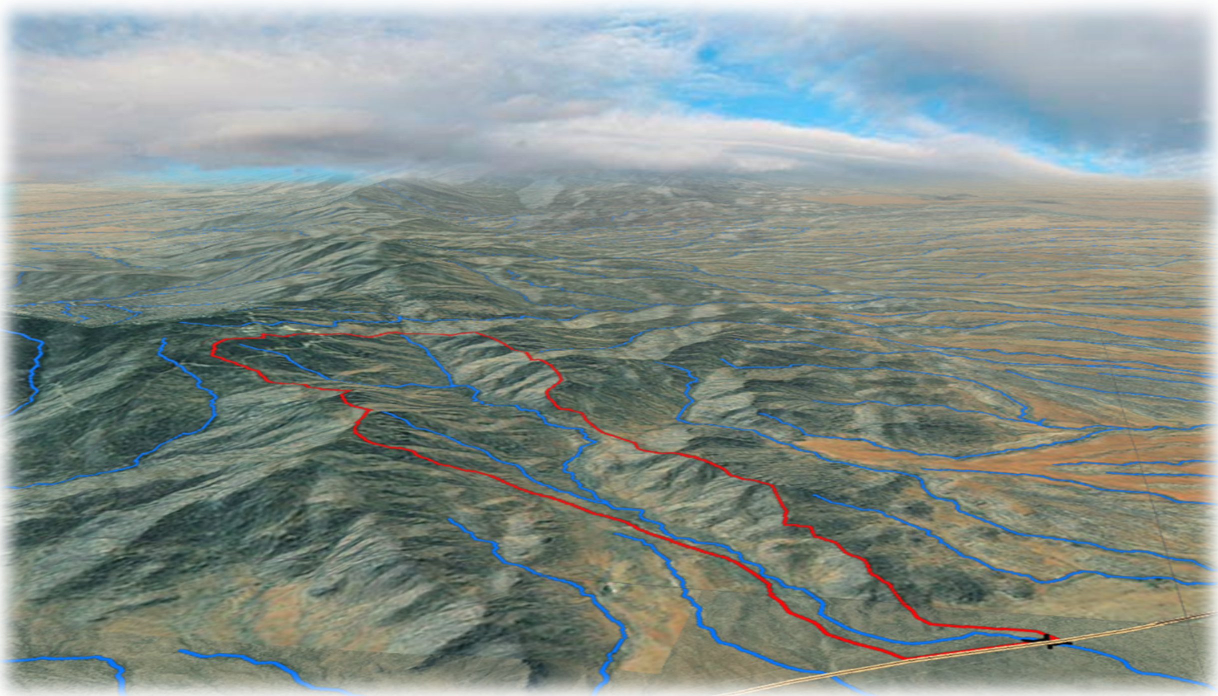
ARIZONA DEPARTMENT OF TRANSPORTATION

Highway Drainage Design Manual



Volume 2

Hydrology

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Intermodal Transportation

ACKNOWLEDGEMENTS

This manual was originally prepared in 1993 by NBS/Lowry Engineers & Planners and George V. Sabol Consulting Engineers under the direction of the Arizona Transportation Research Center (ATRC). The 1993 Manual was the result of an extensive research and validation effort. The methods and procedures within the manual represented the state of the practice at that time and represented a significant advancement in the practice of hydrology for highway drainage design in Arizona. The original manual has been in continuous use since then with no significant updates until now. The work done by the original project team and authors, particularly Dr. George Sabol as the principal author, remains foundational to the current work. The current project team preparing this update would like to acknowledge this earlier work.

The Arizona Department of Transportation (ADOT) also acknowledges the many contributors whose effort went into the successful completion of this hydrology manual update project. Although this manual is a revision of the existing manual and current state of practice preferred by ADOT, much effort went into deeply analyzing its methods to make recommendations for their adequacy and reasonableness for continuous use or to recommend the use of alternate methods. Although other techniques and procedures have been recommended and added, the existing methods which have served ADOT with good results for 20 years have been retained. ADOT solicited the review assistance of various local flood control agencies around the state and the Hydraulics Office of the Federal Highways Administration; to those who responded, their diligence is much appreciated. The review panel of experts, which guided the technical development, is acknowledged.

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PREFACE

INTRODUCTION

The Arizona Department of Transportation (ADOT) issued its Highway Drainage Design Manual-Hydrology (1993 Manual) in 1993. This update is intended to incorporate advances in the state of the practice that have been developed since 1993.

The ADOT Highway Drainage Design Manual (DDM) consists of the following three volumes:

- Volume 1 - Policy & Guidelines
- Volume 2 - Hydrology
- Volume 3 - Hydraulics

DDM Volume 1 - Policy & Guidelines, is the base document that points to the other documents as appropriate.

DDM Volume 2 - Hydrology, is intended to provide guidance for the performance of flood hydrology for ADOT drainage design. Two analytic methods are presented to determine design discharges; those two methods are to be used mainly for ungaged watersheds. The two analytic methods are:

1. The Rational Method. Used for uniform drainage areas that are less than 160 acres in size.
2. Rainfall-Runoff Modeling. Used for any size drainage area.

The rainfall-runoff modeling guidance is structured to be compatible with the U.S. Army Corps of Engineers' HEC-HMS Flood Hydrology program, as well as the FLO-2D two-dimensional flow model by FLO-2D Software, Inc. For rainfall-runoff modeling, this manual should be used in conjunction with the HEC-HMS or FLO-2D User's Manual, as applicable. The content of this manual assumes a familiarity and basic understanding of the HEC-HMS or FLO-2D program and modeling procedures.

A flood frequency analysis procedure is also provided for computing flood magnitude-frequency relations where systematic stream gage records of sufficient length are available. The flood frequency analysis procedure can be used, where appropriate, to (1) estimate the design flood peak discharge, (2) provide estimates of flood peak discharges for the calibration or verification of rainfall-runoff models, (3) provide regional estimates of flood magnitudes that can be used to check or substantiate other methods to estimate flood magnitudes or to develop regional flood discharge relations, or (4) perform other hydrologic studies, such as the investigation of flood magnitudes from snowmelt to be used as baseflow to a watershed rainfall-runoff model.

This manual was prepared for use by engineers and/or hydrologists who are trained and experienced in the fundamentals of hydrology in general, and flood hydrology in particular. Other users should work under the direct supervision and guidance of appropriately qualified personnel.

SUMMARY OF MANUAL CHANGES

Many of the procedures from the 1993 Manual have been retained within this update. However, four key areas of updates are included as follows:

- Update Rainfall from NOAA Atlas 2 to NOAA Atlas 14
- Update the manual document to be more user-friendly by; 1) providing an electronic version suitable for online use and/or distribution and 2) providing electronic tools and datasets to assist in application of the procedures described in this manual
- Migrate from HEC-1 to HEC-HMS as the computerized hydrologic modeling platform
- Provide guidance for application of two-dimensional modeling (FLO-2D) where appropriate

Users of this manual are invited to submit comments, suggestions, or findings of errors. This information should be addressed to:

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USE OF GIS & PRE-PROCESSOR APPLICATIONS FOR HYDROLOGY

One of the significant changes in application of hydrologic methods since completion of the 1993 Manual is the proliferation of GIS tools and data pre-processors, such as WMS and HEC-GeoHMS, to assist in developing computer model inputs. This change has resulted in increased efficiency and improved precision and detail of model input as compared to the labor-intensive desktop methods that were primarily in use in 1993. Although the use of these tools is encouraged for users of this manual, particular methods are not identified or required. The important consideration in the selection and use of pre-processor tools is that the data and procedures specified in this manual are used and that the resulting input can be duplicated and checked by other independent methods. The Rainfall Averaging and Rational Method tools provided as resources have been developed using procedures from the manual. General guidance on the usage and application of the Rainfall Averaging and Rational Method tools are included in Chapter 1 and Chapter 2, respectively, and within the help menus provided in each tool.

REQUIREMENTS FOR DRAINAGE REPORTS

Hydrology documentation for a project normally includes the narrative of the procedures used and a compilation of the input and output data; the computations and the results of the analysis. This documentation makes up part of the overall drainage report for a project, along with the hydraulic documentation and other project documents. The requirements for drainage reports are fully spelled out in the DDM Volume 3 - Hydraulics, Chapter 2, which should be consulted separately.

QUICK REFERENCE GUIDE

The following Quick Reference Guide provides the hydrology procedures contained within this manual as well as data sources and software applications and tools that are available for use in hydrologic computations.

QUICK REFERENCE GUIDE		
TOPIC	PROCEDURES	DATA SOURCES & SOFTWARE
RAINFALL	NOAA Atlas 14 Rainfall Data (Section 1.1.1) NOAA data processing procedures (Section 1.1.2)	NOAA Web Data[1] ADOT Rainfall Averaging Tool with NOAA14 Rainfall
RATIONAL METHOD	Rational Method (Section 2.2) ADOT Rational Method Tool (Section 2.1)	ADOT Rational Method Tool
RAINFALL LOSSES	Green and Ampt Loss Method (Section 3.2) Initial & Constant Loss Rate Method (Section 3.5) HEC-HMS Application (Section 3.3)	Statewide Soil Data HEC-HMS FLO-2D
UNIT HYDROGRAPHS	Clark Unit Hydrograph (Section 4.2) HEC-HMS Application (Section 4.2.4)	HEC-HMS
CHANNEL ROUTING	Muskingum-Cunge, Kinematic Wave, and Modified Puls Routing (Section 5.2)	HEC-HMS
STORAGE ROUTING	Level Pool Storage Routing (Section 6.2)	HEC-HMS
TRANSMISSION LOSSES	Transmission Losses (Section 7.2)	HEC-HMS
HEC-HMS MODELING	HEC-HMS Modeling Guidance (Chapter 8) HEC-HMS Model Review Checklist (Section 8.3)	HEC-HMS
FLO-2D MODELING	FLO-2D Modeling Guidance (Chapter 9) FLO-2D Model Review Guidance (Section 9.3)	FLO-2D
FLOOD-FREQUENCY ANALYSIS	Flood Frequency Analysis Procedure (Chapter 10)	
REGIONAL REGRESSION EQUATIONS	Regional Regression Equation Guidance (Chapter 11)	

[1] NWS-NOAA Rainfall Data Website: http://hdsc.nws.noaa.gov/hdsc/pfds/pfds_map_cont.html

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Chapter 1
RAINFALL

This chapter contains the following details:

- Procedures and instructions to prepare rainfall input to the HEC-HMS program and to generate intensity-duration-frequency curves for use with the Rational Method.
- NOAA Atlas 14 data to develop site-specific rainfall data.

1.1 INTRODUCTION

Analytic methods (Rational Method and rainfall-runoff modeling) require the definition of the rainfall for the desired frequency. For the Rational Method, a site-specific rainfall intensity-duration-frequency (I-D-F) relationship is required. A procedure for developing a site-specific I-D-F relationship using the NOAA Atlas 14 data for any location in Arizona is presented in this section.

For rainfall-runoff modeling (HEC-HMS software), the temporal and spatial distribution of the design rainfall also must be provided. For highway drainage studies in Arizona, a balanced storm rainfall event where the precipitation depths for the various durations within the storm have a consistent exceedance probability is to be used. Within the HEC-HMS software, that rainfall distribution is defined using the Frequency-Based Hypothetical Storm method. The point rainfall depth-duration frequency (D-D-F) statistics that are input into the HEC-HMS software are automatically adjusted for the rainfall depth-area relation by procedures built into the HEC-HMS program. The Frequency-Based Hypothetical Storm methodology is described in U.S. Army Corps of Engineers HEC-HMS Technical Reference Manual (2000).

1.1.1 Source of Design Rainfall Information

The rainfall D-D-F statistics for Arizona are derived from information in National Oceanic and Atmospheric Administration (NOAA) Atlas 14, Volume I, Arizona (Bonnin, et al., 2006). For ADOT projects, the mean precipitation frequency estimates shall be used.

1.1.2 GIS Tools and Datasets for Rainfall

The NOAA's National Weather Service (NWS) provides the GIS datasets of NOAA Atlas 14 rainfall. The data are available from the NWS Hydrometeorological Design Studies Center website: http://www.nws.noaa.gov/oh/hdsc/index.html. The data can be downloaded in ASCII grid format for various storm events and durations. These data include the mean NOAA Atlas 14 precipitation frequency estimates, as well as the upper and lower bounds of the 90% confidence interval. The mean NOAA Atlas 14 precipitation frequency estimates for the design storm durations are to be used in the development of the D-D-F table. The D-D-F table developed in this manner is then used in the development of watershed hydrology using procedures such as the Rational Method or rainfall-runoff modeling as explained in Section 1.3.

In some instances, the precipitation frequency values across a given watershed may vary widely. Such variation is more likely for larger watersheds and watersheds with significant elevation differences. The spatial variation in the NOAA Atlas 14 statistics should be reviewed before selecting the representative watershed average rainfall values. To facilitate the computation of representative watershed rainfall values, ADOT has developed a rainfall averaging tool that incorporates the NOAA Atlas 14 data. This tool can be used to determine average precipitation values over specified areas. The average precipitation values are computed by this tool using areas defined and input into the tool using GIS shapefiles. The average computations are performed for a range of storm frequencies and durations and the results are provided in a table format.

1.2 PROCEDURE

Rational Method

When using the Rational Method, a site-specific D-D-F graph or table should be used.

HEC-HMS Modeling

When using the HEC-HMS model, the rainfall data is input into the software using the frequency storm option. The storm duration to be used is 24 hours.

Rainfall statistics developed using the procedures in this section are dependent upon the information provided in NOAA Atlas 14. The frequency storm is a simplified and idealized representation of the temporal distribution of rainfall. It is intended to be used to estimate design discharges for highway drainage facilities. The frequency storm does not necessarily represent the temporal distribution of any historic storm in Arizona.

Both partial duration and annual maximum series datasets are available from NOAA Atlas 14. Given the default meteorologic model setup for the frequency storm option in HEC-HMS and the potential application to shorter frequency design storms, **the partial duration statistics should be used**. HEC-HMS performs the conversion to annual duration output internal to the software.

Site specific I-D-F graphs (Chapter 2) should be used with the Rational Method for most small watersheds in Arizona. Similarly, the frequency storm should be used as input to the HEC-HMS software for larger watersheds in Arizona.

For very large watersheds (possibly as large as or larger than 500 square miles), where the time of concentration (T_c) exceeds 24 hours, a longer duration rainfall distribution (or other project specific distribution) should be developed and used. Procedures for estimating the watershed time of concentration are described in Chapter 4.

1.3 INSTRUCTIONS

1.3.1 Rational Method Site Specific I-D-F Graph

The I-D-F graph/table may be downloaded directly from the NOAA NWS website. The application of the I-D-F table in the Rational Method is presented in **Chapter 2**.

1.3.2 HEC-HMS Rainfall Input-Frequency Storm

1. Develop the rainfall D-D-F table using the NOAA NWS website or the NOAA Atlas 14 GIS datasets from the NOAA NWS website. It is recommended that averaging of the rainfall values obtained from the GIS datasets be performed when the precipitation values vary greatly within a watershed. Averaging the precipitation depths will provide representative precipitation values. The ADOT Rainfall Averaging Tool may be used to perform this task. Details on the ADOT Rainfall Averaging Tool are presented in **Section 1.4** Example 1.

2. Create a Frequency Storm with HEC-HMS software by entering the rainfall values as shown on **Figure 1–1**.

3. Set the Intensity Duration to 5 minutes.

4. Set the Storm Duration as 24-hours or 1 day.

5. Set the Intensity Position to 50 percent.

6. Input the watershed average point rainfall depths for each duration from 5 minutes to 1 day.

7. Set the storm area as the total drainage area to the point of design. Alternatively, if multiple sub-basins are used in the model to multiple points of design, leave the Storm Area blank and let HEC-HMS compute the areal reduction based on the sub-basin size to each point of design.

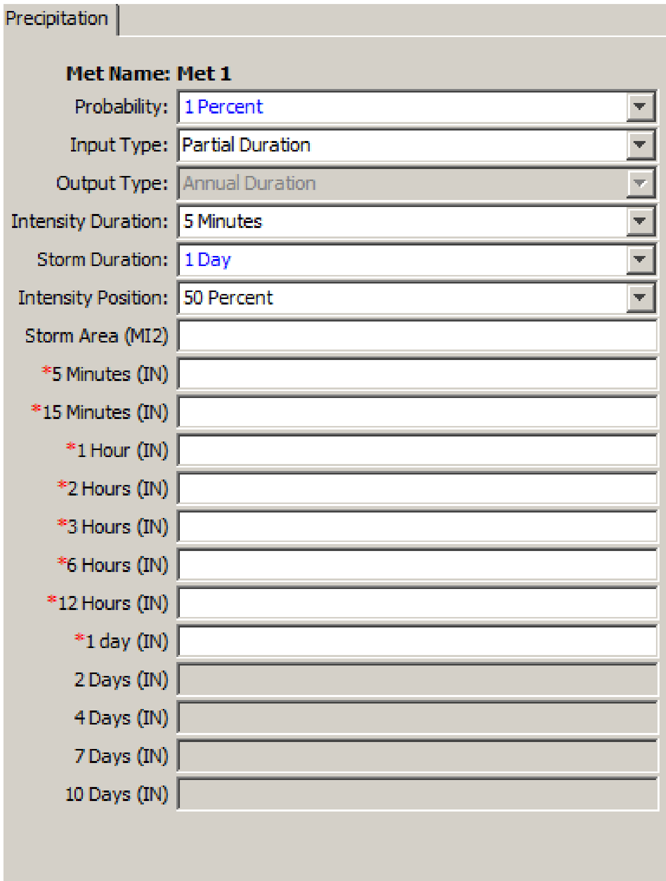


Figure 1–1 Example of Precipitation Tab in HEC-HMS Component Editor

1.4 EXAMPLE

Example No. 1-1 Rainfall Estimation

Problem:

An example watershed is shown in **Figure 1–2**. Perform the following steps for a single point (using the watershed centroid) and multiple points using rainfall averaging techniques:

1. Develop the D-D-F table for storm frequencies: 1-year, 2-year, 5-year, 10-year, 25-year, 50-year, 100-year, and 500-year and durations: 5-minutes, 10-minutes, 15-minutes, 30-minutes, 60-minutes, 2-hours, 3-hours, 6-hours, 12-hours and 24-hours.

2. Develop the I-D-F table using the D-D-F table.

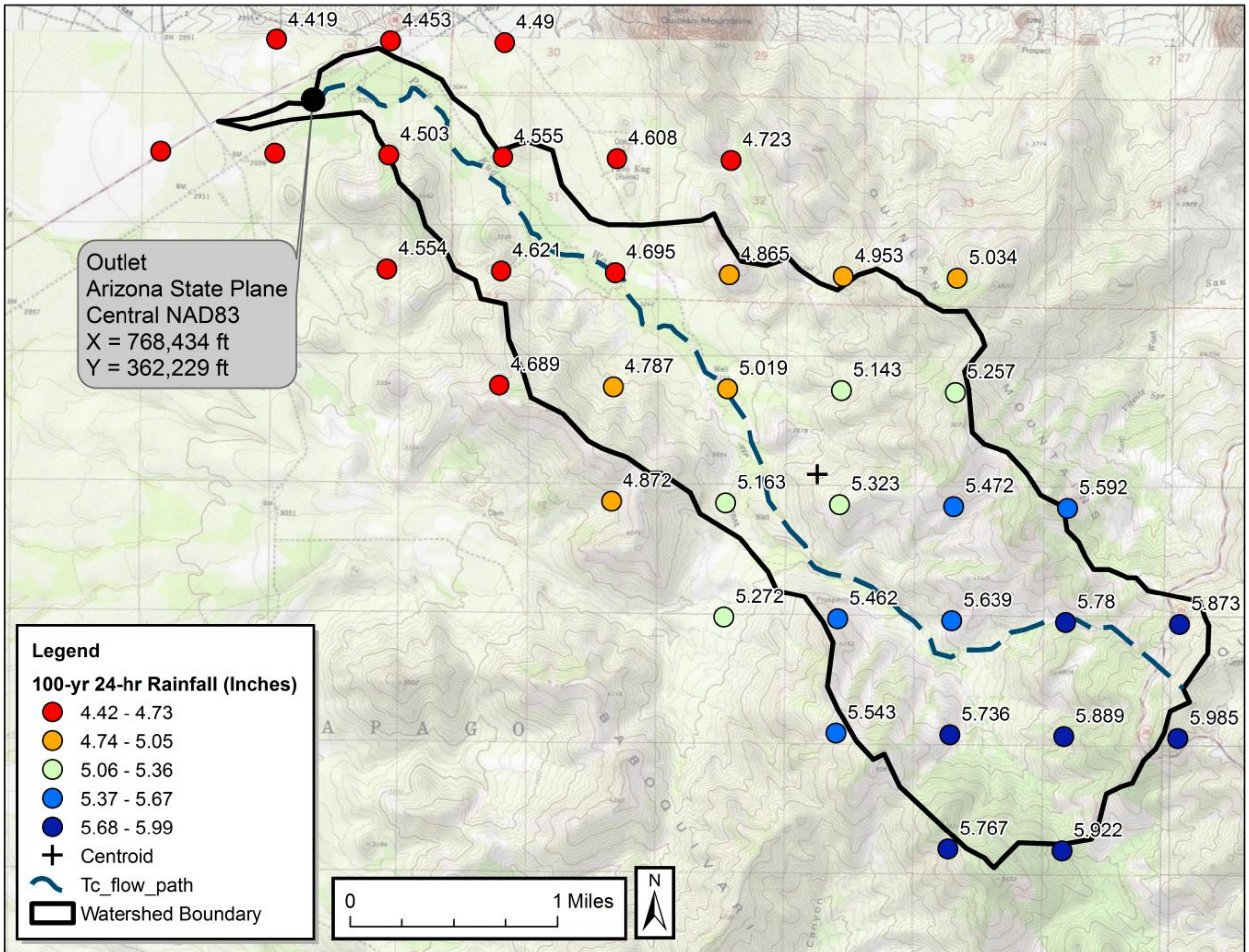


Figure 1–2 Pavo Kug Wash Watershed Rainfall Estimation

Solutions:

Single Point at Watershed Centroid

1. Determine the latitude and longitude of the watershed centroid or point of interest. For Pavo Kug Wash, the watershed centroid (Figure 1–2) is located at

Latitude	Longitude
31.9698 degrees N	-111.6542 degrees W

2. Using a web browser navigate to the NOAA NWS website or click on the link provided.

   http://hdsc.nws.noaa.gov/hdsc/pfds/pfds_map_cont.html

3. Select type of data requested under Data Description.

   Data type: precipitation depth (for a D-D-F output) or precipitation intensity (for an I-D-F output).

Units: English

Time Series type: partial duration

4. Enter in the latitude/longitude under Select Location. Once latitude/longitude is entered, click on Submit.

5. D-D-F/I-D-F output data for selected location can be downloaded as a text file.

Table 1–1 Single Point Pavo Kug Wash Watershed D-D-F Table

Duration	Average Precipitation values (inches)							
	1-year	2-year	5-year	10-year	25-year	50-year	100-year	500-year
5-min	0.33	0.42	0.56	0.66	0.79	0.88	0.97	1.17
10-min	0.5	0.65	0.86	1	1.2	1.34	1.47	1.77
15-min	0.62	0.8	1.06	1.25	1.49	1.66	1.83	2.2
30-min	0.83	1.08	1.43	1.68	2	2.23	2.46	2.96
60-min	1.03	1.34	1.77	2.08	2.48	2.76	3.04	3.66
2-hour	1.15	1.48	1.94	2.28	2.74	3.1	3.45	4.28
3-hour	1.21	1.53	1.98	2.33	2.82	3.21	3.61	4.58
6-hour	1.4	1.76	2.23	2.61	3.13	3.55	3.99	5.04
12-hour	1.63	2.05	2.58	3	3.6	4.06	4.55	5.71
24-hour	2.01	2.51	3.12	3.61	4.28	4.79	5.32	6.59

Table 1–2 Single Point Pavo Kug Wash Watershed I-D-F Table

Duration	Average Rainfall Intensity (inches/hour)							
	1-year	2-year	5-year	10-year	25-year	50-year	100-year	500-year
5-min	3.94	5.1	6.74	7.93	9.44	10.54	11.62	13.98
10-min	2.99	3.88	5.14	6.03	7.19	8.02	8.84	10.64
15-min	2.47	3.21	4.24	4.99	5.94	6.63	7.3	8.79
30-min	1.66	2.16	2.86	3.36	4	4.46	4.92	5.92
60-min	1.03	1.34	1.77	2.08	2.48	2.76	3.04	3.66
2-hour	0.58	0.74	0.97	1.14	1.37	1.55	1.73	2.14
3-hour	0.4	0.51	0.66	0.78	0.94	1.07	1.2	1.53
6-hour	0.23	0.29	0.37	0.44	0.52	0.59	0.67	0.84
12-hour	0.14	0.17	0.21	0.25	0.3	0.34	0.38	0.47
24-hour	0.08	0.1	0.13	0.15	0.18	0.2	0.22	0.27

Rainfall Averaging using GIS precipitation datasets.

This solution assumes the user has knowledge of GIS applications and data manipulation.

1. Obtain the GIS precipitation datasets (in .asc format) for the Semiarid Southwest (sw) from the NOAA NWS website or click the link provided below. There are separate data coverages for each combination of storm frequency and duration. As mentioned in Step 1 of the Example problem statement, there are 8 different storm frequencies and 10 different storm durations. Therefore, 80 (8 times 10) different data coverages should be obtained from the NOAA website. (http://hdsc.nws.noaa.gov/hdsc/pfds/pfds_gis.html)

2. Convert the data coverages obtained from the NOAA website to the same geographical projection as that of the watershed boundary.

3. Determine which data points from the coverage are inside and/or near the watershed boundary.

4. Obtain drainage area precipitation values by performing the arithmetic average on the data values obtained for all the locations identified in Step 3.

5. Repeat Step 4 for all 80 different combinations of storm frequency and duration.

6. Alternatively, the D-D-F table can be generated using the ADOT Rainfall Averaging Tool. A GIS watershed boundary file along with the ADOT Rainfall Averaging Tool was used to determine the D-D-F data for the watershed. The D-D-F table for the watershed shown in **Figure 1–2** is developed using ADOT Rainfall Averaging Tool. The resulting D-D-F table is presented in **Table 1–3**.

Table 1–3 Rainfall Averaged Pavo Kug Wash Watershed D-D-F Table

	Average Precipitation values (inches)							
Duration	1-year	2-year	5-year	10-year	25-year	50-year	100-year	500-year
5-min	0.313	0.406	0.537	0.632	0.754	0.843	0.931	1.125
10-min	0.476	0.618	0.817	0.962	1.148	1.283	1.416	1.712
15-min	0.590	0.766	1.013	1.192	1.423	1.591	1.756	2.122
30-min	0.794	1.031	1.364	1.605	1.917	2.142	2.365	2.858
60-min	0.983	1.276	1.689	1.987	2.372	2.651	2.926	3.537
2-hour	1.104	1.420	1.855	2.187	2.635	2.976	3.324	4.128
3-hour	1.159	1.473	1.899	2.239	2.716	3.087	3.476	4.423
6-hour	1.348	1.696	2.142	2.509	3.019	3.423	3.847	4.862
12-hour	1.563	1.964	2.469	2.880	3.451	3.897	4.362	5.480
24-hour	1.923	2.401	2.988	3.458	4.097	4.591	5.098	6.310

7. The I-D-F table for the watershed shown in Figure 1–2 is developed using D-D-F table from Step 6 of the solution. The precipitation value is divided by the duration value to get the rainfall intensity. The resulting I-D-F table is presented in Table 1–4.

Table 1–4 Rainfall Averaged Pavo Kug Wash Watershed I-D-F Table.

	Average Rainfall Intensity (inches/hour)							
Duration	**1-year**	**2-year**	**5-year**	**10-year**	**25-year**	**50-year**	**100-year**	**500-year**
5-min	3.756	4.872	6.444	7.584	9.048	10.116	11.172	13.500
10-min	2.856	3.708	4.902	5.772	6.888	7.698	8.496	10.272
15-min	2.360	3.064	4.052	4.768	5.692	6.364	7.024	8.488
30-min	1.588	2.062	2.728	3.210	3.834	4.284	4.730	5.716
60-min	0.983	1.276	1.689	1.987	2.372	2.651	2.926	3.537
2-hour	0.552	0.710	0.928	1.094	1.318	1.488	1.662	2.064
3-hour	0.386	0.491	0.633	0.746	0.905	1.029	1.159	1.474
6-hour	0.225	0.283	0.357	0.418	0.503	0.571	0.641	0.810
12-hour	0.130	0.164	0.206	0.240	0.288	0.325	0.364	0.457
24-hour	0.080	0.100	0.125	0.144	0.171	0.191	0.212	0.263

Chapter 2
RATIONAL METHOD

This chapter contains the following details:

- Procedures and instructions for using the Rational Method. These procedures and instructions include the development of site-specific I-D-F curves, a time of concentration equation, and graphs for the selection of the runoff coefficient.

2.1 INTRODUCTION

The Rational Method relates rainfall intensity, a runoff coefficient and a drainage area size to the direct runoff from the drainage area.

Three basic assumptions of the Rational Method are:

1. The frequency of the storm runoff is the same as the frequency of the rainfall producing the runoff (i.e., 25-year runoff event results from a 25-year rainfall event).
2. The peak runoff occurs when all parts of the drainage area are contributing to the runoff.
3. Rainfall is uniform over the watershed.

The Rational Method can be used to estimate peak discharges and runoff volume for uniform drainage areas of 160 acres or less. The Rational Method is usually used to size drainage structures for the peak discharge of a selected return period.

The Rational Method is based on the equation:

$$Q = CiA \tag{2.1}$$

where:
Q = the peak discharge of selected return period (cfs),
C = the runoff coefficient,
i = the average rainfall intensity of calculated rainfall duration for the selected rainfall return period (inches/hr), and
A = the contributing drainage area (acres).

ADOT has developed the ADOT Rational Method software tool to facilitate the computation of discharges using the Rational Method. This tool incorporates the ADOT Rational Method as described in this manual. The tool provides an easy-to-use graphical interface with integrated help menus that can be used as aids for entering the various input parameters and reviewing the computed discharges. The computations are performed for a full range of storm frequencies (2-yr through 100-yr). The ADOT Rational Method tool is available through the ADOT web site.

2.2 PROCEDURE

1. The runoff coefficient (*C*) should be selected based on the character of the existing land surface or the projected character of the land surface under future development conditions. In some situations, it may be necessary to estimate *C* for both existing and future conditions.

2. Land-use must be carefully considered because the evaluation of land-use will affect both the estimation of *C* and also the estimation of the watershed time of concentration (T_c).

3. The peak discharge (*Q*) is generally quite sensitive to the calculation of time of concentration (T_c). Therefore, care must be exercised in obtaining the most appropriate estimate of $T_{c.}$

4. Both *C* and the rainfall intensity (*i*) vary with return period. Therefore, *C* and *i* should be estimated separately for each return period evaluated.

5. Since the T_c equation is a function of rainfall intensity (*i*), T_c will also vary for different flood return periods.

2.2.1 Applications and Limitations

1. The total drainage area <u>must be less than</u> or equal to 160 acres.

2. T_c shall <u>not</u> exceed 60 minutes (maximum T_c). The 10 minute rainfall intensity *i* shall be used when the computed T_c is less than 10 minutes.

3. The land-use of the contributing area must be consistent over the entire drainage area for each concentration point. The user should delineate drainage areas with a single land-use where possible. If there are minor variations in land-use, the *C* coefficient may be weighted to reflect the different conditions in the watershed.

4. For watersheds with minor variations in land-use types, the watershed resistance coefficient (K_b) value should be selected based on the predominant land-use type.

5. The contributing drainage area cannot have drainage structures or other facilities in the area that would require flood routing to correctly estimate the discharge at the point of interest. If flood routing is necessary, use HEC-HMS.

6. Drainage areas that do not meet the above conditions will require the use of HEC-HMS or FLO-2D to estimate flood discharges.

2.2.2 Estimation of Area (A)

An adequate topographic map is needed to define the drainage boundary and to estimate the drainage area (A), in acres. The map should be supplemented with aerial photographs, especially if the area is developed. If the area is presently undeveloped but will undergo development, then land development plans and maps should be obtained because these may indicate a change in the drainage boundary due to road construction or land grade changes.

The delineation of the drainage boundary must be carefully determined. The contributing drainage area for a lower intensity storm does not always coincide with the drainage area for more intense storms. This discrepancy is particularly true for urban areas where roads can form a drainage boundary for small storms but more intense storm runoff can cross roadway crowns, curbs, and so forth, resulting in a larger contributing area. Floods on active alluvial fans and in distributary flow systems can result in increased contributing drainage areas during larger and more intense storms. It is generally prudent to consider the largest reasonable drainage area in such situations.

2.2.3 Estimation of Rainfall Intensity (i)

The rainfall intensity (i) in **Equation 2.1** is the average intensity in inches/hour for the period of maximum rainfall of a specified return period (frequency) having a duration equal to the time of concentration (T_c) for the drainage area. The frequency is usually specified by the design criteria or standard for the intended application. The rainfall intensity (i) is obtained from an I-D-F graph. A site-specific I-D-F graph should be developed using procedures described in **Chapter 1**.

If the calculated T_c is less than 10 minutes the 10 minute intensity should be used to calculate the peak flow.

2.2.4 Estimation of Time of Concentration (T_c)

Time of concentration for the Rational Method is estimated by the following equation:

$$T_C = 11.4\, L^{0.5}\, {K_b}^{0.52}\, S^{-0.31} i^{-0.38} \qquad 2.2$$

Note: Reference Papadakis and Kazan, 1987.

where:
- T_c = the time of concentration (hours),
- L = the length of the longest flow path (miles),
- K_b = the watershed resistance coefficient,
- S = the slope of the longest flow path (ft/mile), and
- i = the average rainfall intensity for a duration of rainfall equal to T_c (inches/hr) (the same i as **Equation 2.1** unless T_c is less than 10 minutes, in which case the i of **Equation 2.1** is for a 10-minute duration).

The longest flow path will be estimated from the best available map and the length (L) measured from the map.

The slope (S), in ft/mile, will be calculated by one of following methods:

1. If the longest flow path has a uniform gradient with no appreciable grade breaks, then the slope is calculated by **Equation 2.3**:

$$S = \frac{H}{L} \tag{2.3}$$

where: H = the change in elevation along L (feet), and
L = the length of the longest flow path (miles).

2. If the longest flow path does not have a uniform gradient or has distinct grade breaks, then the slope is calculated by **Equation 2.4**:

$$S = 5{,}280 \left(\frac{d}{j}\right)^2 \tag{2.4}$$

where: d = the length of the longest flow path (feet),
j = $\sum \left(\frac{{d_i}^3}{H_i}\right)^{1/2}$

Note: Reference, Pima County Department of Transportation and Flood Control District, September 1979.

and d_i = an incremental change in length along the longest flow path (feet) and
H_i = an incremental change in elevation for each length segment d_i (feet).

If the longest flow path has a distinct vertical or near vertical grade break, then height of the vertical or near vertical grade break should be removed from the slope calculation.

The resistance coefficient (K_b) is selected from **Table 2–1**. Use of **Table 2–1** requires a classification as to the predominant landform and a determination of the nature of runoff, whether in a defined drainage network of rills, gullies, channels, etc., or predominantly as overland flow.

Table 2–1 Resistance Coefficient (K_b) For Use With The Rational Method T_c Equation

Description of Landform	K_b Defined Drainage Network	K_b Overland Flow Only
Mountain, with forest and dense ground cover (average slopes – 50% or greater)	0.15	0.30
Mountain, with rough rock and boulder cover (average slopes – 50% or greater)	0.12	0.25
Foothills (average slopes – 10% to 50%)	0.10	0.20
Alluvial fans, Pediments and Rangelands (average slopes – 10% or less)	0.05	0.10
Irrigation Pastures [a]	-	0.20
Tilled Agricultural Fields [a]	-	0.08
URBAN		
Residential, L is less than 1,000 ft[b]	0.04	
Residential, L is greater than 1,000 ft[b]	0.025	
Grass; parks, cemeteries, etc. [a]	-	0.20
Bare ground; playgrounds, etc. [a]	-	0.08
Paved; parking lots, etc. [a]	-	0.02

Notes: a – No defined drainage network.
b – L is length in T_c equation. Streets serve as drainage network.

The solution for T_c using **Equation 2.2** is an iterative process, since the determination of (i) requires the knowledge of the value of T_c. Therefore, **Equation 2.2** will be solved by a successive approximation procedure. After L, K_b, and S are estimated and after the appropriate I-D-F graph is selected or prepared, a value for T_c will be estimated (a trial value) and (i) will be read from the I-D-F graph for the corresponding value of duration equivalent to T_c. That (i) will be used in **Equation 2.2** and T_c will be calculated. If the calculated value of T_c does not equal the trial value of T_c, then the process is repeated until the calculated and trial values of T_c are acceptably close (a difference of less than one minute is acceptable). The Rational Method should not be used if the calculated T_c is greater than 60 minutes. The ADOT Rational Method Tool can be used to perform the computation.

2.2.5 Selection of Runoff Coefficient (C)

The runoff coefficient (C) is selected from **Figure 2–1** through **Figure 2–6** depending on the watershed characteristics. **Figure 2–1** is the C graph to be used for urbanized (developed) watersheds. Select the appropriate curve in **Figure 2–1** based on an estimate of the percent of effective impervious area in the watershed. Effective impervious area is that area that will drain directly to the outlet without flowing over a pervious area (Refer to **Chapter 3**, **Section 3.1** and **Table 3–2**, for general discussion of estimating effective impervious areas). **Figure 2–1** through **Figure 2–6** are to be used for undeveloped (natural) watersheds in Arizona. In these figures, the C graphs are shown as functions of Hydrologic Soil Group (HSG) and percent vegetation cover. The Hydrologic Soil Group is used to classify soil according to its infiltration rate. The Hydrologic Soil Groups, as defined by USDA, Natural Resources Conservation Service (NRCS), 2007 are:

HSG	Definition
A	Soils in this group have low runoff potential when thoroughly wet. Water is transmitted freely through the soil. Group A soils typically have less than 10 percent clay and more than 90 percent sand or gravel and have gravel or sand textures. Some soils having loamy sand, sandy loam, loam or silt loam textures may be placed in this group if they are well aggregated, of low bulk density, or contain greater than 35 percent rock fragments.
B	Soils in this group have moderately low runoff potential when thoroughly wet. Water transmission through the soil is unimpeded. Group B soils typically have between 10 and 20 percent clay and 50 to 90 percent sand and have loamy sand or sandy loam textures. Some soils having loam, silt loam, silt, or sandy clay loam textures may be placed in this group if they are well aggregated, of low bulk density, or contain greater than 35 percent rock fragments.
C	Soils in this group have moderately high runoff potential when thoroughly wet. Water transmission through the soil is somewhat restricted. Group C soils typically have between 20 and 40 percent clay and less than 50 percent sand and have loam, silt loam, sandy clay loam, clay loam, and silty clay loam textures. Some soils having clay, silty clay, or sandy clay textures may be placed in this group if they are well aggregated, of low bulk density, or contain greater than 35 percent rock fragments.
D	Soils in this group have high runoff potential when thoroughly wet. Water movement through the soil is restricted or very restricted. Group D soils typically have greater than 40 percent clay, less than 50 percent sand, and have clayey textures. In some areas, they also have high shrink-swell potential. All soils with a depth to a water impermeable layer less than 50 centimeters [20 inches] and all soils with a water table within 60 centimeters [24 inches] of the surface are in this group, although some may have a dual classification, as described in the next section, if they can be adequately drained.

The percent vegetation cover is the percent of land surface that is covered by vegetation. Vegetation cover is evaluated on plant basal area for grasses and forbs, and on canopy cover for trees and shrubs. (More detailed information on estimating vegetative cover is provided in Appendix C).

Information on Hydrologic Soil Group and percent vegetation cover can usually be obtained from the detailed NRCS soil surveys. When detailed soil surveys are not available for the watershed, then the NRCS general soil maps and accompanying reports for each county are to be used. A site visit is encouraged to confirm watershed and soil conditions.

A different C value for existing conditions and for anticipated future conditions may be needed if the watershed is undergoing development. Estimation of peak discharges for various conditions of development in the drainage area or for different flood frequencies will also require separate estimates of T_c for each assumed land-use condition and for each flood return period.

For watersheds with varying land-uses, the *C* value should be weighted to reflect the different conditions that exist within the watershed using **Equation 2.5**. Additional guidance is provided in **Section 2.2.1**.

$$C_w = \left(\sum_{i=1}^{n} C_i A_i\right) \Big/ A_t \qquad 2.5$$

where:

C_w	=	area-weighted C value for subbasin,
n	=	number of subareas within the subbasin,
C_i	=	sub-area C value,
A_i	=	area of sub-area (acres) and,
A_t	=	total area of the subbasin (acres).

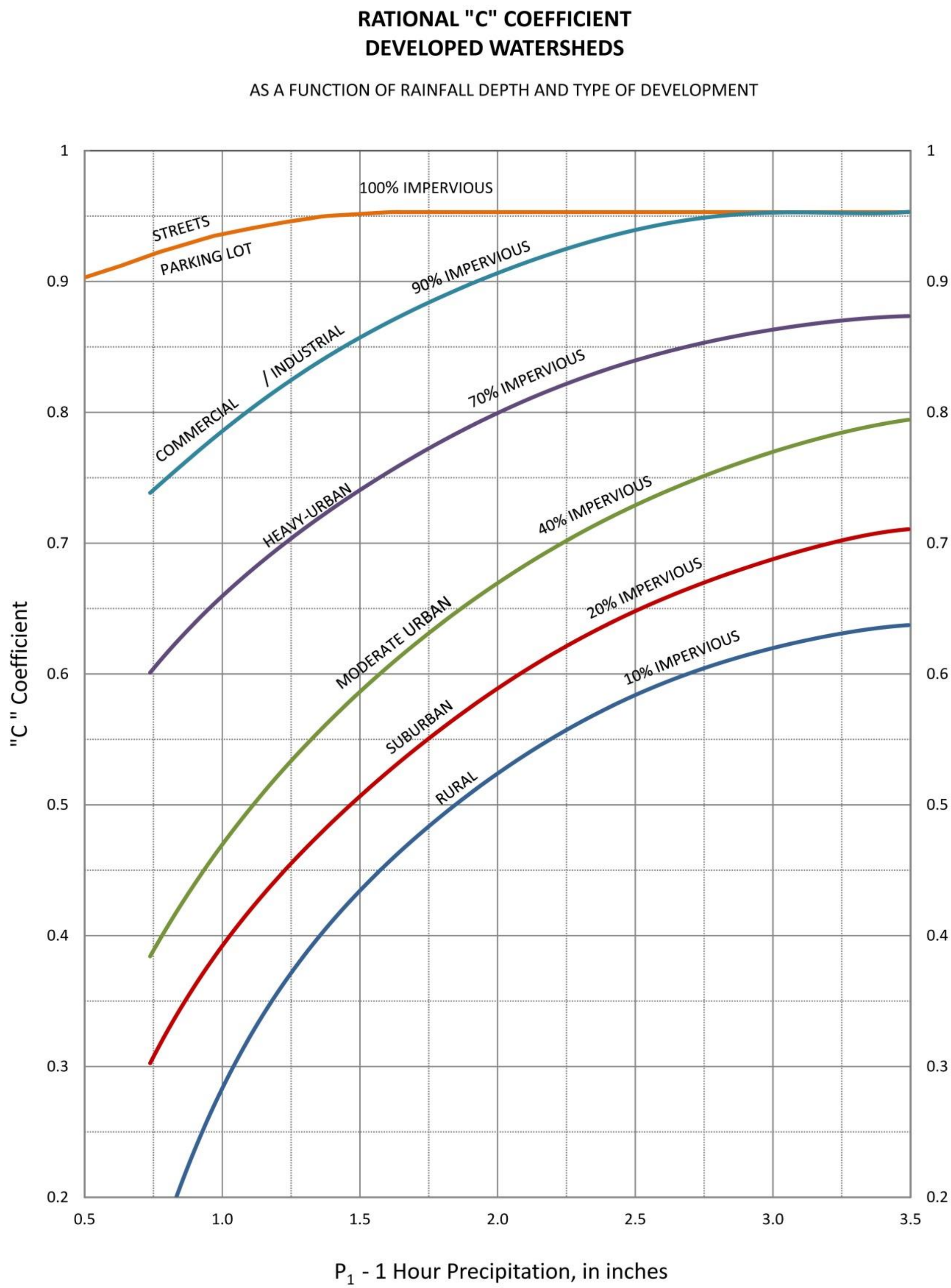


Figure 2–1 Rational "C" Coefficient – Developed Watersheds

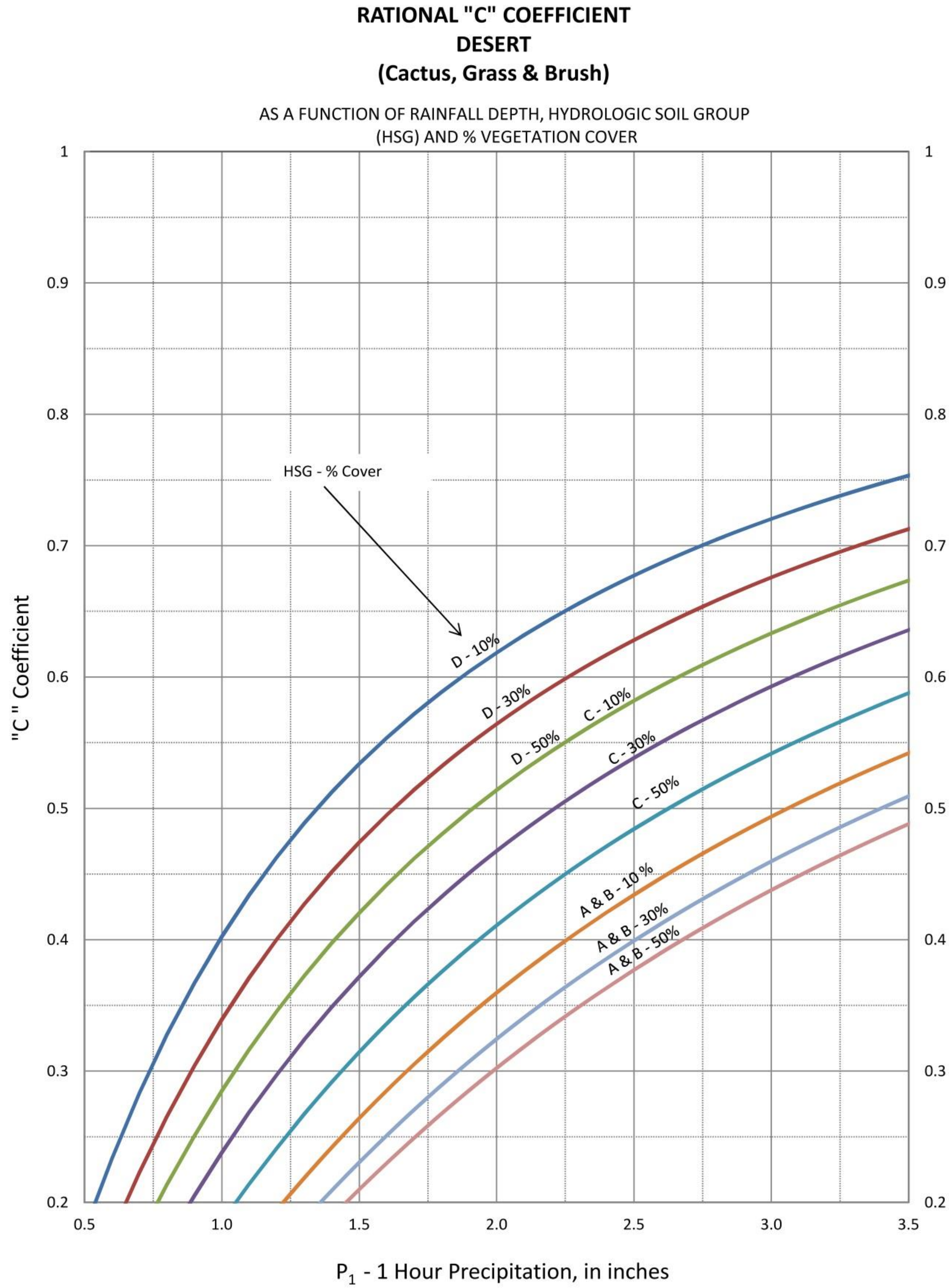


Figure 2–2 Rational "C" Coefficient – Desert

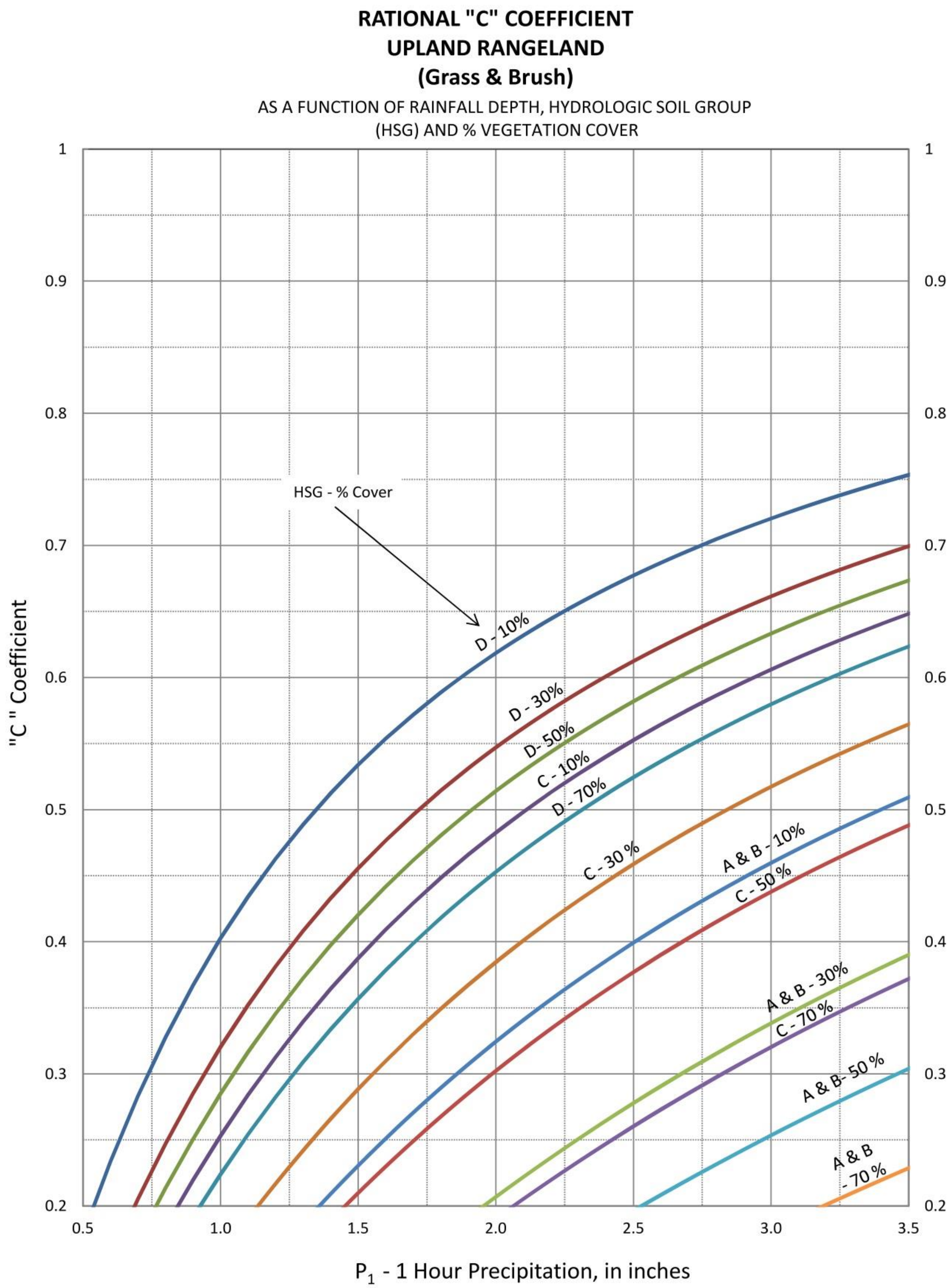


Figure 2–3 Rational "C" Coefficient – Upland Rangeland

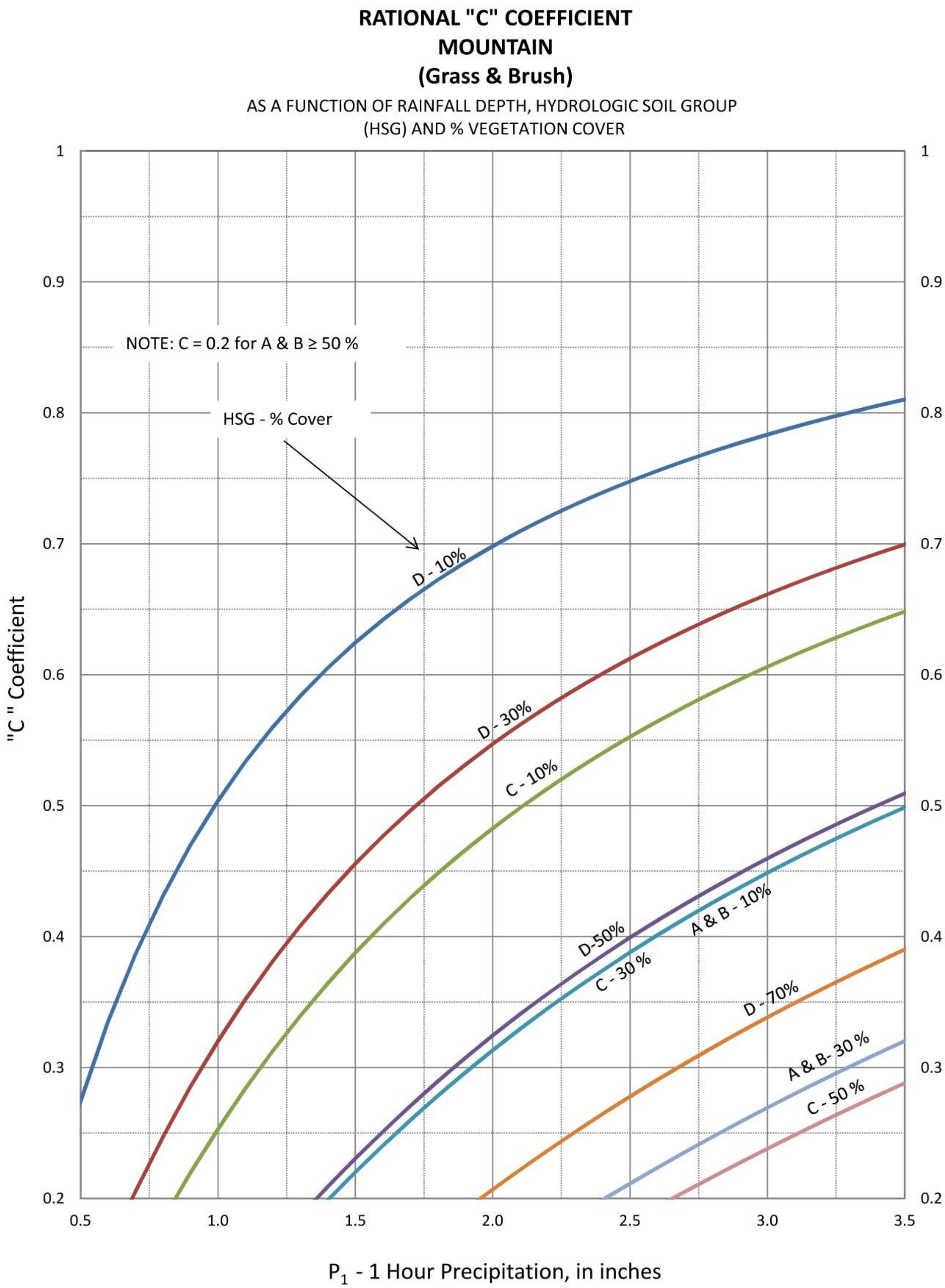


Figure 2–4 Rational "C" Coefficient – Mountain (Grass & Brush)

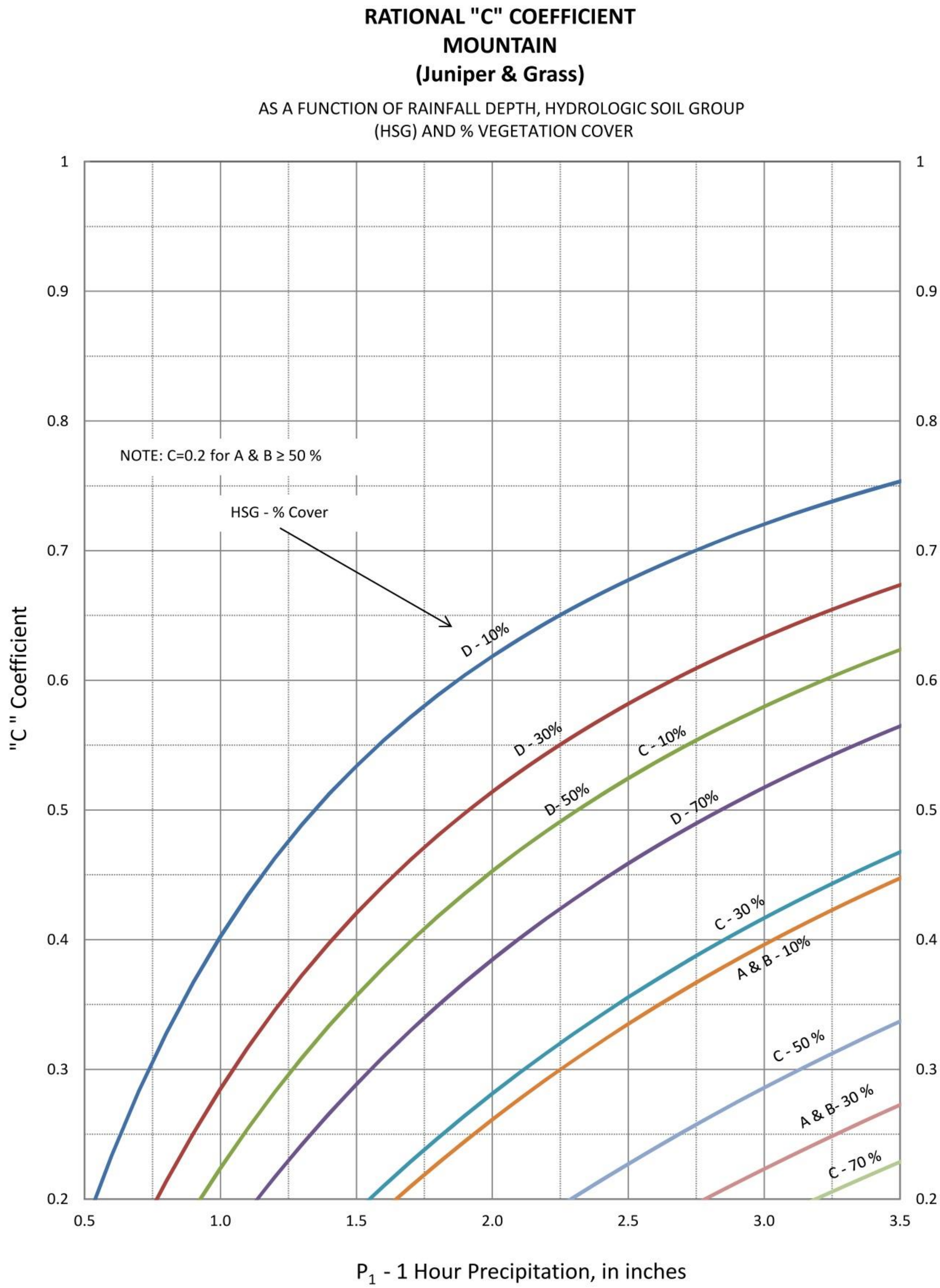


Figure 2–5 Rational "C" Coefficient – Mountain (Juniper & Grass)

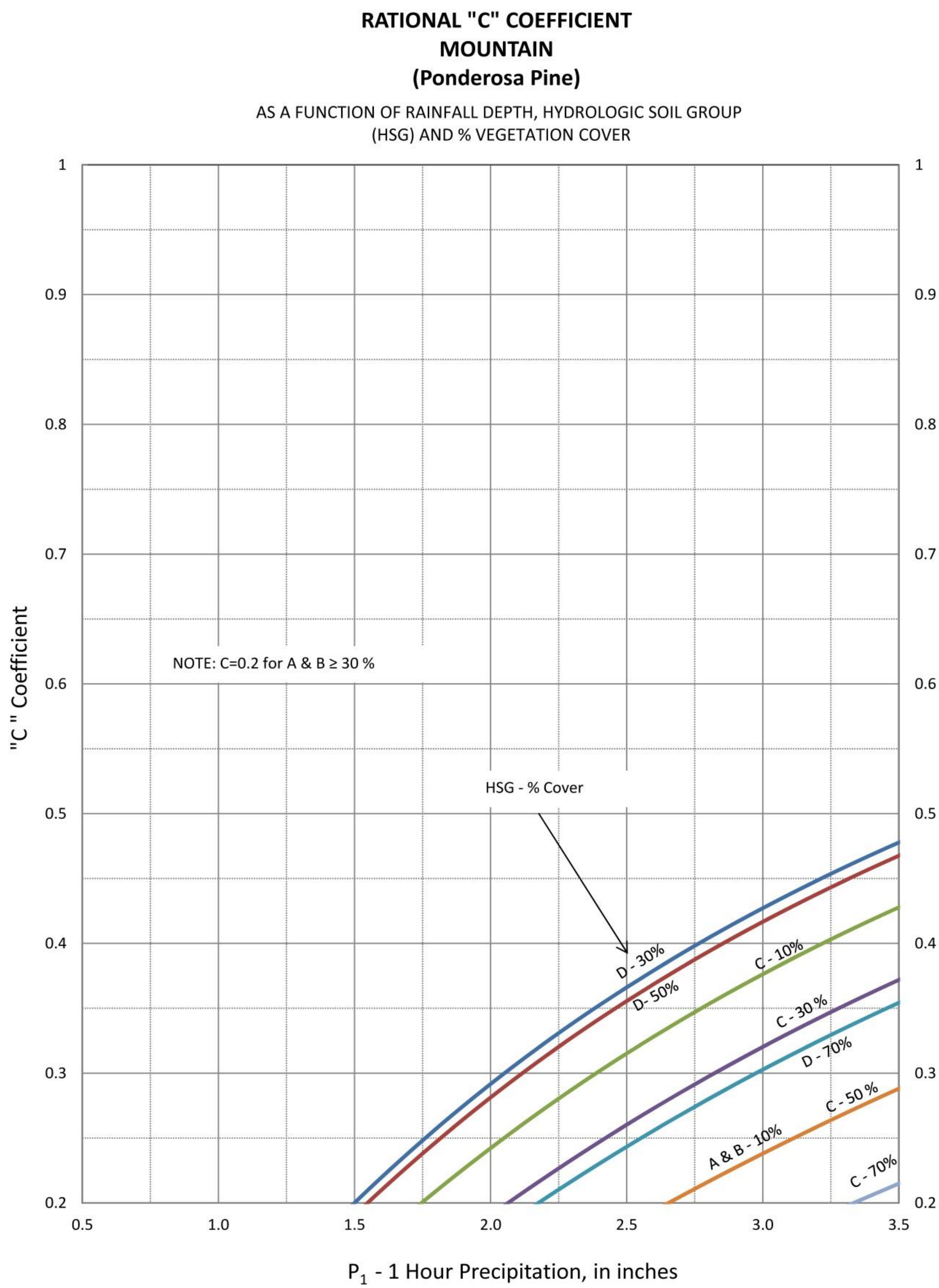


Figure 2–6 Rational "C" Coefficient – Mountain (Ponderosa Pine)

2.2.6 Estimation of Hydrograph Shape

Estimation of a hydrograph shape for Rational Method computations is presented in ADOT Drainage Design Manual, Volume 3 – Hydraulics.

2.3 RATIONAL METHOD INSTRUCTIONS

2.3.1 Estimating Peak Discharge

1. Determine the size of the contributing drainage area (A), in acres.

2. Develop the I-D-F graph using the NOAA website.

3. Select the desired return period(s). See ADOT Drainage Design Manual Volume 1 - Policy & Guidelines for Drainage Frequency Classes.

4. Determine the 1-hour rainfall depth (P_1) for each return period.

5. Determine the length of the longest flow path (L), and watershed resistance coefficient (K_b) as outlined in **Section 2.2.4**.

6. Calculate the slope as outlined in **Section 2.2.4** using **Equation 2.3** or **2.4**.

7. Solve T_c (**Equation 2.2**) by a successive approximation procedure. After L, K_b, and S are estimated and after the appropriate I-D-F graph is selected or prepared, a value for T_C will be estimated (a trial value) and (i) will be read from the I-D-F graph for the corresponding value of duration = T_c. That (i) will be used in **Equation 2.2** and T_c will be calculated. If the calculated value of T_c does not equal the trial value of T_c, then the process is repeated until the calculated and trial values of T_c are acceptably close (a difference of less than 1 minute is acceptable). T_c shall not exceed 60 minutes nor shall a T_c less than 10 minutes be used.

8. Estimate C:

   a. If the watershed is developed, use **Figure 2–1**. This will require an estimate of development type and percent effective impervious area. C is selected as a function of P_1 and type of development.

   b. If the watershed is undeveloped, use **Figure 2–2** through **Figure 2–6**. This will require an estimate of Hydrologic Soil Group (HSG) for the underlying soils types, A through D, as determined from NRCS soils reports, as well as an estimate of percent vegetation cover, as described in Appendix C. C is selected as a function of P_1 and HSG percent vegetation cover.

   c. If the watershed is comprised of more than one land-use type, use **Equation 2.5** and guidance provided in **Section 2.2** to determine a composite weighted C value for the watershed.

9. Calculate the peak discharge using **Equation 2.1**.

2.4 EXAMPLE

Example No. 2-1 Rational Method

Problem:

Calculate the 100-year peak discharge for a 60 acre, single-family residential (about 20% effective impervious area) watershed in Tucson. The following are the watershed characteristics:

A = 60 acres

S = 25 ft/mile

L = 0.7 miles

The following were obtained for the watershed:

P_1= 2.5 inches from the NOAA Atlas 14

K_b = 0.025 from **Table 2–1**

C = 0.65 from **Figure 2–1**

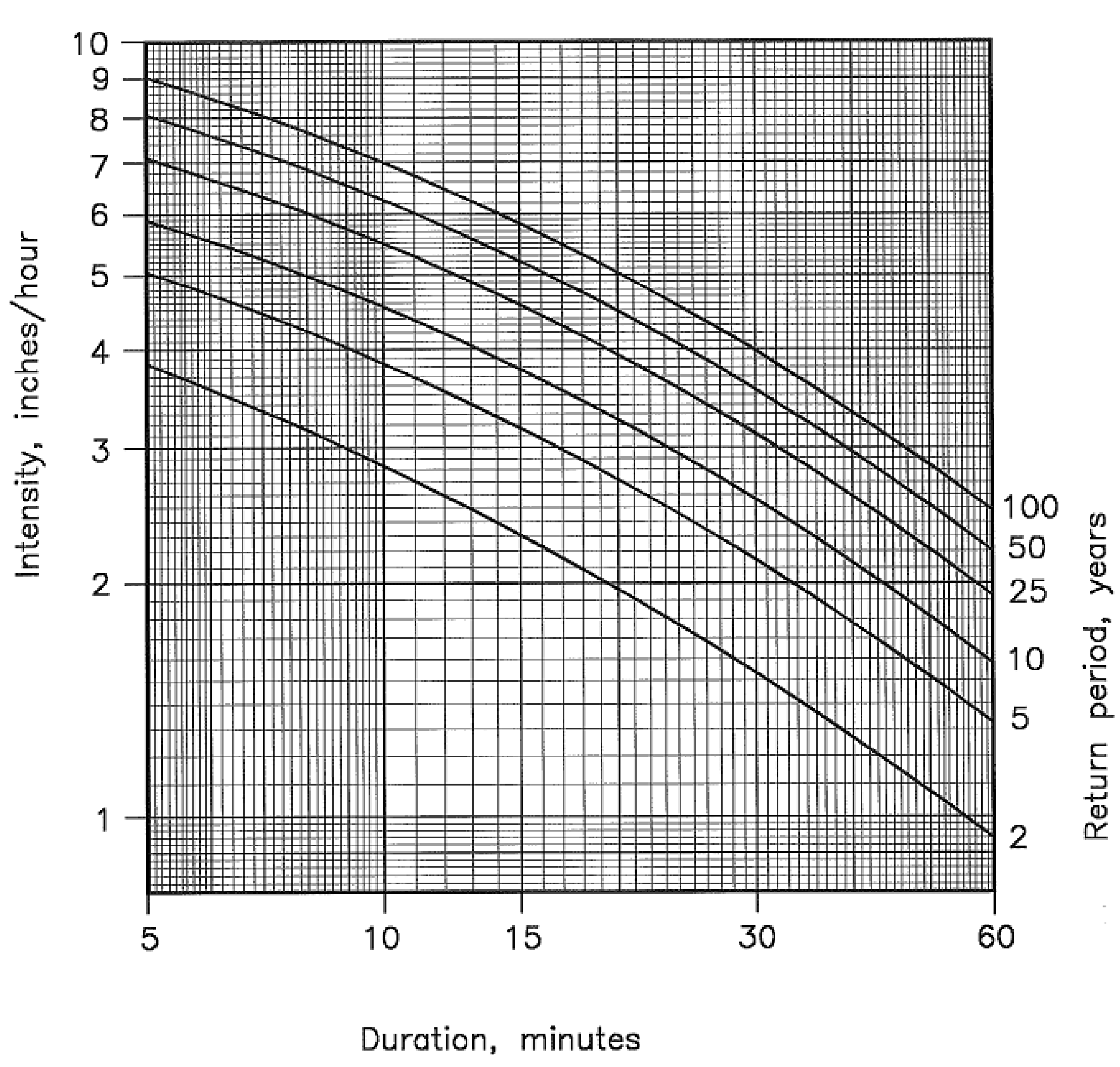


Figure 2–7 Rainfall I-D-F graph for Rational Method Example

Solution #1, Direct Computation:

This example is solved using a site-specific I-D-F graph (Figure 2–7)

Solve for T_c:

$$T_c = 11.4\, L^{0.5}\, {K_b}^{0.52}\, S^{-0.31} i^{-0.38}$$

$$T_c = 11.4\, (0.7)^{0.5}\, (0.025)^{0.52}\, 25^{-0.31} i^{-0.38}$$

$$T_c = 0.52\, i^{-0.38}$$

Table 2–2 Successive Approximation for T_c		
Trial T_c, hr	i in/hr	Calculated T_c, hr
0.75	3.0	0.34
0.30	5.4	0.27
0.27	5.8	0.26 OK

Calculate Q:

$$Q = C\,i\,A$$

$$= (0.64)(5.8)(60)$$

$$= 223 \text{ cfs}$$

Solution #2, ADOT Rational Method Tool:

This example is solved using the ADOT Rational Method Tool.

Watershed Data Input screen:

Drainage Area Type:	Developed (Figure 2–1)
% Impervious:	20
Drainage Area (Acres):	60
Slope of Longest Flow Path (ft/mile):	25 (Elevation Change 17.5 ft.)
Length of longest flow path (miles):	0.7

Parameters for Resistance Coefficient (K_b) Estimation (Table 2–1)

Landform Type:	Urban – Residential, L > 1000 ft
Drainage/Flow Type:	Defined Drainage Network

Subbasin Data C-Factor Input Tab

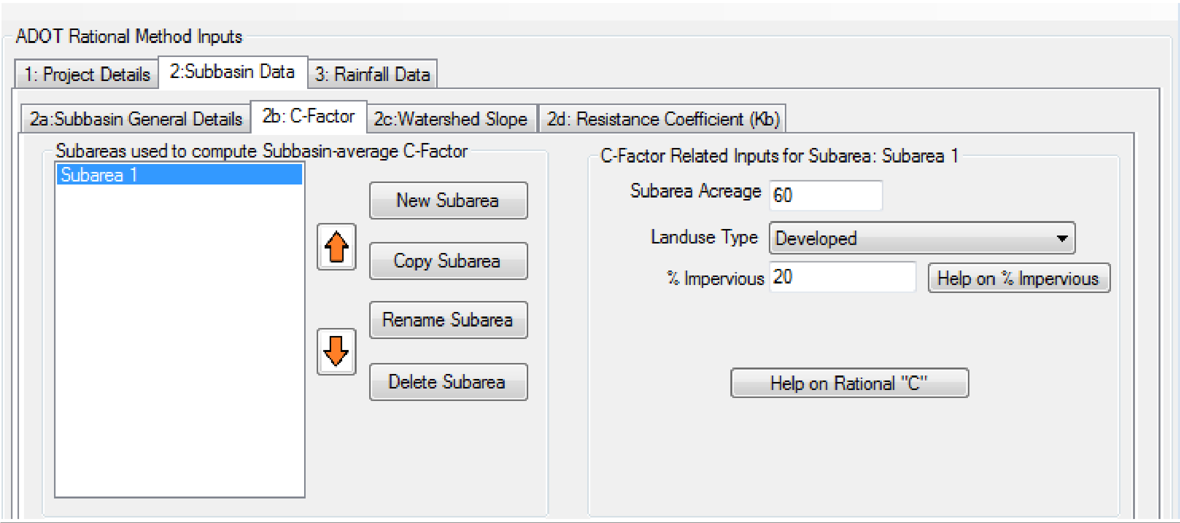


Figure 2–8 ADOT Rational Method Tool, C-Factor Input Tab

Subbasin Data Watershed Slope Input Tab

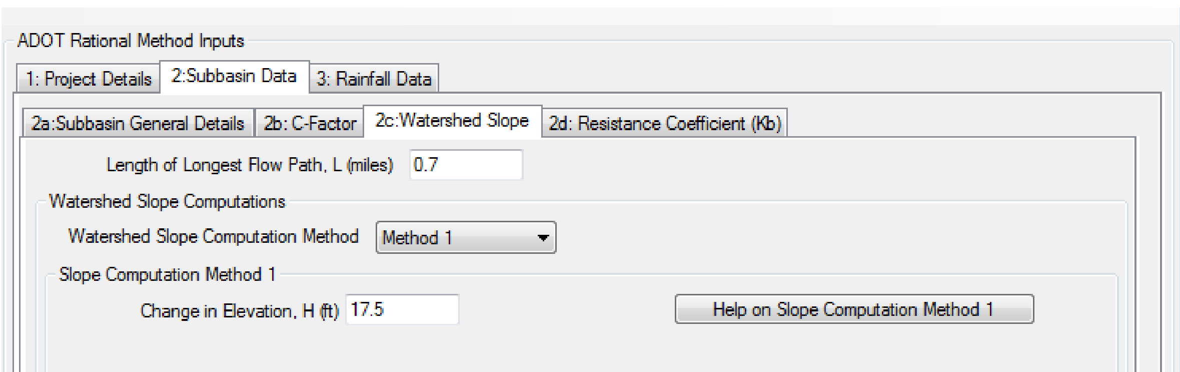


Figure 2–9 ADOT Rational Method Tool, Watershed Slope Input Tab

Subbasin Data Resistance Coefficient (K_b) Input Tab

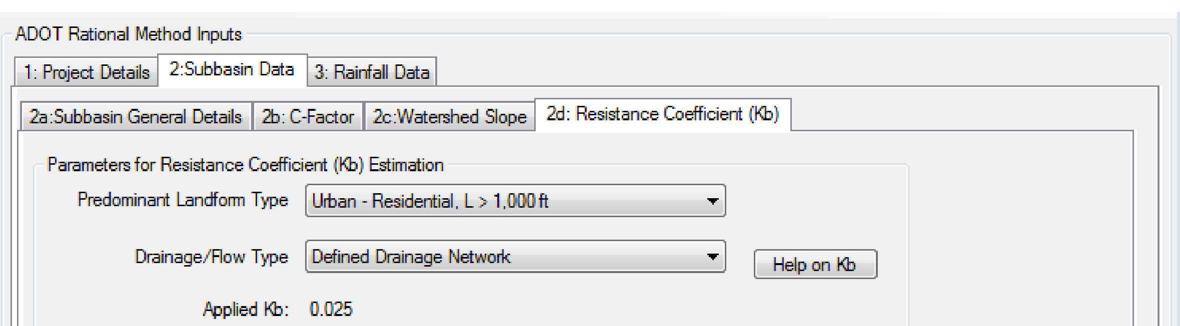
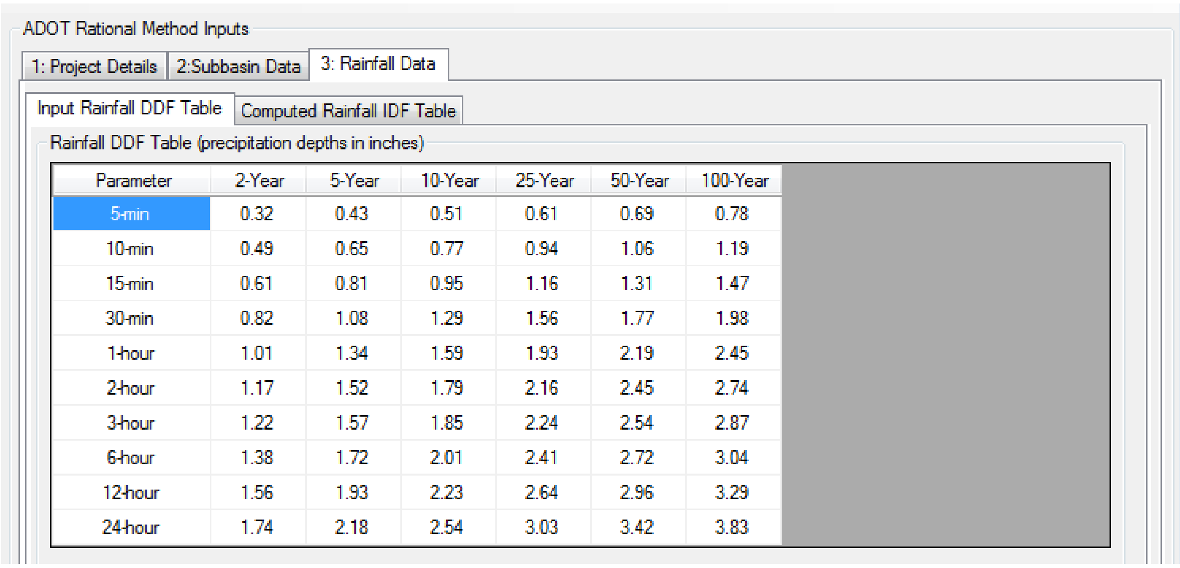


Figure 2–10 ADOT Rational Method Tool, Resistance Coefficient (K_b) Input Tab

Rainfall Data Input Tab



Parameter	2-Year	5-Year	10-Year	25-Year	50-Year	100-Year
5-min	0.32	0.43	0.51	0.61	0.69	0.78
10-min	0.49	0.65	0.77	0.94	1.06	1.19
15-min	0.61	0.81	0.95	1.16	1.31	1.47
30-min	0.82	1.08	1.29	1.56	1.77	1.98
1-hour	1.01	1.34	1.59	1.93	2.19	2.45
2-hour	1.17	1.52	1.79	2.16	2.45	2.74
3-hour	1.22	1.57	1.85	2.24	2.54	2.87
6-hour	1.38	1.72	2.01	2.41	2.72	3.04
12-hour	1.56	1.93	2.23	2.64	2.96	3.29
24-hour	1.74	2.18	2.54	3.03	3.42	3.83

Figure 2–11 ADOT Rational Method Tool, Rainfall Data Input Tab

Table 2–3 ADOT Rational Method Tool Results Summary

Parameter	2-Year	5-Year	10-Year	25-Year	50-Year	100-Year
Discharge-*Q* (cfs)	50	80	107	152	184	221
Rational Coefficient-*C*	0.40	0.47	0.52	0.58	0.61	0.64
Rainfall intensity-*i* (in/hr)	1.95	2.81	3.44	4.36	5.04	5.76
Time of Concentration- T_c (minutes)	24.1	21	19.4	17.7	16.8	15.9

Chapter 3
RAINFALL LOSSES

This chapter contains the following details:

- The Green and Ampt Loss Rate method (Green & Ampt) is the primary method for computation of rainfall losses. Procedures for development of watershed average Green and Ampt parameters for use in HEC-HMS.
- Initial and Constant Loss Rate method (Initial & Constant) is the secondary method for computing rainfall losses.
- Example input and computations.

3.1 INTRODUCTION

This chapter presents the use of the Green and Ampt loss parameters as the primary method for computation of rainfall losses in HEC-HMS. The recommended values for Green and Ampt parameters are based on work by Saxton & Rawls (2006), which is an extension of the original work by Rawls, Brackensiek, & Miller (1983) used to develop Green and Ampt parameters in the 1993 ADOT Hydrology Manual.

To assist application of the new Green and Ampt parameters and procedures, a GIS database containing the new loss parameters for the digital Natural Resource Conservation Service (NRCS) detailed soil surveys (SSURGO) and the Statewide (GSM) Soil Survey (STATSGO) was developed for all of Arizona using the Saxton & Rawls procedures. Tables of the computed map unit values are provided in Appendix B. GIS datasets for SSURGO and STATSGO soils surveys listed in Table B-1 can be downloaded at:

http://www.azdot.gov/business/engineering-and-construction/roadway-engineering/drainage-design.

The Initial and Constant Loss method is the secondary loss rate method, and is recommended for areas where Green and Ampt is not appropriate. The Initial and Constant loss rate method is to be used for watersheds or subbasins where rainfall losses are known to be controlled by factors other than soil texture and vegetation cover, such as in sand or cinder soils.

Rainfall excess is that portion of the total rainfall depth that drains directly from the land surface by overland flow. In terms of mass balance, precipitation less rainfall losses equals rainfall excess. Rainfall losses are the sum of all the portions of precipitation that do not become direct runoff. These include interception losses, surface depression storage, and infiltration.

The design rainfall is determined from the procedures in the Rainfall section. This chapter provides procedures to estimate the rainfall loss, so that the runoff volume from the applied rainfall can be determined. This chapter is only applicable when performing rainfall-runoff modeling with the HEC-HMS program. When using the Rational Method, it is not necessary to estimate rainfall losses by the procedures in this chapter because the "*C*" factor accounts for the effect of rainfall loss on the peak discharge and runoff volume.

The primary rainfall loss method requires the estimation of the surface retention loss (**Table 3–1**) and the rainfall infiltration loss by the Green and Ampt procedure in HEC-HMS. The Saxton & Rawls (2006) method is used to develop the Green and Ampt parameters needed for HEC-HMS. The Green and Ampt parameters, saturated hydraulic conductivity, soil suction, and soil moisture retention are computed as a function of soil properties as determined from the NRCS soil survey map unit descriptions. The Saxton & Rawls (2006) equations for these parameters incorporate the primary textural classification, as well as the percentages of gravel, sand, clay and organic matter. Hydraulic conductivity should be adjusted for the effects of vegetation ground cover, but is limited to a maximum value of 2.0 inches/hour for sand and loamy sand soils as indicated on **Figure 3–1**.

The secondary loss rate method, the Initial and Constant, is to be used for watersheds or subbasins where rainfall losses are known to be controlled by factors other than soil texture and vegetation cover, or for watersheds that are predominantly composed of sand. For example, the land surface of upland watersheds of the San Francisco Mountains near Flagstaff is generally composed of volcanic cinder overlain by forest duff. In such conditions, the Green and Ampt loss equation is not appropriate because infiltration is not controlled by soil texture, and infiltration rates may be as high as five inches per hour or more. Use of the secondary method requires adequate data or appropriate studies to verify the initial and constant loss rate parameters or to calibrate the model of the watershed.

To facilitate application of Green and Ampt, detailed and statewide NRCS soil data for Arizona were assembled and processed. GIS shapefiles of the NRCS soil surveys were produced with attribute data for each soil map unit, including saturated hydraulic conductivity, soil suction, and saturated soil moisture content and initial moisture content values for wilting point (dry) and field capacity (normal) conditions.

These datasets can be used as a source for Green and Ampt parameters. Although the use of GIS tools is not required, using the GIS data provided with this manual will facilitate the modeling process.

Tables with Green and Ampt parameters for each map unit are provided in Appendix B. GIS datasets for SSURGO and STATSGO soils surveys listed in Table B-1 can be downloaded at http://www.azdot.gov/business/engineering-and-construction/roadway-engineering/drainage-design

3.2 GREEN AND AMPT LOSS RATE METHOD

Parameter values for design should be based on reasonable estimates of watershed conditions that would minimize rainfall losses. A range of watershed conditions may need to be identified for evaluation of runoff.

Two sources of information are used to determine bare ground hydraulic conductivity, soil suction and initial moisture content for use in the Green and Ampt infiltration equation. The primary source to be used, is the detailed soil survey data provided in this manual. The secondary source, for areas where the detailed soil surveys are not available, is the general soil map data provided in this manual. Loss rate parameter values are provided in Appendix B for each soil map unit in each soil survey in Arizona.

Green and Ampt parameters are defined as follows:

Initial Content (Dry)	Volumetric soil moisture content expressed as wilting point at start of rainfall, in inches
Initial Content (Normal)	Volumetric soil moisture content expressed as field capacity at start of rainfall, in inches
Saturated Content	Volumetric soil moisture content at saturation, in inches
Suction	Wetting front capillary suction, in inches
Conductivity	Bare ground effective hydraulic conductivity at natural saturation, in inches/hour
Impervious %	Effective impervious area, in percent

Most drainage areas or subbasins will be composed of several subareas of different soil map units. Therefore, the modeler needs to determine composite values for the Green and Ampt parameters to be applied to the drainage areas or subbasin. For the conductivity and soil suction parameters, the composite value is determined using the average of the area-weighted logarithms of the individual subarea values. Composite values of soil moisture content, surface retention loss, and effective impervious area values shall be computed by a simple arithmetic area-weighting procedure.

The composite $\overline{Conductivity}$ is calculated by Equation 3.1:

$$\overline{\text{Conductivity}} = \text{antilog}\left(\frac{\sum A_i \log Conductivity_i}{A_T}\right) \tag{3.1}$$

where:

$\overline{Conductivity}$	=	composite hydraulic conductivity , (inches/hour),
$Conductivity_i$	=	hydraulic conductivity of the soil in a subarea, (inches/hour),
A_i	=	size of a subarea, and
A_T	=	size of the drainage area or modeling subbasin.

A correction of conductivity for vegetation cover (Figure 3–1) is made after the composite value of conductivity is determined (Equation 3.1).

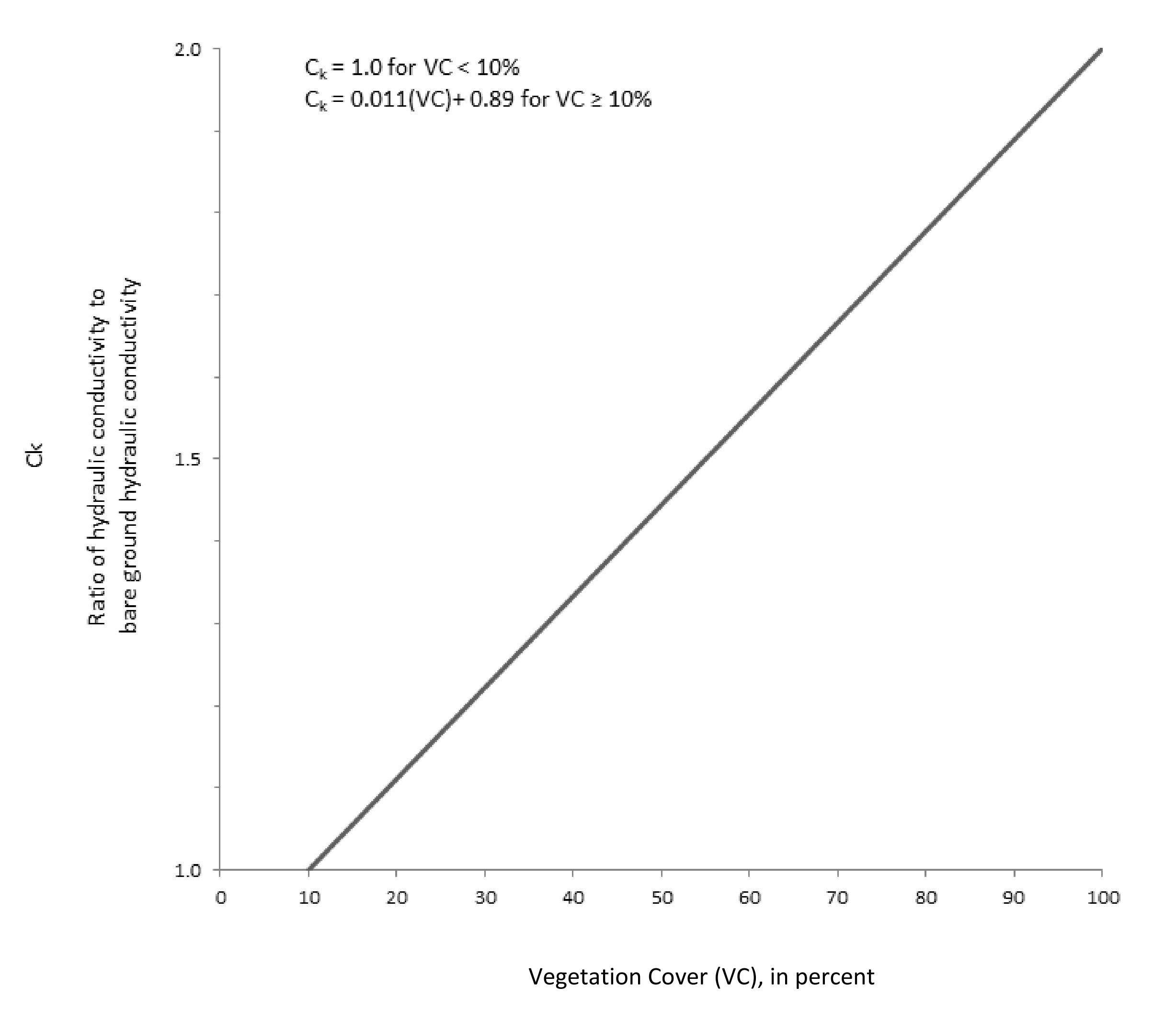


Figure 3–1 Effect of Vegetation Cover on Hydraulic Conductivity (source: DDM for Maricopa County, Volume 1, Hydrology)

Note that the adjusted conductivity is limited to two inches per hour (2 in/hr). In no case should a conductivity value greater than two be used.

Composite soil suction is computed using a log-averaging method in the same manner as conductivity, as shown in Equation 3.2 and with guidance presented in Section 3.3.1:

$$\overline{\text{Suction}} = \text{antilog}\left(\frac{\sum A_i \, log \, Suction_i}{A_T}\right) \quad 3.2$$

where:

$\overline{\text{Suction}}$	=	composite soil suction, (inches),
$Suction_i$	=	conductivity of the soil in a subarea, (inches),
A_i	=	size of a subarea, and
A_T	=	size of the drainage area or modeling subbasin.

Composite soil moisture contents for both initial and saturated condition are computed using a simple area-weighted procedure, as shown in Equation 3.2 and with guidance presented in Section 3.3.3.

$$\overline{SMC} = \frac{\sum A_i SMC_i}{A_T} \qquad 3.3$$

where:

where:	$\overline{SMC}$	=	composite soil moisture content, (inches),
	SMC_i	=	soil moisture content of the soil in a subarea,
	A_i	=	size of a subarea, and
	A_T	=	size of the drainage area or modeling subbasin.

Equation 3.3 applies to both the initial and saturated soil moisture content. The initial soil moisture content should be selected based on land cover type. The initial soil moisture content for natural areas should be the wilting point (dry). For most urban land cover types, the field capacity (normal) value should be used for the initial soil content. For irrigated agricultural areas, the initial soil moisture content should be set equal to the saturated content value.

Composite surface retention losses (*IA*) are computed using a simple area weighted procedure as shown in Equation 3.4 and with guidance presented in Section 3.3.1.

$$\overline{IA} = \frac{\sum A_i IA_i}{A_T} \qquad 3.4$$

where:	$\overline{IA}$	=	composite surface retention loss,
	IA_i	=	surface retention loss on the soil in a subarea,
	A_i	=	size of a subarea, and
	A_T	=	size of the drainage area or modeling subbasin.

Composite effective impervious area (Impervious %) is computed using a simple area weighted procedure as shown in Equation 3.5 and with guidance presented in Section 3.3.3.

$$\overline{Impervious\ \%} = \frac{\sum A_i\ Impervious\ \%_i}{A_T} \qquad 3.5$$

where:	$\overline{Impervious\ \%}$	=	composite Impervious %,
	$Impervious\ \%_i$	=	Impervious % on the soil in a subarea,
	A_i	=	size of a subarea, and
	A_T	=	size of the drainage area or modeling subbasin.

3.3 APPLICATION OF GREEN AND AMPT IN HEC-HMS

In HEC-HMS, the selection of Green and Ampt as the “Loss Method” is made on the “Subbasin” tab within the Basin Model. The values of the Green and Ampt parameters are specified under the “Loss” tab for each subbasin. Figure 3–2 through Figure 3–4 show an example of the recommended Green and Ampt loss parameter input for HEC-HMS (Figure 4–3, pg. 4-9). Figure 3–2 shows an HEC-HMS data entry screen with the Simple Surface “Surface Method” and Green and Ampt “Loss Method” selected on the “Subbasin” tab. Figure 3–3 shows the surface retention storage input on the “Surface” tab. Figure 3–4 shows an example of the Green and Ampt parameters on the “Loss” tab. Some additional discussion for input on each tab is provided in the following sections.

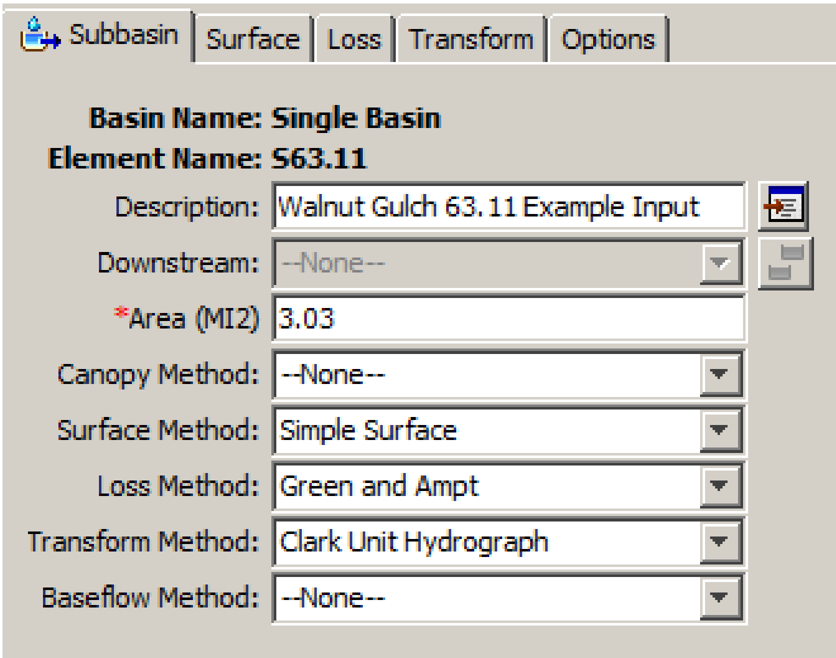


Figure 3–2 Example of Subbasin Tab in the HEC-HMS Component Editor

3.3.1 Surface Retention Losses (Surface Method & Surface Tab)

In HEC-HMS 3.5, the surface retention loss has two parameters specified on the “Surface” Tab – “initial storage (%)” and “max storage (in)”. The “max storage (in)” is to be taken as the sum of all initial losses including surface depression storage and interception losses. For the single event, frequency modeling that is the focus of this manual, lumping initial storage losses as surface retention is adequate.

Interception losses could be specified separately, if known separately, by additional use of the Canopy Method. However, the tables from existing guidance generally include both losses together. Therefore, the more simplified approach of lumping all initial losses into the “Surface Method” is recommended. In special circumstances where interception losses are believed to be significant and separable from surface depression losses, the "Canopy Method" may also be applicable. These circumstances might include heavily forested areas where significant tree canopy captures additional rainfall preventing it from reaching the ground. Separating the two types of initial losses may be necessary for continuous modeling or forensic modeling of storms with multiple rainfall bursts and periods of little or no rainfall in between. In these cases, HEC-HMS is able to drain the surface depression storage during a simulation and make it available again for subsequent rainfall losses as compared to interception (canopy) losses that are filled only once in a single simulation.

The "Initial Storage (%)" will generally be taken as zero (0) percent for drainage design applications. For saturated soil conditions such as agricultural fields, the initial storage should be set at 100 percent.

Surface loss parameters for use with this manual are provided in **Table 3–1**. Additional sources can be found in Table 6-1 in USACE EM 1110-2-1417 (USACE, 1996), Table 7.7 in the Drainage Design Manual for Mohave County (MCFCD, 2009), and Table 4.2 in Drainage Design Manual for Maricopa County, Hydrology (FCDMC, 2013).

Table 3–1 Surface Retention Loss (Max Storage) for Various Land Surfaces in Arizona.

Land-use and/or Surface Cover	Surface Retention Loss (Max Storage), inches
Natural	
Desert and rangeland, flat slopes	0.35
Desert and rangeland, hill slopes	0.15
Mountain, with vegetated surface	0.25
Developed (Residential and Commercial)	
Lawn and turf	0.20
Desert landscape	0.10
Pavement	0.05
Agricultural	
Tilled fields and irrigated pasture	0.50

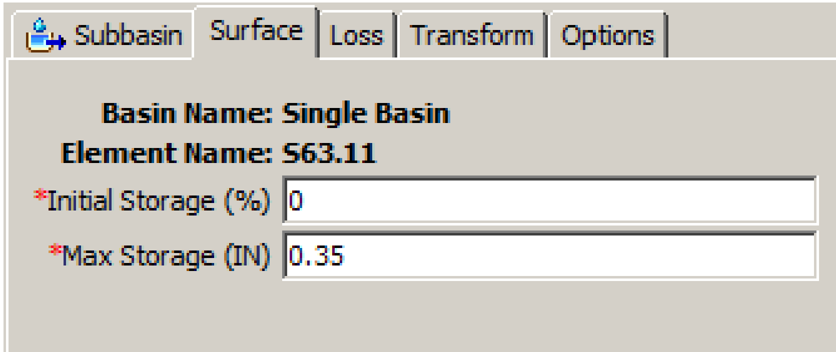


Figure 3–3 Example of Surface Tab in the HEC-HMS Component Editor

3.3.2 Soil Moisture Content (Initial and Saturated)

In HEC-HMS, the Initial Content is the initial saturation in the soil at the beginning of the simulation in terms of volume ratio. The Saturated Content is the maximum water holding capacity in terms of volume ratio, which is often assumed to be equivalent to the total porosity of the soil. Three sets of initial moisture conditions are recommended:

1. Dry – for non-irrigated lands such as desert and rangeland
2. Normal – for irrigated lawn, turf, and permanent pasture
3. Saturated – for irrigated agricultural lands

In HEC-HMS, the initial content for the dry condition shall be set to the wilting point moisture content. For the normal condition, initial content shall be set to the field capacity moisture content. For the wet or saturated condition, the initial saturation equals the saturated content.

3.3.3 Conductivity and Soil Suction

The soil map unit conductivity values presented in this manual are based on the controlling soil horizon of the upper six (6) inches of the soil. Conductivity values for individual soil types are computed based on data in the NRCS soil surveys.

3.3.4 Effective Impervious Area

Effective impervious area is the proportion of the subbasin where runoff is directly connected to the subbasin outlet. Therefore, all of the rainfall that falls on that portion of the subbasin contributes directly to runoff with no rainfall loss. All precipitation for that portion of the subbasin becomes rainfall excess. Usually, effective impervious surfaces are areas such as roof tops, parking lots, and streets where the runoff does not cross any pervious surface before reaching the subbasin outlet.

For undeveloped areas, the effective impervious area is often zero percent. However, in some watersheds there could be extensive rock outcrop that would greatly increase the effective imperviousness of the watershed. Care must be exercised when estimating effective natural impervious area from rock outcrop. Often the rock outcrop is relatively small in terms of the total drainage area and occurs as isolated units surrounded by soils of relatively high infiltration capacities. Relatively small, isolated rock outcrops should not be considered to be effective impervious area because runoff must pass over pervious surfaces before reaching the point of discharge concentration. For watersheds that have significant, contiguous rock outcrop, it may be necessary to establish those areas as their own subbasins so that the direct runoff can be estimated and then routed with channel transmission losses, if appropriate to the point of interest. Finally, the effectiveness of such impervious bedrock areas probably varies with rainfall intensity. That is, for the same watershed the amount of effective impervious area is probably greater for the 100-year storm than for the 2-year storm. Paved roads through undeveloped watersheds will not normally contribute to effective impervious area unless the road serves as a conveyance to the watershed outlet.

For areas that are presently undeveloped, but for which discharge estimates for urbanized conditions are required, estimates of effective impervious area should be obtained based on regional planning and local land-use zoning. Estimates of the effective impervious area for urbanizing areas should be selected from local guidance, if available, along with the general guidance that is provided in **Table 3–2**.

Although **Table 3–2** is to be used as the primary source for impervious values, additional guidance for selection of effective impervious area can also be found in the following: (1) Table 6-6 in EM 1110-2-1417 (USACE, 1996), (2) Table 7.7 in Drainage Design Manual for Mohave County (MCFCD, 2009), and (3) Table 4.2 in the Drainage Design Manual for Maricopa County (FCDMC, 2013). These additional resources may be especially useful when working within those jurisdictions.

For the Walnut Gulch Experimental Watershed 63.11 example, neither urban nor natural rock outcrop imperviousness was present. Therefore, the percent effective imperviousness shown on the example "Loss" tab on **Figure 3–4** is zero percent.

The estimate of effective impervious area for urbanizing areas should be based on ultimate development in the watershed. **Table 3–2** provides guidance on selection of effective impervious values for various urban land-uses.

Table 3–2 General Guidance for Selecting Effective Impervious Area for Urban Land Uses

Land-use	Effective Impervious Area (percent)	
	Mean	Range
Single-Family Residential		
1/4 acre	30	23 – 38
1/3 acre	22	15 – 30
1/2 acre	17	9 – 25
1 acre	14	8 – 20
2 acre	12	7 – 20
Multi-Family Residential	54	42 – 65
Commercial	85	51 – 98
Industrial	59	46 – 72

3.4 PROCEDURES – GREEN AND AMPT

Many of the procedures for computing and developing watershed loss parameters may be done within GIS or some model preprocessor. The instructions below are generic and could be conceptualized as steps in a computerized process using GIS or other computer software.

1. Prepare a base map of the drainage area delineating modeling subbasins.
2. Delineate subareas of different soil map units on the base map. Determine the Green and Ampt parameters for each map unit (based on the tables or GIS data provided with this manual) within each subarea and also assign a land-use or surface cover to each subarea.
3. Determine the size of each subbasin and size of each subarea within each subbasin. For clarification, the *subbasin* is the hydrologic subbasin and the *subarea* is the soil map unit area within each subbasin.
4. Estimate the effective impervious area for each subarea (Table 3–2).
5. Calculate the area-weighted imperviousness for the drainage area or each subbasin.
6. Estimate the surface retention loss (IA) for the drainage area or each subarea (Table 3–1).
7. Calculate the area-weighted surface retention loss value for the drainage area or each subbasin.
8. If the drainage area or subbasin consists of a single map unit, then select conductivity, soil suction, and the appropriate soil moisture content for that soil map unit. If the drainage area or subbasin consists of subareas of different soil map units, then calculate the

composite values for conductivity (Equation 3.1), suction (Equation 3.2) and initial and saturated soil moisture content (Equation 3.3).

9. Estimate the percent vegetation cover and determine the hydraulic conductivity correction factor (C_k) (Figure 3–1).

10. Apply correction factors (C_k) from Step 9 to the value of conductivity from Step 8.

11. The area-weighted values of the surface loss on the “Surface” tab and “Initial Content”, “Saturated Content”, “Suction”, “Conductivity”, and “Impervious” for the drainage area or each subbasin are entered on the “Loss” Tab in the HEC-HMS input for each subbasin.

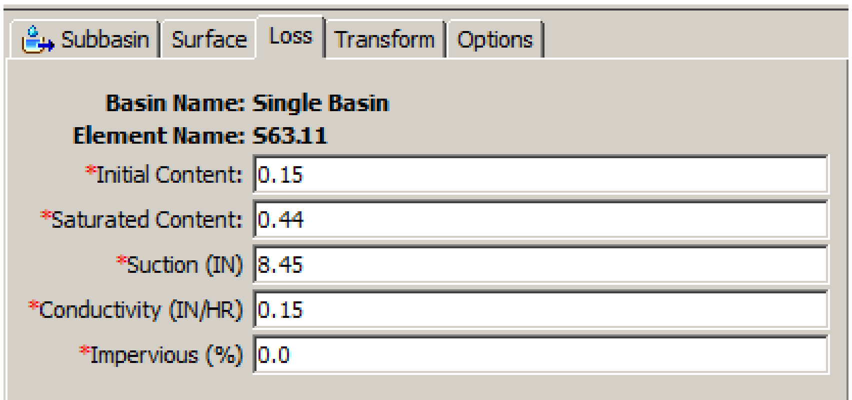


Figure 3–4 Example of Loss Tab in HEC-HMS Component Editor for Green and Ampt Loss Method

3.5 INITIAL AND CONSTANT LOSS RATE METHOD

The Initial and Constant Loss rate method is a simplified rainfall loss estimation method and should be used only when it is known that soil texture does not control infiltration rate. This method also must be used only if adequate calibration or verification is available to justify that uniform loss rates exceed the Saxton & Rawls conductivity values. When using the Initial and Constant Loss rate method, the initial loss is defined as the sum of surface retention loss plus the initial infiltration loss that accrues before surface runoff is produced.

When using the Initial and Constant loss rate method, both the initial loss and the constant loss rate must be estimated. Because this method is to be used for special cases, such as drainage areas and subbasins composed predominantly of sand or volcanic cinders, the estimation of the loss rate parameters will require model calibration, results of regional studies, or other valid techniques. While it is not possible to provide complete guidance in the selection of these parameters, the following general guidance is provided:

1. Since this method is only to be used for special cases, the constant loss rate will either be very low for nearly impervious surfaces or possibly quite high for exceptionally fast draining (porous) land surfaces. For land surfaces with very low infiltration rates, the constant loss

rate will probably be 0.05 inches per hour or less. For sand, a constant loss rate of 0.5 to 1.0 inch per hour or larger would be reasonable. Higher constant loss rate values for sand and other surfaces are possible; however, use of high values will require special studies since they will produce little to no runoff.

2. The selection of the initial loss can be made on the basis of calibration or special studies at the same time the constant loss rate is estimated. Alternatively, the initial loss can be estimated by use of the NRCS Curve Number (CN) equations for estimating initial abstraction, in inches, written as:

$$\text{Initial Loss} = \frac{200}{CN} - 2 \qquad 3.6$$

Estimates of CN for the drainage area or subbasin should be made by referring to NRCS publications such as TR-55 or NEH Part 630. Equation 3.6 provides an acceptable estimate of the initial loss in many cases, however its use will have to be judiciously applied and carefully considered in all cases.

Additional guidance for selection of initial and constant loss parameters is also found in the following publications: (1) Section 4.4.2 of the Drainage Design Manual for Maricopa County, Hydrology (FCDMC, 2013), Table 4.3 for Uniform Loss Rates; (2) Section 7.4.3 of the Drainage Design Manual for Mohave County (MCFCD, 2009), Table 7.10, and (3) ADWR's State Standard 10-07 (ADWR, 2007).

3.6 APPLICATION OF INITIAL AND CONSTANT LOSS IN HEC-HMS

The main parameters in the initial and constant loss are the initial loss and the constant loss rate. This method only should be used in special cases in which the losses are not determined by soil texture or where the subbasin consists of predominantly sand. Due to the uniqueness of each case, only the following general guidance is provided:

1. A constant loss value of 0.05 inches per hour or less is to be used for land surfaces with low infiltration or nearly impervious surfaces.

2. For sand, a constant of 0.5 to 1.0 inches per hour is reasonable. Higher values are possible but will require special studies.

3. The initial loss value can be estimated from calibration or special studies in conjunction with the estimation of the constant loss.

4. Alternatively, since the initial loss amount is equivalent to initial abstraction, it can be estimated using NRCS CN equations: STRTL = 200/CN – 2. Estimates of CN for the subbasin should be made by consulting various publications of the NRCS, particularly, TR-55 and National Engineering Handbook Part 630.

In HEC-HMS, the Initial and Constant is selected as the "Loss Method" under Basin Models (see Figure 3–5) while the loss parameters are entered under the "Loss" tab (see Figure 3–6). Effective impervious area can also be specified with this loss method.

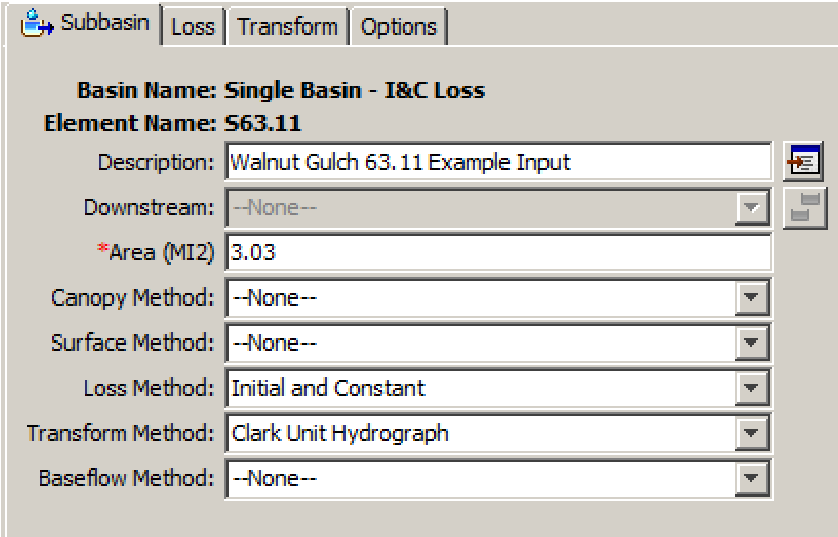


Figure 3–5 Example of Subbasin Tab in HEC-HMS Component Editor for Initial and Constant Loss Method

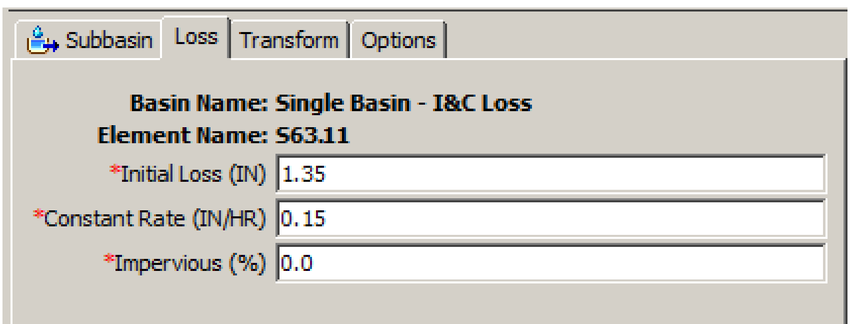


Figure 3–6 Example of Loss Tab in HEC-HMS Component Editor for Initial and Constant Loss Method

3.7 PROCEDURES - INITIAL AND CONSTANT LOSS

1. Prepare a base map of the drainage area that delineates model subbasins.
2. Delineate subareas of different infiltration rates (constant loss rates) on the base map.
3. Determine the size of each subbasin and size of each subarea within each subbasin.
4. Estimate the effective impervious area for the drainage area or each subarea (**Table 3–2**).
5. Estimate the initial loss for the drainage area or each subarea by regional studies or calibration. Alternatively, **Equation 3.6** can be used to estimate or to check the initial loss value.
6. Estimate the constant loss rate for the drainage area or each subarea by regional studies or calibration.
7. Calculate the area-weighted values of percent effective impervious area, initial loss, and constant loss for the drainage area or each subbasin.
8. The area-weighted values of percent effective impervious area, initial loss, and constant loss for the drainage area or each subbasin are entered on the “Loss Tab” in the HEC-HMS input for each subbasin.

Chapter 4
UNIT HYDROGRAPHS

This chapter contains the following details:

- Procedures for the development of Clark unit hydrograph time of concentration and storage coefficient.
- Example HEC-HMS input using the Clark unit hydrograph method.

4.1 INTRODUCTION

A unit hydrograph is defined as a plot of flow versus time that results from one inch of direct runoff during a storm of a specified duration for a particular watershed. Every watershed has a different unit hydrograph that reflects its unique physiography, topography, land-use, and other characteristics. Different unit hydrographs will be produced for the same watershed for different durations of rainfall excess. For example, a unit hydrograph for a particular watershed can be developed for rainfall excess durations of 5-minutes, 15-minutes, 1-hour, or 6-hours, up to the upper duration, as described later.

Only a few watersheds in Arizona will have an adequate database of rainfall and runoff records from which to develop unit hydrographs. Therefore, indirect methods usually will be used to develop synthetic unit hydrographs. Several procedures are available to develop synthetic unit hydrographs, and virtually all of these procedures are empirical. The selection of a synthetic unit hydrograph procedure should be made such that the database for the empirical development is representative of the study watershed.

The unit hydrograph itself is a lumped parameter in that it represents the composite effects of all of the watershed and storm characteristics that dictate the rate of rainfall excess runoff from the watershed. Although there are numerous watershed and storm characteristics that determine the shape of a unit hydrograph, only a limited number of those characteristics can be quantified and used to calculate a unit hydrograph. One or more unit hydrograph parameters (depending on the selection of synthetic unit hydrograph procedure) are needed to calculate a unit hydrograph.

The concept of the unit hydrograph is used to route the time increments of rainfall excess from the watershed to the watershed outlet. For the purposes of ADOT hydrologic modeling, the Clark unit hydrograph method is the recommended synthetic unit hydrograph procedure. Unit hydrograph procedures other than the Clark procedure can be used for specific applications, but will require justification and prior approval by the ADOT Drainage Group.

4.2 PROCEDURE

The Clark unit hydrograph requires the estimation of two parameters - the time of concentration (T_c) and the storage coefficient (R). Sections 4.2.1 and 4.2.2 describe the procedures that are to be used to calculate these parameters, as well as the guidelines in Section 4.2.5 that are to be used to select the unit hydrograph duration and model control time interval.

4.2.1 Time of Concentration

Time of concentration is the travel time during the corresponding period of most intense rainfall excess for a flood wave to travel from the hydraulically most distant point in the watershed to the point of interest (concentration point). Three time of concentration (T_c) equations are to be used, depending on the type of watershed - desert/mountain, agricultural fields, or urban. The recommended T_c equations are:

Desert/mountain:

$$T_c = 2.4A^{0.1}L^{0.25}L_{ca}^{0.25}S^{-0.2} \tag{4.1}$$

Agricultural:

$$T_c = 7.2A^{0.1}L^{0.25}L_{ca}^{0.25}S^{-0.2} \tag{4.2}$$

Urban:

$$T_c = 3.2A^{0.1}L^{0.25}L_{ca}^{0.25}S^{-0.14}RTIMP^{-0.36} \tag{4.3}$$

where:

where:	T_c	=	time of concentration, in hours,
	A	=	area, in square miles,
	S	=	watercourse slope, in ft/mile,
	L	=	Length of the watercourse to the hydraulically most distance point, in miles,
	L_{ca}	=	length measured from the concentration point along L to a point on L that is perpendicular to the watershed centroid, in miles, and
	$RTIMP$	=	effective impervious area, in percent.

In using Equations 4.1 through 4.3, the following points should be noted and observed:

1. The area (A) will be determined from the best available map. The delineation of the drainage boundary must be carefully performed, and special care must be taken where there is little topographic relief. In urban areas, land grading and road construction can produce drainage boundaries that separate runoff from contributing areas during small and lower intensity storms. However, larger and more intense storms, such as the design storm, can produce runoff depths that can cross these intermediate drainage boundaries,

resulting in a larger total contributing area. Similarly, floods on active alluvial fans and in distributary flow systems can result in changes in contributing areas during larger and more intense storms. For such areas, it is generally prudent to consider the largest reasonable drainage area in these situations.

2. Determination of the hydraulically most distant point in a watershed will define both *L* and *S*. Often, the hydraulically most distant point is determined as the point along the watershed boundary that has the longest flow path to the watershed outlet (or subbasin concentration point). This outcome is generally true when the topography is relatively uniform throughout the watershed. However, there are situations in which the longest flow path (*L*) does not define the hydraulically most distant point. Occasionally, especially in mountainous areas, a point with a shorter flow path may have an appreciably flatter slope (*S*) such that the shorter flow path defines the hydraulically most distant point. For watersheds with multiple choices for the hydraulically most distant point, the T_c should be calculated for each point and the largest T_c should be used.

3. Slope (*S*) is the average slope calculated by dividing the difference in elevation between the hydraulically most distant point and the watershed outlet by the watercourse length (*L*). This method will usually be used to calculate *S*. However, there are situations in which special consideration should be given to calculating *S* and to further subdividing the watershed into subbasins that reflect a more uniform value of *S*. For example, if there is dramatic change in watercourse slope in the watershed, then the use of a multiple subbasin model should be considered with change in watercourse slope used in delineating the subbasins. There will also be situations in which the watercourse contains vertical or nearly vertical drops (mountain rims, rock outcrop, etc.). In these situations, plotting of the watercourse profile will usually identify nearly vertical changes in the channel bed. When calculating the average slope, subtract the accumulative elevation differential that occurs in nearly vertical drops from the overall elevation differential prior to calculating *S*.

4. L_{ca} is measured along *L* to a point on *L* that is essentially perpendicular to the watershed centroid. This is a shape factor in the T_c equation. Occasionally, the shape of agricultural fields or urban subbasins is nearly rectangular, possibly resulting in two different dimensions for L_{ca}. In the case of such nearly rectangular (and therefore, nearly symmetrical) watersheds or subbasins L_{ca}, can usually be satisfactorily estimated as 0.5L.

5. *RTIMP* is the effective impervious area. This is the same value that was determined for the watershed as discussed in the Rainfall Losses chapter. *RTIMP* is used to estimate T_c for urban watersheds only (**Equation 4.3**).

6. Ideally, the selection of the watershed or subbasin boundaries can be made so that the area represents a hydrologically uniform region that is essentially all desert/mountain, or agricultural fields, or urban for those situations, the T_c equations (**Equations 4.1** through **4.3**) can be applied directly. However, there will be situations where the watershed or modeling subbasin is a mixture of two or three of those types. In those cases, the T_c

equation (**Equations 4.1** through **4.3**) is selected based on the watershed type that contains the greatest portion of *L*.

4.2.2 Storage Coefficient

The storage coefficient is a Clark unit hydrograph parameter that relates the effects of direct runoff storage in the watershed to unit hydrograph shape. The equation for estimating the storage coefficient (*R*) is:

$$\mathrm{R} = 0.37 T_c^{1.11} L^{0.8} A^{-0.57} \qquad 4.4$$

where *R* is in hours and the other variables are as defined for the T_c equations.

4.2.3 Applications and Limitations

The Clark unit hydrograph can be used for virtually any watershed that will be encountered in Arizona. However, there may be situations where use of another unit hydrograph will be warranted. For example, rainfall and runoff data may be available for the watershed or a nearby hydrologically similar watershed to develop a unique unit hydrograph. In those cases, the developed unit hydrograph would be input to HEC-HMS using the user-specified unit hydrograph option. In other situations, a unit hydrograph at or near the desired location may have been developed for another project. That unit hydrograph or unit hydrograph procedure may be preferable to the recommended Clark unit hydrograph procedure for that application. If other unit hydrographs or unit hydrograph procedures are determined to be more applicable for a certain situation, they should be used, but this must be approved in advance by ADOT.

Equations 4.1 through **4.3** were derived for use in estimating the time of concentration for floods with design return periods that are typical for highway drainage structures. Use of these equations may result in time of concentration estimates that are too short for floods of return period less than 25-year and too long for floods of return period appreciably greater than 100-year. This problem can occur because of the effect that runoff magnitude has on the hydraulic efficiency (runoff velocity) of watersheds. Therefore, if **Equations 4.1** through **4.3** are used to estimate the time of concentration for floods of return period appreciably greater than the 100-year, then the time of concentration should be reduced (by as much as 25 percent for very large, rare floods); similarly, for estimating the time of concentration for floods of return period less than the 25-year, then the time of concentration should be increased (by as much as 100 percent for very frequent flooding, such as the 2-year). Since R (**Equation 4.4**) is a function of T_c, the R value should be recalculated if T_c is adjusted for return period.

4.2.4 Application in HEC-HMS

The recommended unit hydrograph procedures are implemented in HEC-HMS as follows:

1. Select the Clark Unit Hydrograph option as the “Transform Method” in the “Subbasin” tab (see **Figure 4–1**).

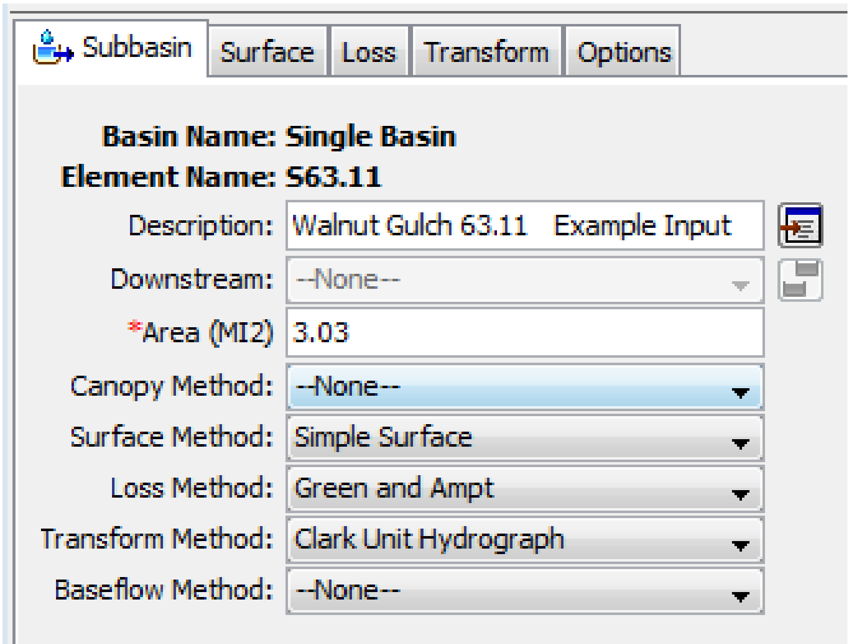


Figure 4–1 Example of Subbasin Tab in HEC-HMS Component Editor

2. Enter the computed "Time of Concentration" and "Storage Coefficient" in the "Transform" tab for each subbasin (see example in Figure 4–2).

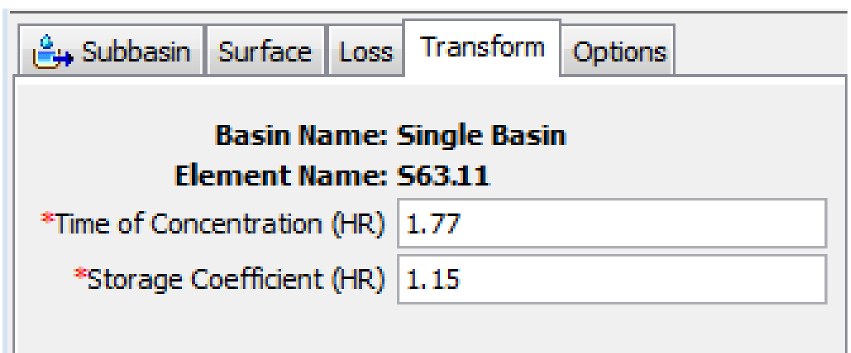


Figure 4–2 Example of Transform Tab in HEC-HMS Component Editor for Clark Unit Hydrograph

4.2.5 Model Time Interval Requirements

The duration of the unit hydrograph (or all unit hydrographs in a multiple subbasin model) is related to the model time interval. In HEC-HMS, the model time interval is specified under

"Control Specifications" in the "Time Interval" pull down. In general, the model time interval will be selected according to the following criteria:

1. Time interval = five (5) minutes for a 24-hour storm duration

2. Time interval should not exceed 0.25 T_c for the subbasin with the shortest T_c.

However, there may be special situations in which the following additional rules should be considered in the selection of the model time interval:

1. Time interval = 0.15 T_c usually provides adequate definition of the hydrograph peak with an optimum number of hydrograph coordinate calculations.

2. Time interval = 0.25 T_c is the maximum value for the time steps.

3. Time interval for a multiple subbasin model should be selected based on the smallest T_c value for any of the subbasins in the model.

4.3 INSTRUCTIONS

1. Delineate the watershed boundaries on the watershed base map.

2. Trace the paths of the major watercourses in the watershed on the base map.

3. If the watershed has more than one land-use, define the areas of the different land-use types. For example:

   a. Desert/mountain

   b. Agricultural

   c. Urban

4. Determine whether the watershed can be treated as a single, hydrologically homogeneous watershed, or if it must be divided into modeling subbasins. This decision should consider the following factors:

   a. Topography (and channel slope)

   b. Land-use

   c. Diversity of soil texture (from Rainfall Losses chapter)

   d. Occurrence of rock outcrop

   e. Existence of drainage and flow control structures within the watershed (detention/retention basins, elevated highway cross-drainage structures, channelized and improved watercourses, and so forth)

   f. Shape of the watershed

   g. Needs of the hydrologic model, such as investigation and planning for future highway drainage structures

5. If the watershed is to be divided into modeling subbasins, use the information from Steps 2, 3, and 4 to delineate the subbasin boundaries.

6. For the watershed or each modeling subbasin, determine the following:

   A – area, in square miles,
   L – length of the flow path to the hydraulically most distant point, in miles,
   L_{ca} – length along L to a point opposite the centroid, in miles,
   S – average slope of L, in ft/mile, and
   RTIMP – effective impervious area, in percent.

7. Calculate T_c depending on the type of watershed:

For desert/mountain:

$$T_c = 2.4\, A^{0.1}\, L^{0.25}\, L_{ca}{}^{0.25}\, S^{-0.2}$$

For agricultural fields:

$$T_c = 7.2\, A^{0.1}\, L^{0.25}\, L_{ca}{}^{0.25}\, S^{-0.2}$$

For urban areas:

$$T_c = 3.2\, A^{0.1}\, L^{0.25}\, L_{ca}{}^{0.25}\, S^{-0.14}\, RTIMP^{-0.36}$$

8. Calculate R:

$$R = 0.37\, T_c^{1.11}\, L^{0.8}\, A^{-0.57}$$

9. Enter the values of T_c and R in the “Transform” tab for the watershed or each subbasin in HEC-HMS.

4.4 EXAMPLE

Example No. 4-1 Clark Unit Hydrograph Parameters for Rangeland Watershed

Problem: Develop the Clark unit hydrograph parameters for the Walnut Gulch Experimental Watershed 63.11 near Tombstone, Arizona.

Solution:

1. The watershed map shows the following:
   a. Watershed boundary
   b. Flow path to the hydraulically most distant point
   c. Location of the basin centroid

2. The following are measured from the map or computed from GIS:

A = 3.03 square miles
L = 4.14 miles
L_{ca} = 1.96 miles
S = 111 ft/mile

3. The watershed is desert/rangeland.

4. Calculate T_c using the desert/mountain T_c equation:

$$T_c = 2.4\, A^{0.1}\, L^{0.25}\, L_{ca}{}^{0.25}\, S^{-0.2}$$
$$T_c = 2.4\, (3.03^{0.1})(4.14^{0.25})(1.96^{0.25})(111^{-0.2})$$
$$T_c = 1.76 \text{ hr}$$

5. Calculate R:

$$R = 0.37\ T_c^{1.11}\ L^{0.8}\ A^{-0.57}$$
$$R = 0.37\ (1.76^{1.11})(4.14^{0.8})(3.03^{-0.57})$$
$$R = 1.15\ \text{hr}$$

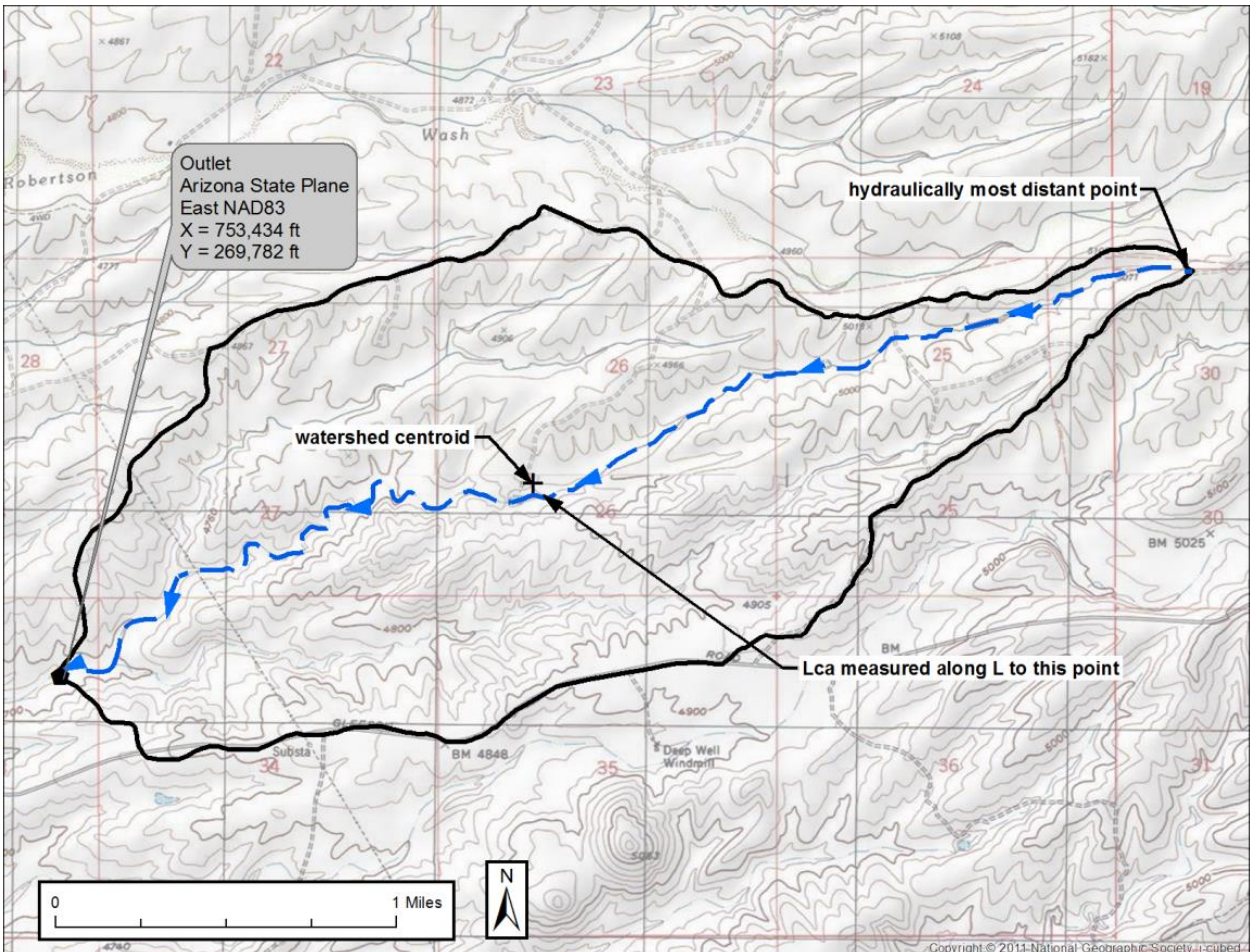


Figure 4–3 Example Map for Walnut Gulch Experimental Watershed 63.11 Near Tombstone, Arizona

Example No. 4-2 Clark Unit Hydrograph Parameters for Urban Watershed

Problem: Develop the Clark unit hydrograph parameters for the Tucson Arroyo, Tucson, Arizona watershed.

Solution:

1. The watershed map shows the following:

   a. Watershed boundary

   b. Flow path to the hydraulically most distant point

   c. Location of the basin centroid

2. The following are measured from the map or computed in GIS:

   A = 8.01 square miles
   L = 6.14 miles
   L_{ca} = 2.51 miles
   S = 38 ft/mile
   RTIMP = 20%

3. The watershed is urban residential with some commercial/industrial areas and a park and golf course.

4. Calculate T_C using the urban T_C equation:

   $$T_c = 3.2\ A^{0.1}\ L^{0.25}\ L_{ca}^{\ 0.25}\ S^{-0.14}\ RTIMP^{-0.36}$$
   $$T_c = 3.2\ (8.01^{0.1})(6.14^{0.25})(2.51^{0.25})(38^{-0.14})(20^{-0.36})$$
   $$T_c = 1.60\ \text{hr}$$

5. Calculate R:

   $$R = 0.37\ Tc^{1.11}\ L^{0.8}\ A^{-0.57}$$
   $$R = 0.37\ (1.60^{1.11})(6.14^{0.8})(8.01^{-0.57})$$
   $$R = 0.81\ \text{hr}$$

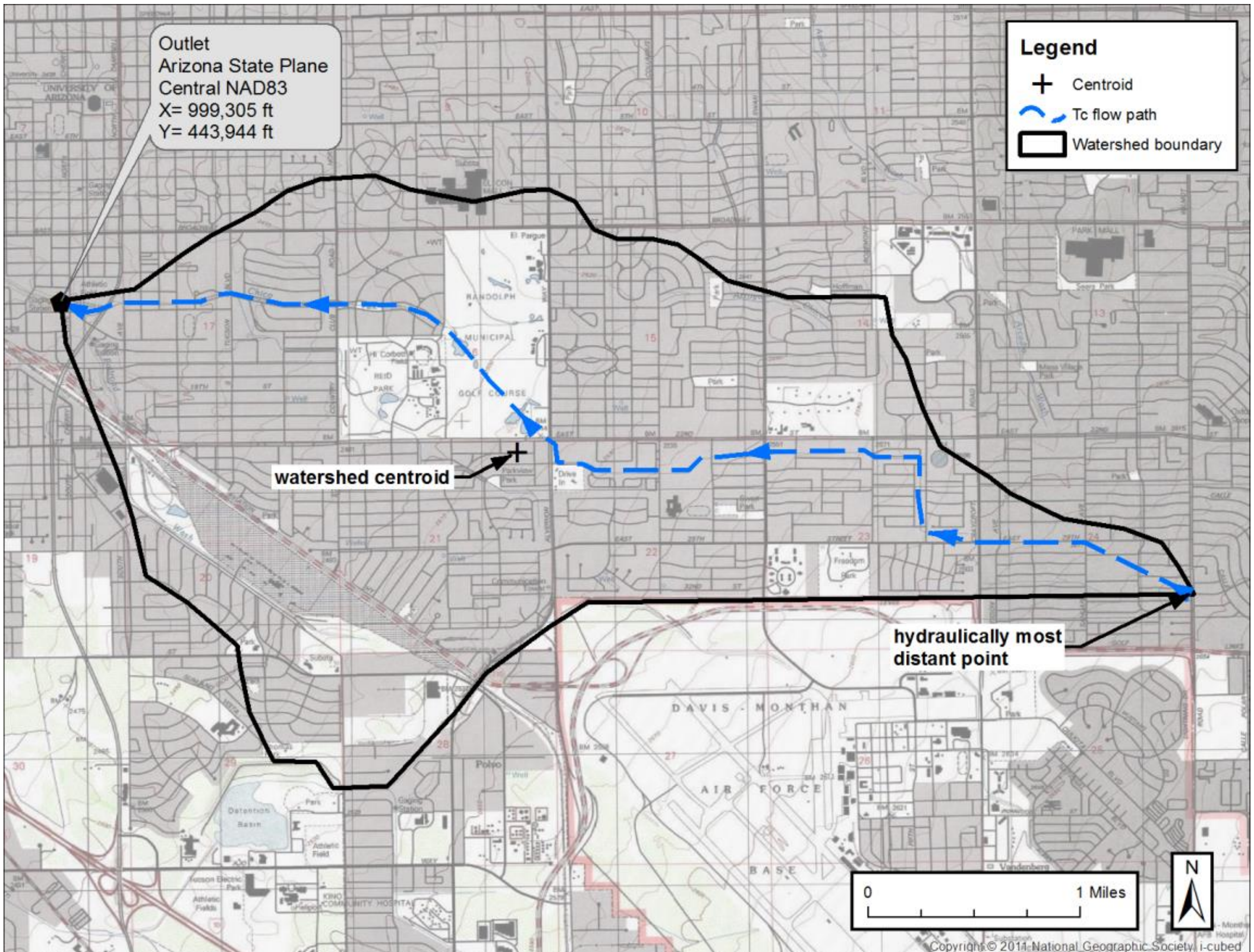


Figure 4–4 Example Map for Tucson Arroyo Watershed, Tucson, Arizona

Chapter 5
CHANNEL ROUTING

This chapter contains the following details:

- Methods for performing hydrologic channel routing.
- The primary method is Muskingum-Cunge. In certain situations, Kinematic Wave and Modified Puls methods may be applicable.
- Example HEC-HMS input for each method.

5.1 INTRODUCTION

Channel routing describes the changes in the shape and timing of a flood wave (hydrograph) as it moves down a watercourse. As a flood wave passes through a river reach, the peak of the outflow hydrograph is usually attenuated and delayed due to flow resistance in the channel and the storage capacity of the channel and its floodplain. Channel routing is used in flood hydrology models, such as HEC-HMS, when the watershed is modeled with multiple subbasins and runoff from the upper subbasins must be routed through channels within downstream subbasins to the watershed outlet. Channel routing may not be required for short reaches when the travel time through the reach is shorter than the computation interval being used. Many channel routing methods are available, but only three methods are recommended for highway drainage in Arizona – the Muskingum-Cunge method, the Kinematic Wave method, and the Modified Puls method.

5.2 PROCEDURE

The Muskingum-Cunge method will be the most commonly applied method for highway drainage modeling in Arizona. The Kinematic Wave method is recommended for some urban settings or reaches with smooth, uniform, constructed channels. The Modified Puls method is recommended for channel reaches with significant backwater effects or where channel storage-discharge relationships are readily available.

5.2.1 Applications and Limitations

Channel routing is to be used in multiple subbasin models when the runoff from the upper subbasins pass through a watercourse, or a system of watercourses, to an outlet downstream concentration point. Channel routing should be used in models when a major component of watershed runoff (an inflow hydrograph) enters a relatively long channel and must flow through that channel to the watershed outlet or to a point along the channel where a flood hydrograph is desired. In those situations, the peak of the outflow hydrograph is usually attenuated and delayed compared with that of the inflow hydrograph. When channel routing travel time is shorter than the model computation interval, the routing reach may be excluded from the model structure. Further discussion on channel routing reach lengths is provided in **Section 8.2.4.5**.

The Muskingum-Cunge routing method will be used in most instances. If significant backwater effects exist within a reach, the Modified Puls method should be used. The Kinematic Wave method may also be used for routing through uniform constructed channels. HEC-HMS Technical Reference Manual (USACE, 2000) provides additional guidance on selection of appropriate routing methods.

The following sections present the implementation of the Muskingum-Cunge, Kinematic Wave, and Modified Puls routing methods in HEC-HMS.

5.2.2 Muskingum-Cunge

Figure 5–1 through **Figure 5–3** shows the HEC-HMS input form for the Muskingum-Cunge routing method. The routing method is selected on the Routing Method pull down menu shown in **Figure 5–1**. **Figure 5–2** shows the required and optional parameters for the Muskingum-Cunge routing method on the “Routing” tab. **Figure 5–3** shows the “Paired Data” tab for use when entering cross section data.

The “Reach tab” (**Figure 5–1**) includes:

Reach Description

The modeler may provide a text description of the reach. The description will appear in the output and graphics.

Downstream

This section contains the downstream connection node in the HEC-HMS model, and typically is a subbasin concentration point, junction, or concentration point.

Routing Method

The Muskingum-Cunge routing method is selected on this drop down menu.

Loss/Gain Method

Typically the Loss/Gain Method is set to “none.” The Loss option can be used when transmission losses need to be included (see **Chapter 7**).

The Routing tab (**Figure 5–2**) includes: Time Step Method

The Time Step Method is a new parameter available in HEC-HMS. It is recommended that the “Automatic Fixed Interval” method be selected on the first line of the “Routing Data Entry” tab (**Figure 5–2**) for use with the methods presented in this manual.

Routing Reach Length (Length)

The routing reach length is the length of the channel or major flow path along which the hydrograph will be routed. The reach length should be measured on the best available map. In the HEC-HMS model, the units of reach length are feet.

Energy Grade Line Slope (Slope)

The slope of the energy grade line is not normally known without detailed step-backwater modeling. The Muskingum-Cunge Method assumes normal flow conditions, for which the energy slope is parallel to the channel bed slope. The channel bed slope is calculated by dividing the difference in bed elevation between the upper and lower ends of the routing reach by the reach length. In the HEC-HMS model, the units of slope are feet/foot.

Manning's Roughness Coefficient (n)

Manning's roughness coefficient, n, is a measure of the flow resistance of a channel or overbank flow area. The flow resistance is affected by many factors including size of bed material, bed form, irregularities in the cross section, depth of flow, vegetation, channel alignment, channel shape, obstructions to flow, and quantity of sediment being transported in suspension or as bed load. In general, all factors that retard flow and increase turbulent mixing tend to increase n.

The HEC-HMS application of the Muskingum-Cunge method requires that an n value be estimated for the channel and the right and left floodplain elements. The value for channel n can be selected from **Table 5–1**. For overbank floodplains, the value of n is selected from **Table 5–2**. Further refinement of Manning's n value for the effect of surface irregularities, obstructions to flow, vegetation, variations in channel cross section, and meandering of the main channel are discussed further in ADOT Highway Drainage Design Manual, Hydraulics.

Cross Section

The channel geometry is to be provided by an 8-point cross section. That cross section is to be representative of the hydraulic characteristics throughout the routing reach. The cross section should contain the maximum discharge. Multiple reaches may be needed if the hydraulic characteristics vary significantly and/or the reach is long relative to the computation interval being used in the model. Considerable judgment is necessary in defining the representative 8-point cross section. The guidance in the HEC-HMS User's Manual should be followed when defining an 8-point cross section. The coordinates (Station and Elevation) can be to any base datum. Specifically, the vertical dimensions (Elevation) do not need to correspond to land surface elevation or any elevation for any location along the routing reach.

Examples of the optional 8-point cross section input data are also shown in **Figure 5–1** through **Figure 5–5**.

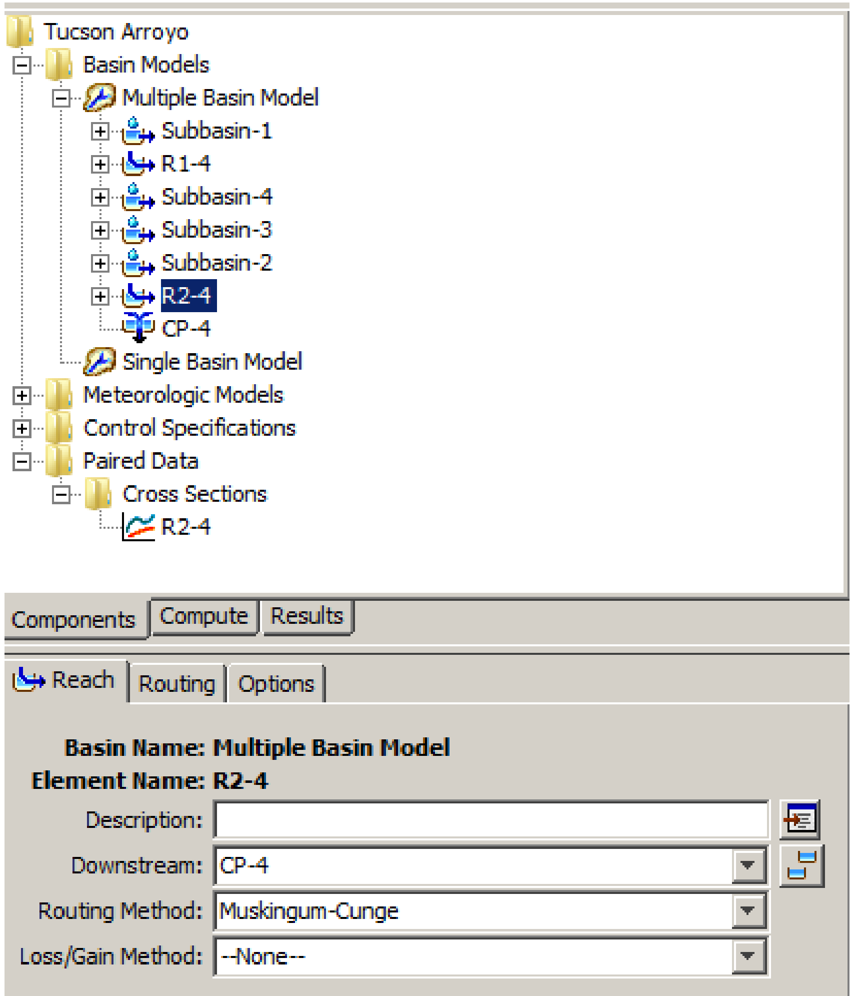


Figure 5–1 Example of the Reach Tab in the Component Editor for the Muskingum-Cunge Method

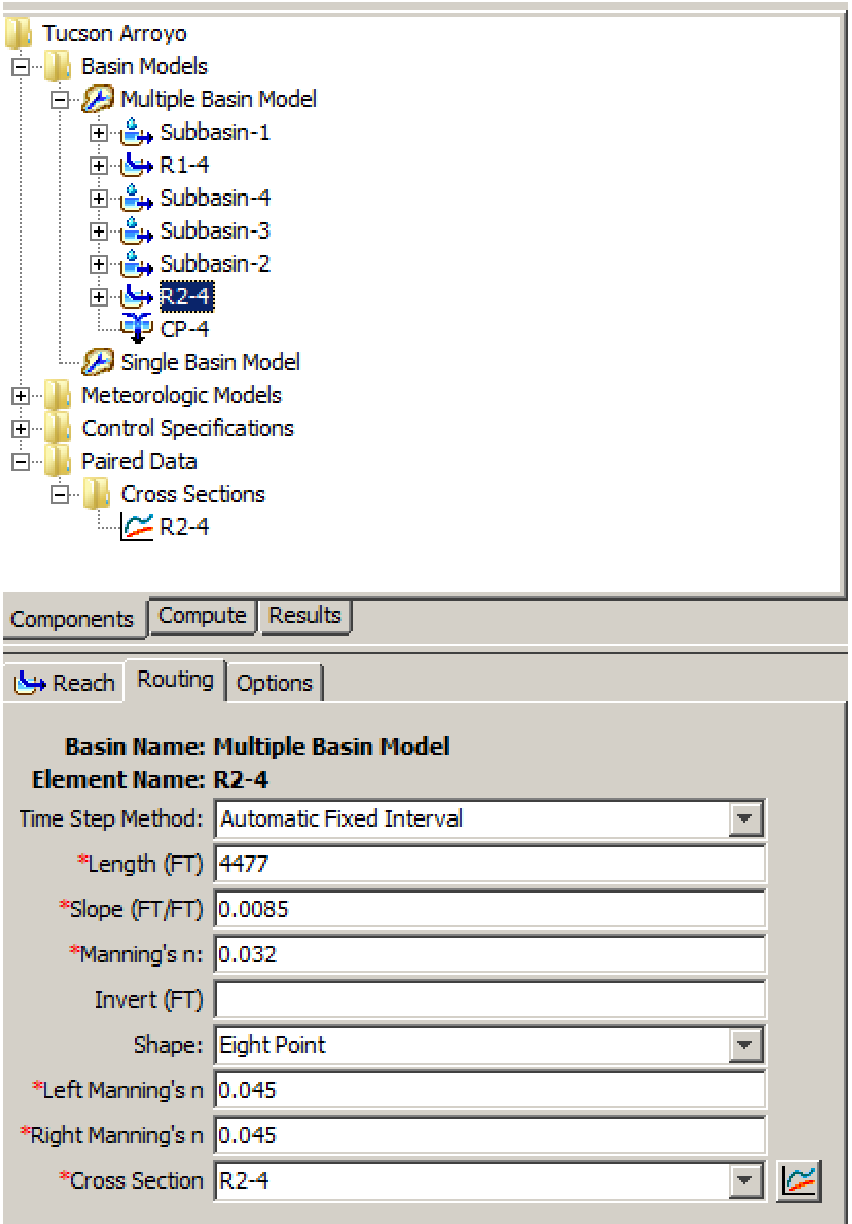


Figure 5–2 Example of the Routing Tab in the Component Editor for the Muskingum-Cunge Method

Table 5–1 Base Values for Manning's Roughness Coefficient for Straight, Uniform, Stable Channels (from Thomsen and Hjalmarson, 1991)

Channel Material	Size of Bed Material		Base Value, n	
	(mm)	(in)	Benson and Dalrymple (1967)[a]	Chow (1959)[b]
Concrete	--	--	0.012-0.018	0.011
Rock cut	--	--	--	0.025
Firm Soil	--	--	0.025-0.032	0.020
Coarse Sand	1-2	--	0.026-0.035	--
Fine Gravel	--	--	--	0.024
Gravel	2-64	0.08-2.50	0.028-0.035	--
Coarse Gravel	--	--	--	0.028
Cobble	64-256	2.50-10.0	0.030-0.050	--
Boulder	>256	>10.0	0.040-0.070	--

Notes: a - Straight uniform channel.
b - Smoothest channel attainable in indicated material.

Table 5–2 Values of Manning's n for Floodplains (from Thomsen and Hjalmarson, 1991)

Description	Minimum	Normal	Maximum
Pasture, no brush:			
Short grass	0.025	0.030	0.035
High grass	0.030	0.035	0.050
Cultivated areas:			
No crop	0.020	0.030	0.040
Mature row crops	0.025	0.035	0.045
Mature field crops	0.030	0.040	0.050
Brush:			
Scattered brush, heavy weeds	0.035	0.050	0.070
Light brush and trees, in winter	0.035	0.050	0.060
Light brush and trees, in summer	0.04	0.060	0.080
Medium to dense brush, in winter	0.045	0.070	0.110
Medium to dense brush, in summer	0.07	0.100	0.160
Trees:			
Dense willows, summer, straight	0.011	0.150	0.200
Cleared land with tree stumps, no sprouts	0.030	0.040	0.050
Same as above, but heavy growth of sprouts	0.050	0.060	0.080
Heavy stand of timber, a few downed trees, little undergrowth, flood stage below branches	0.080	0.100	0.120
Same as above, but with flood stage reaching branches	0.100	0.120	0.160

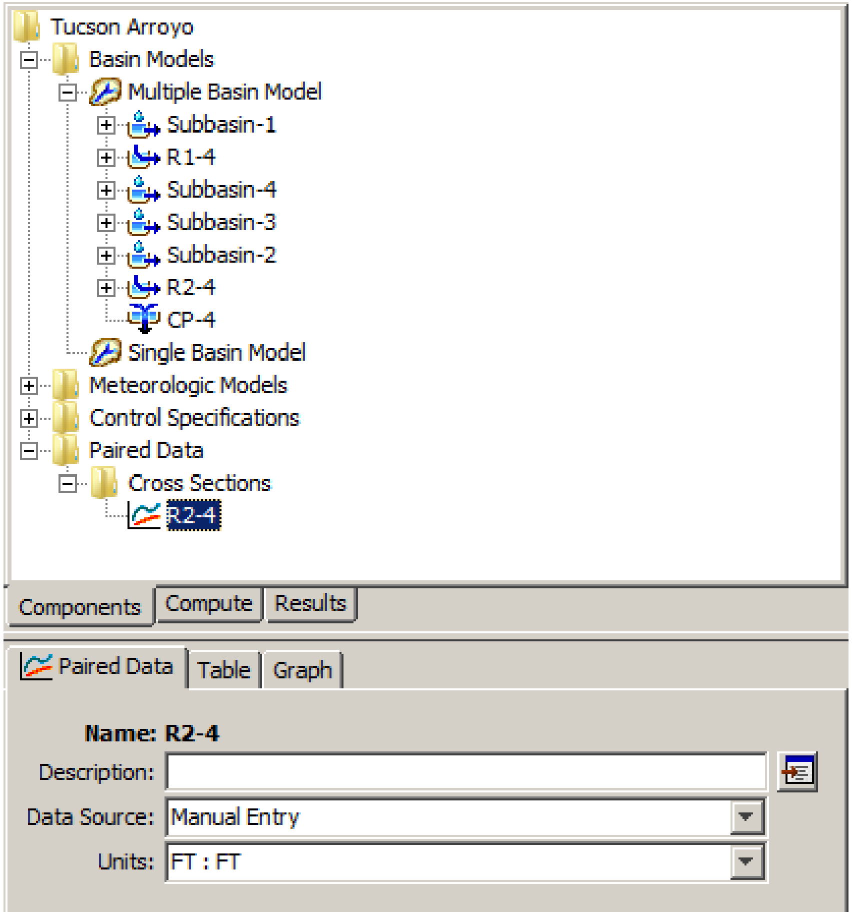


Figure 5–3 Example of the Paired Data Tab in the Component Editor for the Muskingum-Cunge Method

Paired Data | Table | Graph

Station (FT)	Elevation (FT)
0.0	101.0
50.0	101.0
100.0	100.5
120.0	100.0
140.0	100.0
150.0	100.5
200.0	101.0
250.0	101.0

Figure 5–4 Example of the Table Tab in the Component Editor for the Muskingum-Cunge Method

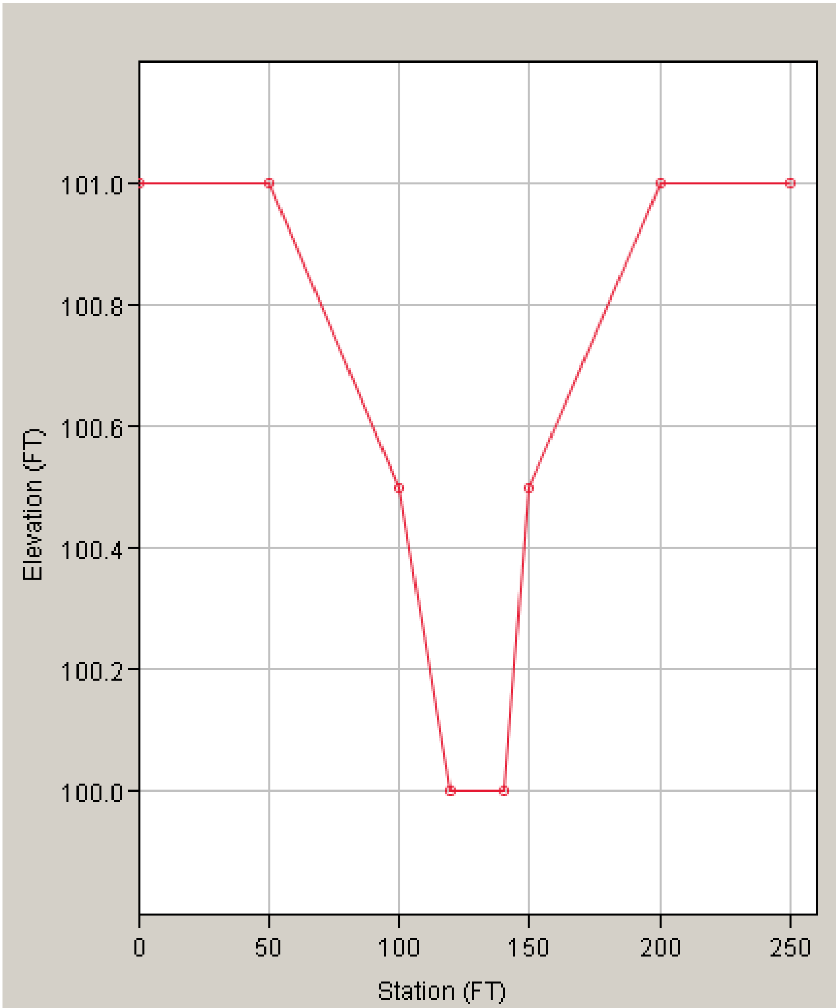


Figure 5–5 Example of the Graph Tab in the Component Editor for the Muskingum-Cunge Method

5.2.3 Kinematic Wave

The Kinematic Wave routing method in HEC-HMS approximates the full unsteady flow equations by ignoring inertial and pressure forces. It also assumes that the energy slope equals the bed slope. It is best suited to fairly steep streams that have been modified to have regular shapes and slopes. Figure 5–6 and Figure 5–7 show an example of the HEC-HMS input form for the Kinematic Wave channel routing method. The routing method is selected from the “Routing Method” pull down menu on the “Reach” tab. The “Routing” tab contains the required and optional input to be specified for each routing reach.

The "Reach" tab (Figure 5–6) includes:

Reach Description

The modeler may provide a text description of the reach. The description will appear in the output and graphics.

Downstream

This section contains the downstream connection node in the HEC-HMS model, and typically is a subbasin concentration point, junction, or concentration point.

Routing Method

The Kinematic Wave routing method is selected on this drop down menu.

Loss/Gain Method

Since the Kinematic Wave method will be used primarily in urban settings or with constructed channels, transmission losses should not be included.

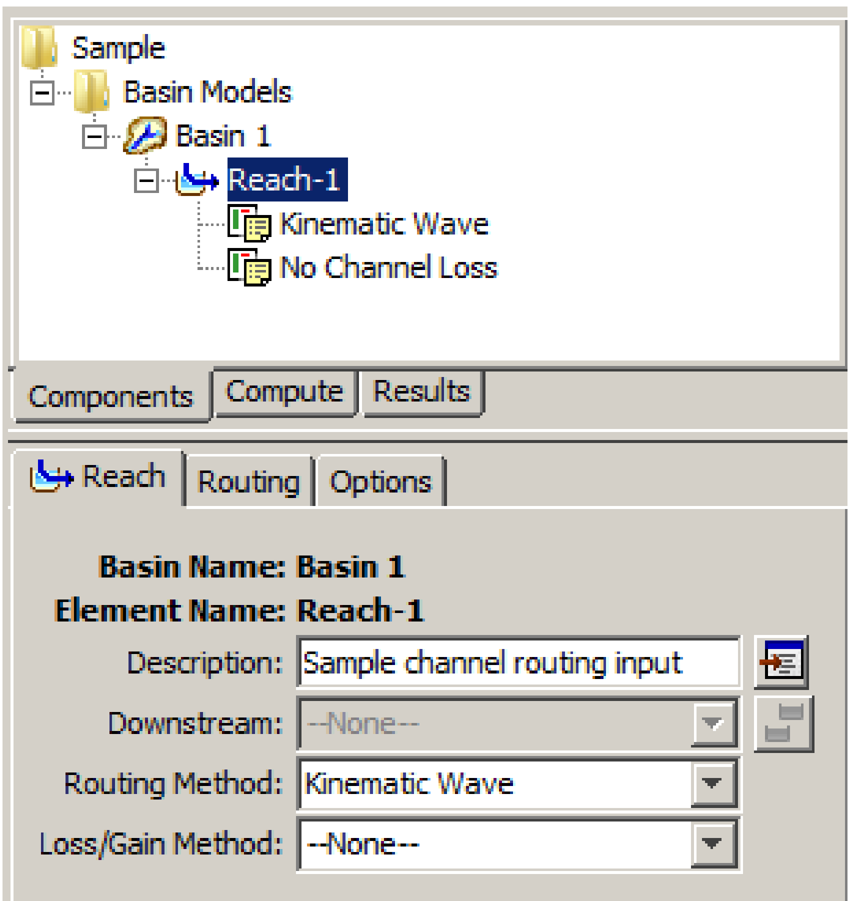


Figure 5–6 Example of the Reach Tab in the Component Editor for the Kinematic Wave Method

The "Routing" tab (Figure 5–7) includes:

Length, Slope, & Manning's n

Length, slope, and Manning's n are the same as used for Muskingum-Cunge.

Subreaches

The subreaches parameter default value is two (2) but may be optionally increased to assist the program in computation of the correct distance step used in the internal routing calculations.

Shape

Available shapes for the channel routing reach include circle, deep, rectangle, trapezoid, and triangle. The "deep" option is for flow conditions in which the flow depth is approximately equal to the flow width. Note that none of the shape options include a floodplain that is topographically or hydraulically separate from the channel. Therefore, the Kinematic Wave method should not be used for routing reaches where such conditions exist.

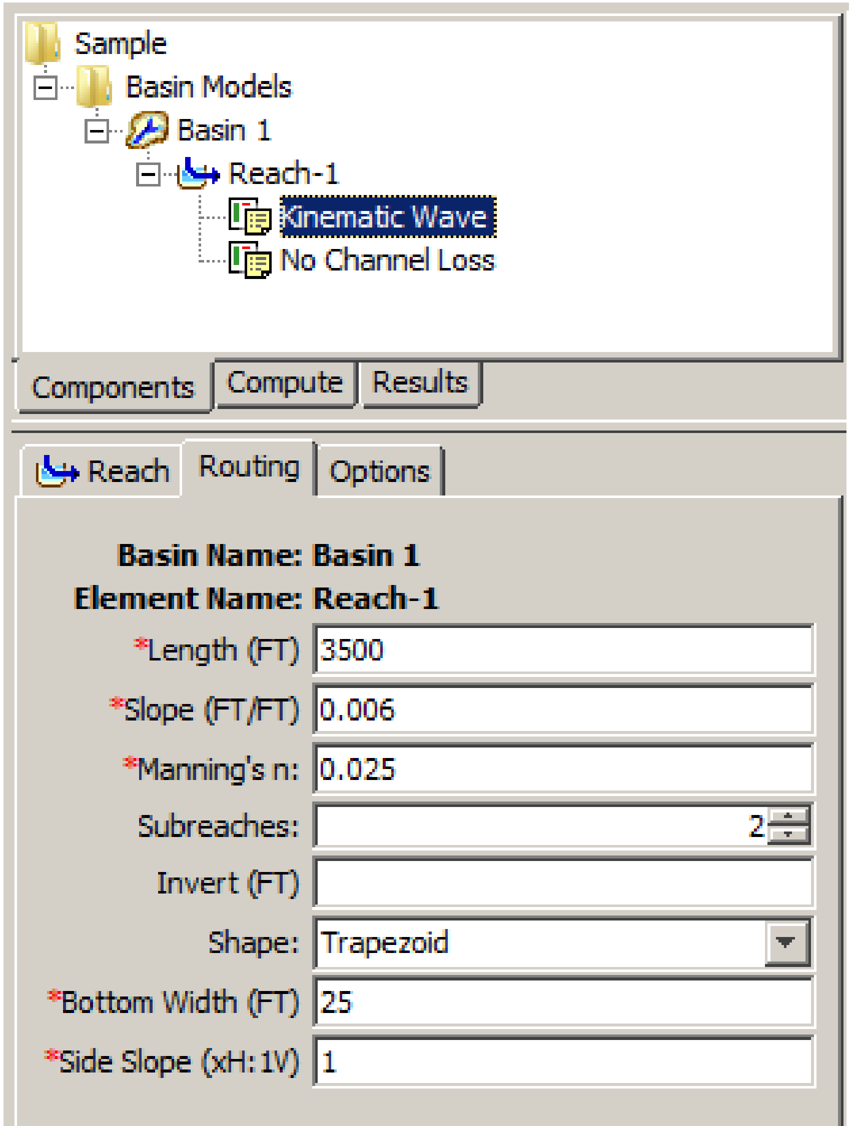


Figure 5–7 Example of the Routing Tab in the Component Editor for the Kinematic Wave Method

5.2.4 Modified Puls

The Modified Puls routing method uses conservation of mass and a relationship between storage and discharge to route flow through a stream reach. It is especially useful in representing routing reaches with backwater effects such as irregular natural streams that cannot be adequately characterized by a single cross-section and slope. Figure 5–8 and Figure 5–9 show an example of the HEC-HMS input for the Modified Puls routing method.

The “Reach” tab (Figure 5–8) includes:

Reach Description

The modeler may provide a text description of the reach. The description will appear in the output and graphics.

Downstream

This section contains the downstream connection node in the HEC-HMS model, and typically is a subbasin concentration point, junction, or concentration point.

Routing Method

The Modified Puls routing method is selected on this drop down menu.

Loss/Gain Method

Typically the Loss/Gain Method is set to “none.” The Loss option can be used when transmission losses need to be included (see Chapter 7).

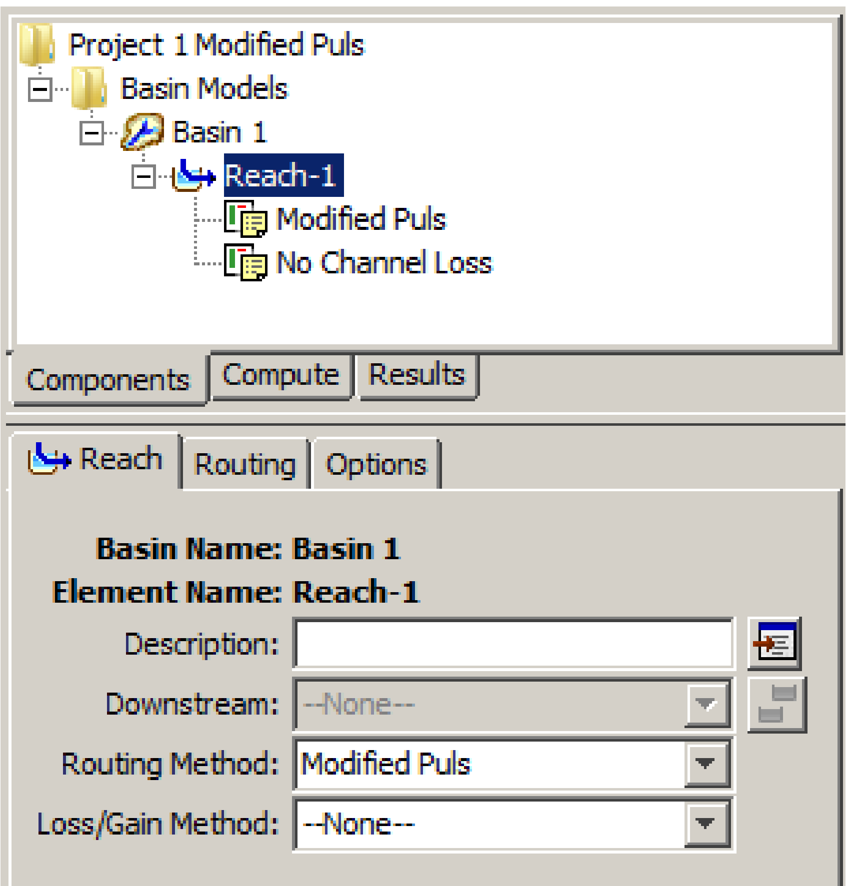


Figure 5–8 Example of the Reach Tab in the Component Editor for the Modified Puls Method

The “Routing” tab (**Figure 5–9**) includes:

Storage-Discharge Function

This table relates routing reach storage (in acre-feet) to outflow discharge (cfs). The data for this table are added using the Paired Data Manager.

Subreaches

The number of subreaches in the Modified Puls routing method can be estimated using:

Number of subreaches = routing reach length/(average velocity x model time interval)

Initial

The initial condition represents the flow in the channel at the start of the routing computation. The manual recommends the use of a dry channel (Discharge = 0 cfs) for channels in Arizona unless a regular base flow is appropriate. This can be modeled in HEC-HMS by specifying the Initial condition as inflow = outflow on the “Routing” tab (see **Figure 5–9**).

Elevation-Discharge Function

The Elevation-Discharge function is a table relating routing reach elevation, or stage (in feet), to outflow discharge (cfs). The data for this table are added using the Paired Data Manager.

Invert

The invert is an optional input that specifies the lowest elevation in the routing reach. The flow depth can then be computed and reported based on the invert elevation and the routed flow elevation.

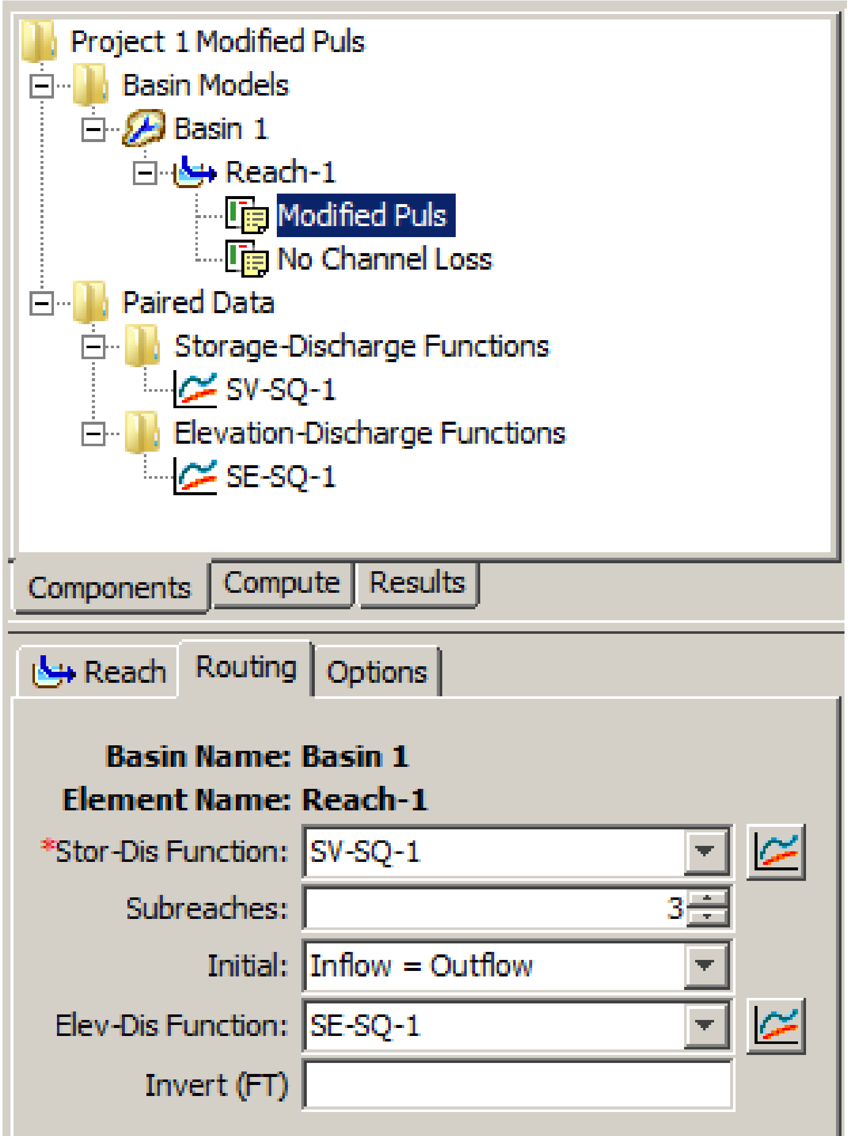


Figure 5–9 Example of the Routing Tab in the Component Editor for the Modified Puls Method

Chapter 6
STORAGE ROUTING

This chapter contains the following details:

- Recommendations and examples of level pool storage routing in HEC-HMS.

6.1 INTRODUCTION

Storage routing should be used when runoff is temporarily detained by the storage capacity or outlet characteristics of a structure such that the flow rate and timing of the outflow is significantly different than that of the inflow. Storage routing is required when runoff passes through retention/detention basins or ponding areas upstream of drainage facilities such as highway cross-drainage structures, particularly where the highway is elevated; where culverts, railroad drainage facilities, or bridges restrict flow rates; or where pump stations exist. Level-pool reservoir routing is used for all of these applications. HEC-HMS performs level-pool storage routing using input data that describe the storage capacity and discharge relations of the storage area and its outlet works. Generally, ADOT does not consider storage impacts upstream from culverts as part of culvert sizing.

6.2 PROCEDURE

For storage routing, topographic, design, and/or as-built information must be available to prepare the necessary input. Due to the diversity of situations for which storage routing can be performed, only general guidance is provided.

6.2.1 Stage-Storage Relation

To perform a storage routing, a relation describing the storage volume relative to water surface elevation must be provided. This description is obtained by one of two methods: 1) examining water surface elevation and its corresponding storage volume (elevation-storage rating curve), or 2) consider water surface elevation and its corresponding surface area (elevation-area rating curve). These data are entered in HEC-HMS using the Paired Data Manager as Elevation-Storage Functions or Elevation-Area Functions. Either method is acceptable. To some extent the selection of the method depends upon the information available. If surface area data are provided, the storage volume is calculated by the HEC-HMS program using the conic formula.

6.2.2 Stage-Discharge Relation

A relation describing the discharge at the outlet(s) of the storage area as a function of water elevation also must be provided. Discharges corresponding to water elevations are entered in the Paired Data Manager. Stage-discharge relations can be established using design reports or hydraulic analysis using software such as HY-8 (FHWA, 2011). Stage-discharge relations can also be computed directly by HEC-HMS by entering the outlet structure elevations, geometry, and discharge coefficients for pipes and spillways.

6.2.3 Structure Overtopping

Structure overtopping can be modeled in HEC-HMS as an outflow structure using the "Dam Tops" option on the "Reservoir" tab when the Outflow Structures method is selected. Level and non-level overtopping weirs can be described for each "Dam Top" specified. For the non-level option, a cross section must be defined in the "Paired Data Manager".

6.2.4 Pump Stations

Pump stations can also be included in HEC-HMS as outflow structures associated with a reservoir. Like Dam Tops, pumps are selected on the "Reservoir" tab and parameterized on the "Pump" tab.

6.2.5 Applications and Limitations

1. Define the stage-storage relation from the best available maps or survey data and input the relation in Elevation-Storage or Elevation-Area Functions in the Paired Data Manager.

2. Define the stage-discharge relation for the outflow structure using an Elevation-Discharge Function in the Paired Data Manager. Alternatively, the Outflow Structures Method on the "Reservoir" tab can be used to define hydraulic outflows.

   Elevation-Discharge Function should be used to define the complete discharge rating curve for all types of discharge through (or over) the structure. These input calculations should be performed for each of the different types of discharge that could occur. A composite discharge rating curve should then be developed by adding together all applicable discharges that occur at any given elevation. This discharge rating curve should extend above the maximum reservoir water surface elevation achieved during the routing operation.

   Also, the same Elevations points should be included in each Function curve entered into the Paired Data Manager. For example, the Elevation-Storage Function should include the same elevations as the Elevation-Discharge Function and so forth. Consistency will ensure proper correlation in HEC-HMS between the two relationships when it interpolates results through the entire routing of the hydrograph.

3. If pump stations are included, and if the pump station capability of the HEC-HMS program is adequate for the analysis, provide pump station information on the "Pump" tab using the Outflow Structure method.

6.3 EXAMPLE

Example No. 6-1 Storage Routing

Determine the storage routing input for a 4 barrel 10' x 5' x 226' CBC as shown in the plan and profile sketch. Include discharge capacity for road overtopping in the stage-discharge rating curve.

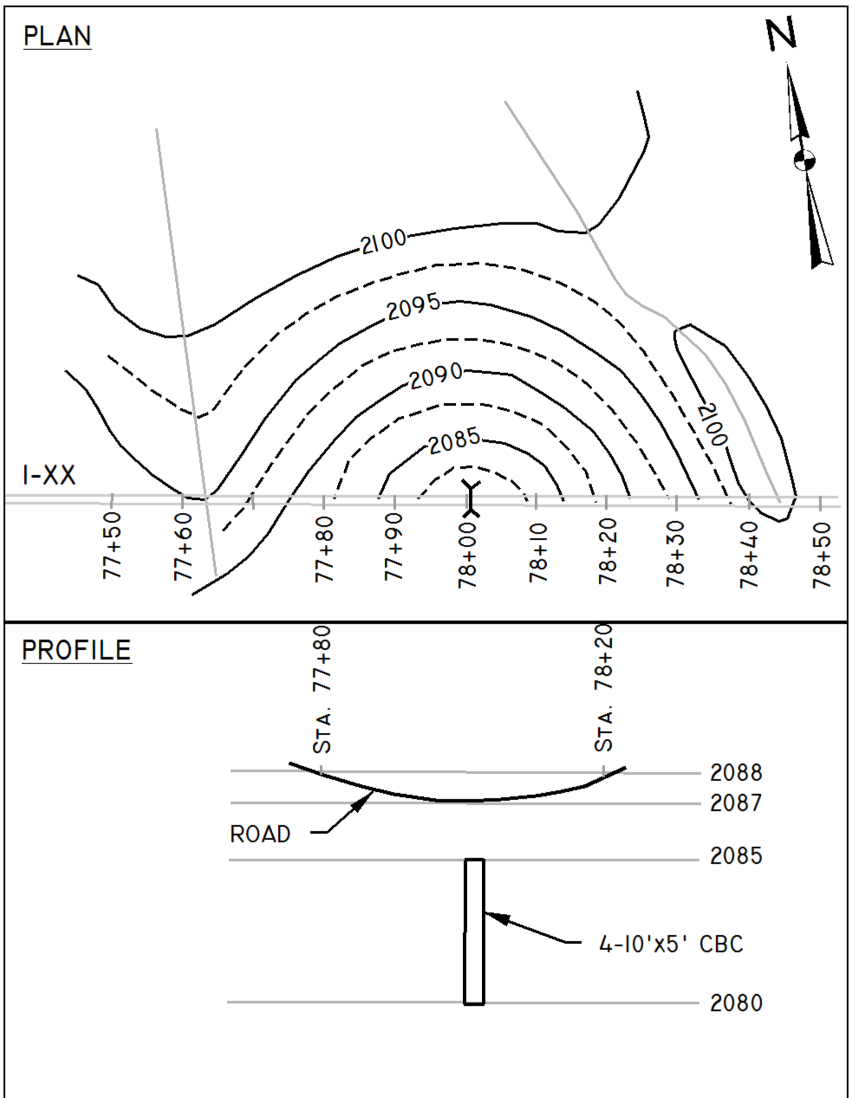


Figure 6–1 Plan and Profile for Reservoir Routing

Table 6–1 Example 6-1 Stage-Storage Relation	
Elevation (feet)	**Area (acres)**
2080.0	0
2081.0	2
2082.0	8
2082.5	12
2083.0	17
2084.0	29
2085.0	44
2086.0	60
2087.0	78
2087.5	89
2088.0	101

Table 6–2 Example 6-1 Stage-Storage Calculation		
Elevation (feet)	**Volume Calculation**	**Volume (ac-feet)**
2080.0	0	0
2081.0	(2+0)/2 ac * 1 ft	1
2082.0	1 ac-ft + [(8 + 2)/2 ac * 1 ft]	6
2082.5	6 ac-ft + [(12 + 8)/2 ac * 0.5 ft]	11
2083.0	11 ac-ft + [(17 + 12)/2 ac * 0.5 ft]	18
2084.0	18 ac-ft + [(29 + 17)/2 ac * 1 ft]	41
2085.0	41 ac-ft + [(44 + 29)/2 ac * 1 ft]	78
2086.0	78 ac-ft + [(60 + 44)/2 ac * 1 ft]	130
2087.0	130 ac-ft + [(78 + 60)/2 ac * 1 ft]	199
2087.5	199 ac-ft + [(89 + 78)/2 ac * 0.5 ft]	241
2088.0	241 ac-ft + [(101 + 89)/2 ac * 0.5 ft]	288

Table 6–3 Example 6-1 Stage-Discharge Relation

Elevation (feet)	Discharge, cfs		
	CBC	Overtopping	Combined
2080.0	0	0	0
2081.0	130	0	130
2082.0	350	0	350
2082.5	480	0	480
2083.0	630	0	630
2084.0	950	0	950
2085.0	1290	0	1290
2086.0	1630	0	1630
2087.0	1930	0	1930
2087.5	2070	750	2820
2088.0	2200	3240	5440

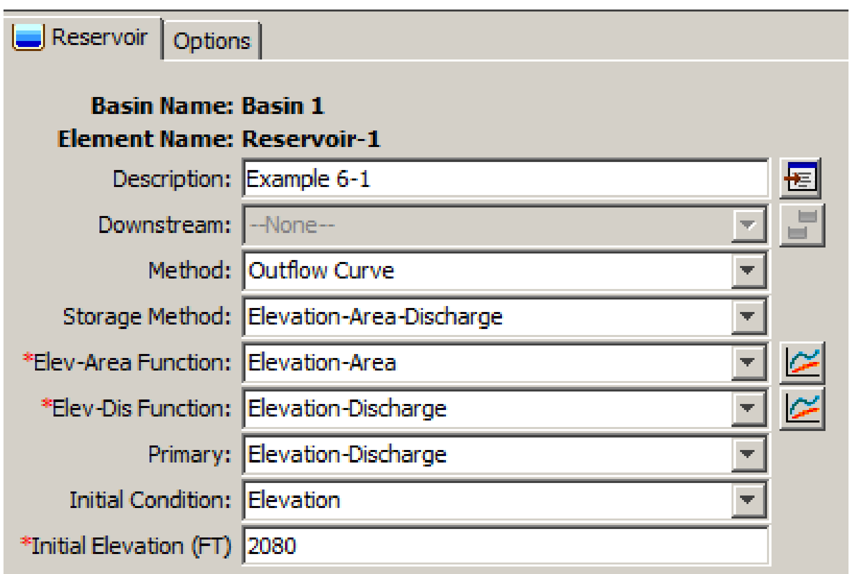


Figure 6–2 Example of the Reservoir Tab in the Component Editor for the Reservoir Routing Method

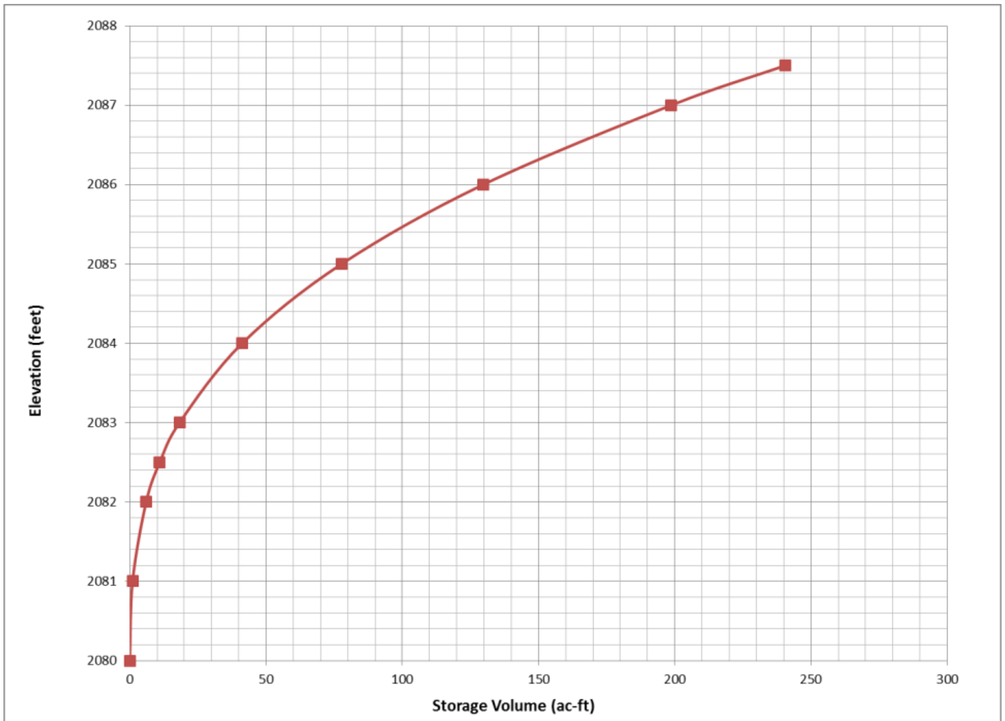


Figure 6–3 Example of Stage-Storage Plot

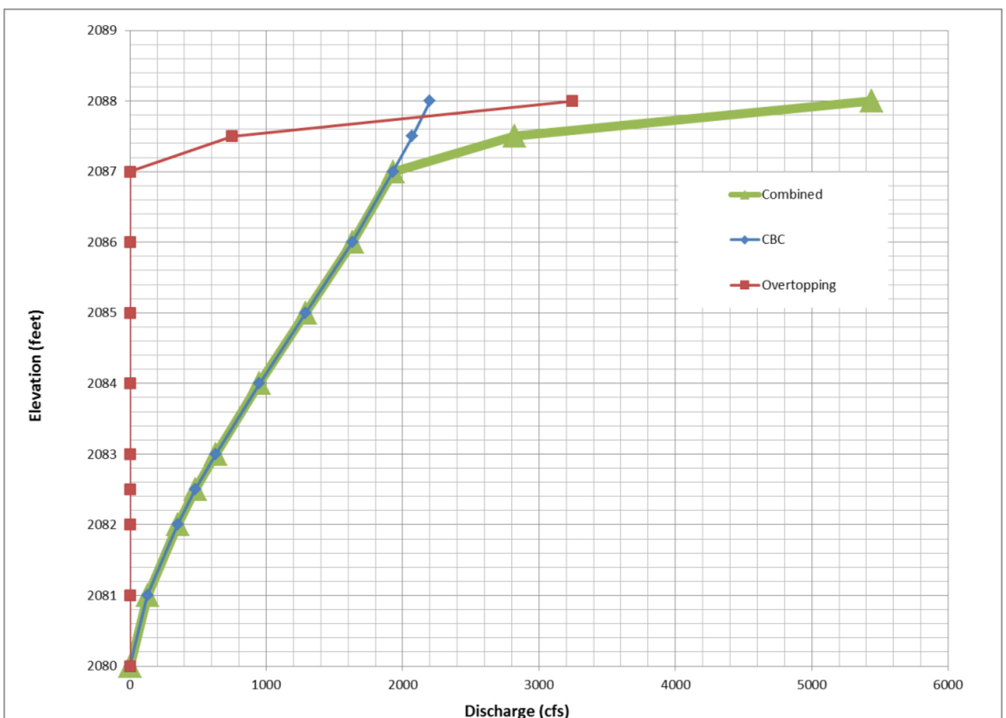


Figure 6–4 Example of Stage-Discharge Plot

Chapter 7
TRANSMISSION LOSSES

This chapter contains the following details:

- Transmission loss, or percolation, through the channel bed.
- Guidelines on when to use transmission losses.

7.1 INTRODUCTION

Incorporation of transmission losses in a watershed rainfall-runoff model requires the approval of ADOT. Storm runoff and floods in Arizona are usually attenuated through the effects of channel and storage routing, but often they are also diminished due to the percolation of water into the bed, banks, and overbank floodplains as the flood wave is conveyed along watercourses. These losses are called transmission losses, which accrue separately from the rainfall losses. Transmission losses can result in significant reductions of runoff volume, especially on very long reaches with wide channels and floodplains with high percolation rates. In narrow confined channels in impermeable soils, transmission losses may be negligible.

The magnitude of transmission loss (both volumetric and peak discharge) is dependent upon factors such as the antecedent moisture condition of a watercourse; the textural characteristics of the bed, bank, and floodplain sediments; the channel geometry (wetted perimeter); the depth to bedrock; the depth to the ground water table; the duration of flow; and the hydrograph shape, as shown in **Table 7–1**. Some of these factors may also vary temporarily. For a watercourse that is initially dry and is composed of coarse, granular material, the initial percolation rate can be very high. However, the percolation rate diminishes during passage of the flood and would eventually reach a steady-state rate if the flow continues long enough.

Although it is recognized that transmission losses can be an important element in performing rainfall-runoff modeling, particularly for ephemeral watercourses in Arizona, procedures and reliable data for estimating transmission losses are generally not available. Therefore, except for situations in which transmission losses should clearly be incorporated in the analysis, the estimation of these losses will not usually be incorporated in rainfall-runoff models. The incorporation of transmission losses in a watershed rainfall-runoff model should be approved in advance by ADOT, and the procedure and assumptions used to estimate such losses should be clearly documented.

Table 7–1 Factors that Affect Transmission Losses

Factors that Decrease Transmission Losses	Factors that Increase Transmission Losses
Clay soils	Sandy soils
Narrow floodplains	Wide floodplains
Steep channel slopes (faster travel time)	Flat channel slopes (slower travel time)
Shallow depth to bedrock	No bedrock present
High antecedent moisture of channel bed	Low antecedent moisture of channel bed
Short duration flows	Long duration flows

Transmission losses can be modeled in HEC-HMS using the Percolation Loss/Gain option within the Muskingum-Cunge and Modified Puls routing methods. The loss/gain option is selected on the channel routing data entry screens for each routing reach by selecting of the “Loss/Gain Method” as percolation or constant. The percolation “Loss/Gain Method” should be used. Conductivity values for the soil units underlying the channel and floodplain can be used to estimate percolation rates. Ranges of typical transmission loss values are provided in Table 7–2 for various bed material types.

The “Constant” loss is not recommended for hydrologic modeling because of the subjectivity involved with selecting the constant loss rates. The recommended percolation rate method is physically-based and should result in better estimates of transmission losses.

7.2 PROCEDURE

In general, transmission losses will be more significant, and may be added to a rainfall-runoff model, if the following conditions exist:

1. The bed, banks, and overbank floodplains of the watercourse are composed of coarse, granular material. Materials such as cobble, gravel, sandy gravel, gravelly sand, sand, and sandy loam are all indicators that appreciable transmission losses can occur.

2. There is a relatively long channel routing reach over which transmission losses might occur. The channel routing reach should at least be long enough that significant attenuation occurs without consideration of transmission losses.

3. The channel and floodplain routing reach is wide, with a high width/depth ratio, and with a large surface area of highly permeable soils.

4. The watercourse is ephemeral, and it is reasonable to assume that the watercourse is dry before the onset of the storm.

5. The bed of the watercourse is not underlain by impermeable material, such as bedrock, calcium carbonate (caliche), or clay-rich sediments, that would inhibit the sustained percolation of water into the bed of the watercourse.

6. The depth to ground water is great enough to not inhibit the sustained percolation of water into the bed of the watercourse.

7. The hydrograph volume is low relative to the volume of potential transmission loss.

If the above conditions are met, then the incorporation of transmission losses into the model may be considered, and the following two other factors should be considered before proceeding:

1. Adequate information must be available to provide input for the selected routing method, so that the percolation rate can be satisfactorily estimated.

2. Incorporation of transmission losses will require a multiple subbasin model with defined routing reaches. Transmission losses are calculated for the routing reaches.

In HEC-HMS, transmission loss data are entered under “Loss/Gain Method” in the “Reach” tab for each routing reach. Although measured percolation rates have been found to be highly variable, Table 7–2 provides some guidance for their selection. If using the 8-point cross section option or a stage-discharge function as part of the routing input, the elevation of the channel invert should correspond to the lowest elevation used in the 8-point cross section or stage-discharge function for that routing reach.

Table 7–2 Percolation Rates for Various Channel Bed Materials (from SCS National Engineering Handbook Section 4, Chapter 19, Transmission Losses, by L.J. Lane)

Bed Material	Transmission Loss Class	Percolation Rate (inches/hour ≈ cfs/acre)
Very clean gravel and large sand	Very high	>5
Clean sand and gravel, field conditions	High	2.0 – 5.0
Sand and gravel mixture with low silt-clay content	Moderately high	1.0 – 3.0
Sand and gravel mixture with high silt-clay content	Moderate	0.25 – 1.0
Consolidated bed material; high silt-clay content	Insignificant to low	0.001 – 0.1

Chapter 8
MODELING GUIDANCE FOR HEC-HMS

This chapter contains the following details:

- A summary of model application guidance.
- An outline of the modeling process.

8.1 INTRODUCTION

The rainfall-runoff modeling procedures outlined in this manual are intended for use with the HEC-HMS Hydrologic Modeling System (U.S. Army Corps of Engineers, 2010), which is available from the Hydrologic Engineering Center's website (http://www.hec.usace.army.mil/). HEC-HMS (ver. 3.5), as of fall 2012, continues to be advanced and supported by USACE. The software is free and is widely used for drainage design across the United States.

This chapter contains an overview of the major theoretical assumptions upon which the HEC-HMS computer program is based, and the resultant limitations. Watershed modeling techniques are presented, and these are related to some of the common errors made when using the HEC-HMS program. A modelers/reviewer's checklist is presented for use by both ADOT engineers and ADOT consultants in developing and reviewing HEC-HMS watershed models.

8.1.1 Assumptions and Limitations of HEC-HMS

Proficiency in use of the HEC-HMS program requires an understanding and appreciation of the following basic underlying model assumptions and limitations:

Deterministic:

The HEC-HMS program treats the rainfall-runoff process as deterministic. Randomness of the process (within both the temporal and spatial domain) is not considered. The effects of natural variability can be investigated by making numerous runs of a HEC-HMS model with changes to input variables.

Lumped Parameter:

Many of the model parameters (for example, the Green and Ampt infiltration parameters) represent spatial averages of highly variable characteristics. These are "lumped" parameters that are intended to represent average conditions for a watershed subarea, not values at all specific points in the watershed.

Unsteady Flow:

The flow rates forecasted by the model vary with time as reported in the resulting hydrographs at each computational location within the model.

8.2 WATERSHED MODELING

8.2.1 Modeling Process

The following general steps are encouraged in performing rainfall-runoff modeling:

1. Collect all pertinent information for the watershed:
    a. Maps
    b. Aerial photographs
    c. Soil surveys/data
    d. Land-use maps/data/reports
    e. Reports of flooding
    f. Streamflow data (if available)
    g. Other flood study reports (FEMA, county, etc.)
2. Prepare a watershed base map using the best available map(s)/data and the most practical map scale.
3. Perform a preliminary subbasin delineation using best available topographic data and aerial photographs.
4. Conduct a field reconnaissance.
5. Finalize the subbasin delineation.
6. Prepare the rainfall input.
7. Prepare a preliminary model schematic diagram.
8. Prepare the rainfall loss input.
9. Prepare the unit hydrograph input.
10. Prepare all routing input.
11. Prepare HEC-HMS model components (Meteorologic, Basin, and Control).
12. Execute the HEC-HMS model.

13. Debug, calibrate, and refine the model to best represent actual watershed conditions.

14. Iterate revisions until results are satisfactory.

15. Execute the final HEC-HMS model.

16. Make final model verifications.

17. Prepare a report.

8.2.2 Model Logic

A schematic diagram for multiple subbasin models should be prepared and included as a part of the final report. The model logic diagram symbolically depicts the order of combining and routing hydrographs. The supporting data to be included in a tabular format include:

1. Subbasin data (subbasin name, area, T_c)

2. Channel routing data (length, slope, average "n" value, base width and/or other dimensions, transmission loss rate)

3. Storage routing data (stage, storage values)

The model diagram is also depicted within the HEC-HMS input (**Figure 8–1**). Use of GIS data can facilitate more accurate and easily understood visual presentation of the model logic structure within the HEC-HMS input data itself. These data can be added as background layers in the Basin Model view in HEC-HMS.

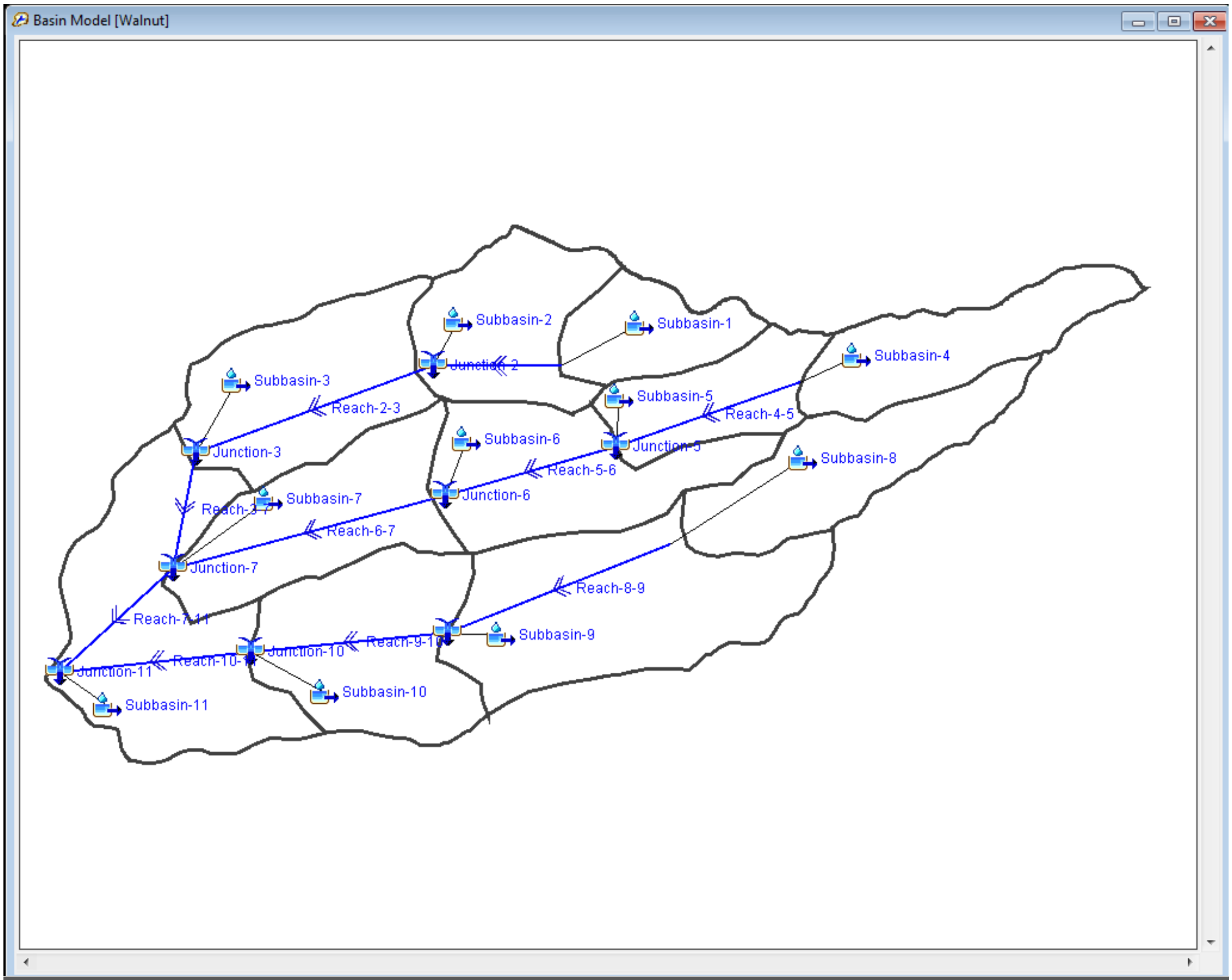


Figure 8–1 Example of Model Diagram in the HEC-HMS Component Editor

8.2.3 Model Simulation Time and Computation Time Interval

The model simulation time period and computation time interval are specified in the Control Specifications in HEC-HMS.

The model simulation time period should span at least 24 hours to cover the entire rainfall event duration. Additional time may be required for larger watersheds where routing of the peak discharge through the entire model area extends beyond a 24-hour period. Model hydrographs should be plotted and examined at the downstream model limits to determine if a sufficient model time period has been simulated.

Generally, a model time interval of 5 minutes will be used. However, the time interval should also be checked against the time of concentration (T_c) guidelines discussed in Section 8.2.4.3.

8.2.4 Subbasin Delineation

The process of breaking down a watershed into subbasins should be done with careful consideration given to the factors listed below. Defining these factors prior to beginning the delineation will help to ensure that the model remains within the limitations of the methodology used. It will also help avoid extensive revisions after the fact. These factors are as follows:

8.2.4.1 Concentration Points

Identify locations where peak flow rates or runoff volumes are desired. The following locations, as a minimum, should be considered:

1. Confluences of watercourses where a significant change in peak discharge may occur.
2. Existing or proposed drainage structures.
3. Crossings of major collector or arterial streets.
4. Jurisdictional boundaries.

8.2.4.2 Subbasin Size

Using the concentration point locations, estimate a target average subbasin size to strive for, and estimate the smallest expected subbasin. Excessive subbasin division is discouraged. Additional criteria for subbasin size are described in **Section 8.2.4.4**.

8.2.4.3 Time of Concentration

Estimate the time of concentration (T_c) for the smallest subbasin. Using this value, determine the integer number of minutes for the computation interval. A computation interval of 0.15 * T_c will provide adequate definition of the hydrograph peak. Per guidance in **Chapter 4**, the computation interval should not exceed 0.25 × T_c for the subbasin with the shortest T_c. A computation interval of 5 minutes will usually be adequate to meet both criteria.

8.2.4.4 Homogeneity

Considerations for subbasin homogeneity, in order to meet the Lumped Parameter assumption are:

1. The subbasin sizes should be as uniform as possible.
2. Each subbasin should have nearly homogeneous land-use and surface characteristics. For example, mountain, hillslope, and valley areas should be separated into individual subbasins wherever possible.
3. Soils and vegetation characteristics for each subbasin should be as homogeneous as is reasonably possible.

The average subbasin size may need to be adjusted (addition of concentration points) as required, in order to satisfy the key assumptions upon which the HEC-HMS model is based.

8.2.4.5 Routing Lengths

The length of the channel reaches defined as a result of the subbasin delineation should be considered while breaking down the watershed. If short reaches are required in the watershed subdivision, combine hydrographs directly rather than route through a reach that is too short.

The Muskingum-Cunge method recommended in this manual uses the automatic fixed time interval method which does not require the number of subreaches to be input. See **Section 5.2.1** for additional discussion.

The Kinematic Wave method requires an initial estimate of the number of subreaches to determine the correct distance step used during the routing calculations. The default value in HEC-HMS is 2 but may be optionally increased if needed.

When using the Modified Puls routing method for routing a hydrograph through a channel reach, a key user input parameter is the number of subreaches. The number of subreaches affects attenuation. One subreach produces the greatest attenuation and a large number of subreaches produce little or no attenuation. The number of subreaches should be determined as a function of the reach length, travel time, and computation interval. A good way to estimate the number of subreaches is to divide the total reach length by the flow velocity and the computation interval. Remember to account for proper units. The actual travel time computed by HEC-HMS should be compared to the assumed flow velocity and the number of subreaches adjusted if needed. See **Section 5.2.4** for additional discussion.

8.2.5 Precipitation

For a multiple subbasin model, the storm area must be specified for the Frequency Storm so that the correct rainfall depth-area reduction factor will be applied. Normally, this is the total drainage area to the primary concentration point at the model outlet. If design discharges are needed at internal concentration points within the basin model, then either several different models will need to be developed (one for each concentration point of interest) or the Depth-Area Analysis option can be used. The internal points of interest can be added as analysis points to obtain the correct areal reduction for rainfall to each point of interest. The HEC-HMS output must be carefully examined to obtain the correct results for each point of interest when using the Depth-Area Analysis option. Consult the instructions in the HEC-HMS User's Manual for additional information about use of the Depth-Area Analysis option in conjunction with the Frequency Storm.

8.2.6 Rainfall Losses

This manual uses lumped parameter rainfall loss rate information, which is intended to be evenly distributed within each subbasin.

The percent impervious value (RTIMP) is the percent of the subbasin area for which one hundred percent runoff will be computed. This means that the impervious area is assumed to be hydraulically connected to the concentration point. This parameter should be used with care. For urban areas, RTIMP is usually less than the total impervious surface area. Natural rock outcrop is not often directly connected to the watershed outlet. Therefore, the total rock outcrop area reported in the soil survey descriptions is seldom equal to the effective impervious area.

8.2.7 Time of Concentration (T_c)

Some watersheds may require estimation of several possible paths to the hydraulically most distant point to find the largest T_c value. This path will often have the longest flow path at the flattest slope.

Since the unit hydrograph method is extremely sensitive to the T_c parameter, every estimate should be checked for reasonableness. Because of the numerous watershed characteristics that influence T_c, verification of this parameter can be difficult. However, an evaluation of average flow velocities through a subbasin can yield worthwhile information on the validity of the computed T_c value.

Any attempt to verify T_c calculations by using an average flow velocity analysis should be pursued with caution. Due to the large influence that overland flow travel time has on the subbasin T_c, an average flow velocity that is computed as simply L/T_c, where L is the length of the subbasin watercourse to the hydraulically most distant point, will normally yield an average velocity that will appear unrealistically low for the open channel flow component of the T_c value. Since overland flow velocities are normally on the order of a few tenths of a foot per second, they can consume a very large proportion of the time of concentration for a subbasin.

Case studies have shown that it is not unusual for a simple L/T_c calculation to produce average flow velocities that are on the order of 2 to 3 fps for channels with slopes in excess of three percent. Such low velocities would not normally be considered reasonable for the channel component in steep-sloped watersheds.

Accordingly, a velocity analysis of T_c should consider separating the open channel flow contribution of T_c from the overland flow portion of T_c. Average velocities can be computed for each flow regime and then applied to the flow path length that would be associated with each of these regimes. By dividing the flow path length for each regime by the average velocity for each regime, a travel time can be computed for each flow regime. The total subbasin travel time computed by such an approach should be similar in magnitude to the estimated T_c value.

The following guidelines are suggested for computing the travel times for each flow regime:

<u>Open Channel Flow</u>

1. Use a 4-point trapezoidal cross-section to approximate the average main channel geometry for the subbasin. The approximate cross-sectional geometry, depth, and roughness should be based on field inspections whenever possible.

2. Record the channel slope value that was used for the T_c calculation.

3. Apply the data from Steps 1 and 2 to Manning's equation to compute the average channel velocity that is associated with the bankfull discharge of the channel.

4. Record the length (L) of the subbasin watercourse that was used for the T_c calculation.

5. Compute the open channel travel time by dividing the watercourse length from Step 4 by the average velocity from Step 3.

Overland Flow

Compute the overland flow travel time with the following equation:

$$T_{OF} = \frac{0.007(nL)^{0.8}}{(P_2)^{0.5}S^{0.4}} \qquad 8.1$$

where:
T_{OF} = overland flow travel time (hours),
n = overland flow roughness,
L = overland flow length (feet),
P_2 = 2-year, 24-hour rainfall (inches), and
S = overland flow slope (feet/feet).

Equation 8.1 is taken from Technical Release 55 (SCS, 1986). Guidelines for selecting the overland flow roughness (n) are provided in the SCS reference, as well as in Table 14 of the HEC-HMS Technical Reference Manual. Overland flow lengths are generally less than 300 feet.

8.2.8 Hydrograph Operations

The primary hydrograph operations available with the HEC-HMS program, other than routing options, are combining and diverting of hydrographs. The combine operation is performed with a junction element on the number of specified hydrographs connected with the junction. If intermediate combination results are desired, multiple Junction elements will be needed.

Hydrograph diversions may be used to simulate flow splits that may occur at street intersections, at elevated highways, or at distributary channel bifurcations. The diversion operation is performed using a Diversion element in HEC-HMS. Key points to remember about this operation are:

1. The preferred method to define a split uses a discharge rating table defined in the Paired Data Manager. A maximum flow rate or maximum volume cutoff option may also be specified if needed.

2. It is important to check the shape of diverted hydrographs for oscillations and to verify that the expected results are obtained.

3. When the diverted outflow from a Diversion element is combined into another downstream element, the drainage area associated with the diverted hydrograph is zero. Similarly, the continuing flow will retain the entire contributing area at the inflow to the

Diversion element. When using diversions with the Depth-Area Analysis, carefully evaluate the impact of the drainage area for each point downstream of any diversions. Separate models may be needed to develop correct flow rates for internal concentration points downstream of diversions.

8.2.9 Channel Routing

As presented in Chapter 5, the Muskingum-Cunge is the preferred channel routing method. In cases of uniform constructed channels, the Kinematic Wave method may be used. Finally, in reaches with significant backwater effects, the Modified Puls method should be used. Some general considerations when implementing these channel routing methods are discussed in the following sections.

8.2.9.1 Number of Subreaches

The Muskingum-Cunge method uses the automatic fixed time interval method which does not require the number of subreaches to be input.

The Kinematic Wave method requires an initial estimate of the number of subreaches to determine the correct distance step used during the routing calculations. The default value in HEC-HMS is two, but may be increased if needed.

As discussed in **Section 8.2.4.5**, when using the Modified Puls routing method for channel routing, the user must input the number of subreaches. The number of subreaches affects attenuation. One subreach produces the greatest attenuation, and a large number of subreaches results in little or no attenuation. The number of subreaches should be determined as a function of the reach length, travel time, and computation interval. A good way to estimate the number of subreaches is to divide the total reach length by the flow velocity and the computation interval. The actual travel time computed by HEC-HMS should be compared to the assumed flow velocity and the number of subreaches adjusted as needed. See **Section 5.2.4** for additional discussion.

8.2.9.2 Channel Geometry

When using the Muskingum-Cunge method, an eight-point cross section may be specified to describe the routing reach. Considerations for selection of the appropriate cross section, which should be checked by field reconnaissance when possible, are:

1. All eight points on the cross section should be meaningful.
2. Be sure there is sufficient hydraulic capacity to convey the peak flow without overtopping the section.
3. Be sure that the cross section is representative of the average characteristics of the reach. If there are significant variations in section geometry, the reach should be broken down into multiple shorter reaches.

4. Verify that the Manning's n values for the cross section are representative of the average characteristics of the reach. If there are significant variations in roughness, the reach should be broken down into multiple shorter reaches.

8.2.10 Reservoir Routing

Modeling of reservoirs and detention basins can be accomplished using a Reservoir element in HEC-HMS. It is recommended that low level outlets, spillways, and structure overtopping be modeled using an elevation-discharge rating curve input in the Paired Data Manager. The rating curve should be developed using appropriate manual or external software calculation methods.

8.3 MODELER'S/REVIEWER'S CHECKLIST

The following is a checklist for the HEC-HMS input and output.

8.3.1 HEC-HMS Input

8.3.1.1 General

HEC-HMS has description fields for every input parameter and element. Liberal use of these descriptive fields should be used to facilitate understanding of the project-specific and location-specific context of each model component and model element. Logical naming conventions should also be established to make it easier for the modeler and model reviewer to understand the structure and organization of the model.

8.3.1.2 Control Specifications

1. Time Interval – make sure the time interval specified conforms to the recommendations relative to the shortest time of concentration (Section 8.2.4.3 and Chapter 4).

2. Start and End Time – make sure at least 24 hours is specified. Additional time may be needed to ensure adequate hydrograph routing for larger areas or longer routing reaches.

8.3.1.3 Basin Models

1. Basin models should be logically named and described.

2. Separate basin models should be developed for each point of design unless using the Depth-Area Analysis.

3. Background layers, such as topographic maps or aerial photographs, should be added to the desktop component of the HEC-HMS model to facilitate understanding of the model structure.

8.3.1.4 Meteorological Models

1. The Meteorological Model defined should use the HEC-HMS Frequency Storm option.

2. The event frequency defined should match the design frequency required for the project.

3. The Input type may be Partial Duration series data when using partial duration statistics from NOAA Atlas 14.

4. A one day storm duration should be specified with the Intensity position at 50 percent.

5. If a multiple subbasin model is used, the total watershed area, in square miles, should be specified as the storm area.

8.3.1.5 Rainfall Loss Records

Green and Ampt Method

Seven parameters are needed for each subbasin to define the Green and Ampt rainfall loss parameters using the recommended methods in this manual. Remember they are lumped parameters representing the average condition for the entire subbasin. Critical values are the conductivity value, in inches/hour and the imperviousness, given in percent. The impervious area is the directly connected impervious area, in percent. No rainfall losses are calculated for the impervious area.

Initial and Constant Loss Method

This method is only to be used if the Green and Ampt method is inappropriate. Again, imperviousness represents the directly connected impervious area, in percent. No rainfall losses are calculated for this area.

8.3.1.6 Unit Hydrograph Input

The use of the Clark unit hydrograph method is recommended. Two parameters are computed – time of concentration, T_c, and a storage coefficient, R, according to the equations presented in **Chapter 4**. The computation time interval specified in the Control Specifications for each Basin Model should meet the requirements presented in **Section 8.2.3**.

8.3.1.7 Junctions

Junctions are specified in HEC-HMS to combine or add two or more hydrographs together. When more than two hydrographs require combining, multiple junctions may be desired in order to obtain intermediate combined results directly. Alternatively, the intermediate flows can be computed externally by adding the tabular hydrographs reported in the time-series output for the junction.

8.3.1.8 Channel Routing

Input varies according to selected channel routing method. For the preferred routing method, Muskingum-Cunge, special note of the following input should be taken:

Length

The length, in feet, represents the flow length for most of the flow during the hydrograph, which may be longer or shorter than the low flow thalweg distance.

Slope

The routing reach slope should be representative of the entire routing reach. If the channel slope or cross section varies significantly along the routing reach, subdivide the reach into multiple channel routing reaches.

Manning's n-values

Channel and overbank roughness n values should be representative of the entire reach over the range of expected flows. Initial results for flow depths should be examined and roughness values adjusted if necessary.

Cross section tables

1. Cross sections are defined in the Paired Data Manager.
2. Cross section stations and elevations are input in feet. Sequential station (x-axis) values must increase in magnitude.
3. The cross section must be "typical" for the routing reach.
4. View the plotted cross section to verify shape and input.
5. The defined cross section must have adequate capacity to contain the peak discharge. If not, the model will extend the two end stations vertically, and this is usually inappropriate for broad, shallow overbanks in Arizona, and will result in underestimating attenuation.
6. Care must be exercised in defining the channel geometry to avoid including ineffective flow areas.

8.3.1.9 Storage Routings

Storage routings are specified as Reservoir elements in HEC-HMS. Most of the input data are provided as tables for elevation, storage volume, and discharge entered via the Paired Data Manager. The paired relationships should be plotted to help make sure the input is correctly entered. Discharges may also be specified using built in functions in HEC-HMS for structures such as low level outlets and weirs. However, the preferred method is to define the stage-discharge functions externally using hand calculations or other hydraulic software. Storage volumes may also be computed internally in HEC-HMS using elevation-area relationships, although external computation of elevation-volume relationships is preferred.

It is suggested that the names and description fields be used in the Paired Data Manager so that the correct tables can be assigned to the correct storage routings with minimal confusion.

8.3.1.10 Transmission losses

The preferred method for transmission losses is to specify the unit area percolation rate, in cfs/acre, using the Percolation Loss/Gain Method. The inflow and outflow runoff volumes and hydrographs should be checked to verify if the selected percolation rate is reasonable. Incorporation of transmission losses in a watershed rainfall-runoff model requires the approval of ADOT.

8.3.2 HEC-HMS Output

Much of the HEC-HMS output can be viewed in table form, as graphs, or other plot summaries within HEC-HMS from the “Results” tab. Data can also be viewed from the .DSS file with HEC DSSVUE or from the DSS viewer in HEC-RAS. These tools are especially helpful if the user wants access to the time-series flow velocity or stage data for channel routings.

8.3.2.1 Basin Map/Schematic

Check the basin map (aka model schematic window). Follow the schematic on the watershed map and see if it is correct and reasonable.

Make sure all the model elements are connected and in the proper sequence. All upstream subareas must be combined before routing through a downstream channel.

Make sure that any diverted hydrographs have been accounted for.

8.3.2.2 Area

Check the accuracy of the total drainage area. Normally, for basins with a single outlet, the easiest way is to check the last number on the "area" column in the Global Summary table. For basins with several outlets, the contributing area for each outlet may have to be added together and then checked for accuracy.

Previous studies of the watershed may also prove useful for comparison of areas.

8.3.2.3 Rainfall Losses

Check the runoff volume in the Global Summary table. Check the runoff volume column for inconsistency. Inconsistencies in estimated losses must be examined. If any subbasins report zero or a very small number, the more detailed output for that subbasin should be examined. Check the time-series results, the total rainfall, total losses and total runoff. Then check the input for any offending subbasins and revise as needed.

8.3.2.4 Routing

1. Check the applicability of the routing methodology.

2. Check that the outflow is not greater than the inflow.

3. Check for instability in the outflow hydrograph by examining the plotted hydrograph for oscillations or irregular shapes.

4. When using the 8-point cross section option, check to see that the flow is contained within the channel by examining the Computed Stage plot in the "Results" tab for each routing. HEC-HMS will extend the end of the cross section vertically if the channel cross section area is not large enough. Extend the cross section if needed.

5. Check travel time. Travel time can be translated back to velocity or wave celerity. If the travel time seems too long or too short, examine the input parameters for the routing. The computed flow velocity throughout the hydrograph can be plotted for each routing in the "Results" tab.

6. Routing procedures will normally result in some attenuation of the peak flow unless using the Kinematic Wave method. The amount of attenuation (or lack of) should be checked for reasonableness.

7. Routing will not only attenuate the flow, but will also delay the peaks and therefore will separate the inflow and outflow hydrographs in time. This separation of peaks can have a substantial effect when combining hydrographs and on the resulting peak at the outlet. Choosing short reaches or using large computation time intervals will cause the peak time to default to the nearest time interval, which can be zero (instantaneous translation of the hydrograph through the reach). The cumulative effect of these actions may result in substantial error. Plotted hydrographs should be examined in the "Results" tab.

8.3.2.5 Peak Runoff

Since HEC-HMS does not provide a summary table showing unit discharge (cfs/square mile), it is recommended that reviewers develop this information themselves. Unit discharges could be used to compare flows from one subbasin with another. Since unit discharge depends on many factors such as area, slope, losses, and so forth, this comparison may be difficult. However, large differences in unit discharge should alert the reviewer to check the input for discrepancies.

8.3.2.6 Time to Peak

Check the time to peak (T_p) column in the Global Summary table:

1. Generally T_p's are expected to increase with drainage area size. If all the T_p's appear to coincide or are very close, the computation time interval must be examined or changed and routing operations should be changed.

2. Check that the T_p's occur after the most intense portion of the rainfall period (12:00 to 12:05 if the 1 day storm is started at midnight).

8.3.2.7 Volumes

Check the output to determine if the volume of runoff is reasonable. This assessment may prove to be somewhat difficult since there are very few "yard sticks" developed for comparing runoff volumes. Experience and published reports should be relied upon to determine if the runoff volumes are reasonable. HEC-HMS allows easy viewing of computed runoff volumes in both total acre-feet and inches per unit area. The percentage of rainfall converted to runoff can then also be easily computed from the runoff in inches to evaluate whether computed runoff volumes are reasonable.

8.3.2.8 General

1. Compare the peak flows and unit discharges (peak flow/drainage area) against available data for the region. Inconsistencies in unit discharges may indicate to the reviewer that errors exist in the HEC-HMS input.

2. Keep the subbasin areas as uniform as possible. Otherwise, it is easy to overestimate the peaks for small subbasins and underestimate the peaks for large subbasins.

3. Separate mountainous areas from the adjacent valleys. In many topographically complex watersheds, much of the peak discharge is generated in the steep mountainous terrain, but is attenuated as it is conveyed through the flatter, less confined valley floor. Mixing the two areas in a single subbasin may lead to incorrect results.

4. Peak discharge is highly dependent on the time of concentration. Predicted flow volume is more sensitive to loss functions.

5. When calibrating a model, make sure that parameter adjustments realistically reflect watershed conditions and probable causes of modeling discrepancies. For example, rainfall losses should not be adjusted where time of concentration is the major cause of the differences between modeled and known hydrographs.

6. Time of concentration and lag are not interchangeable. It is important to use them properly since peak flows are extremely sensitive to these parameters.

7. Manning's friction coefficient for routing must be used properly for main channel and overbanks. If sheet flooding is present, the n values must be adjusted accordingly to account for broad, shallow flow.

8. When comparing existing versus proposed conditions, all the model parameters (rainfall losses, unit hydrographs, routing, and so forth) must be adjusted accordingly. For example, modeling future urbanization involves more than just increasing the subbasin imperviousness. The potential effects of development on time of concentration, watershed boundaries, channel routing, rainfall losses, and reservoir storage should also be modeled.

Chapter 9
MODELING GUIDANCE FOR FLO-2D

This chapter contains the following details:

- Guidance on when to use two-dimensional versus one-dimensional modeling.
- Modeling guidance for developing FLO-2D models.

9.1 INTRODUCTION

FLO-2D is a dynamic two-dimensional hydrologic and hydraulic model that conserves volume as it routes hydrographs over a system of square grid elements. The model routes runoff over the grid using the full dynamic wave momentum equation and a central finite difference routing scheme. The flood wave progression is affected by the surface topography and roughness values (Manning's n-values) associated with land use characteristics.

This section includes guidelines to be used when modeling hydrology using FLO-2D.

The guidance provided in this manual is written for FLO-2D software Version 2009.06.

Some of the key concepts behind hydrological modeling using FLO-2D are summarized below:

1. FLO-2D is based on mass conservation. It models physical processes of water flow by solving the full dynamic wave momentum equation.
2. The momentum equation is solved by computing the average flow velocity across a grid element boundary one direction at a time.
3. FLO-2D incorporates a variable explicit time-stepping scheme enabling relatively fast simulations.
4. Overland flow on unconfined surfaces is simulated using eight possible flow directions from any given grid cell.
5. Flood wave attenuation can be analyzed with hydrograph routing.
6. The flow regime can vary between subcritical and supercritical both spatially and temporally within the same model.
7. Flow over adverse slopes and backwater effects can be simulated.
8. Hydrologic phenomena such as rainfall, infiltration losses and runoff can be modeled.

9. Because FLO-2D is a two-dimensional routing model, branching, distributary, split, and sheet flow can be simulated, as well as flow in multiple channels.

10. Channel flow can be routed with either a rectangular or trapezoidal geometry or natural cross section data.

11. Streets are modeled as shallow rectangular channels.

12. The effects of flow obstructions such as buildings, walls and levees that limit storage or modify flow paths can be modeled.

13. Hydraulic structures such as bridges, culverts and storm drains are modeled using user-defined rating curves.

14. While the number of grid and channel elements and most array components can be considered unlimited, the model run-times may limit the size of the model domain and selection of the grid cell size.

15. Computations are performed at each grid element using the specified input parameters and the results computed at each grid element within the model computational domain.

The general procedure for developing distributed hydrology FLO-2D models involves the following steps:

1. Determination of study area and delineation of the model boundary, which is referred to as the computational domain.

2. Discretization of the area within the model boundary into smaller units known as grid cells.

3. Estimation of model parameters such as topographical elevation, point rainfall depth, and rainfall loss parameters to provide cell-average values as input at each grid cell within the computational domain.

4. Incorporation of other structural components such as channels, culverts, embankments, roadways, and so forth into the distributed model. This process usually involves implementation of the hydraulic parameters where flow transfer can occur between the structure and the hydrology model at various specified cell locations. For example, flow along a channel can be set up in such a manner where channel inflows occur at the upstream end of the channel from neighboring cells, channel overflow occurs from cells adjacent to the channel banks and channel outflow occurs at the cells adjacent to the downstream end of the channel.

5. Sources and sinks are used to insert and remove flow volume at selected cell locations.

6. The hydrologic computations are performed using the principle of mass conservation and use of time-stepping procedures. The results from FLO-2D provide a flow pattern

description both that varies spatially as well as temporally. The post-processing tool such as MAPPER can be used to visualize and process the FLO-2D modeling results.

The FLO-2D software is developed and improved in a continuous fashion. Therefore, the FLO-2D website (http://www.flo-2d.com) should be checked for the latest version and an authorization request be submitted to ADOT for approval prior to use of the model on ADOT projects. The version number and build number are to be used to track the specific version of the model used for the project. It is also recommended that the specific software executable version also be supplied with the project model input and results when delivered to ADOT.

9.1.1 When and Where to Apply 2-D (vs. 1-D)

Two-dimensional hydrologic modeling is to be performed for watersheds where flow patterns are expected to be complex. These include active alluvial fans, distributary flow areas, sheet flooding areas, or split flow channels with uncertain flow conditions. Two-dimensional models are also recommended for complex urbanized conditions where most of the runoff is distributed through street networks with numerous splits, joins, obstructions and diversions.

While it is anticipated that, in general, two-dimensional modeling using FLO-2D will provide better results than one-dimensional models in complex watersheds, the quality of the results is dependent on the quality and implementation of the data input and the inherent capabilities of the two-dimensional model, as well as the presentation of the results. In other words, the improvements in the results from the discretization process heavily depend on the spatial accuracy of all the input parameters. Therefore, it is critical to ensure data accuracy across all the input parameters. In addition, the computational procedures rely on numerical algorithms which can produce erroneous results due to the limitations in the algorithms used. For example, under certain conditions, it is possible that the principle of mass conservation can be violated during the time-stepping process, leading to incorrect estimation of flow distribution. Therefore, it is important that engineering judgment be applied to ensure that the results from the FLO-2D model are reasonable.

The following sections discuss program specific application for FLO-2D. While the detail provided is specific to the FLO-2D software, many of the issues discussed are applicable to any 2D model.

9.2 WATERSHED MODELING

9.2.1 FLO-2D Grid

The FLO-2D computational domain determines the modeling area boundary within which the simulations are performed. The selection of the computational domain can have significant impact in model run-times. Therefore, the study area boundary should be determined such that the following criteria are met:

1. All relevant data should be available for use with reasonable accuracy within the entire study area. In other words, the availability of data should be ensured before finalizing the computational domain so that successful simulation can be performed for the entire

computational domain. Some of the key data needed within the study include topography, rainfall, land use, and soils, as well as structures. Additional guidelines on these data requirements are presented in the following sections.

2. The computational domain should encompass the area of interest with a sufficient buffer area. This buffer is recommended to avoid any modeling related inaccuracies that may occur near the model boundaries. Sufficient buffer area is particularly important at locations where inflow occurs. Grid elements that have specified inflows should be located at adequate distance from the model boundary to ensure that all inflows enter the model study area and do not leave the computational domain due to proximity to the boundary. Initial simulations should be used to refine the computational domain boundary. The flow depth results from these trial simulations along the computational boundary should be reviewed to identify areas with significant flow depth along the boundaries. The boundary should be expanded in these areas unless these represent areas of possible significant flow out of the model study area.

3. The computational domain should avoid non-contributing areas wherever possible to minimize computation time. The computational domain should be fine-tuned based on initial simulations to eliminate such areas.

9.2.1.1 Size and Number of Grid Elements

The FLO-2D computational domain is discretized into uniform-sized square grid elements. Grid size is defined by the side length of each grid. In general, the grid size should be made as small as needed to accurately portray the terrain and desired level of detail of results. In practice, the grid size represents a balance between model run time and the accuracy of the results. This is due to the fact that while a smaller grid size generally produces better resolution, it also increases model run time due to the increased total number of grid cells for which computations must be made. FLO-2D also has some other practical limits on grid size related to the travel time, or flux, across a single grid element. Currently, a lower limit of 15 feet grid size is recommended. For smaller grid sizes, the flow passes across the grid too quickly, resulting in numerical instability problems.

The computational capability of the computer available to the modeler also should be considered in selecting the grid size. Grid size should be chosen to represent a compromise between reasonable accuracy and model run time. In addition, the accuracy of the available data, especially the topography, should also be considered in arriving at the model grid size. For example, improvements in modeling results will not be achieved by reduction in grid size when the accuracy of the topography is poor. In large flood events, topographic variability will not significantly affect the water surface if the entire grid element and its neighbors are completely inundated. When simulating shallow flow, steep slopes and smaller discharges, smaller grid elements should be used.

The selected grid size must adequately simulate the extent of flooding for all the major conveyance features present within the model boundary for the select storm frequency and duration. A comparison of elevations from the model-grids and elevations from other original topographic sources (i.e. ground survey) should be made at key locations to verify that the geometry of the modeled surface adequately reflects the topography.

In summary, the selection of grid size requires a careful examination of the project goals and schedule, the accuracy of available data and capabilities of available computational resources.

9.2.1.2 Grid Element Elevation

FLO-2D grid input requires that the best available topographic data be discretized to arrive at a representative terrain elevation at each grid element location. The Grid Developer's System (GDS) component of the FLO-2D software can compute the representative grid elevation from the topographic data using interpolation and the inverse distance weighting procedure. The grid element size significantly influences the level of resolution obtained from the discretization process. The grid elevations generated by the GDS software should be verified to ensure proper representation of the underlying topography. To ensure this, the following procedures are recommended:

1. A comparison of the elevation at the cell midpoint from the topographic surface and grid element elevation should be made at all grid elements. All values above certain tolerances should be reviewed and checked to ensure accuracy.

2. Contours of same contour interval as the underlying topographic data should be generated from the assigned grid elevation data. A visual comparison of these contours to the contours from the underlying topography should be made. All locations with mismatches should be identified and grid elevation adjustments made as needed.

In addition to the checks listed above, a review of floodplain maximum flow depths should be made to check for anomalies. For example, some locations may show high or low values of flow depth where the opposite might be expected. Such a review can often reveal a problem spot in the grid base elevations. Similarly, significant flows may occur in places where no such flows are anticipated due to minor inaccuracies in the topography or modeling errors. Alternatively, flows may not occur in places where flows are anticipated. Under these three scenarios, manual adjustments to the grid elevations may be required. All such modifications should be clearly documented and approved by ADOT.

9.2.1.3 Grid Element Roughness

FLO-2D handles flow resistance due to roughness in a unique manner. Floodplain roughness is handled through a stepped process and is defined by the following factors:

1. Floodplain Roughness Coefficient: This parameter is defined as the basic description of roughness for flow depths over 3.0 feet. The value is entered in the FPLAIN.DAT file and is specific for each grid element. This coefficient can be altered automatically internally by FLO-2D by applying a Limiting Froude Number.

2. Limiting Froude Number: This parameter is globally assigned as FROUDL in the CONT.DAT file. This parameter determines the automated adjustment to the floodplain roughness coefficient which is used to prevent flow from exceeding a specific Froude Number by individually adjusting the floodplain roughness for each element and each time step. FLO-2D will report on the adjustments in the FPLAIN.RGH and CHAN.RGH output files which should be reviewed and used in determining appropriate roughness coefficients. It is recommended that these changes be reviewed visually in conjunction with other FLO-2D results such as flow depth.

3. Shallow Roughness Coefficient: This parameter is used to specify flow roughness during very shallow flows. The parameter is assigned globally as SHALLOWN in the CONT.DAT file. The minimum value is 0.05, and the model will default to 0.1 if lower values are entered.

4. Depth Varied Roughness: This parameter is a global coefficient with default status of on, but can be turned off (AMANN=-99 in CONT.DAT file). This parameter is used in order to improve the timing of the flood wave progression through the grid system as described in the FLO-2D Data Input Manual (p. 43).

5. It is common modeling practice to adjust roughness values, including the shallow-n value, to "fine tune" a hydrologic/hydraulic model. Setting the FLO-2D parameter values for SHALLOWN and AMANN to 0 and -99, respectively, allows the user to "turn off" the shallow-n computations in the model. When the FLO-2D shallow-n option is not used, floodplain roughness values assigned to each grid element in the FPLAIN.DAT file are utilized for flow computations for all flow depths.

Table 9–1 summarizes the FLO-2D roughness scheme and parameters. The modeler should consider the information in Table 9–1 when selecting and refining n-values for a FLO-2D model.

Table 9–1 Grid Element Roughness Rules

Grid Element Roughness Rules (ft.)	Roughness Defined by	Applied Roughness Value
0.0<d<0.2	Shallow Roughness	n=SHALLOWN
0.2<d<0.5	Shallow Roughness	n=SHALLOWN/2
0.5<d<3	Depth Varied Roughness	n=nb*1.5*e-(0.4*d/3)
3<d	Floodplain Roughness	n= nb (the FPLAIN.DAT value)

(Adapted from FLO-2D Data Input Manual)

9.2.1.4 Grid Element Area Reduction Factor (ARF) and Width Reduction Factors (WRF)

The ARF/WRF mechanisms within FLO-2D are used to block and divert flows between and across individual grid cells. These blockages typically occur due to structures or walls. While the ARF/WRF mechanism is powerful in redirecting flows around structures and so forth, care should be taken to ensure that they function as intended. Flow depth results from trial run simulations should be used to refine the ARF/WRF values to realistically model flow blockage due to structures and walls.

9.2.2 Inflow Hydrographs

Inflows can be input as hydrographs to the model grid at user-specified locations anywhere within the FLO-2D model domain. Since a hydrograph may indicate the input of significant flow at a specific location, it is critical that the inflow occurs over a reasonable surface area to avoid numerical inaccuracies and instabilities. Obtaining a reasonable surface area can be achieved by spreading the inflow over a set of adjacent cells by using the criterion $Q_{peak}/A_{surface}$ approximately equal to 1.0 where Q_{peak} is the peak discharge value in the hydrograph and $A_{surface}$ represents the total grid cell area over which the hydrograph is applied.

In addition, it is also necessary to ensure that the computational domain boundary is not too close to the inflow location. It is possible that some of the inflow may inadvertently leave the model or “pile up” unrealistically if the computational domain is too close to the boundary. After the initial trial runs, it should be verified that the computational boundary is adequate to ensure that this error does not occur.

Finally, grid elements specified in INFLOW.DAT should not be used in specified other special conditions such as in ARF.DAT or HYSTRUC.DAT. That is, inflows should not be applied to grid elements that contain blockages or other special hydraulic features. This misapplication can result in undesirable numerical problems and instabilities.

9.2.3 Rainfall

Point precipitation values entered into FLO-2D are usually obtained by discretization of the values from NOAA Atlas 14. Proper care must be taken that the discretization process is performed without loss of accuracy compared to the original NOAA Atlas 14 data. The need for areal reduction of point rainfall should be evaluated based on the size of the watershed and locations where design flow rates are required. Engineering judgment must be applied in the selection of an appropriate areal reduction factor. In many cases, no areal reduction may be the best solution, as in the case of small watersheds and areas of complex distributary flow.

9.2.4 Rainfall Losses

FLO-2D software has the capability to model rainfall losses using the Green and Ampt procedure, as well as the SCS curve number method. ADOT guidelines require that rainfall losses be computed using the Green and Ampt procedure. The values of Green and Ampt parameters in the FLO-2D model should be based on the ADOT soils and GIS coverages following similar guidance as for development of these parameters in HEC-HMS. The FLO-2D Green and Ampt parameters have different names than are used in HEC-HMS. The XKSAT parameter corresponds to the conductivity value used in HEC-HMS. The PSIF parameter corresponds to the Soil Suction term used in HEC-HMS. The DTHETA parameter is the difference between the initial and saturated soil moisture content in HEC-HMS where DTHETA (dry) is equivalent to Saturated Content minus Wilting Point and DTHETA (normal) is equivalent to Saturated Content minus Field Capacity. The ABSTRINF parameter corresponds to the Max Storage parameter on the "Surface" tab in HEC-HMS. RTIMP corresponds to the Percent Impervious in HEC-HMS. The only difference between FLO-2D and HEC-HMS' implementation of the Green-Ampt loss rate methodology is that for small grid cell sizes in FLO-2D subbasins parameter weighting is unnecessary. Adequate care should be taken to ensure that the soils and land-uses reflect the project study conditions.

Initial and Constant Loss Rate method is not available in FLO-2D. Therefore, modeling of rainfall losses in areas where use of Initial and Constant methods would be appropriate should be carefully evaluated. If the areas are small in spatial extent, use of rainfall losses for adjacent areas may be acceptable.

Two additional FLO-2D parameters that influence the rainfall losses are porosity (POROS) and shallow-n-value (SHALLOWN). The POROS value should be set to zero when the Green and Ampt parameters are assigned using ADOT soils GIS dataset.

It is recommended that the initial loss values be based on land use classifications. It should be noted that the FLO-2D software increases initial losses by including an additional depression storage value (TOL value) assigned in the TOLER.DAT input file. Surface depression storage occurs prior to the beginning of infiltration. Therefore, to eliminate the possible double-counting of the initial losses by the value of TOL, ABSTRINF values should be reduced by the TOL value. Note that the TOL value is specified in feet while the ABSTRINF values are specified in inches.

The output file FPINFILTRATION.OUT contains the total cumulative infiltration by grid element. The spatial distribution of cumulative infiltration values in this file should be reviewed for reasonable spread of the infiltration throughout the watershed.

The output file SUMMARY.OUT should be reviewed to see if the total and percentage loss due to infiltration compare reasonably to expectations. If too much or too little volume is infiltrated, adjustment of SHALLOWN, or XKSAT values may be warranted.

9.2.5 Hydraulic Structures

FLO-2D modeling software includes the capability of flow changes that occur due to the presence of structures within the model study area. An inventory of structures that can impact flow distribution within the study area should be performed and included. These include any structure that can act as a source, sink, diversion, storage, or any other form of attenuation. Historical literature search, field surveys and aerial photography should be used to identify all such structures. These structures should be implemented in the FLO-2D model input or an explanation of their exclusion should be provided.

9.2.5.1 Channels

All channels that convey significant flow should be identified. The decision to model channels using the CHAN.DAT file should be based on the project goals, topography and grid element size. While a reasonable flow simulation within the channel can be obtained by using smaller grid element size, the use of CHAN.DAT to model the channel conveyance will result in higher accuracy in terms of flow attenuation. There is, however, significant drawback in terms of longer model run times and numerical instabilities. It is recommended that several initial model runs be performed to evaluate the need to use CHAN.DAT. It is sometimes possible to arrive at reasonable flow estimates by using a grid element size that is significantly smaller than the channel geometry. This approach can significantly reduce model run times and avoid the numerical instability issues resulting from the use of CHAN.DAT.

When modeling a channel using CHAN.DAT, care should be taken to ensure the specified channel alignments do not result in abrupt changes in the longitudinal slope of the channel. It is recommended the PROFILES component of the FLO-2D software be used to ensure that abrupt changes in the channel profiles are avoided. When the channel top-width is larger than a single grid element, care should be taken to ensure that the channel is coded to the correct elements, such that it extends into the proper adjacent elements. The FLO-2D Grid Developer System (GDS) can facilitate accurate coding of channel elements. The GDS is a GIS integrated software tool included with FLO-2D that is used to facilitate the creation of all required model run data.

Upon completion of the simulation, the channel flow should be reviewed for surging, as indicated in the VELTIMEC.OUT and CHANMAX.OUT files, or by scanning the channel element hydrographs in the HYDROG program. Volume conservation may be impacted, mostly due to data errors, when the channel is implemented using CHAN.DAT. The SUMMARY.OUT and CHVOLUME.OUT files should be reviewed for potential volume conservation issues. In such scenarios, it is recommended all channel inputs be reviewed for accuracy. Other recommendations for troubleshooting include:

1. Consider use of the NOFLOC variable to specify contiguous grid elements that do not share discharge.
2. Eliminate very short channel lengths (XLEN) in CHAN.DAT.
3. Slow the model execution by decreasing WAVEMAX and DEPTOL.
4. Adjust the flow area by smoothing a transition reach by adding interpolated cross-sections.
5. FLO-2D User’s Manual says that volume conservation within 0.001 percent or less will be sufficiently accurate. Under certain circumstances, such high levels of volume conservation may not be possible. Under such conditions, with engineering judgment and justification, a lower level of volume conservation may be acceptable.

The FLO-2D software may internally change the channel roughness values to improve model stability. These values are presented in the CHAN.RGH file. The values in CHAN.RGH must be reviewed to ensure that the values used within FLO-2D are appropriate.

9.2.5.2 Levees

The FLO-2D input file, LEVEE.DAT, allows the simulation of levees and/or walls within a study area. FLO-2D also computes flow diversion along levee alignments. In addition, a levee failure mode can be used to model levee failures. Care should be taken to ensure that the flow exchange between grid elements has been properly restricted to adequately simulate levee function. The input file should be verified to ensure that there are no “leaks” or “breaks” in the levee. Modeling results must be reviewed carefully to ensure anticipated behavior when simulating levee/wall features.

9.2.5.3 Other Hydraulic Structures

Hydraulic structures are implemented within FLO-2D using discharge rating curves which share discharge between two specified channel elements and/or floodplain elements. Such rating curves can be used to represent bridges, culverts, weirs, and spillways. The rating curve is computed externally to FLO-2D using software such as HY-8. The rating curves and other hydraulic structure data are entered in the HYSTRUC.DAT file.

The rating curve at low stages must have adequate resolution to correctly model shallow flows. With inadequate resolution, flow diverted by the hydraulic structure component can be too high, resulting in absorption of all flows that enter the grid element, which leads to surging. Similarly, it must be verified that the rating table extends to an adequately high discharge that covers the entire range of flows. The head reference elevation, HEADREFEL, is the elevation above which the headwater depth is determined for the rating table. If the HEADREFEL is set to zero, the model will use the elevation value set in the FPLAIN.DAT input file.

FLO-2D generates the file HYCROSS.OUT that contains the hydrographs at hydraulic structures. These hydrographs must be reviewed to ensure that the model is not surging. Surging is displayed as abrupt and non-smooth patterns in the hydrographs.

9.2.6 Outflows

Outflows in a FLO-2D model occur when a flow leaves the computational domain through grid elements identified in the OUTFLOW.DAT file. It is recommended that initial modeling results be used to identify possible outflow locations along the computational boundary and the OUTFLOW.DAT file be modified to include all outflow locations. In general, there will be visible evidence of ponding and high flow depths in areas along the boundary where the grid elements are not specified as outflow nodes. These areas can be identified during initial model runs and rectified by including the relevant outflow nodes into the OUTFLOW.DAT file. After rectification, the results should be reviewed once again to ensure the outflow leaves the model in a reasonable manner.

It is also important not to use the outflow grid elements in other input files such as ARF.DAT or HYSTRUC.DAT.

9.2.7 Numerical Controls and Tolerances

The numerical controls and tolerances are specified in the input file called TOLER.DAT. The TOL is the parameter that simulates depression storage. The default value for TOL is 0.1 feet. That is, 1.2 inches of rainfall or inflow will be stored indefinitely on each grid cell prior to generating any runoff from that cell. This value is fairly high and should be reviewed carefully with respect to the topography within the model domain. It is recommended for most applications that a smaller value, in the range of 0.001 to 0.03, be used for TOL. The selection of the TOL value for a specific model should be made relative to the smallest initial abstraction (IA) amount in the model domain. Furthermore, caution should be used in setting TOL in the TOLER.DAT file if initial abstraction is being modeled in the INFIL.DAT file, to avoid double counting. In addition, it must be noted that initial abstraction (IA) is specified in inches and TOL is specified in feet within FLO-2D.

DEPTOL is the tolerance value for percent change in the channel flow depth for a given time step. While performing the computations, FLO-2D determines whether the computed flow-depth exceeds the DEPTOL value. If the value is exceeded, then the time-step reduced and computations are performed again with the reduced time step. DEPTOL values of 0.1 to 0.2 feet are reasonable for use within FLO-2D models. This numerical stability control performed by DEPTOL can be turned off by assigning a value of zero.

The parameter WAVEMAX represents the maximum value of the numerical stability coefficient for full dynamic wave flood routing. The initial value of WAVEMAX can be set as 1.0. Final model runs should use a value of 0.25. Negative values of WAVEMAX can be used to allow FLO-2D to make small adjustments to Manning's n values instead of the time-step. The use of WAVEMAX value greater than 100 turns off this stability control.

During a typical FLO-2D model development process, it may be necessary to experiment with short duration simulations to determine which combination of the stability criteria results in the fastest stable model.

9.2.8 Cross-Section Outputs

The FPXSEC.DAT input file specifies the cross-sections along which the hydrographs are desired. The cross-sections can be oriented in any of 8-directions, and the flow direction along which the output is desired is also specified. In general, it is important to specify flow direction perpendicular to the cross-section, except in unusual situations.

The results related to the cross-sections are output by FLO-2D into the file CROSSMAX.OUT and HYCROSS.OUT. The length and orientation of the cross-section should be verified to ensure that the extent of cross-section captures the desired cross-sectional area of interest. For example, if the cross sections are too short, the cross sections may underestimate the magnitude.

The values of the peak discharges in CROSSMAX.OUT should be reviewed to ensure that they are reasonable. The hydrographs presented in HYCROSS.OUT should also be checked for abnormalities such as surging.

9.2.9 Model Control

The FLO-2D input file CONT.DAT file can be used to specify some of the model run controls. This file should be reviewed to ensure that all appropriate components such as rainfall, infiltration, and so forth are turned on. This file also specifies the model run duration in the variable SIMULT. The value of SIMULT should be verified during initial runs to ensure that the total simulation duration is adequate to allow the entire hydrograph to pass through the entire study area within the watershed.

9.3 FLO-2D MODEL OUTPUT REVIEW

Numerical instabilities can influence the results as well as performance of the FLO-2D model. The SUMMARY.OUT file generated by FLO-2D provides an overall summary of the model simulation. This file should be reviewed to verify a successful completion of the project simulation for the entire duration specified in CONT.DAT. The SUMMARY.OUT also presents the volume budget distribution which can be reviewed to confirm conservation of volume. If a rainfall simulation is performed, the infiltration and rainfall volumes should also be reviewed to ensure that they are within a reasonable range.

The TIME.OUT presents the list of grid elements where the time steps are reduced to satisfy numerical stability criteria. The grid elements with excessive number of time step decrements should be reviewed and adjustments made accordingly. Further guidance on this issue is presented in the FLO-2D Data input manual.

The output files ROUGH.OUT and/or the FPLAIN.RGH files should be reviewed to determine magnitude of Manning’s n-values adjustments made in order to satisfy the Froude number limitation and other model stability criteria.

When reviewing the results in MAXPLOT or MAPPER, the maximum floodplain velocities should be checked for unreasonably high velocities. Excessively high velocities may reflect numerical surging and can be reduced by increasing n-values or adjusting floodplain elevations.

Chapter 10
FLOOD FREQUENCY ANALYSIS

This chapter contains the following details:

- Flood frequency analysis for stream gage data of varying durations and continuity.

10.1 INTRODUCTION

For gaged watersheds where systematic stream gaging records of sufficient length are available, flood frequency analysis can be used to compute flood magnitude frequency relations. The resulting flood magnitude-frequency relation can be used to:

1. Estimate the design flood peak discharge.
2. Provide estimates of flood peak discharges for the calibration or verification of rainfall-runoff models.
3. Provide estimates of flood magnitudes that can be used to check other methods to estimate flood magnitudes or to develop regional flood discharge relations from multiple stations.
4. Perform other hydrologic studies, such as the investigation of flood magnitudes from snowmelt to be used as base flow to a watershed rainfall-runoff model.

While the US Geological Survey has previously completed statistical summaries (USGS, cf, Pope, Rigas, and Smith, 1998) for many gaging stations in Arizona, those analyses are not recommended for drainage design for ADOT. Rather, new analyses should be performed using the procedures outlined in this Chapter.

10.2 PROCEDURE

1. The procedure requires the compilation of recorded, estimated, and historic annual peak discharge data that are generally collected by federal agencies, but on occasion are available through or augmented by state, county, or local agencies. Therefore, an important component of such an analysis involves the careful and complete documentation of all available flood data. In addition, historic flood information must be sought out and compiled.

2. The procedure is a graphical analysis that requires considerable interpretation and judgment. Many of the data collection and analytic procedures can be conducted by less experienced personnel. However, it is advisable that such an individual work under the direct supervision of an experienced practitioner.

3. The procedures, outlined in this section, are taken from research reports, hydrologic studies, and other professional publications. The key sources of this procedure are provided with some additional explanation in the separate Documentation Manual. Users of this procedure should familiarize themselves with the background and theory by studying Reich, 1976 and Reich and Renard, 1981 and other pertinent literature.

10.2.1 Applications and Limitations

1. A minimum of 10-years of continuous, systematic data are required to perform the recommended procedure.

2. Since the accuracy of flood-frequency relationships is directly related to the record length used to derive the relationship, the user should be aware that the reliability of peak discharge estimates will decrease when the flood return interval associated with such a discharge exceeds twice the record length.

3. Flood discharge records must be carefully inspected and evaluated prior to their adoption for analysis. For example, the construction of a dam upstream of a gaging station prior to or during the period of record, or the progressive urbanization of the upstream watershed will require special treatment of the data, discussed in the Preliminary Data Analysis Section of this chapter, prior to its analysis or rejection of the data for analysis.

4. A flood frequency analysis provides flood magnitude-frequency relations that are representative of conditions in the watershed for the period of recorded or historic data. These conditions may or may not be representative of conditions that are desired for design purposes. If the past conditions of the watershed are not representative of desired design conditions, then rainfall-runoff modeling of the watershed will be required; however, knowledge of the past flood frequency relation would be valuable in the development and calibration of the rainfall runoff model.

5. Flood data are by nature extremely variable. Even relatively long records of data may not represent the true occurrence of floods that may be anticipated. In addition, such data may not reflect long-term trends or cycles in the hydrologic processes. Flood records either may not reflect adequately large floods (leading to under design) or may contain one or more exceptionally large and truly rare floods (leading to overdesign). No matter how good the data, the interpretation of the flood frequency relation must be made with the full understanding of the uncertainty of the data and the associated risk involved. For this reason, a procedure to place confidence limits regarding the flood frequency relation is provided.

6. Many other theoretical and practical limitations and applications to this procedure apply which are expected to be understood and appreciated by the users of this procedure and the users of the results. Appropriate design considerations must be made in regard to the accepted risk and the consequences of failure and/or overdesign.

10.2.2 Data

Two types of peak discharge data are to be collected: 1) systematic records, and 2) historic data.

10.2.2.1 Systematic Records

Systematic records are stream discharge data that are systematically observed and recorded at stream gaging stations that have continuous recorders or crest-stage gages. Often, these stations have flood peaks that were estimated for large floods during periods when the gage was not operated, and such flood estimates are generally considered as part of the systematic record. The major source of systematic data for Arizona is the records of the U.S. Geological Survey (USGS). The published records of the USGS can be used to obtain much of these data, although the USGS should be consulted to obtain more recent, unpublished data and to confer with USGS personnel as to the quality of the data and regarding possible other sources of data or related studies. Additional stream discharge data may be available from state or local flood control and water supply agencies. Systematic records can be continuous, broken, or incomplete.

10.2.2.2 Continuous Records

Continuous records are those for which annual flood peak discharges are available from the data collection agency for each water year for the entire period of record.

10.2.2.3 Broken Records

Broken records are those for which annual flood peak discharges are available for two or more distinct periods that are separated by periods for which data were not obtained because of conditions not related to flooding, such as temporarily discontinued gaging stations. For broken records, the length of the systematic record is the sum of the individual periods of data collection. Broken records need to be carefully investigated to assure that physical changes in the watershed that would affect flood magnitudes did not occur.

10.2.2.4 Incomplete Records

Incomplete records refer to records in which one or more annual flood peak discharges are missing because they were either too high or too low to record, or the gage was temporarily out of operation because of flooding or other natural causes. Missing high and low flow data require different treatment. When high flood discharges are not recorded, there is usually information available from which the peak discharge can be estimated. The collecting agency will usually provide such estimates, which are usually so noted in their records. These high flood estimates should be noted in the data compilation forms. This information can be used in considering the accuracy of the plotted data point. Missing low flows can be treated as zero flows (see the Special Cases in Data Treatment, Zero Flow Years).

10.2.2.5 Historic Data

At many locations, particularly near urban areas, there is information about major floods which occurred either before or after the period of systematic data collection. This information can often be used to make estimates of peak discharge. Also, such data often define an extended period during which the largest floods, either recorded or historic, are known. The USGS includes some historic flood information in its published reports and computer files. Additional information can sometimes be obtained from the files of other agencies, extracted from newspaper files or gathered with intensive inquiry and investigation near the site for which the flood frequency information is needed. Historic flood information should be obtained and documented whenever possible, particularly when the systematic record is relatively short. Use of historic data assures that estimates are consistent with local experience and improves the frequency determinations.

10.2.3 Extraordinary Floods

Extraordinary floods are floods with magnitudes that are considerably higher than the vast majority of floods in the record. Extraordinary floods can be either systematic or historic. Many historic floods, by virtue of the fact that they were noted during a period when systematic data were not collected, are also extraordinary floods. Three situations are used to classify floods as extraordinary: (1) when the flood magnitude is determined to be a high outlier, as described later, (2) when certain floods from the systematic record are larger than any historic flood, and (3) when peak discharges from the systematic record are known to be larger than other, non-recorded, annual peak discharges for a period extending to some year prior to the start of the systematic record or for a period after a systematic record was discontinued.

10.2.4 Illustrative Flood Series and Definitions

Figure 10-1 illustrates a series of systematic and historic flood data and includes the definitions and variables used in this section. In this example, a flood study is to be performed for which flooding information is available through 1990. A broken, systematic record exists for 1940 through 1945, and 1950 through 1980. An historic flood occurred in 1915 which is known to be the largest since 1890. Another historic flood occurred in 1986 after the gage was discontinued. The 1974 flood is extraordinary because it is larger than the 1986 flood. The high outlier limit was calculated, and the 1960 flood exceeds that magnitude; therefore, it also is extraordinary. A zero flow year occurred in 1971. The low outlier limit was calculated, and the 1951 flood is less than that magnitude; therefore, it is treated as a zero flow year. The following are the values to be used in this flood frequency analysis:

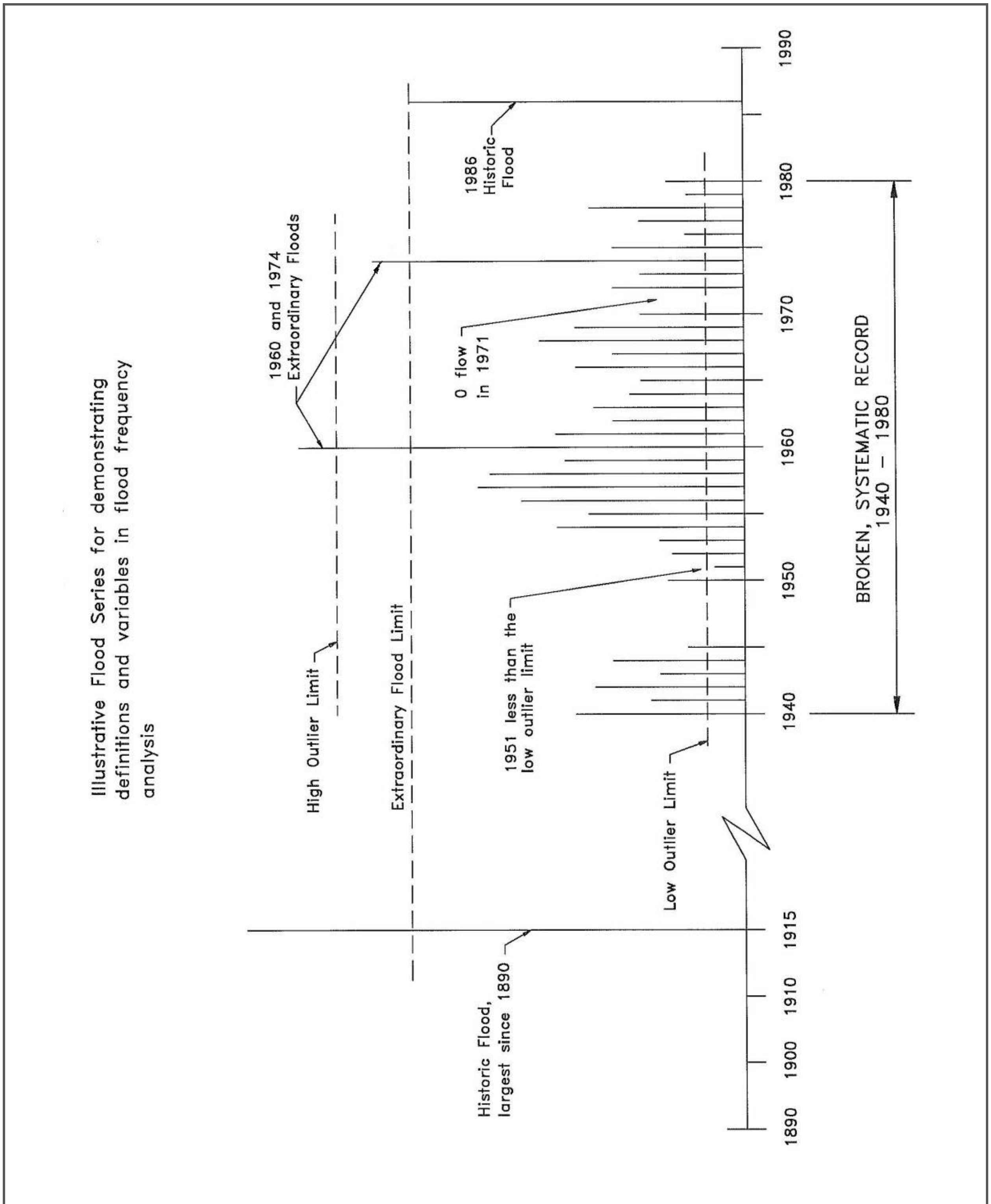


Figure 10-1 Illustrative Flood Series for Demonstrating Definitions and Variables in Flood Frequency Analysis

Effective record length (N) (See Section 10.2.7.2 for definition.)

N = 1890 through 1990 = 101-years
Note: The effective record length is extended to 1990 because of the presence of historic data and extraordinary floods in the record which are known to not have been exceeded during 1981 through 1985 and 1987 through 1990.

Length of systematic record (N_t)

N_t = 1940 through 1945 and 1950 through 1980 = 37 years

Zero flow years (Z)

Zero flow (1971) = 1 year Flow less than low outlier (1951) = 1 year
Z = 1 + 1

Effective length of systematic record (N_s)

$N_s = N_t - Z$
N_s = 37 - 2 = 35 years

Number of historic floods (not in systematic record) (h)

1915 and 1986
h = 2 years

Number of extraordinary floods (in systematic record) (e)

1960 and 1974
e = 2 years

Total number of historic plus extraordinary floods (k)

$k = h + e$
k = 2 + 2 = 4 years

Number of systematic plus historic data (N_g)

$N_g = N_s + h$
N_g = 35 + 2 = 37 years

The use of these variables is defined in the following paragraphs.

10.2.5 Data Compilation

The data that are collected are to be compiled in a table with the following headings: water year; the annual peak discharge (cfs); date of peak discharge; source of data; whether flood was caused by rainfall (R); snowmelt (S), rainfall on snowmelt (R/S), or uncertain (U); and any necessary comment concerning the quality of the data or nature of the flood. A data compilation form is shown in Figure 10-2.

10.2.6 Preliminary Data Analysis

A time series graph of flood peak discharge as a function of water year should be prepared to investigate the stationarity of the flood record. Nonstationarity is indicated either by trends in the magnitudes of the floods, or by sudden discontinuities in flood magnitudes, or by a change in the scatter of the flood magnitudes. Either a bar graph or a line connecting the points, or both types of graphs can be used. A bar graph is more effective than a line graph when showing historic floods or broken records where large time gaps may exist. Line graphs often are better at demonstrating trends or cycles in time series of flood peaks. Only data that exhibit stationarity are to be used in the flood analysis. Therefore, investigate the graph(s) and the history of the watershed and gaging station to determine if there are reasons to question the stationarity of the flood record. Other, more complex statistical methods can be used to test for stationarity if the time series graph(s) and other investigations indicate that nonstationarity may exist (Kite, 1988; Buchberger, 1981; and Reich and de Roulhac, 1985); however, such tests and others are beyond the scope of this manual, and they are not contained in the manual. Nonstationarity can be caused by the construction of upstream dams or other man-made activities affecting flood magnitude, progressive urban development in the watershed, diversions into or out of the river, or long-term and cyclic atmospheric processes. The discharge records often provide information to judge whether man-made activities are responsible for changes in the flood records.

ARIZONA DEPARTMENT OF TRANSPORTATION
HYDROLOGIC DESIGN DATA

Project No. ______________________ TRACS No. ______________________
Project Name ______________________ Date ______________________
Location/Station ______________________
Designer ______________________ Checker ______________________

FLOOD FREQUENCY ANALYSIS
DATA COMPILATION FORM

Page _1_ of ____

Gage Station Name ______________________
Gage Station No. ______________________ **Drainage Area** __________**sq. mi.**
Period of Systematic Record ______________________

WATER YEAR (1)	ANNUAL PEAK DISCHARGE (cfs) (2)	DATE (3)	FLOOD TYPE[a] (4)	COMMENTS (5)

a - rainfall (R), snowmelt (S), rain on snow (R/S), uncertain (U), other (X) - note in comments

Figure 10-2 Flood Frequency Analysis Data Compilation Form

ARIZONA DEPARTMENT OF TRANSPORTATION
HYDROLOGIC DESIGN DATA

Project No. ______________________ TRACS No. ______________________
Project Name ______________________ Date ______________________
Location/Station ______________________
Designer ______________________ Checker ______________________

FLOOD FREQUENCY ANALYSIS
DATA COMPILATION FORM

Page ____ of ____

WATER YEAR (1)	ANNUAL PEAK DISCHARGE (cfs) (2)	DATE (3)	FLOOD [a] TYPE (4)	COMMENTS (5)

a - rainfall (R), snowmelt (S), rain on snow (R/S), uncertain (U), other (X) - note in comments

Figure 10-2 Flood Frequency Analysis Data Compilation Form (continued)

The second preliminary analysis, important for rivers that drain mountainous watersheds in Arizona, requires determining the cause of the flood discharge. Floods in Arizona are normally caused by rainfall, snowmelt, or rainfall on snowmelt. It is necessary to distinguish the cause of the floods to avoid mixed populations in the flood frequency analysis. Often the cause of the flood peak discharge can be determined by simply considering the date of the flood. During the spring and fall it may not be possible to make this simple determination. Often this judgment can be made by inspecting the daily discharge records for the days immediately prior to and after the flood date. In other cases, inspecting the flood stage hydrograph record, consulting meteorological data (rainfall and temperature), referring to flood reports, talking to local authorities, or using other means may be necessary. The data compilation (**Figure 10-2**) should document the cause of the flood.

10.2.7 Plotting Position

Two plotting position equations are recommended. The first is to be used for systematic data of continuous, broken, and incomplete records. The second is to be used for records containing historic and/or extraordinary data. Both plotting position equations are demonstrated with examples. **Equation 10.1** relates the exceedance probability (P_e), to the flood return period (T_r), in years, is:

$$T_r = \frac{1}{P_e} \qquad 10.1$$

10.2.7.1 Systematic Data Equation

For systematic data, the plotting position equation is (Cunnane, 1978):

$$P_e = \frac{m - 0.4}{N_s + 0.2} \qquad 10.2$$

where: P_e = the exceedance probability of a flood event,
m = the rank of each flood in descending magnitude order, and
N_s = the effective length of systematic record.
Note: If zero flow years (or low outliers) exist, then **Equation 10.8** must be used along with **Equation 10.2**.

10.2.7.2 Historic or Extraordinary Floods plus Systematic Data Equation

For flood records containing one or more historic data and/or extraordinary floods, the plotting position equation is (Guo, 1990):

$$P_e = \left(\frac{m - 0.4}{N_s + 0.2}\right)\left(\frac{k}{N}\right)$$

For m = 1, ….., k

$$P_e = \left(\frac{k}{N}\right) + \left(\frac{N-k}{N}\right)\left(\frac{m-k-0.4}{N-k-0.2}\right)\left(\frac{N-k}{N_s+e}\right) \qquad 10.3$$

For m = k + 1, …, N_g

where:

P_e	=	the probability of flood exceedance,
m	=	the rank of each flood event (from 1 to N_g) in descending magnitude order,
N	=	the effective record length. (This is usually the number of years for the period from the first historic flood to the last year of the systematic record, or the number of years between the year that an extraordinary flood has not been exceeded prior to the start of systematic data collection) to the end of the systematic data or the present year of analysis, if appropriate. Some judgment will be necessary in certain cases in selecting the effective record length for records containing extraordinary floods (see Example 10.3, Hassayampa River near Wickenburg, Arizona),
N_s	=	the number of years in the systematic record, less zero flow years and low outlier years, Note: If zero flow years (or low outliers) exist, then Equation 10.8 must be used along with Equation 10.3.
h	=	the number of historic data,
e	=	the number of extraordinary floods in the systematic record,
k	=	the number of historic plus extraordinary floods, and
N_g	=	the number of systematic plus historic data, $N_g = N_s + h$.

10.2.8 Use of Plotting Position Equation

The compiled flood data (Figure 10-2) are ranked from largest to smallest using the form in Figure 10-3. The plotting position is calculated using either Equation 10.2 or 10.3, as appropriate. There may be other data investigations or special treatments to the data that need to be considered or undertaken prior to the calculation of the plotting position. These special cases involve mixed populations of floods from rainfall and snowmelt, records containing zero flow (or low flow) years, and records that may contain high or low flow outliers. Discussion of these special cases is contained in Section 10.2.11.

10.2.9 Graph Papers

The graphical analysis is to be performed by plotting the annual peak discharges corresponding to a specified plotting position on the following probability papers: log normal (LN), extreme value (EV), and log extreme value (LEV). These probability papers were devised to graphically portray data that are from a specific probability distribution. The following graph paper forms are provided for this purpose:

	Figure
log-normal, 2 cycle	Figure 10-4
log-normal, 3 ½ cycle	Figure 10-5
extreme value	Figure 10-6
Log-extreme value, 2 cycle	Figure 10-7
Log-extreme value, 3 ½ cycle	Figure 10-8

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Project No. ______________________ TRACS No. ______________________
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FLOOD FREQUENCY ANALYSIS
PLOTTING POSITION CALCULATION FORM Page _1_ of ____

Gage Station Name ______________________
Gage Station No. ______________________ **Drainage Area** __________**sq. mi.**
Period of Systematic Record ______________________

Check if the data contains any of the following:

Broken Record _____ Mixed Population _____ High Outliers _____
Historic or Extraordinary Data _____ Zero Flow Year _____ Low Outliers _____

Document the plotting position equation or data treatment on a separate sheet.

FLOOD PEAK DISCHARGE (cfs) (1)	RANK (2)	PLOTTING POSITION	
		P_e (3)	T_r (4)

Figure 10-3 Flood Frequency Analysis Plotting Position Calculation Form

ARIZONA DEPARTMENT OF TRANSPORTATION
HYDROLOGIC DESIGN DATA

Project No. ______________________ TRACS No. ______________________
Project Name ______________________ Date ______________________
Location/Station ______________________
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FLOOD FREQUENCY ANALYSIS
PLOTTING POSITION CALCULATION FORM

Page ____ of ____

FLOOD PEAK DISCHARGE (cfs) (1)	RANK (2)	PLOTTING POSITION	
		P_e (3)	T_r (4)

Figure 10-3 Flood Frequency Analysis Plotting Position Calculation Form (continued)

LOG-NORMAL 2 CYCLE GRAPH PAPER

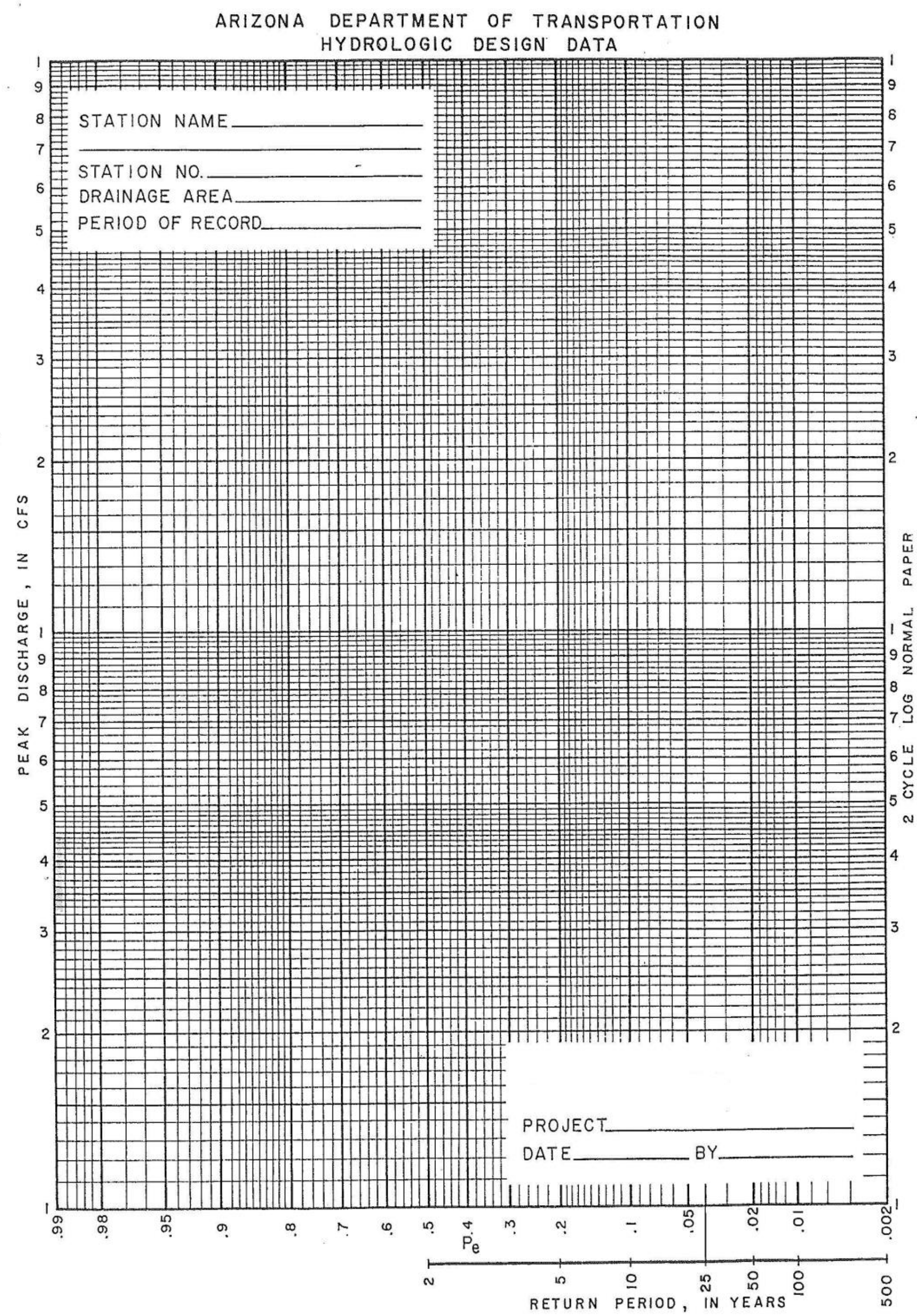

Figure 10-4 Log-Normal 2 Cycle Graph Paper

LOG-NORMAL 3½ CYCLE GRAPH PAPER

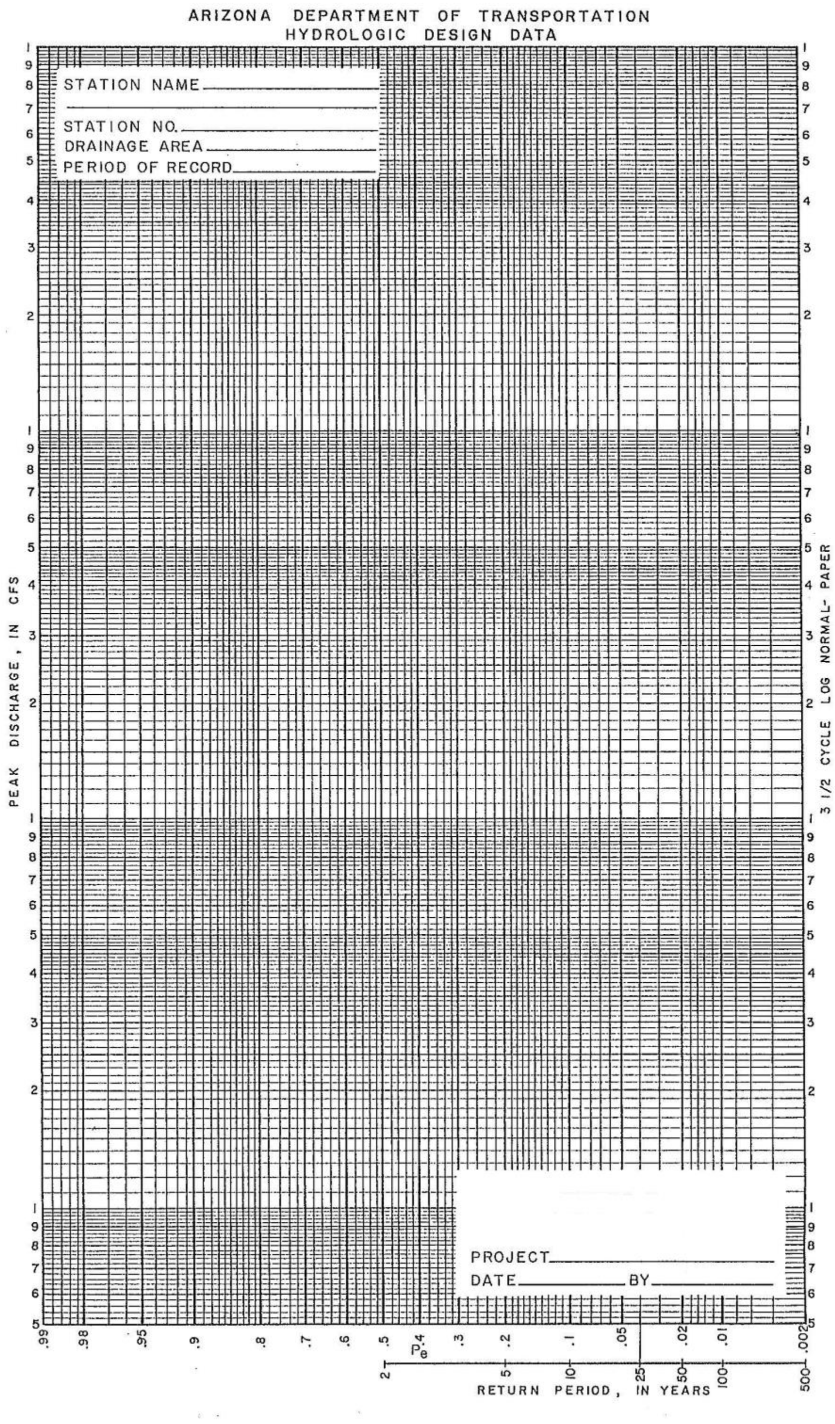

Figure 10-5 Log-Normal 3 ½ Cycle Graph Paper

EXTREME VALUE GRAPH PAPER

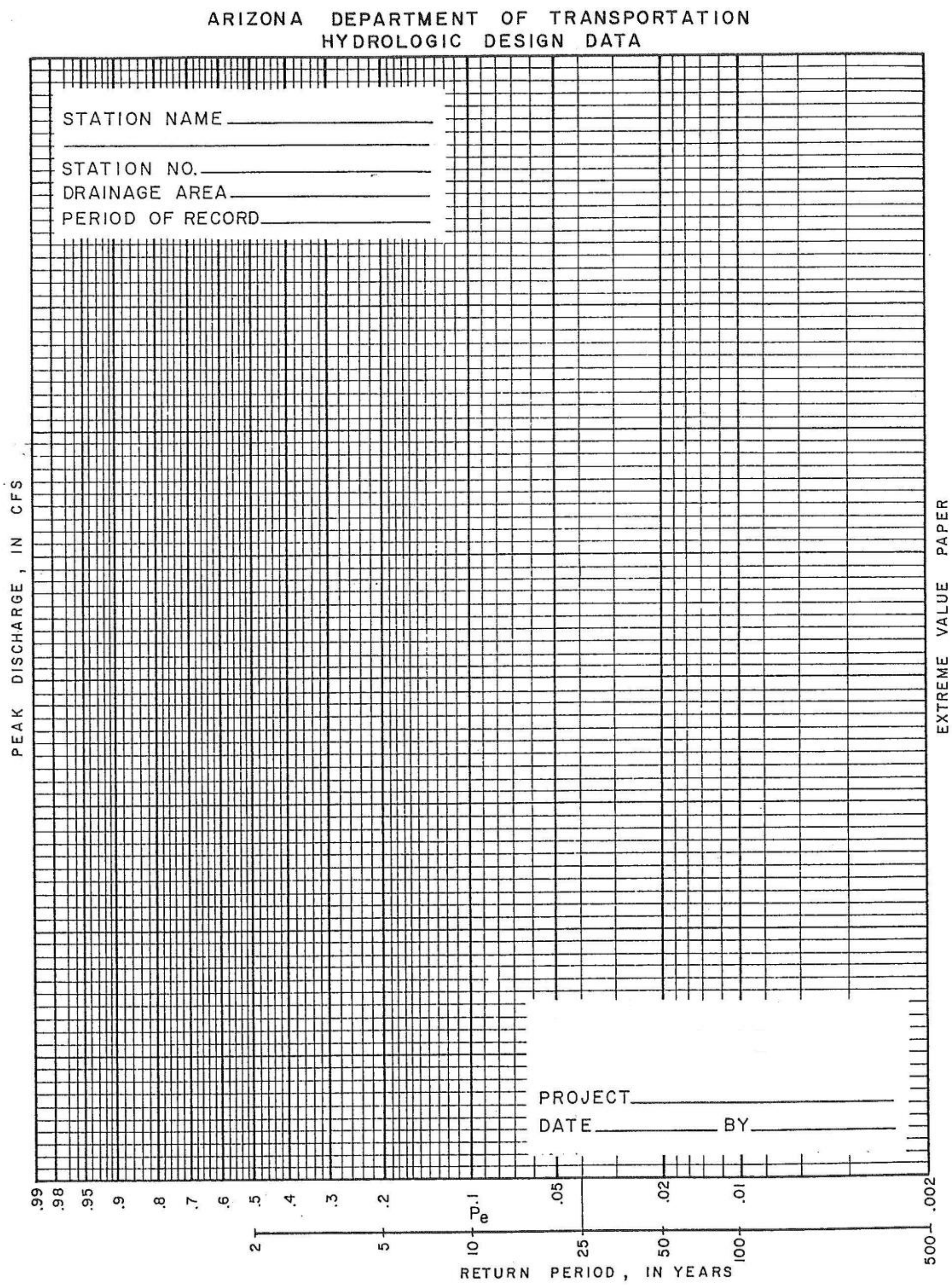

Figure 10-6 Extreme Value Graph Paper

LOG-EXTREME VALUE 2 CYCLE GRAPH PAPER

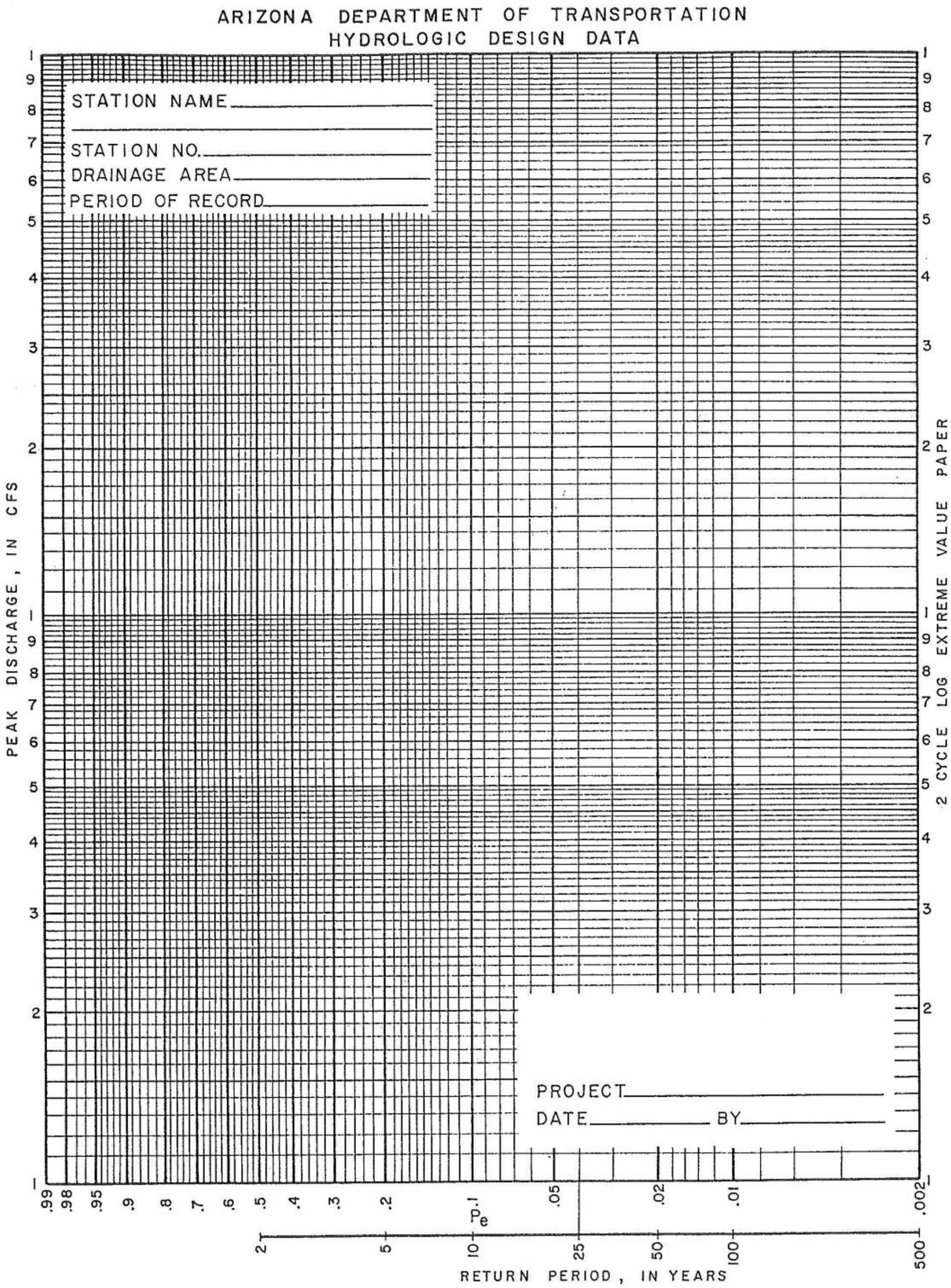

Figure 10-7 Log-Extreme Value 2 Cycle Graph Paper

LOG-EXTREME VALUE 3½ CYCLE PAPER

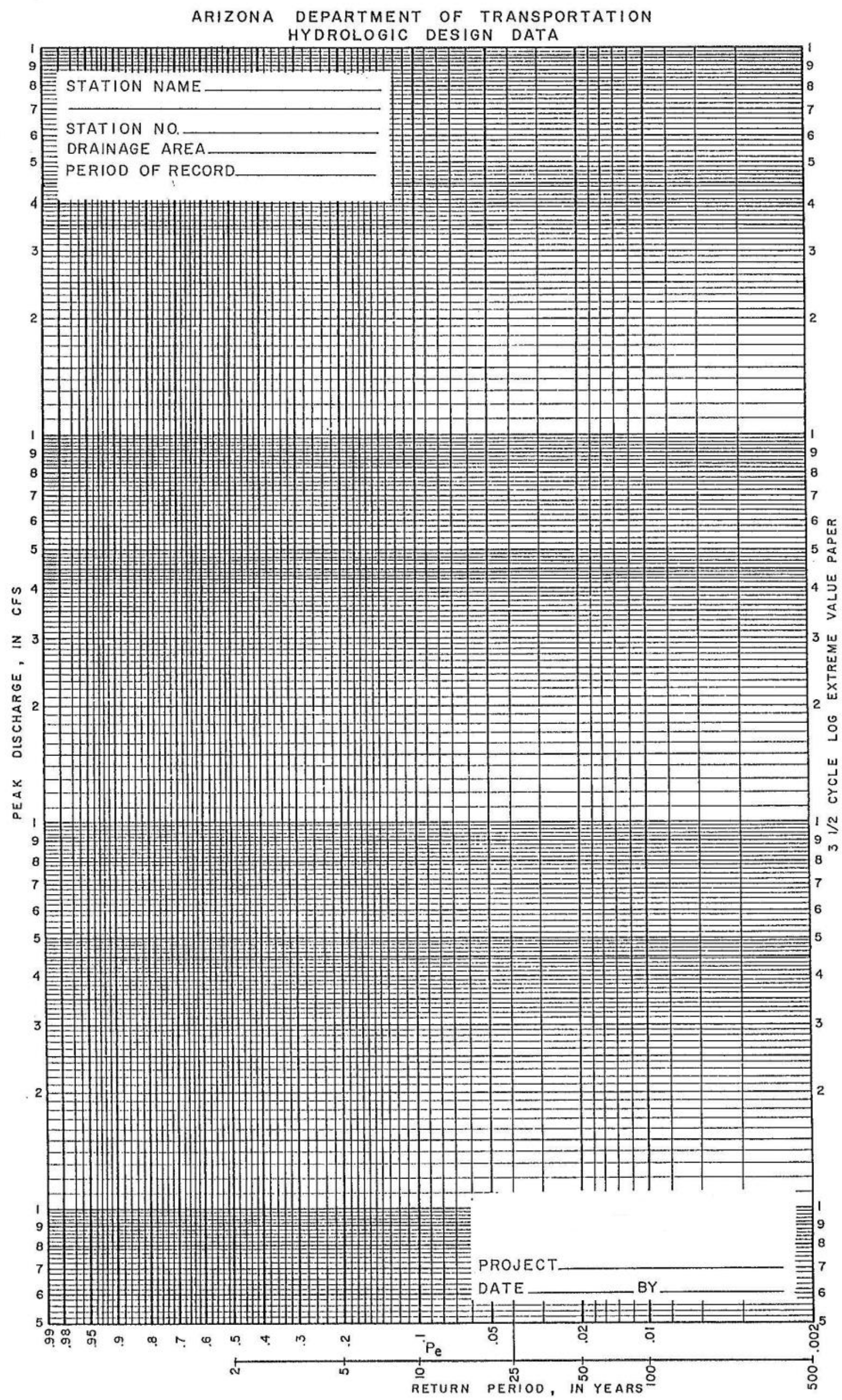

Figure 10-8 Log-Extreme Value 3 ½ Cycle Graph Paper

10.2.10 Plotting Data on Graph Paper

The flood frequency data (Figure 10-3) are plotted on all three types of graph paper- LN, EV, and LEV (Figure 10-4 through Figure 10-8). The intent of this multiple plotting process is to identify the graph paper for which the data plots most nearly as a straight line. Fitting a straight line to the data is necessary so that the line can be extended beyond the range of plotted data points. If the data points appear to be curved instead of a straight line, it is an indication that the data do not follow the probability distribution for which the graph paper was prepared. In this case a curved line must not be fitted through the data points since the extension of curved lines by graphical methods is subjective, leading to increased uncertainty in the flood estimates, and lack of reproducibility among various users.

Several general cases can be observed in the plotting of the data on the graph paper: Case 1- the data can plot very nearly as a straight line on one of the graph papers and not as a straight line on the other two, Case 2- the data can plot nearly linearly, and equally as well, on two or three of the graph papers, and Case 3- the data do not plot as a straight line (even for the high discharge range) on any of the graph papers. This graphical analysis occasionally results in Case 1 above for which the analysis and interpretation is greatly facilitated. However, often the analysis results in either Case 2 or 3 for which the analysis and interpretation is complicated, or, in some rare cases, beyond interpretation by these techniques.

The following are offered as guidelines and suggestions in performing graphical flood frequency analyses and in refining the art of performing such analyses:

1. Read and study the literature that is available on this topic. Of particular value are the papers by Reich (1976) and Reich and Renard (1981). These papers are included in the Documentation Manual and are available through ADOT.

2. Figure 10-9 (King, 1971) provides guidance in the shape of data of unknown probability distribution when plotted on the three recommended graph papers. Notice that when the unknown distribution of the data is the same as the distribution of the graph paper, the data plots as a straight line (the desired situation). Use of Figure 10-9 can help identify the most appropriate graph paper by comparing the general shape of the plotted points to the shape of the lines in Figure 10-9.

3. Some deviation of individual points from the straight line is acceptable. Large flood magnitudes (maybe the largest and second largest events) will often deviate from a linear relation on any graph paper. This is often, though not a general rule, the results of estimation error of such large flood magnitudes that exceed the limits of the gaging station rating curve.

4. Although three probability distribution graph papers are recommended, use of other graph paper for other probability distributions is not precluded. If linearity is not achieved with one of the three recommended graph papers, then consideration might be given to others described by King (1971). A more comprehensive set of comparative graphs (as shown in

Figure 10-9) is presented by King to aid in the selection of alternative graph papers. Alternatively, if linearity is not achieved by the described procedure, then analytic flood frequency procedures can be considered.

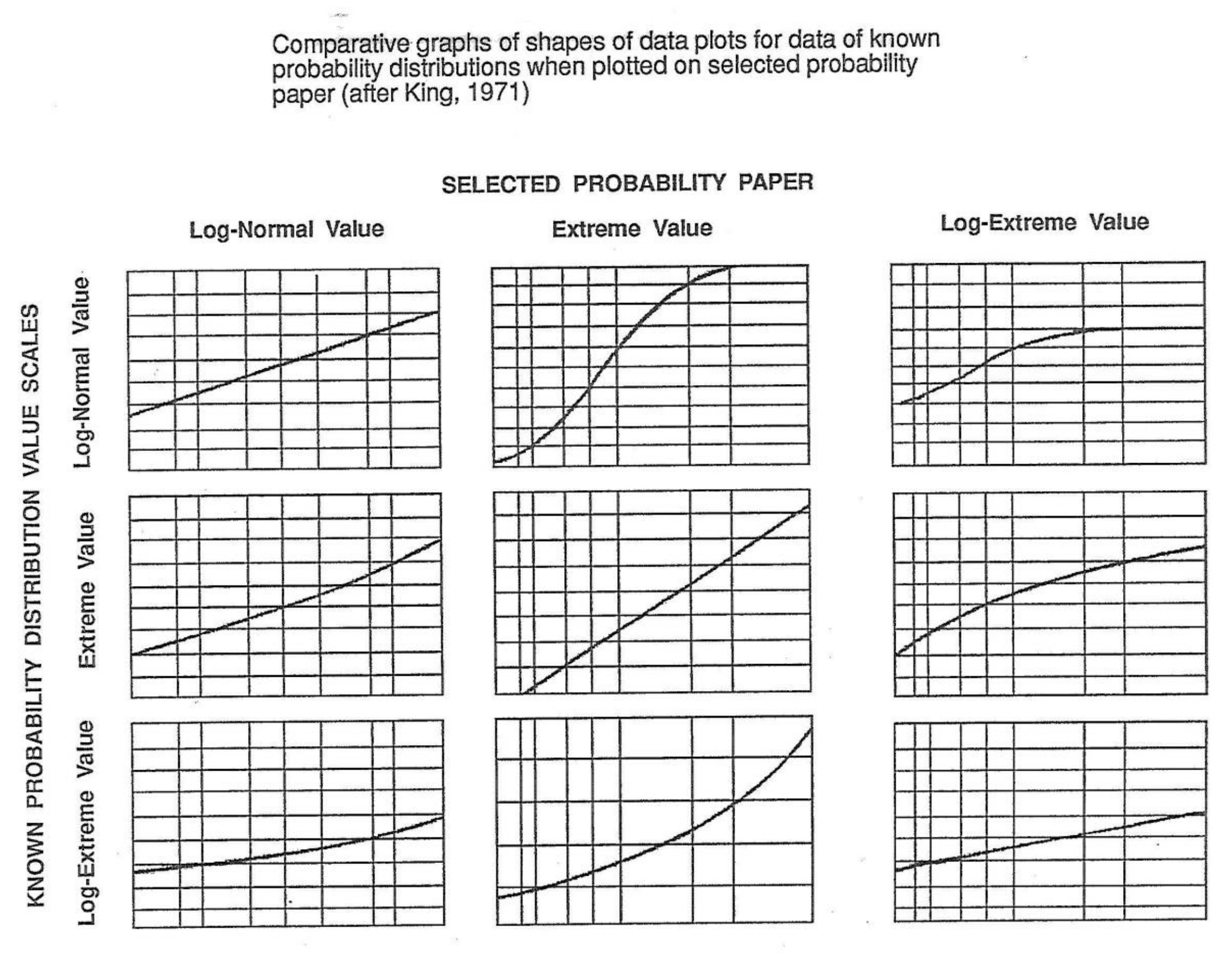

Figure 10-9 Comparative Graphs

5. In some situations the data may plot as two straight lines (one for the smaller flood discharges and another for the larger discharges). This result may be indicative of a mixed population of rainfall and snowmelt floods, or different regimen of rainfall events, one for local storms covering only partial areas of the watershed and another for general storms or larger areal extent local storms. If further investigations indicate a mixed population, then treat accordingly (see Special Cases). Otherwise, fit the straight line to the larger flood events.

6. Use hydrologic judgment, based on regional experience with flooding and specialized training, to fit straight lines to the data with emphasis given to the larger half (P_e than 0.5), or so (P_e less than 0.1 in extreme cases), of the observed floods.

7. Small flood events (P_e greater than 0.5), if they deviate from an otherwise linear relation on the graph paper, need not be considered when attempting to estimate the large floods.

8. Deviations can be expected in even the best data sets; such deviations will occur about the "best fit" line. Some data points will be above the line and some below the line. These results are acceptable as long as the data points appear to be linearly arrayed rather than curvilinearly arrayed. If use of more than one graph paper indicates linearity, select the graph with the least scatter about the line.

9. When it is difficult to select the best choice of graph paper; that is, having similar linearity (or lack of) and similar data scatter about the line, it may be possible to review or perform a flood frequency analysis for a regional and hydrologically similar watershed with better quality data. Such an analysis may indicate a clear choice of governing probability distribution and a valid reason to accept the comparable graph paper for the watershed being studied.

10.2.11 Special Cases in Data Treatment

Three relatively common hydrologic factors may need to be considered, and the data treated accordingly, before proceeding with the graphical flood frequency analyses. These factors need to be considered after the data are compiled and after the preliminary data analyses are performed. These hydrologic factors and the appropriate data treatments involve (1) mixed populations, (2) high and low flow outliers, and (3) zero flow years.

10.2.11.1 Mixed Populations

Mixed populations result when floods are the result of two or more distinct and independent hydrologic events, such as floods from rainfall runoff and floods from snowmelt.

If mixed populations are indicated, then the data treatment and graphical analysis should proceed as follows:

1. Separate the data according to cause of flood (typically either rainfall or snowmelt).

2. Perform separate flood frequency analyses, as previously described. The graphical analyses may result in the use of different graph papers for each flooding type.

   Note: The length of record of systematic data will be different in each case. For example, if 30 years of systematic data are available with 10-years of rainfall floods and 20-years of snowmelt floods, then for the rainfall floods N_s= 10 and m = 1,…., 10 in **Equation 10.2**, and for snowmelt floods N_s = 20 and m = 1,…., 20.

3. Construct a composite flood frequency relation by using conditional probability (Haan, 1977). Mathematically this is (using a mixed population of rainfall (R) and snowmelt (S) floods):

$$P_e = \mathrm{P}(\mathrm{Q} > \mathrm{Q}_0) = [P(Q > Q_0|R)][P(R)] + [P(Q > Q_0|S)][P(S)] \qquad 10.4$$

   Equation 10.4 states that the probability of a flood (Q) being larger than a selected magnitude (Q_0) (the probability of exceedance) is equal to the probability of that flood exceedance given that the flood was caused by rainfall ($P(Q > Q_0|R)$) (from the rainfall flood frequency graph) times the probability of a rainfall flood ($P(R)$ = number of rainfall floods divided by the total number of floods), plus the probability of that flood exceedance given that the flood was caused by snowmelt ($P(Q > Q_0|S)$) (from the snowmelt flood frequency graph) times the probability of a snowmelt flood ($P(S)$ = number of snowmelt floods divided by the total number of floods). Use of Equation 10.4 will result in a flood sequence of magnitudes (Q_0) and associated probabilities of exceedance (P_e).

4. The graphical flood frequency procedure is then repeated using the new sequence of flood magnitudes (Q_0) and plotting positions (P_e) from Step 3, above. That is, graphical analysis is used to identify the graph paper (probability distribution) for which this new flood sequence plots as a straight line. The graph paper will usually, but not always, be the same as the kind used for either rainfall or snowmelt that had the larger floods.

10.2.11.2 Outliers

Outliers are data points which depart significantly from the trend of the remaining data. The retention, modification, or deletion of these outliers can significantly affect the graphical analysis, especially for small samples. All procedures for treating outliers ultimately require judgment involving both mathematical and hydrologic considerations. The detection and treatment of high and low outliers are described below.

The following equation is used to detect high outliers (U.S. Water Resources Council, 1981):

$$log\ Q_H = \overline{log\ Q} + K_N S \qquad 10.5$$

where:

$log\ Q_H$	=	high outlier threshold in log units,
$\overline{log\ Q}$	=	mean of the logarithms of systematic peaks (log Q's) excluding zero flood events,
K_N	=	value from **Table 10–1** for sample size N_s,
N_s	=	the number of years in the systematic record, less zero flow years and low outlier years, and
S	=	standard deviation of log Q's calculated by

$$S = \left[\frac{\sum(logQ_i)^2 - (\sum logQ_i)^2/N_s}{N_s - 1}\right]^{0.5}$$

where Q_i are the annual peak discharges, and N_s is the effective length of systematic record.

If the logarithms of peak discharges in a sample are greater than log Q_H in **Equation 10.5** then they are considered high outliers. Flood peaks considered high outliers should be compared with historic data, flood information at nearby sites, and thoroughly investigated. High outliers can be deleted from the record if the data can be irrefutably determined to be in error, otherwise treat high outliers as extraordinary data. Deletion of high outliers would result in the record being treated as a broken record. The treatment of all extraordinary flood data, and high outliers should be well documented in the analysis.

Table 10–1 contains one sided 10 percent significance level K_N values for a normal distribution (U.S. Water Resources Council, 1981).

Table 10–1 Outlier Test K_N Values

Sample Size N_S	K_N	Sample size N_S	K_N	Sample size N_S	K_N	Sample size N_S	K_N
10	2.036	45	2.727	80	2.940	115	3.064
11	2.088	46	2.736	81	2.945	116	3.067
12	2.134	47	2.744	82	2.949	117	3.070
13	2.175	48	2.753	83	2.953	118	3.073
14	2.213	49	2.760	64	2.957	119	3.075
15	2.247	50	2.768	85	2.961	120	3.078
16	2.279	51	2.775	86	2.966	121	3.081
17	2.309	52	2.783	87	2.970	122	3.083
18	2.335	53	2.790	88	2.973	123	3.086
19	2.361	54	2.798	89	2.977	124	3.089
20	2.385	55	2.804	90	2.981	125	3.092
21	2.408	56	2.811	91	2.984	126	3.095
22	2.429	57	2.818	92	2.989	127	3.097
23	2.448	58	2.824	93	2.993	128	3.100
24	2.467	59	2.831	94	2.996	129	3.102
25	2.486	60	2.837	95	3.000	130	3.104
26	2.502	61	2.842	96	3.003	131	3.107
27	2.519	62	2.849	97	3.006	132	3.109
28	2.534	83	2.854	98	3.011	133	3.112
29	2.549	64	2.860	99	3.014	134	3.114
30	2.563	65	2.866	100	3.017	135	3.116
31	2.577	66	2.871	101	3.021	136	3.119
32	2.591	67	2.877	102	3.024	137	3.122
33	2.604	68	2.883	103	3.027	138	3.124
34	2.616	69	2.888	104	3.030	139	3.126
35	2.628	70	2.893	105	3.033	140	3.129
36	2.639	71	2.897	106	3.037	141	3.131
37	2.650	72	2.903	107	3.040	142	3.133
38	2.661	73	2.908	108	3.043	143	3.135
39	2.671	74	2.912	109	3.046	144	3.138
40	2.682	75	2.917	110	3.049	145	3.140
41	2.692	76	2.922	111	3.052	146	3.142
42	2.700	77	2.927	112	3.055	147	3.144
43	2.710	78	2.931	113	3.058	148	3.146
44	2.719	79	2.935	114	3.061	149	3.148

The following equation is used to detect low outliers (U.S. Water Resources Council, 1981):

$$\log Q_L = \overline{\log Q} - K_N S \qquad 10.6$$

where: $\log Q_L$ = low outlier threshold in log units and the other terms are as defined for Equation 10.5.

If the logarithms of any annual peak discharges in a sample are less than *log* Q_L in Equation 10.6, then they are considered low outliers. Flood peaks considered low outliers are treated as zero flow years.

10.2.11.3 Zero Flow Years

Some gaged watersheds in Arizona have no flow for the entire year. The annual flood peak discharge data for these watersheds will have one or more zero flood values, and this will preclude the plotting of these zeros on the logarithmic graph papers (LN and LEV). The concept of conditional probability (Haan, 1977) is used to treat data containing zero flow years, as follows:

1. After the data are compiled and tabulated, the probability of an annual flood (non-zero data year) is calculated by:

$$P_f = \frac{N_t - Z}{N_t} = \frac{N_s}{N_t} \qquad 10.7$$

where: P_f = Probability of an annual flood,
N_t = length of systematic record including the number of zero flow years ($Nt = Ns + Z$), and
Z = number of years with zero flow

2. Rank the flood events and calculate the plotting position (P_e) using either Equation 10.2 (systematic data only) or Equation 10.3 (systematic plus historic and/or extraordinary data), with the zero flow data removed with either equation.

3. Calculate the conditional plotting position (P_z):

$$P_z = P_e \, x \, P_f \qquad 10.8$$

where: P_z = represents the plotting position for the flood data,
P_e = the probability of flood exceedance given that flooding has occurred (Equation 10.2 for systematic data only or Equation 10.3 for systematic plus historic and/or extraordinary data), and
P_f = calculated by Equation 10.7.

4. Perform the graphic flood frequency analysis as previously described using P_z as the plotting position.

10.2.12 Confidence Limits

In performing a flood frequency analysis by the graphical method or by mathematical methods, the analyst is attempting to estimate the "true" magnitudes of floods of selected return periods from a relatively small sample (record length) of observed floods. Because of the random occurrence of floods at a given location and because of the inherent variation of flood magnitudes within different periods of flood records, there cannot be certainty that the estimated flood magnitudes represent the unknown, but true flood magnitudes. For this reason, it is often prudent to calculate upper and lower confidence limits on the flood magnitudes. Such confidence limits provide a specified degree of probability that the ''true'' flood magnitudes lie between those calculated confidence limits.

Higher probability for the confidence limits results in a wider band about the best fit straight line on the selected graph paper. For example, in the extreme case, a 100 percent probability for the confidence limits would result in an upper limit for flood magnitudes of all return periods at infinity and a lower limit at zero. Such results are neither practical nor informative. There is not established criterion in the profession for confidence level probabilities. A maximum confidence level probability of 0.99 and minimum confidence level probabilities of 0.80 are occasionally used. A more popular range for confidence level is from 0.95 to 0.85. For most applications, a confidence level of 0.90 should be reasonable.

Using a confidence level of 0.90 means that there is a 90 percent chance that the true discharge for a given flood frequency (return period) will lie within the band defined by the upper and lower confidence limits. Or alternatively, there is a 5 percent chance that the true discharge for a given flood frequency is greater than that defined by the upper confidence limit and a 5 percent chance that it is less than that defined by the lower confidence limit.

Procedures were developed to place confidence limits about the best fit straight lines for all three probability distributions (LN, EV, and LEV) based on probability concepts as described by Kite (1988). An explanation of those concepts, or a discussion of those procedures, goes beyond the scope of this manual. Work sheets for establishing upper and lower confidence limits are provided in **Figure 10-10** through **Figure 10-12** for use with the LN, EV, and LEV distributions, respectively. In **Figure 10-10** through **Figure 10-12** is a variable, N_c. This variable is the number of data points that were used to fit the straight line on the probability graph paper. If all of the data were used in fitting the line, then $N_c = N_g$ (systematic data only) or $N_c = N_g$ (systematic plus historic data). However, if there is a break in the fitted straight line and if only the larger flood events are used to define the flood frequency relation, then N_c = the number of data points used to define the straight line region of the flood frequency relation.

ARIZONA DEPARTMENT OF TRANSPORTATION
HYDROLOGIC DESIGN DATA

Project No. ____________________ TRACS No. ____________________
Project Name ____________________ Date ____________________
Location/Station ____
Designer ____________________ Checker ____________________

FLOOD FREQUENCY ANALYSIS
WORK SHEET FOR LOG-NORMAL CONFIDENCE LIMITS

Gage Station Name ____
Gage Station No. ____________________

Confidence Level $(C.L.)$ = __________%

$Q_{2\text{-yr}} =$ __________cfs

$\alpha = \dfrac{100 - C.L.}{100} =$ __________

$Q_{100\text{-yr}} =$ __________cfs

$U_{1-\frac{\alpha}{2}} =$ __________

$N_C =$ __________

$\overline{Y} = \log_{10}(Q_{2-yr}) = \log_{10}(\qquad) =$ __________

$S_{ln} = \dfrac{\log_{10} Q_{100-yr} - \log_{10} Q_{2-yr}}{2.327} = \dfrac{\log_{10}(\quad) - \log_{10}(\quad)}{2.327} =$ __________

T Years (1)	$U_{1-\frac{1}{T}}$ (2)	Y_T (a) (3)	S_T (b) (4)	Limits (c) Upper (5)	Limits (c) Lower (6)
2	0.0				
5	0.842				
10	1.282				
25	1.751				
50	2.052				
100	2.327				

(a) $Y_T = \overline{Y} + U_{1-\frac{1}{T}} S_{ln}$

(c) $Q_L = 10^{(Y_T \pm U_{1-\frac{\alpha}{2}} S_T)}$

(b) $S_T = \left[\left(\dfrac{S_{ln}^{\ 2}}{N_c}\right)\left(1 + .5\, U^2_{1-\frac{1}{T}}\right)\right]^{\frac{1}{2}}$

Figure 10-10 Work Sheet for Log-Normal Confidence Limits

ARIZONA DEPARTMENT OF TRANSPORTATION
HYDROLOGIC DESIGN DATA

Project No. ______________________ TRACS No. ______________________
Project Name ______________________ Date ______________________
Location/Station ______________________
Designer ______________________ Checker ______________________

FLOOD FREQUENCY ANALYSIS
WORK SHEET FOR EXTREME VALUE CONFIDENCE LIMITS

Gage Station Name ______________________
Gage Station No. ______________________

Confidence Level $(C.L.)$ = __________%

$Q_{2\text{-yr}}$ = __________cfs $\qquad \alpha = \dfrac{100-C.L.}{100}$ = __________

$Q_{100\text{-yr}}$ = __________cfs $\qquad U_{1-\frac{\alpha}{2}}$ = __________

N_C = __________

$$A = \frac{Q_{100-yr} - Q_{2-yr}}{4.2336} = \frac{(\quad) - (\quad)}{4.2336} = \_\_\_\_\_\_$$

$$B = Q_{2-yr} - .3665\,A = (\quad) - .3665(\quad) = \_\_\_\_\_\_$$

$$\overline{Q} = B + .5772A = (\quad) + .5772(\quad) = \_\_\_\_\_\_$$

$$S_{ev} = \frac{A}{.7797} = \frac{(\quad)}{.7797} = \_\_\_\_\_\_$$

T Years (1)	K (2)	Z (a) (3)	S_T (b) (4)	Q_T (c) (5)	Limits (d) Upper (6)	Lower (7)
2	-.1643	.9179				
5	.7195	1.5458				
10	1.3046	2.0878				
25	2.0438	2.8149				
50	2.5923	3.3684				
100	3.1367	3.9240				

(a) $Z = (1.0 + 1.1396K + 1.1K^2)^{\frac{1}{2}}$

(b) $S_T = S_{ev}\left(\dfrac{Z}{N_c^{\frac{1}{2}}}\right)$

(c) $Q_T = \overline{Q} + KS_{ev}$

(d) $Q_L = Q_T \pm U_{1-\frac{\alpha}{2}} S_T$

Figure 10-11 Work Sheet for Extreme Value Confidence Limits

ARIZONA DEPARTMENT OF TRANSPORTATION
HYDROLOGIC DESIGN DATA

Project No. ______________________ TRACS No. ______________________
Project Name ______________________ Date ______________________
Location/Station ______________________
Designer ______________________ Checker ______________________

FLOOD FREQUENCY ANALYSIS
WORK SHEET FOR LOG-EXTREME VALUE CONFIDENCE LIMITS

Gage Station Name ______________________
Gage Station No. ______________________

Confidence Level $(C.L.)$ = __________%

$Q_{2\text{-yr}}$ = __________cfs

$\alpha = \dfrac{100-C.L.}{100}$ = __________

$Q_{100\text{-yr}}$ = __________cfs

$U_{1-\frac{\alpha}{2}}$ = __________

N_C = __________

$$A = \frac{\log_{10} Q_{100-yr} - \log_{10} Q_{2-yr}}{4.2336} = \frac{\log_{10}(\quad) - \log_{10}(\quad)}{4.2336} = \_\_\_\_\_\_$$

$$B = \log_{10} Q_{2-yr} - .3665\,A = \log_{10}(\quad) - .3665(\quad) = \_\_\_\_\_\_$$

$$\overline{Y} = B + .5772A = (\quad) + .5772(\quad) = \_\_\_\_\_\_$$

$$S_{lev} = \frac{A}{.7797} = \frac{(\quad)}{.7797} = \_\_\_\_\_\_$$

T Years (1)	K (2)	Z (a) (3)	S_T (b) (4)	Y_T (c) (5)	Limits (d)	
					Upper (6)	Lower (7)
2	-.1643	.9179				
5	.7195	1.5458				
10	1.3046	2.0878				
25	2.0438	2.8149				
50	2.5923	3.3684				
100	3.1367	3.9240				

(a) $$Z = (1.0 + 1.1396K + 1.1K^2)^{\frac{1}{2}}$$

(b) $$S_T = S_{lev}\left(\frac{Z}{N_c^{\frac{1}{2}}}\right)$$

(c) $$Y_T = \overline{Y} + KS_{lev}$$

(d) $$Q_L = 10^{(Y_T \pm U_{1-\frac{\alpha}{2}} S_T)}$$

Figure 10-12 Work Sheet for Log-Extreme Value Confidence Limits

10.3 INSTRUCTIONS

10.3.1 Graphical Flood Frequency Analysis

The following general steps are to be performed for the graphical flood frequency analysis as described:

1. Compile all systematic and historic data (**Figure 10-2**).
2. Compile related flood information, regional studies, and so forth.
3. Perform preliminary data analyses to investigate stationarity of the data, presence of mixed populations, and so forth.
4. Investigate the occurrence of high or low flow outliers and treat accordingly.
5. Identify extraordinary floods in the systematic record and count the number (e).
6. Tabulate the following parameters:
   a. Effective record length (N)
   b. Length of systematic record (N_t)
   c. Number of zero flow years and low flow outliers (Z)
   d. Effective length of systematic record (N_s)
   e. Number of historic data (h)
7. Calculate $N_g = N_s + h$
8. Treat for zero flow years, if they occur.
9. Prepare the data series for mixed populations, if such exists.
10. Rank the data (**Figure 10-3**) and calculate the plotting position according to the following:

Type of Data Series	Equation
Systematic data only	**10.2**
Systematic plus historic and/or extraordinary data	**10.3**
Data with zero flow years	**10.8**

11. Perform the graphical analysis as described herein.

10.3.2 Confidence Limits

The following general steps are to be performed when calculating the confidence limits:

1. Select the appropriate work sheet (Figure 10-10 through Figure 10-12) depending on which probability distribution (LN, EV, or LEV, respectively) was selected as the best fit for the flood frequency analysis.

2. Select the desired probability for the confidence level. The value $u_{1-\alpha/2}$ from the following list is used depending on the selected confidence level:

Confidence Level, %	$u_{1-\alpha/2}$
99	2.575
95	1.960
90	1.645
85	1.439
80	1.282

3. Extend the best fit straight line on the graph paper to intersect the 2-year return period, if it does not already extend to that return period.

4. Read the 2-year and 100-year flood discharges from the best fit straight line or the extension of that line.

5. Determine N_c:

   a. If the straight line extends over the entire range of data points, then

   b. $N_c = N_s$ where only systematic data exist, or

   c. $N_c = N_g$ where systematic plus historic and/or extraordinary data exist.

   d. If the data plots such that the straight line is fit only to the larger flood discharges, then N_c = number of data points used to define the straight line.

6. Using the values from Steps 2, 4, and 5 complete the calculations shown in the work sheets.

Note: If the best fit straight line had to be extended to read the 2-through 10-year return period flood magnitudes, then the confidence limits should not be calculated for that extended portion of the straight line.

7. Plot the upper and lower confidence limit points on the graph with the best fit line and draw a curved line through each set of points.

10.4 EXAMPLES

Four examples of flood frequency analyses are listed below and provided in Appendix D. These examples are included to demonstrate the application of the procedures. They are arranged from the simplest to the more complex analyses.

1. Example 10-1, Agua Fria River near Mayer, Arizona, demonstrates a fairly simple analysis requiring no special treatment of the data.

2. Example 10-2, Cave Creek near Cave Creek, Arizona, demonstrates a dataset that contains zero flow years -a fairly common occurrence for streams in Arizona.

3. Example 10-3, Hassayampa River near Wickenburg, Arizona, demonstrates a dataset containing historic data and extraordinary floods. The effective record length was extended beyond the length of the systematic record.

4. Example 10-4, Santa Cruz River near Lochiel, Arizona, demonstrates a dataset containing a low outlier and extraordinary floods. The effective length of record was extended beyond the length of the systematic record.

Chapter 11
REGRESSION EQUATIONS

This chapter contains the following details:

- Procedures and instructions to calculate peak discharges using regional regression equations.

11.1 INTRODUCTION

In this chapter, regression equations are presented for determining flood discharges for watersheds where rainfall-runoff modeling methods are unwarranted or when flood frequency data are unavailable. Regression equations may also be used as an indirect means for verification of discharges generated by analytic methods.

In addition to methods presented herein, the USGS has consolidated regional flood frequency analysis in a simple to use web based tool called StreamStats. StreamStats is a Geographic Information System (GIS) application created by the USGS that provides users with access to an assortment of analytical tools that are useful for water-resources planning and management. (http://streamstats.usgs.gov/index.html).

11.2 PROCEDURE

A regression equation procedure is provided for obtaining estimates of peak discharges for watersheds in Arizona.

11.2.1 Regional Regression Equations

An analysis of streamflow data was performed for a study area comprised of Arizona, Nevada, Utah, and parts of New Mexico, Colorado, Wyoming, Texas, Idaho, Oregon, and California. That analysis resulted in 16 sets of regional regression equations for the study area. Seven of the regions are in Arizona. These regional regression equations can be used to estimate flood magnitude-frequencies for watersheds in Arizona. Regional Regression equations may be used for verification of discharges generated by analytical methods only and not for design of highway drainage structures.

Figure 11-1 is used to determine in which of the six regions (R8, R10, R11, R12, R13, or R14) in Arizona the watershed lies. The Region 1 equations apply to any area in Arizona that is at elevation 7,500 feet or higher.

For each of the seven regions, regression equations are provided to estimate flood peak discharges for frequencies of 2-, 5-, 10-, 25-, 50-, and 100-years. Use of the regression equations is recommended only if the values of the independent variables for the watershed of interest are within the range of the database used to derive the specific regression equation. For this purpose, scatter diagrams of the values of the independent variables for each set of regression equations are provided in USGS Water Supply Paper 2433, Methods for Estimating magnitude and Frequency of Floods in the Southwest United States. To use a specific regression equation, the values of the independent variables should plot within the "cloud of common values" for the data points.

The regional regression equations are functions of drainage area and one or two other independent variable(s) that describe a key watershed characteristic. The abbreviation for each of the variables used in the equations for Arizona and the method for measuring the variable are defined as follows:

1. AREA is the drainage area, in square miles.
2. ELEV is the mean basin elevation, in thousands of feet above mean sea level.
3. PREC is the normal annual precipitation, in inches, for 1931 through 1960 (Figure 11-2). Usually PREC can be selected from Figure 11-2 at the centroid of the watershed area.
4. EVAP is the mean annual free water-surface evaporation, in inches (Farnsworth and others, 1982). The EVAP value at the study-site location is used, not the value at the centroid of the watershed area or the grid-sampled average value for the watershed.

For each defined flood region in Arizona, the flood magnitude-frequency regression equation is shown in a table. The table, corresponding independent variable scatter diagram, and 100-year peak discharge versus drainage area graph for each region in Arizona are listed below:

Region	Table No. for Regression Equation	Figure No. for Independent Variable Scatter Diagrams	Figure No. for 100-year Discharge vs. Area Graph
1	Table 11–1	Figure 11-4	Figure 11-5
8	Table 11–2	Figure 11-6	Figure 11-7
10	Table 11–3	NA	Figure 11-8
11	Table 11–4	Figure 11-9	Figure 11-10
12	Table 11–5	Figure 11-11	Figure 11-12
13	Table 11–6	NA	Figure 11-13
14	Table 11–7	Figure 11-14	Figure 11-15

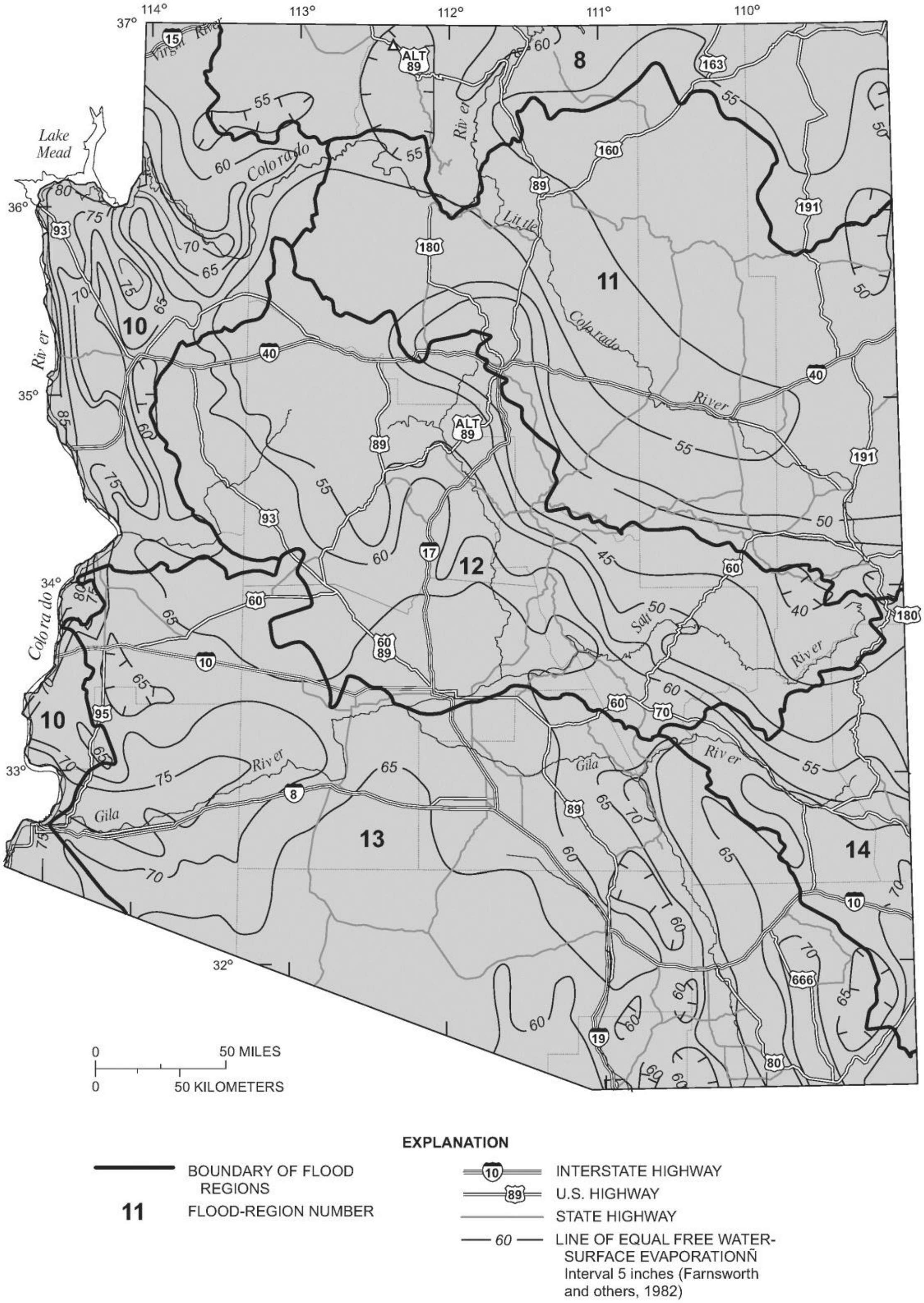

Figure 11-1 Hydrologic Flood Regions for Arizona

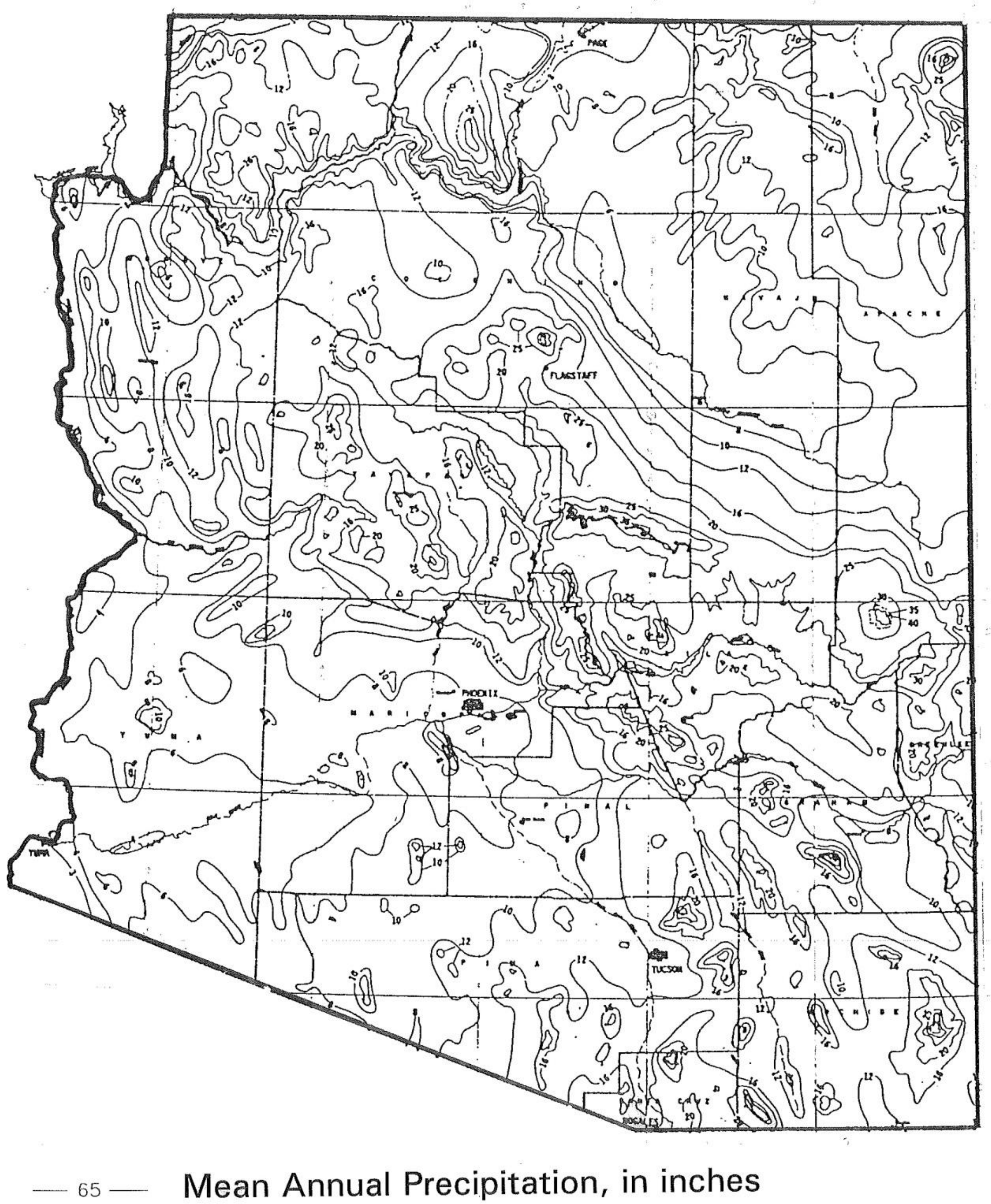

Figure 11-2 Mean Annual Precipitation (PREC)

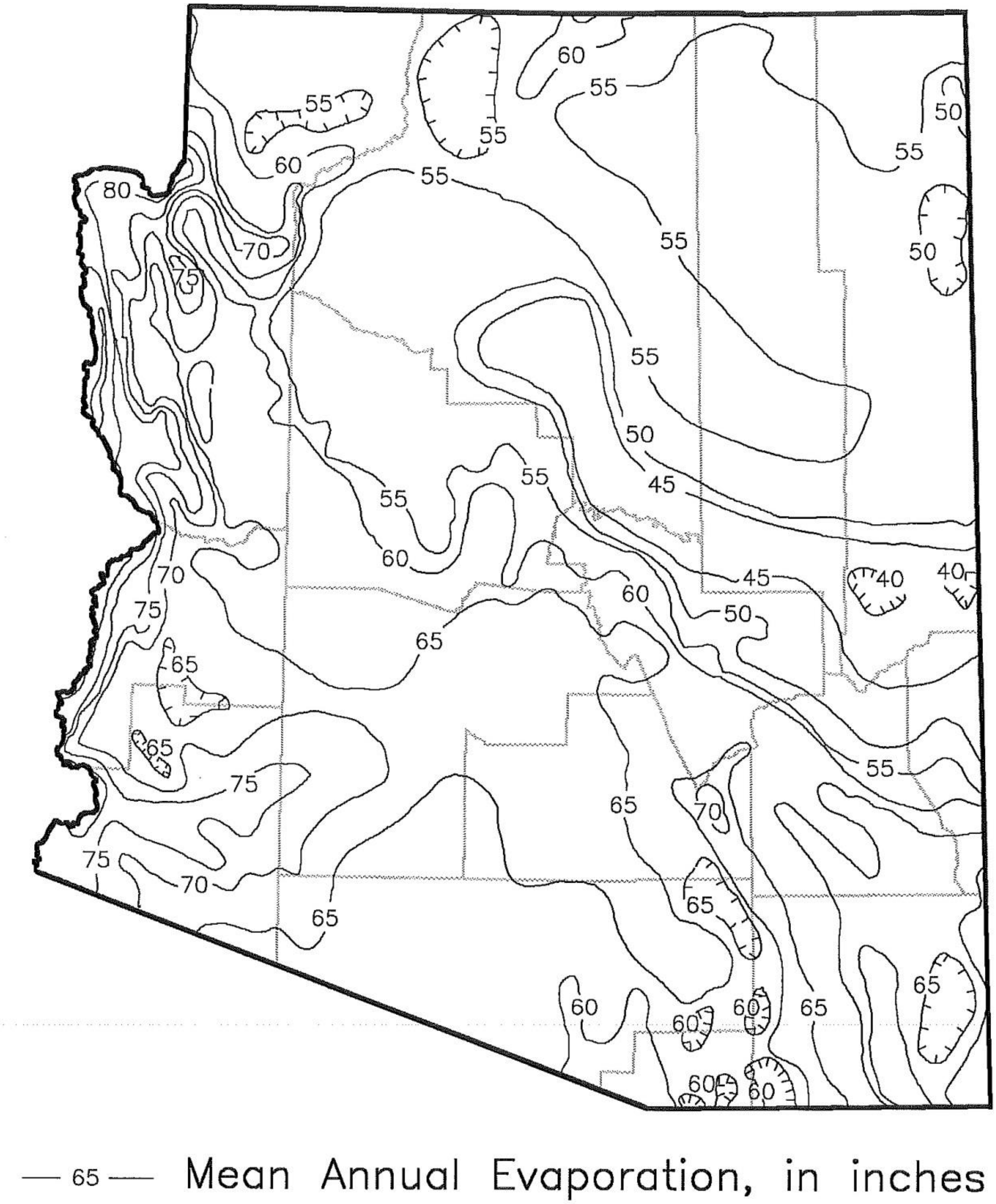

Figure 11-3 Mean Annual Evaporation (EVAP)

Table 11–1 Flood Magnitude-Frequency Relations for the High Elevation Region (R1)

Recurrence interval, in years	Equation	Average standard error of prediction, in percent
2	$Q = 0.124\ AREA^{0.845} PREC^{1.44}$	59
5	$Q = 0.629\ AREA^{0.807} PREC^{1.12}$	52
10	$Q = 1.43\ AREA^{0.786} PREC^{0.958}$	48
25	$Q = 3.08\ AREA^{0.768} PREC^{0.811}$	46
50	$Q = 4.75\ AREA^{0.758} PREC^{0.732}$	46
100	$Q = 6.78\ AREA^{0.750} PREC^{0.668}$	46

Equation: Q, peak discharge, in cubic feet per second; AREA, drainage area, in square miles; and PREC, mean annual precipitation, in inches.

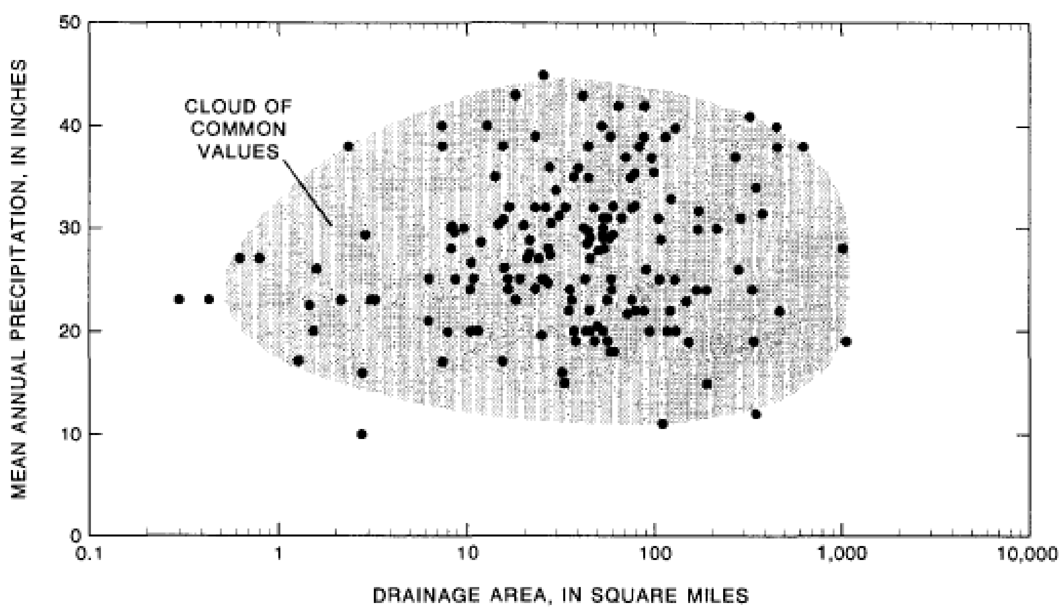

Figure 11-4 Scatter diagram of Independent Variables for R1 Regression Equation

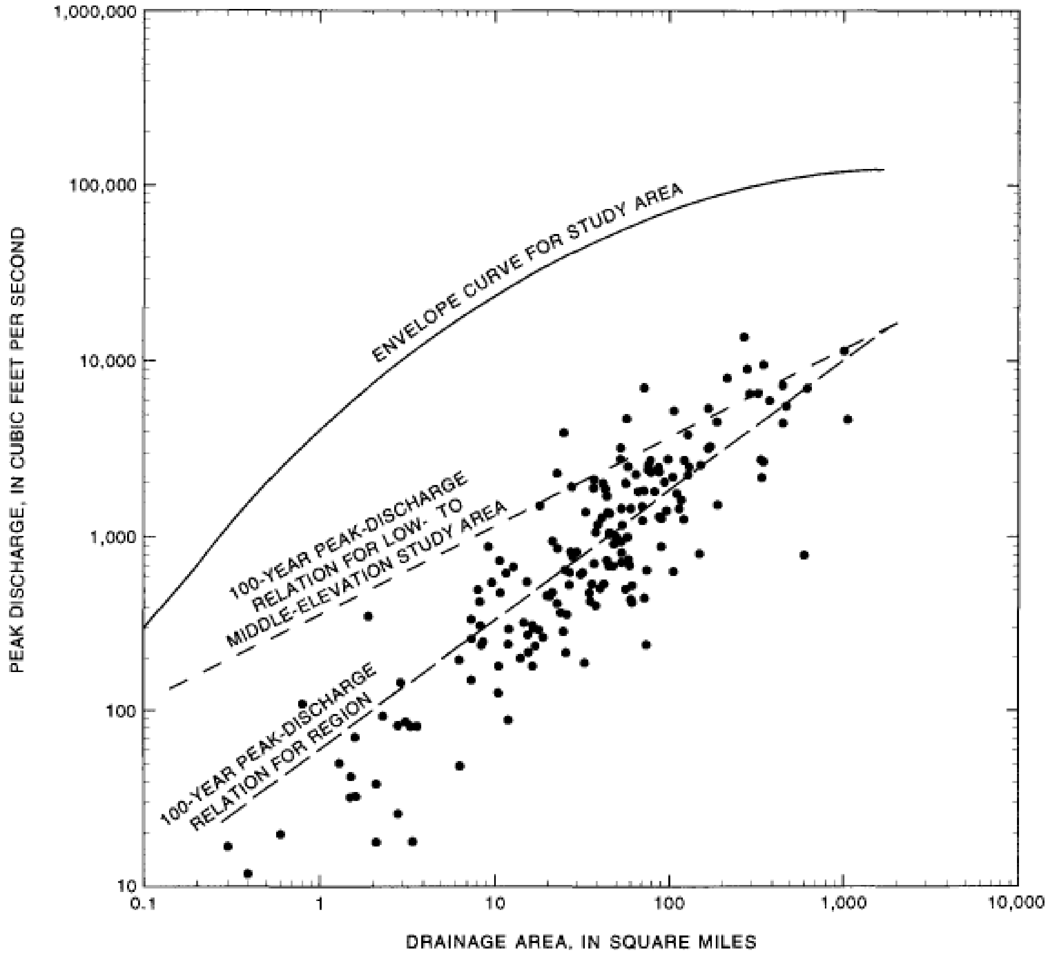

Figure 11-5 Data Points and 100-year Peak Discharge Relation for R1

Table 11–2 Flood Magnitude-Frequency Relations for the Four Corners Region (R8)		
Recurrence interval, in years	**Equation**	**Average standard error of prediction, in percent**
2	$Q = 598\ AREA^{0.501}(ELEV/1{,}000)^{-1.02}$	72
5	$Q = 2{,}620\ AREA^{0.449}(ELEV/1{,}000)^{-1.28}$	62
10	$Q = 5{,}310\ AREA^{0.425}(ELEV/1{,}000)^{-1.40}$	57
25	$Q = 10{,}500\ AREA^{0.403}(ELEV/1{,}000)^{-1.49}$	54
50	$Q = 16{,}000\ AREA^{0.390}(ELEV/1{,}000)^{-1.54}$	53
100	$Q = 23{,}300\ AREA^{0.377}(ELEV/1{,}000)^{-1.59}$	53
Equation: Q, peak discharge, in cubic feet per second; AREA, drainage area, in square miles; and ELEV, mean basin elevation, in feet.		

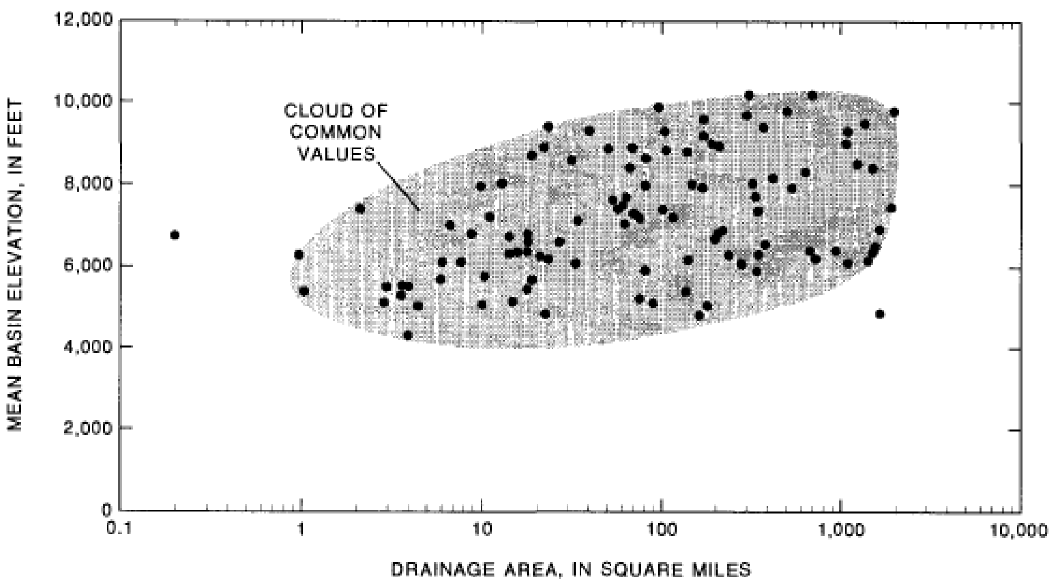

Figure 11-6 Scatter Diagram of Independent Variables for R8 Regression Equation

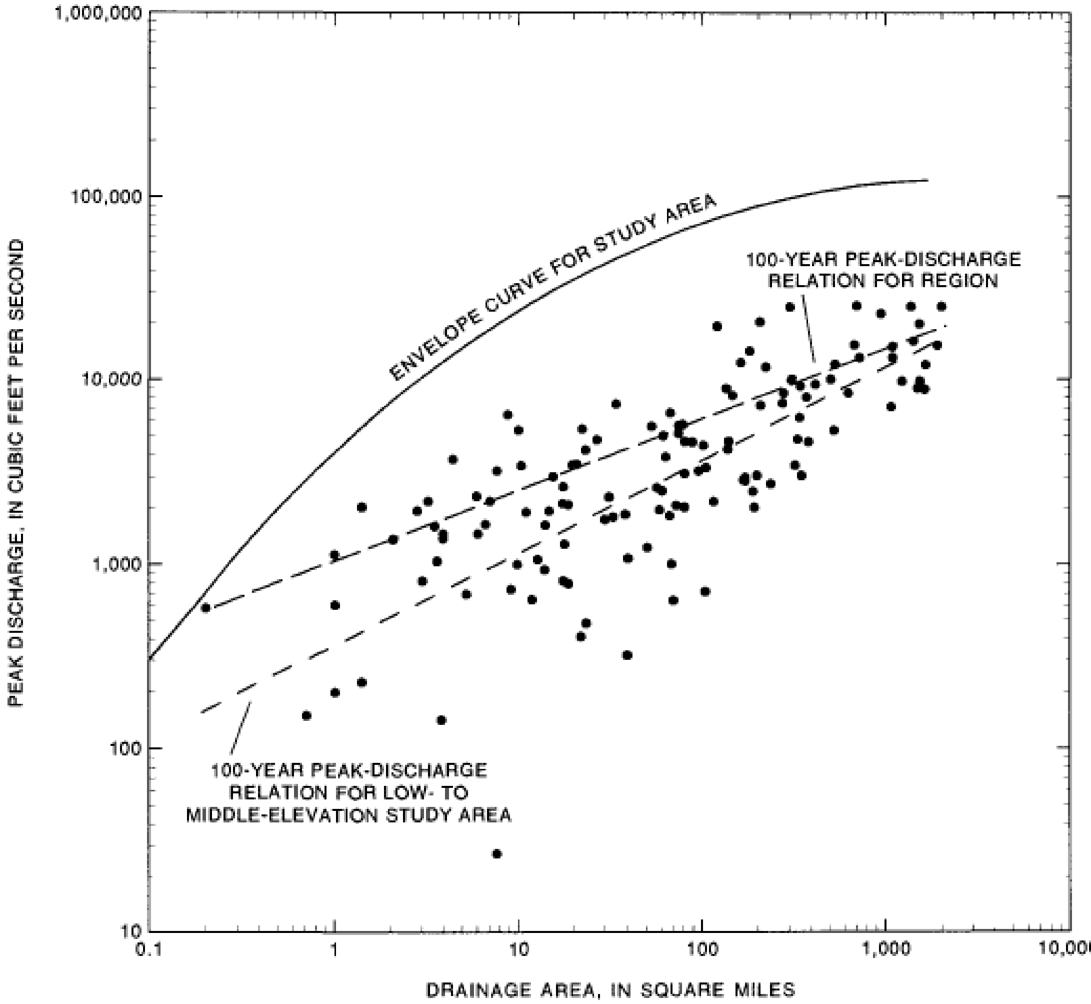

Figure 11-7 Data Points and 100-year Peak Discharge Relation for R8

Table 11–3 Flood Magnitude-Frequency Relations for the Southern Great Basin Region (R10)

Recurrence interval in years	Equation	Estimated Average standard error of regression, in log units
2	$Q = 12\ AREA^{0.58}$	1.140
5	$Q = 85\ AREA^{0.59}$	0.602
10	$Q = 200\ AREA^{0.62}$	0.675
25	$Q = 400\ AREA^{0.65}$	0.949
50	$Q = 590\ AREA^{0.67}$	0.928
100	$Q = 850\ AREA^{0.69}$	1.230

Equation: Q, peak discharge, in cubic feet per second; and AREA, drainage area, in square miles.

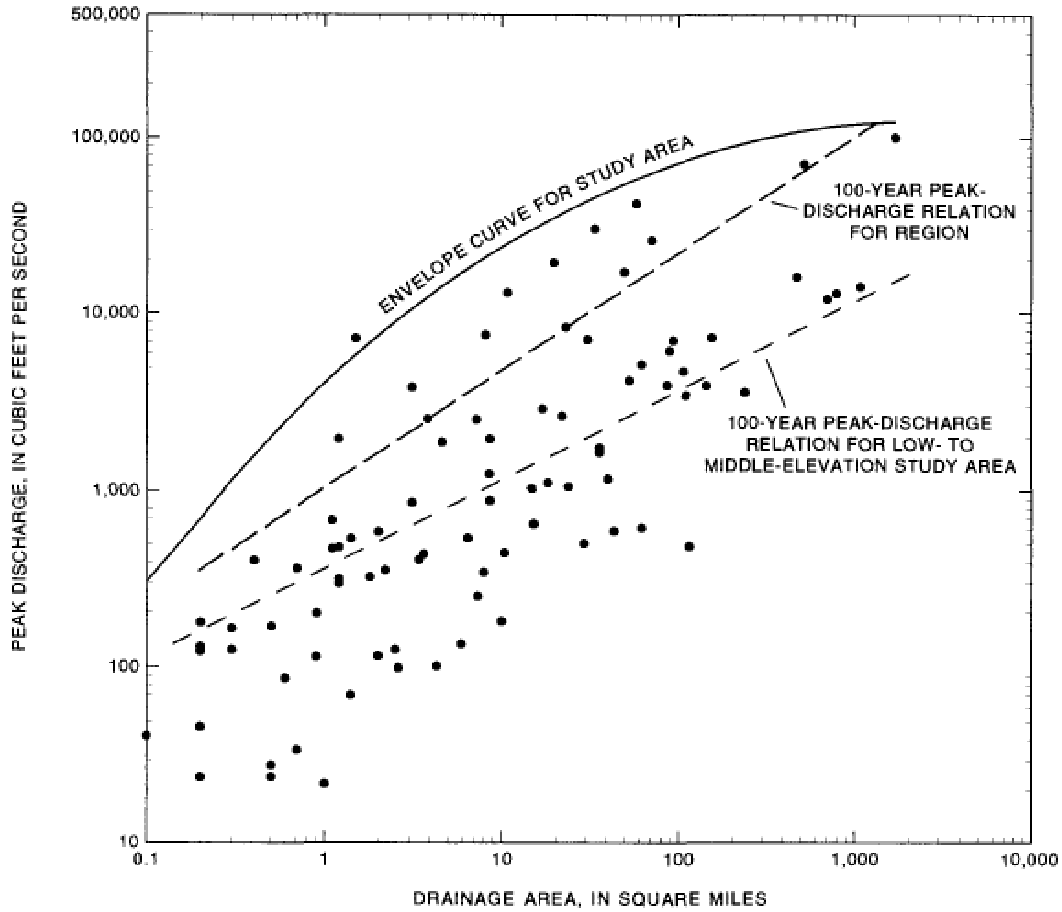

Figure 11-8 Data Points and 100-year Peak Discharge Relation for R10

Table 11–4 Flood Magnitude-Frequency Relations for the Northeastern Arizona Region (R11)

Recurrence interval, in years	Equation	Estimated average standard error of regression, in log units
2	$Q = 26\ AREA^{0.62}$	0.609
5	$Q = 130\ AREA^{0.56}$	0.309
10	$Q = 0.10\ AREA^{0.52} EVAP^{2.0}$	0.296
25	$Q = 0.17\ AREA^{0.52} EVAP^{2.0}$	0.191
50	$Q = 0.24\ AREA^{0.54} EVAP^{2.0}$	0.294
100	$Q = 0.27\ AREA^{0.58} EVAP^{2.0}$	0.863

Equation: Q, peak discharge, in cubic feet per second; AREA, drainage area, in square miles; and EVAP, mean annual evaporation, in inches.

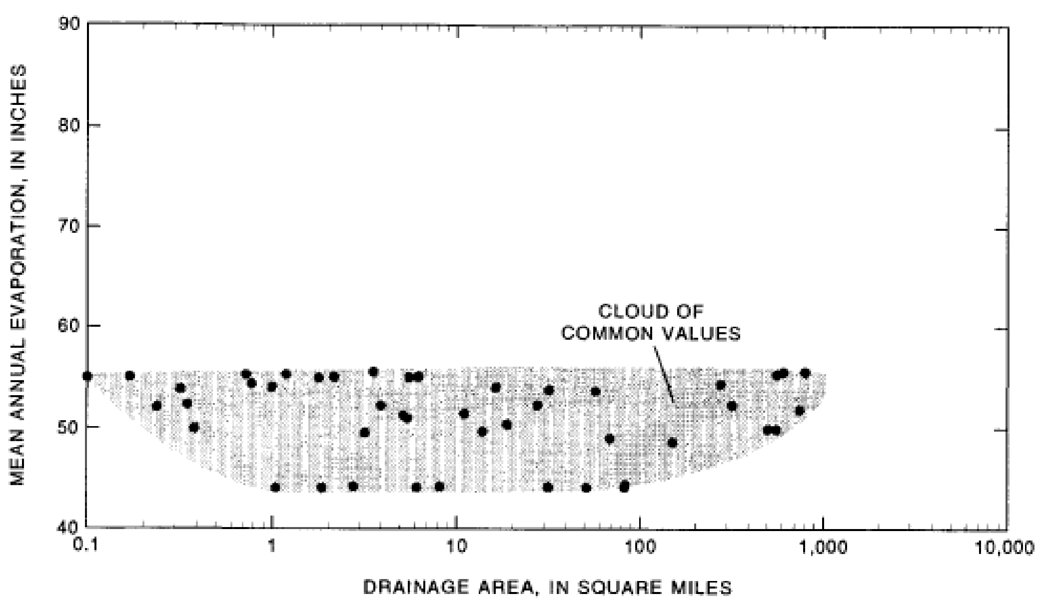

Figure 11-9 Scatter Diagram of Independent Variables for R11 Regression Equation

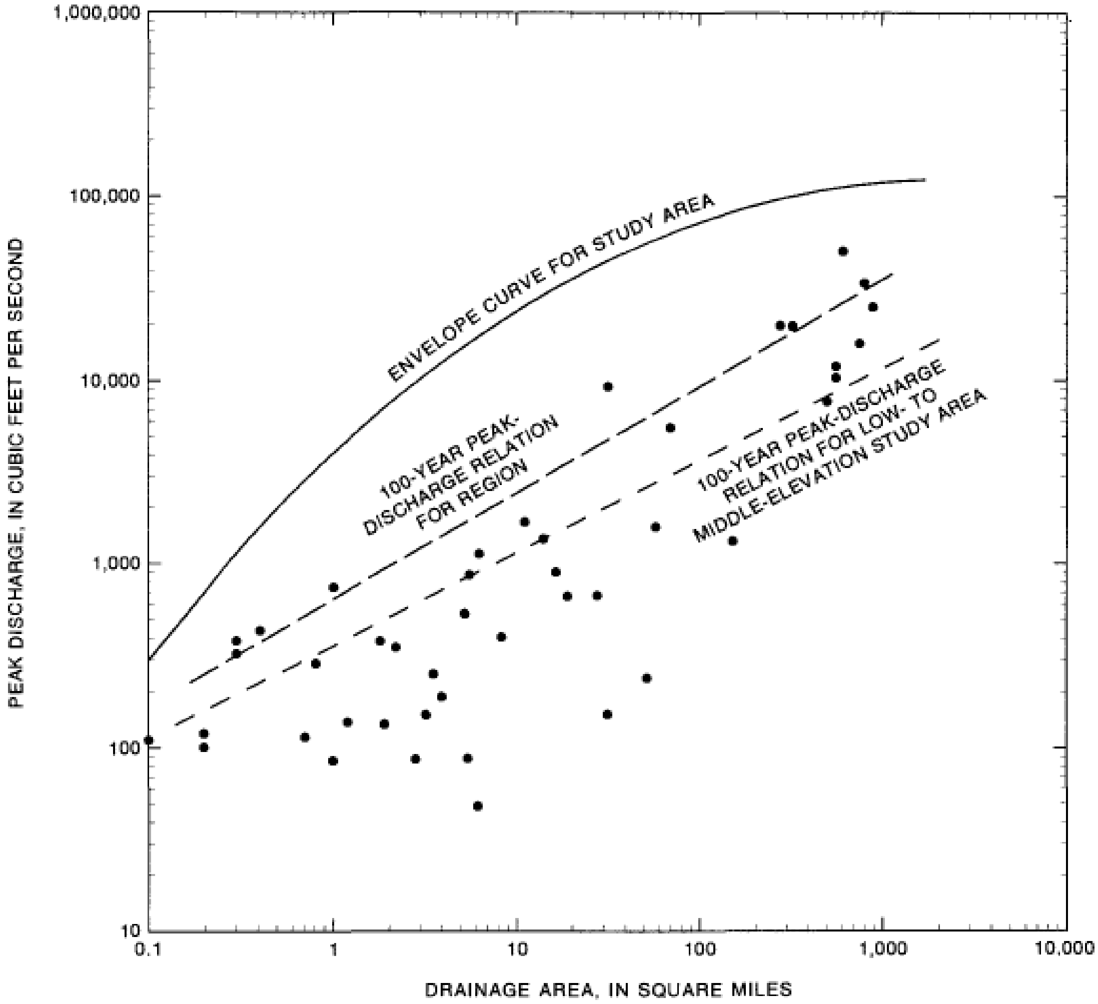

Figure 11-10 Data Points and 100-year Peak Discharge Relation for R11

Table 11–5 Flood Magnitude-Frequency Relations for the Central Arizona Region (R12)

Recurrence interval, in years	Equation	Average standard error of prediction, in percent
2	$Q = 41.1\ AREA^{0.629}$	105
5	$Q = 238\ AREA^{0.687} ELEV^{-0.358}$	68
10	$Q = 479\ AREA^{0.661} ELEV^{-0.398}$	52
25	$Q = 942\ AREA^{0.630} ELEV^{-0.383}$	40
50	$Q = 10^{(7.36-4.17\ AREA^{-0.08})}\ (ELEV/1{,}000)^{-0.440}$	37
100	$Q = 10^{(6.55-3.17\ AREA^{-0.11})}\ (ELEV/1{,}000)^{-0.454}$	39
Equation: Q, peak discharge, in cubic feet per second; AREA, drainage area, in square miles; and ELEV, mean basin elevation, in feet.		

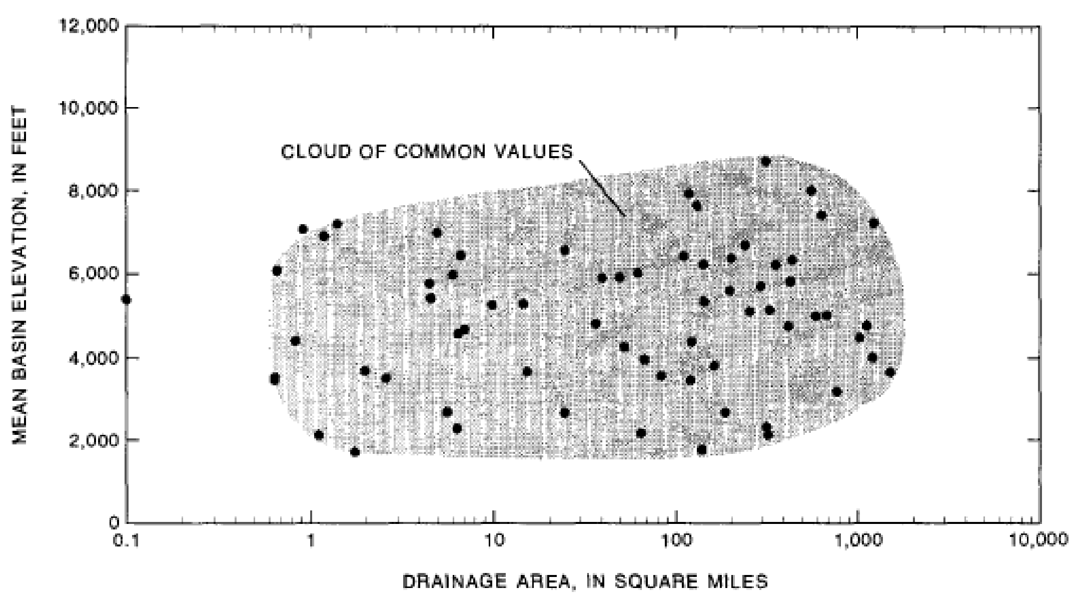

Figure 11-11 Scatter Diagram of Independent Variables for R12 Regression Equation

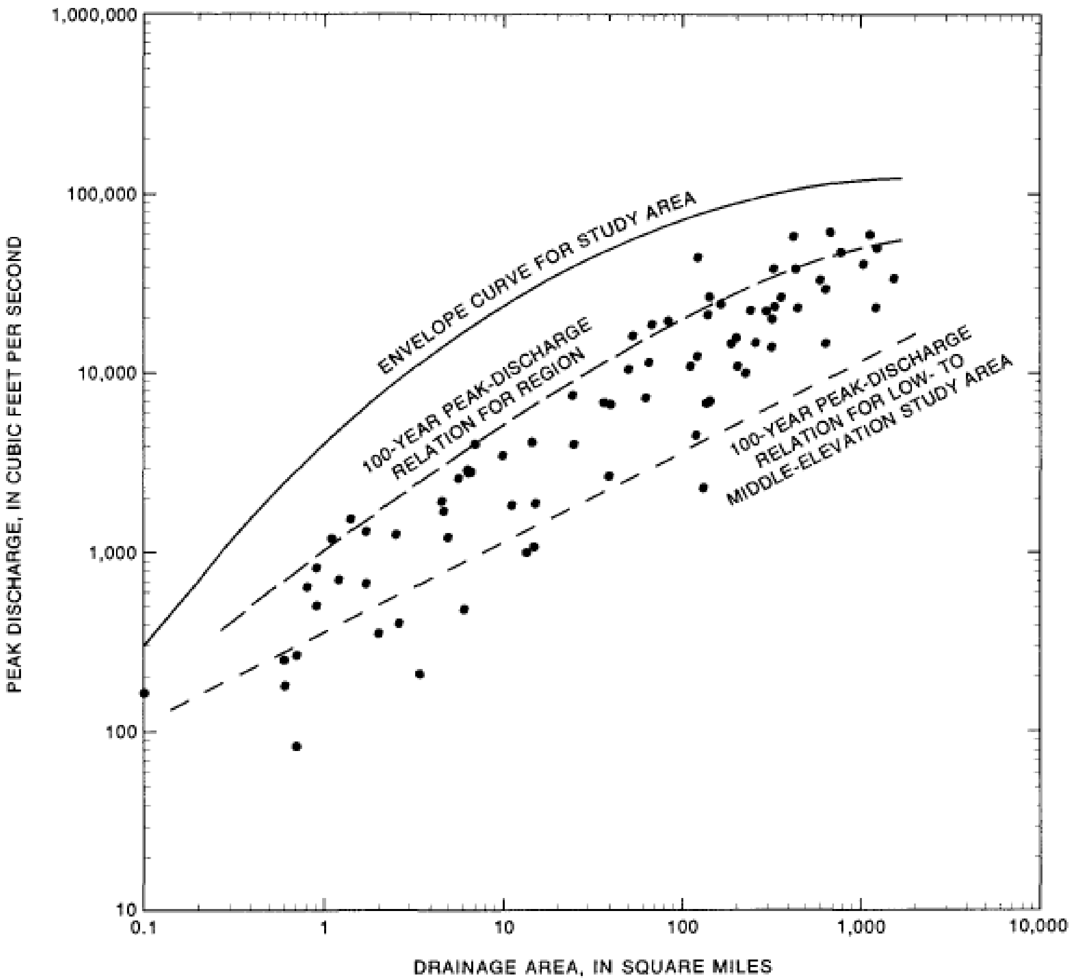

Figure 11-12 Data Points and 100-year Peak Discharge Relation for R12

Table 11–6 Flood Magnitude-Frequency Relations for the Southern Arizona Region (R13)

Recurrence interval, in years	Equation	Average standard error of prediction, in percent
2	$Q = 10^{(6.38 - 4.29\ AREA^{-0.06})}$	57
5	$Q = 10^{(5.78 - 3.31\ AREA^{-0.08})}$	40
10	$Q = 10^{(5.68 - 3.02\ AREA^{-0.09})}$	37
25	$Q = 10^{(5.64 - 2.78\ AREA^{-0.10})}$	39
50	$Q = 10^{(5.57 - 2.59\ AREA^{-0.11})}$	43
100	$Q = 10^{(5.52 - 2.42\ AREA^{-0.12})}$	48

Equation: *Q*, peak discharge, in cubic feet per second; and AREA, drainage area, in square miles;

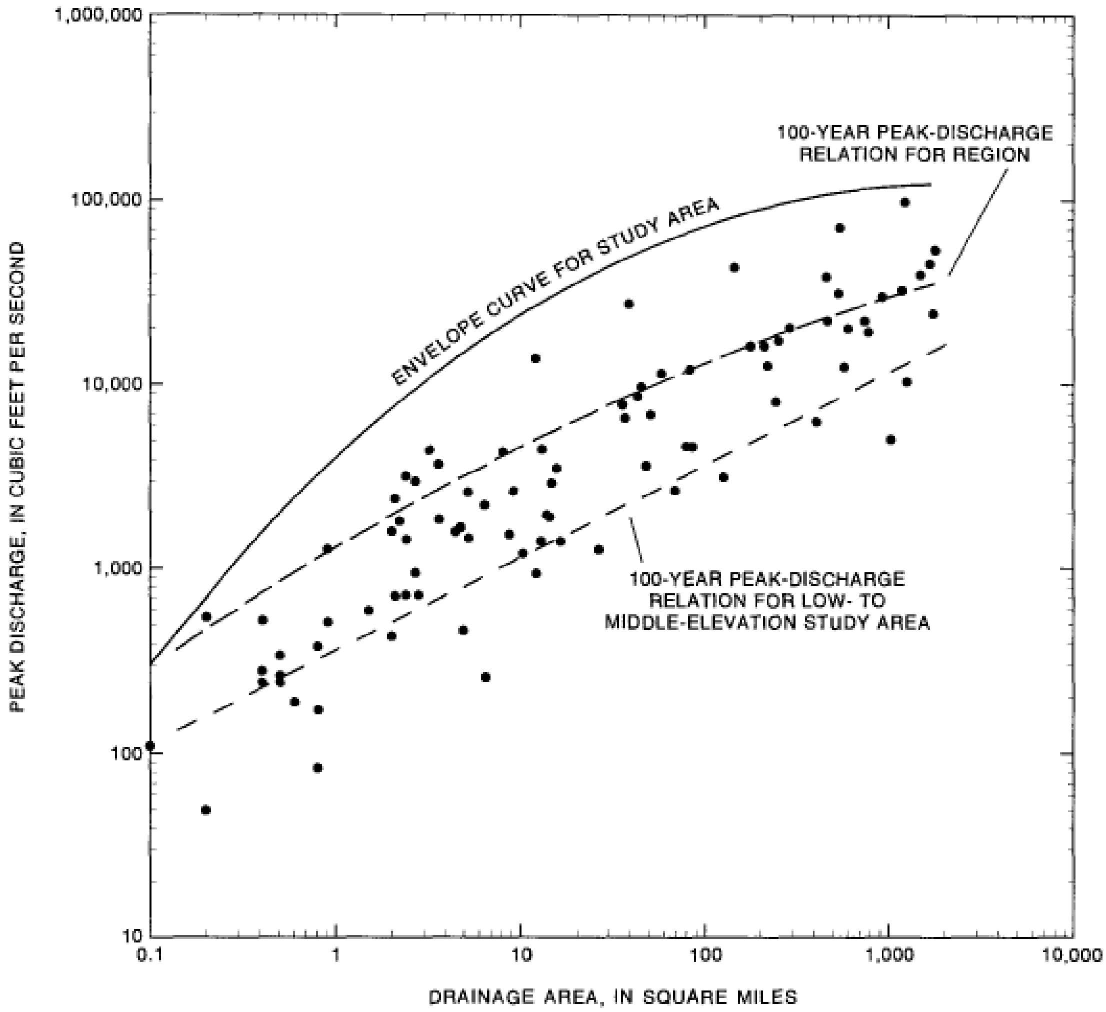

Figure 11-13 Data Points and 100-year Peak Discharge Relation for R13

Table 11–7 Flood Magnitude-Frequency Relations for the Upper Gila Basin Region (R14)

Recurrence interval, in years	Equation	Average standard error of prediction, in percent
2	$Q = 583\ AREA^{0.588}(ELEV/1{,}000)^{-1.30}$	74
5	$Q = 618\ AREA^{0.524}(ELEV/1{,}000)^{-0.70}$	63
10	$Q = 361\ AREA^{0.464}$	65
25	$Q = 581\ AREA^{0.462}$	63
50	$Q = 779\ AREA^{0.462}$	64
100	$Q = 1{,}010\ AREA^{0.463}$	66
Equation: Q, peak discharge, in cubic feet per second; AREA, drainage area, in square miles; and ELEV, mean basin elevation, in feet.		

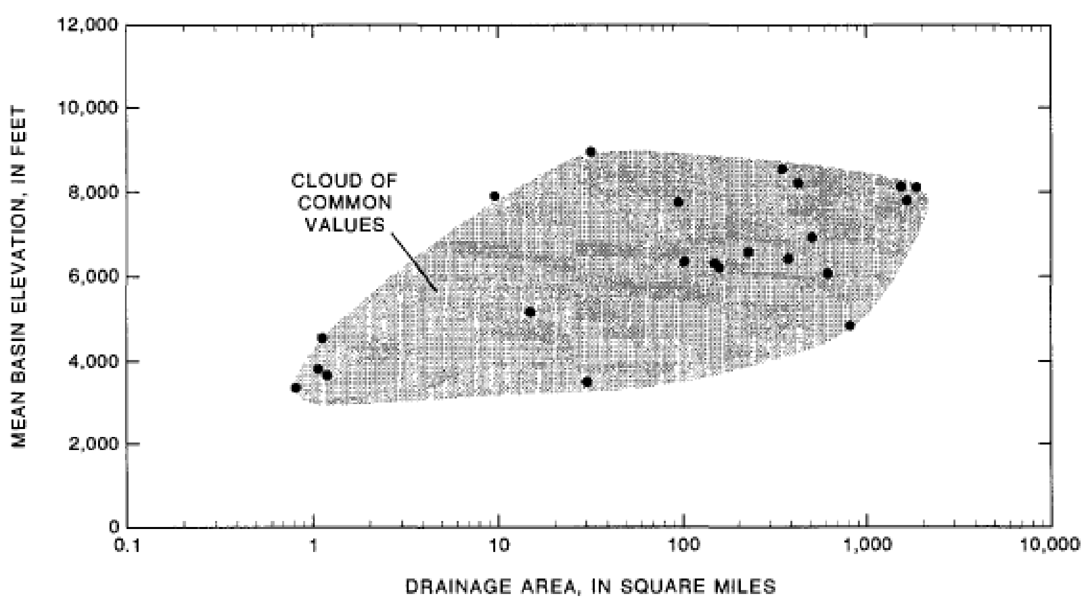

Figure 11-14 Scatter Diagram of Independent Variables for R14 Regression Equation

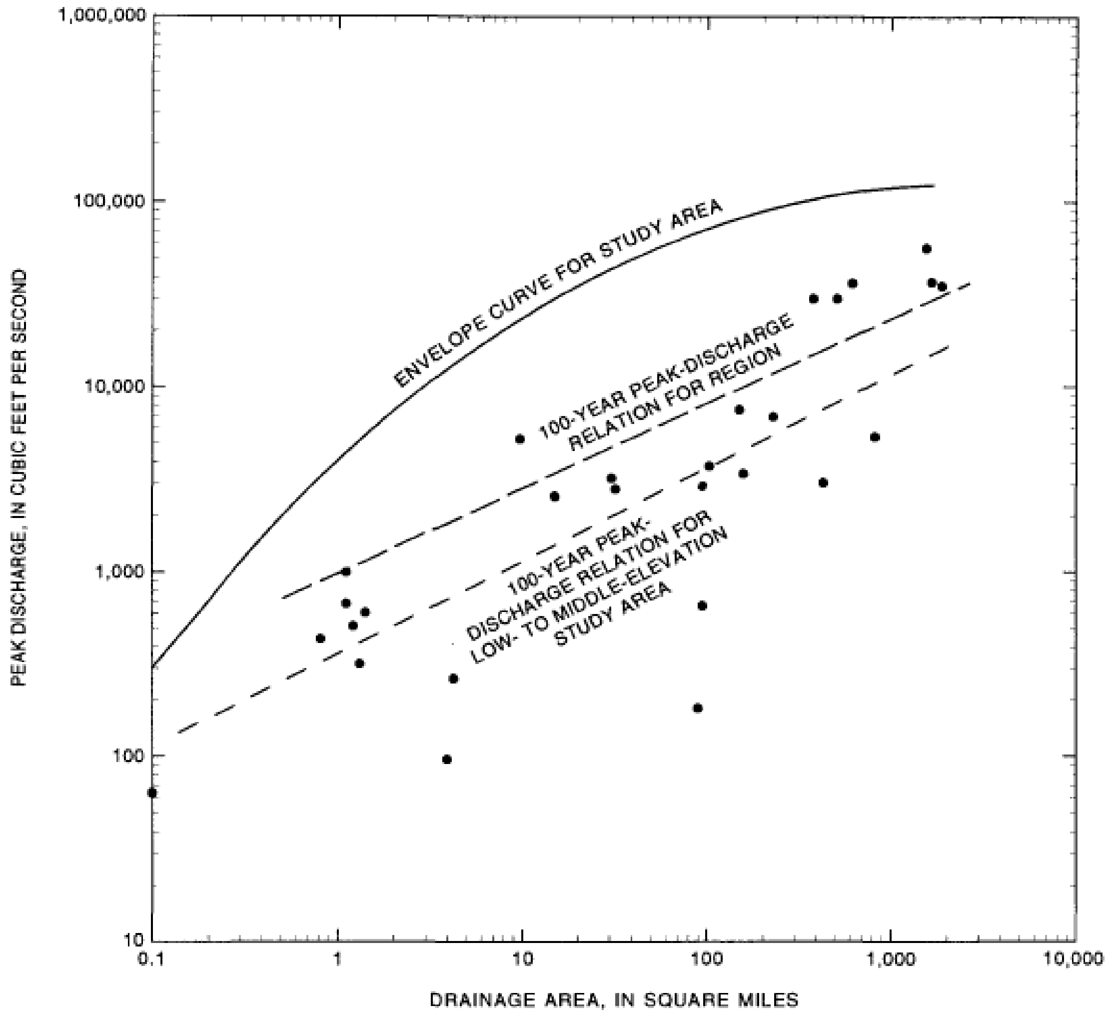

Figure 11-15 Data Points and 100-year Peak Discharge Relation for R14

11.2.2 Applications and Limitations

Limitations exist for the use of the Regional Regression Equations based on values of the watershed characteristics as compared to the values of watershed characteristics that were used to derive these regional regression equations. The interpretation and evaluation of the results of these methods must be conducted with awareness of several factors.

1. It must be noted that these are empirical methods and the results are only applicable to watersheds that are hydrologically similar to the database used to derive the particular method. Refer to the independent variable scatter diagrams when using the Regional Regression Equations.

2. The bulk of the data in all of these methods are for undeveloped watersheds. Urbanized watersheds can have significantly higher discharges than the results that are predicted by any of these methods.

3. This method produces discharge values that are statistically based averages for watersheds in the database. Conditions can exist in any watershed that would produce flood discharges either larger than or smaller than those indicated by these methods. Watershed characteristics that should be considered when comparing the results of the regional regression equations to results by analytic methods and/or flood frequency analysis are as follows:

    a. The occurrence and extent of rock outcrop in the watershed

    b. Watershed slopes that are either exceptionally flat or steep

    c. Soil and vegetation conditions that are conducive to low rainfall losses, such as clay soils, thin soil horizons underlain by rock or clay layers, denuded watersheds (forest and range fires), and disturbed land

    d. Soil and vegetation conditions that are conducive to high rainfall losses, such as sandy soil, volcanic cinder, forest duff, tilled agricultural land, and irrigated turf

    e. Land-use, especially urbanization, but also mining, large scale construction activity, timber harvesting, and over-grazing

    f. Transmission losses that may occur in the watercourses

    g. The existence of distributary flow areas

    h. Upstream water regulation or diversion

Refer to USGS Water Supply Paper 2433, Methods for Estimating Magnitude and Frequency of Floods in the Southwestern United States, 1997 and Arizona Department of Water Resources (ADWR) State Standard 2, 1996 for further discussion on the application of regression equations.

Chapter 12
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