NOTE:Appendices A-1, A-2 and A-3 will be combined into one appendix.

APPENDIX A

National Engineering Handbook, Part 630, Hydrology (NEH630)

The Soil Conservation Service (SCS, now NRCS) first developed the National Engineering Handbook, Section 4, Hydrology (NEH-4) as a means of documenting the procedures which were being developed for evaluating the hydrology of watersheds in watershed planning projects. Prior to this time (the early 1950s), there was no comprehensive guidance available for such projects. Later, the SCS became the Natural Resources Conservation Service (NRCS) and the NEH-4 was renamed to be the National Engineering Handbook, Part 630, Hydrology (NEH630).

NEH630 documents the technical aspects of the hydrologic methodologies used to develop runoff hydrographs, to a limited extent, to route such hydrographs. The NRCS NEH Part 630 can be downloaded by chapter from the NRCS eDirectives web-site at: **http://directives.sc.egov.usda.gov/viewDirective.aspx?hid=21422**

Technical Release No. 20: Project Formulation – Hydrology (TR-20)

Technical Release No. 20, Project Formulation – Hydrology (TR-20) was developed in the 1960s to automate the hydrologic evaluation of large multi-sub-area watersheds using procedures found in the National Engineering Handbook, Section 4, Hydrology (NEH-4) TR-20 was originally issued as a mainframe computer program designed to run on a Harris mainframe system. In the 1980s, TR-20 was updated to run in a disk-operating system (DOS) environment on a personal computer (PC). Eventually, TR-20 was updated to run in a Windows environment on the PC. In this iteration, the computation engine, TR-20, sits behind the graphical user interface, WinTR-20 which allows users to enter, edit, and display input date; run the TR-20 model; and display output.

TR-20 develops full hydrographs at user specified locations throughout a watershed and allows the user to route the hydrographs through stream channels and structures based on user input rating curves. The TR-20 model has been updated over time to take advantage of advances and updates in hydrologic science. One example of this is the procedure used to route hydrographs through stream channels. The original TR-20 utilized the convex routing procedure. Later that was replaced by the Att-Kin method. The current TR-20/WinTR-20 uses the Muskingum-Cunge method.
The current WinTR-20 computer program and documentation can be downloaded from the following web-site: **http://www.nrcs.usda.gov/wps/portal/nrcs/!ut/p/c4/04_SB-8K8xLLM9MSSzPy8xBz9CP0os_hAE-3NjV08fEwOLsCAXA09PMx_HlBcLY3cDA_2CbEdFAJZtSik!/?ss=16&navtype=TOPNAVIGATION&cid=stelprdb1042198&navid=860000000000000&pnavid=null&position=Not%2520Yet%2520Determined.Html&ttype=detail&pname=USDA%2520NRCS%2520-%2520Natural%2520Resources%2520Conservation%2520Service%2520-%2520National%2520Design,%2520Construction,%2520and%2520Soil%2520Mechanics%2520Center**

Technical Release No. 55 in,Urban Hydrology for Small Watersheds (TR-55)

Technical Release No. 55, Urban Hydrology for Small Watersheds (TR-55) was originally developed in 1975 in response to an increased focus on the analysis of small urbanizing watersheds. The procedures found in the SCS-TP 149, A *Method for Estimating Volume and Rate of Runoff in Small Watersheds* (which later morphed into the Engineering Field Manual Chapter 2, Estimating Runoff Volume and Peak Discharge), and the methodologies found in the NEH-4 were focused on agricultural watersheds.

The curve number tables found in NEH-4 and TP-149 did not cover urban or urbanizing areas. TR-55 expanded the curve number tables to include urban and urbanizing areas.

TR-55 was developed as a manual method by utilizing multiple runs of TR-20 to develop generalized tables and graphs from the output to cover a range of watershed conditions, primarily restricted by time of concentration. Contrary to popular belief, TR-55 was not limited to watersheds of a specific size, but instead was limited to watersheds with times of concentration ranging from 0.1 to 5 hours

. Additionally, the 1975 version of TR-55 covered only areas for which the Type II rainfall distribution was appropriate. The 1986 version of the TR-55 added generalized curves and tables for Types I, IA, and III rainfall distributions and expanded the range of applicability for time of concentration up to 10 hours. A DOS based TR55 computer program was also developed in the 1980s. This computer program was a sort of spreadsheet based program that mirrored the published document.

TR-55 gives the user an estimate of runoff volume and a peak discharge estimate, or in the case of the tabular method, a partial hydrograph bracketing the peak discharge.

NRCS no longer supports (updates) TR-55 and no longer encourages its use. We do understand that TR-55 has gained widespread acceptance and use, so, while it is not available as an official NRCS directive, we do still make it available for download. Information on downloading the 1986 TR-55 and accompanying computer program can be found at the following web-site **http://www.nrcs.usda.gov/wps/portal/nrcs/detailfull/national/technical/alphabetical/water/hydrology/?&cid=stelprdb1042922** which provides links for this and other computer programs no longer supported by NRCS.

The updated WinTR-55 computer program, Small Watershed Hydrology, was developed as a windows update/replacement to the DOS based TR55 computer program. While TR55 was based on generalized tables and graphs to provide an estimate of peak discharge and allowed the user to develop a partial hydrograph, the WinTR-55 computer program uses the latest TR-20 computational engine (behind the WinTR-55 graphical user interface) to compute full hydrographs.

The limitations, now including a drainage area size limitation, placed on WinTR-55 were done so in order to limit its use to watershed similar to those that could be modeled with the DOS based TR55.

A more complete discussion of the differences between TR55 and WinTR-55 can be found in a technical paper The New USDA-NRCS WinTR-55 Small Watershed Hydrology Model by Claudia Scheer and Karl Visser, and presented at the 2002 Federal Interagency Hydrologic Modeling Conference, Las Vegas, NV can be found in the conference proceedings (pp. 404-410) through the following web-site: **http://acwi.gov/hydrology/mtsconfwkshops/conf_proceedings/index.html/** (Please note that the web-links referenced in the paper (including the e-mail addresses) are no longer valid).

The WinTR-55 computer program can be downloaded at:
http://www.nrcs.usda.gov/wps/portal/nrcs/!ut/p/c4/04_SB8K8xLLM9MSSzPy8xBz9CP0os_hAE3NjV08fEwOLsCAXA09P-Mx_HIBcLY3cDA_2CbEdFAJZtSik!/?ss=16&navtype=TOPNAVIGATION&cid=stelprdb1042198&navid=860000000000000&pnavid=null&position=Not%2520Yet%2520Determined.Html&ttype=detail&pname=USDA%2520NRCS%2520-%2520Natural%2520Resources%2520Conservation%2520Service%2520-%2520National%2520Design,%2520Construction,%2520and%2520Soil%2

520Mechanics%2520Center

Additional information on all NRCS hydrologic tools and methodologies can be found at: **http://www.nrcs.usda.gov/wps/portal/nrcs/main/national/technical/alphabetical/water/hydrology.** The "Tools and Models" and "Technical Information" links specifically link to a great deal of additional information.

Win TR-20

Background

Using a 24- hr design storm distribution is standard practice in Win TR-20. In order to best reflect the updated NOAA Atlas 14 precipication data, a site specific distribution iis developed based on the text file download from the NOAA Atlas 14 website. The 24- hr design storm distribution is developed based on maximixing the rainfall during and duration from 5-minutes to 24-hours. The duration from 5 minutes to 24 hours are centered on 12 hours and extended symmetrically for the periods before and after 12 hours. Investigations were conducted which showed that regional storm distribution similar to the prior standard NRCS storm distributions (Type 1, Type IA, Type II and Type III) are not feaseible in states covered by NOAA Atlas 14.

To DATA SMOOTHING TECHNIQUE - several mathematical techniques were investigated to determine a computationally efficient, accurate, practical, stable and robust procedure. Since the generated hydrograph is primarily dependent on the relationship of precipitation intensity with duration, this relationship is what is smoothed. This relationship of intensity (inches/hour) and duration is based on a factor defined as incremental intensity. Incremental intensity is defined as the difference in precipitation divided by the difference in duration. The incremental intensity for the 5-minute duration is equal to the 5-minute precipitation divided by ½ and has the units of inches per hour (or mm/hour in metric units). The incremental intensity for the 10 minute duration is the 10-minute precipitation minus the 5 and 10 minutes in units of hours. Each incremental intensity is calculated based on the difference in precipitation divided by the difference in duration. Incremental intensity is calculated and smoothed for each return period independently.

The final smoothing procedure keeps the 5-minute, 60-minute and 24-hour precipitation unchanged from the original NOAA Atlas 14 values. 10,15,30 and 120-minute and 3,6,12-hour values are open to adjustment. The incremental intensity for the 5-minute duration is unchanged. A straight line on the log-log plot extends from 5-minute to 60–minute duration. A second straight line segment on the log-log plot extends from the 60-minute value to 24-hour value.

CONCLUSION and SUMMARY – the user has the option to develop storm distributions based on the original NOAA Atlas 14 data or smoothed data. Comparing hydrographs generated by original and smoothed data indicated that with the smoothed data, peak discharges may vary by as much as plus or minus 10%.

Overview

The Win TR-20 System Controller/Editor allows running on any of the system components (TR-20 model, input convertor, import NOAA Atlas data, and HEC-RAS reformatter) as well as editing a WinTR-20 input file. The Controller/Editor is organized following the input sections described in the user documentation. For editing, each WinTR-20 input section has its own entry window which is accessible by clicking the input section name on the main window. In addition to the input section entry windows, there are entry windows for locally added land used identifiers (w/ runoff curve numbers by hydrologic soil group) and locally added soils (w/ applicable hydrologic soil groups). Entry windows for these two local additions are accessible from the File pull down on main window.

Help Facilities

Help windows of a general nature on the program system are available via the new user button (available at program start up) or from the Help pull down on the main window. All of the Help windows are available from the pull down while only selected ones are available via the “New User ? Click Here” button.

The data entry window that allow for entry and/or editing of input data contain additional Help in the form of information about the current window and the information about each variable to be entered. This Help is available by clicking the window or variable name on the entry window. A Help box opens in the lower left corner of the entry window and displays the window or variable name, its description and range of values (if appropriate). Only window and variable names shown in yellow have such help available. A second click on the window or variable names closes the Help box.

Getting Started

To EDIT Win TR-20 INPUT FILE- Select one of the first three File pull down choices (New WinTR-20 File, Open Existing WinTR-20 File, and Re-Open Last Session) on the main window. No matter which of the three are selected, the WinTR-20 Identifier entry window appears. Make sure the proper input unit system (English or metric) is selected. Once the information on the window is completed, accept the data by clicking the “Accept Changes (Close)” button. The WinTR-20 Identifier Window will close leaving the main window. Continue by clicking (selecting) another input section entry window from the list on the main window. To save data entered, use the Save or SaveAs selections on the File pull down. Remember to save early and save often.

To CONVERT OLD TR-20 INPUT FILE – Select Convert Old Data from the File pull down on the main window. Then select the file name to be converted to start the converter. When the converter run is complete, either the Error File (indicating a problem with converting the data) will displayed or the WinTR-20 Identifier entry window will open for editing the converted data.

To REFORMAT HEC-RAS from the File pull down on the main window. Then select the HEC-RAS output file name to be reformatted. If a WinTR-20 input file is currently loaded, the choice to either add to the current data or start a new file can also be made. After reformatting is complete, either the Error File will be displayed or the WinTR-20 Identifier entry window will open for editing the file containing the reformatted data.

To IMPORT NOAA ATLAS 14 DATA – Select Import NOAA Data from the File pull down menu on the main window. Then enter NOAA Atlas text file. Do not try to import NOAA Data into WinTR-20 input file if currently loaded, the data will be deleted and substituted with NOAA Atlas data. It is recommended to open a new file to include only NOAA Atlas data.

To RUN WinTR-20 input file using the EDIT, CONVERT, and/or REFORMAT techniques described above. Select WinTR-20 from the Run pull down on the main window to run WinTR-20 model. When the run is complete either the Error file or WinTR-20 output file will be displayed. (Note: The Run pull down is ONLY available if the current data has a file name (not”>Untitled<”) and the data has not been modified since it was loaded or saved. If Run is not displayed, then save the current data to make the Run pull down visible.)

EFH2 peak discharge determination

A program for determining peak discharge as prescribed by Engineering Field Handbook Chapter 2. Required information includes watershed characteristics (drainage area, curve number, hydraulic length, watershed slope) and rainfall amount and distribution.

This program has restricted applications. May be applied when:

- Watershed is accurately represented by a single runoff curve number between 40 and 98.
- Watershed area is between 1 and 2,000 acres.
- Watershed hydraulic length is between 200 and 26,000 feet.
- Average watershed slope is between 0.5 and 64%.
- No valley or reservoir routing is required.
- Urban land use within the watershed does not exceed 10%.

For complete information please visit: http://www.nrcs.usda.gov/wps/portal/nrcs/detailfull/national/water/manage/?cid=stelprdb1042921

~~APPENDIX A-1~~

~~ESTIMATING RUNOFF FROM URBAN AREAS~~

~~This appendix contains the USDA Natural Resources Conservation Service's TR-55, Urban Hydrology for Small Watersheds. The pages, tables, and figures listed in the contents are applicable to this section only. This information and TR-55 software is also available on the Internet at~~

~~http://www. ftw. nrcs.usda.gov/tech_tools.html~~

Preface

Technical Release 55 (TR-55) presents simplified procedures to calculate storm runoff volume, peak rate of discharge, hydrographs, and storage volumes required for floodwater reservoirs. These procedures are applicable in small watersheds, especially urbanizing watersheds, in the United States. First issued by the Soil Conservation Service (SCS) in January 1975, TR-55 incorporates current SCS procedures. This revision includes results of recent research and other changes based on experience with use of the original edition.

The major revisions and additions are-

1. A flow chart for selecting the appropriate procedure;
2. Three additional rain distributions;
3. Expansion of the chapter on runoff curve numbers;
4. A procedure for calculating travel times of sheet flow;
5. Deletion of a chapter on peak discharges;
6. Modifications to the Graphical Peak Discharge method and Tabular Hydrograph method;
7. A new storage routing procedure;
8. Features of the TR-55 computer program; and
9. Worksheets.

This revision was prepared by Roger Cronshey, Hydraulic Engineer, Hydrology Unit, SCS, Washington, DC; Dr. Richard H. McCuen, Professor of Civil Engineering, University of Maryland, College Park, MD; Norman Miller, Head, Hydrology Unit, SCS, Washington, DC; Dr. Walter Rawls, Hydrologist, Agricultural Research Service, Beltsville, MD; Sam Robbins (deceased), formerly Hydraulic Engineer, SCS, South National Technical Center (NTC), Fort Worth, TX; and Don Woodward, Hydraulic Engineer, SCS, Northeast NTC, Chester, PA. Valuable contributions were made by John Chenoweth, Stan Hamilton, William Merkel, Robert Rallison (ret.), Harvey Richardson, Wendell Styner, other SCS hydraulic engineers, and Teresa Seeman.

Revised June 1986

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Metric conversions

The English system of units is used in this TR. To convert to the International System of units (metric), use the following factors:

From English unit	To metric unit	Multiply by
Acre	Hectare	0.405
Square mile	Square kilometer	2.59
Cubic feet per second	Cubic meters per second	0.0283
Inch	Millimeter	25.4
Feet per second	Meters per second	0.3048
Acre-foot	Cubic meter	1233.489
Cubic foot	Cubic meter	0.0283

Perform rounding operations as appropriate to indicate the same level of precision as that of the original measurement. For example:

1. A stream discharge is recorded in cubic feet per second with three significant digits.
2. Convert stream discharge to cubic meters per second by multiplying by 0.0283.
3. Round to enough significant digits so that, when converting back to cubic feet per second, you obtain the original value (step 1) with three significant digits.

Definitions of symbols

Symbol	Unit	Definition
a	ft^2	Cross sectional flow area
A_m	mi^2	Drainage area
CN		Runoff curve number
CN_c		Composite runoff curve number
CN_p		Pervious runoff curve number
E_{max}		Maximum stage
F_p		Pond and swamp adjustment factor
H_w	ft	Head over weir crest
I_a	in	Initial abstraction
L	ft	Flow length
L_w	ft	Weir crest length
m		Number of flow segments
n		Manning's roughness coefficient
P	in	Rainfall
P_{imp}		Percent imperviousness
P_2	in	Two-year frequency, 24-hour rainfall
p_w	ft	Wetted perimeter
q	cfs	Hydrograph coordinate
q_i	cfs	Peak inflow discharge
q_o	cfs	Peak outflow discharge
q_p	cfs	Peak discharge
q_t	csm/in	Tabular hydrograph unit discharge
q_u	csm/in	Unit peak discharge
Q	in	Runoff
r	ft	Hydraulic radius
R		Ratio of unconnected impervious area to total impervious area
s	ft/ft	Slope of hydraulic grade line
S	in	Potential maximum retention after runoff begins
t	hr	Hydrograph time
T_c	hr	Time of concentration
T_p	hr	Time to peak
T_t	hr	Travel time
V	ft/s	Average velocity
V_r	acre-ft, ft^3, or watershed-inch	Runoff volume
V_s	acre-ft, ft^3, or watershed-inch	Storage volume

Chapter 1: Introduction

The conversion of rural land to urban land usually increases erosion and the discharge and volume of storm runoff in a watershed. It also causes other problems that affect soil and water. As part of programs established to alleviate these problems, engineers increasingly must assess the probable effects of urban development, as well as design and implement measures that will minimize its adverse effects.

Technical Release 55 (TR-55) presents simplified procedures for estimating runoff and peak discharges in small watersheds. In selecting the appropriate procedure, consider the scope and complexity of the problem, the available data, and the acceptable level of error. While this TR gives special emphasis to urban and urbanizing watersheds, the procedures apply to any small watershed in which certain limitations are met.

Effects of urban development

An urban or urbanizing watershed is one in which impervious surfaces cover or will soon cover a considerable area. Impervious surfaces include roads, sidewalks, parking lots, and buildings. Natural flow paths in the watershed may be replaced or supplemented by paved gutters, storm sewers, or other elements of artificial drainage.

Hydrologic studies to determine runoff and peak discharge should ideally be based on long-term stationary streamflow records for the area. Such records are seldom available for small drainage areas. Even where they are available, accurate statistical analysis of them is usually impossible because of the conversion of land to urban uses during the period of record. It therefore is necessary to estimate peak discharges with hydrologic models based on measurable watershed characteristics. Only through an understanding of these characteristics and experience in using these models can we make sound judgments on how to alter model parameters to reflect changing watershed conditions.

Urbanization changes a watershed's response to precipitation. The most common effects are reduced infiltration and decreased travel time, which significantly increase peak discharges and runoff. Runoff is determined primarily by the amount of precipitation and by infiltration characteristics related to soil type, soil moisture, antecedent rainfall, cover type, impervious surfaces, and surface retention. Travel time is determined primarily by slope, length of flow path, depth of flow, and roughness of flow surfaces. Peak discharges are based on the relationship of these parameters and on the total drainage area of the watershed, the location of the development, the effect of any flood control works or other natural or manmade storage, and the time distribution of rainfall during a given storm event.

The model described in TR-55 begins with a rainfall amount uniformly imposed on the watershed over a specified time distribution. Mass rainfall is converted to mass runoff by using a runoff curve number (CN). CN is based on soils, plant cover, amount of impervious areas, interception, and surface storage. Runoff is then transformed into a hydrograph by using unit hydrograph theory and routing procedures that depend on runoff travel time through segments of the watershed.

For a description of the hydrograph development method used by SCS, see chapter 16 of the SCS National Engineering Handbook, Section 4-Hydrology (NEH-4) (SCS 1985). The routing method (Modified Att-Kin) is explained in appendixes G and H of draft Technical Release 20 (TR-20) (SCS 1983).

Rainfall

TR-55 includes four regional rainfall time distributions. See appendix B for a discussion of how these distributions were developed.

All four distributions are for a 24-hour period. This period was chosen because of the general availability of daily rainfall data that were used to estimate 24-hour rainfall amounts. The 24-hour duration spans most of the applications of TR-55.

One critical parameter in the model is time of concentration (T_c), which is the time it takes for runoff to travel to a point of interest from the hydraulically most distant point. Normally a rainfall duration equal to or greater than T_c is used. Therefore, the rainfall distributions were designed to contain the intensity of any duration of rainfall for the frequency of the event chosen. That is, if the 10-year frequency, 24-hour rainfall is used, the most intense hour will approximate the 10-year, 1-hour rainfall volume.

Runoff

To estimate runoff from storm rainfall, SCS uses the Runoff Curve Number (CN) method (see chapters 4 through 10 of NEH-4, SCS 1985). Determination of CN depends on the watershed's soil and cover conditions, which the model represents as hydrologic soil group, cover type, treatment, and hydrologic condition. Chapter 2 of this TR discusses the effect of urban development on CN and explains how to use CN to estimate runoff.

Time parameters

Chapter 3 describes a method for estimating the parameters used to distribute the runoff into a hydrograph. The method is based on velocities of flow through segments of the watershed. Two major parameters are time of concentration (T_c) and travel time of flow through the segments (T_t). These and the other parameters used are the same as those used in accepted hydraulic analyses of open channels.

Many methods are empirically derived from actual runoff hydrographs and watershed characteristics. The method in chapter 3 was chosen because it is basic; however, other methods may be used.

Peak discharge and hydrographs

Chapter 4 describes a method for approximating peak rates of discharge, and chapter 5 describes a method for obtaining or routing hydrographs. Both methods were derived from hydrographs prepared by procedures outlined in chapter 16 of NEH-4 (SCS 1985). The computations were made with a computerized SCS hydrologic model, TR-20 (SCS 1983).

The methods in chapters 4 and 5 should be used in accordance with specific guidelines. If basic data are improperly prepared or adjustments not properly used, errors will result.

Storage effects

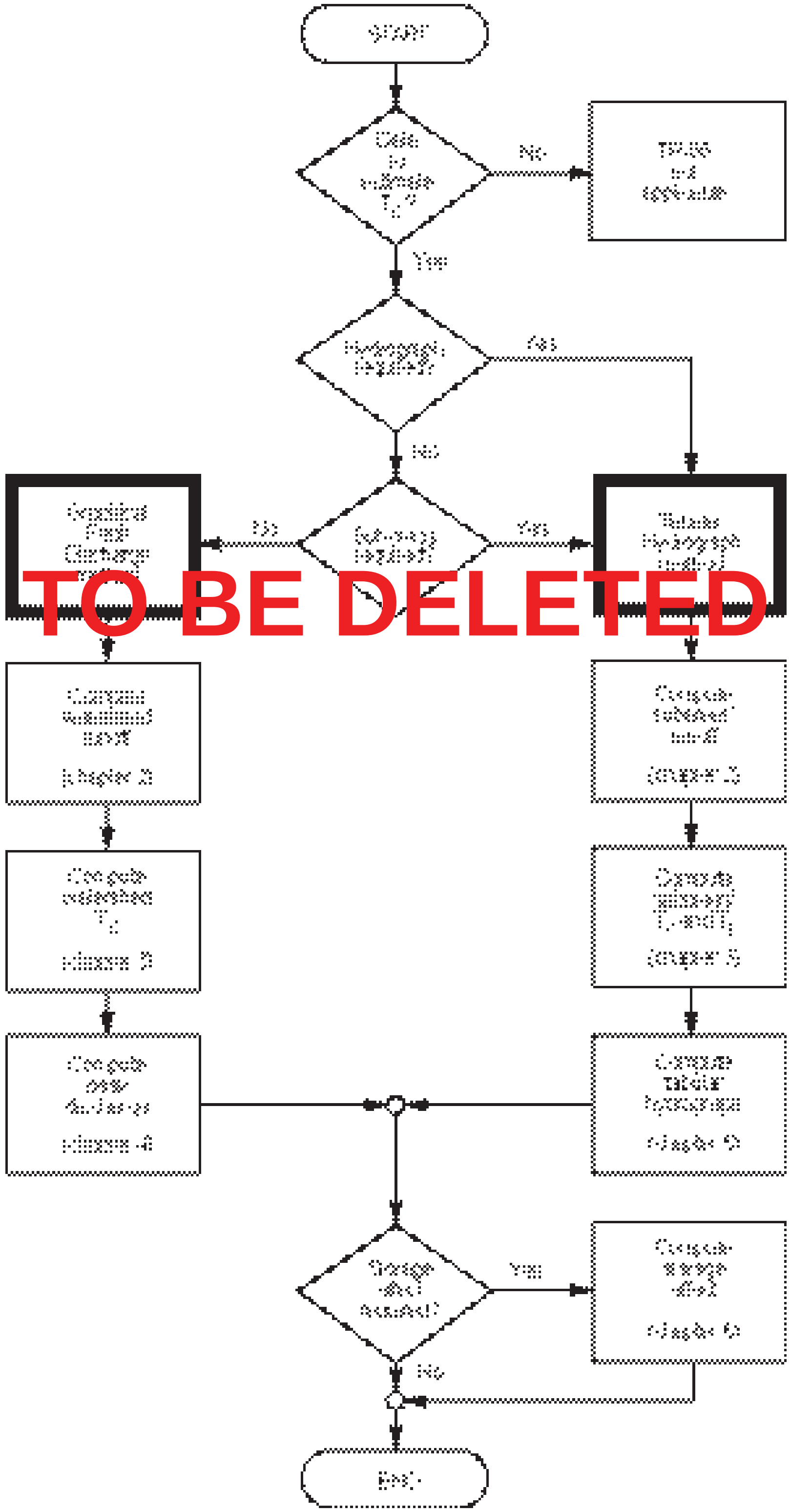
Chapter 6 outlines procedures to account for the effect of detention-type storage. It provides a shortcut method to estimate temporary flood storage based on hydrologic data developed from the Graphical Peak Discharge or Tabular Hydrograph methods.

By increasing runoff and decreasing travel times, urbanization can be expected to increase downstream peak discharges. Chapter 6 discusses how flood detention can modify the hydrograph so that, ideally, downstream peak discharge is reduced approximately to the predevelopment condition. The shortcuts in chapter 6 are useful in sizing a basin even though the final design may require a more detailed analysis.

Selecting the appropriate procedures

Figure 1-1 is a flow chart that shows how to select the appropriate procedures to use in TR-55. In the figure, the diamond-shaped box labeled "Subareas required?" directs the user to the appropriate method based on whether the watershed needs to be divided into subareas. Watershed subdivision is required when significantly different conditions affecting runoff or timing are present in the watershed - for example, if the watershed has widely differing curve numbers or nonhomogeneous slope patterns.

Limitations



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~~Figure 1-1. - Flow Chart for Selecting the Appropriate Procedures in TR-55~~

~~To save time, the procedures in TR-55 are simplified by assumptions about some parameters. These simplifications, however, limit the use of the procedures and can provide results that are less accurate than more detailed methods. The user should examine the sensitivity of the analysis being conducted to a variation of the peak discharge or hydrograph. To ensure that the degree of error is tolerable, specific limitations are given in chapters 2 through 6. Additional general constraints to the use of TR-55 are as follows:~~

- ~~The methods in this TR are based on open and unconfined flow over land or in channels. For large events during which flow is divided between sewer and overland flow, more information about hydraulics than is presented here is needed to determine T_c. After flow enters a closed system, the discharge can be assumed constant until another flow is encountered at a junction or another inlet.~~

- ~~Both the Graphical Peak Discharge and Tabular Hydrograph methods are derived from TR-20 (SCS 1983) output. Their accuracy is comparable; they differ only in their products. The use of T_c permits them to be used for any size watershed within the scope of the curves or tables. The Graphical method (chapter 4) is used only for hydrologically homogeneous watersheds because the procedure is limited to a single watershed subarea. The Tabular method (chapter 5) can be used for a heterogeneous watershed that is divided into a number of homogeneous subwatersheds. Hydrographs for the subwatersheds can be routed and added.~~

- ~~The approximate storage-routing curves (chapter 6) should not be used if the adjustment for ponding (chapter 4) is used. These storage-routing curves, like the peak discharge and hydrograph procedures, are generalizations derived from TR-20 routings.~~

SCS Runoff Curve Number method

The SCS Runoff Curve Number (CN) method is described in detail in NEH-4 (SCS 1985). The SCS runoff equation is

$$Q = \frac{(P - I_a)^2}{(P - I_a) + S} \qquad \text{[Eq. 2-1]}$$

where

Q = runoff (in),
P = rainfall (in),
S = potential maximum retention after runoff begins (in), and
I_a = initial abstraction (in).

Initial abstraction (I_a) is all losses before runoff begins. It includes water retained in surface depressions, water intercepted by vegetation, evaporation, and infiltration. I_a is highly variable but generally is correlated with soil and cover parameters. Through studies of many small agricultural watersheds, I_a was found to be approximated by the following empirical equation:

$$I_a = 0.2S. \qquad \text{[Eq. 2-2]}$$

By removing I_a as an independent parameter, this approximation allows use of a combination of S and P to produce a unique runoff amount. Substituting equation 2-2 into equation 2-1 gives

$$Q = \frac{(P - 0.2S)^2}{(P + 0.8S)}. \qquad \text{[Eq. 2-3]}$$

S is related to the soil and cover conditions of the watershed through the CN. CN has a range of 0 to 100, and S is related to CN by

$$S = \frac{1000}{CN} - 10. \qquad \text{[Eq. 2-4]}$$

Figure 2-1 and table 2-1 solve equations 2-3 and 2-4 for a range of CN's and rainfall.

Factors considered in determining runoff curve numbers

The major factors that determine CN are the hydrologic soil group (HSG), cover type, treatment, hydrologic condition, and antecedent runoff condition (ARC). Another factor considered is whether impervious areas outlet directly to the drainage system (connected) or whether the flow spreads over pervious areas before entering the drainage system (unconnected). Figure 2-2 is provided to aid in selecting the appropriate figure or table for determining curve numbers.

CN's in table 2-2 (a to d) represent average antecedent runoff condition for urban, cultivated agricultural, other agricultural, and arid and semiarid rangeland uses. Table 2-2 assumes impervious areas are directly connected. The following sections explain how to determine CN's and how to modify them for urban conditions.

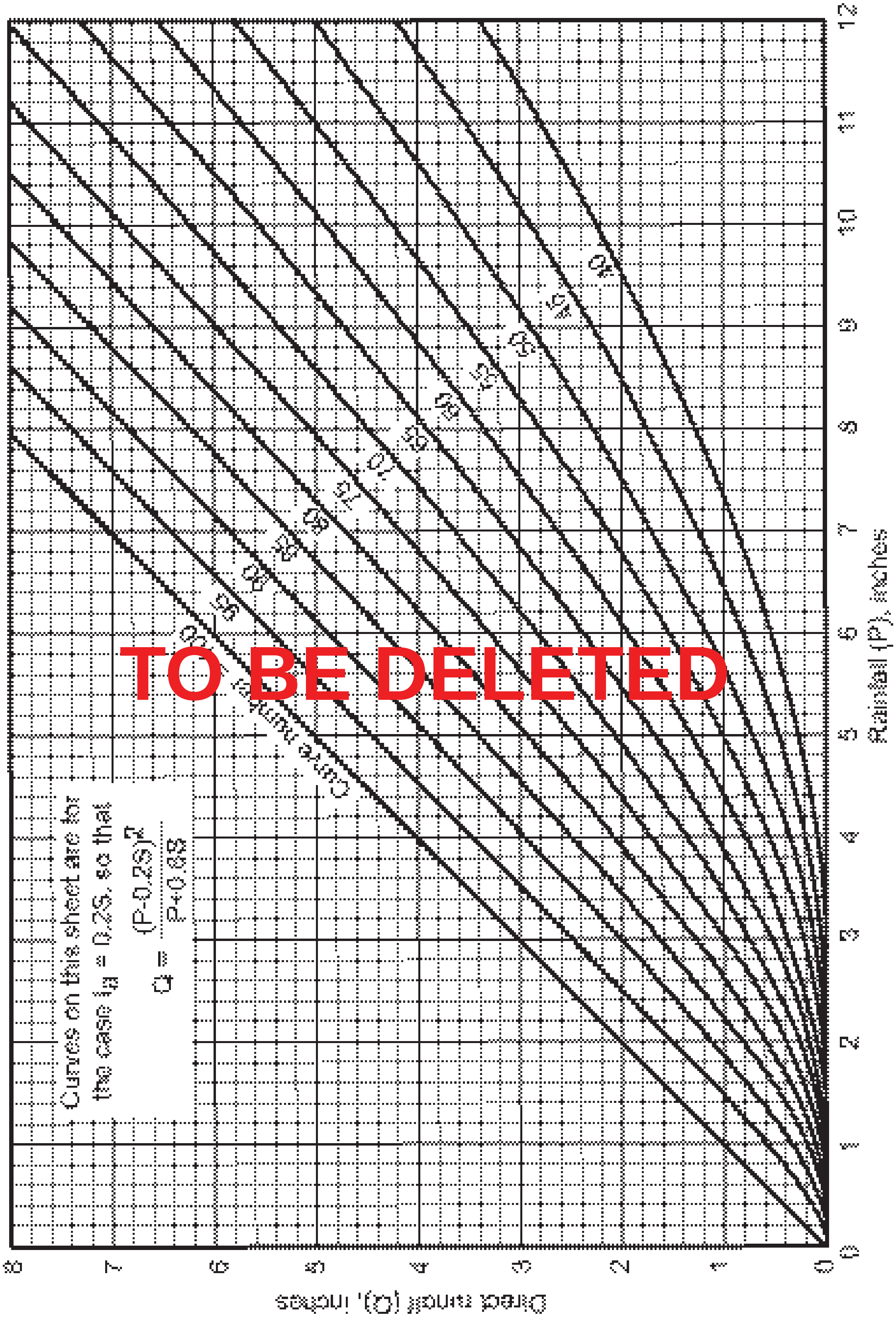
Hydrologic soil groups

Infiltration rates of soils vary widely and are affected by subsurface permeability as well as surface intake rates. Soils are classified into four HSG's (A, B, C, and D) according to their minimum infiltration rate, which is obtained for bare soil after prolonged wetting. Appendix A defines the four groups, and the Manual for Erosion and Sediment Control in Georgia, Appendix B-1, provides a list of most of the soils and their group classifications. The soils in the area of interest may be identified from a soil survey report, which can be obtained from local SCS offices or soil and water conservation district offices.

Most urban areas are only partially covered by impervious surfaces: the soil remains an important factor in runoff estimates. Urbanization has a greater effect on runoff in watersheds with soils having high infiltration rates (sands and gravels) than in watersheds predominantly of silts and clays, which generally have low infiltration rates.

Any disturbance of a soil profile can significantly change its infiltration characteristics. With urbanization, native soil profiles may be mixed or removed or fill material from other areas may be introduced. Therefore, a method based on soil texture is given in appendix A for determining the HSG classification for disturbed soils.

Cover type



TO BE DELETED

~~Figure 2-1. - Solution of Runoff Equation~~

~~Table 2-2 addresses most cover types, such as vegetation, bare soil, and impervious surfaces. There are a number of methods for determining cover type. The most common are field reconnaissance, aerial photographs, and land use maps.~~

~~Treatment~~

~~Treatment is a cover type modifier (used only in table 2-2b) to describe the management of cultivated agricultural lands. It includes mechanical practices, such as contouring and terracing, and management practices, such as crop rotations and reduced or no tillage.~~

~~Hydrologic condition~~

~~Hydrologic condition indicates the effects of cover type and treatment on infiltration and runoff and is generally estimated from density of plant and residue cover on sample areas. Good hydrologic condition indicates that the soil usually has a low runoff potential for that specific hydrologic soil group, cover type, and treatment. Some factors to consider in estimating the effect of cover on infiltration and runoff are (a) canopy or density of lawns, crops, or other vegetative areas; (b) amount of year-round cover; (c) amount of grass or close-seeded legumes in rotations; (d) percent of residue cover; and (e) degree of surface roughness.~~

~~Antecedent runoff condition~~

Table 2-1. - Runoff Depth for Selected CN's and Rainfall Amounts[1]

TO BE DELETED

Rainfall	Runoff depth for curve number of-											
	40	45	50	55	60	65	70	75	80	85	90	95	98
						inches							
1.0	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.03	0.08	0.17	0.32	0.56	0.79
1.2	.00	.00	.00	.00	.00	.00	.03	.07	.15	.27	.46	.74	.99
1.4	.00	.00	.00	.00	.00	.02	.06	.13	.24	.39	.61	.92	1.18
1.6	.00	.00	.00	.00	.01	.05	.11	.20	.34	.52	.76	1.11	1.38
1.8	.00	.00	.00	.00	.03	.09	.17	.29	.44	.65	.93	1.29	1.58
2.0	.00	.00	.00	.02	.06	.14	.24	.38	.56	.80	1.09	1.48	1.77
2.5	.00	.00	.02	.08	.17	.30	.46	.65	.89	1.18	1.53	1.96	2.27
3.0	.00	.02	.09	.19	.33	.51	.71	.96	1.25	1.59	1.98	2.45	2.77
3.5	.02	.08	.20	.35	.53	.75	1.01	1.30	1.64	2.02	2.45	2.94	3.27
4.0	.06	.18	.33	.53	.76	1.03	1.33	1.67	2.04	2.46	2.92	3.43	3.77
4.5	.14	.30	.50	.74	1.02	1.33	1.67	2.05	2.46	2.91	3.40	3.92	4.26
5.0	.24	.44	.69	.98	1.30	1.65	2.04	2.45	2.89	3.37	3.88	4.42	4.76
6.0	.50	.80	1.14	1.52	1.92	2.35	2.81	3.28	3.78	4.30	4.85	5.41	5.76
7.0	.84	1.24	1.68	2.12	2.60	3.10	3.62	4.15	4.69	5.25	5.82	6.41	6.76
8.0	1.25	1.74	2.25	2.78	3.33	3.89	4.46	5.04	5.63	6.21	6.81	7.40	7.76
9.0	1.71	2.29	2.88	3.49	4.10	4.72	5.33	5.95	6.57	7.18	7.79	8.40	8.76
10.0	2.23	2.89	3.56	4.23	4.90	5.56	6.22	6.88	7.52	8.16	8.78	9.40	9.76
11.0	2.78	3.52	4.26	5.00	5.72	6.43	7.13	7.81	8.48	9.13	9.77	10.39	10.76
12.0	3.38	4.19	5.00	5.79	6.56	7.32	8.05	8.76	9.45	10.11	10.76	11.39	11.76
13.0	4.00	4.89	5.76	6.61	7.42	8.21	8.98	9.71	10.42	11.10	11.76	12.39	12.76
14.0	4.65	5.62	6.55	7.44	8.30	9.12	9.91	10.67	11.39	12.08	12.75	13.39	13.76
15.0	5.33	6.36	7.35	8.29	9.19	10.04	10.85	11.63	12.37	13.07	13.74	14.39	14.76

[1] Interpolate the values shown to obtain runoff depths for CN's or rainfall amounts not shown.

TO BE DELETED

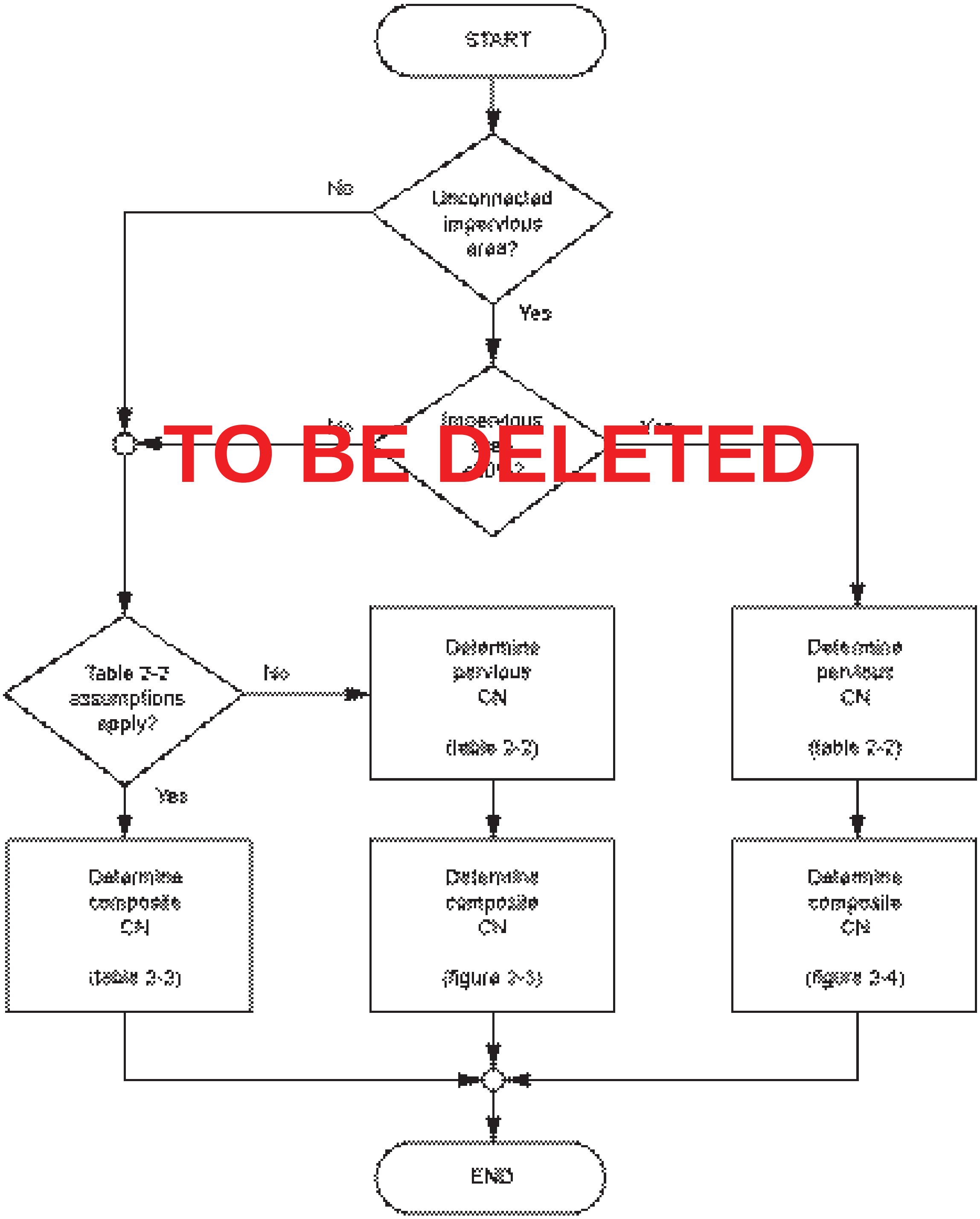


Figure 2-2. - Flow Chart for Selecting the Appropriate Figure or Table for Determining Runoff Curve Numbers

Table 2-2a. - Runoff Curve Numbers for Urban Areas[1]

Cover description		Curve numbers for hydrologic soil group-			
Cover type and hydrologic condition	Average percent impervious area[2]	A	B	C	D
Fully developed urban areas (vegetation established)					
Open space (lawns, parks, golf courses, cemeteries, etc.)[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, roofs, driveways, etc. (excluding right-of-way)		98	98	98	98
Streets and roads:					
Paved; curbs and storm sewers (excluding right-of-way)		98	98	98	98
Paved; open ditches (including right-of-way)		83	89	92	93
Gravel (including right-of-way)		76	85	89	91
Dirt (including right-of-way)		72	82	87	89
Western desert urban areas:					
Natural desert landscaping (pervious areas only)[4]		63	77	85	88
Artificial desert landscaping (impervious weed barrier, desert shrub with 1- to 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)[5]		77	86	91	94
Idle lands (CN's are determined using cover types similar to those in table 2-2c).					

[1] Average runoff condition, and $I_a = 0.2S$.
[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. CN's for other combinations of conditions may be computed using figure 2-3 or 2-4.
[3] CN's shown are equivalent to those of pasture. Composite CN's may be computed for other combinations of open space cover type.
[4] Composite CN's for natural desert landscaping should be computed using figures 2-3 or 2-4 based on the impervious area percentage (CN = 98) and the pervious area CN. The pervious area CN's are assumed equivalent to desert shrub in poor hydrologic condition.
[5] Composite CN's to use for the design of temporary measures during grading and construction should be computed using figure 2-3 or 2-4, based on the degree of development (impervious area percentage) and the CN's for the newly graded pervious areas.

Table 2-2b. - Runoff Curve Numbers for Cultivated Agricultural Lands[1]

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	62	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	SR	Poor	66	77	85	89
or broadcast		Good	58	72	81	85
legumes or	C	Poor	64	75	83	85
rotation		Good	55	69	78	83
meadow	C&T	Poor	63	73	80	83
		Good	51	67	76	80

[1] Average runoff condition, and $I_a = 0.2S$.
[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 ≥ 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 2-2c. - Runoff Curve Numbers for Other Agricultural Lands[1]

Cover description		Curve numbers for hydrologic soil group-			
Cover type	Hydrologic condition	A	B	C	D
Pasture, grassland, or range - continuous forage for grazing.[2]	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	[4]30	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, and $I_a = 0.2S$.

[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 conditions may be computed from the CN's for woods and pasture.

[6] Poor: Forest litter, small trees, 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.

Table 2-2d. - Runoff Curve Numbers for Arid and Semiarid Rangelands[1]

Cover description		Curve numbers for hydrologic soil group-			
Cover type	Hydrologic condition[2]	A[3]	B	C	D
Herbaceous - mixture of grass, weeds, and low-growing brush, with brush the minor element.	Poor		80	87	93
	Fair		71	81	89
	Good		62	74	85
Oak - aspen-mountain brush mixture of oak brush, aspen, mountain mahogany, bitter brush, maple, and other brush.	Poor		66	74	79
	Fair		48	57	63
	Good		30	41	48
Pinyon - juniper-pinyon, juniper, or both; grass understory.	Poor		75	85	89
	Fair		58	73	80
	Good		41	61	71
Sagebrush with grass understory.	Poor		67	80	85
	Fair		51	63	70
	Good		35	47	55
Desert shrub - major plants include saltbush, greasewood, creosotebush, blackbrush, bursage, palo verde, mesquite, and cactus.	Poor	63	77	85	88
	Fair	55	72	81	86
	Good	49	68	79	84

1 Average runoff condition, and $I_a = 0.2S$. For range in humid regions, use table 2-2c.

2 Poor: <30% ground cover (litter, grass, and brush overstory).
Fair: 30 to 70% ground cover.
Good: > 70% ground cover.

3 Curve numbers for group A have been developed only for desert shrub.

The index of runoff potential before a storm event is the antecedent runoff condition (ARC). ARC is an attempt to account for the variation in CN at a site from storm to storm. CN for the average ARC at a site is the median value as taken from sample rainfall and runoff data. The CN's in table 2-2 are for the average ARC, which is used primarily for design applications. See NEH-4 (SCS 1985) and Rallison and Miller (1981) for more detailed discussion of storm-to-storm variation and a demonstration of upper and lower enveloping curves.

Urban impervious area modifications

Several factors, such as the percentage of impervious area and the means of conveying runoff from impervious areas to the drainage system, should be considered in computing CN for urban areas (Rawls et al., 1981). For example, do the impervious areas connect directly to the drainage system, or do they outlet onto lawns or other pervious areas where infiltration can occur?

Connected impervious areas

An impervious area is considered connected if runoff from it flows directly into the drainage system. It is also considered connected if runoff from it occurs as concentrated shallow flow that runs over a pervious area and then into a drainage system.

Urban CN's (table 2-2a) were developed for typical land use relationships based on specific assumed percentages of impervious area. These CN values were developed on the assumptions that (a) pervious urban areas are equivalent to pasture in good hydrologic condition and (b) impervious areas have a CN of 98 and are directly connected to the drainage system. Some assumed percentages of impervious area are shown in table 2-2a.

If all of the impervious area is directly connected to the drainage system, but the impervious area percentages or the pervious land use assumptions in table 2-2a are not applicable, use figure 2-3 to compute a composite CN. For example, table 2-2a gives a CN of 70 for a 1/2-acre lot in HSG B, with an assumed impervious area of 25 percent. However, if the lot has 20 percent impervious area and a pervious area CN of 61, the composite CN obtained from figure 2-3 is 68. The CN difference between 70 and 68 reflects the difference in percent impervious area.

Unconnected impervious areas

Runoff from these areas is spread over a pervious area as sheet flow. To determine CN when all or part of the impervious area is not directly connected to the drainage system, (1) use figure 2-4 if total impervious area is less than 30 percent or (2) use figure 2-3 if the total impervious area is equal to or greater than 30 percent, because the absorptive capacity of the remaining pervious areas will not significantly affect runoff.

When impervious area is less than 30 percent, obtain the composite CN by entering the right half of figure 2-4 with the percentage of total impervious area and the ratio of total unconnected impervious area to total impervious area. Then move left to the appropriate pervious CN and read down to find the composite CN. For example, for a 1/2-acre lot with 20 percent total impervious area (75 percent of which is unconnected) and pervious CN of 61, the composite CN from figure 2-4 is 66. If all of the impervious area is connected, the resulting CN (from figure 2-3) would be 68.

Runoff

When CN and the amount of rainfall have been determined for the watershed, determine runoff by using

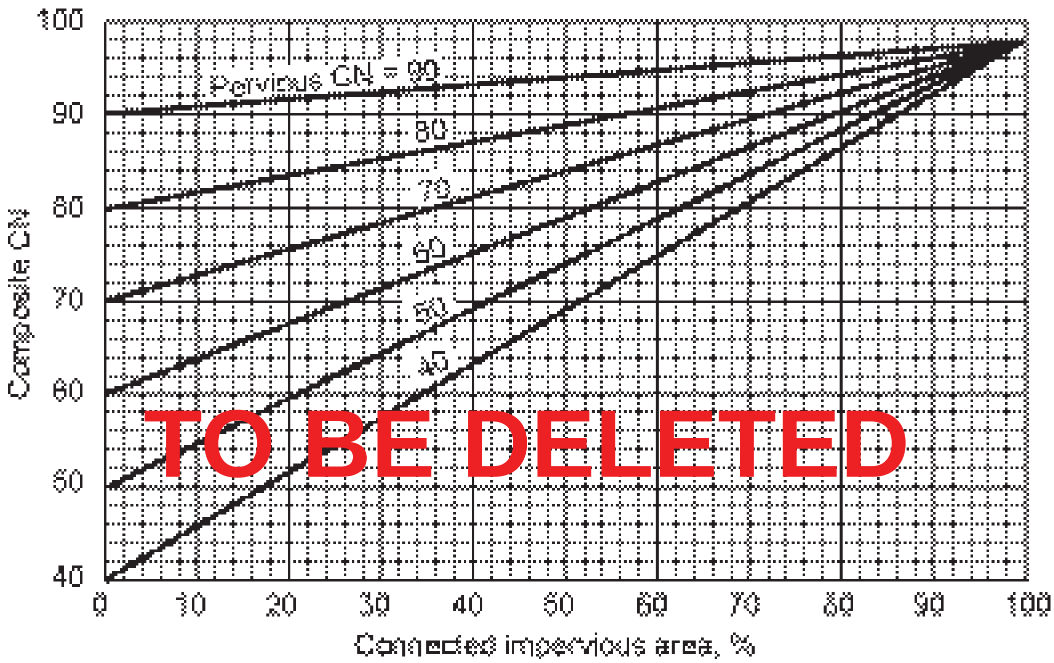


Figure 2-3. - Composite CN with Connected Impervious Area

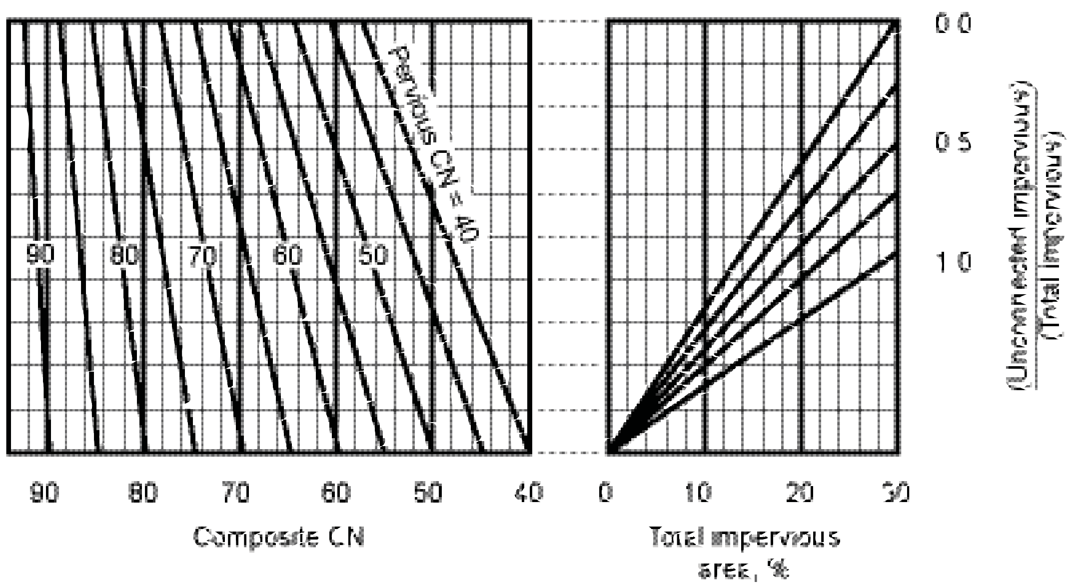


Figure 2-4. - Composite CN with Unconnected Impervious Areas and Total Impervious Area Less Than 30%

figure 2-1, table 2-1, or equations 2-3 and 2-4. The runoff is usually rounded to the nearest hundredth of an inch.

Limitations

- Curve numbers describe average conditions that are useful for design purposes. If the rainfall event used is a historical storm, the modeling accuracy decreases.

- Use the runoff curve number equation with caution when recreating specific features of an actual storm. The equation does not contain an expression for time and, therefore, does not account for rainfall duration or intensity.

- The user should understand the assumption reflected in the initial abstraction term (I_a) and should ascertain that the assumption applies to the situation. I_a, which consists of interception, initial infiltration, surface depression storage, evapotranspiration, and other factors, was generalized as 0.2S based on data from agricultural watersheds (S is the potential maximum retention after runoff begins). This approximation can be especially important in an urban application because the combination of impervious areas with pervious areas can imply a significant initial loss that may not take place. The opposite effect, a greater initial loss, can occur if the impervious areas have surface depressions that store some runoff. To use a relationship other than $I_a = 0.2S$, one must redevelop equation 2-3, figure 2-1, table 2-1, and table 2-2 by using the original rainfall-runoff data to establish new S or CN relationships for each cover and hydrologic soil group.

- Runoff from snowmelt or rain on frozen ground cannot be estimated using these procedures.

- The CN procedure is less accurate when runoff is less than 0.5 inch. As a check, use another procedure to determine runoff.

- The SCS runoff procedures apply only to direct surface runoff: do not overlook large sources of subsurface flow or high ground water levels that contribute to runoff. These conditions are often related to HSG A soils and forest areas that have been assigned relatively low CN's in table 2-2. Good judgment and experience based on stream gage records are needed to adjust CN's as conditions warrant.

- When the weighted CN is less than 40, use another procedure to determine runoff.

Examples

Four examples illustrate the procedure for computing runoff curve number (CN) and runoff (Q) in inches. Worksheet 2 in appendix D is provided to assist TR-55 users. Figures 2-5 to 2-8 represent the use of worksheet 2 for each example. All four examples are based on the same watershed and the same storm event.

The watershed covers 250 acres in Dyer County, northwestern Tennessee. Seventy percent (175 acres) is a Loring soil, which is in hydrologic soil group C. Thirty percent (75 acres) is a Memphis soil, which is in group B. The event is a 25-year frequency, 24-hour storm with total rainfall of 6 inches.

Cover type and conditions in the watershed are different for each example. The examples, therefore, illustrate how to compute CN and Q for various situations of proposed, planned, or present development.

Example 2-1

The present cover type is pasture in good hydrologic condition. (See figure 2-5 for worksheet 2 information.)

Example 2-2

Seventy percent (175 acres) of the watershed, consisting of all the Memphis soil and 100 acres of the Loring soil, is 1/2-acre residential lots with lawns in good hydrologic condition. The rest of the watershed is scattered open space in good hydrologic condition. (See figure 2-6.)

Example 2-3

This example is the same as example 2-2, except that the 1/2-acre lots have a total impervious area of 35 percent. For these lots, the pervious area is lawns in good hydrologic condition. Since the impervious area percentage differs from the percentage assumed in table 2-2, use figure 2-3 to compute CN. (See figure 2-7.)

Example 2-4

This example is also based on example 2-2, except that 50 percent of the impervious area associated with the 1/2-acre lots on the Loring soil is "unconnected," that is, it is not directly connected to the drainage system. For these lots, the pervious area CN (lawn, good condition) is 74 and the impervious area is 25 percent. Use figure 2-4 to compute the CN for these lots. CN's for the 1/2-acre lots on Memphis soil and the open space on Loring soil are the same as those in example 2-2. (See figure 2-8.)

Worksheet 2: Runoff curve number and runoff

Project Heavenly Acres   By WJR   Date 10/1/05

Location Somewhere, Georgia   Checked NM   Date 10/3/05

Circle one: (Present)   Developed

1. Runoff curve number (CN)

TO BE DELETED

Soil name and hydrologic group (appendix A)	Cover description (cover type, treatment, and hydrologic condition; percent impervious; unconnected/connected impervious area ratio)	CN 1/ Table 2-2	CN 1/ Fig. 2-3	CN 1/ Fig. 2-4	Area ☐ acres ☐ mi² ☑ %	Product of CN x area
Memphis, B	Pasture, good condition	61			30	1830
Loring, C	Pasture, good condition	74			70	5180
				Totals =	100	7010

1/ Use only one CN source per line.

CN (weighted) = $\frac{\text{total product}}{\text{total area}} = \frac{7010}{100} = 70.1$;   Use CN = 70

2. Runoff

		Storm #1	Storm #2	Storm #3
Frequency	yr	25		
Rainfall, P (24-hour)	in	6.0		
Runoff, Q (Use P and CN with table 2-1, fig. 2-1, or eqs. 2-3 and 2-4.)	in	2.81		

~~Figure 2-5. - Worksheet 2 for Example 2-1~~

Worksheet 2: Runoff curve number and runoff

Project Heavenly Acres By WJR Date 10/1/05

Location Somewhere, Georgia Checked TJM Date 10/3/05

Circle one: Present (Developed) 175 acres residential

1. Runoff curve number (CN)

TO BE DELETED

Soil name and hydrologic group (appendix A)	Cover description (cover type, treatment, and hydrologic condition; percent impervious; unconnected/connected impervious area ratio)	CN 1/ Table 2-2	CN 1/ Fig. 2-3	CN 1/ Fig. 2-4	Area ☐ acres ☐ mi2 ☐ %	Product of CN x area
Memphis, B	25% impervious 1/2 acre lots, good condition	70			75	5250
Loring, C	25% impervious 1/2 acre lots, good condition	80			100	8000
Loring, C	Open space, good condition	74			75	5550
			Totals =		250	18,800

1/ Use only on CN source per line.

CN (weighted) = $\frac{\text{total product}}{\text{total area}} = \frac{18,800}{250} = 75.2$; Use CN = 75

2. Runoff

		Storm #1	Storm #2	Storm #3
Frequency	yr	25		
Rainfall, P (24-hour)	in	6.0		
Runoff, Q (Use P and CN with table 2-1, fig. 2-1, or eqs. 2-3 and 2-4.)	in	3.28		

~~Figure 2-6. - Worksheet 2 for Example 2-2~~

Worksheet 2: Runoff curve number and runoff

Project Heavenly Acres By WJR Date 10/1/05

Location Somewhere, Georgia Checked TAM Date 10/3/05

Circle one: Present (Developed)

1. Runoff curve number (CN)

TO BE DELETED

Soil name and hydrologic group (appendix A)	Cover description (cover type, treatment, and hydrologic condition; percent impervious; unconnected/connected impervious area ratio)	CN [1/] Table 2-2	Fig. 2-3	Fig. 2-4	Area ☒ acres ☐ mi2 ☐ %	Product of CN x area
Memphis, B	1/2 acre lots, good condition 35% impervious		74		75	5550
Loring, C	1/2 acre lots, good condition 35% impervious		82		100	8200
Loring, C	Open Space, good condition	74			75	5550
				Totals =	250	19,300

1/ Use only one CN source per line.

CN (weighted) = total product / total area = 19,300 / 250 = 77.2 ; Use CN = 77

2. Runoff

		Storm #1	Storm #2	Storm #3
Frequency	yr	25		
Rainfall, P (24-hour)	in	6.0		
Runoff, Q (Use P and CN with table 2-1, fig. 2-1, or eqs. 2-3 and 2-4.)	in	3.48		

Figure 2-7. - Worksheet 2 for Example 2-3

Worksheet 2: Runoff curve number and runoff

Project: Heavenly Acres By: WJR Date: 10/1/05

Location: Somewhere, Georgia Checked: TM Date: 10/3/05

Circle one: Present (Developed)

TO BE DELETED

1. Runoff curve number (CN)

Soil name and hydrologic group (appendix A)	Cover description (cover type, treatment, and hydrologic condition; percent impervious; unconnected/connected impervious area ratio)	CN 1/ Table 2-2	CN 1/ Fig. 2-3	CN 1/ Fig. 2-4	Area ☒ acres ☐ mi² ☐ %	Product of CN x area
Memphis, B	25% connected impervious ½ acre lots, good condition	70			75	5250
Loring, C	25% impervious with 50% unconnected ½ acre lots, good condition			78	100	7800
Loring, C	Open space, good condition	74			75	5550

1/ Use only one CN source per line.

Totals = 250 | 18,600

CN (weighted) = total product / total area = 18,600 / 250 = 74.4 ; Use CN = 74

2. Runoff

		Storm #1	Storm #2	Storm #3
Frequency	yr	25		
Rainfall, P (24-hour)	in	6.0		
Runoff, Q (Use P and CN with table 2-1, fig. 2-1, or eqs. 2-3 and 2-4.)	in	3.19		

Figure 2-8. - Worksheet 2 for Example 2-4

Chapter 3: Time of concentration and travel time

Travel time (T_t) is the time it takes water to travel from one location to another in a watershed. T_t is a component of time of concentration (T_c), which is the time for runoff to travel from the hydraulically most distant point of the watershed to a point of interest within the watershed. T_c is computed by summing all the travel times for consecutive components of the drainage conveyance system.

T_c influences the shape and peak of the runoff hydrograph. Urbanization usually decreases T_c, thereby increasing the peak discharge. But T_c can be increased as a result of (a) ponding behind small or inadequate drainage systems, including storm drain inlets and road culverts, or (b) reduction of land slope through grading.

Factors affecting time of concentration and travel time

Surface roughness

One of the most significant effects of urban development on flow velocity is less retardance to flow. That is, undeveloped areas with very slow and shallow overland flow through vegetation become modified by urban development: the flow is then delivered to streets, gutters, and storm sewers that transport runoff downstream more rapidly. Travel time through the watershed is generally decreased.

Channel shape and flow patterns

In small non-urban watersheds, much of the travel time results from overland flow in upstream areas. Typically, urbanization reduces overland flow lengths by conveying storm runoff into a channel as soon as possible. Since channel designs have efficient hydraulic characteristics, runoff flow velocity increases and travel time decreases.

Slope

Slopes may be increased or decreased by urbanization, depending on the extent of site grading or the extent to which storm sewers and street ditches are used in the design of the water management system. Slope will tend to increase when channels are straightened and decrease when overland flow is directed through storm sewers, street gutters, and diversions.

Computation of travel time and time of concentration

Water moves through a watershed as sheet flow, shallow concentrated flow, open channel flow, or some combination of these. The type that occurs is a function of the conveyance system and is best determined by field inspection.

Travel time (T_t) is the ratio of flow length to flow velocity:

$$T_t = \frac{L}{3600\,V} \qquad \text{[Eq. 3-1]}$$

where

T_t = travel time (hr),
L = flow length (ft),
V = average velocity (ft/s), and
3600 = conversion factor from seconds to hours.

Time of concentration (T_c) is the sum of T_t values for the various consecutive flow segments:

$$T_c = T_{t1} + T_{t2} + \dots T_{tm} \qquad \text{[Eq. 3-2]}$$

where

T_c = time of concentration (hr) and
m = number of flow segments.

Sheet flow

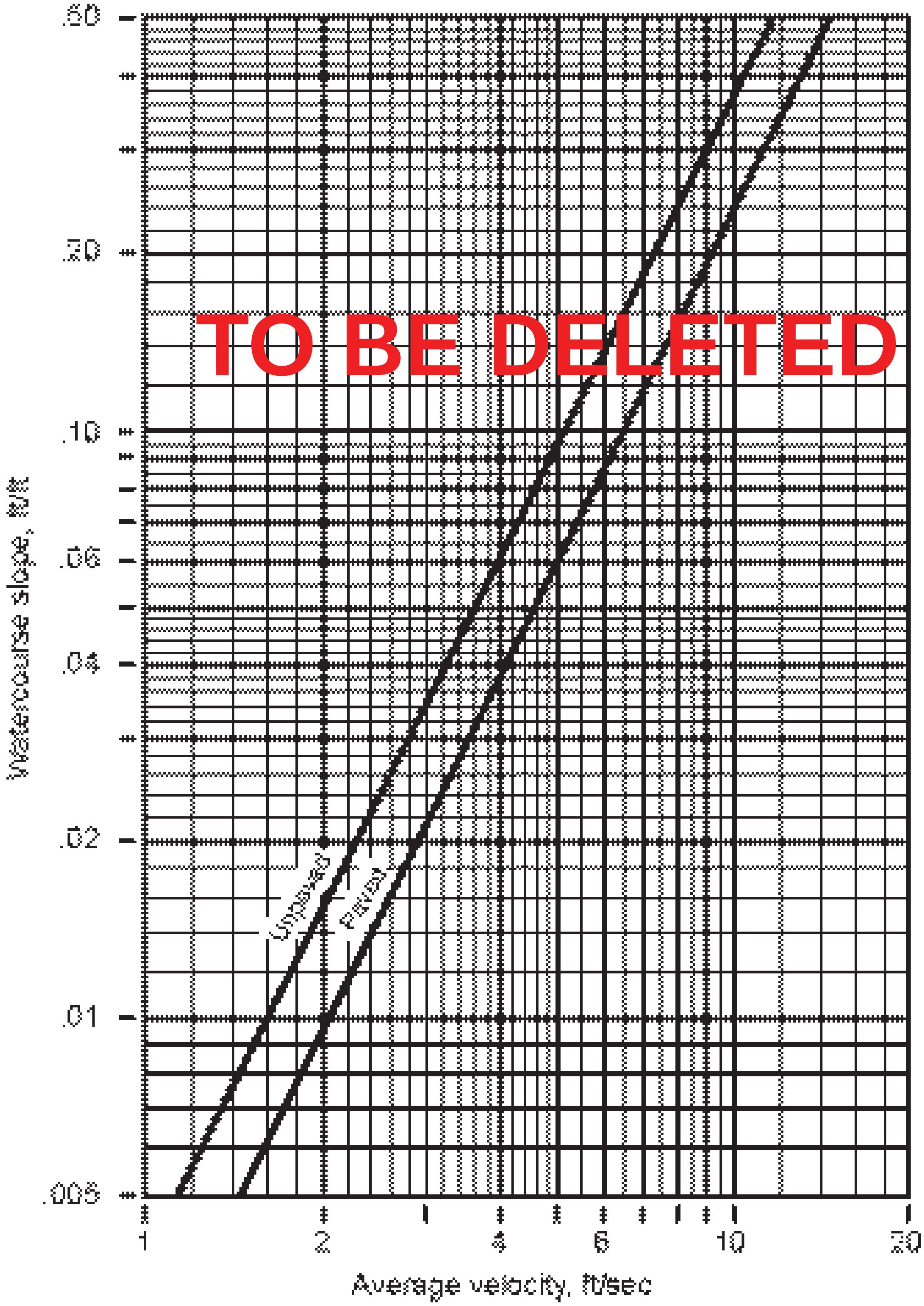


Figure 3-1. - Average Velocities for Estimating Travel Time for Shallow Concentrated Flow

Sheet flow is flow over plane surfaces. It usually occurs in the headwater of streams. With sheet flow, the friction value (Manning's n) is an effective roughness coefficient that includes the effect of raindrop impact; drag over the plane surface; obstacles such as litter, crop ridges, and rocks; and erosion and transportation of sediment. These n values are for very shallow flow depths of about 0.1 foot or so. Table 3-1 gives Manning's n values for sheet flow for various surface conditions.

For sheet flow of less than 300 feet, use Manning's kinematic solution (Overton and Meadows 1976) to compute T_t:

$$T_t = \frac{0.007 (nL)^{0.8}}{(P_2)^{0.5} s^{0.4}} \quad \text{[Eq. 3-3]}$$

where

T_t = travel time (hr),
n = Manning's roughness coefficient (table 3-1),
L = flow length (ft),
P_2 = 2-year, 24-hour rainfall (in), and
s = slope of hydraulic grade line (land slope, ft/ft).

This simplified form of the Manning's kinematic solution is based on the following: (1) shallow steady uniform flow, (2) constant intensity of rainfall excess (that part of a rain available for runoff), (3) rainfall duration of 24 hours, and (4) minor effect of infiltration on travel time. Rainfall depth can be obtained from appendix B.

Shallow concentrated flow

After a maximum of 300 feet, sheet flow usually becomes shallow concentrated flow. The average velocity for this flow can be determined from figure 3-1, in which average velocity is a function of watercourse slope and type of channel. For slopes less than 0.005 ft/ft, use equations given in appendix F for figure 3-1. Tillage can affect the direction of shallow concentrated flow. Flow may not always be directly down the watershed slope if tillage runs across the slope.

After determining average velocity in figure 3-1, use equation 3-1 to estimate travel time for the shallow concentrated flow segment.

Open channels

Open channels are assumed to begin where surveyed cross section information has been obtained, where channels are visible on aerial photographs, or where blue lines (indicating streams) appear on United States Geological Survey (USGS) quadrangle sheets. Manning's equation or water surface profile information can be used to estimate average flow velocity. Average flow velocity is usually determined for bank-full elevation.

Table 3-1. - Roughness Coefficients (Manning's n) for Sheet Flow

Surface description	n[1]
Smooth surfaces (concrete, asphalt, gravel, or bare soil)	0.011
Fallow (no residue)	0.05
Cultivated soils:	
Residue cover ≥20%	0.06
Residue cover >20%	0.17
Grass:	
Short grass prairie	0.15
Dense grasses[2]	0.24
Bermudagrass	0.41
Range (natural)	0.13
Woods:[3]	
Light underbrush	0.40
Dense underbrush	0.80

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

Manning's equation is

$$V = \frac{1.49\, r^{2/3} s^{1/2}}{n} \quad \text{[Eq. 3-4]}$$

where

V = average velocity (ft/s),
r = hydraulic radius (ft) and is equal to a/p_w,
a = cross sectional flow area (ft^2),
p_w = wetted perimeter (ft),
s = slope of the hydraulic grade line (channel slope, ft/ft), and
n = Manning's roughness coefficient for open channel flow.

Manning's n values for open channel flow can be obtained from standard textbooks such as Chow (1959) or Linsley et al. (1982). After average velocity is computed using equation 3-4, T_t for the channel segment can be estimated using equation 3-1.

Reservoirs or lakes

Sometimes it is necessary to estimate the velocity of flow through a reservoir or lake at the outlet of a watershed. This travel time is normally very small and can be assumed as zero.

Limitations

- Manning's kinematic solution should not be used for sheet flow longer than 300 feet. Equation 3-3 was developed for use with the four standard rainfall intensity-duration relationships.

- In watersheds with storm sewers, carefully identify the appropriate hydraulic flow path to estimate T_c. Storm sewers generally handle only a small portion of a large event. The rest of the peak flow travels by streets, lawns, and so on, to the outlet. Consult a standard hydraulics textbook to determine average velocity in pipes for either pressure or nonpressure flow.

- The minimum T_c used in TR-55 is 0.1 hour.

- A culvert or bridge can act as a reservoir outlet if there is significant storage behind it. The procedures in TR-55 can be used to determine the peak flow upstream of the culvert. Detailed storage routing procedures should be used to determine the outflow through the culvert.

Example 3-1

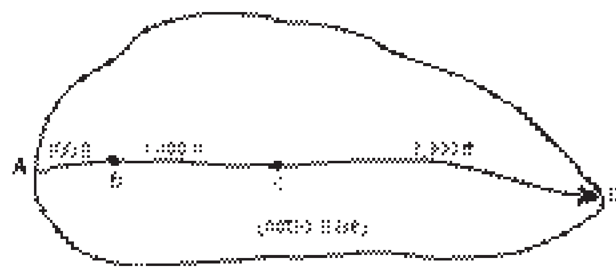
The sketch below shows a watershed in Dyer County, northwestern Tennessee. The problem is to compute T_c at the outlet of the watershed (point D). The 2-year 24-hour rainfall depth is 3.6 inches. All three types of flow occur from the hydraulically most distant point (A) to the point of interest (D). To compute T_c, first determine T_t for each segment from the following information:

Segment AB: Sheet flow; dense grass; slope (s) = 0.01 ft/ft; and length (L) = 100 ft.

Segment BC: Shallow concentrated flow; unpaved; s = 0.01 ft/ft; and L = 1400 ft.

Segment CD: Channel flow; Manning's n =.05; flow area (a) = 27 ft^2; wetted perimeter (p_w) = 28.2 ft; s = 0.005 ft/ft; and L = 7300 ft.

See figure 3-2 for the computations made on worksheet 3.



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Worksheet 3: Time of concentration (T_c) or travel time (T_t)

Project: Heavenly Acres By: DW Date: 10/6/05

Location: Somewhere, Georgia Checked: [illegible] Date: 10/8/05

Circle one: Present (Developed)

Circle one: (T_c) T_t through subarea

Notes: Space for as many as two segments per flow type can be used for each worksheet.

Include a map, schematic, or description of flow segments

Sheet flow (Applicable to T_c only)

		Unit			
	Segment ID		AB		
1.	Surface description (table 3-1)		Dense Grass		
2.	Manning's roughness coeff., n (table 3-1)		0.24		
3.	Flow length, L (total L ≤ 300 ft)	ft	100		
4.	Two-yr 24-hr rainfall, P_2	in	3.6		
5.	Land slope, s	ft/ft	0.01		
6.	$T_t = \frac{0.007\,(nL)^{0.8}}{P_2^{\,0.5}\, s^{0.4}}$ Compute T_t	hr	0.30	+	= 0.30

Shallow concentrated flow

		Unit			
	Segment ID		BC		
7.	Surface description (paved or unpaved)		Unpaved		
8.	Flow length, L	ft	1400		
9.	Watercourse slope, s	ft/ft	0.01		
10.	Average velocity, V (figure 3-1)	ft/s	1.6		
11.	$T_t = \frac{L}{3600\,V}$ Compute T_t	hr	0.24	+	= 0.24

Channel flow

		Unit			
	Segment ID		CD		
12.	Cross sectional flow area, a	ft^2	27		
13.	Wetted perimeter, p_w	ft	28.2		
14.	Hydraulic radius, $r = \frac{a}{p_w}$ Compute r	ft	0.957		
15.	Channel slope, s	ft/ft	0.005		
16.	Manning's roughness coeff., n		0.05		
17.	$V = \frac{1.49\, r^{2/3}\, s^{1/2}}{n}$ Compute V	ft/s	2.05		
18.	Flow length, L	ft	7300		
19.	$T_t = \frac{L}{3600\,V}$ Compute T_t	hr	0.99	+	= 0.99
20.	Watershed or subarea T_c or T_t (add T_t in steps 6, 11, and 19)	hr			1.53

Figure 3-2. - Worksheet 3 for Example 3-1

Chapter 4: Graphical Peak Discharge method

This chapter presents the Graphical Peak Discharge method for computing peak discharge from rural and urban areas. The Graphical method was developed from hydrograph analyses using TR-20, "Computer Program for Project Formulation - Hydrology" (SCS 1983). The peak discharge equation used is

$$q_p = q_u A_m Q F_p \qquad \text{[Eq. 4-1]}$$

where

- q_p = peak discharge (cfs);
- q_u = unit peak discharge (csm/in);
- A_m = drainage area (mi^2);
- Q = runoff (in); and
- F_p = pond and swamp adjustment factor.

The input requirements for the Graphical method are as follows: (1) T_c (hr), (2) drainage area (mi^2), (3) appropriate rainfall distribution (I, IA, II, or III), (4) 24-hour rainfall (in), and (5) CN. If pond and swamp areas are spread throughout the watershed and are not considered in the T_c computation, an adjustment for pond and swamp areas is also needed.

Peak discharge computation

For a selected rainfall frequency, the 24-hour rainfall (P) is obtained from appendix B or more detailed local precipitation maps. CN and total runoff (Q) for the watershed are computed according to the methods outlined in chapter 2. The CN is used to determine the initial abstraction (I_a) from table 4-1. I_a/P is then computed.

If the computed I_a/P ratio is outside the range shown in exhibit 4 (4-I, 4-IA, 4-II, and 4-III) for the rainfall distribution of interest, then the limiting value should be used. If the ratio falls between the limiting values, use linear interpolation. Figure 4-1 illustrates the sensitivity of I_a/P to CN and P.

Peak discharge per square mile per inch of runoff (q_u) is obtained from exhibit 4-I, 4-IA, 4-II, or 4-III by using T_c (chapter 3), rainfall distribution type, and I_a/P ratio. The pond and swamp adjustment factor is obtained from table 4-2 (rounded to the nearest table value). Use worksheet 4 in appendix D to aid in computing the peak discharge using the Graphical method.

Limitations

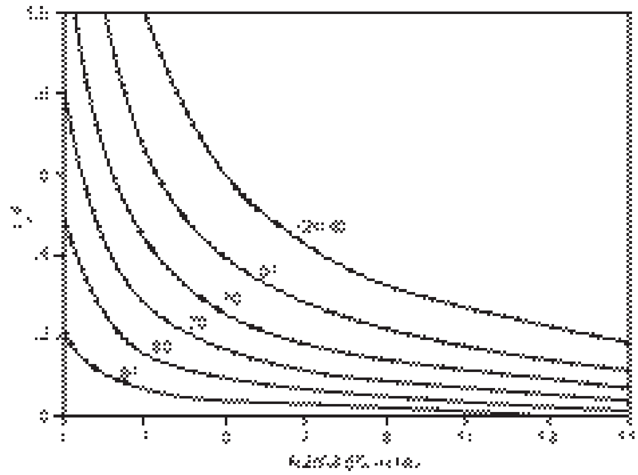

Figure 4-1. - Variation of I_a/P for P and CN

Table 4-1. - I_a Values for Runoff Curve Numbers

CURVE NUMBER	I_a (IN)	CURVE NUMBER	I_a (IN)
40	3.000	70	0.857
41	2.878	71	0.817
42	2.762	72	0.778
43	2.651	73	0.740
44	2.545	74	0.703
45	2.444	75	0.667
46	2.348	76	0.632
47	2.255	77	0.597
48	2.167	78	0.564
49	2.082	79	0.532
50	2.000	80	0.500
51	1.922	81	0.469
52	1.846	82	0.439
53	1.774	83	0.410
54	1.704	84	0.381
55	1.636	85	0.353
56	1.571	86	0.326
57	1.509	87	0.299
58	1.448	88	0.273
59	1.390	89	0.247
60	1.333	90	0.222
61	1.279	91	0.198
62	1.226	92	0.174
63	1.175	93	0.151
64	1.125	94	0.128
65	1.077	95	0.105
66	1.030	96	0.083
67	0.985	97	0.062
68	0.941	98	0.041
69	0.899		

Table 4-2. - Adjustment Factor (Fp) for Pond and Swamp Areas that are Spread throughout the Watershed

Percentage of pond and swamp areas	F_p
0	1.00
0.2	0.97
1.0	0.87
3.0	0.75
5.0	0.72

The Graphical method provides a determination of peak discharge only. If a hydrograph is needed or watershed subdivision is required, use the Tabular Hydrograph method (chapter 5). Use TR-20 if the watershed is very complex or a higher degree of accuracy is required.

- The watershed must be hydrologically homogeneous, that is, describable by one CN. Land use, soils, and cover are distributed uniformly throughout the watershed.
- The watershed may have only one main stream or, if more than one, the branches must have nearly equal T_c's.
- The method cannot perform valley or reservoir routing.
- The F_p factor can be applied only for ponds or swamps that are not in the T_c flow path.
- Accuracy of peak discharge estimated by this method will be reduced if I_a/P values are used that are outside the range given in exhibit 4. The limiting I_a/P values are recommended for use.
- This method should be used only if the weighted CN is greater than 40.
- When this method is used to develop estimates of peak discharge for both present and developed conditions of a watershed, use the same procedure for estimating T_c.
- T_c values with this method may range from 0.1 to 10 hours.

Example 4-1

Compute the 25-year peak discharge for the 250-acre watershed described in examples 2-2 and 3-1. Figure 4-2 shows how worksheet 4 is used to compute q_p as 345 cfs.

Worksheet 4: Graphical Peak Discharge method

Project Heavenly Acres By RHM Date 10/15/05

Location: Somewhere, Georgia Checked TTM Date 10/17/05

Circle one: Present (Developed)

1. Data:

Drainage area . . . . A_m = 0.39 mi^2 (acres/640)

Runoff curve number . . . CN = 75 (From worksheet 2) Figure 2-6

Time of concentration . . . . . T_c = 1.53 (From worksheet 3) Figure 3-2

Rainfall distribution type = II (I, IA, II, III)

Pond and swamp areas spread throughout watershed . . . . = -- percent of A_m (-- acres or mi^2 covered)

	Unit	Storm #1	Storm #2	Storm #3
2. Frequency	yr	25		
3. Rainfall, P (24-hour)	in	6.0		
4. Initial abstraction, I_a . . . . . (Use CN with table 4-1.)	in	0.667		
5. Compute I_a/P		0.11		
6. Unit peak discharge, q_u . . . . . (Use T_c and I_a/P with exhibit 4-__)	csm/in	270		
7. Runoff, Q . . . . . (From worksheet 2).	in	3.28		
8. Pond and swamp adjustment factor, F_p . . . . . (Use percent pond and swamp area with table 4-2. Factor is 1.0 for zero percent pond and swamp area.)		1.0		
9. Peak discharge, q_p . . . . . (Where $q_p = q_u A_m Q F_p$)	cfs	345		

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~~Figure 4-2. - Worksheet 4 for Example 4-1~~

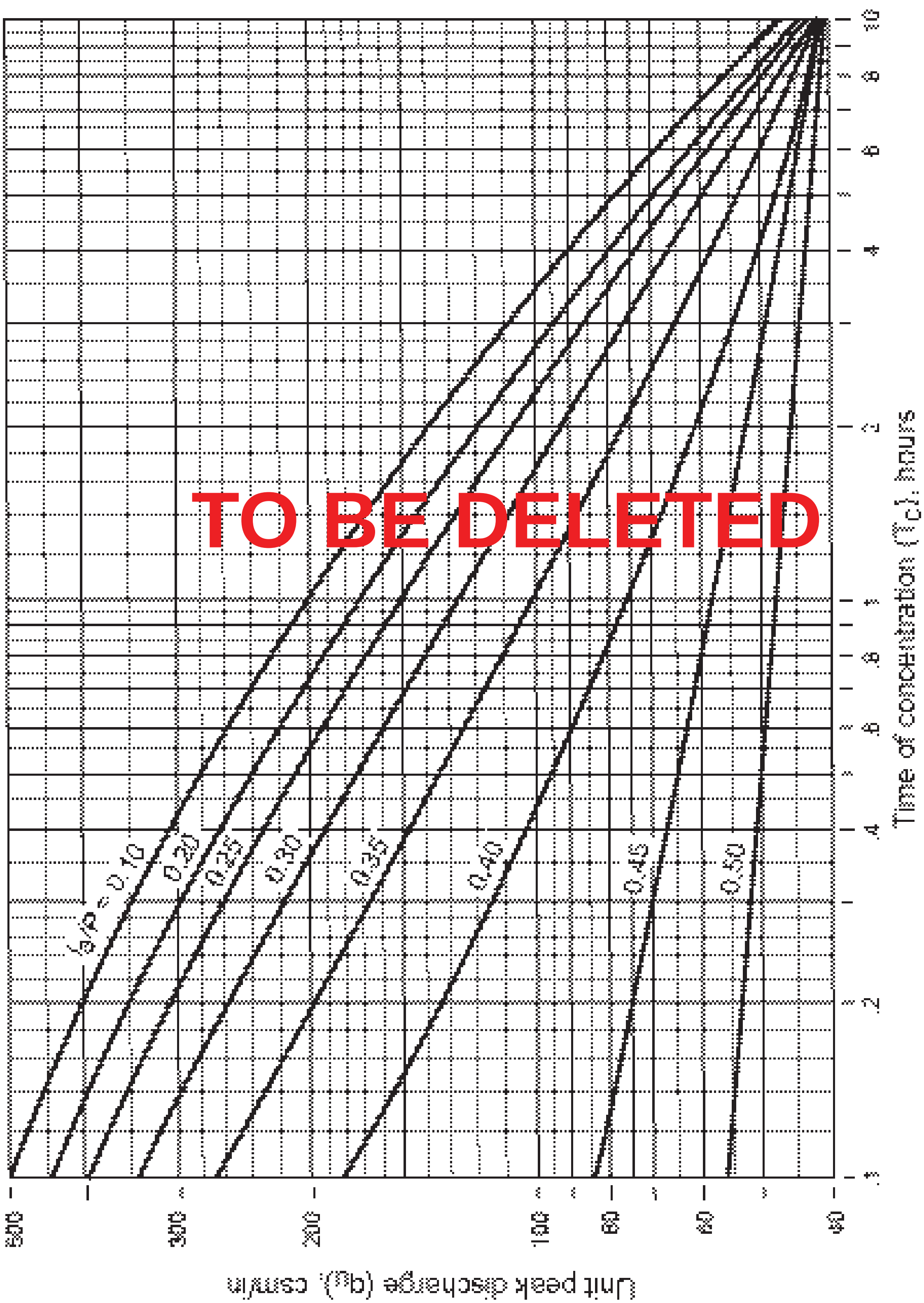

Exhibit 4-I: Unit Peak Discharge (q_u) for SCS Type I Rainfall Distribution

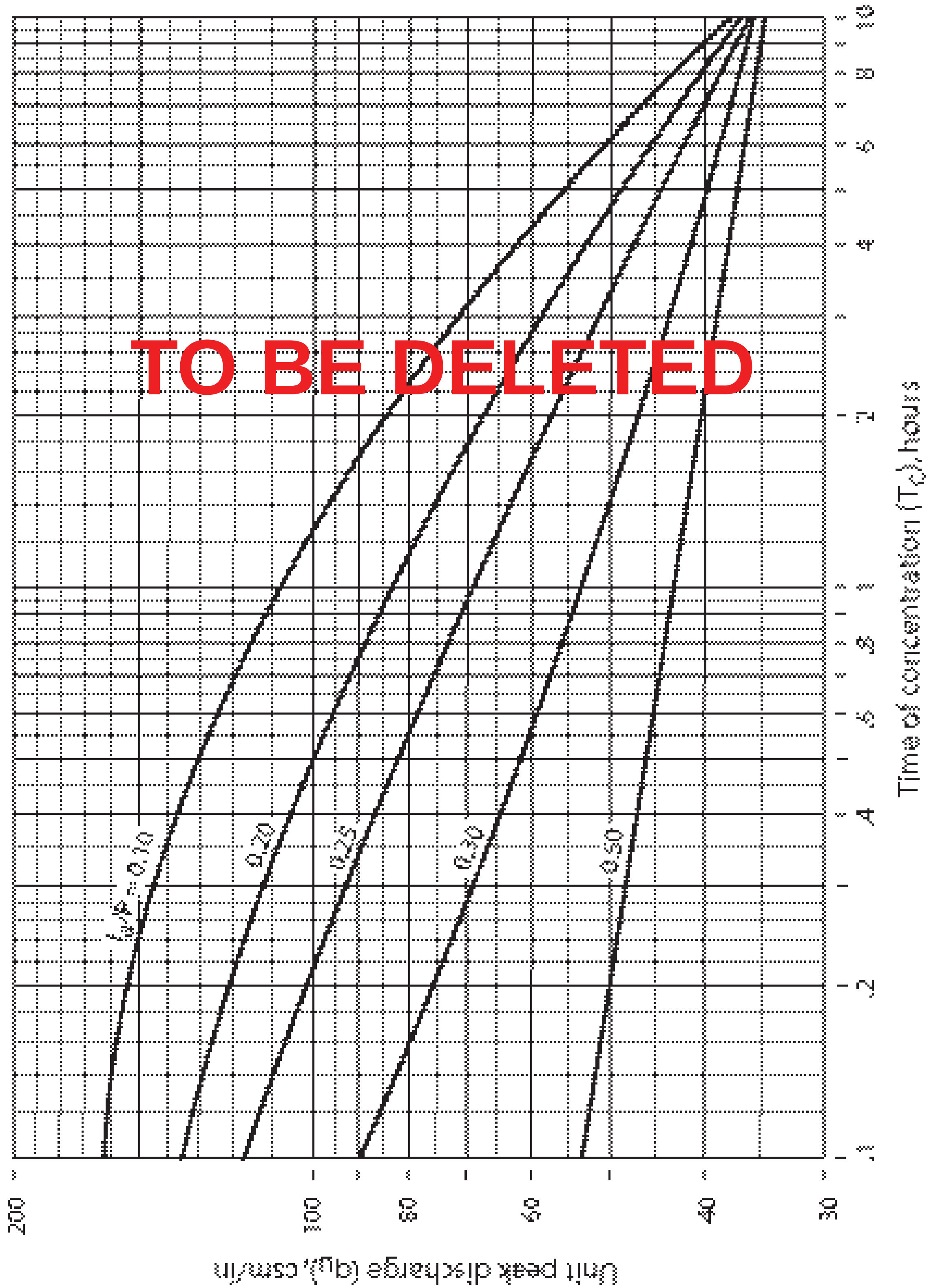

Exhibit 4-IA: Unit Peak Discharge (q_u) for SCS Type IA Rainfall Distribution

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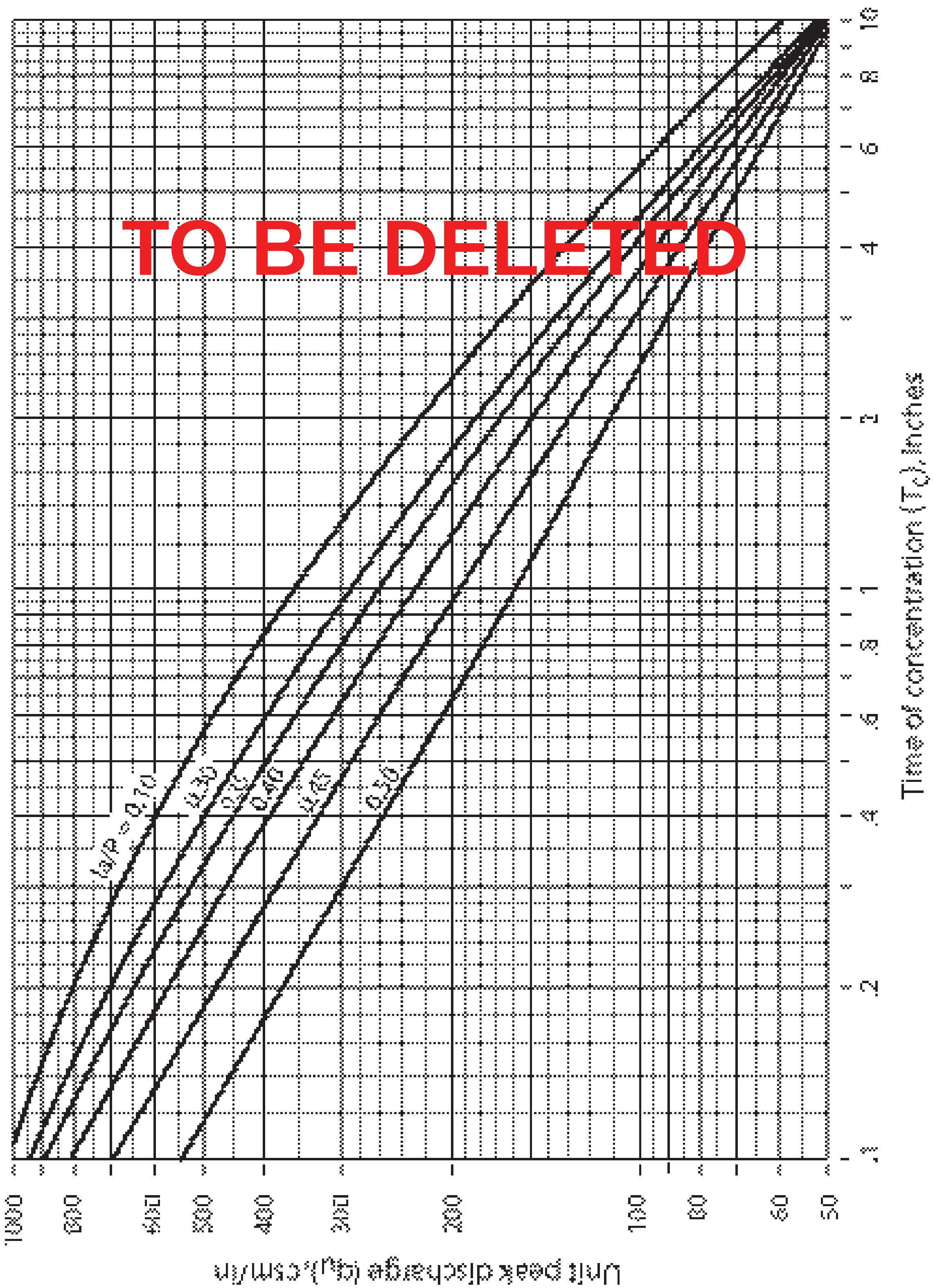

Exhibit 4-II: Unit Peak Discharge (q_u) for SCS Type II Rainfall Distribution

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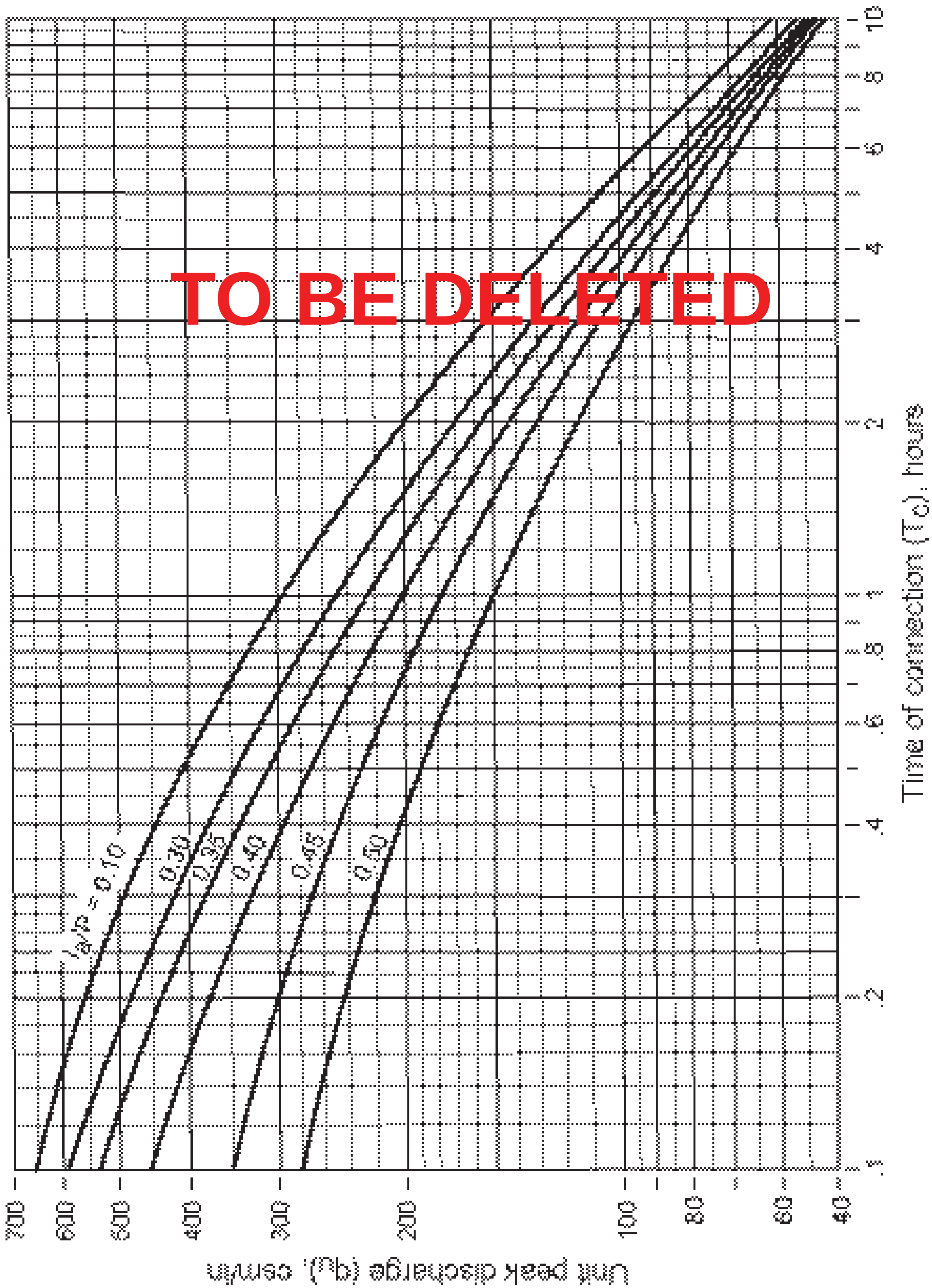


~~**Exhibit 4-III: Unit Peak Discharge (q_u) for SCS Type III Rainfall Distribution**~~

Chapter 5: Tabular Hydrograph method

This chapter presents the Tabular Hydrograph method of computing peak discharges from rural and urban areas, using time of concentration (T_c) and travel time (T_t) from a subarea as inputs. This method approximates TR-20, a more detailed hydrograph procedure (SCS 1983).

The Tabular method can develop partial composite flood hydrographs at any point in a watershed by dividing the watershed into homogeneous subareas. In this manner, the method can estimate runoff from nonhomogeneous watersheds. The method is especially applicable for estimating the effects of land use change in a portion of a watershed. It can also be used to estimate the effects of proposed structures.

Input data needed to develop a partial composite flood hydrograph include (1) 24-hour rainfall (in), (2) appropriate rainfall distribution (I, IA, II, or III), (3) CN, (4) T_c (hr), (5) T_t (hr), and (6) drainage area (mi^2).

Tabular Hydrograph method exhibits

Exhibit 5 (5-I, 5-IA, 5-II, and 5-III) shows tabular discharge values for the various rainfall distributions. Tabular discharges expressed in csm/in (cubic feet of discharge per second per square mile of watershed per inch of runoff) are given for a range of subarea T_c's from 0.1 to 2 hours and reach T_t's from 0 to 3 hours.

The exhibit was developed by computing hydrographs for 1 square mile of drainage area for selected T_c's and routing them through stream reaches with the range of T_t's indicated. The Modified Att-Kin method for reach routing, formulated by SCS in the late 1970's, was used to compute the tabular hydrographs (Comer et al., 1981). A CN of 75 and rainfall amounts generating appropriate I_a/P ratios were used. The resulting runoff estimate was used to convert the hydrographs in exhibits 5-I through 5-III to cubic feet per second per square mile per inch of runoff.

An assumption in development of the tabular hydrographs is that all discharges for a stream reach flow at the same velocity. By this assumption, the subarea flood hydrographs may be routed separately and added at the reference point. The tabular hydrographs in exhibit 5 are prerouted hydrographs. For T_t's other than zero, the tabular discharge values represent the contribution from a single subarea to the composite hydrograph at T_t downstream.

Information required for Tabular Hydrograph method

The following information is required for the Tabular method:

1. Subdivision of the watershed into areas that are relatively homogeneous and have convenient routing reaches.
2. Drainage area of each subarea in square miles.
3. T_c for each subarea in hours. The procedure for estimating T_c is outlined in chapter 3. Worksheet 3 (appendix D) can be used to calculate T_c.
4. T_t for each routing reach in hours. The procedure for estimating T_t is outlined in chapter 3. Worksheet 3 can be used to calculate T_t through a subarea for shallow concentrated and open channel flow.
5. Weighted CN for each subarea. Table 2-2 shows CN's for individual hydrologic soil cover combinations. Worksheet 2 can be used to calculate the weighted runoff curve number.
6. Appropriate rainfall distribution according to figure B-2 (appendix B).
7. The 24-hour rainfall for the selected frequency. Appendix B contains rainfall maps for various frequencies (figures B-3 to B-8).
8. Total runoff (Q) in inches computed from CN and rainfall.
9. I_a for each subarea from table 5-1, which is the same as table 4-1.
10. Ratio of I_a/P for each subarea. If the ratio for the rainfall distribution of interest is outside the range shown in exhibit 5, use the limiting value.

Development of composite flood hydrograph

This section describes the procedure for developing the peak discharge and selected discharge values of a composite flood hydrograph.

Selecting T_e and T_t

First, use worksheet 5a to develop a summary of basic watershed data by subarea. Then use worksheet 5b to develop a tabular hydrograph discharge summary; this summary displays the effect of individual subarea hydrographs as routed to the watershed point of interest. Use ΣT_t for each subarea as the total reach travel time from that subarea through the watershed to the point of interest. Compute the hydrograph coordinates for selected ΣT_t's using the appropriate sheets in exhibit 5. The flow at any time is

$$q = q_t A_m Q \qquad \text{[Eq. 5-1]}$$

where

q = hydrograph coordinate (cfs) at hydrograph time t;
q_t = tabular hydrograph unit discharge from exhibit 5 (csm/in);
A_m = drainage area of individual subarea (mi^2); and
Q = runoff (in).

Since the timing of peak discharge changes with T_e and T_t, interpolation of peak discharge for T_e and T_t values for use in exhibit 5 is not recommended. Interpolation may result in an estimate of peak discharge that would be invalid because it would be lower than either of the hydrographs. Therefore, round the actual values of T_e and T_t to values presented in exhibit 5. Perform this rounding so that the sum of the selected table values is close to the sum of actual T_e and T_t. An acceptable procedure is to select the results of one of three rounding operations:

1. Round T_e and T_t separately to the nearest table value and sum;
2. Round T_e down and T_t up to nearest table value and sum; and
3. Round T_e up and T_t down to nearest table value and sum.

From these three alternatives, choose the pair of rounded T_e and T_t values whose sum is closest to the sum of the actual T_e and T_t. If two rounding methods produce sums equally close to the actual sum, use the combination in which rounded T_e is closest to actual T_e. An illustration of the rounding procedure is as follows:

	Actual values	Table values by rounding method 1	2	3
T_c	1.1	1.0	1.0	1.25
T_t	1.7	1.5	2.0	1.5
Sum	2.8	2.5	3.0	2.75

In this instance, the results from method 3 would be selected because the sum 2.75 is closest to the actual sum of 2.8.

Selecting I_a/P

The computed I_a/P value can be rounded to the nearest I_a/P value in exhibits 5-I through 5-III, or the hydrograph values (csm/in) can be linearly interpolated because I_a/P interpolation generally involves peaks that occur at the same time.

Summing for the composite hydrograph

Table 5-1. - I_a Values for Runoff Curve Numbers

Curve number	I_a (in)	Curve number	I_a (in)
40	3.000	70	0.857
41	2.878	71	0.817
42	2.762	72	0.778
43	2.651	73	0.740
44	2.545	74	0.703
45	2.444	75	0.667
46	2.348	76	0.632
47	2.255	77	0.597
48	2.167	78	0.564
49	2.082	79	0.532
50	2.000	80	0.500
51	1.922	81	0.469
52	1.846	82	0.439
53	1.774	83	0.410
54	1.704	84	0.381
55	1.636	85	0.353
56	1.571	86	0.326
57	1.509	87	0.299
58	1.448	88	0.273
59	1.390	89	0.247
60	1.333	90	0.222
61	1.279	91	0.198
62	1.226	92	0.174
63	1.175	93	0.151
64	1.125	94	0.128
65	1.077	95	0.105
66	1.030	96	0.083
67	0.985	97	0.062
68	0.941	98	0.041
69	0.899		

The composite hydrograph is the summation of prerouted individual subarea hydrographs at each time shown on worksheet 5b. Only the times encompassing the expected maximum composite discharge are summed to define a portion of the composite hydrograph.

If desired, the entire composite hydrograph can be approximated by linear extrapolation as follows:

1. Set up a table similar to worksheet 5b. Include on this table the full range of hydrograph times displayed in exhibit 5.
2. Compute the subarea discharge values for those times and insert them in the table.
3. Sum the values to obtain the composite hydrograph.
4. Apply linear extrapolation to the first two points and the last two points of the composite hydrograph. The volume under this approximation of the entire composite hydrograph may differ from the computed runoff volume.

Limitations

The Tabular method is used to determine peak flows and hydrographs within a watershed. However, its accuracy decreases as the complexity of the watershed increases. If you want to compare present and developed conditions of a watershed, use the same procedure for estimating T_e for both conditions.

Use the TR-20 computer program (SCS 1983) instead of the Tabular method if any of the following conditions applies:

- T_t is greater than 3 hours (largest T_t in exhibit 5).
- T_e is greater than 2 hours (largest T_e in exhibit 5).
- Drainage areas of individual subareas differ by a factor of 5 or more.
- The entire composite flood hydrograph or entire runoff volume is required for detailed flood routings. The hydrograph based on extrapolation is only an approximation of the entire hydrograph.
- The time of peak discharge must be more accurate than that obtained through the Tabular method.

The composite flood hydrograph should be compared with actual stream gage data where possible. The instantaneous peak flow value from the composite flood hydrograph can be compared with data from USGS curves of peak flow versus drainage area.

Examples

A developer proposes to put a subdivision, Fallswood, in subareas 5, 6, and 7 of a watershed in Dyer County, northwestern Tennessee (see sketch below). Before approving the developer's proposal, the planning board wants to know how the development would affect the 25-year peak discharge at the downstream end of subarea 7. The rainfall distribution is type II (figure B-2), and the 24-hour rainfall (P) is 6.0 inches (figure B-6).

Example 5-1

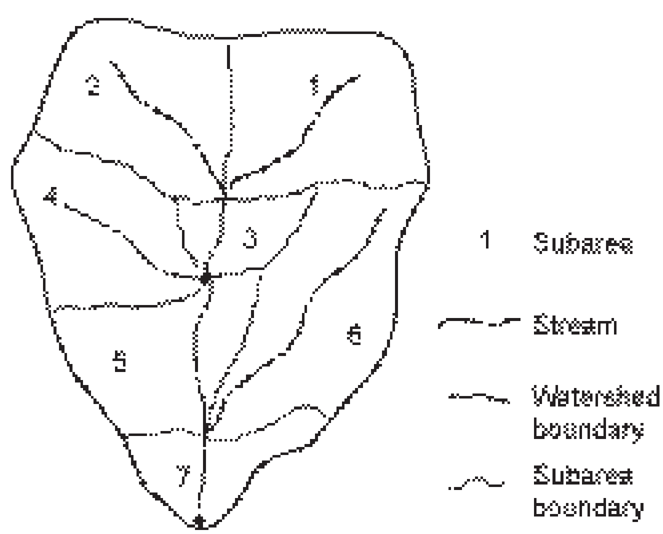
Compute the 25-year frequency peak discharge at the downstream end of subarea 7 for present conditions, using worksheets 5a and 5b. To do this, first calculate the present condition CN, T_e, and T_t for each subarea, using the procedures in chapters 2 and 3. Enter the values on worksheet 5a (figure 5-1).

Next, compute the prerouted hydrograph points for each subarea hydrograph over a range of time near the peak discharge using worksheet 5b (figure 5-2) and the appropriate exhibit 5. For example, for subarea 4, in which T_e = 0.75 hr, refer to sheet 6 of exhibit 5-II. With ΣT_t of 2.00 hr (the sum of downstream travel time through subareas 5 and 7 to the outlet) and I_a/P of 0.1, the routed peak discharge of subarea 4 at the outlet of subarea 7 occurs at 14.6 hr and is 274 csm/in. Solving equation 5-1 with appropriate values provides the peak discharge (q) for subarea 4 at 14.6 hr:

$$q = q_t(A_m Q) = (274)(0.70) = 192 \text{ cfs.}$$

Once all the prerouted subarea hydrographs have been tabulated on worksheet 5b, sum each of the time columns to obtain the composite hydrograph. The resulting 25-year frequency peak discharge is 720 cfs at 14.3 hr (figure 5-2).

Example 5-2



Compute the 25-year frequency peak discharge at the downstream end of subarea 7 for the developed conditions, using worksheets 5a and 5b.

First, calculate the developed condition CN, T_c, and T_t for each subarea, using the procedures in chapters 2 and 3. Enter the values on worksheet 5a (figure 5-3).

Next, compute the prerouted hydrograph points for each subarea hydrograph over a range of time near the peak discharge using worksheet 5b (figure 5-4) and the appropriate exhibit 5. For example, for subarea 6, in which T_c = 1.0 hr, refer to sheet 7 of exhibit 5-II. With ΣT_t of 0.5 hr (downstream travel time through subarea 7 to the outlet) and I_a/P of 0.1, the peak discharge of subarea 6 at the outlet of the watershed occurs at 13.2 hr and is 311 csm/in. Solving equation 5-1 provides the peak discharge (q):

$$q = q_t(A_m Q) = (311)(1.31) = 407 \text{ cfs.}$$

Once all the prerouted subarea hydrographs have been tabulated on worksheet 5b, sum each of the time columns to obtain the composite hydrograph. The resulting 25-year frequency peak discharge is 872 cfs at 13.6 hr (figure 54).

Comparison

According to the results of the two examples, the proposed subdivision at the downstream end of subarea 7 is expected to increase peak discharge from 720 to 872 cfs and to decrease the time to peak from 14.3 to 13.6 hr.

Worksheet 5a: Basic watershed data

Project Fallswood Location Somewhere, Georgia By DW Date 10/1/05

Circle one: (Present) Developed Frequency (yr) 25 Checked 7/24 Date 10/3/05

TO BE DELETED

Subarea name	Drainage area A_m (mi²)	Time of concentration T_c (hr)	Travel time through subarea T_t (hr)	Downstream subarea names	Travel time summation to outlet ΣT_t (hr)	24-hr Rainfall P (in)	Runoff curve number CN	Runoff Q (in)	A_mQ (mi²-in)	Initial abstraction I_a (in)	I_a/P
1	0.30	1.50	--	3,5,7	2.50	6.0	65	2.35	0.71	1.077	0.18
2	0.20	1.25	--	3,5,7	2.50	6.0	70	2.80	0.56	0.857	0.14
3	0.10	0.50	0.50	5,7	2.00	[illegible]	75	3.28	0.33	0.667	0.11
4	0.25	0.75	--	5,7	2.00	6.0	70	2.80	0.70	0.857	0.14
5	0.20	1.50	1.25	7	0.75	6.0	75	3.28	0.66	0.667	0.11
6	0.40	1.50	--	7	0.75	6.0	70	2.80	1.12	0.857	0.14
7	0.20	1.25	0.75	--	0	6.0	75	3.28	0.66	0.667	0.11

From worksheet 3 (Tc, Tt) · From worksheet 2 (CN, Q) · From table 5-1 (Ia)

Figure 5-1. - Worksheet 5a for Example 5-1

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Worksheet 5b: Tabular hydrograph discharge summary

Project Fallswood Location Somewhere, Georgia By DW Date 10/1/05

Circle one: (Present) Developed Frequency(yr) 25 Checked TTM Date 10/3/05

Subarea name	Sub-area Tc (hr)	ΣTt to outlet (hr)	I_a/P	A_mQ (mi²-in)	12.7	12.8	13.0	13.2	13.4	13.6	13.8	14.0	14.3	14.6	15.0	15.5
	Basic watershed data used 1/				Select and enter hydrograph times in hours from exhibit 5-II 2/											
					Discharges at selected hydrograph times 3/ (cfs)											
1	1.50	2.50	0.10	0.71	4	4	5	6	6	8	10	13	24	[illegible]	100	149
2	1.25	2.50	0.10	0.56	3	4	4	6	7	8	11	16	32	64	110	127
3	0.50	2.00	0.10	0.33	5	5	6	8	12	21	41	67	98	92	60	29
4	0.75	2.00	0.10	0.70	8	9	11	14	20	34	62	106	172	[illegible]	149	81
5	1.50	0.75	0.10	0.66	21	28	50	83	118	147	158	154	127	[illegible]	67	44
6	1.50	0.75	0.10	1.12	36	47	85	140	200	249	269	261	214	166	114	75
7	1.25	0	0.10	0.66	169	187	205	176	140	108	85	69	51	40	31	24
Composite hydrograph at outlet					246	284	366	433	503	575	636	686	720	[illegible]	631	529

1/ Worksheet 5a. Rounded as needed for use with exhibit 5.
2/ Enter rainfall distribution type used.
3/ Hydrograph discharge for selected times is A_mQ multiplied by tabular discharge from appropriate exhibit 5.

Figure 5-2. - Worksheet 5b for Example 5-1

TO BE DELETED

Worksheet 5a: Basic watershed data

Project Fallswood   Location Somewhere, Georgia   By DW   Date 10/1/05

Circle one: Present (Developed)   Frequency(yr) 25   Checked TDM   Date 10/3/05

Subarea name	Drainage area A_m (mi²)	Time of concentration T_c (hr)	Travel time through subarea T_t (hr)	Downstream subarea names	Travel time summation to outlet ΣT_t (hr)	24-hr Rainfall P (in)	Runoff curve number CN	Runoff Q (in)	$A_m Q$ (mi²-in)	Initial abstraction I_a (in)	I_a/P
1	0.30	1.50	--	3,5,7	2.[illegible]	6.0	65	2.35	0.71	1.077	0.18
2	0.20	1.25	--	3,5,7	2.[illegible]	6.0	70	2.80	0.56	0.857	0.14
3	0.10	0.50	0.50	5,7	1.5[illegible]	6.0	75	3.28	0.33	0.667	0.11
4	0.25	0.75	--	5,7	1.5[illegible]	6.0	70	2.80	0.70	0.857	0.14
5	0.20	1.50	1.00	7	0.5[illegible]	6.0	85	4.31	0.86	0.353	0.06
6	0.40	1.00	--	7	0.[illegible]	6.0	75	3.28	1.31	0.857	0.14
7	0.20	0.75	0.50	--	[illegible]	6.0	90	4.85	0.97	0.222	0.04

From worksheet 3 (Tc, Tt) — From worksheet 2 (CN, Q) — From table 5-1 (I_a)

Figure 5-3. - Worksheet 5a for Example 5-2

Worksheet 5b: Tabular hydrograph discharge summary

Project Fallswood   Location Somewhere, Georgia   By DW   Date 10/1/05

Circle one: Present (Developed)   Frequency(yr) 25   Checked 7771   Date 10/5/05

Subarea name	Basic watershed data used 1/				Select and enter hydrograph times in hours from exhibit 5-II 2/											
	Subarea T_c (hr)	ΣT_t to outlet (hr)	I_a/P	A_mQ (mi²-in)	12.7	12.8	13.0	13.2	13.4	13.6	13.8	14.0	14.3	14.6	15.0	15.5
					Discharges at selected hydrograph times 3/ (cfs)											
1	1.50	2.00	0.10	0.71	6	6	7	9	11	16	24	40	78	122	155	133
2	1.25	2.00	0.10	0.56	6	6	7	9	12	[illegible]	33	55	96	132	132	87
3	0.50	1.50	0.10	0.33	8	9	14	29	58	[illegible]	106	102	74	46	25	16
4	0.75	1.50	0.10	0.70	13	14	19	32	63	114	169	207	193	143	83	46
5	1.50	0.50	0.10	0.86	51	69	117	167	205	214	202	175	132	99	70	48
6	1.00	0.50	0.10	1.31	149	208	331	407	393	[illegible]	255	195	134	97	69	52
7	0.75	0	0.10	0.97	398	358	244	167	119	[illegible]	72	59	48	40	34	30
Composite hydrograph at outlet					631	670	739	820	861	[illegible]	861	833	755	679	568	412

1/ Worksheet 5a. Rounded as needed for use with exhibit 5.
2/ Enter rainfall distribution type used.
3/ Hydrograph discharge for selected times is A_mQ multiplied by tabular discharge from appropriate exhibit 5.

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~~Figure 5-4. - Worksheet 5b for Example 5-2~~

APPENDIX A-2

PEAK DISCHARGES

NRCS CHART METHOD

INTRODUCTION

A quick and reliable method of computing peak discharges from drainage areas 1 to 2,000 acres in size is given in Figures A-2.3 through A-2.5, p. A-2-3 through A-2-5. The charts were prepared for the solution of the general relationships and are based on type-II rainfall distribution.

Type-II storms occur in regions where the high rates of runoff from small areas are usually generated from summer thunderstorms.

This chapter presents a method of adjusting peak discharges obtained from the charts to reflect the increase in peak discharge due to urbanization. Additional methods for interpolating or adjusting peak discharges for conditions not found on the charts or not represented by the general equations in this chapter are given later in this chapter.

MODIFICATION OF PEAK DISCHARGE DUE TO URBANIZATION

Research in the area of urban hydrology is developing rapidly. Research to date has been sufficient to identify the parameters that are affected by urbanization and to derive limited empirical relationships between those parameters for both agriculture and urban watersheds. The time to peak for urban watersheds is affected by a decrease in lag or time of concentration as described in TR-55 (Appendix A-1).

Figures A-2.1 and A-2.2 give factors for adjusting peaks calculated from Figures A-2.3 to A-2.5 based on the same parameters that affect watershed lag and time of concentration. The factors are applied to the peak using future-condition runoff curve numbers as follows:

Q_{MOD} = Q [Factor imp] [Factor hlm](Eq. A-2.1)

where

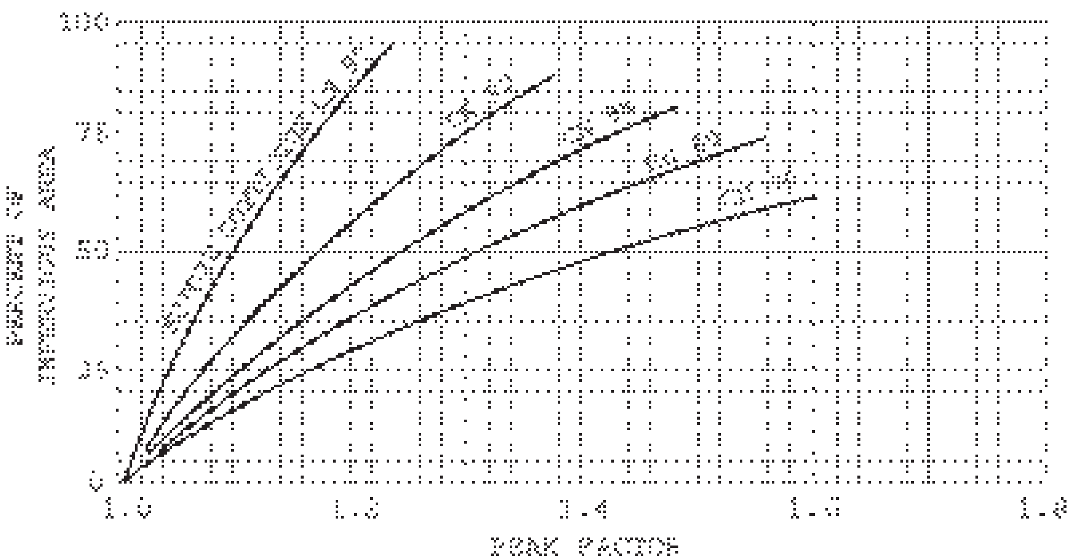


Figure A-2.1 - Factors for Adjusting Peak Discharges for a Given Future-Condition Runoff Curve Number Based on the Percentage of Impervious Area in the Watershed

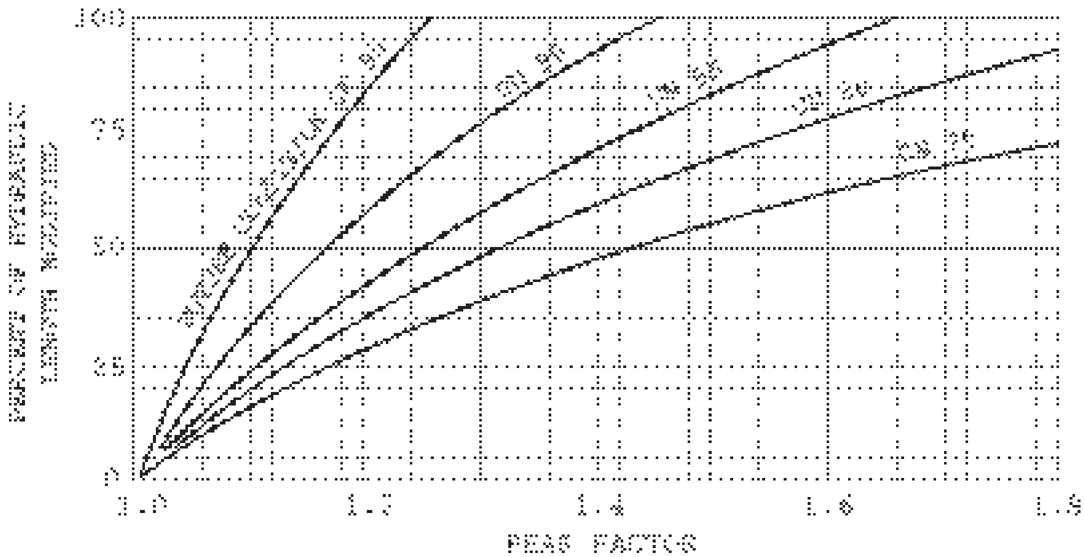


Figure A-2.2 - Factors for Adjusting Peak Discharges for a Given Future-Condition Runoff Curve Number Based on the Percentage of Hydraulic Length Modified.

Q_{MOD} = modified discharge due to urbanization

Q = Discharge for future CN using charts (Figures A-2.3, A-2.4 or A-2.5)

$Factor_{IMP}$ = adjustment factor for percent impervious areas

$Factor_{HLM}$ = adjustment factor for percent of hydraulic length modified.

Example A-2.1

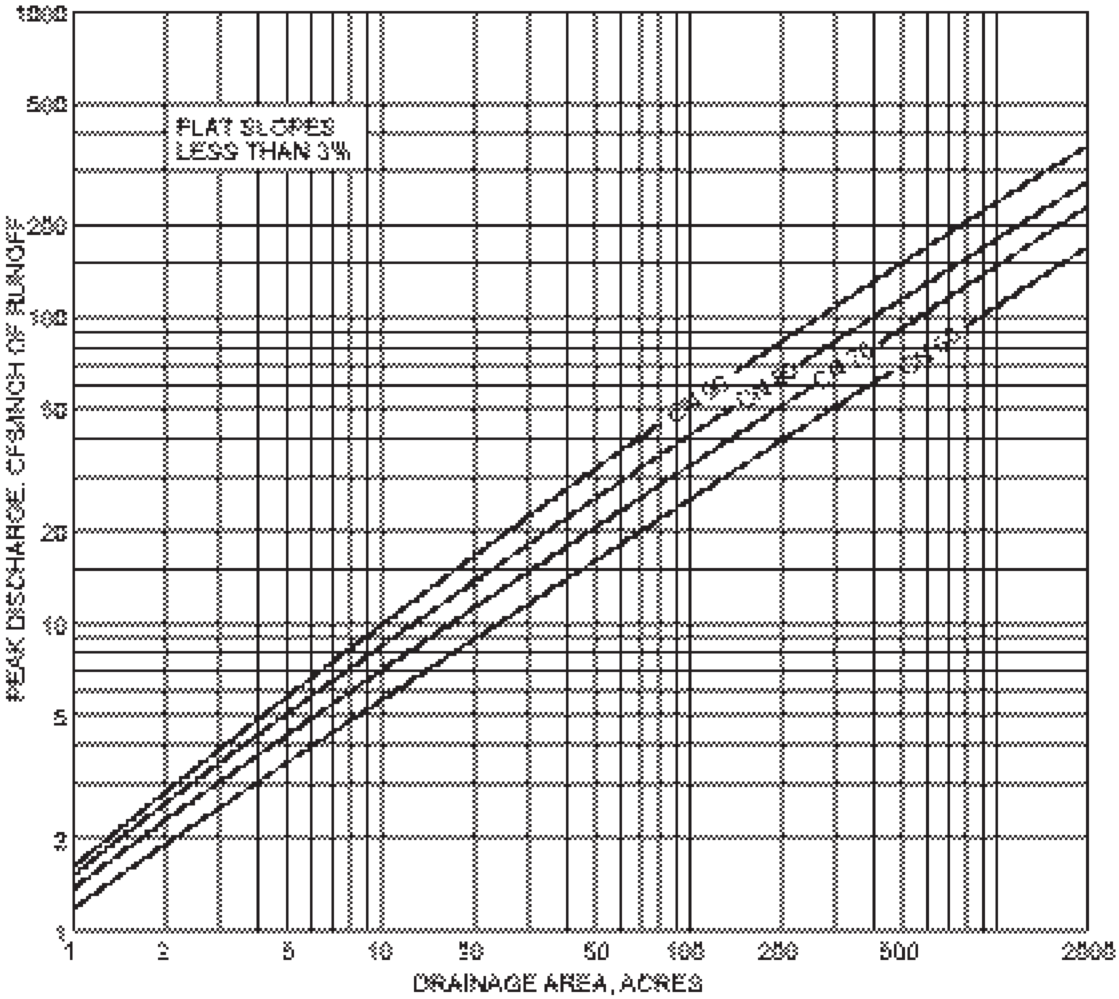
A 300-acre watershed is to be developed. The run-off curve number for the proposed development is computed to be 80. Approximately 60 percent of the hydraulic length will be modified by the installation of street gutters and storm drains to the watershed outlet. Approximately 30 percent of the watershed will be impervious. The average watershed slope is estimated to be 4 percent. Compute the present-condition and anticipated future-condition peak discharge for a 50-year/24-hour storm event with 5 inches of rainfall. The present-condition runoff curve number is 75.

1. From TR-55, Table 2-1 (Appendix A-1), the runoff for present condition is 2.45 inches and for future conditions is 2.89 inches.
2. From the chart for moderate slope in Figure A-2.4 (CN = 75), the present condition peak discharge is 120 cfs (cubic feet per second) per inch of runoff. The peak discharge is then 120 x 2.45 or 294 cfs.
3. From the chart for moderate slope in Figure A-2.4 (CN = 80), the future-condition base discharge for (CN = 80) is 133 cfs per inch of runoff. The base discharge is then 133 x 2.89 or 384 cfs.
4. From Figure A-2.1 with 30 percent impervious area and future runoff curve number of 80, read peak factor = 1.16.
5. From Figure A-2.2, with 60 percent of the hydraulic length modified and future-condition curve number of 80, read peak factor = 1.42.
6. Future-condition peak discharge is:

   384 (1.16)(1.42) = 633 cfs
7. The effect of this proposal development is to increase the peak discharge from 294 to 633 cfs.

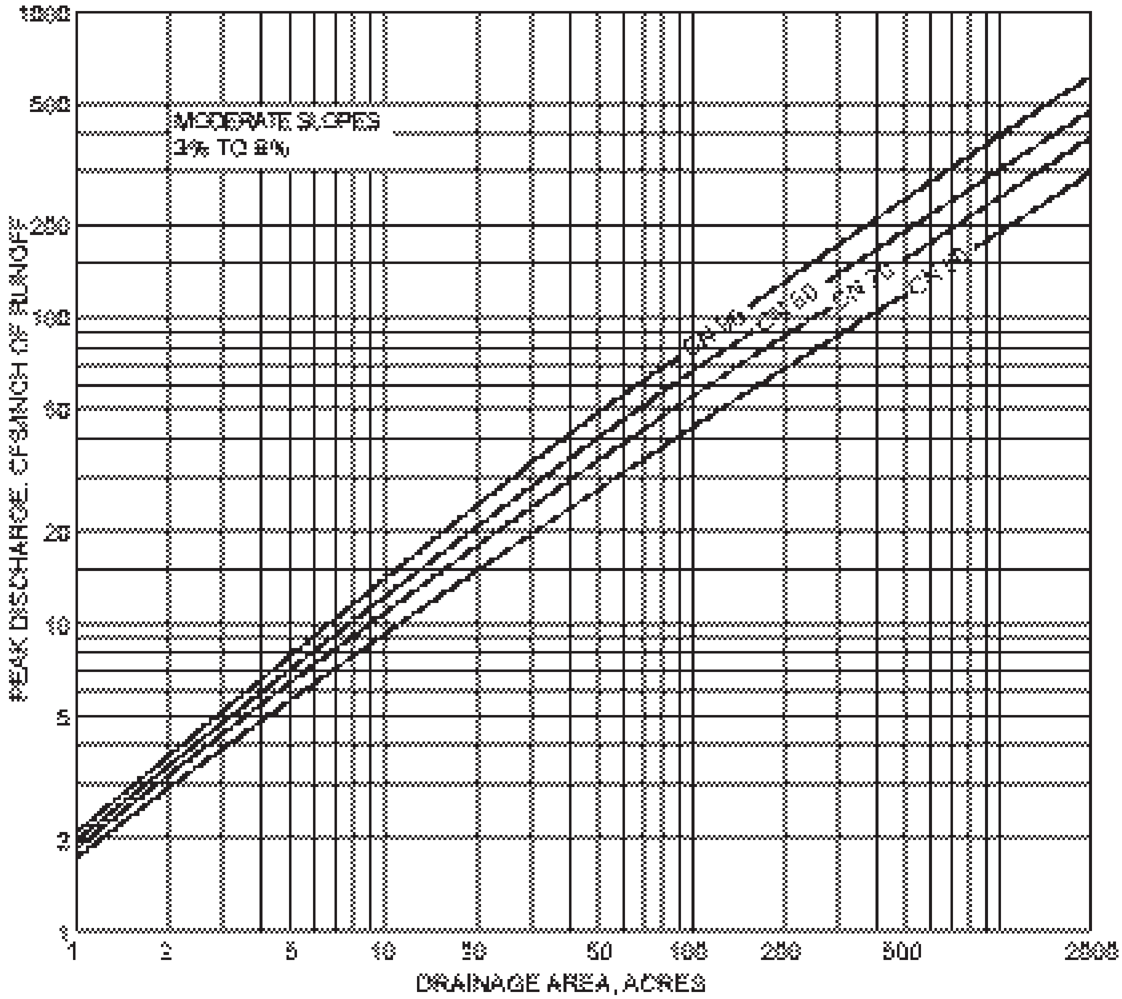
ADJUSTMENT FACTORS FOR PEAKS DETERMINED USING FIGURES A-2.3 THROUGH A-2.5

This section describes methods for adjust-



PEAK RATES OF DISCHARGE FOR SMALL WATERSHEDS
(24 HOUR, TYPE II STORM DISTRIBUTION)

~~Figure A-2.3~~



PEAK RATES OF DISCHARGE FOR SMALL WATERSHEDS (24 HOUR, TYPE II STORM DISTRIBUTION)

~~Figure A-2.4~~

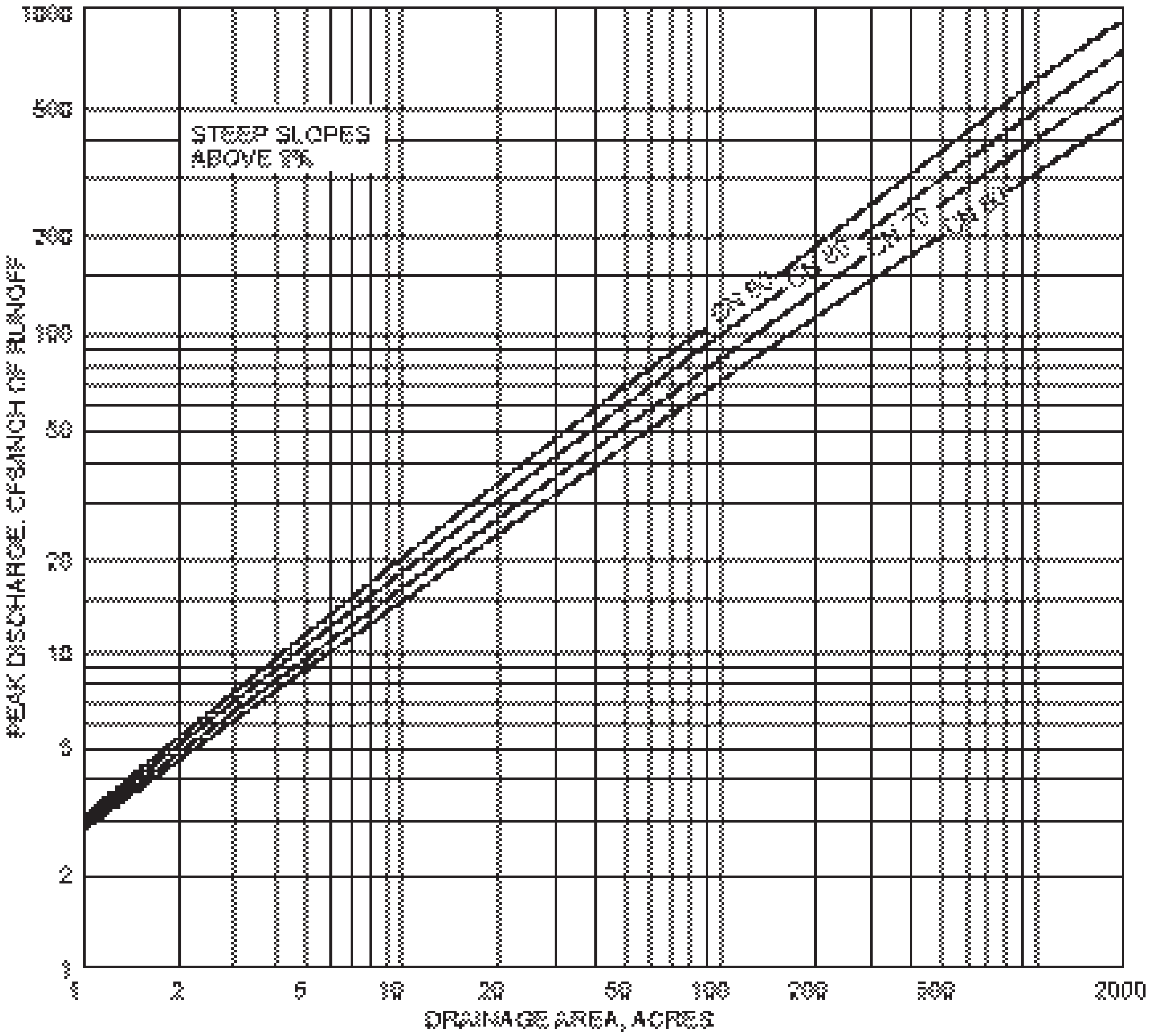


Figure A-2.5. - Peak Rates of Discharge for Small Watersheds (24-Hour, Type II Storm Distribu-

ing peak rates of discharge for ranges of flat, moderate, and steep slopes; for conditions where swamps or ponding areas exist; and for conditions where the watershed shape factor (l/w) varies significantly from that used in the development of the charts of Figures A-2.3 through A-2.5.

SLOPE INTERPOLATION

Table A-2.1 provides interpolation factors to be used in determining peak rates of discharge for specific slopes within ranges of flat, moderate, and steep slopes for a range of drainage areas. Figure A-2.3, for FLAT slopes is based on 1-percent slope, Figure A-2.4, for MODERATE slopes on 4-percent slope, and Figure A-2.5 for STEEP slopes on 16-percent slope. For slopes other than 1, 4, and 16 percent, use the factors shown in Table A-2.1 to modify the peak discharges.

Example A-2.2

Compute the peak discharge for a 1,000-acre watershed with an average watershed slope of 7 percent and a runoff curve number (CN) of 80 for central Lee County, 2-year/24-hour storm.

1. Determine the peak discharge for a watershed with a moderate slope (4 percent). From Figure A-2.4, read a peak discharge of 295 cfs per inch of runoff for 1,000 acres and a CN of 80. From Figure A-2.8, Lee County has a P value of 4.0 inches. From TR-55, Table 2-1 (Appendix A-1) find 2.04 inches of runoff from 4 inches of rainfall and a CN of 80. The peak discharge is then 295 x 2.04 or 602 cfs.

Table A-2.1 - Slope Adjustment Factors by Drainage Areas

Slope (percent)	10 acres	20 acres	50 acres	100 acres	200 acres	500 acres	1,000 acres	2,000 acres
FLAT SLOPES								
0.1	0.49	0.47	0.44	0.43	0.42	0.41	0.41	0.40
0.2	.61	.59	.56	.55	.54	.53	.53	.52
0.3	.69	.67	.65	.64	.63	.62	.62	.61
0.4	.76	.74	.72	.71	.70	.69	.69	.69
0.5	.82	.80	.78	.77	.77	.76	.76	.76
0.7	.90	.89	.88	.87	.87	.87	.87	.87
1.0	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
1.5	1.13	1.14	1.14	1.15	1.16	1.17	1.17	1.17
2.0	1.21	1.24	1.26	1.28	1.29	1.30	1.31	1.31
MODERATE SLOPES								
3	.93	.92	.91	.90	.90	.90	.89	.89
4	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
5	1.04	1.05	1.07	1.08	1.08	1.08	1.09	1.09
6	1.07	1.10	1.12	1.14	1.15	1.16	1.17	1.17
7	1.09	1.13	1.18	1.21	1.22	1.23	1.23	1.24
STEEP SLOPES								
8	.92	.88	.84	.81	.80	.78	.78	.77
9	.94	.90	.86	.84	.83	.82	.81	.81
10	.96	.92	.88	.87	.86	.85	.84	.84
11	.96	.94	.91	.90	.89	.88	.87	.87
12	.97	.95	.93	.92	.91	.90	.90	.90
13	.97	.97	.95	.94	.94	.93	.93	.92
14	.98	.98	.97	.96	.96	.96	.95	.95
15	.99	.99	.99	.98	.98	.98	.98	.98
16	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
20	1.03	1.04	1.05	1.06	1.07	1.08	1.09	1.10

2. Determine the interpolation factor. From Table A-2.1, find 7-percent slope under MODERATE heading and read an interpolation factor of 1.23 for a drainage area of 1,000 acres. (The peak from a 1,000-acre watershed with a watershed slope of 7 percent is 1.23 times greater than for an average watershed slope of 4 percent.)

3. Determine the peak discharge of 7-percent slope.

q = (602)(1.23) = 740 cfs

Examples A-2.3

Compute the peak discharge for a 15-acre watershed with an average slope of 0.5 percent and a runoff curve number of 80 for 4 inches of rainfall.

1. Determine the peak discharge for a watershed with a flat slope (1 percent). From Figure A-2.3 read a peak discharge of 11.2 cfs per inch of runoff for 15 acres and a CN of 80. From Table A-2.1, find 2.04 inches of runoff for 4 inches of rainfall and a CN of 80. The peak discharge is then 11.2 x 2.04 or 23 cfs.

2. Determine the interpolation factor. From Ta-

Table A-2.2. - Peak Flow Adjustment Factors where Ponding and Swampy Areas occur at the

Ratio of drainage area to ponding and swampy area	Percentage of ponding and swampy area	Storm frequency (years)						
		1	2	5	10	25	50	100
500	0.2	0.91	0.92	0.94	0.95	0.96	0.97	0.98
200	.5	.85	.86	.87	.88	.90	.92	.93
100	1.0	.79	.80	.81	.83	.85	.87	.89
50	2.0	.73	.74	.75	.76	.79	.82	.86
40	2.5	.68	.69	.70	.72	.75	.78	.82
30	3.3	.63	.64	.65	.67	.71	.75	.78
20	5.0	.58	.59	.61	.63	.67	.71	.75
15	6.7	.56	.57	.58	.60	.64	.67	.71

Table A-2.3. - Peak Flow Adjustment Factors where Ponding and Swampy Areas are Spread Throughout the Watershed or occur in Central Parts of the Watershed

Ratio of drainage area to ponding and swampy area	Percentage of ponding and swampy area	Storm frequency (years)						
		1	2	5	10	25	50	100
500	0.2	0.93	0.94	0.95	0.96	0.97	0.98	0.99
200	.5	.87	.88	.89	.90	.91	.92	.94
100	1.0	.83	.83	.84	.86	.87	.88	.90
50	2.0	.77	.78	.79	.81	.83	.85	.87
40	2.5	.72	.73	.74	.76	.78	.81	.84
30	3.3	.68	.69	.70	.71	.74	.77	.81
20	5.0	.64	.65	.66	.68	.72	.75	.78
15	6.7	.61	.62	.63	.65	.69	.72	.75
10	10.0	.57	.58	.59	.61	.65	.68	.71

Table A-2.4. - Peak Flow Adjustment Factors where Ponding and Swampy Areas are Located only in Upper Reaches if the Watershed

Ratio of drainage area to ponding and swampy area	Percentage of ponding and swampy area	Storm frequency (years)						
		1	2	5	10	25	50	100
500	0.2	0.95	0.96	0.97	0.98	0.98	0.99	0.99
200	.5	.92	.93	.94	.94	.95	.96	.97
100	1.0	.89	.90	.91	.92	.93	.94	.95
50	2.0	.86	.87	.88	.88	.90	.91	.93
40	2.5	.84	.85	.85	.86	.88	.89	.91
30	3.3	.81	.82	.83	.84	.86	.88	.89
20	5.0	.79	.80	.81	.82	.84	.86	.88
15	6.7	.77	.78	.79	.80	.82	.84	.86

ble A-2.1 find 0.5-percent slope under FLAT heading. Read a slope interpolation factor of 0.81 interpolated between the values for 10 acres and 20 acres.

3. Determine the peak discharge for 0.5-percent slope.

q = (23)(.81) = 19 cfs

ADJUSTMENT FACTORS FOR SWAMPY AND PONDING AREAS

Peak flows determined from Figure A-2.3 through A-2.5 assume that the topography is such that surface flow into ditches, drains, and streams is approximately uniform. On very flat areas and where ponding or swampy areas occur in the watershed, a considerable amount of the surface runoff may be retained in temporary storage. The peak rate of runoff should be reduced to reflect this condition. Tables A-2.2, A-2.3, and A-2.4 provide adjustment factors to determine this reduction based on the ratio of the ponding or swampy area to the total watershed area for a range of storm frequencies.

Table A-2.2 contains adjustment factors to be used when the ponding or swampy areas are located in the path of flow in the vicinity of the design point. Table A-2.3 contains adjustment factors to be used when a significant amount of the flow from the total watershed passes through ponding or swampy areas and these areas are spread throughout the watershed. Table A-2.4 contains adjustment factors to be used when a significant amount of the flow, passes through ponding or swampy areas located in the upper reaches of the watershed.

These conditions may occur in a proposed

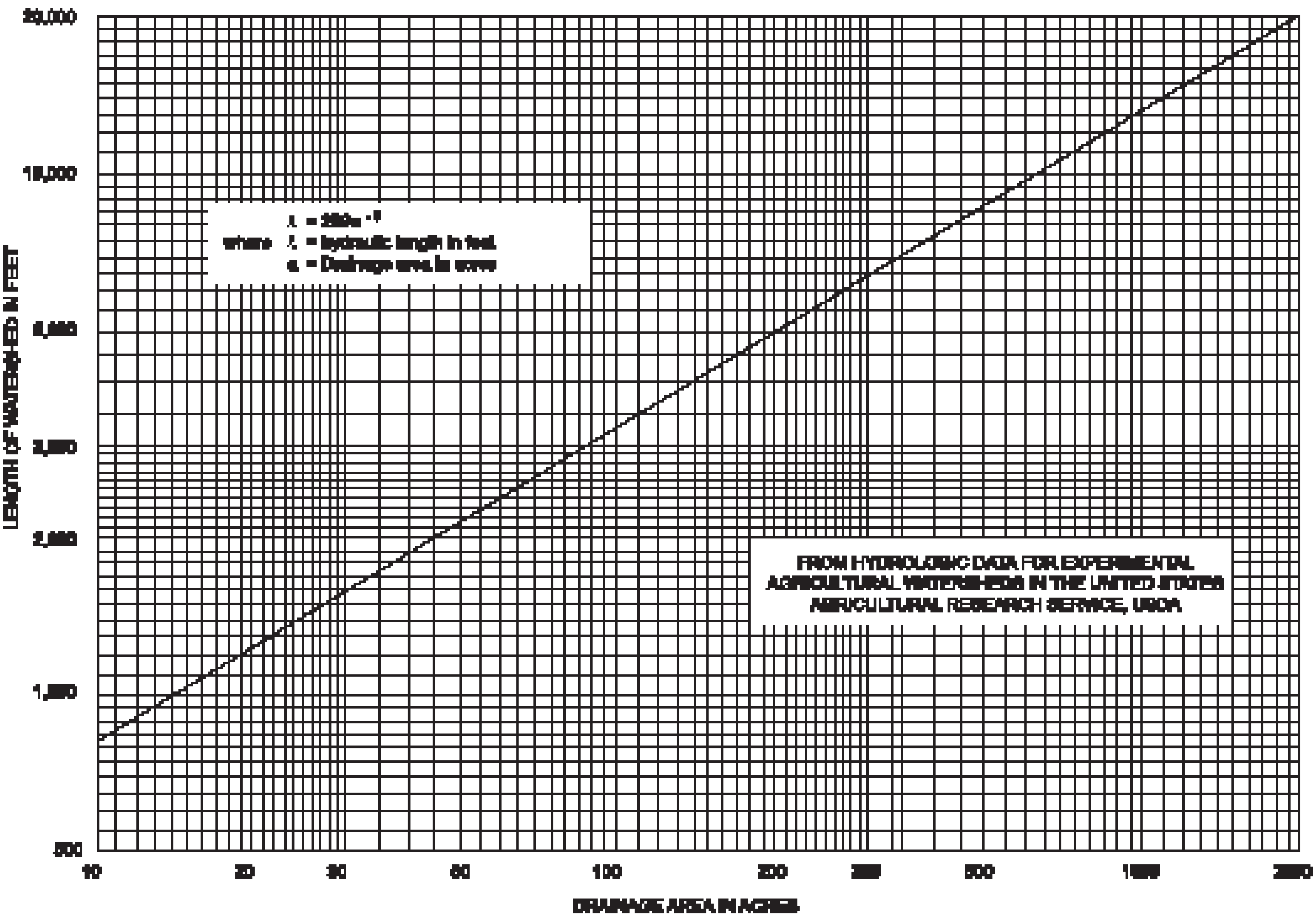


Figure A-2.6 - Hydraulic Length and Drainage Area Relationship or existing urban or suburban area and the adjustment factors from Tables A-2.2, A-2.3, or A-2.4 should be applied after the peaks have been adjusted for the effects of urbanization.

Example A-2.4

A 5-acre pond is located at the downstream end of a 100-acre watershed in which a housing development is proposed. The average watershed slope is 4 percent and the present-condition curve number is 75. After the installation of the housing development, 30 percent of the watershed will be impervious and 50 percent of the hydraulic length will be modified. The future-condition curve number is estimated to be 80. For a 100-year storm 24-hour duration in central Glascock County, determine the present-condition and future-condition peak discharges downstream of the pond.

1. Determine the present-condition peak discharge assuming the pond is not in place: From Figure A-2.4, find the peak discharge to be 59 cfs per inch of runoff. From Figure A-2.13, the rainfall for central Glascock County is 8 inches. From TR-55, Table 2.1 (Appendix A-1) find the runoff to be 5.04 inches. The peak discharge is 59 x 5.04 or 297 cfs.
2. Determine the ponding adjustment factor: Since the pond is at the lower end of the watershed, use Table A-2.2. The ratio of the drainage area to pond area is 100/5 or 20. For a 100-year frequency event, the adjustment factor is 0.75.
3. Compute the present-condition peak discharge:

   Q = 0.75 (297) = 233 cfs
4. Compute the basic future-condition peak discharge: From Figure A-2.4, find the peak discharge to be 65 cfs per inch of runoff. From TR-55, Table 2-1, (Appendix A-1), Find the runoff to be 5.62 inches The peak discharge is then 65 x 5.62 or 365 cfs.
5. Determine the modification factors for proposed urbanization: Taken from Figures A-2.1 and A-2.2 for a curve number of 80: impervious factor = 1.16; hydraulic length factor = 1.31; urbanization factor = (1.16) (1.31) = 1.52.
6. Compute the future condition peak discharge:

   q = 1.52 (365) = 555 cfs
7. Compute the future-condition peak below the pond: From step 2 the ponding factor is 0.75.

   q = 0.75 (555) = 416 cfs

ADJUSTMENT FOR WATERSHED SHAPE FACTOR

The equation used in computing peak discharges from Figures A-2.3 through A-2.5 was based in part on a relationship between the hydraulic length and the watershed area from Agricultural Research Services's studies on small experimental watersheds. Figure A-2.6 shows the best fit line relating length to drainage area. The equation of the line is $l = 209a^{0.6}$. A watershed shape factor, l/w (where w is the average width of the watershed), is then fixed for any given drainage area. For example, for drainage areas of 10, 100, and 1,000 acres, the watershed shape factor is 1.58, 2.51, and 3.98, respectively.

There are watersheds that deviate considerably from these relationships. The peaks can be modified for other shape factors. The procedure is as follows:

1. Determine the hydraulic length of the watershed and compute "equivalent" drainage area using $l = 209a^{0.6}$ or Figure A-2.6.
2. Determine the "equivalent" peak flow from the charts for the "equivalent" drainage area.
3. Compute the "actual" peak discharge for the watershed by multiplying the equivalent peak discharge by the ratio of actual drainage area to the equivalent drainage area.

The factors for modifying the peak for

urbanization can then be applied to the revised peak discharge.

Example A-2.5

From a topographic map the hydraulic length of a 100-acre watershed with moderate slopes and a CN of 75 was measured to be 2,200 feet. Determine the peak discharge for a 6-inch, 24-hour rainfall.

1. Determine the equivalent drainage area for a watershed with a hydraulic length of 2,200 feet. From Figure A-2.6, read 51 acres. (Note that in a 100-acre watershed, the hydraulic length would be 3,300 feet from Figure A-2.6).

2. Determine the "equivalent" peak flow from Figure A-2.4 for a drainage area of 51 acres and a CN of 75. Read 37 cfs per inch of runoff. From TR-55 Table 2-1 (Appendix A-1), find the runoff to be 3.28 or 121 cfs.

3. Compute the actual peak discharge for 100 acres.

$$\text{actual discharge} = \text{equivalent discharge}\left(\frac{\text{actual drainage area}}{\text{equivalent drainage area}}\right)$$

$$q = 121\left(\frac{100}{51}\right) = 237 \text{ cfs}$$

The peak discharge for the 100-acre watershed with a hydraulic length of 2,200 feet is 237 cfs (versus 194 cfs for a "normal" 100-acre watershed). Adjustments to this peak discharge for urbanization can be made using factors discussed on page A-2-1.

4. The procedure in steps 1, 2, and 3 can be used to determine peak discharges when the actual hydraulic length is longer than that shown on Figure A-2.6. For example, if the actual length were 4,500 feet instead of 3,300 feet, the equivalent area would be 170 acres, as shown in Figure A-2.6.

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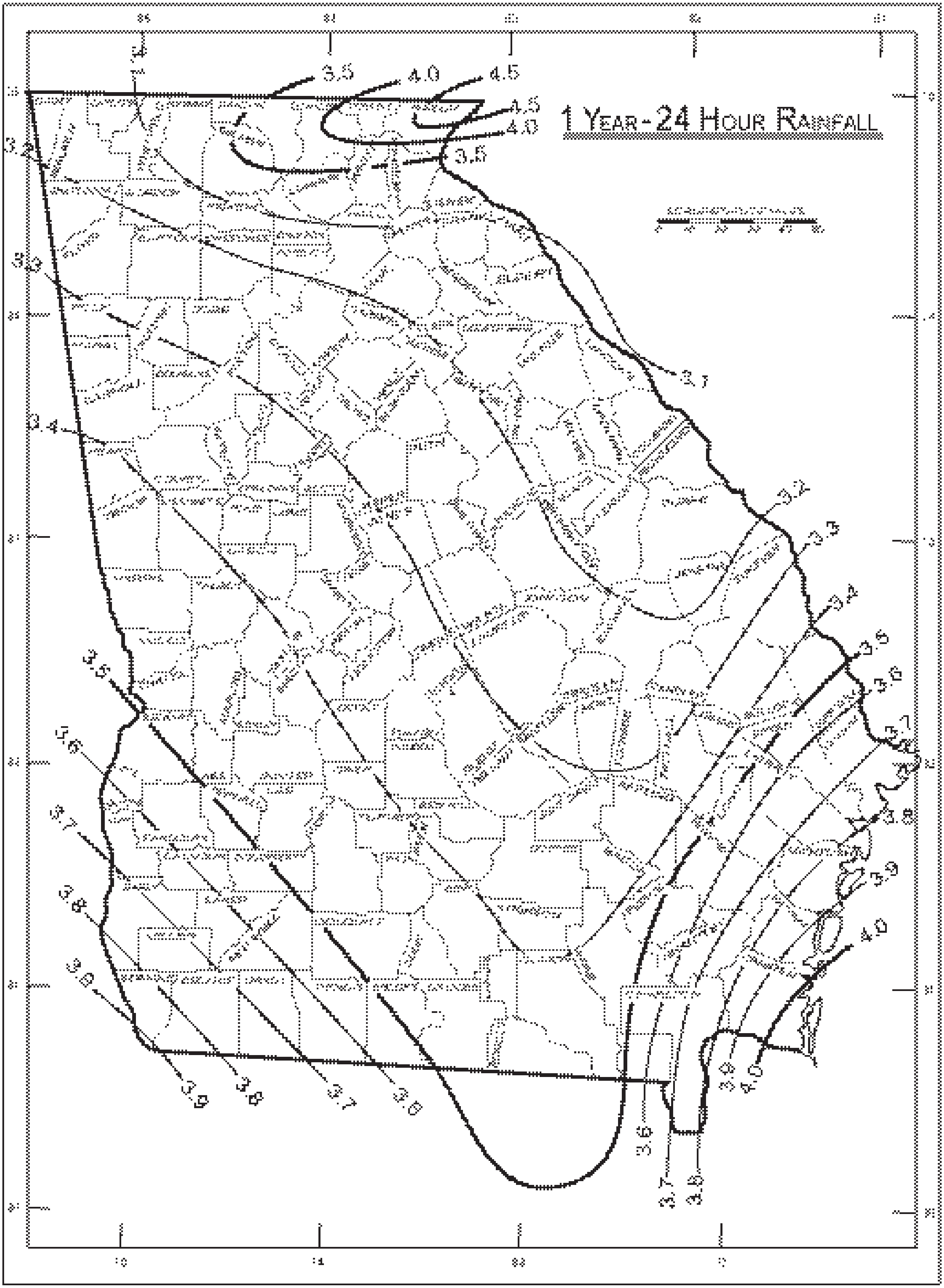

Figure A-2.7. - Total Rainfall (P) for 1-Year/24-Hour Storm

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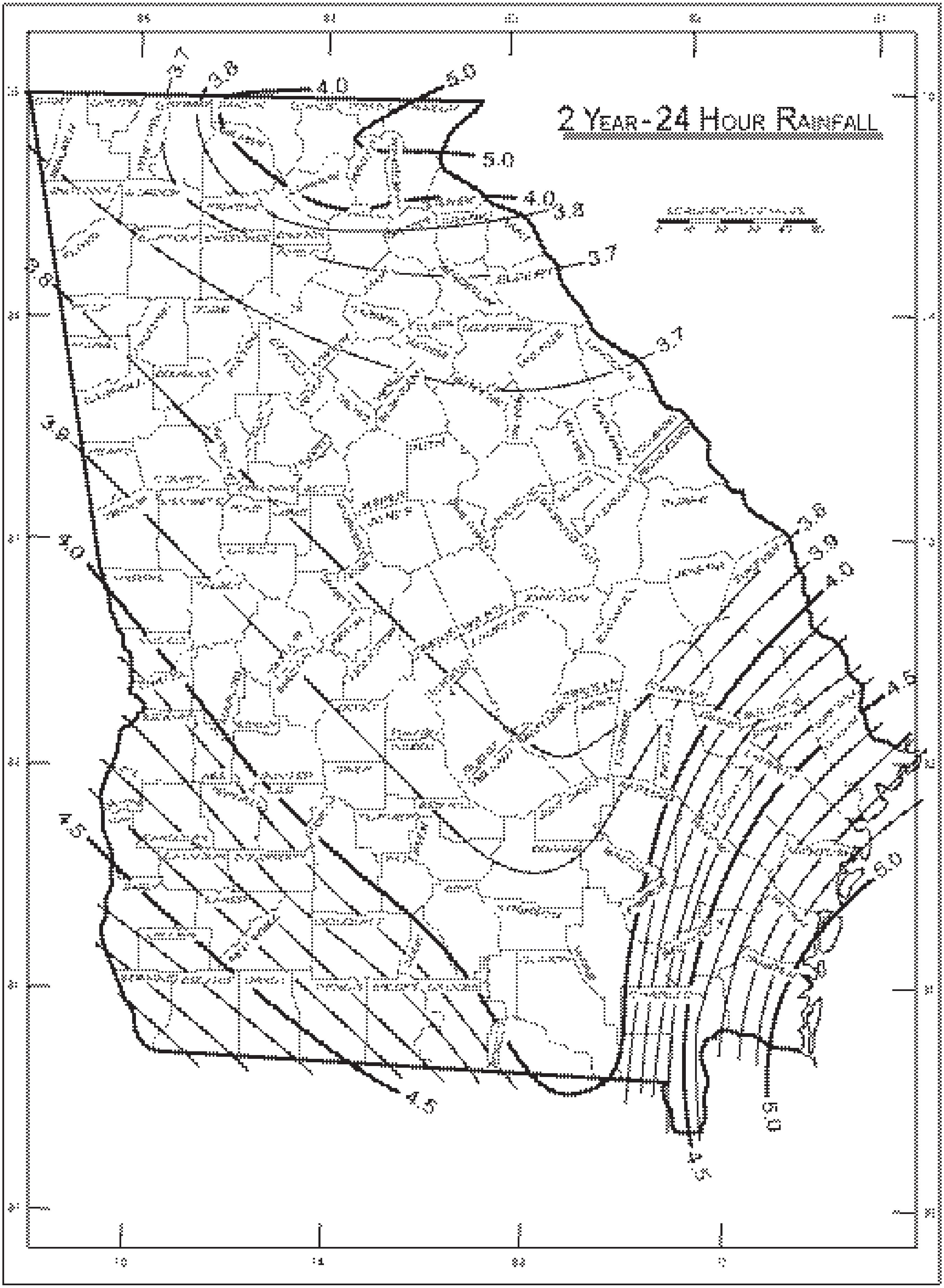

Figure A-2.8. - Total Rainfall (P) for 2-Year/24-Hour Storm

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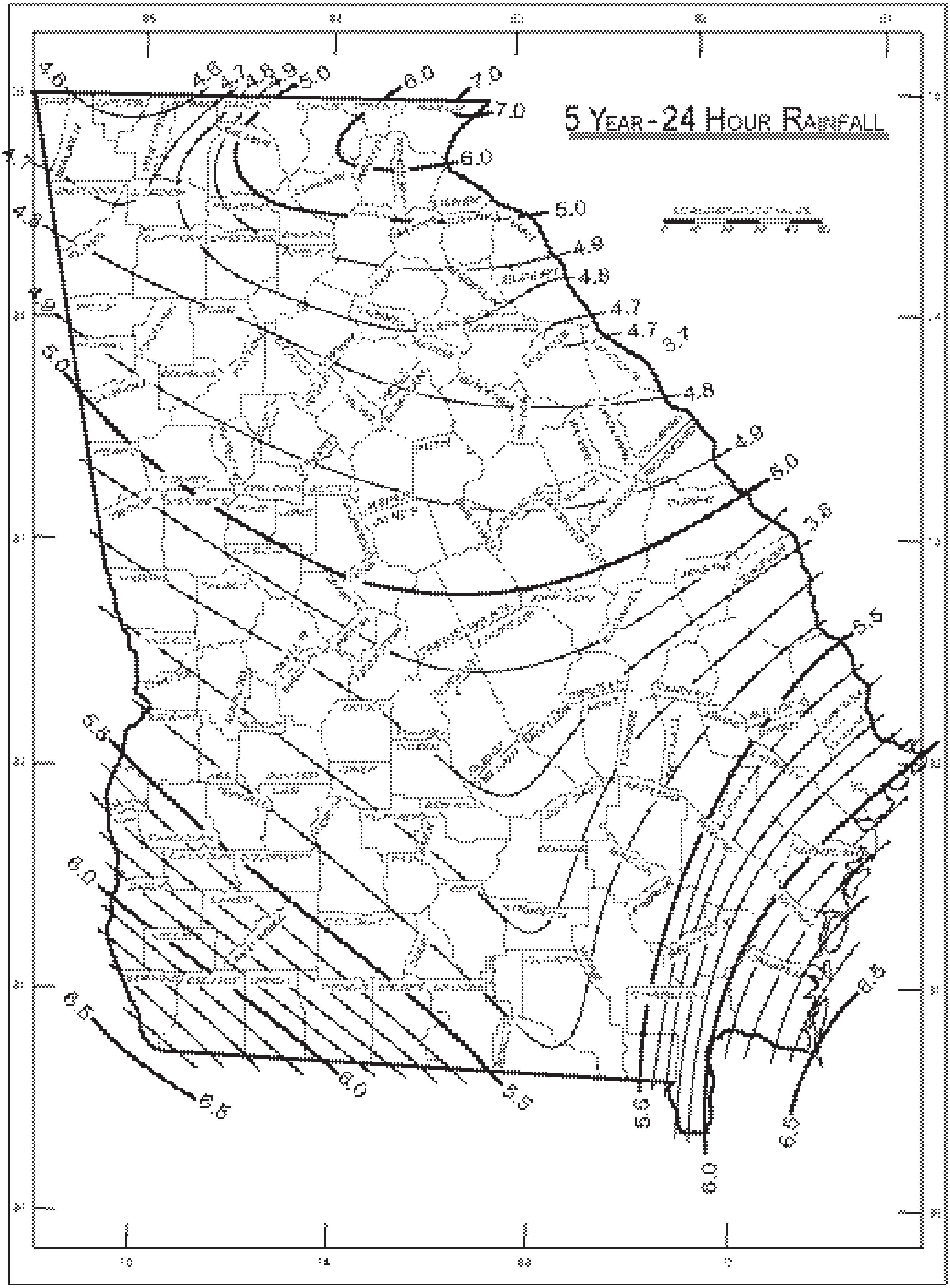

~~Figure A-2.9. - Total Rainfall (P) for 5-Year/24-Hour Storm~~

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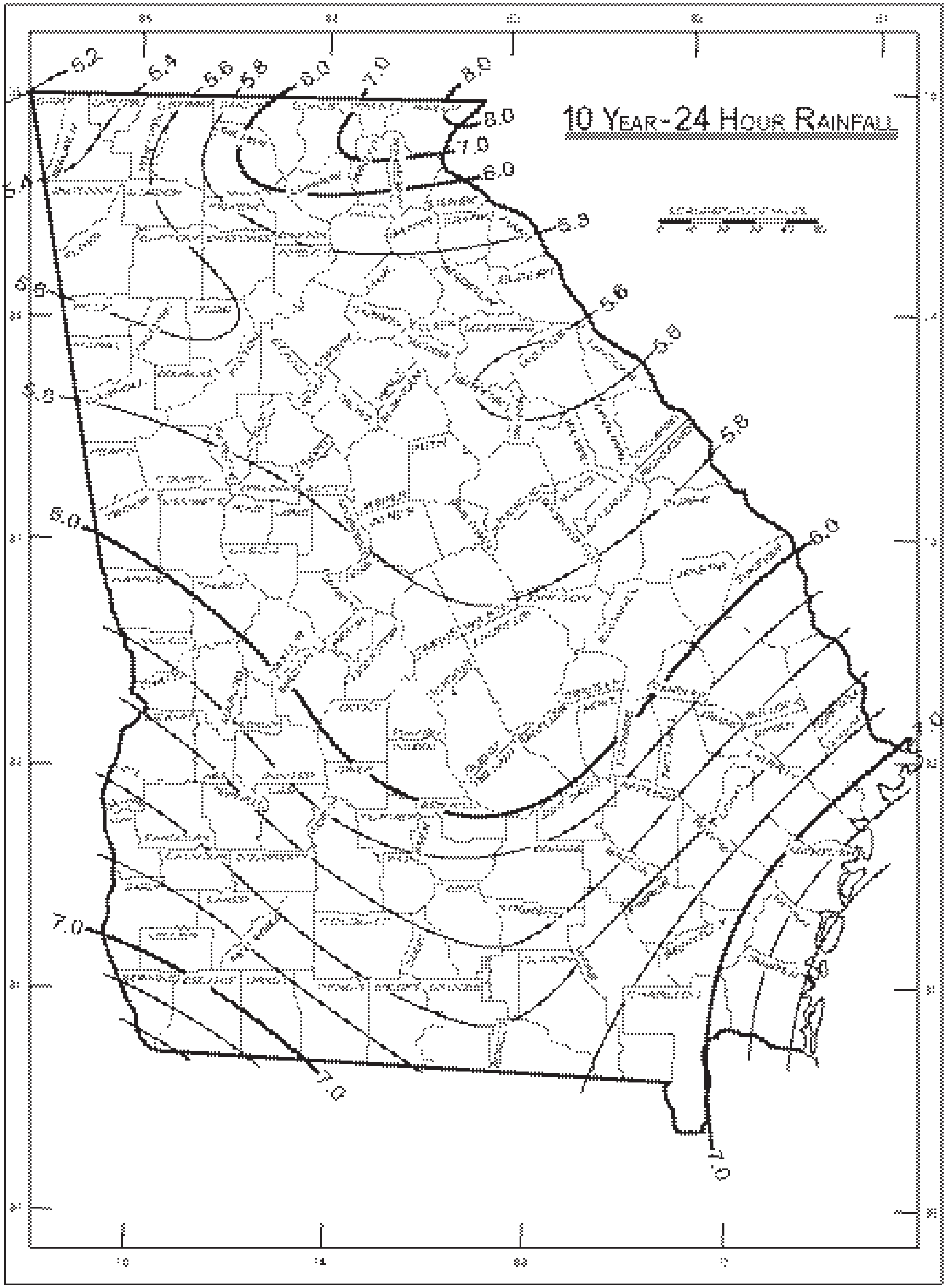

Figure A-2.10. - Total Rainfall (P) for 10-Year/24-Hour Storm

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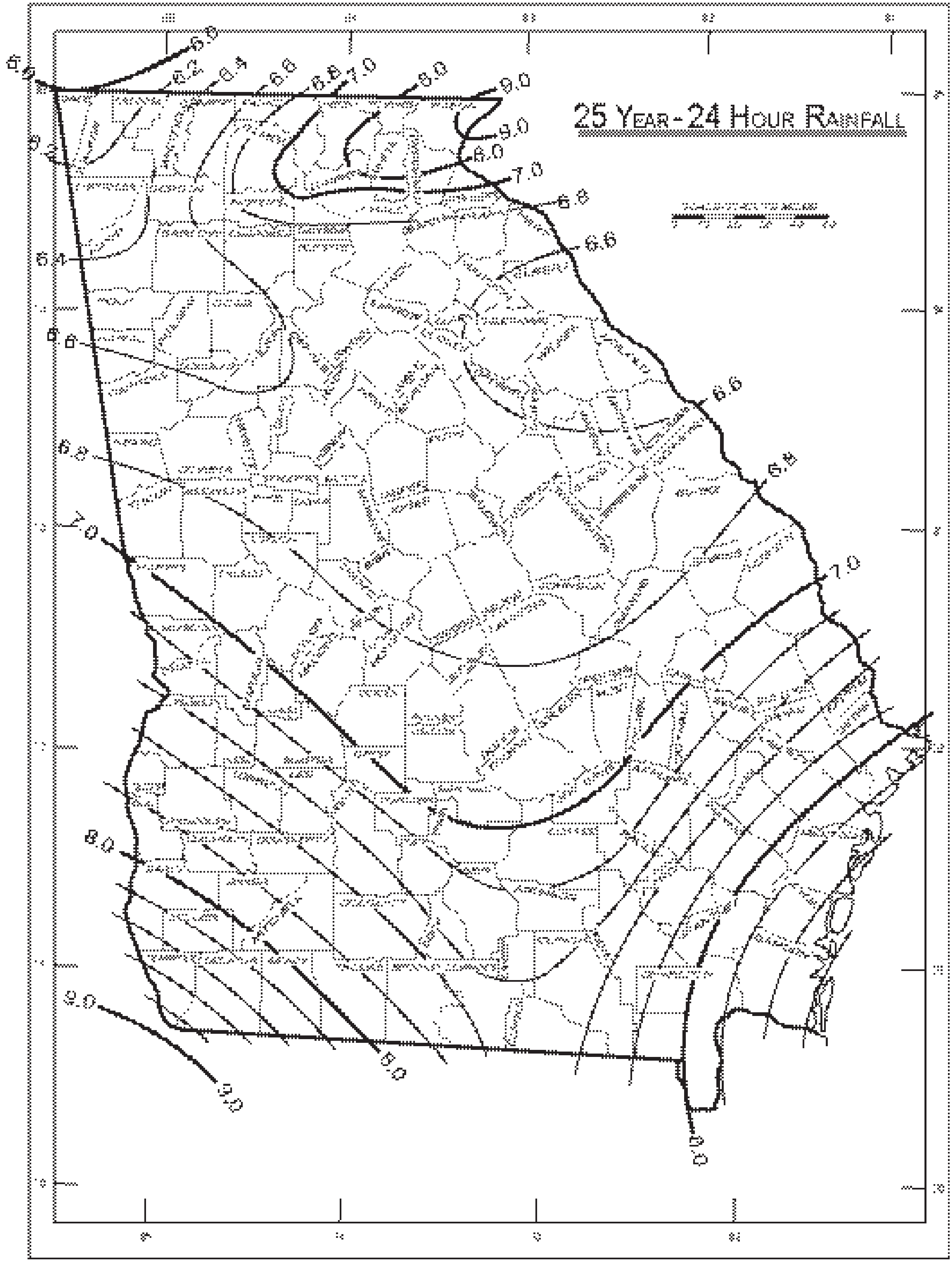
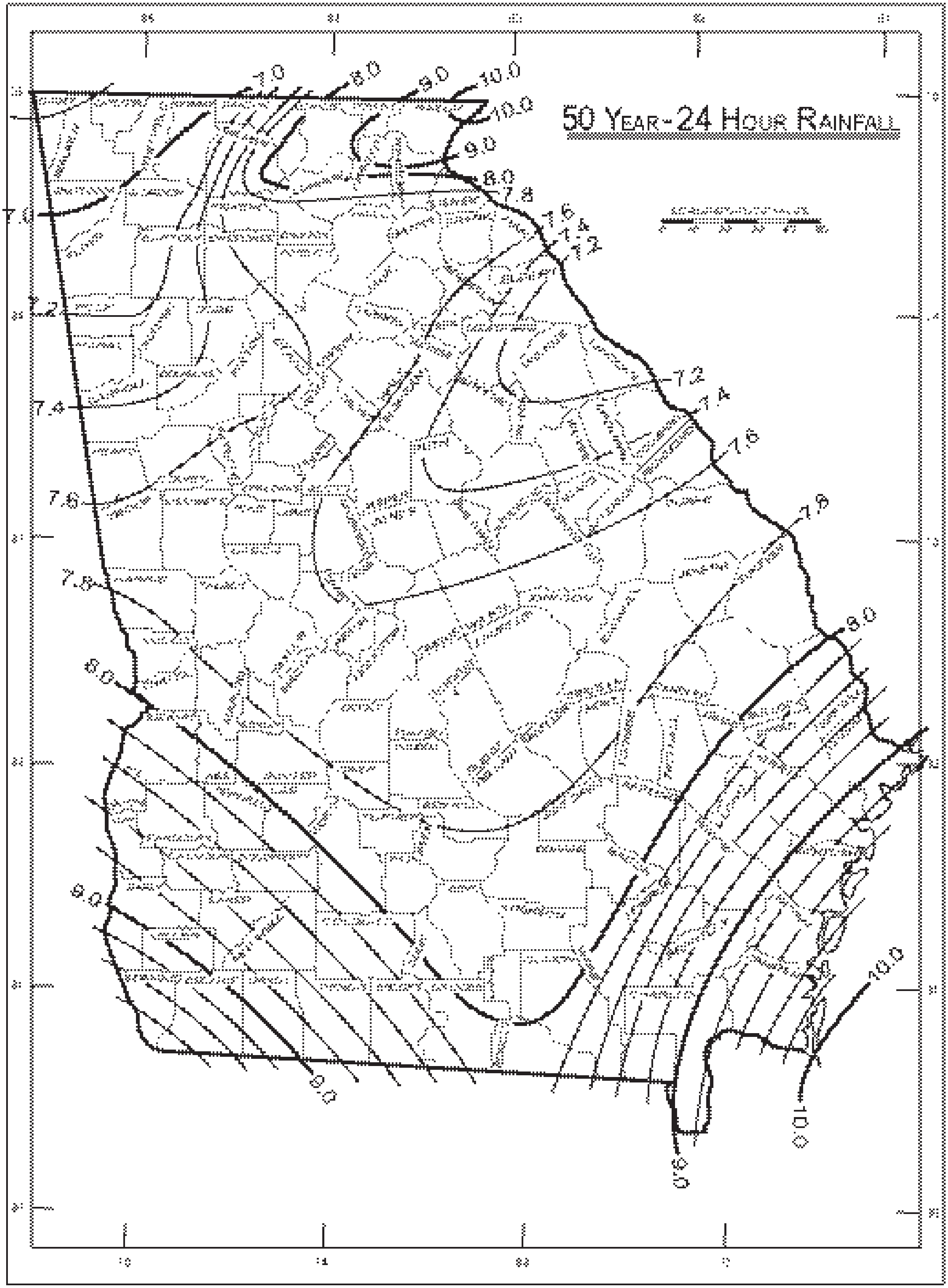


Figure A-2.11. - Total Rainfall (P) for 25-Year/24-Hour Storm

GEORGIA



~~Figure A-2.12. - Total Rainfall (P) for 50-Year/24-Hour Storm~~

GEORGIA

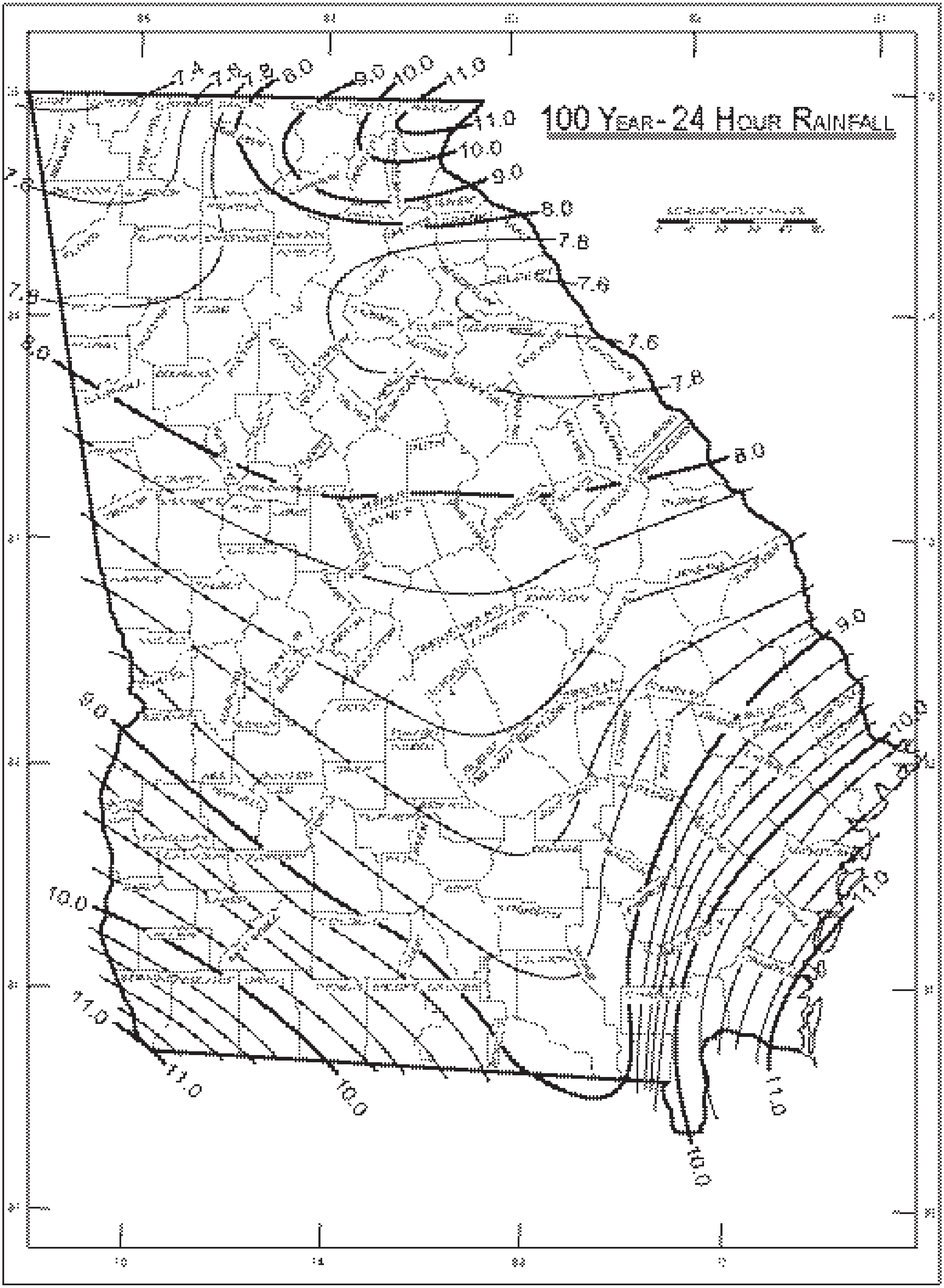


Figure A-2.13. - Total Rainfall (P) for 100-Year/24-Hour Storm

APPENDIX A-2

PEAK DISCHARGES

NRCS CHART METHOD

INTRODUCTION

A quick and reliable method of computing peak discharges from drainage areas 1 to 2,000 acres in size is given in Figures A-2.3 through A-2.5, p. A-2-3 through A-2-5. The charts were prepared for the solution of the general relationships and are based on type-II rainfall distribution.

Type-II storms occur in regions where the high rates of runoff from small areas are usually generated from summer thunderstorms.

This chapter presents a method of adjusting peak discharges obtained from the charts to reflect the increase in peak discharge due to urbanization. Additional methods for interpolating or adjusting peak discharges for conditions not found on the charts or not represented by the general equations in this chapter are given later in this chapter.

MODIFICATION OF PEAK DISCHARGE DUE TO URBANIZATION

Research in the area of urban hydrology is developing rapidly. Research to date has been sufficient to identify the parameters that are affected by urbanization and to derive limited empirical relationships between those parameters for both agriculture and urban watersheds. The time to peak for urban watersheds is affected by a decrease in lag or time of concentration as described in TR-55 (Appendix A-1).

Figures A-2.1 and A-2.2 give factors for adjusting peaks calculated from Figures A-2.3 to A-2.5 based on the same parameters that affect watershed lag and time of concentration. The factors are applied to the peak using future-condition runoff curve numbers as follows:

Q_{MOD} = Q [Factor imp] [Factor hlm](Eq. A-2.1)

where

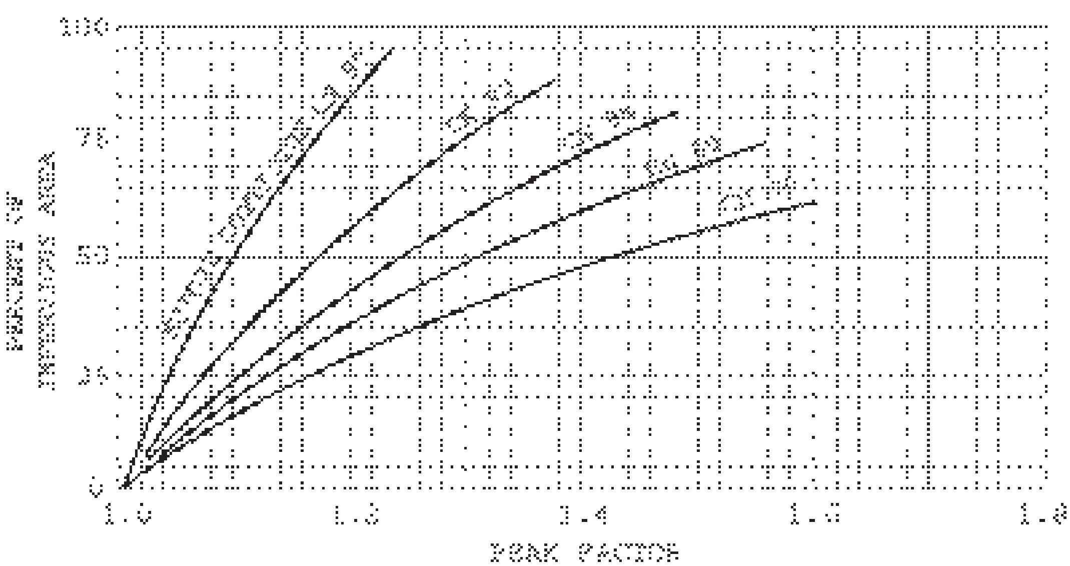
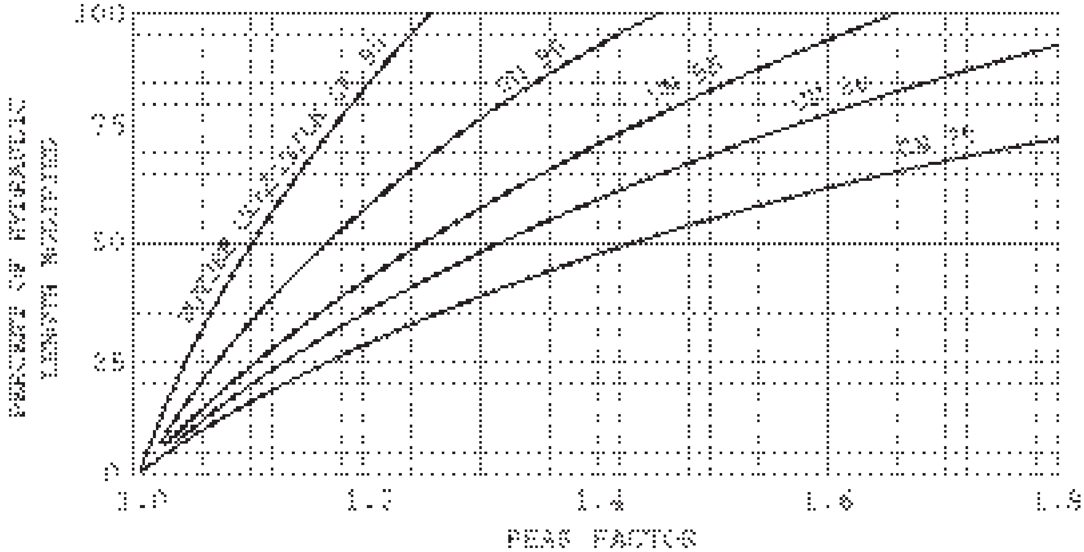


Figure A-2.1 - Factors for Adjusting Peak Discharges for a Given Future-Condition Runoff Curve Number Based on the Percentage of Impervious Area in the Watershed



~~Figure A-2.2 - Factors for Adjusting Peak Discharges for a Given Future-Condition Runoff Curve Number Based on the Percentage of Hydraulic Length Modified.~~

~~Q_{MOD} = modified discharge due to urbanization~~

~~Q = Discharge for future CN using charts (Figures A-2.3, A-2.4 or A-2.5)~~

~~$Factor_{IMP}$ = adjustment factor for percent impervious areas~~

~~$Factor_{HLM}$ = adjustment factor for percent of hydraulic length modified.~~

~~Example A-2.1~~

~~A 300-acre watershed is to be developed. The run-off curve number for the proposed development is computed to be 80. Approximately 60 percent of the hydraulic length will be modified by the installation of street gutters and storm drains to the watershed outlet. Approximately 30 percent of the watershed will be impervious. The average watershed slope is estimated to be 4 percent. Compute the present-condition and anticipated future-condition peak discharge for a 50-year/24-hour storm event with 5 inches of rainfall. The present-condition runoff curve number is 75.~~

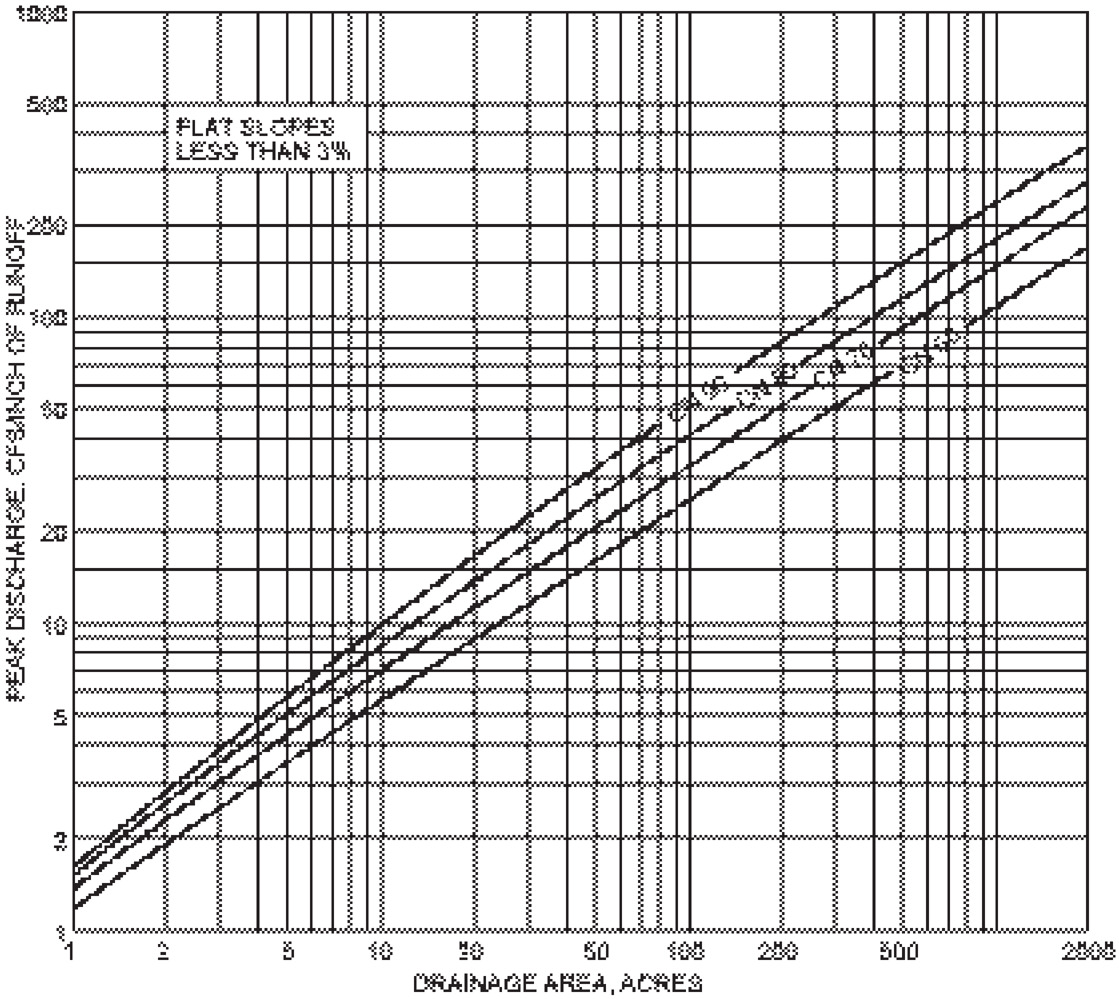
1. ~~From TR-55, Table 2-1 (Appendix A-1), the runoff for present condition is 2.45 inches and for future conditions is 2.89 inches.~~
2. ~~From the chart for moderate slope in Figure A-2.4 (CN = 75), the present condition peak discharge is 120 cfs (cubic feet per second) per inch of runoff. The peak discharge is then 120 x 2.45 or 294 cfs.~~
3. ~~From the chart for moderate slope in Figure A-2.4 (CN = 80), the future-condition base discharge for (CN = 80) is 133 cfs per inch of runoff. The base discharge is then 133 x 2.89 or 384 cfs.~~
4. ~~From Figure A-2.1 with 30 percent impervious area and future runoff curve number of 80, read peak factor = 1.16.~~
5. ~~From Figure A-2.2, with 60 percent of the hydraulic length modified and future-condition curve number of 80, read peak factor = 1.42.~~
6. ~~Future-condition peak discharge is:~~

~~384 (1.16)(1.42) = 633 cfs~~

7. ~~The effect of this proposal development is to increase the peak discharge from 294 to 633 cfs.~~

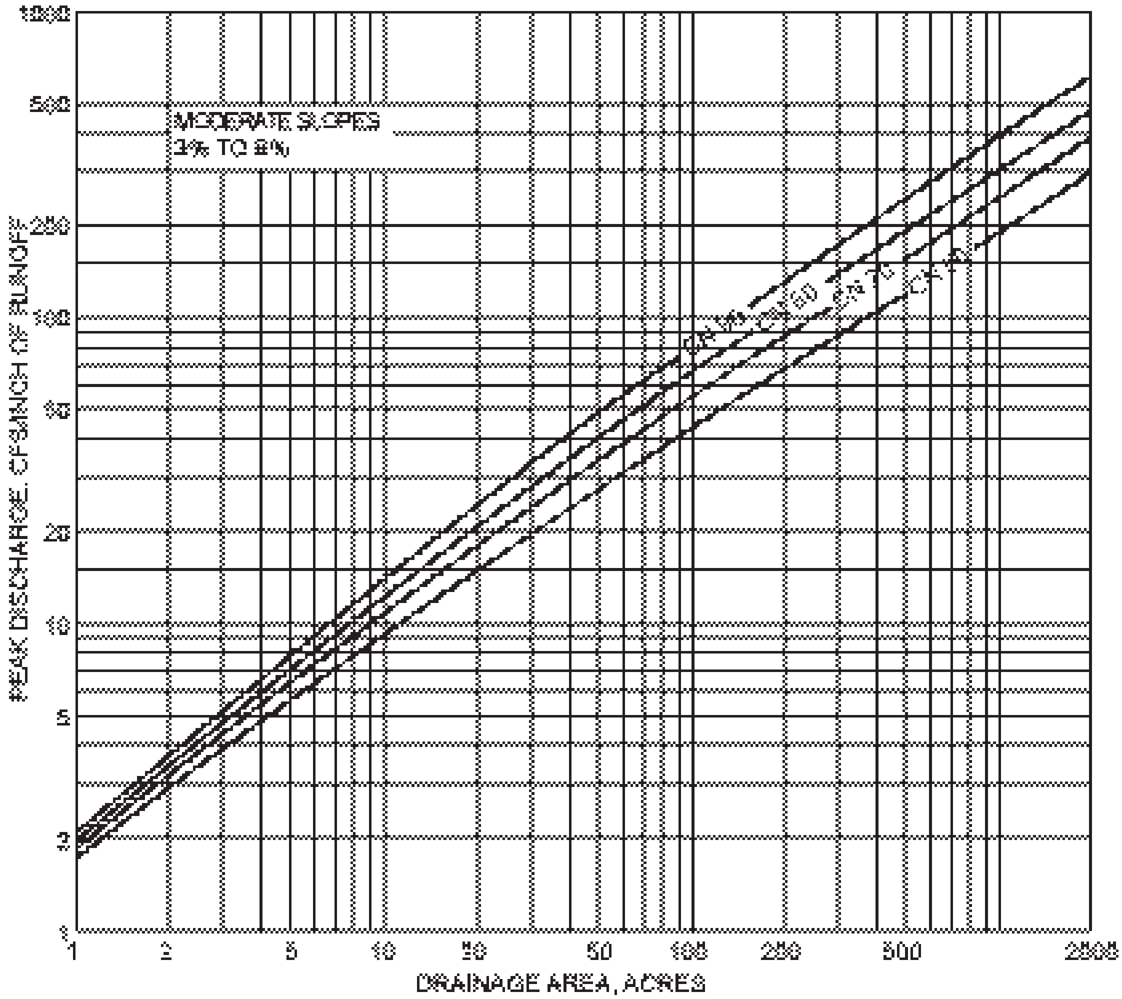
~~ADJUSTMENT FACTORS FOR PEAKS DETERMINED USING FIGURES A-2.3 THROUGH A-2.5~~

~~This section describes methods for adjust-~~



PEAK RATES OF DISCHARGE FOR SMALL WATERSHEDS
(24 HOUR, TYPE II STORM DISTRIBUTION)

Figure A-2.3



PEAK RATES OF DISCHARGE FOR SMALL WATERSHEDS
(24 HOUR, TYPE II STORM DISTRIBUTION)

~~Figure A-2.4~~

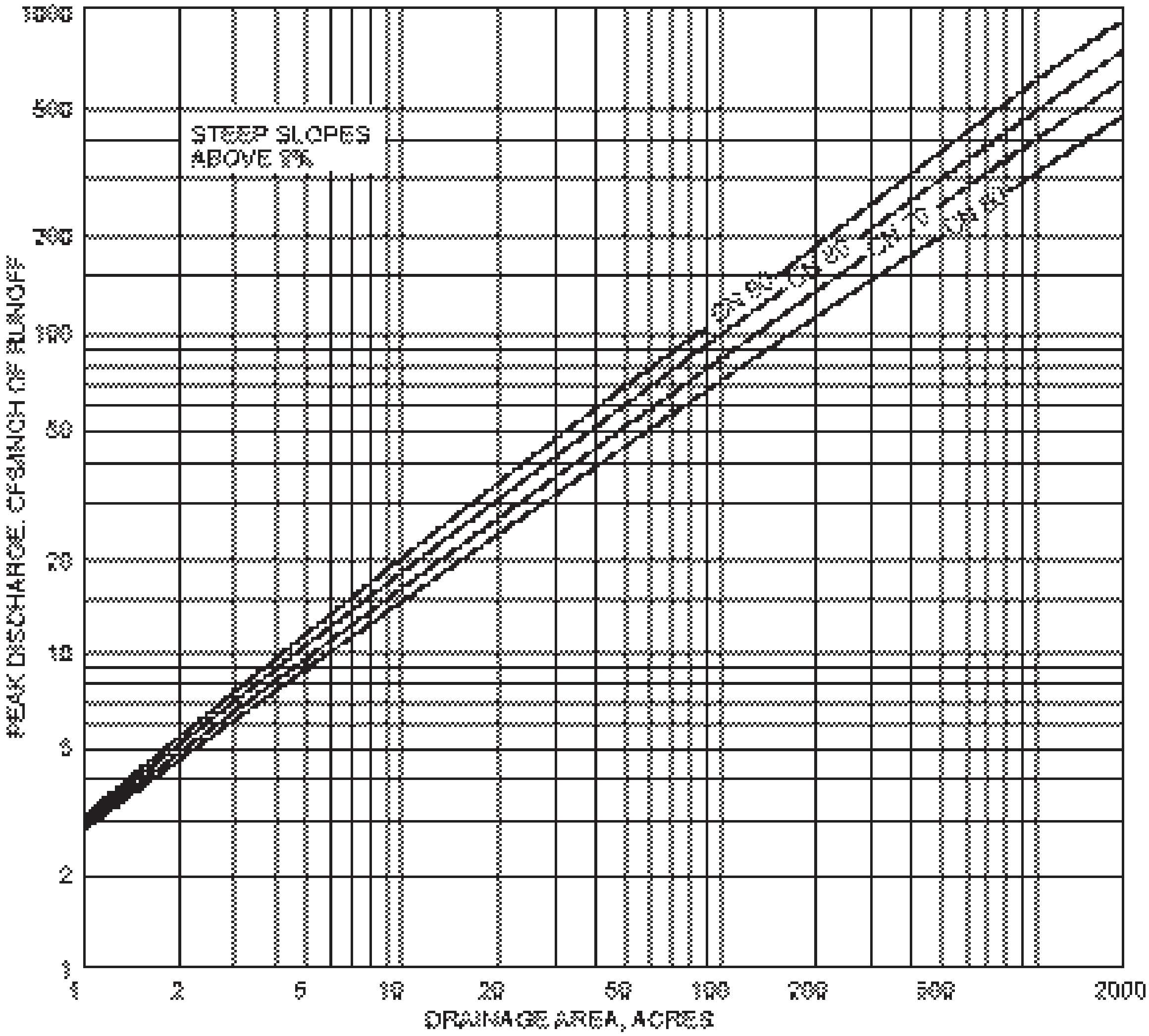


Figure A-2.5. - Peak Rates of Discharge for Small Watersheds (24-Hour, Type II Storm Distribu-

ing peak rates of discharge for ranges of flat, moderate, and steep slopes; for conditions where swamps or ponding areas exist; and for conditions where the watershed shape factor (l/w) varies significantly from that used in the development of the charts of Figures A-2.3 through A-2.5.

SLOPE INTERPOLATION

Table A-2.1 provides interpolation factors to be used in determining peak rates of discharge for specific slopes within ranges of flat, moderate, and steep slopes for a range of drainage areas. Figure A-2.3, for FLAT slopes is based on 1-percent slope, Figure A-2.4, for MODERATE slopes on 4-percent slope, and Figure A-2.5 for STEEP slopes on 16-percent slope. For slopes other than 1, 4, and 16 percent, use the factors shown in Table A-2.1 to modify the peak discharges.

Example A-2.2

Compute the peak discharge for a 1,000-acre watershed with an average watershed slope of 7 percent and a runoff curve number (CN) of 80 for central Lee County, 2-year/24-hour storm.

1. Determine the peak discharge for a watershed with a moderate slope (4 percent). From Figure A-2.4, read a peak discharge of 295 cfs per inch of runoff for 1,000 acres and a CN of 80. From Figure A-2.8, Lee County has a P value of 4.0 inches. From TR-55, Table 2-1 (Appendix A-1) find 2.04 inches of runoff from 4 inches of rainfall and a CN of 80. The peak discharge is then 295 x 2.04 or 602 cfs.

Table A-2.1 - Slope Adjustment Factors by Drainage Areas

Slope (percent)	10 acres	20 acres	50 acres	100 acres	200 acres	500 acres	1,000 acres	2,000 acres
FLAT SLOPES								
0.1	0.49	0.47	0.44	0.43	0.42	0.41	0.41	0.40
0.2	.61	.59	.56	.55	.54	.53	.53	.52
0.3	.69	.67	.65	.64	.63	.62	.62	.61
0.4	.76	.74	.72	.71	.70	.69	.69	.69
0.5	.82	.80	.78	.77	.77	.76	.76	.76
0.7	.90	.89	.88	.87	.87	.87	.87	.87
1.0	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
1.5	1.13	1.14	1.14	1.15	1.16	1.17	1.17	1.17
2.0	1.21	1.24	1.26	1.28	1.29	1.30	1.31	1.31
MODERATE SLOPES								
3	.93	.92	.91	.90	.90	.90	.89	.89
4	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
5	1.04	1.05	1.07	1.08	1.08	1.08	1.09	1.09
6	1.07	1.10	1.12	1.14	1.15	1.16	1.17	1.17
7	1.09	1.13	1.18	1.21	1.22	1.23	1.23	1.24
STEEP SLOPES								
8	.92	.88	.84	.81	.80	.78	.78	.77
9	.94	.90	.86	.84	.83	.82	.81	.81
10	.96	.92	.88	.87	.86	.85	.84	.84
11	.96	.94	.91	.90	.89	.88	.87	.87
12	.97	.95	.93	.92	.91	.90	.90	.90
13	.97	.97	.95	.94	.94	.93	.93	.92
14	.98	.98	.97	.96	.96	.96	.95	.95
15	.99	.99	.99	.98	.98	.98	.98	.98
16	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
20	1.03	1.04	1.05	1.06	1.07	1.08	1.09	1.10

2. Determine the interpolation factor. From Table A-2.1, find 7-percent slope under MODERATE heading and read an interpolation factor of 1.23 for a drainage area of 1,000 acres. (The peak from a 1,000-acre watershed with a watershed slope of 7 percent is 1.23 times greater than for an average watershed slope of 4 percent.)

3. Determine the peak discharge of 7-percent slope.

q = (602)(1.23) = 740 cfs

Examples A-2.3

Compute the peak discharge for a 15-acre watershed with an average slope of 0.5 percent and a runoff curve number of 80 for 4 inches of rainfall.

1. Determine the peak discharge for a watershed with a flat slope (1 percent). From Figure A-2.3 read a peak discharge of 11.2 cfs per inch of runoff for 15 acres and a CN of 80. From Table A-2.1, find 2.04 inches of runoff for 4 inches of rainfall and a CN of 80. The peak discharge is then 11.2 x 2.04 or 23 cfs.

2. Determine the interpolation factor. From Ta-

Table A-2.2. - Peak Flow Adjustment Factors where Ponding and Swampy Areas occur at the

Ratio of drainage area to ponding and swampy area	Percentage of ponding and swampy area	Storm frequency (years)						
		1	2	5	10	25	50	100
500	0.2	0.91	0.92	0.94	0.95	0.96	0.97	0.98
200	.5	.85	.86	.87	.88	.90	.92	.93
100	1.0	.79	.80	.81	.83	.85	.87	.89
50	2.0	.73	.74	.75	.76	.79	.82	.86
40	2.5	.68	.69	.70	.72	.75	.78	.82
30	3.3	.63	.64	.65	.67	.71	.75	.78
20	5.0	.58	.59	.61	.63	.67	.71	.75
15	6.7	.56	.57	.58	.60	.64	.67	.71

Table A-2.3. - Peak Flow Adjustment Factors where Ponding and Swampy Areas are Spread Throughout the Watershed or occur in Central Parts of the Watershed

Ratio of drainage area to ponding and swampy area	Percentage of ponding and swampy area	Storm frequency (years)						
		1	2	5	10	25	50	100
500	0.2	0.93	0.94	0.95	0.96	0.97	0.98	0.99
200	.5	.87	.88	.89	.90	.91	.92	.94
100	1.0	.83	.83	.84	.86	.87	.88	.90
50	2.0	.77	.78	.79	.81	.83	.85	.87
40	2.5	.72	.73	.74	.76	.78	.81	.84
30	3.3	.68	.69	.70	.71	.74	.77	.81
20	5.0	.64	.65	.66	.68	.72	.75	.78
15	6.7	.61	.62	.63	.65	.69	.72	.75
10	10.0	.57	.58	.59	.61	.65	.68	.71

Table A-2.4. - Peak Flow Adjustment Factors where Ponding and Swampy Areas are Located only in Upper Reaches if the Watershed

Ratio of drainage area to ponding and swampy area	Percentage of ponding and swampy area	Storm frequency (years)						
		1	2	5	10	25	50	100
500	0.2	0.95	0.96	0.97	0.98	0.98	0.99	0.99
200	.5	.92	.93	.94	.94	.95	.96	.97
100	1.0	.89	.90	.91	.92	.93	.94	.95
50	2.0	.86	.87	.88	.88	.90	.91	.93
40	2.5	.84	.85	.85	.86	.88	.89	.91
30	3.3	.81	.82	.83	.84	.86	.88	.89
20	5.0	.79	.80	.81	.82	.84	.86	.88
15	6.7	.77	.78	.79	.80	.82	.84	.86

ble A-2.1 find 0.5-percent slope under FLAT heading. Read a slope interpolation factor of 0.81 interpolated between the values for 10 acres and 20 acres.

3. Determine the peak discharge for 0.5-percent slope.

q = (23)(.81) = 19 cfs

ADJUSTMENT FACTORS FOR SWAMPY AND PONDING AREAS

Peak flows determined from Figure A-2.3 through A-2.5 assume that the topography is such that surface flow into ditches, drains, and streams is approximately uniform. On very flat areas and where ponding or swampy areas occur in the watershed, a considerable amount of the surface runoff may be retained in temporary storage. The peak rate of runoff should be reduced to reflect this condition. Tables A-2.2, A-2.3, and A-2.4 provide adjustment factors to determine this reduction based on the ratio of the ponding or swampy area to the total watershed area for a range of storm frequencies.

Table A-2.2 contains adjustment factors to be used when the ponding or swampy areas are located in the path of flow in the vicinity of the design point. Table A-2.3 contains adjustment factors to be used when a significant amount of the flow from the total watershed passes through ponding or swampy areas and these areas are spread throughout the watershed. Table A-2.4 contains adjustment factors to be used when a significant amount of the flow, passes through ponding or swampy areas located in the upper reaches of the watershed.

These conditions may occur in a proposed

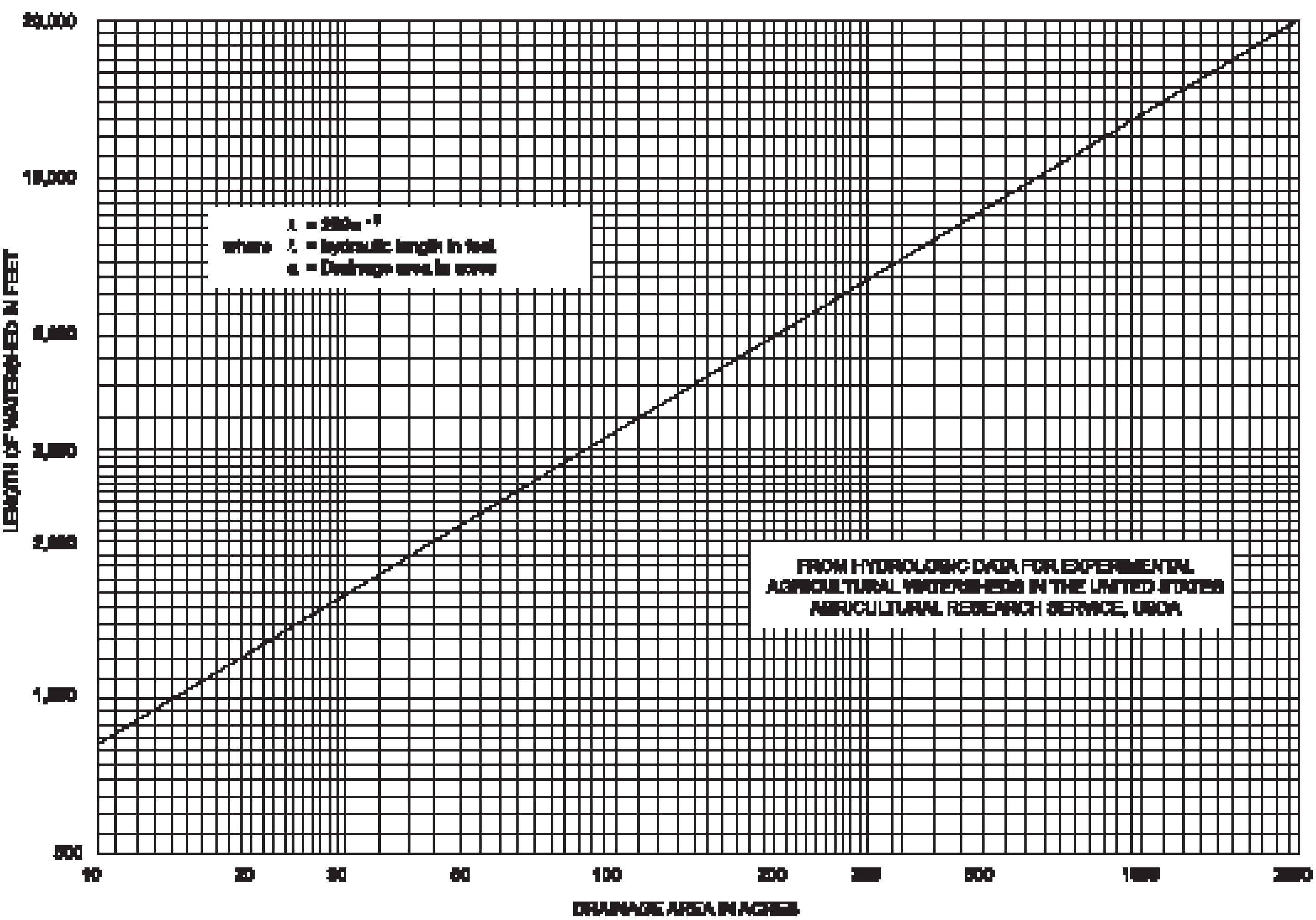


Figure A-2.6 - Hydraulic Length and Drainage Area Relationship or existing urban or suburban area and the adjustment factors from Tables A-2.2, A-2.3, or A-2.4 should be applied after the peaks have been adjusted for the effects of urbanization.

Example A-2.4

A 5-acre pond is located at the downstream end of a 100-acre watershed in which a housing development is proposed. The average watershed slope is 4 percent and the present-condition curve number is 75. After the installation of the housing development, 30 percent of the watershed will be impervious and 50 percent of the hydraulic length will be modified. The future-condition curve number is estimated to be 80. For a 100-year storm 24-hour duration in central Glascock County, determine the present-condition and future-condition peak discharges downstream of the pond.

1. Determine the present-condition peak discharge assuming the pond is not in place: From Figure A-2.4, find the peak discharge to be 59 cfs per inch of runoff. From Figure A-2.13, the rainfall for central Glascock County is 8 inches. From TR-55, Table 2.1 (Appendix A-1) find the runoff to be 5.04 inches. The peak discharge is 59 x 5.04 or 297 cfs.

2. Determine the ponding adjustment factor: Since the pond is at the lower end of the watershed, use Table A-2.2. The ratio of the drainage area to pond area is 100/5 or 20. For a 100-year frequency event, the adjustment factor is 0.75.

3. Compute the present-condition peak discharge:

   Q = 0.75 (297) = 233 cfs

4. Compute the basic future-condition peak discharge: From Figure A-2.4, find the peak discharge to be 65 cfs per inch of runoff. From TR-55, Table 2-1, (Appendix A-1), Find the runoff to be 5.62 inches The peak discharge is then 65 x 5.62 or 365 cfs.

5. Determine the modification factors for proposed urbanization: Taken from Figures A-2.1 and A-2.2 for a curve number of 80: impervious factor = 1.16; hydraulic length factor = 1.31; urbanization factor = (1.16) (1.31) = 1.52.

6. Compute the future condition peak discharge:

   q = 1.52 (365) = 555 cfs

7. Compute the future-condition peak below the pond: From step 2 the ponding factor is 0.75.

   q = 0.75 (555) = 416 cfs

ADJUSTMENT FOR WATERSHED SHAPE FACTOR

The equation used in computing peak discharges from Figures A-2.3 through A-2.5 was based in part on a relationship between the hydraulic length and the watershed area from Agricultural Research Services's studies on small experimental watersheds. Figure A-2.6 shows the best fit line relating length to drainage area. The equation of the line is $l = 209a^{0.6}$. A watershed shape factor, l/w (where w is the average width of the watershed), is then fixed for any given drainage area. For example, for drainage areas of 10, 100, and 1,000 acres, the watershed shape factor is 1.58, 2.51, and 3.98, respectively.

There are watersheds that deviate considerably from these relationships. The peaks can be modified for other shape factors. The procedure is as follows:

1. Determine the hydraulic length of the watershed and compute "equivalent" drainage area using $l = 209a^{0.6}$ or Figure A-2.6.

2. Determine the "equivalent" peak flow from the charts for the "equivalent" drainage area.

3. Compute the "actual" peak discharge for the watershed by multiplying the equivalent peak discharge by the ratio of actual drainage area to the equivalent drainage area.

The factors for modifying the peak for

urbanization can then be applied to the revised peak discharge.

Example A-2.5

From a topographic map the hydraulic length of a 100-acre watershed with moderate slopes and a CN of 75 was measured to be 2,200 feet. Determine the peak discharge for a 6-inch, 24-hour rainfall.

1. Determine the equivalent drainage area for a watershed with a hydraulic length of 2,200 feet. From Figure A-2.6, read 51 acres. (Note that in a 100-acre watershed, the hydraulic length would be 3,300 feet from Figure A-2.6).

2. Determine the "equivalent" peak flow from Figure A-2.4 for a drainage area of 51 acres and a CN of 75. Read 37 cfs per inch of runoff. From TR-55 Table 2-1 (Appendix A-1), find the runoff to be 3.28 or 121 cfs.

3. Compute the actual peak discharge for 100 acres.

$$\text{actual discharge} = \text{equivalent discharge} \left(\frac{\text{actual drainage area}}{\text{equivalent drainage area}}\right)$$

$$q = 121 \left(\frac{100}{51}\right) = 237 \text{ cfs}$$

The peak discharge for the 100-acre watershed with a hydraulic length of 2,200 feet is 237 cfs (versus 194 cfs for a "normal" 100-acre watershed). Adjustments to this peak discharge for urbanization can be made using factors discussed on page A-2-1.

4. The procedure in steps 1, 2, and 3 can be used to determine peak discharges when the actual hydraulic length is longer than that shown on Figure A-2.6. For example, if the actual length were 4,500 feet instead of 3,300 feet, the equivalent area would be 170 acres, as shown in Figure A-2.6.