B-48

FINAL REPORT

STUDY OF PEAK RATES OF RUNOFF IN EASTERN COLORADO AND ADJACENT AREAS

by

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Prepared under the Sponsorship of

The Hydraulic Research Division

U. S. Bureau of Public Roads

Carl F. Izzard, Chief

and

The Colorado Department of Highways

Mark U. Watrous, Chief Engineer

May 1960

CER60RAS31

CORRECTIONS TO CER60RAS31

"STUDY OF PEAK RATES OF RUNOFF IN EASTERN COLORADO

AND ADJACENT AREAS"

The following pen and ink changes should be made to Report Number CER60RAS31:

1. Page 4. Revise paragraph one of "Checking results" to read as follows:

   "1. Comparing the estimates of Q_{10} from Fig. 3 and Q_{10} determined from values of unit discharge from Fig. 4 for the area being considered. (CAUTION: See section "Limitations on Use of Fig. 4 on page 12.)"

2. Page 8 - Revise third line to read as follows:

   "1. The estimate of Q_{10} determined from the unit discharge values of Fig. 4 was within $\pm$ 25 per cent of ..."

3. Page 9 - "Checking results", First paragraph - Revise to read as follows:

   "1. Comparing the estimate of Q_{10} from Fig. 7 with the values of Q_{10} obtained from the <u>unit discharge</u> values shown in Fig. 4. (CAUTION: See section "Limitations on Use of Fig. 4 on page 12.")"

4. Page 11 - Revise line 1 of first paragraph in "SIGNIFICANCE OF CHECKS" to read as follows:

   "1. The estimate of Q_{10} determined from unit discharge values of Fig. 4 was within $\pm$ 25 per cent ..."

Corrections to Report CER60RAS31 - continued:

5. Pages ii and viii - Tables 4 and 5 do not have page numbers. They are inserted between pages 34 and 44.

6. Fig. 13 - Revise the caption to read as follows:

   "Relations Between Total Channel Length Measured from 1:250,000 Scale U.S. Geological Survey Maps and from Colorado Highway Maps, Scale 1" = 1 mile."

7. Table 11 on page 75 - The ninth column from the right-hand side of the table should be marked so as to be included in "Channel Slope", rather than in "Overland Slope".

8. Page viii - Add

   13. Summary of Basic Data . . . . 92.

9. Page 75 - Table 11. Watershed number 20 should be "Rock Creek at Parks, Nebraska."

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TABLE OF CONTENTS

TABLE OF CONTENTS (cont'd)

TABLE OF CONTENTS (cont'd)

TABLE OF CONTENTS (cont'd)

LIST OF FIGURES

LIST OF FIGURES (cont'd)

LIST OF FIGURES (cont'd)

LIST OF TABLES

ABSTRACT

A study was made of the factors affecting peak rates of runoff in the semi-arid region of eastern Colorado and adjacent areas. Within this region, annual maximum floods, on watersheds less than 1000 square miles, are usually the result of intense rainfall over a limited area. The investigation reveals that peak rates of runoff from ungaged watersheds can be estimated from parameters of watershed area, channel slope, and a soil infiltration index in the region east of the Rocky Mountain Foothills.

In the Rocky Mountain Foothills, estimates of peak rates of runoff from ungaged watersheds can be made using watershed contributing area, elevation, and location.

Design procedures for estimating peak rates of runoff in these regions are illustrated by examples.

Results of the studies used to develop these design procedures are presented in summary form.

I. INTRODUCTION

Economical design of highway drainage structures requires a knowledge of the magnitude and frequency of peak rates of runoff. In most cases records of peak rates of runoff are not available at the proposed construction site.

For this reason it was desired to develop techniques for estimating the magnitude and frequency of peak rates of runoff from ungaged watersheds.

A study was made of peak rates of runoff in eastern Colorado and adjacent areas for the purpose of developing such techniques.

Results are presented in two reports, "Procedures for Estimating Peak Rates of Runoff in Eastern Colorado and Adjacent Areas," (CER60RAS30), and "Study of Peak Rates of Runoff in Eastern Colorado and Adjacent Areas," (CER60RAS31). In the first report information is presented which is considered necessary for the design engineer in making estimates of peak rates of runoff. The second report includes the same material as the first, plus additional detailed information on the important results of related studies made in the development of the design procedures.

The organization of both reports is similar. Procedures for making estimates of the magnitude and frequency of peak rates of runoff are described and illustrated, after which the results of related studies are presented. The primary difference in the two reports is that the first gives only a brief summary of these related studies.

II. OBJECTIVES

The objectives of the study were:

1. To evaluate the influence of certain hydrologic, physiographic, and meteorologic parameters on peak rates of runoff.
2. To develop techniques for predicting magnitude and frequency of floods in semi-arid areas (as typified by eastern Colorado and adjacent areas) on ungaged watersheds having contributing areas less than 1000 square miles. The criterion for acceptable accuracy for these techniques is that at least two-thirds of the estimates must not depart from observed values by more than 25 per cent of the estimated value.*

*This criterion for accuracy was recommended by the sponsors. "Per cent of error" is defined by

$$\text{Per cent error} = \frac{Q_{estimated} - Q_{actual}}{Q_{estimated}} \times 100$$

III. ESTIMATING PEAK RATES OF RUNOFF

AREAS OF APPLICATION

Procedures are presented for estimating peak rates of runoff in two separate physiographic areas. The first procedure is applicable in an area of the high plains in eastern Colorado and adjacent areas designated "D-13" and "D-20" in Fig. 1. The second procedure is applicable to the Rocky Mountain Foothills region labeled "E-5" in Fig. 1. The second procedure is also applicable to the shaded portion of the "E-8" area in Fig. 1. Design procedures and background studies for the "E-5" area plus the portion of the Southern Rocky Mountains that is shaded in Fig. 1 are identified as the "E-5" area throughout this report.

PEAK RATES OF RUNOFF FROM THE D-13 and D-20 AREAS

Desired result - Q_N Peak rate of runoff to be expected in N Number of Years.

Data required - For the "D-13" and "D-20" areas of Fig. 1, the following basic data are required for estimating peak rates of runoff:

A	watershed contributing area (square miles)
E_{CS}	elevation (feet MSL) at the construction site
L	length of the longest river channel (miles)
$E_{0.9L}$	elevation (feet MSL) at a point 0.9L upstream from the construction site
I	a soil infiltration index.

Procedure - The procedure for making estimates of the peak rates of runoff having recurrence intervals of 10 years (Q_{10}) is as follows:

1. Determine the parameters A, L, E_{CS}, and $E_{0.9L}$ from the appropriate topographic map or site survey.
2. Determine the soil infiltration index, I, from Fig. 2.
3. Compute the slope parameter

$$S_{0.9L} = \frac{E_{0.9L} - E_{CS}}{0.9L}$$

4. Enter Fig. 3 with A, $S_{0.9L}$, and I and estimate Q_{10}, the required estimate of the peak rate of runoff having a recurrence interval of 10 years (Q_{10}).
5. For estimates of the peak rate of runoff for a recurrence interval greater than 10 years, multiply the estimate of Q_{10} by the appropriate ratio of Q_N/Q_{10} shown in Fig. 3.
6. Check the accuracy of the estimate of Q_{10} by the methods described in the following section.

Checking results - Several methods of checking the estimate of Q_{10} from Fig. 3 are available to the design engineer. They are:

1. Comparing the estimate of Q_{10} from Fig. 3 with the values of unit discharge determined from Fig. 4 for the area being considered. (CAUTION: See section "Limitations on Use of Fig. 4.")
2. Comparing the estimate of Q_{10} from Fig. 3 with the maximum and minimum recommended value of Q_{10} from Fig. 5. The small circles shown in Fig. 5 are the actual values of Q_{10}

for the watersheds that were used in deriving the relation shown in Fig. 3. The maximum curve on Fig. 5 represents the maximum Q_{10} obtained from the parameters used to derive Fig. 3. This does not imply that higher values of these parameters might not be encountered, but rather that these combinations have not occurred for testing on gaged watersheds.

3. Determining whether the ungaged watershed under investigation is similar to the gaged watersheds used to develop Fig. 3 by using the test for representativeness given in Chapter IV (Watershed Characteristics). If the representativeness test indicates that the watershed under investigation is similar to the gaged watersheds used in deriving Fig. 3, then considerable confidence can be placed in the design estimate derived from Fig. 3. If however, the representativeness test indicates (on the basis of area, slope, location, and precipitation) that the ungaged watershed under consideration is not similar to those used in deriving Fig. 3, then less confidence can be placed in the estimate derived from Fig. 3. Details of the representativeness test are given in Chapter IV (Watershed Characteristics).

Degree of accuracy to be expected - Fig. 6 shows the cumulative relative frequency of errors of estimate of the peak rate of runoff having a 10-year recurrence interval (Q_{10}) that can be expected from use of Fig. 3. Fig. 6 shows, for example, that use of Fig. 3 gave errors of estimate exceeding 25 per cent for about 20 per cent of the cases. It also shows that errors of estimate exceeding 50 per cent can be expected about 10 per cent of the time.

Examples - The following example illustrates the design procedure for estimating peak rates of runoff from watersheds in the D-13 and D-20 areas.

Assume that Federal Highway 24 in Colorado is to be a link in the Federal Inter-State System. A new bridge is to be constructed for four-lane divided traffic across Spring Creek two miles west of Stratton, Colorado. Part of the highway design problem is to determine Q_{10}, Q_{25}, and Q_{50}.

SOLUTION: By means of a topographic map (U.S. Geological Survey, Scale 1:250,000)# and the soil map of Fig. 2, the following information is obtained:

A = 144 square miles

$E_{0.9L}$ = 4,980 feet

E_{CS} = 4,345 feet

L = 38 miles

I = 5.3 (If the watershed being considered contains more than one soil type, determine "I" by the method described in Chapter IV of this report "Effect of Soil Type.")

Compute the slope, $S_{0.9L}$, by

$$S_{0.9L} = \frac{E_{0.9L} - E_{CS}}{0.9L} = \frac{635}{34.2} = 18.6 \text{ ft/mi.}$$

For $A = 144$ sq. mi., $S_{0.9L} = 18.6$ ft/mi., and $I = 5.3$ the graph of Fig. 3 gives

$$Q_{10} = 2150 \text{ cfs}$$

#Note that this procedure is applicable only to this map. Refer to Fig. 13 for use of Colorado Highway Department County maps (Scale 1/2" = 1 mile).

Apply the appropriate ratios of Fig. 3 to obtain

$$Q_{25} = (Q_{10})(1.66) = 3560 \text{ cfs}$$

$$Q_{50} = (Q_{10})(2.15) = 4620 \text{ cfs}$$

These are the required design estimates of Q_{10}, Q_{25}, and Q_{50}.

CHECK: The value of Q_{10} may be checked by one or all of the following methods:

1. Regional distribution of <u>unit</u> <u>discharge</u> (Q_{10}/A). At the location of the construction site (Longitude 102° 38', Latitude 39° 18') read from Fig. 4 the value of $Q_{10}/A = 15$. (Interpolated between isolines of $Q_{10}/A = 10$ and 20.)

   Then $Q_{10} = 15A$

   $$Q_{10} = (15)(144) = 2160 \text{ cfs.}$$

   (CAUTION: See section "Limitation on Use of Fig. 4.")

2. Recommended maximum and minimum peak rates of runoff. For A = 144 square miles, Fig. 5 gives a recommended maximum value of Q_{10} of 6500 cfs., and a recommended minimum value of Q_{10} of 210 cfs.

3. Determination of representativeness

   From the procedure described and illustrated in Chapter IV (Watershed Characteristics), this ungaged watershed is determined to be similar to the gaged watersheds used to derive the relationships of Fig. 3.

SIGNIFICANCE OF CHECKS: The estimate of design discharge for this watershed may be assumed to be of acceptable accuracy for the following reasons:

1. The estimate of Q_{10} from Fig. 4 was within $\pm$ 25 per cent of the estimate of Q_{10} from Fig. 3.
2. The estimate of Q_{10} from Fig. 3 fell within the recommended maximum and minimum discharges shown on Fig. 5.
3. The watershed was determined to be representative.

PEAK RATES OF RUNOFF FROM THE E-5 AREA

<u>Desired result</u> - Q_N , the peak rate of runoff to be expected in "N" number of years.

<u>Data required</u> - For the area marked "E-5" and the shaded portion of E-8 in Fig. 1, the following data are required for estimating peak rates of runoff:

A watershed contributing area, square miles

$E_{0.5L}$ elevation on the main channel (feet MSL) half-way between the construction site and the headwaters.

<u>Procedure</u> - The procedure for making estimates of the peak rates of runoff having recurrence intervals of ten years (Q_{10}), from watersheds in this area is as follows:

1. Determine the parameters A and $E_{0.5L}$ from the appropriate topographic map or site survey.
2. Note the latitude and longitude of the construction site.
3. Enter Fig. 7 with these parameters to obtain an estimate of Q_{10}.

4. For estimates of the peak rate of runoff for a recurrence interval greater than 10 years, multiply the estimate of Q_{10} by the appropriate ratio of Q_N/Q_{10} shown in Fig. 7.
5. Check the estimate of Q_{10} by the methods described in the following sections.

Checking Results - Two methods of checking the estimate of Q_{10} from Fig. 7 are available to the design engineer. They are:

1. Comparing the estimate of Q_{10} from Fig. 7 with the values of unit discharge shown in Fig. 4. (CAUTION: See section "Limitations on Use of Fig. 4.")
2. Comparing the estimate of Q_{10} from Fig. 7 with the maximum and minimum recommended value of Q_{10} from Fig. 8. The small circles shown in Fig. 8 are the actual values of Q_{10} for the watersheds that were used in deriving the relation shown in Fig. 7. The maximum curve in Fig. 8 represents the maximum Q_{10} obtained from the parameters used to derive Fig. 7. This does not imply that higher values of these parameters might not be encountered, but rather that these combinations have not been tested by gaged watersheds.

Degree of Accuracy to be Expected - Fig. 9 shows the cumulative frequency of errors of estimate of the peak rate of runoff having a 10-year recurrence interval (Q_{10}) that can be expected from use of Fig. 7. Fig. 9 shows, for example, that use of Fig. 7 gave errors of estimate less than $\pm$ 25 per cent for about 78 per cent of the cases. It also shows that errors of estimate exceeding 50 per cent can be expected slightly more than 10 per cent of the time.

Example - The following example illustrates the design procedure for estimating peak rates of runoff from watersheds in the E-5 Area.

U. S. Highway 285 west of Denver is to be relocated along a less sinuous route through the mountains. Approximately a mile north of Tinytown, Colorado, the highway crosses South Turkey Creek. To determine what size of box or large pipe culvert will be adequate, values for Q_{10}, Q_{25}, and Q_{50} are desired.

SOLUTION: By means of a topographical map (U.S. Geological Survey scale 1:250,000),# the following information is obtained.

$A = 48$ square miles

$E_{0.5L} = 7800$ feet msl

Location of construction site: 105° 14'W, 39° 37'N.

Enter Fig. 7 with $A = 48$, $E_{0.5L} = 7800$, and latitude 39° 37'N, and read $Q_{10} = 840$ cfs.

Apply the appropriate ratios in Fig. 7 to obtain

$$Q_{25} = (Q_{10})(1.66) = (840)(1.66) = 1400 \text{ cfs}$$

$$Q_{50} = (Q_{10})(2.15) = (840)(2.15) = 1800 \text{ cfs}$$

These are the required design estimates of Q_{10}, Q_{25}, and Q_{50}.

CHECK: The value of Q_{10} may be checked by either or both of the following methods:

#Note that this procedure is applicable only to this map. Refer to Fig. 13 for use of Colorado Highway Department County maps (Scale 1/2" = 1 mile).

1. Regional distribution of unit discharge, Q_{10}/A . At the location of the construction site (105° 14'W, 39° 37'N), read from Fig. 4 the value of $Q_{10}/A \approx 14$

   Then $Q_{10} = 14A = 14(48) \approx 675$ cfs

   (CAUTION: See section "Limitation on Use of Fig. 4.")

2. Recommended maximum and minimum peak rates of runoff. For A = 48 square miles, Fig. 8 gives a recommended maximum value of Q_{10} of 1220 cfs and a recommended minimum of Q_{10} of 420 cfs.

SIGNIFICANCE OF CHECKS: Both methods of checking indicate that the estimates of design discharge are reasonable, because:

1. The estimate of Q_{10} from Fig. 4 was within $\pm$ 25 per cent of the estimate of Q_{10} from Fig. 7.
2. The estimate of Q_{10} from Fig. 7 fell within the recommended limits of maximum and minimum discharge shown on Fig. 8.

LIMITATIONS AND PRECAUTIONS

Limitations in Basic Data - In the D-13 and D-20 areas runoff records had the following limitations:

1. Only a few runoff records for watershed areas less than 100 square miles were available, and
2. Only a few of all the runoff records were for a period of time greater than 20 years.

Therefore, a primary need in obtaining improved estimates of peak rates of runoff from small watersheds is the establishment of additional gaging stations--recording and non-recording--on watersheds having contributing areas less than 100 square miles.

Limitations of Extrapolation Techniques for Floods having a Recurrence Interval Greater than 40 Years - From flood frequency studies of watersheds in and near the study area, it was determined that the peak rates of runoff from floods having a 40-year recurrence interval (Q_{40}) were approximately twice as big as floods having a 10-year recurrence interval (Q_{10}). The ratios Q_N/Q_{10} shown on Figs. 3 and 7 were determined by plotting the ratios of Q_N/Q_{10} for $N = 10$ and 40 on extremal-probability paper and connecting the points with a straight line. Intermediate points were determined by interpolation. Values of Q_N/Q_{10} for recurrence intervals of 45 and 50 years were determined by extrapolation of the straight line.

The possible inaccuracy that may result from such an extrapolation technique should be recognized, since the basic data used to derive Figs. 3 and 7 were mostly derived from records less than 40 years in length.

Limitations on Watershed Size - Design procedures presented in this report are valid for watersheds having a drainage area of 1000 square miles or less. Since the basic data used in developing the design charts for the D-13 and D-20 areas were mostly larger than 100 square miles, the portions of Fig. 3 for areas less than 100 square miles are shown in dashed lines to indicate reduced confidence in the estimates of Q_{10} from watersheds of this size.

Limitations on Use of Fig. 4 - Although the isolines on Fig. 4 were drawn after a qualitative consideration of slope, elevation, soil type, and precipitation, no consideration could be given to the effect of area on unit discharge. THEREFORE, FIG. 4 SHOULD NOT BE USED AS A DESIGN CHART, IT SHOULD BE USED ONLY AS A QUALITATIVE CHECK OF THE RESULTS FROM FIGS. 3 AND 7.

It has not been possible to establish any consistent relationship between unit discharge and watershed size in the study area.

IV. INVESTIGATION OF FACTORS RELATED TO PEAK RATES OF RUNOFF

DELIMITATION OF REGION OF STUDY

Primary emphasis in this study was given to the region in eastern Colorado and adjacent areas shown in Fig. 14. The areas noted in Fig. 14 as D-13, D-20, E-5, and E-8 are areas established by the Soil Conservation Service as having similar physiographic features and similar problems in soil conservation. The D-13 area is called the "Northern Brown Plains." The D-20 area is called the "Plains of the Upper Arkansas and Purgatorie Rivers." The E-5 area is called the "Rocky Mountain Foothills." The E-8 area is called the "Southern Rocky Mountains." Brief descriptions of these areas, as given in an unpublished manuscript of the Soil Conservation Service, follow:

D-13 Northern Brown Plains

> The Northern Brown Plains occupy a total area of 48,938,000 acres located in northeastern Colorado, northwestern Kansas, southeastern Wyoming, western Nebraska, and a small area extends into south central South Dakota... It has a relief that is characterized by nearly level to gently rolling tableland areas that break off into steeply rolling valley slopes. In the eastern part of Colorado and southwestern Nebraska there are several relatively large areas of sandhills...
>
> The average annual precipitation is about 14 to 18 inches... Rainfall is quite variable.. (with) the greater portion of the precipitation falling (at high rates) with high runoff and erosion rates...
>
> In the area as a whole 42 per cent of the land is in cultivation and 54 per cent is in range...

Soils of the area are.. of four types and all of them can be found in each of the four states. They are: (1) deep medium textured soils on nearly level tableland areas; (2) medium depth, medium textured soils on upland..; (3) shallow medium textured soils and gravel; and (4) sandy soils on aeolian sand deposits.

D-20 Plains of Upper Arkansas and Purgatorie Rivers

This area is located in southeastern Colorado and covers an area of 6,795,000 acres. The relief is undulating to rolling, 4,000 to 5,000 feet elevation above sea level. Rainfall is variable, 11 to 14 inches (annually). Soils are shallow to moderately deep, medium to moderately heavy textured on range land.

...Erosion--slight sheet erosion on much of the area. Severe in local areas having poor cover...

Seven per cent cultivated... 90 per cent grassland classed as semi-arid grazind land..., 3 per cent miscellaneous, no forest.

The area designated as "E-5" in this report includes the Rocky Mountain Foothills and a part of the Southern Rocky Mountains in Colorado. The parts of the Southern Rocky Mountains included in this study are east of the Sangre de Christo range in southern Colorado, and east of the Continental Divide in central and northern Colorado, and Southern Wyoming. This region is shown in Fig. 1. This area constitutes a thin strip along the headwaters of the Canadian River, Arkansas River, South Platte River, and North Platte River drainages. It extends from 35° latitude in New Mexico, through Colorado to 43° latitude in Wyoming.

Descriptions of these regions from an unpublished manuscript of the Soil Conservation Service follow:

Rocky Mountain Foothills

Along the eastern base of the Rocky Mountains uplift and associated structural ranges in Wyoming and Colorado is a transition zone of limited linear dimension and with individualistic climate, soils, and topography. Total area is 10,014,536 acres.

...Characteristics topography is hogback and cuesta. Elevations range from about 5,500 to 8,000 feet.

Two precipitation zones are evident... The lower elevations receive about 10-14 inches per annum and the upper 15-19 inches. Maximum approach 21 inches and minimums about 7 inches per annum. Torrential rainstorms of violent, but short, duration, encourage considerable runoff from depleted ranges.

...Valley lands associated with the hills have deep moderately permeable soils very favorable to crop production.

...In contrast, the soils on the hogbacks are shallow and often times stony.

...The major land use is grazing; the minor production of supplemental feed on irrigated land. Approximately ten per cent of the area is cropped and the remaining portion provides spring, fall and some winter pasture...

There are no extensive areas of timbered lands in the foothills area.

Southern Rocky Mountains

The southern Rocky Mountains (E-8)... are high mountain country cut by deep narrow valleys with steep slopes and canyons along the streams. Elevations range from 6,000 to 14,000 feet. Upper timber line is about 10,000 feet above sea level, and many of the higher peaks are snow capped throughout the season. The precipitation is 12 to 30 inches per annum governed largely by elevation...

Native vegetation is sub-alpine confierous forest and alpine tundra...

Land use: summer range; 40 per cent grassland, 52 per cent forest, 2 per cent cultivated, 6 per cent miscellaneous, largely above timber line consisting mainly of barren stony land.

SEASONAL DISTRIBUTION OF ANNUAL MAXIMUM FLOOD EVENTS

To determine the effect of elevation and watershed area on the seasonal distribution of annual maximum flood events; runoff records for 62 stations in the North Platte, South Platte,Republican, Arkansas, and

Colorado River watershed covering all of Colorado except the San Luis Valley, were analyzed.

The stations were first divided into three nearly equal groups according to elevation. These groups were then divided into three more groups according to watershed area, making a total of nine classes with varying numbers of cases in each class. A graph of each class was then plotted using accumulated frequency of annual maximum flood events in per cent vs. month of occurrence of the maximum flood event. Results were then analyzed on the basis of the dates of occurrence of 67 per cent of all flood events. The results are given in Table 1.

Table 1. TABULATION OF DATES OF OCCURRENCE OF 67 PER CENT OF ALL ANNUAL FLOOD EVENTS AS A FUNCTION OF ELEVATION AND WATERSHED SIZE.

Elevation Class#	Area Class Sq. Mi.	Approximate Date of Occurrence	Average Date
H	1-127	21 June	
M	"	30 May	11 June
L	"	12 June	
H	139-448	7 June	
M	"	12 June	14 June
L	"	24 July	
H	460-1770	1 July	
M	"	9 June	23 June
L	"	29 June	

H = High elevation, range: 7800 - 11,000 ft msl
M = Medium elevation, range: 6090 - 7680 ft msl
L = Low elevation, range: 2800 - 6080 ft msl

#Elevation refers to elevation of the gaging station, or minimum elevation of the watershed.

One may draw the following conclusions from Table 1.

1. The average date of 67 per cent of annual maximum floods becomes later with an increase in watershed size.
2. For watersheds of 139 - 448 square miles, the date of 67 per cent of annual maximum floods becomes later with a decrease in elevation.
3. For watersheds less than 139 square miles and between 460 - 1770 square miles, the date of occurrence of 67 per cent of annual maximum floods becomes later with decreasing elevation below 7680 feet msl.

In addition to the analysis shown in Table 1, iso-lines were drawn on maps showing equal times of occurrence of 67 per cent of annual maximum flood events. These maps indicate that the dates of occurrence of flood events are later on the plains than in the mountainous areas. This can be interpreted in terms of summertime rains as a cause of flood events on the plains, as compared to snow melt, or a combination of snow melt and rain as a cause of flood events in the mountain areas.

CHARACTERISTICS OF PRECIPITATION ASSOCIATED WITH ANNUAL MAXIMUM FLOODS

To gain an insight into the precipitation characteristics associated with annual maximum flood events, the rainfall distribution associated with annual floods for nine stations in eastern Colorado was studied.

Annual maximum peak flows from contributing watersheds of not more than 1000 square miles were recorded for the period 1930-1950. For each flood event, the amount of precipitation at raingage stations--recording or non-recording--on or near the basin was determined. The precipitation data were then given a weight, as follows: If 0.1 inch per day or more fell at a raingage station, a weight 1 was given; if less than 0.1 inch was recorded, a weight of 0.5 was given; and zero rainfall was given a weight of zero.

The drainage basin was divided into sub-areas by the Thiessen method using the foregoing weighted values to compute the per cent of basin area covered by precipitation for the given flood event. A weight of "1" was used when the entire sub-area received rainfall. The ratio of the number of the annual maximum floods associated with 100 per cent coverage of watershed to the total number of flood events was then expressed as a per cent. This value was then plotted against basin area, as shown in Fig. 15. Fig. 15 shows that for watersheds with contributing areas larger than about 900 square miles, two-thirds or more of the annual maximum flood events are associated with rains which cover the entire basin. For watersheds with contributing areas less than about 50 square miles, one-third or less of the annual maximum floods are caused by such rainfalls.

These data suggest that most of the peak flows from watersheds less than 50 square miles are probably the result of high intensity rains covering a limited area.

The characteristics of such extreme rain events was studied in greater detail. The results are given in the section "Unit Peak Flow as a Function of Watershed Size."

CORRELATION OF PRECIPITATION WITH PHYSIOGRAPHIC PARAMETERS

Attempts were made to correlate certain physiographic parameters with precipitation parameters, as had been done by Spreen (1) for western Colorado, where mean seasonal and annual precipitation was correlated to factors of elevation, exposure, and zone. Results indicated that a statistically significant correlation could be obtained between mean monthly rainfall (the month of May was used in the study) and simple parameters of location (latitude, longitude, and elevation).

Details of this correlation follow:

Dependent variable:

Y = mean monthly precipitation for May, inches.

Independent variables:

X_1 = latitude, less 30 degrees.

X_2 = longitude, less 100 degrees.

X_3 = elevation, in 10 thousands of feet.

Station groupings:

Group 1: Nineteen (19) stations in Colorado in the Arkansas River drainage.

Group 2: Twenty-nine (29) stations in the Platte and Kansas drainage in Colorado.

Results included the regression equation, the correlation coefficient, $\overline{R}$, the standard error of estimate, $\overline{S}$, and the standard deviation, σ, of the individual coefficients.

Results:

Group 1:

$$Y = 2.99 - 0.045X_1 - 0.55X_2 + 2.95X_3$$

$\overline{R}$ = .72 (Significant at 99 per cent level)

$\overline{S} = .38$

$\sigma_1 = .17^{\#}$

$\sigma_2 = .12$

$\sigma_3 = .73$

Group 2:

$$Y = 3.33 + 0.03X_1 + 0.15X_2 - 3.43X_3$$

$\overline{R}$ = 0.67 (Significant at 99 per cent level)

$\overline{S} = 0.37$

$\sigma_1 = .13^{\#}$

$\sigma_2 = .09$

$\sigma_3 = .90$

USE OF WEATHER RADAR DATA TO PROVIDE INCREASED AREAL COVERAGE OF RAINFALL EVENTS

Attempts were made to utilize two types of weather radar data to extend the areal coverage for individual rainfall events. Hand-drawn sketches of the Plan-Position Indicator (PPI) scope from a 5.5 cm set

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#X_1, latitude is not a significant parameter.

used by United Air Lines in Denver, and sketches of a PPI scope reconstructed from coded descriptions of radar echo data from a 3.0 cm set at Lowry Air Force Base in Denver were studied. It was concluded that the data in this form were not suitable for the intended purpose.

The lack of correlation between radar echo intensity and rainfall intensity could be caused by any combination of the following factors:

1. Error in drawing the sketches of the PPI scope.
2. Non-linearities in the scope presentation.
3. Problems in relating the time of the echo to the time of the clock-hourly precipitation.
4. The problem of evaporation of raindrops between the cloud base and the ground, typical of the high-based clouds of this area.

ESTIMATES OF CLOCK-HOURLY PRECIPITATION FROM PRECIPITATION AMOUNTS OF LONGER DURATION

The foregoing study on the characteristics of rainfall associated with annual maximum flood events indicated that high-intensity, short-duration rainfall over small areas was probably associated with annual maximum flood events on small watersheds.

Records of short-duration rainfall are available in the published records of clock-hourly rainfall, derived from records of recording raingages. However, these recording gages are fewer in number than non-recording gages; and in addition, the length of record from non-recording gages is usually longer than from the recording gages. For these reasons, it would be desirable to utilize the more plentiful data from non-recording gages in studies of precipitation related to peak rates of runoff.

A study was made to evaluate methods for making estimates of clock-hourly precipitation for a given recurrence interval from parameters derived from non-recording raingages in an area in eastern Colorado.

The area studied included part of Colorado east of the Continental Divide in the vicinity of Denver. The region of the study was divided into five separate sub-areas, each containing five or more recording raingage stations. Precipitation data were obtained from published records of the U. S. Weather Bureau.

"Relative Wetness" ratios were defined as follows:

$$R_{ij} = \frac{\text{2 year freq. max. hourly precipitation in inches at station } i}{\text{2 year freq. max. hourly precipitation in inches at station } j}$$

$$d_{ij} = \frac{\text{2 year freq. max. daily precipitation in inches at station } i}{\text{2 year freq. max. daily precipitation in inches at station } j}$$

$$M_{ij} = \frac{\text{2 year freq. max. monthly precipitation in inches at station } i}{\text{2 year freq. max. monthly precipitation in inches at station } j}$$

$$A_{ij} = \frac{\text{2 year freq. max. annual precipitation in inches at station } i}{\text{2 year freq. max. annual precipitation in inches at station } j}$$

The subscript is an index notation with $i = 1, 2, 3, \ldots n-1$, and $j = 2, 3, \ldots\ldots n$, where n is the total number of stations in the sub-area.

From records from the selected stations for the period 1948-1957, annual maximum and summer seasonal (May-August) maximum precipitation values for each of the above durations were compiled for each station. Using Gumbel plotting positions, the precipitation amounts having a 2-year recurrence interval were determined for hourly, daily, monthly, and annual values. Stations within each sub-area were then ranked suitably to yield values of d_{ij}, M_{ij}, $A_{ij} \leq 1.0$. Values of R_{ij}, d_{ij}, M_{ij}, and A_{ij} were then computed. (Note that the subscript "s" refers to seasonal values--May through August--and the subscript "a" refers to annual values.)

R_{ij} vs. d_{ij}, R_{ij} vs. M_{ij}, and R_{ij} vs. A_{ij} were plotted both for annual and seasonal values for each sub-area. Assuming an equation of the form $y = mx$, the best-fit lines were computed by the method of least squares, and the departures of the individual plotted points from the fitted regressions were computed. A distribution-of-error curve was prepared for each of these plots. These error charts gave a measure of the dispersion of the data from the fitted regression curves. Results were presumed to be of acceptable accuracy if 67 per cent of the data fell within $\pm$ 25 per cent of the fitted regression.

Examination of 25 such error curves revealed that only three (3) had errors greater than $\pm$ 25 per cent for 67 per cent of the plotted points. This indicates generally acceptable accuracy for the technique. To further delimit the dispersion of the data from the best-fit lines, the areas between the ordinate and the distribution-of-error curves for 0 to 67 per cent of each of the samples were determined. Using these areas as a measure of dispersion, the various combinations or relative wetness ratios were arranged in order of increasing error as shown in Table 2. Numbers shown in Table 2 indicate planimeter readings.

Table 2. RANKING OF RELATIVE WETNESS RATIO IN ORDER OF INCREASING ERROR OF ESTIMATE** OF R_{ij}.

Area	1 (least error)		2		3		4		5 (most error)	
I	Rs/ds	39.5	Ra/Ma	41.0	Rs/Ms	43.0	Ra/da*	43.0	Ra/A	57.0
II	Rs/Ms	18.0	Rs/ds	26.0	Ra/da	29.0	Ra/Ma	39.5	Ra/A	45.0
III	Ra/A	5.0	Ra/Ma	10.0	Rs/Ms	23.0	Ra/da	28.0	Rs/ds	30.0
IV	Rs/Ms	10.0	Rs/ds	13.5	Ra/da	19.0	Ra/A	23.0	Ra/Ma	32.0
V	Rs/ds	30.0	Ra/A*	37.5	Rs/Ms	50.0	Ra/Ma*	56.0	Ra/da	63.0

** Based on areas between ordinate and distribution-of-error curve for 0-67 per cent of the sample.

* Indicates error greater than 67 per cent of the sample within ± 25 per cent.

It will be noted from Table 2 that for four out of five cases, the best estimates resulted from use of seasonal data. Conversely, for four out of five cases, the worst estimates were associated with use of annual data.

Since R_{ij} vs. d_{ij} (seasonal basis) showed least error for sub-areas I and V and gave accuracy better than 67 per cent of the sample within ± 25 per cent for each of the other three areas, R_{ij} vs. d_{ij} was chosen as the relation for use throughout all five sub-areas, plus an additional sub-area No. VI. Location of the sub-areas is shown in Fig. 16. The relation between R_{ij} and d_{ij} for each of the sub-areas I through VI is shown in Fig. 17.

From this study, the following conclusions may be drawn:

1. Summer season (May through August) and annual precipitation values can serve as suitable parameters for making estimates of clock-hourly precipitation amounts having a two-year recurrence interval.

2. Seasonal precipitation parameters give slightly more accurate estimates of clock-hourly precipitation than annual parameters in the areas studied.

3. The study was confined to 2-year values because of the shortness of the records of clock-hourly precipitation. However, it seems reasonable that similar relations would apply to longer durations.

4. The "Relative Wetness" study indicates that sufficient correlation exists between clock-hourly and 24-hour precipitation amounts that the latter can be used to provide an acceptable estimate of the former. This principle was the basis of the decision to prepare maps of 24-hour precipitation amounts for correlation with runoff. The procedure for preparation of maps of 24-hour precipitation amounts is given in the following section.

PRECIPITATION MAPS OF 24-HOUR PRECIPITATION AMOUNTS HAVING RECURRENCE INTERVALS OF 2, 5, 10, 25, and 50 YEARS

Results of the study described in the preceding section indicated that 24-hour precipitation amounts should serve as suitable parameters for estimating short-duration, high-intensity rainfall which is related to peak rates of runoff from small watersheds.

A study was made to prepare maps of 24-hour precipitation amounts having recurrence intervals of 2, 5, 10, 25, and 50 years. The area included in the study consisted of the eastern part of Colorado, western Kansas and Nebraska, and portions of South Dakota, Wyoming, Texas, Oklahoma, and New Mexico, bounded approximately by the 96th and 105th meridians and the 36th and 45th parallels.

Records were collected from nine (9) first-order stations and 63 cooperative stations within this area. The stations were chosen primarily on the basis of uniform areal distribution and adequate length of record. The period of record was generally 30 years (1918-1947) with a break in record of one (1) year or less. Changes in locations and other factors that might affect the quality of the records from the cooperative stations were checked by referring to the "Records of Substation History," published by the U. S. Weather Bureau (2).

The data that were collected consisted of the "daily rainfall amounts in inches" as reported in records of Climatological Data published by the U. S. Weather Bureau. No corrections to reported amounts were made for differences in times of reading the rainfall amounts.

For the nine (9) first-order stations, additional data were collected in order to obtain a record compatible with those in "Technical Paper No. 25" (3) of the U. S. Weather Bureau. The additional data covered 9 years (1909-1917) for seven (7) stations and 15 years (1903-1917) for two (2) stations.

The annual series of maximum daily rainfall values were tabulated for the 72 stations, and a frequency analysis was made by employing the Gumbel Extreme Value method.

The regression equation (4) used was

$$X = \overline{X} - \left(\frac{Sx}{\sigma_N}\right) \overline{Y}_N + \left(\frac{Sx}{\sigma_N}\right) y$$

where

X is the extreme rainfall with a specified frequency

$\overline{X}$ is the sample mean of extreme value series, $\overline{X} = \frac{\Sigma X}{N}$

σ_N is the population standard deviation of reduced extreme values and is a function only of the sample size N. For N = 30 years, $\sigma_N = 1.1124$

Sx is the sample standard deviation of the extreme value series

$$Sx = \sqrt{\frac{\Sigma X^2 - \frac{(\Sigma X)^2}{N}}{N - 1}}$$

$\overline{Y}_N$ is the population mean of reduced variates as a function of sample size N, for N = 30 years, $\overline{Y}_N = 0.5362$

Y is the reduced variate, (linear division on abscissa scale)

Recurrence Interval Years	Y
2	0.36
5	1.50
10	2.22
25	3.20
50	3.88

A regression curve together with observed annual maximum values were plotted on Gumbel frequency paper for each station.

Maximum 24-hour rainfall amounts were available only for the first-order stations. Since the maximum 24-hour precipitation was considered more significant than the maximum daily rainfall for the runoff study, methods were sought for converting the former to the latter. It was found that the conversion factors that were developed were not dependent on geographic locations of the stations. Therefore, average ratios that were derived for the nine (9) first-order stations were assumed to apply to any station throughout the region studied.

Two methods of accomplishing the conversion from maximum daily precipitation to maximum 24-hour precipitation were examined in detail.

Method one - In the first method, the ratio of the maximum 24-hour values to the maximum daily value was computed for each frequency at each station. These ratios were obtained for each of the nine first-order stations for frequencies of 2, 5, 10, 25, and 50 years. The average ratios for the nine stations were as follows:

Frequency, years	2	5	10	25	50
Average ratio of $\frac{\text{Max. 24-hour precip.}}{\text{Max. daily precip.}}$	1.182	1.170	1.161	1.153	1.160

These values were obtained from analysis of records 40 to 45 years in length at the first-order stations. Since the period of record from all of the cooperative stations as distinguished from the first-order stations was only 30 years, a correction factor was required for each frequency to correct for the difference in the length of record. This factor was determined by computing the ratio of the daily precipitation values for the period corresponding to the long records with that for the 30-year records. The average correction factors were as follows:

Frequency, years	2	5	10	25	50
Average ratio $\frac{\text{Max. daily precip. (Long term)}}{\text{Max. daily precip. (30 years)}}$	1.022	0.995	0.991	0.987	0.985

The product of the two ratios for each frequency yields the ratio of the maximum 24-hour precipitation to the maximum daily precipitation (30-year data). The appropriate ratios are as follows:

Frequency, years	2	5	10	25	50
$\frac{\text{Max. 24-hour precip.}}{\text{Max. daily precip. from 30-year data}}$	1.208	1.164	1.151	1.138	1.143

Method Two - In the second method the quotient of the maximum 24-hour values with 5, 10, 25, and 50-year frequencies were divided by the maximum 24-hour values with a 2-year frequency. The average quotients for the nine first-order stations are as follows:

Frequency, years	2	5	10	25	50
Average quotient $\frac{\text{Max. 24-hour value}}{\text{2-year 24-hr value}}$	1.0	1.43	1.70	2.06	2.34

These values plot as a straight line on Gumbel paper. To use the second method, requires that the first method be employed to obtain the maximum 24-hour precipitation value with a 2-year frequency. Values for other frequencies are obtained by multiplying this value with the quotient $\frac{\text{Max. 24-hour value}}{\text{2-year 24-hour value}}$ from the preceding table.

A comparison of the results of the use of these two methods is given in Table 3. The departures from "true" values were computed with respect to the values given in U. S. Weather Bureau Technical Paper No. 25, which were assumed to be the correct values. A "+" sign indicates that the computed value is an overestimation, and vice versa.

Table 3 shows that both methods give approximately equal accuracy, and that the average errors are about 4 to 5 per cent. The first method was considered to be slightly better than the second, and was used to compute maximum 24-hour precipitation amount with various frequencies for all 72 stations.

These computed values were used to prepared isohyetal maps for 2, 5, 10, 25, and 50-year recurrence intervals in Fig. 10 and Figs. 18-21.

It is of interest to note that the isohyetal patterns for the 25 and 50-year frequencies show a greater variation than those for the 2-year frequency. This variation is such that high values of precipitation for the longer recurrence intervals seem to be associated with ridges of higher surface elevation.

This phenomenon suggests that the physical mechanism for the thunderstorms that produce the heavy rains may be related to topography to a greater degree than those storms which do not produce such heavy rains.

Table 3. COMPARISON OF TWO METHODS OF DETERMINING MAXIMUM 24-HOUR PRECIPITATION FOR NINE FIRST-ORDER WEATHER BUREAU STATIONS

Frequency	2-yr		5-yr		10-yr		25-yr		50-yr	
Method	I	II	I	II	I	II	I	II	I	II
Denver Colo.	- .05" -3.4 %	- .05" -3.4 %	- .11" -5.4 %	+ .02" +1.0 %	- .12" -5.0 %	+ .16" +6.7 %	- .19" - 6.6 %	+ .09" + 3.1 %	- .21" - 6.5 %	+ .13" +4.0 %
Pueblo Colo.	- .01" -0.7 %	- .01" -0.7 %	0 0	+ .05" +2.5 %	- .02" -0.8 %	+ .05" +2.1 %	- .03" 1.0 %	+ .07" + 2.4 %	- .02" - 0.6 %	+ .11" +3.4 %
Concordia Kansas	- .19" -7.2 %	- .19" -7.2 %	- .20" -5.6 %	- .10" -2.8 %	- .31" -7.2 %	- .16" -3.7 %	- .44" - 8.3 %	- .23" - 4.4 %	- .54" - 9.0 %	- .27" -4.5 %
Dodge City Kansas	+ .13" +5.8 %	+ .13" +5.8 %	- .02" -0.6 %	+ .18" +5.6 %	- .09" -2.3 %	+ .22" +5.7 %	- .08" - 1.8 %	+ .36" + 7.9 %	- .25" - 4.7 %	+ .31" +5.9 %
Wichita Kansas	+ .18" +5.6 %	+ .18" +5.6 %	+ .37" +8.1 %	+ .34" +7.5 %	+ .36" +6.5 %	+ .29" +5.2 %	+ .70" +10.8 %	+ .56" + 8.6 %	+ .84" +11.5 %	+ .68" +9.3 %
N. Platte Nebraska	+ .13" +6.4 %	+ .13" +6.4 %	- .01" -0.3 %	- .05" -1.6 %	+ .13" +3.6 '	+ .06" +1.7 %	+ .04" + 0.9 %	- .08" + 1.8 %	+ .03" + 0.6 %	- .13" -2.5 %
Valentine Nebraska	0 0	0 0	- .03" -1.1 %	- .04" -1.4 %	- .03" -0.9 %	- .05" -1.5 %	+ .03" + 0.8 '	- .01" - 0.2 %	+ .09" + 2.0 %	+ .04" +0.9 %
Rapid City S. Dakota	0 0	0 0	+ .09" +2.9 %	- .20" -6.4 %	+ .24" +6.4 %	- .25" -6.7 %	+ .28" + 6.0 %	- .48" -10.2 %	+ .42" + 8.0 %	- .51" -9.6 %
Cheyenne Wyoming	+ .10" +7.0 %	+ .10" +7.0 %	+ .07" +3.4 %	+ .13" +6.4 %	+ .09" +3.8 %	+ .18" +7.5 %	+ .11" + 3.8 %	+ .25" + 8.7 %	+ .14" + 4.3 %	+ .32" +9.9 %
Average %	4.0	4.0	3.0	3.9	4.1	4.5	4.4	5.2	5.2	5.6

IDENTIFICATION OF GAGING STATIONS USED IN THE STUDY

Records of peak rates of runoff were collected from stations within and near the D-13, D-20, and E-5 areas. The criteria used in selecting stations suitable for inclusion in the analysis were:

1. The length of record was equal to or greater than 7 years for stations in the D-13, D-20, and E-5 areas; and 23 years for stations outside of the problem area.
2. Records of annual maximum stream flow had to be derived from recording gages only. Records derived from staff gage readings were discarded.
3. Records were not used if there were more than four years break in records
4. No record was utilized where there had been a change in location of site greater than two miles up or downstream.
5. The watershed contributing area was not more than 1500 square miles for stations within the problem areas, and 2000 square miles for stations outside the problem area.
6. No significant artificial flow control existed for high flows.

Stations included in the study that were located outside of problem areas E-5, D-13, and D-20 fell into four general geographic locations: northwest, east, southeast, and southwest of the problem areas D-13 and D-20, defined respectively by the following locations:

Northwest: 45 to 49 degrees north by 106 to 113 degrees west.

East: 37 to 43 degrees north by 94 to 100 degrees west.

Southeast: 29 to 35 degrees north by 94 to 101 degrees west.

Southwest: 34 to 38 degrees north by 102 to 107 degrees west.

Identification of gaging stations inside the problem areas is given in Tables 4 and 5. A map of the locations of the stations used to develop the design charts is shown in Fig. 22.

TABLE 4. IDENTIFICATION OF GAGING STATIONS USED IN THE STUDY

Stations Inside D-13 & D-20 Problem Areas, Including Fringes

Serial Number	Name	Refer to U.S.G.S. Water Supply Paper	Location		Drainage Area in Sq. Mile		Period of Record in Years
			Longitude	Latitude	Nominal	Contributing	
1	Fountain Creek at Pueblo, Colo.	159-1311	104-35-40	38-16-20	926	------	17
3	Apishapa River near Fowler, Colo.	184-1311	103-59	38-05	1125	------	20
4	Timpas Creek near Rocky Ford, Colo.	186-1311	103-43-20	37-57-20	451	------	9
5	Horse Creek near Sugar City, Colo.	191-1311	103-37-40	38-14-10	1080	------	8
9	Rawhide Creek near Lingle, Wyo.	126-1310	104-19-20	42-07-30	510	------	23
10	Blue Creek near Lewellen, Nebr.	155-1310	102-10	41-20	267	------	24
11	Birdwood Creek near Hershey, Nebr.	165-1310	101-04	41-13	286	------	23
12	Cherry Creek near Franktown, Colo.	201-1310	104-45-50	39-21-30	172	------	19
13	Cherry Creek near Melvin, Colo.	202-1310	104-49-15	39-36-20	369	------	19
14	Cherry Creek below Cherry Creek Dam, Colo.	203-1310	104-51-40	39-39-10	386	------	9
15	Cherry Creek at Denver, Colorado	204-1310	105-00-08	39-44-58	420	------	16
16	Lodgepole Creek at Bushnell, Nebr.	288-1310	103-51	41-14	1090	------	20
18	North Fork Republican River at Colorado-Nebraska State Line	387-1310	102-03-05	40-04-10	320	------	24
19	Buffalo Creek near Haigler, Nebr.	388-1310	101-52-15	40-02-45	180	21	18
20	Rock Creek near Parks, Nebr.	389-1310	101-43-40	40-02-30	180	14	12
22	Frenchman Creek below Champion, Nebr.	400-1310	101-43-10	40-28-00	940	570	22
23	Sappa Creek near Oberlin, Kansas	420-1310	100-32-02	39-48-45	1040	unknown	15
24	White River at Crawford, Nebr.	332-1439	103-25	42-41	313	------	17
25	Niobrara River above Box Butte Reservoir, Nebr.	343-1439	103-10-15	42-27-35	1400	980	9
31	Pumpkin Creek near Bridgeport, Nebr.	151-1310	103-02	41-38	1080	------	24
33	Landsman Creek near Hale, Colo.	392-1310	102-14-50	39-34-40	450	------	9
34	South Fork Republican River near Idalia, Colo.	391-1310	102-14-30	39-37-00	1300	------	8
35	Cottonwood Creek at Wendover, Wyo.	99-1310	104-52-33	42-19-32	159	------	23
36	Frenchman Creek near Hamlet, Nebr.	403-1310	101-12-50	40-22-30	1480	960	24
37	Purgatoire River at Trinidad, Colo.	193-1311	104-30-30	37-10-15	795	------	37
38	Vermejo River near Dawson, New Mexico	323-1311	104-47-05	36-40-50	301	------	30
40	Sheep Creek near Morrill, Nebr.	137-1310	103-56	41-58	?	------	18
41	Dry Spotted Tail Creek at Mitchell, Nebr.	139-1310	103-50	41-57	?	------	10
42	Tub Spring near Scottsbluff, Nebr.	141-1310	103-43	41-55	?	------	10
43	Winter Creek near Scottsbluff, Nebr.	143-1310	103-37	41-52	?	------	20
44	Ninemile Drain near Mcgrew, Nebr.	146-1310	103-24	41-46	?	------	20
45	Bayard Sugar Factory Drain near Bayard, Nebr.	147-1310	103-19	41-44	?	------	20
46	Red Willow Creek near Bayard, Nebr.	149-1310	103-15	41-43	?	------	20
47	Bijou Creek near Wiggins, Colo.	282-1310	104-02-08	40-14-53	1420	------	7
48	Buffalo Creek near Darr, Nebr.	300-1310	99-50-00	40-54-00	63	------	10
49	Buffalo Creek near Overton, Nebr.	301-1310	99-30-20	40-44	175	------	9
50	Elm Creek near Overton, Nebr.	302-1310	99-30-20	40-50-40	31	------	12
51	Wood River near Riverdale, Nebr.	305-1310	99-11-50	40-47-50	379	------	12
52	Wood River near Gibbon, Nebr.	306-1310	98-48-00	40-46-10	572	------	10
53	Middle Loup River at Dunning, Nebr.	312-1310	100-06-20	41-49-50	1760	80	9
54	Middle Loup River at Arcadia, Nebr.	318-1310	99-08-10	41-25-20	4730	820	19
55	South Loup River at Ravenna, Nebr.	322-1310	98-54-45	41-00-35	1660	890	14
56	Mud Creek near Sweetwater, Nebr.	324-1310	98-59-45	41-02-05	678	------	11
57	Oak Creek near Dannebrog, Nebr.	326-1310	98-38-30	41-07-00	122	------	8
58	Arikaree River at Haigler, Nebr.	385-1310	101-57-25	40-01-30	1460	------	19
59	South Fork Republican River near Hale, Colo.	393-1310	102-09-45	39-37-25	?	------	9
60	Frenchman Creek near Imperial, Nebr.	401-1310	101-37-30	40-25-20	1220	760	17
61	Frenchman Creek near Enders, Nebr.	402-1310	101-30-35	40-25-05	1300	820	12
62	Frenchman Creek at Palisade, Nebr.	404-1310	101-07-40	40-20-50	1500	980	8
63	Stinking Water Creek near Wauneta, Nebr.	405-1310	101-19-50	40-29-20	1260	340	10
64	Stinking Water Creek near Palisade, Nebr.	406-1310	101-06-50	40-22-10	1390	430	9
65	Blackwood Creek near Culbertson, Nebr.	408-1310	100-48-25	40-14-05	290	------	12
66	Driftwood Creek near McCook, Nebr.	409-1310	100-39-40	40-08-50	360	------	13
67	Red Willow Creek near McCook, Nebr.	411-1310	100-39	40-21	600	300	7
68	Red Willow Creek near Red Willow, Nebr.	412-1310	100-30-00	40-14-10	710	400	18
69	Medicine Creek at Maywood, Nebr.	290-1440	100-36-40	40-39-20	207	------	8
70	Fox Creek at Curtis, Nebr.	292-1440	100-29-20	40-38-00	77	------	8
71	Dry Creek near Curtis, Nebr.	293-1440	100-26-40	40-38-05	20	------	8
72	Medicine Creek above Harry Strunk Lake, Nebr.	413-1310	100-19-20	40-30-10	?	------	9
73	Mitchell Creek above Harry Strunk Lake, Nebr.	414-1310	100-15-25	40-28-20	53	------	9
74	Medicine Creek at Cambridge, Nebr.	417-1310	100-10-35	40-17-55	1070	680	20
75	Muddy Creek at Arapahoe, Nebr.	300-1440	99-54-40	40-18-20	243	------	8
76	Prairie Dog Creek at Norton, Kans.	426-1310	99-53	39-50	721	------	15
77	Cottonwood Creek near Bloomington, Nebr.	311-1440	99-03-55	40-05-10	17	------	7
78	Rose Creek near Wallace, Kans.	441-1310	101-38	38-53	28	------	7
79	North Fork Smoky Hill River near McAllaster, Kans.	442-1310	101-22	39-01	670	------	7
80	Big Creek near Hays, Kans.	448-1310	99-19	38-51	594	------	13
81	Bow Creek near Stockton, Kans.	351-1440	99-17	39-34	337	------	8
82	North Fork Solomon River at Kirwin, Kans.	461-1310	99-07	39-40	1360	------	11
83	Fountain Creek near Fountain, Colo.	158-1311	104-40-13	38-36-08	676	------	16
84	St. Charles River near Pueblo, Colo.	163-1311	104-31-40	38-12-20	468	------	12
85	Apishapa River near Aguilar, Colo.	181-1311	104-39-50	37-22-50	126	------	11
86	Purgatoire River near Alfalfa, Colo.	194-1311	104-07-30	37-11-30	1320	------	7
87	Cimarron River near Guy, New Mexico	240-1311	103-25-25	36-59-15	545	------	17
88	Canadian River near Hebron, New Mexico	316-1311	104-27-45	36-47-10	229	------	12
89	White River below Cottonwood Creek near Whitney, Nebr.	333-1439	103-10-05	42-48-35	676	------	8

TABLE 4. IDENTIFICATION OF GAGING STATIONS USED IN THE STUDY

Stations Outside D-13 & D-20 Problem Areas

Serial Number	Name	Refer to U.S.G.S. Water Supply Paper	Location		Drainage Area in Sq. Mile		Period of Records in Years
			Longitude	Latitude	Nominal	Contributing	
101	Floyd River at James, Iowa	3-1310	96-18-45	42-34-30	882	------	22
102	Elkhorn River at Neligh, Nebr.	352-1310	98-01-40	42-07-20	2200	1800	25
103	Tarkio River at Fairfax, Mo.	371-1310	95-24-20	40-20-20	508	------	34
104	Nodaway River near Burlington Junction, Mo.	377-1310	95-05-20	40-26-40	1240	------	34
105	Little Blue River near Endicott, Nebr.	478-1310	97-08-10	40-05-10	2340	------	28
106	Soldier Creek near Topeka, Kans.	484-1310	95-43	39-06	268	------	25
107	Delaware River at Valley Falls, Kans.	485-1310	95-27	39-21	922	------	35
108	Wakarusa River near Lawrence, Kans.	487-1310	95-16	38-55	458	------	28
109	Stranger Creek near Tonganoxia, Kans.	488-1310	95-01-08	39-06-06	406	------	28
110	Marais des Cygnes River near Ottawa, Kans.	524-1310	95-15	38-37	1250	------	38
111	Pawnee River near Larned, Kans.	219-1311	99-20	38-11	2148	2010	32
112	Little Arkansas River at Valley Center, Kansas	224-1311	97-23	37-50	1327	1250	34
113	Walnut River at Winfield, Kans.	229-1311	97-00	37-14	1840	------	35
114	Spring River near Waco, Mo.	296-1311	94-33-55	37-14-45	1164	------	26
115	Rayado Creek at Sauble Ranch, near Cimarron, New Mexico	333-1311	104-58	36-22	65	------	33
116	Cimarron River at Springer, New Mexico	338-1311	104-35-50	36-21-30	1032	------	27
117	Mora River near Golondrinas, New Mexico	346-1311	105-09-30	35-53-40	273	------	26
118	Coyote Creek near Golondrinas, New Mexico	348-1311	105-09-50	35-54-40	257	------	28
119	Mora River near Shoemaker, New Mexico	351-1311	104-47	35-48	1104	1033	39
120	Mountain Fork River near Eagletown, Okla.	533-1311	94-37	34-03	787	------	27
121	Kiamichi River near Belzoni, Okla.	528-1311	95-29	34-12	1423	------	31
122	Judith River near Utica, Mont.	67-1439	110-14	46-54	331	------	36
123	Musselshell River at Harlowton, Mont.	75-1439	109-51	46-26	1130	------	42
124	Flatwillow Creek near Flatwillow, Mont.	81-1439	108-37	46-47	195	------	34
125	South Fork Milk River near International Boundary	86-1439	112-32-20	49-00	433	------	43
126	North Fork Milk River above St. Mary Canal near Browning, Mont.	87-1439	113-03	48-59	62	------	36
127	North Fork Milk River near International Boundary	88-1439	112-58	49-02	101	------	42
128	Battle Creek at International Boundary	99-1439	109-25-20	49-00-10	726	------	39
129	Woodpile Coulee near International Boundary	100-1439	109-31-50	48-59-00	70	------	27
130	East Fork Battle Creek near International Boundary	101-1439	109-08	48-58	95	------	26
131	Whitewater Creek near International Boundary	104-1439	107-51	48-57	300	------	29
132	Clarks Fork at Chance, Mont.	143-1439	109-05	45-00	1140	------	22
133	Bull Lake Creek near Lenore, Wyo.	154-1439	109-01-20	43-14-33	222	------	39
134	Greybull River at Meeteetse, Wyo.	181-1439	108-52-35	44-09-20	690	------	36
135	Goose Creek near Sheridan, Wyo.	205-1439	107-11	44-42	120	------	27
136	Clear Fork Trinity River at Fort Worth, Texas	75-1442	97-21	32-44	526	--------	24
137	Middle Concho River near Tankersly, Texas	172-1442	100-36-50	31-22-35	1280	1128	26
138	North Concho River near Carlsbad, Texas	175-1442	100-39	31-36	1533	1410	26
139	Pecan Bayou At Brownwood, Texas	188-1442	98-58-30	31-44-10	1614	--------	26
140	North Llano River near Junction, Texas	195-1442	99-47	30-30	914	--------	31
141	Llano River near Junction, Texas	196-1442	99-44	30-30	1874	--------	24
142	Guadalupe River near Spring Branch, Tex.	213-1442	98-23	29-51-40	1282	--------	30
143	Guadalupe River above Comal River at New Braunfels, Tex.	214-1442	98-06-40	29-42-55	1516	--------	28
144	Blanco River at Wimberley, Tex.	216-1442	98-04	29-59	364	--------	27
145	Plum Creek near Luling, Tex.	219-1442	97-37	29-42	356	--------	26
146	Cibolo Creek near Falls City, Tex.	229-1442	97-56	29-01	831	--------	24
147	Nueces River at Laguna, Tex.	234-1442	99-59-50	29-25-45	764	--------	29
148	Frio River at Concan, Tex.	239-1442	99-42	29-29	405	--------	23
149	Rio Grande near Del Norte, Colo.	259-1442	106-27-30	37-41-20	1320	--------	46
150	Cunejos River near Mogote, Colo.	278-1442	106-11-20	37-03-20	282	--------	44
151	Red River near Questa, New Mexico	298-1442	105-34	36-42-10	112	--------	25
152	Santa Fe River near Santa Fe, New Mexico	318-1442	105-50-35	35-41-10	20	--------	25
153	Blue Water Creek near Bluewater, New Mexico	333-1442	108-01-40	35-17-50	235	--------	25
154	Pecos River near Pecos, New Mexico	352-1442	105-41	35-42-25	---	189	25
155	Pecos River near Anton Chico, New Mexico	353-1442	105-06-20	35-10-50	---	1050	23
156	Gallinas River near Montezuma, New Mexico	354-1442	105-19-10	35-39	84	--------	24
157	Gallinas River at Montezuma, New Mexico	355-1442	105-16-30	35-39-15	87	--------	23
158	Mimbres River near Mimbres, New Mexico	403-1442	107-59	32-52-20	152	--------	25

TABLE 4 IDENTIFICATION OF GAGING STATIONS USED IN THE STUDY

E-5 PROBLEM AREA

Serial Number	Name	Refer to U.S.G.S. Water Supply Paper	Location		Drainage Area in Sq. Mi.	Period of Record in Years
			Longitude	Latitude		
200	Illinois Creek near Rand, Colo.	30-1310	106-11-00	40-27-00	71	9
201	Willow Creek near Rand, Colo.	31-1310	106-13-00	40-28-00	71	9
202	Canadian River at Cowdrey, Colo.	34-1310	106-19-00	40-52-00	174	11
204	French Creek near French, Wyo.	39-1310	106-31-00	41-12-30	60	9
205	Deer Creek at Glenrock, Wyo.	85-1310	105-52-02	42-51-42	216	22
206	LaPrele Creek near Douglas, Wyo.	88-1310	105-36-00	42-40-00	146	38
207	Horseshoe Creek near Gelndo, Wyo.	96-1310	104-58-11	42-27-09	203	28
208	Cottonwood Creek at Wendover, Wyo.	99-1310	104-52-33	42-19-32	159	23
209	Laramie River near Glendevey, Colo.	103-1310	105-52-40	40-48-00	101	38
210	Laramie River near Jelm, Wyo	105-1310	106-00-50	41-00-10	297	47
211	Little Laramie River near Filmore, Wyo.	111-1310	106-02-30	41-17-20	155	19
212	Little Laramie River at Two Rivers, Wyo.	112-1310	105-43-50	41-28-10	310	36
213	North Fork South Platte River below Geneva Creek at Grant, Colo.	189-1310	105-39-28	39-27-28	127	8
214	North Fork South Platte River at South Platte, Colo.	191-1310	105-10-30	39-24-30	484	47
215	Bear Creek at Morrison, Colo.	198-1310	105-11-40	39-39-10	165	24
216	Turkey Creek near Morrison, Colo.	199-1310	105-10-05	39-38-10	49.4	12
217	Cherry Creek near Franktown, Colo.	201-1310	104-45-50	39-21-30	172	19
218	St. Vrain Creek at Lyons, Colo.	223-1310	105-15-40	40-13-10	226	35
219	Boulder Creek at mouth, near Longmont, Colo.	235-1310	105-01-00	40-08-05	512	28
220	Cache la Poudre River at mouth of canyon near Ft. Collins, Colo.	272-1310	105-13-00	40-39-55	1,048	50
221	Middle Crow Creek near Hecla, Wyo.	276-1310	105-15-10	41-10-30	23	25
222	South Crow Creek near Hecla, Wyo.	277-1310	105-12-00	41-07-40	16	24
223	Grape Creek near Westcliff, Colo.	143-1311	105-30-00	38-11-00	320	30
224	Hverfano River at Manzanares Crossing, near Redwing, Colo.	168-1311	105-21-10	37-43-40	73	33
225	Cucharas River at Boyd Ranch near Le Veta, Colo.	174-1311	105-03-00	37-25-00	56	21
226	Apishapa River near Aguilar, Colo.	181-1311	104-39-50	37-22-50	126	11
227	Purgatoire River at Trinidad, Colo.	193-1311	104-30-30	37-10-15	795	37
228	Vermejo River near Dawson, N. M.	323-1311	104-47-05	36-40-50	301	30
229	Six Mile Creek near Eagle Nest, New Mexico	326-1311	105-16-15	36-31-10	11	24
230	Ponil Creek near Cimarron, N. M.	332-1311	104-56-55	36-34-35	171	12
231	Mora River near Golondrinas, N. M.	346-1311	105-09-30	35-53-40	273	31
232	Coyote Creek near Golondrinas, N. M.	348-1311	105-09-50	35-54-40	257	28
233	Mora River near Shoemaker, N. M.	351-1311	104-47-00	35-48-00	1,104	39

TABLE 5. GAGING STATIONS IN ALPHABETICAL ORDER

Name	Serial Number	Refer. to USGS Water Supply Paper
Apishapa River near Aguilar, Colo.	85	181-1311
Apishapa River near Aguilar, Colo.	226	181-1311
Apishapa River near Fowler, Colo.	3	184-1311
Arikaree River at Haigler, Nebr.	58	385-1310
Battle Creek at International Boundary	128	99-1439
Bayard Sugar Factory Drain near Bayard, Nebr.	45	147-1310
Bear Creek at Morrison, Colo.	215	198-1310
Big Creek near Hays, Kans.	80	448-1310
Bijou Creek near Wiggins, Colo.	47	282-1310
Birdwood Creek near Hershey, Nebr.	11	165-1310
Blackwood Creek near Culbertson, Nebr.	65	408-1310
Blanco River at Wimberley, Tex.	144	216-1442
Blue Creek near Lewellen, Nebr.	10	155-1310
Bluewater Creek near Bluewater, N. M.	153	333-1442
Boulder Creek at mouth, near Longmont, Colo.	219	235-1310
Bow Creek near Stockton, Kans.	81	351-1440
Buffalo Creek near Darr, Nebr.	48	300-1310
Buffalo Creek near Haigler, Nebr.	19	388-1310
Buffalo Creek near Overton, Nebr.	49	301-1310
Bull Lake Creek near Lenore, Wyo.	133	154-1439
Cache la Poudre River at mouth of Canyon near Fort Collins, Colo.	220	272-1310
Canadian River near Hebron, N. M.	88	316-1311
Canadian River at Cowdrey, Colo.	202	34-1310
Cherry Creek below Cherry Creek Dam, Colo.	14	203-1310
Cherry Creek at Denver, Colo.	15	204-1310
Cherry Creek near Franktown, Colo.	12	201-1310
Cherry Creek near Franktown, Colo.	217	201-1310
Cherry Creek near Melvin, Colo.	13	202-1310
Cibolo Creek near Falls City, Tex.	146	229-1442
Cimarron River near Guy, N. M.	87	240-1311
Cimarron River at Springer, N. M.	116	338-1311
Clarks Fork at Chance, Mont.	132	143-1439
Clear Fork Trinity River at Fort Worth, Tex.	136	75-1442
Cottonwood Creek near Bloomington, Nebr.	77	311-1440
Cottonwood Creek at Wendover, Wyo.	35	99-1310
Cottonwood Creek at Wendover, Wyo.	208	99-1310
Cucharas River at Boyd Ranch near Le Veta, Colo.	225	174-1311
Coyote Creek near Golondrinas, N. M.	118	348-1311
Coyote Creek near Golondrinas, N. M.	232	348-1311
Cunejo River near Mogote, Colo.	150	278-1442
Deer Creek at Glenrock, Wyo.	205	85-1310

TABLE 5. GAGING STATIONS IN ALPHABETICAL ORDER (Cont'd)

Delaware River at Valley Falls, Kans.	107	485-1310
Driftwood Creek near McCook, Nebr.	66	409-1310
Dry Creek near Curtis, Nebr.	71	293-1440
Dry Spotted Tail Creek at Mitchell, Nebr.	41	139-1310
East Fork Battle Creek near International Boundary	130	101-1439
Elkhorn River at Neligh, Nebr.	102	352-1310
Elm Creek near Overton, Nebr.	50	302-1310
Flatwillow Creek near Flatwillow, Mont.	124	81-1439
Floyd River at James, Iowa	101	3-1310
Fountain Creek near Fountain, Colo.	83	158-1311
Fountain Creek at Pueblo, Colo.	1	159-1311
Fox Creek at Curtis, Nebr.	70	292-1440
French Creek near French, Wyo.	204	39-1310
Frenchman Creek below Champion, Nebr.	22	400-1310
Frenchman Creek near Enders, Nebr.	61	402-1310
Frenchman Creek near Hamlet, Nebr.	36	403-1310
Frenchman Creek near Imperial, Nebr.	60	401-1310
Frenchman Creek at Palisade, Nebr.	62	404-1310
Frio River at Concan, Tex.	148	239-1442
Gallinas River at Montezuma, N. M.	157	355-1442
Gallinas River near Montezuma, N. M.	156	354-1442
Goose Creek near Sheridan, Wyo.	135	205-1439
Grape Creek near West Cliff, Colo.	223	143-1311
Greybull River at Meeteetse, Wyo.	134	181-1439
Guadalupe River above Comal River at New Braunfels, Tex.	143	214-1442
Guadalupe River near Spring Branch, Tex.	142	213-1442
Horse Creek near Sugar City, Colo.	5	191-1311
Horseshoe Creek near Glendo, Wyo.	207	96-1310
Hverfano River at Manzanares Crossing near Red Wing, Colo.	224	168-1311
Illinois Creek near Rand, Colo.	200	30-1310
Judith River near Utica, Mont.	122	67-1439
Kiamichi River near Belzoni, Okla.	121	528-1311
La Prele Creek near Douglas, Wyo.	206	88-1310
Landsman Creek near Hale, Colo.	33	392-1310
Laramie River near Glendevey, Colo.	209	103-1310
Laramie River near Jelm, Wyo.	210	105-1310
Little Arkansas River at Valley Center, Kans.	112	224-1311
Little Blue near Endicott, Nebr.	105	478-1310
Little Laramie River near Filmore, Wyo.	211	111-1310
Little Laramie River at Two Rivers, Wyo.	212	112-1310
Llano River near Junction, Tex.	141	196-1442
Lodgepole Creek at Bushnell, Nebr.	16	288-1310
Marais des Cygnes River near Ottowa, Kans.	110	524-1310
Medicine Creek at Cambridge, Nebr.	74	417-1310
Medicine Creek above Harry Strunk Lake, Nebr.	72	413-1310
Medicine Creek at Maywood, Nebr.	69	290-1440
Middle Concho River near Tankersly, Tex.	137	172-1442

TABLE 5. GAGING STATIONS IN ALPHABETICAL ORDER (Cont'd)

Middle Crow Creek near Hecla, Wyo.	221	276-1310
Middle Loup River at Arcadia, Nebr.	54	318-1310
Middle Loup River at Dunning, Nebr.	53	312-1310
Mimbres River near Mimbres, N. M.	158	403-1442
Mitchell Creek above Harry Strunk Lake, Nebr.	73	414-1310
Mora River near Golondrinas, N. M.	117	346-1311
Mora River near Golondrinas, N. M.	231	346-1311
Mora River near Shoemaker, N. M.	119	351-1311
Mora River near Shoemaker, N. M.	233	351-1311
Mountain Fork River near Eagletown, Okla.	120	533-1311
Mud Creek near Sweetwater, Nebr.	56	324-1310
Muddy Creek at Arapahoe, Nebr.	75	300-1440
Musselshell River at Harlowton, Mont.	123	75-1439
Ninemile Drain near Mcgrew, Nebr.	44	146-1310
Niobrara River above Box Butte Reservoir, Nebr.	25	343-1439
Nodaway River near Burlington Junction, Mo.	104	377-1310
North Concho River near Carlsbad, Tex.	138	175-1442
North Fork Milk River near International Boundary	127	88-1439
North Fork Milk River above St. Mary Canal near Browning, Mont.	126	87-1439
N.F.Republican River at Colorado-Nebraska State Line	18	387-1310
North Fork Smoky Hill River near McAllaster, Kans.	79	442-1310
North Fork Solomon River at Kirwin, Kans.	82	461-1310
North Fork South Platte River below Geneva Creek at Grant, Colo.	213	189-1310
North Fork South Platte River at South Platte, Colo.	214	191-1310
North Llano River near Junction, Tex.	140	195-1442
Nueces River at Laguna, Tex.	147	234-1442
Oak Creek near Dannebrog, Nebr.	57	326-1310
Pawnee River near Larned, Kans.	111	219-1311
Pecan Bayou at Brownwood, Tex.	139	188-1442
Pecos River near Anton Chico, N. M.	155	353-1442
Pecos River near Pecos, N. M.	154	352-1442
Plum Creek near Luling, Tex.	145	219-1442
Ponil Creek near Cimarron, New Mexico	230	332-1311
Prairie Dog Creek at Norton, Kans.	76	426-1310
Pumpkin Creek near Bridgeport, Nebr.	31	151-1310
Purgatoire River near Alfalfa, Colo.	86	194-1311
Purgatoire River at Trinidad, Colo.	37	193-1311
Purgatoire River at Trinidad, Colo.	227	193-1311
Rawhide Creek near Lingle, Wyo.	9	126-1310
Rayado Creek at Sauble Ranch, near Cimarron, N.M.	115	333-1311
Red River near Questa, N.M.	151	298-1442
Red Willow Creek near Bayard, Nebr.	46	149-1310
Red Willow Creek near McCook, Nebr.	67	411-1310
Red Willow Creek near Red Willow, Nebr.	68	412-1310
Rio Grande near Del Norte, Colo.	149	259-1442

TABLE 5. GAGING STATIONS IN ALPHABETICAL ORDER (Cont'd)

Rock Creek near Parks, Nebr.	20	389-1310
Rose Creek near Wallace, Kans.	78	441-1310
Santa Fe River near Santa Fe, N. M.	152	318-1442
Sappa Creek near Oberlin, Kans.	23	420-1310
Six Mile Creek near Eagle Nest, New Mexico	229	326-1311
Soldier Creek near Topeka, Kans.	106	484-1310
South Crow Creek near Hecla, Wyo.	222	277-1310
South Fork Milk River near International Boundary	125	86-1439
South Fork Republican River near Hale, Colo.	59	393-1310
South Fork Republican River near Idalia, Colo.	34	391-1310
South Loup River at Ravenna, Nebr.	55	322-1310
Sheep Creek near Morrill, Nebr.	40	137-1310
Spring River near Waco, Mo.	114	296-1311
St. Charles River near Pueblo, Colo.	84	163-1311
St. Vrain Creek at Lyons, Colo.	218	223-1310
Stinking Water Creek near Palisade, Nebr.	64	406-1310
Stinking Water Creek near Wauneta, Nebr.	63	405-1310
Stranger Creek near Tonganoxia, Kans.	109	488-1310
Tarkio River at Fairfax, Mo.	103	371-1310
Timpas Creek near Rocky Ford, Colo.	4	186-1311
Tub Spring near Scottsbluff, Nebr.	42	141-1310
Turkey Creek near Morrison, Colo.	216	199-1310
Vermejo River near Dawson, N. M.	38	323-1311
Vermejo River near Dawson, N. M.	228	323-1311
Wakarusa River near Lawrence, Kans.	108	487-1310
Walnut River at Winfield, Kans.	113	229-1311
White River below Cottonwood Creek near Whitney, Nebr.	89	333-1439
White River at Crawford, Nebr.	24	332-1439
Whitewater Creek near International Boundary	131	104-1439
Willow Creek near Rand, Colo.	201	31-1310
Winter Creek near Scottsbluff, Nebr.	43	143-1310
Wood River near Gibbon, Nebr.	52	306-1310
Wood River near Riverdale, Nebr.	51	305-1310
Woodpile Coulee near International Boundary	129	100-1439

FREQUENCY ANALYSIS

After all records of peak rates of runoff had been collected that met the criteria described in the foregoing section, the flood events were then plotted on Gumbel extreme-value paper.

The data were then analyzed on the basis of techniques developed by Potter (5) and Benson (6). Potter's method approximates an array of points on Gumbel paper by two straight lines, giving a "dog-leg" in those cases for which the plotted points are nonlinear. Benson's method consists of drawing a curved line by eye that best fits the array of plotted points.

Drawing a curved line on Gumbel paper departs significantly from the straight line that theoretically should represent extreme values. Acceptance of a curved line on Gumbel paper implies the existence of a limiting discharge for a curve that is concave downward or of a limiting recurrence interval for a curve that is concave upward. While a limitation on the maximum possible discharge may be possible on physical reasoning, a more common occurrence in the area studied was a curve that was concave upward.

The method of Potter in fitting two straight lines to the plotted points on Gumbel paper does not suffer these limitations, although for some records, difficulty was experienced in obtaining a suitable fit for the data with two straight lines.

A comparison was made of these two methods of analysis, as applied to watersheds in the D-13 and D-20 problem areas. The comparison is described in the following section.

Potter's method of analysis was used for stations in the E-5 area.

RELATIONS BETWEEN SHORT-TERM AND LONG-TERM PEAK RATES OF RUNOFF

A study was made to compare the analysis technique of Potter and Benson on records of runoff from the D-13 and D-20 areas.

Benson's technique was applied to records of runoff from seventeen watersheds in the D-13 and D-20 problem areas with discharges having a recurrence interval of 2 years (Q_2), 5 years (Q_5), 10 years (Q_{10}), and 15 years (Q_{15}). Estimates were made of Q_{25}.

Because of the lack of long-term records, attempts were made to relate short-term discharge values (Q_2, Q_5, Q_{10}) to longer-term discharge values (Q_{15}, Q_{25}) for stations with records of suitable length. Logarithmic plots were made of Q_2 vs Q_5, Q_5 vs Q_{10}, Q_5 vs Q_{15}, Q_5 vs Q_{25}, and Q_{10} vs Q_{25}. Of these combinations, Q_5 vs Q_{25} and Q_{10} vs Q_{25} were considered to have the greatest potential usefulness. The departures from the fitted regressions of Q_{25} on Q_5 and Q_{25} on Q_{10} was such that more than 67 per cent of the sample had an error of less than $\pm$ 25 per cent, the criterion of suitable accuracy followed in this study.

In order to make estimates of peak rates of discharge for recurrence intervals greater than 25 years, it was considered necessary to utilize records having longer records than those which were available in the D-13 and D-20 areas. The success in relating Q_{10} to Q_{25} as described above suggested that a sample of longer records from outside the study area could yield usable relations between floods of short and long-term frequencies that would be applicable to the study area.

A sample was selected and relations were sought between short-term and long-term peak rates of runoff, using both the Potter and the Benson analysis techniques.

A sample was selected from the records available outside of the E-5, D-13 and D-20 areas. (Locations of these stations are given in Table 4.)

The following criteria were used in choosing a sample from the 58 available records.

1. Equal numbers of stations were desired from each location.
2. Equal numbers of stations were desired from watersheds less than 500 square miles and from watersheds larger than 500 square miles.
3. Equal numbers of stations were desired from different lithologic areas having the following classifications:
   a. Sandstone and shale.
   b. Glacial drift and loess.
   c. Unclassified.

Using these criteria, a total sample of 22 stations was selected from the 58 records available. Of these 22 stations, 19 were suitable for the Benson method of analysis; three stations being discarded because of extreme irregularities in the plotted curve on the Gumbel paper. The same 3 stations were discarded in utilizing the Potter method because the upper and lower frequency curves were nearly parallel. This gave a discontinuous curve utilizing the Potter method. In addition, two other stations were discarded for utilization by the Potter method because of an excess error in approximating the plotted points with the two straight lines by the "dog-leg" method. For these two stations, the accumulated percentage error in representing the data with the "dog-leg" was greater than $\pm$ 25 per cent for 2/3 of all plotted points having a recurrence interval of 10 years or more.

The next procedure was to compare the errors resulting from each of the two methods of curve fitting. Using the 17 stations that remained, the accumulated error curve was plotted for both methods. The distribution of error curve is shown in Fig. 23. It will be noted that both methods gave a good representation of the plotted points having a recurrence interval greater than 10 years. Approximately 95 per cent of the sample was within $\pm$ 17 per cent error for both methods.

An attempt was made to group the data from the regions outside of D-13 and D-20 problem areas by geographic areas and by geological parent material classifications. Variations in the relation between Q_{10} vs Q_{40} (Benson's method) and Q_{10U} vs Q_{40U} (determined from the upper frequency curve by Potter's method) were considered to be sufficiently small to permit grouping together the data from northwest, southwest, and east of the problem area. Data from these locations were grouped together. A plot of Q_{10} vs Q_{40} (Benson method) is given in Fig. 24. Plots of Q_{10L} vs Q_{40U} and Q_{10U} vs Q_{40U} are given in Fig. 25. (Q_{10L} was determined from Potter's lower frequency curve). Fig. 26 shows the distribution of error curves for both methods. Examination of Fig. 26 shows that a smaller error results from use of the Benson method, which gives 94 per cent of the sample having less than $\pm$ 25 per cent error.

The relations shown in Fig. 24 and Fig. 25 were derived from geographic locations outside of the D-13 and D-20 problem areas. The problem remained to compare this type of relation from outside the D-13 and D-20 areas with that inside the same area. Fig. 27 shows the relation between Q_{10} vs Q_{25} for points inside and outside the D-13 and D-20 problem areas. Since the points from inside the study area appear to be consistent with those northwest, east, and southwest of the study area, the assumption was made that the relation between Q_{10} and Q_{40} as shown in Fig. 24 also applied inside the D-13 and D-20 areas.

A comparison of values of Q_{10} and Q_{10U} for stations inside and outside the D-13 and D-20 problem areas is shown in Fig. 28. Based on this comparison it was concluded that differences between Q_{10} and Q_{10U} as used in this study were not significant.

Comparison of Figs. 24 and 25b show that for a given estimate of Q_{10} (or Q_{10U}), the difference between the resulting estimate of Q_{40} and Q_{40U} is less than 25 per cent for nearly all the range of values shown on Fig. 25.

For these reasons the estimate of Q_{10} and Q_{40}, obtained as described above, are considered to be consistent with estimates of Q_{10U} and Q_{40U}.

From flood frequency studies of watersheds in and near the study area it was determined that the peak rates of runoff from floods having a 40-year recurrence interval (Q_{40}) were approximately twice as big as floods having a 10-year recurrence interval (Q_{10}). The ratios of Q_N/Q_{10} shown on Figs. 3 and 5 were determined by plotting the ratios of Q_N/Q_{10} for N = 10 and 40 on extremal probability paper and correcting the points with a straight line. Intermediate points were determined by interpolation. Values of Q_N/Q_{10} for recurrence intervals of 45 and 50 years were determined by extrapolation of the straight line.

The possible inaccuracy that may result from such an extrapolation technique should be recognized, since the basic data used to derive Figs. 3 and 5 were mostly derived from records less than 40 years in length.

PHYSICAL FACTORS CAUSING BREAKS OR "DOG-LEGS" ON GUMBEL PLOTS OF PEAK RATES OF RUNOFF

The correct determination, from past data, of the peak flows which can be expected with given frequencies is essential in establishing a relationship between the peak flows and the various parameters which caused them. Extreme flood values when plotted on Gumbel extreme value plotting paper produce, in general, a straight line. However, for many streams the plot deviates sharply from the straight line. In the semi-arid areas they are almost always concave upward, with a break or "dog-leg" at about the 10 to 15-year period. The plotted points with recurrence intervals larger than this usually form a second straight line that deviates from the original straight line.

Objective - The objective of this investigation was to study the physical conditions associated with the breaks or "dog-legs" for plots on Gumbel paper of peak rates of runoff for streams in a semi-arid region of the Western Great Plains.

Procedure - The straight line which is generally re-established above the dog-leg, suggests that the peak flows below and above the break come from a different sample population. It has been reasoned that this new population results from physical conditions that exist for just this portion of the data samples. For example, the ground water conditions might be different for the cases when these peak flows occur or there might be difference in rainfall intensity or condition of ground cover. The procedure in this investigation has been to study carefully the physical conditions which might affect the population sample for peak flow cases above and below the break in order to note differences which might cause the two populations. The following specific physical conditions have been studied in relation to the "dog-legs":

1. The daily precipitation frequency distribution.
2. The physical differences which might exist as a function of season.
3. The antecedent stream flow and soil moisture conditions.
4. The characteristics of the unit hydrographs for peak flow cases.
5. Rainfall intensities.

Table 6 summarizes the sources of data used in making these studies.

Individual Investigations

Comparisons of peak rates of flow and daily precipitation frequency - The precipitation frequency plots of daily precipitation for stations corresponding to the respective locations of the watershed that were studied were essentially linear and did not reveal breaks or "dog-legs" as found on the Gumbel plots of peak flow. While the dog-legs in the peak flow frequency plots were not associated with frequency discontinuities in daily precipitation, it cannot be concluded that they might not be associated with other discontinuities in precipitation characteristics, as, for example, in rainfall intensity.

<u>Seasonal Distribution of Major Floods</u> - An investigation of 53 major floods above the dog-legs of the 13 watersheds studied shows that these floods followed immediately after heavy rains and that snowmelt contribution was not an important factor.

<u>Hydrograph Comparison for Peak Flows</u> - Stream flow preceding and following peak flows as determined from the stream hydrographs show no important differences between cases falling above or below the dog-legs. This indicates that base flow or ground water did not make unusual contributions to peak flows in the cases above the dog-legs.

<u>Unit hydrograph comparisons</u> - The unit hydrographs were compared for storms that plotted both above and below the break in the dog-leg of the frequency curve.

For the cases studied, no significant differences could be found in the relations between volume and peak rates of flow.

It was noted, however, that the rate of rise of the unit hydrograph was faster for those cases that plotted on the upper limb of the dog-leg than on the lower. This indicates that at least one of the physical differences in the peak flow population above and below the break in the dog-legs is the greater intensity of water accumulation in a short period for flows that plot in the upper limb of the dog-leg. This could be expected to result primarily from greater precipitation intensity.

Intensity of Rainfall - Comparison of rainfall intensities associated peak flow dog-legs was made by utilizing reports of hourly precipitation. This type of study is handicapped by the sparsity of hourly-recording stations whose records correspond to the years when peak flows occurred. In some cases it has been possible to estimate the hourly contribution of daily precipitation for stations in the study area from nearby hourly reporting stations affected by the same storm. These studies show rather consistently higher rainfall intensities for peak flow cases occurring above the dog-legs.

Fifteen independent, large storms which produced daily precipitation of around 2.00" or more, at individual stations in northeastern Colorado in the 1950's were studied in some detail. Eight of these 15 storms received 80 per cent, or more, of the total storm precipitation during two hours, while in the other 7 storms 80 per cent of the storm total did not accumulate until 15 hours had elapsed. The eight high-intensity storms averaged 2.29" per 24 hours, but had an average hourly intensity of 0.97" for the highest two-hour interval. The eight low-intensity storms had an average of 2.58" per 24 hours, but had an average hourly intensity of only .14" for the highest two-hour interval. These high-intensity and low-intensity storms obviously result from different weather situations which produce different rainfall intensity populations. The peak flows below and above the dog-legs on the flood frequency plots appear, at least in many cases, to be related to these rainfall intensity populations.

TABLE 6. SUMMARY OF DATA USED IN STUDYING PHYSICAL FACTORS CAUSING BREAKS OR "DOG-LEGS" ON GUMBEL PLOTS OF PEAK RATES OF RUNOFF

Watershed Identification (Serial Number) (See Table 4.)	Factors Considered				
	Freq. Distribution of Daily Precipitation	Seasonal Affects	Antecedent Conditions	Characteristics of Unit Hydrograph	Rainfall Intensity
1	x				
10		x	x		x
16		x	x		x
18	x	x	x	x	x
22		x	x		x
23	x	x	x	x	x
25		x	x		
33		x	x		
36	x				
43	x				
44	x				
45	x				
46	x				
48		x	x		x
50		x	x		x
54	x				
57	x				
66			x		x

(continued on next page)

TABLE 6 - Continued

SUMMARY OF DATA USED IN STUDYING PHYSICAL FACTORS CAUSING BREAKS OR "DOG-LEGS" ON GUMBEL PLOTS OF PEAK RATES OF RUNOFF

Watershed Identification (Serial Number) (See Table 4.)	Factors Considered				
	Freq. Distribution of Daily Precipitation	Seasonal Affects	Antecedent Conditions	Characteristics of Unit Hydograph	Rainfall Intensity
76	x	x	x		x
87			x		
88	x				
101		x	x		
102	x	x	x		
103			x		
105	x				
106			x		
108			x		
109			x		
111	x				
112	x				
113	x				
115		x			

Summary - The "dog-legs" on flood frequency plots for the Western Great Plains do not result, to any major degree, from variations in the following conditions:

1. Antecedent stream flow or ground water conditions
2. Annual maximum daily precipitation
3. Snowmelt contributions to the hydrograph

While there are probably other physical causes, discontinuities in rainfall intensity appears to be a major cause for the discontinuities or "dog-legs" found on the flood frequency plots for watersheds in the Western Great Plains.

REGIONAL DISTRIBUTION OF UNIT DISCHARGE (Q_{10}/A)

Regional Distribution of Unit (Q_{10}/sq. mi.) 10-year Peak Flows

Numerous parameters which might affect peak flows from small watersheds in the D-13, D-20, and E-5 areas have been investigated. Slope, elevation and soil type parameters in the D-13, D-20 areas are the only ones with sufficient relationship to peak flows to be statistically significant. These and other parameters, such as land use, precipitation, etc., which must be important in the control of peak flow, all have regional distributions which could be presented on the basis of isolines connecting points of equal values. The proper integration of these various parameters should produce a chart showing the regional distribution of unit peak flows. Only "watershed area" is not amenable to a regional presentation of areas of equal values. Other studies indicate however, that contributing area plays only a minor role in determining peak rates of runoff for small watersheds in the semi-arid area of the Western Great Plains. (See the section.)

Fig. 4 was prepared to show the regional distribution of unit peak flows in cfs per square mile at the construction site. The values of cfs per square mile taken from the isoline of equal values at a construction site can be multiplied by the area of the watershed above this point to estimate the 10-year peak flow which can be expected at that site. This applies only to those portions of streams and rivers having an area of 1200 square miles or less, above the construction site. The values obtained by this method can be utilized as a first approximation of the 10-year peak flow or can be used as a check on the peak flow computed from methods outlined in Chapter III. Estimates for longer period peak flows can be obtained by multiplying the 10-year values obtained from the map of unit flows by the same ratios as shown in Figs. 3 and 7.

Since the isolines of unit flow have been drawn on the basis of computed values for all suitably gaged watersheds in the region, it was not practical to prepare estimates of error for these watersheds from Fig. 4. It is pertinent to point out, however, that the isolines of unit flow on Fig. 4 present patterns that might reasonably be expected from consideration of parameters that should affect peak flows such as; slope, elevation, soil type, and precipitation.

UNIT PEAK FLOW AS A FUNCTION OF WATERSHED SIZE

Unit Flow as Indicated by Peak Flow Design Charts - In the preparation of the design charts for estimating peak flows for ungaged watersheds using parameters of area and slope, it was noted that the best fit relationship indicated that unit flow (Q_{10}/A) appeared to increase as the size of the area increased. This is in contrast to the decrease in unit flow per area that would be expected and which is apparent in established curves for maximum peak flows per unit area as a function of watershed size. This is of particular importance in the D-13 and D-20 study since conclusions as to peak flows which might be expected for areas less than 100 square miles have to be based on only two cases and on an extrapolation from larger areas.

Unit Flow as a Function of Watershed Size - As a verification that the indications of the design curves were real, 10-year peak flows per unit area were catagorized in groups, according to watershed size, for 79 watersheds in the general area of the central and Western Great Plains. The catagories used were 0-199 square miles, 200-399, 400-599, 600-799, 800-999, and 1,000 to 1,500 square miles. Fig. 29 shows the relationship of unit flow as a function of watershed size for 10-year peak flow for these watersheds. This indicates an increase in unit flows as a function of watershed size as watershed size increases to between 400 and 800 square miles, after which unit flows decrease with further size increases. Despite their greater potential for large unit flows as a result of their potentially greater average areal amounts of precipitation and their decreased times of concentration, small watersheds do not, in general, produce the greatest peak flow during 10-year time intervals on the Western Great Plains.

Statistical Relationship Between Unit Flow and Watershed Area - A further investigation of the relationship between 10-year unit flows and watershed area was made by correlating unit flow and watershed area for watersheds in both the D-13, D-20, and the E-5 areas.

The correlation coefficient for the D-13, D-20 area was -0.16 and for the E-5 area was +.28. The logarithmic correlation coefficient for the D-13 and D-20 areas was -0.07. None of these coefficients are statistically significant, showing that no consistent decrease in unit peak flows with area occurs in the sample data.

Probability of Extreme Rainfall on Small Watersheds - Peak flows in this region occur with short periods of intense rainfall. If these heavy rainfall cases are more or less randomly distributed throughout the area, and if peak flows result from heavy rain over only a portion of the watershed, larger watersheds up to some critical limits would have a greater probability of obtaining a large rainfall, and consequently a peak flow event, in relatively short intervals. This appears to be the condition that exists in the semi-arid area of the Western Great Plains. The larger peak flows expected over small watersheds would be realized even in these semi-arid areas after a sufficiently long period of time. The period of time required could be expected to increase with the aridity of the area.

Several investigations have been made to see if the heavy rainfall events are more or less randomly distributed in the area of northeastern Colorado.

Large rainfall events - In one of these investigations all heavy rainfall events were tabulated for 21 stations having records for the period 1919-1958. The heavy rainfall events considered were 24-hour amounts, equivalent to 2.00" or greater. An adjustment was made for the increased precipitation expected with distance east by computing a value corresponding to 2.0" for each station on the basis of the ratio of the 5-year expected value to the median 5-year expected value for the 21 stations. There were 235 cases of rainfall equivalent to 2.00" or more observed at the 21 stations during the 40-year interval. This is an average of 11.29 cases at each of the 21 stations during the 40-year period. The smallest number of cases observed for any station, was 4 and the greatest was 20. The standard deviation of the observed number of cases exceeding 2.0" was 4.55 cases. There were 15 cases (66 per cent) within ± one standard deviation of the mean, and all 21 cases were within ± two standard deviations. (This compares to theoretical values of 68 and 95 per cent, respectively.) This indicates that, when the adjustment is made for the change in precipitation zone, the expectancy of extreme daily rainfalls at different stations, located at least 20 miles apart, is similar to that which would be expected by chance.

Synthesizing precipitation frequency plots - A further study considered the synthesizing of frequency plots from data from several stations. If the chance of extreme rainfall is randomly distributed, it should be possible, after the correction for broadscale regional changes in precipitation, to synthesize the long-period frequency of extreme value precipitation from extreme values observed for shorter periods at a number of stations.

This was undertaken using annual extreme values of 24-hour precipitation from Denver, Greeley, Cheyenne, Ft. Morgan, Akron, and Sterling. An adjustment was made, as described in the previous section, for increased precipitation expectancy with distance east. A synthetic precipitation record was prepared from these stations and compared to the values obtained from the actual 30 years of data. Consecutive 5-year intervals from 1919 to 1947 and 12 randomly selected 5-year combinations were used for these six stations to compute synthetic frequency distribution from which 2, 5, 10, and 25-year precipitation values were determined. The computed mean 25-year value from the 18 combinations was 3.98" at Akron. The standard error was .44". All combinations were within 2 standard errors of the mean. Two thirds of the 18 cases were within one standard error, as would be expected. The 25-year value for Akron computed from 30 years of data at Akron is 3.95" - almost identical with the mean of 18 combinations computed from 6 separate stations, each having 5 years of record. It is well within the range of values that could be expected by chance.

This close correspondence between the 25-year precipitation amounts computed from the synthetic record and the 30-years of actual data at Akron shows that rainfall amounts greater than 2.0" per day are distributed randomly in time and space in this region. The same result was found when other stations were used for similar comparisons.

Simultaneous occurrence of extreme rainfall - An additional investigation was made of the occurrence of extreme rainfalls at various stations. Comparisons were made of the actual simultaneous occurrences of extreme rainfall events and the chance expectancy of such simultaneous occurrences.

If the occurrence of 10-year precipitation amounts are independent, then the probability of obtaining 10-year values at the two stations in the same year is their product, or $(1/10)(1/10) = 1/100$. In this investigation the actual simultaneous occurrences of 10-year precipitation amounts at pairs of stations for all combinations of 13 stations was tested. The observed average probability of occurrence of 10-year precipitation values at two stations in the same year for all the combinations of the 13 stations is 1/86, not far different than the 1/100 to be expected by chance. the departure of the actual observed value, 1/86, from the chance probability, 1/100, is in the direction that would be expected if some degree of dependence did exist between extreme rainfall events, although by only a small amounts.

Conclusions Covering Unit Peak Flow as a Function of Watershed Size - There is considerable evidence to indicate that for the Western Great Plains the 10-year return period unit peak flows increase as watershed size increases to between 400 and 800 square miles after which unit flows decrease as would be expected. Various studies show that large rainfalls are randomly distributed. This being the case, the smaller the area the smaller the probability that an extreme rainfall event will occur in a finite period of time. The smaller values of unit peak flow per unit area for small areas apparently results from the smaller probability of these areas receiving an extreme rainfall in the finite period of time specified.

EFFECT OF DIVERSIONS ON PEAK RATES OF RUNOFF

A study was made to investigate the effect of diversions for irrigation on magnitude and frequency of peak rates of runoff from selected small watersheds in Colorado. The watersheds studied were located on the eastern slope of the Rocky Mountains and are considered representative of the condition under which irrigation water is in short supply and a heavy demand exists for irrigation water.

The data selected for analysis were from selected small watersheds in the D-13 and D-20 problem areas in Colorado. After consultation with Mr. L. R. Burgess, Chief Hydrographer in the State Engineer's office, it was concluded that the best watersheds for study included Fountain Creek above Pueblo, and Cherry Creek above Melvin and above Franktown Bear Creek, the Cache la Poudre River, and the St. Vrain Creek were also investigated.

Records of diversion for irrigation are maintained in the Water Commissioner's Field Books which are on file in the State Engineer's office. These records include the daily diversion rate, the first and last date water was used, total days water was diverted, total volume of water used, irrigated area, and the water district.

Additional details of diversions, such as the names and locations of ditches, the method of operation of the irrigation system, and the location and types of measuring devices, were obtained from individual water commissioners.

The momentary maximum discharge for each water year and the date of its occurrence for each station was listed from Water Supply Papers of the U. S. Geological Survey. The locations of all ditches were examined in order to make sure those ditches were above the gaging stations.

Fig. 30 shows that the time required for an isolated storm to travel from the headwaters to the gaging station, both for Cherry Creek and Fountain Creek, is less than 24 hours. Therefore, the diversion data were computed as of the date of maximum peak flow, without any adjustment for travel time from the diversion point to the gaging station.

The summation of the diversions from the streams by all ditches or canals and the evaluation of these diversions as a percentage of the peak flow are given in Tables 7, 8, and 9.

A plot of the annual maximum peak rates of runoff vs diversions expressed as per cent of peak flow for watersheds smaller than 1000 square miles is given in Fig. 31.

A similar plot for watersheds larger than 1000 square miles is given in Fig. 32.

Table 7 shows that in 18 out of 20 cases the total diversions for irrigation from Fountain Creek near Pueblo were less than three (3) per cent. Only for the two lowest peak flows are the diversions greater than ten (10) per cent of the annual maximum peak flow. Hence, the effect of diversion for irrigation on the peak rates of runoff may be neglected.

Tables 8 and 9 show that in all cases the total diversions are less than three (3) per cent of the annual maximum peak flows.

Since Cherry Creek and Fountain Creek are intermittent streams, the peak rates of runoff were probably caused by large storms which gave higher rates of flow than the rate of water used for irrigation.

A plot of annual maximum peak flow vs diversion for irrigation as per cent of peak flow from large watersheds (greater than 1000 square miles) is shown in Fig. 32. Table 10 shows that the effect of diversion for irrigation on magnitude and frequency of floods from small watersheds is small except for those waterhseds which exceed 1000 square miles for which floods are mostly caused by snowmelt.

Figs. 31 and 32 also show that the smaller peak flow, the more the effect of diversion. If there is any effect of diversion, it probably would have an effect for recurrence intervals of 5 years or less.

From this study, it may be concluded that there is no significant effect of diversion for irrigation on magnitude and frequency of peak rates of runoff from small watersheds in semi-arid and arid areas, in which streams are of the intermittent type, i.e., peak rates of runoff were caused by rainstorms.

Significant amounts of diversion were found in those larger watersheds (size greater than 1000 square miles) for which snowmelt is an important factor in affecting peak rates of runoff.

TABLE 7. SUM OF DIVERSIONS FOR IRRIGATION FROM FOUNTAIN CREEK ABOVE PUEBLO AS A FRACTION OF THE ANNUAL MAXIMUM PEAK RATES OF RUNOFF

Station: Fountain Creek at Pueblo, Colorado

Water Year	Annual Maximum Peak Rates of Runoff in cfs	Diversions for Irrigation	
		Total in cfs	Per cent of Peak Flow
1958	3,750	97.5	2.60
1957	6,180	91.0	1.47
1956	5,250	34.5	0.66
1955	11,500	72.3	0.63
1954	5,800	25.8	0.45
1953	3,730	65.0	1.74
1952	5,170	72.9	1.41
1951	11,600	57.9	0.50
1950	9,600	117.2	1.22
1949	1,590	184.0	11.55
1948	9,290	142.7	1.53
1947	5,880	86.9	1.48
1946	16,500	193.4	1.17
1945	17,800	112.0	0.63
1944	12,900	117.5	0.91
1943	324	168.1	52.00
1942	11,000	128.0	1.16
1935	35,000	99.6	0.28
1925	2,500	84.5	3.34
1924	12,000	47.0	0.39

TABLE 8. SUM OF DIVERSION FOR IRRIGATION FROM CHERRY CREEK ABOVE MELVIN AS A FRACTION OF THE ANNUAL MAXIMUM PEAK RATES OF RUNOFF

Station: Cherry Creek near Melvin, Colorado

Water Year	Annual Maximum Peak Rates of Runoff in cfs.	Diversion for Irrigation	
		Total in cfs	Per cent of Peak flow.
1958	5,000	6.5	0.13
1957	9,950	23.5	0.24
1956	5,310	10.2	0.19
1955	4,510	2.7	0.06
1954	611	2.0	0.33
1953	1,670	2.0	0.12
1952	321	0	0
1951	1,040	0	0
1950	1,450	0	0
1949	1,420	10.4	0.73
1948	3,760	7.7	0.21
1947	1,790	3.5	0.20
1946	17,600	11.5	0.06
1945	10,700	11.8	0.11
1944	1,380	8.4	0.61
1943	3,580	8.0	0.22
1942	2,220	6.5	0.29
1941	2,390	6.0	0.25
1940	4,500	0.5	0.01

TABLE 9. SUM OF DIVERSIONS FOR IRRIGATION FROM CHERRY CREEK ABOVE FRANKTOWN AS A FRACTION OF THE ANNUAL MAXIMUM PEAK RATES OF RUNOFF.

Station: Cherry Creek near Franktown, Colorado

Water Year	Annual Maximum Peak Rates of Runoff in cfs.	Diversion for Irrigation	
		Total in cfs	Per cent of Peak Flow
1958	165	3.7	2.24
1957	5,380	9.9	0.19
1956	3,380	3.7	0.11
1955	990	2.7	0.34
1954	2,620	2.0	0.08
1953	455	2.0	0.44
1952	1,350	0	0
1951	81	0	0
1950	146	0	0
1949	1,080	3.7	0.34
1948	1,220	0	0
1947	928	0.5	0.05
1946	1,470	5.5	0.37
1945	9,170	2.2	0.02
1944	390	4.4	1.14
1943	198	1.7	0.86
1942	3,620	1.0	0.03
1941	4,700	0.5	0
1940	2,000	2.7	0.13

TABLE 10. A COMPARISON OF DRAINAGE AREAS AND DIVERSIONS FOR IRRIGATION

Station	Drainage Area Sq. Mi.	Max. diversions for irrigation % of peak flow	Remarks
Bear Creek at Morrison, Colo.	165	Very little	1. Information from Water Commissioner 2. In E-5 area.
Cherry Creek near Franktown, Colo.[1]	172	< 2.24	
St. Vrain Creek at Lyons, Colo.	226	Very little	1. Information from Water Commissioner 2. In E-5 area.
Bear Creek at Mouth at Sheridan, Colo.[2]	264	< 5	
Cherry Creek near Melvin, Colo.[3]	369	< 0.73	
Fountain Creek at Pueblo, Colo.[4]	926	< 3.34	Only for the two lowest peak flows have diversions been greater than 10 per cent
St. Vrain Creek at mouth near Platteville, Colo.[5]	1000	Some effect on lowest rates of peak flow	Flow partly regulated by many small reservoirs.
Cache la Poudre River near Ft. Collins, Colo.	1048	little	1. Information from Water Commissioner 2. In E-5 area.
Cache la Poudre River near Greeley, Colo.[5]	1840	> 12.6	The peak flows were also affected by trans-basin, trans-mountain diversion, and diversion for municipal use.

[1] 19 year period 1940 - 1958
[2] 8 highest and 2 lowest peak flow year from 1936 - 1957
[3] 19 year period 1940 - 1958
[4] 20 year period 1924 - 25, 1935, 1942 - 1958
[5] 8 highest and 2 lowest peak flow years from 1936 - 1957

EFFECT OF SOIL TYPE

One of the factors that can affect peak rates of runoff is the infiltration rate of the surface soil. If the infiltration rate is high at the beginning of a storm and continues to be high as the storm progresses, less runoff will result than would be the case for soils with low infiltration rates.

A study was made to establish a relation between a soil infiltration index and peak rates of runoff. The soil infiltration index was developed from consideration of soils data from Nebraska (7), Kansas (8), and Colorado (9). The infiltration index follows the soil classification system of the Soil Conservation Service (10) as follows:

Group A - (Lowest runoff potential) Includes deep sands with very little silt and clay, also deep, rapidly permeable loess.

Group B - Mostly sandy soils less deep than A and loess less deep or less aggregated than A, but the group as a whole has above average infiltration after thorough wetting.

Group C - Comprises shallow soils and soils containing considerable clay and colloid, though less than those of group D. The group has below average infiltration after pre-saturation.

Group D - (Highest runoff potential) Include mostly clays of high swelling per cent, but the group also includes some shallow soils with nearly impermeable subhorizons near the surface.

These soil groups were assigned arbitrarily weighted index numbers related to infiltration as follows:

Group A - Index Number 16 (highest infiltration rate)

Group B - Index Number 8

Group C - Index Number 4

Group D - Index Number 1 (lowest infiltration rate)

The procedure followed in determining a "soil index" for a particular soil type is illustrated in the following example:

Soil type : Anselmo - Keith - Bush

From the hydrologic groupings of soil series, Anselmo is in group A which gives an index number of 16; Keith is in group B, which gives an index number of 8; and Bush is in group B, which also gives an index number of 8. The assigned soil index "I" of this group is

$$I = \frac{(16+8+8)}{3} = 10.67$$

This procedure was followed to obtain soil infiltration indices for the study area. The results are shown in Fig. 2.

To determine the weighted soil index, "I", for a particular watershed, the watershed is plotted on the modified soils map shown in Fig. 2. The percentage of the total area of the watershed with each soil index value is then determined. The weighted soil index for the entire watershed is then the summation of the products of the percentages and the weighted index for each group included in the watershed.

The correlation coefficient between the soil indices "I" and unit discharges (Q_{10}/A) for watersheds 1, 3, 4, 10, 11, 12, 13, 16, 18, 19, 20, 22, 25, 31, 33, and 34 (See Table 11) was -0.539, significant at the 95% level.

This result indicates that there probably is a real relationship between the soil index "I" and peak rates of runoff. The soils index "I" was therefore included as a parameter to be used in the design procedures for estimating peak rates of flow, as described in sections III and V of this report.

An attempt was made to relate the soils indices "I" to the Highway Research Board classification system in use in analysis of soil borings by the Colorado Department of Highways. It was not possible to correlate these two methods of description of soil characteristics. Probable reasons for this lack of correlation include the following:

1. Primary emphasis in the analysis of soil borings is to determine the suitability of the soil profile as a construction material.
2. The soil borings may be taken in locations not representative of the surface soil mantle within the area.
3. Greater emphasis is placed on the soil profile below the surface layer than on the soil mantle at the surface.

For these reasons the soil indices as presented in Fig. 2, are considered to be better suited for use in correlations with runoff than the Highway Research Board system used by the Colorado Highway Department. The soil indices of Fig. 2 are based on the upper layer of the soil surface which more nearly represents the relatively heterogeneous material comprising the soil mantle. Since most peak rates of runoff occur in connection with high intensity rains of short duration, the infiltration characteristics of the surface layers of soil exert a greater influence on peak rates of runoff than the infiltration characteristics of the remainder of the soil profile.

EFFECT OF WATERSHED SLOPE

Preliminary studies indicated that a slope parameter $S_{0.9L}$ was a significant factor related to peak rates of runoff.

Work by Benson (11) indicated that channel slope showed considerable promise as a factor for explaining variations in peak rates of runoff from New England Watersheds. Watersheds from inside the study area were examined and dimensional and dimensionless plots were made of the channel profiles.

Each of three slope parameters were used in conjunction with contributing area in a graphical correlation process to derive a relation suitable for use in estimating Q_{10}. Following Benson's work (5), a slope parameter $S_{.9L}$ was defined by

$$S_{.9L} = \frac{E_{0.9L} - E_{CS}}{D}$$

Where $E_{0.9L}$ = elevation in feet 9/10 of the length of the watercourse upstream from the construction site.

E_{CS} = elevation in feet at the construction site.

D = distance in miles along the watercourse between these locations.

The second slope parameter utilized was the "T" factor, suggested in conversations with Mr. W. D. Potter (12) . The "T" factor (indicating a measure of "time of travel,") is defined as follows:

$$T = T_1 + T_2 = \frac{0.3L}{\sqrt{S_1}} + \frac{0.7L}{\sqrt{S_2}}$$

where

$$\sqrt{S_1} = \sqrt{\frac{E_{HW} - E_{0.7}}{0.3L}}$$

and

$$\sqrt{S_2} = \sqrt{\frac{E_{0.7L} - E_{CS}}{0.7L}}$$

Where the sumbols have the following meanings:

E_{HW} = elevation (feet msl) at the headwaters of the watershed

$E_{0.7L}$ = elevation (feet msl) at a point 0.7 of the distance from the construction site to the headwaters, measured along the watercourse.

E_{CS} = elevation (feet msl) at the construction site.

L = distance (in miles) between construction site and headwaters.

A third slope parameter was defined by

$$S_{0.5L} = \frac{E_{0.5L} - E_{CS}}{0.5L}$$

where $E_{0.5L}$ and E_{CS} are the elevations in feet at the point 0.5 the length of the watercourse and at the construction site, respectively, and L has the same meaning as above.

These slope parameters were used in conjunction with contributing area "A" (in square miles) in a graphical correlation process to estimate Q_{10}. It was found that using $A^{0.90}$ vs Q_{10} with the slope parameter $S_{0.9L}$ gave the best relationship.

A study was made of other methods of measuring slope in order to find a slope parameter which was simple to determine and also highly correlated with unit discharge (Q_{10}/A).

After consideration of a number of different methods of determining slope, three were considered to be sufficiently simple to compute to warrant detailed correlation studies with unit discharge. They were

1. Channel Slope, S

2. Overland Slope, S_{LS}

3. Oblique Overland Slope, S_{OO}

Definition sketches for each of these methods of slope measurement are given in Fig. 33.

The methods for determining each of these values of slope are as follows:

Channel Slope, S, (See Fig. 33a)

1. Find the channel slope between the gaging station and the respective points 0.1, 0.2, 0.3...1.0 times the length from the gaging station and the headwaters.

Overland Slope, S_{LS}, (See Fig. 33b)

1. Along the stream, find the 0.25L, 0.50L, and 0.75L points along the stream.
2. Draw contours through these points.
3. Find the points on these contours midway between the main channel and the boundary.
4. Find the slopes between these midpoints and the next point downstream found from step 1.

Oblique Overland Slope, S_{OO}, (See Fig. 29c)

1. Follow steps 1-3 as for determination of overland slope.
2. Measure the oblique overland slope between the midpoints on the contour of 0.25L, 0.50L, 0.75L, and the gaging station.

These three measures of slope were computed for 16 watersheds in the D-13 and D-20 areas. The results are given in Table 11.

TABLE 11 SLOPES OF SIXTEEN WATERSHEDS, BASED ON THREE TECHNIQUES
(VALUES OF SLOPE ARE IN FEET PER MILE)

No.	Watershed Name	Channel Slope, S										Overland Slope, S_{LS}					Oblique Overland Slope, S_{OO}		
		0.1L	0.2L	0.3L	0.4L	0.5L	0.6L	0.7L	0.8L	0.9L	1.0L	0.25L	0.50L	0.75L	1.0L	Total	0.25L	0.50L	0.75L
1	Fountain Creek at Pueblo, Colo.	29.4	27.2	26.0	25.7	26.5	27.5	29.0	31.3	32.7	39.7	35.2	45.4	70.8	102.7	63.5	35.2	34.9	39.7
3	Apishapa River near Fowler, Colo.	15.3	17.8	22.1	22.4	23.9	24.2	26.0	30.2	37.3	92.5	28.4	55.2	44.0	137.0	66.1	28.4	35.5	71.5
4	Timpas Creek near Rocky Ford, Colo.	2.2	19.6	23.2	22.8	22.8	26.5	26.1	28.0	29.2	32.8	28.0	31.0	46.7	49.7	38.9	28.0	28.2	34.2
10	Blue Creek near Lewellen, Nebr.	10.2	10.7	10.9	11.1	13.4	13.6	12.9	12.8	12.3	14.0	28.4	113.2	54.6	28.7	56.3	28.4	26.2	25.2
11	Birdwood Creek near Hershey, Nebr.	6.7	10.0	10.0	10.0	9.4	9.5	9.1	10.0	10.4	13.3	23.5	75.5	16.9	34.2	37.5	23.5	21.7	26.2
12	Cherry Creek near Franktown, Colo.	115.3	86.5	70.5	62.5	56.9	53.8	51.6	51.9	51.3	53.9	143.0	62.4	49.6	63.1	79.5	143.0	80.9	65.2
13	Cherry Creek near Melvin, Colo.	30.7	31.9	31.4	32.5	44.8	44.0	43.8	44.8	45.0	47.2	56.3	81.7	53.3	54.1	61.4	56.3	52.7	48.6
16	Lodgepole Creek at Bushnell, Nebr.	35.1	31.4	25.9	24.7	23.6	24.1	24.8	25.5	30.3	41.8	32.2	28.0	40.6	56.2	39.3	32.2	30.8	32.0
18	North Fork Republican River	23.8	21.8	19.8	18.9	18.3	17.2	18.7	19.9	19.9	22.2	37.9	129.4	67.5	43.6	69.5	37.9	33.1	41.6
19	Buffalo Creek near Haigler, Nebr.	4.8	14.4	16.0	16.8	18.3	21.7	25.4	26.5	27.8	29.8	34.8	112.7	90.0	45.9	70.7	34.8	54.8	39.0
20	Rock Creeks at Parks, Nebr.	10.0	12.5	13.3	12.5	13.0	12.5	12.9	12.5	16.7	20.0	28.0	160.0	35.7	9.6	58.3	28.0	27.1	27.8
22	Frenchman Creek at Culbertson, Nebr.	9.2	9.2	8.0	10.6	11.3	11.9	12.3	12.3	12.5	12.5	20.9	16.8	17.3	14.5	17.4	20.9	14.6	14.9
24	White River at Crawford, Nebr.	17.9	21.4	25.0	28.6	32.8	37.5	37.2	36.6	37.7	43.6	45.1	91.3	43.0	60.8	60.0	45.1	76.7	96.9
25	Niobrara River above Box Butte, Res.	10.9	10.4	10.5	10.6	10.7	10.4	10.7	10.7	11.3	12.5	24.9	85.8	19.7	24.9	38.8	24.9	24.2	16.2
31	Pumpkin Creek near Bridgeport, Nebr.	9.4	13.1	12.5	12.7	13.5	13.5	13.4	13.9	14.4	16.0	25.9	39.9	44.0	37.2	36.7	25.9	26.6	27.4
33	Landsman Creek near Hale, Colo.	20.4	18.4	19.8	18.9	19.0	19.8	20.1	20.4	19.7	17.8	39.8	24.2	40.1	26.4	32.6	39.8	20.8	23.9

The coefficients of correlation between these measures of slope and unit discharge (Q_{10}/A) were computed. The results are shown in Table 12.

TABLE 12. CORRELATION COEFFICIENTS BETWEEN MEASURE OF SLOPE AND UNIT DISCHARGE, Q_{10}/A.

Position	Channel Slope S	Overland Slope S_{LS}	Oblique Overland Slope S_{OO}
0.1 L			
0.2 L	.793***		
.25L		.783**	.783***
0.3 L			
0.4 L	.888***		
0.5 L	.918***	-.130	.653**
0.6 L	.915***		
0.7 L	.903***		
.75L		.883***	.533*
0.8 L	.908***		
0.9 L	.870***		
1.0 L	.597*		

*Significant at .05 level.
**Significant at .01 level.
***Significant at .001 level.

From this study, it was concluded that the channel slope, S, was best correlated to unit discharge, and that the channel slope measured between the construction site and any position between 0.5L and 0.9L gave approximately the same degree of correlation with unit discharge.

WATERSHED CHARACTERISTICS AS A REPRESENTATIVENESS TEST FOR CHECKING DESIGN ESTIMATES OF PEAK RATES OF RUNOFF

In the early part of the study it was determined from multiple correlation techniques that peak rates of runoff having a 10-year recurrence interval (Q_{10}) could be estimated from either of the following:

1. A combination of the parameters of contributing area, in square miles; drainage density, in miles per square mile; and location (longitude minus latitude).
2. A combination of the parameters of contributing area, in square miles and slope, $S_{0.9L}$, in feet per mile. (The slope measured between the construction site and a point 9/10 of the length from the construction site to the headwaters.)

A method was sought to determine whether a particular ungaged watershed under consideration could be considered similar to the gaged watersheds used in deriving the relations used for estimating Q_{10}. The ideal procedure to follow would be to relate various physical characteristics of watersheds to discharge. However, because only limited discharge data were available, it was necessary to relate the physical characteristics to a parameter that was related to discharge which could be obtained from a large number of watersheds in the region studied. As noted above, $S_{0.9L}$ was such a parameter, hence, the procedure followed was to select a large sample of ungaged watersheds and search for typical relationships of physical characteristics that could be related to $S_{0.9L}$, which in turn could reasonably be expected to be related to Q_{10} . This procedure permitted examination and analysis of a much larger sample of data than would have been possible from analysis of gaged watersheds only.

An attempt was made to relate the slope parameter $S_{0.9L}$ to the following independent variables:

A , Contributing area, square miles.

ΣL , Total length of channel including tributaries in the watershed, obtained by measuring the total length of the blue lines in the watershed on the 1:250,000 scale maps# of the area prepared by the U. S. Geological Survey.

L_L , Difference in degrees between the longitude and the latitude at the construction site.

I , A soil infiltration index, ranging from unity for a clay soil to 16 for a sandy soil. (See Fig. 2).

P_{10} , The 24-hour amount of precipitation having a recurrence interval of 10 years. (See Fig. 10).

A zone of environment was also used in the graphical correlation analysis. The zones were:

1. The Upper Republican River Basin
2. The Arkansas River Basin
3. The South Platte River Basin

Fifty-two (52) ungaged watersheds in certain portions of the D-13 and D-20 areas in eastern Colorado were used as the dependent sample to

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For consistent results only this map or the Colorado Highway Department County Maps (Scale 1/2" = 1 mile) should be used. See Fig. 13.

derive the relation for estimating $S_{0.9L}$ from A, ΣL, L_L, and P_{10} as shown in Fig. 11. An independent sample of 18 ungaged watersheds was used to check the accuracy of estimate of $S_{0.9L}$ from Fig. 11.

Fig. 12 shows the cumulative relative frequency of error of estimate for Fig. 11. It will be noted that approximately 67 per cent of the cases gave errors less than about 22 per cent for the dependent sample, and 67 per cent of the cases in the independent sample gave errors less than 18 per cent.

These results indicate that the parameter $S_{0.9L}$ can be estimated with acceptable accuracy from the given watershed characteristics. Furthermore, success in estimating $S_{0.9L}$ from the watershed characteristics suggests that the relation shown in Fig. 11 can serve as a test for determining whether or not the runoff characteristics of a particular watershed under investigation is similar to the watersheds in the region from which the design chart (Fig. 3) was derived. This assumption is supported by the fact that the factors used to estimate $S_{0.9L}$ from Fig. 11 (Area, drainage density, location, soil infiltration characteristics, and precipitation) are all factors which reasonably could be expected to influence peak rates of runoff. Hence if a given ungaged watershed under consideration is found to be similar (on the basis of the aforementioned characteristics) to the gaged watersheds for which Fig. 3 was developed, it is reasonable to expect that runoff characteristics would also be similar. Unfortunately it was not possible to test this assumption with an adequate sample of data from gaged watersheds.

It should be noted, however, that for ten (10) out of twelve (12) watersheds in eastern Colorado, the departure of the measured value of $S_{0.9L}$ from the value of $S_{0.9L}$ estimated from Fig. 11 did not exceed 25 per cent of the estimated value. For two watersheds the error of estimate of $S_{0.9L}$ from Fig. 11 exceeded 25 per cent.

This leads to the criterion for determining whether or not a particular watershed is representative of those from which **Fig.** 3 was developed: IF THE ESTIMATED $S_{0.9L}$ FROM FIG. 11 DOES NOT DEPART FROM THE MEASURED VALUE OF $S_{0.9L}$ BY MORE THAN 25 PER CENT OF THE ESTIMATED VALUE, THE WATERSHED MAY BE REGARDED AS REPRESENTATIVE. Greater confidence can be placed in the results of use of Fig. 3 when a watershed is determined to be representative.

The procedure for determining the representativeness of a watershed is as follows:

1. Determine if the watershed falls in the area of application - the procedure is applicable in the D-13 and D-20 areas only as shown in Fig. 1.
2. Determine from topographic maps (scale 1:250,000) prepared by the U. S. Geological Survey, the following:

   A , Contributing area, square miles.

   ΣL , Total length of channel, including tributaries in the watershed, miles, represented by the blue lines on the U.S.G.S. maps of scale 1:250,000.

   L_L , Location, longitude minus latitude of the construction site.

NOTE: Total channel length can also be obtained from the county highway maps, (Scale 1/2" = 1 mile) prepared by the Colorado Department of Highways. The relation between the total channel length as determined from these maps and the 1:250,000 U.S.G.S. maps is shown in Fig. 13.

3. Determine the soil infiltration index, I , from Fig. 2.

4. Determine the precipitation parameter, P_{10} , from Fig. 10.

5. With these parameters, enter Fig. 11 and obtain an estimate of $S_{0.9L}$. If the watershed is in the South Platte Basin and the estimate of $S_{0.9L}$ exceeds 22 feet per mile, determine an adjusted value of the estimate, $S_{0.9L}{}^{*}$, by the relation#,

$$S_{0.9L}{}^{*} = 2.3 \; S_{0.9L} - 28.8$$

6. Measure the actual $S_{0.9L}$ from topographic maps or from a site survey.

7. Compute the per cent of error.

$$\text{Per cent error} = \frac{S_{0.9L} \text{ est.} - S_{0.9L} \text{ actual}}{S_{0.9L} \text{ est.}} \times 100$$

8. Accept the watershed as representative if the per cent of error of estimate does not exceed 25 per cent.

The following examples illustrate this procedure.

See insert in Fig. 11.

EXAMPLE OF CHECKING FOR REPRESENTATIVENESS

From the example given in Chapter III (D-13 and D-20 Areas), we have given the following:

$$A = 144 \text{ square miles.}$$

$$E_{0,9L} = 4,980 \text{ ft.}$$

$$E_{CS} = 4,345 \text{ ft.}$$

$$L = 38 \text{ miles.}$$

$$I = 5.3$$

$$S_{0.9L} = 18.6 \text{ ft/mile.}$$

By means of topographical map (U.S. Geological Survey, scale 1:250,000) the following additional information is obtained:

$$\Sigma L = 104 \text{ miles}$$

$$L_L = 102^\circ 38' - 39^\circ 18' = 63^\circ 20'$$

From Fig. 10, obtain

$$P_{10} = 3.13 \text{ inches.}$$

For $A = 144$ square miles, $\Sigma L = 104$ miles, $I = 5.3$,

$L_L = 63^\circ 20'$, and $P_{10} = 3.13$ inches

one obtains from Fig. 11 the estimate $S_{0.9L} = 21$ ft/mile.

The per cent of error between the measured $S_{0.9L}$ and the estimated $S_{0.9L}$ obtained from Fig. 11 is

$$\text{Per cent of error} = \frac{21 - 18.6}{21} \times 100 = \frac{240}{21} = 11.4 \text{ per cent.}$$

Since 11.4 per cent $<$ 25 per cent, the watershed is representative.

Since this watershed is accepted as being representative, one can place more confidence in the estimate of Q_{10} from Fig. 3 than would have been the case had the watershed not been representative. Conversely, if the per cent of error between the measured $S_{0.9L}$ and the estimate of $S_{0.9L}$ from Fig. 11 would have exceeded 25 per cent, one should be cautious in accepting the estimate of Q_{10} from Fig. 3.

OTHER FACTORS

In addition to the factors discussed previously, the following watershed and meteorological parameters were determined for each of the watersheds in the D-13 and D-20 problem areas.

1. The contributing area of the watershed, as listed in the U. S. Geological Survey Water Supply Papers.
2. A location factor, defined as the difference in degrees between the mean longitude and the mean latitude at the centroid of the watershed as determined by eye.
3. A drainage density factor, defined as the total length of channels in miles as indicated by the blue lines on 1:250,000 scale maps of the area prepared by the U. S. Geological Survey, divided by the contributing area in square miles as defined in item 1.
4. The mean elevation of the watershed, an average of the highest and lowest elevations.
5. The mean longitude in degrees at the centroid of the watershed as determined by eye.
6. The mean latitude in degrees at the centroid of the watershed as determined by eye.
7. The ratio of width of the watershed divided by its length. The length of the watershed was the distance from the gaging station to the furthest point. The width was defined as the contributing area divided by this length.

8. A compactness ratio, defined as the circumference of the circle having the same area as the watershed, divided by the total perimeter of the watershed.
9. The over-all slope of the watershed in feet per mile, determined by dividing the elevation difference between gaging station and headwater (in feet) by the distance (in miles) between these two points.
10. The slope in feet per mile for the upper and lower halves of the watershed, determined as for item 9.
11. A precipitation parameter which was the 2-year, 1-day point rainfall in inches, at the station nearest the centroid of the watershed.
12. A precipitation parameter which was the 5-year, 1-day point rainfall in inches, at the station nearest the centroid of the watershed.
13. A precipitation parameter which was a 5-year, 1-day point rainfall in inches, expressed as an average of stations in and near the watershed.
14. A precipitation parameter which was a 5-year, 1-day point rainfall in inches, expressed as area rainfall with an appropriate reduction from point-rainfall.

Attempts were made to use these factors in preparation of design charts for estimating peak rates of runoff. Of these factors, only items 1, 2, 3, 9, and 10 were considered suitable for further analysis.

Details of development of design charts from these and other significant factors are given in the following section.

A summary of the basic data is given in Table 13.

V. DEVELOPMENT OF DESIGN CHARTS FROM SIGNIFICANT FACTORS AND ESTIMATE OF ERROR CURVES

The procedures used for estimating peak flows utilize relationships established between peak flows and certain physical parameters on gaged watersheds for which past records are available. It is then assumed that these relationships hold for ungaged watersheds having similar characteristics and that relationships which existed in the past will also hold in the future.

Graphical correlation techniques were utilized to establish the relationship between 10-year peak flows and parameters by which they are influenced.

The parameters most strongly affecting peak flows and utilized in the graphical correlation for D-13, D-20 are area, slope, as measured between the gaging station and the 0.9 channel length, and an infiltration index. Parameters used for the E-5 area are area, elevation, and latitudinal location.

Attempts were made to include a precipitation parameter in the graphical correlation procedure for estimating Q_{10}. These attempts failed. The reason for this failure probably is due to the relative homogeniety of extreme precipitation events throughout the region studied. (See Figs. 10, and 18-21.)

Distribution of error curves for estimates obtained from these graphical correlations are shown in Fig. 6 for the D-13, D-20 areas, and in Fig. 9 for the E-5 area. The curves show the per cent of time that errors of certain amounts have occurred in the sample tested. These curves were prepared by accumulating the errors to be expected as the number of cases with increasing amounts of error are added.

It can be noted that the per cent of error to be expected is not excessive in about 90 per cent of the cases for the D-13, D-20 area and in about 80 per cent of the cases for the E-5 area.

It must be borne in mind that the excessive errors in about 10 per cent of the cases for D-13, D-20; and in about 20 per cent of the cases for E-5 could, and in some cases probably are, the result of non-representative samples in the test data and not necessarily real errors of such magnitude from the design graphs. The cases giving large errors in the D-13, D-20 areas, for example, are both for cases in which the watershed area was greater than the 1000 square miles for which the design chart is recommended. For the largest error, the value of Q_{10} from frequency analysis is in doubt since the total length of record was only eight years.

VI. DISCUSSION

RESEARCH NEEDS

Needs for additional records from small watersheds - There is a shortage of suitable records of runoff from watersheds having a contributing area less than 100 square miles. This is particularly true in the region east of the Rocky Mountain Foothills.

One of the most valuable contributions to knowledge in the field of small watershed hydrology would be the establishment of additional gaging stations to obtain records of runoff from watersheds having contributing areas less than 100 square miles.

Methods for combining records by the station-year technique to synthesize long-term records of runoff - In the plains area east of the Rocky Mountain Foothills most annual peak flows are caused by high intensity rains which cover only a limited area. This fact indicates that the rains which cause most of the annual peak flows are independent events, and that the center of any storm has an equal probability of passing over any one watershed. If it were possible to identify watersheds having similar runoff characteristics and to obtain records of peak rates of flow from a number of such similar watersheds, it would be possible to synthesize a long-term record by the station-year technique. This procedure would have the distinct advantage of reducing the lengths of record required. The procedure described in this report for determining the representativeness of a watershed is one method by which the degree of similarity of various watersheds can be compared.

Further study of this problem, plus the establishment of suitable gaging stations on watersheds determined to have similar characteristics, would be valuable.

Precipitation characteristics - Studies of precipitation in eastern Colorado indicate that the annual maximum precipitation events exhibit independence in time if the spacing of the measuring points is at least 20 miles. Further study of precipitation characteristics is desirable in order to identify the limiting values of space, time, and geographic location wherein this independence of precipitation events is valid. (For example, preliminary studies have shown that in eastern Colorado the annual maximum 24-hour rainfalls are essentially independent events if the spacing of the stations is at least 20 miles.) Questions on which study would be profitable include: What is the closest spacing for which this is true? Does this apply to other geographic locations, and to other rainfall durations? Does it apply to precipitation amounts other than annual maximum values?

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TABLE 13 SUMMARY OF BASIC DATA - D-13 and D-20 AREAS

Ser. No.	Name of Watershed	Location		Contrib. Area	$S_{0.9L}$	Infiltration Index	Q_{10}		
		Longitude	Latitude				Est. from Fig. 3	From Freq. Anal.	Error
		(Degree°	Min.')	Sq. mi.	Ft/mi.	I	cfs	cfs	%
1	Fountain Creek at Pueblo, Colo.	104°-35'	38°-16'	926	35.2	5.7	18,200	16,300	+ 10.4
3	Apishapa River near Fowler, Colo.	103 -59	38 -05	1125	35.5	7.2	12,300	15,000	- 22.0
4	Timpas Creek near Rocky Ford, Colo.	103 -43	37 -57	451	24.3	6.3	7,800	9,500	- 21.8
10	Blue Creek near Lewellen, Nebr.	102 -10	41 -20	267	13.7	12.8	850	600	+ 29.4
11	Birdwood Creek near Hershey, Nebr.	101 -04	41 -13	286	10.7	8.4	980	1,100	- 12.2
12	Cherry Creek near Franktown, Colo.	104 -45	39 -21	172	53.3	6.0	6,800	6,400	+ 5.9
13	Cherry Creek near Melvin, Colo.	104 -49	39 -36	369	42.3	6.0	11,600	11,600	0.0
16	Lodgepole Creek at Bushnell, Nebr.	103 -51	41 -14	1090	27.3	7.8	7,800	5,900	+ 24.4
18	N. Fork Republican River at Colo. - Nebr. State Line	102 -03	40 -04	130	18.5	11.3	1,150	1,400	- 20.8
19	Bufallo Creek near Haigler, Nebr.	101 -52	40 -02	21	27.9	16.0	115	112	+ 2.6

TABLE 13 SUMMARY OF BASIC DATA - D-13 and D-20 AREAS (cont'd)

Ser. No.	Name of Watershed	Location: Longitude (Degree°)	Location: Latitude (Min.')	Contrib. Area Sq. Mi.	$S_{0.9L}$ Ft/mi.	Infiltration Index I	Q10: Est. from Fig. 3 cfs	Q10: From Freq. Anal. cfs	Error %
20	Rock Creek near Parks, Nebr.	101°-43'	40°-02'	14	19.0	16.0	66	68	- 3.0
22	Frenchman Creek below Champion, Nebr.	101 -43	40 -28	570	13.1	11.5	1,350	1,660	- 23.0
25	Niobrara River above Box Butte Reservoir, Nebr.	103 -10	42 -27	980	10.6	6.3	1,450	1,100	+ 24.2
31	Pumpkin Creek near Bridgeport, Nebr.	103 -02	41 -38	1080	13.8	10.4	1,820	740	+ 59.4
33	Landsman Creek near Hale, Colo.	102 -14	39 -34	450	17.7	5.6	5,200	5,050	+ 2.9
34	S. Fork Republican River near Idalia, Colo.	102 -14	39 -37	1300	19.3	7.1	6,800	17,000	-150.00

TABLE 13. SUMMARY OF BASIC DATA - E - 5 AREA

Ser. No.	Name of Watershed	Location Longitude (Degree°)	Latitude (Min.')	Contributing Area (Square Mile)	Elevation $E_{0.5L}$ (Ft/Mi.)	Q_{10} Est. From Fig.7 (cfs)	Q_{10} From Freq. Anal. (cfs)	Error (%)
200	Illinois Creek near Rand, Colo.	106° 11'	40° 27'	71	8,950	620	690	-11.3
205	Deer Creek at Glenrock, Wyo.	105° 52'	42° 52'	216	6,500	2,000	1,900	5.0
206	La Prele Creek near Douglas, Wyo.	105° 36'	42° 40'	146	6,400	1,100	980	10.9
213	N. Fork South Platte below Geneva Creek at Grant, Colo.	105° 39'	39° 27'	127	9,400	670	825	-23.1
214	N. Fork South Platte River at South Platte, Colo.	105° 11'	39° 25'	484	7,900	2,750	1,580	42.6
215	Bear Creek at Morrison, Colo.	105°12'	39° 39'	165	7,200	3,500	3,400	2.9
216	Turkey Creek near Morrison, Colo.	105° 10'	39° 38'	49.4	7,300	960	880	8.3
217	Cherry Creek near Franktown, Colo.	104° 46'	39° 22'	172	6,900	5,200	6,200	-19.2
218	St. Vrain Creek at Lyons, Colo.	105° 16'	40° 13'	226	7,800	2,400	2,700	-12.5
221	Middle Crow Creek near Hecla, Wyo.	105° 15'	41° 11'	23	7,950	131	163	-24.4
222	South Crow Creek near Hecla, Wyo.	105° 12'	41° 08'	16	7,550	121	66	45.5

TABLE 13. SUMMARY OF BASIC DATA - E - 5 AREA (Cont'd)

Ser. No.	Name of Watershed	Location		Contributing Area	Elevation $E_{0.5L}$	Q_{10}		
		Longitude	Latitude			Est. From Fig. 7	From Freq. Anal.	Error
		(Degree° Min.')		Square Mile	Ft/Mi.	cfs	cfs	%
224	Hverfano River at Manzanares Crossing near Redwing, Colo.	105° 21'	37° 44'	73	9,300	480	1,240	-158.5
225	Cucharas River at Boyd Ranch near Le Veta, Colo.	105° 03'	37° 25'	56	8,800	490	442	9.8
226	Apishapa River near Aguilar, Colo.	104° 40'	37° 23'	126	7,300	2,600	4,750	82.7
227	Purgatoire River at Trinidad, Colo.	104° 31'	37° 10'	795	6,900	23,400	23,400	0
228	Vermejo River near Dawson, N. Mex.	104° 47'	36° 41'	301	7,200	6,200	5,540	10.6
229	Six Mile Creek near Eagle Nest N. Mex.	105° 16'	36° 31'	11	8,700	127	112	11.8
230	Ponil Creek near Cimarron, N. Mex.	104° 57'	36° 35'	171	7,800	1,950	1,735	11.0
231	Mora Creek near Golondrinas, N. Mexico	105° 10'	35° 54'	273	7,500	3,800	3,550	6.6

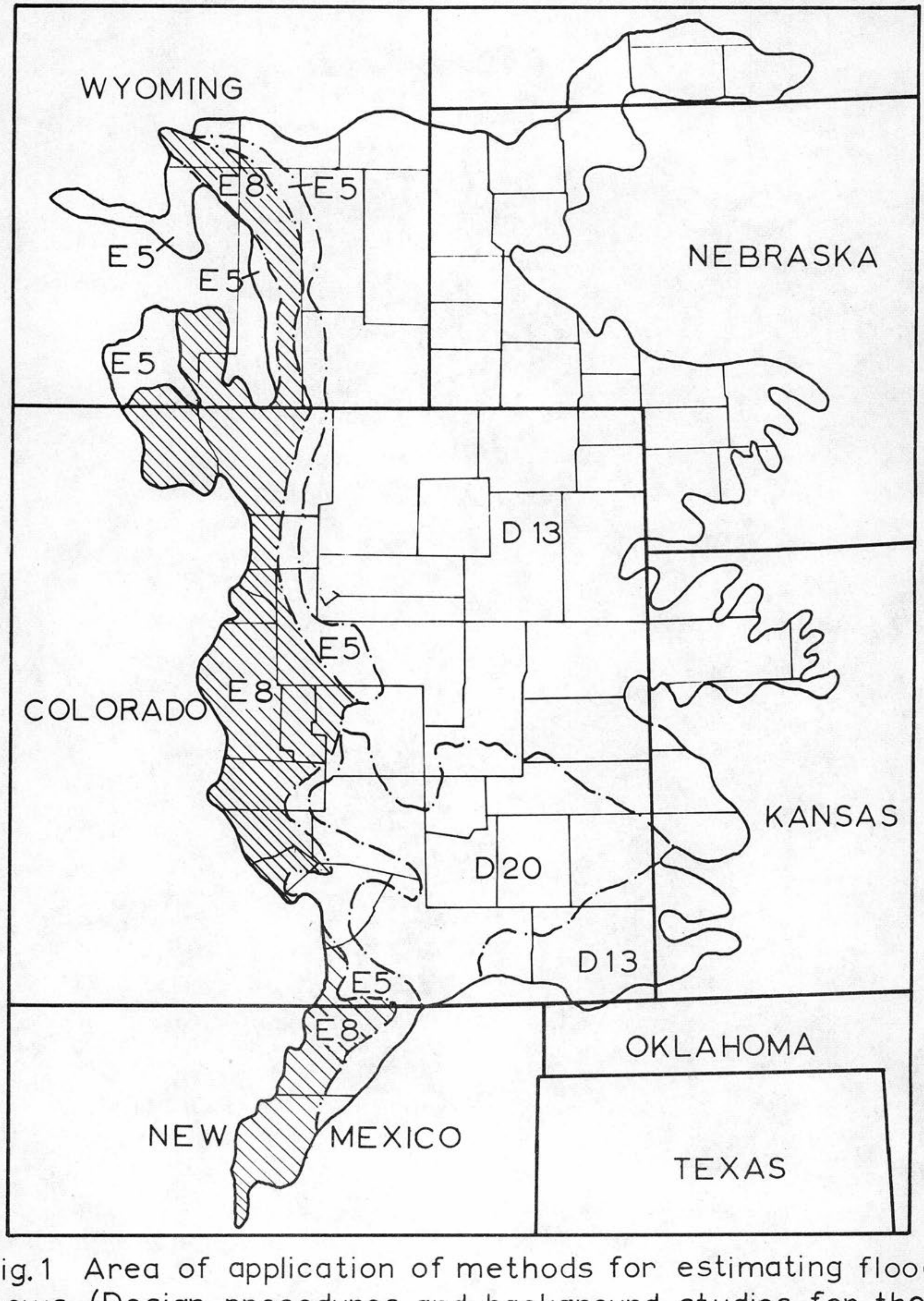


Fig. 1 Area of application of methods for estimating flood flows. (Design procedures and background studies for the combination of the E-5 areas plus the shaded part of the E-8 area are identified throughout the report as the "E-5" Area.)

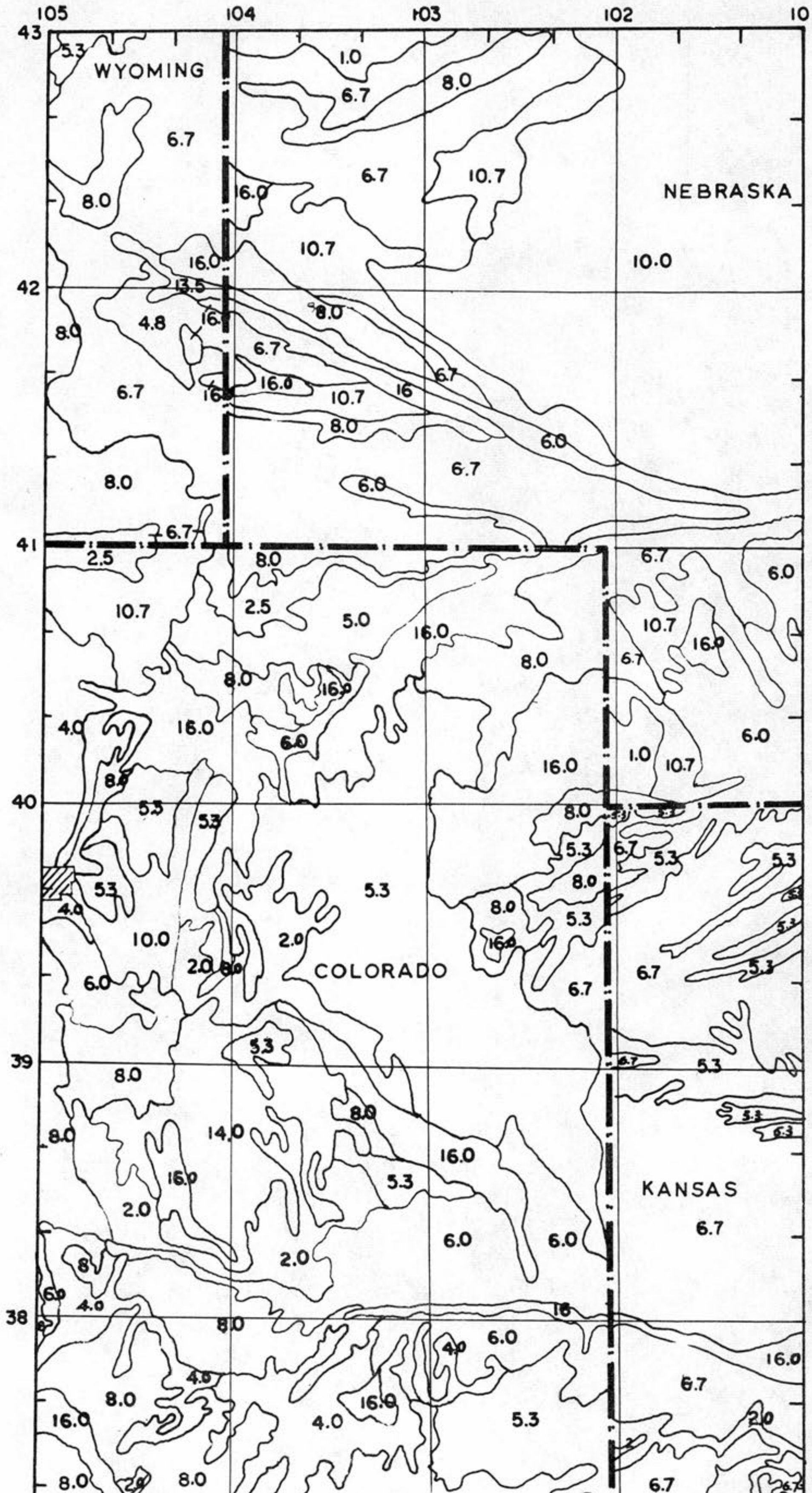


Fig. 2. Soil infiltration indicies for study area based on an arbitrary numbering scale of 16 for high infiltration rates (sandy soils) and unity for low infiltration rates (clay soils).

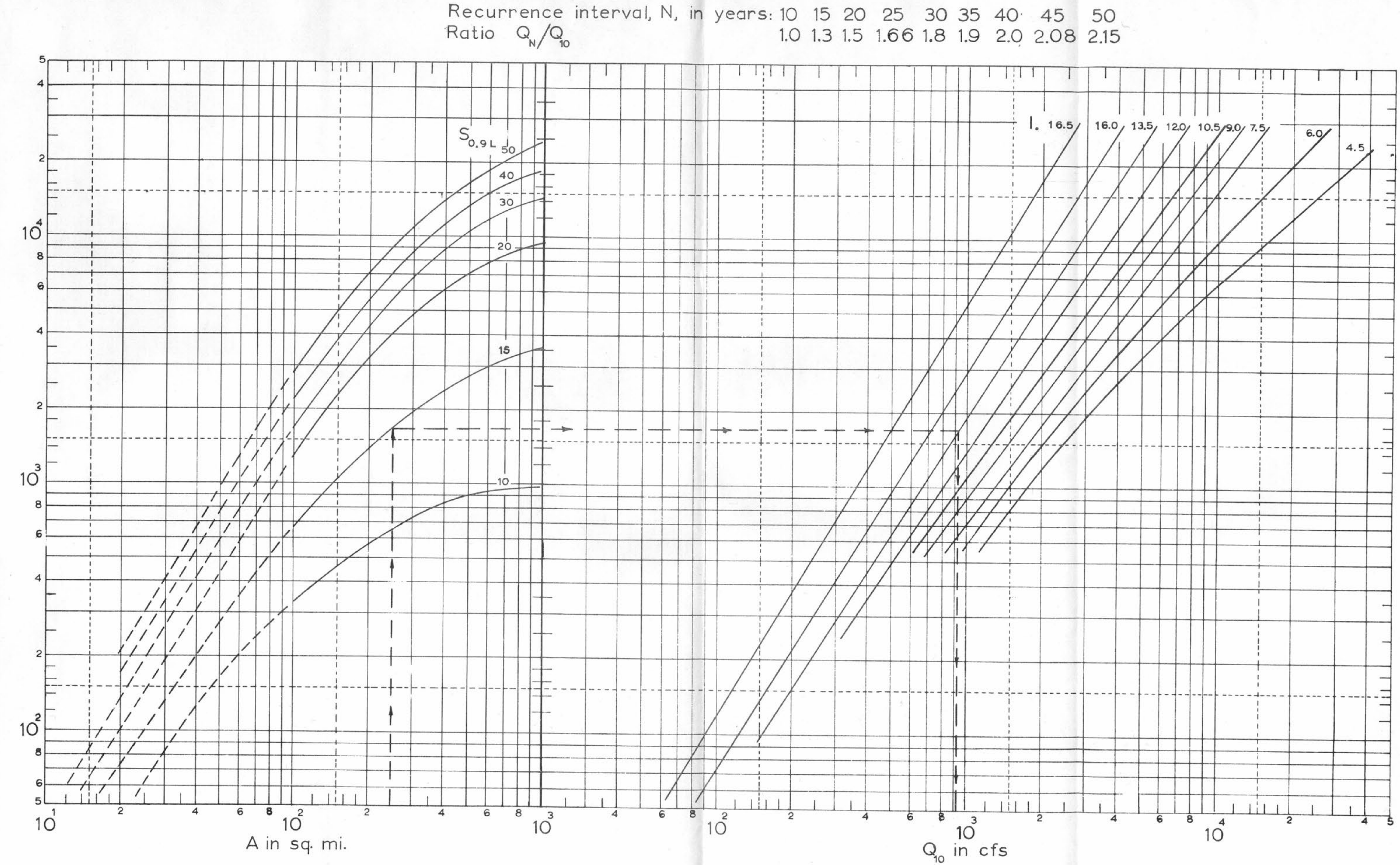


Fig 3 Peak rates of runoff determined from watershed contributing area, a slope parameter and an infiltration index (D-13 and D-20 areas).

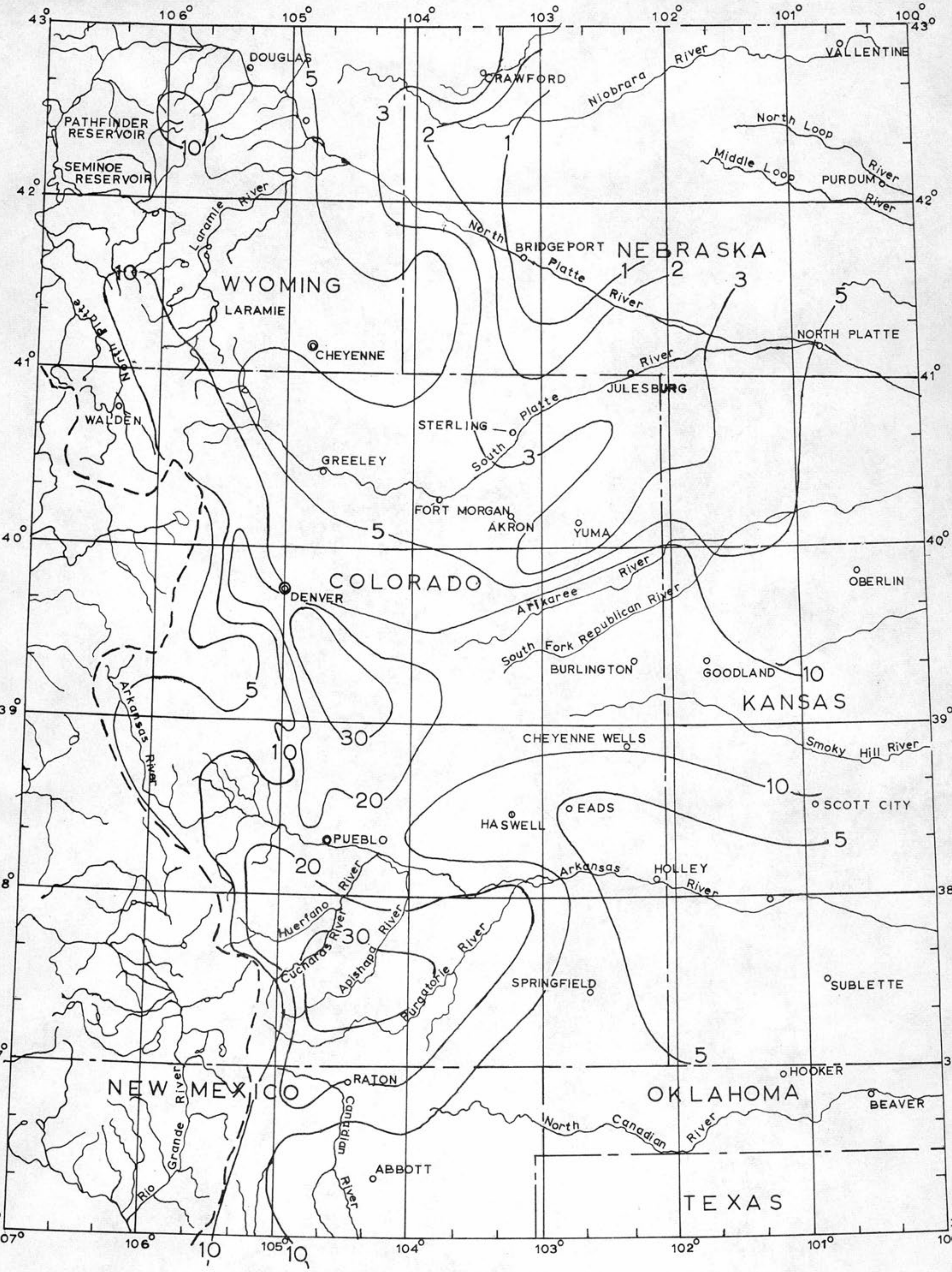


Fig. 4. Regional distribution of unit discharge values (Q_{10}/A) within the study area. Values shown are cfs per square mile.

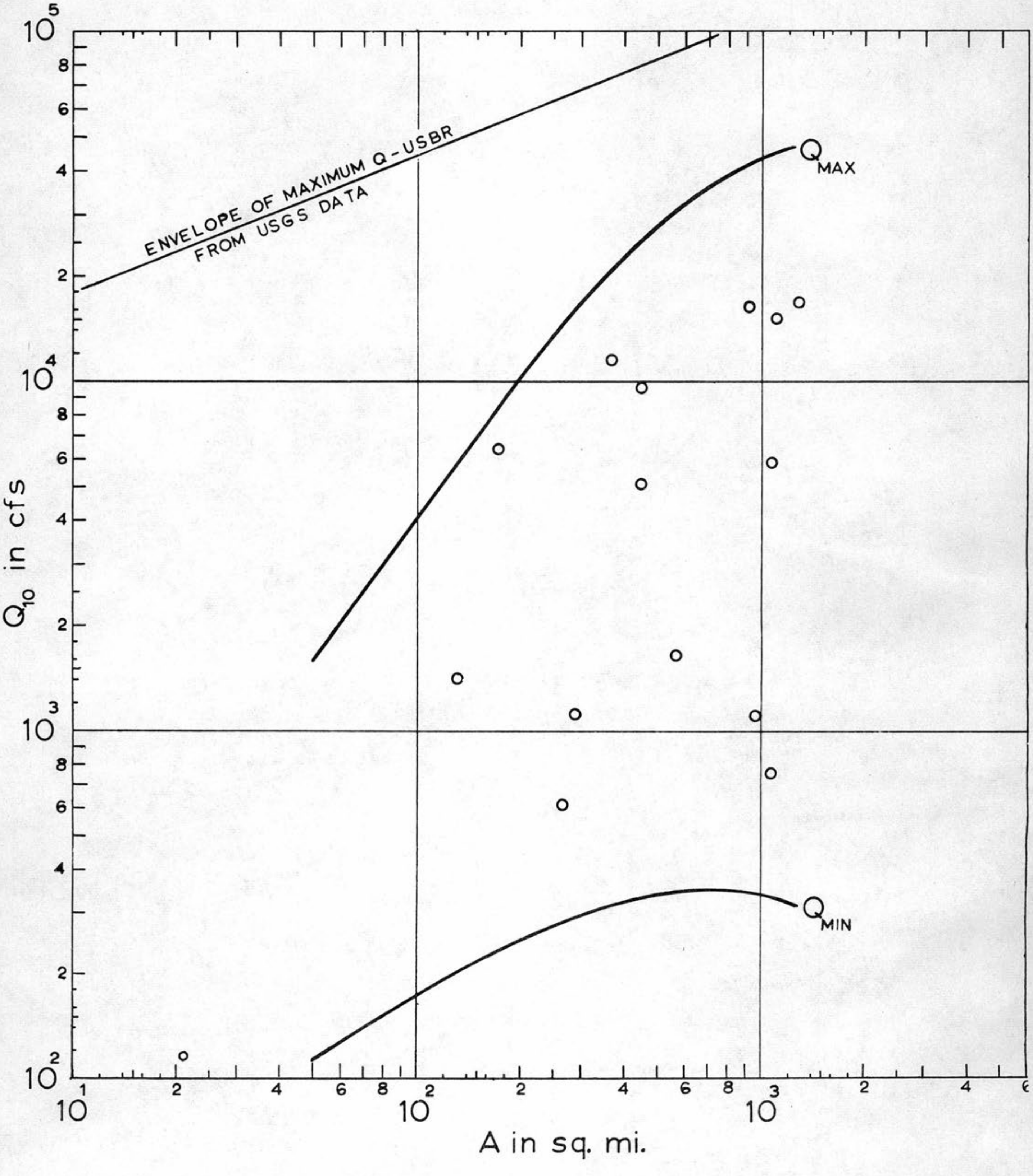


Fig. 5. Recommended maximum and minimum values of Q_{10} of watershed contributing area (D-13 and D-20).

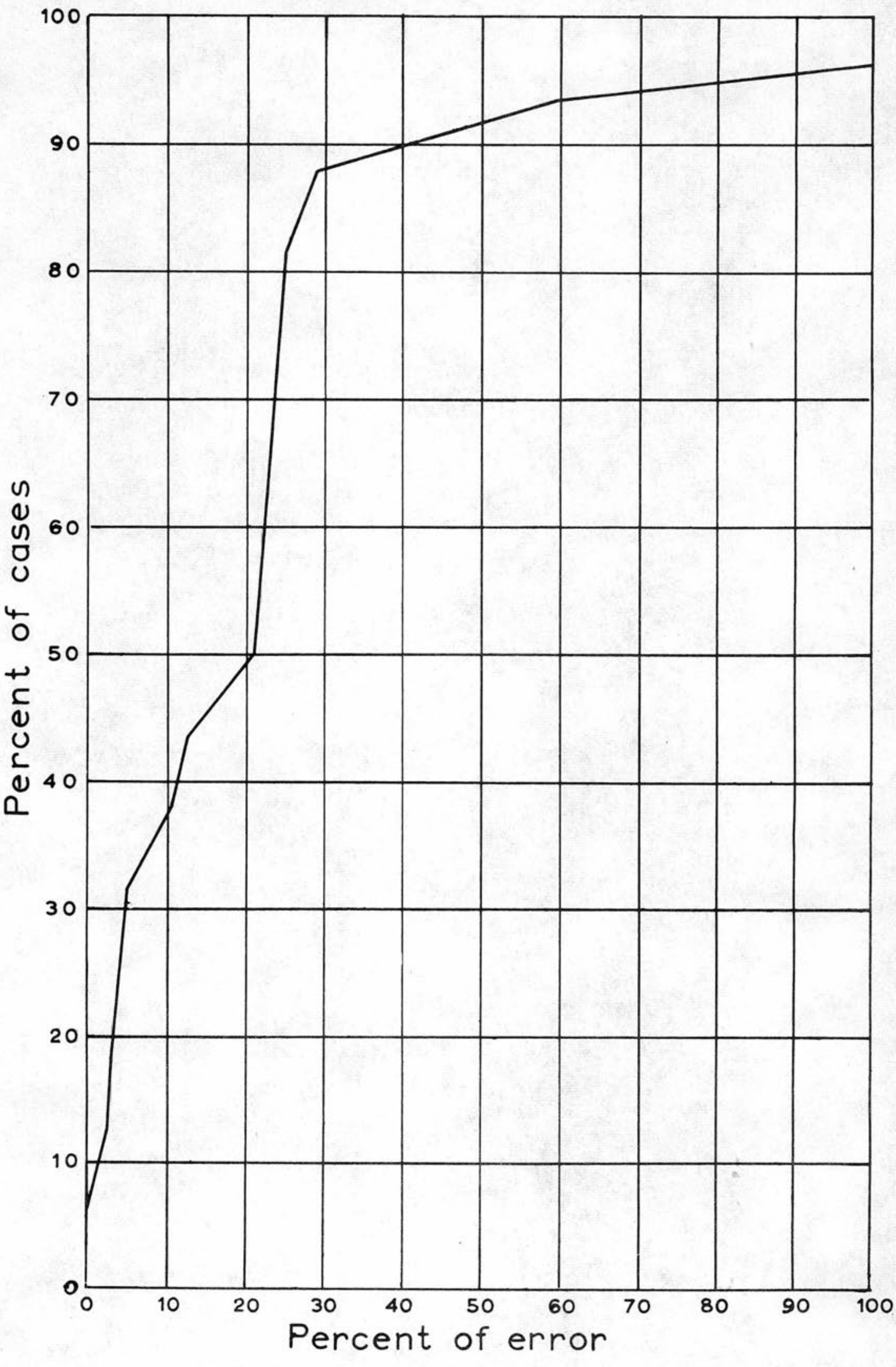


Fig. 6. Cumulative relative frequency of errors of estimate of Q_{10} from Fig. 3.

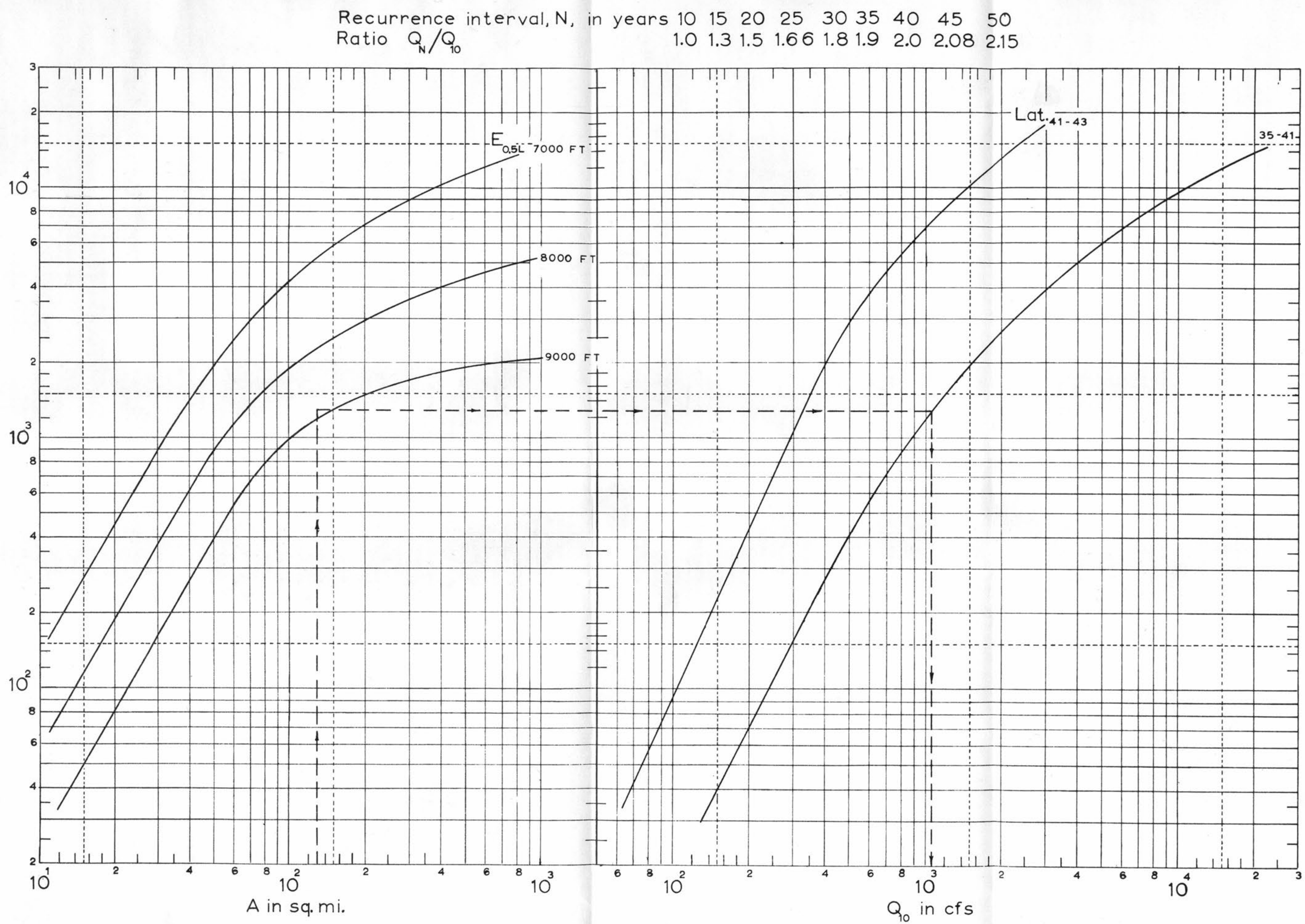


Fig. 7 Peak rates of runoff determined from watershed contributing area, elevation and location (E-5 area).

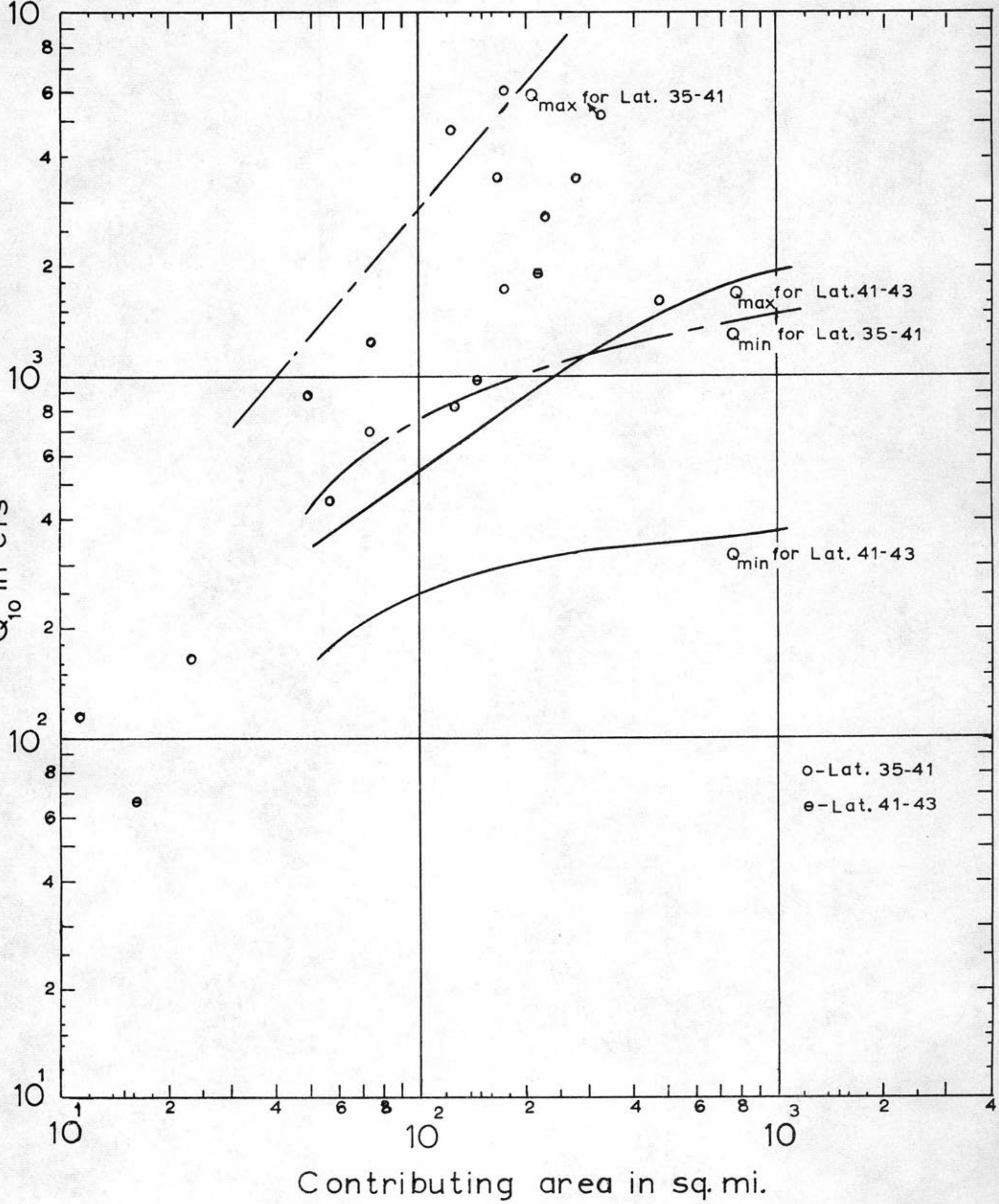


Fig. 8. Recommended maximum and minimum values of Q_{10} as a function of watershed size (E-5 area).

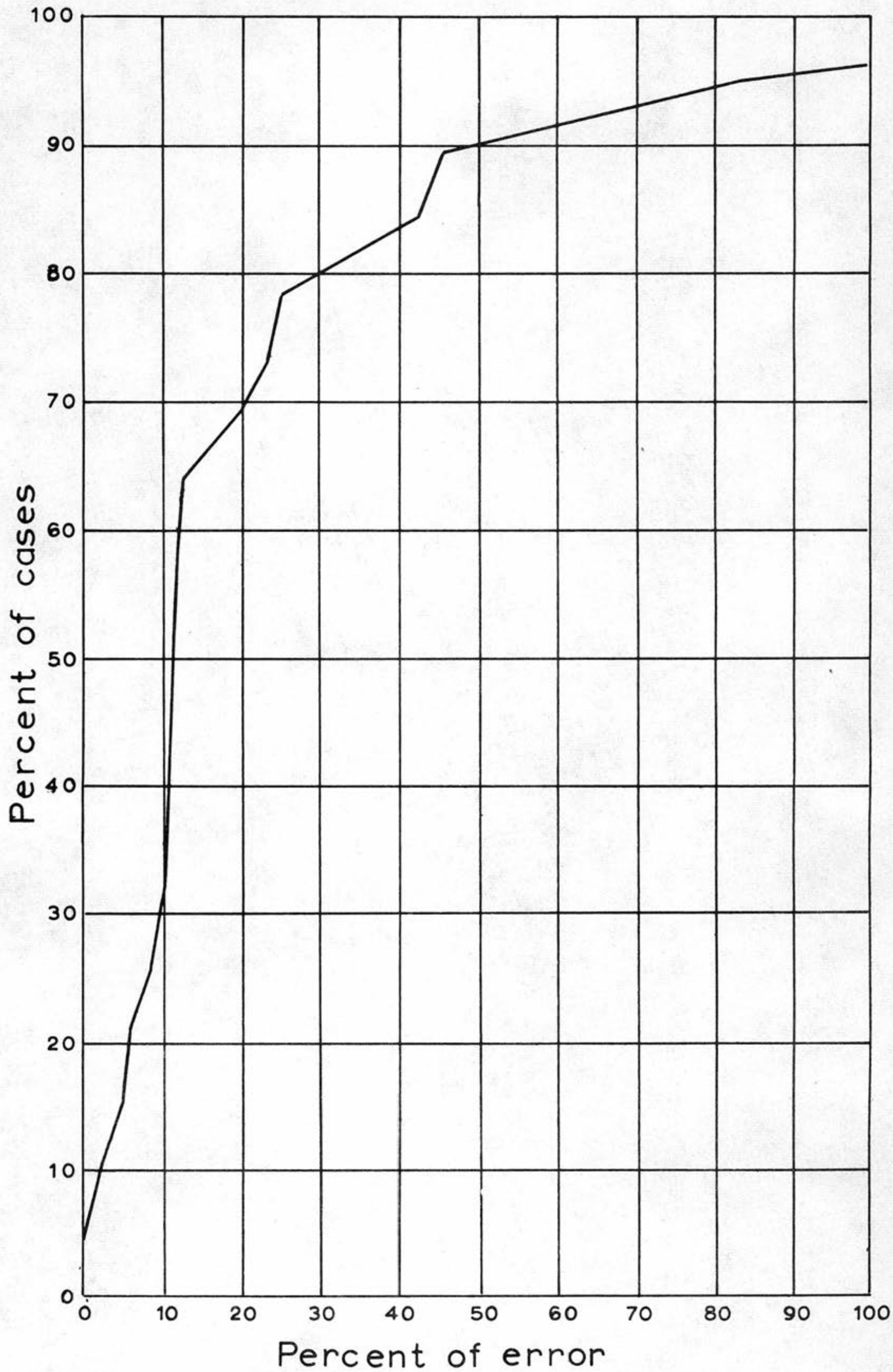


Fig. 9. Cumulative relative frequency of error of estimate of Q_{10} from Fig. 7.

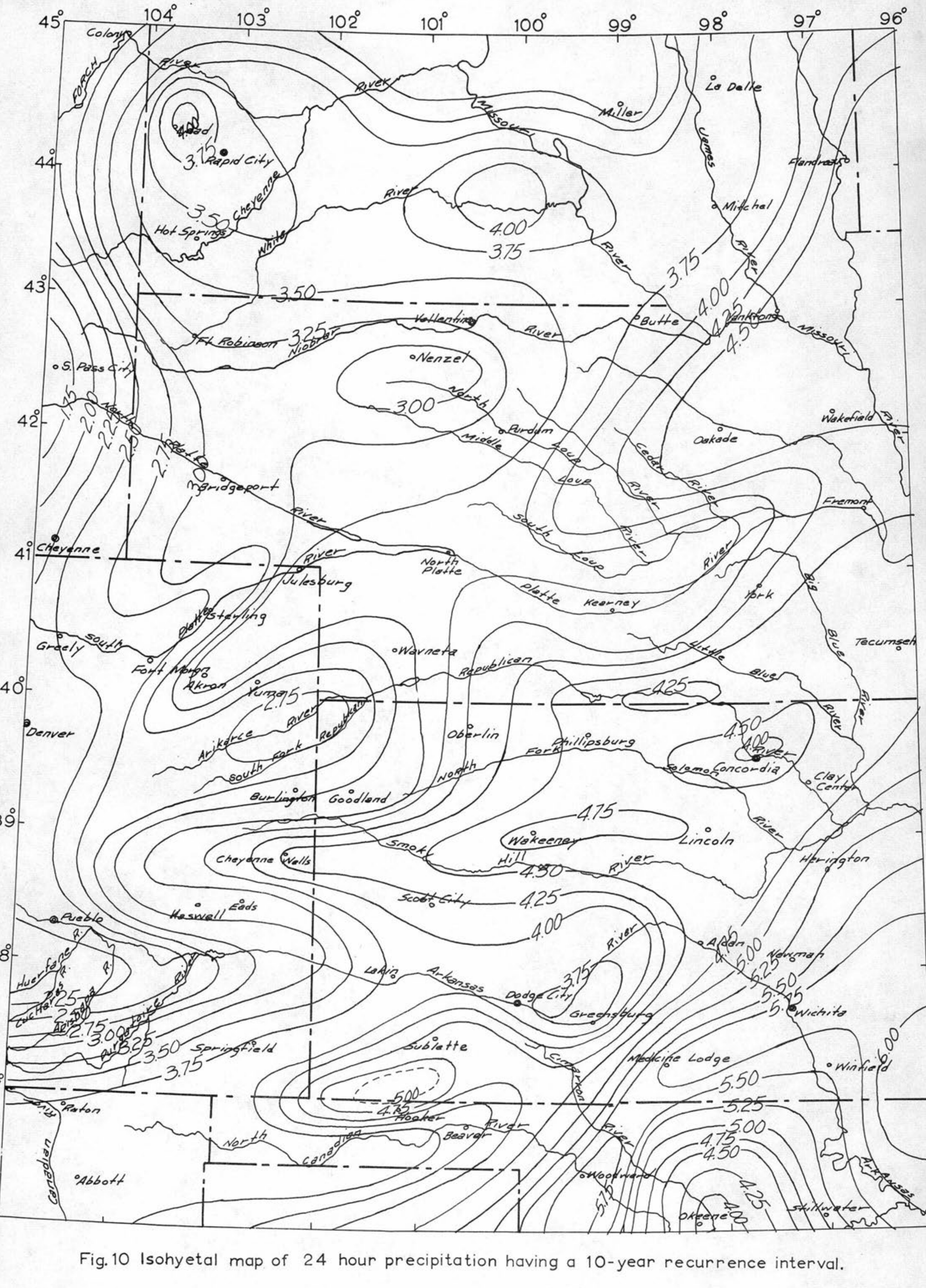


Fig. 10 Isohyetal map of 24 hour precipitation having a 10-year recurrence interval.

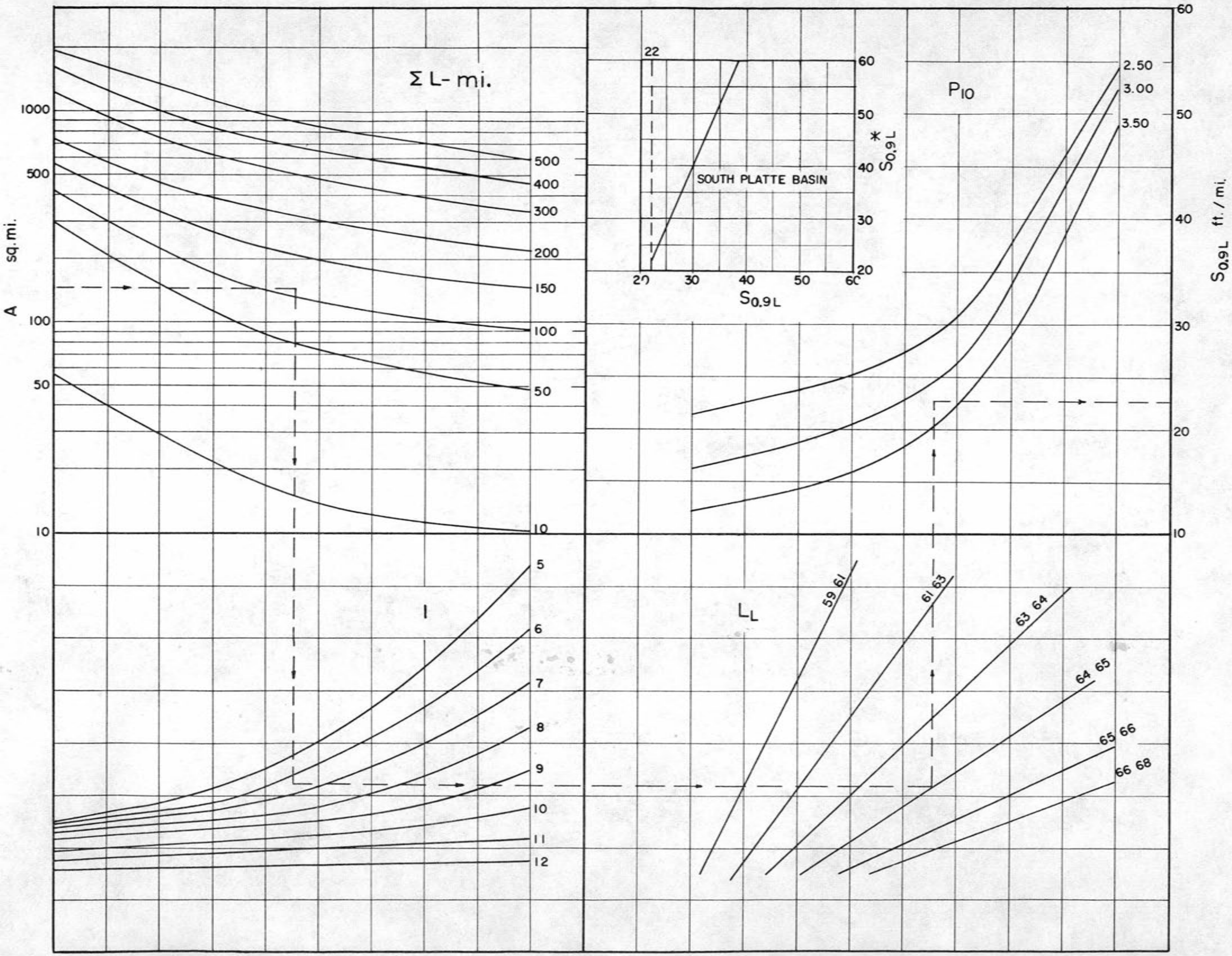


Fig.11 Relations for estimating S from A, ΣL, I, L_L, and P_{10}

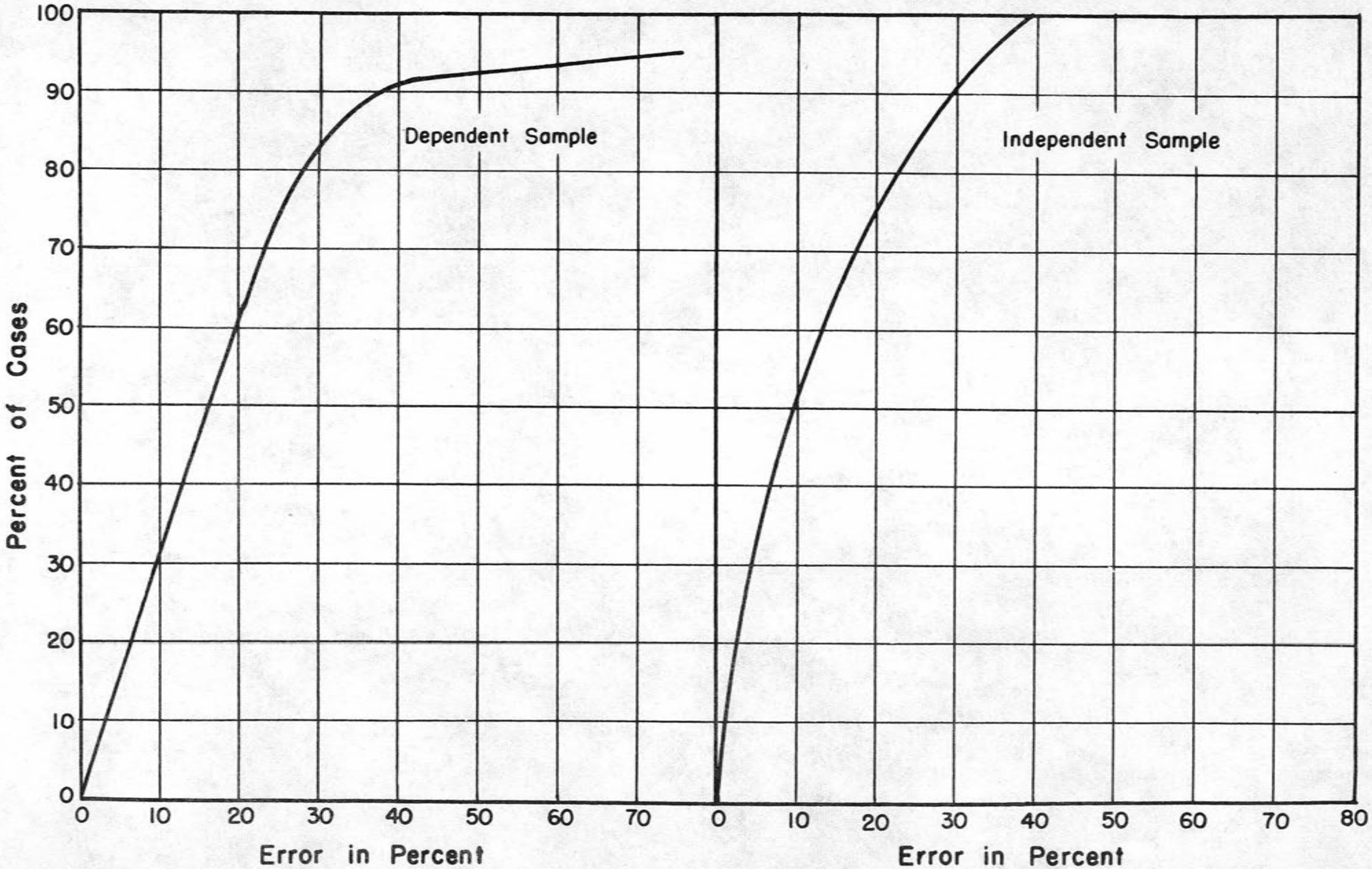


Fig. 12 Cumulative relative frequency of errors of estimate from Fig. 11.

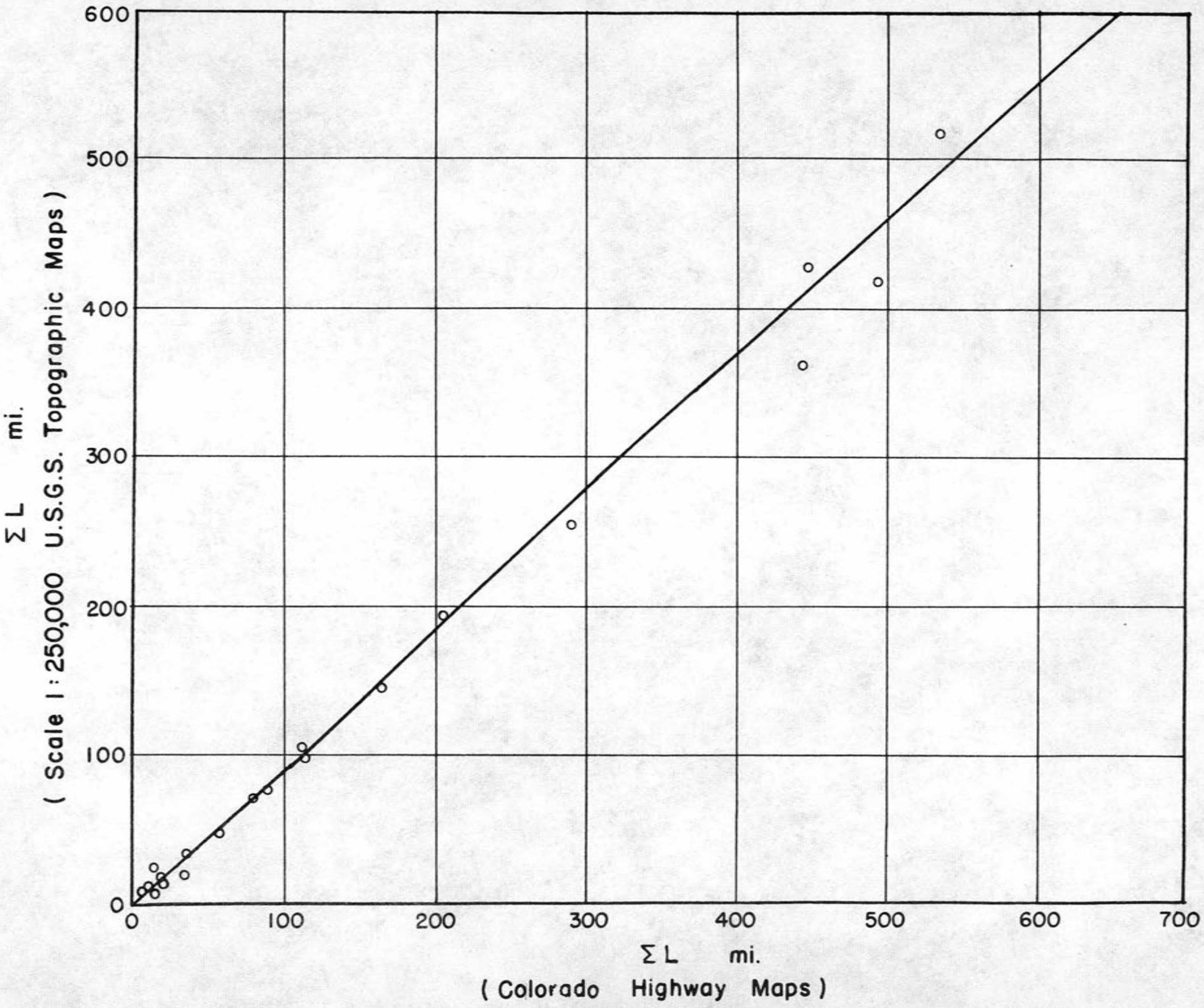


Fig. 13 Relations between the total channel length measured from 1:250,000 scale topographic maps and from Colorado Highway maps.

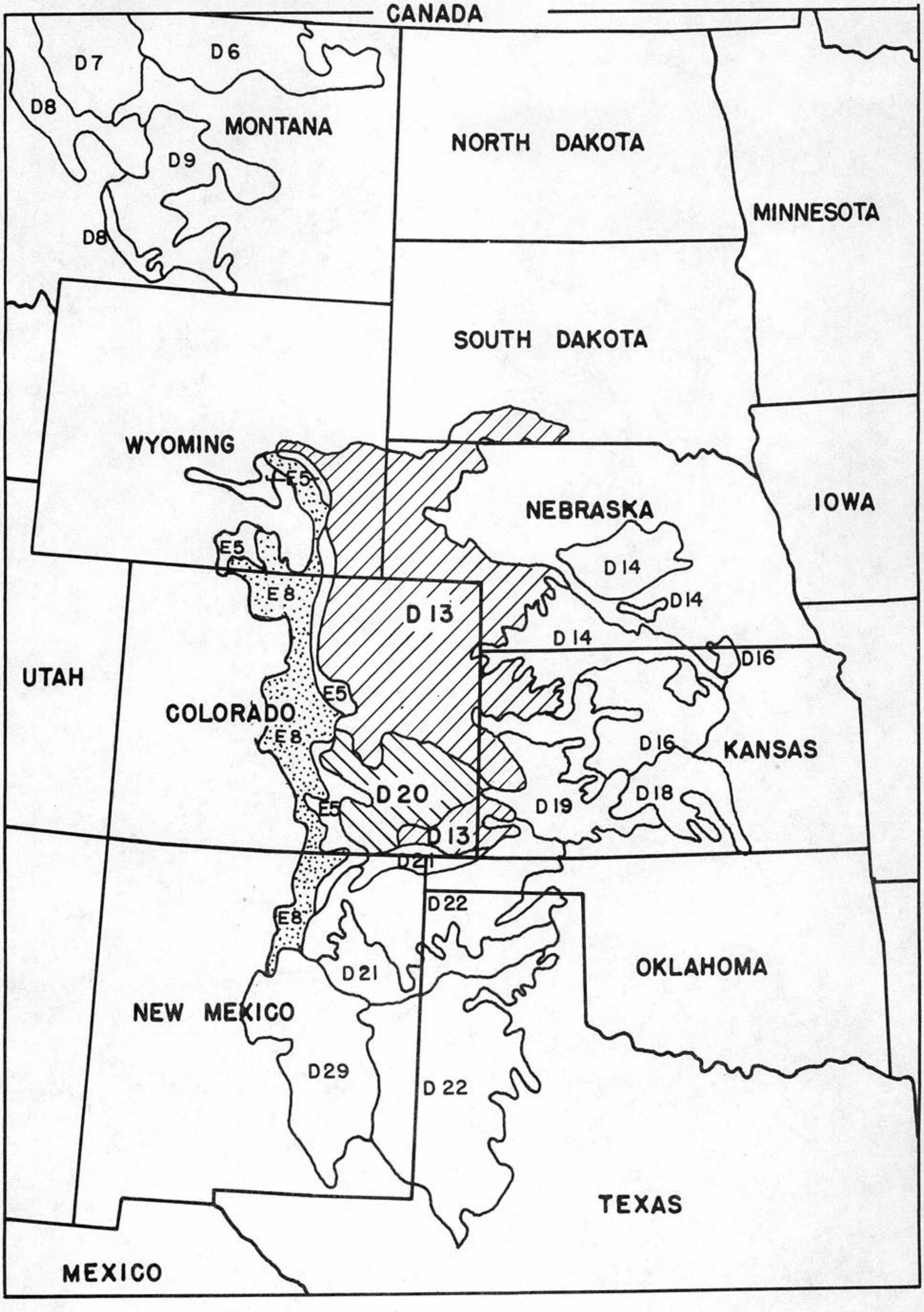


Fig. 14. Location of Study Area.

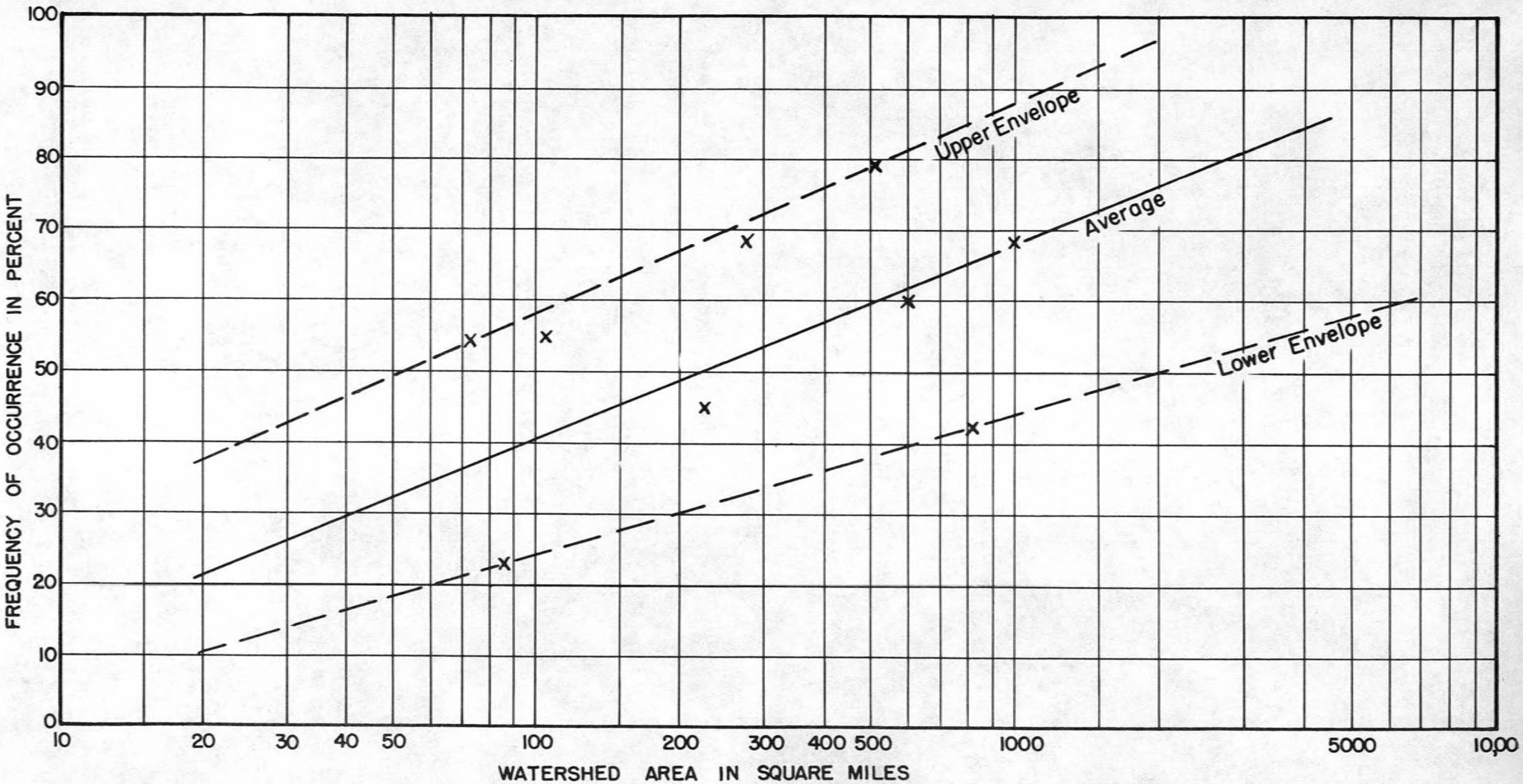


Fig. 15. Frequency of Occurrence of Complete Areal Rainfall Coverage Associated with Annual Maximum Flood Events as a Function of Area for Nine Watersheds in Eastern Colorado.

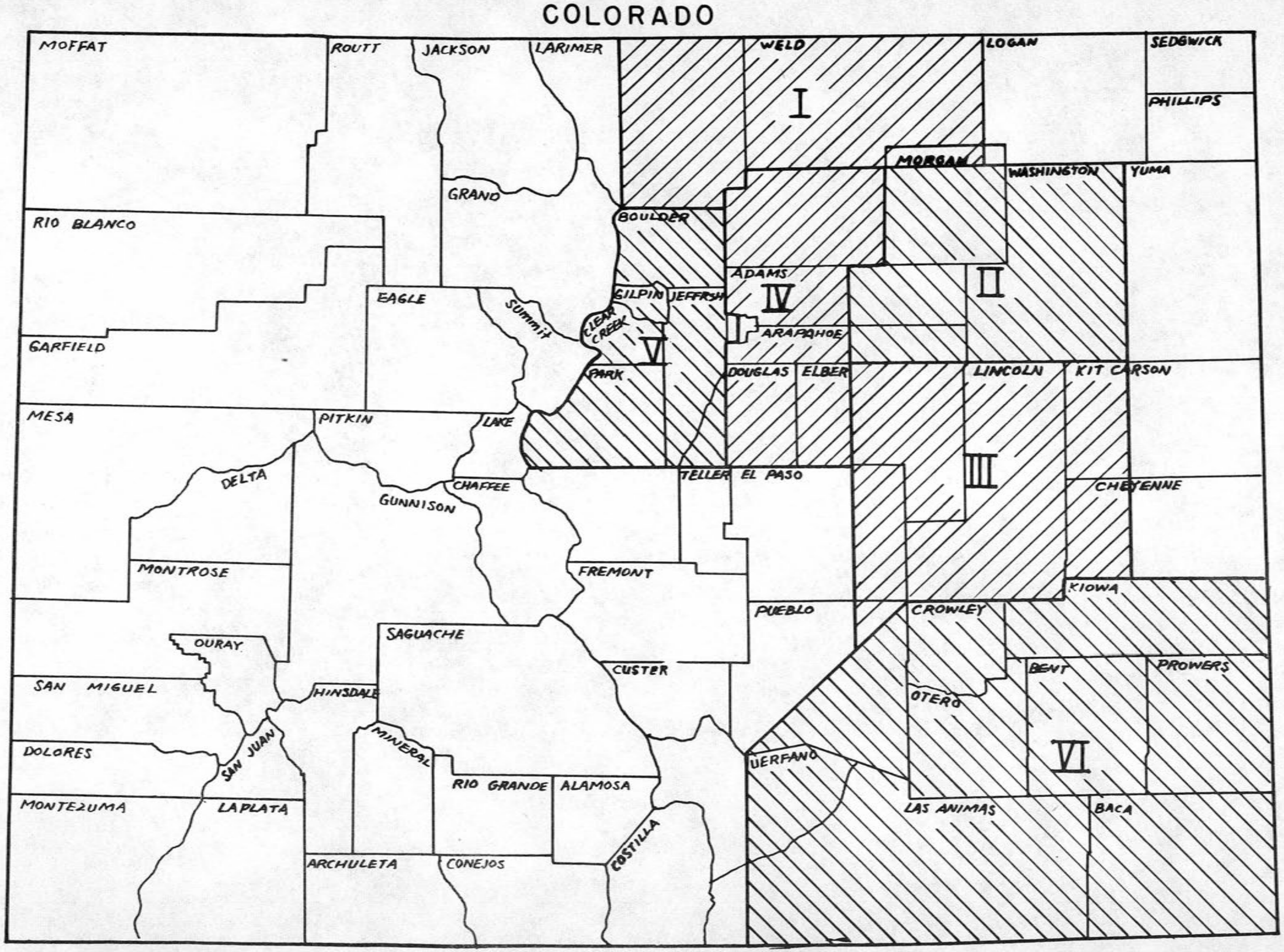


Fig. 16. Location Map for Relative Wetness Study.

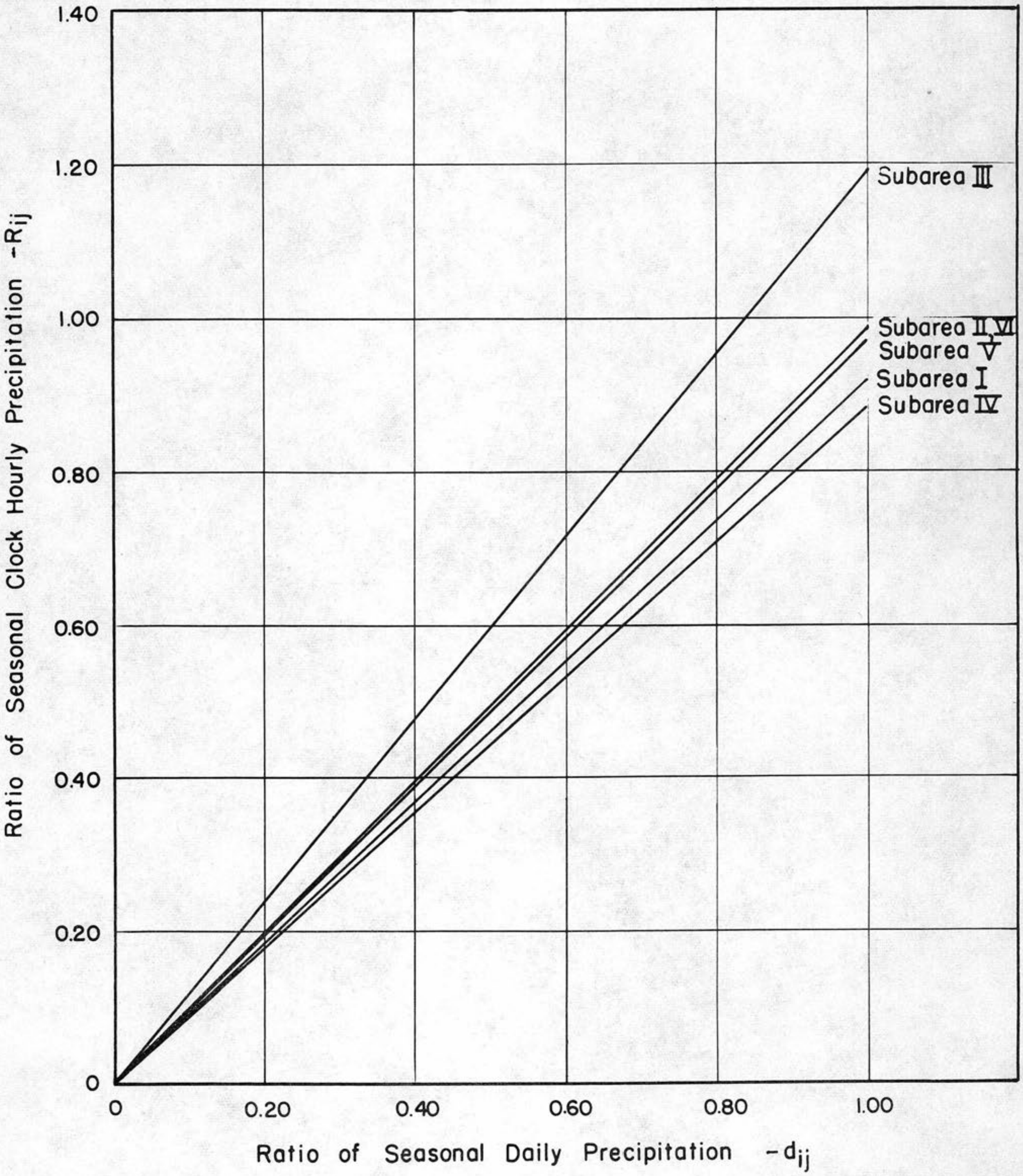


Fig. 17. Relations Between R_{ij} and d_{ij} for Subareas I-VI Shown in Fig. 16. (Seasonal 2 Year Values.)

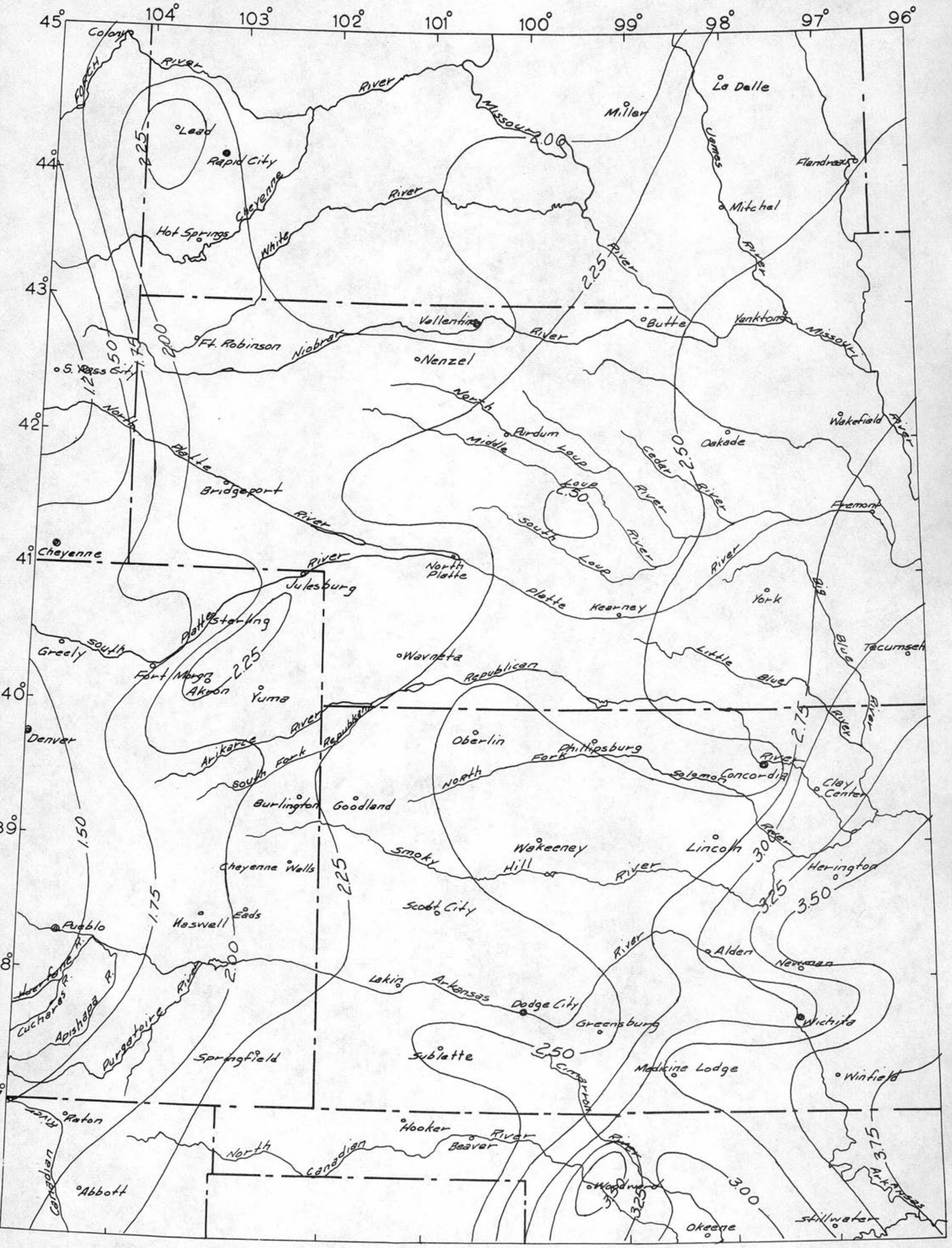


Fig.18 Isohyetal map of 24 hour precipitation having a recurrence interval of 2 years.

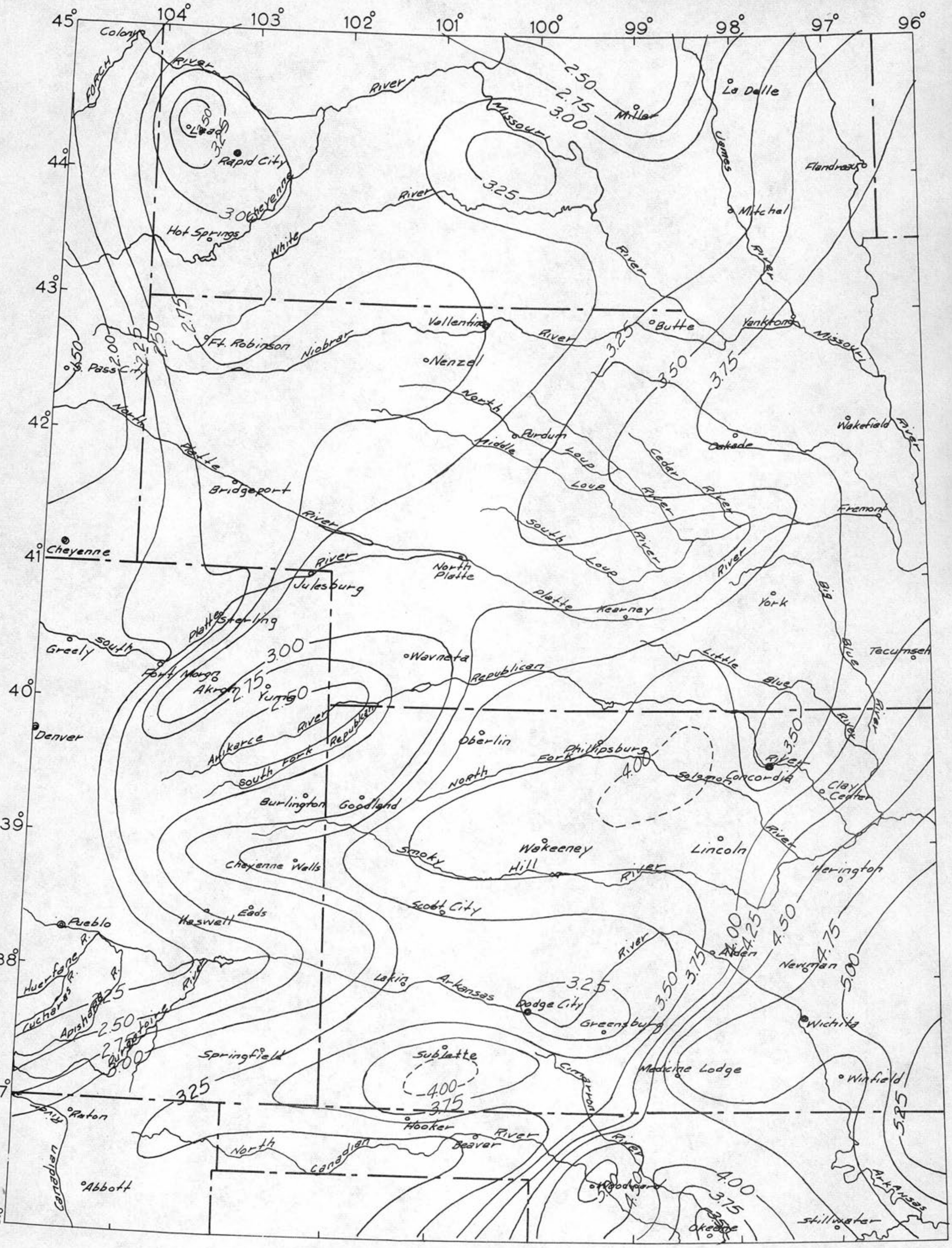


Fig.19 Isohyetal map of 24 hour precipitation having a recurrence interval of 5 years.

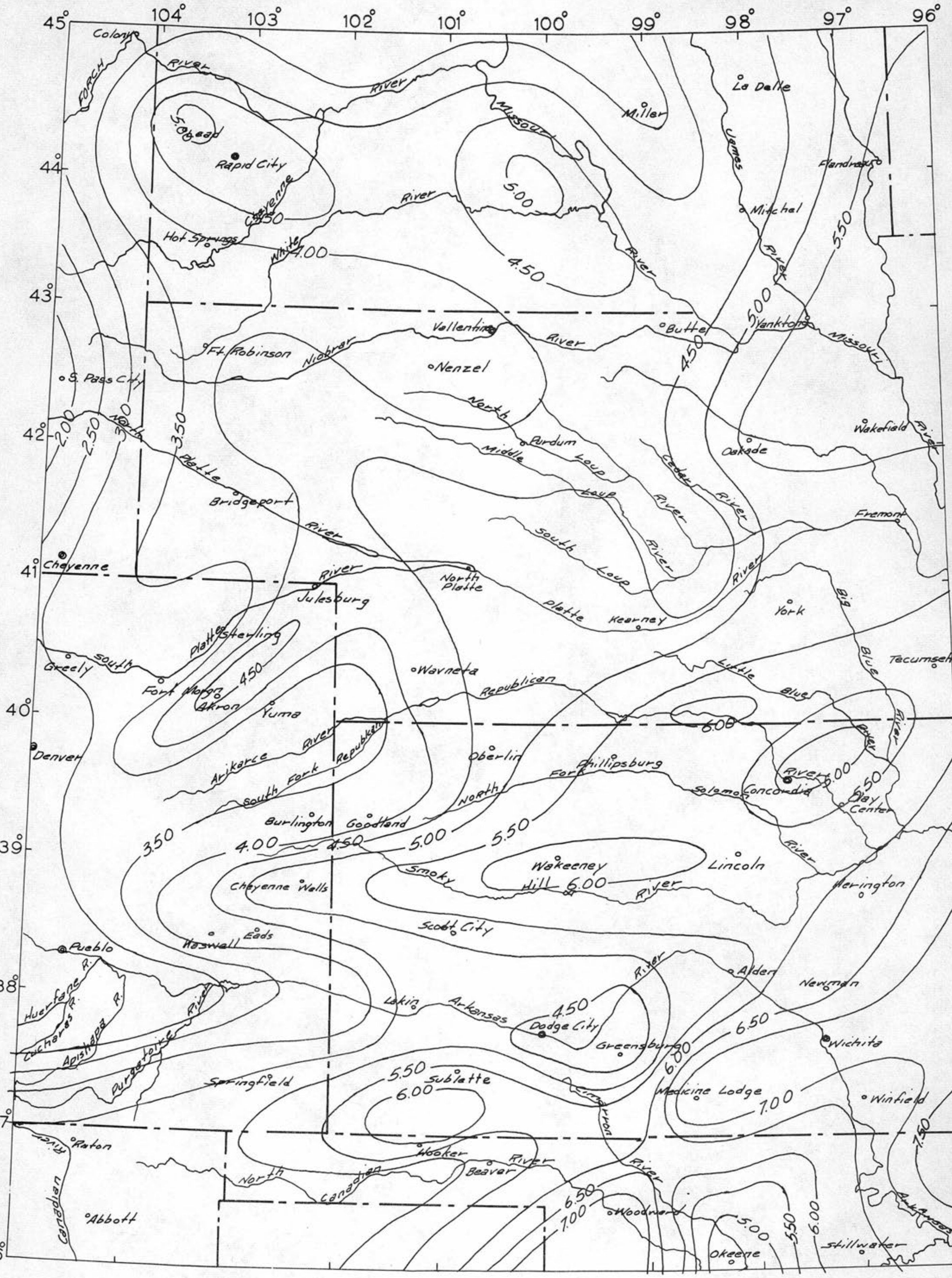


Fig. 20 Isohyetal map of 24 hour precipitation having a recurrence interval of 25 years.

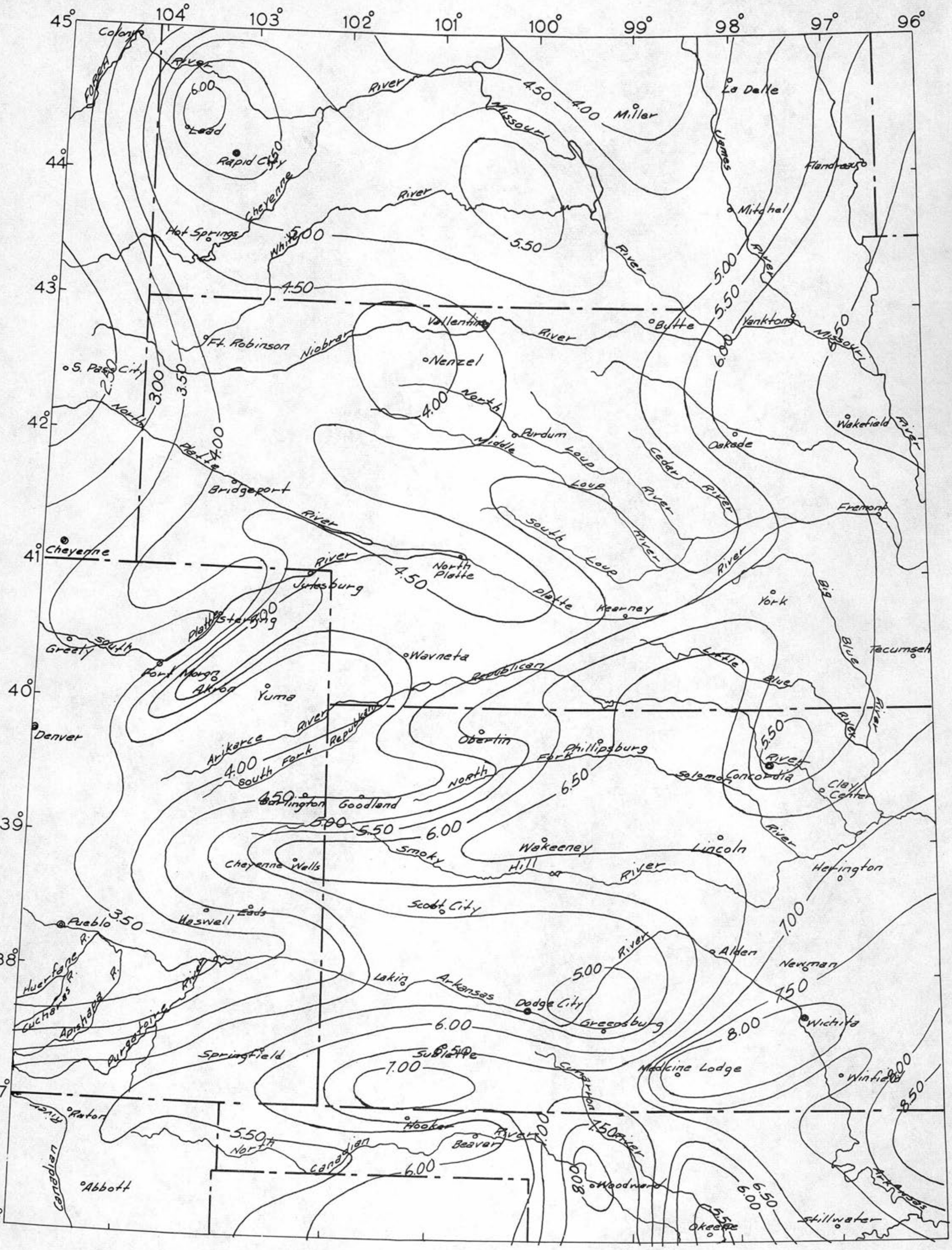


Fig. 21 Isohyetal map of 24 hour precipitation having a recurrence interval of 50 years.

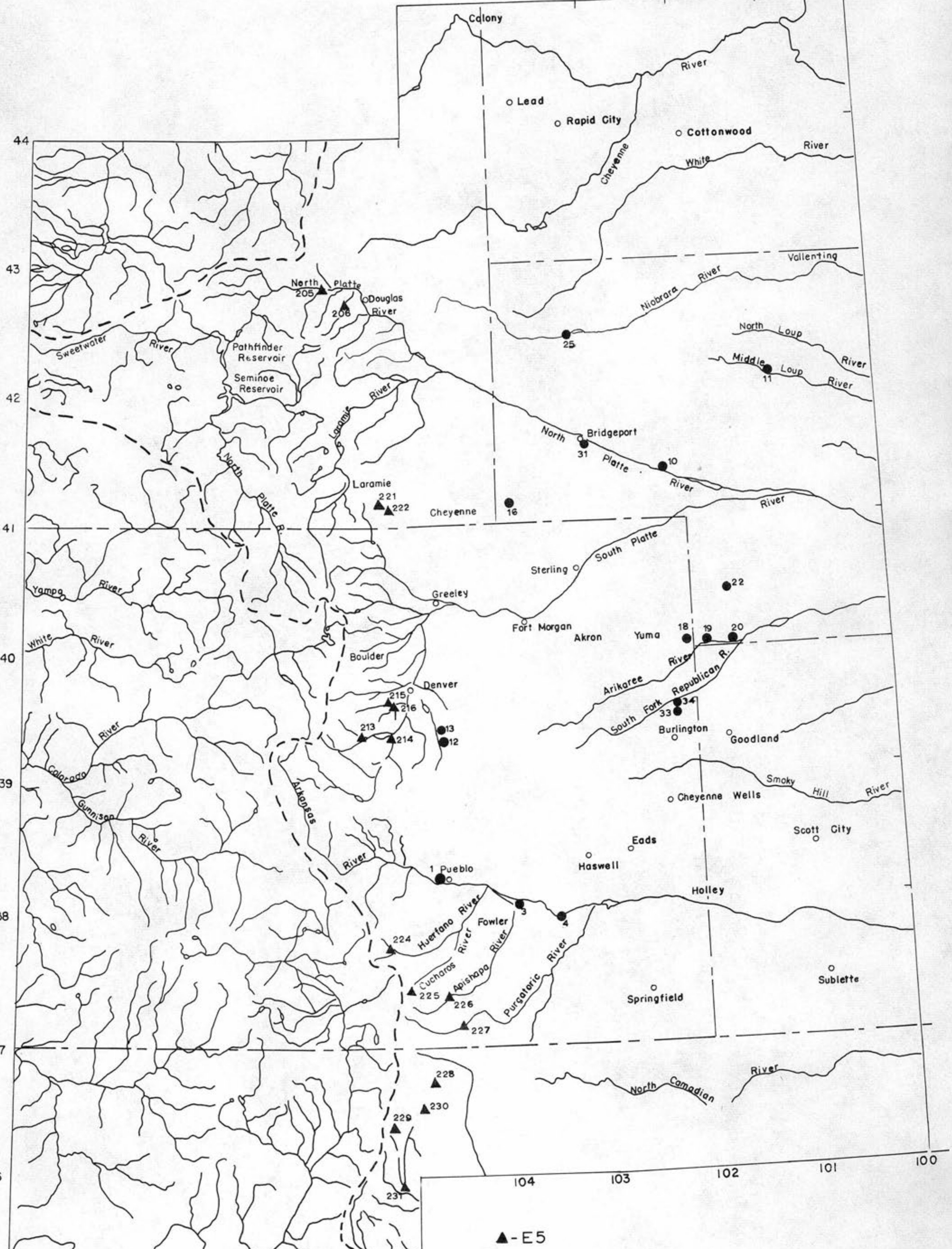


Fig. 22 Map showing location of gaging stations.

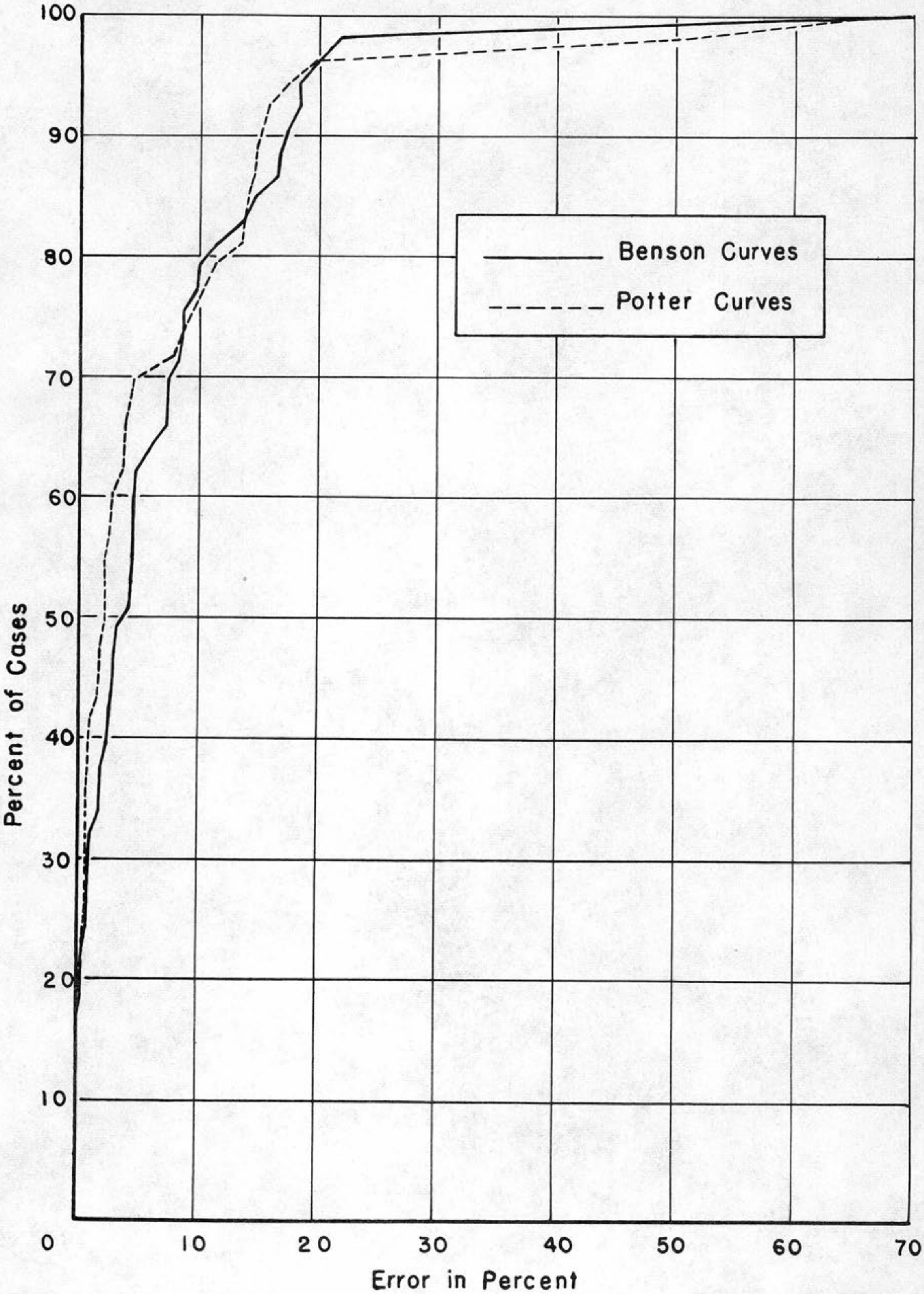


Fig. 23. Distribution of Error Curves Showing Departure of Plotted Points for Recurrence Intervals Greater Than 10 Years From "Benson" and "Potter" Type Curves on Gumbel Frequency Paper.

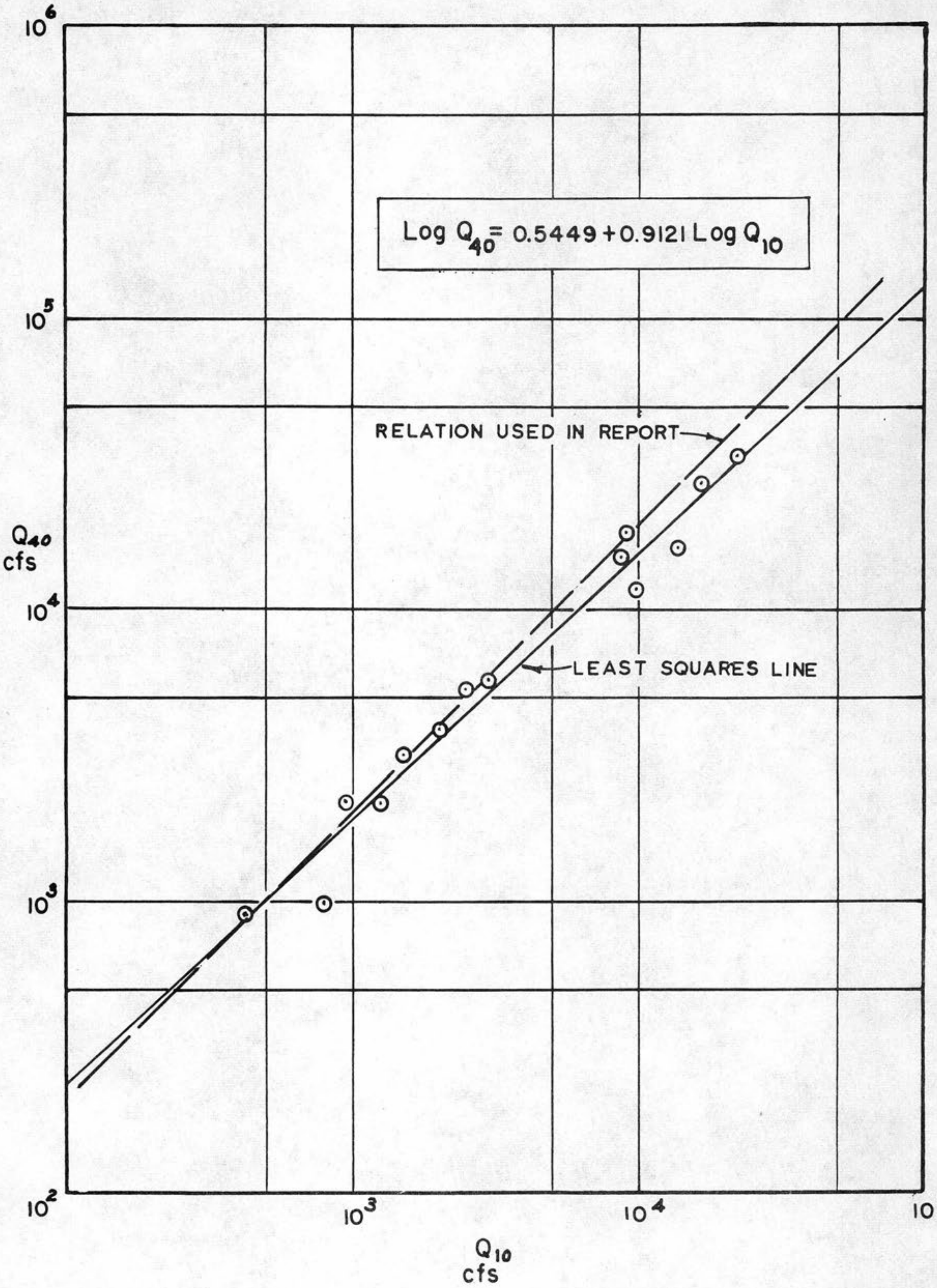


Fig. 24. Relation Between Q_{10} and Q_{40} (Benson method) for Selected Stations Outside D-13 and D-20 Problem Areas. (The solid line is a best-fit line by the method of least squares. The dashed line shows the relation used in this report.)

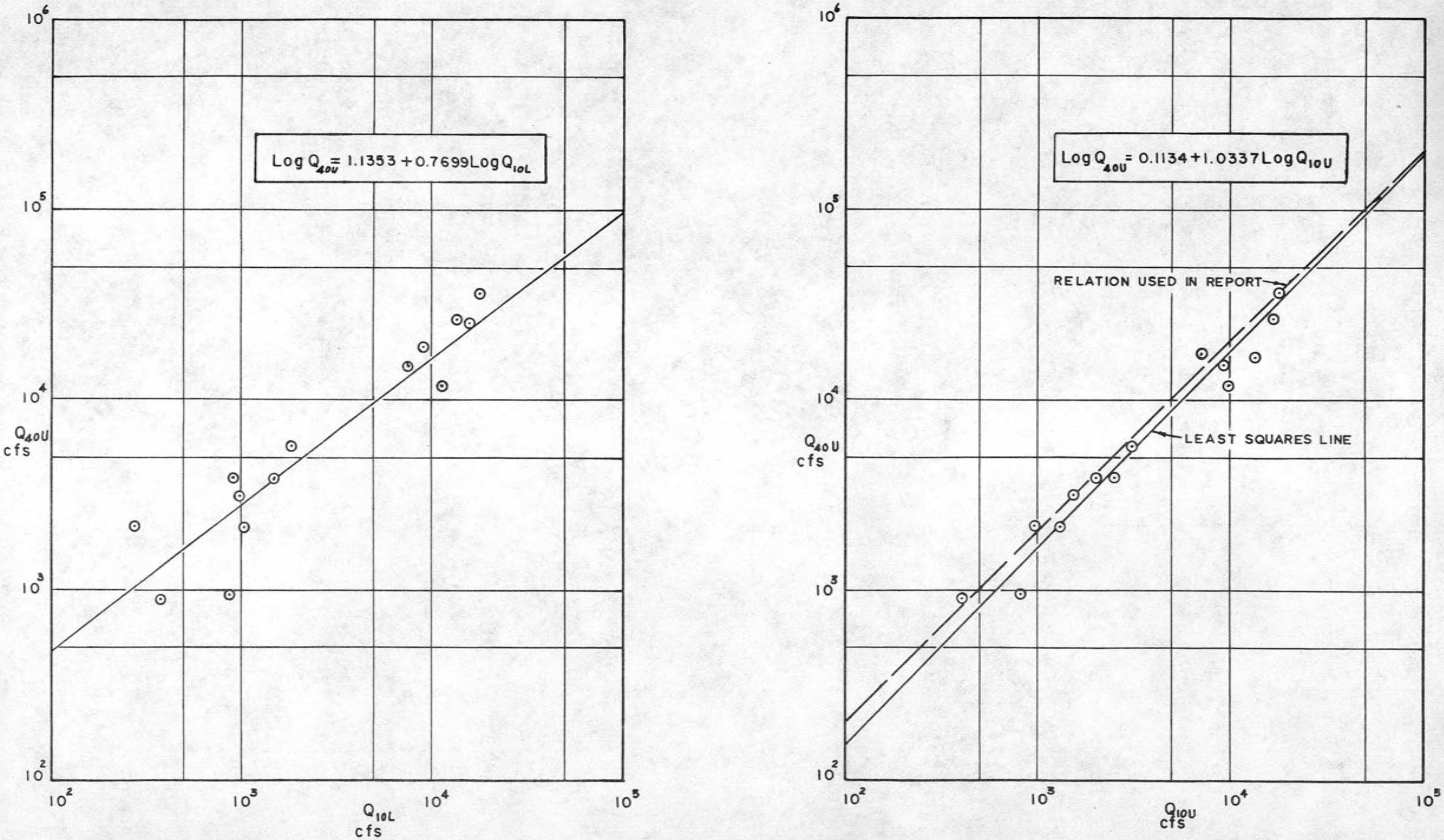


Fig. 25 Relation Between Q_{10L} and Q_{40U}, Q_{10U} and Q_{40U} (Potter method) for Selected Stations Outside D-13 and D-20 Problem Areas. (The solid lines were determined by the method of least squares. The dashed line shows the relation used in this report.)

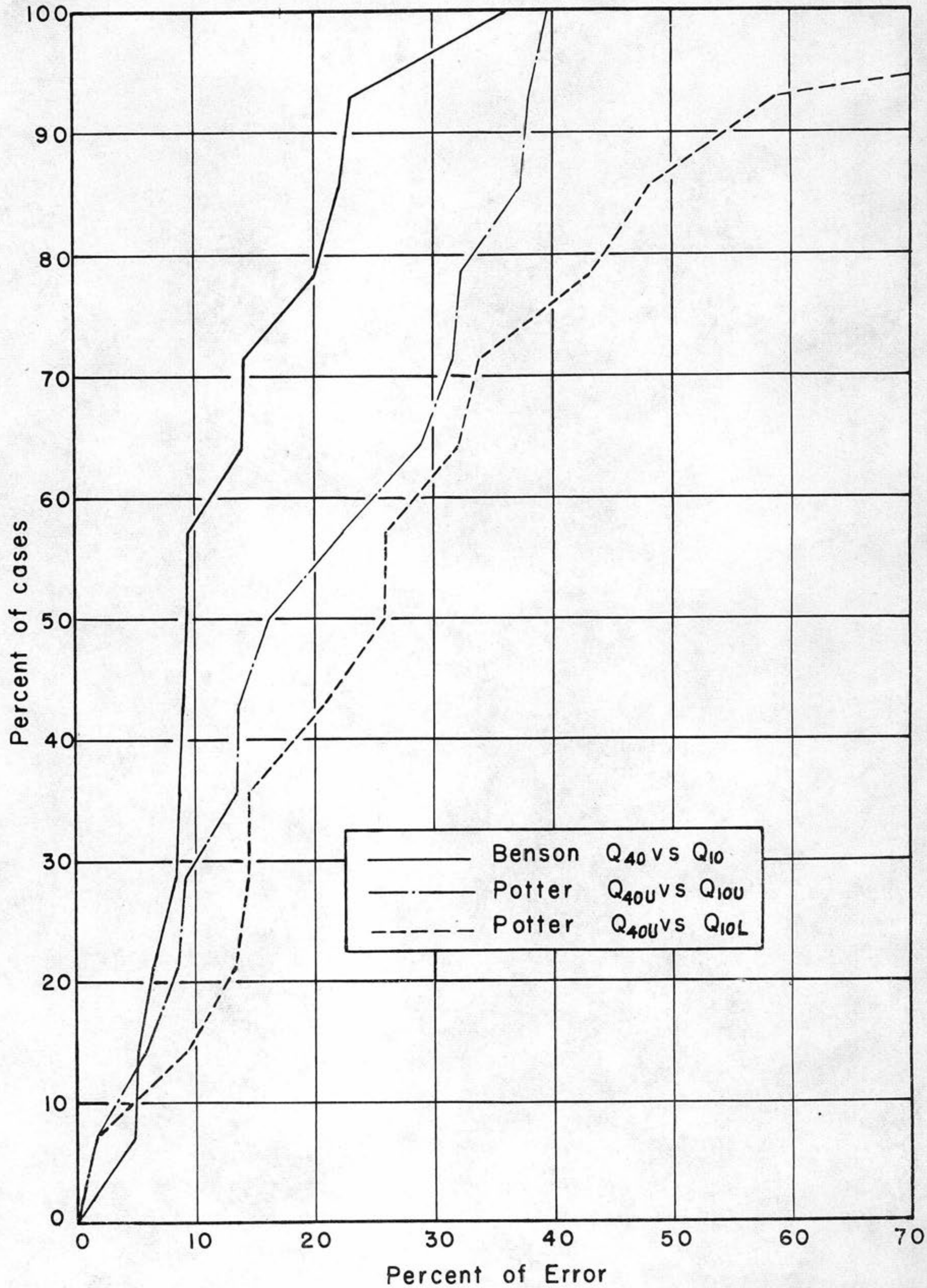


Fig. 26. Distribution of Error Curves for the Relations Shown in Figures 24 and 25.

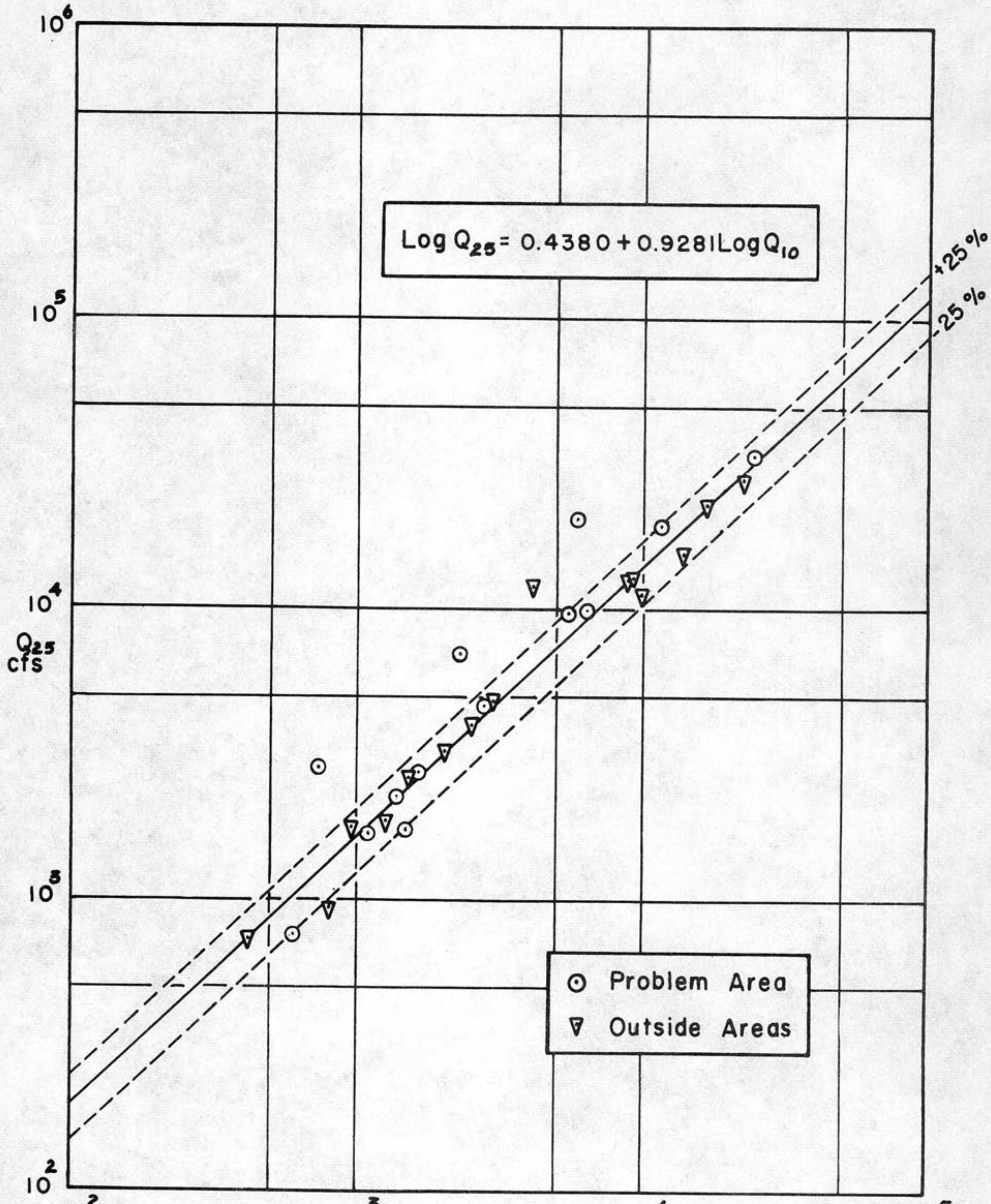


Fig. 27. Comparison of Q_{10} vs Q_{25} Relation for Stations Inside and Outside of the D-13 and D-20 Problem Areas.

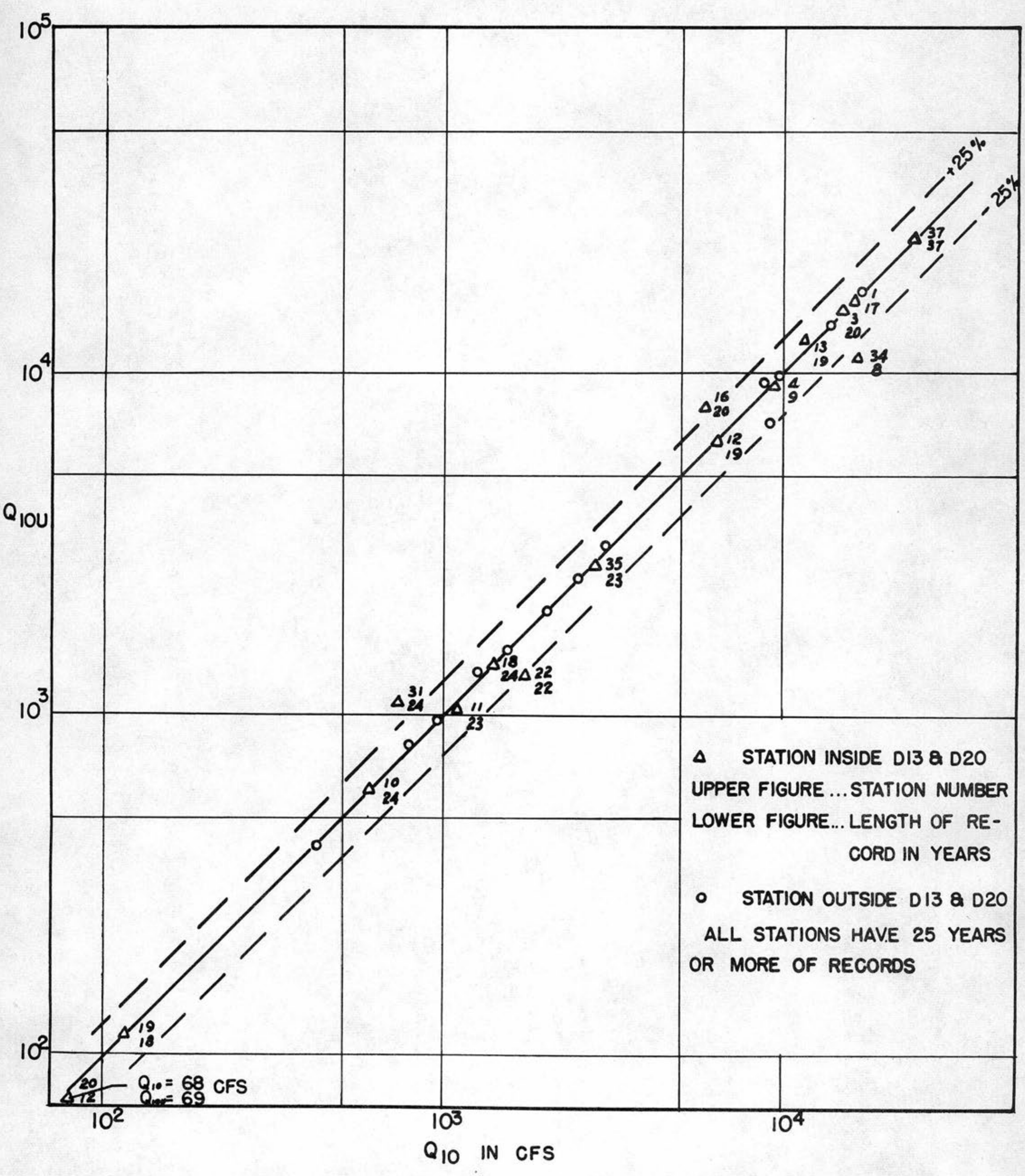


Fig. 28. Comparison of Q_{10} and Q_{10U} for Stations Inside and Outside the D-13 and D-20 Problem Areas.

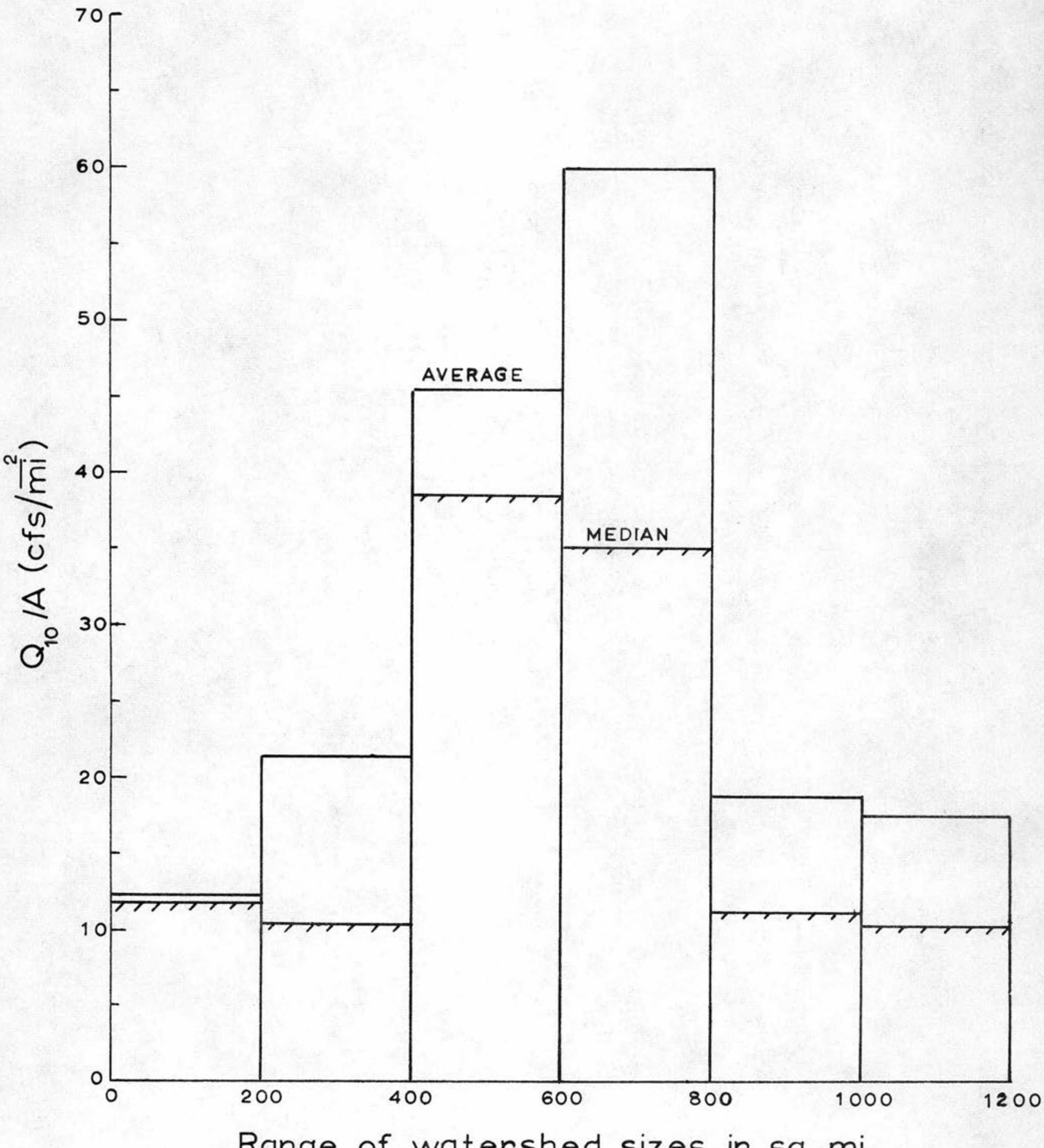


Fig. 29 Unit flow Q_{10}/A as a function of watershed size for 79 watersheds in the Western and Central Great Plains.

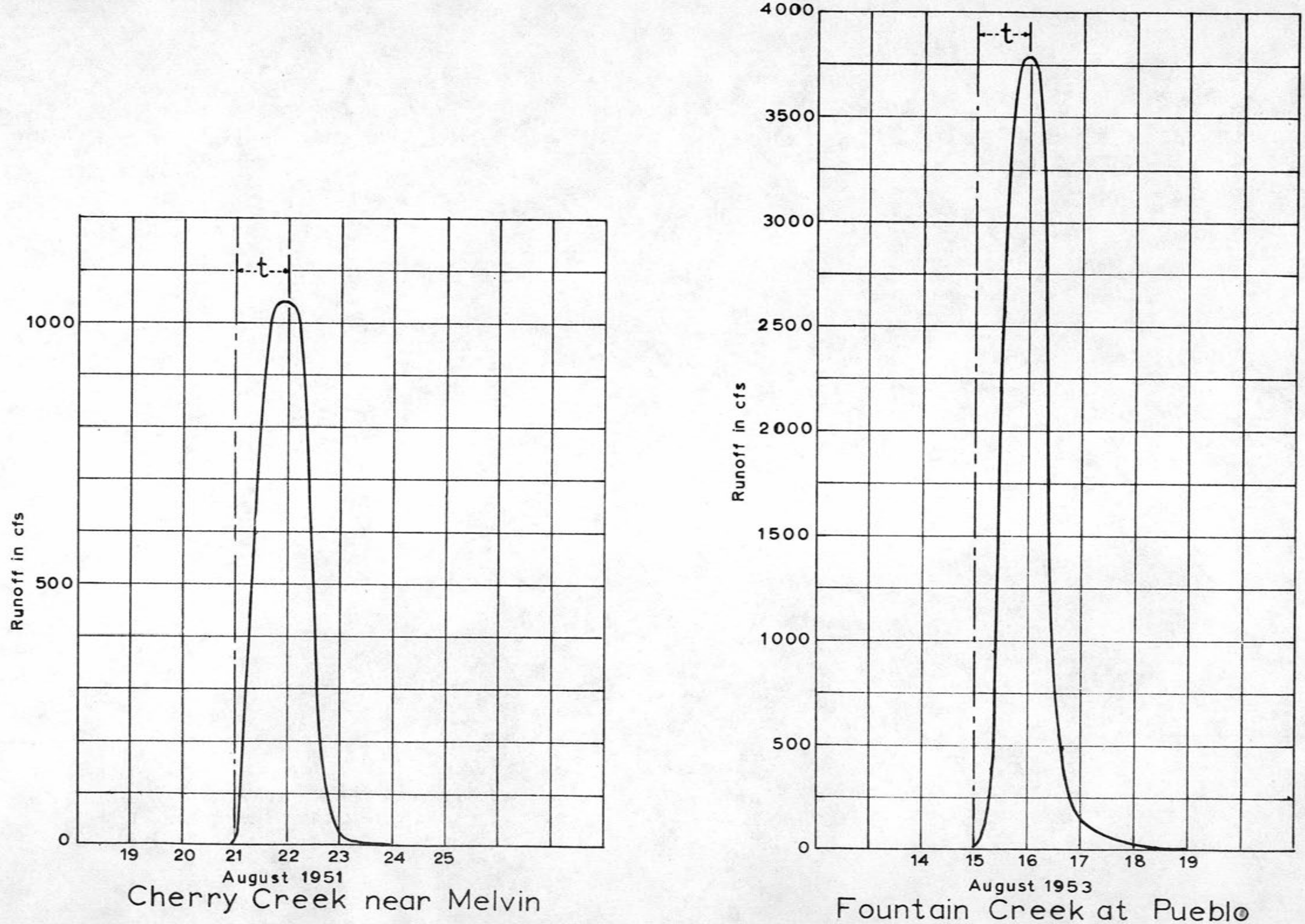


Fig. 30 Hydrographs for Cherry Creek and Fountain Creek.

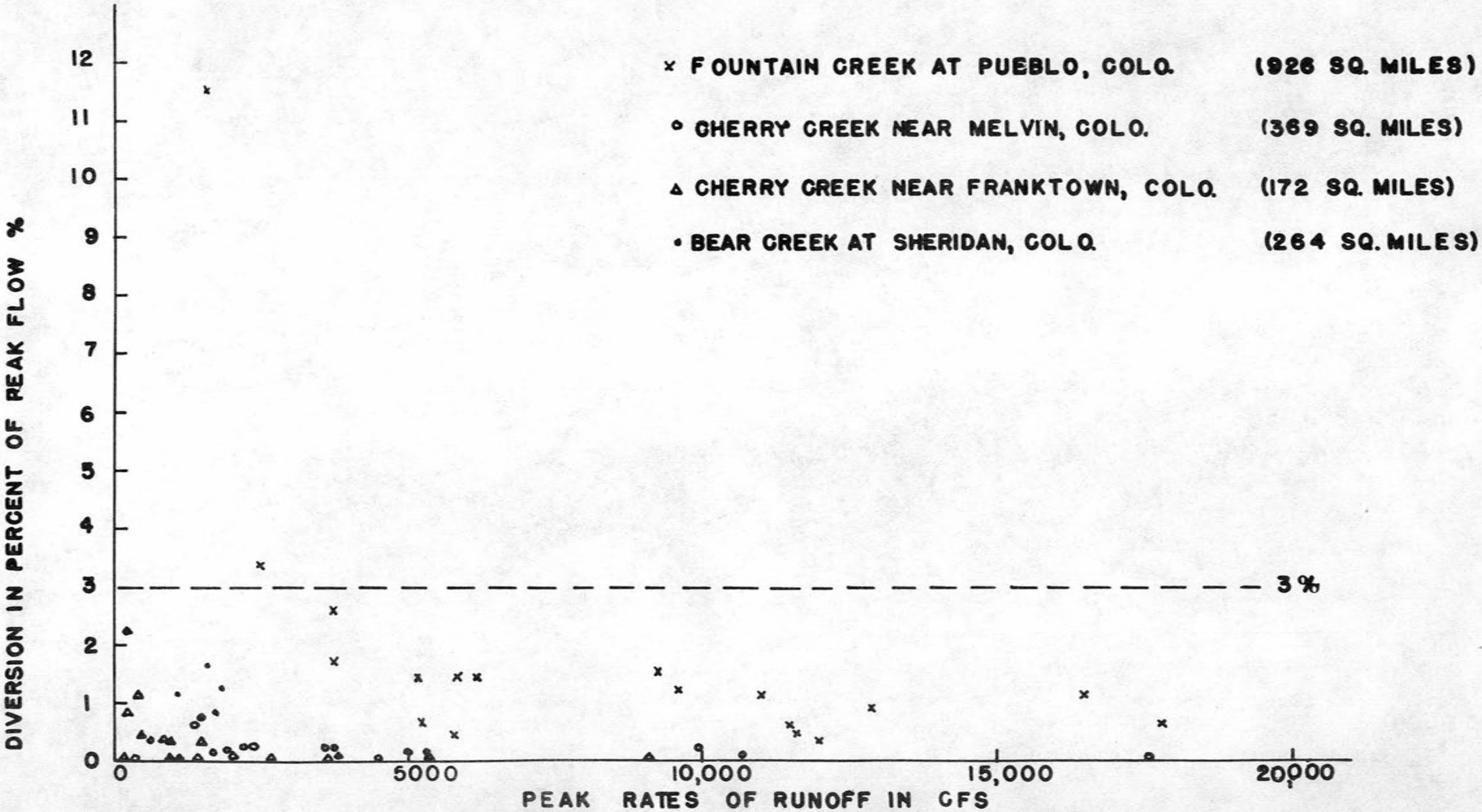


FIG. 31 DIVERSION FOR IRRIGATION AS A FRACTION OF PEAK RATES OF RUNOFF FOR FOUR STREAMS IN EASTERN COLORADO

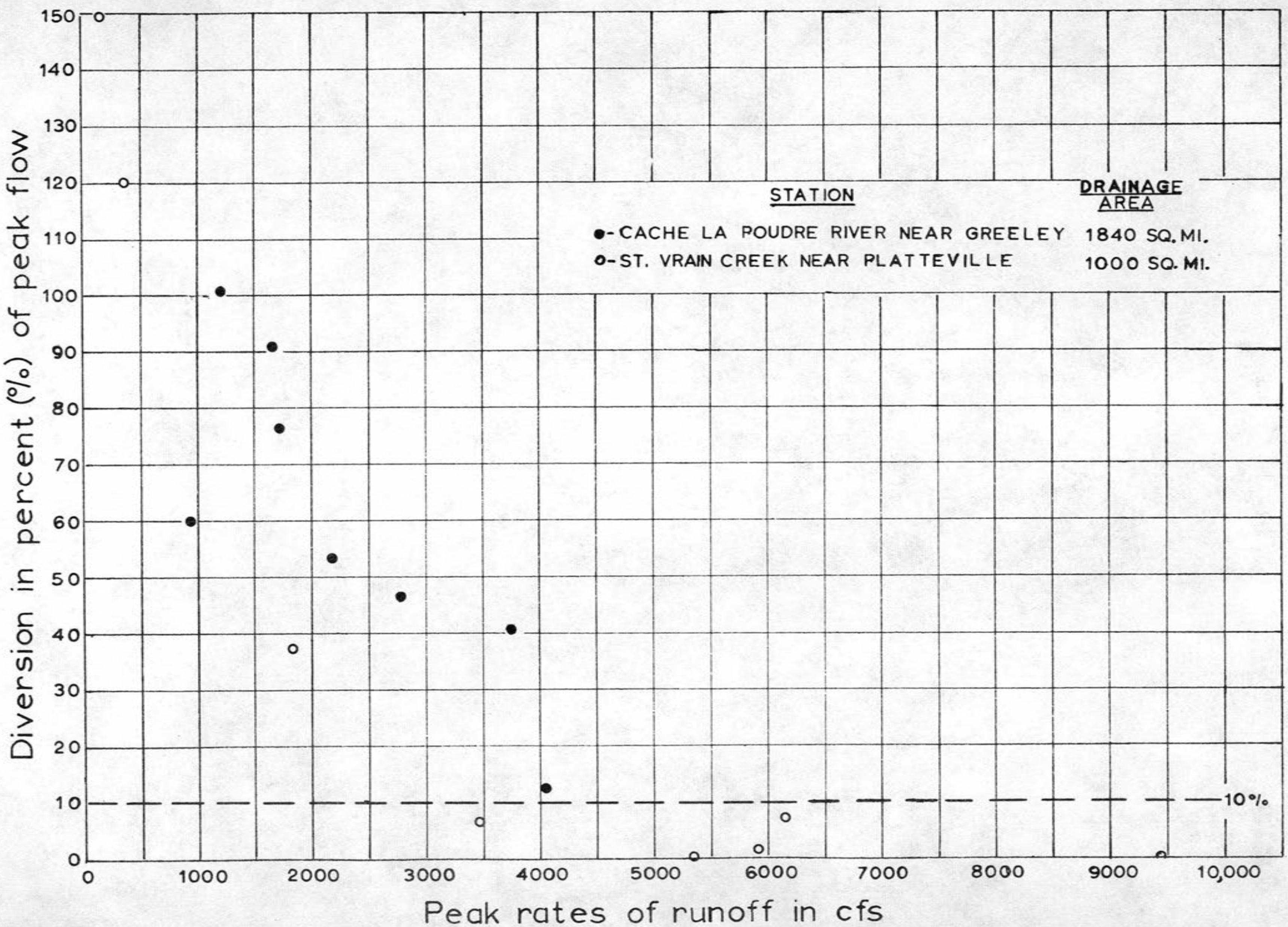
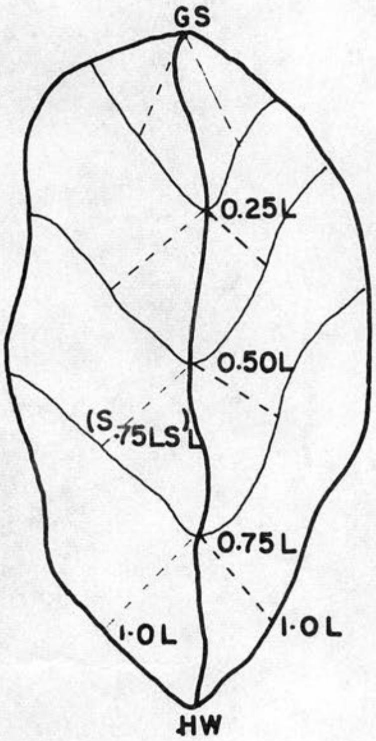
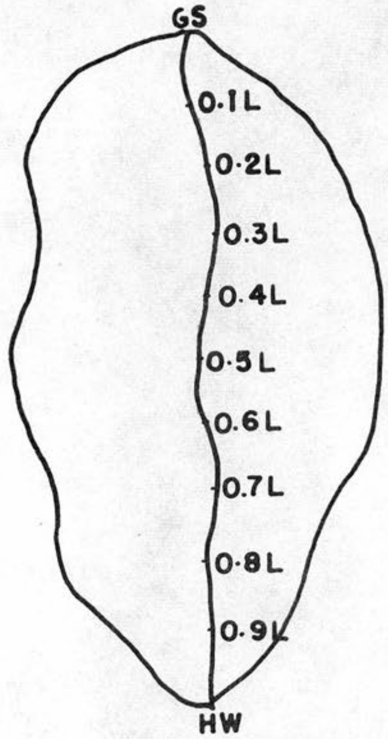


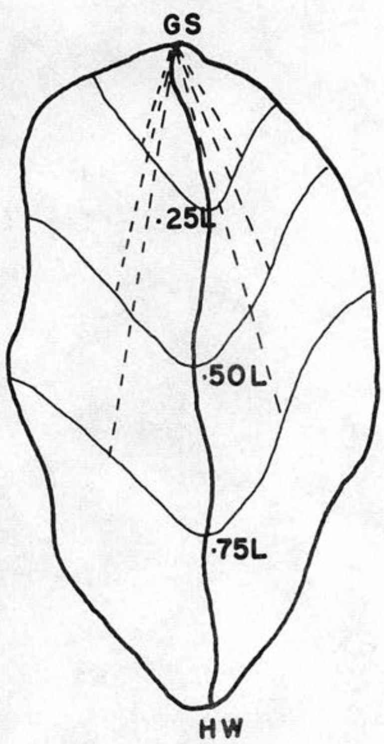
Fig. 32 Diversions for irrigation as a fraction of peak rates of runoff for the Cache la Poudre River and the St. Vrain Creek.



S_{LS}



S



S_{oo}

S	CHANNEL SLOPE
S_{LS}	OVERLAND SLOPE
S_{oo}	OBLIQUE OVERLAND SLOPE
C S	GAGING STATION
HW	HEAD WATER
L	MAIN CHANNEL LENGTH

Fig. 33 Definition sketches for determination of channel slope, S , overland slope, S_{LS} , and oblique overland slope (S_{00}).