ARIZONA DEPARTMENT OF TRANSPORTATION

REPORT NUMBER: FHWA-AZ94-442

HIGHWAY DRAINAGE DESIGN MANUAL
HYDROLOGY
METRIC EDITION

Final Report

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December, 1994

35-250

Prepared for:
Arizona Department of Transportation
206 South 17th Avenue
Phoenix, Arizona 85007
in cooperation with
U.S. Department of Transportation
Federal Highway Administration

Technical Report Documentation Page

1. Report No.	2. Government Accession No.	3. Recipient's Catalog No.
FHWA-AZ-94-442		

4. Title and Subtitle
HIGHWAY DRAINAGE DESIGN MANUAL HYDROLOGY METRIC VERSION

5. Report Date
December 1994

6. Performing Organization Code

7. Author
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8. Performing Organization Report No.

9. Performing Organization Name and Address
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10. Work Unit No.

11. Contract or Grant No.
SPR-PL-1(47)442

12. Sponsoring Agency Name and Address
ARIZONA DEPARTMENT OF TRANSPORTATION
206 S. 17TH AVENUE
PHOENIX, ARIZONA 85007

13. Type of Report & Period Covered

14. Sponsoring Agency Code

15. Supplementary Notes
Prepared in cooperation with the U.S. Department of Transportation, Federal Highway Administration

16. Abstract
This manual is an update and metric conversion of the Arizona Department of Transportations Highway Drainage Design Manual Hydrology

17. Key Words
Drainage Design, Manual, Hydrology

18. Distribution Statement

23. Registrant's Seal
Registered Professional Engineer (Civil)
CERTIFICATE NO.
17928
GEORGE V.
SABOL
Date Signed 18 Jan 95
ARIZONA, U.S.A.

19. Security Classification	20. Security Classification	21. No. of Pages	22. Price
Unclassified	Unclassified		

METRIC (SI*) CONVERSION FACTORS

APPROXIMATE CONVERSIONS TO SI UNITS

Symbol	When You Know	Multiply By	To Find	Symbol
		LENGTH		
in	inches	2.54	centimeters	cm
ft	feet	0.3048	meters	m
yd	yards	0.914	meters	m
mi	miles	1.61	kilometers	km
		AREA		
in^2	square inches	6.452	centimeters squared	cm^2
ft^2	square feet	0.0929	meters squared	m^2
yd^2	square yards	0.836	meters squared	m^2
mi^2	square miles	2.59	kilometers squared	km^2
ac	acres	0.395	hectares	ha
		MASS (weight)		
oz	ounces	28.35	grams	g
lb	pounds	0.454	kilograms	kg
T	short tons (2000 lb)	0.907	megagrams	Mg
		VOLUME		
fl oz	fluid ounces	29.57	millimeters	mL
gal	gallons	3.785	liters	L
ft^3	cubic feet	0.0328	meters cubed	m^3
yd^3	cubic yards	0.765	meters cubed	m^3

Note: Volumes greater than 1000 L shall be shown in m^3.

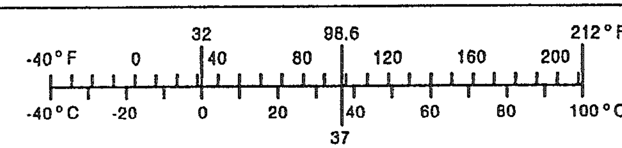
Symbol	When You Know	Multiply By	To Find	Symbol
		TEMPERATURE (exact)		
°F	Fahrenheit temperature	5/9 (after subtracting 32)	Celsius temperature	°C

These factors conform to the requirement of FHWA Order 5190.1A

*SI is the symbol for the International System of Measurements

APPROXIMATE CONVERSIONS TO SI UNITS

Symbol	When You Know	Multiply By	To Find	Symbol
		LENGTH		
mm	millimeters	0.039	inches	in
m	meters	3.28	feet	ft
yd	meters	1.09	yards	yd
km	kilometers	0.621	miles	mi
		AREA		
mm^2	millimeters squared	0.0016	square inches	in^2
m^2	meters squared	10.764	square feet	ft^2
yd^2	kilometers squared	0.39	square miles	mi^2
ha	hectares (10,000 m^2)	2.53	acres	ac
		MASS (weight)		
g	grams	0.0353	ounces	oz
kg	kilograms	2.205	pounds	lb
Mg	megagrams (1000 kg)	1.103	short tons	T
		VOLUME		
mL	millimeters	0.034	fluid ounces	fl oz
L	liters	0.264	gallons	gal
m^3	meters cubed	35.315	cubic feet	ft^3
m^3	meters cubed	1.308	cubic yards	yd^3
		TEMPERATURE (exact)		
°C	Celsius temperature	9/5 (then add 32)	Fahrenheit temperature	°F



PREFACE TO THE METRIC EDITION

The Highway Drainage Design Manual, Hydrology, was originally produced with English units of measurement (dated March 1993 with revisions dated April 1994). The Metric Edition is essentially the same manual, but with numeric values in metric units of measurement.

The Highway Drainage Design Manual, Hydrology, is intended to provide guidance for the performance of flood hydrology for Arizona Department of Transportation (ADOT) drainage design. Two analytic methods are presented, herein, to determine design discharges, and those two methods are to be used mainly for ungaged watersheds. The two analytic methods are; (1) the Rational Method that can be used for uniform drainage areas that are not larger than 160 acres in size, and (2) rainfall-runoff modeling for any size drainage area. The rainfall-runoff modeling guidance is structured to be compatible with the HEC-1 Flood Hydrology program by the U.S. Army Corps of Engineers. For rainfall-runoff modeling, this manual should be used in conjunction with the HEC-1 Users Manual, and the contents of this manual assumes a familiarity and basic understanding of the HEC-1 program and modeling procedures.

A flood frequency analysis procedure is provided for computing flood magnitude-frequency relations where systematic stream gaging 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.

Three indirect methods are presented for estimating flood peak discharges. Results by either analytic methods or flood frequency analysis should always be compared and evaluated by indirect methods. There may be cases where the flood discharges by all three methods (analytic, flood frequency analysis, and indirect) can be obtained and compared prior to making a selection of design discharge.

This manual was prepared for use by engineers and/or hydrologists that 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.

The information in the manual is presented in the following Sections and Chapters:

SECTION I - RAINFALL

Chapter 1 - Rainfall Procedures and instructions are provided to prepare rainfall input to the HEC-1 program, and to generate intensity-duration-frequency curves for use with the Rational Method.

SECTION II - RATIONAL METHOD

Chapter 2 - Rational Method Procedures and instructions are provided for using the Rational Method. This includes two general intensity-duration-frequency curves, a time of concentration equation, and graphs for the selection of the runoff coefficient.

SECTION III - RAINFALL-RUNOFF MODELING

Chapter 3 - Rainfall Losses The method to be used to estimate rainfall losses by the Green and Ampt equation is presented.

Chapter 4 - Unit Hydrographs The Clark unit hydrograph is recommended and procedures to calculate the unit hydrograph parameters are presented.

Chapter 5 - Channel Routing Recommendations and instructions for channel routing are presented.

Chapter 6 - Storage Routing Recommendations and instructions for storage routing are presented.

Chapter 7 - Transmission Losses A discussion of channel transmission losses and guidance on when to incorporate transmission losses into a rainfall-runoff model are presented.

Chapter 8 - Modeling Technique and General Guidance for Using HEC-1 Applicability, assumptions and limitations of the HEC-1 program, general guidance for watershed modeling, and a modeler's/reviewer's checklist are provided.

SECTION IV - FLOOD FREQUENCY ANALYSIS

Chapter 9 - Flood Frequency Analysis Procedures and instructions are provided, along with worksheets and graph paper, for performing graphical flood frequency analyses. A procedure for placing confidence limits about the flood frequency line is provided.

SECTION V - INDIRECT METHODS FOR DISCHARGE VERIFICATION

Chapter 10 - Indirect Methods for Discharge Verification Three methods are presented for checking and "verifying" peak discharges that are obtained by the analytic methods (Rational Method and rainfall-runoff modeling), and by flood frequency analysis.

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CHAPTER 1
RAINFALL

1.1 INTRODUCTION

1.1.1 General Discussion

Analytic methods (Rational Method and rainfall-runoff modeling using the HEC-1 program) require the definition of the rainfall for the desired flood frequency. For the Rational Method, a rainfall intensity-duration-frequency (I-D-F) graph is required. Generalized I-D-F graphs for 2 zones in Arizona are provided for the Rational Method (Chapter 2). There may be situations when a site-specific I-D-F graph is to be used with the Rational Method, and a procedure for developing a site-specific I-D-F graph for any location in Arizona is presented in this section.

For rainfall-runoff modeling (HEC-1 program), the temporal and spatial distribution of the design rainfall must be provided. For highway drainage studies in Arizona, a symmetric nesting of rainfall depths for specified intra-storm durations is used. That rainfall distribution is called the hypothetical distribution, and when using the HEC-1 program, input is provided in the PH record. The point rainfall depth-duration-frequency (D-D-F) statistics that are input in the PH record are automatically adjusted for the rainfall depth-area relation by procedures built into the HEC-1 program. The hypothetical distribution methodology is described in U.S. Army Corps of Engineers, Training Document No. 15 (1982).

1.1.2 Source of Design Rainfall Information

The rainfall depth-duration-frequency statistics for Arizona are derived from information in NOAA Atlas 2, Volume VIII, Arizona (Miller and others, 1973). The short-duration (less than 1-hour) rainfall ratios are from Arkell and Richards (1986). The depth-area reduction curves are those from the NOAA Atlas 2. The NOAA Atlas 2 presents point rainfall depth-duration-frequency values as isopluvial maps in English units. Therefore, it is necessary to convert the rainfall depths from the NOAA Atlas 2 from English units, in inches, to metric units, in millimeters.

1.2 PROCEDURE

1.2.1 General Considerations

Rational Method - When using the Rational Method, either one of the two generalized I-D-F graphs, one for Zone 6 and one for Zone 8 (see Chapter 2 - Rational Method), or a site-specific I-D-F graph is used. The T-year, 1-hour rainfall depth is used with the Rational Method, where T indicates the desired design flood return period.

HEC-1 Modeling - When using the HEC-1 model, the rainfall input is provided in the PH record. The storm duration to be used depends on the total watershed area as follows:

1. If the total watershed area is less than or equal to 2.5 square kilometers, the design storm duration is 6 hours.

2. If the total watershed area is greater than 2.5 square kilometers, the design storm duration is 24 hours.

Arkell and Richards (1986) determined that the short-duration (less than 1-hour) rainfall ratios, as shown in the NOAA Atlas 2 series, are not appropriate for the entire western United States. They identified zones that have different short-duration rainfall ratios and provided those ratios for each zone. Arizona contains two zones (Zone 6 and Zone 8) as shown in **Figure 1-1**. The short-duration rainfall ratios for those two zones are shown in **Table 1-1**. Use of those ratios will affect the short-duration rainfall depths and rainfall intensities as compared to the values that would be obtained using the ratios in the NOAA Atlas 2. The short-duration rainfall ratio from Arkell and Richards (1986) along with the isopluvial maps and other information from the NOAA Atlas 2 are used to define design rainfall for Arizona.

FIGURE 1-1

SHORT-DURATION RAINFALL RATIO ZONES FOR ARIZONA

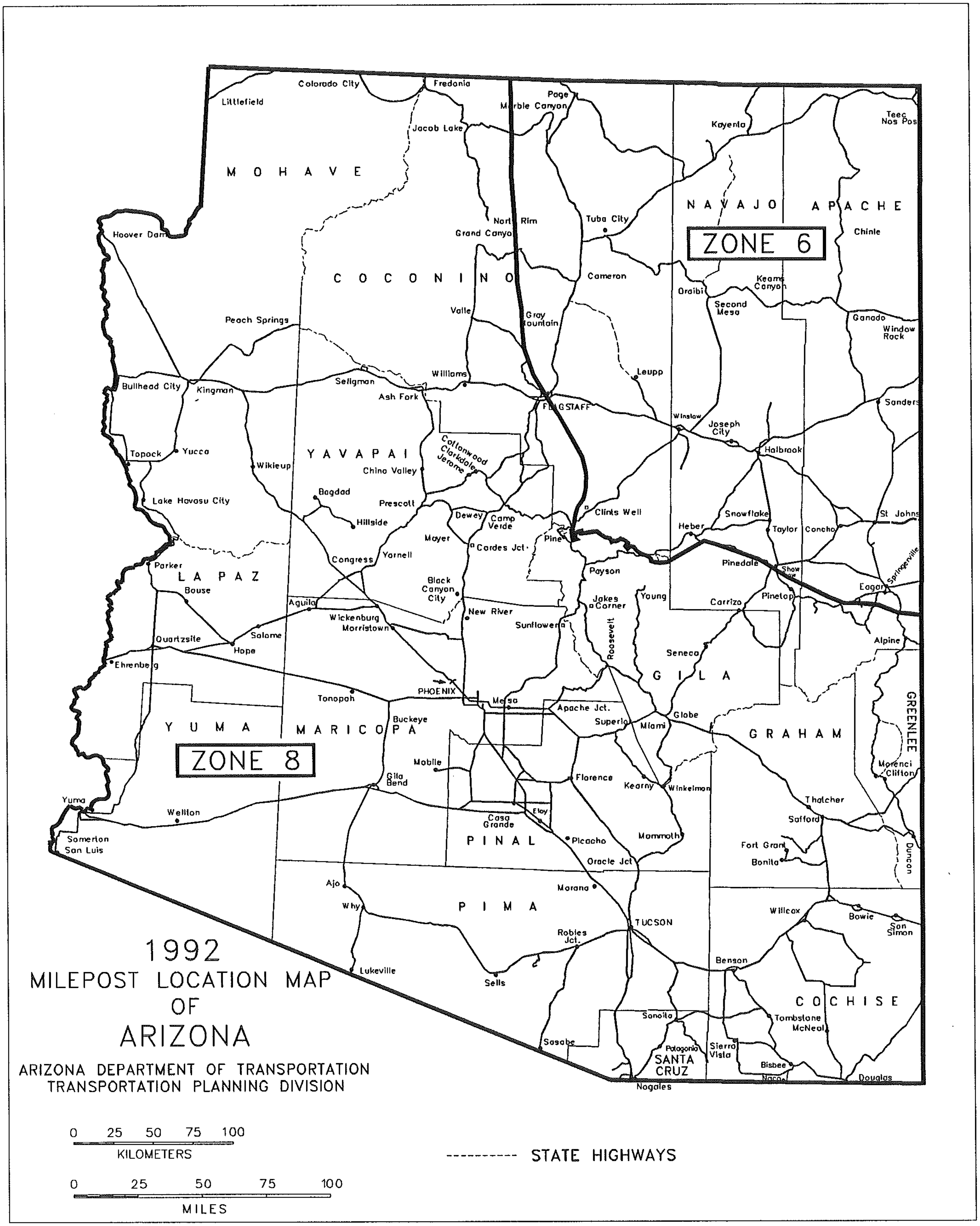


TABLE 1-1

SHORT DURATION RAINFALL RATIOS FOR ARIZONA
(Arkell and Richards, 1986)

	RATIOS TO 1-HOUR RAINFALL DEPTH							
	2-Year Return Period				100-Year Return Period			
	Duration, in minutes				Duration, in minutes			
Zone	**5**	**10**	**15**	**30**	**5**	**10**	**15**	**30**
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)
6	.35	.54	.65	.83	.32	.50	.62	.81
8	.34	.51	.62	.82	.30	.46	.59	.80

A rainfall depth-duration-frequency (D-D-F) table must be developed prior to coding input in the PH record or developing a site-specific I-D-F graph. The D-D-F statistics can be calculated by use of the PREFRE Program (U.S. Bureau of Reclamation, 1988) (with input in English units and the conversion of results to metric units) or by the following procedure and equations:

1. Determine the following point rainfall depth-duration-frequency values for the watershed using the isopluvial maps in **Appendix B:**
   a. 2-year, 6-hour $(P_{2,\,6'})$
   b. 2-year, 24-hour $(P_{2,\,24'})$
   c. 100-year, 6-hour $(P_{100,\,6'})$
   d. 100-year, 24-hour $(P_{100,\,24'})$

   Note: 5" denotes 5 minutes, etc.
   1' denotes 1 hour, etc.

   1. If the watershed is small or if there is little variation in the isopluvial lines for the drainage area, then the rainfall values can be taken from the isopluvial maps at the centroid of the watershed. If the watershed is large enough to indicate significant variation in rainfall depth throughout the watershed, calculate the area weighted rainfall values. Area- weighted rainfall values are calculated by laying a transparent watershed map and grid over each of

the isopluvial maps. The point rainfall values are read at each grid intersection (a minimum of 10) and these are averaged.

2. For watersheds that are to be divided into modeling subbasins and which contain numerous isopluvial lines (nonuniform rainfall characteristics), consideration should be given to developing separate D-D-F tables for each modeling subbasin. Multiple PH records (one for each subbasin) would be used in the HEC-1 model to improve the distribution of rainfall over the watershed.

2. Convert the point rainfall depth-duration-frequency values from inches to millimeters by the following:

$$P_{T,t} \text{ in millimeters} = (25.4)\ P_{T,t} \text{ in inches}$$

3. Compute the following rainfall statistics:

a. 2-year, 1-hour $\quad P_{2,1'} = -0.279 + \dfrac{.942\ (P_{2,6'})^2}{P_{2,24'}}$

b. 100-year, 1-hour $\quad P_{100,1'} = 12.55 + \dfrac{.755\ (P_{100,6'})^2}{P_{100,24'}}$

4. Compute the following rainfall statistics:

a. 2-year, 2-hour $\quad P_{2,2'} = .341\ (P_{2,6'}) + .659\ (P_{2,1'})$

b. 2-year, 3-hour $\quad P_{2,3'} = .569\ (P_{2,6'}) + .431\ (P_{2,1'})$

c. 2-year, 12-hour $\quad P_{2,12'} = .500\ (P_{2,6'}) + .500\ (P_{2,24'})$

d. 100-year, 2-hour $\quad P_{100,2'} = .341\ (P_{100,6'}) + .659\ (P_{100,1'})$

e. 100-year, 3-hour $\quad P_{100,3'} = .569\ (P_{100,6'}) + .431\ (P_{100,1'})$

f. 100-year, 12-hour $\quad P_{100,12'} = .500\ (P_{100,6'}) + .500\ (P_{100,24'})$

Note: 5" denotes 5 minutes, etc.
1' denotes 1 hour, etc.

5. Determine the short-duration rainfall zone, **Figure 1-1**.

6. Determine the 2-year and 100-year short-duration rainfall ratios, **Table 1-1.**

7. Compute the short-duration rainfall statistics according to the following:

		Zone 6	Zone 8
2-yr, 5-min	$P_{2, 5"}$ =	.35 ($P_{2, 1'}$)	.34 ($P_{2, 1'}$)
2-yr, 10-min	$P_{2, 10"}$ =	.54 ($P_{2, 1'}$)	.51 ($P_{2, 1'}$)
2-yr, 15-min	$P_{2, 15"}$ =	.65 ($P_{2, 1'}$)	.62 ($P_{2, 1'}$)
2-yr, 30-min	$P_{2, 30"}$ =	.83 ($P_{2, 1'}$)	.82 ($P_{2, 1'}$)
100-yr, 5-min	$P_{100, 5"}$ =	.32 ($P_{100, 1'}$)	.30 ($P_{100, 1'}$)
100-yr, 10-min	$P_{100, 10"}$ =	.50 ($P_{100, 1'}$)	.46 ($P_{100, 1'}$)
100-yr, 15-min	$P_{100, 15"}$ =	.62 ($P_{100, 1'}$)	.59 ($P_{100, 1'}$)
100-yr, 30-min	$P_{100, 30"}$ =	.81 ($P_{100, 1'}$)	.80 ($P_{100, 1'}$)

8. Compute rainfall statistics for other frequencies (T-year) and other durations (t-min/hour) by the following:

   a. 5-year, t-min/hour $P_{5,t} = .674\ (P_{2,t}) + .278\ (P_{100,t})$

   b. 10-year, t-min/hour $P_{10,t} = .496\ (P_{2,t}) + .449\ (P_{100,t})$

   c. 25-year, t-min/hour $P_{25,t} = .293\ (P_{2,t}) + .669\ (P_{100,t})$

   d. 50-year, t-min/hour $P_{50,t} = .146\ (P_{2,t}) + .835\ (P_{100,t})$

   e. 500-year, t-min/hour $P_{500,t} = -.337\ (P_{2,t}) + 1.381\ (P_{100,t})$

   Note: 5" denotes 5 minutes, etc.
   1' denotes 1 hour, etc.

The values derived from the NOAA Atlas 2 are point rainfall depths. These must be converted to equivalent uniform depth of rainfall for the entire watershed, and this is accomplished with a set of depth-area reduction curves. Use of the PH record with the HEC-1 program will result in automatic adjustment of the point rainfall values that are coded into the PH record. Do <u>not</u> convert the point rainfall depths to equivalent uniform depths of rainfall in the PH record or there will be double reduction of the point rainfall depths using this procedure.

1.2.2 Applications and Limitations

The rainfall statistics that are developed by procedures in this section are dependent upon the information that is provided in the NOAA Atlas 2 (Miller and others, 1973). The potential deficiencies of that information are recognized. However, until a similar, comprehensive and accepted source of rainfall information for Arizona becomes available, the NOAA Atlas 2 will be used for highway drainage studies in Arizona.

The hypothetical distribution is a simplified and idealized representation of the temporal distribution of rainfall. It is intended for use to estimate design discharges for highway drainage facilities. It does not necessarily represent the temporal distribution of any historical storm in Arizona. The use of that distribution for design purposes does provide reasonable assurance that design discharges of specified frequency are produced regardless of the size of the watershed.

For very large watersheds (possibly as large or larger than 1,300 square kilometers (500 square miles)), where the time of concentration (T_C) exceeds 24 hours, a longer duration hypothetical distribution (or other project specific distribution) should be developed and used. Procedures for estimating the watershed time of concentration are contained in Chapter 4 - Unit Hydrographs.

In general, the hypothetical distribution can be used, as input to the HEC-1 program, for highway drainage design purposes in Arizona. Similarly, the two generalized I-D-F graphs (see Chapter 2 - Rational Method) can be used with the Rational Method (within the limitations specified in that section) for most small watersheds in Arizona.

1.3 INSTRUCTIONS

1.3.1 HEC-1 Rainfall Input - PH Record

1. Develop the rainfall depth-duration-frequency (D-D-F) statistics for the desired flood frequency using the D-D-F Worksheet **(Figure 1-2)**, or the PREFRE Program.

2. Code the rainfall input in the PH record:

   a. Field 1, PFREQ

   If the analysis is for flood frequency of 2-, 5-, or 10-year, insert the following value in Field 1:

Flood Frequency	Value of PFREQ in Field 1
2-year	50
5-year	20
10-year	10

For all other flood frequencies, Field 1 is left blank.

   b. Field 2, TRSDA

   Insert the <u>total</u> watershed area (not subbasin area), in square kilometers, in Field 2. For watersheds with non-uniform rainfall characteristics, i.e. those requiring multiple PH records, the <u>total</u> watershed area is to be input to all PH records.

   c. Fields 3 through 10, PNHR(I)

   1) If the total watershed area is less than or equal to 2.5 square kilometers, insert the rainfall depth, in millimeters, for each duration of the selected flood frequency in the appropriate field:

Field	Rainfall Duration
3	5-minute
4	15-minute
5	1-hour
6	2-hour
7	3-hour
8	6-hour

2) If the total watershed area is greater than 2.5 square kilometers, complete Fields 3 through 8, as above, and insert the additional rainfall depths in Fields 9 and 10:

Field	Rainfall Duration
9	12-hour
10	24-hour

1.3.2 Rational Method - Site-Specific I-D-F Graph

This procedure will be used if one of the two generalized I-D-F graphs (see Chapter 2 - Rational Method) is not to be used.

1. Develop the rainfall depth-duration-frequency (D-D-F) statistics for the desired flood frequency or frequencies using the D-D-F Worksheet, **Figure 1-2**, or the PREFRE Program. (When using the PREFRE Program, input is in English units and results must be converted to metric units.)

2. Divide each rainfall depth by its corresponding duration, in hours. Tabulate these rainfall intensities, in millimeters per hour, using the I-D-F Worksheet, **Figure 1-3.**

3. Plot the rainfall intensities for each rainfall frequency versus the rainfall duration, in minutes, on log-log graph paper.

ARIZONA DEPARTMENT OF TRANSPORTATION
HYDROLOGIC DESIGN DATA

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

FIGURE 1-2 Sheet 1 of 4

RAINFALL DEPTH-DURATION-FREQUENCY (D-D-F) WORKSHEET

PART A

Determine rainfall depths from the isopluvial maps **(Appendix B)** and convert inches to millimeters:

	Rainfall depth, in inches	**Rainfall depth, in millimeters**
2-year, 6-hour	$P_{2,6'}$ = ________	$P_{2,6'}$ = ________
2-year, 24-hour	$P_{2,24'}$ = ________	$P_{2,24'}$ = ________
100-year, 6-hour	$P_{100,6'}$ = ________	$P_{100,6'}$ = ________
100-year, 24-hour	$P_{100,24'}$ = ________	$P_{100,24'}$ = ________

Note: $P_{T,t}$ in millimeters = (25.4) $P_{T,t}$ in inches

PART B

Compute the following:

2-year, 1-hour	$-0.279 + \frac{.942\,(P_{2,6'})^2}{(P_{2,24'})} = -0.279 + \frac{.942\,(\quad)^2}{(\quad)}$	$P_{2,1'}$ = ______
100-year, 1-hour	$12.55 + \frac{.755\,(P_{100,6'})^2}{(P_{100,24'})} = 12.55 + \frac{.755\,(\quad)^2}{(\quad)}$	$P_{100,1'}$ = ______
2-year, 2-hour	$.341(P_{2,6'}) + .659(P_{2,1'}) = .341(\quad) + .659(\quad)$	$P_{2,2'}$ = ______
2-year, 3-hour	$.569(P_{2,6'}) + .431(P_{2,1'}) = .569(\quad) + .431(\quad)$	$P_{2,3'}$ = ______
2-year, 12-hour	$.500(P_{2,6'}) + .500(P_{2,24'}) = .500(\quad) + .500(\quad)$	$P_{2,12'}$ = ______
100-year, 2-hour	$.341(P_{100,6'}) + .659(P_{100,1'}) = .341(\quad) + .659(\quad)$	$P_{100,2'}$ = ______
100-year, 3-hour	$.569(P_{100,6'}) + .431(P_{100,1'}) = .569(\quad) + .431(\quad)$	$P_{100,3'}$ = ______
100-year, 12-hour	$.500(P_{100,6'}) + .500(P_{100,24'}) = .500(\quad) + .500(\quad)$	$P_{100,12'}$ = ______

Note: 5" denotes 5 minutes, etc.; 1' denotes 1 hour, etc.

RAINFALL DEPTH-DURATION-FREQUENCY (D-D-F) WORKSHEET
(Continued)

PART C

Determine the short-duration rainfall zone (**Figure 1-1**):

Zone = __________

Determine the short-duration rainfall ratios (**Table 1-1**):

Duration (minutes)	Ratio	
	2-Year	100-Year
5	A = ________	E = __________
10	B = ________	F = __________
15	C = ________	G = __________
30	D = ________	H = __________

Compute the following:

2-year, 5-min	(A) ($P_{2,1'}$) = ( ) ( )	$P_{2,5''}$ = ________
2-year, 10-min	(B) ($P_{2,1'}$) = ( ) ( )	$P_{2,10''}$ = ________
2-year, 15-min	(C) ($P_{2,1'}$) = ( ) ( )	$P_{2,15''}$ = ________
2-year, 30-min	(D) ($P_{2,1'}$) = ( ) ( )	$P_{2,30''}$ = ________
100-year, 5-min	(E) ($P_{100,1'}$) = ( ) ( )	$P_{100,5''}$ = ________
100-year, 10-min	(F) ($P_{100,1'}$) = ( ) ( )	$P_{100,10''}$ = ________
100-year, 15-min	(G) ($P_{100,1'}$) = ( ) ( )	$P_{100,15''}$ = ________
100-year, 30-min	(H) ($P_{100,1'}$) = ( ) ( )	$P_{100,30''}$ = ________

Note: 5" denotes 5 minutes, etc.; 1′ denotes 1 hour, etc.

FIGURE 1-2
RAINFALL DEPTH-DURATION-FREQUENCY (D-D-F) WORKSHEET
(Continued)

PART D

For any flood frequency (T-yr) other than 2-year or 100-year, calculate the rainfall depth for each rainfall duration (*t*) by the following equation:

$$P_{T,t} = (X)(P_{2,t}) + (Y)(P_{100,t})$$

where X and Y for a selected frequency (T-yr) are:

Frequency (T-yr)	X	Y
5-year	.674	.278
10-year	.496	.449
25-year	.293	.669
50-year	.146	.835
500-year	-.337	1.381

Selected frequency (T-yr) = ________ X = ________ Y = ________

5-min	$(X)(P_{2,5''}) + (Y)(P_{100,5''})$ = ( )( ) + ( )( )	$P_{\_\_,5''}$ = ______
10-min	$(X)(P_{2,10''}) + (Y)(P_{100,10''})$ = ( )( ) + ( )( )	$P_{\_\_,10''}$ = ______
15-min	$(X)(P_{2,15''}) + (Y)(P_{100,15''})$ = ( )( ) + ( )( )	$P_{\_\_,15''}$ = ______
30-min	$(X)(P_{2,30''}) + (Y)(P_{100,30''})$ = ( )( ) + ( )( )	$P_{\_\_,30''}$ = ______
1-hour	$(X)(P_{2,1'}) + (Y)(P_{100,1'})$ = ( )( ) + ( )( )	$P_{\_\_,1'}$ = ______
2-hour	$(X)(P_{2,2'}) + (Y)(P_{100,2'})$ = ( )( ) + ( )( )	$P_{\_\_,2'}$ = ______
3-hour	$(X)(P_{2,3'}) + (Y)(P_{100,3'})$ = ( )( ) + ( )( )	$P_{\_\_,3'}$ = ______
6-hour	$(X)(P_{2,6'}) + (Y)(P_{100,6'})$ = ( )( ) + ( )( )	$P_{\_\_,6'}$ = ______
12-hour	$(X)(P_{2,12'}) + (Y)(P_{100,12'})$ = ( )( ) + ( )( )	$P_{\_\_,12'}$ = ______
24-hour	$(X)(P_{2,24'}) + (Y)(P_{100,24'})$ = ( )( ) + ( )( )	$P_{\_\_,24'}$ = ______

Note: 5" denotes 5 minutes, etc.; 1' denotes 1 hour, etc.

FIGURE 1-2
RAINFALL DEPTH-DURATION-FREQUENCY (D-D-F) WORKSHEET
(Continued)

PART E

Tabulate the rainfall Depth-Duration-Frequency statistics below:

Duration	Rainfall depth, in millimeters Frequency, in years						
	2	5	10	25	50	100	500
5-min.							
10-min.*							
15-min.							
30-min.*							
1-hour							
2-hour							
3-hour							
6-hour							
12-hour							
24-hour							

* - Note: 10-min. and 30-min. values are not coded into the PH record.
5" denotes 5 minutes, etc.; 1′ denotes 1 hour, etc.

ARIZONA DEPARTMENT OF TRANSPORTATION
HYDROLOGIC DESIGN DATA

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

FIGURE 1-3
RAINFALL INTENSITY-DURATION-FREQUENCY (I-D-F) WORKSHEET

Divide each rainfall depth from the D-D-F Worksheet (**Figure 1-2 Part E**) by each corresponding duration, in hours, and tabulate below:

Duration	Rainfall intensity, in millimeters/hour Frequency, in years						
	2	5	10	25	50	100	500
5-min.							
10-min.							
15-min.							
30-min.							
1-hour							
2-hour							
3-hour							
6-hour							
12-hour							
24-hour							

Note: 5" denotes 5 minutes, etc.; 1' denotes 1 hour, etc.

EXAMPLE 1-1

RAINFALL DEPTH-DURATION-FREQUENCY (D-D-F) TABLE

Problem:

Develop a Rainfall Depth-Duration-Frequency (D-D-F) table for Bisbee, Arizona.

Solution:

The D-D-F Worksheets (**Figure 1-2, Parts A - E**) are used as follows:

Project No. Example 1-1 TRACS No. ______
Project Name D-D-F Table for Bisbee, Az Date ______
Location/Station ______
Designer ______ Checker ______

EXAMPLE 1-1

RAINFALL DEPTH-DURATION-FREQUENCY (D-D-F) WORKSHEET

PART A

Determine rainfall depths from the isopluvial maps **(Appendix B)**:

	Rainfall depth, in inches	Rainfall depth, in millimeters
2-year, 6-hour	$P_{2,6'}$ = 1.62	$P_{2,6'}$ = 41.1
2-year, 24-hour	$P_{2,24'}$ = 1.99	$P_{2,24'}$ = 50.5
100-year, 6-hour	$P_{100,6'}$ = 3.56	$P_{100,6'}$ = 90.4
100-year, 24-hour	$P_{100,24'}$ = 4.25	$P_{100,24'}$ = 108.0

Note: $P_{T,t}$ in millimeters = (25.4) $P_{T,t}$ in inches

PART B

Compute the following:

2-year, 1-hour	$-0.279 + \frac{.942\,(P_{2,6'})^2}{(P_{2,24'})} = -0.279 + \frac{.942\,(41.1)^2}{(50.5)}$	$P_{2,1'}$ = 31.2
100-year, 1-hour	$12.55 + \frac{.755\,(P_{100,6'})^2}{(P_{100,24'})} = 12.55 + \frac{.755\,(90.4)^2}{(108.0)}$	$P_{100,1'}$ = 69.7
2-year, 2-hour	$.341(P_{2,6'}) + .659(P_{2,1'}) = .341(41.1) + .659(31.2)$	$P_{2,2'}$ = 34.6
2-year, 3-hour	$.569(P_{2,6'}) + .431(P_{2,1'}) = .569(41.1) + .431(31.2)$	$P_{2,3'}$ = 36.8
2-year, 12-hour	$.500(P_{2,6'}) + .500(P_{2,24'}) = .500(41.1) + .500(50.5)$	$P_{2,12'}$ = 45.8
100-year, 2-hour	$.341(P_{100,6'}) + .659(P_{100,1'}) = .341(90.4) + .659(69.7)$	$P_{100,2'}$ = 76.8
100-year, 3-hour	$.569(P_{100,6'}) + .431(P_{100,1'}) = .569(90.4) + .431(69.7)$	$P_{100,3'}$ = 81.5
100-year, 12-hour	$.500(P_{100,6'}) + .500(P_{100,24'}) = .500(90.4) + .500(108.0)$	$P_{100,12'}$ = 99.2

Note: 5" denotes 5 minutes, etc.; 1′ denotes 1 hour, etc.

FIGURE 1-2 Sheet 1 of 4

RAINFALL DEPTH-DURATION-FREQUENCY (D-D-F) WORKSHEET

PART C

Determine the short-duration rainfall zone (**Figure 1-1**):

Zone = 8

Determine the short-duration rainfall ratios (**Table 1-1**):

Duration (minutes)	Ratio	
	2-Year	100-Year
5	A = 0.34	E = 0.30
10	B = 0.51	F = 0.46
15	C = 0.62	G = 0.59
30	D = 0.82	H = 0.80

Compute the following:

2-year, 5-min	(A) ($P_{2,1'}$) = (0.34)(31.2)	$P_{2,5''}$	= 10.6
2-year, 10-min	(B) ($P_{2,1'}$) = (0.51)(31.2)	$P_{2,10''}$	= 15.9
2-year, 15-min	(C) ($P_{2,1'}$) = (0.62)(31.2)	$P_{2,15''}$	= 19.3
2-year, 30-min	(D) ($P_{2,1'}$) = (0.82)(31.2)	$P_{2,30''}$	= 25.6
100-year, 5-min	(E) ($P_{100,1'}$) = (0.30)(69.7)	$P_{100,5''}$	= 20.9
100-year, 10-min	(F) ($P_{100,1'}$) = (0.46)(69.7)	$P_{100,10''}$	= 32.1
100-year, 15-min	(G) ($P_{100,1'}$) = (0.59)(69.7)	$P_{100,15''}$	= 41.1
100-year, 30-min	(H) ($P_{100,1'}$) = (0.80)(69.7)	$P_{100,30''}$	= 55.8

Note: 5" denotes 5 minutes, etc.; 1′ denotes 1 hour, etc.

FIGURE 1-2 Sheet 2 of 4

RAINFALL DEPTH-DURATION-FREQUENCY (D-D-F) WORKSHEET

PART D

For any flood frequency (T-yr) other than 2-year or 100-year, calculate the rainfall depth for each rainfall duration (t) by the following equation:

$$P_{T,t} = (X)(P_{2,t}) + (Y)(P_{100,t})$$

where X and Y for a selected frequency (T-yr) are:

Frequency (T-yr)	X	Y
5-year	.674	.278
10-year	.496	.449
25-year	.293	.669
50-year	.146	.835
500-year	-.337	1.381

Selected frequency (T-yr) = 5-year X = 0.674 Y = 0.278

5-min	$(X)(P_{2,5''}) + (Y)(P_{100,5''})$ = (0.674)(10.6) + (0.278)(20.9)	$P_{5,5''}$	= 13.0
10-min	$(X)(P_{2,10''}) + (Y)(P_{100,10''})$ = (0.674)(15.9) + (0.278)(32.1)	$P_{5,10''}$	= 19.6
15-min	$(X)(P_{2,15''}) + (Y)(P_{100,15''})$ = (0.674)(19.3) + (0.278)(41.1)	$P_{5,15''}$	= 24.4
30-min	$(X)(P_{2,30''}) + (Y)(P_{100,30''})$ = (0.674)(25.6) + (0.278)(55.8)	$P_{5,30''}$	= 32.8
1-hour	$(X)(P_{2,1'}) + (Y)(P_{100,1'})$ = (0.674)(31.2) + (0.278)(69.7)	$P_{5,1'}$	= 40.4
2-hour	$(X)(P_{2,2'}) + (Y)(P_{100,2'})$ = (0.674)(34.6) + (0.278)(76.8)	$P_{5,2'}$	= 44.7
3-hour	$(X)(P_{2,3'}) + (Y)(P_{100,3'})$ = (0.674)(36.8) + (0.278)(81.5)	$P_{5,3'}$	= 47.5
6-hour	$(X)(P_{2,6'}) + (Y)(P_{100,6'})$ = (0.674)(41.1) + (0.278)(90.4)	$P_{5,6'}$	= 52.8
12-hour	$(X)(P_{2,12'}) + (Y)(P_{100,12'})$ = (0.674)(45.8) + (0.278)(99.2)	$P_{5,12'}$	= 58.4
24-hour	$(X)(P_{2,24'}) + (Y)(P_{100,24'})$ = (0.674)(50.5) + (0.278)(108.0)	$P_{5,24'}$	= 64.1

Note: 5" denotes 5 minutes, etc.; 1′ denotes 1 hour, etc.

FIGURE 1-2 Sheet 3 of 4

RAINFALL DEPTH-DURATION-FREQUENCY (D-D-F) WORKSHEET

PART D

For any flood frequency (T-yr) other than 2-year or 100-year, calculate the rainfall depth for each rainfall duration (*t*) by the following equation:

$$P_{T,t} = (X)(P_{2,t}) + (Y)(P_{100,t})$$

where X and Y for a selected frequency (T-yr) are:

Frequency (T-yr)	X	Y
5-year	.674	.278
10-year	.496	.449
25-year	.293	.669
50-year	.146	.835
500-year	-.337	1.381

Selected frequency (T-yr) = 10-year X = 0.496 Y = 0.449

5-min	$(X)(P_{2,5''}) + (Y)(P_{100,5''})$ = (0.496)(10.6) + (0.449)(20.9)	$P_{10,5''}$	= 14.6
10-min	$(X)(P_{2,10''}) + (Y)(P_{100,10''})$ = (0.496)(15.9) + (0.449)(32.1)	$P_{10,10''}$	= 22.3
15-min	$(X)(P_{2,15''}) + (Y)(P_{100,15''})$ = (0.496)(19.3) + (0.449)(41.1)	$P_{10,15''}$	= 28.0
30-min	$(X)(P_{2,30''}) + (Y)(P_{100,30''})$ = (0.496)(25.6) + (0.449)(55.8)	$P_{10,30''}$	= 37.8
1-hour	$(X)(P_{2,1'}) + (Y)(P_{100,1'})$ = (0.496)(31.2) + (0.449)(69.7)	$P_{10,1'}$	= 46.8
2-hour	$(X)(P_{2,2'}) + (Y)(P_{100,2'})$ = (0.496)(34.6) + (0.449)(76.8)	$P_{10,2'}$	= 51.6
3-hour	$(X)(P_{2,3'}) + (Y)(P_{100,3'})$ = (0.496)(36.8) + (0.449)(81.5)	$P_{10,3'}$	= 54.8
6-hour	$(X)(P_{2,6'}) + (Y)(P_{100,6'})$ = (0.496)(41.1) + (0.449)(90.4)	$P_{10,6'}$	= 61.0
12-hour	$(X)(P_{2,12'}) + (Y)(P_{100,12'})$ = (0.496)(45.8) + (0.449)(99.2)	$P_{10,12'}$	= 67.3
24-hour	$(X)(P_{2,24'}) + (Y)(P_{100,24'})$ = (0.496)(50.5) + (0.449)(108.0)	$P_{10,24'}$	= 73.5

Note: 5" denotes 5 minutes, etc.; 1' denotes 1 hour, etc.

FIGURE 1-2 Sheet 3 of 4

RAINFALL DEPTH-DURATION-FREQUENCY (D-D-F) WORKSHEET

PART D

For any flood frequency (T-yr) other than 2-year or 100-year, calculate the rainfall depth for each rainfall duration (*t*) by the following equation:

$$P_{T,t} = (X)(P_{2,t}) + (Y)(P_{100,t})$$

where X and Y for a selected frequency (T-yr) are:

Frequency (T-yr)	X	Y
5-year	.674	.278
10-year	.496	.449
25-year	.293	.669
50-year	.146	.835
500-year	-.337	1.381

Selected frequency (T-yr) = 25-year X = 0.293 Y = 0.669

5-min	$(X)(P_{2,5''}) + (Y)(P_{100,5''})$	= (0.293)(10.6) + (0.669)(20.9)	$P_{25,5''}$ = 17.1
10-min	$(X)(P_{2,10''}) + (Y)(P_{100,10''})$	= (0.293)(15.9) + (0.669)(32.1)	$P_{25,10''}$ = 26.1
15-min	$(X)(P_{2,15''}) + (Y)(P_{100,15''})$	= (0.293)(19.3) + (0.669)(41.1)	$P_{25,15''}$ = 33.2
30-min	$(X)(P_{2,30''}) + (Y)(P_{100,30''})$	= (0.293)(25.6) + (0.669)(55.8)	$P_{25,30''}$ = 44.8
1-hour	$(X)(P_{2,1'}) + (Y)(P_{100,1'})$	= (0.293)(31.2) + (0.669)(69.7)	$P_{25,1'}$ = 55.8
2-hour	$(X)(P_{2,2'}) + (Y)(P_{100,2'})$	= (0.293)(34.6) + (0.669)(76.8)	$P_{25,2'}$ = 61.5
3-hour	$(X)(P_{2,3'}) + (Y)(P_{100,3'})$	= (0.293)(36.8) + (0.669)(81.5)	$P_{25,3'}$ = 65.3
6-hour	$(X)(P_{2,6'}) + (Y)(P_{100,6'})$	= (0.293)(41.1) + (0.669)(90.4)	$P_{25,6'}$ = 72.5
12-hour	$(X)(P_{2,12'}) + (Y)(P_{100,12'})$	= (0.293)(45.8) + (0.669)(99.2)	$P_{25,12'}$ = 79.8
24-hour	$(X)(P_{2,24'}) + (Y)(P_{100,24'})$	= (0.293)(50.5) + (0.669)(108.0)	$P_{25,24'}$ = 87.0

Note: 5" denotes 5 minutes, etc.; 1' denotes 1 hour, etc.

FIGURE 1-2 Sheet 3 of 4

RAINFALL DEPTH-DURATION-FREQUENCY (D-D-F) WORKSHEET

PART D

For any flood frequency (T-yr) other than 2-year or 100-year, calculate the rainfall depth for each rainfall duration (*t*) by the following equation:

$$P_{T,t} = (X)(P_{2,t}) + (Y)(P_{100,t})$$

where X and Y for a selected frequency (T-yr) are:

Frequency (T-yr)	X	Y
5-year	.674	.278
10-year	.496	.449
25-year	.293	.669
50-year	.146	.835
500-year	-.337	1.381

Selected frequency (T-yr) = 50-year X = 0.146 Y = 0.835

5-min	$(X)(P_{2,5"}) + (Y)(P_{100,5"})$	= (0.146)(10.6) + (0.835)(20.9)	$P_{50,5"}$ = 19.0
10-min	$(X)(P_{2,10"}) + (Y)(P_{100,10"})$	= (0.146)(15.9) + (0.835)(32.1)	$P_{50,10"}$ = 29.1
15-min	$(X)(P_{2,15"}) + (Y)(P_{100,15"})$	= (0.146)(19.3) + (0.835)(41.1)	$P_{50,15"}$ = 37.1
30-min	$(X)(P_{2,30"}) + (Y)(P_{100,30"})$	= (0.146)(25.6) + (0.835)(55.8)	$P_{50,30"}$ = 50.3
1-hour	$(X)(P_{2,1'}) + (Y)(P_{100,1'})$	= (0.146)(31.2) + (0.835)(69.7)	$P_{50,1'}$ = 62.8
2-hour	$(X)(P_{2,2'}) + (Y)(P_{100,2'})$	= (0.146)(34.6) + (0.835)(76.8)	$P_{50,2'}$ = 69.2
3-hour	$(X)(P_{2,3'}) + (Y)(P_{100,3'})$	= (0.146)(36.8) + (0.835)(81.5)	$P_{50,3'}$ = 73.4
6-hour	$(X)(P_{2,6'}) + (Y)(P_{100,6'})$	= (0.146)(41.1) + (0.835)(90.4)	$P_{50,6'}$ = 81.5
12-hour	$(X)(P_{2,12'}) + (Y)(P_{100,12'})$	= (0.146)(45.8) + (0.835)(99.2)	$P_{50,12'}$ = 89.5
24-hour	$(X)(P_{2,24'}) + (Y)(P_{100,24'})$	= (0.146)(50.5) + (0.835)(108.0)	$P_{50,24'}$ = 97.6

Note: 5" denotes 5 minutes, etc.; 1' denotes 1 hour, etc.

FIGURE 1-2 Sheet 3 of 4

RAINFALL DEPTH-DURATION-FREQUENCY (D-D-F) WORKSHEET

PART D

For any flood frequency (T-yr) other than 2-year or 100-year, calculate the rainfall depth for each rainfall duration (*t*) by the following equation:

$$P_{T,t} = (X)(P_{2,t}) + (Y)(P_{100,t})$$

where X and Y for a selected frequency (T-yr) are:

Frequency (T-yr)	X	Y
5-year	.674	.278
10-year	.496	.449
25-year	.293	.669
50-year	.146	.835
500-year	-.337	1.381

Selected frequency (T-yr) = 500-year X = -0.337 Y = 1.381

5-min	$(X)(P_{2,5''}) + (Y)(P_{100,5''})$ = (-0.337)(10.6) + (1.381)(20.9)	$P_{500,5''}$	= 25.3
10-min	$(X)(P_{2,10''}) + (Y)(P_{100,10''})$ = (-0.337)(15.9) + (1.381)(32.1)	$P_{500,10''}$	= 39.0
15-min	$(X)(P_{2,15''}) + (Y)(P_{100,15''})$ = (-0.337)(19.3) + (1.381)(41.1)	$P_{500,15''}$	= 50.3
30-min	$(X)(P_{2,30''}) + (Y)(P_{100,30''})$ = (-0.337)(25.6) + (1.381)(55.8)	$P_{500,30''}$	= 68.4
1-hour	$(X)(P_{2,1'}) + (Y)(P_{100,1'})$ = (-0.337)(31.2) + (1.381)(69.7)	$P_{500,1'}$	= 85.7
2-hour	$(X)(P_{2,2'}) + (Y)(P_{100,2'})$ = (-0.337)(34.6) + (1.381)(76.8)	$P_{500,2'}$	= 94.4
3-hour	$(X)(P_{2,3'}) + (Y)(P_{100,3'})$ = (-0.337)(36.8) + (1.381)(81.5)	$P_{500,3'}$	= 100.1
6-hour	$(X)(P_{2,6'}) + (Y)(P_{100,6'})$ = (-0.337)(41.1) + (1.381)(90.4)	$P_{500,6'}$	= 111.0
12-hour	$(X)(P_{2,12'}) + (Y)(P_{100,12'})$ = (-0.337)(45.8) + (1.381)(99.2)	$P_{500,12'}$	= 121.6
24-hour	$(X)(P_{2,24'}) + (Y)(P_{100,24'})$ = (-0.337)(50.5) + (1.381)(108.0)	$P_{500,24'}$	= 132.1

Note: 5" denotes 5 minutes, etc.; 1′ denotes 1 hour, etc.

FIGURE 1-2 Sheet 3 of 4

RAINFALL DEPTH-DURATION-FREQUENCY (D-D-F) WORKSHEET

PART E

Tabulate the rainfall Depth-Duration-Frequency statistics below:

Duration	Rainfall depth, in millimeters						
	Frequency, In years						
	2	5	10	25	50	100	500
5-min.	10.6	13.0	14.6	17.1	19.0	20.9	25.3
10-min.*	15.9	19.6	22.3	26.1	29.1	32.1	39.0
15-min.	19.3	24.4	28.0	33.2	37.1	41.1	50.3
30-min.*	25.6	32.8	37.8	44.8	50.3	55.8	68.4
1-hour	31.2	40.4	46.8	55.8	62.8	69.7	85.7
2-hour	34.6	44.7	51.6	61.5	69.2	76.8	94.4
3-hour	36.8	47.5	54.8	65.3	73.4	81.5	100.1
6-hour	41.1	52.8	61.0	72.5	81.5	90.4	111.0
12-hour	45.8	58.4	67.3	79.8	89.5	99.2	121.6
24-hour	50.5	64.1	73.5	87.0	97.6	108.0	132.1

* - Note: 10-min. and 30-min. values are not coded into the PH record.
5" denotes 5 minutes, etc.; 1' denotes 1 hour, etc.

FIGURE 1-2 Sheet 4 of 4

EXAMPLE 1-2
PH RECORD CODING

Problem:

Code a PH record for a watershed at Bisbee, Arizona for various flood frequencies and watershed sizes.

Solution:

The D-D-F table of the required rainfall depth-duration-frequency (D-D-F) statistics is first prepared (See **Example 1-1**).

a. For a 100-yr, 6-hr flood and 2.0 square kilometer watershed:

Field

	1	2	3	4	5	6	7	8	9	10
PH		1.94	20.9	41.1	69.7	76.8	81.5	90.4		

b. For a 5-yr, 6-hr flood and 2.0 square kilometer watershed:

Field

	1	2	3	4	5	6	7	8	9	10
PH	20	1.94	13.0	24.4	40.4	44.7	47.5	52.8		

c. For a 50-yr, 6-hr flood and a 47 square kilometer watershed:

Field

	1	2	3	4	5	6	7	8	9	10
PH		47	19.0	37.1	62.8	69.2	73.4	81.5	89.5	97.6

EXAMPLE 1-3
RAINFALL INTENSITY-DURATION-FREQUENCY (I-D-F) TABLE

Problem:

Develop a site-specific Intensity-Duration-Frequency (I-D-F) graph for Bisbee, Arizona.

Solution:

The D-D-F table is first produced (See **Example 1-1**). Then the I-D-F Worksheet (**Figure 1-4**) is used. The rainfall intensities, in inches per hour, are plotted against corresponding rainfall durations, in hours, on log-log paper.

ARIZONA DEPARTMENT OF TRANSPORTATION
HYDROLOGIC DESIGN DATA

Project No. EXAMPLE 1-3 TRACS No. ______
Project Name I-D-F GRAPH Date ______
Location/Station BISBEE, ARIZONA
Designer ______ Checker ______

EXAMPLE 1-3

RAINFALL INTENSITY-DURATION-FREQUENCY (I-D-F) WORKSHEET

Divide each rainfall depth from the D-D-F Worksheet **(Figure 1-3 - Part E)**, by each corresponding duration, in hours, and tabulate below:

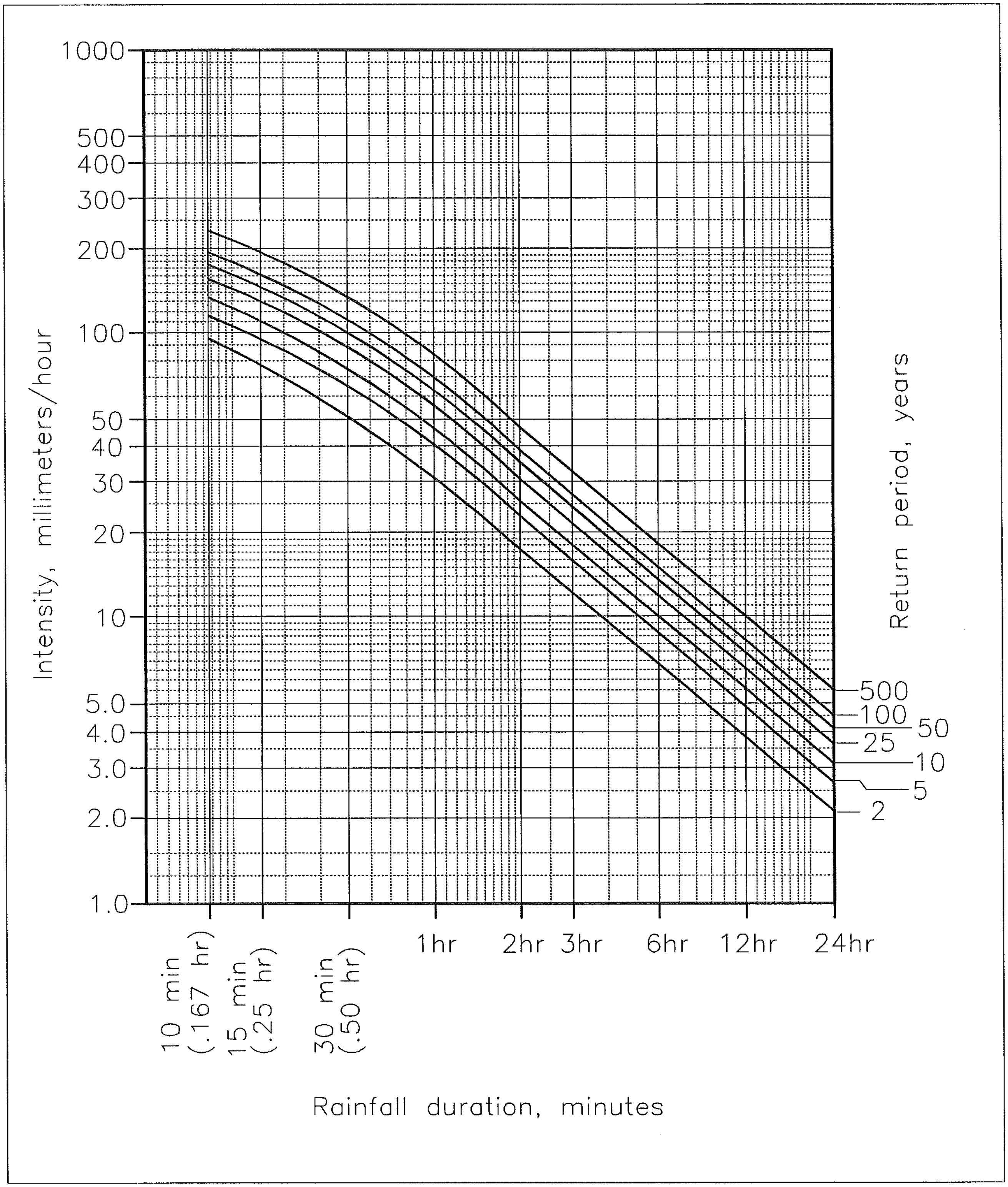
Duration	Rainfall intensity, in millimeters/hour						
	Frequency, in years						
	2	5	10	25	50	100	500
5-min.	127.2	156.0	175.2	205.2	228.0	250.8	303.6
10-min.	95.4	117.6	133.8	156.6	174.6	192.6	234.0
15-min.	77.2	97.6	112.0	132.8	148.4	164.4	201.2
30-min.	51.2	65.6	75.6	89.6	100.6	111.6	136.8
1-hour	31.2	40.4	46.8	55.8	62.8	69.7	85.7
2-hour	17.3	22.4	25.8	30.8	34.6	38.4	47.2
3-hour	12.3	15.8	18.3	21.8	24.5	27.2	33.4
6-hour	6.9	8.8	10.2	12.1	13.6	15.1	18.5
12-hour	3.8	4.9	5.6	6.7	7.5	8.3	10.1
24-hour	2.1	2.7	3.1	3.6	4.1	4.5	5.5

* - Note: 10-min. and 30-min. values are not coded into the PH record.
5" denotes 5 minutes, etc.; 1' denotes 1 hour, etc.

FIGURE 1-3

EXAMPLE 1-3

RAINFALL INTENSITY-DURATION-FREQUENCY
SITE SPECIFIC I-D-F GRAPH FOR BISBEE, ARIZONA



CHAPTER 2
RATIONAL METHOD

2.1 INTRODUCTION

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

Three basic assumptions of the Rational Method are:

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

2.1.1 General Discussion

The Rational Method, as presented herein, can be used to estimate peak discharges, the runoff hydrograph shape, and runoff volume for small, uniform drainage areas that are not larger than 65 hectares (160 acres) in size. The method is usually used to size drainage structures for the peak discharge of a selected return period. An extension of the basic method is provided to estimate the shape of the runoff hydrograph if it is necessary to design retention/detention facilities and/or to design drainage facilities that will require routing of the runoff hydrograph through the structure.

The Rational Method is based on the equation: $Q = C\, i\, A / 363$ (2-1)

where
- Q = the peak discharge, in m^3/s, of selected return period,
- C = the runoff coefficient,
- i = the average rainfall intensity, in millimeters/hr, of calculated rainfall duration for the selected rainfall return period, and
- A = the contributing drainage area, in hectares.

2.2 PROCEDURE

2.2.1 General Considerations

1. Depending on the intended application, 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 T_c and care must be exercised in obtaining the most appropriate estimate of T_c.

4. Both C and the rainfall intensity (i) will vary if peak discharges for different flood return periods are desired.

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

2.2.2 Applications and Limitations

1. The total drainage area must be less than or equal to 65 hectares (160 acres).

2. T_c shall not exceed 60 minutes.

3. The land-use of the contributing area must be fairly consistent over the entire area; that is, the area should not consist of a large percentage of two or more land-uses, such as 50 percent commercial and 50 percent undeveloped. This will lead to inconsistent estimates of T_c (and therefore i) and errors in selecting the most appropriate C coefficient.

4. 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.

5. Drainage areas that do not meet the above conditions will require the use of an appropriate rainfall-runoff model (the HEC-1 Program) to estimate flood discharges.

2.2.3 Estimation of Area (A)

An adequate topographic map of the drainage area and surrounding land is needed to define the drainage boundary and to estimate the area (A), in hectares. The map should be supplemented with aerial photographs, if available, especially if the area is developed. If the area is presently undeveloped but is to undergo development, then the land development plan and maps should be obtained because these may indicate a change in the drainage boundary due to road construction or land grade changes. If development plans are not available, then land-use should be based on current zoning of the area.

The delineation of the drainage boundary needs to 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 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, etc. resulting in a larger contributing area. Floods on alluvial fans (active and inactive) 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.

2.2.4 Estimation of Rainfall Intensity (i)

The intensity (i) in Equation 2-1 is the average rainfall intensity in millimeters/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 according to a design criteria or standard for the intended application. The rainfall intensity (i) is obtained from an intensity-duration-frequency (I-D-F) graph. Two methods can be used for obtaining I-D-F information: 1) two generalized I-D-F graphs are provided that can be used for any site in Arizona, and 2) a site-specific I-D-F graph can be developed, if desired. The two generalized I-D-F graphs are shown in **Figure 2-1** for Zone 6, and **Figure 2-2** for Zone 8, respectively. The delineation of the two rainfall zones for Arizona is shown in **Figure 1-1** of Chapter 1 - Rainfall. Procedures for developing a site-specific I-D-F graph are described in Chapter 1.

The intensity (i) in Equation 2-1 is the average rainfall intensity for rainfall of a selected return period from an I-D-F graph for a rainfall duration that is equal to the time of concentration (T_c) as calculated according to the procedure described below. A minimum rainfall duration of 10 minutes is to be used if the calculated T_c is less than 10 minutes. The Rational Method should not be used if the calculated T_c is greater than 60 minutes.

2.2.5 Estimation of Time of Concentration (T_c)

Time of concentration (T_c) is to be calculated by Equation 2-2:

$$T_C = 18.3\ L^{0.5}\ K_b^{0.52}\ S^{-0.31}\ i^{-0.38} \tag{2-2}$$

Note: Reference Papadakis and Kazan, 1987.

where

- T_c = the time of concentration, in hours,
- L = the length of the longest flow path, in kilometers,
- K_b = the watershed resistance coefficient,
- S = the slope of the longest flow path, in meters/kilometer, and
- i = the average rainfall intensity, in millimeters/hr, for a duration of rainfall equal to T_c (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.

FIGURE 2-1
GENERALIZED I-D-F GRAPH FOR ZONE 6 OF ARIZONA

Example: For a selected 10-year return period, P_1 = 50 millimeters. T_c is calculated as 20 minutes. Therefore, (i) = 106 mm/hr.

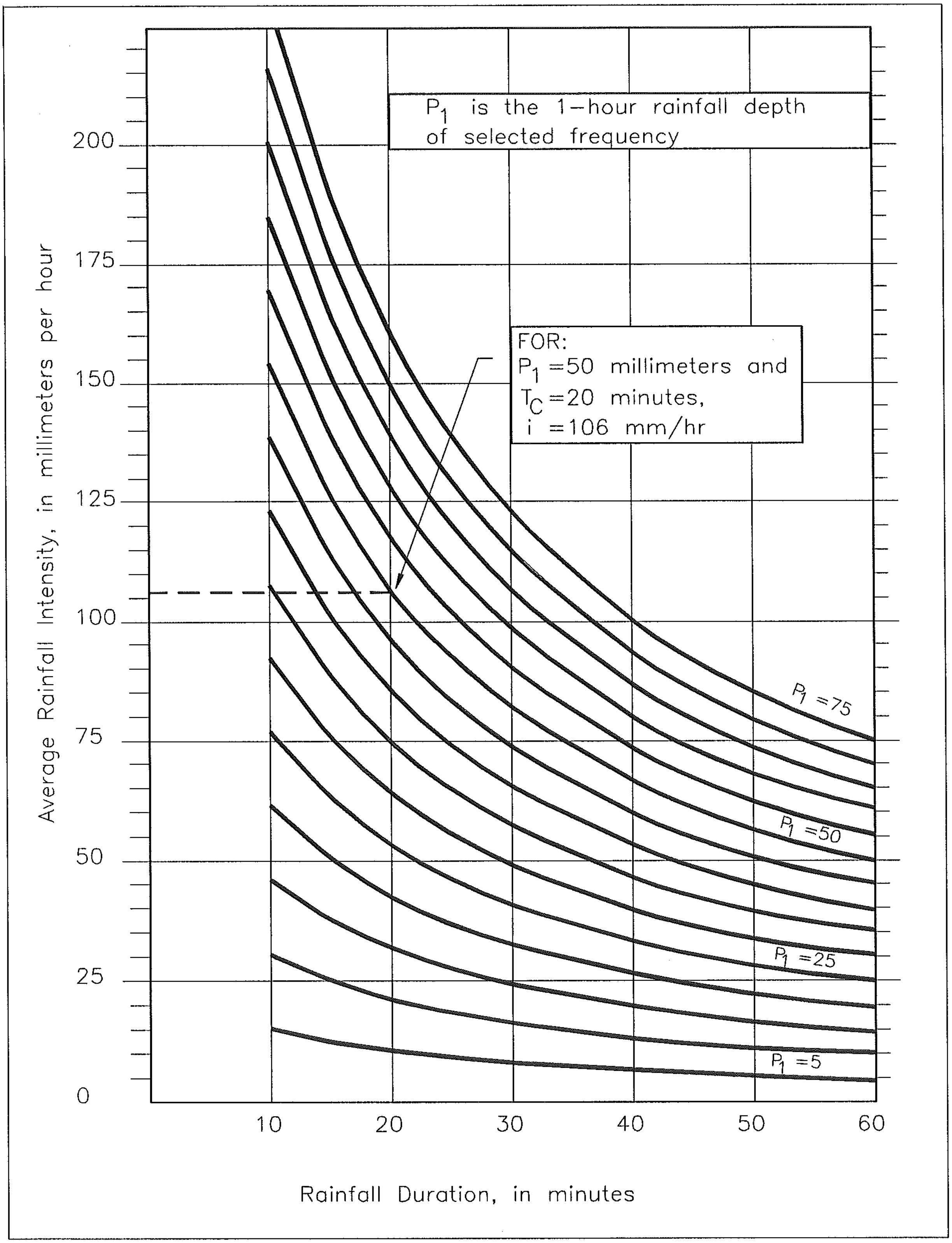
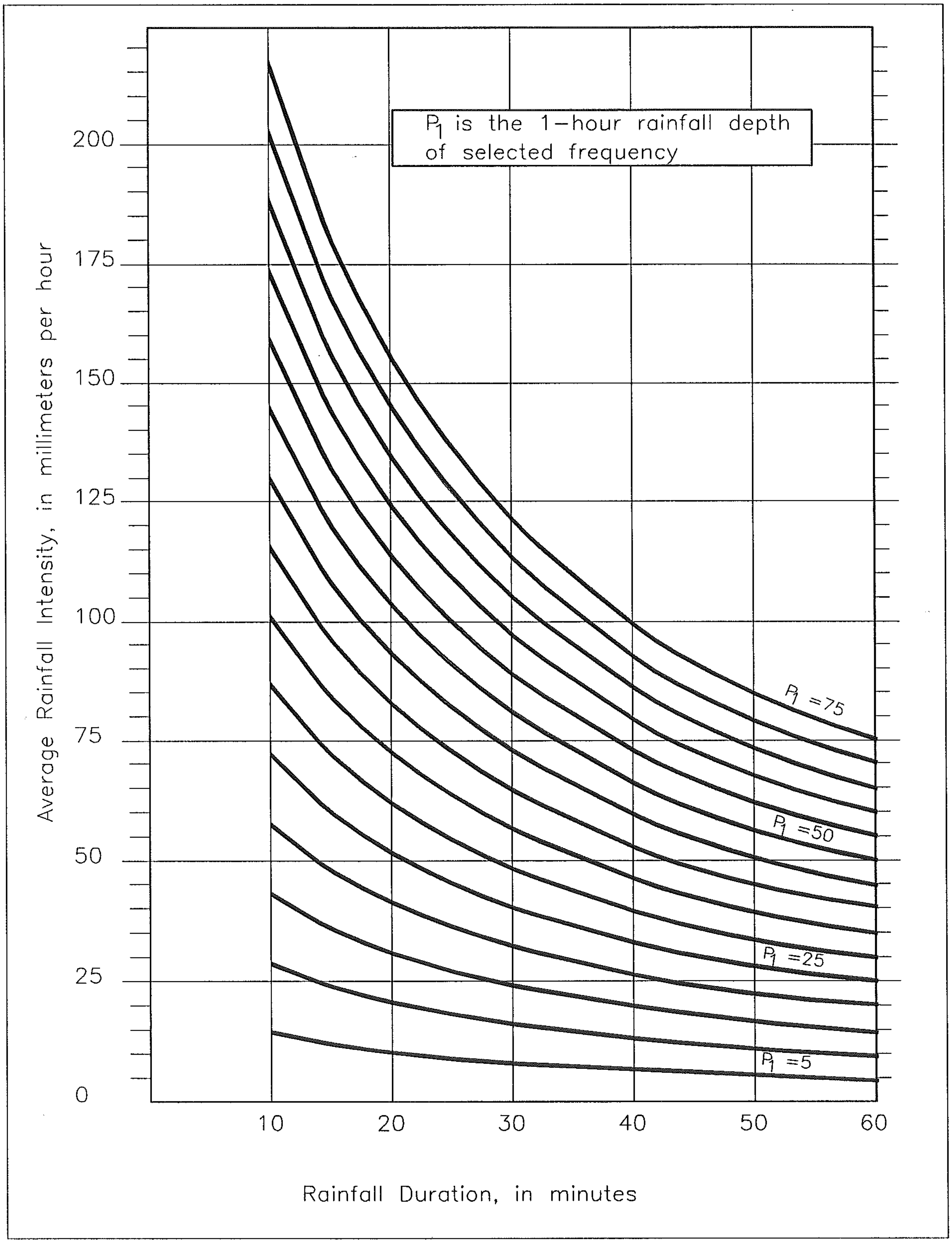


FIGURE 2-2
GENERALIZED I-D-F GRAPH FOR ZONE 8 OF ARIZONA



The slope (S), in meters/kilometer, will be calculated by one of two 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, in meters, along L, and
L = as defined in Equation 2-2.

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:

$$S = 1,000 \left(\frac{d}{j}\right)^2 \tag{2-4}$$

where $d = 1,000 \times L$

$$j = \sum \left(\frac{d_i{}^3}{H_i}\right)^{1/2}$$

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

and d_i = an incremental change in length, in <u>meters</u>, along the longest flowpath and
H_i = an incremental change in elevation, in meters, for each length segment, d_i .

The resistance coefficient (K_b) is selected from **Table 2-1**. Use of **Table 2-1** requires a classification as to the 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	Overland Flow Only
Mountain, with forest and dense ground cover (overland slopes - 50% or greater)	0.15	0.30
Mountain, with rough rock and boulder cover (overland slopes - 50% or greater)	0.12	0.25
Foothills (overland slopes - 10% to 50%)	0.10	0.20
Alluvial fans, Pediments and Rangeland (overland slopes - 10% or less)	0.05	0.10
Irrigated Pasture [a]	----	0.20
Tilled Agricultural Fields [a]	----	0.08
Urban		
Residential, L is less than 300 meters [b]	0.04	----
Residential, L is greater than 300 meters [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 the T_c equation. Streets serve as drainage network.

The solution of 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 trial-and-error 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 10 percent should be acceptable).

2.2.6 Selection of Runoff Coefficient (C)

The runoff coefficient (C) is selected from **Figure 2-3** through **Figure 2-8** depending on the classification of the nature of the watershed. **Figure 2-3** is the C graph to be used for urbanized (developed) watersheds. Select the appropriate curve in **Figure 2-3** 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 pervious area. (Refer to Chapter 3 - Rainfall Losses, 3.1.1 and Table 3-3, for discussion of effective impervious areas.) **Figure 2-4** through **Figure 2-8** are to be used for undeveloped (natural) watersheds in Arizona, and 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, Soil Conservation Service (SCS), 1972 are:

HSG	Definition
A	Soils having high infiltration rates even when thoroughly wetted and consisting chiefly of deep, well to excessively drained sands and gravels. These soils have a high rate of water transmission.
B	Soils having moderate infiltration rates when thoroughly wetted and consisting chiefly of moderately deep to deep, moderately well to well drained soils with moderately fine to moderately coarse textures. These soils have a moderate rate of water transmission.

<u>HSG</u>	<u>Definition</u>
C	Soils having slow infiltration rates when thoroughly wetted and consisting chiefly of soils with a layer that impedes downward movement of water, or soils with moderately fine to fine texture. These soils have a slow rate of water transmission.
D	Soils having very slow infiltration rates when thoroughly wetted and consisting chiefly of clay soils with a high swelling potential, soils with a permanent high water table, soils with a claypan at or near the surface, and shallow soils over nearly impervious material. These soils have a very slow rate of water transmission.

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 (see Appendix C).

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

It may be required to select the appropriate C value for existing conditions and another C value for anticipated future conditions, if the watershed is undergoing development. Estimation of peak discharges for various conditions of development in the drainage area or for different periods will also require separate estimates of T_c for each existing or assumed land-use condition and for each flood return period.

2.2.7 Estimation of Hydrograph Shape

This procedure is to be used where routing of the storm inflow through the drainage structure is desired, such as for the design of a detention basin or pump station. The procedure is based on synthesizing a hydrograph from the peak discharge estimated by the Rational Method and by the use of some dimensionless hydrograph shapes from TR-55 (Soil Conservation Service, 1986). Two sets of dimensionless hydrographs are provided; one set is for use with urbanized watersheds (**Table 2-2**), and the other set is for use with undeveloped watersheds (**Table 2-3**, Both sets of dimensionless unit hydrographs are functions of T_c.

TABLE 2-2
URBAN WATERSHED - COORDINATES (q_t) OF DIMENSIONLESS HYDROGRAPH TO BE USED WITH THE RATIONAL METHOD

Time[a]	q_t values									
	T_c, in hours									
hours	0.17	.18 - .25	.26 - .35	.36 - .45	.46 - .62	.63 - .88	.89 - 1.12	1.13 - 1.38	1.39 - 1.75	1.76 - 2.5
0.0	0	0	0	0	0	0	0	0	0	0
1.0	24	23	20	18	17	13	11	10	9	7
1.3	34	31	28	25	23	18	15	13	11	9
1.6	53	47	41	36	32	24	20	18	15	12
1.9	334	209	118	77	57	36	29	25	21	16
2.0	647	403	235	141	94	46	35	29	25	18
2.1	**1010**	739	447	271	170	68	47	38	31	21
2.2	623	**800**	676	468	308	115	72	54	41	27
2.3	217	481	**676**	**592**	467	194	112	81	58	36
2.4	147	250	459	574	**529**	294	168	118	82	49
2.5	123	166	283	431	507	380	231	163	112	64
2.6	104	128	196	298	402	**424**	289	213	147	82
2.7	86	102	146	216	297	410	329	256	184	104
2.8	76	86	114	163	226	369	**357**	284	216	127
3.0	66	70	80	104	140	252	313	**311**	255	171
3.2	57	61	66	77	96	172	239	266	**275**	201
3.4	51	54	57	63	74	123	175	212	236	**226**
3.6	46	49	51	55	61	93	133	163	198	205
3.8	42	44	46	49	53	74	103	129	159	193
4.0	38	40	42	44	47	61	83	104	129	171
4.3	34	35	37	38	41	49	63	78	98	132
4.6	32	33	33	34	36	41	50	61	76	105
5.0	29	30	31	31	32	35	40	47	57	79
5.5	26	27	28	28	29	31	33	37	43	58
6.0	23	24	24	25	26	27	29	31	35	45
6.5	21	21	22	22	23	24	26	27	30	36
7.0	20	20	20	21	21	22	23	24	25	30
7.5	19	19	19	20	20	20	21	22	23	26
8.0	18	18	18	18	19	19	20	20	21	23
9.0	15	16	16	16	16	17	17	18	18	20
10.0	13	13	13	14	14	15	15	16	16	17
12.0	12	12	12	12	12	12	12	12	12	13
16.0	0	0	0	0	0	0	0	1	1	3

Reference: TR-55 (1986), Exhibit 5-II for IA/P = 0.10 and Travel Time = 0.0

Notes:

a - Time is the TR-55 hydrograph time minus 10 hours.

b - The maximum unit peak discharge, q_{tmax}, is underlined for each hydrograph.

TABLE 2-3
UNDEVELOPED WATERSHED - COORDINATES (q_t) OF DIMENSIONLESS HYDROGRAPH TO BE USED WITH THE RATIONAL METHOD

Time[a]	q_t values									
	T_c, in hours									
hours	0.17	.18 - .25	.26 - .35	.36 - .45	.46 - .62	.63 - .88	.89 - 1.12	1.13 - 1.38	1.39 - 1.75	1.76 - 2.5
0.0	0	0	0	0	0	0	0	0	0	0
1.0	0	0	0	0	0	0	0	0	0	0
1.3	0	0	0	0	0	0	0	0	0	0
1.6	0	0	0	0	0	0	0	0	0	0
1.9	0	0	0	0	0	0	0	0	0	0
2.0	70	7	1	0	0	0	0	0	0	0
2.1	539	98	25	7	2	0	0	0	0	0
2.2	377	371	151	59	26	2	1	1	0	0
2.3	196	322	299	168	89	16	7	5	3	1
2.4	171	221	277	245	170	45	21	13	8	4
2.5	154	182	219	257	217	92	42	26	16	8
2.6	134	158	187	213	229	137	71	44	27	13
2.7	117	137	162	186	200	166	101	68	42	20
2.8	108	120	141	163	179	185	126	91	59	28
3.0	99	104	113	128	144	170	160	125	92	51
3.2	89	94	100	109	119	146	154	142	116	73
3.4	83	86	90	96	104	125	138	142	128	92
3.6	77	80	84	88	93	110	123	128	130	104
3.8	72	74	73	81	85	98	110	117	121	111
4.0	67	69	72	75	78	89	100	107	112	112
4.3	61	62	65	67	70	79	87	94	100	106
4.6	59	60	61	62	64	70	77	83	90	97
5.0	56	57	58	58	59	63	67	72	78	86
5.5	51	52	53	54	55	58	60	63	67	75
6.0	46	47	48	50	51	53	55	57	60	66
6.5	43	44	44	45	46	48	50	52	55	60
7.0	42	42	42	43	43	44	46	47	50	54
7.5	40	40	41	41	41	42	43	44	46	49
8.0	38	39	39	39	40	41	41	42	43	46
9.0	34	35	35	35	36	37	38	38	39	40
10.0	30	30	31	31	32	33	34	34	35	37
12.0	28	28	28	28	28	28	28	28	29	30
16.0	0	0	0	0	0	0	1	2	4	7

Reference: TR-55 (1986), Exhibit 5-II for IA/P = 0.50 and Travel Time = 0.0

Notes:

[a] - Time is the TR-55 hydrograph time minus 10 hours.

[b] - The maximum unit peak discharge, q_{tmax}, is underlined for each hydrograph.

2.3 INSTRUCTIONS

A. For estimating peak discharge:

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

2. Decide whether the generalized I-D-F graphs will be used or whether a site-specific I-D-F graph will be developed.

    a. If the generalized I-D-F graphs are to be used, determine the Zone from **Figure 1-1** of Chapter 1 - Rainfall. Use the I-D-F graph of **Figure 2-1** if the watershed is in Zone 6, and use **Figure 2-2** if the watershed is in Zone 8.

    b. If a site-specific I-D-F graph is to be used, develop the I-D-F graph by procedures in Chapter 1 - Rainfall.

3. Select the desired return period(s).

4. Determine the 1-hour rainfall depth (P_1) for each return period.
   Note: P_1 = 1-hr rainfall intensity times 1 hour.

5. Estimate the time of concentration (T_c), for each return period, by Equation 2-2.

6. Select the rainfall intensity (i) from the I-D-F graph at a duration equal to T_c which is the value of (i) used in the solution of Equation 2-2 (but not less than 10 minutes).

7. Estimate C:

    a. If the watershed is developed, use **Figure 2-3**. This will require an appraisal 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 **Figures 2-4** through **2-8**. This will require an appraisal of Hydrologic Soil Group (HSG), A through D, from Soil Conservation Service (SCS) soils reports, and an estimate of percent vegetation cover. C is selected as a function of P_1, and HSG-percent vegetation cover.

8. Calculate the peak discharge by Equation 2-1.

B. For estimating a runoff hydrograph:

1. Calculate Q according to the above instructions.

2. Select the appropriate dimensionless hydrograph coordinates to use from **Table 2-2** or **Table 2-3**. The selection is based on T_c (round to the nearest T_c value in the tables) and on whether the drainage area is urbanized or undeveloped.

3. Read the maximum unit peak discharge, q_{tmax}, for the selected dimensionless hydrograph and computed T_c value in either **Table 2-2** or **Table 2-3**.

4. Calculate: $K = Q/q_{tmax}$.

5. Tabulate the time and q_t values from either **Table 2-2** or **Table 2-3** and multiply each q_t by K.

$$q = Kq_t$$

6. Plot the hydrograph discharge (q) versus time.

7. Draw a smooth hydrograph. This may require extending the rising limb of the hydrograph to intersect the 0 discharge axis.

FIGURE 2-3
RATIONAL "C" COEFFICIENT
DEVELOPED WATERSHEDS

AS A FUNCTION OF RAINFALL DEPTH AND TYPE OF DEVELOPMENT

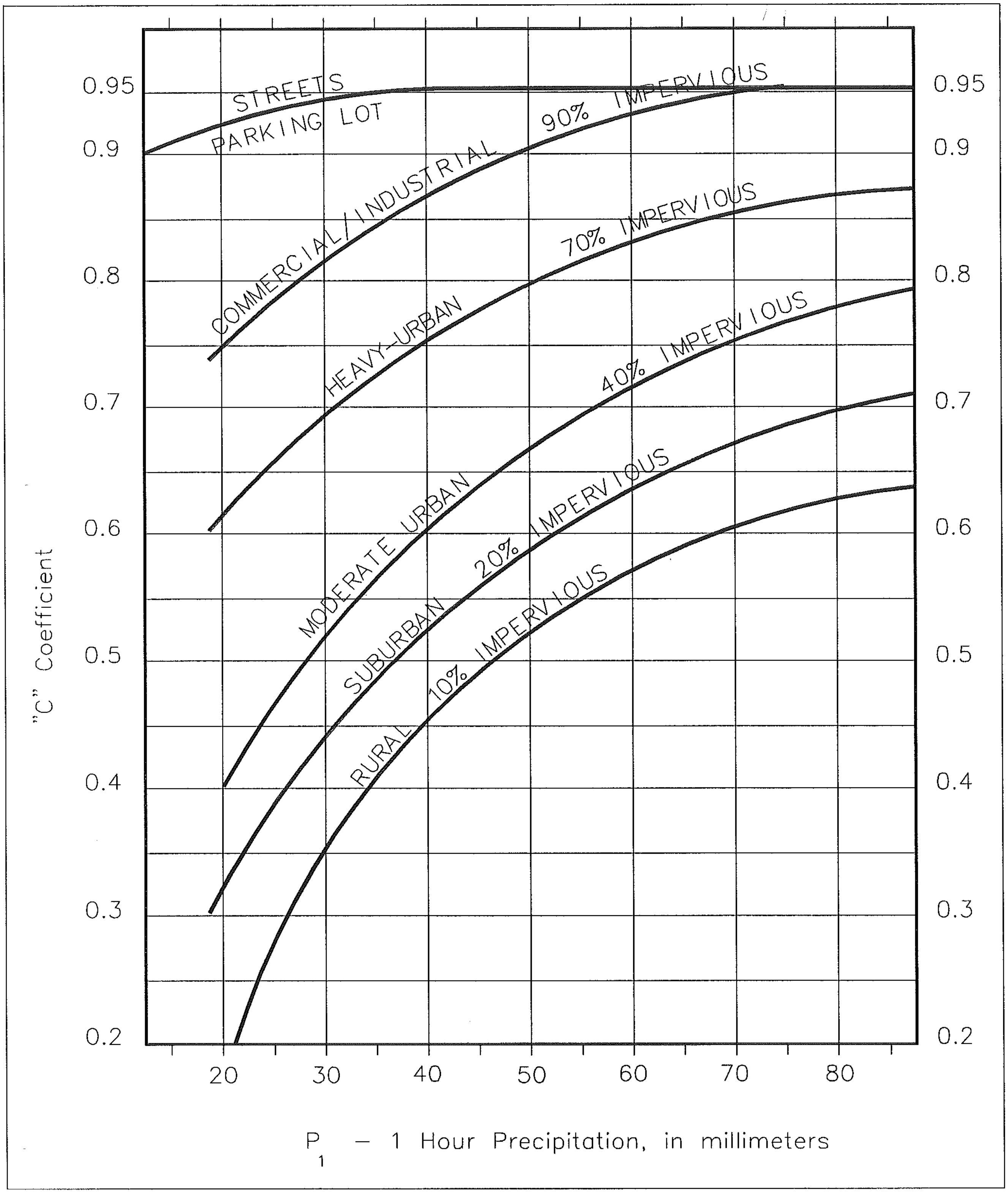


FIGURE 2-4
RATIONAL "C" COEFFICIENT
DESERT
(CACTUS, GRASS & BRUSH)

AS A FUNCTION OF RAINFALL DEPTH, HYDROLOGIC SOIL GROUP (HSG), AND % OF VEGETATION COVER.

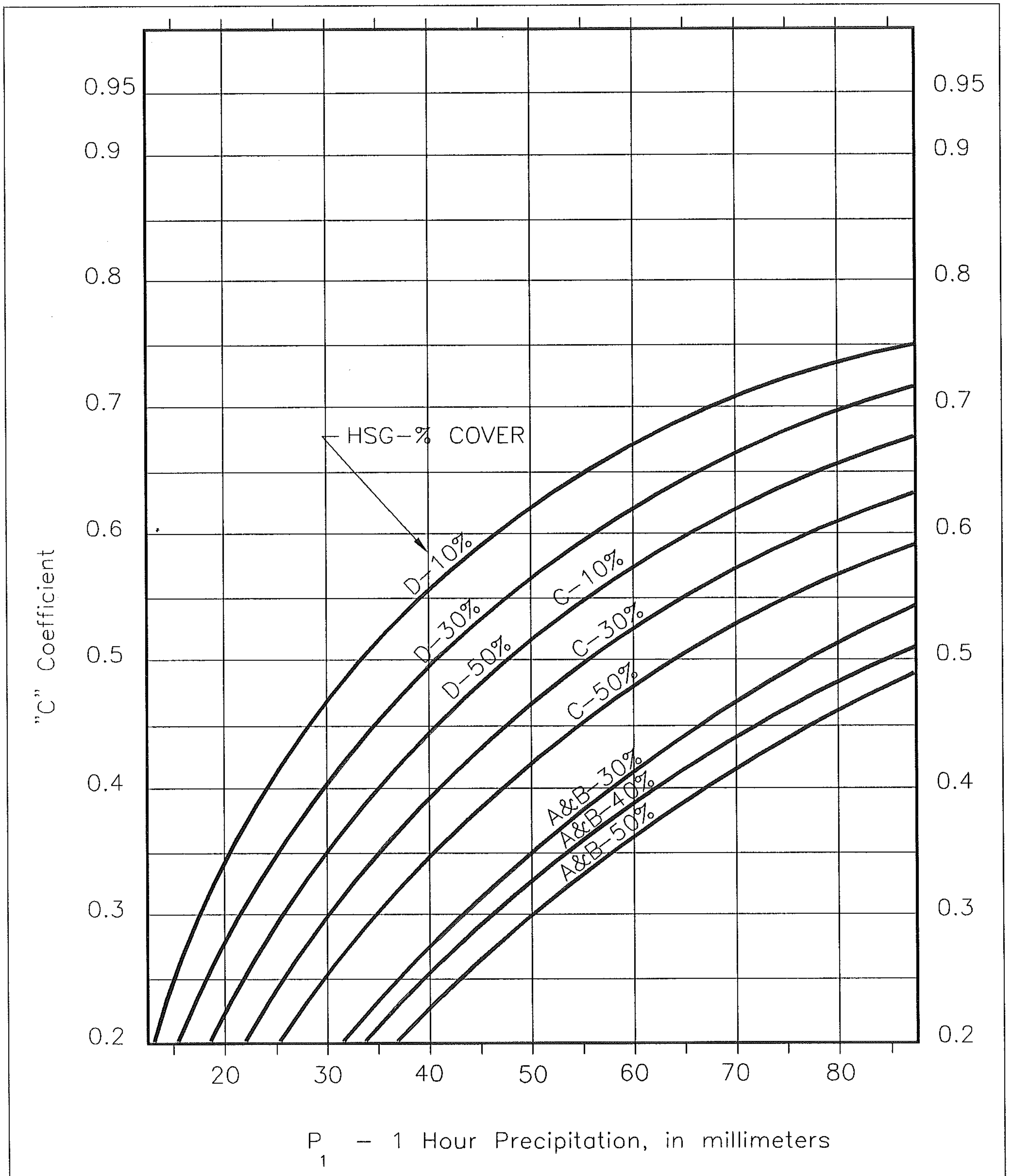


FIGURE 2-5
RATIONAL "C" COEFFICIENT
UPLAND RANGELAND
(GRASS & BRUSH)

AS A FUNCTION OF RAINFALL DEPTH, HYDROLOGIC SOIL GROUP (HSG), AND % OF VEGETATION COVER

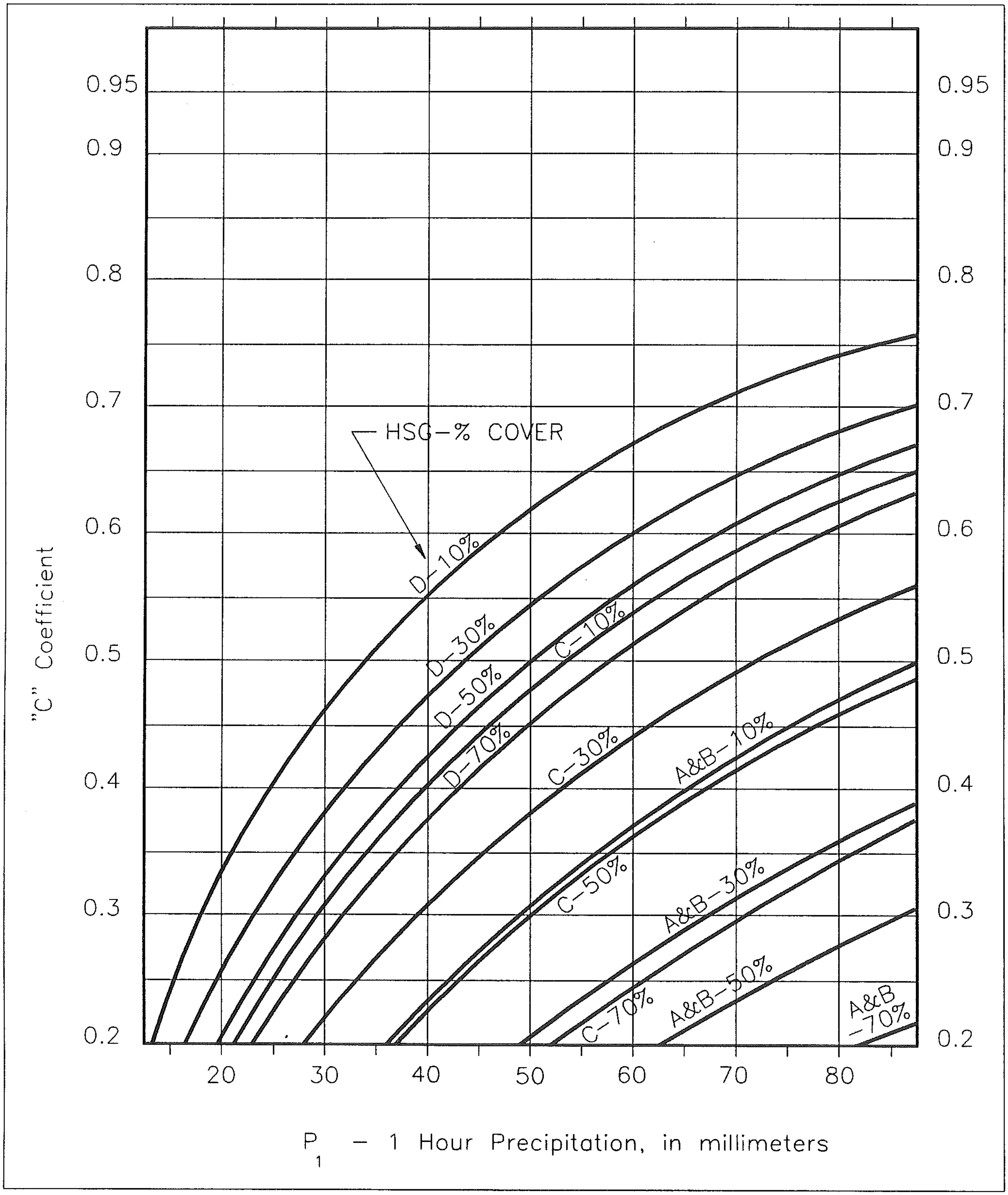


FIGURE 2-6
RATIONAL "C" COEFFICIENT
MOUNTAIN
(GRASS & BRUSH)

AS A FUNCTION OF RAINFALL DEPTH, HYDROLOGIC SOIL GROUP (HSG), AND % OF VEGETATION COVER

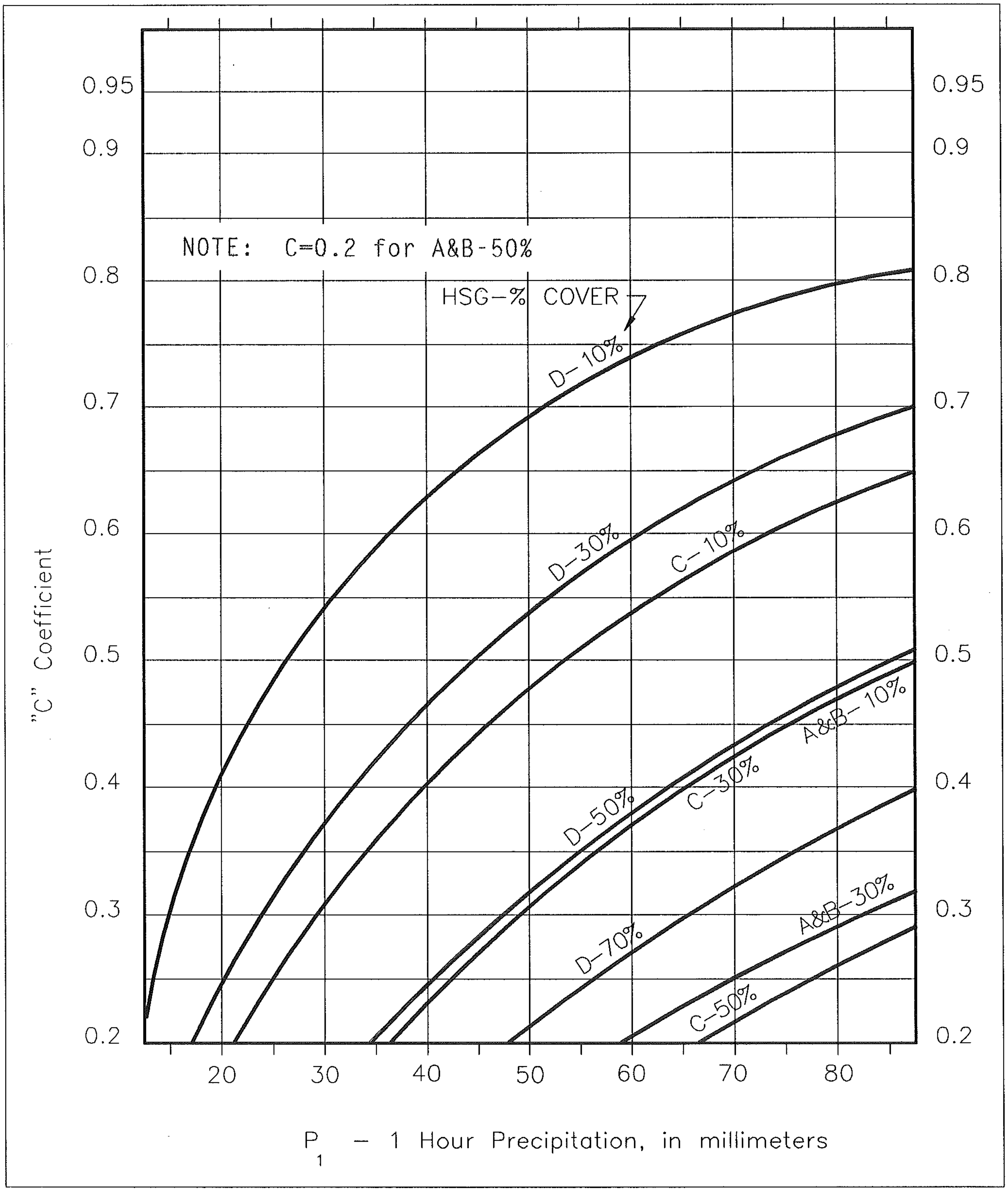


FIGURE 2-7
RATIONAL "C" COEFFICIENT
MOUNTAIN
(JUNIPER & GRASS)

AS A FUNCTION OF RAINFALL DEPTH, HYDROLOGIC SOIL GROUP (HSG), AND % OF VEGETATION COVER

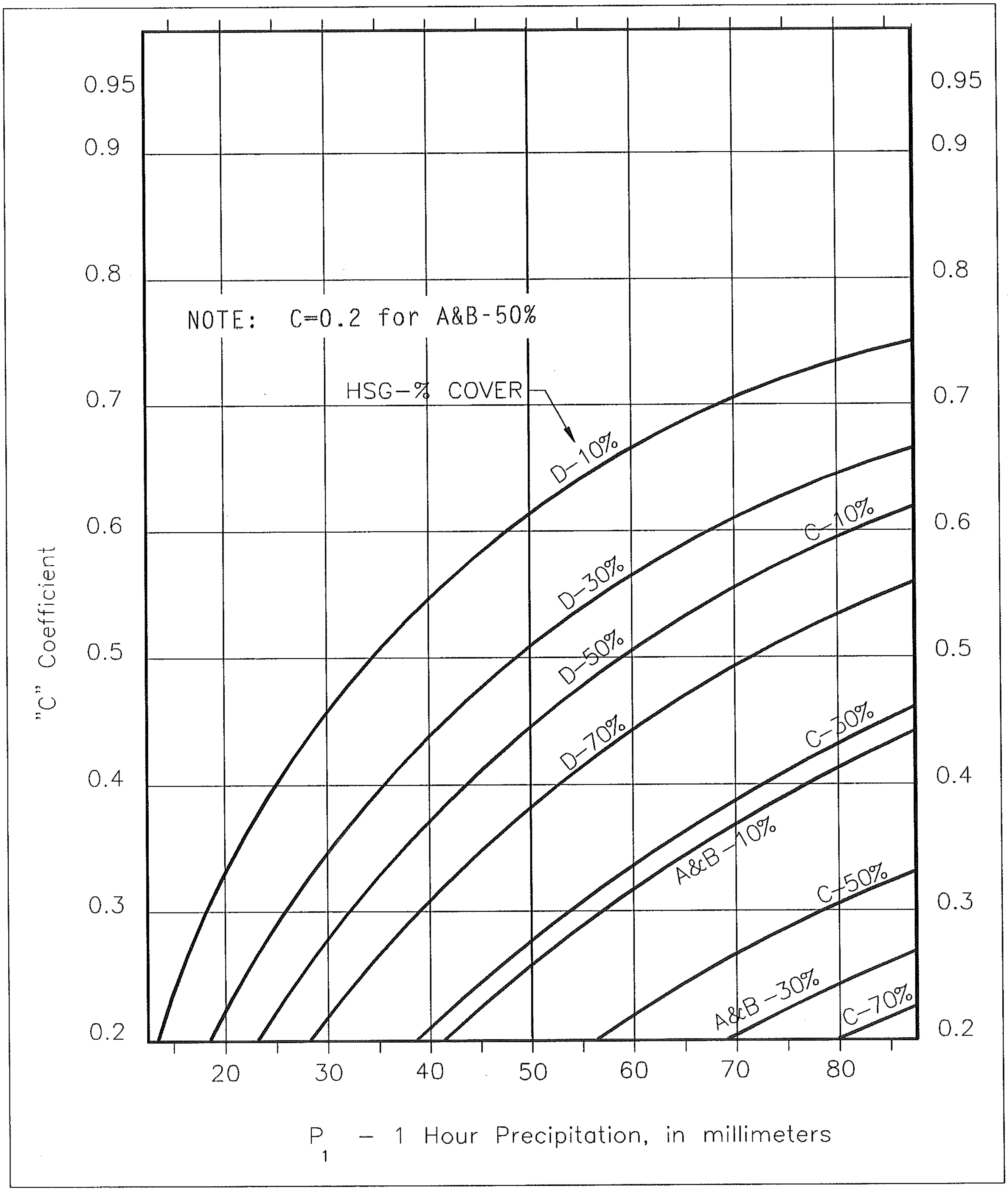
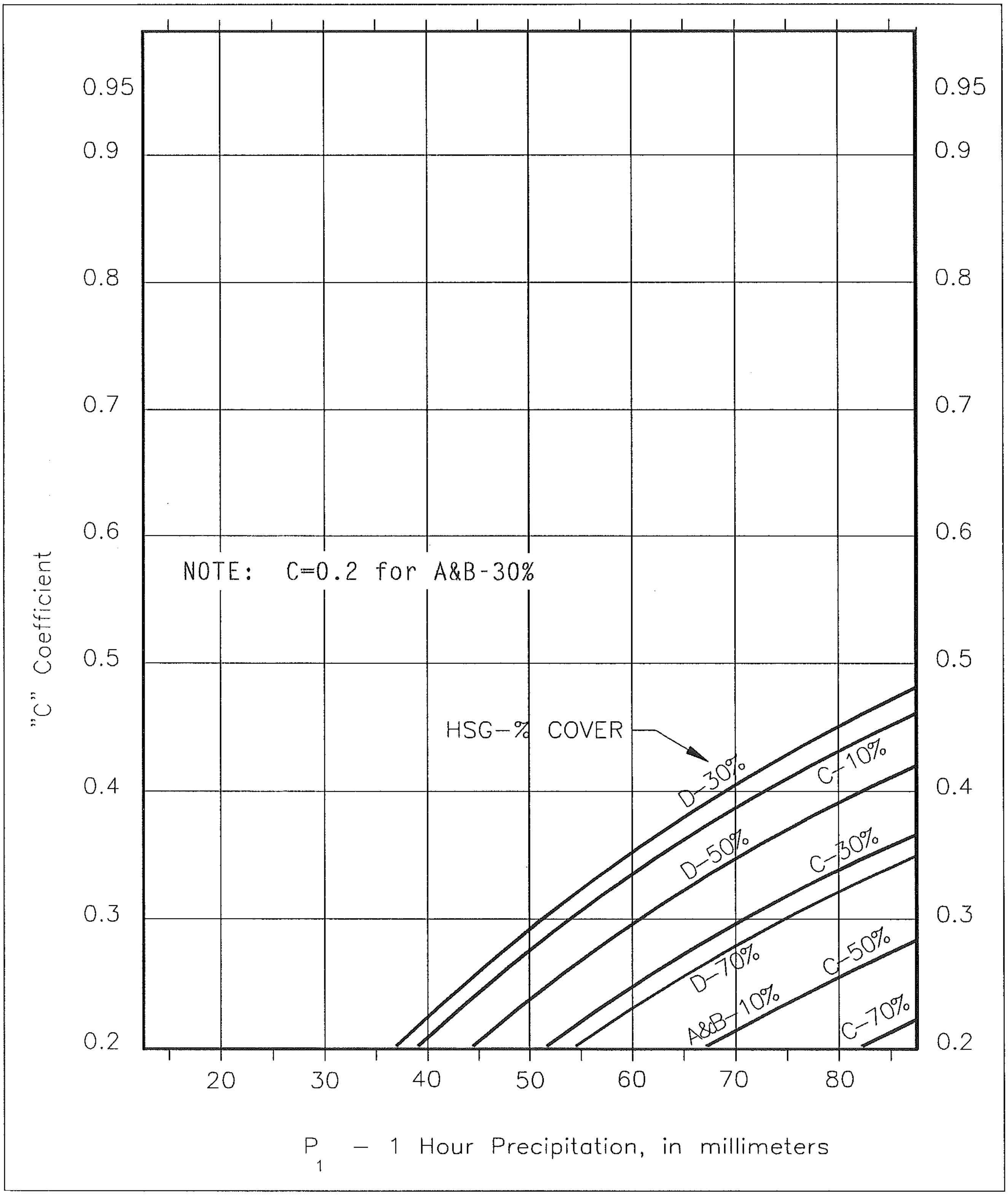


FIGURE 2-8
RATIONAL "C" COEFFICIENT
MOUNTAIN
(PONDEROSA PINE)

AS A FUNCTION OF RAINFALL DEPTH, HYDROLOGIC SOIL GROUP (HSG),
AND % OF VEGETATION COVER



Problem:

Calculate the 100-year peak discharge and estimate the runoff hydrograph for a 25 hectare, single-family residential (about 20% effective impervious area) watershed in Phoenix. The following are the watershed characteristics:

A = 25 hectares

S = 5m/km

L = 1 km

The following were obtained for the watershed:

P_1 = 62.5 mm (converted from NOAA Atlas data, Appendix B)

K_b = .025 from **Table 2-1**

C = .65 from **Figure 2-3**

Solution:

This example is solved using A) a site-specific I-D-F graph, and B) using the generalized I-D-F graph.

A) Using the site specific I-D-F graph (shown):

Solve for T_c :

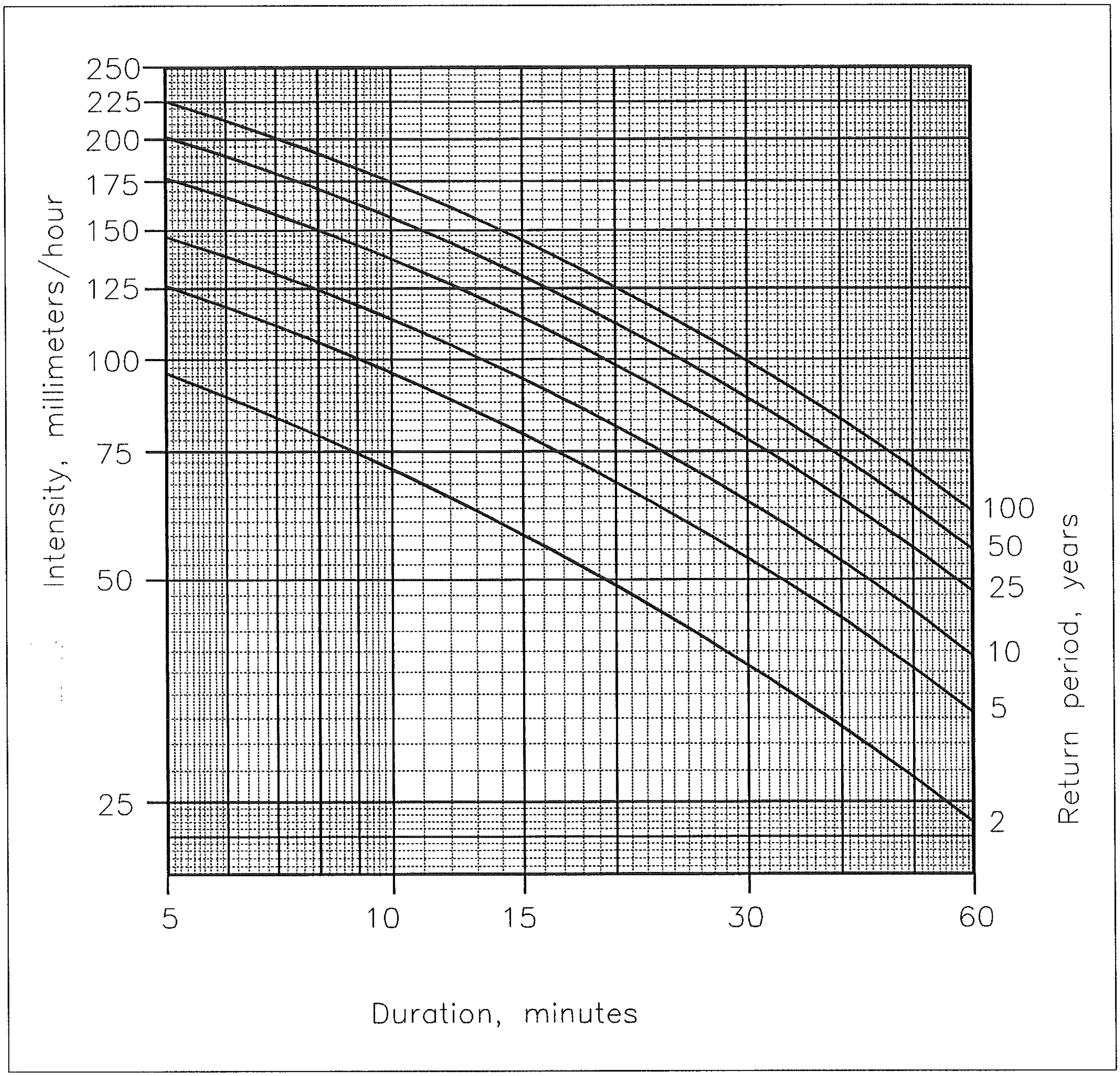
$$T_c = 18.3\ L^{Q5}\ K_b^{.52}\ S^{-.31}\ i^{-.38}$$

$$T_c = 18.3\ (1^{Q5})\ (.025^{.52})\ (5^{-.31})\ i^{-.38}$$

$$= 1.63\ I^{-.38}$$

Trial T_c, hr	i,mm/hr	Calculated T_c, hr
.75	77	.31
.30	133	.25
.24	148	.24 OK

RAINFALL INTENSITY-DURATION-FREQUENCY



Calculate Q:

$$Q = C\,i\,A\,/363$$
$$= (0.65)\,(148)\,(25)\,/363$$
$$= 6.6\ m^3/s$$

B) Using the generalized I-D-F graph (**Figure 2-2** for Zone 8):

Solve for T_c:

$$T_c = 1.63\ i^{-.38}$$

Trial T_c, hr	i,mm/hr	Calculated T_c, hr
.33 (19.8 minutes)	130	.25
.24 (14.4 minutes)	156	.24 OK

Calculate Q:

$$Q = C\,i\,A\,/363$$
$$= (0.65)\,(156)\,(25)\,/363$$
$$= 7.0\ m^3/s$$

The hydrograph shape is calculated using the Q that was calculated using the generalized I-D-F graph.

EXAMPLE 2-1

Estimate the hydrograph shape:

Use the urban, dimensionless hydrograph from **Table 2-2** for T_c = .18 to .25 hr.

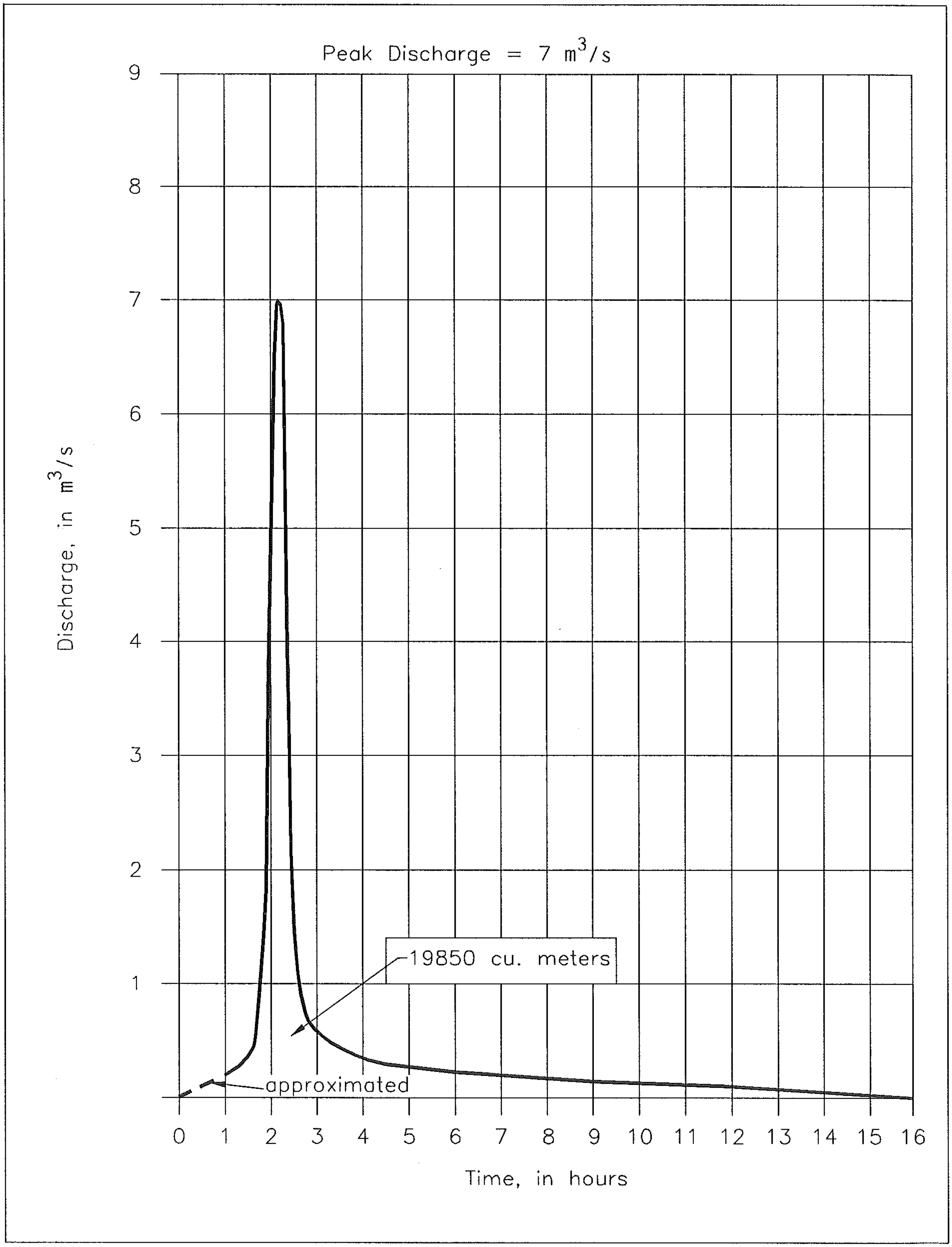
q_{tmax} = 800

$$K = \frac{Q}{q_{tmax}} = \frac{7.0}{800} = .009$$

Tabulated Time (hr)	Dimensionless Hydrograph q_t (m^3/s)	Runoff Hydrograph $q_i = Kq_t$ (m^3/s)	Average Discharge $\frac{q_i + q_i + 1}{2} = \bar{q}$ (m^3/s)	Volume Calculation $\bar{q}\,(\Delta t)$ ($m^3/s{\cdot}hr$)
1.0	23	0.20		
1.3	31	0.27	0.24	0.07
1.6	47	0.41	0.34	0.10
1.9	209	1.83	1.12	0.34
2.0	403	3.53	2.68	0.27
2.1	739	6.47	5.00	0.50
2.2	800	7.00	6.73	0.67
2.3	481	4.21	5.60	0.56
2.4	250	2.19	3.20	0.32
2.5	166	1.45	1.82	0.18
2.6	128	1.12	1.29	0.13
2.7	102	0.89	1.01	0.10
2.8	86	0.75	0.82	0.08
3.0	70	0.61	0.68	0.14
3.2	61	0.53	0.57	0.11
3.4	54	0.47	0.50	0.10
3.6	49	0.43	0.45	0.09
3.8	44	0.39	0.41	0.08
4.0	40	0.35	0.37	0.07
4.3	35	0.31	0.33	0.10
4.6	33	0.29	0.30	0.09
5.0	30	0.26	0.28	0.11
5.5	27	0.24	0.25	0.12
6.0	24	0.21	0.22	0.11
6.5	21	0.18	0.20	0.10
7.0	20	0.18	0.18	0.09
7.5	19	0.17	0.17	0.09
8.0	18	0.16	0.16	0.08
9.0	16	0.14	0.15	0.15
10.0	13	0.11	0.13	0.13
12.0	12	0.11	0.11	0.22
16.0	0	0.00	0.05	0.21
				5.5 m^3/s-hr (19,850 cubic meters)

EXAMPLE 2-1

PEAK DISCHARGE



CHAPTER 3
RAINFALL LOSSES

3.1 INTRODUCTION

3.1.1 General Discussion

Rainfall excess is that portion of the total rainfall depth that drains directly from the land surface by overland flow. By a mass balance, rainfall excess plus rainfall losses equals precipitation.

This chapter is only applicable when performing rainfall-runoff modeling with the HEC-1 program. The design rainfall is determined from the procedures in the Rainfall section, and this chapter provides procedures to estimate the runoff from the applied rainfall. 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.

One of two methods shall be used to estimate rainfall losses; the primary method is to be used for the majority of cases, and the secondary method is to be used only for special cases when it is determined that the primary method is inappropriate. The primary method requires the estimation of the surface retention loss (**Table 3-1**) and the estimation of the rainfall infiltration loss by the Green and Ampt equation. The Green and Ampt equation parameters are estimated as a function of soil texture (**Table 3-2**). This classification system places soil into one of 12 texture classes based on the size gradation of the soil according to percentage sand, silt, and clay (**Figure 3-1**). One of the Green and Ampt equation parameters (hydraulic conductivity) can be adjusted for the effects of vegetation ground cover (**Figure 3-2**). Correction for vegetation ground cover is not to be made if the soil is either sand or loamy sand, and this is because the use of such a correction could result in overestimation of the losses due to infiltration.

TABLE 3-1

SURFACE RETENTION LOSS FOR VARIOUS LAND SURFACES IN ARIZONA

(To be used with the Green and Ampt Infiltration Equation
for estimating rainfall losses.)

Land-use and/or Surface Cover (1)	Surface Retention Loss (IA) millimeters (2)
Natural	
Desert and rangeland, flat slope	9
Desert and rangeland, hill slopes	4
Mountain, with vegetated surface	6
Developed (Residential and Commercial)	
Lawn and turf	5
Desert Landscape	3
Pavement	2
Agricultural	
Tilled fields and irrigated pasture	13

TABLE 3-2

GREEN AND AMPT INFILTRATION EQUATION LOSS RATE PARAMETER VALUES FOR BARE GROUND

Soil Texture Classification	DTHETA[a] Dry	DTHETA[a] Normal	DTHETA[a] Saturated	XKSAT mm/hr	PSIF millimeters
(1)	(2)	(3)	(4)	(5)	(6)
sand[b]	.35	.30	0	120.0	50
loamy sand	.35	.30	0	30.0	61
sandy loam	.35	.25	0	11.0	110
loam	.35	.25	0	6.5	89
silt loam	.40	.25	0	3.4	170
silt	.35	.15	0	2.5	190
sandy clay loam	.25	.15	0	1.5	220
clay loam	.25	.15	0	1.0	210
silty clay loam	.30	.15	0	1.0	270
sandy clay	.20	.10	0	0.6	240
silty clay	.20	.10	0	0.5	290
clay	.15	.05	0	0.3	320

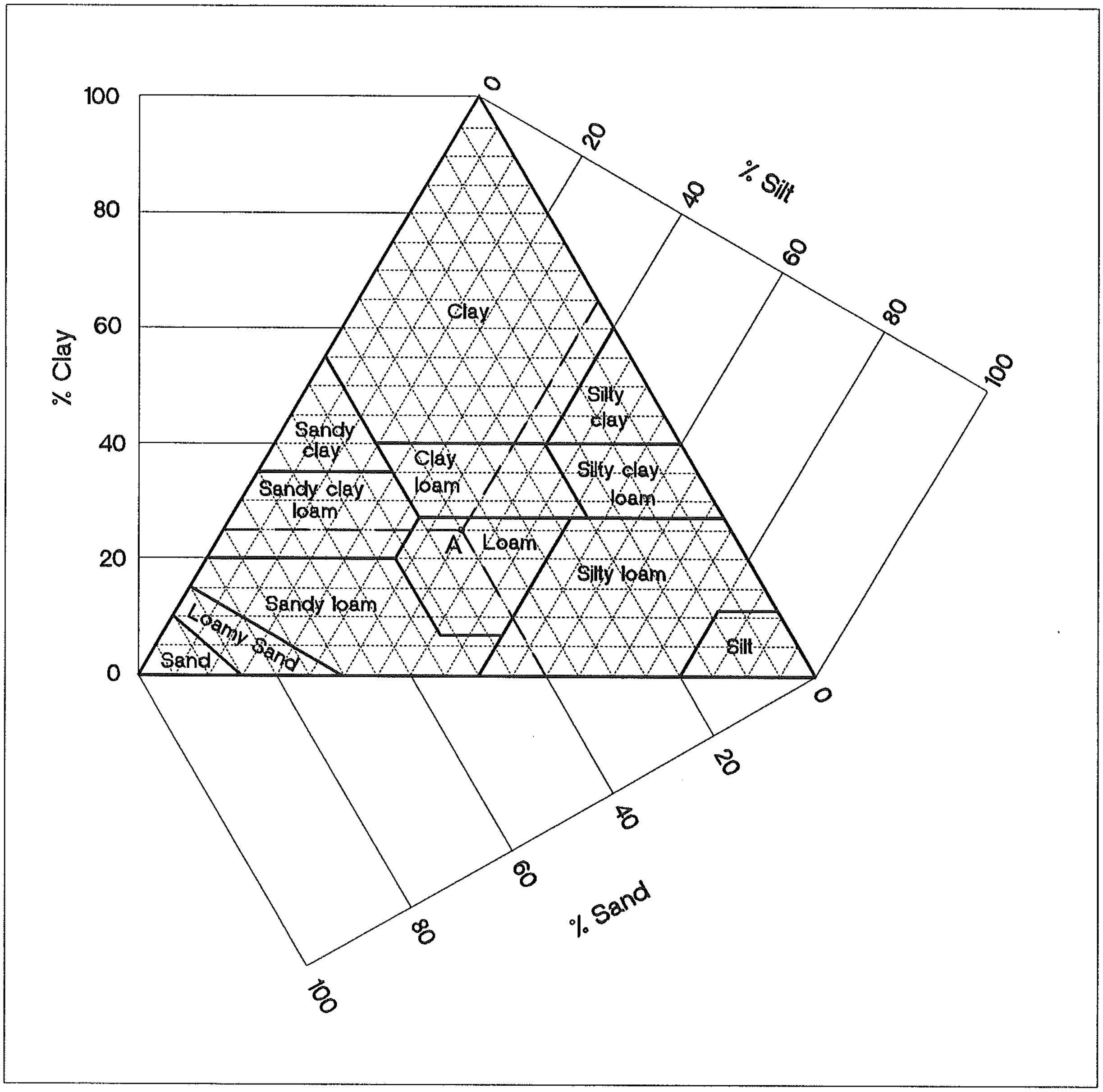
[a] Selection of DTHETA:
Dry - for non-irrigated lands such as desert and rangeland
Normal - for irrigated lawn, turf, and permanent pasture
Saturated - for irrigated agricultural lands

[b] The use of the Green and Ampt Infiltration Equation for drainage areas or subbasins that are predominantly sand should be avoided and the IL+ULR method should be used.

FIGURE 3-1

SOIL TEXTURE CLASSIFICATION

TRIANGLE



Reference: U.S. Department of Agriculture

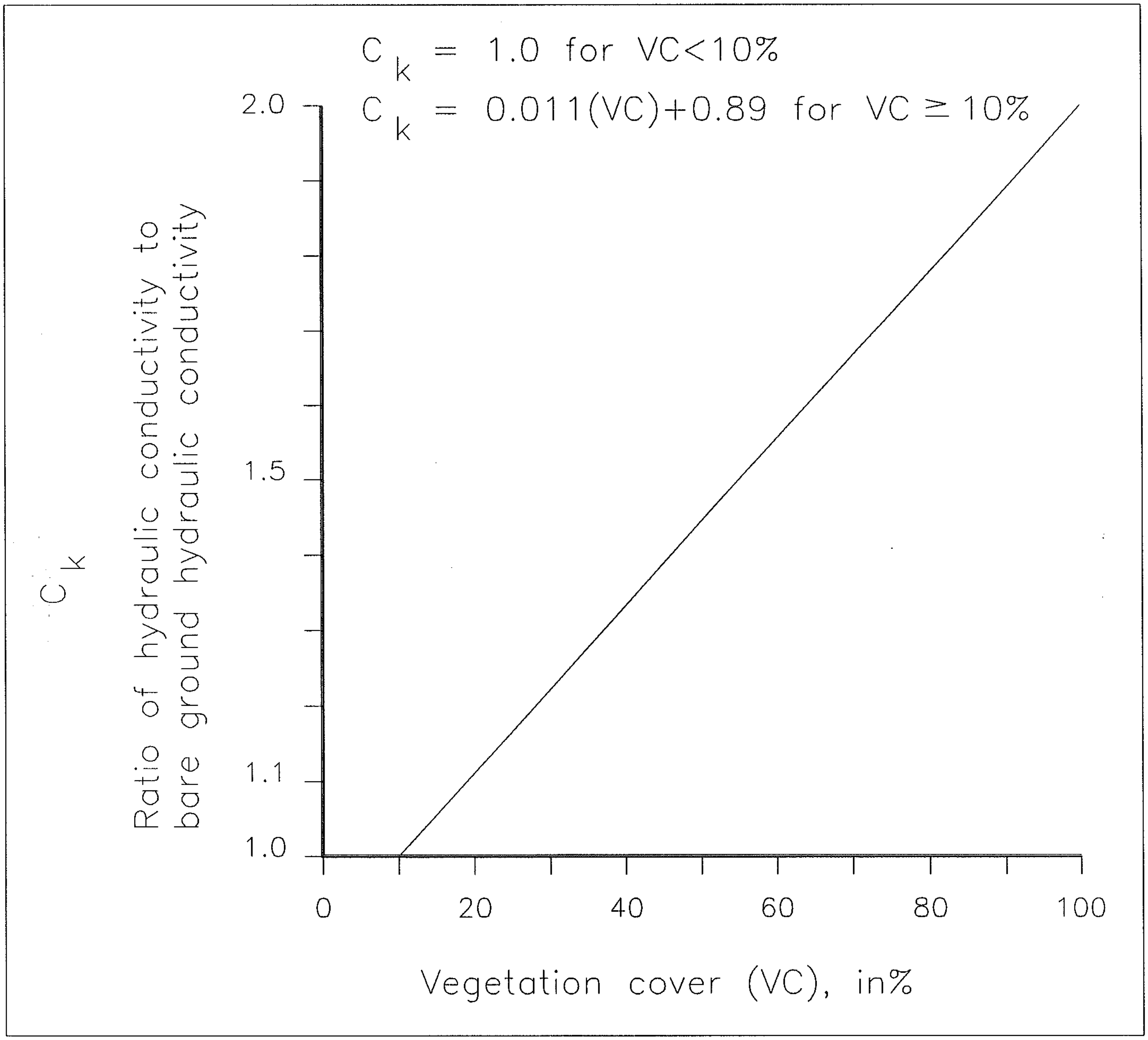
Definitions:
Clay - mineral soil particles less than 0.002 mm in diameter.
Silt - mineral soil particles that range in diameter from 0.002 mm to 0.05 mm.
Sand - mineral soil particles that range in diameter from 0.05 mm to 2.0 mm.

Example: Point A is a soil composed of 40% sand, 35% silt, and 25% clay. It is classified as a loam.

FIGURE 3-2

EFFECT OF VEGETATION COVER ON HYDRAULIC CONDUCTIVITY FOR HYDRAULIC SOIL GROUPS B, C, AND D, AND FOR ALL SOIL TEXTURES EXCEPT SAND AND LOAMY SAND

(Reference - Drainage Design Manual for Maricopa County, Arizona, Volume I, Hydrology)



The secondary method requires the estimation of the initial loss and an uniform loss rate (IL+ULR method). The secondary 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, or for watersheds that are predominantly composed of sand; for example, the land surface of upland watersheds of the San Francisco Mountains near Flagstaff are generally composed of volcanic cinder overlain by forest duff and the Green and Ampt equation is not appropriate. Infiltration is not controlled by soil texture in such watersheds and infiltration rates may be as high as 125 millimeters (5 inches) per hour or more. Use of the secondary method requires adequate data or appropriate studies to verify the IL+ULR parameters or to calibrate the model of the watershed.

Both the primary and the secondary methods require the estimation of the impervious area of the watershed. Impervious area (or nearly impervious area) is composed of rock outcrop, paved roads, parking lots, roof tops, and so forth. When performing watershed modeling with the HEC-1 program, the impervious area is to be the effective (directly connected) impervious area (see definitions). For urbanized areas, the effective impervious area should be estimated from aerial photographs with guidance as provided in **Table 3-3**. For areas that are presently undeveloped but for which flood estimates are desired for future urbanized conditions, estimates of effective impervious area should be obtained based on regional planning and land-use zoning as determined by the local jurisdiction. 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 3-3**. For undeveloped areas, the effective impervious area is often 0 percent. However, in some watersheds there could be extensive rock outcrop that would greatly increase the imperviousness of the watershed. Care must be exercised when estimating effective impervious area for rock outcrop. Often the rock outcrop is relatively small (in terms of the total drainage area) and is of isolated units surrounded by soils of relatively high infiltration capacities. Relatively small, isolated rock outcrop should not be considered as 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 subbasins so that the direct runoff can be estimated and then routed (with channel transmission losses, if appropriate) to the point of interest. Paved roads through undeveloped watersheds will not normally contribute to effective impervious area unless the road serves as a conveyance to the watershed outlet.

TABLE 3-3

GENERAL GUIDANCE FOR SELECTING EFFECTIVE IMPERVIOUS AREA (RTIMP)

Land-Use	Effective Impervious Area, in percent	
	Mean	Range
(1)	(2)	(3)
Single-Family Residential		
0.10 hectares (1/4 acre)	30	23-38
0.15 hectares (1/3 acre)	22	15-30
0.20 hectares (1/2 acre)	17	9-25
0.50 hectares (1 acre)	14	8-20
1.0 hectare (2 acres)	12	7-20
Multi-Family Residential	54	42-65
Commercial	85	51-98
Industrial	59	46-72

3.2 PROCEDURE

3.2.1 General Considerations

1. Infiltration is the movement of water from the land surface into and through the upper horizon of soil. Percolation is the movement of water through the underlying soil or geologic strata subsequent to infiltration. Infiltration can be controlled by percolation if the soil does not have a sustained drainage capacity to provide access for more infiltrated water. However, the extent by which percolation can restrict infiltration for design rainfalls in Arizona needs to be carefully considered. For example, shallow soils with high infiltration rates that overlay nearly impervious material can be placed in hydrologic soil group D in SCS soil surveys. The soil texture, vegetation cover, and depth of the surface horizon of soil and the properties of the underlying horizons of soil need to be considered when estimating the infiltration rate. Surface soils that are more than 150 millimeters (6 inches) thick should generally be considered adequate to contain infiltrated rainfall for up to the 100-year rainfall in Arizona without the subsoil restricting the infiltration rate.

This is because most common soils have porosities that range from about 25 to 35 percent, and therefore 150 millimeters of soil with a porosity of 30 percent can absorb about 45 millimeters (150 millimeters times 30 percent) of rainfall infiltration. It is unlikely that more soil moisture storage is needed for storms up to the 100-year return period in Arizona. Accordingly, in estimating the Green and Ampt infiltration parameters in Arizona, for up to the 100-year rainfall, the top 150 millimeters of soil should be considered. If the top 150 millimeter horizon is uniform soil or nearly uniform, then select the Green and Ampt parameters (**Table 3-2**) for that soil texture. If the top 150 millimeter horizon is layered with different soil textures, then select the Green and Ampt parameters (**Table 3-2**) for the soil texture with the lowest hydraulic conductivity (XKSAT).

2. Parameter values for design should be based on reasonable estimates of watershed conditions that would minimize rainfall losses. The estimate of impervious area (RTIMP) for urbanizing areas should be based on ultimate development in the watershed.

3. Two sources of information are to be used to classify soil texture for the purpose of estimating Green and Ampt infiltration equation parameters. The primary source that is to be used for the watershed, when it is available, are the detailed soil surveys that are prepared by the USDA, Soil Conservation Service (SCS). When detailed soil surveys are not available for the watershed, then the general soil maps and accompanying reports prepared by the SCS for each county in Arizona are to be used.

4. Most drainage areas or modeling subbasins will be composed of several subareas containing soils of different texture; and therefore, there may be the need to determine composite values for the Green and Ampt parameters to be applied to the drainage areas or each modeling subbasin. The procedure that is to be used is to average the area-weighted logarithms of the individual subarea XKSAT values and to select the PSIF and DTHETA values from a graph.

The composite XKSAT is calculated by Equation 3-1:

$$\overline{XKSAT} = \text{antilog}\left(\frac{\Sigma\, A_i \log XKSAT_i}{A_T}\right) \tag{3-1}$$

where $\overline{XKSAT}$ = composite hydraulic conductivity (XKSAT), in millimeters/hour,

$XKSAT_i$ = hydraulic conductivity (**Table 3-2**) of the soil in a subarea, in millimeters/hour.

A_i = size of a subarea, and

A_T = size of the drainage area or modeling subbasin.

After $\overline{XKSAT}$ is calculated, the values of PSIF and DTHETA (normal or dry) are selected from **Figure 3-3** at the corresponding value of $\overline{XKSAT}$.

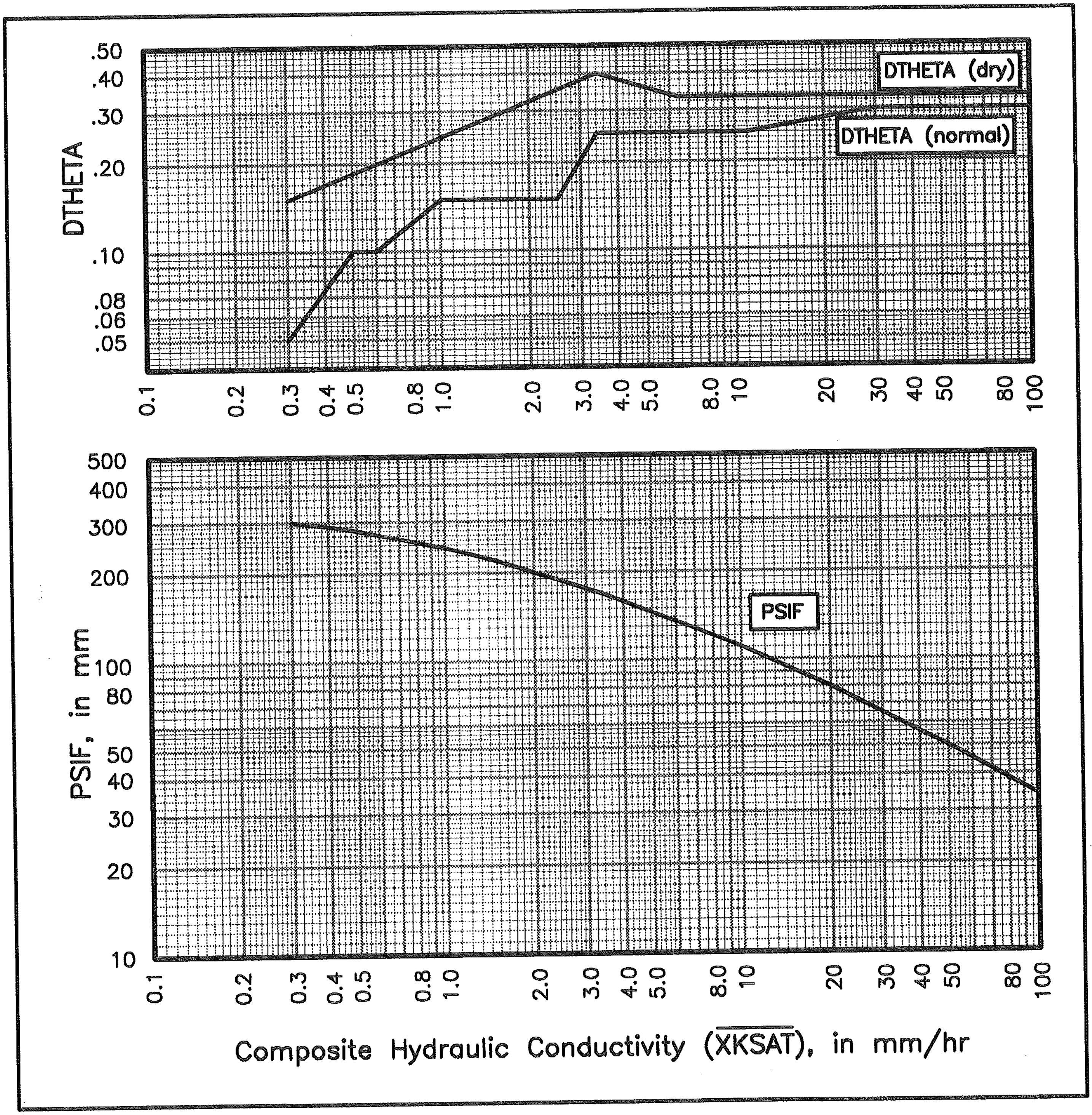
5. The composite values for PSIF and DTHETA (**Figure 3-3**) are determined from the composite value of XKSAT prior to making the correction of XKSAT for vegetation cover. Correction of XKSAT for vegetation cover (**Figure 3-2**) is made after the composite value of XKSAT is determined (Equation 3-1).

6. There are conceptual and computational differences between the Green and Ampt infiltration equation method and the IL+ULR method for estimating rainfall losses. When using the IL+ULR method, the initial loss (STRTL) is defined as the sum of surface retention loss (IA) plus initial infiltration loss that accrues before surface runoff is produced, and this is equivalent to initial abstraction (see definitions). When using the Green and Ampt infiltration equation method, the initial abstraction is calculated based on the input of both the surface retention loss (IA) and the infiltration parameters (XKSAT, PSIF, and DTHETA).

FIGURE 3-3

COMPOSITE VALUES OF PSIF AND DTHETA AS A FUNCTION OF $\overline{XKSAT}$

(To be used for Area Weighted Averaging of Green and Ampt Parameter Values)



7. When using the IL+ULR method, both the initial loss (STRTL) and the uniform loss rate (CNSTL) must be estimated. Because this method is to be used for special cases where infiltration is not controlled by soil texture or for drainage areas and subbasins that are predominantly sand, the estimation of the parameters will require model calibration, results of regional studies, or other valid techniques. It is not possible to provide complete guidance in the selection of these parameters, however, some general guidance is provided.

   a. Because this method is only to be used for special cases, the uniform loss rate (CNSTL) 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 value of CNSTL will probably be 1.5 millimeters per hour or less. For sand, a CNSTL of 10 to 25 millimeters per hour or larger would be reasonable. Higher values of CNSTL for sand and other surfaces are possible, however use of high values of CNSTL will require special studies.

   b. The selection of the initial loss (STRTL) can be made on the basis of calibration or special studies at the same time that CNSTL is estimated. Alternatively, since STRTL is equivalent to initial abstraction, STRTL (in millimeters) can be estimated by use of the SCS CN equations for estimating initial abstraction, written as:

$$STRTL = \frac{5080}{CN} - 50.8 \qquad (3\text{-}2)$$

Estimates of CN for the drainage area or subbasin should be made by referring to various publications of the SCS, particularly TR-55. Equation 3-2 should provide a fairly good estimate of STRTL in many cases, however its use will have to be judiciously applied and carefully considered in all cases.

3.2.2 Applications and Limitations

The Green and Ampt infiltration equation, along with an estimate of the surface retention loss can be used to estimate rainfall losses for most areas of Arizona with confidence. Most soils in Arizona are loamy sand, sandy loam, loam, or silt loam for which the Green and Ampt infiltration equation parameters from **Table 3-2** should apply. Silt, as a soil texture, is relatively rare and it is not expected that significant areas will be encountered. The finer soil textures (those with "clay" in the classification name) occur in Arizona but not usually over large areas; however, these soils have relatively low infiltration rates (XKSAT). Use of the Green and Ampt infiltration equation parameters for the finer soil textures may be somewhat conservative, and therefore their use should be appropriate for most design flood estimation purposes. Sand, as a soil texture, is also relatively rare and it has a very high infiltration rate (XKSAT). Therefore, when encountering large areas that have soils that are classified as sand, it is possible that estimates of rainfall losses with the Green and Ampt equation would be too large and the IL+ULR method should be used. Ideally, rainfall-runoff data or streamgage data would be available for model calibration of loss rate parameters in those cases. Alternatively, regional studies or extrapolation of results from similar watersheds can be used to estimate the IL+ULR parameters for sand.

In general, the Green and Ampt infiltration equation with an estimate of the surface retention loss should be used for most drainage areas in Arizona. The IL+ULR method should be used for drainage areas where soil texture does not control the infiltration rate (such as volcanic cinder) or where the soil texture of the drainage area is predominantly sand. Calibration data or results of regional studies are necessary to justify the selection of parameters for the IL+ULR method.

3.2.3 Determination of Soil Texture

The normal method to estimate infiltration losses requires the classification of soil according to soil texture (**Figure 3-1**). Two sources of information are available in Arizona to determine the soil texture. The following procedure should be applied when determining soil texture from these sources.

3.2.3.1 SCS Soil Survey: For limited areas of Arizona:

1. Locate the watershed boundaries and subbasin boundaries on the detailed soil maps.

2. List the map symbol and soil name for each soil that is contained within the watershed boundaries.

3. Read the description of each of the soil series and each mapping unit. Try to identify the soil texture that best describes each soil (or the top 150 millimeters (6 inches) of layered soils).

4. Consult soil properties tables of the soil survey, and from the columns for soil depth and dominant texture, make the final selection of soil texture that will control the infiltration rate. The size gradation data that is provided in the tables can also be used to assist in selecting the soil texture. Many of the soils in Arizona contain significant quantities of gravel, and the adjective "gravelly," when used in conjunction with the soil texture, can either be disregarded when it is used in conjunction with "sandy," that is, gravelly sandy loam can be taken as equivalent to sandy loam; or "gravelly" can be used as a replacement for "sandy" when used alone, that is, gravelly clay can be taken as equivalent to sandy clay. Similarly, adjectives such as "very fine" and "very coarse," usually used in association with sand, can be disregarded in determining soil texture classification.

3.2.3.2 General Soil Map: For each County in Arizona:

1. Locate the watershed boundaries and subbasin boundaries on the general soil map. (Since these maps are 1:500,000 scale, it may only be possible to locate the watershed.)

2. Identify the soil association(s) from the map.

3. Read the description of each soil which will identify the soil texture and soil depths.

4. Consult the soil properties tables of the general soils report, and from the columns for soil depth and texture make the final selection of soil texture that will control the infiltration rate. Comments regarding the use of adjectives such as "gravelly," and "very fine" or "very course" are the same as item 4 above.

3.3 INSTRUCTIONS

3.3.1 Green and Ampt Infiltration Equation based on Soil Texture

1. Prepare a base map of the drainage area delineating modeling subbasins, if used.

2. Delineate subareas of different soils on the base map. Determine the soil texture for 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.

4. Estimate the impervious area (RTIMP) for each subarea (**Table 3-3**).

5. Calculate the area weighted RTIMP 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 value of IA for the drainage area or each subbasin.

8. If the drainage area or subbasin consists of soil of the same textural class, then select XKSAT, PSIF, and DTHETA for that soil texture (**Table 3-2**). Proceed to Step 10.

9. If the drainage area or subbasin consists of subareas of different soil textural classes, then calculate the composite value of XKSAT (Equation 3-1), and select the composite values of PSIF and DTHETA (**Figure 3-3**).

10. Estimate the percent vegetation cover and determine the hydraulic conductivity (XKSAT) correction factor (C_k) **(Figure 3-2)**.

11. Apply correction factors (C_k) from Step 10 to the value of XKSAT from either Step 8 or Step 9.

12. The area weighted values of RTIMP, IA, XKSAT, PSIF, and DTHETA for the drainage area or each subbasin are entered on the LG record of the HEC-1 input file.

3.3.2 Initial Loss plus Uniform Loss Rate (IL+ULR)

The following method can be used only when it is known that soil texture does not control infiltration rate. This method must be used with adequate calibration or verification to justify the use of uniform loss rates that may exceed the hydraulic conductivities shown in **Table 3-2.**

1. Prepare a base map of the drainage area delineating modeling subbasins, if used.

2. Delineate subareas of different infiltration rates (uniform loss rates) on the base map. 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.

4. Estimate the impervious area (RTIMP) for the drainage area or each subarea **(Table 3-3)**.

5. Estimate the initial loss (STRTL) for the drainage area or each subarea by regional studies or calibration. Alternatively, Equation 3-2 can be used to estimate or to check the value of STRTL.

6. Estimate the uniform loss rate (CNSTL) for the drainage area or each subarea by regional studies or calibration.

7. Calculate the area weighted values of RTIMP, STRTL, and CNSTL for the drainage area or each subbasin.

8. The area weighted values of RTIMP, STRTL, and CNSTL for the drainage area or each subbasin are entered on the LU record of the HEC-1 input file.

EXAMPLE 3-1

ESTIMATION OF RAINFALL LOSS PARAMETERS FOR GREEN AND AMPT METHOD, YOUNGTOWN, ARIZONA

Problem:

The rainfall loss parameters are estimated for a 0.34 square kilometers drainage area in Youngtown, Arizona. A drainage area is delineated on a topographic map, as shown in the accompanying figure. The drainage area is nearly all single-family residential with about a 0.1 hectare or slightly smaller lot size. About 50 percent of the residential lots are irrigated turf, although some lawns are in poor condition and the vegetation cover is estimated as 75 percent. The other 50 percent of the residential lots are desert landscaped.

The rainfall loss parameters are estimated as follows:

1. RTIMP is 30 percent for a 0.1 hectare lot size **(Table 3-3)**.

2. IA is based on 50 percent lawn (IA = 5 mm) and 50 percent desert landscape (IA = 3 mm) **(Table 3-1)**. The area-weighted IA is:

   IA = (5)(.50) + (3)(.50) = 4 mm.

3. The soil composition of the watershed and soil texture classifications are as follows:

Soil Symbol	Soil Name	Hydrologic Soil Group	Soil Texture	XKSAT (Table 3-2)	% Area
LcA	Laveen loam	B	loam	6.5	50
PeA	Perryville gravelly loam	B	sandy loam	11	38
Vf	Vecont clay	D	clay	.3	12

4. The composite value of XKSAT is calculated (Equation 3-1):

$$\overline{XKSAT} = \text{antilog}\ [(.50)\log 6.5 + (.38)\log 11 + (.12)\log .3]$$

$$\overline{XKSAT} = 5.5\ \text{mm/hr}$$

5. The composite values of PSIF and DTHETA are estimated (**Figure 3-3**):

PSIF = 138 mm

DTHETA = .25 for lawn (50%)

= .36 for desert landscaping (50%)

$\overline{\text{DTHETA}}$ = (.25)(.50) + (.36)(.50)

= .31

6. The vegetation correction factor (C_k) (**Figure 3-2**) is calculated based on 50 percent lawn at 75 percent cover.

VC = (.50)(75) = 38 percent

C_k = .011(38) + .89

= 1.31

7. The $\overline{\text{XKSAT}}$ is adjusted for vegetation cover:

XKSAT = (1.31)(5.5) = 7.2 mm/hr

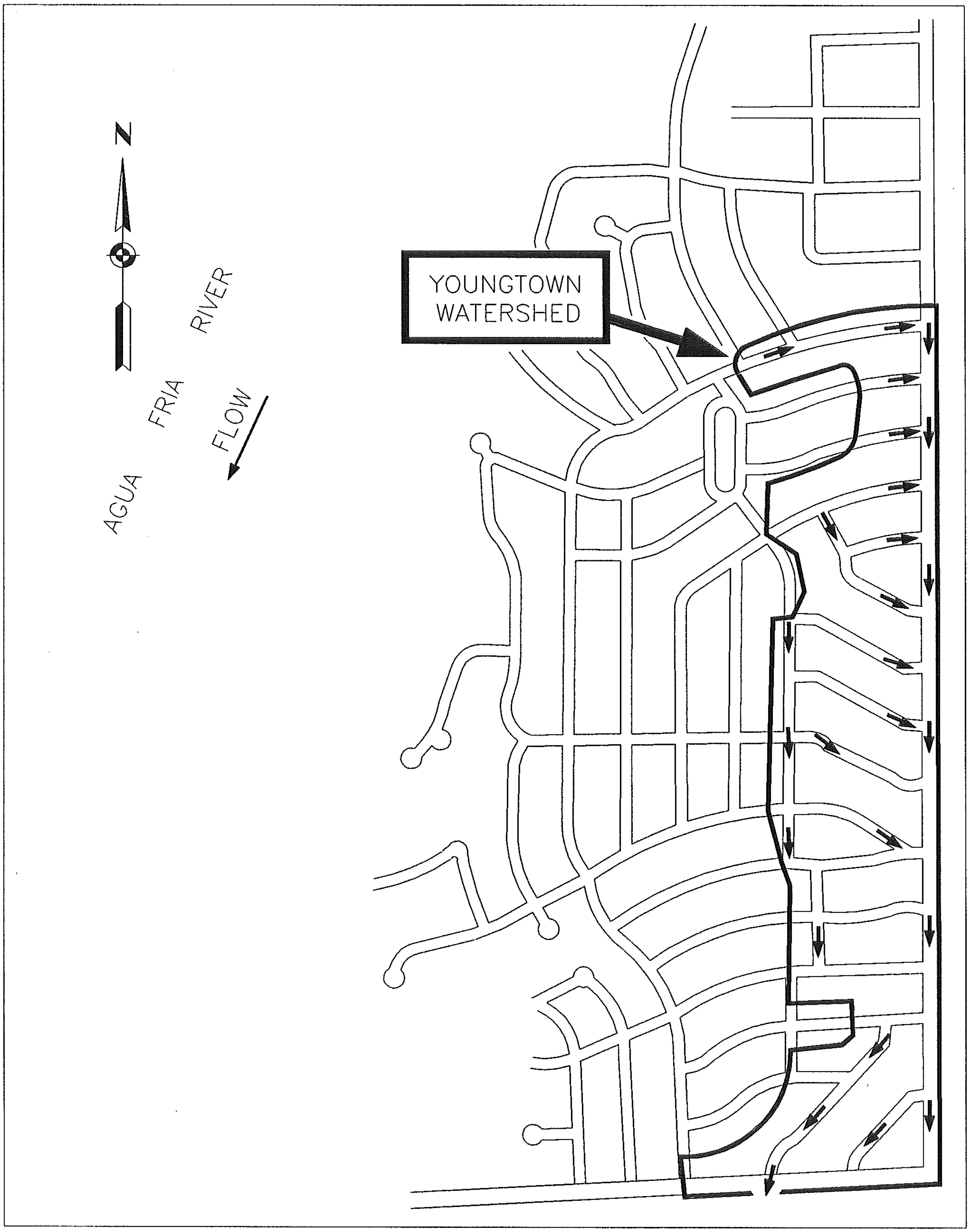
8. The LG record is coded as follows:

LG, IA, DTHETA, PSIF, XKSAT, RTIMP

LG, 4, .31, 138, 7.2, 30

YOUNGTOWN WATERSHED

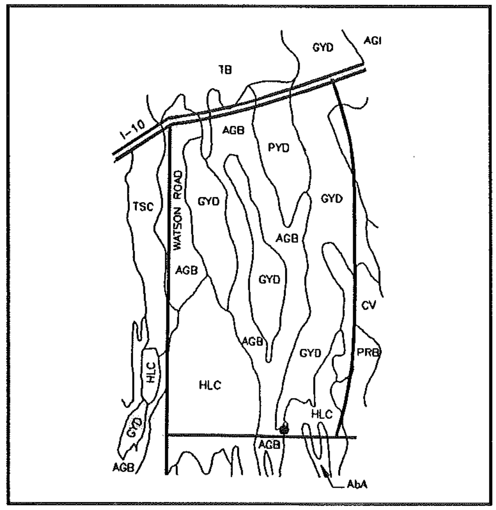
(Example 3-1)



AREA WEIGHTED AVERAGE GREEN AND AMPT PARAMETERS FOR THE SUBBASIN NEAR BUCKEYE, ARIZONA

Problem:

Determine the area weighted average Green and Ampt parameters for the subbasin near Buckeye, Arizona. Adjust XKSAT for 20 percent vegetation coverage.



Solution:

Use of the SCS Soil Survey of Maricopa County, Arizona, Central Part and planimetering of subareas result in the following:

Soil Symbol	Soil Name	Textural Class	XKSAT mm/hr (Table 3-2)	Area Sq. Km.
GYD	Gunsight - Rillito Complex	Sandy Loam	11.0	0.83
AGB	Antho - Carizo Complex	Sandy Loam	11.0	0.75
HLC	Harqua - Gunsight Complex	Clay Loam	1.0	0.62
PYD	Pinamt - Tremant Complex	Sandy Clay Loam	1.5	0.18
CY	Coolidge - Laveen Association	Sandy Loam	11.0	0.05
TSC	Tremant - Rillito Complex	Sandy Clay Loam	1.5	0.05
PRB	Perryville - Rillito Complex	Sandy Loam	11.0	0.03
TB	Torrifluvents	Loamy Sand	30.0	0.03
AbA	Antho Sandy Loam	Sandy Loam	11.0	0.03

Total Area = 2.57

Area of Sandy Loam (XKSAT = 11) = 1.69
Area of Sandy Clay (XKSAT = 1.5) = 0.23
Area of Clay Loam (XKSAT = 1.0) = 0.62
Area of Loamy Sand (XKSAT = 30) = 0.03

$$\overline{XKSAT} = \text{antilog}\left[\frac{1.69(\log 11) + .23(\log 1.5) + .62(\log 1.0) + .03(\log 30)}{2.57}\right]$$

$$\overline{XKSAT} = 5.2 \text{ mm/hr}$$

PSIF = 141 mm **(Figure 3-3)**
DTHETA (dry) = .37 **(Figure 3-3)**
XKSAT (adjusted by **Figure 3-2**) = 5.2[.011(20) + .89] = 5.7 mm/hr

CHAPTER 4
UNIT HYDROGRAPHS

4.1 INTRODUCTION

4.1.1 General Discussion

A unit hydrograph is defined as the hydrograph of 10 millimeters of direct runoff from a storm of a specified duration for a particular watershed. Every watershed will have a different unit hydrograph that reflects the physiography, topography, land-use, and other unique characteristics of the individual watershed. 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 a rainfall excess duration of 5-minutes, or 15-minutes, or 1-hour, or 6-hours, etc. Any duration can be selected for unit hydrograph development as long as an upper limit for the unit hydrograph duration is not exceeded. Guidelines for the determination of the upper limit of unit hydrograph duration are provided in a later section.

Only a few watersheds in Arizona will have an adequate data base (rainfall and runoff records) from which to develop unit hydrographs. Therefore, indirect methods usually will be used to develop unit hydrographs. Such unit hydrographs are called 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 data base 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 (or modeling subbasin) to the watershed outlet (or modeling concentration point). The synthetic unit hydrograph procedure that is recommended is the Clark unit hydrograph. Procedures are provided, herein, to estimate the three Clark unit hydrograph parameters and these are entered on the UC and UA records of HEC-1. Unit hydrograph procedures other than the Clark procedure can be used for specific applications, however, this will require justification and approval by ADOT for such use.

4.2 PROCEDURE

4.2.1 General Considerations

The Clark unit hydrograph requires the estimation of three parameters; the time of concentration (T_C), the storage coefficient (R), and a time-area relation. Sub-sections 4.2.1.1 through 4.2.1.4 describe the procedures that are to be used to calculate these parameters, and the guidelines that are to be used to select the unit hydrograph duration and computation interval (NMIN).

4.2.1.1 Time of Concentration: Time of concentration is the travel time, during the corresponding period of most intense rainfall excess, for a floodwave 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

$$T_c = 1.2\, A^{.1}\, L^{.25}\, L_{ca}^{.25}\, S^{-.2} \qquad (4\text{-}1)$$

agricultural fields

$$T_c = k\, A^{.1}\, L^{.25}\, L_{ca}^{.25}\, S^{-.2} \qquad (4\text{-}2)$$

urban

$$T_c = 1.8\, A^{.1} L^{.25}\, L_{ca}^{.25}\, S^{-.14}\, RTIMP^{-.36} \qquad (4\text{-}3)$$

where T_c	=	time of concentration, in hours
A	=	area, in square kilometers
S	=	watercourse slope, in m/km
L	=	length of the watercourse to the hydraulically most distant point, in kilometers
L_{ca}	=	length measured from the concentration point along L to a point on L that is perpendicular to the watershed centroid, in kilometers
RTIMP	=	effective impervious area, in percent, and
k	=	agricultural field T_c factor as discussed below in item 7.

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 needs to 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 from this Manual, can produce runoff depths that can cross these intermediate drainage boundaries resulting in a larger total contributing area. Similarly, floods on alluvial fans (active and inactive) and in distributary flow systems can result in increased contributing areas during larger and more intense storms. For such areas, it is generally prudent to consider the largest <u>reasonable</u> drainage area in these situations.

2. Determination of the hydraulically most distant point 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 is generally true where the topography is relatively uniform throughout the watershed. However, there are situations where 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 where special consideration should be given to calculating S and to dividing the watershed into subbasins. For example, if there is dramatic change in watercourse slope throughout 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 where the watercourse contains vertical or nearly vertical drops (mountain rims, headcuts, rock outcrop, and so forth). 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 are nearly rectangular in shape and this may result 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 ½L.

5. RTIMP is the effective impervious area. This is the same value that was determined for the watershed by the procedures 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, and for those situations, the T_c 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 (4-1 through 4-3) is selected based on the watershed type that contains the greatest portion of L. The effects of a

mixture of watershed types is accounted for by the selection of the time-area relation (to be discussed in a later section).

7. When using the T_C equation for agricultural fields (Equation 4-2), the factor k must be selected based on a qualitative evaluation of the hydraulics of surface runoff through the agricultural area. In general, k ranges from a minimum of about 1.2 to a maximum of about 3.6. A low value of k (approximately 1.2) is used if the following combination of conditions is present in the field: the field is at natural grade (not leveled), irrigation berms are <u>not</u> normally present (irrigated by methods other than flooding), cropping patterns are such as to offer little additional resistance to sheetflow, and a well defined form of surface drainage network exists. Large, permanent pastures that are irrigated by subirrigation or sprinklers would typically have low k values.

   Conversely, a high value of k (approximately 3.6) is used if the following combination of conditions is present in the field: the field has been leveled and land terraces exist, fields are typically bermed for irrigation purposes, crops are typically planted in furrows or the cropping patterns offer substantial resistance to sheetflow, and the surface drainage network is nonexistent or poorly defined. Often, cotton fields represent a case where high values of k should be used. Judgement must be used in selecting the most appropriate value of k. For example, a field reconnaissance may indicate fallow fields with no irrigation berms; however, if the area has a vibrant agricultural economy (particularly areas of double cropping during the year) then it can probably be safely assumed that typical agricultural practices should be assumed in the evaluation. Care must be used in agricultural areas that are adjacent to urban expansion. In those cases, agricultural practices can cease for many years prior to urban development leaving the fields fallow (with subsequent low k values). In such cases, it may be prudent, or required, to treat the agricultural areas as urban under the assumption of reasonable future conditions.

4.2.1.2 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:

$$R = 0.44 \; T_c^{1.11} L^{.80} A^{-.57} \tag{4-4}$$

where R is in hours and the variables are as defined for the T_C equations.

4.2.1.3 Time-Area Relation: The time-area relation is a graphical parameter that specifies the accumulated area of the watershed that is contributing runoff to the outlet of the watershed at any time. Two methods can be used to develop a time-area relation: 1) by analysis of the watershed to define incremental runoff producing areas that have equal incremental travel times to the outflow location, or 2) by use of synthetic time-area relations. The development of a time-area relation by analysis of the watershed is a difficult task and well-defined and reliable procedures for this task are not available. Unless the watershed has an extremely unusual shape, or has several distinct areas of dramatically different land-use, this analysis should not be undertaken. In general, synthetic time-area relations can be used in Arizona.

The dimensionless, synthetic time-area relations that can be used in Arizona are shown in **Figure 4-1** and the coordinate values of the curves are listed in **Table 4-1**. Curve A should be used if the land-use in the watershed or subbasin is urban or predominantly urban. Curve C should be used if the land-use in the watershed or subbasin is desert/rangeland or is mostly desert/rangeland with some mountains in the watershed and/or some irrigated agricultural fields interspersed in the lowlands. Curve B should be used for all other situations.

Curve B is the default time-area relation in HEC-1 and will be used with the Clark unit hydrograph if a time-area relation (UA record) is not supplied. Curves A and C are dimensionless and these curves are input to HEC-1 by inserting the percent of total area values from **Table 4-1** in the UA record.

TABLE 4-1

VALUES OF THE DIMENSIONLESS SYNTHETIC TIME-AREA RELATIONS FOR THE CLARK UNIT HYDROGRAPH

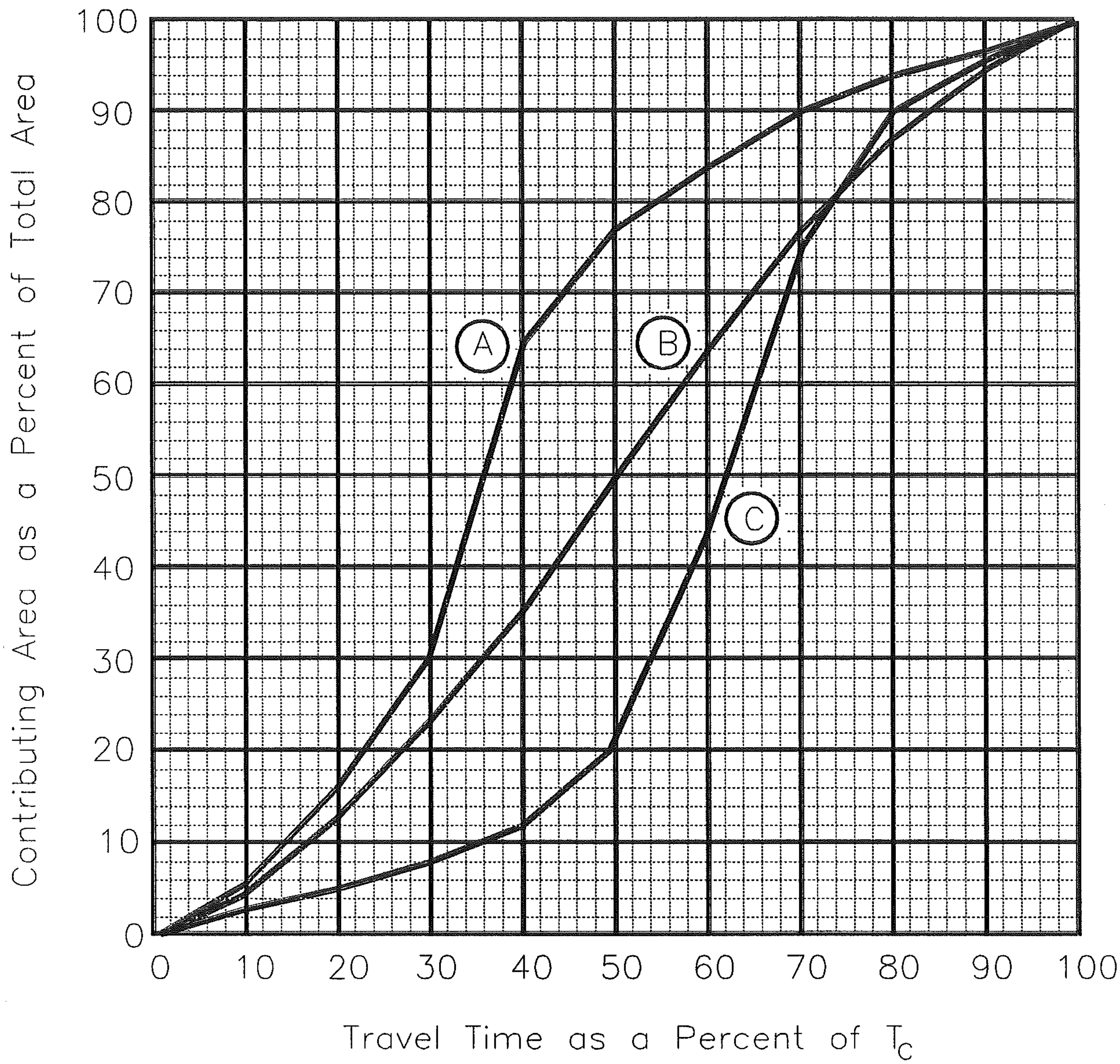
	Contributing Area, as a Percent of Total Area[a]		
Travel Time, as a percent of T_c	A	B[b]	C
(1)	(2)	(3)	(4)
0	0	0.0	0
10	5	4.5	3
20	16	12.6	5
30	30	23.2	8
40	65	35.8	12
50	77	50.0	20
60	84	64.2	43
70	90	76.8	75
80	94	87.4	90
90	97	95.5	96
100	100	100.0	100

[a] The dimensionless Synthetic Time-Area relations should be selected as follows:
A - The land-use in the watershed or subbasin is urban or predominantly urban.
B - All watersheds or subbasins other than those defined for use of curves A or C.
C - The land-use in the watershed or subbasin is desert/rangeland or is mostly desert/rangeland with some mountains in the watershed and/or some irrigated agricultural fields interspersed in the lowlands.

[b] Curve B is the HEC-1 default Time-Area relation and the UA record is not needed as input to the HEC-1 model.

FIGURE 4-1

SYNTHETIC TIME-AREA RELATION



Curve A– Urban
Curve B– HEC–1 Default Relation
Curve C– Desert/Rangeland

4.2.1.4 Duration: The duration of the unit hydrograph (or all unit hydrographs in a multiple subbasin model) is specified in HEC-1 in the IT record as NMIN. In general, NMIN will be selected according to the following criteria:

NMIN = 2 minutes for a 6-hour storm duration (drainage area less than or equal to 2.5 square kilometers), and

NMIN = 5 minutes for a 24-hour storm duration (drainage area greater than 2.5 square kilometers).

Note: NMIN should not exceed .25 T_c for the subbasin with the shortest T_c.

However, there may be special situations (see Chapter 8 Modeling Techniques and General Guidance for using HEC-1, 8.2.4.3) where a NMIN, other than as defined above, is to be used. In those situations, the following rules should be considered:

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

2. NMIN = 0.25 T_c is the maximum value for NMIN.

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

4.2.2 Applications and Limitations

The Clark unit hydrograph, as described herein, 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 unit hydrograph, and in those cases, the developed unit hydrograph would be input to HEC-1 by use of UI records. 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. However, deviations from the procedures in this Manual should be discussed with ADOT and approval received for deviations from the recommended procedures before incorporating such deviations into the project hydrology analysis.

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 (25-year to 100-year). 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 is 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.

4.3 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:

   urban
   desert/rangeland
   mountain
   irrigated agriculture

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, etc.),

f. shape of the watershed, and

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 kilometers

L - length of the flow path to the hydraulically most distant point, in kilometers

L_{ca} - length along L to a point opposite the centroid, in kilometers

S - average slope of L, in m/km

RTIMP- effective impervious area, in percent.

7. Calculate T_c depending on the type of watershed:

desert/mountain

$$T_c = 1.2\ A^{.1}\ L^{.25}\ L_{ca}^{.25}\ S^{-.2}$$

agricultural fields

$$T_c = k\ A^{.1}\ L^{.25}\ L_{ca}^{.25}\ S^{-.2}$$

where k ranges from a minimum of about 1.2 to a maximum of about 3.6

urban

$$T_c = 1.8\ A^{.1}\ L^{.25}\ L_{ca}^{.25}\ S^{-.14}\ RTIMP^{-.36}$$

8. Calculate R:

$$R = 0.44\ T_c^{1.11}\ L^{.80}\ A^{-.57}$$

9. Enter the values of T_c and R in the UC record for the watershed or each subbasin.

10. Determine whether the time-area relation will be developed from an analysis of the watershed or whether a dimensionless synthetic time-area relation will be used.

    a. If the time-area relation is to be determined by analytic means, proceed with the analysis and input the incremental areas (or percentages of total area) in the UA record.

    b. If the dimensionless synthetic time-area relations are to be used (**Figure 4-1 and Table 4-1**),

        i. use the values for Curve A in the UA record if the watershed or subbasin is urban or predominantly urban,

        ii. use the values for Curve C in the UA record if the watershed or subbasin is desert/rangeland or is mostly desert/rangeland with some mountains and/or some irrigated agricultural fields interspersed in the lowlands, and

        iii. use Curve B for all other applications (Curve B is the HEC-1 default relation and the UA record is not needed).

EXAMPLE 4-1

CLARK UNIT HYDROGRAPH PARAMETERS

FOR RANGELAND WATERSHED

Problem:

Develop the Clark unit hydrograph parameters for the Walnut Gulch Experimental Watershed 63.011 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:

   $A = 8.24$ square kilometers

   $L = 6.44$ kilometers

   $L_{ca} = 2.90$ kilometers

   $S = 18.9$ m/km

3. The watershed is desert/rangeland.

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

   $$T_c = 1.2\,A^{.1}\,L^{.25}\,L_{ca}^{.25}\,S^{-.2}$$

   $$T_c = 1.2(8.24^{.1})(6.44^{.25})(2.90^{.25})(18.9^{-.2})$$

   $$T_c = 1.71 \text{ hr}$$

5. Calculate R:

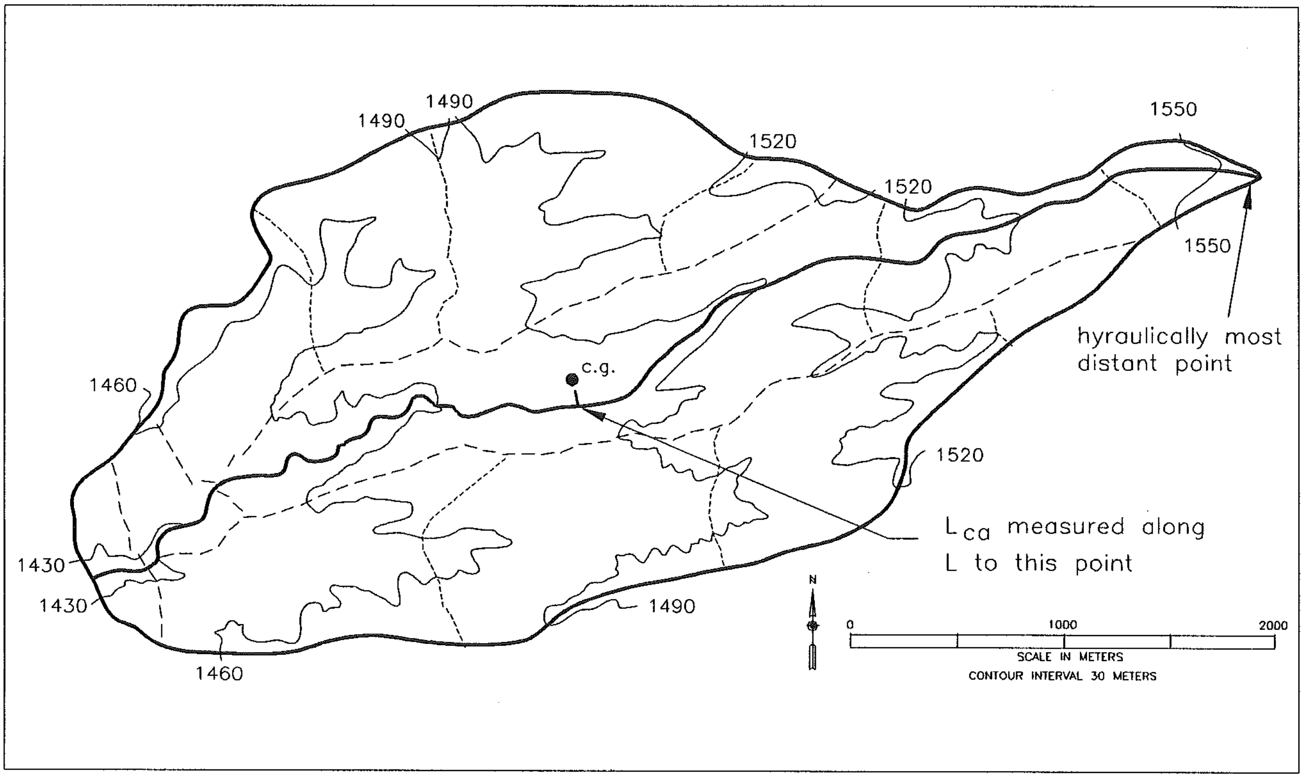
   $$R = 0.44\,T_c^{1.11}\,L^{.80}\,A^{-.57}$$

   $$R = 0.44\,(1.71^{1.11})(6.44^{.80})(8.24^{-.57})$$

   $$R = 1.06 \text{ hr}$$

6. The desert/rangeland dimensionless synthetic time-area relation (Curve C) is used.

MAP FOR WALNUT GULCH EXPERIMENTAL WATERSHED 63.011
NEAR TOMBSTONE, ARIZONA



EXAMPLE 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:

   A = 21.03 square kilometers

   L = 9.98 kilometers

   L_{ca} = 4.35 kilometers

   S = 7.1 m/km

   RTIMP = 20.2%

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:

   $$T_c = 1.8\, A^{.1}\, L^{.25}\, L_{ca}^{.25}\, S^{-.14}\, RTIMP^{-.36}$$

   $$T_c = 1.8\, (21.03^{.1})(9.98^{.25})(4.35^{.25})(7.1^{-.14})(20.2^{-.36})$$

   $$T_c = 1.61 \text{ hr}$$

5. Calculate R:

   $$R = 0.44\, T_c^{1.11}\, L^{.80}\, A^{-.57}$$

   $$R = 0.44\, (1.61^{1.11})(9.98^{.80})(21.03^{-.57})$$

   $$R = 0.83 \text{ hr}$$

6. The urban dimensionless synthetic time-area relation (Curve A) is used.

MAP FOR TUCSON ARROYO WATERSHED

TUCSON, ARIZONA

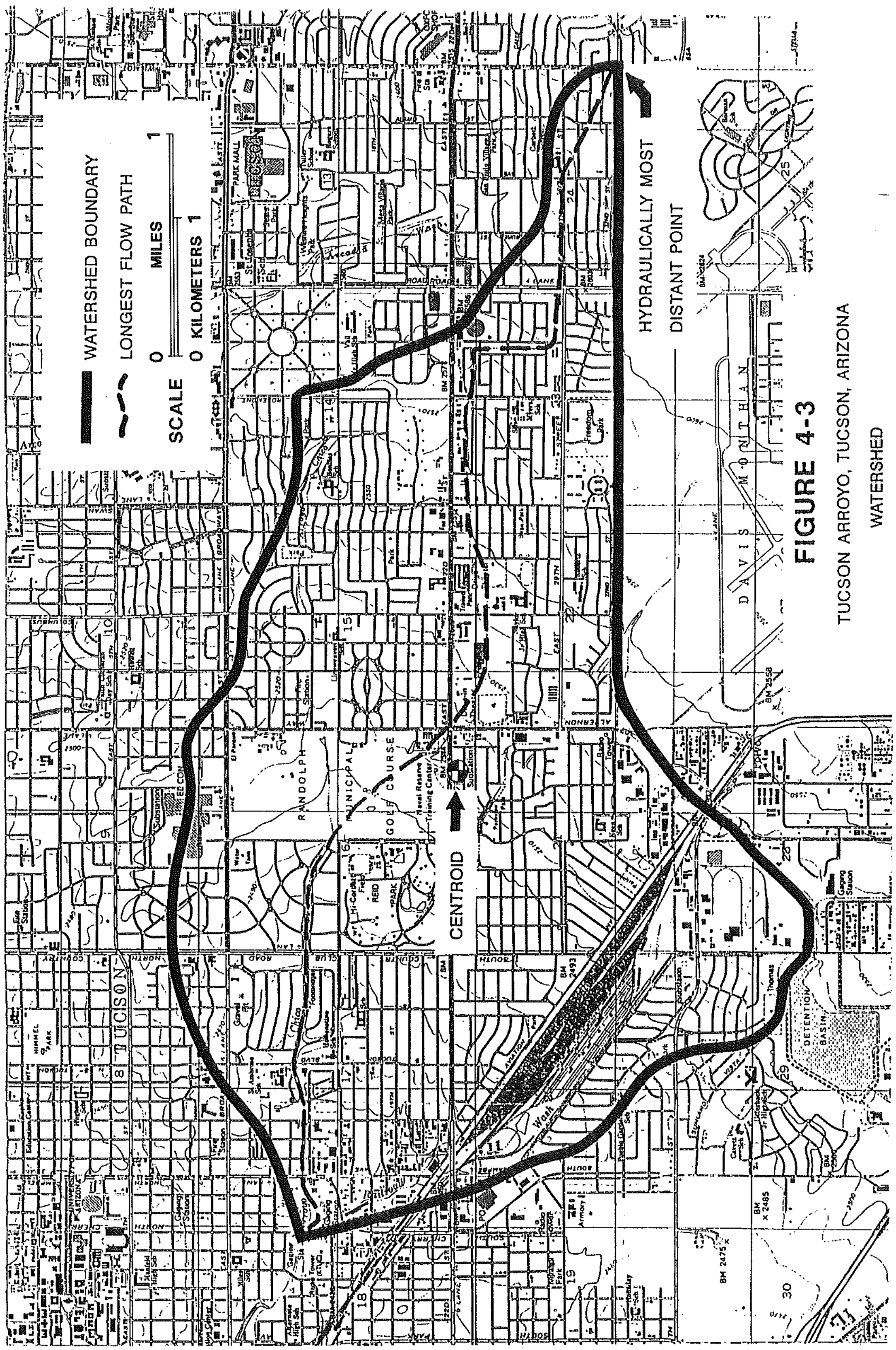


FIGURE 4-3

TUCSON ARROYO, TUCSON, ARIZONA WATERSHED

CHAPTER 5
CHANNEL ROUTING

5.1 INTRODUCTION

5.1.1 General Discussion

Channel routing describes the movement of a flood wave (hydrograph) 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 river reach. Channel routing is used in flood hydrology models, such as HEC-1, when the watershed is modeled with multiple subbasins and runoff from the upper subbasins must be routed through a channel, or system of channels, to the watershed outlet. Several methods are available for channel routing. The method that is recommended for the majority of channel routing applications for highway drainage in Arizona is the Normal Depth method.

5.2 PROCEDURE

The recommended procedure for routing is the Normal Depth method and that method should be used unless there is good cause for deviation from this recommendation. The following procedure is for the Normal Depth method, however, the information can often be used to assist in defining routing input for other methods.

For Normal Depth routing, data must be provided for the number of steps in the routing calculation, the initial condition of the flow in the channel, channel resistance coefficients, and channel geometry. Much of this data is normally obtained from appropriate maps and/or field survey data.

5.2.1 General Considerations

5.2.1.1 Number of Computation Steps (NSTPS): This is the number of computation steps that will be used in the Normal Depth routing calculation. The Normal Depth route operation in HEC-1 is accomplished by use of a single 8-point cross section which is selected to be typical of the routing reach. Storage routing is accomplished by using wedge-storage for subreaches. The subreach length is the distance traveled by the flood wave during one computation time interval (NMIN). The number of necessary subreaches corresponds to NSTPS, which must be an integer. NSTPS can be estimated by reach length/average velocity/NMIN. (See Chapter 8 Modeling Techniques and General Guidance for using HEC-1, 8.2.4.5, for additional guidance in selecting NSTPS.)

5.2.1.2 Initial Flow Condition (ITYP and RSVRIC): These define the initial condition of the flow in the channel at the start of the routing computation. Normally the initial condition that is used is the discharge in the channel and this will often be 0 (dry channel) for channels in Arizona. If the channel is expected to have flow in the channel prior to the modeled storm, or a baseflow, then use the appropriate discharge data. The channel water surface elevation at the start of the routing computation can be used, if desired instead of initial discharge conditions.

5.2.1.3 Routing Reach Length (RLNTH): This is the length of the channel or major flow path. The length will be measured on the best available map. The units of RLNTH are meters.

5.2.1.4 Energy Grade Line Slope (SEL): This is the slope of the energy grade line and is not normally known. For normal flow, it is parallel to the channel bed slope. It is usually estimated as the channel bed slope, calculated by dividing the difference in bed elevation between the upper and lower ends of the watercourse by the routing reach length. The units of SEL are m/m.

5.2.1.5 Manning's Roughness Coefficient (n): The 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 n for a channel can be computed by

$$n = (n_0 + n_1 + n_2 + n_3 + n_4)\, m_5 \tag{5-1}$$

where n_0 is the base value for a straight, uniform, stable channel, n_1 is a value for the effect of surface irregularities, n_2 is a value to account for obstructions to flow, n_3 is a value for vegetation, n_4 is a value to account for variations in channel cross section, and m_5 is a correction factor to account for meandering of the main channel.

The value for n_0 can be selected from **Table 5-1**. The adjustment factors (n_1, n_2, n_3, n_4, and m_5) can be selected from **Table 5-2**.

For overbank floodplains, the value of n is selected from **Table 5-3**.

The Manning's roughness coefficient for the main channel is designated as ANCH, for the left overbank it is ANL, and for the right overbank it is ANR according to HEC-1 nomenclature.

5.2.1.6 Channel Geometry: 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. Considerable judgement is necessary in defining the representative 8-point cross section. The guidance in the HEC-1 User's Manual should be followed when defining an 8-point cross section. The coordinates (X and Y) can be to any base datum. Specifically, the vertical dimensions (Y) do not need to correspond to land surface elevation or any elevation for any location along the routing reach.

TABLE 5-1

BASE VALUES (n_0) OF MANNING'S ROUGHNESS COEFFICIENT FOR STRAIGHT, UNIFORM, STABLE CHANNELS

(from Thomsen and Hjalmarson, 1991)

Channel Material	Size of Bed Material		Base Values, n_0	
	Millimeters	Inches	Benson and Dalrymple (1967)[a]	Chow (1959)[b]
Concrete	-------	-------	0.012-0.018	0.011
Rock Cut	-------	-------	-----------	.025
Firm Soil	-------	-------	.025- .032	.020
Coarse Sand	1-2	-------	.026- .035	-----
Fine Gravel	-------	-------	-----------	.024
Gravel	2-64	0.08- 2.5	.028- .035	-----
Coarse Gravel	-------	-------	-----------	.028
Cobble	64-256	2.50-10.0	.030- .050	-----
Boulder	>256	>10.0	.040- .070	-----

[a] Straight uniform channel.

[b] Smoothest channel attainable in indicated material.

TABLE 5-2

ADJUSTMENT FACTORS (n_1, n_2, n_3, n_4 and m_5) FOR THE DETERMINATION OF OVERALL MANNING'S n VALUE

(from Thomsen and Hjalmarson, 1991)

Channel Conditions	Manning's n adjustment[a]	Example
Degree of irregularity:	n_1	
Smooth	0.000	Smoothest channel attainable in given bed material.
Minor	.001 - .005	Channels with slightly eroded or scoured side slopes.
Moderate	.006 - .010	Channels with moderately sloughed or eroded side slopes.
Severe	.011 - .020	Channels with badly sloughed banks; unshaped, jagged, and irregular surfaces of channels in rock.
Effects of obstruction[b]:	n_2	
Negligible	.000 - .004	A few scattered obstructions, which include debris deposits, stumps, exposed roots, logs, piers, or isolated boulders, that occupy less than 5 percent of the cross-sectional area.
Minor	.005 - .015	Obstructions occupy 5 to 15 percent of the cross-sectional area and the spacing between obstructions is such that the sphere of influence around one obstruction does not extend to the sphere of influence around another obstruction. Smaller adjustments are used for curved smooth-surfaced objects than are used for sharp-edged angular objects.
Appreciable	.020 - .030	Obstructions occupy from 15 to 50 percent of the cross-sectional area or the space between obstructions is small enough to cause the effects of several obstructions to be additive, thereby blocking an equivalent part of a cross section.
Severe	.040 - .060	Obstructions occupy more than 50 percent of the cross-sectional area or the space between obstructions is small enough to cause turbulence across most of the cross section.

a Adjustments for degree of irregularity, variations in cross section, effect of obstructions, and vegetation are added to the base n value before multiplying by the adjustment for meander.

b Conditions considered in other steps must not be reevaluated or duplicated in this section.

Channel Conditions	Manning's n adjustment[c]	Example
Vegetation:	n_3	
Small	.002 - .010	Dense growths of flexible turf grass, such as Bermuda, or weeds where the average depth of flow is at least two times the height of the vegetation; supple tree seedlings such as willow, cottonwood, arrow weed, or saltcedar, where the average depth of flow is at least three times the height of the vegetation.
Medium	.010 - .025	Grass or weeds where the average depth of flow is from one to two times the height of the vegetation; moderately dense stemmy grass, weeds, or tree seedlings, where the average depth of flow is from two to three times the height of the vegetation; moderately dense brush, similar to 1- to 2-year-old saltcedar in the dormant season, along the banks and to no significant vegetation along the channel bottoms where the hydraulic radius exceeds 600 millimeters.
Large	.025 - .050	Turf grass or weeds where the average depth to flow is about equal to the height of vegetation; small trees intergrown with some weeds and brush where the hydraulic radius exceeds 600 millimeters.
Very Large	.050 - .100	Turf grass or weeds where the average depth of flow is less than half the height of vegetation; small bushy trees intergrown with weeds along side slopes of dense cattails growing along channel bottom; trees intergrown with weeds and brush.
Variations in channel cross section:	n_4	
Gradual	.000	Size and shape of cross sections change gradually.
Alternating (Occasionally)	.001 - .005	Large and small cross sections alternate occasionally, or the main flow occasionally shifts from side to side owing to changes in cross-sectional shape.
Alternating (Frequently)	.010 - .015	Large and small cross sections alternate frequently, or the main flow frequently shifts from side to side owing to changes in cross-sectional shape.

[c] Adjustments for degree of irregularity, variations in cross section, effect of obstructions, and vegetation are added to the base n value before multiplying by the adjustment for meander.

Channel Conditions	Manning's n adjustment[d]	Example
Degree of meandering[e]:	m_5	
Minor	1.00	Ratio of the meander length to the straight length of the channel reach is 1.0 to 1.2.
Appreciable	1.15	Ratio of the meander length to the straight length of the channel is 1.2 to 1.5.
Severe	1.30	Ratio of the meander length to the straight length of the channel is greater than 1.5.

[d] Adjustments for degree of irregularity, variations in cross section, effect of obstructions, and vegetation are added to the base n value before multiplying by the adjustment for meander.

[e] Adjustment values apply to flow confined in the channel and do not apply where downvalley flow crosses meanders. The adjustment is a multiplier.

TABLE 5-3

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	.030	.035	.050
Cultivated areas:			
No crop	.020	.030	.040
Mature row crops	.025	.035	.045
Mature field crops	.030	.040	.050
Brush:			
Scattered brush, heavy weeds	.035	.050	.070
Light brush and trees, in winter	.035	.050	.060
Light brush and trees, in summer	.040	.060	.080
Medium to dense brush, in winter	.045	.070	.110
Medium to dense brush, in summer	.070	.100	.160
Trees:			
Dense willows, summer, straight	.110	.150	.200
Cleared land with tree stumps, no sprouts	.030	.040	.050
Same as above, but heavy growth of sprouts	.050	.060	.080
Heavy stand of timber, a few down trees, little undergrowth, flood stage below branches	.080	.100	.120
Same as above, but with flood stage reaching branches	.100	.120	.160

5.2.2 Applications and Limitations

Channel routing is to be used in multiple subbasin models when the runoff from the upper subbasins passes through a watercourse, or a system of watercourses, to the watershed outlet. 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.

The Normal Depth method, that is available in the HEC-1 program, is usually an appropriate routing method for use in watercourses in Arizona. It should be used where routing effects (peak attenuation and delay) are expected. Other methods may be more appropriate or more practical in certain applications. For example, the Kinematic Wave channel routing method can often be used with comparable accuracy for constructed urban channels, including storm drains, and for short, steep natural channels. The Muskingum method may be appropriate for certain rivers if data are available to determine the two parameters (K and X) by analysis, or by HEC-1 optimization from recorded hydrographs, or if other information is available to yield reliable estimates of K and X. The Muskingum-Cunge method is also available and it can be used in certain applications. However, the Muskingum-Cunge method can produce unreliable results, particularly for wide, shallow watercourses, especially with steep slopes. The use of the Muskingum-Cunge method must be applied with caution, and results carefully reviewed before acceptance. Also, the Muskingum-Cunge method is not amenable for channel routing if channel transmission losses (by the recommended method, see Chapter 7 - Transmission Losses) are to be included in the watershed model. In general, however, the Normal Depth method is to be used.

One of the most critical aspects of watershed modeling using subbasins and channel routing is the selection of channel routing lengths (RLNTH). The numeric procedure used in routing calculations requires that the travel time through each routing reach be a multiple of the selected computation interval (NMIN). For this reason, the selection of too short a RLNTH could result in the computation of zero travel time through the routing reach (instantaneous translation of the flood wave through the reach). This could result in erroneously large peak discharges at downstream concentration points in the watershed model. A watershed model of numerous small subbasins and connecting short routing reaches can result in progressively larger overestimation of peak discharges in a downstream direction producing grossly overestimated peak discharge at the watershed outlet. Chapter 8 - Modeling Techniques and General Guidance for using HEC-1, 8.2.4.5, should be consulted prior to watershed delineation to avoid problems with channel routing lengths that are too short.

5.3 INSTRUCTIONS

The following steps should be used with the Normal Depth routing method:

1. From the watershed base map, identify the routing reaches. (See Chapter 8 - Modeling Techniques and General Guidance for using HEC-1, 8.2.4.5 for additional guidance.)

2. Compile information on the characteristics of those reaches (detailed topographic maps to define channel geometry, photographs of the channels and overbanks, other hydrologic reports for the area, etc.)

3. Conduct a field reconnaissance of the watershed and routing reaches, if practical. Observe and note the characteristics of the routing reaches; variations in the channel cross sections, irregularity of the channel, and degree of meandering of the main channel. Determine the hydraulically representative section of the routing reaches. Make note of and photograph the representative sections paying particular attention to flow resistance characteristics; bed material, obstructions to flow (rock outcrop, boulders, debris, etc.), and vegetation in the channel and overbank floodplains. If adequate maps are not available to define the channel geometry of the representative sections, field surveys or field measurements can be made of the channel and overbank floodplains.

4. Prepare a sketch of the representative section of each routing reach, and prepare the channel geometry input (RX and RY records).

5. Estimate the main channel roughness coefficient, ANCH, by use of Equation 5-1:
   a. select the base value, n_0, from **Table 5-1**, and
   b. select the adjustment factors, n_1, n_2, n_3, n_4, and m_5 from **Table 5-2**.

6. If an 8-point cross section is used that contains overbank floodplains, select the n for each of the overbanks (ANL and ANR) from **Table 5-3**.

7. Measure the routing reach length, RLNTH, from the base map.

8. Estimate the energy gradient (SEL), by calculating the channel bed slope from the base map.

9. Input the routing information into the RS, RC, RX and RY records.

EXAMPLE 5-1
NORMAL DEPTH CHANNEL ROUTING

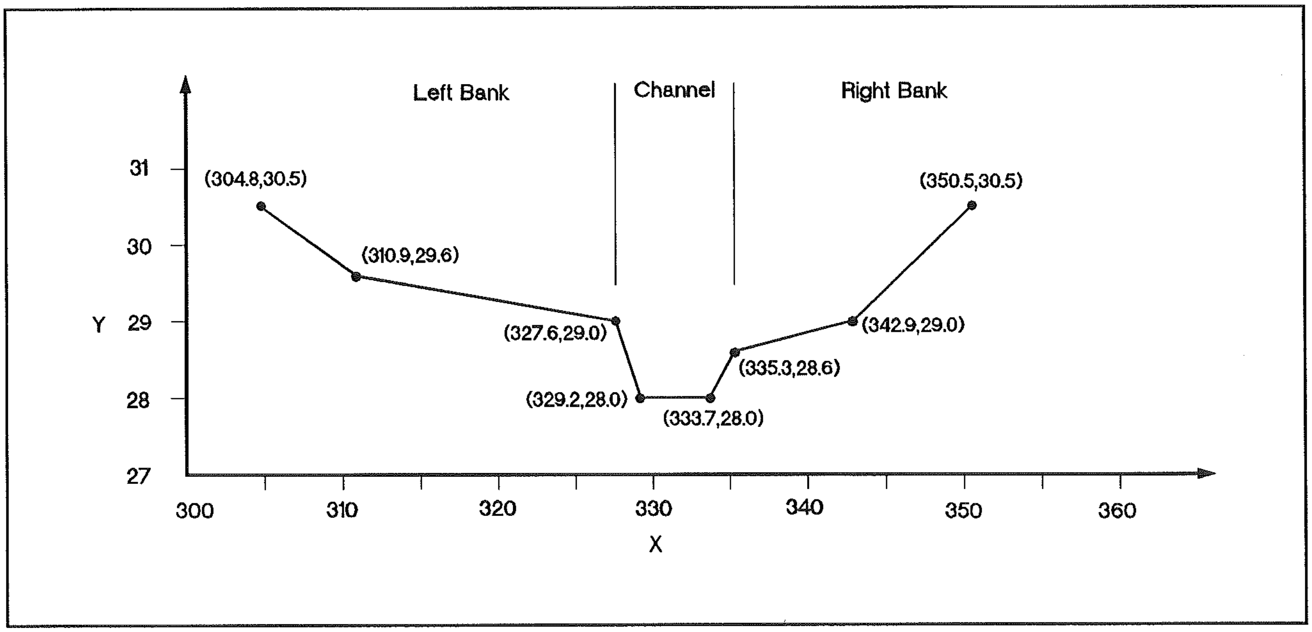
Problem:

Determine the Normal Depth routing parameters for the routing reach, A to B, shown in the routing reach map (Page 5-14). A site reconnaissance was conducted and a representative 8-point cross section, as shown below, was selected. The watercourse is normally dry except during storms. (USGS map in English units)

Conversion Table from English to Metric Units

Channel Length = 4,300 ft = 1,311 m
Channel Bed Slope = 122 ft/mi = 23.1 m/km

Channel Cross Section	STA(ft)	ELEV (ft)	STA(m)	ELEV(m)
	1000	100	304.8	30.5
	1020	97	310.9	29.6
	1075	95	327.6	29.0
	1080	92	329.2	28.0
	1095	92	333.7	28.0
	1100	94	335.3	28.6
	1125	95	342.9	29.0
	1150	100	350.5	30.5



Solution:

The model NMIN = 5 minutes.
Length of routing reach, RLNTH = 1,311 m
Channel bed slope, SEL = 23.1 m/km = 0.023 m/m

Estimate NSTPS:

The mean discharge velocity (V) is estimated as 2.13 m/sec.

$$\text{NSTPS} = \frac{\text{RLNTH}}{\text{V} \times 60 \times \text{NMIN}}$$

$$= \frac{1311}{(2.13)(60)(5)}$$

$$= 2.05 \quad \text{(use NSTPS = 2)}$$

Determination of main channel ANCH: (Tables 5-1 and 5-2)

- Channel material is coarse gravel . . . . n_0 = 0.028
- Channel banks are moderately irregular . . . . n_1 = 0.01
- Obstructions in the channel are minor . . . . n_2 = 0.01
- Vegetation in the channel is negligible . . . . n_3 = 0.0
- Variation in channel cross section is gradual . . . . n_4 = 0.0
- degree of meandering is minor . . . . m_5 = 1.0

$$\begin{aligned} \text{ANCH} &= (n_0 + n_1 + n_2 + n_3 + n_4)m_5 \\ &= (.028 + .01 + .01 + 0 + 0)1.0 \\ &= .048 \end{aligned}$$

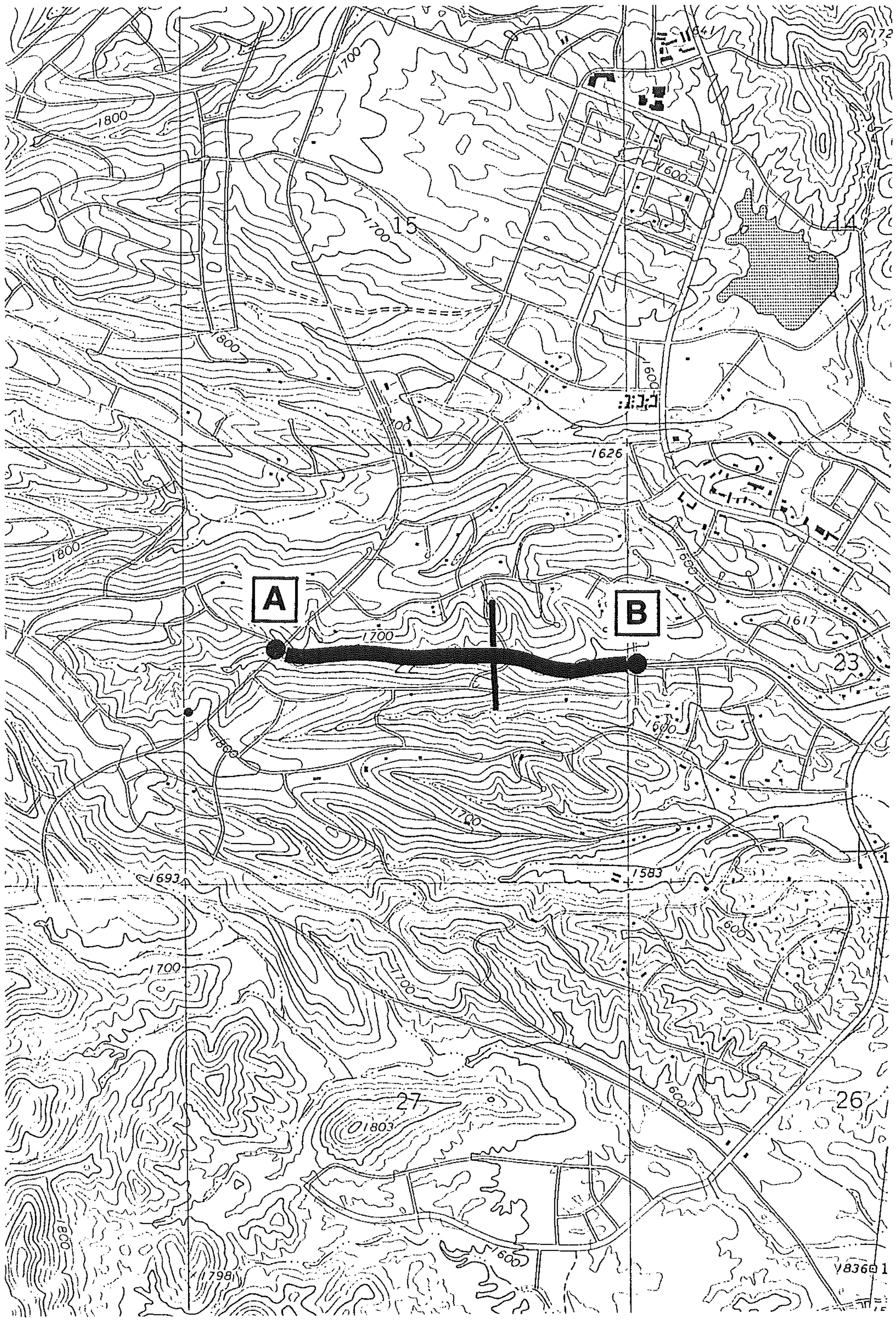
Determination of overbank n's: (Tables 5-3)

- Left overbank has medium to dense brush . . . . ANL = 0.08
- Right overbank has light brush . . . . ANR = 0.06

The HEC-1 records, using the 8-point section, are:

	FIELD									
	1	2	3	4	5	6	7	8	9	10
RS	2	Flow	0							
RC	.08	.048	.06	1311	.023					
RX	304.8	310.9	327.6	329.2	333.7	335.3	342.9	350.5		
RY	30.5	29.6	29.0	28.0	28.0	28.6	29.0	30.5		

ROUTING REACH MAP FOR A CHANNEL IN FOUNTAIN HILLS, ARIZONA



CHAPTER 6
STORAGE ROUTING

6.1 INTRODUCTION

6.1.1. General Discussion

Storage routing will be used when inflow to a structure is temporarily detained by the storage capacity and/or outlet characteristics of the structure such that the outflow is significantly different than the inflow in terms of flow rate and time. Storage routing is required when flow is routed through retention/detention basins; where flow passes through drainage facilities such as highway cross-drainage structures (particularly where the highway is elevated on earthen fill); where culverts, railroad drainage facilities, and some bridges restrict flow rates; and pump stations.

Level-pool reservoir routing is used for these applications. Information must be provided on various combinations of HEC-1 input records to describe the storage capacity and discharge relations of the structure and its outlet works.

6.2 PROCEDURE

6.2.1 General Considerations

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

6.2.1.1 Stage-Storage Relation: A relation describing the storage volume that is obtained with a specified water surface elevation must be provided. This is accomplished by one of two methods: 1) water stage (SE record) and corresponding storage volume (SV record), or 2) water stage (SE record) and corresponding surface area for the stored water to that elevation (SA record). Either method is acceptable and to some extent the selection depends upon the information that is available. If surface area data (SA records) are provided, the storage volume is calculated during the execution of the HEC-1 program.

6.2.1.2 Stage-Discharge Relation: A relation describing the discharge through the structure as a function of stage of water behind the structure must be provided. Discharges are entered on SQ records that correspond to water stages of the SE records. Stage-discharge relations are established by hydraulic analysis of the structure or from design reports.

6.2.1.3 Structure Overtopping: There are situations where structures can be overtopped due to inflow that exceeds the stage-storage-discharge relations. This can happen in a variety of situations such as elevated highway embankments with cross-drainage structures that cannot pass the required inflow. Often in such cases, the excess inflow will overtop the structure, and in those cases, the ST record can be used to model the flow that would pass over the structure; however, an overtopping discharge rating curve is the recommended method. The SQ record, in that case, is for the combined discharge through the structure plus overtopping discharge.

6.2.1.4 Pump Stations: A pump station may be included as a part of storage routing to withdraw water from the structure at that point. Pumped water leaves the study area unless it is retrieved and inserted in the model at another point. This can occur at depressed road intersections where the pumped water is released to a drainage structure outside of the intersection drainage boundaries. Pump stations can be modeled with WP and WR records. Pump station operation where multiple pumps and/or variable pump capacity is required to be modeled <u>cannot</u> be adequately modeled with HEC-1. In such cases, more sophisticated pump station models should be used. The HEC-1 model can usually be used successfully to provide the inflow hydrograph for the pump station analysis.

6.3 INSTRUCTIONS

1. Define the stage-storage relation from the most appropriate maps and input the relation in SE and SV records, or in SE and SA records.

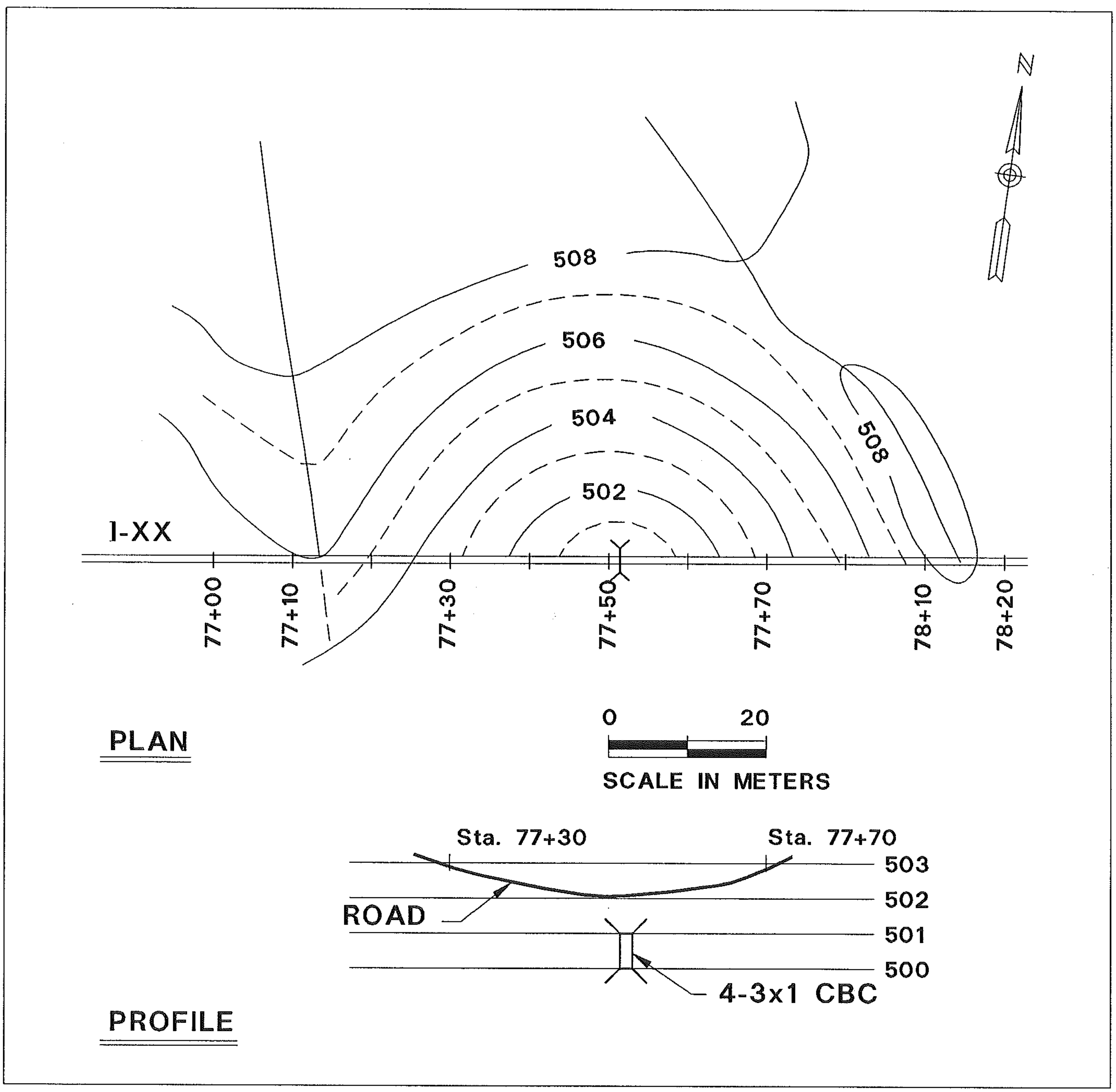
2. Define the stage-discharge relation for the outflow through the structure by use of the SQ record. Care must be taken if the structure is subject to emergency spillway flows or overtopping. The use of an SQ record will suppress all data entered on an SS record (spillway characteristics). However, flows taken from an SQ record will be added to any flows computed from the ST record (top-of-dam overflow).

   The recommended approach is to use SQ/SE records to define the complete discharge rating curve for all types of discharge through (or over) the structure. These input calculations should be performed manually 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.

3. If pump stations are included, and if the pump station capability of the HEC-1 program is adequate for the analysis, provide pump station information in WP and WR records.

EXAMPLE 6-1
STORAGE ROUTING

Determine the storage routing input for a 4 barrel 3 meter x 1 meter x 50 meter CBC as shown in the plan and profile sketch. Discharge capacity for road overtopping is to be included in the stage-discharge rating curve.



Stage-Storage Relation:

STAGE-AREA (from planimetered plan)	
Elevation, meters	Area, 1,000 sq. meters
500	0
501	50
502	75
503	360

Stage-Storage Calculation:

@ El. 500 Vol. = 0.0 1000 cubic meters

@ El. 501 Vol. = (50)(1)/2 = 25.0 1000 cubic meters

@ El. 502 Vol. = 25 + (50)(1) + (175 - 50)(1)/2 = 137.5 1000 cubic meters

@ El. 503 Vol. = 137.5 + (175)(1) + (360 - 75)(1)/2 = 455.0 1000 cubic meters

Stage-Discharge Relation:

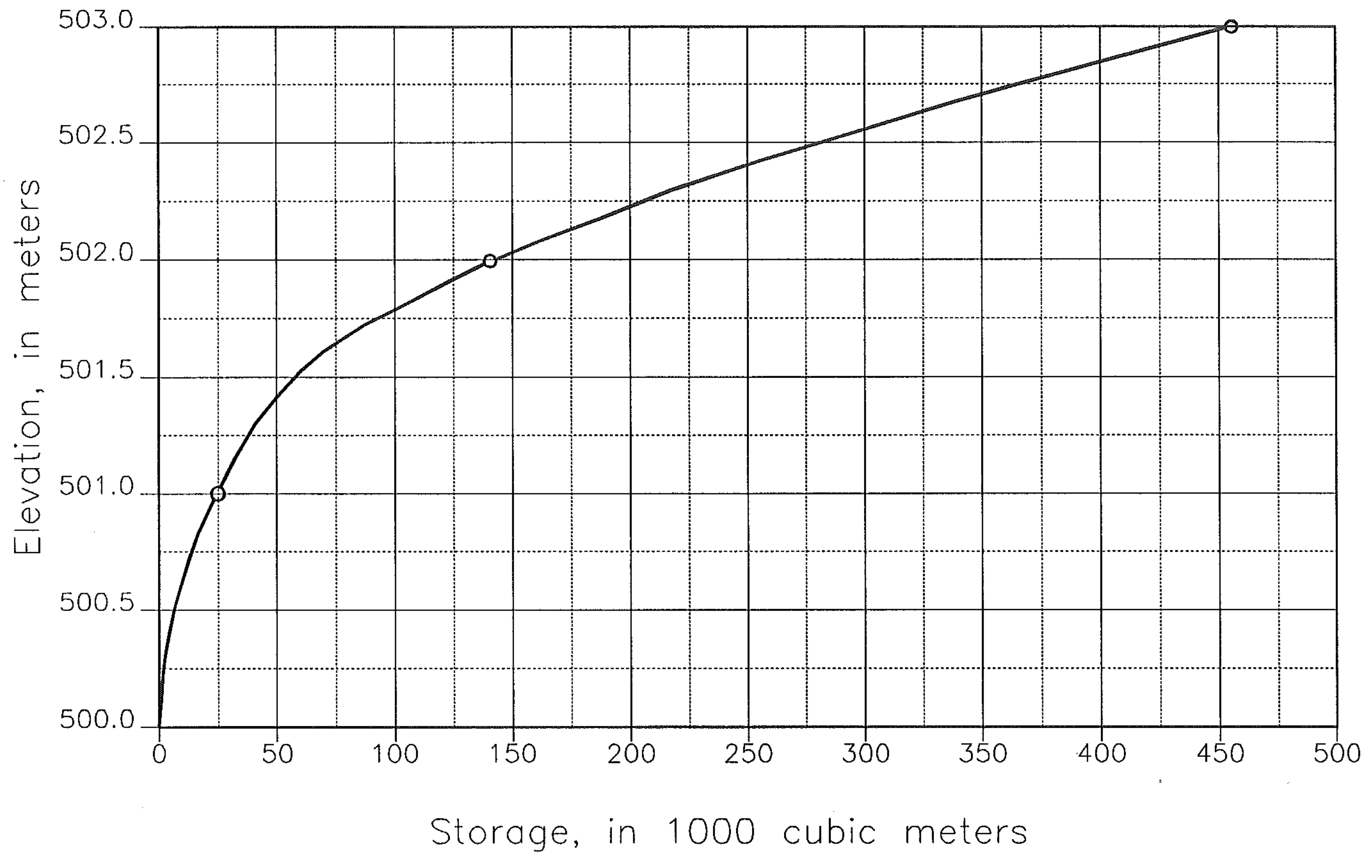
Elevation, meters	DISCHARGE, in m^3/s		
	CBC	Overtopping	Combined
500.00	0	0	0
500.54	8	0	8
500.87	16	0	16
501.18	24	0	24
501.56	32	0	32
501.96	39	1	40
502.23	43	13	56
502.32	44	20	64
502.40	45	27	72
502.48	46	34	80

HEC-1 Input:

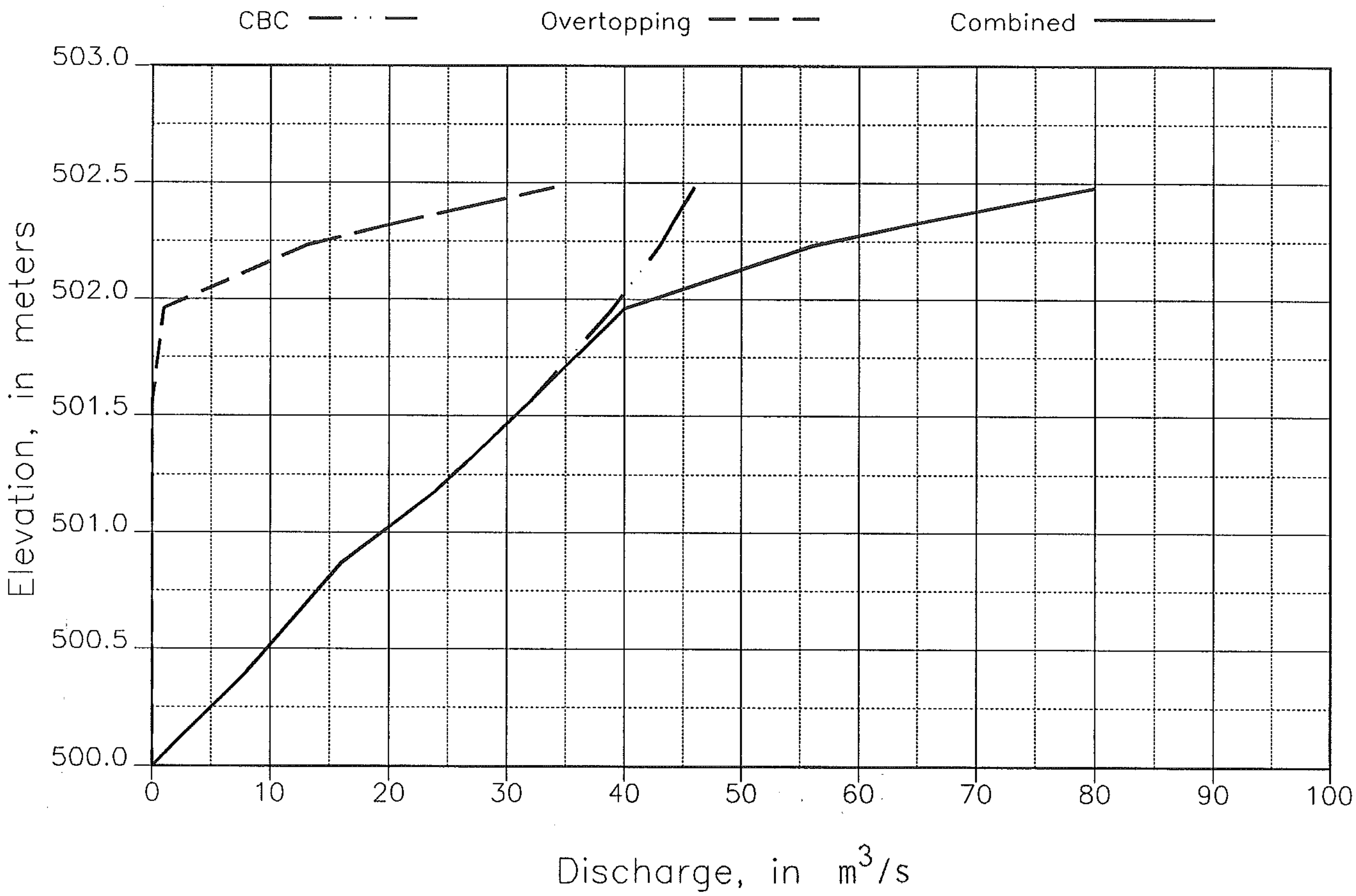
FIELD

	1	2	3	4	5	6	7	8	9	10
SV	0	13.50	21.75	45.25	88.00	133.00	210.53	239.10	264.50	289.90
SQ	0	8	16	24	32	40	56	64	72	80
SE	500	500.54	500.87	501.18	501.56	501.96	502.23	502.32	502.40	502.48

EXAMPLE 6-1
STAGE-STORAGE



EXAMPLE 6-1
STAGE-DISCHARGE



CHAPTER 7
TRANSMISSION LOSSES

7.1 INTRODUCTION

7.1.1 General Discussion

Storm runoff and floods in Arizona are usually attenuated through the effects of channel and storage routing, but they are often also diminished due to the percolation of water into the bed, banks, and overbank floodplains of the watercourses. These losses in the watercourses are transmission losses, and these are losses that accrue in the watershed in addition to the rainfall losses on the land surface. Transmission losses can, and often do, result in a significant reduction in the runoff volume. Often, transmission losses only result in a relatively small reduction in flood peak discharge; however, there are situations, such as very long, wide channels with high percolation rates, where the flood peak discharges are dramatically reduced.

The magnitude of transmission loss (both volumetric and peak discharge) is dependent upon the antecedent conditions of the watercourse; characteristics of the bed, bank, and overbank materials; channel geometry (wetted perimeter); depth to bedrock; depth to the ground water table; duration of flow; and hydrograph shape. 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 poor. Therefore, except for situations where 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 by ADOT and the procedure and assumptions clearly documented.

Two options in the HEC-1 program are available for estimating transmission losses. Both options use the RL record. The recommended option uses an estimated channel percolation rate (PERCRT) and must be used with the channel storage routing option (RS record). The second option estimates the transmission loss as a constant loss (QLOSS), in m^3/s, plus a ratio (CLOSS) of the remaining flow after subtracting QLOSS. The second method can be used with any of the HEC-1 channel routing options, however, that method is not recommended for general use because of the very subjective decisions that will need to be made in selecting QLOSS and CLOSS. The recommended method is physically-based and should result in better estimates of transmission losses, if adequate estimates can be made of the percolation rate and if the necessary storage routing information can be satisfactorily represented.

7.2 PROCEDURE

7.2.1 General Considerations

The following conditions should be met for the consideration of the incorporation of transmission losses into a rainfall-runoff model of a watershed:

1. The bed, banks, and overbank floodplains of the watercourse are composed of coarse, granular material. Material 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 total length of watercourse that is composed of coarse, granular material.

3. The watercourse is ephemeral and it is prudent to assume that the watercourse is dry before the onset of the storm.

4. The bed of the watercourse is not underlain by material, such as bedrock, that would inhibit the sustained percolation of water into the bed of the watercourse.

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

If the above conditions are met, then the incorporation of transmission losses into the model should be considered. At this point, two other factors should be considered before proceeding:

1. Incorporation of transmission losses will require a multiple subbasin model with defined routing reaches. Transmission losses will be calculated for the routing reaches. Use of the recommended option for calculating transmission losses with the HEC-1 program will require storage routing. Transmission losses will be considered only if a multiple subbasin model is acceptable.

2. Adequate information must be available to provide input for the storage routing method, and the percolation rate can be satisfactorily estimated.

If the above conditions are met, and if it is determined that modeling of transmission losses are vital and practical to the development of a rainfall-runoff model, then proceed to incorporate transmission losses in the model. This will require input of the necessary normal depth storage routing information on RC, RX, and RY records.

The transmission loss will be calculated using information from the RL record (PERCRT and ELVINV). Very little guidance is available for estimating the percolation rates (PERCRT), which can vary from more than 2,500 millimeters (100 inches) per hour to less than 25 millimeters (1 inch) per hour. **Table 7-1** provides some guidance for the percolation rate that can be expected in channels of various bed materials. The elevation of the channel invert (ELVINV) must correspond to the lowest elevation that is used in the 8-point cross section for that routing reach.

TABLE 7-1

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 PERCRT millimeters/hr
1. Very clean gravel and large sand	Very High	>130
2. Clean sand and gravel, field conditions	High	50 - 130
3. Sand and gravel mixture with low silt-clay content	Moderately High	25 - 75
4. Sand and gravel mixture with high silt-clay content	Moderate	5 - 25
5. Consolidated bed material; high silt-clay content	Insignificant to Low	0.025 - 2.5

CHAPTER 8

MODELING TECHNIQUE AND GENERAL GUIDANCE FOR USING HEC-1

8.1 INTRODUCTION

8.1.1 General Discussion

Practical application of the rainfall-runoff modeling procedures in this manual can be accomplished through use of the HEC-1 Flood Hydrograph Package (U.S. Army Corps of Engineers, 1990). This computer program, which is available from the National Technical Information Service and several commercial program vendors, provides modeling capability for the hydrologic procedures that are specified in this manual.

This chapter contains an overview of the major theoretical assumptions upon which the HEC-1 computer program is based, and the resultant limitations. Watershed modeling techniques are presented, and these are related to some of the common coding errors that are often made when using the HEC-1 program. A modeler's/reviewer's checklist is presented for use by both ADOT engineers and ADOT consultants in developing and reviewing HEC-1 watershed models.

A user's working knowledge of the following areas is assumed:

1. Surface water hydrology and watershed modeling.
2. Basic input data structure for the HEC-1 program.
3. Procedures presented in this manual.

8.1.2 Applicable HEC-1 Versions

There are many versions of the HEC-1 computer program available and in use. Care should be taken by the user to obtain and use a version containing the desired capabilities. The HEC-1 program was originally developed by the U.S. Army Corps of Engineers, Hydrologic Engineering Center (HEC) in 1967. Since that time, there have been seven significant updates and numerous error corrections. The program was originally written for main frame computers and has since been ported to a number of different platforms. This discussion is specific to the PC versions. The following is a brief synopsis of the releases made since 1988:

1988 Version -

1. The Green-Ampt infiltration equation was added as an option.
2. The Kinematic Wave runoff computations were improved.
3. All the main-frame computer options were made available in the PC version.
4. A program bug is present in the application of the Green and Ampt equation in combination with the JD record option.

1990 Version -

1. Muskingum-Cunge channel routing was added as an option.
2. Detention basin modeling capabilities were improved.
3. The Green and Ampt error from the 1988 version was corrected.
4. A program bug is present in the Kinematic Wave runoff procedure when using the JR record option. Hydrographs do not combine properly.

1991 Version -

1. This version is specific to the 80386/80486 microprocessors and requires a minimum of 2.5 megabytes of total memory, or 640 kilobytes of memory and 3 megabytes of disk space.
2. The Kinematic Wave error from the 1990 version was fixed.
3. The number of hydrograph ordinates available was increased from 300 to 2,000.

A 1990 or later version of the HEC-1 program should be used for ADOT rainfall-runoff watershed modeling purposes. The 1988 version is acceptable for single-basin models that do not require channel routing.

8.1.3 Assumptions and Limitations of HEC-1

Proficiency in use of the HEC-1 program requires an understanding and appreciation of the basic underlying assumptions and limitations. The key assumptions of the program are as follows:

8.1.3.1 Deterministic: The rainfall-runoff process is stochastic, however, the HEC-1 program treats the 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-1 model with changes to input variables.

8.1.3.2 Lumped Parameters: Many of the model parameters, for example the Green and Ampt infiltration parameters, represent spatial averages. These are "lumped" parameters that are intended to represent average conditions for a watershed subarea, not values at a point in the watershed.

8.1.3.3 Unsteady Flow: The flow rates forecasted by the model vary with time.

The key limitations of the program are as follows:

1. **Single Storm:** A single storm event is modeled. Provisions are not available for soil moisture recovery between independent storms or between bursts of rainfall within a single storm.

2. **Hydrologic Routing:** All routing (channel and storage) is by hydrologic methods. Hydraulic routing (the use of the St. Venant equations) is not performed.

3. **Results:** The results are in terms of discharges and runoff volumes. Accurate water stages are not provided for channel flow. The water stages for reservoir routing do meet the standards of the profession for accuracy (except in the tailwater reach of the reservoir where gradually varied flow would exist).

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
   d. land-use maps/reports
   e. reports of flooding
   f. streamflow data (if available)
   g. reports of other flood studies (FEMA, county, etc.)

2. Prepare a watershed base map using the best available map and most practical map scale.

3. Perform a preliminary subbasin delineation.

4. Conduct a field reconnaissance.

5. Finalize the subbasin delineation.

6. Prepare the rainfall input.

7. Prepare the rainfall loss input.

8. Prepare the unit hydrograph input.

9. Prepare all routing input.

10. Prepare a preliminary logic diagram.

11. Prepare HEC-1 input file.

12. Debug and calibrate the model, where possible.

13. Execute the HEC-1 model.

14. Check results using indirect methods for discharge verification (Chapter 10).

15. Evaluate the model and results based on available information.

16. Revise the model, as appropriate, to best represent actual watershed conditions. Model sophistication, such as incorporation of transmission losses, is usually added to the model at this point.

17. Execute the final HEC-1 model.

18. Make final model verifications and evaluations.

19. Revise the logic diagram.

20. Prepare a report.

8.2.2 HEC-1 Logic Diagram

A schematic diagram for multiple subbasin models should be prepared and included as a part of the final report. This diagram symbolically depicts the order of combining and routing hydrographs. The data to be included are:

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

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

3. Storage routing data (maximum stage, maximum storage)

8.2.3 Model Time Base and Computation Interval

The model time base and computation interval are controlled by the NMIN and NQ variables which are input in the IT record. These variables are defined as:

NMIN - The integer number of minutes in the tabulation interval used to define the spacing of the hydrograph ordinates. This variable sets the definition of the hydrograph. Too large a value will result in inaccuracies in peak discharge and runoff volume estimates.

The following criteria are recommended for NMIN:

NMIN = 2 minutes for a 6-hour storm duration (drainage area less than or equal to 2.5 square kilometers), and

NMIN = 5 minute for a 24-hour storm duration (drainage area greater than 2.5 square kilometers).

NQ - NQ is the integer number of hydrograph ordinates to be computed. There are a maximum of 300 allowed for the normal MSDOS version, and 2,000 for the extended memory MSDOS version. The total time base for the model is therefore NQ x NMIN, and this product must be greater than the total storm duration specified on the PH record.

When using a 24-hour storm duration and NMIN = 5 minutes, NQ will normally be 300. If NMIN is larger than 5 minutes, NQ can often be less than 300. If NMIN is less than 5 minutes, then NQ must be greater than 300 and the extended memory MSDOS version must be used.

When using a 6-hour storm duration and NMIN = 2 minutes, NQ can usually be set at 200. If NMIN is larger than 2 minutes, NQ can be less than 200. If NMIN is 1 minute, then NQ must be greater than 300 and the extended memory MSDOS version must be used.

Note: See Section 8.3.1.1, Item 2.c. for guidance on inspection of HEC-1 output for determination of the adequacy of the NMIN and NQ values, and guidance on alternative selections of NMIN and NQ.

8.2.4 Subbasin Delineation

The process of breaking down a watershed into subbasins should be done with careful consideration given to several critical factors. 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. Drainage structures and flood retarding structures.

3. Crossing of watercourses with 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.

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 (NMIN) for the computation interval, which will usually be either 2 minutes or 5 minutes, and estimate the number of hydrograph ordinates (NQ) required.

Note: Verify that the required NMIN and NQ estimates can be accommodated with the version of HEC-1 proposed for use.

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 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-1 model is based.

8.2.4.5 Routing Lengths: The length of the channel reaches defined as a result of the delineation should be considered while breaking down the watershed. A key parameter used in routing a hydrograph through a channel reach is the number of steps (NSTPS). Although this is most important for channel storage routing using the Normal Depth option, it is also a good check to use when applying the Muskingum-Cunge method. The minimum reach length should satisfy the following expression:

$$L = NSTPS \cdot V_{avg} \cdot 60 \cdot NMIN \qquad (8\text{-}1)$$

where: L = the minimum reach length, in meters.
$NSTPS$ = a minimum of 1, but preferably more than 1.
V_{avg} = an estimate of the average velocity, in m/sec.

Equation 8-1 is intended to be used as a guide in estimating the minimum channel routing length ($RLNTH_{min}$) before delineating subbasins in a multibasin watershed model. The use of Equation 8-1 to estimate the minimum reach length in the model can improve modeling accuracy and will minimize routing instability warnings in the model output. Section 5.2.2 should be consulted for discussion of problems that may result if this recommendation is not followed.

8.2.5 Precipitation and Rainfall Distributions

Field 1 of the PH record is coded if the model is used to estimate the 2-, 5-, or 10-year flood magnitudes, otherwise it is left blank. This is done to correct the partial-duration rainfall statistics from the NOAA Atlas 2 to annual-duration rainfall statistics. No correction is needed for other flood frequencies. Field 2 can be left blank for a single-basin model. For a multiple subbasin model, Field 2 must contain the total watershed area (not the subbasin area) so that the correct rainfall depth-area reduction factor will be applied. If design discharges are needed at existing internal concentration points in the model, then either several different models will need to be developed (one for each concentration point of interest) or the JD record option can be used. Instructions in the HEC-1 User's Manual for use of the JD record option in conjunction with the PH record for rainfall should be consulted. Insert the correct precipitation values in Fields 3 through 8 of the PH record for a 6-hour storm, or use Fields 3 through 10 of the PH record for a 24-hour storm.

8.2.6 Rainfall Losses

Keep in mind that the rainfall loss parameters are averages, assumed to be evenly distributed, for the 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 the effective impervious area which is usually less than the total impervious area. Rock outcrop is not often directly connected to the watershed outlet. Care must be exercised when estimating RTIMP for rock outcrop.

8.2.7 Time of Concentration

Certain watersheds may require estimation of several T_C's for different hydraulically most distant points. Use the largest T_C value that is calculated for the different flow paths that are considered.

Since the unit hydrograph method is extremely sensitive to the T_C parameter, every time of concentration 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 hundredths of a meter 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 0.5 to 1.0 meters per second for channels with slopes in excess of three percent. Such low velocities would not normally be considered reasonable for such steep-sloped channels.

Accordingly, a velocity analysis approach 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 <u>total</u> 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:

8.2.7.1 Open Channel Flow:

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.

8.2.7.2 Overland Flow:

1. Compute the overland flow travel time with the following equation:

$$T_{OF} = \frac{0.091\ (nL\ )^{0.8}}{(P_2)^{0.5} S^{\ 0.4}} \tag{8-2}$$

where T_{OF} = overland flow travel time (hours)

n = overland flow roughness

L = overland flow length (meters)

P_2 = 2-year, 24-hour rainfall (millimeters)

S = overland flow slope (meters/meters)

Equation 8-2 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 the HEC-1 User's Manual. Overland flow lengths are generally less than 100 meters (300 feet).

8.2.8 Hydrograph Operations

The primary hydrograph operations available with the HEC-1 program, other than routing options, are combining and diverting of hydrographs. The combine operation is performed on the number of specified hydrographs starting with the most recent operation and extending sequentially back to previous operations. Key points to remember when using this operation are:

1. The maximum number of hydrograph locations that can be displayed using the DIAGRAM option of HEC-1 is nine.

2. The maximum number of hydrographs which can be combined at one time is five.

3. The total watershed area of the combined hydrographs may be entered manually in Field 2 of the HC record.

Hydrograph diversions may be used to simulate flow splits such as might occur at street intersections, over elevated highways, or at distributary channel apexes. Key points to remember about this operation are:

1. The split is done using a discharge rating table for the diversion with a maximum volume cutoff option.

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

3. When a diverted hydrograph is recalled into the stack, the drainage area associated with the hydrograph is zero. The HEC-1 summary tables will reflect incorrect areas unless the area is corrected using the manual area input option (Field 2 of the HC record) for the first combine operation downstream of the recalled hydrograph.

8.2.9 Channel Routing

The channel routing option specified for use in this manual is the Normal Depth method. The following are considerations for use of the Normal Depth channel routing option:

8.2.9.1 Number of Calculation Steps: The NSTPS parameter must be selected with care. Normally, this parameter may be estimated iteratively as follows:

1. Make an initial estimate of NSTPS for each reach using an assumed average velocity for the peak discharge.

2. Run the model and calculate the discharge velocity for each reach. This velocity can be approximated by either of two methods.

   The most accurate, and preferred, method is to perform a normal depth calculation using Manning's equation. The normal depth calculation should use the same channel data that is entered on the RC, RX and RY records in the HEC-1 model. The average peak discharge between the upstream and downstream routing locations (obtained from the first run of the model) should be used for the velocity calculation.

A more simplified and less time consuming method (although less accurate than the previous method) is to estimate the discharge velocity by dividing the routing length on the RC record by the difference between "Time of Peak" at the upstream and downstream routing limits. The "Time of Peak" values are listed in the Runoff Summary of HEC-1 output file.

The accuracy of this second method is subject to compromise because of program rounding protocol when printing the "Time of Peak". The times to peak are based on multiples of the user selected computation interval (NMIN). Errors are created when the actual routing time is not an exact multiple of NMIN.

3. Estimate the new NSTPS values for each reach based on the calculated discharge velocity. Update and run the HEC-1 model.

4. Perform Steps 2 and 3 until the NSTPS values stabilize. This normally occurs within three iterations.

8.2.9.2 Channel Geometry: Considerations, which should be checked by field reconnaissance, when possible, for the Normal Depth method 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.

8.2.9.3 HEC-1 Warnings: A common warning message is the following:

WARNING Modified Puls Routing May Be Numerically Unstable For Outflows Between "Q_1" to "Q_2".

When this warning occurs, the following steps should be taken:

1. Examine the outflow hydrograph for oscillations and check the outflow peak against the inflow peak to be sure that the routed peak did not increase in magnitude. If these checks are satisfactory, then the warning can generally be considered to be satisfactorily addressed.

2. The NMIN variable can be reduced until the warning message goes away, or the calculated peak lies outside the specified range. However, when changing the NMIN value remember that this may affect other input parameters such as NQ and NSTPS.

8.2.10 Reservoir Routing

Modeling of reservoirs and detention basins can be accomplished using the modified Puls storage routing option of HEC-1. It is recommended that low level outlets, spillways, and structure overtopping be modeled using a discharge rating curve (SQ and SE records). The rating curve should be developed using appropriate manual calculation methods.

8.3 MODELER'S/REVIEWER'S CHECKLIST

The following is a checklist for the usual HEC-1 records that are used in watershed modeling using the procedures in this manual.

8.3.1 HEC-1 Input

8.3.1.1. Job Initialization Records:

1. ID Records
   a. The first ID record should contain the project name/number, modeler's name, and date of analysis.
   b. Additional ID records should be used to document the analysis, i.e., special model input, unique assumptions, unusual watershed conditions, etc.
   c. Revisions should be clearly identified on subsequent ID records.

2. IT Record
   a. NMIN: In general, NMIN will be selected as follows:

      NMIN = 2 minutes for a 6-hour storm duration, and

      NMIN = 5 minutes for a 24-hour storm duration.

      There may be situations requiring a different selection for NMIN. NMIN should not exceed 0.25 T_C for the subbasin with the shortest time of concentration (T_C). NMIN should be an integer. NMIN cannot be less than 1 minute.
   b. IDATE and ITIME: These records identify the date and time of the start of rainfall. These fields normally will be left blank when using the PH record for precipitation.
   c. NQ: In general, NQ will be selected as follows:

      NQ = 200 for a 6-hour storm duration, and

      NQ = 300 for a 24-hour storm duration.

      However, there may be situations requiring a different selection for NQ. Therefore, inspect the HEC-1 output for each subbasin to verify that the last discharge that is tabulated for the tail of the hydrograph is less than about 5 percent of the peak discharge for that hydrograph. If it is not, then either NQ or NMIN or both must be increased. The following must be observed when increasing either NQ or NMIN:

1. NQ cannot exceed 300 unless the extended memory MSDOS version of HEC-1 is used. Therefore, when using the 24-hour storm duration, either NMIN must be increased or the extended memory MSDOS version must be used if the discharge tail of the hydrograph does not recede to less than 5 percent of the peak discharge.

2. NMIN should not exceed 0.25 T_C for the subbasin with the shortest time of concentration (T_C).

Note: Refer to Section 8.2.3 for additional discussion.

3. IO Record
   a. IPRT: Level 3 or lower is suggested for IPRT for model development and review, since some error messages may not be printed with higher output levels. Levels 4 or 5 can be used for final (report) runs to minimize output length.

4. IM Record

   This record is required by HEC-1 for metric unit input and output.

8.3.1.2 Basin Records:

1. BA Record
   a. TAREA: This is the total contributing watershed area, in square kilometers, for a single-basin model, or the subbasin area for a multiple subbasin model.

2. BF Record
   a. Stream baseflow, in m^3/s, can be added to the runoff hydrograph to reflect desired conditions such as flow antecedent to the storm, upstream reservoir release, etc.
   b. Use of BF for a subbasin should be reset to zero (or other value) for the following subbasin or the previous BF value will be carried over to <u>each</u> subsequent subbasin.

8.3.1.3 Precipitation Record:

1. PH Record
   a. If flood estimation is for 2-, 5- or 10-year floods, the correct value of PFREQ must be used in Field 1 and left blank for other flood frequencies.
   b. If a multiple subbasin model is used, TRSDA is the total watershed area, in square kilometers, and Field 2 must be used.
   c. The correct rainfall depths are inserted in Fields 3 through 8 if the total watershed area (not subbasin area) is 2.5 square kilometers or smaller (6-hour storm duration).
   d. The correct rainfall depths are inserted in Fields 3 through 10 if the total watershed area is larger than 2.5 square kilometers (24-hour storm duration).

8.3.1.4 Rainfall Loss Records:

1. LG Record
   a. IA: This value is surface retention loss, in millimeters. This is less than initial abstraction.
   b. DTHETA, PSIF and XKSAT: These are the area weighted values of the Green and Ampt parameters.
   c. RTIMP: This is the directly connected impervious area, in percent. No rainfall losses are calculated for this area.

2. LU Record
   a. This method is only to be used if the Green and Ampt method is inappropriate.
   b. STRTL: This value is the sum, in millimeters, of surface retention loss (IA) and the initial infiltration loss prior to surface ponding. This is equivalent to initial abstraction.
   c. CNSTL: This value is the equivalent uniform loss rate, in millimeters per hour.
   d. RTIMP: This is the directly connected impervious area, in percent. No rainfall loses are calculated for this area.

8.3.1.5 Unit Hydrograph Records:

1. For a multiple subbasin model, all subbasin unit hydrographs have a duration equal to NMIN.

2. UC Record
   a. T_C: This is the basin or subbasin time of concentration, in hours. Check that this value is reasonable for the basin or subbasin.
   b. R: This is the storage coefficient, in hours.

3. UA Record
   a. Check that the correct UA values are used. If a UA record is not supplied, the HEC-1 default time-area relation is used.

8.3.1.6 Hydrograph Operation Record:

1. HC Record
   a. No more than five hydrographs can be combined at any time.
   b. No more than nine hanging hydrographs can be carried on a schematic diagram.
   c. TAREA: This is the total area, in square kilometers. It is usually left blank. TAREA should be specified if a previously diverted hydrograph is to be added at that point.

8.3.1.7 Channel Routing Records:

1. RS Record
   a. NSTPS: Number of steps to be used in the Normal Depth channel routing. (See Sections 8.2.4.5 and 8.2.9.1)
   b. ITYP: Insert FLOW indicating that the discharge for the beginning of the first time period is specified in the next field.
   c. RSVRIC: The discharge value, in m^3/s, corresponding to the desired starting condition at the beginning of the routing operation (often 0 for conditions in Arizona unless the stream or river is assumed to have baseflow).

2. RC Record

   a. ANL, ANCH and ANR: These channel roughness n values should be reasonable and inserted in the record in the correct order.

   b. RLNTH: Same as L in RS record.

   c. SEL: Same as S in RS record.

   d. ELMAX: Not usually used. May be left blank.

3. RX and RY Records

   a. All eight stations must be used.

   b. Values are in meters.

   c. Sequential values on the RX record must not decrease in magnitude.

   d. The cross section must be "typical" for the routing reach.

   e. The defined cross section must have adequate capacity to contain the peak discharge. If not, HEC-1 will extend the two end stations vertically, and this is usually inappropriate for broad, shallow overbanks in Arizona.

   f. Care must be exercised in defining the channel geometry to avoid non-effective flow areas.

8.3.1.8 Storage Routing Records:

1. RS Record

   a. NSTPS: This is the number of steps used in the calculation. NSTPS = 1 for reservoir storage routing. NSTPS must be calculated if this method is used for Normal Depth channel routing.

   b. ITYP: Use STOR if the initial condition of the reservoir will be indicated by an existing storage volume. Use FLOW if the initial condition of the reservoir or channel will be identified by an existing discharge. Use ELEV if the initial condition of the reservoir or channel will be identified by an existing water surface elevation.

   c. RSVRIC: This is the value of the initial routing condition (storage, in 1,000 cubic meters; discharge, in m^3/s; or elevation, in meters) as indicated by ITYP.

2. SV/SA Records

   a. When using the SV record, RCAP is storage volume, in 1,000 cubic meters, corresponding to the elevation value in the same Field in the following SE record.

   b. When using the SA record, RAREA is surface area, in 1,000 square meters, corresponding to the elevation value in the same Field in the following SE record.

3. SE Record

   a. This record is placed immediately after either an SV or SA record.

   b. ELEV: This is the water surface elevation, in meters, corresponding to values in the same Field of either the SV or SA record.

   c. SV/SA and SE values should correspond to an established volume/area versus elevation rating curve.

4. SQ Record

   a. This record is used to define a stage-discharge relation. DISQ is discharge, in m^3/s, corresponding to the previous SV/SA and SE records, or a separate SE record for use with the SQ record <u>only</u> can be placed immediately after the SQ record.

8.3.1.9 Transmission Losses Record

1. RL Record

   a. The preferred method is by specifying the unit area percolation rate (PERCRT), in $m^3/s/1{,}000$ square meters, in Field 3. If that method is used, the Muskingum-Cunge channel routing method <u>cannot</u> be used. Storage routing (also called Normal Depth for channel routing, RS record) must be used.

   b. ELVINV: This is the lowest elevation on the 8-point section geometry (RY record). Transmission losses will not be calculated if this value is not specified.

8.3.2 HEC-1 Output

8.3.2.1 Errors: All error messages must be checked. Output level (IPRT) 3 or less must be entered on the IO record for all error messages to appear. The HEC-1 manual contains a section explaining the error messages and how to correct them.

8.3.2.2 Diagram: Check the schematic. Follow the diagram on the watershed map and see if it is correct and reasonable.

1. Make sure there are no "hanging hydrographs" left.

2. Make sure that all of the diverted hydrographs have been accounted for.

3. Make sure that all of the subareas are attached and are being combined in the proper sequence. All upstream subareas must be combined before routing through a downstream channel.

8.3.2.3 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 HEC-1 summary table. For basins with several outlets, the contributing area for each outlet may have to be added together and then checked for accuracy.

If USGS streamgages are present in the watershed, the HEC-1 area above the gage concentration point should be compared to USGS published reports. Previous studies of the watershed may also prove useful for comparison of areas.

When a diverted hydrograph is returned (HC record), the area associated with it must also be returned (Field 2), if the user desires the HEC-1 output summary to reflect accurate basin areas at downstream concentration points that combines the diverted hydrograph with other HEC-1 operations.

8.3.2.4 Losses: Look through the output for each subbasin. Check the total rainfall, total losses and total runoff. If zero or a very small number is noticed in any of these columns, the input for that subbasin must be examined. It is possible to drop a loss record (LG, LU) and not get an error statement in the output. Check the loss columns for inconsistency. Inconsistencies in estimated losses must be examined.

8.3.2.5. Routing:

1. Check the applicability of the routing methodology applied.

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

3. Check for instability in the outflow hydrograph. This can be done by using level 1 (IPRT) output or by plotting the hydrograph.

4. Check to see that the flow is contained within the channel. HEC-1 will normally extend the banks vertically if the channel cross section area is not large enough.

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. Routing steps in the input can be checked against the output velocity.

6. Routing procedures will normally result in some attenuation of the peak flow. This 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 them 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 0 (instantaneous translation of the hydrograph through the reach). The cumulative effect of this may result in substantial error.

8.3.2.6 Peak Runoff: Since HEC-1 does not have a summary table showing unit discharge (m^3/s/square kilometer), 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, etc., this comparison may be difficult. However, large differences in unit discharge should alert the reviewer to check the input for discrepancies.

8.3.2.7 Time to Peak: Check the time to peak column in the HEC-1 summary table:

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

2. Check that the Tp's occur after the most intense portion of the rainfall period (more than half the duration of the rainfall using the PH record).

8.3.2.8 Volumes: Check the output to determine if the volume of runoff is reasonable. This 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.

8.3.2.9 General:

1. Compare the peak flows and unit discharges against available data for the area. Inconsistencies in these discharges may indicate to the reviewer that errors exist in the HEC-1 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. Most of the peak is generated from hill slopes and attenuated in the valley. Mixing the two may cause incorrect results.

4. Peaks are most affected by the time of concentration. Volumes are most sensitive to loss functions.

5. When calibrating the HEC-1 model, make sure adjustments are made properly. For example, losses should not be adjusted where time of concentration is the major cause of the differences.

6. Time of concentration and lag time 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 flow is present, the n values must be adjusted accordingly.

8. When comparing existing versus proposed conditions, all the model parameters (rainfall losses, unit hydrographs, routing, etc.) must be adjusted accordingly. Proposed storm sewer pipe flows are more efficient than surface flows and can increase peak discharges. For more frequent storms, where depth of flow is small, introducing street networks may effect the flow paths. This may require a re-examination of subbasin boundaries.

CHAPTER 9
FLOOD FREQUENCY ANALYSIS

9.1 INTRODUCTION

9.1.1 General Discussion

Flood frequency analysis is a procedure for computing flood magnitude frequency relations where systematic stream gaging records of sufficient length are available. The result of such an analysis, as presented herein, is a graph of peak discharge as a function of return period. This graph can be used to estimate the flood magnitude for selected return periods, generally between 2-year and 100-year. 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 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.

9.2 PROCEDURE

9.2.1 General Considerations

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 <u>all</u> 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 judgement. 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 procedures to be applied are summarized, herein, and do not contain technical discussion or extensive instructions. 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.

9.2.2 Applications and Limitations

1. A minimum of 10 years of continuous, systematic data is 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 the 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 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. This 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; although, knowledge of the past flood frequency relation would be valuable in the development and calibration of the rainfall-runoff model.

5. Flood data have extremely large natural variability and 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 adequate large floods (leading to underdesign) 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 about 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.

9.2.3 Data

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

9.2.3.1 Systematic Records: These 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 this systematic data for Arizona are the records of the U.S. Geological Survey (USGS). The published records of the USGS can be used to obtain much of this data, although the USGS should be consulted to obtain more recent, unpublished data and to confer with USGS personnel on the quality of the data and on possible other sources of data or related studies. Additional stream discharge data may be available from state agencies, such as the Arizona Department of Water Resources, and county or local agencies. Systematic records can be continuous, broken, or incomplete.

9.2.3.2 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.

9.2.3.3 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 did not occur that would affect the flood magnitudes.

9.2.3.4 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 cause. Missing high and low flow data require different treatment. When high flood discharges were not recorded, there is usually information available from which the peak discharge can be estimated. The collecting agency will usually provide such estimates and these are usually so noted in the records of the agency. 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).

9.2.3.5 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, <u>such data often defines an extended period during which the largest floods, either recorded or historic, are known</u>. 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 or extracted from newspaper files or by 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 where the systematic record is relatively short. Use of historic data assures that estimates are consistent with local experience and improves the frequency determinations.

9.2.4 Extraordinary Floods

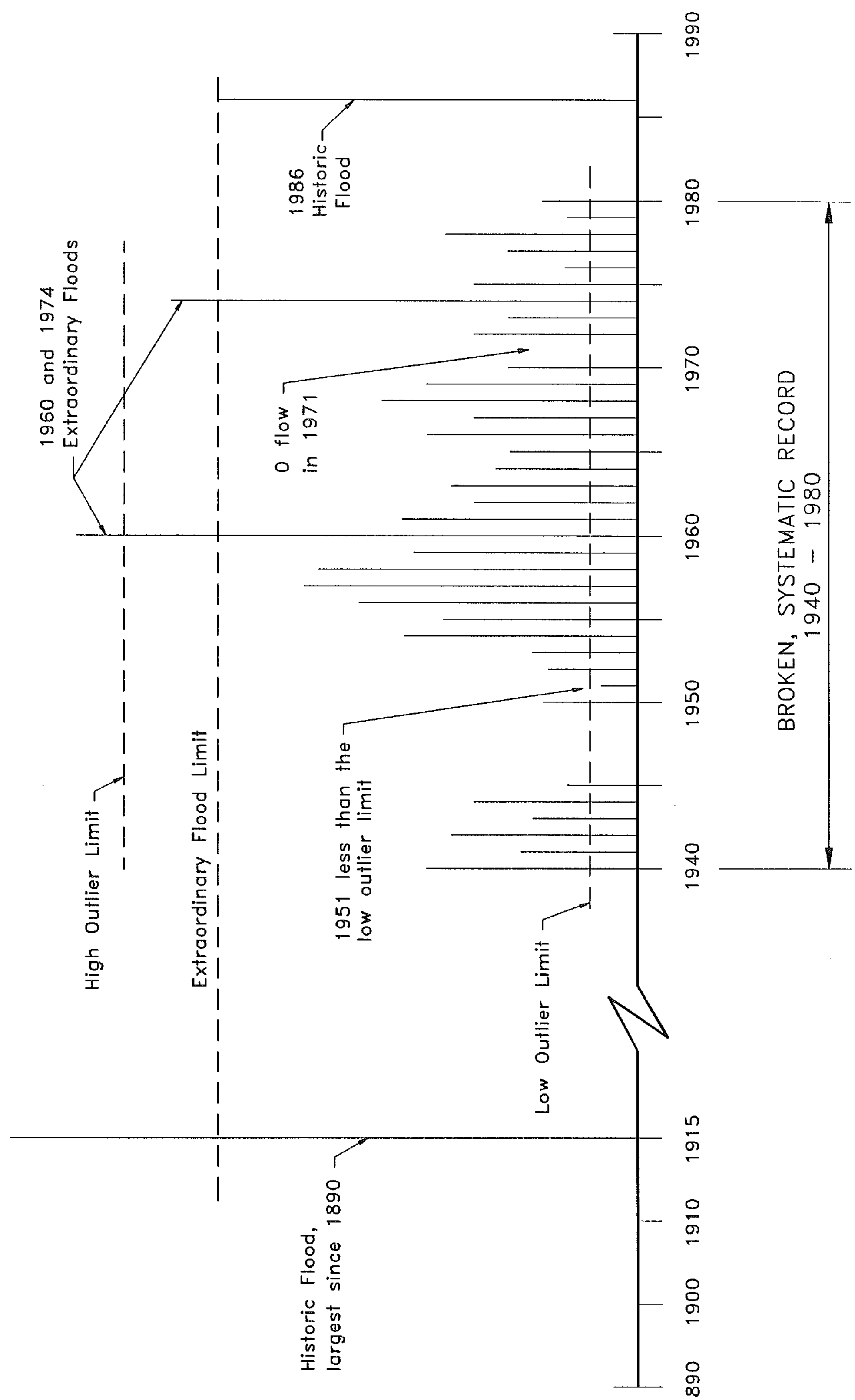
Extraordinary floods are floods with magnitudes that are considerably higher than the vast majority of floods in the record. Extraordinary floods can be either systematic or historic. Most historic floods, by virtue of the fact that they were noted during a period when systematic data were not collected, 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.

9.2.5 Illustrative Flood Series and Definitions

Figure 9-1 illustrates a series of systematic and historic flood data. This illustration demonstrates the definitions and variables that are 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, inclusive. 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 larger than the 1986 flood and therefore the 1974 flood is extraordinary. The high outlier limit was calculated and the 1960 flood exceeds that magnitude and 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 and therefore it is treated as a zero flow year.

FIGURE 9 – 1

Illustrative Flood Series for demonstrating definitions and variables in flood frequency analysis



The following are the values to be used in this flood frequency analysis:

Effective record length (N) (See 9.2.8.2 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)

$N_s = N_t - Z$

$= 37 - 2 = 35$ 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 (h)

$k = h + e$

$= 2 + 2 = 4$ years

Number of systematic plus historic data (N_g)

$N_g = N_s + h$

$= 35 + 2 = 37$ years

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

9.2.6 Data Compilation

The data that are collected are to be compiled in a table with the following headings: water year; annual peak discharge (cfs); annual peak discharge (m^3/s); 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. Since it is probable that most annual discharge data available to date will be in the English units of cubic feet per second (cfs), a column in those units is provided for the compilation of this data. The English unit data will be converted to cubic meters per second (m^3/s) and entered into the next column. A data compilation form is shown in **Figure 9-2**, and a cfs to m^3/s conversion equation is provided in that form.

9.2.7 Preliminary Data Analysis

A time series graph of flood peak discharge as a function of water year will 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 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.

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FIGURE 9-2
FLOOD FREQUENCY ANALYSIS
DATA COMPILATION FORM

Page _1_ of ___

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

WATER YEAR (1)	ANNUAL PEAK DISCHARGE (cfs) (2)	ANNUAL PEAK DISCHARGE (m^3/s) (3)	DATE (4)	FLOOD TYPE[a] (5)	COMMENTS (6)

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

Note: (3) = 0.0283 * (2) for cfs to m^3/s conversion

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HYDROLOGIC DESIGN DATA

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FLOOD FREQUENCY ANALYSIS
DATA COMPILATION FORM

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WATER YEAR (1)	ANNUAL PEAK DISCHARGE (cfs) (2)	ANNUAL PEAK DISCHARGE (m^3/s) (3)	DATE (4)	FLOOD TYPE [a] (5)	COMMENTS (6)

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

Note: (3) = 0.0283 * (2) for cfs to m^3/s conversion

FIGURE 9-2 Continued

The second preliminary analysis, that will be important for rivers that drain mountainous watersheds in Arizona, is the determination of the cause of the flood discharge. Floods in Arizona will normally be 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 and often this judgement can be made by inspecting the daily discharge records for the days immediately prior to and after the flood date. In other cases, it may be necessary to inspect the flood stage hydrograph record, consult meteorologic data (rainfall and temperature), refer to flood reports, talk to local authorities, or use other means to make this judgement. The data compilation (**Figure 9-2**) should document the cause of the flood.

9.2.8 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. The use of both plotting position equations are demonstrated with examples. The equation relating the exceedance probability (P_e), to the flood return period (T_r), in years, is:

$$T_r = 1/P_e \tag{9-1}$$

9.2.8.1 Systematic Data Equation: For systematic data, the plotting position equation is (Cunnane, 1978):

$$P_e = \frac{m - .4}{N_s + .2} \tag{9-2}$$

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 9-8 must be used along with Equation 9-2.

9.2.8.2 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):

$$P_e = \left(\frac{m - .4}{k + .2}\right)\left(\frac{k}{N}\right)$$

for m = 1, ..., k

$$P_e = \frac{k}{N} + \left(\frac{N - k}{N}\right)\left(\frac{m - k - .4}{N - k + .2}\right)\left(\frac{N - k}{N_s \quad e}\right) \qquad (9\text{-}3)$$

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 judgement will be necessary in certain cases in selecting the effective record length for records containing extraordinary floods (see Example No 9-3, 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 9-8 must be used along with Equation 9-3.

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, $N_g = N_s + h$.

9.2.9 Use of Plotting Position Equation

The compiled flood data (**Figure 9-2**) are ranked from largest to smallest using the form in **Figure 9-3**. The plotting position is calculated by either Equation 9-2 or 9-3, 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 a later section.

9.2.10 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	**9-4**
log-normal, 3 1/2 cycle	**9-5**
extreme value	**9-6**
log-extreme value, 2 cycle	**9-7**
log-extreme value, 3 1/2 cycle	**9-8**

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FIGURE 9-3
FLOOD FREQUENCY ANALYSIS
PLOTTING POSITION CALCULATION FORM Page 1 of ____

Gage Station Name ______________________
Gage Station No. ______________________ **Drainage Area** __________**sq. km.**
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 (m^3/s) (1)	RANK (2)	PLOTTING POSITION	
		P_e (3)	T_r (4)

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FLOOD FREQUENCY ANALYSIS
PLOTTING POSITION CALCULATION FORM

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FLOOD PEAK DISCHARGE (m^3/s) (1)	RANK (2)	PLOTTING POSITION	
		P_e (3)	T_r (4)

FIGURE 9-3 Continued

FIGURE 9-4
LOG-NORMAL 2 CYCLE GRAPH PAPER

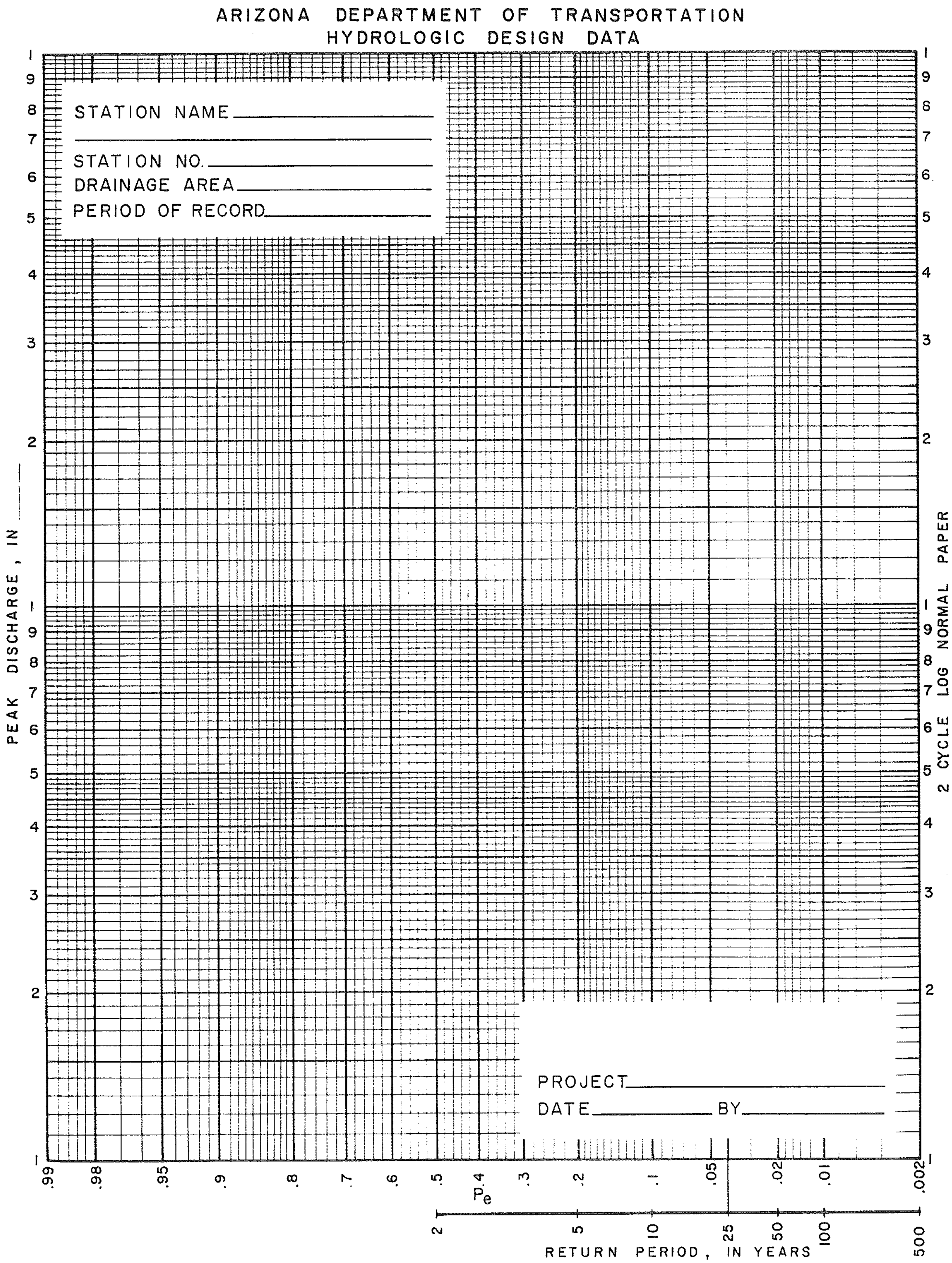


FIGURE 9-5
LOG-NORMAL 3½ CYCLE GRAPH PAPER

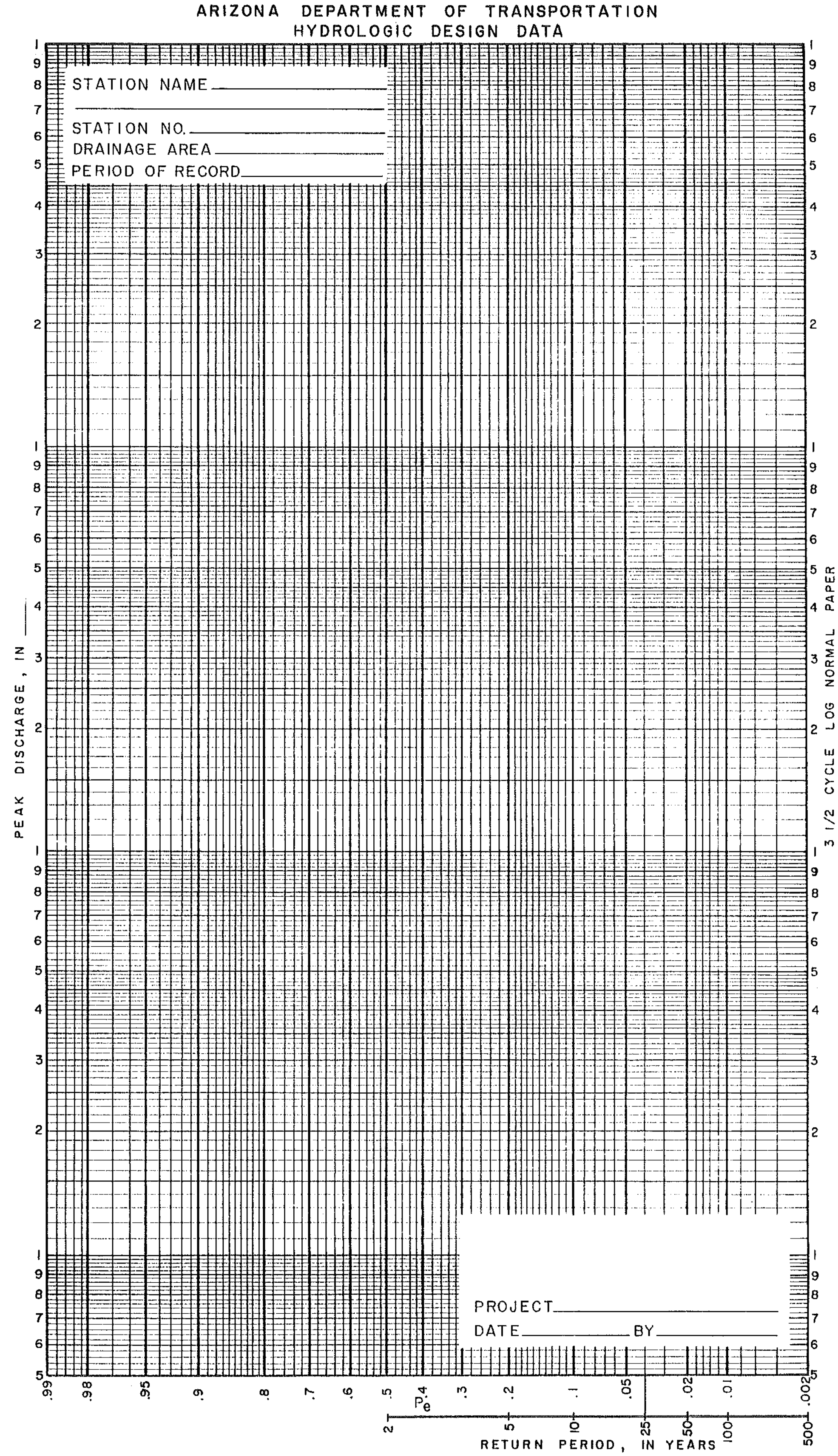


FIGURE 9-6
EXTREME VALUE GRAPH PAPER

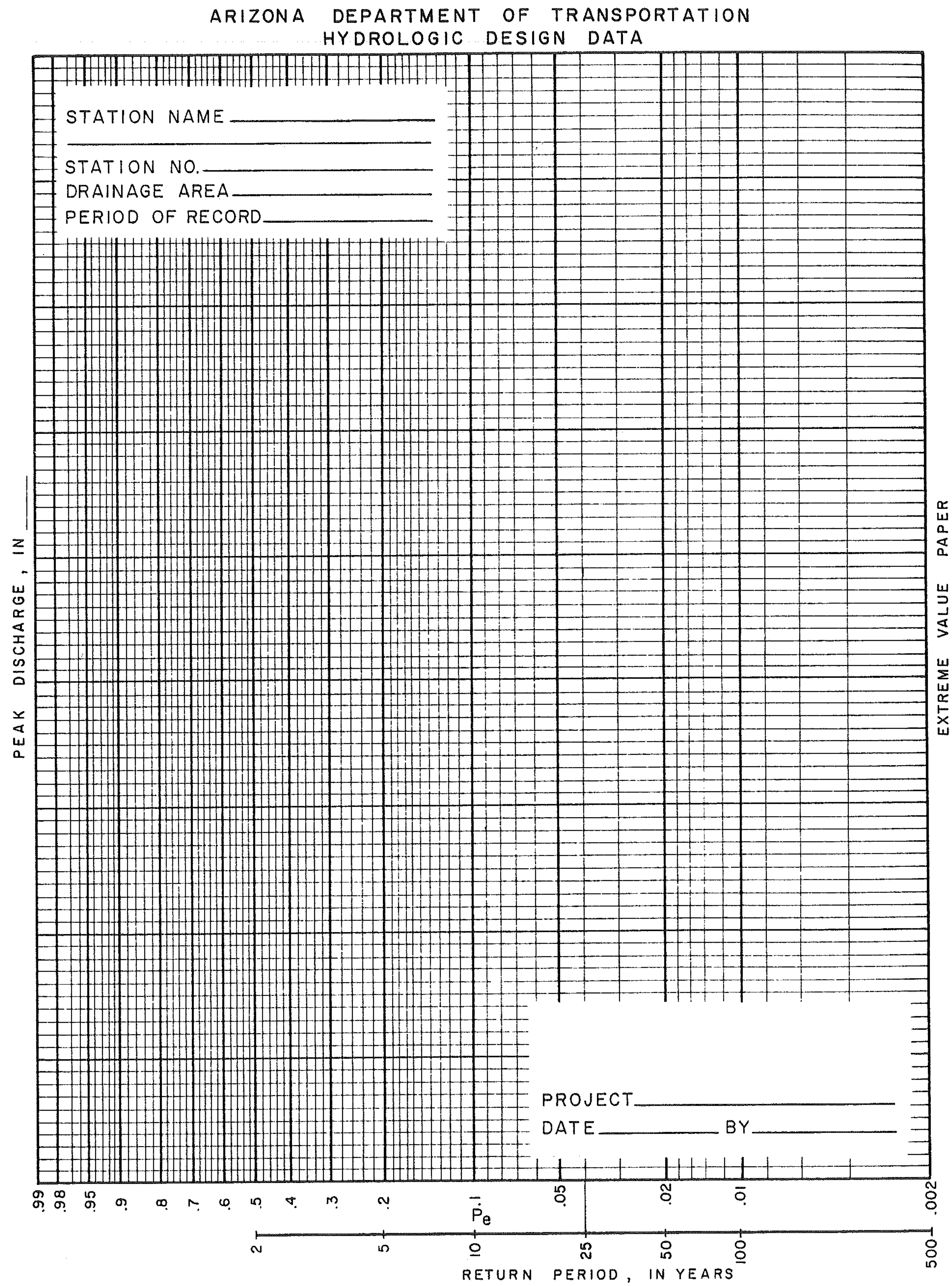


FIGURE 9-7
LOG-EXTREME VALUE 2 CYCLE GRAPH PAPER

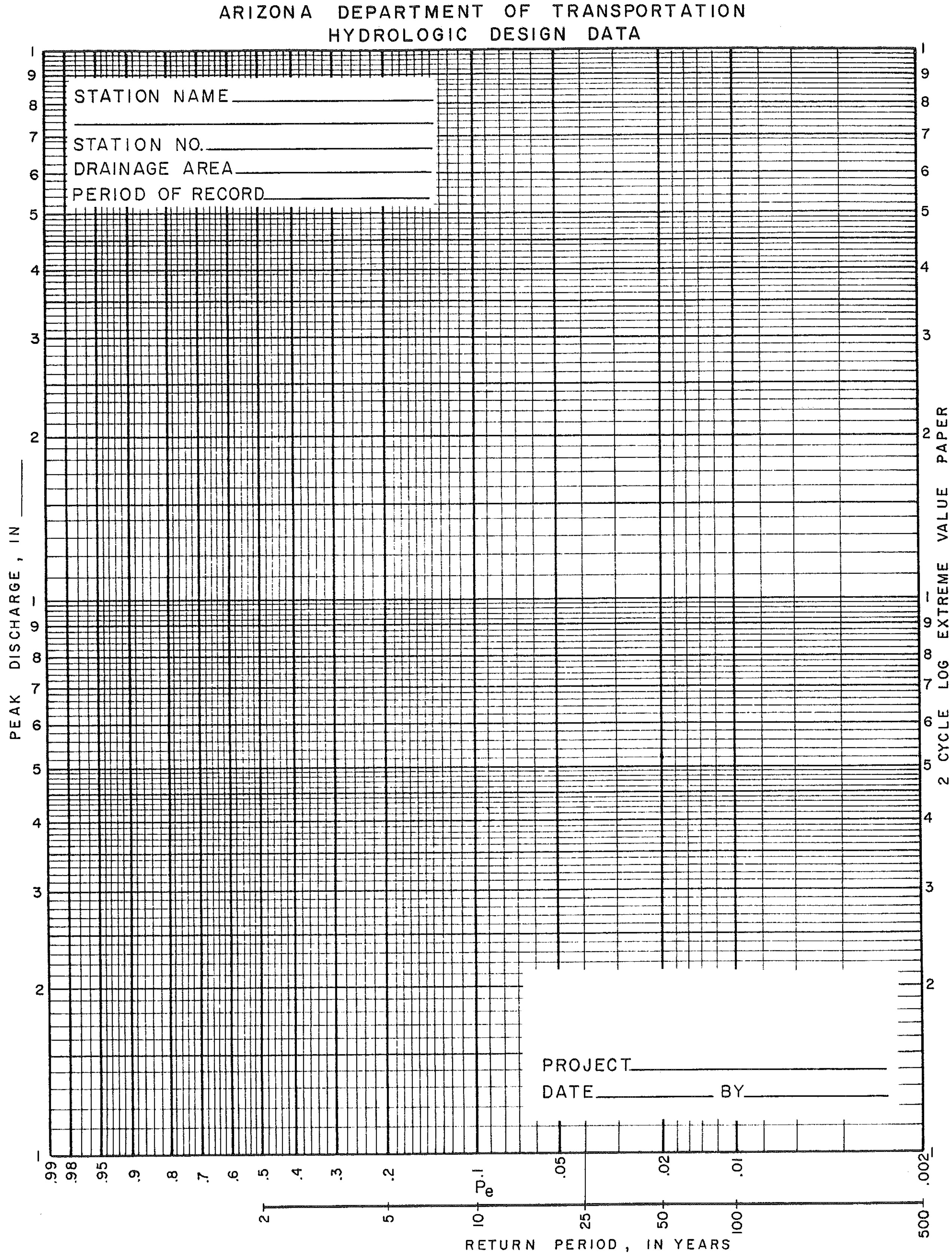
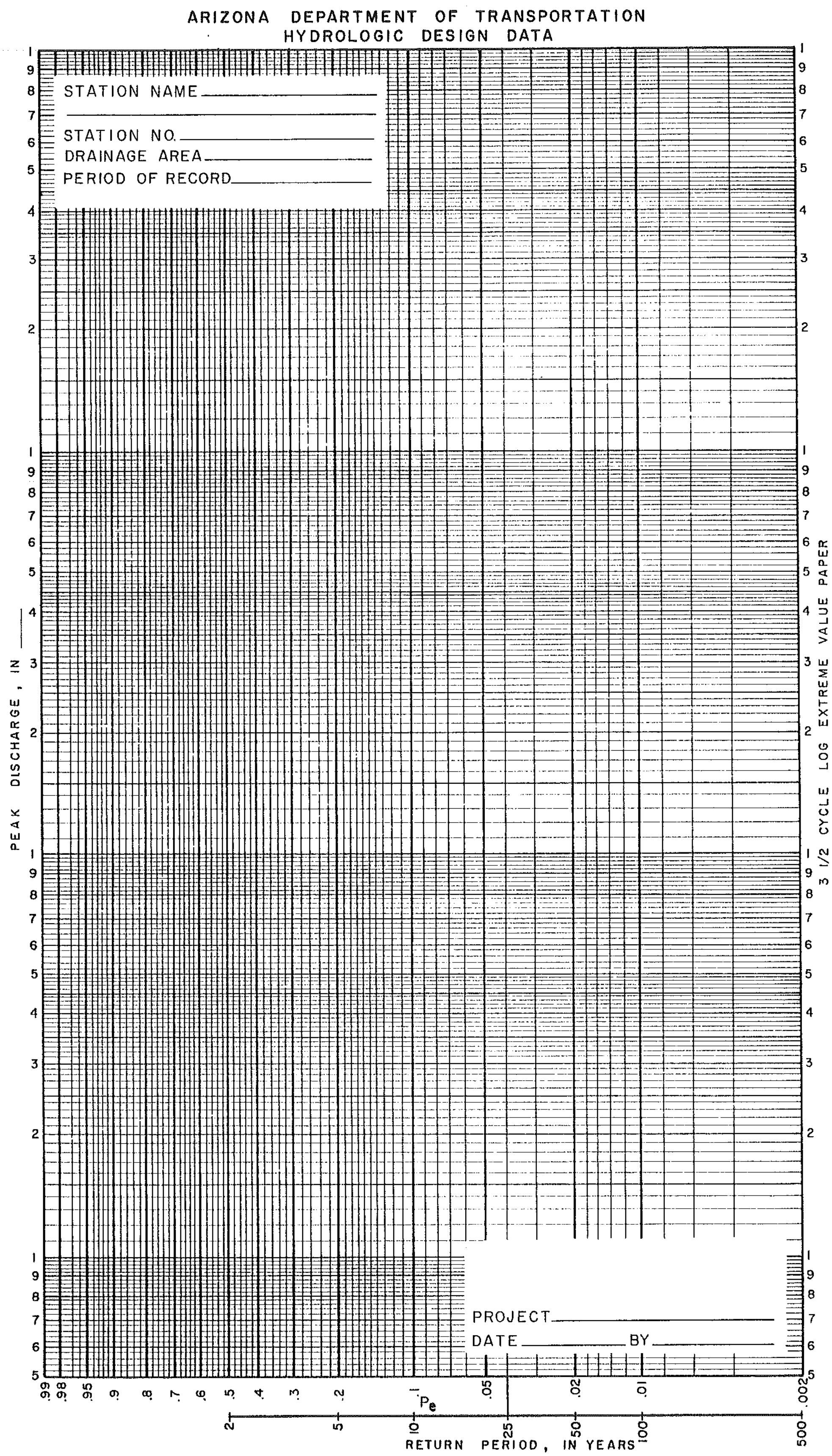


FIGURE 9-8
LOG-EXTREME VALUE 3½ CYCLE PAPER



9.2.11 Plotting Data on Graph Paper

The flood frequency data (**Figure 9-3**) are plotted on all three types of graph paper; LN, EV, and LEV (**Figures 9-4 through 9-8**). 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: (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, (2) the data can plot nearly linearly, and equally as well, on two or three of the graph papers, and (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). Those papers are included in the Documentation Manual and are available through ADOT.

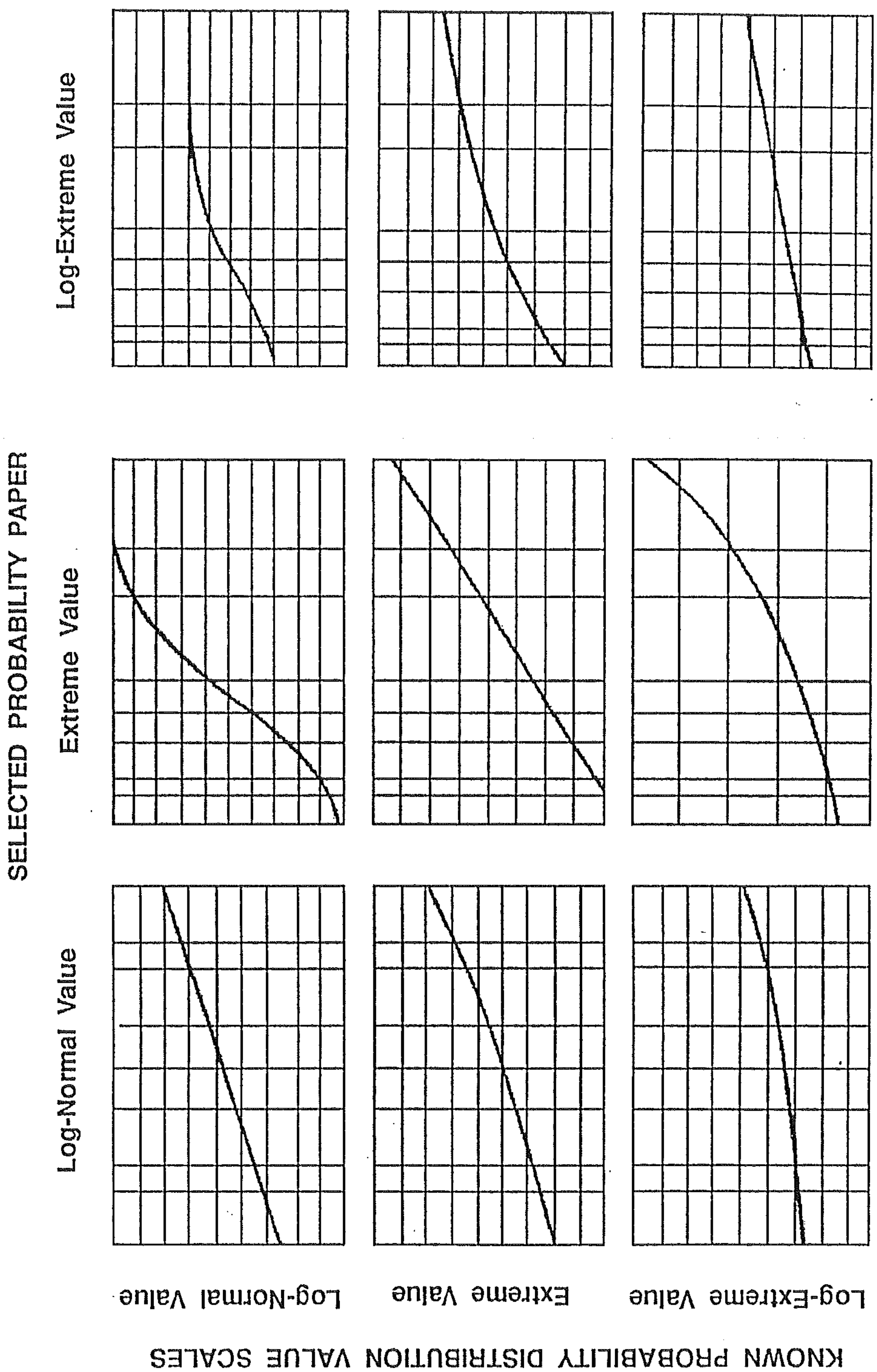
2. **Figure 9-9** (King, 1971) provides guidance in the shape of data of unknown probability distribution when plotted on the three recommended graph papers. Notice that when the unknown distribution of the data is the same as the distribution of the graph paper, the data plots as a straight line (the desired situation). Use of **Figure 9-9** 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 9-9**.

FIGURE 9-9

COMPARATIVE GRAPHS

FIGURE 9-9

Comparative graphs of shapes of data plots for data of known probability distributions when plotted on selected probability paper (after King, 1971)



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 result of estimation error of such large flood magnitudes that exceed the limits of the gaging station rating curve.

4. Three probability distribution graph papers are recommended but this does not preclude use of other graph paper for other probability distributions. 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 9-9**) 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.

5. There will be situations where the data may plot as two straight lines (one for the smaller flood discharges and another for the larger discharges). This 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 judgement, 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 less 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, and such deviations will occur about the "best fit" line. Some data points will be above the line and some below the line, and this is acceptable as long as the data points appear to be linearly arrayed rather than curvelinearly 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.

9.2.12 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 <u>after</u> 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.

9.2.12.1 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 9-2, 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):

$$P_e = P(Q > Q_o) = [P(Q > Q_o \mid R)][P(R)] + [P(Q > Q_o \mid S)][P(S)] \qquad (9\text{-}4)$$

Equation 9-4 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 ($P(Q > Q_0 \mid R)$) (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 ($P(Q > Q_0 \mid S)$) (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 9-4 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. This will usually, but not always, be the same graph paper that was used for either rainfall or snowmelt that had the larger floods.

9.2.12.2 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):

$$\log Q_H = \overline{\log Q} + K_N S \tag{9-5}$$

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 9-1** for sample size N_s, and

S = standard deviation of log Q's calculated by

$$S = \left[\frac{\sum (\log Q_i)^2 - (\sum \log Q_i)^2/N_s}{N_s - 1}\right]^{.5}$$

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 9-5 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 9-1
FLOOD FREQUENCY ANALYSIS
OUTLIER TEST K_N VALUES

10 PERCENT SIGNIFICANCE LEVEL K_N VALUES

The table below contains one sided 10 percent significance level K_N values for a normal distribution (U.S. Water Resources Council, 1981).

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	84	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	63	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):

$$\log Q_L = \overline{\log Q} - K_N S \tag{9-6}$$

where $\log Q_L$ = low outlier threshold in log units and the other terms are as defined for Equation 9-5.

If the logarithms of any annual peak discharges in a sample are less than $\log Q_L$ in Equation 9-6, then they are considered low outliers. Flood peaks considered low outliers are treated as zero flow years.

9.2.12.3 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:

$$P_f = \frac{N_t - Z}{N_t} = \frac{N_s}{N_t} \tag{9-7}$$

where P_f = probability of an annual flood,
N_t = length of systematic record including the number of zero flow years ($N_t = N_s + Z$), and
Z = number of years with zero flow.

2. Rank the flood events and calculate the plotting position (P_e) using either Equation 9-2 (systematic data only) or Equation 9-3 (systematic plus historic and/or extraordinary data), with the zero flow data removed with either equation.

3. Calculate the conditional plotting position (P_z):

$$P_z = P_e \times P_f \tag{9-8}$$

where P_z = the plotting position for the flood data,
P_e = the probability of flood exceedance given that flooding has occurred (Equation 9-2 for systematic data only or Equation 9-3 for systematic plus historic and/or extraordinary data), and
P_f = calculated by Equation 9-7.

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

9.2.13 Confidence Limits

In performing a flood frequency analysis by the graphical method, as described, 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 nature 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; which obviously is not practical or informative. There is not an established criteria 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 **Figures 9-10** through **9-12** for use with the LN, EV, and LEV distributions, respectively. In **Figures 9-10** through **9-12** 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 $N_c = N_s$ (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.

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FIGURE 9-10
FLOOD FREQUENCY ANALYSIS
WORK SHEET FOR LOG-NORMAL CONFIDENCE LIMITS

Gage Station Name ______________________
Gage Station No. ______________________

Confidence Level (C.L.) = __________%

$Q_{2\text{-yr}}$ = __________ m^3/s

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

$Q_{100\text{-yr}}$ = __________ m^3/s

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

N_c = __________

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

$$S_{ln} = \frac{\log_{10} Q_{100\text{-yr}} - \log_{10} Q_{2\text{-yr}}}{2.327} = \frac{\log_{10}(\quad) - \log_{10}(\quad)}{2.327} = \text{\_\_\_\_\_\_\_\_}$$

T Years (1)	$U_{1-\frac{1}{T}}$ (2)	Y_T (a) (3)	S_T (b) (4)	Limits, in m^3/s (c) Upper (5)	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) $S_T = \left[\left(\dfrac{S_{ln}^{\,2}}{N_c}\right)\left(1 + .5\, U^2_{1-\frac{1}{T}}\right)\right]^{\frac{1}{2}}$

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FIGURE 9-11
FLOOD FREQUENCY ANALYSIS
WORK SHEET FOR EXTREME VALUE CONFIDENCE LIMITS

Gage Station Name ______________________
Gage Station No. ______________________

Confidence Level (C.L.) = ________%

$Q_{2\text{-yr}}$ = ________ m^3/s $\alpha = \frac{100 - C.L.}{100}$ = ________

$Q_{100\text{-yr}}$ = ________ m^3/s $U_{1-\frac{\alpha}{2}}$ = ________

N_c = ________

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

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

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

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

T Years (1)	K (2)	Z (a) (3)	S_T (b) (4)	Q_T (c) (5)	Limits, in m^3/s (d) Upper (6)	Lower (7)
2	-.1643	.9179				
5	.7195	1.545				
10	1.3046	2.087				
25	2.0438	2.814				
50	2.5923	3.368				
100	3.1367	3.924				

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

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

(c) $Q_T = \overline{Q} + K\,S_{ev}$

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

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FIGURE 9-12
FLOOD FREQUENCY ANALYSIS
WORK SHEET FOR LOG-EXTREME VALUE CONFIDENCE LIMITS

Gage Station Name ____
Gage Station No. ________________________

Confidence Level (C.L.) = __________%

$Q_{2\text{-yr}}$ = __________ m^3/s $\qquad \alpha = \dfrac{100 - C.L.}{100}$ = __________

$Q_{100\text{-yr}}$ = __________ m^3/s $\qquad U_{1-\frac{\alpha}{2}}$ = __________

N_c = __________

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

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

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

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

T Years (1)	K (2)	Z (a) (3)	S_T (b) (4)	Y_T (c) (5)	Limits, in m^3/s (d)	
					Upper (6)	Lower (7)
2	-.1643	.9179				
5	.7195	1.545				
10	1.3046	2.087				
25	2.0438	2.814				
50	2.5923	3.368				
100	3.1367	3.924				

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

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

(c) $Y_T = \overline{Y} + K\,S_{lev}$

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

9.3 INSTRUCTIONS

9.3.1 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 9-2**).

2. Compile related flood information, regional studies, etc.

3. Perform preliminary data analyses to investigate stationarity of the data, presence of mixed populations, etc.

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 9-3**) and calculate the plotting position according to the following:

Type of Data Series	Equation
Systematic data only	9-2
Systematic plus historic and/or extraordinary data	9-3
Data with zero flow years	9-8

11. Perform the graphical analysis as described herein.

9.3.2 Confidence Limits

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

1. Select the appropriate work sheet (**Figures 9-10** through **9-12**) 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 of $u_{1-\alpha/2}$ 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

      $N_c = N_s$, where only systematic data exist, or
      $N_c = N_g$, where systematic plus historic and/or extraordinary data exist.

   b. 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.

9.4 EXAMPLES

In the following, four examples of flood frequency analyses are provided. These examples are included to demonstrate the application of the procedures. They are arranged from the simplest to the more complex analyses.

1. Example 9-1, Agua Fria River near Mayer, Arizona, demonstrates a fairly simple analysis requiring no special treatment of the data.

2. Example 9-2, Cave Creek near Cave Creek, Arizona, demonstrates a data set that contains zero flow years - a fairly common occurrence for streams in Arizona.

3. Example 9-3, Hassayampa River near Wickenburg, Arizona, demonstrates a data set containing historic data and extraordinary floods. The effective record length was extended beyond the length of the systematic record.

4. Example 9-4, Santa Cruz River near Lochiel, Arizona, demonstrates a data set containing a low outlier and extraordinary floods. The effective length of record was extended beyond the length of the systematic record.

FLOOD FREQUENCY ANALYSIS EXAMPLE No. 9-1

Station Name - Agua Fria River near Mayer, Arizona
Station Number - 09512500
Drainage Area - 1,523 square kilometers
Period of Record - 1940 through 1989

Flood Data

A continuous, 50 year systematic record is available, and the entire record was used in the analysis. All annual floods are considered to be caused by rainfall. There are no historic data. There are no zero flow years. The high and low floods of record are 937 m^3/s (1980) and 21 m^3/s (1974), respectively. The record is considered stationary.

Flood Frequency Analysis

The high outlier limit is calculated at 1,331 m^3/s, and no high outliers are identified. The low outlier limit is calculated at 18 m^3/s, and no low outliers are identified. No extraordinary floods are identified.

The length of the systematic record is for the period 1940 through 1989 (N_t = 50). There are no zero flow years or low outliers (Z = 0), and the effective length of the systematic record is 50 years ($N_s = N_t - Z = 50 - 0 = 50$). There is no special treatment in calculating the plotting positions.

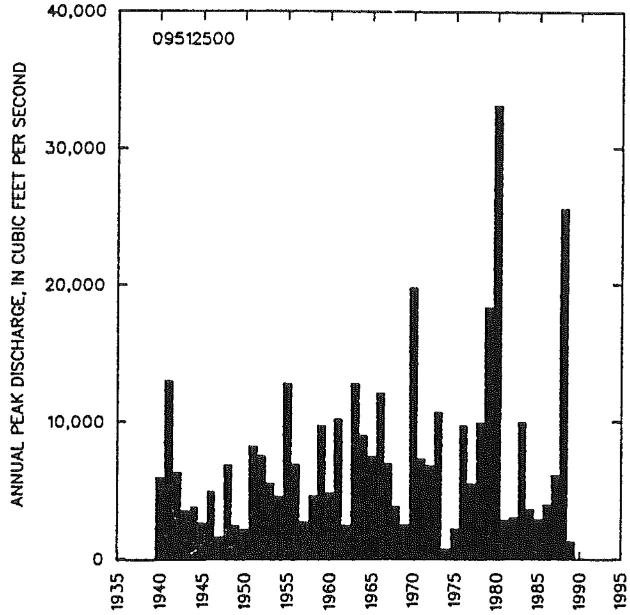
The annual flood peak discharges are plotted on the three probability papers at their respective plotting positions. The extreme value (EV) graph shows a concave up form to the data points, and a linear trend to data with P_e less than about 0.17. The log-extreme value (LEV) graph shows a concave down form to the data points, and a linear trend to data with P_e less than about 0.31. The log-normal (LN) graph shows a good linear trend to the data points for all but the smallest flood peak discharges. The LN is selected as the best representation of the probability distribution of floods with return periods that are equal to or longer than 2 years.

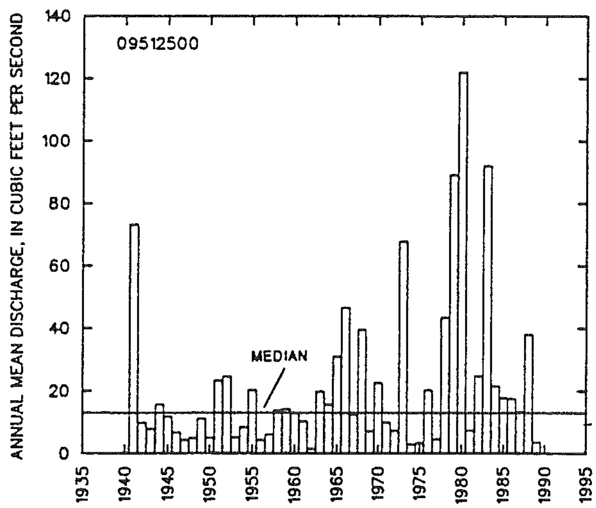
Confidence limits are set about the LN best fit line. The 43 largest floods (N_c = 43) are used to establish the best fit line. The estimated 100-yr flood peak discharge is 1,048 m^3/s with 90 percent upper and lower confidence limits of 1,555 m^3/s and 708 m^3/s, respectively.

Discussion

This example illustrates a flood frequency analysis that does not require any special treatment of the data. The LN graph provides the best straight line fit to the data. This is an example of a clear choice of the best graph to select. The range for the confidence limits is relatively tight because the 43 largest floods can be used to establish the best fit line.

09512500 AGUA FRIA RIVER NEAR MAYER, AZ--CONTINUED





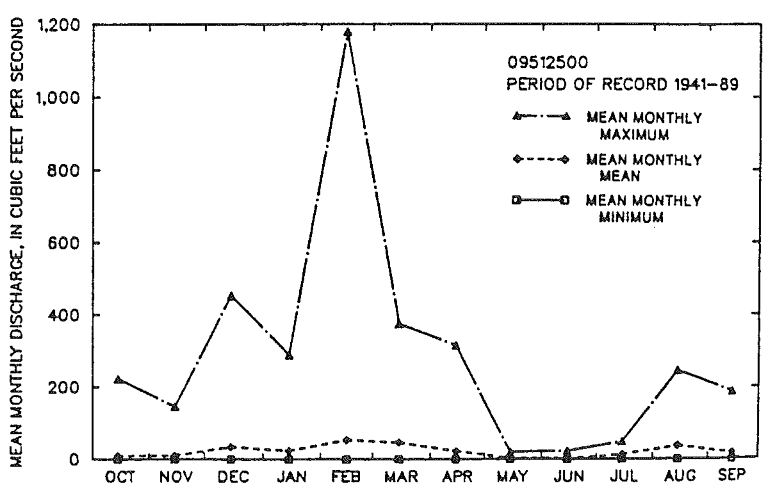


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FIGURE 9-2
FLOOD FREQUENCY ANALYSIS
DATA COMPILATION FORM

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Gage Station Name AGUA FRIA RIVER NEAR MAYER, AZ
Gage Station No. 09152500 **Drainage Area** 1523 **sq. km.**
Period of Systematic Record 1940 - 1989

WATER YEAR (1)	ANNUAL PEAK DISCHARGE (cfs) (2)	ANNUAL PEAK DISCHARGE (m^3/s) (3)	DATE (4)	FLOOD TYPE [a] (5)	COMMENTS (6)
1940	5920	168	26 JUNE 40	R	
41	13000	368	1 MAR 41	R	
42	6280	178	6 AUG 42	R	
43	3500	99	25 SEPT 43	R	
44	3810	108	16 SEPT 44	R	
45	2620	74	27 JULY 45	R	
46	4930	140	22 JULY 46	R	
47	1610	46	16 AUG 47	R	
48	6830	193	4 AUG 48	R	
49	2460	70	13 JAN 49	R	
50	2170	61	17 JULY 50	R	
51	8180	232	28 AUG 51	R	
52	7500	212	18 JAN 52	R	
53	5510	156	8 JULY 53	R	
54	4570	129	3 SEPT 54	R	
55	12800	362	3 AUG 55	R	
56	6880	195	25 JULY 56	R	
57	2710	77	13 AUG 57	R	
58	4620	131	21 JUNE 58	R	
59	9700	275	4 AUG 59	R	
60	4820	137	8 AUG 60	R	
61	10200	289	22 JULY 61	R	

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

Note: (3) = 0.0283 * (2) for cfs to m^3/s conversion

09512500 AGUA FRIA RIVER NEAR MAYER, AZ

LOCATION.--Lat 34°18'55", long 112°03'48", in NW¼SE¼ sec.20, T.11 N., R.3 E., Yavapai County, Hydrologic Unit 15070102, on left bank at Sycamore damsite, 700 ft downstream from Big Bug Creek and 12 mi southeast of Mayer.

DRAINAGE AREA.--585 mi ^{2}.

REMARKS.--Diversions above station for mining and irrigation of about 600 acres. Perry Canal, which previously headed 300 ft above the gage, was washed out on July 11, 1977, and was not rebuilt.

ANNUAL PEAK DISCHARGE

WATER YEAR	DATE	ANNUAL PEAK DISCHARGE (FT^3/S)	WATER YEAR	DATE	ANNUAL PEAK DISCHARGE (FT^3/S)
1940	06-26-40	5,920	1965	04-04-65	7,470
1941	03-01-41	13,000	1966	12-22-65	12,100
1942	08-06-42	6,280	1967	08-19-67	6,960
1943	09-25-43	3,500	1968	12-19-67	3,850
1944	09-16-44	3,810	1969	08-07-69	2,490
1945	07-27-45	2,620	1970	09-05-70	19,800
1946	07-22-46	4,930	1971	08-25-71	7,280
1947	08-16-47	1,610	1972	08-12-72	6,800
1948	08-04-48	6,830	1973	10-07-72	10,700
1949	01-13-49	2,460	1974	07-20-74	740
1950	07-17-50	2,170	1975	07-27-75	2,190
1951	08-28-51	8,180	1976	02-09-76	9,700
1952	01-18-52	7,500	1977	08-23-77	5,480
1953	07-08-53	5,510	1978	03-01-78	9,900
1954	09-03-54	4,570	1979	12-18-78	18,300
1955	08-03-55	12,800	1980	02-19-80	33,100
1956	07-25-56	6,880	1981	09-23-81	2,850
1957	08-13-57	2,710	1982	09-10-82	3,040
1958	06-21-58	4,620	1983	09-23-83	9,940
1959	08-04-59	9,700	1984	08-14-84	3,620
1960	08-08-60	4,820	1985	12-27-84	2,880
1961	07-22-61	10,200	1986	11-26-85	3,970
1962	09-13-62	2,470	1987	10-11-86	6,070
1963	08-19-63	12,800	1988	08-29-88	25,500
1964	07-24-64	9,000	1989	08-18-89	1,280

BASIN CHARACTERISTICS

MAIN CHANNEL SLOPE (FT/MI)	STREAM LENGTH (MI)	MEAN BASIN ELEVA-TION (FT)	FORESTED AREA (PERCENT)	SOIL INDEX	MEAN ANNUAL PRECIPI-TATION (IN)	RAINFALL INTENSITY, 24-HOUR	
						2-YEAR (IN)	50-YEAR (IN)
56.9	37.5	5,000	3.4	1.3	16.7	2.1	4.3

09512500 AGUA FRIA RIVER NEAR MAYER, AZ--Continued

MEAN MONTHLY AND ANNUAL DISCHARGES 1941-89

MONTH	MAXIMUM (FT^3/S)	MINIMUM (FT^3/S)	MEAN (FT^3/S)	STANDARD DEVIATION (FT^3/S)	COEFFICIENT OF VARIATION	PERCENT OF ANNUAL RUNOFF
OCTOBER	223	0.14	10	33	3.2	3.7
NOVEMBER	146	0.10	10	25	2.4	3.8
DECEMBER	453	0.08	34	87	2.6	12.6
JANUARY	288	0.07	23	50	2.2	8.5
FEBRUARY	1,180	0.02	53	173	3.3	19.7
MARCH	373	0.01	46	83	1.8	17.2
APRIL	314	0.00	22	58	2.7	8.0
MAY	20	0.03	3.1	5.1	1.6	1.1
JUNE	23	0.01	2.3	3.7	1.7	0.8
JULY	48	0.15	12	13	1.0	4.5
AUGUST	244	0.31	37	52	1.4	13.7
SEPTEMBER	187	0.20	17	36	2.1	6.3
ANNUAL	122	1.5	22	26	1.2	100

MAGNITUDE AND PROBABILITY OF ANNUAL LOW FLOW BASED ON PERIOD OF RECORD 1941-89

PERIOD (CONSECUTIVE DAYS)	DISCHARGE, IN FT^3/S, FOR INDICATED RECURRENCE INTERVAL, IN YEARS, AND NON-EXCEEDANCE PROBABILITY, IN PERCENT: 2 50%	5 20%	10 10%	20 5%	50 2%	100† 1%
1	0.00	0.00	0.00	0.00	0.00	0.00
3	0.00	0.00	0.00	0.00	0.00	0.00
7	0.00	0.00	0.00	0.00	0.00	0.00
14	0.00	0.00	0.00	0.00	0.00	0.00
30	0.00	0.00	0.00	0.00	0.00	0.00
60	0.57	0.19	0.11	0.06	0.03	0.02
90	0.90	0.29	0.16	0.09	0.05	0.03
120	1.9	0.66	0.34	0.19	0.09	0.05
183	4.4	1.6	0.85	0.48	0.24	0.15

MAGNITUDE AND PROBABILITY OF INSTANTANEOUS PEAK FLOW BASED ON PERIOD OF RECORD 1940-89

DISCHARGE, IN FT^3/S, FOR INDICATED RECURRENCE INTERVAL IN YEARS, AND EXCEEDANCE PROBABILITY, IN PERCENT: 2 50%	5 20%	10 10%	25 4%	50 2%	100 1%
5,920	10,600	14,500	20,500	25,800	31,700

WEIGHTED SKEW (LOGS)= 0.16
MEAN (LOGS)= 3.78
STANDARD DEV. (LOGS)= 0.30

MAGNITUDE AND PROBABILITY OF ANNUAL HIGH FLOW BASED ON PERIOD OF RECORD 1941-89

PERIOD (CONSECUTIVE DAYS)	DISCHARGE, IN FT^3/S, FOR INDICATED RECURRENCE INTERVAL, IN YEARS, AND EXCEEDANCE PROBABILITY, IN PERCENT: 2 50%	5 20%	10 10%	25 4%	50 2%	100† 1%
1	793	2,000	3,290	5,670	8,110	11,200
3	388	998	1,680	2,970	4,340	6,150
7	216	564	946	1,660	2,390	3,350
15	130	333	549	943	1,340	1,850
30	83	211	343	574	799	1,070
60	53	134	216	356	489	649
90	38	95	155	258	359	483

DURATION TABLE OF DAILY MEAN FLOW FOR PERIOD OF RECORD 1941-89

DISCHARGE, IN FT^3/S, WHICH WAS EQUALED OR EXCEEDED FOR INDICATED PERCENT OF TIME

1%	5%	10%	15%	20%	30%	40%	50%	60%	70%	80%	90%	95%	98%	99%	99.5%	99.9%
393	70	20	10	6.9	4.2	2.8	1.9	1.3	0.81	0.51	0.21	0.14	0.10	0.00	0.00	0.00

† Reliability of values in column is uncertain, and potential errors are large.

ARIZONA DEPARTMENT OF TRANSPORTATION
HYDROLOGIC DESIGN DATA

Project No. ______ TRACS No. ______
Project Name ______ Date ______
Location/Station AGUA FRIA RIVER NEAR MAYER, AZ
Designer ______ Checker ______

FLOOD FREQUENCY ANALYSIS
DATA COMPILATION FORM

Page 2 of 3

WATER YEAR (1)	ANNUAL PEAK DISCHARGE (cfs) (2)	ANNUAL PEAK DISCHARGE (m^3/s) (3)	DATE (4)	FLOOD TYPE [a] (5)	COMMENTS (6)
1962	2470	70	13 SEPT 62	R	
63	12800	362	19 AUG 63	R	
64	9000	255	24 JULY 64	R	
65	7470	211	4 APR 65	R	
66	12100	343	22 DEC 65	R	
67	6960	197	19 AUG 67	R	
68	3850	109	19 DEC 67	R	
69	2490	71	7 AUG 69	R	
70	19800	561	5 SEPT 70	R	
71	7280	206	25 AUG 71	R	
72	6800	193	12 AUG 72	R	
73	10700	303	7 OCT 72	R	
74	740	21	20 JULY 74	R	
75	2190	62	27 JULY 75	R	
76	9700	275	9 FEB 76	R	
77	5480	155	23 AUG 77	R	
78	9900	280	1 MAR 78	R	
79	18300	518	18 DEC 78	R	
80	33100	937	19 FEB 80	R	
81	2850	81	23 SEPT 81	R	
82	3040	86	10 SEPT 82	R	
83	9940	282	23 SEPT 83	R	
84	3620	103	14 AUG 84	R	
85	2880	82	27 DEC 84	R	
86	3970	112	26 NOV 85	R	
87	6070	172	11 OCT 86	R	
88	25500	722	29 AUG 88	R	

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

Note: (3) = 0.0283 * (2) for cfs to m^3/s conversion

FIGURE 9-2 Continued

ARIZONA DEPARTMENT OF TRANSPORTATION
HYDROLOGIC DESIGN DATA

Project No. ________ TRACS No. ________
Project Name ________ Date ________
Location/Station AGUA FRIA RIVER NEAR MAYER AZ
Designer ________ Checker ________

FLOOD FREQUENCY ANALYSIS
DATA COMPILATION FORM

Page 3 of 3

WATER YEAR (1)	ANNUAL PEAK DISCHARGE (cfs) (2)	ANNUAL PEAK DISCHARGE (m^3/s) (3)	DATE (4)	FLOOD TYPE[a] (5)	COMMENTS (6)
1989	1280	36	18 AUG 89	R	
		ARITH.	LOG10		
		$\bar{Q}$ = 210	2.2004		
		S = 175	0.3334		
		Δ = 50			

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

Note: (3) = 0.0283 * (2) for cfs to m^3/s conversion

FIGURE 9-2 Continued

ARIZONA DEPARTMENT OF TRANSPORTATION
HYDROLOGIC DESIGN DATA

Project No. ______________ TRACS No. ______________
Project Name ______________ Date ______________
Location/Station AGUA FRIA RIVER NEAR MAYER AZ
Designer ______________ Checker ______________

TEST FOR HIGH & LOW OUTLIERS

$\overline{\text{LOG } Q} = 2.2004$ $\quad N = 50$

$\text{LOG } S = 0.3334$ $\quad K_N = 2.768$

- HIGH OUTLIERS

$$\text{LOG } Q_H = \overline{\text{LOG } Q} + K_N \text{ LOG } S$$
$$= 2.2004 + 2.768(0.3334)$$
$$= 3.1233$$
$$\underline{Q_H = 1328 \text{ m}^3/\text{s}}$$

THERE ARE NO Q's > 1328 m^3/s ∴ NO HIGH OUTLIERS

- LOW OUTLIERS

$$\text{LOG } Q_L = \overline{\text{LOG } Q} - K_N \text{ LOG } S$$
$$= 2.2004 - 2.768(0.3334)$$
$$= 1.2775$$
$$\underline{Q_L = 19 \text{ m}^3/\text{s}}$$

THERE ARE NO Q's < 19 m^3/s ∴ NO LOW OUTLIERS

ARIZONA DEPARTMENT OF TRANSPORTATION
HYDROLOGIC DESIGN DATA

Project No. ______ TRACS No. ______
Project Name ______ Date ______
Location/Station AGUA FRIA RIVER NEAR MAYER AZ
Designer ______ Checker ______

A. THE ANNUAL FLOOD PEAK DISCHARGE DATA SET CONTAINS:

- NO ZERO FLOW YEARS, AND
- NO LOW OUTLIERS, AND
- NO HIGH OUTLIERS, AND
- NO HISTORIC DATA, AND
- NO EXTRAORDINARY FLOODS.

PLOTTING POSITION EQUATION

$$P_e = \frac{m-0.4}{N_s+0.2} \qquad \text{FOR } m = 1 \ldots N_s$$

WHERE LENGTH OF SYSTEMATIC RECORD, $N_t = 50$

EFFECTIVE LENGTH OF SYSTEMATIC RECORD, $N_s = N_t = 50$

$$P_e = \frac{m-0.4}{50+0.2} = 0.0199\,(m-0.4) \qquad \text{FOR } m = 1 \ldots 50$$

AT $m = 1$, $P_e = 0.0199\,(1-0.4) = 0.0120 \Rightarrow T_r = 84^{\text{YRS}}$

ARIZONA DEPARTMENT OF TRANSPORTATION
HYDROLOGIC DESIGN DATA

Project No. ______________ TRACS No. ______________
Project Name ______________ Date ______________
Location/Station AGUA FRIA RIVER NEAR MAYER, AZ
Designer ______________ Checker ______________

FIGURE 9-3
FLOOD FREQUENCY ANALYSIS
PLOTTING POSITION CALCULATION FORM

Page 1 of 3

Gage Station Name AGUA FRIA RIVER NEAR MAYER, AZ
Gage Station No. 09512500 **Drainage Area** 1523 **sq. km.**
Period of Systematic Record 1940 - 1989

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 (m^3/s) (1)	RANK (2)	PLOTTING POSITION	
		P_e (3)	T_r (4)
937	1	0.012	83.7
722	2	0.032	31.3
561	3	0.052	19.2
518	4	0.072	13.9
368	5	0.092	10.8
362	6	0.112	8.9
362	7	0.131	7.6
343	8	0.151	6.6
303	9	0.171	5.8
289	10	0.191	5.2
282	11	0.211	4.7
280	12	0.231	4.3
275	13	0.251	4.0
275	14	0.271	3.7
255	15	0.291	3.4
232	16	0.311	3.2
212	17	0.331	3.0
211	18	0.351	2.8
206	19	0.371	2.7

ARIZONA DEPARTMENT OF TRANSPORTATION
HYDROLOGIC DESIGN DATA

Project No. ______________ TRACS No. ______________
Project Name ______________ Date ______________
Location/Station AGUA FRIA RIVER NEAR MAYER AZ
Designer ______________ Checker ______________

FLOOD FREQUENCY ANALYSIS
PLOTTING POSITION CALCULATION FORM

Page 2 of 3

FLOOD PEAK DISCHARGE (m^3/s) (1)	RANK (2)	PLOTTING POSITION	
		P_e (3)	T_r (4)
197	20	0.390	2.6
195	21	0.410	2.4
193	22	0.430	2.3
193	23	0.450	2.22
178	24	0.470	2.13
172	25	0.490	2.04
168	26	0.510	1.96
156	27	0.530	1.87
155	28	0.550	1.82
140	29	0.570	1.75
137	30	0.590	1.69
131	31	0.610	1.64
129	32	0.629	1.59
112	33	0.649	1.54
109	34	0.669	1.49
108	35	0.689	1.45
103	36	0.709	1.41
99	37	0.729	1.37
86	38	0.749	1.34
82	39	0.769	1.30
81	40	0.789	1.27
77	41	0.809	1.24
74	42	0.829	1.21
71	43	0.849	1.18
70	44	0.869	1.15
70	45	0.888	1.13
62	46	0.908	1.10
61	47	0.928	1.08

FIGURE 9-3 Continued

ARIZONA DEPARTMENT OF TRANSPORTATION
HYDROLOGIC DESIGN DATA

Project No. ______ TRACS No. ______
Project Name ______ Date ______
Location/Station AGUA FRIA RIVER NEAR MAYER, AZ
Designer ______ Checker ______

FLOOD FREQUENCY ANALYSIS
PLOTTING POSITION CALCULATION FORM

Page 3 of 3

FLOOD PEAK DISCHARGE (m^3/s) (1)	RANK (2)	PLOTTING POSITION P_e (3)	T_r (4)
46	48	0.948	1.05
36	49	0.968	1.03
21	50	0.988	1.01

FIGURE 9-3 Continued

ARIZONA DEPARTMENT OF TRANSPORTATION
HYDROLOGIC DESIGN DATA

Project No. ______________________ TRACS No. ______________________
Project Name ______________________ Date ______________________
Location/Station AGUA FRIA RIVER NEAR MAYER, AZ
Designer ______________________ Checker ______________________

FIGURE 9-10
FLOOD FREQUENCY ANALYSIS
WORK SHEET FOR LOG-NORMAL CONFIDENCE LIMITS

Gage Station Name AGUA FRIA RIVER NEAR MAYER, AZ
Gage Station No. 09512500

Confidence Level (C.L.) = 90 %

$Q_{2\text{-yr}}$ = 150 m^3/s $\alpha = \frac{100 - C.L.}{100}$ = 0.1

$Q_{100\text{-yr}}$ = 980 m^3/s $U_{1-\frac{\alpha}{2}}$ = 1.645

N_C = 43

$\overline{Y} = \log_{10}(Q_{2-yr}) = \log_{10}(150)$ = 2.1761

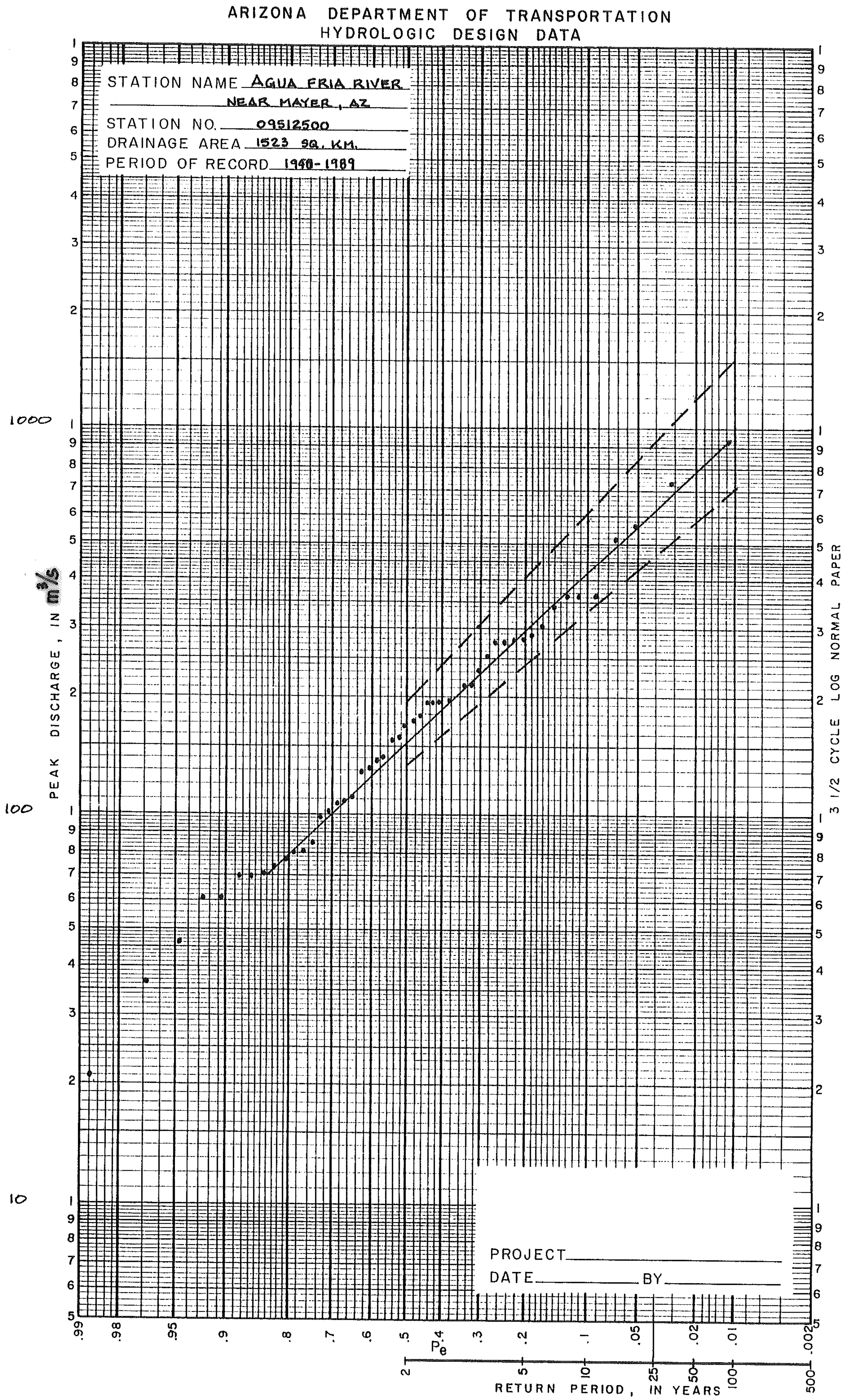
$S_{ln} = \frac{\log_{10} Q_{100-yr} - \log_{10} Q_{2-yr}}{2.327} = \frac{\log_{10}(980) - \log_{10}(150)}{2.327}$ = 0.3503

T Years (1)	$U_{1-\frac{1}{T}}$ (2)	Y_T (a) (3)	S_T (b) (4)	Limits, in m^3/s (c) Upper (5)	Lower (6)
2	0.0	2.1761	0.0534	184	123
5	0.842	2.4711	0.0622	314	234
10	1.282	2.6252	0.0721	554	321
25	1.751	2.7895	0.0850	850	446
50	2.052	2.8949	0.0941	1121	550
100	2.327	2.9912	0.1029	1447	663

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

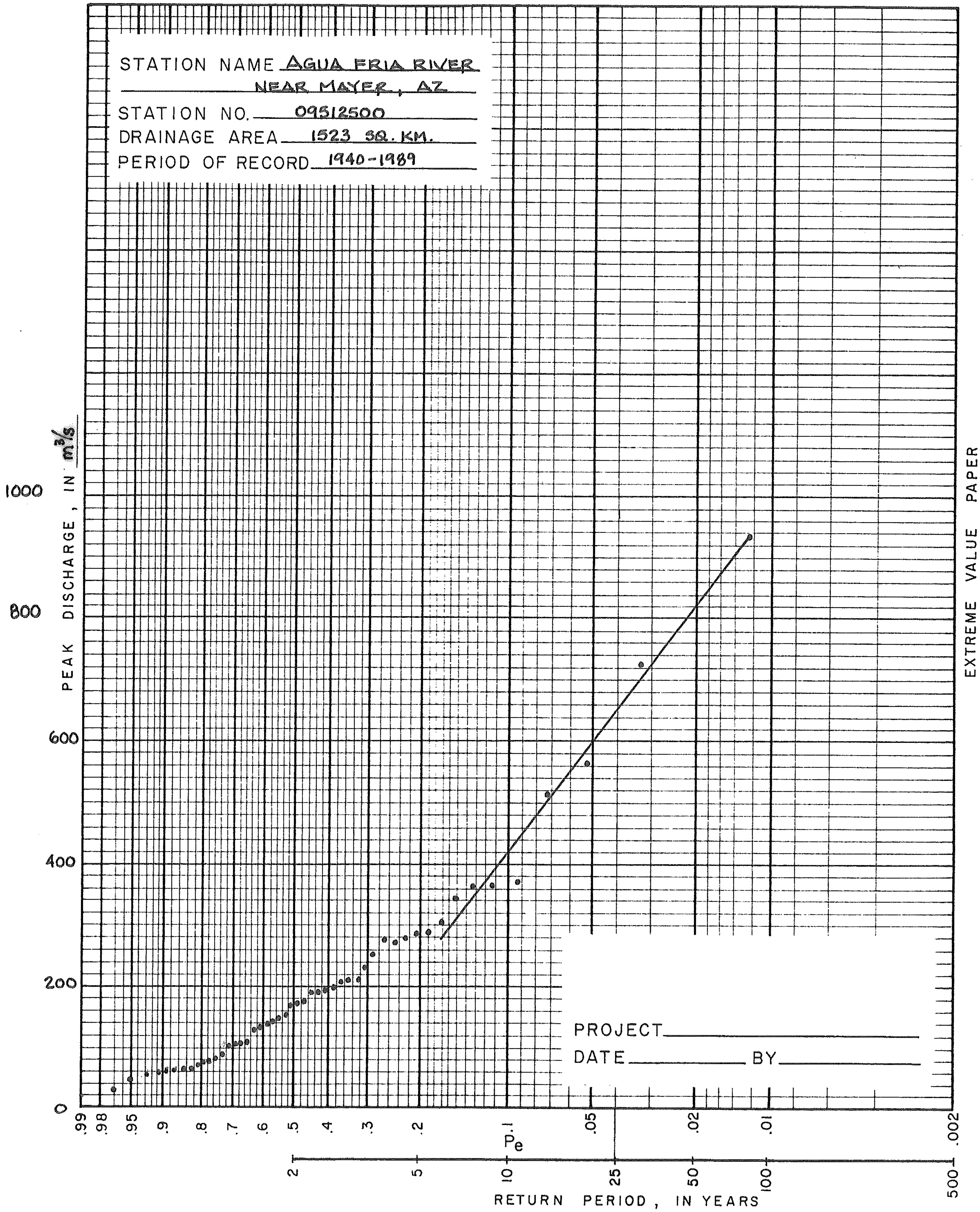
(c) $Q_L = 10^{(Y_T \pm U_{1-\frac{\alpha}{2}} S_T)}$

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

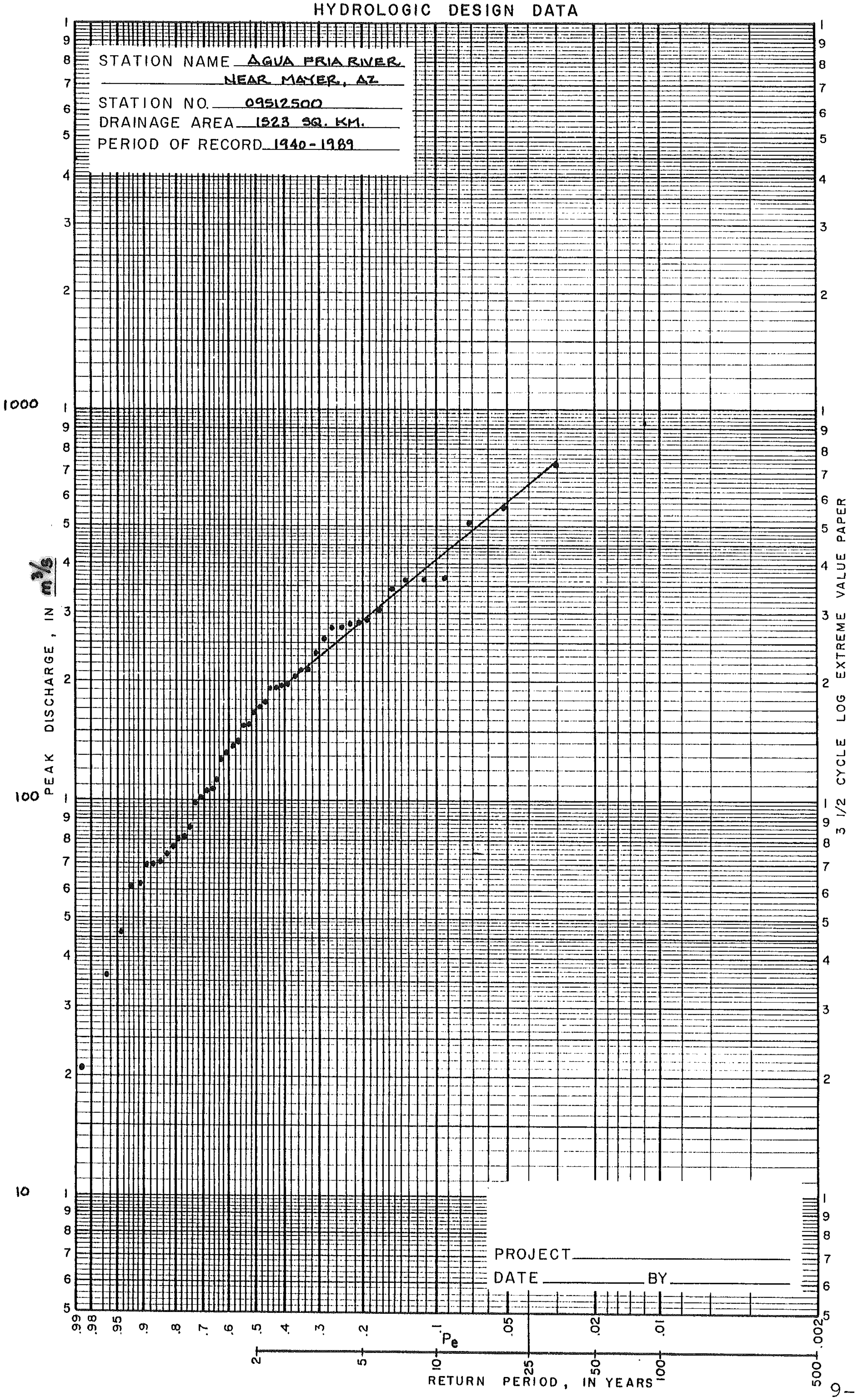


ARIZONA DEPARTMENT OF TRANSPORTATION
HYDROLOGIC DESIGN DATA
STATION NAME AGUA FRIA RIVER NEAR MAYER, AZ
STATION NO. 09512500
DRAINAGE AREA 1523 SQ. KM.
PERIOD OF RECORD 1940-1989
PEAK DISCHARGE, IN m³/s
1000
100
10
3 1/2 CYCLE LOG NORMAL PAPER
PROJECT
DATE
BY
P_e
RETURN PERIOD, IN YEARS

ARIZONA DEPARTMENT OF TRANSPORTATION
HYDROLOGIC DESIGN DATA



ARIZONA DEPARTMENT OF TRANSPORTATION
HYDROLOGIC DESIGN DATA



FLOOD FREQUENCY ANALYSIS EXAMPLE No. 9-2

Station Name - Cave Creek near Cave Creek, Arizona
Station Number - 09512300
Drainage Area - 313 square kilometers
Period of Record - 1958 through 1979 and 1981 through 1989

Flood Data

A broken, 31 year systematic record is available, and the entire record was used in the analysis. All annual floods are considered to be caused by rainfall. There are no historic data. Zero flow years occurred in 1969, 1977, 1981, 1987 and 1989. The high and low floods (other than zero flow years) of record are 351 m^3/s (1968) and 4.2 m^3/s (1984), respectively. The record is considered stationary.

Flood Frequency Analysis

The high outlier limit is calculated at 974 m^3/s, and no high outliers are identified. The low outlier limit is calculated at 2.4 m^3/s, and no low outliers are identified. No extraordinary floods are identified.

The data set contains zero flow years. The length of the broken, systematic record is for the period 1958 through 1979, and 1981 through 1989 ($N_t = 31$). There are five zero flow years ($Z = 5$). The effective length of the systematic record is 26 years ($N_s = N_t - Z = 31 - 5 = 26$). These parameters are used in calculating the plotting positions.

The annual flood peak discharges are plotted on the three probability papers at their respective plotting positions. The log-normal (LN) graph shows a concave down trend to the data and a poor linear trend to the data with P_e smaller than about 0.34. The log-extreme value (LEV) graph is also concave down and a linear trend to data with P_e smaller than about 0.18. The extreme value (EV) graph shows a good linear trend for data with P_e less than about 0.34. The EV graph is accepted as the best representation of the probability distribution of floods with return periods that are longer than about 3 years.

Confidence limits are set about the EV best fit line. The 11 largest floods ($N_c = 11$) are used to establish the best fit line. The estimated 100-yr flood peak discharge is 413 m^3/s with 90 percent upper and lower confidence limits of 640 m^3/s and 188 m^3/s, respectively.

Discussion

This example illustrates a flood frequency analysis for a data set containing five zero flow years. The EV graph provides the best fit straight line to the large floods (P_e less than 0.34). This is a fairly clear choice of the best graph. The EV graph shows a linear trend for the 11 largest floods. The range for the confidence limits is broad because only the 11 largest floods can be used to establish the best fit line.

09512300 CAVE CREEK NEAR CAVE CREEK, AZ

LOCATION.--Lat 33°47'00", long 112°00'24", in SW¼ sec.12, T.5 N., R.3 E., Maricopa County, Hydrologic Unit 15060106, on left bank, 200 ft upstream from Prescott-to-Mesa transmission line, 5 mi southwest of town of Cave Creek, and 5.0 mi upstream from Cave Creek Dam.

DRAINAGE AREA.--121 mi^2.

ANNUAL PEAK DISCHARGE

WATER YEAR	DATE	ANNUAL PEAK DISCHARGE (FT^3/S)	WATER YEAR	DATE	ANNUAL PEAK DISCHARGE (FT^3/S)
1958	09-12-58	5,680	1974	08-05-74	1,390
1959	08-05-59	3,590	1975	11-02-74	856
1960	10-29-59	8,570	1976	02-09-76	1,260
1961	09-17-61	696	1977	00-00-77	0
1962	12-16-61	280	1978	03-02-78	7,500
1963	08-06-63	1,510	1979	12-18-78	6,900
1964	08-02-64	3,120	1981	00-00-81	0
1965	07-16-65	610	1982	10-02-81	1,200
1966	12-22-65	6,000	1983	03-03-83	1,420
1967	09-06-67	1,800	1984	08-09-84	148
1968	12-19-67	12,400	1985	12-27-84	910
1969	00-00-69	0	1986	07-22-86	1,350
1970	09-05-70	2,700	1987	00-00-87	0
1971	08-04-71	364	1988	08-21-88	170
1972	07-17-72	3,950	1989	00-00-89	0
1973	10-19-72	3,950			

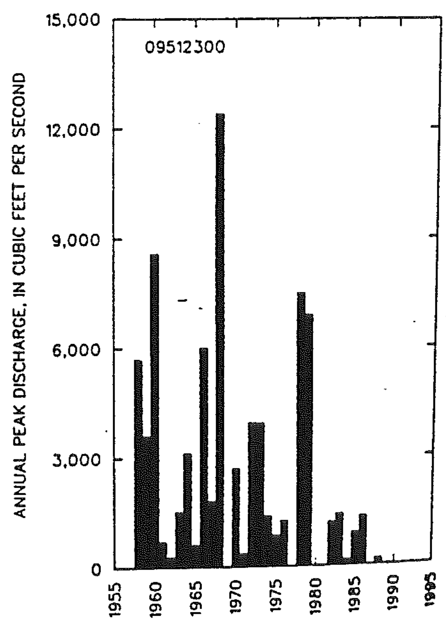
MAGNITUDE AND PROBABILITY OF INSTANTANEOUS PEAK FLOW BASED ON PERIOD OF RECORD 1958-79, 1981-86

DISCHARGE, IN FT^3/S, FOR INDICATED RECURRENCE INTERVAL IN YEARS, AND EXCEEDANCE PROBABILITY, IN PERCENT

2 50%	5 20%	10 10%	25 4%	50 2%	100† 1%
1,740	4,320	6,870	11,200	15,200	20,000

WEIGHTED SKEW (LOGS)= -0.12
MEAN (LOGS)= 3.23
STANDARD DEV. (LOGS)= 0.48

† Reliability of values in column is uncertain, and potential errors are large.



BASIN CHARACTERISTICS

MAIN CHANNEL SLOPE (FT/MI)	STREAM LENGTH (MI)	MEAN BASIN ELEVATION (FT)	FORESTED AREA (PERCENT)	SOIL INDEX	MEAN ANNUAL PRECIPITATION (IN)	RAINFALL INTENSITY, 24-HOUR 2-YEAR (IN)	RAINFALL INTENSITY, 24-HOUR 50-YEAR (IN)
123	18.4	3,470	0.1	1.17	15.7	2.3	4.4

ARIZONA DEPARTMENT OF TRANSPORTATION
HYDROLOGIC DESIGN DATA

Project No. ____________ TRACS No. ____________
Project Name ____________ Date ____________
Location/Station CAVE CREEK NEAR CAVE CREEK, AZ
Designer ____________ Checker ____________

FIGURE 9-2
FLOOD FREQUENCY ANALYSIS
DATA COMPILATION FORM

Gage Station Name CAVE CREEK NEAR CAVE CREEK, AZ
Gage Station No. 09512300 **Drainage Area** 313 **sq. km.**
Period of Systematic Record 1958-1979, 1981-1986, 1988-1989

WATER YEAR (1)	ANNUAL PEAK DISCHARGE (cfs) (2)	ANNUAL PEAK DISCHARGE (m^3/s) (3)	DATE (4)	FLOOD TYPE [a] (5)	COMMENTS (6)
1958	5680	161	12 SEPT 58	R	
59	3590	102	5 AUG 59	R	
60	8570	243	29 OCT 59	R	
61	696	20	17 SEPT 61	R	
62	280	8	16 DEC 61	R	
63	1510	43	6 AUG 63	R	
64	3120	88	2 AUG 64	R	
65	610	17	16 JULY 65	R	
66	6000	170	22 DEC 65	R	
67	1800	51	6 SEPT 67	R	
68	12400	351	19 DEC 67	R	
69	0	0	—		ZERO FLOW YEAR
70	2700	76	5 SEPT 70	R	
71	364	10	4 AUG 71	R	
72	3950	112	17 JULY 72	R	
73	3950	112	19 OCT 72	R	
74	1390	39	5 AUG 74	R	
75	856	24	2 NOV 74	R	
76	1260	36	9 FEB 76	R	ZERO FLOW YEAR
77	0	0	—		
78	7500	212	2 MAR 78	R	
79	6900	195	18 DEC 78	R	

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

Note: (3) = 0.0283 * (2) for cfs to m^3/s conversion

ARIZONA DEPARTMENT OF TRANSPORTATION
HYDROLOGIC DESIGN DATA

Project No. ______________ TRACS No. ______________
Project Name ______________ Date ______________
Location/Station CAVE CREEK NEAR CAVE CREEK, AZ
Designer ______________ Checker ______________

FLOOD FREQUENCY ANALYSIS
DATA COMPILATION FORM

Page ____ of ____

WATER YEAR (1)	ANNUAL PEAK DISCHARGE (cfs) (2)	ANNUAL PEAK DISCHARGE (m^3/s) (3)	DATE (4)	FLOOD TYPE[a] (5)	COMMENTS (6)
1980	—	—	—	—	BROKEN
81	0	0	—		ZERO FLOW YEAR
82	1200	34	2 OCT 81	R	
83	1420	40	3 MAR 83	R	
84	148	4	9 AUG 84	R	
85	910	26	27 DEC 84	R	
86	1350	38	22 JULY 86	R	
87	0	0	—		ZERO FLOW YEAR
88	170	5	21 AUG 88	R	
89	0	0	—		ZERO FLOW YEAR

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

Note: (3) = 0.0283 * (2) for cfs to m^3/s conversion

FIGURE 9-2 Continued

ARIZONA DEPARTMENT OF TRANSPORTATION
HYDROLOGIC DESIGN DATA

Project No. ______ TRACS No. ______
Project Name ______ Date ______
Location/Station CAVE CREEK NEAR CAVE CREEK, AZ
Designer ______ Checker ______

A. TEST FOR HIGH & LOW OUTLIERS

$\overline{\text{LOG } Q} = 1.6796$ $N = 26$

$\text{LOG } S = 0.5233$ $K_N = 2.502$

- HIGH OUTLIERS

$$\text{LOG } Q_H = \overline{\text{LOG } Q} + K_N \text{ LOG } S$$
$$= 1.6796 + 2.502(0.5233)$$
$$= 2.989$$
$$Q_H = 975 \text{ m}^3/\text{s}$$

THERE ARE NO Q's > 975 m^3/s ∴ NO HIGH OUTLIERS

- LOW OUTLIERS

$$\text{LOG } Q_L = \overline{\text{LOG } Q} - K_N \text{ LOG } S$$
$$= 1.6796 - 2.502(0.5233)$$
$$= 0.3703$$
$$Q_L = 2.3 \text{ m}^3/\text{s}$$

THERE ARE NO Q's < 2.3 m^3/s ∴ NO LOW OUTLIERS

ARIZONA DEPARTMENT OF TRANSPORTATION
HYDROLOGIC DESIGN DATA

Project No. ______ TRACS No. ______
Project Name ______ Date ______
Location/Station CAVE CREEK NEAR CAVE CREEK, AZ
Designer ______ Checker ______

B. THE ANNUAL FLOOD PEAK DISCHARGE DATA SET CONTAINS:

- ZERO FLOWS YEARS, AND/OR
- LOW OUTLIERS, AND
- NO HIGH OUTLIERS, AND
- NO HISTORIC DATA, AND
- NO EXTRAORDINARY FLOODS.

PLOTTING POSITION EQUATION

$$P_e = \left(\frac{N_t - Z}{N_t}\right)\left(\frac{m-0.4}{N_s+0.2}\right) \quad \text{FOR } m=1 \ldots N_s$$

WHERE LENGTH OF SYSTEMATIC RECORD; $N_t = 31$
NUMBER OF ZERO FLOW YEARS AND/OR
NUMBER OF LOW QUALIFIERS; $Z = 5$
EFFECTIVE LENGTH OF SYSTEMATIC RECORD; $N_s = N_t - Z = 26$

$$P_e = \left(\frac{26}{31}\right)\left(\frac{m-0.4}{26+0.2}\right) = 0.0320(m-0.4) \quad \text{FOR } m=1 \ldots 26$$

AT $m=1$, $P_e = 0.0320(1-0.4) = 0.0192 \Rightarrow T_r = 52^{\text{YRS}}$

ARIZONA DEPARTMENT OF TRANSPORTATION
HYDROLOGIC DESIGN DATA

Project No. ______ TRACS No. ______
Project Name ______ Date ______
Location/Station CAVE CREEK NEAR CAVE CREEK, AZ
Designer ______ Checker ______

FIGURE 9-3
FLOOD FREQUENCY ANALYSIS
PLOTTING POSITION CALCULATION FORM

Page 1 of 2

Gage Station Name CAVE CREEK NEAR CAVE CREEK, AZ
Gage Station No. 09512300 **Drainage Area** 313 **sq. km.**
Period of Systematic Record 1958-1979, 1981-1986, 1988-1989

Check if the data contains any of the following:

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

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

FLOOD PEAK DISCHARGE (m^3/s) (1)	RANK (2)	PLOTTING POSITION P_e (3)	T_r (4)
351	1	0.0192	52.1
243	2	0.0512	19.5
212	3	0.0832	12.0
195	4	0.1152	8.7
170	5	0.1472	6.8
161	6	0.1792	5.6
112	7	0.2112	4.7
112	8	0.2432	4.1
102	9	0.2752	3.6
88	10	0.3072	3.3
76	11	0.3392	2.9
51	12	0.3712	2.7
43	13	0.4032	2.5
40	14	0.4352	2.3
39	15	0.4672	2.1
38	16	0.4992	2.0
36	17	0.5312	1.9
34	18	0.5632	1.8
26	19	0.5952	1.7

ARIZONA DEPARTMENT OF TRANSPORTATION
HYDROLOGIC DESIGN DATA

Project No. ______ TRACS No. ______
Project Name ______ Date ______
Location/Station CAVE CREEK NEAR CAVE CREEK, AZ
Designer ______ Checker ______

FLOOD FREQUENCY ANALYSIS
PLOTTING POSITION CALCULATION FORM

Page 2 of 2

FLOOD PEAK DISCHARGE (m^3/s) (1)	RANK (2)	PLOTTING POSITION	
		P_e (3)	T_r (4)
24	20	0.6272	1.6
20	21	0.6592	1.5
17	22	0.6912	1.45
10	23	0.7232	1.38
8	24	0.7552	1.32
5	25	0.7872	1.27
4	26	0.8192	1.22

FIGURE 9-3 Continued

ARIZONA DEPARTMENT OF TRANSPORTATION
HYDROLOGIC DESIGN DATA

Project No. ______ TRACS No. ______
Project Name ______ Date ______
Location/Station CAVE CREEK NEAR CAVE CREEK, AZ
Designer ______ Checker ______

FIGURE 9-11
FLOOD FREQUENCY ANALYSIS
WORK SHEET FOR EXTREME VALUE CONFIDENCE LIMITS

Gage Station Name CAVE CREEK NEAR CAVE CREEK, AZ
Gage Station No. 09512300

Confidence Level (C.L.) = 90 %

$Q_{2\text{-yr}}$ = 25 m^3/s $\qquad \alpha = \frac{100 - C.L.}{100} = 0.1$

$Q_{100\text{-yr}}$ = 412 m^3/s $\qquad U_{1-\frac{\alpha}{2}} = 1.645$

$N_C = 16$

$$A = \frac{Q_{100-yr} - Q_{2-yr}}{4.2336} = \frac{(412) - (25)}{4.2336} = 91.4116$$

$$B = Q_{2-yr} - .3665\ A = (25) - .3665\ (91.4116) = -8.5024$$

$$\overline{Q} = B + .5772\ A = (-8.5024) + .5772\ (91.4116) = 44.2604$$

$$S_{ev} = \frac{A}{.7797} = \frac{(91.4116)}{.7797} = 117.2395$$

T Years (1)	K (2)	Z (a) (3)	S_T (b) (4)	Q_T (c) (5)	Limits, in m^3/s (d) Upper (6)	Lower (7)
2	-.1643	.9179	26.9035	24.9980	69	-19
5	.7195	1.545	45.3072	128.6142	203	54
10	1.3046	2.087	61.1932	197.2111	298	96
25	2.0438	2.814	82.5044	283.8745	420	148
50	2.5923	3.368	98.7274	348.1804	511	186
100	3.1367	3.924	115.0119	412.0055	601	223

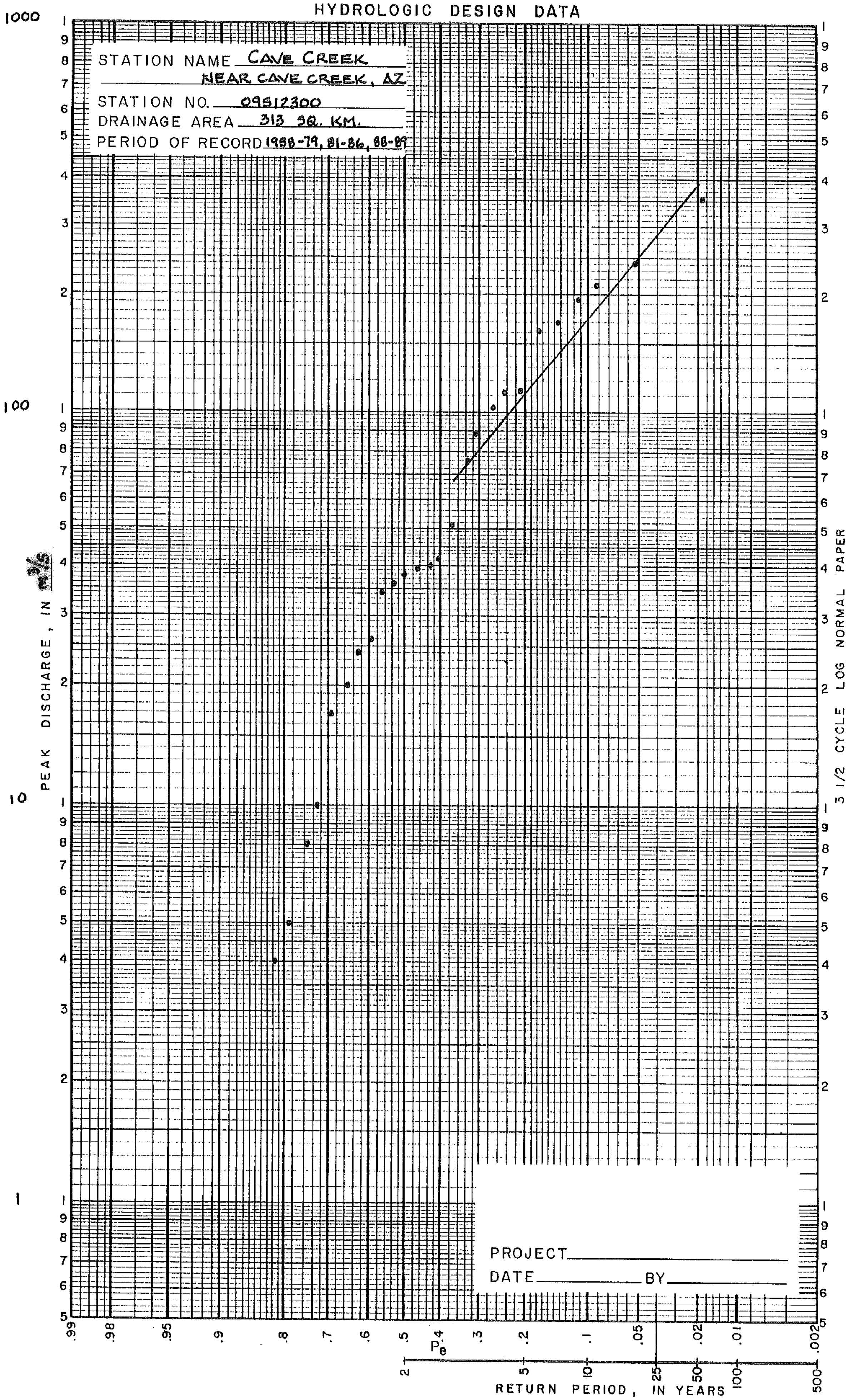
(a) $Z = (1.0 + 1.1396\ K + 1.1K^2)^{\frac{1}{2}}$

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

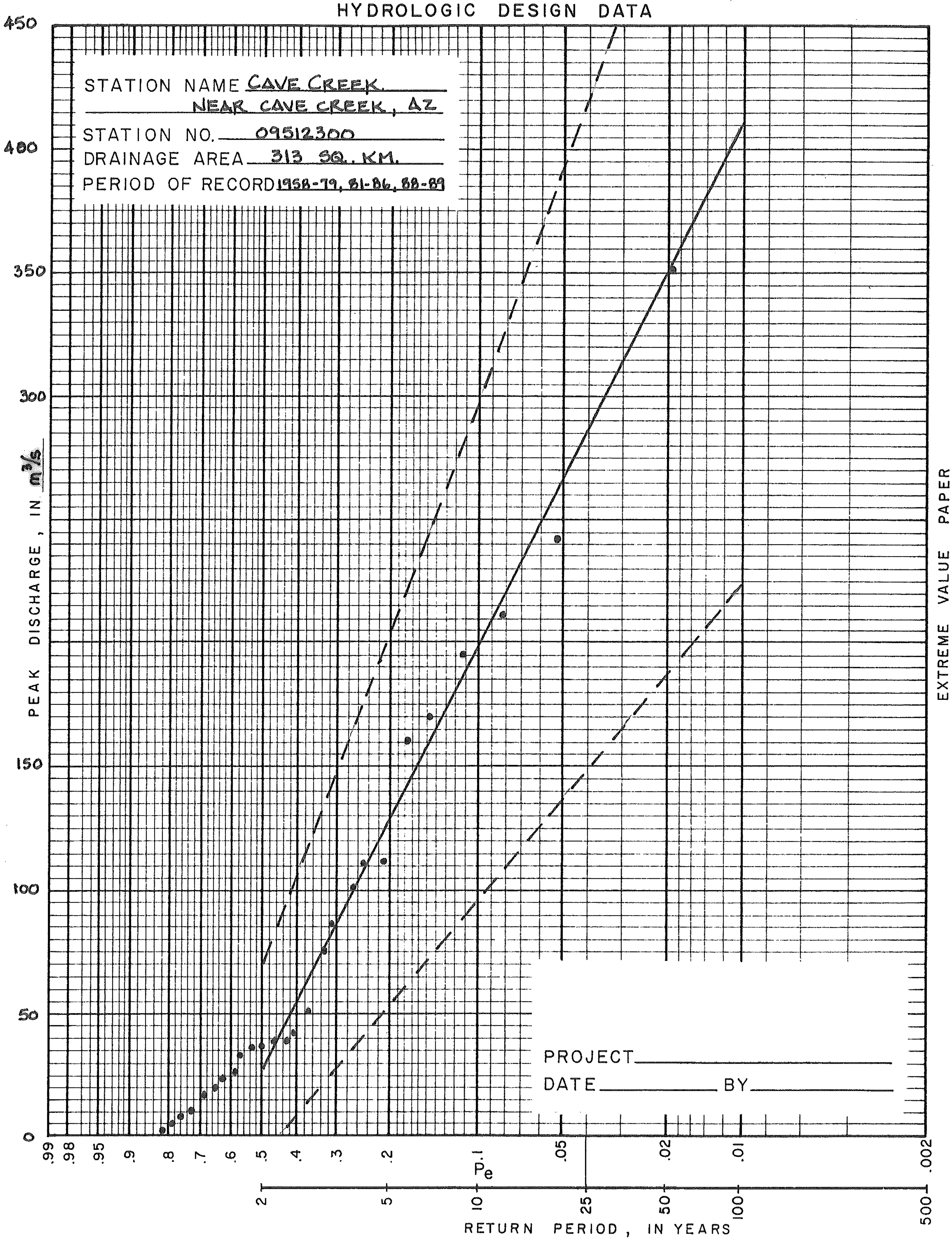
(b) $S_T = S_{ev} \left(\frac{Z}{N_c^{\frac{1}{2}}} \right)$

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

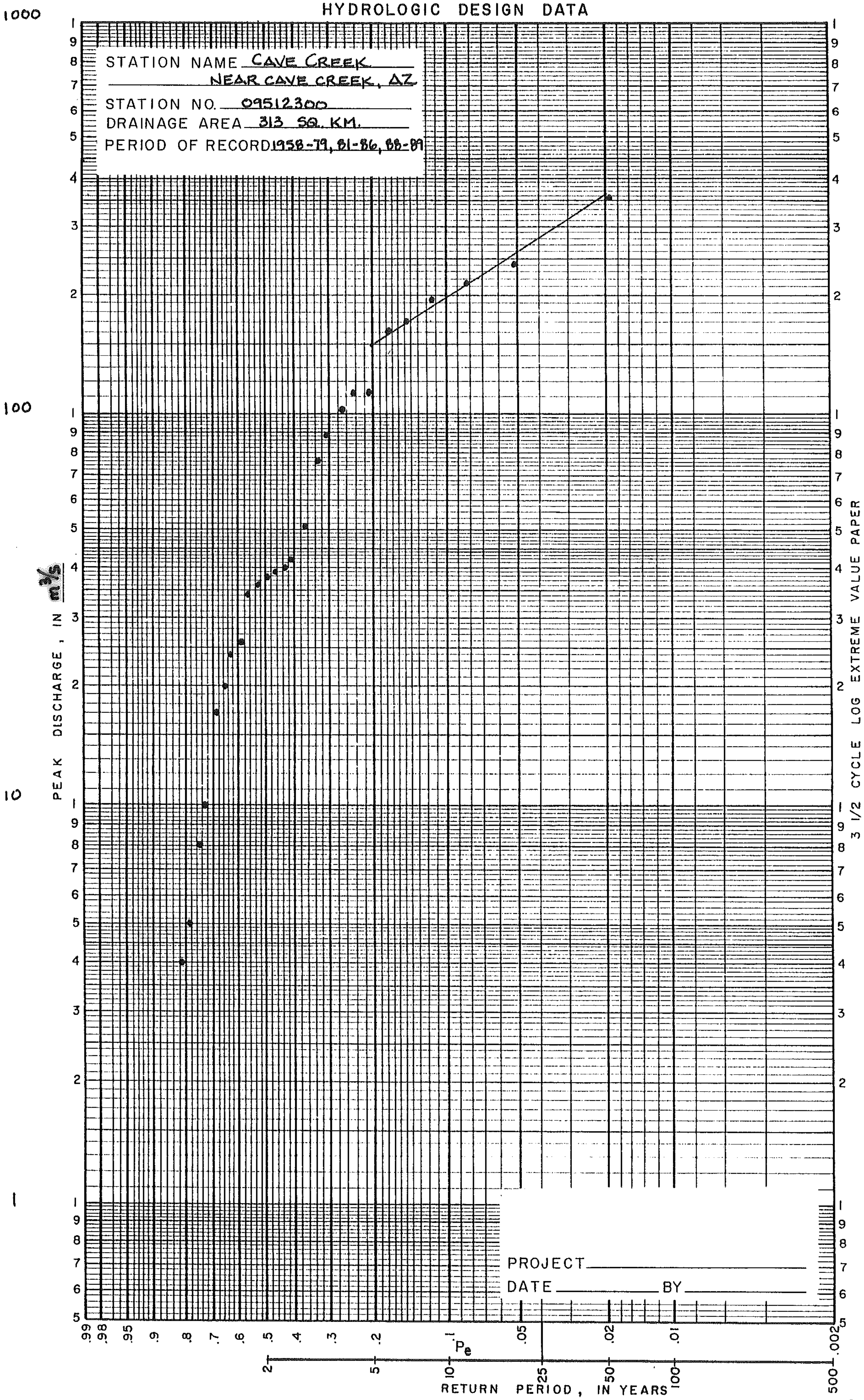
ARIZONA DEPARTMENT OF TRANSPORTATION
HYDROLOGIC DESIGN DATA



ARIZONA DEPARTMENT OF TRANSPORTATION
HYDROLOGIC DESIGN DATA



ARIZONA DEPARTMENT OF TRANSPORTATION
HYDROLOGIC DESIGN DATA



FLOOD FREQUENCY ANALYSIS EXAMPLE No. 9-3

Station Name - Hassayampa River near Wickenburg, Arizona
Station Number - 09515500
Drainage Area - 1,080 square kilometers
Period of Record - 1938, 1946 through 1982

Flood Data

A broken, 38 year systematic record is available, and the entire record was used in the analysis. All annual floods are considered to be caused by rainfall. There are no zero flow years. The high and low floods of record are 1,642 m^3/s (1970) and 4.4 m^3/s (1975), respectively. The 1925 (722 m^3/s), 1927 (765 m^3/s), and 1937 (623 m^3/s) floods are indicated in the records of the U.S. Geological Survey (USGS) as historic data. The 1951 flood (765 m^3/s) is indicated in the records of the USGS as being the largest since 1927. The 1970 flood (1,642 m^3/s) is indicated in the records of the USGS as being the largest since 1890. The record is considered stationary.

Flood Frequency Analysis

The high outlier limit is calculated at 3,680 m^3/s, and no high outliers are identified. The low outlier limit is calculated at 3 m^3/s, and no low outliers are identified. Extraordinary floods are identified for 1951 (765 m^3/s) and 1970 (1,640 m^3/s) because these floods, from the systematic record, are known to be larger than any flood since 1927 and 1890, respectively, prior to the start of the systematic record. The 1980 flood (680 m^3/s) is also extraordinary because it is larger than the 1937 historic data (623 m^3/s). The station was discontinued after 1982; however, the USGS records that were used are for a period through 1989. Because of the presence of historic data and extraordinary floods, the effective length of record can be extended, and because of the information that is available, the record can be extended at both ends of the record. The record can be extended backward to 1980 because the USGS records indicate that the largest flood of record (1,640 m^3/s) is the largest since 1890. The record can also be extended for the period 1982 to 1989 because estimated floods would be reported by the USGS, or others, for that period if floods had occurred that were as large as or larger than any of the six historic and extraordinary floods (623 m^3/s to 1,640 m^3/s).

The effective record length, as previously described, is for the period 1890 through 1989 ($N = 100$). The length of the systematic record is for the period 1938 and 1946 through 1982 ($N_t = 38$). There are no zero flow years or low outliers ($Z = 0$), and the effective length of the systematic record is 38 years ($N_s = N_t - Z = 38 - 0 = 38$). There are three historic floods ($h = 3$), and there are three extraordinary floods in the systematic record ($e = 3$). The sum of historic plus extraordinary floods is six ($k = h + e = 3 + 3 = 6$). There are 41 systematic plus historic floods ($N_G = N_s + h = 38 + 3 = 41$). The parameters are used in calculating the plotting positions.

The annual flood peak discharges are plotted on the three probability papers at their respective plotting positions. The extreme value (EV) graph does not show a linear trend. The log-extreme value (LEV) graph shows a concave down trend to the data points, and a weak linear trend to data with P_e less than 0.42. The log-normal (LN) shows a slight break in the data points at about $P_e = 0.45$, and a reasonable linear trend for the data points with P_e less than 0.42. The LN graph is selected as the best representation of the probability distribution of floods with return periods that are longer than about 3 years.

Confidence limits are set about the LN best fit line. The 20 largest floods ($N_c = 20$) are used to establish the best fit line. The estimated 100-yr flood peak discharge is 1,190 m^3/s with 90 percent upper and lower confidence limits of 2,520 m^3/s and 561 m^3/s, respectively.

Discussion

This example illustrates a flood frequency analysis for a data set containing historic data and extraordinary floods. The effective record length was extended beyond the length of the systematic record. The LN graph is selected as the best straight line fit to the 20 largest floods. This is a clear choice of the best graph paper to select. The range for the confidence limits is somewhat broad because only the 20 largest floods can be used to establish the best fit line.

09515500 HASSAYAMPA RIVER AT BOX DAMSITE, NEAR WICKENBURG, AZ

LOCATION.--Lat 34°02'42", long 112°42'33", in SW¼SE¼ sec.7, T.8 N., R.4 W., Yavapai County, Hydrologic Unit 15070103, on right bank at Box damsite, 5.5 mi northeast of Wickenburg.

DRAINAGE AREA.--417 mi ^{2}.

REMARKS.--Small diversions for irrigation and mining above station.

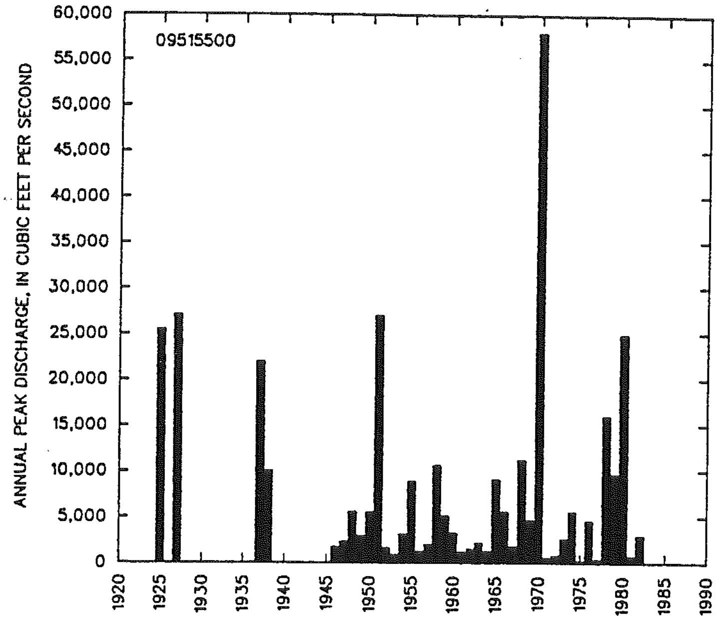
ANNUAL PEAK DISCHARGE

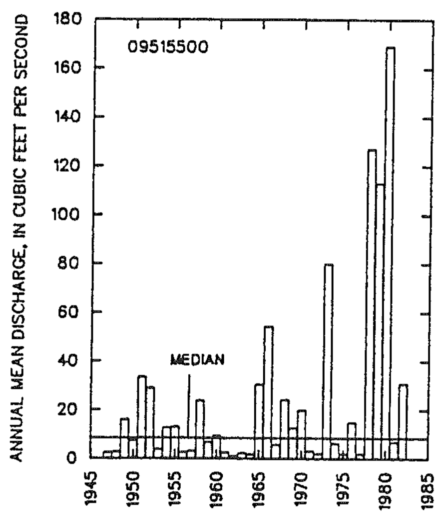
WATER YEAR	DATE	ANNUAL PEAK DISCHARGE (FT^3/S)	DISCHARGE CODES	WATER YEAR	DATE	ANNUAL PEAK DISCHARGE (FT^3/S)	DISCHARGE CODES
1925	09-19-25	25,500	HP	1963	08-17-63	2,150	
1927	02-16-27	27,100	HP	1964	07-14-64	1,230	
1937	02-07-37	22,000	HP	1965	09-02-65	9,060	
1938	03-03-38	10,000		1966	12-10-65	5,560	
1946	08-11-46	1,710		1967	12-07-66	1,740	
1947	08-08-47	2,300		1968	12-19-67	11,200	
1948	08-05-48	5,600		1969	09-13-69	4,630	
1949	09-26-49	2,910		1970	09-05-70	[2]58,000	
1950	10-18-49	5,500		1971	08-25-71	556	
1951	08-29-51	[1]27,000		1972	08-27-72	800	
1952	12-30-51	1,590		1973	10-07-72	2,600	
1953	07-18-53	865		1974	07-20-74	5,560	
1954	03-23-54	3,090		1975	07-28-75	154	
1955	07-23-55	8,840		1976	02-09-76	4,560	
1956	08-18-56	1,210		1977	08-15-77	315	
1957	08-10-57	1,980		1978	03-02-78	16,000	
1958	09-05-58	10,600		1979	03-28-79	9,640	
1959	08-24-59	5,110		1980	02-19-80	24,900	
1960	12-26-59	3,210		1981	07-10-81	698	
1961	08-19-61	1,150		1982	03-15-82	2,940	
1962	09-21-62	1,510					

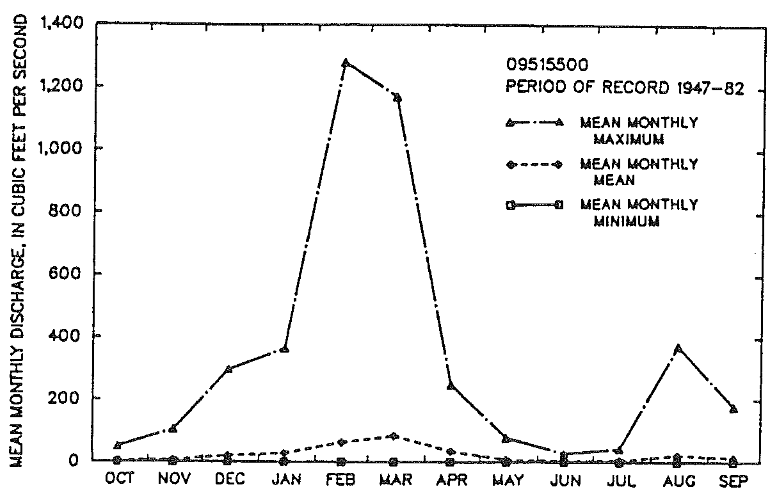
[1] Highest since 1927.
[2] Highest since 1890.

BASIN CHARACTERISTICS

MAIN CHANNEL SLOPE (FT/MI)	STREAM LENGTH (MI)	MEAN BASIN ELEVA-TION (FT)	FORESTED AREA (PERCENT)	SOIL INDEX	MEAN ANNUAL PRECIPI-TATION (IN)	RAINFALL INTENSITY, 24-HOUR 2-YEAR (IN)	RAINFALL INTENSITY, 24-HOUR 50-YEAR (IN)
71.0	45.0	4,750	9.6	1.0	19.3	2.4	4.7


09515500
ANNUAL PEAK DISCHARGE, IN CUBIC FEET PER SECOND
60,000
55,000
50,000
45,000
40,000
35,000
30,000
25,000
20,000
15,000
10,000
5,000
0
1920 1925 1930 1935 1940 1945 1950 1955 1960 1965 1970 1975 1980 1985 1990


09515500
ANNUAL MEAN DISCHARGE, IN CUBIC FEET PER SECOND
180
160
140
120
100
80
60
40
20
0
MEDIAN
1945 1950 1955 1960 1965 1970 1975 1980 1985


09515500
PERIOD OF RECORD 1947-82
MEAN MONTHLY MAXIMUM
MEAN MONTHLY MEAN
MEAN MONTHLY MINIMUM
MEAN MONTHLY DISCHARGE, IN CUBIC FEET PER SECOND
1,400
1,200
1,000
800
600
400
200
0
OCT NOV DEC JAN FEB MAR APR MAY JUN JUL AUG SEP

ARIZONA DEPARTMENT OF TRANSPORTATION
HYDROLOGIC DESIGN DATA

Project No. ______ TRACS No. ______
Project Name ______ Date ______
Location/Station HASSAYAMPA RIVER NEAR WICKENBURG, AZ
Designer ______ Checker ______

FIGURE 9-2
FLOOD FREQUENCY ANALYSIS
DATA COMPILATION FORM

Page 1 of ____

Gage Station Name HASSAYAMPA RIVER NEAR WICKENBURG, AZ
Gage Station No. 09515500 **Drainage Area** 1080 **sq. km.**
Period of Systematic Record 1938, 1946 - 1982

WATER YEAR (1)	ANNUAL PEAK DISCHARGE (cfs) (2)	ANNUAL PEAK DISCHARGE (m^3/s) (3)	DATE (4)	FLOOD TYPE [a] (5)	COMMENTS (6)
1925	25500	722	19 SEPT 25	R	HISTORIC
1927	27100	767	16 FEB 27	R	HISTORIC
1937	22000	623	7 FEB 37	R	HISTORIC
1938	10000	283	3 MAR 38	R	
1939-45	—	—	—		BROKEN
1946	1710	48	11 AUG 46	R	
47	2300	65	8 AUG 47	R	
48	5600	159	5 AUG 48	R	
49	2910	82	26 SEPT 49	R	
50	5500	156	18 OCT 49	R	
51	27000	765	29 AUG 51	R	EXTRAORDINARY
52	1590	45	30 DEC 51	R	
53	865	24	18 JULY 53	R	
54	3090	88	23 MAR 54	R	
55	8840	250	23 JULY 55	R	
56	1210	34	18 AUG 56	R	
57	1980	56	10 AUG 57	R	
58	10600	300	5 SEPT 58	R	
59	5110	145	24 AUG 59	R	
60	3210	91	26 DEC 59	R	
61	1150	33	19 AUG 61	R	
62	1510	43	21 SEPT 62	R	

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

Note: (3) = 0.0283 * (2) for cfs to m^3/s conversion

ARIZONA DEPARTMENT OF TRANSPORTATION
HYDROLOGIC DESIGN DATA

Project No. ____ TRACS No. ____
Project Name ____ Date ____
Location/Station HASSAYAMPA RIVER NEAR WICKENBURG, AZ
Designer ____ Checker ____

FLOOD FREQUENCY ANALYSIS DATA COMPILATION FORM

Page 2 of 2

WATER YEAR (1)	ANNUAL PEAK DISCHARGE (cfs) (2)	ANNUAL PEAK DISCHARGE (m^3/s) (3)	DATE (4)	FLOOD TYPE[a] (5)	COMMENTS (6)
1963	2150	61	17 AUG 63	R	
64	1230	35	14 JULY 64	R	
65	9060	257	2 SEPT 65	R	
66	5560	157	10 DEC 65	R	
67	1740	49	7 DEC 66	R	
68	11200	317	19 DEC 67	R	
69	4630	131	13 SEPT 69	R	
70	58000	1643	5 SEPT 70	R	EXTRAORDINARY
71	556	16	25 AUG 71	R	
72	800	23	24 AUG 72	R	
73	2600	74	7 OCT 72	R	
74	5560	157	20 JULY 74	R	
75	154	4	28 JULY 75	R	
76	4560	129	9 FEB 76	R	
77	315	9	15 AUG 77	R	
78	16000	453	2 MAR 78	R	
79	9640	273	28 MAR 79	R	
80	24900	705	19 FEB 80	R	EXTRAORDINARY
81	698	20	10 JULY 81	R	
82	2940	83	15 MAR 82	R	

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

Note: (3) = 0.0283 * (2) for cfs to m^3/s conversion

FIGURE 9-2 Continued

ARIZONA DEPARTMENT OF TRANSPORTATION
HYDROLOGIC DESIGN DATA

Project No. ______________________ TRACS No. ______________________
Project Name ______________________ Date ______________________
Location/Station HASSAYAMPA RIVER NEAR WICKENBURG, AZ
Designer ______________________ Checker ______________________

A. TEST FOR HIGH & LOW OUTLIERS

$\overline{LOG\,Q} = 2.0249$ $N_G = 41$
$LOG\,S = 0.5726$ $K_N = 2.692$

- HIGH OUTLIERS

$$
\begin{aligned}
LOG\,Q_H &= \overline{LOG\,Q} + K_N\,LOG\,S \\
&= 2.0249 + 2.692(0.5726) \\
&= 3.5643 \\
\underline{Q_H} &= \underline{3684\ m^3/s}
\end{aligned}
$$

THERE ARE NO Q'S > 3684 m^3/s ∴ NO HIGH OUTLIERS

- LOW OUTLIERS

$$
\begin{aligned}
LOG\,Q_L &= \overline{LOG\,Q} - K_N\,LOG\,S \\
&= 2.0249 - 2.692(0.5726) \\
&= 0.4835 \\
\underline{Q_L} &= \underline{3\ m^3/s}
\end{aligned}
$$

THERE ARE NO Q'S < 3 m^3/s ∴ NO LOW OUTLIERS

ARIZONA DEPARTMENT OF TRANSPORTATION
HYDROLOGIC DESIGN DATA

Project No. ____________ TRACS No. ____________
Project Name ____________ Date ____________
Location/Station HASSAYAMPA RIVER NEAR WICKENBURG, AZ
Designer ____________ Checker ____________

B. THE ANNUAL PEAK DISCHARGE DATA SET CONTAINS:

- NO ZERO FLOW YEARS, AND
- NO LOW OUTLIERS, AND
- NO HIGH OUTLIERS, AND/OR
- HISTORIC DATA, AND/OR
- EXTRAORDINARY FLOODS.

PLOTTING POSITION EQUATION

$$P_e = \left(\frac{m-0.4}{K+0.2}\right)\left(\frac{K}{N}\right) \quad \text{FOR } m = 1 \ldots K$$

$$P_e = \frac{K}{N} + \left(\frac{N-K}{N}\right)\left(\frac{m-K-0.4}{N-K+0.2}\right)\left(\frac{N-K}{N_s-e}\right) \quad \text{FOR } m = K+1 \ldots N_G$$

WHERE EFFECTIVE RECORD LENGTH, $N = 100$
LENGTH OF SYSTEMATIC RECORD, $N_t = 38$
EFFECTIVE LENGTH OF SYSTEMATIC RECORD, $N_s = N_t = 38$
NUMBER OF HISTORIC FLOODS, $h = 3$
NUMBER OF EXTRAORDINARY FLOODS IN THE SYSTEMATIC RECORD, $e = 3$

$K = h + e = 6$

$N_G = N_s + h = 41$

$$P_e = \left(\frac{m-0.4}{6+0.2}\right)\left(\frac{6}{100}\right) = 0.0097\,(m-0.4) \quad \text{FOR } m = 1 \ldots 6$$

$$P_e = \frac{6}{100} + \left(\frac{100-6}{100}\right)\left(\frac{m-6-0.4}{100-6+0.2}\right)\left(\frac{100-6}{38-3}\right) = 0.06 + 0.0268\,(m-6.4)$$

FOR $m = 7 \ldots 41$

AT $m = 1$, $P_e = 0.0097(1-0.4) = 0.0058$

AT $m = 7$, $P_e = 0.06 + 0.0268(7-6.4) = 0.0761$

ARIZONA DEPARTMENT OF TRANSPORTATION
HYDROLOGIC DESIGN DATA

Project No. ______ TRACS No. ______
Project Name ______ Date ______
Location/Station HASSAYAMPA RIVER NEAR WICKENBURG, AZ
Designer ______ Checker ______

FIGURE 9-3
FLOOD FREQUENCY ANALYSIS
PLOTTING POSITION CALCULATION FORM

Page 1 of 2

Gage Station Name HASSAYAMPA RIVER NEAR WICKENBURG, AZ
Gage Station No. 09515500 **Drainage Area** 1080 **sq. km.**
Period of Systematic Record 1938, 1946-1982

Check if the data contains any of the following:

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

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

FLOOD PEAK DISCHARGE (m^3/s) (1)	RANK (2)	PLOTTING POSITION	
		P_e (3)	T_r (4)
1643	1	0.0058	172
767	2	0.0155	64
765	3	0.0252	40
722	4	0.0348	29
705	5	0.0445	22
623	6	0.0542	18
453	7	0.0761	13
317	8	0.1029	9.7
300	9	0.1297	7.7
283	10	0.1565	6.4
273	11	0.1833	5.4
257	12	0.2101	4.8
250	13	0.2369	4.2
159	14	0.2637	3.8
157	15	0.2905	3.4
157	16	0.3173	3.2
156	17	0.3441	2.9
145	18	0.3709	2.7
131	19	0.3977	2.5

ARIZONA DEPARTMENT OF TRANSPORTATION
HYDROLOGIC DESIGN DATA

Project No. ____________ TRACS No. ____________
Project Name ____________ Date ____________
Location/Station HASSAYAMPA RIVER NEAR WICKENBURG, AZ
Designer ____________ Checker ____________

FLOOD FREQUENCY ANALYSIS
PLOTTING POSITION CALCULATION FORM

Page 2 of 2

FLOOD PEAK DISCHARGE (m^3/s) (1)	RANK (2)	PLOTTING POSITION P_e (3)	PLOTTING POSITION T_r (4)
129	20	0.4245	2.4
91	21	0.4513	2.2
88	22	0.4781	2.1
83	23	0.5049	1.98
82	24	0.5317	1.88
74	25	0.5585	1.79
65	26	0.5853	1.71
61	27	0.6121	1.63
56	28	0.6389	1.56
49	29	0.6657	1.50
48	30	0.6925	1.44
45	31	0.7193	1.39
43	32	0.7461	1.34
35	33	0.7729	1.29
34	34	0.7997	1.25
33	35	0.8265	1.21
24	36	0.8533	1.17
23	37	0.8801	1.13
20	38	0.9069	1.10
16	39	0.9337	1.07
9	40	0.9605	1.04
4	41	0.9873	1.01

FIGURE 9-3 Continued

ARIZONA DEPARTMENT OF TRANSPORTATION
HYDROLOGIC DESIGN DATA

Project No. ________ TRACS No. ________

Project Name ________ Date ________

Location/Station HASSAYAMPA RIVER NEAR WICKENBURG, AZ

Designer ________ Checker ________

FIGURE 9-10
FLOOD FREQUENCY ANALYSIS
WORKSHEET FOR LOG-NORMAL CONFIDENCE LIMITS

Gage Station Name HASSAYAMPA RIVER NEAR WICKENBURG, AZ

Gage Station No. 09515500

Confidence Level (C.L.) = 90 %

$Q_{2\text{-yr}}$ = 84 m^3/s

$\alpha = \frac{100 - C.L.}{100}$ = 0.1

$Q_{100\text{-yr}}$ = 1300 m^3/s

$U_{1-\frac{\alpha}{2}}$ = 1.645

N_C = 23

$\overline{Y} = \log_{10}(Q_{2-yr}) = \log_{10}(84)$ = 1.9243

$S_{ln} = \frac{\log_{10} Q_{100-yr} - \log_{10} Q_{2-yr}}{2.327} = \frac{\log_{10}(1300) - \log_{10}(84)}{2.327}$ = 0.5112

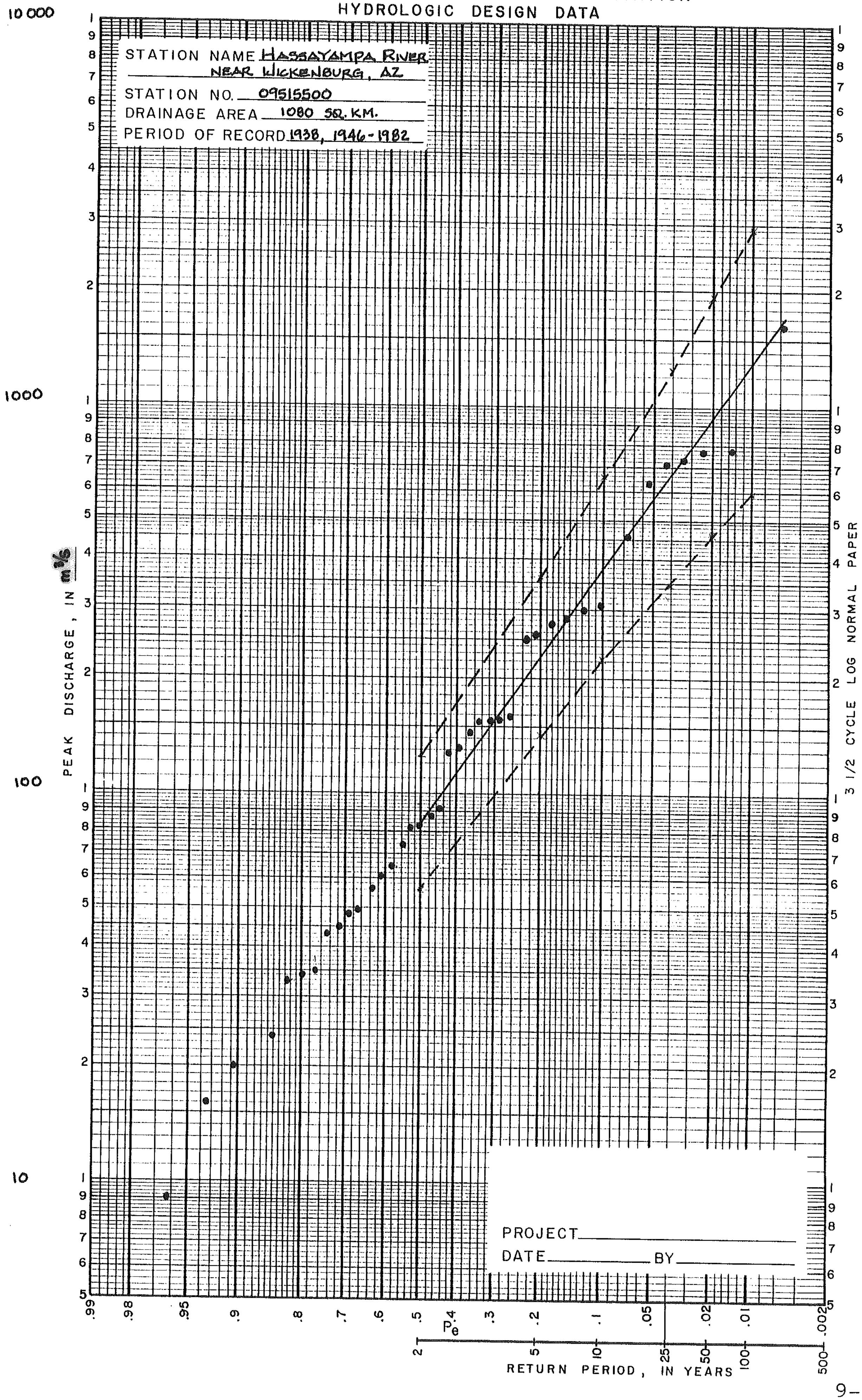
T Years (1)	$U_{1-\frac{1}{T}}$ (2)	Y_T (a) (3)	S_T (b) (4)	Limits, in m^3/s (c) Upper (5)	Lower (6)
2	0.0	1.9243	0.1066	125	56
5	0.842	2.3547	0.1241	362	141
10	1.282	2.5809	0.1439	657	221
25	1.751	2.8194	0.1696	1254	347
50	2.052	2.9733	0.1878	1915	462
100	2.327	3.1139	0.2052	2828	598

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

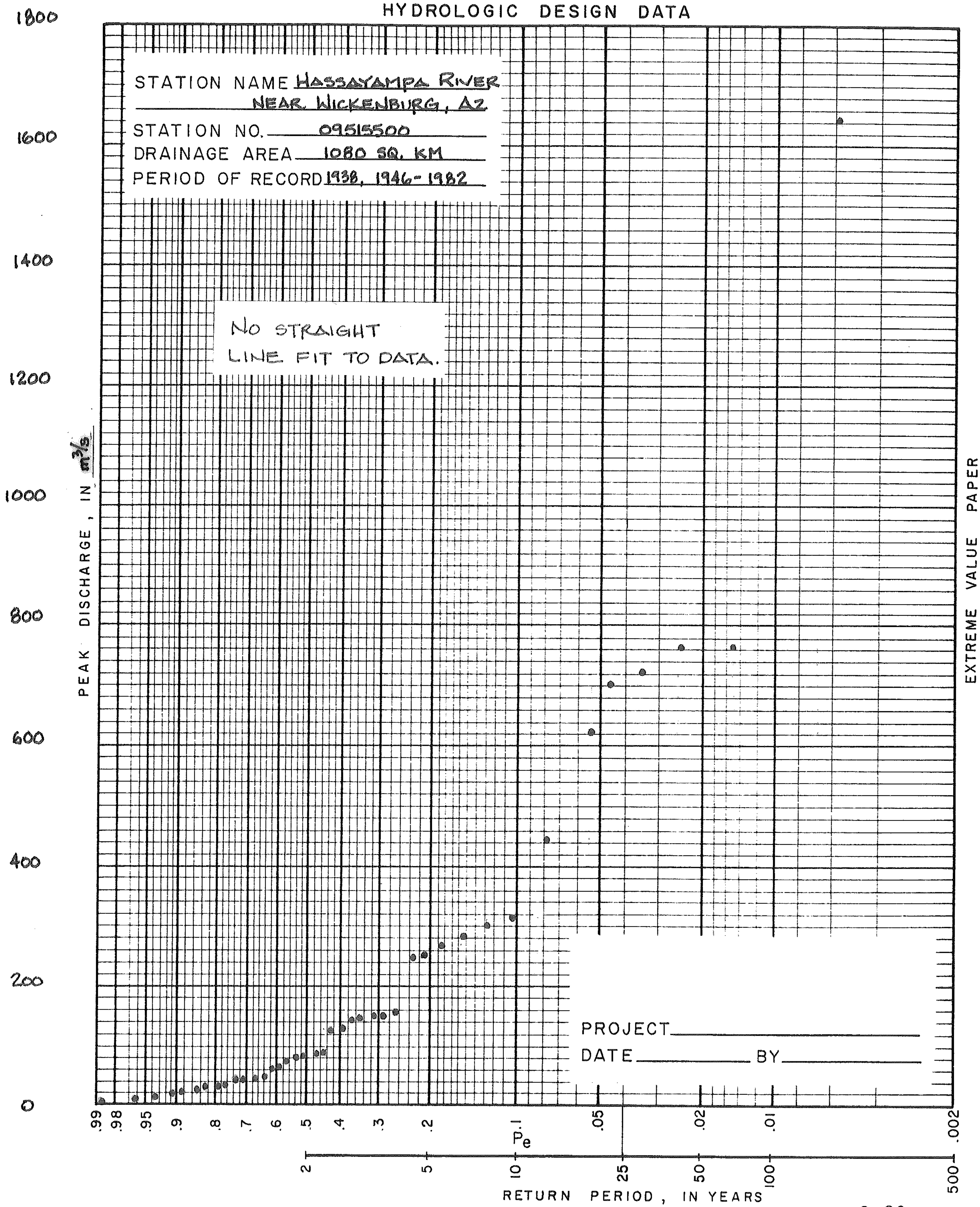
(c) $Q_L = 10^{(Y_T \pm U_{1-\frac{\alpha}{2}} S_T)}$

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

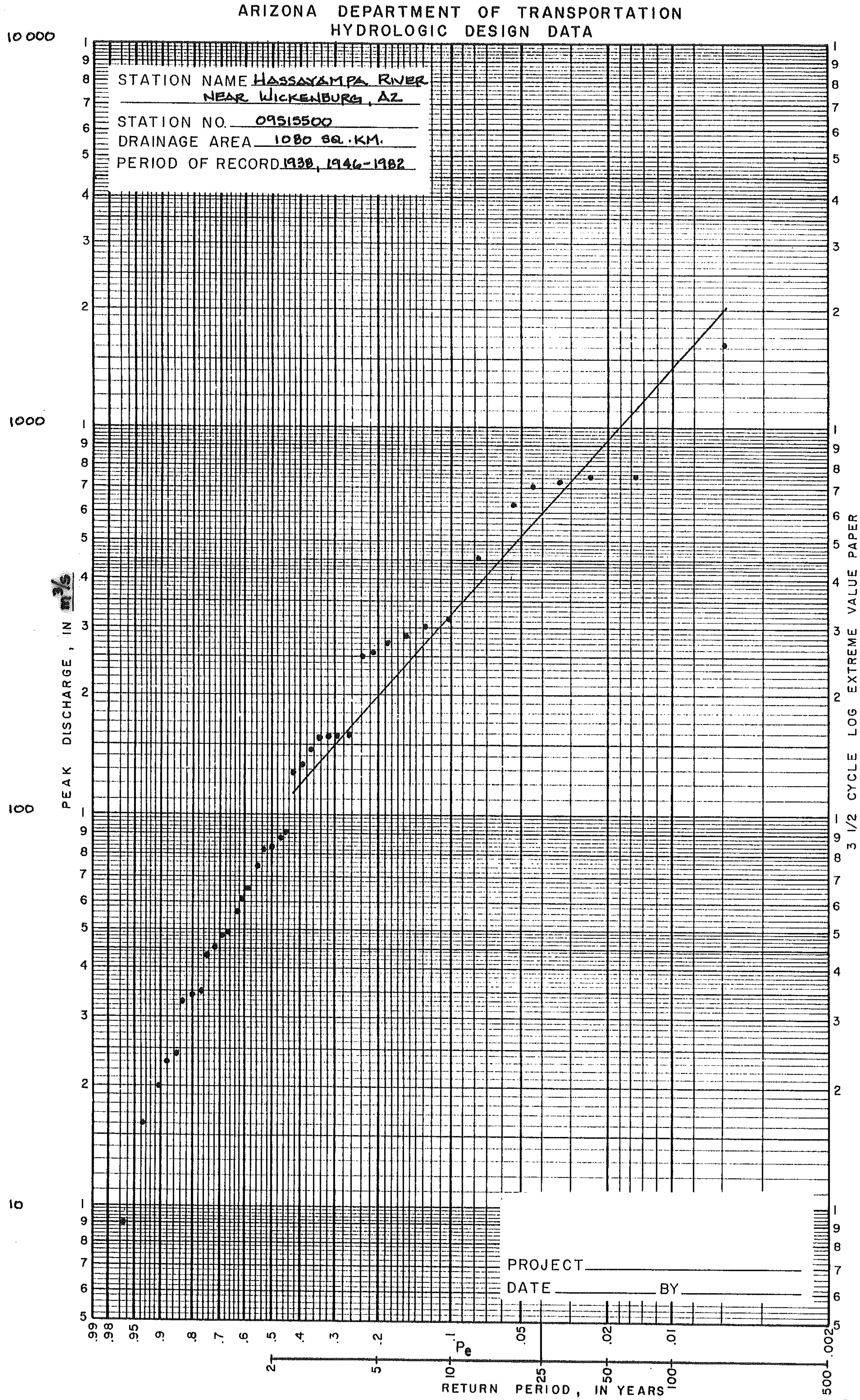
ARIZONA DEPARTMENT OF TRANSPORTATION
HYDROLOGIC DESIGN DATA



ARIZONA DEPARTMENT OF TRANSPORTATION
HYDROLOGIC DESIGN DATA



9-80


ARIZONA DEPARTMENT OF TRANSPORTATION
HYDROLOGIC DESIGN DATA
STATION NAME Hassayampa River Near Wickenburg, AZ
STATION NO. 09515500
DRAINAGE AREA 1080 SQ. KM.
PERIOD OF RECORD 1938, 1946-1982
PEAK DISCHARGE, IN m3/s
3 1/2 CYCLE LOG EXTREME VALUE PAPER
PROJECT
DATE
BY
Pe
RETURN PERIOD, IN YEARS

FLOOD FREQUENCY ANALYSIS EXAMPLE No. 9-4

Station Name - Santa Cruz River near Lochiel, Arizona
Station Number - 09480000
Drainage Area - 213 square kilometers
Period of Record - 1949 through 1989

Flood Data

A continuous, 41 year systematic record is available, and the entire record was used in the analysis. All annual floods are considered to be caused by rainfall. There are no historic data. There are no zero flow years. The high and low floods of record are 340 m^3/s (1978 and 1984) and 0.2 m^3/s (1962), respectively. Two floods of 340 m^3/s in 1978 and 1984, are indicated in the records of the U.S. Geological Survey as being the largest since 1926. The record is considered stationary.

Flood Frequency Analysis

The high outlier limit is calculated at 990 m^3/s, and no high outliers are identified. The low outlier limit is calculated at 1.4 m^3/s, and a low outlier is identified for 1962 (0.2 m^3/s). Extraordinary floods are identified for 1978 and 1984 (340 m^3/s each) because these floods, from the systematic record, are known to be larger than any flood since 1926, prior to the start of the systematic record.

The data set contains a low outlier and extraordinary floods. The effective record length is the period 1926 through 1989 ($N = 64$). The length of the systematic record is the period 1949 through 1989 ($N_t = 41$). There is one low outlier ($Z = 1$), and the effective length of the systematic record is 40 years ($N_s = N_t - Z = 41 - 1 = 40$). There are no historic data ($h = 0$), but there are two extraordinary floods ($e = 2$); and, ($k = h + e = 0 + 2 = 2$). There are 40 systematic plus historic floods ($N_G = N_s + h = 40 + 0 = 40$). These parameters are used in calculating the plotting positions.

The annual flood peak discharges are plotted on the three probability papers at their respective plotting positions. The extreme value (EV) graph does not show a linear relation for the two largest floods. The log-extreme value (LEV) graph indicates a concave down trend to the data. The log-normal (LN) graph indicates a reasonably good linear fit for virtually all of the data. The two largest floods, being at the same magnitude, makes it impossible for those two points to lie in a straight line with the other data. The LN graph is clearly the best linear fit to the data, and it represents the probability distribution of floods with return periods that are equal to or longer than 2 years.

Confidence limits are set about the LN best fit line. The 40 largest floods (N_c = 40) are used to establish the best fit line. The estimated 100-yr flood peak discharge is 340 m^3/s with 90 percent upper and lower confidence limits of 540 m^3/s and 212 m^3/s, respectively.

Discussion

This example illustrates a flood frequency analysis for a data set containing a low outlier and extraordinary floods. The effective length of record was extended beyond the length of the systematic record. The LN graph is selected as the best straight line fit to the data. This is an example of a clear choice of the best graph paper to select. The data are nearly linear with little scatter about the line. The range of the confidence limits is tight because all 40 data points are used to establish the best fit line.

GILA RIVER BASIN

09480000 SANTA CRUZ RIVER NEAR LOCHIEL, AZ

LOCATION.--Lat 31°21'19", long 110°35'20", in SW; sec.11, T.24 S., R.17 E. (unsurveyed), Santa Cruz County, Hydrologic Unit 15050301, on southern border of Spanish land grant of San Rafael, near left bank on downstream side of pier of bridge on county road, 1.7 mi upstream from international boundary and 2.5 mi northeast of Lochiel.

DRAINAGE AREA.--82.2 mi^2.

REMARKS.-Small diversions for irrigation of 200 acres above station, mostly by pumping from ground water.

ANNUAL PEAK DISCHARGE

WATER YEAR	DATE	ANNUAL PEAK DISCHARGE (FT^3/S)	WATER YEAR	DATE	ANNUAL PEAK DISCHARGE (FT^3/S)
1949	09-13-49	1,650	1970	08-03-70	880
1950	07-30-50	4,520	1971	08-10-71	2,830
1951	08-02-51	2,560	1972	07-16-72	2,070
1952	08-16-52	550	1973	06-30-73	1,490
1953	07-14-53	3,320	1974	08-04-74	1,730
1954	07-22-54	1,570	1975	07-22-75	3,330
1955	08-06-55	4,300	1976	07-22-76	3,540
1956	07-17-56	1,360	1977	09-05-77	1,130
1957	08-09-57	688	1978	10-09-77	[1]12,000
1958	08-07-58	380	1979	01-25-79	1,060
1959	08-14-59	243	1980	06-30-80	406
1960	07-30-60	625	1981	07-15-81	1,110
1961	08-08-61	1,120	1982	08-11-82	2,640
1962	07-29-62	7.6	1983	03-04-83	1,120
1963	08-25-63	2,390	1984	08-15-84	12,000
1964	09-09-64	2,330	1985	07-19-85	850
1965	09-12-65	4,810	1986	08-29-86	4,210
1966	08-18-66	1,780	1987	08-10-87	291
1967	08-03-67	1,870	1988	08-23-88	804
1968	12-20-67	986	1989	08-04-89	871
1969	08-05-69	484			

[1]Highest since 1926.

BASIN CHARACTERISTICS

MAIN CHANNEL SLOPE (FT/MI)	STREAM LENGTH (MI)	MEAN BASIN ELEVATION (FT)	FORESTED AREA (PERCENT)	SOIL INDEX	MEAN ANNUAL PRECIPITATION (IN)	RAINFALL INTENSITY, 24-HOUR	
						2-YEAR (IN)	50-YEAR (IN)
42.2	12.0	5,150	31.0	2.3	18.2	1.9	4.3

09480000 SANTA CRUZ RIVER NEAR LOCHIEL, AZ--Continued

MEAN MONTHLY AND ANNUAL DISCHARGES 1950-89

MONTH	MAXIMUM (FT^3/S)	MINIMUM (FT^3/S)	MEAN (FT^3/S)	STANDARD DEVIATION (FT^3/S)	COEFFICIENT OF VARIATION	PERCENT OF ANNUAL RUNOFF
OCTOBER	77	0.00	5.2	17	3.2	11.1
NOVEMBER	6.8	0.00	1.1	1.5	1.4	2.3
DECEMBER	18	0.00	1.8	3.7	2.0	3.9
JANUARY	47	0.02	2.7	8.3	3.1	5.7
FEBRUARY	18	0.03	1.7	3.4	2.0	3.6
MARCH	34	0.01	1.9	5.6	2.9	4.0
APRIL	5.2	0.00	0.74	1.2	1.6	1.6
MAY	2.8	0.00	0.39	0.67	1.7	0.8
JUNE	2.8	0.00	0.30	0.65	2.2	0.6
JULY	69	0.03	8.4	16	1.8	17.8
AUGUST	187	0.00	17	38	2.2	37.0
SEPTEMBER	44	0.00	5.3	9.5	1.8	11.4
ANNUAL	29	0.31	3.9	5.3	1.3	100

MAGNITUDE AND PROBABILITY OF ANNUAL LOW FLOW BASED ON PERIOD OF RECORD 1950-89

PERIOD (CONSECUTIVE DAYS)	DISCHARGE, IN FT^3/S, FOR INDICATED RECURRENCE INTERVAL, IN YEARS, AND NON-EXCEEDANCE PROBABILITY, IN PERCENT					
	2 50%	5 20%	10 10%	20 5%	50 2%	100† 1%
1	0.00	0.00	0.00	0.00	0.00	0.00
3	0.00	0.00	0.00	0.00	0.00	0.00
7	0.00	0.00	0.00	0.00	0.00	0.00
14	0.00	0.00	0.00	0.00	0.00	0.00
30	0.00	0.00	0.00	0.00	0.00	0.00
60	0.00	0.00	0.00	0.00	0.00	0.06
90	0.00	0.00	0.00	0.00	0.01	0.10
120	0.00	0.00	0.00	0.05	0.12	0.41
183	0.74	0.21	0.10	0.05	0.02	0.01

MAGNITUDE AND PROBABILITY OF INSTANTANEOUS PEAK FLOW BASED ON PERIOD OF RECORD 1949-89

DISCHARGE, IN FT^3/S, FOR INDICATED RECURRENCE INTERVAL IN YEARS, AND EXCEEDANCE PROBABILITY, IN PERCENT					
2 50%	5 20%	10 10%	25 4%	50 2%	100† 1%
1,460	2,950	4,330	6,590	8,700	11,200

WEIGHTED SKEW (LOGS)= 0.20
MEAN (LOGS)= 3.17
STANDARD DEV. (LOGS)= 0.35

MAGNITUDE AND PROBABILITY OF ANNUAL HIGH FLOW BASED ON PERIOD OF RECORD 1950-89

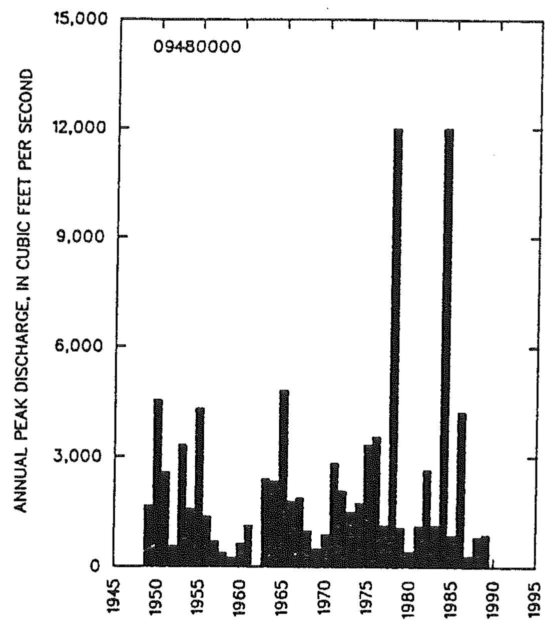
PERIOD (CONSECUTIVE DAYS)	DISCHARGE, IN FT^3/S, FOR INDICATED RECURRENCE INTERVAL, IN YEARS, AND EXCEEDANCE PROBABILITY, IN PERCENT					
	2 50%	5 20%	10 10%	25 4%	50 2%	100† 1%
1	170	439	661	963	1,190	1,410
3	75	211	343	553	735	937
7	38	114	196	341	482	651
15	22	66	115	202	290	398
30	14	41	72	130	190	267
60	8.7	25	43	77	114	161
90	6.3	17	30	54	80	114

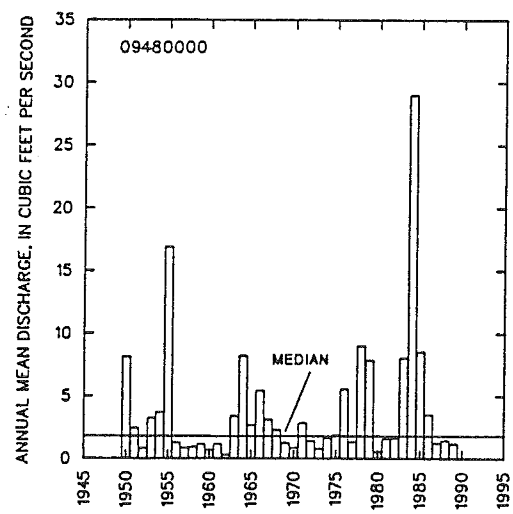
DURATION TABLE OF DAILY MEAN FLOW FOR PERIOD OF RECORD 1950-89

DISCHARGE, IN FT^3/S, WHICH WAS EQUALED OR EXCEEDED FOR INDICATED PERCENT OF TIME																
1%	5%	10%	15%	20%	30%	40%	50%	60%	70%	80%	90%	95%	98%	99%	99.5%	99.9%
59	9.1	4.3	2.5	1.6	0.95	0.64	0.45	0.30	0.20	0.10	0.00	0.00	0.00	0.00	0.00	0.00

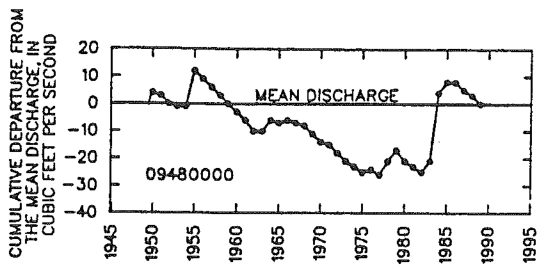
† Reliability of values in column is uncertain, and potential errors are large.

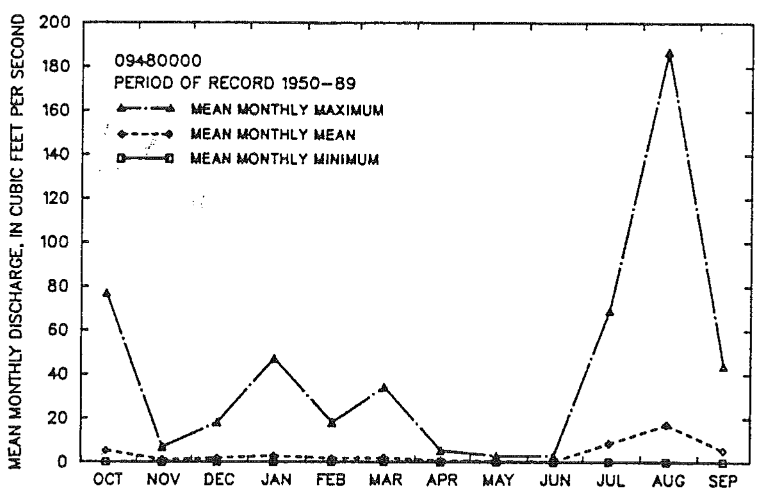
09480000 SANTA CRUZ RIVER NEAR LOCHIEL, AZ--CONTINUED





09480000 SANTA CRUZ RIVER NEAR LOCHIEL, AZ--Continued





ARIZONA DEPARTMENT OF TRANSPORTATION
HYDROLOGIC DESIGN DATA

Project No. ______ TRACS No. ______
Project Name ______ Date ______
Location/Station SANTA CRUZ RIVER LOCHIEL, AZ
Designer ______ Checker ______

FIGURE 9-2
FLOOD FREQUENCY ANALYSIS
DATA COMPILATION FORM

Gage Station Name SANTA CRUZ RIVER NEAR LOCHIEL, AZ
Gage Station No. 09480000 **Drainage Area** 213 **sq. km.**
Period of Systematic Record 1949-1989

WATER YEAR (1)	ANNUAL PEAK DISCHARGE (cfs) (2)	ANNUAL PEAK DISCHARGE (m^3/s) (3)	DATE (4)	FLOOD TYPE [a] (5)	COMMENTS (6)
1949	1650	47	13 SEPT 49	R	
50	4520	128	30 JULY 50	R	
51	2560	72	2 AUG 51	R	
52	550	16	16 AUG 52	R	
53	3320	94	14 JULY 53	R	
54	1570	44	22 JULY 54	R	
55	4300	122	6 AUG 55	R	
56	1360	39	17 JULY 56	R	
57	688	19	9 AUG 57	R	
58	380	11	7 AUG 58	R	
59	243	7	14 AUG 59	R	
60	625	18	30 JULY 60	R	
61	1120	32	8 JULY 61	R	
62	8	0.2	29 JULY 62	R	
63	2390	68	25 AUG 63	R	
64	2330	66	9 SEPT 64	R	
65	4810	136	12 SEPT 65	R	
66	1780	50	18 AUG 66	R	
67	1870	53	3 AUG 67	R	
68	986	28	20 DEC 67	R	
69	484	14	5 AUG 69	R	
70	880	25	3 AUG 70	R	

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

Note: (3) = 0.0283 * (2) for cfs to m^3/s conversion

ARIZONA DEPARTMENT OF TRANSPORTATION
HYDROLOGIC DESIGN DATA

Project No. ______________________ TRACS No. ______________________
Project Name ______________________ Date ______________________
Location/Station SANTA CRUZ RIVER NEAR LOCHIEL, AZ
Designer ______________________ Checker ______________________

FLOOD FREQUENCY ANALYSIS
DATA COMPILATION FORM

Page 2 of 2

WATER YEAR (1)	ANNUAL PEAK DISCHARGE (cfs) (2)	ANNUAL PEAK DISCHARGE (m^3/s) (3)	DATE (4)	FLOOD TYPE [a] (5)	COMMENTS (6)
1971	2830	80	10 AUG 71	R	
72	2070	59	16 JULY 72	R	
73	1490	42	30 JUNE 73	R	
74	1730	49	4 AUG 74	R	
75	3330	94	22 JULY 75	R	
76	3540	100	22 JULY 76	R	
77	1130	32	5 SEPT 77	R	
78	12000	340	9 OCT 77	R	EXTRAORDINARY
79	1060	30	25 JAN 79	R	
80	406	11	30 JUN 80	R	
81	1110	31	15 JULY 81	R	
82	2640	75	11 AUG 82	R	
83	1120	32	4 MAR 83	R	
84	12000	340	15 AUG 84	R	EXTRAORDINARY
85	850	24	17 JULY 85	R	
86	4210	119	29 AUG 86	R	
87	291	8	10 AUG 87	R	
88	804	23	23 AUG 88	R	
89	871	25	4 AUG 89	R	

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

Note: (3) = 0.0283 * (2) for cfs to m^3/s conversion

FIGURE 9-2 Continued

ARIZONA DEPARTMENT OF TRANSPORTATION
HYDROLOGIC DESIGN DATA

Project No. ______ TRACS No. ______
Project Name ______ Date ______
Location/Station SANTA CRUZ RIVER NEAR LOCHIEL, AZ
Designer ______ Checker ______

A. TEST FOR HIGH & LOW OUTLIERS

$\overline{\text{LOG } Q} = 1.5766 \qquad N_G = 41$

$\text{LOG } S = 0.5276 \qquad K_N = 2.692$

- HIGH OUTLIERS

$\text{LOG } Q_H = \overline{\text{LOG } Q} + K_N \text{ LOG } S$

$= 1.5766 + 2.692\,(0.5276)$

$= 2.9969$

$\underline{Q_H = 993 \text{ m}^3/\text{s}}$

THERE ARE NO Q's > 993 m³/s ∴ NO HIGH OUTLIERS

- LOW OUTLIERS

$\text{LOG } Q_L = \overline{\text{LOG } Q} - K_N \text{ LOG } S$

$= 1.5766 - 2.692\,(0.5276)$

$= 0.1563$

$\underline{Q_L = 1.4 \text{ m}^3/\text{s}}$

* 1962 ⇒ 0.2 m³/s ONE LOW OUTLIER ⇒ ZERO FLOW YEAR

ARIZONA DEPARTMENT OF TRANSPORTATION
HYDROLOGIC DESIGN DATA

Project No. ______ TRACS No. ______
Project Name ______ Date ______
Location/Station SANTA CRUZ RIVER NEAR LOCHIEL, AZ
Designer ______ Checker ______

B. THE ANNUAL FLOOD PEAK DISCHARGE DATA SET CONTAINS:

- ZERO FLOW YEARS, AND/OR
- LOW OUTLIERS, AND
- NO HIGH OUTLIERS, AND
- NO HISTORIC DATA, AND
- EXTRAORDINARY FLOODS.

PLOTTING POSITION EQUATION

$$P_e = \left(\frac{N_t - Z}{N_t}\right)\left(\frac{m-0.4}{K+0.2}\right)\left(\frac{K}{N}\right) \quad \text{FOR } m = 1 \ldots K$$

$$P_e = \left(\frac{N_t - Z}{N_t}\right)\left(\frac{K}{N} + \left(\frac{N-K}{N}\right)\right)\left(\frac{m-K-0.4}{N-K+0.2}\right)\left(\frac{N-K}{N_s-e}\right) \quad \text{FOR } m = K+1 \ldots N_G$$

WHERE EFFECTIVE RECORD LENGTH, $N = 64$

LENGTH OF SYSTEMATIC RECORD, $N_t = 41$

NUMBER OF ZERO FLOW YEARS, AND/OR NUMBER OF LOW OUTLIERS, $Z = 1$

EFFECTIVE LENGTH OF SYSTEMATIC RECORD, $N_s = N_t - Z = 40$

NUMBER OF HISTORIC FLOODS, $h = 0$

NUMBER OF EXTRAORDINARY FLOODS, $e = 2$

$K = h + e = 2$

$N_G = N_s + h = 40$

$$P_e = \left(\frac{41-1}{41}\right)\left(\frac{m-0.4}{2+0.2}\right)\left(\frac{2}{64}\right) = 0.0139(m-0.4) \quad \text{FOR } m = 1 \ldots 2$$

$$P_e = \left(\frac{41-1}{41}\right)\left(\frac{2}{64} + \frac{64-2}{64}\right)\left(\frac{m-2-0.4}{64-2+0.2}\right)\left(\frac{64-2}{40-2}\right) = 0.0305 + 0.0248(m-2.4)$$

FOR $m = 3 \ldots 40$

AT $m = 1$; $P_e = 0.0139(1-0.4) = 0.0083$

AT $m = 3$; $P_e = 0.0305 + 0.0248(3-2.4) = 0.0454$

ARIZONA DEPARTMENT OF TRANSPORTATION
HYDROLOGIC DESIGN DATA

Project No. ______ TRACS No. ______
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FIGURE 9-3
FLOOD FREQUENCY ANALYSIS
PLOTTING POSITION CALCULATION FORM

Page 1 of 2

Gage Station Name SANTA CRUZ RIVER NEAR LOCHIEL, AZ
Gage Station No. 09480000 **Drainage Area** 213 **sq. km.**
Period of Systematic Record 1949-1989

Check if the data contains any of the following:

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

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

FLOOD PEAK DISCHARGE (m^3/s) (1)	RANK (2)	PLOTTING POSITION P_e (3)	T_r (4)
340	1	0.0083	120
340	2	0.0222	45
136	3	0.0454	22
128	4	0.0702	14
122	5	0.0949	10.5
119	6	0.1197	8.4
100	7	0.1445	7.0
94	8	0.1693	5.9
94	9	0.1941	5.1
80	10	0.2189	4.6
75	11	0.2437	4.1
72	12	0.2685	3.7
68	13	0.2935	3.4
66	14	0.3181	3.1
59	15	0.3429	2.9
53	16	0.3677	2.71
50	17	0.3924	2.54
49	18	0.4172	2.40
47	19	0.4420	2.26

ARIZONA DEPARTMENT OF TRANSPORTATION
HYDROLOGIC DESIGN DATA

Project No. ______________________ TRACS No. ______________________
Project Name ______________________ Date ______________________
Location/Station SANTA CRUZ RIVER NEAR LOCHIEL, AZ
Designer ______________________ Checker ______________________

FLOOD FREQUENCY ANALYSIS
PLOTTING POSITION CALCULATION FORM Page 2 of 2

FLOOD PEAK DISCHARGE (m^3/s) (1)	RANK (2)	PLOTTING POSITION	
		P_e (3)	T_r (4)
44	20	0.4668	2.14
42	21	0.4916	2.03
39	22	0.5164	1.93
32	23	0.5412	1.85
32	24	0.5660	1.77
32	25	0.5908	1.69
31	26	0.6156	1.63
30	27	0.6404	1.56
28	28	0.6652	1.50
25	29	0.6899	1.45
25	30	0.7147	1.40
24	31	0.7395	1.35
23	32	0.7643	1.31
19	33	0.7891	1.27
18	34	0.8139	1.23
16	35	0.8387	1.19
14	36	0.8635	1.16
11	37	0.8883	1.13
11	38	0.9131	1.10
8	39	0.9379	1.07
7	40	0.9627	1.01

FIGURE 9-3 Continued

Project No. ______ TRACS No. ______
Project Name ______ Date ______
Location/Station SANTA CRUZ RIVER NEAR LOCHIEL, AZ
Designer ______ Checker ______

FIGURE 9-10
FLOOD FREQUENCY ANALYSIS
WORK SHEET FOR LOG-NORMAL CONFIDENCE LIMITS

Gage Station Name SANTA CRUZ RIVER NEAR LOCHIEL, AZ
Gage Station No. 09480000

Confidence Level (C.L.) = 90 %

$Q_{2\text{-yr}}$ = 38 m^3/s $\alpha = \frac{100 - C.L.}{100}$ = 0.1

$Q_{100\text{-yr}}$ = 340 m^3/s $U_{1-\frac{\alpha}{2}}$ = 1.645

N_C = 40

$\overline{Y} = \log_{10}(Q_{2-yr}) = \log_{10}(38)$ = 1.5798

$$S_{ln} = \frac{\log_{10} Q_{100-yr} - \log_{10} Q_{2-yr}}{2.327} = \frac{\log_{10}(340) - \log_{10}(38)}{2.327} = 0.4090$$

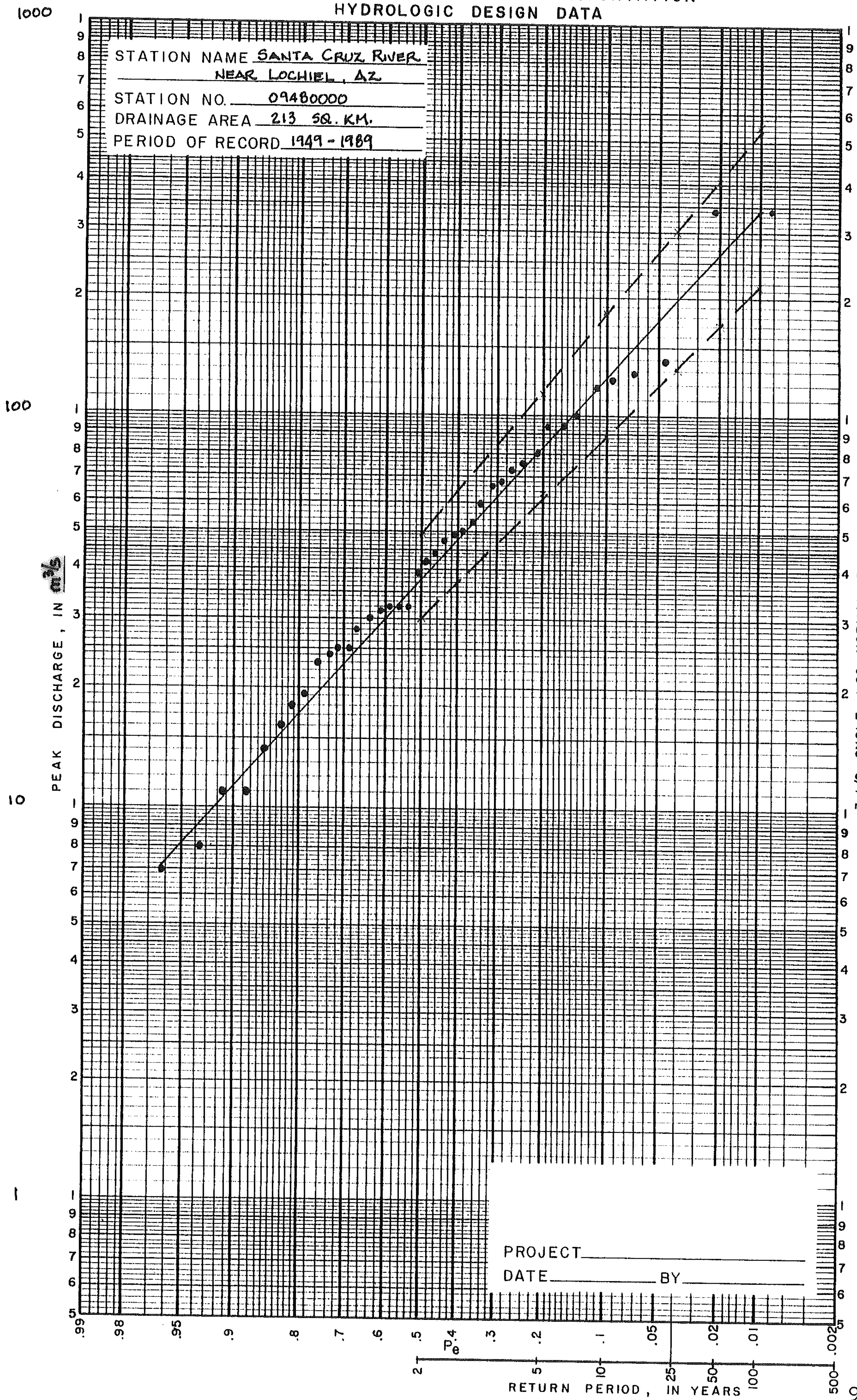
T Years (1)	$U_{1-\frac{1}{T}}$ (2)	Y_T (a) (3)	S_T (b) (4)	Limits, in m^3/s (c) Upper (5)	Lower (6)
2	0.0	1.5798	0.0647	49	30
5	0.842	1.9242	0.0753	112	63
10	1.282	2.1041	0.0936	181	89
25	1.751	2.2960	0.1104	300	130
50	2.052	2.4191	0.1140	404	170
100	2.327	2.5315	0.1245	545	212

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

ARIZONA DEPARTMENT OF TRANSPORTATION
HYDROLOGIC DESIGN DATA



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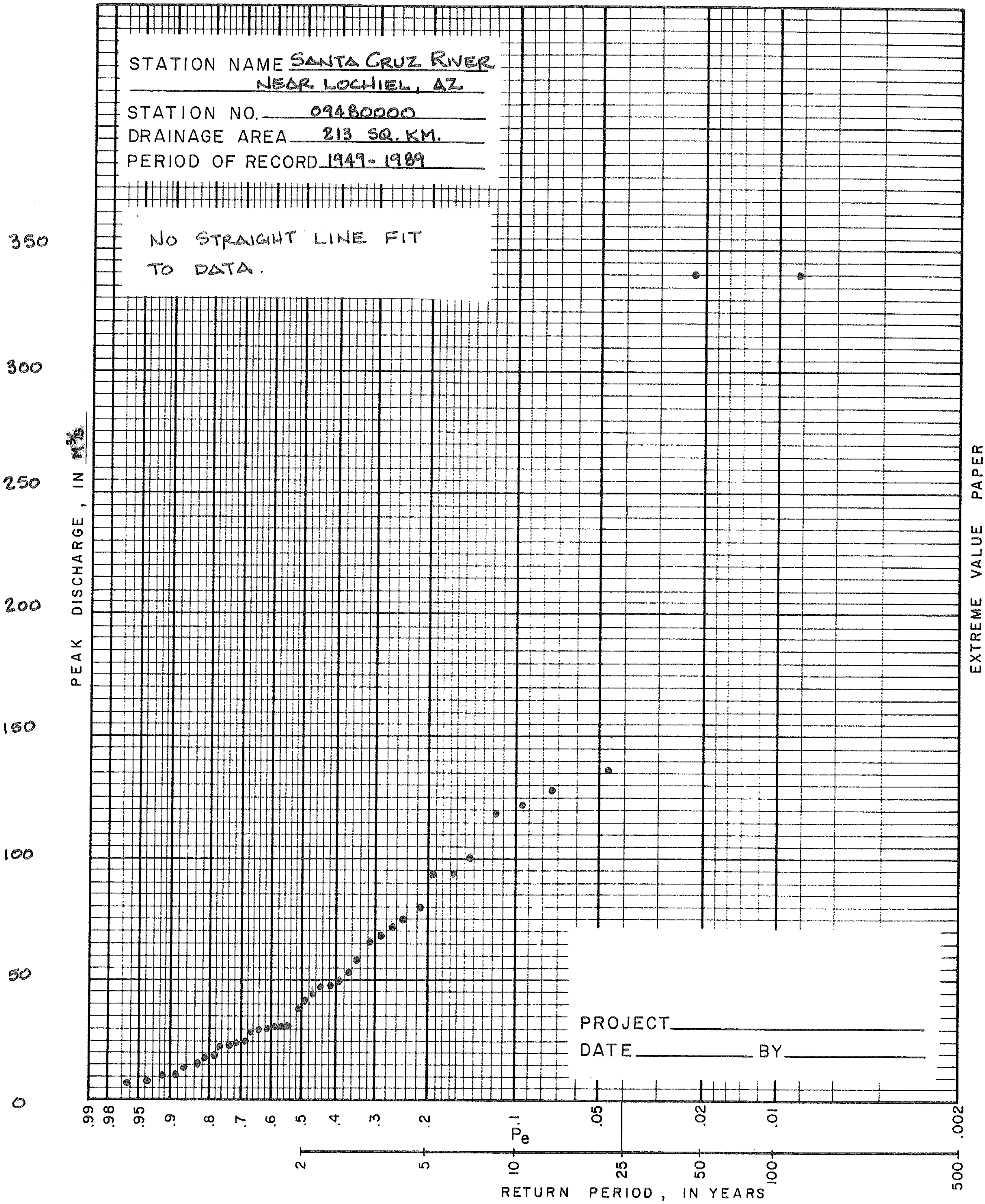
HYDROLOGIC DESIGN DATA



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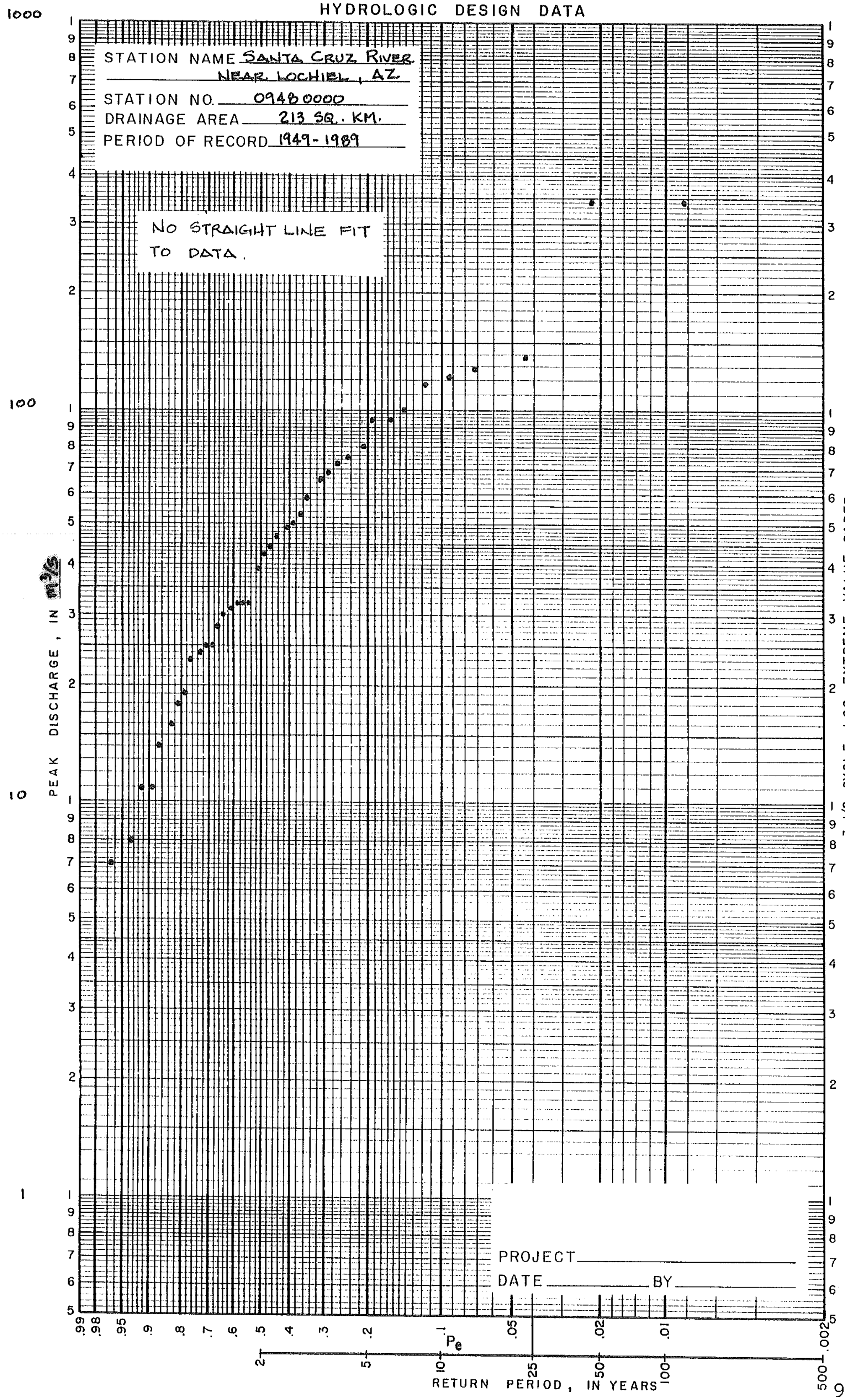
HYDROLOGIC DESIGN DATA



CHAPTER 10
INDIRECT METHODS FOR DISCHARGE VERIFICATION

10.1 INTRODUCTION

10.1.1 General Discussion

The estimation of peak discharges by analytic methods (the Rational Method or by rainfall-runoff modeling (HEC-1 program)) is based on various assumptions, and in the case of HEC-1 modeling, requires the correct input of numerous model input. Therefore, the resulting peak discharges that are computed by analytic methods should always be verified, to the extent possible, to guard against erroneous design discharges that can result from questionable assumptions and/or faulty model input.

Since the majority of discharge estimates are made for ungaged watersheds, usually only indirect methods can be used to check the discharge estimates obtained from either the Rational Method or rainfall-runoff modeling. When the watershed is gaged, or is near a gaging station, a flood frequency analysis can be performed and the results of that analysis can be used for design or used to check the results from analytic methods. The results of flood frequency analyses, because of variability of flooding in both the time and space regime, and because of uncertainties in the data and the analytic procedures, should also be checked by indirect methods.

True verification of design discharges cannot be made by any of the methods (analytic methods, flood frequency analyses, or indirect methods) because for none of these methods is there "absolute assurance" that the discharges that are obtained are the "true" representations of the flood discharge for a given frequency of flooding. However, the results of the various methods, when compared against each other and when qualitatively evaluated, can provide a basis for either acceptance or rejection of specific estimates of design discharges for watersheds in Arizona.

In this chapter, three indirect methods are presented for "verifying" flood discharges that are obtained by either analytic methods or by flood frequency analyses. Results by either analytic methods or flood frequency analysis should always be compared and evaluated by indirect methods. There may be cases, for certain watersheds, where the flood discharges by all three methods (analytic, flood frequency analysis, and indirect) can be obtained and compared prior to making a selection of design discharge.

10.2 PROCEDURE

10.2.1 General Considerations

Three procedures are provided for obtaining indirect estimates of peak discharges for watersheds in Arizona:

1. A graph of numerous unit peak discharge versus drainage area curves,

2. Five graphs of estimated 100-year discharges and maximum recorded discharges versus drainage area for gaged watersheds in Arizona, and

3. Regression equations and data graphs for seven flood regions in Arizona.

In general, all three procedures should be used when verifying the results of analytic methods and/or flood frequency analyses.

10.2.2 Indirect Method No. 1 - Unit Peak Discharge Curves

Figure 10-1 presents 10 unit peak discharge relations and envelope curves. A brief description of each of those curves follows:

A - An envelope curve, based on a compilation of unusual flood discharges in the United States and abroad (data prior to 1941), by Craeger and others (1945).

B - An envelope curve of extreme floods in Arizona and the Rocky Mountain region developed by Matthai and published by Roeske (1978).

C - A 100-year peak discharge relation developed for Arizona from an analysis by Malvick (1980).

D - An envelope curve of peak streamflow data developed for Arizona by Malvick (1980).

E - An envelope curve of peak streamflow data for the Little Colorado River basin in Northern Arizona developed by Crippen (1982).

F - An envelope curve of peak streamflow data for Central and Southern Arizona developed by Crippen (1982).

G - A 100-year peak discharge relation for Southeastern Arizona developed by Eychaner (1984).

H - A 100-year peak discharge envelope curve for Southeastern Arizona developed by Boughton and others (1987).

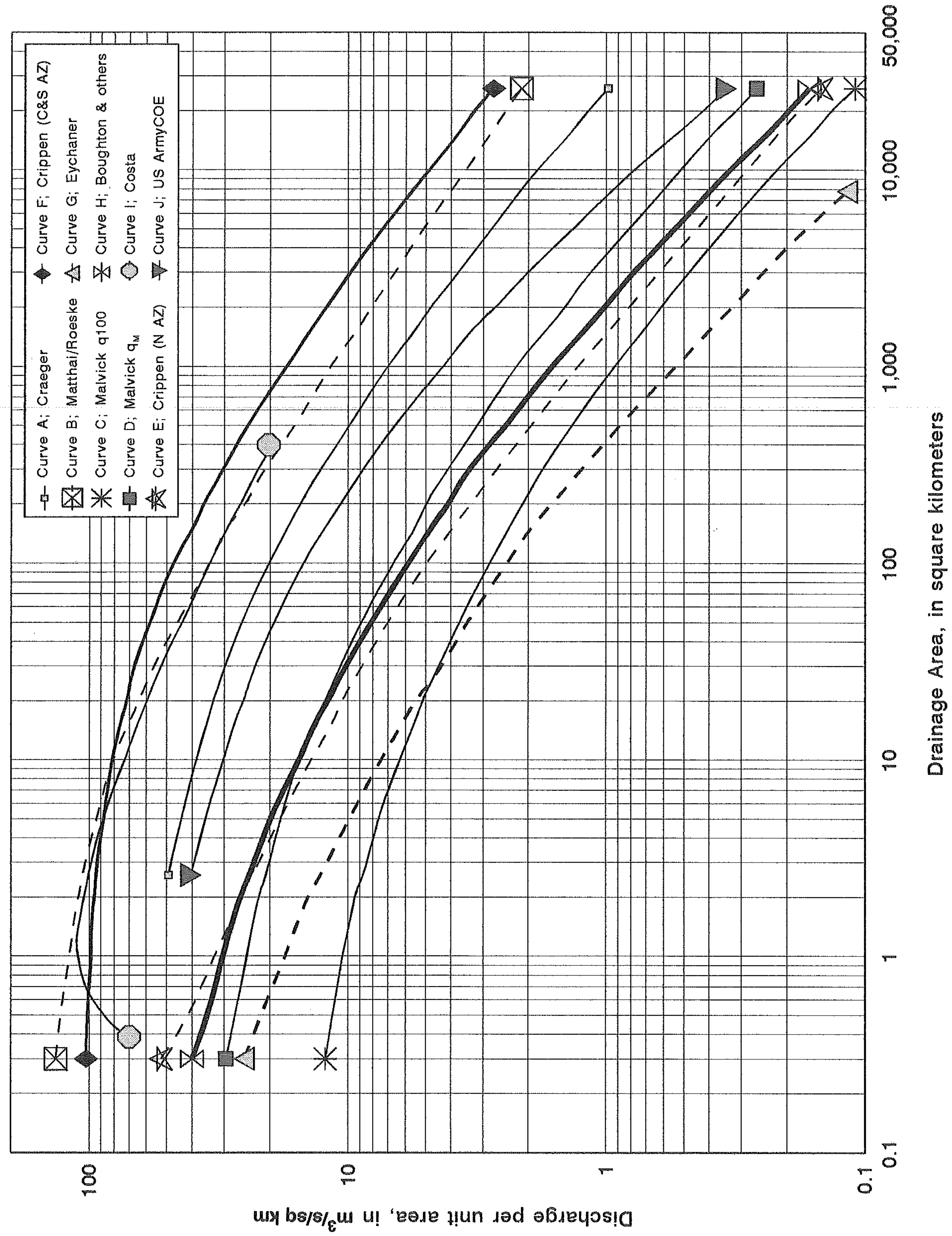
I - An envelope curve of the largest floods in the semi-arid Western United States developed by Costa (1987).

J - An envelope curve of peak discharges for Arizona, Nevada and New Mexico developed by the U.S. Army Corps of Engineers (1988).

When using **Figure 10-1**, it must be noted that the curves represent different data sets for different hydrologic regions. Seven of the curves represent envelopes of <u>maximum observed flood discharges</u> (Curves A, B, D, E, F, I and J), one is a <u>100-year discharge envelope</u> (Curve H), and two are <u>100-year discharge relations</u> (Curves C and G). The curves of most interest in evaluating 100-year peak discharges for Arizona are C, G, and H.

FIGURE 10-1

PEAK DISCHARGE RELATIONS AND ENVELOPE CURVES



10.2.3 Indirect Method No. 2 - USGS Data for Arizona

The U.S. Geological Survey (USGS) provides streamflow and statistical data for 138 continuous-record streamflow-gaging stations and 176 partial-record gaging stations in Arizona (Garrett and Gellenbeck, 1991). The streamflow data were analyzed by the USGS by Log-Pearson Type 3 (LP3) analyses and flood magnitude-frequency statistics are provided in the report along with the maximum recorded discharge for each of the stations. **Figure 10-2** is a plot of the 100-year peak discharge (from LP3 analyses) and the maximum recorded discharge for each gaging station versus drainage area (for stations with drainage areas smaller than 5,000 square kilometers (2,000 square miles). Lines were fit to the two data sets by least-squares of the log-transformed data. The equation for the 100-year peak discharge (Q_{100}) line is:

$$Q_{100} = 14.4A^{.54} \tag{10-1}$$

and, the equation for the maximum recorded discharge (Q_M) line is:

$$Q_M = 5.81\,A^{.62} \tag{10-2}$$

where Q_{100} and Q_M are in m^3/s, and A is in square kilometers in both equations.

The discharge relations for Curves C-Roeske, G-Eychaner, and H-Boughton (converted to discharge rather than unit discharge) are also shown in **Figure 10-2**.

As an aid to using **Figure 10-2**, that figure is reproduced with larger drainage area scales in **Figures 10-3** through **10-6**. Those larger scale plots of the data also show 75 percent tolerance limit lines about the 100-year discharge line (Equation 10-1). The tolerance limits are a statistical measure of the spread of the data about that line.

A listing of the data that was used to produce **Figures 10-2** through **10-6** is shown in **Table 10-1**. This table includes USGS streamflow-gaging station numbers, the associated drainage areas, the 100-year flood peak discharge estimates by LP3, and the maximum recorded peak discharges. Watershed characteristics for each of these gaging stations is provided in the USGS report (Garrett and Gellenbeck, 1991). Maps of Arizona showing the locations of the gaging stations for this data compilation are shown in **Figures 10-7** and **10-8**.

FIGURE 10-2

100-YEAR PEAK DISCHARGE BY LP3 ANALYSIS (LP3 Q100) AND MAXIMUM RECORDED DISCHARGE (Q_M RECORD) vs. DRAINAGE AREA FOR 0.1 TO 5,000 SQUARE KILOMETERS

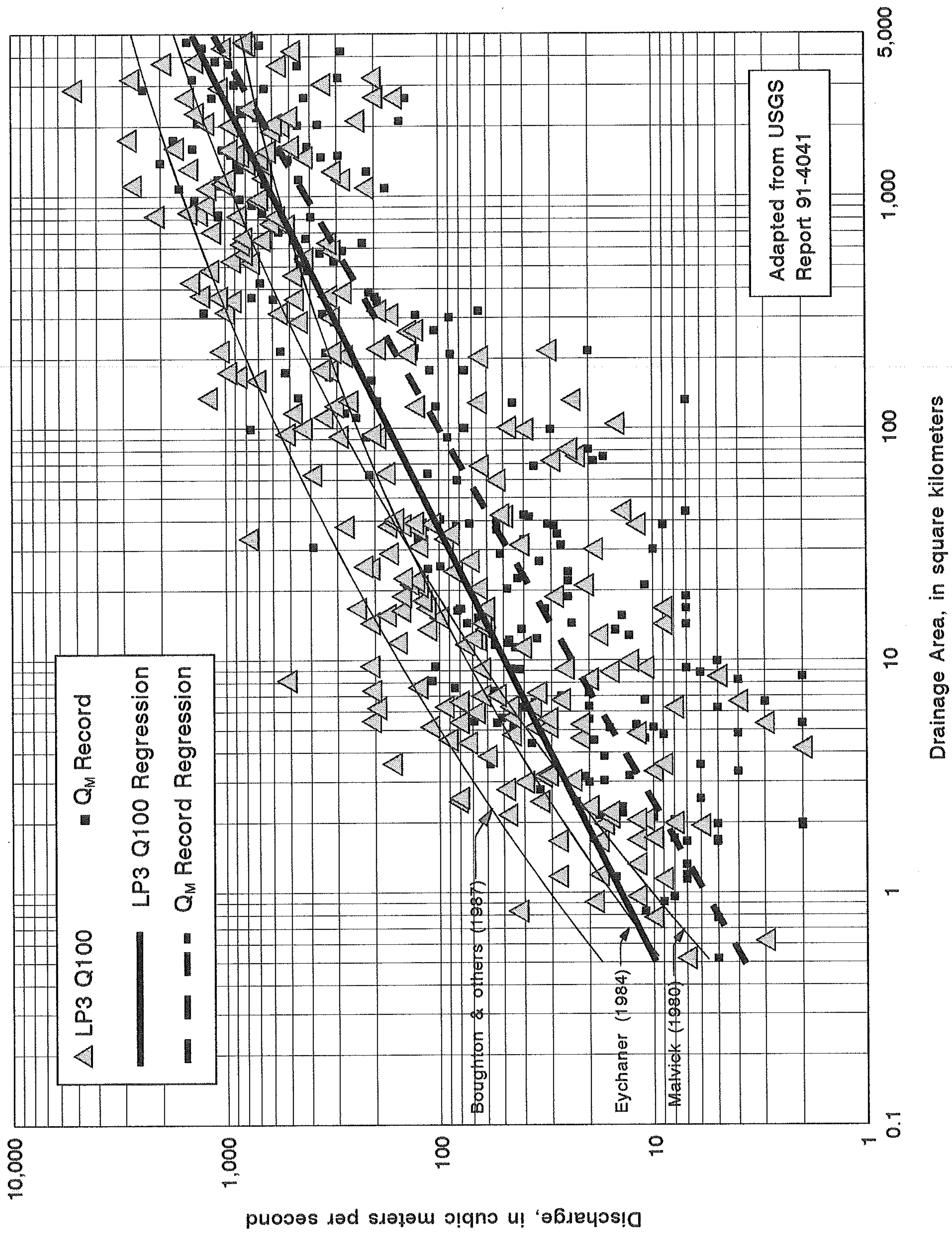


FIGURE 10-3

100-YEAR PEAK DISCHARGE BY LP3 ANALYSIS (LP3 Q100) AND MAXIMUM RECORDED DISCHARGE (Q_M RECORD) vs. DRAINAGE AREA FOR 0.1 TO 5.0 SQUARE KILOMETERS

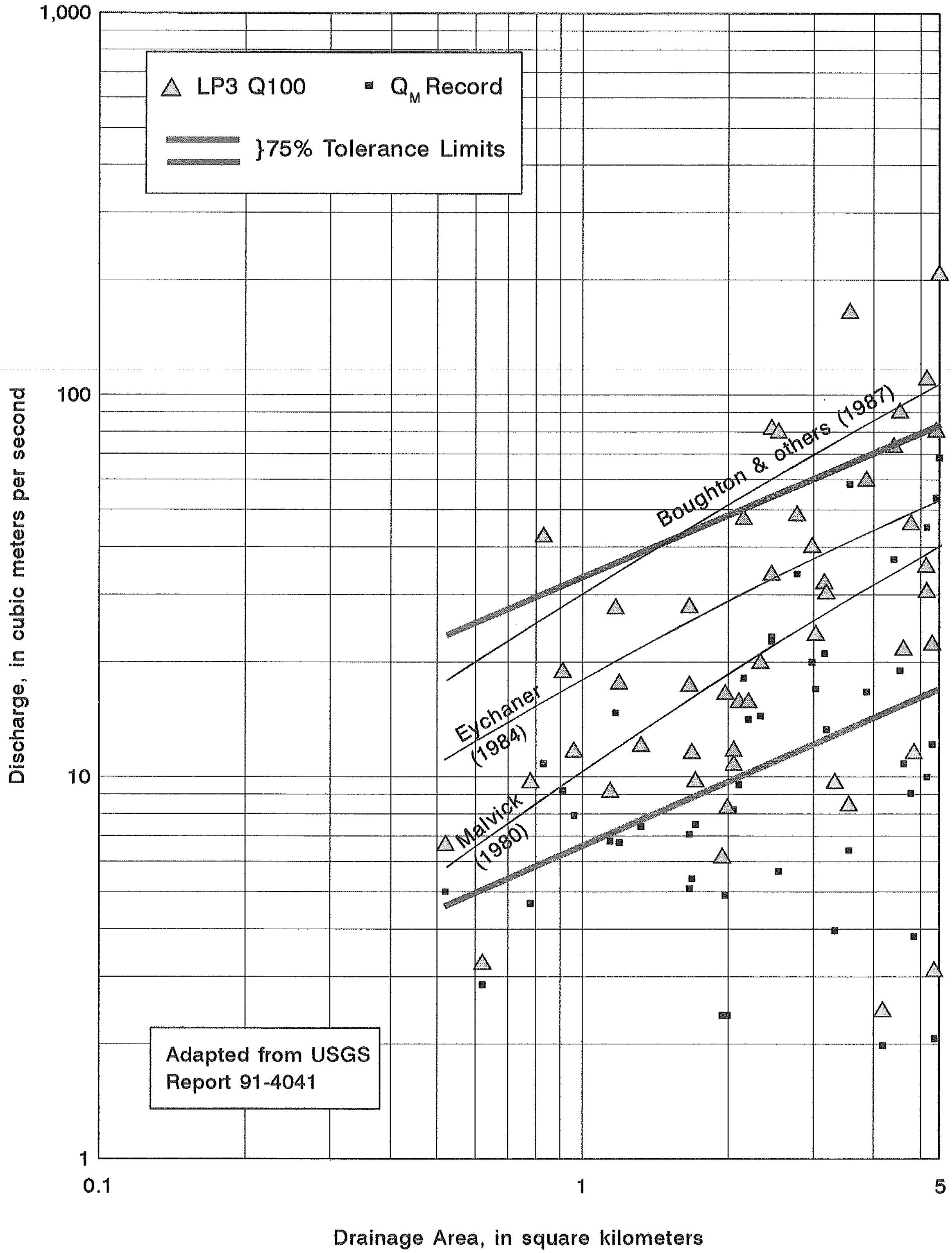


FIGURE 10-4

100-YEAR PEAK DISCHARGE BY LP3 ANALYSIS (LP3 Q100) AND MAXIMUM RECORDED DISCHARGE (Q_M RECORD) vs. DRAINAGE AREA FOR 1 TO 50 SQUARE KILOMETERS

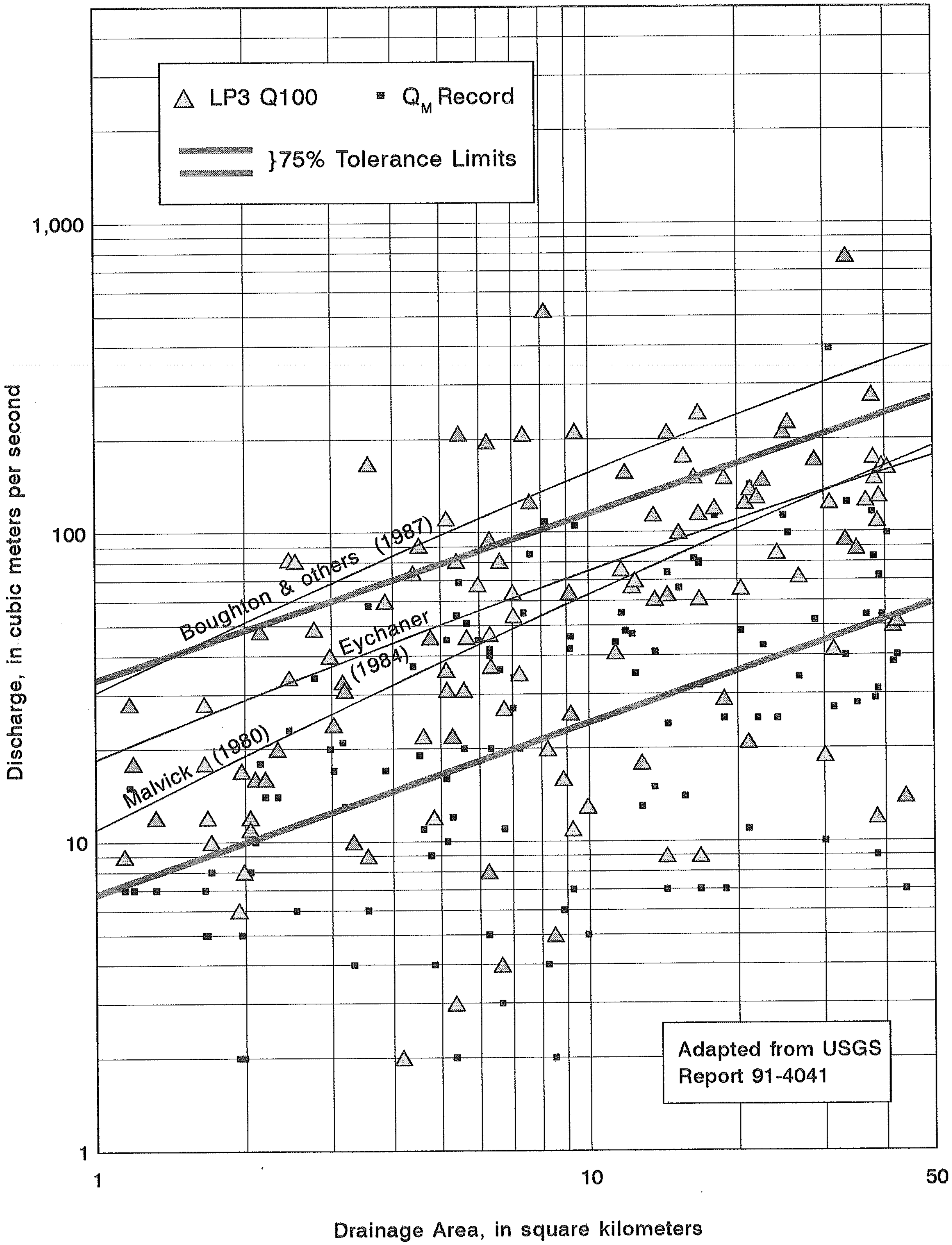


FIGURE 10-5

100-YEAR PEAK DISCHARGE BY LP3 ANALYSIS (LP3 Q100) AND MAXIMUM RECORDED DISCHARGE (Q_M RECORD) vs. DRAINAGE AREA FOR 10 TO 500 SQUARE KILOMETERS

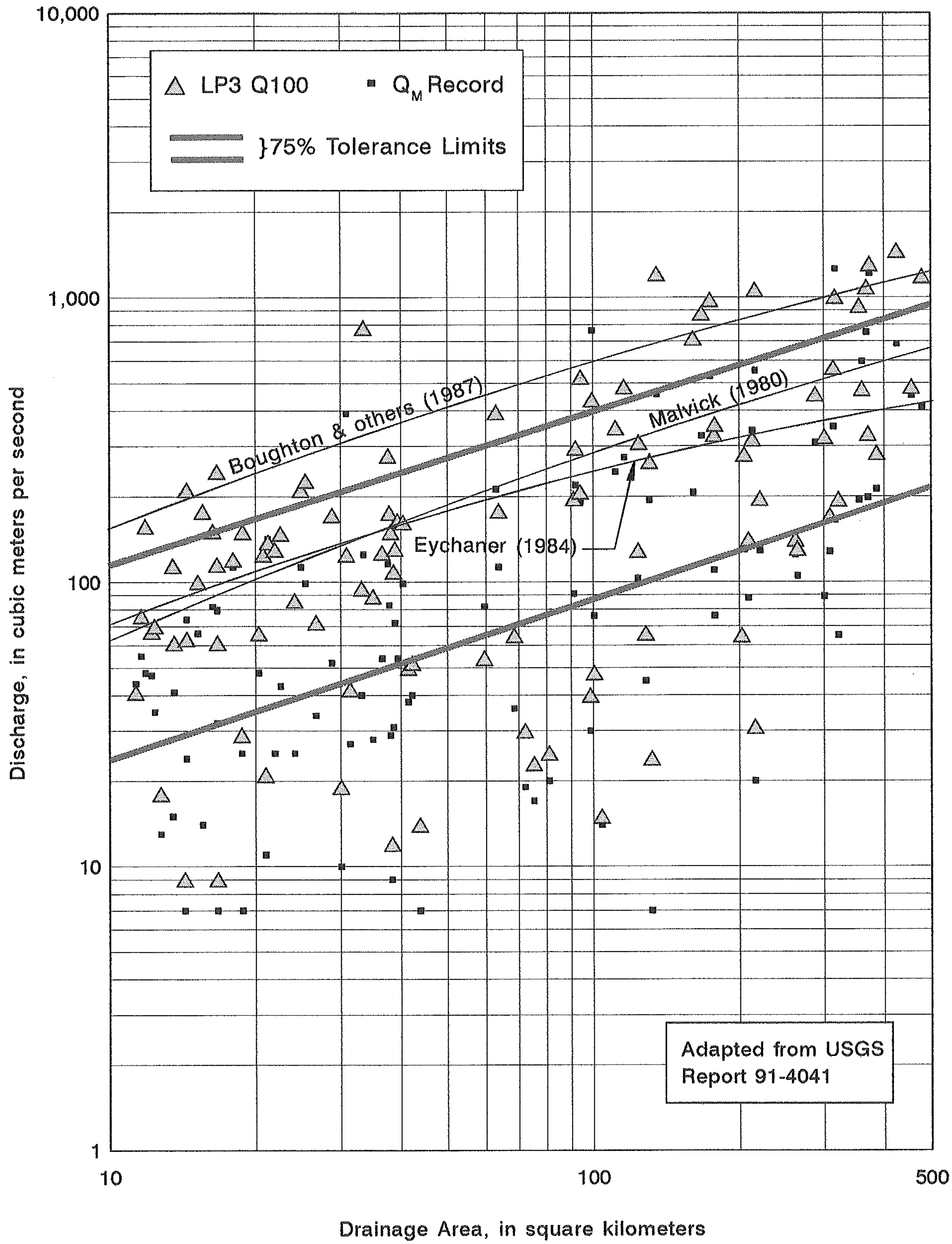


FIGURE 10-6

100-YEAR PEAK DISCHARGE BY LP3 ANALYSIS (LP3 Q100) AND MAXIMUM RECORDED DISCHARGE (Q_M RECORD) vs. DRAINAGE AREA FOR 100 TO 5,000 SQUARE KILOMETERS

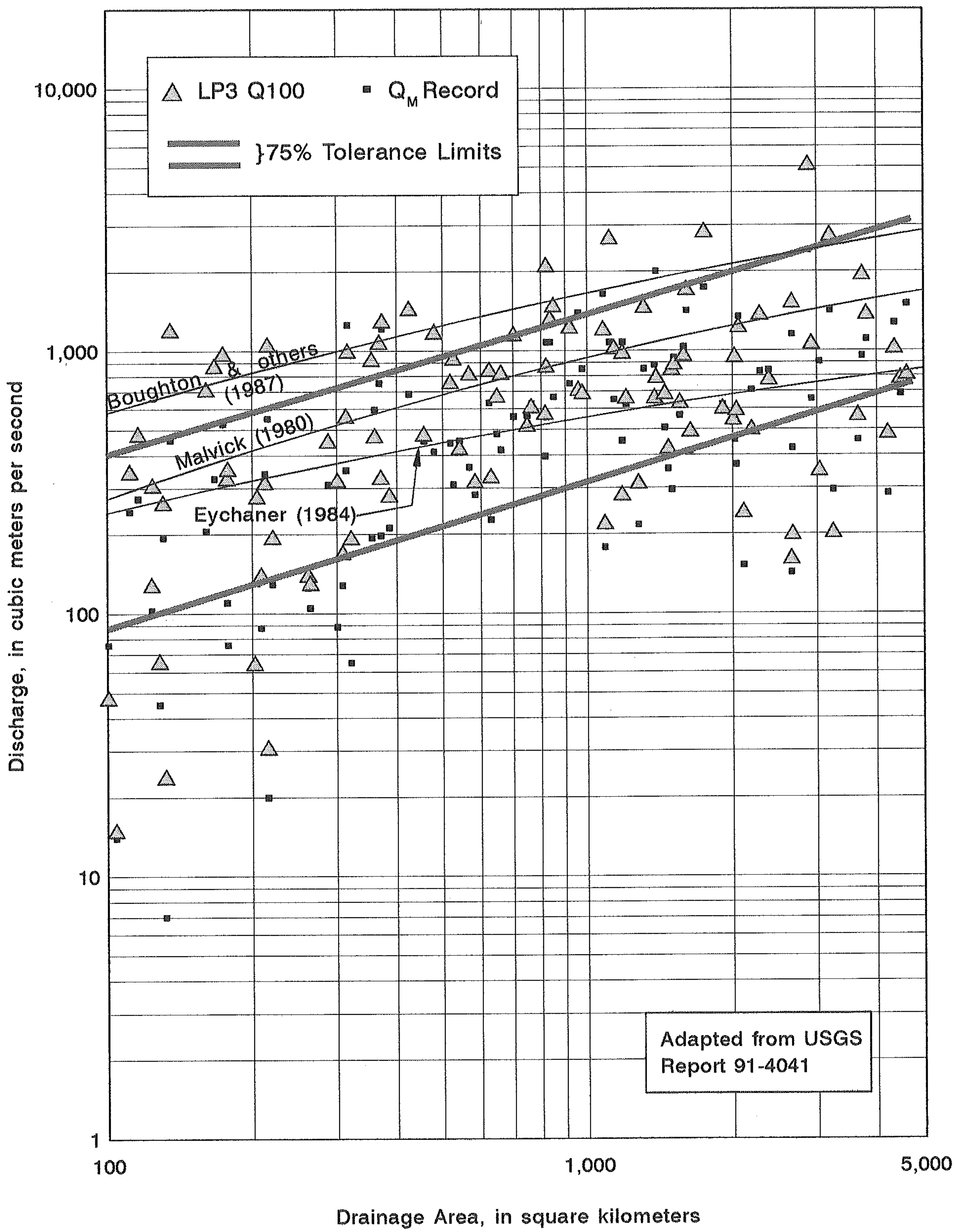


TABLE 10-1

USGS DATA LISTING FOR WATERSHEDS WITH DRAINAGE AREAS BETWEEN 0.5 AND 5,000 SQUARE KILOMETERS (ORDERED BY INCREASING DRAINAGE AREA)

Drainage Area sq km	Gage #	LP3 Q100 m^3/s	Qm Record m^3/s
0.52	404310	7	5
0.62	384200	3	3
0.78	429510	10	5
0.83	400200	43	11
0.91	385800	19	9
0.96	478600	12	8
1.14	520110	9	7
1.17	487140	28	15
1.19	483040	18	7
1.32	479200	12	7
1.66	505900	18	5
1.66	424700	28	7
1.68	536350	12	5
1.71	498600	10	8
1.94	503740	6	2
1.97	536100	17	5
1.99	428545	8	2
2.05	401245	12	8
2.05	471600	11	11
2.10	482330	16	10
2.15	468300	48	18
2.20	504100	16	14
2.33	520300	20	14
2.46	512420	82	23
2.46	483010	34	23
2.54	379980	81	6
2.77	512700	49	34
2.98	504400	40	20
3.03	483042	24	17
3.16	396400	33	21
3.19	419590	31	13
3.32	395100	10	4
3.55	379060	9	6
3.57	379100	167	58
3.86	520230	60	17
4.17	489080	2	2
4.40	424430	74	37
4.53	512200	91	19
4.61	400560	22	11
4.77	427700	46	9
4.84	400680	12	4
5.13	429150	36	16
5.15	520400	111	45
5.15	424410	31	10
5.28	483200	22	12
5.34	400660	3	2
5.39	483250	81	54
5.46	483030	209	69
5.57	485950	31	20
5.65	520160	46	51
5.96	482950	68	45
6.22	472400	197	91
6.24	400740	8	5
6.29	483025	95	42
6.29	519600	47	40
6.32	487400	37	20
6.60	496800	81	36
6.63	429400	4	3
6.73	510170	27	11
7.02	471700	64	27
7.04	485550	54	34
7.23	517200	35	20
7.38	403800	208	55
7.61	482480	126	85
8.16	404350	521	108
8.24	403930	20	4
8.50	400910	5	2
8.86	505600	16	6
9.14	483045	64	42
9.17	383020	26	46
9.25	400530	11	7
9.40	473200	212	105
9.92	404050	13	5
11.32	473600	41	44
11.63	510100	76	55
11.86	510070	157	48
12.22	520130	67	47
12.41	507700	70	35
12.77	485900	18	13
13.52	392800	114	15
13.60	470900	61	41
14.30	400700	9	7
14.43	515800	211	24
14.43	400580	63	74
15.23	379560	100	66
15.57	502700	177	14
16.34	516600	151	82
16.68	498900	115	80
16.68	507600	244	79
16.71	400565	61	32
16.73	484510	9	7
18.00	424480	120	113
18.75	482410	29	25
18.83	415050	150	7
20.33	400100	66	48
20.77	472100	125	123
21.00	400650	21	11
21.24	483000	138	142
21.94	423760	130	25
22.53	520100	148	43

TABLE 10-1
USGS DATA LISTING FOR WATERSHEDS WITH DRAINAGE AREAS BETWEEN 0.5 AND 5,000 SQUARE KILOMETERS (ORDERED BY INCREASING DRAINAGE AREA)
(Continued)

Drainage Area sq km	Gage #	LP3 Q100 m^3/s	Qm Record m^3/s	Drainage Area sq km	Gage #	LP3 Q100 m^3/s	Qm Record m^3/s
24.09	400290	86	25	178.19	519750	357	76
24.81	485570	211	113	202.54	491000	65	65
25.38	510080	227	99	204.87	537200	280	130
26.68	481700	72	34	209.01	379030	141	88
28.75	513820	172	52	212.90	480000	317	340
30.04	444100	19	10	215.75	513800	1062	552
30.82	487100	125	391	215.75	383500	31	20
31.34	520200	42	27	220.67	517280	196	129
33.15	488600	95	40	261.59	403000	141	125
33.41	519780	782	125	264.18	445500	131	105
34.97	424407	89	28	287.49	505200	456	309
36.52	484580	127	54	300.44	519760	323	89
37.56	503750	278	116	308.21	489700	171	128
37.81	428550	175	83	313.39	512300	566	351
38.07	423900	150	29	315.98	498870	1003	1257
38.33	489200	12	9	321.16	503800	195	65
38.59	503720	109	31	354.83	516800	932	194
38.85	456400	131	72	360.01	512100	476	595
39.37	510180	164	54	367.78	505350	1082	753
40.40	478200	162	99	370.37	424200	331	198
41.44	371100	50	38	372.96	478500	1306	1215
42.22	484200	52	40	385.91	446000	283	212
43.77	383600	14	7	424.76	510200	1456	685
59.57	482400	54	82	455.84	481750	484	453
62.94	501300	394	212	479.15	513835	1184	413
63.71	505300	178	113	518.00	497980	765	445
68.64	482420	65	36	525.77	496000	940	309
72.26	397800	30	19	541.31	481500	428	453
75.37	383400	23	17	567.21	484500	824	360
81.07	423780	25	20	582.75	494300	320	283
91.17	467120	196	91	624.19	505800	850	634
91.95	484000	295	219	629.37	520170	334	227
94.02	503000	207	189	647.50	486300	677	481
94.28	508300	524	193	660.45	502800	827	419
98.68	489070	40	30	701.89	397500	1161	561
99.46	484570	436	765	748.51	484560	524	566
100.49	492400	48	76	764.05	497800	617	629
104.12	490800	15	14	815.85	489100	581	396
111.37	483100	348	244	821.03	513890	2127	1076
116.03	485000	484	274	821.03	398500	881	558
123.80	517400	129	103	836.57	513910	1334	1076
124.32	505250	309	297	849.52	507980	1495	666
128.46	400300	66	45	919.45	504500	1238	748
130.80	484590	265	194	958.30	404340	716	1388
132.09	400600	24	7	976.43	446500	697	850
135.46	510150	1209	456	1080.03	515500	1218	1643
160.84	497900	716	206	1087.80	514200	222	178
167.31	513860	878	326	1113.70	498800	2705	1076
174.31	513780	980	527	1137.01	496500	1031	651
177.67	390500	329	110	1181.04	388400	286	453

TABLE 10-1
USGS DATA LISTING FOR WATERSHEDS WITH DRAINAGE AREAS BETWEEN 0.5 AND 5,000 SQUARE KILOMETERS (ORDERED BY INCREASING DRAINAGE AREA)
(Continued)

Drainage Area sq km	Gage #	LP3 Q100 m^3/s	Qm Record m^3/s	Drainage Area sq km	Gage #	LP3 Q100 m^3/s	Qm Record m^3/s
1183.63	484600	991	1076	2108.26	456000	245	152
1204.35	486800	668	623	2191.14	393500	507	708
1276.87	395900	317	217	2279.20	513970	1388	830
1310.54	444200	1481	850	2377.62	486000	784	841
1380.47	480500	668	878	2649.57	537500	163	143
1390.83	473000	799	2005	2657.34	468500	1543	1150
1450.40	489499	694	507	2662.52	403780	202	425
1473.71	535100	430	354	2874.90	512800	5154	2407
1499.61	401220	855	295	2921.52	424900	1073	654
1515.15	512500	898	937	3030.30	487250	354	906
1559.18	485500	640	566	3190.88	490500	2773	1416
1587.67	447000	969	1031	3237.50	535300	205	295
1608.39	399000	1725	1416	3651.90	382000	572	456
1636.88	494000	498	413	3727.01	425500	1971	952
1748.25	499000	2860	1739	3807.30	517000	1393	1104
1908.83	470500	609	623	4219.11	401260	490	286
2009.84	487000	552	541	4356.38	482000	1034	1274
2022.79	398000	957	456	4480.70	471550	793	685
2038.33	423820	600	368	4615.38	488500	821	1504
2061.64	516500	1243	1345				

FIGURE 10-7

LOCATION OF CONTINUOUS-GAGING STATIONS

(From Garrett and Gellenbeck, 1991)

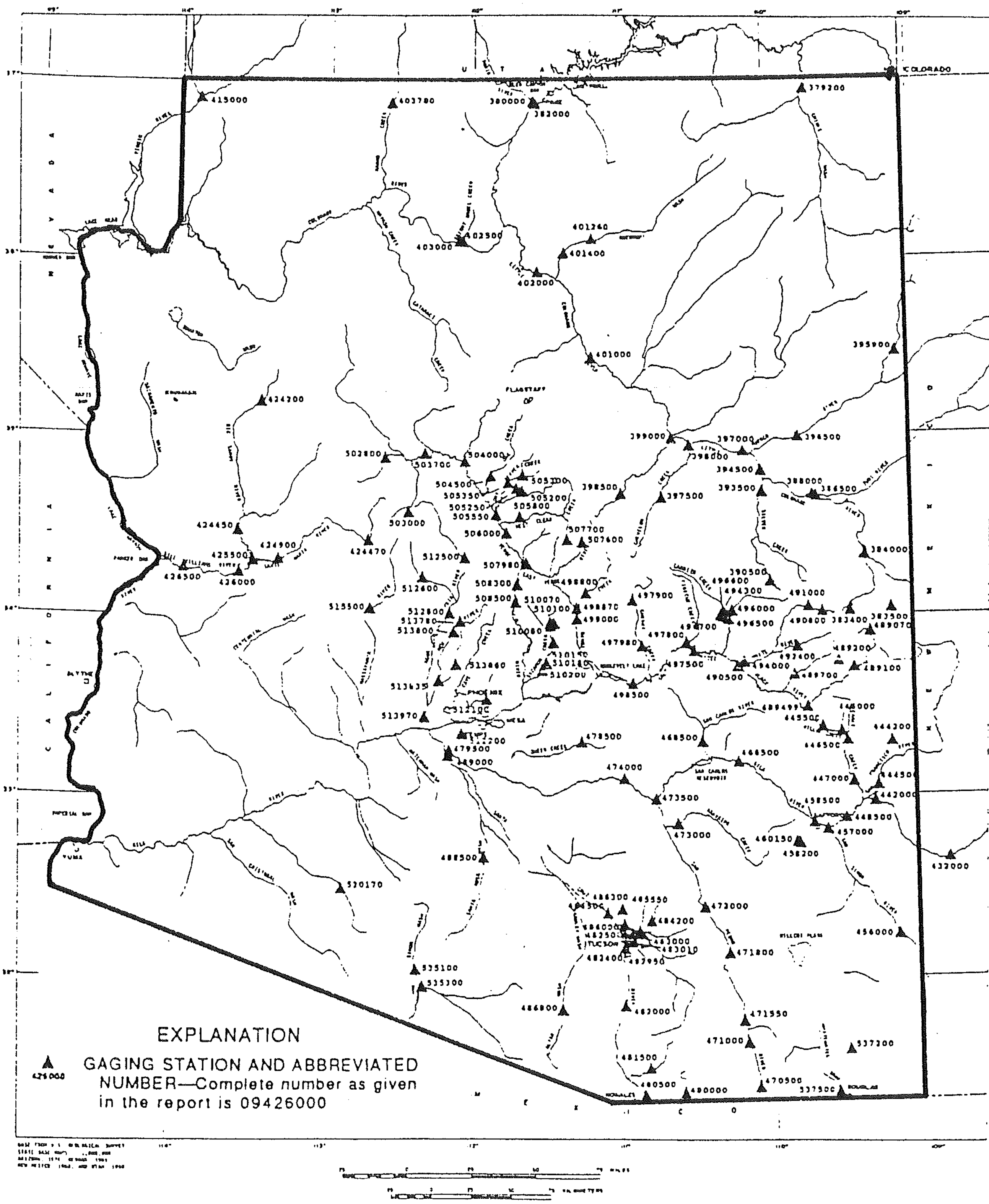
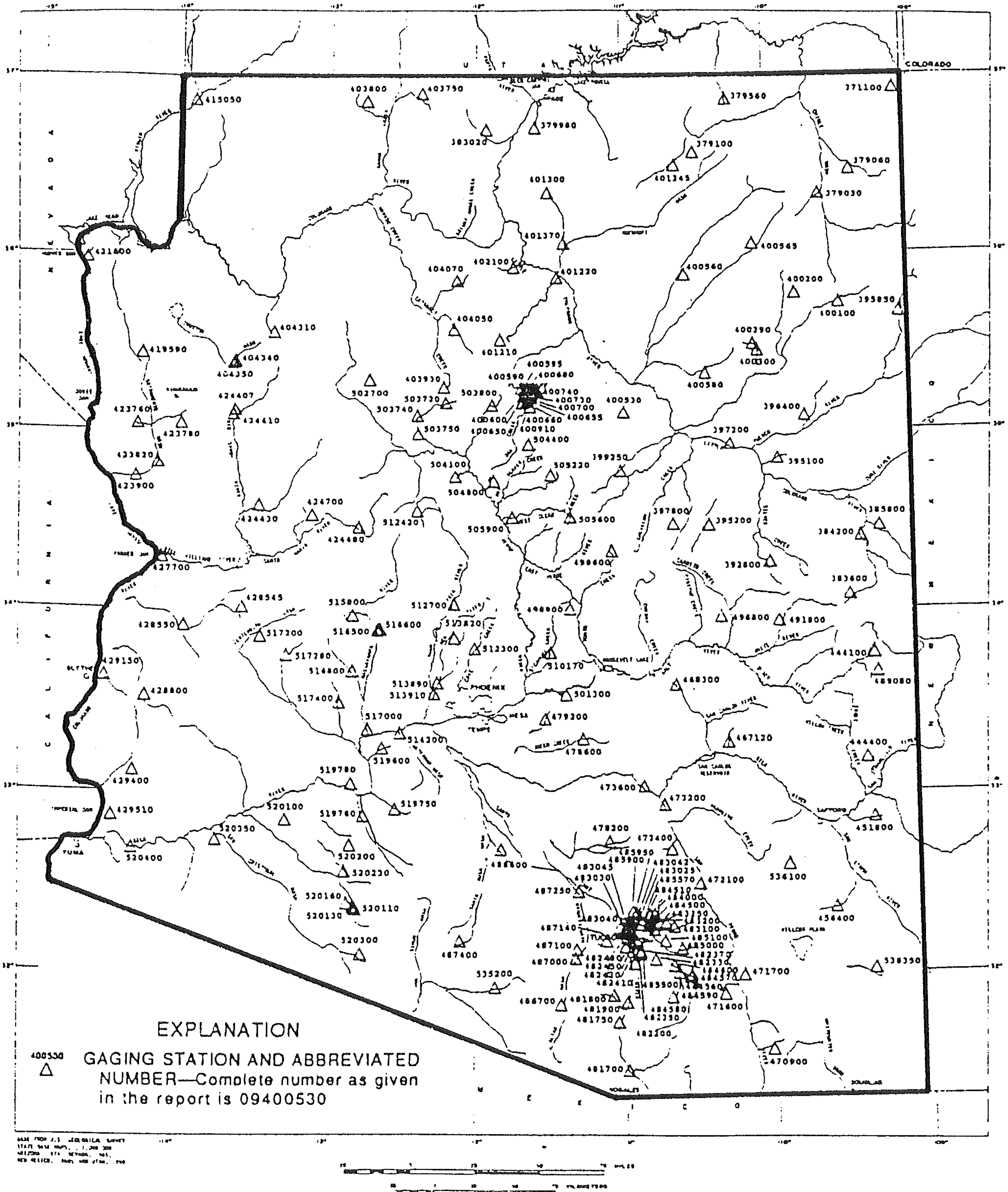


FIGURE 10-8

LOCATION OF CREST-STAGE GAGES

(From Garrett and Gellenbeck, 1991)



10.2.4 Indirect Method No. 3 - Regional Regression Equations

An analysis was performed of streamflow data for a study area comprised of Arizona, Nevada, Utah, and parts of New Mexico, Colorado, Wyoming, Texas, Idaho, Oregon, and California (Thomas, Hjalmarson, and Waltemeyer, 1994). 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.

Figure 10-9 is used to determine if the watershed is in one of the six regions (R8, R10, R11, R12, R13, or R14) in Arizona. If the mean basis elevation is above 2,300 meters (7,500 feet), then the watershed is in the High-Elevation Region (R1).

For each of the seven regions, regression equations (in English units) 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 <u>only</u> if the values of the independent variables for the watershed of interest are within the range of the data base 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. 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 usually one other independent variable. The regional regression equations were derived using English units, and two of the independent variables (PREC and EVAP, as described below) are obtained from figures that are presented in English units. It is impractical to convert these figures to metric units; therefore, the regional regression equations are to be used in English units and the results (in cfs) are to be converted to metric units (in m^3/s). Appropriate unit conversion equations are provided with each table of the regional regression equations. 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, and is determined by planimetering the contributing drainage area on the largest scale topographic map available.

2. ELEV is the mean basin elevation, in thousands of feet above mean sea level, and is determined by placing a transparent grid over the largest scale topographic map available. The elevation at each grid intersection within the drainage-area boundary is determined and elevations are averaged. The grid size should be selected so that at least 20 elevation points are sampled in the basin. As many as 100 points may be needed for large basins.

3. PREC is the normal annual precipitation, in inches, for 1931 through 1960 **(Figure 10-10)**. Usually PREC can be selected from **Figure 10-10** at the centroid of the watershed area. For large watersheds, PREC should be determined from **Figure 10-10** by a grid-sampling method as used for determining ELEV.

4. EVAP is the mean annual free water-surface evaporation, in inches, (Farnsworth and others, 1982) **(Figure 10-11)**. 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.

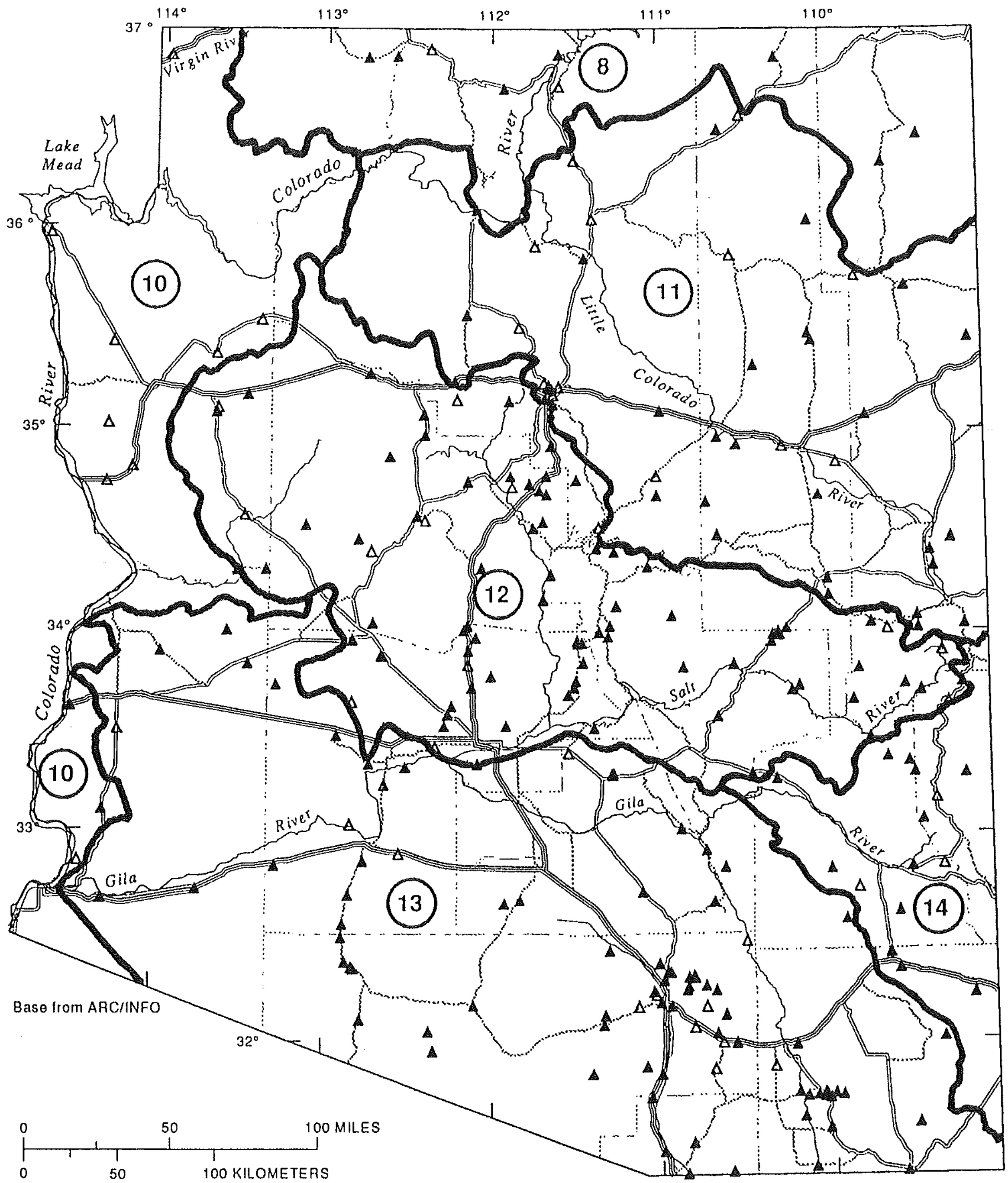
Also provided for each set of regression equations are graphs of the 100-year (LP3) flood peak discharge versus drainage area. A line depicting the relation between the 100-year peak discharge (computed from the regional regression equation) and drainage area is shown on each of those graphs.

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 equations	Figure No. for independent variable scatter diagram	Figure No. for 100-year peak discharge vs area graph
1	10-2	10-12	10-13
8	10-3	10-14	10-15
10	10-4	NA	10-16
11	10-5	10-17	10-18
12	10-6	10-19	10-20
13	10-7	NA	10-21
14	10-8	10-22	10-23

FIGURE 10-9

FLOOD REGIONS IN ARIZONA



EXPLANATION

BOUNDARY OF FLOOD REGIONS

11 FLOOD-REGION NUMBER

INTERSTATE HIGHWAY

U.S. HIGHWAY

STATE HIGHWAY

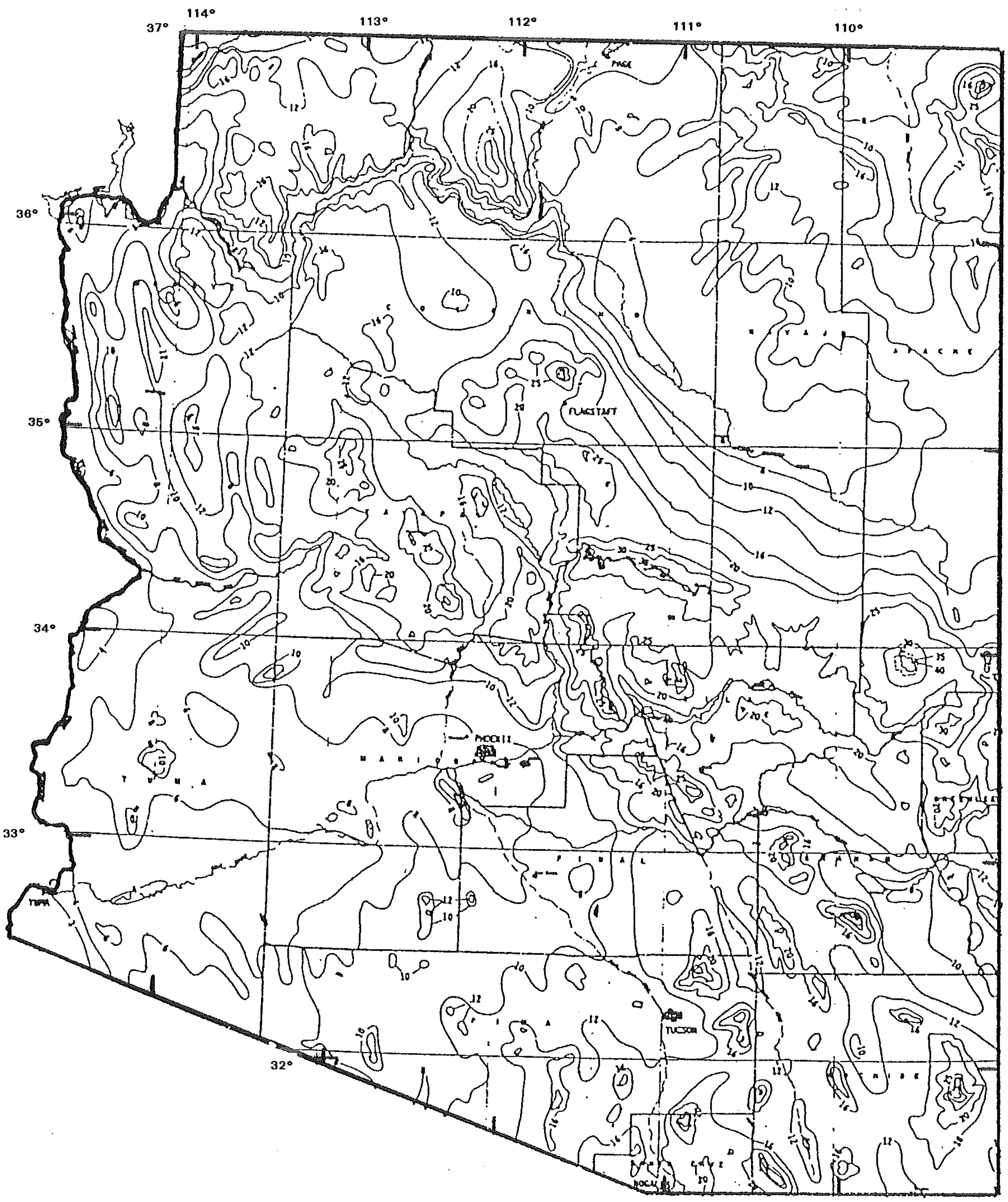
GAGING STATION

▲ Station relation defined

Δ Station relation undefined

FIGURE 10-10

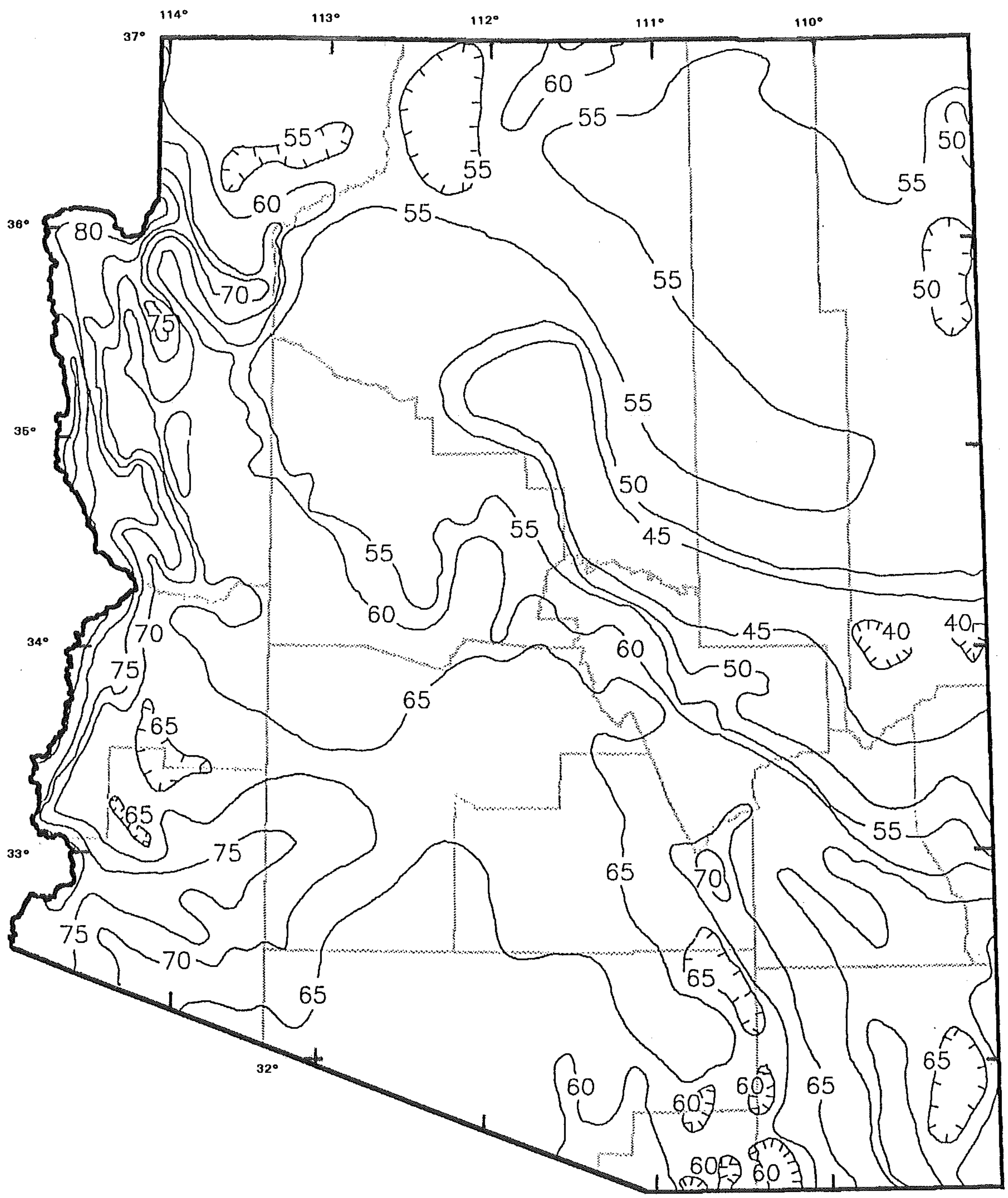
MEAN ANNUAL PRECIPITATION (PREC), 1931-60



— 65 — Mean Annual Precipitation, in inches

FIGURE 10-11

MEAN ANNUAL EVAPORATION (EVAP)



— 65 — Mean Annual Evaporation, in inches

TABLE 10-2

FLOOD MAGNITUDE-FREQUENCY RELATIONS FOR THE HIGH ELEVATION REGION (R1)

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

Note: Convert AREA, in square kilometers, to square miles by multiplying by 0.386. PREC, in inches, is obtained from Figure 10-10.

Recurrence interval, in years	Equation	Average standard error of model, 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

Note: Convert Q, in cfs, to m^3/s by multiplying by 0.028.

FIGURE 10-12

SCATTER DIAGRAM OF INDEPENDENT VARIABLES FOR R1 REGRESSION EQUATION

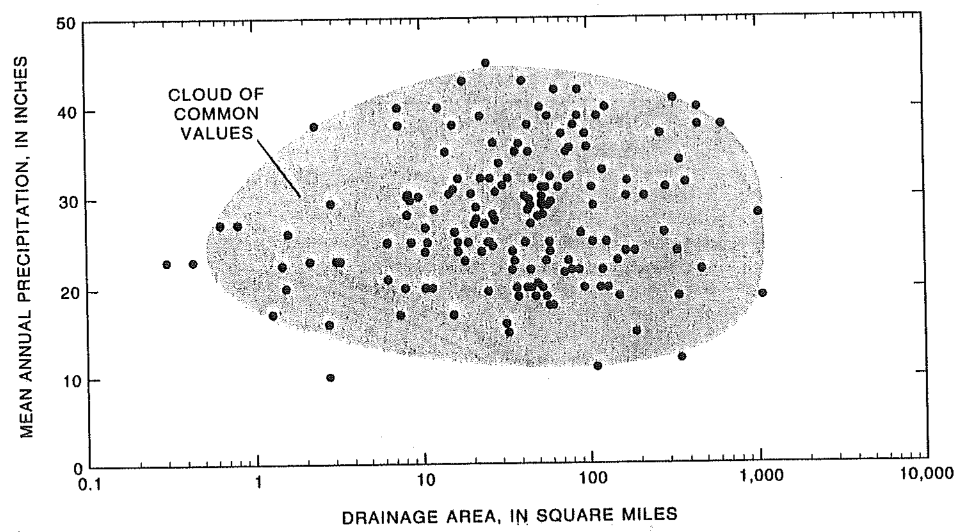


FIGURE 10-13

Q_{100} DATA POINTS AND 100-YEAR PEAK DISCHARGE RELATION FOR R1

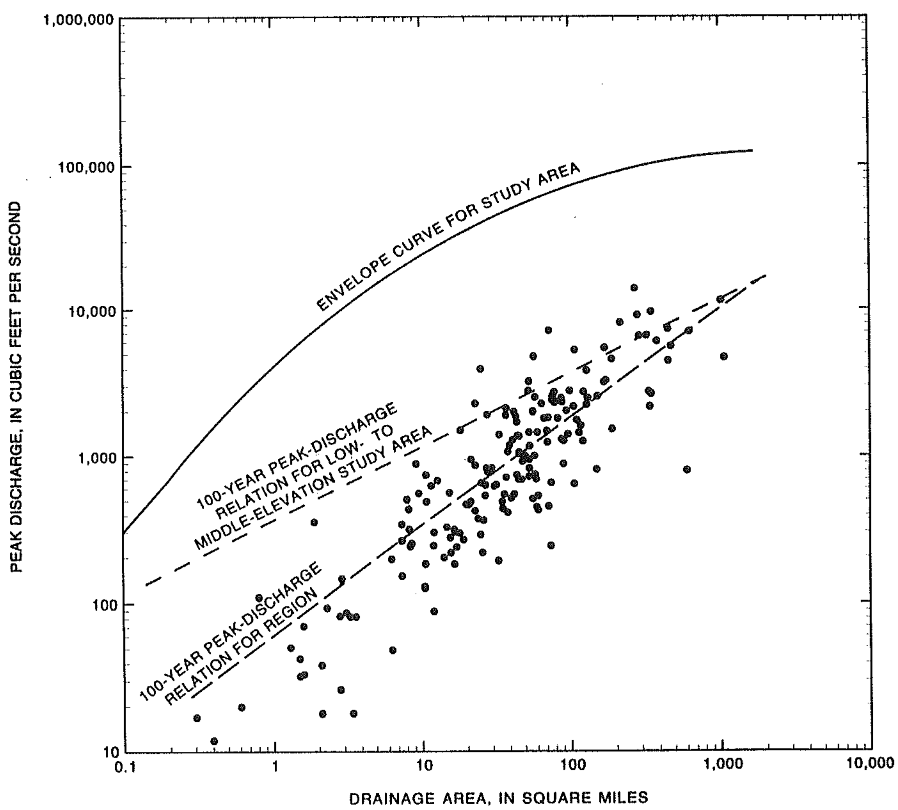


TABLE 10-3

FLOOD MAGNITUDE-FREQUENCY RELATIONS FOR THE FOUR CORNERS REGION (R8)

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

Note: Convert AREA, in square kilometers, to square miles by multiplying by 0.386. Convert ELEV, in meters, to feet by multiplying by 3.28.

Recurrence interval, in years	Equation	Average standard error of model, 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

Note: Convert Q, in cfs, to m^3/s to multiplying by 0.028.

FIGURE 10-14

SCATTER DIAGRAM OF INDEPENDENT VARIABLES FOR R8 REGRESSION EQUATION

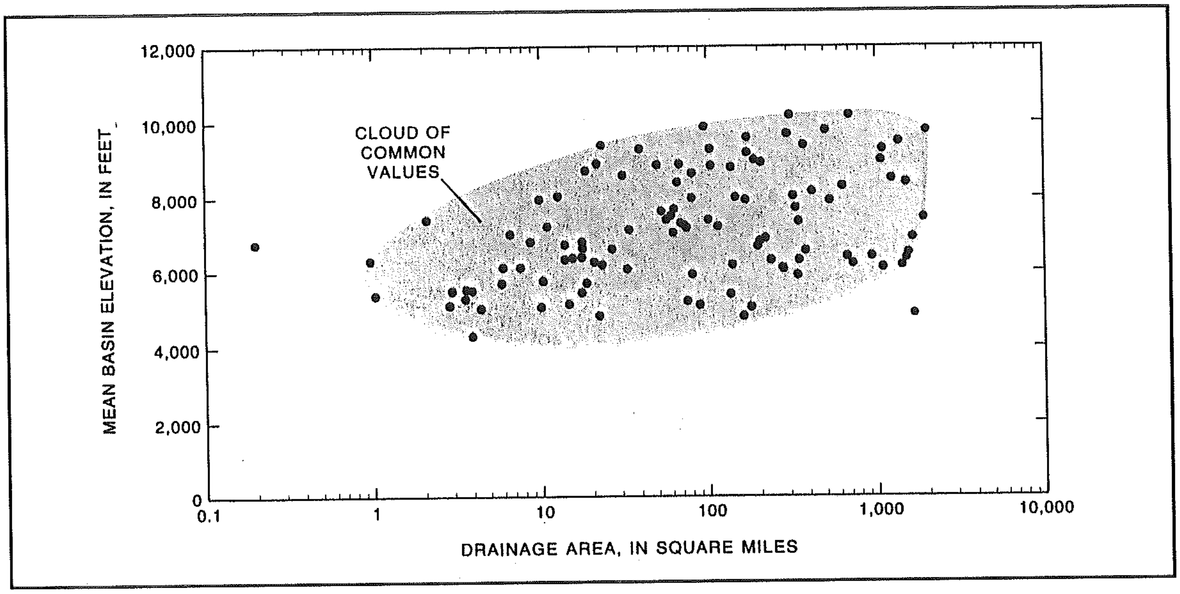


FIGURE 10-15

Q_{100} DATA POINTS AND 100-YEAR PEAK DISCHARGE RELATION FOR R8

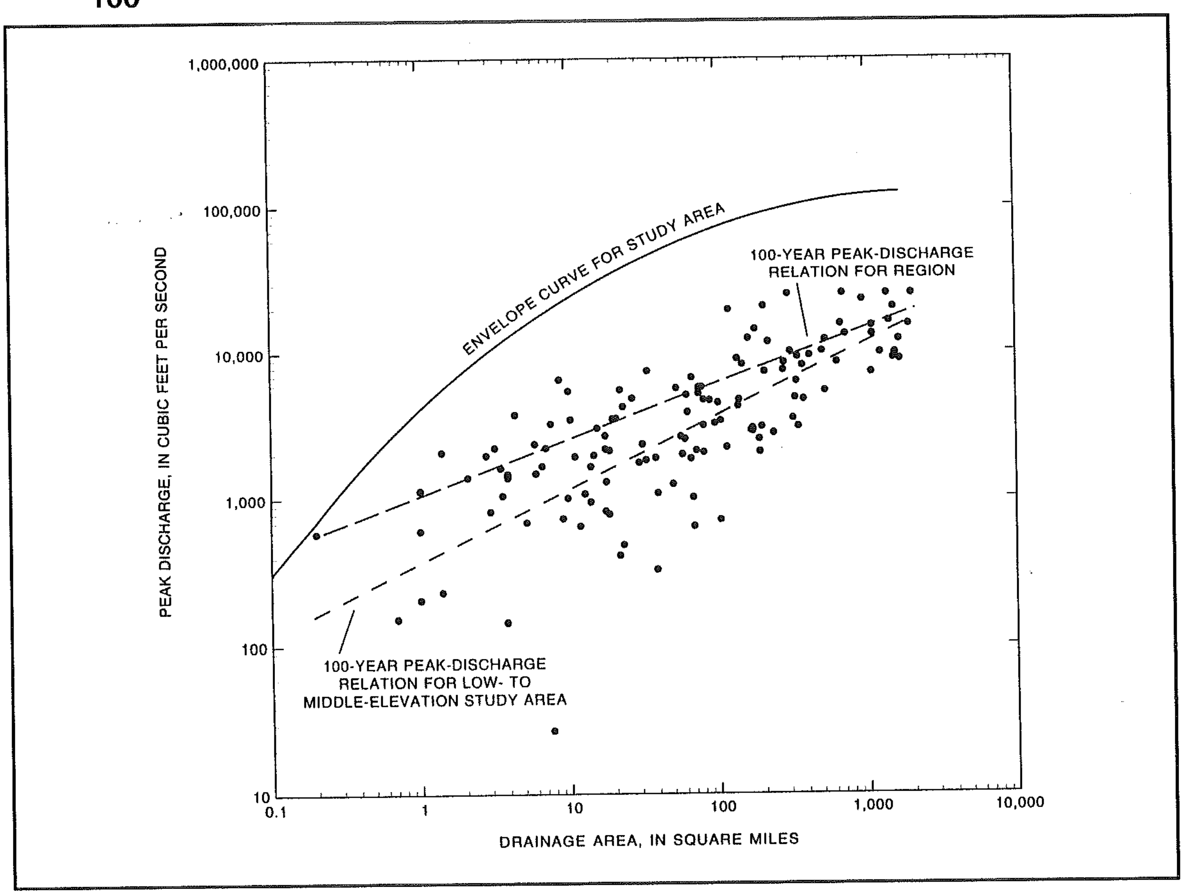


TABLE 10-4

FLOOD MAGNITUDE-FREQUENCY RELATIONS FOR THE SOUTHERN GREAT BASIN REGION (R10)

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

Note: Convert AREA, in square kilometers, to square miles by multiplying by 0.386.

Recurrence interval, in years	Equation	Estimated Average standard error of regression, in log units
2	$Q = 12\ AREA^{0.58}$	1.140
5	$Q = 85\ AREA^{0.59}$	.602
10	$Q = 200\ AREA^{0.62}$	.675
25	$Q = 400\ AREA^{0.65}$	.949
50	$Q = 590\ AREA^{0.67}$	.928
100	$Q = 850\ AREA^{0.67}$	1.230

Note: Convert Q, in cfs, to m^3/s by multiplying by 0.028.

FIGURE 10-16

Q_{100} DATA POINTS AND 100-YEAR PEAK DISCHARGE RELATION FOR R10

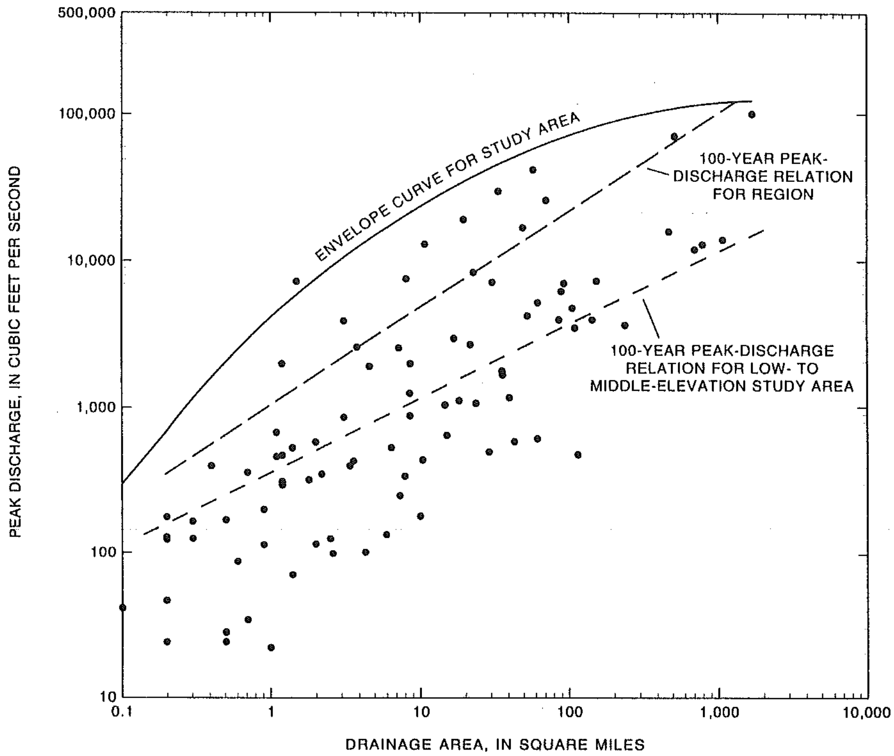


TABLE 10-5

FLOOD MAGNITUDE-FREQUENCY RELATIONS FOR THE NORTHEAST ARIZONA REGION (R11)

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

Note: Convert AREA, in square kilometers, to square miles by multiplying by 0.386. EVAP, in inches, is obtained from Figure 10-11.

Recurrence interval, in years	Equation	Estimated Average standard error of regression, in log units
2	$Q = 26\ AREA^{0.62}$	.609
5	$Q = 130\ AREA^{0.56}$	.309
10	$Q = 0.10\ AREA^{0.52}\ EVAP^{2.0}$	.296
25	$Q = 0.17\ AREA^{0.52}\ EVAP^{2.0}$	.191
50	$Q = 0.24\ AREA^{0.54}\ EVAP^{2.0}$	.294
100	$Q = 0.27\ AREA^{0.58}\ EVAP^{2.0}$	.863

Note: Convert Q, in cfs, to m^3/s by multiplying by 0.028.

FIGURE 10-17

SCATTER DIAGRAM OF INDEPENDENT VARIABLES FOR R11 REGRESSION EQUATION

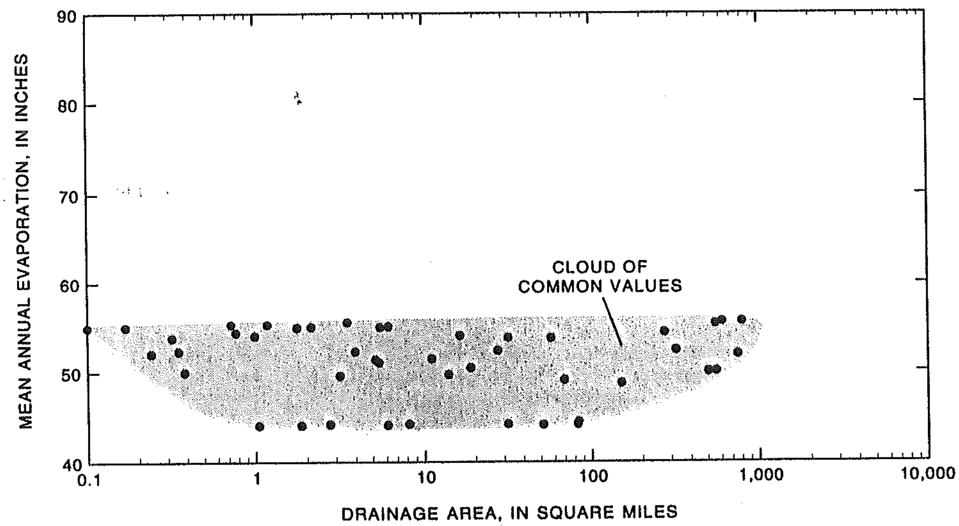


FIGURE 10-18

Q_{100} DATA POINTS AND 100-YEAR PEAK DISCHARGE RELATION FOR R11

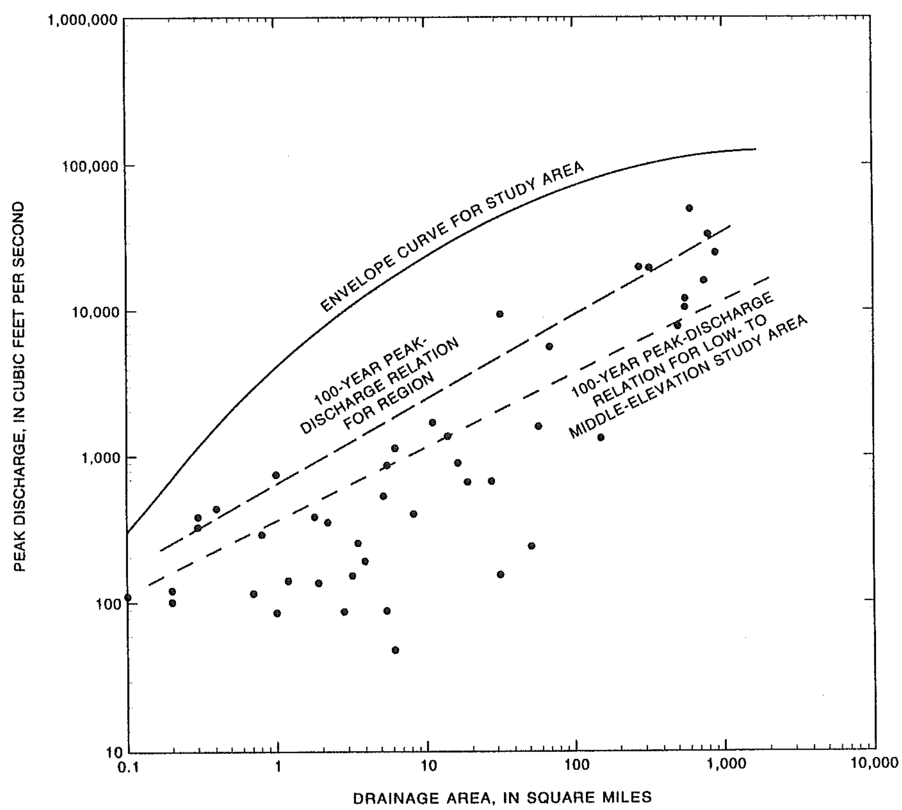


TABLE 10-6

FLOOD MAGNITUDE-FREQUENCY RELATIONS FOR THE CENTRAL ARIZONA REGION (R12)

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

Note: Convert AREA, in square kilometers, to square miles by multiplying by 0.386. Convert ELEV, in meters, to feet by multiplying by 3.28.

Recurrence interval, in years	Equation	Average standard error of model, in percent
2	$Q = 41.1\ AREA^{0.629}$	105
5	$Q = 238\ AREA^{0.687}\ (ELEV/1{,}000)^{-0.358}$	68
10	$Q = 479\ AREA^{0.661}\ (ELEV/1{,}000)^{-0.398}$	52
25	$Q = 942\ AREA^{0.630}\ (ELEV/1{,}000)^{-0.383}$	40
50	$LOG\ Q = 7.36 - 4.17\ AREA^{-0.08} - 0.440\ LOG\ (ELEV/1{,}000)$	37
100	$LOG\ Q = 6.55 - 3.17\ AREA^{-0.11} - 0.454\ LOG\ (ELEV/1{,}000)$	39

Note: Convert Q, in cfs, to m^3/s by multiplying by 0.028.

FIGURE 10-19

SCATTER DIAGRAM OF INDEPENDENT VARIABLES FOR R12 REGRESSION EQUATION

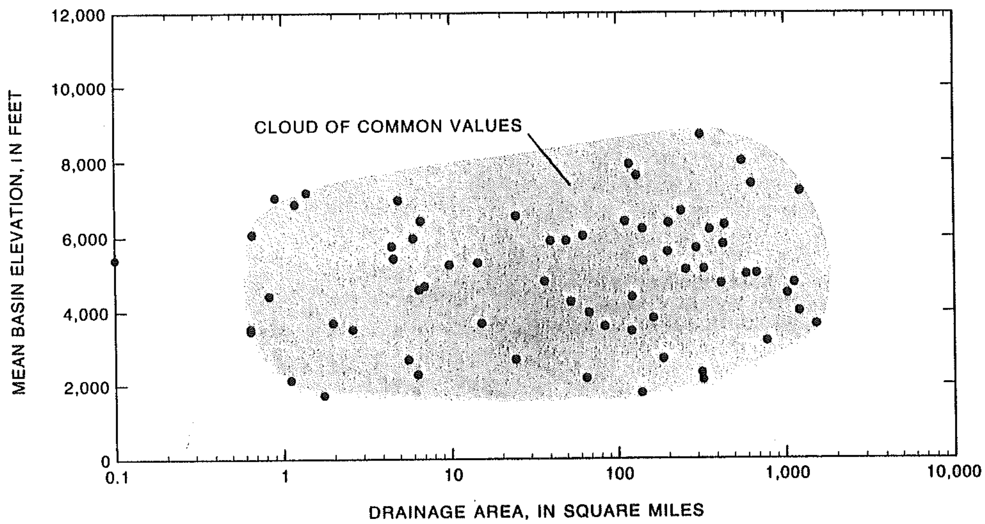


FIGURE 10-20

Q_{100} DATA POINTS AND 100-YEAR PEAK DISCHARGE RELATION FOR R12

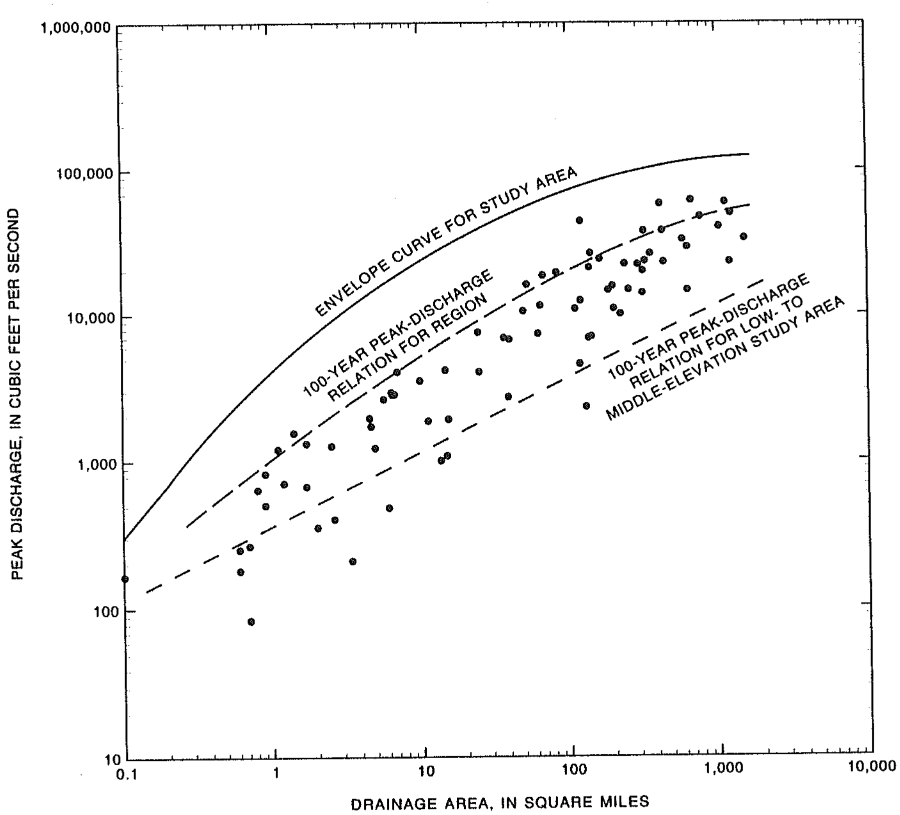


TABLE 10-7

FLOOD MAGNITUDE-FREQUENCY RELATIONS FOR THE SOUTHERN ARIZONA REGION (R13)

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

Note: Convert AREA, in square kilometers, to square miles by multiplying by 0.386.

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

Note: Convert Q, in cfs, to m^3/s by multiplying by 0.028.

FIGURE 10-21

Q_{100} DATA POINTS AND 100-YEAR PEAK DISCHARGE RELATION FOR R13

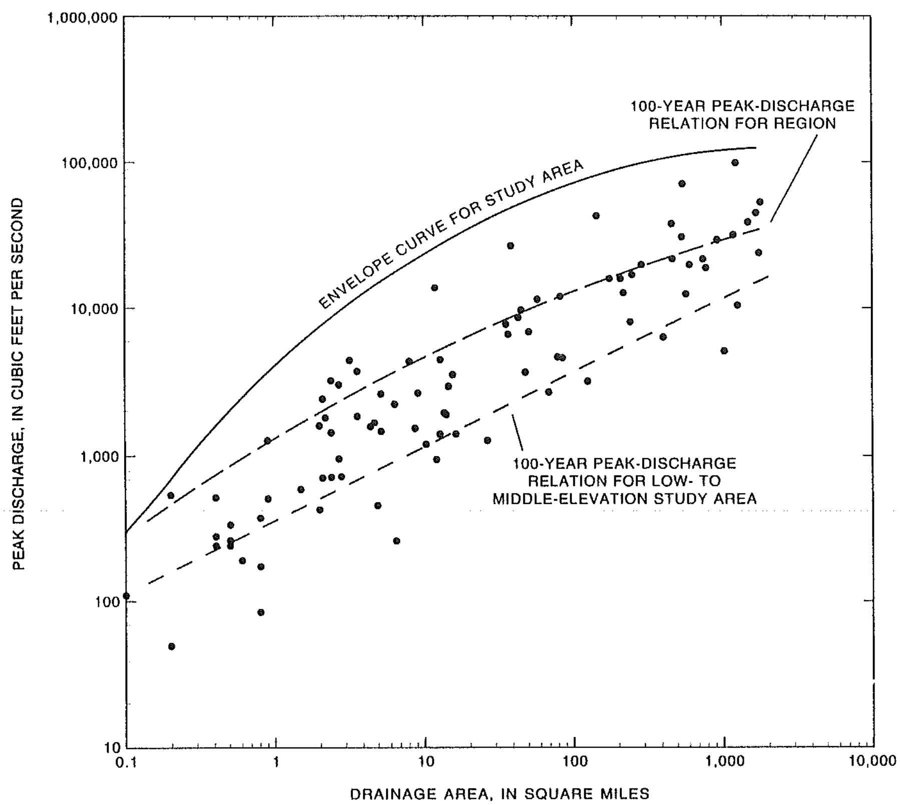


TABLE 10-8

FLOOD MAGNITUDE-FREQUENCY RELATIONS FOR THE UPPER GILA BASIN REGION (R14)

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

Note: Convert AREA, in square kilometers, to square miles by multiplying by 0.386. Convert ELEV, in meters, to feet by multiplying by 3.28.

Recurrence interval, in years	Equation	Average standard error of model, in percent
2	$Q = 583\ AREA^{0.588}\ (ELEV/1{,}000)^{-1.3}$	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

Note: Convert Q, in cfs, to m^3/s by multiplying by 0.028.

FIGURE 10-22

SCATTER DIAGRAM OF INDEPENDENT VARIABLES FOR R14 REGRESSION EQUATION

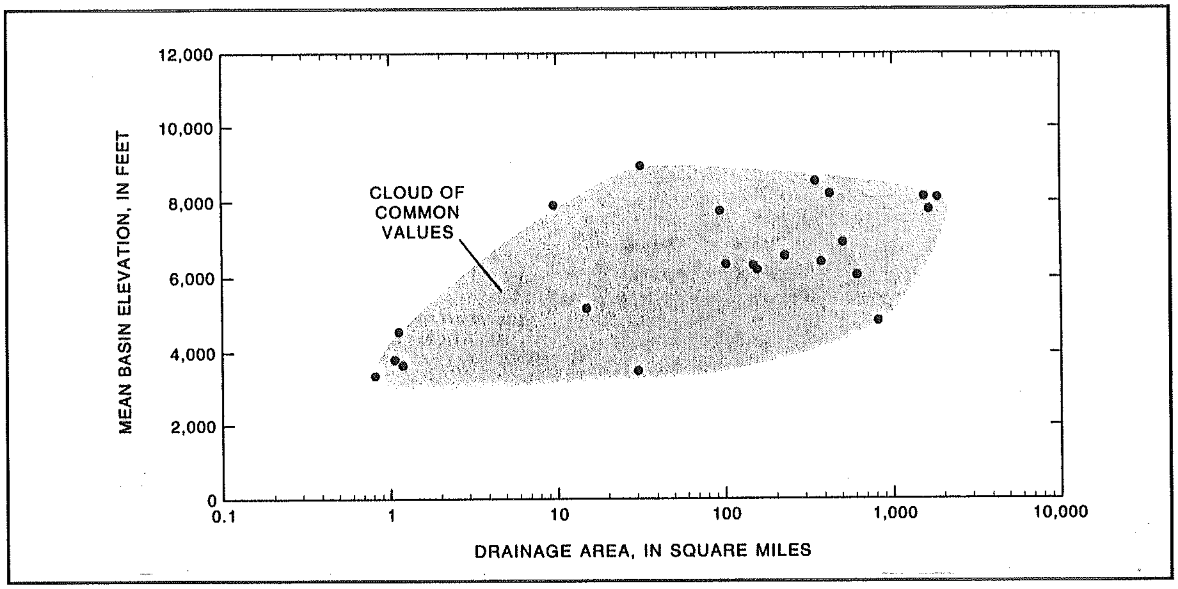
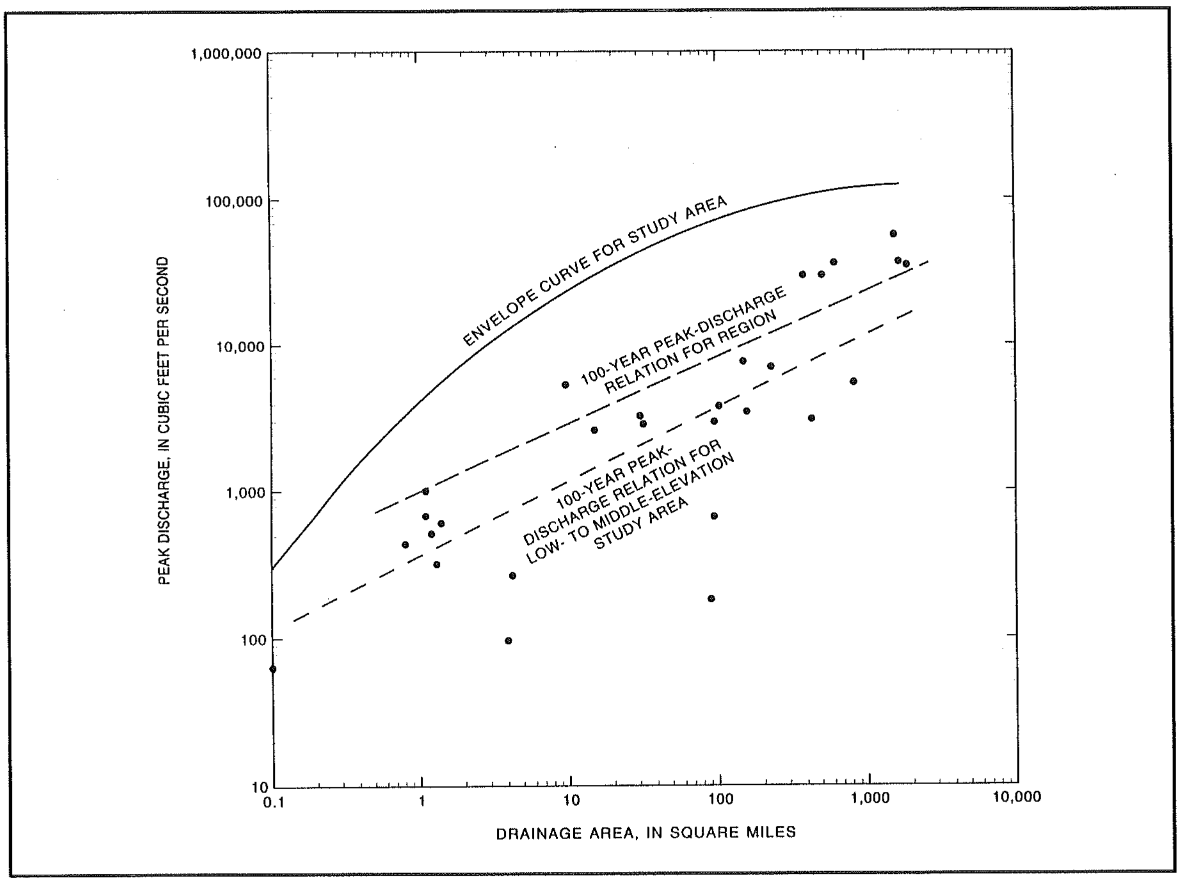


FIGURE 10-23

Q_{100} DATA POINTS AND 100-YEAR PEAK DISCHARGE RELATION FOR R14



10.2.5 Applications and Limitations

The three indirect methods can be applied to any watershed in Arizona, gaged or ungaged. 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 data base used to derive the particular method. Refer to the independent variable scatter diagrams when using the Regional Regression Equations.

2. The majority of the data in all three 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. These methods (other than envelope curves) produce discharge values that are statistically based averages for watersheds in the data base. 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 indirect methods to results by analytic methods and/or flood frequency analysis are:

    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 <u>low</u> 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 <u>high</u> 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, and

h. upstream water regulation or diversion.

10.3 INSTRUCTIONS

The following instructions should be followed for verifying peak discharges that are derived by either analytic methods (Rational Method or rainfall-runoff modeling) or flood frequency analyses (collectively these are called primary peak discharge estimates in the Instructions) with peak discharges that are developed by indirect methods (called secondary peak discharge estimates).

A. Compute Primary Peak Discharge:

1. The primary peak discharge will be calculated by either the Rational Method, rainfall-runoff modeling (HEC-1), or flood frequency analysis according to procedures contained within this Manual.

B. Verification with Unit Peak Discharge Curves:

1. For a given watershed of drainage area (A), in square kilometers, divide the 100-year primary peak discharge estimate by A.

2. Plot the unit peak discharge from Step B.1 on a copy of **Figure 10-1**. Note the location of the plotted point in relation to the various curves in that figure. Particular attention should be given to Curves C, G, and H.

3. Tabulate the primary unit peak discharge estimate and the secondary unit peak discharge estimates from curves C, G, and H.

C. Verification with USGS Data for Arizona:

1. Calculate the 100-year secondary peak discharge estimate by Equation 10-1.

2. Select **Figure 10-3** through **10-6** according to watershed drainage area size, and plot the 100-year primary peak discharge estimate on a copy of that figure.

3. Using watershed drainage area size as a guide, identify gaged watersheds of the same approximate size from **Table 10-1**. Tabulate peak discharge

statistics, maximum recorded peak discharges, and watershed characteristics for those gaged watersheds by using the USGS report (Garrett and Gellenbeck, 1991). Compare these to the primary peak discharge estimates and watershed characteristics for the watershed of interest.

D. Verification with Regional Regression Equations:

1. Calculate the average watershed elevation (ELEV).

2. Determine whether the watershed is in the High Elevation Region (R1) (mean basin elevation above 2,300 meters (7,500 feet)). If the watershed is in R1, proceed to Step D.3. If the watershed is not in R1, determine the flood region **(Figure 10-9)**, and then proceed to Step D.3.

3. Depending on the flood region, calculate the applicable values of the independent variables for the watershed, i.e., AREA, ELEV, PREC, and EVAP.

   AREA and ELEV must be converted to English units if they are measured in metric units.

   PREC is determined using a grid-sample average of values for the watershed (**Figure 10-10**).

   EVAP is determined for the study-site location (**Figure 10-11**).

4. Check the values of the independent variables using the appropriate scatter diagram to determine if the values of the variables are in the "cloud of common values." (Proceed with the analysis regardless of the outcome, but clearly note if the variable values are not within the "cloud of common values.")

5. Calculate the secondary peak discharge estimates using the applicable regression equations for the flood region within which the project site is located.

6. Plot the 100-year primary peak discharge estimate on a copy of the appropriate Q_{100} data points and 100-year peak discharge relation graph **(Figures 10-13, 10-15, etc.)**

7. Convert the peak discharge estimates by the regional regression equations from cfs to cms by multiplying by 0.028.

8. Tabulate the primary and secondary peak discharge estimates from this method.

E. For all three Indirect Methods:

1. Quantitatively and qualitatively analyze the results of the primary and the secondary peak discharge estimates. Address watershed characteristics that may explain differences between the primary and secondary estimates.

2. Prepare a summary of results by all methods and a qualitative evaluation of the results.

APPENDIX A

GLOSSARY

GLOSSARY

annual flood - The maximum instantaneous peak discharge in each year of record.

annual flood series - A sequence of annual floods.

attenuate - To reduce the flood peak discharge and lengthen the time base of the flood wave.

baseflow - Discharge in a river prior to the onset of direct runoff from a rainfall event.

bed form - The irregularities of the channel bed that are larger than the largest bed material particles.

bed load - Fluvial material moving on or near the bed of the watercourse.

bed material - Fluvial material that exists in appreciable quantities in the bed of the watercourse.

broken record - A systematic record which is divided into separate continuous segments because of discontinuation of recording for a year or longer.

concentration point - A physical location in a watershed where all surface runoff must pass to exit the watershed.

direct runoff - The same as rainfall excess.

distribution - Function describing the frequency with which random events of various magnitudes occur.

drainage area - The total area contributing to surface runoff at a point of interest (flow concentration point).

duration - Used either as the length of time for rainfall, such as a 6-hour storm, or as length of time for rainfall excess, such as used to specify the duration of rainfall excess for a unit hydrograph.

effective impervious area - The portion of a land area, expressed in percent of total land area, that will drain directly to the outlet of the drainage area without flowing over pervious area. This is often called directly connected impervious area.

exceedance probability - Probability that a flood discharge will exceed a specified magnitude in a given time period, usually one year unless otherwise indicated.

frequency - The measure of the probability of occurrence or exceedance of a flood magnitude in a number of observations.

historic data - Record of major floods which occurred either before or after the period of systematic data collection.

homogeneity - Records from the same population.

hydrograph - A continuous plot of instantaneous discharge versus time.

hydrologic soil group - A classification system developed by the SCS to place soils into one of four groups based on runoff potential.

impervious area - The portion of a land area, expressed in percent of total land area, that has a negligible infiltration rate. Impervious area can be natural, such as rock outcrop and the surface of permanent water bodies; or man-made, such as paved areas, roofs, and so forth.

incomplete record - A streamflow record in which some peak flows are missing because they were too low or high to measure, or the gage was out of operation for a short period because of flooding, instrument malfunction, or similar reason.

infiltration - The rate of movement, in millimeters per hour, of rainfall from the land surface into and through the surface soil.

initial abstraction - The accumulative loss, due to all mechanisms, of all rainfall from the start of rainfall to the point in time when surface runoff begins. This is equivalent to the initial loss (STRTL) in the IL+ULR method.

outlier - Outliers (extreme events) are data points which depart from the trend of the rest of data.

percolation - The rate of movement, in millimeters per hour, of water through the underlying soil or geologic strata subsequent to infiltration.

physiography - The physical geography of a watershed.

population - The entire (usually infinite) number of data from which a sample is taken or collected. The total number of past, present, and future floods at a location on a river is the population of floods for that location even if the floods are not measured or recorded. The frequency distribution of the population defines the underlying probability model from which the sample of annual floods arise.

rainfall excess - The equivalent uniform depth of runoff, in millimeters, that drains from the land surface. Rainfall excess equals rainfall minus rainfall losses.

rainfall losses - The sum of rainfall that is lost to surface runoff due to interception, depression storage, evaporation, infiltration, and other mechanisms. Rainfall loss is expressed as an equivalent uniform depth, in millimeters.

reach - A relatively short length of channel or watercourse.

record length - The number of years of record.

return period - The average number of years between occurrences of a hydrological event of a given or greater magnitude. In an annual flood series, the average number of years in which a flood of a given size is exceeded as an annual maximum.

routing - A procedure by which an inflow hydrograph is modified by the effects of flow resistance and storage to simulate an outflow hydrograph from the system.

soil - The layer of inorganic particulate matter covering the earth's surface. It can and does contain organic matter and often supports vegetation. For the purpose of estimating rainfall losses, only the upper horizon (generally about the top 150 millimeters of soil) will be considered. Underlying soil horizons or other strata will generally not affect rainfall losses in Arizona for storms of 100 year magnitude or less.

soil texture - The classification of soil into groups according to percentage of sand, silt, and clay, as used by the U.S. Department of Agriculture (Figure 3-1).

sand - Soil composed of particles in the 0.05 millimeters to 2.0 millimeters size range.

silt - Soil composed of particles in the 0.002 millimeters to 0.05 millimeters size range.

clay - Soil composed of particles smaller than 0.002 millimeters.

stationarity - The statistical properties of the annual flood series do not change with time.

storage coefficient - A Clark unit hydrograph parameter that relates the effects of direct runoff storage on the watershed to unit hydrograph shape.

subarea - A portion of a drainage area or subbasin that is delineated according to a physical feature such as soil texture or land-use.

subbasin - A portion of a drainage area that is determined according to the internal surface drainage pattern. A drainage area can often be divided into subbasins for modeling purposes.

surface retention loss - The depth of rainfall loss, in millimeters, due to all factors other than infiltration.

systematic record - Data from a stream gaging station for which flood discharges are systematically observed and recorded.

time of concentration - The travel time, during the corresponding period of most intense rainfall excess, for a floodwave to travel from the hydraulically most distant point in the watershed to the point of interest (concentration point).

topography - The surface features of a watershed.

unit hydrograph - The hydrograph of 10 millimeters of direct runoff from a storm of a specified duration for a particular watershed.

vegetation cover - The percentage 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.

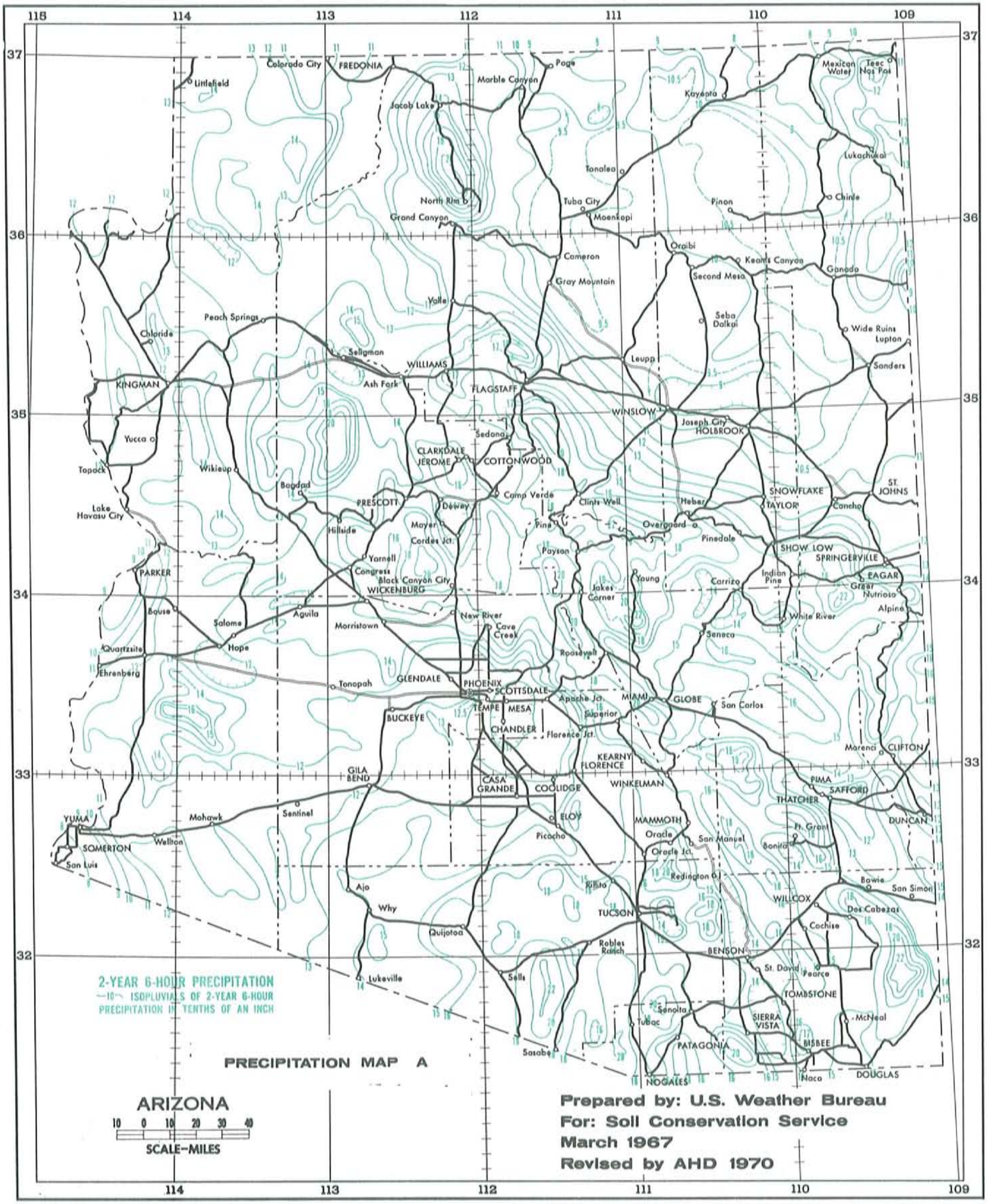
water year - The water accounting year; in the U.S., from 1 October through 30 September. The year specified is the calendar year for January of the period.

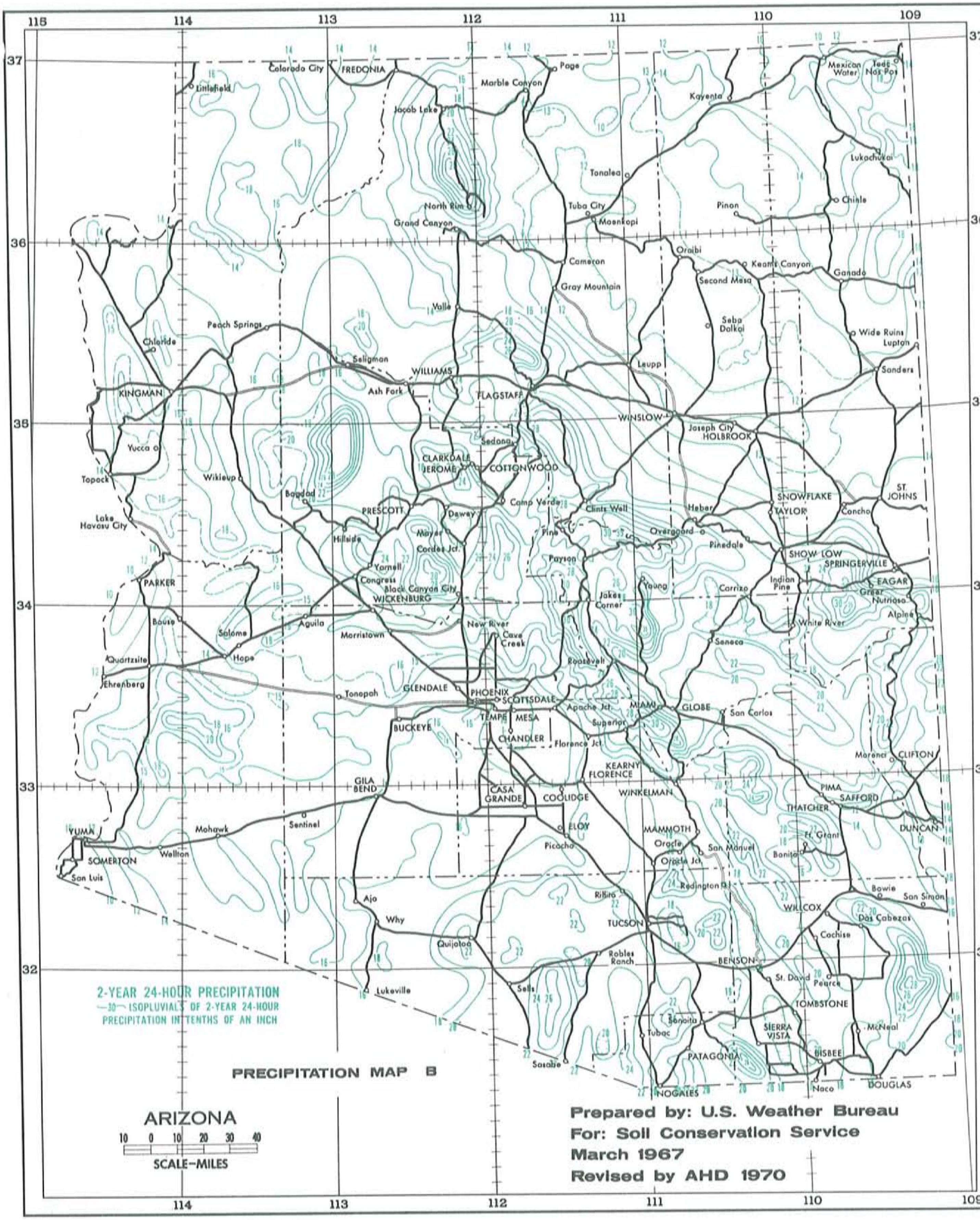
watercourse - An overland flow path that is defined by topography; such as a river, stream, channel, ditch, wash, swale, etc.

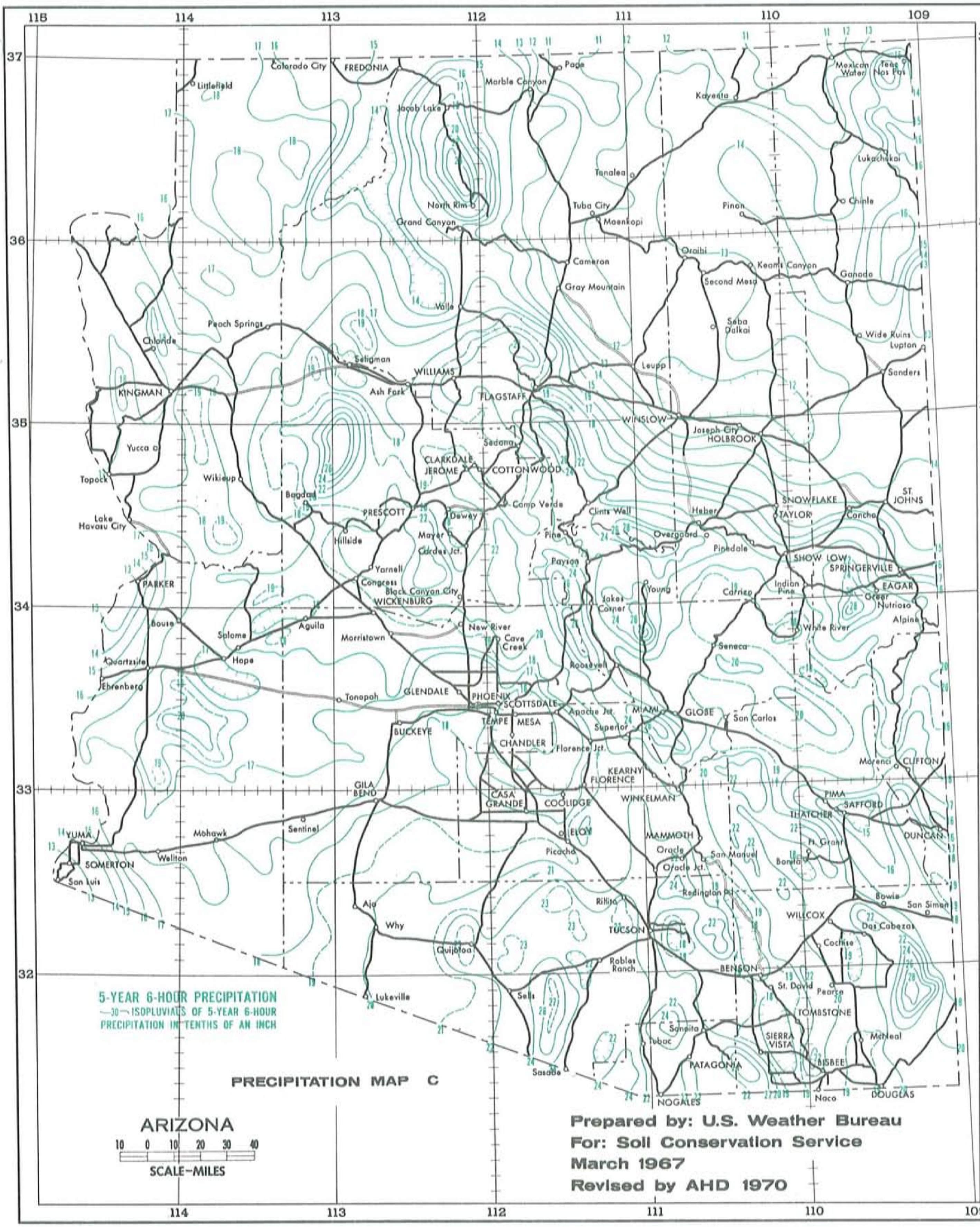
watershed - The area within definable boundaries where all direct runoff drains to a common outlet.

APPENDIX B

NOAA ATLAS 2 ISOPLUVIAL MAPS


2-YEAR 6-HOUR PRECIPITATION
—10— ISOPLUVIALS OF 2-YEAR 6-HOUR PRECIPITATION IN TENTHS OF AN INCH
PRECIPITATION MAP A
ARIZONA
SCALE—MILES
Prepared by: U.S. Weather Bureau
For: Soil Conservation Service
March 1967
Revised by AHD 1970
Littlefield
Colorado City
FREDONIA
Page
Marble Canyon
Jacob Lake
Kayenta
Mexican Water
Teec Nos Pos
Lukachukai
Tonalea
North Rim
Tuba City
Pinon
Chinle
Grand Canyon
Moenkopi
Oraibi
Keams Canyon
Cameron
Second Mesa
Ganado
Gray Mountain
Seba Dalkai
Valle
Peach Springs
Wide Ruins
Lupton
Chloride
Seligman
WILLIAMS
Leupp
Sanders
KINGMAN
Ash Fork
FLAGSTAFF
WINSLOW
Joseph City
HOLBROOK
Yucca
Sedona
CLARKDALE
JEROME
COTTONWOOD
Topock
Wikieup
Bagdad
Camp Verde
SNOWFLAKE
ST. JOHNS
Clints Well
Heber
TAYLOR
Concho
Lake Havasu City
PRESCOTT
Dewey
Mayer
Pine
Overgaard
Pinedale
Hillside
Cordes Jct.
Yarnell
SHOW LOW
SPRINGERVILLE
EAGAR
PARKER
Congress
Payson
Black Canyon City
WICKENBURG
Young
Jakes Corner
Carrizo
Indian Pine
Greer
Nutrioso
Alpine
Bouse
Aguila
New River
White River
Salome
Morristown
Cave Creek
Seneca
Quartzsite
Hope
Roosevelt
Ehrenberg
Tonopah
GLENDALE
PHOENIX
SCOTTSDALE
Apache Jct.
MIAMI
GLOBE
San Carlos
TEMPE
MESA
Superior
BUCKEYE
CHANDLER
Florence Jct.
Morenci
CLIFTON
KEARNY
FLORENCE
GILA BEND
CASA GRANDE
COOLIDGE
WINKELMAN
PIMA
SAFFORD
THATCHER
Sentinel
YUMA
Mohawk
ELOY
MAMMOTH
DUNCAN
Ft. Grant
Oracle
San Manuel
Picacho
Bonita
SOMERTON
Wellton
Oracle Jct.
San Luis
Redington
Bowie
San Simon
Ajo
Rillito
WILLCOX
Dos Cabezas
Why
TUCSON
Cochise
Quijotoa
Robles Ranch
BENSON
St. David
Pearce
Lukeville
Sells
TOMBSTONE
Sonoita
McNeal
SIERRA VISTA
Tubac
PATAGONIA
BISBEE
Sasabe
NOGALES
Naco
DOUGLAS


2-YEAR 24-HOUR PRECIPITATION
—30— ISOPLUVIALS OF 2-YEAR 24-HOUR PRECIPITATION IN TENTHS OF AN INCH
PRECIPITATION MAP B
ARIZONA
10 0 10 20 30 40
SCALE–MILES
Prepared by: U.S. Weather Bureau
For: Soil Conservation Service
March 1967
Revised by AHD 1970


5-YEAR 6-HOUR PRECIPITATION
—30— ISOPLUVIALS OF 5-YEAR 6-HOUR PRECIPITATION IN TENTHS OF AN INCH
PRECIPITATION MAP C
ARIZONA
10 0 10 20 30 40
SCALE–MILES
Prepared by: U.S. Weather Bureau
For: Soil Conservation Service
March 1967
Revised by AHD 1970
115
114
113
112
111
110
109
37
36
35
34
33
32
Littlefield
Colorado City
FREDONIA
Page
Marble Canyon
Jacob Lake
Kayenta
Mexican Water
Teec Nos Pos
Lukachukai
Chinle
Tonalea
North Rim
Tuba City
Moenkopi
Grand Canyon
Pinon
Oraibi
Keams Canyon
Second Mesa
Ganado
Cameron
Gray Mountain
Seba Dalkai
Valle
Peach Springs
Chloride
Wide Ruins
Lupton
Seligman
WILLIAMS
Leupp
Sanders
KINGMAN
Ash Fork
FLAGSTAFF
WINSLOW
Joseph City
HOLBROOK
Sedona
Yucca
CLARKDALE
JEROME
COTTONWOOD
Topock
Wikieup
Bagdad
Camp Verde
PRESCOTT
Dewey
Clints Well
Heber
SNOWFLAKE
TAYLOR
ST. JOHNS
Concho
Lake Havasu City
Hillside
Mayer
Cordes Jct.
Pine
Overgaard
Pinedale
Payson
SHOW LOW
SPRINGERVILLE
Yarnell
Congress
Black Canyon City
PARKER
WICKENBURG
Young
Carrizo
Indian Pine
EAGAR
Greer
Nutrioso
Jakes Corner
Alpine
Bouse
Aguila
New River
White River
Salome
Morristown
Cave Creek
Seneca
Quartzsite
Hope
Roosevelt
Ehrenberg
Tonopah
GLENDALE
PHOENIX
SCOTTSDALE
Apache Jct.
MIAMI
GLOBE
San Carlos
TEMPE
MESA
Superior
BUCKEYE
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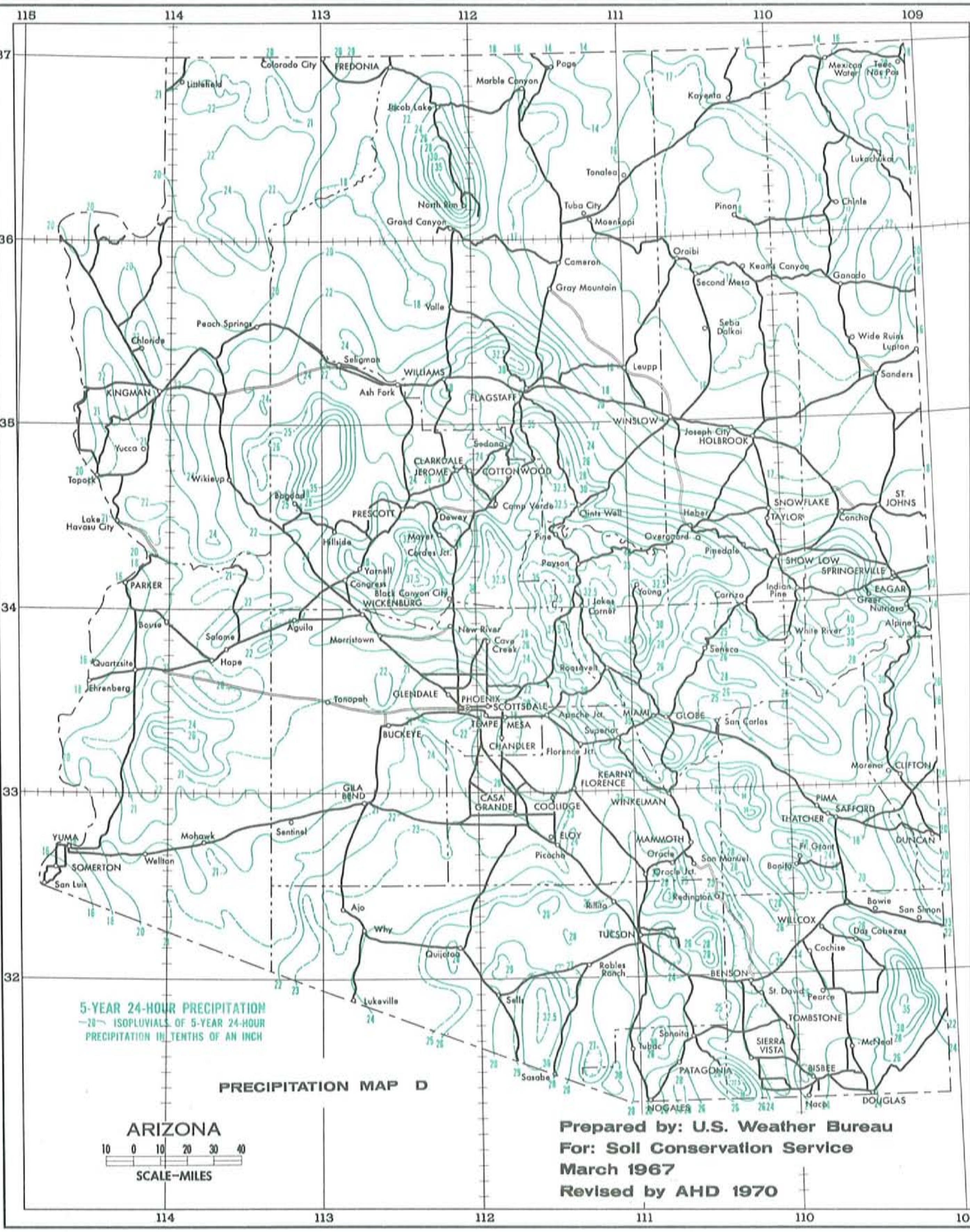

5-YEAR 24-HOUR PRECIPITATION
—20— ISOPLUVIALS OF 5-YEAR 24-HOUR PRECIPITATION IN TENTHS OF AN INCH
PRECIPITATION MAP D
ARIZONA
10 0 10 20 30 40
SCALE–MILES
Prepared by: U.S. Weather Bureau
For: Soil Conservation Service
March 1967
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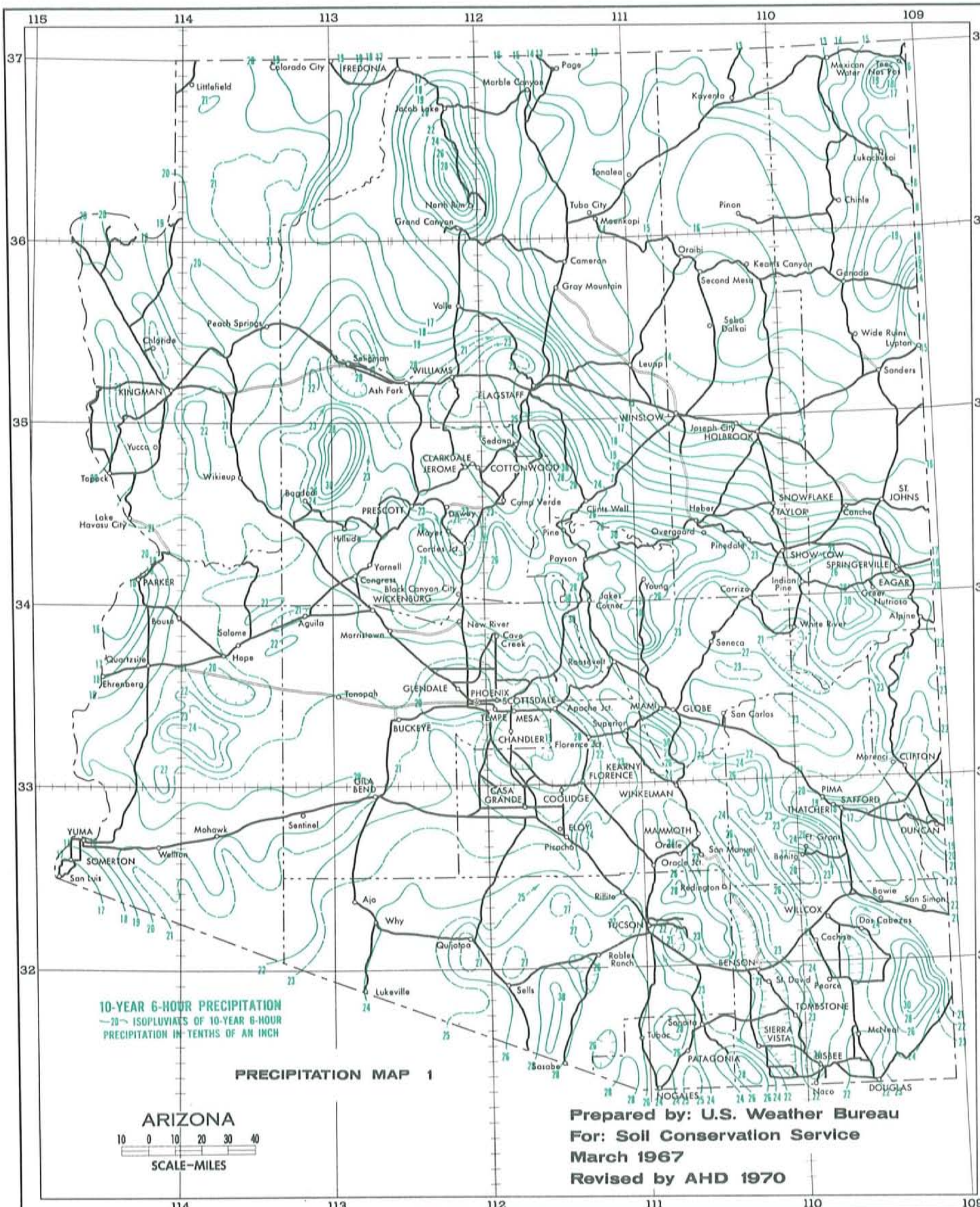
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—20— ISOPLUVIALS OF 10-YEAR 6-HOUR PRECIPITATION IN TENTHS OF AN INCH
PRECIPITATION MAP 1
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10 0 10 20 30 40
SCALE–MILES
Prepared by: U.S. Weather Bureau
For: Soil Conservation Service
March 1967
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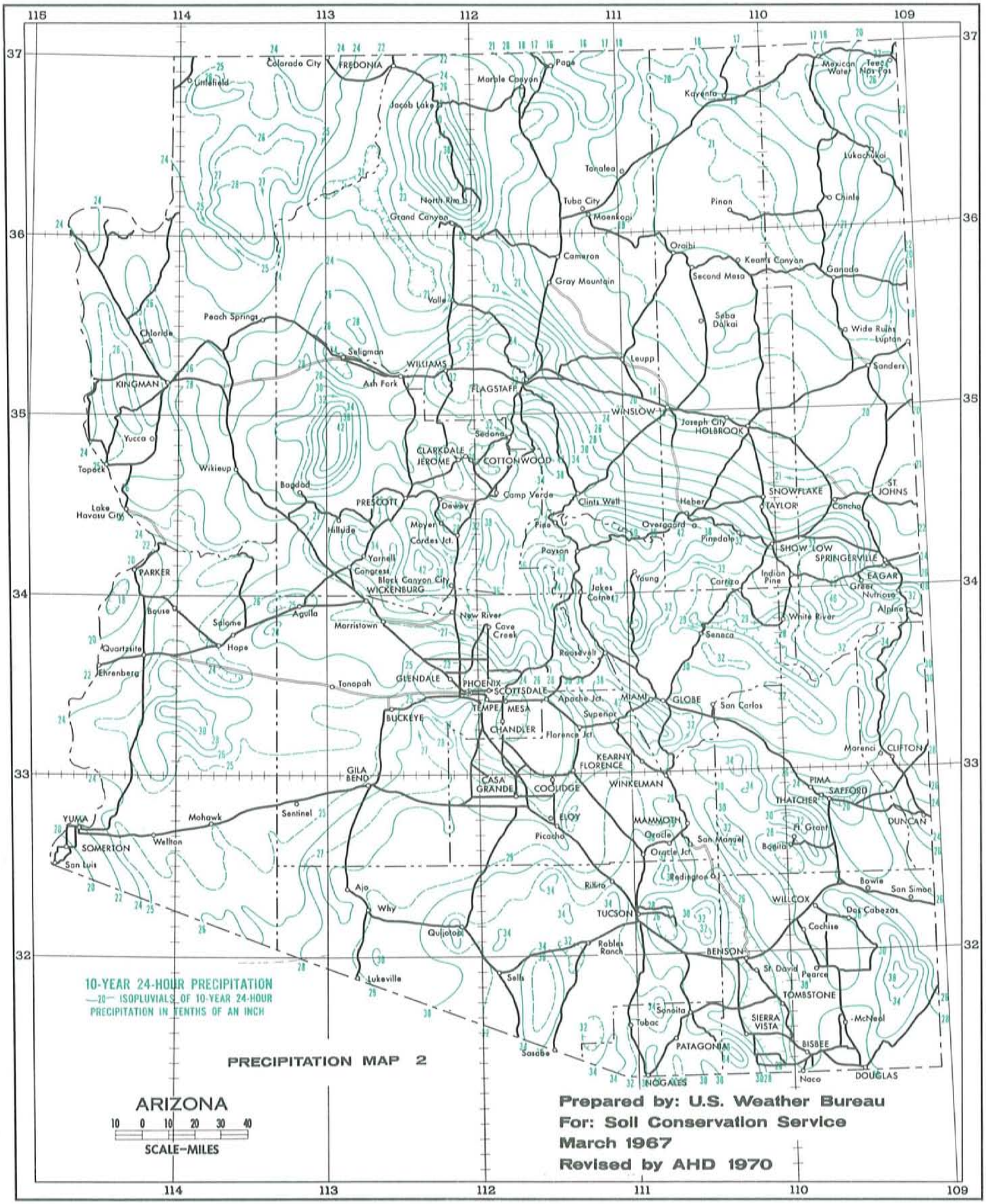
10-YEAR 24-HOUR PRECIPITATION
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SCALE-MILES
Prepared by: U.S. Weather Bureau
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March 1967
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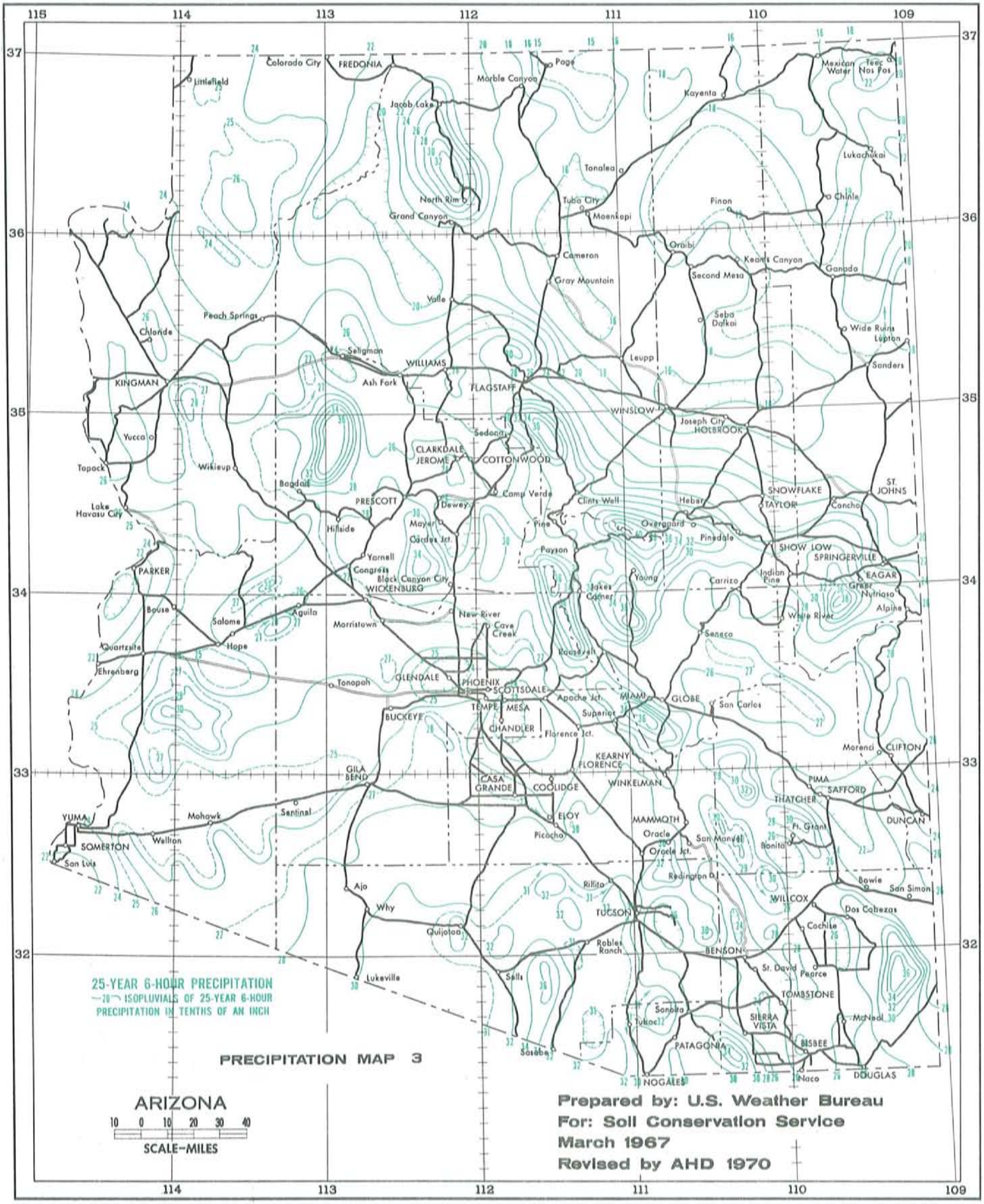
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—20— ISOPLUVIALS OF 25-YEAR 6-HOUR PRECIPITATION IN TENTHS OF AN INCH
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SCALE—MILES
Prepared by: U.S. Weather Bureau
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March 1967
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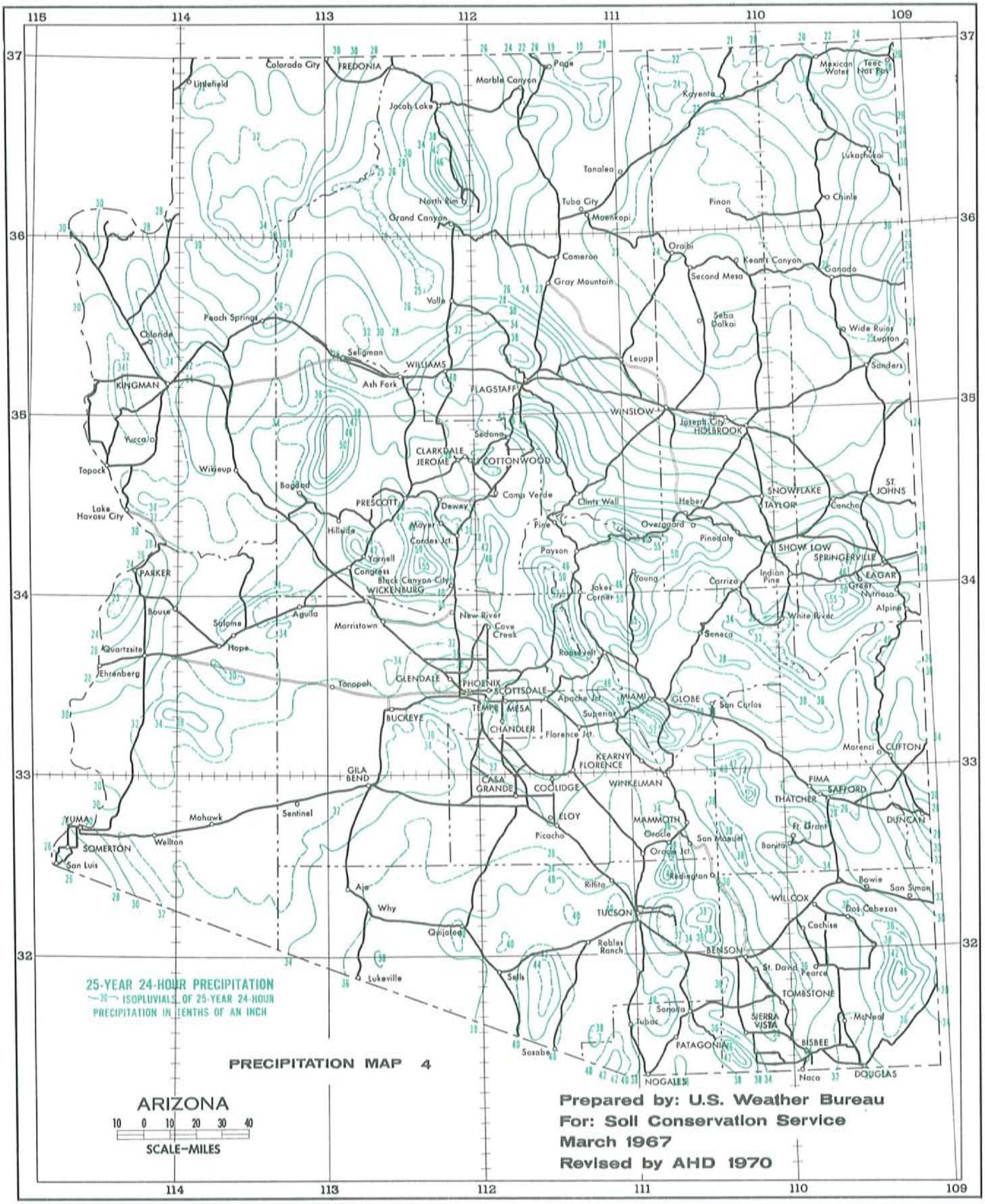
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—30— ISOPLUVIALS OF 25-YEAR 24-HOUR PRECIPITATION IN TENTHS OF AN INCH
PRECIPITATION MAP 4
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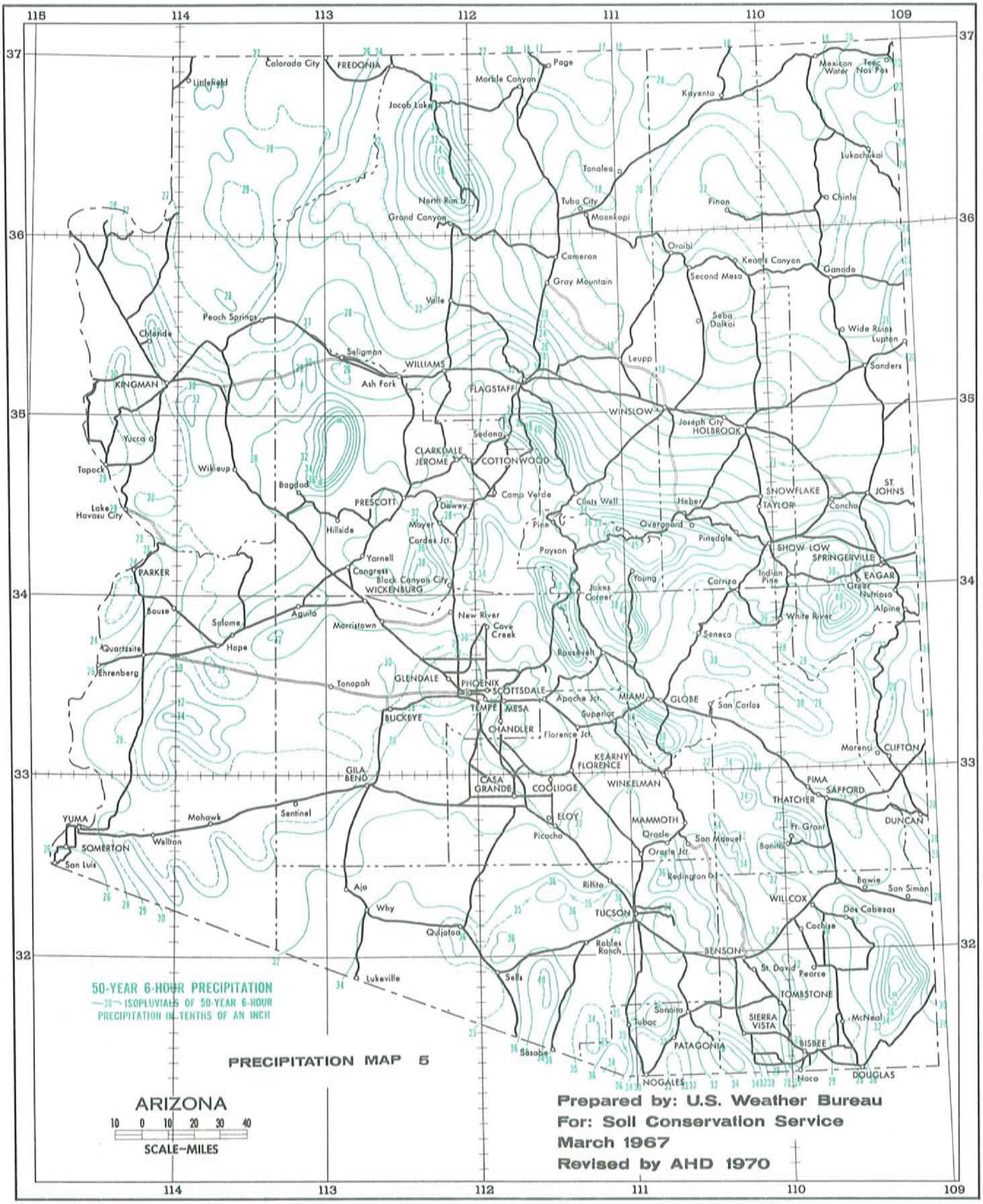
10 0 10 20 30 40
SCALE—MILES
Prepared by: U.S. Weather Bureau
For: Soil Conservation Service
March 1967
Revised by AHD 1970


50-YEAR 6-HOUR PRECIPITATION
—20— ISOPLUVIALS OF 50-YEAR 6-HOUR PRECIPITATION IN TENTHS OF AN INCH
PRECIPITATION MAP 5
ARIZONA
10 0 10 20 30 40
SCALE–MILES
Prepared by: U.S. Weather Bureau
For: Soil Conservation Service
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50-YEAR 24-HOUR PRECIPITATION
—30— ISOPLUVIALS OF 50-YEAR 24-HOUR PRECIPITATION IN TENTHS OF AN INCH
PRECIPITATION MAP 6
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10 0 10 20 30 40
SCALE–MILES
Prepared by: U.S. Weather Bureau
For: Soil Conservation Service
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Revised by AHD 1970
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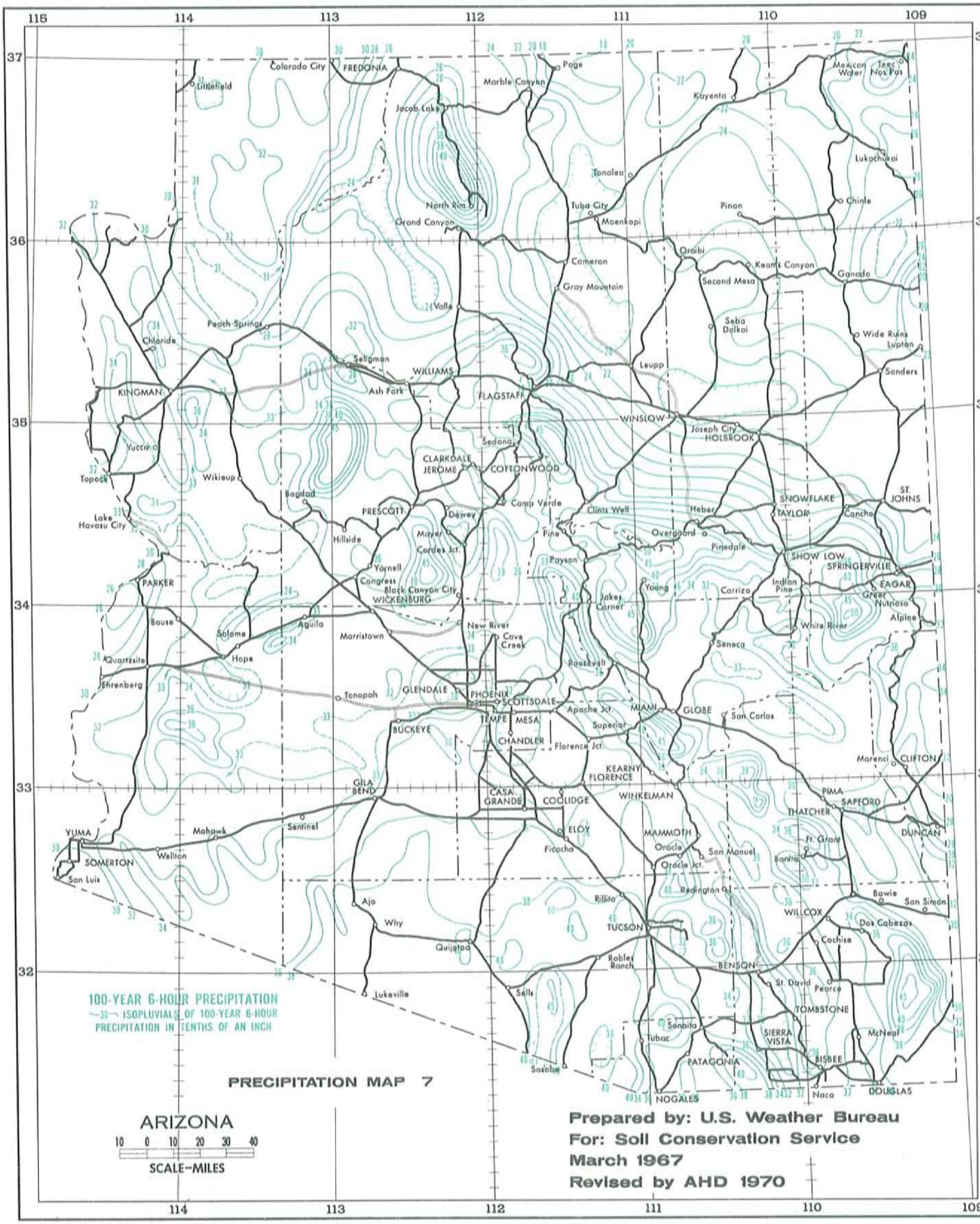
100-YEAR 6-HOUR PRECIPITATION
—30— ISOPLUVIALS OF 100-YEAR 6-HOUR PRECIPITATION IN TENTHS OF AN INCH
PRECIPITATION MAP 7
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10 0 10 20 30 40
SCALE-MILES
Prepared by: U.S. Weather Bureau
For: Soil Conservation Service
March 1967
Revised by AHD 1970
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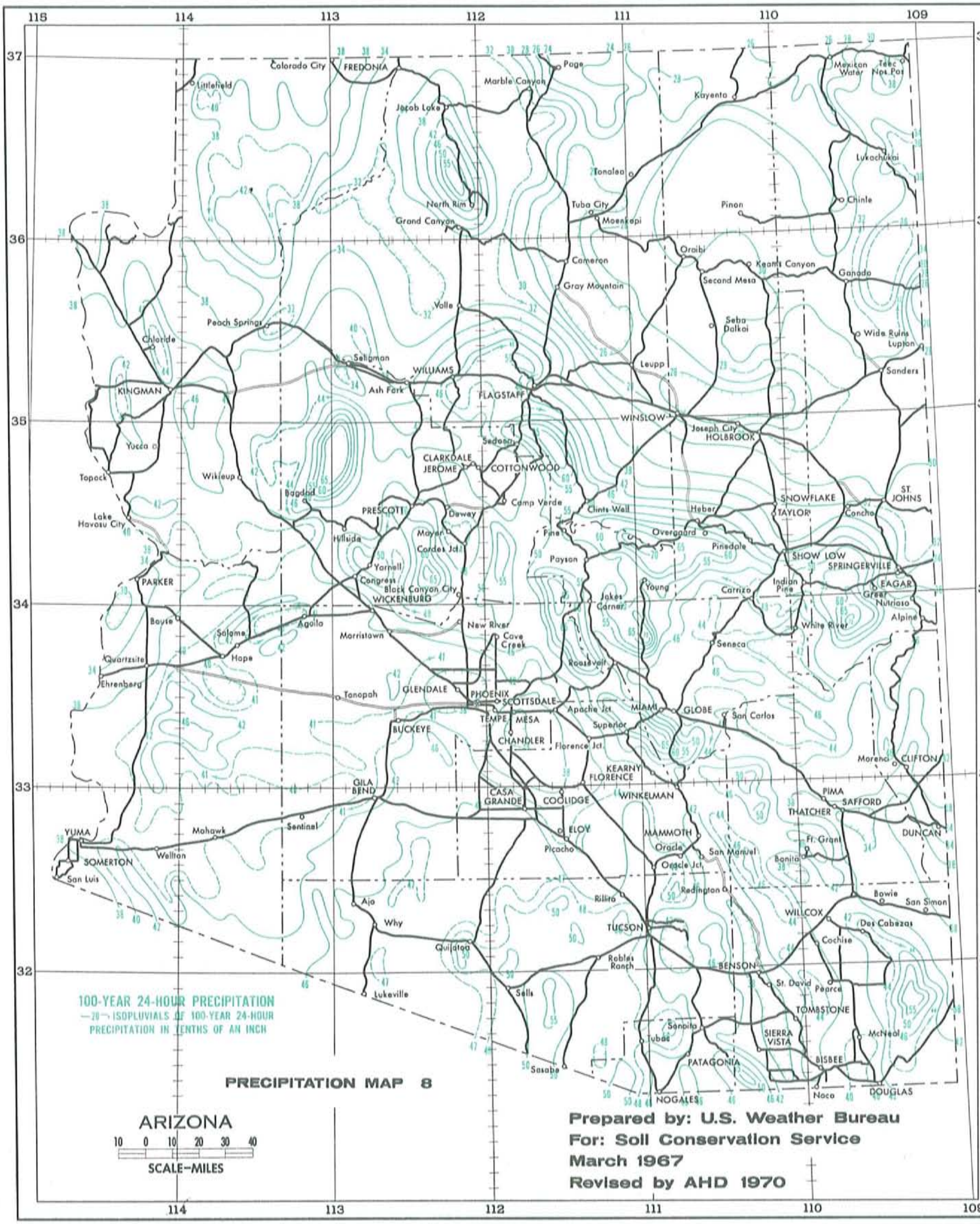
100-YEAR 24-HOUR PRECIPITATION
—20— ISOPLUVIALS OF 100-YEAR 24-HOUR PRECIPITATION IN TENTHS OF AN INCH
PRECIPITATION MAP 8
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SCALE-MILES
Prepared by: U.S. Weather Bureau
For: Soil Conservation Service
March 1967
Revised by AHD 1970
Colorado City
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APPENDIX C

ESTIMATION OF VEGETATION COVER

ESTIMATION OF VEGETATION COVER

An estimate of percent vegetation cover is needed when selecting the Rational Method runoff coefficient (C) from **Figures 2-4** through **2-8**, and for adjusting the XKSAT value with the Green and Ampt infiltration equation (**Figure 3-2**). The following information is provided to assist in the estimation of percent vegetation cover.

1. The percent vegetation cover is the percent of the land surface that is covered by vegetation. Vegetation cover is evaluated on plant basal area for grasses and forbs (broad leaf plants that are generally called flowers and weeds), and on canopy cover for trees and shrubs. Vegetation litter, if significant, should be considered as vegetation cover.

2. Vegetation types in Arizona, that basically affect the runoff process, are often divided into the following groups:

   Desert Brush: includes such plants a mesquite, creosote bush, black bush, catclaw, cactus, etc. - desert brush is typical of lower elevations and low annual rainfall.

   Herbaceous: includes short desert grasses with some brush, herbaceous is typical of intermediate elevations and higher annual rainfall than desert areas.

   Mountain Brush: mountain brush mixtures of oak, aspen, mountain mahogany, manzanita, bitter brush, maple, etc. - mountain brush is typical of intermediate elevations and generally higher annual rainfall than herbaceous areas.

   Juniper-Grass: juniper areas mixed with varying grass cover that is generally heavier than desert grasses due to higher annual precipitation - typical of higher elevations.

   Ponderosa Pine: ponderosa pine forests typical of high elevations and high annual precipitation - found along the Mogollon Rim, the Kaibab Plateau, the White Mountains, etc.

3. If one-half or more of the drainage area has a given vegetation type consider all the drainage area as having that vegetation type. If the vegetative type appears about equally

divided among all types of hydrologic cover, consider it all as herbaceous as this results in average values.

4. The Soil Conservation Service determines vegetation cover density by field surveys of carefully selected locations within the drainage area. However, for highway drainage design where runoff from numerous small drainage areas is to be determined, an approximation of the vegetative cover based on visual observation will be adequate.

   Three broad ranges of vegetative cover density have been established.

Poor	0 - 20% vegetative cover
Fair	20% - 40% vegetative cover
Good	40% + vegetative cover

   Some representative values for vegetative cover densities have been determined and are shown in the following photographs:

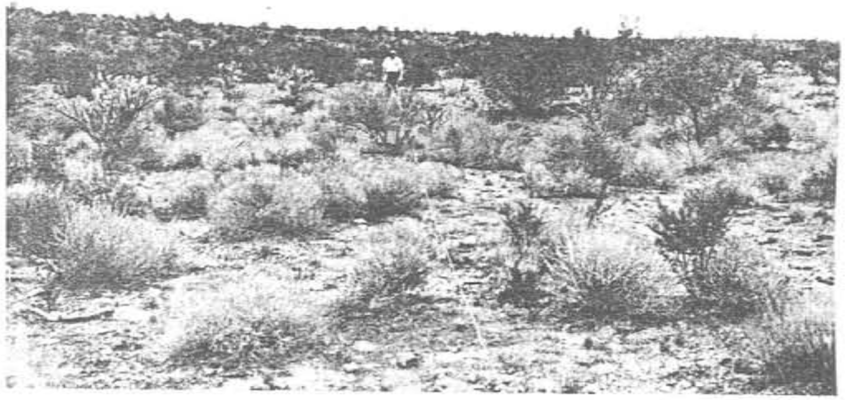


Photo 1

Location:	Highway 89 near Congress
Vegetation Type:	Desert Brush
Cover Density:	10%, Poor
Soil Group:	C

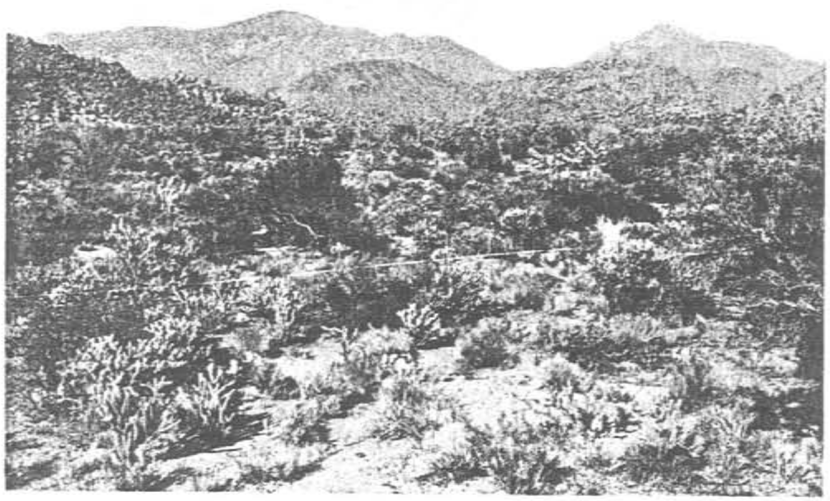
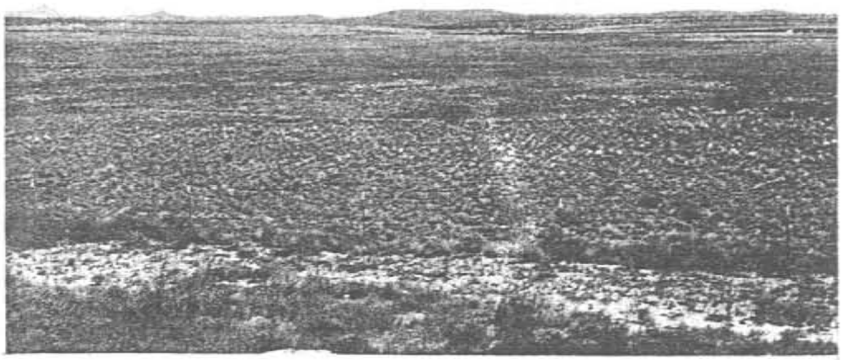


Photo 2

Location:	Hualapai Mtns. near Yucca
Vegetation Type:	Desert Brush
Cover Density:	30%, Fair
Soil Group:	D

NOTE: Vegetative cover density greater than 40% for desert brush is not found in Arizona.

#3



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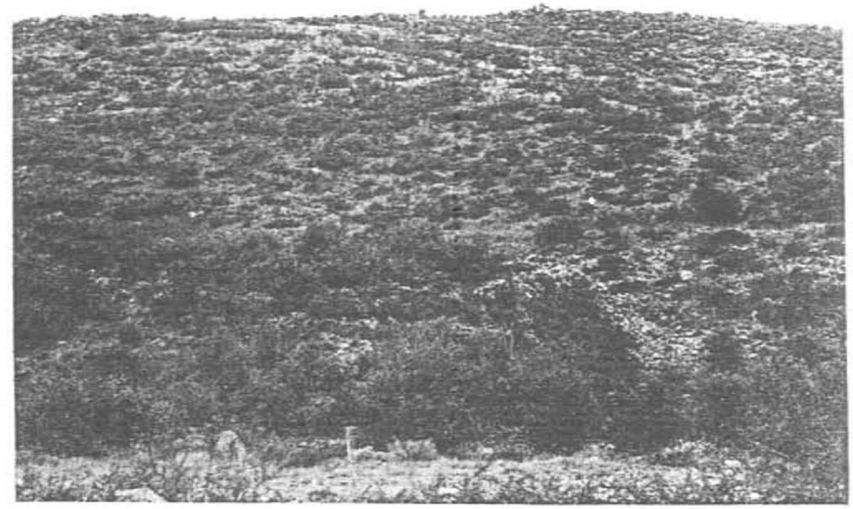

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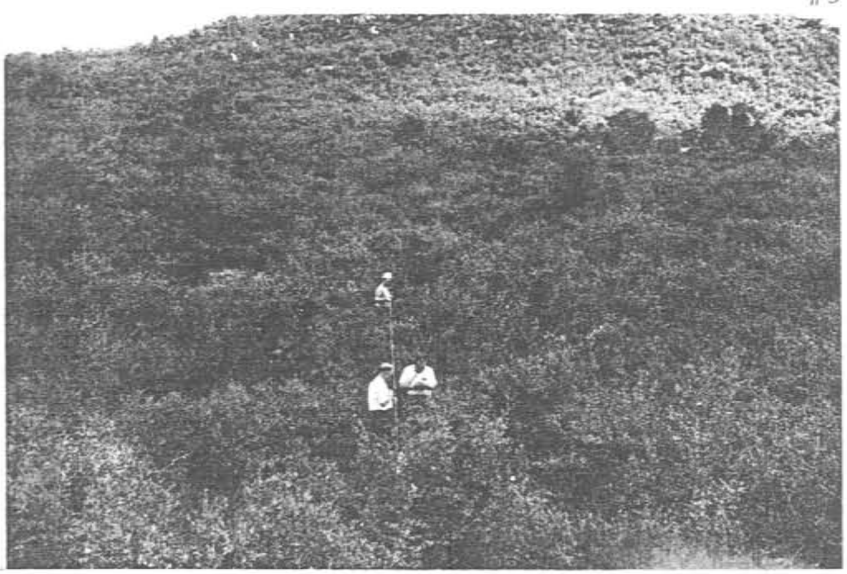


Photo 3	Location:	I-40 near Seligman
	Vegetation Type:	Herbaceous
	Cover Density:	15%, Poor
	Soil Group:	C
Photo 4	Location:	County Road near Wagoner
	Vegetation Type:	Mountain Brush
	Cover Density:	24%, Fair
	Soil Group:	D
Photo 5	Location:	Highway 89 near Wilhoit
	Vegetation Type:	Mountain Brush
	Cover Density:	75%, Good
	Soil Group:	D

Photo 6

Location:	I-40 near Ashfork
Vegetation Type:	Juniper-Grass
Cover Density:	29%, Fair
Soil Group:	C

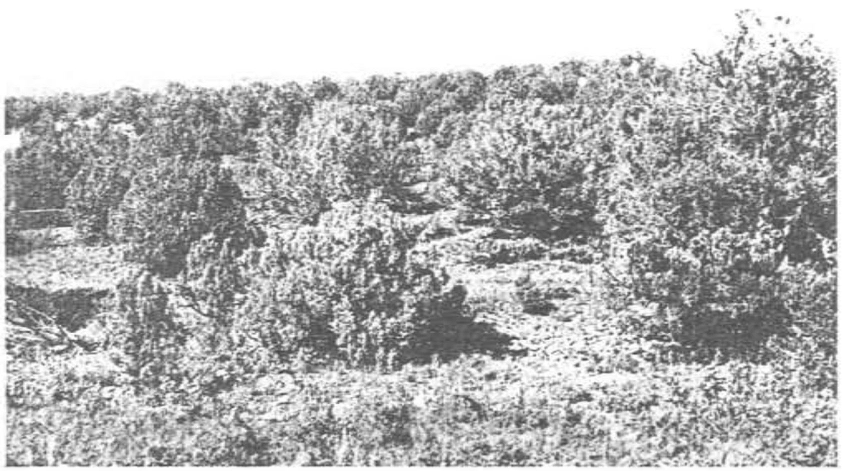


Photo 7

Location:	I-17 near Stoneman Lake
Vegetation Type:	Juniper-Grass
Cover Density:	63%, Good
Soil Group:	B

The vegetative cover densities shown in Photos 1-7 have been determined in the following manner:

1) An area representing the typical vegetative cover density for the drainage area is selected.

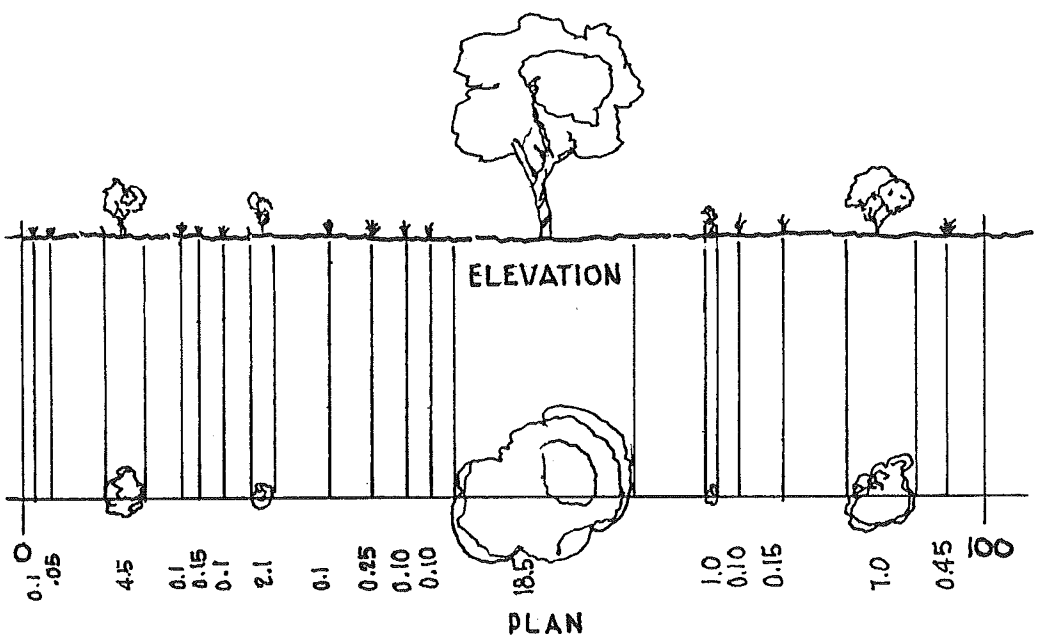
2) A 100 foot chain is stretched out between two posts, approximately 3 ft. above ground level.

3) The intercepts of the vegetative cover along the 100 ft. length are noted.

4) The total distances covered by vegetation and litter along the 100 ft. length are summed up and represent the percent of vegetative cover for the selected area.

5) Several determinations may have to be made to compute the average percent of cover for the drainage area.

The following sketch illustrates the field procedure:



Vegetative
Cover = .1+.05+4.5+.1+.15+.1+2.1+.1+.25+.1+.1+18.5+1.0+.1+.15+7.0+.45
Density = 34.85%

APPENDIX D

WORKSHEETS AND PROBABILITY PAPER

ARIZONA DEPARTMENT OF TRANSPORTATION
HYDROLOGIC DESIGN DATA

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

RAINFALL DEPTH-DURATION-FREQUENCY (D-D-F) WORKSHEET

PART A

Determine rainfall depths from the isopluvial maps and convert inches to millimeters:

	Rainfall depth, in inches	Rainfall depth, in millimeters
2-year, 6-hour	$P_{2,6'}$ = ________	$P_{2,6'}$ = ________
2-year, 24-hour	$P_{2,24'}$ = ________	$P_{2,24'}$ = ________
100-year, 6-hour	$P_{100,6'}$ = ________	$P_{100,6'}$ = ________
100-year, 24-hour	$P_{100,24'}$ = ________	$P_{100,24'}$ = ________

Note: $P_{T,t}$ in millimeters = (25.4) $P_{T,t}$ in inches

PART B

Compute the following:

2-year, 1-hour	$-0.279 + \dfrac{.942\,(P_{2,6'})^2}{(P_{2,24'})} = -0.279 + \dfrac{.942\,(\quad)^2}{(\quad)}$	$P_{2,1'}$ = ______
100-year, 1-hour	$12.55 + \dfrac{.755\,(P_{100,6'})^2}{(P_{100,24'})} = 12.55 + \dfrac{.755\,(\quad)^2}{(\quad)}$	$P_{100,1'}$ = ______
2-year, 2-hour	$.341(P_{2,6'}) + .659(P_{2,1'}) = .341(\quad) + .659(\quad)$	$P_{2,2'}$ = ______
2-year, 3-hour	$.569(P_{2,6'}) + .431(P_{2,1'}) = .569(\quad) + .431(\quad)$	$P_{2,3'}$ = ______
2-year, 12-hour	$.500(P_{2,6'}) + .500(P_{2,24'}) = .500(\quad) + .500(\quad)$	$P_{2,12'}$ = ______
100-year, 2-hour	$.341(P_{100,6'}) + .659(P_{100,1'}) = .341(\quad) + .659(\quad)$	$P_{100,2'}$ = ______
100-year, 3-hour	$.569(P_{100,6'}) + .431(P_{100,1'}) = .569(\quad) + .431(\quad)$	$P_{100,3'}$ = ______
100-year, 12-hour	$.500(P_{100,6'}) + .500(P_{100,24'}) = .500(\quad) + .500(\quad)$	$P_{100,12'}$ = ______

Note: 5" denotes 5 minutes, etc.; 1' denotes 1 hour, etc.

RAINFALL DEPTH-DURATION-FREQUENCY (D-D-F) WORKSHEET
(Continued)

PART C

Determine the short-duration rainfall zone:

Zone = ________

Determine the short-duration rainfall ratios:

Duration (minutes)	Ratio	
	2-Year	100-Year
5	A = ______	E = ______
10	B = ______	F = ______
15	C = ______	G = ______
30	D = ______	H = ______

Compute the following:

2-year, 5-min	(A) ($P_{2,1'}$) = ( ) ( )	$P_{2,5"}$ = ______
2-year, 10-min	(B) ($P_{2,1'}$) = ( ) ( )	$P_{2,10"}$ = ______
2-year, 15-min	(C) ($P_{2,1'}$) = ( ) ( )	$P_{2,15"}$ = ______
2-year, 30-min	(D) ($P_{2,1'}$) = ( ) ( )	$P_{2,30"}$ = ______
100-year, 5-min	(E) ($P_{100,1'}$) = ( ) ( )	$P_{100,5"}$ = ______
100-year, 10-min	(F) ($P_{100,1'}$) = ( ) ( )	$P_{100,10"}$ = ______
100-year, 15-min	(G) ($P_{100,1'}$) = ( ) ( )	$P_{100,15"}$ = ______
100-year, 30-min	(H) ($P_{100,1'}$) = ( ) ( )	$P_{100,30"}$ = ______

Note: 5" denotes 5 minutes, etc.; 1' denotes 1 hour, etc.

RAINFALL DEPTH-DURATION-FREQUENCY (D-D-F) WORKSHEET
(Continued)

PART D

For any flood frequency (T-yr) other than 2-year or 100-year, calculate the rainfall depth for each rainfall duration (*t*) by the following equation:

$$P_{T,t} = (X)(P_{2,t}) + (Y)(P_{100,t})$$

where X and Y for a selected frequency (T-yr) are:

Frequency (T-yr)	X	Y
5-year	.674	.278
10-year	.496	.449
25-year	.293	.669
50-year	.146	.835
500-year	-.337	1.381

Selected frequency (T-yr) = __________   X = __________   Y = __________

5-min	$(X)(P_{2,5''}) + (Y)(P_{100,5''})$ = ( )( ) + ( )( )	$P_{\_\_,5''}$ = ______
10-min	$(X)(P_{2,10''}) + (Y)(P_{100,10''})$ = ( )( ) + ( )( )	$P_{\_\_,10''}$ = ______
15-min	$(X)(P_{2,15''}) + (Y)(P_{100,15''})$ = ( )( ) + ( )( )	$P_{\_\_,15''}$ = ______
30-min	$(X)(P_{2,30''}) + (Y)(P_{100,30''})$ = ( )( ) + ( )( )	$P_{\_\_,30''}$ = ______
1-hour	$(X)(P_{2,1'}) + (Y)(P_{100,1'})$ = ( )( ) + ( )( )	$P_{\_\_,1'}$ = ______
2-hour	$(X)(P_{2,2'}) + (Y)(P_{100,2'})$ = ( )( ) + ( )( )	$P_{\_\_,2'}$ = ______
3-hour	$(X)(P_{2,3'}) + (Y)(P_{100,3'})$ = ( )( ) + ( )( )	$P_{\_\_,3'}$ = ______
6-hour	$(X)(P_{2,6'}) + (Y)(P_{100,6'})$ = ( )( ) + ( )( )	$P_{\_\_,6'}$ = ______
12-hour	$(X)(P_{2,12'}) + (Y)(P_{100,12'})$ = ( )( ) + ( )( )	$P_{\_\_,12'}$ = ______
24-hour	$(X)(P_{2,24'}) + (Y)(P_{100,24'})$ = ( )( ) + ( )( )	$P_{\_\_,24'}$ = ______

Note: 5" denotes 5 minutes, etc.; 1′ denotes 1 hour, etc.

RAINFALL DEPTH-DURATION-FREQUENCY (D-D-F) WORKSHEET
(Continued)

PART E

Tabulate the rainfall Depth-Duration-Frequency statistics below:

Duration	Rainfall depth, in millimeters — Frequency, in years						
	2	5	10	25	50	100	500
5-min.							
10-min.*							
15-min.							
30-min.*							
1-hour							
2-hour							
3-hour							
6-hour							
12-hour							
24-hour							

* - Note: 10-min. and 30-min. values are not coded into the PH record.
5" denotes 5 minutes, etc.; 1′ denotes 1 hour, etc.

ARIZONA DEPARTMENT OF TRANSPORTATION
HYDROLOGIC DESIGN DATA

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

RAINFALL INTENSITY-DURATION-FREQUENCY (I-D-F) WORKSHEET

Divide each rainfall depth from the D-D-F Worksheet by each corresponding duration, in hours, and tabulate below:

Duration	Rainfall intensity, in millimeters/hour — Frequency, in years						
	2	5	10	25	50	100	500
5-min.							
10-min.							
15-min.							
30-min.							
1-hour							
2-hour							
3-hour							
6-hour							
12-hour							
24-hour							

Note: 5" denotes 5 minutes, etc.; 1' denotes 1 hour, etc.

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. km.**
Period of Systematic Record ______________________

WATER YEAR (1)	ANNUAL PEAK DISCHARGE (cfs) (2)	ANNUAL PEAK DISCHARGE (m^3/s) (3)	DATE (4)	FLOOD TYPE [a] (5)	COMMENTS (6)

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

Note: (3) = 0.0283 * (2) for cfs to m^3/s conversion

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)	ANNUAL PEAK DISCHARGE (m^3/s) (3)	DATE (4)	FLOOD TYPE [a] (5)	COMMENTS (6)

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

Note: (3) = 0.0283 * (2) for cfs to m^3/s conversion

ARIZONA DEPARTMENT OF TRANSPORTATION
HYDROLOGIC DESIGN DATA

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

FLOOD FREQUENCY ANALYSIS
PLOTTING POSITION CALCULATION FORM

Page _1_ of ____

Gage Station Name ______________________
Gage Station No. ______________________ **Drainage Area** ____________**sq. km.**
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 (m^3/s) (1)	RANK (2)	PLOTTING POSITION	
		P_e (3)	T_r (4)

ARIZONA DEPARTMENT OF TRANSPORTATION
HYDROLOGIC DESIGN DATA

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

FLOOD FREQUENCY ANALYSIS
PLOTTING POSITION CALCULATION FORM

Page ____ of ____

FLOOD PEAK DISCHARGE (m^3/s) (1)	RANK (2)	PLOTTING POSITION	
		P_e (3)	T_r (4)

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.) = ____________%

$Q_{2\text{-yr}}$ = ____________ m^3/s

$Q_{100\text{-yr}}$ = ____________ m^3/s

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

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

N_c = ____________

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

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

T Years (1)	$U_{1-\frac{1}{T}}$ (2)	Y_T (a) (3)	S_T (b) (4)	Limits, in m^3/s (c) Upper (5)	 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) $S_T = \left[\left(\dfrac{S_{ln}^{\,2}}{N_c}\right)\left(1 + .5\, U^2_{1-\frac{1}{T}}\right)\right]^{\frac{1}{2}}$

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.) = __________ %

$Q_{2\text{-yr}}$ = __________ m^3/s

$Q_{100\text{-yr}}$ = __________ m^3/s

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

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

N_c = __________

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

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

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

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

T Years (1)	K (2)	Z (a) (3)	S_T (b) (4)	Q_T (c) (5)	Limits, in m^3/s (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) $Z = (1.0 + 1.1396\ K + 1.1K^2)^{\frac{1}{2}}$

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

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

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

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.) = __________%

$Q_{2\text{-yr}}$ = __________ m^3/s

$Q_{100\text{-yr}}$ = __________ m^3/s

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

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

N_c = __________

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

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

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

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

T Years (1)	K (2)	Z (a) (3)	S_T (b) (4)	Y_T (c) (5)	Limits, in m^3/s (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				

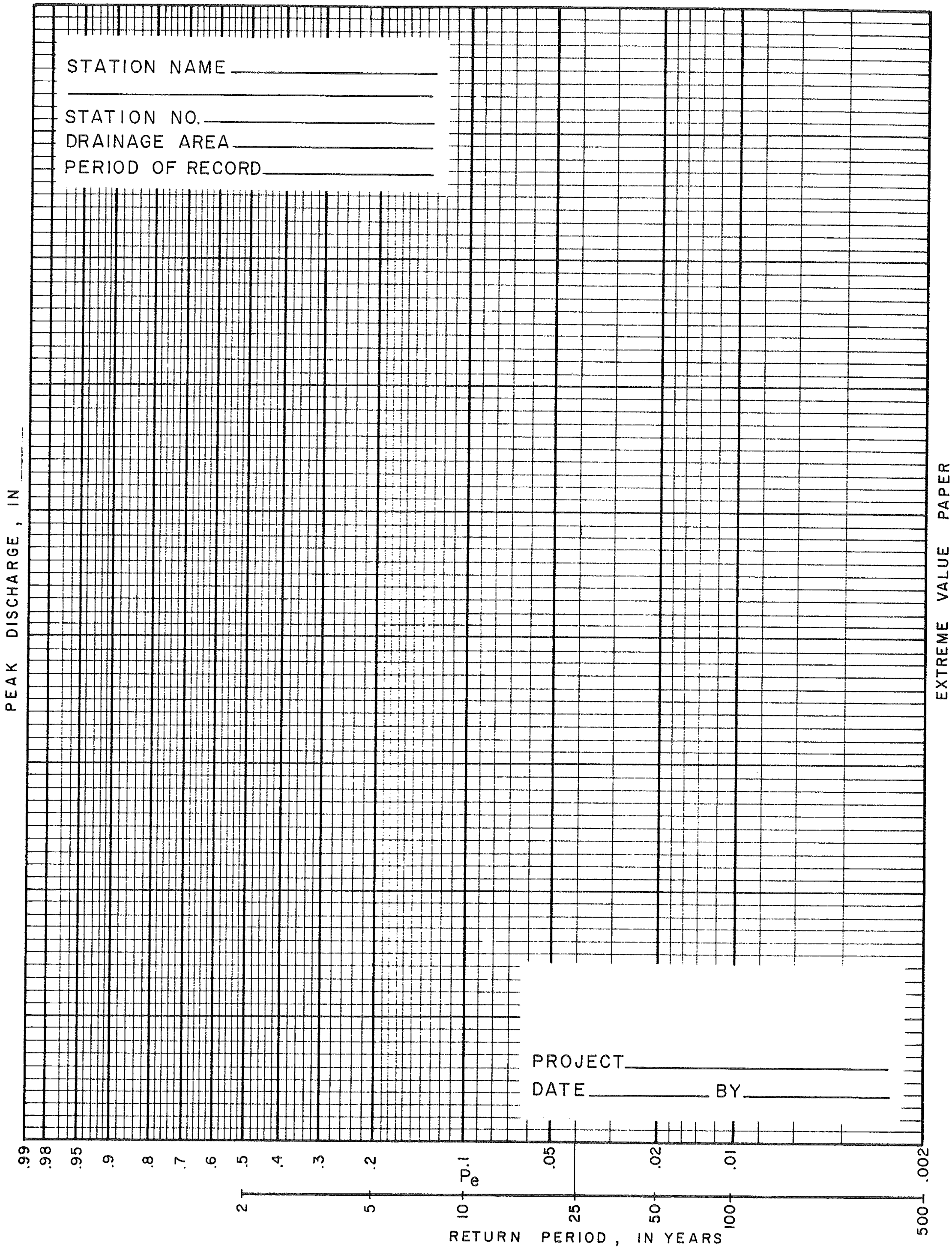
(a) $$Z = (1.0 + 1.1396\ K + 1.1K^2)^{\frac{1}{2}}$$

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

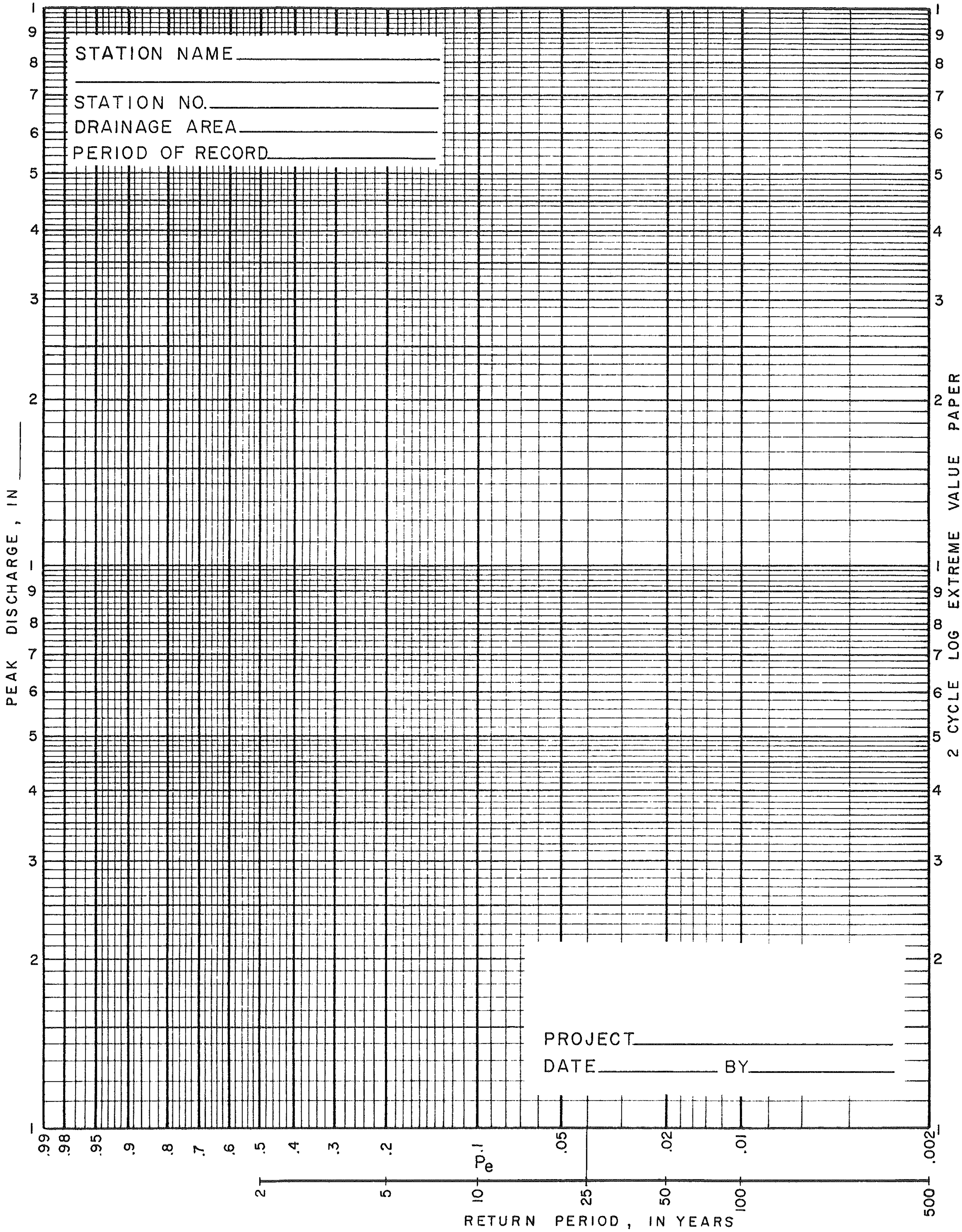
(c) $$Y_T = \overline{Y} + K\,S_{lev}$$

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

ARIZONA DEPARTMENT OF TRANSPORTATION
HYDROLOGIC DESIGN DATA



ARIZONA DEPARTMENT OF TRANSPORTATION
HYDROLOGIC DESIGN DATA



ARIZONA DEPARTMENT OF TRANSPORTATION
HYDROLOGIC DESIGN DATA

STATION NAME ______________________

____

STATION NO. ______________________

DRAINAGE AREA ______________________

PERIOD OF RECORD ______________________

PEAK DISCHARGE , IN ____

1 9 8 7 6 5 4 3 2 1 9 8 7 6 5 4 3 2 1

2 CYCLE LOG NORMAL PAPER

PROJECT ______________________

DATE ____________ BY ____________

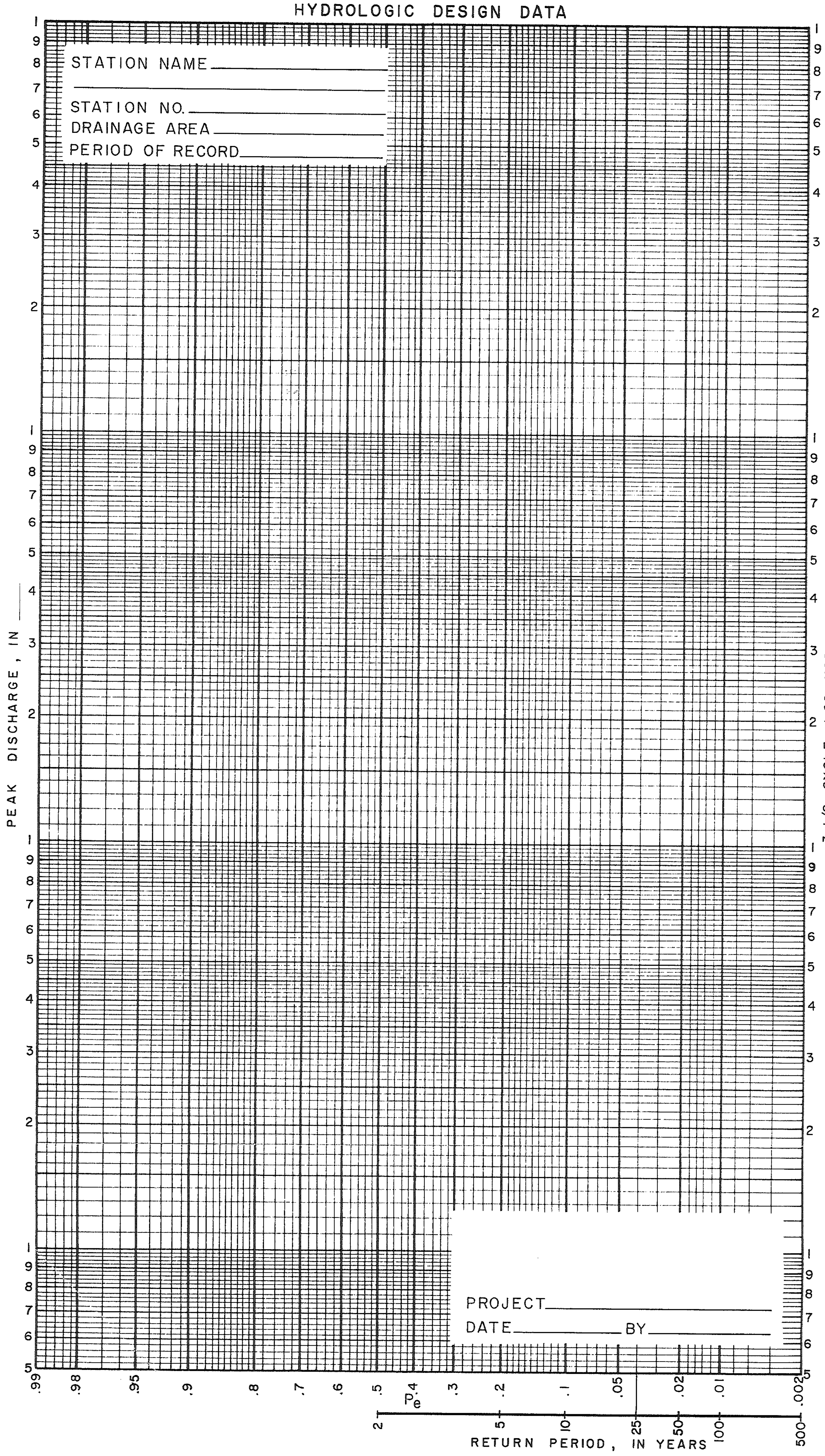
.99 .98 .95 .9 .8 .7 .6 .5 .4 .3 .2 .1 .05 .02 .01 .002

P_e

2 5 10 25 50 100 500

RETURN PERIOD, IN YEARS

ARIZONA DEPARTMENT OF TRANSPORTATION
HYDROLOGIC DESIGN DATA



ARIZONA DEPARTMENT OF TRANSPORTATION
HYDROLOGIC DESIGN DATA

STATION NAME ____________________
____
STATION NO. ____________________
DRAINAGE AREA ____________________
PERIOD OF RECORD ____________________

PEAK DISCHARGE , IN ____

3 1/2 CYCLE LOG EXTREME VALUE PAPER

1 9 8 7 6 5 4 3 2 1 9 8 7 6 5 4 3 2 1 9 8 7 6 5 4 3 2 1 9 8 7 6 5

PROJECT ____________________
DATE ____________ BY ____________

.99 .98 .95 .9 .8 .7 .6 .5 .4 .3 .2 .1 .05 .02 .01 .002

P_e

2 5 10 25 50 100 500

RETURN PERIOD , IN YEARS

APPENDIX E

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