Chapter 7

HYDROLOGY

ODOT ROADWAY DRAINAGE MANUAL

November 2014

Chapter 7
HYDROLOGY

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7.1 HYDROLOGIC DESIGN GUIDELINES

7.1.1 Definition

Hydrology is defined as: "The science and study concerned with the occurrence, circulation, distribution and properties of the waters of the earth and its atmosphere, including precipitation, runoff and groundwater." 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 discharge in cubic feet per second (cfs). 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.

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 hydraulics 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 is available on the factors influencing the rainfall-runoff relationship to expect precise solutions.

The following sections summarize ODOT practices that relate to hydrology.

7.1.2 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:

- drainage basin characteristics (e.g., size, shape, slope, land use, geology, soil type, surface infiltration, storage);
- stream channel characteristics (e.g., geometry and configuration, natural and artificial controls, channel modification, aggradation/degradation, ice and debris);
- floodplain characteristics; and
- meteorological characteristics (e.g., precipitation amounts and type, storm cell size and distribution characteristics, storm direction, time rate of precipitation (hyetograph)).

7.1.3 Site Data

Because hydrologic considerations can influence the selection of a highway corridor and the alternative routes within the corridor, the hydraulics 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.” 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.”

7.1.4 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.

7.1.5 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 “Administration” and Chapter 4 “Planning and Location” discuss coordination in more detail.

7.1.6 Sources of Information and Documentation

The type and source of information available for hydrologic analysis will vary from site to site. The hydraulics designer is responsible for determining what information is available and applicable to a particular analysis. Chapter 5 “Data Collection” provides a comprehensive list of data sources.

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

7.1.7 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.

7.1.8 Design Flood Frequency

A design flood frequency should be selected commensurate with the facilities cost, amount of traffic, potential flood hazard to property, expected level of service, political considerations and budgetary constraints, as well as the magnitude and risk associated with damages from larger flood events. The design flood frequencies used in the design of cross/side drain structures and storm sewer systems for different classification of State Highway, as recommended by ODOT, are as shown in Figures 7.1-A and 7.1-B.

Roadway Classification	Exceedence Probability (%)	Return Period (Year)
Interstate, Freeways (Urban/Rural)[1]	2%	50
Principal Arterial	2%	50
Minor Arterial System with AADT > 3000 VPD	2%	50
Minor Arterial System with AADT ≤ 3000 VPD	4%	25
Collector System with AADT > 3000 VPD	4%	25
Collector System with AADT ≤ 3000 VPD	10%	10
Local Road System[2]	20%–10%	5–10

Notes:

1. *Federal regulation requires Interstate highways to be provided with protection from the two percent flood event. Underpasses and depressed roadways should also be designed to accommodate the two percent flood. Where no embankment overflow relief is available, drainage structures should be designed for at least the one percent or 100-yr event.*
2. *At the discretion of the designer, based on Risk Analysis and Design Hourly Volume (DHV).*

Figure 7.1-A — ODOT DESIGN STORM SELECTION GUIDELINES FOR ROADWAY CROSS/SIDE DRAIN STRUCTURES

Roadway Classification (Rural, Suburban and Urban)	Location	Return Period
Freeways and Arterials	on grade	10-year
Freeways and Arterials	at sag	50-year
Collectors	on grade	10-year
Collectors	at sag	10-year
Local Roads and Streets, AADT > 250 VPD	on grade	10-year
Load Roads and Streets, AADT < 250 VPD	on grade	5-year
Local Roads and Streets, any AADT	at sag	10-year

Note: *To lessen the possibility of a pressure flow in the storm drain system, the hydraulics designer should design the inlet and outlet conduit system from the true sump (where all runoff must be handled by the storm sewer system) forward on the 2% return frequency (50-year storm). The tailwater elevation or depth of floor in the receiving stream or culvert should also be considered.*

Figure 7.1-B — ODOT DESIGN STORM SELECTION GUIDELINES FOR STORM DRAINS

7.1.9 Risk Assessment

When roadway overtopping is allowed, hydrologic analysis should include the determination of several flood frequencies for use in the hydraulic design. These frequencies are used to size different drainage facilities so as to allow for an optimum design, which considers both risk of damage and construction costs. ODOT design standards will accommodate most design locations. See Appendix 7.A for situations that should be screened using risk assessment.

7.1.10 Review Flood Frequency

The use of the Review Flood Frequency in the hydraulic analysis of the proposed structure is required only when ODOT needs to comply with State/Federal Regulatory Agency(ies) requirements. See Section 7.4.4 for additional information on Review Flood Frequency.

Remember that the Review Flood Frequency is not and will not be used in designing the size of the structure.

7.2 SYMBOLS AND DEFINITIONS

To provide consistency within this Chapter, the symbols in Figure 7.2-A will be used. These symbols have gained wide use in hydrologic publications.

Symbol	Definition	Units
A	Drainage area	acre, square mile
A	Cross sectional area	square ft
BDF	Basin development factor	—
C	Runoff coefficient	—
C_f	Frequency factor	—
CA	Contributing drainage area	square mile
CN	NRCS-runoff curve number	—
C_v, C_p	Physiographic coefficients	—
d	Time interval	hr
D_m	Mean depth of lake or reservoir	ft
En	Equivalent years of record	years
F_p	Adjustment factor for pond/swamp	—
I	Rainfall intensity	in/hr
IA	Percent of impervious area	%
I_a	Initial abstraction from total rainfall	in
K	Frequency factor	—
L	Length of mainstream to furthest divide or Lag time	ft
L_{ca}	Length of mainstream to centroid	ft
n	Manning's roughness coefficient	—
N	Number of years of flood record	years
P	Accumulated rainfall, rainfall depth (NRCS method)	in
Q	Rate of runoff	cfs
Q	Direct runoff (NRCS method)	in
q	Storm runoff during a time interval	in
R	Hydraulic radius	ft
RL	Regression constant	—
RI	Recurrence interval	years
RQ	Equivalent rural peak runoff rate	cfs
S, SL or Y	Ground slope, main channel slope	ft/ft, ft/mile or %
S	Potential maximum retention storage (NRCS method)	in
ST	Basin storage factor	%
T_B	Time base of unit hydrograph	hr
t_c	Time of concentration	min or hr
T_L	Lag time	hr
T_r	Snyder's duration of excess rainfall	hr
T_t	Travel time	min
UQ	Urban peak runoff rate	cfs
V	Velocity	fps
V	Runoff volume	acre-ft
WP	Wetted perimeter	ft
X	Logarithm of the annual peak	—

Figure 7.2-A — SYMBOLS AND DEFINITIONS

The following definitions are important to hydrologic analyses and apply to most methods. Some of the terms (e.g., AMC and lag time) apply to the NRCS method discussed in Section 7.7.

1. Antecedent Moisture Conditions (AMC). 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. The following AMC conditions are defined and used by NRCS:

   - AMC I: low moisture dry
   - AMC II: average moisture
   - AMC III: high moisture due to heavy rainfall

2. Depression/Retention Storage. Depression/Retention 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 (see NRCS method).

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 river stage is the water surface elevation 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 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* (1) and HDS-2 (2).

7.3 HYDROLOGIC ANALYSIS PROCEDURE

Figure 7.3-A provides the process that should be followed in determining discharges for roadway drainage structures. The following should be considered before the analysis is started:

- If the existing structure is currently a span bridge or a bridge box, ODOT Bridge Division will perform the hydrologic/hydraulic analyses.
- If the existing structure is currently a roadway structure, ODOT Roadway Division will perform the hydrologic/ hydraulic analyses. Sometimes existing structures are undersized and a bridge structure may be necessary. The hydraulics designer should consider the drainage area, estimated design storm peak runoff (Q), FEMA Floodplain Maps and estimate of structure size (e.g., bridge structure if span is equal to or greater than 20 ft; roadway structure if span greater than 20 ft) for hydrologic/hydraulic analyses.
- If there is no existing drainage structure, then the hydraulics designer will make a preliminary determination if the structure will be a roadway structure or a bridge structure. The roadway hydraulics designer should consider the drainage area, estimated design storm peak runoff discharge, FEMA Floodplain Maps and estimate of structure size (bridge structure if span is equal to or greater than 20 ft; roadway structure if span less than 20 ft) for hydrologic/hydraulic analyses.
- The Bridge Division designs overflow structures adjacent to or in the proximity of a bridge structure and detour drainage structures.

Figure 7.3-A presents the steps needed for the hydrologic analysis and the designs that will use the hydrologic estimates.

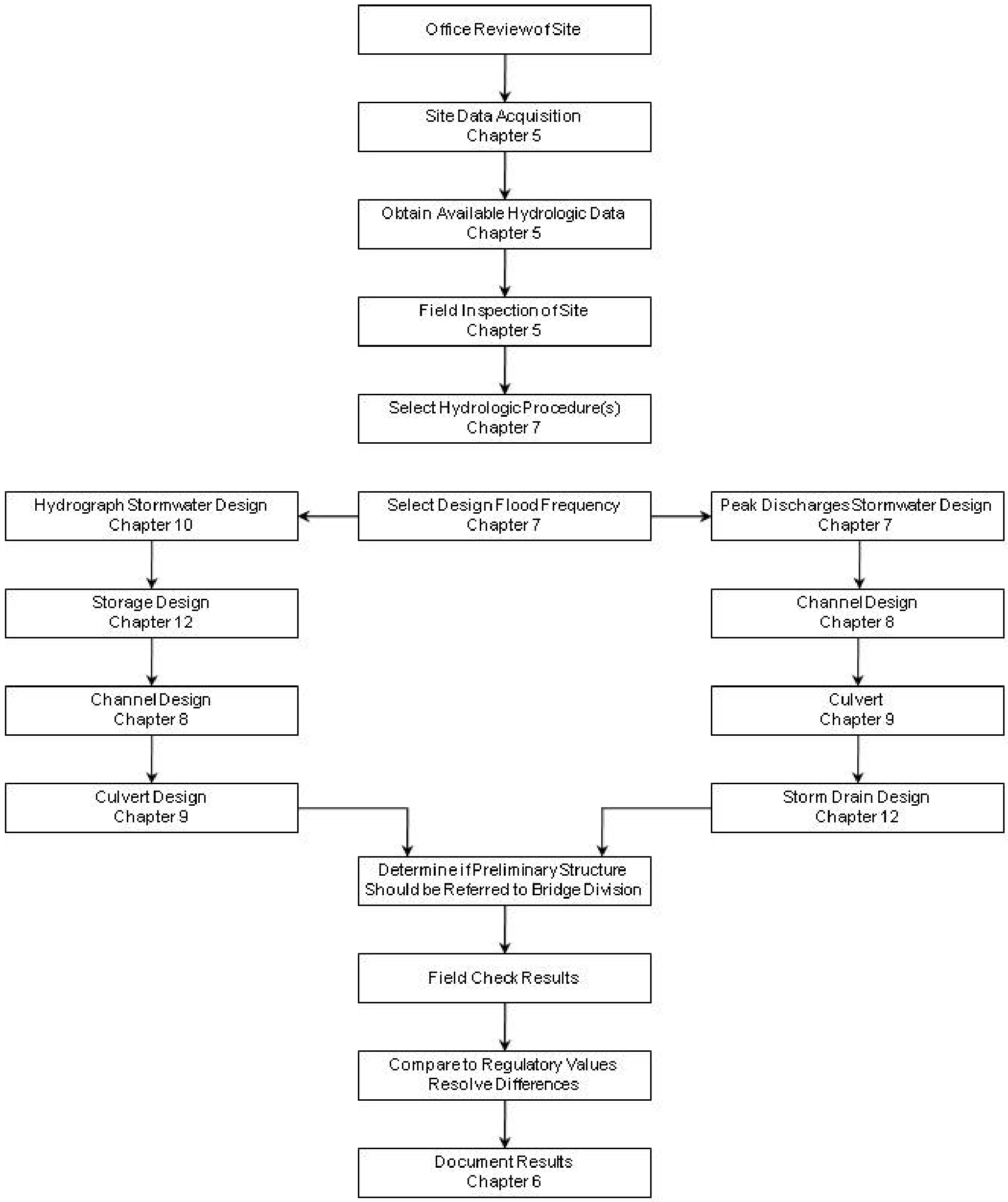


Figure 7.3-A — HYDROLOGIC ANALYSIS PROCEDURE FLOWCHART

7.4 FLOOD FREQUENCY

7.4.1 Overview

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 hydraulics designer should note that the ten-year flood is not one that will necessarily be equaled or exceeded every five 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. The same reasoning applies to floods with other return periods.

7.4.2 Design Flood 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 flood frequency should be selected. The design flood 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 hydraulics designer should consider the potential land use on nearby property that could reasonably occur over the anticipated life of the drainage facility.

Figures 7.1-A and 7.1-B present a guide of recommended design flood frequencies expressed as return periods adopted by ODOT for the various drainage facilities on streets and highways. By definition, roadway overtopping does not occur for the design flood frequency. Consider the following Sections when considering a design frequency.

7.4.2.1 Cross/Side Drain

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

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

Based on these design criteria, a design involving temporary roadway overtopping for floods larger than the design event is acceptable practice.

The cross drain should not overtop unless the design return period is being conveyed. The side drain should convey the design return period but may overtop prior to reaching this conveyance. In no circumstance should a side drain overtop so that it impacts the adjacent roadway.

7.4.2.2 Storm Drains

A storm drain should be designed to accommodate a discharge with a given return period for the following circumstances. The design should be such that the storm runoff does not:

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

7.4.2.3 NFIP-Mapped Floodplains

When the proposed roadway drainage structure is located in FEMA NFIP-Mapped floodplain/floodway, the hydraulics designer should analyze it with the 100-year (base) flood to make sure that the change of the backwater elevation is within limits permitted by FEMA.

7.4.2.4 Risk Assessment/Analysis

When risk assessment is required by 23 CFR 650.115(a), the hydraulics designer should refer to the performance curves analysis of the proposed drainage structure. See Appendix 7.A for more detail.

7.4.3 <u>Design Frequency for Temporary On-Site Traffic Detours/Diversions</u>

The diversion should be designed for the design frequency determined using the risk rating procedure provided in Appendix 7.B. Factors in the estimation process include annual average daily traffic (AADT), loss of life, property damage, alternative detour length, height above streambed, drainage area and traffic interruptions. Where practical, the traffic diversion profile grade should be low enough for overtopping at higher flood frequencies without creating excessive backwater. Where upstream insurable buildings could be affected, the traffic diversion and drainage structures should be sized to prevent an increase in the upstream Q_{100} water surface elevations over existing conditions. FEMA requirements should be met where applicable.

7.4.4 <u>Review Flood Frequency</u>

The hydraulics designer should only use the review flood frequency (overtopping flood or base flood) to perform the analysis of the culvert when:

- a risk assessment or analysis of the culvert is required by 23 CFR 650.115(a), or
- the culvert is located in a FEMA National Flood Insurance Program mapped floodplain.

See Chapter 15 “Permits” to confirm that the backwater caused by the culvert would not exceed 1ft over the existing base (100-year flood elevation).

7.4.5 Rainfall vs Flood Frequency

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

7.4.6 Rainfall Intensity-Duration-Frequency (IDF) Curves

Rainfall data of the State of Oklahoma used for design are obtained from the USGS Water Resources Investigation (WRI) Report 99-4232, Depth-Duration frequency of Precipitation for Oklahoma (3) that was published in 1999.

This detailed data was analyzed by Te Ngo who produced the IDF curves for 8 zones in Oklahoma (4). The Rainfall Intensity-Duration-Frequency (IDF) curves are used in the Rational Method (Section 7.6).

The isohyetal curves for the NRCS method (Section 7.13) for the State of Oklahoma are also based on this same report (3).

7.5 SELECTION OF HYDROLOGIC METHOD

7.5.1 Overview

Estimating peak discharges of various recurrence intervals is one of the most challenging decisions faced by hydraulics 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 10 years of continuous or synthesized data. The estimated discharge will increase in reliability with more data. The State of Oklahoma has numerous gaged locations that can be found at the USGS website: http://waterdata.usgs.gov/ok/nwis/rt. The accepted way to estimate the peak runoff at a gaged site is to use the USGS regression equations (5) or the procedures of Bulletin 17B (6), which are discussed in Section 7.10.

2. Ungaged Sites. The site is not near a gaging station and no streamflow record is available. This situation is very common in Oklahoma. 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 (see Section 7.8).

7.5.2 Peak Flow Rate or Hydrograph

The peak runoff rate for design conditions is generally adequate for highway conveyance systems (e.g., cross drain, side drain, storm drains, open channels). However, if the design will include flood routing (e.g., storage basins, detention/retention pond, complex conveyance networks), a flood hydrograph is usually required. The development of runoff hydrographs (typically more complex than estimating peak discharges) will be based on the NRCS method (see Chapter 16 “Hydraulic Software”). Other synthetic Unit Hydrographs (e.g., Snyder, Clark, hypothetical) could be used only with the approval from the ODOT engineers supervising the project or from the Roadway Drainage Engineer.

7.5.3 Typical ODOT Hydrologic Methods

Figure 7.7-A shows the methods recommended by ODOT in the hydrologic design. These methods are explained in detail in the referenced sections. The hydraulics designer should compare the results calculated with the information obtained from survey books, gaging stations, high water marks, etc. The hydraulics designer should use engineering judgment in choosing the most reasonable result for the design.

Because the drainage area of a Roadway drainage structure is rarely greater than 2000 acres, only three methods: Rational, NRCS and USGS regression equations will be discussed in detail in this chapter.

Watershed Area	Method Recommended	Section
From 0 to less than 200 acres	Rational Method	7.6
From 200 to less than 640 acres	Rational Method NRCS TR-55 Method	7.6 7.7
From 640 acres to 2510 square miles	NRCS TR-55 Method USGS Regression Equations USGS StreamStats Program	7.7 7.8 7.9
Greater than 2510 square miles	Log-Pearson III	7.10

Notes:

1. *The hydraulics designer should contact the ODOT Bridge Division for bridge hydraulic/hydrologic design criteria for watersheds exceeding roadway structures.*

2. *Where two or more methods are recommended, the hydraulics designer should use engineering judgment and field data to determine the best method. All decisions should be documented in the hydraulic/hydrology report, see Chapter 6 “Documentation.”*

Figure 7.5-A — GUIDELINES ON PEAK DISCHARGE DESIGN METHODS

7.6 RATIONAL METHOD

7.6.1 Introduction

The Rational Method, first introduced in 1889, is recommended for estimating the design storm peak runoff for areas up to 640 acres. The Rational Method was modified in the 1980's to include a runoff coefficient correction tied to the flood frequency. This Modified Rational Method is used by ODOT.

7.6.2 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.

7.6.3 Characteristics

Characteristics of the Rational Method that limit its use to 640 acres include:

1. 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. This assumption limits the size of the drainage basin that can be evaluated by the Rational method. For large drainage areas, the time of concentration can be so large that constant rainfall intensities for such long periods do not occur and shorter more intense rainfalls can produce larger peak flows.
2. Peak Discharge Frequency. The frequency of peak discharges is the same as that of the rainfall intensity for the given time of concentration. Frequencies of peak discharges depend on rainfall frequencies, antecedent moisture conditions and the response characteristics of the drainage system. For small and largely impervious areas, rainfall

frequency is the dominant factor. For larger drainage basins, the response characteristics control, for drainage basins with few imperious surfaces (less urban development), antecedent moisture conditions usually govern, especially for rainfall events with a return period of 10 years or less.

3. Runoff. The fraction of rainfall that becomes runoff (C) is independent of rainfall intensity or volume. This assumption is reasonable for impervious areas, such as streets, rooftops and parking lots. For pervious areas, the fraction of runoff varies with rainfall intensity and the accumulated volume of rainfall. Thus, the art necessary for application of the Rational Method involves the selection of a coefficient that is appropriate for the storm, soil and land use conditions.

4. Peak Rate. The peak rate of runoff is sufficient information for the decision. Modern drainage practice often includes detention of urban storm runoff to reduce the peak rate of runoff downstream. With only the peak rate of runoff, the Rational method severely limits the evaluation of design alternatives available in urban and in some instances, rural drainage design. The use of the Rational triangular hydrograph is not recommended in the design of the detention/retention pond.

7.6.4 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:

$$Q = C_f CIA \qquad \text{Equation 7.6(1)}$$

Where:

Q = maximum rate of runoff, cfs

C = runoff coefficient representing a ratio of runoff to rainfall

C_f = 1.0 for 10-year or less recurrence interval
1.1 for 25-year
1.2 for 50-year
1.25 for 100-year

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

A = drainage area tributary to the design location, acres

Note: If the product of C_f and C is greater than 1.0, use 1.0 in equation 7.6(1).

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 hydraulics 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.

7.6.5 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 hydraulics 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 7.6-A for a listing of C values. For variation of C with slope; use the following figures:

Surface Type	Figure
Woodland areas	7.6-B
Pasture and commercial area	7.6-C
Residential areas	7.6-D
Paved areas	7.6-E
Cultivation area	7.6-F

7.6.6 Time of Concentration

The time of concentration is the time required for water to flow from the most remote point on the watershed boundary to the proposed structure. The most remote point is defined in terms of travel time, not distance. t_c is defined as the summation of the time for overland flow, T_o, plus the time for channel or gutter flow, T_f:

$$t_c = T_o + T_f \qquad \text{Equation 7.6(2)}$$

7.6.6.1 Overland Flow

The time of concentration for overland flow, T_o, is the time required for the water to run from the most remote point on the watershed boundary to the beginning point of the channel. Once the overland flow characteristics (distance, slope, ground cover) have been defined, the time of concentration for the overland flow can be computed by Equation 7.6(3):

$$T_o = \frac{k\left(L_o^{0.37}\right)}{S_o^{0.20}} \qquad \text{Equation 7.6(3)}$$

Where:

T_o = Time of concentration for overland flow, minutes

L_o = Length of overland flow path, ft

S_o = Slope of the overland flow path, ft

k = Dimensionless coefficient, a factor of the retardation of the conveyance of the water through the drainage area (overland) and is depending on the overland ground cover

Values of k for various overland ground covers are:

Surface Type	k
Concrete, Asphalt	0.372
Commercial	0.445
Residential	0.511
Rocky, Bare Soil	0.604
Cultivated	0.775
Woodland, Thin Grass	0.942
Average Pasture	1.040
Tall Grass	1.130

7.6.6.2 Channel Flow

The time of concentration for channel flow, T_f, is the time required for water to flow in the channel from the beginning point of the channel to the proposed structure. Once the channel length and slope have been defined, the time of concentration, T_f, for channel flow can be computed by the equation:

$$T_f = \frac{k'\left(L_f^{0.77}\right)}{S_f^{0.385}} \qquad \text{Equation 7.6(4)}$$

Where:

T_f = Time of concentration for channel flow, minutes
L_f = Channel length, ft
S_f = Channel slope, ft/ft
k' = Dimensionless coefficient

Values of k′ for various channel conditions are:

Channel Type	k′
Straight, clean stream	0.00592
Average stream, few obstruction	0.00835
Meandering stream with pools	0.01020
V-ditch	0.01252

7.6.6.3 Minimum Time of Concentration

ODOT has adopted the following for minimum time of concentration for the Rational method only:

- use a minimum t_c = 5 minutes for the densely populated, steep slopes (> 4%) urban areas; and
- use a minimum t_c = 10 minutes for rural areas or well-developed, flat slopes (< 1.00%) urban areas.

7.6.7 Rainfall Intensity

The rainfall intensity (I) is the average rainfall rate (in/hr) 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. Oklahoma IDF curves for 8 zones (see Figure 7.6-G) were developed by Te Ngo (4). The report contains the data used and equations for each curve for each of the following zones:

- Zone 1 – 8 counties (Atoka, Bryan, Choctaw, Latimer, Leflore, McCurtain, Pittsburg and Pushmataha), see Figure 7.6-H
- Zone 2 – 6 counties (Adair, Cherokee, Haskell, McIntosh, Muskogee and Sequoyah), see Figure 7.6-I
- Zone 3 – 7 counties (Craig, Delaware, Mayes, Nowata, Ottawa, Rogers and Wagoner), see Figure 7.6-J
- Zone 4 – 18 counties (Creek, Garfield, Grant, Hughes, Kay, Kingfisher, Lincoln, Logan, Noble, Okfuskee, Okmulgee, Osage, Payne, Pawnee, Pottawatomie, Seminole, Tulsa and Washington), see Figure 7.6-K
- Zone 5 – 19 counties (Caddo, Canadian, Carter, Cleveland, Coal, Comanche, Cotton, Garvin, Grady, Jefferson, Johnston, Love, Marshall, McClain, Murray, Oklahoma, Pontotoc, Stephens and Tillman), see Figure 7.6-L
- Zone 6 – 10 counties (Beckham, Blaine, Custer, Dewey, Harmon, Jackson, Kiowa, Roger Mills and Washita), see Figure 7.6-M
- Zone 7 – 7 counties (Alfalfa, Beaver, Ellis, Harper, Major, Woods and Woodward), see Figure 7.6-N
- Zone 8 – 2 counties (Cimarron and Texas), see Figure 7.6-O

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
Agricultural land:	
Bare, packed soil, smooth	0.30 – 0.60
Bare, packed soil, rough	0.20 – 0.50
Cultivated rows:	
Heavy soil, no crop	0.30 – 0.60
Heavy soil, with crop	0.20 – 0.50
Sandy soil, no crop	0.20 – 0.40
Sandy soil, with crop	0.10 – 0.25
Pasture:	
Heavy soil	0.15 – 0.45
Sandy soil	0.05 – 0.25
Woodland	0.05 – 0.25
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

Source: FHWA HDS 2 (2)

Note: Use lower values for large areas; use higher values for steep slope.

Figure 7.6-A — RUNOFF COEFFICIENTS FOR THE RATIONAL EQUATION

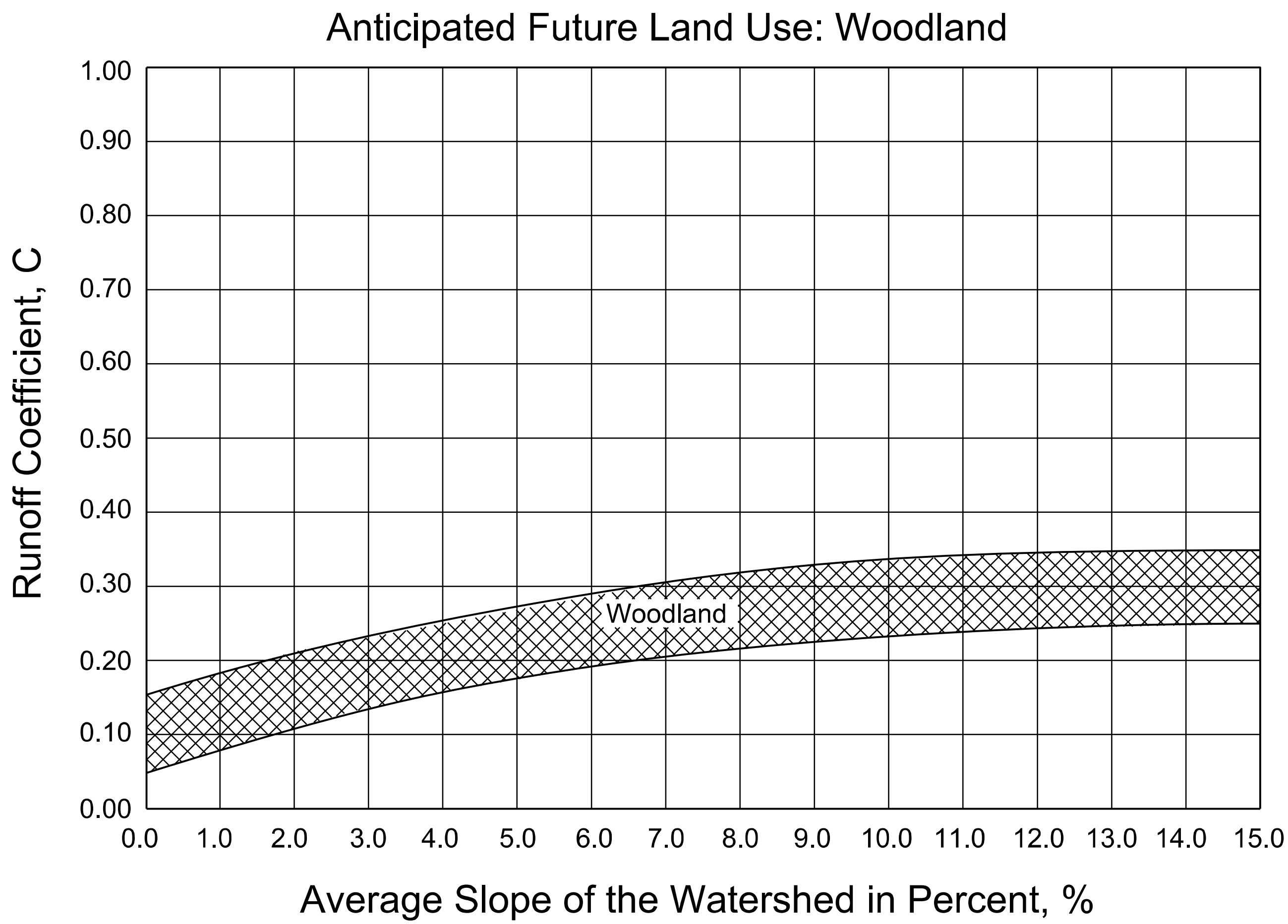


Figure 7.6-B — RUNOFF COEFFICIENT, C, FOR WOODLAND AREA

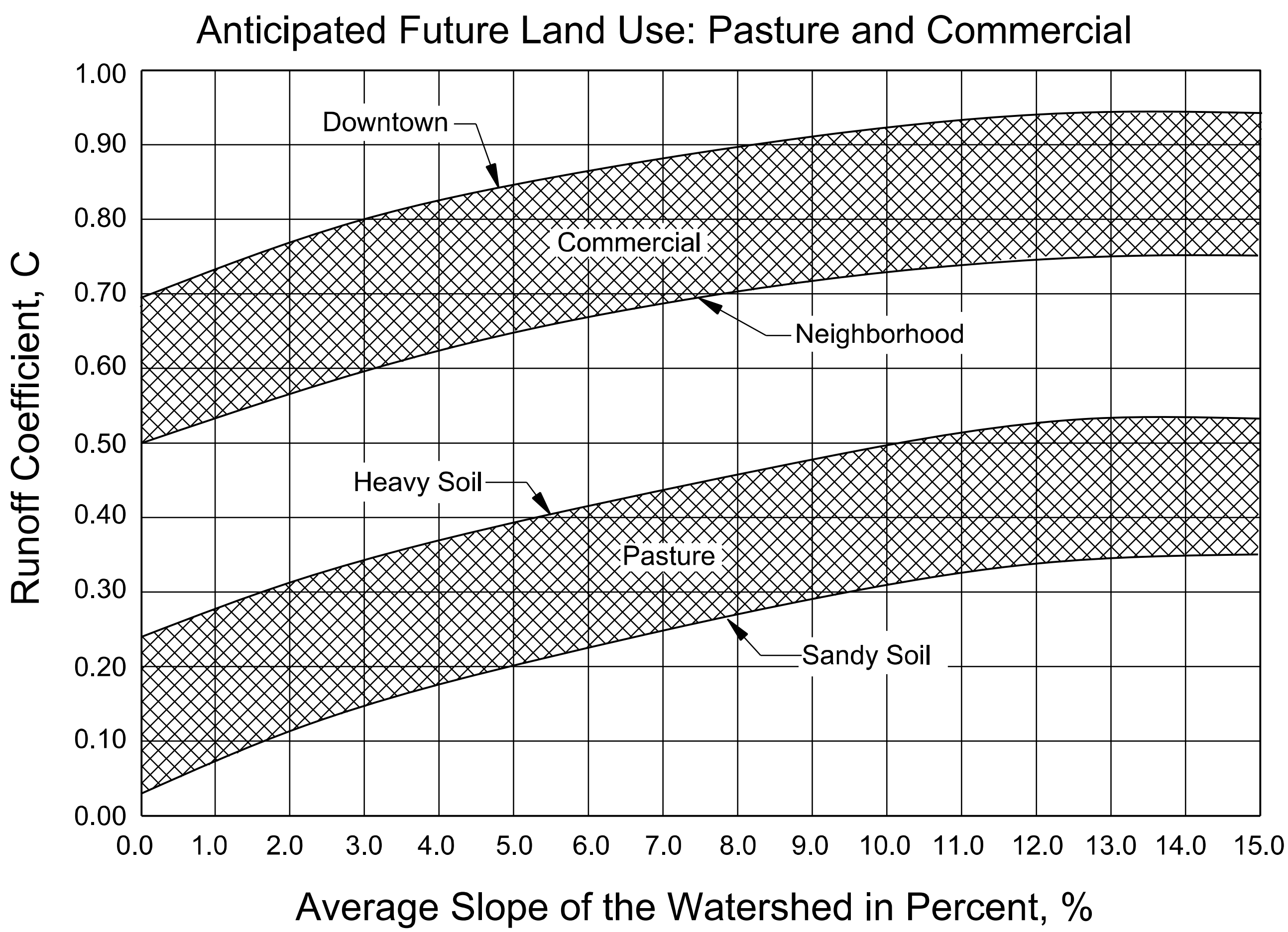


Figure 7.6-C — RUNOFF COEFFICIENT, C, FOR PASTURE AND COMMERCIAL AREA

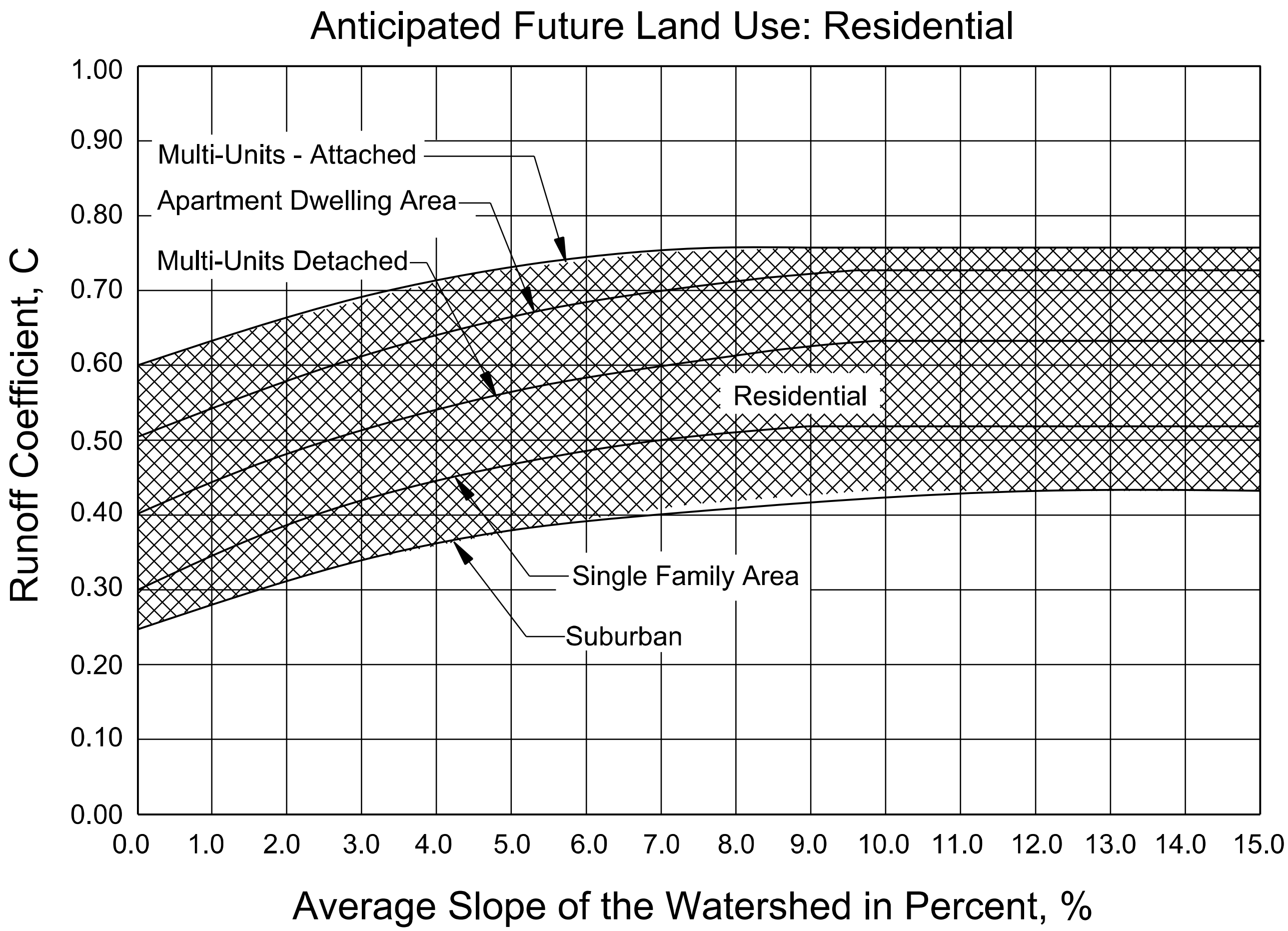


Figure 7.6-D — RUNOFF COEFFICIENT, C, FOR RESIDENTIAL AREA

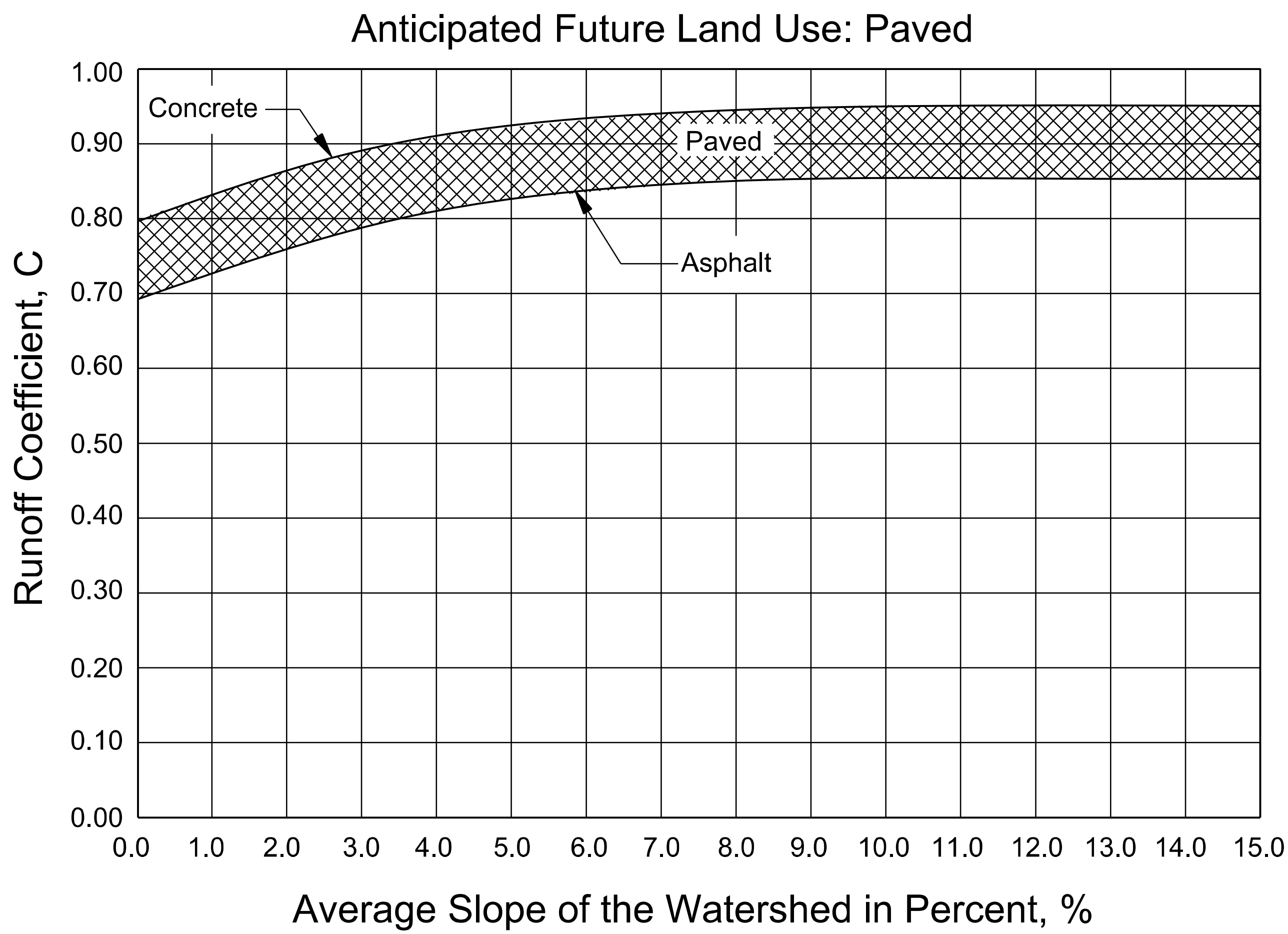


Figure 7.6-E — RUNOFF COEFFICIENT, C, FOR PAVED AREA

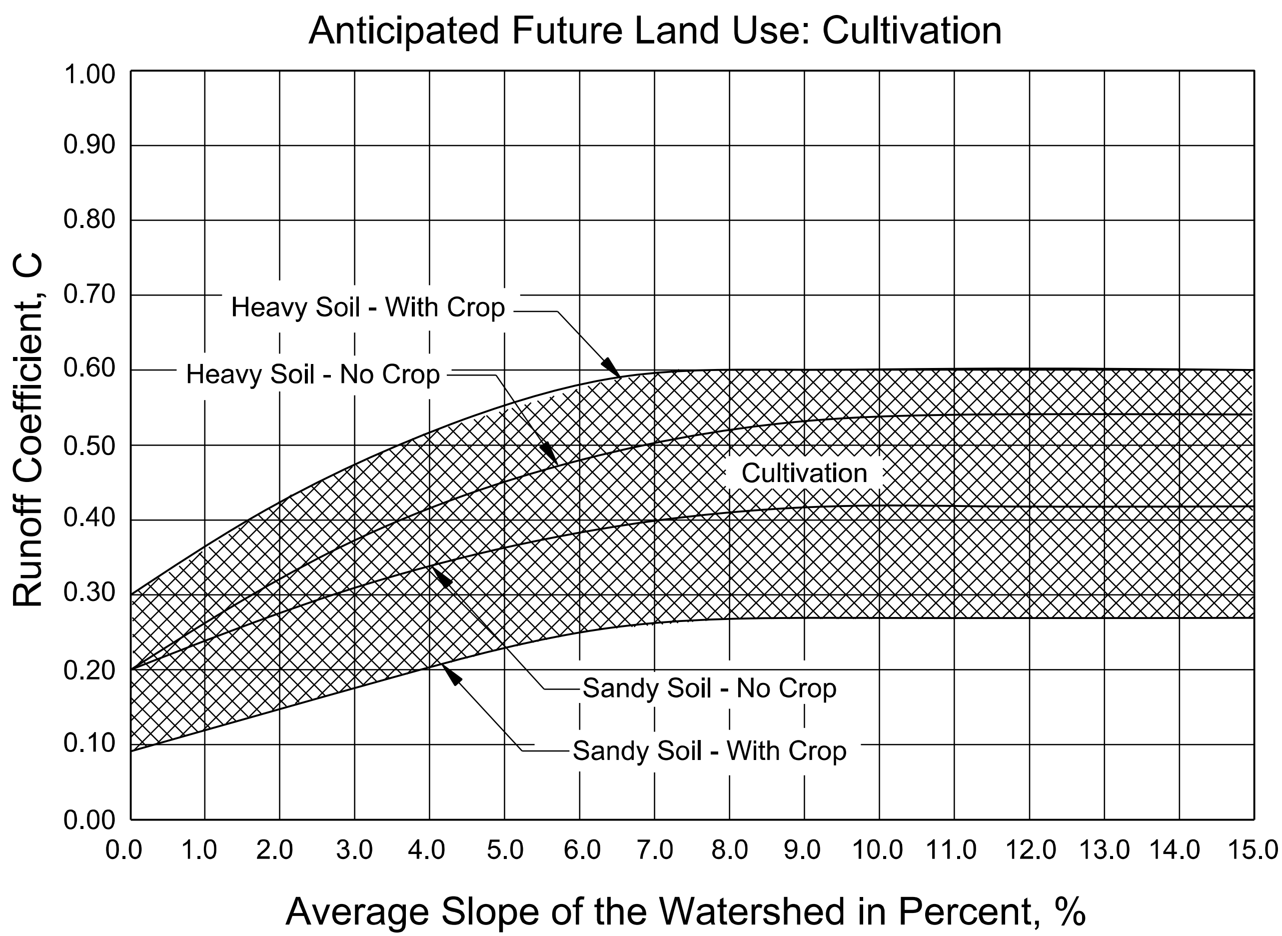


Figure 7.6-F — RUNOFF COEFFICIENT, C, FOR CULTIVATION AREA

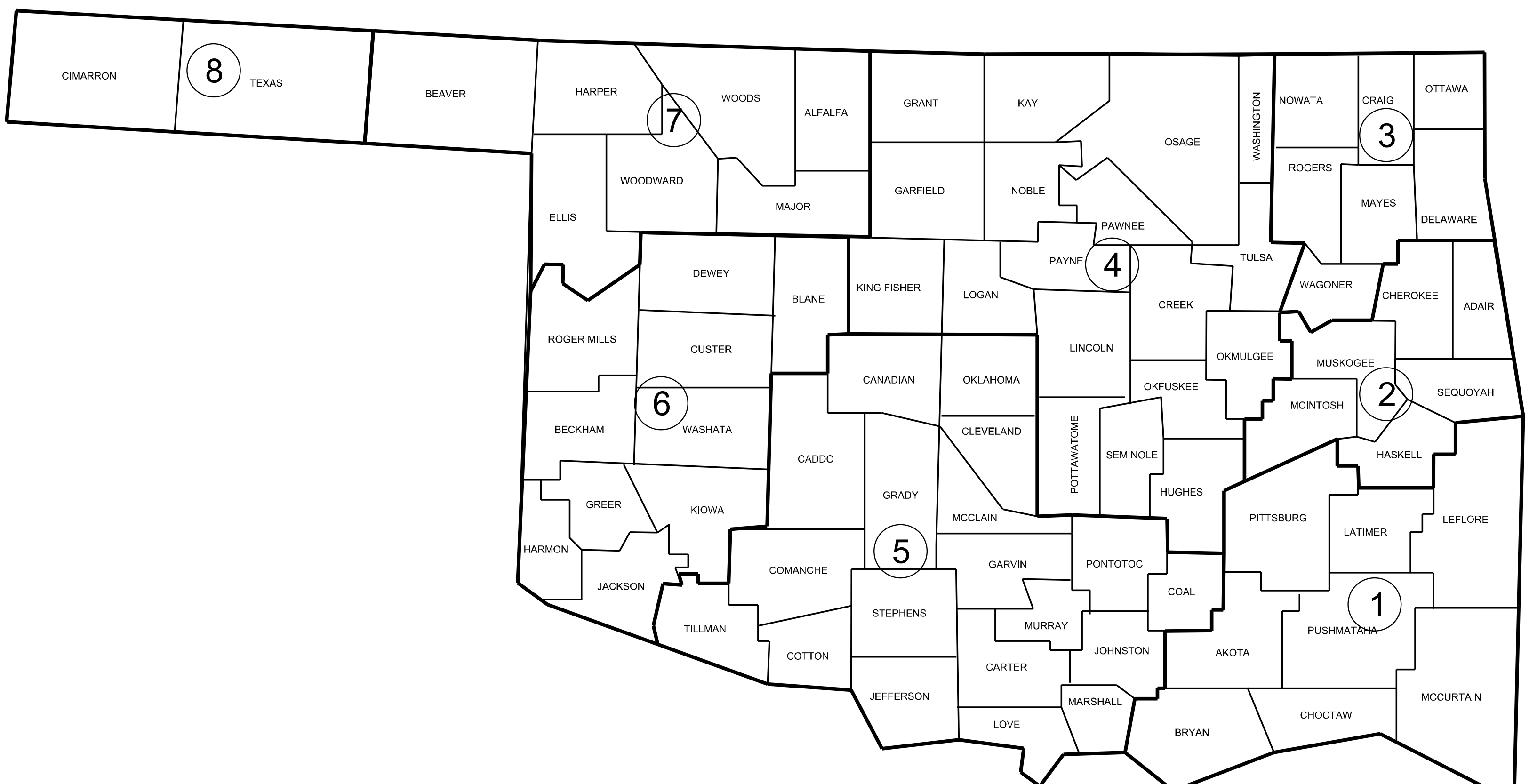
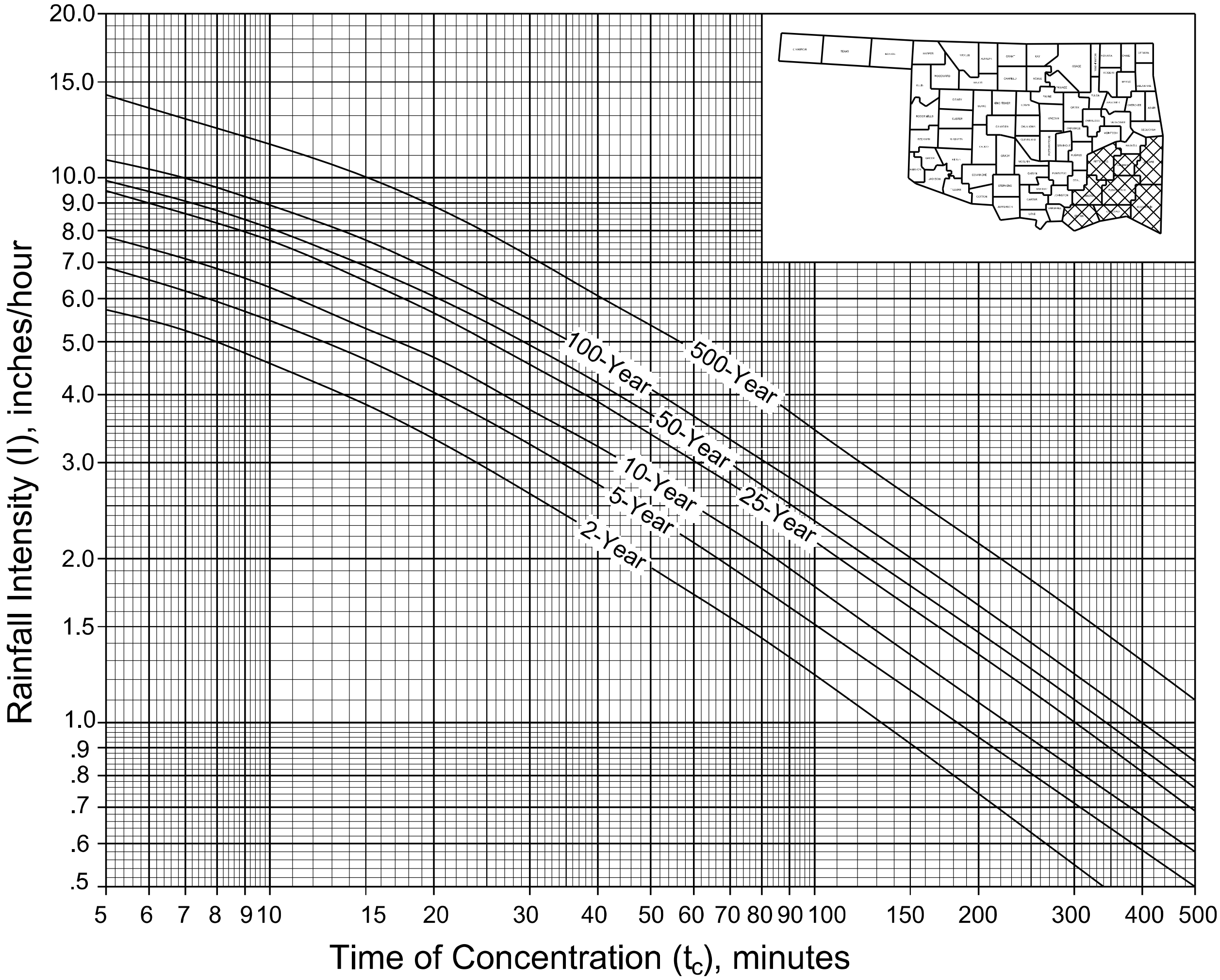


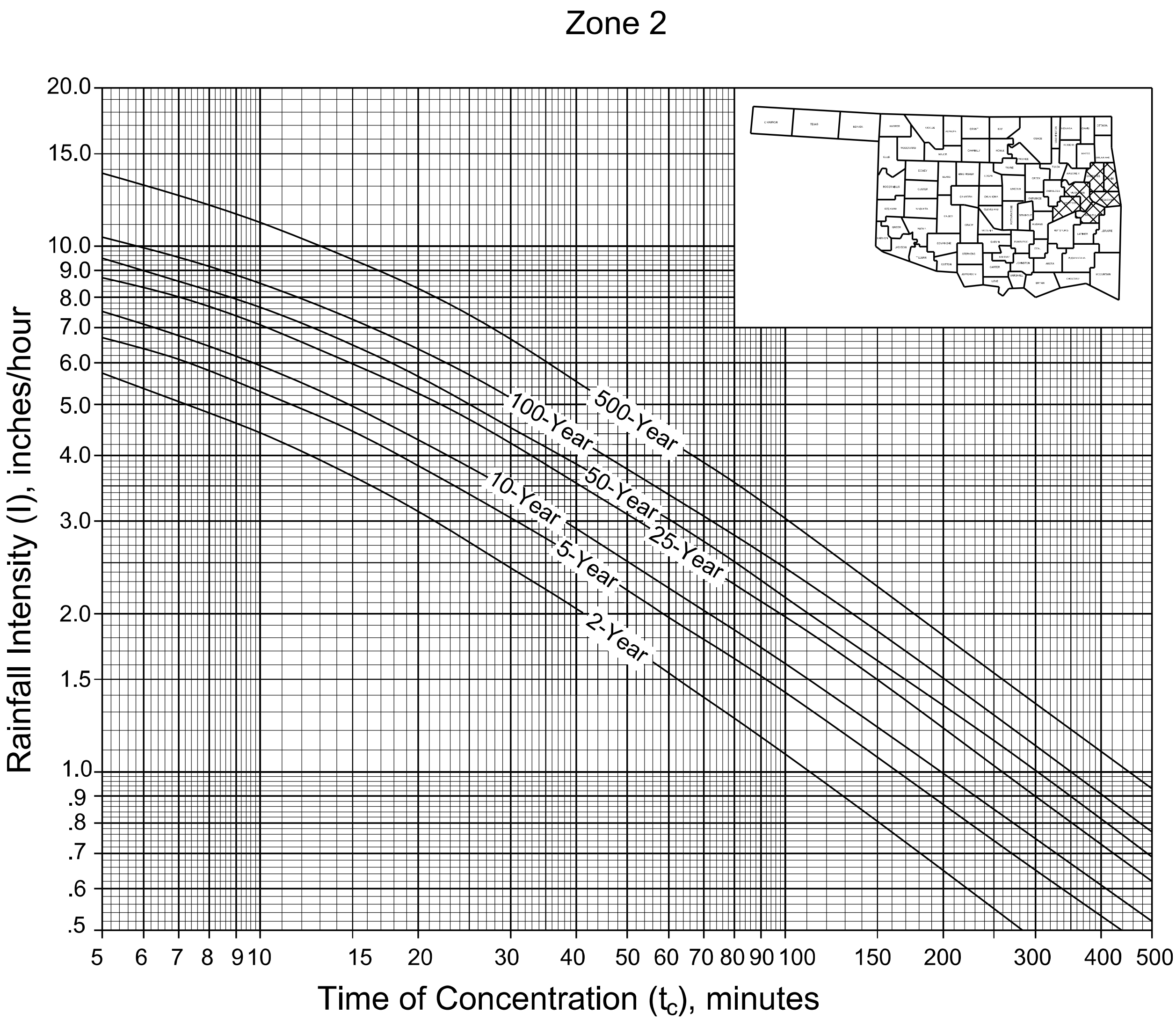
Figure 7.6-G — IDF CURVE ZONES IN OKLAHOMA

Zone 1



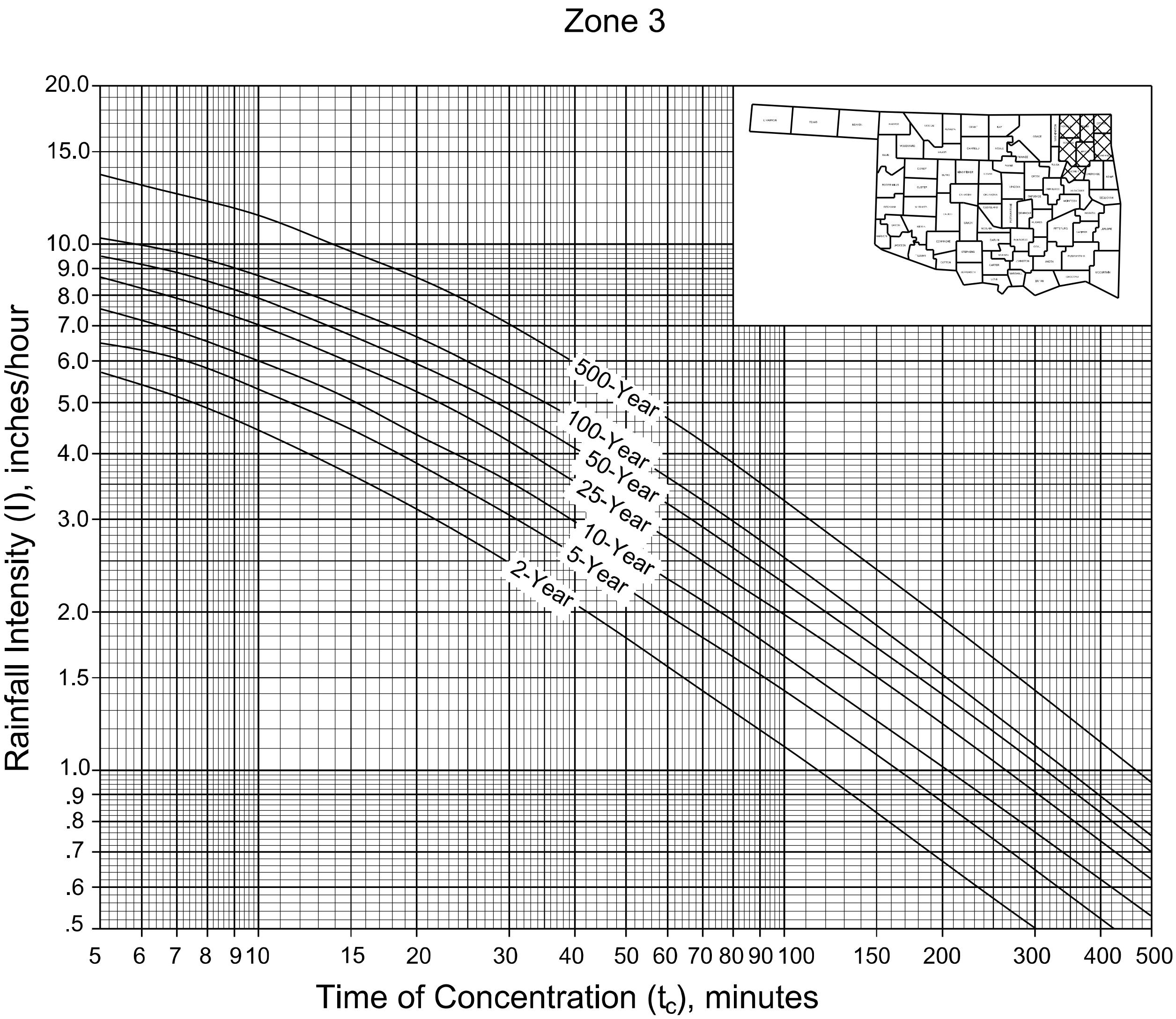
Source: (4)

Figure 7.6-H — ZONE 1 IDF CURVE (Atoka, Bryan, Choctaw, Latimer, Leflore, McCurtain, Pittsburg and Pushmataha Counties)



Source: (4)

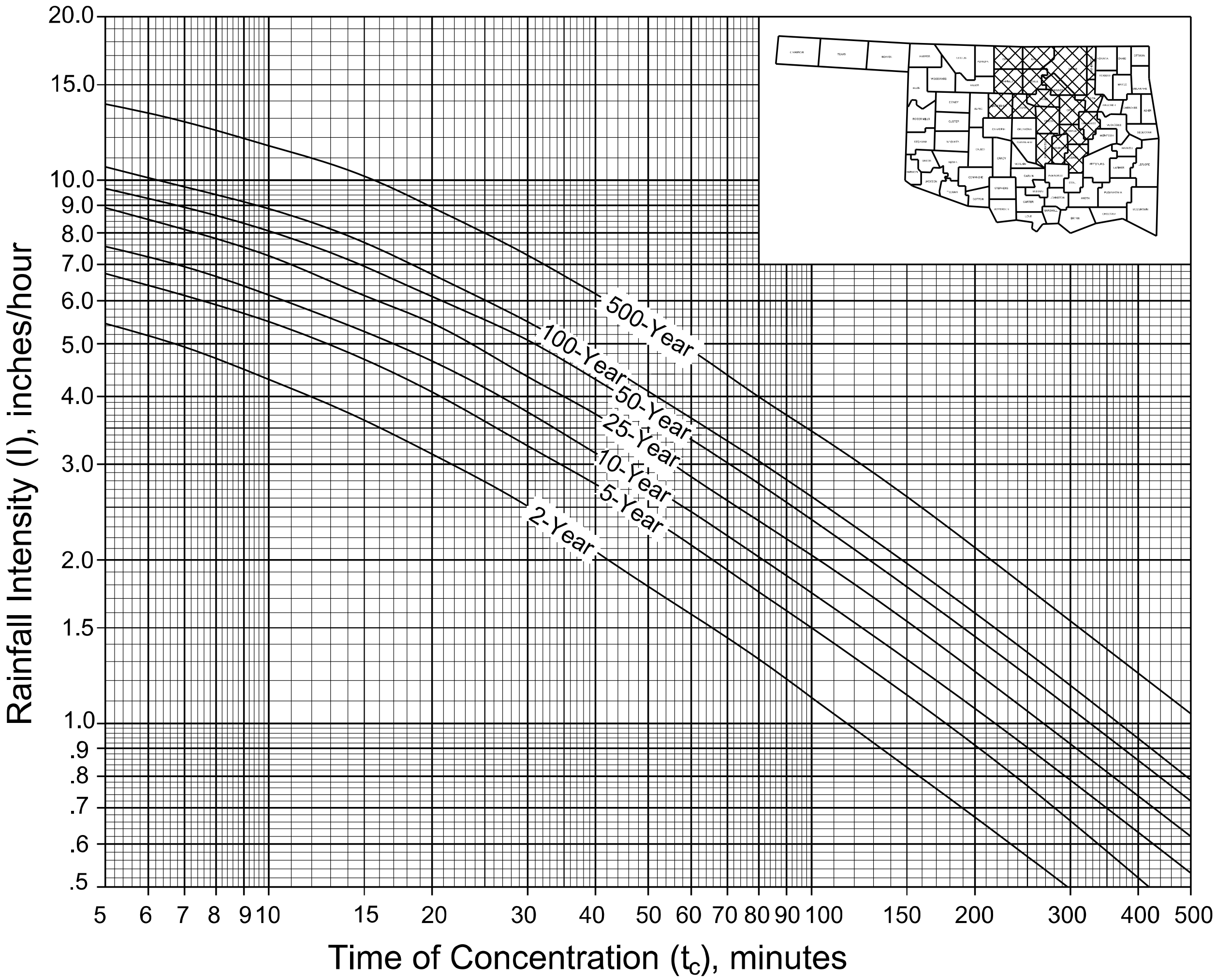
Figure 7.6-I — ZONE 2 IDF CURVE (Adair, Cherokee, Haskell, McIntosh, Muskogee and Sequoyah Counties)



Source: (4)

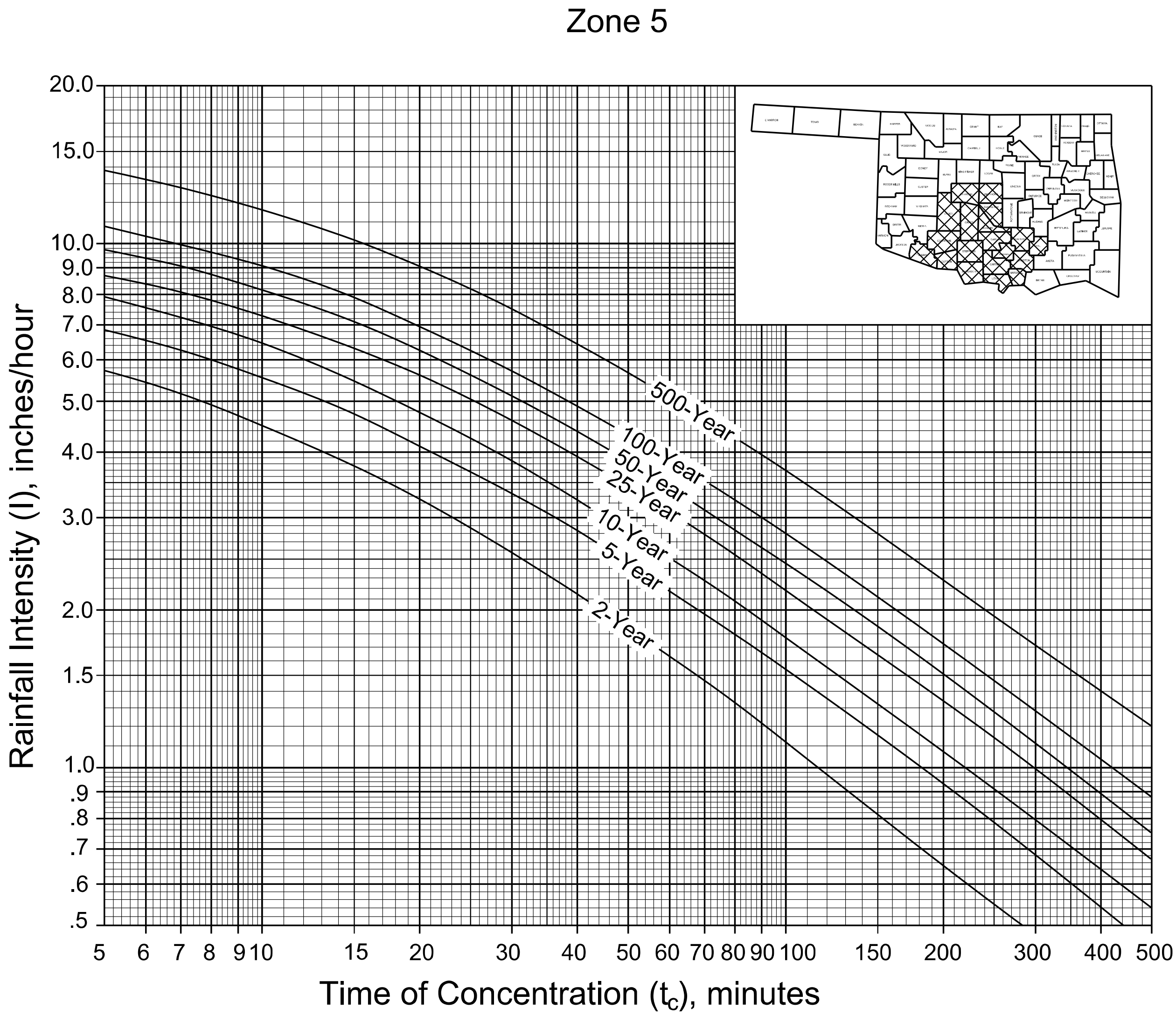
Figure 7.6-J — ZONE 3 IDF CURVE (Craig, Delaware, Mayes, Nowata, Ottawa, Rogers and Wagoner Counties)

Zone 4



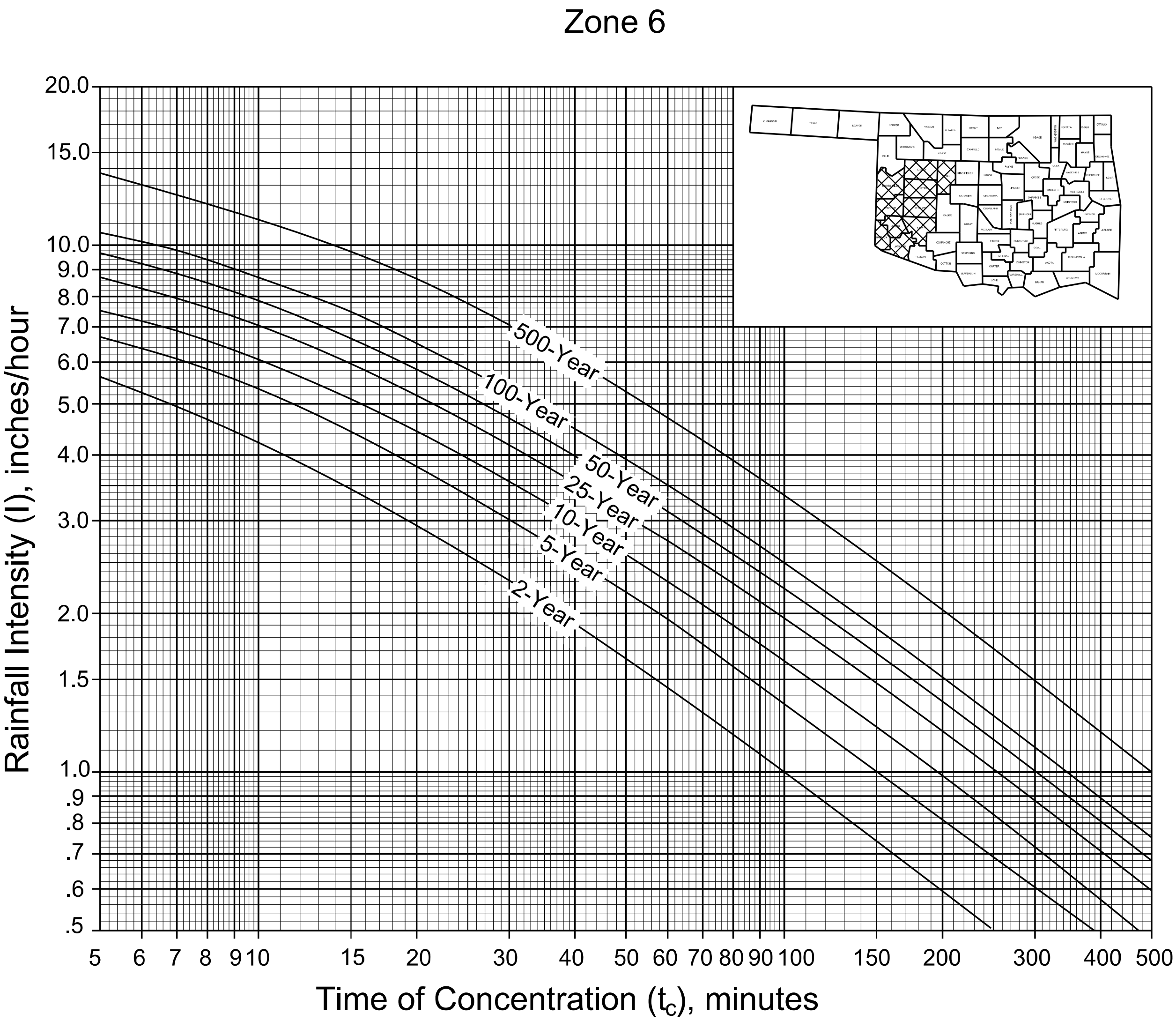
Source: (4)

Figure 7.6-K— ZONE 4 IDF CURVE (Creek, Garfield, Grant, Hughes, Kay, Kingfisher, Lincoln, Logan, Noble, Okfuskee, Okmulgee, Osage, Payne, Pawnee, Pottawatomie, Seminole, Tulsa and Washington Counties)



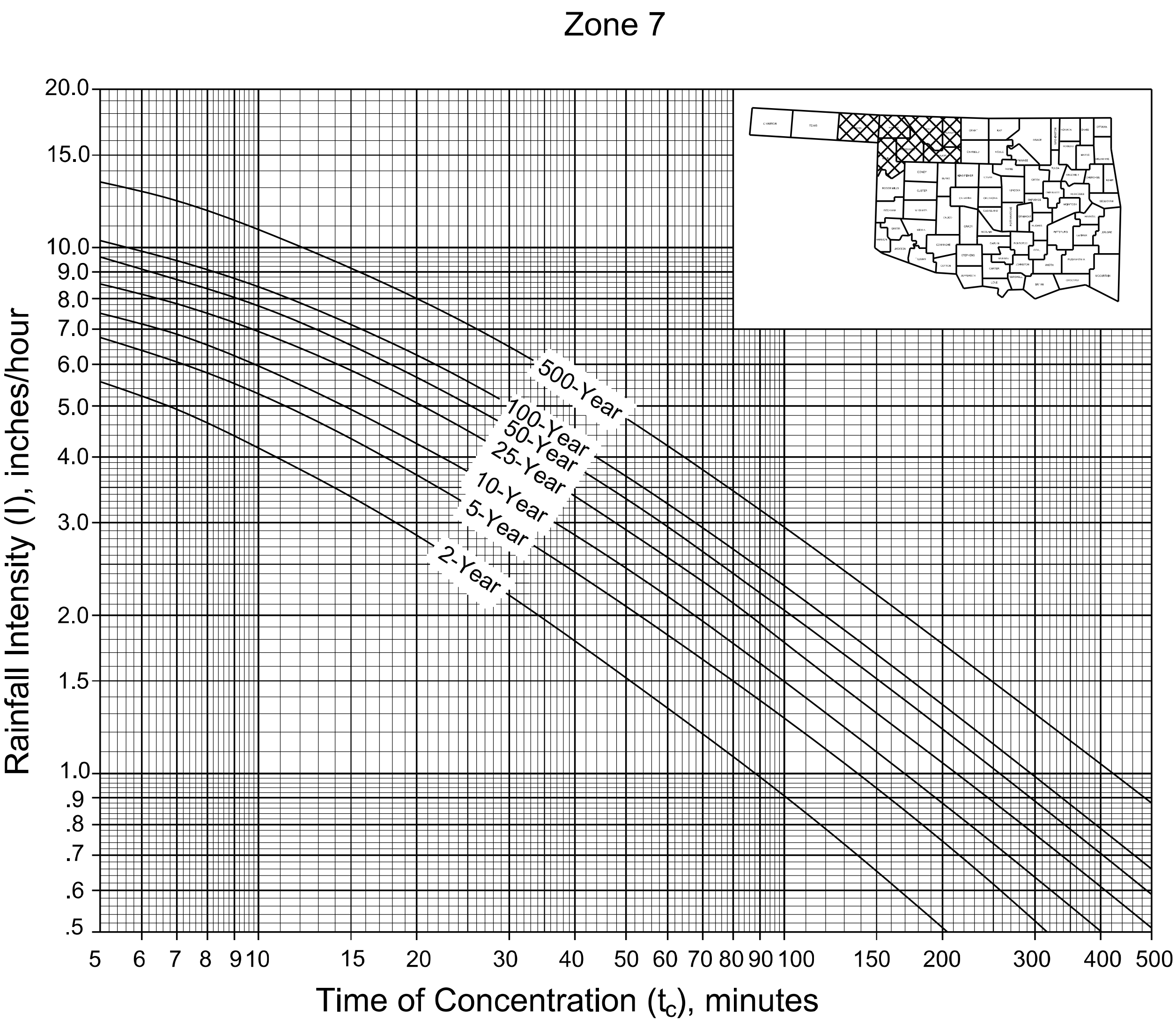
Source: (4)

Figure 7.6-L — ZONE 5 IDF CURVE (Caddo, Canadian, Carter, Cleveland, Coal, Comanche, Cotton, Garvin, Grady, Jefferson, Johnston, Love, Marshall, McClain, Murray, Oklahoma, Pontotoc, Stephens and Tillman Counties)



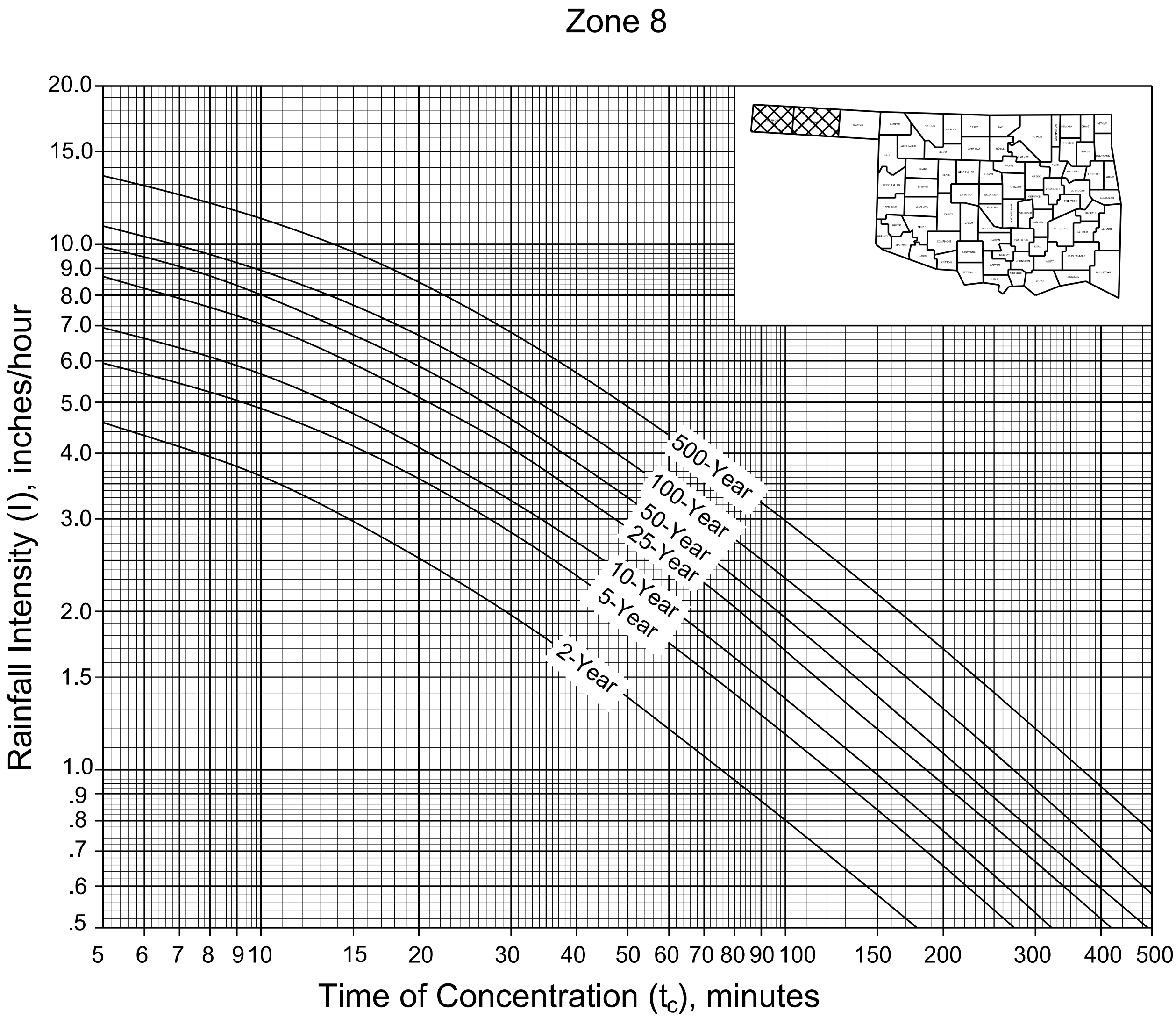
Source: (4)

Figure 7.6-M — ZONE 6 IDF CURVE (Beckham, Blaine, Custer, Dewey, Harmon, Jackson, Kiowa, Roger Mills and Washita Counties)



Source: (4)

Figure 7.6-N — ZONE 7 IDF CURVE (Alfalfa, Beaver, Ellis, Harper, Major, Woods and Woodward Counties)



Source: (4)

Figure 7.6-O — ZONE 8 IDF CURVE (Cimarron and Texas Counties)

Zone	Values	2-year	5-year	10-year	25-year	50-year	100-year	500-year
1	a	43	47	58	70	77	91	126
	b	9	9	10	10	11	12	13
	c	0.76	0.73	0.74	0.74	0.74	0.75	0.76
2	a	42	45	49	67	70	83	120
	b	8	8	8	10	10	11	11
	c	0.78	0.74	0.73	0.75	0.74	0.75	0.78
3	a	41	45	53	71	91	126	169
	b	8	8	9	11	13	16	16
	c	0.77	0.74	0.74	0.76	0.78	0.82	0.83
4	a	46	59	69	81	106	116	153
	b	10	11	12	12	15	15	15
	c	0.79	0.78	0.78	0.78	0.80	0.80	0.80
5	a	53	64	74	93	104	108	130
	b	10	12	12	15	15	15	15
	c	0.82	0.79	0.79	0.79	0.79	0.77	0.75
6	a	40	53	67	81	88	104	148
	b	7	9	11	12	12	13	15
	c	0.79	0.78	0.79	0.79	0.78	0.79	0.80
7	a	44	59	75	90	98	110	129
	b	7	9	11	12	12	13	12
	c	0.83	0.82	0.83	0.83	0.82	0.82	0.80
8	a	64	97	113	140	160	205	240
	b	12	15	15	15	15	18	18
	c	0.93	0.93	0.93	0.93	0.93	0.94	0.92

Source: (4)

Figure 7.6-P — IDF CURVE COEFFICIENTS for $I = a/(t_c + b)^c$

7.6.8 Design Procedure

The following steps are used to compute the peak discharge by the Rational Method. Since the results of using the Rational Method are very sensitive to the parameters that are used; the hydraulics designer must use good engineering judgment in estimating the values that are used in the method.

Step 1. Determine the boundary of the watershed and its area. The two most common ODOT methods for determining the boundary and the area of a small watershed are:

- USGS Maps. Printed 7.5 or 15 minute quadrangle maps with contour lines. Once the watershed boundary has been determined, the area of the watershed can be measured with a planimeter. When greater accuracy is required, especially for small or flat areas, a field survey (photogrammetric or survey crew) should be requested.

- Field Survey. Figure 7.6-Q is an example of the field survey data provided by the ODOT Survey Division field crews. Although field survey information is more accurate, the survey effort should be commensurate with the sensitivity or importance of the site. Cost, crew availability and time required are some of the factors that limit the use of field survey.

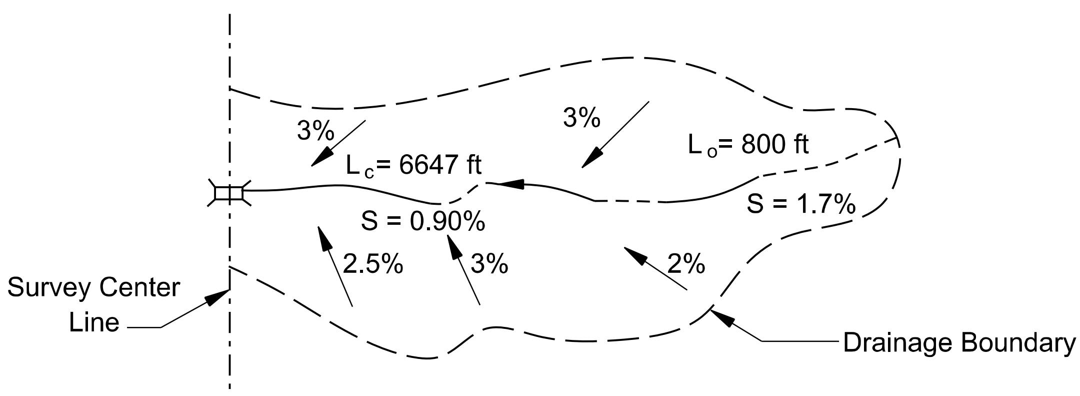


Figure 7.6-Q — EXAMPLE OF FIELD SURVEY INFORMATION

Step 2. Compute the average slope of the watershed.

The average slope of the watershed is computed using the following procedure and is illustrated in Figure 7.6-R:

a. Divide the watershed into subareas that have the same slope and compute the area A_n of each subarea in acres.

b. Compute the different slopes S_n of the watershed. The slope is the ratio in percent of the difference in elevation to the length of a straight line that connects a point on the watershed boundary with a point on the main channel. The line should be as perpendicular to the main channel, as possible. The average slope is computed using equation 7.6(2).

$$S = (A_1S_1 + A_2S_2 + A_nS_n)/A_t \qquad \text{Equation 7.6(5)}$$

Where:

S	=	average slope of the watershed, %
S_n	=	slope of subarea n, %
A_n	=	area of subarea n, acres
A_t	=	total area of the watershed, acres

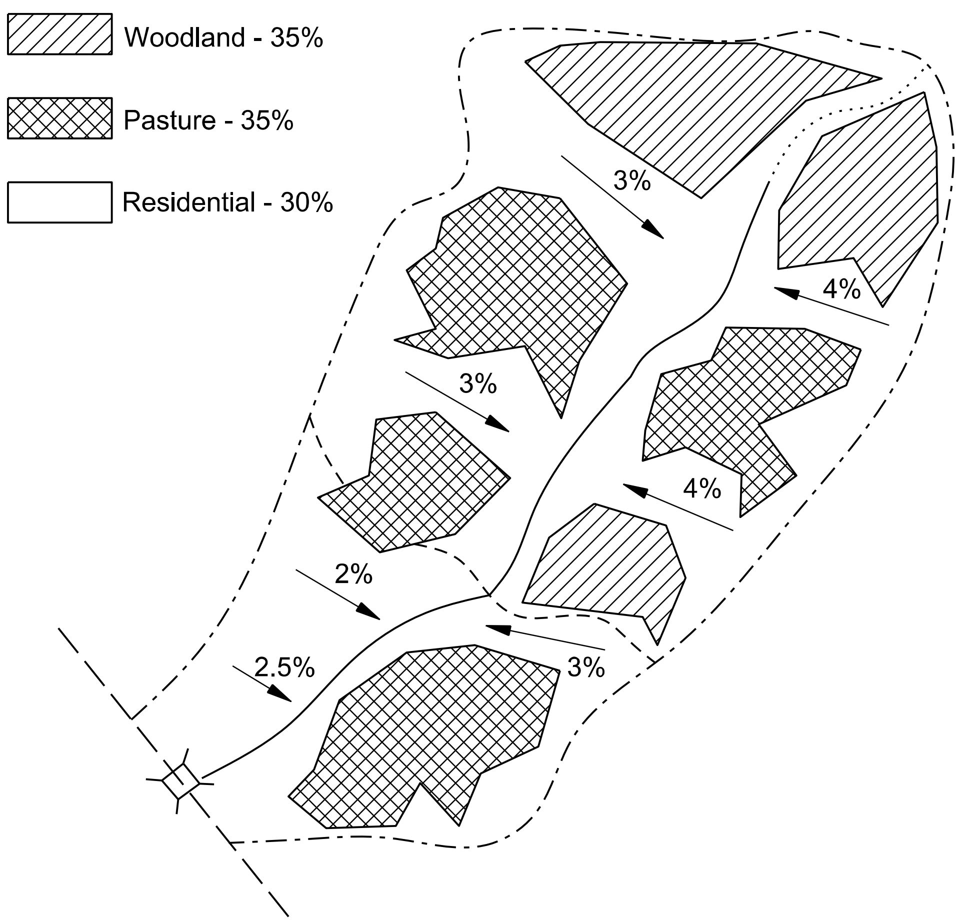


Figure 7.6-R — DIFFERENT SLOPES AND LAND USE TYPES IN A TYPICAL WATERSHED

Step 3. Determine the anticipated future land use (AFLU) for all the subareas identified in Step 2.

Figure 7.6-A lists the potential categories of land use and the associated runoff coefficient. If the AFLU will be the same, proceed to Step 4. If the land use is likely to change, assign the most likely land use; e.g. if pasture is near an urban area and adjacent to a residential area assume that the pasture will become residential. In

this way, the drainage structure should be capable of handling the future development.

Step 4. Determine the runoff coefficient C for each subarea.

The runoff coefficient C is defined as the ratio of the rainfall runoff to the total rainfall precipitation. The runoff coefficient is dimensionless and is dependent on the land use and the slope of the watershed. The values of C vary from zero (no runoff) to a maximum of 1 (100% runoff). If the watershed has only one type of land use, C can be determined directly from Figures 7.6-A through Figure 7.6-F. If there are several types of land use, the following procedure should be used to determine a weighted runoff coefficient:

a. Divide the watershed into subareas that have the same type of land use (see Figure 7.6-U). Measure the area A_n of each subarea in acres.

b. Use Figures 7.6-A through Figure 7.6-F to determine the runoff coefficient C_n of each subarea, based on the average slope of the watershed (Step 2).

c. Compute the weighted coefficient C using equation 7.6(3).

$$C = (A_1C_1 + A_2C_2 + A_nC_n)/A_t \qquad \text{Equation 7.6(6)}$$

Where:

C	=	weighted runoff coefficient of the watershed
C_n	=	runoff coefficient of subarea n, %
A_n	=	area of subarea n, acres
A_t	=	total area of the watershed, acres

Step 5. Compute the time of concentration, t_c.

The time of concentration is the time required for water to flow from the hydraulically mostly remote point of the drainage area to the point where the drainage structure is located. Section 7.6.6 lists the equations for calculating t_c. Two common errors should be avoided when calculating t_c:

- In some cases, runoff from a portion of the drainage area, which is highly impervious, may result in a greater peak discharge than would occur if the entire area were considered. In this case, only the impervious area should be considered and the rest of the area disregarded. However, various combinations should be investigated to determine which condition yields the largest discharge.

- When designing a drainage system, the overland flow path is not necessarily perpendicular to the contours shown on available mapping. Often the land may be graded and swales will intercept the natural contour and conduct the water to the street which reduces the t_c. If the overland flow path is long (greater than 2000 ft), consider treating the part that exceeds 2000 ft as channel flow.

Step 6. Determine the design frequency.

The recommended design frequency is shown in Figures 7.1-A and 7.1-B.

Step 7. Determine the rainfall Intensity, I in/hr.

The rainfall intensity, I, is the average rainfall rate in inches per hr for a duration equal to the time of concentration t_c for a selected design frequency. Locate the hydrological zone of the site (see section 7.6.7). For a given duration (t_c from Step 5) and design frequency from Step 6, the design rainfall intensity, I, can be read from the corresponding zone IDF curve (Figures 7.6-H through 7.6-O) or can be computed using Figure 7.6-P.

Step 8. Compute the design discharge Q_n.

The design discharge can be calculated using Equation 7.6(1) using C from Step 4, I from Step 7 and area from Step 1. If the design frequency is greater than 10-year, then consider increasing the discharge with the appropriate adjustment factor.

The information supplied below is typical of the type of information found in a survey book. Of course, information may vary from what is shown here and should be examined carefully to determine its usefulness.

a. D.A. = 293.6 acres

b. Average slope of D.A.: 3% = 70%, 2.5% = 10%, 2% = 20%

   Average slope of D.A. = 0.0275% or 2.75%

c. Length of channel flow = 1.3 miles (6647 ft); Slope = 0.90%

   Length of overland flow = 800 ft; Slope = 1.7%

d. Average slope of channel = 0.9%

e. D.A. cover: 25% cultivation, 75% pasture

f. Highwater elevation = 1286.3 ft; occurred in 1956 or 1957. A local resident, Mr. John Doe, observed extreme highwater overtopping the centerline of SH-90 by 1 ft at the culvert location. Mr. John Doe has lived in this area for over 50 years. His residence is approximately 0.5 miles west of the culvert location.

g. Channel notes: Channel has a dirt and grass flowline, few obstructions

h. Ravine Section: None taken

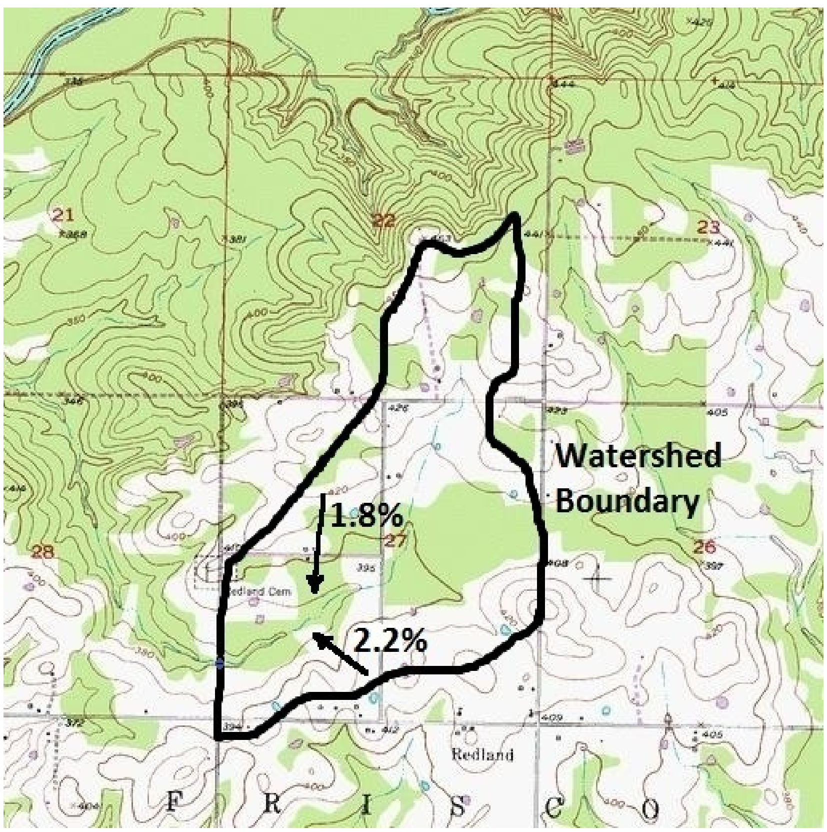
i. Location: Cleveland County

7.6.9 Design Example

A drainage structure is proposed at the southern end of the drainage basin, which is outlined on the OK USGS 7.5 minutes quadrangle map "SCHULTS, OK" (dated 1950, photo revised in 1975), shown in Step 1 below. The location is on a No Name Creek, which is 2 miles west and 4.5 miles north of the City of Haworth, located mostly in Section 27 of T-7-S and R-25-E in McCurtain County, OK. Compute the 10-year and 100-year peak discharges for the design of a culvert using the rational formula.

Step 1. Determine the boundary of the watershed and its area.

The watershed boundary is shown on the OK USGS 7.5 minutes quadrangle map, "SCHULTS, OK." The area of the watershed was measured with a planimeter and determined to be 593 acres. No NRCS flood control structures are within the watershed.



Step 2. Compute the average slope of the watershed.

As shown on the map, the area north of the stream has a slope of 1.8% and about 50% of the area. The area south of the stream has a slope of 2.2% for the remaining 50% of the watershed.

The average slope of the watershed is:

$$S = 0.5(2.2) + 0.5(1.8) = 2.0\%$$

<u>Step 3</u>. Determine the anticipated future land use (AFLU).

From the USGS map and aerial photos, the anticipated future land is assumed to be:

- 50% of the area will be woodland
- 50% of the area will be pasture

<u>Step 4</u>. Determine the runoff coefficient C for each subarea.

Figure 7.6-B for woodland gives C = 0.2 for the average slope of 2%.
Figure 7.6-C for pasture gives C = 0.3 for the average slope of 2%.

From Equation 7.6(3), the weighted C is:

$C = (A_1C_1 + A_2C_2)/A$
$C = [(0.5A)(0.02) + (0.5A)(0.3)]/A = 0.25$

<u>Step 5</u>. Compute the time of concentration, t_c.

The flow path consists of the following:

- 2100 ft of overland flow through pasture with a slope of 0.018 ft/ft
- 7000 ft of channel flow at an average slope of 0.00764 ft/ft

Sheet Flow is calculated using the following equation:

$$T_o = \frac{k\left(L_o^{0.37}\right)}{S_o^{0.20}}$$

Where:

k = 1.04 for average grass
S_o = 0.018 ft/ft
L_o = 2100 ft

$$T_o = \frac{1.04\ (2100\ \text{ft})^{0.37}}{(0.018)^{0.20}} = 39\ \text{minutes}$$

Channel flow is calculated using the following equation:

$$T_f = \frac{k'\left(L_f^{0.77}\right)}{S_{fo}^{0.385}}$$

Where:

L_f = 7000 ft
S_f = 0.00764 ft/ft
K' = 0.00835 for average stream

$$T_f = \frac{0.00835\left(7000^{0.77}\right)}{0.00764^{0.385}} = 50 \text{ minutes}$$

$t_c = T_o + T_f = 39 + 50 = 89$ minutes

Step 6. Determine the design frequency

The design frequencies are 10-year and 100-year return period.

Step 7. Determine the rainfall Intensity, I in/hr

From Figure 7.6-L – Zone 1 for McCurtain County:

$I_{10} = 1.9$ in/hr
$I_{100} = 2.9$ in/hr

Step 8. Compute the design discharge Q_n

The design discharge is calculated using Equation 7.6(1).

Intensity (I) is calculated using Figure 7.6-P

- $C_f = 1.0$ for 10-year and 1.25 for 100-year
- C = 0.25 from Step 4
- $I_{10} = 1.94$ in/hr and $I_{100} = 2.85$ in/hr
- A = 593 acres from Step 1

$Q = C_f CIA$
$Q_{10} = 1.0(0.25)(1.94)(593) = 287.61$ cfs
$Q_{100} = 1.25(0.25)(2.85)(593) = 528.14$ cfs

7.7 NRCS TR-55 PEAK DISCHARGE METHOD

For many peak discharge estimation methods, the input includes variables to reflect the size of the contributing area, the amount of rainfall, the potential watershed storage and the time-area distribution of the watershed. These are often translated into input variables such as the drainage area, the depth of rainfall, an index reflecting land use and soil type and the time of concentration. The NRCS peak discharge method (7) is typical of many peak discharge methods that are based on input such as that described. The overview provided in this section is taken from HDS 2 (2).

The NRCS method can also be used to produce a hydrograph (see Chapter 16 “Hydraulic Software”).

7.7.1 Runoff Depth Estimation

The volume of storm runoff can depend on a number of factors. Certainly, the volume of rainfall will be an important factor. For very large watersheds, the volume of runoff from one storm event may depend on rainfall that occurred during previous storm events. However, when using the design storm approach, the assumption of storm independence is quite common.

In addition to rainfall, other factors affect the volume of runoff. A common assumption in hydrologic modeling is that the rainfall available for runoff is separated into three parts—direct (or storm) runoff, initial abstraction and losses. Factors that affect the split between losses and direct runoff include the volume of rainfall, land cover and use, soil type and antecedent moisture conditions. Land cover and land use will determine the amount of depression and interception storage.

7.7.2 Runoff Curve Number

The NRCS cover complex classification consists of three factors—land use, treatment or practice and hydrologic condition. Many different land uses are identified in the following tables for estimating runoff curve numbers.

- Figure 7.7-A is for urban areas
- Figure 7.7-B is for cultivated agricultural lands - Agricultural land uses are often subdivided by treatment or practices (e.g., contoured or straight row); this separation reflects the different hydrologic runoff potential that is associated with variation in land treatment. The hydrologic condition reflects the level of land management; it is separated into three classes—poor, fair and good.
- Figure 7.7-C is for other agricultural lands
- Figure 7.7-D is for arid and semi-arid range lands

Urban Area[1] Cover Description		**Curve Numbers for Hydrologic Soil Group**			
		A	**B**	**C**	**D**
Fully developed urban areas[1] (vegetation established)					
Open Space (Lawns, parks, golf courses, cemeteries, etc.) [3]					
Good condition; grass cover on 75% or more of the area		39	61	74	80
Fair condition; grass cover on 50% to 75% of the area		49	69	79	84
Poor condition; grass cover on 50% or less of the area		68	79	86	89
Imperious Areas					
Paved parking lots, roofs, driveways, etc. (excl. right-of-way)		98	98	98	98
Streets and roads					
Paved with curbs and storm sewers (excl. right-of-way)		98	98	98	98
Gravel (including right-of-way)		76	85	89	91
Dirt (including right-of-way)		72	82	87	89
Paved with open ditches (including right-of-way)		83	89	92	93
Western Desert Urban Areas:					
Natural desert landscaping (pervious areas only) [4]		63	77	85	88
Artificial desert landscaping (impervious weed barrier, desert shrub with 1-in to 2-in sand or gravel mulch and basin borders)		96	96	96	96
Urban Districts	Average imperious[2]				
Commercial and business areas	85	89	92	94	95
Industrial districts	72	81	88	91	93
Residential: Average Lot Size					
$^1/_8$ acre or less (town houses)	65	77	85	90	92
$^1/_4$ acre	38	61	75	83	87
$^1/_3$ acre	30	57	72	81	86
$^1/_2$ acre	25	54	70	80	85
1 acre	20	51	68	79	84
2 acre	12	46	65	77	82
Developing Urban Areas					
Newly graded area (pervious areas only), no vegetation established)[5]		77	86	91	94

Figure 7.7-A — RUNOFF CURVE NUMBERS FOR URBAN AREAS

Notes:

1. *Average runoff condition, and Ia = 0.2S.*
2. *The average percent impervious area shown was used to develop the composite CN's. Other assumptions are as follows: impervious areas are directly connected to the drainage system, impervious areas have a CN of 98 and pervious areas are considered equivalent to open space in good hydrologic condition. CN's for other combinations of conditions may be computed using Figure 7.7-E.*
3. *CN's shown are equivalent to those of pasture. Composite CN's may be computed for other combinations of open space cover type.*
4. *Composite CN's for natural desert landscaping should be computed using Figure 7.7-E based on the impervious area percentage (CN = 98) and the pervious area CN. The pervious area CN's are assumed equivalent to desert shrub in poor hydrologic condition.*
5. *Composite CN's to use for the design of temporary measures during grading and construction should be computed using Figure 7.7-E based on the degree of development (impervious area percentage) and the CN's for the newly graded pervious areas.*

Figure 7.7-A — RUNOFF CURVE NUMBERS FOR URBAN AREAS
(continued)

Cultivated Agricultural Lands[1] Cover Type		Hydrologic Condition[3]	Curve Numbers for Hydrologic Soil Group			
Cover type	Treatment[2]		A	B	C	D
Fallow	Bare soil		77	86	91	94
	Crop residue cover (CR)	Poor	76	85	90	93
		Good	74	83	88	90
Row crops	Straight row (SR)	Poor	72	81	88	91
		Good	67	78	85	89
	SR + CR	Poor	71	80	87	90
		Good	64	75	82	85
	Contoured (C)	Poor	70	79	84	88
		Good	65	75	82	86
	C + CR	Poor	69	78	83	87
		Good	64	74	81	85
	Contoured and terraced (C&T)	Poor	66	74	80	82
		Good	62	71	78	81
	C&T + CR	Poor	65	73	79	81
		Good	61	70	77	80
Small grain	SR	Poor	65	76	84	88
		Good	63	75	83	87
	SR + CR	Poor	64	75	83	86
		Good	60	72	80	84
	C	Poor	63	74	82	85
		Good	61	73	81	84
	C + CR	Poor	62	73	81	84
		Good	60	72	80	83
	C&T	Poor	61	72	79	82
		Good	59	70	78	81
	C&T + CR	Poor	60	71	78	81
		Good	58	69	77	80
Close-seeded or broadcast legumes or rotation meadows	SR	Poor	66	77	85	89
		Good	58	72	81	85
	C	Poor	64	75	83	85
		Good	55	69	78	83
	C&T	Poor	63	73	80	83
		Good	57	67	76	80

Notes:

1. *Average runoff condition and I_a=0.2S*
2. *Crop residue cover applies only if residue is on at least 5% of the surface throughout the year.*
3. *Hydraulic condition is based on combination factors that affect infiltration and runoff, including*
   a. *density and canopy of vegetative areas,*
   b. *amount of year-round cover,*
   c. *amount of grass or close-seeded legumes,*
   d. *percent of residue cover on the land surface (good ≥ 20%), and*
   e. *degree of surface roughness.*

Poor = Factors impair infiltration and tend to increase runoff.

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

Figure 7.7-B — RUNOFF CURVE NUMBERS FOR CULTIVATED AGRICULTURAL LANDS

Other Agricultural Lands[1] Cover Descriptions	Hydrologic Condition[3]	Curve Numbers for Hydrologic Soil Group			
		A	B	C	D
Pasture, grassland or range - continuous forage for grazing[2]	Poor	68	79	86	89
	Fair	49	69	79	84
	Good	39	61	74	80
Meadow - continuous grass, protected from grazing and generally moved for hay	------	30	58	71	78
Brush - brush-weed-grass mixture with brush the major element[3]	Poor	48	67	77	83
	Fair	35	56	70	77
	Good	30[4]	48	65	73
Woods[6]	Poor	45	66	77	83
	Fair	36	60	73	79
	Good	30[4]	55	70	77
Woods - grass combination (orchard or tree farm)[5]	Poor	57	73	82	86
	Fair	43	65	76	82
	Good	32	58	72	79
Farmsteads - buildings, lanes driveways and surrounding lots	------	59	74	82	86

Notes:

1. *Average runoff condition, and Ia = 0.2S.*
2. *Poor: < 50% ground cover or heavily grazed with no mulch*
   Fair: 50% to 75% ground cover and not heavily grazed
   Good: > 75% ground cover and lightly or only occasionally grazed
3. *Poor: < 50% ground cover*
   Fair: 50% to 75% ground cover
   Good: > 75% ground cover
4. *Actual curve number is less than 30; use CN = 30 for runoff computations.*
5. *CN's shown were computed for areas with 50% woods and 50% grass (pasture) cover. Other combinations of conditions may be computed from the CN's for woods and pasture.*
6. *Poor: Forest litter, small trees and brush are destroyed by heavy grazing or regular burning.*
   Fair: Woods are grazed but not burned and some forest litter covers the soil.
   Good: Woods are protected from grazing and litter and brush adequately cover the soil

Figure 7.7-C — RUNOFF CURVE NUMBERS FOR OTHER AGRICULTURAL LANDS

Arid and Semi-arid Range Lands[1] Cover Type	Hydrologic Condition[2]	Curve Numbers for Hydrologic Soil Group			
		A[3]	B	C	D
Herbaceous - mixture of grass, weeds and low-growing brush, with brush the minor element	Poor		80	87	93
	Fair		71	81	89
	Good		62	74	85
Oak-aspen - mountain brush mixture of oak brush, aspen, mountain mahogany, bitter brush, maple and other brush	Poor		66	74	79
	Fair		48	57	63
	Good		30	41	48
Pinyon - juniper - pinyon, juniper or both; grass understory	Poor		75	85	89
	Fair		58	73	80
	Good		41	61	71
Sagebrush with grass understory	Poor		67	80	85
	Fair		51	63	70
	Good		35	47	55
Desert shrub - major plants include saltbush, greasewood, creosotebush, blackbrush, bursage, palo verde, mesquite and cactus	Poor	63	77	85	88
	Fair	55	72	81	86
	Good	49	68	79	84

Notes:

1. *Average runoff condition and Ia, = 0.2S. For range in humid regions, use table 7.7-C.*
2. *Poor: < 30% ground cover (litter, grass and brush overstory).*
   Fair: 30% to 70% ground cover.
   Good: > 70% ground cover.
3. *Curve numbers for group A have been developed only for desert shrub.*

Figure 7.7-D — RUNOFF CURVE NUMBERS FOR ARID AND SEMI-ARID RANGE LANDS

7.7.2.1 Hydrologic Soil Group

The NRCS assigns a curve number (CN) value for the different land uses, treatments and hydrologic conditions; separate values are given for each soil group (see Figures 7.7-A through 7.7-D). For example, the CN for a wooded area with good cover and soil group B is 55; for soil group C, the CN would increase to 70. If the cover (on soil group B) is poor, the CN will be 66. The CN is used in Section 7.7.9, Step 5.

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

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

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

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

7.7.2.2 Effect of Connected Impervious Urban Areas on Curve Numbers

An urban impervious area is considered connected if:

- its runoff flows directly into the drainage system, or
- its runoff occurs as concentrated shallow flow that runs over a pervious area then into a drainage system.

The Curve Numbers for Urban areas (including Commercial, Industrial and residential districts) as shown in Figure 7.7-A are based on a specific percent of connected impervious areas

For example, the CN values for commercial land use in this table are based on the assumption that 85% of this commercial land site is connected impervious area and the remaining 15% is open space with grass cover in good condition.

The weighted CN value for this site could be compute by the following equation:

$$CN_w = CN_p\,(1 - f) + f(98) \qquad \text{Equation 7.7(1)}$$

in which f is the fraction (not percent) of imperviousness.

Figure 7.7-A shows that:

- The CN value for the connected impervious portion is 98, and
- The CN values for the pervious portion (open space, grass cover, good condition) for hydrologic soil groups A, B, C and D are 39, 61, 74 and 80, respectively.

From Equation 7.7(1), the CN values for commercial land use with 85% imperviousness are:

A soil: 39(0.15) + 98(0.85) = 89
B soil: 61(0.15) + 98(0.85) = 92
C soil: 74(0.15) + 98(0.85) = 94
D soil: 80(0.15) + 98(0.85) = 95

These are the same values shown in Figure 7.7-A. Equation 7.7(1) can be placed in graphical form (see Figure 7.7-E(a)). By entering with the percent of connected imperviousness and moving vertically to the pervious area CN, the composite CN can be read.

The examples above show that the weighted CN value for a commercial land with 85% connected impervious area and 15% pervious area type B soil is 92.

However, for a commercial land area with 60% connected imperviousness and the remainder 40% a B soil with CN = 61, the composite CN would only be:

$$CN_w = 61(0.4) + 98(0.6) = 83$$

The same value can be obtained from Figure 7.7-E(a).

7.7.2.3 Effect of Unconnected Impervious Area on Curve Numbers

An urban impervious area is considered unconnected if:

- its runoff is spread over a pervious area as sheet flow, or
- all or part of its runoff is not discharged into a drainage system.

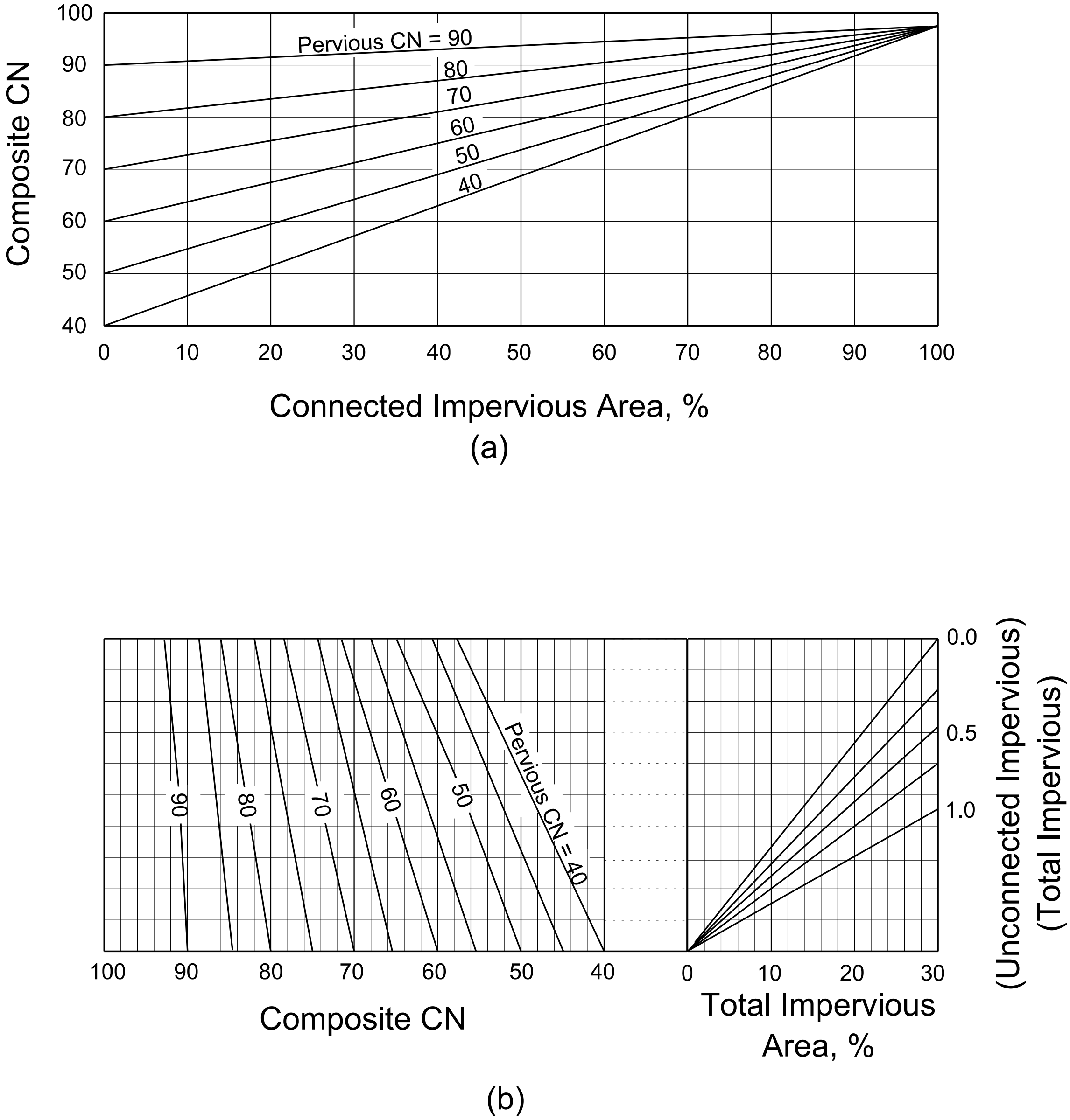
Many local drainage policies are requiring runoff that occurs from certain types of impervious land cover (i.e., rooftops, driveways, patios) to be directed to pervious surfaces rather than being connected to storm drain systems. Such a policy is based on the belief that disconnecting these impervious areas will require smaller and less costly drainage systems and lead both to increased groundwater recharge and to improvements in water quality. If disconnecting some impervious surfaces will reduce both the peak runoff rates and volumes of direct flood runoff, credit should be given in the design of drainage systems. The effect of disconnecting impervious surfaces on runoff rates and volumes can be accounted for by modifying the CN.

There are three variables involved in the adjustment—the pervious area CN, the percent of impervious area and the percent of the imperviousness that is unconnected. Because Figure 7.7-E(a) for computing composite CN values is based on the pervious area CN and the percent of imperviousness, a correction factor was developed to compute the composite CN for unconnected urban impervious areas. The correction is a function of the percent of unconnected imperviousness, which is shown in Figure 7.7-E(b). The use of the correction is limited to drainage areas having percent of imperviousness that are less than 30%.

As an alternative to Figure 7.7-E(b), the composite curve number (CN_c) can be computed by:

$$CN_c = CN_p + (P_1/100)\ (98 - CN_p)(1 - 0.5R) \text{ for } P_1 \leq 30\ \% \qquad \text{Equation 7.7(2)}$$

in which P_i is the percent imperviousness and R is the ratio of unconnected impervious area to the total impervious area. Equation 7.7(2), like Figure 7.7-E(a), is limited to where the total imperviousness (P_i) is less than 30%.



(a) all imperviousness area connected to storm drains
(b) some imperviousness area not connected to storm drain

Figure 7.7-E — COMPOSITE CURVE NUMBER ESTIMATION

7.7.3 Time of Concentration and Lag time

There are three types of flow that are needed to be considered in computing the time of concentration in NRCS method, as show in the table below.

Type of Flow	Recommended Equations
Sheet Flow	Equation 7.7(4) or 7.7(5)
Shallow Concentrated Flow	Equations 7.7(6) and 7.7(7) or 7.7(8)
Channel or Pipe Flow	Equations 7.7(6) and 7.7(9)

Figure 7.7-F — 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 7.7(3). The t_c may include all the travel time types, a few types or just a single type:

$$t_c = T_{t(sheet)} + T_{t(shallow)} + T_{t(channel)} + T_{t(pipe)}$$

Equation 7.7(3)

7.7.3.1 Sheet Flow Travel Time

Sheet flow is a shallow mass of runoff on a plane surface with the depth uniform across the sloping surface. 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 (2). The original form of the kinematic wave travel time (in English units) is:

$$T_{t(sheet)} = \frac{0.93}{I^{0.4}}\left(\frac{nL}{\sqrt{S}}\right)^{0.6}$$

Equation 7.7(4)

Where:

$T_{t(sheet)}$ = sheet flow travel time, minutes

n = roughness coefficient (see Figure 7.7-G). Remember that this is NOT Manning's roughness coefficient.

L = flow length, ft

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

S = slope of the surface, ft/ft

To avoid the necessity to solve for t_c iteratively, NRCS TR-55 (7) uses the following variation of the kinematic wave equation:

$$T_{t(sheet)} = \frac{0.42}{P_2^{0.5}}\left(\frac{nL}{\sqrt{S}}\right)^{0.8}$$ Equation 7.7(5)

Where:

P_2 = 2-year, 24-hr rainfall depth, in (see Figure 7.7-L)

NRCS TR-55 (7) recommends an upper limit of L = 300 ft for using this equation.

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

Notes:

1. *The n values are a composite of information compiled by Engman* (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 7.7-G — ROUGHNESS COEFFICIENTS FOR SHEET FLOW

7.7.3.2 Travel Time for Shallow Concentrate Flow, Channel Flow and Pipe Flow

The velocity equation can be used to estimate travel times for 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):

$$T_{t(type)} = \frac{L}{60V}$$ Equation 7.7(6)

Where:

$T_{t(type)}$ = Travel flow time for shallow concentrated T_t (shallow), for pipe flow T_t (pipe) or for channel flow T_t (channel), minutes.

L = flow length, ft

V = flow velocity in shallow concentrated segment, in channel or in pipe, fps

7.7.3.3 Velocity for Shallow Concentrated Flow

The land use of the shallow concentrate flow segment can be divided into two types: paved or unpaved.

From TR-55 (7), the flow velocity V for the paved shallow concentrate segment could be computed by the following equations:

$$V = 20.3282\left(S^{0.5}\right)$$ Equation 7.7(7)

For the unpaved shallow concentrated segment, use the following equation to compute V:

$$V = 16.1345\left(S^{0.5}\right)$$ Equation 7.7(8)

Where:

V = Flow velocity for shallow concentrate segment, fps
S = Slope of the shallow concentrated segment, ft/ft

7.7.3.4 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. For channel, average flow velocity is usually determined for bank-full condition.

Manning's equation is:

$$V = \frac{1.486\ R^{2/3}\ S^{1/2}}{n}$$ Equation 7.7(9)

Where:

V	=	average velocity, fps
R	=	hydraulic radius, ft (equal to A/WP)
A	=	cross sectional flow area, square 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 7.7(9), the travel time for the channel segment or pipe can be calculated from Equation 7.7(6).

The lag time, L, is equal to 0.6 time the time of concentration t_c (L= 0.6 t_c) and is computed in hr. This lag time L is required in computing the peak discharges.

7.7.4 Parameter (I_a/P)

I_a/P is a parameter that is necessary to estimate peak discharge rates. I_a denotes the initial abstraction and P is the 24-hr rainfall depth for a selected return period. The I_a/P value can be obtained from Figure 7.7-H for a given CN and P. For a given 24-hr rainfall distribution, I_a/P represents the fraction of rainfall that must occur before runoff begins.

Rainfall (in)	Curve Number											
	40	**45**	**50**	**55**	**60**	**65**	**70**	**75**	**80**	**85**	**90**	**95**
0.4	*	*	*	*	*	*	*	*	*	*	*	0.27
0.8	*	*	*	*	*	*	*	*	*	0.45	0.28	0.13
1.2	*	*	*	*	*	*	*	*	0.42	0.30	0.19	+
1.6	*	*	*	*	*	*	*	0.42	0.32	0.22	0.14	+
2.0	*	*	*	*	*	*	0.44	0.34	0.25	0.18	0.11	+
2.4	*	*	*	*	*	0.46	0.36	0.28	0.21	.15	+	+
2.8	*	*	*	*	0.48	0.39	0.31	0.24	0.18	.13	+	+
3.1	*	*	*	*	0.42	0.34	0.27	0.21	0.16	.11	+	+
3.5	*	*	*	0.46	0.38	0.30	0.24	0.19	0.14	.10	+	+
3.9	*	*	*	0.42	0.34	0.27	0.22	0.17	0.13	+	+	+
4.3	*	*	0.46	0.38	0.31	0.25	0.20	0.15	0.12	+	+	+
4.7	*	*	0.42	0.35	0.28	0.23	0.18	0.14	0.11	+	+	+
5.1	*	0.48	0.39	0.32	0.26	0.21	0.17	0.13	0.10	+	+	+
5.5	*	0.44	0.36	0.30	0.24	0.20	0.16	0.12	+	+	+	+
5.9	*	0.41	0.34	0.28	0.23	0.18	0.15	0.11	+	+	+	+
6.3	0.48	0.39	0.32	0.26	0.21	0.17	0.14	0.11	+	+	+	+
6.7	0.45	0.37	0.30	0.24	0.20	0.16	0.13	0.10	+	+	+	+
7.1	0.42	0.34	0.28	0.23	0.19	0.15	0.12	+	+	+	+	+
7.5	0.40	0.33	0.27	0.22	0.18	0.14	0.11	+	+	+	+	+
7.9	0.38	0.31	0.25	0.21	0.17	0.14	0.11	+	+	+	+	+
8.3	0.36	0.30	0.24	0.20	0.16	0.13	0.10	+	+	+	+	+
8.7	0.35	0.28	0.23	0.19	0.15	0.12	0.10	+	+	+	+	+
9.1	0.33	0.27	0.22	0.18	0.15	0.12	+	+	+	+	+	+
9.4	0.32	0.26	0.21	0.17	0.14	0.11	+	+	+	+	+	+
9.8	0.30	0.25	0.20	0.17	0.14	0.11	+	+	+	+	+	+
10.2	0.29	0.24	0.20	0.16	0.13	0.11	+	+	+	+	+	+
10.6	0.28	0.23	0.19	0.15	0.13	0.10	+	+	+	+	+	+
11.0	0.27	0.22	0.18	0.15	0.12	0.10	+	+	+	+	+	+
11.4	0.26	0.21	0.18	0.14	0.12	+	+	+	+	+	+	+
11.8	0.25	0.21	0.17	0.14	0.11	+	+	+	+	+	+	+
12.2	0.25	0.20	0.16	0.13	0.11	+	+	+	+	+	+	+
12.6	0.24	0.19	0.16	0.13	0.11	+	+	+	+	+	+	+
13.0	0.23	0.19	0.15	0.13	0.10	+	+	+	+	+	+	+
13.4	0.22	0.18	0.15	0.12	0.10	+	+	+	+	+	+	+
13.8	0.22	0.18	0.15	0.12	0.10	+	+	+	+	+	+	+
14.2	0.21	0.17	0.14	0.12	+	+	+	+	+	+	+	+
14.6	0.21	0.17	0.14	0.11	+	+	+	+	+	+	+	+
15.0	0.20	0.16	0.13	0.11	+	+	+	+	+	+	+	+
15.4	0.20	0.16	0.13	0.11	+	+	+	+	+	+	+	+
15.7	0.19	0.16	0.13	0.10	+	+	+	+	+	+	+	+

** Signifies that I_a/P = 0.50 should be used*
+Signifies that I_a/P = 0.50 should be used

Figure 7.7-H — I_A/P FOR SELECTED RAINFALL DEPTHS AND CURVE NUMBERS

7.7.5 Peak Discharge Estimation

The following equation can be used to compute a peak discharge with the NRCS method:

$$q_p = q_u A Q \qquad \text{Equation 7.7(10)}$$

in which q_p is the peak discharge in cfs, q_u the unit peak discharge in cfs/square mile/in of runoff, A is the drainage area in square miles and Q is the depth of runoff from Equation 7.7(11).

$$Q = (P - I_a)^2/(P - I_a) + S \qquad \text{Equation 7.7(11)}$$

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 = (1000/CN) - 10, in

The unit peak discharge is obtained from the following equation that requires the time of concentration (t_c) in hrs and the initial abstraction/rainfall (I_a/P) ratio as input:

$$q_u = 10^{co + c1 \log Tc + c2 [\log(Tc)]^2} \qquad \text{Equation 7.7(12)}$$

in which the values of C_o, C_1 and C_2 are given in Figure 7.7-I for various I_a/P ratios. The runoff depth (Q) is obtained from Equation 7.7(11) and is a function of the depth of rainfall P and the runoff CN. The I_a/P ratio is obtained either directly by $I_a = 0.2S$ or from Figure 7.7-H; the ratio is a function of the CN and the depth of rainfall.

The peak discharge obtained from Equation 7.7(12) assumes that the topography is such that surface flow into ditches, drains and streams is relatively unimpeded. Where ponding or swampy areas occur in the watershed, a considerable amount of the surface runoff may be retained in temporary storage. The peak discharge rate should be reduced to reflect this condition of increased storage. Values of the pond and swamp adjustment factor (F_p) are provided in Figure 7.7-J. The adjustment factor values are a function of the percent of the total watershed area in ponds and swamps (PPS). If the watershed includes significant portions of pond and swamp storage, then the peak discharge of Equation 7.7(13) can be adjusted using the following:

$$q_a = q_a F_p \qquad \text{Equation 7.7(13)}$$

in which q_a is the adjusted peak discharge in cfs

Rainfall Type	I_a/P	C_0	C_1	C_2
I	0.10	2.30550	-0.51429	-0.11750
	0.20	2.23537	-0.50387	-0.08929
	0.25	2.18219	-0.48488	-0.06589
	0.30	2.10624	-0.45695	-0.02835
	0.35	2.00303	-0.40769	0.01983
	0.40	1.87733	-0.32274	0.05754
	0.45	1.76312	-0.15644	0.00453
	0.50	1.67889	-0.06930	0.0
IA	0.10	2.03250	-0.31583	-0.13748
	0.20	1.91978	-0.28215	-0.07020
	0.25	1.83842	-0.25543	-0.02597
	0.30	1.72657	-0.19826	0.02633
	0.50	1.63417	-0.09100	0.0
II	0.10	2.55323	-0.61512	-0.16403
	0.30	2.46532	-0.62257	-0.11657
	0.35	2.41896	-0.61594	-0.08820
	0.40	2.36409	-0.59857	-0.05621
	0.45	2.29238	-0.57005	-0.02281
	0.50	2.20282	-0.51599	-0.01259
III	0.10	2.47317	-0.51848	-0.17083
	0.30	2.39628	-0.51202	-0.13245
	0.35	2.35477	-0.49735	-0.11985
	0.40	2.30726	-0.46541	-0.11094
	0.45	2.24876	-0.41314	-0.11508
	0.50	2.17772	-0.36803	-0.09525

Figure 7.7-I — COEFFICIENTS FOR NRCS PEAK DISCHARGE METHOD (EQUATION 7.7(12))

Area of pond and swamp (%)	F_p
0	1.00
0.2	0.97
1.0	0.87
3.0	0.75
5.0	0.72

Figure 7.7-J — ADJUSTMENT FACTOR (F_p) FOR POND AND SWAMP AREAS THAT ARE SPREAD THROUGHOUT THE WATERSHED

7.7.6 Design Procedure

The following steps are used to compute the peak discharge by the NRCS method. Because the results of using the NRCS method are very sensitive to the parameters that are used; the hydraulics designer must use good engineering judgment in estimating the values that are used in the method.

Step 1. Determine the boundary of the watershed and its area.

The two most common ODOT methods for determining the boundary and the area of a small watershed are:

- USGS Maps. Printed 7.5 or 15 minute quadrangle maps with contour lines. Once the watershed boundary has been determined, the area of the watershed can be measured with a planimeter. When greater accuracy is required, especially for small or flat areas, a field survey (photogrammetric or survey crew) should be requested.

- Field Survey. Figure 7.6-T is an example of the field survey data provided by the ODOT Survey Division field crews. Although field survey information is more accurate, the survey effort should be commensurate with the sensitivity or importance of the site. Cost, crew availability and time required are some of the factors which limit the use of field survey.

Step 2. Compute the average slope of the watershed.

The average slope of the watershed is computed using the procedure in Section 7.6.8 and is illustrated in Figure 7.6-Q.

Step 3. Determine the anticipated future land use (AFLU) for all the subareas identified in Step 2.

If the AFLU will be the same, proceed to Step 4. If the land use is likely to change, assign the most likely land use; e.g. if pasture is near an urban area and adjacent to a residential area assume that the pasture will become residential. In this way, the drainage structure should be capable of handling the future development.

Step 4. Define the type of rainfall and the amount of rainfall.

- Rainfall Type. There are four NRCS types of rainfall distribution is the United States, but Oklahoma only has Type II and Type III. Most of the state is Type II (see Figure 7.7-K).

  - Figure 7.7-K shows that Type III rainfall distribution applies to 14 counties, including Adair, Atoka, Bryan, Cherokee, Choctaw, Haskell, Latimer, Leflore, McCurtain, McIntosh, Muskogee, Pittsburg, Pushmataha and Sequoyah.

- o Figure 7.6-G shows that these 14 counties are also located in the Hydrological Zone 1 and 2 in the State of Oklahoma (see also Section 7.6.7 - Rational Method).

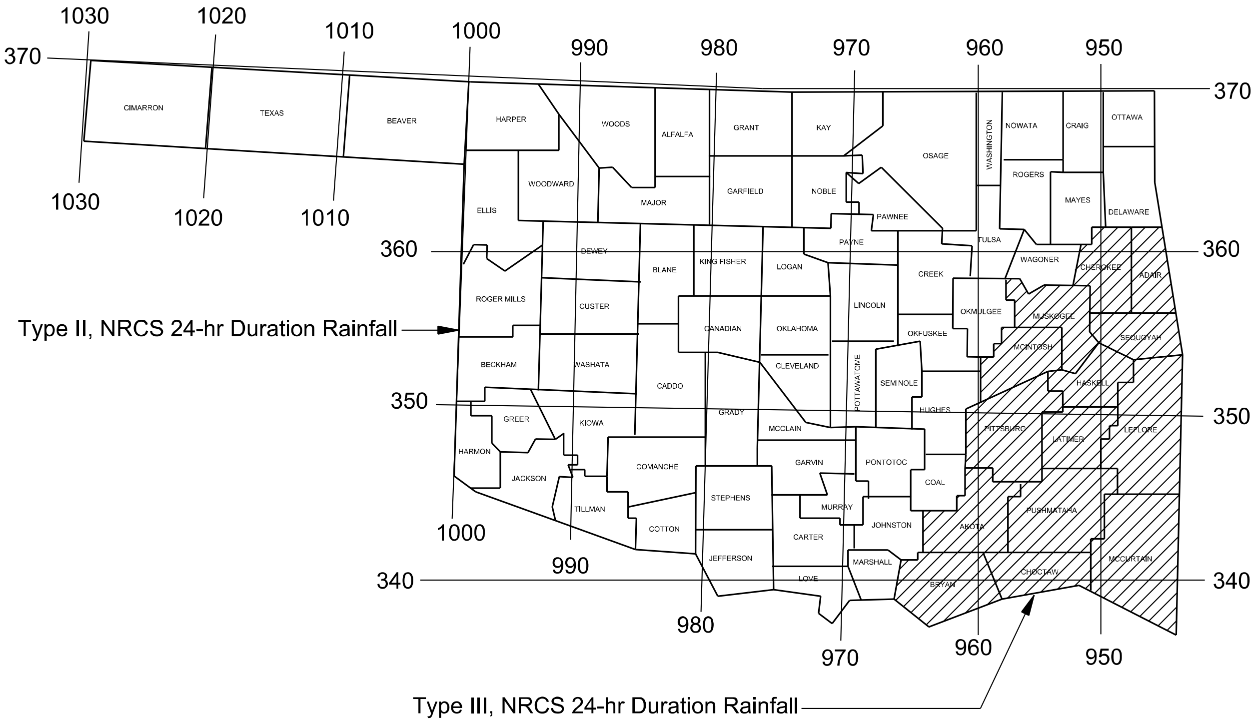


Figure 7.7-K — NRCS 24-HR RAINFALL DISTRIBUTION TYPES IN OKLAHOMA

- Rainfall Amount. The rainfall data used in the NRCS method is based on a 24-hr/1day storm event obtained from the USGS Water Resources Investigation Report 99-4232 (3). For conservative reasons, the hydraulics designer should use the larger values of the 24 hr or the 1 day rainfall data in the computation of the peak discharges.

  - o Figures 7.7-L (2-year) through Figures 7.7-R (500-year) can be used to determine the amount of expected 24-hr rainfall in inches.

  - o Figures 7.7-S (2-year) through Figures 7.7-X (500-year) can be used to determine the amount of expected 1 day rainfall in inches.

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Step 5. Determine the runoff curve number (CN) for the watershed.

The NRCS uses a combination of soil conditions and land-use (ground cover) to assign a runoff factor to an area. These runoff factors, called runoff curve numbers (CN), indicate the runoff potential of an area when the soil is not frozen. The higher the CN, the higher is the runoff potential. Soil properties influence the relationship between rainfall and runoff by affecting the rate of infiltration. The infiltration is the movement of the water through the soil surface into the soil. The NRCS has divided soils into the four hydrologic soil groups A, B, C and D based on infiltration rates described in Section 7.7.2.1. CN is determined using the following steps:

a. Determine Soil Types in the watershed. The hydraulics designer should consult the NRCS soil survey book of the County where the watershed is located. The NRCS, in cooperation with the Oklahoma Agricultural Experiment Station, has published the Soil Survey books of the 77 counties of Oklahoma. These books are stored in the Erosion Section, Roadway Design Division and are available online on the NRCS website.

b. Determine Hydrologic Soil Group. Once the soil type has been determined, the hydraulics designer should refer to Exhibit A in the TR-55 manual which is available online (7) to determine the hydrologic soil group.

c. Find CN. The hydrologic soil group and the ground cover type is used to find the CN for an AMC II (average moisture conditions, see Figure 7.7-Z) from one of the following figures:

- Figure 7.7-A is for urban areas
- Figure 7.7-B is for cultivated agricultural lands
- Figure 7.7-C is for other agricultural lands
- Figure 7.7-D is for arid and semi-arid range lands

d. Adjust for Moisture Condition. If the soil condition is normally dry or wet, the CN can be adjusted using Figure 7.7-AA.

e. Adjust for Imperious Areas. If the ground cover is made up in part with connected or unconnected impervious areas, the CN can be adjusted with Figure 7.7-E. An area is connected if runoff flows directly into the drainage system.

A sample of the TR-55 Runoff Curve Number and Runoff computation sheets are included in Appendix 7.C.

Step 6. Determine the direct runoff (Q).

Assuming I_a = 0.2S and substituting in Equation 7.7(11) the direct runoff is:

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

Where:

Q	=	accumulated direct runoff, in
P	=	accumulated rainfall (potential maximum runoff), in
S	=	potential maximum retention = (1000/CN) - 10, in

Step 7. Calculate the time of concentration (t_c)

In the NRCS method, the time of concentration (t_c) is defined to be the time required for water to travel from the most hydraulically distant point in a watershed to its outlet. Compute the time of concentration (t_c) using equation 7.7(3) of section 7.7.3.

A sample of the TR-55 time of concentration computation sheet is included in Appendix 7.C.

If information about the channel cross section data is not available and the watershed area is less than 2000 acres, the hydraulics designer could use the following equation to compute the lag time of the watershed (9):

$$L = L_m^{0.8} \ (S+1)^{0.7} / \left(1900\ Y^{0.5}\right) \qquad \text{Equation 7.7(14)}$$

Where:

L	=	lag time, hrs
L_m	=	length of mainstream to farthest divide, ft
Y	=	average slope of the watershed, %
S	=	potential maximum retention = (1000/CN) - 10, in

Then:

$$t_c = 1.67L \qquad \text{Equation 7.7(15)}$$

Step 8. Compute the unit peak discharge (q_u)

The unit peak discharge (q_u) is computed with Equation 7.7(12):

$$q_u = 10^{co\ +\ c1 \log Tc\ +\ c2\ [\log(Tc)]^2}$$

Where:

C_0, C_1, C_2 are constants from Figure 7.7-I

t_c	=	time of concentration from Step 7, hrs
q_u	=	unit peak discharge, cfs/square mile/in

Step 9. Compute the peak discharge (q_p)

The peak discharge (q_p) is computed with Equation 7.7(10):

$$q_p = q_u A Q$$

Where:

q_u	=	unit peak discharge in cfs/square mile/in of runoff
A	=	drainage area in square miles
Q	=	depth of runoff in in from Step 6

Step 10. Peak discharge adjustment for pond and swamp.

If the area in ponds or swamps is greater than 10%, the peak discharge from Step 9 should be adjusted with Equation 7.7(13):

$q_a = q_p F_p$

Where:

q_p = peak discharge in cfs/square mile/in of runoff from Step 9
F_p = pond and swamp adjustment from Figure 7.7-J

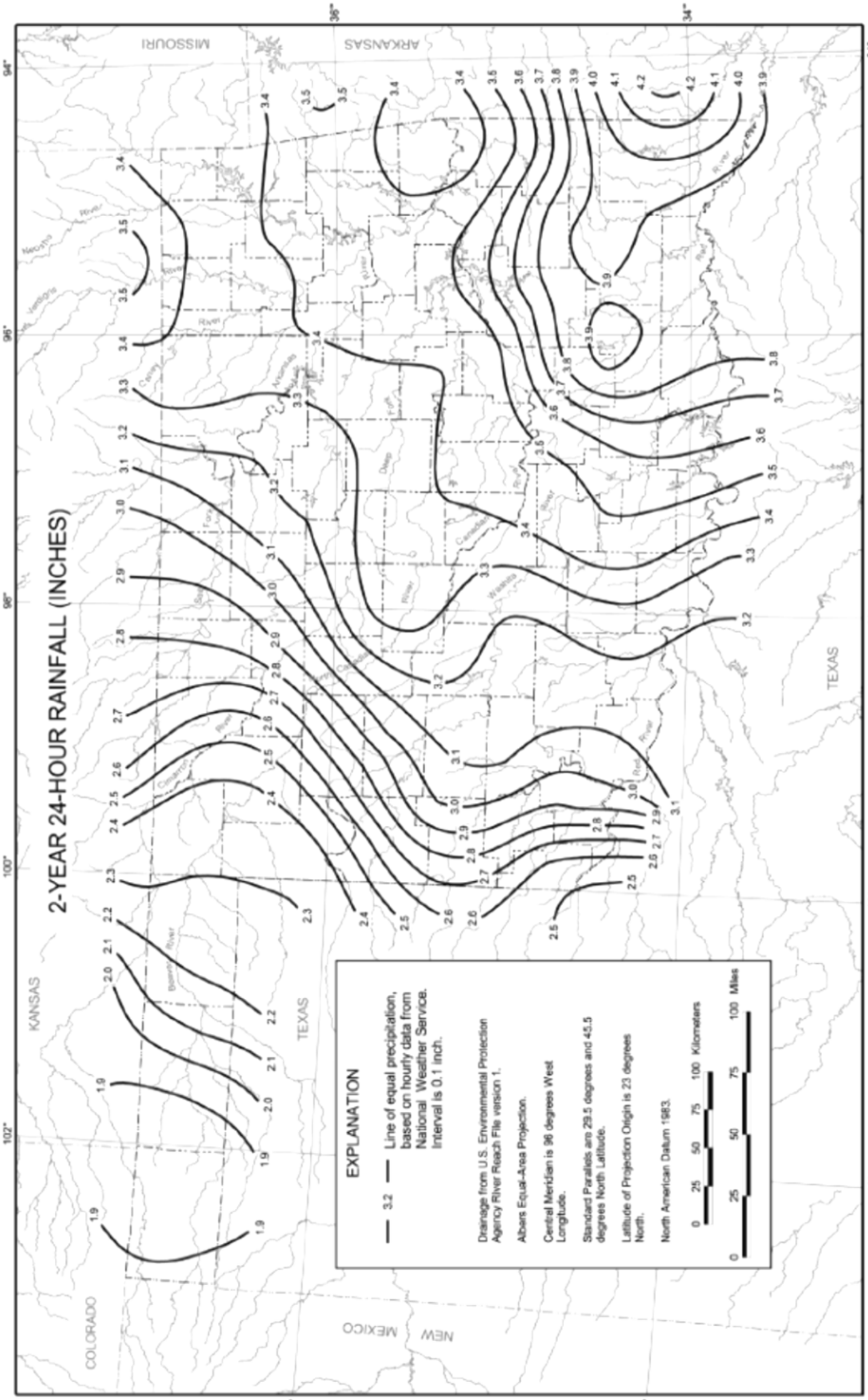
Step 11. Check the result using USACE HEC-HMS program.

The hydraulics designer could check the above manual computation by using the USACE HEC-1 or HEC-HMS program, using the NRCS hydrograph method option.

Step 12. Evaluate the result.

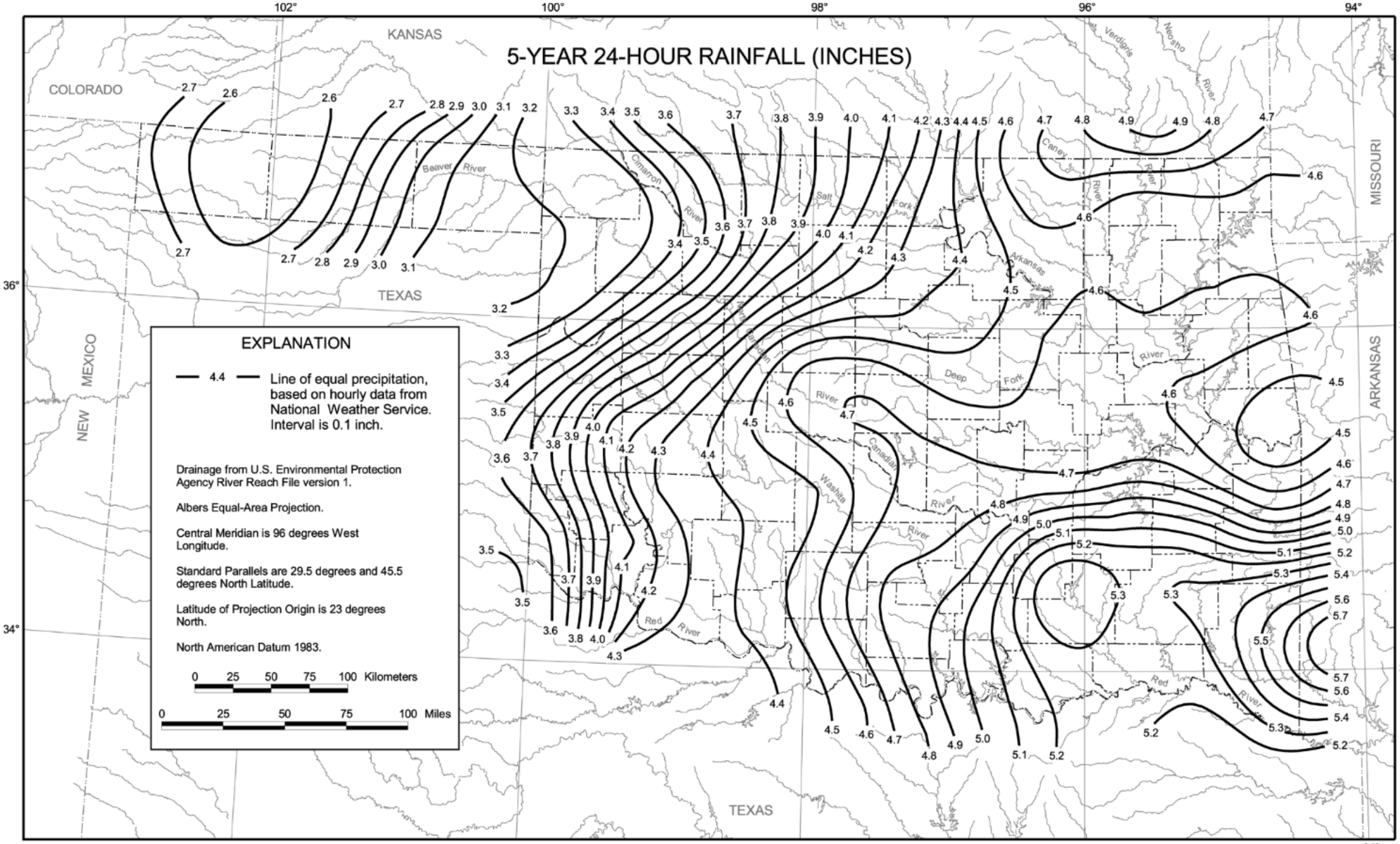
The choice of using the peak discharges computed by either the NRCS method or the Rational method in the design should be based on additional factors, including but not limited to:

- The hydraulics characteristics of the existing structure.
- The highwater marks recorded at the site and the rainfall frequencies corresponding with these highwater marks.



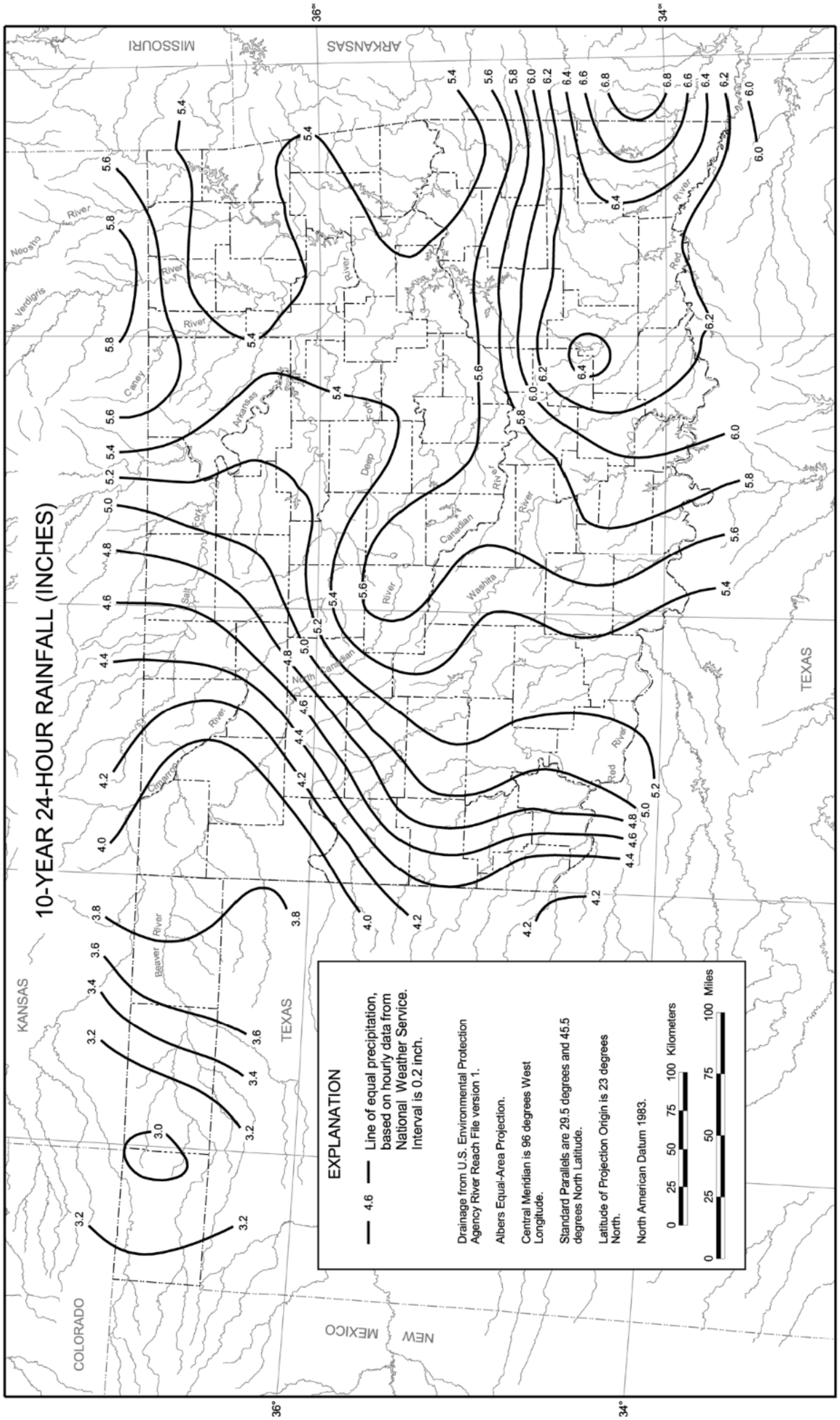
Source: USGS (3)

Figure 7.7-L — OKLAHOMA 2-YEAR, 24-HR RAINFALL (in)



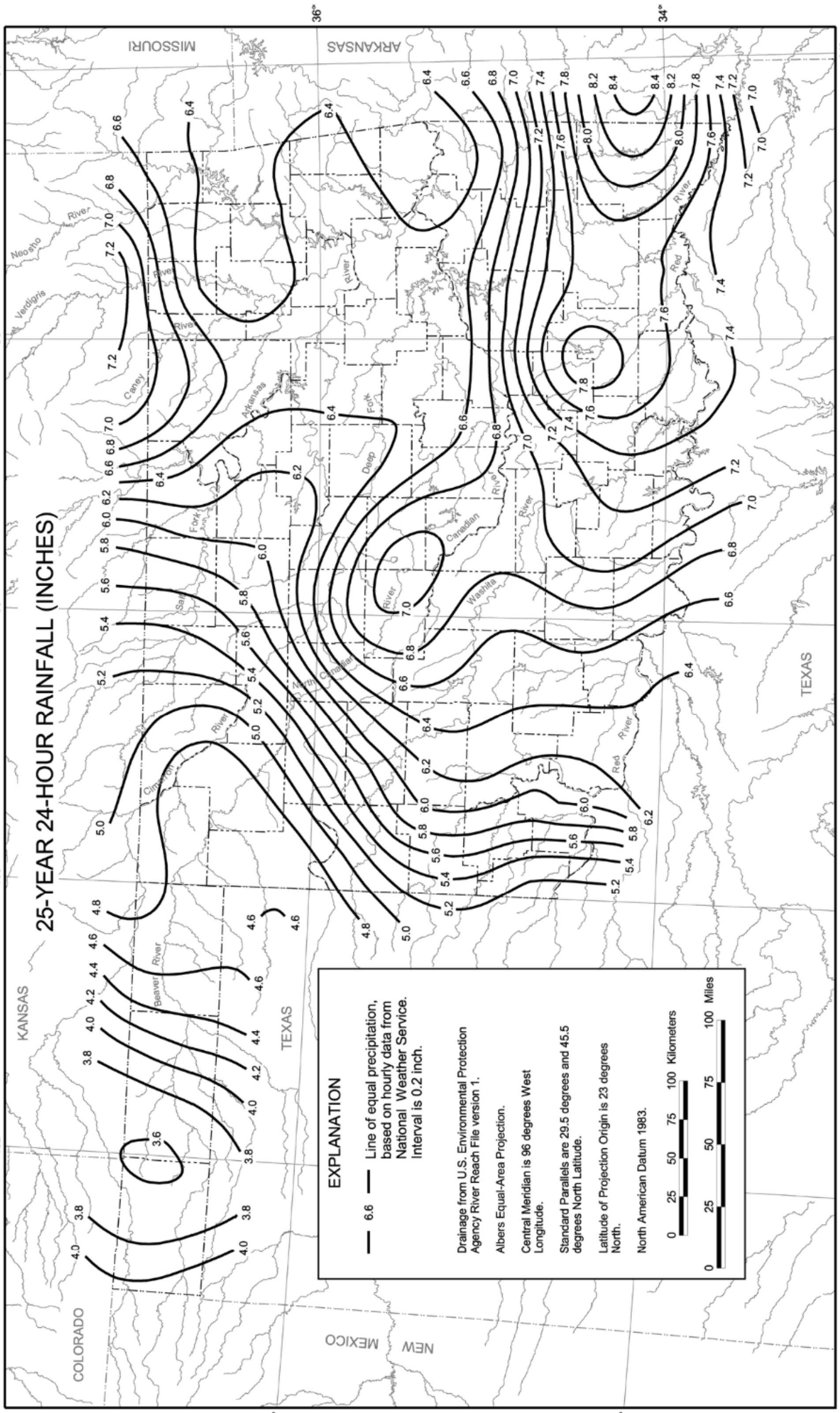
Source: USGS (3)

Figure 7.7-M — OKLAHOMA 5-YEAR, 24-HR RAINFALL (in)



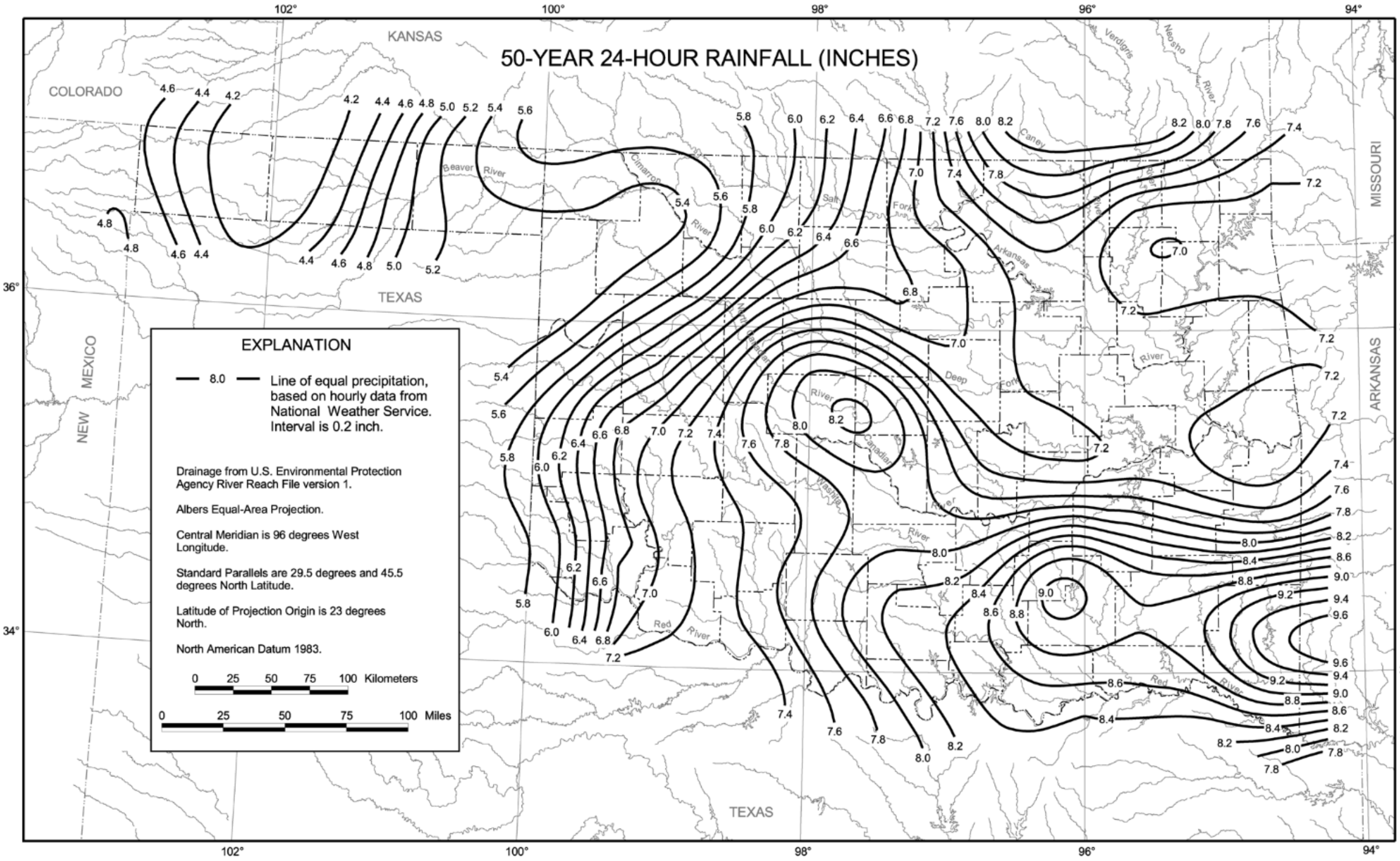
Source: USGS (3)

Figure 7.7-N — OKLAHOMA 10-YEAR, 24-HR RAINFALL (in)



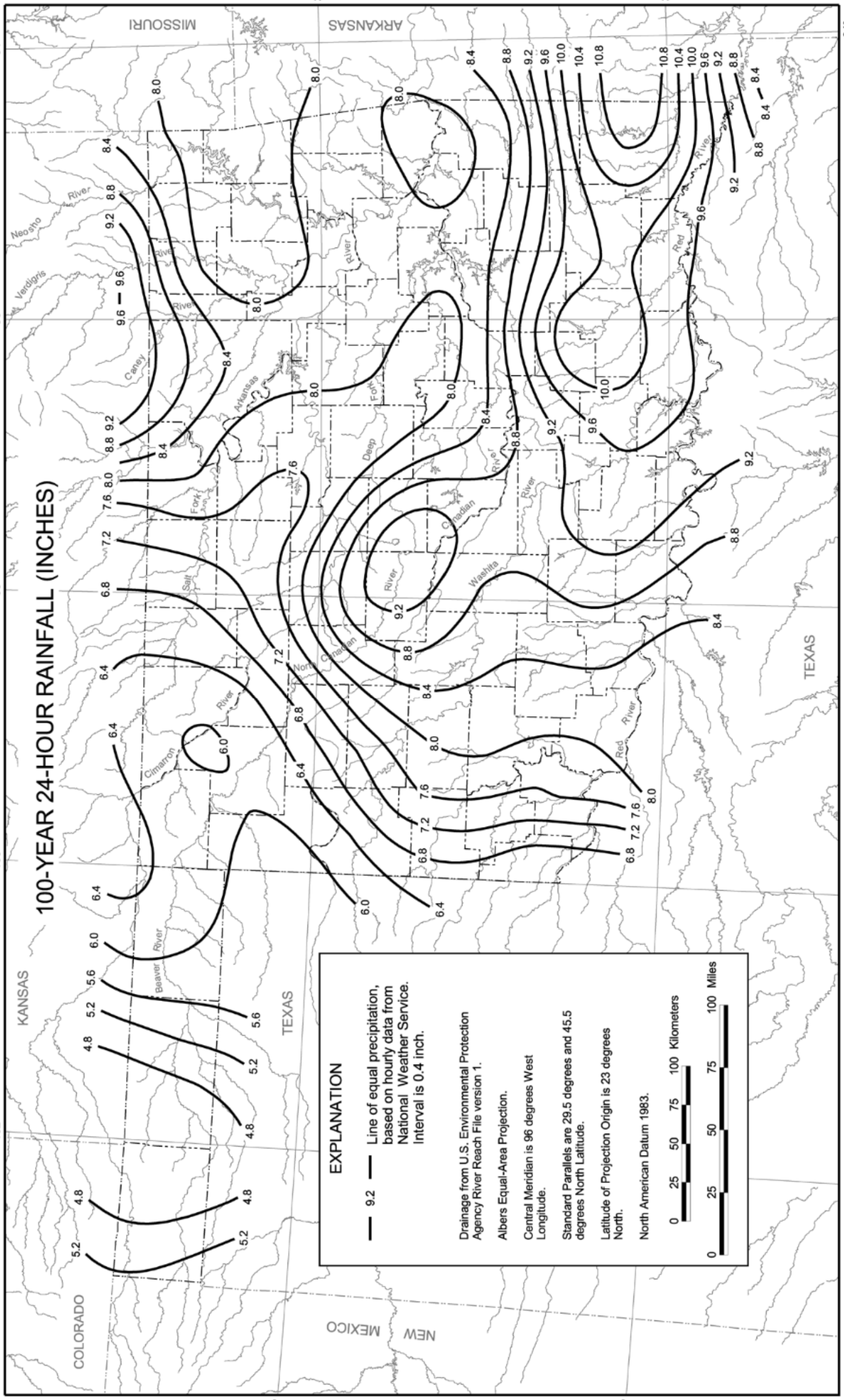
Source: USGS (3)

Figure 7.7-O — OKLAHOMA 25-YEAR, 24-HR RAINFALL (in)



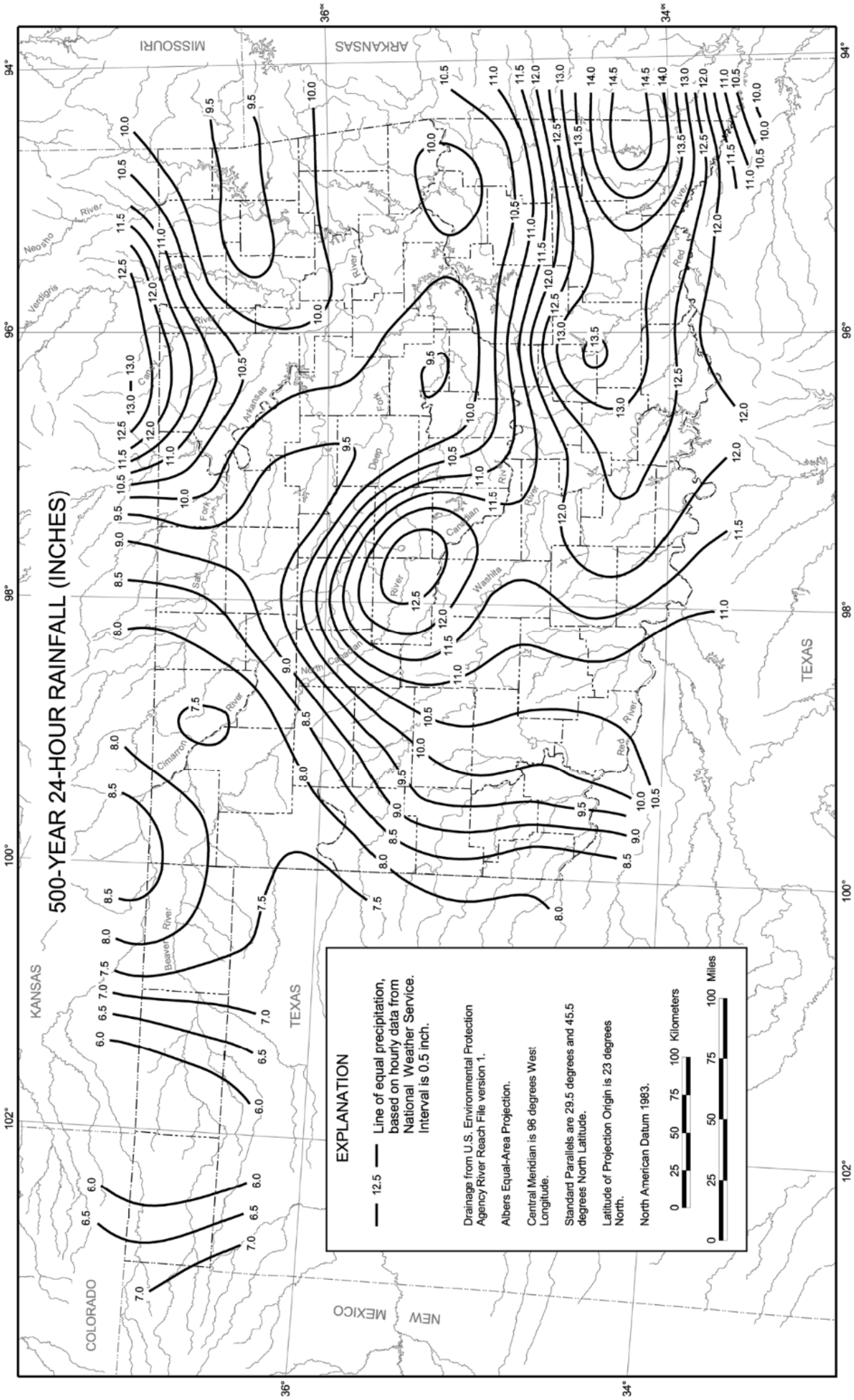
Source: USGS (3)

Figure 7.7-P — OKLAHOMA 50-YEAR, 24-HR RAINFALL (in)



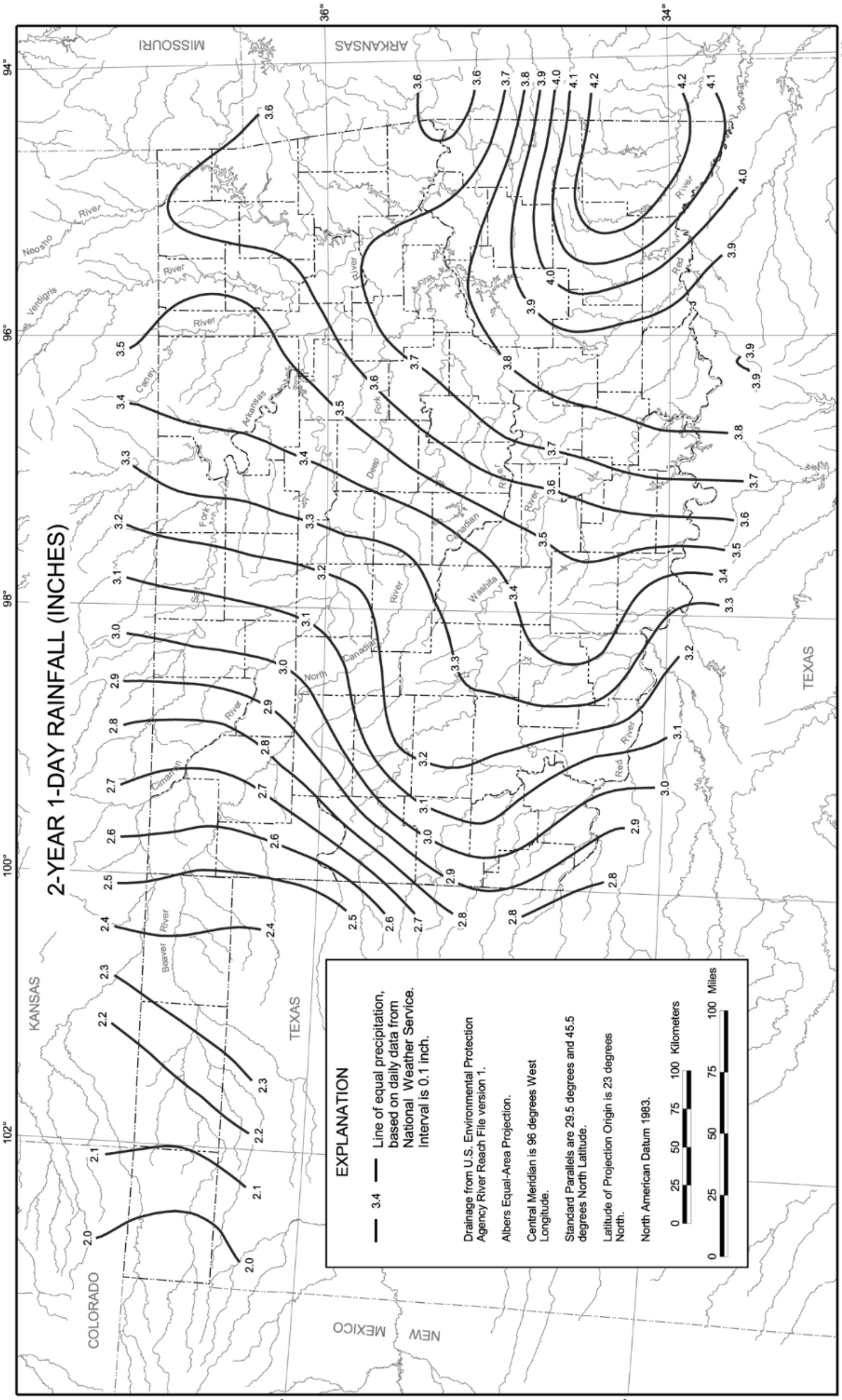
Source: USGS (3)

Figure 7.7-Q — OKLAHOMA 100-YEAR, 24-HR RAINFALL (in)



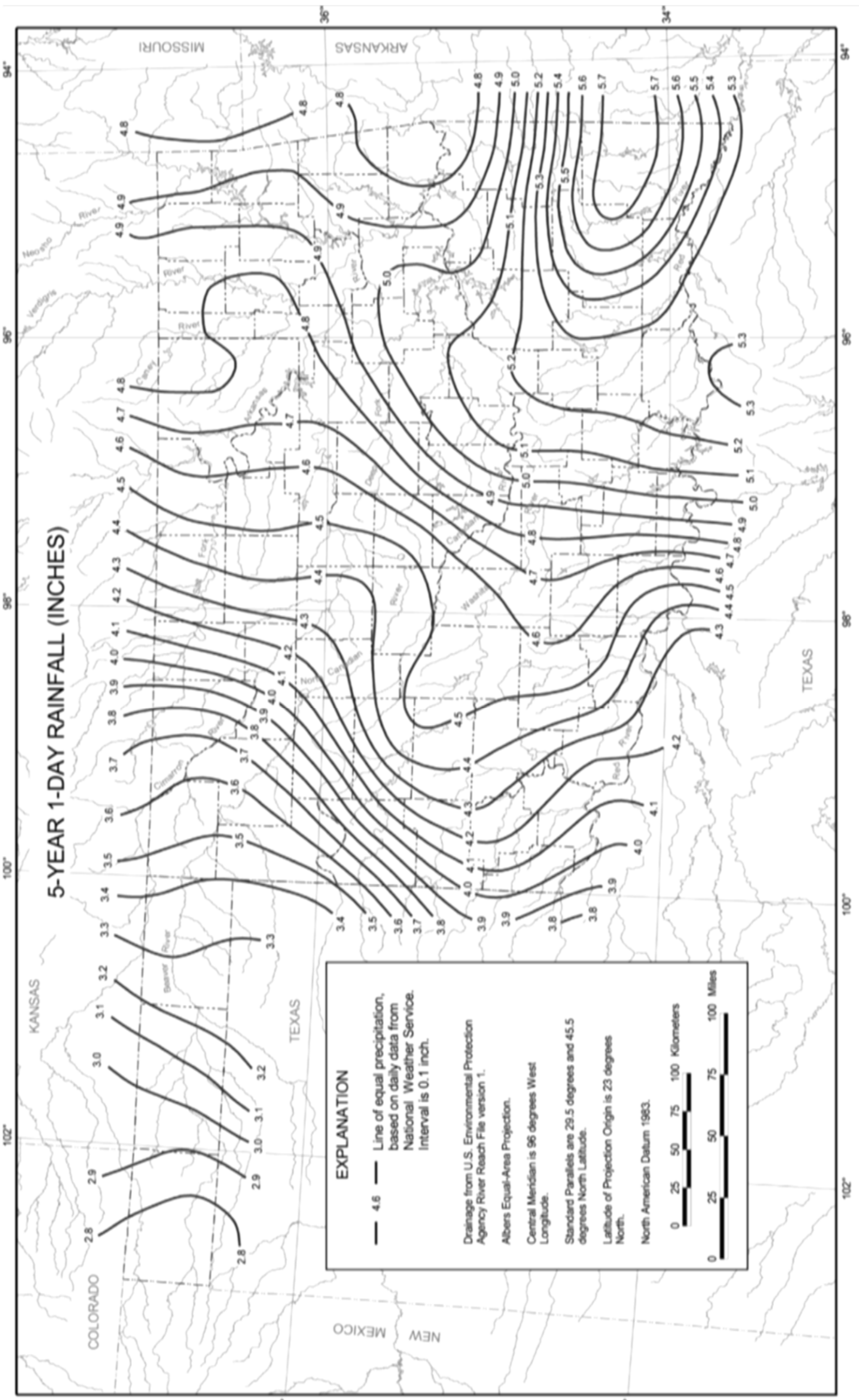
Source: USGS (10)

Figure 7.7-R — OKLAHOMA 500-YEAR, 24-HR RAINFALL (in)



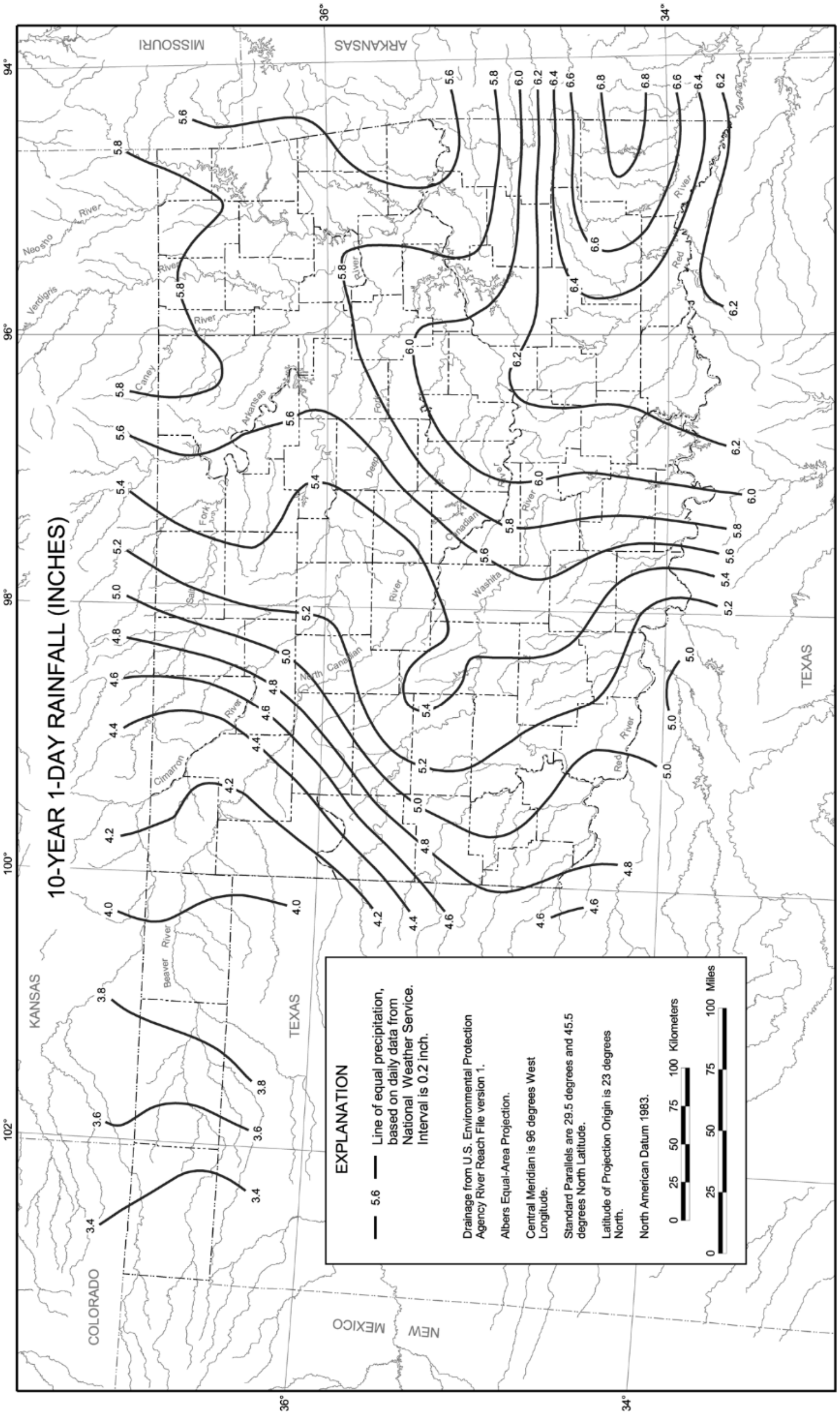
Source: USGS (3)

Figure 7.7-S — OKLAHOMA 2-YEAR, 1-Day RAINFALL (in)



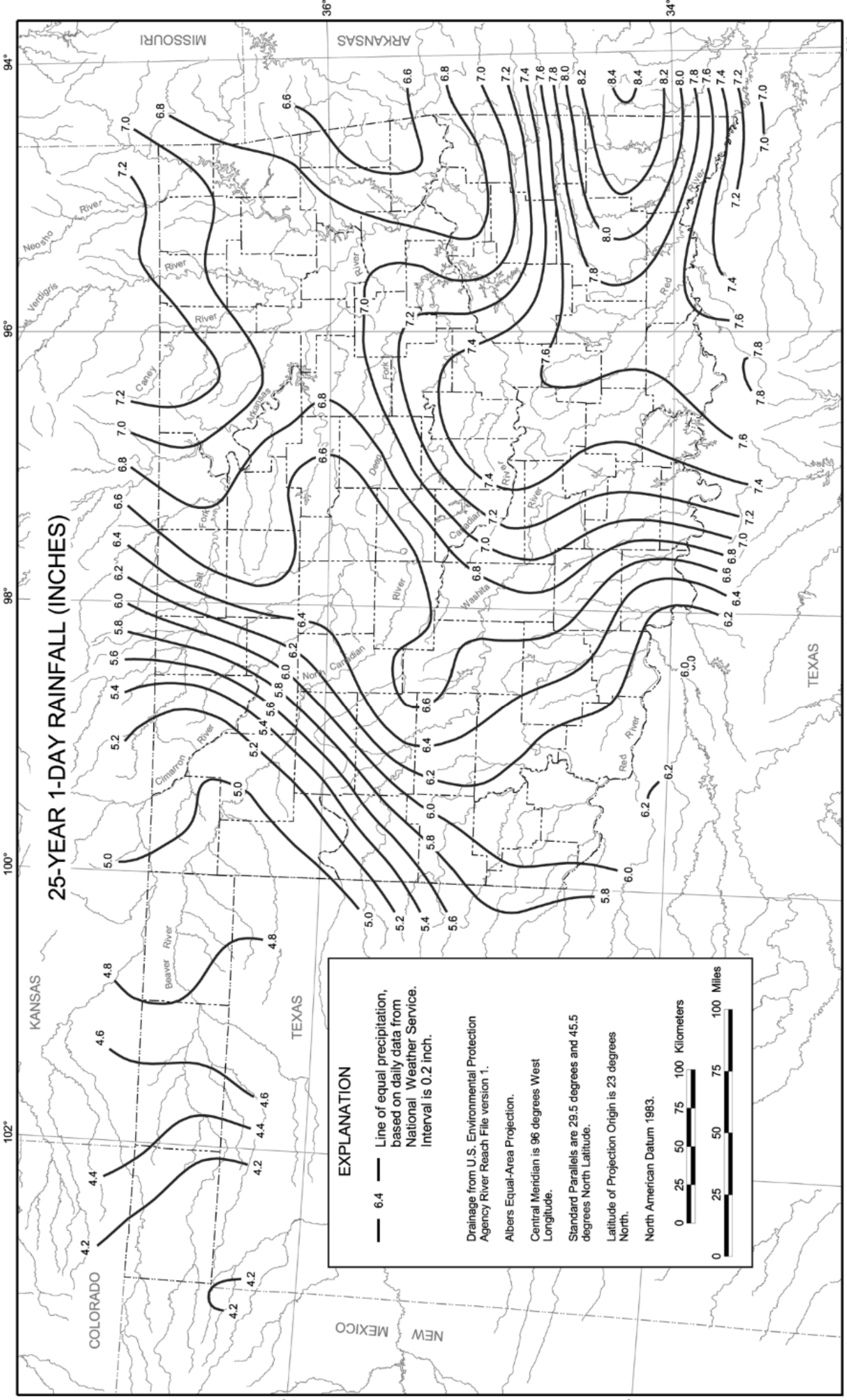
Source: USGS (3)

Figure 7.7-T — OKLAHOMA 5-YEAR, 1-Day RAINFALL (in)



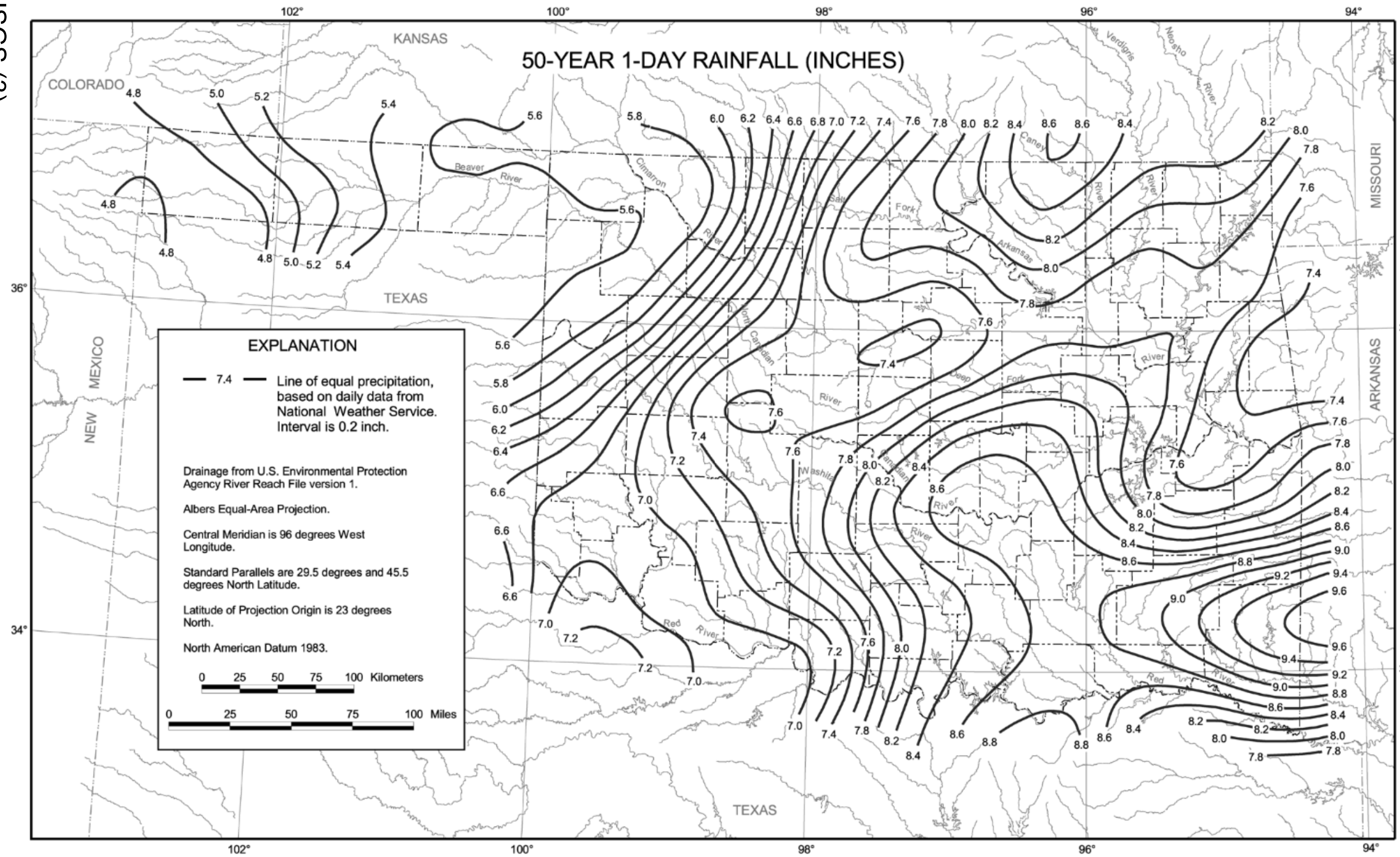
Source: USGS (3)

Figure 7.7-U — OKLAHOMA 10-YEAR, 1-Day RAINFALL (in)



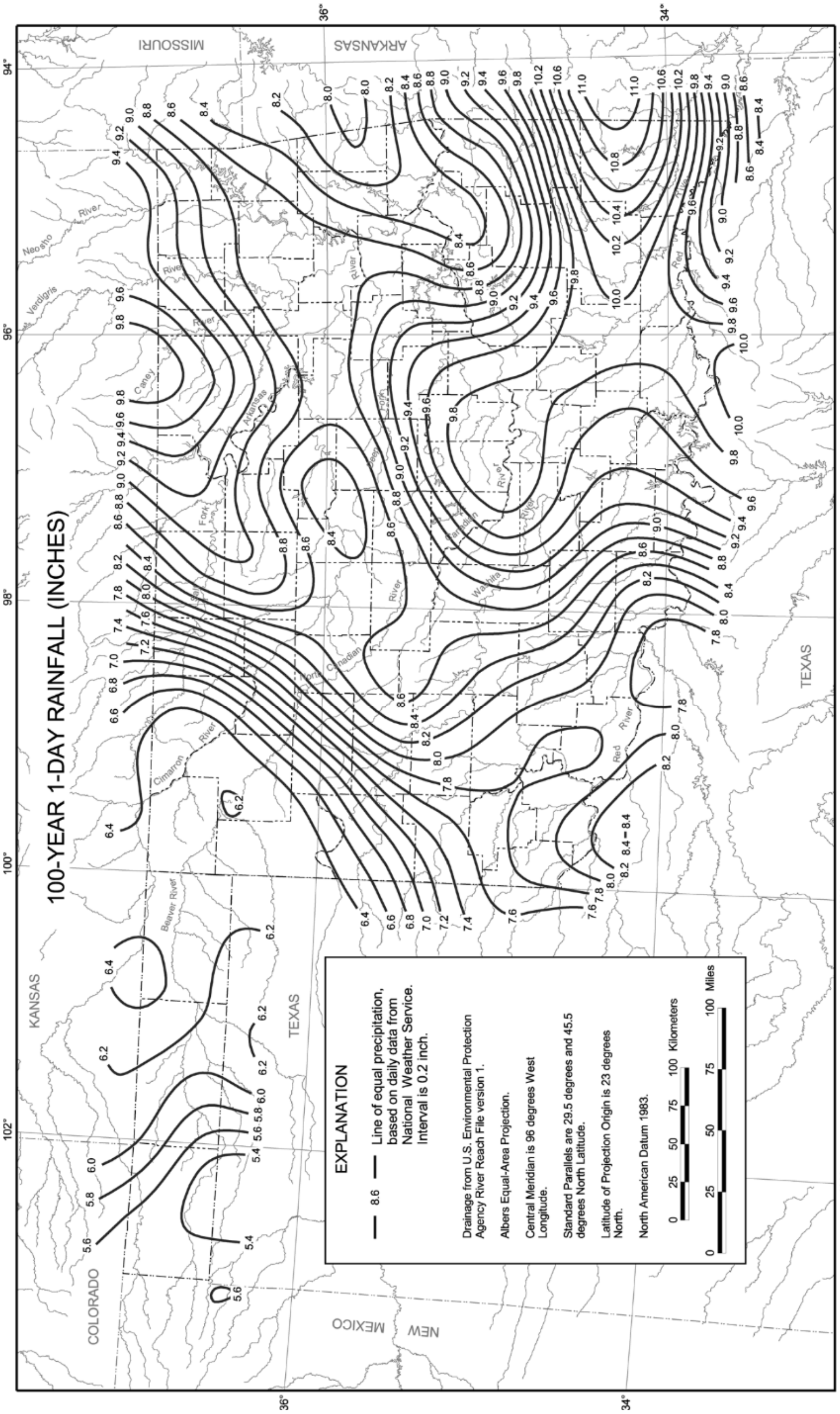
Source: USGS (3)

Figure 7.7-V — OKLAHOMA 25-YEAR, 1-Day RAINFALL (in)



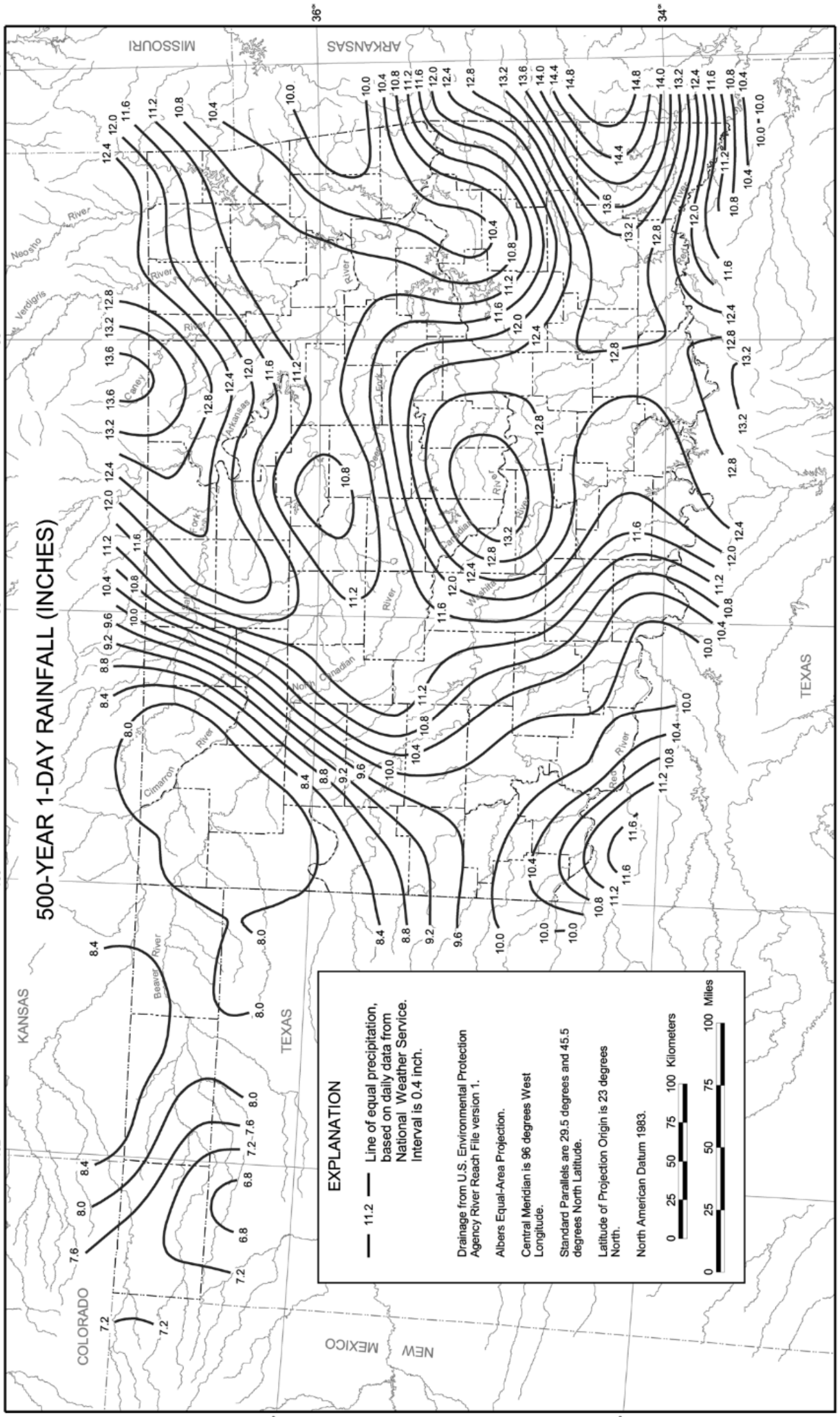
Source: USGS (3)

Figure 7.7-W — OKLAHOMA 50-YEAR, 1-Day RAINFALL (in)



Source: USGS (3)

Figure 7.7-X — OKLAHOMA 100-YEAR, 1-Day RAINFALL (in)



Source: USGS (3)

Figure 7.7-Y — OKLAHOMA 500-YEAR, 1-Day RAINFALL (in)

Antecedent Condition	Condition's Description	Growing Season Antecedent Rainfall	Dormant Season Antecedent Rainfall
Dry	An optimum condition of watershed soils, where soils are dry but not to the wilting point and when satisfactory plowing or cultivation takes place	Less than $1^1/_2$ in	Less than ½ in
Average	The average case for annual floods	$1^1/_2$ in to 2 in	$^1/_2$ in to 1 in
Wet	When a heavy rainfall or light rainfall and low temperatures, have occurred during the five days previous to a given storm	Over 2 in	Over 1 in

Source: NRCS (10)

Figure 7.7-Z — RAINFALL GROUPS FOR ANTECEDENT SOIL MOISTURE CONDITIONS DURING GROWING AND DORMANT SEASONS

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

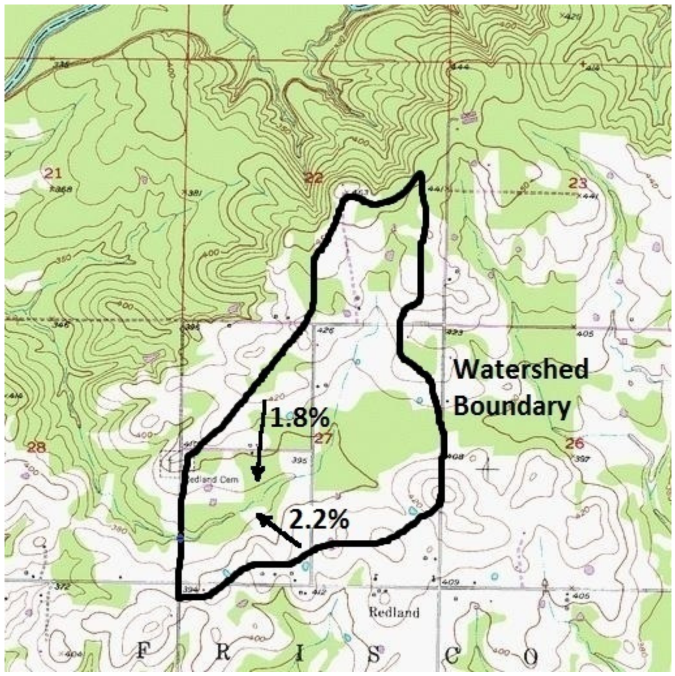
Source: NRCS (10)

Figure 7.7-AA — CONVERSION FROM AVERAGE ANTECEDENT MOISTURE CONDITIONS TO DRY AND WET CONDITIONS

7.7.7 Example Problem—NRCS TR-55 Peak Discharge Method

Following is an example problem illustrates the application of the NRCS TR-55 Peak Discharge Method to calculate the 10-year and 100-year peak discharges for the same watershed used in the Rational Method example (Section 7.6.9).

Step 1. Determine the boundary of the watershed and its area.



The watershed boundary is shown on the "SHULTS.OK" USGS 7.5 minutes quadrangle map. The area of the watershed was measured with a planimeter and determined to be 593 acres. No NRCS flood control structures are within the watershed.

Step 2. Compute the average slope of the watershed.

As shown on the map, the area north of the stream has a slope of 1.8% and about 50% of the area. The area south of the stream has a slope of 2.2% for the remaining 50% of the watershed.

The average slope of the watershed is:

$$S = 0.5(2.2) + 0.5(1.8) = 2.0\%$$

Step 3. Determine the anticipated future land use (AFLU).

From the USGS map and aerial photos, the anticipated future land is assumed to be:

- 50% of the area will be woodland
- 50% of the area will be pasture

<u>Step 4</u>. Define the Type of rainfall and the amount of rainfall.

- Rainfall Type
  - o McCurtain County is in Zone 1
  - o Use NRCS type III rainfall
- Rainfall Amount
  - o Site is at Latitude: N33.9°, Longitude: W94.7°
  - o Figure 7.7-L gives a 24-hr P10 = 6.50 in at the site
  - o Figure 7.7-O gives a 24-hr P100 = 10.00 in at the site

<u>Step 5</u>. Determine the runoff curve number (CN) for the watershed.

a. Determine Soil Types in the watershed

Use the NRCS online soil map to outline the watershed and generate the percent of the area that is each soil group (http://websoilsurvey.nrcs.usda.gov/app/):

The online tool generates the following table for the area of interest (AOI):

McCurtain County, Oklahoma (OK 089)			
Map Unit Symbol	**Map Unit Name**	**Acres in AOI**	**Percent of AOI**
Ad	Adaton loam, 0% to 3% slopes	188.2	31.4%
Bk	Boggy-Pushmataha complex, 0% to 1% slopes, frequently flooded	5.5	0.9%
KuB	Kullit fine sandy loam, 1% to 3% slopes	119.7	19.9%
MuC	Muslogee loam, 1% to 3% slopes	108.9	31.0%
RuD	Ruston fine sandy loam, 3% to 8% slopes	62.5	10.4%
SwE	Swirk-Hollywood complex, 5% to 20% slopes	10.1	1.7%
TkC	Tiak-Ruston complex, 1% to 5% slopes	25.4	4.2%
TkE	Tiak-Ruston complex, 5% to 15%	3.0	0.5%
Totals for Area of Interest		**600.3**	**100.0%**

b. Determine Hydrologic Soil Group

Find the soil name and hydrologic group in Exhibit A of 2nd Edition of TR-55.

c. Find CN

Assign a cover type using the soil map photo.

Soil	Soil Name	Group	Cover Description	CN	Percent	CN(%)
Ad	Adaton	D	Pasture, good	80	15.7	1256
Ad	Adaton	D	Woodland, good	77	15.7	1208.9
Bk	Boggy	C	Woodland, good	79	0.9	71.1
KuB	Kullit	B	Woodland, fair	60	19.9	1194
MuB	Muskogee	C	Woodland, good	79	15.5	1224.5
MuB	Muskogee	C	Pasture, fair	79	15.5	1224.5
RuD	Ruston	B	Pasture, good	61	10.4	634.4
SwE	Swink	D	Pasture, good	80	1.7	136
TkC	Tiak	C	Woodland, fair	73	4.2	306.6
TkE	Tiak	C	Woodland, fair	73	0.5	36.5
					100	7292.5

The weighted CN = sum of CN(%)/100 = 7292.5/100 = 72.9
Use CN = 73 for design

d. Adjust for Moisture Condition – no adjustment needed

e. Adjust for Imperious Areas – no adjustment needed

Step 6. Compute the direct runoff, Q

$S = (1000/CN) - 10 = (1000/73) – 10 = 3.7$ in
$Q = (P – 0.2S)^2/(P + 0.8S)$
$Q_{10} = [6.50 – 0.2(3.7)]^2/[6.50 + 0.8(3.7)] = 33.18/9.46 = 3.51$ in
$Q_{100} = [10.00 – 0.2(3.7)]^2/[10.00 + 0.8(3.7)] = 85.75/12.96 = 6.62$ in

<u>Step 7</u>. Compute the time of concentration, t_c

Method 1 (see Section 7.7.3)

The flow path consists of the following:

- 300 ft of sheet flow through pasture with grass cover with a slope of 0.018 ft/ft.
- 1800 ft of shallow concentrated flow on unpaved pasture with a slope of 0.018 ft/ft.
- 7000 ft of channel flow at an average slope of 0.00764 ft/ft. The survey shows the hydraulics characteristics of the channel at bank full depth are as follows:

Size	Trapezoidal Channel
Bottom width	15.00 ft
Side slope right	3:1
Side slope left	3:1
Bank full depth	4.00 ft
Area of opening at bank full depth	124.00 ft^2
Wetted perimeter	47.99 ft
Hydraulics radius	2.58
Channel slope	0.00764 ft/ft
Manning n (average stream with trees within channel)	0.95

Sheet Flow is calculated using Equation 7.7(5), which requires:

- n = 0.15 for short grass prairie (pasture) from Figure 7.7-B
- P_2 = 4 in from Figure 7.7-I for McCurtain County, OK
- S = 0.018 ft/ft
- L = 300 ft (maximum)

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

$T_{t(sheet)}$ = 0.21(104.9) = 22 minutes

Shallow concentrated flow is calculated using Equation 7.7(6) for the remaining 1800 ft of flow through the pasture, using the unpaved equation 7.7(8):

S = 0.018 ft/ft
V = 16.1345((0.018^0.5) = 2.16 fps
T_t (shallow concentrated flow) = L/60V = 1800/[60(2.16)] = 13.89 minutes

Channel flow is calculated using Equation 7.7(6) and 7.7(9) and based on the above hydraulics characteristics of the 7000 ft of channel flow at bank full depth:

$V = (1.486/0.095)(2.58\text{^}(2/3))*(0.00764\text{^}0.5)$

$V = 2.57$ fps

$T_{t(channel)} = L/60V = 7000/[60(2.57)] = 45.40$ minutes

$t_c = T_{t(sheet)} + T_{t(shallow)} + T_{t(channel)}$

$t_c = 22 + 13.89 + 45.40 = 81.29$ minutes = 1.35 hrs

ODOT Worksheet for TR-55 Time of Concentration (T_c) or Travel Time (T_t)

Hydraulics Designer: Te Anh Ngo	**Date:** 16 April 2013
Project: NRCS Example 1	**Stream:** No Name Creek
Location: McCurtain County between Section 27 & 28, T7S, R5E	

Sheet Flow	Segment 1	Segment 2	Sum 1 & 2
Surface description (Figure 7.7-A to D)	Grass		
Manning's n (Figure 7.7-G)	0.150		
Flow length, L (L≤ 300 ft), ft	300		
Two-year 24-hr rainfall, in	4.00		
Land Slope, ft/ft	0.018		
T_t, hours (Equation 7.7(4))	0.37		0.37

Shallow Concentrated Flow	Segment 1	Segment 2	Sum 1 & 2
Surface description	Unpaved		
Flow length, L, ft	1800		
Watercourse slope, ft/ft	0.0180		
Average velocity, fps (Eq. 7.7(8) or (9))	2.16		
T_t, = L/(3600V) hours	0.23		0.23

Channel Flow	Segment 1	Segment 2	Sum 1 & 2
Cross sectional flow area, sq ft	124		
Wetted Perimeter, P, ft	47.99		
Hydraulic Radius, ft	2.58		
Channel Slope, ft/ft	0.00764		
Manning, n	0.095		
Velocity, fps (Equation 7.7(10))	2.57		
Flow length, ft	7000		
T_t, = L/(3600V) hours	0.76		0.76

Total Time of Concentration (T_c) or Travel Time (T_t) =	1.35	hours
Lag time = 0.6 T_c =	0.812	hours

Figure 7.7-BB — COMPUTATION OF TIME OF CONCENTRATION t_C

Method 2 (NRCS TP-149)

Because the area of this watershed is only 593 acres (< 2000 acres), the hydraulics designer could use Equation 7.7(14) to compute the lag time:

$$L = [(L_m^{0.8})((S + 1)^{0.7})]/[(1900(Y^{0.5})], \text{ hr}$$
$$L = [(9100^{0.8})((3.7 + 1)^{0.7})]/[1900(2^{0.5})] = 1469.7(2.95)/2687 = 1.6 \text{ hr}$$

Where:

L_m = length of mainstream to farthest divide = 9100 ft
Y = average slope of the watershed = 2% from Step 2
S = potential maximum retention = (1000/CN) – 10 = 3.7 in
L = lag time, hr
t_c = 1.67L = 1.67(1.6) = 2.67 hr

The computation by the two methods above shows that there is significant difference between the value of the time of concentration (t_c) from Method 1 (t_c = 1.35 hr.) and from method 2 (t_c = 2.67 hr.).

However, the time of concentration (t_c) computed from Method 1 (t_c = 1.35 hr) is closer to that computed by the Rational method (t_c = 89 minutes = 1.48 hr (see Section 7.6.9) than that computed by method 2 (t_c = 2.67 hr.)

Therefore, the hydraulics designer should use the time of concentration t_c = 1.35 hr for design.

Step 8. Compute the unit peak discharge (q_u)
The unit peak discharge (q_u) is computed with Equation 7.8(12):

$$q_u = 10^{co + c1 \log Tc + c2 [\log(Tc)]^2}$$

Where:

q_u = unit peak discharge, cfs/square mile/in
t_c = time of concentration from Step 7
C_0, C_1, C_2 are constants from Figure 7.7-I

- $I_a = 0.2S = 0.2(3.7) = 0.74$
- $I_a/P_{10} = 0.74/6.50 = .11$
- $I_a/P_{100} = 0.74/10.00 = .07$

- Use coefficients for Rainfall Type III and $I_a/P = 0.1$

- $C_0 = 2.47317$, $C_1 = -0.51848$, $C_2 = -0.17083$
- $x = C_0 + C_1[\log(t_c)] + [C_2\log(t_c)]^2$
- $x = 2.47317 - 0.51848[\log(1.35)] - 0.17083[\log(1.35)]^2$
- $x = 2.47317 - 0.06758 – 0.00290 = 2.40269$

- $q_u = 10^x = 102.40269 = 252.75$ cfs/square mile/in

Step 9. Compute the peak discharge (q_p)

The peak discharge (q_p) is computed with Equation 7.7(10):

$$q_p = q_u AQ$$

Where:

q_u = unit peak discharge in cfs/square mile/in. of runoff
A = drainage area in square miles = 593 acres/640 = 0.926 square miles
Q = depth of runoff in in from Step 6

$$q_{p10} = q_u AQ_{10} = (252.75)(0.926)(3.51) = 821 \text{ cfs}$$

$$q_{p100} = q_u AQ_{100} = (252.75)(0.926)(6.62) = 1549 \text{ cfs}$$

Step 10. Peak discharge adjustment for pond and swamp.

Because the area in ponds or swamps is less than 10%, the peak discharge from Step 9 is used without adjustment.

Step 11. Check the result using USACE HEC-HMS program.

The hydraulics designer could check the above manual computation by using the USACE HEC-1 or HEC-HMS program, using the NRCS hydrograph method option. Copies of the USACE HEC-1 and HECHMS runs are as shown below.

Step 12. Evaluate the result.

The computation above shows that for this same example watershed, the peak discharges obtained from the NRCS method are much larger than that computed by the Rational method. This fact does not indicate that the NRCS is more conservative or better than the Rational method.

The choice of using the peak discharges computed by either the NRCS method or the Rational Method in the design should be based on additional factors, including but not limited to:

- The hydraulics characteristics of the existing structure.
- The highwater marks recorded at the site and the rainfall frequencies corresponding with these highwater marks.

Assume that the survey information shows that the highwater mark observed at the site is at elevation 100.00 on the 28 of May of the year 1981.

A check with the National Weather Bureau shows that is recorded a total rainfall amount of 6.5 inches on that date.

A check with a local resident who has lived more than 30 year in the area confirmed that the duration of this rainfall is about 6 hours.

Based on this information, a check with the USGS-WRI-99-4232 shows that the rainfall that caused this highwater mark may have the frequency of 100-year return period.

The hydraulics designer should try to recreate the highwater mark by analyzing the existing structure, using the 100-year discharge data from the NRCS and Rational methods.

Assuming the analysis shows that the 100-year elevation using the NRCS method Q_{100} is 101.50 ft and that by the Rational method Q_{100} is 99.50 ft.

Because the highwater mark elevation computed by the Rational method (El. 99.50) is much closer to the real highwater mark recorded at the site (El. 100.00) than that computed by the NRCS method (El. 101.50), the hydraulics designer should use the peak discharges computed by the Rational method in the design.

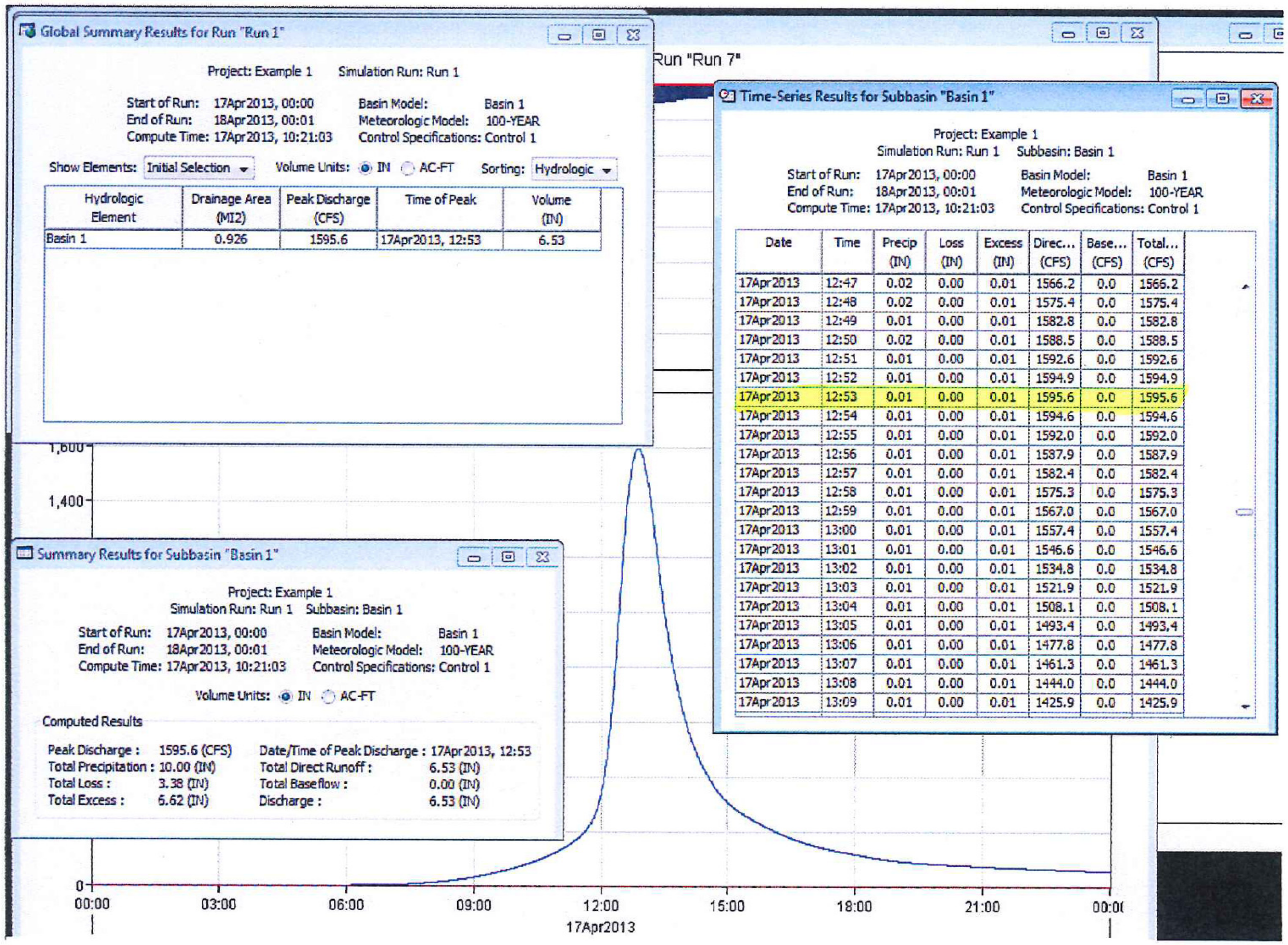


Figure 7.7-CC — HEC-HMS OUTPUT - NRCS EXAMPLE - 100-YEAR

```
FLOOD HYDROGRAPH PACKAGE(HEC-1)VERSION 4.0.1E MAY 1991
USACE,HYDROLOGIC ENGINEERING CENTER, 609 SECOND STREET, DAVIS, CALIFORNIA 95616
RUN DATE 04-16-2013 TIME 4:00PM
                                        HEC-1 INPUT                                          PAGE 1

     LINE           ID.......1.......2.......3.......4.......5.......6.......7.......8.......9......10
        1           ID        Flood routing - Run 1 - 04-15-2013 NRCS Method Example2
        2           ID        Type III Rainfall Distribution
        3           ID        100 year storm - 24 hour duration - 10.00
        4           ID
        5           ID
        6           ID        Use SCS 6-minute type II distribution storm
        7           IT       6 15APR13       0     300 15APR13    2400
        8           IO       5
        9           PG       1   10.00
       10           IN       6 15APR13       0     300 15APR13    2400
       11           PC  .00010  .00200  .00300  .00400  .00500  .00600  .00700  .00800  .00900  .01000
       12           PC  .01100  .01200  .01300  .01400  .01500  .01600  .01700  .01800  .01900  .02000
       13           PC  .02100  .02200  .02300  .02400  .02500  .02600  .02700  .02800  .03000  .03100
       14           PC  .03200  .03300  .03400  .03500  .03700  .03800  .03900  .04000  .04200  .04300
       15           PC  .04400  .04600  .04700  .04800  .05000  .05100  .05200  .05400  .05500  .05700
       16           PC  .05800  .06000  .06100  .06300  .06400  .06600  .06700  .06900  .07000  .07200
       17           PC  .07400  .07500  .07700  .07900  .08100  .08300  .08400  .08600  .08800  .09000
       18           PC  .09300  .09500  .09700  .09900  .10200  .10400  .10600  .10900  .11000  .11400
       19           PC  .11700  .11900  .12200  .12500  .12800  .13200  .13500  .13800  .14200  .14600
       20           PC  .15000  .15300  .15800  .16200  .16600  .17000  .17500  .17900  .18400  .18900
       21           PC  .19400  .19900  .20500  .21100  .21600  .22300  .22900  .23600  .24300  .25000
       22           PC  .25800  .26600  .27600  .28700  .29800  .31400  .33900  .37300  .41600  .50000
       23           PC  .58400  .62700  .66100  .68600  .70200  .71300  .72400  .73400  .74200  .75000
       24           PC  .75700  .76400  .77100  .77700  .78400  .78900  .79500  .80100  .80600  .81100
       25           PC  .81600  .82100  .82500  .83000  .83400  .83800  .84200  .84700  .85000  .85400
       26           PC  .85800  .86200  .86500  .86800  .87200  .87500  .87800  .88100  .88300  .88600
       27           PC  .88900  .89100  .89400  .89600  .89800  .90100  .90300  .90500  .90700  .91000
       28           PC  .91200  .91400  .91600  .91800  .91900  .92100  .92300  .92500  .92600  .92800
       29           PC  .93000  .93100  .93300  .93400  .93600  .93700  .93900  .94000  .94200  .94300
       30           PC  .94500  .94600  .94800  .94900  .95000  .95200  .95300  .95400  .95600  .95700
       31           PC  .95800  .96000  .96100  .96200  .96300  .96500  .96600  .96700  .96800  .96900
       32           PC  .97100  .97200  .97300  .97400  .97500  .97600  .97700  .97900  .98000  .98100
       33           PC  .98200  .98300  .98400  .98500  .98600  .98700  .98800  .98900  .99000  .99100
       34           PC  .99200  .99300  .99400  .99500  .99600  .99700  .99750  .99800  .99900  1.0000
       35           JR    PREC   0.400   0.540   0.650   0.780   0.880   1.000   1.250
       36           KK       1  Drainage area A1 = 593 ac= 0.92656 sq.miles
       37           KM          SCS Unit Hydrograp -
       38           BA  .92656
       39           PR       1
       40           PW       1
       41           LS       0   73.00
       42           UD   0.812
       43           ZZ
```

Figure 7.7-DD — USACE HEC-1 PROGRAM OUTPUT

Figure 7.7-DD — USACE HEC-1 PROGRAM OUTPUT (CONTINUED)

```
                    SCHEMATIC DIAGRAM OF STREAM NETWORK
INPUT
 LINE      (V) ROUTING          (--->) DIVERSION OR PUMP FLOW
  NO.      (.) CONNECTOR        (<---) RETURN OF DIVERTED OR PUMPED FLOW
  36           1
                                 Flood routing - Run 1 - 04-15-2013 NRCS Method Example2
                                 Type III Rainfall Distribution
                                 100 year storm - 24 hour duration - 10.00
                                 Use SCS 6-minute type II distribution storm

   8 IO          OUTPUT CONTROL VARIABLES
                       IPRNT           5  PRINT CONTROL
                       IPLOT           0  PLOT CONTROL
                       QSCAL          0.   HYDROGRAPH PLOT SCALE

     IT          HYDROGRAPH TIME DATA
                        NMIN           6  MINUTES IN COMPUTATION INTERVAL
                       IDATE     15APR13  STARTING DATE
                       ITIME        0000  STARTING TIME
                          NQ,        241  NUMBER OF HYDROGRAPH ORDINATES
                      NDDATE     16APR13  ENDING DATE
                      NDTIME        0000  ENDING TIME
                      ICENT           19  CENTURY MARK

                   COMPUTATION INTERVAL    0.10 HOURS
                        TOTAL TIME BASE   24.00 HOURS

          ENGLISH UNITS
               DRAINAGE AREA          SQUARE MILES
               PRECIPITATION DEPTH    INCHES
               LENGTH, ELEVATION      FEET
               FLOW                   CUBIC FEET PER SECOND
               STORAGE VOLUME         ACRE-FEET
               SURFACE AREA           ACRES
               TEMPERATURE            DEGREES FAHRENHEIT

     JP          MULTI-PLAN OPTION
                       NPLAN           1 NUMBER OF PLANS

     JR          MULTI-RATIO OPTION
                     RATIOS OF PRECIPITATION
                   0.40      0.54      0.65      0.78      0.88      1.00      1.25

               PEAK FLOW AND STAGE (END-OF-PERIOD) SUMMARY FOR MULTIPLE PLAN-RATIO ECONOMIC COMPUTATIONS
                                 FLOWS IN CUBIC FEET PER SECOND, AREA IN SQUARE MILES
                                                 TIME TO PEAK IN HOURS

                                                          RATIOS APPLIED TO PRECIPITATION
OPERATION       STATION     AREA    PLAN           RATIO 1  RATIO 2  RATIO 3  RATIO 4  RATIO 5  RATIO 6  RATIO 7
                                                      0.40     0.54     0.65     0.78     0.88     1.00     1.25

HYDROGRAPH AT
+                     1     0.93       1   FLOW       342.     600.     817.    1082.    1290.    1542.    2071.
                                           TIME      12.90    12.80    12.80    12.80    12.80    12.80    12.80

*** NORMAL END OF HEC-1 ***
```

7.8 USGS REGRESSION EQUATIONS (SIR-2010-5137)

7.8.1 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 hydraulics designers; see TRR No. 896 (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. Oklahoma has regression equations for rural areas (see Section 7.8.2) and for urban areas (see Section 7.8.3).

7.8.2 Oklahoma Rural Regression Equations

7.8.2.1 Introduction

In 2010, the US Geological Survey published USGS Scientific Investigation Report (SIR) 2010-5137 "Methods for Estimating Magnitude and Frequency of Peak Streamflows for Unregulated Streams in Oklahoma" (5). Using Log-Pearson Type III procedures, the Report provides peak-flow frequency estimates for standard recurrence intervals from 2 years to 500 years for 231 gaging stations in and near Oklahoma. For stations used in this study, the data was collected in the water year 2008. In the analysis, only stations with at least 8 years of flood peak data were used. This Section presents an application-oriented treatment of the Oklahoma rural regression equations based on SIR 2010-5137 (i.e., the Section reproduces the necessary information to calculate the peak discharge at ungaged sites).

7.8.2.2 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 mapping software may also be used to determine the drainage area.

2. Non-Contributing Areas. Drainage areas that do not flow toward the outlet of any hydrologic unit 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. Such areas may be large enough to be designated as hydrologic units at any level of the hierarchy if they are within the size range for a given level.

   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 hydrologic unit or 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 hierarchical level being delineated or if they are scattered throughout a drainage area, they should be considered as part of the encompassing delineated hydrologic unit. Note the size of the non-contributing area in the attribute file for the hydrologic unit containing the non-contributing area. Isolated non-contributing areas larger than 3000 acres should be delineated.

In addition, the user should consider the storage covered by lakes, ponds, wetlands, etc., in the watershed. 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. <u>Main-Channel Slope (S), ft per mile</u>. 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.

4. <u>Precipitation (P), in</u>. The mean-annual precipitation, which can be taken from Figure 7.8-A.

7.8.2.3 Regression Equations for Rural Ungaged Sites

Figure 7.8-B presents the equations for rural ungaged sites in Oklahoma.

7.8.2.4 Limitations on Use of Rural Regression Equations

The following limitations should be considered when using the rural regression equations to compute peak-flow frequencies for Oklahoma 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 7.8.3 for urban areas.

- The rural equations should be used only for basin and climatic characteristics that are within the range of characteristics used to develop the regression equations:

    - contributing drainage area (CA) from 1.00 square mile to 2510 square mile;
    - precipitation (P) from16.6 in to 62.1 in; and
    - channel slope (S) from 1.98 ft/mi to 342 ft/mi.

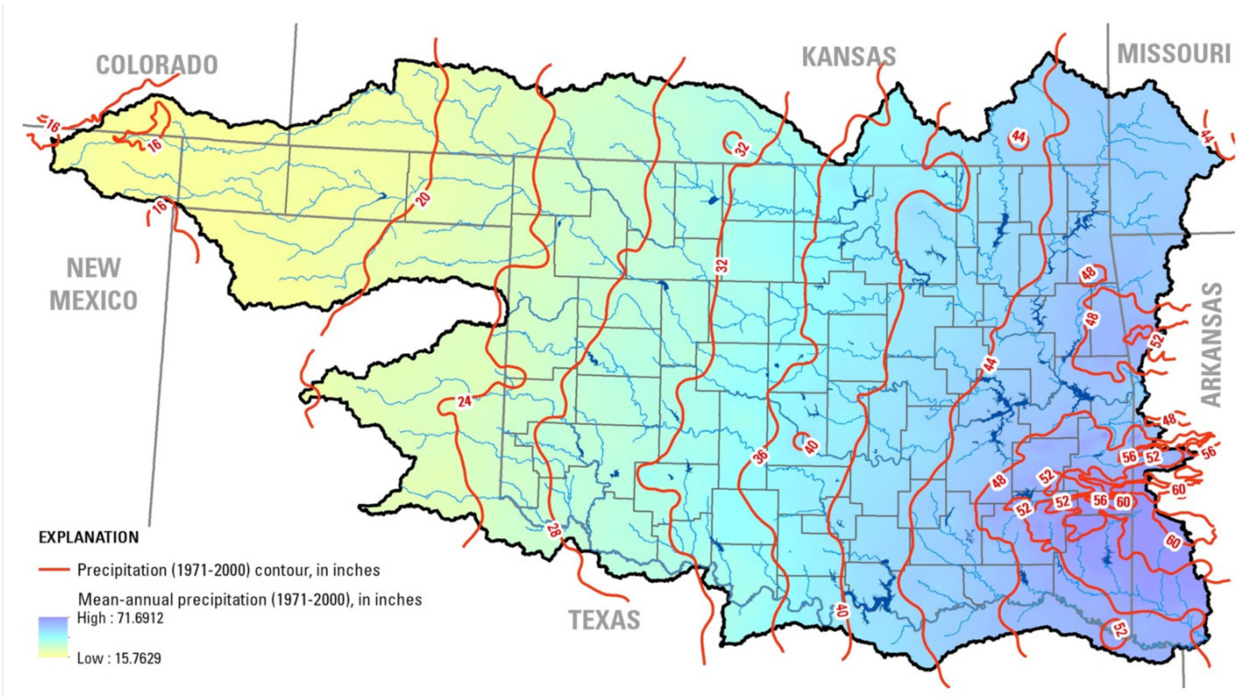


Figure 7.8-A — MEAN ANNUAL PRECIPITATION

Recurrence Interval (years)	Equation	Coefficient of Determination R^2	Average Standard Error of Prediction (%)
2	$Q_2 = 0.064\ CA^{0.66}\ P^{2.06}\ S^{0.16}$	92	47
5	$Q_5 = 0.574\ CA^{0.66}\ P^{1.63}\ S^{0.19}$	95	35
10	$Q_{10} = 1.74\ CA^{0.66}\ P^{1.42}\ S^{0.21}$	96	32
25	$Q_{25} = 4.9\ CA^{0.66}\ P^{1.24}\ S^{0.23}$	95	35
50	$Q_{50} = 13.18\ CA^{0.66}\ P^{1.05}\ S^{0.21}$	95	34
100	$Q_{100} = 26.9\ CA^{0.65}\ P^{0.92}\ S^{0.21}$	95	36
500	$Q_{500} = 126\ CA^{0.64}\ P^{0.64}\ S^{0.19}$	92	43

Figure 7.8-B — RURAL REGRESSION EQUATIONS FOR OKLAHOMA

7.8.2.5 Adjustment for Floodwater Retarding Structures

The equations of Section 7.8.2.3 can be used when the percent of the drainage area regulated by USACE dam/NRCS floodwater retarding structure(s) is not greater than 86% of the total basin. This is accomplished by substituting the area of the drainage basin unregulated (A_u) for the total basin area (CA) in the equations; e.g., if CA = 155 square miles, A_u = 100 square miles, then 55 square miles are regulated by a dam. In this case, A_u = 100 square miles is used in the regression equation. The mean channel slope (S) for the whole basin should be used.

When the percent of regulated drainage area is greater than 86%, the hydraulics designer should use flood routing techniques to calculate the outflow from these dam/flood control structure(s).

7.8.2.6 Example Problem

Determine the 50-year design flood and 100-year review flood discharges for the drainage basin associated with USGS Gage 07160350 Skeleton Creek at Enid, Oklahoma that was used for the gaged site example in Section 7.8.2.3. The basin characteristics from SIR 2010-5137 (5), Table 1, Site 38 are:

- contributing drainage area (CA) is 69.95 square mile
- mean annual precipitation (P) is 34 in
- Stream slope (S) is 12.63 ft/mile

Step 1. Identify the basin characteristics that are necessary to solve the regression equation for the applicable hydrologic sub-region — CA = 69.95, P = 34 and S = 12.63.

Step 2. Select the design frequency from Figure 7.1-A. Use the 50-year design flood and 100-year review flood discharges.

Step 3. Calculate the peak-flow discharge from the applicable regression equation in Figure 7.9-B for the selected design frequencies:

$$\begin{aligned} Q_{50} &= 13.18\ CA^{0.66}\ P^{1.05}\ S^{0.21} \\ &= 13.18\ (69.95)^{0.66}\ (34)^{1.05}\ (12.63)^{0.21} \\ &= 13.18\ (16.50)(40.56)(1.70) \\ &= 15{,}024 \text{ cfs} \end{aligned}$$

$$\begin{aligned} Q_{100} &= 26.9\ CA^{0.65}\ P^{0.92}\ S^{0.21} \\ &= 26.9\ (69.95)^{0.65}\ (34)^{0.92}\ (12.63)^{0.21} \\ &= 26.9\ (15.82)(25.64)(1.70) \\ &= 18{,}549 \text{ cfs} \end{aligned}$$

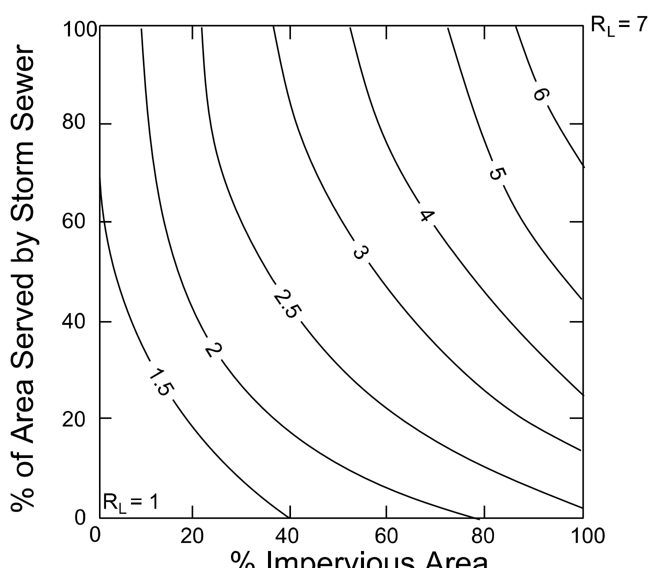
Step 4. The hydraulics designer could also uses the USGS StreamStats program in computing the peak discharges from this location, see Section 7.9.

7.8.3 Oklahoma Urban Regression Equations

The following procedure is applied as an adjustment to the rural regression equations discussed in Section 7.8.2. Since the procedure is ODOT standard practice, it was included in SIR 2010-5137 (5). The percent of the basin that is impervious and the percent of the basin served by storm sewers is required in addition to the variables needed for ungaged sites on unregulated streams to estimate flood magnitude and frequency for ungaged sites on urban streams. The percent of the basin that is impervious can be determined from the StreamStats web application, aerial photographs, recent USGS topographic maps or field surveys. The percent of the basin served by storm sewers should be determined from the best available storm sewer and drainage map. After the percent of the area impervious and area served by storm sewers are obtained, RL, the urban adjustment factor, is obtained from Figure 7.8-D (Leopold, 1968). The urban adjustment factor, RL, is the ratio of the mean annual flood in urban areas to that in rural areas. The following equations (Figure 7.8-C) computed by USGS) can be used to adjust estimates from rural equations (Section 7.8.2) to urban areas (12):

Recurrence Interval (years)	Urban Equation
2	$Q_{2U} = (RL)\ Q_2$
5	$Q_{5U} = 1.60(RL-1)\ Q_2 + 0.167(7-RL)\ Q_5$
10	$Q_{10U} = 1.87(RL-1)\ Q_2 + 0.167(7-RL)\ Q_{10}$
25	$Q_{25U} = 2.21(RL-1)\ Q_2 + 0.167(7-RL)\ Q_{25}$
50	$Q_{50U} = 2.46(RL-1)\ Q_2 + 0.167(7-RL)\ Q_{50}$
100	$Q_{100U} = 2.72(RL-1)\ Q_2 + 0.167(7-RL)\ Q_{100}$
500	$Q_{500U} = 3.30(RL-1)\ Q_2 + 0.167(7-RL)\ Q_{500}$

Figure 7.8-C — URBAN REGRESSION EQUATIONS FOR OKLAHOMA



Source: USGS Circular 554 (13)

Figure 7.8-D — URBAN ADJUSTMENT FACTOR (RL)

7.9 USGS STREAMSTATS PROGRAM

The StreamStats program for Oklahoma was developed in cooperation with the ODOT and the Oklahoma Water Resources Board (OWRB) and was made available to the public on April 15, 2010.

StreamStats determines drainage-basin boundaries by using digital elevation data. It incorporates statewide USGS Rural regression equations for estimating instantaneous peak flows that have recurrence interval of 2, 5, 20, 25, 50, 200 and 500.

The hydraulics designer can access the StreamStats program for Oklahoma at the following website: http://water.usgs.gov/osw/streamstats/oklahoma.html.

The hydraulics designer can use the StreamStats program to compute the peak discharges from watersheds with area ranging from 1.00 square miles up to 2510 square miles.

7.9.1 Example Problem

Determine the 50-year design flood and 100-year review flood discharges for the drainage basin associated with USGS Gage 07160350 Skeleton Creek at Enid, Oklahoma that was used for the USGS regression equation example, section 7.8.2.6. The basin characteristics from SIR 2010-5137 (5), Table 1, Site 38 are:

- contributing drainage area (CA) is 69.95 square mile,
- mean annual precipitation (P) is 34 in, and
- stream slope (S) is 12.63 ft/mi.

Use the link above to enter the site and use the interactive map to identify gage sites in the project location. In this case, the gage is located SE of Enid. Once the gage is located, use the tool to mark the location of interest. The basin characteristic tool can be used to generate Section 7.9.1.1 and the discharge tool can be used to generate the peak flows shown in Section 7.9.1.2. These reports were cut and pasted from the Streamstats, but they can also be printed.

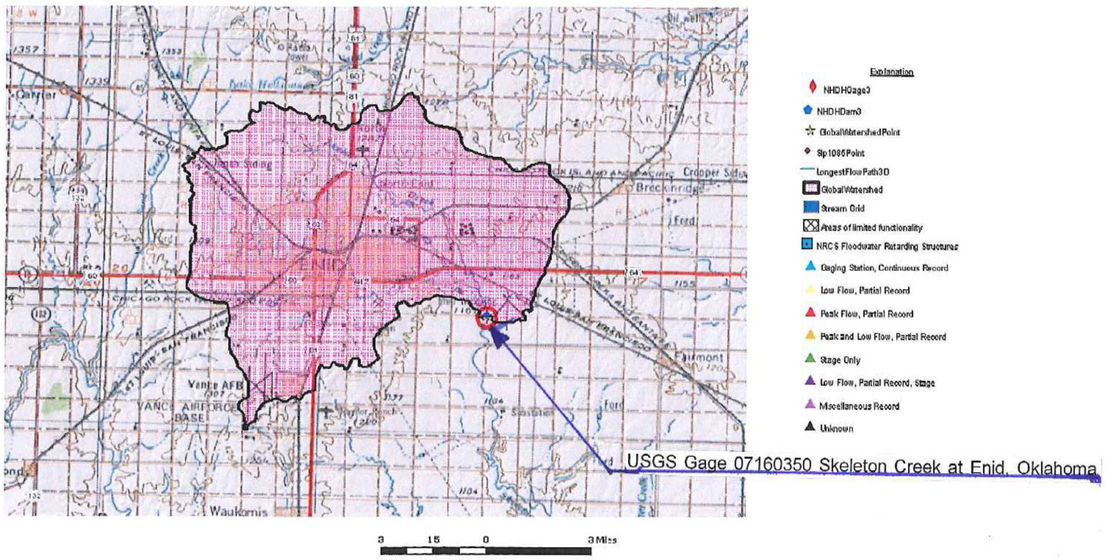
7.9.1.1 Basin Characteristics Report from Streamstats

Date: Tue Oct 23 2012 11:13:06 Mountain Daylight Time
NAD27 Latitude: 36.3759 (36 22 33)
NAD27 Longitude: -97.8005 (-97 48 02)
NAD83 Latitude: 36.3759 (36 22 33)
NAD83 Longitude: -97.8008 (-97 48 03)

USGS StreamStats Page 1 of 1

USGS
Oklahoma StreamStats

StreamStats Print Page



4/18/2013 9:23:26 AM

http://streamstatsags.cr.usgs.gov/ok_ss/default.aspx?stabbr=ok&dt=1366297905996 4/18/2013

Parameter	Value
Contributing drainage area, in square miles	69.96
Contributing drainage area minus the area upstream of NRCS floodwater-retarding structures, in square miles	69.96
Percent of contributing drainage area regulated by NRCS floodwater-retarding structures	0.0
Mean basin elevation, in feet	1260
Elevation at outlet in feet	1110
Mean basin slope in percent, computed from 10-meter DEM	1.60
Longest-flow-path 10-85 slope in feet per mile	12.6
Soil permeability from STATSGO, in inches per hour	2.290
Area-weighted mean annual precipitation (1971-2000), in inches	34.01
Mean annual precipitation (1961-1990) at outlet in inches	32
Mean annual precipitation (1971-2000) at outlet in inches	34.1
Mean June-October precipitation (1971-2000) at outlet in inches	16.7
Mean December-May precipitation (1971-2000) at outlet in inches	17.3
February mean precipitation (1971-2000) at outlet in inches	1.58
March mean precipitation (1971-2000) at outlet in inches	2.7
April mean precipitation (1971-2000) at outlet in inches	3.24
May mean precipitation (1971-2000) at outlet in inches	4.91
June mean precipitation (1971-2000) at outlet in inches	4.35
December mean precipitation (1971-2000) at outlet in inches	1.47
Forest canopy in percent	2.16
Impervious cover in percent	12.34
Percentage of urban land cover determined from NLCD 2001 land cover dataset	35.1

7.9.1.2 Streamstats Ungaged Site Report

Date: Tue Oct 23 2012 11:15:58 Mountain Daylight Time
Site Location: Oklahoma
NAD27 Latitude: 36.3759 (36 22 33)
NAD27 Longitude: -97.8005 (-97 48 02)
NAD83 Latitude: 36.3759 (36 22 33)
NAD83 Longitude: -97.8008 (-97 48 03)
Drainage Area: 69.96 mi^2
Percent Urban: 35.1%
Percent Impervious: 12.34%

Peak-Flow Basin Characteristics			
Parameter	Value	Regression Equation Valid Range	
		Min.	Max.
Contributing Drainage Area (square miles)	70	0.1	2510
Stream Slope 10 and 85 Method ft per mi (feet per mi)	12.6	1.98	342
Mean Annual Precipitation (inches)	34.01	16.6	62.1

Floodwater Retarding Structure Regulated Peak-Flow Basin Characteristics			
Parameter	Value	Regression Equation Valid Range	
		Min.	Max.
Unregulated Drainage Area (square miles)	70	0.1	2510
Stream Slope 10 and 85 Method ft per mi (feet per mi)	12.6	1.98	342
Mean Annual Precipitation (inches)	34.01	16.6	62.1

Flow-Duration Basin Characteristics			
Parameter	Value	Regression Equation Valid Range	
		Min.	Max.
Contributing Drainage Area (square miles)	70	3.65	9056
Jun to Oct Gage Precipitation (inches)	16.7	13	17.9
Mean Basin Elevation (feet)	1260	1121	3188
Stream Slope 10 and 85 Method ft per mi (feet per mi)	12.6	3.73	39.3
Mean June Precipitation (inches)	4.35	3.36	4.44
Elevation of Gage (feet)	1110	923	1866
Mean Basin Slope from 10 m DEM (percent)	1.60 (below min value)	1.76	8.26
Nov to May Gage Precipitation (inches	17.3	12.5	19.2
Average Soil Permeability (inches per hour)	2.290	0.26	3.44
Percent Area Under Canopy (percent)	2.16	0.28	15.5
Mean April Precipitation (inches)	3.24	2.16	3.64
Mean Annual Precipitation at Gage (inches)	34.1	26.1	35.8

Warning: Some parameters are outside the suggested range. Estimates will be extrapolations with unknown errors.

Peak-Flow Streamflow Statistics					
Statistic	Flow (ft^3/s)	Prediction Error (percent)	Equivalent Years of Record	90% Prediction Interval	
				Min.	Max.
PK2	2260	47	2		
PK5	4810	35	5		
PK10	7310	32	8		
PK25	11,500	35	9		
PK50	15,000	34	11		
PK100	18,600	36	12		
PK500	29,500	43	12		

7.10 LOG-PEARSON III

Referring to the Glossary Chapter of the AASHTO *Highway Drainage Guidelines*, 4th Edition, 2007, the Log-Pearson III method, developed by Karl Pearson, is defined as "a series of probability functions to fit virtually any distribution. Although these functions have only slight theoretical basis, they have been used widely in practical statistical works, often in hydrological frequency analysis. The Log-Pearson III method is a skew distribution with limited range in the left direction, usually bell shape but maybe J-shape.

Because the Log-Pearson III is rarely used in the computation of the discharges of roadway drainage structure, no details of this method are provided in this Chapter. Procedures for this method can be found in USGS Bulletin 17B (6) and FHWA HDS 2 (2).

7.11 REFERENCES

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APPENDIX 7.A
RISK ASSESSMENT FORMS

7.A.1 Introduction

FHWA has published the following clarification of 23 CFR 659 Subpart A: Design Standards for Flood Plain Encroachments (23 CFR 650.115(a)(1)):

"The intent of the statement, "as appropriate, a risk analysis or assessment," in Section 23 CFR 650.115(a)(1) is to allow judgment as to the detail of design studies. Where site conditions or structural requirements substantially limit practicable design alternatives, the conventional hydraulic analysis coupled with a risk assessment should meet the requirements of the design standards. Where site conditions permit a range of design alternatives and flood losses are anticipated, an abbreviated or partial risk analysis may be appropriate. We would anticipate that use of the full scale detailed economic (risk) analysis as described in HEC 17 (14) would not be necessary for normal stream crossings, but would apply to unusual, complex or high cost encroachments involving flood losses."

ODOT practice, which is outlined below, is to apply risk assessment if needed. If a site-specific risk assessment indicates that a risk analysis may be useful, it will be considered.

7.A.1.1 No Risk Assessment Needed

No risk assessment documentation is necessary for the following encroachments, which are considered to have minimal or acceptable risk:

- if the ODOT design criteria are used and if no insurable buildings are in the 100-year floodplain, or
- if the encroachment is designed consistent with NFIP criteria.

7.A.1.2 Consider Risk Assessment

The consideration of the potential impacts constitutes an assessment of risk for the specific site. The evaluation of risk is a two-stage process. The initial step, identified as risk assessment, is qualitative. Figure 7.A-1 is a "Preliminary Risk Assessment Form" for documenting that an encroachment has been screened for unusual risk (15). In almost all cases, where the risks are low and/or threshold design values can be met, it is unnecessary to perform a detailed assessment. Where the risks are assumed to be high and/or threshold design values cannot be met, the second stage of the process is to perform a more detailed assessment using Figure 7.A-2 "Design Risk Assessment Form," which documents threshold values that should be met by the hydraulic design (15). If the design criteria are not flexible or if significant risk exists, consider a risk analysis.

7.A.1.3 Consider Risk Analysis

The risk analysis process determines the least-total-expected-cost (LTEC) alternative using the analytical procedures provided in HEC 17 (14). This analysis provides a comparison between various alternatives developed in response to considerations such as environmental, regulatory and political constraints.

The evaluation of the consequence of risk associated with the probability of flooding attributed to a stream crossing system is a tool by which site-specific design criteria can be developed. This evaluation considers capital cost, traffic service, environmental and property impacts and hazards to human life. It is necessary to document the risk to the structure.

The following forms from Chapter 10 of the AASHTO *Model Drainage Manual* (15) should be used to assess risk.

Figure 7.A-1 — PRELIMINARY RISK ASSESSMENT FORM

Design Risk Assessment Checklist

LOCATION

County ______________________________ Sec. _______ Twp ______ Range ______________
Over (River, Cr, Dr, Ditch) ___________________________ Road No. _________________
Project No. _________________________ PCN Number ________________________________
Assessment Prepared by ___________________________ Date _____________________

1. HYDROLOGIC EVALUATION

A. Nearest Gaging Station on this stream ___________ (None ______) __________________

____

Are flood studies available on this stream? ______________________________________

Flood Data
Drainage Area ____________________ Method used to compute Q _________________
Q_{10} _______ cfs, Est. Bkwtr. ft Q_{25} ____cfs, Est. Bkwtr. _______ ft
Q_{50} _______ cfs, Est. Bkwtr. ft Q_{100} ___ cfs, Est. Bkwtr. _______ ft
Q_{500} _______ cfs, or Overtopping ______ cfs Est. Bkwtr. _________ ft

B. Does the crossing require outside Agency approval? Yes ___ No. ___

List Agencies: ____

____

2. PROPERTY-RELATED EVALUATIONS

A. Damage potential: Low _________________ Moderate ________ High ________
List buildings in floodplain _____________________ Location ___________________
Floor Elevation ____
Upstream Land Use ____
Anticipate any change? ____

B. Any flood zoning? (NFIP Studies, etc.) Yes _____ No _____

Type of Study ____
Base Flood Elevation _______________________________ (100 year)
Regulatory Floodway Width ___________________________ (As noted in NFIP studies)

Comments; ____

____

3. ENVIRONMENTAL CONSIDERATIONS

A. List commitments in Environmental Documents that affect Hydraulic Design. (None ________)

____

____

Figure 7.A-2 — DESIGN RISK ASSESSMENT FORM

Figure 7.A-2 — DESIGN RISK ASSESSMENT FORM
(Continued)

Figure 7.A-2 — DESIGN RISK ASSESSMENT FORM
(Continued)

APPENDIX 7.B
DESIGN FREQUENCY FOR TEMPORARY ON-SITE TRAFFIC DETOURS/DIVERSIONS

7.B.1 Introduction

Temporary hydraulic facilities include all channels, culverts or bridges that are required for haul roads, channel relocations, culvert installations, bridge construction, temporary roads or detours. These designs will be included in the contract plans for the project. Typically, the design flood frequency recommended for temporary hydraulic facilities is much lower than that used for permanent hydraulic facilities. This Appendix presents the procedures used in determining the frequency for temporary hydraulic facilities.

As is the case for highway stream crossings, temporary hydraulic facilities should accommodate floods larger than the “design” event to:

- avoid undue liability for damages from excessive backwater, and
- reduce the probability of losing the temporary hydraulic facility during a larger flood.

This can be achieved by:

- providing a low roadway profile that allows the temporary roadway to be overtopped without creating excessive velocities or backwaters;
- posting warnings that the road is expected to be submerged during certain rainfall events for undetermined lengths of time; and
- anchoring the temporary hydraulic facility, if needed.

7.B.2 Design Procedures

The selection of a design flood frequency for temporary hydraulic facilities involves consideration of several factors as discussed in Section 7.B.3. These selection factors are captured and weighted individually (see Figure 7.B-1) within an Impact Rating Value (IRV). The Total Impact Rating Value, which represents the sum total of all pertinent factors at a given crossing (see Figure 7.B-2), is used in Figure 7.B-3 to determine Percent Design Risk. The selection of design flood frequency for temporary hydraulic facilities (see Figure 7.B-4) is then based upon the Percent Design Risk and on the anticipated time of use in months. When the design point falls between curves, this figure can be used conservatively by sliding to the right and using the higher frequency event.

Selection Factor			Impact Rating Value (IRV)		
AADT	Urban	AADT	0 – 400	401 – 1500	> 1500
		IRV	1	2	3
	Suburban	AADT	0 – 750	751 – 1500	> 1500
		IRV	1	2	3
	Rural	AADT	0 – 1500	1501 –3000	> 3000
		IRV	1	2	3
Loss of Life (cross-checked with AADT)	Yes →		15	30	45
	No →		1	2	3
Property Damage (cross-checked with AADT)	IRV for residential, commercial, industrial areas, wastewater, storm water and water supply systems.		10	20	30
	IRV for croplands, parking and recreational areas.		5	10	15
	IRV for all others: Pasture, meadow, bare soil, etc.		1	2	3
Detour Length	Length (mile)		< 5	5 – 9	> 9
	IRV		1	2	3
Height Above Streambed	Height (ft)		< 10	10 – 20	> 20
	IRV		1	2	3
Drainage Area	Area (mile^2)		< 1	1 – 65	> 65
	IRV		1	2	3
Traffic Interruption (see instructions)			IRV for AADT multiplied by IRV for Detour Length.		

Figure 7.B-1 — RATING SELECTION

7.B.3 Selection Factors

The major factors to determine the Impact Rating Value (IRV) are:

1. Average Daily Traffic. The average number of vehicles traveling through the area in both directions in a 24-hour period, also referred to as Vehicles Per Day (VPD). Figure 7.B-1 shows that the IRV is not only dependent on the AADT but also on the location of the highway.

2. Loss of Life. If there is a potential loss of life caused by the destruction of the temporary drainage structure or by washout of the temporary roadway, the IRV due to this factor will be equal to the roadway AADT IRV multiplied by 15.

   If there is NO potential loss of life caused by the failure of the temporary drainage structure or by washout of the temporary roadway, the IRV due to this factor will be equal to the roadway AADT IRV only.

3. Property Damage. This factor accounts for property damages attributed to the destruction/loss of the temporary drainage structure or by the roadway overtopping flood to private and public structures (residential, commercial or manufacturing); appurtenances such as sewage treatment and water supply systems; and utility structures either above or below ground. The Property Damage IRV is equal to the roadway AADT IRV multiplied by 10.

   The property damage impact rating caused by the destruction of the temporary drainage structure or by the roadway overtopping flood to active cropland, parking lots and recreational areas is equal to the roadway AADT IRV multiplied by 5.

   All other areas (pasture, meadow, bare land, etc.) should have the same rating as the roadway AADT IRV.

4. Detour Length. The length in miles of an emergency detour by other roads in the event the temporary facility is not functional.

5. Height Above Streambed. The difference in elevation in feet between the traveled way and the bed of the waterway.

6. Drainage Area. The total drainage area contributing runoff to the temporary hydraulic facility, in mi^2.

7. Traffic Interruption. Includes consideration for emergency supplies and rescue, delays, alternative routes, busses, etc. Short-duration flooding of a low-volume roadway might be acceptable. If the duration of flooding is long (more than one day) and there is a quality alternative route nearby, then the flooding of a higher volume highway might also be acceptable. The Traffic Interruption IRV is determined by the Detour Length IRV multiplied by the Roadway AADT IRV.

Considering the broad breadth and width of temporary hydraulic structures and their construction surroundings, this procedure and the list of seven selection factors presented here cannot possibly capture all potential scenarios and all pertinent design considerations. The procedure addresses the standard, base set of design considerations and constraints that impact the recommended design event for a typical temporary crossing.

These seven factors should not be considered all inclusive. There may be site-specific considerations that are not captured by this procedure. An example would be an upstream flood control structure, such as a US Army Corps reservoir, that releases a known discharge on an annual or semi-annual basis. In that case, user judgment is required to adjust the procedure results or possibly replace it entirely by adopting the Corps' discharge. Similarly, the design parameters or breakpoints in Figure 7.B-1 used to generate IRVs should not be considered unchangeable. For example, some detours may be sufficiently long in the judgment of the user to justify increasing the Detour Length IRV. The increase would subsequently increase the Traffic Interruption IRV and, ultimately, size the temporary structure to a larger design event.

The user is encouraged to apply judgment to each crossing, to add selection factors as required and to integrate site-specific considerations as needed to tailor or modify the IRV weighting process. This procedure should be used cautiously and in conjunction with risk analysis that compares temporary roadway and waterway opening alternatives across a reasonable and supportable range of design discharges.

Selection Factor	**Impact Rating Value**
1. AADT	
2. Loss of Life	
3. Property Damage	
4. Detour Length	
5. Height Above Streambed	
6. Drainage Area	
7. Traffic Interruption	
TOTAL IMPACT RATING VALUE	

Figure 7.B-2 — IMPACT RATING TABLE

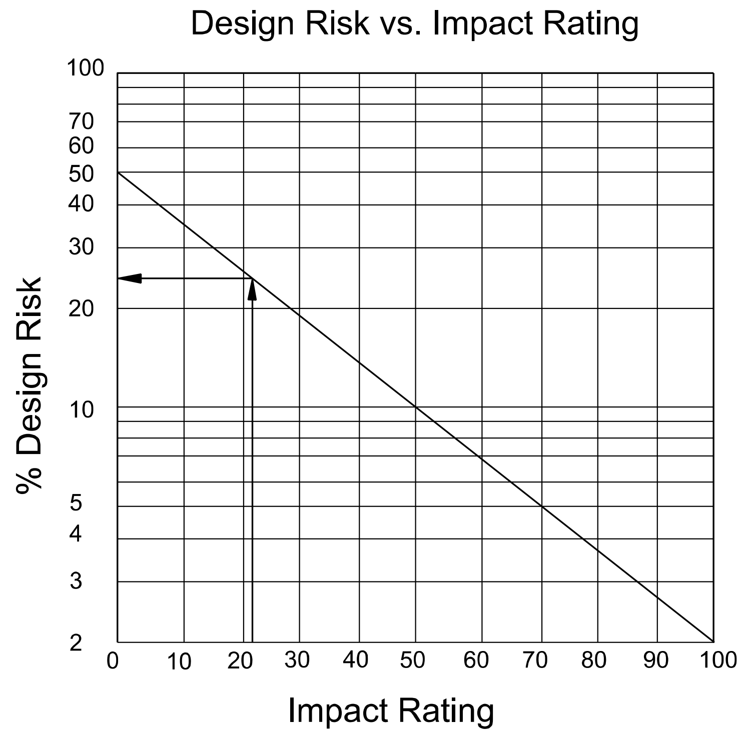

Figure 7.B-3 — DESIGN RISK VS. TOTAL IMPACT RATING VALUE

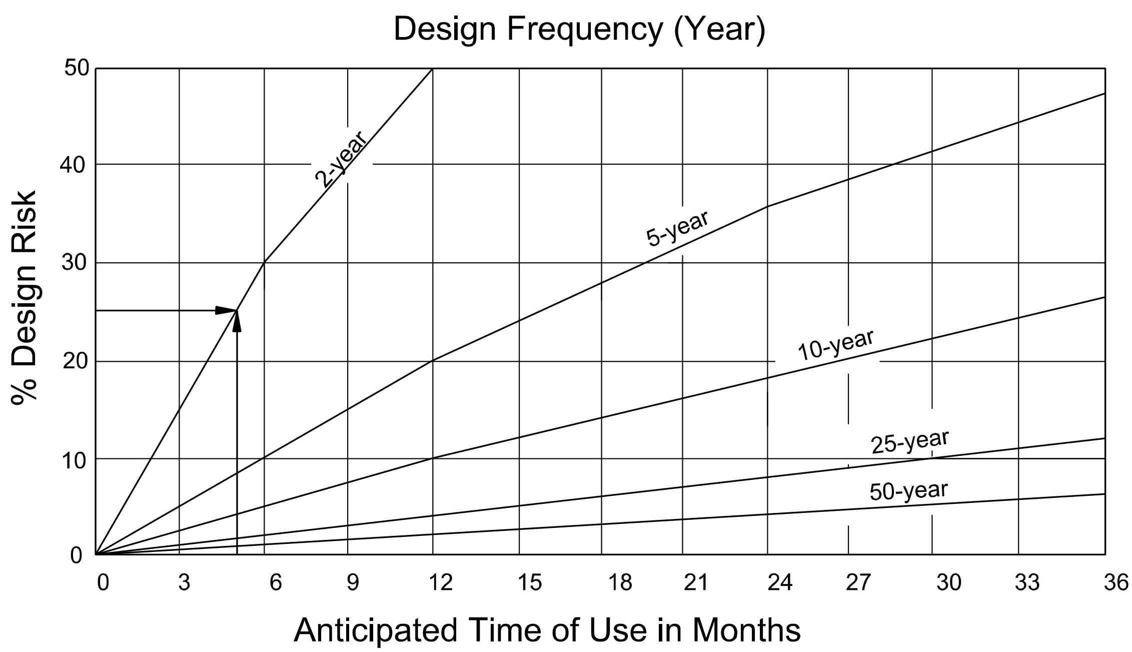

Figure 7.B-4 — RECOMMENDED DESIGN FREQUENCY

7.B.4 Example

The following example illustrates the procedure to determine the design frequency recommended for a temporary hydraulic facility.

Given: A section of a rural roadway will be widened. There is an existing 8 ft × 4 ft × 40 ft box culvert with a drainage area of 320 acres that must be replaced. A temporary structure and roadway will be provided on the downstream side of the existing roadway. The AADT of the highway is 2000 vehicles per day (VPD). The top of the temporary roadway is approximately 8 ft above the streambed. The land use on the upstream side of the proposed temporary hydraulic facility is predominantly croplands. If the temporary roadway becomes non-functional, the detour length is approximately 6 mile. The danger of loss of life due to the destruction of the temporary hydraulic facility is minimal. The anticipated use of the temporary runaround facility is five months.

Problem: Find the design frequency for the temporary hydraulic facility.

Solution:

A. Compute the Impact Rating Value (IRV) based on Figure 7.B-1:

- For a rural roadway with AADT of 2000 vpd, the IRV is 2.
- The IRV for no loss of life with this type of highway is 2.
- The Property Damage IRV is 10 (croplands).
- For Detour Length equal to 6 mi, the IRV is 2.
- For height above streambed of 8 ft, the IRV is 1.
- For Drainage Area = 320 acres, the IRV is 1.
- The IRV for traffic interruption is the product of the AADT IRV (2) times the Detour Length IRV (2). The product is 4.

B. Total Impact Rating Value (IRV)

The Total Impact Rating Value = 22, as shown in Figure 7.B-5.

C. Compute the Percent Design Risk Value:

From Figure 7.B-3, for a Total Impact Rating Value = 22, the value of the Percent Design Risk is 25%.

D. Compute the Design Frequency:

From Figure 7.B-4, for a Percent Design Risk of 25% and a construction time of five months, the recommended design frequency for the temporary hydraulic facility is a two-year return period.

Selection Factor	Impact Rating Value
1. Roadway AADT	2
2. Loss of Life	2
3. Property Damage	10
4. Detour Length	2
5. Height Above Streambed	1
6. Drainage Area	1
7. Traffic Interruption	4
TOTAL IMPACT RATING VALUE	22

Figure 7.B-5 — IMPACT RATING TABLE (Example Problem)

7.B.5 Important Notes

The goal of the guidelines and example above is to provide the hydraulics designer with information on how to come up with the recommended design frequency for the temporary drainage structure.

However, to avoid any detrimental effects that may be caused by the temporary drainage structure to up-stream's properties, it is recommended that the finish grade of the temporary road located within the vicinity of the temporary drainage structure should be lower than that of the permanent road.

No matter whether the temporary drainage structure is located on the upstream or downstream side of the permanent drainage structure, the hydraulics designer should make sure that the headwater elevation of the temporary drainage structure should be lower or equal to that of the permanent structure at:

- the design flood of the permanent drainage structure, if the site is not located in a FEMA regulated floodplain, or
- the base (100-year) flood of the permanent drainage structure, if the site is located in a FEMA regulated floodplain.

APPENDIX 7.C
ODOT WORKSHEETS FOR NRCS TR-55

ODOT Worksheet for Runoff Curve Number and Runoff

Hydraulics Designer:						
Project:			**Stream:**			
Location:						
Runoff Curve Number (CN)						
		CN				
Soil Name and Hydrologic Group	**Cover Description**	**Figures 7.7-A to 7.7-D**	**Figures 7.7-E(a)**	**Figures 7.7-E(b)**	**Area Percent**	**Product of CN x Area**
				TOTAL		
CN (weighted) = Total Product/Total Area =						
CN (use) =						

The header row also contains **Date:** in the right half.

Runoff				
Rainfall Frequency	**24-hr Rainfall Figures 7.7-L to 7.7-R**	**Ratio**	**1-day Rainfall Figures 7.7-T to 7.7-Y**	**Ratio**
2-Year				
5-year				
10-year				
25-year				
50-year				
100-year				
500-year				

Figure 7.C-1 — ODOT WORKSHEET FOR NRCS TR-55 CURVE NUMBER

ODOT Worksheet for TR-55 Time of Concentration (T_c) or Travel Time (T_t)

Hydraulics Designer:	**Date:**
Project:	**Stream:**
Location:	

Sheet Flow	Segment 1	Segment 2	Sum 1 & 2
Surface description (Figure 7.7-A to D)			
Manning's n (Figure 7.7-G)			
Flow length, L (L≤ 300 ft), ft			
Two-year 24-hr rainfall, in			
Land Slope, ft/ft			
T_t, hours (Equation 7.7(4))			

Shallow Concentrated Flow	Segment 1	Segment 2	Sum 1 & 2
Surface description			
Flow length, L, ft			
Watercourse slope, ft/ft			
Average velocity, fps (Eq. 7.7(8) or (9))			
T_t, = L/(3600V) hours			

Channel Flow	Segment 1	Segment 2	Sum 1 & 2
Cross sectional flow area, sq ft			
Wetted Perimeter, P, ft			
Hydraulic Radius, ft			
Channel Slope, ft/ft			
Manning, n			
Velocity, fps (Equation 7.7(10))			
Flow length, ft			
T_t, = L/(3600V) hours			

Total Time of Concentration (T_c) or Travel Time (T_t) =		hours
Lag time = 0.6 T_c =		hours

Figure 7.C-2 — ODOT Worksheet for TR-55 Time of Concentration (T_c) or Travel Time (T_t)