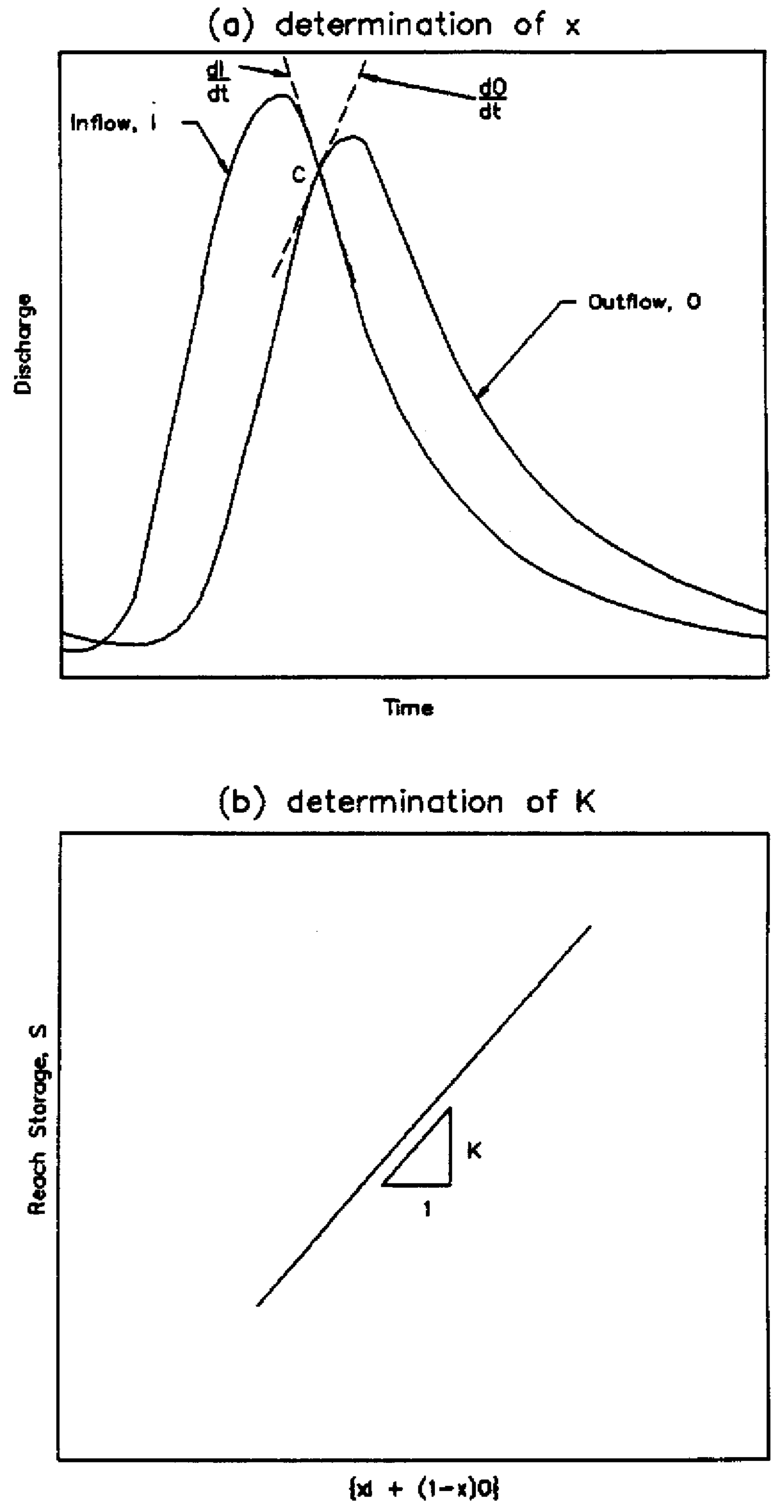

Figure 6.3. Theoretical determination of Muskingum coefficients, k and x.

the usual method for hand calculations. Estimation of x and k is further illustrated in Example 6.2.

The routing period, Δt, is usually chosen to be some convenient value between k/3 and k (Viessman, et. al., 1977, pg. 233). However, Chow (1964, pg. 25-42) recommends that Δt fall between 2kx and k. In most cases, one should probably use the more narrow range.

The Muskingum method relies on the assumption that reach storage can be calculated by equation (6.12), which implies

1. *Reach outflow is directly proportional to the reach prism storage.* While this relation is not exact, it is a reasonable approximation. As long as assumption 2 holds, the same proportionality can be used from case to case.

2. *The parameters k and x are constant during and between events.* This eliminates the use of the Muskingum method when automatic controls, or backwater effects exist within the reach. With a constant reach storage, outflow can be considerably reduced if backwater conditions exist, changing the value of k.

The Muskingum method is widely accepted and used. The major limitation of the Muskingum method is that data from a previous event is required. Of particular concern in South Florida applications is the inability of the method to include backwater effects.

Example 6.2: Application of the Muskingum Method. Table 6.3 shows hydrographs for two events which occurred on the same river reach. For the first event, on June 23, data for both inflow and outflow were available. However, during the second event, outflow was not recorded. Using the Muskingum method, and assuming an initial outflow on August 4 of 36 cfs, the August 4 outflow will be estimated.

Estimation of parameters x and k. The x and k parameters must be estimated from the previous event. In Table 6.4, incremental storage, S_i, is calculated using a trapezoidal approximation. This is accomplished by summing the incremental areas between the inflow and outflow hydrographs. Figure 6.4 shows the June 23 hydrographs, with the area between divided into increments. The area prior to the hydrograph intersection is positive (i.e. I-O); the area after is negative. The shaded incremental area in Figure 6.4, is given by

$$S_i = \left\{\frac{I_2 + I_1}{2} - \frac{O_2 + O_1}{2}\right\}(t_2 - t_1)$$

where S_i is the incremental storage. Consulting Table 6.3 for t_2 = 30 hours, t_1 = 24 hours and substituting

$$S_i = \{(575 + 717)/2 - (149 + 326)/2\}(30 - 24)$$

$$= 2451 \text{ cfs-hours}$$

This is shown in Table 6.4 along with the remainder of the storage calculations. Shown also in Table 6.4 are calculated values of $\{xI + (1\text{-}x)O\}$ for x = 0.2 and x =

TABLE 6.3. HYDROGRAPHS FOR EXAMPLE 6.2

Time (hours)	June 23 Inflow (cfs)	June 23 Outflow (cfs)	August 4 Inflow (cfs)
0	36	58	66
6	43	46	
12	121	42	250
18	346	61	
24	575	149	550
30	717	326	
36	741	536	595
42	612	674	
48	440	681	420
54	328	560	
60	251	437	295
66	196	341	
72	153	272	210
78	124	218	
84	101	180	147
90	84	150	
96	71	124	100
102	60	104	
108	52	86	74
114	46	73	
120	41	62	60
126	37	52	

0.4. These are plotted, with a curve for x = 0.5, in Figure 6.5. The choice for x should be 0.4, since its curve is nearest to a straight line. From Figure 6.5, we see that

$$K = 4000/300$$
$$= 13.3$$

Routing of new inflow. With the estimated coefficients from above, a proper, but convenient, routing interval, Δt, needs to be selected. The most useful routing period would be 12 hours, since the August 4 inflow was recorded at 12 hour intervals. This

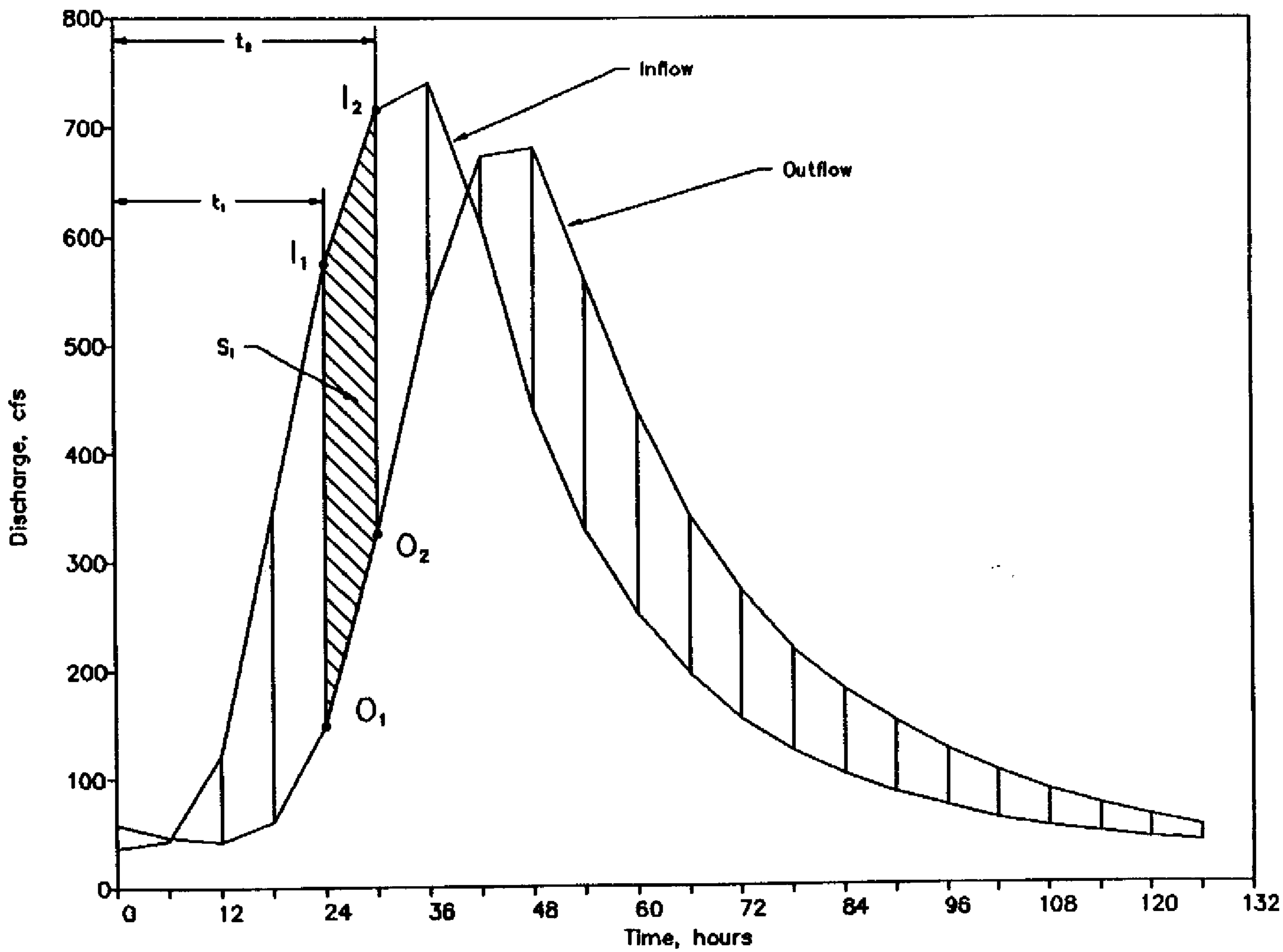

Figure 6.4. Determination of reach storage for June 23 event in Example 6.2.

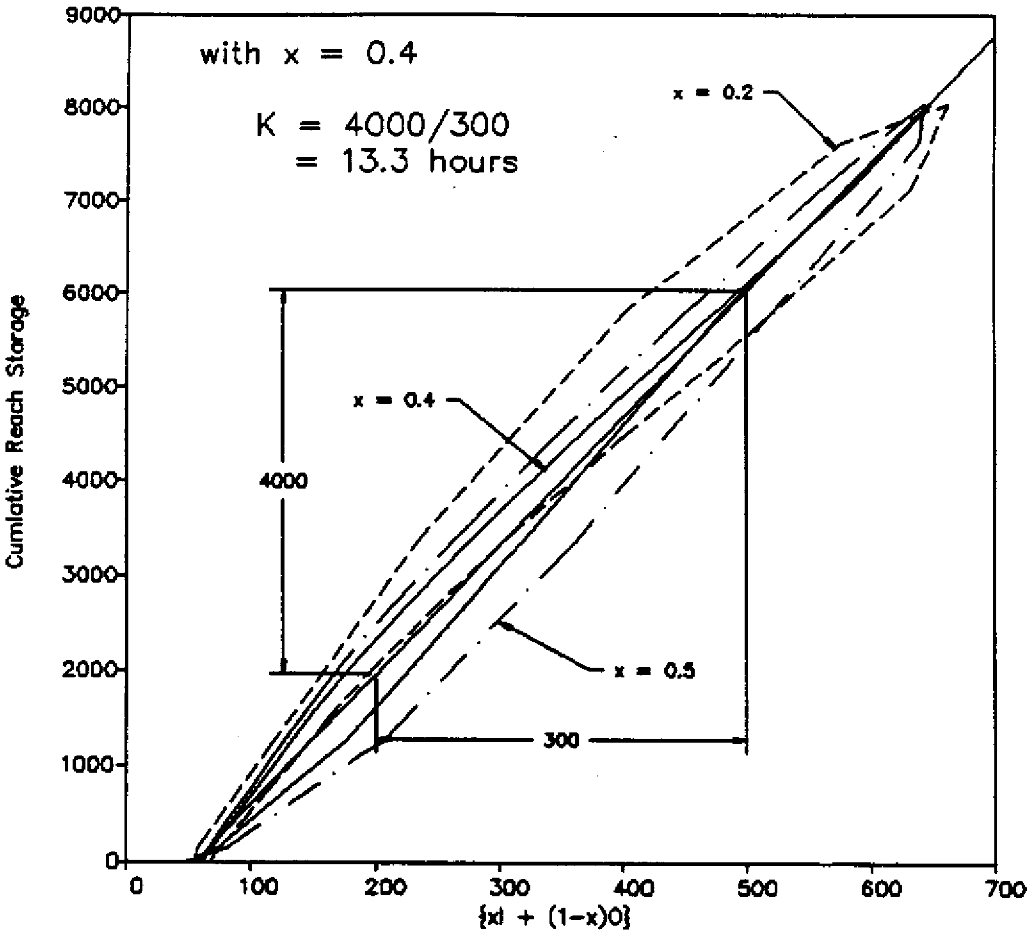

Figure 6.5. Plots of S versus {xI + (1-x)O} in Example 6.2 for several choices of x .

TABLE 6.4. CALCULATION OF REACH STORAGE AND {xI + (1-x)O} FOR EXAMPLE 6.2

Time (hours)	Change in Storage (cfs-hrs)	Accumulated Reach Storage (cfs-hrs)	{xI + (1-x)O} x = 0.2 (cfs)	{xI + (1-x)O} x = 0.4 (cfs)
0	0	0	54	49
6	-75	-75	45	45
12	228	153	58	74
18	1092	1245	118	175
24	2133	3378	234	319
30	2451	5829	404	482
36	1788	7617	577	618
42	429	8046	662	649
48	-909	7137	633	585
54	-1419	5718	514	467
60	-1254	4464	400	363
66	-993	3471	312	283
72	-792	2679	248	224
78	-639	2040	199	180
84	-519	1521	164	148
90	-435	1086	137	124
96	-357	729	113	103
102	-291	438	95	86
108	-234	204	79	72
114	-183	21	68	62
120	-144	-123	58	54
126	-108	-231	49	46

is less than the maximum stipulated ($k = 13.3$ hours). The routing interval should also be greater than

$$k/3 = 4.4 \text{ hours}$$

and

$$2kx = 2(13.3)(0.4) = 10.6 \text{ hours},$$

which it is. Then by equation (6.14),

$$C_1 = -\frac{\{(13.3hr)(0.4) - 0.5(12\ hr)\}}{\{13.3\ hr - (13.3\ hr)(0.4) + 0.5(12\ hr)\}}$$

$$= 0.049$$

by equation (6.15),

$$C_2 = \frac{\{(13.3hr)(0.4) + 0.5(12\ hr)\}}{\{13.3\ hr - (13.3\ hr)(0.4) + 0.5(12\ hr)\}}$$

$$= 0.81$$

and by equation (6.16),

$$C_3 = \frac{\{13.3\ hr - (13.3hr)(0.4) - 0.5(12\ hr)\}}{\{13.3\ hr - (13.3\ hr)(0.4) + 0.5(12\ hr)\}}$$

$$= 0.14$$

The estimated outflow hydrograph for the August 4 event is shown in Table 6.5. For illustration, outflow for routing period 4 will be calculated. Using equation (6.13),

$$O_4 = 0.049(595 \text{ cfs}) + 0.86(550 \text{ cfs}) + 0.14(183 \text{ cfs})$$

$$= 29 + 446 + 26 \text{ cfs}$$

$$= 501 \text{ cfs} \qquad \text{(as shown in Table 6.5)}$$

TABLE 6.5. CALCULATION OF OUTFLOW FOR EXAMPLE 6.2, AUGUST 4 EVENT.

Time (hours)	Actual August 4 Inflow (cfs)	C_1I_2 (cfs)	C_2I_1 (cfs)	C_3O_1 (cfs)	Calculated August 4 Outflow (cfs)
0	56				56
12	66	3	45	8	56
24	250	12	54	7	73
36	550	27	203	7	237
48	595	29	446	26	501
60	420	21	482	62	565
72	295	14	340	73	427
84	210	10	239	56	305
96	147	7	170	40	217
108	100	5	119	28	152
120	74	4	81	20	105
132	60	3	60	14	77
144	51	2	49	10	61
156	46	2	41	8	51

6.2.3. CONVEX METHOD

The Convex Method was developed by the SCS and is discussed in the most recent NEH-4 (USDA-SCS, 1985). The method relies on what the SCS terms the "routing principle". The routing principle says that for a reach of *proper length*, L, and a *specific flood wave travel time*, Δt,

$$\begin{aligned} &for\ I_1 \geq O_1,\ then\ I_1 \geq O_2 \geq O_1 \\ &for\ I_1 \leq O_1,\ then\ I_1 \leq O_2 \leq O_1 \end{aligned} \tag{6.18}$$

where

I_1 = inflow at time t_1;

O_1 = outflow at time t_1;

O_2 = outflow at time t_2; and

Δt = $t_2 - t_1$

Figure 6.6 shows reach input and output hydrographs for which equation (6.18) holds. The relationship in equation (6.18) can be seen in both the rising and falling limbs of the hydrographs.

The basic working relation for the Convex method is

$$O_2 = (1 - C)O_1 + CI_1 \tag{6.19}$$

where C is a routing coefficient and ranges from 0 to 1.0. C is estimated by the empirical relation

$$C = \frac{V}{V + 1.7} \tag{6.20}$$

where V is a steady flow velocity for the reach. The selection of V is critical to the Convex method. The SCS recommends three methods for its determination:

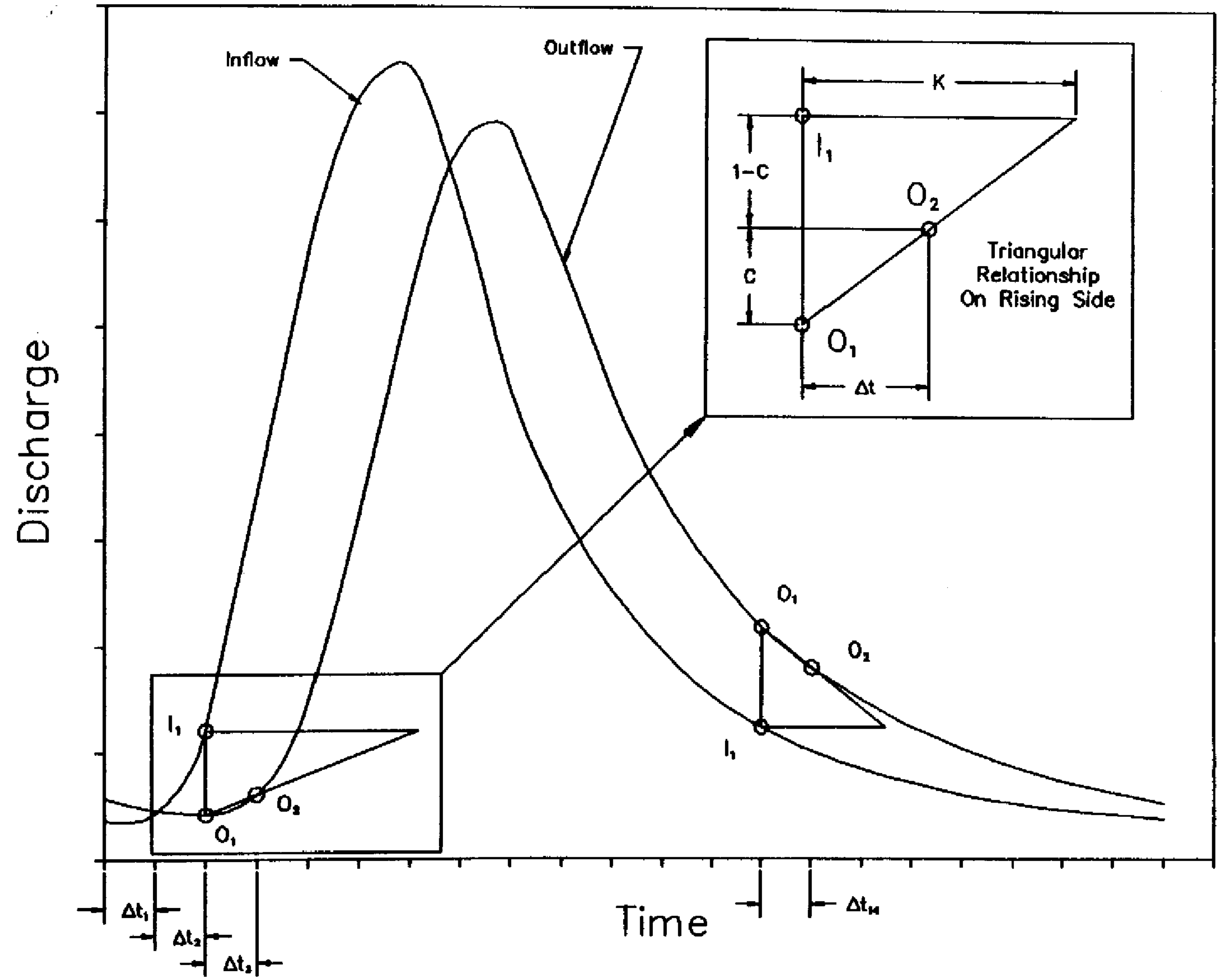

Figure 6.6. Illustration of the "routing principle", and triangular relation, used in the Convex method.

- The first is for use in a computer program. Steady flow velocities, from Manning's equation (Section 2.5), e.g., are calculated for all flows greater than 50 percent of the inflow hydrograph peak. The average of these velocities is used as V.

- The second process is used when a computer is not available. V is calculated as the steady flow velocity for 75 percent of the inflow peak.

- With the final method, C is calculated as 2x, where x is the weighting factor from the Muskingum method (Section 6.2.2).

It can be seen immediately that V, and hence C, will change for each inflow hydrograph to be routed.

The flood wave travel time, Δt, in the Convex method also changes with each inflow hydrograph, since it depends on the value of C. Selection of a routing interval is based on the triangular relationship depicted in the inset of Figure 6.6. By similar triangles:

$$\Delta t = CK \tag{6.21}$$

where K is the travel time corresponding to the steady-flow velocity, V, and can be estimated by

$$K = T_t = \frac{L}{3600V} \tag{6.20}$$

where

$K =$ steady flow travel time, hours;

$L =$ reach length, in feet; and

$V =$ steady flow velocity, feet per second.

The routing constant, k, from the Muskingum method (Section 6.3.2) can also be used to estimate K for the Convex method. Values of C, K, and Δt, calculated as described, are valid only for one inflow hydrograph and one reach.

If C and K, as calculated above, are known, there can be only one valid routing interval, Δt. This Δt can be inconvenient. If it is to be modified to a more convenient value, K and C must be modified accordingly. The SCS presents two possible ways to adjust Δt to a convenient value:

- *method 1:* fixing the reach length, L, and changing the routing coefficient, C, or

- *method 2:* fixing the routing coefficient and changing the reach length.

In the first method, a new routing coefficient, C^*, is chosen by

$$C^* = 1 - (1 - C)^a \tag{6.23a}$$

where

$$a = \frac{\Delta t^* + 0.5\Delta t}{1.5\Delta t} \tag{6.23b}$$

and

$\Delta t^* =$ desired routing interval, in hours and

$\Delta t =$ Flood wave travel time previously calculated.

The desired routing interval should be selected so that it is less than 1/5 of the time to peak.

In the second method, a new reach length, L*, is calculated by

$$L^* = \frac{3600 \Delta t^* V}{C} \tag{6.24}$$

where L* is the length of the first subreach. If L* is less than the given L, then the channel is effectively divided into subreaches. The first subreach is routed using L* and Δt* from equation (6.24). The remainder of the reach must be analyzed separately, using the calculated outflow from the upstream subreach as inflow. If L*, calculated by equation (6.24), is larger than the initial L, adjustment of the routing period should be made by equation (6.23). Example 6.3 illustrates the use of the Convex method.

Two aspects of the Convex Method distinguish it from other hydrologic channel routing methods based on the continuity equation:

1. The computed outflow begins one routing interval after inflow begins, which is the case for real channels. The Convex Method can then account for travel time within a reach. The Modified Puls and Muskingum methods predict outflow to begin during the same routing interval as inflow begins, which ignores travel time within a reach.

2. For any time step, the inflow for the next routing interval is not included in the Convex Method working equation, as do the Modified Puls and Muskingum methods. This allows the Convex Method to be used for forecasting purposes. That is, the method can predict the outflow for a given routing interval using only the inflow and outflow from the previous routing interval.

The Convex Method is generally used for routing hydrographs through stream reaches with negligible storage effects. It is useful in urban hydrology applications, where channel routing processes usually involve relatively short reaches of improved channel. The method was included in early versions of the TR-20 project formulation program (Section 5.8), in 1964 and 1972, but the most recent TR-20 (USDA-SCS, 1983) does not include it as an option.

Example 6.3: Application of the Convex Method. A certain channel reach is 75,000 feet long with a trapezoidal cross-section having a base width, b, of 12 ft and side slopes of z = 3.5 to 1. A bottom slope, So, of 0.01 percent and Manning's n of 0.03 are assumed. An input hydrograph for the reach is shown in Table 6.6. Route this input through the reach using the Convex method.

Calculation of steady flow velocity. In order to calculate the Convex method coefficients, a steady-flow velocity must be estimated. Examining the inflow hydrograph, in Table 6.6, we see that the peak flow is 595 cfs. A steady flow velocity will be calculated using 75 percent of the peak flow, or

$$Q = 0.75(595 \text{ cfs}) = 446 \text{ cfs}$$

TABLE 6.6. INFLOW HYDROGRAPH FOR EXAMPLE 6.3

Time, hr	Inflow, cfs	Time, hr.	Inflow, cfs
0	0	54	210
6	10	60	147
12	36	66	100
18	66	72	74
24	250	78	60
30	550	84	51
36	595	90	46
42	420	96	20
48	295	102	0

and Manning's equation:

$$V = \frac{Q}{A} = \frac{1.49}{n} R_h^{2/3} S_o^{1/2} \tag{6.25}$$

where, for a trapezoidal section,

$$R_h = \frac{(b + zy)y}{b + 2y(1 + z^2)^{\frac{1}{2}}}$$

and

$$A = (b + zy)y$$

where y is the depth of flow. With b = 12 ft, and z = 3.5, equation (6.25) is iteratively solved to yield

y = 8.1 ft

so that,

V = 1.37 fps

Calculation of C, K, and Δt. The routing coefficient, C, is calculated by equation (6.20) as

$$C = \frac{1.37\ fps}{1.37 + 1.7\ fps}$$

$$= 0.447$$

The steady flow travel time is given by equation (6.22):

$$K = \frac{75000\ ft}{(1.37\ ft)(3600\ sec/hr)}$$

$$= 15.21\ hours$$

And finally, the flood wave travel time, Δt is calculated using equation (6.21):

$$\begin{aligned} \Delta t &= (0.447)(15.21\ \text{hours}) \\ &= 6.79\ \text{hours} \end{aligned}$$

The inflow hydrograph has points at 6 hour intervals. A more appropriate routing interval would then be

$$\Delta t^* = 6\ \text{hours.}$$

The SCS outlines two methods by which the routing interval can be changed. Both are presented.

Adjust Routing Interval - Method 1. In the first method, the length of the reach remains constant and a new routing coefficient, C*, calculated. The new C is calculated by equation (6.23), where

$$a = \frac{(6\ hr) + 0.5(6.79\ hr)}{1.5(6.79\ hr)}$$

$$= 0.923$$

so that

$$C^{*} = 1 - (1 - 0.447)^{0.923}$$

$$= 0.421$$

Using this coefficient, the routing is completed in Table 6.7. The important item to note is the times calculated for outflow. The original Δt is defined as the "flood wave travel time". If that definition is to remain valid, outflow must begin at $\Delta t = 6.79$ hours. The remaining outflows are spaced at the chosen routing interval, Δt^*. This is shown in the last column of Table 6.7. As a sample, the outflow for routing period 4 is calculated, by equation (6.19), as

$$O_4 = (1 - C^{*})O_3 + CI$$

$$= (1 - 0.421)(4\ cfs) + 0.421(36\ cfs)$$

$$= 18\ cfs$$

<u>*Adjust Routing Interval - Method 2*</u>. In the second method for adjusting the routing period, the given reach is broken into subreaches, in this case two subreaches. The proper length, L^*, for the first subreach is given by equation (6.24):

$$L^{*} = \frac{(6\ hr)(1.37\ fps)}{0.447}\ \frac{3600\ sec}{hr}$$

$$= 66{,}201\ feet$$

The routing for the first subreach is carried out in Table 6.8. The outflow for routing interval 4 will be calculated as an example. Using equation (6.19), for routing interval 4 (superscripts denote subreach number),

$$O_4^1 = (1 - C)O_3^1 + CI_3$$

$$= (1 - 0.447)(4\ cfs) + 0.447(36\ cfs)$$

$= 18.3\ cfs\ \ (as\ shown\ in\ Table\ 6.8)$

The length of the second subreach is then

$$L_2 = L - L^*$$
$$= 75{,}000 - 66{,}201 \text{ feet}$$
$$= 8{,}799 \text{ feet}$$

which is too short to subdivide further, and we must resort to method 1, and calculate a new routing coefficient. The flood wave travel time for subreach is

$$\Delta t_2 = \frac{CL_2}{V}$$

$$= \frac{(0.447)(8799\ feet)}{(1.37\ fps)(3600\ sec/hr)}$$

$$= 0.80\ hours$$

The routing coefficient is then calculated by equation (6.23), using Δt_2 in place of Δt:

$$a_2 = \frac{\Delta t^* + 0.5\Delta t_2}{1.5\Delta t_2}$$

$$= \frac{(6\ hr) + 0.5(0.80\ hr)}{1.5(0.78\ hr)}$$

$$= 5.33$$

and

$$C_2^* = 1 - (1 - 0.447)^{5.33} = 0.96$$

The outflow from subreach 1 is the inflow to subreach 2. Routing for subreach is completed and shown in Table 6.9. Here again the outflow time must be adjusted to reflect the previously defined flood wave time, Δt_2. As an example, the outflow for routing period 4 is calculated by equation (6.19):

$$O_4^2 = (1 - C_2^*)O_3^2 + C_2^*O_3^1$$

$$= (1 - 0.96)(0.0\ cfs) + 0.96(4 cfs)$$

$$= 4\ cfs$$

The results of both methods are compared in Figure 6.7. There are differences in the resulting outflow hydrographs, though not large, between the two methods. Deciding between Method 1 and Method 2 for adjusting the Convex method routing interval is probably a matter of convenience.

TABLE 6.7. ROUTING RESULTS FROM EXAMPLE 6.3 USING METHOD 1 FOR RE-CALCULATING THE ROUTING INTERVAL.

Routing Interval	Inflow Time hr	Inflow cfs	Outflow cfs	Outflow Time hr
1	0	0	0	0
2	6	10	0	6.79
3	12	36	4	12.79
4	18	66	18	18.79
5	24	250	38	24.79
6	30	550	127	30.79
7	36	595	305	36.79
8	42	420	427	42.79
9	48	295	424	48.79
10	54	210	370	54.79
11	60	100	237	60.79
12	66	74	179	66.79
13	72	60	135	72.79
14	78	51	103	78.79
15	84	46	81	84.79
16	90	20	66	90.79
17	96	0	47	96.79

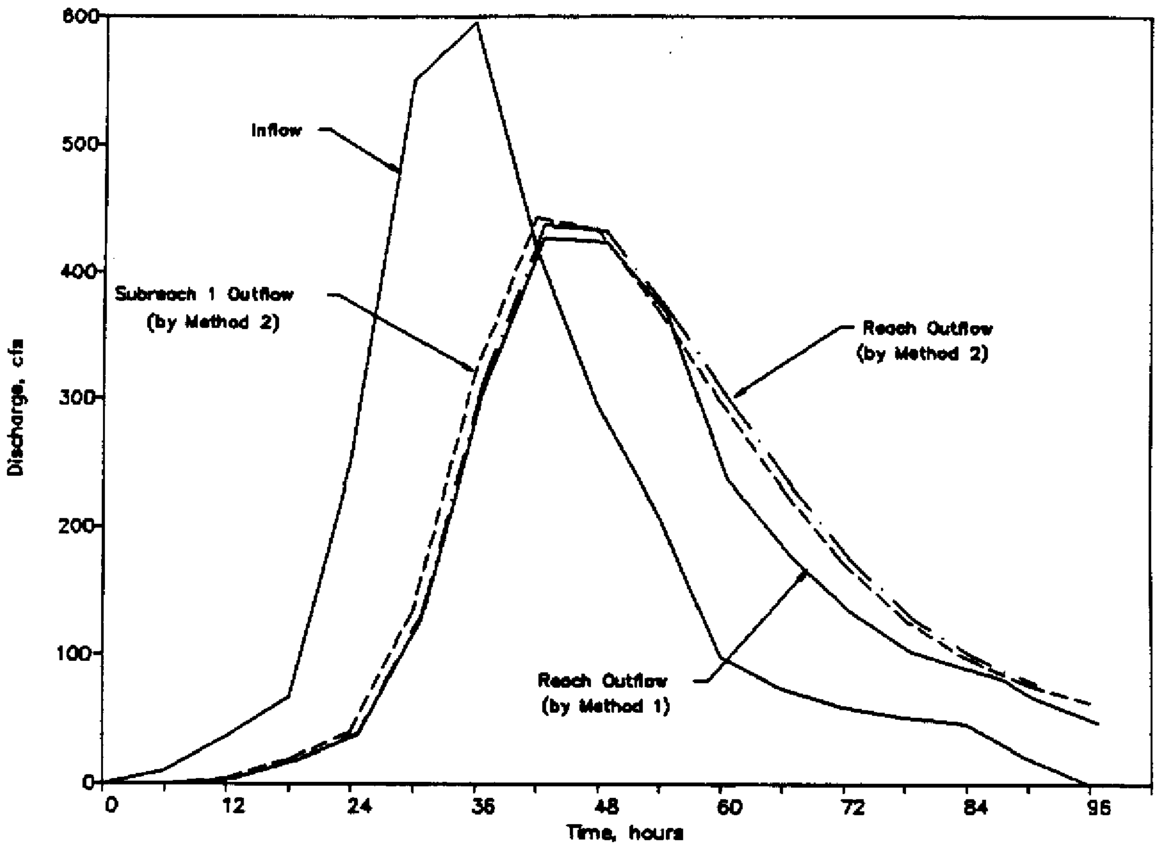

Figure 6.7. Results of Convex method routing completed in Example 6.3.

TABLE 6.8. ROUTING RESULTS FROM EXAMPLE 6.3 USING METHOD 2 FOR ADJUSTING THE ROUTING PERIOD.

Interval	Time (hr)	Inflow (cfs)	Outflow Subreach 1 (cfs)	Outflow Subreach 2 (cfs)	Subreach 2 Outflow Time (hr)
1	0	0	0	0	--
2	6	10	0	0	0
3	12	36	4	0	6.79
4	18	66	19	4	12.79
5	24	250	40	18	18.79
6	30	550	134	39	24.79
7	36	595	320	130	30.79
8	42	420	443	312	36.79
9	48	295	433	438	42.79
10	54	210	371	433	48.79
11	60	100	299	374	54.79
12	66	74	210	302	60.79
13	72	60	149	214	66.79
14	78	51	109	152	72.79
15	84	46	83	111	78.79
16	90	20	67	84	84.79
17	96	0	46	67	90.79

6.3. Hydraulic Routing Methods

6.3.1. GENERAL

Hydraulic routing methods use the Saint Venant equations, in conservative or nonconservative form, to describe the translation of a flood wave through a channel reach. Ideally, any routing problem would be analyzed with a complete solution of the Saint Venant equations (e.g. equations (6.2) and (6.5) in their entirety). However, this is usually either impractical or not necessary. In practice, certain terms of equation (6.5) are assumed unimportant and ignored, and the resulting methods are classified based on which terms have not been used in calculations. One-dimensional hydraulic routing methods are classified into

- kinematic wave,
- diffusion wave, and
- dynamic wave.

Table 6.9 summarizes which terms are included in equation (6.5) for each of these methods.

In all but the most simple cases, a numerical solution (i.e. finding Q, y, or V as a function of x and t) to equations (6.2) and (6.5) is required. The solution requires that certain information be supplied: *initial conditions* and *boundary conditions*. Initial conditions are those conditions (Q, y, V) known to exist at a certain position, x, at the start of the simulation: upstream and downstream stages, for example. Boundary conditions are those (Q, y, V) conditions known to exist at a certain position, x, but which vary with time: an upstream or downstream hydrograph, or a section rating curve, for example. For all simplifications of the Saint Venant equations, solutions

TABLE 6.9. FORCES AND TERMS INCLUDED IN THE CONSERVATION OF MOMENTUM RELATION, EQUATION (6.5), FOR EACH TYPE OF HYDRAULIC ROUTING.

Forces	Local Inertia	Convective Inertia	Pressure	Friction	Gravity
Included Terms	$A^{-1}\{\partial Q/\partial t\}$	$A^{-1}\{\partial(Q^2/A)/\partial x\}$	$g(\partial y/\partial x)$	gSf	-gSo
Hydraulic Routing Method					
kinematic wave				☒	☒
diffusion wave			☒	☒	☒
dynamic wave	☒	☒	☒	☒	☒

require two initial conditions. The number of boundary conditions required, however, will vary depending on the assumptions made.

6.3.1.1. Kinematic Wave

Kinematic wave routing is the simplest and least accurate of the hydraulic routing techniques. The only forces considered are those of friction and gravity, so equation (6.5) is reduced to

$$S_f = S_o \tag{6.26}$$

Numerical solutions to equation (6.26) require only one boundary condition. Typically, this is supplied by flow conditions at the upstream section of the reach. By using only upstream conditions, the kinematic wave method eliminates the need for a downstream boundary condition and, thereby, the mechanism which could be used to account for backwater effects. The approach also implies that channel slopes are steep and the flood wave propagates in the downstream direction only. The reader should refer to Ponce, et. al. (1978) for a complete discussion of the limitations of the kinematic wave approach. The application of the kinematic wave to flat sloped land can cause significant error.

The kinematic approach is also limited since it does not provide any peak attenuation to the flood wave. That is, the hydrograph given as the upstream boundary condition is merely translated through the reach, its shape remains the same. The applicability of the kinematic wave method in South Florida is probably quite limited, and needs further investigation.

6.3.1.2. Diffusion Wave

Diffusion wave routing ignores the "inertial" force terms of equation (6.5), and is called by some "zero-inertia" routing. Equation (6.5), after making the diffusion wave assumptions, is reduced to

$$\frac{dy}{dx} = S_f - S_o \tag{6.27}$$

This is a substantial improvement over the kinematic wave assumptions. Inclusion of the pressure term in the diffusion wave model provides a means by which backwater effects and flood peak attenuation can be described. The method does not account for waves traveling upstream due to downstream disturbances (e.g. automatic gate control, tidal waves, for example), and only steep slopes are considered without error (see Ponce, et. al., 1978).

The diffusion wave approach requires two boundary conditions for a unique solution to equation (6.27). Hence, simultaneous or iterative numerical solutions are required.

6.3.1.3. Dynamic Wave

Dynamic wave routing makes use of the momentum equation in its entirety. Dynamic wave models are applicable to nearly every situation. They are limited, however, by the large amount of information required. Dynamic wave routing is applied when both inertial and pressure forces are important: with mild or flat channel slopes; backwater or changing downstream conditions; and flow reversal (tides).

6.3.2. DYNAMIC WAVE OPERATIONAL MODEL (DWOPER)

DWOPER was developed during the 1970's by the National Weather Service (Fread, 1978). DWOPER is applicable to unsteady flows which are subject to backwater effects, tides, inflow from large tributaries, and when channel bottom slopes are mild.

DWOPER uses the one-dimensional dynamic wave form of the Saint Venant equations, and an implicit finite difference solution technique. Key features of the model are

- generalized information input allowing application to rivers with a variety of physical features;
- ability to use large time steps for slowly varying flows;
- use of irregularly spaced cross-sections along the river system;
- efficient automatic calibration features, for determining optimum roughness coefficients in channel networks

The latest version of the DWOPER is called NETWORK DWOPER (Fread, 1984) and is applicable to storm sewer systems for urban runoff analysis and system design.

DWOPER requires two boundary conditions for each main channel and one upstream boundary condition for each tributary. For the upstream end, this would be a stage or discharge hydrograph. Stage hydrographs, discharge hydrographs, or looped discharge rating curves are required at the downstream section. Initial conditions require a specified discharge and stage at each cross section. Initial stage and discharge at intermediate cross sections can be generated by DWOPER (using a steady-state backwater calculation) if the initial conditions are a steady upstream flow and downstream stage.

Supercritical flow can be analyzed by DWOPER, however, care should be taken whenever there is a transition from supercritical to subcritical. This may change the downstream boundary conditions. In this case, the river reach should be divided into two or more reaches of the same flow regime.

6.3.3. EPA STORMWATER MANAGEMENT MODEL (SWMM)

The original EPA stormwater Management Model was developed from 1969 to 1971. This model was one of the first sophisticated computer models for analyzing both water quantity and non-point source pollution problems in urban areas. The model has been continually maintained and updated, and is the best known and most widely used of the available urban runoff quantity/quality models.

The model structure is constructed in the form of "blocks", as shown in Figure 6.8. The Runoff, Transport, and Extended Transport blocks are of importance here. These are the runoff quantity models, and are discussed below.

6.3.3.1. Runoff Block

The Runoff Block simulates overland flow by storage routing using Manning's equation and the continuity equation. The simulation can be either event oriented or continuous. The method assumes that the hydraulic radius is equal to the depth of flow (i.e. the flow surface is much wider than it is deep), and the depth of flow is constant along the length of the overland flow plane during a given time interval. Depression storage is treated in such a way that overland flow occurs over the entire reach only after depression storage is satisfied. For impervious areas, the depression storage is assigned as zero to simulate immediate runoff. However, care must be taken to insure that impervious areas are hydraulically (directly) connected to the drainage system, otherwise, they should not be treated as impervious. Infiltration on the pervious areas is represented by the Horton's or Green-Ampt equation (see Section 4.5). The SWMM gutter/pipes system can only receive a concentrated inflow, and not a distributed inflow. The total runoff inflow to the gutter/pipes system is computed by flow in the unit width multiplied by the width of the subcatchment. An equivalent width of the subcatchment can be used to adjust the shape of the hydrograph to the recorded one. Generally, the Runoff Block is well suited for the simulation of overland and small pipe or channel flow in the upper regions of the storm sewer system where the assumptions of uniform flow hold and no backwater effects exist.

6.3.3.2. Transport Block

The specific function of the transport subsystem of the storm sewer network is to route surface runoff hydrographs through the network of channels and/or pipes, junctions, flow diversion structures, and storage basins of the main drainage system to the receiving water outfall.

The Transport Block flow routing technique in this block categorizes a sewer system into certain types of "elements". All elements are represented by link and node combinations to form a conceptual representation of the system. Elements may be conduits, manholes, lift stations(pumps), overflow structures. Conduits can be circular, rectangular, or horseshoe shaped. Links may be conduits or open channels, and nodes can be manholes, pumps, and overflow structures. Systems that branch in the downstream direction are modeled using "flow divider" elements. Flow routing then proceeds downstream through all elements during each increment in time until the storm hydrographs have been passed through the system.

The continuity equation (conservation of mass) is applied to each node during each time step, and the kinematic wave approach is applied to each link. As a consequence, backwater effects are not modeled and downstream conditions, such as tide gates and diversion structures are assumed to have no affect on upstream conditions.

6.3.3.3. Extended Transport (EXTRAN) Block

The EXTRAN Block was developed by Camp, Dresser and McKee during 1973 from their study of the proposed master plan for control of combined sewer overflow in San Francisco. EXTRAN has been part of the SWMM package since 1976. The latest documentation of EXTRAN was published in 1981 as a separate addendum to the Version III SWMM Model user's manual published in 1981.

The EXTRAN Block uses a link-node description of the sewer system as with the Transport Block. Properties associated with the links are roughness, cross-sectional area, hydraulic radius, and surface width. Six different types of conduits (links) can be handled by EXTRAN. They are circular, rectangular, horseshoe, egg, basket handle, and trapezoidal. Flow devices such as orifices, weirs, pumps, tide gates, transverse weirs with or without tide gates and side flow weirs with or without tide gates can also be handled by the EXTRAN Block.

The Saint Venant equations [one-dimensional, see equations (6.2) and (6.5)] are applied in the links for flow routing (dynamic wave). Nodes in the EXTRAN Block are the storage elements of the system and correspond to manholes or pipe junctions in the physical system. The variables associated with a node are volume, head, and surface area. The continuity equation is applied to each node during each time step. On-line and off-line storage tanks in the physical system can be handled in the EXTRAN Block as well.

The EXTRAN Block is more applicable to flow networks where surges, flow reversal, and backwater conditions are likely to occur. Some limitations for EXTRAN's use have been noted: (1) Headloss is not explicitly accounted for at transitions (e.g. conduit expansions or contractions, or bridges); and (2) Computational errors occur during surcharging at manholes with different inlet and outlet elevations. It is also more computationally expensive, and more information about the system is required.

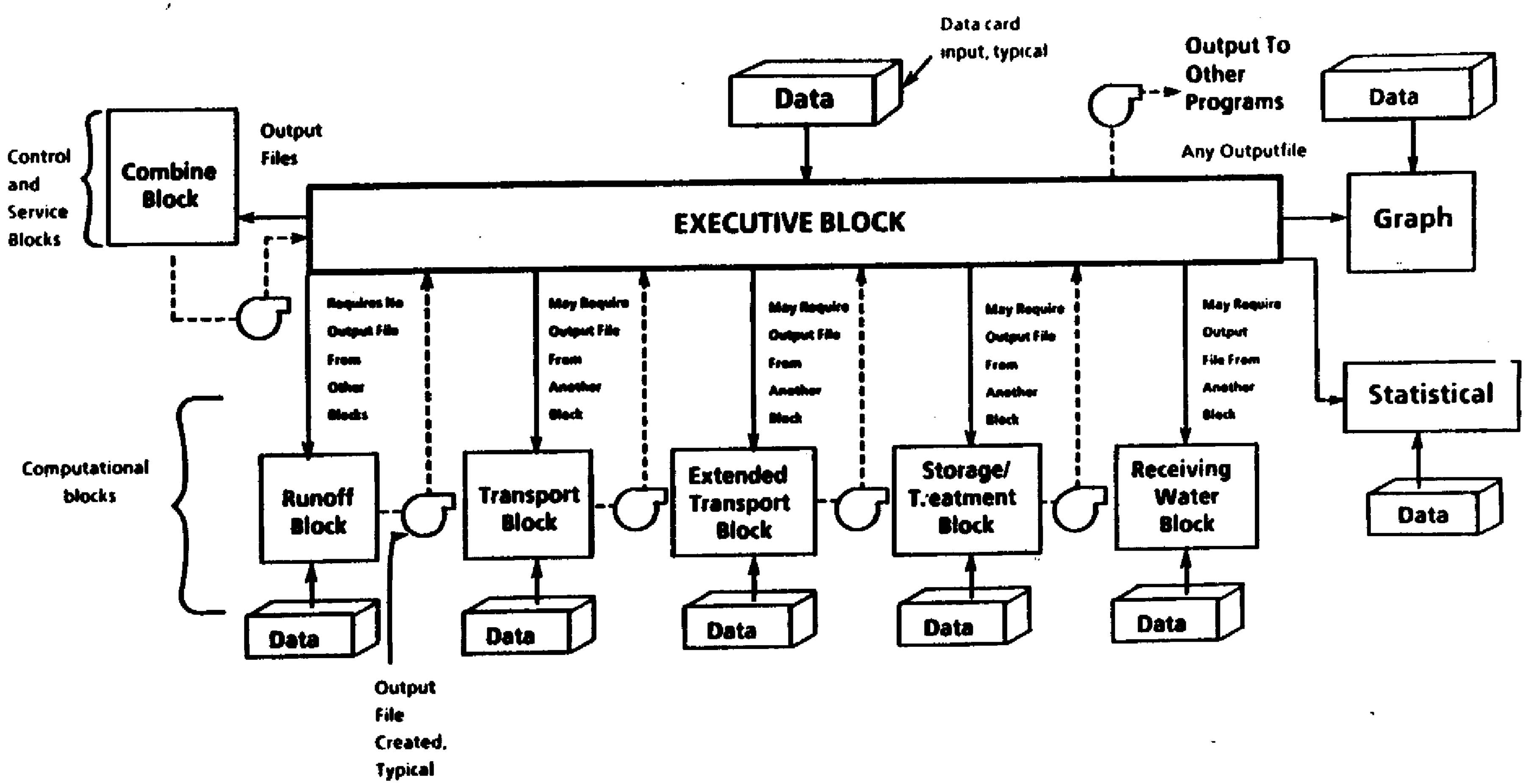

Figure 6.8. Schematic of EPA's Stormwater Management Model.

APPENDIX

A. SCS Hydrologic Soil Groups

The Soil Conservation Service, in conjunction with their Curve Number methods (Sections 2.7 and 4.1), uses a general system of soil classification. In the system, the soil is classified by a letter, A through D, based on the infiltration rate of the soil when it is wet. This classification reflects the runoff potential of the soil during a storm, e.g. a high infiltration capacity leads to a low runoff potential. Briefly, the SCS hydrologic soil groups are as follows:

Group A: These soils are characterized by a high rate of water transmission. They maintain a high infiltration rate even when thoroughly wet. Most are deep, coarse-grained sands or gravel, which are well drained.

Group B: These usually medium to fine textured soils maintain only a moderate infiltration rate when wet. They are typically deep or moderately deep, but have a lower water transmission rate than the soils classified as group A.

Group C: These soils typically have a layer which limits the movement of water downward, or have a fine to moderately fine texture. When wet, their infiltration rate is slow.

Group D: These soils have a very slow infiltration rate when wet. Typically they are clay soils with a high shrink-swell potential. They can also be soils which have a permanently high water table, or hardpan, near the soil surface. The transmission rate of the soils is very low.

In some cases a soil is given a dual classification: one for a drained and another for the undrained profile. These include those profiles classified as A/D (the drained soil is classified in group A and the undrained soil is in group D), or B/D. These classifications are used when there is a limiting layer within the subsoil. This layer can be either a hardpan or high water table. When the profile above this "limiting" layer is dry, infiltration can proceed at a high rate. As the profile becomes wet that infiltration rate slows. Most of these soils have a natural water tables within one foot of the ground surface. This dual classification can be an important factor when a CN is selected. If the area under consideration is drained, or the hardpan has been damaged, the hydrologic soil group may change, which affects CN. The SCS makes no recommendations for CN selection with dual classified soils. Table A.1 shows a suggested scheme.

TABLE A.1. SELECTION OF HYDROLOGIC GROUP FOR SOILS GIVEN A DUAL CLASSIFICATION. These values are subject to revision when more data become available.

Dual Class	Depth to Water (feet)	Depth to Restricted Layer* (feet)	Soil Group
A/D	>4	0 - 1	D
		1 - 2	C
		2 - 4	B
		>4	A
	3 - 4	0 - 1	D
		1 - 2	C
		>3	B
	2 - 3	0 - 1	D
		1 - 2	D
		>2	C
	1 - 2	0 - 2	D
B/D	>4	0 - 1	D
		1 - 2	C
		>2	B
	3 - 4	0 - 1	D
		1 - 2	C
		>2	B
	2 - 3	0 - 1	D
		1 - 2	D
		>2	B
	<2	0 - 2	D

* The "restricted layer" may be either a hardpan, a clay subsoil, or bedrock.

NOMENCLATURE

The following is a partial list of the symbols and abbreviations used in this review. The ones listed here are used consistently throughout. Other symbols may be defined, or these redefined, for use in a particular section.

Δt	time interval, routing period
α	friction coefficient (Akan's Method - Section 2.8)
β	peak rate factor (DUH's - Section 5.2)
β	Horton equation (Section 4.5.2) exponential decay factor
θ	volumetric soil water content
θ_i	initial volumetric soil water content
θs	Green-Ampt volumetric soil water content above the wetting front (Section 4.5.1)
ϕ	soil porosity, angle of channel bottom from horizontal, or parameters for the Tracor method (Section 5.2.3)
A, A_i	area, area for subbasin i
AMC	Antecedent Moisture Condition
C, C_i	constant or runoff coefficient, runoff coefficient for subbasin i
cfs	cubic feet per second
CN, CN', CN_i	Curve Number, same for use with the modified CN method (Section 2.7), same for the ith subbasin
csm/in	cfs per square mile per inch of runoff
D	excess rainfall duration for unit hydrographs (Section 5)
DCUH	Dimensionless Curvilinear Unit Hydrograph (Section 5.2.1.1)
DTUH	Dimensionless Triangular Unit Hydrograph (Section 5.2.1.2)

DUH	Dimensionless Unit Hydrograph
f	depth of infiltration , or infiltration rate
f_c	final infiltration rate
f_o	initial infiltration rate
f_p, $f_p(t)$	infiltration capacity, usually expressed as a function of time t, in inches per hour
F_p, F_p'	swamp and pond factor (SCS peak discharge methods - Section 3.2 to 3.4)
fps	feet per second
$F_p(t)$	cumulative infiltration prior to time t.
GDCUH, GDUH	General Dimensionless(Curvilinear) Unit Hydrograph (Section 5.2.2)
I	reach inflow or rainfall intensity
i	rainfall intensity
I(t)	rate of runoff - SBUH (Section 5.4) "Instantaneous Hydrograph"
I_a	initial abstraction, used in SCS methods
IMP	fraction of basin surface area which is impervious
K	with DUH's the reciprocal of the dimensionless area under the DUH; a storage or other constant for routing methods; or hydraulic conductivity
K'	dimensionless soil parameter (Akan's Method - Section 2.8)
k_o	a constant (Akan's Method - Section 2.8)
K_s	Green-Ampt hydraulic conductivity above the wetting front
L	Runoff flow path length, or reach length
L_g	lag time
n	Manning's roughness coefficient (Manning's n)
NEH-4	National Engineering Handbook, Section 4 (SCS)
O	reach outflow

P, P_i	Precipitation depth, usually 24-hour duration; precipitation depth which has a i year return frequency
P'	dimensionless soil parameter (Akan's Method - Section 2.8)
$P(t)$	precipitation depth at time t
P_f	Green-Ampt soil water tension at the wetting front (Akan's method - Section 2.8, Section 4.5.1)
$Q, Q(t)$	discharge or peak discharge, discharge at time t
q_p	peak discharge (hydrographs - Section 5)
q_u, q_u'	unit peak discharge (discharge per unit area)
R, R_h	hydraulic radius or runoff volume (hydrographs)
$R(t)$	runoff rate at time t
R_{imp}	runoff from impervious basin area (SBUH - Section 5.4)
R_o	total runoff volume, or excess rainfall, usually expressed as a depth of water spread evenly over the basin
R_{perv}	runoff rate from pervious basin surface (SBUH - Section 5.4)
S	surface detention/retention factor, or potential abstraction (SCS). Usually expressed in terms of inches of storage spread evenly over the basin. For the routing methods, reach storage.
SBUH	Santa Barbara Urban Hydrograph (Section 5.4)
SCS	Soil Conservation Service
S_{DWT}	basin storage as a function of water table depth only (SFWMD Runoff Volume Procedure - Section 4.2)
SFWMD	South Florida Water Management District
S_i	initial degree of saturation (Akan's Method - Section 2.8), or reach storage for the ith routing period.
S_o	land slope, ft/ft

t .... time

t_b .... base time for SCS Triangular UH (Section 5.2.1.2)

T_c .... Time of Concentration

T_c' .... dimensionless time of concentration (Akan's Method - Section 2.8)

T_e .... equilibrium time (Akan's Method - Section 2.8)

t_p .... time to peak discharge (hydrographs - Section 5); time to ponding (infiltration methods - Section 4.5)

t_r .... recession time for SCS Triangular UH (Section 5.2.1.2)

T_t, T_{t_i} .... travel time, travel time for the ith reach along the flow path

USACE .... U. S. Army Corps of Engineers

UH .... unit hydrograph

V .... flow velocity

x .... dimensionless area under a DUH

GLOSSARY

abstraction .... natural or artificial means by which a portion of precipitation is lost during the runoff process.

antecedent conditions .... those pertinent basin conditions (e.g. moisture, vegetation, ground water levels, etc.) which exist prior to the runoff event to be examined.

attenuation .... alterations of a (flood) wave hydrograph shape which occur during the translation of the wave downstream. The typical alterations are a lower peak, and a longer duration at the downstream end.

basin .... the area which contributes to the surface water outflow at the defining outflow point.

conservation of mass .... a law of fluid flow physics which states that matter can be neither created nor destroyed.

conservation of momentum .... a law of physics which is a statement of Newton's Second Law for fluid flow - the sum of the external forces acting on a control volume is equal to the rate of change in the control volume's momentum.

continuity .... an extension of the conservation of mass principle for incompressible fluids (fluids which have a constant density); simply stated it means that *volume* is conserved.

control volume .... an imaginary parcel of fluid which serves to isolate an incremental volume in order to analyze it mathematically.

depression storage .... a term used to describe surface runoff which is held temporarily in small puddles or ponds within the basin; a type of abstraction.

detention .... that portion of rainfall which becomes runoff from the basin after being held within the basin as depression storage.

evapotranspiration (ET) .... surface or ground water transferred the atmosphere by free surface evaporation, vegetative transpiration, or both.

groundwater .... water stored in a saturated zone beneath the soil surface.

hydraulic routing .... routing techniques which are based on the physics of fluid flow using principles of both conservation of momentum and conservation of mass.

hydrograph .... a graphical or tabular representation of stage or flow versus time.

hydrologic routing .... routing methods which are based strictly on the conservation of mass principle.

infiltration .... that portion of rainfall which moves into surface soil layers.

infiltration capacity .... the maximum rate, usually expressed as a function of time, at which water can infiltrate the soil.

interception .... that portion of rainfall which is trapped and stored on vegetative or other basin surfaces. This usually evaporates.

interflow .... that portion of infiltration which moves laterally within the surface soil layers and eventually returns to the surface.

lag time .... the time from the centroid of an excess rainfall distribution to the peak flow of the corresponding runoff hydrograph.

peak discharge .... the maximum discharge rate which occurs during a runoff event.

percolation .... that portion of infiltration which moves into lower soil layers and eventually into ground water.

porosity .... the ratio of a soils volume of voids (volume taken by water and air) to the volume of soil solids.

rainfall excess .... direct runoff; that portion of actual rainfall which leaves the basin via surface flow. Alternatively, actual rainfall less runoff losses (infiltration, ET, etc.), usually expressed as a depth.

reach .... a specific length of waterway.

retention .... that portion of rainfall which remains in the watershed following a runoff event. This water is trapped in depression storage, or has infiltrated, or evaporated.

return frequency .... a measure of the likelihood of an event expressed as a number between 0 and 1. Return frequency represents the likelihood of experiencing a hydrologic event, having a given or greater magnitude, in any given year.

return period .... a measure of the likelihood of an event. Return period represents the average time between occurrences of hydrologic events of the same or greater magnitude.

routing	analysis methods by which a hydrograph at one point in a system, or basin, is predicted from a known hydrograph at another point within the same system.
runoff volume	the total amount of runoff which results from a rainfall event, usually expressed in volume units. "runoff volume" and excess rainfall are sometimes used interchangeably.
runoff	that portion of rainfall which leaves a basin via surface flow.
soil storage	water which is held in the soil in an unsaturated zone.
time to ponding	the time during a rainfall event when the rainfall intensity becomes larger than the infiltration capacity, i.e. water can not be infiltrated as fast as it is being supplied.
time of concentration	the time during a storm which must elapse before the entire watershed area contributes to the surface outflow.
unit peak discharge	the maximum discharge which occurs during a runoff event, expressed on a unit area basis.
watershed	see "basin".

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