TABLE OF CONTENTS

LIST OF FIGURES

CHAPTER TWENTY-NINE

HYDROLOGY

29-1.0 HYDROLOGIC DESIGN POLICIES

29-1.01 Introduction

The following is a summary of policies which apply to hydrologic analysis. For more information, see the AASHTO *Highway Drainage Guidelines*.

29-1.02 Surveys

Hydrologic considerations can influence the selection of a highway corridor and the alternative routes within the corridor. Studies and investigations should be performed, including the consideration of the environmental and ecological impact of the project. The magnitude and complexity of these studies should be commensurate with the importance and magnitude of the project and the problems encountered. The data to be included in these surveys or studies include topographic maps, aerial photographs, streamflow records, historical high-water elevations, flood discharges, or locations of hydraulic features such as reservoirs, water projects, or designated or regulatory floodplain areas.

29-1.03 Flood-Hazard Areas

A hydrologic analysis is a prerequisite to identifying flood-hazard areas and determining the locations at which construction and maintenance will be unusually expensive or hazardous.

29-1.04 Coordination

Interagency coordination is necessary because many levels of government plan, design, and construct highway and water resource projects which can have an effect on each other. Agencies can share data and experiences within project areas to assist in the completion of accurate hydrologic analyses. The agencies include the Indiana Department of Natural Resources (IDNR), U.S. Fish and Wildlife Service (USFS), U.S. Army Corps of Engineers (USACOE), watershed management organizations, Natural Resources Conservation Service (NRCS), U.S. Geological Survey (USGS), and counties and cities.

29-1.05 Documentation

The design of a highway drainage facility should be adequately documented. It is often necessary to refer to plans or specifications long after the actual construction has been completed. Documentation should include final computations, method of analysis selected, drainage area map, designer's name and date, project correspondence relative to hydraulic considerations, and permit information. See Section 28-5.0 for Department guidelines on documentation for hydrologic information.

29-1.06 Evaluation of Runoff Factors

For each hydrologic analysis, the following must be evaluated and included if they will have a significant effect on the final results.

1. Drainage-basin characteristics including size, shape, slope, land use, geology, soil type, surface infiltration, and storage.

2. Stream channel characteristics including geometry and configuration, natural and artificial controls, channel modification, aggradations or degradation, and ice and debris.

3. Floodplain characteristics.

4. Meteorological characteristics such as precipitation amounts and type, distribution characteristics, and time rate of precipitation (hyetograph).

5. Where appropriate, the designer should evaluate future land use changes that can occur during the service life of the proposed facility and that can result in an inadequate drainage system.

29-1.07 Flood History

Each hydrologic analysis must consider the flood history of the area and the effect of such historical floods on each existing or proposed structure. The flood history must include the historical floods and the flood history for each existing structure.

29-2.0 OVERVIEW

29-2.01 Introduction

The analysis of the peak rate of runoff, volume of runoff, and time distribution of flow is fundamental to the design of a drainage facility. The design of each highway drainage facility requires the determination of discharge-frequency relationships. One design may require a peak-flow rate while another requires a runoff hydrograph providing an estimate of runoff volume. The peak-flow rate is used in the design of a bridge, culvert, roadside ditch, or small storm-sewer system. A drainage system involving detention storage, pumping stations, or large or complex storm-sewer systems require the development of a runoff hydrograph.

Errors in the estimates will result in a structure that is either undersized and causes more drainage problems, or oversized and costs more than necessary. A hydrologic analysis is only an approximation. The relationship between the amount of precipitation on a drainage basin and the amount of runoff from the basin is complex. Insufficient data is available concerning the factors influencing the rural and urban rainfall-runoff relationship to expect exact solutions.

29-2.02 Definition

Hydrology is defined as a science which explores the interrelationship between water on and under the earth and in the atmosphere. For this *Manual*, hydrology will address estimating flood magnitudes as the result of precipitation. In the design of a highway-drainage structure, floods are considered in terms of peak runoff or discharge in cubic feet per second (ft^3/s) and hydrographs as discharge per time. For a structure which is designed to control the volume of runoff (e.g., detention storage facility) or where flood routing through a culvert is used, the entire discharge hydrograph will be of interest. Wetland hydrology, the water-related driving force to create a wetland, is addressed in the AASHTO *Highway Drainage Guidelines*, Volume X.

29-2.03 Factors Affecting a Flood

In the hydrologic analysis for a drainage structure, the designer must recognize that there are many variables that affect a flood. Some of the factors which must be considered on an individual site-by-site basis include the following:

1. rainfall amount and storm distribution;
2. drainage area size, shape, and orientation;
3. ground cover and soil type;
4. slopes of terrain and streams;
5. antecedent moisture condition;
6. storage potential (overbank, pond, wetlands, reservoir, channel, etc.);

7. watershed-development potential;
8. type of precipitation (rain, snow, hail, or combinations thereof); and
9. elevation and mixed-population events.

29-2.04 Sources of Information

The type and source of information available for hydrologic analysis will vary from site to site, and it is the responsibility of the designer to determine what information is available and applicable to each analysis. Sources of information include, but are not limited to the following:

1. discharge information, IDNR;
2. topographic maps, USGS;
3. county soil maps, NRCS;
4. stream flow data and regression equations, USGS;
5. hydrology studies, NRCS;
6. flood insurance studies, IDNR;
7. watershed data studies done by other units of government;
8. rainfall data, U.S. Weather Bureau;
9. aerial photos, INDOT;
10. flood data, U.S. Army Corps of Engineers;
11. site visits; and
12. State, county, or local maps, as appropriate.

29-3.0 SYMBOLS AND DEFINITIONS

To provide consistency within this Chapter and throughout this *Manual*, the symbols in Figure 29-3A, Hydrologic Symbols and Definitions, will be used. These symbols have been selected because of their widespread use in hydrologic publications.

29-4.0 CONCEPT DEFINITIONS [REV. JAN. 2011]

The following discusses concepts which will be important in a hydrologic analysis. These concepts will be used throughout the remainder of this Chapter in addressing different aspects of hydrologic studies.

1. Antecedent Moisture Conditions. These are the soil-moisture conditions of the watershed at the beginning of a storm. These conditions affect the volume of runoff generated by a specific storm event. They affect the peak discharge only in the lower range of flood

magnitude (i.e., below about the 15-year event threshold). As flooding becomes rarer, antecedent moisture has a rapidly-decreasing influence on runoff.

2. Depression Storage. This consists of the natural depressions within a watershed which store runoff. After the depression storage is filled, runoff will commence.

3. Frequency. This is the number of times a flood of a given magnitude can be expected to occur on average over a long period of time. Frequency analysis is the estimation of peak discharges for various recurrence intervals. Frequency can also be expressed by means of probability. Probability analysis seeks to define the flood flow with a probability of being equaled or exceeded in a given year.

4. Hydraulic Roughness. This is a composite of the physical characteristics which influence the flow of water across the earth's surface, whether natural or channelized. It affects both the time response of a watershed and drainage channel and the channel storage characteristics.

5. Hydrograph. This is a graph of the time distribution of runoff from a watershed.

6. Hyetographs. This is a graph of the time distribution of rainfall over a watershed.

7. Infiltration. This is a complex process of allowing runoff to penetrate the ground surface and flow through the upper soil surface. The infiltration curve is a graph of the time distribution at which this occurs.

8. Interception. This consists of storage of rainfall on foliage or another intercepting surface during a rainfall event.

9. Lag Time. This is the time from the centroid of the excess rainfall to the peak of the hydrograph.

10. Peak Discharge. Also identified as peak flow, is the maximum rate of flow of water passing a given point during or after a rainfall event or snowmelt.

11. Rainfall Excess. This is the water available to runoff after interception, depression storage, and infiltration requirements have been satisfied.

12. Rainfall Intensity. This is the amount of rainfall occurring in a unit of time, converted to its equivalent in inches per hour.

13. Recurrence Interval. This is the average number of years between occurrences of a discharge or rainfall that equals or exceeds the given magnitude.

14. Runoff. This is the portion of the precipitation which runs off the surface of a drainage area after all abstractions are accounted for.

15. Runoff Coefficient. This is a factor representing the portion of runoff resulting from a unit rainfall. It is dependent on topography, land use, and soil characteristics.

16. Stage. The stage of a river is the elevation of the water surface above an elevation datum.

17. Time of Concentration. This is the time required for a drop of water falling on the hydraulically most-remote point in the watershed to travel through the watershed to the point under investigation.

18. Ungaged Stream Site. This is a location at which no systematic records are available for actual stream flow.

19. Unit Hydrograph. This is the direct runoff hydrograph resulting from a rainfall event which has a specific temporal and spatial distribution, which lasts for a specific duration, and which has unit volume (or results from a unit depth of rainfall). The ordinates of the unit hydrograph are such that the volume of direct runoff represented by the area under the hydrograph is equal to one inch of runoff from the drainage area. If a unit hydrograph is shown with units of cubic feet per second, it is implied that the ordinates are cubic feet per second per inch of direct runoff.

20. Volumetric Runoff Coefficient, R_V. This is used in water-quality calculations. It represents the portion of rainfall that becomes runoff and is dependent on percentage of impervious cover.

21. Water-Quality Volume. This is the treatment volume or accumulated direct runoff depth that should be treated to remove a significant percentage of the stormwater pollution load, or approximately 80% of the average annual post-development total suspended solids loading. Water-quality volume is identified as WQ_V for units of acre-feet, or Q_{WV} for units of inches. Water-quality volume is used to design a detention-based water-quality-treatment system such as a stormwater wetland, wet detention pond, or water-quality swale.

22. Water-Quality-Treatment Rate, Q_{WA}. This is the design flow rate for flow through a water-quality system. A properly designed flow-through system, such as an infiltration trench or hydrodynamic separator, should be able to satisfy pollutant-removal

requirements at the design-flow rate, and should be able to bypass inflows greater than the design treatment rate.

For a more-complete discussion of these concepts and others related to hydrologic analysis, the designer should see *Hydrologic Design For Highways*, Federal Highway Administration, Hydraulic Design Series 2, 1995; and *Guidelines for Hydrology - Volume II Highway Drainage Guidelines*, Task Force On Hydrology and Hydraulics, AASHTO Highway Subcommittee on Design.

29-5.0 DESIGN FREQUENCY

29-5.01 Overview

Because it is not economically feasible to design a structure for the maximum runoff that a watershed is capable of producing, a design frequency must be established. The design frequency for a given flood is defined as the reciprocal of the probability or chance that a flood will be equaled or exceeded in a given year. If a flood has a 20 percent chance of being equaled or exceeded each year over a long period of time, the flood will be equaled or exceeded on average once every five years. This is called the return period or recurrence interval (RI). Thus, the exceedance probability equals 100/RI. The 5-year flood is not one that will necessarily be equaled or exceeded every five years. There is a 20 percent chance that the flood will be equaled or exceeded in a given year. Therefore, the 5-year flood can conceivably occur in several consecutive years. The same reasoning applies to a flood with another return period.

INDOT has related design frequency to roadway serviceability. Roadway serviceability may be defined as travel lanes open to traffic with no floodwaters encroaching into the travel lanes during a design storm. The higher functional classifications require design-flood frequencies of less-frequent storms than the lower functional classifications.

29-5.02 Design Frequency

The design frequency used to design a hydraulic facility is determined by the type, size, and location of the structure. The following applies to the design frequency for the indicated drainage application.

1. Cross Drainage. A drainage facility should be designed to accommodate a discharge with a given return period(s) for the following circumstances. The design should be such that the backwater (the headwater) caused by the structure for the design storm does not cause the following:

a. significantly increases the flood hazard for property;
b. overtop the highway; or
c. exceeds a certain depth on the highway embankment.

Based on these design criteria, a design involving temporary roadway overtopping for a flood larger than the design event is acceptable practice. If overtopping is allowed, the structure may be designed to accommodate a flood of a lesser frequency without overtopping.

2. Storm Drain. A storm drain should be designed to accommodate a discharge with a given return period(s) for the following circumstances. The design shall be such that the storm runoff does not cause the following:

   a. significantly increases the flood hazard for property;
   b. encroach onto the street or highway so as to cause a significant traffic hazard; or
   c. limit traffic, emergency vehicles, or pedestrian movement to an unreasonable extent.

   Based on these design criteria, a design involving temporary street or road inundation for a flood larger than the design event is acceptable practice.

See Figure 29-5A, Design Frequency (Return Period – Years).

29-5.03 Review Frequency

Where appropriate, the design of a hydraulic structure should include an assessment of flood hazards inherent in the proposed facility for frequencies other than the design frequency. After sizing a drainage facility using a flood or possibly the hydrograph corresponding to the design frequency, it is necessary to review the proposed facility with a base discharge. This is done to ensure that there are no unexpected flood hazards inherent in the proposed facility. Where the design Q is less than Q_{100}, the review flood should be the 100-year event. If available, discharges should be obtained from the coordinated discharge curves, which are shown in the IDNR publication *Coordinated Discharges of Selected Streams in Indiana* and in FEMA/NFIP publications.

Potential impacts to consider include possible flood damage due to a high embankment where overtopping is not practical, backup due to the presence of a median barrier or noise wall, or flood damage due to a storm-sewer backup. Potential scour damage to a bridge substructure should be reviewed for the 500-year frequency.

29-5.04 Rainfall Curves

Rainfall data are available for many geographic areas. From these data, rainfall intensity-duration-frequency (IDF) curves have been developed for the commonly-used design frequencies. The IDF curves are shown in Section 29-8. The curves have been developed using HYDRAIN's HYDRO module, and they are based on National Weather Service (NWS) technical memorandum, HYDRO-35. HYDRO may be used to develop the IDF for a specific location with known latitude and longitude for a duration up to 60 min.

29-6.0 HYDROLOGIC PROCEDURE SELECTION

29-6.01 Overview

Streamflow measurements for determining a flood frequency relationship at a site are usually unavailable. Therefore, it is accepted practice to estimate peak runoff rates and hydrographs using statistical or empirical methods. Results from using several methods should be compared, not averaged. The designer should review the design discharge for other structures on the stream and historical data and consider previous studies including flood-insurance studies. INDOT's practice is to use the discharge that best reflects local project conditions with the reasons documented. The following discusses INDOT's use for each procedure.

29-6.02 Peak-Flow Rate or Hydrograph

A consideration of peak-runoff rate for the design condition is adequate for a conveyance system such as a storm drain or open channel. However, if the design must include flood routing (e.g., storage basin, complex conveyance network), a flood hydrograph is required. Although the development of a runoff hydrograph (more complex than estimating peak-runoff rate) is accomplished using computer programs, some methods are adaptable to nomographs or other desktop procedures.

29-6.03 Hydrologic Methods

Where feasible, for a large structure, more than one method of computing discharge should be checked, comparing the results to what other structures in the area are designed for and the historical data for the area. Engineering judgment should then be used to select the discharge. If practical, the method should be calibrated to local conditions and tested for accuracy and reliability.

Figure 29-6A, Selection of Discharge Computation Method, summarizes the recommended hydrologic methods currently acceptable for use and their application in the design of a highway structure. The following provides additional guidance on the selection of hydrologic methods.

1. IDNR Coordinated Discharge Curves. This is the preferred method for a stream for which the information is available. The reference is *Coordinated Discharges of Selected Streams in Indiana*.

2. IDNR Letter of Discharge. The IDNR Letter of Discharge must be prepared for a structure that requires a Construction in a Floodway Permit.

3. NRCS (formerly SCS) Unit Hydrograph Method (TR-20). This method can be used to determine peak discharge and hydrograph in a rural area for a given basin size.

4. HEC I. This hydrograph method can be used to determine peak discharge and hydrograph in a rural area for a given basin size, or in an urban area with large watersheds.

5. Indiana USGS Regression Equations. This method can be used in a rural area for estimating if no other method is available.

6. Rational Method. This is the preferred method for a developed area. It can be used for a drainage area of less than 100 acres in an urban area or that of less than 200 acres in a rural area.

7. FEMA. The 100-year discharge specified in the applicable FEMA flood-insurance study should be used to analyze impacts of a proposed crossing on a regulatory floodway. However, if the discharge is considered outdated, the discharge based on current methods may be used subject to receiving the necessary regulatory approvals.

8. Frequency Analysis of Stream-Gaging Records. The IDNR Division of Water maintains a database of discharges for various frequencies computed using methodologies included in Water Resources Council Bulletin 17B. Comparisons of discharges computed for nearby gages can be of value.

29-7.0 TIME OF CONCENTRATION

29-7.01 Overview

The time of concentration, t_C, is the time required for water to flow from the hydraulically most-remote point of the drainage area to the point under investigation. Time of concentration is an important variable in many hydrologic methods, including the Rational and Natural Resources Conservation Service (formerly SCS) procedures. For the same size watershed, the shorter the t_C, the larger the peak discharge.

29-7.02 Procedure

Water moves through a watershed as a combination of overland and channelized flow. The type that occurs is a function of the conveyance system and is best determined by field inspection. In designing a drainage system, the overland flow path is not necessarily perpendicular to the contours shown on available mapping. The land will often be graded, and swales will intercept the natural contour and conduct the water to the streets which reduces the time of concentration. The overland-flow path should be less than 100 ft.

29-7.02(01) Available Methods

See Figure 29-7A, Methods for Calculating Time of Concentration.

29-7.02(02) Selection of Method

The methods included in this Chapter are applicable for both the Rational Equation and the NRCS (formerly SCS) Peak Flow or Hydrograph Methods. In the Rational Equation, t_C is expressed in minutes. In the NRCS procedures, t_C is expressed in hours.

To choose a method, consider the conditions for which the equation was developed and how they compare to the drainage area being designed. If NRCS methods will be used to compute discharge, t_C should be determined using the methods recommended by the NRCS.

29-7.02(03) Total Time of Concentration

To obtain the total time of concentration, the channel-flow time must be calculated and added to the overland-flow time. After first determining the average flow velocity in the pipe or channel, the travel time, t_t, is obtained by dividing velocity into the pipe or channel length.

$$t_t = \frac{L}{60V} \qquad \text{(Equation 29-7.1)}$$

Where:
t_t = travel time, min
L = length which runoff must travel, ft
V = estimated or calculated velocity, ft/s

The total time of concentration is as follows:

$$t_C = t_O + t_t \qquad \text{(Equation 29-7.2)}$$

Where:
t_C = total time of concentration
t_O = overland flow time
t_t = travel time

29-7.02(04) Storm-Drainage System

For a storm-drainage system, the time of concentration for an area consists of an inlet time plus the time of flow in a closed conduit or open channel to the design point. Inlet time is the sum of the time required for water to move across the pavement or overland back of the curb to the gutter, plus the time required for flow to move through the length of gutter to the inlet. If the total time of concentration for pavement-drainage inlets is less than 5 min, a minimum of 5 min should be used to estimate the duration of rainfall.

29-7.03 NRCS Curve Number

The Natural Resources Conservation Service (NRCS) (formerly SCS) Curve Number method may be used to estimate the total time of concentration for a small rural area of 3 acres to 2000 acres (NRCS, 1989). Use Equation 29-7.3 to estimate the time of concentration from a natural, homogeneous watershed with the same curve number as follows:

$$t_C = \frac{(L)^{0.8}\left(\frac{1000}{CN} - 9\right)^{0.7}}{1140Y^{0.5}} \qquad \text{(Equation 29-7.3)}$$

Where:
t_C = time of concentration, h
L = length of mainstream to farthest divide, ft
Y = average watershed slope, %
CN = NRCS curve number (Section 29-10)

The above equation should only be used for a rural watershed with a flow length between 200 ft and 26,000 ft (5 mi), and an average watershed slope between 0.5% and 64%. This method is included in HYDRO as an option for calculating t_C.

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Example 29-7.1

Given: L = 660 ft Y = 2 % CN = 77

Find: Time of concentration, t_C, using Equation 29-7.3.

Solution:
$$t_C = \frac{(660)^{0.8}\left(\frac{1000}{77} - 9\right)^{0.7}}{1140(2)^{0.5}} = 0.29\ h = 17\ \text{min}$$

* * * * * * * * * *

29-7.04 Kinematic Wave Equation

HEC No. 12 (FHWA, 1984) recommends the kinematic wave equation as the most-realistic method for estimating overland flow time of concentration. The equation is as follows:

$$t_O = \frac{0.93 L^{0.6} n^{0.6}}{(Ci)^{0.4} S^{0.3}} \qquad \text{(Equation 29-7.4)}$$

Where:
- t_O = time of overland flow, min
- L = overland flow length, ft
- n = Manning roughness coefficient
- C = runoff coefficient
- i = rainfall rate, in./h
- S = average slope of the overland area, decimal

In using the equation, both the time of concentration and rainfall intensity are unknown and iteration is required. A value for i is first assumed and the related time of concentration found. The assumed rainfall intensity must then be checked against the rainfall Intensity-Duration-Frequency curve for the frequency of the event chosen for the particular design problem, and the procedure repeated until the assumed rainfall intensity approximately agrees with the intensity associated with the time of concentration. This method is included in HYDRO.

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Example 29-7.2

Given: L = 150 ft
S = 0.02
n = 0.24 (dense grass)
C = 0.40 (impervious soil with turf)
Design frequency = 10 yr
Location: Indianapolis

Find: Overland flow time, t_O, using Equation 29-7.4.

Solution:

1. Assume i = 4.8 in./h and calculate t_O as follows:

$$t_O = \frac{0.93(150)^{0.6}(0.24)^{0.6}}{[(0.40)(4.8)]^{0.4}(0.02)^{0.3}} = 20 \text{ min}$$

2. Based on calculated t_O, find i from Figure 29-8D, Rainfall Intensity-Duration-Frequency Curve (Indianapolis):

i = 4 in./h

3. Calculate t_O as follows:

$$t_O = \frac{0.93(150)^{0.6}(0.24)^{0.6}}{[(0.40)(4)]^{0.4}(0.02)^{0.3}} = 21 \text{ min}$$

4. Based on calculated t_O, find i from Figure 29-8D.

i = 3.92 in./h, assumed value was 4 in./h; therefore, t_O = 21 min

29-7.05 Manning's Kinematic Solution

For sheet (overland) flow of less than 300 ft, TR-55 (NRCS, 1986) recommends Manning's kinematic solution (Overton and Meadows, 1976) to compute t_O. This method is included in the TR-55 computer program. The equation is as follows:

$$t_O = \frac{0.42(nL)^{0.8}}{(P_2)^{0.5} S^{0.4}} \qquad \text{(Equation 29-7.5)}$$

Where:

- t_O = overland flow time, min
- n = Manning's roughness coefficient, Figure 29-7B, Roughness Coefficients for the Rational Formula
- L = flow length, ft
- P_2 = 2-year, 24-h rainfall, in (from TP-40)
- S = slope of hydraulic grade line (land slope), decimal

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 (rain available for runoff);
3. rainfall duration of 24 h; and
4. minor effect of infiltration on travel time.

This overland time of concentration is acceptable for use within the TR-20 hydrologic methodology.

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<u>Example 29-7.3</u>

Given:
L = 150 ft
S = 0.02
n = 0.24 (dense grass)
Location: Indianapolis

Find: Overland flow time, t_O, using Equation 29-7.5.

Solution:

1. For Indianapolis, P_2 = 2.6 in, from TP 40.

2. Determine t_O as follows:

$$t_O = \frac{0.42[(0.24)(150)]^{0.8}}{(2.6)^{0.5}(0.02)^{0.4}} = 0.36\ h = 21.8\ \text{min}$$

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29-7.06 Federal Aviation Administration Method

For design conditions that do not involve complex drainage conditions, the Federal Aviation Administration Equation (FAA, 1970) can be used to estimate overland flow time. Equation 29-7.6 was developed from airport-drainage data, and it is best suited for a small drainage area with fairly homogeneous surfaces. For each drainage area, the distance is determined from the inlet to the most remote point in the tributary area. From a topographic map, the average slope is determined for the same distance. Figures 29-8A and 29-8B, Runoff Coefficients for the Rational Formula, provide values for the Rational Method runoff coefficient, *C*.

$$t_O = \frac{(1.1 - C)L^{0.5}}{2.63 S^{0.33}} \qquad \text{(Equation 29-7.6)}$$

Where:
- t_O = overland flow travel time, min
- L = overland flow path length, ft
- S = slope of overland flow path, decimal
- C = Rational Method runoff coefficient (Figures 29-8A and 29-8B)

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Example 29-7.4

Given:
L = 150 ft
S = 0.02
Surface: grass

Find: Overland flow time, t_O, using Equation 29-7.6

Solution:

1. Determine C from Figure 29-8A.
   For lawn, heavy soil, 2% to 7% slope, use C = 0.18.

2. Determine t_O as follows:

$$t_O = \frac{(1.1 - 0.18)(150)^{0.5}}{2.63(0.02)^{0.33}} = 16 \text{ min}$$

* * * * * * * * * *

29-7.07 NRCS Upland Method

The Upland Method (NRCS, 1972) can be used to determine flow velocity to estimate time of concentration. This method relates watershed slope and surface to flow velocity. HYDRO includes the Grassed Waterway relationship to calculate channel travel time. TR-55 (1986) includes the relations for Grassed Waterway for an unpaved area and paved area to determine the travel time for shallow concentrated flow as follows:

Unpaved $V = 16.393\ S^{0.5}$ (Equation 29-7.7)

Paved $V = 20.653\ S^{0.5}$ (Equation 29-7.8)

Where:
V = average velocity, ft/s
S = slope of hydraulic grade line (watercourse slope), decimal

* * * * * * * * * *

Example 29-7.5

Given:
L = 500 ft
S = 0.025 (gutter slope)
Surface: concrete (paved)

Find: Gutter travel time, t_t (shallow concentrated flow)

Solution:

1. Determine V from Equation 29-7.8 as follows:

$$V = 20.653\ (0.025)^{0.5} = 3.27 \text{ ft/s}$$

2. Determine t_t from Equation 29-7.1 as follows:

$$t_t = \frac{500\ ft}{(3.27\ ft/s)(60\,\text{sec/min})} = 2.6 \text{ min}$$

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29-7.08 Triangular Gutter Flow

The travel time for gutter flow can be estimated using an average velocity of the flow. Equation 29-7.9 can be used to determine the velocity in a triangular gutter section given the watercourse slope, gutter cross slope, and water spread.

$$V = \frac{1.12}{n} S^{0.5} S_x^{0.67} T^{0.67} \qquad \text{(Equation 29-7.9)}$$

Where:

V	=	flow velocity in gutter, ft/s
n	=	Manning's roughness coefficient for sheet flow (Figure 29-7B)
S	=	longitudinal slope, decimal
S_x	=	gutter cross slope, decimal
T	=	water spread, ft

For a triangular channel with uniform inflow per length and zero flow at the upstream end, the average velocity will occur where the spread is 65% of the maximum. HYDRO includes this method as an option to determine travel time.

* * * * * * * * * *

Example 29-7.6

Given:
S = 0.025 (longitudinal slope)
S_x = 0.02 (cross slope)
T = 10 ft (design spread at inlet)
L = 500 ft (flow length)
n = 0.016 (concrete)

Find: Travel time of flow in gutter

Solution:

1. Use $T_{avg} = 2.17 T_{design}$ = (2.17)(3) = 6.51 ft

2. From Equation 29-7.9:

$$V = \frac{1.12}{0.016}(0.025)^{0.5}(0.02)^{0.67}(6.51)^{0.67} = 2.82 \text{ ft/s}$$

3. From Equation 29-7.1:

$$t_t = \frac{500\,ft}{(2.82\,ft/s)(60\sec/\min)} = 2.9\min$$

* * * * * * * * * *

29-7.09 Mannings' Equation

In a watershed with storm drains or channels, the travel time must be added to the overland flow time to find the total time of concentration where appropriate. The velocity can be determined using Manning's equation as follows:

$$V = \frac{1.486}{n} R^{2/3} S^{1/2} \qquad \text{(Equation 29-7.10)}$$

Where: V = mean velocity of flow, ft/s
n = Manning's roughness coefficient
R = hydraulic radius = Area/Wetted Perimeter (ft)
S = slope of the hydraulic grade line, decimal

29-7.09(01) Pipe Flow

For ordinary conditions, a storm drain should be sized assuming that it will flow full or almost full for the design discharge. For non-pressure flow, the velocity can be determined using Manning's equation. For a circular pipe flowing full, the equation becomes the following:

$$V = \frac{0.593}{n} D^{2/3} S^{1/2} \qquad \text{(Equation 29-7.11)}$$

Where: D = diameter of circular pipe, ft

Pipe flow charts can be used to determine the velocity for either full or partially-full flow conditions.

29-7.09(02) Open Channel

An open channel is assumed to begin where the surveyed cross-section information has been obtained, where the channel is visible on an aerial photograph, or where a blue line (indicating a stream) appears on a United States Geological Survey (USGS) quadrangle sheet. Manning's equation or the water-surface profile information can be used to estimate average flow velocity. Equation 29-7.10 can be used to determine the average flow velocity. It is determined for bank-full elevation.

29-7.10 Continuity Equation

If the pipes of a storm-drainage system will operate under pressure flow, the continuity equation should be used to determine velocity as follows:

$$V = Q/A \qquad \text{(Equation 29-7.12)}$$

Where: V = mean velocity of flow, ft/s
Q = discharge in pipe, ft^3/s
A = area of pipe, ft^2

29-7.11 Reservoir or Lake

It may be necessary to compute t_C for a watershed having a relatively large body of water within its flow path. Therefore, t_C is computed to the upstream end of the lake or reservoir. For the body of water, the travel time is computed using the following equation (King, 1967).

$$V_w = (gD_m)^{0.5} \qquad \text{(Equation 29-7.13)}$$

Where: V_w = the wave velocity across the water, ft/s
g = the acceleration due to gravity, or 32.2 ft/s^2
D_m = mean depth of lake or reservoir, ft

V_w will be 8.3 ft/s to 30 ft/s. Equation 29-7.13 only estimates travel time across the lake. It does not account for the travel time involved with the passage of the inflow hydrograph through spillway storage and the reservoir or lake outlet. This time is much longer and is added to the travel time across the lake. The travel time through lake storage and its outlet can be determined by the storage routing procedures described in Chapter Thirty-five.

Equation 29-7.13 can be used for a swamp with considerable open water, but where the vegetation or debris is relatively thick (less than about 25 percent open water) Manning's equation is more appropriate.

29-7.12 Kerby's Equation

The time of concentration for overland flow using Kerby's Equation is calculated as follows:

$$t_O = K\left(LNS^{-0.5}\right)^{0.467} \qquad \text{(Equation 29-7.14)}$$

Where:
- t_O = time of overland flow, min
- K = 0.67
- L = length of flow, ft
- N = retardance roughness coefficient (Figure 29-7C)
- S = average slope of overland flow, decimal

The length used in the equation, *L*, is the straight-line distance from the most-distant point of the watershed to the outlet, measured parallel to the slope of the land until a well-defined channel is reached. A watershed of less than 10 acres was used to calibrate the model. The slope was less than 1%, the *N* value was 0.8 or less, and surface flow dominated.

* * * * * * * * * *

Example 29-7.7

Given:
- L = 666.7 ft
- S = 0.5%
- Grass cover
- N = 0.40 from Figure 29-7C

Find: Overland t_O using Kerby's Equation.

Solution: Using Equation 29-7.14:

$$t_O = 0.67\left[(666.7)(0.40)(0.005)^{-0.5}\right]^{0.467} = 38.8 \text{ min}$$

* * * * * * * * * *

29-7.13 Kirpich's Equation

Kirpich's Equation is an empirical watershed equation based on data which account for length, slope, and soil cover. It derives from work done to determine the rate of runoff from a small agricultural watershed. The Equation is considered applicable to a watershed from 3 ac to 200 ac.

Kirpich's Equation is expressed as follows:

$$t_C = 0.0078\left(L^{0.77}\right)H^{-0.385} \qquad \text{(Equation 29-7.15)}$$

Where:
- t_C = time of concentration, h
- L = length of the longest waterway from the point in question to the basin divide, ft
- H = difference in elevation between the point in question and the basin divide (omitting drops due to gully overfills, waterfalls, etc.), ft

Kirpich's Equation works well for a natural, rural basin with well-defined channels, for overland flow on bare earth, or for a mowed earth roadside channel. Using the Equation, a paved basin and a forested one will have identical times of concentration if the lengths and reliefs are the same. This cannot occur; therefore the Equation should be adjusted if it is used elsewhere using the following guidelines:

For overland flow on a grass surface, multiply t_C by 2.0.
For overland flow on a concrete or asphalt surface, multiply t_C by 0.4.
For flow in a concrete-lined channel, multiply t_C by 0.2.

The application of Kirpich's Equation to a basin is as follows:

1 Compute the length, L, in feet between the basin divide and the point in question.

2. Compute the relief, H, in feet between the basin divide and the point in question. The elevation of the basin divide should represent an average of the elevations in the immediate vicinity of the termination point of the longest watercourse. This procedure avoids bias in the t_C computation due to an isolated peak in the headwater area. The elevation of the site should be interpolated between successive contours crossing the stream.

Compute t_C in hours using Equation 29-7.15.

Apply an adjustment factor, if applicable, based on surface type.

The t_C produced by Step 4 is appropriate for an urban area or a steep area. For the use of Kirpich's Equation, steep is defined as an overall basin slope of greater than 0.6% to 0.7%. For other than an urban area or a steep area, the t_C produced by Step 4 should be divided by 0.6. Because a basin may not clearly be rural or urban, or flat or steep, divide Kirpich's Equation by 0.8.

* * * * * * * * * *

Example 29-7.8

Given: L = 2666.7 ft
H = 33 ft
Surface: grass

Find: Time of concentration, t_C, using Kirpich's Equation, 29-7.15.

Solution:

1. Using Equation 29-7.15:

$$t_C = 0.0078\left(2666.7^{0.77}\right)\left(0.0125\right)^{-0.385} = 18.3\,\text{min}$$

2. For overland flow on a grass surface, multiply t_C by 2.0 as follows:

t_C = 2 (18.3) = 37 min

3. The basin slope = 33.3 / 2666.7 = 1.25%. Therefore, this is defined as steep for the use of Kirpich's Equation. No other adjustments are necessary. Therefore t_C = 37 min.

t_C = 2 (0.31) = 0.62 h = 37 min

29-8.0 RATIONAL METHOD

29-8.01 Introduction

The Rational Method is used to calculate the peak flow from a small drainage area. It is recommended for estimating the design-storm peak runoff for a rural area of up to 200 ac or an urban area of up to 100 ac.

29-8.02 Application

The precautions to be considered in applying the Rational Method are as follows.

1. The first step in applying the Rational Method is to obtain a topographic map and to define the boundaries of the drainage area under study. A field inspection of the area should also be made to determine if the natural drainage divides have been altered.

2. Restrictions to the natural flow such as highway crossings or dams that exist in the drainage area should be investigated to determine how they affect the design flows.

3. The charts, graphs, and tables included herein are not intended to replace reasonable and prudent engineering judgment which should permeate each step in the design process.

29-8.03 Characteristics

The Rational-Method formula applies a developed area with a significant amount of pavement, gutters, or storm sewers. The assumptions within the Rational Method include the following.

1. Basin Size. The rate of runoff resulting from rainfall intensity is a maximum if the rainfall intensity lasts as long as or longer than the time of concentration. That is, the entire drainage area does not contribute to the peak discharge until the time of concentration has elapsed.

   This assumption limits the size of the drainage basin that can be evaluated by the Rational Method. For a large drainage area, the time of concentration can be so large that constant rainfall intensity for such a long period does not occur, and a shorter, more-intense rainfall can produce a larger peak flow.

2. Frequency of Peak Discharge. The frequency of peak discharges is the same as that for the rainfall intensity for the given time of concentration.

   The frequency of peak discharge depends on rainfall frequency, antecedent moisture conditions in the watershed, and the response characteristics of the drainage system. For a small and largely impervious area, rainfall frequency is the dominant factor. For a larger drainage basin, the response characteristics control.

3. Runoff. The fraction of rainfall that becomes runoff, C, is independent of rainfall intensity or volume.

   The assumption is reasonable for an impervious area such as a street, rooftop, or parking lot. For a pervious area, the fraction of runoff varies with rainfall intensity and the accumulated volume of rainfall. The selected runoff coefficient must be appropriate for the storm, soil, and land-use conditions.

4. Peak Rate. The peak rate of runoff is sufficient information for design.

29-8.04 Equation

The Rational-Method formula estimates the peak rate of runoff at a location in a watershed as a function of the drainage area, runoff coefficient, and mean rainfall intensity for duration equal to the time of concentration. Because the result of using the Rational-Method formula to estimate peak discharge is sensitive to the parameters used, the designer must use engineering judgment in estimating values that are used in the Method. The formula is expressed as follows:

$$Q = CIA \quad \text{(Equation 29-8.1)}$$

Where: Q = maximum rate of runoff, ft^3/s

C = runoff coefficient representing a ratio of runoff to rainfall

I = average rainfall intensity for a duration equal to the time of concentration for a selected return period, in./h

A = drainage area tributary to the design location, acres.

Due to assumptions made in the formula, C can vary depending on the design storm. The value for C can be expressed as follows:

$$C = kC_1$$

Where: k = factor to adjust formula. It should be taken as follows:

Design Storm	k
2- to 10-year	1.0
25-year	1.1
50-year	1.2
100-year	1.25

C_1 = runoff coefficient representing a ratio of runoff to rainfall.

29-8.05 Time of Concentration

The time of concentration is the time required for water to flow from the hydraulically most-remote point of the drainage area to the point under investigation. Use of the Rational Formula requires the time of concentration, t_C, for each design point within the drainage basin to determine the rainfall intensity. Section 29-7.0 provides the methods for computing time of concentration.

29-8.05(01) Storm-Drainage System

For a storm-drainage system, the designer is interested in two different times of concentration, one for inlet spacing and one for pipe sizing. There is a difference between the two times as discussed in the following.

1. Inlet Spacing. The time of concentration, t_C, for inlet spacing is the time for water to flow from the hydraulically most-distant point of the drainage area to the inlet, which is known as the inlet time. This is the sum of the time required for water to move across the pavement or overland back of the curb to the gutter, plus the time required for flow to move through the length of the gutter to the inlet. For pavement drainage, if the total time of concentration to the upstream inlet is less than 5 min, a minimum t_C of 5 min should be used to estimate the intensity of rainfall. The time of concentration for the second downstream inlet and each succeeding inlet should be determined independently, the same as the first inlet. Travel time between inlets is not considered.

2. Pipe Sizing. The time of concentration for a point on a storm drain is the inlet time for the inlet at the upper end of the line plus the time of flow through the storm drain from the upper end of the storm drain to the point in question. If there is more than one source of runoff to a given point in a storm-drainage system, the longest t_C is used to estimate the rainfall intensity, I. There can be an exception to this, for example, where there is a large inflow area at some point along the system, the t_C for that area may produce a larger discharge than the t_C for the summed area with the longer t_C. The designer should be aware of this possibility if joining drainage areas and determining which drainage area governs.

29-8.05(02) Common Errors

Two common errors should be avoided when calculating t_c. First, runoff from a portion of the drainage area which is highly impervious may result in a greater peak discharge than can occur if the entire area is considered. An adjustment can be made to the drainage area by disregarding those areas where flow time is too slow to add to the peak discharge. It may be necessary to estimate several different times of concentration to determine the design flow that is critical for a specific application.

Second, the overland flow path is not necessarily perpendicular to the contours shown on available mapping. The land may be graded and swales will intercept the natural contour and conduct the water to the streets which reduces the time of concentration. The overland flow path should not exceed 200 ft in an urban area or 300 ft in a rural area.

29-8.06 Runoff Coefficient

The runoff coefficient, *C*, requires engineering judgment and an understanding by the designer. A typical coefficient represents the integrated effects of many drainage basin parameters. The selected value must be appropriate for the storm, soil, and land-use conditions.

Two sets of runoff coefficients for various types of surfaces are shown in Figures 29-8A and 29-8B. The designer may select a runoff coefficient from either set as deemed appropriate for the specific site application. The total *CA* value should be based on a ratio of the drainage areas associated with each *C* value as follows:

$$\text{Total } CA = A_1C_1 + A_2C_2 + A_3C_3 \ldots \qquad \text{(Equation 29-8.2)}$$

The coefficients provided in Figures 29-8A and 29-8B are applicable to a storm of five- to ten-year frequency. A less frequent, higher intensity storm will require a higher coefficient because infiltration and other losses have a proportionately smaller effect on runoff (Wright, McLaughlin, 1969).

As the slope of the drainage basin increases, the selected *C* value should also increase. This is because, as the slope of the drainage area increases, the velocity of overland and channel flow will increase allowing less opportunity for water to infiltrate the ground surface. Thus, more of the rainfall will become runoff from the drainage area.

Figure 29-8A, Runoff Coefficient for the Rational Formula, provides an example for the calculation of a weighted runoff coefficient.

29-8.07 Rainfall Intensity

The rainfall intensity, *I*, is the average rainfall rate in inches per hour for a duration equal to the time of concentration for a selected return period. Once a return period has been selected for design and a time of concentration calculated for the drainage area, the rainfall intensity can be determined from the rainfall Intensity-Duration-Frequency (IDF) curves.

IDF curves are located at the NOAA website, http://hdsc.nws.noaa.gov/hdsc/pfds/orb/in_pfds.html.

29-8.08 Rational-Method Example Problem

The following example problem illustrates the application of the Rational Method to estimate the peak discharge. The peak runoff is needed at the storm-sewer catch basin for a 10-yr return period.

Step 1: Determine site data.

The following data were measured from a topographic map and field survey:

Residential area (single family) = 1.725 ac
Pavement area (concrete) = 0.3 ac

Length of overland flow = 150 ft Average overland slope = 2.0%
Length of concrete gutter = 500 ft Slope of gutter = 0.025

Step 2: Choose runoff coefficient from Figure 29-8A and find total CA.

Land Use	Area	Runoff Coefficient	CA
Residential (single family)	1.725	0.40	0.69
Concrete Pavement	0.3	0.90	0.27
	Total Weighted CA =		0.96

Step 3: Calculate the total time of concentration to the inlet.

Overland flow time: t_O calculated using the kinematic wave equation in Example 29-7.2 to be 14 min

Channel flow time: t_t calculated using the triangular gutter method in Example 29-7.6 to be 3 min

Total Time of Concentration: $t_C = t_O + t_t = 14 + 3 = 17$ min

Step 4: Find Rainfall Intensity, *I*, from NOAA website, http://hdsc.nws.noaa.gov/hdsc/pfds/orb/in_pfds.html. Click on location on map corresponding to the peak discharge. The information is shown in English measurement units only. For this example,

Indianapolis
10-year return period
Duration = t_C = 17 min.

Interpolation:
Solve for *x*,

$$\frac{(30-17)}{(1.62-x)} = \frac{(17-15)}{(x-1.16)}$$

x = 1.22 in. for 17 min, which converts to 4.31 in./h

I_{10} = 4.31 in./h from NOAA website

Step 5: Compute Peak Runoff.

$Q = CIA = (0.96)(4.31) = 4.14$ ft^3/s

29-9.0 USGS REGRESSION EQUATIONS

The USGS procedure cannot be used for the final design.

29-9.01 Introduction

This Section provides equations for estimating the magnitude and frequency of a flood at an ungaged site on a regulated rural stream. They are based on the USGS publication *Techniques for Estimating Magnitude and Frequency of Floods on Streams in Indiana* (Water-Resources Investigations Report 84-4134). The equations were developed by multiple-regression analysis of basin characteristics and peak-flow statistical data from 242 gaged locations in Indiana, Ohio and Illinois. The State was divided into seven areas on the basis of the regression analysis. A set of equations for estimating peak discharge with recurrence intervals of 2, 10, 25, 50, and 100 years was developed for each area. Significant basin characteristics in the equations are drainage area, channel length, channel slope, mean annual precipitation, storage, precipitation intensity, and a runoff coefficient. Standard errors of estimate for the equations range from 24 percent to 45 percent.

This Section also provides methods for estimating flood magnitude and frequency at a site on a gaged stream.

29-9.02 Hydrologic Regions

Regression analyses use stream-gage data to define hydrologic regions. These are geographic regions which have very similar flood frequency relationships, and as such commonly display similar characteristics. Because of the distance between stream gages, the regional boundaries cannot be considered as precise. Figure 29-9A, Areas for Selecting Flood-Frequency USGS Estimating Equations, shows the hydrologic regional boundaries.

Problems related to hydrologic boundaries may occur in selecting the appropriate regression equation. The watershed of interest may lie partially within two or more hydrologic regions, or it may lie totally within a hydrologic region but close to a hydrologic region boundary. A field visit is recommended to first collect all available historical flood data and to compare the project watershed characteristics with those of the abutting hydrologic regions.

29-9.03 Basin Characteristics

The basin characteristics that are required for use of the USGS equations are defined as follows:

1. Design Discharge, Q_t, (cubic feet per second). The peak discharge for the specified design flood frequency, t.

2. Drainage Area, *DA*, square miles. The area contributing directly to runoff at the study site. Draw an outline of the drainage areas on a topographic map and use a planimeter to determine the area.

3. Main-Channel Slope, *SL*, feet per mile. The slope of the streambed between points that are 10% and 85% of the distance from the location on the stream to the basin divide. Determine from topographic maps to nearest 0.1 ft/mi.

4. Channel Length, *L*, miles. The distance measured along the main channel from the location on the stream to the basin divide is determined from topographic maps to the nearest 0.1 mile.

5. Storage,*STOR*, percent. The percentage of drainage area covered by lakes, ponds, or wetlands.

6. Mean Annual Precipitation, *PREC*, inches. The 1941-70 average annual precipitation is determined from Figure 29-9B, Mean Annual Precipitation (1941-70), (Stewart, 1983). A constant of 30 in. is subtracted from the characteristic *PREC* for use in the estimating equations. Plot the basin centroid determined from Figure 29-9B and determine the mean annual precipitation for that point by interpolating between lines of equal precipitation.

7. Precipitation Intensity, $I_{24,2}$, inches. The maximum 24-h precipitation having a recurrence interval of 2 yr is determined from Figure 29-9C, Two-Year, 24-Hour Projection, (Hershfield, 1961).

8. Runoff Coefficient, *RC*. A coefficient that relates storm runoff to soil permeability by means of major hydrologic soil groups is determined from Figure 29-9D, Major Hydrologic Soil Groups, (Davis, 1975). Values range from 0.3 for hydrologic soil group A to 1.0 for hydrologic soil group E.

29-9.04 Regression Equations for Ungaged Site

Figure 29-9E, Production Equations, Standard Errors of the Estimate, *SEE*, and Equivalent Years of Record, *EY*, provides the equations for an ungaged site for each of the seven geographic areas (see Figure 29-9A). Figure 29-9F, Ranges of Area Basin Characteristics for USGS Regression Equations, provides the ranges for application of each basin characteristic to each of the geographic areas.

29-9.05 Procedure

Follow this procedure for the USGS Method.

Step 1: From Figure 29-9A, locate the area for the site.

Step 2: From *Techniques for Estimating Magnitude and Frequency of Floods on Streams in Indiana*, USGS Report 84-4134, Figure 1 and Table 4, determine if the study site is at a gaged site or on a gaged stream.

Step 3: If the site is on a gaged stream, go to Step 6.

Step 4: Determine the basin characteristics necessary to solve the regression equation from Figure 29-9E (Prediction Equations).

Step 5: Use the appropriate equation from Figure 29-9E to solve for the required discharge.

Step 6: If the site is at a gaged location, the weighted estimate of Q_t from the USGS Report Table 4should be used.

Step 7: If the drainage area of an ungaged site on a gaged stream is less than 50% or greater than 150% of the drainage area of a gaged site on the same stream, the discharge should be estimated from the appropriate equation in Figure 29-9E as if the site were on an ungaged stream. Go to Step 4.

Step 8: If the drainage area of an ungaged site on a gaged stream is between 50% and 150% of the drainage area of a gaged site on the same stream, the discharge should be an estimate calculated from both gaged data (USGS Report Table 4) and estimating equations (Figure 29-9E). An estimate of the process is as follows:

a. Compute the ratio as follows:

$$R = \frac{Q_{TW}}{Q_{TR}}$$

Where: Q_{TW} = weighted estimate of T-year flood at gaged site
Q_{TR} = regression equation estimate of T-year flood at gaged site
Both can be obtained from USGS Report, Table 4

b. Compute weighting factor as follows:

$$R_W = R - \left(\frac{2\Delta A}{A_G}\right)(R-1)$$

Where: R = ratio defined in Step 8.a.
ΔA = absolute value of the difference between the drainage areas (DA) of the gaged and ungaged sites

$A_G = DA$ of gaged site

c. Compute T-year peak discharge at the ungaged site as follows:

$$Q_T = Q_{TR} R_W$$

Where: Q_{TR} = regression equation estimate of T-year flood at ungaged site

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Example 29-9.1: Ungaged Stream

Given: Location: Brown County
$DA = 6.94$ mi^2 (ungaged site)
$L = 4.40$ mi
Elevation of channel at 10% of length (0.4 mi) = 652′
Elevation of channel at 85% of length (3.7 mi) = 824′
Distance between points = 3.7 - 0.4 = 3.3 mi
Channel Slope, $SL = \frac{824 - 652}{3.3} = 52.1\ ft/mi$
$I_{24,2}$ = 3.05 in., from Figure 29-9C

Find: The 100-year discharge.

Solution:

Step 1. Determine area where site is located. From Figure 29-9A, Brown County is located in Area 3 for an ungaged stream.

Step 2. From Figure 29-9E, the regression equation for Q_{100} in Area 3 is as follows:

$$Q_{100} = 181\left(DA^{0.779}\right)\left(SL^{0.466}\right)\left(I_{24,2} - 2.5\right)^{0.831}$$

Step 3. Substitute the values of basin characteristics as follows:

$$Q_{100} = 181(6.94)^{0.779}(52.1)^{0.466}(3.05 - 2.5)^{0.831} = 3140\ cfs$$

Example 29-9.2 (Gaged Stream)

Given: Gaging Station 03366500 on the Muscatatuck River near Deputy;
Ungaged site on Muscatatuck River downstream from gaging station.
Basin characteristics (ungaged site) are as follows:

DA_U = 359 mi^2
SL = 6.2 ft/mi
L = 68.8 mi
$I_{24,2}$ = 3.00 in.

Find: Q_{100} at gaging station
Q_{100} at ungaged site downstream

Solution:

Step 1: From USGS Report, Table 4, three values are given for Q_{100} as follows:

a. 40,900 ft^3/s, from flood frequency analysis of observed station data
b. 44,600 ft^3/s, from regression equation
c. 41,200 ft^3/s, from weighting the station and area estimates

Select the weighted value as the best estimate. Therefore, Q_{100} = 41,200 ft^3/s at the gaging site.

Step 2: From USGS Report, Figure 1, and Figure 29-9A, the gaging station is located in Area 4. The regression equation for Q_{100} in Area 4 is as follows:

$$Q_{100} = 32\left(DA^{0.565}\right)\left(SL^{0.705}\right)\left(L^{0.730}\right)\left(I_{24,2} - 2.5\right)^{0.464}$$

Step 3: Substitute the values of basin characteristics as follows:

$$Q_{100} = 32(359)^{0.565}(6.2)^{0.705}(68.8)^{0.730}(3.00 - 2.5)^{0.464} = 51{,}200 \text{ ft}^3/\text{s}$$

Step 4: Compute the ratio of the ungaged drainage area to the gaged drainage area as:

$$\frac{DA_U}{DA_G} = \frac{359}{293} = 1.22$$

1.22 is greater than 0.5 and less than 1.5, therefore the ratio is acceptable.

Step 5: Compute the gaged-site discharge ratio as follows:

$$R = \frac{Q_{TW}}{Q_{TR}} = \frac{41{,}200}{44{,}600} = 0.924$$

Step 6: Compute the weighting factor as follows:

$$R_W = R - \frac{2\Delta A}{A_G}(R-1)$$

$$R_W = 0.924 - \left[\frac{2(359-293)}{293}\right](0.924-1) = 0.958$$

Step 7: Reduce regression value by weighting factor as follows:

$$Q_T = (51{,}200)(0.958) = 49{,}000 \text{ ft}^3/\text{s}$$

29-10.0 NRCS UNIT HYDROGRAPH

29-10.01 Introduction

Techniques developed by the U.S. Natural Resources Conservation Service (formerly the Soil Conservation Service) for calculating rate of runoff require the same data as for the Rational Method: drainage area, a runoff factor, time of concentration, and rainfall intensity. The NRCS approach, however, is more sophisticated because it also considers the time distribution of the rainfall, the initial rainfall losses to interception and depression storage, and an infiltration rate that decreases during the course of a storm. In the NRCS method, the direct runoff can be calculated for a given storm, either real or fabricated, by subtracting infiltration and other losses from the rainfall amount to obtain the precipitation excess. Details of the methodology can be found in the *NRCS National Engineering Handbook*, Section 4.

29-10.02 Application

A unit hydrograph and a dimensionless-unit hydrograph are used in the NRCS procedure. A unit hydrograph represents the time distribution of flow resulting from 1 in. of direct runoff occurring over the watershed in a specified time. A dimensionless-unit hydrograph represents the composite of many unit hydrographs. The dimensionless-unit hydrograph is plotted in nondimensional units of time versus time to peak and discharge at given time versus peak discharge.

Characteristics of the dimensionless hydrograph vary with the size, shape, and slope of the tributary drainage area. The most significant characteristics affecting the dimensionless hydrograph shape are the basin lag and the peak discharge for a given rainfall. Basin lag is the time from the center of mass of rainfall excess to the hydrograph peak. Steep slopes, a compact shape, and an efficient drainage network tend to make lag time short and peaks high. Flat slopes, an elongated shape, and an inefficient drainage network tend to make lag time long and peaks low.

29-10.03 Equations and Concepts

The following discussion outlines the equations and basic concepts utilized in the NRCS method.

1. Drainage Area. The drainage area of a watershed is determined from topographic maps and field surveys. For a large drainage area, it may be necessary to divide the area into subdrainage areas to account for major land-use changes, obtain analysis results at different points within the drainage area, or locate stormwater drainage facilities and assess their effects on the flood flows. A field inspection of existing or proposed drainage systems should be made to determine if the natural drainage divides have been altered. These alterations could make significant changes in the size and slope of the subdrainage areas.

2. Rainfall. See Figure 29-10A, Huff Distribution of Design Rainfall (50% Probability of Design Rainfall). Quartile II should be used. The rainfall intensity for the given duration and return period should be multiplied by the duration to determine rainfall depth.

3. Rainfall-Runoff Equation. A relationship between accumulated rainfall and accumulated runoff has been derived by NRCS from experimental plots for numerous soils and vegetative-cover conditions. Data for land-treatment measures, such as contouring and terracing, from experimental watersheds have been included. Equation 29-10.1 was developed for a small watershed for which only daily rainfall and watershed data are ordinarily available. It was developed from recorded storm data that included total amount of rainfall in a calendar day but not its distribution with respect to time. The NRCS runoff equation is therefore a method of estimating direct runoff from 24-h or 1-day storm rainfall. The equation is as follows:

$$Q = \frac{(P - I_a)^2}{(P - I_a) + S} \qquad \text{(Equation 29-10.1)}$$

Where: Q = accumulated direct runoff, in.

P = accumulated rainfall (potential maximum runoff), in.

I_a = initial abstraction including surface storage, interception, and infiltration prior to runoff, in.

S = potential maximum retention, in.

The relationship between I_a and S was developed from experimental watershed data. It removes the necessity for estimating I_a for common usage. The empirical relationship used in the NRCS runoff equation is as follows:

$$I_a = 0.2S \qquad \text{(Equation 29-10.2)}$$

Substituting 0.2S for I_a in Equation 29-10.1, the NRCS rainfall-runoff equation becomes the following:

$$Q = \frac{(P - 0.2S)^2}{(P + 0.8S)} \qquad \text{(Equation 29-10.3)}$$

Figure 29-10B, NCRS Relation Between Direct Runoff, Curve Number and Precipitation, shows a graphical solution of Equation 29-10.3 which enables the precipitation excess from a storm to be obtained if the total rainfall and watershed curve number are known.

29-10.04 Procedure

The following is a discussion of procedures that are used in the hydrograph method along with recommended tables and figures.

29-10.04(01) Runoff Factor

In a hydrograph application, runoff is referred to as rainfall excess or effective rainfall defined as the amount by which rainfall exceeds the capability of the land to infiltrate or otherwise retain the rainwater. The principal physical watershed characteristics affecting the relationship between rainfall and runoff are land use, land treatment, soil types, and land slope.

Land use is the watershed cover, and it includes both agricultural and nonagricultural uses. Items such as type of vegetation, water surfaces, roads, roofs, etc., are all part of the land use. Land treatment applies to agricultural land use, and it includes mechanical practices such as contouring or terracing, and management practices such as rotation of crops.

The NRCS uses a combination of soil conditions and land-use (ground cover) to assign a runoff factor to an area. The runoff factors, called runoff curve numbers, *CN*, indicate the runoff potential of an area when the soil is not frozen. The higher the *CN*, the higher the runoff potential.

Soil properties influence the relationship between rainfall and runoff by affecting the rate of infiltration. The NRCS has divided soils into four hydrologic soil groups, A, B, C, and D, based on infiltration rates. The groups were previously described for the Rational Method. The applicable NRCS soil classification maps may be obtained from the appropriate county agency.

The effect of urbanization on the natural hydrologic soil group should be considered. If heavy equipment can be expected to compact the soil during construction or if grading will mix the surface and subsurface soils, appropriate changes should be made in the soil group selected. Also, runoff curve numbers vary with the antecedent soil moisture conditions, defined as the amount of rainfall occurring in a selected period preceding a given storm. The greater the antecedent rainfall, the more direct runoff there is from a given storm. A 5-day period is used as the minimum for estimating antecedent moisture conditions. Antecedent soil moisture conditions also vary during a storm. Heavy rain falling on dry soil can change the soil moisture condition from dry to average to wet during the storm period.

The following figures provide a series of runoff factors. Figures 29-10C provides runoff curve numbers for an urban area, Figure 29-10D provides those for an undeveloped area, and Figure 29-10E provides those for agricultural land. The tables are based on the average antecedent moisture condition: soils are neither very wet nor very dry once the design storm begins. Curve numbers should be selected only after a field inspection of the watershed and a review of zoning and soil maps. Figure 29-10F provides conversion from average antecedent moisture conditions. Figure 29-10G provides rainfall groups for antecedent soil moisture conditions during growing and dormant seasons.

29-10.04(02) Time of Concentration

The average slope within the watershed together with the overall length and retardance of overland flow are the factors affecting the runoff rate through the watershed. In the NRCS method, time of concentration, t_C, is defined as the time required for water to travel from the hydraulically most-distant point in a watershed to its outlet. Lag, L, can be considered as a weighted time of concentration and is related to the physical properties of a watershed, such as area, length, and slope. The NRCS derived the following empirical relationship between lag and time of concentration.

$$L = 0.6t_C \qquad \text{(Equation 29-10.4)}$$

In an urban area of less than 2000 ac, a curve-number method can be used to estimate the time of concentration from watershed lag. In this method, the lag for the runoff from an increment of excess rainfall can be considered as the time between the center of mass of the excess rainfall increment and the peak of its incremental outflow hydrograph. The equation developed by NRCS to estimate lag is as follows:

$$L = \frac{\left([L_m]^{0.8}(S+1)^{0.7}\right)}{\left(1900Y^{0.5}\right)} \qquad \text{(Equation 29-10.5)}$$

Where:

L	=	lag, h
L_m	=	length of mainstream to farthest divide, ft
Y	=	average slope of watershed, %
S	=	$\left(\left[\frac{1000}{CN}\right]-10\right)$, in.
CN	=	NRCS curve number

The lag time can be corrected for the effects of urbanization by using Figure 29-10H, Factors for Adjusting Lag if Impervious Areas Occur in Watershed, and Figure 29-10 I, Factors for Adjusting Lag if Main Channel has been Hydraulically Improved. The amount of modification to the hydraulic flow length must be determined from topographic maps or aerial photographs following a field inspection of the area. The modification to the hydraulic flow length not only includes pipes or channels but also the length of flow in streets or drives.

After the lag time is adjusted for the effects of urbanization, Equation 29-10.4 can be used to calculate the time of concentration for use in the NRCS method. Section 29-7.0 provides an alternative procedure for travel time and time of concentration estimation.

29-10.04(03) Triangular Hydrograph Equation

The triangular hydrograph is a practical representation of excess runoff with only one rise, one peak, and one recession. Its geometric configuration can be described mathematically, which makes it useful in estimating discharge rate. The NRCS developed Equation 29-10.6 to estimate the peak rate of discharge for an increment of runoff, as follows:

$$Q_p = \frac{[0.208A(q)]}{\left(\frac{d}{2}+L\right)} \qquad \text{(Equation 29-10.6)}$$

Where: Q_p = peak rate of discharge, ft^3/s
A = area, mi^2
q = storm runoff during time interval, in
d = time interval, h
L = watershed lag, h

Equation 29-10.6 can be used to estimate the peak discharge for the unit hydrograph which can then be used to estimate the peak discharge and hydrograph from the entire watershed.

The constant 0.208, or peak-rate factor, is valid for the NRCS dimensionless-unit hydrograph. A change in the dimensionless-unit hydrograph reflecting a change in the percent of volume under the rising side will cause a corresponding change in the shape factor associated with the triangular hydrograph and therefore a change in the constant 0.208. The constant has been known to vary from about 0.258 in steep terrain to 0.129 in flat swampy terrain.

29-10.05 TR-20 Example Problem, NRCS Method

The Natural Resources Conservation Service has developed the computerized program TR-20 to perform the calculations for the NRCS hydrologic methodology. The program is in English units of measurement, therefore all input and output data must be in English units. To run TR-20, the user must input the data as follows:

1. desired design frequency;
2. runoff Curve Number;
3. average watershed slope;
4. stream cross-section data;
5. reach length;
6. structure data;
7. cumulative rainfall table including desired time increment;
8. antecedent moisture condition;
9. storm characteristics;

10. routing instructions; and
11. desired output.

Figure 29-10A provides a sample output from TR-20.

29-11.0 HEC 1

HEC 1 was developed by the U.S. Army Corps of Engineers in 1967. It is also a hydrograph-oriented program with the capability to compute, combine, and route hydrographs through a system of subareas. HEC 1 can be utilized without regard to basin size. Input requirements are similar to TR-20. A detailed description of HEC 1 is not provided here because of its similarities to TR-20.

29-12.0 REFERENCES

1. AASHTO, *Highway Drainage Guidelines*, Volume II.

2. Federal Highway Administration, HYDRAIN Documentation, 1990.

3. USGS, Water-Resources Investigations Report 84-4134, *Techniques For Estimating Magnitude and Frequency of Floods on Streams in Indiana*.

4. AASHTO, *Model Drainage Manual*, 1991 and 1997.

5. Indiana DNR, *Coordinated Discharges of Selected Streams in Indiana*.

6. Federal Highway Administration, Hydraulic Engineering Circular No. 19, *Hydrology*, 1994.

7. Water Resources Council, Bulletin 17B, *Guidelines for Determining Flood Flow Frequency*, 1981.

8. Highway Extension and Research Project for Indiana Counties and Cities, H-94-6, *Storm Water Drainage Manual*, July 1994.

9. Indiana Department of Natural Resources, Division of Water, *Hydrology and Hydraulics in Indiana, Volume 1*, January 1986.

Symbol	Definition	Unit
A	Drainage area	ac, mi^2
BDF	Basin development factor	%
C	Runoff coefficient	-
C_f	Frequency factor	-
CN	NRCS-runoff curve number	-
C_t, C_p	Physiographic coefficients	-
d	Time interval	h
DH	Difference in elevation	ft
I or i	Rainfall intensity	in./h
IA	Percentage of impervious area	%
I_a	Initial abstraction from total rainfall	in.
K	Frequency factor for a particular return period and skew	-
L	Lag	h
L_m	Length of mainstream to furthest divide	ft
L_{ca}	Length along main channel to a point opposite the watershed centroid	mi
M	Rank of a flood within a long record	-
n	Manning roughness coefficient	-
N	Number of years of flood record	years
P	Accumulated rainfall	in
Q	Rate of runoff	ft^3/s
q	Storm runoff during a time interval	in.
R	Hydraulic radius	ft
RC	Regression constant	-
RQ	Equivalent rural peak runoff rate	ft^3/s
S or Y	Ground slope	in./in., ft/mi or %
S	Potential maximum retention storage	in.
NRCS	Natural Resources Conservation Service	-
SL	Main channel slope	ft/ft
S_L	Standard deviation of the logarithms of the peak annual floods	-
ST	Basin storage factor	%
T_B	Time base of unit hydrograph	h
t_C	Time of concentration	min or h
T_L	Lag time	h
T_r	Snyder's duration of excess rainfall	h
UQ	Urban peak runoff rate	ft^3/s
V	Velocity	ft/s
X	Logarithm of the annual peak	–

HYDROLOGIC SYMBOLS AND DEFINITIONS

Figure 29-3A

Highway Classification	Drainage Appurtenance							
	Roadway Serviceability	Bridge Waterway Openings		Roadway Cross Culverts		Storm Drain Systems (2)		Open Channels (4)
		Allowable Backwater	Allowable Velocity	Allowable Backwater	Allowable Velocity	Design for Inlet Spacing And And Trunk Line (Gravity)	Check for HGL on Trunk Line (Pressure Flow)	
Freeways	100	100	100	100	50	50	N/A	10
Multilane Non-Freeways	100	100	100	100	50	10	50	10
Two-Lane Facilities (3) AADT ≥ 3000	100	100	100	100	50	10	50	10
3000 > AADT ≥ 1000	25	100	100	100	25	10	50	10
AADT < 1000	10	100	100	100	10	10	50	10
Driveways (1)	10	N/A	N/A	100	10	N/A	N/A	N/A
Bridge Decks (Non-Freeways)	N/A	N/A	N/A	N/A	N/A	10	N/A	N/A
Ramps	100	100	100	100	50	10	50	10

(1) An overtopping area should be provided at the driveway entrance to accommodate floods larger than the design flood. The designer should check the capacity of the driveway culvert for the design frequency indicated for the roadway cross culvert to ensure that there will be no overtopping of the roadway. The design storm for allowable backwater for driveway pipes will match the allowable backwater for the facility the driveway is connected to.

(2) *Inlet spacing for pavement drainage is based on both the recurrence interval and the allowable spread of water in the gutter. See Figure 36-7A for the criteria for allowable spread.*

(3) Traffic volumes are for a 20-year projection.

(4) Side ditches only. Relocated streams parallel to the road will be designed for a 100-year event.

General Note: Where existing drainage appurtenances can accommodate less frequent recurrence intervals, no reduction in hydraulic capacity nor serviceability is allowable.

DESIGN FREQUENCY

(Return Period - Years)

Figure 29-5A

Facility Description(5)	Methodology				
	IDNR Coordinated Curves	TR-20	HEC I	USGS Regression Equations	Rational Method (6)
Stream Flow, Channel, Bridge, Large Culvert	(1)	(2)	(2)	(4)	
Small Culvert		(1)	(1)	(4)	(2)
Storm Drain, Roadside Ditch, Roadside Culvert, Inlet Spacing		(2)	(2)		(1)
Pumping Station		(3)	(3)		(2)
Detention Basin		(3)	(3)		(4)

Note: TR-55 methodology is acceptable for time of concentration only.

(1) *Preferred method.*

(2) *Alternative if (1) is not applicable.*

(3) *Preferred for complex facility or if hydrograph is required.*

(4) *Method may be used for preliminary evaluation.*

(5) *The IDNR Letter of Discharge is required for a project that requires an IDNR Permit.*

(6) *The Rational Method can only be used for a drainage area of less than 100 ac in an urban area or less than 200 ac in a rural area.*

SELECTION OF DISCHARGE COMPUTATION METHOD

Figure 29-6A

Method	Comments
NRCS Curve Number	Overland and channelized flow time for rural area (2 to 200 ac). Included in HYDRO.
Kinematic Wave	Overland flow time. Requires iterative solution and is included in HYDRO and HEC 1.
Manning's Kinematic Solution	Overland flow time. The maximum flow length is 300 ft. Included in TR-55.
Federal Aviation Administration	Overland flow time; developed for airport.
NRCS Upland Method	Flow velocity for overland and shallow concentrated flow. Included in TR-55 for shallow, concentrated flow. Grassy waterway method included in HYDRO.
Triangular Gutter Flow	Flow velocity in gutter. Included in HYDRO.
Manning's Equation	Flow velocity for non-pressure flow in pipe or open channel.
Continuity Equation	Flow velocity for pressure flow in pipe.
Reservoir and Lake	Wave velocity across water; does not account for travel time of hydrograph through storage.
Kerby	Overland flow time. Watershed < 10 ac, slopes < 1%, and N value ≤ 0.8 were used to calibrate model.
Kirpich	Good estimate of overall flow time for steep, wooded watershed. Can be applied to flatter basin by dividing result by 0.6.

METHODS FOR CALCULATING TIME OF CONCENTRATION

Figure 29-7A

Surface Description		*n*[1]
Smooth Surface (concrete, asphalt, gravel, bare soil)		0.011
Fallow (no residue)		0.05
Cultivated Soil	Residue cover ≤ 20%	0.06
	Residue cover > 20%	0.17
Grass	Short-grass prairie	0.15
	Dense grass[2]	0.24
	Bermuda grass	0.41
Range (natural)		0.13
Woods[3]	Light underbrush	0.40
	Dense underbrush	0.80

1 *The* n *value is a composite of information complied by Engman (1986).*

2 *Includes species such as weeping lovegrass, bluegrass, buffalo grass, blue grama grass, or native grass mixtures.*

3 *In selecting* n*, consider cover to a height of about 12 in. This is the only part of the plant cover that will obstruct sheet flow.*

ROUGHNESS COEFFICIENT (MANNING'S *n*) FOR SHEET FLOW

Figure 29-7B

Type of Surface	*n*
Smooth, impervious surface	0.02
Smooth, bare, packed soil	0.10
Poor grass, cultivated row crops, or moderately rough, bare surface	0.20
Deciduous timberland	0.60
Pasture or overage grass	0.40
Conifer timberland, deciduous timberland with deep forest litter, or dense grass	0.80

VALUES OF *n* FOR KERBY'S FORMULA

Figure 29-7C

TYPE OF SURFACE	RUNOFF COEFFICIENT

<u>Rural Area</u>

Concrete or sheet asphalt pavement	0.8 - 0.9
Asphalt macadam pavement	0.6 - 0.8
Gravel roadway or shoulder	0.4 - 0.6
Bare earth	0.2 - 0.9
Steep grassed area (2:1 slope)	0.5 - 0.7
Turf meadow	0.1 - 0.4
Forested area	0.1 - 0.3
Cultivated field	0.2 - 0.4

<u>Urban Area</u>

Watertight roof surface	0.75 - 0.95
Asphalt or concrete pavement	0.80 - 0.95
Traffic-bound pavement	0.70 - 0.90
Gravel roadway	0.35 - 0.70
Impervious soil, heavy	0.40 - 0.65
Impervious soil, with turf	0.30 - 0.55
Slightly pervious soil	0.15 - 0.40
Slightly pervious soil, with turf	0.10 - 0.30
Moderately pervious soil	0.05 - 0.20
Moderately pervious soil, with turf	0.00 - 0.10

<u>Example of Weighted *C* Factor</u>

5% watertight roof surface	5% x 0.85 = 0.04
10% asphalt or concrete pavement	10% x 0.9 = 0.09
10% traffic-bound pavement	10% x 0.8 = 0.08
50% slightly impervious soil	50% x 0.4 = 0.20
15% slightly impervious soil, with turf	15% x 0.2 = 0.03
10% moderately pervious soil	10% x 0.1 = <u>0.01</u>

WEIGHTED *C* FACTOR = 0.45

RUNOFF COEFFICIENT FOR THE RATIONAL FORMULA

Figure 29-8A

URBAN AREA

Type of Drainage Area			Runoff Coefficient
Business	Downtown		0.70 - 0.95
	Neighborhood		0.50 - 0.70
Residential	Single-family unit		0.30 - 0.50
	Multi-family unit	Detached	0.40 - 0.60
		Attached	0.60 - 0.75
	Suburban		0.25 - 0.40
	Apartment dwelling		0.50 - 0.70
Industrial	Light		0.50 - 0.80
	Heavy		0.60 - 0.90
Park or Cemetery			0.10 - 0.25
Playground			0.20 - 0.35
Railroad yard			0.20 - 0.40
Unimproved			0.10 - 0.30
Lawn	Sandy soil	flat, 2%	0.05 - 0.10
		average, 2 - 7%	0.10 - 0.15
		steep, 7%	0.15 - 0.20
	Heavy soil	flat, 2%	0.13 - 0.17
		average, 2 - 7%	0.18 - 0.22
		steep, 7%	0.25 - 0.35
Street	Asphalt		0.70 - 0.95
	Concrete		0.80 - 0.95
	Brick		0.70 - 0.85
Drive or walk			0.75 - 0.85
Roof			0.75 - 0.95

RUNOFF COEFFICIENTS FOR THE RATIONAL FORMULA

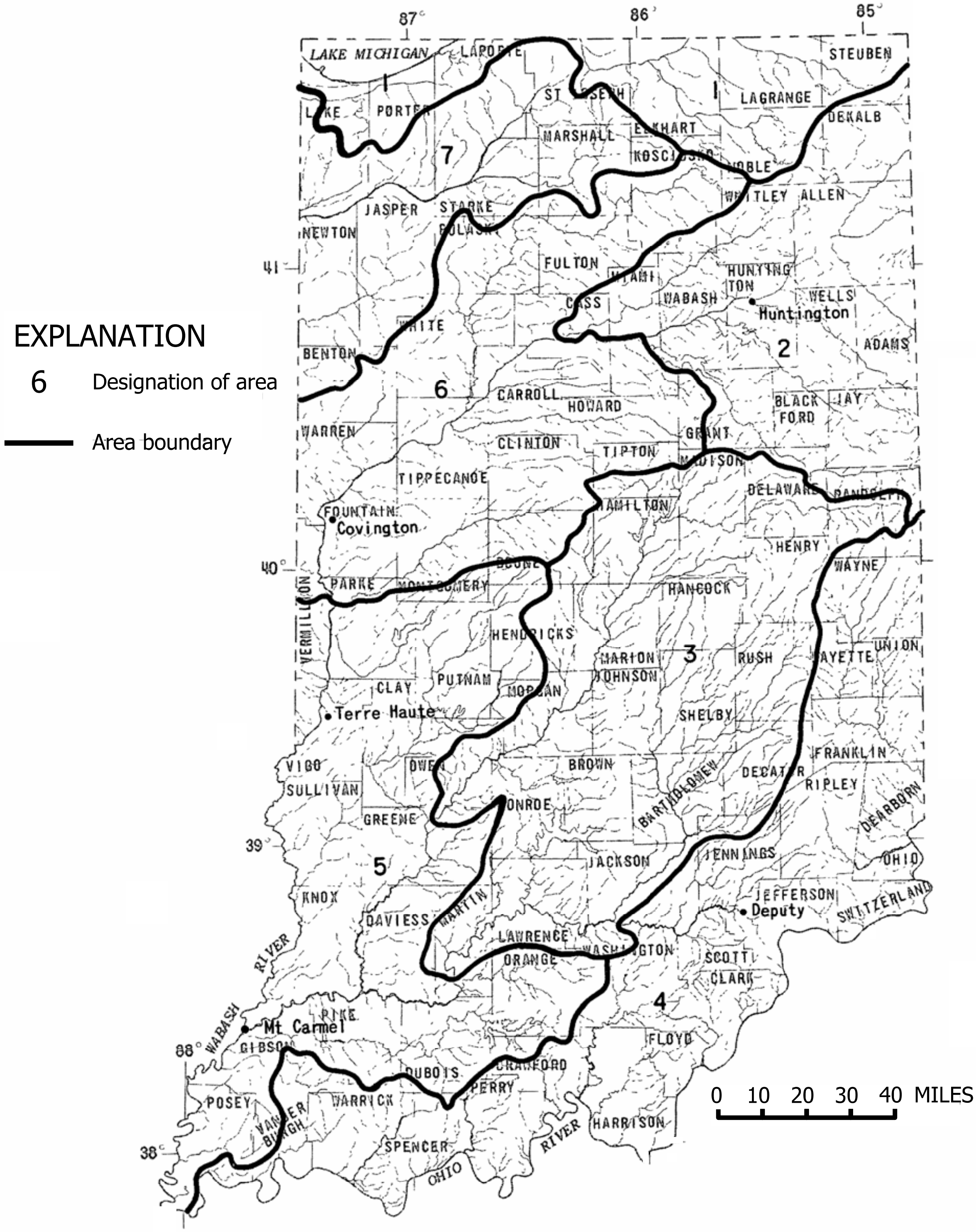
Figure 29-8B

RURAL AREA

Soil Type	Watershed Cover		
	Cultivated	Pasture	Woodlands
With above-average infiltration rate; usually sandy or gravelly.	0.20	0.15	0.10
With average infiltration rate; no clay pans, loams, or similar soils.	0.40	0.35	0.30
With below-average infiltration rate; and heavy clay soils or soils with a clay pan near the surface, and shallow soils above impervious rock	0.50	0.45	0.40

RUNOFF COEFFICIENTS FOR THE RATIONAL FORMULA

Figure 29-8B (contd.)

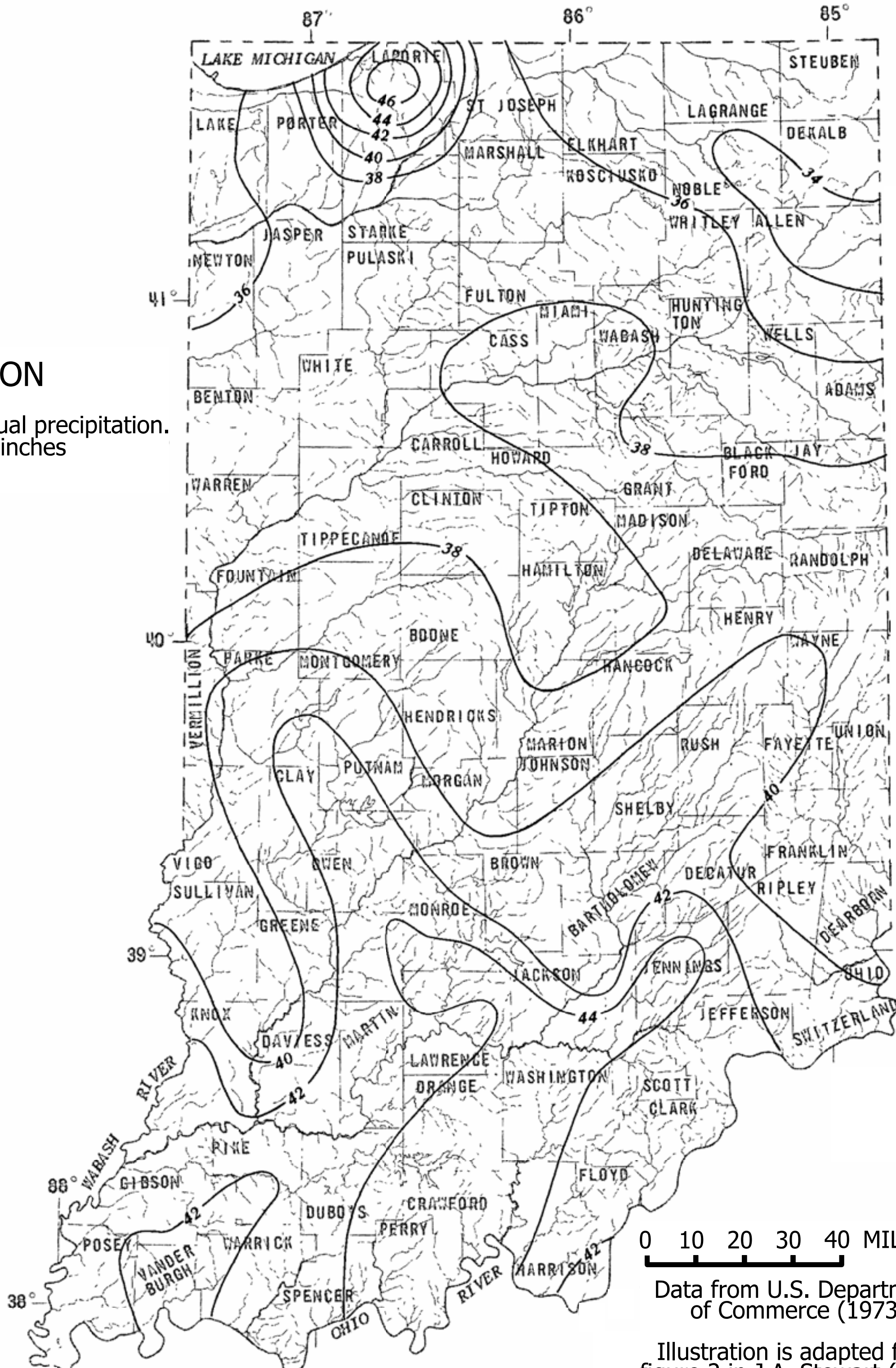


AREAS FOR SELECTING FLOOR-FREQUENCY
USGS ESTIMATING EQUATIONS

Figure 29-9A

EXPLANATION

–42– Line of equal precipitation. Interval 2 inches

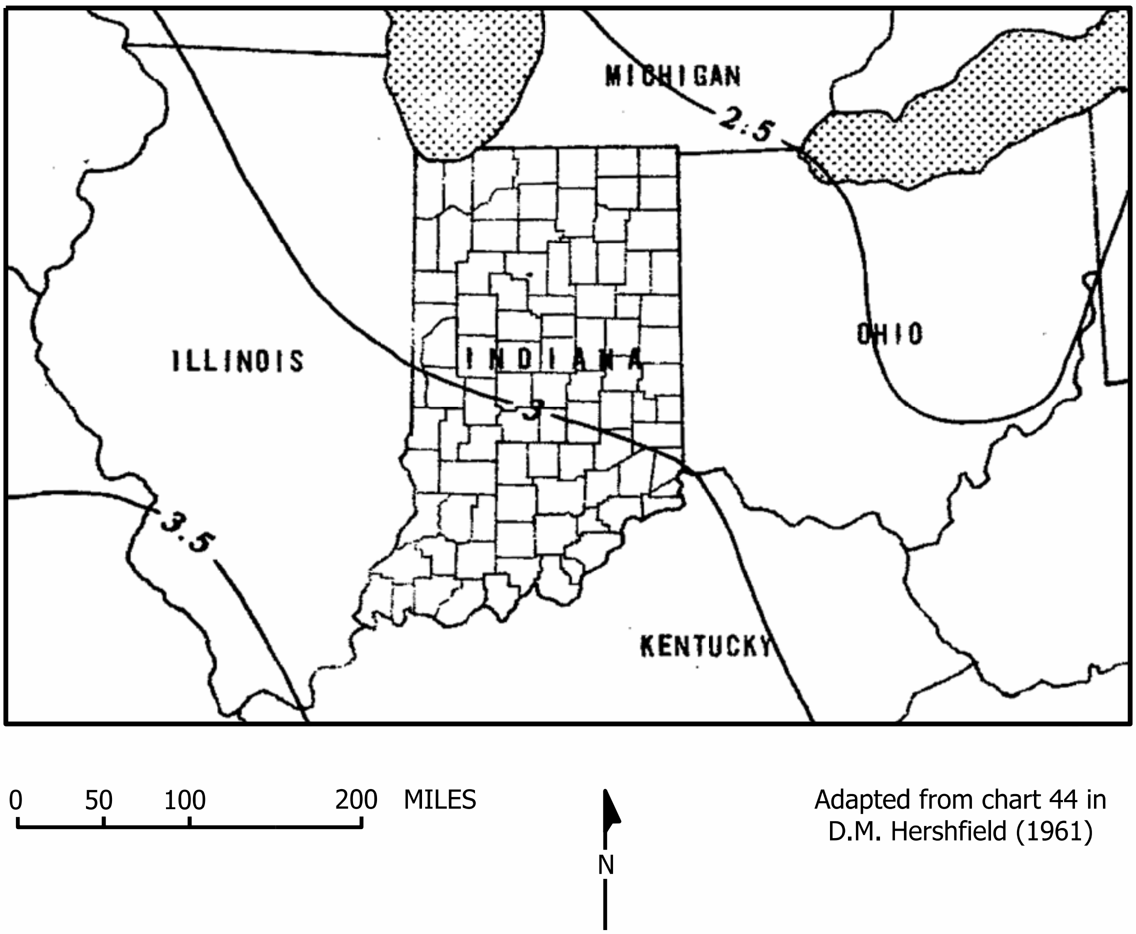


Data from U.S. Department of Commerce (1973)

Illustration is adapted from figure 2 in J.A. Stewart (1983)

MEAN ANNUAL PRECIPITATION (1941-1970)

Figure 29-9B



Adapted from chart 44 in D.M. Hershfield (1961)

EXPLANATION

—3.5— Line of equal precipitation. Interval 0.5 inches

TWO-YEAR, 24-HOUR PRECIPITATION

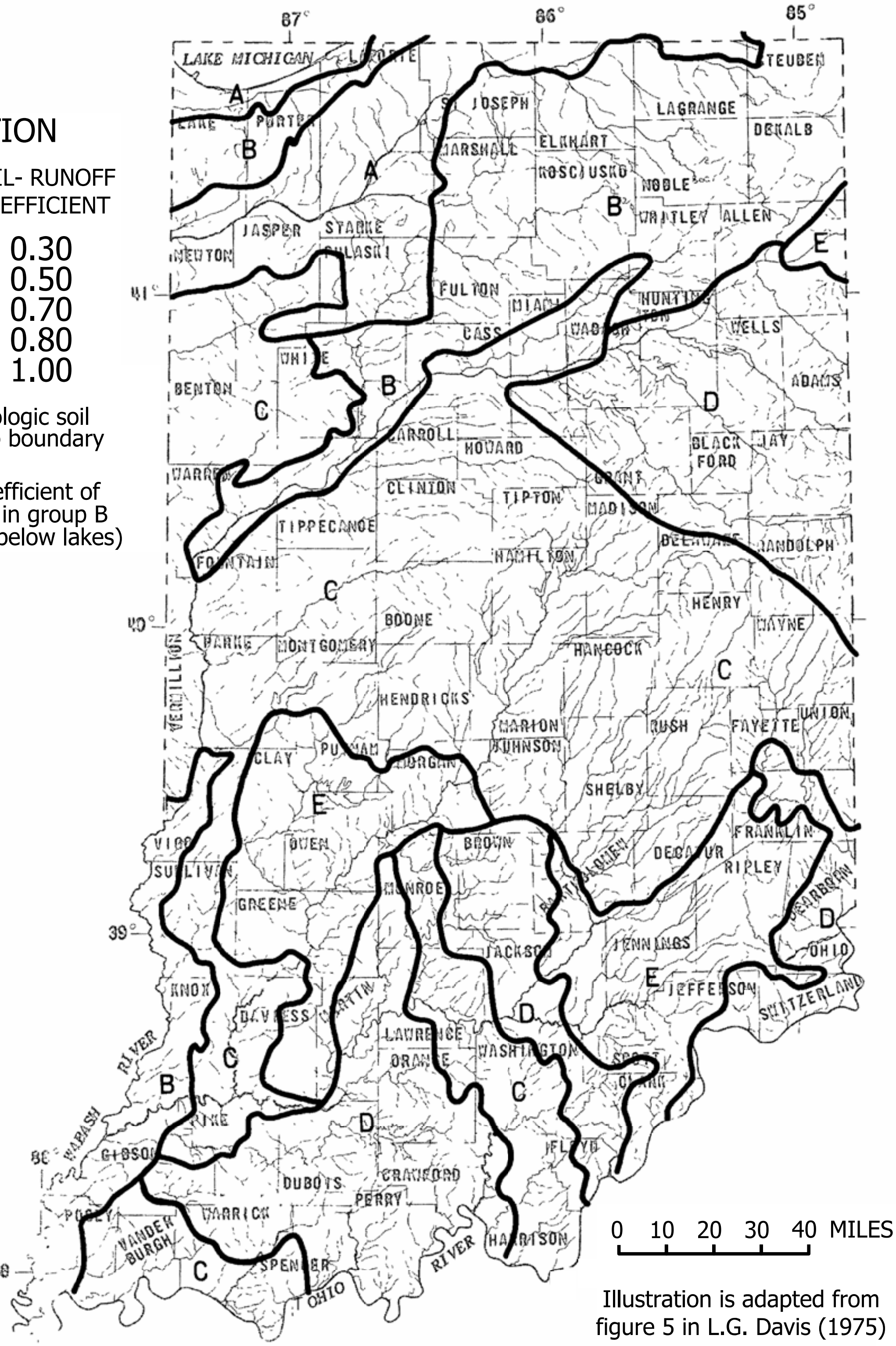
Figure 29-9C

EXPLANATION

HYDROLOGIC SOIL GROUP	SOIL- RUNOFF COEFFICIENT
A	0.30
B	0.50
C	0.70
D	0.80
E	1.00

Hydrologic soil group boundary

(Use a runoff coefficient of 0.30 for streams in group B that are directly below lakes)



MAJOR HYDROLOGIC SOIL GROUPS

Figure 29-9D

Equations	SEE	EY
AREA 1 (16 Stations)		
$Q_2 = 6.72\ DA^{0.714}\ (STOR + 1)^{-0.289}\ (PREC - 30)^{0.965}$	27%	3
$Q_{10} = 10.3\ DA^{0.701}\ (STOR + 1)^{-0.262}\ (PREC - 30)^{1.060}$	35%	3
$Q_{25} = 11.8\ DA^{0.697}\ (STOR + 1)^{-0.253}\ (PREC - 30)^{1.093}$	39%	3
$Q_{50} = 12.9\ DA^{0.696}\ (STOR + 1)^{-0.248}\ (PREC - 30)^{1.114}$	42%	4
$Q_{100} = 13.8\ DA^{0.695}\ (STOR + 1)^{-0.243}\ (PREC - 30)^{1.132}$	45%	5
AREA 2 (31 Stations)		
$Q_2 = 26.4\ DA^{0.708}\ (STOR + 1)^{-0.207}\ RC^{0.479}\ (PREC - 30)^{0.653}$	24%	4
$Q_{10} = 61.8\ DA^{0.655}\ (STOR + 1)^{-0.312}\ RC^{0.697}\ (PREC - 30)^{0.696}$	28%	4
$Q_{25} = 85.0\ DA^{0.635}\ (STOR + 1)^{-0.357}\ RC^{0.782}\ (PREC - 30)^{0.702}$	31%	5
$Q_{50} = 106\ DA^{0.619}\ (STOR + 1)^{-0.391}\ RC^{0.859}\ (PREC - 30)^{0.707}$	35%	6
$Q_{100} = 127\ DA^{0.608}\ (STOR + 1)^{-0.418}\ RC^{0.902}\ (PREC - 30)^{0.708}$	37%	7
AREA 3 (60 Stations)		
$Q_2 = 102\ DA^{0.758}\ SL^{0.273}\ (I_{24,2} - 2.5)^{0.948}$	36%	3
$Q_{10} = 141\ DA^{0.772}\ SL^{0.384}\ (I_{24,2} - 2.5)^{0.894}$	34%	4
$Q_{25} = 158\ DA^{0.776}\ SL^{0.423}\ (I_{24,2} - 2.5)^{0.868}$	36%	5
$Q_{50} = 170\ DA^{0.777}\ SL^{0.445}\ (I_{24,2} - 2.5)^{0.847}$	37%	7
$Q_{100} = 181\ DA^{0.779}\ SL^{0.466}\ (I_{24,2} - 2.5)^{0.831}$	39%	9
AREA 4 (46 Stations)		
$Q_2 = 16.8\ DA^{0.435}\ SL^{0.528}\ L^{0.860}\ (I_{24,2} - 2.5)^{0.459}$	31%	3
$Q_{10} = 24.1\ DA^{0.517}\ SL^{0.628}\ L^{0.769}\ (I_{24,2} - 2.5)^{0.445}$	30%	6
$Q_{25} = 27.4\ DA^{0.545}\ SL^{0.664}\ L^{0.741}\ (I_{24,2} - 2.5)^{0.448}$	32%	7
$Q_{50} = 29.6\ DA^{0.554}\ SL^{0.687}\ L^{0.738}\ (I_{24,2} - 2.5)^{0.458}$	34%	9
$Q_{100} = 32.0\ DA^{0.565}\ SL^{0.705}\ L^{0.730}\ (I_{24,2} - 2.5)^{0.464}$	37%	11

Q_t = Peak Discharge (ft^3/s)
DA = Drainage Area (mi^2)
SL = Channel Slope (ft/mi)
L = Channel Length (mi)
SEE = Standard Error of Estimate
STOR = Lakes, Ponds, Wetlands (%)
PREC = Average Annual Precipitation (in.)
$I_{24,2}$ = Max. 24-h, 2-yr Precipitation
RC = Runoff Coefficient
EY = Equivalent Years of Record

PREDICTION EQUATIONS, STANDARD ERRORS OF ESTIMATE, AND EQUIVALENT YEARS OF RECORD

Figure 29-9E

Equations	SEE	EY
AREA 5 (35 Stations)		
$Q_2 = 45.5\ DA^{0.760}\ SL^{0.390}$	30%	3
$Q_{10} = 67.7\ DA^{0.780}\ SL^{0.469}$	33%	5
$Q_{25} = 77.0\ DA^{0.790}\ SL^{0.499}$	36%	5
$Q_{50} = 83.8\ DA^{0.805}\ SL^{0.516}$	39%	7
$Q_{100} = 91.2\ DA^{0.811}\ SL^{0.529}$	42%	8
AREA 6 (32 Stations)		
$Q_2 = 681\ DA^{0.691}\ RC^{0.856}\ (I_{24,2} - 2.5)^{1.771}$	27%	5
$Q_{10} = 2{,}177\ DA^{0.662}\ RC^{0.865}\ (I_{24,2} - 2.5)^{1.980}$	29%	7
$Q_{25} = 3{,}165\ DA^{0.598}\ RC^{0.852}\ (I_{24,2} - 2.5)^{2.035}$	32%	7
$Q_{50} = 3{,}908\ DA^{0.584}\ RC^{0.849}\ (I_{24,2} - 2.5)^{2.049}$	34%	10
$Q_{100} = 4{,}734\ DA^{0.570}\ RC^{0.834}\ (I_{24,2} - 2.5)^{2.068}$	37%	12
AREA 7 (22 Stations)		
$Q_2 = 22.6\ DA^{0.468}\ SL^{0.414}\ L^{0.624}\ RC^{0.846}$	26%	3
$Q_{10} = 45.7\ DA^{0.350}\ SL^{0.439}\ L^{0.726}\ RC^{0.862}$	29%	4
$Q_{25} = 56.4\ DA^{0.318}\ SL^{0.458}\ L^{0.754}\ RC^{0.862}$	32%	4
$Q_{50} = 63.6\ DA^{0.300}\ SL^{0.473}\ L^{0.770}\ RC^{0.860}$	35%	5
$Q_{100} = 70.1\ DA^{0.285}\ SL^{0.488}\ L^{0.785}\ RC^{0.854}$	38%	6

Q_t =	Peak Discharge (ft^3/s)	STOR =	Lakes, Ponds, Wetlands (%)
DA =	Drainage Area (mi^2)	PREC =	Average Annual Precipitation (in.)
SL =	Channel Slope (ft/mi)	$I_{24,2}$ =	Max. 24-h, 2-yr Precipitation
L =	Channel Length (mi)	RC =	Runoff Coefficient
SEE =	Standard Error of Estimate	EY =	Equivalent Years of Record

PREDICTION EQUATIONS, STANDARD ERRORS OF ESTIMATE, AND EQUIVALENT YEARS OF RECORD

Figure 29-9E (contd.)

Area	*DA* (mi^2)	SLOPE (ft/mi)	$I_{24,2}$ (in.)	*STOR* (%)	*RC*	LENGTH (mi)	*PREC* (in.)	No. of Gaging Stations
1	0.17 - 3370			0 - 13.3				16
2	0.17 - 1967			0 - 4.1	0.5 - 0.8		34 - 46	31
3	0.31 - 4927	2.0 - 149	2.85 - 3.15				34 - 39	60
4	0.07 - 1224	2.4 - 267	2.80 - 3.30			0.3 - 77.1		46
5	0.04 - 11,125	1.2 - 236						35
6	0.10 - 856		2.70 - 3.00		0.3 - 0.8			32
7	0.39 - 1578	0.9 - 39.7			0.3 - 0.7	1.1 - 78.6		22

RANGE OF AREA-BASIN CHARACTERISTICS
FOR USGS REGRESSION EQUATION

Figure 29-9F

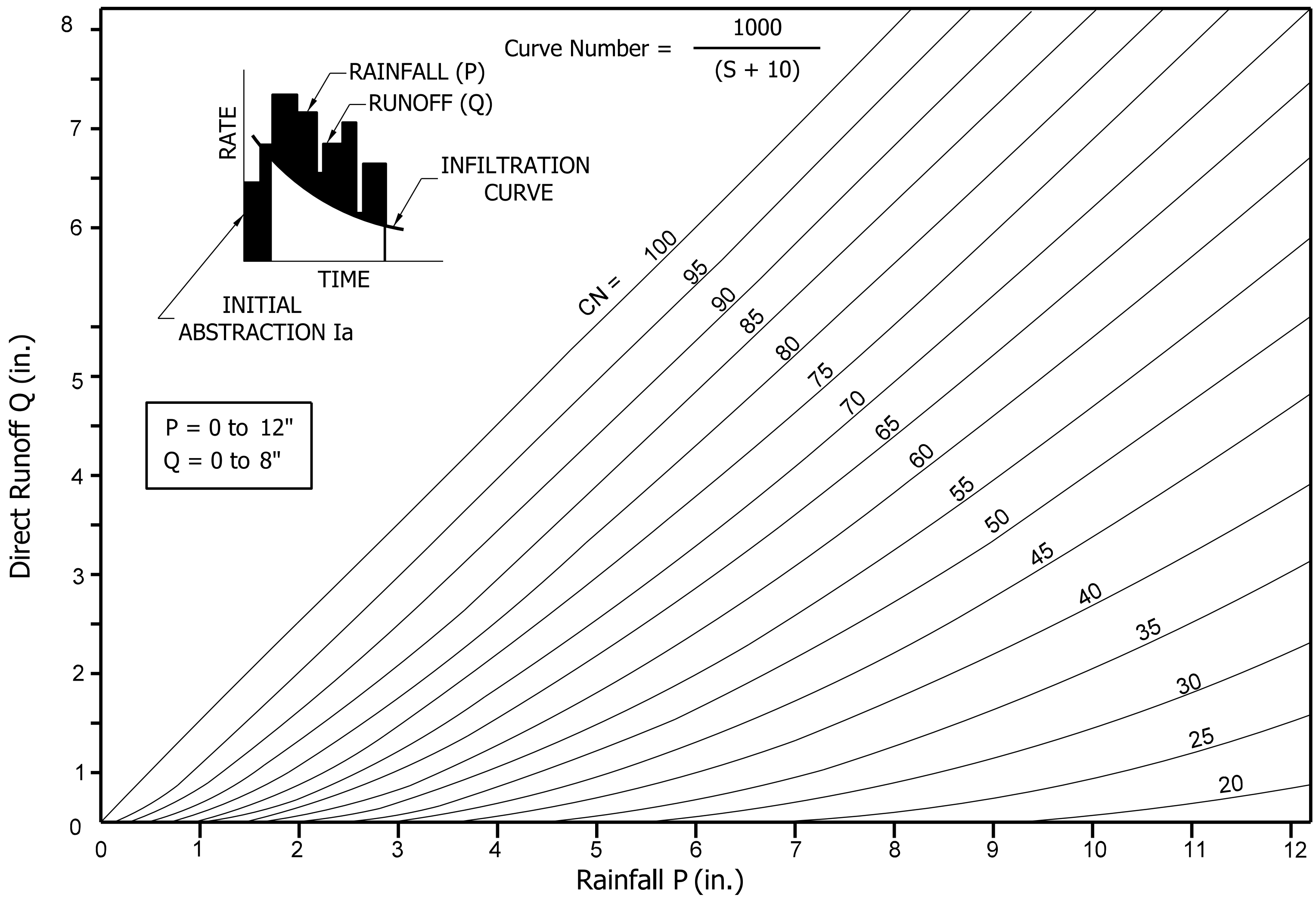
Each Indiana Station Contains Four Quartiles

% Storm Time	Indianapolis				Evansville				Fort Wayne				South Bend			
	I	II	III	IV	I	II	III	IV	I	II	III	IV	I	II	III	IV
0	0.0	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
10	20.00	6.50	5.26	6.67	22.82	6.28	5.13	6.92	20.00	6.67	6.00	7.14	20.00	7.50	7.00	8.26
20	40.80	18.13	11.55	14.25	44.69	17.33	11.11	14.04	41.11	17.14	12.23	14.23	40.00	18.57	13.33	16.35
30	54.95	35.85	17.06	20.00	57.11	33.33	16.67	20.51	54.83	34.17	18.86	20.00	51.67	34.00	20.00	22.73
40	62.50	52.94	24.24	26.09	65.33	53.09	25.44	27.06	62.00	52.18	26.15	25.71	60.89	51.43	27.50	28.50
50	68.75	67.86	37.78	33.33	71.43	69.57	37.93	34.21	68.42	66.67	38.46	33.33	67.35	66.67	39.13	34.04
60	76.67	76.52	58.33	40.00	78.15	78.57	57.39	40.91	75.00	76.36	57.23	38.00	75.00	75.17	58.46	40.20
70	83.05	83.81	78.03	50.00	84.66	85.60	77.44	50.79	81.62	84.29	76.11	48.50	80.83	82.32	75.98	50.00
80	89.70	90.67	88.68	68.57	90.00	91.72	88.54	69.70	87.50	90.00	87.69	68.24	86.67	88.89	86.79	67.50
90	95.00	95.89	95.29	88.37	95.36	96.50	95.88	89.36	93.75	95.56	95.08	87.88	92.89	94.78	94.17	87.50
100	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00

Note: Quartile II is recommended for use.

HUFF DISTRIBUTION OF DESIGN RAINFALL
(50% Probability Curve Ordinates)

Figure 29-10A



NRCS RELATION BETWEEN DIRECT RUNOFF, CURVE NUMBER AND PRECIPITATION

Figure 29-10B

Cover Description		Curve Number for Hydrologic Soil Group			
Cover Type and Hydrologic Condition	Average Percent Impervious Area [2]	A	B	C	D
Fully-developed urban area (vegetation established)					
Open space (lawn, park, golf course, cemetery, etc.) [2]					
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 Area:					
Paved parking lot, roof, drive, etc. (excluding right of way)		98	98	98	98
Street or Road:					
Paved; curbs and storm drains (excluding right of way)		98	98	98	98
Paved; open ditches (including right of way)		83	89	92	93
Gravel (including right of way)		76	85	89	91
Dirt (including right of way)		72	82	87	89
Urban Area:					
Commercial or Business	85	89	92	94	95
Industrial	72	81	88	91	93
Residential Area by Average Lot Size:					
0.12 ac or less (townhouse)	65	77	85	90	92
0.25 ac	38	61	75	83	87
0.33 ac	30	57	72	81	86
0.50 ac	25	54	70	80	85
1.00 ac	20	51	68	79	84
2.00 ac	12	46	65	77	82
Developing Urban Area					
Newly-graded area (pervious area only, no vegetation)		77	86	91	94
Idle land (CN is determined using cover types similar to those shown in Figure 29-10E).					

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

2 *The average percent impervious area shown was used to develop the composite CN. Other assumptions are as follows: An impervious area is directly connected to the drainage system and has a CN of 98; a pervious area is considered equivalent to open space in good hydrologic condition. If the impervious area is not connected, the NRCS method has an adjustment to reduce the effect.*

3 *CN shown is equivalent to that for a pasture. A composite CN may be computed for other combinations of open-space cover type.*

RUNOFF CURVE NUMBER FOR URBAN AREA [1]

Figure 29-10C

Cover Description Cover Type and Hydrologic Condition	Curve Number for Hydrologic Soil Group			
	A	B	C	D
Cultivated Land (Row Crops):				
With conservation treatment	62	71	78	81
Without conservation treatment	72	81	88	91
Pasture or Range Land:				
Poor condition	68	79	86	89
Good condition	39	61	74	80
Meadow:				
Good condition	30	58	71	78
Wood or Forest Land:				
Thin stand, poor cover, no mulch	45	66	77	83
Good cover	25	55	70	77

RUNOFF CURVE NUMBER FOR UNDEVELOPED AREA

Figure 29-10D

Cover Description	Curve Number for Hydrologic Soil Group			
Cover Type and Hydrologic Condition	A	B	C	D
Pasture, grassland or range with continuous forage for grazing:				
Poor	68	79	86	89
Fair	49	69	79	84
Good	39	61	74	80
Meadow with continuous grass, protected from grazing and generally mowed for hay	30	58	71	78
Brush/brush-weed-grass mixture with brush being the major element:				
Poor	48	67	77	83
Fair	35	56	70	77
Good	30	48	65	73
Woods and grass combination (orchard or tree farm):				
Poor	57	73	82	86
Fair	43	65	76	82
Good	32	58	72	79
Woods:				
Poor	45	66	77	83
Fair	36	60	73	79
Good	30	55	70	77
Farmsteads	59	74	82	86

RUNOFF CURVE NUMBER FOR AGRICULTURAL LAND

Figure 29-10E

CN for Average Condition	Corresponding CN	
	Dry	Wet
100	100	100
95	87	98
90	78	96
85	70	94
80	63	91
75	57	88
70	51	85
65	45	82
60	40	78
55	35	74
50	31	70
45	26	65
40	22	60
35	18	55
30	15	50
25	12	43
15	6	30
5	2	13

Source: USDA Natural Resources Conservation Service TP-149 (NRCS-TP-149), A Method for Estimating Volume and Rate of Runoff in Small Watersheds, *revised April 1973.*

CONVERSION FROM AVERAGE ANTECEDENT MOISTURE CONDITIONS

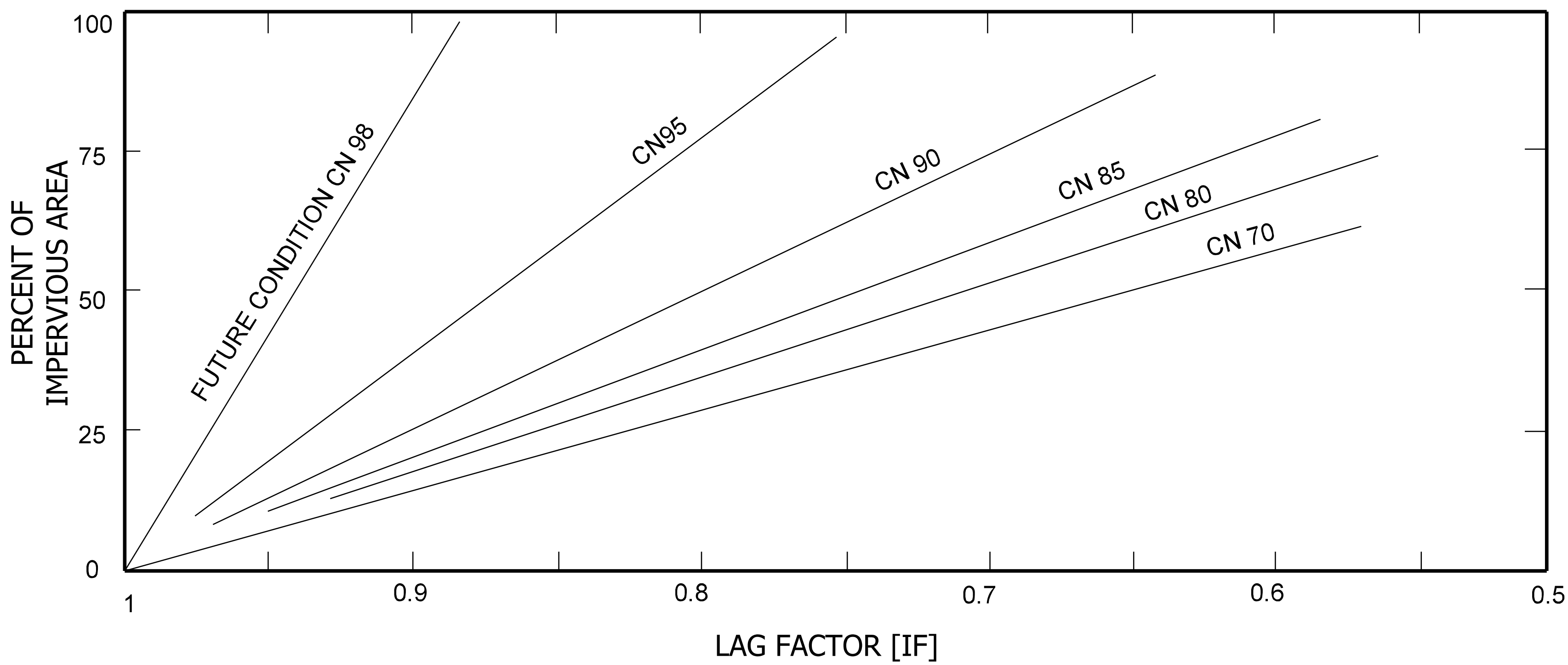
Figure 29-10F

Antecedent Condition	Condition Description	Growing Season 5-day Antecedent Rainfall	Dormant Season 5-day Antecedent Rainfall
Dry	An optimum condition of watershed soils, where soils are dry but not to the wilting point and when satisfactory plowing or cultivation takes place	Less than 1.5 in.	Less than 0.5 in.
Average	The average situation for an annual flood	1.5 in. ≤ rainfall < 2 in.	0.5 in. ≤ rainfall < 1 in.
Wet	Where a heavy rainfall, or light rainfall and low temperatures, have occurred during the five days previous to a given storm	Over 2 in.	Over 1 in.

Source: Natural Resources Conservation Service

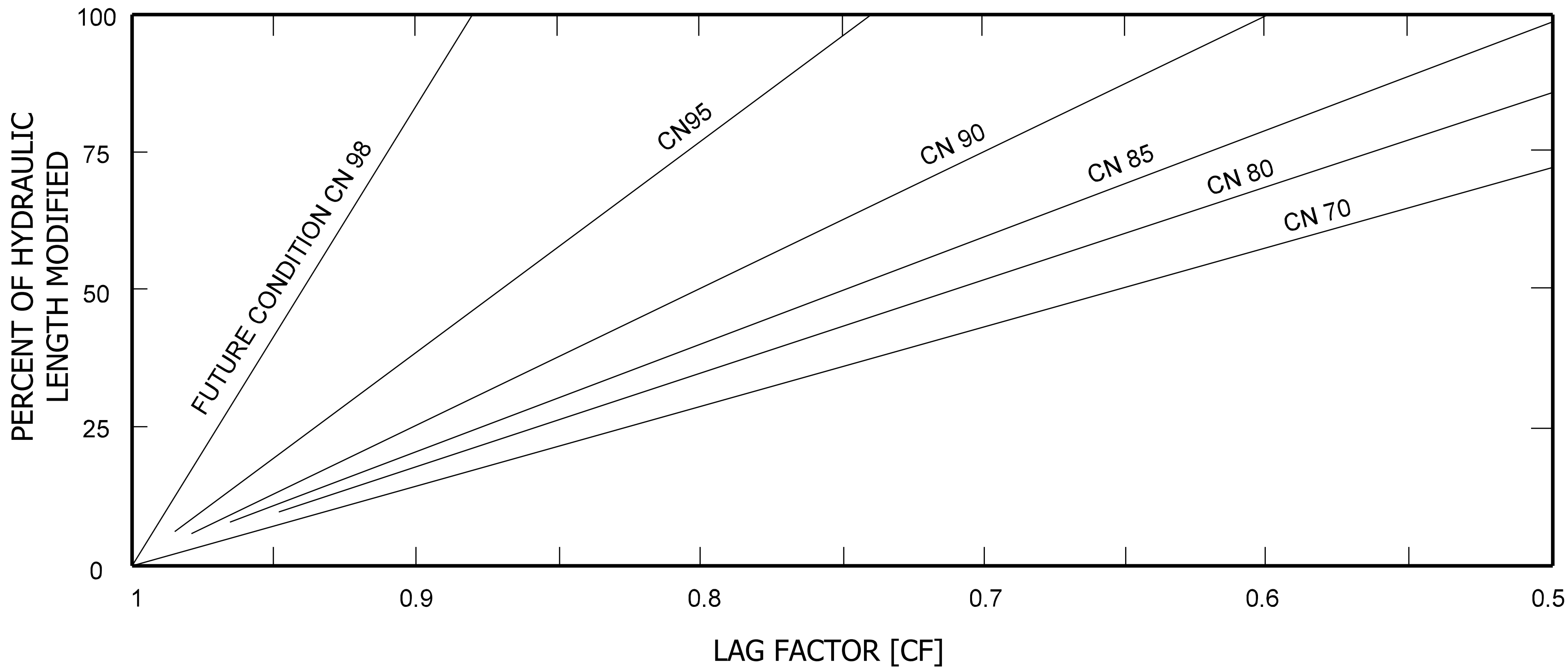
RAINFALL GROUP FOR ANTECEDENT SOIL MOISTURE CONDITION DURING GROWING OR DORMANT SEASON

Figure 29-10G



FACTORS FOR ADJUSTING LAG
(When Impervious Areas Occur in Watershed)

Figure 29-10H



FACTORS FOR ADJUSTING LAG
(When Main Channel Has Been Hydraulically Improved)

Figure 29-10I

```
*****************80-80 LIST OF INPUT DATA FOR TR-20 HYDROLOGY*****************

JOB TR-20                    FULLPRINT         SUMMARY
TITLE 123 EXAMPLE PROBLEM  LICK CREEK AND LITTLE LICK CREEK AT US 150
TITLE 000 100 YEAR STORMS   HUFF 2ND QUARTILE FOR EVANSVILLE
 5 RAINFL 8             .05
 8          0.000       0.040       0.063       0.120       0.173
 8          0.260       0.333       0.430       0.530       0.610
 8          0.696       0.740       0.786       0.820       0.856
 8          0.880       0.917       0.950       0.965       0.980
 8          1.00        1.000       1.000       1.000       1.000
 9 ENDTBL
 6 RUNOFF 1 001       5 1.51        75.2        0.90        1 0   0 0 1
 6 RUNOFF 1 001       6 0.59        78.5        0.60        1 0   0 0 1
 6 ADDHYD 4 001   5 6 7                                     1 0   0 0 1
   ENDATA
 7 INCREM 6             .1
 7 COMPUT 7 001    001  0.          1.67        0.25        8 2  01  01
   ENDCMP 1
 7 COMPUT 7 001    001  0.          2.37        0.5         8 2  02  02
   ENDCMP 1
 7 COMPUT 7 001    001  0.          3.10        1.          8 2  03  03
   ENDCMP 1
 7 COMPUT 7 001    001  0.          3.48        2.          8 2  04  04
   ENDCMP 1
   ENDJOB 2
0*****************************END OF 80-80 LIST******************************
1


 TR20 XEQ 03-25-98  12:23       EXAMPLE PROBLEM  LICK CREEK AND LITTLE LICK CREEK AT US 150                     JOB  1    PASS   1
      REV PC 09/83(.2)          100 YEAR STORMS   HUFF 2ND QUARTILE FOR EVANSVILLE                                         PAGE   1


 EXECUTIVE CONTROL OPERATION INCREM                                                                            RECORD ID
+                                         MAIN TIME INCREMENT =   .10 HOURS


 EXECUTIVE CONTROL OPERATION COMPUT                                                                            RECORD ID
+                                         FROM XSECTION   1
+                                                              TO XSECTION   1
        STARTING TIME =    .00    RAIN DEPTH =  1.67    RAIN DURATION=    .25    RAIN TABLE NO.= 8     ANT. MOIST. COND= 2
        ALTERNATE NO.= 1          STORM NO.= 1    MAIN TIME INCREMENT =   .10 HOURS


 OPERATION RUNOFF   CROSS SECTION    1
           OUTPUT HYDROGRAPH=  5
           AREA=    1.51 SQ MI    INPUT RUNOFF CURVE= 75.    TIME OF CONCENTRATION=    .90 HOURS
           INTERNAL HYDROGRAPH TIME INCREMENT=  .0125 HOURS

           PEAK TIME(HRS)                PEAK DISCHARGE(CFS)            PEAK ELEVATION(FEET)
                .76                           285.37                        (RUNOFF)

   RUNOFF VOLUME ABOVE BASEFLOW =    .24 WATERSHED INCHES,     230.90 CFS-HRS,     19.08 ACRE-FEET;    BASEFLOW =     .00 CFS


 OPERATION RUNOFF   CROSS SECTION    1
           OUTPUT HYDROGRAPH=  6
           AREA=     .59 SQ MI    INPUT RUNOFF CURVE= 79.    TIME OF CONCENTRATION=    .60 HOURS
           INTERNAL HYDROGRAPH TIME INCREMENT=  .0125 HOURS

           PEAK TIME(HRS)                PEAK DISCHARGE(CFS)            PEAK ELEVATION(FEET)
                .55                           225.53                        (RUNOFF)

   RUNOFF VOLUME ABOVE BASEFLOW =    .33 WATERSHED INCHES,     124.14 CFS-HRS,     10.26 ACRE-FEET;    BASEFLOW =     .00 CFS


 OPERATION ADDHYD   CROSS SECTION    1
           INPUT HYDROGRAPHS= 5,6      OUTPUT HYDROGRAPH= 7

           PEAK TIME(HRS)                PEAK DISCHARGE(CFS)            PEAK ELEVATION(FEET)
                .65                           475.30                        (NULL)

   RUNOFF VOLUME ABOVE BASEFLOW =    .26 WATERSHED INCHES,     355.05 CFS-HRS,     29.34 ACRE-FEET;    BASEFLOW =     .00 CFS


 EXECUTIVE CONTROL OPERATION ENDCMP                                                                            RECORD ID
+                                         COMPUTATIONS COMPLETED FOR PASS    1


1


 TR20 XEQ 03-25-98  12:23       EXAMPLE PROBLEM  LICK CREEK AND LITTLE LICK CREEK AT US 150                     JOB  1    PASS   2
      REV PC 09/83(.2)          100 YEAR STORMS   HUFF 2ND QUARTILE FOR EVANSVILLE                                         PAGE   2
```

TR 20 EXAMPLE PROBLEM

Figure 29-10J

```
EXECUTIVE CONTROL OPERATION COMPUT                                                                    RECORD ID
+                                      FROM XSECTION   1
+                                                      TO XSECTION   1
      STARTING TIME =   .00    RAIN DEPTH =  2.37    RAIN DURATION=   .50    RAIN TABLE NO.= 8    ANT. MOIST. COND= 2
      ALTERNATE NO.= 2         STORM NO.= 2    MAIN TIME INCREMENT =   .10 HOURS

OPERATION RUNOFF   CROSS SECTION   1
        OUTPUT HYDROGRAPH=  5
        AREA=   1.51 SQ MI   INPUT RUNOFF CURVE= 75.    TIME OF CONCENTRATION=   .90 HOURS
        INTERNAL HYDROGRAPH TIME INCREMENT=  .0250 HOURS

        PEAK TIME(HRS)              PEAK DISCHARGE(CFS)             PEAK ELEVATION(FEET)
           .90                           679.29                         (RUNOFF)

 RUNOFF VOLUME ABOVE BASEFLOW =   .58 WATERSHED INCHES,    569.28 CFS-HRS,    47.05 ACRE-FEET;   BASEFLOW =    .00 CFS

OPERATION RUNOFF   CROSS SECTION   1
        OUTPUT HYDROGRAPH=  6
        AREA=    .59 SQ MI   INPUT RUNOFF CURVE= 79.    TIME OF CONCENTRATION=   .60 HOURS
        INTERNAL HYDROGRAPH TIME INCREMENT=  .0250 HOURS

        PEAK TIME(HRS)              PEAK DISCHARGE(CFS)             PEAK ELEVATION(FEET)
           .70                           464.96                         (RUNOFF)

 RUNOFF VOLUME ABOVE BASEFLOW =   .73 WATERSHED INCHES,    277.23 CFS-HRS,    22.91 ACRE-FEET;   BASEFLOW =    .00 CFS

OPERATION ADDHYD   CROSS SECTION   1
        INPUT HYDROGRAPHS= 5,6      OUTPUT HYDROGRAPH= 7

        PEAK TIME(HRS)              PEAK DISCHARGE(CFS)             PEAK ELEVATION(FEET)
           .80                          1061.99                          (NULL)

 RUNOFF VOLUME ABOVE BASEFLOW =   .62 WATERSHED INCHES,    846.52 CFS-HRS,    69.96 ACRE-FEET;   BASEFLOW =    .00 CFS

EXECUTIVE CONTROL OPERATION ENDCMP                                                                    RECORD ID
+                                     COMPUTATIONS COMPLETED FOR PASS   2

1

TR20 XEQ 03-25-98  12:23      EXAMPLE PROBLEM  LICK CREEK AND LITTLE LICK CREEK AT US 150              JOB  1    PASS   3
    REV PC 09/83(.2)          100 YEAR STORMS   HUFF 2ND QUARTILE FOR EVANSVILLE                                  PAGE   3

EXECUTIVE CONTROL OPERATION COMPUT                                                                    RECORD ID
+                                      FROM XSECTION   1
+                                                      TO XSECTION   1
      STARTING TIME =   .00    RAIN DEPTH =  3.10    RAIN DURATION=  1.00    RAIN TABLE NO.= 8    ANT. MOIST. COND= 2
      ALTERNATE NO.= 3         STORM NO.= 3    MAIN TIME INCREMENT =   .10 HOURS

OPERATION RUNOFF   CROSS SECTION   1
        OUTPUT HYDROGRAPH=  5
        AREA=   1.51 SQ MI   INPUT RUNOFF CURVE= 75.    TIME OF CONCENTRATION=   .90 HOURS
        INTERNAL HYDROGRAPH TIME INCREMENT=  .0500 HOURS

        PEAK TIME(HRS)              PEAK DISCHARGE(CFS)             PEAK ELEVATION(FEET)
          1.18                          1037.90                         (RUNOFF)

 RUNOFF VOLUME ABOVE BASEFLOW =  1.04 WATERSHED INCHES,   1011.27 CFS-HRS,    83.57 ACRE-FEET;   BASEFLOW =    .00 CFS

OPERATION RUNOFF   CROSS SECTION   1
        OUTPUT HYDROGRAPH=  6
        AREA=    .59 SQ MI   INPUT RUNOFF CURVE= 79.    TIME OF CONCENTRATION=   .60 HOURS
        INTERNAL HYDROGRAPH TIME INCREMENT=  .0500 HOURS

        PEAK TIME(HRS)              PEAK DISCHARGE(CFS)             PEAK ELEVATION(FEET)
           .93                           591.65                         (RUNOFF)

 RUNOFF VOLUME ABOVE BASEFLOW =  1.23 WATERSHED INCHES,    468.61 CFS-HRS,    38.73 ACRE-FEET;   BASEFLOW =    .00 CFS

OPERATION ADDHYD   CROSS SECTION   1
        INPUT HYDROGRAPHS= 5,6      OUTPUT HYDROGRAPH= 7

        PEAK TIME(HRS)              PEAK DISCHARGE(CFS)             PEAK ELEVATION(FEET)
          1.09                          1541.21                          (NULL)

 RUNOFF VOLUME ABOVE BASEFLOW =  1.09 WATERSHED INCHES,   1479.88 CFS-HRS,   122.30 ACRE-FEET;   BASEFLOW =    .00 CFS
```

TR 20 EXAMPLE PROBLEM
(continued)
Figure 29-10J

```
 EXECUTIVE CONTROL OPERATION ENDCMP                                                                     RECORD ID
+                                            COMPUTATIONS COMPLETED FOR PASS    3

1

 TR20 XEQ 03-25-98  12:23        EXAMPLE PROBLEM  LICK CREEK AND LITTLE LICK CREEK AT US 150                 JOB  1   PASS   4
      REV PC 09/83(.2)           100 YEAR STORMS   HUFF 2ND QUARTILE FOR EVANSVILLE                                    PAGE   4

 EXECUTIVE CONTROL OPERATION COMPUT                                                                     RECORD ID
+                                            FROM XSECTION   1
+                                                          TO XSECTION   1
        STARTING TIME =    .00     RAIN DEPTH =  3.48    RAIN DURATION=  2.00    RAIN TABLE NO.= 8    ANT. MOIST. COND= 2
        ALTERNATE NO.= 4           STORM NO.= 4    MAIN TIME INCREMENT =   .10 HOURS

 OPERATION RUNOFF   CROSS SECTION   1
            OUTPUT HYDROGRAPH=  5
            AREA=   1.51 SQ MI   INPUT RUNOFF CURVE= 75.    TIME OF CONCENTRATION=   .90 HOURS
            INTERNAL HYDROGRAPH TIME INCREMENT=  .1000 HOURS

            PEAK TIME(HRS)              PEAK DISCHARGE(CFS)            PEAK ELEVATION(FEET)
                1.58                          899.88                        (RUNOFF)

   RUNOFF VOLUME ABOVE BASEFLOW =  1.30 WATERSHED INCHES,    1265.85 CFS-HRS,    104.61 ACRE-FEET;   BASEFLOW =    .00 CFS

 OPERATION RUNOFF   CROSS SECTION   1
            OUTPUT HYDROGRAPH=  6
            AREA=    .59 SQ MI   INPUT RUNOFF CURVE= 79.    TIME OF CONCENTRATION=   .60 HOURS
            INTERNAL HYDROGRAPH TIME INCREMENT=  .0800 HOURS

            PEAK TIME(HRS)              PEAK DISCHARGE(CFS)            PEAK ELEVATION(FEET)
                1.29                          484.48                        (RUNOFF)

   RUNOFF VOLUME ABOVE BASEFLOW =  1.51 WATERSHED INCHES,     576.72 CFS-HRS,     47.66 ACRE-FEET;   BASEFLOW =    .00 CFS

 OPERATION ADDHYD   CROSS SECTION   1
            INPUT HYDROGRAPHS= 5,6      OUTPUT HYDROGRAPH= 7

            PEAK TIME(HRS)              PEAK DISCHARGE(CFS)            PEAK ELEVATION(FEET)
                1.48                         1326.73                        (NULL)

   RUNOFF VOLUME ABOVE BASEFLOW =  1.36 WATERSHED INCHES,    1842.57 CFS-HRS,    152.27 ACRE-FEET;   BASEFLOW =    .00 CFS

 EXECUTIVE CONTROL OPERATION ENDCMP                                                                     RECORD ID
+                                            COMPUTATIONS COMPLETED FOR PASS    4

 EXECUTIVE CONTROL OPERATION ENDJOB                                                                     RECORD ID
1

 TR20 XEQ 03-25-98  12:23        EXAMPLE PROBLEM  LICK CREEK AND LITTLE LICK CREEK AT US 150                 JOB  1   SUMMARY
      REV PC 09/83(.2)           100 YEAR STORMS   HUFF 2ND QUARTILE FOR EVANSVILLE                                    PAGE   5
```

SUMMARY TABLE 1 - SELECTED RESULTS OF STANDARD AND EXECUTIVE CONTROL INSTRUCTIONS IN THE ORDER PERFORMED
(A STAR(*) AFTER THE PEAK DISCHARGE TIME AND RATE (CFS) VALUES INDICATES A FLAT TOP HYDROGRAPH
A QUESTION MARK(?) INDICATES A HYDROGRAPH WITH PEAK AS LAST POINT.)

SECTION/ STRUCTURE ID	STANDARD CONTROL OPERATION	DRAINAGE AREA (SQ MI)	RAIN TABLE #	ANTEC MOIST COND	MAIN TIME INCREM (HR)	PRECIPITATION BEGIN (HR)	PRECIPITATION AMOUNT (IN)	PRECIPITATION DURATION (HR)	RUNOFF AMOUNT (IN)	PEAK DISCHARGE ELEVATION (FT)	PEAK DISCHARGE TIME (HR)	PEAK DISCHARGE RATE (CFS)	PEAK DISCHARGE RATE (CSM)
ALTERNATE 1 STORM 1													
XSECTION 1	RUNOFF	1.51	8	2	.10	.0	1.67	.25	.24	---	.70	285.37	189.0
XSECTION 1	RUNOFF	.59	8	2	.10	.0	1.67	.25	.33	---	.55	225.53	382.3
XSECTION 1	ADDHYD	2.10	8	2	.10	.0	1.67	.25	.26	---	.65	475.30	226.3
ALTERNATE 2 STORM 2													
XSECTION 1	RUNOFF	1.51	8	2	.10	.0	2.37	.50	.58	---	.90	679.29	449.9
XSECTION 1	RUNOFF	.59	8	2	.10	.0	2.37	.50	.73	---	.70	464.96	788.1
XSECTION 1	ADDHYD	2.10	8	2	.10	.0	2.37	.50	.62	---	.80	1061.99	505.7
ALTERNATE 3 STORM 3													

TR 20 EXAMPLE PROBLEM
(continued)
Figure 29-10J

XSECTION	1	RUNOFF	1.51	8	2	.10	.0	3.10	1.00	1.04	---	1.18	1037.90	687.3
XSECTION	1	RUNOFF	.59	8	2	.10	.0	3.10	1.00	1.23	---	.93	591.65	1002.8
XSECTION	1	ADDHYD	2.10	8	2	.10	.0	3.10	1.00	1.09	---	1.09	1541.21	733.9

ALTERNATE 4 STORM 4

XSECTION	1	RUNOFF	1.51	8	2	.10	.0	3.48	2.00	1.30	---	1.58	899.88	595.9
XSECTION	1	RUNOFF	.59	8	2	.10	.0	3.48	2.00	1.51	---	1.29	484.48	821.2
XSECTION	1	ADDHYD	2.10	8	2	.10	.0	3.48	2.00	1.36	---	1.48	1326.73	8

1

TR20 XEQ 03-25-98 12:23 EXAMPLE PROBLEM LICK CREEK AND LITTLE LICK CREEK AT US 150 JOB 1 SUMMARY
REV PC 09/83(.2) 100 YEAR STORMS HUFF 2ND QUARTILE FOR EVANSVILLE PAGE 6

SUMMARY TABLE 3 - DISCHARGE (CFS) AT XSECTIONS AND STRUCTURES FOR ALL STORMS AND ALTERNATES

XSECTION/ STRUCTURE ID	DRAINAGE AREA (SQ MI)	STORM NUMBERS.......... 1	2	3	4
0 XSECTION 1	2.10				
ALTERNATE 1		475.30	.00	.00	.00
ALTERNATE 2		.00	1061.99	.00	.00
ALTERNATE 3		.00	.00	1541.21	.00
ALTERNATE 4		.00	.00	.00	1326.73

1END OF 1 JOBS IN THIS RUN

TR 20 EXAMPLE PROBLEM
(continued)
Figure 29-10J