CHAPTER 7

HYDROLOGY

7.1 Hydrologic Design Policies

Introduction
7.1.1

Following is a summary of policies which shall be followed for hydrologic analysis. For a more detailed discussion refer to the publication, Highway Drainage Guidelines, published by the American Association of State Highway and Transportation Officials.

Surveys
7.1.2

Since hydrologic considerations can influence the selection of a highway corridor and the alternate routes within the corridor, studies and investigations, including consideration of the environmental and ecological impact of the project, shall be undertaken. Also special studies and investigations may be required at sensitive locations. The magnitude and complexity of these studies shall be commensurate with the importance and magnitude of the project and problems encountered. Typical data to be included in such surveys or studies are: topographic maps, aerial photographs, stream flow records, historical high-water elevations, flood discharges, and locations of hydraulic features such as reservoirs, water projects, and designated or regulatory floodplain areas. **See the MDT Survey Manual for specific requirements. Form HYD-1 is MDT's permanent record and is required as described in the Survey Manual.**

Flood Hazards
7.1.3

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

Coordination
7.1.4

Since many levels of government plan, design, and construct highway and water resource projects which might have an affect on each other, interagency coordination is desirable and often necessary. In addition agencies can share data and experiences within project areas to assist in the completion of accurate hydrologic analysis.

Documentation
7.1.5

Experience indicates that the design of highway drainage facilities should be adequately documented. Frequently, it is necessary to refer to plans and specifications long after the actual construction has been completed. Thus it is necessary to fully document the results of all hydrologic analysis.

Factors Affecting Flood Runoff
7.1.6

For all hydrologic analysis, the following factors shall be evaluated and included when they will have a significant effect on the final results.

- Drainage basin characteristics including: size, shape, slope, land use, geology, soil type, surface infiltration, and storage.
- Stream channel characteristics including: geometry and configuration, natural and artificial controls, channel modification, aggradation - degradation, and ice and debris.
- Flood plain characteristics.

- Meteorological characteristics such as precipitation amounts and type (rain, snow, hail, or combinations thereof), storm cell size and distribution characteristics, storm direction, and time rate of precipitation (hyetograph).

Flood History 7.1.7

All hydrologic analysis shall consider the flood history of the area and the effect of these historical floods on existing and proposed structures. The flood history shall include the historical floods and the flood history of any existing structures. **This information should be obtained from the local Maintenance Section for interstate and primary roadways and from the County for secondary and off-system roadways.**

Hydrologic Method 7.1.8

Many hydrologic methods are available. The methods to be used and the circumstances for their use are listed below. If possible the method should be calibrated to local conditions and tested for accuracy and reliability.

Approved Methods 7.1.9

For drainage areas greater than 1 square mile (2.5 square kilometers), a brief hydrologic report shall be written describing the area, runoff characteristics, historical events, methods used in determining design flow, and the reason a particular method was used or not used. On many projects, one report will suffice for all drainages; however, a report on an individual drainage may be written if conditions warrant.

The majority of drainage areas greater than 1 square mile (2.5 square kilometers) will require a composite analysis. Approximate discharges of existing culverts and bridges along with field comments on adequacy shall be used to temper the actual flow used in design. In reviewing historical data, the designer shall review USGS Water Resources Summaries, USGS open-file reports (e.g. floods of 1948, 1952, 1964, etc.), MDT's Drainage Structure Flood Summaries, and USGS cooperative Bridge Site or culvert studies. These may reveal some peak flows not published in the gaging station summaries. The 500-year flood (for scour analysis) should be determined by the same methods as the other return period floods, not by using 1.7 times the 100-year flood, as noted in FHWA HEC-18. At least two of the following methods should be analyzed for all drainages, and compared to the flood history for the existing structure (i.e., what flow overtops the structure and how often has this occurred). The USGS regression equations and the Regional Frequency Analysis should be two of the methods used. Other methods may be used where appropriate.

- **Current regression equations developed by the USGS; or by James Nallick (for drainage areas less than 1 square mile (250 hectares) east of the Continental Divide).**

- **Current channel geometry equations developed by the USGS.**
- **Where there is a gaging station on the stream with at least 10 years of record, a log-Pearson III analysis of the gage data shall be completed.**
- **Regional Frequency Analysis using surrounding gages.**
- **Regional Regression Analysis using MDT's regression program.**
- **The 100-year discharges specified in the FEMA Flood Insurance Study.**
- **The SCS Curve Number Method (also called the Soil-Cover-Complex Method) for drainages less than 3 square miles (8 square kilometers).**
- **The Rational Method may be used only for drainage areas less than 200 acres (80 hectares).**
- **Suitable hydrograph methods may be used for routing calculations to attenuate peak flows, after the peak flow has been determined.**

Design Frequency 7.1.10

A design frequency should be selected commensurate with the facilities cost, amount of traffic, potential flood hazard to property, expected level of service, political considerations, and budgetary constraints as well as the magnitude and risk associated with damages from larger flood events. With long highway routes having no practical detour, where many sites are subject to independent flood events, it may be necessary to increase the design frequency at each site to avoid frequent route interruptions from floods. In selecting a design frequency, potential upstream land use should be considered which could reasonably occur over the anticipated life of the drainage facility. **See Appendix A for specific criteria relating to design frequency.**

Risk Assessment 7.1.11

Hydrologic analysis should include the determination of several design flood frequencies for use in the hydraulic design. These frequencies are used to size different drainage facilities so as to allow for an optimum design, which considers both risk of damage and construction cost. Consideration should be given to what frequency flood was used to design other structures along a highway corridor. **MDT's abbreviated analysis for Risk Assessment is presented in Procedure Memoranda 11 and 11B, which are included as part of Appendix A.**

Review Frequency 7.1.12

All proposed structures obtained using the selected design frequency shall be reviewed using **the base flood and the overtopping or the 500-year flood, whichever is smaller,** to ensure that there are no unexpected flood hazards. **For bridges, an assessment should be made for the 500-year flood if overtopping is not practicable.**

7.2 Overview

Introduction
7.2.1

The analysis of the peak rate of runoff, volume of runoff, and time distribution of flow is fundamental to the design of drainage facilities. 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. On the other hand, it must be realized that any 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, and too little data are available on the factors influencing the rural and urban rainfall-runoff relationship to expect exact solutions.

Definition
7.2.2

Hydrology is generally defined as a science dealing with the interrelationship between water on and under the earth and in the atmosphere. For the purpose of this manual, hydrology will deal with estimating flood magnitudes as the result of precipitation. In the design of highway drainage structures, floods are usually considered in terms of peak runoff or discharge in cubic feet per second (cfs) and hydrographs as discharge per time. For structures which are designed to control volume of runoff, like detention storage facilities, or where flood routing through culverts is used, then the entire discharge hydrograph will be of interest.

Factors Affecting Floods
7.2.3

In the hydrologic analysis for a drainage structure, it must be recognized that there are many variable factors that affect floods. Some of the factors which need to be recognized and considered on an individual site by site basis are such things as:

- rainfall amount and storm distribution,
- drainage area size, shape and orientation,
- ground cover,
- type of soil,
- slopes of terrain and stream(s),
- antecedent moisture condition,
- storage potential (over bank, ponds, wetlands, reservoirs, channel, etc.),
- watershed development potential,
- type of precipitation (rain, snow, hail, or combinations thereof),
- elevation, and
- mixed population events.

Sources of Information
7.2.4

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 a particular analysis. A comprehensive list of data sources is included in the Data Collection Chapter of this handbook.

7.3 Symbols and Definitions

To provide consistency within this chapter as well as throughout this manual the following symbols will be used. These symbols were selected because of their wide use in hydrologic publications.

Table 7-1 Symbols and Definitions

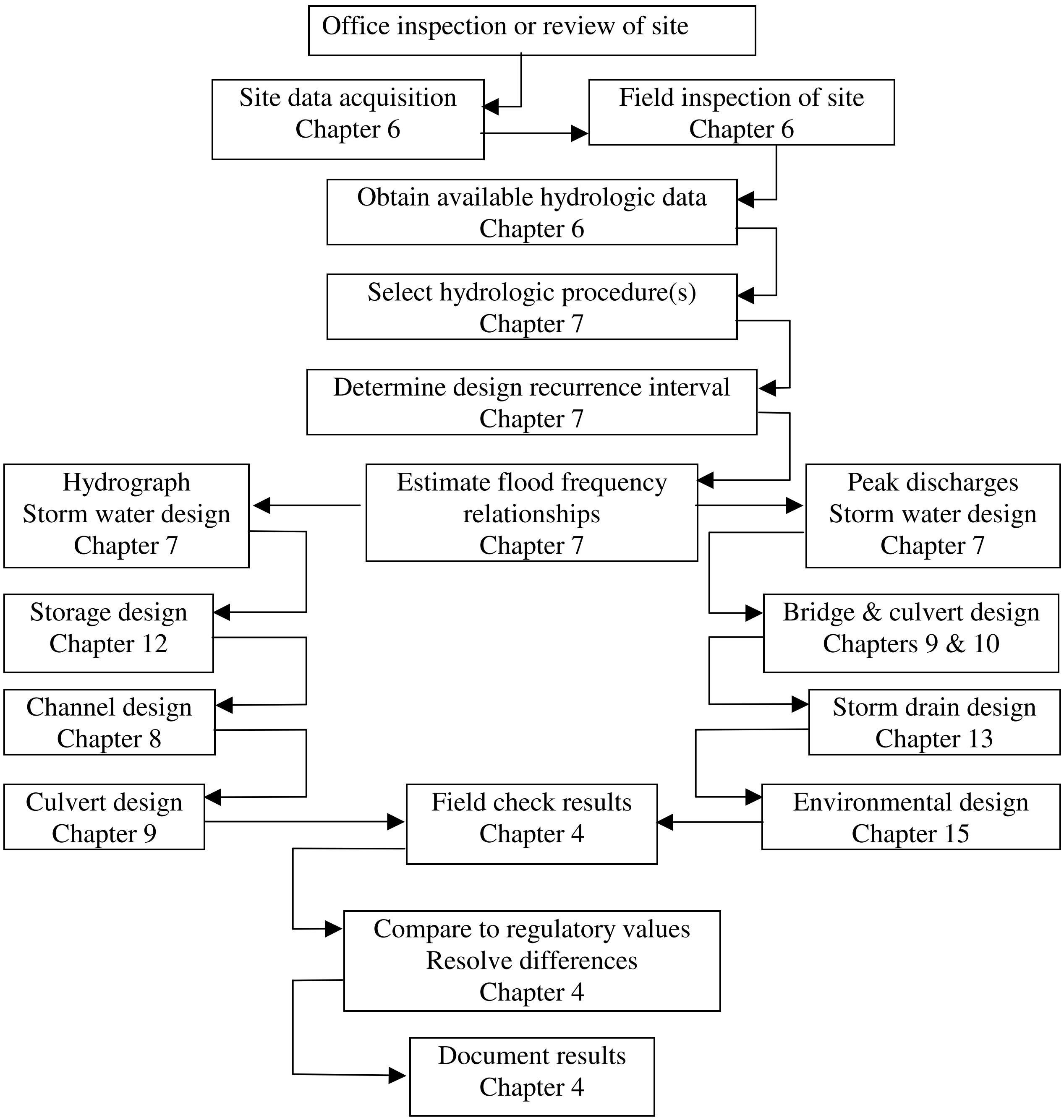
Symbol	Definition	Units
A	Drainage area	acres, sq.mi.
BDF	Basin development factor	%
C	Runoff coefficient	–
C_f	Frequency factor	–
CN	SCS-runoff curve number	–
C_t, C_p	Physiographic coefficients	–
d	Time interval	hours
DH	Difference in elevation	ft
I	Runoff intensity	in./hr
IA	Percentage of impervious area	%
I_a	Initial abstraction from total rainfall	in
K	Frequency factor for a particular return period and skew	–
L	Lag	hours
l	Length of mainstream to furthest divide	ft
L_{ca}	Length along main channel to a point opposite the watershed centroid	miles
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	cfs
q	Storm runoff during a time interval	in
R	Hydraulic radius	ft
RC	Regression constant	–
RQ	Equivalent rural peak runoff rate	cfs
S or Y	Ground slope	ft/ft or %
S	Potential maximum retention storage	
SCS	Soil Conservation Service	–
SL	Main channel slope	ft/mile
S_L	Standard deviation of the logarithms of the peak annual floods	–

Table 7-1 Symbols and Definitions

Symbol	Definition	Units
ST	Basin storage factor	%
T_B	Time base of unit hydrograph	hours
t_c or T_c	Time of concentration	min or hours
T_L	Lag time	hours
T_r	Snyder's duration of excess rainfall	hours
UQ	Urban peak runoff rate	cfs
V	Velocity	ft/s
X	Logarithm of the annual peak	–

7.4 Hydrologic Analysis Procedure Flowchart

The hydrologic analysis procedure flowchart shows the steps needed for the hydrologic analysis, and the designs that will use the hydrologic estimates.



7.5 Concept Definitions

Introduction	Following are discussions of concepts which will be important in a hydrologic analysis. These concepts will be used throughout the remainder of this chapter in dealing with different aspects of hydrologic studies.
Antecedent Moisture Conditions	Antecedent moisture conditions are the soil moisture conditions of the watershed at the beginning of a storm. These conditions affect the volume of runoff generated by a particular storm event. Notably they affect the peak discharge only in the lower range of flood magnitudes -- say below about the 15-year event threshold. As floods become more rare, antecedent moisture has a rapidly decreasing influence on runoff.
Depression Storage	Depression storage is the natural depressions within a watershed which store runoff. Generally after the depression storage is filled runoff will commence.
Frequency	Frequency is the number of times a flood of a given magnitude can be expected to occur on an average over a long period of time. Frequency analysis is then the estimation of peak discharges for various recurrence intervals. Another way to express frequency is with probability. Probability analysis seeks to define the flood flow with a probability of being equaled or exceeded in any year.
Hydraulic Roughness	Hydraulic roughness 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 as well as the channel storage characteristics.
Hydrograph	The hydrograph is a graph of the time distribution of runoff from a watershed.
Hyetographs	The hyetograph is a graph of the time distribution of rainfall over a watershed.
Infiltration	Infiltration 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.
Interception	Storage of rainfall on foliage and other intercepting surfaces during a rainfall event is called interception storage.
Lag Time	The lag time is defined as the time from the centroid of the excess rainfall to the peak of the hydrograph.

Concept Definitions (continued)

Peak Discharge	The peak discharge, sometimes called peak flow, is the maximum rate of flow of water passing a given point during or after a rainfall event or snowmelt.
Rainfall Excess	The rainfall excess is the water available to runoff after interception, depression storage and infiltration have been satisfied.
Stage	The stage of a river is the elevation of the water surface above some elevation datum.
Time Of Concentration	The time of concentration is the time it takes the drop of water falling on the hydraulically most remote point in the watershed to travel through the watershed to the outlet.
Unit Hydrograph	A unit hydrograph is the direct runoff hydrograph resulting from a rainfall event which has a specific temporal and spatial distribution and which lasts for a unit duration of time. 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.
References	For a more complete discussion of these concepts and others related to hydrologic analysis, the reader is referred to - Hydrology, Federal Highway Administration, Hydraulic Engineering Circular No. 19, October 1984, and Guidelines for Hydrology - Volume II Highway Drainage Guidelines, prepared by the Task Force On Hydrology and Hydraulics, AASHTO Highway Subcommittee on Design.

7.6 Design Frequency

Overview
7.6.1

Since it is not economically feasible to design a structure for the maximum runoff a watershed is capable of producing, a design frequency must be established. The frequency with which a given flood can be expected to occur is the reciprocal of the probability or chance that the 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 an average of once every five years. This is called the return period or recurrence interval (RI). Thus the exceedence probability equals 100/RI.

The designer should note that 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 any year; therefore, the 5-year flood could conceivably occur in several consecutive years. The same reasoning applies to floods with other return periods.

Design Frequency
7.6.2

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

- increase the flood hazard significantly for property,
- overtop the highway, or
- exceed a certain depth on the highway embankment.

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

<u>Storm Drains</u>: A storm drain shall 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:

- increase the flood hazard significantly for property,
- encroach on to the street or highway so as to cause a significant traffic hazard, or
- limit traffic, emerging vehicle, or pedestrian movement to an unreasonable extent.

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

Review Frequency 7.6.3

After sizing a drainage facility using a flood and sometimes the hydrograph corresponding to the design frequency, it shall be necessary to review this proposed facility with a base discharge. This is done to insure that there are no unexpected flood hazards inherent in the proposed facility(ies). The review flood shall be the 100-year event. In some cases, a flood event larger than the 100-year flood is used for analysis to ensure the safety of the drainage structure and downstream development.

Frequency Table 7.6.4

Appendix A presents a guide of preferred design frequencies to be used by MDT for the various drainage facilities on streets and highways.

Rainfall vs. Flood Frequency 7.6.5

Drainage structures are designed based on some flood frequency. However, certain hydrologic procedures use rainfall and rainfall frequency as the basic input. Thus it is commonly assumed that the 10-year rainfall will produce the 10-year flood. Depending on antecedent soil moisture conditions, and other hydrologic parameters this may be true or there may not be a direct relationship between rainfall and flood frequency.

Rainfall Curves 7.6.6

Rainfall data are available for many geographic areas. From these data, rainfall intensity values have been developed for the commonly used design frequencies. Appendix B at the end of this chapter contains the values available at this time for the MDT.

Discharge Determination 7.6.7

Estimating peak discharges of various recurrence intervals is one of the most common engineering challenges faced by drainage facility designers. The problem can be divided into two general categories:

- Gaged sites - the site is at or near a gaging station and the streamflow record is of sufficient length to be used to provide estimates of peak discharges. A complete record is defined as one having at least **20** years of continuous or synthesized data. In Montana, this is a relatively rare situation.

- Ungaged sites - the site is not near a gaging station and no streamflow record is available. This situation is very common in Montana.

This chapter will address hydrologic procedures that can be used for both categories.

7.7 Hydrologic Procedure Selection

Overview
7.7.1

Streamflow measurements for determining a flood frequency relationship at a site are usually unavailable; in such cases, it is accepted practice to estimate peak runoff rates and hydrographs using statistical or empirical methods. In general results from using several methods should be compared, not averaged. MDT practice shall be to use the discharge that best reflects local project conditions with the reasons documented. The MDT use for each procedure is outlined with each hydrologic procedure given below.

Peak Flow Rates or Hydrographs
7.7.2

A consideration of peak runoff rates for design conditions is generally adequate for conveyance systems such as storm drains or open channels. However, if the design must include flood routing (e.g., storage basins or complex conveyance networks), a flood hydrograph is usually required. Although the development of runoff hydrographs (typically more complex than estimating peak runoff rates) is often accomplished using computer programs, some methods are adaptable to nomographs or other desktop procedures.

Hydrologic Procedures
7.7.3

The methods presented in this manual were selected to reflect the methods commonly used by the MDT Hydraulics Section.

Analysis of Stream Gage Data - where stream gage data are available they can be used to develop peak discharges and hydrographs.

Rational Method - Provides peak runoff rates for small urban and rural watersheds less than **200 acres (80 hectares)**, but is best suited to urban storm drain systems. It should be used with caution if the time of concentration exceeds 30 minutes. Rainfall is a necessary input.

Regression Equations - Peak flow can be calculated by using regression equations developed for specific geographic regions. The equations are in the form of a log-log formula, where the dependent variable would be the peak flow for a given frequency, and the independent variables may be variables such as area, basin elevation, channel geometry, and other meteorological, physical or site specific data. **MDT will evaluate at least one of these procedures for all designs where the drainage area is within the limits of the equations.**

Log Pearson III Flood Frequency - With at least 25 years of continuous stream gage data the log Pearson III is considered to be the most reliable method for estimating flood frequency relationships. **This procedure should be evaluated for all sites where at least 10 years of data are available.** Data can be obtained from the Helena U.S.G.S. office.

Regional Frequency Analysis - This method consists of a plot of drainage area vs. flow using data from nearby gages, and a line of best fit, to determine the relationship between these two variables.

Hydrologic Procedures (continued)

<u>Regional Regression Analysis</u> - This method is similar to the Regional Frequency Analysis, except that the line of best fit is determined using the MDT Regression Program.

<u>Flood Insurance Study (FIS)</u> - Where a FEMA Flood Insurance Study has been completed, various return period flows will be published in the Study. These flows should always be reported in the Hydraulic Report, and will generally be used as the design flows. When several other reliable methods indicate that the predicted flows are greater than the FIS flows, the higher values should be used for design, but the FIS flow should be checked for compliance with floodplain criteria. If several other reliable methods indicate that the predicted flows are less than the FIS flows, the design should be evaluated at both the FIS flow and the lower flow.

<u>SCS Curve Number Method</u> - Also known as the Soil-Cover-Complex Method, this method requires rainfall data, soil type, and an estimate of the Curve Number to develop peak flow rates.

<u>Nallick Regression Equations</u> - These regression equations were developed by James Nallick, in cooperation with MDT. They are applicable to small drainage basins in the plains of eastern Montana, and utilize drainage area, average annual precipitation, and one-hour precipitation intensity as parameters.

<u>Wyoming Unit Hydrograph</u> - The USGS, in cooperation with the Wyoming State Highway Department, developed a unit hydrograph procedure for Wyoming. This procedure utilizes drainage area, basin slope, maximum basin relief and channel slope as parameters.

<u>Montana Unit Hydrograph</u> - The USGS, in cooperation with the Montana Department of Natural Resources and Conservation, developed a unit hydrograph procedure for large floods at ungaged sites in Montana. This procedure utilizes drainage area, main channel length, main channel slope, and distance from basin centroid to mouth, as parameters. These parameters are used to develop values for the Clark unit hydrograph method and the dimensionless unit hydrograph method.

<u>SCS Synthetic Unit Hydrograph</u> - The Soil Conservation Service has developed a synthetic unit hydrograph procedure which has been widely used for developing rural and urban hydrographs. The unit hydrograph used by the SCS method is based upon an analysis of a large number of natural unit hydrographs from a broad cross section of geographic locations and hydrologic regions. Rainfall is a necessary input.

7.8 Calibration

Definition 7.8.1

Calibration is a process of varying the parameters or coefficients of a hydrologic method so that it will estimate peak discharges and hydrographs consistent with local rainfall and streamflow data.

Hydrologic Accuracy 7.8.2

The accuracy of the hydrologic estimates will have a major affect on the design of drainage or flood control facilities. Although it might be argued that one hydrologic procedure is more accurate than another, practice has shown that all of the methods discussed in this chapter can, if calibrated, produce acceptable results consistent with observed or measured events. What should be emphasized is the need to calibrate the method for local conditions. This calibration process can result in much more accurate and consistent estimates of peak flows and hydrographs.

Calibration Process 7.8.3

The calibration process can vary depending on the data or information available for a local area. **Comparison of several different hydrologic methods is usually a satisfactory technique for calibrating peak flow hydrology.**

1. **The predicted hydrology should be compared to the known floods of record or overtopping events. For example, if an existing bridge would overtop at a flow equal to the predicted 50 year flow, and overtopping has occurred four times in the last 30 years, the hydrology should be considered suspect. Or, if the two largest floods of record are the 500-year flow and the 2000-year flow based on the predicted hydrology, the hydrology should also be considered suspect. The reverse could also be true. It may be known, based on discussions with the maintenance people, that the existing structure has not overtopped in the last 30 years, and the predicted hydrology indicates that the overtopping flow is the 2-year flow, this hydrology could also be suspect.**

2. **Comparison of the predicted hydrology to the channel capacity should also be done. Natural channels will typically contain between the 2-year flow and the 10-year flow within the channel banks.**

3. If streamflow data are available for an area, the hydrologic procedures can be calibrated to these data. The process would involve generating peak discharges and hydrographs for different input conditions (e.g., slope, area, antecedent soil moisture conditions) and comparing these results to the gaged data. Changes in the model would then be made to improve the estimated values as compared to the measured values.

Calibration Process (continued)

4. After changing the variables or parameters in the hydrologic procedure the results should be checked against another similar gaged stream or another portion of the streamflow data that were not used for calibration.

5. If some local agency has developed procedures or equations for an area based on streamflow data, general hydrologic procedures can be calibrated to these local procedures. In this way the general hydrologic procedures can be used for a greater range of conditions (e.g., land uses, size, slope).

6. The calibration process should only be undertaken by personnel highly qualified in hydrologic procedures and design.

7. *Should it be necessary to use unreasonable values for variables in order for the model to produce reasonable results, then the model should be considered suspect* and its use carefully considered (e.g., having to use terrain variables that are obviously dissimilar to the geographic area in order to calibrate to measured discharges or hydrographs).

7.9 Rational Method

Introduction
7.9.1

The rational method is recommended for estimating the design storm peak runoff for areas as large as **200 acres (80 hectares)**. This method, while first introduced in 1889, is still used in many engineering offices in the United States. Even though it has frequently come under criticism for its simplistic approach, no other drainage design method has received such widespread use.

Application
7.9.2

Some precautions should be considered when applying the rational method.

- The first step in applying the rational method is to obtain a good topographic map and define the boundaries of the drainage area in question. A field inspection of the area should also be made to determine if the natural drainage divides have been altered.

- In determining the runoff coefficient C value for the drainage area, thought should be given to future changes in land use that might occur during the service life of the proposed facility that could result in an inadequate drainage system. Also, the effects of upstream detention facilities may be taken into account.

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

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

Characteristics
7.9.3

Characteristics of the rational method which limit its use to **200 acres (80 hectares)** include:

1. The rate of runoff resulting from any rainfall intensity is a maximum when the rainfall intensity lasts as long 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 large drainage areas, the time of concentration can be so large that constant rainfall intensities for such long periods do not occur and shorter more intense rainfalls can produce larger peak flows. Further, in semi arid and arid regions, storm cells are relatively small with extreme intensity variations thus making the rational method inappropriate for watersheds greater than about **200 acres (80 hectares)**.

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

Frequencies of peak discharges depend on rainfall frequencies, antecedent moisture conditions in the watershed, and the response characteristics of the drainage system. For small and largely impervious areas, rainfall frequency is the dominant factor. For larger drainage basins, the response characteristics control. For drainage areas with few impervious surfaces (less urban development), antecedent moisture conditions usually govern, especially for rainfall events with a return period of 10 years or less.

3. The fraction of rainfall that becomes runoff (C) is independent of rainfall intensity or volume.

The assumption is reasonable for impervious areas, such as streets, rooftops, and parking lots. For pervious areas, the fraction of runoff varies with rainfall intensity and the accumulated volume of rainfall. Thus, the art necessary for application of the rational method involves the selection of a coefficient that is appropriate for the storm, soil, and land use conditions. Many guidelines and tables have been established, but seldom, if ever, have they been supported with empirical evidence.

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

Modern drainage practice often includes detention of urban storm runoff to reduce the peak rate of runoff downstream. With only the peak rate of runoff, the rational method severely limits the evaluation of design alternatives available in urban and in some instances, rural drainage design.

Equation 7.9.4

The rational formula estimates the peak rate of runoff at any location in a watershed as a function of the drainage area, runoff coefficient, and mean rainfall intensity for a duration equal to the time of concentration (the time required for water to flow from the most remote point of the basin to the location being analyzed). The rational formula is expressed as follows:

$$Q = CIA \qquad 7.1$$

where:

Q = maximum rate of runoff, cfs
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/hr
A = drainage area tributary to the design location, acres

Infrequent Storm 7.9.5

The coefficients given in Tables 7-4 through 7-6 are applicable for storms of 5-yr to 10-yr frequencies. Less frequent, higher intensity storms will require modification of the coefficient because infiltration and other losses have a proportionally smaller effect on runoff (Wright-McLaughlin 1969). The adjustment of the rational method for use with major storms can be made by multiplying the right side of the rational formula by a frequency factor C_f. The rational formula now becomes:

$$Q = Cc_f IA \qquad 7.2$$

C_f values are listed below. The product of C_f times C shall not exceed 1.0.

Table 7-2 Frequency Factors For Rational Formula

Recurrence Interval (years)	C_f
25	1.1
50	1.2
100	1.25

Procedures 7.9.6

The results of using the rational formula to estimate peak discharges is very sensitive to the parameters that are used. The designer must use good engineering judgment in estimating values that are used in the method. Following is a discussion of the different variables used in the rational method.

Time of Concentration 7.9.6.1

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. The duration of rainfall is then set equal to the time of concentration and is used to estimate the design average rainfall intensity (I). For a specific drainage basin, the time of concentration 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 time required for runoff to flow over the surface to the nearest inlet and is primarily a function of the length of overland flow, the slope of the drainage basin, and surface cover. Pipe or open channel flow time can be estimated from the hydraulic properties of the conduit or channel. An alternative way to estimate the overland flow time is to use Figure 7-1 to estimate overland flow velocity and divide the velocity into the overland travel distance.

For design conditions that do not involve complex drainage conditions, Figure 7-2 can be used to estimate inlet time. 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. The runoff coefficient (C) is determined by the procedure described in a subsequent section of this chapter.

To obtain the total time of concentration, the pipe or open channel flow time must be calculated and added to the inlet time. After first determining the average flow velocity in the pipe or channel, the travel time is obtained by dividing velocity into the pipe or channel length. Velocity can be estimated by using the Manning's equation.

Time of concentration is an important variable in most hydrologic methods. Several methods are available for estimating t_c. Appendix D (Travel Time Estimation) at the end of this chapter describes the method from the SCS Technical Release No. 55 (2nd Edition).

Common Errors 7.9.6.2

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

Second, when designing a drainage system, the overland flow path is not necessarily perpendicular to the contours shown on available mapping. Often the land will be graded and swales will intercept the natural contour and conduct the water to the streets which reduces the time of concentration. Care should be exercised in selecting overland flow paths in excess of 200 feet in urban areas and 400 feet in rural areas.

Rainfall Intensity
7.9.6.3

The rainfall intensity (I) is the average rainfall rate in/hr for a duration equal to the time of concentration for a selected return period. Once a particular return period has been selected for design and a time of concentration calculated for the drainage area, the rainfall intensity can be determined from **Rainfall-Intensity tables. An example of such a table is given in Figure 7-3; other tables for use by the MDT are given in Appendix B.**

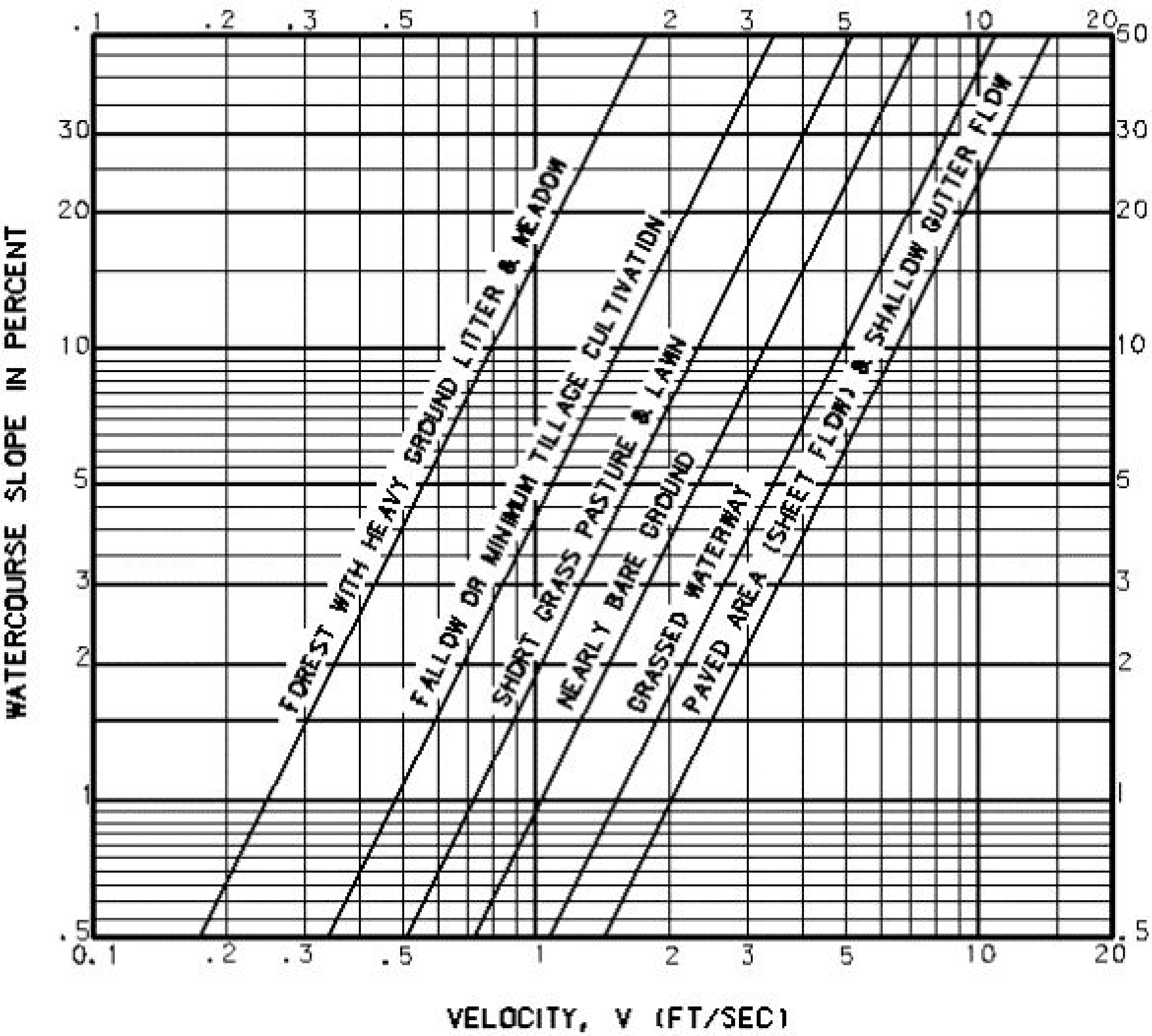


Figure 7-1 Velocities for Upland Method
of Estimating Time of Concentration
Source: HEC No. 19, FHWA

Distance = 385 feet
slope = 1.0%
C = 0.70
Overland Flow Time = 15.0 minutes

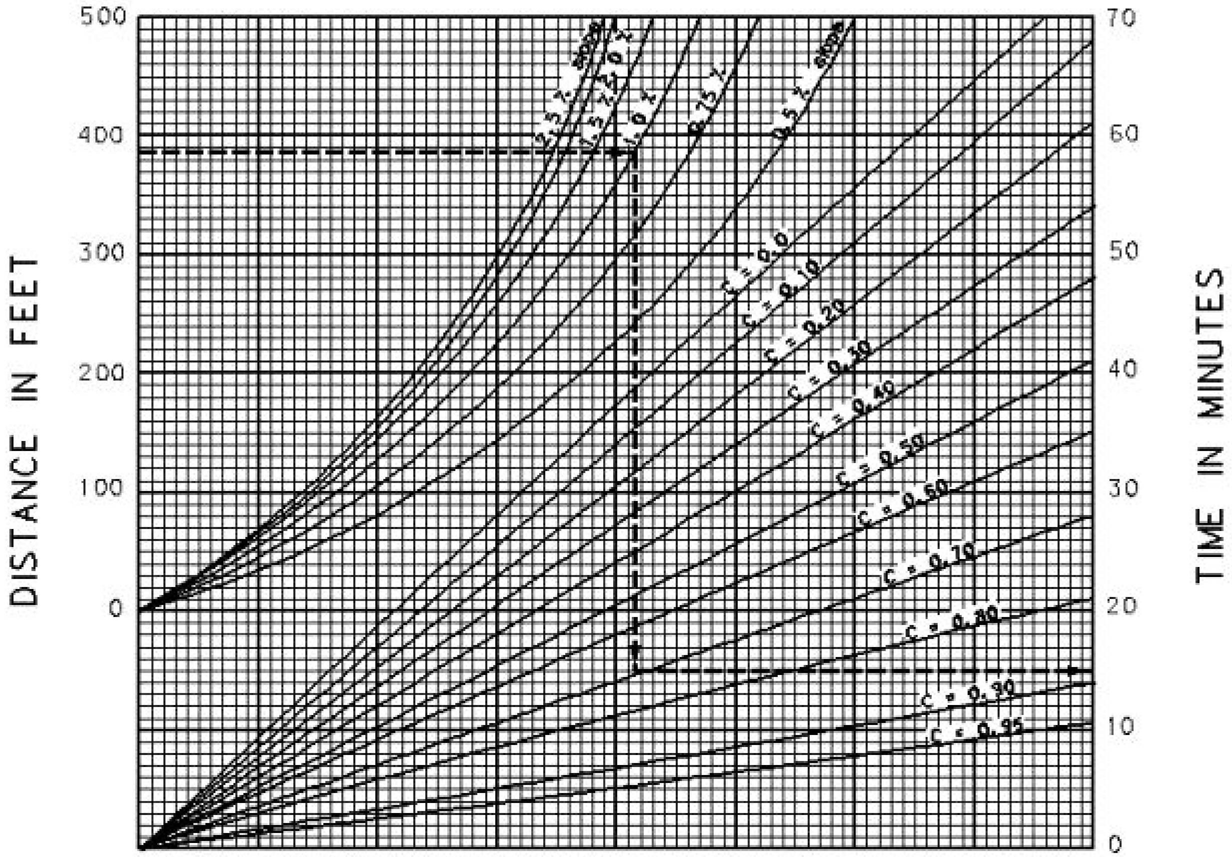


Figure 7-2 Overland Time Of Flow
Source: Airport Drainage, Federal Aviation Administration, 1965

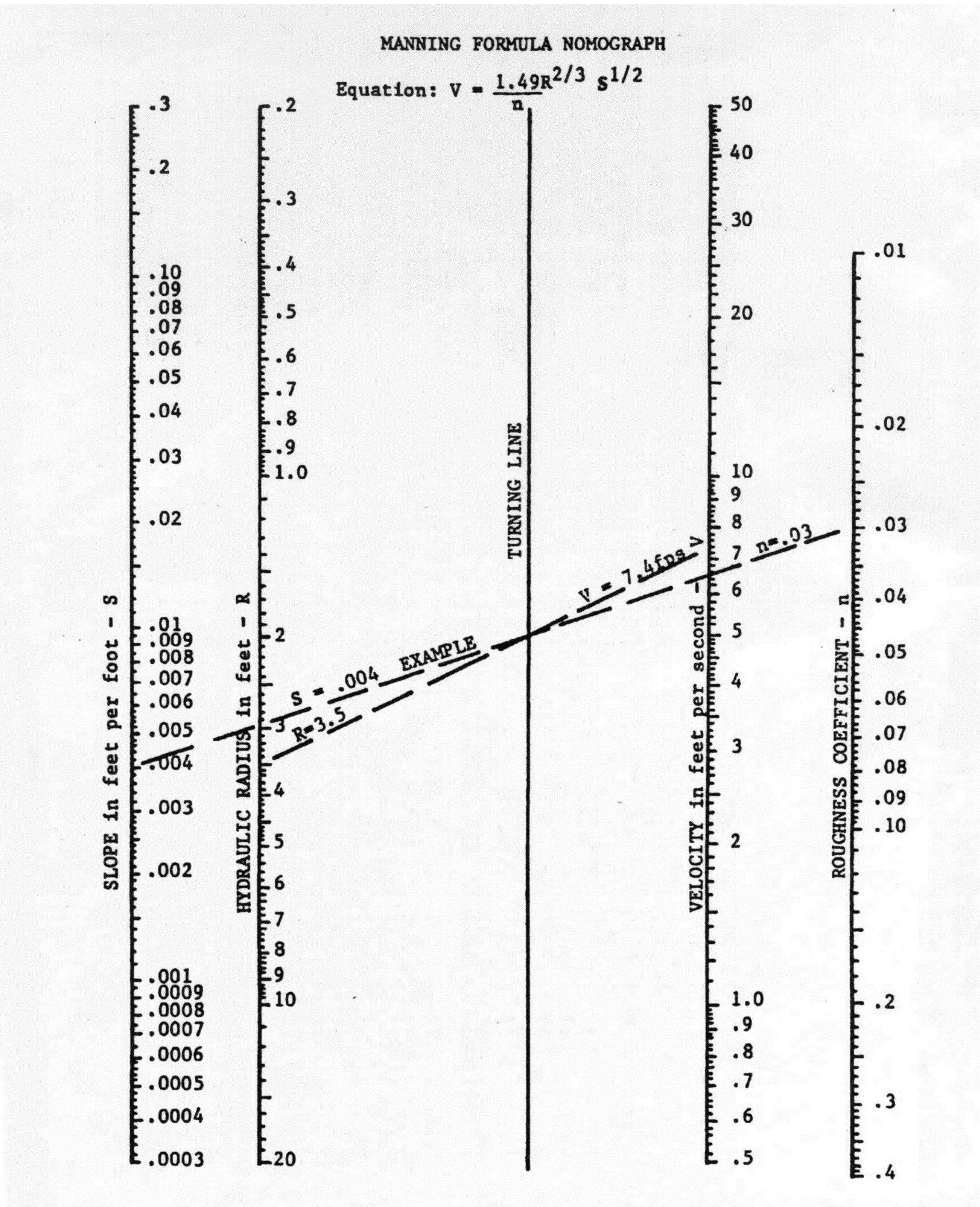


Figure 7–3 Manning Formula for Travel Time

Precipitation Intensity Values, in Inches/Hour

Return Period and Duration	Helena	Kalispell	Missoula
2 years			
5 minutes	2.60	2.06	2.09
10 minutes	1.87	1.60	1.49
15 minutes	1.46	1.32	1.19
30 minutes	0.90	0.82	0.70
60 minutes	0.52	0.48	0.41
5 years			
5 minutes	3.56	2.95	2.90
10 minutes	2.70	2.24	2.14
15 minutes	2.08	1.87	1.78
30 minutes	1.25	1.20	1.03
60 minutes	0.71	0.71	0.60
10 years			
5 minutes	4.21	3.55	3.46
10 minutes	3.25	2.66	2.57
15 minutes	2.50	2.24	2.17
30 minutes	1.49	1.45	1.25
60 minutes	0.84	0.86	0.72
25 years			
5 minutes	5.10	4.36	4.21
10 minutes	4.01	3.25	3.17
15 minutes	3.08	2.74	2.70
30 minutes	1.82	1.79	1.56
60 minutes	1.02	1.08	0.88
50 years			
5 minutes	5.80	4.99	4.80
10 minutes	4.60	3.70	3.63
15 minutes	3.52	3.14	3.11
30 minutes	2.08	2.06	1.79
60 minutes	1.16	1.24	1.01

Figure 7-3 Intensity Duration Table

Runoff Coefficient 7.9.6.4

The runoff coefficient (C) is the variable of the rational method least susceptible to precise determination and requires judgment and understanding on the part of the designer. While engineering judgment will always be required in the selection of runoff coefficients, a typical coefficient represents the integrated effects of many drainage basin parameters, the following discussion considers only the effects of soil groups, land use, and average land slope.

Three methods for determining the runoff coefficient are presented based on soil groups and land slope (Tables 7-3 and 7-4), land use (Table 7-5), and a composite coefficient for complex watersheds (Table 7-6).

Table 7-4 gives the recommended coefficient (C) of runoff for pervious surfaces by selected hydrologic soil groupings and slope ranges. From this table the C values for non-urban areas such as forest land, agricultural land, and open space can be determined. Soil properties influence the relationship between runoff and rainfall since soils have differing rates of infiltration. Infiltration is the movement of water through the soil surface into the soil. Based on infiltration rates, the Soil Conservation Service (SCS) has divided soils into four hydrologic soil groups as follows:

Group A — Soils having a low runoff potential due to high infiltration rates. These soils consist primarily of deep, well drained sands and gravels.

Group B — Soils having a moderately low runoff potential due to moderate infiltration rates. These soils consist primarily of moderately deep to deep, moderately well to well drained soils with moderately fine to moderately coarse textures.

Group C — Soils having a moderately high runoff potential due to slow infiltration rates. These soils consist primarily of soils in which a layer exists near the surface that impedes the downward movement of water or soils with moderately fine to fine texture.

Group D — Soils having a high runoff potential due to very slow infiltration rates. These soils consist primarily of clays with high swelling potential, soils with permanently high water tables, soils with a clay pan or clay layer at or near the surface, and shallow soils over nearly impervious parent material.

Runoff Coefficient (continued)

As an example, a list of soils for **Treasure County, Montana** and their hydrologic classification is presented in Table 7-3. **References for other soil surveys available in Montana are given in Appendix C at the end of this chapter.**

As the slope of the drainage basin increases, the selected C value should also increase. This is caused by the fact that 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.

It is often desirable to develop a composite runoff coefficient based on the percentage of different types of surface in the drainage area. Composites can be made with Tables 7-4 and 7-5. At a more detailed level composites can be made with Table 7-4 and the coefficients with respect to surface type given in Table 7-6. The composite procedure can be applied to an entire drainage area or to typical "sample" blocks as a guide to selection of reasonable values of the coefficient for an entire area.

Runoff Coefficient (continued)

Table 7-3
Hydrologic Soils Groups for Treasure County, Montana

Series Name	Hydrologic Group	Series Name	Hydrologic Group
Arvada	D	Laurel	D
Bainville	C	Lismas	D
Banks	A	Lohmiller	C
Beaverton	B	Marias	D
Beckton	D	McKenzie	D
Bew	C	McRae	B
Bowdoin	D	Midway	D
Briggsdale	C	Nihill	B
Cherry	C	Nunn	C
Cushman	C	Pierre	D
Dwyer	A	Promise	D
Edgar	B	Renohill	C
Fattig	C	Sage	D
Flasher	D	Shonkin	D
Fort Collins	B	Terry	C
Gilt Edge	D	Travessilla	D
Glendive	B	Treasure	B
Havre	B	Tullock	C
Hesper	B	Wanetta	B
Hoven	D	Wibaux	B
Hysham	D	Winnett	D

Source: Soil Survey, Treasure County, Montana, Series 1957, No. 22 and Soil Conservation Service Technical Release 55, "Urban Hydrology for Small Watersheds."

Table 7-4
Recommended Coefficient of Runoff for Pervious Surfaces
by Selected Hydrologic Soil Groupings and Slope Ranges

Slope	A	B	C	D
Flat (0–1%)	0.04–0.09	0.07–0.12	0.11–0.16	0.15–0.20
Average (2–6%)	0.09-0.14	0.12-0.17	0.16-0.21	0.20-0.25
Steep (Over 6%)	0.13-0.18	0.18-0.24	0.23-0.31	0.28-0.38

Source: Storm Drainage Design Manual, Erie and Niagara Counties Regional Planning Board

Runoff Coefficient (continued)

Table 7-5
Recommended Coefficient of Runoff Values
for Various Selected Land Uses

Description of Area		Runoff Coefficients
Business: Downtown areas		0.70-0.95
Neighborhood areas		0.50-0.70
Residential:	Single-family areas	0.30-0.50
	Multi units, detached	0.40-0.60
	Multi units, attached	0.60-0.75
	Suburban	0.25-0.40
Residential (1.2 acre lots or more)		0.30-0.45
Apartment dwelling areas		0.50-0.70
Industrial:	Light areas	0.50-0.80
	Heavy areas	0.60-0.90
Parks, cemeteries		0.10-0.25
Playgrounds		0.20-0.40
Railroad yard areas		0.20-0.40
Unimproved areas		0.10-0.30

Source: Hydrology, Federal Highway Administration, HEC No. 19, 1984

Table 7-6 Coefficients For Composite Runoff Analysis

Surface		Runoff Coefficients
Street:	Asphalt	0.70-0.95
	Concrete	0.80-0.95
Drives and walks		0.75-0.85
Roofs		0.75-0.95

Source: Hydrology, Federal Highway Administration, HEC No. 19, 1984

7.10 Example Problem – Rational Method

Following is an example problem which illustrates the application of the Rational Method to estimate peak discharges.

Preliminary estimates of the maximum rate of runoff are needed at the inlet to a culvert **near Missoula** for a 10-yr and 50-yr return period.

<u>Site Data</u>

From a topographic map and field survey, the area of the drainage basin upstream from the point in question is found to be 90 acres. In addition the following data were measured:

Length of overland flow = 150 ft
Average overland slope = 2.0%
Length of main basin channel = 2300 ft
Estimated Roughness coefficient (n) of channel = 0.090
Slope of channel - .018 ft/ft = 1.8%

<u>Land Use And Soil Data</u>

From existing land use maps, land use for the drainage basin was estimated to be:

Residential (single family)	80%
Undeveloped (2% slope)	20%

For the undeveloped area the soil group was determined from a SCS Map to be:

Group B	100%

From existing land use maps, the land use for the overland flow area at the head of the basin was estimated to be:

Undeveloped (Soil Group B, 2.5%)	100%

<u>Overland Flow</u>

A runoff coefficient (C) for the overland flow area is determined from Table 7-4 to be .12.

Time of Concentration

From Figure 7-2 with an overland flow length of 150 ft, slope of 2.0% and a C of .12, the inlet time is 18 min. Channel flow velocity is determined from the Manning's equation to be 3.5 ft/s (n = 0.090, R = 1.97 and S = .018). Therefore,

Flow Time = (2300 ft)/(3.5 ft/s)(60 s/min) = 10.95 min

and t_c = 18 + 10.95 = 28.95 min - **use 30 min**

Rainfall Intensity

From Figure 7-3, **for Missoula,** with duration equal to **30 min**,

I_{10} (10-yr return period) = **1.25** in/hr

I_{50} (**50**-yr return period) = **1.79** in/hr

Runoff Coefficient

A weighted runoff coefficient (C) for the total drainage area is determined in the following table by utilizing the values from Tables 7-4 and 7-5.

Land Use	(1) Percent of Total Land Area	(2) Runoff Coefficient	(3) Weighted Runoff Coefficient*
Residential (single family)	.80	.40	.32
Undeveloped (Soil Group B)	.20	.12	.02
Total Weighted Runoff Coefficient			.34

* Column 3 equals column 1 multiplied by column 2.

Example Problem – Rational Method (continued)

Peak Runoff

From the rational equation:

Q_{10} = CIA = .34 x **1.25** x 90 = **38** cfs

Q_{50} = C_fCIA = 1.20 x .34 x **1.79** x 90= **66** cfs

These are the estimates of peak runoff for a 10-yr and **50**-yr design storm for the given basin.

7.11 USGS Rural Regression Equations

Introduction
7.11.1

Regional regression equations are a commonly accepted method for estimating peak flows at ungaged sites or sites with insufficient data. Also, they have been shown to be accurate, reliable, and easy to use as well as providing consistent findings when applied by different hydraulic engineers (Newton and Herrin, 1982). The first method provided in this manual is termed the basin characteristics method. The second method is based on channel geometry characteristics (Wahl, 1983). Regression studies are statistical practices used to develop runoff equations. These equations are used to relate such things as the peak flow at a specified recurrence interval to the basin's physiographic, hydrologic and meteorological characteristics or, as an alternative, to the channel geometry characteristics at a site. As such it should be noted that the regression analyses are separate studies by the USGS and are not part of the analysis contained in this manual for devising a flood-frequency curve at an ungaged site, i.e., the regression analysis only provides the equation for application.

MDT
Application
7.11.2

The USGS basin characteristics regression equations shall be routinely used by MDT. At least one additional method shall also be used for comparison. Where there is stream gage data, the findings from a Log Pearson III method will generally govern should they vary significantly from those obtained using the rural regression equations, and provided there is at least **20** years of continuous stream gage record. Where there is less stream gage record, reasonable and prudent judgment along with consideration of the standard regression error shall be used in reaching a design decision.

Characteristics
7.11.3

<u>Basin Characteristics Methods</u> - The primary characteristics commonly include the drainage area above the point of interest as an independent variable. The remaining watershed characteristics are much more varied and depend upon the statistical significance of such variables as the **mean basin elevation index and the basin high elevation index. The only meteorological characteristic considered by the USGS as an independent variable is mean annual precipitation.** In the study by the USGS to devise the rural regression equations, the statistical significance of each prospective independent variable was determined and those that were statistically insignificant were eliminated from further consideration in the hydrologic analyses for projects.

Characteristics
(continued)

<u>Channel Geometry Method</u> - The primary characteristics include **active channel width and bankfull width. These geometric features are measured as described in USGS Water Resources Investigation Report 03–4308.** Additionally, channel geometry measurements must always be obtained beyond the influence of any man-made facilities as well as within a reach that has not incurred major damage from a recent flood; i.e., no extreme bank erosion ("gutted" channel) or "head cuts."

Mixed Population Floods 7.11.4

Mixed population floods are those derived from two (or more) causative factors; e.g., rainfall on a snow pack. To evaluate the effect of such occurrences requires reasonable and prudent judgment. Most of Montana can be subject to mixed population floods.

Hydrologic Regions 7.11.5

Regression analyses use stream gage data to define hydrologic regions. These are geographic regions having very similar flood frequency relationships and, as such, commonly display similar watershed, channel and meteorological characteristics; often termed hydrologically homogeneous geographic areas. Because of the distance between stream gages and sometimes due to the foregoing mixed flood population events, the regional boundaries cannot be considered as precise. **The current USGS report on Estimating Magnitudes and Frequency of Floods in Montana should be consulted for these boundaries.**

Problems related to hydrologic boundaries may occur in selecting the appropriate regression equation. First, the watershed of interest may lie partly within two or more hydrologic regions. A problem occurs when a watershed lies totally within a hydrologic region, but close to a hydrologic region boundary. In these instances care must be exercised in using regression equations. A field visit is recommended to first collect all available historical flood data as well as to compare the project's watershed characteristics with those of the abutting hydrologic regions. The channel geometry characteristics method may be useful in resolving problems in selecting the appropriate hydrologic region.

Typical Equations 7.11.6

Typical multiple log linear and log curvilinear equations utilized in regional flood studies commonly take the following formats. The format used by the USGS are the linear format.

$$Y_t = KX_1^{a1}X_2^{a2} \; ... \; X_n^{an} \quad \text{(linear form)}$$

Y_t is the dependent variable, X_1, X_2, ... , X_n are independent variables, K is a regression constant, and a_1, a_2, ..., a_n are regression coefficients. The dependent variable is normally the peak discharge for a given return period or some other property of the particular flood frequency. Independent variables for the foregoing two equation formats are selected to characterize either the watershed and its meteorologic conditions or, as an alternative, the channel geometry at a site. Note, the constants K, and a_1, a_2, ..., a_n are determined in the regression analysis and as such, are not determined as part of the hydrologic analysis for a site; i.e., they are "givens."

Procedure 7.11.7

The following procedure shall be used in applying the USGS regression equations.

Watersheds Having No Boundary Or Mixed Population Complexities - Collect the data from such things as maps, field visits, surveys, and aerial photos as well as the enclosed figures and apply the following equations. Hydrologic regions, regression constants, coefficients and standard errors are in the USGS Report. Compute a flood frequency curve to include the **2, 5, 10, 25, 50, 100 and 500 year events.**

Watersheds Transcending Hydrologic Region Boundaries - Where a watershed transcends hydrologic boundaries, the total discharge at a site for the foregoing recurrence intervals shall be determined **by the procedure described in the USGS Report.**

Transferring Gaged Data - Gaged data may be transferred to an ungaged site of interest provided such data are nearby, i.e., within the same hydrologic region, and there are no major tributaries or diversions between the gage and the site of interest. These procedures make use of the constants obtained in developing the regression equations. **These procedures are described in the USGS Report.**

7.12 Example Problem – Discharges From USGS Regression Equations

Examples are shown in the appropriate USGS Reports.

7.13 Regional Regression Analysis

Introduction 7.13.1

The Regional Regression Analysis is an analysis similar to that completed in developing the USGS Rural Regression Equations discussed previously. The primary differences are that a limited number of nearby gages (generally 4 to 10) are used, and the parameters are usually limited to those used in the USGS Rural Regression Equations for the Geographic Region that includes the basin being studied. For a unique basin, other variables can be considered.

MDT Application 7.13.2

The Regional Regression Analysis can be used for any drainage basin. The selected nearby gages should include drainage areas that are both smaller and larger than the basin being considered. For example, for a drainage basin of 10 square miles, the gages used should include areas between 2 and 50 square miles. Selection of the gages should also include consideration of basin orientation, shape and other physical or meteorological characteristics.

A Regional Regression Analysis or a Regional Frequency Analysis should be done for any drainage area that is greater than one square mile in area, unless there is a USGS gage on the stream that has a long period of record (at least 20 years).

Procedure 7.13.3

The gages selected for this analysis should be as close as possible to the study area, and should generally all be within the same geographic region. A log-Pearson III analysis of the gage data should be completed for each gage, and the results used for the Regional Regression. The MDT Regression Program uses the data supplied to generate an equation of the form:

$$Q = X^a Y^b C$$

where Q is the peak flow for the return period of interest, X and Y are basin parameters described in the USGS Report, C is a multiplication constant, and a and b are exponents. The values for a, b and C are developed by the MDT Regression Program.

The MDT Regression Program also calculates goodness of fit for each of the gages used, and the correlation coefficient and standard error for the given regression equation. The values from this analysis should be strongly considered for use when the standard error is lower than the standard error for the USGS Regression Equations.

7.14 Example Problem – Regional Regression Analysis

Estimate the 50-year and 100-year peak discharges for a 6.5 square mile drainage area, near Roundup, with a mean basin elevation of 3400 feet. The basin is in the East-Central Plains Region, as defined by USGS WRIR 92-4048.

Select nearby gages for the analysis. These will include:

Gage Number	Stream Name	Area	Years of Record
061257	Big Coulee nr Lavina	232	15
061263	Currant Cr nr Roundup	220	15
061264.7	Halfbreed Cr nr Klein	53.2	11
061271	S Willow Cr Trib nr Roundup	1.38	15
061272	Musselshell R Trib nr Musselshell	10.8	15
061275.2	Home Cr nr Sumatra	1.98	20
061275.7	Butts Coulee nr Melstone	6.71	30

The first five gages listed are discontinued crest-stage gages. The peak flow values for these five gages are therefore taken directly from the table in USGS WRIR 92-4048. The last two gages are active gages, and a log-Pearson III analysis needs to be completed. The peak flow values for all seven gages are listed below:

Gage Number	Area	Q50yr	Q100yr	Mean Elev.
061257	232	2430	3700	42307
061263	220	2270	3280	4250
061264.7	53.2	929	1520	3870
061271	1.38	733	992	3590
061272	10.8	518	677	3300
061275.2	1.98	364	482	3190
061275.2	1.98	364	482	3190
061275.7	6.71	492	619	3000

Two different regression computations were completed, one using only drainage area as an independent variable, and one using drainage area and mean basin elevation (in thousands of feet) as the two independent variables. A sample output for the 100-year flow with two independent variables is shown below:

REGRESSION
NEAR ROUNDUP
THE CORRELATION COEFFICIENT = .979
THE STANDARD ERROR = 20.507%
Q100 = AREA^ .122 E^ 4.165 x 4.077

Sta. Number	Calculated Q100	Actual Q100	G.F.	Area	E
1257	3216.3	3700	1.15	232	4.23
1263	3258.9	3280	1.006	220	4.25
1264.7	1855.8	1520	.819	53.2	3.87
1271	869.9	992	1.14	1.38	3.59
1272	786.9	677	.86	10.8	3.3
1275.2	555.7	482	.867	1.98	3.19
1275.7	499.3	619	1.24	6.71	3

The equations from the Regional Regression Analysis are as follows:

Q50 = AREA^ .311 x 352.492	(Standard error = 40.7%)
Q100 = AREA^ .341 x 452.521	(Standard error = 42.8%)
Q50 = AREA^ .117 E^ 3.68 x 5.495	(Standard error = 24.8%)
Q100 = AREA^ .122 E^ 4.165 x 4.077	(Standard error = 20.5%)

The standard error of the USGS Regression Equations for this Region are 59% for the 50-year peak and 61% for the 100-year peak. The Regional Regression Equations have a much lower standard error, and therefore should be strongly considered for design purposes.

For an area of 6.5 square miles, with E = 3.4, these equations yield the following peak flows:

Using area only:	Q50 = 631 cfs	Q100 = 857 cfs
Using area and mean elevation:	Q50 = 618 cfs	Q100 = 838 cfs

Using USGS Regression Equations:	**Q50 = 640 cfs**	**Q100 = 903 cfs**

The close agreement between the Regional Regression Analysis values and those from the USGS Regression Equations provides additional credibility to the analysis.

7.15 Regional Frequency Analysis

Introduction 7.15.1

The Regional Frequency Analysis is very similar to the Regional Regression Analysis except that a graphical line of best fit is developed.

MDT Application 7.15.2

As with the basin characteristics regression equations for peak discharge, a Regional Frequency Analysis shall be routinely used for comparison by MDT. Where there is stream gage data, the findings from a Log Pearson III analysis will generally govern should they vary significantly from those obtained using the Regional Frequency Analysis, provided there is at least 20 years of continuous stream gage record.

Procedure 7.15.3

A limited number of nearby gages (generally 4 to 10) are selected. These gages should be in the same USGS Geographic Region as the basin being analyzed. The selected gages should include drainage areas that are both smaller and larger than the basin being considered. For example, for a drainage basin of 10 square miles, the gages used should include areas between 2 and 50 square miles. Selection of the gages should also include consideration of basin orientation, shape and other physical or meteorological characteristics.

A plot of the logarithm of drainage area vs. logarithm of peak flow is made. The line of best fit is then drawn on this plot. The line of best fit can be drawn giving more consideration to points that represent gages with long periods of record. More consideration can also be given to points that represent drainage basins that are similar in orientation, shape, or other physical or meteorological characteristics.

The peak flow for the drainage being analyzed can be determined using the drainage area of the basin and the line of best fit from the Regional Frequency Analysis.

7.16 Example Problem – Regional Frequency Analysis

Estimate the 50-year and 100-year discharges for a 4.1 square mile drainage area, near Bozeman, with a mean basin elevation of 4,820 feet. The basin is in the Upper Yellowstone-Central Mountain Region, as defined by USGS WRIR 92-4048.

Select nearby gages for the analysis. These will include:

Gage Number	Stream Name	Area	Years of Record
060430	Taylor Creek nr Grayling	98.0	11
060432	Squaw Creek nr Gallatin Gateway	40.4	17
060433	Logger Creek nr Gallatin Gateway	2.48	29
060435	Gallatin River nr Gallatin Gateway	825	60
060465	Rocky Creek nr Bozeman	49.0	32
060467	Pitcher Creek nr Bozeman	2.33	16
060470	Bear Canyon nr Bozeman	17.0	18
060480	East Gallatin River at Bozeman	148	22
060485	Bridger Creek nr Bozeman	62.5	27
061935	Shields River at Clyde Park	543	41
061940	Brackett Creek nr Clyde Park	57.9	27

A map showing the location of these gages is in Figure 7-5. The return period flow values for these gages were taken directly from the table in USGS WRIR 92-4048. The 50-year and 100-year flows for all eleven gages are listed below:

Gage Number	Area	Q50yr	Q100yr	Mean Elev.
060430	98.0	1150	1200	8320
060432	40.4	723	830	7440
060433	2.48	62	74	7120
060435	825	9780	10,700	7960
060465	49.0	1180	1380	6110
060467	2.33	119	162	5680
060470	17.0	513	604	6690
060480	148	1800	2140	6210
060485	62.5	1090	1320	6540
061935	543	3950	4810	6090
061940	57.9	1110	1440	6140

The 50-year and 100-year flow values are then plotted as shown on Figure 7-6. A line of best fit is then drawn through the points. This line of best fit is then used to determine the flow values for the 4.1 square mile drainage area. The 50-year flow is 210 cfs, and the 100-year flow is 240 cfs.

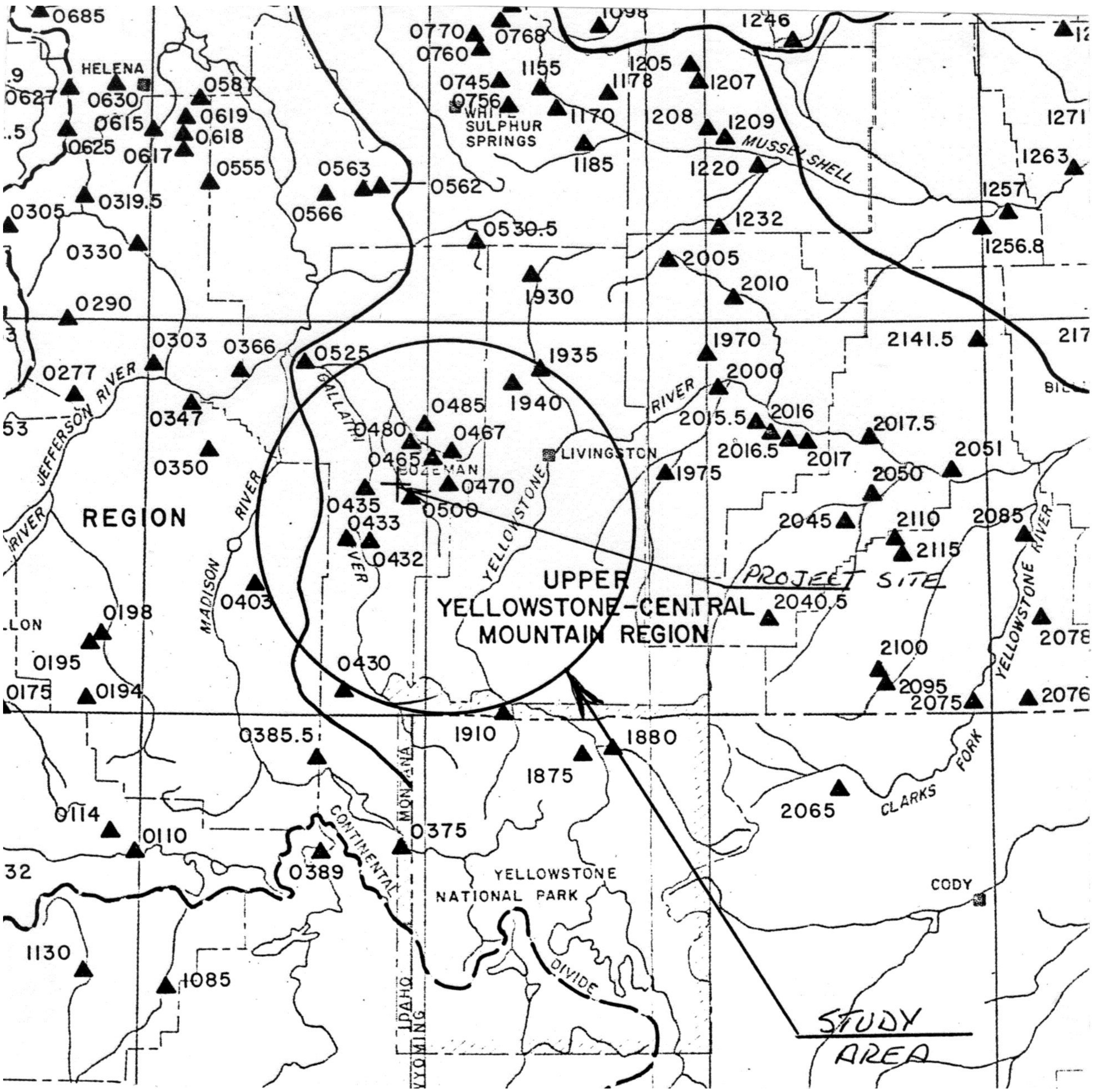


Figure 7-5 Regional Frequency Analysis
Location of Surrounding Gages

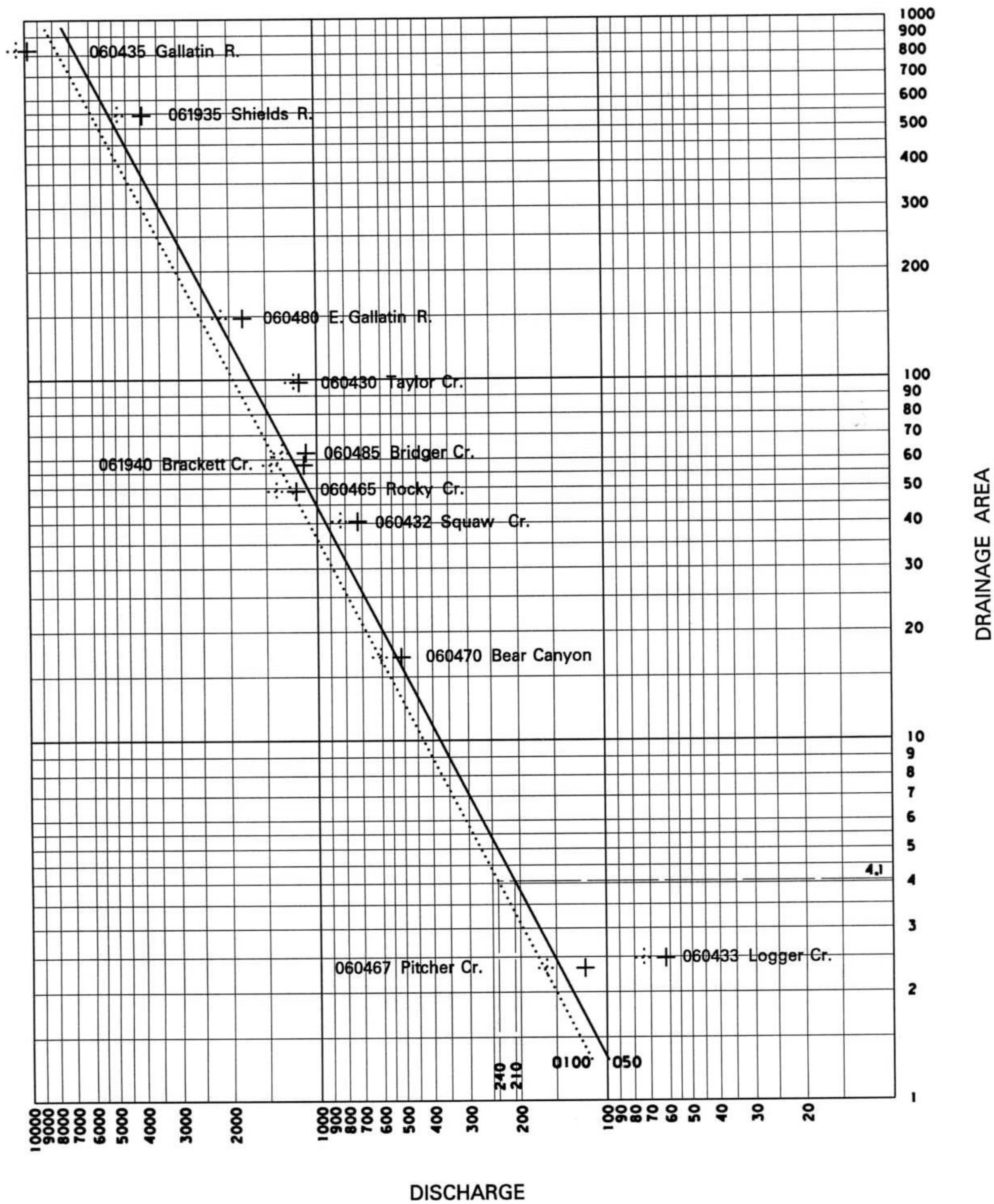


Figure 7-6 Regional Frequency Analysis Plot

7.17 Nallick Regression Equations

Introduction 7.17.1

A study by James Nallick, a graduate student at Montana State University, in cooperation with MDT, provided regression equations to be used for small drainage basins (less than 1 square mile (250 hectares)). These equations are appropriate for the plains areas of Montana east of the continental divide. The parameters determined to be of significance in the analysis were drainage area, average annual precipitation, and one-hour precipitation intensity.

MDT Application 7.17.2

The Nallick Regression Equations can be used for any small drainage basin on the plains of eastern Montana. The results of the equations should be compared to results from other methods, and historical data, to determine the most appropriate hydrology.

Equations 7.17.3

The Nallick Regression Equations are as follows:

$$Q_2 = 2{,}442\ A^{0.6986}\ P^{-1.9916}\ I_{25}^{2.3244}$$

$$Q_5 = 17{,}539\ A^{0.6357}\ P^{-2.3436}\ I_{25}^{2.0561}$$

$$Q_{10} = 51{,}964\ A^{0.6154}\ P^{-2.5528}\ I_{25}^{1.8896}$$

$$Q_{25} = 112{,}176\ A^{0.5937}\ P^{-2.6276}\ I_{25}^{1.6507}$$

$$Q_{50} = 148{,}559\ A^{0.5724}\ P^{-2.5958}\ I_{25}^{1.4537}$$

$$Q_{100} = 198{,}244\ A^{0.5569}\ P^{-2.5800}\ I_{25}^{1.2582}$$

where:

Q_i = peak flow rate (in cfs) for the i^{th} year return interval in years
A = area (square miles)
P = average annual precipitation (inches)
I_{25} = 25-year, maximum one-hour rainfall (inches)

Procedure 7.17.4

The average annual precipitation values (P) are taken from the SCS document <u>Average Annual Precipitation, Montana</u> (1977). The 25-year, maximum one-hour rainfall values (I_{25}) can be determined from a nearby rainfall gage (see Appendix B for a listing of gages and rainfall intensities).

7.18 Analysis of Stream Gage Data

Introduction
7.18.1

Many gaging stations exist throughout Montana where data can be obtained and used for hydrologic studies. If a project is located near one of these gages and the gaging record is of sufficient length in time, a frequency analysis **shall** be made according to the following discussion. The most important aspect of applicable station records is the series of annual peak discharges. It is possible to apply a frequency analysis to that data for the derivation of flood-frequency curves. Such curves can then be used in several different ways.

- If the subject site is at or very near the gaging site and on the same stream and watershed, the discharge for a specific frequency from the flood-frequency curve can be used directly.

- **If the drainage area at the subject site is between 0.5 and 1.5 times the drainage area at the gage and on the same stream and watershed, the discharge for a specific frequency from the flood-frequency curve can be determined by the gage transfer method described in the USGS Rural Regression Equations Report.**

- If the facility site is nearby or representative of a watershed with similar hydrologic characteristics, transposition of frequency discharges is possible.

- If the flood-frequency curve is from one of a group of several gaging stations comprising a hydrologic region, then regional regression relations may be derived **(see previous discussion on Regional Regression Analysis**).

Application
7.18.2

The MDT shall use the stream gage analysis findings for design when there is sufficient years of measured stream gage record unless **there is a FEMA Flood Insurance Study or the Log Pearson Type III analysis is questionable, due to a short period of record or a non-continuous record.** The preferred method for analyzing stream gage data is the Log Pearson Type III method. Outliers shall be placed into perspective using the procedure found in Water Resources Council Bulletin 17B.

Characteristics
7.18.3

The gaging station or stations most representative of a site that are in the vicinity or hydrologic region of a proposed structure should be selected. A record of 10 years **is considered the minimum for the analysis and 20 years is generally required for an analysis to be used without comparisons to other hydrologic methods. A plot of the results of the log-Pearson fit should always be completed, to determine how well the predicted values fit the data.** Stations with records which include controlled watershed runoff should be **used only below the controlling structure, on the gaged stream,** since the natural flood events may not have been recorded. The following steps may then be taken to develop a flood-frequency curve representing the data.

The analysis of gaged data permits an estimate of the peak discharge for the desired return period at a particular site. Experience has shown that statistical frequency distributions may be more representative of naturally occurring floods and can be reliable when used for prediction. Although several different distributions are used for frequency analysis, experience has shown the log-Person Type III distribution to be one of the most useful. The log-Pearson III distribution and the process of fitting it to a particular data sample are described in detail in Water Resources Council Bulletin 17B, "Guidelines for Determining Flood Flow Frequency", 1981. The following abbreviated procedure is taken from that publication.

The log-Pearson fit to the data can be plotted on standard log probability paper by computing several values of Q for different return periods. If the skew of the sample data happens to be equal to zero, the plot of the log-Pearson fit to the data will be a straight line. If the skew is negative the plot will be a curve with a downward concavity. If the skew is positive, the plot will be a curve with upward concavity.

In the course of preparing a frequency analysis for a particular watershed, the designer will undoubtedly encounter situations where further adjustments to the data are necessary. Special handling of outliers, historical data, incomplete data, and zero flow years is covered in detail in Bulletin 17B.

The computer system HYDRAIN provides the Log Pearson III flood frequency analysis. **MDT also has an in-house program that performs the log-Pearson Type III analysis.** Both analyses follow the WRC Bulletin 17B guidelines for the calculation of a log-Pearson frequency curve based on the mean, standard deviation, and skewness of the logarithms of the recorded annual peak flows.

Skew
7.18.4

There are two alternative methods for determining the value of the skew coefficient to be used in calculating the log-Pearson curve fit. The value of skew that is calculated directly from the gage data using the above formula is called the station skew. This value may not be a true representation of the actual skew of the data if the period of record is short or if there are extreme events in the period of record. **USGS Water Resources Investigations Report 92-4048** contains a map of generalized skew coefficients of the logarithms of annual maximum stream flows throughout **Montana**.

Often, the station skew and the generalized skew can be combined to provide a better estimate for a given sample of flood data. Bulletin 17B outlines a procedure for combining the station skew and the generalized skew to provide a weighted skew.

Special
Considerations
7.18.5

The following types of data records may require special considerations, as described in WRC Bulletin 17B:

- **Broken Record - Annual peaks for certain years may be missing because of conditions not related to flood magnitude, such as gage removal. In this case, the different record segments are analyzed as a continuous record with length equal to the sum of both records, unless there is some physical change in the watershed between segments which may make the total record nonhomogeneous.**

- **Incomplete Record - An incomplete record refers to a stream flow record in which some peak flows are missing because they were too low or too high to record, or the gage was out of operation for a short period because of flooding. Missing high and low data require different treatment.**

  When one or more high annual peaks during the period of systematic record have not been recorded, there is usually information available from which the peak discharge can be estimated. In most instances, the data collecting agency routinely provides such estimates. If not, and such an estimate is made as part of the flood frequency analysis, it should be documented.

  At some crest gage sites the bottom of the gage is not reached in some years. For this situation, use of a conditional probability adjustment is recommended.

Special Considerations (continued)

- **Zero Flood Years - Some streams in arid regions have no flow for the entire year. Thus, the annual flood series for these streams will have one or more zero flood values. This precludes the normal statistical analysis of the data using the recommended log-Pearson Type III distribution because the logarithm of zero is minus infinity. A conditional probability adjustment is recommended for determining frequency curves for records with zero flood years. This procedure should only be used when not over 25 percent of the total record consists of zero flows.**

- **Historic Flood Data - Information which indicates that any flood peaks which occurred before, during, or after the systematic record are maximums in an extended period of time should be used in frequency computations. The treatment of all historic flood data should be well documented.**

Transposition of Data 7.18.6

The transposition of design discharges from one basin to another basin with similar hydrologic characteristics is accomplished by multiplying the design discharge by the direct ratio of the respective drainage areas raised to the **power indicated in the USGS Rural Regression Equation Report for the appropriate geographic region.** Thus on streams where no gaging station is in existence, records of gaging stations in a nearby **(within 50 miles, or 80 kilometers)** hydrologically similar watershed may be used. The discharge for such an ungaged stream may be determined by the transposition of records using a similar procedure. Following is an example using an exponent of 0.7.

Watershed	Q_{25}	Area, sq.mi.
Gaged Watershed	62,000	737
Ungaged Watershed	–	450

Ungaged: $Q_{25} = 62{,}000\ (450/737)^{0.7} = 43{,}895$ cfs

7.19 Example Problem – Analysis of Stream Gage Data

Estimate the 10-year and 100-year flows for a bridge crossing of the Teton River in Teton County, southwest of Collins. The drainage area at the bridge crossing is 813 square miles.

There is a USGS gage on the Teton River near Dutton (gage number 0610800). The drainage area at the gage is 1307 square miles. The USGS Report 92-4048 lists a 10-year flow of 5440 cfs and a 100-year flow of 29,900 cfs. This includes a period of record of 33 years, through 1988. Discussions with the USGS in Helena indicated that one of the records was a zero flow, due to ice. This record should have been deleted from the analysis. The discussions also indicated that the 1964 flood flow of 71,300 cfs was treated as an historic flood and the largest flood in a 100-year period. Based on the information received from the USGS, along with additional flow records from 1989 through 1992, the gage data was re-analyzed.

The MDT in-house program for log-Pearson Type III analysis was used. The output for this gage is shown in Figure 7-7. The 10-year flow at the gage is 5374 cfs, and the 100-year flow is 28,112 cfs.

These flows then need to be transferred to the bridge site. The gage transfer equation in the USGS Report 92-4048 is:

$$Q_{tu} = Q_{tg} * (A_u/A_g)^a$$

where Q_{tu} = **flow at the ungaged site,**
Q_{tg} = **flow at the gaged site,**
A_u = **drainage area at the ungaged site,**
A_g = **drainage area at the gaged site,**
a = **exponent from USGS Report 92-4048, for desired return period (NW Foothills Region).**

The 10-year flow at the bridge is:

$$Q_{tu} = 5374 * (813/1307)^{0.47} = 4300 \text{ cfs}$$

The 100-year flow at the bridge is:

$$Q_{tu} = 28{,}112 * (813/1307)^{0.48} = 22{,}383 \text{ cfs}$$

N.W. FOOT HILLS REGION

LOG PEARSON TYPE III
BULLETIN 17B

TETON RIVER NEAR DUTTON

GAGE NUMBER 6108000

THERE ARE 35 YEARS OF RECORD

Discharge (CFS)	Rank	Plotting Position	Discharge (CFS)	Rank	Plotting Position	Discharge (CFS)	Rank	Plotting Position
16000	1	0.03	1690	13	0.36	552	25	0.69
8580	2	0.06	1570	14	0.39	410	26	0.72
7290	3	0.08	1310	15	0.42	379	27	0.75
4340	4	0.11	1200	16	0.44	376	28	0.78
3230	5	0.14	1100	17	0.47	304	29	0.81
2540	6	0.17	1040	18	0.50	276	30	0.83
2490	7	0.19	1040	19	0.53	272	31	0.86
2300	8	0.22	1000	20	0.56	258	32	0.89
2180	9	0.25	975	21	0.58	178	33	0.92
2000	10	0.28	875	22	0.61	154	34	0.94
1730	11	0.31	744	23	0.64	124	35	0.97
1700	12	0.33	623	24	0.67			

The generalized skew for this station is	.5
The low outlier threshold value is	49.67
No flows were found above the threshold value of	21558.41

Adjusted Probability for 0 Flows	Discharge Computed	Discharge Weighted Low Outliers	Adjusted Probability Outliers	Discharge Without (Years)	Return Period	Normal Probabilities	Final Discharge
0.5000	993	970	0.5000	970	2	0.5000	976
0.4296	1219	1191	0.4296	1191	2.33	0.4296	1210
0.1000	4646	4699	0.1000	4699	10	0.1000	5374
0.0400	8463	8849	0.0400	8849	25	0.0400	10961
0.0200	12592	13536	0.0200	13536	50	0.0200	17830
0.0100	18126	20057	0.0100	20057	100	0.0100	28112
0.0050	25442	29011	0.0050	29011	200	0.0050	43284
0.0020	38469	45917	0.0020	45917	500	0.0020	74445
SKEW	0.2143	0.3368		0.3368			0.5024
STD. DEV.	0.5009	0.5009		0.5009			0.5268
MEAN	3.0149	3.0149		3.0149			3.0333

The high outliers and historic records have been considered the largest flows in 100 years

Figure 7-7 MDT Log-Pearson Program Output

7.20 SCS Curve Number Method

Introduction
7.20.1

Techniques developed by the U. S. Soil Conservation Service for calculating rates of runoff require the same basic data as the Rational Method: drainage area, a runoff factor, time of concentration, and rainfall. The SCS approach, however, is more sophisticated in that it considers also 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. With the SCS method, the direct runoff can be calculated for any storm, either real or fabricated, by subtracting infiltration and other losses from the rainfall to obtain the precipitation excess. Details of the methodology can be found in the SCS National Engineering Handbook, Section 4.

Application
7.20.2

The SCS Curve Number Method (also referred to as the Soil-Cover-Complex Method) is applicable to drainage areas less than 3 square miles (8 square kilometers). It generates runoff from rainfall events only - snowmelt peaks are not considered.

Equations and Concepts
7.20.3

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

Drainage Area - The drainage area of a watershed is determined from topographic maps and field surveys. For large drainage areas it might be necessary to divide the area into sub-drainage 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.

Rainfall - The SCS method is based on a 24-hour storm event which has a **standard** time distribution. The Type II storm distribution is a "typical" time distribution which the SCS has prepared from rainfall records for Montana. Figure 7-7 shows this distribution. **Type I and Type IA storm distributions are also appropriate for parts of Montana. To select the appropriate storm distribution, it is necessary to determine the ratio of the 6-hour rainfall (P_6) to the 24-hour rainfall (P_{24}). The 6-hour and 24-hour rainfall values should be taken from NOAA Atlas 2 for the frequency of the design storm desired. The following table should be used to select the distribution:**

Type IA Distribution: $P_6/P_{24} < 0.518$
Type I Distribution: $P_6/P_{24} = 0.518$ to 0.639
Type II Distribution: $P_6/P_{24} = 0.640$ to 0.767

For a ratio greater than 0.767, this method should not be used.

Concepts (continued)

Rainfall-Runoff Equation - A relationship between accumulated rainfall and accumulated runoff was derived by SCS from experimental plots for numerous soils and vegetative cover conditions. Data for land-treatment measures, such as contouring and terracing, from experimental watersheds were included. The equation was developed mainly for small watersheds 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 SCS runoff equation is therefore a method of estimating direct runoff from 24-hour or 1-day storm rainfall. The equation is:

$$\mathbf{Q = (P - I_a)^2 / (P - I_a) + S} \qquad \mathbf{(7.20)}$$

Where:

Q = accumulated direct runoff, inches

P = accumulated rainfall (potential maximum runoff), inches

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

S = potential maximum retention, inches

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 SCS runoff equation is:

$$\mathbf{I_a = 0.2S} \qquad \mathbf{(7.21)}$$

Substituting 0.2S for I_a in equation 7.20, the SCS rainfall-runoff equation becomes:

$$\mathbf{Q = (P - 0.2S)^2 / (P + 0.8S)} \qquad \mathbf{(7.22)}$$

The value of S can be determined from the Curve Number (CN) using equation 7.23.

$$\mathbf{S = (1000/CN) - 10} \qquad \mathbf{(7.23)}$$

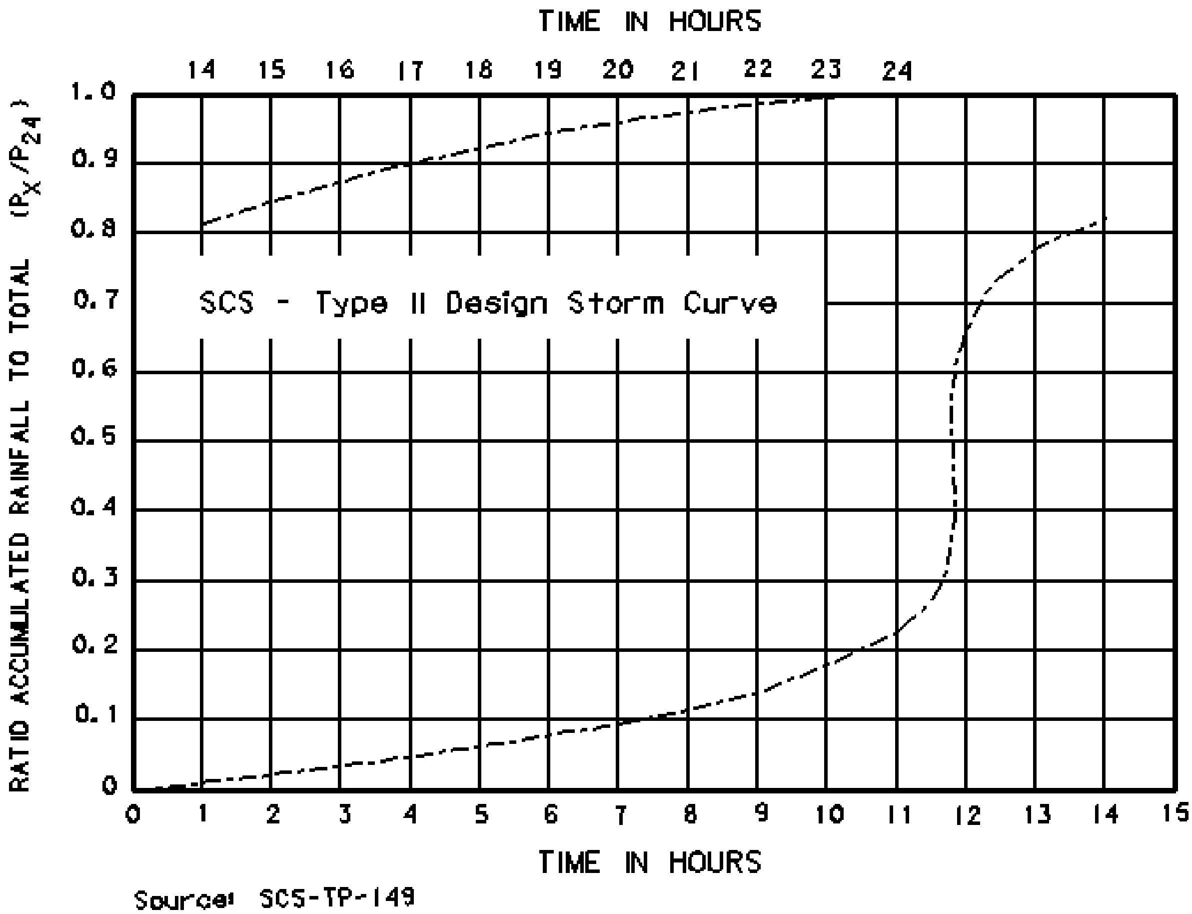


Figure 7-8 Type II Design Storm Curve

Procedures
7.20.4

Following is a discussion of procedures that are used in the **Curve Number Method** and recommended tables and figures.

Runoff Factor
7.20.4.1

In **the Curve Number Method**, runoff is often referred to as rainfall excess or effective rainfall - all defined as the amount by which rainfall exceeds the capability of the land to infiltrate or otherwise retain the rain water. 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 non-agricultural uses. Items such as type of vegetation, water surfaces, roads, roofs, etc. are all part of the land use. Land treatment applies mainly to agricultural land use, and it includes mechanical practices such as contouring or terracing and management practices such as rotation of crops.

The SCS uses a combination of soil conditions and land-use (ground cover) to assign a runoff factor to an area. These 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 is the runoff potential.

Soil properties influence the relationship between rainfall and runoff by affecting the rate of infiltration. The SCS has divided soils into four hydrologic soil groups based on infiltration rates (Groups A, B, C, and D). These groups were previously described for the Rational Formula.

Consideration should be given to the effects of urbanization on the natural hydrologic soil group. 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. In general, the greater the antecedent rainfall, the more direct runoff there is from a given storm. A five-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 a dry soil can change the soil moisture condition from dry to average to wet during the storm period.

Runoff Factor (continued

The following pages give a series of tables related to runoff factors. The first tables (Tables 7-8 - 7-11) gives curve numbers for various land uses. These tables are based on an average antecedent moisture condition i.e., soils that are neither very wet nor very dry when 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. Table 7-12 gives conversion factors to convert average curve numbers to wet and dry curve numbers. Table 7-13 gives the antecedent conditions for the three classifications.

Table 7-8 Runoff Curve Numbers[1]

Urban Areas

Cover Description		Curve Numbers for Hydrologic Soil Groups			
Cover Type and Hydrologic Condition	Average Percent Impervious Area[2]	A	B	C	D
Open space (lawns, parks, golf courses, cemeteries, etc.)[3]					
Poor condition (grass cover)	<50%	68	79	86	89
Fair condition (grass cover)	50% to 75%	49	69	79	84
Good condition (grass cover)	>75%	39	61	74	80
Impervious areas:					
Paved parking lots, roofs, driveways, etc. (excluding right-of-way)		98	98	98	98
Streets and roads:					
Paved; curbs and storm 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
Western desert urban areas:					
Natural desert landscaping (pervious areas only)		63	77	85	88

Table 7-8 Runoff Curve Numbers[1]

Urban Areas

Cover Description		Curve Numbers for Hydrologic Soil Groups			
Cover Type and Hydrologic Condition	Average Percent Impervious Area[2]	A	B	C	D
Artificial desert landscaping (impervious weed barrier, desert shrub with 1- to 2-inch sand or gravel mulch and basin borders)		96	96	96	96
Urban districts:					
Commercial and business	85	89	92	94	95
Industrial	72	81	88	91	93
Residential districts by average lot size:					
1/8 acre or less (town houses)	65	77	85	90	92
1/4 acre	38	61	75	83	87
1/3 acre	30	57	72	81	86
1/2 acre	25	54	70	80	85
1 acre	20	51	68	79	84
2 acres	12	46	65	77	82
Developing urban areas Newly graded areas (pervious areas only, no vegetation)		77	86	91	94

[1] Average runoff condition, and $I_a = 0.2S$

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

[3] CN's shown are equivalent to those of pasture. Composite CN's may be computed for other combinations of open space cover type.

Runoff Factor (continued)

Table 7-9 Cultivated Agricultural Land[1]

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

[1] Average runoff condition, and $I_a = 0.2S$.

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

Runoff Factor (continued)

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

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

Row crops are typically sugar beets and corn, whereas wheat, oats and barley would be classified as small grain.

Table 7-10 Other Agricultural Lands[1]

Cover Description		Curve Numbers for Hydrologic Soil Group			
Cover Type	Hydrologic Condition	A	B	C	D
Pasture, grassland, or range-continuous forage for grazing[2]	Poor	68	79	86	89
	Fair	49	69	79	84
	Good	39	61	74	80
Meadow--continuous grass, protected from grazing and generally mowed for hay	–-	30	58	71	78
Brush--brush-weed-grass mixture with brush the major element[3]	Poor	48	67	77	83
	Fair	35	56	70	77
	Good	[4]30	48	65	73
Woods--grass combination (orchard or tree farm)[5]	Poor	57	73	82	86
	Fair	43	65	76	82
	Good	32	58	72	79
Woods[6]	Poor	45	66	77	83
	Fair	36	60	73	79
	Good	[4]30	55	70	77
Farmsteads--buildings, lanes, driveways, and surrounding lots	–-	59	74	82	86

[1] Average runoff condition, and $I_a = 0.2S$

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

Runoff Factor (continued)

[3] Poor: < 50% ground cover
Fair: 50 to 75% ground cover
Good: > 75% ground cover

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

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

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

Runoff Factor (continued)

Table 7–11 Arid and Semiarid Rangelands[1]

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

[1] Average runoff condition, and $I_a = 0.2S$

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

[3] Curve numbers for Group A have been developed only for desert shrub

Runoff Factor (continued)

Table 7-12 Conversion From Average Antecedent Moisture Conditions To Dry And Wet Conditions

CN For Average Conditions	Corresponding CN's For Dry	Corresponding CN's For 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

Source: USDA Soil Conservation Service TP-149 (SCS-TP-149), "a Method for Estimating Volume and Rate of Runoff in Small Watersheds," revised April 1973.

Runoff Factor (continued)

Table 7-13 Rainfall Groups For Antecedent Soil Moisture Conditions During Growing And Dormant Seasons

Antecedent Condition	Conditions Description	Growing Season Five-Day Antecedent Rainfall	Dormant Season Five-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 1.4 inches	Less than 0.5 inches
Average	The average case for annual floods	1.4 to 2.1 inches	0.5 to 1.1 inches
Wet	When a heavy rainfall, or light rainfall and low temperatures, have occurred during the five days previous to a given storm	Over 2.1 inches	Over 1.1 inches

Source: Soil Conservation Service

Time of Concentration 7.20.4.2

The average slope within the watershed together with the overall length and retardance of overland flow are the major factors affecting the runoff rate through the watershed. In the SCS method, time of concentration (t_c) is defined to be the time required for water to travel from the most hydraulically distant point in a watershed to its outlet.

In small urban areas (less than **3 square miles or 8 square kilometers**), a curve number method can be used to estimate the time of concentration. The equation developed by SCS to estimate time of concentration is:

$$\mathbf{t_c = (\mathit{l}^{0.8}\ (S + 1)^{0.7}) / (1140\ Y^{0.5})} \qquad \textbf{(7.24)}$$

Where:

$\mathbf{t_c}$	**= time of concentration, hrs**
l	= length of mainstream to farthest divide, ft
Y	= average slope of watershed, %
S	= 1000/CN - 10
CN	= SCS curve number

The average watershed slope (Y) is the slope of the land and not the watercourse. It can be determined from soil survey data or topographic maps. The average watershed slope can be determined using the following relationship:

$$\mathbf{Y = (100*C*I)/A} \qquad \textbf{(7.25)}$$

Where:

Y	**= average watershed slope in percent**
A	**= drainage area in square feet**
I	**= contour interval in feet**
C	**= total contour length in feet.**

Flow length (*l*) is the longest flow path in the watershed from the watershed divide to the outlet. It is the total path water travels overland and in small channels on the way to the outlet.

I_a/P Ratio
7.20.4.3

The watershed CN is used to determine the initial abstraction (I_a) using the equation:

$$I_a = 0.2 * [(1000/CN)-10].$$

The I_a/P ratio is a parameter that indicates how much of the total rainfall is needed to satisfy the initial abstraction. The precipitation value to be used in this ratio is the 24-hour rainfall for the return period of interest. The larger the I_a/P ratio, the lower the unit peak discharge (q_u) for a given time of concentration. This indicates that if initial abstraction is a high portion of rainfall, the peak discharge will be lower. Thus, the I_a/P ratio is greater for smaller storms.

If the computed I_a/P ratio is outside the range of 0.1 to 0.5, then the limiting values should be used; i.e., use 0.1 if less than 0.1 and 0.5 if greater than 0.5.

Estimating Peak Discharge
7.20.4.4

The unit peak discharge (q_u) is obtained from Figures 7-8, 7-9 or 7-10, depending on the rainfall type. The time of concentration and I_a/P values are needed to obtain a value for q_u from the Figures. The peak discharge (q_p) is computed as the product of the unit peak discharge (q_u), the drainage area (A) and the runoff (Q):

$$q_p = q_u * A * Q.$$

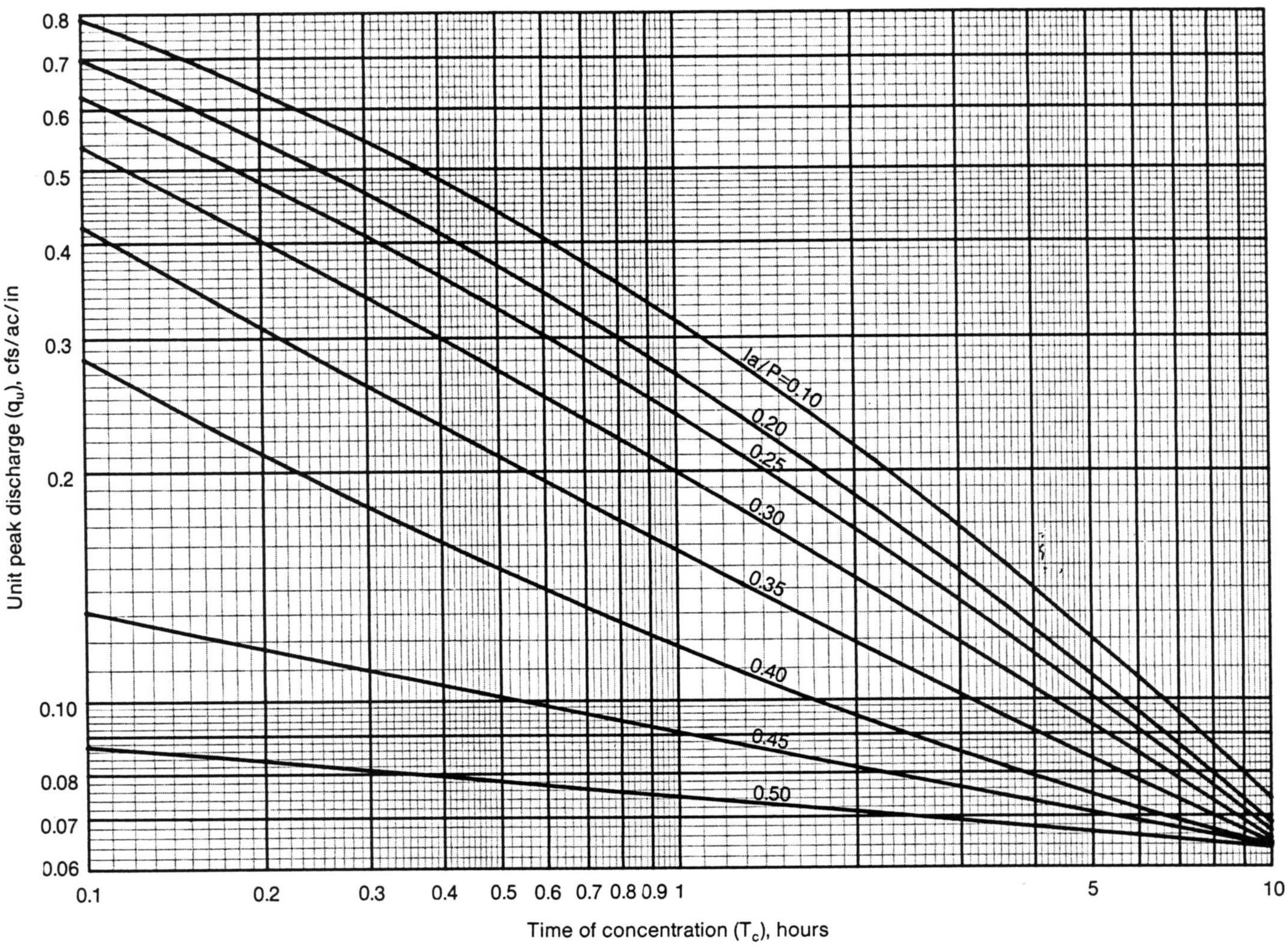


Figure 7-8 Unit Peak Discharge (q_u)
for SCS Type I Rainfall Distribution

Source: **Soil Conservation Service**

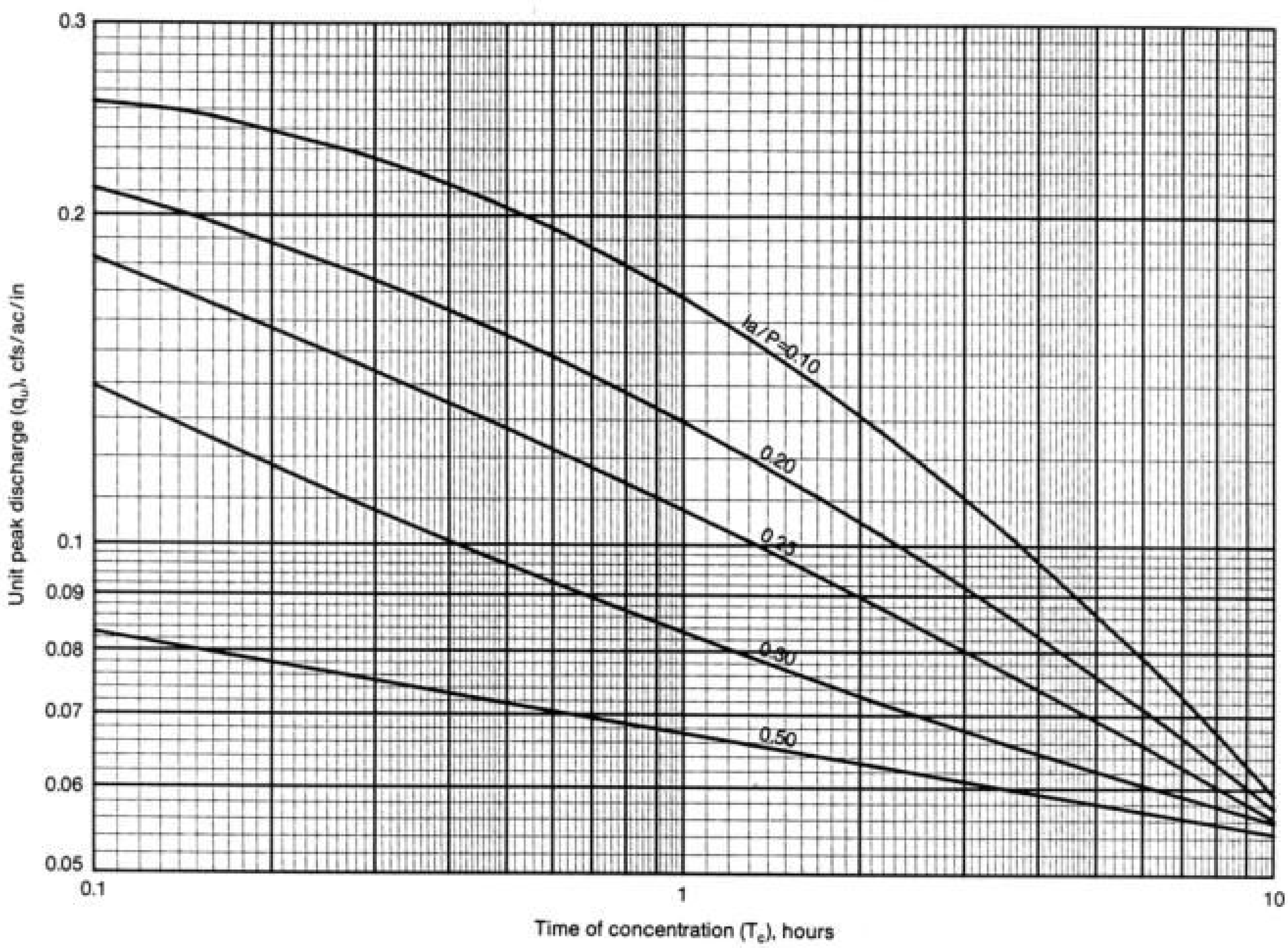


Figure 7-9 Unit Peak Discharge (q_u)
for SCS Type IA Rainfall Distribution

Source: **Soil Conservation Service**

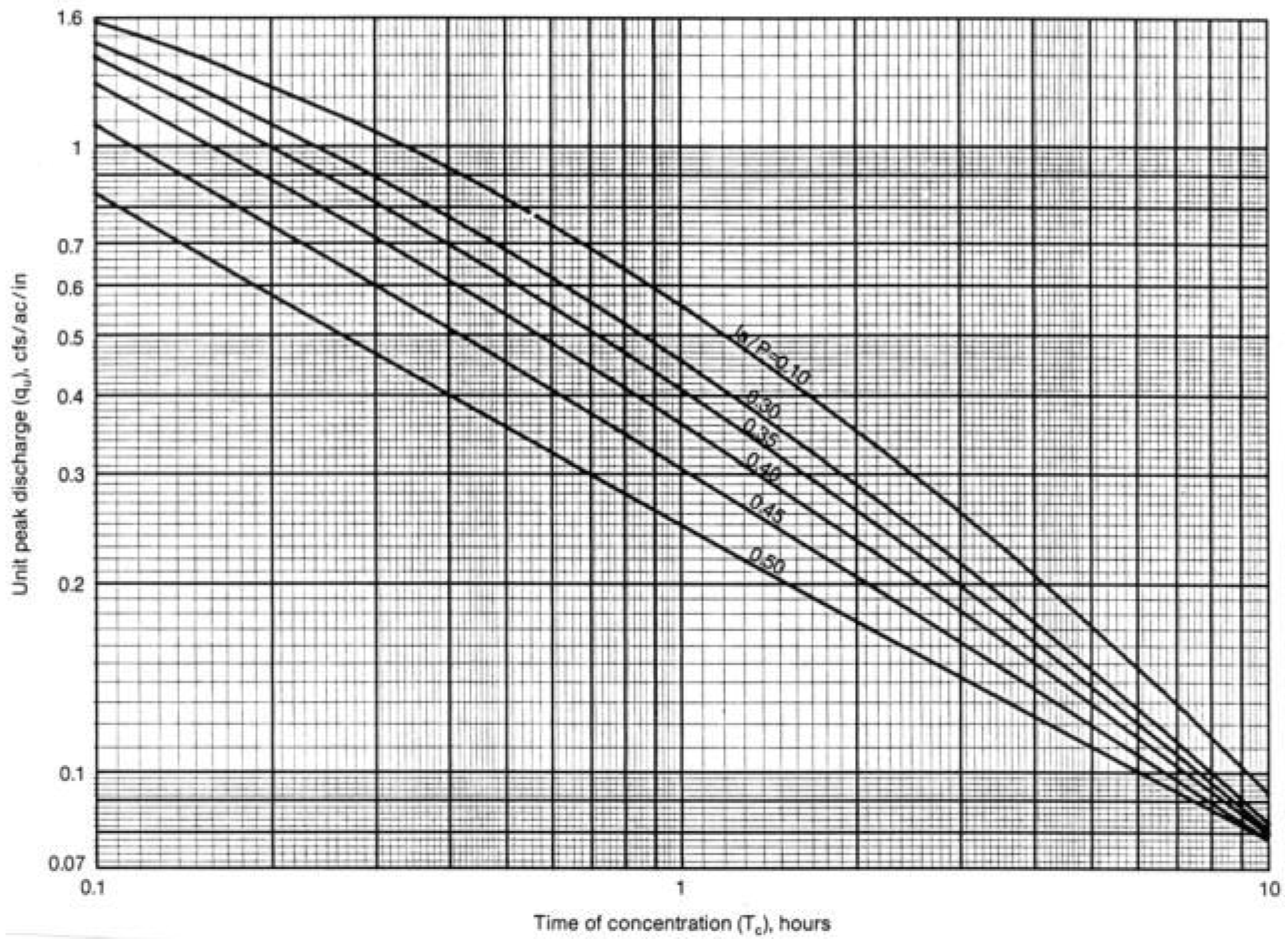


Figure 7-10 Unit Peak Discharge (q_u)
for SCS Type II Rainfall Distribution

Source: **Soil Conservation Service**

7.21 Example Problem – SCS Curve Number Method

Estimate the 50-year and 100-year peak discharges for a 0.49 square mile drainage area, about 11 miles north of Laurel. The following additional parameters are determined:

50-year, 6-hour Rainfall	**=**	**2.0 inches (from NOAA Atlas 2)**
50-year, 24-hour Rainfall	**=**	**3.0 inches (from NOAA Atlas 2)**
100-year, 6-hour Rainfall	**=**	**2.2 inches (from NOAA Atlas 2)**
100-year, 24-hour Rainfall	**=**	**3.4 inches (from NOAA Atlas 2)**

Soil Type is Bainville (from Soil Survey of Yellowstone County)
Hydrologic Soil Group is C (from SCS National Engineering Handbook, Section 4, Hydrology)
The drainage is primarily cultivated agricultural land, small grain, contoured, good condition (from aerial photographs)
Length of mainstream to farthest divide = 10,000 feet (from USGS topographic map - Two Pine School)

Contour Interval = 20 feet (from USGS topographic map)
Total contour length = 18,000 feet (from USGS topographic map)

The Runoff Curve Number (CN) is 81, from Table 7-9.
The potential maximum retention (S), in inches is:

$$S = (1000/CN) - 10$$

$$= (1000/81) - 10$$

$$= 2.35$$

The direct runoff is determined from the equation:

$$Q = (P - 0.2S)^2/(P + 0.8S)$$

$$= [3.0 - (0.2 * 2.35)]^2/[3.0 + (0.8 * 2.35)] \quad \text{(50-year)}$$

$$= 1.31 \text{ inches} \quad \text{(50-year)}$$

$$Q = [3.4 - (0.2 * 2.35)]^2/[3.4 + (0.8 * 2.35)] \quad \text{(100-year)}$$

$$= 1.63 \text{ inches} \quad \text{(100-year)}$$

The average watershed slope is given by the equation:

$$Y = (100*C*I)/A \tag{7.25}$$

Where:

Y = average watershed slope in percent
A = drainage area in square feet
I = contour interval in feet
C = total contour length in feet.

Therefore, the average watershed slope is:

$$Y = (100 * 18{,}000 * 20)/ (0.49 * 5280^2)$$

$$= 2.6\%$$

The time of concentration is determined from the equation:

$$t_c = (l^{0.8} (S + 1)^{0.7}) / (1140\ Y^{0.5}) \tag{7.24}$$

Where:

t_c = time of concentration, hrs
l = length of mainstream to farthest divide, ft
Y = average slope of watershed, %
S = 1000/CN - 10
CN = SCS curve number

Therefore the time of concentration is:

$$t_c = (10{,}000^{0.8} * (2.35+1)^{0.7})/(1140 * 2.6^{0.5})$$

$$= 2.01 \text{ hours}$$

The initial abstraction (I_a) is determined from the equation:

$$I_a = 0.2 * [(1000/CN)-10].$$

$$= 0.2 * [(1000/81)-10]$$

$$= 0.47 \text{ inch}$$

The I_a/P ratio for the 50-year and 100-year return periods are:

I_a/P = 0.47/3.0 = 0.16 (50 year)

I_a/P = 0.47/3.4 = 0.14 (100 year)

The P_6/P_{24} ratio for each return period is:

P_6/P_{24} = 2.0/3.0 = 0.667 (50-year)

P_6/P_{24} = 2.2/3.4 = 0.647 (100-year)

Both of these P_6/P_{24} ratios fall into the range of a Type II Distribution design storm. The unit peak discharge can then be determined from Figure 7-10.

q_u **= 0.33 cfs/acre/inch (50-year)**

q_u **= 0.34 cfs/acre/inch (100-year)**

The peak discharge is then computed from the equation:

$q_u = q_u * A * Q$

$q_u = 0.33 * (0.49 * 640) * 1.31$

= 136 cfs (50-year)

$q_u = 0.34 * (0.49 * 640) * 1.63$

= 174 cfs (100-year)

7.22 Methods for Estimating Low Flow

Low flow estimates are sometimes required to determine the design flows for fish passage. Numerous publications, most by the USGS, are available to assist in this determination. The following chronological list includes a description of the type of information available in each publication and a description of the area of the state that this information is applicable. The designer must determine the applicability of the method on a case by case basis.

- **Methods for Estimating Monthly Streamflow Characteristics at Ungaged Sites in Western Montana, USGS Water Supply Paper 2365, 1990. Provides three methods for estimating mean monthly discharge and 90, 70, 50, and 10 percent exceedance values for each month. Prepared in cooperation with BIA and CS&K Tribes. Applicable to Montana west of the Continental Divide.**

- **Estimates of Monthly Streamflow Characteristics at Selected Sites in the Upper Missouri River Basin, Montana, Base Period Water Years 1937-1986, USGS Water Resources Investigation Report 89-4082, 1989. Provides three methods for estimating mean monthly and 90, 80, 50 and 20 percent exceedance values for each month. Prepared in cooperation with Montana Department of Fish, Wildlife and Parks. Applicable to the Missouri River basin above Fort Peck Dam.**

- **Estimates of Monthly Streamflow Characteristics for Selected Sites in the Musselshell River Basin, Montana, Base Period Water Years 1937-86, USGS Water Resources Investigation Report 89-4165, 1989. Provides four methods for estimating mean monthly values. Prepared in cooperation with Lower Musselshell Conservation District and Montana Department of Natural Resources and Conservation.**

- **A Method for Estimating Mean and Low Flows of Streams in National Forests of Montana, USGS Water Resources Investigations Report 85-4071, 1985. Provides three methods for estimating mean annual discharge, and one method for estimating 95 percent and 80 percent annual exceedance values. Prepared in cooperation with Montana Reserved Water Rights Compact Commission and U.S. Department of Agriculture, Forest Service. Applicable to national forest lands west of longitude 109°.**

- **Streamflow Characteristics of the Yellowstone River Basin, Montana, Through September 1982, USGS Water Resources Investigations Report 84-4063, 1984. Provides gaging station data for annual low flow and annual high flow periods. No estimating techniques presented for ungaged streams. Prepared in cooperation with the U.S. Bureau of Land Management. Applicable to gaging stations in the Yellowstone River basin.**

- **A Method for Estimating Mean Annual Runoff of Ungaged Streams Based on Basin Characteristics in Central and Eastern Montana, USGS Water Resources Investigations Report 84-4143, 1984. Provides three methods for estimating mean annual flow. Prepared in cooperation with the U.S. Bureau of Land Management. Applicable to central and eastern Montana, approximately east of 110 longitude.**

- **Mean Annual Runoff and Peak Flow Estimates Based on Channel Geometry of Streams in Northeastern and Western Montana, USGS Water Resources Investigations Report 83-4046, 1983. Provides two methods for estimating mean annual flow and peak discharges. Prepared in cooperation with the U.S. Bureau of Land Management, U.S. Forest Service and Montana Department of Natural Resources and Conservation. Applicable to western and northeastern Montana.**

- **Mean Annual Runoff and Peak Flow Estimates Based on Channel Geometry of Streams in Southeastern Montana, USGS Water Resources Investigations Report 82-4092, 1983. Provides two methods for estimating mean annual flow and peak discharges. Prepared in cooperation with the U.S. Bureau of Land Management. Applicable to southeastern Montana.**

- **A Procedure for Estimating Flow-Duration Curves for Ungaged Mountainous and High Plains Streams in Montana, by A.B. Cunningham and D.A. Peterson, Department of Civil Engineering, Montana State University, June 1983. Provides one method for estimating flow-durations curves for ungaged, continuously flowing streams (not applicable to ephemeral streams). Prepared for The U.S. Office of Water Research and Technology, Montana Department of Natural Resources and Conservation, Montana Department of Fish, Wildlife and Parks, and Foundation for Montana Trout. Applicable to all portions of the state, on perennial streams.**

- **<u>Streamflow Characteristics of the Upper Columbia River Basin, Montana, Through September 1979</u>, USGS Water Resources Investigations 81-82, 1982. Provides gaging station data for annual low flow and annual high flow periods. No estimating techniques presented for ungaged streams. Prepared in cooperation with the Montana Department of Natural Resources and Conservation. Applicable to gaging stations in the Columbia River drainage basin in Montana.**

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U.S. Geological Survey, Open-File Report 74-38, Floods of January 15-17, 1974 in Northwestern Montana.

U.S. Geological Survey, Open-File Report 76-424, Floods of May through July, 1975 along the Continental Divide in Montana.

U.S. Geological Survey, Open-File Report 78-429, Floods of June 4 and 12, 1976 at Culbertson, Montana.

U.S. Geological Survey, Open-File Report 78-985, Data for Floods of May 1978 in Northeastern Wyoming and Southeastern Montana.

U.S. Geological Survey, Water Resources Investigations 82-33, Floods of May 1981 in West-Central Montana.

PROCEDURE MEMORANDUM NO. 11A **Date: November 5, 1984**
ECONOMIC ASSESSMENTS/ANALYSES **Updated: September 1995**

Introduction A.1

MDT Guidelines for selecting design flood frequencies are contained in Procedure Memorandum No. 11A, issued November 5, 1984. This Procedure Memorandum is repeated here, with slight modifications to reflect current requirements.

The design flood is defined in the Federal Aid Policy Guide, 23 CFR 650A. A preliminary or trial design flood shall be established using the following procedure and adjusted as necessary upon completion of the final design. For simplicity these flood values will merely be referred to as the design flood.

The design flood will be established based upon consideration of three major factors. These are 1) ADT; 2) detour length; and 3) functional classification of the route. However, when the route is used by emergency vehicles or as an evacuation route a minimum design flood will be established as provided below.

Definitions A.2

ADT: Average Daily Traffic for the design year of the project.

Route Classification: Routes will be considered only as arterial, major collector, or minor collectors in the order listed below:

1 - Arterial: Interstate and Primary Highways
2 - Major collectors: Secondary highways and certain urban routes.
3 - Minor collectors: Local or county roads and remaining urban routes.

Detour length: Detour length shall be measured between the nearest logical junction or termini on each side of a proposed crossing along a route of an equal or higher classification level minus the distance between the termini for the design route. Nearest logical termini are frequently two towns or populations centers.

Procedure A.3

The minimum design flood for preliminary design of urban, arterial and collector route stream crossings shall be determined based on Table A-1.

Table A-1 Design Flood Selection Guidelines

	<10 yr	10 yr	25 yr	50 yr
ADT	<50	50–399	400–3000	>3000
ADT x Miles of Detour	<1000	1000–3999	4000–15,000	>15,000

PROCEDURE MEMORANDUM NO. 11A **Date: November 5, 1984**
ECONOMIC ASSESSMENTS/ANALYSES **Updated: September 1995**

The design flood may be the larger of the values obtained by using ADT or by using ADT times miles of detour. A variance from the return period in Table A-1 may be considered where costs, flood duration, and road networks indicate that a higher or lower level of service is justifiable.

The minimum design flood for all interstate highway crossings shall be the 50 year flood (2% chance flood).

The minimum design flood for routes specifically designated as community evacuation routes or used daily by emergency vehicles should be the 25 year flood (4% chance flood). For crossings where a new bridge (or other structures of similar cost) will be designed, the minimum design flood should be the 10-year flood (10% chance flood).

Facilities on rural highways such as underpasses, depressed roadways, etc. where no overflow relief is available should be designed for the same return period as the remainder of the roadway.

PROCEDURE MEMORANDUM NO. 11 — Date: November 5, 1984
ECONOMIC ASSESSMENTS/ANALYSES — Updated: September 1995

GENERAL
The design of hydraulic structures for highway crossings of rivers and streams includes giving consideration to factors such as natural floodplain values, estimated construction costs, design costs and costs attributable to the facility over its lifetime as well as the hydraulic (stage-discharge/backwater) analysis. Many potentially serious design problems can be eliminated or minimized by the hydraulics engineer through the input and documentation provided during the Location and Environmental phases (refer to Procedure Memorandum No. 3, in the Appendix of Chapter 5).

Because of the many factors which can become involved in the hydrologic and hydraulic designs for highway crossings, the engineer must be able to exercise considerable engineering judgment in arriving at the final design. For this reason a case approach shall be used for hydraulic designs rather than a plan (i.e., statewide standards) approach so the judgment exercised by the engineer is not limited. Additional discussions on this subject can be found in the Hydraulic Analyses for Location and Design of Bridges, Volume VII, Highway Drainage Guidelines, AASHTO, 1992, and should be reviewed by the hydraulic engineer periodically.

The guidelines set forth in this procedure establish a systematic method for evaluating or screening major factors in the design of highway crossings and is referred to as an economic assessment (risk assessment). When this assessment or screening process indicates that a more detailed evaluation is warranted, an economic analysis (risk analysis) shall be performed. Guidelines for performing these economic analyses are contained in the FHWA's Hydraulic Circular No. 17 (HEC 17), "The Design of Encroachments on Floodplains Using Risk Analysis," dated April 1981.

PROCEDURE
The following steps outline the major factors which are to be considered to complete the design/assessment for drainage structures.

1. **Determine Design Flood Frequency: The design flood frequency is the basis for the preliminary or trial design. The subsequent assessment of factors will be based upon this trial design. Guidelines for selecting the design flood frequency are contained in Procedure Memorandum No. 11A (in Appendix A of Chapter 7).**

2. **Determine waterway opening (bridge or culvert) necessary to satisfy site constraints and to accommodate the trial design flood while satisfying the following criteria:**
   a) **No roadway overtopping**
   b) **No backwater damage to adjacent property**
   c) **Headwater depths for culverts will not generally exceed the values listed in Table A-2. For arch pipes, the allowable headwater shall be based on the rise (R) of the pipe, and the pipe size category will be the equivalent round pipe size. The ratios in Table A-2 may be exceeded based on sound engineering judgment, and with the approval of the Hydraulics Section.**

TABLE A-2
Maximum Allowable Headwater

Pipe Size	HW @ design flow	HW @ 100-year flow
≤ 42"	< 3.0 D or 3.0 R	< 4.0 D or 4.0 R
48"-108"	< 1.5 D or 1.5 R	< D + 5' or R + 5'
≥ 120"	< D + 2' or R + 2'	< D + 4' or R + 4'

3. **Assess the trial structure under higher flows following the guidelines in Procedure Memorandum No. 11B (in Appendix A of Chapter 7).**

4. **Complete file documentation using FORM HYD 4, Parts 1 and 2, or a narrative Hydraulics Report and FORM HYD 4, Part 2, in accordance with Federal Aid Policy Guide, 23 CFR 650A and applicable procedures.**

APPLICABILITY

The risk assessment procedure (FORM HYD 4, Part 2) shall be applicable as listed below. At the designer's discretion, this assessment procedure may be used for minor drainage crossings not listed below.

1. **All drainage crossings in urban or suburban areas requiring culverts larger than 24-inches in diameter or equivalent.**

2. **All drainage crossings in rural areas where there is potential significant backwater damage, or where there are potential significant traffic related losses caused by overtopping at the 100-year flow at a depth of 6 inches or greater.**

3. **All bridge crossings.**

PROCEDURE MEMORANDUM NO. 11B **Date: November 5, 1984**
HYD FORM 4 **Updated: September 1995**

GENERAL

HYD FORM 4 (DRAINAGE CROSSING HYDRAULIC REPORT) shall be utilized for the formal file documentation for all crossings requiring evaluation per Procedure Memorandum No. 11. This form should also be utilized for any other crossings where the designer feels that site conditions warrant added evaluation or documentation. The form is generally self explanatory; however, the following are provided for emphasis, clarification or information.

1. **This form will be a working document subject to changes necessary to reflect overall project development. The use of notes on the form to reflect such changes and attachments are encouraged.**

2. **The "greatest flood" is defined as the 500-year flood for bridge crossings and the 200-year flood for other crossings, based on normal state-of-the-art capabilities.**

3. **Because of the high proportion of crossing sites where site constraints are a major consideration in establishing the minimum waterway opening or type of structure, these evaluations have always been an integral part of the preliminary hydraulic analysis. These will continue to be evaluated prior to performing the economic assessment on the trial structure. In some cases such as fisheries and delineated floodplains (also see Procedure Memorandum No. 12), added costs or cost effectiveness must be determined before using the constraint as absolute.**

4. **In addition to the specific information listed for item C.6. in Part 1, this item should mention special design features (improved inlets, guide banks, etc.) or design options which have not been finalized or included in the cost estimate.**

5. **Table A-3 is provided for use in item A.2.c) in Part 2.**

6. **LTEC is defined as Least Total Expected Cost to the public. If any of the yes LTEC answers under Section D are checked, then further analysis of this item using the LTEC process as presented in the FHWA's HEC 17 is necessary or further justification why it is not required.**

PROCEDURE MEMORANDUM NO. 11B **Date: November 5, 1984**
HYD FORM 4 **Updated: September 1995**

TABLE A–3
TRAFFIC RISK

Assumptions:

Traffic Make-up:	**cars**	**70%**
	small trucks	**20%**
	semis	**10%**
Running Costs:	**cars**	**$0.30/mile**
	small trucks	**$0.65/mile**
	semis	**$1.00/mile**

Value of Lost Time: **$14/hr/occupant**
1.25 occupants/vehicle

Average Detour Time: **2 days**

Detour Speed: **50 mph**

From the above assumptions, the traffic risk can be found from the following equation:

Traffic Risk = 1.58 x ADT x Length of Detour (in miles)

To find Annual Traffic Risk, multiply Traffic Risk by the probability of overtopping.

Example:

Given: **ADT = 1000 vehicles/day**
Detour = 30 miles
Overtopping = 50 year frequency

Solution: **Traffic Risk = 1.58 x 1000 x 30 = $47,400**
Probability of Overtopping = 1/50 = 0.02
Annual Traffic Risk = $47,400 x 0.02 = $948

Data Sources: IRS allowable rate for passenger cars; average MDT truck rental rate schedule for small trucks; FHWA Motor Carriers Division for semis; Wyoming Water Research Center (December 1993, for Wyoming DOT) for lost time cost.

DRAINAGE CROSSING HYDRAULIC REPORT

GENERAL DATA

Project Name ____________________ No. ____________________

Stream Name ____________________ Sta. ____________________

Designer ____________________ Date ____________________

Current ADT ____________________ Projected ADT (20__) ____________________

Projected ADT x Detour Length __________

Is this route used regularly by emergency vehicles?____________________

A. SITE DATA

1. General Information (Include in file/permanent records)
   a) Location Map - Indicate crossing (also attached to report)
   b) Completed FORM HYD 1
   c) Photographs

2. Existing Bridge/Culvert
   a) Size structure ____________________
   b) Type ____________________
   c) Number & length of spans ____________________
   d) Does overtopping of road, basin divide or relief structure(s) occur?__________ If yes, explain ____________________
      i - Elevation ____________________
      ii - Magnitude ____________________
      iii - Frequency ____________________
      iv - Approximate backwater at overtopping flood ____________________

3. Discuss Factors Affecting Water Stages
   High water from other streams, reservoirs, lakes, divided flow, etc.
   ____
   ____

4. River Type
   a) Meandering or braided ____________________
   b) Brief discussion of aggradation/degradation, bank erosion and stream stability in vicinity ____________________
   ____
   ____

5. Channel Slope __________ ft/ft. How was slope determined?
   ____

6. Are design considerations for fish passage necessary?__________

B. <u>HYDROLOGIC ANALYSIS</u>

1. Drainage Area __________ Sq. Mi.
   How was drainage area determined? ____________________
   ____
   ____
   Describe drainage basin parameters ____________________
   ____
   ____

2. Flood Frequency/Magnitude Analysis
   a) Discuss methods considered and used to determine flows & trial design flood: ____________________
   ____
   ____
   b) Attach Flood Frequency curves, regression analysis, USGS equation results, etc.

B. HYDROLOGIC ANALYSIS - (continued)

c) Complete showing trial design flood, 100 yr. & "Greatest Flood" (200 yr. for pipes or 500 yr. for bridges)

______% ______yr. ___________c.f.s. (Trial Design Flood)

1% 100 yr. ___________c.f.s. (Base Flood)

______% ______yr. ___________c.f.s. ("Greatest" Flood)

3. Historic Flood Data:

Elevation ____________________________ Flow ________________________

Frequency ____________________________ Date ________________________

Cause ____

____

C. TRIAL HYDRAULIC ANALYSIS & DESIGN CONSTRAINTS

1. Fish passage considerations, discuss ______________________________________

____

____

____

2. Ice and debris considerations, discuss______________________________________

____

____

____

3. Aggradation/degradation considerations, discuss___________________________

____

____

____

4. Delineated floodplain considerations, discuss______________________________

____

____

____

C. <u>HYDRAULIC ANALYSIS & DESIGN CONSTRAINTS</u> - (continued)

5. Other considerations, discuss ____

____

____

____

6. Size, type, approximate cost and other relative information on trial design structure: ____

____

____

7. Magnitude and percent chance of "overtopping" flood, when applicable
_______________ cfs; ______________ percent chance
Minimum roadway or basin divide overflow elevation______________________
Check where overtopping will occur: Roadway _______ Basin Divide ________
Discuss location and possible effects of overflow _________________________

____

____

8. Flood of Section 650.115(a)(1)(i)
________________ cfs _______________________ percent chance
Water surface elevations (with backwater) _________________________________

9. Highwater elevation at which damages are likely to occur ___________________
Describe ____

____

____

DRAINAGE CROSSING RISK ASSESSMENT

A. ECONOMIC ASSESSMENT

1. Backwater Damage - Major flood damage in this section refers to shopping centers, hospitals, commercial plants, residences, cultivated cropland, etc.
   a) Is there major flood damage potential for the flood of Section 650.115(a)(1)(i) of 23 CFR 650?
      No _________ (Go to 2) Yes _________(Go to 1b)
   b) Will there be flood damage potential to residence(s) or other buildings during a 100 yr. flood?
      Yes _________ (Go to 1c); No ___ (Go to 2) but discuss _____________
   c) Could this flood damage occur even if the roadway crossing wasn't there?
      Yes _________ (Go to 1d); No _________ (Go to 1e)
   d) Could this flood damage be significantly increased by the backwater caused by the proposed crossing?
      Yes _________ (Go to 1e); No _________ (Go to 2)
   e) Could the stream crossing be designed in such a manner so as to minimize this potential flood damage?
      Yes _________ (Go to 1f); No _________ (Go to 2)
   f) Does the value of the building(s) and/or its contents have sufficient value to justify further evaluation of risk and potential flood damage?
      Yes _________ (LTEC-(Go to 2)); No _________ (Go to 2)

2) Traffic Related Losses
   a) Is the roadway overtopping flood greater than the "Greatest" flood?
      Yes _________ (Go to 3); No _________ (Go to 2b)
   b) Is the roadway overtopping flood greater than the overtopping flood for the existing facility?
      Yes _________ (Go to 2c); No _________ (Go to 2c)

A. ECONOMIC ASSESSMENT - (continued)

c) Does the annual risk cost for traffic related costs exceed 10% of the annual capital costs?

Yes ________ (LTEC-(Go to 3)); No _________ (Go to 3)

Annual Risk Cost ____

(See PM #11B)

Annual Capital Cost ____

(Based on 50 yr. service life and "Discount rate for water resources planning", published in the Federal Register; 7.125% discount rate, as of 11/7/97, for FY 1998)

3) Roadway and/or Structure Repair Costs

a) Is the overtopping flood less than a 100 yr. frequency flood?

Yes ________ (Go to 3b); No _________ (Go to 3e)

b) Compare tailwater (TW) elevation with the roadway sag point elevation for the overtopping flood. Is erosion of the embankment a concern?

No _________ (Go to 3e); Yes ________ (Go to 3c)

c) Will the cost of protecting the roadway and/or embankment from severe damage caused by overtopping exceed the cost of providing additional culverts or bridge capacity?

No _________ (Go to 3e); Yes ________ (LTEC-(Go to 4))

d) Is there damage potential to the structure caused by scour, ice, debris, or other means during the lesser of the overtopping flood or the 100 year flood?

No _________ (Go to 4); Yes ________ (Go to 3e)

e) Will the cost of protecting the structure from damage exceed the cost of providing additional culverts or bridge capacity?

No _________ (Go to 4); Yes ________ (LTEC-(Go to 5))

A. ECONOMIC ASSESSMENT - (continued)

4) In your opinion, are there any other factors which you feel should require further study through a risk analysis?

No _________ (Go to 5); Yes _________ (LTEC)

5) Complete design, file documentation and transmit recommendations to appropriate section.

B. HYDRAULIC RECOMMENDATIONS

1) Size, type, location, alignment, elevation, etc., for structure

____

____

____

____

2) Special considerations (bank protection, channel changes, guide banks, ditch blocks, roadway sags, etc.) ____________________

____

____

____

Appendix B – Rainfall Curves

Short-duration precipitation intensity values have been determined by Mark Peterson of MDT's Hydraulic Section for the seven first-order weather stations in Montana. These weather stations are located at Billings, Glasgow, Great Falls, Havre, Helena, Kalispell and Missoula. Table B-1 shows the period of record for each station, and the source of the data. Precipitation intensities for durations of 5, 10, 15, 30 and 60 minutes are shown in Table B-3.

TABLE B-1
PERIOD OF RECORD
OF AVAILABLE DATA

Station	Max. short-duration from local stations	Max. short–duration from published data	Excessive Precip. from published data
Billings	1941-1948 1951-1984	1985-1993	1949-1950
Glasgow		1973-1980 1982-1993	1958-1970 1972
Great Falls	1973-1984	1985-1993	1943-1972
Havre	1973-1984	1985-1987	1908-1934 1936-1940 1942-1972
Helena	1951-1984	1985-1993	1909-1950
Kalispell	1953-1984	1985-1993	1907 1910-1937 1939-1949
Missoula	1936-1984	1985-1993	

One-hour precipitation data for 104 weather service stations in Montana are shown in Table B-4. These 104 stations include the seven first order stations. The values shown in Table B-4 are different from the values in Table B-3 due to the different period of record, and the difference between the highest rainfall in any 60-minute increment (for example, between 2:17 p.m. and 3:17 p.m.) and the highest rainfall in a one-hour increment (for example, between 2 p.m. and 3 p.m.). If durations less than one-hour are required for stations where short-duration data is not available, the ratios from a nearby first order weather station can be used, or the one-hour intensity values can be multiplied by the statewide averages shown in Table B-2 to obtain the 5, 10, 15 and 30 minute values. For example, the 10-year, 5-minute intensity for Alzada would be the 10-year, 1-hour intensity of 0.95 times 4.7 = 4.5 inches per hour. The 25-year, 15-minute intensity for Alzada would be the 25-year, 1-hour intensity of 1.15 x 2.8 = 3.2 inches per hour.

TABLE B-2
STATEWIDE AVERAGES FOR
SHORT-DURATION INTENSITIES

Duration	Multiply 1-hour intensity by:
5 minutes	4.7
10 minutes	3.4
15 minutes	2.8
30 minutes	1.7

The following is a list of stations in each county, with number of years of record in parentheses, included in Table B-4. A map of these stations is shown in Figure B-1.

Beaverhead*-Dillon 9 miles south(43), Dillon Airport(45), Elkhorn Hot Springs(39), Gibbons Pass(45), Lakeview(44), Lima(45), Wisdom(43); *Big Horn*-Decker(10), Lodge Grass(42), Yellowtail Dam (24); *Blaine*-Hays(16); *Broadwater*-Townsend(43); *Carbon*-Bridger(43); *Carter*-Alzada (43), Ekalaka(42); *Cascade*-Great Falls(43), Millegan(43), Neihart(26), Simms(14); *Chouteau*-Highwood(31), Iliad(41), Russell(22); *Custer*-Ismay(43), Miles City(36); *Daniels*-Scobey(37); *Dawson*-Bloomfield(32), Glendive(42); *Fergus*-Hilger(43), Lewistown (43); *Flathead*-Essex(16), Kalispell Airport(40), Polebridge(34), Summit(45), West Glacier (22); *Gallatin*-Belgrade Airport(32), Bozeman 6 miles west(16), Hebgen Dam(44), Logan(21), Willow Creek(23); *Garfield*-Cohagen(43); *Glacier*-Browning(41), Cut Bank(43); *Golden Valley*-Lavina(42); *Granite*-Drummond(43), Philipsburg Ranger Station(14); *Hill*-Havre(29); *Jefferson*-Boulder(44), Cardwell(14), Whitehall(29); *Judith Basin*-Stanford(10), Utica(10); *Lake*-Round Butte(45), Swan Lake(44); *Lewis & Clark*-Augusta(42), Helena(43), Holter Dam(43), Lincoln Ranger Station(20); *Liberty*-Joplin(43); *Lincoln*-Eureka Ranger Station(32), Libby Ranger Station (42); *Madison*-Cameron(45), Silver Star(45); *Meagher*-Kings Hill(22), Martinsdale(50), White Sulphur Springs(18); *Mineral*-Haugan(36), St. Regis(22); *Missoula*-Lolo Hot Springs(22), Missoula(45), Seeley Lake Ranger Station(45); *Park*-Cooke City(23), Corwin Springs(20), Livingston(49); *Petroleum*-Dovetail(35), Winnett(43); *Phillips*-Content (43), Dodson(39), Zortman(23); *Powell*-Ovando (22); *Powder River*-Broadus(49), Ridge(22); *Prairie*-Terry(51); *Ravalli*-Darby(22); *Roosevelt*-Bredette(51), Froid(42), Wolf Point(10); *Rosebud*-Ashland Ranger Station(42), Vananda (43); *Sanders*-Lonepine(19), Plains Ranger Station(43); *Sheridan*-Reserve(33), Westby(41); *Silver Bow*-Butte 8 miles south(39), Divide (44); *Stillwater*-Molt(43), Reedpoint(43); *Teton*-Choteau(51), Gibson Dam(43); *Toole*-Shelby (41); *Valley*-Baylor(43), Fort Peck(34), Glasgow(34); *Yellowstone*-Billings(43), Custer(28).

TABLE B-3
Precipitation Intensity Values, in inches/hour

Return Period and Duration	Billings	Glasgow	Great Falls	Havre
2 years				
5 minutes	3.08	3.91	3.26	2.51
10 minutes	2.26	2.78	2.29	1.93
15 minutes	1.82	2.18	1.84	1.59
30 minutes	1.08	1.40	1.22	1.03
60 minutes	0.62	0.86	0.72	0.64
5 years				
5 minutes	4.58	5.70	4.09	3.38
10 minutes	3.19	3.92	3.05	2.67
15 minutes	2.58	3.14	2.48	2.24
30 minutes	1.58	2.01	1.60	1.44
60 minutes	0.87	1.21	0.93	0.89
10 years				
5 minutes	5.58	6.88	4.90	3.98
10 minutes	3.81	4.68	3.56	3.17
15 minutes	3.10	3.77	2.91	2.68
30 minutes	1.91	2.41	1.86	1.72
60 minutes	1.04	1.44	1.08	1.07
25 years				
5 minutes	6.92	8.50	5.92	4.81
10 minutes	4.66	5.73	4.28	3.86
15 minutes	3.80	4.65	3.52	3.27
30 minutes	2.37	2.97	2.22	2.10
60 minutes	1.27	1.76	1.28	1.30
50 years				
5 minutes	7.97	9.74	6.66	5.46
10 minutes	5.32	6.53	4.83	4.40
15 minutes	4.34	5.32	3.98	3.74
30 minutes	2.71	3.39	2.50	2.39
60 minutes	1.45	2.01	1.44	1.49
100 years				
5 minutes	9.01	10.99	7.42	6.10
10 minutes	5.98	7.34	5.39	4.93
15 minutes	4.88	6.00	4.45	4.20
30 minutes	3.06	3.82	2.78	2.69
60 minutes	1.63	2.25	1.60	1.67

TABLE B–3
Precipitation Intensity Values, in inches/hour

Return Period and Duration	Helena	Kalispell	Missoula
2 years			
5 minutes	2.60	2.06	2.09
10 minutes	1.87	1.60	1.49
15 minutes	1.46	1.32	1.19
30 minutes	0.90	0.82	0.70
60 minutes	0.52	0.48	0.41
5 years			
5 minutes	3.56	2.95	2.90
10 minutes	2.70	2.24	2.14
15 minutes	2.08	1.87	1.78
30 minutes	1.25	1.20	1.03
60 minutes	0.71	0.71	0.60
10 years			
5 minutes	4.21	3.55	3.46
10 minutes	3.25	2.66	2.57
15 minutes	2.50	2.24	2.17
30 minutes	1.49	1.45	1.25
60 minutes	0.84	0.86	0.72
25 years			
5 minutes	5.10	4.36	4.21
10 minutes	4.01	3.25	3.17
15 minutes	3.08	2.74	2.70
30 minutes	1.82	1.79	1.56
60 minutes	1.02	1.08	0.88
50 years			
5 minutes	5.80	4.99	4.80
10 minutes	4.60	3.70	3.63
15 minutes	3.52	3.14	3.11
30 minutes	2.08	2.06	1.79
60 minutes	1.16	1.24	1.01
100 years			
5 minutes	6.48	5.62	5.39
10 minutes	5.18	4.15	4.09
15 minutes	3.96	3.52	3.52
30 minutes	2.33	2.32	2.02
60 minutes	1.30	1.40	1.14

TABLE B-4
ONE-HOUR PRECIPITATION
Return Period (Years)

Station	2	5	10	25	50	100
Alzada	0.60	0.81	0.95	1.15	1.30	1.45
Ashland Ranger Station	0.62	0.87	1.05	1.28	1.46	1.64
Augusta	0.59	0.89	1.09	1.36	1.57	1.78
Baylor	0.63	0.96	1.18	1.47	1.70	1.92
Belgrade Airport	0.37	0.58	0.72	0.91	1.05	1.19
Billings	0.54	0.76	0.91	1.10	1.26	1.41
Bloomfield	0.77	1.03	1.20	1.45	1.64	1.83
Boulder	0.41	0.57	0.67	0.82	0.93	1.04
Bozeman 6 miles W	0.37	0.50	0.58	0.70	0.80	0.89
Bredette	0.67	0.92	1.08	1.31	1.49	1.66
Bridger	0.33	0.43	0.50	0.59	0.67	0.74
Broadus	0.64	0.88	1.04	1.26	1.43	1.60
Browning	0.42	0.55	0.65	0.77	0.87	0.96
Butte 8 miles S	0.41	0.58	0.70	0.85	0.97	1.09
Cameron	0.38	0.56	0.68	0.83	0.96	1.08
Cardwell	0.40	0.54	0.63	0.76	0.86	0.96
Choteau	0.50	0.68	0.80	0.97	1.11	1.24
Clark Canyon Dam	0.35	0.47	0.55	0.66	0.74	0.83
Cohagen	0.53	0.77	0.93	1.14	1.31	1.47
Content	0.62	0.93	1.13	1.41	1.63	1.84
Cooke City	0.34	0.44	0.51	0.60	0.67	0.75
Corwin Springs	0.34	0.41	0.45	0.52	0.57	0.63
Custer	0.50	0.76	0.94	1.17	1.35	1.53
Cut Bank	0.35	0.50	0.61	0.75	0.86	0.97
Darby	0.36	0.44	0.49	0.57	0.64	0.70
Decker	0.67	0.90	1.07	1.30	1.48	1.66

TABLE B-4
ONE-HOUR PRECIPITATION
Return Period (Years)

Station	2	5	10	25	50	100
Dillon Airport	0.35	0.46	0.53	0.63	0.71	0.78
Dillon 9 miles S	0.36	0.48	0.57	0.69	0.78	0.87
Divide	0.39	0.48	0.55	0.64	0.72	0.79
Dodson	0.53	0.74	0.88	1.08	1.23	1.38
Dovetail	0.44	0.68	0.84	1.05	1.21	1.37
Drummond	0.40	0.54	0.63	0.76	0.86	0.96
Ekalaka	0.67	0.87	1.02	1.22	1.38	1.53
Elkhorn Hot Springs	0.32	0.39	0.44	0.51	0.56	0.62
Essex	0.34	0.47	0.55	0.66	0.75	0.84
Eureka Ranger Station	0.39	0.52	0.61	0.73	0.83	0.92
Fort Peck	0.71	1.07	1.31	1.64	1.89	2.14
Froid	0.64	0.84	0.97	1.16	1.31	1.45
Gibbons Pass	0.39	0.53	0.63	0.77	0.87	0.98
Gibson Dam	0.40	0.55	0.65	0.78	0.89	0.99
Glasgow	0.69	0.99	1.18	1.45	1.66	1.87
Glendive	0.72	1.09	1.34	1.68	1.94	2.20
Great Falls	0.58	0.77	0.89	1.07	1.21	1.34
Haugan	0.37	0.51	0.60	0.73	0.83	0.93
Havre	0.46	0.64	0.77	0.93	1.06	1.19
Hays	0.40	0.50	0.58	0.67	0.76	0.84
Hebgen Dam	0.41	0.53	0.61	0.72	0.81	0.90
Helena	0.47	0.70	0.84	1.04	1.19	1.35
Highwood	0.46	0.61	0.72	0.86	0.97	1.08
Hilger	0.53	0.72	0.84	1.02	1.16	1.29
Holter Dam	0.43	0.66	0.80	1.00	1.15	1.31
Iliad	0.44	0.60	0.72	0.88	1.00	1.12

TABLE B-4
ONE-HOUR PRECIPITATION
Return Period (Years)

Station	2	5	10	25	50	100
Ismay	0.73	0.93	1.07	1.26	1.42	1.57
Joplin	0.47	0.70	0.84	1.04	1.20	1.35
Kalispell Airport	0.40	0.72	0.92	1.19	1.40	1.61
Kings Hill	0.45	0.79	1.01	1.30	1.53	1.75
Lakeview	0.43	0.56	0.65	0.77	0.87	0.96
Lavina	0.53	0.72	0.85	1.02	1.16	1.30
Lewistown	0.57	0.77	0.91	1.10	1.25	1.40
Libby Ranger Station	0.36	0.49	0.59	0.71	0.81	0.91
Lima	0.38	0.50	0.58	0.69	0.78	0.87
Lincoln Ranger Station	0.42	0.56	0.66	0.80	0.90	1.01
Livingston	0.40	0.72	0.92	1.19	1.39	1.60
Lodge Grass	0.55	0.79	0.94	1.15	1.31	1.47
Logan	0.43	0.58	0.69	0.83	0.94	1.05
Lolo Hot Springs	0.39	0.53	0.63	0.77	0.88	0.99
Lonepine	0.26	0.34	0.40	0.47	0.53	0.59
Martinsdale	0.47	0.68	0.82	1.01	1.15	1.30
Miles City	0.61	0.86	1.03	1.27	1.45	1.64
Millegan	0.51	0.79	0.97	1.22	1.40	1.59
Missoula	0.38	0.54	0.64	0.79	0.90	1.01
Molt 6 miles SW	0.45	0.71	0.88	1.11	1.29	1.46
Niehart	0.50	0.62	0.71	0.83	0.93	1.03
Ovando	0.43	0.57	0.66	0.78	0.88	0.98
Philipsburg Ranger Station	0.41	0.51	0.58	0.67	0.75	0.83
Plains Ranger Station	0.34	0.43	0.50	0.59	0.66	0.73

TABLE B-4
ONE-HOUR PRECIPITATION
Return Period (Years)

Station	2	5	10	25	50	100
Polebridge	0.34	0.44	0.50	0.59	0.66	0.74
Reedpoint	0.49	0.64	0.80	0.97	1.11	1.24
Reserve 14 miles W	0.63	0.95	1.15	1.44	1.65	1.87
Ridge	0.70	0.93	1.08	1.30	1.47	1.64
Round Butte	0.39	0.47	0.53	0.62	0.69	0.76
Russell	0.47	0.64	0.76	0.91	1.03	1.15
St. Regis	0.41	0.68	0.86	1.10	1.28	1.46
Scobey	0.50	0.68	0.80	0.96	1.09	1.22
Seeley Lake Ranger Station	0.42	0.55	0.65	0.78	0.88	0.98
Shelby	0.47	0.68	0.81	0.98	1.12	1.26
Silver Star	0.33	0.46	0.54	0.66	0.76	0.85
Simms	0.51	0.70	0.82	0.98	1.11	1.25
Stanford	0.56	0.70	0.80	0.94	1.05	1.16
Summit	0.36	0.45	0.51	0.60	0.67	0.74
Swan Lake	0.38	0.47	0.54	0.63	0.71	0.78
Terry 25 miles NW	0.69	0.94	1.10	1.34	1.52	1.71
Townsend 12 miles ENE	0.42	0.54	0.63	0.75	0.84	0.94
Utica	0.64	0.85	1.00	1.20	1.35	1.51
Vananda	0.54	0.86	1.07	1.35	1.56	1.77
Westby	0.62	0.87	1.04	1.28	1.46	1.64
West Glacier	0.41	0.50	0.57	0.66	0.74	0.81
Whitehall	0.47	0.64	0.75	0.91	1.03	1.16
White Sulphur Springs	0.46	0.60	0.71	0.85	0.96	1.07
Willow Creek	0.39	0.47	0.53	0.62	0.69	0.75

TABLE B-4
ONE-HOUR PRECIPITATION
Return Period (Years)

Station	2	5	10	25	50	100
Winnett 11 miles ESE	0.52	0.73	0.87	1.06	1.21	1.36
Wisdom	0.32	0.46	0.55	0.67	0.77	0.86
Wolf Point	0.60	0.77	0.90	1.07	1.21	1.34
Yellowtail Dam	0.55	0.78	0.94	1.14	1.30	1.46
Zortman	0.58	0.79	0.93	1.13	1.28	1.44

Figure B–1
Location of Precipitation Stations

Daily Precipitation Values

An analysis of the daily rainfall records at 291 stations was completed in early 1995. The data analyzed included data through 1993. The results of the analysis are shown in Table B-5. The values in these tables can be used to determine an appropriate nearby station having one-hour precipitation values or short duration precipitation values, shown in Table B-4.

For example, for a project near Anaconda, the nearest stations with one-hour values are Butte 8 miles South, Divide 2 miles Northwest and Philipsburg Ranger Station. Daily precipitation values are available for the Philipsburg Ranger Station, for a station at the Butte Airport, and for a station East Anaconda.There are also values available for Anaconda, but it has only 10 years of record,compared to 72 years of record for East Anaconda. The various return period values for the three stations are shown below.

Station	**2 yr.**	**5 yr.**	**10 yr.**	**25 yr.**	**50 yr.**	**100 yr.**
East Anaconda	1.07	1.40	1.63	1.95	2.19	2.44
Butte Airport	1.07	1.42	1.66	1.99	2.25	2.51
Philipsburg–Ranger Station	1.24	1.88	2.30	2.88	3.32	3.76

The daily values for the station East Anaconda are very close to those of the Butte Airport, so using one-hour values from Butte 8 miles South for a project near Anaconda appears to be reasonable.

DAILY PRECIPITATION

The following is a list of stations in each county, with number of years of record in parentheses, included in the daily precipitation table. All stations with at least 10 years of record are included, although data from stations with short periods of record should be used with extreme caution.

Beaverhead - Apex 2 NW (26), Dillon Airport (41), Dillon Western Montana College (93), Glen 4 N (25), Grant 4 ESE (34), Jackson 1 SE (31), Lakeview (42), Lima (65), Monida (37), Wisdom (42), Wise River 3 WNW (38); ***Big Horn -*** Busby (38), Crow Agency (82), Decker (32), Hardin (36), Kirby 1 S (15), Pryor (40), Wyola 1 SW (44), Yellowtail Dam (30); ***Blaine -*** Chinook (37), Cleveland 5 ENE (24), Harlem 4 W (37), Hays (15), Hogeland 7 WSW (27), Turner (40); ***Broadwater -*** Deep Creek Pass (13), Toston 1 W (21), Townsend (41); ***Carbon -*** Bridger (70), Edgar 9 SE (23),Joliet (39), Red Lodge 1 NW (88), Roberts 1 N (39); ***Carter -*** Albion (32), Belltower (40), Ekalaka (86), Ridgeway 1 S (35); ***Cascade -*** Cascade 5 S (85), Great Falls (56), Great Falls Airport (43), Neihart 8 NNW(26), Power 6 SE (38), Ulm 8 SE Truly (15); ***Chouteau -*** Big Sandy (59), Brady 24 SE (31), Carter (10), Fort Benton (44), Geraldine (42), Iliad (23), Loma 1 WNW (39), Lonesome Lake (25), Shonkin (39); ***Custer -*** Miles City (80), Miles City Airport (55), Mizpah 4 NNW (41), Powderville 8 NNE (28), Volborg (36); ***Daniels -*** Scobey (42); ***Dawson -*** Glendive (92), Lindsay (40), Richey (21); ***Deer Lodge –*** Anaconda (10), East Anaconda (72), Silver Lake (33); ***Fallon –*** Baker (44), Knobs (40), MacKenzie (42), Plevna (72), Webster 3 E (29); ***Fergus -*** Denton (43), Grassrange (40), Lewistown 10 S (42), Lewistown Airport (85), Roy 8 NE (44), Roy 24 NE Mobridge (28), Winifred (40); Flathead – Columbia Falls 5 SW (48), Creston (43), Hungry Horse Dam (36), Kalispell (24), Kalispell Airport (90), Kila (14), Olney (25), Pleasant Valley (24), Polebridge (38), Summit (28),West Glacier (41), Whitefish (39); ***Gallatin –*** Anceney (15), Belgrade Airport (51), Bozeman 6 W (25), Bozeman 12 NE (41), Bozeman – MSU (97), Gallatin Gateway 10 SSW (37), Gallatin Gateway 26 S (21), Hebgen Dam (43), Manhattan (29), Menard (36), Trident (54), West Yellowstone (54); ***Garfield –*** Haxby 18 SW (38), Jordan (65), Mosby 2 ENE (28), Mosby 18 N (23); ***Glacier –*** Babb 6 NE (44), Browning (80), Cut Bank (81), Del Bonita (34), East Glacier (38), St. Mary (11), Santa Rita 14 N (28); ***Golden Valley –*** Barber (39), Ryegate (28); ***Granite -*** Drummond Airport (59), Philipsburg Ranger Station (84); ***Hill –*** Fort Assinniboine (74), Gildford (32), Havre (97), Kremlin (41), Rocky Boy (27), Rudyard 27 NE (25), Simpson 6 NW (43); ***Jefferson –*** Cardwell (12), Whitehall (29); ***Judith Basin –*** Hobson (29), Moccasin Experiment Station (82), Raynesford 2 NNW (34), Stanford (57), Utica 11 WSW (24); ***Lake –*** Bigfork 13 S (47), Polson (67), Polson Kerr Dam (39), St. Ignatius (82); ***Lewis and Clark*** – Augusta (87), Austin (39), Gibson Dam (42), Helena 6 N (29), Helena (99), Holter Dam (43), Lincoln Ranger Station (43), Marysville (12), Rogers Pass 9 NNE (24); ***Liberty –*** Chester (41), Joplin (37), Tiber Dam (39); ***Lincoln –*** Eureka Ranger Station (32), Fortine 1 N (80), Libby Dam (23), Libby 1 NE Ranger Station (80), Libby 32 SSE (40), Troy (28), Troy 18 N (26); ***Madison –*** Alder (14), Alder 17 South (36), Alder Ruby Dam (10), Ennis (42), Norris 3 ENE (20), Norris Madison Pump House (85), Pony (25), Twin Bridges (42), Virginia City (44);

McCone - Brockway 3 WSW (30), Circle (28), Circle 7 N (15), Vida 6 NE (39); ***Meagher –*** Fort Logan 3 ESE (39), Lennep 6 WSW (33), Martinsdale 3 NNW (38), White Sulphur Springs (69); ***Mineral –*** Haugan 3 E (76), St. Regis Ranger Station (26); ***Missoula -*** Alberton (14), Lindbergh Lake (30), Lolo Hot Springs (14),Missoula 2 N (82), Missoula Airport (43), Potomac (22), Seeley Lake Ranger Station (42); ***Musselshell -*** Melstone (43), Roundup (65); ***Park***-Cooke City 2 W (17), Emigrant (15), Gardiner (25), Jardine (20), Livingston (70), Livingston Airport (44), Livingston 12 S (41), Springdale (34), Wilsall (14), Wilsall 8 ENE (34); ***Petroleum –*** Flatwillow 4 ENE (77), Winnett 5 NNE (22); ***Phillips –*** Forks 4 NNE (75), Harb (34), Loring 10 N (19), Malta (82), Malta 35 S (28), Phillips 1 S (17), Saco 1 NNW (26), Telegraph Creek (23), Whitewater (38); ***Pondera –*** Conrad (68), Dupuyer 7 WNW (12), Valier (75); ***Powder River –*** Biddle (36), Biddle 8 SW (28), Broadus (44), Moorhead 9 NE (29), Otter 9 SSW (28), Sonnette 2 WNW (31); ***Powell –*** Deer Lodge 3 W (27), Elliston (25), Ovando (65), Ovando 9 SSE (15), Ovando 7 WNW (14); ***Prairie –*** Mildred (75), Terry (30), Terry 21 NNW (27); ***Ravalli –*** Conner (17), Darby (43), Hamilton (84), Stevensville (77), Sula 3 ENE (35), Western Agriculture Research Center (27); ***Richland –*** Fairview (23), Lambert (12), Nohly 4 NW (30), Savage (85), Sidney (53); ***Roosevelt –*** Bredette (41), Culbertson (84), Poplar 2 E (84), Wolf Point (41); ***Rosebud –*** Birney (34), Brandenburg (35), Colstrip (39), Forsyth (43), Ingomar 11 NE (39), Lame Deer 3 W (34), Rock Springs (41); ***Sanders –*** Heron 2 NW (76), Hot Springs (16), Lonepine 1 WNW (45), Paradise (11), Thompson Falls (80), Trout Creek Ranger Station (67); ***Sheridan –*** Medicine Lake 3 SE (64), Plentywood (34), Raymond Border Station (32), Redstone (41),Westby (37); ***Silver Bow –*** Butte School of Mines (10),Butte Airport (89); ***Stillwater –*** Columbus (42), Fishtail (41), Mystic Lake (44), Rapelje 4 S (81); ***Sweet Grass –*** Big Timber (79), Gibson 2 NE (39), Melville 4 W (37); ***Teton –*** Blackleaf (38), Choteau Airport (77), Fairfield (44), Pendroy 2 NNW (36); ***Toole –*** Dunkirk 15 NNE (64), Ethridge (39), Galata 16 SSW (42), Goldbutte 7 N (44), Shelby Airport (35), Sunburst 8 E (41); ***Treasure –*** Hysham (40), Hysham 25 SSE (24); ***Valley -*** Fort Peck Power Plant (44), Frazer (16), Glasgow (86), Glasgow 15 NW (37), Hinsdale (41), Lustre 4 NNW (44), Opheim 10 N (36), Opheim 16 SE (44); ***Wheatland –*** Harlowtown (42), Judith Gap (37), Judith Gap 13 E (26); ***Wibaux –*** Carlyle 12 NW (30), Wibaux 2 E (42); ***Yellowstone –*** Ballantine (69), Billings Water Plant (89), Billings Airport (44), Broadview (33), Huntley Experiment Station (81), Laurel (33).

TABLE B–5 DAILY PRECIPITATION (24 HR.) RETURN PERIOD (YEARS)						
Station	**2**	**5**	**10**	**25**	**50**	**100**
Alberton	1.00	1.30	1.50	1.79	2.01	2.23
Albion	1.51	1.96	2.27	2.71	3.05	3.39
Alder	1.01	1.23	1.38	1.60	1.78	1.95
Alder 17 South	1.19	1.44	1.62	1.88	2.08	2.28
Alder Ruby Dam	1.07	1.42	1.66	1.99	2.25	2.50
Anaconda	1.10	1.29	1.44	1.65	1.82	1.99
Anceney	1.30	1.57	1.77	2.06	2.29	2.51
Apex 2 NW	0.96	1.26	1.47	1.77	1.99	2.22
Augusta	1.54	2.14	2.55	3.10	3.53	3.96
Austin	1.21	1.65	1.95	2.36	2.69	3.01
Babb 6 NE	1.81	2.45	2.89	3.50	3.97	4.44
Baker	1.43	1.93	2.27	2.74	3.10	3.46
Ballantine	1.27	1.77	2.11	2.58	2.94	3.30
Barber	1.34	1.78	2.09	2.51	2.84	3.17
Belgrade Airport	1.15	1.59	1.88	2.29	2.61	2.92
Belltower	1.45	1.99	2.35	2.86	3.25	3.63
Biddle	1.45	2.02	2.41	2.93	3.34	3.75
Biddle 8 SW	1.73	2.33	2.74	3.30	3.74	4.17
Big Sandy	1.54	2.44	3.02	3.81	4.41	5.02
Big Timber	1.54	2.06	2.41	2.91	3.29	3.67
Bigfork 13 S	1.57	2.23	2.67	3.28	3.75	4.22
Billings Airport	1.36	1.81	2.12	2.55	2.88	3.22
Billings Water Plant	1.52	2.19	2.64	3.25	3.73	4.20
Birney	1.35	1.73	2.00	2.38	2.68	2.97
Blackleaf	1.55	2.25	2.71	3.34	3.83	4.32
Bozeman 12 NE	1.90	2.47	2.87	3.42	3.85	4.28
Bozeman 6 W	1.27	1.64	1.89	2.24	2.52	2.79
Bozeman-MSU	1.29	1.61	1.84	2.16	2.42	2.67
Brady 24 SE	1.32	1.67	1.91	2.25	2.52	2.79
Brandenberg	1.39	1.77	2.04	2.42	2.71	3.00
Bredette	1.49	1.96	2.29	2.74	3.10	3.45
Bridger	1.35	1.89	2.24	2.74	3.12	3.50
Broadus	1.45	1.81	2.07	2.44	2.72	3.01
Broadview	1.41	1.85	2.15	2.57	2.90	3.23

TABLE B–5 DAILY PRECIPITATION (24 HR.) RETURN PERIOD (YEARS)						
Station	**2**	**5**	**10**	**25**	**50**	**100**
Brockway 3 WSW	1.59	2.17	2.56	3.10	3.51	3.93
Browning	1.60	2.39	2.92	3.63	4.18	4.73
Busby	1.48	2.06	2.44	2.97	3.38	3.80
Butte Airport	1.07	1.42	1.66	1.99	2.25	2.51
Butte School of Mines	1.01	1.18	1.31	1.50	1.65	1.80
Cardwell	1.20	1.53	1.75	2.07	2.32	2.57
Carlyle 12 NW	1.71	2.22	2.57	3.06	3.44	3.83
Carter	1.56	1.88	2.12	2.46	2.73	2.99
Cascade 5 S	1.63	2.29	2.74	3.35	3.83	4.30
Chester	1.19	1.56	1.81	2.16	2.44	2.72
Chinook	1.53	2.28	2.78	3.46	3.98	4.50
Choteau Airport	1.42	2.04	2.45	3.02	3.46	3.89
Circle	1.60	2.09	2.43	2.91	3.28	3.65
Circle 7 N	1.42	1.93	2.28	2.76	3.13	3.50
Cleveland	1.98	2.49	2.85	3.36	3.76	4.16
Colstrip	1.42	1.86	2.16	2.59	2.92	3.25
Columbia Falls 5 SW	1.35	1.77	2.06	2.46	2.78	3.09
Columbus	1.63	2.15	2.50	2.99	3.37	3.76
Conner	1.16	1.31	1.43	1.61	1.76	1.90
Conrad	1.44	2.00	2.38	2.90	3.30	3.70
Cooke City 2 W	1.16	1.32	1.44	1.63	1.78	1.94
Creston	1.41	1.67	1.87	2.16	2.38	2.61
Crow Agency	1.51	2.09	2.48	3.01	3.43	3.84
Culbertson	1.59	2.12	2.49	2.99	3.39	3.78
Cut Bank Airport	1.38	1.89	2.22	2.70	3.06	3.43
Darby	1.15	1.33	1.46	1.67	1.83	1.99
Decker	1.41	1.95	2.32	2.82	3.21	3.60
Deep Creek Pass	1.41	1.95	2.31	2.82	3.20	3.59
Deer Lodge 3 W	0.91	1.12	1.27	1.48	1.65	1.82
Del Bonita	1.65	2.19	2.55	3.07	3.46	3.86
Denton	1.50	1.97	2.30	2.75	3.11	3.46
Dillon Airport	1.00	1.21	1.36	1.58	1.75	1.93
Dillon WMC	1.20	1.61	1.89	2.28	2.58	2.88
Drummond Airport	0.98	1.22	1.39	1.63	1.81	2.00

TABLE B–5
DAILY PRECIPITATION (24 HR.)
RETURN PERIOD (YEARS)

Station	2	5	10	25	50	100
Dunkirk 15 NNE	1.24	1.66	1.95	2.35	2.66	2.97
Dupuyer 7 WNW	1.28	1.96	2.40	3.00	3.47	3.93
East Anaconda	1.07	1.40	1.63	1.95	2.19	2.44
East Glacier	1.79	2.61	3.15	3.89	4.46	5.03
Edgar 9 SE	1.93	3.12	3.90	4.94	5.74	6.54
Ekalaka	1.59	2.23	2.67	3.27	3.73	4.19
Elliston	1.14	1.51	1.76	2.10	2.38	2.65
Emigrant	0.87	1.00	1.10	1.24	1.36	1.48
Ennis	1.07	1.36	1.56	1.85	2.07	2.30
Ethridge	1.35	1.72	1.98	2.35	2.64	2.93
Eureka Ranger Station	1.03	1.24	1.40	1.63	1.80	1.98
Fairfield	1.41	1.80	2.07	2.45	2.75	3.04
Fairview	1.63	2.17	2.53	3.05	3.44	3.84
Fishtail	1.79	2.33	2.70	3.23	3.64	4.05
Flatwillow 4 ENE	1.34	1.79	2.10	2.53	2.87	3.20
Forks 4 NNE	1.58	2.01	2.31	2.73	3.07	3.40
Forsyth	1.42	1.82	2.10	2.50	2.81	3.12
Fort Assinniboine	1.38	1.92	2.29	2.79	3.18	3.56
Fort Benton	1.56	2.03	2.35	2.80	3.14	3.49
Fort Logan 3 ESE	1.04	1.34	1.55	1.84	2.06	2.29
Fort Peck Power Plant	1.58	2.29	2.76	3.40	3.90	4.39
Fortine 1 N	1.20	1.54	1.78	2.12	2.38	2.64
Frazer	1.69	2.28	2.68	3.24	3.67	4.10
Galata 16 SSW	1.44	1.86	2.15	2.56	2.88	3.20
Gallatin Gateway 10 SSW	1.67	2.17	2.52	3.00	3.38	3.76
Gallatin Gateway 26 S	1.23	1.55	1.78	2.09	2.34	2.59
Gardiner	0.94	1.20	1.39	1.65	1.85	2.05
Geraldine	1.65	2.27	2.69	3.28	3.73	4.17
Gibson 2 NE	1.50	1.79	2.00	2.31	2.55	2.80
Gibson Dam	1.96	2.95	3.60	4.49	5.18	5.86
Gilford	1.46	1.83	2.09	2.45	2.74	3.03
Glasgow	1.49	2.12	2.54	3.11	3.55	3.99
Glasgow 15 NW	1.56	2.34	2.86	3.56	4.10	4.63
Glen 4 N	0.89	1.08	1.21	1.41	1.56	1.72

TABLE B–5 DAILY PRECIPITATION (24 HR.) RETURN PERIOD (YEARS)						
Station	**2**	**5**	**10**	**25**	**50**	**100**
Glendive	1.66	2.33	2.78	3.40	3.88	4.36
Goldbutte 7 N	1.52	1.92	2.21	2.61	2.92	3.23
Grant 4 ESE	0.95	1.20	1.38	1.63	1.82	2.02
Grass Range	1.55	2.03	2.35	2.81	3.17	3.53
Great Falls	1.42	2.06	2.48	3.06	3.51	3.95
Great Falls Airport	1.48	1.86	2.13	2.51	2.80	3.10
Hamilton	1.05	1.38	1.61	1.93	2.18	2.43
Harb	1.41	2.03	2.45	3.01	3.45	3.88
Hardin	1.34	1.90	2.27	2.78	3.18	3.57
Harlem 4 W	1.44	2.21	2.71	3.40	3.92	4.45
Harlowtown	1.36	1.68	1.91	2.24	2.49	2.75
Haugan 3 E	1.46	1.90	2.21	2.63	2.96	3.29
Havre	1.31	1.80	2.12	2.57	2.92	3.27
Haxby 18 SW	1.51	2.11	2.52	3.08	3.51	3.94
Hays	1.33	1.76	2.05	2.46	2.77	3.09
Hebgen Dam	1.35	1.57	1.74	1.99	2.19	2.39
Helena	1.08	1.47	1.73	2.09	2.38	2.66
Helena 6 N	1.03	1.40	1.65	2.00	2.27	2.54
Heron 2 NW	1.64	1.99	2.24	2.60	2.89	3.17
Hinsdale	1.66	2.25	2.65	3.21	3.64	4.07
Hobson	1.43	1.89	2.21	2.65	3.00	3.34
Hogeland 7 WSW	1.51	2.11	2.52	3.08	3.51	3.94
Holter Dam	1.29	1.63	1.87	2.21	2.47	2.73
Hot Springs	1.09	1.32	1.49	1.73	1.92	2.11
Hungry Horse Dam	1.66	1.98	2.21	2.54	2.81	3.08
Huntley Experiment Station	1.34	1.77	2.07	2.48	2.80	3.12
Hysham	1.50	1.90	2.17	2.56	2.87	3.17
Hysham 25 SSE	1.32	1.63	1.85	2.16	2.41	2.65
Iliad	1.45	2.01	2.39	2.91	3.32	3.72
Ingomar 11 NE	1.35	1.77	2.05	2.45	2.76	3.07
Jackson 1 SE	0.97	1.21	1.39	1.63	1.83	2.02
Jardine	1.25	2.06	2.58	3.29	3.83	4.37
Joliet	1.68	2.31	2.74	3.33	3.79	4.24
Joplin	1.34	1.82	2.14	2.60	2.95	3.29

TABLE B–5 DAILY PRECIPITATION (24 HR.) RETURN PERIOD (YEARS)						
Station	**2**	**5**	**10**	**25**	**50**	**100**
Jordan	1.49	2.10	2.50	3.06	3.49	3.92
Judith Gap	1.47	2.11	2.54	3.13	3.58	4.03
Judith Gap 13 E	1.52	2.01	2.35	2.83	3.19	3.56
Kalispell	1.09	1.37	1.56	1.84	2.06	2.28
Kalispell Airport	1.08	1.44	1.69	2.03	2.30	2.56
Kila	1.14	1.40	1.58	1.84	2.05	2.25
Kirby 1 S	1.76	2.57	3.10	3.83	4.39	4.96
Knobs	1.47	1.84	2.10	2.48	2.77	3.06
Kremlin	1.50	2.05	2.42	2.93	3.33	3.73
Lakeview	1.40	1.71	1.94	2.26	2.51	2.76
Lambert	1.80	2.52	3.01	3.68	4.19	4.71
Lame Deer 3 W	1.42	1.91	2.24	2.71	3.07	3.43
Laurel	1.53	2.05	2.40	2.90	3.28	3.66
Lennep 6 WSW	1.21	1.50	1.71	2.01	2.24	2.47
Lewistown 10 S	1.94	2.55	2.97	3.55	4.00	4.45
Lewistown Airport	1.60	2.13	2.50	3.01	3.40	3.79
Libby 1 NE Ranger Station	1.12	1.42	1.62	1.92	2.15	2.38
Libby 32 SSE	1.30	1.56	1.75	2.03	2.25	2.46
Libby Dam	1.07	1.32	1.49	1.75	1.95	2.15
Lima	1.00	1.42	1.70	2.09	2.39	2.68
Lincoln Ranger Station	1.27	1.62	1.86	2.21	2.48	2.74
Lindbergh Lake	1.47	1.94	2.26	2.71	3.06	3.41
Lindsay	1.71	2.27	2.66	3.19	3.60	4.02
Livingston	1.26	1.71	2.02	2.44	2.77	3.10
Livingston 12 S	1.39	1.83	2.14	2.56	2.89	3.22
Livingston Airport	1.25	1.62	1.88	2.24	2.52	2.80
Lolo Hot Springs 2 NE	1.27	1.54	1.74	2.02	2.24	2.46
Loma 1 WNW	1.48	1.90	2.19	2.60	2.92	3.24
Lonepine 1 WNW	0.89	1.08	1.22	1.41	1.57	1.72
Lonesome Lake	1.40	2.06	2.50	3.10	3.56	4.02
Loring 10 N	1.69	2.18	2.51	2.99	3.36	3.73
Lustre 4 NNW	1.39	1.70	1.92	2.24	2.49	2.74
MacKenzie	1.57	1.97	2.25	2.65	2.96	3.27
Malta	1.52	2.13	2.53	3.09	3.53	3.96

TABLE B–5 DAILY PRECIPITATION (24 HR.) RETURN PERIOD (YEARS)						
Station	**2**	**5**	**10**	**25**	**50**	**100**
Malta 35 S	1.68	2.40	2.88	3.53	4.04	4.55
Manhattan	1.11	1.34	1.50	1.74	1.92	2.11
Martinsdale 3 NNW	1.21	1.55	1.79	2.12	2.38	2.64
Marysville	1.41	1.86	2.16	2.59	2.92	3.26
Medicine Lake 3 SE	1.53	2.08	2.45	2.97	3.37	3.77
Melstone	1.45	1.85	2.14	2.53	2.84	3.15
Melville 4 W	1.61	1.97	2.23	2.60	2.89	3.18
Menard 3 NE	1.25	1.54	1.75	2.05	2.29	2.52
Mildred	1.62	2.22	2.63	3.19	3.62	4.05
Miles City	1.41	1.99	2.38	2.92	3.33	3.75
Miles City Airport	1.48	1.95	2.28	2.73	3.08	3.44
Missoula 2 N	1.15	1.49	1.72	2.05	2.30	2.56
Missoula Airport	1.01	1.28	1.47	1.73	1.94	2.15
Mizpah 4 NNW	1.52	2.01	2.35	2.82	3.18	3.54
Moccasin Experiment Station	1.33	1.81	2.14	2.59	2.93	3.28
Monida	1.07	1.30	1.46	1.70	1.89	2.07
Moorhead 9 NE	1.57	2.13	2.50	3.03	3.43	3.84
Mosby 18 N	1.52	2.04	2.40	2.89	3.27	3.66
Mosby 2 ENE	1.64	2.13	2.46	2.94	3.31	3.68
Mystic Lake	1.70	2.23	2.59	3.09	3.49	3.88
Neihart 8 NNW	1.81	2.41	2.83	3.40	3.85	4.29
Nohly 4 NW	1.69	2.19	2.54	3.03	3.41	3.78
Norris 3 ENE	1.46	2.01	2.38	2.89	3.29	3.69
Norris Madison Pump House	1.54	2.08	2.45	2.96	3.36	3.76
Olney	1.42	1.69	1.89	2.18	2.41	2.63
Opheim 10 N	1.48	1.87	2.13	2.52	2.82	3.12
Opheim 16 SE	1.55	1.97	2.27	2.69	3.01	3.34
Otter 9 SSW	1.84	2.27	2.58	3.02	3.36	3.71
Ovando	1.16	1.69	2.04	2.53	2.90	3.27
Ovando 7 WNW	1.02	1.25	1.41	1.64	1.83	2.01
Ovando 9 SSE	1.01	1.38	1.58	1.87	2.09	2.31
Paradise	1.44	1.94	2.28	2.75	3.12	3.48
Pendroy NNW	1.58	2.07	2.41	2.88	3.25	3.62
Philipsburg Ranger Station	1.24	1.88	2.30	2.88	3.32	3.76

TABLE B–5 DAILY PRECIPITATION (24 HR.) RETURN PERIOD (YEARS)						
Station	**2**	**5**	**10**	**25**	**50**	**100**
Phillips 1 S	1.44	1.81	2.06	2.43	2.72	3.00
Pleasant Valley	1.29	1.62	1.84	2.16	2.41	2.66
Plentywood	1.51	1.92	2.20	2.61	2.92	3.23
Plevna	1.52	1.92	2.21	2.61	2.92	3.23
Polebridge	1.22	1.51	1.72	2.02	2.26	2.49
Polson	1.22	1.63	1.91	2.31	2.61	2.92
Polson Kerr Dam	1.25	1.60	1.85	2.19	2.46	2.73
Pony	1.46	1.68	1.85	2.10	2.31	2.51
Poplar 2 E	1.66	2.19	2.55	3.06	3.45	3.85
Potomac	1.06	1.20	1.30	1.47	1.60	1.73
Powderville 8 NNE	1.57	2.15	2.54	3.09	3.51	3.93
Power 6 SE	1.34	1.70	1.95	2.30	2.58	2.85
Pryor	1.77	2.48	2.96	3.62	4.12	4.63
Rapelje 4 S	1.42	1.83	2.11	2.52	2.83	3.14
Raymond Border Station	1.60	2.07	2.40	2.86	3.21	3.57
Raynesford 2 NNW	1.64	2.22	2.62	3.17	3.59	4.02
Red Lodge 1 NW	1.95	2.45	2.80	3.30	3.69	4.07
Redstone	1.48	1.91	2.21	2.63	2.95	3.28
Richey	1.52	1.91	2.18	2.57	2.87	3.17
Ridgeway 1 S	1.52	1.93	2.21	2.62	2.93	3.25
Roberts 1 N	1.71	2.28	2.66	3.20	3.61	4.03
Rock Springs	1.30	1.73	2.02	2.43	2.75	3.06
Rocky Boy	2.10	2.99	3.58	4.40	5.03	5.66
Rogers Pass 9 NNE	1.81	2.21	2.50	2.92	3.25	3.58
Roundup	1.34	1.82	2.13	2.58	2.92	3.27
Roy 24 NE Mobridge	1.61	2.16	2.53	3.06	3.47	3.87
Roy 8 NE	1.51	1.91	2.18	2.57	2.87	3.18
Rudyard 27 N	1.28	1.61	1.85	2.18	2.44	2.70
Ryegate 18 NNW	1.43	1.69	1.88	2.17	2.39	2.62
Saco 1 NNW	1.43	1.91	2.23	2.69	3.04	3.39
Santa Rita	1.56	2.04	2.37	2.83	3.19	3.55
Savage	1.65	2.21	2.60	3.13	3.54	3.95
Scobey	1.51	1.87	2.13	2.49	2.78	3.07
Seeley Lake Ranger Station	1.17	1.40	1.56	1.80	1.99	2.17

TABLE B–5 DAILY PRECIPITATION (24 HR.) RETURN PERIOD (YEARS)						
Station	**2**	**5**	**10**	**25**	**50**	**100**
Shelby Airport	1.35	1.85	2.18	2.64	3.00	3.36
Shonkin 7 S	2.84	3.78	4.42	5.32	6.02	6.72
Sidney	1.63	2.14	2.49	2.98	3.36	3.74
Silver Lake	1.21	1.52	1.75	2.06	2.31	2.55
Simpson 6 NW	1.39	1.86	2.17	2.61	2.95	3.29
Sonnette 2 WSW	1.54	2.02	2.34	2.80	3.16	3.51
Springdale	1.33	1.69	1.94	2.29	2.57	2.84
St. Ignatius	1.26	1.69	1.98	2.39	2.71	3.03
St. Mary	2.00	3.04	3.73	4.66	5.37	6.09
St. Regis Ranger Station	1.11	1.33	1.49	1.73	1.92	2.10
Stanford	1.44	1.83	2.11	2.49	2.80	3.10
Stevensville	0.99	1.26	1.46	1.73	1.94	2.15
Sula 3 ENE	1.04	1.24	1.39	1.60	1.77	1.94
Summit	2.24	3.47	4.28	5.38	6.22	7.06
Sunburst 8 E	1.41	1.82	2.10	2.51	2.82	3.13
Telegraph Creek	1.58	2.24	2.68	3.28	3.75	4.21
Terry	1.52	1.87	2.12	2.48	2.76	3.04
Terry 21 NNW	1.86	2.64	3.16	3.87	4.42	4.97
Thompson Falls	1.22	1.60	1.86	2.23	2.52	2.80
Tiber Dam	1.27	1.60	1.83	2.16	2.41	2.67
Toston 1 W	1.09	1.33	1.50	1.74	1.93	2.12
Townsend	0.99	1.27	1.46	1.73	1.95	2.16
Trident	1.13	1.44	1.66	1.97	2.21	2.45
Trout Creek Ranger Station	1.59	1.89	2.11	2.43	2.69	2.94
Troy	1.43	1.74	1.96	2.28	2.54	2.79
Troy 18 N	1.79	2.22	2.52	2.95	3.29	3.63
Turner	1.64	2.30	2.74	3.35	3.82	4.29
Twin Bridges	0.92	1.16	1.33	1.57	1.76	1.94
Ulm 8 SE Truly	1.91	2.45	2.83	3.36	3.78	4.19
Utica 11 WSW	1.50	1.96	2.28	2.72	3.07	3.41
Valier	1.56	2.19	2.61	3.20	3.65	4.10
Vida 6 NE	1.84	2.46	2.88	3.46	3.91	4.36
Virginia City	1.14	1.33	1.48	1.70	1.87	2.04
Volborg	1.51	2.07	2.44	2.96	3.36	3.77

TABLE B–5 DAILY PRECIPITATION (24 HR.) RETURN PERIOD (YEARS)						
Station	**2**	**5**	**10**	**25**	**50**	**100**
Webster 3 E	1.68	2.16	2.49	2.97	3.33	3.70
West Glacier	1.58	1.92	2.16	2.51	2.79	3.06
West Yellowstone	1.20	1.46	1.65	1.92	2.13	2.34
Westby	1.57	2.13	2.50	3.02	3.42	3.83
Western Agriculture Research Center	0.95	1.22	1.41	1.68	1.89	2.10
White Sulphur Springs	1.14	1.47	1.69	2.01	2.26	2.51
Whitefish	1.47	1.83	2.08	2.44	2.72	3.00
Whitehall	1.00	1.24	1.41	1.65	1.84	2.03
Whitewater	1.51	1.90	2.18	2.57	2.87	3.18
Wibaux 2 E	1.64	2.17	2.53	3.03	3.42	3.81
Wilsall	1.10	1.46	1.71	2.05	2.32	2.59
Wilsall 8 ENE	1.58	2.05	2.37	2.83	3.18	3.54
Winifred	1.57	2.02	2.34	2.78	3.12	3.46
Winnett 5 NNE	1.37	1.78	2.07	2.47	2.78	3.10
Wisdom	0.94	1.17	1.34	1.57	1.75	1.93
Wise River 3 WNW	0.98	1.24	1.42	1.68	1.88	2.08
Wolf Point	1.65	2.10	2.41	2.86	3.21	3.55
Wyola 1 SW	1.55	1.95	2.24	2.64	2.95	3.26
Yellowtail Dam	1.92	2.82	3.41	4.22	4.85	5.48

Appendix C – Soil Classifications

The following soil surveys have been published by the U.S. Department of Agriculture, Soil Conservation Service. The information in these soil surveys, along with Section 4, Hydrology, of the SCS National Engineering Handbook, can be used to determine the Hydrologic Soil Group(s) for a drainage basin.

Soil Survey of Big Horn County Area, Montana, December 1977

Soil Survey, Bitterroot Valley Area, Montana, May 1959

Soil Survey of Blaine County and part of Phillips County, Montana, April, 1986

Soil Survey of Broadwater County Area, Montana, April 1977

Soil Survey of Carbon County Area, Montana, February 1975

Soil Survey of Cascade County Area, Montana, January 1982

Soil Survey of Dawson County, Montana, January 1976

Soil Survey of Fergus County, Montana, June 1988

Soil Survey of Upper Flathead Valley Area, Montana, September 1960

Soil Survey of Glacier County and Part of Pondera County, Montana, March 1980

Soil Survey of Judith Basin Area, Montana, January 1967

Soil Survey of Helena Valley part of Lewis and Clark County, Montana

Soil Survey of Madison County Area, Montana, September 1989

Soil Survey of McCone County, Montana, July 1984

Soil Survey of Petroleum County, Montana January 1993

Soil Survey of Powder River Area, Montana, June 1971

Soil Survey of Richland County, Montana, August 1980

Soil Survey of Roosevelt and Daniels Counties, Montana, May 1985

Soil Survey of Sheridan County, Montana, June 1977

Soil Survey of Stillwater County Area, Montana, July 1980

Soil Survey of Treasure County, Montana, February 1967

Soil Survey of Valley County, Montana, September 1984

Soil Survey, Wibaux County, Montana, December 1958

Soil Survey of Yellowstone County, Montana, March 1972

The following are reconnaissance surveys, which can be used to determine hydrologic soil types, when more complete soil surveys are not available:

Soil Survey (Reconnaissance), Central Montana, February 1953, including Petroleum, Musselshell, Fergus, Golden Valley, Wheatland, Judith Basin, Meagher, Cascade, Broadwater and Lewis and Clark Counties

Soils of Chouteau County, October 1931

Soils of Cascade County, March 1937

Soils of Broadwater County, March 1944

Soil Survey of The Lower Flathead Valley Area, Montana, 1929, including the Mission, Jocko and Camas Valleys of Lake, Sanders and Missoula Counties

Soil Survey of The Gallatin Valley Area, Montana, 1931

Soils of Golden Valley County, April 1942

Soils of Hill County, May 1931

Soils of Judith Basin County, December 1937

Soils of Meagher County, February 1944

Soil Survey of The Milk River Area, Montana, 1928, including parts of Hill, Blaine, Phillips and Valley Counties.

Soil Survey, The Upper Musselshell Valley Area, Montana, November 1943, including parts of Wheatland County and Meagher County

Soils of Musselshell County, August 1939

Soils of Petroleum County, August 1938

Soils of Pondera County, June 1934

Soil Survey of the Valier Irrigation Project (Pondera County), December 1928

Soils of Richland County, November 1955

Soils of Stillwater County, March 1957

Soil Survey of the Sun River Irrigation Project, September 1927, including parts of Teton, Cascade and Lewis and Clark Counties

Soils of Sweet Grass County, May 1956

Soils of Teton County, January 1937

Soils of Toole and Liberty Counties, April 1933

Soils of Wheatland County, February 1943

Soil Survey, The Lower Yellowstone Valley Area, Montana, September 1939, including parts of Prairie, Dawson and Richland Counties

Soil Survey, The Middle Yellowstone Valley Area, Montana, August 1940, including parts of Treasure, Rosebud and Custer Counties

Soils of Lewis and Clark County, July 1947

Soil Survey (Reconnaissance) of The Northern Plains of Montana, 1929, including all of Sheridan, Daniels, Roosevelt, Valley, Phillips, Blaine, Hill, Liberty, Toole and Chouteau Counties, and the greater parts of Glacier, Pondera and Teton Counties

Mapping for the Soil Surveys for the following areas have been completed, but the surveys have not yet been published. Information on soils in these areas are available from the local SCS Office or from the State SCS Office in Bozeman.

Carter County
Chouteau County Area
Fallon County
Hill County
Liberty County
Prairie County
Toole County
Lake County Area
Lewis & Clark County Area
Missoula County Area
Upper Clark Fork River Area and Parts of Powell, Granite and Deer Lodge Counties
Rosebud County Area and Part of Big Horn County
Choteau-Conrad Area and Parts of Teton and Pondera Counties

Mapping for the Soils Surveys for the following areas is in progress as of April 1993. Information on soils in these areas may be available from the local SCS Office.

Custer County
Musselshell County
Granite County Area
Gallatin County Area
Jefferson County Area and part of Silver Bow County
Phillips County Area
Sanders County Area and Parts of Flathead and Lincoln Counties

Appendix D – Travel Time Estimation

Introduction D.1

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

Following is a discussion of procedures and equations for calculating travel time and time of concentration.

Travel Time D.2

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

Travel time is the ratio of flow length to flow velocity:

$$\mathbf{T_t = L/(3600V)} \qquad \textbf{(D1)}$$

Where:

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

Time of Concentration D.3

The time of concentration is the sum of T_t values for the various consecutive flow segments:

$$\mathbf{T_c = T_{t1} + T_{t2} + \ldots\ T_{tm}} \qquad \textbf{(D2)}$$

Where:

T_c = time of concentration, hr
m = number of flow segments.

Sheet Flow

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

Sheet Flow (Continued)

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

$$\mathbf{T_t = [0.007\ (nL)^{0.8} / (P_2)^{0.5} s^{0.4}]} \qquad \textbf{(D3)}$$

Where:

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

Table D-1 - Roughness Coefficients (Manning's n) For Sheet Flow

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

1 The n values are a composite of information complied by Engman (1986).

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

3 When selecting n, consider cover to a height of about 0.1 ft. This is the only part of the plant cover that will obstruct sheet flow.

Shallow Concentrated Flow D.5

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 hours, and
4. minor effect of infiltration on travel time.

Another approach is to use the kinematic wave equation. For details on using this equation consult the publication by R. M. Regan, A Nomograph Based on Kinematic Wave Theory for Determining Time of Concentration for Overland Flow, Report Number 44, Civil Engineering Department, University of Maryland at College Park, 1971.

After a maximum of 300 feet, sheet flow usually becomes shallow concentrated flow. The average velocity for this flow can be determined from Figure D-1 on the next page, in which average velocity is a function of watercourse slope and type of channel. For slopes less than 0.005 ft/ft, use equations given below for Figure D-1.

Average velocities for estimating travel time for shallow concentrated flow using Figure D-1.

Unpaved $V = 16.1345(s)^{0.5}$ **(D4)**
Paved $V = 20.3282(s)^{0.5}$ **(D5)**

Where:

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

These two equations are based on the solution of Manning's equation with different assumptions for n (Manning's roughness coefficient) and r (hydraulic radius, ft). For unpaved areas, n is 0.05 and r is 0.4; for paved areas, n is 0.025 and r is 0.2.

After determining average velocity using Figure D-1, use equation D1 to estimate travel time for the shallow concentrated flow segment.

Open Channels D.6

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

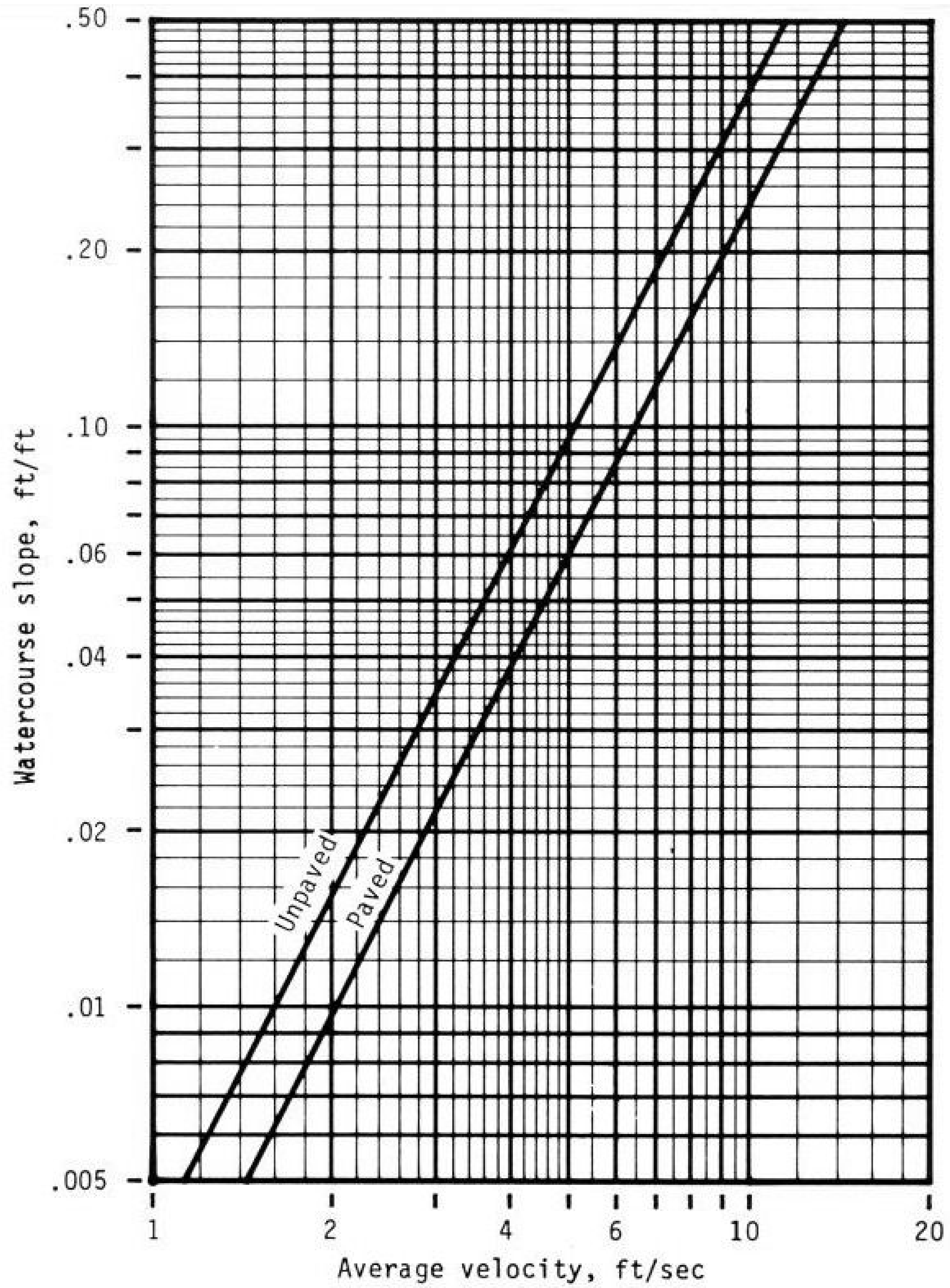

Figure D–1 Average Velocities for Estimating Travel Time for Shallow Concentrated Flow
Source: SCS TR–55

Open Channels (continued)

Manning's equation is

$$\mathbf{V = (1.49\ r^{2/3}\ s^{1/2})/n} \qquad \textbf{(D6)}$$

where:

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

After average velocity is computed using equation D6, T^t for the channel segment can be estimated using equation D1.

Reservoir or Lake D.7

Sometimes it is necessary to compute a T_c for a watershed having a relatively large body of water in the flow path. In such cases, T_c is computed to the upstream end of the lake or reservoir, and for the body of water the travel time is computed using the equation:

$$\mathbf{V_w = (gD_m)^{0.5}} \qquad \textbf{(D7)}$$

Where:

- V_w = the wave velocity across the water, ft/s
- g = 32.2 ft/s^2
- D_m = mean depth of lake or reservoir, ft

Generally, V_w will be high ((8 - 30 ft/s)).

One must not overlook the fact that equation D7 only provides for estimating travel time across the lake and for the inflow hydrograph to the lake's outlet. 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 generally 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 storage routing procedures.

Limitations
D.8

For additional discussion of equation D7 see King's Handbook of Hydraulics, fourth edition, page 8-50, or Elementary Mechanics of Fluids, by Hunter Rouse, John Wiley and Sons, Inc., 1946, page 142.

Equation D7 can be used for swamps with much open water, but where the vegetation or debris is relatively thick ((less than about 25 percent open water)), Manning's equation is more appropriate.

- Manning's kinematic solution should not be used for sheet flow longer than 300 feet. Equation D3 was developed for use with the four standard rainfall intensity-duration relationships.

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

- A culvert or bridge can act as a reservoir outlet if there is significant storage behind it. Detailed storage routing procedures should be used to determine the outflow through the culvert.