

Kern County
Hydrology Manual

Prepared by T.V. Hromadka II, Ph.D.,
PH Professor,
California State University of Fullerton

Foreword and Acknowledgements

The County of Kern Hydrology Manual represents an update of hydrologic techniques over those identified for use in the Kern County Subdivision Standards, and the incorporation of the NOAA Atlas II Isohyetal Maps.

Revisions include (i) an updated rational method, developed to better estimate peak flow rates obtained from rainfall-runoff relationships, (ii) Unit Hydrograph procedures, including methods for low loss rate calculations, and depth-area curves, (iii) inclusions of hydrologic models for channel and pipeflow routing, and flow-by basin routing, (iv) extended design storms for detention basin testing, (v) link-node model guidelines, and (vi) desert hydrology criteria and parameter selection for the calculation of losses and the unit hydrograph.

Acknowledgements are made to the members of the Kern County Hydrology Manual Advisory Committee (alphabetically) Harry Afshar (representing the City of Bakersfield), David Fiddler (representing the Kern County Public Works Department), Robert Lusich (representing The California Counsel of Civil Engineers and Land Surveyors), Richard Meyer (representing The American Society of Civil Engineers), and Darrel Sorenson (representing The Kern County Water Agency).

Acknowledgements are also made to the Kern County Planning and Development Service's Staff, (alphabetically) Clark Farr (Engineer 3, Editor in Charge), Richard Lloyd (Engineering Technician, Graphics).

Additional acknowledgements are made to Glenda Hudson, of Hudson Writing Consultants, for her suggestions that have made this manual easier to use, and to Alejandro Gonzaga, Engineering Aide, for converting the original printed manual to digital form.

Special acknowledgements are made to Joe DeVries (Ph.D., RCE, University of California, Davis), Rick McCuen (Ph.D., University of Maryland), and Bob Whitley (Ph.D., University of California, Irvine) for their technical assistance.

TABLE OF CONTENTS

Section	Title	Page
A	**INTRODUCTION**	A-1
A.1.	About This Manual	A-1
A.2.	Goals	A-2
A.3.	Summary of Contents	A-2
B	**PRECIPITATION**	B-1
B.1.	Definitions	B-1
B.2.	Classification of Precipitation	B-2
B.3.	Mechanisms for Cooling	B-2
B.4.	Precipitation Depth-Duration-Frequency	B-2
B.5.	Measurement and Synthesis of Precipitation Data	B-3
B.6.	Point Precipitation	B-4
B.7.	Random Nature of Point-Precipitation Data	B-4
B.8.	Event Depth-Duration	B-4
B.9.	Intensity-Duration Curves	B-5
B.10.	Synthetic 24-Hour Critical Storm Pattern	B-6
B.11.	Longer Duration Rainfall Data	B-7
C	**LOSSES**	C-1
C.1.	Definitions	C-1
C.2.	Watershed Losses	C-1
C.3.	Hydrologic Soil Groups	C-2
C.3.1	Soil Maps	C-2
C.4.	Soil Cover and Hydrologic Conditions	C-3
C.5.	Watershed Development Conditions	C-3
C.6.	Antecedent Moisture Condition (AMC)	C-3

Section	Title	Page
C.7.	Estimation of Loss Rates	C-3
C.7.1.	Estimation of Initial Abstraction (Ia)	C-4
C.7.2.	Estimation of Storm Runoff Yield	C-4
C.7.3.	Low Loss Rate, F*	C-5
C.7.4.	Infiltration Rates	C-5
C.7.5.	Estimation of Catchment Maximum Loss Rates, F_m	C-6
C.7.6.	Design Storm Loss Rates	C-6
C.8.	Desert Hydrology Loss Rates	C-6
C.8.1.	Hydrologic Soil-Cover Complex	C-7
C.8.2.	Hydrologic Cover Types	C-7
C.8.3.	Hydrologic Cover Density	C-7
D	**RATIONAL METHOD**	D-1
D.1	Definitions	D-1
D.2.	Rational Method Equation	D-2
D.3.	Limitations of the Rational Method	D-3
D.3.1	Validity of Relationship	D-3
D.3.2	Areal Limitations of Rational Method	D-3
D.3.3	Selection Considerations of Runoff Coefficient	D-3
D.3.4	Other Loss Adjustments	D-3
D.3.5	Homogeneity of Watershed	D-3
D.3.6	Compensating for Anomalous Results	D-4
D.4.	Critical Duration (Time of Concentration)	D-4
D.4.1	Explanation of Time of Concentration	D-4
D.4.2	Variables Effecting Time of Concentration	D-4

Section	Title	Page
D.4.3	Time of Concentration for Intermediate Recurrence Intervals	D-4
D.4.4	Determination of Time of Concentration	D-5
D.5.	Watershed Study Intensity-Duration Curves	D-6
D.6.	Runoff Coefficient	D-7
D.7.	Peak Flow Rate Formula	D-7
D.8.	Drainage Area	D-8
D.9.	Rational Method Confluence Analysis	D-9
D.10.	Rational Method Tc Calculations for Unit Hydrograph Studies	D-11
D.11.	Required Format	D-12
D.12.	Instructions for Rational Method Hydrology Calculations	D-12
D.13.	Example: Peak Flow Rate Determination	D-26
E	**THE UNIT HYDROGRAPH METHOD FOR CATCHMENT RUNOFF HYDROGRAPHS**	E-1
E.1.	Definitions	E-1
E.2.	Background	E-2
E.3.	Determination of Synthetic Distribution Graphs	E-3
E.3.1	Application of Lag and S-Graphs	E-6
E.4.	Development of the Synthetic Unit Hydrograph	E-6
E.5.	Design Storm Point Precipitation	E-7
E.5.1.	Point Precipitation Adjustment for Watershed Area	E-8
E.6.	Design Storm Pattern	E-8
E.7.	Design Storm Loss Rates	E-8
E.7.1.	Maximum Loss Rate, F_m	E-8
E.7.2.	Low Loss Rate, F*	E-9

Section	Title	Page
E.8.	Runoff Volume Estimation	E-10
E.9.	Baseflows	E-10
E.10.	Alternative Runoff Volume Hydrograph Methods	E-10
E.11.	Required Format	E-10
E.12.	Instructions for Synthetic Unit Hydrograph Method Hydrology Calculations	E-10
E.13.	Example: Application of Lag Equation	E-30
E.14.	Example: Runoff Volume Estimation	E-31
E.15.	Example: Unit Hydrograph Determination	E-32
F	**FLOW-THROUGH BASIN ANALYSIS**	F-1
F.1.	Definitions	F-1
F.2.	Introduction	F-1
F.2.1	Reservoir and Stream Flow Routing	F-1
F.2.2	Unit Hydrograph Approach	F-1
F.3.	Detention Basin Analysis	F-2
F.3.1	Detention Basin Routing Procedure	F-2
F.4.	Extended Design Storm Criteria	F-4
F.5.	Required Format	F-4
F.6.	Example: Detention Basin Hydrograph Routing	F-9
G	**FLOW-BY BASIN ANALYSIS (HYDROGRAPH SEPARATION)**	G-1
G.1.	Definitions	G-1
G.2.	Introduction	G-1
G.3.	Extended Design Storm Criteria	G-1
G.4.	Flow-By Basin Volume Analysis: Weir Structure Efficiency	G-1
H	**STREAMFLOW ROUTING**	H-1

Section	Title	Page
H.1.	Definitions	H-1
H.2.	Introduction	H-1
H.3.	Convex Routing Method for Unsteady Open Channel Flow	H-1
H.4.	Example: Convex Channel Routing	H-5
I	**A PIPEFLOW ROUTING MODEL**	I-1
I.1.	Definitions	I-1
I.2.	Introduction	I-1
I.3.	Pipeflow Routing Process	I-1
J	**SMALL AREA RUNOFF HYDROGRAPH DEVELOPMENT**	J-1
J.1.	Definitions	J-1
J.2.	Introduction	J-1
J.3.	Runoff Hydrograph Criterion	J-1
J.3.1	Depth-area Adjustment	J-1
J.3.2	Design Storm Pattern Development	J-1
J.3.3	Loss Rates	J-1
J.3.4	Unit Hydrograph	J-2
J.3.5	Convolution	J-2
J.4.	Example: Small Area Runoff Hydrograph Development	J-4
K	**WATERSHED MODELING GUIDELINES**	K-1
K.1.	Definitions	K-1
K.2.	Introduction	K-1
K.3.	Single Area Runoff Hydrograph Development	K-1
K.4.	Complex Watershed Runoff Hydrograph Development	K-2
K.5.	Use of Watershed Model Computer Programs	K-3

Section	Title	Page
K.6.	Single Area Runoff Hydrograph Comparison Criteria	K-3

APPENDICES

I	Detention Basin Considerations
II	Equations for Curves and Figures
III	List of Equations and Variables Used in the Hydrology Manual
IV	Development of Hydrologic Design Criteria
V	Brief Overview of the Recent History of Flooding in Kern County
VI	Methodological Applications Policies
VII	Alternative Rational Method, With "C" Factor Curves

BIBLIOGRAPHY

REFERENCE LIST

LIST OF FIGURES

Figure		Page
B	Rainfall Isohyet Index Map	B-9
B-1 through B-55	Isohyetal Maps for Kern County	B-10
B-56	Approximate Regional Delineation for Slope of Short Duration Intensity-Duration Curves	B-65
C-1	Soil Cover Type Definition	C-9
C-2	Curve Numbers for Pervious Areas	C-10
C-3	Actual Impervious Cover for Developed Areas	C-12
C-4	SCS 24-hour Storm Rainfall-Runoff Relationships	C-13
C-5	Infiltration Rate for Pervious Areas Versus SCS Curve Numbers	C-14
C-6	Design Storm Loss Function	C-15
C-7	Hydrologic Soil Cover Complexes and Associated Curve Numbers	C-16
D-1	Time of Concentration Nomograph for Initial Subarea	D-15
D-2a	V vs Q: Natural Mountain Channel	D-16
D-2b	V vs Q: Half-Width Local Street, Residential	D-17
D-2c	V vs Q: Half-Width Local Street, Commercial	D-18
D-2d	V vs Q: Half-Width Major/Secondary Highway	D-19
D-3	Rainfall Depth Versus Return Period for Partial Duration Series	D-20
D-4	Intensity-Duration Curves Calculation Sheet	D-21
D-5	Rational Method Confluence Analysis (Summation of Runoff Hydrographs)	D-22
D-6	Rational Method Tc Calculations for Unit Hydrograph Studies	D-23

Figure		Page
D-7	Typical Report Format for Rational Method Study	D-24
D-8	Sample Rational Method Study Form	D-25
D-9	Example Problem: Drainage System Map	D-27
D-10	Example Problem: Rainfall Depth Versus Return Period for Partial Duration Series	D-28
D-11a	Example Problem: Intensity-Duration Curves, Calculation Sheet, 5 Year Return Frequency	D-29
D-11b	Example Problem: Intensity-Duration Curves, Calculation Sheet, 10 Year Return Frequency	D-30
D-11c	Example Problem: Intensity-Duration Curves, Calculation Sheet, 100 Year Return Frequency	D-31
D-12a	Example Problem: Rational Method Calculation Sheet, 100 Year Return Frequency	D-35
D-12b	Example Problem: Rational Method Calculation Sheet, 10 Year Return Frequency	D-37
D-12c	Example Problem: Rational Method Calculation Sheet, 5 Year Return Frequency	D-38
E-1	Derivation of a Synthetic Unit Hydrograph	E-14
E-2	Basin Factor Descriptions	E-15
E-3a	S-Graph for Valley: Developed	E-16
E-3b	S-Graph for Foothill Areas	E-17
E-3c	S-Graph for Mountain Areas	E-18
E-3d	S-Graph for Valley: Undeveloped	E-19
E-3e	S-Graph for Desert Areas	E-20
E-4	Design Storm Depth Area Curves	E-21
E-5a	Design Critical Storm Peak 1-hour Pattern	E-22
E-5b	Design Critical Storm Peak 3-hour Pattern	E-23
E-5c	Design Critical Storm Peak 24-hour Pattern	E-24

Figure		Page
E-6	Typical Report Format for Unit Hydrograph Study	E-25
E-7	Watershed Information Form	E-26
E-8	Area-Averaged Mass Rainfall Plotting Sheet	E-27
E-9	Flood Hydrograph Calculation Form	E-28
E-10	Derivation of Runoff Hydrograph	E-29
E-11	Example Problem: Watershed Boundary	E-33
E-12	Example Problem: Watershed Information Form	E-34
E-13	Example Problem: Area-Averaged Mass Rainfall Plotting Sheet	E-35
E-14	Example Problem: Intensity-Duration Curves, Calculation Sheet	E-36
E-15	Example Problem: Area-Averaged Mass Rainfall Plotting Sheet	E-36
E-16	Example Problem: Watershed Loss Determination Information Map	E-38
E-17	Example Problem: Synthetic 3-hour Critical Storm	E-43
E-18	Example Problem: S-Graph for Valley: Developed	E-44
E-19	Example Problem: Unit Distribution Graph	E-45
E-20	Example Problem: Flood Hydrograph Calculation Form	E-46
E-21	Example Problem: 3-hour Storm Runoff Hydrograph	E-47
F-1	Detention Basin Inflow and Outflow Hydrographs	F-5
F-2	Extended Design Storm: Flow-through Basin	F-6
F-3	Required Report Format for Detention Basin Study	F-7
F-4	Detention Basin Information Form	F-8
F-5	Example Problem: Detention Basin Outflow (cfs) as a Function of Depth (ft)	F-10

Figure		Page
F-6	Example Problem: Detention Basin Storage (AF) as a Function of Depth (ft)	F-11
F-7	Example Problem: Detention Basin Information Form	F-12
F-8	Example Problem: Storage-Indication Curve	F-13
G-1	Flow-by Basin Concept and Elements	G-3
G-2	Flow-by Basin Critical Storm Pattern (3-day Example)	G-4
H-1	Assumed Routing Coefficient Geometric Relationships	H-4
I-1	Pipeflow Modeling Elements	I-2
J-1	Small Area Unit Hydrograph	J-3
J-2	Example Problem: Small Area Runoff Hydrograph	J-6

LIST OF TABLES

Table		Page
B.1.	Multiday Rainfall Mass Ratios for Kern County	B-10
D.1.	K_T Values for Common Return Intervals	D-6
D.2.	Area-Averaged F_m Computation	D-10
D.3a.	Example Problem: Calculations of Subarea Yield Rates; 5 Year	D-37
D.3b.	Example Problem: Calculations of Subarea Yield Rates; 10 Year	D-38
D.3c.	Example Problem: Calculations of Subarea Yield Rates; 100 Year	D-39
E.1.	Example Problem: Runoff Volume Estimation Assumptions	E-36
E.2.	Example Problem: Runoff Volume Estimation Yield Calculations	E-36
E.3.	Example Problem: Unit Hydrograph Study, Unit Rainfall Determination	E-45
E.4.	Example Problem: Unit Hydrograph Study, Watershed Loss Determination, F_m	E-46
E.5.	Example Problem: Unit Hydrograph Study, Watershed Loss Determination, -Y	E-47
E.6.	Example Problem: Unit Hydrograph Study, 3-hour Storm Effective Rainfall Determination	E-48
F.1.	Example Problem: Storage-Indication Curve Development	F-10
F.2.	Example Problem: Basin Routing Tabulation	F-15
H.1.	Convex Routing Example Problem Solution	H-7
J.1.	Example Problem Results	J-5

SECTION A

INTRODUCTION

A.1. ABOUT THIS MANUAL

This manual provides the computational techniques and criteria for the estimation of runoff, discharges, and volumes for use in hydrology study submittals to the County of Kern.

In general, each section is laid out in the following manner:

Overview: This segment will include an outline of materials to be covered in the section as well as a cross-reference to other pertinent sections. This overview segment will be enclosed in a box at the beginning of each section.

Definitions: Those terms required for understanding the section will be defined.

Text: This segment will explain the techniques or concepts necessary to perform the desired task or hydrologic method.

Instructions: Some sections have detailed methodological procedures included to assist the user in the preparation of the needed work. When such procedural instructions are provided they will be placed after the text on **BLUE** paper.

Tables and Figures: All the sections related tables and figures are, in general, located after the text, or instruction sections when applicable, on **YELLOW** paper.

Examples: Example problems, expounding upon the methods or concepts germane to the section, are found after the tables and figures on **GREEN** paper (Note: Oversized sheets are on white paper).

Equations: All equations described in the section are attached at the end of the section, for ready reference, on **PINK** paper.

Related Equations: Those equations that are related to the topics discussed in the section, but are described in detail in other sections, are included at the end of the section on **PINK** paper.

Throughout the text certain key phrases or terms are highlighted and *italicized* to assist the user in finding pertinent information. Typically these key phrases or terms will also be defined within the definitions segment of the section.

A.2. GOALS

The goal of the County is to provide flood protection, for all habitable structures and other non-floodproofed structures, consistent with the Kern County Floodplain Management Ordinance. Additionally, the Kern County "Land Development Standards" specify drainage system design and flood protection and drainage design criterion for roadways, catch basins, sump design, and other conditions.

A.3. SUMMARY OF CONTENTS

- Precipitation and losses information common to both the Rational Method and the Unit Hydrograph procedure for developing flow rates are contained in Sections B and C, respectively.
- Specific guidelines for application of the Rational Method are contained in Section D.
- Section E contains the procedures for developing runoff hydrographs using the unit hydrograph method.
- Sections F through I contain guidelines for application of various flood routing methods.
- The development of runoff hydrographs for small areas is discussed in Section J and watershed modeling guidelines are provided in Section K.
- Appendix I provides a discussion of environmental considerations related to detention basin design.
- Appendix II provides data and/or equations for the curves and figures used in this manual.
- Appendix III provides a list of equations and variables used in this manual.
- Appendix IV is a discussion of the development of the hydrologic design criteria used in the manual.
- Appendix V provides a brief history of recent flood events in the County.
- Appendix VI provides a list of policies for the application, or modification, of the methodologies described in this manual.
- Appendix VII provides an alternative Rational Method based upon the use of "C" factor curves.

SECTION B

PRECIPITATION

OVERVIEW

This section will cover precipitation depth and intensity computation methods and data. It will discuss:

- Development of Intensity-Duration Curves
- Preparation of Synthetic 24-hour Critical Storm Pattern
- Multiday Design Storm Criterion

Section C presents Loss computations. Section D and E address watershed flow determination processes.

For other important information pertinent to this section, refer to Figures D-3 and D-4 in Section D.

B.1. DEFINITIONS

Critical duration: The critical duration of a design storm event for a hydraulic structure is essentially the "time of concentration," which is the time for water deposited at the most remote part of a watershed to flow to the structure, outlet, or spillway.

Duration: The specified length of storm time under study. Duration may be expressed in any time unit such as seconds, minutes, hours, days, or season.

Exceedance (cumulative) probability: The probability that a precipitation event of a specified depth and duration will be exceeded in one year.

Frequency: The frequency of occurrence of events with the specified precipitation depth and duration. This is expressed in terms of either the return period or exceedance probability, both of which are defined below.

Intensity-duration: Dividing precipitation depth by duration, an average intensity for a specified duration is obtained.

Precipitation depth: The amount of precipitation occurring during a specified duration of storm time. Precipitation depth is usually expressed in units of inches.

Return period (recurrence interval): The long term average number of years between occurrences of an event of a given depth and duration, either equaled or exceeded.

B.2. CLASSIFICATION OF PRECIPITATION

In this manual, references are made to several types of storm precipitation mechanisms which may occur in Kern County watersheds. The classification of storm precipitation is generally attributed to the mechanism which elevates the moisture laden air mass to be subsequently cooled. ***Frontal precipitation***, for example, occurs when moisture laden warm air is elevated (along a frontal surface) over a region of cooler air. Likewise, non-frontal precipitation implies that the storm precipitation is not attributed to fronts. ***Convective precipitation*** is caused by the upwards migration of warm air through dense cold air or, for example, thunderstorms over the desert. Related rainfall intensities range from light showers to catastrophic cloudbursts. ***Orographic precipitation*** describes the process of moisture laden air being cooled due to the elevation of the air mass over mountain barriers.

B.3. MECHANISMS FOR COOLING

Each of the above-discussed elevation mechanisms involves a cooling process which is also associated with the resulting precipitation. ***Cyclonic cooling***, for example, is generally classified according to the association with frontal and non-frontal effects.

The non-frontal grouping is associated with the convergence and resulting elevation of air accompanied with a low pressure area. Non-frontal cyclonic precipitation of extratropical origin is associated with rainfall (or snows) of moderate intensities and of long duration. This type of storm may last for a few days and deliver 1 to in excess of 6 inches of precipitation. In contrast, non-frontal cyclonic precipitation of tropical origin may result in over double the rainfall in a period of half the duration.

Orographic cooling occurs when moisture laden air is elevated along a mountain barrier, allowing the air to expand and cool at the upper elevation lower pressure. Because of the mountain barrier, a "rain shadow" may occur on the lee side of the barrier. Generally, the windward sides of mountain barriers are more cloudy, have more precipitation, and have smaller temperature ranges than the leeward side.

When vertical instability of moisture laden air is produced by surface heating, a convection current results. ***Convective precipitation*** is generally associated with very short duration events of high intensity, and can result in catastrophic flooding.

B.4. PRECIPITATION DEPTH-DURATION-FREQUENCY

Due to the apparent randomness of precipitation patterns and intensities, a strictly deterministic analysis of precipitation quantities is not possible and, consequently, a statistical evaluation is generally used.

In hydrologic analysis, the rainfall intensity is usually the most important parameter. This value relates both precipitation volume to storm duration and also relates storm runoff to storm precipitation with the intensity being an upper bound to the watershed runoff rate. However, in order to provide a reasonable level of flood protection, the statistical concept of ***return frequency*** is utilized which aids in assigning a probabilistic meaning to a precipitation event.

The ***exceedance probability*** **(P)** and ***return period*** **(T)** are related by

$$P=1/T \tag{B.1}$$

It can be argued that a 100-year precipitation event, for example, will not necessarily occur once in every 100 years but actually has a finite probability that it will occur in several consecutive years.

B.5. MEASUREMENT AND SYNTHESIS OF PRECIPITATION DATA

Of interest for hydrologic studies are the maximum intensities of precipitation possible throughout a watershed. Given a long history of such maximum rainfall intensities for various durations of time, a reasonable statistical interpretation can be made of the data to determine estimates of maximum rainfall intensities or depths as a function of storm duration and of return frequency. Sources of precipitation data include the National Oceanic and Atmospheric Administration (NOAA) - National Weather Service, U.S. Army Corps of Engineer, U.S. Geological Survey, and other private and governmental cooperative weather observers.

For each automatic recording rainfall gauge, the precipitation records are analyzed to determine the ***annual maximum rainfall depth*** for several durations of interest (e.g., 5 minutes, 10 minutes, 15 minutes, etc.). This data can then be arranged in an increasing order of magnitude for each storm duration for the history of the rain gauge, and plotted on normal probability paper. From this accumulation of rainfall depth-duration data, various statistical models can be applied to assign a return frequency (or period) to the known data values and to estimate maximum rainfall depth-duration values for typically unmeasured higher return frequencies (e.g., the 100-year return frequency). The resulting data for each rain gauge is generally termed ***"point precipitation"*** values to distinguish them from average values for large areas.

The rainfall data can generally be assumed to underestimate the true maximum point rainfall intensities for the following reasons:

1) Storm events seldom locate their peak intensities over rain gauges

2) The rain gauge network is widely distributed (allowing small intense rainfall events to miss the gauge network)

3) Mechanical defects of the gauging devices and due to other unknown variables such as wind effects.

B.6. POINT PRECIPITATION

The County has prepared ***isohyetal maps*** (Figures B-1 through B-55) showing 2-year 6- and 24-hour (Figures B-1 through B-22), and 100-year 6- and 24-hour (Figures B-34 through B-55) point precipitation values. These maps are based on the "NOAA Atlas 2, Precipitation-Frequency Atlas of the Western United States, Volume XI-California" published by the National Weather Service in 1973. A correlation to local rain gauge data and an interpretation of short duration precipitation data was prepared for the County of Kern. (A comparison of the NOAA Atlas 2 data and the State of California Department of Water Resources (DWR) precipitation data was made for several Kern County locations. The data sets compare favorably. For example, NOAA Atlas 2, 100-year point precipitation values are on the average 8 percent higher for the 1-hour duration to 1 to 4 percent lower than the DWR data for the 6- and 24-hour durations, respectively.) Figures B-1 through B-55 can be found at the end of this section.

The ***isohyetal maps*** may be used to estimate the precipitation data needed to prepare rational method (Section D) or unit hydrograph (Section E) hydrology studies. Any deviation in the use of the provided rainfall data must be approved by the County. One-hour point precipitation values can be estimated using the equations (NOAA Atlas 2) shown in Section B.9, Intensity-Duration Curves. Point precipitation values for other return frequencies can be estimated using the diagram (NOAA Atlas 2) of Figure D-3.

B.7. RANDOM NATURE OF POINT-PRECIPITATION DATA

When rain gauge records are examined to identify relationships with respect to time, an extremely wide range of variations are found. These random variations are so great that they essentially obscure any long-term pattern or periodicity which may exist. To utilize point precipitation data, therefore, a combination of probabilistic and deterministic methods is needed. The duration and magnitude of individual storm events is assumed probabilistic, while the internal assemblage of the storm may be essentially deterministic. Additionally, the origin of such storm events (e.g., convective, orographic) adds to the difficulty of developing a comprehensive analysis.

B.8. EVENT DEPTH-DURATION

For most hydrologic study purposes, the important relationship is that of precipitation depth for any rainfall event of a given duration. As discussed in the NOAA Atlas 2, this relationship will include total precipitation from storms of the given duration, and will also include the depths from independent continuous partial storm durations. This information can be represented by event depth-duration curves which are constructed by ranking in the order of decreasing rainfall depth all storm events of some common duration from a subject rain gauge. From the position of a ***precipitation depth***, an estimate can be made of the number of years during which the event (of a given duration) will be equaled or exceeded.

After constructing event depth-duration curves, a second set of precipitation depth-duration (or intensity-duration) curves can be developed which, for a given return frequency, represent the maximum precipitation depth (or intensity) which can occur from any storm as a function of duration.

B.9. INTENSITY-DURATION CURVES

Intensity-duration data is required for use with the rational method. This data is usually presented in the form of curves of rainfall intensity in inches per hour versus storm duration in minutes. Intensity-duration data for durations under 3 hours tends to plot in a straight line on log-log paper, and the curves for various return periods tend to run parallel to one another.

Intensity-duration curves can be developed for a watershed by estimating the appropriate area-averaged one-hour point precipitation values from the following NOAA Atlas 2 equations:

Coast Ranges:	Y2 = 0.111+0.545 (X1)(X1/X2)	(B.2)
	Y100 = 0.221+0.885 (Y2)(X3/X1) **or**	(B.3)
	Y100 = 0.221+0.098 (X3/X1) + 0.482 (X3)(X1/X2)	(B.4)
San Joaquin Valley:	Y2 = 0.107+0.315 (X1)	(B.5)
	Y100 = -0.391+1.224 (Y2)(X3/X1) +0.043 (X5) **or**	(B.6)
	Y100 = -0.391+0.131 (X3/X1) +0.386(X3)+0.043 (X5)	(B.7)
Sierra Nevada (Tehachapi) Mtns:	Y2 = 0.126+0.561 (X1)(X1/X2)	(B.8)
	Y100 = -0.030+0.816 (Y2)(X3/X1) +0.063 (X5) **or**	(B.9)
	Y100 = -0.030+0.103 (X3/X1) + 0.458 (X3)(X1/X2) + 0.063 (X5)	(B.10)
Desert Region:	Y2 = 0.005+0.852 (X1)(X1/X2)	(B.11)
	Y100 = 0.322+0.789 (X3)(X3/X4)	(B.12)

List of Variables

Y2 = 2-year 1-hour estimated value

Y100 = 100-year 1-hour estimated value

X1 = 2-year 6-hour value from precipitation-frequency maps

X2 = 2-year 24-hour value from precipitation-frequency maps

X3 = 100-year 6-hour value from precipitation-frequency maps

X4 = 100-year 24-hour value from precipitation-frequency maps

X5 = Latitude (in decimals) minus 32

The approximate regional boundary locations are delineated on Figure B-56. The determination of exact regional boundaries can be highly subjective and is, therefore, left to the hydrologist under consultation with the County.

Intermediate return frequency point precipitation values can be estimated using Figure D-3.

Using Figure D-4, the one-hour point precipitation value is plotted and a straight line is drawn with the appropriate slope. Generally, a slope of 0.65 may be used for watersheds in the San Joaquin Valley, 0.60 in the Desert region, 0.55 in the Antelope Valley, Tehachapi Mountains, and Coast Ranges, and 0.45 in the northerly Sierra Nevada Mountain Region of the County. From Figure D-4, a minimum duration of 5 minutes is used for rational method studies.

B.10. SYNTHETIC 24-HOUR CRITICAL STORM PATTERN

The U.S. Department of Agriculture Soil Conservation Service (SCS) developed dimensionless critical storm patterns using the U.S. National Weather Service's (NWS) rainfall frequency atlases (ref. 2). The rainfall frequency data for areas less than 400 square miles, for durations to 24 hours, and for frequencies from 1 to 100 years were used.

These critical storm patterns are based on the generalized precipitation depth-duration-frequency relationships shown in technical publications of the NWS, and precipitation depths for durations from 1 minute to 24 hours were used to derive the storm patterns. Using increments of 30 minutes, incremental precipitation depths were determined. For example, the 30-minute depth was subtracted from the 1-hour depth and the 1-hour depth was subtracted from the 1.5-hour depth. The storm patterns were formed by arranging the 30-minute incremental depths such that the maximum 30-minute depth is contained within the maximum 1-hour depth, and the maximum 1-hour depth is contained within the maximum 1.5-hour depth and so forth. Because all of the critical

precipitation depths are contained within the storm pattern, the critical storm patterns may be assumed appropriate for designs on both small and large watersheds (ref. 2).

The agency's design storm pattern is based upon a modification of the SCS 24-hour storm pattern. The design storm pattern provides a representation of local precipitation depth-duration-frequency tendencies by constructing the several nested intervals to fit local recorded rainfall data. Additionally, the SCS storm pattern is further modified to include the necessary adjustments (reduction in shorter duration point precipitation values) due to watershed area effects. The procedures used to construct the 24-hour storm pattern and determine the associated rainfall depths adjusted for depth-area follow the U.S. Army Corps of Engineers methods as published in the HEC Training Document No. 15 (ref. 10). Details of the 24-hour storm pattern and the necessary adjustments for depth-area effects are contained in Section E.

B.11. LONGER DURATION RAINFALL DATA

The County's design storm criteria extend to a multiday design storm when needed to evaluate detention basin characteristics (Sections F and G). The following tabulation provides a ratio of daily rainfalls to the peak 24-hour mass rainfall. (Table B.1 is a weighted average relationship developed from the Bakersfield, Glennville-Fulton, Tehachapi, Mojave, and Inyokern rain gauges.)

TABLE B.1. MULTIDAY RAINFALL MASS RATIOS FOR KERN COUNTY

Rainfall Duration	Ratio to Peak 24-hours
Peak 24-hours	1
(Peak 48-hours)-(Peak 24-hours)	0.21
(Peak 72-hours)-(Peak 48-hours)	0.11
(Peak 96-hours)-(Peak 72-hours)	0.07
(Peak 120-hours)-(Peak 96-hours)	0.05

RAINFALL ISOHYET INDEX MAP

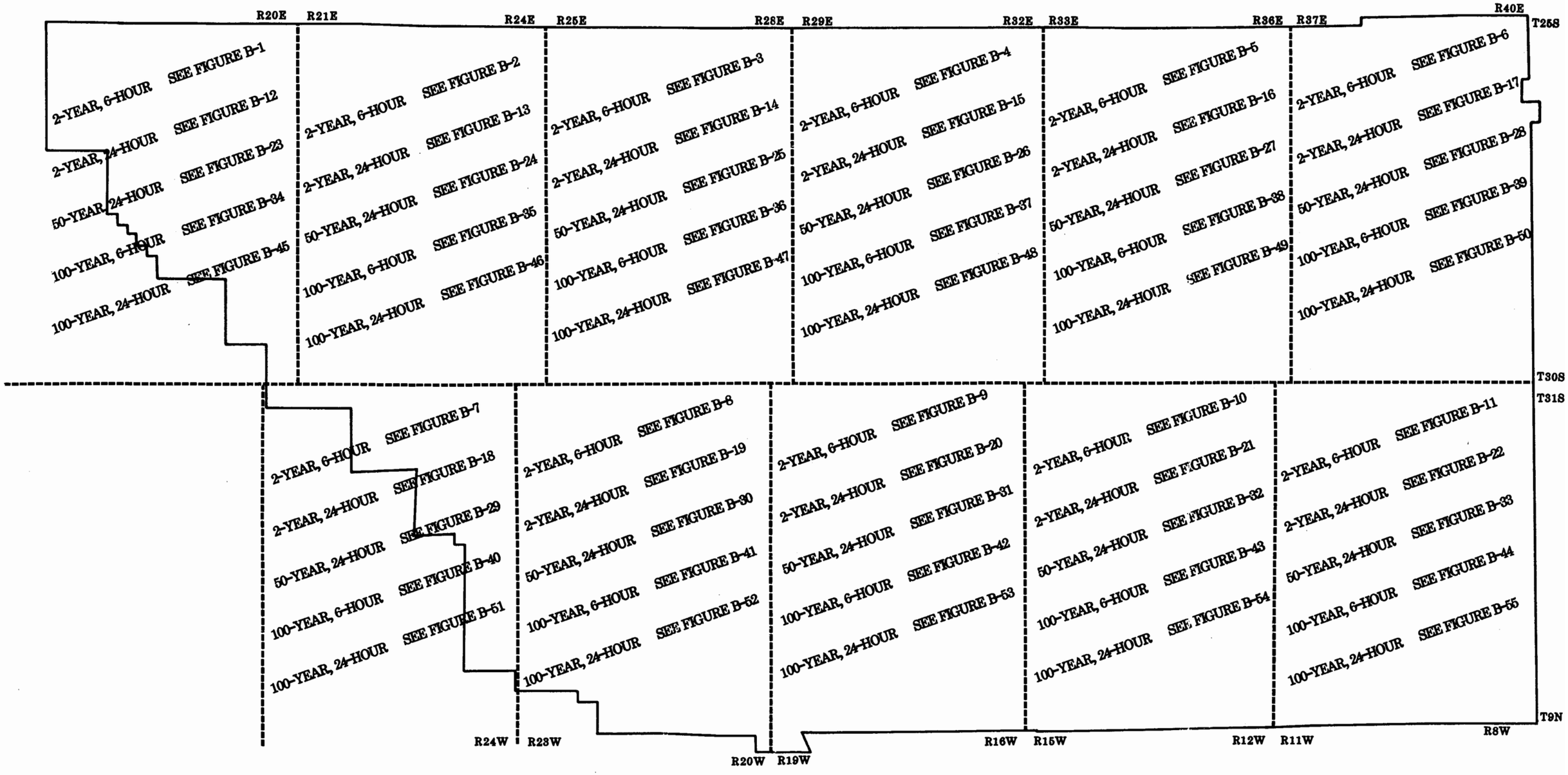


FIGURE B

Rainfall Isohyet Index Page

Township 25S-30S Range 20E

Figure	Map	Page
B-1	2-year 6-hour	B-12
B-12	2-year 24-hour	B-23
B-23	50-year 24-hour	B-34
B-34	100-year 6-hour	B-45
B-45	100-year 24-hour	B-56

Township 25S-30S Range 21E-24E

B-2	2-year 6-hour	B-13
B-13	2-year 24-hour	B-24
B-24	50-year 24-hour	B-35
B-35	100-year 6-hour	B-46
B-46	100-year 24-hour	B-57

Township 25S-30S Range 25E-28E

B-3	2-year 6-hour	B-14
B-14	2-year 24-hour	B-25
B-25	50-year 24-hour	B-36
B-36	100-year 6-hour	B-47
B-47	100-year 24-hour	B-58

Township 25S-30S Range 29E-32E

B-4	2-year 6-hour	B-15
B-15	2-year 24-hour	B-26
B-26	50-year 24-hour	B-37
B-37	100-year 6-hour	B-48
B-48	100-year 24-hour	B-59

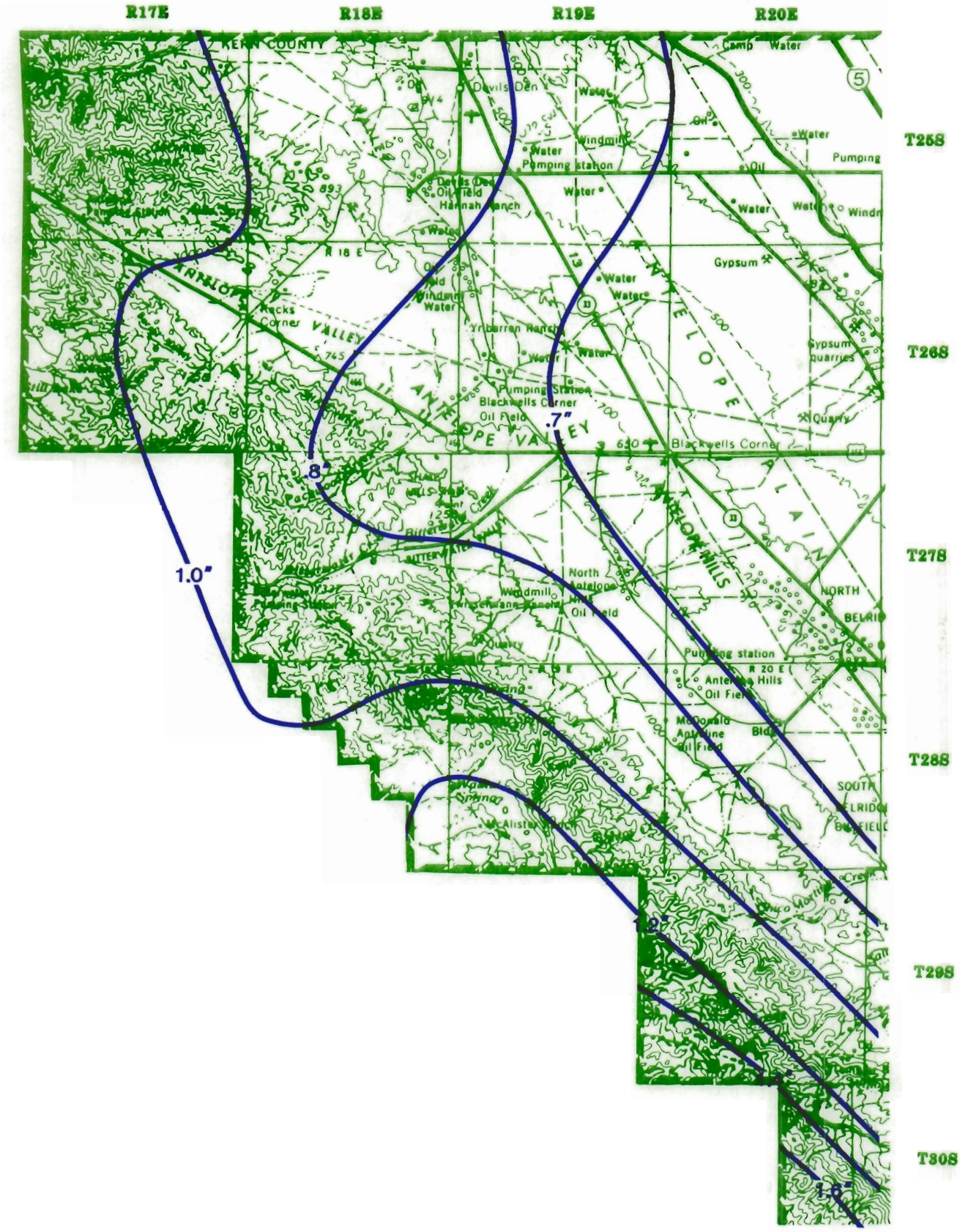
Township 25S-30S Range 33E-36E

B-5	2-year 6-hour	B-16
B-16	2-year 24-hour	B-27
B-27	50-year 24-hour	B-38
B-38	100-year 6-hour	B-49
B-49	100-year 24-hour	B-60

Township 25S-30S Range 37E-40E

B-6	2-year 6-hour	B-17
B-17	2-year 24-hour	B-28
B-28	50-year 24-hour	B-39

B-39	100-year 6-hour	B-50
B-50	100-year 24-hour	B-61
Township 31S-9N Range 24W		
B-7	2-year 6-hour	B-18
B-18	2-year 24-hour	B-29
B-29	50-year 24-hour	B-40
B-40	100-year 6-hour	B-51
B-51	100-year 24-hour	B-62
Township 31S-9N Range 23W-20W		
B-8	2-year 6-hour	B-19
B-19	2-year 24-hour	B-30
B-30	50-year 24-hour	B-41
B-41	100-year 6-hour	B-52
B-52	100-year 24-hour	B-63
Township 31S-9N Range 19W-16W		
B-9	2-year 6-hour	B-20
B-20	2-year 24-hour	B-31
B-31	50-year 24-hour	B-42
B-42	100-year 6-hour	B-53
B-53	100-year 24-hour	B-64
Township 31S-9N Range 15W-12W		
B-10	2-year 6-hour	B-21
B-21	2-year 24-hour	B-32
B-32	50-year 24-hour	B-43
B-43	100-year 6-hour	B-54
B-54	100-year 24-hour	B-65
Township 31S-9N Range 11W-8W		
B-11	2-year 6-hour	B-22
B-22	2-year 24-hour	B-33
B-33	50-year 24-hour	B-44
B-44	100-year 6-hour	B-55
B-55	100-year 24-hour	B-66



KERN COUNTY HYDROLOGY MANUAL

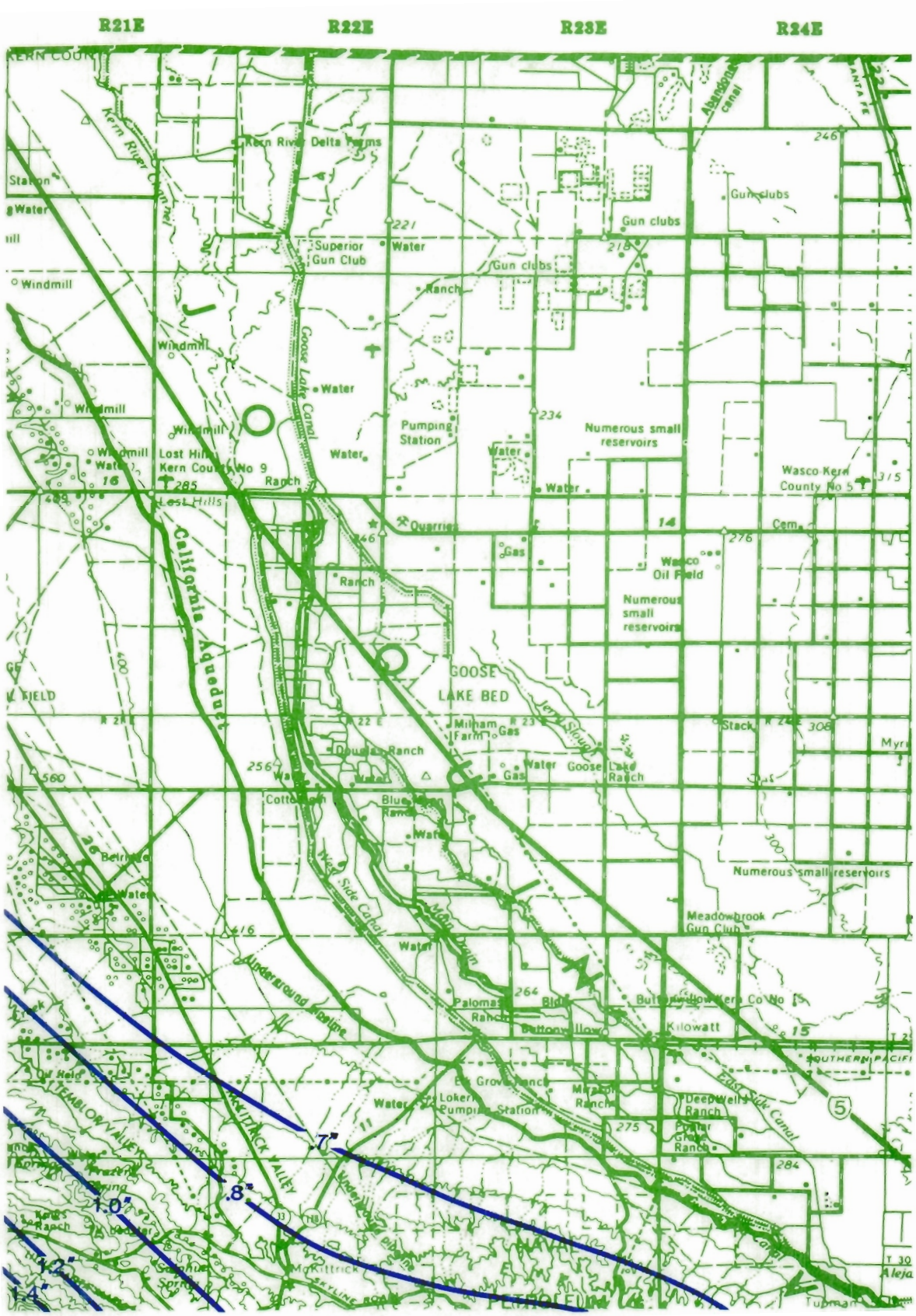
RAINFALL ISOHYET MAP

APPROXIMATE SCALE
1"=4 miles

2-Year, 6-Hour Isohyet

NORTH

FIGURE B-1



KERN COUNTY HYDROLOGY MANUAL

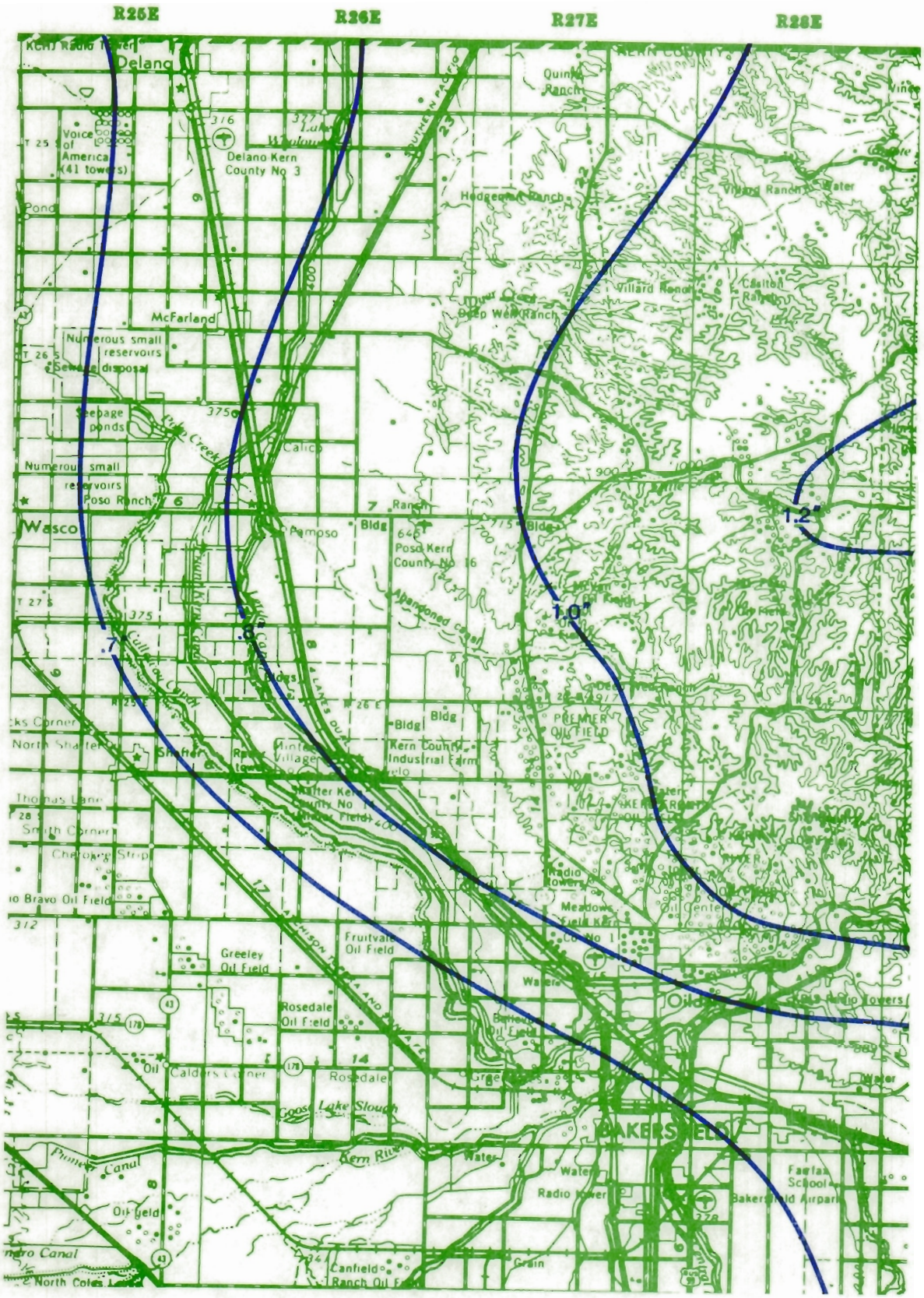
RAINFALL ISOHYET MAP

APPROXIMATE SCALE
1"=4 miles

2-Year, 6-Hour Isohyet

NORTH

FIGURE B-2



KERN COUNTY HYDROLOGY MANUAL

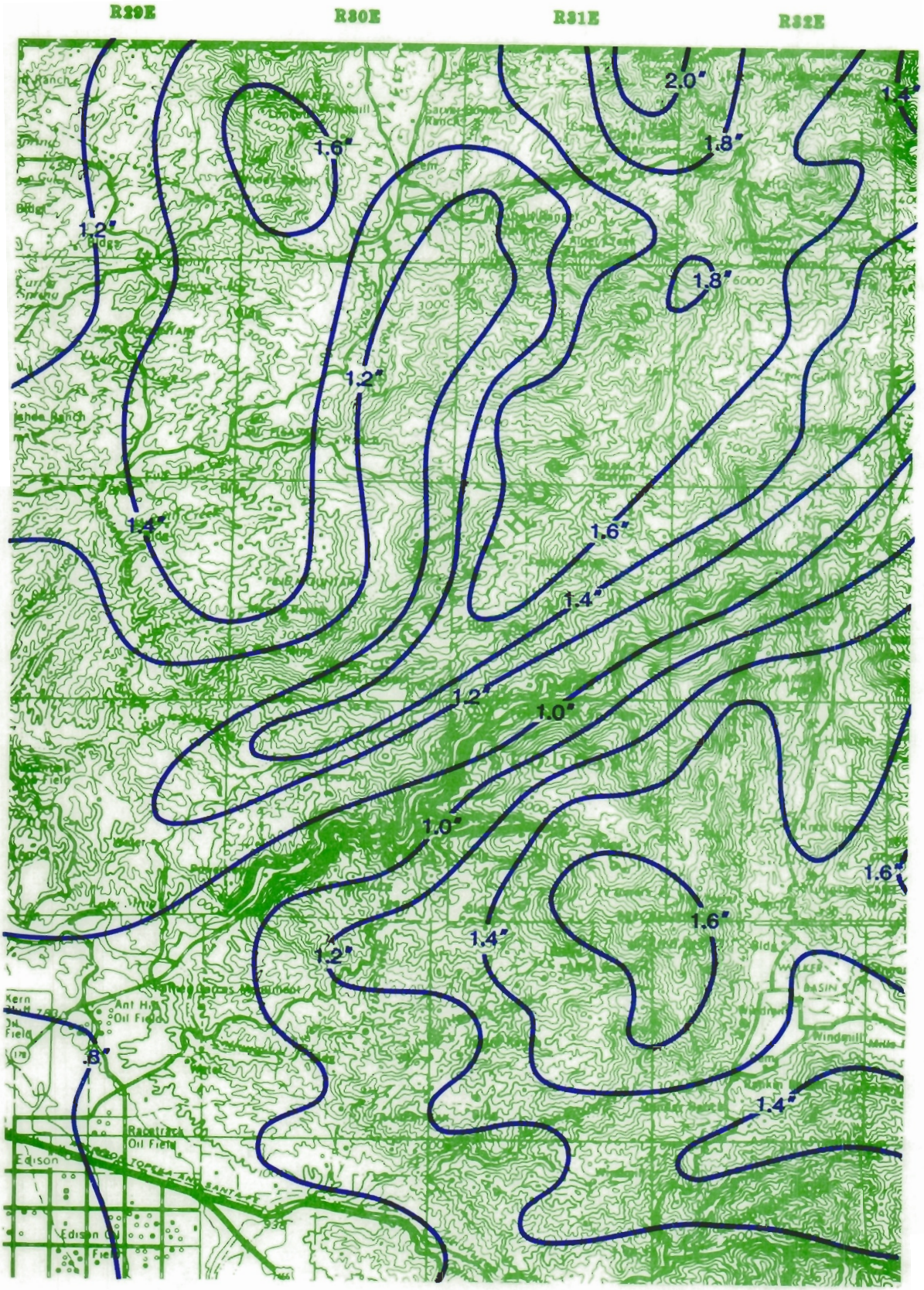
RAINFALL ISOHYET MAP

APPROXIMATE SCALE
1"=4 miles

2-Year, 6-Hour Isohyet

NORTH

FIGURE B-3



KERN COUNTY HYDROLOGY MANUAL

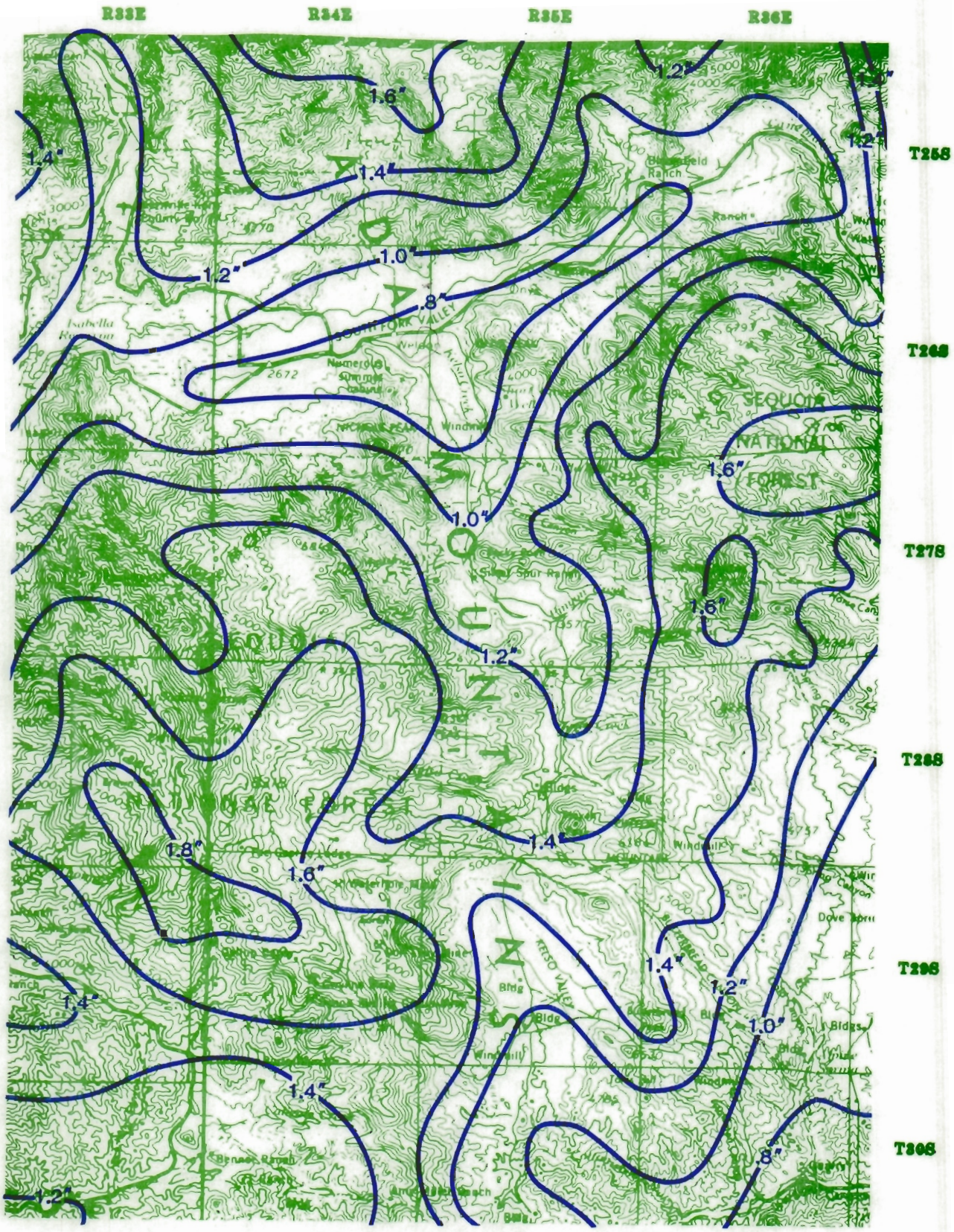
RAINFALL ISOHYET MAP

APPROXIMATE SCALE
1"=4 miles

2-Year, 6-Hour Isohyet

NORTH

FIGURE B-4



KERN COUNTY HYDROLOGY MANUAL

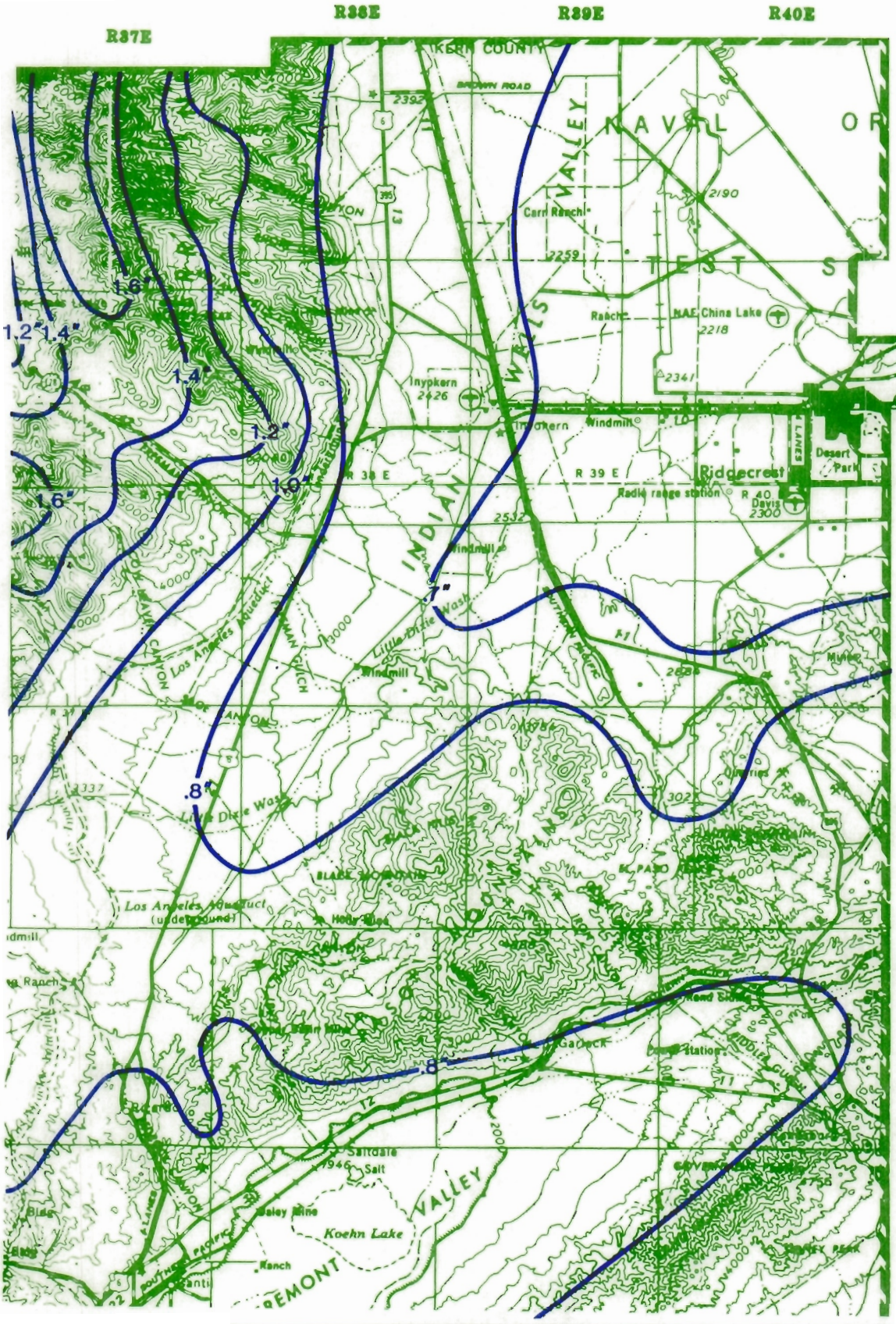
RAINFALL ISOHYET MAP

APPROXIMATE SCALE
1"=4 miles

2-Year, 6-Hour Isohyet

NORTH

FIGURE B-5



KERN COUNTY HYDROLOGY MANUAL

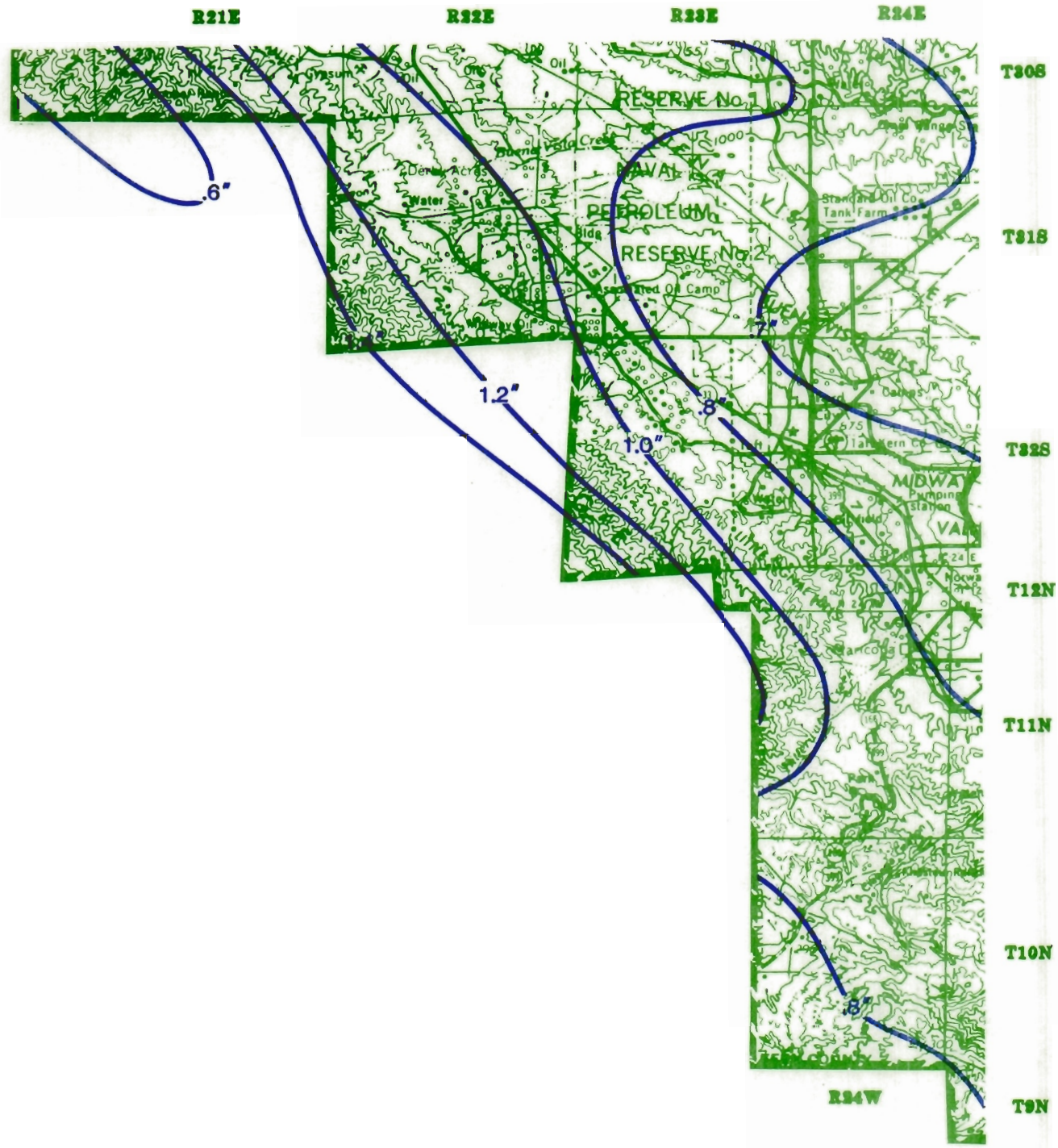
RAINFALL ISOHYET MAP

APPROXIMATE SCALE
1"=4 miles

2-Year, 6-Hour Isohyet

NORTH

FIGURE B-6



KERN COUNTY HYDROLOGY MANUAL

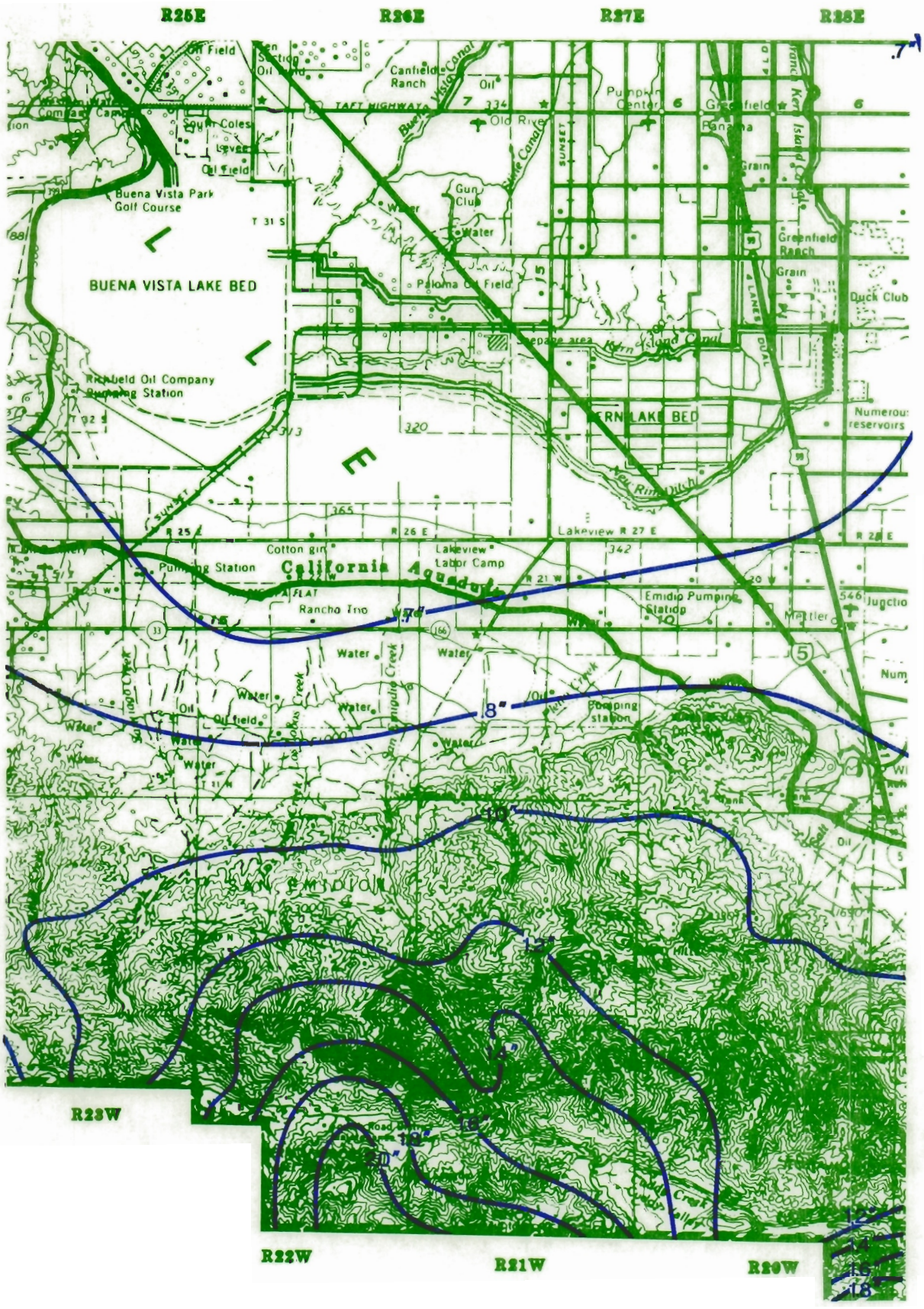
RAINFALL ISOHYET MAP

APPROXIMATE SCALE
1"=4 miles

2-Year, 6-Hour Isohyet

NORTH

FIGURE: B-7



KERN COUNTY HYDROLOGY MANUAL

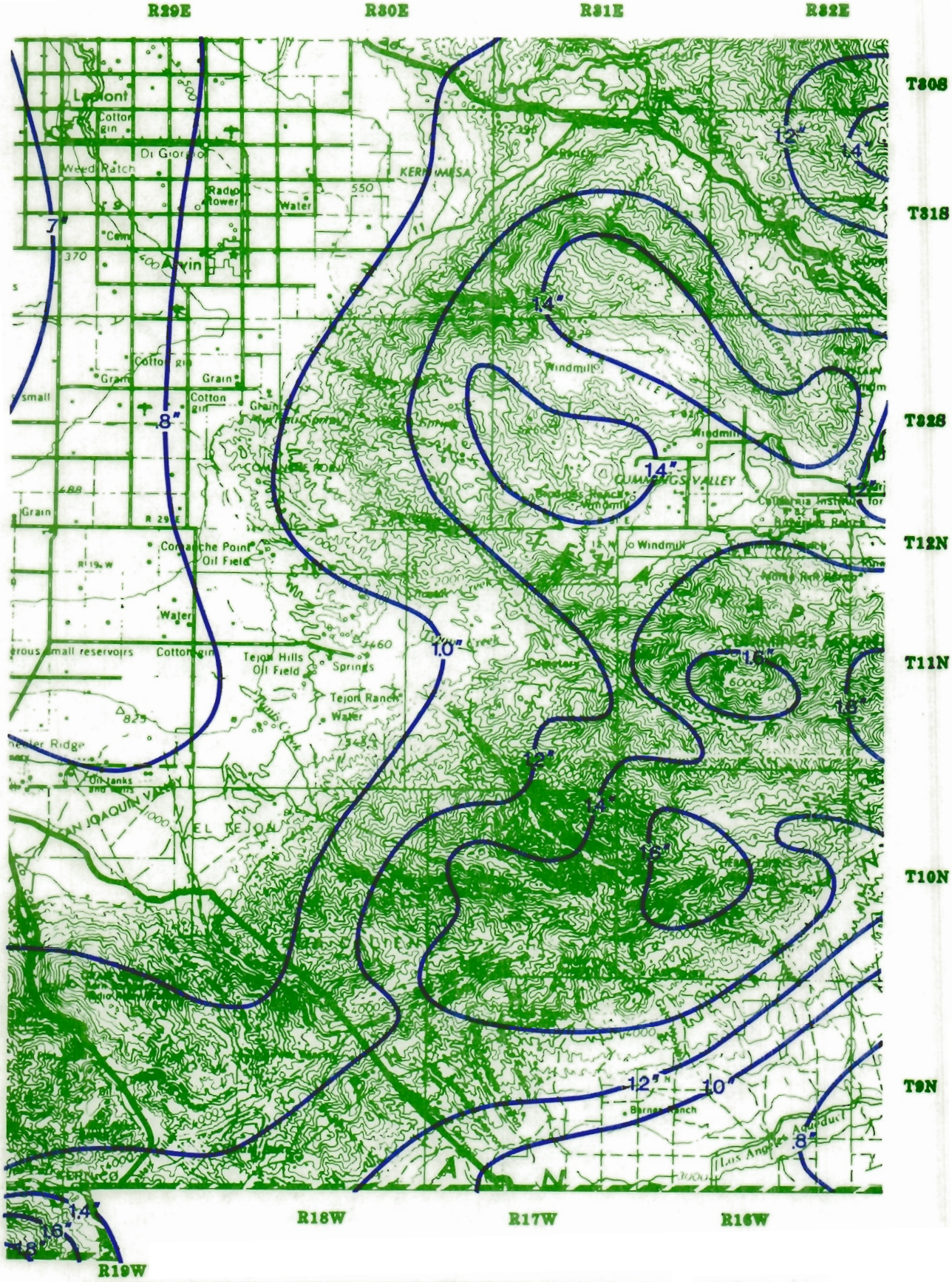
RAINFALL ISOHYET MAP

APPROXIMATE SCALE
1"=4 miles

2-Year, 6-Hour Isohyet

NORTH

FIGURE B-8



KERN COUNTY HYDROLOGY MANUAL

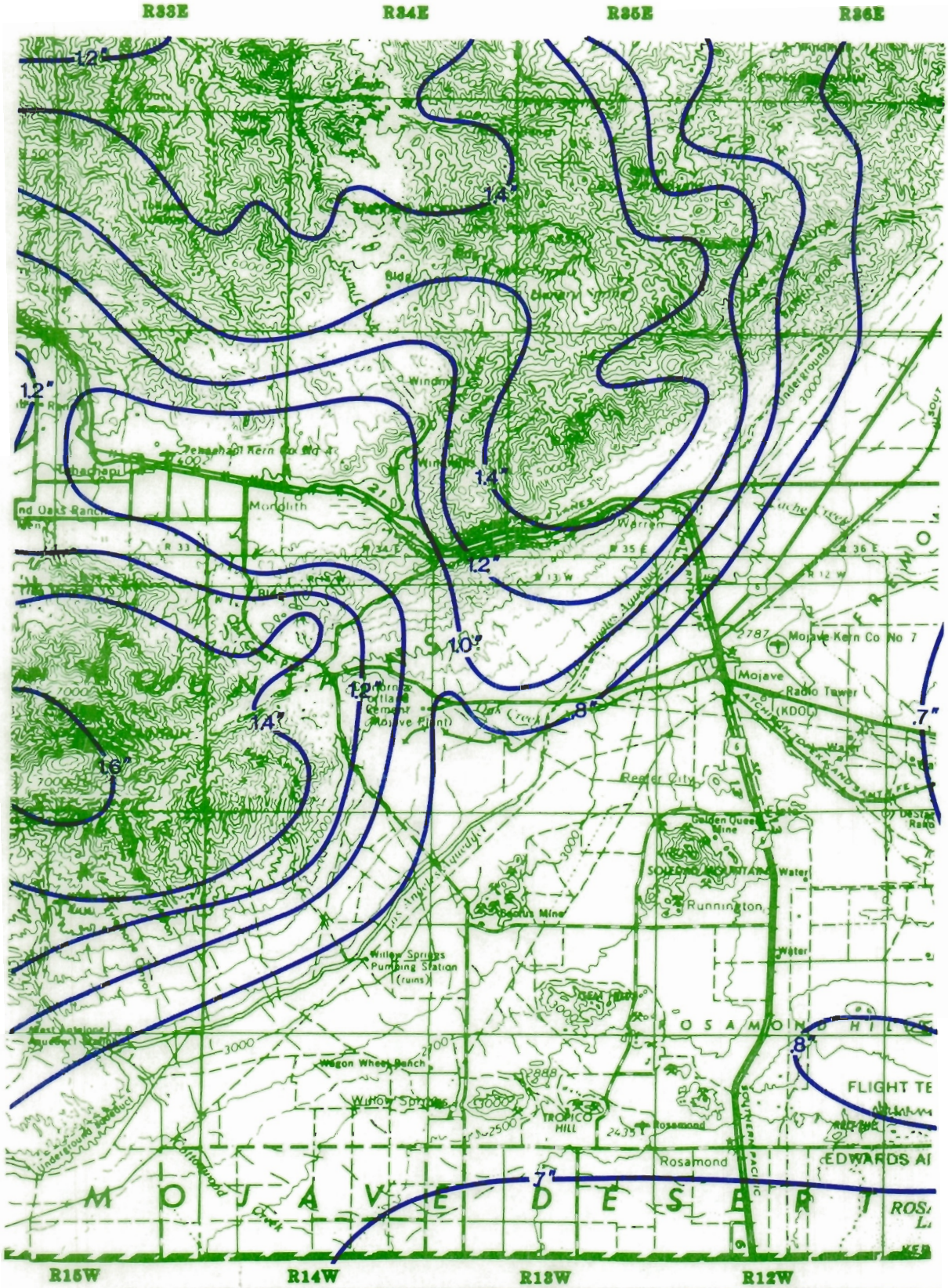
RAINFALL ISOHYET MAP

APPROXIMATE SCALE
1"=4 miles

2-Year, 6-Hour Isohyet

NORTH

FIGURE B-9



KERN COUNTY HYDROLOGY MANUAL

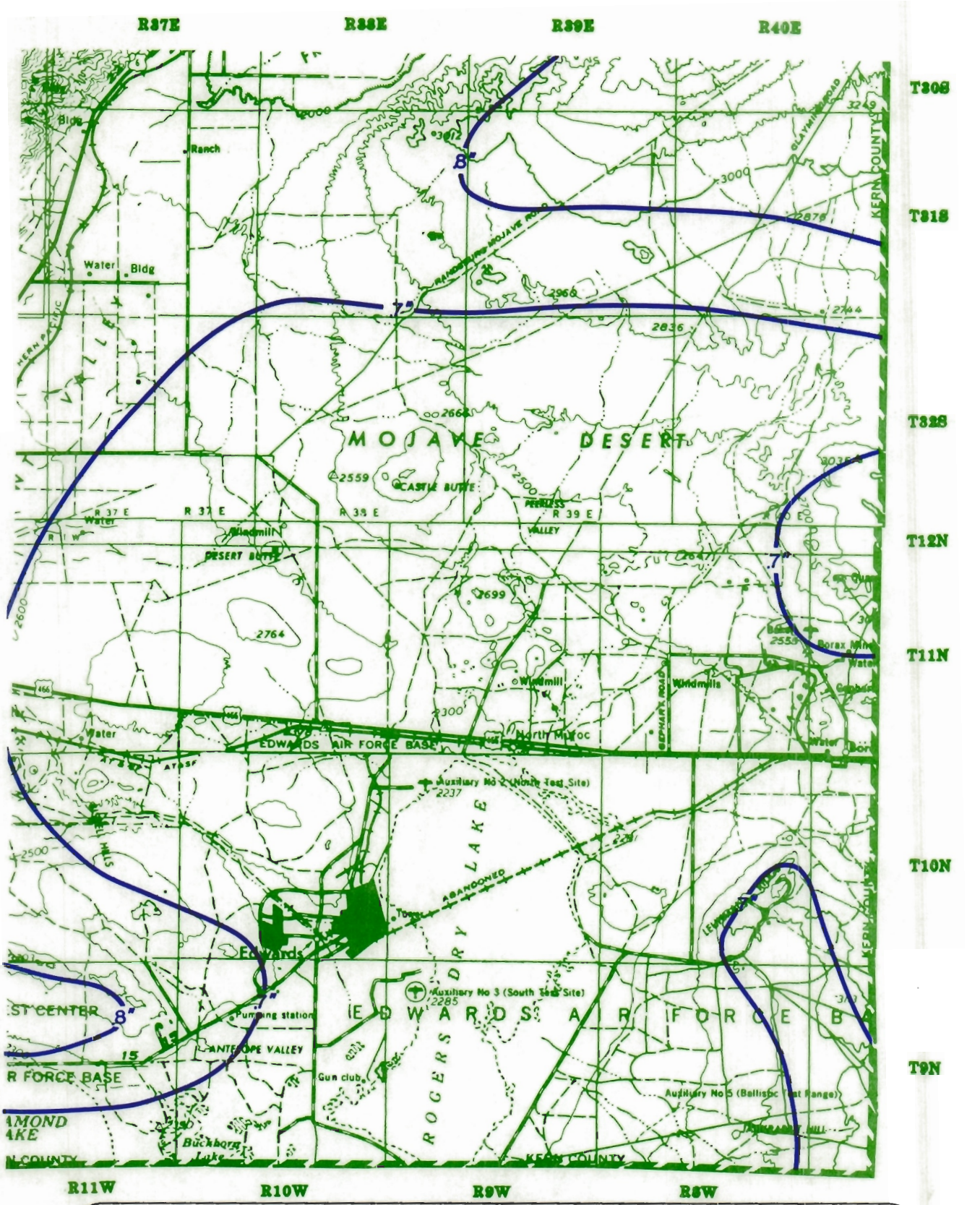
RAINFALL ISOHYET MAP

APPROXIMATE SCALE
1"=4 miles

2-Year, 6-Hour Isohyet

NORTH

FIGURE B-10



KERN COUNTY HYDROLOGY MANUAL

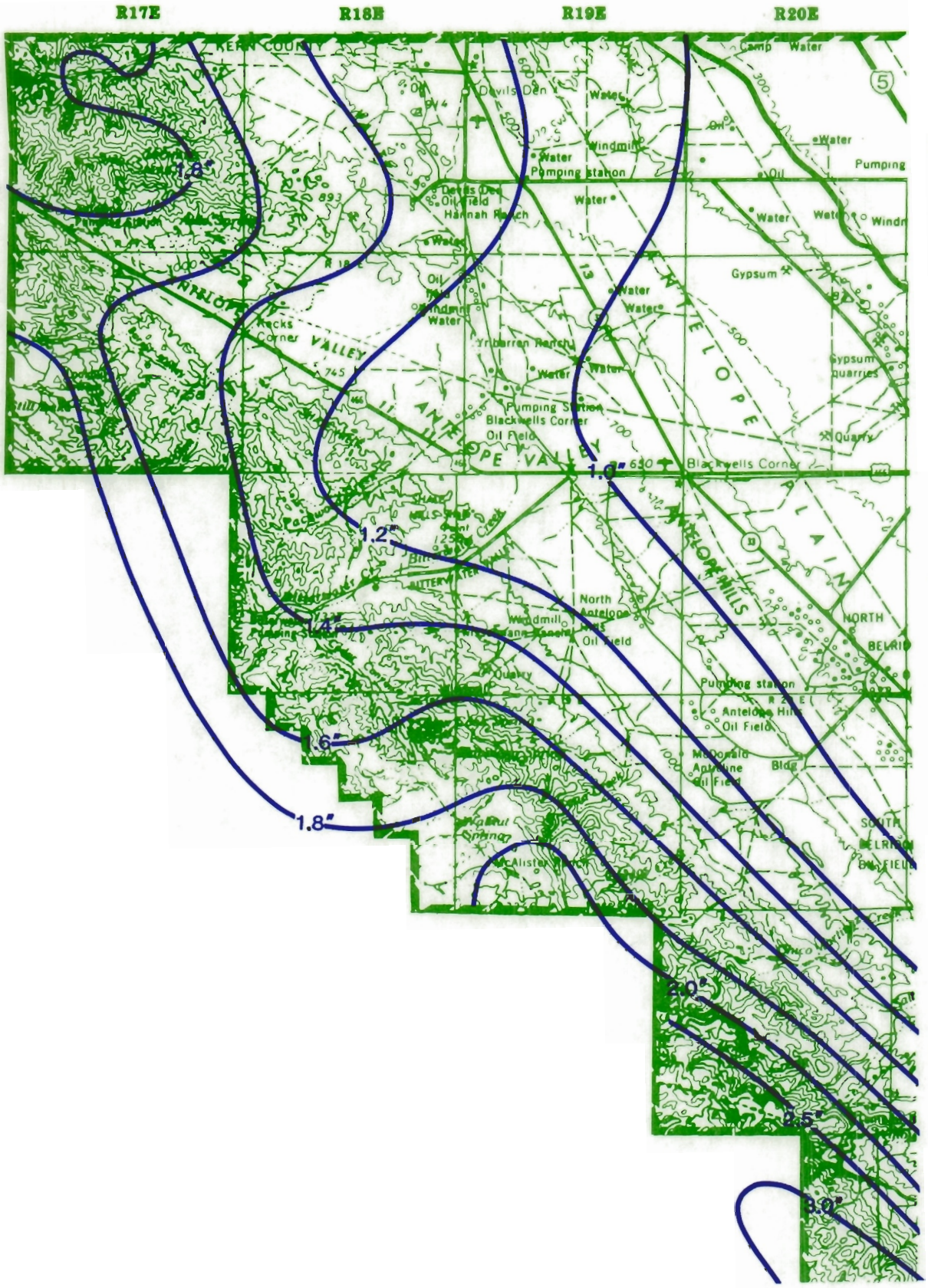
RAINFALL ISOHYET MAP

APPROXIMATE SCALE
1"=4 miles

2-Year, 6-Hour Isohyet

NORTH

FIGURE B-11



KERN COUNTY HYDROLOGY MANUAL

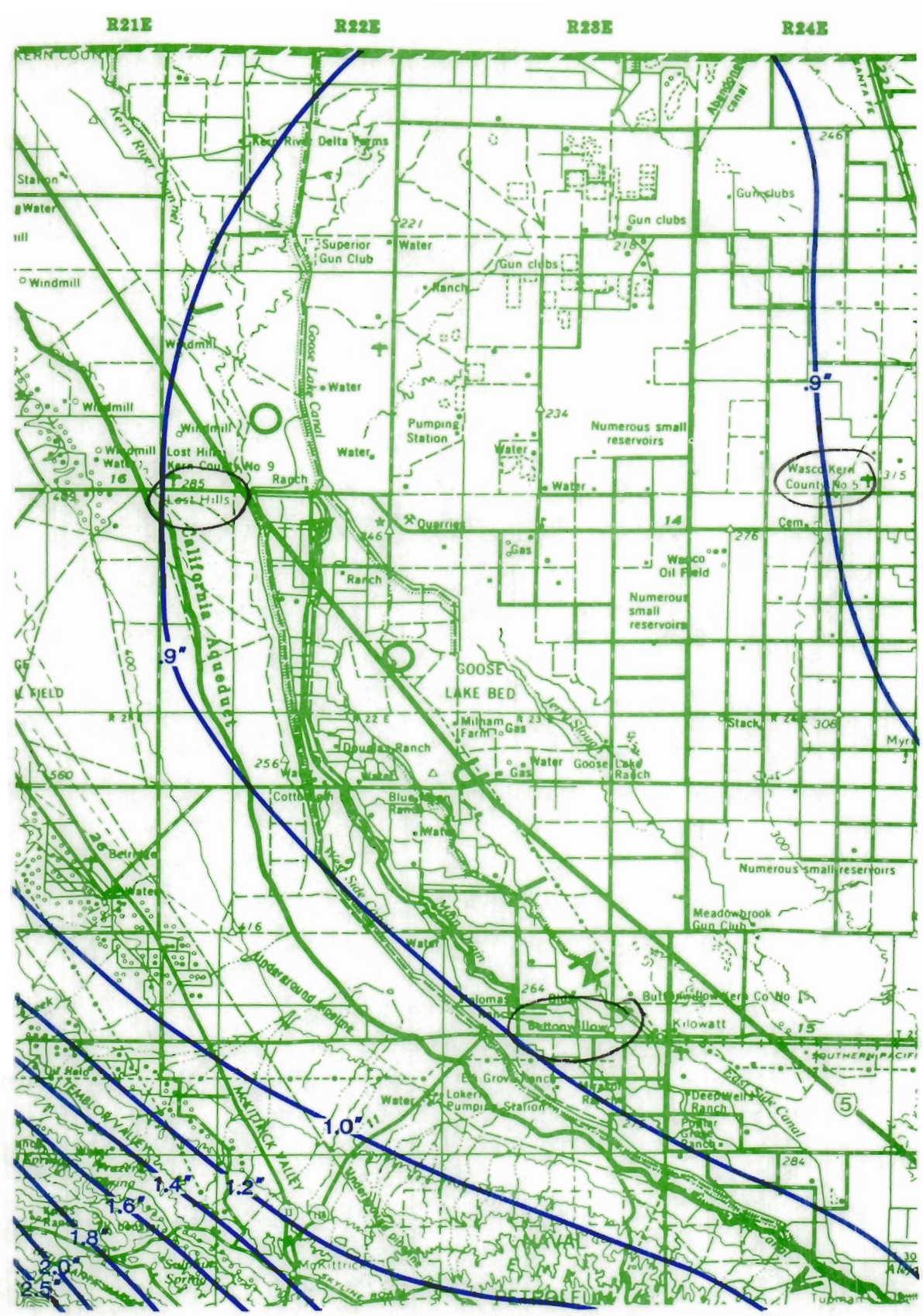
RAINFALL ISOHYET MAP

APPROXIMATE SCALE
1"=4 miles

2-Year, 24-Hour Isohyet

NORTH

FIGURE B-12



KERN COUNTY HYDROLOGY MANUAL

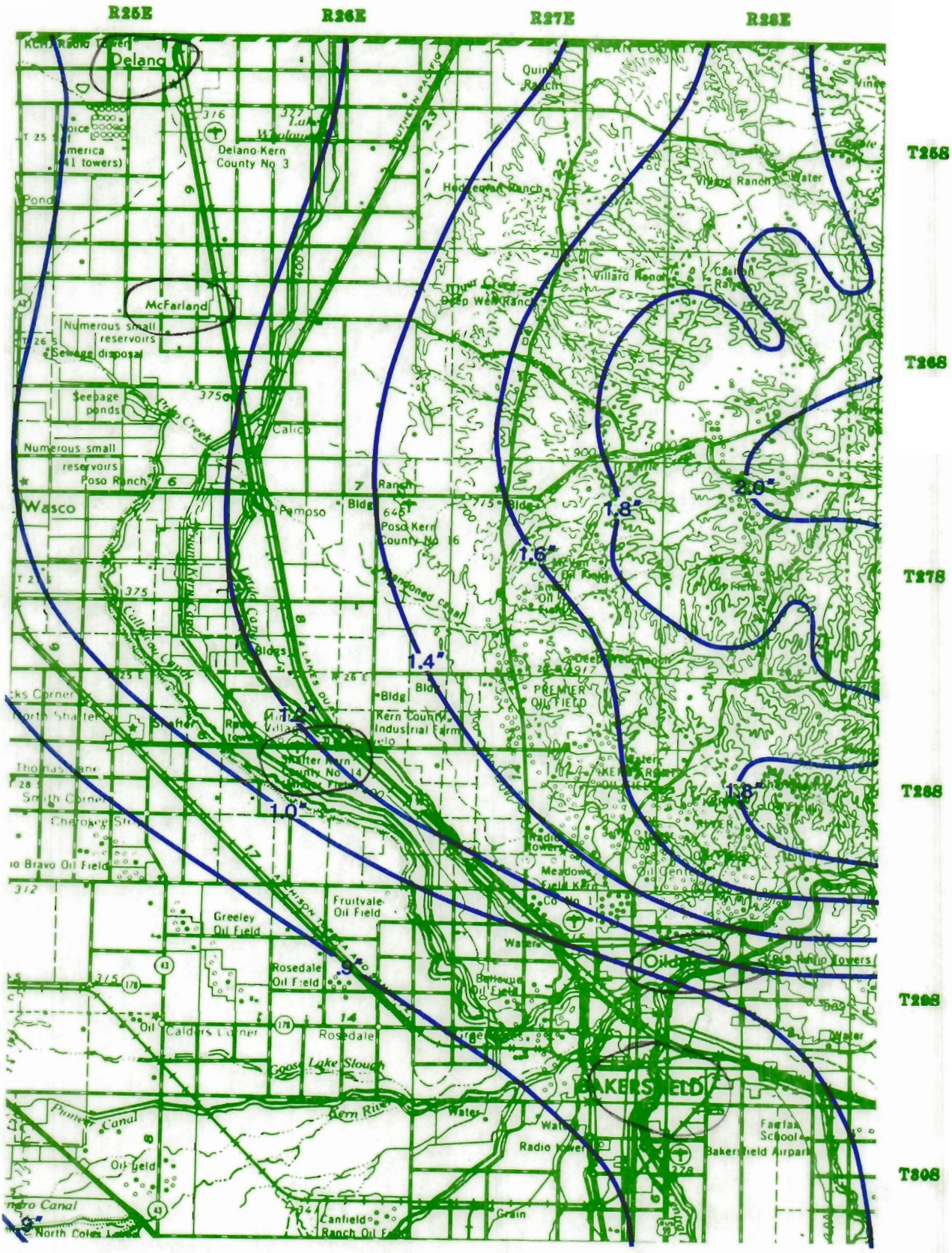
RAINFALL ISOHYET MAP

APPROXIMATE SCALE
1"=4 miles

2-Year, 24-Hour Isohyet

NORTH

FIGURE B-13



KERN COUNTY HYDROLOGY MANUAL

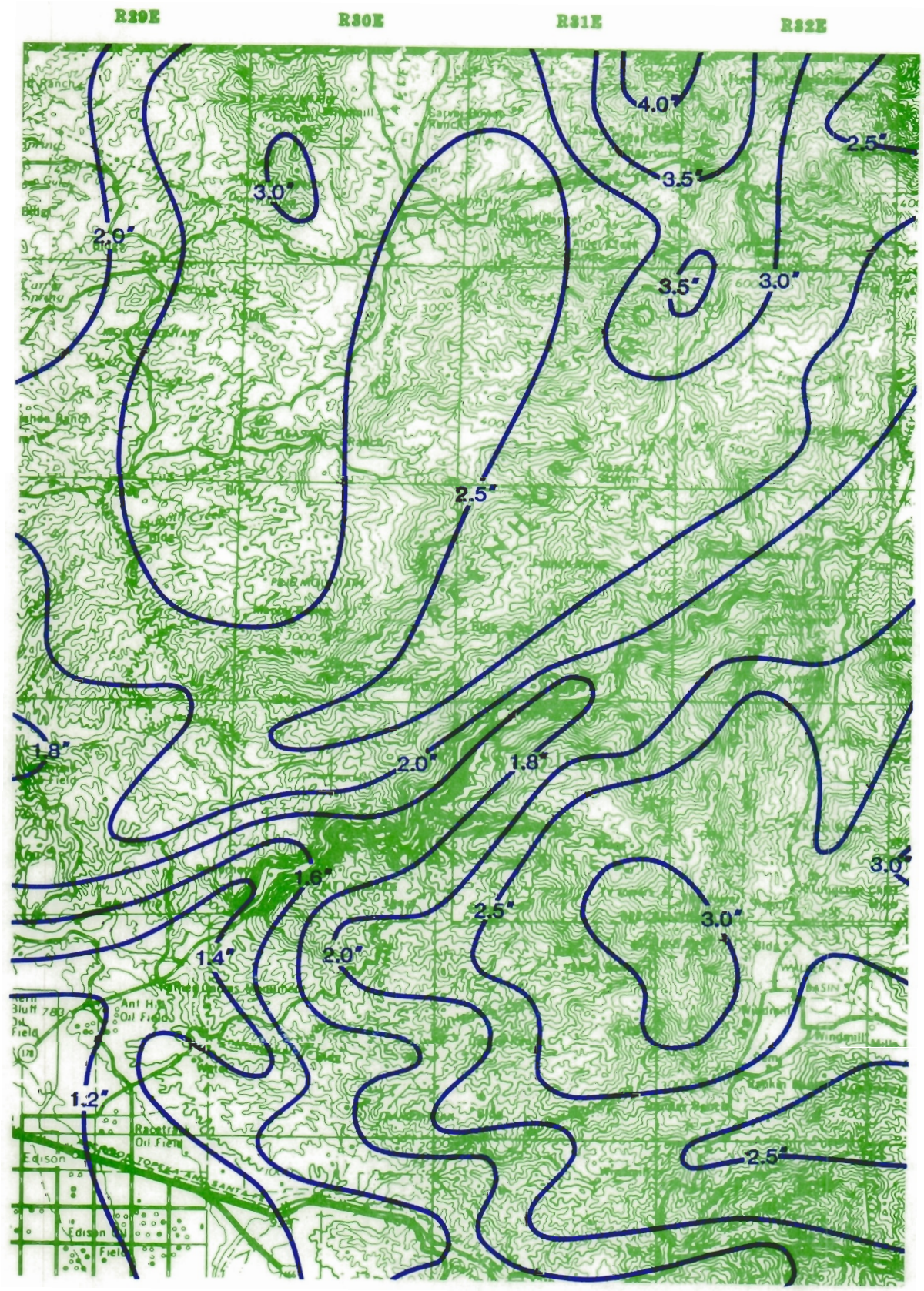
RAINFALL ISOHYET MAP

APPROXIMATE SCALE
1"=4 miles

2-Year, 24-Hour Isohyet

NORTH

FIGURE B-14



KERN COUNTY HYDROLOGY MANUAL

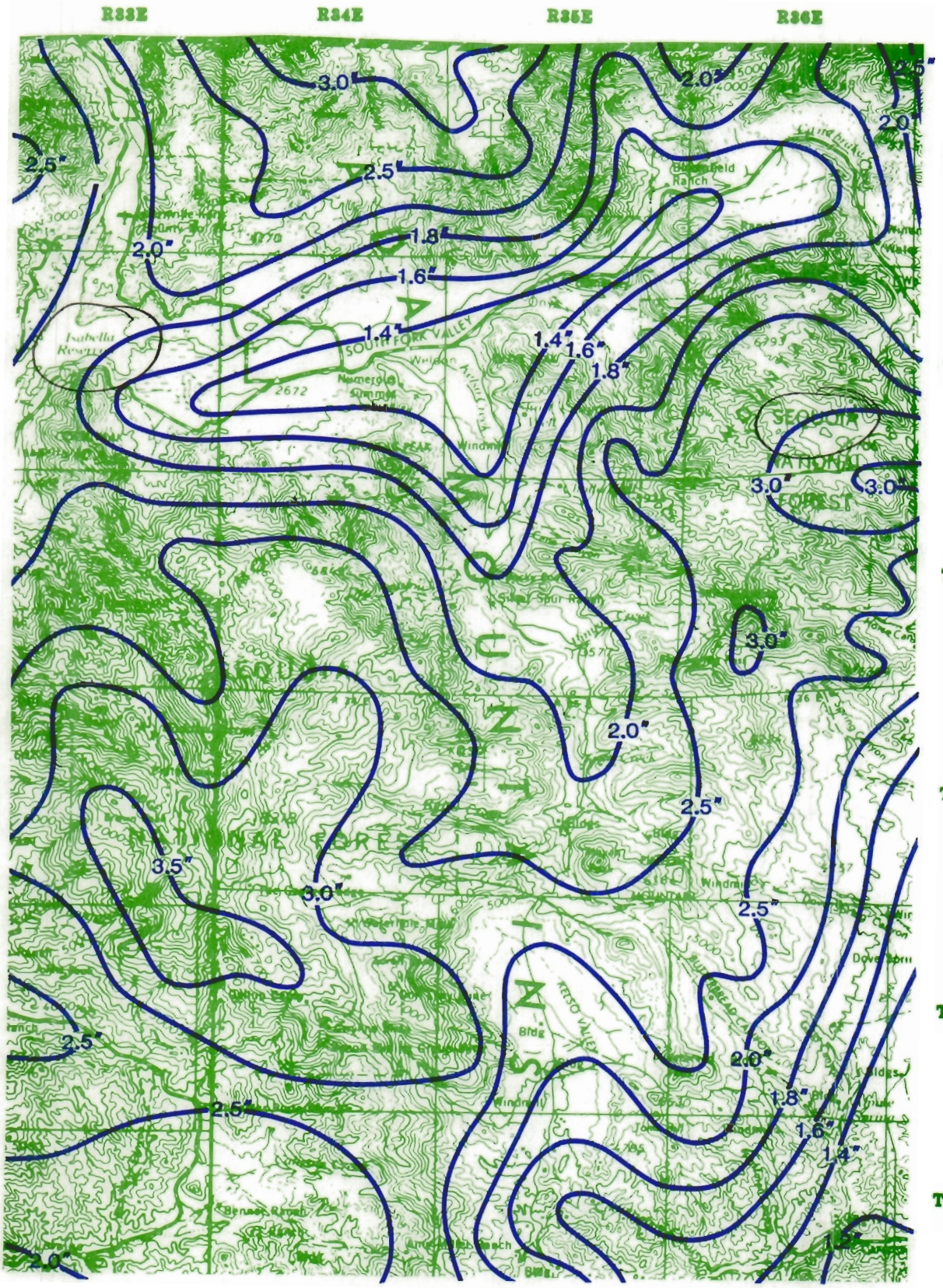
RAINFALL ISOHYET MAP

APPROXIMATE SCALE
1"=4 miles

2-Year, 24-Hour Isohyet

NORTH

FIGURE B-15



KERN COUNTY HYDROLOGY MANUAL

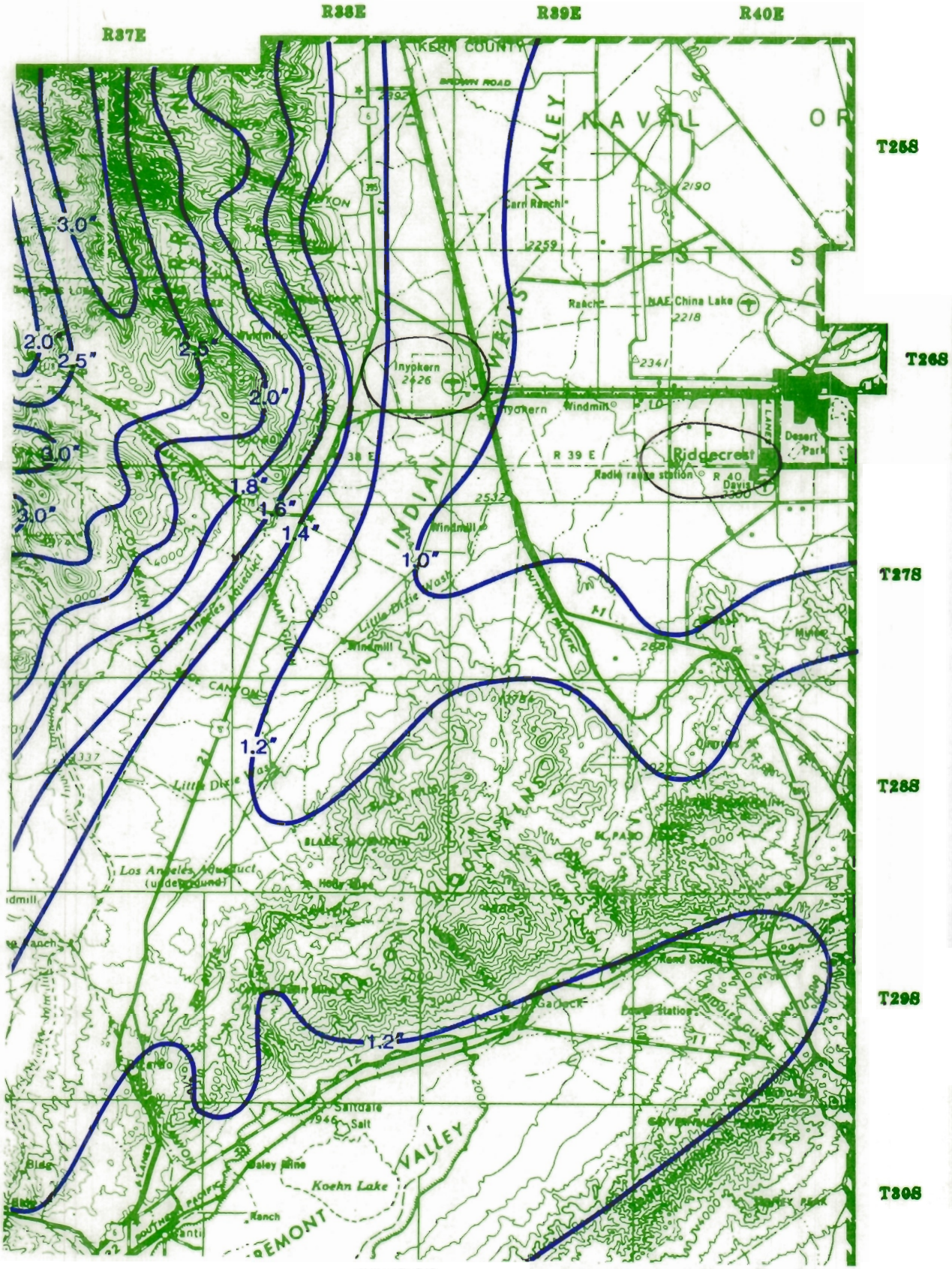
RAINFALL ISOHYET MAP

APPROXIMATE SCALE
1"=4 miles

2-Year, 24-Hour Isohyet

NORTH

FIGURE B-16



KERN COUNTY HYDROLOGY MANUAL

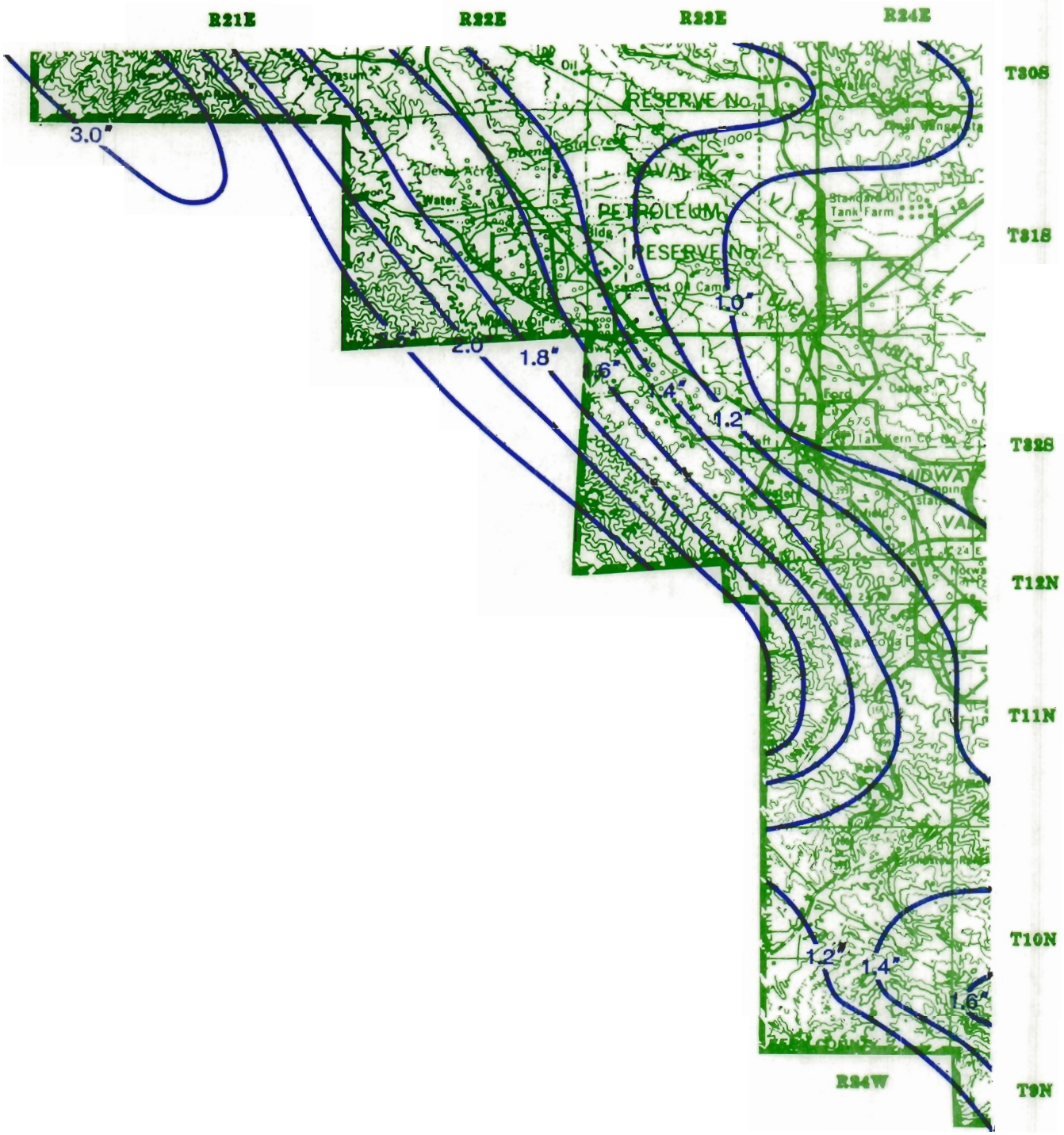
RAINFALL ISOHYET MAP

APPROXIMATE SCALE
1"=4 miles

2-Year, 24-Hour Isohyet

NORTH

FIGURE B-17



KERN COUNTY HYDROLOGY MANUAL

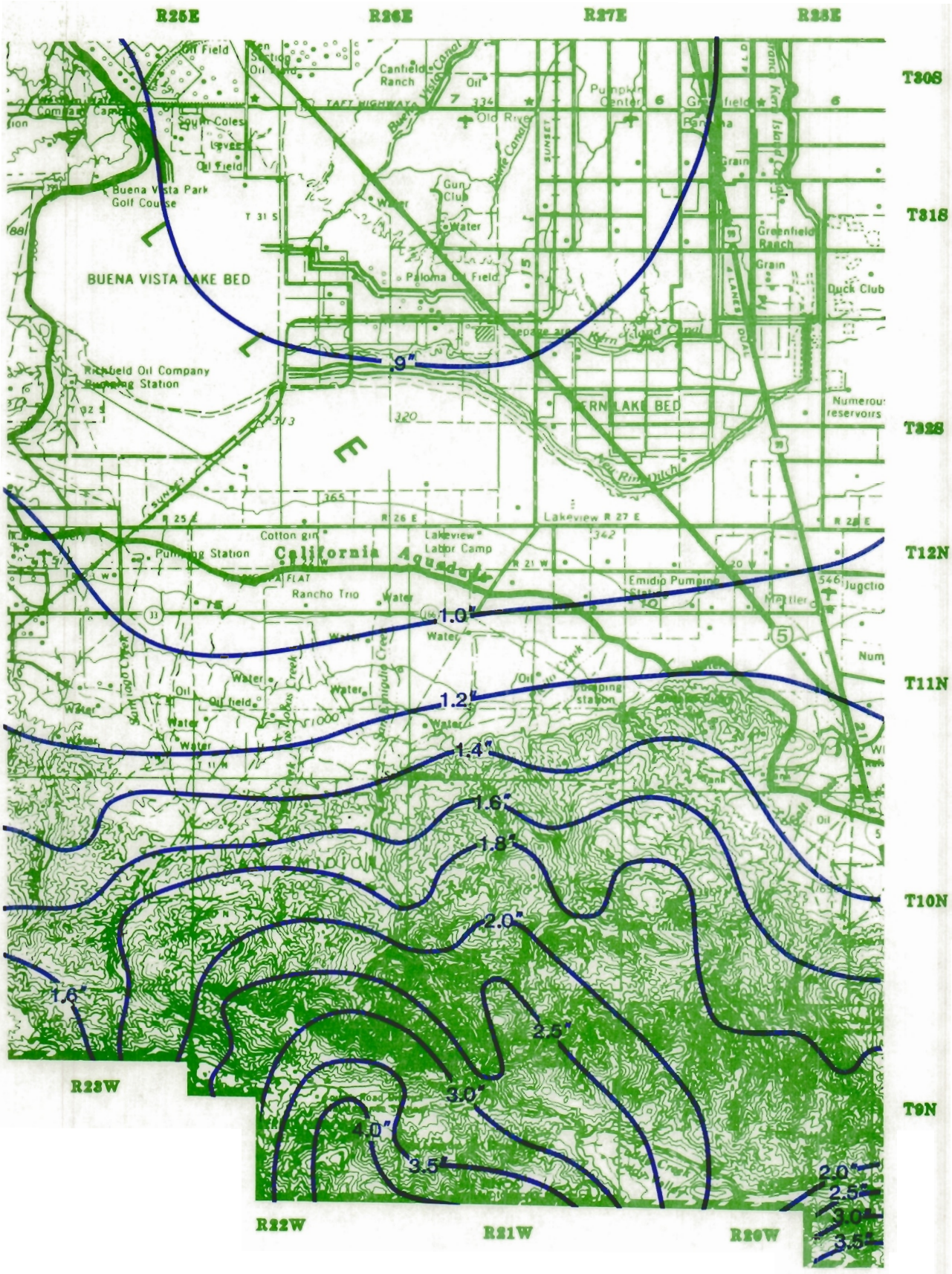
RAINFALL ISOHYET MAP

APPROXIMATE SCALE
1"=4 miles

2-Year, 24-Hour Isohyet

NORTH

FIGURE B-18



KERN COUNTY HYDROLOGY MANUAL

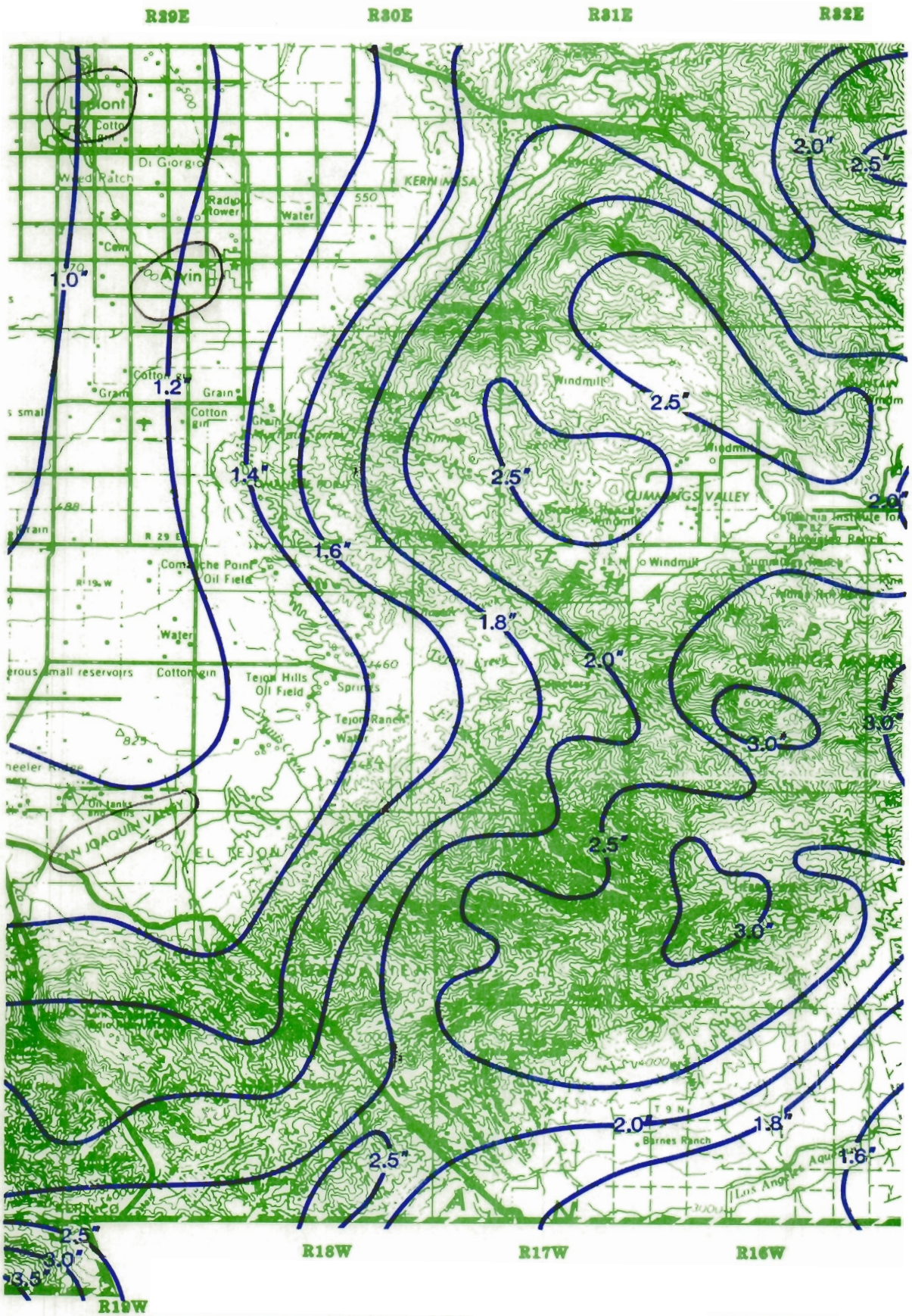
RAINFALL ISOHYET MAP

APPROXIMATE SCALE
1"=4 miles

2-Year, 24-Hour Isohyet

NORTH

FIGURE B-19



KERN COUNTY HYDROLOGY MANUAL

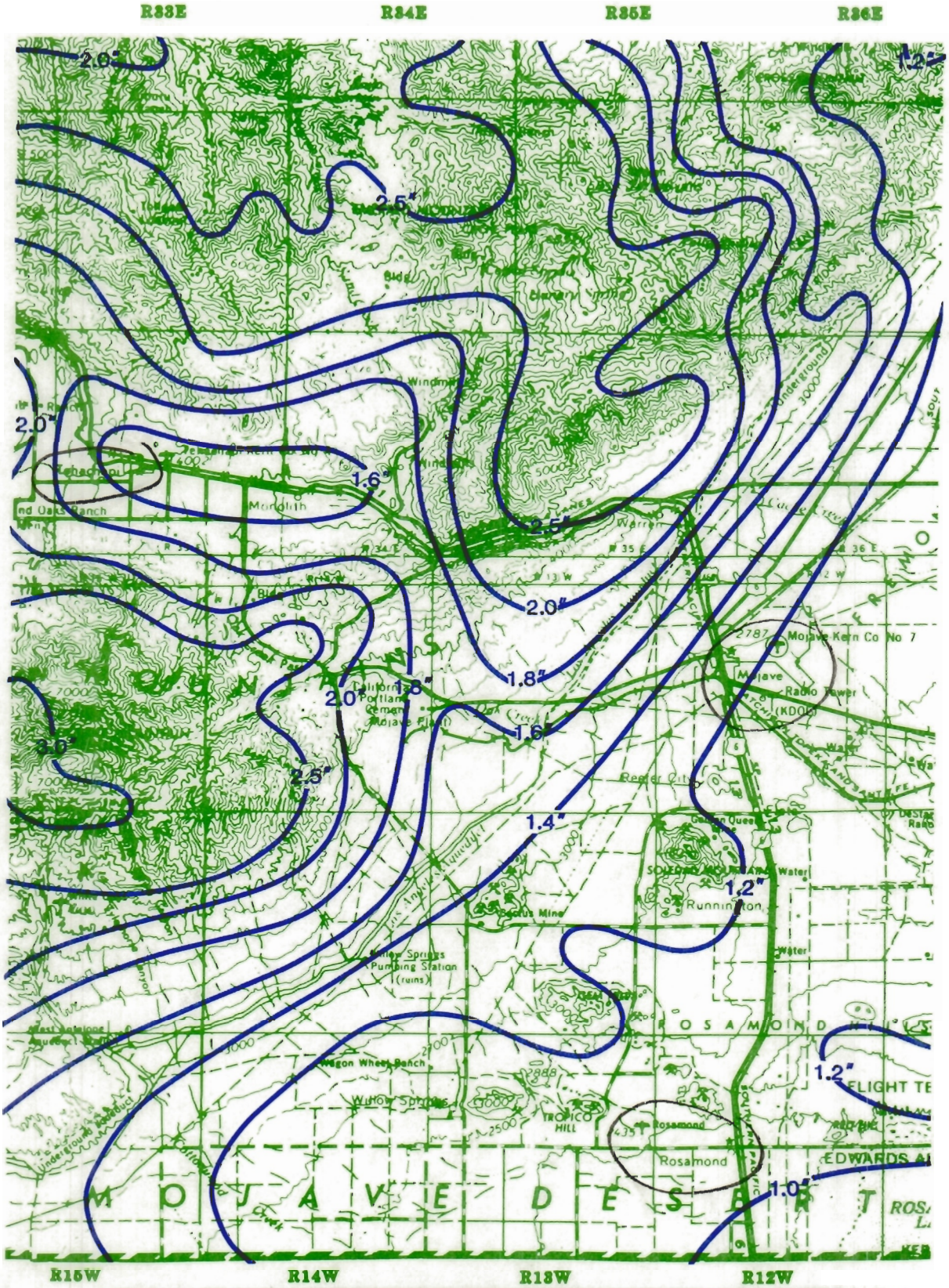
RAINFALL ISOHYET MAP

APPROXIMATE SCALE
1"=4 miles

2-Year, 24-Hour Isohyet

NORTH

FIGURE B-20



KERN COUNTY HYDROLOGY MANUAL

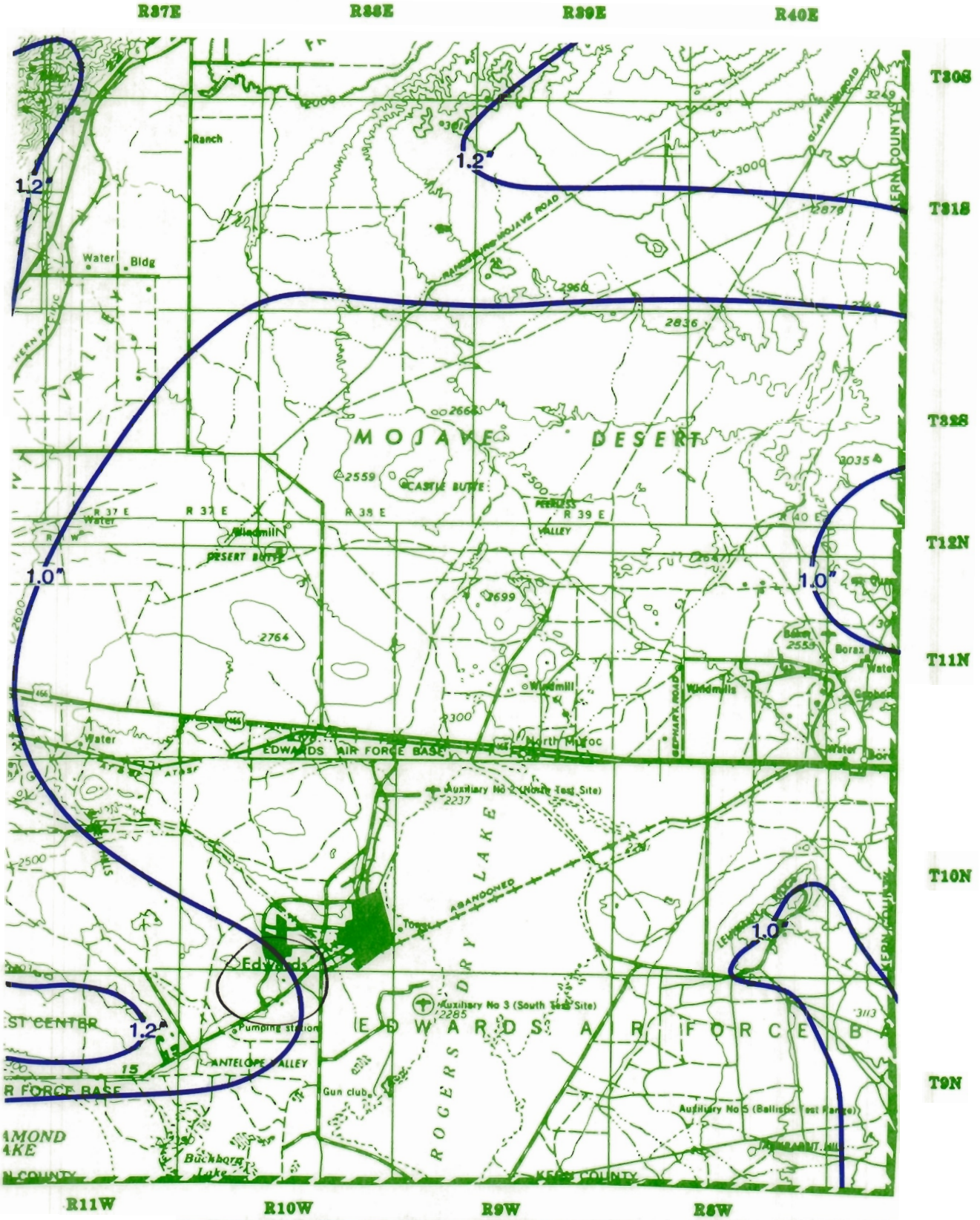
RAINFALL ISOHYET MAP

APPROXIMATE SCALE
1"=4 miles

2-Year, 24-Hour Isohyet

NORTH

FIGURE B-21



KERN COUNTY HYDROLOGY MANUAL

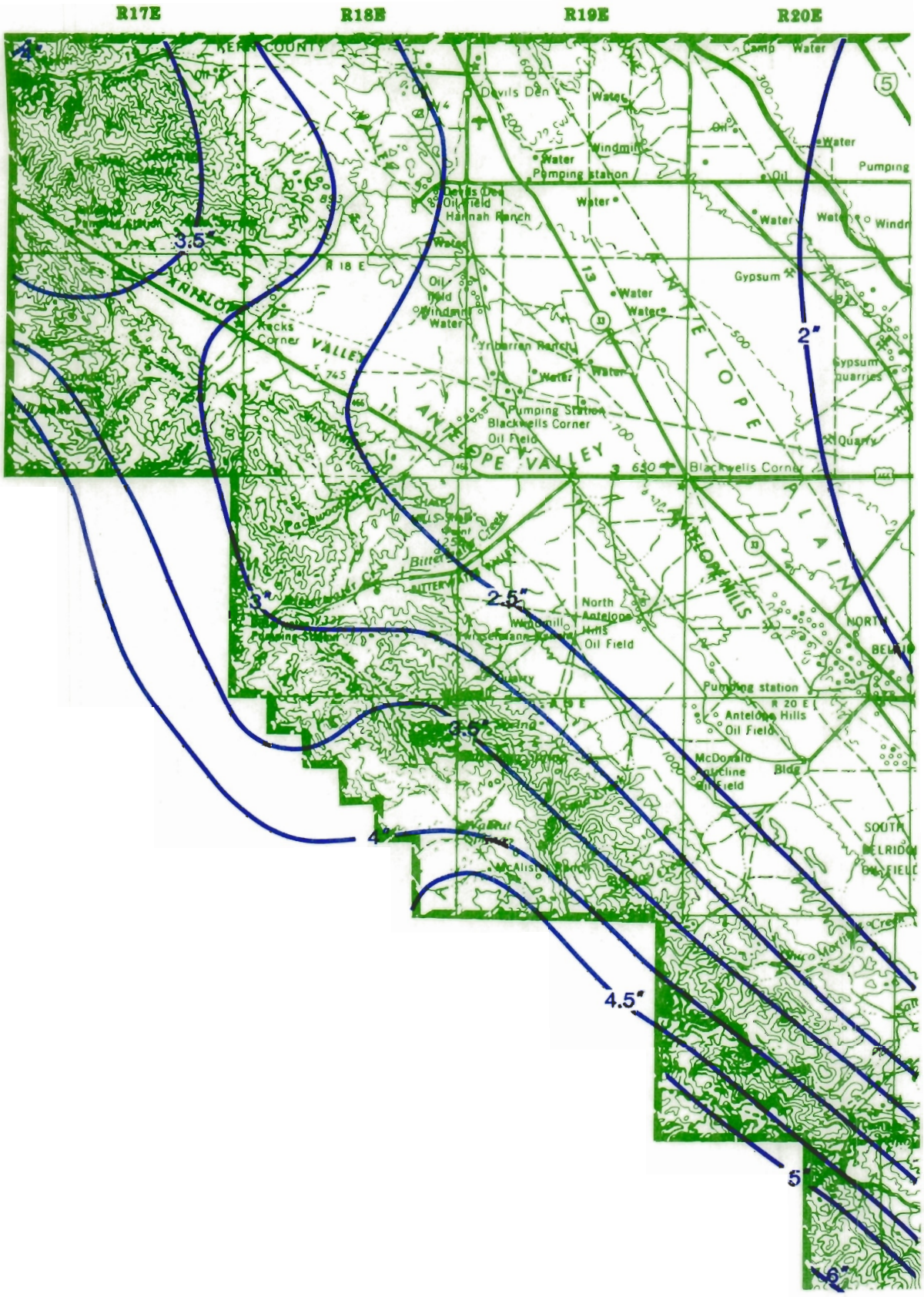
RAINFALL ISOHYET MAP

APPROXIMATE SCALE
1"=4 miles

2-Year, 24-Hour Isohyet

NORTH

FIGURE B- 22



KERN COUNTY HYDROLOGY MANUAL

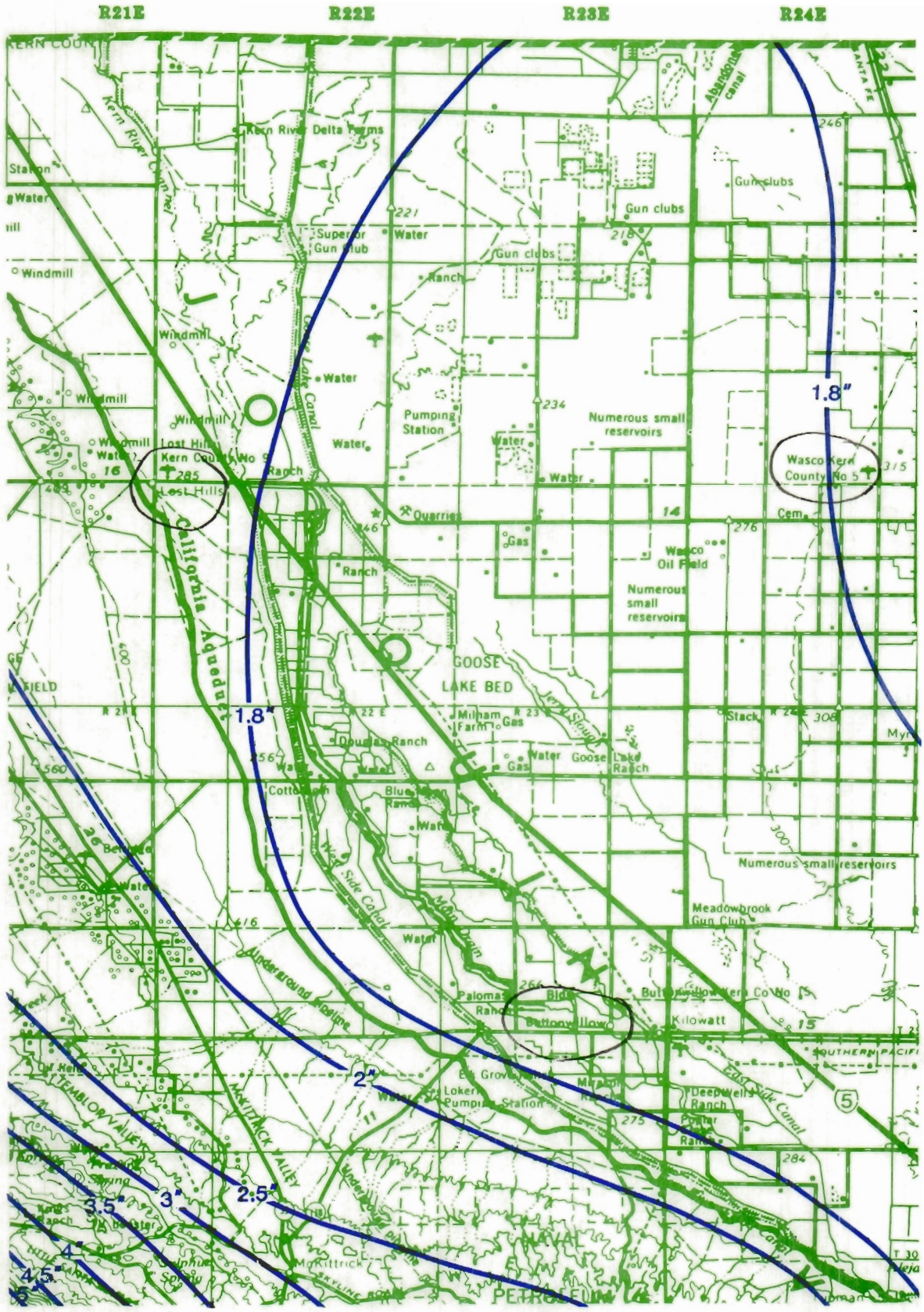
RAINFALL ISOHYET MAP

APPROXIMATE SCALE
1"=4 miles

50-Year, 24-Hour Isohyet

NORTH

FIGURE B-23



KERN COUNTY HYDROLOGY MANUAL

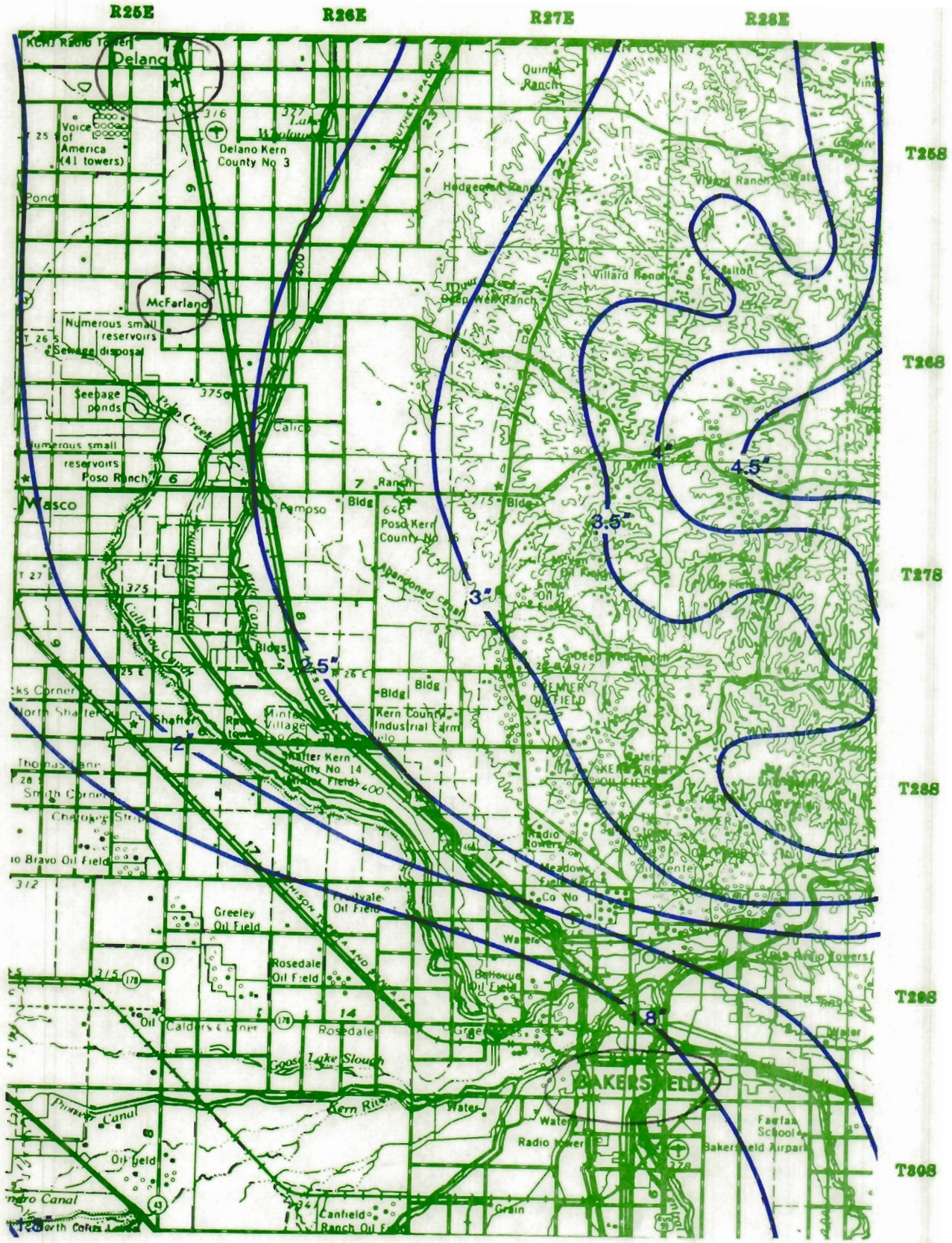
RAINFALL ISOHYET MAP

APPROXIMATE SCALE
1"=4 miles

——— 50-Year, 24-Hour Isohyet

NORTH

FIGURE B-24



KERN COUNTY HYDROLOGY MANUAL

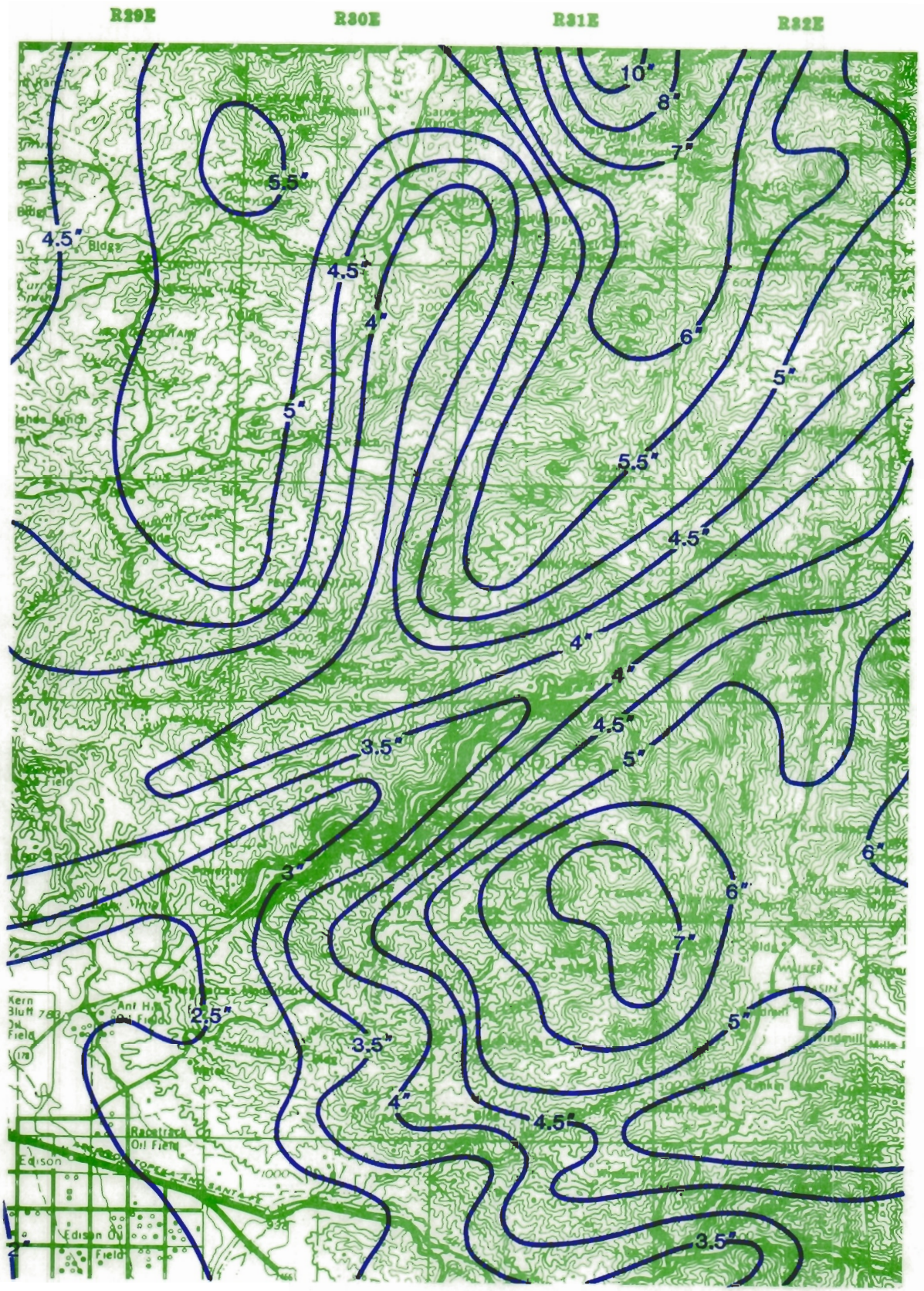
RAINFALL ISOHYET MAP

APPROXIMATE SCALE
1"=4 miles

——— 50-Year, 24-Hour Isohyet

NORTH

FIGURE B-25



KERN COUNTY HYDROLOGY MANUAL

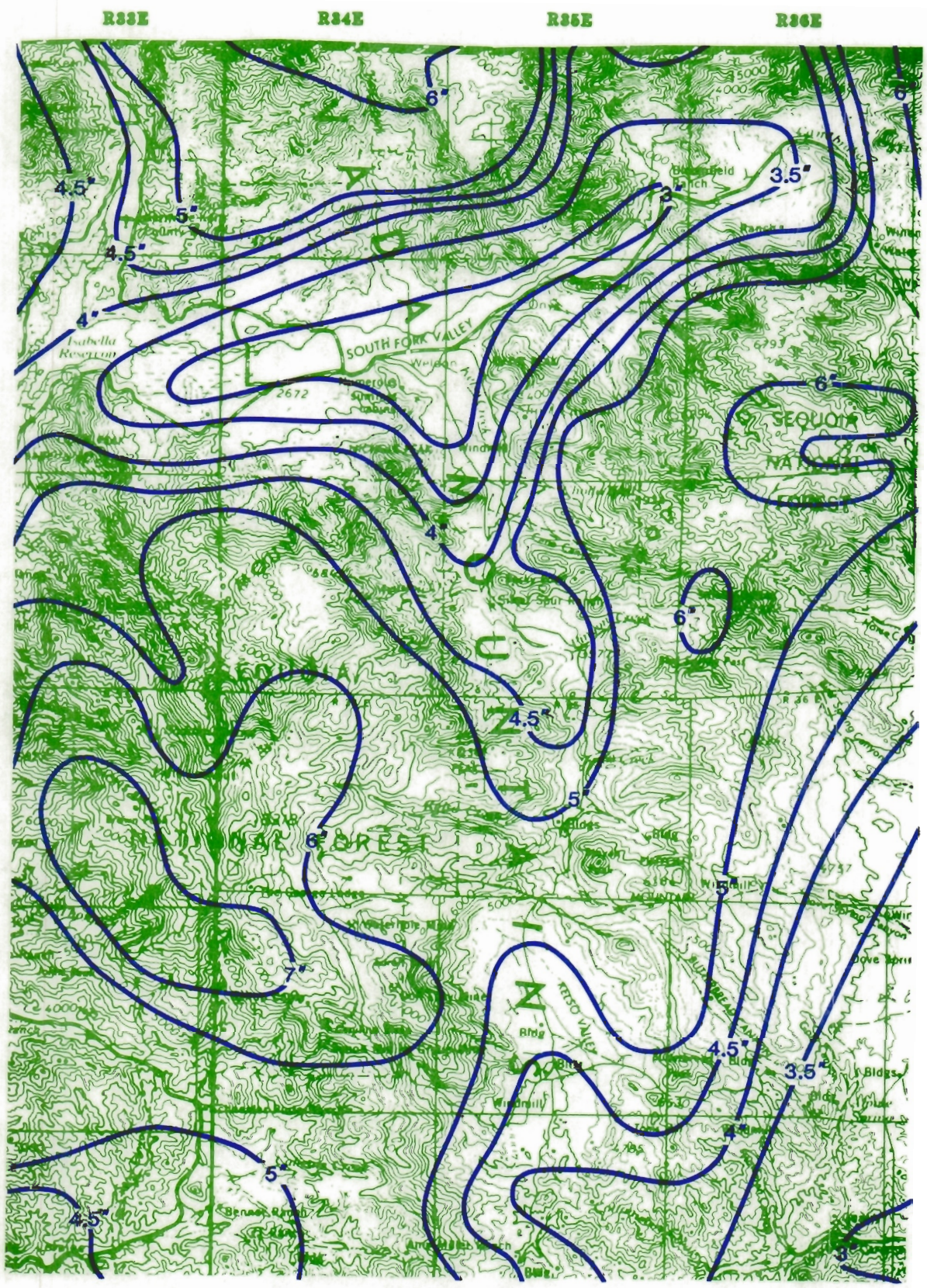
RAINFALL ISOHYET MAP

APPROXIMATE SCALE
1"=4 miles

——— 50-Year, 24-Hour Isohyet

NORTH

FIGURE B-26



KERN COUNTY HYDROLOGY MANUAL

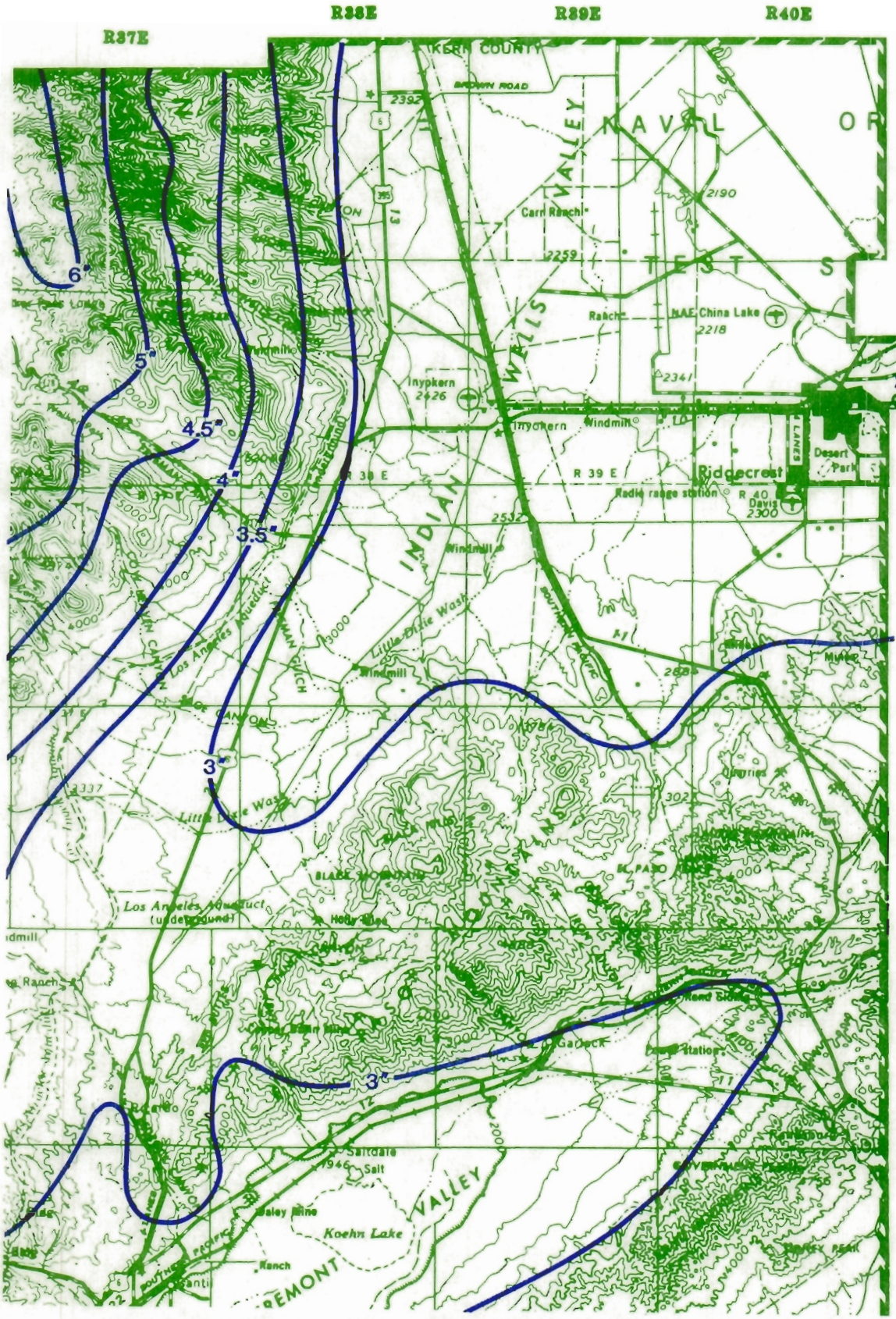
RAINFALL ISOHYET MAP

APPROXIMATE SCALE
1"=4 miles

——— 50--Year, 24-Hour Isohyet

NORTH

FIGURE B-27



KERN COUNTY HYDROLOGY MANUAL

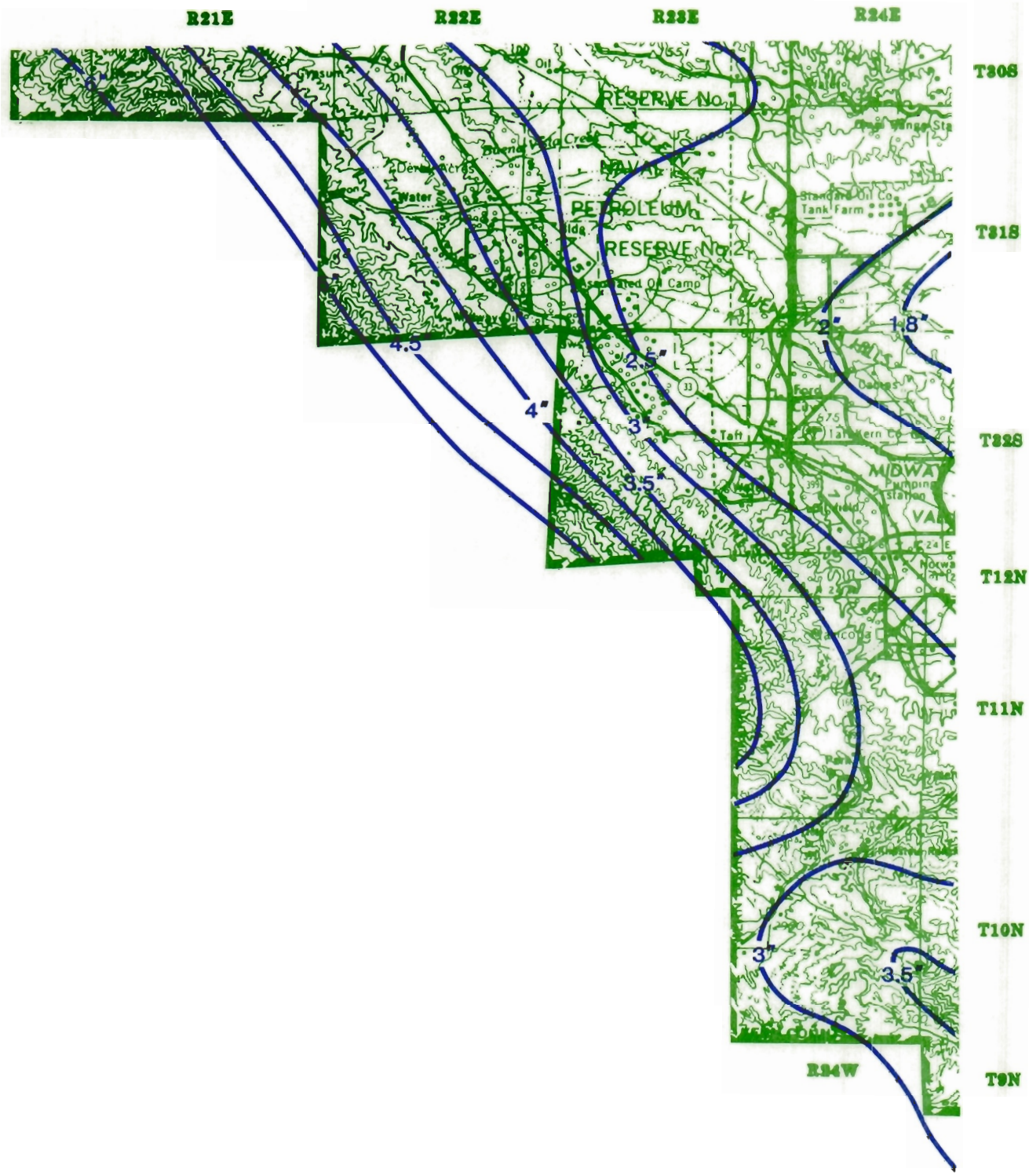
RAINFALL ISOHYET MAP

APPROXIMATE SCALE
1"=4 miles

50-Year, 24-Hour Isohyet

NORTH

FIGURE B-28



KERN COUNTY HYDROLOGY MANUAL

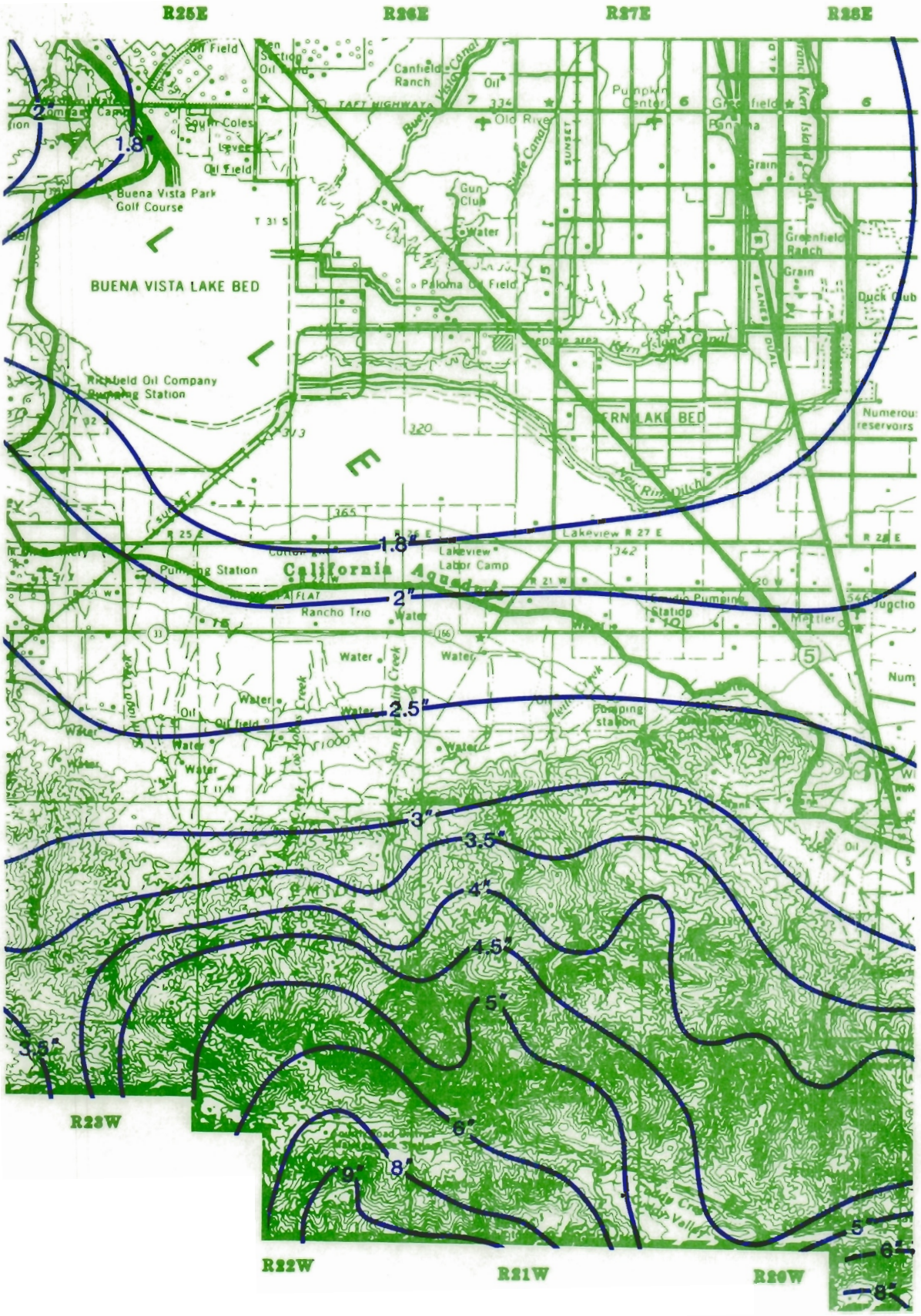
RAINFALL ISOHYET MAP

APPROXIMATE SCALE
1"=4 miles

50-Year, 24-Hour Isohyet

NORTH

FIGURE B-29



KERN COUNTY HYDROLOGY MANUAL

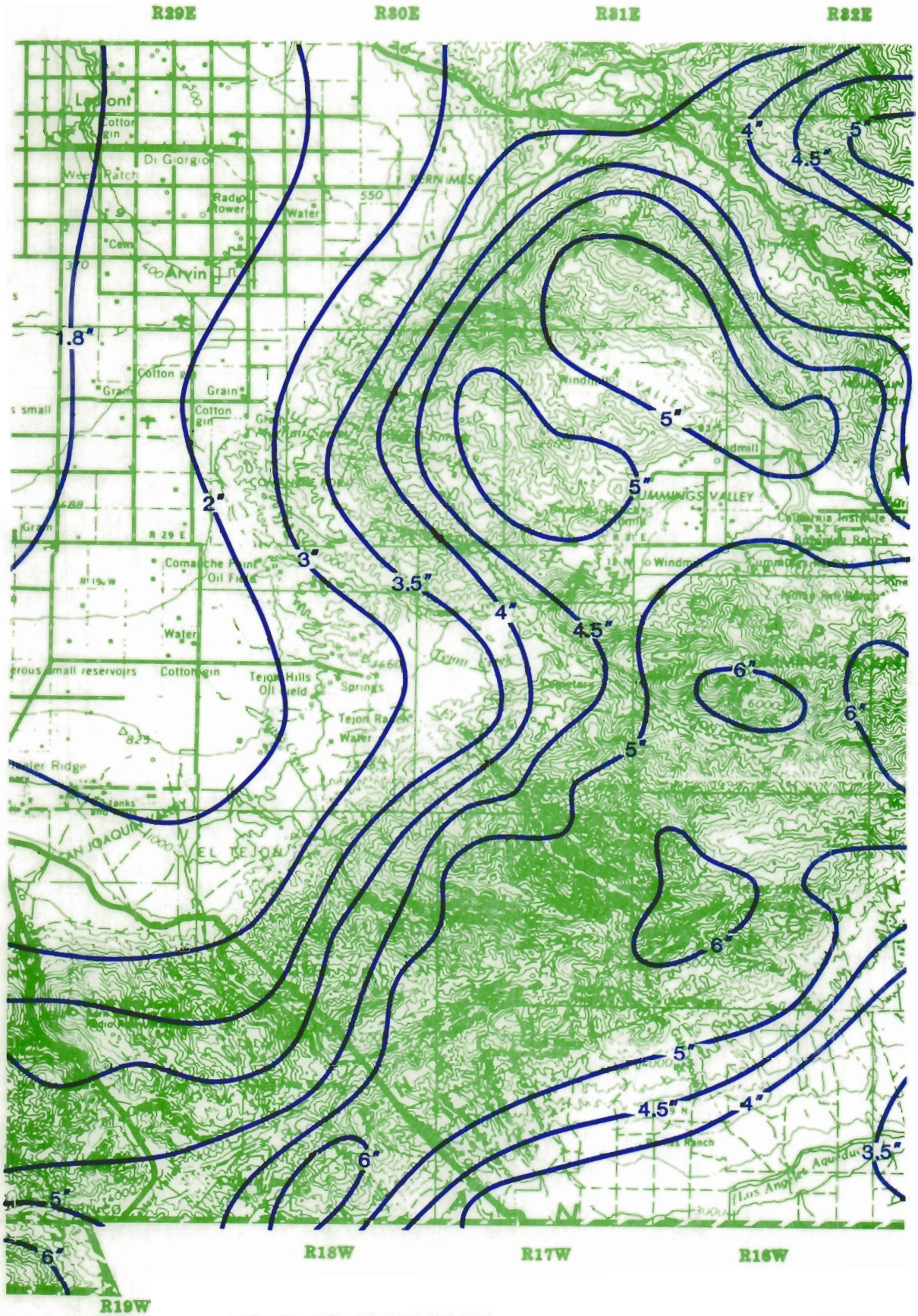
RAINFALL ISOHYET MAP

APPROXIMATE SCALE
1"=4 miles

50-Year, 24-Hour Isohyet

NORTH

FIGURE B-30



KERN COUNTY HYDROLOGY MANUAL

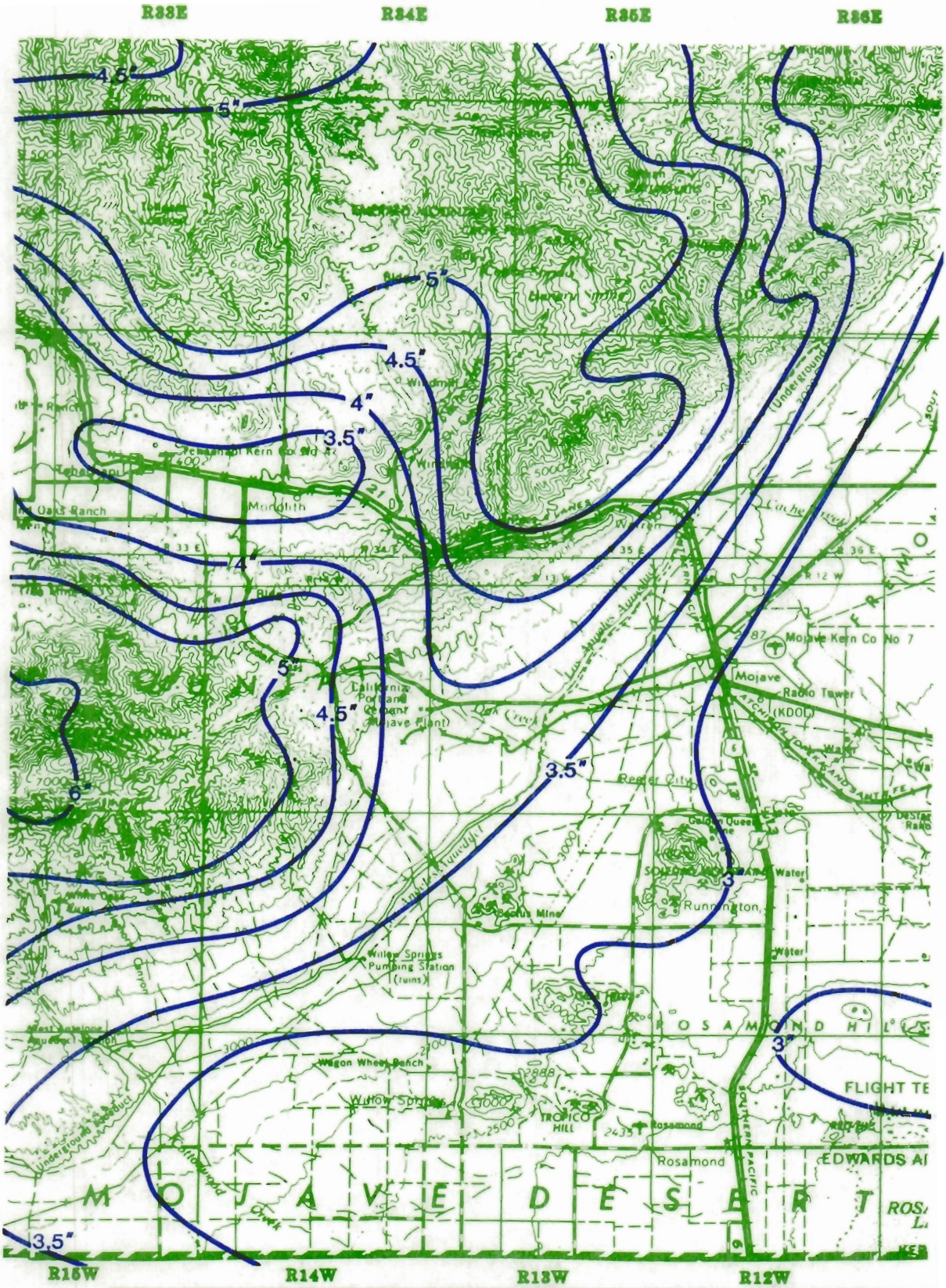
RAINFALL ISOHYET MAP

APPROXIMATE SCALE
1"=4 miles

50-Year, 24-Hour Isohyet

NORTH

FIGURE B-31



KERN COUNTY HYDROLOGY MANUAL

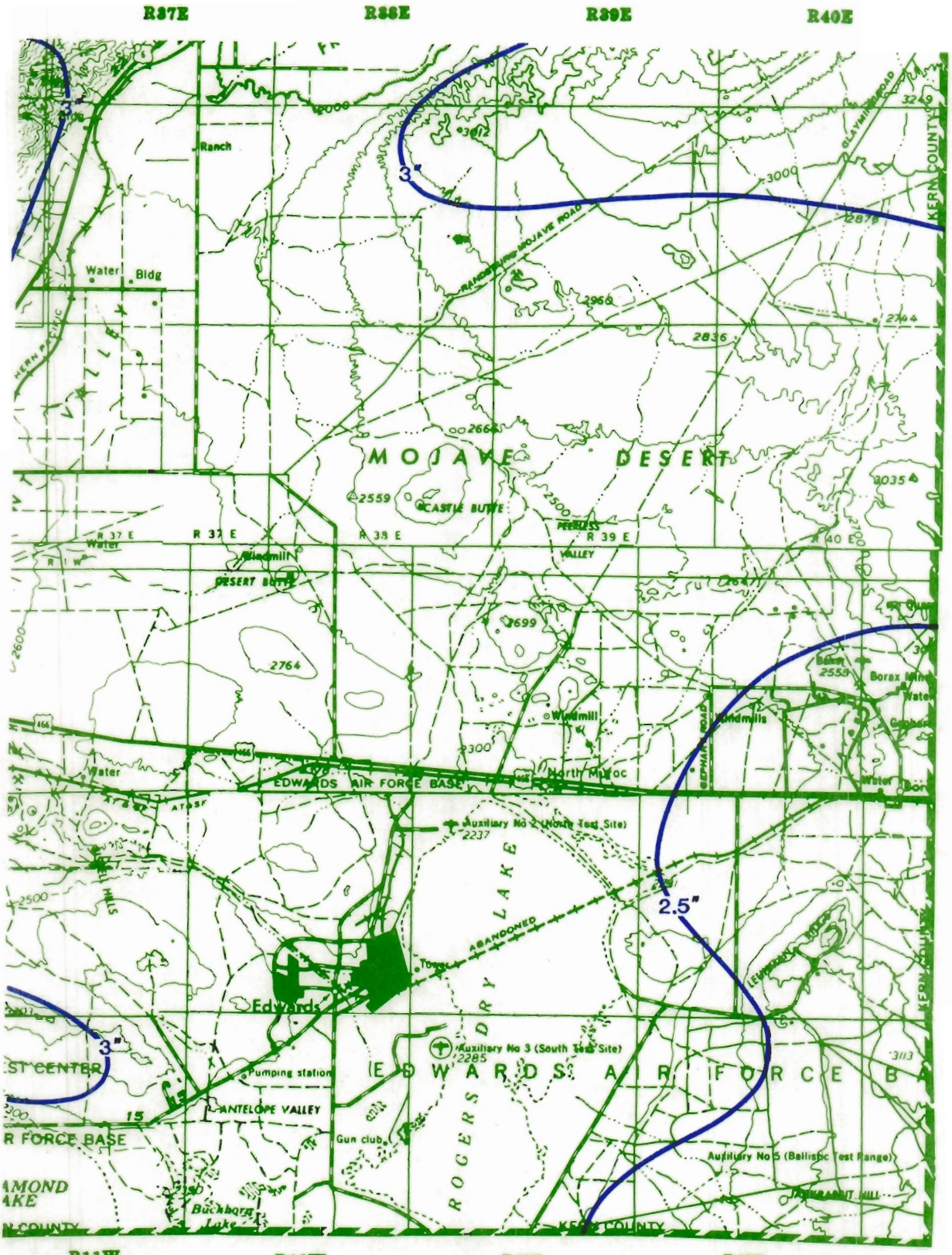
RAINFALL ISOHYET MAP

APPROXIMATE SCALE
1" = 4 miles

50-Year, 24-Hour Isohyet

NORTH

FIGURE B-32



KERN COUNTY HYDROLOGY MANUAL

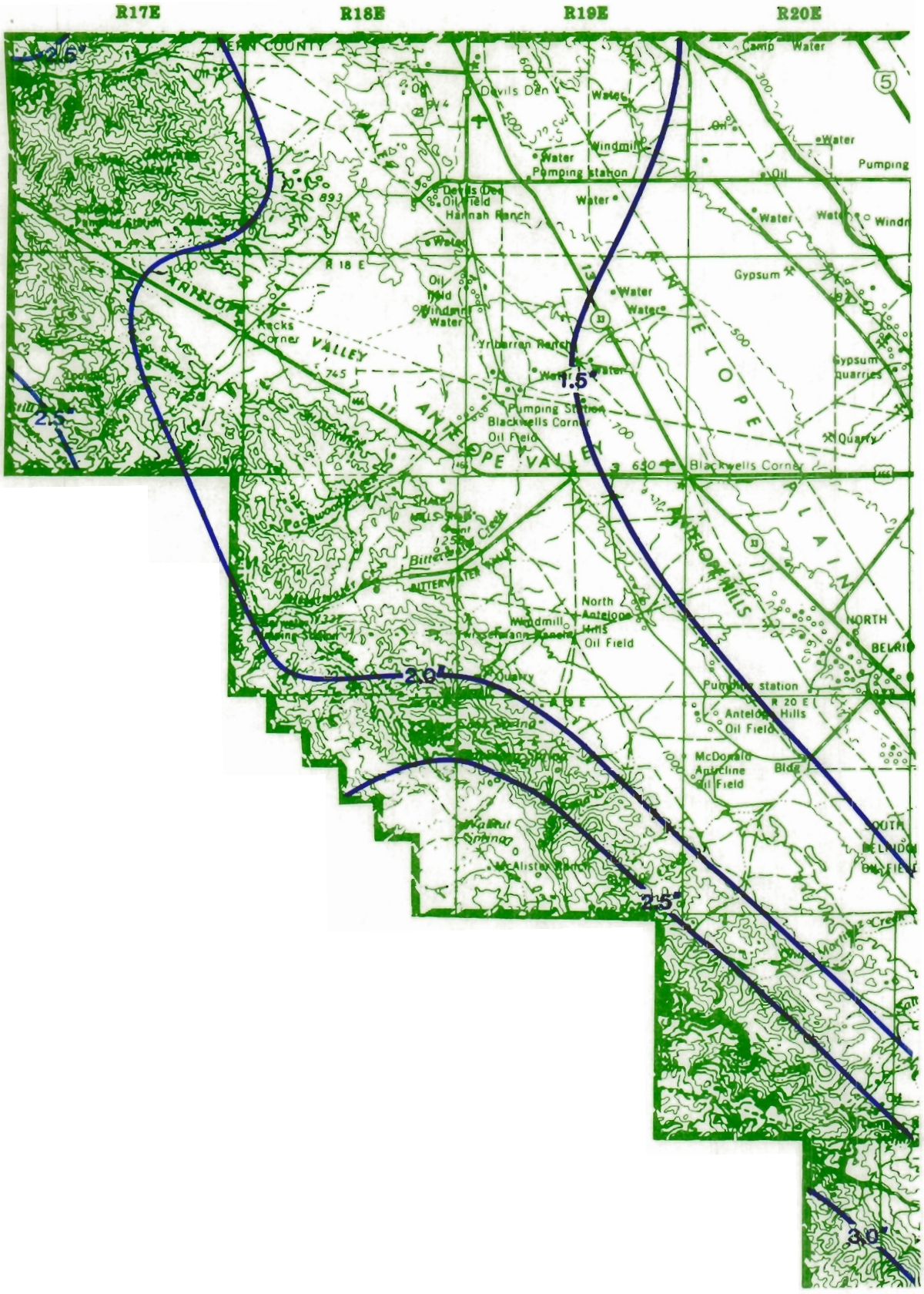
RAINFALL ISOHYET MAP

APPROXIMATE SCALE
1"=4 miles

——— 50-Year, 24-Hour Isohyet

NORTH

FIGURE B-33



KERN COUNTY HYDROLOGY MANUAL

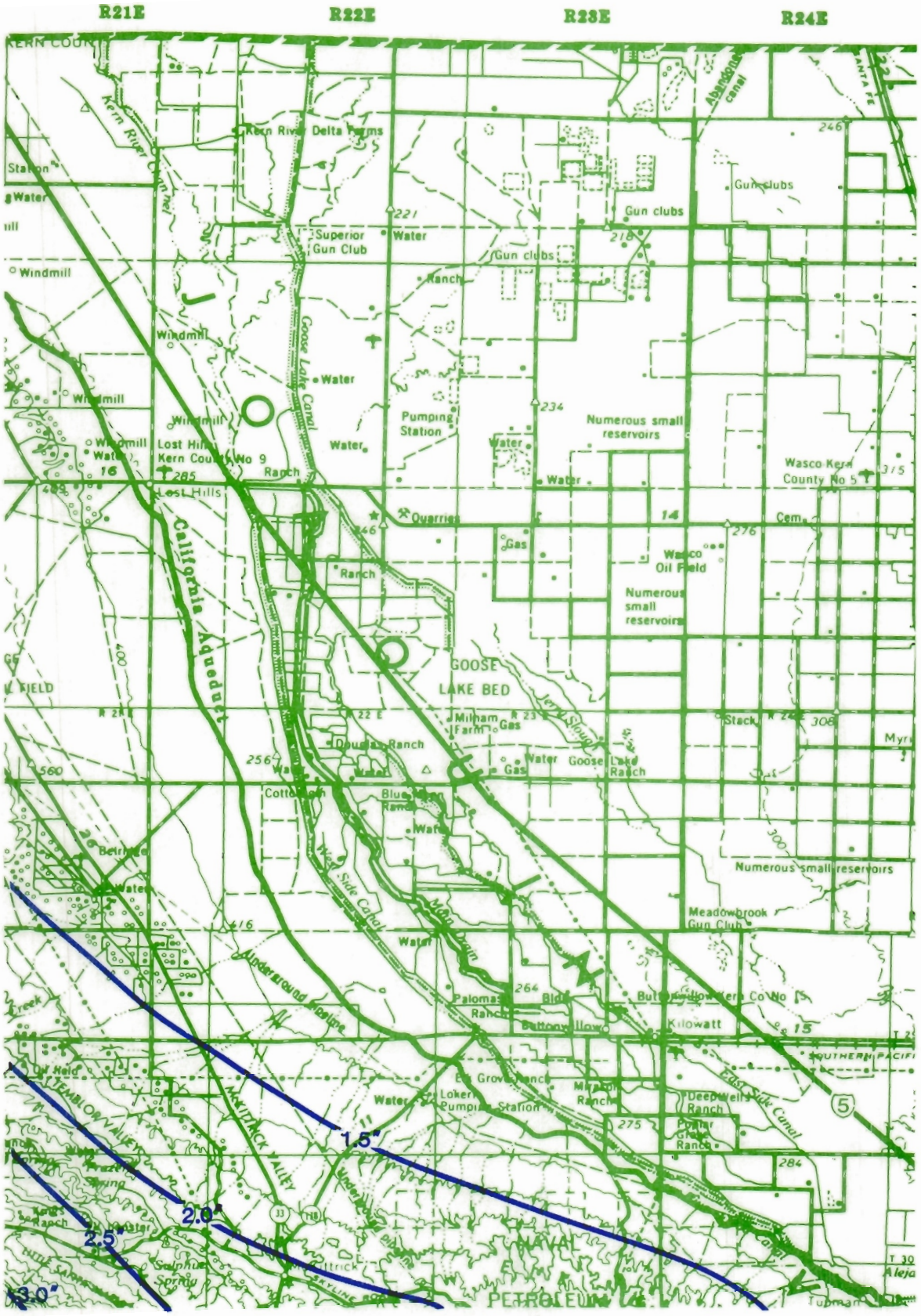
RAINFALL ISOHYET MAP

APPROXIMATE SCALE
1"=4 miles

100-Year, 6-Hour Isohyet

NORTH

FIGURE B-34



KERN COUNTY HYDROLOGY MANUAL

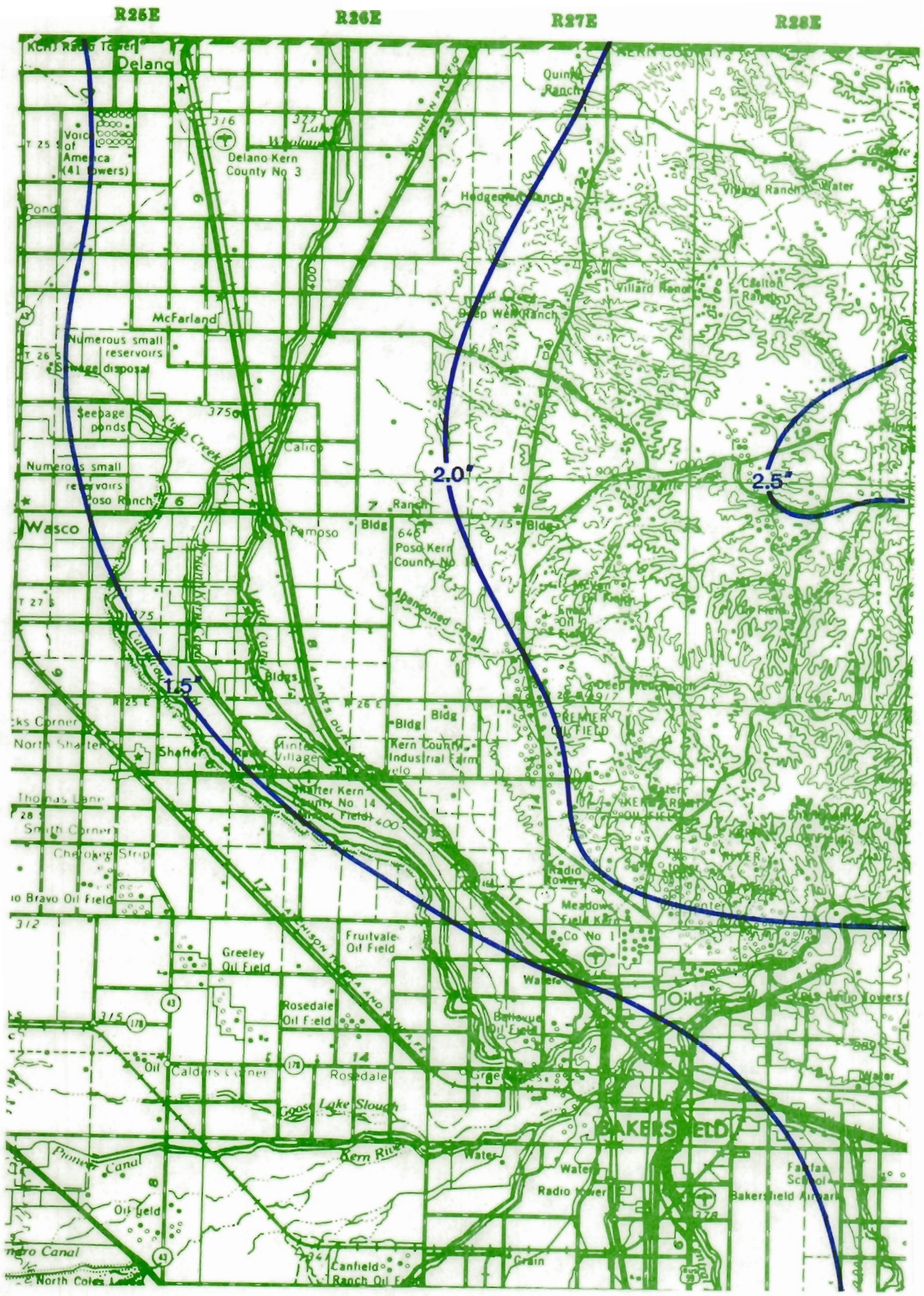
RAINFALL ISOHYET MAP

APPROXIMATE SCALE
1"=4 miles

100-Year, 6-Hour Isohyet

NORTH

FIGURE B-35



KERN COUNTY HYDROLOGY MANUAL

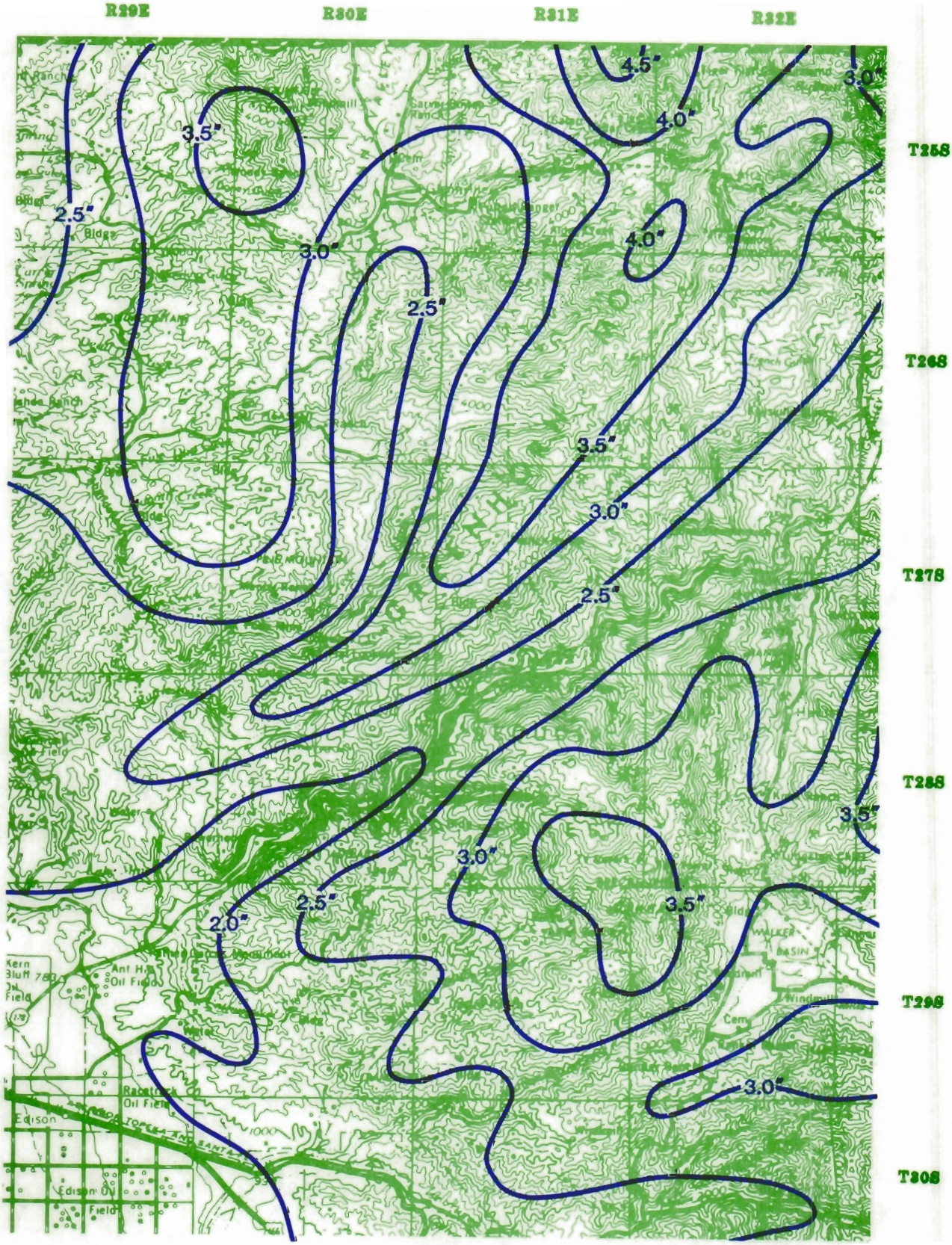
RAINFALL ISOHYET MAP

APPROXIMATE SCALE
1"=4 miles

——— 100-Year, 6-Hour Isohyet

NORTH

FIGURE B-36



KERN COUNTY HYDROLOGY MANUAL

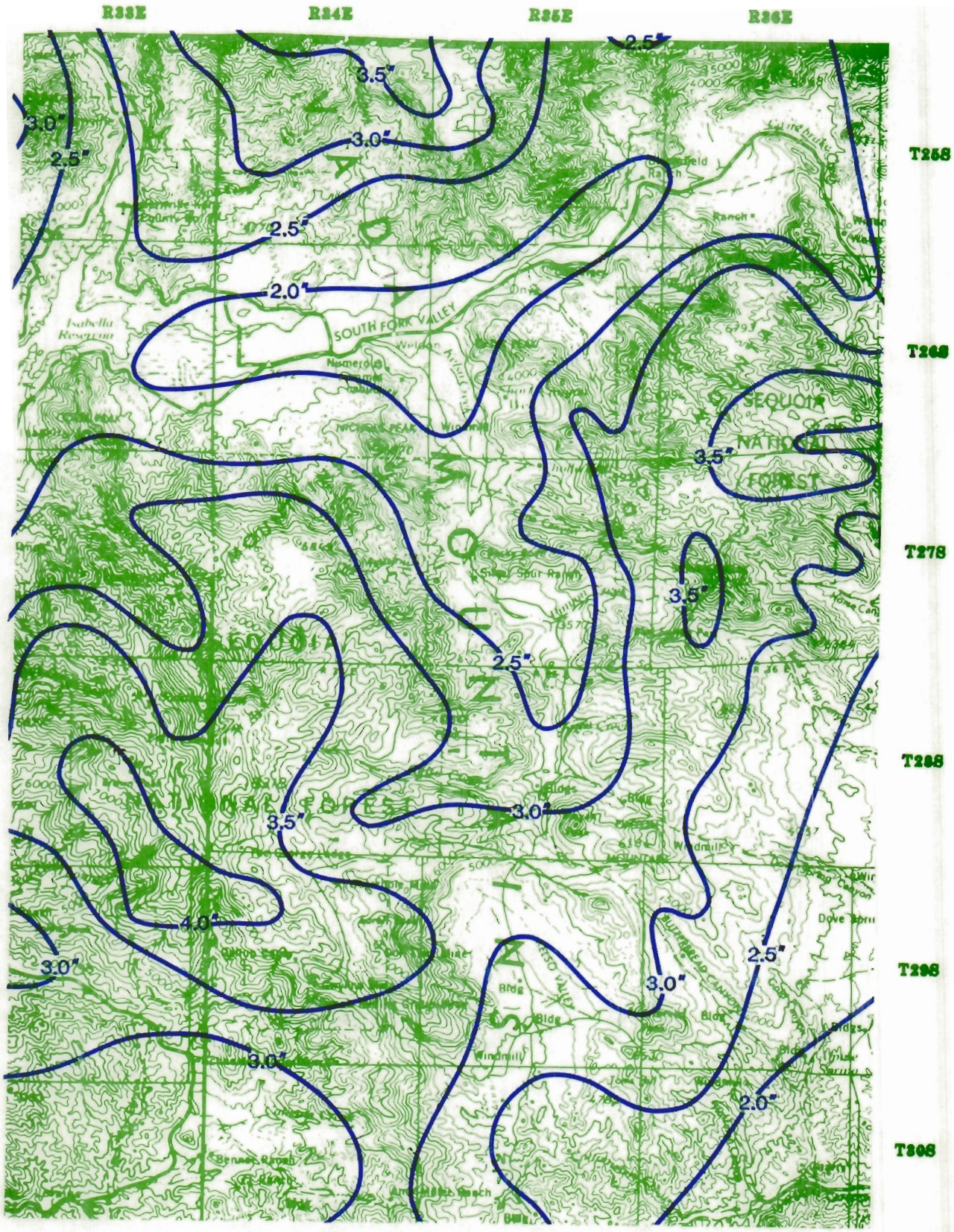
RAINFALL ISOHYET MAP

APPROXIMATE SCALE
1"=4 miles

100-Year, 6-Hour Isohyet

NORTH

FIGURE B-37



KERN COUNTY HYDROLOGY MANUAL

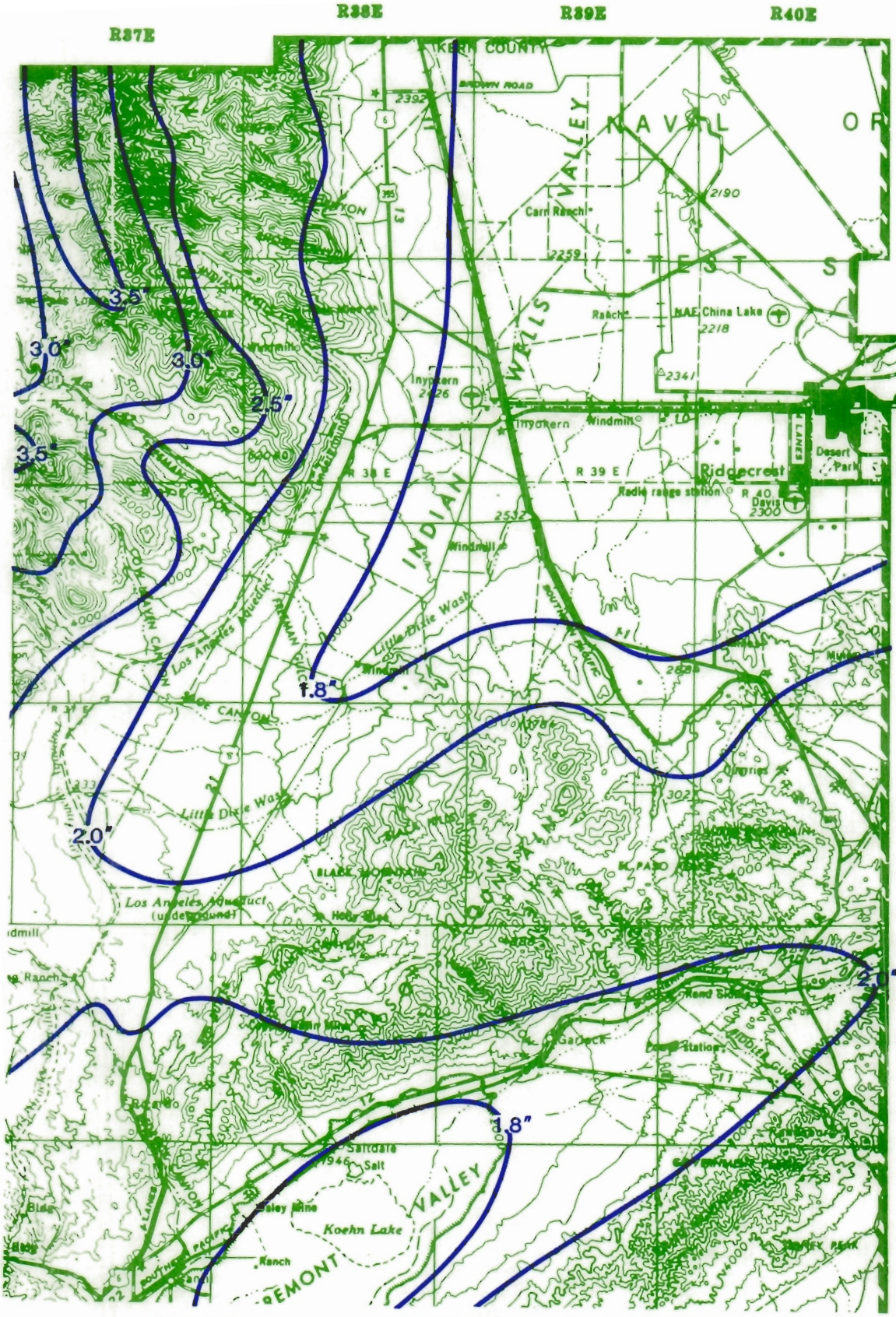
RAINFALL ISOHYET MAP

APPROXIMATE SCALE
1"=4 miles

——— 100-Year, 6-Hour Isohyet

NORTH

FIGURE B-38



KERN COUNTY HYDROLOGY MANUAL

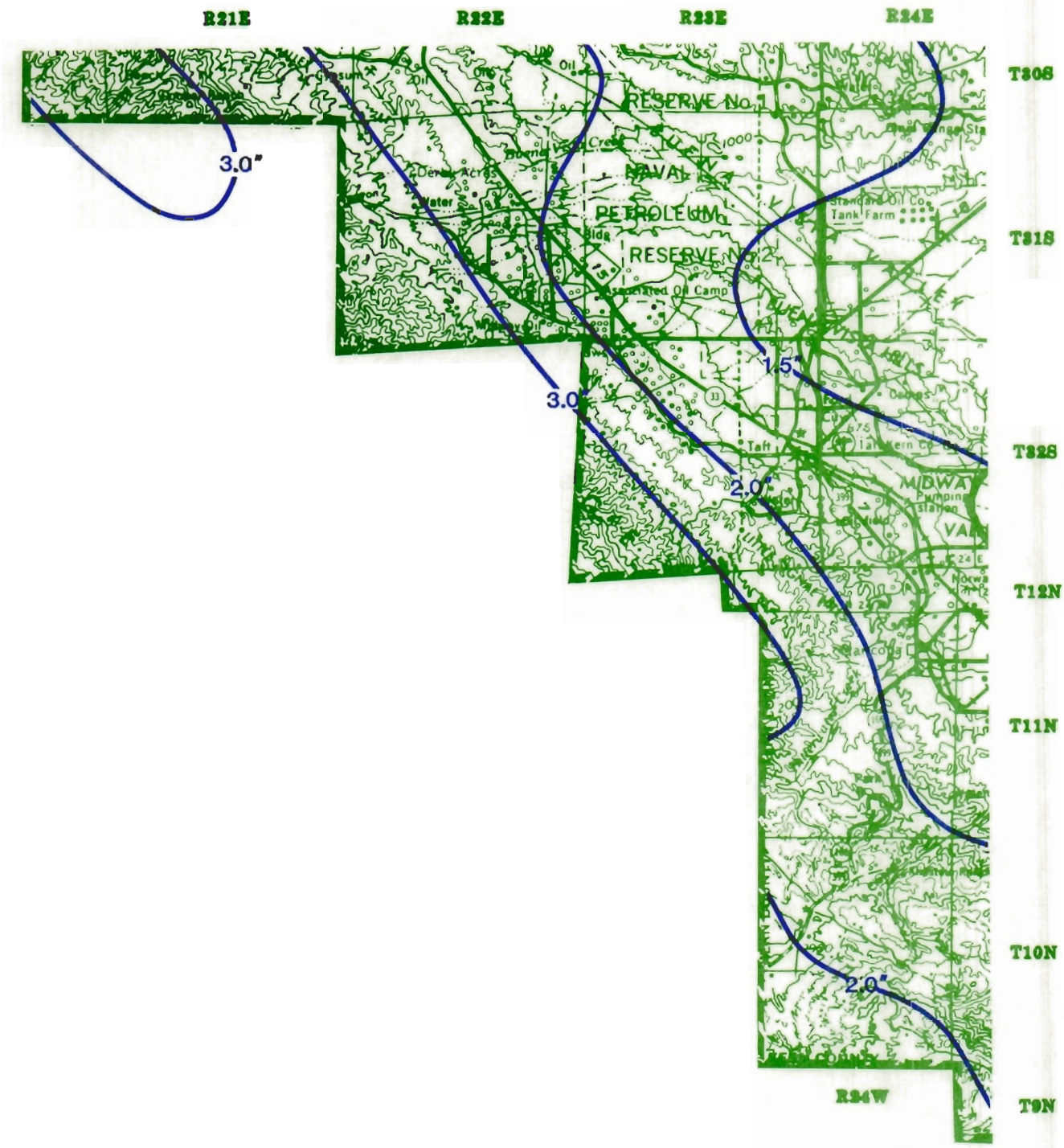
RAINFALL ISOHYET MAP

APPROXIMATE SCALE
1"=4 miles

100-Year, 6-Hour Isohyet

NORTH

FIGURE B-39



KERN COUNTY HYDROLOGY MANUAL

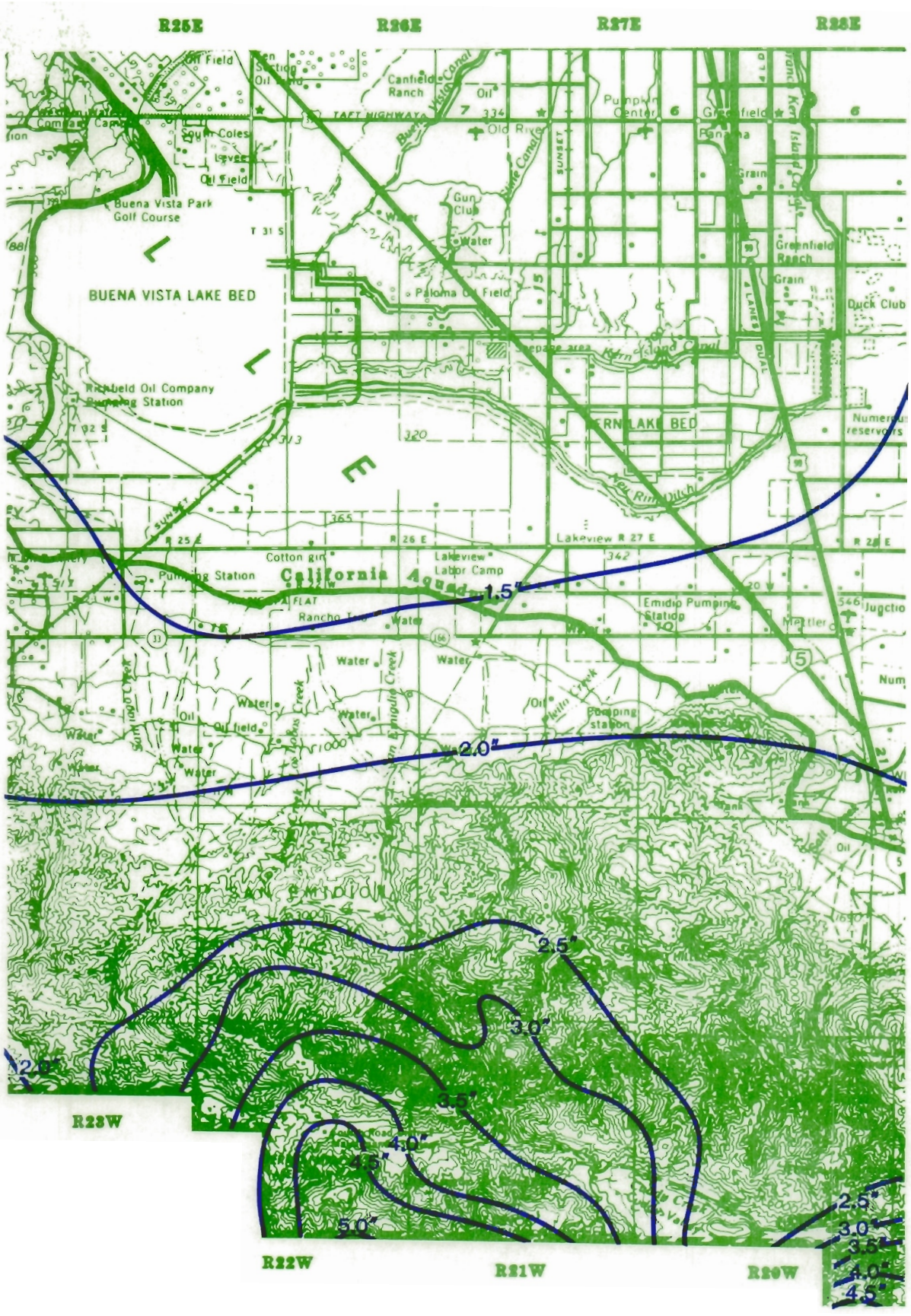
RAINFALL ISOHYET MAP

APPROXIMATE SCALE
1"=4 miles

——— 100-Year, 6-Hour Isohyet

NORTH

FIGURE B-40



KERN COUNTY HYDROLOGY MANUAL

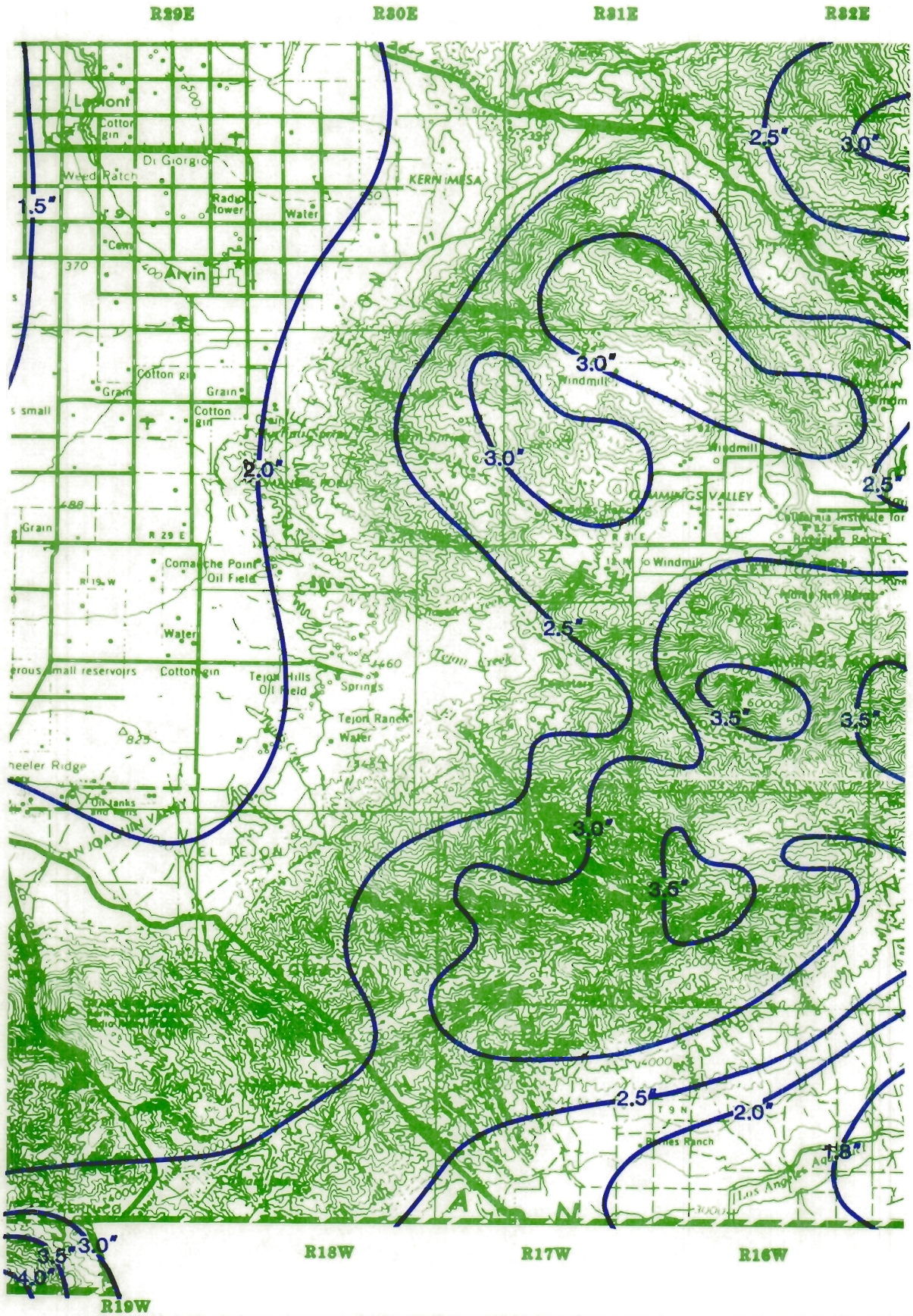
RAINFALL ISOHYET MAP

APPROXIMATE SCALE
1"=4 miles

100-Year, 6-Hour Isohyet

NORTH

FIGURE B-41



KERN COUNTY HYDROLOGY MANUAL

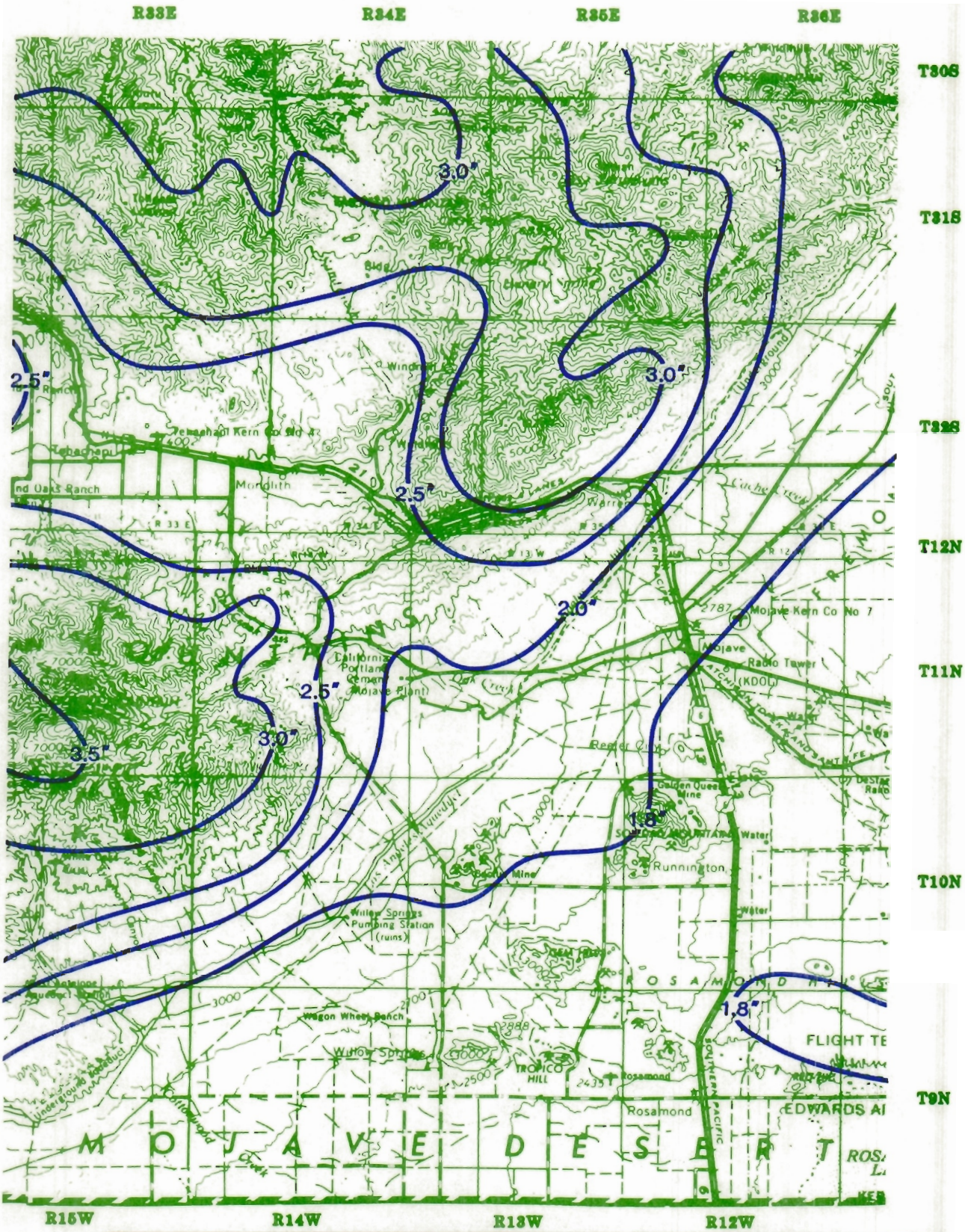
RAINFALL ISOHYET MAP

APPROXIMATE SCALE
1"=4 miles

100-Year, 6-Hour Isohyet

NORTH

FIGURE B-42



KERN COUNTY HYDROLOGY MANUAL

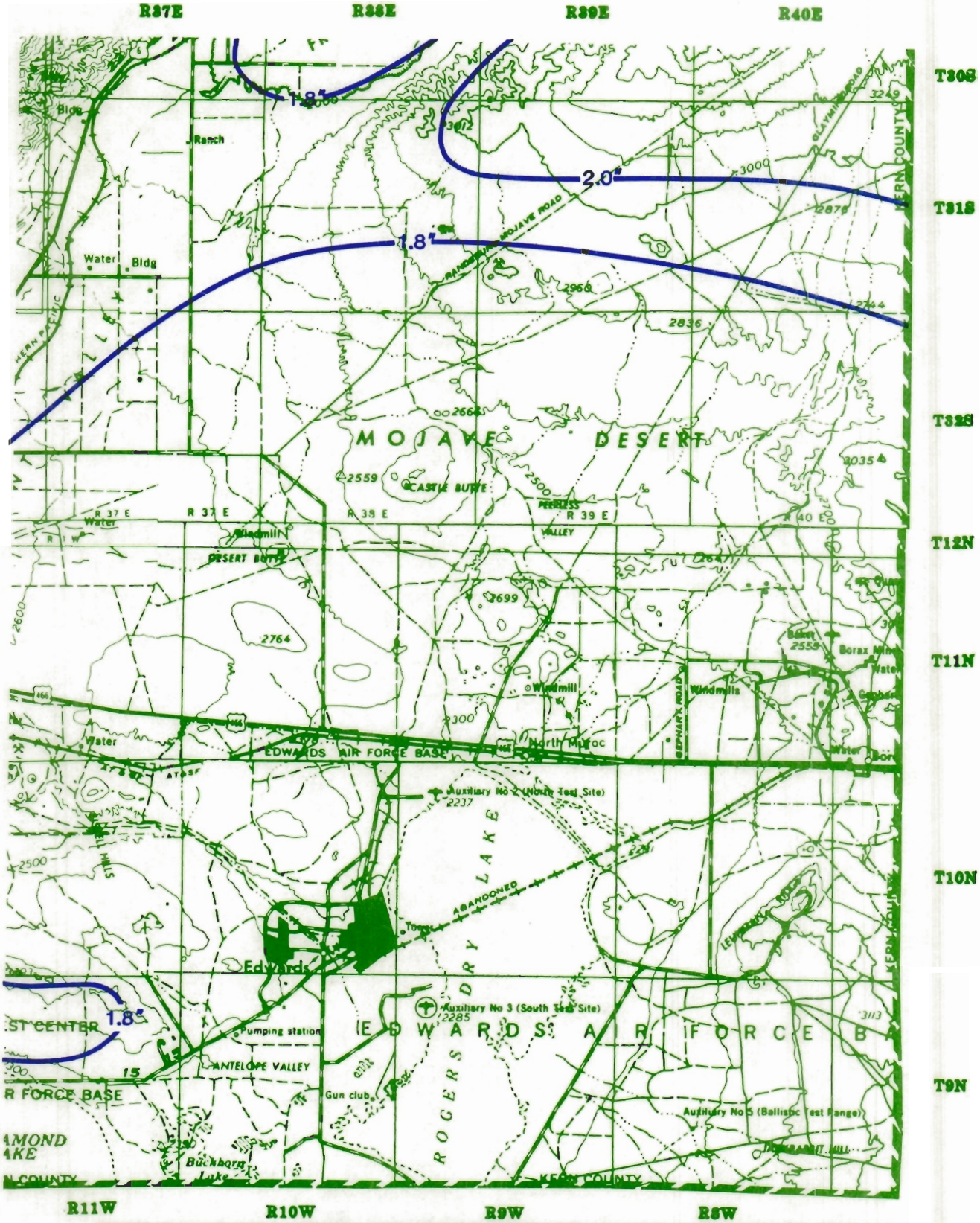
RAINFALL ISOHYET MAP

APPROXIMATE SCALE
1"=4 miles

——— 100-Year, 6-Hour Isohyet

NORTH

FIGURE B-43



KERN COUNTY HYDROLOGY MANUAL

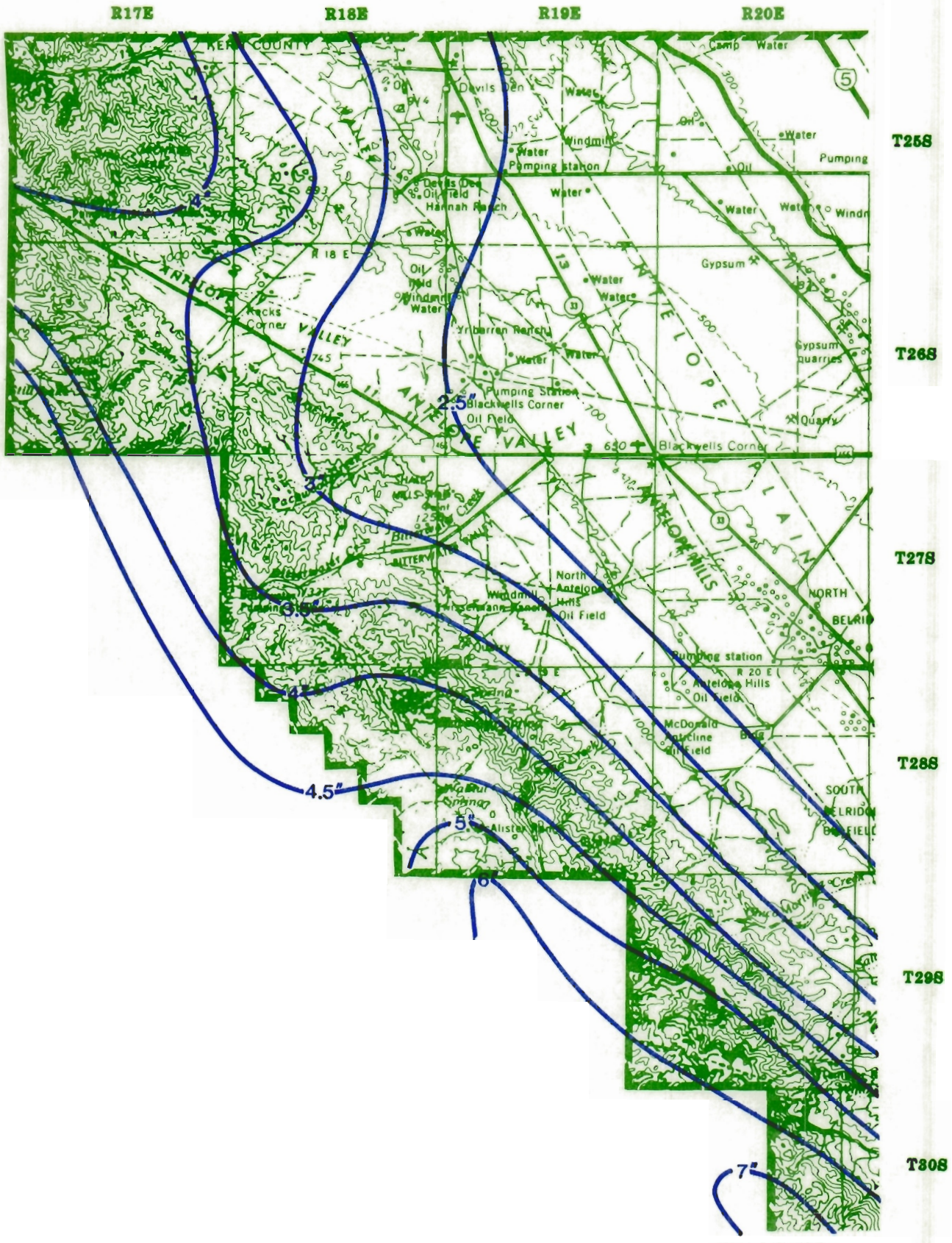
RAINFALL ISOHYET MAP

APPROXIMATE SCALE
1"=4 miles

100-Year, 6-Hour Isohyet

NORTH

FIGURE B-44



KERN COUNTY HYDROLOGY MANUAL

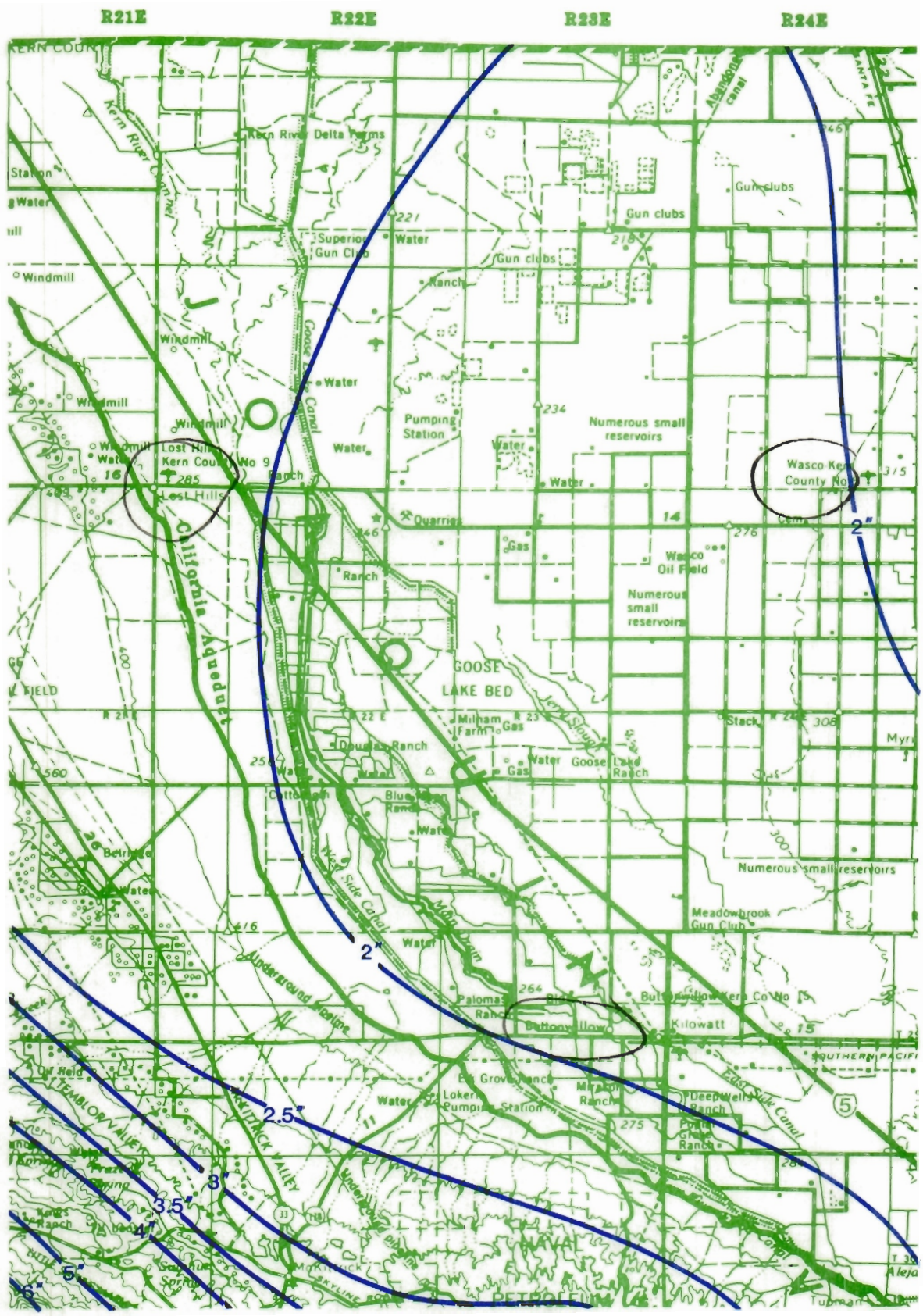
RAINFALL ISOHYET MAP

APPROXIMATE SCALE
1"=4 miles

100-Year, 24-Hour Isohyet

NORTH

FIGURE B-45



KERN COUNTY HYDROLOGY MANUAL

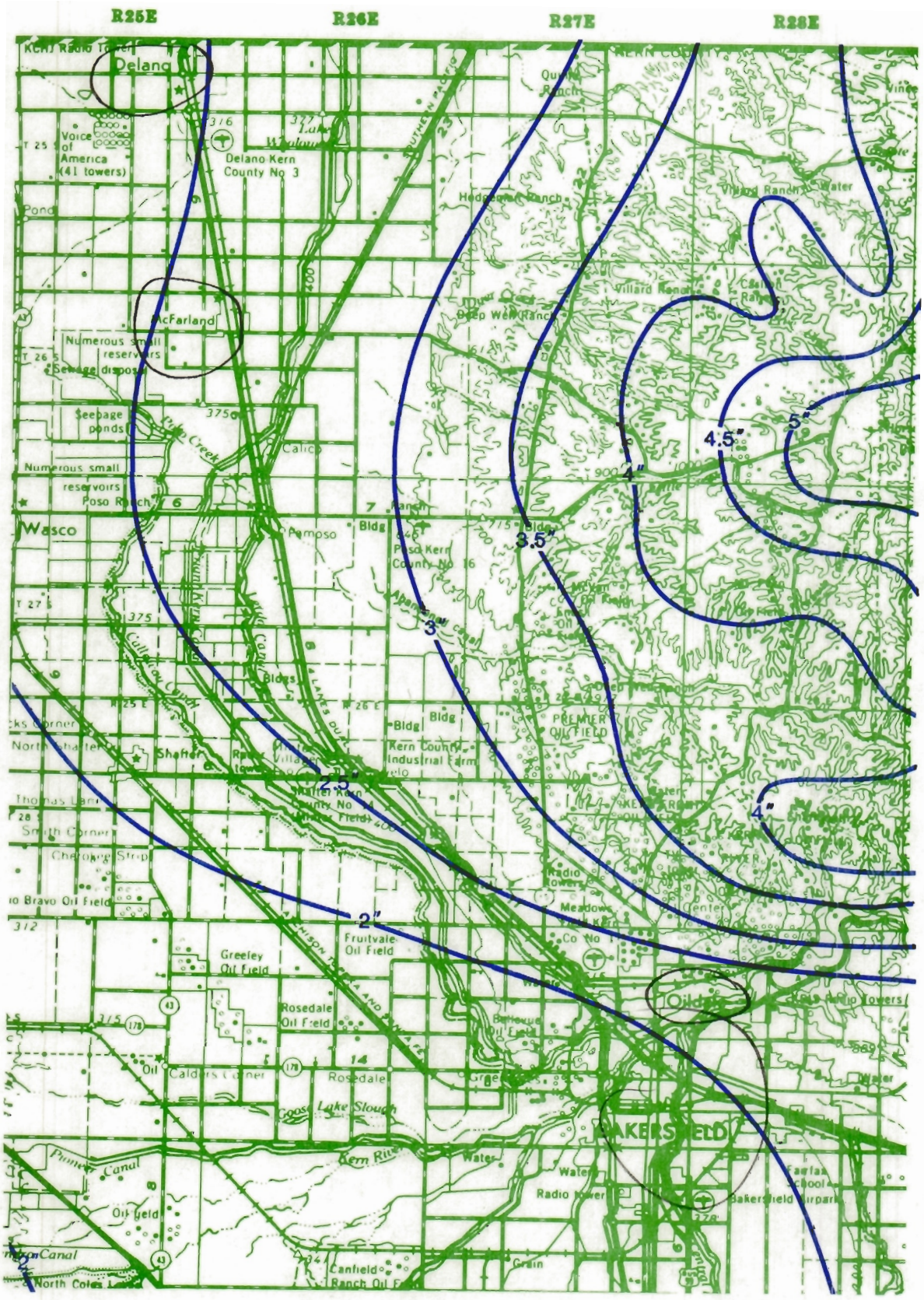
RAINFALL ISOHYET MAP

APPROXIMATE SCALE
1"=4 miles

100-Year, 24-Hour Isohyet

NORTH

FIGURE B-46



KERN COUNTY HYDROLOGY MANUAL

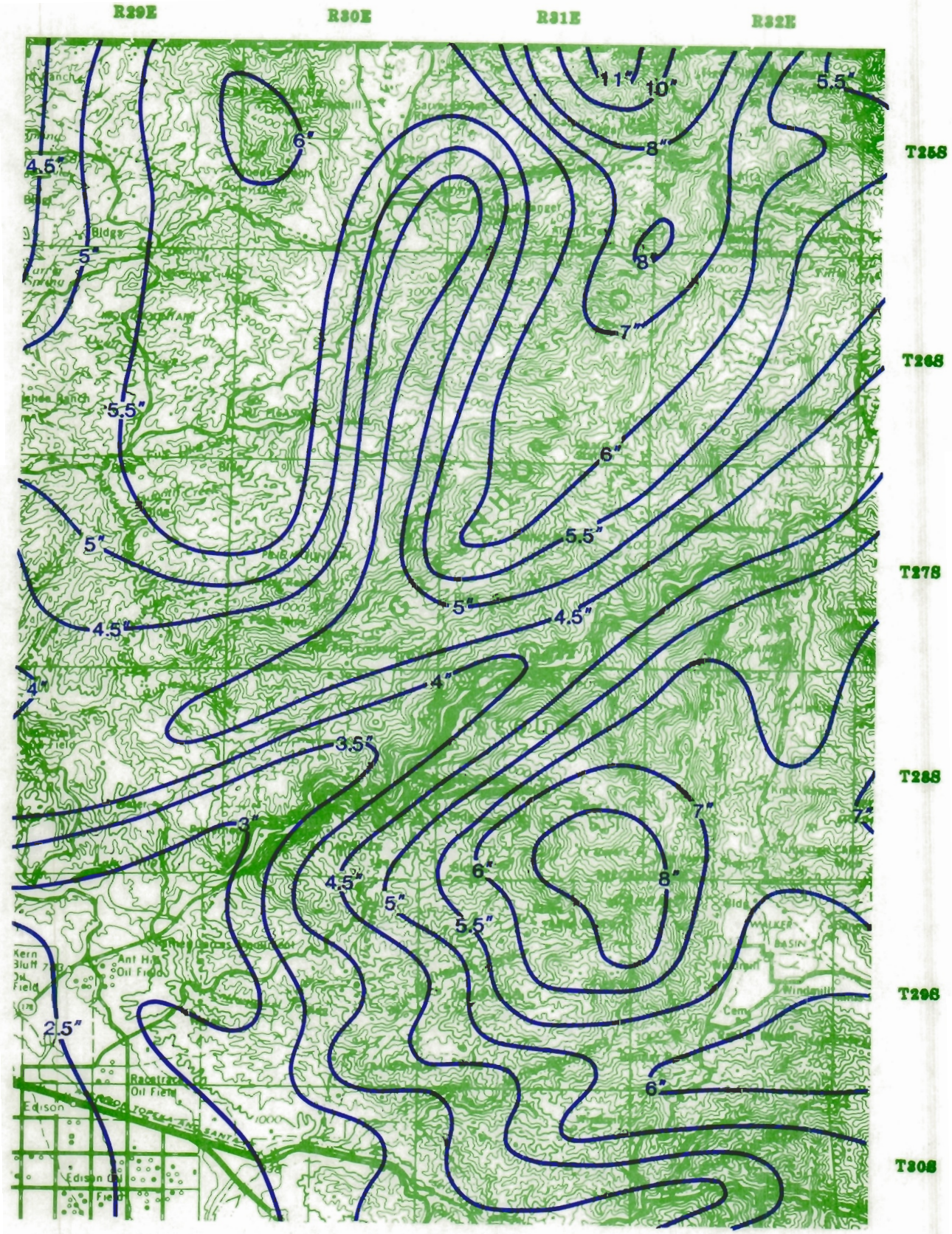
RAINFALL ISOHYET MAP

APPROXIMATE SCALE
1"=4 miles

100-Year, 24-Hour Isohyet

NORTH

FIGURE B-47



KERN COUNTY HYDROLOGY MANUAL

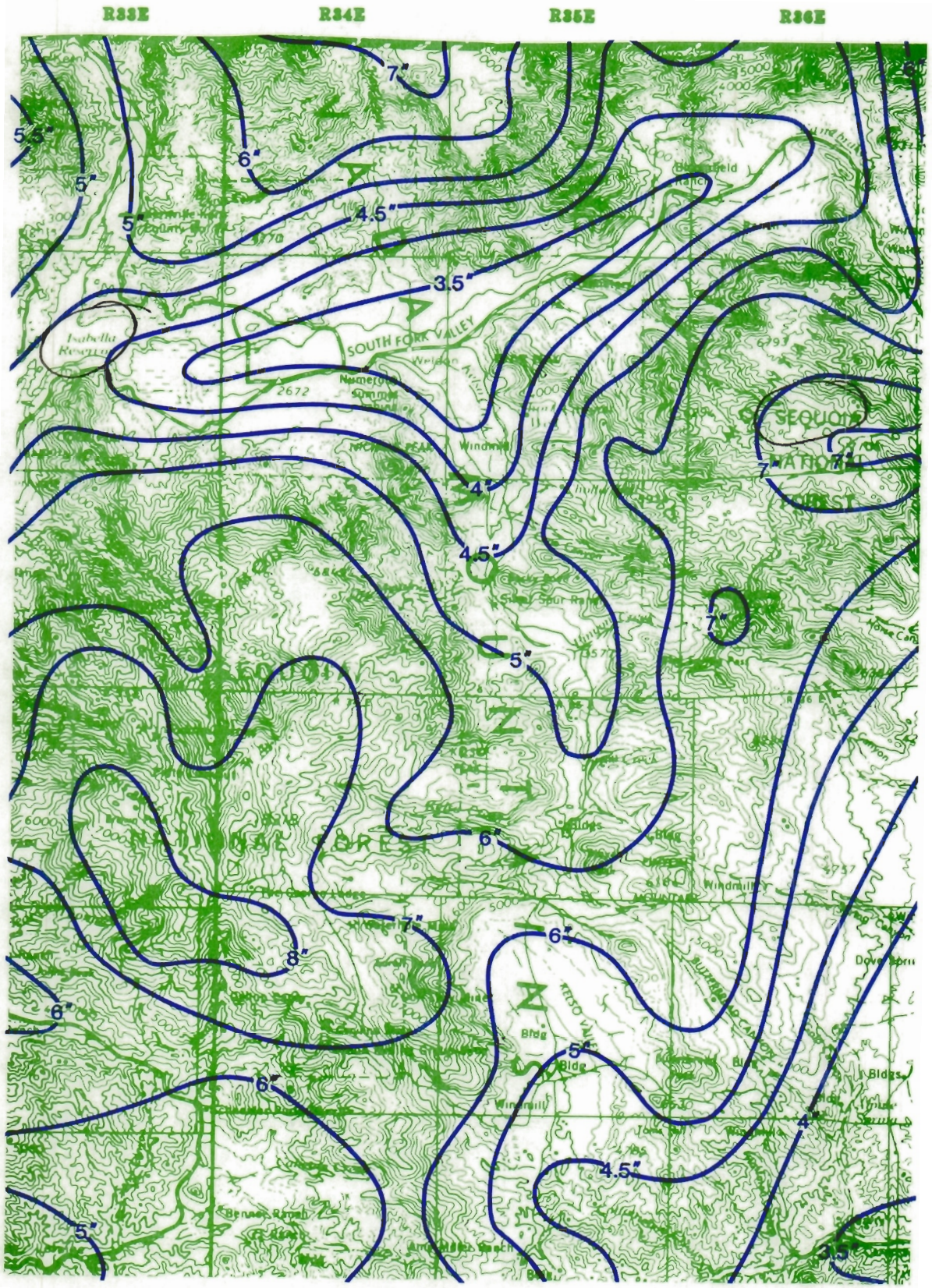
RAINFALL ISOHYET MAP

APPROXIMATE SCALE
1"=4 miles

——— 100-Year, 24-Hour Isohyet

NORTH

FIGURE B--48



KERN COUNTY HYDROLOGY MANUAL

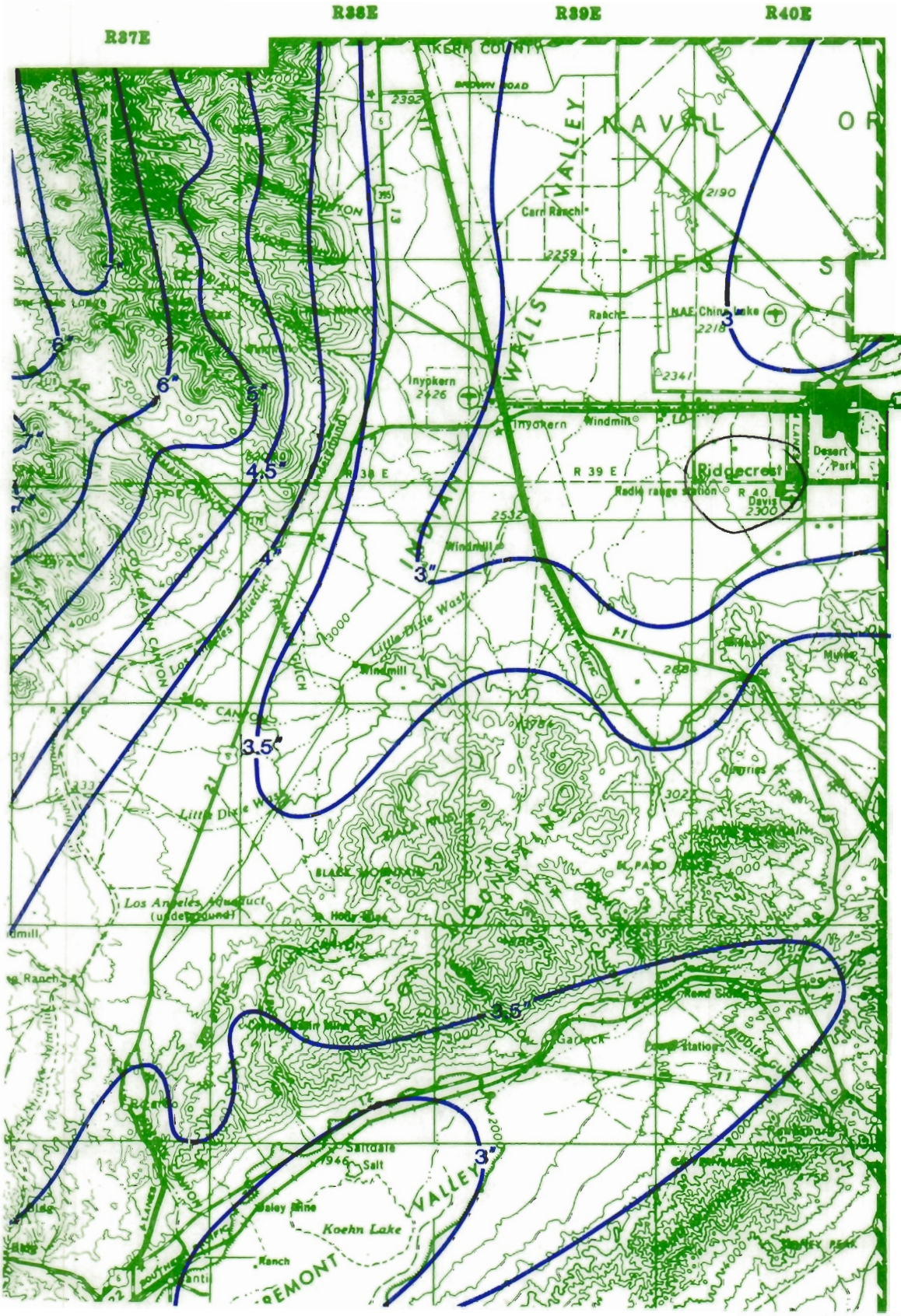
RAINFALL ISOHYET MAP

APPROXIMATE SCALE
1"=4 miles

100-Year, 24-Hour Isohyet

NORTH

FIGURE B-49



KERN COUNTY HYDROLOGY MANUAL

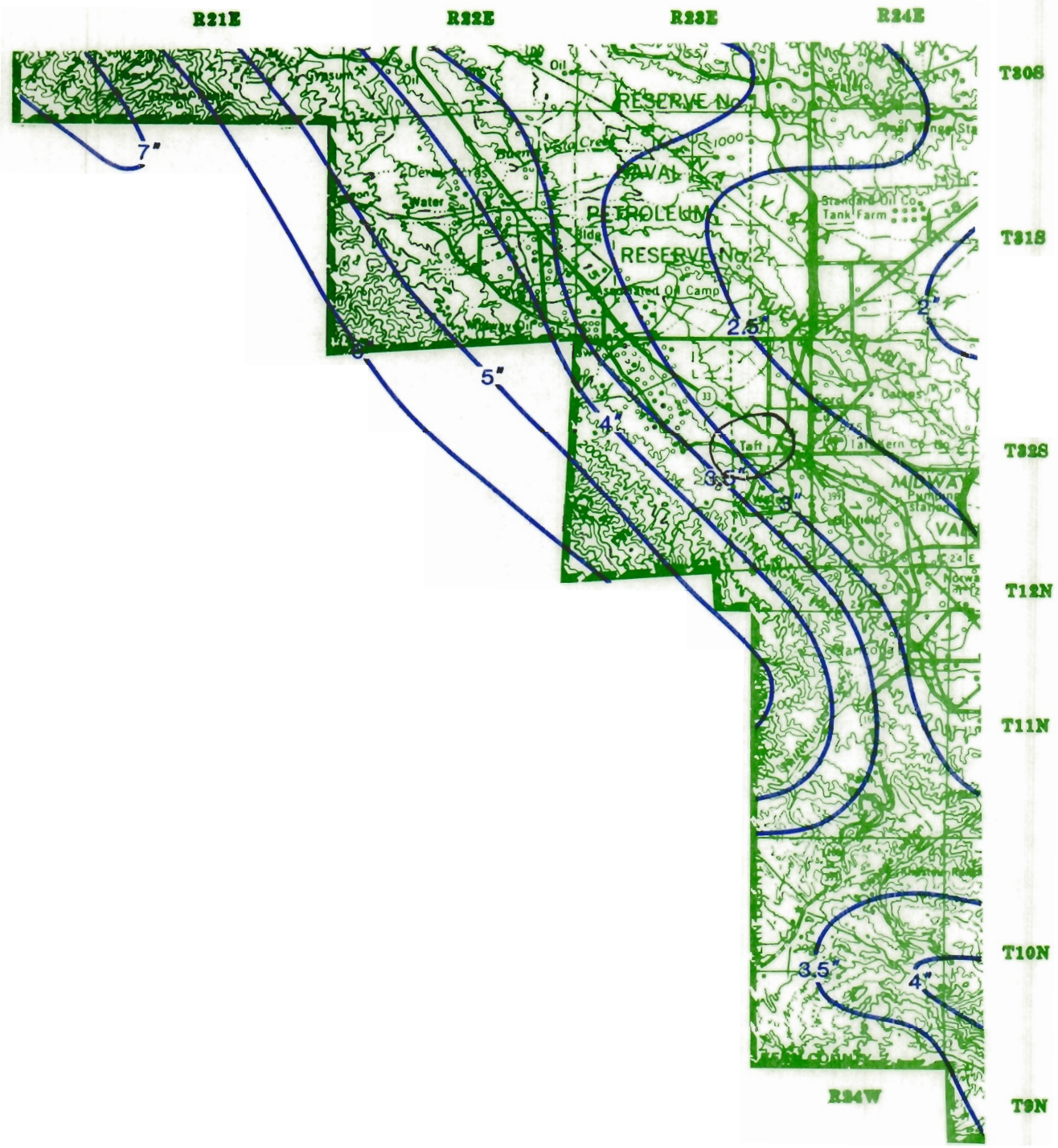
RAINFALL ISOHYET MAP

APPROXIMATE SCALE
1"=4 miles

——— 100-Year, 24-Hour Isohyet

NORTH

FIGURE B-50



KERN COUNTY HYDROLOGY MANUAL

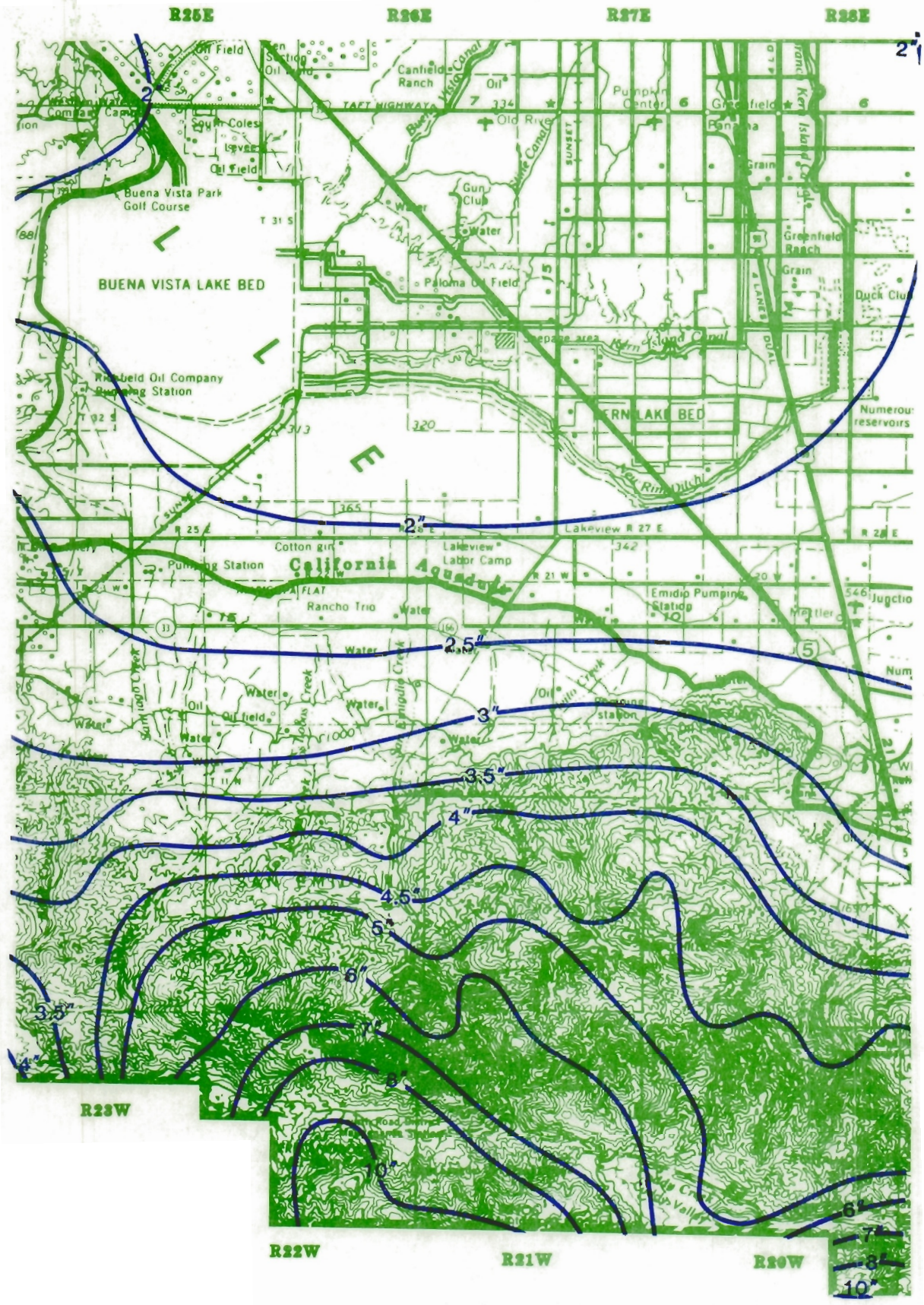
RAINFALL ISOHYET MAP

APPROXIMATE SCALE
1"=4 miles

——— 100-Year, 24-Hour Isohyet

NORTH

FIGURE B-51



KERN COUNTY HYDROLOGY MANUAL

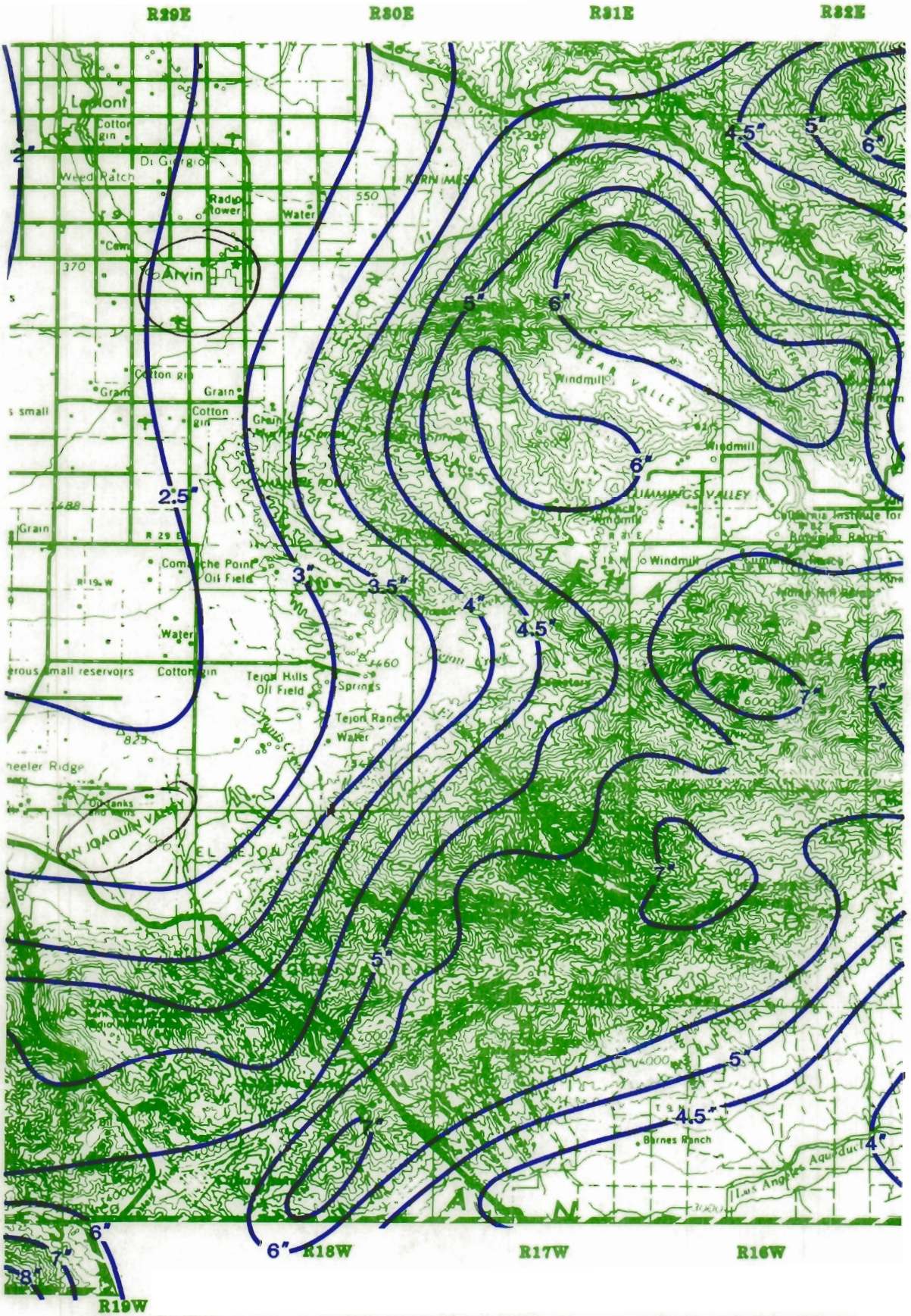
RAINFALL ISOHYET MAP

APPROXIMATE SCALE
1"=4 miles

100-Year, 24-Hour Isohyet

NORTH

FIGURE B-52



KERN COUNTY HYDROLOGY MANUAL

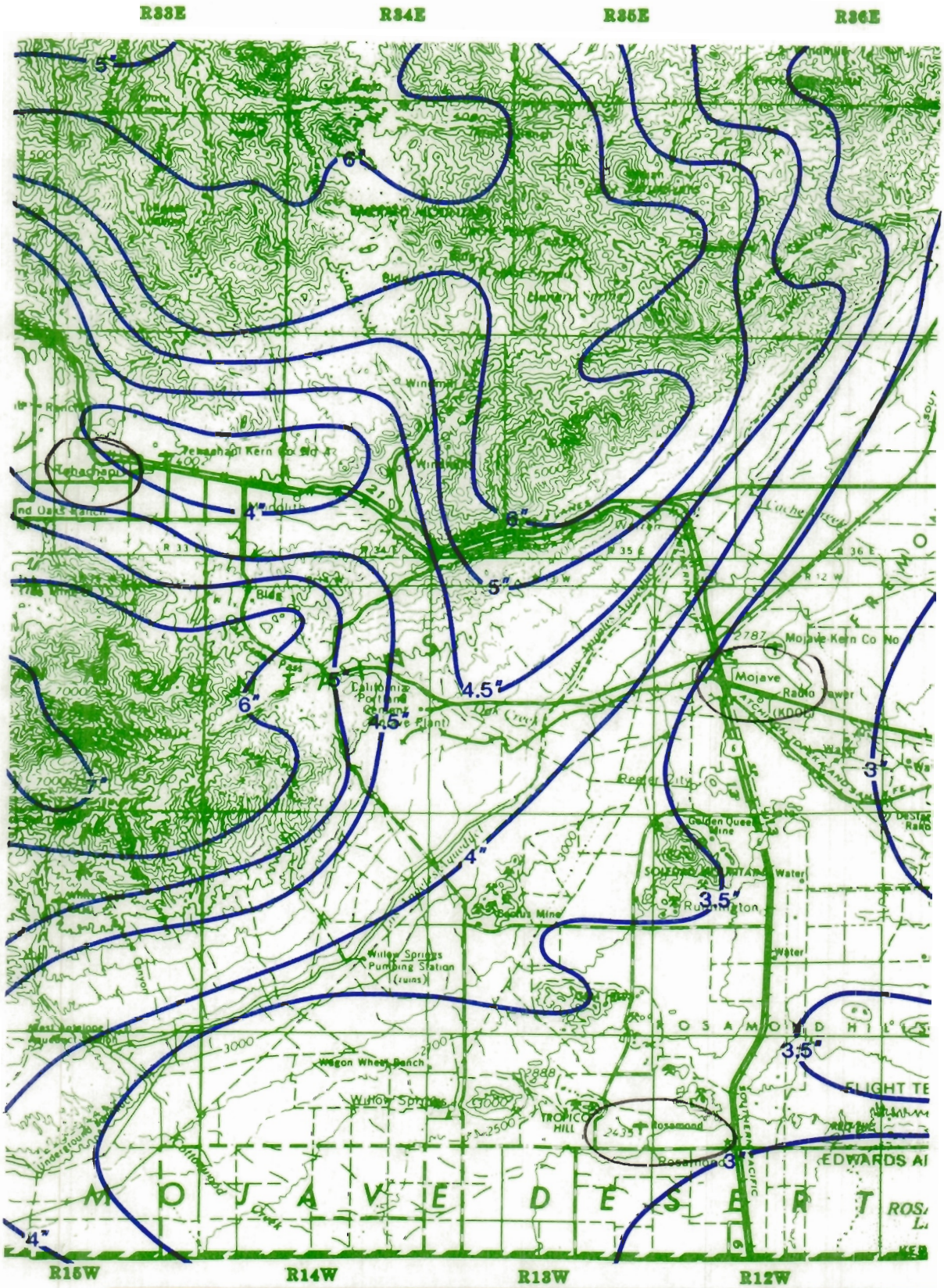
RAINFALL ISOHYET MAP

APPROXIMATE SCALE
1"=4 miles

——— 100-Year, 24-Hour Isohyet

NORTH

FIGURE B-53



KERN COUNTY HYDROLOGY MANUAL

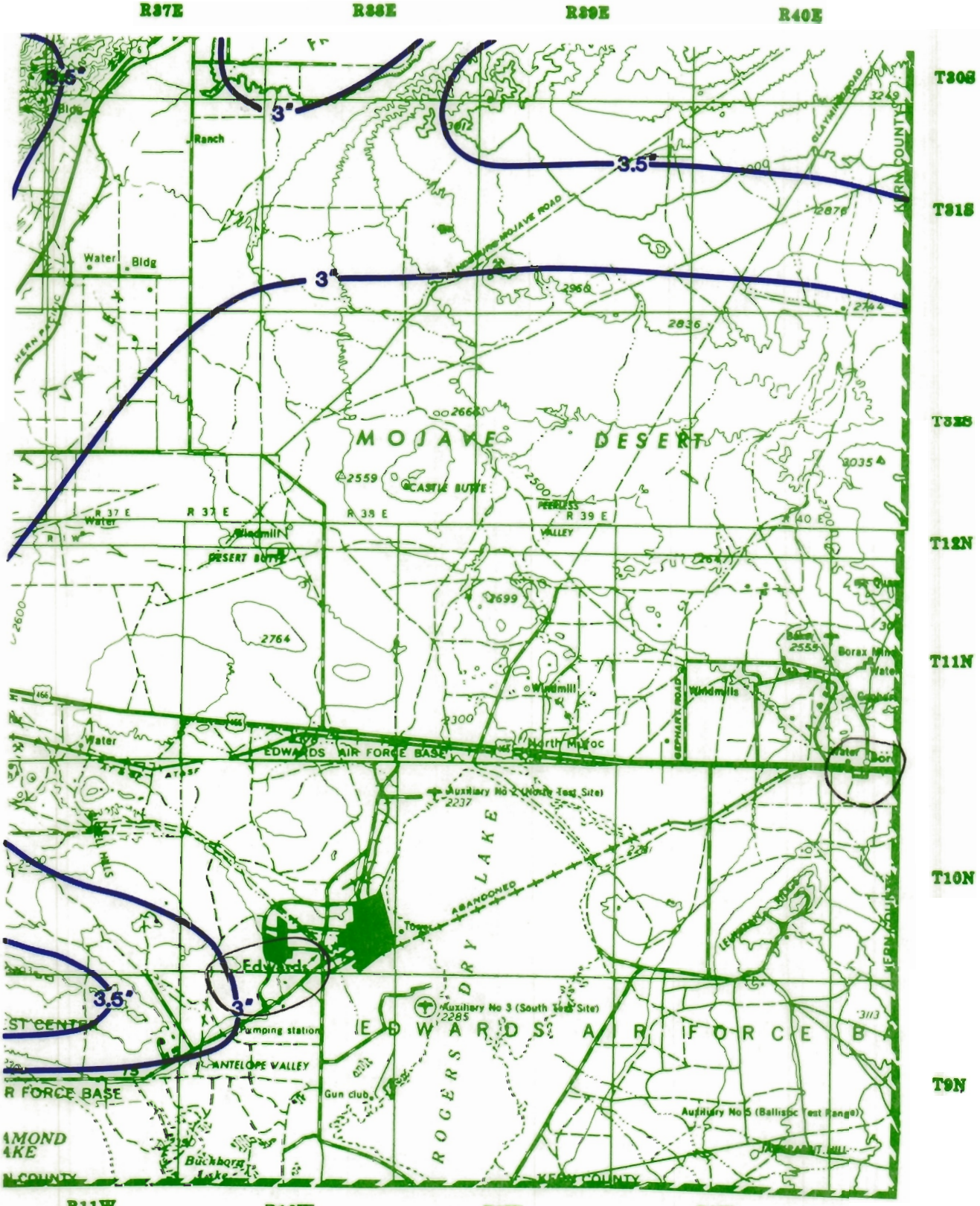
RAINFALL ISOHYET MAP

APPROXIMATE SCALE
1"=4 miles

100-Year, 24-Hour Isohyet

NORTH

FIGURE B-54



KERN COUNTY HYDROLOGY MANUAL

RAINFALL ISOHYET MAP

APPROXIMATE SCALE
1"=4 miles

——— 100-Year, 24-Hour Isohyet

NORTH

FIGURE B-55

Approximate Regional Delineation for Slope of Short Duration Intensity-Duration Curve

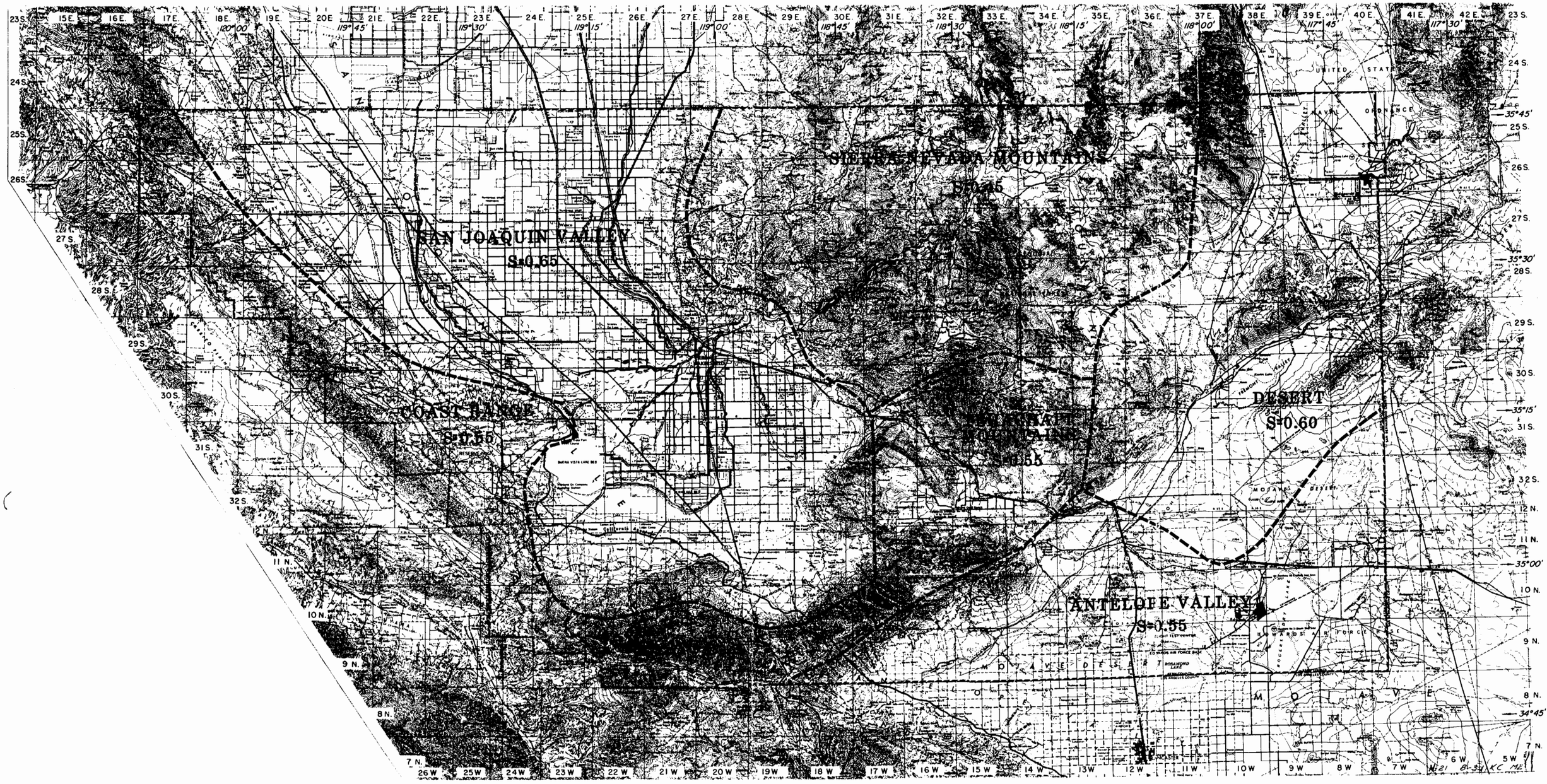


FIGURE B-56

Section B
Equations

List of Equations:

Equation No.	Equation		
B.1	P = 1/T		
B.2	Y2	=	0.111+0.545 (X1)(X1/X2)
B.3	Y100	=	0.221+0.885 (Y2)(X3/X1)
B.4	**or** Y100	=	0.221+0.098 (X3/X1)+0.482 (X3)(X1/X2)
B.5	Y2	=	0.107+0.315 (X1)
B.6	Y100	=	-0.391+1.224 (Y2)(X3/X1)+0.043 (X5)
B.7	**or** Y100	=	-0.391+0.131 (X3/X1)+0.386(X3) +0.043 (X5)
B.8	Y2	=	0.126+0.561 (X1)(X1/X2)
B.9	Y100	=	-0.030+0.816 (Y2)(X3/X1)+0.063 (X5)
B.10	**or** Y100	=	-0.030+0.103 (X3/X1)+0.458 (X3)(X1/X2) +0.063 (X5)
B.11	Y2	=	0.005+0.852 (X1)(X1/X2)
B.12	Y100	=	0.322+0.789 (X3)(X3/X4)

SECTION C

LOSSES

OVERVIEW

This section will cover watershed loss computation methods and data. It will discuss:

- Hydrologic Soil Groups
- Watershed Development Conditions
- Desert Hydrology Loss Rates

Section B presents precipitation estimation procedures and data. Sections D and E address watershed flow determination processes.

C.1. DEFINITIONS

Antecedent Moisture Condition (AMC):	The degree of wetness of a watershed at the beginning of a storm.
Infiltration:	The process of water entering the soil surface and percolating downward into the soil where it is stored during a precipitation event.
Initial Abstraction:	All the losses, including routing losses, except infiltration.
Impervious:	That portion of the land surface which is impenetrable to the passage of water. This area is usually associated with either manmade conditions (i.e. structures, paving, etc...) or natural conditions (i.e. rock outcroppings).
Pervious:	That portion of the land surface which is permeable to the passage of water. This area is usually associated with natural top soil materials.
Runoff Yield:	That percentage of precipitation which contributes to surface flow.
Ultimate Development:	That highest watershed development density specified in the Kern County General Plan, Specific Plan, or current zoning.

C.2. WATERSHED LOSSES

Watershed losses are considered to be depression storage, vegetation interception and transpiration, minor amounts of evaporation, and infiltration. The stored soil water, resulting from infiltration, may be consumptively used by vegetation, percolate further downward to groundwater storage, or exit the soil surface as seeps or springs. Seepage from stream bank storage is the primary source of baseflow which is derived from prior precipitation events. For modeling purposes, watershed losses are grouped into two components: namely, (i) ***infiltration***, and (ii) ***initial abstraction***.

Volume losses due to channel routing are only applicable after the rainfall event has ceased. During rainfall the initial abstraction accounts for these volume losses. Attenuation of flood hydrographs should be performed in accordance with Section H, Streamflow Routing.

C.3. HYDROLOGIC SOIL GROUPS

The major factor affecting loss rates is the nature of the soil itself. The soil surface characteristics, its ability to transmit water to subsurface layers, and total storage capacity, are all major factors in controlling the infiltration rate and initial abstraction parameter values of a particular soil. Soils are classified into four hydrologic soil groups as follows (refs. 2, 3):

Group A: ***Low runoff potential.*** Soils having high infiltration rates even when thoroughly wetted and consisting chiefly of deep, well-drained sands or gravels. These soils have a high rate of water transmission

Group B: Soils having ***moderate infiltration*** rates when thoroughly wetted and consisting chiefly of moderately deep to deep, moderately well to well-drained sandy-loam soils with moderately fine to moderately coarse textures. These soils have a moderate rate of water transmission.

Group C: Soils having ***slow infiltration*** rates when thoroughly wetted and consisting chiefly of silty-loam 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.

Group D: ***High runoff potential***. 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 or clay layer at or near the surface, and shallow soils over nearly impervious material. These soils have a very slow rate of water transmission.

C.3.1. Soil Maps

Detailed soil surveys have been prepared by the Soil Conservation Service for some areas of Kern County. An approximate soil survey has been prepared for the entire County. These soil surveys are available for review from the County.

C.4. SOIL COVER AND HYDROLOGIC CONDITIONS

The types of vegetation or ground cover on a watershed, and the quality or density of that cover, have a major impact on the infiltration capacity of a given soil. Definitions of specific cover types are provided in Figure C-1. Further refinement in the cover type descriptions is provided by the definition of cover quality as follows:

POOR: Heavily grazed or regularly burned areas. Less than 50 percent of the ground surface is protected by plant cover or brush and tree canopy.

FAIR: Moderate cover with 50 percent to 75 percent of the ground surface protected by vegetation.

GOOD: Heavy or dense cover with more than 75 percent of the ground surface protected by vegetation.

In most cases, watershed existing conditions cover type and quality can be readily determined by a field review of a watershed. Figure C-2 provides the CN values for various types and quality of ground cover. Impervious areas shall be assigned a CN of 98. It is noted that for ultimately developed conditions, the CN for urban landscaping (turf) is provided in Figure C-2.

C.5. WATERSHED DEVELOPMENT CONDITIONS

Ultimate development of the watershed should normally be assumed since watershed urbanization is reasonably likely within the expected life of most hydraulic facilities. Long range master plans for the County and incorporated cities should be reviewed to ensure that reasonable land use assumptions are made for the ultimate development of the watershed. A field review shall also be made to confirm existing use and drainage patterns. Particular attention shall be paid to existing and proposed landscape practices, as it is common in some areas to use ornamental gravels underlain by impervious plastic materials in place of lawns and shrubs. Appropriate actual impervious percentages can then be selected from Figure C-3. It should be noted that the recommended values from these figures are for average conditions and, therefore, some adjustment for particular applications may be required.

C.6. ANTECEDENT MOISTURE CONDITION (AMC)

The Soil Conservation Service (SCS) does not recommend the use of differing AMC in its recent release of TR-55. Therefore, this manual does not consider the use of AMC I or III. All curve numbers are based upon a moderate runoff potential, AMC II.

C.7. ESTIMATION OF LOSS RATES

In estimating loss rates for design hydrology, a watershed ***curve number*** **(CN)** is determined for each soil-cover complex within the watershed using Figure C-2. The working range of CN values is between 0 and 98, where a low CN indicates low runoff potential (high infiltration), and a high CN indicates high runoff potential (low infiltration). Selection of a CN takes into account the major factors affecting loss rates on pervious surfaces including the hydrologic soil group, cover type and cover quality.

Also included in the CN selection are the effects of "initial abstraction" (Ia) which represents the combined effects of other effective rainfall losses including depression storage, vegetation interception, evaporation, and transpiration, among other factors.

C.7.1. Estimation of Initial Abstraction (Ia)

The initial abstraction (Ia) for an area is a function of land use, treatment, and condition; interception; depression storage; and antecedent soil moisture. An estimate for Ia is given by the SCS as

$$Ia = 0.2S \tag{C.1}$$

where S is an estimate of total soil capacity given by

$$S = \frac{1000}{CN} - 10 \tag{C.2}$$

where CN is the area curve number.

C.7.2. Estimation of Storm Runoff Yield

Given the CN for a subarea A_j, the corresponding 24-hour storm runoff yield fraction, Y_j, is estimated by

$$Y_j = \frac{(P_{24} - Ia)^2}{(P_{24} - Ia + S) \cdot P_{24}} \tag{C.3}$$

where

Y_j = 24-hour storm runoff yield fraction for subarea A_j

P_{24} = 24-hour storm rainfall

Ia = initial abstraction from (C.1)

S = see (C.2)

It is noted that should Ia be greater than P_{24} in (C.3), then Y_j is defined to be zero. In this manual, the notation Y and Y_j will represent the ***runoff yield fraction***, rather than the volume of runoff.

If the area under study contains several (say m) CN designations, then the yield, Y, for the total area must represent the net effect of the several curve numbers. By weighing each of the subarea yield values according to the respective areas,

$$Y = \frac{Y_1 A_1 + \ldots + Y_m A_m}{A_1 + A_2 + \ldots + A_m} \tag{C.4}$$

where each Y_j follows from (C.3).

C.7.3. Low Loss Rate, F*

In design storm runoff hydrograph studies, the following formula is used to estimate that portion of rainfall to be attributed to watershed losses:

$$\overline{Y} = 1 - Y \tag{C.5}$$

where

$\overline{Y}$ = catchment low loss fraction

Y = catchment 24-hour storm runoff yield fraction computed from (C.4)

Using the low loss fraction, $\overline{Y}$, the corresponding low loss rate, F*, is given by

$$F^* = \overline{Y} \cdot I \tag{C.6}$$

where I is the rainfall intensity and F* has units of inches/hour. Use of F* enables the design storm 24-hour storm runoff yield to approximate the yield values obtained from the CN approach (see Figure C-4).

C.7.4. Infiltration Rates

Soil infiltration rates have been estimated for each of the soil groups by laboratory studies and measurements. These measurements show that an initially dry soil will have an associated infiltration rate which essentially decreases with time as the soil becomes wetted. As the soil is subjected to continual heavy rainfall, this infiltration rate approaches a minimum (usually within about 30 minutes) which represents the infiltration capacity of the soil.

When sufficient stream gauge information is available, infiltration rates for unit hydrograph hydrology can be estimated from a study of rainfall-runoff relationships of major storms. Where such data is not available, infiltration rates for pervious areas as a function of CN can be estimated using Figures C-2 and C-5. Loss rates for pervious areas estimated from the Figure C-5 curve was found to be generally consistent with values developed from rainfall-runoff reconstitution studies in San Bernardino County, California watersheds. Due to the similarities in hydrologic and meteorological conditions in Kern and San Bernardino Counties, the curves of Figure C-5 can be used in estimating infiltration rates for Kern County watersheds.

C.7.5. Estimation of Catchment Maximum Loss Rates, F_m

The infiltration rate selected from Figure C-5 applies to the ***pervious*** area fraction of the watershed. The infiltration rate assumed for an ***impervious*** surface is 0.0 inch/hour. The maximum loss rate, F_m, for a catchment is therefore given by

$$F_m = a_p F_p \qquad (C.7)$$

where a_p is the pervious area fraction, and F_p is the infiltration rate for the pervious area.

Should a catchment contain several F_p values, the composite F_m value is determined as a simple area average of the several F_m values.

C.7.6. Design Storm Loss Rates

In design storm runoff hydrograph studies, a 24-hour duration storm pattern is used to develop the time distribution of effective rainfall over the watershed. The effective rainfall quantities are determined by subtracting the watershed losses from the design storm rainfall.

The loss rate used for a particular catchment is a combination of the maximum loss rate F_m and the low loss rate F*. F* is used as the loss rate unless F* exceeds F_m, in which case F_m is used as the loss rate. That is, F_m serves as the maximum loss rate. Typically in 100-year storm studies, F* serves as the loss rate for the entire storm pattern except for the most intense rainfalls where F_m would apply. However, for lower frequency storm studies such as the 5-year return event, F* often applies for the entire 24-hour storm pattern. The example problem of Section E provides an illustration in the use of F* and F_m values. Figure C-6 illustrates the loss rate function used with the design storm.

C.8. DESERT HYDROLOGY LOSS RATES

For desert catchments, a field investigation will be required for most hydrologic studies to determine the pertinent drainage area characteristics. The extent of the required field study will depend on the size of the drainage area and the complexity of the drainage problem. Many features such as channel-flow pattern, flow distribution, channel diversions, and type and density of the vegetative cover, may not be apparent from

contour maps or air photos. The field study may also indicate the need for low level air photos to solve complex drainage problems.

C.8.1. Hydrologic Soil-Cover Complex

A determination of vegetative cover types, hydrologic condition of cover types, and hydrologic soil group must be made for the drainage area. Ordinarily only broad categories of soils and cover types are delineated. Hydrologic soil cover complexes most commonly encountered in desert areas are given in Figure C-7 together with the associated curve numbers (CN). Curve numbers and, consequently, direct runoff varies with vegetation type, vegetative cover density, and hydrologic soil group.

C.8.2. Hydrologic Cover Types

Vegetative types are divided into the following groups:

Desert Brush: Includes such plants as 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.

C.8.3. Hydrologic Cover Density

Hydrologic cover density is defined as the percent of the ground surface covered by the crown canopy of live plants and litter.

Three broad ranges of vegetative cover density have been established:

Poor: 0% - 20% Vegetative cover

Fair: 20% - 40% Vegetative cover

Good: Over 40% Vegetative cover

TABLE C.1. F_m (in/hr) VALUES FOR TYPICAL COVER TYPES

		SOIL GROUP			
	a_p (1)	A	B	C	D
NATURAL:					
Barren	1	0.41	0.27	0.18	0.14
Row Crops (good)	1	0.59	0.41	0.29	0.22
Grass (fair)	1	0.82	0.56	0.4	0.31
Orchards (fair)	1	0.88	0.62	0.43	0.34
Woodland (fair)	1	0.95	0.69	0.5	0.4
URBAN:					
Residential (1 DU/AC)	0.8	0.76	0.56	0.39	0.3
Residential (2 DU/AC)	0.7	0.65	0.49	0.34	0.27
Residential (4 DU/AC)	0.6	0.56	0.42	0.29	0.23
Residential (10 DU/AC)	0.4	0.37	0.28	0.19	0.15
Condominium	0.35	0.32	0.24	0.17	0.13
Mobilehome Park	0.25	0.23	0.17	0.12	0.1
Apartments	0.2	0.19	0.14	0.1	0.08
Commercial/Industrial	0.1	0.09	0.07	0.05	0.04

NOTES:

(1) Recommended a_p values from Figure C-3

(2) AMC II assumed for all F_m values

(3) CN values obtained from Figure C-2

(4) DU/AC = dwelling unit per acre

Residential Landscaping (Lawn, Shrubs, etc.) - The pervious portions of commercial establishments, single and multiple family dwellings, trailer parks and schools where the predominant land cover is lawn, shrubbery and trees.

Row Crops - Lettuce, tomatoes, beets, tulips or any field crop planted in rows far enough apart that most of the soil surface is exposed to rainfall impact throughout the growing season. At plowing, planting and harvest times it is equivalent to fallow.

Small Grain - Wheat, oats, barley, flax, etc. planted in rows close enough that the soil surface is not exposed except during planting and shortly thereafter.

Legumes - Alfalfa, sweetclover, timothy, etc. and combinations are either planted in close rows or broadcast.

Fallow - Fallow land is land plowed but not yet seeded or tilled.

Woodland - grass - Areas with an open cover of broadleaf or coniferous trees usually live oak and pines, with the intervening ground space occupied by annual grasses or weeds. The trees may occur singly or in small clumps. Canopy density, the amount of ground surface shaded at high noon, is from 20 to 50 percent.

Woodland - Areas on which coniferous or broadleaf trees predominate. The canopy density is at least 50 percent. Open areas may have a cover of annual or perennial grasses or of brush. Herbaceous plant cover under the trees is usually sparse because of leaf or needle litter accumulation.

Chaparral - Land on which the principal vegetation consists of evergreen shrubs with broad, hard, stiff leaves such as manzonita, ceanothus and scrub oak. The brush cover is usually dense or moderately dense. Diffusely branched evergreen shrubs with fine needle-like leaves, such as chamise and redchank, with dense high growth are also included in this soil cover.

Annual Grass - Land on which the principal vegetation consists of annual grasses and weeds such as annual bromes, wild barley, soft chess, ryegrass and filaree.

Irrigated Pasture - Irrigated land planted to perennial grasses and legumes for production of forage and which is cultivated only to establish or renew the stand of plants. Dry land pasture is considered as annual grass.

Meadow - Land areas with seasonally high water table, locally called cienegas. Principal vegetation consists of sod-forming grasses interspersed with other plants.

Orchard (Deciduous) - Land planted to such deciduous trees as apples, apricots, pears, walnuts, and almonds.

Orchard (Evergreen) - Land planted to evergreen trees which include citrus and avocados and coniferous plantings.

Turf - Golf courses, parks and similar lands where the predominant cover is irrigated mowed close-grown turf grass. Parks in which trees are dense may be classified as woodland.

KERN COUNTY

HYDROLOGY MANUAL

SCS
COVER TYPE
DESCRIPTIONS

FIGURE C-1

Curve[1] Numbers of Hydrologic Soil-Cover Complexes For Pervious Areas-AMC II

Cover Type (3)	Quality of Cover (2)	Soil Group A	B	C	D
NATURAL COVERS -					
Barren (Rockland, eroded and graded land)		77	86	91	94
Chaparral, Broadleaf (Manzonita, ceanothus and scrub oak)	Poor	53	70	80	85
	Fair	40	63	75	81
	Good	31	57	71	78
Chaparral, Narrowleaf (Chamise and Redskank)	Poor	71	82	88	91
	Fair	55	72	81	86
Grass, Annual or Perennial	Poor	68	79	86	89
	Fair	49	69	79	84
	Good	39	61	74	80
Meadows or Cienagas (Areas with seasonally high water table, principal vegatation is sod forming grass)	Poor	63	77	85	88
	Fair	51	70	80	84
	Good	30	58	71	78
Open Brush (Soft wood shrubs-buckwheat,sage,etc.)	Poor	62	76	84	88
	Fair	46	66	77	83
	Good	41	63	75	81
Woodland (4) (Coniferous or broadleaf trees predominate. Canopy density is at least 50 percent)	Poor	45	66	77	83
	Fair	36	60	73	79
	Good	30	55	70	77
Woodland, Grass (Coniferous or broadleaf trees with canopy density from 20 to 50 percent)	Poor	57	73	82	86
	Fair	43	65	76	82
	Good	32	58	72	79
URBAN COVERS -					
Residential or Commercial Landscaping (Lawns, shrubs, etc.)	Good	39	61	74	80
Turf (Irrigated and mowed grass)	Poor	68	79	86	89
	Fair	49	69	79	84
	Good	39	61	74	80

KERN COUNTY
Hydrology Manual

CURVE NUMBERS FOR PERVIOUS AREAS

Curve[1] Numbers of Hydrologic Soil-Cover Complexes For Pervious Areas-AMC II

Cover Type (3)	Quality of Cover (2)	Soil Group A	B	C	D
AGRICULTURAL COVERS -					
Fallow (Bare Soil)		77	86	91	94
Close Seeded (alfalfa, sweetclover, timothy, etc.)	Poor	66	77	85	89
	Good	58	72	81	85
Orchards, Evergreen (Citrus, avacodos, etc.)	Poor	57	73	82	86
	Fair	43	65	76	82
	Good	32	58	72	79
Pasture (Grassland or range, continuos forage for grazing)	Poor	68	79	86	89
	Fair	49	69	79	84
	Good	39	61	74	80
Row Crops (Straight row, non-contoured)	Poor	72	81	88	91
	Good	67	78	85	89
Small Grain (Straight row, non-contoured)	Poor	65	76	84	88
	Good	63	75	83	87

Notes:

1. Average runoff condition, Ia = 0.2(S)

2. Poor: Heavily grazed, regularly burned areas, or areas of high burn potential. Less than 50 percent of the ground surface is protected by plant cover or brush and tree canopy.

   Fair: Moderate cover with 50 percent to 75 percent of the ground surface protected. In wooded areas the woods are grazed but not burned, and some forest litter covers the soil.

   Good: Heavy or dense cover with more than 75 percent of the ground surface protected. In wooded areas the woods are protected from grazing, litter and brush adequately cover soil.

3. See Figure C-1 for definition of cover types.

KERN COUNTY
Hydrology Manual

CURVE NUMBERS FOR PERVIOUS AREAS

ACTUAL IMPERVIOUS COVER

Land Use	Range-Percent	Recommended Value For Average Conditions-Percent (1)
Natural or Agriculture	0 - 0	0
Public Park	10 - 25	15
School	30 - 50	40
Single Family Residential: (2)		
2.5 acre lots	5 - 15	10
1 acre lots	10 - 25	20
2-3 DU/acre	20 - 40	30
3-5 DU/acre	30 - 50	40
5-8 DU/acre	35 - 55	50
8-10 DU/acre	50 - 70	60
More than 10 DU/acre	65 - 90	80
Multiple Family Residential:		
Condominiums	45 - 70	65
Apartments	65 - 90	80
Mobile Home Parks	60 - 85	75
Commercial, Downtown Business or Industrial	80 - 100	90

Notes:

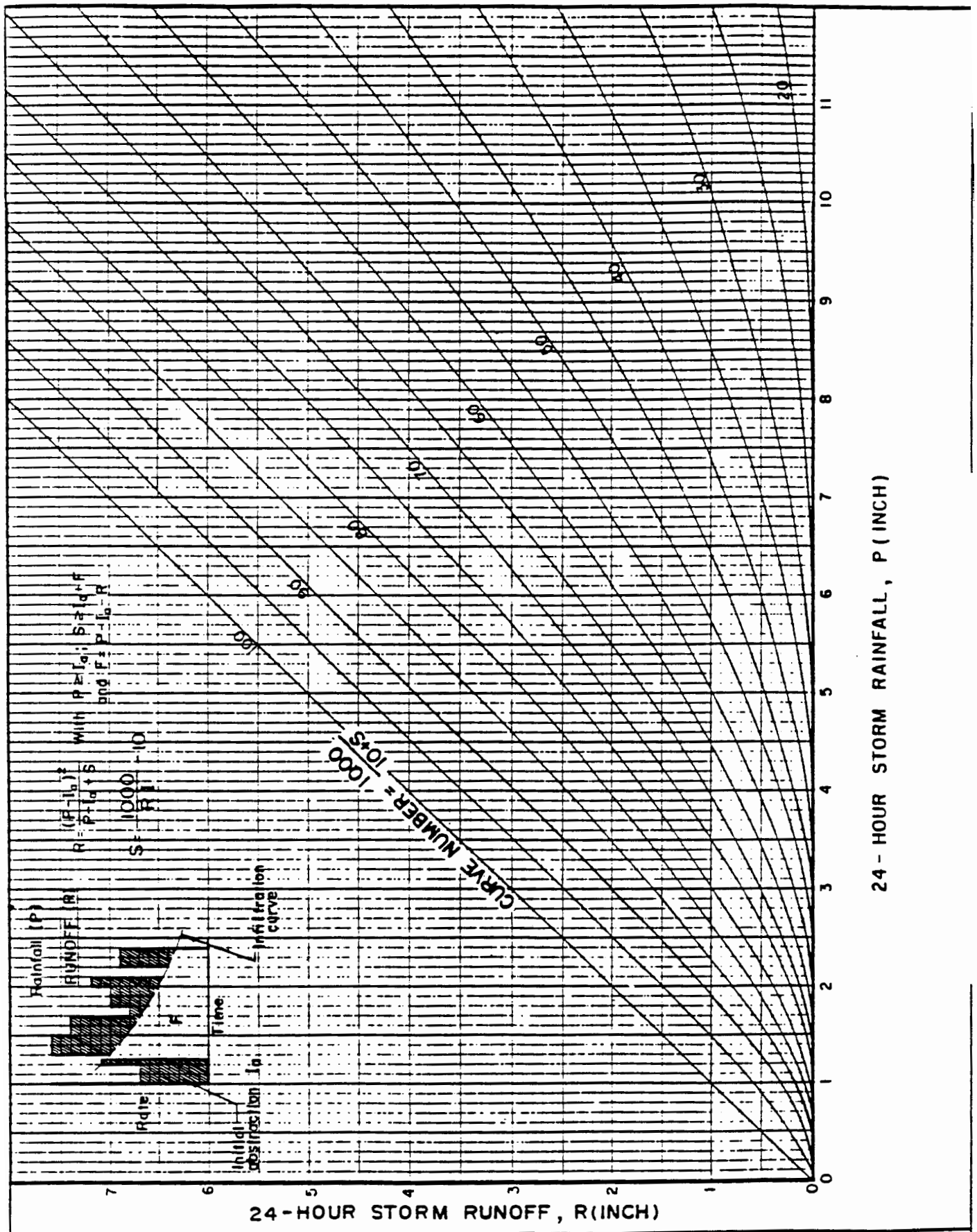
1) Recommended values are based on average conditions which may not apply to a particular study. The percentage impervious may vary greatly even on comparable study sized lots due to differences in in dwelling size, improvements, etc. Landscape practices should also be considered as it is common in some areas to use ornamental gravels underlain by impervious plastic materials in place of lawns and shrubs. A field investigation of a study area shall always be made, and a review of aerial photos, where available, may assist in estimating the percentage of impervious cover in developed areas.

2) For typical equestrian subdivisions increase impervious area 5% over the values recommended in the table above.

KERN COUNTY
Hydrology Manual

ACTUAL IMPERVIOUS COVER FOR DEVELOPED AREAS

FIGURE C-3

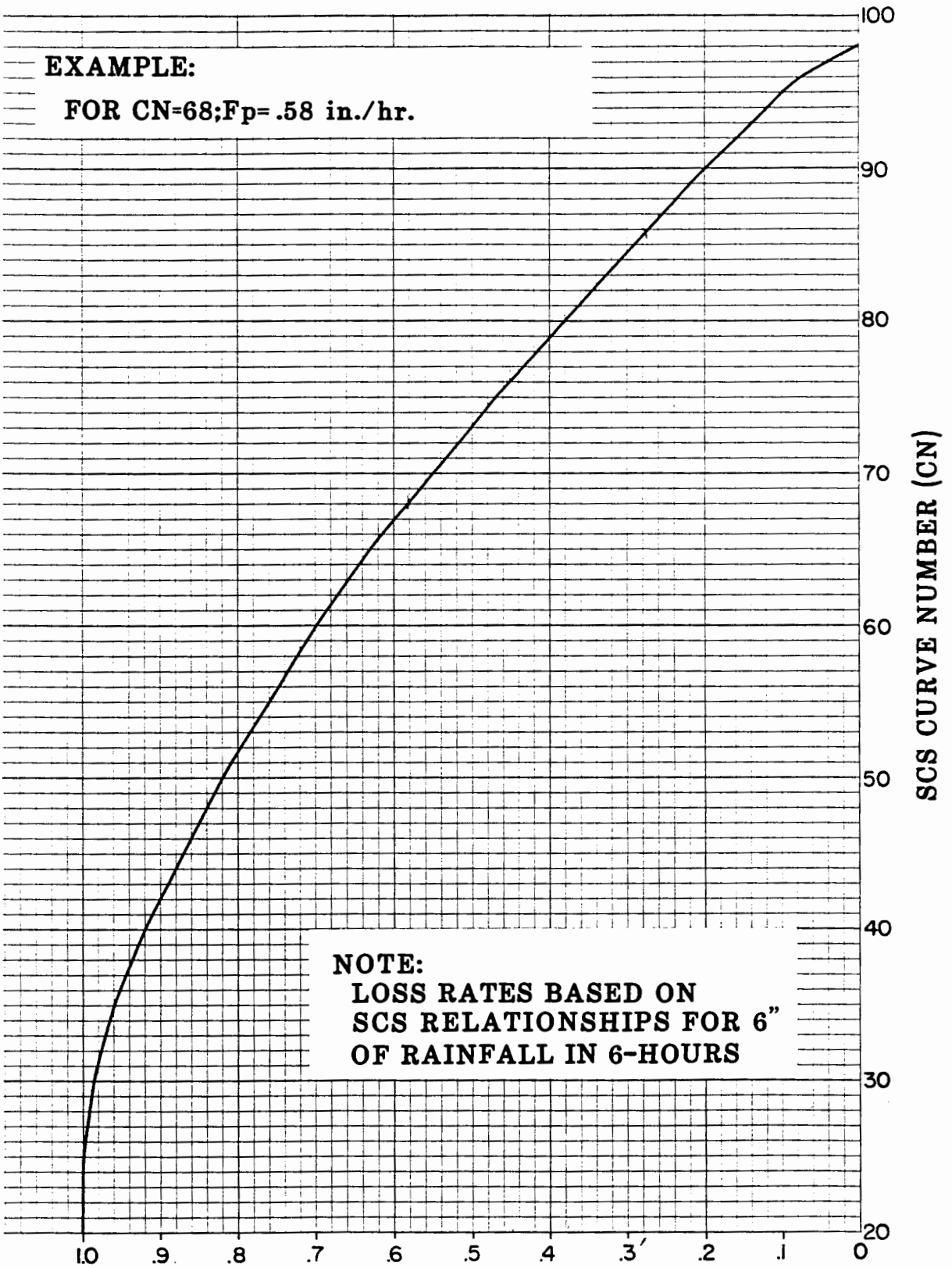


KERN COUNTY

HYDROLOGY MANUAL

SCS 24 - HOUR STORM RAINFALL - RUNOFF RELATIONSHIPS

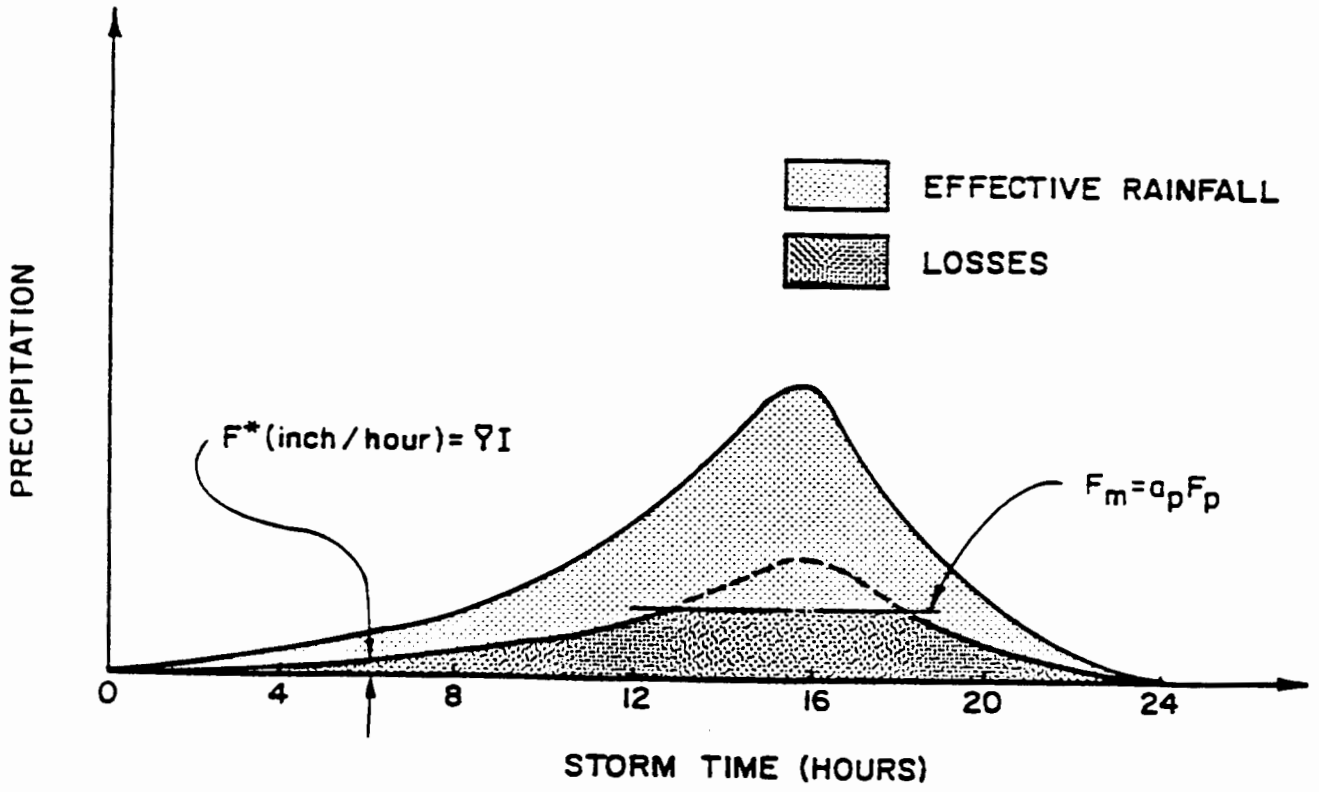
FIGURE C-4



KERN COUNTY
Hydrology Manual

INFILTRATION RATE FOR PERVIOUS AREAS VERSUS SCS CURVE NUMBERS

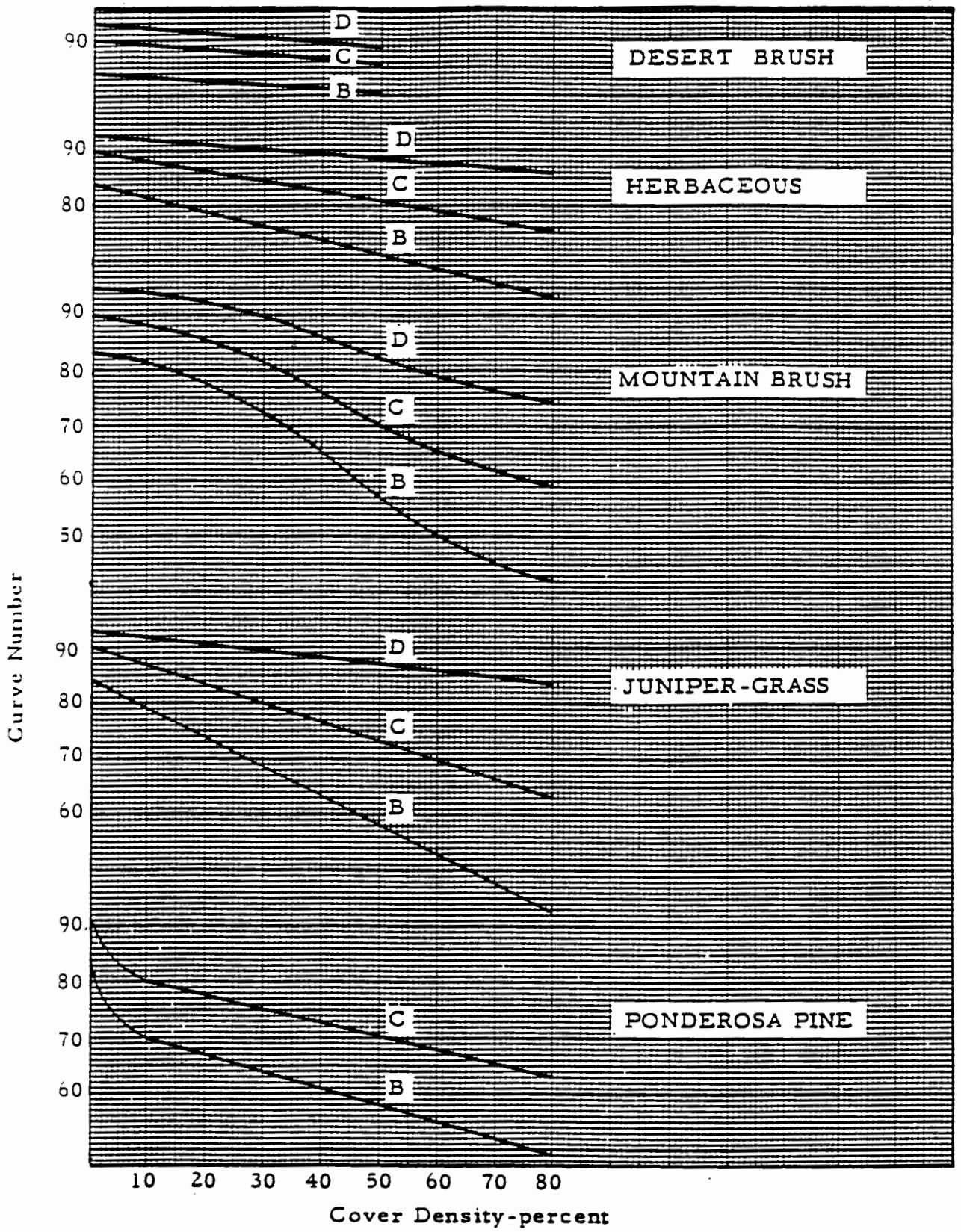
FIGURE C-5



KERN COUNTY
HYDROLOGY MANUAL

DESIGN STORM
LOSS FUNCTION

FIGURE C-6



KERN COUNTY

HYDROLOGY MANUAL

HYDROLOGIC SOIL COVER COMPLEXES AND ASSOCIATED CURVE NUMBERS

FIGURE C-7

Section C
Equations

List of Equations

Equation No.	Equation
C.1	$Ia = 0.2S$
C.2	$S = \frac{1000}{CN} - 10$
C.3	$Y_j = \frac{(P_{24} - Ia)^2}{(P_{24} - Ia + S)P_{24}}$
C.4	$Y = \frac{Y_1 A_1 + ... + Y_m A_m}{A_1 + A_2 + ... + A_m}$
C.5	$\overline{Y} = 1 - Y$
C.6	$F* = \overline{Y} \cdot I$
C.7	$Fm = a_p F_p$

Variables

Ia	Initial Abstraction (inches)
S	Estimate soil capacity
CN	Area Curve Number
Y_j	Yield fraction for 24-hour storm runoff
Y	Yield
$\overline{Y}$	Catchment low loss fraction

SECTION D

RATIONAL METHOD

OVERVIEW

This section will cover the use and application of the Rational Method in hydrologic studies. It will discuss:

- Determination of time-of-concentration
- Peak flow rate calculation
- Confluence analysis
- Time of concentration calculations for Unit Hydrograph studies
- Required study format

Section B should be reviewed for precipitation calculations. Section C should be reviewed to Loss determination. Section J uses the methods described in this section to develop small area runoff hydrographs.

For other important information pertinent to this section refer to B.9., C.7.5., and Equation C.4.

D.1. DEFINITIONS

Average Peak Flow:	In general, the average of the peak flow rates estimated at the upstream and downstream points of the subarea.
Confluence Analysis:	The estimation of the peak flow rate at the junction of two or more streams.
Critical Duration:	The event duration based upon the time of concentration of the drainage area, in the rational method.
Initial Subarea:	The upper most subarea of the watershed, with limits of 1000 feet maximum length and 10 acres maximum area, upon which the assumption of overland flow effects dominate.
Intensity Duration:	Dividing precipitation depth by duration, an average intensity for a specified duration is obtained.
Rational Method:	That hydrologic method that equates peak flow rate to rainfall intensity, runoff coefficient, and drainage area. The method is called rational because the units of the quantities involved are numerically consistent approximately.

Runoff Coefficient:	The ratio of the rate of runoff to the rate of rainfall at an average intensity when the total drainage area is contributing.
Runoff Yield Fraction:	The ratio of that percentage of precipitation which contributes to surface flow to the total event precipitation.
Time of Concentration:	The interval of time required for the flow at a given point to become a maximum under a uniform rainfall intensity.
Travel Time:	The flow distance through a subarea divided by the velocity of flow associated with average peak flow rate passing through the subarea.

D.2. RATIONAL METHOD EQUATION

The ***rational method*** was originally developed to estimate peak discharges from small (less than one square mile) urban and developed areas and its use should normally be limited to those conditions. The rational method equation relates rainfall intensity, a runoff coefficient, and drainage area size to the peak runoff from the drainage area. This relationship is expressed by the equation:

$$Q = CIA \qquad (D.1)$$

where

- Q = the peak discharge in cubic feet per second (cfs)
- C = a runoff coefficient representing the ratio of runoff depth to rainfall depth (dimensionless)
- I = the time-averaged rainfall intensity for a storm duration equal to the time of concentration (inches/hour)
- A = drainage area (acres)

The values of the ***runoff coefficient*** (C) and the ***rainfall intensity*** (I) are based on a study of drainage area characteristics such as type and condition of the runoff surfaces and the time of concentration. These factors and the limitations of the rational method equation are discussed in the following sections. Drainage area (A) may be determined by planimetering a suitable topographic map of the project area.

Data required for the computation of peak discharge by the rational method are: (i) rainfall intensity (I) for a storm of specified duration and selected design frequency; (ii) drainage area characteristics of size (A), shape, slope; and (iii) a runoff coefficient (C).

D.3. LIMITATIONS OF THE RATIONAL METHOD

D.3.1 Validity of Relationship

The validity of the relationship expressed by the rational method equation holds true only if certain assumptions are reasonably correct and limitations of the method are observed. Two basic assumptions are that (i) the frequency of a storm runoff is the same as the frequency of the rainfall producing this runoff; i.e., a 25-year recurrence interval rainfall will provide a 25-year recurrence interval storm runoff, and (ii) that the peak runoff occurs when all effective parts of the drainage area are contributing to the runoff.

D.3.2 Areal Limitations of Rational Method

The rational method equation is only applicable where the rainfall intensity (I) can be assumed to be uniformly distributed over the drainage area at a uniform rate throughout the duration of the storm. This condition is generally assumed to reasonably apply to small areas of less than 640 acres. Beyond this limit, the rainfall distribution may vary considerably from the point values given in rainfall isohyetal maps and the rational method equation may be inappropriate. For this reason, the rational method will not be used to compute design flows for areas greater than 640 acres unless first approved by the County.

D.3.3 Selection Considerations of Runoff Coefficient

The selection of the runoff coefficient (C) is another major limitation for the rational method equation. For small urban and developed areas, the runoff coefficient can be reasonably well estimated from field and aerial photo studies. For larger areas where the determination of the runoff coefficient is to be based on vegetation type, cover density, the infiltration capacity of the ground surface, and the slope of the drainage area, an estimate of the runoff coefficient may be subject to a much greater error due to the variability of the drainage area characteristics.

D.3.4 Other Loss Adjustments

Rainfall losses due to evaporation, transpiration, depression, and channel storage are inadequately evaluated, and may appreciably affect the estimate of the watershed peak rate of runoff, especially in natural cover and desert catchment areas. The effects of depth-area-duration (or depth-area) factors are not accounted for in the simple intensity-duration curve used for rational method studies. For large drainage areas, the absence of depth-area adjustments can result in significant differences in the estimate of the average depth of catchment point rainfalls.

D.3.5 Homogeneity of Watershed

The Rational Method as applied in this manual, also assumes a general homogeneity of loss rates throughout the watershed. In those cases where anomalous results occur due

to a lack of homogeneity, alternative methods of flow determination, under consultation with the county, shall be considered.

D.3.6 Compensating for Result Anomalies

Since the Rational Method does not compensate for peak flow rate reduction by use of transmission losses, attenuation, or routing losses, peak flow rate reduction through a subarea should be considered anomalous and thus the previous upstream flow rate should be used.

D.4. CRITICAL DURATION (TIME OF CONCENTRATION)

The ***critical duration*** of the storm rainfall required in the rational method equation is based on the time of concentration of the drainage area.

D.4.1 Explanation of Time of Concentration

The ***time of concentration*** (Tc) is defined as the interval of time (in minutes) required for the flow at a given point to become a maximum under a uniform rainfall intensity. Often this occurs when all effective parts of the drainage area are contributing to the flow. Generally, the time of concentration is the interval of time from the beginning of rainfall for water from the hydraulically most remote portion of the drainage area to reach the point of concentration; e.g., the inlet of the drainage structure.

D.4.2 Variables Effecting Time of Concentration

The time of concentration is a function of many variables including the length of the flow path from the most remote point of an area to the concentration point, the slope and other characteristics of natural and improved channels in the area, the loss rate characteristics of the soil, and the extent and type of development.

D.4.3 Time of Concentration for Intermediate Recurrence Intervals

Time of concentration estimates are generally computed as a function of peak flow rates corresponding to the subject return frequency storm event. For rational method studies based on this manual, the time of concentration for intermediate recurrence intervals should be assumed to be a function of the time of concentration corresponding to the "100 Year" flow. This relationship is defined in (D.2). The values of K_T, with respect to the common return periods (T), are given in Table D.1.

$$Tc(T) = Tc(100)*K_T \qquad (D.2)$$

Table D.1

K_T Values in (Equation D.2),for Common Return Intervals

Return Period (T)	K_T Value (Equation D.2)
2	1.32
5	1.2
10	1.14
25	1.07
50	1.03
100	1

K_T values should not be applied to ***initial subarea*** time of concentration values.

D.4.4 Determination of Time of Concentration

The time of concentration for an ***initial subarea*** may be estimated from the nomograph of Figure D-1. The time of concentration for the next downstream subarea is computed by adding to the initial Tc, the time required for the computed peak flow to travel to the next concentration point. Time of concentration is computed for each subsequent subarea by computing the runoff peak flow rate travel time between subareas and adding to the cumulative sum. Points of concentration should be selected downstream of the initial subarea such that subarea travel times are less than 3 minutes and 5 minutes for Tc values of less than 30 minutes and 60 minutes, respectively. After a Tc of one hour, subarea travel times should be limited to less than 10 minutes.

When the flow is concentrated in curb and gutters, drainage channels or conduits, the flow velocity may be estimated by the well-known Manning's equation

$$V = \frac{1.49}{n} \cdot R^{2/3} \cdot S^{1/2} \qquad \text{(D.3)}$$

where

V = mean velocity (fps)

n = Manning coefficient of roughness

R = hydraulic radius (feet)

S = energy slope which equals to the conduit invert slope for uniform flow

Figures D-2a (Natural Mountain Channel), D-2b (Local Street Residential, half-width), D-2c (Local Street Commercial, half-width), D-2d (Major/Secondary Highway, half-width) are included to assist in the determination of velocity versus flow. (Note: The applicability of use of Figure D-2a shall be verified by the hydrologist).

The ***travel time*** will then be the flow distance divided by the velocity of flow.

Computations of travel time through subareas which continually add runoff to the peak flow (e.g., streetflow) should be based on the average peak flow through the subarea. This ***average peak flow*** is generally a simple average of the peak flow rates estimated at the upstream and downstream points of the subarea.

The initial subarea Tc estimation often is the most significant factor leading to the Tc computation of a small watershed. Small development studies typically utilize only initial subarea estimations due to the small subarea sizes. Larger study areas generally show high sensitivity to the initial subarea Tc. Consequently, judgment is needed when developing initial subarea Tc estimates. The nomograph of Figure D-1 is based on the Kirpich formula and relates an initial subarea Tc to subarea slope and development type. It is assumed in the nomograph that overland flow effects dominate the travel time hydraulics.

It is noted that the Tc computation procedure is based upon the summation of an initial subarea time of concentration with the several travel times estimated by normal depth flow-velocities of the peak flow rates through subsequent subareas.

D.5. WATERSHED STUDY INTENSITY-DURATION CURVES

Rainfall intensity (I) is determined using ***intensity-duration*** curves which are appropriate for the study watershed.

The County prepared isohyetal maps (Figures B-1 through B-55) showing 2-year 6- and 24-hour, and 100-year 6- and 24-hour point precipitation values. These maps are based on the "NOAA Atlas 2, Precipitation-Frequency Atlas of the Western United States, Volume XI-California" published by the National Weather Service in 1973. A correlation to local rain gauge data and an interpretation of short duration precipitation data was prepared for the County of Kern. Figures B-1 through B-55 can be found at the end of Section B.

One-hour point precipitation values can be estimated using the equations (NOAA Atlas 2) shown in Section B.9, Intensity-Duration Curves.

Point precipitation values for other frequencies can be estimated using the diagram (NOAA Atlas 2) on Figure D-3.

Using Figure D-4, the 1-hour point precipitation value is plotted and a straight line is drawn with the appropriate slope. Generally, a slope of 0.65 may be used for watersheds

in the San Joaquin Valley, 0.60 in the Desert region, 0.55 in the Antelope Valley, Tehachapi Mountains, and Coast Ranges, and 0.45 in the Northern Sierra Nevada Mountains of the County. From Figure D-4, a minimum duration of 5 minutes is used for rational method studies.

D.6. RUNOFF COEFFICIENT

The ***runoff coefficient*** (C) is the ratio of rate of runoff to the rate of rainfall at an ***average intensity*** (I) when the total drainage area is contributing. The selection of the runoff coefficient depends on rainfall intensity, drainage area slope, type and amount of vegetative cover, infiltration capacity of the ground surface, and various other factors.

Since one acre-inch/hour is equal to 1.008 cfs, the rational formula is used to estimate a peak flow rate in cfs. The runoff coefficient is assumed to be a function of the impervious and pervious area fractions, an infiltration rate, F_p, for the pervious area, and the effects of watershed detention. Runoff coefficient curves can be developed using the relationship:

$$C = \begin{cases} 0.90\ (a_i + ap(I-F_p)/I), \text{ for I greater than } F_p \\ 0.90\ a_i, \text{ for I less than or equal to } F_p \end{cases} \quad \text{(D.4)}$$

where the proportion factor of 0.90 is a calibration constant determined by an average fit between the rational method and design storm unit hydrograph (see Section E) peak flow rate estimates, and where

C = runoff coefficient

I = rainfall intensity (inches/hour)

F_p = infiltration rate for pervious areas (inches/hour) (see Section C.7.4)

a_i = ratio of impervious area to total area (decimal fraction)

a_p = ratio of pervious area to total area (decimal fraction), ($a_p = 1 - a_i$)

D.7. PEAK FLOW RATE FORMULA

Combining Equations (D.1) and (D.4);

$Q = 0.90\ (a_i + a_p(I-F_p)/I)IA$

$= 0.90\ (a_iI + a_pI - a_pF_p)A$

$= 0.90\ (I(a_i+a_p) - a_pF_p)A$

$a_i + a_p = 1$; $a_pF_p = F_m$; Therefore:

$$Q = .90\ (I - F_m)A \quad \text{(D.5)}$$

where $F_m = a_p F_p$ (see Section C.7.5), and where in (D.5) it is understood that I is greater than F_p (otherwise $Q = .90\, a_i IA$).

The Rational Method should be considered as an estimator of the peak flow of the catchment runoff hydrograph; therefore, the minimum flow from a catchment will be a function of the 24-hour storm ***runoff yield fraction***, Y, as calculated by equations (C.3) and (C.4) such that,

$$Q(\text{min}) = .9YIA \qquad \text{(D.6)}$$

In (D.5), F_m represents the loss rate for the total watershed tributary to the point of concentration. Should the tributary area contain several runoff surfaces, an area-averaged F_m is calculated. Table D.2 illustrates such an area-averaged F_m computation.

TABLE D.2. AREA-AVERAGED F_m COMPUTATION DATA

Subarea Number	a_p	F_p (inch/hour)	Area (Acres)	Area Weighing
(1)	(2)	(3)	(4)	(5)
1	0.6	0.4	8	1.92
2	0.8	0.3	12	2.88
3	0.75	0.25	11	2.06
4	0.1	0.2	15	0.3
5	0.5	0.25	16	2
		Sum	62	9.16

From Table D.2, the area-averaged maximum loss rate F_m, is given by $F_m = (9.16)/(62) = 0.146$ inch/hour, say 0.15.

D.8. DRAINAGE AREA

The contributing drainage area may be determined from topographic contour maps, aerial photos, and field surveys. Watershed divides are then drawn on a suitable topographic map and the enclosed drainage area is determined by planimeter or other methods.

In areas where lateral and transverse slopes on the watershed are very mild, the nominal watershed area (or drainage subdivision) runoff may "cascade" under severe rainfall. That is, when the divide between one watershed and another is defined by a low relief feature such as the crown of a road, the runoff from such a watershed may "spill over" into the adjacent watershed or watershed subdivision. This may occur, for example,

when gutter capacity is exceeded thereby increasing runoff contributions at downstream or adjacent concentration points above those anticipated by analysis of the nominal or "low flow" drainage boundaries. The possibility of such cascading shall be considered and accounted for by the hydrologist.

The hydrologist should avoid the inclusion of ineffective flow areas (i.e. long "fingerlets" of area) in the initial subarea of the watershed. The addition of such areas tends to add to the time of concentration while contributing a disproportionately small watershed area. The result will be falsely lower peak flow rate estimates for the watershed as a whole.

D.9. RATIONAL METHOD CONFLUENCE ANALYSIS

In most studies, the calculation of peak flow rates along a main channel or stream involves only the direct application of (D.5). Such studies typically involve the inclusion of subarea runoff to the stream where the effect on the stream peak flow rate is relatively minor and, consequently, only (D.5) is needed for the analysis.

At the junction of two or more streams, however, the estimation of the peak flow rate involves a ***confluence analysis*** of the associated runoff hydrographs (see Sections E and J). The equations defined for confluence analysis assume that the catchment peak flow rates, determined from the rational method, are only portions of larger hydrograph events.

For the confluence of two streams, let T_1, I_1, F_{m1}, Y_1, A_1, and Q_1, be the time of concentration, rainfall intensity, area-averaged loss rate, area averaged runoff yield fraction, catchment area, and peak flow rate for stream 1 while T_2, I_2, F_{m2}, Y_2, A_2, and Q_2 correspond to stream 2. Also let Q_1 be less than Q_2. Finally, let T_p, A_p, and Q_p be the resulting confluence estimates for Tc, area, and peak flow rate, respectively. Then two cases are possible:

*Case 1: $T_1 = T_2$. The runoff hydrographs must both peak at $T_p = T_1 = T_2$. And $Q_p = Q_1+Q_2$ for a total contributing area of $A_p = A_1+A_2$.

*Case 2: $T_1 \neq T_2$. In this case, the assumption of the summing of two runoff hydrographs must be considered. Generally, Q_p will occur at time T_2, and the peak flow rate estimates are calculated as follows:

$$T_p = T_2$$

$$Q_p = Q_2 + \text{Adjusted}[Q_1] \qquad \text{(D.7)}$$

The general equation for estimating the Adjusted[Q_1] is as follows:

$$\text{Adjusted}[Q_1] = Q_1 * \text{Minimum of} \quad \frac{\text{Maximum of } [(I_2 - Fm_1),\ (Y_1 I_2)]}{\text{Maximum of } [(I_1 - Fm_1),\ (Y_1 I_1)]}\ ,\ \frac{(T_2+T_1)}{2T_1} \qquad \text{(D.8)}$$

Thus, if Q_1 has a shorter time of concentration than Q_2 (i.e T_1 is less than T_2), four possibilities exist:

1) If $I_2 - F_{m1} > Y_1 I_2$ and $I_1 - F_{m1} > Y_1 I_1$ then:

$$Adjusted[Q_1] = \frac{I_2 - Fm_1}{I_1 - Fm_1} \cdot Q_1 \qquad \text{(D.8a)}$$

It is noted that the confluence peak Q of (D.7), with substitution from (D.8a), equals the peak flow rate estimated from direct use of (D.5). Additionally, the total contributing area is $A_p = A_1 + A_2$.

2) When $T_2 > T_1$, no value of Y_1 can exist which will meet the conditions; $I_2 - F_{m1} > Y_1 I_2$ and $I_1 - F_{m1} < Y_1 I_1$ simultaneously.

3) If $I_2 - F_{m1} < Y_1 I_2$ and $I_1 - F_{m1} > Y_1 I_1$ then:

$$Adjusted[Q_1] = \frac{Y_1 \cdot I_2}{I_1 - Fm_1} \cdot Q_1 \qquad \text{(D.8b)}$$

The total contributing area is $A_p = A_1 + A_2$

4) If $I_2 - F_{m1} < Y_1 I_2$ and $I_1 - F_{m1} < Y_1 I_1$ then:

$$Adjusted[Q_1] = \frac{I_2}{I_1} \cdot Q_1 \qquad \text{(D.8c)}$$

The total contributing area is $A_p = A_1 + A_2$

Also, if Q_2 has a shorter time of concentration than Q_1, (i.e T_1 is greater than T_2), then:

$$Adjusted[Q_1] = \frac{T_2 + T_1}{2T_1} \cdot Q_1 \qquad \text{(D.8d)}$$

This assumes that the shape of the antecedent leg of the hydrograph, near the peak, is linear and that flow is occurring from the catchment prior to the rational method peak determination. This linearity is a function of Tc and area, only, and not intensity.

The assumption of a linear relationship between catchment Tc and area requires the adjustment of the catchment tributary to stream 1, so that for downstream study purposes, the adjusted catchment area corresponds to the subject peak flow rate.

It is noted that in the adjusted peak flow rate estimate of (D.8), the critical duration is $T_p = T_2$. Consequently, the peak flow rate contribution from the adjusted catchment area of stream 1 must reflect the higher rainfall intensity corresponding to time T_2 rather than time T_1. Use of (D.8) results in a peak flow which equals the governing rational method peak flow rate estimate from (D.5) applied to the adjusted catchment area computed by (D.9).

The adjusted area equation is derived as,

$$A_p = A_2 + \text{Adjusted}[A_1] \qquad \text{(D.9)}$$

When $I_2\text{-}F_{m1} > Y_1 I_2$ and $I_1\text{-}F_{m1} > Y_1 I_1$, from (D.8):

$$Adjusted[A_1] = \frac{(T_2 + T_1) \cdot (I_1 - F_{m1})}{2T_1 \cdot (I_2 - F_{m1})} \cdot A_1 \qquad \text{(D.9a)}$$

When $I_2\text{-}F_{m1} > Y_1 I_2$ and $I_1\text{-}F_{m1} < Y_1 I_1$, from (D.8):

$$Adjusted[A_1] = \frac{(T_2 + T_1) \cdot (Y_1 \cdot I_1)}{2T_1 \cdot (I_2 - F_{m1})} \cdot A_1 \qquad \text{(D.9b)}$$

When $I_2\text{-}F_{m1} < Y_1 I_2$ and $I_1\text{-}F_{m1} < Y_1 I_1$, from (D.8):

$$Adjusted[A_1] = \frac{(T_2 + T_1) \cdot I_1}{2T_1 \cdot I_2} \cdot A_1 \qquad \text{(D.9c)}$$

D.10. RATIONAL METHOD Tc CALCULATIONS FOR UNIT HYDROGRAPH STUDIES

Although the peak flow rate formula should generally not be used for catchments larger than one square mile, the rational method can be used to estimate Tc values for larger areas. That is, the rational method Tc estimate for an area of approximately one square mile, plus the sum of travel times (using normal depth flow velocities for the average peak flow rate derived from the runoff hydrograph) for subsequent flow reaches

may be adequate for use in the design storm unit hydrograph studies of Section E. Generally, downstream reach lengths should be chosen such that:

reach travel time <3 minutes for Tc <30 minutes

reach travel time <5 minutes for Tc <60 minutes

reach travel time <10 minutes for Tc >60 minutes

Figure D-6 illustrates the use of peak flow rates computed by the design storm unit hydrograph method of Section E, for estimation of Tc values at downstream points of concentration.

D.11. REQUIRED FORMAT

Figure D-7 illustrates the required format for the submittal of rational method hydrology studies. All rational method calculations must be summarized on the form shown in Figure D-8.

D.12. INSTRUCTIONS FOR RATIONAL METHOD HYDROLOGY CALCULATIONS

Note: The time of concentration (Tc) for intermediate recurrence intervals should be assumed to be a function of the Tc corresponding to the "100 year" flow. Therefore, this procedure should be applied to the determination of the "100 year" flow. All other required intermediate recurrence interval Tc's, other than for initial subareas, should be based upon Equation D.2 using the values shown in Table D.1.

1. On a topographic map of the drainage area, draw the study drainage system and designate subareas tributary to the various points of concentration (see example problem).

2. Prepare the Intensity-Duration Curve, Figure D-4, appropriate to the area under study.

   2a. Determine the 2-Year, 6-Hour precipitation depth (X1), using Figures B-1 to B-11.

   2b. Determine the 2-Year, 24-Hour precipitation depth (X2), using Figures B-12 to B-22.

   2c. Determine the 100-Year, 6-Hour precipitation depth (X3), using Figures B-34 to B-44.

   2d. Determine the 100-Year, 24-Hour precipitation depth (X4), using Figures B-45 to B-55.

   2e. Determine the Latitude (in decimals) at the watershed centroid, minus 32 (X5), using the USGS Quadrangle maps.

   2f. Calculate the 2-Year, 1-Hour (Y2) and 100-Year, 1-hour (Y100) precipitation depths, using Equations B.2 to B.12.

   2g. Determine the appropriate rainfall depth for the required return interval, using Figure D-3.

   2h. Determine the slope (S) of the Intensity-Duration curve, using Figure B-56.

   2i. Plot the Intensity-Duration curve, using the 1-hour rainfall depth determined in step 2g, the slope (S) from step 2h, and Figure D-4.

3. Determine the initial time of concentration, (Tc), using Figure D-1. The initial subarea should be <u>less than 10 acres,</u> have a flow path of <u>less than 1,000 feet,</u> and generally should be the most upstream subarea of the watershed drainage system.

4. Using the time of concentration, determine (I) (intensity of rainfall in inches per hour) from the appropriate intensity-duration curve for the particular area under study using Figure D-4.

5. Calculate the area-averaged maximum loss rate, F_m, which corresponds to the soil group, cover complex, and imperviousness of the drainage subarea. Loss rates for the pervious area, F_p, follow from Section C.7.4.

6. Calculate the area-averaged runoff yield fraction, Y, using equation (C.4).

7. Determine the area (A, acres) of the <u>total</u> watershed tributary to the point of concentration. Because the rational method computational results are sensitive to the subarea size definitions (especially in the most upstream reaches of the watershed), limit the size of subareas to allow for a gradual increase in subarea size as the study progresses downstream. The method is sensitive to large differences in successive subarea shapes, and lengths of reaches where travel times are estimated. Points of concentration should be selected downstream of the initial subarea such that subarea travel times are less than 3 minutes and 5 minutes for Tc values less than 30 minutes and 60 minutes, respectively. After a Tc of one hour, subarea travel times should be limited to less than 10 minutes.

8a. Compute $Q = .90\ (I-F_m)A$ for the point of concentration.

8b. Compute $Q(min) = .9YIA$ for the point of concentration.

8c. Use the higher of the values determined in steps 8a or 8b.

8d. Should the computed Q be less than the previous upstream point of concentration Q, use the upstream Q value.

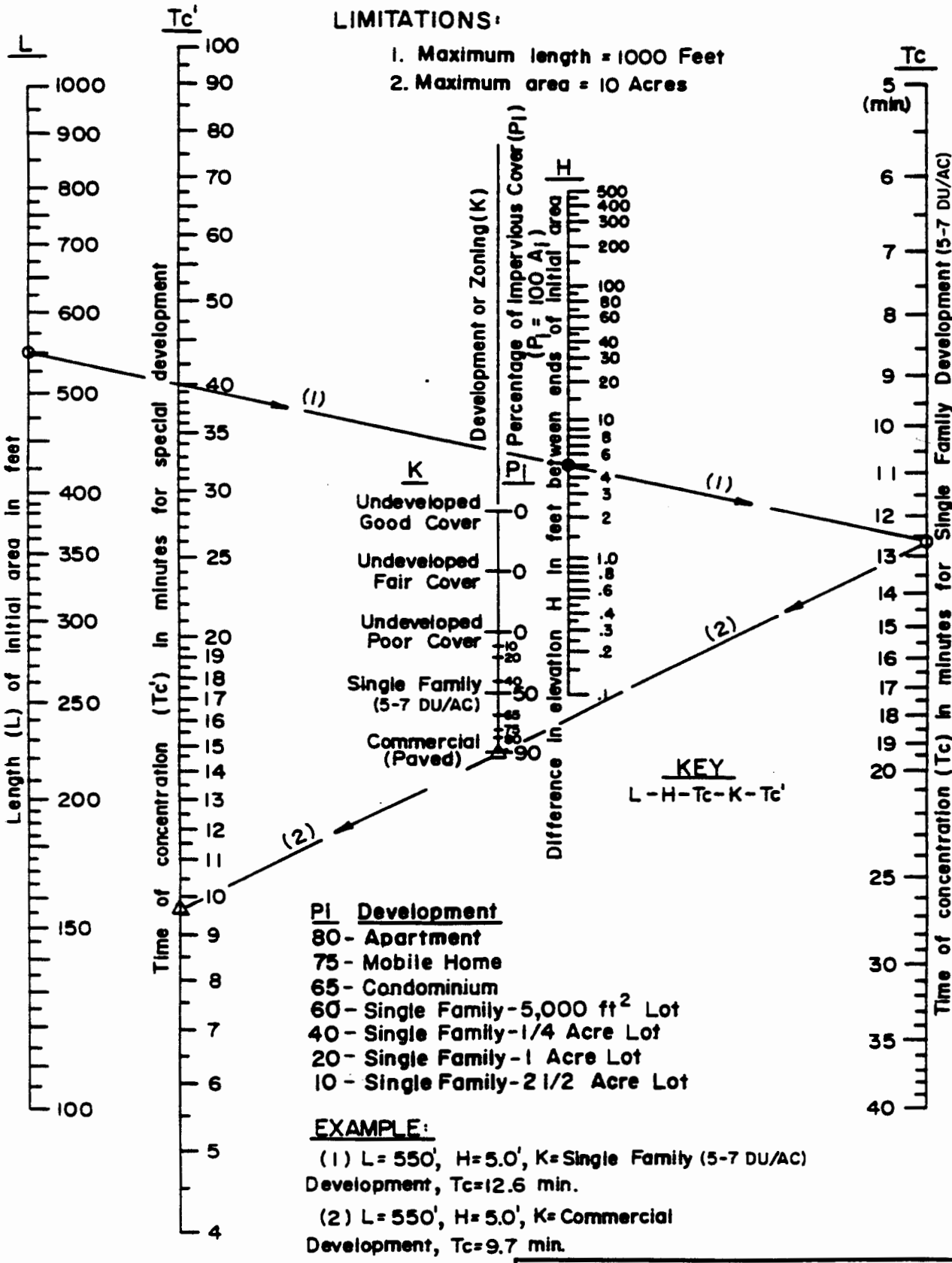
9. Measure the length that the peak runoff must travel to the point of concentration of the next downstream subarea. Determine the average velocity of flow in this reach using the mean peak Q in the appropriate type of conveyance being considered (natural channel, street, pipe, or open channel) using Manning's formula.

   Using the reach length and average flow velocity, compute the travel time and add to the time of concentration from the upstream subarea to determine the downstream point time of concentration.

10. Calculate Q for the new point of concentration using Steps 4 through 8 and the new time of concentration. Determine the time of concentration for the next downstream subarea using Step 9. Continue the above computation procedure downstream until a junction with a lateral drain is reached.

11. Start at the upstream end of the lateral and compute its Q down to the junction with the main line, using the methods outlined in the previous steps.

12. Compute the peak Q at the junction (confluence analyses (see Section D.9)) and evaluate the sensitivity of the computed results to using the other Q and Tc values determined. That is, the downstream estimated peak Q values may be higher had a lower Q and lower Tc value been used at an upstream confluence point. The largest Q is, therefore, estimated along the entire watershed main channel.



KERN COUNTY
HYDROLOGY MANUAL

TIME OF CONCENTRATION NOMOGRAPH FOR INITIAL SUBAREA

Figure D-1

Flow Chart
Natural Mountain Channel

Velocity (V) fps

Flow (Q) cfs

Natural Mountain Channel
cross-section

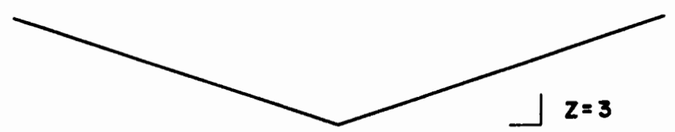


Figure D-2a

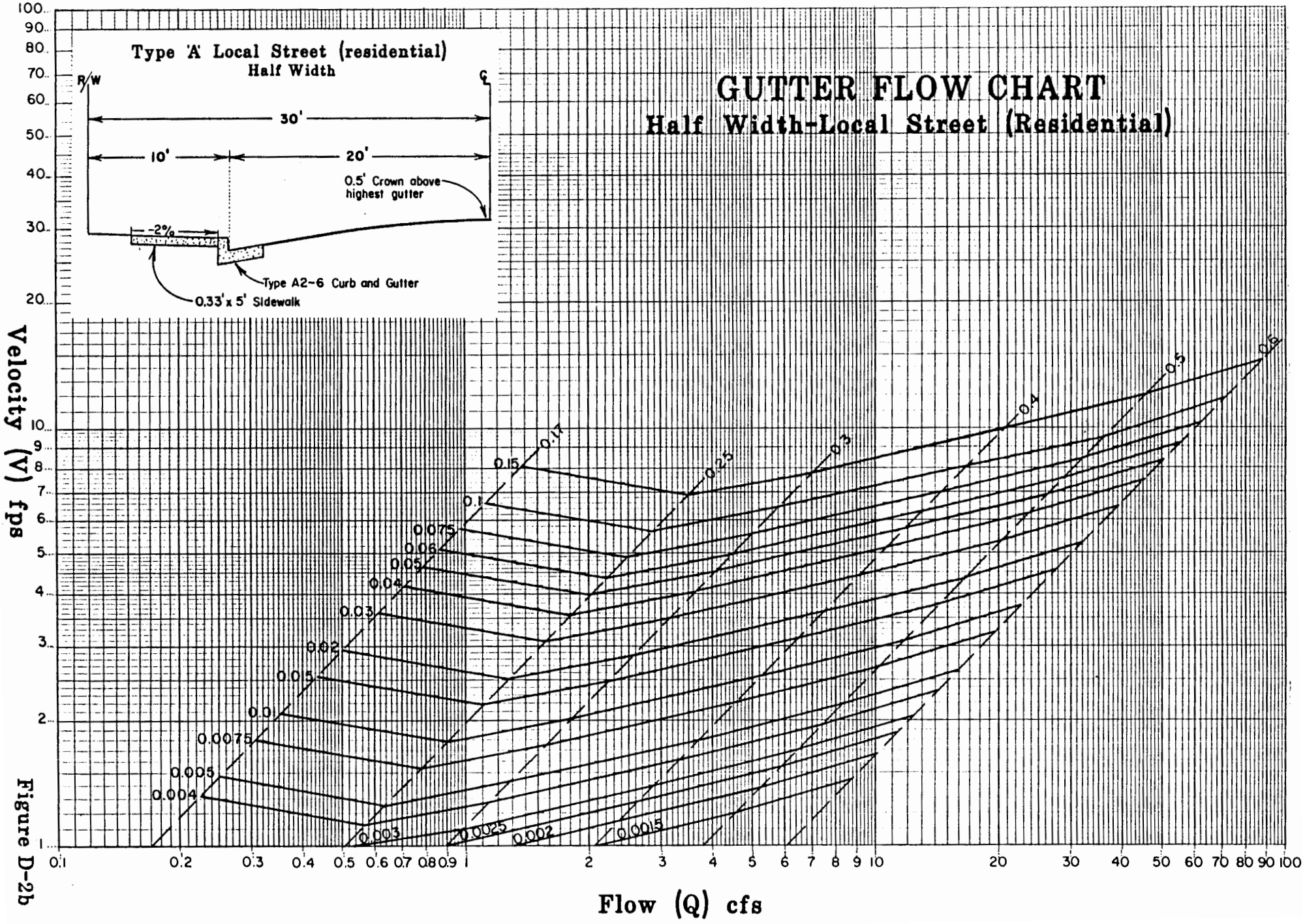

Type 'A' Local Street (residential)
Half Width
R/W
℄
30'
10'
20'
0.5' Crown above highest gutter
-2%
Type A2-6 Curb and Gutter
0.33' x 5' Sidewalk
GUTTER FLOW CHART
Half Width-Local Street (Residential)
Velocity (V) fps
Flow (Q) cfs
0.17
0.15
0.1
0.075
0.06
0.05
0.04
0.03
0.02
0.015
0.01
0.0075
0.005
0.004
0.003
0.0025
0.002
0.0015
0.25
0.3
0.4
0.5
0.6

Figure D-2b

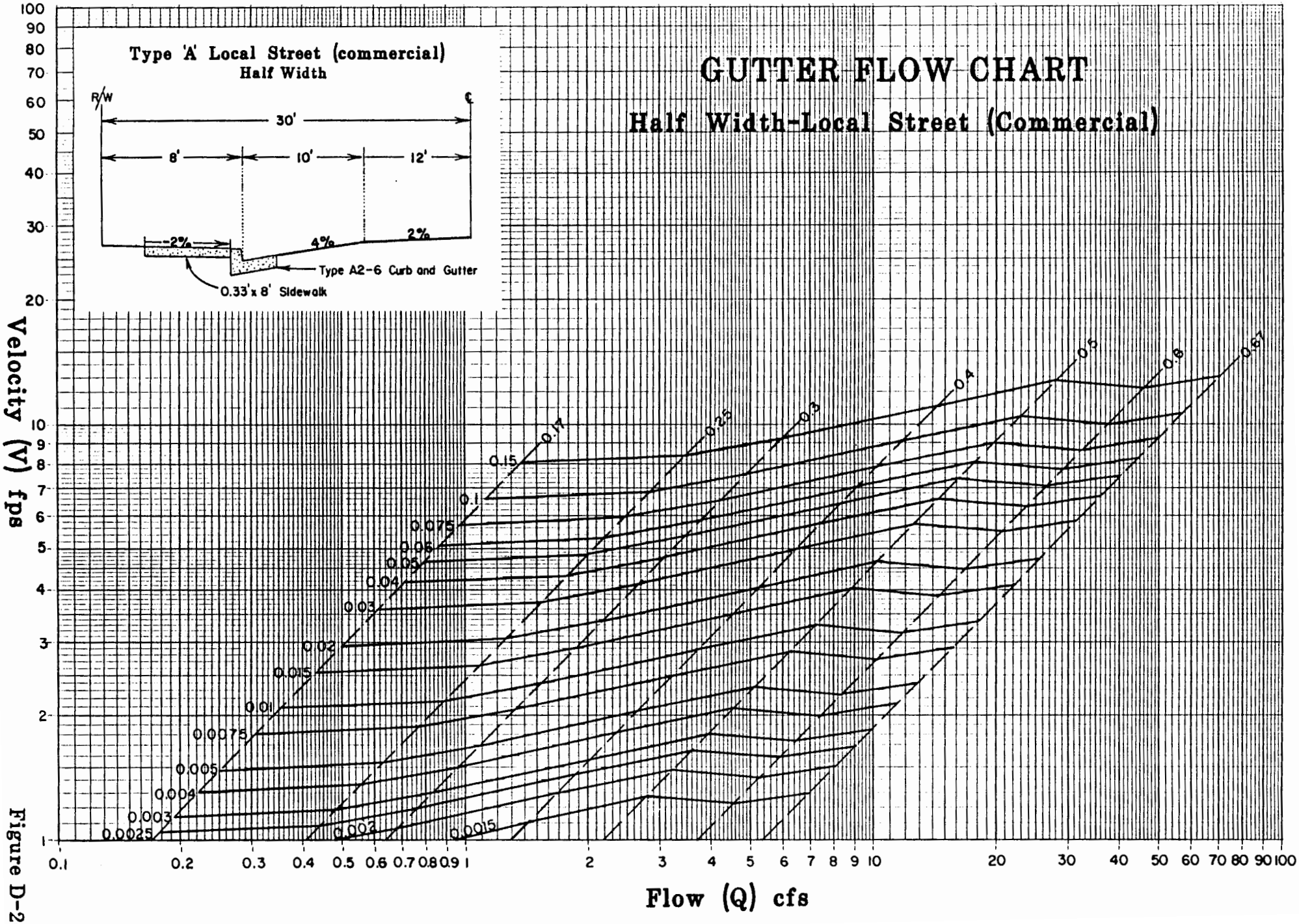

GUTTER FLOW CHART
Half Width-Local Street (Commercial)
Type 'A' Local Street (commercial)
Half Width
R/W
℄
30'
8'
10'
12'
2%
4%
2%
Type A2-6 Curb and Gutter
0.33'x 8' Sidewalk
Velocity (V) fps
Flow (Q) cfs
100
90
80
70
60
50
40
30
20
10
9
8
7
6
5
4
3
2
1
0.1
0.2
0.3
0.4
0.5
0.6
0.7
0.8
0.9
1
2
3
4
5
6
7
8
9
10
20
30
40
50
60
70
80
90
100
0.5
0.8
0.67
0.4
0.25
0.3
0.17
0.15
0.1
0.075
0.06
0.05
0.04
0.03
0.02
0.015
0.01
0.0075
0.005
0.004
0.003
0.0025
0.002
0.0015

Figure D-2c

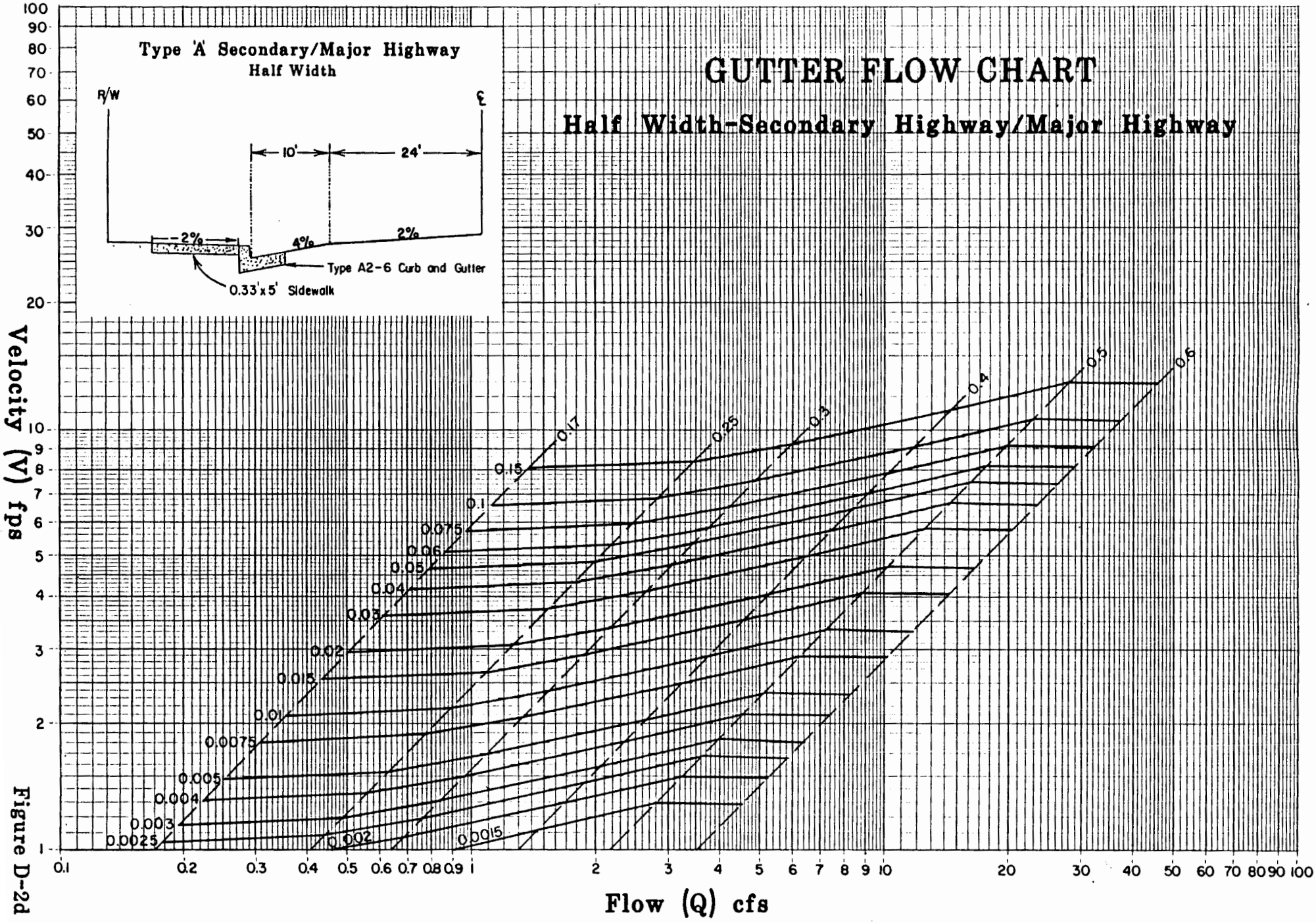
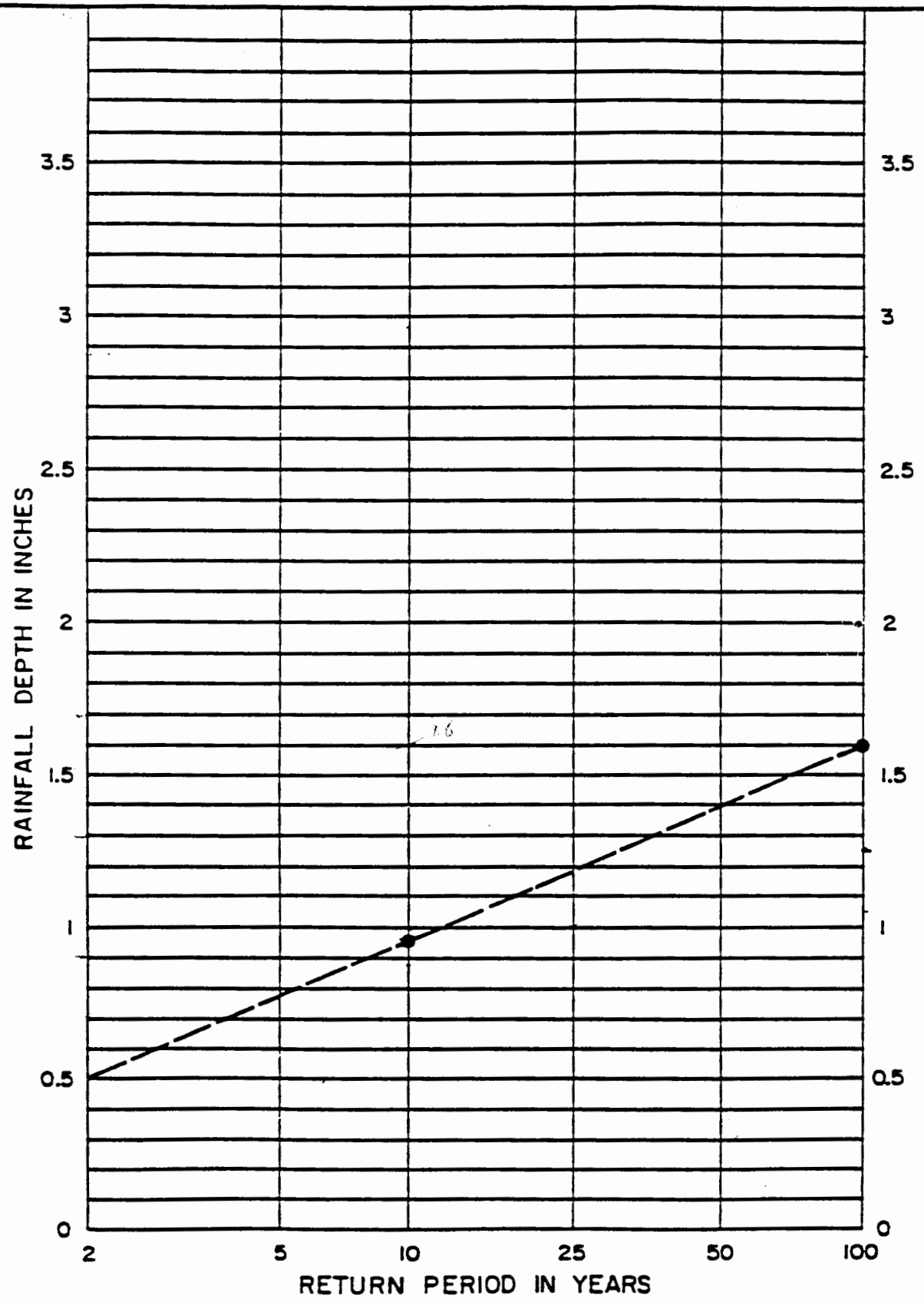

GUTTER FLOW CHART
Half Width-Secondary Highway/Major Highway
Type 'A' Secondary/Major Highway
Half Width
R/W
℄
10'
24'
2%
4%
2%
Type A2-6 Curb and Gutter
0.33'x5' Sidewalk
Velocity (V) fps
Flow (Q) cfs
0.6
0.5
0.4
0.3
0.25
0.17
0.15
0.1
0.075
0.06
0.05
0.04
0.03
0.02
0.015
0.01
0.0075
0.005
0.004
0.003
0.0025
0.002
0.0015

Figure D-2d



NOTE:

1. FOR INTERMEDIATE RETURN PERIODS PLOT 10-YEAR AND 100-YEAR ONE HOUR VALUES FROM MAPS, THEN CONNECT POINTS AND READ VALUE FOR DESIRED RETURN PERIOD. FOR EXAMPLE GIVEN 10-YEAR ONE HOUR = 0.95" AND 100-YEAR ONE HOUR = 1.60", 25-YEAR ONE HOUR = 1.18".

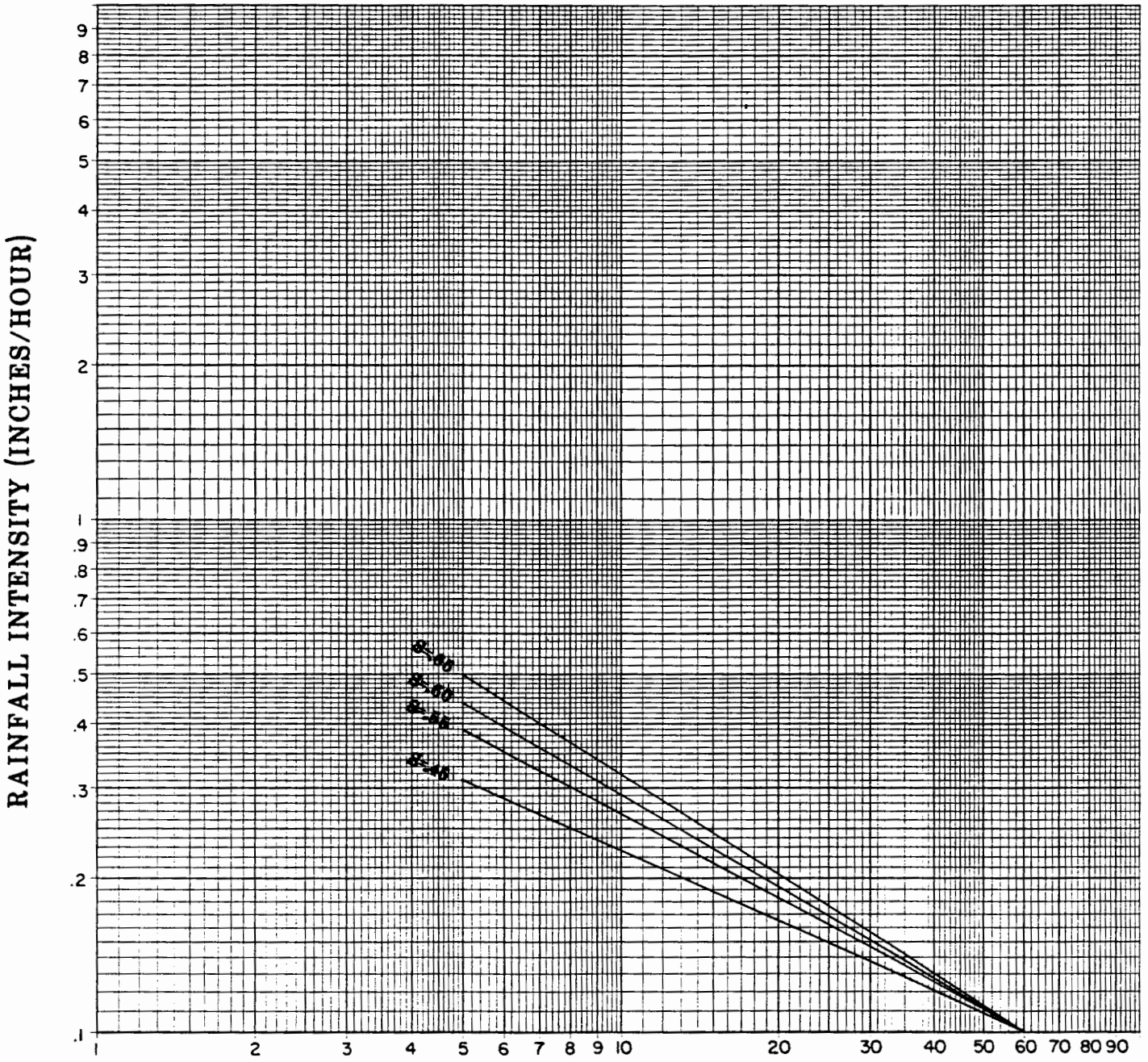
REFERENCE: NOAA ATLAS 2, VOLUME XI - CAL., 1973

KERN COUNTY

HYDROLOGY MANUAL

RAINFALL DEPTH VERSUS RETURN PERIOD FOR PARTIAL DURATION SERIES

FIGURE D-3



DESIGN STORM FREQUENCY = _____ YEARS

ONE HOUR POINT RAINFALL= _____ INCHES

LOG-LOG SLOPE= _____

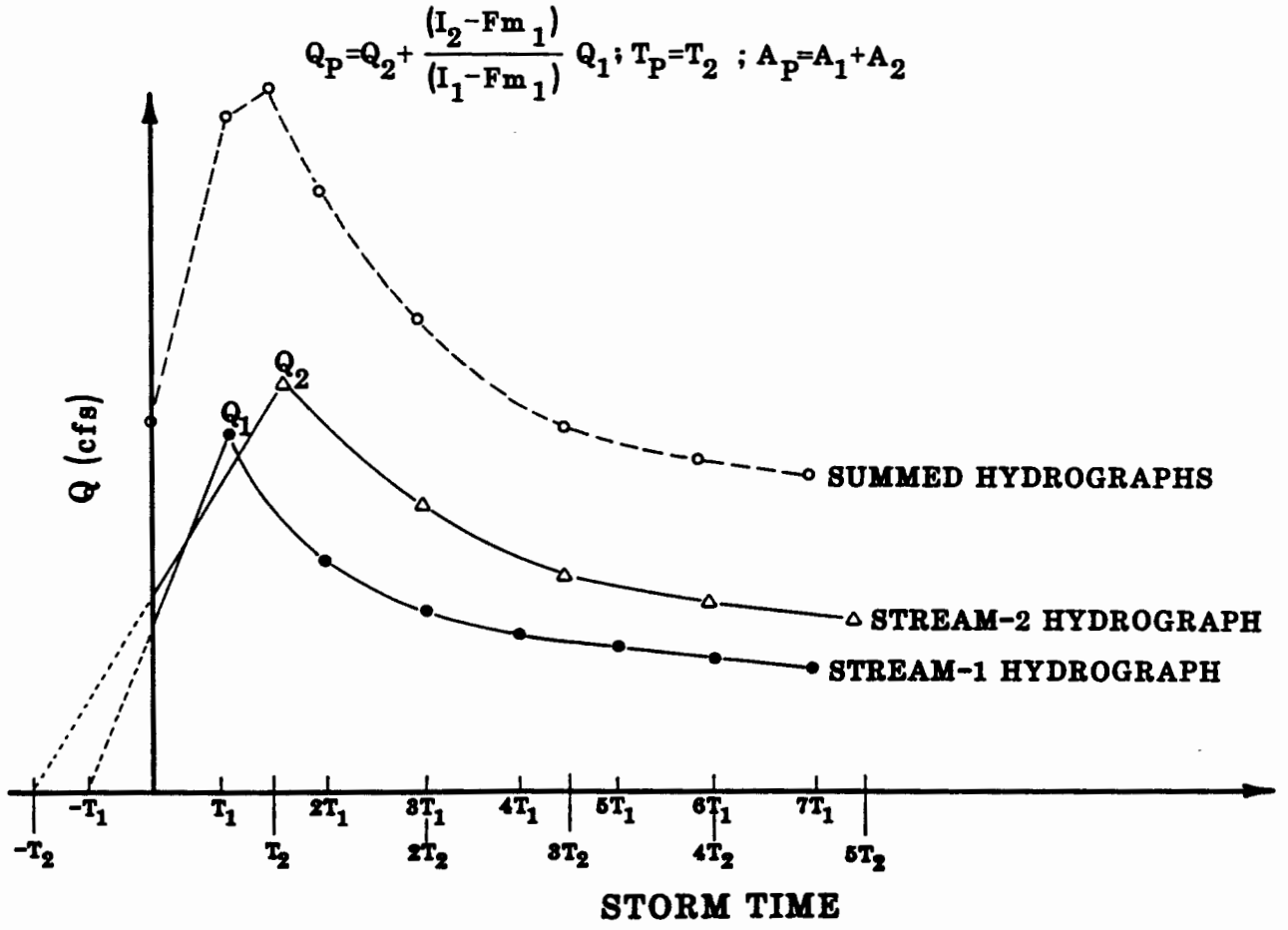
PROJECT LOCATION= _____

KERN COUNTY
Hydrology Manual

INTENSITY-DURATION CURVES

CALCULATION SHEET

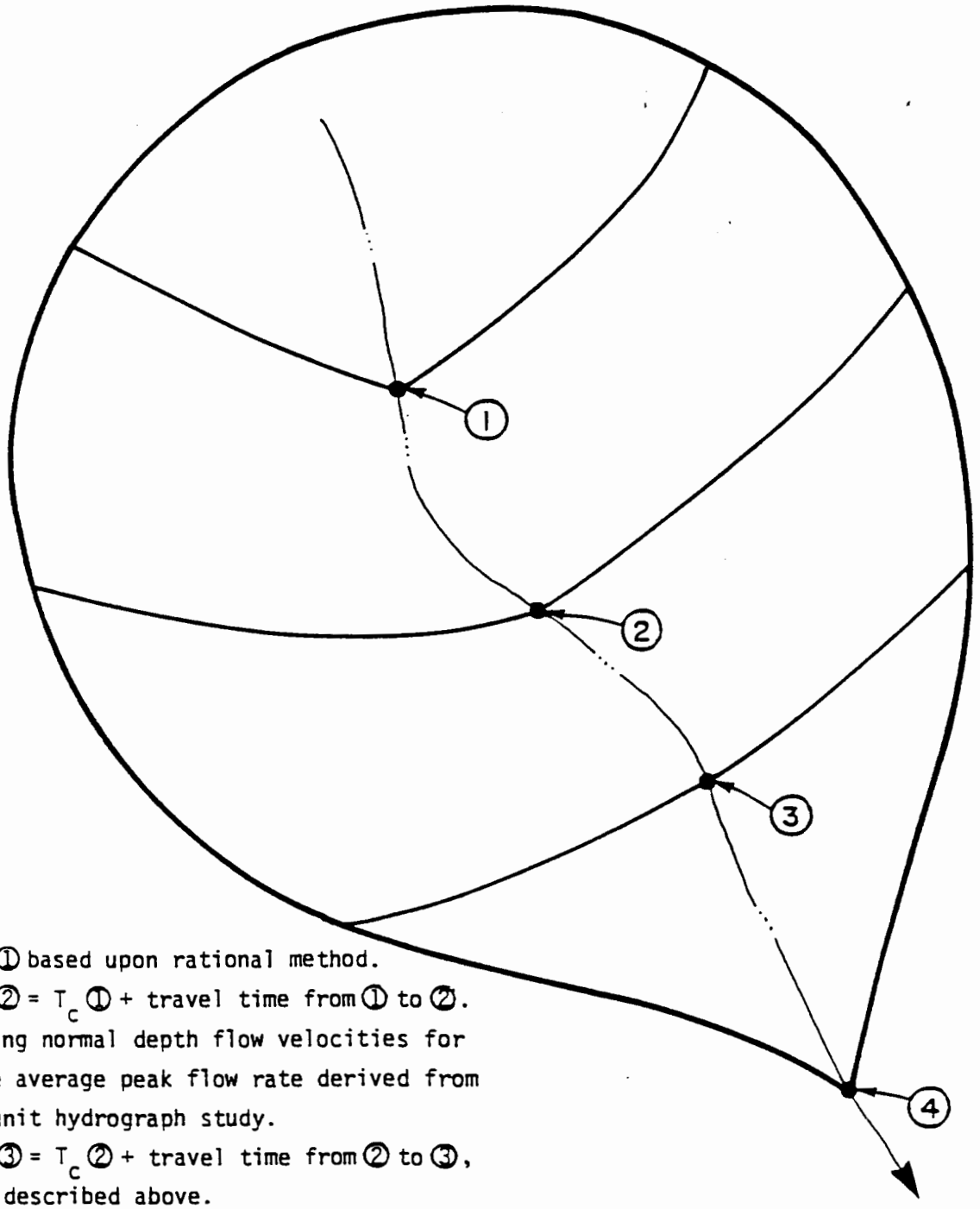
FIGURE D-4



KERN COUNTY
Hydrology Manual

RATIONAL METHOD
CONFLUENCE ANALYSIS
(Summation of Runoff Hydrographs)

FIGURE D-5

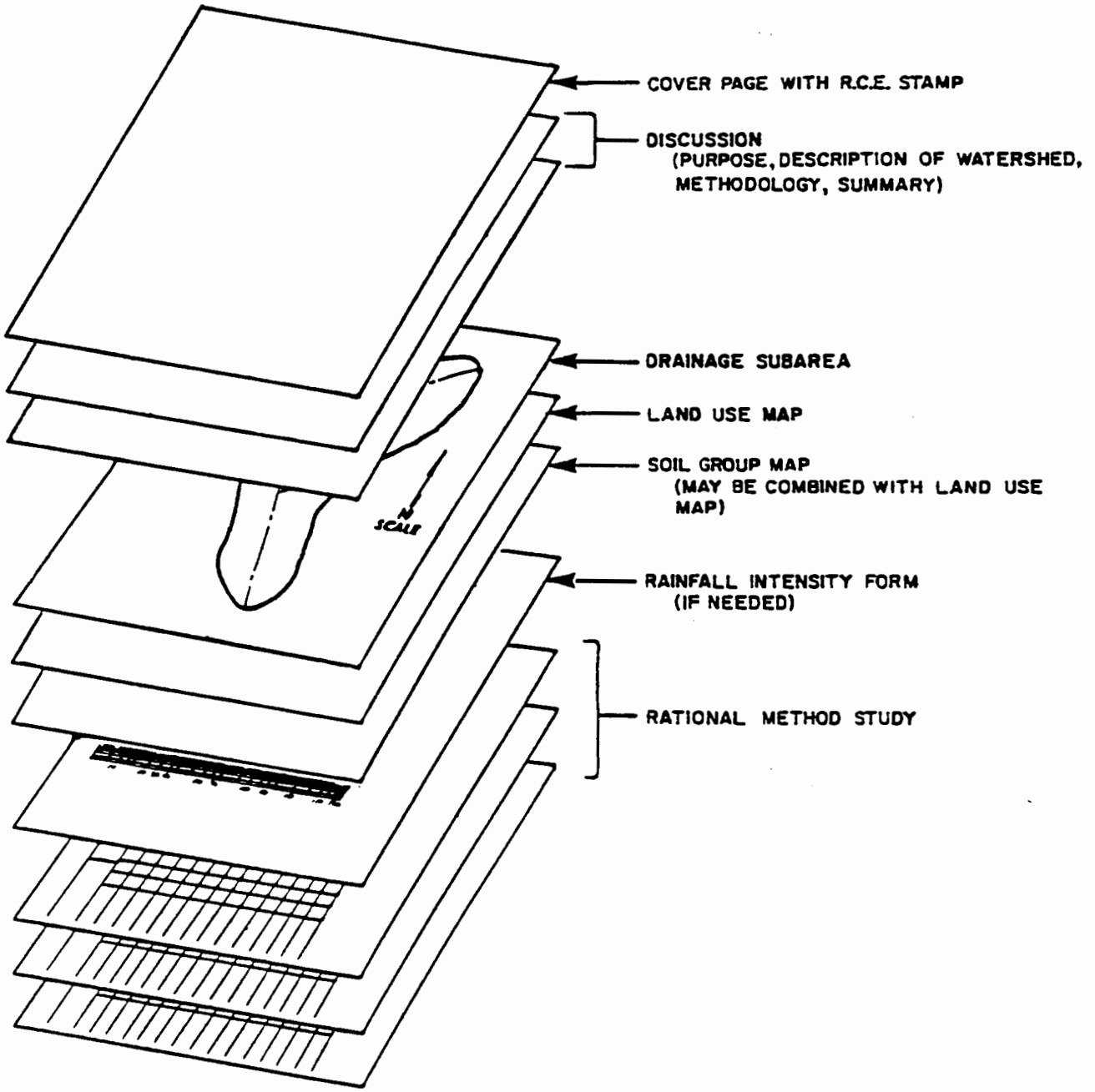


1. T_c ① based upon rational method.
2. T_c ② = T_c ① + travel time from ① to ②. using normal depth flow velocities for the average peak flow rate derived from a unit hydrograph study.
3. T_c ③ = T_c ② + travel time from ② to ③, as described above.
4. T_c ④ = T_c ③ + travel time from ③ to ④, as described above.
5. In general, reach lengths should be chosen such that:
   travel time < 3 minutes for T_c < 30 minutes
   travel time < 5 minutes for T_c < 60 minutes
   travel time < 10 minutes for T_c < 60 minutes

KERN COUNTY
HYDROLOGY MANUAL

RATIONAL METHOD Tc CALCULATIONS FOR UNIT HYDROGRAPH STUDIES

FIGURE D-6



KERN COUNTY

HYDROLOGY MANUAL

TYPICAL REPORT FORMAT FOR RATIONAL METHOD STUDY

FIGURE D-7

KERN COUNTY HYDROLOGY MANUAL: Rational Method Study Form

Study Name: ______________________________ 1-Hour Rainfall (inches)= _____ Slope= _____

Storm Return Interval: ______ 24-Hour Rainfall (inches)= _____ K(T)= _____

Calculated by ____________________ Date ________

Checked by ____________________ Date ________

Page ____ of ______

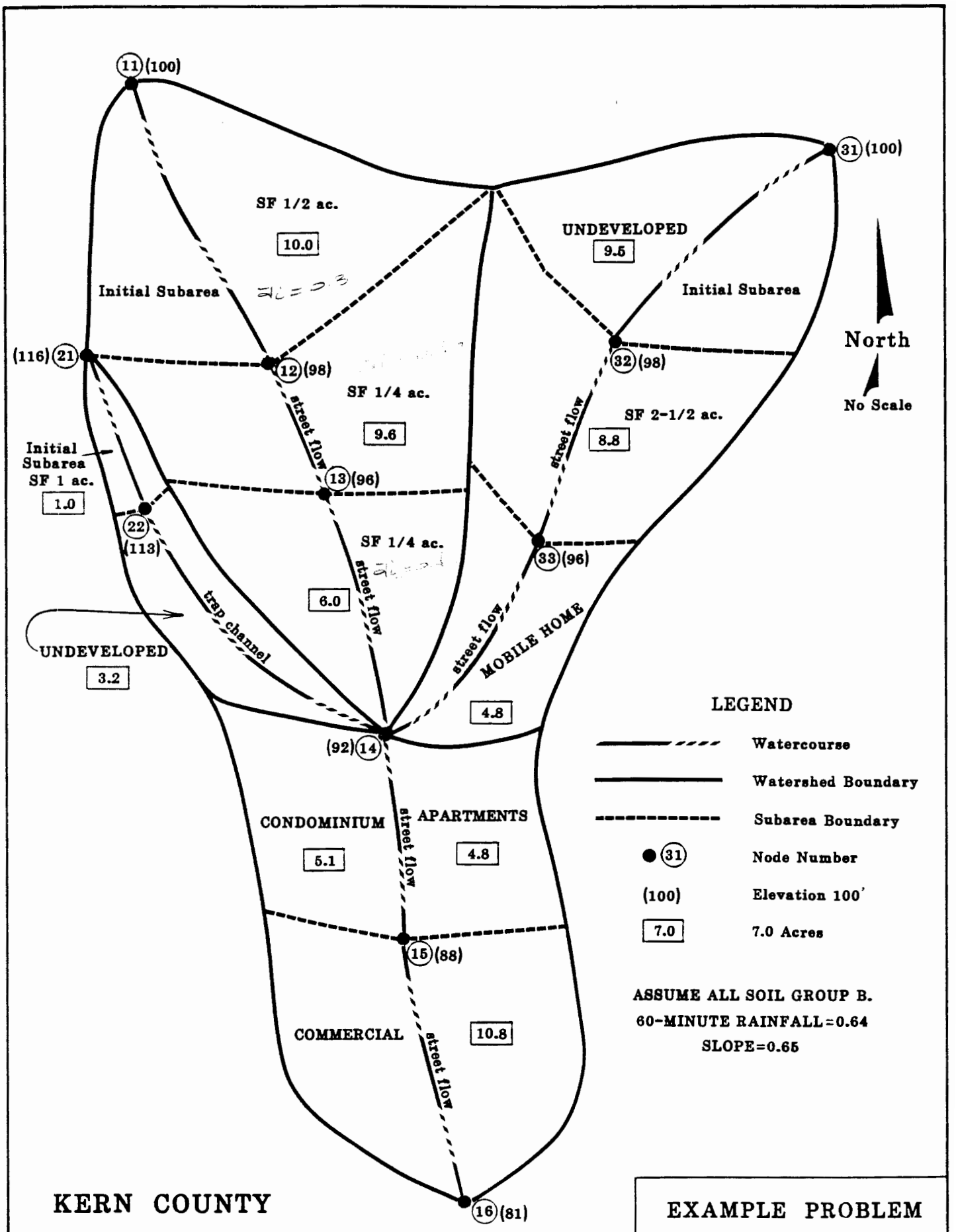
Concentration Point	Area (Acres)		Soil Type	Development Type	Percent Impervious		Pervious		T(t) minutes	Tc minutes	I in/hr	Fm in/hr	Fm average	Y in/hr	Y average	Q minimum	Q calc'd	Q Total	Flow Path Length ft.	Elevation Difference ft.	Slope ft./ft.	Velocity V ft./sec.	Hydraulics and Notes
	Subarea	Total			ai	ai avg.	CN	Fp value															

D.13. EXAMPLE: PEAK FLOW RATE DETERMINATION

The following example problem illustrates the format required for rational method hydrology studies. In the following, an example watershed is analyzed using the rational method approach. The example problem presentation contains the following information:

Description

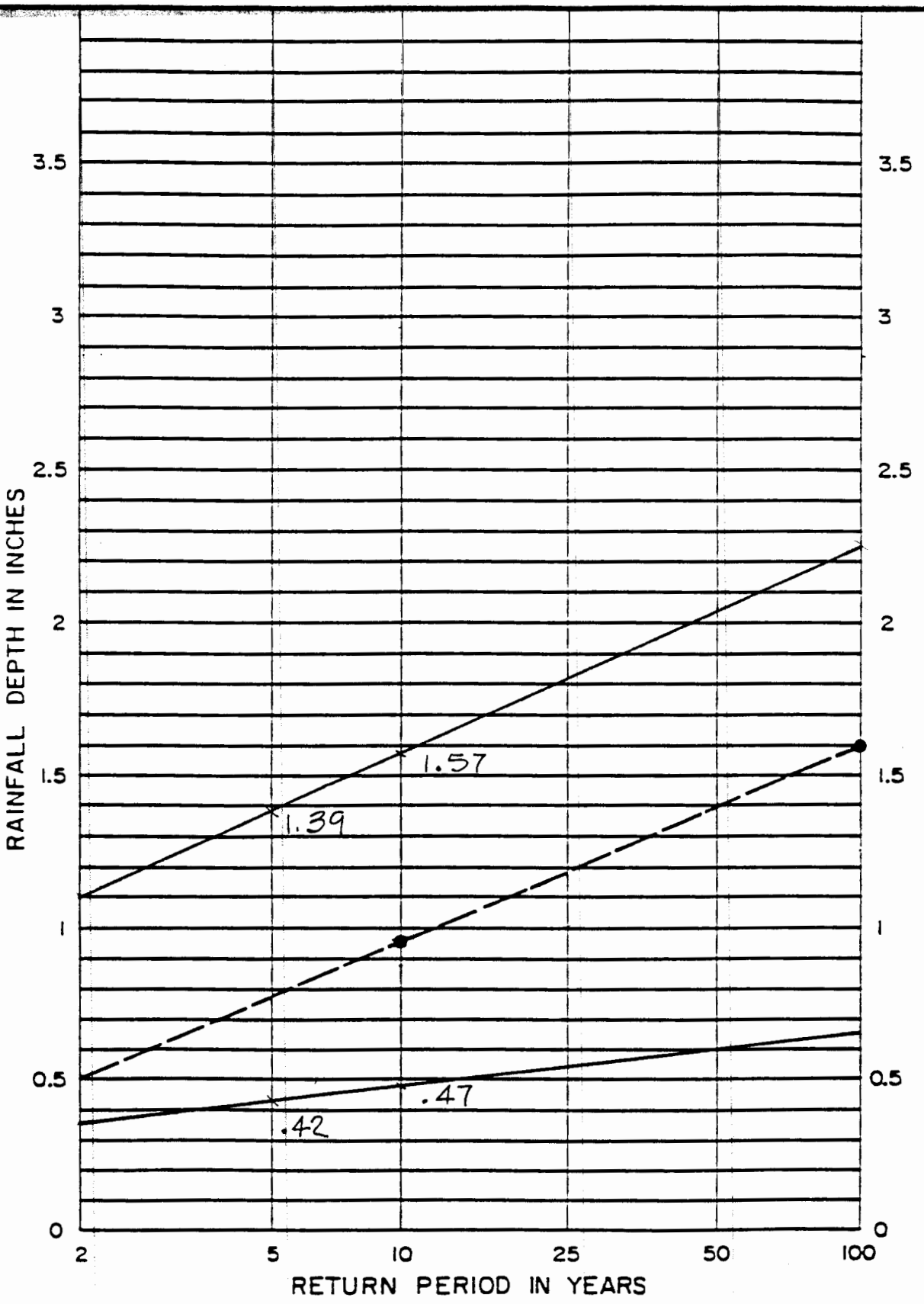
- Example Problem Drainage System Map (Figure D-9)
- Example Problem Rainfall Depth vs Return Period for Partial Duration Series (Figure D-10)
- Example Problem Intensity - Duration Precipitation Form, 5 year Return Frequency (Figure D-11a)
- Example Problem Intensity - Duration Precipitation Form, 10 year Return Frequency (Figure D-11b)
- Example Problem Intensity - Duration Precipitation Form, 100 year Return Frequency (Figure D-11c)
- Example Calculations of Subarea Yield Rates: 5 Year (Table D-3a)
- Example Calculations of Subarea Yield Rates: 10 Year (Table D-3b)
- Example Calculations of Subarea Yield Rates: 100 Year (Table D-3c)
- Example Problem Rational Method Calculation Sheets: 100 Year (Figure D-12a)
- Example Problem Rational Method Calculation Sheets: 10 Year (Figure D-12b)
- Example Problem Rational Method Calculation Sheets: 5 Year (Figure D-12c)



KERN COUNTY
Hydrology Manual

EXAMPLE PROBLEM
DRAINAGE SYSTEM

FIGURE D-9



NOTE:

1. FOR INTERMEDIATE RETURN PERIODS PLOT 10-YEAR AND 100-YEAR ONE HOUR VALUES FROM MAPS, THEN CONNECT POINTS AND READ VALUE FOR DESIRED RETURN PERIOD. FOR EXAMPLE GIVEN 10-YEAR ONE HOUR = 0.95" AND 100-YEAR ONE HOUR = 1.60", 25-YEAR ONE HOUR = 1.18".

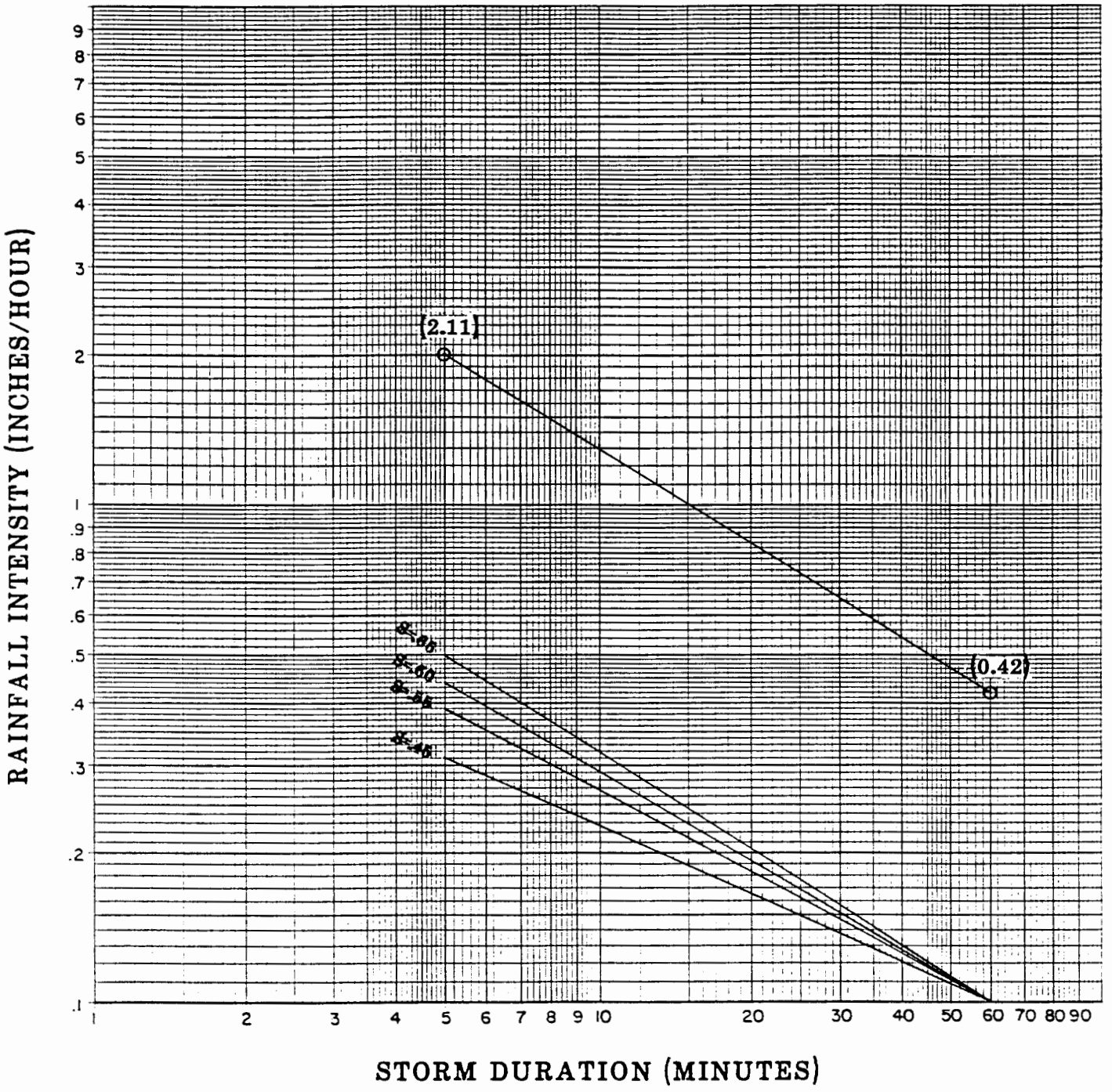
REFERENCE: NOAA ATLAS 2, VOLUME XI - CAL., 1973

KERN COUNTY

HYDROLOGY MANUAL

RAINFALL DEPTH VERSUS RETURN PERIOD FOR PARTIAL DURATION SERIES

FIGURE D-10



DESIGN STORM FREQUENCY = 5 YEARS

ONE HOUR POINT RAINFALL= .42 INCHES

LOG-LOG SLOPE= 0.65

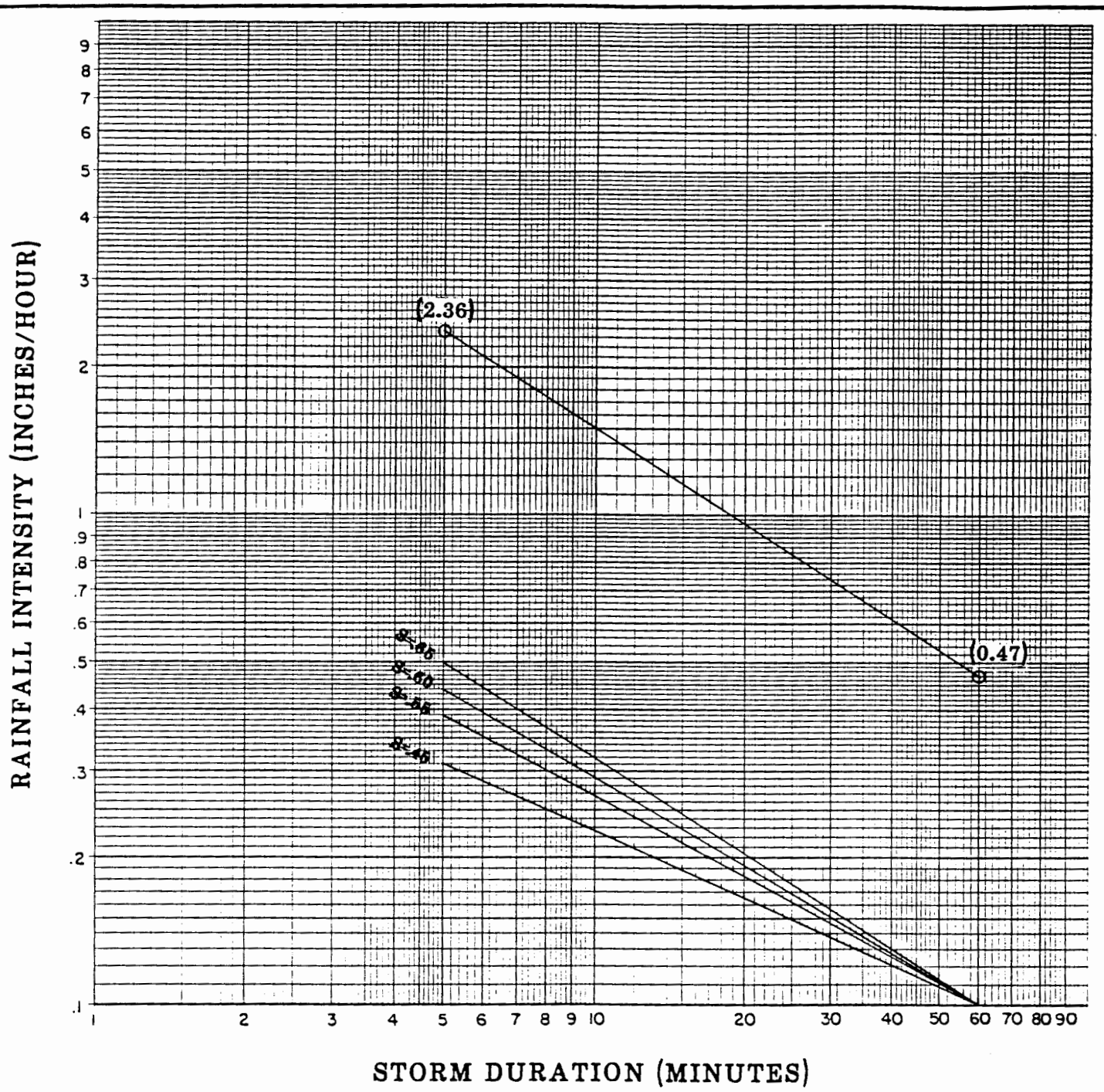
PROJECT LOCATION= EXAMPLE

KERN COUNTY
Hydrology Manual

INTENSITY-DURATION CURVES

CALCULATION SHEET

FIGURE D-11a



DESIGN STORM FREQUENCY = 10 YEARS

ONE HOUR POINT RAINFALL= .47 INCHES

LOG-LOG SLOPE= 0.65

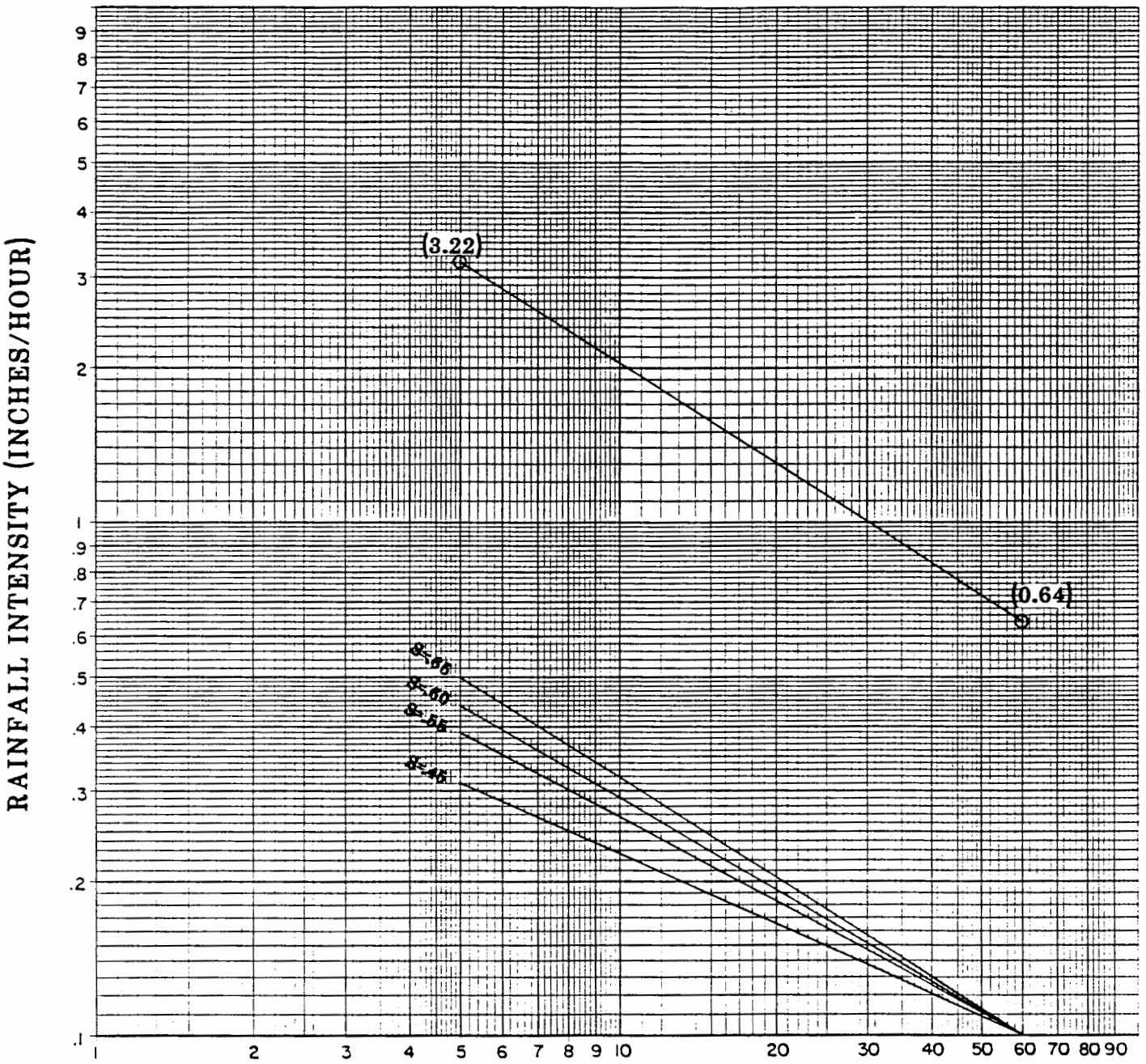
PROJECT LOCATION= EXAMPLE

KERN COUNTY
Hydrology Manual

INTENSITY-DURATION CURVES

CALCULATION SHEET

FIGURE D-11b



DESIGN STORM FREQUENCY = 100 YEARS

ONE HOUR POINT RAINFALL= .64 INCHES

LOG-LOG SLOPE= 0.65

PROJECT LOCATION= EXAMPLE

KERN COUNTY
Hydrology Manual

INTENSITY-DURATION CURVES

CALCULATION SHEET

FIGURE D-11c

TABLE D-3a

Calculation of Subarea Yield Rates: 5 yr Study Name: Example

General Equation for Yield:

$$Y = (P(24)-Ia)^2/[(P(24)-Ia+S)(P(24)]$$

P(24) = 1.39 S(imp) = 0.20

Y(imp) = 0.84 Ia(imp) = 0.04

Point of Concen-tration	Percent Imp-ai	Perv. CN	Perv. S	Perv. Ia	Perv. Y	Net Y
12	0.3	61	6.39	1.28	0	0.25
13	0.4	61	6.39	1.28	0	0.34
14(1)	0.4	61	6.39	1.28	0	0.34
15	0.72	61	6.39	1.28	0	0.61
16	0.9	61	6.39	1.28	0	0.76
22	0.2	70	4.29	0.86	0.04	0.2
14(2)	0	70	4.29	0.86	0.04	0.04
32	0	70	4.29	0.86	0.04	0.04
33	0.1	70	4.29	0.86	0.04	0.12

Note: ai(15) = (.65(5.1)+.8(4.8))/9.9 = 0.72

Fp(CN=61) = 0.68

Fp(CN=70) = 0.55

TABLE D-3b

Calculation of Subarea Yield Rates: 10 yr Study Name: Example

General Equation for Yield:

$$Y = (P(24)-Ia)^2/[(P(24)-Ia+S)(P(24)]$$

P(24) = 1.57 S(imp) = 0.20

Y(imp) = 0.86 Ia(imp) = 0.04

Point of Concen-tration	Percent Imp-ai	Perv. CN	Perv. S	Perv. Ia	Perv. Y	Net Y
12	0.3	61	6.39	1.28	0.01	0.26
13	0.4	61	6.39	1.28	0.01	0.35
14(1)	0.4	61	6.39	1.28	0.01	0.35
15	0.72	61	6.39	1.28	0.01	0.62
16	0.9	61	6.39	1.28	0.01	0.77
22	0.2	70	4.29	0.86	0.06	0.22
14(2)	0	70	4.29	0.86	0.06	0.06
32	0	70	4.29	0.86	0.06	0.06
33	0.1	70	4.29	0.86	0.06	0.14
14(3)	0.75	61	6.39	1.28	0.01	0.65

Note: ai(15) = (.65(5.1)+.8(4.8))/9.9 = 0.72

Fp(CN=61) = 0.68

Fp(CN=70) = 0.55

TABLE D-3c

Calculation of Subarea Yield Rates: 100 yr Study Name: Example

General Equation for Yield:

$$Y = (P(24)-Ia)^2/[(P(24)-Ia+S)(P(24)]$$

P(24) = 2.25 S(imp) = 0.20

Ia(imp) = 0.04 Y(imp) = 0.90

Point of Concen-tration	Percent Imp-ai	Perv. CN	Perv. S	Perv. Ia	Perv. Y	Net Y
12	0.30	61	6.39	1.28	0.06	0.31
13	0.40	61	6.39	1.28	0.06	0.39
14(1)	0.40	61	6.39	1.28	0.06	0.39
15	0.72	61	6.39	1.28	0.06	0.66
16	0.90	61	6.39	1.28	0.06	0.81
22	0.20	70	4.29	0.86	0.15	0.30
14(2)	0.00	70	4.29	0.86	0.15	0.15
32	0.00	70	4.29	0.86	0.15	0.15
33	0.10	70	4.29	0.86	0.15	0.23
14(3)	0.75	61	6.39	1.28	0.06	0.69

Note: ai(15) = (.65(5.1)+.8(4.8))/9.9 = 0.72

Fp(CN=61) = 0.68

Fp(CN=70) = 0.55

KERN COUNTY HYDROLOGY MANUAL: Rational Method Study Form

Study Name: EXAMPLE | 1-Hour Rainfall (inches)= 0.64 | Slope= 0.65

Storm Return Interval: 100 YR. | 24-Hour Rainfall (inches)= 2.25 | K(T)= 1.00

Calculated by FPM Date 1/92

Checked by CEF Date

Concentration Point	Area (Acres) Subarea	Area (Acres) Total	Soil Type	Development Type	Percent Impervious ai / ai avg.	Pervious CN / Fp value	T(t) minutes	Tc minutes	I in/hr	Fm in/hr	Fm average	Y in/hr	Y average	Q minimum	Q calc'd	Q Total	Flow Path Length ft.	Elevation Difference ft.	Slope ft./ft.	Velocity V ft./sec.	Hydraulics and Notes
JUNCTION AT 14: T_2 < T_1 THEREFORE: ADJUSTED $[Q_1] = \left[\frac{T_2+T_1}{2T_1}\right]Q_1$ = $\frac{(27.5)}{2(49.8)}$																	(5.6)	=4.3			
Q_p = 16.4 + 4.3 = 20.7																					
SINCE $T_2 < T_1$ AN AREA ADJUSTMENT IS REQUIRED																					
$I_1 - FM_1 > Y_1 I_1$; $I_2 - FM_1$							=1.06	.45 =	.61 >	$Y_1 I_2$ = .29(1.06)=	.31			THEREFORE	ADJUST $[A_1] = \left(\frac{T_2-T_1}{2T_1}\right)\left(\frac{I_1-FM_1}{I_2-FM_1}\right)A_1$						
A_p = 29.8 + $\left(\frac{27.5}{2(49.8)} + \frac{49.8}{1}\right)$							(.72-1.06)	(.45-.45)	23.1	= 29.8 +	7.9 =	37.7		ACRES							
FM_p = (29.8 (.45) + 7.9 (.45))							/37.7 =	0.45	; Y_p = (29.8 (.34) +	7.9 (	.29))	/37.7 =	.33				; I_p =	1.06	; T_p =	27.5	
14	-	37.7	-	-	-	-		27.5	1.06	-	.45	-	.33	-	-	20.7					
15	9.9	47.6	B	CONDO APART.	72 / -	61 / .68	2.9	30.4	1.00	.19	.39	.66	.40	17.1	26.1	26.1	550	4	.0073	3.2	LOCAL STREET FIG. D-26
16	10.8	58.4	B	COMM.	90 / -	61 / .68	3.2	33.6	.94	.07	.33	.69	.45	22.2	32.1	32.1	700	7	.01	3.7	LOCAL STREET FIG. D-26

FIGURE D-12a (2 of 2)

KERN COUNTY HYDROLOGY MANUAL: Rational Method Study Form

Study Name: EXAMPLE 1-Hour Rainfall (inches)= 0.64 Slope= 0.65

Storm Return Interval: 100 YR. 24-Hour Rainfall (inches)= 2.25 K(T)= 1.00

Calculated by FPM Date 1/92

Checked by CEF Date

Concentration Point	Area (Acres) Subarea	Area (Acres) Total	Soil Type	Development Type	Percent Impervious ai / ai avg.	Pervious CN / Fp value	T(t) minutes	Tc minutes	I in/hr	Fm in/hr	Fm average	Y in/hr	Y average	Q minimum	Q calc'd	Q Total	Flow Path Length ft.	Elevation Difference ft.	Slope ft./ft.	Velocity V ft./sec.	Hydraulics and Notes
JUNCTION AT 14: $T_2 < T_1$ THEREFORE: ADJUSTED $[Q_1] = \left[\frac{T_2 + T_1}{2T_1}\right] Q_1 = \frac{(27.5)}{2(49.8)}$																(49.8)	(5.6)	=4.3			
$Q_p = 16.4 + 4.3 = 20.7$																					
SINCE $T_2 < T_1$ AN AREA ADJUSTMENT IS REQUIRED																					
$I_1 - FM_1 > Y_1 I_1$; $I_2 - FM_1 = 1.06$								.45 =	.61 >	$Y_1 I_2 = .29(1.06) = .31$				THEREFORE	ADJUST $[A_1] = \left(\frac{T_2 - T_1}{2T_1}\right)\left(\frac{I_1 - FM_1}{I_2 - FM_1}\right) A_1$						
$Ap = 29.8 + \left(\frac{27.5 + 49.8}{2(49.8)}\right)$							(.72-.45) (1.06-.45)	23.1	= 29.8 +	7.9 =	37.7			ACRES							
$FMp = (29.8(.45) + 7.9(.45))/37.7 =$								0.45	; Yp = (29.8(.34) +	7.9(.29))/37.7 =		.33					; Ip =	1.06	; Tp =	27.5	
14	-	37.7	-	-	-	-		27.5	1.06	-	.45	-	.33	-	-	20.7					
15	9.9	47.6	B	CONDO APART.	72 / -	61 / .68	2.9	30.4	1.00	.19	.39	.66	.40	17.1	26.1	26.1	550	4	.0073	3.2	LOCAL STREET FIG. D-26
16	10.8	58.4	B	COMM.	90 / -	61 / .68	3.2	33.6	.94	.07	.33	.69	.45	22.2	32.1	32.1	700	7	.01	3.7	LOCAL STREET FIG. D-26

FIGURE D-12a (2 of 2)

KERN COUNTY HYDROLOGY MANUAL: Rational Method Study Form

Study Name: EXAMPLE 1-Hour Rainfall (inches)= 0.47 Slope= 0.65

Storm Return Interval: 10 YR. 24-Hour Rainfall (inches)= 1.57 K(T)= 1.14

Calculated by FPM Date 1/92

Checked by CEF Date

Page 1 of 1

Concentration Point	Area (Acres) Subarea	Area (Acres) Total	Soil Type	Development Type	Percent Impervious ai / ai avg.	Pervious CN / Fp value	T(t) minutes	Tc minutes	I in/hr	Fm in/hr	Fm average	Y in/hr	Y average	Q minimum	Q calc'd	Q Total	Flow Path Length ft.	Elevation Difference ft.	Slope ft./ft.	Velocity V ft./sec.	Hydraulics and Notes
16		58.4						38.3	.63		.33		.42	13.9	15.8	15.8					
	T_c = 33.6 x	1.14 = 38.3																			
			PT. 12	PT. 13			PT. 14(1)	PT. 22		PT. 14(2)			PT. 32			PT. 32		PT. 14(3)			PT. 15 PT. 16
	Y =	.26	(10) + .35	(9.6)	+ .35		(6.0)	+ .22	(1.0)	+ .06	(3.2)	+ .06	(9.5)	(7.9/23.1)	+ .14	(8.8)(7.9/23.1)	+ .65	(4.8)(7.9/23.1)	+ .62	(9.9)	+ .77 (10.8)
												58.4									
	Y = .42																				

FIGURE D-12b

KERN COUNTY HYDROLOGY MANUAL: Rational Method Study Form

Study Name: EXAMPLE

Storm Return Interval: 5YR.

1-Hour Rainfall (inches) = 0.42 Slope = 0.65

24-Hour Rainfall (inches) = 1.29 K(T) = 1.20

Calculated by FPM Date 1/92

Checked by CEF Date

Page 1 of 1

Concentration Point	Area (Acres) Subarea	Area (Acres) Total	Soil Type	Development Type	Percent Impervious ai / ai avg.	Pervious CN / Fp value	T(t) minutes	Tc minutes	I in/hr	Fm in/hr	Fm average	Y in/hr	Y average	Q minimum	Q calc'd	Q Total	Flow Path Length ft.	Elevation Difference ft.	Slope ft./ft.	Velocity V ft./sec.	Hydraulics and Notes
33		18.3			5			51.2	.47		.52		.08	0.6	.4	0.6					
	Y = .04 (9.5) + .12 (8.8) =					.08															
		18.3																			
	TC = 42.7 X 1.2 =	51.2																			
	QCALC'D = .9 I ai A (SINCE I = FM X 0)																				
	= .9 (.47) (.05) 18.3																				
	= 0.39 USE 0.4																				

FIGURE D-12c

Section D
List of Equations and Variables used in the Hydrology Manual

D.1	Q	=	CIA
D.2	Tc(T)	=	$Tc(100)\cdot K^{T}$
D.3	$V = \frac{1.49}{n}\cdot R^{2/3}\cdot S^{1/2}$		
D.4	C =		$0.90(a_i + ap(I - F_p)/I)$, for I greater than F_p $0.90a_i$, for I less than or equal to F_p
D.5	Q	=	$0.90\ (I - F_m)A$
D.6	Q(min)	=	$0.90\cdot YIA$
D.7	$Q_p = Q_2 + Adjusted[Q_1]$		
D.8	Adjusted[Q]= Q1*Minimum of		Maximum of $[(I_2 - Fm_1),\ (Y_1I_2)]$ (T_2+T_1) ----------, ---------- Maximum of $[(I_1 - Fm_1),\ (Y_1I_1)]$ $2T_1$
D.8a	$Adjusted[Q_1] = \frac{I_2 - Fm_1}{I_1 - Fm_1}\cdot Q_1$		
D.8b	$Adjusted[Q_1] = \frac{Y_1\cdot I_2}{I_1 - Fm_1}\cdot Q_1$		
D.8c	$Adjusted[Q_1] = \frac{I_2}{I_1}\cdot Q_1$		
D.8d	$Adjusted[Q_1] = \frac{T_2 + T_1}{2\cdot T_1}\cdot Q_1$		
D.9	A_p	=	A_2 + Adjusted[A_1]
D.9a	$Adjusted[A_1] = \frac{(T_2 + T_1)(I_1 - F_{m1})}{2T_1(I_2 - F_{m1})}\cdot A_1$		
D.9b	$Adjusted[A_1] = \frac{(T_2 + T_1)\cdot(Y_1\cdot I_1)}{2T_1\cdot(I_2 - F_{m1})}\cdot A_1$		
D.9c	$Adjusted[A_1] = \frac{(T_2 + T_1)\cdot I_1}{2T_1\cdot I_2}\cdot A_1$		

INDEX SHEET

for

TOPOGRAPHIC MAPS

ERN COUNTY, CALIFORNIA

1 — 30 Min. QUADS

1-2 — 15 Min. QUADS

1-2-A — 7 1/2 Min. QUADS

ap revised to date: March 1972

NOTE

This map intended primarily for delineating the boundary lines of districts, may include some inconsistencies, particularly in the eastern portion due to irregularities in Government Surveys which have not yet been remedied, but which do not interfere with its purpose.

Prepared By:

OFFICE of the COUNTY SURVEYOR

SANTA BARBARA COUNTY

VENTURA COUNTY

LOS ANGELES

SECTION E

THE UNIT HYDROGRAPH METHOD FOR

CATCHMENT RUNOFF HYDROGRAPHS

OVERVIEW

This section will cover the use and application of the Unit Hydrograph Method for hydrology studies. It will discuss:

- Determination of synthetic distribution graphs
- Calculation of lag
- Development of synthetic unit hydrographs
- Design storm patterns
- Runoff volume estimation
- Required study format

Section B should be reviewed for precipitation calculations. Section C should be reviewed for loss determination. Section D provides the methodology for determining the time of concentration. Section F, G, H, and I provide hydrograph routing criterion.

For other important information pertinent to this section refer to sections B.6. through B.10., Section C.7., Section D.10., and figures C-3, C-4, C-5, and D-3.

E.1. DEFINITIONS

Distribution graph: Is a unit hydrograph whose ordinates are expressed in terms of percent of ultimate discharge. A distribution graph is generally developed as a block graph with each block representing its associated percent of unit runoff which occurs during the specified unit time. The unit time used in the distribution graph is identical to the unit time specified for the unit hydrograph.

Effective rainfall: Is that part of rainfall that runs off in a relatively brief time period. (Here, the brief time period is selected sufficiently small such that the significant hydrologic effects are adequately represented by the time period's average values.) Effective rainfall is the total rainfall less infiltration, evaporation, transpiration, absorption, and detention.

Lag: for a watershed is the time (hours) from the beginning of a continuous series of unit period effective rainfalls over the watershed area (tributary to a point of concentration) to the instant when the rate of resulting tributary watershed runoff (at the point of concentration) equals 50

percent of the ultimate rate of the resulting runoff.

S-Graph: Is a summation hydrograph developed by plotting watershed discharge expressed in percent of ultimate discharge as a function of time expressed in percent of lag.

Summation hydrograph: for a point of concentration on a given stream is a curve (hydrograph) showing the time distribution of the rates of runoff that would result from a continuous series of unit period effective rainfalls over the tributary watershed upstream of the subject point of concentration. The ordinates of the summation hydrograph are expressed in percent of the ultimate discharge.

Ultimate discharge: Is the maximum rate of watershed runoff which can result from a specified effective rainfall intensity. Ultimate discharge from a watershed occurs when the rate of runoff on the summation hydrograph is equivalent to the rate of effective rainfall. For an effective rainfall rate of one inch occurring in a unit period of one hour, the ultimate discharge is 645 cfs for every square mile of watershed. Ultimate discharge for different unit periods is given by dividing 645 by the unit period in hours, and multiplying by the watershed area in square miles.

Unit hydrograph: (or unit graph) for a point of concentration on a watershed (catchment) stream is a curve (hydrograph) showing the time distribution of rates of runoff which results from one inch of effective rainfall during a unit period of time over the tributary watershed upstream of the point of concentration. The unit effective rainfall is generally assumed to occur as an equivalent constant rainfall intensity during a specified unit period of time (such as 5, 10, 15, or 30 minutes). Figure E-1 illustrates the general formulation of the unit hydrograph.

E.2. BACKGROUND

The ***unit hydrograph*** **(UH)** method assumes that watershed discharge is related to the total volume of runoff, and that the time factors which affect the unit hydrograph shape are invariant, and that watershed storm rainfall-runoff relationships are characterized by watershed area, slope, and shape factors. The UH method is used to estimate the time distribution of watershed runoff in drainage basins where stream gauge information is either unavailable or inadequate to justify statistical interpretation (refs. 4-10). The unit hydrograph method for determining the time distribution of runoff is used for hydrology studies of all Kern County watersheds in excess of approximately one square mile in area. For watersheds less than one square mile in area, the small area runoff hydrograph development procedures described in Section J, should be used.

E.3. DETERMINATION OF SYNTHETIC DISTRIBUTION GRAPHS

Adequate storm rainfall and watershed runoff information are available for the determination of ***distribution graphs*** for several streams in Southern California. The distribution graphs for each of the gauged streams can be determined by trial-and-error attempts to duplicate the runoff hydrographs produced by major storm events (i.e., reconstitution studies). The derived distribution graphs are then verified by using them to reproduce runoff hydrographs from other major storm events.

The method of determining synthetic distribution graphs is used to estimate the time distribution of watershed runoff in drainage basins where stream gauge data is inadequate. The procedure develops a time distribution of runoff based on the properties of distribution graphs from several gauged watersheds (refs. 4-10).

It is assumed that the drainage areas within a given region are physiographically and hydrologically similar. Because no two drainage areas have identical hydrologic characteristics, the runoff patterns from these areas are generally dissimilar and the distribution graphs of these areas may differ considerably. Therefore, direct transposition of distribution graphs from one watershed to another is usually precluded. However, most distribution graphs exhibit certain similarities which the introduction of a factor called "lag" will bring the arrangement of ordinates along the bases of distribution graphs into a generally consistent relationship. ***Lag***, which was initially defined in the literature as the time difference in phase between salient features of the rainfall and runoff rate curves, is an empirical expression of the hydrologic characteristics of a watershed in terms of time. Details of the determination of lag for watersheds where the time distributions of runoff are known and of the use of lag in developing synthetic distribution graphs are discussed in the following:

1. Summation Hydrograph - The first step in determining lag for a watershed is the determination of the summation hydrograph, which is the hydrograph of runoff that would result from the continuous generation of unit effective rainfall over the watershed. The ordinates of summation hydrographs are expressed in percent of ultimate discharge and a summation hydrograph for a point of concentration is determined by adding a continuous series of identical distribution graphs out of phase one unit period. On such a hydrograph, the time required to reach maximum (ultimate) discharge is equal to the length of the base of one distribution graph less one unit period.

2. Lag - Lag for a watershed can be defined as the elapsed time (in hours) from the beginning of unit effective rainfall to the instant that the summation hydrograph for the point of concentration reaches 50 percent of ultimate discharge. When the lags determined from summation hydrographs for several gauged watersheds are correlated to the hydrologic characteristics of the watersheds, an empirical relationship is usually apparent. This relationship can then be used to determine the lags for comparable drainage areas for which the hydrologic characteristics can be determined, but for which the distribution graphs for concentration points cannot

be determined because of inadequate hydrologic data. By comparing lag values (obtained from the analysis of rainfall-runoff data) to catchment time of concentration Tc values estimated from a detailed rational method analysis (Section D.9), a relationship is readily determined,

$$lag = 0.8Tc \qquad (E.1)$$

It is noted that the rational method time of concentration, used for the estimation of basin lag time, is a critical parameter in the unit hydrograph method. Extreme care must be taken in the evaluation of the catchment Tc in order to reduce uncertainty, and enable "reproducibility" of this parameter. Section D.9 provides the procedure for estimating Tc for use in unit hydrograph studies using the rational method.

(E.1) is used in all unit hydrograph studies where sufficient topographic information is available to compute the time of concentration, Tc. It is noted that due to Tc being the sum of the rational method's initial subarea Tc and the subsequent downstream reach hydraulic travel times, Tc values will vary depending on the return frequency of rainfall used in the analysis. That is, a 2-year storm estimated Tc value typically is longer in duration than a 100-year storm estimated Tc value. Consequently, when computing the lag corresponding to a T-year design storm event, the Tc is estimated using the 100-year intensity-duration rainfalls and modified by the K_T factors shown in Table D-1. For certain large scale natural condition catchment studies, the County may consider the use of the lag relationship given by the empirical formula:

$$\text{lag (hours)} = C_t\,((L \cdot L_{ca})/S^{0.5})^m \qquad (E.2)$$

where

C_t = a constant (determined by regional flood reconstitution studies)

L = length of longest watercourse (miles)

L_{ca} = length along longest watercourse, measured upstream to a point opposite center of area (miles)

S = overall slope of drainage area between the headwaters and the collection point (feet per mile)

m = a constant determined by regional flood reconstitution studies

It is then assumed that there exists a relationship between watershed lag and the quotient $((L \cdot L_{ca})/S^{0.5}))^m$. This relationship is given by the above empirical formula for lag when

C_t = 24$\bar{n}$; ($\bar{n}$ is the visually estimated basin factor of all collection streams and watershed channels, see Figure E-2)

m = 0.38

3. S-graph - After lag factors are determined for several gauged watersheds, the next step in determining synthetic distribution graphs is the development of S-graphs, which are summation hydrographs modified so that the percent of ultimate discharge is related to time expressed in percent of lag. The derivation of an S-graph is identical to the derivation of a summation hydrograph, except that the factor of lag has been introduced. Time in percent of lag has been used to determine S-graphs for five major groupings of watersheds.

   Five S-graphs are used for unit hydrograph development in Kern County. These S-graphs are entitled Valley:Developed, Valley:Undeveloped, Foothill, Mountain, and Desert (Figures E-3a, b, c, d, and e). In conformity with the definition of lag, each S-graph reaches 50 percent of ultimate discharge at 100 percent of lag. The average of the several S-graphs determined for mountain watersheds is assumed to be applicable to the mountain drainage basins with unknown runoff characteristics. Similarly the average of the S-graphs determined for valley watersheds is assumed to be applicable to the valley drainage basins, and so forth.

   - The Foothill S-graph is only for watersheds characterized by natural channels that are sharply incised in canyon bottoms; i.e., overbank flows are confined near the defined channel.
   - The Mountain S-graph is only for watersheds characterized by natural channels with numerous plunging flow reaches and lodged boulders/debris.
   - The Valley:Undeveloped S-graph is for natural watersheds whose channels are not sharply incised, i.e., where overbank flows may spread widely from the defined channel.
   - The Valley:Developed S-graph is for all watershed conditions where prismatic channels exist or are to be provided for conveyance of T-year peak flows. This S-graph is representative of fully urbanized catchments where the ultimately planned, free-draining (no detention facilities) storm drains and major flood control channels are operative.
   - The Valley:Undeveloped S-graph may be more appropriate than the Valley:Developed S-graph, should the catchment be fully developed but the storm drain system is generally deficient, with significant ponding and storage effects occurring interior of the catchment.
   - The Desert S-graph is for watersheds within the desert regions of the county.

E.3.1. Application of Lag and S-graphs

Using the rational method (Section D), the watershed time of concentration (Section D.10) is computed and lag is determined using (E.1). A unit time is selected (generally 15 to 25 percent of the lag) and accumulated unit time periods are expressed as accumulated percentages of the watershed lag. These percentages of lag are used in superimposing a "block" graph on the appropriate average ***S-graph*** for the watershed and the resulting pattern is used in determining the accumulated mean percentage of ***ultimate discharge*** for each accumulated unit time (see example problem). Because these accumulated mean percentages represent the accumulated mean percentages for the synthetic distribution graph for the watershed, the mean percentage for successive unit periods are determined by a series of subtractions.

E.4. DEVELOPMENT OF THE SYNTHETIC UNIT HYDROGRAPH

For watersheds where stream gauge data is inadequate, it may generally be assumed that a synthetic unit hydrograph adequately approximates the time distribution of runoff at the subject watershed point of concentration. From the above discussion, a method to develop a synthetic unit hydrograph is described in the following steps:

1. Estimate the watershed lag using topographic information and a rational method Tc calculation based on the appropriate T-year rainfall (see Section D.10).

2. Select a unit period to be used for the hydrograph analysis. This unit period will be used for development of design critical storm unit rainfalls and the runoff hydrograph. The unit period is generally chosen to be within 15 and 25 percent of the watershed lag in order to provide sufficient definition of the unit hydrograph.

3. Choose an S-graph which is appropriate for the catchment being studied.

4. The appropriate watershed S-graph can be approximated by a block graph where the base of each block is the selected unit period percentage of lag (Step 2) and the ordinate of each block is the time-averaged percentage of ultimate discharge (from the S-graph) for that unit period. The area of each block equals the area under the S-graph for each respective unit period. Consequently, at the end of each unit period, the total area under the S-graph equals the sum of the areas of the equivalent unit period blocks.

5. The unit distribution block graph is developed by computing the difference between the ordinates (percentage of ultimate discharge) assigned to the unit period blocks used to approximate the S-graph of Step 4. This is equivalent to computing the difference between the ordinates of two S-graphs that have been offset by one unit period.

6. The final step to develop the synthetic unit hydrograph (or unit graph) is to multiply the ordinates of the distribution block graph (Step 5) by the factor K, the ultimate discharge. The ultimate discharge is defined by

$$K\ (cfs) = 645\ A/T \qquad (E.3)$$

where

A = drainage area (square miles)

T = unit time period (hours)

E.5. DESIGN STORM POINT PRECIPITATION

The County's prescribed level of flood protection is obtained by using T-year rainfalls for the development of the T-year runoff hydrograph. Based on the NOAA Atlas 2, the County has prepared rainfall isohyetal maps to be used to analyze point precipitation patterns within a watershed (see Section B.6).

It may generally be assumed that the 3-hour point precipitation value within a watershed is related to the 1-hour and 6-hour point precipitation by straight line interpolation on log-log paper.

The shorter duration (less than 3 hours) rainfall intensity-duration data, used in rational method studies, plot as a straight line on standard log-log paper. Using the appropriate 1-hour point precipitation value, an estimate of the peak 5- and 30-minute point rainfalls can be computed using the procedures outlined in Section D (see Figure D-3).

The peak 5-minute, 30-minute, 1-hour, 6-hour, and 24-hour area-averaged point precipitation values are required in order to develop the design storm pattern to be used to compute the watershed runoff hydrograph for the subject rainfall return frequency. Unit-hydrograph studies to be submitted for review should contain the following point precipitation information:

1. Two separate maps showing the watershed 6-hour and 24-hour isohyetals for both the 2-year and 100-year storm return intervals.

2. Calculations of the area-averaged point precipitation value within the watershed for the peak 6-hour and 24-hour durations for both the 2-year and 100-year storm return intervals.

3. Calculations using the appropriate equations in Section B.9, estimating the 1-hour area-averaged point precipitation value for both the 2-year and 100-year storm return intervals.

4. Calculations using Figure D-3, estimating the 1-hour, 6-hour, and 24-hour area-averaged point precipitation values for the appropriate storm return frequency.

5. Calculations using standard log-log paper (Figure E-8), plotting the area-averaged point precipitation values for the peak 1 hour and 6 hours of rainfall. Assuming a

straight line relationship, the area-averaged 3-hour point precipitation value for the watershed should be estimated.

6. Calculations using the log-log paper (Figure D-4), estimating the area-averaged 5-minute and 30-minute point precipitation values for the watershed.

E.5.1. Point Precipitation Adjustment for Watershed Area

In the unit hydrograph approach, the volume of watershed runoff is related to the watershed average depth of precipitation rather than the depth at specific points within the watershed. Depth-area curves approximately relate the average of all point precipitation values for a specific duration and frequency to the average depth of precipitation within the subject watershed for the same duration and frequency (Figure E-4).

E.6. DESIGN STORM PATTERN

The design storm pattern is based upon a single synthetic 24-hour critical storm pattern which includes the peak rainfall intensities estimated for the 5-minute, 30-minute, 1-hour, 3-hour, 6-hour, and 24-hour durations. The storm pattern is developed from the watershed area-averaged point precipitation values, and modified incrementally according to the depth-area curves of Figure E-4. The assignment of peak rainfall values within the synthetic critical storm pattern is shown in Figure E-5 (a, b, c) (refs.2, 4, 10).

For large watersheds (e.g., 5 square miles or larger) or for detention basin studies, the entire 24-hour synthetic storm pattern may be required for hydrologic study purposes. For small watersheds (less than 5 square miles) where only peak runoff rates are required, the peak 3 hours of the 24-hour synthetic storm pattern generally can be used for study purposes, ignoring the remaining 21 hours of lower intensity rainfall.

E.7. DESIGN STORM LOSS RATES

Where sufficient stream gauge information is of adequate quantity and quality as determined by the County, loss rates for unit hydrograph hydrology may be estimated from a study of rainfall-runoff relationships of major storms. Where such data is not available, loss rates for pervious areas shall be estimated using the methods of Section C.7.

E.7.1. Maximum Loss Rate, F_m

The maximum loss rate, F_m, for a catchment is computed by

$$F_m = a_p F_p \qquad \text{(E.4)}$$

where

F_m = maximum loss rate (inches/hour)

a_p = pervious area fraction (see Figure C-3)

F_p = maximum loss rate for pervious areas (inches/hour); (see Section C.7.5)

Maximum loss rates, F_m, for runoff hydrograph studies are usually within the range of 0.10 to 0.40 inches per hour in urbanized areas.

E.7.2. Low Loss Rate, F*

During the peak rainfall intensities of the synthetic design storm pattern, the loss rate used to estimate effective rainfall is typically the maximum loss rate, F_m. At lower rainfall intensities, however, a low loss rate, F*, is used for the estimation of effective rainfall. The low loss rate F* is based upon the low loss fraction (Y, defined by (see Section C.7.3)

$$\bar{Y} = 1\text{-}Y \qquad \text{(E.5)}$$

where

$\bar{Y}$ = catchment low loss fraction

Y = catchment 24-hour storm runoff yield (fraction) computed from (C.4)

The corresponding low loss rate based on the $\bar{Y}$ value is

$$F^* = \bar{Y} \cdot I \qquad \text{(E.6)}$$

where

F* = low loss rate (inches/hour)

$\bar{Y}$ = low loss fraction

I = rainfall intensity (inches/hour)

Using both the low loss rate (F*) and the maximum loss rate (Fm) in the determination of effective rainfall will result in 24-hour storm watershed yields (ratio of total runoff to total rainfall) that approximate the values predicted by the curve number approach (see Figures C-4 and C-6).

The low loss fraction should be used to estimate effective rainfall whenever the maximum loss rate, Fm, exceeds F*. In all cases, however, the maximum loss rate is the constant value, Fm. Use of these two loss rate parameters enables the 24-hour design storm runoff hydrograph model to develop peak runoff rates based upon a maximum watershed loss rate (phi index), and also develop 24-hour storm runoff yields which approximate the values obtained from the SCS curve number approach.

E.8. RUNOFF VOLUME ESTIMATION

The catchment yield fraction, Y, can be used to approximate the total runoff volume produced from a 24-hour duration precipitation event without developing the 24-hour storm runoff hydrograph. The basis of this determination is the curve number approach which provides a rainfall-runoff relationship (Figure C-5). Section E.14. Example: Runoff Volume Estimate describes the application of yield fraction to determine the runoff volume.

E.9. BASEFLOWS

Baseflow is usually a minor factor in developing flood hydrographs for relatively rare flood events in Kern County. The primary source for base flow is seepage from stream bank storage derived from prior precipitation events. Generally, during the 100 year event, 10 cfs per watershed square mile is adequate for unlined channels that intercept mountainous regions where many geologic strata are crossed by the stream bed. Baseflow for events with lesser recurrence intervals should be determined under consultation with the County. Baseflow can be included in the watershed runoff by adding to the ordinates for the computed runoff hydrograph. In fully urbanized areas, baseflow can be entirely neglected.

E.10. ALTERNATIVE RUNOFF VOLUME HYDROGRAPH METHODS

The Kern County Hydrology Manual is based on calibration studies performed for San Bernardino and Orange Counties, and adjusted to local watershed conditions, in order that the unit hydrograph hydrologic methods achieve the desired level of protection in estimating the return frequency of flood flows. The introduction of additional parameters, in particular the routing of subarea flows, may alter the calibration formulation, resulting in a failure to achieve the flood control protection level objectives. Because model sensitivity analysis and model calibration is essentially precluded for studies involving ungauged watersheds, any hydrologic study not prepared in accordance with this hydrology manual may be rejected (see Section K).

E.11. REQUIRED FORMAT

Figure E-6 illustrates the required format for submitting unit hydrograph study results for review.

Figure E-7 is to be used to supply the necessary hydrology information to determine the runoff hydrograph. Figure E-8 is used to plot both the unadjusted and adjusted mass rainfall curves. Figure E-9 is the Flood Computation Form.

E.12. INSTRUCTIONS FOR SYNTHETIC UNIT HYDROGRAPH METHOD HYDROLOGY CALCULATIONS

I. Synthetic Unit Hydrograph Development

A. On a USGS topographic quadrangle sheet or other topographic map of suitable scale, outline the watershed boundary.

B. Calculate the catchment time of concentration (Tc) by using a rational method analysis for the T-year storm (see Section D.9).

1. Catchment lag is computed by

$$lag = 0.8Tc$$

2. For certain large scale natural condition catchment studies, the agency may use the lag relationship given by

$$lag\ (hours) = 24n(L \cdot L_{ca}/S^{0.50})^{0.38}$$

where

A = drainage area (square miles)

L = length of longest watercourse (miles)

L_{ca} = length along the longest watercourse, measured upstream to a point opposite the centroid of the area (miles)

H = difference in elevation between the concentration point and the most remote point of the basin (feet)

S = overall slope of longest watercourse between headwaters and concentration point (S = H/L, feet per mile)

n = visually estimated average basin factor from Figure E-2

C. Select a unit time period. To adequately define the unit hydrograph, the unit time period should be about 20 percent of lag time, and never more than 25 percent of lag time. If possible, use the unit time of the synthetic critical storm pattern of 5 minutes.

D. Select the S-graph applicable to the drainage basin (Figures E-3a, b, c, d, e). Determine the average percentage of the ultimate discharge for each unit period. In reading the percentage of ultimate discharge from the S-graph, the <u>average</u> ordinate over the time increment should be determined rather than the mean of the ordinates at the beginning and end of the time increment (see example problem).

E. Compute the unit distribution graph by subtracting from the percentage of ultimate discharge for each unit time period, the percentage of ultimate discharge for the previous time period.

F. Compute the ordinates of the synthetic unit hydrograph (unit graph) by multiplying the distribution graph values by the ultimate discharge K, using:

$$K\ (cfs) = 645\ A/T$$

where

A = drainage area (square miles)

T = unit time period (hours)

II. T-Year Design Storm Pattern Development

A. Using the appropriate T-year point precipitation values from the rainfall isohyetal maps and figures D-3 and E-8, compute the area-averaged precipitation values for the 5-minute, 30-minute, 1-hour, 3-hour, 6-hour, and 24-hour durations.

B. Adjust all point precipitation values for areal effect by using Figure E-4.

C. Develop a synthetic critical storm peak rainfall mass plot using Figure E-8 (see example problem for demonstration).

D. Using the unit interval duration for the unit hydrograph development, calculate the synthetic storm unit interval rainfall quantities by successive subtraction of mass peak rainfall values, each offset in time by one unit period.

E. Arrange the unit rainfall quantities determined in Step D into the critical storm pattern shown in Figures E-5a, b, c. For most hydrology studies, only the peak 3 hours of the synthetic critical storm may need consideration.

III. Runoff Hydrograph Development

A. Find the pervious area loss rates for subareas within the drainage area using Figures C-2 and C-3. Adjust these rates to account for impervious area using the relationship below, and then compute an area-averaged maximum loss rate for the catchment.

$$F_m = a_p F_p$$

where

F_m = maximum loss rate (inches/hour)

a_p = pervious area fraction (decimal percent of total area) (see Figure C-3)

Fp = maximum loss rate for pervious areas fraction (see Section C.7.4)

B. Compute the low loss fraction, ^{-}Y. Use F* in each unit time period where the maximum loss rate F_m exceeds the low loss rate F*, ($F^*=^{-}Y \cdot I$, see Section C.7.3).

C. Compute the unit effective rainfall for each unit time period by subtracting the unit loss from the unit rainfall.

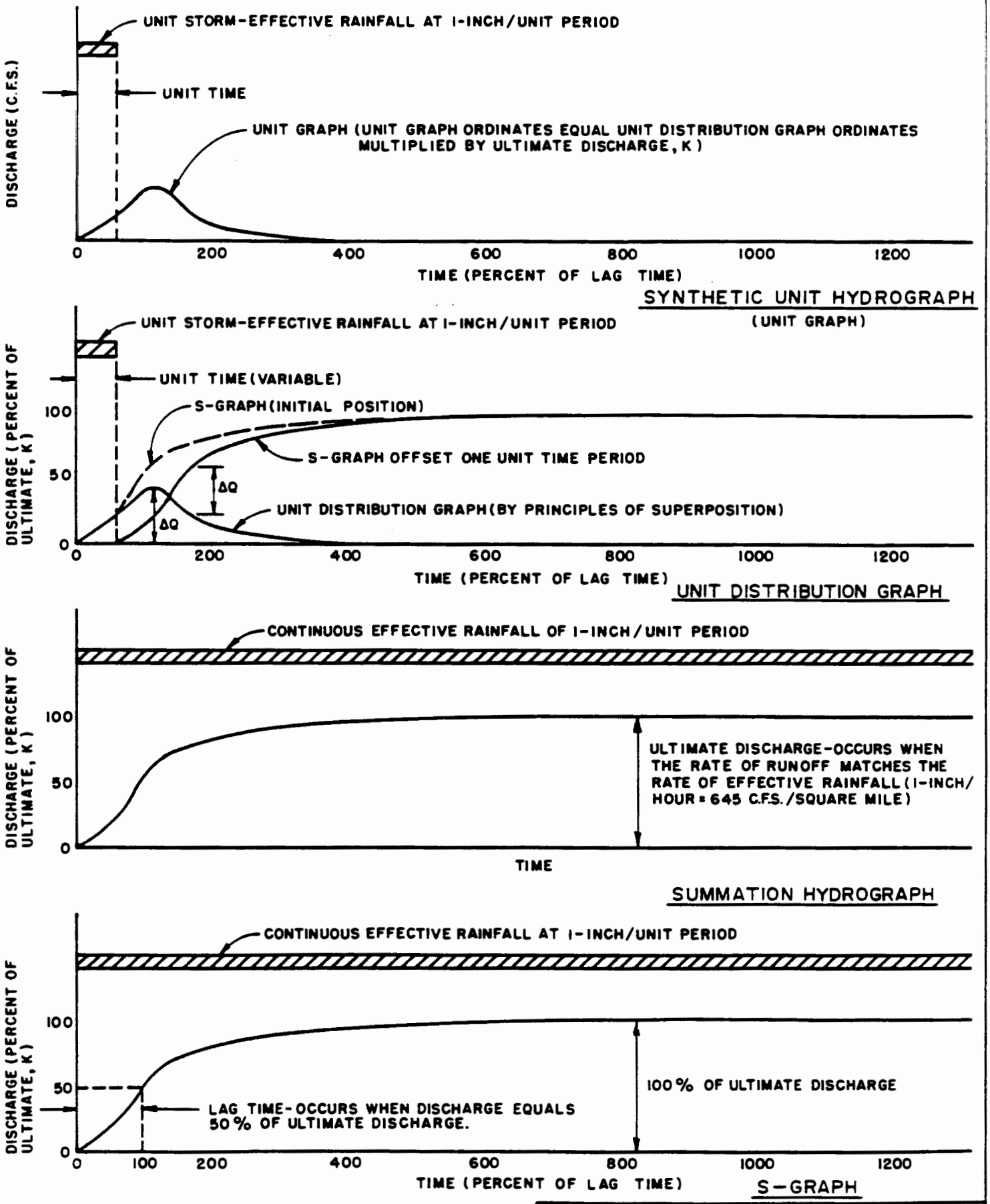
D. Compute the flood hydrograph.

1. Multiply the effective unit rainfall for the first unit time period by each synthetic unit hydrograph value to determine the flood hydrograph which would result from that rainfall increment.

2. Repeat the above process for each succeeding effective rainfall value, advancing the resultant flood hydrographs one unit time period for each computation cycle (see Figure E-10).

3. Sum the flow ordinates found in the steps above to determine the average flow ordinate per unit time period for the design storm flood hydrograph.

E. Add the appropriate base flow to the flood hydrograph ordinates determined in Step D.



KERN COUNTY

HYDROLOGY MANUAL

DERIVATION OF A SYNTHETIC UNIT HYDROGRAPH

FIGURE E-1

$\bar{n}$ = **0.015**

1. Drainage area has fairly uniform, gentle slopes
2. Most watercourses either improved or along paved streets
3. Groundcover consists of some grasses - large % of area impervious
4. Main water course improved channel or conduit

$\bar{n}$ = **0.020**

1. Drainage area has some graded and non-uniform, gentle slopes
2. Over half of the area watercourses are improved or paved streets
3. Groundcover consists of equal amount of grasses and impervious area
4. Main watercourse is partly improved channel or conduit and partly greenbelt (see n = 0.025)

$\bar{n}$ = **0.025**

1. Drainage area is generally rolling with gentle side slopes
2. Some drainage improvements in the area - streets and canals
3. Groundcover consists mostly of scattered brush and grass and small % impervious
4. Main watercourse is straight channels which are turfed or with stony beds and weeds on earth bank (greenbelt type)

$\bar{n}$ = **0.030**

1. Drainage area is generally rolling with rounded ridges and moderate side slopes
2. No drainage improvements exist in the area
3. Groundcover includes scattered brush and grasses
4. Watercourses meander in fairly straight, unimproved channels with some boulders and lodged debris

$\bar{n}$ = **0.040**

1. Drainage area is composed of steep upper canyons with moderate slopes in lower canyons
2. No drainage improvements exist in the area
3. Groundcover is mixed brush and trees with grasses in lower canyons
4. Watercourses have moderate bends and are moderately impeded by boulders and debris with meandering courses

$\bar{n}$ = **0.050**

1. Drainage area is quite rugged with sharp ridges and steep canyons
2. No drainage improvements exist in the area
3. Groundcover, excluding small areas of rock outcrops, includes many trees and considerable underbrush
4. Watercourses meander around sharp bends, over large boulders and considerable debris obstruction

$\bar{n}$ = **0.200**

1. Drainage area has comparatively uniform slopes
2. No drainage improvements exist in the area
3. Groundcover consists of cultivated crops or substantial growths of grass and fairly dense small shrubs, cacti, or similar vegetation
4. Surface characteristics are such that channelization does not occur

KERN COUNTY

HYDROLOGY MANUAL

BASIN FACTOR DESCRIPTIONS

Figure E-2

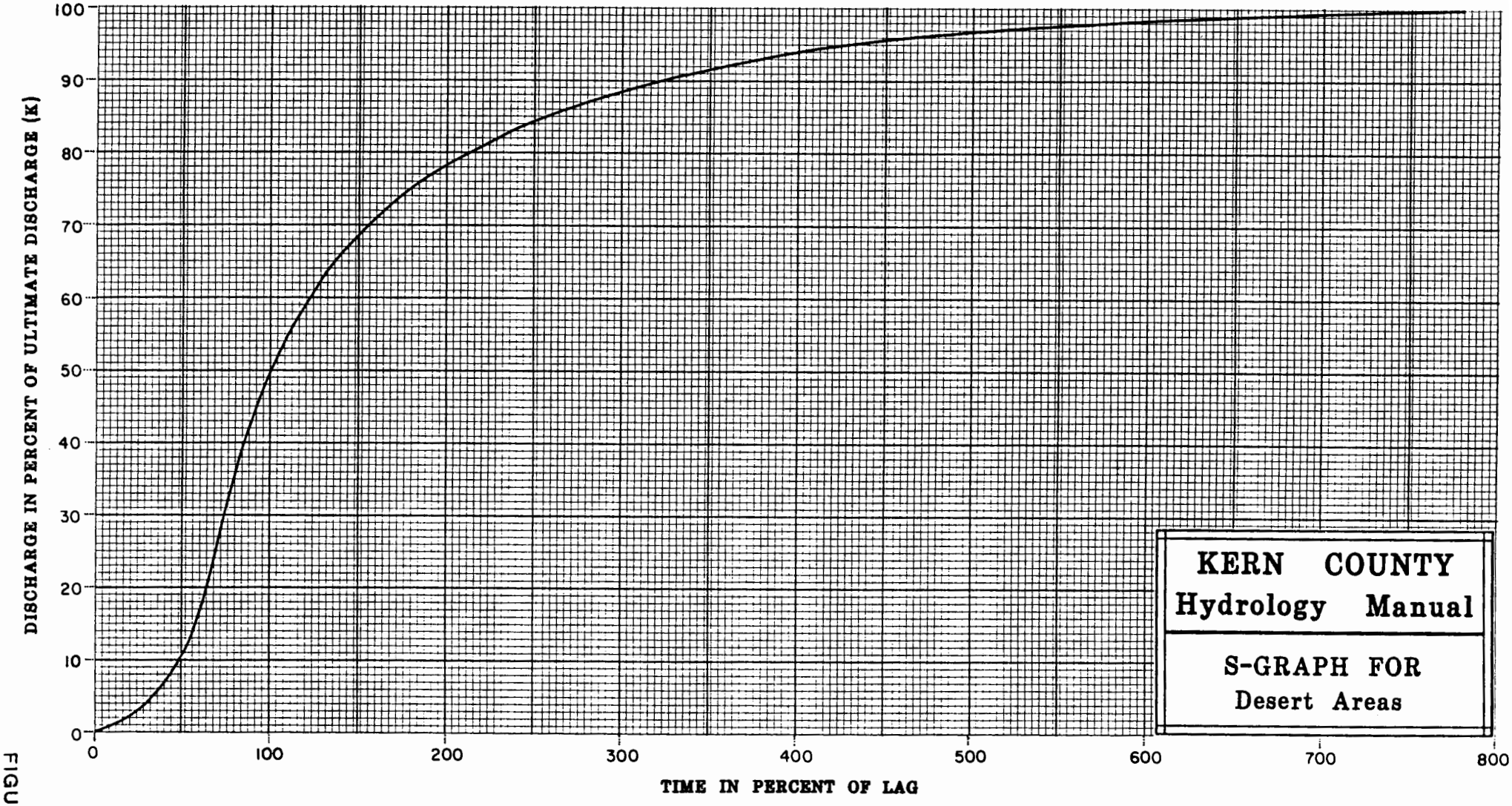

DISCHARGE IN PERCENT OF ULTIMATE DISCHARGE (K)
100
90
80
70
60
50
40
30
20
10
0
0
100
200
300
400
500
600
700
800
TIME IN PERCENT OF LAG
KERN COUNTY
Hydrology Manual
S-GRAPH FOR
Desert Areas

FIGURE E-3a

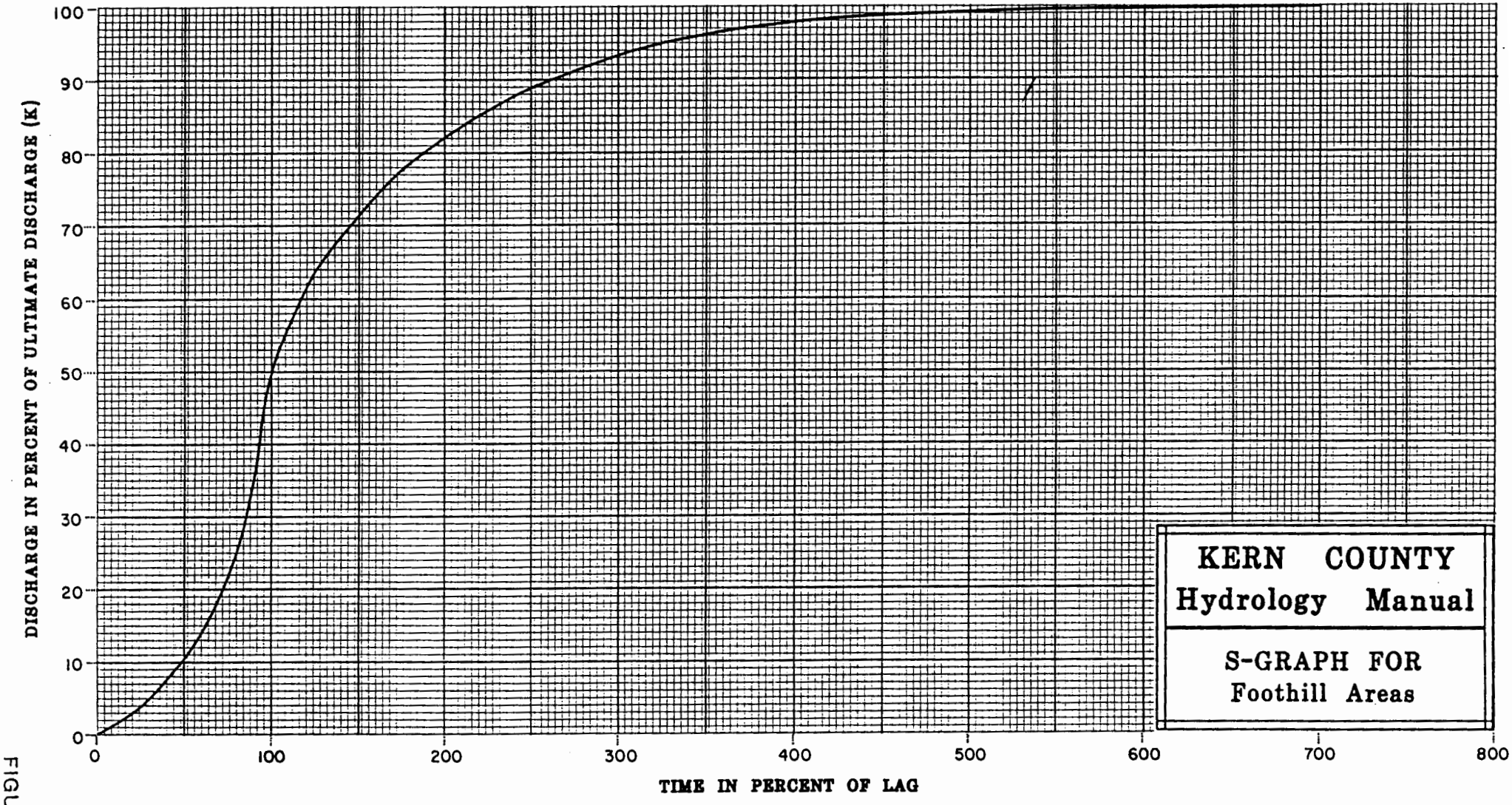

DISCHARGE IN PERCENT OF ULTIMATE DISCHARGE (K)
100
90
80
70
60
50
40
30
20
10
0
0
100
200
300
400
500
600
700
800
TIME IN PERCENT OF LAG
KERN COUNTY
Hydrology Manual
S-GRAPH FOR
Foothill Areas

FIGURE E-3b

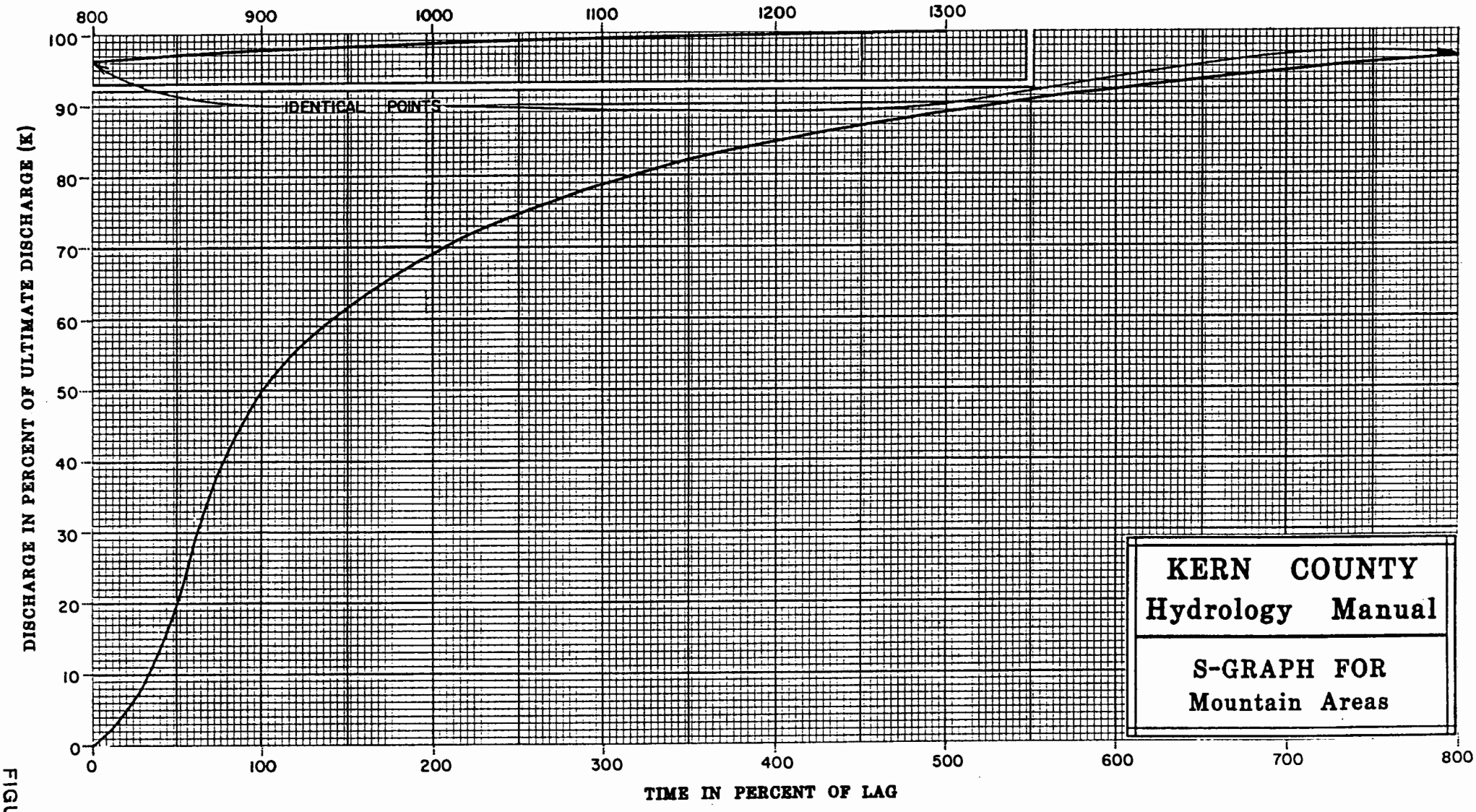

800
900
1000
1100
1200
1300
100
90
80
70
60
50
40
30
20
10
0
IDENTICAL POINTS
DISCHARGE IN PERCENT OF ULTIMATE DISCHARGE (K)
0
100
200
300
400
500
600
700
800
TIME IN PERCENT OF LAG
KERN COUNTY
Hydrology Manual
S-GRAPH FOR
Mountain Areas

FIGURE E-3C

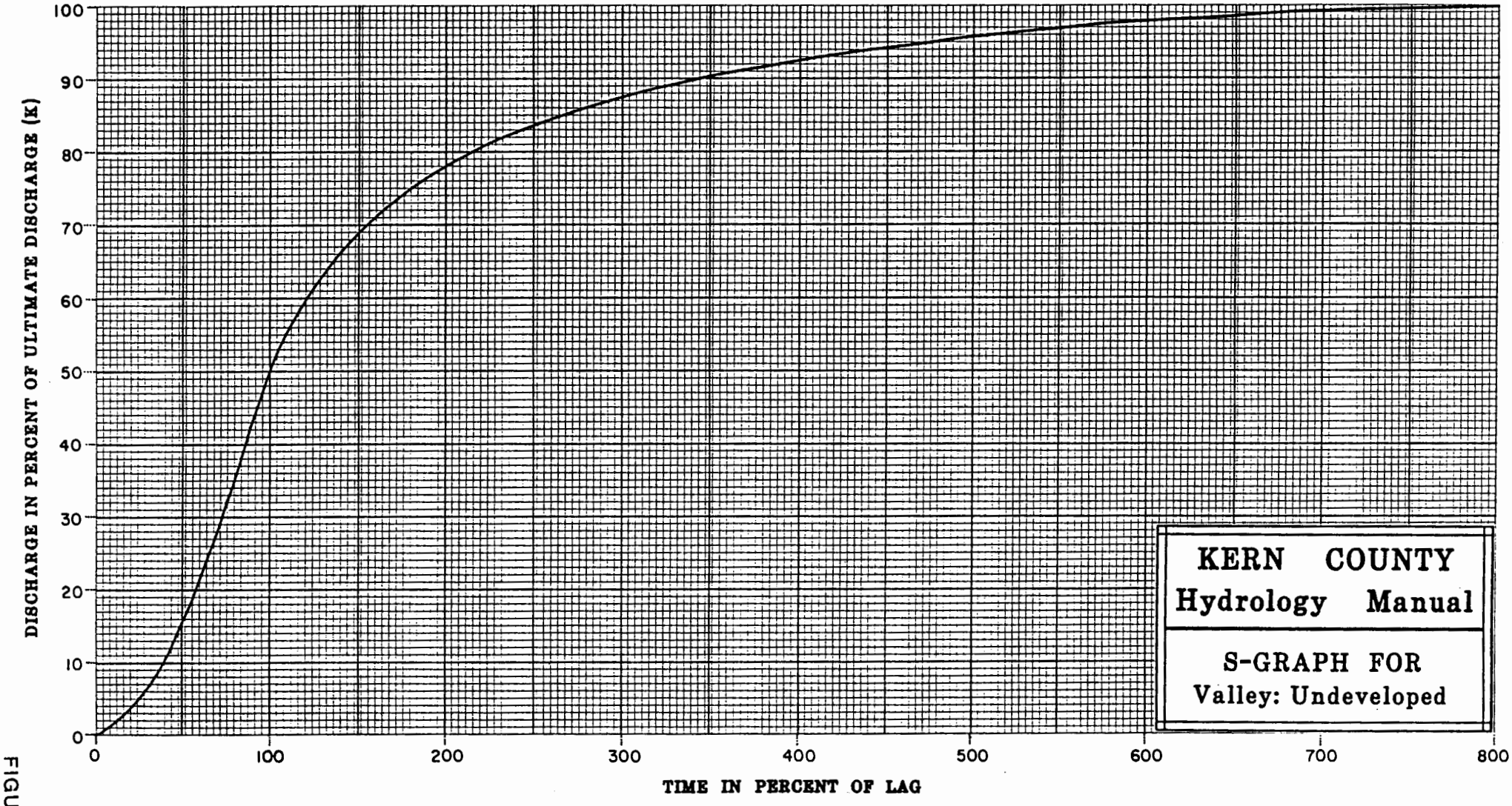

DISCHARGE IN PERCENT OF ULTIMATE DISCHARGE (K)
100
90
80
70
60
50
40
30
20
10
0
0
100
200
300
400
500
600
700
800
TIME IN PERCENT OF LAG
KERN COUNTY
Hydrology Manual
S-GRAPH FOR
Valley: Undeveloped

FIGURE E-3d

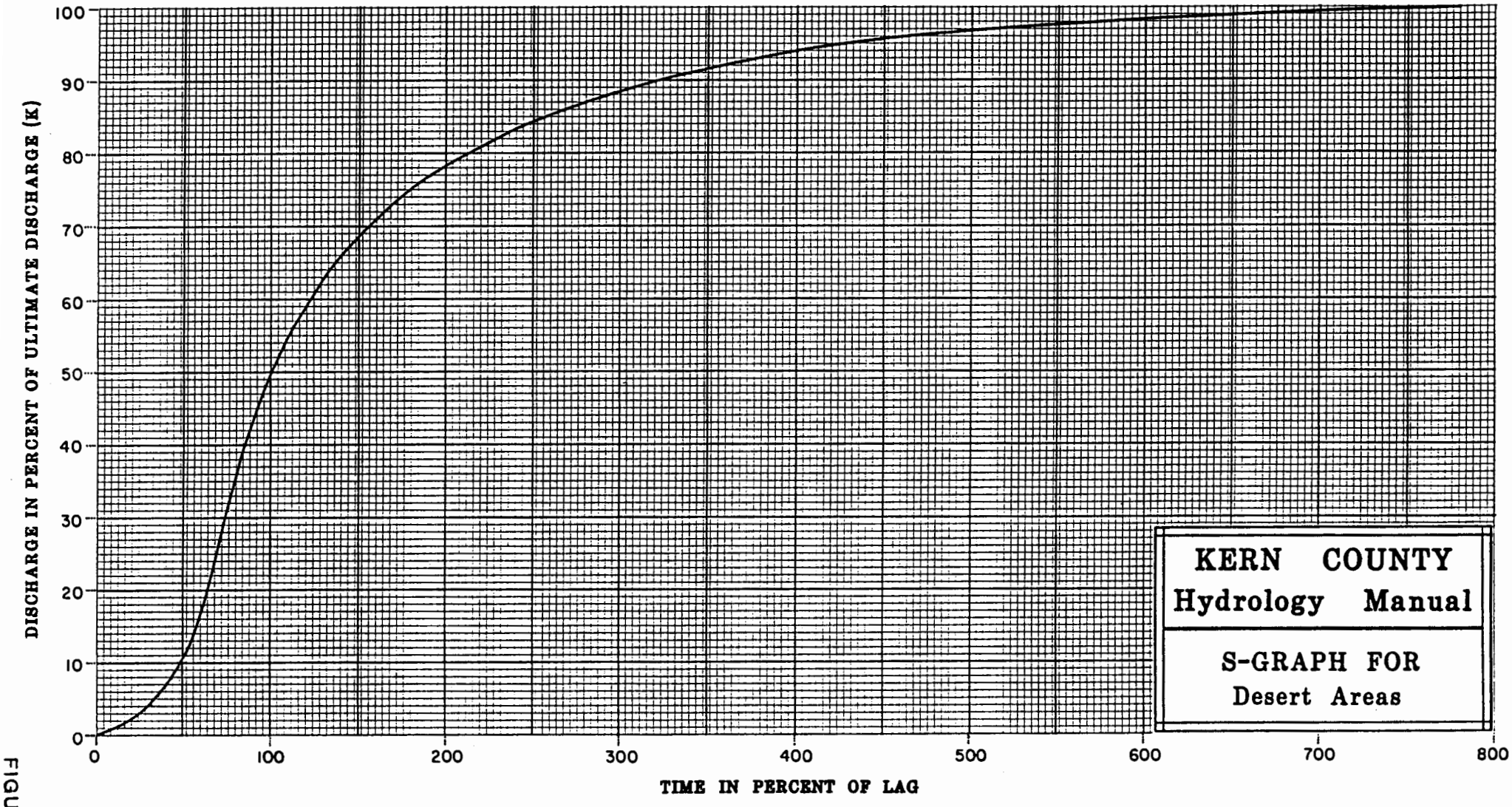
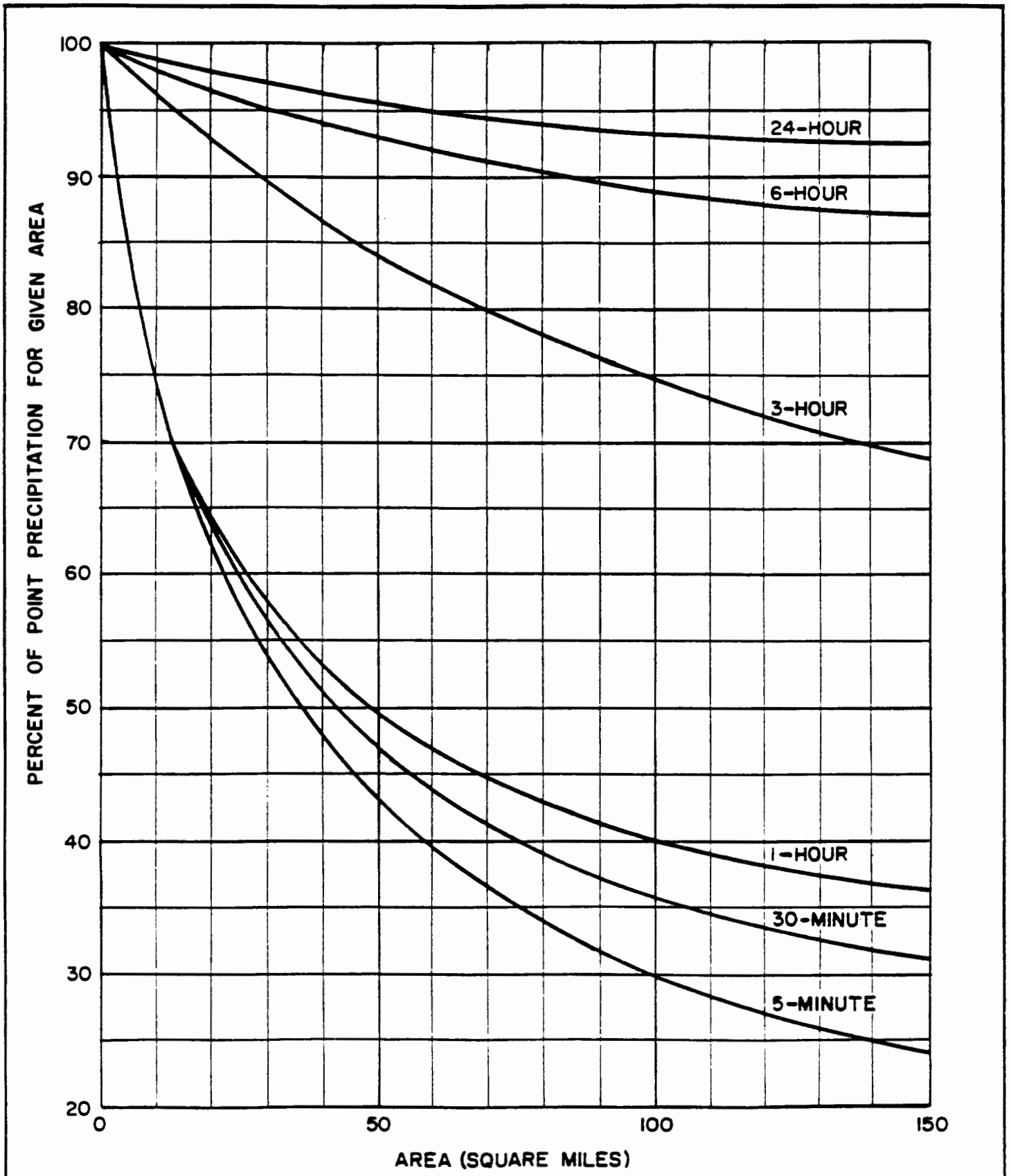

DISCHARGE IN PERCENT OF ULTIMATE DISCHARGE (K)
100
90
80
70
60
50
40
30
20
10
0
0
100
200
300
400
500
600
700
800
TIME IN PERCENT OF LAG
KERN COUNTY
Hydrology Manual
S-GRAPH FOR
Desert Areas

FIGURE E-3e

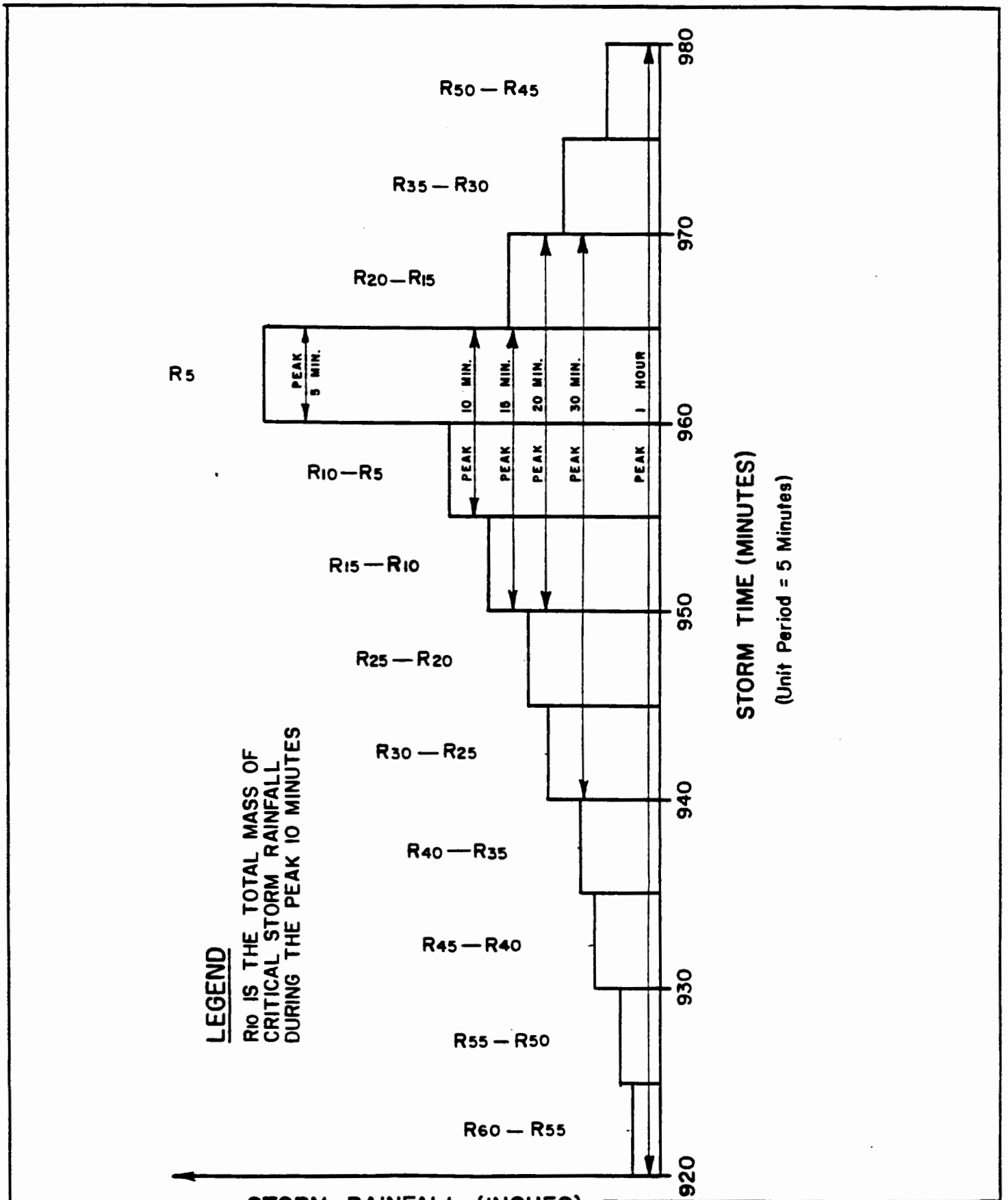


KERN COUNTY

HYDROLOGY MANUAL

DESIGN STORM DEPTH AREA CURVES

Figure E-4

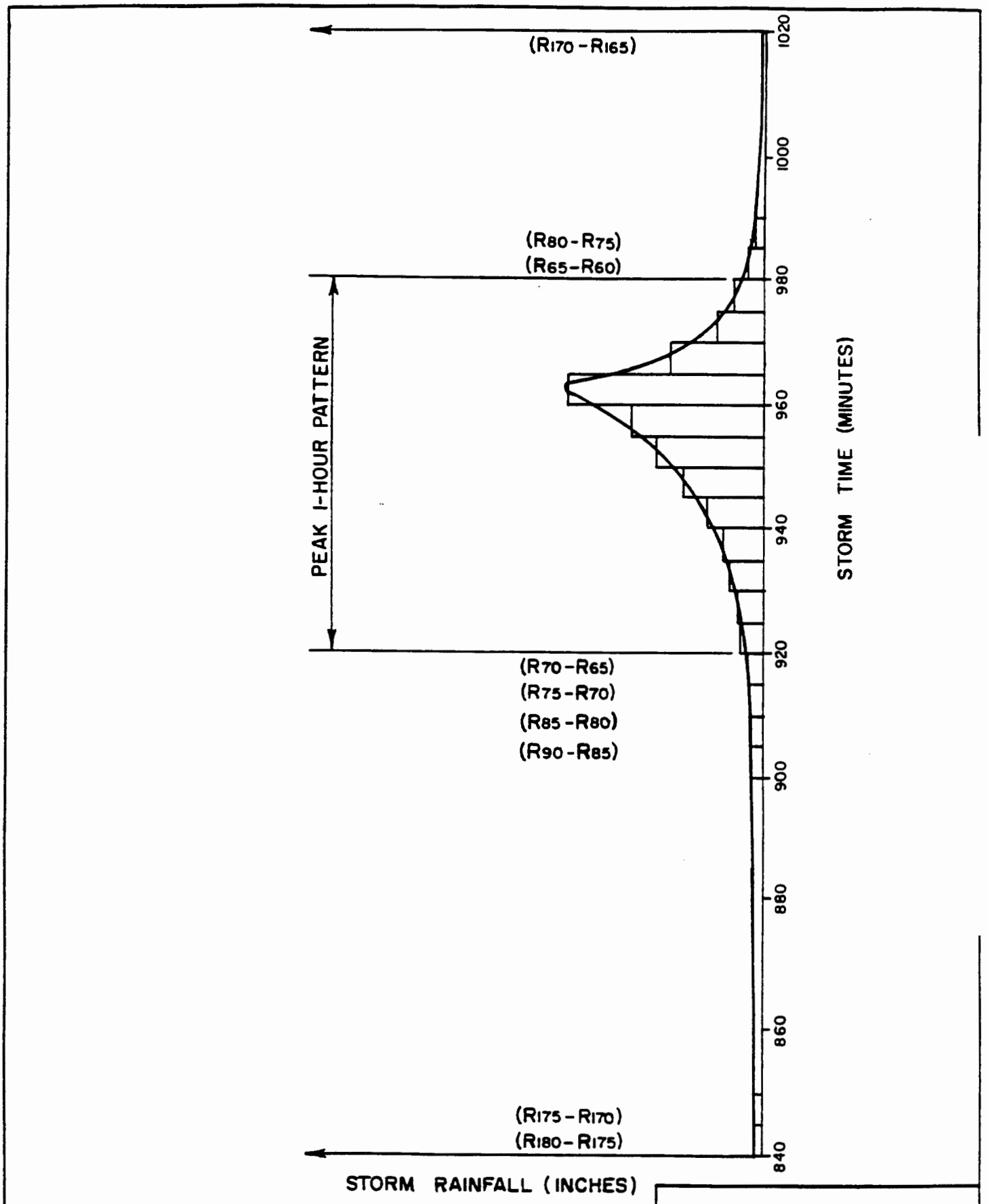


KERN COUNTY

HYDROLOGY MANUAL

DESIGN CRITICAL STORM PEAK 1-HOUR PATTERN

Figure E-5a

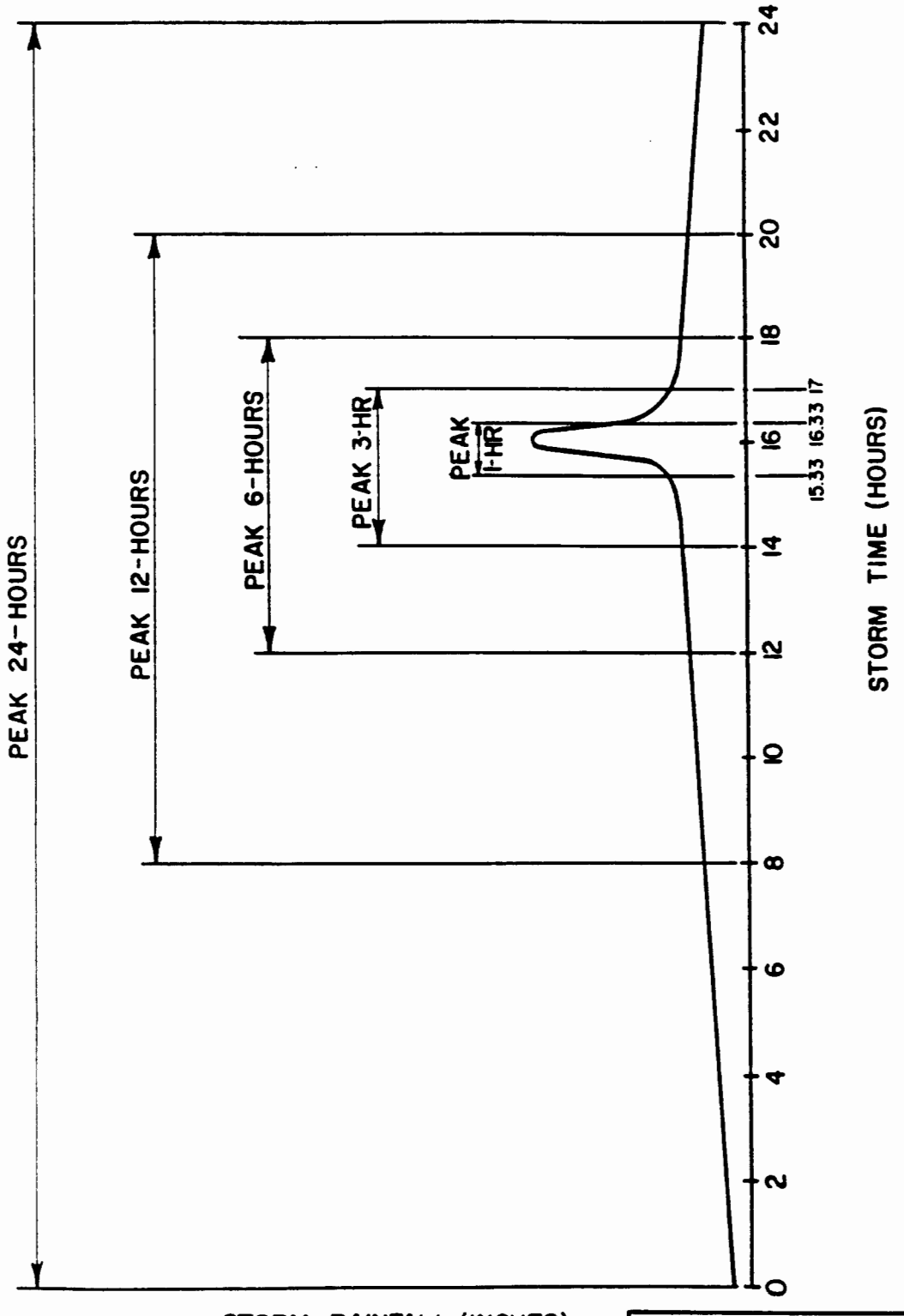


KERN COUNTY

HYDROLOGY MANUAL

DESIGN CRITICAL STORM PEAK 3-HOUR PATTERN

Figure E-5b

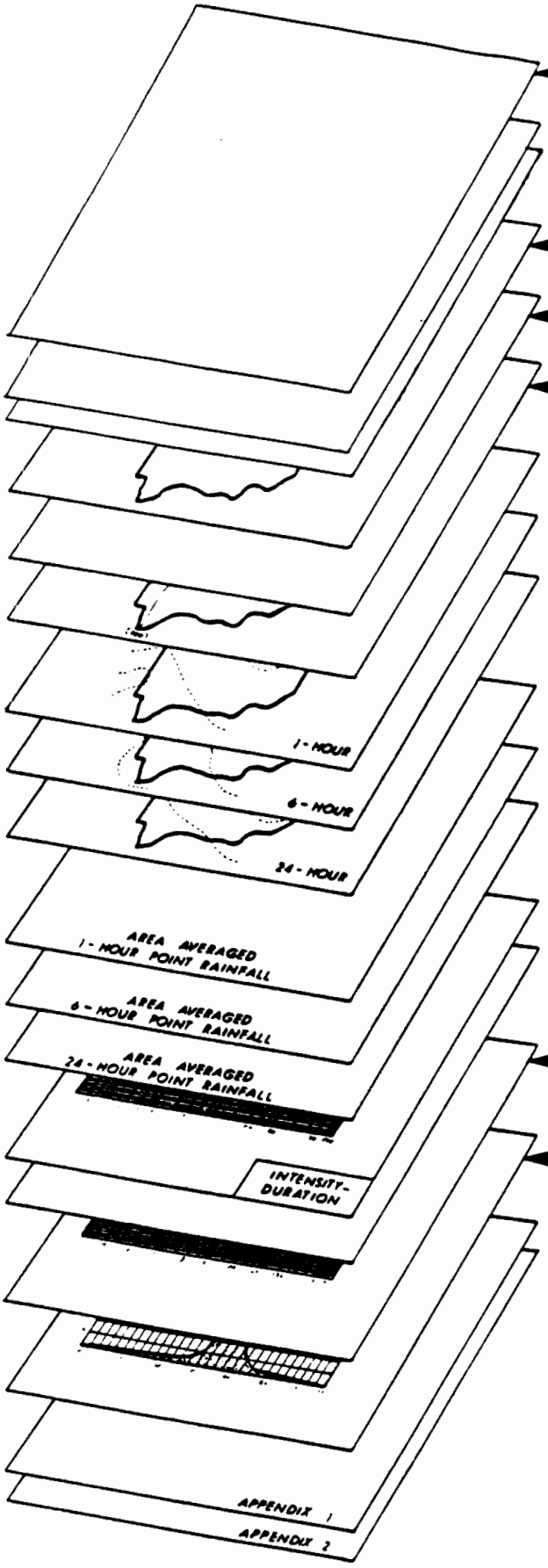


KERN COUNTY

HYDROLOGY MANUAL

DESIGN CRITICAL STORM PEAK 24-HOUR PATTERN

Figure E-5c



COVER PAGE WITH R.C.E. STAMP

DISCUSSION (PURPOSE, DESCRIPTION OF WATERSHED, METHODOLOGY, SUMMARY)

WATERSHED BOUNDARY/TOPOGRAPHIC MAP

WATERSHED INFORMATION FORM (FIG. E-7)

WATERSHED GEOMETRIC INFORMATION MAP

WATERSHED POINT RAINFALL ISOHYETAL MAPS

WATERSHED POINT RAINFALL AREA-AVERAGING CALCULATIONS

WATERSHED AREA AVERAGED POINT RAINFALL ESTIMATIONS FOR PEAK 5 AND 30 MINUTES, AND 3-HOURS OF RAINFALL. (FIG. D-3 AND E-8)

ADJUSTED AND UNADJUSTED MASS RAINFALL PLOTS (FIG. E-8)

WATERSHED RUNOFF HYDROGRAPH (3-HOUR OR 24-HOUR STORM, AS APPROPRIATE)

APPENDICES TO REPORT (e.g.: SOIL LOSS CALCULATIONS, etc.)

KERN COUNTY
HYDROLOGY MANUAL

REQUIRED REPORT FORMAT FOR UNIT HYDROGRAPH STUDY

Figure

PROJECT: _____ **DATE:** _____

ENGINEER: _____

1. Enter the design storm return frequency **(years)** _____

2. Enter catchment lag **(hours)** _____

3. Enter the catchment area **(acres)** _____

4. Enter baseflow **(cfs/square mile)** _____

5. Enter S-Graph proportions **(decimal)**

   Valley: Developed _____
   Foothill _____
   Mountain _____
   Valley: Undeveloped _____
   Desert _____

6. Enter maximum loss rate, F_m **(inch/hour)** _____

7. Enter low loss fraction, $\bar{Y}$ **(decimal)** _____

8. Enter watershed area-averaged 5-minute point rainfall **(inches)*** _____

   Enter watershed area-averaged 30-minute point rainfall **(inches)*** _____

   Enter watershed area-averaged 1-hour point rainfall **(inches)*** _____

   Enter watershed area-averaged 3-hour point rainfall **(inches)*** _____

   Enter watershed area-averaged 6-hour point rainfall **(inches)*** _____

   Enter watershed area-averaged 24-hour point rainfall **(inches)*** _____

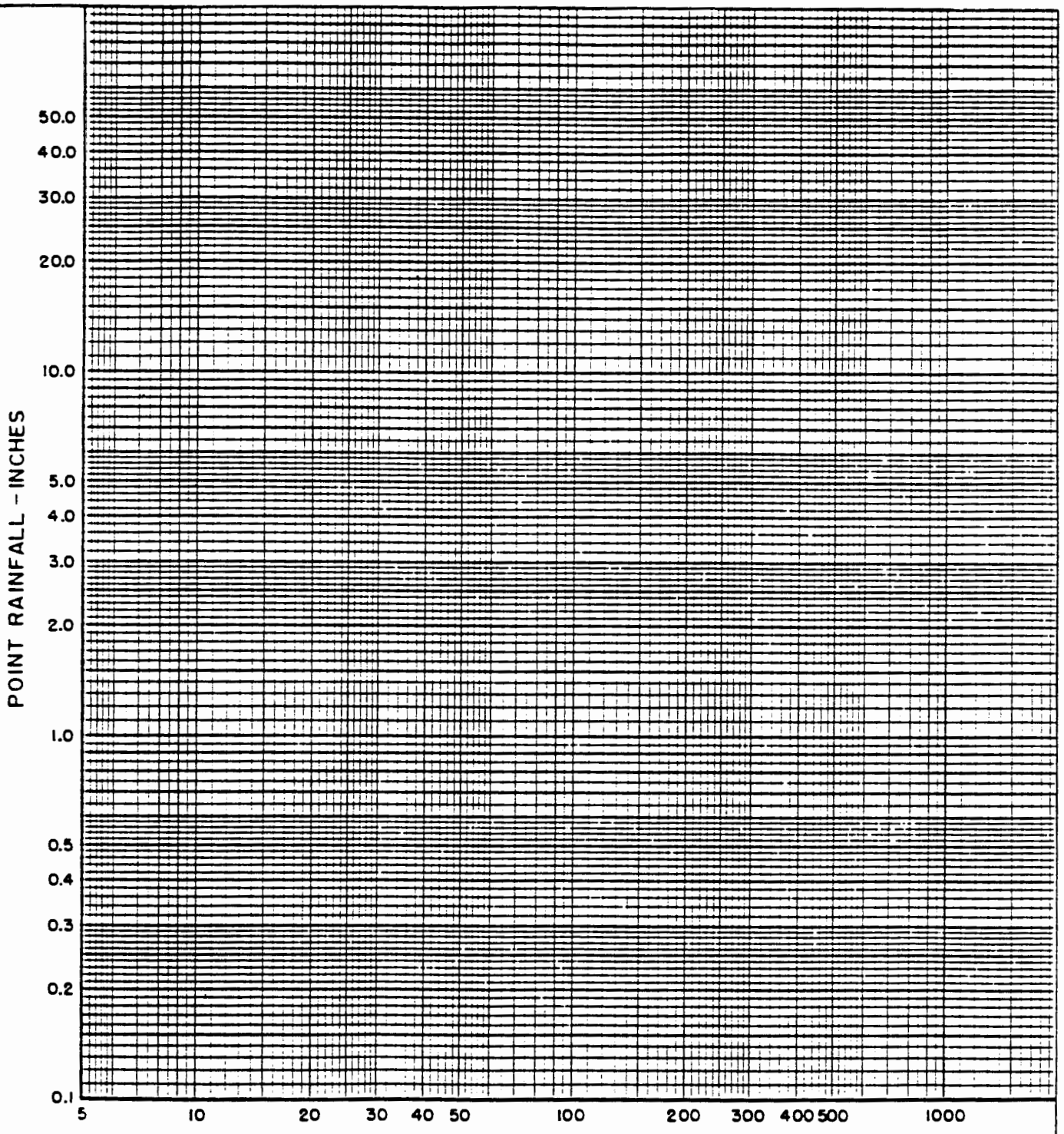
9. Enter 24-hour storm unit interval **(minutes)** _____

*Note: enter values unadjusted by depth-area factors

KERN COUNTY
HYDROLOGY MANUAL

WATERSHED INFORMATION FORM

Figure E-7



PROJECT LOCATION _____

NOTES _____

_____

KERN COUNTY

HYDROLOGY MANUAL

AREA - AVERAGED
MASS RAINFALL
PLOTTING SHEET

FIGURE E-8

KERN COUNTY

HYDROLOGY MANUAL

SYNTHETIC UNIT HYDROGRAPH METHOD

Flood Hydrograph Calculation Form

Project _____

_____

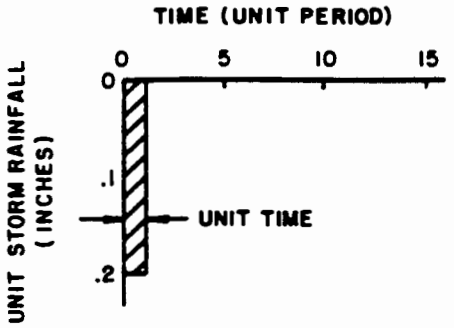
By _____ Date _____

Checked _____ Date _____

Sheet _____

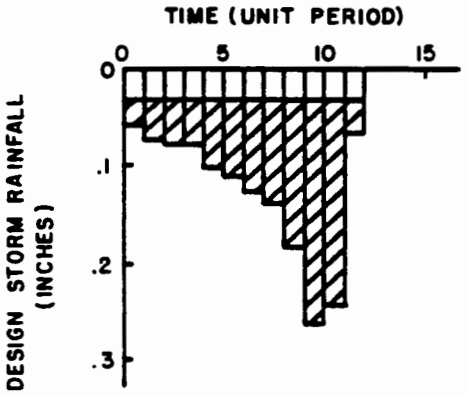
	EFFEC RAIN																								FLOOD HYDRO-GRAPH
UNIT GRAPH	UNIT PER.																								

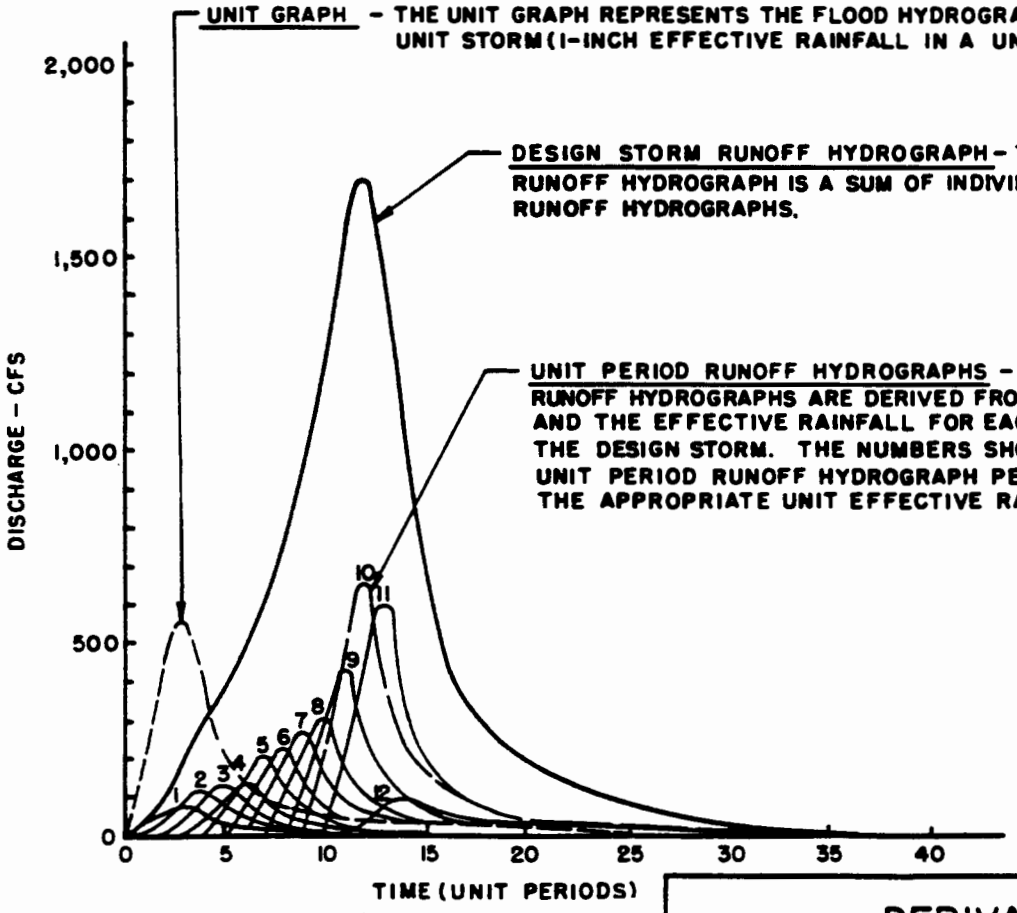
Figure E-9



LEGEND

- RAINFALL LOSS
- EFFECTIVE RAIN





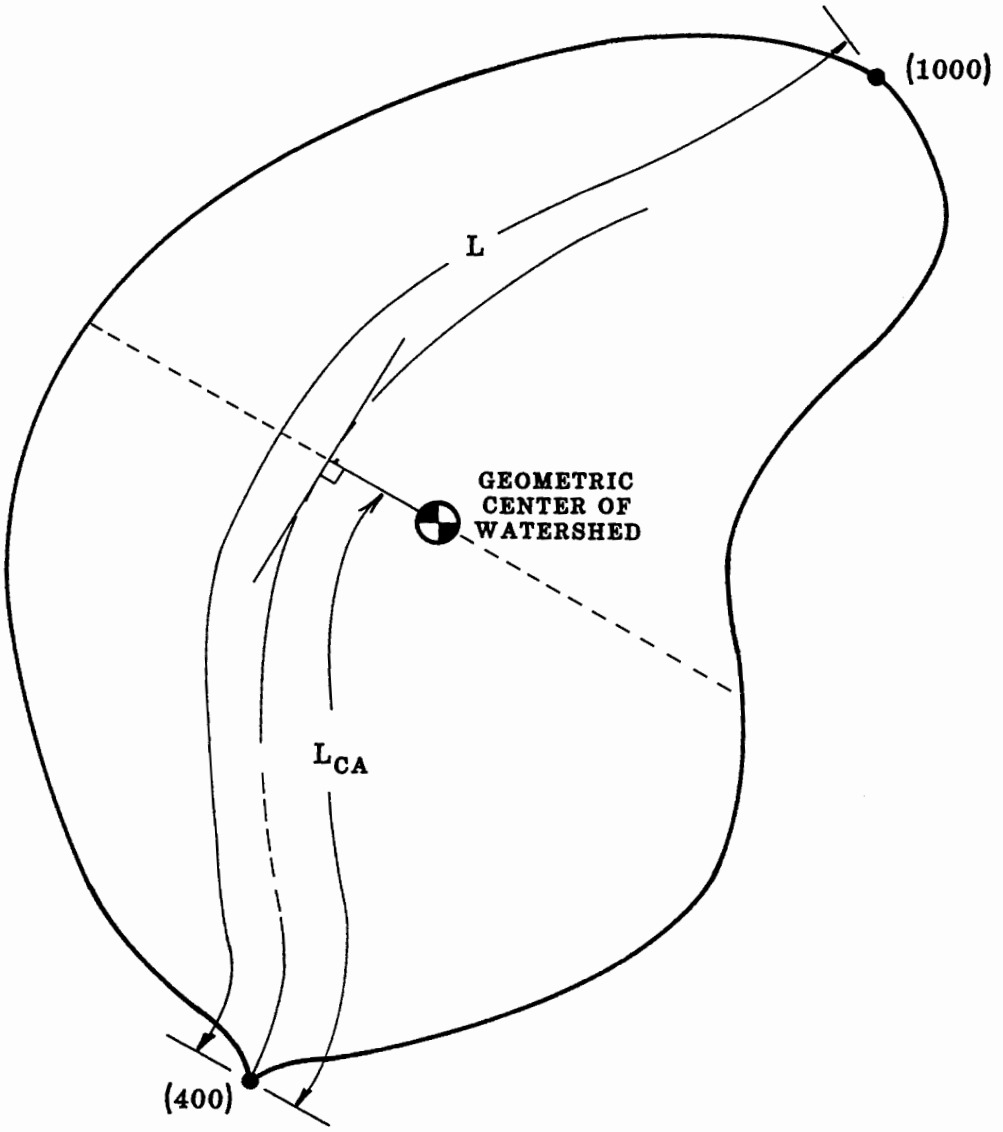
KERN COUNTY

HYDROLOGY MANUAL

DERIVATION OF A RUNOFF HYDROGRAPH

FIGURE E-10

E.13. EXAMPLE: Application of Lag Equation



L = 6.7 (length of longest watercourse)(miles)

L_{CA} = 3.2 (length along longest watercourse, measured upstream to a point opposite center of area)(miles)

$\bar{n}$ = .040 (by visual inspection)

s = (1000-400)/6.7

= 89.6 ft./mile

$$\text{Lag (hours)} = 24\bar{n}\left[(L \cdot L_{CA})/s^{0.5}\right]^{m}$$

$$= 24(.040)\left[(6.7 \cdot 3.2)/(89.6)^{0.5}\right]^{0.38}$$

$$= \underline{\underline{1.31 \text{ hours}}}$$

E.14. Example: Runoff Volume Estimation

The runoff volume from a 24-hour duration precipitation event with a 50-year return frequency is estimated using the low loss fraction. Assume a 1,000-acre watershed with the following land uses, cover complexes, and soil groups (see Figures C-3 and C-4):

TABLE E.1. EXAMPLE PROBLEM: ASSUMPTIONS

Subarea Number	Area Fraction	Land Use and Condition	Soil Group	Pervious CN
1	0.1	meadow: good condition	D	78
2	0.1	woodland: poor cover	C	77
3	0.1	annual grass: good condition	D	80
4	0.8	residential (1/4-acre lots)	C	74

impervious area fraction a_i=0.40

The yield computed for each subarea using (C.3) where, for the 50-year design storm, P_{24} = 1.90 inches (example) as follows:

TABLE E.2. EXAMPLE PROBLEM: YIELD CALCULATIONS

Subarea Numbers	CN	S	Ia	Y	Area Fraction
1	78	2.82	0.56	0.236	0.1
2	77	2.99	0.6	0.208	0.05
3	80	2.5	0.5	0.265	0.05
4a	74	3.51	0.7	0.16	0.48
4b	98	0.2	0.04	0.882	0.32

In Table E.2, subarea 4 is further subdivided into pervious and impervious area fractions as subareas 4a and 4b. Normally, the entire catchment impervious area fraction is lumped together for simplicity.

Using Table E.2, the catchment runoff yield (fraction), Y, is calculated from (C.4) as

Y = (0.226)(0.10) + (0.208)(0.05) + (0.265)(0.05) + (0.160)(0.48) + (0.882)(0.32)

= 0.405

Thus, the 24-hour 50-year design storm runoff volume is estimated to be

VOLUME = (0.405)(1.90-inches)(1000-acres)(1-ft/12-inches)

= 64.1 acre-feet

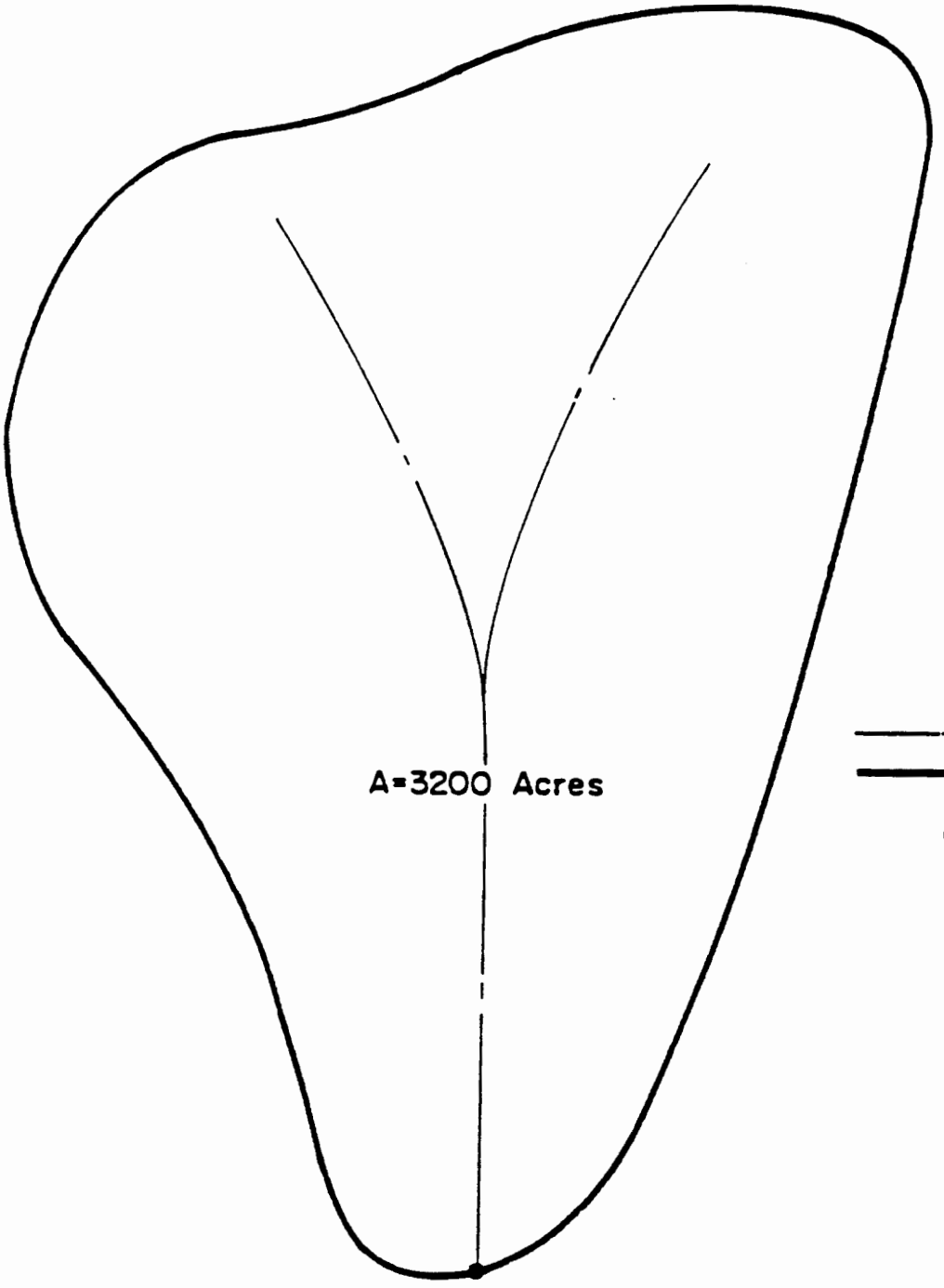
It is noted that the above volume estimate is typically slightly less than the runoff volume developed from a 24-hour 50-year design storm runoff hydrograph analysis because the effects of the maximum loss rate, F_m, were neglected in the calculations.

E.15. EXAMPLE: UNIT HYDROGRAPH METHOD

The following example problem illustrates the format suggested for synthetic unit hydrograph hydrology studies to be submitted for review. In the following, an example watershed is analyzed using the agency unit hydrograph approach. The example problem presentation contains the following information:

Description

- Watershed Map including Boundary and Geometric Information (Figure E-11)
- Watershed Information Form (Figure E-12)
- 3-Hour Area-Averaged Rainfall Estimation (Figure E-13)
- 5-Minute and 30-Minute Area-Averaged Rainfall Estimation (Figure E-14)
- Adjusted and Unadjusted Mass Rainfall Plots (Depth-Area Effects) (Figure E-15)
- Watershed-Loss Information Map (Figure E-16)
- Unit Rainfall Determination (Table E.3.)
- Area-Averaged Maximum Loss Rate (Fm) Determination (Table E.4.)
- Area-Averaged Low Loss Fraction Y) Determination (Table E.5.)
- 3-Hour Storm Unit Rainfall Determination (5-Minute Unit Interval) (Table E.6.)
- Effective Rainfall Determination (Table E.6.)
- 3-Hour Critical Storm (Figure E-17)
- S-Curve Approximation (Figure E-18)
- Unit Hydrograph Determination (Figure E-19)
- Runoff Hydrograph Determination (Figure E-20(a,b))
- Runoff Hydrograph (Figure E-21)



KERN COUNTY

HYDROLOGY MANUAL

EXAMPLE PROBLEM
WATERSHED BOUNDARY

FIGURE E-11

PROJECT: EXAMPLE PROBLEM DATE:

ENGINEER:

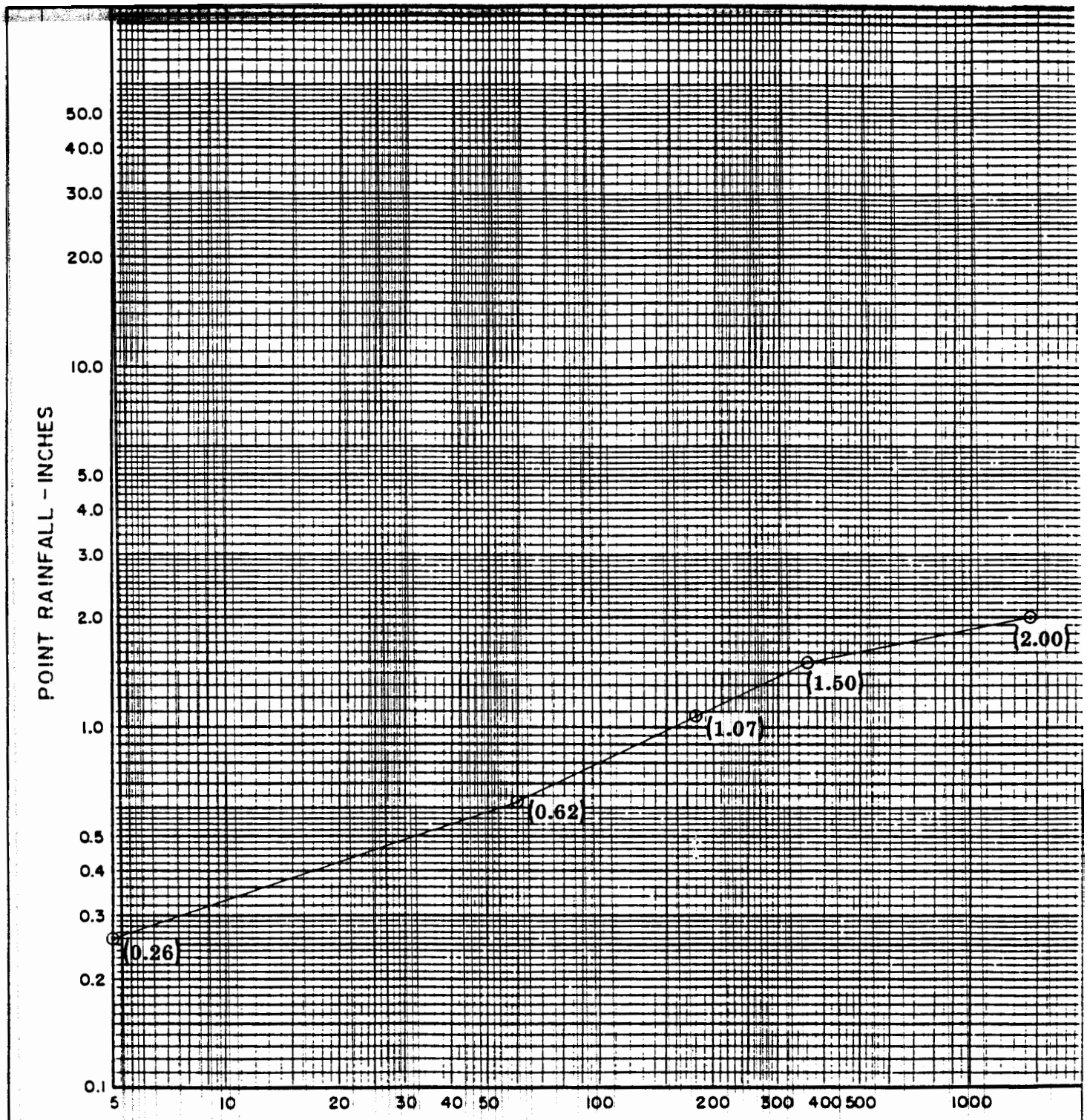
1.	Enter the design storm return frequency **(years)**	100
2.	Enter catchment lag **(hours)**	0.75
3.	Enter the catchment area **(acres)**	3200
4.	Enter baseflow **(cfs/square mile)**	0
5.	Enter S-Graph proportions **(decimal)**	
	Valley: Developed	1.0
	Foothill	0
	Mountain	0
	Valley: Undeveloped	0
	Desert	0
6.	Enter maximum loss rate, F_m **(inch/hour)**	0.28
7.	Enter low loss fraction, $\bar{Y}$ **(decimal)**	0.45
8.	Enter watershed area-averaged 5-minute point rainfall **(inches)***	0.26
	Enter watershed area-averaged 30-minute point rainfall **(inches)***	0.49
	Enter watershed area-averaged 1-hour point rainfall **(inches)***	0.62
	Enter watershed area-averaged 3-hour point rainfall **(inches)***	1.07
	Enter watershed area-averaged 6-hour point rainfall **(inches)***	1.50
	Enter watershed area-averaged 24-hour point rainfall **(inches)***	2.00
9.	Enter 24-hour storm unit interval **(minutes)**	5.0

*Note: enter values unadjusted by depth-area factors

KERN COUNTY
HYDROLOGY MANUAL

WATERSHED
INFORMATION FORM

FIGURE E-12



PROJECT LOCATION EXAMPLE PROBLEM

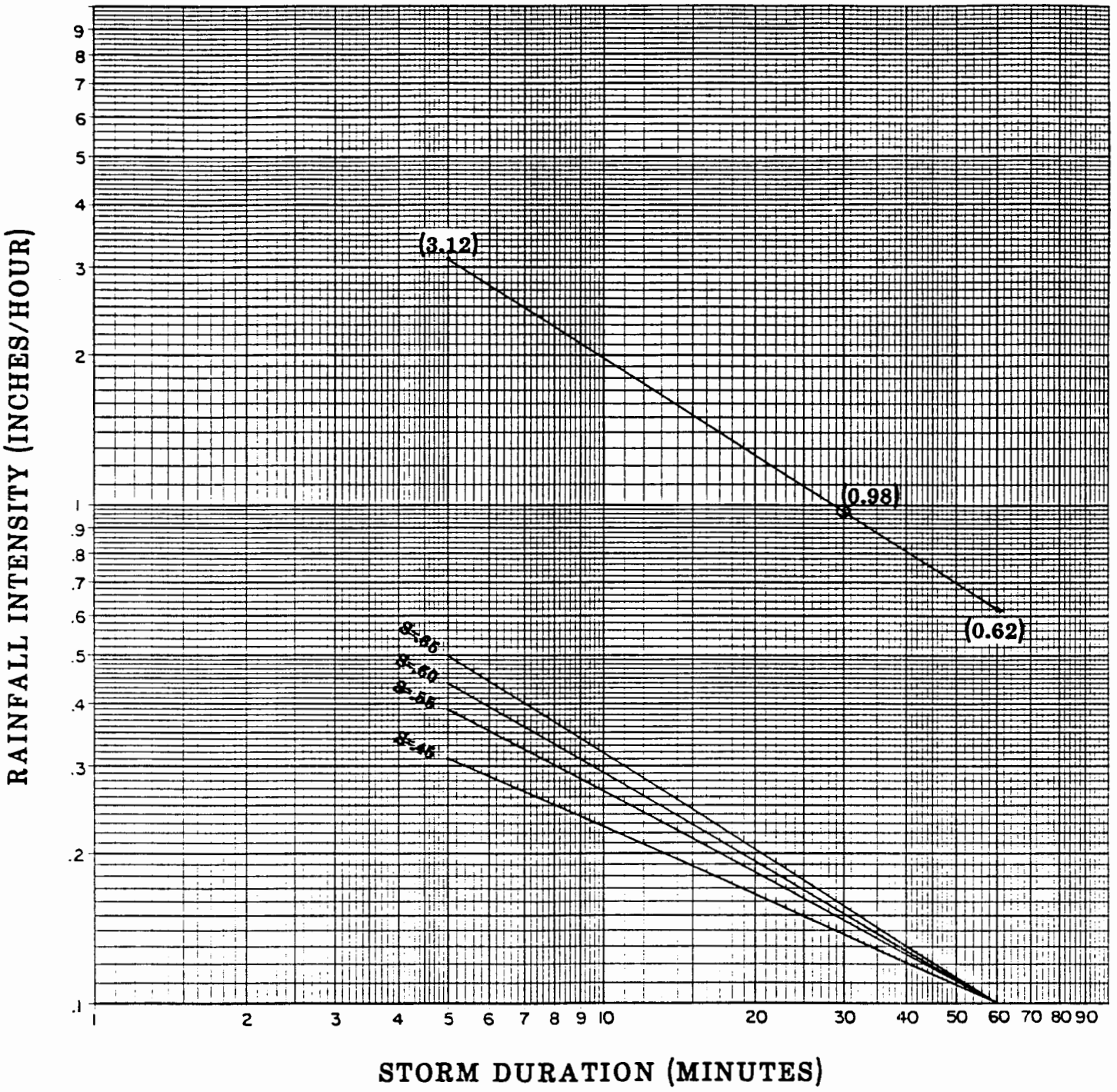
NOTES 100-YR; 1 HOUR = 0.62; 6-HOUR = 1.50; 24-HOUR = 2.00

KERN COUNTY

HYDROLOGY MANUAL

AREA - AVERAGED MASS RAINFALL PLOTTING SHEET

FIGURE E-13



DESIGN STORM FREQUENCY = 100 YEARS

ONE HOUR POINT RAINFALL= 0.62 INCHES

LOG-LOG SLOPE= 0.66

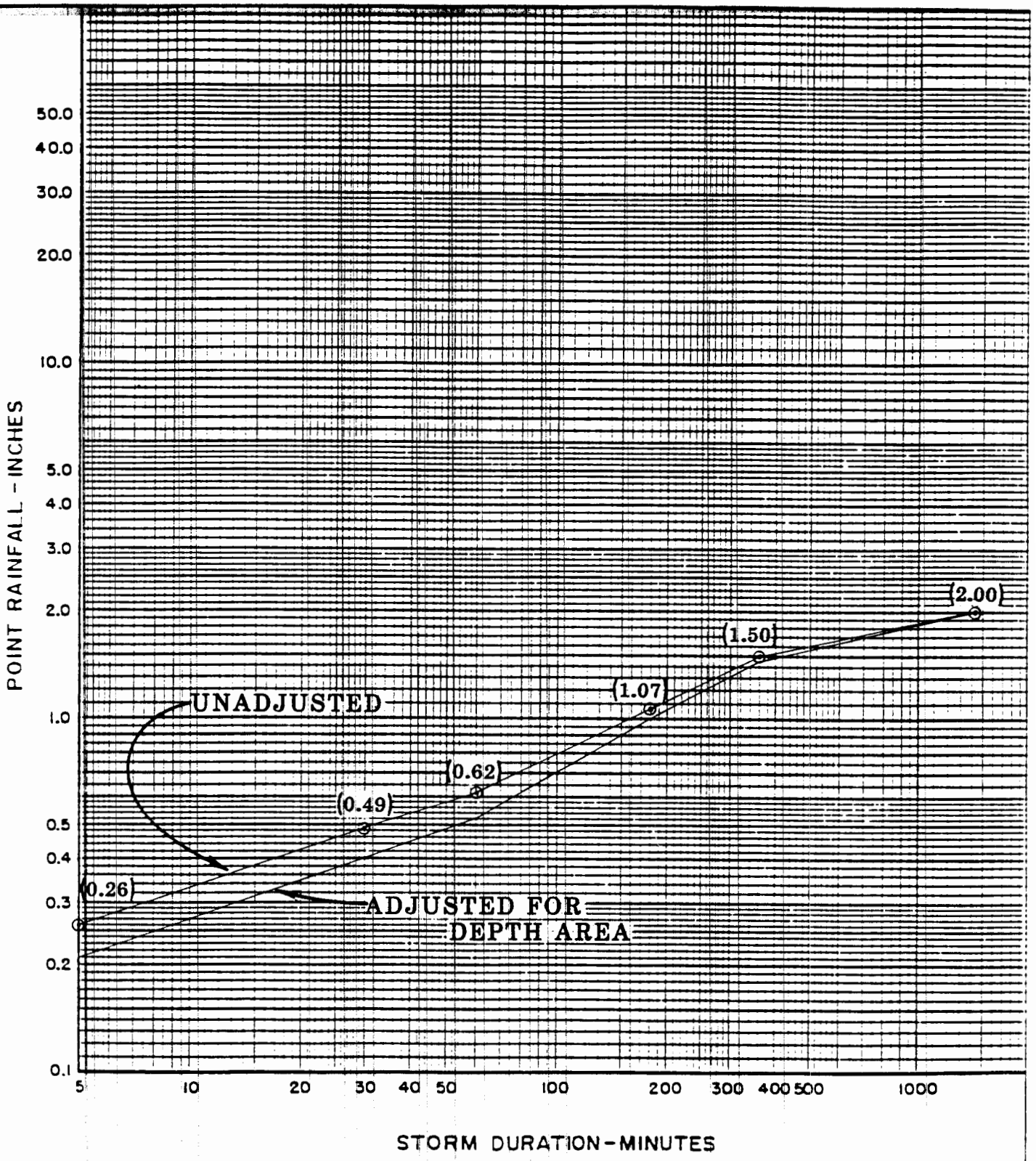
PROJECT LOCATION= EXAMPLE PROBLEM

KERN COUNTY
Hydrology Manual

INTENSITY-DURATION CURVES

CALCULATION SHEET

FIGURE E-14



PROJECT LOCATION EXAMPLE PROBLEM

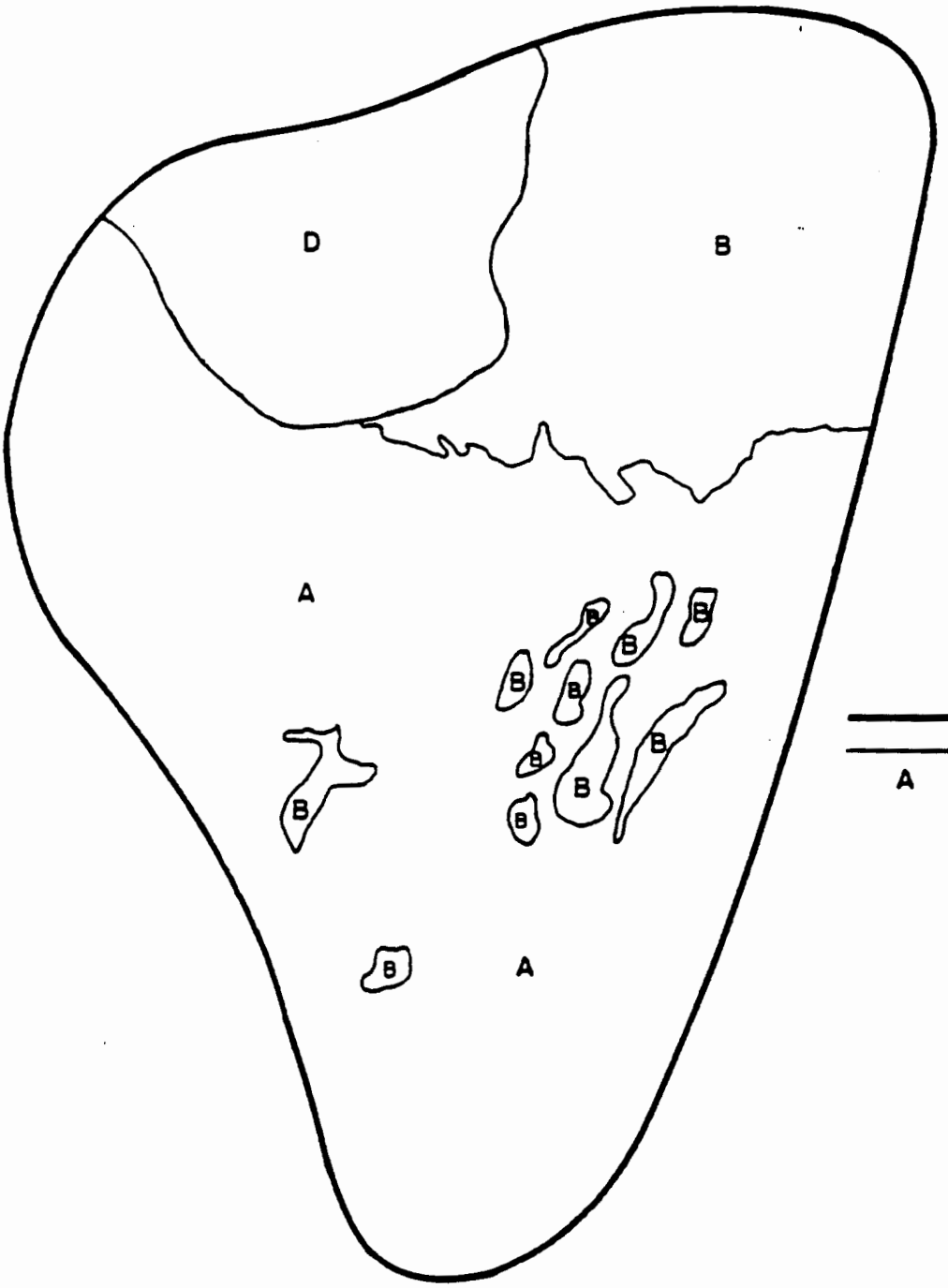
NOTES 100 - YEAR STORM

KERN COUNTY

HYDROLOGY MANUAL

AREA - AVERAGED
MASS RAINFALL
PLOTTING SHEET

FIGURE E-15



LEGEND

Watershed Boundary

Soil Group Boundary

A Soil Group Designation

KERN COUNTY

HYDROLOGY MANUAL

EXAMPLE PROBLEM

WATERSHED-LOSS DETERMINATION

INFORMATION MAP

FIGURE E-16

TABLE E.3.EXAMPLE PROBLEM: UNIT HYDROGRAPH STUDY

UNIT RAINFALL DETERMINATION (Example Unit Period=5 minutes)

Peak Rainfall Unit Rainfall	Intensity	Depth	Adjusted Depth	Unit Number (inches)
1	3.12	0.26	0.22	0.218
2	1.99	0.33	0.28	0.060
3	1.53	0.38	0.32	0.042
4	1.27	0.42	0.35	0.034
5	1.10	0.46	0.38	0.029
6	0.97	0.49	0.41	0.025
7	0.88	0.51	0.43	0.023
8	0.81	0.54	0.45	0.021
9	0.75	0.56	0.47	0.019
10	0.70	0.58	0.49	0.018
11	0.66	0.60	0.51	0.017
12	0.62	0.62	0.52	0.016
13	0.60	0.64	0.54	0.021
14	0.57	0.67	0.56	0.020
15	0.55	0.69	0.58	0.019
16	0.54	0.71	0.60	0.019
17	0.52	0.74	0.62	0.018
18	0.50	0.76	0.64	0.018
19	0.49	0.78	0.65	0.017
20	0.48	0.80	0.67	0.017
21	0.47	0.82	0.69	0.016
22	0.46	0.84	0.70	0.016
23	0.45	0.85	0.72	0.016
24	0.44	0.87	0.73	0.015
25	0.43	0.89	0.75	0.015
26	0.42	0.91	0.76	0.015
27	0.41	0.92	0.78	0.014
28	0.40	0.94	0.79	0.014
29	0.40	0.96	0.80	0.014
30	0.39	0.97	0.82	0.014
31	0.38	0.99	0.83	0.013
32	0.38	1.01	0.84	0.013
33	0.37	1.02	0.86	0.013
34	0.37	1.04	0.87	0.013
35	0.36	1.05	0.88	0.013
36	0.36	1.07	0.90	0.012
TIME = 3 HOURS			TOTAL =	0.90 INCHES

TABLE E.4. EXAMPLE PROBLEM: UNIT HYDROGRAPH STUDY,
WATERSHED LOSS DETERMINATIONS

Area-Averaged Maximum Loss Rate, Fm

1. Using the watershed soil and development characteristics, estimate the area-averaged maximum loss rate:

Land Use and Condition	**Area Fraction**	**Soil Group**	**F_p (in/hr) Fig C.6**	**a_p Fig C.4**	**F_m (in/hr)**
Annual grass; fair cover (100% pervious)	.10	D	0.31	1.0	0.31
Residential: S.F. (1/4 acre) Lots (50% pervious)	.60	B	0.75	0.50	0.38
Commercial: (10% pervious)	.30	B	0.75	0.10	.075

Area-Averaged Adjusted Loss Rate (inch/hour) = 0.28

TABLE E.5. EXAMPLE PROBLEM: UNIT HYDROGRAPH STUDY, WATERSHED LOSS DETERMINATIONS

Area-Averaged Low Loss Fraction, $\bar{Y}$

1. Using the watershed soil and development characteristics, estimate the area-averaged low loss fraction:

Land Use and Condition	Area Fraction	Soil Group	Curve Number (CN) Fig C.3	S[2]	Pervious Area Yield Fraction, Y[3]
Annual grass; fair cover (100% pervious)	.10	D	84	1.90	0.37
Residential: S.F. (1/4 acre) Lots (50% pervious)	.30	B	56	7.86	0.01
	.30	B	98	0.20	0.89
Commercial: (10% pervious)	.03	B	56	7.86	0.01
	.27	B	98	0.20	0.89

Area-Averaged Catchment Yield Fraction (Y) = 0.550

Area Averaged Low Loss Fraction ($\bar{Y}$) = . 45

NOTES:

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

(3) $Y = (P24-0.2S)^2/((P24+0.8S)P24$

(4) $Y = 1-Y$

TABLE E.6. EXAMPLE PROBLEM:

UNIT HYDROGRAPH STUDY, 3-HOUR STORM

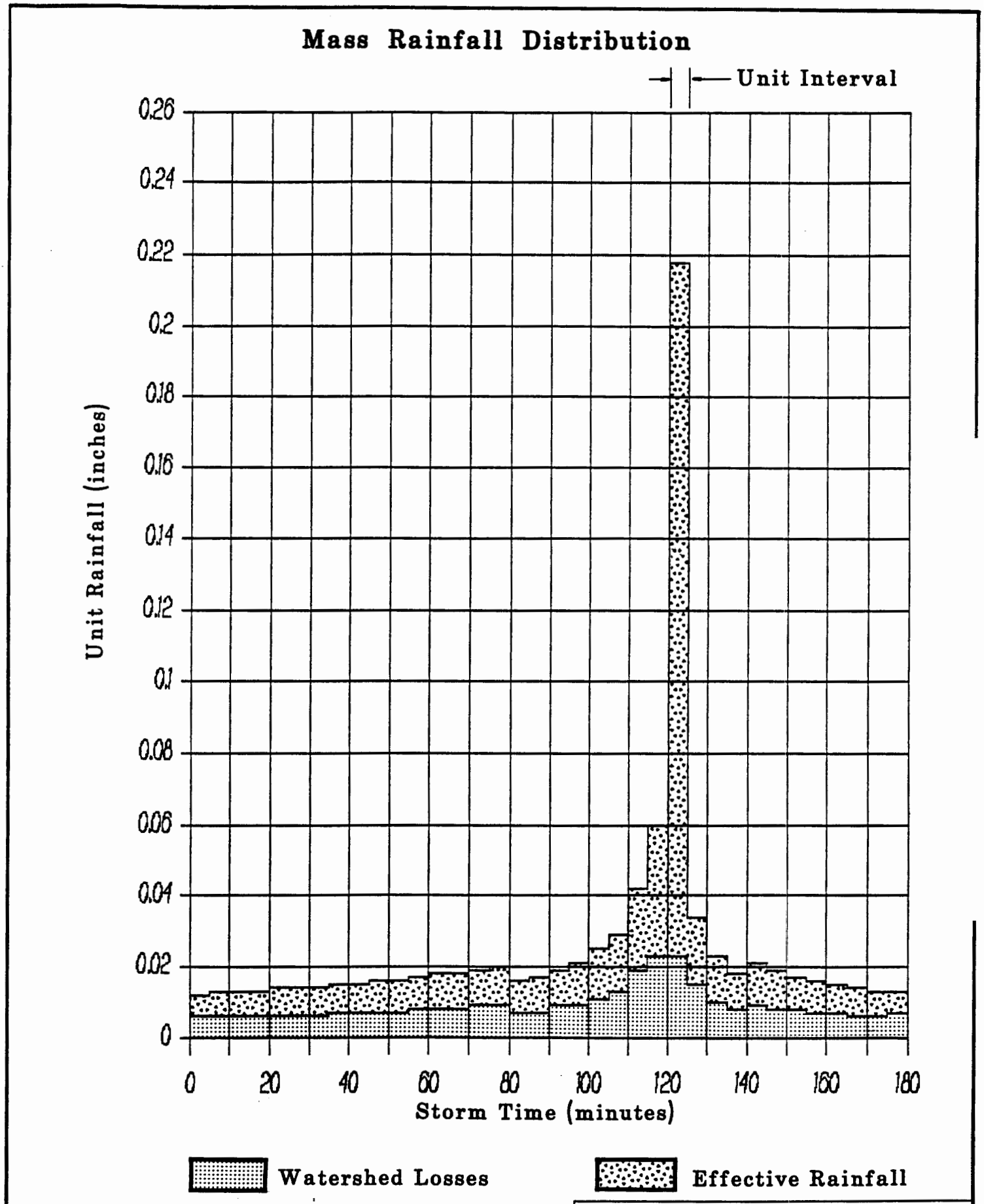
EFFECTIVE RAINFALL DETERMINATION

(Unit Period = 5 minutes)

Unit Period Number	Unit Rainfall (inches)	Unit Loss(inches)	Effective Rainfall (inches)
1	0.012	0.006	0.007
2	0.013	0.006	0.007
3	0.013	0.006	0.007
4	0.013	0.006	0.007
5	0.014	0.006	0.007
6	0.014	0.006	0.008
7	0.014	0.006	0.008
8	0.015	0.007	0.008
9	0.015	0.007	0.008
10	0.016	0.007	0.009
11	0.016	0.007	0.009
12	0.017	0.008	0.009
13	0.018	0.008	0.010
14	0.018	0.008	0.010
15	0.019	0.009	0.011
16	0.020	0.009	0.011
17	0.016	0.007	0.009
18	0.017	0.007	0.009
19	0.019	0.009	0.010
20	0.021	0.009	0.011

Unit Period Number	Unit Rainfall (inches)	Unit Loss(inches)	Effective Rainfall (inches)
21	0.025	0.011	0.014
22	0.029	0.013	0.016
23	0.042	0.019	0.023
24	0.060	0.023*	0.037
25	0.218	0.023*	0.195
26	0.034	0.015	0.019
27	0.023	0.010	0.012
28	0.018	0.008	0.010
29	0.021	0.009	0.012
30	0.019	0.008	0.010
31	0.017	0.008	0.009
32	0.016	0.007	0.009
33	0.015	0.007	0.008
34	0.014	0.006	0.008
35	0.013	0.006	0.007
36	0.016	0.007	0.009
	0.898	0.325	0.573

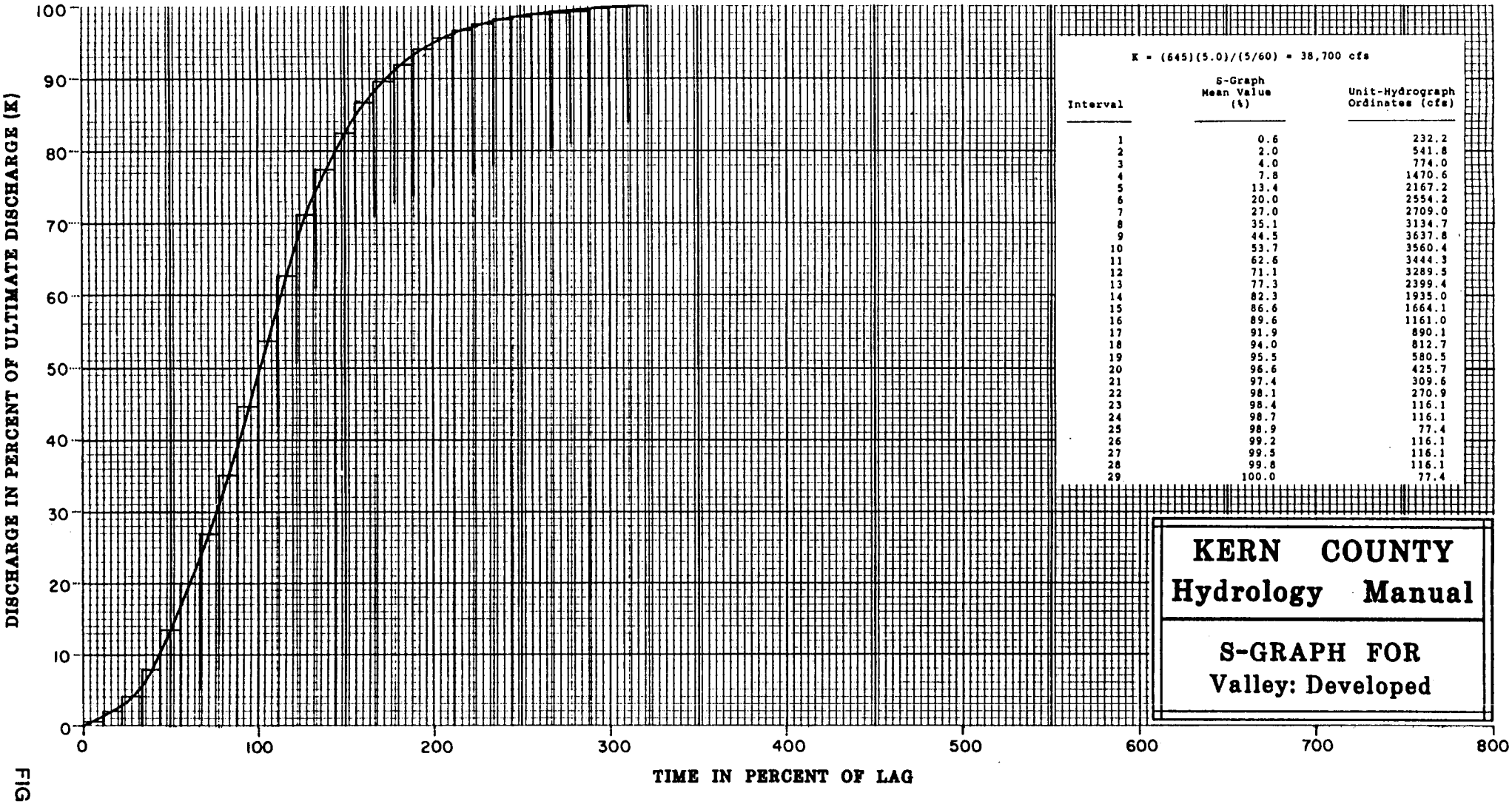
* unit low loss exceeds maximum loss rate, F_m



KERN COUNTY
Hydrology Manual

EXAMPLE
Synthetic 3-Hour Critical Storm

FIGURE E-17



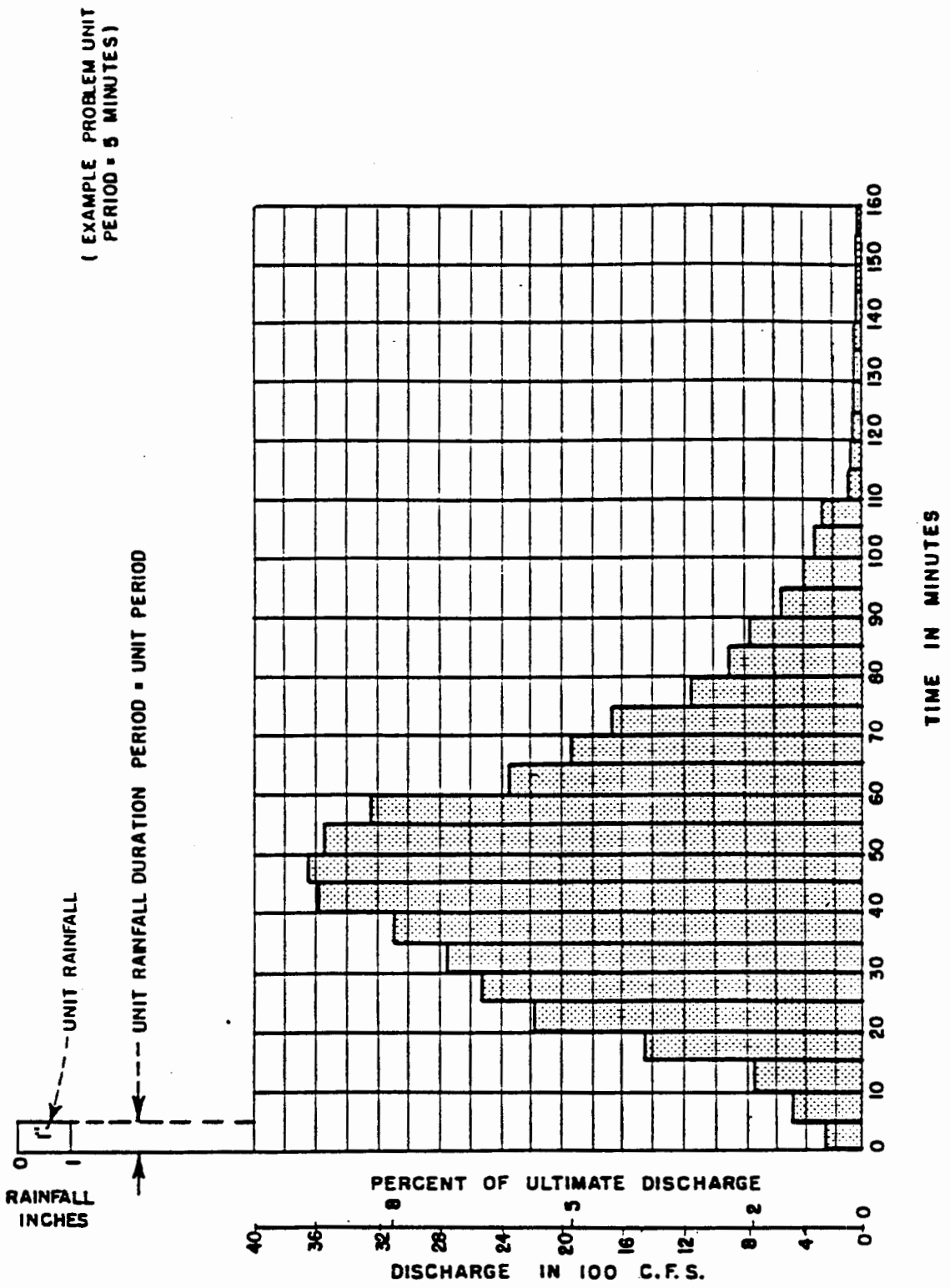
K = (645)(5.0)/(5/60) = 38,700 cfs

Interval	S-Graph Mean Value (%)	Unit-Hydrograph Ordinates (cfs)
1	0.6	232.2
2	2.0	541.8
3	4.0	774.0
4	7.8	1470.6
5	13.4	2167.2
6	20.0	2554.2
7	27.0	2709.0
8	35.1	3134.7
9	44.5	3637.8
10	53.7	3560.4
11	62.6	3444.3
12	71.1	3289.5
13	77.3	2399.4
14	82.3	1935.0
15	86.6	1664.1
16	89.6	1161.0
17	91.9	890.1
18	94.0	812.7
19	95.5	580.5
20	96.6	425.7
21	97.4	309.6
22	98.1	270.9
23	98.4	116.1
24	98.7	116.1
25	98.9	77.4
26	99.2	116.1
27	99.5	116.1
28	99.8	116.1
29	100.0	77.4

KERN COUNTY
Hydrology Manual

S-GRAPH FOR
Valley: Developed

FIGURE E-18



KERN COUNTY

HYDROLOGY MANUAL

EXAMPLE PROBLEM

UNIT DISTRIBUTION GRAPH

FIGURE E-19

KERN COUNTY

HYDROLOGY MANUAL

SYNTHETIC UNIT HYDROGRAPH METHOD

Flood Hydrograph Calculation Form

Project HYDROGRAPH EXAMPLE

By FPM Date 2-26-91

Checked CEF Date 2-26-91

Sheet 1 / 2

UNIT GRAPH	EFFEC RAIN UNIT PER.	.007	.007	.007	.007	.007	.008	.008	.008	.008	.009	.009	.009	.01	.01	.011	.011	.009	.009	.01	.011	.014	.016	.023	.037	FLOOD HYDRO-GRAPH
232.2	50	1.6																								
541.8	1	3.8	1.6																							
774.0		5.4	3.8	1.6																						
1470.6		10.3	5.4	3.8	1.6																					
2167.2		15.2	10.3	5.4	3.8	1.6																				
2554.2		17.9	15.2	10.3	5.4	3.8	1.9																			
2709.0		19.0	17.9	15.2	10.3	5.4	4.3	1.9																		
3134.7		21.9	19.0	17.9	15.2	10.3	6.2	4.3	1.9																	
3637.8		25.5	21.9	19.0	17.9	15.2	11.8	6.2	4.3	1.9																
3560.4		24.9	25.5	21.9	19.0	17.9	17.3	11.8	6.2	4.3	2.1															
3444.3		24.1	24.9	25.5	21.9	19.0	20.4	17.3	11.8	6.2	4.9	2.1														
3289.5		23.0	24.1	24.9	25.5	21.9	21.7	20.4	17.3	11.8	7.0	4.9	2.1													
2399.4		16.8	23.0	24.1	24.9	25.5	25.1	21.7	20.4	17.3	13.2	7.0	4.9	2.3												
1935.0		13.5	16.8	23.0	24.1	24.9	29.1	25.1	21.7	20.4	19.5	13.2	7.0	5.4	2.3											
1664.1		11.6	13.5	16.8	23.0	24.1	28.5	29.1	25.1	21.7	23.0	19.5	13.2	7.7	5.4	2.6										
1161.0		8.1	11.6	13.5	16.8	23.0	27.6	28.5	29.1	25.1	24.4	23.0	19.5	14.7	7.7	6.0	2.6									
890.1		6.2	8.1	11.6	13.5	16.8	26.3	27.6	28.5	29.1	28.2	24.4	23.0	21.7	14.7	8.5	6.0	2.1								
812.7		5.7	6.2	8.1	11.6	13.5	19.2	26.3	27.6	28.5	32.7	28.2	24.4	25.5	21.7	16.2	8.5	4.9	2.1							
580.5		4.1	5.7	6.2	8.1	11.6	15.5	19.2	26.3	27.6	32.0	32.7	28.2	27.1	25.5	23.8	16.2	7.0	4.9	2.3						
425.7		3.0	4.1	5.7	6.2	8.1	13.3	15.5	19.2	26.3	31.0	32.0	32.7	31.3	27.1	28.1	23.8	13.2	7.0	5.4	2.6					
309.6		2.2	3.0	4.1	5.7	6.2	9.3	13.3	15.5	19.2	29.6	31.0	32.0	36.4	31.3	29.8	28.1	19.5	13.2	7.7	6.0	3.3				
270.9		1.9	2.2	3.0	4.1	5.7	7.1	9.3	13.3	15.5	21.6	29.6	31.0	35.6	36.4	34.5	29.8	23.0	19.5	14.7	8.5	7.6	3.7			
116.1		0.8	1.9	2.2	3.0	4.1	6.5	7.1	9.3	13.3	17.4	21.6	29.6	34.4	35.6	40.0	34.5	24.4	23.0	21.7	16.2	10.8	8.7	5.3		
116.1		0.8	0.8	1.9	2.2	3.0	4.6	6.5	7.1	9.3	15.0	17.4	21.6	32.9	34.4	39.2	40.0	28.2	24.4	25.5	23.8	20.6	12.4	12.5	8.6	
77.4		0.5	0.8	0.8	1.9	2.2	3.4	4.6	6.5	7.1	10.4	15.0	17.4	24.0	32.9	37.9	39.2	32.7	28.2	27.1	28.1	30.3	23.5	17.8	20.0	
116.1		0.8	0.5	0.8	0.8	1.9	2.5	3.4	4.6	6.5	8.0	10.4	15.0	19.4	24.0	36.2	37.9	32.0	32.7	31.3	29.8	35.8	34.7	33.8	28.6	
116.1		0.8	0.8	0.5	0.8	0.8	2.2	2.5	3.4	4.6	7.3	8.0	10.4	16.6	19.4	26.4	36.2	31.0	32.0	36.4	34.5	37.9	40.9	49.8	54.4	
116.1		0.8	0.8	0.8	0.5	0.8	0.9	2.2	2.5	3.4	5.2	7.3	8.0	11.6	16.6	21.3	26.4	29.6	31.0	35.6	40.0	43.9	43.3	58.7	80.2	
77.4		0.5	0.8	0.8	0.8	0.5	0.9	0.9	2.2	2.5	3.8	5.2	7.3	8.9	11.6	18.3	21.3	21.6	29.6	34.4	39.2	50.9	50.2	62.3	94.5	
			0.5	0.8	0.8	0.8	0.6	0.9	0.9	2.2	2.8	3.8	5.2	8.1	8.9	12.8	18.3	17.4	21.6	32.9	37.9	49.8	58.2	72.1	100.2	
				0.5	0.8	0.8	0.9	0.6	0.9	0.9	2.4	2.8	3.8	5.8	8.1	9.8	12.8	15.0	17.4	24.0	36.2	48.2	57.0	83.7	116.0	
					0.5	0.8	0.9	0.9	0.6	0.9	1.0	2.4	2.8	4.3	5.8	8.9	9.8	10.4	15.0	19.4	26.4	46.1	55.1	81.9	134.6	
						0.5	0.9	0.9	0.9	0.6	1.0	1.0	2.4	3.1	4.3	6.4	8.9	8.0	10.4	16.6	21.3	33.6	52.6	79.2	131.7	
							0.6	0.9	0.9	0.9	0.7	1.0	1.0	2.7	3.1	4.7	6.4	7.3	8.0	11.6	18.3	27.1	38.4	75.7	127.4	
								0.6	0.9	0.9	1.0	0.7	1.0	1.2	2.7	3.4	4.7	5.2	7.3	8.9	12.8	23.3	31.0	55.2	121.7	
									0.6	0.9	1.0	1.0	0.7	1.2	1.2	3.0	3.4	3.8	5.2	8.1	9.8	16.3	26.6	44.5	88.8	
										0.6	1.0	1.0	1.0	0.8	1.2	1.3	3.0	2.8	3.8	5.8	8.9	12.5	18.6	38.3	71.6	
											0.7	1.0	1.0	1.2	0.8	1.3	1.3	2.4	2.8	4.3	6.4	11.4	14.2	26.7	61.6	
												0.7	1.0	1.2	1.2	0.9	1.3	1.0	2.4	3.1	4.7	8.1	13.0	20.5	43.0	
													0.7	1.2	1.2	1.3	0.9	1.0	1.0	2.7	3.4	6.0	9.3	18.7	32.9	
														0.8	1.2	1.3	1.3	0.7	1.0	1.2	3.0	4.3	6.8	13.4	30.1	
															0.8	1.3	1.3	1.0	0.7	1.2	1.3	3.8	5.0	9.8	21.5	
																0.9	1.3	1.0	1.0	0.8	1.3	1.6	4.3	7.1	15.8	
																	0.9	1.0	1.0	1.2	0.9	1.6	1.9	6.2	11.5	
																		0.7	1.0	1.2	1.3	1.1	1.9	2.7	10.0	
																			0.7	1.2	1.3	1.6	1.2	2.7	4.3	
																				0.8	1.3	1.6	1.9	1.8	4.3	
																					0.9	1.6	1.9	2.7	2.9	
																						1.1	1.9	2.7	4.3	
																							1.2	2.7	4.3	
																								1.8	4.3	
																									2.9	

50

FIGURE E-20(a)

KERN COUNTY

HYDROLOGY MANUAL

SYNTHETIC UNIT HYDROGRAPH METHOD

Flood Hydrograph Calculation Form

Project HYDROGRAPH EXAMPLE

By FPM Date 2-26-91

Checked CEF Date 2-26-91

Sheet 2 / 2

X	EFFEC RAIN																										
UNIT GRAPH	UNIT PER.	.195	.019	.012	.01	.012	.01	.009	.009	.008	.008	.007	.009														FLOOD HYDRO GRAPH
																											51.6
																											55.4
																											60.8
																											71.1
																											86.3
																											104.4
																											123.9
																											146.6
																											173.6
																											200.9
																											228.1
																											254.6
																											276.2
																											296.1
																											314.9
																											331.2
																											346.3
																											361.0
																											374.1
																											385.7
																											396.4
																											407.5
																											421.4
																											442.7
		45.3																									507.8
		105.7	4.4																								591.6
		150.9	10.3	2.8																							671.8
		286.8	14.7	6.5	2.3																						831.9
		422.6	27.9	9.3	5.4	2.8																					987.2
		498.1	41.2	17.6	7.7	6.5	2.3																				1081.2
		580.3	48.5	26.0	14.7	9.3	5.4	2.1																			1132.8
		611.3	51.5	30.7	21.7	17.6	7.7	4.9	2.1																		1226.1
		709.4	59.6	32.5	25.5	26.0	14.7	7.0	4.9	1.9																	1316.1
		694.3	69.1	37.6	27.1	30.7	21.7	13.2	7.0	4.3	1.9																1293.7
		671.6	67.6	43.7	31.3	32.5	25.5	19.5	13.2	6.2	4.3	1.6															1249.8
		641.5	66.4	42.7	36.4	37.6	27.1	23.0	19.5	11.8	6.2	3.8	2.1														1183.2
		467.9	62.5	41.3	35.6	43.7	31.3	24.4	23.0	17.3	11.8	5.4	4.9														991.3
		377.3	45.6	39.5	34.4	42.7	36.4	28.2	24.4	20.4	17.3	10.3	7.0														870.6
		324.5	36.8	28.8	32.9	41.3	35.6	32.7	28.2	21.7	20.4	15.2	13.2														783.4
		226.4	31.6	23.2	24.0	39.5	34.4	32.0	32.7	25.1	21.7	17.9	19.5														658.3
		173.6	22.1	20.0	19.4	28.8	32.9	31.0	32.0	29.1	25.1	19.0	23.0														570.7
		158.5	16.9	13.9	16.6	23.2	24.0	29.6	31.0	28.5	29.1	21.9	24.4														515.2
		113.2	15.4	10.7	11.6	20.0	19.4	21.6	29.6	27.6	28.5	25.5	28.2														436.3
		83.0	11.0	9.8	8.9	13.9	16.6	17.4	21.6	26.3	27.6	24.9	32.7														369.9
		60.4	8.1	7.0	8.1	10.7	11.6	15.0	17.4	19.2	26.3	24.1	32.0														309.7
		52.8	5.9	5.1	5.8	9.8	8.9	10.4	15.0	15.5	19.2	23.0	31.0														265.4
		22.6	5.1	3.7	4.3	7.0	8.1	8.0	10.4	13.3	15.5	16.8	29.6														206.1
		22.6	2.2	3.3	3.1	5.1	5.8	7.3	8.0	9.3	13.3	13.5	21.6														175.0
		15.1	2.2	1.4	2.7	3.7	4.3	5.2	7.3	7.1	9.3	11.6	17.4														147.3
		22.6	1.5	1.4	1.2	3.3	3.1	3.8	5.2	6.5	7.1	8.1	15.0														137.0
		22.6	2.2	0.9	1.2	1.4	2.7	2.8	3.8	4.6	6.5	6.2	10.4														121.6
		22.6	2.2	1.4	0.8	1.4	1.2	2.4	2.8	3.4	4.6	5.7	8.0														109.4
		15.1	2.2	1.4	1.2	0.9	1.2	1.0	2.4	2.5	3.4	4.1	7.3														92.7
			1.5	1.4	1.2	1.4	0.8	1.0	1.0	2.2	2.5	3.0	5.2														71.1
				0.9	1.2	1.4	1.2	0.7	1.0	0.9	2.2	2.2	3.8														65.6
					0.8	1.4	1.2	1.0	0.7	0.9	0.9	1.9	2.8														61.6
						0.9	1.2	1.0	1.0	0.6	0.9	0.8	2.4														59.0
							0.8	1.0	1.0	0.9	0.6	0.8	1.0														56.3
								0.7	1.0	0.9	0.9	0.5	1.0														55.2
									0.7	0.9	0.9	0.8	0.7														54.1
										0.6	0.9	0.8	1.0														53.4
											0.6	0.8	1.0														52.5
												0.5	1.0														51.6
													0.7														50.7

FIGURE E-20(b)

EXAMPLE:

Hydrograph

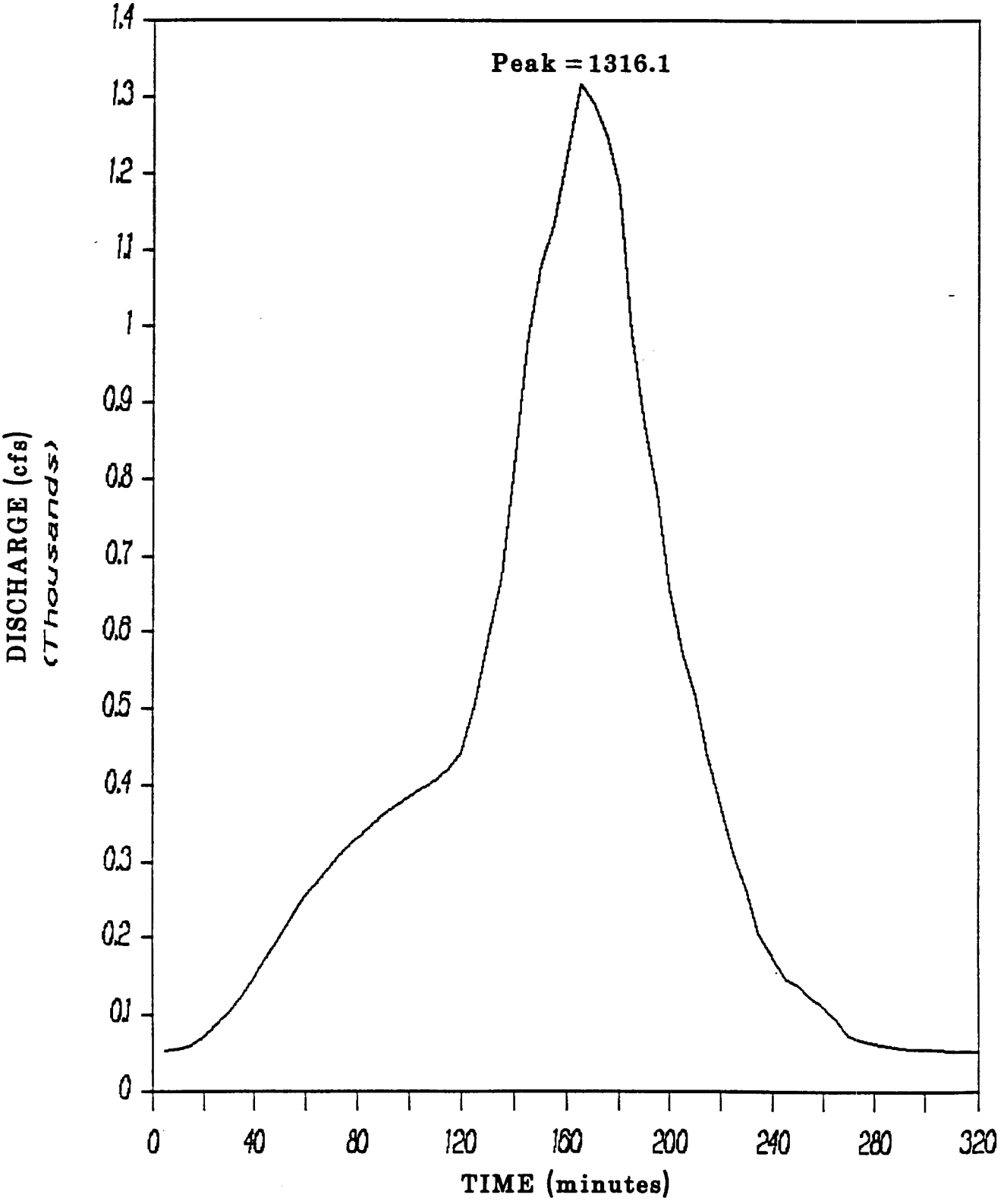


FIGURE E-21

Section E

Equations

List of Equations:

Equation No.	Equation		
E.1	Lag	=	0.8Tc
E.2	Lag(hours)	=	$C_t((L \cdot L_{CA})/S^{0.5})^m$
E.3	K(cfs)	=	645 • A/T
E.4	$\overline{Y} = 1 - Y$		
E.5	$F* = \overline{Y} \cdot I$		
E.6	$F_m = a_p F_p$		

Section B
Related Equations

List of Equations:

Equation No.	Equation		
B.1	P = 1/T		
B.2	Y2	=	0.111+0.545 (X1)(X1/X2)
B.3	Y100	=	0.221+0.885 (Y2)(X3/X1)
B.4	**or** Y100	=	0.221+0.098 (X3/X1)+0.482 (X3)(X1/X2)
B.5	Y2	=	0.107+0.315 (X1)
B.6	Y100	=	-0.391+1.224 (Y2)(X3/X1)+0.043 (X5)
B.7	**or** Y100	=	-0.391+0.131 (X3/X1)+0.386(X3) +0.043 (X5)
B.8	Y2	=	0.126+0.561 (X1)(X1/X2)
B.9	Y100	=	-0.030+0.816 (Y2)(X3/X1)+0.063 (X5)
B.10	**or** Y100	=	-0.030+0.103 (X3/X1)+0.458 (X3)(X1/X2) +0.063 (X5)
B.11	Y2	=	0.005+0.852 (X1)(X1/X2)
B.12	Y100	=	0.322+0.789 (X3)(X3/X4)
C.1	Ia	=	0.2S
C.2	$S = \frac{1000}{CN} - 10$		
C.3	$Y_j = \frac{(P_{24} - Ia)^2}{(P_{24} - Ia + S)P_{24}}$		
C.4	$Y = \frac{Y_1 A_1 + \ldots + Y_m A_m}{A_1 + A_2 + \ldots + A_m}$		

SECTION F

FLOW-THROUGH BASIN ANALYSIS

OVERVIEW

This section will cover the design of flow through basins (detention basins). It will discuss:

- Detention basin routing procedures
- Extended design storm criterion
- Required format

Section E includes a detailed discussion of the unit hydrograph.

Section J provides details of the development of small area hydrographs.

Appendix I discusses the environmental considerations in using detention basins.

For other important information pertinent to this section refer to Table B.1.

F.1. DEFINITIONS

Critical Storm: the design storm pattern under which a detention basin system demonstrates no increase in the required basin volume due to the further extension by a successive day storm.

Detention Basin: a stormwater facility intended to reduce peak flow rates of runoff generated from an upstream watershed and to control peak flows into a downstream area.

F.2. INTRODUCTION

F.2.1 Reservoir and Stream Flow Routing

There are two types of routing that are important in watershed planning; reservoir routing and streamflow routing. Both processes operate upon an inflow hydrograph to produce an outflow hydrograph. In this section, reservoir routing will be presented based on the modified Pul's method. Section H will present the convex method for streamflow routing. It is noted that the County has hydraulic design criteria which must be considered in addition to the hydrologic criteria established in this manual.

F.2.2 Unit Hydrograph Approach

Section E of this hydrology manual includes a detailed discussion of the unit hydrograph approach to be used for hydrologic studies of watersheds. The approach develops a 24-hour duration ***synthetic critical storm pattern*** which is composed of peak rainfall intensities for a specified return frequency (e.g., peak 5-minutes, 30-minutes,

1-hour, 3-hour, 6-hour and 24-hour) nested together with the storm peak 5-minutes of rainfall defined to occur at storm time of hour 16.

Use of this synthetic storm pattern subjects a study watershed to all durations of storm events (of a given rainfall return frequency) up to 24-hours. That is, the 24-hour critical storm pattern for floodwater detention basin studies will include all durations of critical storm events up to 24-hours. Therefore, in many cases only one design storm is necessary for analysis of the detention basin flood routing.

F.3. DETENTION BASIN ANALYSIS

F.3.1. Detention Basin Routing Procedure

The modified Pul's (refs. 2, 3, 5) method may be used for ***detention basin*** routing studies. The basin routing relationships are based upon the formula

$$I - O = \frac{dS}{dt} \tag{F.1}$$

where

I = basin inflow rate (cfs)

O = basin outflow rate (cfs)

dS = change in basin storage during the time step (cubic feet)

dt = time step (sec)

Equation (F.1) is approximated by replacing the variables I and O by an average value during the timestep using

$$I = \frac{I_1 + I_2}{2} \tag{F.2}$$

$$O = \frac{O_1 + O_2}{2} \tag{F.3}$$

$$dS = S_2 - S_1 \tag{F.4}$$

where the subscript 1 indicates the beginning of a time period and subscript 2 indicates the end of the subject time period. Substituting (F.2), (F.3), and (F.4) into the basin routing equation of (F.1) and rearranging terms gives

$$(S_2 + O_2 \cdot \frac{dt}{2}) = (S_1 - O_1 \cdot \frac{dt}{2}) + (I_1 + I_2) \cdot \frac{dt}{2} \quad \text{(F.5)}$$

In (F.5), the right side is known from the previously computed values of storage, S_1, outflow, O_1, and the average basin inflow $(I_1+I_2)/2$ for time step dt.

The solution of the basin routing problem requires the following information:

1. Known initial conditions for basin storage and outflow
2. A routing timestep, dt
3. The basin inflow hydrograph
4. Basin volume versus depth and outflow versus depth relationships

To solve (F.5), a storage indication curve should be developed. Such a curve may be derived from the known storage-discharge relationship by selecting various values of depth and computing (S + O*dt/2) from the associated values for storage and outflow. This quantity is plotted versus outflow, as shown in Figure F-1.

The solution procedure then proceeds with the following steps:

1. Determine the average inflow volume from the inflow hydrograph during the timestep dt; i.e., calculate $(I_1 + I_2) \cdot dt/2$.
2. Compute $S_1 - O_1 \cdot dt/2$ from the assumed initial condition of the basin flow depth or the last computed values of S and O.
3. Use (F.5) to compute $(S_2 + O_2\, dt/2)$.
4. Use the estimate from Step 3 and the storage indication curve (see Figure F-8) to compute O_2.
5. Use O_2 and the known storage versus depth and outflow versus depth relationships to compute S_2.

These five steps are repeated for the next timestep using I_2, O_2, and S_2 as the new values of I_1, O_1, and S_1, respectively. This procedure is repeated until the basin inflow hydrograph has been completely analyzed and basin outflow becomes negligible.

F.4. EXTENDED DESIGN STORM CRITERIA

Due to the interaction of watershed size, Tc, percentage of peak discharge reduction, and basin volume, the critical storm duration is generally not known (in advance) for a watershed flood control system which includes one or several detention basins. Hence, the use of the 24-hour design storm may not be the "***critical***" storm for flow-through detention basin design purposes, and a longer duration design storm may be needed. Figure F-2 illustrates the extended design storm (multiday) for a two-day duration. Longer duration design storms are developed in a similar fashion.

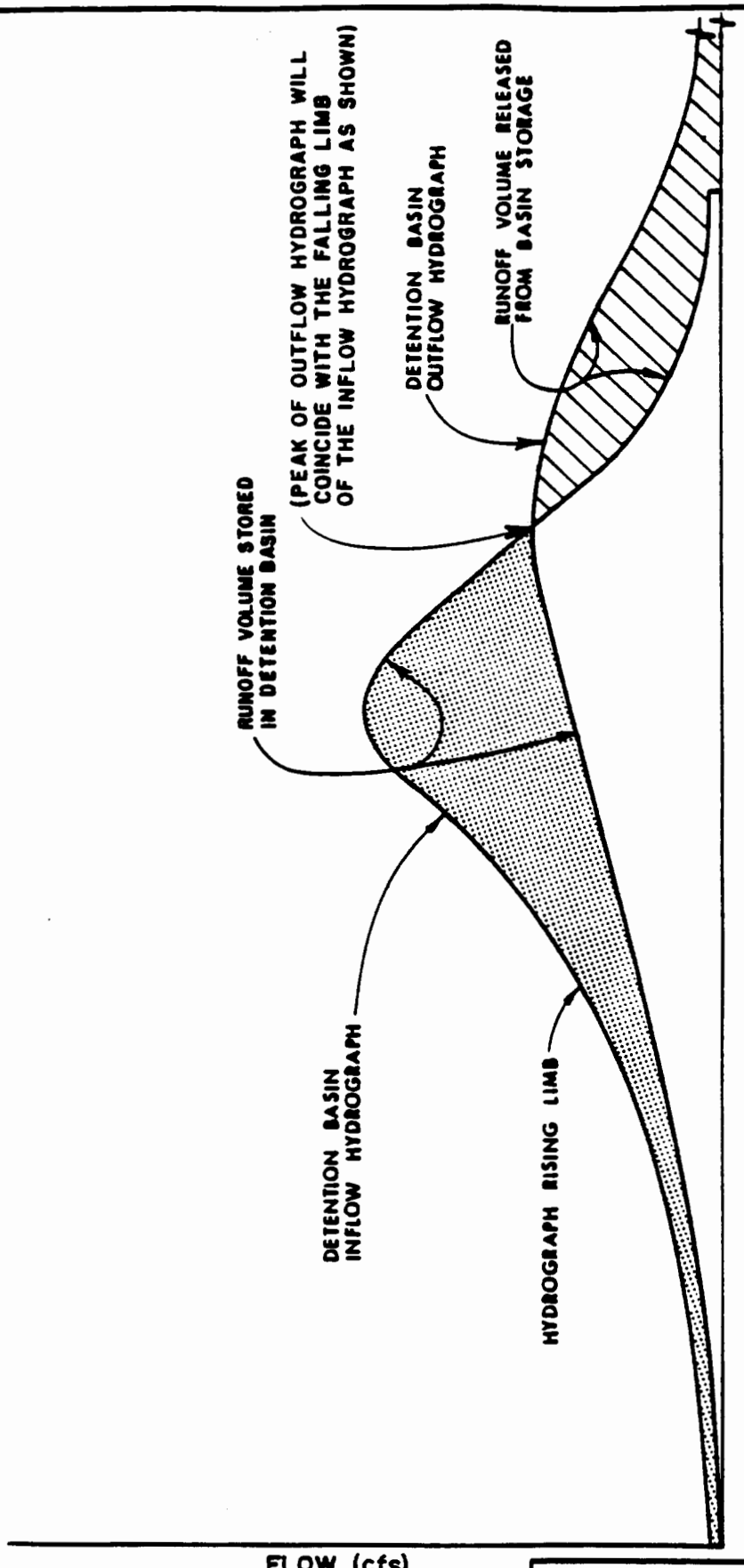
The ***multiday design storm*** utilizes the structure of Figure F-2 for all flow-through detention basin systems. Successive day storms are developed and added in the front of the previously developed design storm patterns until the detention basin system demonstrates no increase in the required basin volume due to the further extension of the design storm pattern. By increasing the basin outlet capacity, the critical duration can be reduced.

From Figure F-2, it is seen that the extended design storm is constructed from an arrangement of rainfalls of identical T-year return frequency. That is, even though a two day or longer duration multiday storm is being used to test the detention basin's level of flood protection, the extended design storm still contains no more than T-year rainfall depths for the extended duration. Each of the 24-hour storm patterns are constructed by a simple scaling of the peak 24-hour design pattern according to a ratio of the respective 24-hour precipitation values, see Table B.1.

F.5. REQUIRED FORMAT

Figure F-3 illustrates the required format for submitting detention basin study results for review.

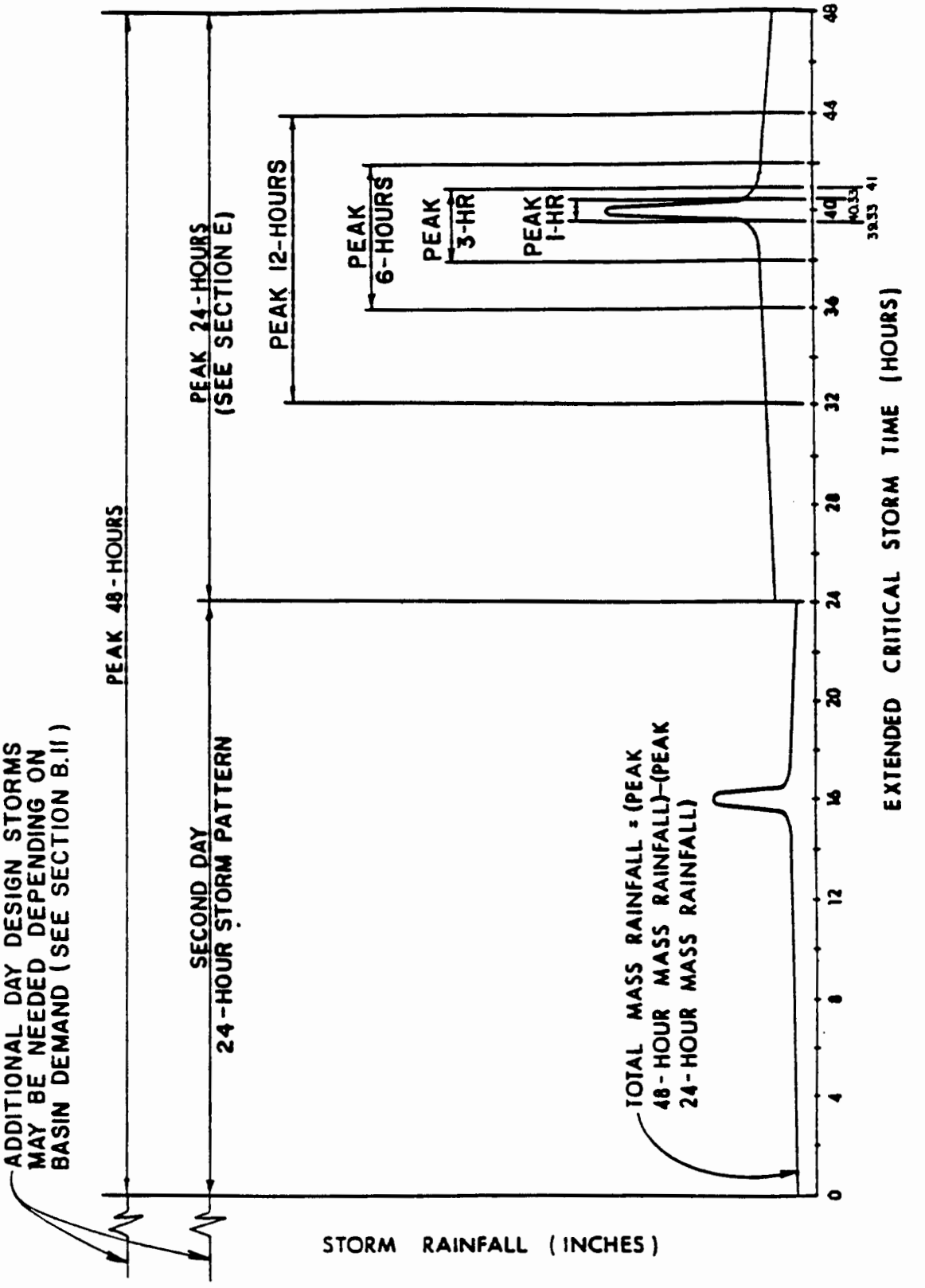
Figure F-4 is to be used to supply the necessary detention basin information to determine the routing results.



KERN COUNTY
HYDROLOGY MANUAL

DETENTION BASIN INFLOW AND OUTFLOW HYDROGRAPHS

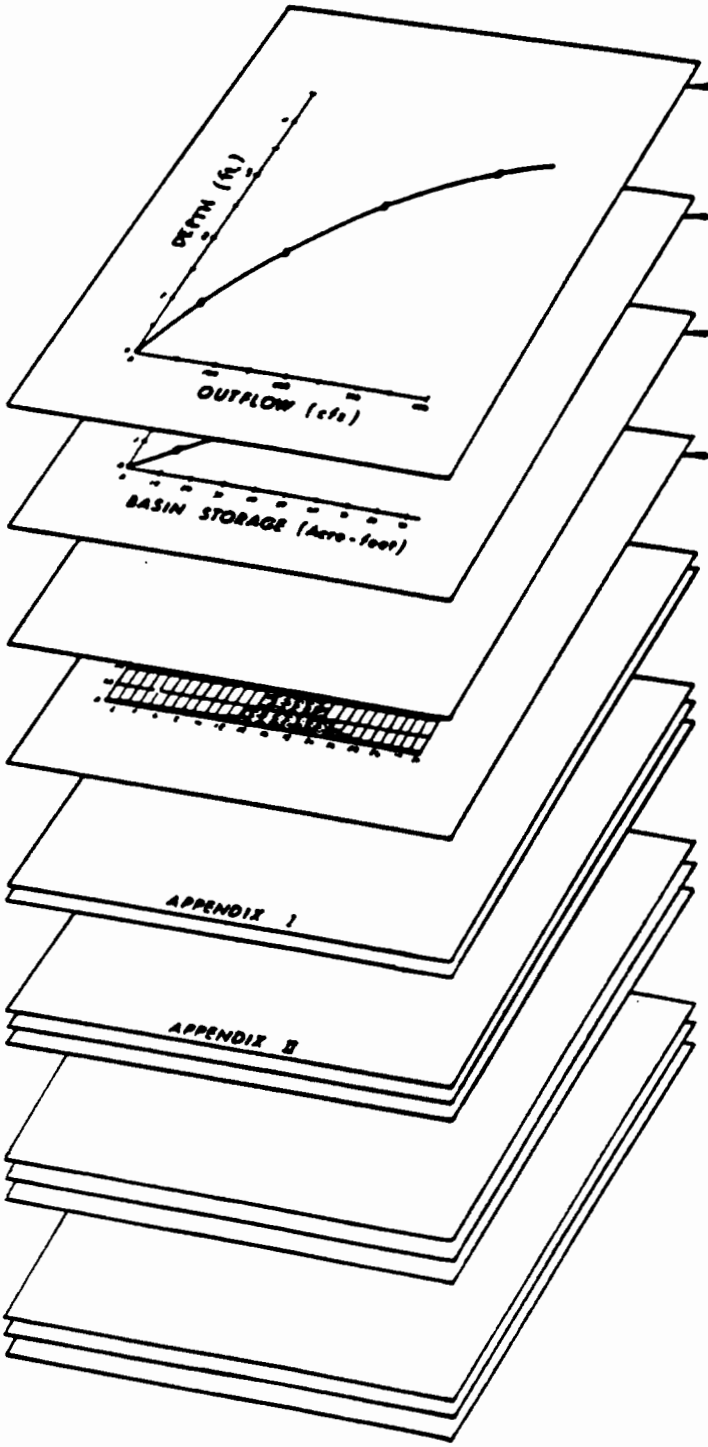
FIGURE F-1



KERN COUNTY
HYDROLOGY MANUAL

EXTENDED DESIGN STORM: FLOW-THROUGH BASIN

FIGURE F-2



DETENTION BASIN OUTFLOW (c.f.s.) AS A FUNCTION OF DEPTH (feet)

DETENTION BASIN STORAGE (Acre - feet) AS A FUNCTION OF DEPTH (feet)

DETENTION BASIN INFORMATION FORM (Fig. F - 8)

DETENTION BASIN INFLOW / OUTFLOW HYDROGRAPHS

UNIT - HYDROGRAPH STUDY (SEE SECTION E FORMAT REQUIREMENTS.)

DETENTION BASIN ROUTING CALCULATIONS (MODIFIED PULS METHOD)

ONE DAY

TWO DAY

ETC.

(UNTIL ADDITIONAL BASIN VOLUME IS NOT REQUIRED)

KERN COUNTY
HYDROLOGY MANUAL

REQUIRED REPORT FORMAT
FOR
DETENTION BASIN STUDY

FIGURE F-3

PROJECT: _____ DATE: _____

ENGINEER: _____

1. Enter the hydrograph unit interval duration **(minutes)** _____

2. Enter total number of basin depth-versus-outflow values **(maximum of 20)** _____

3. Enter basin outflow (cfs) and storage volume (AF) for each basin depth value in the following table. Enter values in order of increasing outflow basin depth:

Entry No.	Water Surface Elevation (FT)	Basin Depth (FT)	Basin Storage (AF)	Basin Outflow (CFS)
1	_____	0.0 (defined)	_____	0.0 (defined)
2	_____	_____	_____	_____
3	_____	_____	_____	_____
4	_____	_____	_____	_____
5	_____	_____	_____	_____
6	_____	_____	_____	_____
7	_____	_____	_____	_____
8	_____	_____	_____	_____
9	_____	_____	_____	_____
10	_____	_____	_____	_____
11	_____	_____	_____	_____
12	_____	_____	_____	_____
13	_____	_____	_____	_____
14	_____	_____	_____	_____
15	_____	_____	_____	_____
16	_____	_____	_____	_____
17	_____	_____	_____	_____
18	_____	_____	_____	_____
19	_____	_____	_____	_____
20	_____	_____	_____	_____

4. Enter assumed initial depth (feet) of water in detention basin _____

KERN COUNTY

HYDROLOGY MANUAL

DETENTION BASIN INFORMATION FORM

FIGURE F-4

F.6. Example: Detention Basin Hydrograph Routing

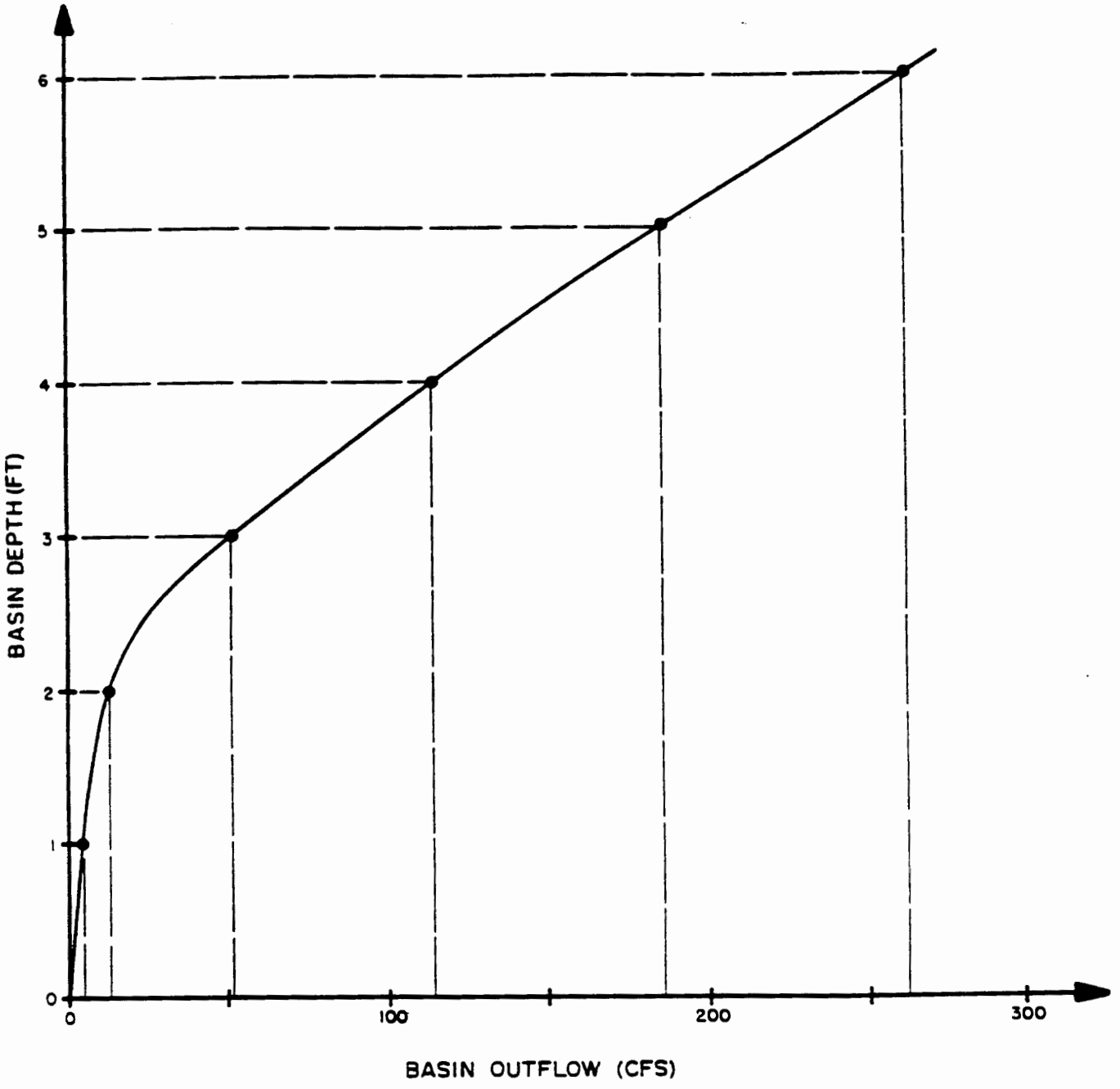
The assumed detention basin depth versus outflow and depth versus storage relationships are shown in Figures F-5 and F-6, respectively. The detention basin information sheet (Figure F-4) is completed in Figure F-7. Using a timestep of 60 minutes (3,600 seconds), the associated storage-indication curve is developed in the following table and plotted in Figure F-8.

TABLE F.1.

EXAMPLE PROBLEM: STORAGE-INDICATION CURVE DEVELOPMENT

Depth	O	S	S-O t/2	S+O t/2
(ft)	(cfs)	(AF)	(AF)	(AF)
0	0	0	0	0
1	4.2	14.4	14.22	14.57
2	12	28.8	28.3	29.3
3	51.7	43.2	41.06	45.34
4	114.7	57.6	52.86	62.34
5	186.8	72	64.28	79.72
6	263.2	86.4	75.52	97.28

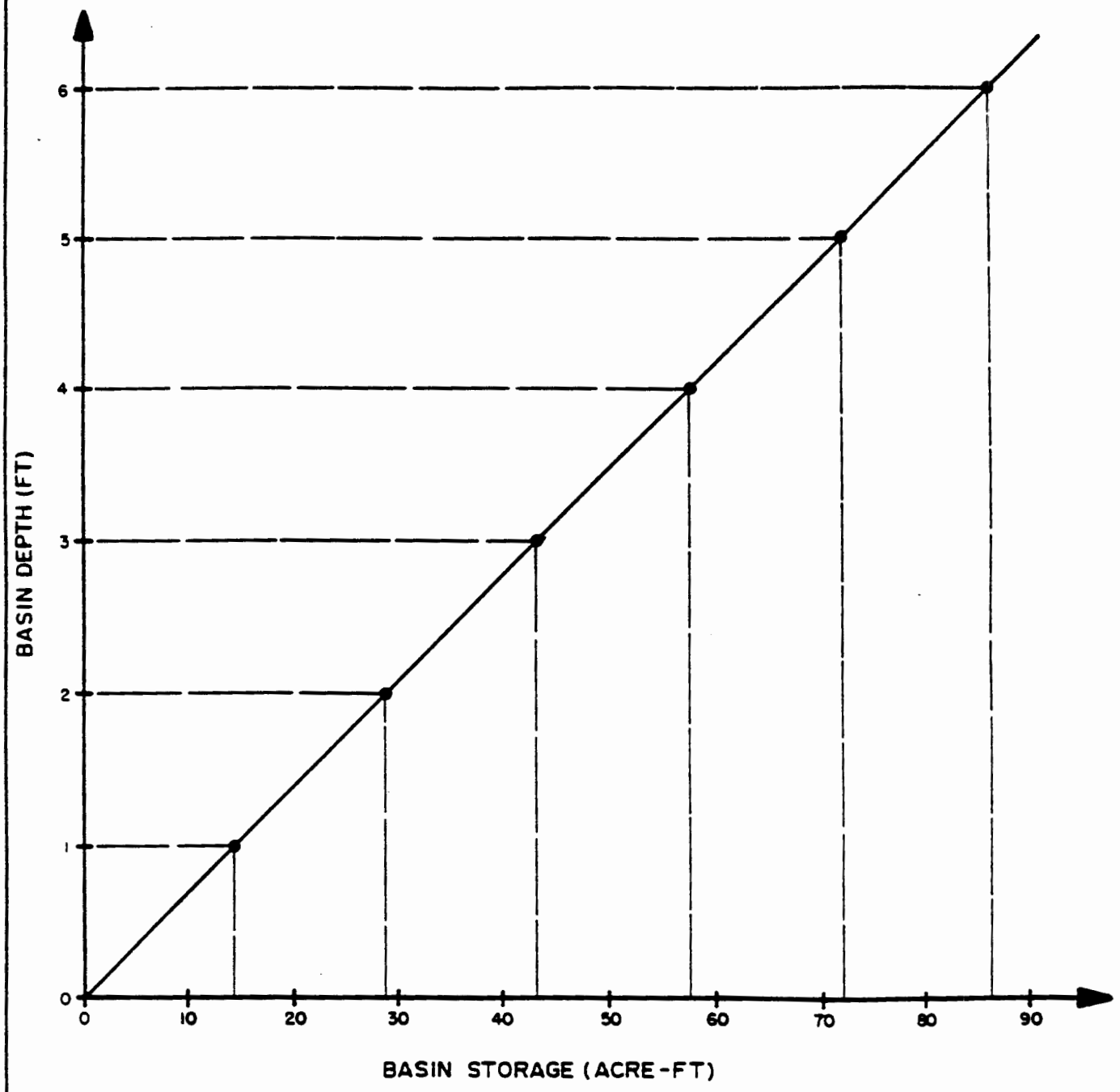
Assuming an initial condition of zero basin outflow and storage, an example basin inflow hydrograph (unit period of 60 minutes) is routed using the modified Pul's method in the tabulation of Table F.2. The 60-minute timestep is used for demonstration purposes only. Typically, a 5-minute timestep is needed in order to adequately describe the runoff hydrograph peak flow rates. Important features of a routed detention basin hydrograph are shown in Figure F-1.



KERN COUNTY
HYDROLOGY MANUAL

EXAMPLE PROBLEM
DETENTION BASIN OUTFLOW (CFS)
AS A FUNCTION OF DEPTH (FT)

FIGURE F-5



KERN COUNTY
HYDROLOGY MANUAL

EXAMPLE PROBLEM
DETENTION BASIN STORAGE (AF)
AS A FUNCTION OF DEPTH (FT)

FIGURE F-6

PROJECT: EXAMPLE PROBLEM DATE:

ENGINEER: A.E.S.

1. Enter the hydrograph unit interval duration **(minutes)** 60

2. Enter total number of basin depth-versus-outflow values **(maximum of 20)** 7

3. Enter basin outflow (cfs) and storage volume (AF) for each basin depth value in the following table. Enter values in order of increasing outflow basin depth:

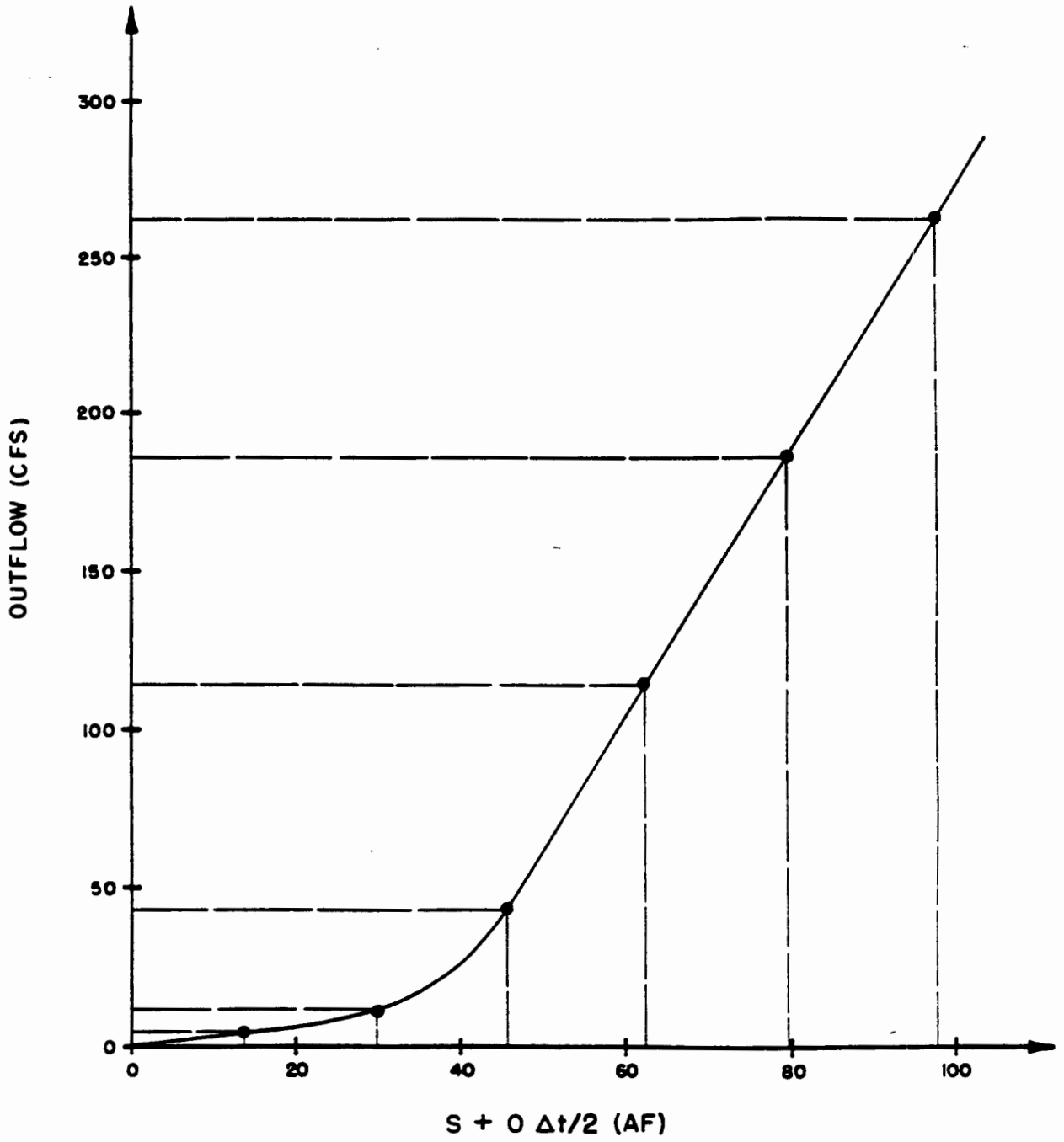
Entry No.	Water Surface Elevation (FT)	Basin Depth (FT)	Basin Storage (AF)	Basin Outflow (CFS)
1	100	0.0 (defined)	0	0.0 (defined)
2	101	1	14.4	4.2
3	102	2	28.8	12.0
4	103	3	43.2	51.7
5	104	4	57.6	114.7
6	105	5	72.0	186.8
7	106	6	86.4	263.2
8				
9				
10				
11				
12				
13				
14				
15				
16				
17				
18				
19				
20				

4. Enter assumed initial depth (feet) of water in detention basin 0

KERN COUNTY
HYDROLOGY MANUAL

DETENTION BASIN
INFORMATION FORM

FIGURE F-7



KERN COUNTY

HYDROLOGY MANUAL

EXAMPLE PROBLEM

STORAGE-INDICATION CURVE

FIGURE F-8

TABLE F.2.

EXAMPLE PROBLEM BASIN ROUTING TABULATION

Time (min.)	Inflow (cfs)	Average Inflow (cfs)	$(I_1+I_2)\ t/2$ (AF)	$S_1-O_1\ t/2$ (AF)	$S_2+O_2\ t/2$ (AF)	Outflow (cfs)	Storage (AF)
0	0					0	0
		30	2.48	0	2.48		
60	60					0.7	2.45
		90	7.44	2.42	9.86		
120	120					2.8	9.74
		200	16.53	9.62	26.16		
180	280					10.3	25.73
		265	21.9	25.31	47.21		
240	250					58.6	44.79
		235	19.42	42.37	61.79		
300	220					112.7	57.14
		170	14.05	52.48	66.53		
360	120					132.1	61.07
		110	9.09	55.61	64.7		
420	100					124.5	59.56
		80	6.61	54.41	61.02		
480	60					109.8	56.48
		30	2.48	51.94	54.42		
540	0					85.4	50.89
		0	0	47.36	47.36		
600	0					59.2	44.91
		0	0	42.46	42.46		

Section F
Equations

List of Equations

Equation No.	Equation
F.1	$I - O = S/dT$
F.2	$I = (I_1 + I_2)/2$
F.3	$O = (O_1 + O_2)/2$
F.4	$dS = S_2 - S_1$
F.5	$S_2 + O_2 \cdot \frac{dT}{2} = (S_1 - O_1 \cdot \frac{dT}{2}) \cdot (I_1 + I_2) \cdot \frac{dT}{2}$

SECTION G

FLOW-BY BASIN ANALYSIS (HYDROGRAPH SEPARATION)

OVERVIEW

This section will cover the design consideration required for the design of flow-by basins. It will discuss:

- Design storm criteria
- Volume analysis

Section E includes a detailed discussion of the unit hydrograph. Section J provides details of the development of small area hydrographs. Appendix I discusses the environmental considerations in using detention basins.

G.1. DEFINITIONS

Critical Storm: that multi-day storm which will guarantee that the basin will have adequate storage capacity remaining when the peak 24-hour storm occurs.

G.2. INTRODUCTION

This process models the effect of a channel flowing by a detention basin or structure that intercepts and diverts away from the channel all runoff flows in excess of some specified flow rate. Although simple in concept, use of a flow-by basin in a flood control system can provide a useful reduction in the runoff hydrograph peak flow rate. Figure G-1 shows the main elements of a typical flow-by basin. For a discussion on the considerations in using detention basins, see the Appendix I.

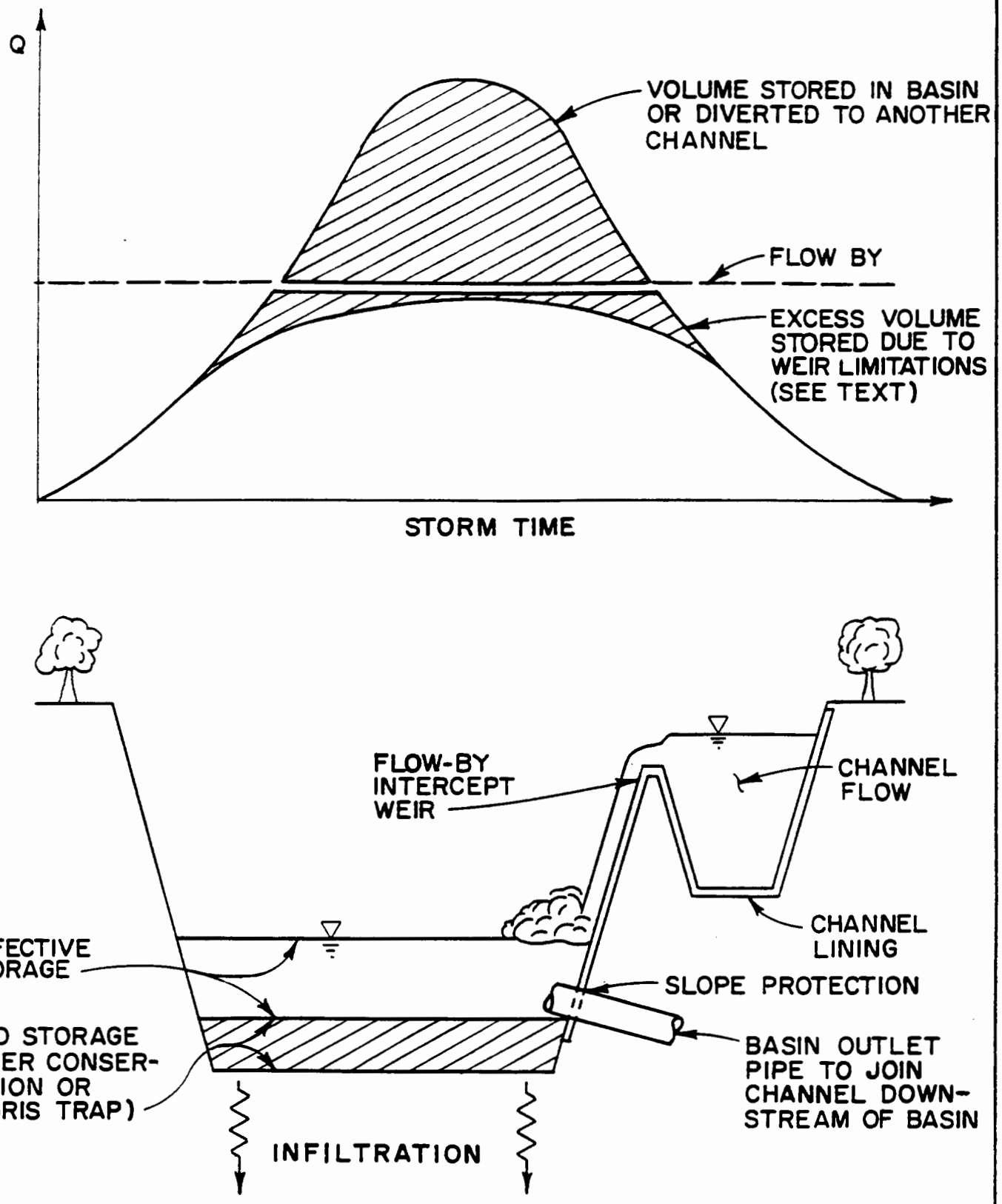
G.3. CRITICAL STORM CRITERIA

A **multiple-day storm** may be required to guarantee that the basin has an adequate storage capacity remaining when a peak 24-hour storm event occurs. For a multiple-day storm condition involving a flow-by basin system, the peak rainfall intensities of the selected T-year return frequency should be incorporated within each 24-hour duration (up to the appropriate mass rainfall volume with the desired return frequency) such as is shown in Figure G-2.

G.4. FLOW-BY BASIN VOLUME ANALYSIS: WEIR STRUCTURE EFFICIENCY

Figure G-1 illustrates a typical weir structure flow-by basin design. Due to the finite weir length, the actual flow-by discharge (i.e., no overflow into the basin) is less than the desired flow-by discharge and, consequently, the actual basin storage requirements are higher than idealized by a simple separation of the hydrograph.

Generally, this volume excess ranges between 20 and 50 percent, depending on the weir length used for the overflow.



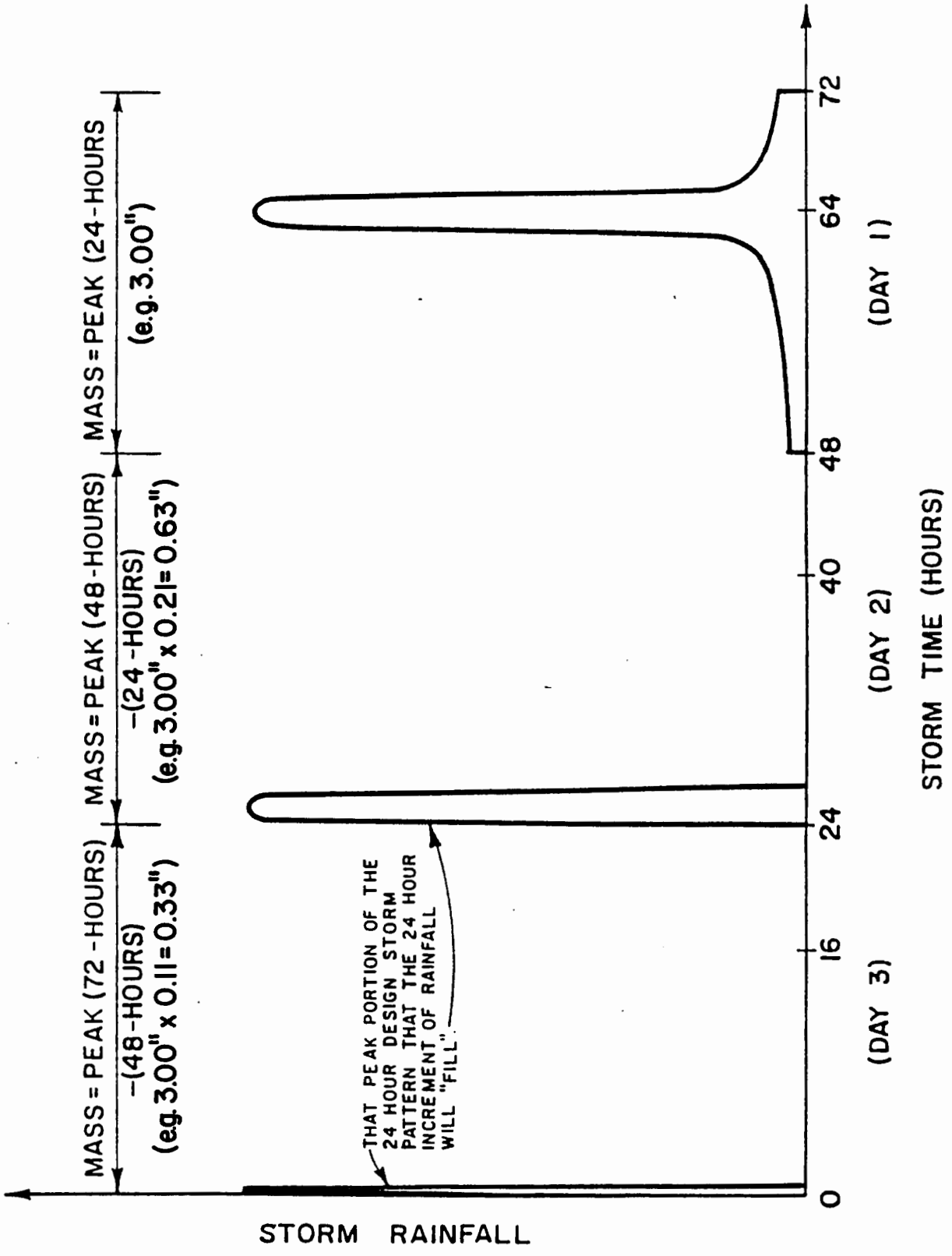
KERN COUNTY

HYDROLOGY MANUAL

FLOW-BY BASIN

CONCEPT AND ELEMENTS

Figure G-1



KERN COUNTY

HYDROLOGY MANUAL

FLOW-BY BASIN CRITICAL STORM PATTERN (3-DAY EXAMPLE)

Figure G-2

SECTION H

STREAMFLOW ROUTING

OVERVIEW

This section will cover the use of the convex routing technic in hydrology studies. It will discuss:

- Method description and method application

Section E includes a detailed discussion of the unit hydrograph. Section J discusses details of the development of small area hydrographs.

H.1. DEFINITIONS

Convex Routing: that method, as described in the "National Engineering Handbook: Section 4", in which it is recognized that the inflow and outflow variables are members of a Convex set (see "An Introduction to Linear Programming," by Charnes, Cooper, Henderson).

Routing Coefficient: The relational coefficient, C, that equates outflow to inflow and flood wave travel time to a selected steady flow travel time through a reach.

H.2. INTRODUCTION

The convex streamflow routing technique is described for use in design storm hydrology studies for watershed flood control systems (refs. 2, 3).

H.3. CONVEX ROUTING METHOD FOR UNSTEADY OPEN CHANNEL FLOW

The governing relationship used in the ***convex routing*** approach is:

$$O_{T+dT} = (1-C)\cdot O_T + C\cdot I_T \qquad \text{(H.1)}$$

where

I_T = hydrograph inflow at time T

O_T = channel outflow at time T

O_{T+dT} = channel outflow at time T + dT

C = a routing coefficient (where C is between 0 and 1)

Rearranging (H.1) gives the explicit statement

$$O_{T+dT} = O_T + C(I_T - O_T) \qquad \text{(H.2)}$$

and

$$C = (O_{T+dT} - O_T)/(I_T - O_T) \quad \text{(H.3)}$$

The **routing coefficient** may be estimated by the empirical relationship (ref.3),

$$C = V/(V + 1.7) \quad \text{(H.4)}$$

where V is a mean flow velocity assumed for the inflow hydrograph. One method of computing V is to calculate the normal depth corresponding to the average flowrate of all unit flows greater than 50 percent of the inflow hydrograph peak flowrate. From this normal depth calculation, V is defined to be the corresponding flow velocity. Thus V represents an average velocity which is used to translate the inflow hydrograph along the total length of the channel. Obviously, other values for V can result depending on the choice of which average flowrate value to be used from the inflow hydrograph, and care must be taken to avoid offsetting the hydrograph peak flow rates at channel confluences by the selection of the channel V parameter.

The **routing timestep**, dT, is given by

$$dT = \frac{C \cdot L}{3600 \cdot V} \quad \text{(H.5)}$$

where C is given by (H.4), L is the **channel length** in feet, and dT is in units of hours.

Because the unit hydrograph analysis base unit period (defined in this section as dT*) is usually different than the dT time period of (H.1) a modification of C is required. Typically, a 5-minute unit period is used in runoff hydrograph studies. For a unit period of 5 minutes, dT* = 0.0833 hours and the **modified routing coefficient** is

$$C^* = 1 - (1 - C)^E \quad \text{(H.6)}$$

where E = (dT* + 0.5dT)/(1.5dT).

To determine the dT which corresponds to (H.1), it is assumed that

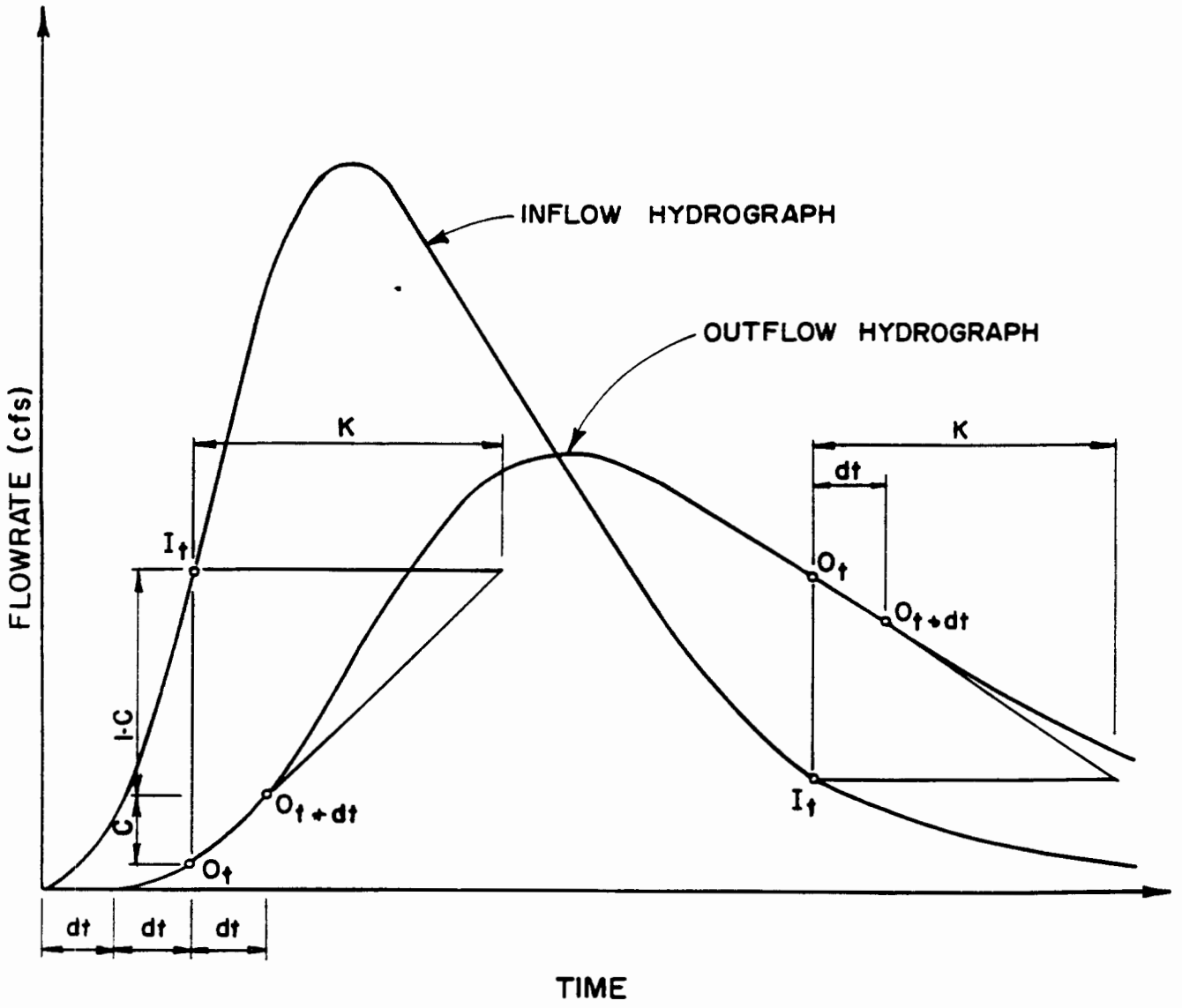
$$\frac{O_{T+dT} - O_T}{I_T - O_T} = \frac{dT}{K} \quad \text{(H.7)}$$

where K is the **channel reach travel time** as estimated from the selection of the inflow hydrograph mean V value and the channel length, L. Figure H-1 illustrates the geometric interpretation of the relationship given by (H.7). Thus

$$dT = CK \qquad \text{(H.8)}$$

A 5-minute unit period is used for all convex routing applications.

An examination of the convex routing method reveals that the entire routing approach is a function of the routing coefficient, C. Consequently, a watershed link-node model composed of m such channel links necessarily includes m channel routing parameters, each with an associated unknown uncertainty function. Additionally, the uncertainty involved in combining the m channel links is further aggravated by the fact that each channel-routed hydrograph is also a function of the number and channel reach lengths used for each channel link. That is, the routed hydrograph through a channel with a length of 20,000 feet will differ from the results of routing a hydrograph through two successive reaches with a length of 10,000 feet, and so forth. Channel routing processes usually involve relatively short reaches of improved channel where storage effects are minor, or where confluences from other channels (or pipes) enter the main channel and a summation of hydrographs occurs.



KERN COUNTY

HYDROLOGY MANUAL

ASSUMED ROUTING COEFFICIENT GEOMETRIC RELATIONSHIPS

Figure H-1

H.4. Example: Convex Channel Routing

The example problem channel is a rectangular concrete section with a base of 10 feet, a Manning friction factor of 0.015, length of 3,000 feet, and a mean slope of 0.005 foot/foot. The problem inflow hydrograph is tabulated in Table H.1. From the table, the average flowrate in excess of the 50-percent peak flowrate value is 767.4 cfs.

Using Manning's equation, the normal depth flow velocity is calculated as

$$V = \frac{1.486 \cdot R_h^{2/3} \cdot S_0^{1/2}}{n} \tag{H.9}$$

where R_h is the hydraulic radius, and S_0 is the channel slope. For the example problem, V is 13.5 fps (feet per second) and the default routing coefficient from (H.4) is C = 0.89. From (H.8),

K = (3000 ft)/(13.5 fps)(3600 sec/hr) = 0.062 hour, 3.7 minutes

dT = (0.89)(0.062) = 0.055 hour, 3.3 minutes

From (H.6), C* is estimated for a timestep of dT* equal to 5 minutes by

$C^* = 1 - (1-0.89)^E = 0.948$

where E = (0.0833 + (0.5)(0.055))/((1.5)(0.055)) = 1.343. Thus the appropriate convex method routing approximation statement is

$$O_{T+dT} = (1 - C^*) \cdot O_{T+dT-dT^*} + C^* I_T \tag{H.10}$$

where for the example problem

$O_{T+dT} = (0.052)O_{T+dT-dT^*} + (0.948)I_T$

TABLE H.1.

CONVEX ROUTING EXAMPLE PROBLEM SOLUTION

Storm Time T (minutes)	Inflow I(T) (cfs)	T+dT-dT* (minutes)	O(T+dT-dT*) (cfs)	O(T+dT) (cfs)	Outflow[1] O(T) (cfs)
0	0				
		3.3	0	0	
5	0.8				0.3
		8.3	0.8	0.8	
10	0.9				0.8
		13.3	0.9	0.9	
15	40.5				13.7
		18.3	38.4	38.4	
20	202.7				91.4
		23.3	194.2	194.2	
25	445.1				275
		28.3	432.1	432.1	
30	602.9				487.1
		33.3	594	594	
35	653.7				613.3
		38.3	650.6	650.6	
40	600.9				634.6
		43.3	603.5	603.5	
45	608				604.9
		48.3	607.8	607.8	
50	917.1				707.5
		53.3	901	901	
55	1186.7				993.1
		58.3	1171.8	1171.8	
60	1001.2				1116.8
		63.3	1010.1	1010.1	
65	763.6				930.6
		68.3	776.4	776.4	
70	714.9				756.6
		73.3	718.1	718.1	

Note 1 : Outflow is to be offset by 3.7 minutes of travel time due to a computed mean flow velocity of 13.5 fps.

Example Calculation:

$O_{T+dT} = (1-C^*)O_{T+dT-dT^*} + C^*I_T$; C*=0.948, dT=3.3, dT*=5

When T=20 min:

I_T =202.7 cfs

T+dT=23.3 min

T+dT-dT*=18.3 min

$O_{T+dT-dT^*}$=38.4 cfs

Therefore, O_{T+dT} = (1-0.948)38.4 + 0.948(202.7) = 194.2 cfs

By interpolation:

$$O_T = 38.4 + (192.4 - 38.4)\frac{20.0 - 18.3}{23.3 - 18.3}$$

= 91.4 cfs @ T = 20 min

SECTION I

A PIPEFLOW ROUTING MODEL

OVERVIEW

This section will cover the design considerations involved in routing hydrographs through pipe systems. It will discuss:

- Pipeflow routing process and criterion.

Section E includes detailed discussions of the unit hydrograph. Section J provides details of the development of small area hydrographs.

I.1. DEFINITIONS

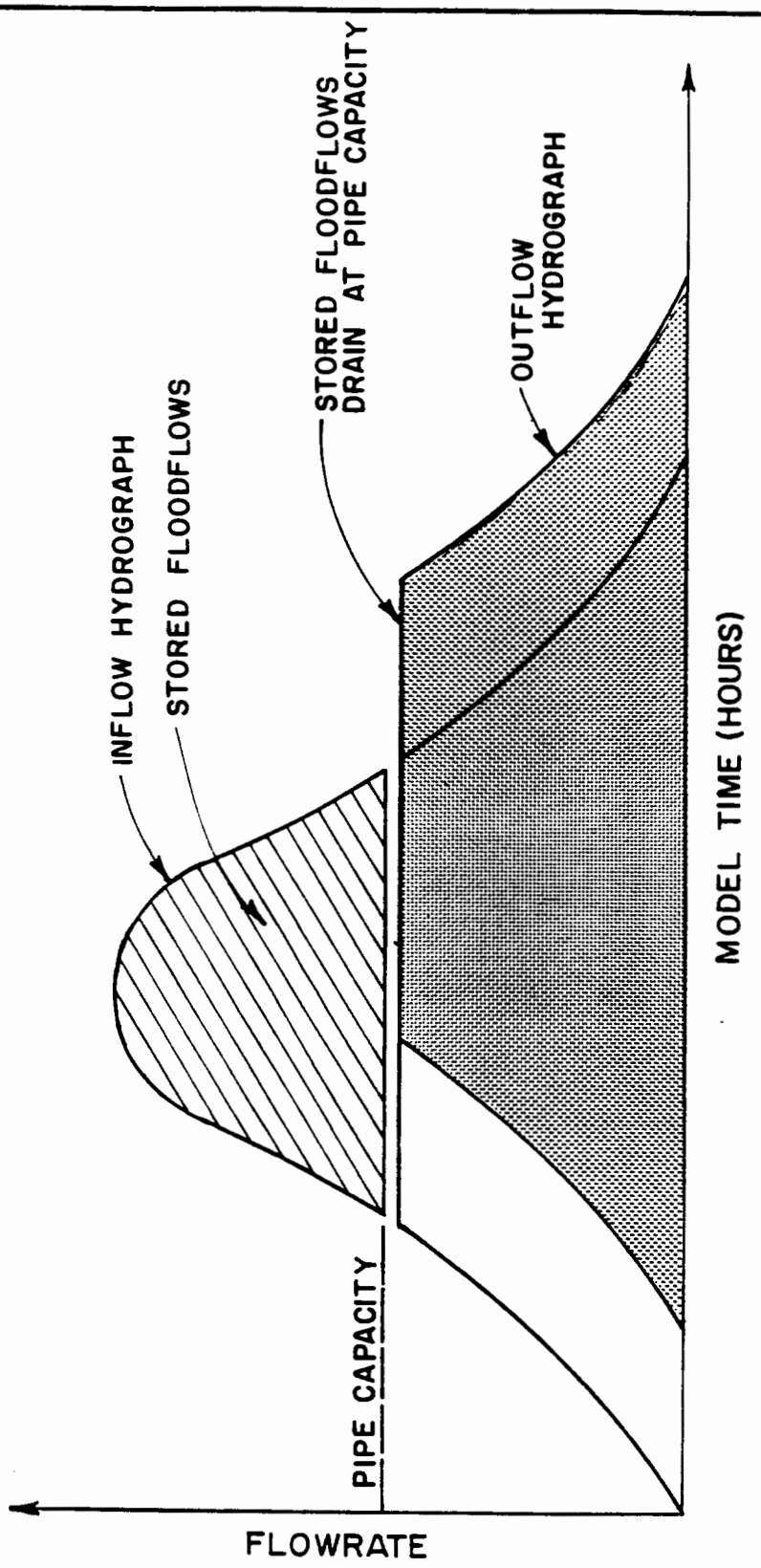
Pipeflow Routing: The process of routing a hydrograph through a storm drain system.

I.2. INTRODUCTION

Similar to the convex routing approximation procedure, the ***pipeflow routing*** model develops an outflow hydrograph from a reach of pipe given an inflow hydrograph and appropriate pipe section data. In the considered pipeflow model, however, a limiting value of outflow is assumed whereby all inflows greater than this pipe capacity are temporarily stored at the upstream endpoint of the pipe. The stored floodwaters subsequently drain into the pipe at a rate equal to the pipe capacity. Where this assumption is not valid, an alternative approach should be used. This modeling approach approximates the ponding of floodwaters where a significant volume of storage is available with a small change in flooding depths. Similar to the convex channel routing method, backwater effects are not included in the modeling approach.

I.3. PIPEFLOW ROUTING PROCESS

The pipeflow routing process is modeled by calculating a normal depth flow velocity for each unit period (e.g., 5-minute) runoff value from the inflow hydrograph, and translating the unit runoff forward in storm time by the appropriate time increment. Generally, flow depths in excess of 0.82 of the pipe diameter are assumed to be sealed. Where sealed pipe flows exist this method is not valid and an alternative approach should be used. Figure I-1 shows the salient features of the pipeflow modeling approach.



KERN COUNTY
HYDROLOGY MANUAL

PIPEFLOW
MODELING ELEMENTS

Figure I-1

SECTION J

SMALL AREA RUNOFF HYDROGRAPH DEVELOPMENT

OVERVIEW

This section will cover preparation of runoff hydrographs from watersheds with areas less than one square mile. It will discuss:

- Depth-Area adjustment
- Design storm pattern
- Unit hydrograph shape
- Convolution of the unit hydrograph with the unit effective rainfall

Section D includes a detailed discussion of the peak flow rate development. Section E includes additional information regarding unit hydrograph development as well as loss rate calculation procedures.

J.1. DEFINITIONS

Small Area: watersheds with areas less than one square mile.

J.2. INTRODUCTION

For watersheds with less than equal to one square mile of watershed, a simpler procedure can be used to develop the design storm runoff hydrograph. In order to avoid the unit hydrograph being too coarse an approximation, a small-area unit hydrograph method is needed. This technique is analogous to the design storm approach of Section E but has simplifications, which are elaborated in this section.

J.3. RUNOFF HYDROGRAPH CRITERION

J.3.1 Depth-area Adjustment

Generally, watersheds with less than or equal to one square mile of drainage area are small enough that depth-area adjustment is not required; i.e., the regionalized point rainfall depths are used without depth-area adjustment.

J.3.2 Design Storm Pattern Development

Using a unit interval equal to the time of concentration (Tc), unit rainfalls are determined by successive subtractions along the mass rainfall plot (see Example).

J.3.3 Loss Rates

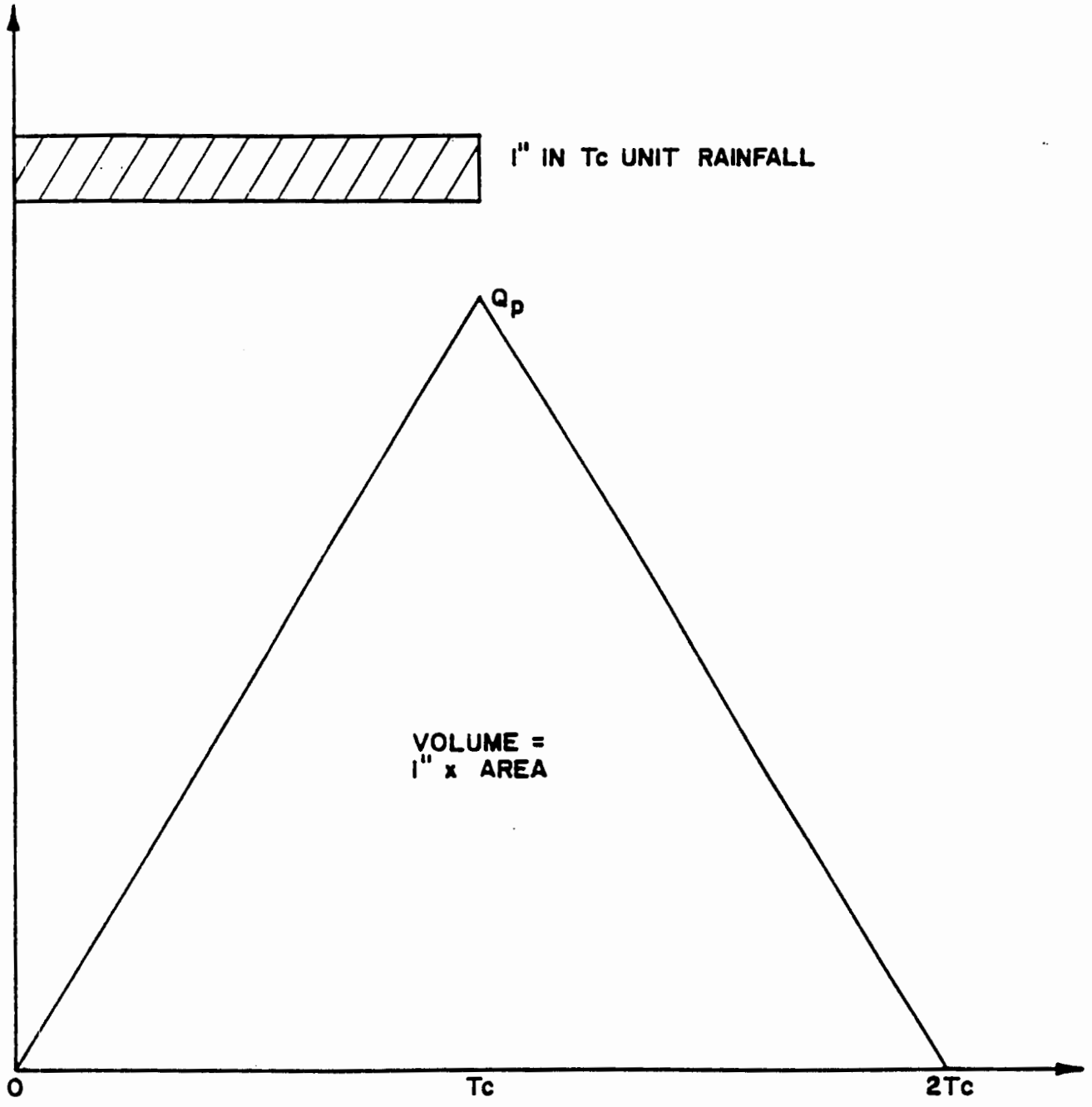
Conforms to Section E.

J.3.4 Unit Hydrograph

The unit hydrograph is defined to be a triangle with a base of 2Tc, and a peak at time of Tc (see Figure J-1). The volume of the unit hydrograph is (1-inch(area). The example problem illustrates the triangle unit hydrograph. Note that, in this case, lag is defined to equal the Tc estimate; (i.e., 50-percent volume occurs at time of Tc). Also note that lag is not computed, but the rational method Tc is used directly.

J.3.5 Convolution

The convolution of the unit hydrograph with the unit effective rainfalls (storm rainfall less losses) is simply the addition of peak runoff values at each of the Tc unit intervals (see example), where peak flow estimates follow from the rational method of Section D.



KERN COUNTY

HYDROLOGY MANUAL

SMALL AREA

UNIT HYDROGRAPH

Figure J-1

J.4. EXAMPLE: SMALL AREA RUNOFF HYDROGRAPH DEVELOPMENT

1. Assume given from a rational method study (see Section D):
   watershed area = 8 acres
   time of concentration = 10 minutes = unit interval
   maximum loss rate (F_m) = 0.12 inches/hour
   runoff yield fraction (Y) = 0.35
   low loss rate (Y-bar) = 0.65
2. D_{10} follows from Figure D-3,
   where
   D_{10} = 10-year frequency depth (inches)
   T = duration (minutes)
3. $Q = 0.9 (I-F_m)A$, or $.9 (I-F^*)A$

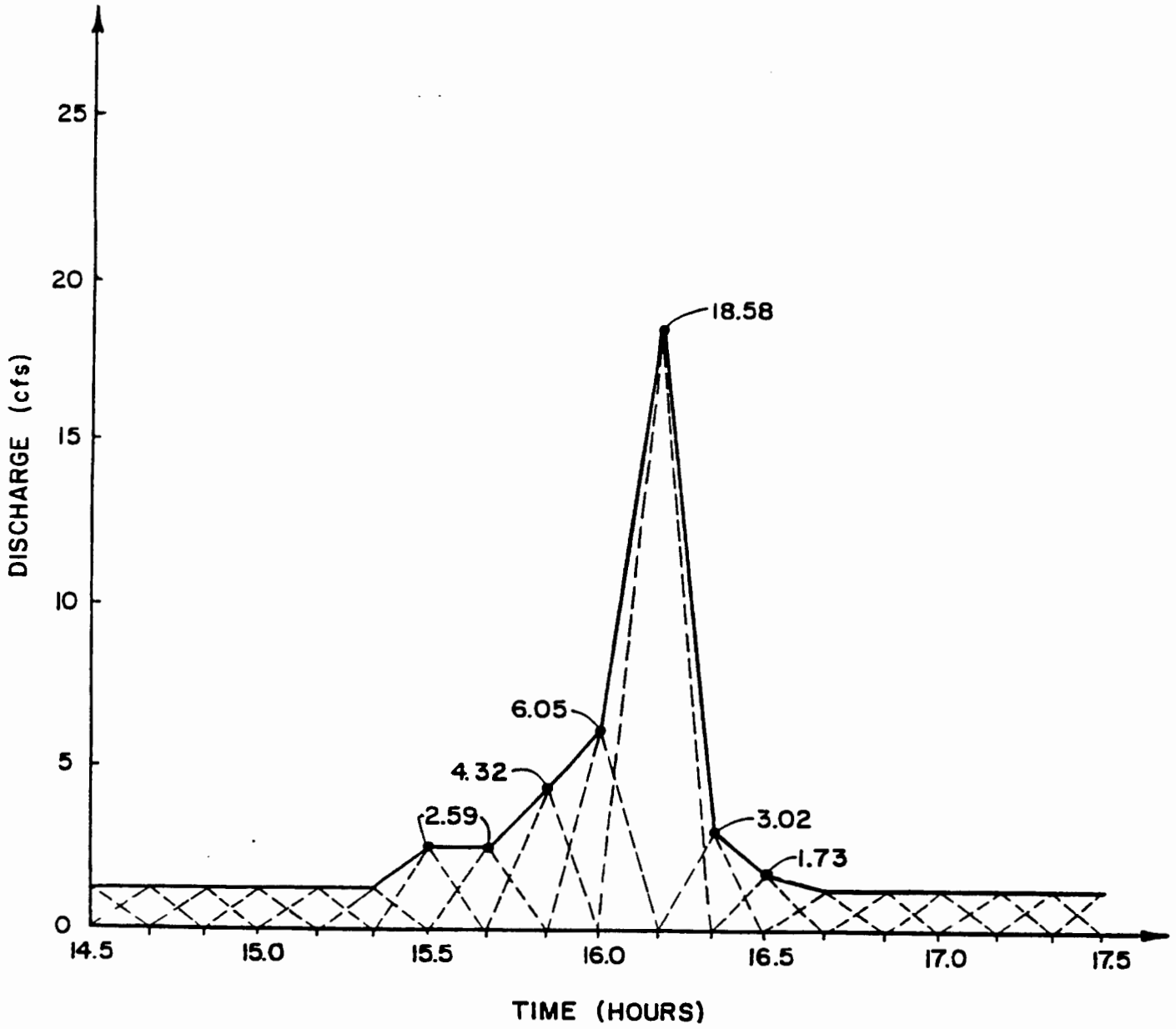
Table J.1

Example Problem Results

Peak Rainfall Unit Number	Mass Rainfall (inches)	Unit Rainfall (inches)	Unit Loss (inches)	Net Rainfall (inches)	Effective Rainfall (inches/hr)	Discharge (Q) (cfs)
1	0.45	0.45	.02*	0.43	2.58	18.58
2	0.61	0.16	.02*	0.14	.84	6.05
3	0.73	0.12	.02*	0.10	.60	4.32
4	0.82	0.09	.02*	0.07	.42	3.02
5	0.90	0.08	.02*	0.06	.36	2.59
6	0.98	0.08	.02*	0.06	.36	2.59
7	1.04	0.06	.02*	0.04	24	1.73
8	1.10	0.06	.02*	0.04	.24	1.73
9	1.16	0.06	.02*	0.04	.24	1.73
10	1.21	0.05	.02*	0.03	.18	1.30
11	1.27	0.06	.02*	0.04	.24	1.73
12	1.31	0.04	.01	0.03	.18	1.30
13	1.36	0.05	.02*	0.03	.18	1.30
14	1.40	0.04	.01	0.03	.18	1.30
15	1.44	0.04	.01	0.03	.18	1.30
16	1.48	0.04	.01	0.03	.18	1.30
17	1.52	0.04	.01	0.03	.18	1.30
18	1.56	0.04	.01	0.03	.18	1.30
19	1.60	0.04	.01	0.03	.18	1.30
20	1.63	0.03	.01	0.02	.12	0.86

*Unit low loss exceeds unit adjusted loss

Discharge to 24 hours is calculated by the above method



KERN COUNTY

HYDROLOGY MANUAL

EXAMPLE PROBLEM
SMALL AREA
RUNOFF HYDROGRAPH

Figure J-2

SECTION K

WATERSHED MODELING GUIDELINES

OVERVIEW

This section will present guidelines for the preparation and application of complex hydrologic models. It will discuss:

- Criterion for the development of complex watershed runoff hydrographs
- Guidelines for the use of watershed model computer programs
- Comparison criteria between Complex and Single area runoff hydrographs

Section E includes a detailed discussion of the unit hydrograph. Sections F, G, H, and I discuss various routing methods.

K.1. DEFINITIONS

Complex Watershed Runoff Hydrograph:	That runoff hydrograph developed for a watershed divided into subareas which are linked together by routing processes.
Single Area Runoff Hydrograph:	That runoff hydrograph developed for a watershed assuming that the watershed acts as a singular unit.

K.2. INTRODUCTION

The previous sections provide the several elements used in developing a link-node watershed model for hydrologic planning purposes. In this section, guidelines are presented for development of complex hydrologic models for the analysis of the design storm condition. The combination of the several submodels described in Sections E through I provides the hydrologist with the modeling capability to analyze complex watershed conditions including variations in runoff production caused by flood control measures and alternative watershed development plans. It is frequently required that the difference in runoff production between existing and the ultimate development conditions be mitigated. For example, in large watersheds the location of greenbelt channels or detention basins can significantly affect the total watershed peak flowrate estimate. Similarly, the planned location of high density development may mitigate the effects of watershed urbanization.

K.3. SINGLE AREA RUNOFF HYDROGRAPH DEVELOPMENT

In many cases, watershed studies involve a free flowing drainage system where storm runoff is collected by major storm drains or street systems and is carried from the watershed by means of a major flood control channel. These watersheds typically have minor storage or detention effects due to detention basins, channel constrictions, or channel capacity (i.e., overbank flow) problems. Additionally, these watersheds have a

time of concentration which approximately equals the watershed critical duration and are comparable to the watersheds from which the S-graphs were derived.

Generally, a **single basin unit hydrograph** study such as illustrated in the example problem of Section E will be appropriate for the development of a design storm runoff hydrograph and peak flow rate (see Sections E and J).

K.4. COMPLEX WATERSHED RUNOFF HYDROGRAPH DEVELOPMENT

For **complex watershed modeling** conditions, the watershed is divided into subareas which are "linked" together by routing processes. Such watersheds are characterized by significant detention or storage effects and large areas of different development or soil loss conditions.

The procedures to be used for the various routing processes are given in the preceding sections. Subarea unit hydrograph and subsequent runoff hydrograph development follows directly from Section E.

i. **Watershed Division into Subareas** - All watershed modeling results differ based on the number and selection of subareas linked together to represent the total watershed. A guideline for the watershed division is to limit subareas such that the largest subarea is no greater than four times the area of the smallest subarea. Generally, subareas are defined which are tributary to detention basins or major channels whose storage routing effects are considered significant. Additionally, subareas should be determined such that the corresponding lag values are between 20 minutes and 2.5 hours; preferably, between 25 minutes and 1.5 hours (the range of lag values used in the calibration effort). Arbitrary subdivision of the watershed into subareas should generally be avoided. It must be remembered that an increase in the watershed subdivision does not necessarily increase the modeling "accuracy" but rather, transfers the model's reliability from the calibrated unit hydrograph and lag relationships to the unknown reliability of the several flow routing submodels used to link together the several subareas.

ii. **Subarea Design Storm Analysis** - Each subarea is subject to the design storm condition. Therefore, all flood control facilities shall be analyzed based on the design storm impacting each subarea independently (see Section E).

iii. **Depth-area Adjustment** - As the watershed area increases, depth-area adjustment is needed based upon the entire tributary area. For example, should a point of concentration have three tributary subareas with a combined area of 6 square miles, then each of the subareas must be reanalyzed for the design storm condition with the depth-area factors based upon the total area of 6 square miles. All routing procedures are also reevaluated based upon the new subarea runoff hydrographs. In

this fashion, each point of concentration has the appropriate depth-area adjustment applied to the design storm.

K.5. USE OF WATERSHED MODEL COMPUTER PROGRAMS

Several single-event unit hydrograph computer models are currently available. For example, the unit hydrograph option of the HEC-1 and TR-20 programs have been used for both small and large watershed master planning. As discussed in Section E, unmodified use of these models are precluded. In the following, guidelines are presented which provide the parameter and design storm restrictions needed to conform the various available watershed models to the design storm conditions described in this manual:

i. **Effective Rainfall Computation** - All watershed loss rates are to conform to the specifications of Section C (i.e., watershed 24-hour storm runoff yields, maximum loss rates (Fm), and low loss fraction (Y)). (Note: TR 20 does not provide for a maximum loss rate (Fm), and therefore may not be applicable for use with the calibrated methods described in this manual).

ii. **Single-Event Design Storm Pattern** - The watershed model is to be based upon the design storm patterns shown in Sections E, F, and G. Depth-area adjustment and rainfall depths are to conform to the requirements of Sections B and E.

iii. **Routing Processes** - Basin and channel routing modeling techniques are to be based upon the modified Pul's and convex methods described in the previous sections. (Full details of these techniques are contained in refs. 2, 3, 5.)

iv. **Complex Watershed Modeling** - The division of the watershed into subareas and the application of depth-area adjustment to tributary area must conform to the guidelines of this section of the manual.

v. **Unit Hydrograph Development** - The development of unit hydrographs must conform to the S-graphs and lag computation procedures described in Section E. Calculation of watershed time of concentration must conform to the rational method procedures of Section D.

vi. **Preproject Meeting** - All complex watershed modeling proposals are to be discussed with the County prior to study submittals for review. This preproject meeting will aid in familiarizing the County with the project, and also aid in checking whether the modeling approach conforms to the hydrology manual.

K.6. SINGLE AREA RUNOFF HYDROGRAPH COMPARISON CRITERIA

When a complex watershed model (e.g., a "link-node" schematic involving subareas linked by channel routing) is to be used, a single area runoff hydrograph

model is also to be developed for comparison purposes. Should detention basins be planned, the complex model without the basins (i.e., "free-draining") is to be compared to a single subarea model.

Should the peak Q from the free-draining complex model be greater than the single area runoff hydrograph model, then the complex model peak Q is to be used as the design Q. The use of a higher Q for design purposes aids in accommodating for the increased uncertainty in the complex model.

APPENDIX I
DETENTION BASIN CONSIDERATIONS

Generally, the main purpose for inclusion of a stormwater detention basin in a flood control system is to reduce peak rates of runoff generated from an upstream watershed and to control peak flows into downstream areas. Some of the advantages and disadvantages of use of detention basins are listed in the following:

BENEFITS	POTENTIAL CONCERNS
Reduce peak rates of runoff to downstream areas.	Detention basins do not reduce total storm runoff volume (unless the groundwater recharge potential is large).
Basin reduces transport of sediments carried in floodwaters.	Maintenance of storage capacity, inflow and outflow facilities is critical.
Reduces size of downstream flood control facilities.	Basins increase the duration of flows which may increase erosion effects downstream from the basin. Downstream erosion may be further increased due to sediment extraction in the basin.
Provides location for groundwater recharge if aquifer contact exists.	Improperly sized and placed basins may aggravate rather than reduce downstream flooding potential (especially in large complex systems).
	Accumulated debris from runoff decreases flood control storage volume in a detention basin.
	Cost of debris removal.
	Detention basins in urban areas may become unsightly and/or vermin infested without intensive maintenance.

The consideration of a detention basin system needs to address the various hydrologic, hydraulic, environmental, and flood control concerns listed above, as well as any other concern which may arise during the course of the project study, and determine the necessary mitigative measures which are acceptable to the County. Of special concern is the interplay between the several components of the total system network. Unplanned placement of detention basins without consideration of other watershed detention basins and tributary watersheds can increase the downstream peak flow rate above the anticipated runoff peak flow rate attained without any detention basins in the watershed.

APPENDIX II

EQUATIONS FOR CURVES AND FIGURES

Figure C-6: Infiltration Rate For Pervious Areas Versus SCS Curve Numbers

CN	Fp
20	1.00
25	1.00
30	0.985
35	0.96
40	0.92
45	0.87
50	0.82
55	0.76
60	0.70
65	0.63
70	0.55
75	0.47
80	0.38
85	0.29
90	0.20
95	0.10
98	0.00

Figure D-1: Time of Concentration

Nomograph For Initial Area

General Equation: $$T_C = K \cdot \left(\frac{L^3}{H} \right)^E$$

L = length of initial subarea flow path (not to exceed 1,000 feet)

E = constant exponent, equal to 0.20

H = drop in elevation along flow path

K = coefficient depending on development type and other factors

Values of K for percent impervious or cover condition

% Impervious	K
90	0.304
80	0.324
75	0.336
65	0.360
60	0.374
50	0.389
40	0.412
30	0.438
20	0.469
15	0.483
10	0.487
0 (Agricultural)	0.525
0 (Poor Cover)	0.525
0 (Fair Cover)	0.706
0 (Good Cover)	0.935

Figure D-2: Rainfall Depth Versus Return Period For Partial Duration Series

General Equation:

$$Y_P = Y_2 + \frac{Y_{100} - Y_2}{263} \cdot K_P$$

P	K_P
5	65
10	108
25	164
50	215
5	65

where:

P	=	return period
K_P	=	constant associated with return period P
Y_2,Y_{100}	=	intensities associated with the 2-year and 100-year return periods, respectively
Y_P	=	intensity associated with return period P

Figure D-3: Intensity-Duration Curves

General Equation: $$Y = Y_{1hr} \cdot \left(\frac{60}{X}\right)^S$$

where: S = Regional Intensity-Duration Slope
X = Duration (in minutes)
Y_{1hr} = Intensity at the 1 hour intensity
Y = Intensity at Duration = X (in inches/hour)

Equation D-9a,b,c: Equation Derivation

The adjusted area equation is derived as,

$$A_p = A_2 + \text{Adjusted}[A_1] \tag{D.9}$$

When $I_2 - F_{m1} > Y_1 I_2$ and $I_1 - F_{m1} > Y_1 I_1$, from (D.8);

$$Adjusted[Q_1] \equiv \frac{T_2 + T_1}{2 \cdot T_1} \cdot Q_1 = \frac{T_2 + T_1}{2 \cdot T_1} \cdot \{0.9 \cdot (I_1 - F_{m1}) \cdot A_1\}$$

$$Adjusted[Q_1] = 0.9 \cdot (I_2 - F_{m1}) \cdot Adjusted[A_1]$$

combining equations,

$$0.9 \cdot (I_2 - F_{m1}) \cdot Adjusted[A_1] = \frac{T_2 - T_1}{2 \cdot T_1} \cdot \{0.9 \cdot (I_1 - F_{m1}) \cdot A_1\}$$

simplifying

$$Adjusted[A_1] = \frac{(T_2 + T_1) \cdot (I_1 - F_{m1})}{2T_1 \cdot (I_2 - F_{m1})} \cdot A_1 \tag{D.9a}$$

When $I_2 - F_{m1} > Y_1 I_2$ and $I_1 - F_{m1} < Y_1 I_1$, from (D.8):

$$Adjusted[Q_1] = \frac{T_2 + T_1}{2T_1} \cdot Q_1 = \frac{T_2 + T_1}{2T_1} \cdot \{0.9 \cdot (Y_1 \cdot I_1) \cdot A_1\}$$

$$Adjusted[Q_1] = 0.9 \cdot (I_2 - F_{m1}) \cdot Adjusted[A_1]$$

Combining equations,

$$0.9 \cdot (I_2 - F_{m1}) \cdot Adjusted[A_1] = \frac{T_2 + T_1}{2T_1} \cdot \{0.9 \cdot (Y_1 \cdot I_1) \cdot A_1\}$$

Simplifying

$$Adjusted[A_1] = \frac{(T_2 + T_1) \cdot (Y_1 \cdot I_1)}{2T_1 \cdot (I_2 - F_{m1})} \cdot A_1 \tag{D.9b}$$

When $I_2 - F_{m1} < Y_1 I_2$ and $I_1 - F_{m1} < Y_1 I_1$, from (D.8):

$$Adjusted[Q_1] = \frac{T_2 + T_1}{2T_1} \cdot Q_1 = \frac{T_2 + T_1}{2T_1} \cdot \{0.9 \cdot (Y_1 \cdot I_1) \cdot A_1\}$$

$$Adjusted[Q_1] = 0.9 \cdot (Y_1 \cdot I_2) \cdot Adjusted[A_1]$$

Combining equations,

$$0.9 \cdot (Y_1 \cdot I_2) \cdot Adjusted[A_1] = \frac{T_2 + T_1}{T_1} \cdot \{0.9 \cdot (Y_1 \cdot I_1) \cdot A_1\}$$

Simplifying,

$$Adjusted[A_1] = \frac{(T_2 + T_1) \cdot I_1}{2 \cdot T_1 \cdot I_2} \cdot A_1 \qquad \text{(D.9c)}$$

Figure E-3a: Valley Developed S-Graph

S-Graph % Volume	% Lag	S-Graph % Volume	% Lag
0	0	52	103
2	17.5	54	106
4	29	56	109.5
6	35.25	58	112
8	40	60	114
10	43.75	62	115
12	47.75	64	117.25
14	51.75	66	120.5
16	54.25	68	123.5
18	58	70	125.25
20	61.75	72	129
22	64.75	74	132
24	67.25	76	136
26	70.75	78	140
28	74.25	80	145
30	76.75	82	149
32	79.75	84	153.5
34	81.75	86	158.75
36	84.25	88	165.25
38	88	90	173.75
40	89.25	92	183.25
42	92	94	193.75
44	94.25	96	209.5
46	96.87	98	235
48	98.75	100	315
50	100		

Figure E-3b: Foothill S-Graph

S-Graph % Volume	% Lag	S-Graph % Volume	% Lag
0	0	52	103
2	15	54	106
4	26	56	109
6	35	58	113
8	41	60	117
10	49	62	120
12	55	64	125
14	60	66	132
16	64	68	138
18	68	70	145
20	72	72	152
22	76	74	159
24	78	76	167
26	81	78	177
28	83	80	188
30	86	82	199
32	87	84	212
34	89	86	226
36	91	88	242
38	92	90	262
40	93	92	284
42	94	94	311
44	95	96	348
46	97	98	410
48	98	100	700
50	100		

Figure E-3c: Mountain S-Graph

S-Graph % Volume	% Lag	S-Graph % Volume	% Lag
0	0	52	106
2	9	54	113
4	17	56	121
6	23	58	131
8	29	60	141
10	33	62	151
12	37	64	163
14	41	66	176
16	44	68	190
18	47	70	206
20	50	72	222
22	52	74	242
24	55	76	264
26	57	78	288
28	59	80	316
30	61	82	346
32	64	84	383
34	67	86	428
36	70	88	479
38	73	90	536
40	77	92	601
42	81	94	678
44	85	96	779
46	90	98	918
48	95	100	1300
50	100		

Figure E-3d: Valley Undeveloped S-Graph

S-Graph % Volume	% Lag	S-Graph % Volume	% Lag
0	0	52	103
2	12	54	107
4	21	56	111
6	28	58	115
8	34	60	120
10	39	62	126
12	44	64	132
14	48	66	138
16	51	68	146
18	55	70	154
20	58	72	164
22	61	74	174
24	64	76	186
26	67	78	200
28	70	80	215
30	73	82	233
32	75	84	254
34	78	86	279
36	81	88	309
38	83	90	343
40	86	92	386
42	89	94	440
44	92	96	508
46	94	98	611
48	97	100	820
50	100		

Figure E-3e: Desert S-Graph

S-Graph % Volume	% Lag	S-Graph % Volume	% Lag
0	0	52	104
2	18	54	108
4	29	56	112
6	37	58	117
8	43	60	122
10	48	62	127
12	53	64	133
14	56	66	140
16	59	68	148
18	62	70	156
20	64	72	165
22	66	74	175
24	68	76	186
26	70	78	199
28	72	80	213
30	74	82	228
32	76	84	246
34	78	86	268
36	80	88	293
38	83	90	323
40	85	92	359
42	88	94	402
44	91	96	466
46	94	98	580
48	97	100	780
50	100		

Figure E-4: Design Storm Depth Area Curves

Area	5-Minute %	30-Minute %	1-Hour %
0	100	100	100
7	80	80	80
13	70	70	70
25	57	59.5	60.5
32.5	52	54.5	56
40	47.5	51	53
50	43	46.8	49.2
57.5	40.2	44.5	47.3
65	38	42.3	45.8
75	35	40	44
82.5	33.3	38.5	42.5
90	31.8	37.2	41.5
100	30	35.5	40.2
107.5	28.8	34.9	39.2
115	27.6	34	38.5
125	26.5	33	37.8
132.5	25.7	32.3	37.2
140	25	31.8	36.8
150	24	31	36.2
Area	**3-Hour %**	**6-Hour %**	**24-Hour %**
0	100	100	100
7	97	98.5	99.1
13	94	97.3	98.3
25	91	95.8	97.4
32.5	88.5	94.9	96.8
40	86.5	94	96.3
50	84	92.9	95.7
57.5	82.3	92.2	95.3
65	80.8	91.5	94.9
75	79	90.6	94.5
82.5	77.5	89.9	94.1
90	76.2	89.4	93.8
100	74.7	88.9	93.5
107.5	73.5	88.5	93.2
115	72.5	88.2	93
125	71.2	87.8	92.8
132.5	70.3	87.5	92.6
140	69.6	87.2	92.5
150	68.8	87	92.3

APPENDIX III

List of Equations and Variables used in the Hydrology Manual

B.1	P = 1/T		
B.2	Y2	=	0.111+0.545 (X1)(X1/X2)
B.3	Y100	=	0.221+0.885 (Y2)(X3/X1)
B.4	or Y100	=	0.221+0.098 (X3/X1) +0.482 (X3)(X1/X2)
B.5	Y2	=	0.107+0.315 (X1)
B.6	Y100	=	-0.391+1.224 (Y2)(X3/X1) +0.043 (X5)
B.7	or Y100	=	-0.391+0.131 (X3/X1) +0.386(X3)+0.043 (X5)
B.8	Y2	=	0.126+0.561 (X1)(X1/X2)
B.9	Y100	=	-0.030+0.816 (Y2)(X3/X1) +0.063 (X5)
B.10	or Y100	=	-0.030+0.103 (X3/X1) +0.458 (X3)(X1/X2) +0.063 (X5)
B.11	Y2	=	0.005+0.852 (X1)(X1/X2)
B.12	Y100	=	0.322+0.789 (X3)(X3/X4)
C.1	Ia	=	0.2S
C.2	$S = \frac{1000}{CN} - 10$		
C.3	$Y_j = \frac{(P_{24} - Ia)^2}{(P_{24} - Ia + S)P_{24}}$		
C.4	$Y = \frac{Y_1 A_1 + \ldots + Y_m A_m}{A_1 + A_2 + \ldots + A_m}$		

C.5	$\overline{Y} = 1 - Y$
C.6	$F^* = \overline{Y} \cdot I$
C.7	$F_m = a_p F_p$
D.1	$Q = CIA$
D.2	$Tc(T) = Tc(100) \cdot K^T$
D.3	$V = \frac{1.49}{n} \cdot R^{2/3} \cdot S^{1/2}$
D.4	$C = \begin{cases} 0.90(a_i + ap(I - F_p)/I), \text{ for I greater than } F_p \\ 0.90a_i, \text{ for I less than or equal to } F_p \end{cases}$
D.5	$Q = 0.90\ (I - F_m)A$
D.6	$Q(min) = 0.90 \cdot YIA$
D.7	$Q_p = Q_2 + Adjusted[Q_1]$
D.8	$Adjusted[Q_1] = Q_1 * \text{Minimum of } \left\{ \frac{\text{Maximum of } [(I_2 - Fm_1),\ (Y_1 I_2)]}{\text{Maximum of } [(I_1 - Fm_1),\ (Y_1 I_1)]},\ \frac{(T_2 + T_1)}{2T_1} \right\}$
D.8a	$Adjusted[Q_1] = \frac{I_2 - Fm_1}{I_1 - Fm_1} \cdot Q_1$
D.8b	$Adjusted[Q_1] = \frac{Y_1 \cdot I_2}{I_1 - Fm_1} \cdot Q_1$
D.8c	$Adjusted[Q_1] = \frac{I_2}{I_1} \cdot Q_1$
D.8d	$Adjusted[Q_1] = \frac{T_2 + T_1}{2 \cdot T_1} \cdot Q_1$
D.9	$A_p = A_2 + Adjusted[A_1]$
D.9a	$Adjusted[A_1] = \frac{(T_2 + T_1)(I_1 - F_{m1})}{2T_1(I_2 - F_{m1})} \cdot A_1$
D.9b	$Adjusted[A_1] = \frac{(T_2 + T_1) \cdot (Y_1 \cdot I_1)}{2T_1 \cdot (I_2 - F_{m1})} \cdot A_1$
D.9c	$Adjusted[A_1] = \frac{(T_2 + T_1) \cdot I_1}{2T_1 \cdot I_2} \cdot A_1$

E.1	Lag	=	0.8Tc
E.2	Lag(hours)	=	$C_t((L \cdot L_{CA})/S^{0.5})m$
E.3	K(cfs)	=	645·A/T
E.4	$\overline{Y} = 1 - Y$		
E.5	$F^* = \overline{Y} \cdot I$		
E.6	$F_m = a_p F_p$		
F.1	I – O	=	S/dT
F.2	I	=	$(I_1 + I_2)/2$
F.3	O	=	$(O_1 + O_2)/2$
F.4	dS	=	$S_2 - S_1$
F.5	$S_2 + O_2 \cdot \frac{dT}{2} = (S_1 - O_1 \cdot \frac{dT}{2}) \cdot (I_1 + I_2) \cdot \frac{dT}{2}$		
H.1	$O_T + d_T$	=	$(1 - C) \cdot O_T + CI_T$
H.2	O_{T+dT}	=	$O_T + C(I_T - O_T)$
H.3	C	=	$(O_{T+dT} - O_T)/(I_T - O_T)$
H.4	C	=	V/(V + 1.7)
H.5	$dT = \frac{CL}{3600 \cdot V}$		
H.6	C*	=	$1 - (1 - C)^E$
H.7	$(O_{T+dT} - O_T)/(I_T - O_T)$	=	dT/K
H.8	dT	=	CK
H.9	$V = \frac{1.486 \cdot R_h^{2/3} \cdot S_0^{1/2}}{n}$		
H.10	O_{T+dT}	=	$(1 - C^*)O_{T+dT-dT^*} + C^*I_T$

List of Variables

P	Exceedance Probability
T	Return Period
Y2	2-year 1-hour estimated value
Y100	100-year 1-hour estimated value
X1	2-year 6-hour value from precipitation-frequency maps
X2	2-year 24-hour value from precipitation-frequency maps
X3	100-year 6-hour value from precipitation-frequency maps
X4	100-year 24-hour value from precipitation-frequency maps
X5	Latitude (in decimals) minus 32
Ia	Initial abstration
S	Soil capacity
CN	Curve number
Y_j	24-hour storm runoff yield fraction for subarea Aj
P_{24}	24-hour storm rainfall
Y	Catchment 24-hour storm runoff yield fraction
$\overline{Y}$	Catchment low loss fraction
F*	Catchment low loss rate
I	Rainfall intensity
F_m	Maximum loss rate
a_p	Pervious area fraction
a_i	Impervious area fraction
F_p	Infiltration rate for the pervious area
Q	Peak discharge
C	Runoff coefficient

A	Drainage area
Tc(T)	Time of concentration for return period, T
Tc(100)	Time of concentration for 100 yr return period
Tc	Time of concentration
K_T	Time adjustment constant
V	Velocity
n	Manning's roughness coefficient
R	Hydraulic radius
s	Slope
Q(min)	Minimum catchment peak discharge
Q_p	Resultant confluence estimate of peak flow rate
T_p	Resultant confluence estimate of time of concentration
A_p	Resultant confluence estimate of catchment area
Q_1	Peak flow rate from stream 1
T_1	Time of concentration for stream 1
I_1	Rainfall intensity for stream 1
F_{m1}	Maximum loss rate for stream 1
Y_1	Runoff Yield fraction for stream 1
A_1	Catchment area for stream 1
Q_2	Peak flow rate from stream 2
T_2	Time of concentration for stream 2
I_2	Rainfall intensity for stream 2
F_{m2}	Maximum loss rate for stream 2
Y_2	Runoff Yield fraction for stream 2
A_2	Catchment area for stream 2

Lag	The elapsed time (in hours) from the beginning of unit effective rainfall to the instant that the summation hydrograph for the point of concentration reaches 50 percent of ultimate discharge.
C_T	A constant (determined by regional flood reconstitution studies)
L	Length of longest watercourse
L_{CA}	Length along the longest watercourse, measured upstream to a point opposite the center of the area.
S	Overall slope of drainage area between the headwaters and the collection point.
m	A constant determined by regional flood reconstitution studies
$\bar{n}$	The visually estimated basin factor
K	Ultimate discharge factor
I	Inflow
O	Outflow
S	Storage
dT	Routing timestep
I_T	Hydrograph inflow at time T
O_T	Channel outflow at time T
O_{T+dT}	Channel outflow at time T+dT
dT*	Unit period
$O_{T+dT-dT}$*	Channel outflow at time T+dT-Dt*
C	Routing coefficient
C*	Modified routing coefficient
E	Routing coefficient modifier

APPENDIX IV

DEVELOPMENT OF HYDROLOGIC DESIGN CRITERIA

Introduction:

The hydrologic procedures proposed for use are the most common, nationally accepted methods available. It has been demonstrated that complex, highly discrete models, typically do not perform any better at determining the peak flow and runoff volume than the more common, less complicated, models described in this manual. It is for this reason that the Modified Rational Method, and the Unit hydrograph method procedures are used in this manual.

It is the intent of this manual to clearly define the application of methodology and link the procedures to the best available data set for calibration purposes.

Hydrologic Procedure Calibration:

The hydrologic procedures are based on the regional design storm rainfall-runoff model calibration study conducted by Dr. T.V. Hromadka of Williamson and Schmid for the Orange County Environmental Management Agency (OCEMA) and San Bernardino County Water Resources Division.

A calibrated design storm rainfall-runoff model should satisfy three major characteristics. First, the runoff portion of the model should reproduce observed flood events with a reasonable accuracy given adequate observed rainfall for the event. For the Orange County regional model, regional unit hydrograph and loss rate relationships were derived from reconstitutions of several observed storm and flood events in Orange and Los Angeles counties. The derived regional relationships were verified using a storm and flood event not used in the reconstitution studies.

Second, the calibrated design storm model should reproduce n-year peak discharge frequency curves from stream gauge data with reasonable accuracy. To accomplish this, first, a storm pattern, time distribution, depth-area-duration reduction relationships, and a source of T-year t-hour precipitation values were selected. A nested duration storm pattern, also referred to as a "balanced" storm in HEC Training Document 15, is used in order to provide uniform return frequency rainfalls for all durations, additionally, the nested duration storm pattern exhibits several similarities to the pattern of a 6-hour storm that occurred over Orange County on March 1, 1983. During that particular storm, recording rain gauges showed near 100-year rainfall intensities for all durations between 15 minutes and 6 hours. Because every watershed is sensitive to a particular duration of rainfall that will produce the peak discharge, a nested duration design storm ensures that each watershed will receive the design frequency depth of rainfall for it's critical duration. In the design storm the maximum 5 minutes of rainfall is nested within the maximum 10 minutes, the maximum 10 minutes within the maximum 15 minutes and so forth until a 24-hour storm pattern is developed. A 2/3, 1/3 time distribution with T-year frequency rainfalls

nested about hour 16 of the 24-hour storm was chosen due to the sensitivity in reservoir or detention basin design with respect to storm pattern shape. Depth area adjustment factors from the March 1943 Sierra Madre storm that occurred in Southern California, for durations between 30 minutes and 3 hours, along with NOAA Atlas 2 factors for durations between 6 and 24 hours were chosen. The Sierra Madre factors were chosen for the shorter durations over the NOAA Atlas factors because the Sierra Madre storm occurred locally and is well documented. These variables then became deterministic components of the model.

The regional loss rate function and unit hydrograph parameters derived from the flood reconstitutions also became deterministic components.

The final deterministic component, AMC, was also selected; an average value of AMC II was used in the calibration study.

The remaining variable, T-year precipitation, was made the only design storm calibration variable; frequency of precipitation was varied until adequate reproduction of T-year peak discharges from stream gauge frequency curves was obtained.

In order to calibrate the design storm for the model so that the peak flow rate estimates represent local flood frequency tendencies, an objective criterion for flood protection must be specified. That is, not only is a 100-year flood, for example, selected as a targeted level of flood protection, but also a level of confidence in estimating (or exceeding) the 100-year flood must be specified. This specification for a hydrologic model becomes then a policy statement for flood control.

The estimation of the T-year flood is a basic problem in hydrology due to the uncertainty in the estimate caused by the uncertain estimation of statistical parameters, such as due to sampling variance. This uncertainty can have a significant effect on the flood design value, and it's quantification is an important aspect of evaluating the risk involved in a chosen level of flood protection. A flood estimate obtained from a typical flood frequency curve only represents an estimate of the true value. The confidence that the value obtained from a flood frequency curve equals or exceeds the true value of the selected return frequency is only 50-percent. Should a value be used that is larger than the flood frequency curve value, then there is an associated increase in confidence achieving the desired level of flood protection. In order to address the sampling uncertainty and reduce the chance of being flooded, Orange and San Bernardino counties both adopted a policy statement objective to use the discharges computed at the 85-percent upper confidence level for the T-year flood as the criterion for flood protection.

The results of the calibration are specified in terms of T-year precipitation needed to produce 50-percent and 85-percent confidence level T-year peak discharges, given all other hydrologic parameters (e.g. timing, losses, etc...) invariant and are as follows:

Table Appendix 4.1

Return Frequency Rainfall Required (Yrs)

50% Confidence	85% Confidence	Return Frequency of Runoff
25	100	100
15	50	50
10	25	25
5	10	10
4	5	5
2	2	2

Third, the calibrated design storm rainfall-runoff model should be transportable to ungauged watersheds with assurance that the computed peak discharges are reasonable estimates of T-year peak discharges. This can be achieved because the parameters that vary with physical characteristics of a watershed are all deterministic; S-graph, lag and loss function. The calibrated design storm provides considerable assurance that with appropriate modification of the deterministic variables, reasonable estimates of n-year peak discharges will result.

Results of Calibration to Selected Kern County Watersheds:

An analysis of three streams (San Emigdio, Oak, Joshua) was performed to check the calibration of the design storm variable, T-year precipitation, given the deterministic variable values obtained by use of the Kern County Hydrology Manual.

The results of the statistical analysis of the stream gauge record for Joshua, Oak, an San Emigdio Creeks are listed in Table Appendix 4.2. The calibration results are listed in Table Appendix 4.3, 4.4, and 4.5. The results from Joshua Creek are considered quite anomalous, resulting from several high flows and a very high percentage of zero flow years, and were therefore not considered in the evaluation of the results.

Table Appendix 4.2

Flood Magnitudes for Selected Exceedence Frequencies

Computed Discharge

Return Frequency	Exceedence Probability	Joshua Creek		Oak Creek		San Emigdio Creek	
		50%	85%	50%	85%	50%	85%
5	0.2	180	500	135	300	--	--
10	0.1	710	1500	440	700	545	1100
25	0.04	2370	4400	1070	1500	1840	2800
50	0.02	4950	8400	1700	2400	3260	5100
100	0.01	8910	14700	2550	3700	5370	8700

Table Appendix 4.3

Design Storm

Calibration Results for Joshua Creek (3.83 miles^5)

Return Frequency of Runoff	Return Frequency of Precipitation	Return Frequency of Precipitation
	50%	85%
100	>100	>100
50	>100	>100
25	100	>100
10	<5	10
5	<5	<5
2	<5	<5

Table Appendix 4.4

Design Storm

Calibration Results for Oak Creek (15.8 miles^5)

Return Frequency of Runoff	Return Frequency of Precipitation 50%	Return Frequency of Precipitation 85%
100	5	25
50	<5	5
25	<5	<5
10	<5	<5
5	<5	<5
2	<5	<5

Table Appendix 4.5

Design Storm

Calibration Results for San Emigdio Creek (48.8 miles^5)

Return Frequency of Runoff	Return Frequency of Precipitation 50%	Return Frequency of Precipitation 85%
100	5	25
50	<5	5
25	<5	<5
10	<5	<5
5	<5	<5
2	<5	<5

The Kern County design storm calibration results are inconsistent with those results obtained from the use of the coastal Los Angeles and Orange County data and the valley San Bernardino County data in that the required T-year precipitation values show considerably higher variance in correlating rainfall to runoff. The rainfall statistical estimates obtained from both the NOAA Atlas II (1973) and DWR reports are both undergoing further study and may change in future updates. The Kern County stream gauge records require considerable time in order to obtain sufficient data to reduce the variance in floodflow estimates.

The Los Angeles County rainfall-runoff data is more substantial than the Kern County data, with more gauges and longer periods of record. Most likely, the two sets of calibration results differed so greatly because the Orange County model was calibrated to stream gauges with a substantially longer period of record including several years of record outside of the "dry" years; whereas, the Kern County stream gauges have a much shorter period with only a few years of record outside of the "dry" years. The Kern County stream gauge data may not be representative of the long term statistical trends of the area, only the collection of additional data will resolve these uncertainty issues.

APPENDIX V

Brief Overview of the Recent History of Flooding in Kern County

Community Description

Kern County lies at the southern end of the San Joaquin Valley, in southern California. The County is bordered on the west by San Luis Obispo and Santa Barbara Counties; on the south by Ventura and Los Angeles Counties; on the east by San Bernardino County; and on the north by Kings, Tulare, and Inyo Counties. Kern County covers 8,064 square miles and is California's third largest county. Elevations range from 205 feet to 8,775 feet.

The following incorporated communities are located within the boundaries of Kern County: Cities of Arvin, Bakersfield, California City, Delano, Maricopa, Ridgecrest, Shafter, Taft, Tehachapi, and Wasco.

The County can be divided into three distinct physiographic zones: valley, desert, and mountain. The valley portion, the western one-third of the County, is its population and agricultural center. Valley crops make Kern County a leading agricultural producer among the nation's counties. The Mojave Desert covers the eastern one-quarter of the County. Major economic activities in the desert depend on military installations — Edwards Air Force Base and the China Lake Naval Weapons Center — and on mining. One of the world's largest borax deposits is located near Boron. The third zone in the County consists of a ring of foothills and mountains. The Temblor Range, part of the Coast Range, borders the western edge of Kern County. The Tehachapi Mountains separate the valley from the Los Angeles basin to the south. The Sierra Nevada Range separates the valley from the desert to the east. The western hills contain a major source of petroleum; Kern County produces one-half of California's annual petroleum output. The Tehachapi and Sierra Nevada Ranges contain a number of year-round and seasonal resorts and produce commercial timber and minerals.

The vegetation and climate of Kern County vary among its three physiographic zones. Generally, the County is classified as desert or semiarid, with hot, dry summers and mild, humid winters. In most areas, 90 percent of the precipitation occurs from November through April.

In the valley, rainfall averages three to seven inches annually. Maximum summer temperatures in Bakersfield average 101 degrees Fahrenheit; there are approximately 110 days per year when temperatures are higher than 90 degrees Fahrenheit. On the average, 16 days per year have temperatures below 32 degrees Fahrenheit. The long growing season of 250 to 300 days allows growing a wide range of crops when irrigation is provided. Major agricultural products of the valley include grapes, cotton, cattle, potatoes, almonds, oranges, forage crops, spring and summer vegetables, and rose plants. Annual production of these commodities is valued at more than $15 million each. Before the

advent of agriculture, the valley floor was covered by grassland. It was one of the most productive wildflower areas of the State.

Because of the topography, the climate of the mountain zone tends to be cooler and wetter than the other zones. The western hills, in the lee of the Coast Range, are semiarid and sparsely vegetated. In contrast, the windward side of the Tehachapi and Sierra Nevada Ranges receive as much as 40 inches of precipitation a year. An average of 30 inches of snow falls at higher elevations. However, most snow cover is intermittent during the winter because of occasional melting. The occurrence of thunderstorms and the potential for intense, local rainfall increases on approaching the Sierra Nevada Range. Whereas the western edge of the valley experiences an average of one such storm each year, the number increases to 15 to 20 per year over the mountains. These occur with greatest frequency in the summer. Four plant communities occur in the mountain zone, depending on site characteristics: oak woodland, coniferous woodland, desert shrub, and grassland.

The desert zone lies in the lee of Tehachapi and Sierra Nevada Ranges. Precipitation averages three to six inches a year, but is extremely variable. Snowfall is infrequent, but may range from one to four inches when it occurs. Snow and winter rains result from North Pacific cyclonic storms. The remainder of the precipitation comes from intense, local thunderstorms and summer tropical storms. The natural vegetation is sparse and consists primarily of creosote and burroweed; Joshua trees occur in some locations.

Early settlement in Kern County was spurred by the discovery of gold in 1851. The initial population was centered in the mountain zone. When Kern County was organized in 1866, the first County seat was Havilah, located approximately 70 miles northeast of Bakersfield. The government center was transferred to Bakersfield in 1873, reflecting the gradual population shift to the valley.

The 1980 population of Kern County was 403,089. This represents approximately a 22-percent increase from the 1970 population of 330,234. Kern County is served by a number of major highways. Interstate Highway 5 and State Highway 99 are oriented along the north/south axis of the valley. State Highway 58 connects the valley to the Mojave Desert, and State Highway 14 and U.S. Highway 395 run east of the Sierra Nevada Range. Maintained County roads exceed 3,362 miles. In addition, there are ten County airports and two railroad lines.

Securing a reliable water supply has been a continuing problem for much of Kern County. Current need in the valley is partly met by supplies from surface-water impoundments, such as Lake Isabella on the Kern River, and from groundwater pumping. Additional water must be imported by the Central Valley Project and the California Aqueduct. Because of the need for drainage and irrigation works, farms in the valley traditionally have been, and continue to be, large. Farm size averages 1,752 acres.

The only major river in the County is the Kern River, with an average annual runoff of 700,000 acre-feet. The Kern River is the chief flooding source in the Bakersfield area.

Isabella Dam controls 2,074 square miles of the Kern River basin, but floodwaters can still accumulate from approximately 333 square miles of watershed downstream of the dam. Releases from Isabella Dam and natural runoff flow through a steep canyon for approximately 30 miles downstream of Isabella Reservoir, then flow southwesterly over the valley floor to Bakersfield. Twenty-one miles downstream of Bakersfield, the Kern River enters Buena Vista Lakebed or flows to Tulare Lake via the Kern River Flood Canal. Flow can also be diverted to the California Aqueduct via the Kern River-California Aqueduct Intertie, completed in 1977.

Many small streams also cause flooding problems in the valley when they discharge from steep canyons. These streams are confined and distinct in upland areas, but tend to spread out into a number of poorly defined drainages on the valley floor. Flow is further disrupted by cultivation and urbanization.

Similar problems occur in the desert area, where ephemeral streams flow across alluvial fans. Drainage channels are poorly defined and can shift from one flood event to the next. Streets, railroads, and other structures divert flows and may increase flooding problems.

Another type of flooding occurs in the mountains, where streams flow through alluvial valleys. In these areas, flows peak quickly and have high velocities.

Principal Flood Problems

Flooding results from a variety of conditions in Kern County. Intense storms can cause high flood peaks that pass rapidly, but can leave severely damaged property in their wake. Spring snow melt often results in high flows that have a longer period of runoff and a lower peak than rain floods. Overflow from poorly defined channels and inadequate culverts and drains can lead to shallow flooding, even from low-intensity rainfall.

Various areas in Kern County have suffered from numerous floods. Severe flooding occurred in November 1950, December 1966, January and February 1969, and September 1975. Localized flash flooding occurred in September 1976, and a large storm produced high flows on various streams in February 1978. A large storm in March 1983, in the mountains of the Caliente Creek watershed, caused extensive flooding of the Caliente Creek area near the City of Arvin and in the unincorporated community of Lamont. Thunderstorms in the Lake Isabella area, August 1984, produced severe damages, most dramatically in the unincorporated communities of Onyx and Weldon. Damages also occurred in the Bodfish and Southlake areas.

A series of storms during November 1950, produced widespread flooding throughout the Sacramento and San Joaquin River basins in California, and the Truckee, Carson, and Walker River basins in California and Nevada. At many localities, these floods were the most destructive experienced over the 40- or 50-year period of record. Peak flow on the Kern River at the First Point gage (located six miles upstream of Bakersfield) was 36,000 cubic feet per second (cfs). This flow occurred four years before Isabella Dam was

constructed and would have a recurrence interval of approximately 670 years under current, regulated conditions. Bakersfield was saved from damage by intensive flood fighting; however, part of neighboring Oildale, the Fruitvale Oil Field, and agricultural areas were flooded, and Gordon's Ferry Bridge was washed out. Approximately 37,000 acres were flooded along the entire length of the Kern River, and flood damage totaled nearly $2,000,000. Some areas along Poso and Caliente Creeks were also flooded.

An intense 48-hour storm on December 4 through 6, 1966, caused severe flooding in a 60- by 100-mile area in the Sierra Nevada Range northeast of Bakersfield. The Kern, Tule, and Kaweah River basins were affected. Damage was extensive in all headwater areas of these three stream systems. The road from Kernville upstream of Johnsondale along the Kern River was obliterated at the outside bank of many river bends, and the pavement was scoured away in other locations. Waterstage recorder structures and the measuring cableways on the Kern River at Kernville and South Fork Kern River near Onyx were destroyed by the flood. Further downstream, Isabella Dam minimized flood damage to the valley. Peak flow along the Kern River was 60,000 cfs, more than twice the previous maximum that occurred in November 1950. The COE estimated that, had Isabella Reservoir not been in operation, flow on the Kern River at the First Point gage would have been approximately 80,000 cfs. Actual flow was nearly 9,300 cfs, which has a recurrence interval of approximately 80 years. Areas along Caliente Creek were also flooded, resulting in $625,800 in damage to urban and rural property.

Storms occurring January 18 through 22 and 24 through 27, 1969, produced heavy rainfall that caused extensive flooding in the San Joaquin Valley. Flows in the Kern River at Kernville were one-third of the record flows in 1966, but were the third or fourth highest of record. A storm that lasted from February 22 to 24 brought heavy rain in the southern and western portions of the San Joaquin Valley. Poso Creek near Oildale had a peak discharge of 6,600 cfs on February 24, 1969. This flow has a recurrence interval of approximately 22 years. Nearly 38,000 acres were inundated by this stream. On February 25, El Paso Wash, just downstream of Inyokern Road in Ridgecrest, had an estimated flood peak of 350 cfs. This flow has a recurrence interval of approximately 13 years.

Major flooding occurred in Kern County on September 9 through 10, 1975. High-intensity thunderstorms passed over the northeastern part of the County causing flooding at Isabella and in the desert areas. Unusually high flood peaks, some estimated to be in excess of the 100-year flood, were gaged from high-water marks on various streams following the storm.

Between September 29 and October 1, 1976, flash flooding occurred within the South Fork Kern River and Kelso Creek Valleys and in the Piute Mountains. Flooding was caused by thunderstorms that moved easterly across the southern area of Lake Isabella. Erskine Creek had a peak discharge of 2,660 cfs causing damage to Uffert Park. This flood has a recurrence interval of approximately 28 years. The Kern County Water Agency Improvement District No. 3 levees along Kelso Creek were severely damaged by a peak discharge of 11,200 cfs. This flow has a recurrence interval of approximately 50 years and is 1.5 to 2 times the levee design flood.

A large storm in February 1978, produce a peak flow of approximately 8,300 cfs on the Kern River at the First Point gage, near Bakersfield. This flow has a recurrence interval of slightly greater than 70 years. Preliminary data from unpublished records of the U.S. Geological Survey and Kern County Water Agency indicate that the peak flow on Poso Creek near Oildale was 3,570 cfs on February 10, 1978. This flow has a recurrence interval of slightly less than 11 years.

Major flood problems on the eastern side of the City of McFarland result from the overflow of Poso Creek and runoff from the mountains east of McFarland. Runoff ponds behind the Friant-Kern Canal and flows southerly along the east canal bank. Both the overflow from Poso Creek and runoff from the canal flow northerly toward McFarland through a siphon beneath State Highway 99. East of State Highway 99, there are overland flows with an average depth of less than one foot.

In early March 1983, the Loraine area was severely flooded, causing damage to many of the structures in the floodplain. The bridge crossing of Indian Creek just above the confluence of Caliente Creek and Indian Creek was completely washed out, as was most of Caliente Creek Road below this confluence point. These problems were mainly due to flows approaching the 100-year discharge.

Flooding problems throughout Kern County are aggravated by undersized or insufficient drainage facilities. Even large culverts and bridges may become constricted by debris during flood events. In addition to flooding from washes, creeks, and rivers, many areas are subject to flooding from flows on alluvial fans. Both unpredictability of flow paths on alluvial fans and typically high sediment loads compound the problem.

APPENDIX VI
Methodological Applications Policies:

Policy #1: There may exist hydrologic cases for which the application of this manual's methodologies is less appropriate than a recommended alternative. Upon submittal of evidence indicating inconsistency of the methodologies described in this manual to create an adequate hydrologic model of the watershed under consideration and proof of consistency with the primary hydrologic parameters of this manual, such methodology may, at the sole discretion of the Director of the administering department, be accepted for use for the specific case intended. Such acceptance for one case will not obligate the department to accept the methodology in any other case.

Policy #1a: The methodologies described in this manual have been calibrated to an 85 percent confidence limit. Any alternative methodology shall be demonstrated to maintain this criterion, unless the methodology has been accepted (including any input parameters) for use in the region intended, by the Federal Emergency Management Agency (FEMA).

Policy #2: Rain gauge data may be used in lieu of NOAA Atlas 2 if the following criterion are true:

(a) The gauge is a recording gage with 1-hour readings.
(b) The gauge has a good record length of 25 years or more.
(c) The project is within close proximity to the gauge (i.e., 3 miles distance, and 200-foot elevation).
(d) All other rain gauges with adequate record length (10+ years) within 15 miles of the project are also considered.

Policy #3: The determination of soil classifications, in areas not mapped by the Soil Conservation Service, is the responsibility of the hydrologist's soils engineer. If the hydrologist does not wish to have a soils classification prepared, the soil shall be assumed to be type "D" for runoff determinations.

Policy #4: Figures APP-6.1 through APP-6.19 may be used as the short duration "Intensity-Duration" curves for the County areas designated as requiring Type "A" improvements, per the Kern County Land Division Ordinance, dated January 1990.

Policy #5: Figure APP-6.20 identifies the regional limits of the USGS Regional Regression Equations. Figure APP-6.21 or Equation APP6.1 may, after consultation and approval by the County for it's use, be used to determine the 100 year return period flow for undeveloped watersheds with areas less than 20 square miles. Approval for use of this chart is at the sole discretion of the County, and will be based upon the showing of special circumstances which warrant the use of this regression equation. Under NO circumstances will this chart be used for developed or developing watersheds.

Policy #6: The HEC-1, with the LAPRE1 data preprocessor, hydrologic computer software are acceptable for use on hydrograph studies for submittal to the County. The following input parameters shall be used:

1) HEC-1:
The RC, with RX and RY, records (normal depth routing), shall be used for hydrograph routing.

2) LAPRE1:

$P -	Field 1, PRECIP	=	15
	Field 2, DRAIN	=	input the basin area
	Field 3 - 8	=	0
$Q -	Field 1 - 6	=	use the information from item 8, Figure E-7
	Field 7,	=	input $\bar{Y}$
	Field 8,	=	input F_m
$U -	Field 1 - 4	=	input values for use with (E.2). BN, the subarea "n" value (Field 4), should be adjusted so that $C_t(\frac{L \cdot L_{CA}}{\sqrt{s}})^m = 0.8 \cdot T$
	Field 5, NNT	=	1, for Mountain
		=	2, for Valley (Undeveloped)
		=	9, for Desert
		=	11, for Foothill
		=	23, for Valley (Developed)
	Field 6, 7	=	0

Policy #7: The requirement that, "Computations of travel time through subareas which continually add runoff to the peak flow (e.g., streetflow) should be based on the average peak flow through the subarea", is considered to be necessary only when a significant increase in velocity occurs. Velocity increases in excess of 10% are considered significant. In general, a 10% increase in velocity is associated with a 75% increase in flow, in street cross-sections.

Policy #8: The calculation of 5-year recurrence interval flows using the rational method may be based upon the use of the following equations;

$Q = 0.8(I - Fm)A$
$Q = 0.8YIA$

These modified equations are based upon an adjustment between the

50% and 85% confidence intervals.

Policy #9: Section 408-1. Retention Basin Design Volume

The design volume of storm water retention basins shall be based upon the runoff from the ISDD five-day storm event and a volume of nuisance water determined by the engineer. No runoff generated on site from the design storm or from nuisance flows will be allowed to leave the site unless downstream drainage disposal facilities exist to handle the flow. The retention of upstream off-site flows shall not be considered to reduce the size of the required on-site retention facilities or mitigate the runoff from the proposed development. An evaluation of the runoff volumes associated with the site in its existing condition shall not reduce the size of the required drainage facilities. The runoff volume from the ISDD five-day storm shall be calculated using the formula:

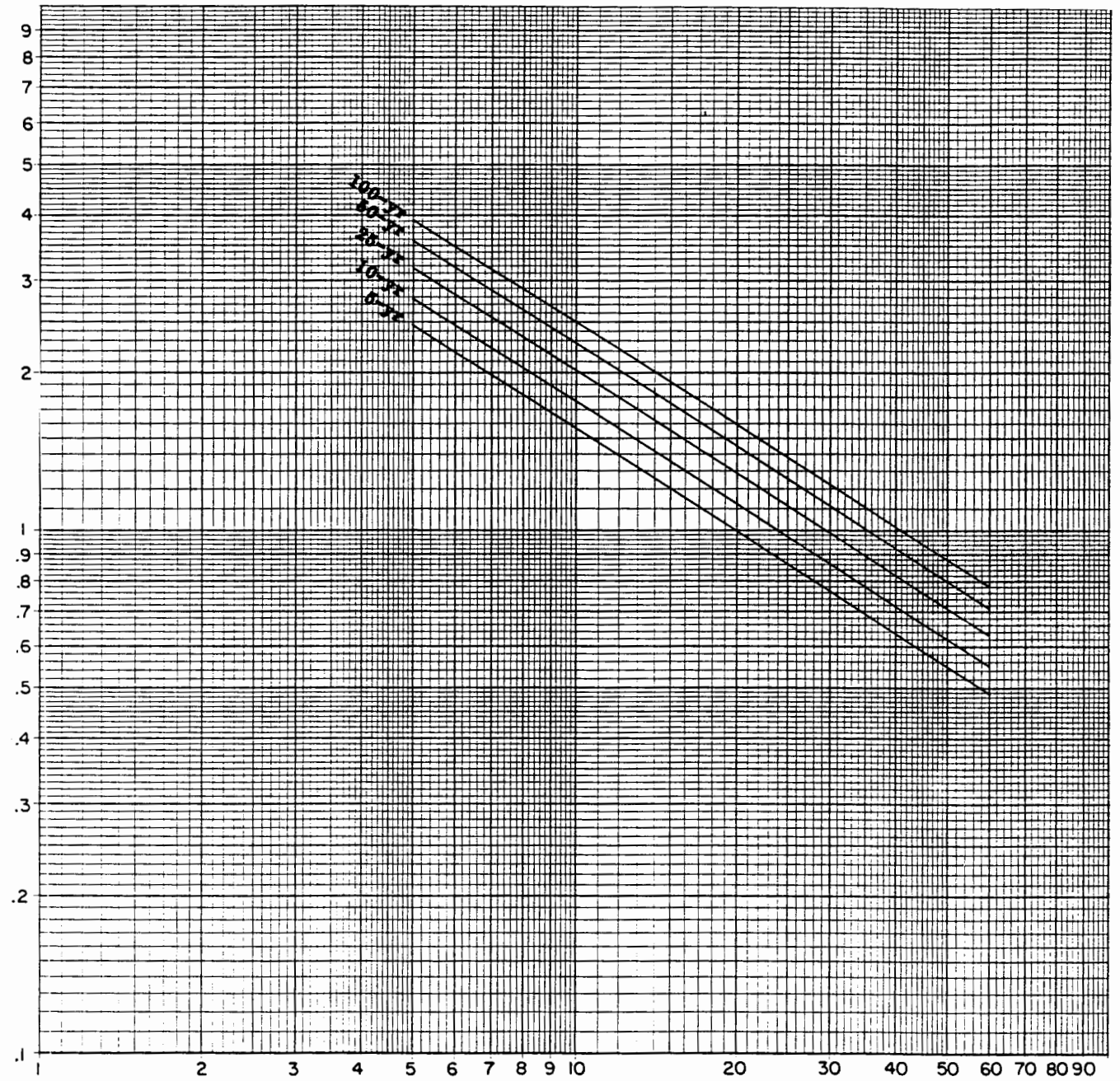
$$\text{Runoff Volume} = .12\ (D_{10})\ (a_i)\ (\text{Area})$$

D_{10} = 10 yr 24-hr. depth of rainfall (in.)

a_i = average percentage of impervious area

Area = Drainage area of total development

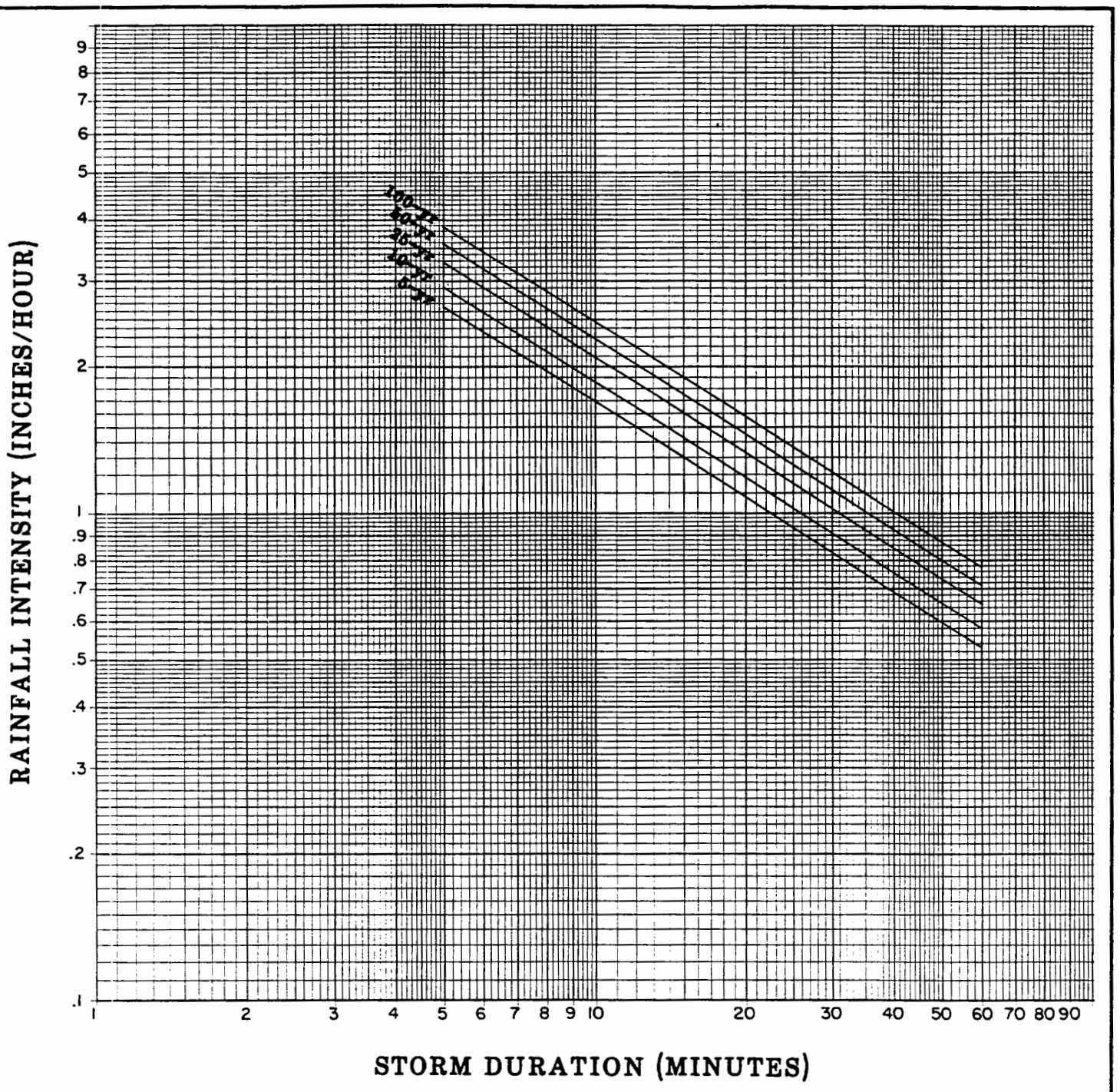
Per Kern County Development Standards, August 1995



PROJECT LOCATION: Arvin

KERN COUNTY
Hydrology Manual

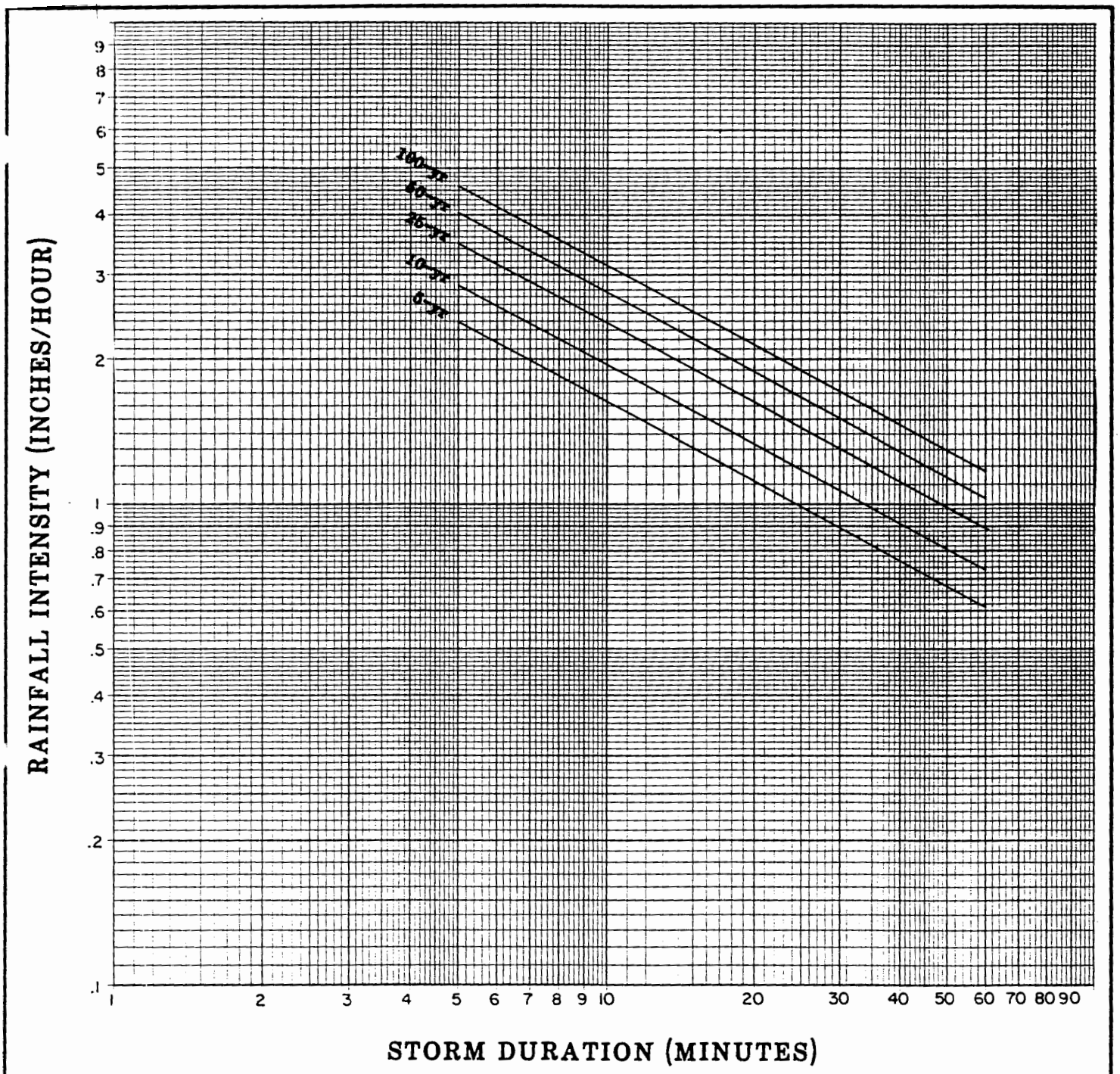
RAINFALL
INTENSITY-DURATION
FREQUENCY (IDF) CHART



PROJECT LOCATION: Bakersfield

KERN COUNTY
Hydrology Manual

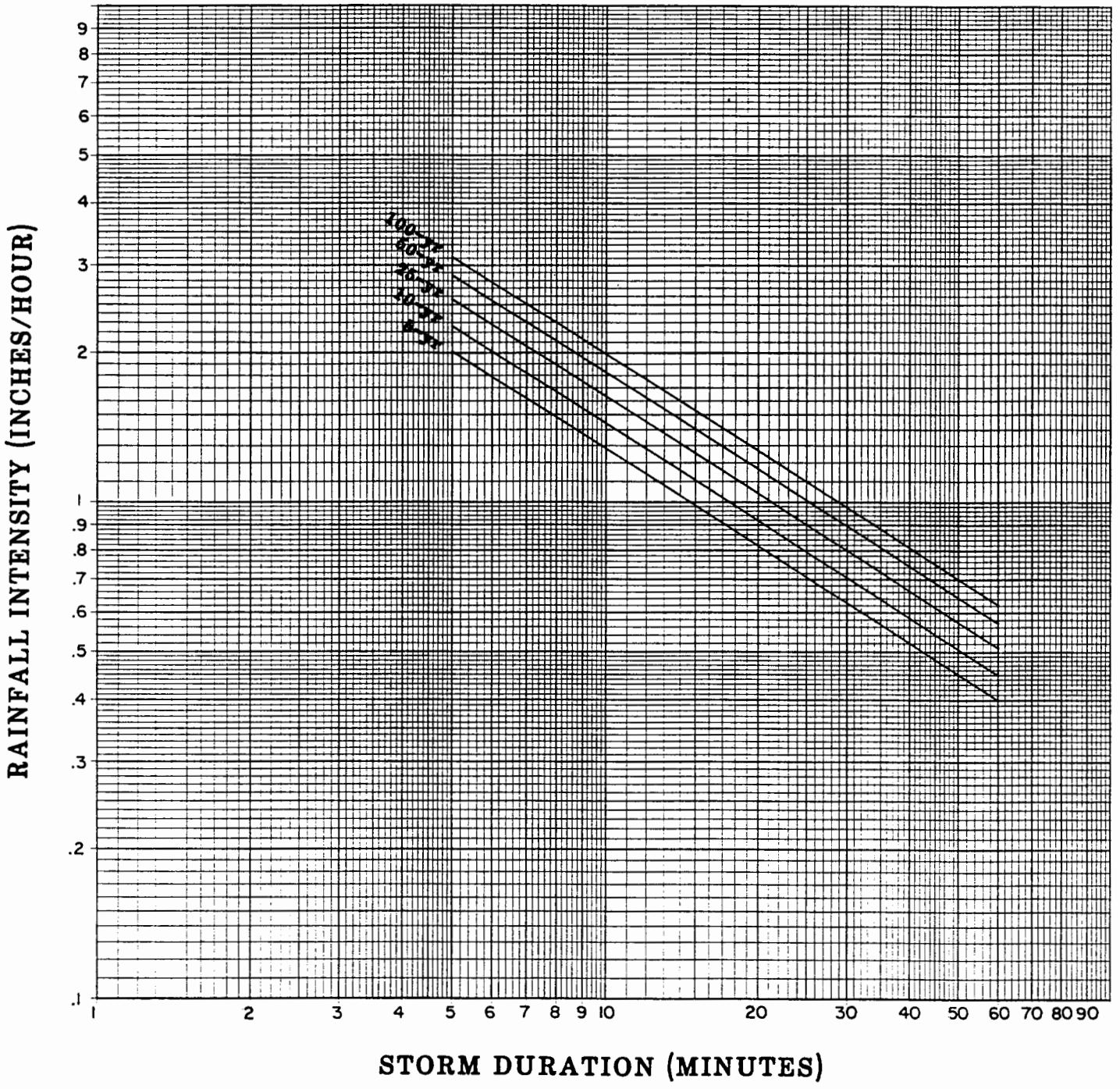
RAINFALL
INTENSITY-DURATION
FREQUENCY (IDF) CHART



PROJECT LOCATION: Boron

KERN COUNTY
Hydrology Manual

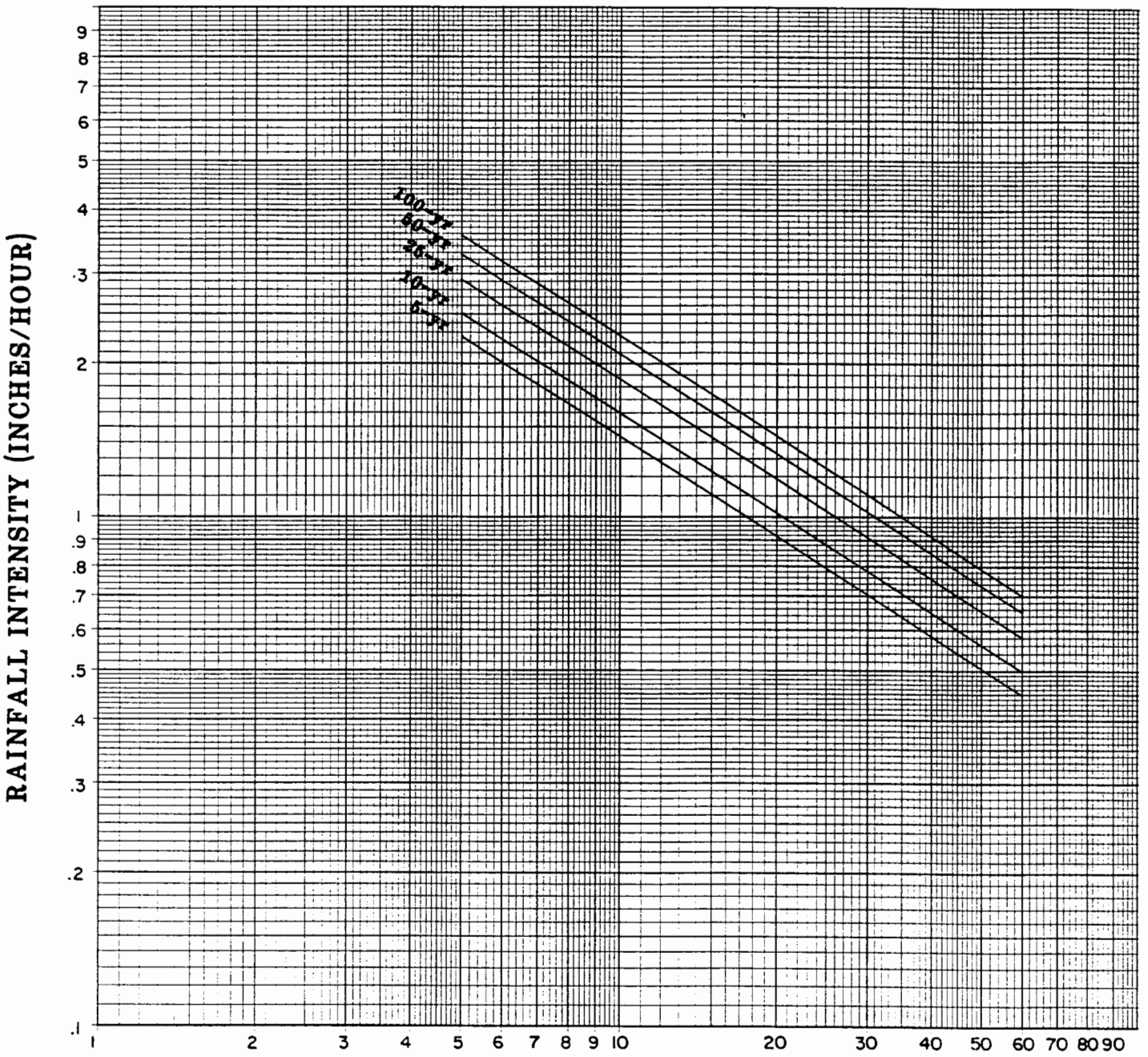
RAINFALL
INTENSITY-DURATION
FREQUENCY (IDF) CHART



PROJECT LOCATION: Buttonwillow

KERN COUNTY
Hydrology Manual

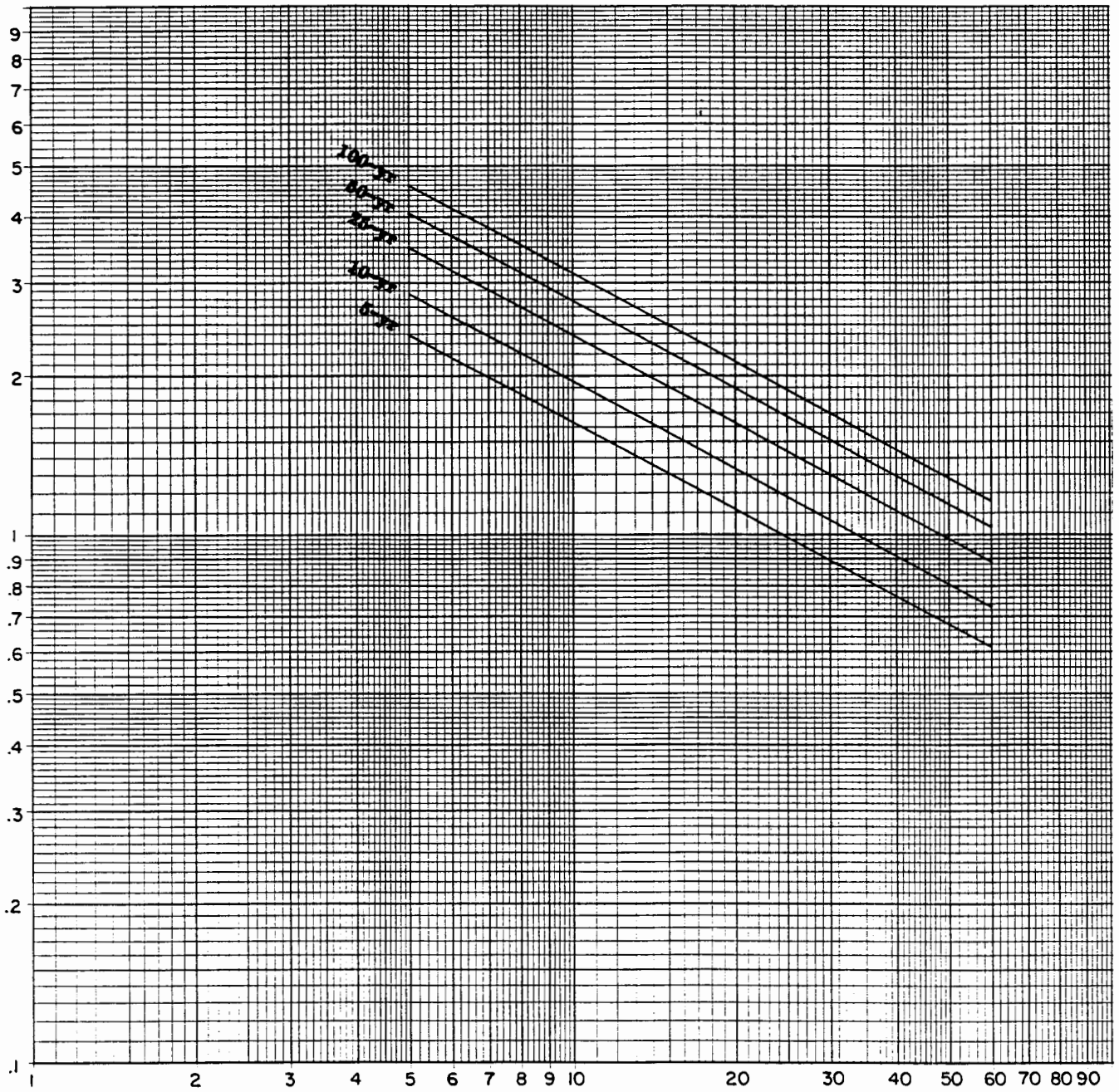
RAINFALL INTENSITY-DURATION FREQUENCY (IDF) CHART



PROJECT LOCATION: Delano

KERN COUNTY
Hydrology Manual

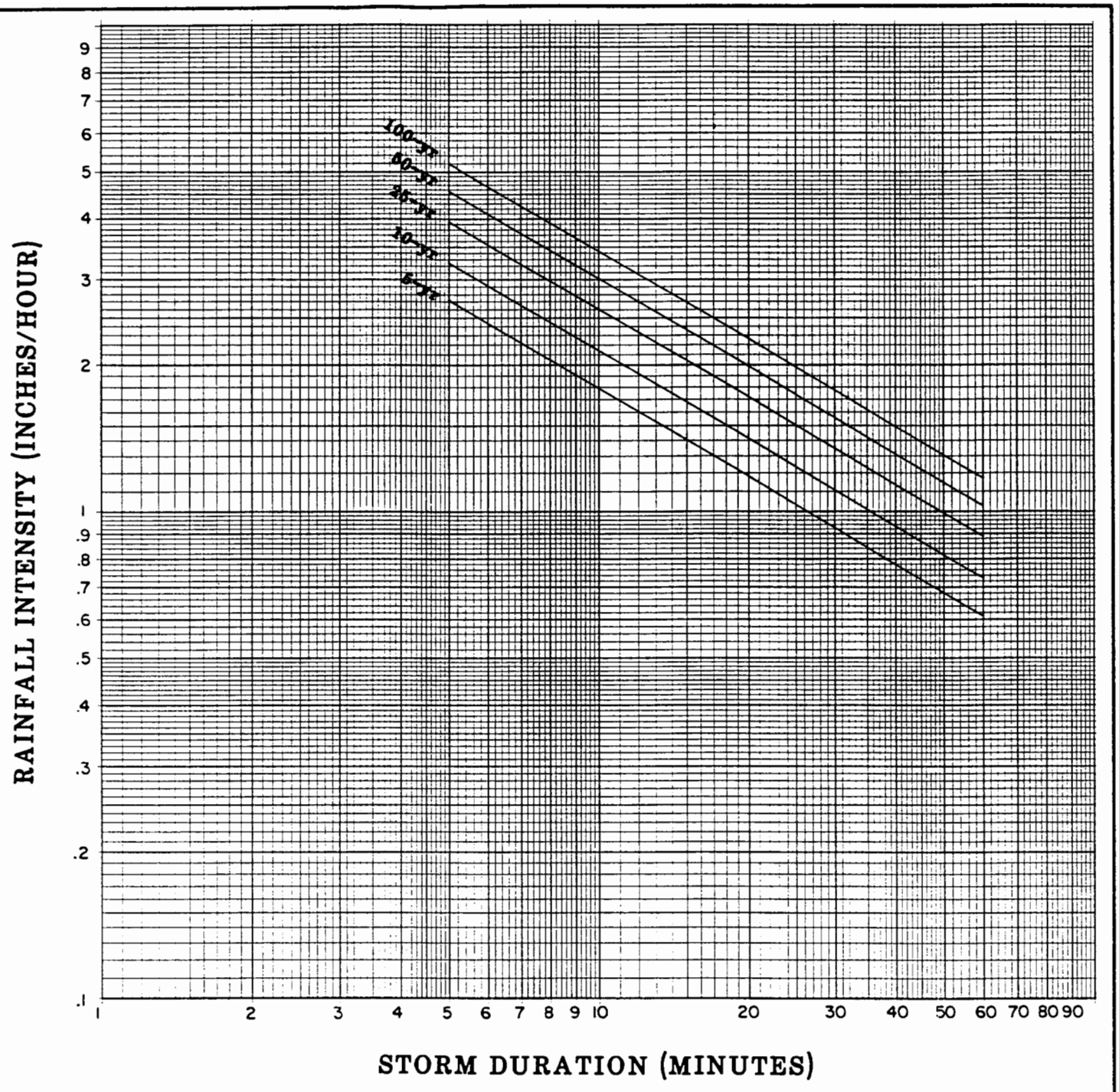
RAINFALL INTENSITY-DURATION FREQUENCY (IDF) CHART



PROJECT LOCATION: Desert Lake

KERN COUNTY
Hydrology Manual

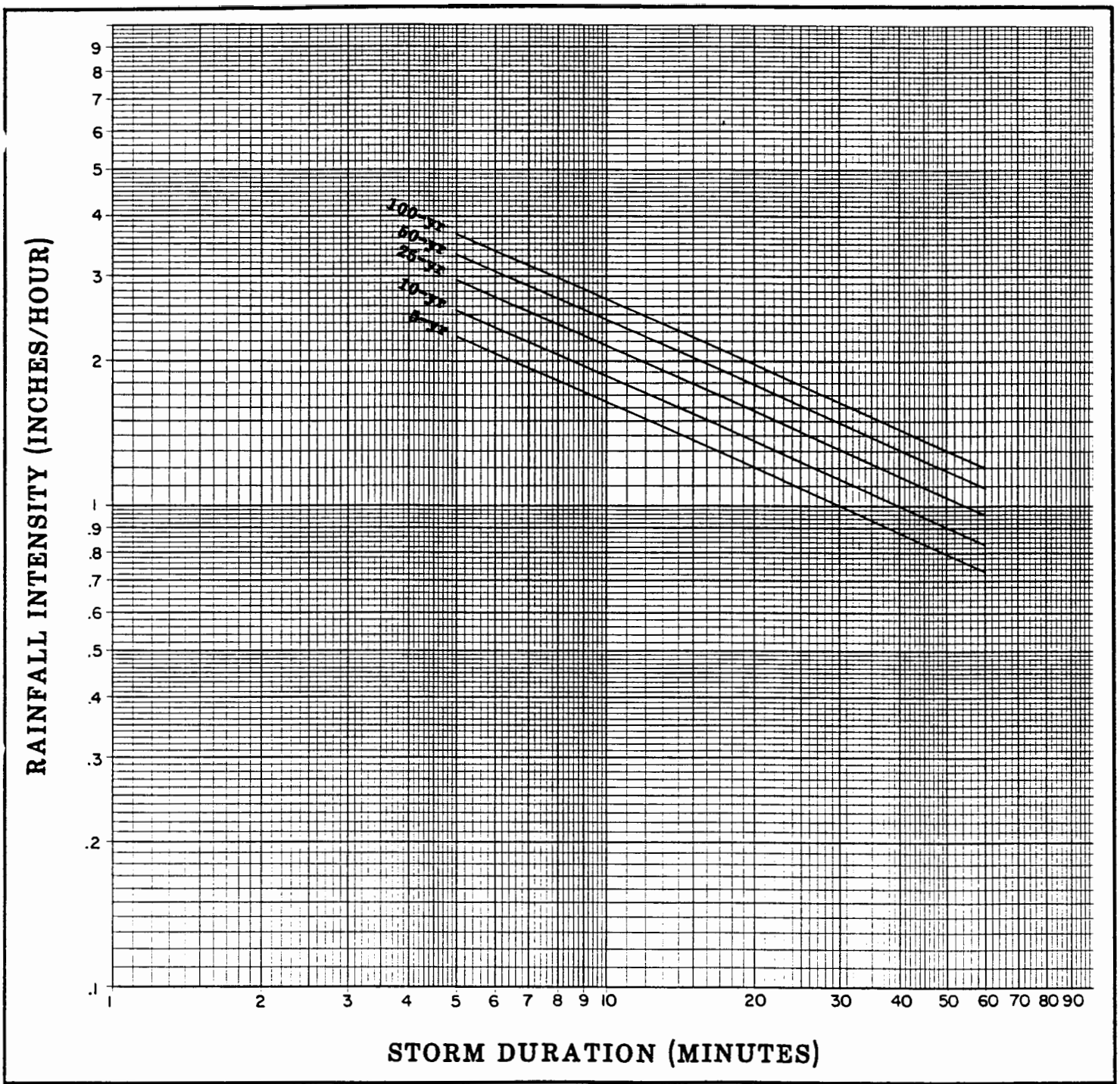
RAINFALL INTENSITY-DURATION FREQUENCY (IDF) CHART



PROJECT LOCATION: Inyokern

KERN COUNTY
Hydrology Manual

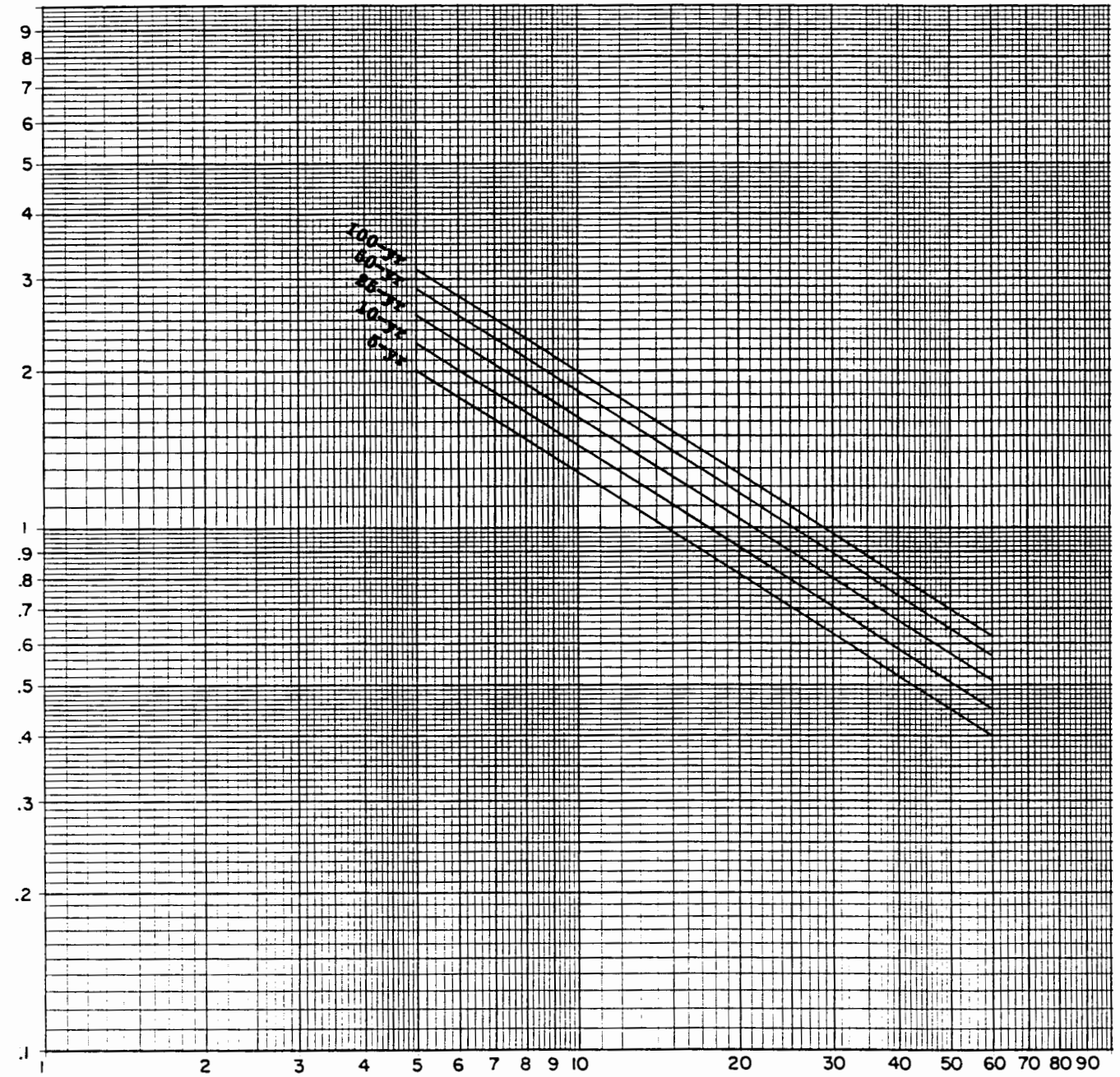
RAINFALL INTENSITY-DURATION FREQUENCY (IDF) CHART



PROJECT LOCATION: Kernville

KERN COUNTY
Hydrology Manual

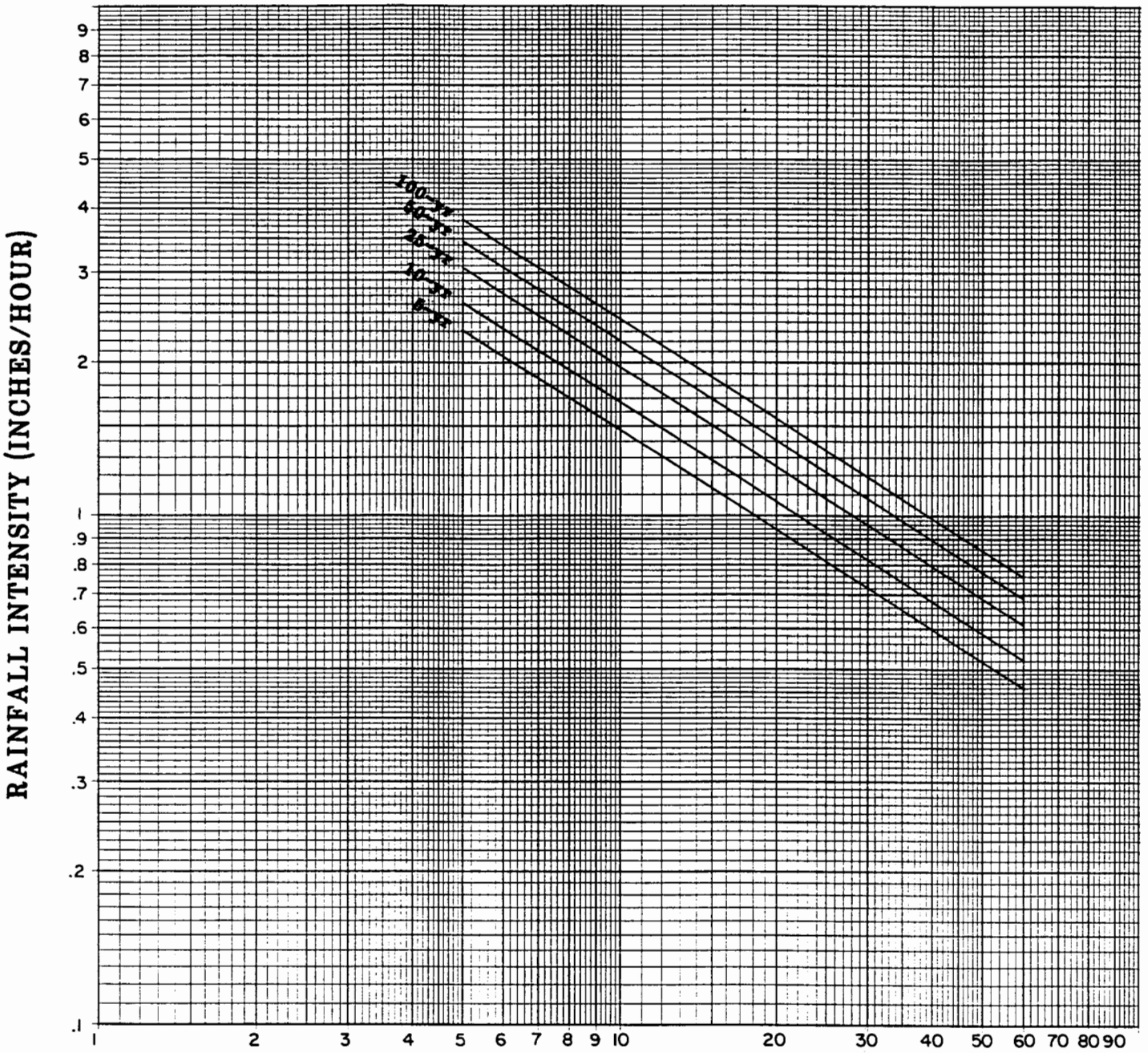
RAINFALL
INTENSITY-DURATION
FREQUENCY (IDF) CHART



PROJECT LOCATION: Lost Hills

KERN COUNTY
Hydrology Manual

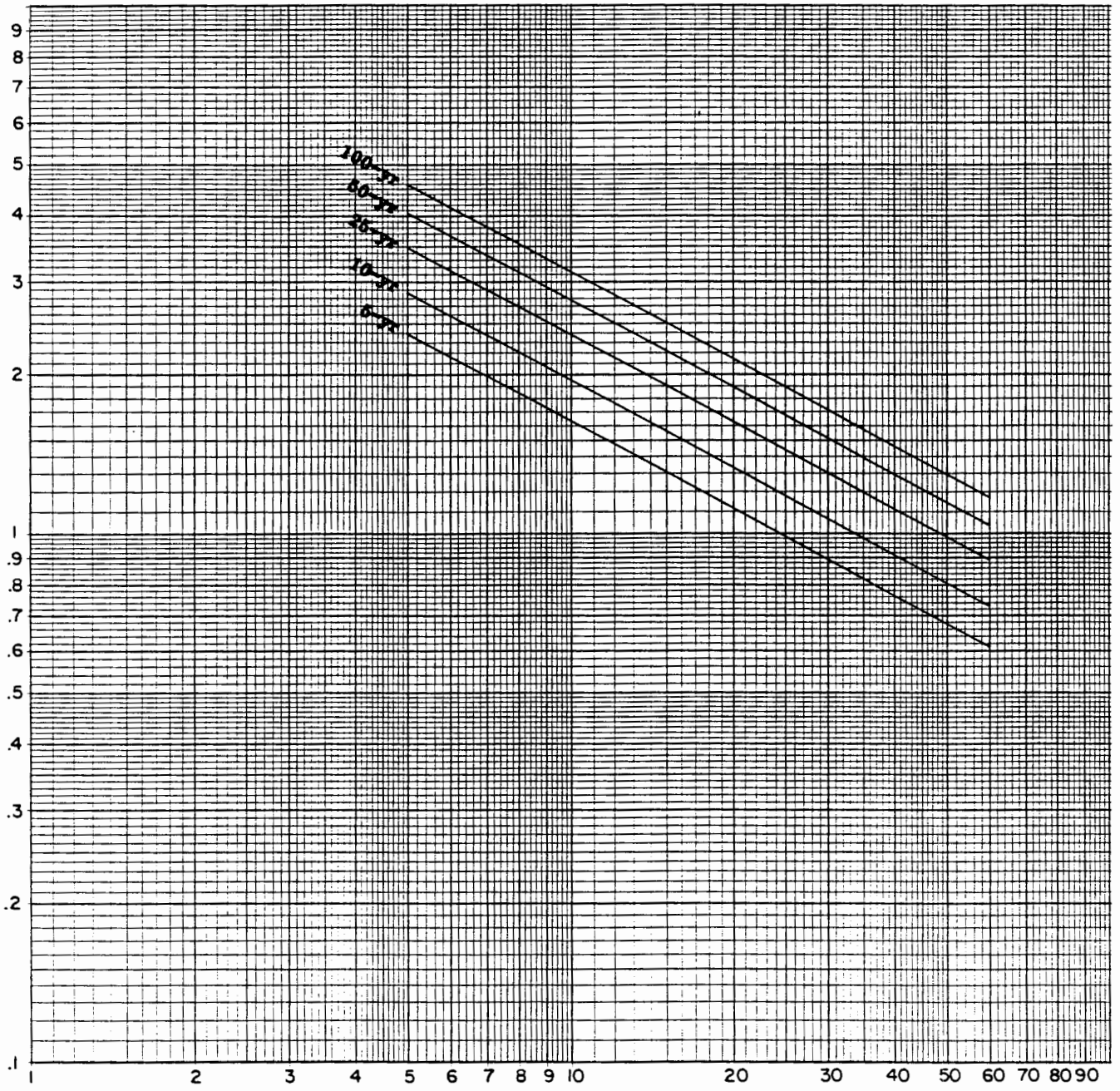
RAINFALL
INTENSITY-DURATION
FREQUENCY (IDF) CHART



PROJECT LOCATION: McFarland

KERN COUNTY
Hydrology Manual

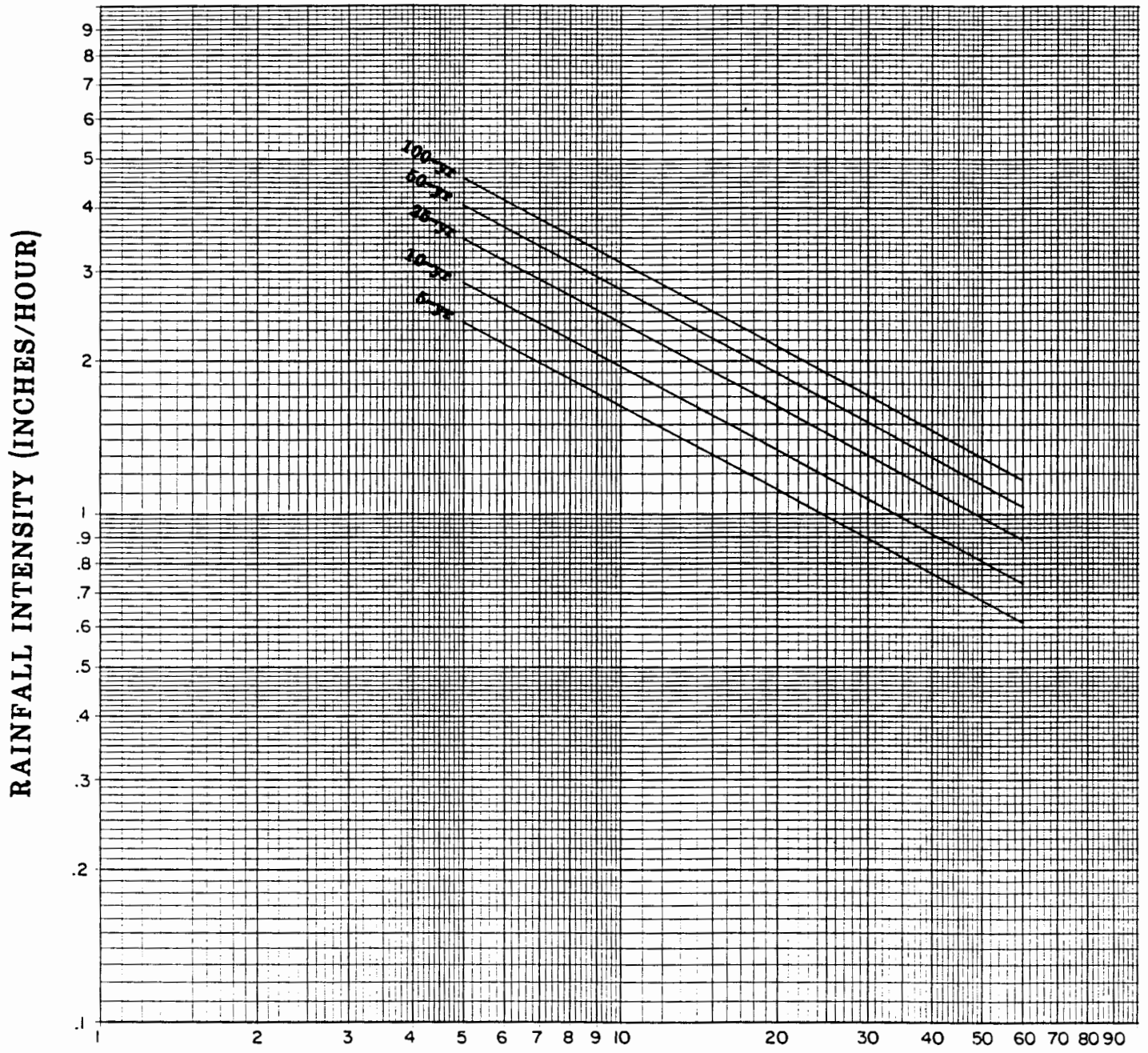
RAINFALL INTENSITY-DURATION FREQUENCY (IDF) CHART



PROJECT LOCATION: **Mojave**

KERN COUNTY
Hydrology Manual

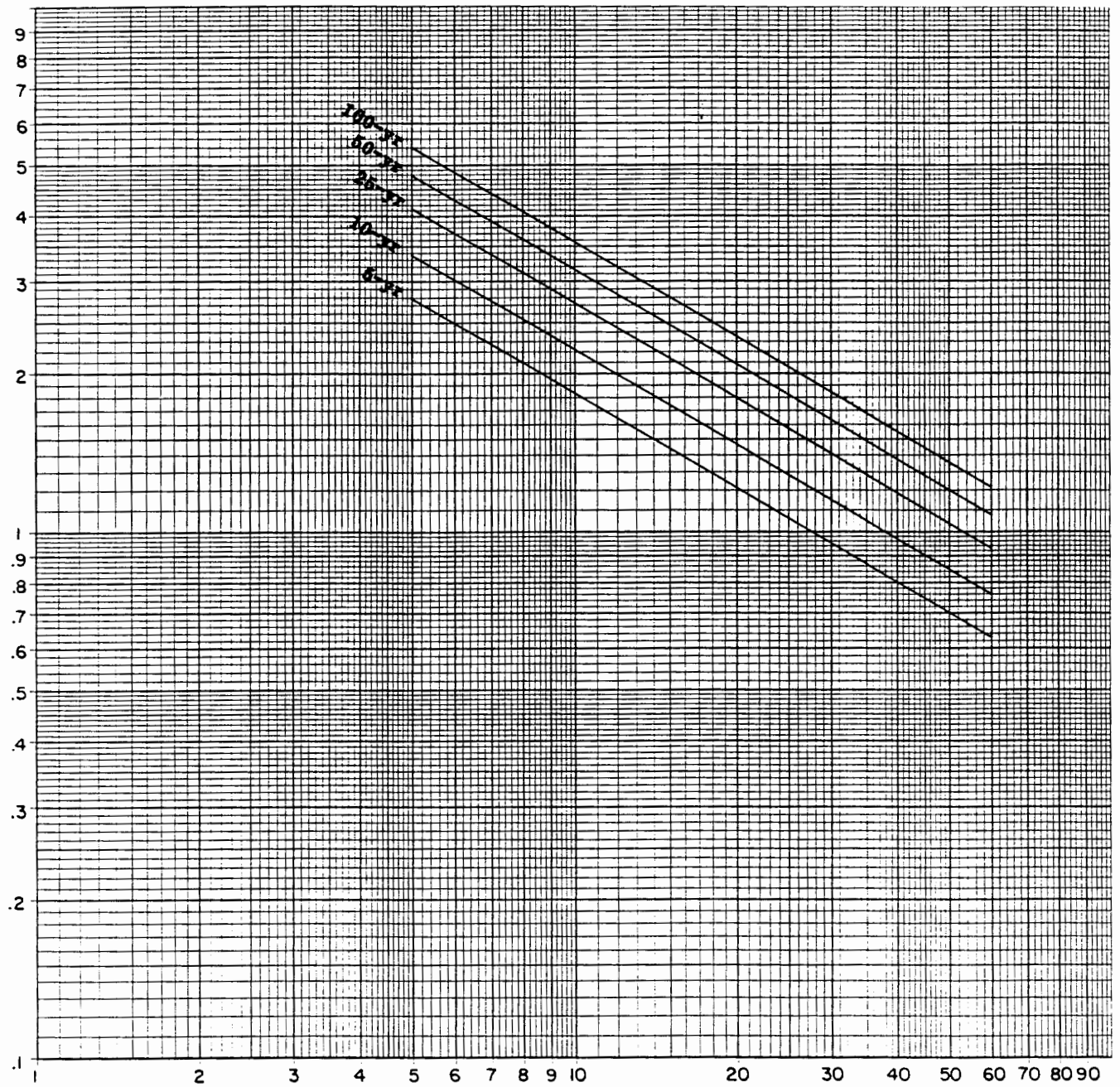
RAINFALL INTENSITY-DURATION FREQUENCY (IDF) CHART



PROJECT LOCATION: North Edwards

KERN COUNTY
Hydrology Manual

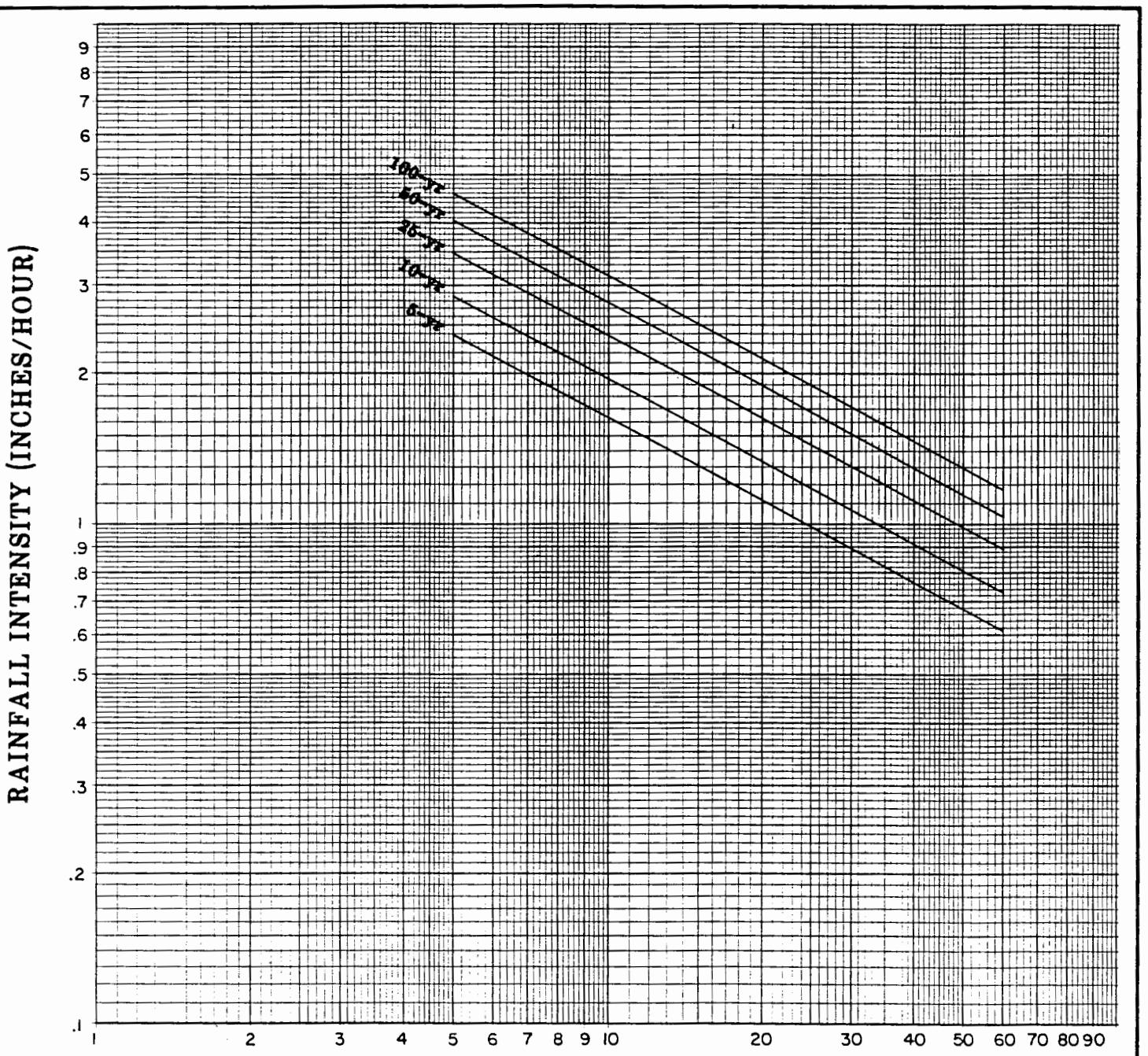
RAINFALL INTENSITY-DURATION FREQUENCY (IDF) CHART



PROJECT LOCATION: Ridgecrest

KERN COUNTY
Hydrology Manual

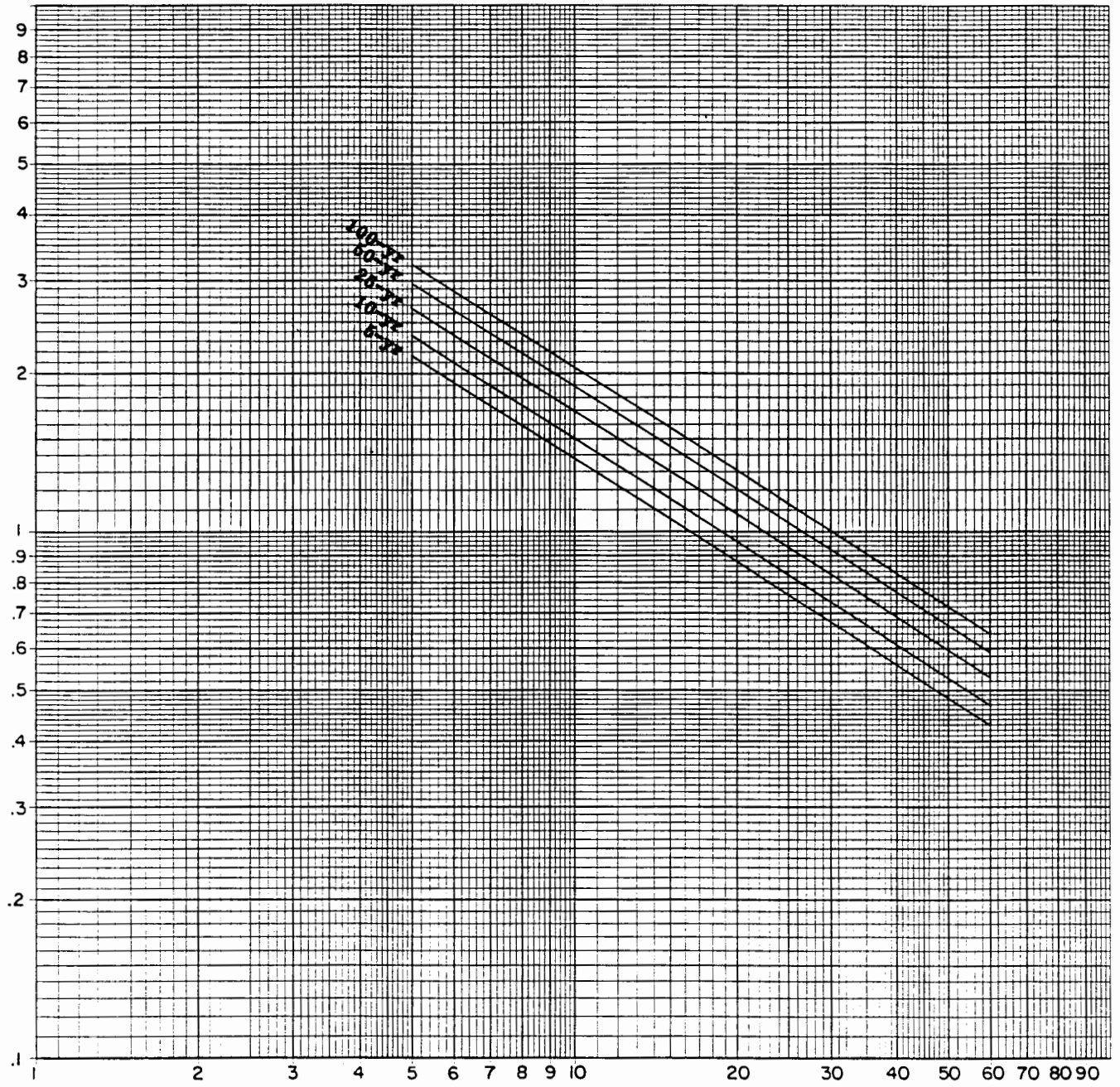
RAINFALL
INTENSITY-DURATION
FREQUENCY (IDF) CHART



PROJECT LOCATION: Rosamond

KERN COUNTY
Hydrology Manual

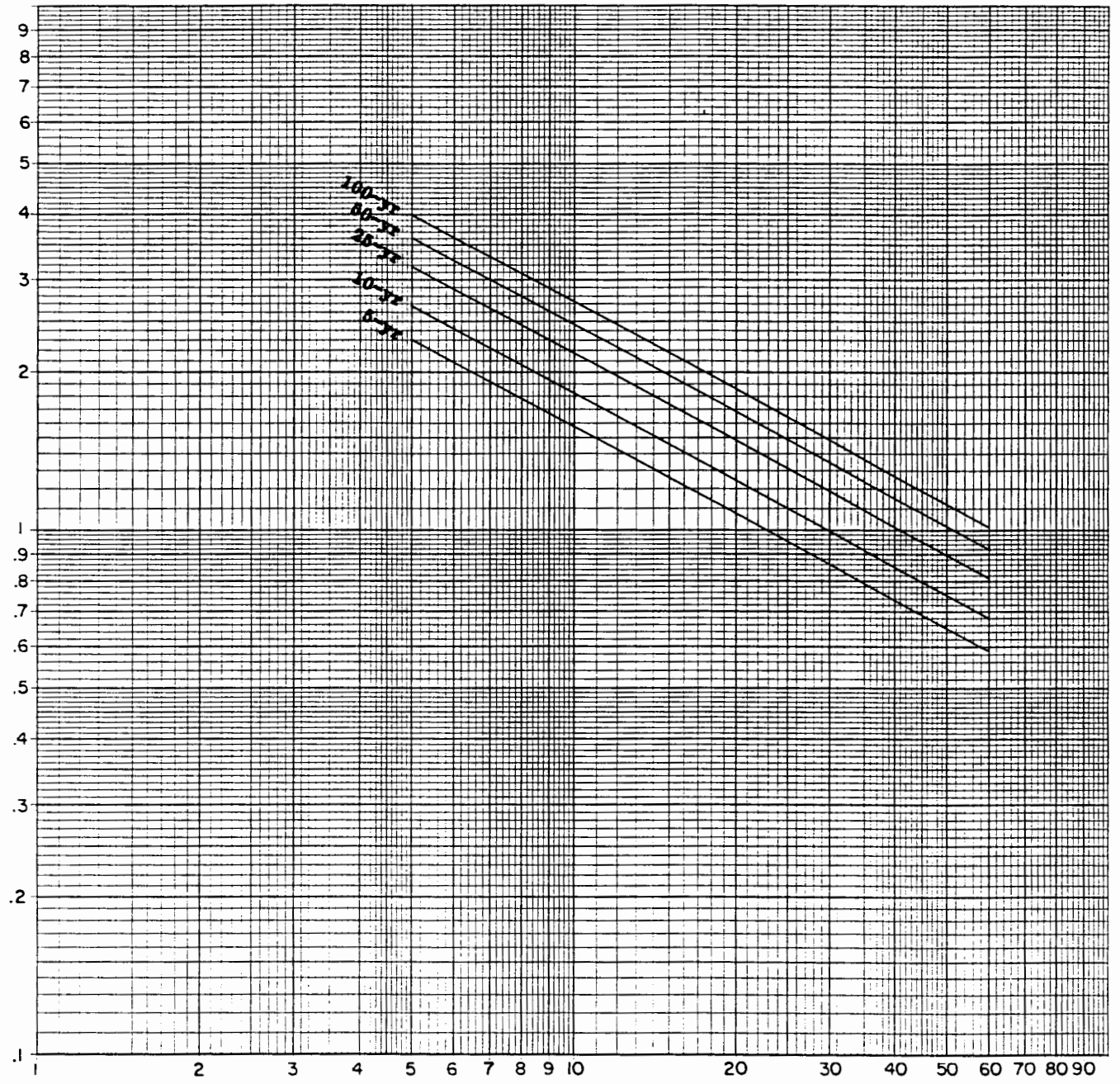
RAINFALL
INTENSITY-DURATION
FREQUENCY (IDF) CHART



PROJECT LOCATION: Shafter

KERN COUNTY
Hydrology Manual

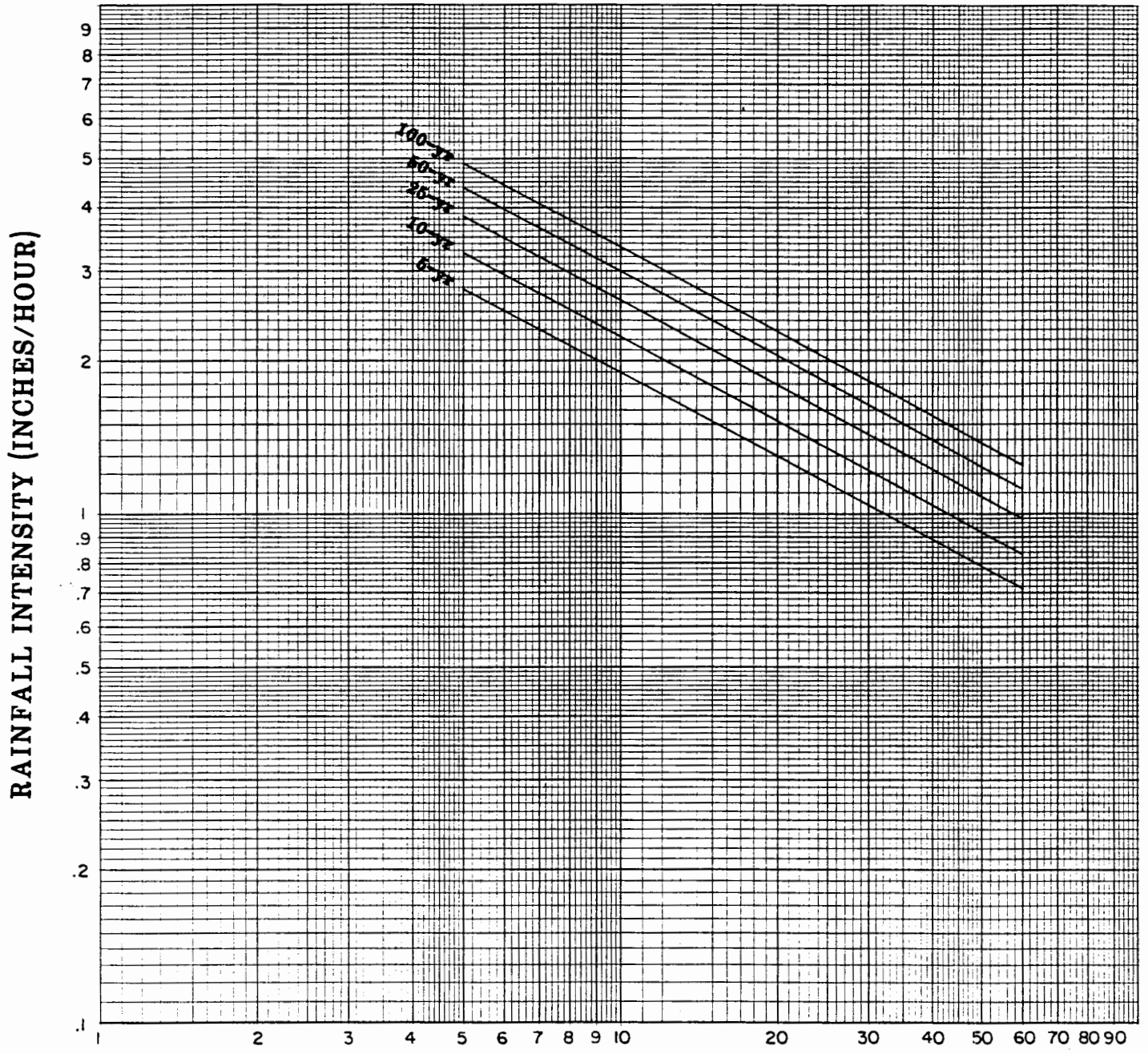
RAINFALL INTENSITY-DURATION FREQUENCY (IDF) CHART



PROJECT LOCATION: Taft

KERN COUNTY
Hydrology Manual

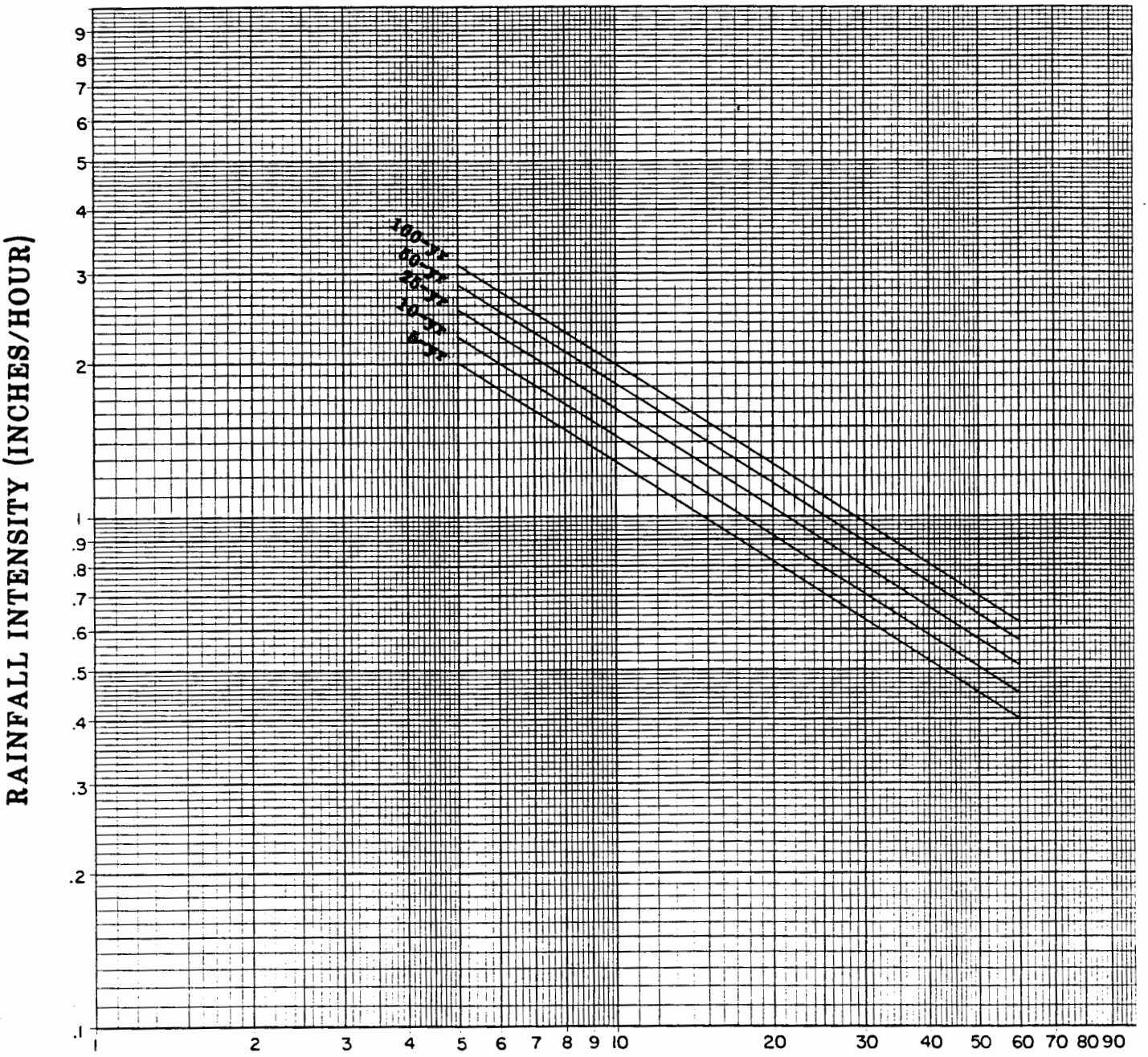
RAINFALL
INTENSITY-DURATION
FREQUENCY (IDF) CHART



PROJECT LOCATION: Tehachapi

KERN COUNTY
Hydrology Manual

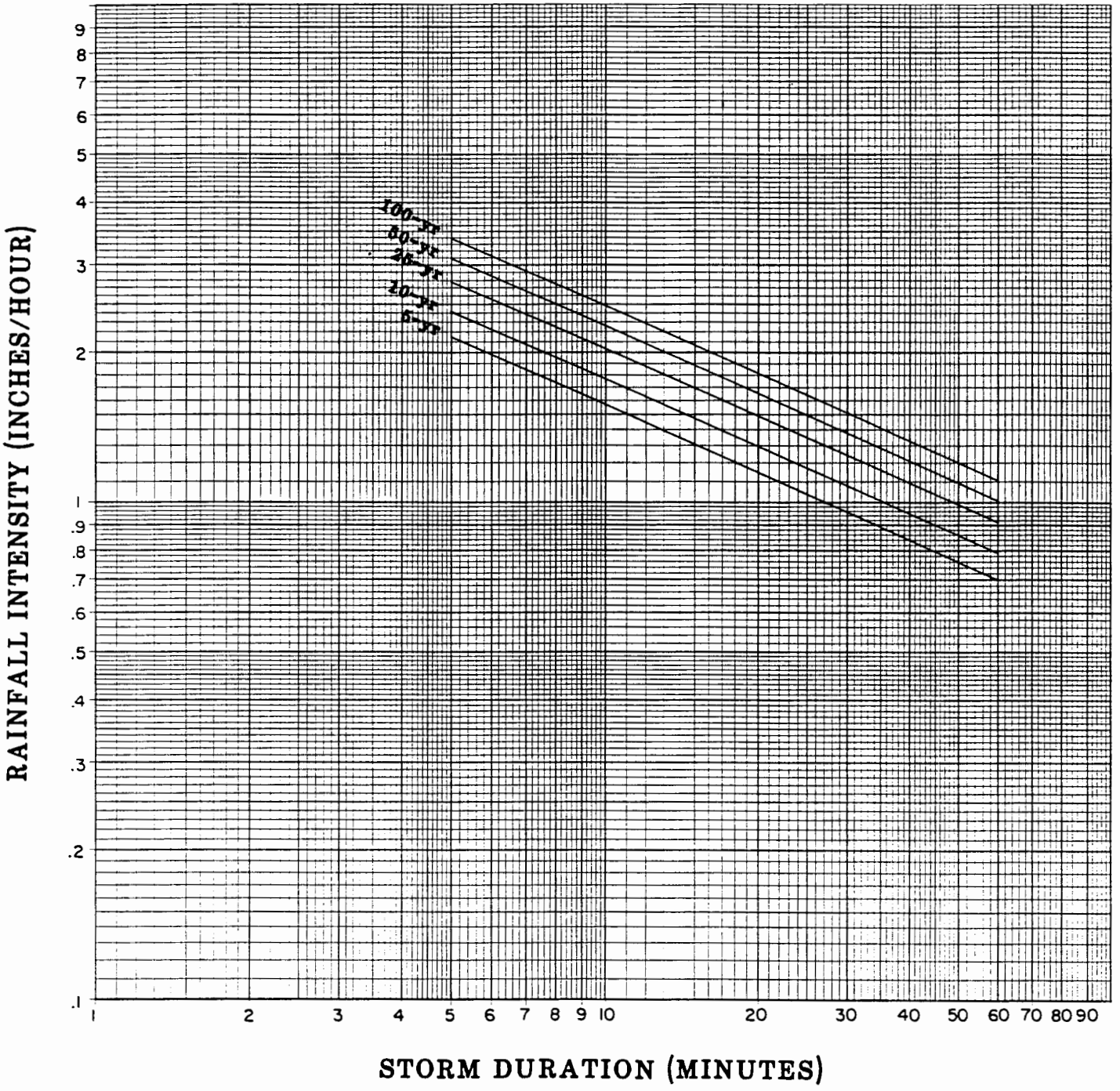
RAINFALL
INTENSITY-DURATION
FREQUENCY (IDF) CHART



PROJECT LOCATION: Wasco

KERN COUNTY
Hydrology Manual

RAINFALL
INTENSITY-DURATION
FREQUENCY (IDF) CHART



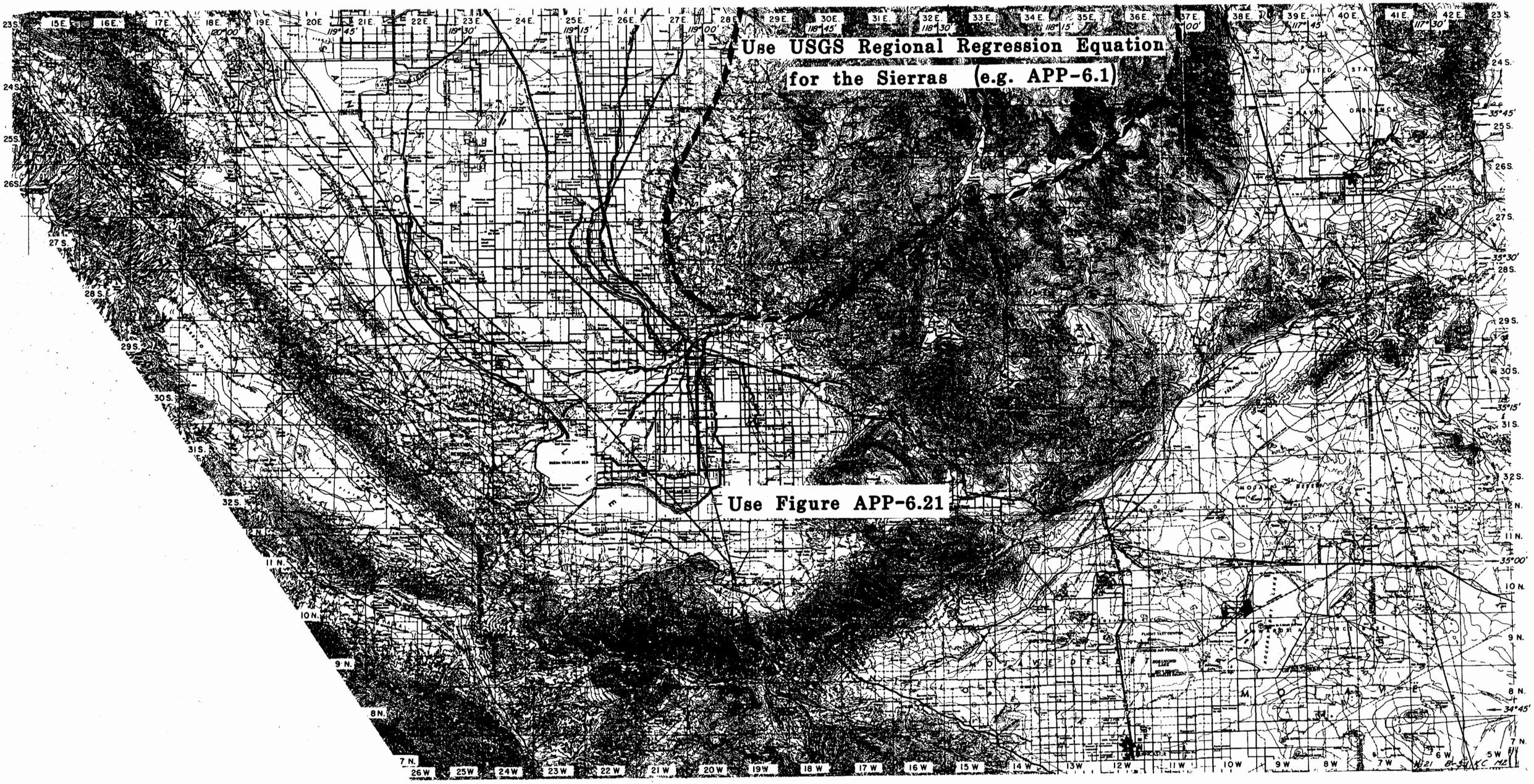
PROJECT LOCATION: Wofford Heights

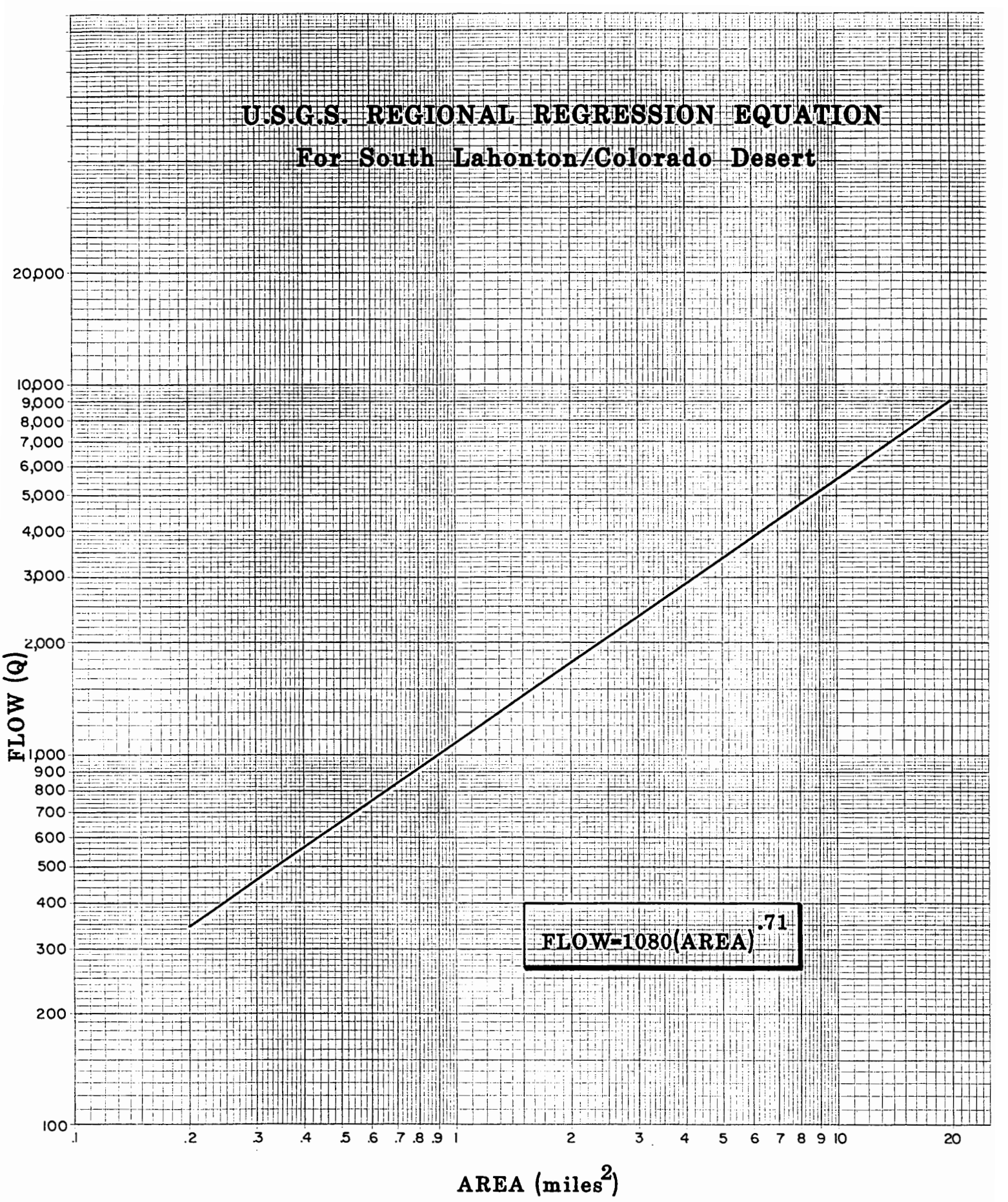
KERN COUNTY
Hydrology Manual

RAINFALL INTENSITY-DURATION FREQUENCY (IDF) CHART

USGS REGIONAL REGRESSION EQUATION

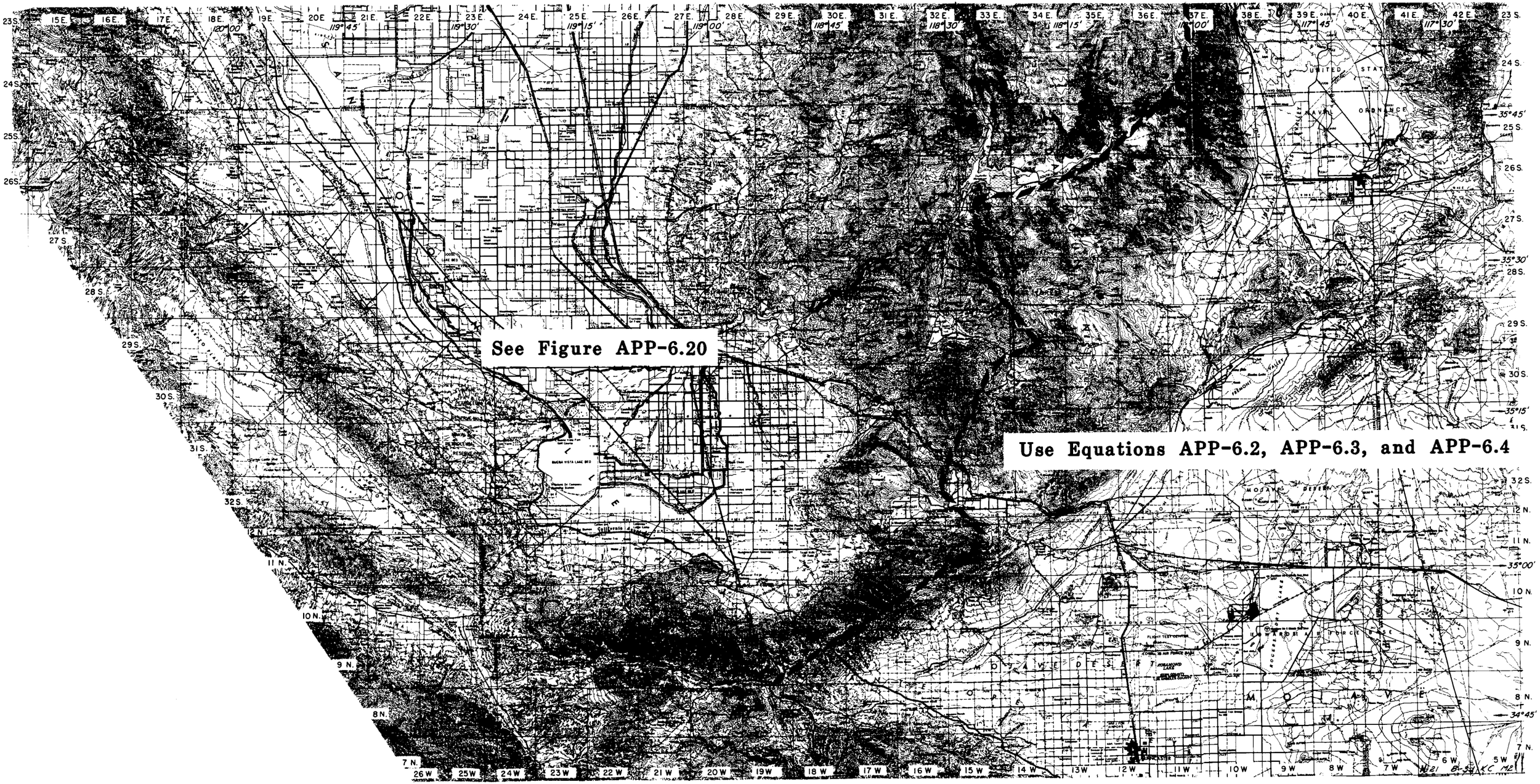
Regional Delineation




U.S.G.S. REGIONAL REGRESSION EQUATION
For South Lahonton/Colorado Desert
FLOW (Q)
20,000
10,000
9,000
8,000
7,000
6,000
5,000
4,000
3,000
2,000
1,000
900
800
700
600
500
400
300
200
100
.1
.2
.3
.4
.5
.6
.7
.8
.9
1
2
3
4
5
6
7
8
9
10
20
AREA (miles^2)
FLOW=1080(AREA)$^{.71}$

REGIONAL REGRESSION EQUATIONS

South Lahonton-Colorado Desert, Antelope Valley Sub-region



EQUATION APP6.1

$$Q_{100} = 15.7 \cdot A^{0.77} \cdot P^{1.02} \cdot H^{-0.43}$$

Where:

A = Drainage area in square miles

P = Mean annual precipitation in inches

H = Altitude index, in thousands of feet, computed as the average of the altitudes at the 10% and 85% points along the main channel used to compute channel slope.

REGIONAL REGRESSION EQUATIONS
South Lahonton-Colorado Desert Region
Antelope Valley Sub-Region[1]

Recurrence Interval, in years	Equation
2	APP6.2: $Q = 12 \cdot A^{0.58}$
5	APP6.3: $Q = 85 \cdot A^{0.59}$
10	APP6.4: $Q = 200 \cdot A^{0.62}$
25	APP6.5: $Q = 400 \cdot A^{0.65}$
50	APP6.6: $Q = 590 \cdot A^{0.67}$
100	APP6.7: $Q = 850 \cdot A^{0.69}$

Where:

A = Drainage area in square miles

[1] Taken from *Methods for Estimating Magnitude and Frequency of Floods in the Southwestern United States,* USGS, Open-File Report 93-419. Tucson, AZ. 1994

Regional Regression Equations
South Lahonton-Colorado Desert Region
Antelope Valley Sub-Region
FEMA 1992

Equation APP6.2	2-year flow	$Q_2 = 6.0 \cdot A^{0.27}$
Equation APP6.3	10-year flow	$Q_{10} = 97 \cdot A^{0.46}$
Equation APP6.4	100-year flow	$Q_{100} = 662 \cdot A^{0.62}$

A = Watershed area in square miles

APPENDIX VII

Alternative Rational Method With "C" Factor Curves

Kern County, recognizing the fact that a general lack of familiarity exists with the use of the Rational Method as described in Section D, of this manual, includes as this appendix an alternative form of Rational Method using "C" factor curves.

The use of a "C" factor curve has certain mathematical limitations which limit the application of the alternative method described in this appendix. If **ANY** of the following criterion are found to be true then this appendix's method **SHALL NOT** be used, and the methods described in Section D shall be applied.

Criterion for using this alternative method:

1) Is there an off-site undeveloped watershed contributory to this development?

2) Is the development non-homogeneous with regards to development type (i.e. a portion 4 DU's per acre, and a portion commercial)?

3) Is the development non-homogeneous with regards to SCS soil type (i.e. a portion type A soil and a portion type B soil)?

4) Is the watershed greater than 640 acres?

If the answer to ANY of the above question is YES, then this method SHALL NOT be used.

These restrictions are necessary for the following reasons:

1) The loss rates which the "C" curves are based upon are a function of SCS curve numbers for landscaped areas. Undeveloped soil cover complexes will, generally have different loss rates, and as such these curves are not applicable to an undeveloped watershed.

2) The existence of a non-homogeneous development type situation requires the generation, and use for downstream catchments, of intermediate curves, and as such voids the curves presented.

3) The existence of a non-homogeneous soil type situation requires the generation, and use for downstream catchments, of intermediate curve sets, and as such voids the curve sets presented.

4) The rational method SHALL NOT be used on watersheds greater than 640 acres.

Alternative Rational Method Procedure

Step 1 Measure the drainage area in acres, the length and fall of the watershed from the divide point to the catchment point in feet.

Step 2 Determine the time of concentration for initial area (not to exceed 10 acres in area or a travel length of 1000 feet) using Figure D-1. When the size of the watershed is large, the watershed shall be subdivided into reaches of reasonable length and the average velocity of flow in each reach used to determine the time through each reach. The sum of the initial time plus the time through each reach will equal the time of concentration for the point in question.

Step 3 Using the procedures described in Section B, or an approved alternative, prepare a short duration Intensity-Duration curve for the development area (see Figure D-4).

Step 4 Determine the intensity (i) value by intercepting the Intensity-Duration curve at the time of concentration.

Step 5 Determine the hydrologic soil group.

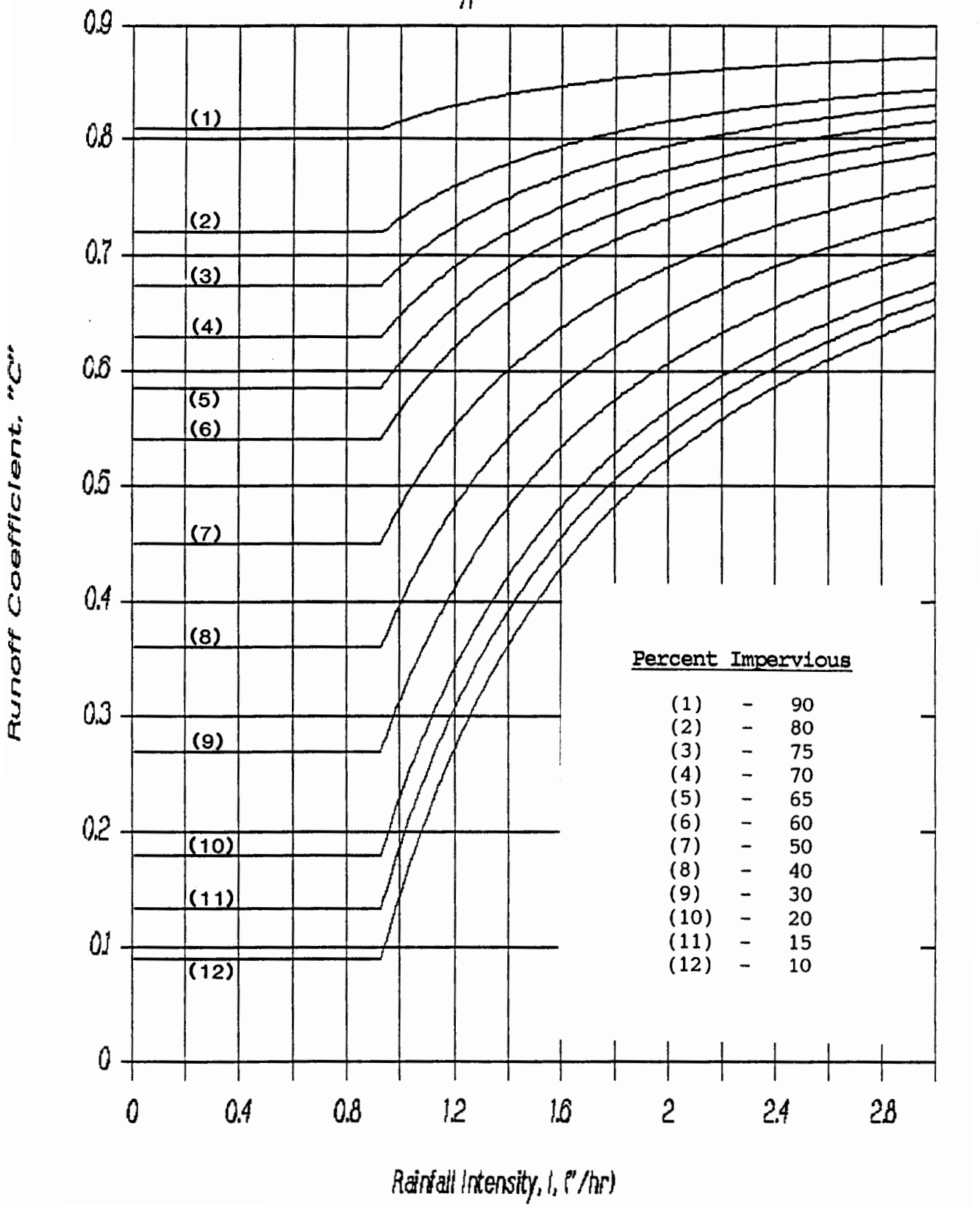
Step 6 Enter the appropriate Runoff Coefficient Value Chart, for soil type determined in Step 5, using the intensity determined in Step 4, intersect the correct development type curve. Read "C" at the ordinate of intersection.

Step 7 Compute $Q=CiA$

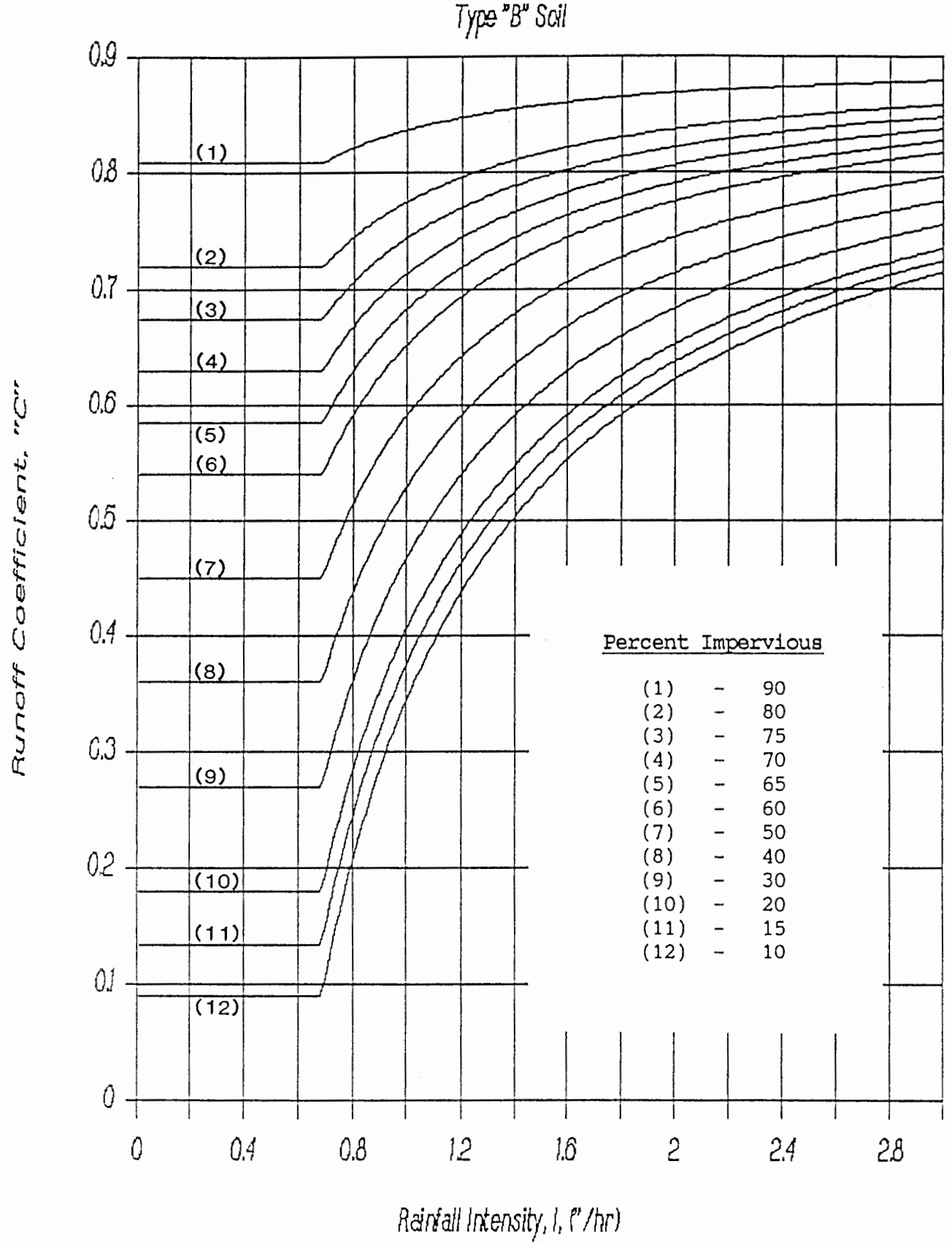
Step 8 Repeat the above steps as necessary for each reach of the watershed.

NOTE: Rational Method reports SHALL be submitted in the form shown on Figure D-7

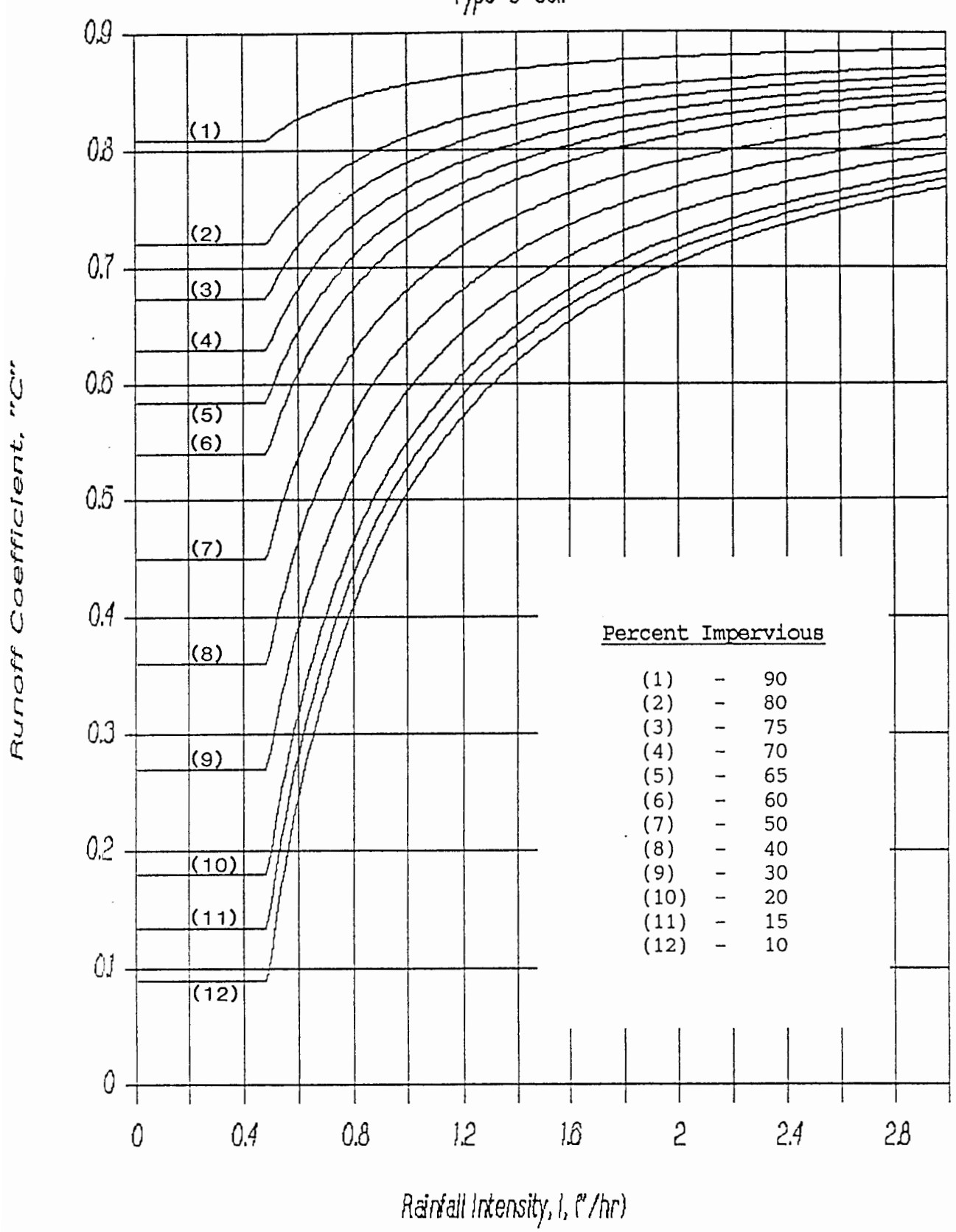
RUNOFF COEFFICIENT VALUES



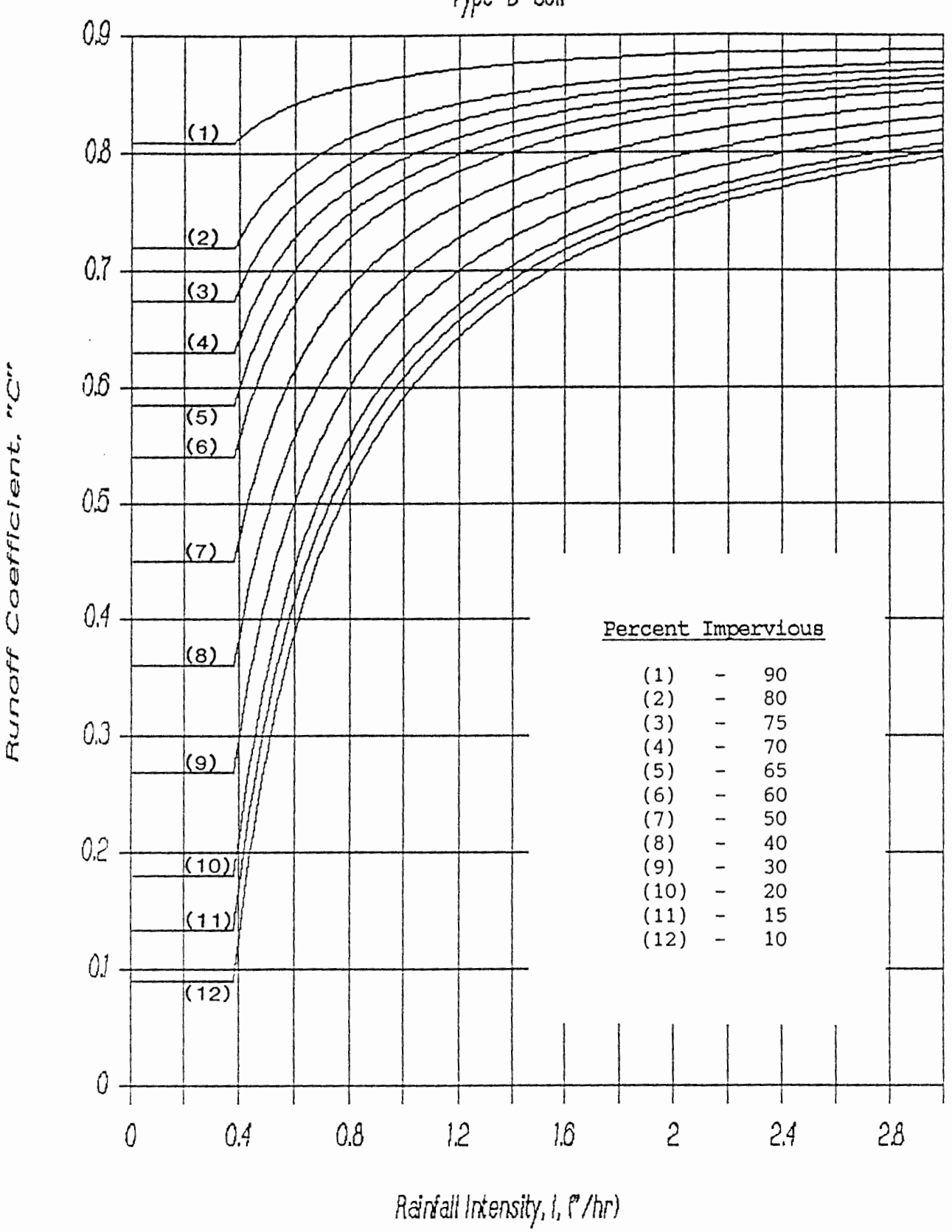
RUNOFF COEFFICIENT VALUES



RUNOFF COEFFICIENT VALUES



RUNOFF COEFFICIENT VALUES



BIBLIOGRAPHY

1. U.S. Department of Commerce, National Oceanic and Atmospheric Administration, National Weather Service, "NOAA Atlas, Precipitation Frequency Atlas of the Western United States, Volume XI-California" (1973)
2. McCuen, R.H., A Guide to Hydrologic Analysis Using SCS Methods, Prentice-Hall, Inc. (1982)
3. U.S. Department of Agriculture, Soil Conservation Service, "National Engineering Handbook," Section 4, Hydrology, Washington, D.C. (1969)
4. Hromadka II, T.V., et al., "A Modified S.C.S. Runoff Hydrograph Method," 10th Int. Symp. on Urban Hydrology, Hydraulics and Sediment Control (1983)
5. Hromadka II, T.V. "Computer Methods in Urban Hydrology: Rational Methods and Unit Hydrograph Methods," Lighthouse Publications, Mission Viejo, California (1983)
6. Langbein, W.B., "Some Channel Storage and Unit Hydrograph Studies," Trans. Am. Geophys. Union, (21), 620 (1940)
7. Sherman, L.K., "Storm Flow from Rainfall by Unit-Graph Method," Eng. News Record (108) (April 1932)
8. Snyder, F.F., "Synthetic Unit Hydrographs," Tans. Am. Geophys. Union, (19), 447 (1938)
9. U.S. Engineer Office, (Los Angeles, California) "Hydrologic of San Gabriel River and the Rio Hondo above Whittler Narrows Flood Control Basin," (December 1944, Rev. July 1946)
10. U.S. Army Corps of Engineers, The Hydrologic Engineering Center, "Hydrologic Analysis of Ungaged Watersheds Using HEC-1," Training Document No. 15 (April 1982)
11. Federal Emergency Management Agency, "Flood Insurance Study, Kern County, California - Unincorporated Areas", (revised September 29, 1989)
12. Hromadka II, T.V., et al, "Derivation of a Regionalized Rainfall Runoff Model to Comput N Year Floods for Orange and SanBernardino Counties," (January 1988)
13. Hromadka II, T.V., "San Bernardino County Desert Hydrology Manual Study, Final Summary Report," (February 24, 1986)

REFERENCE LIST

1. Akan A., Yen B., Diffusion - Wave Flood Routing in Channel Networks, Journal of Hydraulics Division, Proceedings of the American Society of Civil Engineers, Vol. 107, No. HY6 (June 1981)
2. Alonso C., Stochastic Models of Suspended-Sediment Dispersion, Journal of Hydraulic Engineering, Proceedings of the American Society of Civil Engineers, Vol. 107, No. HY6 (June 1981)
3. Beard L., Chang S., Journal of the Hydraulics Division, Urbanization Impact on Streamflow (June 1974)
4. Beard L., Impact of Hydrologic Uncertainties on Flood Insurance, Journal of the Hydrologic Division, American Society of Civil Engineers, Vol. 104, No. Hy11 (November 1978)
5. Bell F., Estimating Design Floods From Extreme Rainfall, Hydrology Papers, Colorado State University, Fort Collins, Colorado, No. 29 (June 1968)
6. Beven K., On the Generalized Kinematic Routing Method, Water Resources Research, Vol. 15, No. 5 (October 1979)
7. Bree T., the General Linear Model With Prior Information, Journal of Hydrology, 39 (1978) 113-127, Elsevier Scientific Publishing Company, Amsterdam, printed in the Netherlands
8. Cermak R., Feldman A., Urban Hydrologic Modeling Using Hec-1/Kinematic Wave, Presented at the 19th Annual AWRA Conference, October 9-13, 1983, San Antonio, Texas
9. Chein J., Sarikelle S., Synthetic Design Hyetograph and Rational Runoff Coefficient, Journal of the Irrigation and Drainage Division, Vol. 102., No. 1R3 (September 1976)
10. Chow V., Kulandaiswamy V., The IUH of General Hydrologic Systems Model, Journal of the Hydraulics Division, Proceedings of the American Society of Civil Engineers, Vol. 108., No. HY7 (July 1982)
11. Crippen J., Envelope Curves for Extreme Flood Events, Journal of the Hydraulic Division, Vol. 108, No. HY10 (October 1982)
12. Dawdy D., Bergman J., Effect of Rainfall Variability on Streamflow Simulation, Water Resources Research, Vol. 5, No. 5 (October 1969)
13. Dawdy D., O'Donnell T., Mathematical Models of Catchment Behavior, Journal of the Hydraulics Division Vol. 91, No. HY4 (July 1965)
14. Debo T., Urban Flood Damage Estimating Curves, Journal of the Hydraulics Division, Vol. 108, No. HY10 (October 1982)
15. Dickinson W., et al., An Experimental Rainfall-Runoff Facility, No. 25, Hydrology Papers, Colorado State University, Fort Collins, Colorado (September 1967)
16. Fleming G., Franz D., Flood Frequency Estimating Techniques for Small Watersheds, Journal of the Hydraulics Division, Vol. 97, No. HY9 (September 1971)
17. Fogel M., Duckstein L., Point Rainfall Frequencies in Convective Storms, Water Resources Research, Vol. 5, No. 6 (December 1969)

18. Garen D., Burges S., Approximate Error Bounds for Simulated Hydrographs, Journal of the Hydraulics Division, Proceedings of the American Society of Civil Engineers, ASCE, Vol. 107, No. HY11 (November 1981)
19. Gundlach A., Adjustment of Peak Discharge Rates for Urbanization, Journal of the Irrigation and Drainage Division, American Society of Civil Engineers, Vol. 104, No. 1R3 (September 1978)
20. Gupta V., Sorooshian S., Uniqueness and Observability of Conceptual Rainfall-Runoff Model Parameters: The Percolation Process Examined, Water Resources Research, Vol. 19, No. 1, Pages 269-276 (February 1983)
21. Hjalmarson H., Flash Flood in Tanque Verde Creek, Tucson, Arizona, Journal of Hydraulic Engineering, Vol. 110, No. 12 (December 1984)
22. Hjeimfelt A., Burwell R., Spatial Variability of Runoff, Journal of Irrigation and Drainage Engineering, Vol. 110, No. 1 (March 1984)
23. Hjeimfelt A., Convolution and the Kinematic Wave Equations, Journal of Hydrology, 75 (1984/1985) 301-309, Elsevier Science Publishers B.V., Amsterdam, printed in the Netherlands
24. Hollis G., The Effect of Urbanization on Floods of Different Recurrence Interval, Water Resources Research, Vol. 11, No. 3 (June 1975)
25. Horberger et al., Shenandoah Watershed Study: Calibration of a Topography-Based, Variable Contributing Area Hydrological Model to a Small Forested Catchment, Water Resources Research, Vol. 21, No. 12 (December 1985)
26. Huang Y., Channel Routing by Finite Difference Method, Journal of the Hydraulics Division, Vol. 104, No. HY10 (October 17, 1977)
27. Hydrological Engineering Center, Corps of Engineers, U.S. Army, Davis, CA, Vol. 5 (March 1975)
28. Hydrology, U.S. Department of Transportation, Federal Highway Administration, Hydraulic Engineering Circular No. 19 (October 1984)
29. Johnston P., Pilgrim D., Parameter Optimization for Watershed Models, Water Resources Research, Vol. 12 No. 3 (June 1976)
30. Katopodes N., Schamber D., Applicability of Dam Break Flood Wave Models, Journal of Hydraulic Engineering, Vol. 109, No. 5 (May 1983)
31. Keeler T., Comparison of Linear Systems and Finite Differences Flow-Routing Techniques, Water Resources Research, Vol. 12, No. 5 (October 1976)
32. Kelway P., The Rainfall Recorder Problem, Journal of Hydrology, 26 (1975) 55-77, Elsevier Scientific Publishing Company, Amsterdam, printed in the Netherlands
33. Kite G., Confidence Limits for Design Events, Water Resources Research, Vol. 11, No. 1 (February 1975)
34. Klemes V., Bulu A., Limited Confidence in Confidence Limits Derived by Operational Stochastic Hydrologic Models, Journal of Hydrology, 42 (1979) 9-22 Elsevier Scientific Publishing Company, Amsterdam, printed in the Netherlands
35. Lee L., Essex T., Urban Headwater Flooding Damage Potential, Journal of Hydraulic Engineering, Vol. 109, No. 4 (April 1983)

36. Loague K., Freeze R., A Comparison of Rainfall-Runoff Modeling Techniques on Small Upland Catchments, Water Resources Research, Vol. 21, No. 2 (February 1984)
37. Mawdsley J., Tagg A., Identification of Unit Hydrographs from Multi-Event Analysis, Journal of Hydrology, 49 (1981) 315-327, Elsevier Scientific Publishing Company, Amsterdam, printed in the Netherlands
38. Mein R.G., Brown B.M., Sensitivity of Optimized Parameters in Watershed Models, Water Resources Research, Vol. 14, No. 2 (April 1978)
39. Mays L., Coles, L., Optimization of Unit Hydrograph Determination, Journal of the Hydraulics Division, American Society of Civil Engineers, Vol. 106, No. HY1 (January 1980)
40. Mays L., Taur C., Unit Hydrograph via Nonlinear Programming, Water Resources, Vol. 18, No. 4, pages 744-752 (August 1982)
41. McCuen R., Bondelid T., Estimating Unit Hydrograph Peak Rate Factors, Journal of Irrigation and Drainage Engineering, Vol. 109, No. 2 (June 1983)
42. McCuen R., et al., Estimating Urban Time of Concentration, Journal of Hydraulic Engineering, Vol. 110, No. 7 (July 1984)
43. McCuen R., et al., SCS Urban Peak Flow Methods, Journal of Hydraulic Engineering, Vol. 110, No. 3 (March 1984)
44. McPherson M., Schneider W., Problems in Modeling Urban Watersheds, Water Resources Research, Vol. 10, No. 3 (June 1974)
45. Nash J., Sutcliffe J., River Flow Forecasting through Conceptual Models Part 1 - A Discussion of Principles, Journal of Hydrology, 10 (1970) 282-290
46. Neff E., How Much Rain Does a Rain Gage Gage?, Journal of Hydrology, 35 (1977) 213-220, Elsevier Scientific Publishing Company, Amsterdam, printed in the Netherlands
47. Osborn H., Hickok R., Variability of Rainfall Affecting Runoff from a Semiarid Rangeland Watershed, Southwest Watershed Research Center Tucson, Arizona, Water Resources Research, Vol. 4, No. 1 (February 1968)
48. Osborn H., Lane, L., Precipitation-Runoff for Very Small Semiarid Rangeland Watersheds, Water Resources Research, Vol. 5, No. 2 (April 1969)
49. Pedersen J., et al., Hydrographs by Single Linear Reservoir Model, Journal of the Hydraulics Division, Vol. 106, No. HY5 (May 1980)
50. Pilgrim D., Travel Times and Nonlinerity of Flood Runoff from Tracer Measurements on a Small Watershed, Water Resources Research, Vol. 12, No. 3 (June 1976)
51. Pitman W., Flow Generation by Catchment Models of Differing Complexity - A Comparison of Performance, Journal of Hydrology, 38 (1978) 59-70, Elsevier Scientific Publishing Company, Amsterdam, printed in the Netherlands
52. Porter J., A Comparison of Hydrologic and Hydraulic Catchment Routing Procedures, Journal of Hydrology, 24 (1975) 333-349
53. Reed D., et al., A Nonlinear Rainfall-Runoff Model, Providing for Variable Lag Time, Journal of Hydrology, 25 (1975) 295-305, North Holland Publishing Company, Amsterdam, printed in the Netherlands

54. Rose F., Hwang G., A Study of Differences between Streamflow Frequency and Rainfall Frequency for Small Rural Watersheds, 1985 International Symposium on Urban Hydrology, Hydraulic Infrastructures and Water Quality Control, University of Kentucky (July 23-25, 1985)
55. Ruh-Ming Ll., et al., Nonlinear Kinematic Wave Approximation for Water Routing, Water Resources Research, Vol. 11, No. 2 (April 1975)
56. Schilling W., Fuchs L., Errors in Stormwater Modeling - A Quantitive Assessment, Journal of Hydraulic Engineering, Vol. 112, No. 2 (February 1986)
57. Scully, D., Bender D., Separation of Rainfall Excess from Total Rainfall, Water Resources Research, Vol. 5, No. 4 (August 1969)
58. Sorooshian S., Gupta V., Automatic Calibration of Conceptual Rainfall-Runoff Models: The Question of Parameter Observability and Uniqueness, Water Resources Research, Vol. 19, No. 1 (February 1983)
59. Stedinger J., Confidence Intervals for Design Events, Journal of Hydraulic Engineering, Vol. 109, No. 1 (January 1983)
60. Stedinger J., Design Events with Specified Flood Risk, Water Resources Research, Vol. 19, No. 2, pages 511-522 (April 1983)
61. Tingsanchali T., Manadndhar S., Analytical Diffusion Model for Flood Routing, Journal of Hydraulic Engineering, Vol. 111, No. 3 (March 1985)
62. Troutman B., An Analysis of Input in Perception-Runoff Models Using Regression with Errors in the Independent Variables, Water Resources Research, Vol. 18., No. 4, pages 947-964 (August 1982)
63. United States Department of the Interior Geological Survey, a Digital Model for Streamflow Routing by Convolution Methods, U.S. Geological Survey Water-Resources Investigations Report 83-4160
64. U.S. Army Corps of Engineers, the Hydrologic Engineering Center, Adoption of Flood Flow Frequency Estimates at Ungaged Locations, Training Document No. 11 (February 1980)
65. U.S. Army Corps of Engineers, the Hydrologic Engineering Center, Comparative Analysis of Flood Routing Methods, Research Document No. 24 (September 1980)
66. U.S. Army Corps of Engineers, the Hydrologic Engineering Center, Continuous Hydrologic Simulation of the West Branch Dupage River above West Chicago: An Application of Hydrocomp's HSP, Research Note No. 6
67. U.S. Army Corps of Engineers, the Hydrologic Engineering Center, Hydrologic Analysis of Ungaged Watersheds Using HEC-1, Training Document No. 15 (April 1982)
68. U.S. Department of Transportation, Federal Highway Administration, Hydrology, Hydraulic Engineering Circular No. 19 (October 1984)
69. U.S. Army Corps of Engineers, the Hydrologic Engineering Center, Introduction and Application of Kinematic Wave Routing Techniques Using HEC-1, Training Document No. 10 (May 1979)
70. U.S. Army Corps of Engineers, the Hydrologic Engineering Center, Testing of Several Runoff Models on an Urban Watershed, Technical Paper No. 59

71. U.S. Department of Agriculture, Soil Conservation Service, National Engineering Handbook Section 4 (NEH-4) 210-V1 Amendment 5, Transmission Losses, Washington, D.C. (March 1, 1983)
72. Wallis J., Wood E., Relative Accuracy of Log Pearson 111 Procedures, Journal of Hydraulic Engineering Vol. 111, No. 7 (July 1985)
73. Watt W., Kidd C., Quurm - A Realistic Urban Runoff Model, Journal of Hydrology, 27 (1975) 225-235, Elsevier Scientific Publishing Company, Amsterdam, printed in the Netherlands
74. Weinmann E., Laurenson E., Approximate Flood Routing Methods: A Review, Journal of the Hydraulics Division, Vol. 105, No. HY12 (December 1979)
75. Whitely, R. and Hromadka II, T.V., "Computing Confidence Intervals for Floods, I," Microsoftware for Engineers, in-press (1986)
76. Whitely, R. and Hromadka II, T.V., "Computing Confidence Intervals for Floods, II," Microsoftware for Engineers, in-press (1986)
77. Williams, D.W., et al., TRRL and Unit Hydrograph Simulations Compared with Measurements in an Urban Catchment, Journal of Hydrology, 48 (1980) 63-70, Elsevier Scientific Publishing Company, Amsterdam, printed in the Netherlands
78. McCuen, R.H., Yen, C.C., and Hromadka II, T.V., "Adjusting Stream Gage Data for Urbanization Effects," Microsoftware for Engineers (1986) in-press
79. Zaghloul N., SWMM Model and Level of Discetization, Journal of the Hydraulics Division, Proceedings of the American Society of Civil Engineers, ASCE, Vol. 107, No. HY11 (November 1981)