Chapter 7

HYDROLOGY

SOUTH DAKOTA DRAINAGE MANUAL

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Chapter 7
HYDROLOGY

7.1 HYDROLOGIC DESIGN GUIDELINES

The guidelines in this Chapter are based on concepts found in Chapter 2 of the AASHTO *Highway Drainage Guidelines* (Reference (1)) and HDS 2 (Reference (2)). The following sections summarize SDDOT practices that relate to hydrology.

7.1.1 Site Data

Because hydrologic considerations can influence the selection of a highway corridor and the alternative routes within the corridor, the designer should include these in the project studies and investigations. The magnitude and complexity of hydrologic studies should be consistent with the importance and magnitude of the project and the problems encountered. See Chapter 4 "Planning and Location (Hydraulics)." Special studies and investigations may be required at sensitive locations. Typical data includes topographic maps, aerial photographs, streamflow records, historical high-water elevations, flood discharges and locations of hydraulic features (e.g., reservoirs, water projects, floodplains, FEMA-mapped floodplains and floodways). See Chapter 5 "Data Collection."

7.1.2 Flood Hazards

A hydrologic analysis is a prerequisite to identifying special flood hazard areas. Then, hydraulic structures may be designed that are cost effective, will require a minimum amount of maintenance and will be safe for the traveling public.

7.1.3 Coordination

Because many levels of government plan, design and construct highway and water resource projects that might have an effect on each other, interagency coordination is usually necessary. In addition, agencies can share data and experiences within project areas to assist in the completion of accurate hydrologic analysis. Chapter 1 "General" and Chapter 4 "Planning and Location (Hydraulics)" discuss coordination in more detail.

7.1.4 Documentation

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 analyses. See Chapter 6 “Documentation” for a further discussion on SDDOT documentation guidelines for hydraulic studies.

7.1.5 Evaluation of Runoff Factors

For all hydrologic analyses, the following factors should be evaluated if they will have a significant effect on the final design:

- 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;
- floodplain characteristics; and
- meteorological characteristics (e.g., precipitation amounts and type, storm cell size and distribution characteristics, storm direction and time rate of precipitation (hyetograph)).

7.1.6 Flood History

All hydrologic analyses should consider the flood history of the area and the effect of these historical floods on existing and proposed structures. The flood history will include the historical floods and the flood history of any existing structures.

7.2 HYDROLOGIC OVERVIEW

7.2.1 Introduction

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 could result in a structure that is either undersized and causes more drainage problems or oversized and costs more than necessary. In contrast, the hydraulic designer should realize 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 rainfall-runoff relationship to expect precise solutions.

7.2.2 Definition

Hydrology is generally defined as a science that addresses the interrelationship between water on and under the earth and in the atmosphere. In this *Manual*, hydrology will address 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 or a hydrograph, which is how the discharge varies with time for a given flood event. For structures that are designed to control the volume of runoff (e.g., detention storage facilities) or where flood routing through culverts is used, then the entire discharge hydrograph will be of interest. Chapter 8 addresses wetlands hydrology, the water-related driving force to create wetlands.

7.2.3 Factors Affecting Floods

In the hydrologic analysis for a drainage structure, there are many variable factors that affect floods. Those that should be considered on an individual site-by-site basis include the following:

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

7.2.4 Sources of Information

The type and source of information available for hydrologic analysis will vary from site to site, and it is the responsibility of the designer to determine what information is available and applicable to a particular analysis. Chapter 5 "Data Collection" provides a comprehensive list of data sources.

7.3 SYMBOLS AND DEFINITIONS

To provide consistency within this Chapter, the symbols in Figure 7.3-A will be used. These symbols have gained wide use in hydrologic publications.

Symbol	Definition	Units
A	Drainage area	ac, sq mi
A	Cross sectional area	sq ft
BDF	Basin development factor	—
C	Runoff coefficient	—
CA	Contributing drainage area	ac, sq mi
CN	NRCS-runoff curve number	—
en	Equivalent years of record	years
i	Rainfall intensity	in/hour
I_a	Initial abstraction from total rainfall	in
k	Land cover factor	—
L	Length of flow	ft
n	Manning's roughness coefficient	—
N	Number of years of flood record	years
P	Accumulated rainfall, rainfall depth	in
PII	Precipitation intensity index	in
Q	Rate of runoff	cfs
q	Storm runoff during a time interval	in
R	Hydraulic radius	ft
RC	Regression constant	—
RI	Recurrence interval	years
RQ	Equivalent rural peak runoff rate	cfs
S	Ground slope, main channel slope	ft/ft, ft/mi or %
S	Potential maximum retention storage	in
S_i	Soil-infiltration index	—
t_c	Time of concentration	minutes or hours
T_L	Lag time	hour
T_t	Travel time	minutes
UQ	Urban peak runoff rate	cfs
V	Velocity	fps
V	Runoff volume	acre-ft
WP	Wetted perimeter	ft
x	Mean exponent	—

Figure 7.3-A — SYMBOLS AND DEFINITIONS

7.4 HYDROLOGIC ANALYSIS PROCEDURE FLOWCHART

Figure 7.4-A presents the steps needed for the hydrologic analysis and the designs that will use the hydrologic estimates.

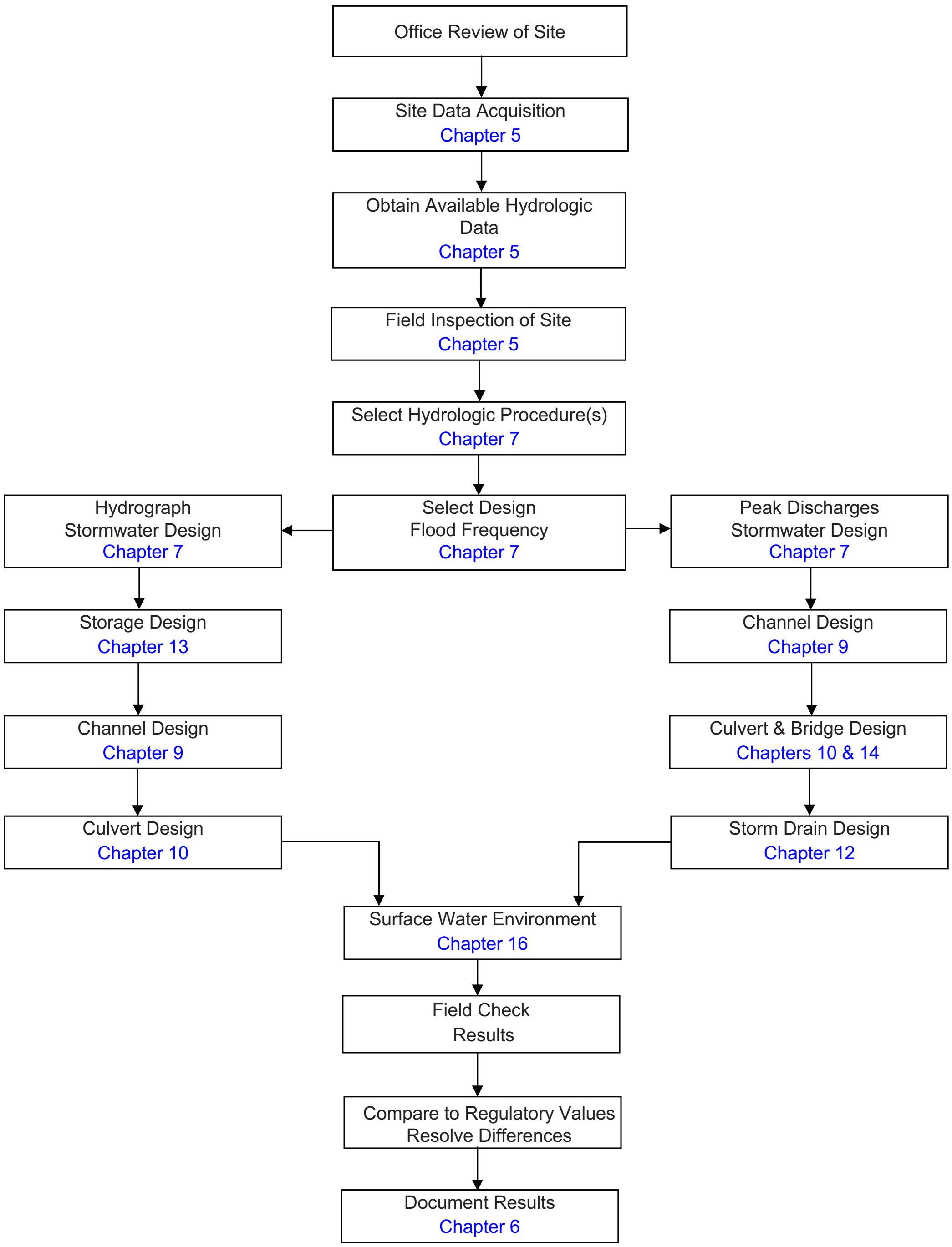


Figure 7.4-A — HYDROLOGIC ANALYSIS PROCEDURE FLOWCHART

7.5 CONCEPT DEFINITIONS

The following are concepts that are important to hydrologic analyses:

1. 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 approximately the 15-year event threshold. As floods become rarer, antecedent moisture has a rapidly decreasing influence on runoff.

2. Depression Storage. Depression storage is the natural depression within a watershed that stores runoff. Generally, after the depression storage is filled, runoff will begin.

3. Flood Frequency. The average time interval between occurrences of a hydrological event of a given or greater magnitude, usually expressed in years; may also be called recurrence interval. Frequency analysis is then the estimation of future peak discharges for various recurrence intervals using the historical record. Another way to express frequency is with probability of exceedence (exceedence probability). Exceedence probability is the likelihood of a given flood flow being equaled or exceeded in any year. Exceedence probability is “one” divided by the return interval, expressed as a percent.

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

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

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

7. 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.

8. Interception. Storage of rainfall on foliage and other intercepting surfaces during a rainfall event is called interception storage.

9. Lag Time. The lag time is defined as the time from the centroid of the excess rainfall to the peak of the hydrograph.

10. 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.

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

12. Stage. The stage of a river is the elevation of the water surface above some elevation datum.

13. Time of Concentration. The time of concentration is the time it takes a drop of water falling on the hydraulically most remote point in the watershed to travel through the watershed to the outlet.

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

For a more complete discussion of these concepts and others related to hydrologic analysis, the reader is referred to Chapter 2 of the AASHTO *Highway Drainage Guidelines* (Reference (1)) and HDS No. 2 (Reference (2)).

7.6 FLOOD FREQUENCY

7.6.1 General

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. For example, if a flood has a 20% chance of being equaled or exceeded each year over a long period of time, the flood will be equaled or exceeded on average once every five years. This is called the return period or recurrence interval (RI). Thus, the exceedence probability equals 100/RI.

The hydraulic designer should note that the ten-year flood is not one that will necessarily be equaled or exceeded every ten years. There is a 10% chance that the flood will be equaled or exceeded in any one year; therefore, the ten-year flood could conceivably occur in several consecutive years.

7.6.2 SDDOT Guidelines and Practices

7.6.2.1 Design Frequency and Headwater

Because it is not economically feasible to design a structure for the maximum runoff a watershed is capable of producing, a design frequency should be selected. The design frequency should be consistent with the facility's highway classification, traffic volume, potential flood hazard to property, expected level of service, political considerations and budgetary constraints. In addition, the hydraulic designer should consider the potential land use on nearby property that could reasonably occur over the anticipated life of the drainage facility.

Figure 7.6-A presents the design flood frequencies expressed as return periods adopted by SDDOT for the various drainage facilities on streets and highways. By definition, roadway overtopping does not occur for the design flood frequency.

7.6.2.2 Design Frequency and Headwater for NFIP-Mapped Floodplains

For a highway encroachment into a NFIP-mapped floodplain (see Section 17.2), the design frequency should be 100 years. The NFIP 100-year discharge may overtop the highway depending on highway classification. The 100-year discharge specified in the applicable FEMA flood insurance study shall be used to analyze impacts of a proposed crossing on a regulatory floodway. However, if this discharge is considered outdated, the discharge based on current methods may be used subject to receiving the necessary regulatory approvals. In general, the SDDOT bridge backwater criteria of 1 ft or less of rise at the 100-year flood should satisfy NFIP requirements and community ordinances. Outside of floodways, culverts may pond water higher than 1 ft over the 100-year flood level if the increased flood level does not affect insurable buildings.

Highway Classification	Return Period (Years) for Drainage Appurtenances					
	Bridge Waterway Openings		Roadway Cross Culverts	Storm Drainage Systems (3)(4)	Roadside Ditches	
	Design Headwater(1)	Scour (2)	Design Headwater(1)	Inlet Spacing(5) & Trunk Line	Design Headwater(1)	Permanent Erosion Protection
Interstate	50	100	50	10 (6)	50	50
US & State Highways	25	100	25	10 (6)	25	25
Local Roads and Streets (7) (ADT ≥ 100)	25	100	25	10	25	25
Local Roads and Streets (ADT < 100)	10	100	10	10	10	10

Notes:

1(a) *The allowable design headwater elevation should not exceed 1 ft below the low subgrade shoulder at the lowest point of the roadway within the drainage basin. For NFIP-mapped floodplains, see Section 7.6.2.2.*

1(b) *The review flood frequency should be 100 years. The headwater elevation for the review flood should not overtop the highway for Interstate and NHS highways. In addition, bridges should not raise backwater more than 1 ft above existing conditions and should provide for at least 2 ft of free board below the lowest superstructure element. For smaller crossings (typically smaller than 1000 acres) on non-NHS highways, review the impacts of the 100-year flood, but overtopping is allowed; however, sensitive sites (urban areas, nearby homes or farmsteads, etc.) may require further analysis regarding the effects of overtopping. Where there is development near a cross culvert, the 100-year event should be reviewed to evaluate the potential impacts.*

1(c) *Ramps should be designed for the lower design frequency of the two intersecting highways, but reviewed with the higher review frequency of the two intersecting highways.*

1(d) *Approaches should be designed for the 10-year flood and reviewed with the review frequency of the highway. The design headwater elevation should not overtop the approach. Approaches should be designed to meet the roadside ditch criteria for the highway. Approaches that do not serve a house may be designed for less than the 10-year flood.*

2 *In addition to the 100-year or worst case up to the 100-year flood, bridge foundations should be evaluated for scour at the super flood (500-year event or that which produces the maximum scour up to the 500-year event) so that the resulting ratio of ultimate to applied loads is greater than 1.0.*

3 *If a storm drain provides the outlet for a cross drain, then the design frequency of the cross drain should be used for the storm drain system downstream from the cross drain inlet.*

4 *If local drainage facilities and practices have provided storm drains of lesser standard, to which the highway system should connect, provide special consideration to whether it is realistic to design the highway system to a higher standard than available outlets.*

5 *Roadway inlets should be designed to meet the spread criteria in Section 12.7.3. Bridge deck inlets should be designed to meet the spread criteria in Section 14.7.3.6.*

6 *For certain sag points, the design frequency should be 50 years. See Section 12.5.2.*

7 *Local roads and streets are considered to be facilities that are not Interstate, US or State highways. For local facilities not eligible for Federal-aid funds, use the design frequencies for ADT < 100.*

Figure 7.6-A — DESIGN FREQUENCY
(Return Period – Years)

Local communities may have standards that should be considered when reviewing drainage facilities. These standards may exceed the NFIP minimums; see Section 17.3.10 for SDDOT policy.

7.6.2.3 Review Frequency and Headwater

All proposed structures sized using the selected design frequency should be reviewed using a review flood or super flood according to the following. The review flood should be used to ensure that there are no unexpected flood hazards and to assess the magnitude and risk associated with damages from larger flood events. Depending on the highway classification and other circumstances, the review flood may be permitted to flow over the highway. At larger structure sites, the SDDOT practice is to try to limit the increase in headwater at the review flood to no more than 1 ft above existing conditions.

For roadway crossings, the review flood frequency is typically the 100-year event. For larger basins and for crossings where overtopping is unlikely, the designer should use the super flood, which is usually the 500-year event but may be the maximum measured flood flow (see Section 7.7.4). In all cases, the designer should assess the damage associated with the review flood. If the increase in damage over the design flood is minor, the facility is satisfactory as designed. If the amount of damage increases substantially, the designer should consider increasing the hydraulic opening to reduce the damage. For example, the design headwater may cause no damage, but a slightly higher headwater may flood a group of homes. In this case, exceeding the design standard would be prudent. The review flood should consider the cumulative impacts of overflow or overtopping from (or to) adjacent basins.

For storm drainage systems see Section 12.5.3.

7.7 SELECTION OF HYDROLOGIC METHOD

7.7.1 Overview

Estimating peak discharges of various recurrence intervals is one of the most challenging decisions faced by hydraulic designers. The problem can be divided into two general categories:

1. 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 25 years of continuous or synthesized data. In South Dakota, this is a relatively rare situation.

2. Ungaged Sites. The site is not near a gaging station and no streamflow record is available. This situation is very common in South Dakota. Because streamflow measurements for determining a flood frequency relationship at a site are usually unavailable, it is accepted practice to estimate peak runoff rates and hydrographs using statistical or empirical methods. In general, results from using more than one method (if applicable to the site) should be compared, not averaged. The Department's practice is to use the discharge that best reflects local project conditions with the reasons documented.

7.7.2 Peak Flow Rates or Hydrographs

A consideration of peak runoff rates for design conditions is generally adequate for conveyance systems (e.g., storm drains or open channels). However, if the design will include flood routing (e.g., storage basins or complex conveyance networks), a flood hydrograph is usually required.

7.7.3 Typical SDDOT Hydrologic Methods

Figure 7.7-A summarizes the general selection and application of those hydrologic methods used with considerable frequency by SDDOT. These methods apply to drainage appurtenances designed by both the Office of Road Design and the Office of Bridge Design. Each of these methods is discussed in subsequent Sections of Chapter 7.

	Title	US Geological Survey (USGS) (Gaged Streams)	US Geological Survey (USGS) (Rural Streams)
	Basic Reference	1. USGS Scientific Investigations Report 2008-5104 "Peak-Flow Frequency Estimates Based on Data through Water Year 2001 for Selected Streamflow-Gaging Stations in South Dakota," 2008 (Reference (3)) 2. See the National Water Information System website	USGS WRI 98-4055 "Techniques for Estimating Peak-Flow Magnitude and Frequency Relations for South Dakota Streams," 1998 (Reference (4))
	Developed By	Sando, S.K.; Driscoll, D.G.; and Parrett, Charles	Steven K. Sando
	South Dakota Drainage Manual Reference	Section 7.8	Section 7.9
APPLICABILITY	Size of Drainage Area	Provided by USGS for each gaged site	Varies by hydrologic subregion; see Figure 7.9-D
APPLICABILITY	Types of Drainage Appurtenances	Bridge waterway openings, large culverts	All drainage structures
APPLICABILITY	General Usage	Use as reference to determine streams that are gaged. If a gage exists, update estimates. See the National Water Information System website.	Use if stream is ungaged.
	Basic Approach	Provides peak discharge for standard design frequencies for 234 gaged sites in South Dakota on natural-flow streams.	Calculates peak discharge for standard design frequencies from data at 197 gaged sites in South Dakota on natural-flow streams to produce regression equations for undeveloped (rural) watersheds.
	Significant Parameters Incorporated	Gaged data used to determine annual maximum discharges.	Gaged data Contributing drainage area Precipitation intensity index Main channel slope *Note: Other basin characteristics evaluated but not found to be explanatory variables.*
	Advantages	Based on gaged data in South Dakota.	Based on gaged data in South Dakota. Most reliable method for ungaged watersheds. Applicable to wide range of drainage area sizes.
	Limitations	Some gaged stations have sparse historical gaged data, as little as 10 years. Drainage areas for (nearby) ungaged sites should be between 50% and 150% of drainage area for gaged sites.	Does not apply to sites on streams: • where watershed is significantly affected by regulation, diversion or urbanization • where storage is significant for the peak discharge of interest • just below a reservoir of any size

Figure 7.7-A — HYDROLOGIC METHODS USED IN SOUTH DAKOTA

Title		US Geological Survey (USGS) (Urban Areas)	Rational Method
Basic Reference		USGS WSP 2207 "Flood Characteristics of Urban Watersheds in the United States — Techniques for Estimating Magnitude and Frequency of Urban Floods," 1983 (Reference (5))	HDS 2, "Highway Hydrology," (Reference (2))
Developed By		V.B. Sauer, W.O. Thomas, V.A. Stricker and K.V. Wilson	Kuichling, 1889
South Dakota Drainage Manual Reference		Section 7.11	Section 7.13
APPLICABILITY	Size of Drainage Area	0.2 sq mi (128 acres) ≤ A ≤ 100 sq mi	A ≤ 200 acres
APPLICABILITY	Types of Drainage Appurtenances	All drainage structures in watersheds that have been developed (channelized, storm drains and curb and gutter)	All drainage structures where the contributing area has a uniform surface cover and texture
APPLICABILITY	General Usage	Use where possible for developed watersheds.	Use to determine pavement discharge and for other surface runoff calculations, primarily to size closed storm drainage systems.
Basic Approach		Calculates peak discharge for standard design frequencies based on a nationwide study with a database of 269 gaged basins in 56 cities and 31 States to produce regression equations that produce a factor to modify the rural regression equations.	Calculates peak discharge for standard design frequencies directly from rainfall data, drainage area size and runoff coefficient.
Significant Parameters Incorporated		Equivalent rural discharge Drainage area size Storage Main channel slope Basin development factor Presence of impervious surfaces	Rainfall intensity Drainage area Runoff characteristics of watershed
Advantages		Modifies the reliable rural regression equations. Can be used to estimate discharge for future development.	Straightforward and easy-to-use. Reasonably accurate for very small, homogenous drainage areas.
Limitations		Not applicable to watersheds less than 0.2 sq mi	Not based on gaged data. Restricted applicability to drainage area size. Neglects storage.

Figure 7.7-A — HYDROLOGIC METHODS USED IN SOUTH DAKOTA (Continued)

	Title	US Geological Survey (USGS) (Hydrograph)	Natural Resources Conservation Service (Hydrograph)
Basic Reference		USGS WRI 80-80 "Techniques for Estimating Flood Peaks, Volumes, and Hydrographs on Small Streams in South Dakota," September 1980 (Reference (6))	WinTR-20 "Project Formulation Hydrology Program System," 2005 (Reference (7))
Developed By		Lawrence D. Becker	NRCS, US Department of Agriculture
South Dakota Drainage Manual Reference		Section 7.15.2	Section 7.15.3
APPLICABILITY	Size of Drainage Area	0.05 sq mi ≤ A ≤ 100 sq mi (peak discharges) 0.05 sq mi ≤ A ≤ 15 sq mi (runoff volume)	0 acres ≤ A ≤ 16,000 acres (25 sq mi) *Note: This limit only applies to TR-55 (Reference (8)). Recommend that A be divided into subwatersheds with uniform cover.*
APPLICABILITY	Types of Drainage Appurtenances	Detention/storage facilities; only use if a hydrograph is needed.	Detention/storage facilities; only use if a hydrograph is needed.
APPLICABILITY	General Usage	Use to develop a hydrograph if storage is significant.	Use to develop a hydrograph if storage is significant.
Basic Approach		Uses gaged data to determine the general characteristics for hydrographs from rural watersheds in South Dakota.	Calculates peak discharge and hydrograph for standard design frequencies from precipitation and drainage area data factoring in water losses from detention, infiltration, evaporation, etc.
Significant Parameters Incorporated		Drainage area Peak discharge Soil-infiltration index Flood volume	Rainfall data Drainage area Antecedent moisture condition Hydrologic properties of soils Vegetative cover Topography
Advantages		Specific to South Dakota	Uses many parameters that impact runoff.
Limitations		Watershed should be essentially natural. Main channel slope should be between 5 ft/mi and 408 ft/mi.	Not based on gaged data. Average slope of drainage area cannot exceed 30%.

Figure 7.7-A — HYDROLOGIC METHODS USED IN SOUTH DAKOTA (Continued)

7.7.4 Maximum Measured Floodflows

USGS Water-Supply Paper 1887 "Maximum Floodflows in the Conterminous United States" by J.R. Crippen and Concrad D. Bue, 1977, which is contained in USGS WRI Report 94-4002 (Reference (9)), provides envelope curves for the maximum measured discharges in the United States. The curves are provided for 17 hydrologic regions. South Dakota falls substantially in Flood Region 11, which can be represented by Equation 7.1. The $Q_{envelope}$ varies from 500 cfs for 0.1 sq mi to 350,000 cfs for 10,000 sq mi:

$$Q_{envelope} = 40800\ A^{0.919}(5 + A^{0.5})^{-1.352} \qquad \text{(Equation 7.1)}$$

where:

A = area, sq mi (A_{max} = 10,000 sq mi)

Equation 7.1 can be used to verify that the design and review floods are reasonable for that size drainage area. This method can also be used to estimate a reasonable discharge for use as the review flood where the highway fill is very high, is costly and is not likely to be overtopped.

7.7.5 Determining Design Discharge

For both roadway and bridge drainage structures, the designer shall apply the methods for determining design discharge in the following order:

1. Determine peak discharge using USGS gaged data (Section 7.8):
   - if the gage is on the same stream, and
   - if the ratio of the gaged drainage area to the ungaged drainage area at the site is between 50% and 150%.
2. Determine peak discharge using USGS regression equations:
   - if the drainage basin is undeveloped, use Section 7.9; or
   - if the drainage basin is partially or fully developed and greater than 200 acres, modify with urban equations (Section 7.11).
3. Determine peak discharge using the Rational Method (Section 7.13):
   - if the drainage area is less than 200 acres, and
   - if the drainage basin has uniform ground cover.

4. Determine hydrograph if storage will be considered (Section 7.15):

   - if drainage area is less than 15 sq mi, use USGS 80-80 (Reference (6)) method; and

   - if the drainage area is greater than 15 sq mi or has more than one cover type, use the NRCS hydrograph method (Reference (7)).

7.8 STREAM GAGE DATA

7.8.1 General

7.8.1.1 Introduction

Many gaging stations exist throughout South Dakota where data can be obtained and used for hydrologic studies. If a stream crossing is located near one of these gages and the gage has at least 10 years of record, a frequency analysis may be applied to that data for the derivation of flood-frequency curves. The designer should study the hydrologic cycles for the site to determine whether the gage data seems reasonable for the period of record. These curves can then be used for several different applications:

- 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 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.

Two methods of estimating flood-frequency curves from stream gage data have commonly been used by highway hydraulic engineers. The first method is a graphical procedure based on the Gumbel distribution; see Reference (2). This distribution was the primary distribution used prior to 1970. The second, a statistical method, makes use of the log-Pearson Type III frequency distribution; see Reference (10).

7.8.1.2 Application

SDDOT uses stream gage data for sizing drainage structures when there are at least 10 years of measured or synthesized stream gage record and no significant changes have occurred or are anticipated in the watershed. The standard method for analyzing stream gage data is the log-Pearson Type III distribution method. If there are outliers, they should be placed into perspective using the procedure found in Bulletin 17B; see Reference (10).

7.8.1.3 Statistical Method Procedure

The log-Pearson Type III distribution is the recommended statistical method. This method is defined by three standard statistical parameters — the mean, standard deviation and coefficient of skew. These parameters are determined from the data sample, which normally consists of the peak annual flows for a period of record.

If a flood-frequency curve is necessary, then by computing several values of Q for different return periods, the log-Pearson fit to the data can be plotted on standard log probability paper. If the skew of the sample data equals 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 downward concavity. If the skew is positive, the plot will be a curve with upward concavity.

7.8.1.4 Skew

There are three alternative methods for determining the value of the skew coefficient to be used in calculating the log-Pearson curve fit:

1. Station Skew. The value of skew that is calculated directly from the gaged data. 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.

2. Generalized Skew. Bulletin 17B contains a map of generalized skew coefficients of the logarithms of annual maximum streamflows throughout the United States and average skew coefficients by one-degree quadrangles over most of the country. The USGS in WRI 98-4055 recommends that these values be used because they will yield slightly higher discharges than if the station skew is used.

3. Weighted Skew. 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.

7.8.2 South Dakota Gaged Streams

7.8.2.1 Introduction

In 2008, the US Geological Survey published USGS Scientific Investigations Report 2008-5104 "Peak-Flow Frequency Estimates Based on Data through Water Year 2001 for Selected Streamflow-Gaging Stations in South Dakota," (Reference (3)). Using log-Pearson Type III procedures and PeakFQ software, the Report provides peak-flow frequency estimates for standard recurrence intervals from 2 years to 500 years for 272 continuous-record and crest-staging gaging stations in South Dakota. Each of the 272 stations had 10 or more years of annual peaks as of water year 2001. In 1998, the US Geological Survey published USGS Water-Resources Investigation 98-4055 "Techniques for Estimating Peak-Flow Magnitude and Frequency Relations for South

Dakota Streams," (Reference (4)). This Report contains results for 197 gages; however, most of the results were taken from the 1996 edition of the 2008 report.

7.8.2.2 Peak-Flow Frequency Estimates

Figure 7.8-A shows the location of selected active gages in 2001. Also, see USGS SIR 2008-5104 for these locations; see Reference (3). If a gage is located on the stream that is crossed by the highway project or is in the vicinity of the highway project, the discharges for the gage can be found in USGS SIR 2008-5104 Appendix 1 in tabular form or in Appendix 2 as graphs. Appendix 1 contains the peak-flow estimates for the following recurrence intervals — 2-year, 5-year, 10-year, 25-year, 50-year, 100-year and 500-year.

The discharges from Reference (3) can be used for design if the gage is at the crossing and the length of record is more than 25 years. If the gage is not at the crossing, the data can be adjusted using the procedure in Section 7.8.2.3. If the record is short, additional years of record should be obtained and the flood-frequency estimate should be updated. Contact the District USGS Office to either obtain an analysis for the gaged data or to obtain the peak-flow data so that a frequency analysis can be prepared for the gage.

7.8.2.3 USGS Nationwide Gaged Data

The gaged data can be obtained using the USGS National Water Information System. Gages can quickly be found if the gage number from either the 2008 or 1998 USGS Report is known. For example, Gage 06442000 on Medicine Knoll Creek near Blunt has a lengthy record of 48 years in the 2008 Report (Location #140). If a Watstore file is downloaded and run with the USGS PeakFQ software (see Chapter 18), the hydraulic designer will produce the results that are shown in the first row. The second row shows the adjusted results from the 2008 report. Because the USGS applied the adjustments, the adjusted discharges should be used.

Frequency Analysis	25-year Peak (cfs)	100-year Peak (cfs)	500-year (cfs)
PeakFQ [1]	4197	13,990	42,320
2008 Report [2]	3480	9980	25,800

1 *Discharges are computed using PeakFQ software.*

2 *Discharge estimates are adjusted values from Appendix 1 of 2008 Report.*

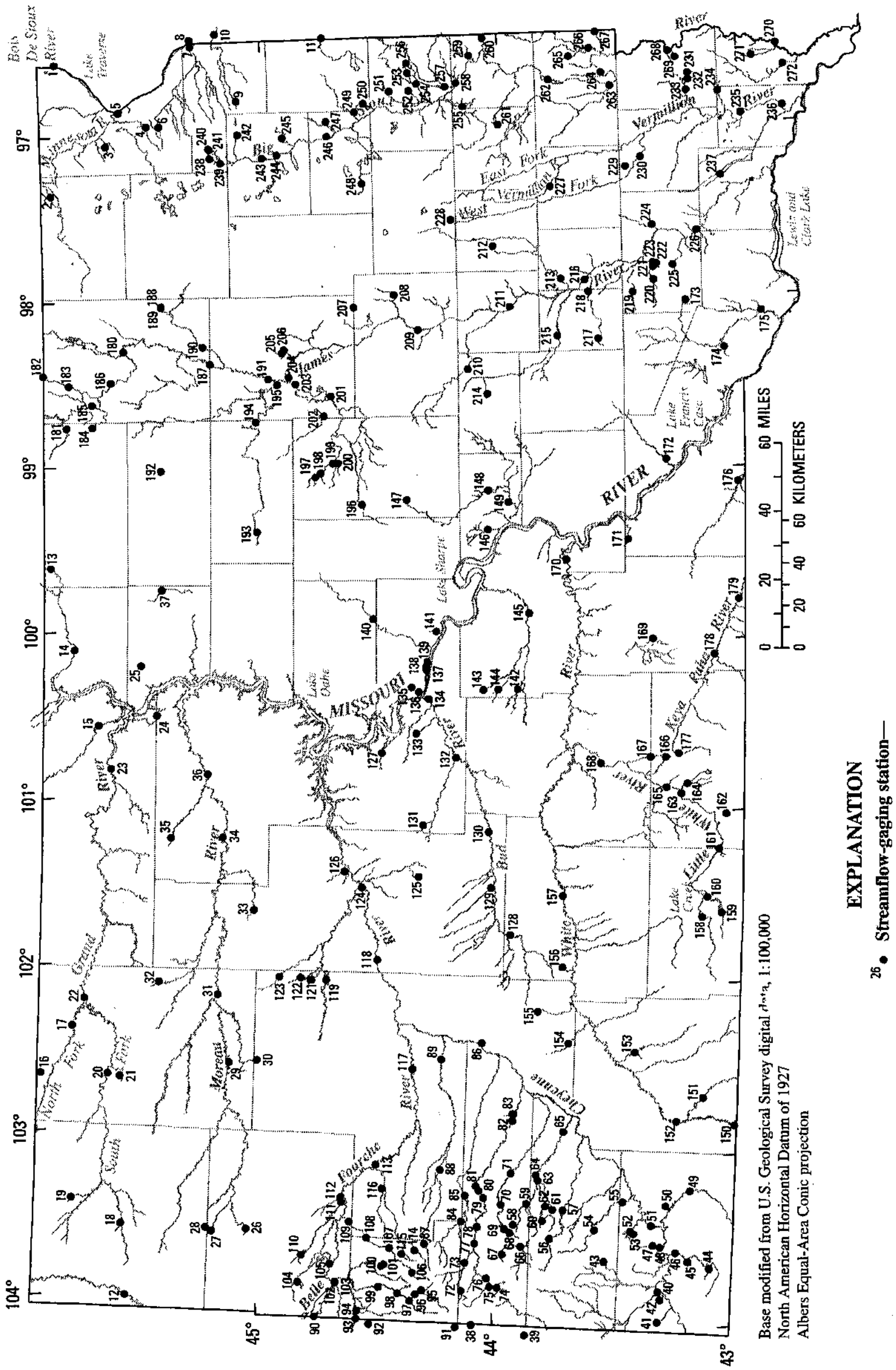


Figure 7.8-A — LOCATION OF ACTIVE GAGING STATIONS IN 2001

The USGS NWIS Web Interface can also be used to determine if a gage exists near the project. The data can be searched in many ways. A list of gages can be produced for a State, county, hydrologic region or latitude-longitude box.

7.8.2.4 Transposition of Data in South Dakota

The transposition of design discharges can be used:

- for an ungaged site on a gaged stream, or
- for an ungaged site on an ungaged stream with basin characteristics similar to those for a gaged site.

This is accomplished by multiplying the design discharge at the gage by the direct ratio of the respective drainage areas raised to the power shown in Figure 7.8-B. These exponents are from USGS WRI 98-4055 (Reference (4)):

$$Q_{T(u)} = Q_{TW(g)} \left(CA_u / CA_g\right)^x \qquad \text{(Equation 7.2)}$$

where:

$Q_{T(u)}$ = peak flow for the ungaged site for a recurrence interval of T years, cfs
$Q_{TW(g)}$ = peak flow for the gaging station for a recurrence interval of T years, cfs
CA_u = contributing drainage area for the ungaged site, sq mi
CA_g = contributing drainage area for the gaging station, sq mi
x = mean exponent for the applicable hydrologic subregion; see Figure 7.8-B

Hydrologic Subregion	Exponent (x)
A	0.529
B	0.615
C	0.569
D	0.545
E	0.691
F	0.654
G	0.689

Note: See Figure 7.9-A for hydrologic subregions in South Dakota.

Figure 7.8-B — EXPONENT FOR HYDROLOGIC SUBREGIONS

Thus, on streams where no gaging station exists, records of gaging stations in nearby hydrologically similar watersheds may be used. The discharge for such an ungaged stream may be determined by the transposition of records using the above procedure. This procedure should be limited to sites that differ in area by no more than 50% from the area of the gaged site. The following is an example using an exponent of 0.689 for Subregion G:

Watershed	Q_{25} (cfs)	Area (sq mi)
Gaged Watershed A	948	105
Ungaged Watershed B	—	80

Q_{25} for Watershed B = 948 $(80/105)^{0.689}$ = 786 cfs

In some streams, especially those draining from the Black Hills, natural peak flows may decrease downstream because of significant floodplain storage or loss of flow in aquifer recharge karst zones. The hydraulic designer is cautioned not to use the transposition approach in such cases.

7.8.2.5 Weighted Peak-Flow Magnitude

The peak-flow estimate determined using the gaged data can be used directly if the length of record is at least 25 years. If the length of record is near the minimum of 10 years, the peak-flow estimate from the gaged site can be adjusted using a weighting procedure with the USGS regression equations from USGS WRI 98-4055 (see Section 7.9). The weighting equation uses the longer equivalent years of record for the regression equations and the actual years of record for the gage to weight the gaged estimate and the regression equation estimate:

$$Q_{TW} = \frac{NQ_{TS} + (en)Q_{TR}}{N + (en)} \quad \text{(Equation 7.3)}$$

where:

Q_{TW} = weighted peak flow for the gaged site for a recurrence interval of T years, cfs

Q_{TS} = station peak flow for the gaged site for a recurrence interval of T years, cfs

N = number of years of station data used to compute Q_{TS}

Q_{TR} = peak flow for a recurrence interval of T years from regional regression equation, cfs (see Figure 7.9-C)

en = equivalent years of record for Q_{TR} from USGS WRI 98-4055 (see Figure 7.9-C)

7.9 RURAL REGRESSION EQUATIONS

7.9.1 Background

7.9.1.1 Introduction

Regional regression equations are a commonly accepted method for estimating peak flows at ungaged sites or sites with insufficient data. Also, they are easy to use and provide consistent findings when applied by different hydraulic engineers; see TRR No. 896 (Reference (11)). Regression studies are statistical practices used to develop runoff equations. These equations are used to relate elements (e.g., peak flow) or some other flood characteristic at a specified recurrence interval to the watershed's physiographic, hydrologic and meteorological characteristics. As such, it should be noted that the regression analysis is a separate study and is not part of the analysis contained in this *Manual* for devising a flood-frequency curve (or hydrograph) at an ungaged site; i.e., the regression analysis only provides the equation for application.

7.9.1.2 National Flood Frequency Program

USGS, in cooperation with FHWA and FEMA, has compiled all current Statewide and urban-area regression equations into the National Flood Frequency (NFF) computer program. This program includes regression equations for estimating flood-peak discharges and techniques for estimating a typical flood hydrograph for a given recurrence interval peak discharge for unregulated rural and urban watersheds. The regression equations for estimating flood-peak discharges for rural, unregulated watersheds have been developed for every State and the Commonwealth of Puerto Rico. Regression equations for estimating urban flood-peak discharges for several urban areas in at least 13 States are also available from the program. For a complete report, consult USGS WRI 94-4002 (Reference (9)).

7.9.2 South Dakota Rural Regression Equations

7.9.2.1 Introduction

In 1998, the US Geological Survey published USGS Water-Resources Investigations Report 98-4055 "Techniques for Estimating Peak-Flow Magnitude and Frequency Relations for South Dakota Streams" (Reference (4)). The USGS study used peak-flow records through water year 1994 for 197 continuous- and partial-record streamflow gaging stations that had 10 or more years of unregulated systematic record. The methodology employed a generalized least-squares regression analysis to relate peak flow to selected basin characteristics for the following recurrence intervals — 2-year, 5-year, 10-year, 25-year, 50-year, 100-year and 500-year — for seven hydrologic subregions in South Dakota.

This Section presents an application-oriented treatment of the South Dakota rural regression equations based on USGS WRI-98-4055; i.e., the Section reproduces the necessary information to calculate the peak discharge at ungaged sites.

7.9.2.2 Hydrologic Subregions

Regression analyses use stream gage data to define hydrologic regions. These are geographic regions that have similar characteristics and, as such, display similar flood-frequency relationships. Figure 7.9-A presents the hydrologic regional boundaries for South Dakota.

When estimating peak-flow frequencies in Subregions C and D, users of the regression equations should especially observe the specific characteristics of the ungaged site relative to the characteristics for the gaging stations used to develop the regression equations for Subregions C and D. Although the ranges in main-channel slopes for gaging stations in Subregions C and D overlap (Figure 7.9-A), sites with main-channel slopes greater than approximately 60 ft/mi generally are much more commonly found in Subregion D than in Subregion C. Conversely, sites with main-channel slopes less than approximately 60 ft/mi are much more commonly found in Subregion C than in Subregion D. Contributing drainage areas for gaging stations in Subregion D tend to be small, with 75% of the stations having drainage areas less than 15 sq mi. Although the largest contributing drainage areas for stations in Subregion D exceed 100 sq mi, users should use extreme caution when applying the equations for Subregion D to contributing drainage areas greater than 15 sq mi because unusually larger peak-flow estimates will result. Users should carefully examine the topographic characteristics of a given ungaged site, and the basin characteristics and peak-flow frequency relations for nearby or topographically similar gaging stations when selecting appropriate regression equations for a specific ungaged location in Subregion C or D.

The estimation of peak-flow magnitudes and frequencies for streams draining the Black Hills is complicated by the fact that some of these streams cross outcrops of fractured limestone bedrock. Large losses in streamflow can occur where streams cross these outcrops. Therefore, caution should be used in applying the regression equations to Subregions F and G to ungaged locations that are immediately downstream from limestone bedrock karst outcrops. This is especially true for estimates of peak flows with smaller recurrence intervals.

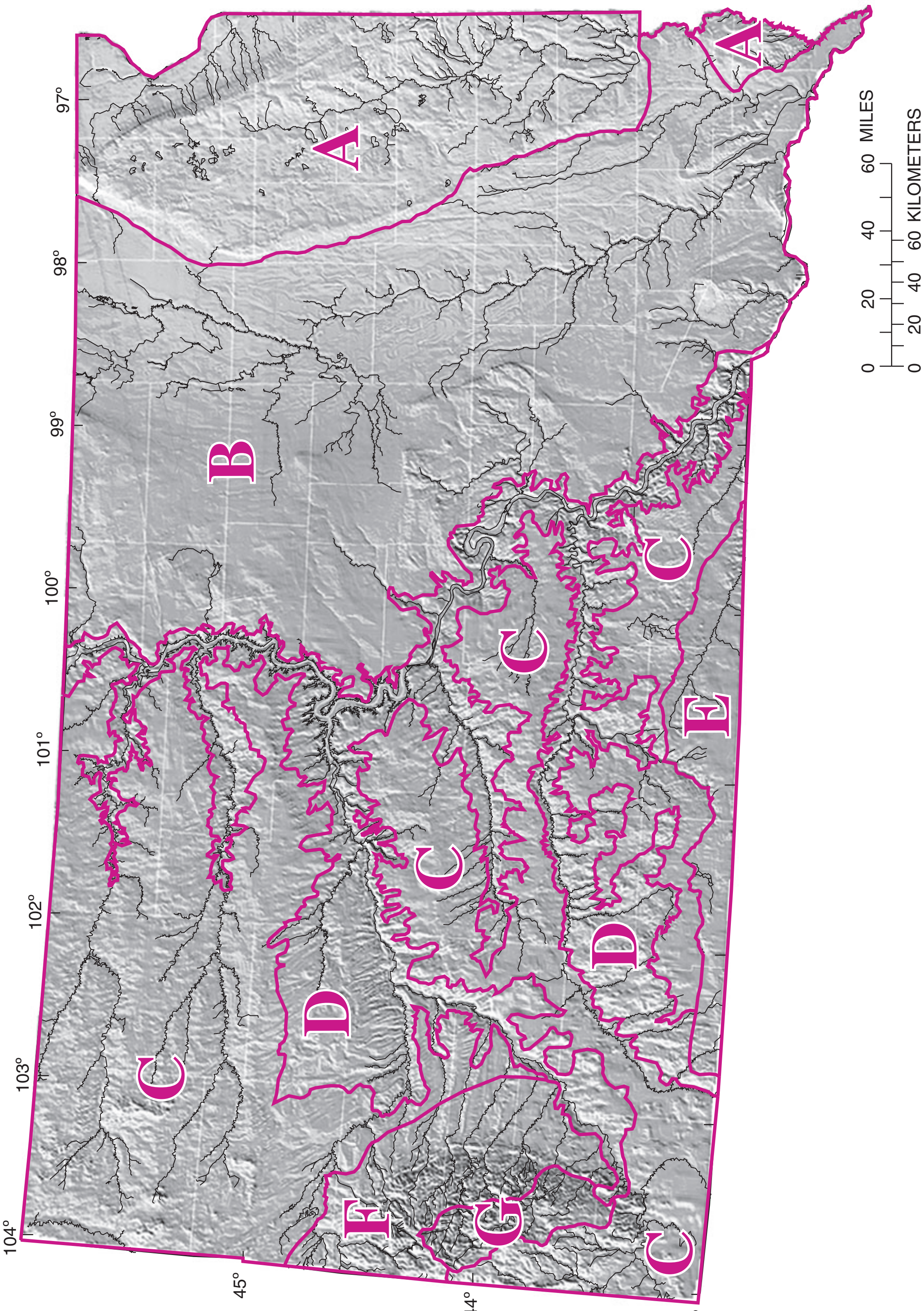


Figure 7.9-A — HYDROLOGIC SUBREGIONS IN SOUTH DAKOTA
(for Use with USGS Rural Regression Equations)

7.9.2.3 Basin Characteristics

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

1. Contributing Drainage Area (CA), Square Miles. The area contributing directly to runoff at the study site. The drainage area can be found by drawing its outline on a topographic map and using a planimeter. Digital methods such as WMS or ArcMap may also be used to determine the drainage area.

2. Non-Contributing Areas. Drainage areas within a watershed that do not contribute runoff directly to a drainage appurtenance are considered non-contributing areas. Such areas may be due to glaciated plains (potholes), enclosed basins, playas, cirques, depression lakes, dry lakebeds or similar landforms.

   Semi-confined basins contributing surface water to another area in wet years, but acting as a sink in dry years, may be considered as a non-contributing area. These types of special situations should be reviewed, coordinated and agreed upon at the State level. Assistance or consultation with climatologists or NOAA on prevailing water/precipitation regimes that may have a long-term influence on non-contributing areas should be explored.

   If non-contributing areas are small and dispersed relative to the encompassing drainage area for the site, they should be considered as part of the encompassing drainage area. If the percent of storage within the total contributing drainage area is significant (greater than 10%), the user should consider using a hydrograph. Some guidance is available at the NRCS website.

3. Main-Channel Slope (S), feet per mile. The slope of the streambed between points that are 10% and 85% of the distance from the location on the stream to the basin divide. Determine from topographic maps (paper copy or digital) to the nearest 0.1 ft/mi. Where two or more major sub-basins join together directly upstream of a crossing, the main basin slope can be weighted based on the slopes for each of its sub-basins. $S_w = (S_1(A_1) + S_2(A_2)) / (A_1 + A_2)$

4. Precipitation Intensity Index (PII), inches. The maximum 24-hour precipitation intensity having a recurrence interval of 2 years (estimated from US Weather Bureau, 1961) minus 1.5, as determined from Figure 7.9-B.

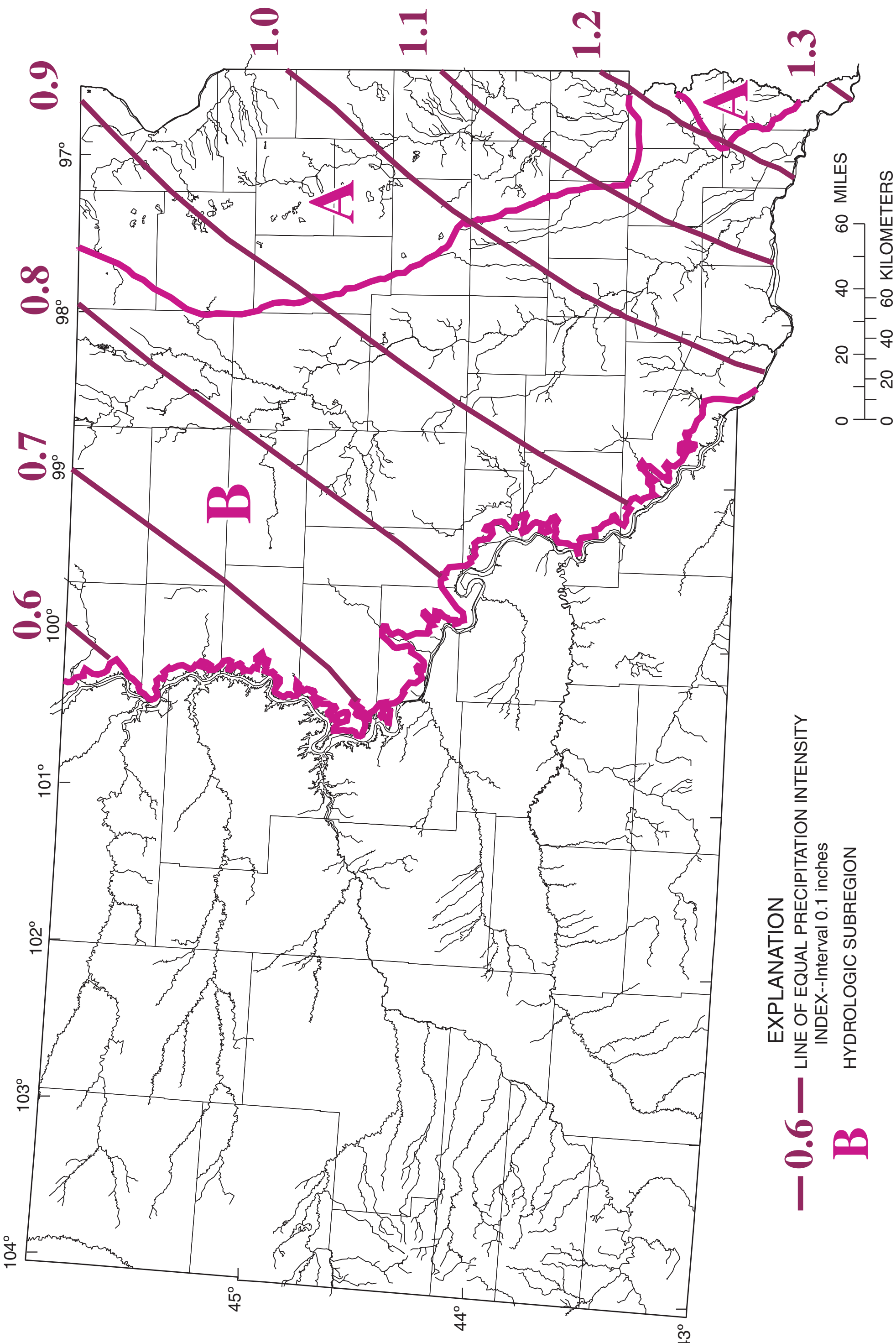
7.9.2.4 Regression Equations for Ungaged Sites

Figure 7.9-C presents the equations for ungaged sites for each of the seven hydrologic subregions in South Dakota from Figure 7.9-A.

7.9.2.5 Limitations on Use of Regression Equations

The following limitations should be considered when using the regression equations to compute peak-flow frequencies for South Dakota streams:

- The rural equations apply to streams that are located in rural watersheds. The equations should not be applied to watersheds substantially affected by urbanization unless modified with the urban equations. See Section 7.11 for urban areas.
- The equations should not be used where dams, flood-detention structures and other man-made works exist that significantly affect the annual peak flows.
- The equations generally should be used only for streams that have drainage areas of less than or equal to 1000 sq mi and for streams that have drainage areas and basin and climatic characteristics that are within the range of characteristics used to develop the regression equations. See Figure 7.9-D. In addition, consider the cautions in Section 7.9.2.2 for Subregions C, D, E, F and G. Subregion E is added to this caution, because the 10 sq mi lower limit is used to develop the regression equation.
- Caution should be exercised for those equations shown in Figure 7.9-C where the standard error of prediction is large (100%). In these cases, the estimate should be checked against the nearest gage shown in SIR 2008-5104 (Reference (3)).



Note: Figure shows isolines of equal value determined by subtracting 1.5 in from the 2-year, 24-hour precipitation values. This figure is only applicable to Subregions A and B.

Figure 7.9-B — PRECIPITATION INTENSITY INDEX (PII)

Recurrence Interval (years)	Equation	Number of Stations Used in Analysis	Standard Error of Estimate (percent)	Average Standard Error of Prediction (percent)	Average Equivalent Years of Record (years)
		Subregion A			
2	$Q = 30.9\ CA^{0.513}\ PII^{6.14}$	55	55	59	4.5
5	$Q = 85.5\ CA^{0.509}\ PII^{5.45}$	55	50	54	6.1
10	$Q = 137\ CA^{0.510}\ PII^{5.12}$	55	50	54	7.8
25	$Q = 218\ CA^{0.513}\ PII^{4.80}$	55	51	56	9.8
50	$Q = 287\ CA^{0.517}\ PII^{4.62}$	55	53	58	11.0
100	$Q = 362\ CA^{0.521}\ PII^{4.47}$	55	55	61	11.9
500	$Q = 553\ CA^{0.531}\ PII^{4.22}$	55	62	69	13.0
		Subregion B			
2	$Q = 18.6\ CA^{0.425}\ PII^{1.10}$	43	60	67	5.4
5	$Q = 51.6\ CA^{0.508}\ PII^{0.835}$	43	57	64	7.1
10	$Q = 86.8\ CA^{0.546}\ PII^{0.764}$	43	59	67	8.7
25	$Q = 148\ CA^{0.584}\ PII^{0.730}$	43	62	72	10.6
50	$Q = 206\ CA^{0.606}\ PII^{0.728}$	43	65	76	11.6
100	$Q = 275\ CA^{0.625}\ PII^{0.742}$	43	69	81	12.4
500	$Q = 480\ CA^{0.661}\ PII^{0.811}$	43	78	93	13.6
		Subregion C			
2	$Q = 25.0\ CA^{0.569}$	48	104	108	1.8
5	$Q = 72.5\ CA^{0.578}$	48	65	67	4.8
10	$Q = 125\ CA^{0.579}$	48	55	58	8.3
25	$Q = 207\ CA^{0.573}$	48	50	53	12.0
50	$Q = 286\ CA^{0.570}$	48	50	53	14.9
100	$Q = 379\ CA^{0.566}$	48	51	55	16.5
500	$Q = 664\ CA^{0.556}$	48	61	65	16.6
		Subregion D			
2	$Q = 78.5\ CA^{0.357}$	17	98	109	2.3
5	$Q = 230\ CA^{0.455}$	17	54	61	7.4
10	$Q = 395\ CA^{0.515}$	17	37	44	17.9
25	$Q = 676\ CA^{0.585}$	17	26	34	39.1
50	$Q = 944\ CA^{0.627}$	17	22	33	52.5
100	$Q = 1270\ CA^{0.663}$	17	22	34	59.2
500	$Q = 2300\ CA^{0.732}$	17	27	41	57.5
		Subregion E			
2	$Q = 12.1\ CA^{0.555}$	10	38	44	4.3
5	$Q = 18.9\ CA^{0.611}$	10	23	28	16.0
10	$Q = 22.6\ CA^{0.653}$	10	20	26	27.0
25	$Q = 27.0\ CA^{0.702}$	10	23	30	30.2
50	$Q = 30.3\ CA^{0.737}$	10	28	36	27.4
100	$Q = 33.6\ CA^{0.769}$	10	34	42	24.2
500	$Q = 41.4\ CA^{0.840}$	10	49	60	18.5

Figure 7.9-C — REGIONAL RURAL REGRESSION EQUATIONS FOR SOUTH DAKOTA

Recurrence Interval (years)	Equation	Number of Stations Used in Analysis	Standard Error of Estimate (percent)	Average Standard Error of Prediction (percent)	Average Equivalent Years of Record (years)
Subregion F					
2	$Q = 0.937\ CA^{0.676} S^{0.447}$	17	93	107	2.6
5	$Q = 0.591\ CA^{0.779} S^{0.745}$	17	71	83	6.0
10	$Q = 0.471\ CA^{0.832} S^{0.907}$	17	61	73	10.5
25	$Q = 0.406\ CA^{0.888} S^{1.06}$	17	53	66	18.4
50	$Q = 0.381\ CA^{0.925} S^{1.16}$	17	50	64	24.6
100	$Q = 0.352\ CA^{0.960} S^{1.25}$	17	49	64	29.4
500	$Q = 0.243\ CA^{1.04} S^{1.47}$	17	58	78	31.2
Subregion G					
2	$Q = 3.46\ CA^{0.650}$	7	41	51	3.9
5	$Q = 7.70\ CA^{0.654}$	7	58	71	3.2
10	$Q = 11.3\ CA^{0.673}$	7	70	87	3.2
25	$Q = 16.5\ CA^{0.704}$	7	86	108	3.3
50	$Q = 21.0\ CA^{0.731}$	7	98	126	3.3
100	$Q = 25.8\ CA^{0.759}$	7	110	144	3.4
500	$Q = 38.5\ CA^{0.826}$	7	141	193	3.5

CA = Contributing drainage area, sq mi
S = Main-channel slope, ft/mi
PII = Precipitation intensity index, in

Figure 7.9-C — REGIONAL RURAL REGRESSION EQUATIONS FOR SOUTH DAKOTA
(Continued)

Hydrologic Subregion	Contributing Drainage Area (sq mi) (CA)	Precipitation Intensity Index (in) (PII)	Main-Channel Slope (ft/mi) (S)
A	0.14 – 983	0.79 – 1.30	—
B	0.22 – 670	0.60 – 1.21	—
C	0.06 – 904	—	—
D	0.11 – 137	—	—
E	10.0 – 760	—	—
F	0.63 – 920	—	29.6 – 460
G	3.81 – 105	—	—

Figure 7.9-D — RANGES OF SUBREGIONAL BASIN CHARACTERISTICS FOR USGS RURAL REGRESSION EQUATIONS

7.10 EXAMPLE PROBLEM — RURAL DISCHARGES FROM REGRESSION EQUATIONS

7.10.1 Problem

Determine the 25-year design flood and 100-year review flood discharges for the drainage basin associated with USGS Gage 6442000 Medicine Knoll Creek near Blunt, South Dakota that was used for the gaged site example in Section 7.8.2.3. The drainage area is 317 sq mi and the precipitation intensity index is 0.76.

7.10.2 Solution

Step 1 From Figure 7.9-A, determine the hydrologic subregion where the site is located — Subregion B.

Step 2 Identify the basin characteristics that are necessary to solve the regression equation for the applicable hydrologic subregion — CA = 317 and PII = 0.76.

Step 3 Select the design frequency from Figure 7.6-A. Use the 25-year design flood and 100-yr review flood discharges.

Step 4 Calculate the peak-flow discharge from the applicable regression equation in Figure 7.9-C for the selected design frequencies:

$$\begin{aligned} Q_{25} &= 148\ CA^{0.584}\ PII^{0.730} \\ &= 148\ (317)^{0.584}\ (0.76)^{0.730} \\ &= 148(28.88)(0.819) \\ &= 3501\ \text{cfs} \end{aligned}$$

$$\begin{aligned} Q_{100} &= 275\ CA^{0.625}\ PII^{0.742} \\ &= 275\ (317)^{0.625}\ (0.76)^{0.742} \\ &= 275(36.57)(0.816) \\ &= 8206\ \text{cfs} \end{aligned}$$

As a comparison, the Q_{25} and Q_{100} discharges determined using the gaged data in Section 7.8.2.3 were 3480 cfs and 9980 cfs.

7.11 URBAN REGRESSION EQUATIONS

7.11.1 Introduction

Regression equations developed by USGS (Reference (5)) as part of a nationwide project can be used to estimate peak runoff for urban watershed conditions. Reference (5) provides two sets of seven-parameter equations and a third set based on three parameters. The equations account for regional runoff variations through the use of the equivalent rural peak runoff rate (RQ). The equations adjust RQ to an urban condition using the basin development factor (BDF) and drainage area (A). The simpler three-parameter equations are recommended for use by HDS 2 (Reference (2)) because they have substantially the same error of estimate.

7.11.2 Application

The hydraulic designer may apply these urban equations for the design of bridges, culverts and similar structures in urban areas, if the contributing watershed is between 0.2 sq mi (128 acres) and 100 sq mi.

7.11.3 Typical Equations and Characteristics

The nationwide equations for urban conditions presented in Figure 7.11-A take the following general form:

$$UQ_T = (RC)A^{b1}\,(13 - BDF)^{b2}\,RQ_T^{\,b3} \qquad \text{(Equation 7.4)}$$

where:

UQ_T = peak discharge for the urban watershed for recurrence interval T, cfs

RC = regression constant

A = contributing drainage area, sq mi

BDF = basin development factor, an index of the prevalence of the drainage aspects of (a) storm drains, (b) channel improvements, (c) impervious channel linings, and (d) curb and gutter streets. The range of BDF is 0 (undeveloped) to 12 (fully developed).

RQ_T = peak discharge for an equivalent rural drainage basin in the same hydrologic area as the urban basin and for recurrence interval T, cfs

b1, b2, b3 = regression exponents

Standard Peak Runoff Equation
$UQ_2 = 13.2A^{0.21}(13 - BDF)^{-0.43} RQ_2^{0.73}$
$UQ_5 = 10.6A^{0.17}(13 - BDF)^{-0.39} RQ_5^{0.78}$
$UQ_{10} = 9.51A^{0.16}(13 - BDF)^{-0.36} RQ_{10}^{0.79}$
$UQ_{25} = 8.68A^{0.15}(13 - BDF)^{-0.34} RQ_{25}^{0.80}$
$UQ_{50} = 8.04A^{0.15}(13 - BDF)^{-0.32} RQ_{50}^{0.81}$
$UQ_{100} = 7.70A^{0.15}(13 - BDF)^{-0.32} RQ_{100}^{0.82}$
$UQ_{500} = 7.47A^{0.16}(13 - BDF)^{-0.30} RQ_{500}^{0.82}$

Figure 7.11-A — USGS NATIONWIDE REGRESSION EQUATIONS FOR URBAN CONDITIONS

7.11.4 Procedure

The procedure is not intended to require precise measurements. A certain amount of subjectivity is involved, and field checking should be performed to obtain the best estimate. The BDF is the sum of the assigned codes; therefore, with three subbasins per basin, and four drainage aspects to which codes are assigned in each subbasin, the maximum value for a fully developed drainage system would be 12. Conversely, a totally undeveloped drainage system would receive a BDF of zero. This rating does not necessarily mean that the basin is unaffected by urbanization. In fact, a basin could be partially urbanized, have some impervious area and have some improvement of secondary tributaries, and still have an assigned BDF of zero.

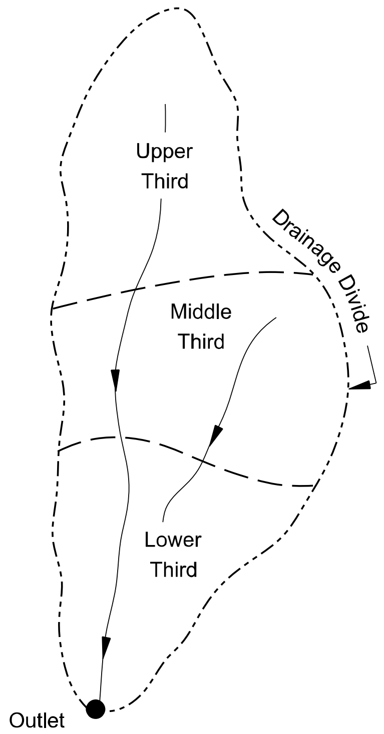
The following steps are used to apply the nationwide equations:

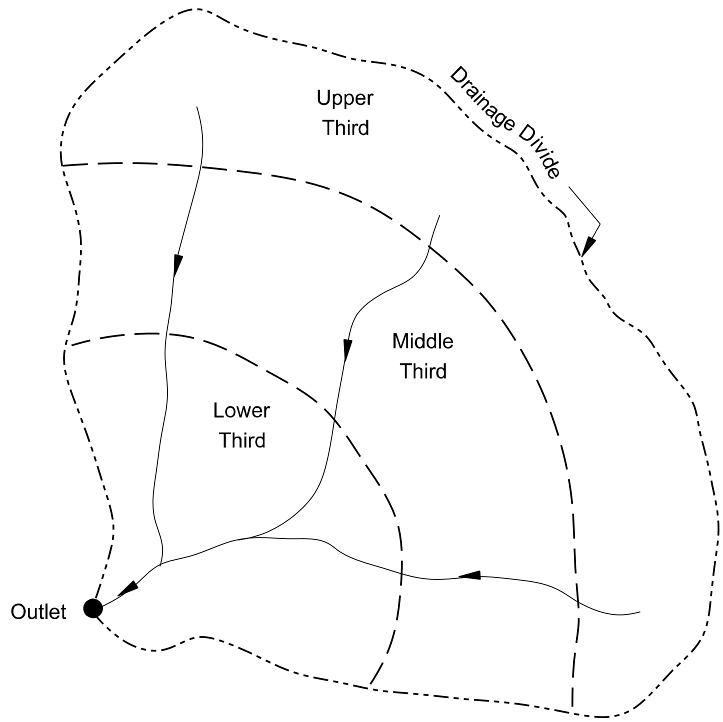
1. Use the applicable USGS rural regression equations in Figure 7.9-C for natural flow conditions in the hydrologic subregion to estimate the peak runoff rate for an equivalent rural drainage basin (RQT). See Section 7.9.

2. Determine input parameters for current or future urban conditions. The BDF should be determined from drainage maps and by field inspection of the watershed.

3. Calculate peak runoff rates for desired return periods.

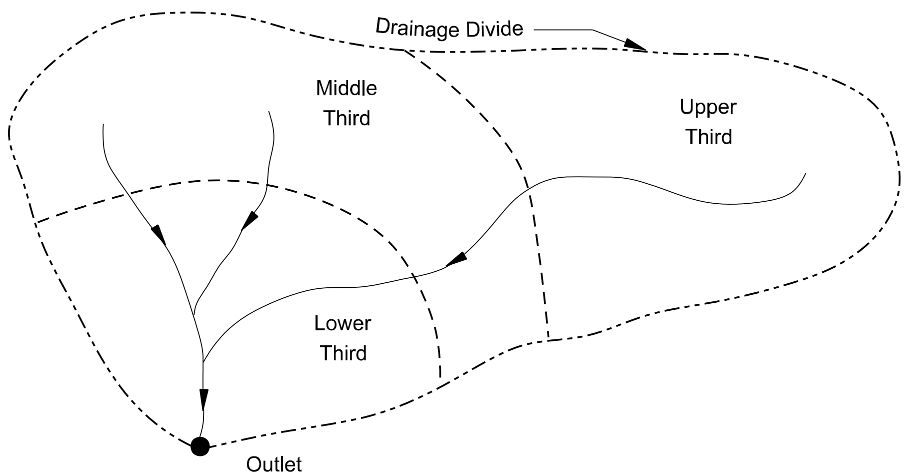
The basin is first divided into thirds (see Figure 7.11-B) and, within each third, four aspects of the drainage system are evaluated and assigned a code as discussed in the following Subsections.





(A) LONG, NARROW BASIN

(B) FAN-SHAPED BASIN



(C) SHORT, WIDE BASIN

Schematics illustrate typical drainage basin shapes and their subdivision into basin thirds. Note that stream-channel distances within any given third of a basin in the examples are approximately equal but, between basin thirds, the distances are not equal to compensate for relative basin width of the thirds.

Figure 7.11-B — BASIN SUBDIVISION

7.11.4.1 Channel Improvements

If channel improvements (e.g., straightening, enlarging, deepening, clearing) are prevalent for the main drainage channels and principal tributaries (those that drain directly into the main channel), then a code of One is assigned. To be considered prevalent, at least 50% of the main drainage channels and principal tributaries should be improved to some degree over natural conditions. If channel improvements are not prevalent, then a code of Zero is assigned.

7.11.4.2 Channel Linings

If more than 50% of the length of the main drainage and principal tributaries has been lined with an impervious material (e.g., concrete), then a code of One is assigned to this aspect. If less than 50% of these channels are lined, then a code of Zero is assigned. The presence of channel linings is a good indication that channel improvements have been performed and signifies a more highly developed drainage system.

7.11.4.3 Storm Drains

A closed storm drainage system is frequently used on the secondary tributaries that receive drainage directly from streets or parking lots. Many of these drains empty into open channels; in some basins, however, they empty into channels enclosed as box or pipe culverts. Where more than 50% of the secondary tributaries within a subbasin consist of storm drains, a code of One is assigned to this aspect; if less than 50%, then a code of Zero. Note that, if 50% or more of the main drainage channels and principal tributaries are enclosed, the aspects of channel improvements and channel linings would also be assigned a code of One.

7.11.4.4 Curb and Gutter

If more than 50% of a subbasin is urbanized, and if more than 50% of the streets and highways in the subbasin are constructed with curbs and gutters, then a code of One would be assigned to this aspect. Otherwise, it would receive a code of Zero. Drainage from curb and gutter streets frequently empties into storm drains.

7.12 EXAMPLE PROBLEM — URBAN REGRESSION EQUATIONS

7.12.1 Problem

The upper 50 sq mi of the watershed that drains to Gage 6442000 (Subbasin B) is being developed. The area has been completely subdivided, has streets with curb and gutter, but is only partially served by storm drains. However, because the drainage structure being designed is for a major highway, full development of the watershed will be assumed.

7.12.2 Solution

Step 1 Use the USGS regression equations in Figure 7.9-C (Subbasin B) for natural flow conditions to estimate the peak runoff rate for an equivalent rural drainage basin (RQ_T) and the PII = 0.76 as determined in Section 7.10:

$$
\begin{aligned}
RQ_{25} &= 148\ CA^{0.584}\ PII^{0.730} \\
&= 148\ (50)^{0.584}\ (0.76)^{0.730} \\
&= 148(9.82)(0.819) \\
&= 1190\ \text{cfs}
\end{aligned}
$$

$$
\begin{aligned}
RQ_{100} &= 275\ CA^{0.625}\ PII^{0.742} \\
&= 275\ (50)^{0.625}\ (0.76)^{0.742} \\
&= 275(11.53)(0.816) \\
&= 2587\ \text{cfs}
\end{aligned}
$$

Step 2 Determine input parameters for current or future urban conditions. The BDF should be determined from drainage maps and by field inspection of the watershed. Because a fully developed condition is being assumed, the BDF will be 12.

Step 3 Calculate peak runoff rates for desired return periods:

$$
\begin{aligned}
UQ_{25} &= 8.68A^{0.15}\ (13 - BDF)^{-0.34}\ RQ_{25}^{0.80} \\
&= 8.68(50)^{0.15}\ (1)\ (1190)^{0.80} \\
&= 8.68(1.8)289 = 4515\ \text{cfs}
\end{aligned}
$$

$$
\begin{aligned}
UQ_{100} &= 7.70.A^{0.15}\ (13 - BDF)^{-0.32}\ RQ_{100}^{0.82} \\
&= 7.70(50)^{0.15}\ (1)\ (2587)^{0.82} \\
&= 7.70(1.8)629 = 8714\ \text{cfs}
\end{aligned}
$$

7.13 RATIONAL METHOD

7.13.1 Introduction

The Rational Method is recommended for estimating the design storm peak runoff for areas up to 200 acres. This method, although 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. The procedures presented in this Section are based on HDS 2 (Reference (2)).

7.13.2 Application

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 surface characteristics for the drainage area, consider any 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 considered.
- Restrictions to the natural flow (e.g., highway crossings and dams that exist in the drainage area) should be investigated to determine how they might affect the design flows.
- The charts, graphs and tables included in this Section are not intended to replace reasonable and prudent engineering judgment that should permeate each step in the design process.

7.13.3 Characteristics

Characteristics of the Rational Method that limit its use to 200 acres include:

- Rainfall Intensity vs. Time of Concentration. The rate of runoff resulting from any rainfall intensity is a maximum when the rainfall intensity lasts as long as or longer than the time of concentration. That is, the entire drainage area does not contribute to the peak discharge until the time of concentration has elapsed.
- Peak Discharge Frequency. The frequency of peak discharges is the same as that of the rainfall intensity for the given time of concentration.

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

- Peak Rate. The peak rate of runoff is sufficient information for the decision.

7.13.4 Equation

The Rational equation 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 equation is expressed as follows:

$$Q = CiA \qquad \text{(Equation 7.5)}$$

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/hour

A = drainage area tributary to the design location, acres

The results of using the Rational Method to estimate peak discharges are very sensitive to the parameters used, especially time of concentration and runoff coefficient. The hydraulic designer should use good engineering judgment in estimating values that are used in the Method. Following is a discussion of the variables used in the Rational Method.

7.13.5 Runoff Coefficient

The runoff coefficient (C) is the variable of the Rational Method least amenable to precise determination and requires the judgment and understanding of the hydraulic designer. Although engineering judgment will always be required in the selection of runoff coefficients, a typical coefficient represents the integrated effects of many drainage basin parameters. See Figure 7.13-A.

Type of Drainage Area	Runoff Coefficient (C)
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
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
Playground	0.20 – 0.40
Railroad yard areas	0.20 – 0.40
Unimproved areas	0.10 – 0.30
Lawns:	
Sandy soil, flat (< 2%)	0.05 – 0.10
Sandy soil, average (2% to 7%)	0.10 – 0.15
Sandy soil, steep (> 7%)	0.15 – 0.20
Heavy soil, flat (< 2%)	0.13 – 0.17
Heavy soil, average (2% to 7%)	0.18 – 0.22
Heavy soil, steep (> 7%)	0.25 – 0.35
Streets:	
Asphalt	0.70 – 0.95
Concrete	0.80 – 0.95
Brick	0.70 – 0.85
Drives and walks	0.75 – 0.85
Roofs	0.75 – 0.95
Roadway Ditches	0.20 – 0.50
Forested Areas	0.10 – 0.30
Meadows	0.10 – 0.40
Pasture Land	0.20 – 0.45
Cultivated Land:	
Sand and gravel, flat (< 2%)	0.20 – 0.30
Sand and gravel, average (2% to 7%)	0.30 – 0.35
Sand and gravel, steep (> 7%)	0.35 – 0.45
Clay and loam, flat (< 2%)	0.30 – 0.45
Clay and loam, average (2% to 7%)	0.45 – 0.55
Clay and loam, steep (> 7%)	0.55 – 0.70

Figure 7.13-A — RUNOFF COEFFICIENTS FOR THE RATIONAL EQUATION (Reference (2))

7.13.6 Time of Concentration

7.13.6.1 General

The time of concentration (t_c) is the time required for water to flow from the hydraulically most remote point of the drainage area to the point under investigation. Factors that affect the time of concentration are the length of flow, the slope of the flow path, and the roughness of the flow path. For flow at the upper reaches of a watershed, rainfall characteristics, most notably the intensity, may also influence the velocity of the runoff.

Various methods can be used to estimate the time of concentration of a watershed. When selecting a method to use in design, it is important to select a method that is appropriate for the flow path. Sheet flow occurs in the upper reaches of a watershed. Such flow occurs over short distances and at shallow depths prior to the point where topography and surface characteristics cause the flow to concentrate in rills and swales. The depth of such flow is usually 0.8 in to 1.2 in or less. Shallow concentrated flow is runoff that occurs in rills and swales and has depths in the range of 1.6 in to 3.9 in. Part of the principal flow path may include pipes or small streams. The travel time through these segments would be computed separately. Velocities in open channels are usually determined assuming bank-full depths.

The type of flow that occurs is a function of the conveyance system and is best determined by field inspection. Development within a watershed may create changes in overland flow paths. The overland flow path is not necessarily perpendicular to the contours shown on the available mapping. Often, the land will be graded and swales will intercept the natural contour and conduct the water to the roads and streets, which reduces the time of concentration. Designers should identify constructed topographic changes and include existing flow characteristics in their design. After the designer has determined the flow type for each area within the watershed, use Figure 7.13-B to determine the equations to calculate the travel time for that area.

Type of Flow	Recommended Equations
Sheet Flow	Equation 7.7 or 7.8
Overland Flow	Equations 7.9 & 7.10
Shallow Concentrated Flow	Equations 7.9 & 7.10
Channel or Pipe Flow	Equations 7.9 & 7.11

Figure 7.13-B — FLOW TYPES AND EQUATIONS FOR TRAVEL TIME

The time of concentration (t_c) for the watershed is calculated by summing the travel times (T_t) for each area using Equation 7.6. The t_c may include all the travel time types, a few types or just a single type:

$$t_c = T_{t(sheet)} + T_{t(overland)} + T_{t(shallow)} + T_{t(channel)} + T_{t(pipe)} \quad \text{(Equation 7.6)}$$

The time of concentration from Equation 7.6 should be used if it is longer than the minimum t_c listed in Section 7.13.6.2. If it is shorter, use the minimum t_c.

7.13.6.2 Minimum Time of Concentration

SDDOT has adopted the following for minimum times of concentration:

- Use a minimum t_c = 5 minutes for paved areas other than bridge decks.
- Use a minimum t_c = 10 minutes for bridge decks.
- Use a minimum t_c = 10 minutes for overland (sheet) flow and unpaved roads.

7.13.6.3 Sheet-Flow Travel Time

Sheet flow is a shallow mass of runoff on a plane surface with the depth uniform across the sloping surface. Typically, flow depths will not exceed 2 in. This type of flow occurs over relatively short distances, rarely more than approximately 300 ft, but most likely less than 80 ft. Sheet flow rates are commonly estimated using a version of the kinematic wave equation (Reference (2)). The original form of the kinematic wave travel time (in English units) is:

$$T_{t(sheet)} = \frac{0.93}{i^{0.4}} \left(\frac{nL}{\sqrt{S}} \right)^{0.6} \quad \text{(Equation 7.7)}$$

where:

T_t = travel time, minutes

n = roughness coefficient (see Figure 7.13-C)

L = flow length, ft

i = rainfall intensity (in/hour) for a storm that has a return period T and duration of t_c

S = slope of the surface, ft/ft

Surface Description	n[1]
Smooth surfaces (concrete, asphalt, gravel, bare soil)	0.011
Fallow (no residue)	0.05
Cultivated soils:	
Plant residue cover ≤ 20%	0.06
Plant 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 compiled in Reference (8) and are specific to overland and sheet flow.*

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

3 *When selecting n, consider cover to a height of approximately 1 in. This is the only part of the plant cover that will obstruct sheet flow.*

Figure 7.13-C — ROUGHNESS COEFFICIENTS (MANNING'S n) FOR SHEET FLOW

The Rational Method assumes that the storm duration equals the time of concentration. Thus, the time of concentration is entered into the IDF curve to find the design intensity. However, for the Rational Method (Equation 7.5), I depends on t_c and t_c is not initially known. Therefore, the computation of t_c is an iterative process. An initial estimate of t_c is assumed and used to obtain I from the intensity-duration-frequency curve for the locality. The t_c is computed from Equation 7.5 and used to check the initial value of I. If they are not the same, then the process is repeated until two successive t_c estimates are the same.

To avoid the necessity to solve for t_c iteratively, NRCS TR-55 (Reference (8)) uses the following variation of the kinematic wave equation:

$$T_{t(sheet)} = \frac{0.42}{P_2^{0.5}}\left(\frac{nL}{\sqrt{S}}\right)^{0.8} \qquad \text{(Equation 7.8)}$$

where:

P_2 = 2-year, 24-hour rainfall depth, in (see Figure 7.13-D)

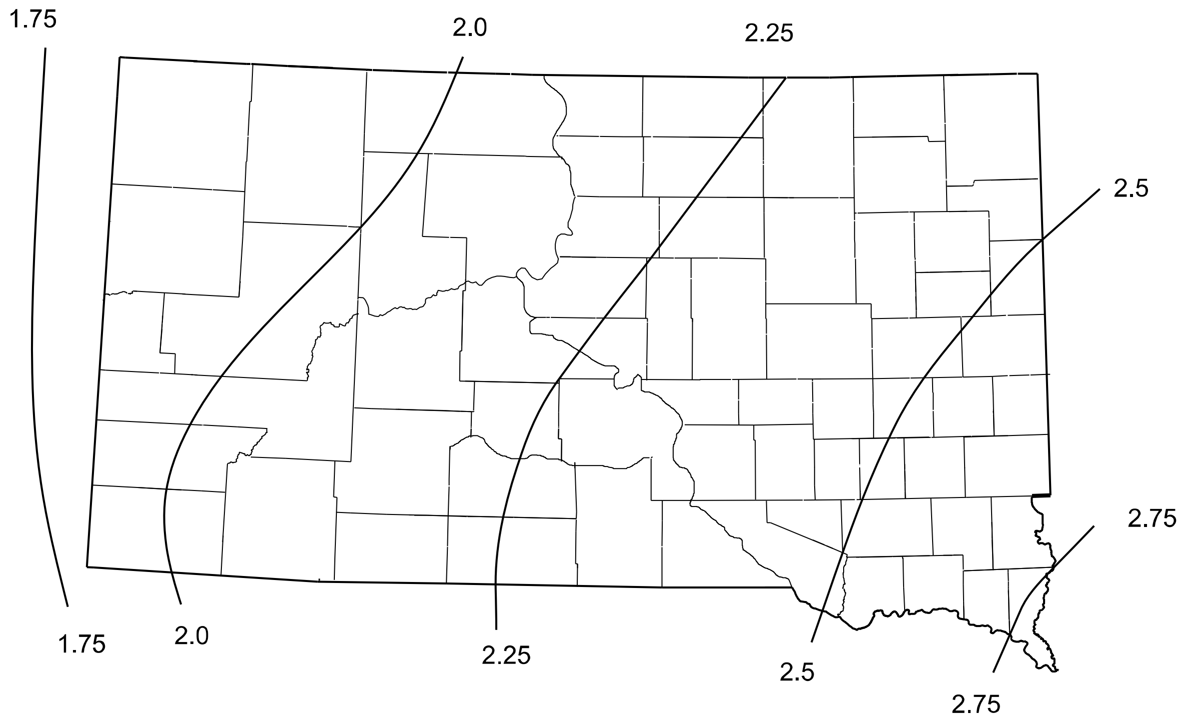


Figure 7.13-D — SOUTH DAKOTA 2-YEAR, 24-HOUR RAINFALL (in) AFTER NOAA TP-40 (See Reference (12))

The other variables are as previously defined. Equation 7.8 is based on an assumed IDF relationship. NRCS TR-55 (Reference (8)) recommends an upper limit of L = 300 ft for using this equation.

7.13.6.4 Travel Time

The velocity equation can be used to estimate travel times for overland flow, shallow concentrated flow, pipe flow or channel flow. It is based on the concept that the travel time (T_t) for a flow segment is a function of the length of flow (L) and the velocity (V):

$$T_{t(type)} = \frac{L}{60V} \qquad \text{(Equation 7.9)}$$

in which L and V have units of ft and fps, respectively. $T_{t(type)}$ has units of minutes and is indicated for overland flow as $T_{t(overland)}$, for shallow concentrated as $T_{t(shallow)}$, for pipe flow as $T_{t(pipe)}$ and for channel flow as $T_{t(channel)}$. The travel time is computed for the principal flow path. Where the principal flow path consists of segments that have different slopes or land covers, the principal flow path should be divided into segments and Equation 7.9 used for each flow segment. The velocity of Equation 7.9 is a function of the type of flow, the roughness of the flow path and the slope of the flow path.

7.13.6.4.1 Velocity for Overland and Shallow Concentrated Flow

After short distances, sheet flow tends to concentrate in rills and then gullies of increasing proportions. Such flow is usually referred to as shallow concentrated flow. The velocity of such flow can be estimated using an empirical relationship between the velocity and the slope:

$$V = 33kS^{0.5} \qquad \text{(Equation 7.10)}$$

in which V is the velocity (fps) and S is the slope (ft/ft). The value of k is a function of the land cover, with values for selected land covers given in Figure 7.13-E.

k	Land Cover/Flow Regime
0.076	Forest with heavy ground litter; hay meadow (overland flow)
0.152	Trash fallow or minimum tillage cultivation; contour or strip cropped; woodland (overland flow)
0.213	Short grass pasture (overland flow)
0.274	Cultivated straight row (overland flow)
0.305	Nearly bare and untilled (overland flow); alluvial fans in western mountain regions
0.457	Grassed waterway (shallow concentrated flow)
0.491	Unpaved (shallow concentrated flow)
0.619	Paved area (shallow concentrated flow); small upland gullies

Figure 7.13-E — INTERCEPT COEFFICIENTS (k) FOR VELOCITY VS. SLOPE RELATIONSHIP OF EQUATION 7.10

After the average velocity is computed using Equation 7.10, the travel time for the channel segment can be calculated from Equation 7.9.

7.13.6.4.2 Velocity for Pipe and Channel Flow

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 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 condition.

Manning's equation is:

$$V = \frac{1.486\, R^{2/3}\, S^{1/2}}{n} \qquad \text{(Equation 7.11)}$$

where:

V	=	average velocity, fps
R	=	hydraulic radius, ft (equal to A/WP)
A	=	cross sectional flow area, sq ft
WP	=	wetted perimeter, ft
S	=	slope of the hydraulic grade line, ft/ft
n	=	Manning's roughness coefficient

After the average velocity is computed using Equation 7.11, the travel time for the channel segment can be calculated from Equation 7.9.

7.13.7 Rainfall Intensity

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

The following South Dakota IDF curves are used in the Rational equation:

- Figure 7.13-G, Locations for IDF Curves in South Dakota;
- Figure 7.13-H, Belle Fourche IDF Curves;
- Figure 7.13-I, Rapid City IDF Curves;
- Figure 7.13-J, Custer IDF Curves;
- Figure 7.13-K, Mobridge IDF Curves;
- Figure 7.13-L, Pierre IDF Curves;
- Figure 7.13-M, Winner IDF Curves;
- Figure 7.13-N, Aberdeen IDF Curves;
- Figure 7.13-O, Huron IDF Curves;
- Figure 7.13-P, Watertown IDF Curves;
- Figure 7.13-Q, Mitchell IDF Curves;
- Figure 7.13-R, Sioux Falls IDF Curves; and
- Figure 7.13-S, Yankton IDF Curves.

HYDRO-35 (Reference (13)), which was published in 1977, was used to calculate the data to construct these IDF curves. The basic data for South Dakota are from 5 stations with average length of record of 60 years and approximately 40 stations with hourly data for the period of 1948 to 1972 (25 years); see Figure 1 of HYDRO-35. In addition to the IDF curves, the rainfall intensity can be read from Figure 7.13-F.

Return Period	Duration (min)	Belle Fourche	Rapid City	Custer	Mobridge	Pierre	Winner	Aberdeen	Huron	Watertown	Mitchell	Sioux Falls	Yankton
2-yr	5	4.2	4.3	4.2	4.5	4.6	4.7	4.7	4.9	5.0	5.0	5.2	5.2
	10	3.2	3.4	3.3	3.6	3.7	3.8	3.8	3.9	4.0	4.0	4.1	4.1
	15	2.6	2.8	2.8	3.0	3.1	3.2	3.2	3.3	3.4	3.4	3.5	3.5
	30	1.6	1.8	1.8	1.9	2.0	2.1	2.1	2.2	2.3	2.3	2.4	2.4
	60	1.0	1.1	1.1	1.2	1.2	1.3	1.3	1.4	1.5	1.5	1.6	1.6
5-yr	5	5.5	5.6	5.5	5.8	5.8	6.0	6.0	6.2	6.2	6.2	6.3	6.3
	10	4.2	4.4	4.4	4.6	4.7	4.9	4.9	5.0	5.1	5.1	5.2	5.2
	15	3.5	3.7	3.7	3.9	4.0	4.1	4.1	4.3	4.3	4.3	4.4	4.4
	30	2.3	2.5	2.5	2.6	2.7	2.8	2.8	2.9	3.0	3.0	3.1	3.1
	60	1.5	1.6	1.6	1.6	1.7	1.8	1.8	1.9	1.9	2.0	2.0	2.0
10-yr	5	6.4	6.5	6.4	6.7	6.7	6.8	6.8	7.0	7.1	7.1	7.1	7.1
	10	4.9	5.1	5.1	5.3	5.5	5.6	5.6	5.8	5.8	5.8	5.9	5.9
	15	4.1	4.3	4.3	4.5	4.6	4.8	4.8	4.9	4.9	4.9	5.0	5.0
	30	2.8	2.9	2.9	3.0	3.1	3.3	3.3	3.4	3.4	3.5	3.6	3.6
	60	1.8	1.9	1.9	1.9	2.0	2.1	2.1	2.2	2.2	2.3	2.4	2.4
25-yr	5	7.7	7.7	7.7	7.9	8.0	8.1	8.1	8.3	8.3	8.3	8.3	8.3
	10	5.9	6.1	6.1	6.3	6.5	6.6	6.6	6.8	6.9	6.9	6.9	6.9
	15	4.9	5.1	5.1	5.3	5.5	5.6	5.6	5.8	5.8	5.8	5.9	5.9
	30	3.4	3.6	3.6	3.7	3.8	3.9	3.9	4.1	4.1	4.2	4.3	4.3
	60	2.2	2.3	2.3	2.4	2.5	2.6	2.6	2.7	2.7	2.8	2.8	2.8
50-yr	5	8.6	8.6	8.6	8.9	9.0	9.1	9.1	9.2	9.3	9.3	9.3	9.3
	10	6.7	6.9	6.9	7.1	7.3	7.5	7.5	7.7	7.7	7.7	7.7	7.7
	15	5.6	5.8	5.8	5.9	6.1	6.3	6.3	6.5	6.5	6.5	6.6	6.6
	30	3.9	4.1	4.1	4.1	4.3	4.5	4.5	4.6	4.6	4.7	4.8	4.8
	60	2.5	2.7	2.7	2.7	2.8	2.9	2.9	3.0	3.0	3.1	3.2	3.2
100-yr	5	9.6	9.6	9.6	9.8	10.0	10.1	10.1	10.2	10.2	10.2	10.2	10.2
	10	7.5	7.6	7.6	7.9	8.1	8.3	8.3	8.5	8.5	8.5	8.5	8.5
	15	6.2	6.4	6.4	6.6	6.8	7.0	7.0	7.2	7.2	7.2	7.3	7.3
	30	4.3	4.6	4.6	4.6	4.8	5.0	5.0	5.1	5.1	5.3	5.3	5.3
	60	2.8	3.0	3.0	3.0	3.2	3.3	3.3	3.4	3.4	3.5	3.6	3.6

Figure 7.13-F — RAINFALL INTENSITY DATA FOR SOUTH DAKOTA

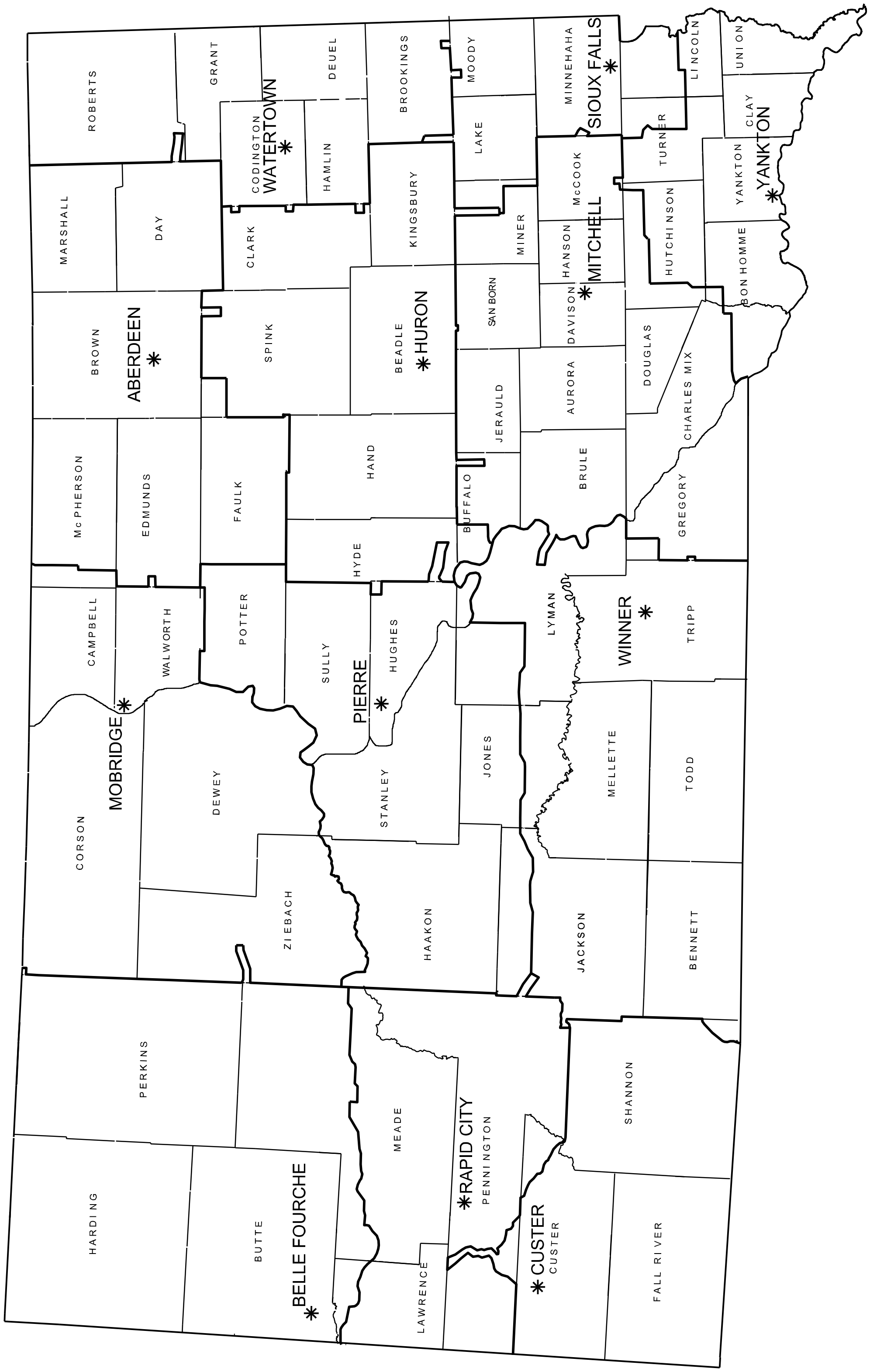
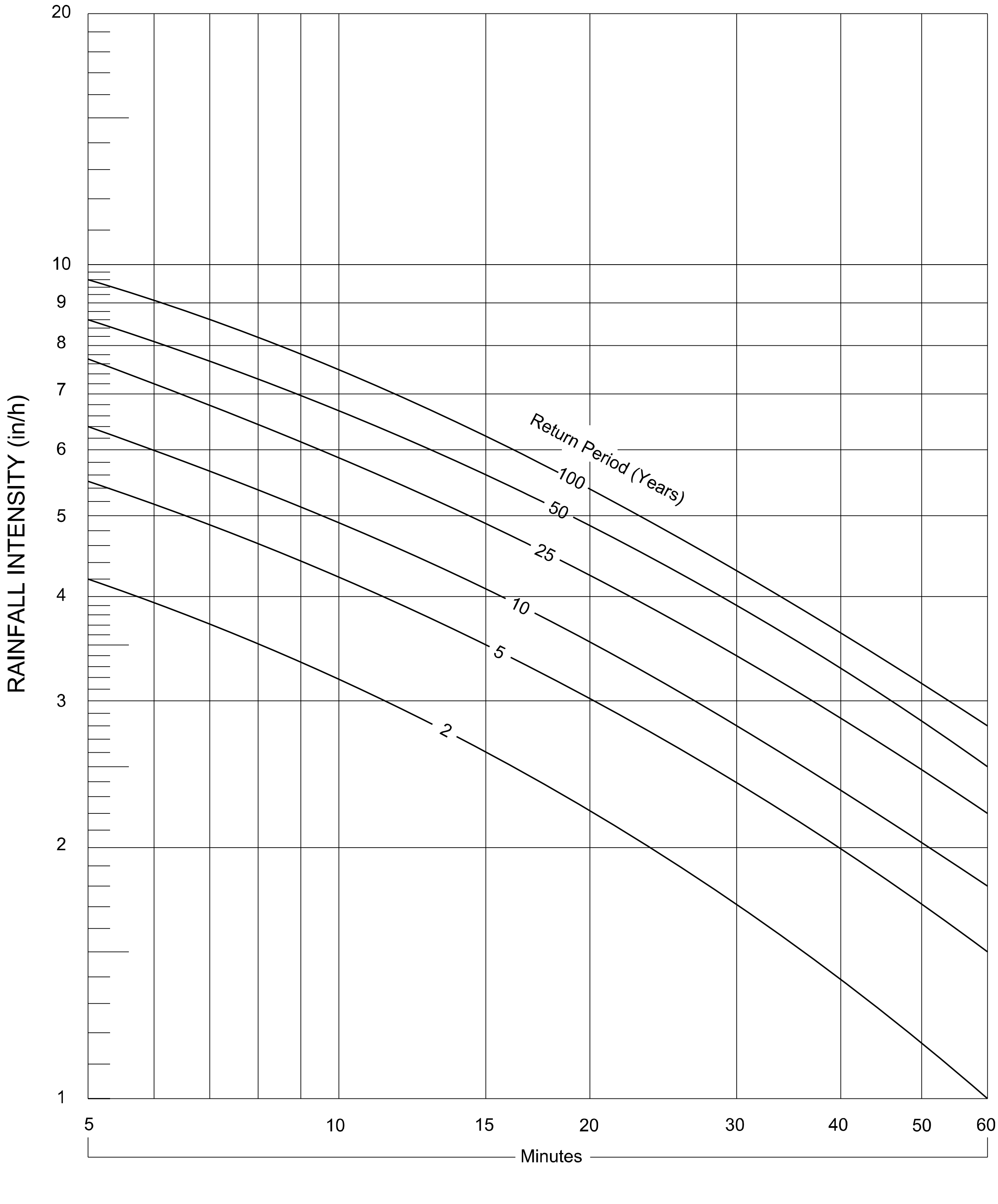
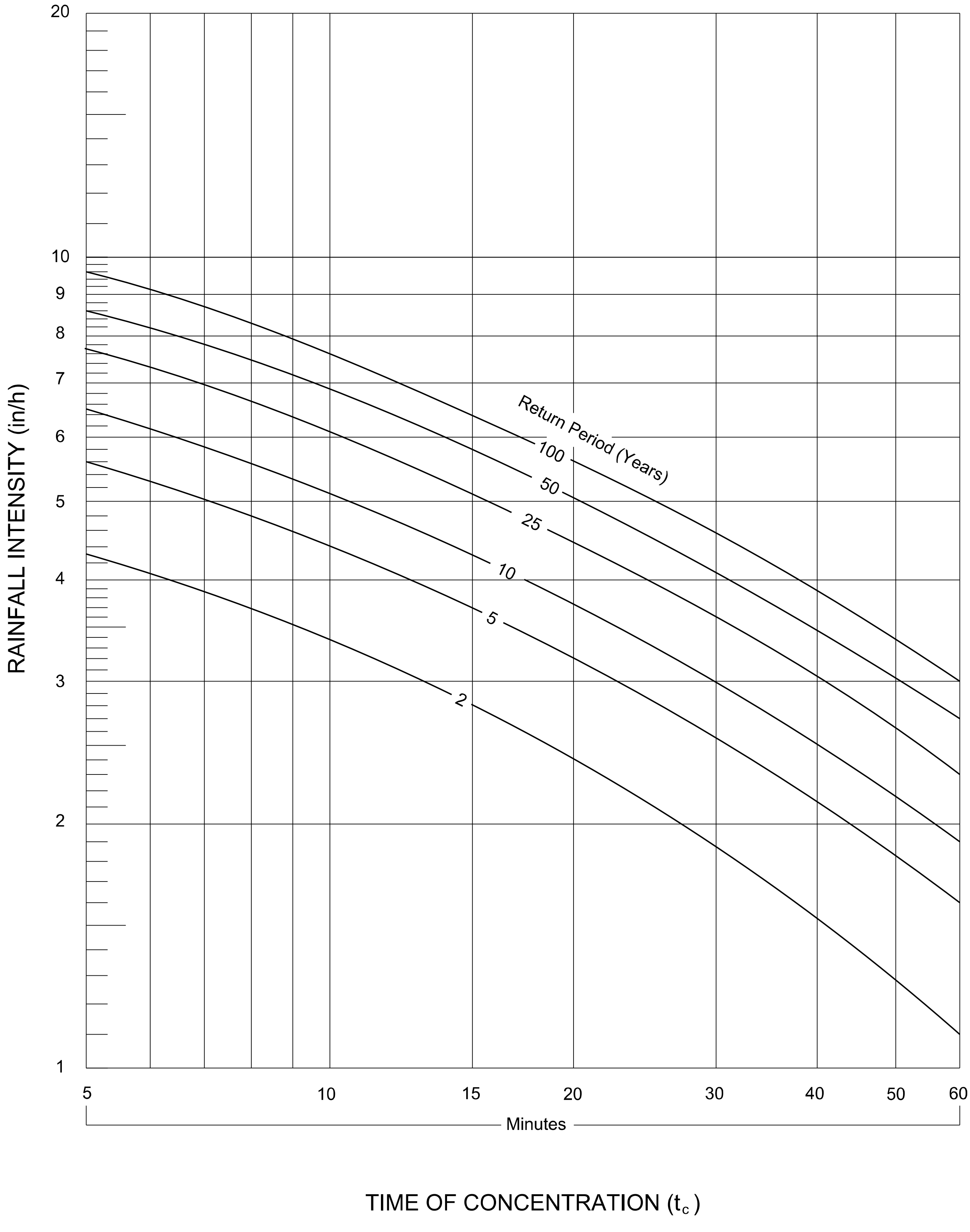


Figure 7.13-G — LOCATIONS FOR IDF CURVES IN SOUTH DAKOTA



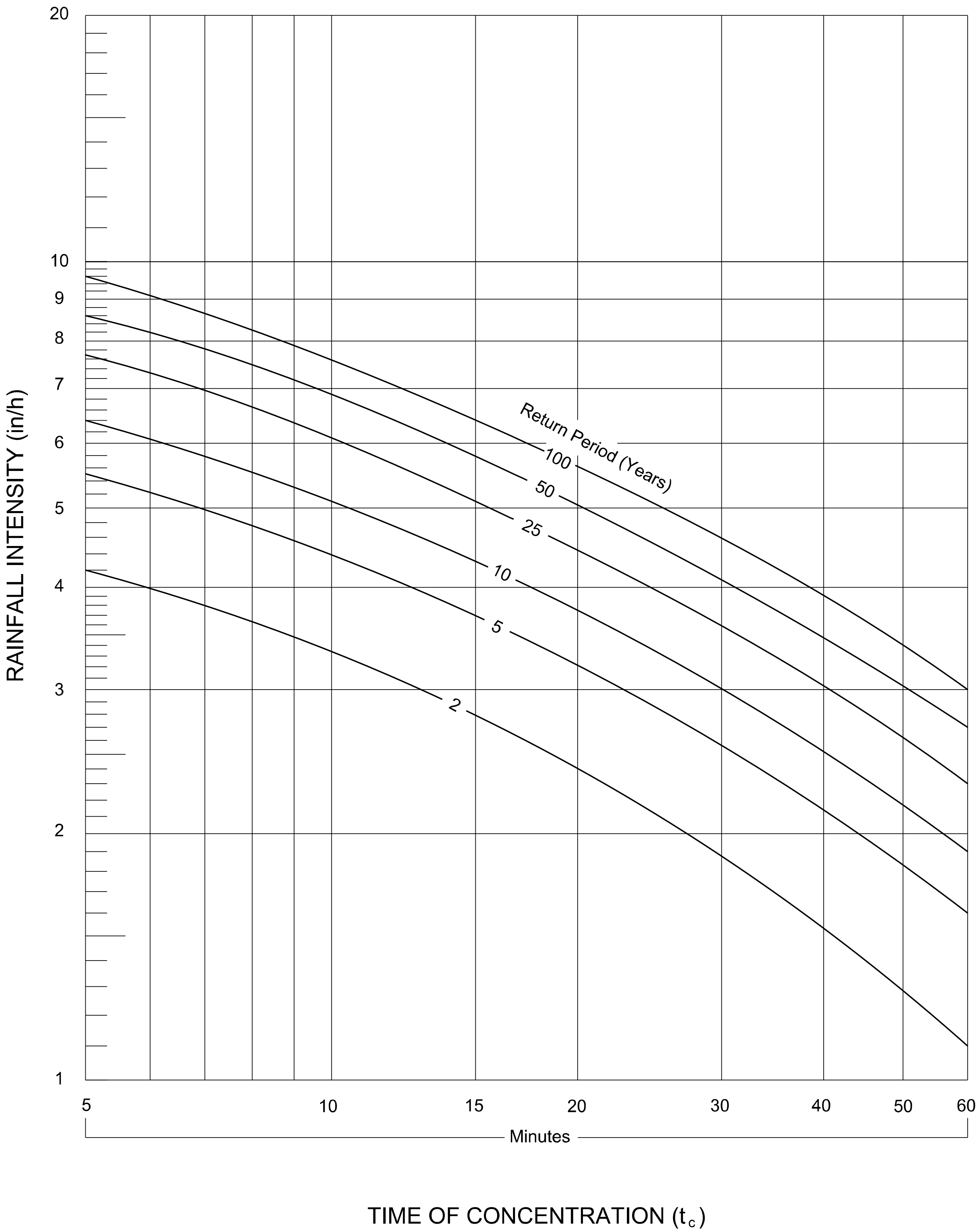
Belle Fourche, SD

Figure 7.13-H — BELLE FOURCHE IDF CURVE



Rapid City, SD

Figure 7.13-I — RAPID CITY IDF CURVE



Custer, SD

Figure 7.13-J — CUSTER IDF CURVE

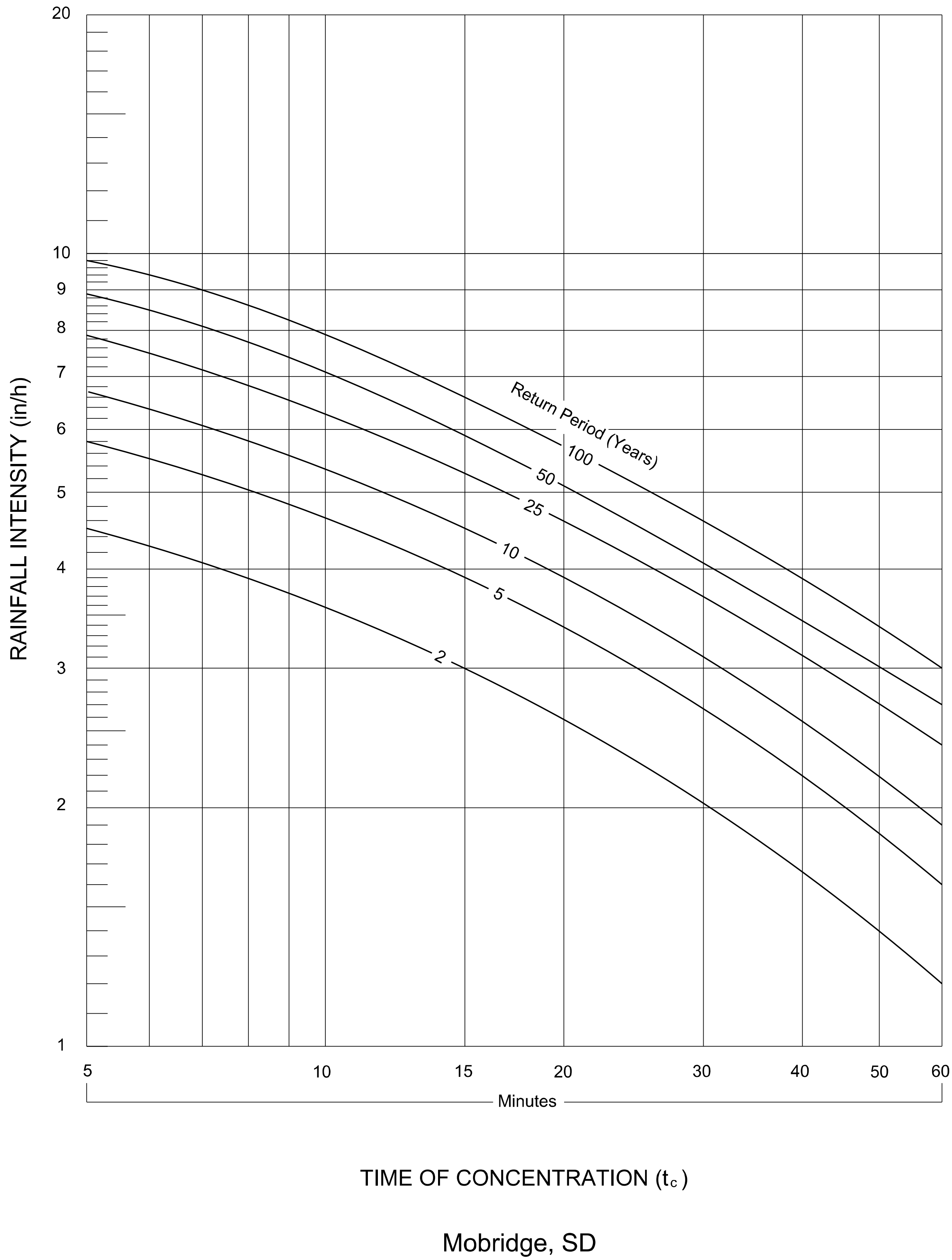
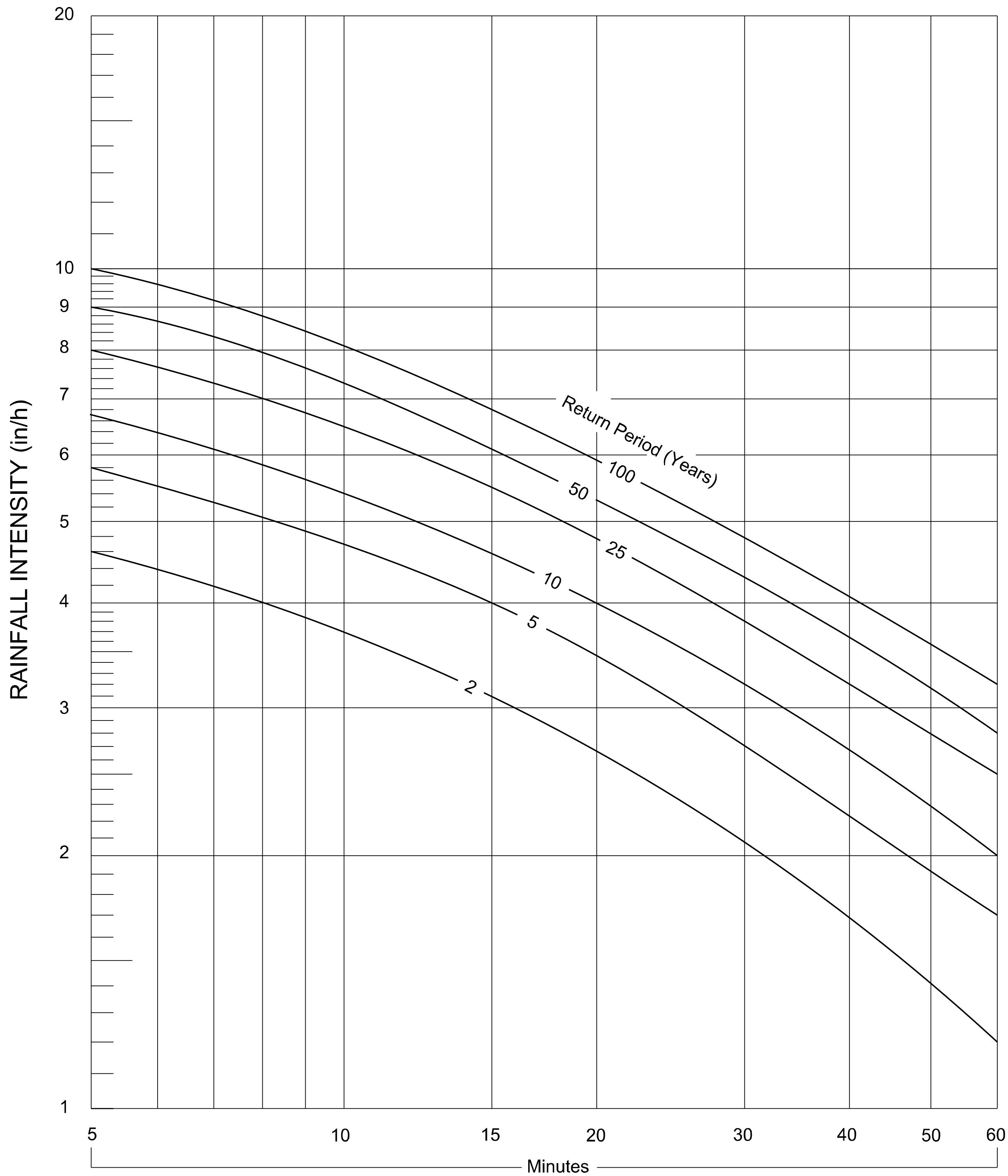


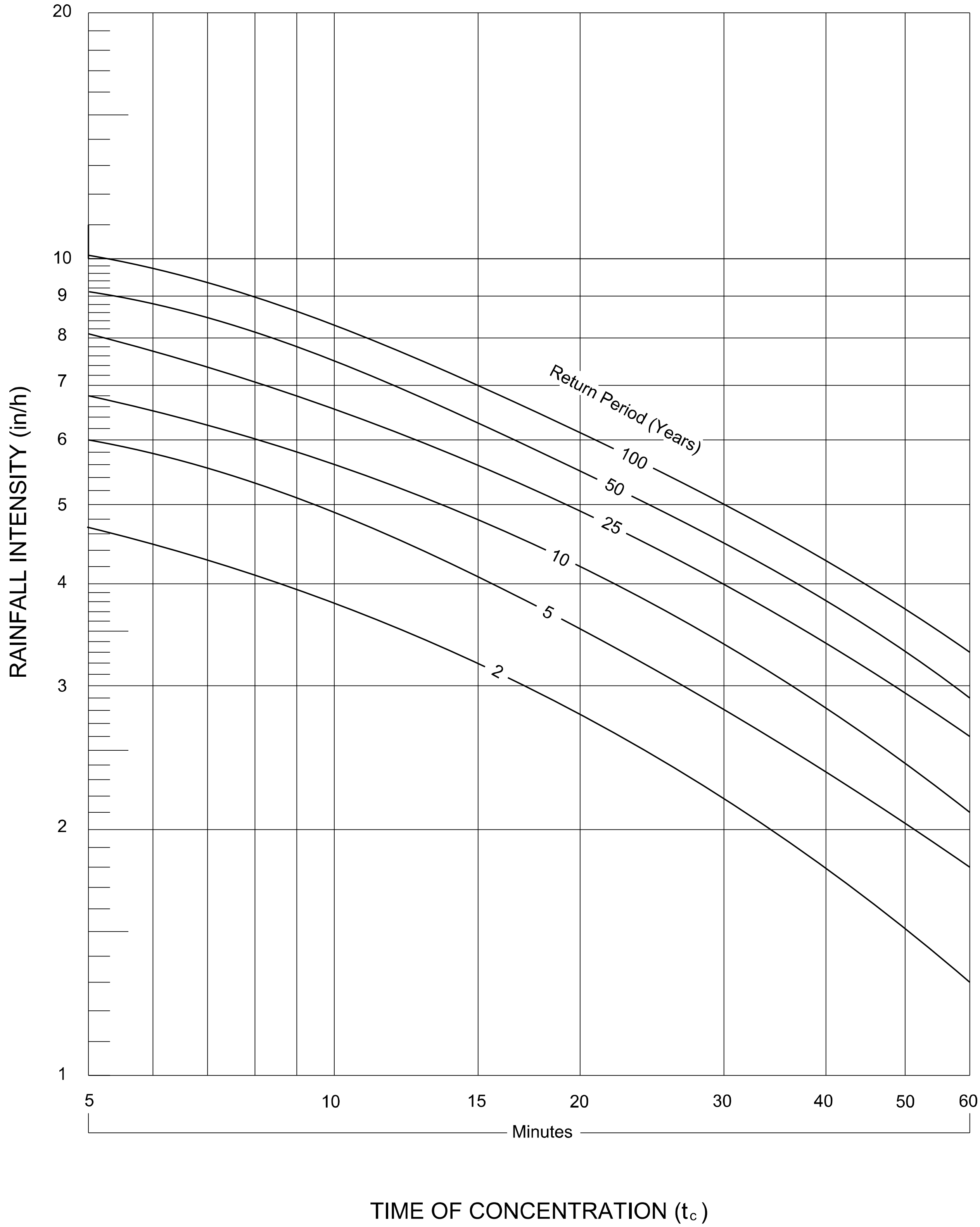
Figure 7.13-K — MOBRIDGE IDF CURVE



TIME OF CONCENTRATION (t_c)

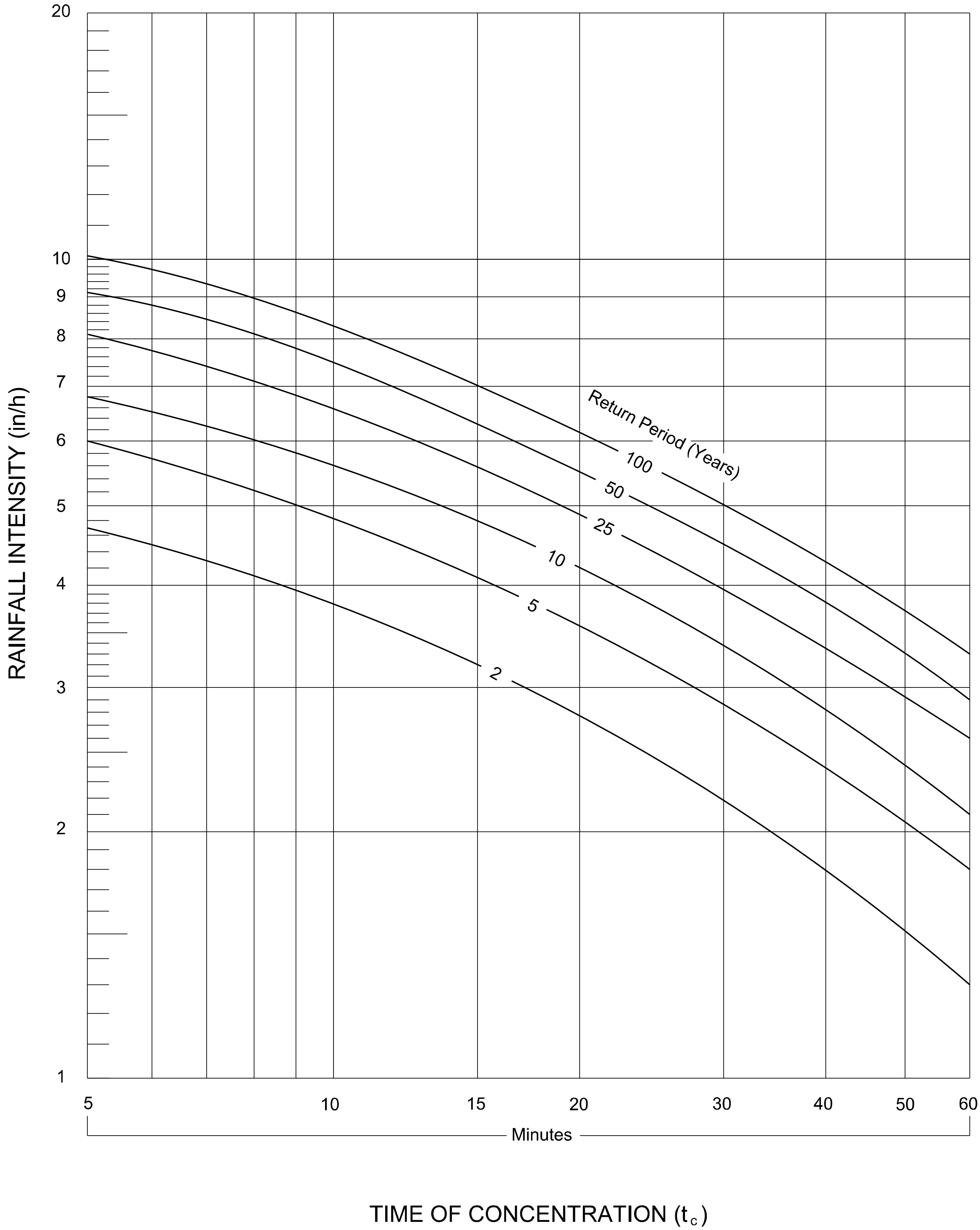
Pierre, SD

Figure 7.13-L — PIERRE IDF CURVE



Winner, SD

Figure 7.13-M — WINNER IDF CURVE



Aberdeen, SD

Figure 7.13-N — ABERDEEN IDF CURVE

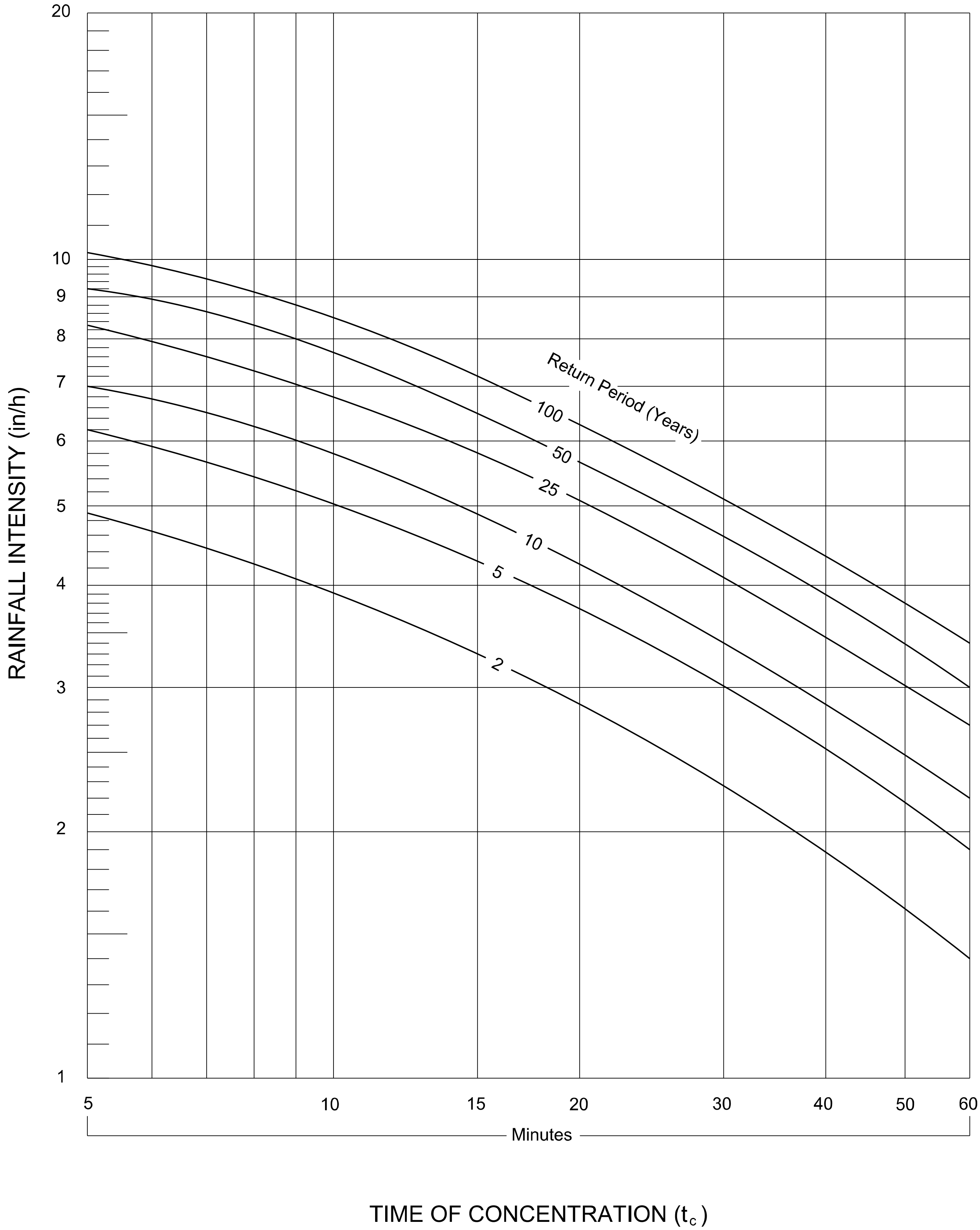
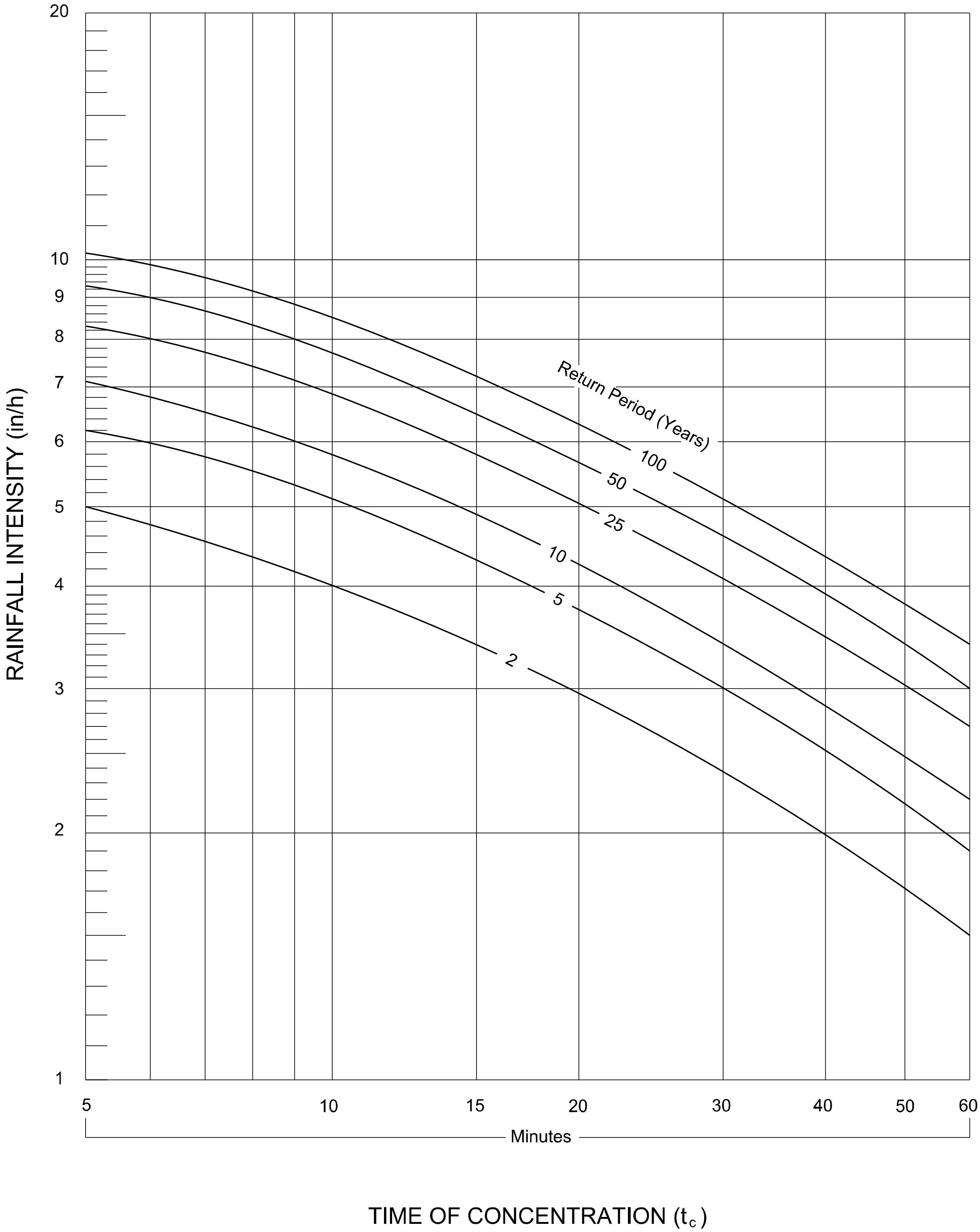


Figure 7.13-O — HURON IDF CURVE



Watertown, SD

Figure 7.13-P — WATERTOWN IDF CURVE

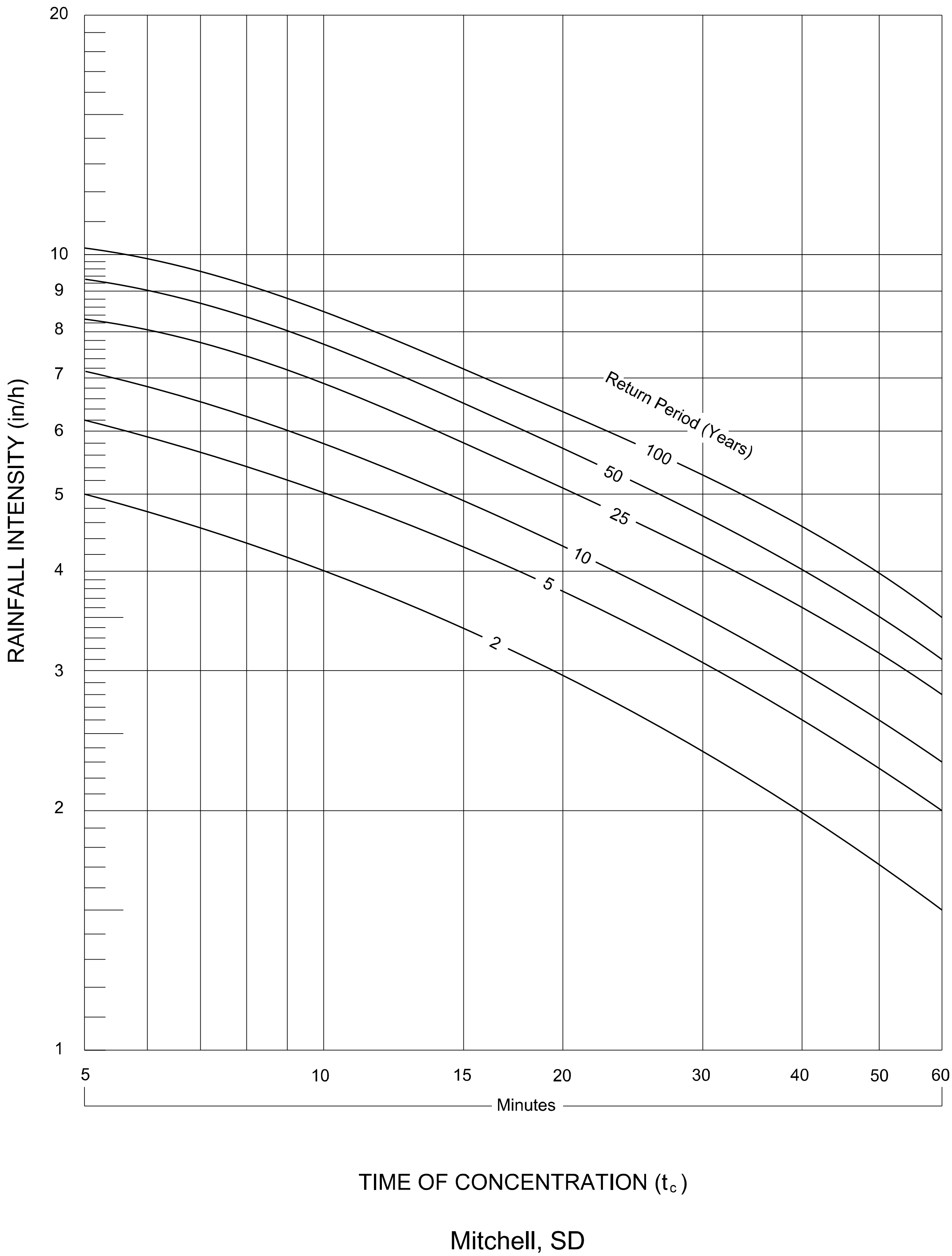
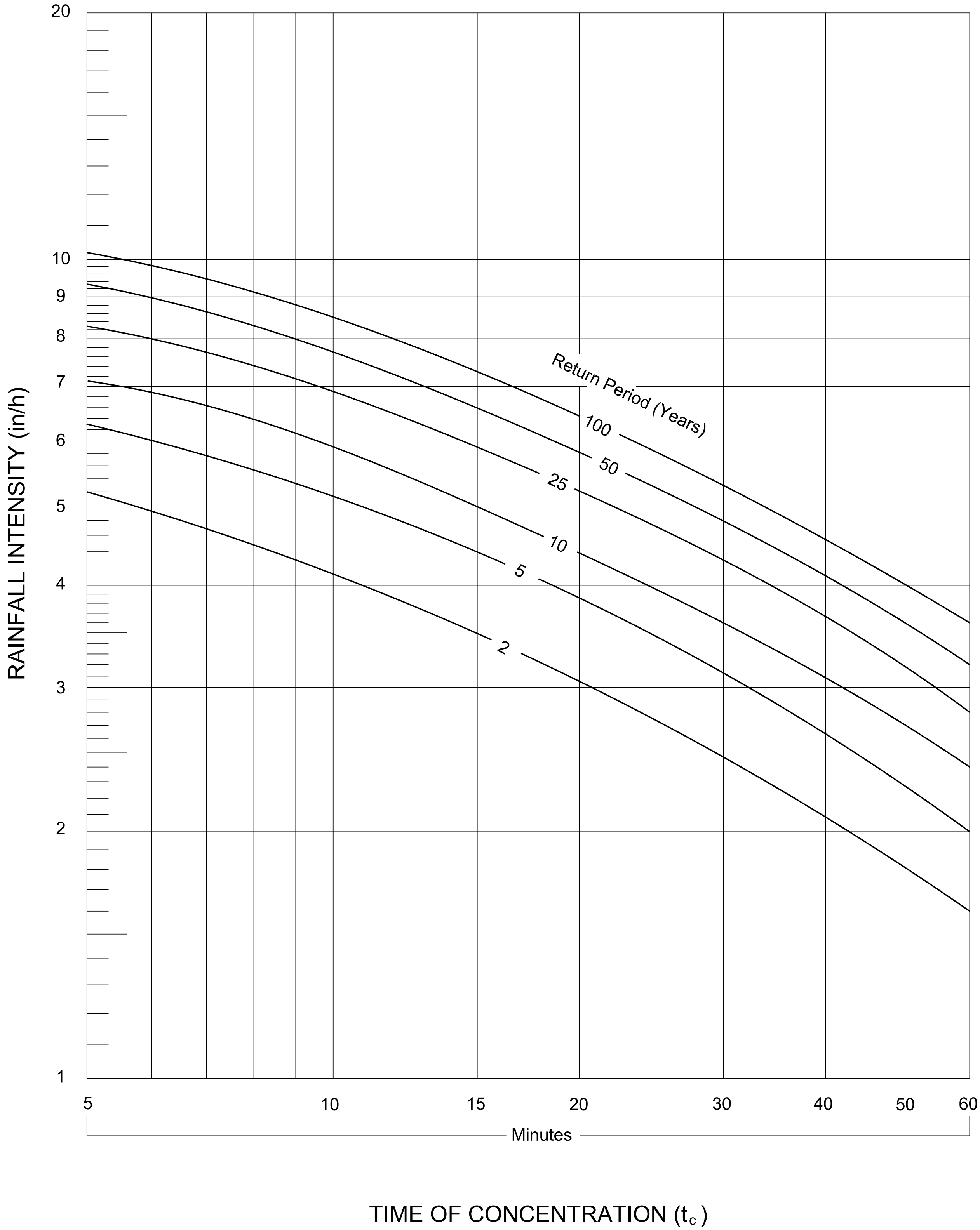


Figure 7.13-Q — MITCHELL IDF CURVE



Sioux Falls, SD

Figure 7.13-R — SIOUX FALLS IDF CURVE

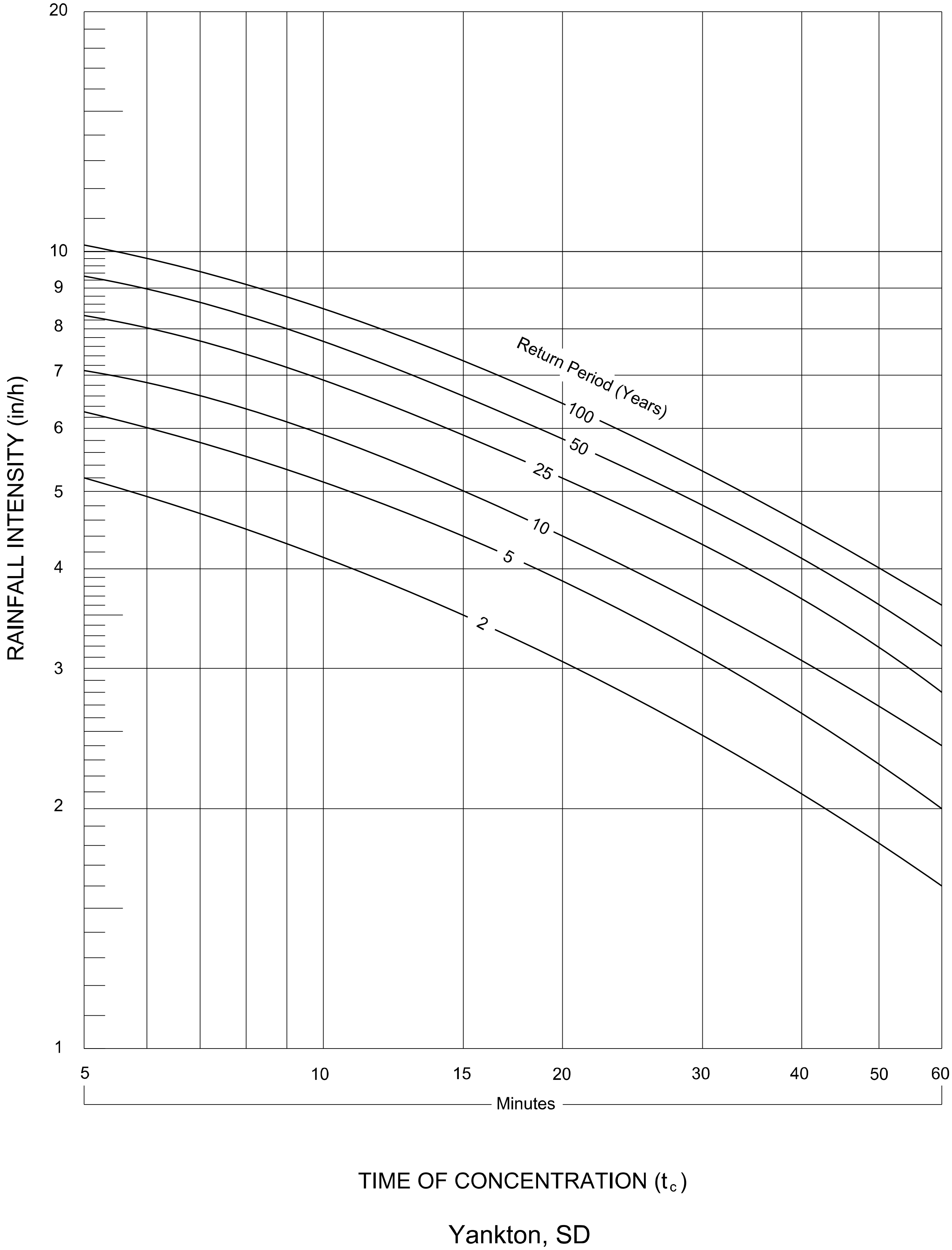


Figure 7.13-S — YANKTON IDF CURVE

7.14 EXAMPLE PROBLEM — RATIONAL METHOD

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

7.14.1 Problem

Estimate the maximum rate of runoff at the inlet to a culvert for the 10-year and 100-year return periods for the following rural site near Rapid City.

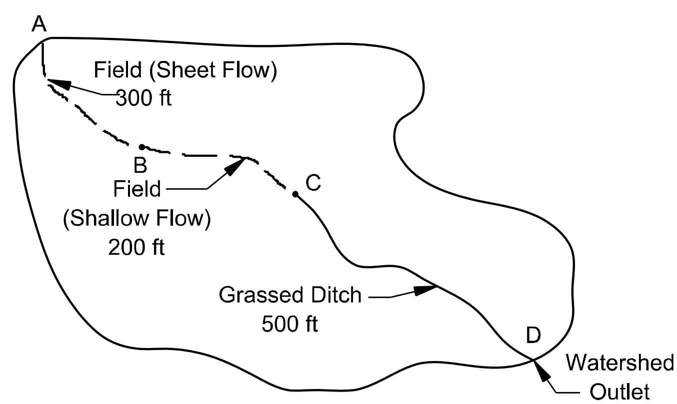


Figure 7.14-A — FIELD NEAR RAPID CITY

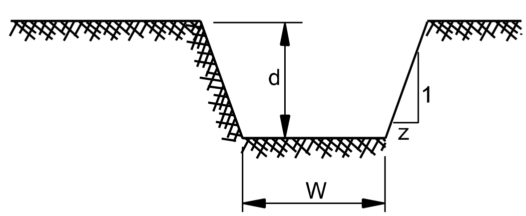


Figure 7.14-B — GRASSED DITCH CROSS SECTION
(z = 5, w = 10 ft, d = 2 ft bank full)

7.14.2 Watershed Characteristics

Site Data

From a topographic map and field survey, the area of the drainage basin upstream from the culvert inlet is 20 acres. In addition, the following data were measured:

- Length of overland (sheet) flow = 500 ft; average overland slope = 0.3% (Assume sheet flow for 300 ft and overland flow for 200 ft)
- Length of grassed waterway = 500 ft; average grassed waterway slope = 0.5%

Land Use

Land use for the drainage basin was observed to be:

- Cultivated fields = 70%
- Slightly pervious soil = 30%

7.14.3 Solution

Step 1 Determine Runoff Coefficient

A weighted runoff coefficient C for the total drainage area is determined in the following table by using the values from Figure 7.13-A:

Land Use	(1) Percent of Total Land Area	(2) Runoff Coefficient (C)	(3) Weighted Runoff Coefficient[1]
Cultivated Fields	70%	0.30	0.21
Slightly Pervious Soils	30%	0.25	0.08
		Total Weighted Runoff	0.29

[1]*Column 3 equals Column 1 multiplied by Column 2.*

An alternative procedure for determining a weighted runoff coefficient is to use CA (Reference (2)). This method is commonly used for storm drainage. See the following table:

Land Use	(1) Runoff Coefficient (C)	Percent of 200 Acres in Land Use	(2) Area (A)	(3) CA
Cultivated Fields	0.30	70%	140	42
Slightly Pervious Soils	0.25	30%	60	15
		Totals	200	57

[1]*Column 3 equals Column 1 multiplied by Column 2.*

Weighted C = (∑ CA)/A = 57/200 = 0.29

Note that both methods provide the same weighted C value.

Step 2 Calculate Time of Concentration

Assume sheet flow for 300 ft from A to B (see Figure 7.14-A), overland flow for 200 ft from B to C and channel flow for the 500 ft of grassed ditch. The travel time is calculated as follows:

a. For sheet flow, use Equation 7.8:

$$T_{t(sheet)} = \frac{0.42}{P_2^{0.5}}\left(\frac{nL}{\sqrt{S}}\right)^{0.8}$$

P_2 = 2 in from Figure 7.13-D

n = 0.06 from Figure 7.13-C for cultivated field (≤ 20% residue cover)

$$T_{t(sheet)} = \frac{0.42}{(2)^{0.5}}\left(\frac{0.06(300)}{\sqrt{0.003}}\right)^{0.8}$$

$T_{t(sheet)}$ = 31 minutes

b. For overland flow, use Equation 7.9; from Figure 7.13-E, k = 0.274 for cultivated straight flow (overland flow):

$V = 33(0.274)(0.3)^{0.5} = 4.95$ fps

$$T_{t(overland)} = \frac{200\text{ ft}}{(4.95\text{ fps})(60\text{ seconds/minute})} = 0.67\text{ minutes}$$

c. For grassed waterway, use Manning's equation and, from Figure 7.13-C, n = 0.15 for short grass prairie:

$$V = \frac{1.486\,R^{2/3}\,S^{1/2}}{n}$$

$$R = \frac{A}{P} = \frac{wd + zd^2}{w + 2d\sqrt{1+z^2}} = \frac{10(2)+5(2)^2}{10+2(2)\sqrt{(1+5^2)}}$$

R = 1.32

$$V = \frac{1.486\,(1.32)^{2/3}(0.005)^{1/2}}{0.15} = 0.84 \text{ fps}$$

$$T_{t(\text{channel})} = \frac{500 \text{ ft}}{(0.84 \text{ fps})(60 \text{ seconds/minute})} = 9.9 \text{ minutes}$$

d. $t_c = T_{t(sheet)} + T_{t(overland)} + T_{t(channel)}$

$t_c = 31 + 1 + 10 = 42$ minutes

Step 3 Determine Rainfall Intensity

From Figure 7.13-I with duration equal to 42 minutes:

i_{10} = 2.4 in/hour
i_{100} = 3.8 in/hour

Step 4 Determine Peak Runoff

From the Rational equation:

Q_{10} = CiA = (0.29)(2.4 in/hour)(20 acres) = 14 cfs
Q_{100} = CiA = (0.29)(3.8 in/hour)(20 acres) = 22 cfs

7.15 HYDROGRAPHS

7.15.1 Applicability

Hydrographs are needed to determine the effect of flow through storage facilities. This Section discusses the NRCS Unit Hydrograph and the Hydrograph from USGS WRI 80-80 (Reference (6)). The Snyder Unit Hydrograph, which is contained in US Army Corps of Engineers software, may also be used.

See Figure 7.7-A for the limitations and applicability of the two hydrograph methods. In general, where either hydrograph is applicable to a site, USGS WRI 80-80 should be the first choice. Other factors to consider include:

- WRI 80-80 is best where there is natural cover (i.e., rural areas). It can still be used where there is a mix of natural and manmade cover.
- In urban areas with little or no natural cover, NRCS must be used.
- To calculate runoff volume, NRCS must be used where the drainage area exceeds 15 sq mi.

7.15.2 Hydrographs on Small Streams in South Dakota

7.15.2.1 General

The dimensionless hydrograph that is contained in USGS Water-Resources Investigation 80-80 "Techniques for Estimating Flood Peaks, Volumes, and Hydrographs on Small Streams in South Dakota" (Reference (6)) should be used if a hydrograph is needed for a rural watershed that is at least 32 acres and less than or equal to 15 sq mi. However, the rural regression peak discharges (Section 7.9) should be substituted for the peak discharge equations contained in WRI 80-80.

7.15.2.2 Flood-Frequency Equations

Estimates of peak discharges should be made using the appropriate rural regression equations in Section 7.9. The runoff volumes for various design frequencies can be computed for natural-flow sites in South Dakota by using the following equations:

$$V_2 = 129\ A^{0.72}\ Si^{-1.74} \quad \text{(Equation 7.12)}$$
$$V_5 = 222\ A^{0.72}\ Si^{-1.69} \quad \text{(Equation 7.13)}$$
$$V_{10} = 296\ A^{0.73}\ Si^{-1.65} \quad \text{(Equation 7.14)}$$
$$V_{25} = 403\ A^{0.75}\ Si^{-1.59} \quad \text{(Equation 7.15)}$$
$$V_{50} = 491\ A^{0.76}\ Si^{-1.54} \quad \text{(Equation 7.16)}$$
$$V_{100} = 589\ A^{0.77}\ Si^{-1.49} \quad \text{(Equation 7.17)}$$

where:

A = contributing drainage area, sq mi
Si = soil-infiltration index from Figure 7.15-A, in

7.15.2.3 Estimation of Flood Hydrograph

The average hydrograph shape for South Dakota streams is defined by the dimensionless set of time and discharge values listed in the following table:

i	**t'_i (time units)**	**q'_i (discharge units)**
1	0	0
2	3	5.6
3	5	13
4	7	25
5	10	49
6	11	57
7	12	60
8	13	59
9	14	55
10	18	38
11	23	23
12	30	12
13	40	5.2
14	50	2.0
15	60	0.5
16	70	0

The average hydrograph is presented in dimensionless units. Determining the likely hydrograph at a particular site requires modifying the time scale (t') and discharge scale (q') of the average hydrograph. Equations (adapted from Craig and Rankl, 1978) used to modify these scales are:

$$t_i = 44.91 \frac{V}{Q} t'_i \qquad \text{(Equation 7.18)}$$

$$q_i = \frac{Q}{60} q'_i \qquad \text{(Equation 7.19)}$$

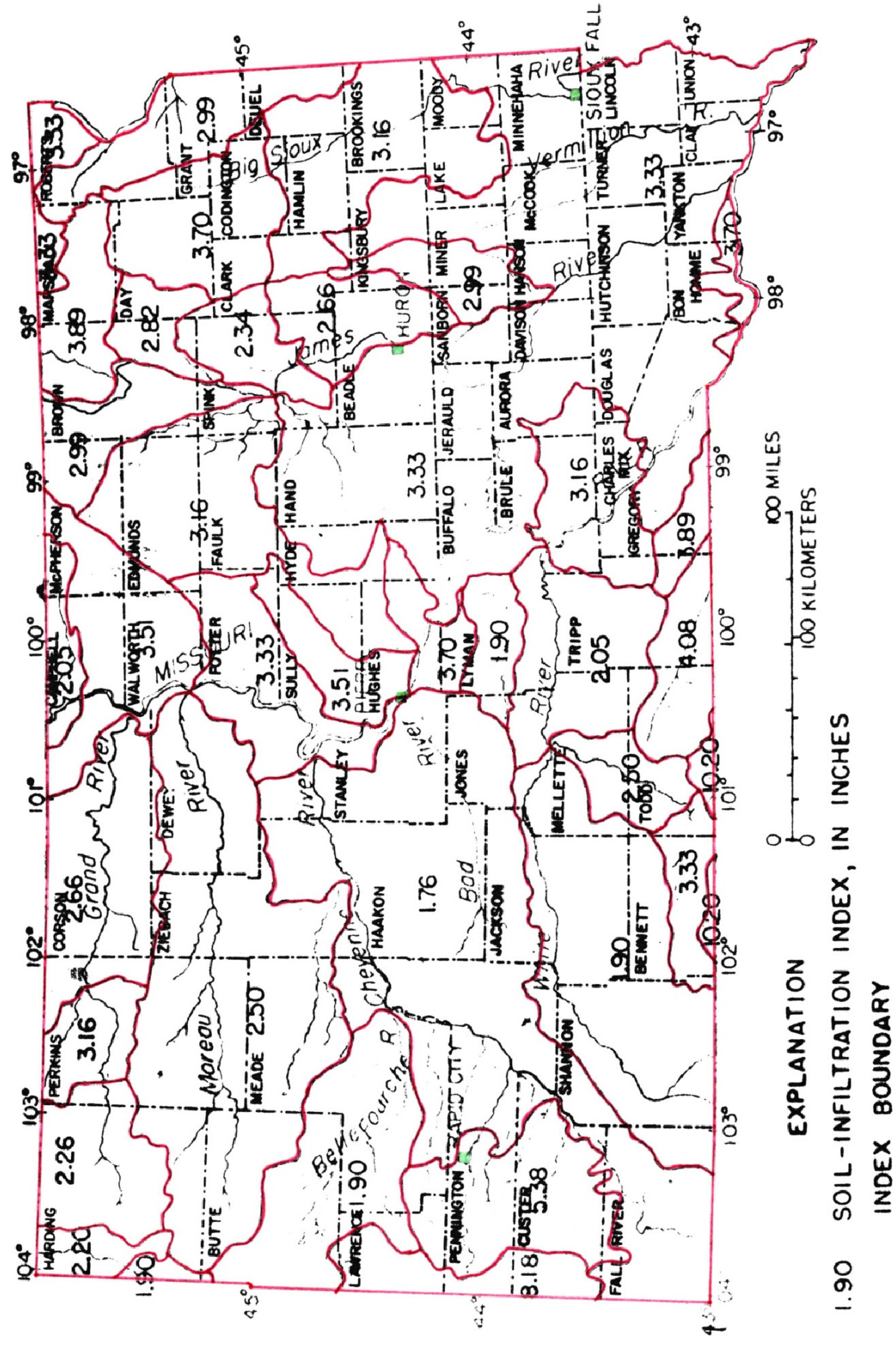


Figure 7.15-A — SOIL-INFILTRATION INDEX

where:

t_i	=	time to instant i from start of runoff, minutes
q_i	=	discharge rate at instant i, in cfs
t'_i	=	dimensionless time value at instant i
q'_i	=	dimensionless discharge rate at instant i
Q	=	peak discharge, cfs
V	=	runoff volume, acre-feet

A complete hydrograph for rain-induced floods of any desired occurrence interval may be defined using estimates of Q and V for the desired recurrence interval to compute all values of t_i and q_i.

7.15.2.4 Example Problem

Determine a synthetic hydrograph for the 25-year flood (1030 cfs) for a 5 sq mi basin on a tributary to the Moreau River (Si = 2.5):

Step 1 Use Q_{25} = 1030 cfs and calculate the V_{25} using Equation 7.15:

$$V_{25} = 403(5)^{0.75}(2.5)^{-1.59} = 314 \text{ acre-feet}$$

Step 2 Use Equations 7.18 and 7.19 to compute instantaneous time (t_i) and discharge (q_i) values needed to define the synthetic hydrograph:

$$t_i = 44.91\frac{V}{Q}t'_i = 44.91\frac{(314)}{(1030)}t'_i = 13.69t'_i \text{ minutes}$$

$$q_i = \frac{Q}{60}q'_i = \frac{1030}{60}q'_i = 17.17q'_i \text{ cfs}$$

Step 3 For i = 3, t'_3 = 5 and q'_3 = 13:

$$t_3 = 13.69t'_3 = (13.69)(5) = 68.5 \text{ minutes}$$

$$q_3 = 17.17q'_3 = (17.17)(13) = 233 \text{ cfs}$$

Required instantaneous data are computed and tabulated as follows:

(i)	Time Units (t'_i)	Time Constant (44.91 V/Q)	Minutes (t_i)	Discharge Units (q'_i)	Discharge Constant (Q/60)	Cubic Feet Per Second (q_i)
1	0	13.69	0	0	17.17	0
2	3	13.69	41	5.6	17.17	96
3	5	13.69	68	13	17.17	223
4	7	13.69	96	25	17.17	429
5	10	13.69	137	49	17.17	841
6	11	13.69	151	57	17.17	979
7	12	13.69	164	60	17.17	1030
8	13	13.69	178	59	17.17	1010
9	14	13.69	192	55	17.17	944
10	18	13.69	246	38	17.17	652
11	23	13.69	315	23	17.17	395
12	30	13.69	411	12	17.17	206
13	40	13.69	548	5.2	17.17	89
14	50	13.69	685	2.0	17.17	34
15	60	13.69	821	0.5	17.17	8.6
16	70	13.69	958	0	17.17	0

Step 4 Plot time (t_i) and discharge (q_i) from previous tabulation to obtain synthetic hydrograph (see Figure 7.15-B).

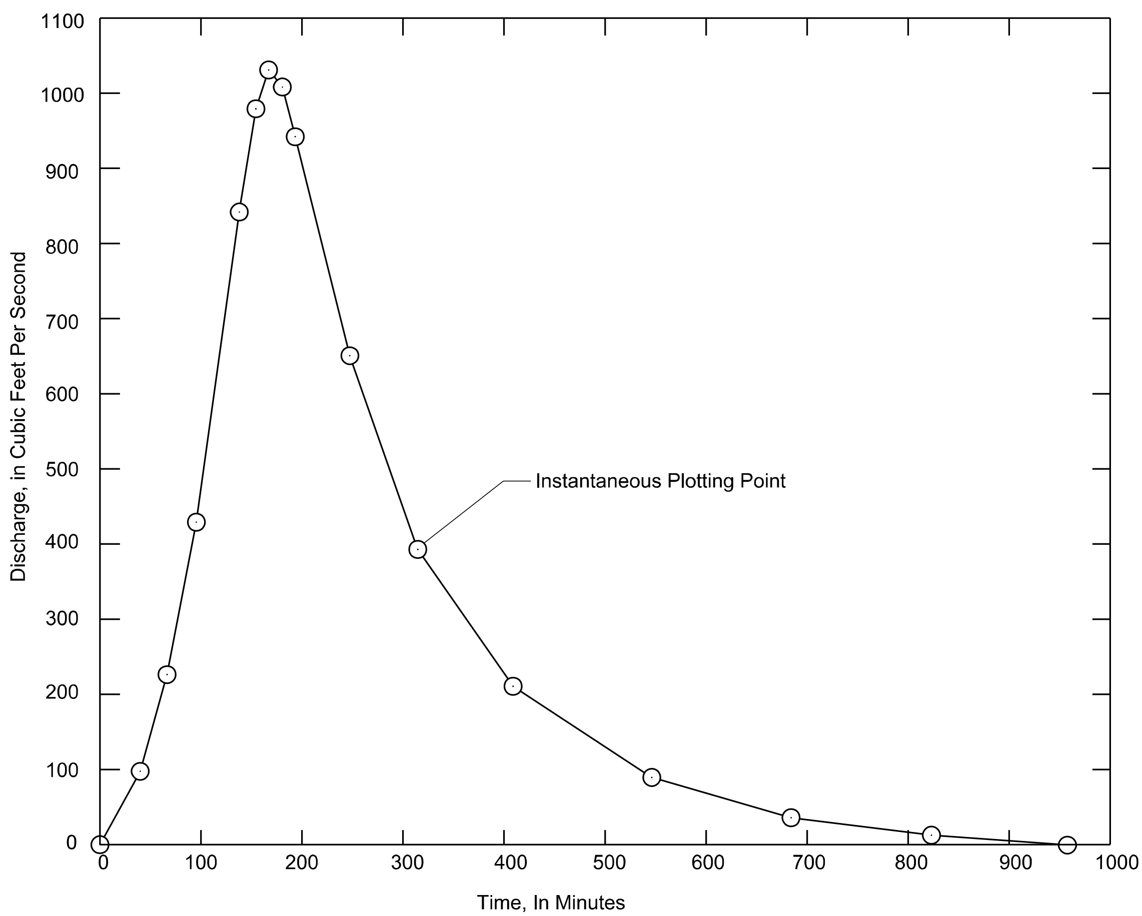


Figure 7.15-B — SYNTHETIC HYDROGRAPH BASED ON MEAN DIMENSIONLESS HYDROGRAPH METHOD

7.15.3 NRCS Unit Hydrograph

7.15.3.1 General

Techniques developed by NRCS 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 NRCS approach, however, is more sophisticated because it also considers the time distribution of the rainfall, the initial rainfall losses to interception and depression storage, and an infiltration rate that decreases during the course of a storm. With the NRCS 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 (NRCS) National Engineering Handbook, Section 4 (Reference (14)) and the associated computer program TR-20 (Reference (7)). The following background is provided so that the designer will use the coefficients appropriate for South Dakota when using the software (see Chapter 18). Example problems can be found in the above references and in HDS 2 (Reference (2)).

7.15.3.2 Equations and Concepts

The following discussion outlines the equations and basic concepts used in the NRCS method:

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

2. Rainfall. The NRCS method is based on a 24-hour storm event that has a Type II time distribution. The Type II storm distribution is a “typical” time distribution that the NRCS has prepared from rainfall records and is applicable to South Dakota.

3. Rainfall-Runoff Equation. A relationship between accumulated rainfall and accumulated runoff was derived by NRCS from experimental plots for numerous soils and vegetative cover conditions. It includes data for land-treatment measures (e.g., contouring, terracing) from experimental watersheds. 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 the total amount of rainfall in a calendar day but not its

distribution with respect to time. The NRCS runoff equation is therefore a method of estimating direct runoff from a 24-hour storm rainfall. The equation is:

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

where:

Q = accumulated direct runoff, in

P = accumulated rainfall (potential maximum runoff), in

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

S = potential maximum retention, in

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

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

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

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

Additional empirical analyses were made to estimate the value of S. The studies found that S was related to soil type, land cover and the hydrologic condition of the watershed. These are represented by the curve number (CN), which is used to estimate S:

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

where:

CN = index that represents the combination of a hydrologic soil group, a land use and treatment class.

The above equations form the basis of the calculations that are performed by the software used to produce hydrographs (see Section 18.2.1). The user will input the appropriate values of CN and P for the above equations plus other parameters to define the watershed to obtain a hydrograph.

7.16 REFERENCES

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(2) Federal Highway Administration, *Highway Hydrology*, Hydraulic Design Series No. 2 (HDS 2), FHWA-NHI-02-001, 2002.

(3) US Geological Survey, Scientific Investigations Report 2008-5104 "Peak-Flow Frequency Estimates Based on Data through Water Year 2001 for Selected Streamflow-Gaging Stations in South Dakota," 2008.

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(5) Sauer, V.B., Thomas, W.O., Stricker, V.A. and Wilson, K.V., "Flood Characteristics of Urban Watersheds in the United States — Techniques for Estimating Magnitude and Frequency of Urban Floods," USGS Water-Supply Paper 2207, 1983.

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(9) US Geological Survey, Water Resources Investigations Report 94-4002, "Nationwide Summary of the U.S. Geological Survey Regression Equations for Estimating Magnitude and Frequency of Floods for Ungaged Sites," 2002.

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(14) Soil Conservation Service, *National Engineering Handbook*, Section 4, “Hydrology,” Natural Resources Conservation Service, Washington, DC, 1972.