The New Mexico State Highway and Transportation Department Drainage Section is pleased to present a comprehensive update to its Drainage Manual. Volume 1 focuses on Hydrology and the prediction of flood flows at highway crossings. A companion document is presently under development which will address drainage structure hydraulics as well as sediment and erosion at highway structures. Together these documents will summarize and standardize methods by which drainage structures are designed for NMSHTD Projects. Comments regarding the content of this document are welcomed, and should be addressed to: Section Head, Drainage Section, NMSHTD, P.O. Box 1149, Santa Fe, NM 87504-1149.

Pete K. Rahn, Secretary
New Mexico State Highway and Transportation Department
# Table of Contents

1. **Introduction** ........................................... 1–1
   1.1 *Drainage Manual Purpose and Use* .................. 1–1
   1.2 *Drainage Design Criteria Guidelines* ............. 1–1
   1.3 *Use of Metric Standards in the Design of NMSHTD Projects* .... 1–2

2. **Basic Requirements for Drainage Studies** ........... 2–1
   2.1 *Drainage Field Inspection* ......................... 2–1
   2.2 *Preliminary Drainage Report* ....................... 2–5
   2.3 *Final Drainage Report* ............................. 2–5
   2.4 *Temporary Erosion and Sediment Control Plan* .... 2–5

3. **Hydrology** ............................................. 3–1
   3.1 *NMSHTD Approach to Hydrologic Analysis* ......... 3–1
   3.2 *Selection of a Hydrologic Method* .................. 3–1
   3.3 *Drainage Basins Without Gage Data* ............... 3–4
      3.3.1 *General Data for Hydrologic Analysis* ....... 3–4
         3.3.1.1 *Drainage Basin Delineation* ............... 3–5
         3.3.1.2 *Rainfall* .................................. 3–7
            3.3.1.2.1 *Rainfall in the Rational Formula* .... 3–7
            3.3.1.2.2 *Rainfall in the Simplified Peak Flow Method* .... 3–16
         3.3.1.2.3 *Rainfall in the SCS Unit Hydrograph Method* ........ 3–16
         3.3.1.3 *Rainfall Losses and Runoff Curve Numbers* .... 3–19
            3.3.1.3.1 *Curve Number Selection* ............ 3–19
            3.3.1.3.2 *Curve Number Weighting* ............. 3–29
         3.3.1.4 *Time of Concentration* .................... 3–30
            3.3.1.4.1 *The Upland Method* ............... 3–32
            3.3.1.4.2 *Time of Concentration by the Kirpich Formula* .... 3–34
            3.3.1.4.3 *The Stream Hydraulic Method* ....... 3–34
### TABLE OF CONTENTS (CONTINUED)

<table>
<thead>
<tr>
<th>Section</th>
<th>Page Number</th>
</tr>
</thead>
<tbody>
<tr>
<td>3.3.2 <strong>RATIONAL FORMULA METHOD</strong></td>
<td>3–36</td>
</tr>
<tr>
<td>3.3.2.1 <strong>APPLICATION OF THE RATIONAL FORMULA</strong></td>
<td>3–37</td>
</tr>
<tr>
<td>3.3.2.1.1 <strong>RATIONAL FORMULA METHOD EXAMPLE PROBLEMS</strong></td>
<td>3–44</td>
</tr>
<tr>
<td>3.3.3 <strong>SIMPLIFIED PEAK FLOW METHOD</strong></td>
<td>3–49</td>
</tr>
<tr>
<td>3.3.3.1 <strong>APPLICATION</strong></td>
<td>3–49</td>
</tr>
<tr>
<td>3.3.3.2 <strong>SIMPLIFIED PEAK FLOW METHOD EXAMPLE PROBLEMS</strong></td>
<td>3–55</td>
</tr>
<tr>
<td>3.3.4 <strong>USGS REGRESSION EQUATIONS FOR NEW MEXICO</strong></td>
<td>3–62</td>
</tr>
<tr>
<td>3.3.4.1 <strong>RURAL PEAK DISCHARGE METHOD</strong></td>
<td>3–64</td>
</tr>
<tr>
<td>3.3.4.2 <strong>URBAN USE OF USGS REGRESSION EQUATIONS</strong></td>
<td>3–72</td>
</tr>
<tr>
<td>3.3.4.3 <strong>BASIN DEVELOPMENT FACTOR</strong></td>
<td>3–72</td>
</tr>
<tr>
<td>3.3.4.4 <strong>THREE PARAMETER ESTIMATING EQUATIONS</strong></td>
<td>3–74</td>
</tr>
<tr>
<td>3.3.4.4.1 <strong>USGS REGRESSION EQUATION EXAMPLE PROBLEMS</strong></td>
<td>3–76</td>
</tr>
<tr>
<td>3.3.5 <strong>SCS UNIT HYDROGRAPH METHOD</strong></td>
<td>3–79</td>
</tr>
<tr>
<td>3.3.5.1 <strong>UNIT HYDROGRAPH PREPARATION</strong></td>
<td>3–79</td>
</tr>
<tr>
<td>3.3.5.2 <strong>APPLICATION OF THE SCS UNIT HYDROGRAPH METHOD</strong></td>
<td>3–85</td>
</tr>
<tr>
<td>3.4 <strong>WATERSHEDS WITH STREAM GAGE DATA</strong></td>
<td>3–86</td>
</tr>
<tr>
<td>3.5 <strong>RISK AND UNCERTAINTY IN HYDROLOGIC ANALYSIS</strong></td>
<td>3–87</td>
</tr>
</tbody>
</table>
TABLE OF CONTENTS (CONTINUED)

FIGURES

<table>
<thead>
<tr>
<th>Figure</th>
<th>Description</th>
<th>Page Number</th>
</tr>
</thead>
<tbody>
<tr>
<td>1-1</td>
<td>NMSHTD Facilities Map</td>
<td>1–3</td>
</tr>
<tr>
<td>2-1</td>
<td>Drainage Structure Field Inspection Form</td>
<td>2–4</td>
</tr>
<tr>
<td>3-1</td>
<td>Methodology Selection Flow Chart, Rural Conditions</td>
<td>3–2</td>
</tr>
<tr>
<td>3-2</td>
<td>Methodology Selection Flow Chart, Urban Conditions</td>
<td>3–3</td>
</tr>
<tr>
<td>3-3</td>
<td>Effect of Drainage Basin Shape on Hydrograph Shape</td>
<td>3–6</td>
</tr>
<tr>
<td>3-4</td>
<td>DDF/IDF Worksheet</td>
<td>3–14</td>
</tr>
<tr>
<td>3-5</td>
<td>Precipitation Intensity Duration Frequency (IDF) Graph</td>
<td>3–15</td>
</tr>
<tr>
<td>3-6</td>
<td>The Modified NOAA–SCS Rainfall Distribution Worksheet</td>
<td>3–18</td>
</tr>
<tr>
<td>3-7</td>
<td>Estimating Ground Cover Density</td>
<td>3–21</td>
</tr>
<tr>
<td>3-8</td>
<td>Hydrologic Soil – Cover Complexes and Associated Curve Numbers</td>
<td>3–22</td>
</tr>
<tr>
<td>3-9</td>
<td>Composite CN for Urban Areas with Connected and Unconnected Impervious Areas</td>
<td>3–28</td>
</tr>
<tr>
<td>3-10</td>
<td>Flow Velocities for Overland and Shallow Concentrated Flows</td>
<td>3–33</td>
</tr>
<tr>
<td>3-11</td>
<td>Rational “C” Coefficient Upland Rangeland (Grass &amp; Brush)</td>
<td>3–38</td>
</tr>
<tr>
<td>3-12</td>
<td>Rational “C” Coefficient Desert (Cactus, Grass &amp; Brush)</td>
<td>3–39</td>
</tr>
<tr>
<td>3-13</td>
<td>Rational “C” Coefficient Mountain (Juniper &amp; Grass)</td>
<td>3–40</td>
</tr>
<tr>
<td>3-14</td>
<td>Rational “C” Coefficient Mountain (Grass &amp; Brush)</td>
<td>3–41</td>
</tr>
<tr>
<td>3-15</td>
<td>Rational “C” Coefficient Mountain (Ponderosa Pine)</td>
<td>3–42</td>
</tr>
<tr>
<td>3-16</td>
<td>Rational “C” Coefficient Developed Watersheds</td>
<td>3–43</td>
</tr>
<tr>
<td>3-17</td>
<td>Estimating Direct Runoff</td>
<td>3–51</td>
</tr>
<tr>
<td>3-18</td>
<td>Unit Peak Discharge for the Simplified Peak Flow Method</td>
<td>3–52</td>
</tr>
<tr>
<td>3-19</td>
<td>Simplified Peak Flow Worksheet</td>
<td>3–54</td>
</tr>
<tr>
<td>3-20</td>
<td>USGS Physiographic Regions</td>
<td>3–63</td>
</tr>
<tr>
<td>3-21</td>
<td>Dividing Urban Drainage Basins Into Thirds</td>
<td>3–73</td>
</tr>
<tr>
<td>3-22</td>
<td>Dimensionless Unit Hydrograph and Mass Curve for SCS Synthetic Hydrograph</td>
<td>3–82</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>3-23</td>
<td>Dimensionless Curvilinear Unit Hydrograph and Equivalent Triangular Hydrograph</td>
<td>3–83</td>
</tr>
<tr>
<td>3-24</td>
<td>Regression Line and Average Standard Error of Prediction Range</td>
<td>3–91</td>
</tr>
</tbody>
</table>
TABLE OF CONTENTS (CONTINUED)

Tables

Table 3-1 — Runoff Curve Numbers for Arid and Semiarid Rangelands 3-23
Table 3-2 — Runoff Curve Numbers for Cultivated Agricultural Lands 3-24
Table 3-3 — Runoff Curve Numbers for Other Agricultural Lands 3-25
Table 3-4 — Runoff Curve Numbers Urban Areas 3-26
Table 3-5 — Conversion from Average Antecedent Moisture Conditions to Dry and Wet Conditions 3-27
Table 3-6 — Time of Concentration Method Selection Chart 3-31
Table 3-7 — USGS Rural Flood Frequency Equations for New Mexico 3-65
Table 3-8 — USGS Small Rural Basin Flood Frequency Equations for New Mexico 3-71
Table 3-9 — USGS Urban Peak Discharge Three Parameter Estimating Equations 3-75
Table 3-10 — Ratios for Dimensionless Unit Hydrograph and Mass Curve, SCS Synthetic Hydrograph 3-84
Table 3-11 — Tabulation of Risk of at Least One Exceedance During the Design Life 3-92

Appendices

Appendix A — Photographic Guide to Hydrologic Conditions A-1
Appendix B — Glossary of Terms B-1
Appendix C — Metric Equations and Conversion Factors C-1
Appendix D — Text References and Other Selected References D-1
Appendix E — NOAA Isopluvial Maps for New Mexico E-1
1 \textbf{INTRODUCTION}

1.1 \textit{DRAINAGE MANUAL PURPOSE AND USE}

The New Mexico State Highway and Transportation Department (NMSHTD) is responsible for the maintenance and construction of a vast network of roads throughout the State of New Mexico. Public safety and prudent investment of public funds in our road network requires that each facility be reasonably protected from a damaging flood. Standard methods of analysis and design have evolved over the past fifty years. Certain methods commonly used by the NMSHTD Drainage Section have proven their validity for use in New Mexico. This Manual summarizes those common methods which have a proven record for use in this state.

The standard methods of Hydrologic analysis presented in this Drainage Manual should be used for all NMSHTD projects. Use of these standard methods will ensure consistency of analysis and design methods to the greatest extent possible. A brief description of each analysis method is included in this Drainage Manual, followed by a step by step procedure to apply the method. Example problems are included to assist the drainage designer. Limitations on the use of each analysis method are also included. This Drainage Manual does not include descriptions of the development or derivation of analysis methods. References are provided for the reader who wishes to review the source documents for each method.

This Drainage Manual specifies which hydrologic analysis method may be used for a particular drainage structure, based on drainage area size and location. By limiting the choice of hydrologic analysis method, a consistent and appropriate level of analysis is assured for every drainage structure, large and small. Despite these efforts to standardize methods, proper drainage analysis and design is not complete without the inclusion of competent engineering judgement. Drainage designers working on NMSHTD projects are expected to apply engineering judgement throughout the design development process. "Does this make sense? Will it work? What are the consequences of a failure? What is the risk associated with keeping the present structure?" These are the kinds of questions which complete the drainage design process once the analytic methods described in this Manual have been performed.

1.2 \textit{DRAINAGE DESIGN CRITERIA GUIDELINES}

Drainage structures within the NMSHTD facilities network must be designed to meet certain minimum standards. Design frequency flood events are selected for each element of the highway drainage system. The magnitude of the design event is consistent with the highway classification, average daily traffic, user safety, risk, and consideration of economic impacts. Each drainage structure is designed to safely pass the appropriate design frequency flood without compromising the entire traveled way. The "appropriate" flood magnitude is a matter of public policy, balancing limited economic resources with the need to provide benefits to the greatest number of facility users. The NMSHTD Policy on Drainage Design Criteria may be found in a separate document of the same title. As a separate document from
this Drainage Manual, it may periodically be revised to accommodate changes in public policy. Users of the NMSHTD Drainage Manual should obtain a current copy of the NMSHTD Policy on Drainage Design Criteria, so that drainage structures are designed for the appropriate flood magnitudes.

1.3 USE OF METRIC STANDARDS IN THE DESIGN OF NMSHTD PROJECTS

The NMSHTD endorses the use of metric or International System of Units (SI) for the analysis and design of NMSHTD projects. All of the drainage design procedures identified in this Drainage Manual were developed in the context of US Standard units of measurement. For this reason the discussion of different methods is provided using US Standard units where required. However, all hydrologic equations are provided in both SI and US Standard formats.

The Drainage Section of the NMSHTD has developed this edition of their Drainage Manual in recognition of the transition period which will occur in the near future. Designers will be increasingly required to perform hydrologic and hydraulic calculations in SI units. Under current guidelines, all projects must be analyzed and designed in SI units by the end of September, 1996. Many designers have developed personal rules of thumb and error checking procedures which are in US Standard units (CFS per acre, ft. per second, etc.). These important design procedures must be carried on in SI units, not abandoned. By providing a Drainage Manual which promotes the use of SI without discarding US Standard, the NMSHTD Drainage Section hopes to promote an orderly transition to SI.

In this transition period, all drainage engineers and designers working on NMSHTD projects are strongly encouraged to use SI units whenever possible in their analyses. Additional SI design aides will be disseminated by the Drainage Section as they become available. We welcome your suggestions for promoting a smooth transition to SI based design. Please send your written comments to: Chief, Drainage Section, NMSHTD, P. O. Box 1149, Santa Fe, NM 87504–1149.
Figure 1-1
NMSHTD Facilities Map
2 **BASIC REQUIREMENTS FOR DRAINAGE STUDIES**

Drainage studies for NMSHTD projects must identify the hydrologic demands and hydraulic requirements of each drainage structure within the project limits. Each study will result in one or more drainage reports, summarizing the drainage improvements associated with the project. The drainage engineer's responsibility usually does not end with the drainage report. Staff engineers within the NMSHTD Drainage Section who prepare drainage reports will usually be responsible for drainage related permits (EPA, COE, FEMA), for development of a Sediment and Erosion Control Plan, and ongoing coordination with other NMSHTD Sections. Similar responsibilities may be required of consultants under contract with the NMSHTD. No matter what the total scope of services include, a drainage study and associated report(s) will be required. This section of the NMSHTD Drainage Manual describes the basic requirements of a drainage study for a NMSHTD project.

Most NMSHTD Projects include a standard set of project development milestones. These standard milestones are shown below. Drainage study elements are shown in bold text, identifying their location in the project development schedule. Specific requirements for these drainage study elements are described in the following sections.

**Typical Project Schedule**

- Preliminary Scoping Report
- Preliminary Field Review
  - **Drainage Field Inspection**
  - **Preliminary Drainage Report**
- Field Design Inspection
  - **Final Drainage Report**
- Grade and Drain Inspection
  - **Temporary Erosion and Sediment Control Plans**
- Plan in Hand
- Plans, Specifications & Estimate

*The drainage field inspection is sometimes combined with the Preliminary Field Review.*

2.1 **DRAINAGE FIELD INSPECTION**

Field inspection of the project from a drainage perspective is a critical element of the drainage study process. A thorough inspection will often reveal design considerations which cannot be deduced from the topographic mapping. *The drainage field inspection should be performed in the preliminary drainage report phase of the project, after basic data collection and after the preliminary hydrologic analysis has been performed.* In this sequence, the field inspection can be used to verify design assumptions, locate existing structures and sizes, and evaluate the potential impacts of proposed drainage improvements. This is an opportunity for the drainage designer to field verify his or her preliminary design.
The basic elements of the drainage field inspection are listed below, with suggestions on things to look for and quantify in the field. The designer will probably develop a list of questions during the preliminary hydrologic analysis which need field verification.

**Figure 2-1** is a field inspection form for drainage structures. This form should be copied and completed in the field for all existing drainage structures. Be sure to allow adequate time for the drainage field inspection, particularly if field surveys of structure inlet – outlet conveyances are planned.

**Field Inspection Suggestions**

**Watershed Conditions**
- verify assumptions used in hydrologic analysis, including:
  - soil types, Hydrologic Soil Group (HSG)
  - land usage
  - vegetation and ground cover density
  - percent impervious
- evidence of flow diversions, stock ponds, etc. not accounted for in analysis

**Existing Structures**
- measure actual structure sizes, wall thickness, etc.
- identify actual locations: use mileposts, stations from as–built plans, distance meters, etc.
- structural condition: look for rust, spalling, cracks, deformed cross section
- structure subsidence: is the vertical alignment okay?
- evidence of outlet erosion and/or inlet sedimentation
- upstream high water marks: (when estimating the magnitude of flow events, an approximate discharge can be calculated using the Slope – Area method)
- evidence of debris accumulation
- channel geometry upstream and downstream
- effectiveness of structure skew, inlet/outlet geometry
- does the existing structure appear capable of passing the design flow?
  - if not, what will happen? roadway overtopping? backwater onto adjacent properties?

**Onsite Drainage Facilities** (within the Right–of–Way) – evaluate how they are functioning
- roadside ditches: vegetation, ditch erosion, cut slopes erosion
- median swales working
- rundowns still working properly
- area inlets and catch basins working
- curbs, gutters diverting flows to desired locations
- is the pavement section being drained adequately?
- erosion of an embankment by pavement runoff
Interview NMSHTD Patrol Foreman

- identify inadequate drainage facility locations
- describe location and magnitude of major flow events
- discuss maintenance procedures including
  - standard practices
  - specific problem spots
  - frequency and timing of maintenance work
- list improvements suggested by Patrol Foreman

Interview Other Individuals as Required – State Police, local property owners, etc.

- be sure to get names, and date of interview

Evaluate Proposed Drainage Improvements

- does the proposed structure seem reasonable?
- does the upstream conveyance reflect the design flow?
- will a backwater condition adversely impact adjoining landowners?
- can the inlet condition be improved with trainer dikes?
- consider the proposed road section and profile for impacts to
  - structure extensions and resulting inlet/outlet locations
  - special designs for high fills or minimal cover conditions
- how will future maintenance operations be affected?
- would a different type of structure improve passage of sediment or debris?
- are debris control measures required?
- are additional drainage improvements needed?
- effectiveness of proposed skew angle

Evaluate Effectiveness of Maintenance Work

- is the pavement surface able to drain effectively?
- does water pond next to the pavement?
- are structure inlets obstructed with debris?
- do grading operations increase ditch or shoulder erosion?

Individual designers will undoubtedly come up with other questions to be answered in the drainage field inspection. However, these suggestions provide a basic list of items which should be evaluated in the field on each NMSHTD project.
## Verify Watershed Conditions

- **Land Use**
- **Vegetation Type**
- **Verify – Effective Drainage Area**
- **Stock Ponds or Detention Facilities**
- **Other Comments**

## Structure Type

- **Size or Span**
- **Clear Height**
- **Structure Skew**
- **Evidence of, Bridge Scour**
- **General Condition of Structure**
- **Erosion**
- **Spalling**
- **Cracking**
- **Barrel Deformation**
- **Other Comments**

## Structure Inlet Conditions

- **Wingwalls**
- **Headwalls**
- **Training Dikes**
- **Upstream Channel**
- **Evidence of, Debris**
- **Evidence of, Ponding**
- **Channel Bed Material**

## Structure Outlet Conditions

- **Wingwalls**
- **Headwalls**
- **Training Dikes**
- **Outlet Apron**
- **Evidence of, Erosion at Outlet**

## General Conditions

- **Calculated Peak Design Flow**
- **Is This Reasonable?**
- **Evidence of Flood Damage to Adjacent Properties**
- **Evidence of Stream Instability Effecting Adjacent Properties**
- **Irrigation Facilities Affected**
- **Environmental Hazards Present**
- **Photos Taken of:**
- **Survey Required:**
- **Items to Research Back at the Office:**
- **Other Comments:**

---

**Project Location:**

**CN#:**

**Date:**

**Inspected by:**

**Structure Location:**

**Project Station:**

---

**Figure 2-1**

Drainage Structure Field Inspection Form
2.2 **Preliminary Drainage Report**

The preliminary drainage report should **summarize** the results of the preliminary drainage analysis. Structure Size recommendations will be reviewed by the NMSHTD Drainage Section, and will be used for field design plans by the Highway Design Section. Basic elements which should be included in the preliminary drainage report are listed below.

- Project Name, location, Project Control Number, etc.
- Drainage area topographic map with structure locations identified
- Identify soil types, vegetation and land use distribution
- Curve Number or Rational Formula “C” calculations
- Time of Concentration calculations
- Summarize the drainage field inspection results, including patrol foreman interview
- Drainage Structure Field Inspection forms
- Summary Table of existing and recommended drainage structure sizes and types
- Identify data sources used in the analysis

The preliminary drainage report should **not** include detailed print outs from hydrologic or hydraulic analyses. However, data generated in the analysis process should be kept on file and made available to the NMSHTD Drainage Section when requested.

2.3 **Final Drainage Report**

The Final Drainage Report is basically a refinement of the Preliminary Drainage report. The Final Drainage Report is not begun until receipt of the preliminary design from the Highway Design Section. The preliminary highway design data must include: preliminary plan and profile sheets, with preliminary grade, typical roadway sections, toe of slope lines, and drainage structure survey data. Modifications to the preliminary hydrologic analysis are completed as required, and final structure sizes are established. A detailed hydraulic analysis (backwater profiles, flow velocities, etc.) is required for bridge structures and for some large culvert locations. Permanent erosion protection design is completed, including riprap design, drainage structure outlet design and analysis of scour depths at critical locations. For watersheds producing high sediment loads, an estimate of upstream sediment transport and sediment continuity at the highway crossing structure may be required.

2.4 **Temporary Erosion and Sediment Control Plan**

Design of temporary erosion and sediment control measures is not included in the preliminary or final drainage report. The drainage designer should refer to the document “National Pollutant Discharge Elimination System Implementation Package,” prepared by the NMSHTD. Contact the NMSHTD Drainage Section in Santa Fe for further information.
3 HYDROLOGY

3.1 NMSHTD APPROACH TO HYDROLOGIC ANALYSIS

The New Mexico State Highway and Transportation Department must provide transportation facilities which are reasonably safe for the public. A safe roadway environment includes properly designed drainage structures. The NMSHTD must design drainage structures to meet minimum design standards, and must do so within certain budgetary constraints. Current minimum design standards for drainage facilities can be found in the document “Drainage Design Criteria for NMSHTD Projects.” This document is available from the NMSHTD Drainage Section, in Santa Fe.

The NMSHTD also recognizes that the effort associated with the design and analysis of drainage structures must be commensurate with the importance of the transportation facility. Small culverts on low volume roads in remote areas normally do not require an exhaustive analysis. For this reason, the NMSHTD has established a hierarchy of drainage analysis methods to ensure that appropriate design methods are used.

It is the goal of the NMSHTD Drainage Section to standardize the hydrologic analysis methods used on NMSHTD projects, requiring the use of standard methods which have a demonstrated performance record in this state. Many hydrologic analysis methods have been used in New Mexico with widely varying results. Some of these methods do not work well in this state, or perhaps are valid only for a particular region of New Mexico. Furthermore, within each hydrologic analysis method there is some range of judgement or interpretation. By standardizing hydrologic analysis methods, a significant amount of confusion and debate will be removed from drainage analyses performed on NMSHTD projects. Guidelines for the use of NMSHTD approved hydrologic analysis methods are provided in this manual, along with visual aides to promote consistency in the selection of curve numbers.

3.2 SELECTION OF A HYDROLOGIC METHOD

The NMSHTD Drainage Section has established certain hydrologic analysis methods to be used on NMSHTD projects. Methods are selected based on drainage area size, and whether or not the highway facility is located in an Urban or Rural area. In general, NMSHTD personnel and consultants to the NMSHTD are required to use the hydrologic methods specified below. The NMSHTD Drainage Section may allow other hydrologic analysis methods to be used, depending on project specific circumstances. Contact the Drainage Section and obtain approval before using a method other than those specified below.

Figures 3-1 and 3-2 are used to select the appropriate hydrologic method for a particular drainage structure. When two or three methods are applicable, the order of preference is shown by a small symbol, O. In areas where a local government agency has a drainage policy which mandates a specific hydrologic analysis method, that hydrologic analysis method shall be used on NMSHTD projects. For example, the AHYMO model using the COMPUTE NMHYD routine is approved for use in Albuquerque, but not in Roswell. When a particular drainage basin is borderline between two size categories, the more detailed analysis method shall be used. At the discretion of the designer, the Unit Hydrograph Method can be substituted for the Simplified Peak Flow method.
RURAL CONDITIONS

Drainage Area
less than
5 sq.mi.

Pavement
Drainage
NPDES Sites

Rational
Method

USGS Statewide ©
Small Basin
Regression
Equations

Drainage Area
greater than
5 sq.mi.

Offsite
Watersheds

Simplified ①
Peak Flow

USGS ①
Regional
Regression
Equations **

Ungaged
Stream

Gaged* Stream

USGS Gage Data  **

* Only gage data from USGS gages will be allowed for use on NMSHTD Projects.
** The NMSHTD may require designers to provide a supplementary Unit Hydrograph calculation for comparison purposes.

Figure 3-1
Methodology Selection
Flow Chart
Rural Conditions
Figure 3-2
Methodology Selection
Flow Chart
Urban Conditions
3.3 **DRAINAGE BASINS WITHOUT GAGE DATA**

The vast majority of drainage structures on New Mexico highways pass flows from watersheds for which there is no measured data on rainfall or runoff. Peak rates of runoff and runoff volumes must therefore be estimated using analytical or parametric (regression) methods. Designers using this manual need not be proficient in statistical analysis procedures. The regression methods specified herein have been developed by the United States Geological Survey (USGS) and can be quickly applied. The analytic methods adopted for use by the NMSHTD are commonly accepted methods which have been used successfully in New Mexico. The Rational Formula is used for very small watersheds. SCS methods including the Unit Hydrograph procedure and the Peak Rate of Discharge for Small Watersheds are used for larger watersheds. In urban areas where established drainage policy dictates a particular hydrologic analysis method, analysis of drainage structures within that jurisdiction will follow the local established method.

Use of specific rain gage data will generally not be allowed on NMSHTD projects. Instead, rainfall data from the National Oceanic and Atmospheric Administration (NOAA) Precipitation – Frequency Atlas (Miller et al, 1973) will be used*. The purpose of this exclusion is to promote the use of regionally adjusted rainfall data, in lieu of reliance on data from a single location. Regional regression analysis techniques were used by NOAA to smooth the delineation of equal precipitation areas, removing some of the uncertainty associated with a single gage location. Use of regionalized rainfall data is particularly important in New Mexico where rainfall can vary dramatically from one location to another nearby location.

### 3.3.1 GENERAL DATA FOR HYDROLOGIC ANALYSIS

Certain characteristics of each drainage basin must be quantified to estimate peak rates of runoff and runoff volumes. Size of a drainage basin is always important. The quantity of rainfall is also important. The time distribution and intensity of rainfall has a direct effect on the rate of runoff. Rainfall lost to ground infiltration, localized depression ponding or plant absorption means less water available for runoff. The slope of the watershed and development of stream channels affects how fast runoff can reach the drainage structure. The following sections of this manual describe these factors in greater detail, and how to quantify them for use in each hydrologic analysis method.

---

*The NOAA Rainfall Atlas is currently being revised (1995). Updated NOAA rainfall data will be used on NMSHTD projects once the revised Atlas is publicly available.*
3.3.1.1 Drainage Basin Delineation

Drainage basins are usually defined graphically using topographic maps. USGS topographic maps at 1:24,000 scale provide adequate detail for NMSHTD projects and are available for most areas of New Mexico. Drainage structures crossing highways are usually located at low spots in the terrain, and are always provided where a stream channel exists. From the drainage structure location, drainage basin boundaries are drawn on the topographic map proceeding uphill such that the boundary encompasses all land which can drain to the crossing structure location. A simple test is to imagine a drop of rain falling on the ground, and to follow the path it takes as it runs downhill. Drainage basin boundary lines are generally drawn perpendicular to the topographic lines, following the ridgetops.

Once the overall drainage basin has been defined, the total drainage area should be measured. A planimeter is commonly used to measure areas from topographic maps. Drainage basin areas may also be measured electronically by digitizing map areas. Some USGS maps are now available in digital format. The historical grid method may also be used, where the basin map is overlaid with a transparent grid and grid rectangles are counted within the basin boundary lines.

Each drainage basin should be qualitatively assessed as follows:

- What hydrologic analysis method is required based on drainage basin size?

- Is one drainage basin okay for analysis purposes, or should we create sub-basins? Considerations might include: drastic changes in land slope, land use and development.

- Is the overall drainage basin shape somewhat consistent with implicit assumptions built into the analytical design methods? Figure 3-3 shows the effects on hydrograph shape from different drainage basin shapes. The designer should consider subdividing drainage basins which are particularly elongated or short and wide.

- Will roads, diversions, ponds or other features within the drainage basin prevent it from behaving as a uniform, homogeneous watershed?

- In flat terrain, are there roads or other development features which act as drainage divides?

When these factors are accounted for, parameters such as Time of Concentration and Runoff Curve Number will more accurately portray the runoff response of the watershed.
Figure 3-3
Effect of Drainage Basin Shape on Hydrograph Shape

Adapted from SCS, NEH-4, 1972
3.3.1.2 RAINFALL

Rainfall data is a necessary input parameter for nearly all runoff computations performed on NMSHTD projects. The quantity of rainfall and the time distribution of the rainfall will both affect the resulting peak rate of runoff. Rainfall data is taken from the NOAA Precipitation – Frequency Atlas (Miller et al, 1973) or from updated NOAA maps when they become available. Figures E-1 through E-12 in APPENDIX E of this manual provide the same NOAA data (1973) with a current (1995) State Highway map. Point precipitation values may be read from these Figures for the design rainfall event.

The designer must first determine the return frequency of the design flood to be used on a particular project or drainage structure. Design frequency floods are listed in a separate document, "Drainage Design Criteria for NMSHTD Projects," which may be obtained from the NMSHTD Drainage Section. Design frequencies are not included in this manual because the design criteria may change over time. Designers should verify that they have the latest Drainage Design Criteria before proceeding with design on NMSHTD projects.

For NMSHTD projects the assumption is made that rainfall frequencies produce equivalent flood frequencies, i.e. the 50–year rainfall event will produce the 50–year runoff event. This assumption is generally valid when all other factors remain constant (antecedent moisture, etc.), particularly for ephemeral stream systems. There are some situations where this assumption may not be correct. In regions of New Mexico where the seasonal snowpack is significant, the designer should evaluate both a rainfall event and a snowmelt/rainfall event as predicted by the USGS rural peak discharge regression equations.

3.3.1.2.1 RAINFALL IN THE RATIONAL FORMULA

Rainfall data must be transformed into an Intensity–Duration–Frequency (IDF) relationship for use in the Rational Formula. Rainfall intensity, i, has units of inches/hour, and changes with the Time of Concentration and design frequency. Specific IDF curves must be prepared for each NMSHTD project location. Generalized IDF curves should not be used. A manual procedure for preparing IDF curves is described below. A computer spreadsheet is used by the NMSHTD Drainage Section to expedite these calculations.
Manual IDF Procedure:

Step 1

Obtain the 6-hour and 24-hour point precipitation depths from Figures E–1 through E–12, or from the current NOAA Atlas. 2-year and 100-year depths are required, along with other return periods needed for the drainage analysis. Enter the values in the Depth–Duration–Frequency (DDF) Worksheet (Figure 3–4). Designers should make blank copies of the DDF/IDF Worksheet and the IDF Graph for use on different projects.

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Note: Cross hatching denotes which values are being entered in the DDF matrix of Figure 3–4.
Step 2

Locate the project in NOAA Region 1 or Region 2, from Figure E-13.

Compute the 1-hour depths for the 2- and 100-year return periods.

In Region 1,

\[ P^{\text{2-yr, 1-hr}} = 0.218 + 0.709 \frac{P^{2}}{P^{2-yr, 24-hr}} \]  
\[ P^{\text{100-yr, 1-hr}} = 1.897 + 0.439 \frac{P^{2}}{P^{100-yr, 24-hr}} - 0.00008 \quad Z \]

where \( Z \) is the average project elevation, in feet.

In Region 2,

\[ P^{\text{2-yr, 1-hr}} = -0.011 + 0.942 \frac{P^{2}}{P^{2-yr, 24-hr}} \]
\[ P^{\text{100-yr, 1-hr}} = 0.494 + 0.755 \frac{P^{2}}{P^{100-yr, 24-hr}} \]

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Step 3

Complete the row of 1-hour depths from the 2- and 100-year end points.

\[
P_{5\text{-yr}, 1\text{-hr}} = 0.770\ P_{2\text{-yr}, 1\text{-hr}} + 0.230\ P_{100\text{-yr}, 1\text{-hr}} \quad (3\text{-5})
\]

\[
P_{10\text{-yr}, 1\text{-hr}} = 0.609\ P_{2\text{-yr}, 1\text{-hr}} + 0.391\ P_{100\text{-yr}, 1\text{-hr}} \quad (3\text{-6})
\]

\[
P_{25\text{-yr}, 1\text{-hr}} = 0.425\ P_{2\text{-yr}, 1\text{-hr}} + 0.575\ P_{100\text{-yr}, 1\text{-hr}} \quad (3\text{-7})
\]

\[
P_{50\text{-yr}, 1\text{-hr}} = 0.241\ P_{2\text{-yr}, 1\text{-hr}} + 0.759\ P_{100\text{-yr}, 1\text{-hr}} \quad (3\text{-8})
\]
Step 4

Compute the 2- and 3-hour depths from the 1- and 6-hour depths for each return period.

For both regions,

\[ P_{2 \text{- hr}} = 0.658 \ P_{1 \text{- hr}} + 0.342 \ P_{6 \text{- hr}} \]  \hspace{1cm} (3-9)

In Region 1,

\[ P_{3 \text{- hr}} = 0.401 \ P_{1 \text{- hr}} + 0.599 \ P_{6 \text{- hr}} \]  \hspace{1cm} (3-10)

In Region 2,

\[ P_{3 \text{- hr}} = 0.428 \ P_{1 \text{- hr}} + 0.572 \ P_{6 \text{- hr}} \]  \hspace{1cm} (3-11)

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Step 5

Complete the 12-hour depth from the 6- and 24-hour depths for each return period.

\[ P_{12-hr} = 0.50 \ P_{6-hr} + 0.50 \ P_{24-hr} \]  

(3–12)

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Step 6

Compute the 5-, 10-, 15- and 30-minute rows from the 1-hour depth for each return period.

\[ P_{5-min} = 0.29 \ P_{1-hr} \]  

(3–13)

\[ P_{10-min} = 0.45 \ P_{1-hr} \]  

(3–14)

\[ P_{15-min} = 0.57 \ P_{1-hr} \]  

(3–15)

\[ P_{30-min} = 0.79 \ P_{1-hr} \]  

(3–16)

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Step 7

Divide the values in the DDF worksheet by the time increment in the left hand column. Enter the resulting value in the IDF worksheet (Figure 3-4) in the corresponding box.

Step 8

Plot the computed rainfall intensity values from the IDF worksheet for all time increments of the desired return periods on the IDF Graph (Figure 3-5). Connect the points to create project specific IDF curves for different return periods.

Step 9

Enter the IDF Graph (Figure 3-5) on the time axis with the Time of Concentration for the drainage basin. Read the corresponding rainfall intensity value from the intensity axis for the design return frequency.
### DEPTH-DURATION-FREQUENCY WORKSHEET

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### INTENSITY-DURATION-FREQUENCY WORKSHEET

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Project Location: ____________________________  Figure 3-4  DDF/IDF  Worksheet
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Date: ____________________________
Computed by: ____________________________
3.3.1.2.2 RAINFALL IN THE SIMPLIFIED PEAK FLOW METHOD

The Simplified Peak Flow method uses the 24-hour total depth of precipitation for the design frequency event. Obtain the 24-hour rainfall depth directly from the appropriate Figure in APPENDIX E. For NMSHTD projects, there is no reduction factor applied to 2-year, 5-year, and 10-year rainfall depths. This represents a slight departure from the original SCS method (SCS, 1985) adding a small measure of safety for frequent return period events.

The time distribution of rainfall is built into the Simplified Peak Flow method. This statewide rainfall distribution varies from 45% to over 85% of the 24-hour rainfall occurring in the peak hour of the storm as the Time of Concentration varies from 10 hours to 0.1 hours respectively.

3.3.1.2.3 RAINFALL IN THE SCS UNIT HYDROGRAPH METHOD

Proper application of this method requires use of a 24-hour rainfall event with the peak precipitation rate occurring at 6 hours. Rainfall data for the SCS Unit Hydrograph method consists of 24-hour point precipitation depths and a rainfall distribution. Point precipitation depths for the design return period may be obtained directly from the Figures in APPENDIX E.

For NMSHTD projects the rainfall distribution used with the SCS Unit Hydrograph method is called the Modified NOAA–SCS rainfall distribution. This Modified NOAA–SCS rainfall distribution is a combination of the peak rainfall intensity defined by NOAA, with an SCS Type II–a storm rearrangement. NOAA 6-hour and 24-hour point precipitation values are used to compute rainfall intensities throughout the hypothetical storm. These rainfall intensities are used to construct a depth–duration–frequency curve. Incremental rainfall depths are then reordered around the storm peak at 6 hours to create the Type II–a distribution.

The Modified NOAA–SCS rainfall distribution adjusts the peak hour rainfall intensity for each location in New Mexico. Peak hour point precipitation ranges from about 55% to almost 80%, depending on location. The original SCS method used a Type II–a distribution, where “a” represents the ratio of the 1-hour point precipitation to the 24-hour point precipitation, in percent. The SCS used a map (1973) to define areas of New Mexico where different rainfall distributions should be used. A Type II–60, Type II–65, Type II–70 or a Type II–75 distribution were defined for different physiographic regions of New Mexico. The procedure given in this manual results in a similar range of rainfall distributions which are less generalized. A comparison of the Modified NOAA–SCS rainfall distribution with “a” values from the original SCS map (1973) shows similar values in most locations around the state (Heggen, 1995, unpublished).

A manual method of computing the Modified NOAA–SCS rainfall distribution is described below. The NMSHTD Drainage Section has developed a spreadsheet to compute the Modified NOAA–SCS rainfall distribution (NMRAIN.WK4), given the 6-hour and 24-hour point precipitation values from Figures E–1 through E–12, or the current NOAA Atlas.
Manual Rainfall Distribution Procedure:

Step 1

Compute the 5-minute through 24-hour depths as described in SECTION 3.3.1.2.1 for the desired return frequency event. Enter the depth values in the rainfall DDF worksheet. Use linear interpolation to find the rainfall depths associated with the time increments listed in column 2 of Figure 3-6.

Step 2

Enter the interpolated depth values in column 3 of the Worksheet. Subtract successive depth values (row 2 minus row 1, row 3 minus row 2, etc.) to obtain the incremental depth values (column 4).

Step 3

Copy incremental depth values from column 4 to column 7 of the worksheet. The first value in column 4 is copied to the cell in column 7 adjacent to the “rearranged n” value of 1 found in column 6, the second value in column 4 goes next to “rearranged n” value of 2, etc.

Step 4

The first value in column 8 will be the same as the first value in column 7. Thereafter, values in column 8 increase by the amount shown in column 7. Beginning at the top of the sheet, add each incremental depth value in column 7 to the previous cumulative depth in column 8 to obtain the new value of cumulative depth for column 8.

Column 8 now contains the rainfall distribution corresponding to the hyetograph time steps shown in column 5.
## The Modified NOAA–SCS Rainfall Distribution Worksheet

<table>
<thead>
<tr>
<th></th>
<th>Time (duration) (hrs)</th>
<th>Cumulative Depth (inches)</th>
<th>Incremental Depth (inches)</th>
<th>Hyetograph time period (hrs)</th>
<th>Rearranged n</th>
<th>Incremental Depth (inches)</th>
<th>Cumulative Depth (inches)</th>
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<tr>
<td>0</td>
<td>0</td>
<td>0.0</td>
<td>0.0</td>
<td>0 – 1.0</td>
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<td></td>
<td></td>
</tr>
<tr>
<td>1</td>
<td>.25</td>
<td></td>
<td></td>
<td>1.0 – 2.0</td>
<td>17</td>
<td></td>
<td></td>
</tr>
<tr>
<td>2</td>
<td>.50</td>
<td></td>
<td></td>
<td>2.0 – 3.0</td>
<td>15</td>
<td></td>
<td></td>
</tr>
<tr>
<td>3</td>
<td>.75</td>
<td></td>
<td></td>
<td>3.0 – 4.0</td>
<td>13</td>
<td></td>
<td></td>
</tr>
<tr>
<td>4</td>
<td>1.0</td>
<td></td>
<td></td>
<td>4.0 – 4.5</td>
<td>11</td>
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<td></td>
</tr>
<tr>
<td>5</td>
<td>1.25</td>
<td></td>
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<td>4.5 – 5.0</td>
<td>9</td>
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<td></td>
</tr>
<tr>
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<td>1.50</td>
<td></td>
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<td>5.0 – 5.25</td>
<td>7</td>
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<td></td>
</tr>
<tr>
<td>7</td>
<td>1.75</td>
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<td>2.0</td>
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<td></td>
<td>5.50 – 5.75</td>
<td>3</td>
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<tr>
<td>9</td>
<td>2.5</td>
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<td>5.75 – 6.0</td>
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<td>6.0 – 6.25</td>
<td>2</td>
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<td></td>
</tr>
<tr>
<td>11</td>
<td>3.5</td>
<td></td>
<td></td>
<td>6.25 – 6.50</td>
<td>4</td>
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</tr>
<tr>
<td>12</td>
<td>4.0</td>
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<td>6.50 – 6.75</td>
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<tr>
<td>13</td>
<td>5.0</td>
<td></td>
<td></td>
<td>6.75 – 7.0</td>
<td>8</td>
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<tr>
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<td>7.0 – 7.5</td>
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<td>7.0</td>
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<td>7.5 – 8.0</td>
<td>12</td>
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<td>8.0</td>
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<td>8.0 – 9.0</td>
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<tr>
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<td>9.0</td>
<td></td>
<td></td>
<td>9.0 – 10.0</td>
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<td></td>
</tr>
<tr>
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<td>10.0</td>
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<td></td>
<td>10.0 – 11.0</td>
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<td></td>
</tr>
<tr>
<td>19</td>
<td>11.0</td>
<td></td>
<td></td>
<td>11.0 – 12.0</td>
<td>20</td>
<td></td>
<td></td>
</tr>
<tr>
<td>20</td>
<td>12.0</td>
<td></td>
<td></td>
<td>12.0 – 14.0</td>
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<td>14.0 – 16.0</td>
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<td>16.0</td>
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<td>16.0 – 18.0</td>
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<tr>
<td>23</td>
<td>18.0</td>
<td></td>
<td></td>
<td>18.0 – 20.0</td>
<td>24</td>
<td></td>
<td></td>
</tr>
<tr>
<td>24</td>
<td>20.0</td>
<td></td>
<td></td>
<td>20.0 – 22.0</td>
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<tr>
<td>25</td>
<td>22.0</td>
<td></td>
<td></td>
<td>22.0 – 24.0</td>
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<td></td>
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<tr>
<td>26</td>
<td>24.0</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Project Location: ____________________________
CN#: _______________________________________
Date: ______________________________________
Computed by: ___________________________ Checked by: ________________________

Figure 3-6
The Modified NOAA–SCS Rainfall Distribution Worksheet
3.3.1.3 Rainfall Losses and Runoff Curve Numbers

Runoff curve numbers are used to quantify rainfall losses such as infiltration, interception and depression storage. Curve numbers are required input for the SCS rainfall runoff models used in this manual: Simplified Peak Flow and SCS Unit Hydrograph methods. In practice, curve numbers range from about 40 to 100, with larger curve numbers representing more runoff. Factors such as land use, ground cover type, hydrologic condition and hydrologic soil group are used to select a curve number.

Methods for selecting a runoff curve number and for making areal adjustments are described below. When carefully followed, these methods will yield a curve number which represents the runoff response of the watershed for the assumed watershed conditions. It is very important that the designer consider what changes will occur in the watershed during the year. The NMSHTD cannot design for anticipated changes in development. However, the designer should account for seasonal variations in vegetation and ground cover. The condition of the watershed may vary dramatically from the date of field reconnaissance to the annual season of largest historic runoff. This problem is most evident in cultivated agricultural areas where 1) the land is planted in row crops that are short or tall depending on plant type and growing season, or 2) the crop has been harvested and the ground is plowed or fallow, or 3) the crop type may be changed from year to year. The designer must exercise engineering judgement to determine the appropriate runoff curve number for a particular drainage basin or sub-basin.

3.3.1.3.1 Curve Number Selection

Primary factors used in the selection of a curve number are described below. The designer must evaluate the watershed in terms of these factors to select an appropriate curve number. Tabulated curve number values are provided in this manual and may also be found in several SCS publications (SCS, 1986). A graphic method for selecting curve numbers in rural areas is provided in Figure 3-8. As an additional resource, photographs of different land uses and ground cover types are provided in Appendix A.

Land Use — categorizes the land into several broad categories of usage, including rangeland, agricultural and urban. Land use is further subdivided by ground cover type and hydrologic condition. Particularly for agricultural land use, the land treatment can be a major consideration (i.e. terracing, crop rotation, etc.). In areas of human activity, compaction of natural soils may change the runoff response. For urban areas the density of development, type of landscaping, treatment of idle land and network of drainage conveyances should all be considered.

Ground Cover Type and Cover Density — describes the type of vegetation in the watershed. Arid rangeland areas may have weeds, grasses, sagebrush, desert shrubs, etc. Areas of greater rainfall may have piñon—juniper, continuous grasses, deciduous or coniferous woods, etc. Agricultural lands may be in pasture, in crops, fallow, etc. In urban areas the ground cover type is closely related with the land use. The percentage of impervious area is the most important factor in urban areas. Figure 3-9 provides a method for adjusting curve numbers to reflect the percent impervious area. Designers should assume that all of the impervious area is "connected." In rural and agricultural areas the ground cover density has a big effect...
on the runoff response of the watershed. For these areas the designer must estimate ground cover type and density at the time of year when large runoff events are most likely to occur. Figure 3–7 shows how to estimate ground cover density.

**Hydrologic Condition** – a “poor” hydrologic condition indicates impaired infiltration and therefore increased runoff. A “good” hydrologic condition indicates factors which encourage infiltration. For agricultural lands the hydrologic condition is a combination of factors including percent ground cover, canopy of vegetation, amount of year-round cover, percent of residue cover on the ground, grazing usage, and degree of roughness. For arid and semi-arid lands the percent ground cover determines the hydrologic condition.

**Hydrologic Soil Group** – categorizes the surface and subsurface soils in terms of their ability to absorb water. Sandy soils tend to fall into group “A,” whereas clay soils and rock outcrops are usually in the “D” group. “A” soils are relatively permeable whereas “D” soils are not. SCS Soil Surveys include aerial photograph maps of soil series, and for each series a hydrologic soil group has been assigned. SCS Soil Surveys are available by county for the majority of New Mexico. Most of the soil surveys were performed through aerial photo interpretation of large areas and detailed field inspections at selected locations. In watershed areas where excavation or extensive reworking of the surface soils has occurred, the designer should use field inspections to confirm the hydrologic soil group of the present surface soils.

**Antecedent Moisture Condition (AMC)** – describes the amount of moisture in the soil at the time rainfall begins. Antecedent moisture is categorized into three conditions: dry (I), average (II) and wet (III). Tables 3–1 through 3–4 list curve number values for various land use categories and average AMC. The assumption of AMC = II is valid for design watershed conditions on NMSHTD projects. For arid lands, an AMC of II may appear conservative, but represents conditions which could reasonably occur in conjunction with the design rainfall event. Occasionally a different AMC may be considered on a specific project. When required, the curve number for an average AMC may be adjusted as shown in Table 3–5.


Types of cover densities for grasses, weeds, and brush. Use basal densities for design.

Standard method of measuring ground cover density.

Figure 3-7
Estimating Ground Cover Density
Desert Brush: Brush–weed and grass mixtures with brush the predominant element. Some typical plants are – Mesquite, Creosote, Yuccas, Sagebrush, Saltbush, etc. This area is typical of lower elevations of desert and semi–desert areas.

Herbaceous: Grass–weed–brush mixtures with brush the minor element. Some typical plants are – Grama, Tobosa, Broom Snakeweeds, Sagebrush, Saltbush, Mesquite, Yucca, etc. This area is typical of lower elevations of desert and semi–desert areas.

Mountain Brush: Mountain brush mixtures of Oak, Mountain Mohagany, Apache Plume, Rabbit Brush, Skunk Brush, Sumac, Cliff Rose, Snowberry, etc. Mountain Brush is typical of intermediate elevations and generally higher annual rainfall than Desert Brush and herbaceous areas.

Juniper – Grass: These areas are mixed with varying amounts of juniper, piñon, grass, and cholla cover, or may be predominantly of one species. Grass cover is generally heavier than desert grasses due to higher annual precipitation. Juniper – Grass is typical of mountain slopes and plateaus of intermediate elevations.

Ponderosa Pine: These are forest lands typical of higher elevations where the principal cover is timber.

Figure 3–8
Hydrologic Soil – Cover Complexes and Associated Curve Numbers

Adapted from SCS, Chapter 2 for NM, 1985
Table 3-1 — Runoff Curve Numbers for Arid and Semiarid Rangelands
Source: USDA SCS, TR-55, 1986

<table>
<thead>
<tr>
<th>Cover Description</th>
<th>Hydrologic Condition</th>
<th>Curve Numbers for Hydrologic Soil Group —</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>A³</td>
<td>B</td>
</tr>
<tr>
<td>Herbaceous — mixture of grass, weeds, and</td>
<td>Poor</td>
<td>80</td>
</tr>
<tr>
<td>low growing brush, with brush the minor element.</td>
<td>Fair</td>
<td>71</td>
</tr>
<tr>
<td></td>
<td>Good</td>
<td>62</td>
</tr>
<tr>
<td>Oak–aspen — mountain brush mixture of oak</td>
<td>Poor</td>
<td>66</td>
</tr>
<tr>
<td>brush, aspen, mountain mahogany, bitter brush, maple, and other brush.</td>
<td>Fair</td>
<td>48</td>
</tr>
<tr>
<td></td>
<td>Good</td>
<td>30</td>
</tr>
<tr>
<td>Piñon, juniper, or both; grass understory.</td>
<td>Poor</td>
<td>75</td>
</tr>
<tr>
<td></td>
<td>Fair</td>
<td>58</td>
</tr>
<tr>
<td></td>
<td>Good</td>
<td>41</td>
</tr>
<tr>
<td>Sagebrush with grass understory.</td>
<td>Poor</td>
<td>67</td>
</tr>
<tr>
<td></td>
<td>Fair</td>
<td>51</td>
</tr>
<tr>
<td></td>
<td>Good</td>
<td>35</td>
</tr>
<tr>
<td>Desert shrub — major plants include saltbush,</td>
<td>Poor</td>
<td>63</td>
</tr>
<tr>
<td>greasewood, creosotebush, blackbrush, bursage,</td>
<td>Fair</td>
<td>55</td>
</tr>
<tr>
<td>palo verde, mesquite, and cactus.</td>
<td>Good</td>
<td>49</td>
</tr>
</tbody>
</table>

¹ Average runoff condition.

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.
Table 3-2 — Runoff Curve Numbers for Cultivated Agricultural Lands

Source: USDA SCS, TR-55, 1986

<table>
<thead>
<tr>
<th>Cover Type</th>
<th>Treatment(^2)</th>
<th>Hydrologic Condition(^3)</th>
<th>A</th>
<th>B</th>
<th>C</th>
<th>D</th>
</tr>
</thead>
<tbody>
<tr>
<td>Fallow</td>
<td>Bare soil</td>
<td>-</td>
<td>77</td>
<td>86</td>
<td>91</td>
<td>94</td>
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<tr>
<td></td>
<td>Crop Residue Cover (CR)</td>
<td>Poor</td>
<td>76</td>
<td>85</td>
<td>90</td>
<td>93</td>
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<tr>
<td></td>
<td></td>
<td>Good</td>
<td>74</td>
<td>83</td>
<td>88</td>
<td>90</td>
</tr>
<tr>
<td>Row crops</td>
<td>Straight Row (SR)</td>
<td>Poor</td>
<td>72</td>
<td>81</td>
<td>88</td>
<td>91</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Good</td>
<td>67</td>
<td>78</td>
<td>85</td>
<td>89</td>
</tr>
<tr>
<td></td>
<td>SR + CR</td>
<td>Poor</td>
<td>71</td>
<td>80</td>
<td>87</td>
<td>90</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Good</td>
<td>64</td>
<td>75</td>
<td>82</td>
<td>85</td>
</tr>
<tr>
<td></td>
<td>Contoured (C)</td>
<td>Poor</td>
<td>70</td>
<td>79</td>
<td>84</td>
<td>88</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Good</td>
<td>65</td>
<td>75</td>
<td>82</td>
<td>86</td>
</tr>
<tr>
<td></td>
<td>C + CR</td>
<td>Poor</td>
<td>69</td>
<td>78</td>
<td>83</td>
<td>87</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Good</td>
<td>64</td>
<td>74</td>
<td>81</td>
<td>85</td>
</tr>
<tr>
<td></td>
<td>Contoured &amp; Terraced (C&amp;T)</td>
<td>Poor</td>
<td>66</td>
<td>74</td>
<td>80</td>
<td>82</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Good</td>
<td>62</td>
<td>71</td>
<td>78</td>
<td>81</td>
</tr>
<tr>
<td></td>
<td>C&amp;T + CR</td>
<td>Poor</td>
<td>65</td>
<td>73</td>
<td>79</td>
<td>81</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Good</td>
<td>61</td>
<td>70</td>
<td>77</td>
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<tr>
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<td>SR</td>
<td>Poor</td>
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<td></td>
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<td>75</td>
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<tr>
<td></td>
<td>SR + CR</td>
<td>Poor</td>
<td>64</td>
<td>75</td>
<td>83</td>
<td>86</td>
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<tr>
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<td>60</td>
<td>72</td>
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<tr>
<td></td>
<td>C</td>
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<td>73</td>
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<td>84</td>
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<tr>
<td></td>
<td>C + CR</td>
<td>Poor</td>
<td>62</td>
<td>73</td>
<td>81</td>
<td>84</td>
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<tr>
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<td>80</td>
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<tr>
<td></td>
<td>C&amp;T</td>
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<td>61</td>
<td>72</td>
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</tr>
<tr>
<td></td>
<td></td>
<td>Good</td>
<td>59</td>
<td>70</td>
<td>78</td>
<td>81</td>
</tr>
<tr>
<td></td>
<td>C&amp;T + CR</td>
<td>Poor</td>
<td>60</td>
<td>71</td>
<td>78</td>
<td>81</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Good</td>
<td>58</td>
<td>69</td>
<td>77</td>
<td>80</td>
</tr>
<tr>
<td>Close-seeded or</td>
<td>SR</td>
<td>Poor</td>
<td>66</td>
<td>77</td>
<td>85</td>
<td>89</td>
</tr>
<tr>
<td>broadcast legumes or</td>
<td></td>
<td>Good</td>
<td>58</td>
<td>72</td>
<td>81</td>
<td>85</td>
</tr>
<tr>
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<td>C</td>
<td>Poor</td>
<td>64</td>
<td>75</td>
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</tr>
<tr>
<td></td>
<td></td>
<td>Good</td>
<td>55</td>
<td>69</td>
<td>78</td>
<td>83</td>
</tr>
<tr>
<td></td>
<td>C&amp;T</td>
<td>Poor</td>
<td>63</td>
<td>73</td>
<td>80</td>
<td>83</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Good</td>
<td>51</td>
<td>67</td>
<td>76</td>
<td>80</td>
</tr>
</tbody>
</table>

\(^1\) Average runoff condition.

\(^2\) Crop residue cover applies only if residue is on at least 5% of the surface throughout the year.

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

Poor: Factors impair infiltration and tend to increase runoff.

Good: Factors encourage average and better than average infiltration and tend to decrease runoff.
Table 3–3 — Runoff Curve Numbers for Other Agricultural Lands

Source: USDA SCS, TR–55, 1986

<table>
<thead>
<tr>
<th>Cover Description</th>
<th>Hydrologic Soil Group –</th>
<th>Curves Numbers</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Cover Type</td>
<td>A</td>
</tr>
<tr>
<td>Pasture, grassland, or range—continuous forage for grazing.</td>
<td>Poor</td>
<td>68</td>
</tr>
<tr>
<td></td>
<td>Fair</td>
<td>49</td>
</tr>
<tr>
<td></td>
<td>Good</td>
<td>39</td>
</tr>
<tr>
<td>Meadow—continuous grass, protected from grazing and generally mowed for hay.</td>
<td>—</td>
<td>30</td>
</tr>
<tr>
<td>Brush–weed–grass mixture with brush the major element.</td>
<td>Poor</td>
<td>48</td>
</tr>
<tr>
<td></td>
<td>Fair</td>
<td>35</td>
</tr>
<tr>
<td></td>
<td>Good</td>
<td>30</td>
</tr>
<tr>
<td>Woods—grass combination (orchard or tree farm).</td>
<td>Poor</td>
<td>57</td>
</tr>
<tr>
<td></td>
<td>Fair</td>
<td>43</td>
</tr>
<tr>
<td></td>
<td>Good</td>
<td>32</td>
</tr>
<tr>
<td>Woods.</td>
<td>Poor</td>
<td>45</td>
</tr>
<tr>
<td></td>
<td>Fair</td>
<td>36</td>
</tr>
<tr>
<td></td>
<td>Good</td>
<td>30</td>
</tr>
<tr>
<td>Farmsteads—buildings, lanes, driveways, and surrounding lots.</td>
<td>—</td>
<td>59</td>
</tr>
</tbody>
</table>

1 Average runoff condition.

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

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

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

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

6 Poor: Forest litter, small trees, and brush are destroyed by heavy grazing or regular burning.
   Fair: Woods are grazed but not burned, and some forest litter covers the soil.
   Good: Woods are protected from grazing, and litter and brush adequately cover the soil.
Table 3-4 — Runoff Curve Numbers Urban Areas

Source: USDA SCS, TR-55, 1986

<table>
<thead>
<tr>
<th>Cover Description</th>
<th>Average Percent Impervious Area&lt;sup&gt;2&lt;/sup&gt;</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cover Type and Hydrologic Condition</td>
<td>A</td>
</tr>
<tr>
<td>Fully developed urban areas (vegetation established)</td>
<td></td>
</tr>
<tr>
<td>Open space (lawns, parks, golf courses, cemeteries, etc.)&lt;sup&gt;3&lt;/sup&gt;:</td>
<td></td>
</tr>
<tr>
<td>Poor condition (grass cover &lt; 50%)</td>
<td>68</td>
</tr>
<tr>
<td>Fair condition (grass cover 50% to 75%)</td>
<td>49</td>
</tr>
<tr>
<td>Good condition (grass cover &gt; 75%)</td>
<td>39</td>
</tr>
<tr>
<td>Impervious areas:</td>
<td></td>
</tr>
<tr>
<td>Paved parking lots, roofs, driveways, etc. (excluding right-of-way)</td>
<td>98</td>
</tr>
<tr>
<td>Streets and roads:</td>
<td></td>
</tr>
<tr>
<td>Paved; curbs and storm sewers (excluding right-of-way)</td>
<td>98</td>
</tr>
<tr>
<td>Paved; open ditches (including right-of-way)</td>
<td>83</td>
</tr>
<tr>
<td>Gravel (including right-of-way)</td>
<td>76</td>
</tr>
<tr>
<td>Dirt (including right-of-way)</td>
<td>72</td>
</tr>
<tr>
<td>Western desert urban areas:</td>
<td></td>
</tr>
<tr>
<td>Natural desert landscaping (pervious areas only)&lt;sup&gt;4&lt;/sup&gt;</td>
<td>63</td>
</tr>
<tr>
<td>Artificial desert landscaping (impervious weed barrier, desert shrub with 1- to 2-inch sand or gravel mulch and basin borders)</td>
<td>96</td>
</tr>
<tr>
<td>Urban districts:</td>
<td></td>
</tr>
<tr>
<td>Commercial and business</td>
<td>85</td>
</tr>
<tr>
<td>Industrial</td>
<td>72</td>
</tr>
<tr>
<td>Residential districts by average lot size:</td>
<td></td>
</tr>
<tr>
<td>1/8 acre or less (town houses)</td>
<td>65</td>
</tr>
<tr>
<td>1/4 acre</td>
<td>38</td>
</tr>
<tr>
<td>1/3 acre</td>
<td>30</td>
</tr>
<tr>
<td>1/2 acre</td>
<td>25</td>
</tr>
<tr>
<td>1 acre</td>
<td>20</td>
</tr>
<tr>
<td>2 acres</td>
<td>12</td>
</tr>
<tr>
<td>Developing urban areas</td>
<td></td>
</tr>
<tr>
<td>Newly graded areas (pervious areas only, no vegetation)&lt;sup&gt;5&lt;/sup&gt;</td>
<td>77</td>
</tr>
<tr>
<td>Vacant lands (CN's are determined using cover types similar to those in Table 3-3)</td>
<td></td>
</tr>
</tbody>
</table>

<sup>1</sup> Average runoff condition.

<sup>2</sup> 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 3.9.

<sup>3</sup> CN's shown are equivalent to those of pasture. Composite CN's may be computed for other combinations of open space cover type.

<sup>4</sup> Composite CN's for natural desert landscaping should be computed using Figure 3.9 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.

<sup>5</sup> Composite CN's to use for the design of temporary measures during grading and construction should be computed using Figure 3.9, based on the degree of development (impervious area percentage) and the CN's for the newly graded pervious areas.
Table 3-5 — Conversion from Average Antecedent Moisture Conditions to Dry and Wet Conditions
Source: USDA SCS, TR-55, 1986

<table>
<thead>
<tr>
<th>CN for Average Conditions</th>
<th>Corresponding CN's for</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Dry</td>
</tr>
<tr>
<td>100</td>
<td>100</td>
</tr>
<tr>
<td>95</td>
<td>87</td>
</tr>
<tr>
<td>90</td>
<td>78</td>
</tr>
<tr>
<td>85</td>
<td>70</td>
</tr>
<tr>
<td>80</td>
<td>63</td>
</tr>
<tr>
<td>75</td>
<td>57</td>
</tr>
<tr>
<td>70</td>
<td>51</td>
</tr>
<tr>
<td>65</td>
<td>45</td>
</tr>
<tr>
<td>60</td>
<td>40</td>
</tr>
<tr>
<td>55</td>
<td>35</td>
</tr>
<tr>
<td>50</td>
<td>31</td>
</tr>
<tr>
<td>45</td>
<td>26</td>
</tr>
<tr>
<td>40</td>
<td>22</td>
</tr>
<tr>
<td>35</td>
<td>18</td>
</tr>
<tr>
<td>30</td>
<td>15</td>
</tr>
<tr>
<td>25</td>
<td>12</td>
</tr>
<tr>
<td>20</td>
<td>10</td>
</tr>
<tr>
<td>15</td>
<td>6</td>
</tr>
<tr>
<td>10</td>
<td>4</td>
</tr>
<tr>
<td>5</td>
<td>2</td>
</tr>
</tbody>
</table>
Figure 3-9
Composite CN for Urban Areas with Connected and Unconnected Impervious Areas

Adapted from SCS. TR-55, 1986
### 3.3.1.3.2 Curve Number Weighting

When hydrologic conditions are consistent throughout the watershed, then use of a single curve number is appropriate. For watersheds where curve numbers vary by 10 or less, an area weighted curve number is sufficient. When curve numbers vary dramatically within the watershed, the designer should consider subdividing the watershed into different drainage sub-basins. An alternative to subdividing a highly variable drainage basin is to use a Runoff weighted curve number. Examples of each curve number weighting procedure are shown below.

#### Area Weighted Curve Number

40% of the drainage basin is characterized by CN = 65 
60% of the drainage basin is characterized by CN = 73

\[
CN = \frac{(.40)(65) + (.60)(73)}{1.00} = 69.8
\]

use CN = 70

#### Runoff Weighted Curve Number

40% of the drainage basin is characterized by CN = 88 
60% of the drainage basin is characterized by CN = 72

Assume a design rainfall event of 2.0 inches.

Use Figure 3–16 to estimate
1.0 inches of direct runoff from the CN = 88 land 
and 0.3 inches of direct runoff from the CN = 72 land 
the average runoff is calculated as

\[
\frac{(.40)(1.0) + (.60)(.03)}{1.00} = 0.58 \text{ inches} \quad \text{average direct runoff}
\]

Use Figure 3–16 to find a runoff weighted curve number of CN = 80

#### Comparison of Methods

Recall that by the area weighted method we would have obtained a CN = 78. 
The difference in this example is approximately 0.1 inches of direct runoff. This difference becomes particularly important for small rainfall amounts where lower CN values may not predict any runoff. In the example above a curve number difference of 2 resulted in a

\[
\frac{0.58 - 0.50}{.50} = .16
\]

the runoff weighted curve number predicts a 16% increase in runoff.

Use the criteria described above to select the best weighting method.
3.3.1.4 Time of Concentration

Time of Concentration is defined as the time required for runoff to travel from the hydraulically most distant part of the watershed to the point of interest. Time of concentration is one of the most important drainage basin characteristics needed to calculate the peak rate of runoff. An accurate estimate of a watershed's time of concentration is crucial to every type of hydrologic modeling.

The method used to calculate time of concentration must be consistent with the method of hydrologic analysis selected for design. Designers working on NMSHTD projects must use the time of concentration methods specified in this section for each hydrologic method. Mixing of methods is not allowed on NMSHTD projects. Table 3-6 defines the correct time of concentration method to be used for each hydrologic method.

Within each watershed the designer must locate the primary watercourse. This is the watercourse that extends from the bottom of the watershed or drainage structure to the most hydraulically remote point in the watershed. Most designers begin at the bottom of the watershed and work their way upstream until the longest watercourse has been found. At the top of the watershed a defined watercourse may not exist. In these areas overland flow will be the dominant flow type. As the runoff proceeds downstream, overland flows will naturally begin to coalesce, gradually concentrating together. Shallow concentrated flow often has enough force to shape small gullies in erosive soils. Gullies eventually gather together until a defined stream channel is formed. The water course is now large enough to be identified on a quadrangle topographic map.

Sections along the primary watercourse should be identified which are hydraulically similar. Time of concentration is estimated for each section of the watercourse. Time of concentration in any given watershed is simply the sum of flow travel times within hydraulically similar reaches along the longest watercourse. Time of concentration is determined from measured reach lengths and estimated average reach velocities. The basic equation for time of concentration is:

\[ T_c = \left( \frac{L_1}{V_1} + \frac{L_2}{V_2} + \frac{L_3}{V_3} + \ldots + \frac{L_n}{V_n} \right) \frac{1}{60} \]

where

- \( T_c \) = Time of concentration, minutes
- \( V_1 \) = Average flow velocity in the uppermost reach of the watercourse, ft./sec.
- \( L_1 \) = Length of the uppermost reach of the watercourse, ft.
- \( V_2, V_3, \ldots \) = Average flow velocities in subsequent reaches progressing downstream, ft./sec.
- \( L_2, L_3, \ldots \) = Lengths of subsequent reaches progressing downstream, ft.
<table>
<thead>
<tr>
<th>Hydrologic Method</th>
<th>Watershed Condition</th>
<th>Time of Concentration Method</th>
</tr>
</thead>
<tbody>
<tr>
<td>Rational Method</td>
<td>Un-gullied Watershed*</td>
<td>Upland Method</td>
</tr>
<tr>
<td></td>
<td>Gullied Watershed*</td>
<td>Kirpich Formula</td>
</tr>
<tr>
<td>Simplified Peak Flow Method</td>
<td>Un-gullied Watershed*</td>
<td>Upland Method</td>
</tr>
<tr>
<td></td>
<td>Gullied Watershed*</td>
<td>Kirpich Formula</td>
</tr>
<tr>
<td></td>
<td>Watershed Partially Gullied</td>
<td>Upland Method for the Un-gullied Portion, then Kirpich Formula for the Gullied Portion**</td>
</tr>
<tr>
<td>USGS Regression Equations</td>
<td></td>
<td>NOT REQUIRED</td>
</tr>
<tr>
<td>Unit Hydrograph Method</td>
<td>No Defined Stream Channel</td>
<td>Upland Method</td>
</tr>
<tr>
<td></td>
<td>Defined Stream Channel</td>
<td>Stream Hydraulic Method</td>
</tr>
<tr>
<td>Approved Urban Method</td>
<td>All Conditions</td>
<td>Use (T_c) Method Specified for the Approved Urban Method***</td>
</tr>
</tbody>
</table>

*A watershed is considered un-gullied if 10% or less of the primary watercourse exhibits gullying.

**Mixing \(T_c\) Methods in a watershed is only allowed with the Simplified Peak Flow Method.

***When using AHYMO with the COMPUTE NM HYD routine, compute the time of concentration in accordance with the City of Albuquerque Design Process Manual. See \textit{Sections 3.2} and \textit{3.3.5} of this manual for limitations on the use of AHYMO.
3.3.1.4.1 **The Upland Method**

The Upland Method is used to estimate travel times for overland flow and shallow concentrated flow conditions. Originally developed by the SCS, the upland method is limited to use in watersheds less than 2000 acres in size, or to the upper reaches of larger watersheds. For NMSHTD projects the Upland Method may be used for computing the time of concentration when using the Rational Method or the Simplified Peak Flow method on an un-gullied watershed.

At the very top of the watershed, sheet flow is the predominant flow regime. The overland flow lines in Figure 3.10 may be used to estimate the velocity of sheet flow. Overland flow continues until the volume of water creates a shallow concentrated flow regime. In erosive soil formations with limited ground cover, the length of overland flow may be so short as to be negligible. Given the slope of the land and some knowledge of the ground cover conditions, Figure 3.10 may be used to estimate the velocity of shallow concentrated flow. For NMSHTD projects, shallow concentrated flow is assumed to occur from the end of overland flow to the bottom of a watershed where there is little or no gullying (10% or less). Where gullying is evident in the majority of the watercourse (by field inspection, or by a blue line on the USGS quadrangle topographic map), time of concentration should be computed by the Kirpich Method for the entire watershed. **When the Simplified Peak Flow method is being used for NMSHTD projects, the Upland Method may be used for the un-gullied portion of the watercourse, in combination with the Kirpich Formula for the gullied sections of the watercourse.**
Note: For watercourses with slopes less than 0.5 percent, use the overland flow velocity given for 0.5 percent, except for shallow concentrated flow where a flatter slope may be considered.

Figure 3-10
Flow Velocities for Overland and Shallow Concentrated Flows

Modified from SCS, NEH-4, 1972
3.3.1.4.2 TIME OF CONCENTRATION BY THE KIRPICH FORMULA

This method is used to calculate time of concentration in gullied watersheds when using the Rational Method or the Simplified Peak Flow Method. The Kirpich Formula should be used when gullying is evident in more than 10% of the primary watercourse. Gullying can be assumed if a blue line appears on the watercourse shown on the USGS quadrangle topographic map. The Kirpich Formula is given as:

\[ T_c = 0.0078 L^{0.77} S^{-0.385} \]  

(3–18)

where

- \( T_c \) = time of concentration, in minutes
- \( L \) = length from drainage to outlet along the primary drainage path, in feet
- \( S \) = average slope of the primary drainage path, in ft./ft.

The Kirpich Formula should generally be used for the entire drainage basin. The exception to this rule occurs when the Simplified Peak Flow Method is being used on NMSHTD projects and the watercourse has a mixture of gullied and un-gullied sections. In these situations, mixing of time of concentration methods is allowed. The Upland Method is used for the ungullied portion of the primary watercourse, and the Kirpich Formula is used for the gullied portion of the watercourse. The two times of concentration are added together to obtain the total time of concentration of the watershed. Typically the Kirpich Formula is only used for that portion of the watercourse shown in blue on the quadrangle topo map. **Mixing of time of concentration methods is only allowed with the Simplified Peak Flow Method for NMSHTD projects.**

3.3.1.4.3 THE STREAM HYDRAULIC METHOD

The stream hydraulic method is used when calculating peak flows by the Unit Hydrograph Method in a watercourse where a defined stream channel is evident (blue line, solid or broken, on a quadrangle topo map). The designer must measure or estimate the hydraulic properties of the stream channel, and must divide the total watercourse into channel reaches which are hydraulically similar. Field reconnaissance measurements of the stream channel are best, however sometimes direct measurements are not possible. The designer must determine the slope, channel cross section and an appropriate hydraulic roughness coefficient for each channel reach. Average slope is often determined from the topographic mapping of the watershed. Channel cross section should be measured in the field whenever possible. Roughness coefficients of the waterway should be based on actual observations of the watercourse or of nearby watercourses which are believed to be similar and which are more accessible.

Time of Concentration by the stream hydraulic method is simply the travel time in the stream channel. Channel flow velocities can be estimated from normal depth calculations for the watercourse. In addition to the average flow velocity, designers should compute the Froude Number of the flow. If the Froude number of the flow exceeds a value of 1.3, then the designer should verify that supercritical flow conditions can actually be sustained. For most earth lined channels the velocity calculation should be recomputed using a larger effective
friction factor until the Froude Number has a value less than 1.3. **Equation 3–19** is Manning's equation arranged to solve for velocity:

\[
V = \frac{1.49}{n} R^{2/3} S^{1/2}
\]

(3–19)

where

- \( V \) = average velocity in the channel, ft./sec.
- \( n \) = Manning's roughness coefficient
- \( R \) = hydraulic radius of the flow, ft.
- \( S \) = average channel slope, ft./ft.

and

\[
R = \frac{A}{P}
\]

\[
F_r = \frac{V}{\sqrt{gD}}
\]

\[
D = \frac{A}{T}
\]

where

- \( A \) = cross section area of the flow, ft.\(^2\)
- \( P \) = the wetted perimeter of the flow, ft.
- \( S \) = gravitational acceleration (\( g = 32.2 \) ft./sec.\(^2\))
- \( D \) = hydraulic depth, ft.
- \( T \) = top width of the flow, ft.

In order to solve Manning's equation for velocity, the designer must calculate or estimate the hydraulic radius (\( R \)). If the flow depth or flow rate is known, then the hydraulic radius may be found directly. However, the usual situation is that neither flow depth nor flow rate are known without first computing the time of concentration. Two procedures are provided below for solving this problem.

**Simplified Procedure**

a. **Wide Shallow Channels**

Use this method for channels where the flow depth is relatively shallow compared to the flow width. When this is true then the hydraulic radius (\( R \)) converges toward a value of 1.0. **Use of \( R = 1.0 \) is acceptable for NMSHTD projects where the stream channel is relatively wide and the flow is shallow.** Larger arroyo systems in alluvial terrain often satisfy this criterion.

b. **Moderate and Narrow Width Channels**

Use this method for all other channels. Estimate the flow depth from high water mark evidence or other available data. For most ephemeral stream channels the flow depth should be in the range of 1 - 4 ft. Where a channel has obvious channel banks in the 1 - 4 ft. height range, use the “bank full” depth. For most ephemeral streams use the bank full depth of the low flow channel. If the physical evidence suggests a flow depth greater than the height of an incised channel bank, use the physical evidence depth but compute the flow velocity based on water in the channel only (no overbank...
flow considered). Use the flow depth and channel cross section geometry to estimate $R$. For estimated flow depths deeper than 4 – 5 ft., the designer should consider using the iterative procedure described below.

Iterative Procedure

For some channel flow conditions the simplified procedure described above may not be adequate. In these cases, the iterative procedure described here must be followed. First, the peak rate of runoff from the watershed is estimated. A good estimate may be obtained using the USGS regression equations for New Mexico (See Section 3.3.4 of this Manual.) The flow rate for the velocity calculation is assumed to be two-thirds of the peak rate. Average channel velocity is calculated from Equation 3-19, using the other hydraulic parameters of the channel. The average channel velocity for each reach is then used to determine the total time of concentration of the watershed. After the peak discharge from the watershed is computed, the designer must reassess the flow rate used to compute an average channel velocity. If the assumed peak discharge is within 10% of the calculated peak discharge, the computed average channel velocity and resulting time of concentration should be reasonably accurate. Often a second iteration is required using two-thirds of the computed peak flow to compute a new average channel velocity. This iterative procedure should be continued until the assumed peak flow rate is within 10% of the computed peak flow rate. Sample Problem No. 5 illustrates the procedure. (Note: use of a computer or calculator program to calculate normal depth will greatly expedite this iterative procedure.)

3.3.2 Rational Formula Method

The Rational Formula Method is a widely accepted procedure for estimating peak rates of runoff from small watersheds. The Rational Formula may be used on NMSHTD projects for roadway drainage facilities and small drainage structures as described in Section 3.2 of this manual. The standard form of the equation is:

$$Q = C i A$$  \(3-20\)

where

- $Q$ = the peak rate of runoff, in cfs
- $C$ = a dimensionless runoff coefficient
- $i$ = the rainfall intensity, in inches/hour
- $A$ = the watershed area, in acres

This equation is in mixed units, however the conversion factor from in–ac/hr to cfs is 1.008 which introduces a negligible error. The Rational Formula has several assumptions implicit to the method, including:

- Peak flow occurs when all of the watershed is contributing runoff
- The rainfall intensity is uniform over a duration of time equal to or greater than $T_c$
- The frequency of the peak flow is equal to the frequency of the rainfall intensity
Limitations for using the Rational Formula on NMSHTD projects include the following:

- Total drainage area no larger than about 150 acres
- Land use must be fairly consistent throughout the watershed
- No drainage channels or other structures in the watershed which would require flood routing
- Time of Concentration does not exceed one hour

### 3.3.2.1 APPLICATION OF THE RATIONAL FORMULA

Measure the watershed area in acres. Construct an Intensity–Duration Frequency (IDF) curve as described in **SECTION 3.3.1.2** of this manual. Compute the Time of Concentration ($T_c$) for the watershed as described in **SECTION 3.3.1.4** of this manual. Enter the appropriate IDF curve (or spreadsheet) with a value of $T_c$ to obtain the design rainfall intensity. When $T_c$ is computed as less than 10 minutes, a minimum rainfall duration of 10 minutes should be used. When $T_c$ is computed as greater than 60 minutes, the Rational Method should not be used.

The runoff coefficient, $C$, is selected from **Figures 3–11 through 3–16**, depending on the ground cover, hydrologic soil group, type of development, and 1–hour rainfall depth for the design return period. Hydrologic soil groups are defined in **SECTION 3.3.1.3** and 1–hour rainfall depths are determined in **SECTION 3.3.1.2** of this manual. **Figures 3–11 through 3–16** show how $C$ varies with 1–hour rainfall depth. This is because $C$ is a function of infiltration and other hydrologic abstractions, relating the peak discharge to the theoretical peak discharge produced by 100% runoff.

When land use or other factors vary significantly throughout the watershed, an area weighted $C$ value should be used. The weighted $C$ value is computed by the equation:

$$
Weighted\ C = \frac{\sum C_i \cdot A_i}{A}
$$

where

- $C_i = C$ value for one part of the watershed
- $A_i = area, A, in$ acres for the corresponding part of the watershed

The designer should select the appropriate **Figure (3–11 through 3–16)** depending on the watershed location (desert, upland range, mountain or urban) and the predominant vegetation type (cactus, brush, grasses, juniper, pine). Enter each **Figure** with the design 1–hour rainfall depth. **Move vertically up through the Figure** until the appropriate curve is found, then move horizontally to find the design $C$ value. The appropriate curve is selected based on the Hydrologic Soil Group (HSG) and the percent ground cover of the vegetation.
As a Function of Rainfall Depth, Hydrologic Soil Group (HSG), and % of Vegetation Cover
Adapted from Arizona DOT Highway Drainage Design Manual, 1993

Figure 3-11
Rational "C" Coefficient
Upland Rangeland
(Grass & Brush)
As a Function of Rainfall Depth, Hydrologic Soil Group (HSG), and % of Vegetation Cover
Adapted from Arizona DOT Highway Drainage Design Manual, 1993

Figure 3-12
Rational “C” Coefficient
Desert
(Cactus, Grass & Brush)
As a Function of Rainfall Depth, Hydrologic Soil Group (HSG), and % of Vegetation Cover
Adapted from Arizona DOT Highway Drainage Design Manual, 1993

Figure 3-13
Rational "C" Coefficient
Mountain (Juniper & Grass)
As a Function of Rainfall Depth, Hydrologic Soil Group (HSG), and % of Vegetation Cover
Adapted from Arizona DOT Highway Drainage Design Manual, 1993

Figure 3-14
Rational "C" Coefficient
Mountain
(Grass & Brush)
As a Function of Rainfall Depth, Hydrologic Soil Group (HSG), and % of Vegetation Cover
Adapted from Arizona DOT Highway Drainage Design Manual, 1993

Figure 3-15
Rational "C" Coefficient
Mountain
(Ponderosa Pine)
As a Function of Rainfall Depth, and Type of Development
Adapted from Arizona DOT Highway Drainage Design Manual, 1993

Figure 3-16
Rational "C" Coefficient
Developed Watersheds
Problem No. 1

Location: Near Santa Fe
Elevation: 7,000 ft.
Watershed: Limited access urban highway, pavement runoff
Design Frequency Flood: 10–year
Watershed Area: 2 acres

Compute the time of concentration.

Total length the drainage swale up to the drainage divide, 550 ft.
Assume:
- paved area sheet flow for 100 ft. at $S = 0.020$ ft./ft.
- overland flow in the drainage swale for 100 ft. at $S = 0.040$ ft./ft.
- shallow concentrated flow for remaining 350 ft. at $S = 0.030$ ft./ft.

Select appropriate velocities from Figure 3–10.

$$T_{c \text{ upland}} = \left( \frac{100 \ \text{ft.}}{2.8 \ \text{ft./sec.}} + \frac{100 \ \text{ft.}}{2.0 \ \text{ft./sec.}} + \frac{350 \ \text{ft.}}{3.4 \ \text{ft./sec.}} \right) \frac{1}{60} = 3.1 \text{ minutes}$$

Use the minimum allowable time of concentration.

$$T_{c} = 10.0 \text{ minutes}$$

Compute the rainfall depth and intensity. Follow the procedures in Section 3.3.1.2.1. (This computation can be done manually, or using the NMSHTD spreadsheet NMRAIN.WK4.)

2–year through 100–year rainfall depths for the 6–hour and 24–hour durations are entered into the spreadsheet NMRAIN.WK4 along with NOAA zone (1 or 2) and elevation. See attached spreadsheet printout.

Obtain rainfall intensity from the spreadsheet computed Intensity–Duration–Frequency (IDF) curve, or from the IDF worksheet if the manual procedure is followed.

The one–hour rainfall depth, $P_1$, is obtained from the DDF table as 1.359 inches. Select a runoff coefficient “C” value from Figure 3–16.

$$C = 0.95$$

Compute the peak rate of runoff from the watershed using the Rational Formula, Equation 3–20.

$$Q = C \cdot i \cdot A$$
$$Q = 0.95 \cdot (3.67) \cdot (2.0)$$
$$Q = 7.0 \ \text{cfs}$$

This is the 10–year design frequency flood.
Project Location: US 285, North of Santa Fe
CN #: XXXX
Computed by: CSP
Date: 11/15/95

NOAA Zone: 2
Elevation: 7000 feet

Return Period for Rainfall Distribution: 50 Year

Rainfall Depths in Inches

<table>
<thead>
<tr>
<th>Duration</th>
<th>2-year</th>
<th>5-year</th>
<th>10-year</th>
<th>25-year</th>
<th>50-year</th>
<th>100-year</th>
</tr>
</thead>
<tbody>
<tr>
<td>6-hour</td>
<td>1.210</td>
<td>1.550</td>
<td>1.800</td>
<td>2.100</td>
<td>2.350</td>
<td>2.690</td>
</tr>
<tr>
<td>24-hour</td>
<td>1.600</td>
<td>2.000</td>
<td>2.300</td>
<td>2.650</td>
<td>2.940</td>
<td>3.300</td>
</tr>
</tbody>
</table>

Depth - Duration - Frequency (DDF) Table

<table>
<thead>
<tr>
<th>Duration</th>
<th>2-year</th>
<th>5-year</th>
<th>10-year</th>
<th>25-year</th>
<th>50-year</th>
<th>100-year</th>
</tr>
</thead>
<tbody>
<tr>
<td>5-min</td>
<td>0.247</td>
<td>0.333</td>
<td>0.394</td>
<td>0.463</td>
<td>0.533</td>
<td>0.623</td>
</tr>
<tr>
<td>10-min</td>
<td>0.383</td>
<td>0.517</td>
<td>0.611</td>
<td>0.719</td>
<td>0.826</td>
<td>0.967</td>
</tr>
<tr>
<td>15-min</td>
<td>0.485</td>
<td>0.655</td>
<td>0.774</td>
<td>0.911</td>
<td>1.047</td>
<td>1.225</td>
</tr>
<tr>
<td>30-min</td>
<td>0.672</td>
<td>0.908</td>
<td>1.073</td>
<td>1.262</td>
<td>1.451</td>
<td>1.698</td>
</tr>
<tr>
<td>60-min</td>
<td>0.851</td>
<td>1.150</td>
<td>1.359</td>
<td>1.598</td>
<td>1.837</td>
<td>2.150</td>
</tr>
<tr>
<td>2-hr</td>
<td>0.974</td>
<td>1.287</td>
<td>1.510</td>
<td>1.769</td>
<td>2.012</td>
<td>2.334</td>
</tr>
<tr>
<td>3-hr</td>
<td>1.056</td>
<td>1.379</td>
<td>1.611</td>
<td>1.885</td>
<td>2.130</td>
<td>2.459</td>
</tr>
<tr>
<td>6-hr</td>
<td>1.210</td>
<td>1.550</td>
<td>1.800</td>
<td>2.100</td>
<td>2.350</td>
<td>2.690</td>
</tr>
<tr>
<td>12-hr</td>
<td>1.405</td>
<td>1.775</td>
<td>2.050</td>
<td>2.375</td>
<td>2.645</td>
<td>2.995</td>
</tr>
<tr>
<td>24-hr</td>
<td>1.600</td>
<td>2.000</td>
<td>2.300</td>
<td>2.650</td>
<td>2.940</td>
<td>3.300</td>
</tr>
</tbody>
</table>
Problem No. 2

Location: Near Roswell  
Elevation: 4,000 ft.  
Watershed: Upland range  
Design Frequency Flood: 50–year  
Watershed Area: 80 acres

Compute the time of concentration.

Total length of the watercourse to the hydraulically most remote point in the drainage basin, 2,100 ft.

Assume: overland flow for 400 ft. at S = 0.050 ft./ft.
shallow concentrated flow for the remaining 1,700 ft. at S = 0.030 ft./ft.

No defined watercourse observed on the quad sheet.

Select appropriate velocities from Figure 3–10.

\[ T_{e_{\text{upland}}} = \left( \frac{400 \text{ ft.}}{2.2 \text{ ft./sec.}} + \frac{1,700 \text{ ft.}}{3.4 \text{ ft./sec.}} \right) \frac{1}{60} = 11.4 \text{ minutes} \]

Use the spreadsheet NMRAIN.WK4 to compute the rainfall data.

From the spreadsheet DDF table, read down the 50–year column
  10 min. depth = 1.145 inches  
  15 min. depth = 1.450 inches

Use linear interpolation to get 11.4 min. depth.

\[ D_{11.4} = 1.145 + (1.450 - 1.145) \left( \frac{1.4}{5} \right) = 1.23 \text{ inches} \]

Rainfall intensity is computed as depth divided by time.

\[ i = \frac{1.23 \text{ inches}}{11.4 \text{ min.} \left( \frac{1 \text{ hour}}{60 \text{ min.}} \right)} = 6.48 \text{ inches/hour} \]

Compute an area–weighted runoff coefficient “C” for the drainage basin.
Read the one–hour rainfall depth from the spreadsheet DDF table as \( P_1 = 2.544 \text{ inches} \).

Subarea 1: 30 acres, upland range, HSG = C, percent vegetation cover estimated as 30%.

From Figure 3–11, C = 0.46
Subarea 2: 50 acres, upland range, HSG = B, 30% vegetation cover.

From Figure 3–11, $C = 0.28$

The area weighted Runoff coefficient is computed as

$$\text{Weighted } C = \frac{(0.46) (30) + (0.28) (50)}{30 + 50} = 0.35$$

Compute the 50-year design frequency flood using the Rational Formula.

$$Q = (0.35) (6.48) (80)$$

$$Q = 181 \text{ cfs}$$
Project Location: US 70, west of Roswell
CN #: XXXX
Computed by: CSP
Date: 11/15/95

NOAA Zone: 1
Elevation: 4000 feet
Return Period for Rainfall Distribution: 50 Year

Rainfall Depths in Inches

<table>
<thead>
<tr>
<th></th>
<th>2-year</th>
<th>5-year</th>
<th>10-year</th>
<th>25-year</th>
<th>50-year</th>
<th>100-year</th>
</tr>
</thead>
<tbody>
<tr>
<td>6-hour</td>
<td>1.580</td>
<td>2.100</td>
<td>2.500</td>
<td>3.100</td>
<td>3.500</td>
<td>3.950</td>
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<tr>
<td>24-hour</td>
<td>1.920</td>
<td>2.650</td>
<td>3.150</td>
<td>3.900</td>
<td>4.400</td>
<td>4.850</td>
</tr>
</tbody>
</table>

Depth - Duration - Frequency (DDF) Table

<table>
<thead>
<tr>
<th></th>
<th>2-year</th>
<th>5-year</th>
<th>10-year</th>
<th>25-year</th>
<th>50-year</th>
<th>100-year</th>
</tr>
</thead>
<tbody>
<tr>
<td>5-min</td>
<td>0.331</td>
<td>0.454</td>
<td>0.540</td>
<td>0.639</td>
<td>0.738</td>
<td>0.867</td>
</tr>
<tr>
<td>10-min</td>
<td>0.513</td>
<td>0.704</td>
<td>0.838</td>
<td>0.991</td>
<td>1.145</td>
<td>1.345</td>
</tr>
<tr>
<td>15-min</td>
<td>0.650</td>
<td>0.892</td>
<td>1.062</td>
<td>1.256</td>
<td>1.450</td>
<td>1.704</td>
</tr>
<tr>
<td>30-min</td>
<td>0.900</td>
<td>1.237</td>
<td>1.472</td>
<td>1.741</td>
<td>2.009</td>
<td>2.362</td>
</tr>
<tr>
<td>60-min</td>
<td>1.140</td>
<td>1.565</td>
<td>1.863</td>
<td>2.203</td>
<td>2.544</td>
<td>2.989</td>
</tr>
<tr>
<td>2-hr</td>
<td>1.290</td>
<td>1.748</td>
<td>2.081</td>
<td>2.510</td>
<td>2.871</td>
<td>3.318</td>
</tr>
<tr>
<td>3-hr</td>
<td>1.403</td>
<td>1.886</td>
<td>2.245</td>
<td>2.740</td>
<td>3.116</td>
<td>3.565</td>
</tr>
<tr>
<td>6-hr</td>
<td>1.580</td>
<td>2.100</td>
<td>2.500</td>
<td>3.100</td>
<td>3.500</td>
<td>3.950</td>
</tr>
<tr>
<td>12-hr</td>
<td>1.750</td>
<td>2.375</td>
<td>2.825</td>
<td>3.500</td>
<td>3.950</td>
<td>4.400</td>
</tr>
<tr>
<td>24-hr</td>
<td>1.920</td>
<td>2.650</td>
<td>3.150</td>
<td>3.900</td>
<td>4.400</td>
<td>4.850</td>
</tr>
</tbody>
</table>

![Intensity - Duration - Frequency Graph](image)
3.3.3  **SIMPLIFIED PEAK FLOW METHOD**

The Simplified Peak Flow method estimates the peak rate of runoff and runoff volume from small to medium size watersheds. This method was developed by the Soil Conservation Service and revised by that agency for use in New Mexico ("Peak Rates of Discharge for Small Watersheds," Chapter 2, SCS, 1985). Infiltration and other losses are estimated using the SCS Curve Number (CN) methodology. Input parameters are consistent with those used in the SCS Unit Hydrograph method. The Simplified Peak Flow method is limited for NMSHTD use to single basins less than 5 square miles in area, and should not be used when $T_c$ exceeds 8.0 hours. This method may be used on NMSHTD projects for those conditions identified in **SECTION 3.2** of this manual. This method should not be used for watersheds with perennial stream flow.

The original Chapter 2 method (SCS, 1973) included unit peak discharge curves for different rainfall distributions, varying from 45% to 85% of the rainfall occurring in the peak hour. After analysis of stream gage data, the 1985 update included only one peak discharge curve, representing a variable rainfall distribution depending on the Time of Concentration of the watershed. Therefore, a separate estimate of rainfall distribution is not required to use this method. The analysis of gage data also showed that the method overestimated peak flows at elevations above 7500 ft. Drainage structures above this elevation should be evaluated by the unit hydrograph or USGS regression equation methods.

**3.3.3.1 APPLICATION**

**Step 1** – Gather Input Data

- Establish the appropriate Design Frequency Flood(s) for analysis
- Estimate the drainage area, A, in acres (**SECTION 3.3.1.1**)
- Compute the Time of Concentration, $T_c$, in hours (**SECTION 3.3.1.4**)
- Determine the appropriate runoff Curve Number, CN, for the drainage basin (**SECTION 3.3.1.3**)
- Obtain the 24-hour rainfall depth, $P_{24}$, for the appropriate design frequency, from **APPENDIX E**
**Step 2** Determine the unit peak discharge, \( q_u \), for the watershed. The unit peak discharge can be read from Figure 3-18, given the time of concentration, or calculated directly by the following equation:

\[
q_u = 0.543 T_c^{-0.812} 10^{-\frac{1}{10} \left( \log (T_c) - 0.3 \right) - \frac{1}{10} \left( \log (T_c) - 0.3 \right)^{1.5}}
\]  

(3-22)

where

- \( q_u \) = unit peak discharge from the watershed, in cfs/ac-in
- \( T_c \) = time of concentration, in hours

**Note:** for \( T_c > 0.5 \) hours, the last term of the equation, \( 10^{-\frac{1}{10} \left( \log (T_c) - 0.3 \right) - \frac{1}{10} \left( \log (T_c) - 0.3 \right)^{1.5}} \), is equal to 1.0

**Step 3**

Calculate the direct runoff from the watershed. The direct runoff is expressed as an average depth of water over the entire watershed, in inches. The direct runoff may be read from Figure 3-17 using the 24-hour rainfall depth \( P_{24} \) in inches, and the runoff curve number, CN. The runoff depth may also be calculated from the following equation:

\[
Q_d = \frac{\left( P_{24} - \frac{200}{CN} + 2 \right)^2}{P_{24} + \frac{800}{CN} - 8}
\]

(3-23)

where

- \( Q_d \) = average runoff depth for the entire watershed, in inches

**Step 4**

Compute the peak discharge from the watershed by the following equation:

\[
Q_p = A \cdot Q_d \cdot q_u
\]

(3-24)

where

- \( Q_p \) = peak discharge, in cfs
- \( A \) = drainage area, in acres

**Step 5**

Compute the runoff volume, if required. The runoff volume is obtained by the equation:

\[
Q_v = \frac{Q_d \cdot A}{12}
\]

(3-25)

where

- \( Q_v \) = runoff volume from the watershed, in ac-ft
Figure 3–17
Estimating Direct Runoff

Adapted from SCS, NEH–4, 1964
Figure 3-18
Unit Peak Discharge for the Simplified Peak Flow Method

Adapted from SCS, Chapter 2 for NM, 1985
Step 6

Estimate Transmission Losses, if required. For watersheds less than 1.0 square miles in size there is no reduction factor applied. Where base flow is observed or known to occur, transmission losses should not be included. For large watersheds with sand or gravel bed channels, transmission losses may need to be considered. To compute transmission losses, follow the procedure in the SCS document NEH-4, Chapter 19, Transmission Losses, 1983.
Simplified Peak Flow Worksheet

Structure Location: ____________________________________________________________

Structure Description: _______________________________________________________

Drainage Area: \( A = \) ______________ acres

Time of Concentration: \( T_c = \) ______________ hours

Weighted Runoff Curve Number: \( CN = \) ______________

Unit Peak Discharge (from Figure 3-18): \( q_u = \) ______________ cfs/ac-in

Design Frequency Flood

_____ -year \hspace{1cm} _____ -year

24-hour Rainfall Depth (APPENDIX E): \( P_{24} = \) __________ in. \hspace{1cm} \( P_{24} = \) __________ in.

Direct Runoff (Figure 3-17):
\( Q_d = \) __________ in. \hspace{1cm} \( Q_d = \) __________ in.

Peak Discharge, \( Q_p = A \cdot Q_d \cdot q_u: \)
\( Q_p = \) __________ cfs \hspace{1cm} \( Q_p = \) __________ cfs

Runoff Volume, \( Q_v = A \cdot Q_d/12: \)
\( Q_v = \) __________ ac-ft \hspace{1cm} \( Q_v = \) __________ ac-ft

Transmission Losses, if applicable (computed by methods in SCS NEH 4, Chapter 19, 1983)

Predicted Runoff Volume:
\( Q_{pv} = \) __________ ac-ft \hspace{1cm} \( Q_{pv} = \) __________ ac-ft

Predicted peak Discharge:
\( Q_{pp} = \) __________ ac-ft \hspace{1cm} \( Q_{pp} = \) __________ ac-ft

Project Location: _______________________________ Figure 3-19
CN#: __________________________________________ Simplified
Date: __________________________________________ Peak Flow
Computed by: ___________________________ Checked by: ______________ Worksheet

PAGE NUMBER 3-54 NMSHTD DRAINAGE MANUAL DECEMBER 1995
3.3.3.2 SIMPLIFIED PEAK FLOW METHOD EXAMPLE PROBLEMS

Problem No. 3

Location: South of Deming, sparse desert brush
Elevation: 4,000 ft.
Design Frequency Flood: 50-year
Watershed Area: 250 acres (< 5 sq. mi., so okay for Simplified Peak Flow Method)
24-hour rainfall depth, 50-year return frequency, from Figure E-11, \( P_{50} = 3.3 \) inches

Compute the time of concentration.

The upper watershed shows significant erosion, with many gullies evident. Assume overland flow occurs for the first 200 ft. at \( S = 0.035 \) ft./ft. Assume shallow concentrated flow occurs for the remaining 600 ft. at \( S = 0.025 \) ft./ft. until a defined stream channel is evident on the quadrangle topographic map. Select appropriate velocities from Figure 3–10.

\[
T_{c\,\text{upland}} = \left( \frac{200}{1.8} + \frac{600}{3.1} \right) \frac{1}{60} = 5.1 \text{ min.}
\]

The defined stream channel is a broad wash where larger flows really spread out. Channel length is measured as 3,000 ft. Bottom width \( \approx 30 \) ft., \( S = 0.015 \) ft./ft., \( n = 0.030 \). For this channel we can use the simplifying assumption that \( R = 1 \).

Compute channel velocity based on Manning's equation.

\[
V = \frac{1.49}{0.030} \left( \frac{3000}{6.1} \right)^{\frac{1}{3}} \left( \frac{0.015}{1} \right) = 6.08 \text{ ft./sec.}
\]

\[
T_{c\,\text{stream hydraulic}} = \left( \frac{3000}{6.1} \right) \frac{1}{60} = 8.2 \text{ min.}
\]

The total time of concentration for the watershed is

\[
T_{c\,\text{watershed}} = 5.1 + 8.2 = 13.3 \text{ minutes} = 0.222 \text{ hours}
\]

\( (0.222 \) hours is less than 8.0 hours, okay to use the simplified peak flow method.)

Select a representative runoff Curve Number.
Vegetation: Desert brush
HSG: A
Hydrologic Condition: poor, minimal ground cover
From Table 3–1, select \( CN = 63 \)

Compute the direct runoff using Equation 3–23 (or obtain \( Q_d \) from Figure 3–17).
\[
Q_d = \frac{\left(3.3 - \left(\frac{200}{63}\right) + 2\right)^2}{3.3 + \left(\frac{800}{63}\right) - 8} = 0.56 \text{ inches}
\]

Because the watershed is less than 1.0 square miles, Transmission Losses are not considered.

The unit peak discharge, \(q_u\), is read from Figure 3-18, or calculated directly by Equation 3-22.

\[q_u = 1.607 \text{ cfs/ac-in}\]

Compute the design frequency peak flow by Equation 3-24.

\[Q_p = (250)(0.56)(1.607)\]
\[Q_p = 225 \text{ cfs}\]
Problem No. 4

Location: North of Crownpoint, gently sloping rangeland
Elevation: 6,500 ft.
Design Frequency Flood: 50–year

Watershed Area: 600 acres (< 5 sq. mi., okay)
24–hour rainfall depth, 50–year return frequency, from Figure E–11, $P_{50} = 2.2$ inches

Compute the time of concentration.

The total length of the watercourse to the hydraulically most remote point in the drainage basin is 7,600 ft.

We are unable to inspect the entire watershed, therefore some assumptions are necessary:

Assume overland flow occurs for 400 ft. at $S = 0.020$ ft./ft.
Shallow concentrated flow is assumed for the remaining 1,200 ft. until a defined stream channel is observed on the quad sheet topo. $S = 0.010$ ft./ft.
Select appropriate velocities from Figure 3–10.

\[
T_{c \text{ upland}} = \left( \frac{200 \text{ ft.}}{1.4 \text{ ft./sec.}} + \frac{200 \text{ ft.}}{1.4 \text{ ft./sec.}} + \frac{1,200 \text{ ft.}}{2.0 \text{ ft./sec.}} \right) \frac{1}{60} = 14.8 \text{ min.}
\]

The remainder of watercourse is a defined stream channel in alluvial material.
Length = 6,000 ft., Slope = 0.010 ft./ft.
The stream channel observed upstream of the highway has the following cross sectional properties:

15 ft. bottom, 1:1 sideslopes, cut banks approximately 4 ft. tall

We estimate Manning's $n = 0.030$, sand bed channel without vegetation.

Use the simplified procedure for moderate and narrow width channels to estimate flow velocity.

Estimate the flow depth from vegetation and old debris, $d = 3.0$ ft.
Flow Area = 45 ft.$^2$
Wetted Perimeter = 21 ft.

The Hydraulic Radius, $R = \frac{A}{P} = \frac{45}{21} = 2.1$
Flow velocity computed by Manning's Equation is

\[ V = \frac{1.49}{n} R^{\frac{2}{3}} S^{\frac{1}{3}} = \frac{1.49}{0.030} (2.1)^{\frac{2}{3}} (0.010)^{\frac{1}{2}} = 8.3 \text{ ft./sec.} \]

\[ T_{c, \text{stream hydraulic}} = \left( \frac{6,000 \text{ ft.}}{8.3 \text{ ft./sec.}} \right) \frac{1}{60} = 12.0 \text{ min.} \]

The total time of concentration for the watershed is

\[ T_{c, \text{watershed}} = 14.8 + 12.0 = 26.8 \text{ minutes} = 0.447 \text{ hours} \]

Select a representative runoff Curve Number.

- Vegetation: Desert brush
- HSG: B
- Hydrologic Condition: 20% ground cover

From Figure 3-8, select CN = 82.5

Compute the direct runoff using Equation 3-23 (or obtain \( Q_d \) from Figure 3-17).

\[ Q_d = \left( \frac{2.2 - \left( \frac{200}{82.5} \right) + 2}{2.2 + \left( \frac{800}{82.5} \right) - 8} \right)^2 = 0.81 \text{ inches} \]

The unit peak discharge, \( q_u \), is read from Figure 3-18, or calculated directly by Equation 3-22.

\[ q_u = 1.037 \text{ cfs/acre} \]

Compute the design frequency peak flow by Equation 3-24.

\[ Q_p = (600) (0.81) (1.037) = 504 \text{ cfs} \]

As a check, compute the normal depth for this discharge.

- for \( Q_p = 504 \text{ cfs} \), normal depth \( d = 3.14 \text{ ft.} \)

This confirms our assumed depth. If the normal depth was substantially different from the assumed value then we would need to revise our \( T_c \) calculation accordingly.
Problem No. 5

Location: Near Chama, forested mountain terrain
Elevation: 7,500 ft.
Design Frequency Flood: 50–year

Watershed Area: 3,000 acres (< 5.0 square miles, okay for Simplified Peak Flow Method)
P₂₄, 50–year = 2.8 inches

Compute the time of concentration.

Unable to inspect the entire watershed, therefore some assumptions are necessary:

Assume overland flow occurs for 400 ft. at S = 0.100 ft./ft.
Shallow concentrated flow is assumed for the remaining 2,200 ft. until a defined stream channel is observed on the quad sheet topo. S = 0.060 ft./ft.
Select appropriate velocities from Figure 3–10.

\[
T_{\text{upland}} = \left( \frac{100 \text{ ft.}}{0.8 \text{ ft./sec.}} + \frac{300 \text{ ft.}}{2.8 \text{ ft./sec.}} + \frac{2,200 \text{ ft.}}{5.0 \text{ ft./sec.}} \right) \frac{1}{60} = 11.2 \text{ min.}
\]

The remainder of watercourse is a defined stream channel.

Since there is not any real data on the stream channel geometry and no good evidence of flow depths, use the iterative procedure.

Estimate the peak discharge using the USGS statewide small basin regression equations.

From Table 3–7 we find the 50–year return frequency equation to be

\[
Q_{\text{small basin}} = 7.92 \times 10^2 \cdot A^{0.45}
\]

\[
Q_{\text{small basin}} = 792 \left( \frac{3,000}{640} \right)^{0.45} = 1,587 \text{ cfs}
\]

For the SCS iterative procedure, the flow rate used to compute channel flow velocity is assumed to be 2/3 of the estimated peak flow.

\[
Q_{\text{velocity}} = \frac{2}{3} (1587) = 1,063 \text{ cfs}
\]

The length of stream channel has been measured as 14,500 ft. from the quad sheet topo.

Assume a channel geometry: 10 ft. bottom, 2:1 sideslopes, n = 0.045, slope = 0.035 ft./ft.

By normal depth calculation, Velocity, V = 12.4 ft./sec.
The travel time is then

\[ T_{c \text{ stream hydraulic}} = \left( \frac{14,500 \text{ ft.}}{12.4 \text{ ft./sec.}} \right) \frac{1}{60} = 19.5 \text{ min.} \]

Total Time of Concentration for the watershed is

\[ T_{c \text{ watershed}} = 11.2 + 19.5 = 30.7 \text{ min.} = 0.512 \text{ hours} \]

Select a representative Runoff Curve Number.

Vegetation: Woods  
HSG: C  
Hydrologic Condition: Fair  
From Table 3–3, choose CN = 73

Compute the direct runoff \( Q_d \) (Equation 3–23), or use Figure 3–17.

\[
Q_d = \left( \frac{2.8 - \left( \frac{200}{73} \right) + 2}{2.8 + \left( \frac{800}{73} \right) - 8} \right)^2 = 0.74 \text{ inches}
\]

Channel seepage was observed, so transmission losses are neglected.

The unit peak discharge is given by Equation 3–22, or may be obtained directly from Figure 3–18.

Since \( T_c = 0.512 \text{ hours} > 0.5 \text{ hours} \), Equation 3–22 reduces to

\[
q_u = (0.543) T_c^{-1.12}
\]
\[
q_u = (0.543) (0.512)^{-1.12}
\]
\[
q_u = 0.94 \text{ cfs/ac–in}
\]

The design frequency peak flow is given by Equation 3–24.

\[
Q_n = (3000) (0.74) (0.94)
\]
\[
Q_n = 2087 \text{ cfs}
\]

Since the calculated \( Q_n \) is more than 20% different than the estimated \( Q_p \), the time of concentration for the stream hydraulic reach should be revised.

\[
Q_{velocity} = \frac{2}{3} (2087) = 1398 \text{ cfs}
\]

For the same channel geometry, \( V = 13.3 \text{ ft./sec.} \).
\[ T_{c \text{ stream hydraulic}} = \left( \frac{14,500 \text{ ft.}}{13.3 \text{ ft./sec.}} \right) \frac{1}{60} = 18.2 \text{ min.} \]

\[ T_{c \text{ watershed}} = 11.2 = 18.2 = 29.4 \text{ min.} = 0.490 \text{ hours} \]

\[ q_a = 0.97 \text{ cfs/acre-in} \]

\[ Q_p = (3000) (0.74) (0.97) \]
\[ Q_p = 2153 \text{ cfs} \]

This peak flow is within 10\% of the \( Q_p \) used to estimate channel flow velocity, so no further iterations are required.

The peak flow calculated using the simplified peak flow method is somewhat larger than the estimated peak flow using the USGS small basin regression equation. Remember that the USGS equation is valid for the entire state, regardless of vegetation or elevation. Since we have used a runoff Curve Number to model the runoff response of this watershed, the \( Q_p \) calculated by the simplified peak flow method is probably better. Also, the observed channel seepage suggests using the higher peak flow value. Use \( Q_p = 2153 \text{ cfs} \) for design.
Stream gage data and associated rainfall data from sites around New Mexico have been compiled by the United States Geological Survey (USGS) (Waltemeyer, 1986; Thomas and Gold, 1982). These watersheds were evaluated to find basin and climatic characteristics which are statistically significant in predicting peak flow rates at the stream gages. Regression equations were developed which predict the peak rate of runoff from watersheds within certain physiographic regions of New Mexico for different return period events.

The most recent set of USGS regression equations for New Mexico (Waltemeyer, 1996) were developed using 201 gaging stations, the majority of which are in New Mexico. Flood discharges for selected exceedance probabilities were determined for each streamflow gaging station. Logarithms of annual peak flows were fitted to a log Pearson Type III probability distribution to develop flood frequency curves according to standard techniques (Interagency Advisory Committee on Water Data, 1982). New Mexico was divided into eight physiographic regions, yielding regression equations with the best data fit. Figure 3-20 shows the eight regions within New Mexico. The NMSHTD has selected these equations for predicting peak rates of runoff for larger NMSHTD drainage basins (see SECTION 3.2). These USGS regression equations may be used in rural areas, or in urban areas as described in SECTION 3.3.4.2. The USGS regression equations are also the preferred hydrologic analysis method when sizing drainage structures for perennial streams.
Figure 3-20
USGS Physiographic Regions

Adapted from USGS, WRI XXXX, 1996
3.3.4.1 RURAL PEAK DISCHARGE METHOD

Peak discharges from rural watersheds may be calculated directly from the equations presented in Table 3-7. Locate the project on Figure 3-20 to determine the appropriate physiographic region. For each watershed in the project, determine the basin characteristic(s) needed for the regression equation. Select the regression equation appropriate for the desired return frequency event. Basin characteristics are defined as follows:

- **A** — drainage area, in square miles
- **E** — mean basin elevation, in feet above sea level
- **E_c** — average channel elevation upstream from the gaging station (use structure location instead of gaging station for NMSHTD projects)
- **P_{24,10}** or **P_{24,25}** — maximum precipitation depths, in inches, (24-hour storm, 10-year or 25-year recurrence interval). Precipitation depths can be found in APPENDIX E.
- **Q_x** — the predicted peak flow value for an x-year return frequency, in cfs. Equations are given for x = 2-, 5-, 10-, 25-, 50-, 100- and 500-year events.

When a drainage basin is located within more than one physiographic region, a weighted averaging technique is used. Compute the peak discharge from each physiographic portion of the watershed for that watershed area only. Then compute an area weighted peak discharge for the entire drainage basin. See example problem No. 8 as an illustration of the area weighting procedure.

Statistically significant basin characteristics vary from one physiographic region to the next. For all of the regions, basin area was found to be a significant basin characteristic. In the Northeast Plains, the Northwest Plateau, the Southeast Plains, and the Southwest Desert regions, basin area was the only significant characteristic used in developing a regression equation. Basin elevation or average channel elevation were found to be significant in most of the Mountain regions and in the Central Mountain Valley region. Characteristics such as main channel slope and length, mean annual precipitation, and percentage of lakes and forests were evaluated by the USGS study. Using these parameters did not improve the final regression equations. Cross-correlation between some basin characteristics reduced the need for multiple independent variables in the regression equations.

The USGS has also developed a set of regression equations for small drainage basins (< 10 sq. mi.). These equations have been generalized for the whole state, and are valid for drainage basins with a mean basin elevation less than 7500 ft. The small basin equations are presented in Table 3-8. Use of the USGS small basin equations on NMSHTD projects should be limited to checks on results from the Simplified Peak Flow Method or comparison with the USGS regional regression equations.
Table 3-7
USGS Rural Flood Frequency Equations for New Mexico

<table>
<thead>
<tr>
<th>Region 1</th>
<th>Northeast Plains</th>
<th>Average Standard Error of Prediction</th>
<th>Log Units</th>
</tr>
</thead>
<tbody>
<tr>
<td>$Q_2 = 1.14 \times 10^2 \cdot A^{0.53}$</td>
<td></td>
<td>0.346</td>
<td>(3–26)</td>
</tr>
<tr>
<td>$Q_5 = 3.07 \times 10^2 \cdot A^{0.50}$</td>
<td></td>
<td>0.301</td>
<td>(3–27)</td>
</tr>
<tr>
<td>$Q_{10} = 5.08 \times 10^2 \cdot A^{0.49}$</td>
<td></td>
<td>0.288</td>
<td>(3–28)</td>
</tr>
<tr>
<td>$Q_{25} = 8.53 \times 10^2 \cdot A^{0.48}$</td>
<td></td>
<td>0.284</td>
<td>(3–29)</td>
</tr>
<tr>
<td>$Q_{50} = 1.18 \times 10^3 \cdot A^{0.48}$</td>
<td></td>
<td>0.285</td>
<td>(3–30)</td>
</tr>
<tr>
<td>$Q_{100} = 1.58 \times 10^3 \cdot A^{0.48}$</td>
<td></td>
<td>0.291</td>
<td>(3–31)</td>
</tr>
<tr>
<td>$Q_{500} = 2.80 \times 10^3 \cdot A^{0.48}$</td>
<td></td>
<td>0.312</td>
<td>(3–32)</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Region 2</th>
<th>Northwest Plateau</th>
<th>Average Standard Error of Prediction</th>
<th>Log Units</th>
</tr>
</thead>
<tbody>
<tr>
<td>$Q_2 = 8.47 \times 10^1 \cdot A^{0.47}$</td>
<td></td>
<td>0.390</td>
<td>(3–33)</td>
</tr>
<tr>
<td>$Q_5 = 1.97 \times 10^2 \cdot A^{0.46}$</td>
<td></td>
<td>0.311</td>
<td>(3–34)</td>
</tr>
<tr>
<td>$Q_{10} = 3.06 \times 10^2 \cdot A^{0.46}$</td>
<td></td>
<td>0.282</td>
<td>(3–35)</td>
</tr>
<tr>
<td>$Q_{25} = 4.86 \times 10^2 \cdot A^{0.45}$</td>
<td></td>
<td>0.262</td>
<td>(3–36)</td>
</tr>
<tr>
<td>$Q_{50} = 6.54 \times 10^2 \cdot A^{0.45}$</td>
<td></td>
<td>0.255</td>
<td>(3–37)</td>
</tr>
<tr>
<td>$Q_{100} = 8.53 \times 10^2 \cdot A^{0.45}$</td>
<td></td>
<td>0.254</td>
<td>(3–38)</td>
</tr>
<tr>
<td>$Q_{500} = 1.45 \times 10^3 \cdot A^{0.45}$</td>
<td></td>
<td>0.265</td>
<td>(3–39)</td>
</tr>
</tbody>
</table>
Table 3-7

USGS Rural Flood Frequency Equations for New Mexico

<table>
<thead>
<tr>
<th>Region 3</th>
<th>Southeast Mountain</th>
<th>Average Standard Error of Prediction</th>
<th>Log Units</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>Q2 = 8.54 x 10^6 • A^{0.60} • (E/1000)^{-5.96}</td>
<td></td>
<td>0.150 (3-40)</td>
</tr>
<tr>
<td></td>
<td>Q5 = 7.14 x 10^7 • A^{0.67} • (E/1000)^{-6.69}</td>
<td></td>
<td>0.163 (3-41)</td>
</tr>
<tr>
<td></td>
<td>Q10 = 1.60 x 10^8 • A^{0.70} • (E/1000)^{-6.94}</td>
<td></td>
<td>0.169 (3-42)</td>
</tr>
<tr>
<td></td>
<td>Q25 = 3.04 x 10^8 • A^{0.75} • (E/1000)^{-7.10}</td>
<td></td>
<td>0.178 (3-43)</td>
</tr>
<tr>
<td></td>
<td>Q50 = 4.15 x 10^8 • A^{0.78} • (E/1000)^{-7.16}</td>
<td></td>
<td>0.187 (3-44)</td>
</tr>
<tr>
<td></td>
<td>Q100 = 5.21 x 10^8 • A^{0.81} • (E/1000)^{-7.19}</td>
<td></td>
<td>0.199 (3-45)</td>
</tr>
<tr>
<td></td>
<td>Q500 = 7.11 x 10^8 • A^{0.87} • (E/1000)^{-7.20}</td>
<td></td>
<td>0.236 (3-46)</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Region 4</th>
<th>Southeast Plains</th>
<th>Average Standard Error of Prediction</th>
<th>Log Units</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Q2 = 8.17 x 10^4 • A^{0.51}</td>
<td></td>
<td>0.538 (3-47)</td>
</tr>
<tr>
<td></td>
<td>Q5 = 2.36 x 10^5 • A^{0.54}</td>
<td></td>
<td>0.424 (3-48)</td>
</tr>
<tr>
<td></td>
<td>Q10 = 4.07 x 10^5 • A^{0.55}</td>
<td></td>
<td>0.374 (3-49)</td>
</tr>
<tr>
<td></td>
<td>Q25 = 7.21 x 10^5 • A^{0.57}</td>
<td></td>
<td>0.326 (3-50)</td>
</tr>
<tr>
<td></td>
<td>Q50 = 1.04 x 10^6 • A^{0.58}</td>
<td></td>
<td>0.300 (3-51)</td>
</tr>
<tr>
<td></td>
<td>Q100 = 1.43 x 10^6 • A^{0.59}</td>
<td></td>
<td>0.282 (3-52)</td>
</tr>
<tr>
<td></td>
<td>Q500 = 2.72 x 10^6 • A^{0.62}</td>
<td></td>
<td>0.262 (3-53)</td>
</tr>
</tbody>
</table>
Table 3-7
USGS Rural Flood Frequency Equations for New Mexico

<table>
<thead>
<tr>
<th>Region 5</th>
<th>Northern Mountain</th>
<th>Average Standard Error of Prediction</th>
<th>Log Units</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>$Q_2$</td>
<td>$8.54 \times 10^2 \cdot A^{0.83} \cdot \left( \frac{E}{1,000} \right)^{2.22} \cdot P_{24,25}^{0.31}$</td>
<td>0.343</td>
<td>(3–54)</td>
</tr>
<tr>
<td>$Q_5$</td>
<td>$7.39 \times 10^3 \cdot A^{0.81} \cdot \left( \frac{E}{1,000} \right)^{3.01} \cdot P_{24,25}^{0.63}$</td>
<td>0.309</td>
<td>(3–55)</td>
</tr>
<tr>
<td>$Q_{10}$</td>
<td>$2.19 \times 10^4 \cdot A^{0.81} \cdot \left( \frac{E}{1,000} \right)^{3.41} \cdot P_{24,25}^{0.81}$</td>
<td>0.297</td>
<td>(3–56)</td>
</tr>
<tr>
<td>$Q_{25}$</td>
<td>$6.90 \times 10^4 \cdot A^{0.80} \cdot \left( \frac{E}{1,000} \right)^{3.85} \cdot P_{24,25}^{1.03}$</td>
<td>0.294</td>
<td>(3–57)</td>
</tr>
<tr>
<td>$Q_{50}$</td>
<td>$1.44 \times 10^5 \cdot A^{0.80} \cdot \left( \frac{E}{1,000} \right)^{4.13} \cdot P_{24,25}^{1.18}$</td>
<td>0.298</td>
<td>(3–58)</td>
</tr>
<tr>
<td>$Q_{100}$</td>
<td>$2.80 \times 10^5 \cdot A^{0.80} \cdot \left( \frac{E}{1,000} \right)^{4.40} \cdot P_{24,25}^{1.33}$</td>
<td>0.306</td>
<td>(3–59)</td>
</tr>
<tr>
<td>$Q_{500}$</td>
<td>$1.10 \times 10^6 \cdot A^{0.80} \cdot \left( \frac{E}{1,000} \right)^{4.95} \cdot P_{24,25}^{1.64}$</td>
<td>0.337</td>
<td>(3–60)</td>
</tr>
</tbody>
</table>
Table 3-7
USGS Rural Flood Frequency Equations for New Mexico

<table>
<thead>
<tr>
<th>Region 6</th>
<th>Average Standard Error of Prediction</th>
<th>Log Units</th>
</tr>
</thead>
<tbody>
<tr>
<td>Central Mountain–Valley</td>
<td></td>
<td></td>
</tr>
<tr>
<td>$Q_2 = 7.47 \times 10^5 \cdot A^{0.50} \cdot \left( \frac{Ec}{1,000} \right)^{-5.28} \cdot P_{24,10}^{1.18}$</td>
<td>0.366</td>
<td>(3–61)</td>
</tr>
<tr>
<td>$Q_5 = 2.57 \times 10^5 \cdot A^{0.47} \cdot \left( \frac{Ec}{1,000} \right)^{-4.49} \cdot P_{24,10}^{1.76}$</td>
<td>0.274</td>
<td>(3–62)</td>
</tr>
<tr>
<td>$Q_{10} = 1.53 \times 10^5 \cdot A^{0.46} \cdot \left( \frac{Ec}{1,000} \right)^{-4.09} \cdot P_{24,10}^{2.06}$</td>
<td>0.231</td>
<td>(3–63)</td>
</tr>
<tr>
<td>$Q_{25} = 8.89 \times 10^4 \cdot A^{0.44} \cdot \left( \frac{Ec}{1,000} \right)^{-3.67} \cdot P_{24,10}^{2.37}$</td>
<td>0.193</td>
<td>(3–64)</td>
</tr>
<tr>
<td>$Q_{50} = 6.11 \times 10^4 \cdot A^{0.43} \cdot \left( \frac{Ec}{1,000} \right)^{-3.38} \cdot P_{24,10}^{2.57}$</td>
<td>0.180</td>
<td>(3–65)</td>
</tr>
<tr>
<td>$Q_{100} = 4.18 \times 10^4 \cdot A^{0.42} \cdot \left( \frac{Ec}{1,000} \right)^{-3.09} \cdot P_{24,10}^{2.74}$</td>
<td>0.173</td>
<td>(3–66)</td>
</tr>
<tr>
<td>$Q_{500} = 1.78 \times 10^4 \cdot A^{0.40} \cdot \left( \frac{Ec}{1,000} \right)^{-2.45} \cdot P_{24,10}^{3.03}$</td>
<td>0.185</td>
<td>(3–67)</td>
</tr>
</tbody>
</table>
### Table 3–7

**USGS Rural Flood Frequency Equations for New Mexico**

<table>
<thead>
<tr>
<th>Region 7</th>
<th>Southwest Desert</th>
<th>Average Standard Error of Prediction</th>
<th>Log Units</th>
</tr>
</thead>
<tbody>
<tr>
<td>$Q_2$</td>
<td>$1.28 \times 10^2 \cdot A^{0.46}$</td>
<td></td>
<td>0.229</td>
</tr>
<tr>
<td>$Q_5$</td>
<td>$2.46 \times 10^2 \cdot A^{0.48}$</td>
<td></td>
<td>0.211</td>
</tr>
<tr>
<td>$Q_{10}$</td>
<td>$3.45 \times 10^2 \cdot A^{0.49}$</td>
<td></td>
<td>0.212</td>
</tr>
<tr>
<td>$Q_{25}$</td>
<td>$4.91 \times 10^2 \cdot A^{0.50}$</td>
<td></td>
<td>0.220</td>
</tr>
<tr>
<td>$Q_{50}$</td>
<td>$6.15 \times 10^2 \cdot A^{0.51}$</td>
<td></td>
<td>0.231</td>
</tr>
<tr>
<td>$Q_{100}$</td>
<td>$7.51 \times 10^2 \cdot A^{0.52}$</td>
<td></td>
<td>0.244</td>
</tr>
<tr>
<td>$Q_{500}$</td>
<td>$1.12 \times 10^3 \cdot A^{0.55}$</td>
<td></td>
<td>0.279</td>
</tr>
</tbody>
</table>
Table 3-7

USGS Rural Flood Frequency Equations for New Mexico

<table>
<thead>
<tr>
<th>Region 8</th>
<th>Southwest Mountain</th>
<th>Average Standard Error of Prediction</th>
<th>Log Units</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
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</tr>
</tbody>
</table>

\[
Q_2 = 2.58 \times 10^7 \cdot A^{0.19} \cdot \left( \frac{Ec}{1,000} \right)^{-0.10} \\
Q_5 = 1.49 \times 10^7 \cdot A^{0.23} \cdot \left( \frac{Ec}{1,000} \right)^{-5.53} \\
Q_{10} = 1.03 \times 10^7 \cdot A^{0.25} \cdot \left( \frac{Ec}{1,000} \right)^{-5.19} \\
Q_{25} = 6.53 \times 10^6 \cdot A^{0.27} \cdot \left( \frac{Ec}{1,000} \right)^{-4.80} \\
Q_{50} = 4.69 \times 10^6 \cdot A^{0.29} \cdot \left( \frac{Ec}{1,000} \right)^{-4.52} \\
Q_{100} = 3.40 \times 10^6 \cdot A^{0.30} \cdot \left( \frac{Ec}{1,000} \right)^{-4.25} \\
Q_{500} = 1.66 \times 10^6 \cdot A^{0.32} \cdot \left( \frac{Ec}{1,000} \right)^{-3.68} \\
\]

Table 3–8

USGS Small Rural Basin Flood Frequency Equations for New Mexico

<table>
<thead>
<tr>
<th>State Wide Application</th>
<th>Average Standard Error of Prediction</th>
<th>Log Units</th>
</tr>
</thead>
<tbody>
<tr>
<td>$Q_2 = 1.07 \times 10^2 \cdot A^{0.39}$</td>
<td>0.413</td>
<td>(3–82)</td>
</tr>
<tr>
<td>$Q_5 = 2.43 \times 10^2 \cdot A^{0.42}$</td>
<td>0.326</td>
<td>(3–83)</td>
</tr>
<tr>
<td>$Q_{10} = 3.74 \times 10^2 \cdot A^{0.43}$</td>
<td>0.292</td>
<td>(3–84)</td>
</tr>
<tr>
<td>$Q_{25} = 5.91 \times 10^2 \cdot A^{0.44}$</td>
<td>0.266</td>
<td>(3–85)</td>
</tr>
<tr>
<td>$Q_{50} = 7.92 \times 10^2 \cdot A^{0.45}$</td>
<td>0.255</td>
<td>(3–86)</td>
</tr>
<tr>
<td>$Q_{100} = 1.03 \times 10^3 \cdot A^{0.46}$</td>
<td>0.248</td>
<td>(3–87)</td>
</tr>
<tr>
<td>$Q_{500} = 1.73 \times 10^3 \cdot A^{0.47}$</td>
<td>0.247</td>
<td>(3–88)</td>
</tr>
</tbody>
</table>

Note: These equations were developed for drainage basins less than 10 sq. mi. and less than 7500 ft. mean basin elevation. See Section 3.2 of this manual for limitations on the use of these equations.
3.3.4.2 **Urban Use of USGS Regression Equations**

USGS regression equations have been developed to predict peak rates of runoff from urban watersheds. The methods use the equivalent rural peak discharge (see **Section 3.3.4.1**) in combination with other basin parameters to predict urban peak discharges. Data from 269 gaged basins in 31 states were analyzed in this USGS study (Sauer et al, 1983). A seven parameter set of regression equations was developed which included factors such as drainage area size, channel slope, rainfall intensity, percent impervious, ponded storage, basin lag time, and a Basin Development Factor (BDF). The resulting equations show consistently strong coefficients of correlation. A simpler set of regression equations was also developed which utilized only three parameters: drainage area size, rural peak discharge, and the BDF. The three parameter USGS regression equations have coefficients of correlation almost as good as the seven parameter equations. The NMSHTD has selected the three parameter equations for urban use because of their ease of use.

3.3.4.3 **Basin Development Factor**

The Basin Development Factor, or BDF, provides a measure of the efficiency of the drainage system. The BDF can readily be determined from drainage maps and field inspections. The procedure is as follows:

**Step 1**

Divide the drainage basin into thirds. **Figure 3–21** shows the schematic division of several drainage basins. Stream distances within any given third are approximately equal; however, stream distances within different thirds may be different. Each third contains approximately one third of the contributing drainage area. Lines dividing the basin into thirds can generally be drawn by eye, without precise measurements.

**Step 2**

Evaluate the efficiency of the drainage system. Four aspects of improvement are described below. Evaluate each third of the basin in terms of the four aspects. For each aspect assign a value of 1 or 0. Sum the values from each third to determine the BDF. Full development and maximum urban effects on peak rates of runoff will occur when the BDF = 12.
Schematic of typical drainage basin shapes and subdivision into basin thirds. Note that stream-channel distances within any given third of a basin in the examples are approximately equal, but between basin thirds the distances are not equal, to compensate for relative basin width of the thirds.

Figure 3-21
Dividing Urban Drainage Basins Into Thirds

Adapted from USGS, WSP 2207, 1983
a. Channel Improvements — If channel improvements such as straightening, enlarging, deepening, and clearing are prevalent for the main drainage channels and principal tributaries (those that drain directly into the main channel), then a value of 1 is assigned. Any or all of these improvements would qualify for a value of 1. To be considered prevalent, at least 50 percent of the main drainage channels and principal tributaries must be improved to some degree over natural conditions. If channel improvements are not prevalent, then a value of zero is assigned.

b. Channel Linings — If more than 50 percent of the length of the main drainage channels and principal tributaries has been lined with an impervious material, such as concrete, then a value of 1 is assigned to this aspect. If less than 50 percent of these channels is lined, then a value of zero is assigned. The presence of channel linings would obviously indicate the presence of channel improvements as well. Therefore, this is an added factor and indicates a more highly developed drainage system.

c. Storm Drains, or Storm Sewers — Storm drains are defined as enclosed drainage structures (usually pipes), frequently used on the secondary tributaries where the drainage is received directly from streets or parking lots. Many of these drains empty into open channels; however, in some basins they empty into channels enclosed as box or pipe culverts. When more than 50 percent of the secondary tributaries within a subarea (third) consists of storm drains, then a value of 1 is assigned to this aspect; if less than 50 percent of the secondary tributaries consists of storm drains, then a value of zero is assigned. It should be noted that if 50 percent or more of the main drainage channels and principal tributaries are enclosed, then the aspects of channel improvements and channel linings would also be assigned a value of 1.

d. Curb-and-Gutter Streets — If more than 50 percent of a subarea (third) is urbanized (covered by residential, commercial, and/or industrial development), and if more than 50 percent of the streets and highways in the subarea are constructed with curbs and gutters, then a value of 1 would be assigned to this aspect. Otherwise, it would receive a value of zero. Drainage from curb-and-gutter streets frequently empties into storm drains.

3.3.4.4 Three Parameter Estimating Equations

Table 3-9 lists the USGS urban regression equations for peak rates of runoff for the 2-year through the 500-year return frequency events. Required input parameters include:

- \( A \) — Drainage area, in square miles
- \( BDF \) — Basin Development Factor, dimensionless
- \( RQ \) — Equivalent rural peak discharge from Table 3-7, in cfs
- \( UQ \) — Urbanized peak discharge, in cfs
### Table 3-9

USGS Urban Peak Discharge

| UO₂ | 13.2 • A^{2.1} • (13 - BDF)^{-3.43} • RQ₂^{.73} | .91 | (3-89) |
| UO₅ | 10.6 • A^{1.7} • (13 - BDF)^{-3.9} • RQ₅^{.78} | .92 | (3-90) |
| UO₁₀ | 9.51 • A^{1.6} • (13 - BDF)^{-3.6} • RQ₁₀^{.79} | .92 | (3-91) |
| UO₂₅ | 8.68 • A^{1.5} • (13 - BDF)^{-3.4} • RQ₂₅^{.80} | .92 | (3-92) |
| UO₅₀ | 8.04 • A^{1.5} • (13 - BDF)^{-3.2} • RQ₅₀^{.81} | .91 | (3-93) |
| UO₁₀₀ | 7.70 • A^{1.5} • (13 - BDF)^{-3.2} • RQ₁₀₀^{.82} | .91 | (3-94) |
| UO₅₀₀ | 7.47 • A^{1.6} • (13 - BDF)^{-3.0} • RQ₅₀₀^{.82} | .89 | (3-95) |
3.3.4.4.1 USGS Regression Equation Example Problems

Problem No. 6

Location: West of Clayton
Design Frequency Flood: 50–year

The project is located in the Northeast Plains physiographic region, Region 1.

From Table 3–6, the 50–year design peak flow is given by Equation 3–30.

\[ Q_{50} = 1.18 \times 10^3 \cdot A^{0.48} \]

Watershed Area, \( A = 47 \) square miles

\[ Q_{50} = 1.18 \times 10^3 \times 47^{0.48} \]

\( Q_{50} = 7,490 \) cfs
Problem No. 7

Location: A developed area in Santa Fe
Elevation: \( E_c = 7,000 \text{ ft.}, \) average elevation of the stream channel
Design Frequency Flood: 50–year

USGS Physiographic Region: 6
Watershed Area: 3.5 sq. mi.
\( P_{24.10} = 2.4 \text{ inches} \)
The rural peak discharge is computed by Equation 3–65.

\[
Q_{50} = 6.11 \times 10^4 (3.5)^{0.43} \left(\frac{7,000}{1,000}\right)^{–0.38} (2.4)^{2.57} = 1383 \text{ cfs}
\]

Use \( Q_{50} = 1380 \text{ cfs} \) because the USGS recognizes only three significant figures.

Now compute the effects of urbanization.

The drainage basin is divided into thirds as described in SECTION 3.3.4.2. Within each third, the basin is evaluated for significant urban development factors, as described in SECTION 3.3.4.3. Scoring for this example watershed is as follows:

<table>
<thead>
<tr>
<th></th>
<th>Upper Third</th>
<th>Middle Third</th>
<th>Lower Third</th>
</tr>
</thead>
<tbody>
<tr>
<td>Channel Improvements</td>
<td>0</td>
<td>1</td>
<td>1</td>
</tr>
<tr>
<td>Channel Linings</td>
<td>0</td>
<td>0</td>
<td>1</td>
</tr>
<tr>
<td>Storm Drains</td>
<td>0</td>
<td>0</td>
<td>1</td>
</tr>
<tr>
<td>Curb &amp; Gutter Streets</td>
<td>0</td>
<td>1</td>
<td>1</td>
</tr>
</tbody>
</table>

The Basin Development Factor, BDF, is the sum of these development values.

\[
BDF = 6
\]

The urban design discharge by the USGS regression equation method is given by Equation 3–93, located in Table 3–7.

\[
UQ_{50} = 8.04 (3.5)^{15} (13 – 6)^{–32} (1380)^{81}
\]

\[
UQ_{50} = 1820 \text{ cfs}
\]

This is the urbanized 50–year design peak flow.
Problem No. 8

Location: Near Capitan
Elevation: \( E = 7,200 \) ft., mean basin elevation
Design Frequency Flood: 50–year

USGS Physiographic Regions: 3 and 4
Watershed Area: 90.2 sq. mi., total
  41.3 sq. mi. in physiographic region 3
  48.9 sq. mi. in physiographic region 4

Compute the 50–year design peak flow for each portion of the watershed.

For physiographic region 3, the peak flow is given by Equation 3-44.

\[
Q_{50} = 4.15 \times 10^8 \cdot A^{0.78} \cdot \left( \frac{E}{1,000} \right)^{-7.16}
\]

\[
Q_{50} = 4.15 \times 10^8 \cdot (41.3)^{0.78} \cdot \left( \frac{7,200}{1,000} \right)^{-7.16}
\]

\[
Q_{50} = 5,500 \text{ cfs}
\]

For physiographic region 4, the peak flow is given by Equation 3-51.

\[
Q_{50} = 1.04 \times 10^3 \cdot A^{0.58}
\]

\[
Q_{50} = 1.04 \times 10^3 \cdot (48.9)^{0.58}
\]

\[
Q_{50} = 9,930 \text{ cfs}
\]

The peak discharge for the entire drainage basin is weighted by drainage area.

\[
Q_{50} = 5,500 \left( \frac{41.3}{90.2} \right) + 9,930 \left( \frac{48.9}{90.2} \right)
\]

\[
Q_{50} = 7,900 \text{ cfs}
\]
3.3.5 **SCS Unit Hydrograph Method**

The SCS synthetic unit hydrograph method has been selected for use on NMSHTD projects, as defined in **SECTION 3.2**. This method should be used whenever peak flow calculations involve multiple (sub) basins. The method should also be used whenever the analysis includes flood routing through detention facilities or long conveyance facilities. Synthetic unit hydrographs can be used to model drainage basins with and without base flow.

A hydrograph is a plot of discharge versus time. Synthetic unit hydrograph methods are used to adjust the shape of a generalized hydrograph to a particular drainage basin, usually at an unaged site. A unit hydrograph is defined as the direct runoff hydrograph, resulting from a rainfall event which has a specific temporal and spatial distribution, and which generates a unit depth of rainfall. The area beneath the unit hydrograph curve is equal to the volume of direct runoff from one inch of excess rainfall over the entire drainage basin. **Figure 3–22** shows a dimensionless unit hydrograph and its associated cumulative mass curve. The SCS unit hydrograph was developed through the analysis of a large number of natural unit hydrographs from a broad cross section of geographic locations and hydrologic regions.

### 3.3.5.1 Unit Hydrograph Preparation

The SCS unit hydrograph method requires only the determination of time to peak and peak discharge to develop a synthetic unit hydrograph for a particular drainage basin. **Figure 3–23** shows the parameters used in the SCS curvilinear unit hydrograph and the equivalent triangular hydrograph.

Peak discharge of the synthetic unit hydrograph is computed with the following equation:

\[
q_p = \frac{K_p \cdot A}{T_p}
\]

(3–96)

where

- \(q_p\) = the unit hydrograph peak discharge, in cfs
- \(A\) = basin area, in square miles
- \(T_p\) = the time to peak of the hydrograph, in hours
- \(K_p\) = an empirical constant

\(K_p\) is an empirically derived constant which varies from 300 in very flat swampy areas to 600 in steep terrain. An average value of \(K_p = 484\) is used unless otherwise indicated.

Time to peak, \(T_p\), is defined as the time from the beginning of rainfall to the peak discharge. The SCS relates \(T_p\) to other hydrograph parameters with the following equations:
\[ T_p = \frac{D}{2} + T_L \]  \hspace{1cm} (3-97)

and

\[ D = 0.133 \, T_c \]  \hspace{1cm} (3-98)

where

\( T_p \) = time to peak of the hydrograph, in hours
\( D \) = duration of excess rainfall, in hours
\( T_L \) = basin lag time, time from the centroid of excess rainfall, in hours
\( T_c \) = time of concentration, in hours

These relationships are shown graphically in Figure 3-23. The SCS also relates lag time to the time of concentration by the equation:

\[ T_L = 0.6 \, T_c \]  \hspace{1cm} (3-99)

Therefore the time to peak, \( T_p \), may be computed by the equation:

\[ T_p = 0.67 \, T_c \]  \hspace{1cm} (3-100)

Time of Concentration, \( T_c \), should be computed by the appropriate method as identified in SECTION 3.3.1.4 of this manual.

The volume of runoff from the unit hydrograph is, by definition, equal to one inch rainfall depth over the entire watershed area. The runoff volume may be computed by the equation:

\[ Q_v = 53.33 \, A \, Q_d \]  \hspace{1cm} (3-101)

where

\( Q_v \) = runoff volume, in acre-ft
\( Q_d \) = average depth of runoff, in inches
\( A \) = basin area, in square miles

The unit peak discharge and time to peak determine the SCS synthetic unit hydrograph shape for a particular drainage basin. Peak discharge rates may be computed for any amount of excess rainfall. Simply multiply the \( q_p \) of the unit hydrograph by the depth of excess rainfall (in inches). This method may also be used to compute all of the flow rates for discrete time steps within the hydrograph, thereby allowing the summation of multiple hydrographs from adjoining basins. Ratios of incremental time, \( t \), to time to peak \( T_p \), and incremental flow rate, \( q \), to peak flow rate, \( q_p \), are tabulated in Table 3-10. These ratios may be used to construct a dimensional unit hydrograph for a particular basin.
Manual construction of unit hydrographs can be tedious, and designers are encouraged to use computer models capable of computing an SCS synthetic unit hydrograph. Computer models acceptable to the NMSHTD Drainage Section include:

- AHYMO: Albuquerque Metropolitan Arroyo Flood Control Authority, 1993
- SC Hydro: Akan Paine, Inc. 1993, distributed through the American Society of Civil Engineers

Note: Designers must use the proper settings within a given computer model in order for the SCS unit hydrograph procedure to be active. Consult the individual user manuals for detailed instructions.
1.0
1.1111111111111111111.1
1.1

q = DISCHARGE AT TIME t
q_p = PEAK DISCHARGE
Q_o = ACCUMULATED VOLUME AT TIME t
Q = TOTAL VOLUME
T = A SELECTED TIME
T_p = TIME FROM BEGINNING OF RISE TO THE PEAK

q / q_p OR Q_o / Q

Dimensionless Unit Hydrograph and Mass Curve for SCS Synthetic Hydrograph

Figure 3-22
Dimensionless Unit Hydrograph and Mass Curve for SCS Synthetic Hydrograph

Adapted from SCS, NEH-1, 1972
Figure 3-23
Dimensionless Curvilinear Unit Hydrograph and Equivalent Triangular Hydrograph

Adapted from SCS, NEH-1, 1972
Table 3-10

Ratios for Dimensionless Unit Hydrograph and Mass Curve, SCS Synthetic Hydrograph

<table>
<thead>
<tr>
<th>Time Ratios ( (t/T_p) )</th>
<th>Discharge Ratios ( (q/q_p) )</th>
<th>Mass Curve Ratios ( (Q_a/Q) )</th>
</tr>
</thead>
<tbody>
<tr>
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<td>1.000</td>
</tr>
</tbody>
</table>

from SCS, 1972
3.3.5.2 APPLICATION OF THE SCS UNIT HYDROGRAPH METHOD

Computer models are the preferred approach for application of the SCS synthetic unit hydrograph method. These computation methods make addition and routing of multiple hydrographs a relatively easy task. The SCS's model, TR-20, has the capability of computing runoff hydrographs from several drainage basins, adding hydrographs together, and estimating travel time for hydrographs once they have entered defined conveyance channels. Different rainfall distributions and total rainfall depths may be used in the model. Other computer models such as HEC-1 and AHYMO have options to perform the same calculations as TR-20, as well as many other features not found in TR-20. Computer programs such as SCSHYDRO perform the most common calculations found in TR-20, but have the advantage of an easier user interface.

All of these rainfall runoff computer models require the same basic input data to compute an SCS synthetic unit hydrograph, including:

- Drainage basin area
- Time of concentration or Lag time
- Runoff Curve number
- Total Rainfall depth
- Rainfall distribution

The precise input format for each of these elements varies with the computer model used. Guidelines for the use of a particular computer model are beyond the scope of this manual. Drainage designers wishing to use computer models for hydrologic analysis are encouraged to obtain licensed copies of the software. Questions on the use of a particular model can then be answered through the user manual or software support network.

There are some basic requirements for use of a hydrologic computer model on an NMSHTD project.

1. The computer model should be approved by the NMSHTD Drainage Section (See SECTION 3.3.5.1 for approved computer models).
2. Time of Concentration must be computed using the Stream Hydraulic Method, and the Upland Method as appropriate. These methods are described in SECTION 3.3.1.4 of this manual.
3. The rainfall distribution used must be the “Modified NOAA–SCS” distribution described in SECTION 3.3.1.2.3 of this manual.
4. Complete input files, routing diagrams, and summary output files only must be included (in an appendix) in every drainage report.
5. The SCS curve number methodology must be used, except in an urban area such as Albuquerque where the local drainage ordinance prescribes a different method.
6. When hydrograph routing is required, the ATT–KIN method (Modified Attenuation–Kinematic) is preferred for use with the SCS Unit Hydrograph procedure. A normal depth or Muskingum–Cunge method may be an acceptable alternative in certain situations.

3.4 Watersheds with Stream Gage Data

When a particular drainage structure is located in a watershed where a USGS streamflow gage exists, the gage data should be used in the determination of the design discharge. Streamflow data from gages other than USGS gages should not be used for design of NMSHTD projects. There are two general scenarios for the location of a streamflow gage:

a) The gage is located at the highway drainage structure.
b) The gage is located up or downstream at some distance from the highway.

The majority of the gage data in New Mexico has been collected by the USGS. For most of their streamflow gage sites, the USGS has computed flood frequency estimates. These estimates can be used directly for sizing a drainage structure, if the gage is located at the highway. The current USGS study of peak streamflows in New Mexico (Waltemeyer, 1996) includes tabulated flood frequency estimates for most USGS gage sites.

When there is a USGS gage at the highway drainage structure this data should be used for design. Waltemeyer (1996) has performed a frequency distribution analysis for most of the USGS streamflow gaging sites, and the resulting flood frequency estimates can be used directly for design. If the gage data set represents a relatively short period of record, a correction weighting procedure is recommended. The gage frequency distribution peak flood estimate is weighted according to length of record and equivalent years from the USGS regression analysis. Waltemeyer (1996) describes a procedure for improving flood frequency estimates at gaged sites, using the USGS regression equations. In the event that the USGS gage at the highway drainage structure was not included in Waltemeyer's study, then a frequency distribution analysis becomes necessary. A comprehensive discussion of frequency analysis is beyond the scope of this manual. There are several publications which describe the process in great detail (Interagency Advisory Committee on Water Data, 1982; US Army Corps of Engineers, 1993). Typically, a log Pearson Type III probability distribution is fit to the set of streamflow data. The use of a partial duration series as opposed to an annual series may depend on what data is available.

When the USGS streamflow gage is located some distance upstream or downstream of the highway, the gage site can still be used to provide a weighted flood frequency estimate. The area weighted correction procedure is described by Waltemeyer (1996). The procedure includes a drainage area ratio adjustment, which can be used when the ratio of ungaged watershed to gaged watershed area is within the limits 0.5 to 1.5.
The area weighted flood frequency estimate procedure described by Waltemeyer (1996) provides the equation:

\[ Q_{pu} = Q_{pg} \left( \frac{A_u}{A_g} \right)^x \]  

(3–102)

where

- \( Q_{pu} \) = weighted peak discharge at the ungaged site, in cfs
- \( Q_{pg} \) = peak discharge at the gaged site, in cfs
- \( A_u \) = drainage area at the ungaged site, in square miles
- \( A_g \) = drainage area at the gaged site, in square miles
- \( x \) = exponent of the drainage area term used in the appropriate equation from Table 3–7.

### 3.5 Risk and Uncertainty in Hydrologic Analysis

Drainage structures are designed to safely pass a certain magnitude flood. On most New Mexico highways the design flood will be the “50–year” design frequency flood. This design flood is theoretically equivalent to the largest flood which will occur at that location on average at least once every fifty years. By designing drainage structures to safely pass relatively rare events, the risk to users of the highway is reduced to an acceptable level.

There is always some chance, or risk, that a flood will occur which exceeds the design flood used to size a particular drainage structure. While it might be desirable to design all drainage structures to pass the largest possible flood, economic realities prevent this from happening. Instead, a level of protection must be provided which is both responsible and reasonable.

Even though the 50–year flood occurs on average at least once every 50 years, there is some small possibility that this flood could occur in any given year. And just because a 50–year flood occurred last year, does not mean that it could not occur again this year. Of course the probability that a particular location would experience back to back 50–year floods is very small, but still possible.

Consider a drainage structure capable of passing the 100–year event, and with a structural design life of 50–years. What is the probability or risk, that the structure will see a 100–year flood (or greater) during its design life? The logical answer might be 1 chance in 2, or 50%. However statistical analyses show that the risk is lower, actually the risk is 39%. Statistically the concept of risk is described by a binomial distribution (Interagency Advisory Committee on Water Data, 1982). Equation 3–103 describes this statistical relationship.
\[ R = 1 - \left( 1 - \frac{1}{T_r} \right)^m \]  
(3-103)

Where

- \( R \) = the risk of the design discharge being exceeded at least once during the design life, in percent
- \( T_r \) = the recurrence interval or frequency of the design flood, in years
- \( m \) = the design life of the structure, in years

Assuming that the structure has a design life of 100 years, then Equation 3-103 predicts that a 100-year flood has only a 63% chance of being equaled or exceeded during the structure's design life. Table 3-11 lists computed values of risk for a range of structure design lives.

Another way of looking at the concept of risk is to define an acceptable level of risk and then compute the design flood which would have to be accommodated by the drainage structure to satisfy that level of risk. Equation 3-103 can be rearranged to solve for the required return period, yielding Equation 3-104:

\[ T_r = \frac{1}{1 - \left( 1 - \frac{R}{100} \right)^{\frac{1}{m}}} \]  
(3-104)

Assume for a moment that a 5% level of risk is desirable. Or stated another way, we have a 95% confidence level that the structure is adequate. Then Equation 3-104 predicts that our structure with the design life of 50 years must be capable of passing the 975-year flood.

\[ T_r = \frac{1}{1 - \left( 1 - \frac{5}{100} \right)^{\frac{1}{50}}} = 975 \text{ years} \]

It becomes apparent that risk cannot be completely eliminated, just reduced to a level acceptable to society. And even if there were unlimited funds to build drainage structures, the ability to accurately calculate the magnitude of flood events decreases as the design flood magnitude increases. All of the current flood prediction methods, whether analytical or parametric, are based on observed flood flows from watersheds with measured response characteristics, and occasionally rain gage data. The effective period of recorded data in New Mexico reaches 100 years in only a few locations. Thus, the prediction of a 975-year flood is done by extrapolating the data, since the desired flood has only a small chance of being included in the data set. The uncertainty in predicted flood flows increases as the return period lengthens.
The accuracy of predicted flood magnitudes up to the 100-year event is certainly much better. For the analytic methods presented in this manual, risk takes the form of uncertainty in the input parameters. Drainage area can be measured by several individuals and the answers should all be within two or three percent. Use of a consistent method to compute time of concentration reduces variability in the estimation of $T_c$. However, the selection of a Rational Formula “C” or an SCS runoff curve number “CN” involves considerable judgment. Even meticulous measurement of watershed areas, land uses, and hydrologic soil groups may not pinpoint the response of the watershed. There is some inherent variability of the data, and of its interpretation, leading to uncertainty in the selection of the correct runoff coefficient or curve number. This uncertainty cannot be quantified, and thus becomes part of the overall risk in predicting peak flood magnitudes.

With the analytic methods in this manual, one approach to qualitatively assess the risk is to perform a sensitivity analysis. This is done by varying a particular input parameter across its range of reasonable values and comparing the resulting range of predicted peak flows. The most sensitive analytic parameter will probably be the curve number CN (or rational formula C). Use the CN value obtained by normal design methods to compute a peak flow, as well as the lowest and highest CN values which the designer believes could occur in the watershed. Now there are three computed peak flow values which estimate the range of most probable peak flood flows. This is not a precise computed range of risk, but it does help to bracket the most likely peak flow value. The central peak flood flow value will often be used to size the structure, while the upper limit peak flood flow can be used to assess the “worst case” headwater or overtopping condition. For heavy traffic roadways, the designer may wish to evaluate the increase in structure size necessary to pass the upper limit peak flood flow.

The USGS parametric (regression) methods included in this drainage manual are statistically derived, and include a measure of the risk associated with their use. Each equation has an associated Average Standard Error of Prediction shown next to each equation in the text. The average standard error of prediction provides a measure of the overall uncertainty of a specific equation’s prediction. Figure 3–24 graphically portrays a data set, the associated regression line (equation) and the average standard error of prediction.
Assume a hypothetical watershed, where we calculate $Q_{50} = 720$ cfs using Equation 3–58. The standard error of prediction is given as 0.298 for Equation 3–58. There is a 67% chance that the true 50-year peak flow will lie within the range of plus or minus one average standard error of prediction.

$$Q_{low_{67}} = 10 \left[ \log(720) - 0.298 \right] = 363 \text{ cfs}$$

$$Q_{high_{67}} = 10 \left[ \log(720) + 0.298 \right] = 1430 \text{ cfs}$$

The 95% range of the average standard error of prediction is computed using two standard errors. The 95% confidence range of predicted peak flows is calculated as:

$$Q_{low_{95}} = 10 \left[ \log(720) - 2(0.298) \right] = 183 \text{ cfs}$$

$$Q_{high_{95}} = 10 \left[ \log(720) + 2(0.298) \right] = 2840 \text{ cfs}$$

Within each physiographic region of New Mexico the watersheds used in the USGS regression analysis have a lot of variability. Nonetheless, the regression equations represent observed flood flows, not theoretical flows predicted by analytic means. Use of the USGS regression equations and their respective average standard error of predictions can be a very powerful tool for quantifying the risk associated with hydrologic analysis.
Figure 3-24
Regression Line and Average Standard Error of Prediction Range
Table 3–11
Tabulation of Risk
of at Least One Exceedance During the Design Life

<table>
<thead>
<tr>
<th>Recurrence Interval</th>
<th>2</th>
<th>5</th>
<th>10</th>
<th>25</th>
<th>50</th>
<th>100</th>
</tr>
</thead>
<tbody>
<tr>
<td>2</td>
<td>0.750</td>
<td>0.970</td>
<td>≈ 1.00</td>
<td>≈ 1.00</td>
<td>≈ 1.00</td>
<td>≈ 1.00</td>
</tr>
<tr>
<td>5</td>
<td>0.360</td>
<td>0.670</td>
<td>0.89</td>
<td>≈ 1.00</td>
<td>≈ 1.00</td>
<td>≈ 1.00</td>
</tr>
<tr>
<td>10</td>
<td>0.190</td>
<td>0.410</td>
<td>0.65</td>
<td>0.93</td>
<td>0.99</td>
<td>≈ 1.00</td>
</tr>
<tr>
<td>25</td>
<td>0.080</td>
<td>0.180</td>
<td>0.34</td>
<td>0.64</td>
<td>0.87</td>
<td>0.98</td>
</tr>
<tr>
<td>50</td>
<td>0.040</td>
<td>0.100</td>
<td>0.18</td>
<td>0.40</td>
<td>0.64</td>
<td>0.87</td>
</tr>
<tr>
<td>100</td>
<td>0.020</td>
<td>0.050</td>
<td>0.10</td>
<td>0.22</td>
<td>0.39</td>
<td>0.63</td>
</tr>
<tr>
<td>500</td>
<td>0.004</td>
<td>0.010</td>
<td>0.02</td>
<td>0.05</td>
<td>0.10</td>
<td>0.18</td>
</tr>
<tr>
<td>1,000</td>
<td>0.002</td>
<td>0.005</td>
<td>0.01</td>
<td>0.02</td>
<td>0.05</td>
<td>0.10</td>
</tr>
</tbody>
</table>
APPENDICES
APPENDIX A — PHOTOGRAPHIC GUIDE TO HYDROLOGIC CONDITIONS
PHOTOGRAPHIC GUIDE TO HYDROLOGIC CONDITIONS

General Notes

1. These photographs are intended as an aide in selecting an appropriate Curve Number.

2. Examples of different cover type and hydrologic condition are provided, along with estimates of percent of cover.

3. Hydrologic soil group values and curve numbers listed below each photograph are for general information purposes only.

4. Actual hydrologic soil group values should be determined from field inspections and Soil Conservation Service soil surveys.

5. Actual curve numbers will vary from one project to the next even though the cover type and hydrologic condition may appear similar to a particular photograph.

6. Cover type and density will vary from one slope aspect to another (i.e., north vs. south). For a particular slope aspect, cover density will vary seasonally throughout the year.

Note: White measuring stick seen in some photos is 6 ft. in length.
Photo No. **1A, General View**

Location: **Near Santa Fe, Santa Fe County**

Aspect: **South Facing**  

Photo Date: **August 1995**

Cover Description: **Piñon and Juniper with Bunch Grass**

Hydrologic Condition: **Poor**  

HSG: **B**  

Percent Cover: **30**

Soil Series: **Pojoaque**  

Soil Description: **Gravely Sand**

Estimated Runoff Curve Number: **75**

---

Photo No. **1B, Ground View**
Photo No. 2A, General View
Location: Northwest Albuquerque, Bernalillo County
Aspect: East Facing     Photo Date: June, 1995
Cover Description: Sagebrush, Saltbush, Weeds, Bunch Grasses
Hydrologic Condition: Fair     HSG: A     Percent Cover: 40%
Soil Series: Bluepoint     Soil Description: Sandy
Estimated Runoff Curve Number: 65
Photo No. 3A, General View
Location: Near Hondo, Lincoln County
Aspect: South Facing   Photo Date: June, 1995
Cover Description: Juniper and Bunch Grasses
Hydrologic Condition: Fair   HSG: D   Percent Cover: 40%
Soil Series: Deama   Soil Description: Rocky
Estimated Runoff Curve Number: 80

Photo No. 3B, Ground View
Photo No. 4A, General View
Location: Near Radium Springs, Doña Ana County
Aspect: Gently Sloping West  Photo Date: June, 1995
Cover Description: Desert Brush, Yucca, Cactus, Bunch Grasses
Hydrologic Condition: Fair  HSG: A  Percent Cover: 30%
Soil Series: Bluepoint  Soil Description: Sandy Loam
Estimated Runoff Curve Number: 60

Photo No. 4B, Ground View
Photo No. 5A, General View
Location: West of San Marcial, Socorro County
Aspect: Southeast Sloping
Photo Date: June, 1995
Cover Description: Desert Shrub, Mesquite
Hydrologic Condition: Poor
Percent Cover: 10%
Soil Series: Nickel
Soil Description: Gravely
Estimated Runoff Curve Number: 85

Photo No. 5B, Ground View
Photo No. 6A, General View
Location: Near Alamogordo, Otero County
Aspect: West Sloping     Photo Date: June, 1995
Cover Description: Desert Brush, Cactus, Bunch Grasses
Hydrologic Condition: Fair     HSG: B     Percent Cover: 40%
Soil Series: Nickel     Soil Description: Sand & Gravel
Estimated Runoff Curve Number: 74

Photo No. 6B, Ground View
Photo No. 7A, General View
Location: South of Taos, Taos County
Aspect: West Sloping    Photo Date: August 1995
Cover Description: Sagebrush and Pinon
Hydrologic Condition: Fair    HSG: C    Percent Cover: 50%
Soil Series: not available    Soil Description: Sandy Clay
Estimated Runoff Curve Number: 67

Photo No. 7B, Ground View
Photo No. 8A, General View
Location: North of Taos, Taos County
Aspect: Level Photo Date: August 1995
Cover Description: Sagebrush and Bunch Grasses
Hydrologic Condition: Fair HSG: C Percent Cover: 60%
Soil Series: not available Soil Description: Silty Clay
Estimated Runoff Curve Number: 63

Photo No. 8B, Ground View
Photo No. 9A, General View
Location: South of Espanola, Rio Arriba County
Aspect: West Facing   Photo Date: August 1995
Cover Description: Rangeland
Hydrologic Condition: Poor   HSG: A   Percent Cover: 20%
Soil Series: not available   Soil Description: Silty Sand
Estimated Runoff Curve Number: 78

Photo No. 9B, Ground View
Photo No. 10A, General View
Location: North Central Mountains Near Taos, Taos County
Aspect: South Sloping  Photo Date: August 1995
Cover Description: Pine Forest
Hydrologic Condition: Good  HSG: C  Percent Cover: 70%
Soil Series: not available  Soil Description: Thin Gravely Topsoil
Estimated Runoff Curve Number: 70

Photo No. 10B, Ground View
Photo No. 11A, General View
Location: Near Cloudcroft, Otero County
Aspect: West Facing       Photo Date: June, 1995
Cover Description: Pine Forest with Partial Brush Understory
Hydrologic Condition: Good       HSG: D       Percent Cover: 80%
Soil Series: Not Mapped       Soil Description: Rocky
Estimated Runoff Curve Number: 77

Photo No. 11B, Ground View
Photo No. 12
Location: North of Ragland, Quay County
Aspect: West Facing  Photo Date: June, 1995
Cover Description: Juniper, Shrubs and Grasses, Rangeland
Hydrologic Condition: Fair  HSG: B  Percent Cover: 60%
Soil Series: Quay Loam  Soil Description: Silty Loam
Estimated Runoff Curve Number: 72

Photo No. 13
Location: Near Grady, Curry County
Aspect: Level  Photo Date: June, 1995
Cover Description: Grassed Rangeland, Heavily Grazed
Hydrologic Condition: Fair  HSG: C  Percent Cover: 75%
Soil Series: Potter  Soil Description: Clay Loam
Estimated Runoff Curve Number: 79
Photo No. 14
Location: South of Broadview, Curry County
Aspect: Nearly Level       Photo Date: June, 1995
Cover Description: Mixed Prairie Grass, Some Weeds
Hydrologic Condition: Good       HSG:  C       Percent Cover: 100%
Soil Series: Potter       Soil Description: Clay Loam
Estimated Runoff Curve Number: 71

Photo No. 15
Location: North of Clovis, Curry County
Aspect: Nearly Level       Photo Date: June, 1995
Cover Description: Dense Prairie Grass
Hydrologic Condition: Good       HSG:  C       Percent Cover: 100%
Soil Series: Potter       Soil Description: Clay Loam
Estimated Runoff Curve Number: 66
Photo No. 16
Location: North of Clovis, Curry County
Aspect: Nearly Level  Photo Date: June, 1995
Cover Description: Agricultural, Tilled, Bare Soil
Hydrologic Condition: Poor  HSG: B  Percent Cover: 0%
Soil Series: Amarillo  Soil Description: Clayey Loam
Estimated Runoff Curve Number: 85

Photo No. 17
Location: Near Doña Ana, Doña Ana County
Aspect: Nearly Level  Photo Date: June, 1995
Cover Description: Row Crops, Young Chili
Hydrologic Condition: Poor  HSG: B  Percent Cover: 10%
Soil Series: Harkey  Soil Description: Silty Loam
Estimated Runoff Curve Number: 79
Photo No. 18
Location: North of Clovis, Curry County
Aspect: Nearly Level     Photo Date: June, 1995
Cover Description: Row Crop, Young Corn
Hydrologic Condition: Fair     HSG: B     Percent Cover: 30%
Soil Series: Amarillo     Soil Description: Clayey Loam
Estimated Runoff Curve Number: 78
APPENDIX B — GLOSSARY OF TERMS
GLOSSARY

Absorption  The act or process of taking in water by inflow of atmospheric vapor, hygroscopic absorption, wetting, infiltration, influent seepage, and gravity flow of streams into sinkholes or other large openings.

Abstraction  That portion of rainfall which does not become runoff. It includes interception, infiltration, and storage in depression. It is affected by land use, land treatment and condition, and antecedent soil moisture.

Accretion  ① A process of accumulation by flowing water whether of silt, sand, pebbles, etc. Accretion may be due to any cause and includes alluviation. ② The gradual building up of a beach by wave action. ③ The gradual building of the channel bottom, bank, or bar due to silting or wave action.

Acre–Foot  The amount of water that will cover 1 acre to a depth of 1 foot. It equals 43,560 cubic feet. Abbreviated ac–ft.

Aggradation  General and progressive upbuilding of the longitudinal profile of a channel by deposition of sediment.

Allowable Headwater  The depth or elevation of impounded water at the entrance to a hydraulic structure, above which flooding or some other unfavorable result could occur.

Alluvial Channel  A channel wholly in alluvium with no bedrock exposed in the channel at low flow or likely to be exposed by erosion during major flow.

Alluvium  Unconsolidated clay, silt, sand, or gravel deposited by a stream in a channel, flood plain, delta, alluvial fan, or pediment.

Annual Flood  The highest peak discharge in a water year.

Annual Series  A frequency series in which only the largest value in each year is used, such as annual floods.

Annual Yield  The total amount of water obtained in a year from a stream, spring, artesian well, etc. Usually expressed in inches depth, acre–feet, millions of gallons, or cubic feet.

Antecedent Moisture Condition (AMC)  The amount of moisture in the soil and plants at the beginning of the storm. Antecedent moisture conditions affect the volume of runoff generated by a particular storm event. As storm magnitudes increase, antecedent moisture has a rapidly decreasing influence on runoff because the soils become saturated.
Area Rainfall  The average rainfall over an area, usually as derived from, or discussed in contrast with point rainfall.

Bank  Lateral boundaries of a channel or stream, as indicated by a scarp, or on the inside of bends, by the streamward edge of permanent vegetal growth.

Base Flow  Stream discharge derived from groundwater sources. Sometimes considered to include flows from regulated lakes or reservoirs. Fluctuates much less than storm runoff.

Basin, Drainage  The area of land drained by a watercourse.

Basin Lag  The amount of time from the centroid of the rainfall hyetograph to the hydrograph peak.

Bed (of a channel or stream)  The part of a channel without permanent vegetation, bounded by banks, over which water normally flows.

Berm  A narrow shelf or ledge; also a form of dike.

Bridge  A structure including supports erected over a depression or an obstruction, such as a watercourse, highway, or railway, and having a tract or passageway for carrying traffic or moving loads. A bridge has an opening measured along the center of the roadway of more than 20 feet between undercopings of abutments or spring lines of arches, or extreme ends of openings for multiple boxes. A bridge may also include multiple pipes, where the clear distance between openings is less than half of the smaller contiguous opening. A bridge is designed hydraulically using the principles of open channel flow to operate with a free water surface, but may be inundated under flood conditions.

Bridge Opening  The cross-sectional area beneath a bridge that is available for conveyance of water.

Capacity  A measure of the ability of a channel or conduit to convey water.

Catch Basin  A structure with a sump for inletting drainage from a gutter or median and discharging the water through a conduit. In common usage it is a grated inlet with or without a sump.

Catchment  The watershed. (Implying all physical characteristics.)

Catchment Area  The area tributary to a lake, stream, or drainage system.
CFS  Abbreviation for cubic feet per second. A unit of water flow. Sometimes called “second feet.”

CMS  Abbreviation for cubic meters per second. A unit of water flow.

Channel  ① The bed and banks that confine the surface flow of a natural or artificial stream. Braided streams have multiple subordinate channels which are within the main stream channel. Anabranch streams have more than one channel. ② The course where a stream of water runs, or the closed course or conduit through which water runs, such as a pipe.

Channel Routing  The process whereby a peak flow and/or its associated streamflow hydrograph is mathematically transposed to another site downstream.

Check Dam  A low structure, dam or weir, across a channel for the control of water stage or velocity, or to control channel erosion.

Control Section  A cross section, such as a bridge crossing, reach of channel, or dam, with limited flow capacity, in which the discharge is related to the upstream water—surface elevation.

Conveyance  A measure, K, of the ability of a stream, channel, or conduit to convey water. In Manning’s formula, \( K = \frac{1.49}{n} A R^{2.5} \).

Cover  The depth of soil above the crown of a pipe or culvert. The vegetation, or vegetational debris such as mulch that exists on the soil surface. The percent of ground cover and cover types are fundamental parameters for determining runoff curve numbers.

Criterion  A standard, rule, or test on which a judgement can be based.

Cross Drainage  The runoff from contributing drainage areas both inside and outside the highway right—of—way and the transmission thereof from the upstream side of the highway facility to the downstream side.

Cross Section  The shape of a channel, stream, or valley, viewed across its axis. In watershed investigations it is determined by a line approximately perpendicular to the main path of water flow, along which measurements of distance and elevation are taken to define the cross—sectional area.

Culvert  A structure which is sometimes designed hydraulically to take advantage of submergence to increase hydraulic capacity. A structure used to convey surface runoff through embankments. A structure, as distinguished from bridges, which is usually covered with embankment and is composed of structural material around the entire perimeter, although some are supported on spread footings with the streambed serving as the bottom of the culvert. Also,
a structure which is 20 feet or less in centerline length between extreme ends of openings for multiple boxes.

**Debris**  
Material transported by the stream, either floating or submerged, such as logs or brush.

**Degradation**  
General and progressive lowering of the longitudinal profile of a channel by erosion.

**Deposition**  
The setting of material from the stream flow onto the bottom.

**Depression Storage**  
The natural depressions within a watershed that store runoff. Generally after the depression storage is filled runoff will commence.

**Design Discharge or Flow**  
The rate of flow for which a facility is designed.

**Design Flood**  
A flood that does not overtop the roadway.

**Design Flood Frequency**  
The recurrence interval that is expected to be accommodated without contravention of the adopted design constraints. The return interval (recurrence interval or reciprocal of probability) used as a basis for the design discharge.

**Design Storm**  
A given rainfall amount, areal distribution, and time distribution, used to estimate runoff. The rainfall amount is either a given frequency (25-year, 50-year, etc.) or a specific large value.

**Detention Basin**  
A basin or reservoir incorporated into the watershed whereby runoff is temporarily stored, thus attenuating the peak of the runoff hydrograph.

**Detour**  
A temporary change in the roadway alignment. It may be localized at a structure or may be along an alternate route.

**Direct Runoff**  
The water that enters the stream channels during a storm or soon after, forming a runoff hydrograph. May consist of rainfall on the stream surface, surface runoff, and seepage of infiltrated water (rapid subsurface flow).

**Discharge**  
The rate of the volume of flow of a stream per unit of time, usually expressed in cfs.

**Drainage Area**  
The area draining into a stream at a given point. The area may be of different sizes for surface runoff, subsurface flow, and base flow, but generally the surface flow area is used as the drainage area.
<table>
<thead>
<tr>
<th>Term</th>
<th>Definition</th>
</tr>
</thead>
<tbody>
<tr>
<td>Drainage Structure</td>
<td>A conduit for conveying storm water away from or under a roadway. Often a closed conduit such as a culvert, but could also be a bridge.</td>
</tr>
<tr>
<td>Effective Duration</td>
<td>The time in a storm during which the water supply for direct runoff is produced. Also used to mean the duration of excess rainfall.</td>
</tr>
<tr>
<td>Ephemeral Stream</td>
<td>A stream or reach of a stream that does not flow continuously for most of the year.</td>
</tr>
<tr>
<td>Erosion</td>
<td>The wearing away or scouring of material in a channel, opening, or outlet works caused by flowing water.</td>
</tr>
<tr>
<td>Evapotranspiration</td>
<td>Plant transpiration plus evaporation from the soil. Difficult to determine separately, therefore used as a unit for study.</td>
</tr>
<tr>
<td>Excess Rainfall</td>
<td>Direct runoff.</td>
</tr>
<tr>
<td>Exfiltration</td>
<td>The process by which stormwater leaks or flows to the surrounding soil through openings in a conduit.</td>
</tr>
<tr>
<td>Flood</td>
<td>In common usage, an event that overflows the normal banks. In technical usage, it refers to a given discharge based, typically, on a statistical analysis of an annual series of events.</td>
</tr>
<tr>
<td>Flood Frequency</td>
<td>The average time interval, in years, in which a given storm or amount of water in a stream will be exceeded.</td>
</tr>
<tr>
<td>Flood of Record</td>
<td>Reference to the maximum estimated or measured discharge that has occurred at a site.</td>
</tr>
<tr>
<td>Floodplain</td>
<td>The alluvial land bordering a stream, formed by stream processes, that is subject to inundation by floods.</td>
</tr>
<tr>
<td>Flood Routing</td>
<td>Determining the changes in a flood hydrograph as it moves downstream through a channel or through a reservoir (called reservoir routing). Graphic or numerical methods are used.</td>
</tr>
<tr>
<td>Flow Distribution</td>
<td>The estimated or measured spatial distribution of the total streamflow.</td>
</tr>
<tr>
<td>Freeboard</td>
<td>The vertical distance between the level of the water surface, usually corresponding to design flow and a point of interest such as a low chord of a bridge beam or specific location on the roadway grade.</td>
</tr>
<tr>
<td>Term</td>
<td>Definition</td>
</tr>
<tr>
<td>------</td>
<td>------------</td>
</tr>
<tr>
<td><strong>Frequency</strong></td>
<td>In analysis of hydrologic data, the recurrence interval is simply called frequency.</td>
</tr>
<tr>
<td><strong>Groundwater</strong></td>
<td>Subsurface water occupying the saturation zone, from which wells and springs are fed. A source of base flow in streams. In a strict sense the term applies only to water below the water table. Also called phreatic water.</td>
</tr>
<tr>
<td><strong>Guide Banks</strong></td>
<td>Embankments built upstream from one or both abutments of a bridge to guide the approaching flow through the waterway opening.</td>
</tr>
<tr>
<td><strong>Gutter</strong></td>
<td>That portion of the roadway section adjacent to the curb which is utilized to convey storm runoff water.</td>
</tr>
<tr>
<td><strong>Headwater</strong></td>
<td>That depth of water impounded upstream of a culvert due to the influence of the culvert constriction, friction, and configuration.</td>
</tr>
<tr>
<td><strong>Highwater Elevation</strong></td>
<td>The water surface elevation that results from the passage of flow. It may be &quot;observed highwater elevation&quot; as a result of an event, or &quot;calculated highwater elevation&quot; as part of a design process.</td>
</tr>
<tr>
<td><strong>Historical Flood</strong></td>
<td>A past flood event of known or estimated magnitude.</td>
</tr>
<tr>
<td><strong>Hydraulic Grade Line</strong></td>
<td>A profile of the piezometric level to which the water would rise in piezometer tubes along a pipe run. In open channel flow, it is the water surface.</td>
</tr>
<tr>
<td><strong>Hydrograph</strong></td>
<td>A graph showing, for a given point on a stream or for a given point in any drainage system, the discharge, stage, velocity or other property of water with respect to time.</td>
</tr>
<tr>
<td><strong>Hydrologic Soil—Cover Complex</strong></td>
<td>A combination of a hydrologic soil group and a type of cover.</td>
</tr>
<tr>
<td><strong>Hydrologic Soil Group</strong></td>
<td>A group of soils having the same runoff potential under similar storm and cover conditions.</td>
</tr>
<tr>
<td><strong>Hydrology</strong></td>
<td>The study of the occurrence, circulation, distribution, and properties of the waters of the earth and its atmosphere.</td>
</tr>
<tr>
<td><strong>Hyetograph</strong></td>
<td>A graphical representation of average rainfall, rainfall-excess rates or volumes over specified areas during successive units of time during a storm.</td>
</tr>
<tr>
<td><strong>Impervious</strong></td>
<td>Impermeable to the movement of water.</td>
</tr>
<tr>
<td>Term</td>
<td>Definition</td>
</tr>
<tr>
<td>----------------------</td>
<td>-------------------------------------------------------------------------------------------------------------------------------------------</td>
</tr>
<tr>
<td><strong>Infiltration</strong></td>
<td>That part of the rainfall that enters the soil. The passage of water through the soil surface into the ground. Used interchangeably herein with the word: percolation.</td>
</tr>
<tr>
<td><strong>Infiltration Rate</strong></td>
<td>The rate at which water enters the soil under a given condition. The rate is usually expressed in inches or centimeters per hour, or feet per day.</td>
</tr>
<tr>
<td><strong>Initial Abstraction (Ia)</strong></td>
<td>When considering surface runoff, Ia is all the rainfall before runoff begins. If considering direct runoff, Ia consists of interception, evaporation, and the soil-water storage that must be exhausted before direct runoff may begin. Sometimes called “initial loss.”</td>
</tr>
<tr>
<td><strong>Intensity</strong></td>
<td>The rate of rainfall upon a watershed, usually expressed in inches per hour.</td>
</tr>
<tr>
<td><strong>Interception</strong></td>
<td>Precipitation retained on plant or plant residue surfaces and finally absorbed, evaporated, or sublimated. That which flows down the plant to the ground is called “stemflow” and not counted as true interception.</td>
</tr>
<tr>
<td><strong>Isohyet</strong></td>
<td>A line on a map, connecting points of equal rainfall amounts.</td>
</tr>
<tr>
<td><strong>Lag Time, Tc</strong></td>
<td>The difference in time between the centroid of the excess rainfall (that rainfall producing runoff) and the peak of the runoff hydrograph. Often estimated as 60 percent of the time of concentration (Tc = 0.6Tc).</td>
</tr>
<tr>
<td><strong>Land Use</strong></td>
<td>A land classification. Cover, such as row crops or pasture, indicates a kind of land use. Roads may also be classified as a separate land use.</td>
</tr>
<tr>
<td><strong>Length</strong></td>
<td>A certain distance within a watershed or along a water course. For Time of Concentration computation, length is defined as the distance from the drainage divide to the point of interest, following primary flow paths.</td>
</tr>
<tr>
<td><strong>Levee</strong></td>
<td>A linear embankment outside a channel for containment of flow.</td>
</tr>
<tr>
<td><strong>Major Structure</strong></td>
<td>A drainage conduit which is larger than a minor structure, yet smaller than a bridge.</td>
</tr>
<tr>
<td><strong>Manning's “n”</strong></td>
<td>A coefficient of roughness, used in a formula for estimating the capacity of a channel to convey water. Generally, “n” values are determined by inspection of the channel.</td>
</tr>
<tr>
<td><strong>Mass Inflow Curve</strong></td>
<td>A graph showing the total cumulative volume of stormwater runoff plotted against time for a given drainage area.</td>
</tr>
<tr>
<td><strong>Minor Structure</strong></td>
<td>A drainage conduit which is equal to or greater than a 48” (1.6 M) circular pipe culvert, or equivalent hydraulic capacity.</td>
</tr>
</tbody>
</table>
Overland Flow
Runoff which makes its way to the watershed outlet without concentrating in gullies and streams (often in the form of sheet flow).

Onsite Drainage
Runoff from within the highway right-of-way, including pavement, medians, and road shoulders. Runoff may be collected by gutters, catch basins, swales, and rundowns.

Partial-Duration Series
A list of all events, such as floods, occurring above a selected base, without regard to the number, within a given period. In the case of floods, the selected base is usually equal to the smallest annual flood, in order to include at least one flood in each year.

Peak Discharge
Maximum discharge rate on a runoff hydrograph.

Permeability
The property of a material that permits appreciable movement of water through it when it is saturated and movement is actuated by hydrostatic pressure of the magnitude normally encountered in natural subsurface water.

Perennial Stream
A stream or reach of a stream that flows continuously for all or most of the year.

Point Rainfall
Rainfall at a single rain gage or location.

Precipitation
The process by which water in liquid or solid state falls from the atmosphere.

Rainfall Excess
The water available to runoff after interception, depression storage, and infiltration have been satisfied.

Rainfall Intensity
Amount of rainfall occurring in a unit of time, converted to its equivalent in inches per hour at the same rate.

Rating Curve
A graphical plot relating stage to discharge.

Recession Curve
The receding portion of a hydrograph, occurring after excess rainfall has stopped.

Recharge Basin
A basin excavated in the earth to receive the discharge from streams or storm drains for the purpose of replenishing groundwater supply.

Regional Analysis
A regional study of gaged watersheds which produces regression equations relating various watershed and climatological parameters to discharge. Use for design of ungaged watershed with similar characteristics.
Regulatory Flood
 Usually the 100–year flood, which was adopted by the Federal Emergency Management Agency (FEMA), as the base flood for floodplain management purposes.

Regulatory Floodway
 The floodplain area that is reserved in an open manner by Federal, State, or local requirements, i.e., unconfined or unobstructed either horizontally or vertically, to provide for the discharge of the base flood so that the cumulative increase in water surface elevation is no more than a designated amount.

Reservoir Routing
 Flood routing of a hydrograph through a reservoir.

Retention Basin
 A basin or reservoir wherein water is stored for regulating a flood. It does not have an uncontrolled outlet. The stored water is disposed by a means such as infiltration, injection (or dry) wells, or by release to the downstream drainage system after the storm event. The release may be through a gate–controlled gravity system or by pumping.

Runoff
 That part of the precipitation which runs off the surface of a drainage area after all abstractions are accounted for.

Runoff Coefficient
 A factor representing the portion of runoff resulting from a unit rainfall. Dependent on terrain and topography.

Rural
 Where the majority of the watershed area is agricultural or lightly used by human activity.

Saturated Soil
 Soil that has its interstices or void spaces filled with water to the point at which runoff occurs.

Sedimentation
 The process involving the deposition of soil particles which have been carried by flood waters.

Skewness
 When data are plotted in a curve on log–normal paper, the curvature is skewness.

Soil Porosity
 The percentage of the soil (or rock) volume that is not occupied by solid particles, including all pore space filled with air and water.

Soil–Water Storage
 The amount of water the soils (including geologic formations) of a watershed will store at a given time. Amounts vary from watershed to watershed. The amount for a given watershed is continually varying as rainfall or evapotranspiration takes place.

Stage
 Height of water surface above a specified datum.
**Storm Duration**  
The period or length of storm.

**Stream Channel**  
The bottom of a topographic area, where runoff collects and flows in a concentrated manner. May be ephemeral or perennial.

**Stream Reach**  
A length of stream channel selected for use in hydraulic or other computations.

**Surface Runoff**  
Total rainfall minus interception, evaporation, infiltration, and surface storage which moves across the ground surface to a stream or depression.

**Surface Storage**  
Stormwater that is contained in surface depressions or basins.

**Surface Water**  
Water appearing on the surface in a diffused state, with no permanent source of supply or regular course, as distinguished from water appearing in water courses, lakes, or ponds.

**Synthetic Hydrograph**  
A hydrograph determined from empirical rules. Usually based on the physical characteristics of the basin. A graph developed for an ungaged drainage area, based on known physical characteristics of the watershed basin.

**Swale**  
A slight depression in the ground surface where water collects.

**Time of Concentration, T\textsubscript{c}**  
The time it takes surface runoff water to travel from the hydraulically most distant point to the watershed outlet. T\textsubscript{c} varies with flood frequency, but is often used as a watershed constant.

**Trainer Dikes**  
Low embankments constructed to guide flows toward a drainage structure opening, often with erosion control revetments.

**Travel Time**  
The average time for water to flow through a reach or other stream or valley length.

**Tributaries**  
Branches of the watershed stream system.

**Ungaged Stream Sites**  
Locations at which no systematic records are available regarding actual streamflows.

**Uniform Flow**  
Flow of constant cross section and average velocity through a reach of channel during an interval of time.

**Unit Hydrograph**  
A hydrograph of a direct runoff resulting from 1 unit of effective rainfall (inch, centimeter) generated uniformly over the watershed area during a specified period of time or duration.
**Urban**  A development condition within a watershed where large, contiguous areas have a significant quantity of impervious surfaces constructed by man. Runoff from an urban area is typically concentrated in roads or defined drainage channels.

**Watercourse**  A channel in which a flow of water occurs, either continuously or intermittently, with some degree of regularity.

**Watershed**  The catchment area for rainfall which is delineated as the drainage area producing runoff. Usually it is assumed that base flow in a stream also comes from the same area.

**Water Table**  The upper surface of the zone of saturation, except where that surface is formed by an impermeable body (perched water table).
APPENDIX C — Metric Equations and Conversion Factors
# Table C-1

**Metric – U.S. Standard Conversion Factors**

<table>
<thead>
<tr>
<th>From Unit</th>
<th>Abbrev.</th>
<th>Multiply by</th>
<th>To obtain Unit</th>
<th>Abbrev.</th>
</tr>
</thead>
<tbody>
<tr>
<td>cubic foot per second</td>
<td>cfs</td>
<td>0.02832</td>
<td>cubic meter per second</td>
<td>m³/s</td>
</tr>
<tr>
<td>foot</td>
<td>ft</td>
<td>0.3048</td>
<td>meter</td>
<td>m</td>
</tr>
<tr>
<td>foot squared</td>
<td>ft²</td>
<td>0.0929</td>
<td>meter squared</td>
<td>m²</td>
</tr>
<tr>
<td>foot cubed</td>
<td>ft³</td>
<td>0.0283</td>
<td>meter cubed</td>
<td>m³</td>
</tr>
<tr>
<td>foot per mile</td>
<td>ft/mi</td>
<td>0.189</td>
<td>meter per kilometer</td>
<td>m/km</td>
</tr>
<tr>
<td>inch</td>
<td>in</td>
<td>25.4</td>
<td>millimeter</td>
<td>mm</td>
</tr>
<tr>
<td>square mile</td>
<td>mi²</td>
<td>2.59</td>
<td>square kilometer</td>
<td>km²</td>
</tr>
<tr>
<td>acre</td>
<td>ac</td>
<td>0.4047</td>
<td>hectare</td>
<td>ha</td>
</tr>
<tr>
<td>foot per second</td>
<td>fps</td>
<td>0.3048</td>
<td>meter per second</td>
<td>m/s</td>
</tr>
<tr>
<td>acre-feet</td>
<td>ac-ft</td>
<td>1232.748</td>
<td>meters cubed</td>
<td>m³</td>
</tr>
</tbody>
</table>
**METRIC EQUATIONS**

NOAA Rainfall Depth Duration Frequency Equations

**Note:** Precipitation depths obtained from the NOAA Atlas (or from **APPENDIX E**) must be converted to millimeters for use of these equations.

In **Region 1**

\[
P_{2-yr, 1-hr} = 5.537 + 0.709 \frac{P_{2-yr, 6-hr}}{P_{2-yr, 24-hr}}
\]

\[
P_{100-yr, 1-hr} = 48.184 + 0.439 \frac{P_{100-yr, 6-hr}}{P_{100-yr, 24-hr}} - 0.00667 Z
\]

where \(Z\) is the average project elevation, in meters.

In **Region 2**

\[
P_{2-yr, 1-hr} = -0.279 + 0.942 \frac{P_{2-yr, 6-hr}}{P_{2-yr, 24-hr}}
\]

\[
P_{100-yr, 1-hr} = 12.548 + 0.755 \frac{P_{2-yr, 6-hr}}{P_{2-yr, 24-hr}}
\]

\[
P_{5-yr, 1-hr} = 0.770 P_{2-yr, 1-hr} + 0.230 P_{100-yr, 1-hr}
\]

\[
P_{10-yr, 1-hr} = 0.609 P_{2-yr, 1-hr} + 0.391 P_{100-yr, 1-hr}
\]

\[
P_{25-yr, 1-hr} = 0.425 P_{2-yr, 1-hr} + 0.575 P_{100-yr, 1-hr}
\]

\[
P_{50-yr, 1-hr} = 0.241 P_{2-yr, 1-hr} + 0.759 P_{100-yr, 1-hr}
\]

\[
P_{2-hr} = 0.658 P_{1-hr} + 0.342 P_{6-hr}
\]

In **Region 1**

\[
P_{3-hr} = 0.401 P_{1-hr} + 0.599 P_{6-hr}
\]
In Region 2
\[ P_{3-hr} = 0.428 P_{1-hr} + 0.572 P_{6-hr} \] 
(3-11M)

\[ P_{12-hr} = 0.50 P_{6-hr} + 0.50 P_{24-hr} \] 
(3-12M)

Time of Concentration

\[ T_c = \left( \frac{L_1}{V_1} + \frac{L_2}{V_2} + \frac{L_3}{V_3} + \ldots + \frac{L_n}{V_n} \right) \frac{1}{60} \]

\[ P_{5-min} = 0.29 P_{1-hr} \] 
(3-13M)

\[ P_{10-min} = 0.45 P_{1-hr} \] 
(3-14M)

\[ P_{15-min} = 0.57 P_{1-hr} \] 
(3-15M)

\[ P_{30-min} = 0.79 P_{1-hr} \] 
(3-16M)

Time of Concentration

\[ T_c = \left( \frac{L_1}{V_1} + \frac{L_2}{V_2} + \frac{L_3}{V_3} + \ldots + \frac{L_n}{V_n} \right) \frac{1}{60} \]

(3-17M)

where

- \( T_c \) = Time of concentration, minutes
- \( V_1 \) = Average flow velocity in the uppermost reach of the watercourse, m./sec.
- \( L_1 \) = Length of the uppermost reach of the watercourse, m.
- \( V_2, V_3, \ldots \) = Average flow velocities in subsequent reaches progressing downstream, m./sec.
- \( L_2, L_3, \ldots \) = Lengths of subsequent reaches progressing downstream, m.

Kirpich Formula

\[ T_c = 0.0195 L^{0.77} S^{-0.385} \] 
(3-18M)

where

- \( T_c \) = time of concentration, in minutes
- \( L \) = length from drainage to outlet along the primary drainage path, in meters
- \( S \) = average slope of the primary drainage path, in m/m
Manning’s Equation

\[ V = \frac{1.003}{n} R^{2/3} S^{1/2} \]  \hspace{1cm} (3-19M)

where

- \( V \) = average velocity in the channel, m./sec.
- \( n \) = Manning’s roughness coefficient
- \( R \) = hydraulic radius of the flow, m.
- \( S \) = average channel slope, m./m.

Rational Formula

\[ Q = 0.0028 \ C \ i \ A \]  \hspace{1cm} (3-20M)

where

- \( Q \) = the peak rate of runoff, in \( m^3/s \)
- \( C \) = a dimensionless runoff coefficient
- \( i \) = the rainfall intensity, in millimeters/hour
- \( A \) = the watershed area, in hectares

\textit{Weighted} \( C = \frac{\sum C_i \cdot A_i}{A} \)  \hspace{1cm} (3-21M)

where

- \( C_i \) = C value for one part of the watershed
- \( A_i \) = area, A, in hectares for the corresponding part of the watershed

Unit Peak Discharge Equation

\[ q_u = 0.0015 \ T_c^{-0.812} 10^{[\log (T_c) + 0.3]} \cdot [\log (T_c) - 0.3]^{1.5} \]  \hspace{1cm} (3-22M)

where

- \( q_u \) = unit peak discharge from the watershed, \( m^3/s/ha-mm \)
- \( T_c \) = time of concentration, in hours
Runoff Depth Equation

\[ Q_d = \frac{P_{24}}{24} - \frac{(5,080/CN)}{(20,320/CN) - 203.2} \]

where

\( Q_d \) = average runoff depth for the entire watershed, in millimeters

\( P_{24} \) = 24-hour rainfall depth, in millimeters

Peak Rate of Runoff Equation

\[ Q_p = A \cdot Q_d \cdot q_u \]

where

\( Q_p \) = peak discharge, in m³/s

\( A \) = drainage area, in hectares

Runoff Volume Equation

\[ Q_v = 10 \cdot Q_d \cdot A \]

where

\( Q_v \) = runoff volume from the watershed, in m³

USGS Regional Regression Equations

- \( A \) — drainage area, in hectares
- \( E \) — mean basin elevation, in meters above sea level
- \( E_c \) — average of channel elevations at 10% and 85% of stream length upstream from the gaging station (use structure location instead of gaging station for NMSHTD projects), in meters
- \( P_{24,10} \) — maximum precipitation depths, in millimeters (24-hour storm, 10-year recurrence interval). Precipitation depths can be found in Appendix E, conversion from inches to millimeters is necessary.
Table C–2

USGS Rural Flood Frequency Equations for New Mexico

Metric Version

<table>
<thead>
<tr>
<th>Region 1</th>
<th>Average Standard Error of Prediction</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Log Units</td>
</tr>
<tr>
<td><strong>Northeast Plains</strong></td>
<td></td>
</tr>
<tr>
<td>$Q_2 = 0.170 \cdot A^{0.53}$</td>
<td>0.346 (3–26M)</td>
</tr>
<tr>
<td>$Q_5 = 0.540 \cdot A^{0.50}$</td>
<td>0.301 (3–27M)</td>
</tr>
<tr>
<td>$Q_{10} = 0.945 \cdot A^{0.49}$</td>
<td>0.288 (3–28M)</td>
</tr>
<tr>
<td>$Q_{25} = 1.68 \cdot A^{0.48}$</td>
<td>0.284 (3–29M)</td>
</tr>
<tr>
<td>$Q_{50} = 2.32 \cdot A^{0.48}$</td>
<td>0.285 (3–30M)</td>
</tr>
<tr>
<td>$Q_{100} = 3.11 \cdot A^{0.48}$</td>
<td>0.291 (3–31M)</td>
</tr>
<tr>
<td>$Q_{500} = 5.51 \cdot A^{0.48}$</td>
<td>0.312 (3–32M)</td>
</tr>
</tbody>
</table>

<p>| Region 2                  |                                      |
| <strong>Northwest Plateau</strong>     |                                      |
| $Q_2 = 0.176 \cdot A^{0.47}$ | 0.390 (3–33M)                        |
| $Q_5 = 0.433 \cdot A^{0.46}$ | 0.311 (3–34M)                        |
| $Q_{10} = 0.672 \cdot A^{0.46}$ | 0.282 (3–35M)                       |
| $Q_{25} = 1.13 \cdot A^{0.45}$ | 0.262 (3–36M)                       |
| $Q_{50} = 1.52 \cdot A^{0.45}$ | 0.255 (3–37M)                       |
| $Q_{100} = 1.98 \cdot A^{0.45}$ | 0.254 (3–38M)                       |
| $Q_{500} = 3.37 \cdot A^{0.45}$ | 0.265 (3–39M)                       |</p>
<table>
<thead>
<tr>
<th>Region 3</th>
<th>Average Standard Error of Prediction</th>
<th>Log Units</th>
<th>Equation Code</th>
</tr>
</thead>
<tbody>
<tr>
<td>Southeast Mountain</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>( Q_2 = 7.25 \cdot A^{0.60} \cdot \left( \frac{E}{1,000} \right)^{-5.96} )</td>
<td>0.150</td>
<td>0.150</td>
<td>(3–40M)</td>
</tr>
<tr>
<td>( Q_5 = 1.73 \cdot A^{0.57} \cdot \left( \frac{E}{1,000} \right)^{-6.69} )</td>
<td>0.163</td>
<td>0.163</td>
<td>(3–41M)</td>
</tr>
<tr>
<td>( Q_{10} = 24.3 \cdot A^{0.70} \cdot \left( \frac{E}{1,000} \right)^{-6.94} )</td>
<td>0.169</td>
<td>0.169</td>
<td>(3–42M)</td>
</tr>
<tr>
<td>( Q_{25} = 28.9 \cdot A^{0.75} \cdot \left( \frac{E}{1,000} \right)^{-7.10} )</td>
<td>0.178</td>
<td>0.178</td>
<td>(3–43M)</td>
</tr>
<tr>
<td>( Q_{50} = 31.1 \cdot A^{0.78} \cdot \left( \frac{E}{1,000} \right)^{-7.16} )</td>
<td>0.187</td>
<td>0.187</td>
<td>(3–44M)</td>
</tr>
<tr>
<td>( Q_{100} = 31.9 \cdot A^{0.81} \cdot \left( \frac{E}{1,000} \right)^{-7.19} )</td>
<td>0.199</td>
<td>0.199</td>
<td>(3–45M)</td>
</tr>
<tr>
<td>( Q_{500} = 30.9 \cdot A^{0.87} \cdot \left( \frac{E}{1,000} \right)^{-7.20} )</td>
<td>0.236</td>
<td>0.236</td>
<td>(3–46M)</td>
</tr>
<tr>
<td>Region 4</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Southeast Plains</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>( Q_2 = 0.136 \cdot A^{0.51} )</td>
<td>0.538</td>
<td>0.538</td>
<td>(3–47M)</td>
</tr>
<tr>
<td>( Q_5 = 0.332 \cdot A^{0.54} )</td>
<td>0.424</td>
<td>0.424</td>
<td>(3–48M)</td>
</tr>
<tr>
<td>( Q_{10} = 0.542 \cdot A^{0.55} )</td>
<td>0.374</td>
<td>0.374</td>
<td>(3–49M)</td>
</tr>
<tr>
<td>( Q_{25} = 0.860 \cdot A^{0.57} )</td>
<td>0.326</td>
<td>0.326</td>
<td>(3–50M)</td>
</tr>
<tr>
<td>( Q_{50} = 1.17 \cdot A^{0.58} )</td>
<td>0.300</td>
<td>0.300</td>
<td>(3–51M)</td>
</tr>
<tr>
<td>( Q_{100} = 1.53 \cdot A^{0.59} )</td>
<td>0.282</td>
<td>0.282</td>
<td>(3–52M)</td>
</tr>
<tr>
<td>( Q_{500} = 2.46 \cdot A^{0.62} )</td>
<td>0.262</td>
<td>0.262</td>
<td>(3–53M)</td>
</tr>
</tbody>
</table>
Table C-2  
USGS Rural Flood Frequency Equations for New Mexico  
Metric Version

<table>
<thead>
<tr>
<th>Region 5</th>
<th>Northern Mountain</th>
<th>Average Standard Error of Prediction</th>
<th>Log Units</th>
</tr>
</thead>
<tbody>
<tr>
<td>$Q_2 = 6.3 \times 10^{-3} \cdot A^{0.83} \cdot \left(\frac{E}{1,000}\right)^{-2.22} \cdot P_{24,25}^{0.31}$</td>
<td></td>
<td>0.343</td>
<td>(3–54M)</td>
</tr>
<tr>
<td>$Q_5 = 8.47 \times 10^{-3} \cdot A^{0.81} \cdot \left(\frac{E}{1,000}\right)^{-3.01} \cdot P_{24,25}^{0.63}$</td>
<td></td>
<td>0.309</td>
<td>(3–55M)</td>
</tr>
<tr>
<td>$Q_{10} = 8.72 \times 10^{-3} \cdot A^{0.81} \cdot \left(\frac{E}{1,000}\right)^{-3.41} \cdot P_{24,25}^{0.81}$</td>
<td></td>
<td>0.297</td>
<td>(3–56M)</td>
</tr>
<tr>
<td>$Q_{25} = 8.45 \times 10^{-3} \cdot A^{0.80} \cdot \left(\frac{E}{1,000}\right)^{-3.85} \cdot P_{24,25}^{1.03}$</td>
<td></td>
<td>0.294</td>
<td>(3–57M)</td>
</tr>
<tr>
<td>$Q_{50} = 7.78 \times 10^{-3} \cdot A^{0.80} \cdot \left(\frac{E}{1,000}\right)^{-4.13} \cdot P_{24,25}^{1.18}$</td>
<td></td>
<td>0.298</td>
<td>(3–58M)</td>
</tr>
<tr>
<td>$Q_{100} = 6.76 \times 10^{-3} \cdot A^{0.80} \cdot \left(\frac{E}{1,000}\right)^{-4.40} \cdot P_{24,25}^{1.33}$</td>
<td></td>
<td>0.306</td>
<td>(3–59M)</td>
</tr>
<tr>
<td>$Q_{500} = 5.07 \times 10^{-3} \cdot A^{0.80} \cdot \left(\frac{E}{1,000}\right)^{-4.95} \cdot P_{24,25}^{1.64}$</td>
<td></td>
<td>0.337</td>
<td>(3–60M)</td>
</tr>
</tbody>
</table>
Table C-2

USGS Rural Flood Frequency Equations for New Mexico

Metric Version

<table>
<thead>
<tr>
<th>Region 6 Central Mountain–Valley</th>
<th>Average Standard Error of Prediction</th>
<th>Log Units</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>(3–61M)</td>
</tr>
<tr>
<td>$Q_2 = 5.45 \times 10^{-2} \cdot A^{0.50} \cdot \left( \frac{Ec}{1,000} \right)^{-5.28} \cdot P_{24,10}^{1.18}$</td>
<td>0.366</td>
<td></td>
</tr>
<tr>
<td>$Q_5 = 8.68 \times 10^{-3} \cdot A^{0.47} \cdot \left( \frac{Ec}{1,000} \right)^{-4.49} \cdot P_{24,10}^{1.76}$</td>
<td>0.274</td>
<td></td>
</tr>
<tr>
<td>$Q_{10} = 3.33 \times 10^{-3} \cdot A^{0.46} \cdot \left( \frac{Ec}{1,000} \right)^{-4.09} \cdot P_{24,10}^{2.06}$</td>
<td>0.231</td>
<td></td>
</tr>
<tr>
<td>$Q_{25} = 1.31 \times 10^{-3} \cdot A^{0.44} \cdot \left( \frac{Ec}{1,000} \right)^{-3.67} \cdot P_{24,10}^{2.37}$</td>
<td>0.193</td>
<td></td>
</tr>
<tr>
<td>$Q_{50} = 7.01 \times 10^{-4} \cdot A^{0.43} \cdot \left( \frac{Ec}{1,000} \right)^{-3.38} \cdot P_{24,10}^{2.57}$</td>
<td>0.180</td>
<td></td>
</tr>
<tr>
<td>$Q_{100} = 4.13 \times 10^{-4} \cdot A^{0.42} \cdot \left( \frac{Ec}{1,000} \right)^{-3.09} \cdot P_{24,10}^{2.74}$</td>
<td>0.173</td>
<td></td>
</tr>
<tr>
<td>$Q_{500} = 1.65 \times 10^{-4} \cdot A^{0.40} \cdot \left( \frac{Ec}{1,000} \right)^{-2.45} \cdot P_{24,10}^{3.03}$</td>
<td>0.185</td>
<td></td>
</tr>
</tbody>
</table>
Table C-2

USGS Rural Flood Frequency Equations for New Mexico

Metric Version

<table>
<thead>
<tr>
<th>Region 7</th>
<th>Southwest Desert</th>
<th>Region 8</th>
<th>Southwest Mountain</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Average Standard Error</td>
<td></td>
<td>Average Standard Error</td>
</tr>
<tr>
<td></td>
<td>of Prediction</td>
<td></td>
<td>of Prediction</td>
</tr>
<tr>
<td></td>
<td>Log Units</td>
<td></td>
<td>Log Units</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>(3-68M)</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>(3-69M)</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>(3-70M)</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>(3-71M)</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>(3-72M)</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>(3-73M)</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>(3-74M)</td>
</tr>
</tbody>
</table>

**Region 7**

Southwest Desert

\[ Q_2 = 0.281 \cdot A^{0.46} \]
\[ Q_5 = 0.484 \cdot A^{0.48} \]
\[ Q_{10} = 0.642 \cdot A^{0.49} \]
\[ Q_{25} = 0.864 \cdot A^{0.30} \]
\[ Q_{50} = 1.02 \cdot A^{0.31} \]
\[ Q_{100} = 1.18 \cdot A^{0.52} \]
\[ Q_{500} = 1.49 \cdot A^{0.55} \]

**Average Standard Error of Prediction**

- Log Units: 0.229
- Log Units: 0.211
- Log Units: 0.212
- Log Units: 0.220
- Log Units: 0.231
- Log Units: 0.244
- Log Units: 0.279

**Region 8**

Southwest Mountain

\[ Q_2 = 181 \cdot A^{0.19} \cdot \left( \frac{Ec}{1000} \right)^{-6.10} \]
\[ Q_5 = 165 \cdot A^{0.23} \cdot \left( \frac{Ec}{1000} \right)^{-5.53} \]
\[ Q_{10} = 153 \cdot A^{0.25} \cdot \left( \frac{Ec}{1000} \right)^{-5.19} \]
\[ Q_{25} = 138 \cdot A^{0.27} \cdot \left( \frac{Ec}{1000} \right)^{-4.80} \]
\[ Q_{50} = 123 \cdot A^{0.29} \cdot \left( \frac{Ec}{1000} \right)^{-4.52} \]
\[ Q_{100} = 117 \cdot A^{0.30} \cdot \left( \frac{Ec}{1000} \right)^{-4.25} \]
\[ Q_{500} = 100 \cdot A^{0.32} \cdot \left( \frac{Ec}{1000} \right)^{-3.68} \]

**Average Standard Error of Prediction**

- Log Units: 0.348
- Log Units: 0.332
- Log Units: 0.321
- Log Units: 0.331
- Log Units: 0.344
- Log Units: 0.359
- Log Units: 0.401
Table C–3
USGS Small Rural Basin Flood Frequency Equations for New Mexico
Metric Version

<table>
<thead>
<tr>
<th>Resistance</th>
<th>Average Standard Error of Prediction</th>
<th>Log Units</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>(3–82M)</td>
</tr>
<tr>
<td></td>
<td></td>
<td>(3–83M)</td>
</tr>
<tr>
<td></td>
<td></td>
<td>(3–84M)</td>
</tr>
<tr>
<td></td>
<td></td>
<td>(3–85M)</td>
</tr>
<tr>
<td></td>
<td></td>
<td>(3–86M)</td>
</tr>
<tr>
<td></td>
<td></td>
<td>(3–87M)</td>
</tr>
<tr>
<td></td>
<td></td>
<td>(3–88M)</td>
</tr>
</tbody>
</table>

Note: These equations were developed for drainage basins less than 10 sq. mi. and less than 7500 ft. mean basin elevation. See Section 3.2 of this manual for limitations on the use of these equations.
Table C-4

USGS Urban Peak Discharge

Metric Version

<table>
<thead>
<tr>
<th>Three Parameter Estimating Equations</th>
<th>Coefficient of Determination ($R^2$)</th>
</tr>
</thead>
<tbody>
<tr>
<td>$UQ_2 = 13.2 \cdot A^{-2.1} \cdot (13 - BDF)^{-4.3} \cdot RQ_{2}^{.73}$</td>
<td>.91 (3–89M)</td>
</tr>
<tr>
<td>$UQ_5 = 10.6 \cdot A^{-1.7} \cdot (13 - BDF)^{-3.9} \cdot RQ_{5}^{.78}$</td>
<td>.92 (3–90M)</td>
</tr>
<tr>
<td>$UQ_{10} = 9.51 \cdot A^{-1.6} \cdot (13 - BDF)^{-3.6} \cdot RQ_{10}^{.79}$</td>
<td>.92 (3–91M)</td>
</tr>
<tr>
<td>$UQ_{25} = 8.68 \cdot A^{-1.5} \cdot (13 - BDF)^{-3.4} \cdot RQ_{25}^{.80}$</td>
<td>.92 (3–92M)</td>
</tr>
<tr>
<td>$UQ_{50} = 8.04 \cdot A^{-1.5} \cdot (13 - BDF)^{-3.2} \cdot RQ_{50}^{.81}$</td>
<td>.91 (3–93M)</td>
</tr>
<tr>
<td>$UQ_{100} = 7.70 \cdot A^{-1.5} \cdot (13 - BDF)^{-3.2} \cdot RQ_{100}^{.82}$</td>
<td>.91 (3–94M)</td>
</tr>
<tr>
<td>$UQ_{500} = 7.47 \cdot A^{-1.6} \cdot (13 - BDF)^{-3.0} \cdot RQ_{500}^{.82}$</td>
<td>.89 (3–95M)</td>
</tr>
</tbody>
</table>
Unit Hydrograph Equations

\[ q_p = \frac{K_p}{T_p} A \]  

(3–96M)

where

- \( q_p \) = the unit hydrograph peak discharge, in \( \text{m}^3/\text{s} \)
- \( A \) = basin area, in hectares
- \( T_p \) = the time to peak of the hydrograph, in hours
- \( K_p \) = an empirical constant

Note: \( K_p \) is an empirically derived constant which varies from (0.0328) in very flat swampy areas to (0.0656) in steep terrain. An average value of \( K_p = 0.0529 \) is used unless otherwise indicated, and approved by the NMSHTD Drainage Section.

\[ T_p = \frac{D}{2} + T_L \]  

(3–97M)

and

\[ D = 0.133 T_c \]  

(3–98M)

where

- \( T_p \) = time to peak of the hydrograph, in hours
- \( D \) = duration of excess rainfall, in hours
- \( T_L \) = basin lag time, time from the centroid of excess rainfall, in hours
- \( T_c \) = time of concentration, in hours

\[ T_L = 0.6 T_c \]  

(3–99M)

\[ T_p = 0.67 T_c \]  

(3–100M)

\[ Q_v = 10 \cdot A \cdot Q_d \]  

(3–101M)

where

- \( Q_v \) = runoff volume, in \( \text{m}^3 \)
- \( Q_d \) = average depth of runoff, in millimeters
- \( A \) = watershed area, in hectares

The weighted flood frequency estimate is given by the equation:

\[ Q_{pu} = Q_{pg} \left( \frac{A_u}{A_g} \right)^x \]  

(3–102M)

where

- \( Q_{pu} \) = weighted peak discharge at the ungaged site, in \( \text{m}^3/\text{s} \)
- \( Q_{pg} \) = peak discharge at the gaged site, in \( \text{m}^3/\text{s} \)
- \( A_u \) = drainage area at the ungaged site, in hectares
- \( A_g \) = drainage area at the gaged site, in hectares
- \( x \) = exponent of the drainage area term used in the appropriate equation from Table C–2.
\[ R = 1 - \left(1 - \frac{1}{T_r}\right)^m (100) \]  

(3–103M)

where

- \( R \) = the risk of the design discharge being exceeded at least once during the design life, in percent
- \( T_r \) = the recurrence interval or frequency of the design flood, in years
- \( m \) = the design life of the structure, in years

\[ T_r = \frac{1}{\left(1 - \frac{R}{100}\right)^{\frac{1}{m}}} \]  

(3–104M)
APPENDIX D — Text References and Other Selected References


