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Technical Report

HYDROGRAPH RISE TIMES

by

Songthara Om Kar

Prepared for

Bureau of Land Management
U. S. Department of The Interior
as one of the reports
on investigations conducted under
Contract No. 14-11-0008-0590-62

Colorado State University
Civil Engineering Department
Fort Collins, Colorado

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ABSTRACT

HYDROGRAPH RISE TIMES

The runoff distribution with respect to time is needed for the design of many hydraulic structures. In most of the presently available techniques of flood peak estimation and design hydrograph prediction, it is necessary to determine characteristic or critical hydrograph times. This study suggests the use of the hydrograph rise time, which is the time between the commencement of runoff and the maximum discharge measured at a stream gaging station, as the time characteristic of a particular watershed. It is shown, however, that rise times within a watershed have considerable variability but a general distribution of values may be applied to all the watersheds used in this study. Because of the inadequacy of the existing rise time prediction methods, several new equations are derived in a stepwise regression analysis by computer. The investigation uses 407 flood events of 47 watersheds in 13 states, which is a larger sample than the data in any previous study of the same type. The conclusions are limited to floods caused by thunderstorms or storms of short duration (two or three hours) which are generally the most critical for small watersheds. Different equations are

obtained for estimating median rise times in humid regions and arid regions.

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June, 1967

RESUME

Les Temps de Montée

Dans la conception de nombreuses structures hydrauliques on a besoin de connaitre la répartition de l'écoulement dans le temps. Les techniques existantes pour la détermination de la pointe de crue et pour la prédiction de l'hydrogramme requièrent le calcul des temps caractéristiques ou critiques de l'hydrogramme. On a suggéré l'usage du temps de montée comme le temps caractéristique d'un bassin versant. Ce temps représente la période allant du commencement de l'écoulement jusqu'à la pointe de crue. Cependant, on a montré que les temps de montée dans un bassin varient considérablement, mais, qu'une distribution générale de valeurs peut être appliquée à tous les bassins étudiés. Comme les méthodes existantes pour déterminer les temps de montée ne sont pas adéquates on a dérivé plusieurs formules nouvelles par analyse régressive en employant un calculateur électronique. Dans cet étude on a utilisé 407 crues dans 47 bassins et 13 états - c'est le plus grand échantillon de données utilisé dans des études de ce genre. Les conclusions sont limitées aux crues causées par les averses ou les pluies de courte durée (deux ou trois heures) qui sont généralement les plus critiques

dans les petits bassins. On a obtenu des formules différentes pour calculer la médiane des temps de montée dans les régions humides et dans les régions arides.

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LIST OF SYMBOLS

Watershed Characteristics *

A	Area	Square miles
C	Compactness coefficient	Dimensionless
D_d	Drainage density	Miles per sq. mi.
F	Form factor	Dimensionless
H	Total fall	Feet
L	Length of main stream	Miles
L_c	Distance to centroid of area	Miles
L_m	Dimensionless mean travel distance	Dimensionless
L_s	Total length of extended streams	Miles
L_t	Average travel distance	Miles
P	Perimeter of catchment	Miles
R_1, R_2, R_3, R_4, R_5	Overland slope	Feet per mile
R_6	Dimensionless overland slope	Dimensionless
S_1, S_2, S_3, S_4	Stream slope	Feet per mile
W	Average width of catchment	Miles

* The definitions of the watershed characteristics are in Appendix A.

Runoff Parameters

API	Antecedent precipitation index	Inch
P_s	Total rainfall	Inch
q	Runoff rate	Inch per hour
P_F	Flood frequency	Decimal

Time Parameters

D	Rainfall duration	Minutes
D_e	Rainfall excess duration	Minutes
T_B	Base time	Hours
T_c	Time of concentration	Hours
T_e	Time to equilibrium	Hours
T_L	Lag time or storage delay time	Hours
T_P	Basin lag or storage delay	Hours
T_T	Translation time	Hours
T_R, T_{R1}, T_{R2}	Rise time *	Hours
T_M, T_{M1}, T_{M2}	Median rise time **	Hours
R_T, R_{T1}, R_{T2}	Relative rise time ***	Dimensionless

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* Specific definition is in Section 2 of Chapter IV.
** Specific definition is in Section 2 of Chapter V.
*** Specific definition is in Section 3 of Chapter V.

Chapter I

INTRODUCTION

Numerous methods are presently available to estimate the total volume of runoff associated with a particular combination of storm rainfall and antecedent conditions (1, 2, 3) *. In many instances, however, the computation of runoff volume is only a preliminary step in the determination of the outflow hydrograph from a basin. The design of hydraulic structures may be cited as a case where the engineer is concerned not only with the total volume but also with the maximum discharge and its time of occurrence.

Various studies have provided methods of estimating flood peaks directly from watershed characteristics such as area, slope, location etc... (4, 5, 6). It appears possible to derive similar relationships for estimating rise time and recession characteristics so that the complete design hydrograph may be synthesized.

For small watersheds, flood frequencies are often derived from rainfall frequencies by techniques such as Unit-graph method (7), "Rational" Formula (8), Soil Conservation Service Hydrograph Families (9), Bureau of Public Roads method (10), Chow's method

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* Numbers in the parentheses refer to the references beginning on page 80.

(11), and Tacitly Maximized Peaks (12). In most of these it is necessary to estimate characteristic or critical hydrograph times that are related to the rise time and effective rainfall durations. These are mainly watershed characteristics, long characteristic times being associated with large or slow-responding watersheds and vice versa.

Possibly no part of the hydrologic literature is more confusing than that devoted to the determination of these times. There are many slightly different concepts, some of which have conflicting interpretations. They include "rise time", "storage delay", "lag", "critical duration", "effective duration", "optimum duration" and "time to peak". Further, there are many suggested methods of estimating these values but most, if not all, are of uncertain reliability (13). This study should clarify the significance of the concepts and provide some recommendations for improved estimates of the required times.

Chapter II

REVIEW OF LITERATURE

As mentioned in the introduction, no part of the hydrologic literature is more confusing than that devoted to the determination of time parameters. This chapter is limited only to the literature survey on rise time or its equivalent.

1. Ramser, 1927 (14)

Ramser conducted a series of rainfall and runoff measurements on six agricultural watersheds ranging in area from 1.25 acres to 112.0 acres, near Jackson, Tennessee. The time of concentration was defined as the time required for the water to flow from the farthest point on the watershed to the gaging station. This was determined by noting the rise time which is the time required for the water in the channel at the gaging station to rise from the low to the maximum stage as recorded by the water-stage recorders. Ramser found that this period varied to some extent for the different rains, depending upon the degree of saturation of the watersheds at the occurrence of the rain that produced the maximum rate of runoff. He also found that the time of concentration would be less if the channel were partially filled with water when the rains of greatest intensity occurred, and would be less for rains of high intensity than for rains of less intensity. However, owing to the inability of the recording instruments to register the time with high accuracy, the time of concentration or rise time was taken to be the same for the different rains on the same watershed.

2. Kirpich, 1940 (20)

Kirpich presented two curves based on data obtained by Ramser (14) for evaluation of time of concentration. In the two curves shown (figures 1 and 2) it is assumed that the time of concentration for any watershed depends on a factor, K, which varies directly with the length of travel and inversely, with the square root of the slope. It could be put into a form of empirical formula (21) as follows:

$$T_C = 0.00013 \frac{L^{0.77}}{S^{0.385}}$$

where L is the length of the basin areas in miles, measured along the water course from the gaging station and in a direct line from the upper end of the watercourse to the farthest point on the drainage basin; and S is the ratio in feet to L of the fall of the basin from the farthest point on the basin to the outlet of runoff, or approximately the average slope of the basin in dimensionless ratio.

Considering the original data obtained by Ramser (14), this T_C is actually the rise time.

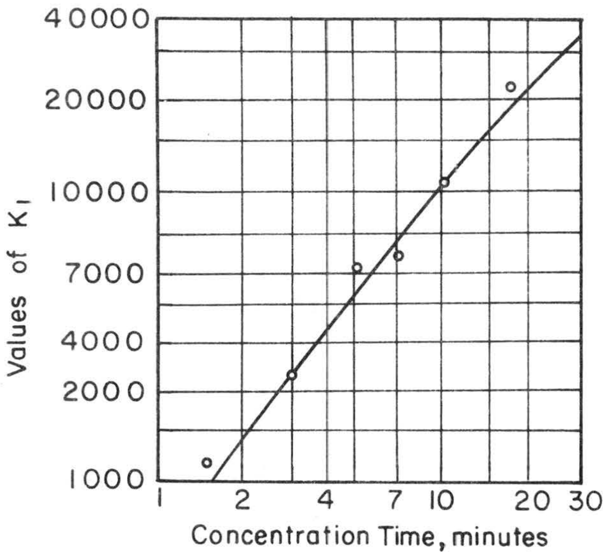


Figure 1. Concentration time of small agricultural watersheds, based on K_1 (after Z. P. Kirpich, 1940)

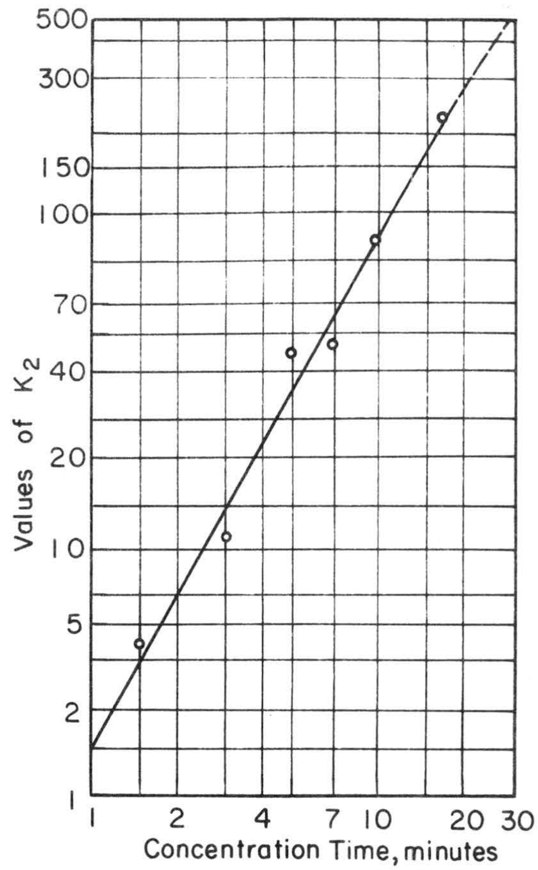


Figure 2. Concentration time of small agricultural watersheds, based on K_2 (after Z.P. Kirpich, 1940)

$K_1 = L/S_1$; $K_2 = L/S_2$

$S_1 = H_1/L$; $S_2 = H_2/A$

H_1 is the difference in elevation in feet between the most remote point and the outlet.

H_2 is the average height of the watershed above the outlet in feet.

L is the length of travel in feet.

A is the area of the watershed in square miles.

These definitions are referred to the work of Kirpich only.

3. Soil Conservation Service, 1957 (9)

The Soil Conservation Service developed a nomograph, figure 3, from Kirpich's paper (20), which is based on Ramser's experimental data (14). In the original study of Ramser's, the values presented were actually the hydrograph rise times although they were called times of concentration. The SCS has apparently overlooked this because in their report they regard the times to peak as different to Ramser's original times of concentration. The SCS nomograph is suggested for estimating the time of concentration T_C in hours knowing the length in feet, L, of the longest waterway from the watershed outlet to the ridge and the difference in elevation, H, between the watershed outlet and the farthermost point, in feet.

The SCS suggested that an average time of concentration can be obtained using the time from beginning of rise to peak, T_R, in the equation

$$T_R = T_C^{0.5} + 0.6\ T_C$$

Figure 4 permits an easy solution for T_R once the T_C is known. T_R is averaged value for simple hydrographs or from field observations of simple hydrographs.

For the design hydrograph, the SCS suggested to estimate the time to peak, T_R, by relating it to the duration of rainfall excess, D_e, and the time of concentration, T_C, of the watershed.

$$T_R = 0.5\ D_e + 0.6\ T_C$$

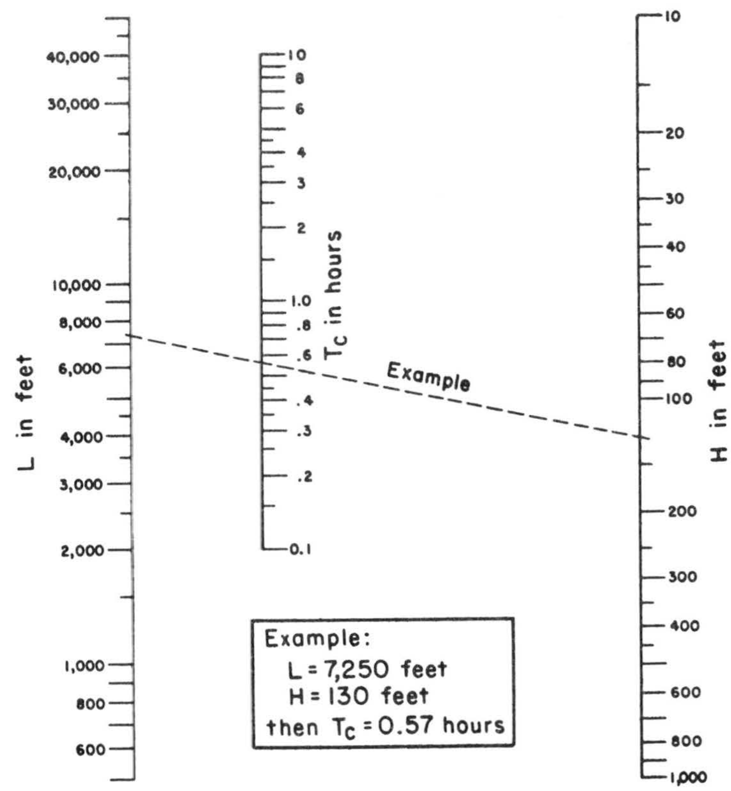


Figure 3. Time of concentration (after SCS-USDA, 1957)

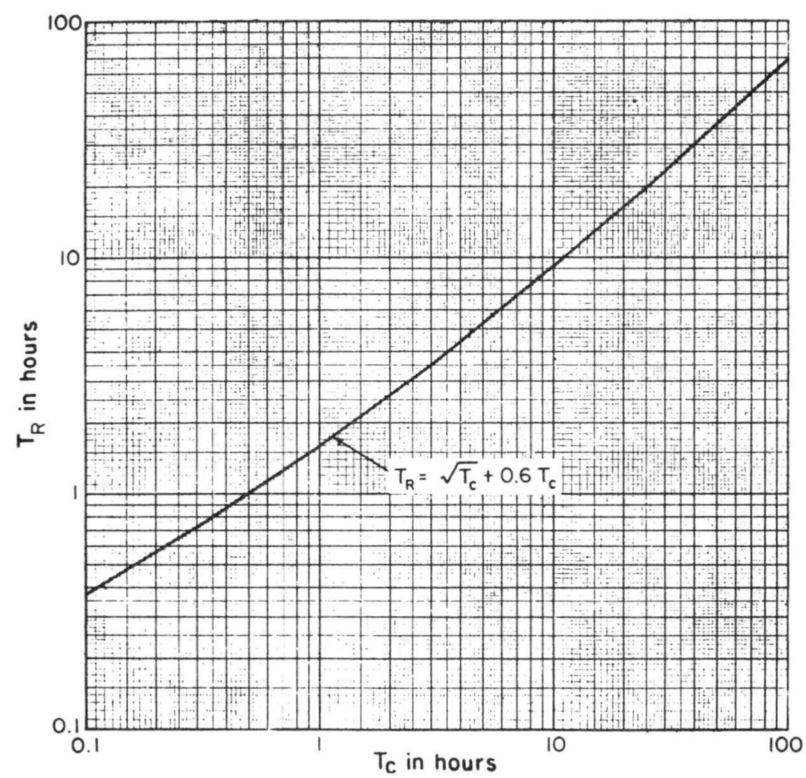


Figure 4. Time to peak (after SCS-USDA, 1957)

4. Minshall, 1960 (1)

Using three Edwardsville watersheds in Illinois with a large number of runoff periods each year, Minshall presented a method for constructing synthetic unit hydrographs for drainage areas from 20 acres up to about 500 acres and for different rainfall intensities.

Time from beginning of rainfall excess to peak rate of runoff, T_R, can be obtained from the nomograph in figure 5 for a particular size area and different rainfall intensities.

5. Gray, 1961 (15)

In 1961, Gray found in his study of lag time for drainage areas, ranging in size from 25 to 50 square miles in the central United States, that time lag could be correlated to the time period of rise of the hydrograph. From 94 selected storms he concluded that :

$$T_L = 0.996\ T_R^{1.005}$$

where T_L was defined as lag time from center of mass of rainfall to peak of runoff, in hours, and T_R was the time period of rise of the hydrograph. He stated that since the coefficient and exponent were sufficiently close to unity, they could be considered numerically equal to one and thus "a given change in T_R produces an equal change in T_L". Since the time of rise was found to be equal to the lag time, he stated that T_R could be used as an important variable in relating the hyetograph to the hydrograph.

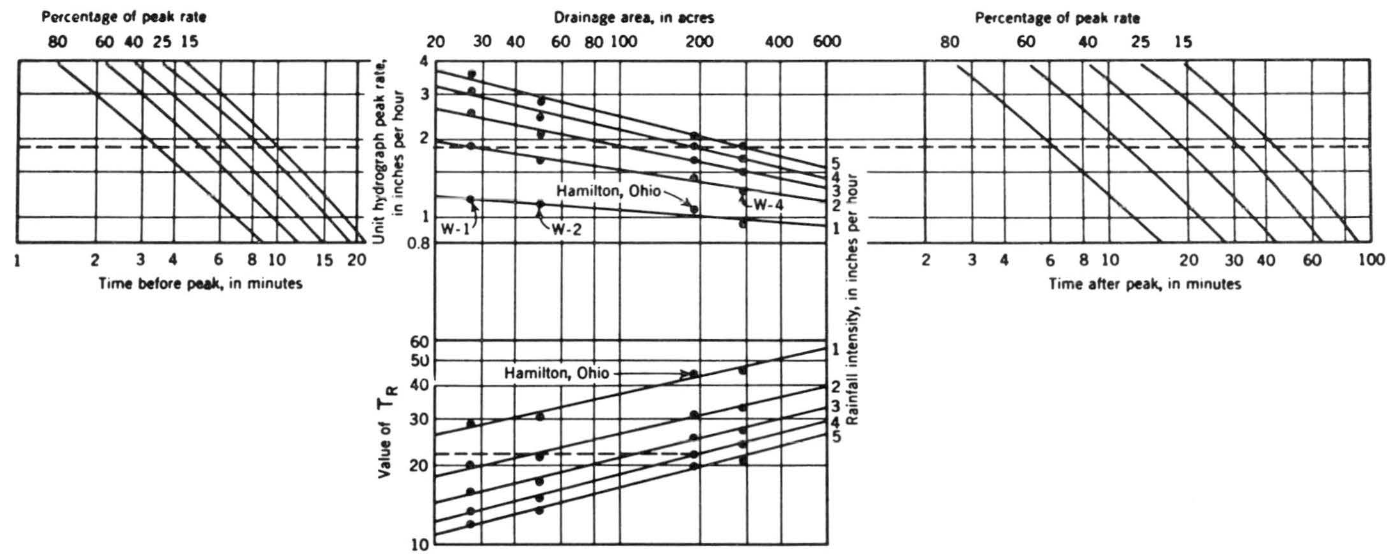


Figure 5. Effect of rainfall intensity and size of watershed on the unit hydrograph (after N. E. Minshall, 1960)

6. Keppel and Renard, 1962 (16)

In a paper by Keppel and Renard an equation for average rise time for the Walnut Gulch watersheds was given as

$$T_R = 25.3\, A_W^{-0.14}$$

where:

T_R = rise time in minutes

A_W = watershed area in square miles.

As explained in that paper, the decrease in rise time with increase of drainage area is probably due to two interrelated factors: the occurrence of most of the transmission losses during rising stages of the flow; and the presence of overriding translatory waves as the flow moves through the channel.

7. Chow, 1962 (11)

Chow, after analyzing the data from 20 small midwestern watersheds, recommended time to peak be determined by the equation:

$$T_R = 0.00236\, (L/S^{0.5})^{0.64}$$

In the foregoing L is the channel length in feet as measured up the main water course to the divide and S is the slope in per cent. The latter is determined by plotting the stream profile and fitting a straight line through the gaging station so that the area between the line and the profile lying below the line is equal to that lying above it. T_R is measured in hours, and assumed to be constant for each watershed.

8. Wu, 1963 (17)

I-Pai Wu made a study to analyze the existing data on 21 small watersheds (less than 100 square miles) distributed throughout the state of Indiana and to determine the relations between the slope of the hydrograph and some identifiable and readily obtainable watershed characteristics. It was noted that the time to peak, T_R, did not vary radically for the same watershed and hence an average time to peak was used.

He found that average time to peak, T_R, is correlated with three measurable watershed characteristics, A (watershed area), L (length of main stream), and S (mean slope of main stream).

$$T_R = 31.42\ A^{1.085}\ L^{-1.233}\ S^{-0.668}$$

Wu suggested that the design storm duration be made equal to the value of T_R. The selection of the duration of storm rainfall is based on the time of concentration of the watershed considering that a storm rainfall duration equal to the time of concentration will result in the maximum rate of discharge. Since the time of concentration of the watershed is difficult to determine, it may be assumed that the design rainfall is uniformly distributed over the entire watershed, in which case the time of concentration is equal to the time to peak.

9. Kibler, 1965 (18)

Using a total of 140 selected storms pertaining to runoff from the Shaver Creek basin, a 3.75 square mile watershed in central Pennsylvania, Kibler attempted an emperical approach. By the method of multiple regression, he developed the following equation:

$$T_R = 5.538\ D^{0.333}$$

in which T_R is defined as total time elapsed between initiation of surface runoff and time of peak discharge, in hours, and D is the storm duration in hours. This equation is considered valid for prediction of time to peak of the March-May storms with average errors of 17.9 per cent. It is, however, limited to the Shaver Creek basin.

10. Machmeier, 1966 (19)

Machmeier developed a mathematical model of a 21.35 square mile watershed with the land and channel characteristics representative of small watersheds in southeastern Minnesota. In one part of his study he attempted to show how the various time parameters are affected by supply rate and duration.

For a duration of one hour, figure 6 shows the effect of the supply rate on the time to peak. For each of the durations used, the time to peak for the model decreases as the supply rate increases.

35% reduction

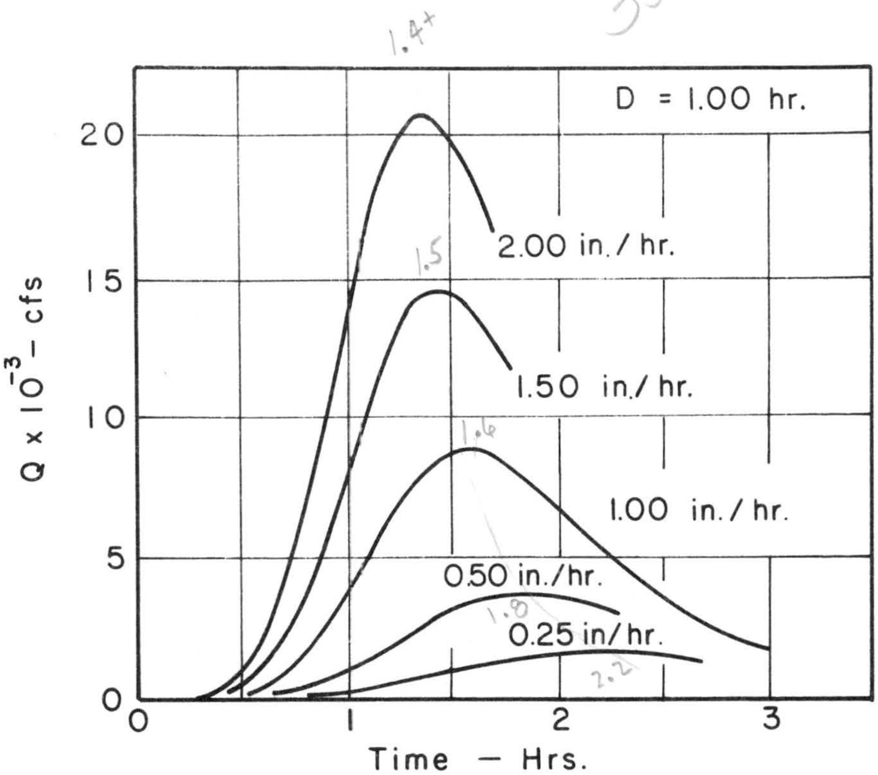


Figure 6. Model watershed hydrographs for various supply rates and a duration of 1.0 hour (after R. E. Machmeier, 1966)

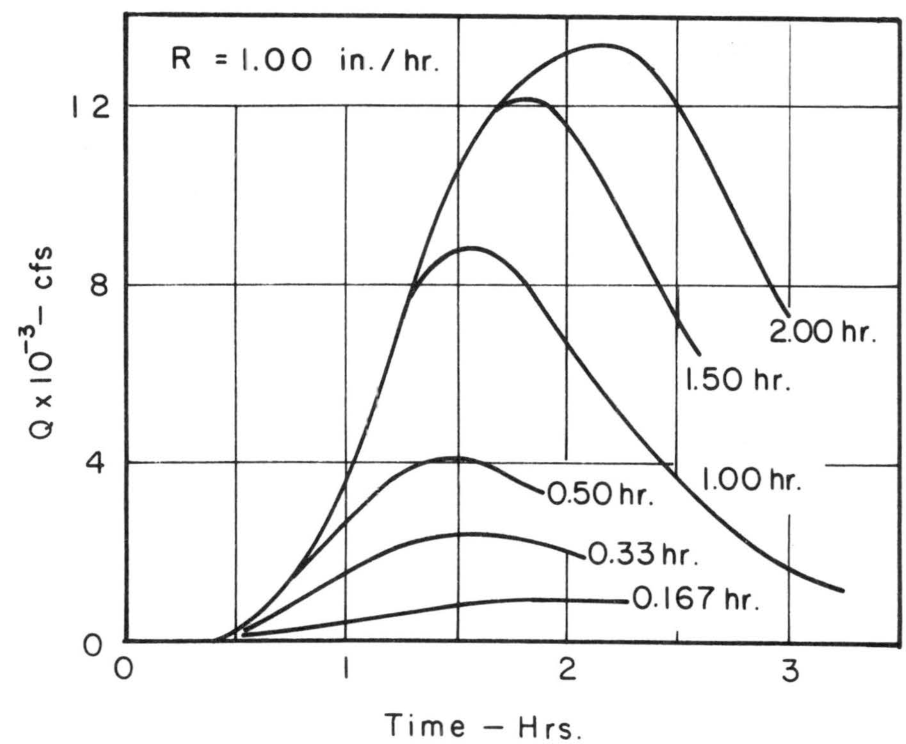


Figure 7. Model watershed hydrographs for various durations and a supply rate of 1.0 in./hr (after R. E. Machmeier, 1966)

Machmeier explained that this nonlinear response is evidently due to the fact that the average velocity in the channel system increases with discharge. According to unit hydrograph theory, the time to peak is not affected by supply rate.

The effect of duration on time to peak is shown in figure 7 for a supply rate of 1.00 inch per hour. Generally, one would expect time to peak to increase with duration. However, the results of this study indicate that time to peak has a minimum value and tends to increase both for longer and shorter durations. This same trend was exhibited for all of the supply rates tested.

All of the hydrologic type equations must remain approximations and individual watersheds will depart appreciably from mean relationships presented previously. Among the factors not easily evaluated in analyzing existing data are: the relative role of overland and channel flow, the effect of intensity and duration of rainfall, the effect of roughness on both overland and channel flow, and the variance in basin morphology as represented by such items as drainage density, channel and land slopes, and channel cross-section throughout the watershed.

CHAPTER III

ANALYSIS AND INTERPRETATION OF HYDROGRAPH RISE TIME

Inability to measure the time most desired by theoretical definition is one of many reasons for confusion in the determination of characteristic times. Rise time, sometimes referred to as time to peak, marks the occurrence of the runoff peak rate; hence, it has a definite physical significance and can be measured easily. Therefore, the rise time will be used in this study as a main-time parameter (or characteristic). When necessary, it can be related to other hydrograph characteristics for predicting the time distribution of runoff.

1. Hydrograph Characteristics

The hydrograph can be regarded as an integral expression of the physiographic and climatic characteristics that govern the relations between rainfall and runoff of a particular drainage basin (21). It shows the time distribution of runoff at the point of measurement, defining the combined effects of the rainfall excess and the complexities of the basin characteristics by a simple empirical curve.

A typical single-peaked hydrograph, figure 8, consists of three parts: the approach segment AB, the rising segment BC and the falling

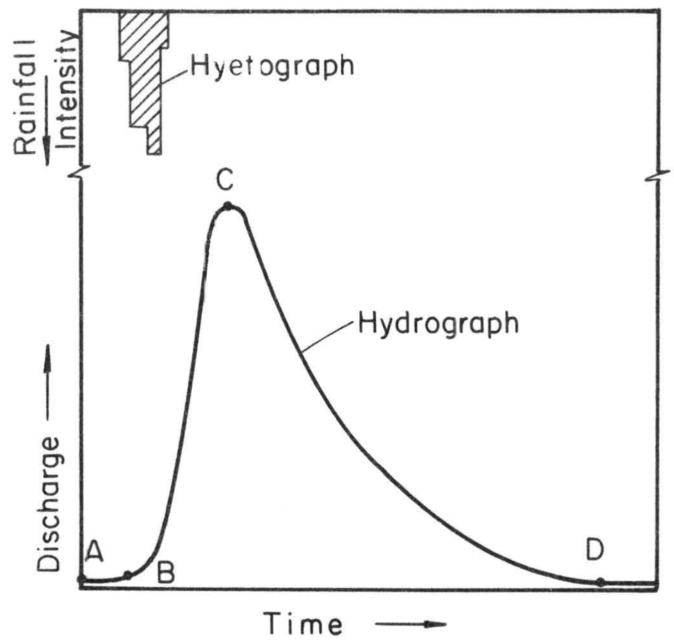
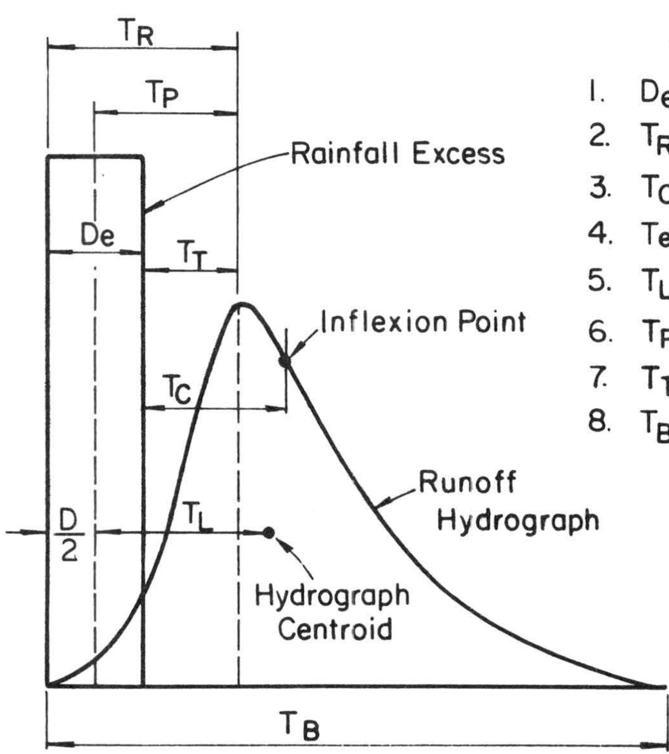


Figure 8. A Typical hydrograph with corresponding hyetograph



Time Parameters

1. D_e = Rainfall Excess Duration
2. T_R = Rise Time
3. T_C = Concentration Time
4. T_e = Equilibrium Time (Fig. 10)
5. T_L = Lag Time or Storage Delay Time
6. T_P = Basin Lag or Storage Delay
7. T_T = Translation Time
8. T_B = Base Time

Figure 9. Definition sketch of selected measures of time scale

segment CD. The approach segment represents antecedent flow before significant runoff-producing rainfall reaches the outlet. The rising segment usually commences immediately after the runoff-producing rainfall commences. It represents increasing rates of runoff caused by the inflow to channel storages from the rainfall. The peak of a hydrograph is the highest concentration of the runoff from a drainage basin. The falling segment represents withdrawal of water from storage after the main rainfall excess has finished. It is usually considered to be largely independent of the time pattern of rainfall excess.

2. Relationship Between Rise Time and Other Hydrograph Characteristics

The relationships between rise time and other hydrograph characteristics are illustrated in figure 9 where T_R is the time difference between the lowest rate of runoff q_i and the peak rate q_o of the hydrograph. It is assumed that the rainfall excess is of uniform intensity and its time of commencement corresponds with q_i.

a. Time of concentration

A most widely used term is the time of concentration, T_C, (3, 8, 14, 20, 22) defined previously in Chapter II. The Soil Conservation Service relates T_C to rise time by the following formulas:

$$\text{Rise time} = 0.5\ D_e + 0.6\ T_C$$

or

$$\text{Rise time} = T_C^{0.5} + 0.6\, T_C$$

where D_e is duration of rainfall excess.

Clark (22) measured T_C as time from end of rainfall excess to point of inflexion on the recession segment of the hydrograph (figure 9). Hence, his relationship between the rise time and time of concentration becomes

$$\text{Rise time} = D_e + T_C \quad .$$

b. <u>Time of equilibrium,</u> T_e (figure 10)

For uniform rainfall intensity, the time of concentration would be the time of equilibrium at which the rate of runoff is equal to the rate of rainfall supply (21). Theoretically, this is the maximum possible rise time. Since rainfall excess does not occur uniformly for any extended period of time, equilibrium flows are seldom encountered. However, for small areas where channel storage can be neglected and the conditions approach two-dimensional overflow (unit width concept) a time of equilibrium can result. Izzard (23) showed that time of equilibrium is a function of channel length, rainfall intensity, and slope and land surface. Morgali (24) cited experimental verification of the time distribution of overland flow based on a finite difference solution by computer of the continuity and momentum equations where boundary conditions include zero flow at upstream end, sub-critical flow at downstream exit, and uniformly added rainfall excess along the channel slope. The time of equilibrium is represented as follows:

$$T_e = C \frac{L^{0.593} n^{0.605}}{i^{0.388} S_o^{0.38}}$$

where T_e = time of equilibrium

C = a constant

L = length of overland flow

n = Manning roughness coefficient

i = intensity of rainfall excess

S_o = slope of surface.

c. Lag time parameters

The best measure of basin lag would be T_L, the time from centroid of rainfall excess to centroid of the resulting runoff hydrograph (figure 9), but usually this value cannot be calculated conveniently (25). In view of this difficulty, various alternative concepts have been introduced. Snyder (26) suggested that basin lag could be represented by T_P, the time from centroid of rainfall to hydrograph peak. Clark (22) utilized T_C, time of concentration, as a measure of maximum lag time in the basin. Clark's definition, in a physical sense, is analogous to the time of concentration of the rational formula but is independent of time of rainfall. Gray (15) found that lag time, T_P, is equal to rise time T_R.

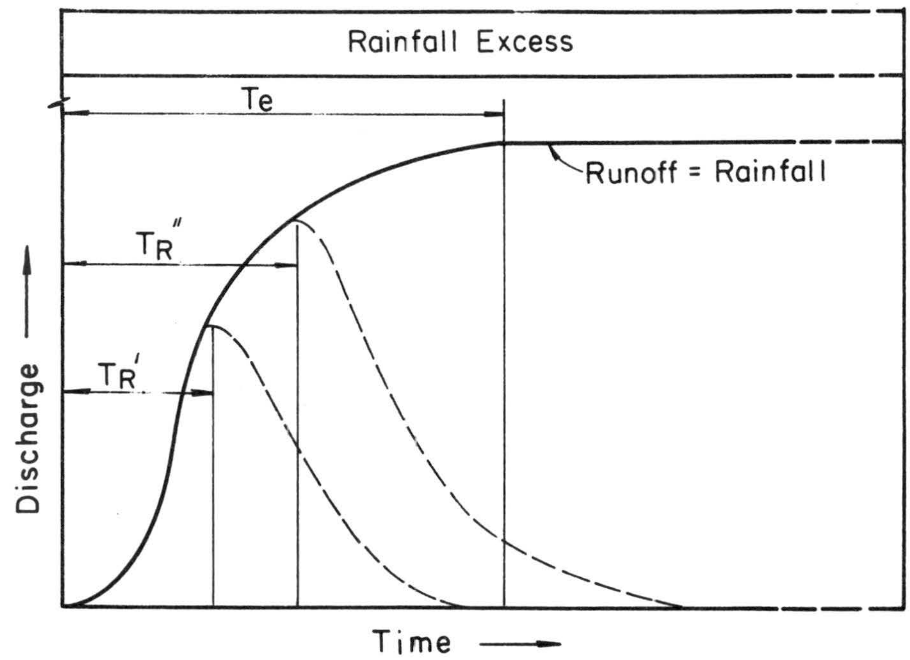


Figure 10. Definition sketch of time to equilibrium

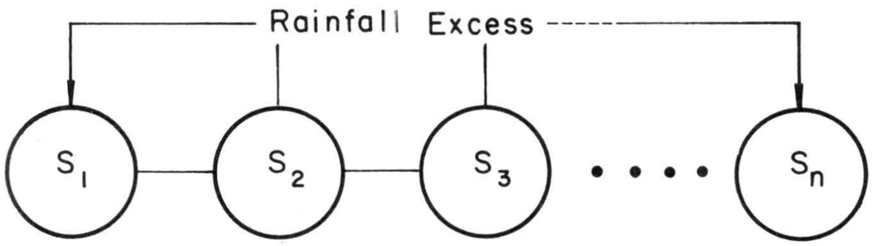


Figure 11. Definition sketch of storages defined by Laurenson

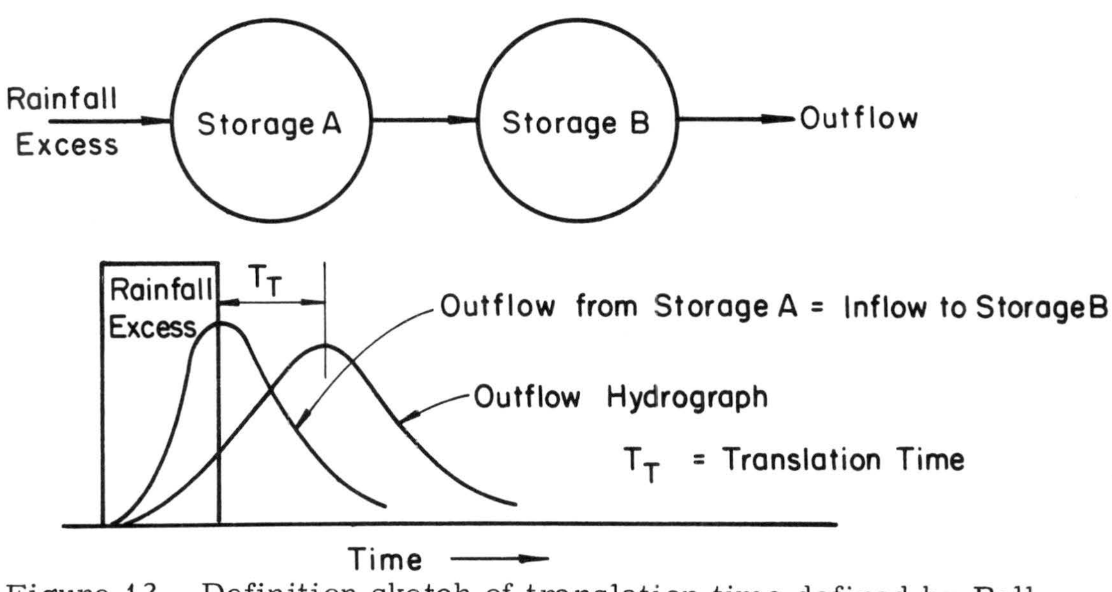


Figure 12. Definition sketch of translation time defined by Bell

Laurenson (27) suggested that a watershed behaves like a number of concentrated storages acting in series (figure 11). Each storage is expressed by

$$S_n = K_n q_n$$

where

S_n = storage volume

q_n = outflow.

He showed that K has the dimensions of time and is equal to the time between centroid of inflow hydrograph and centroid of outflow hydrograph if the storages are linear. When the watershed is treated as a whole, the time, T_L, between the centroid of rainfall excess and the centroid of hydrograph at the outlet is given by:

$$K = K_1 + K_2 + K_3 + \ldots . K_n$$

He suggested that this be regarded as the "average storage delay time" of the watershed.

Bell (28) used two concentrated non-linear storages to simulate streamflow behavior as shown in figure 12. The translation time, T_T, is the time between the peaks of the inflow and outflow hydrographs of storage B. In practice it is derived from the time, T_T, between the end of the main excess rainfall and the peak of the outflow hydrograph (figure 9). It is used for routing the flow through storage B, i.e.

$$q = \frac{S_B}{T_T}$$

where

q = outflow

S_B = volume of B storage

$T_T = f(q)$ = average translation time from q_o to q.

The rise time is therefore equal to duration of rainfall excess plus translation time.

d. Base time

The base time T_B is the base length of the hydrograph (figure 9). In the triangular hydrograph method (3) by the Soil Conservation Service, T_B is related to T_R by the following formula:

$$T_B = 2.67\ T_R$$

where

$$T_R = 0.5\ D_e + 0.6\ T_C.$$

3. Rise Time and the Unit Hydrograph Concept

The unit hydrograph was originated in 1932 by Sherman (7) and has since become widely accepted as a method of predicting extreme floods, given the volume of runoff. Although intended for streams having at least a few years of stream flow records, the method has been extended to ungaged watersheds by means of "synthetic unit hydrographs" and "dimensionless hydrograph" techniques. At the same time, several researchers have extended and formalized the theory mathematically through the use of the "instantaneous unit hydrograph". These developments have been summarized by Linsley et al., (25).

All of these techniques depend on the basic unit hydrograph assumption that the discharge at any time is proportional to the volume of runoff. It follows that unit hydrograph rise time and other runoff time parameters are constant for a given watershed or, in other words, are unaffected by the magnitude of the runoff. The principle of superposition also follows the assumption that, if "excess rainfall" or runoff supply is divided into segments, the unit hydrograph method can be applied to each segment separately and the resulting hydrographs can be added together. Therefore, the rise time of the summation-curve varies only with the storm duration. Since these are characteristics of a linear system, the unit hydrograph is, therefore, based on assumptions of linearity.

In recent years, these assumptions have been questioned. Knowing that the velocities of flowing water and wave celerities in a natural channel usually increase with discharge, one intuitively expects rise time and other time parameters to be affected somewhat by discharge. Studies with laboratory models by Amorocho (29) and Pabst (30) indicate that non-linearity does exist. An analysis of field data by Laurenson (27) showed that lag time varies with discharge, indicating non-linearity. Using a mathematical watershed model Machmeier (19) showed that rise time and other time parameters presently defined should not be considered constant for a given watershed.

4. Variability of Rise Time Within Watersheds

Some studies imply that rise time within watersheds is approximately constant but the previous analysis shows that this is difficult to justify. Unit hydrograph rise times, concentration times, equilibrium times, lag times and translation times are all approximately constant, but the rise time depends on the duration of rainfall excess which varies from storm to storm according to the type of rainfall, antecedent moisture and similar factors. Kibler's study, discussed in the literature review, showed a direct relationship between rise time and rainfall duration.

Rise time is probably also affected by the areal distribution of rainfall. If the storm is centered near the outlet, the time of rise should be shorter than if the storm is centered near the headwaters.

Since there are no previous studies of the range of rise times that might be expected within any particular watershed, it appears desirable that this matter be examined in further detail.

5. Variability of Rise Time Between Watersheds

Watershed characteristics are sources of variability of rise time between watersheds as indicated by studies of basin lag, time of concentration and the other approximately constant factors. Variability in rise time between watersheds can also be studied directly by selecting representative rise times such as average or median. This will be attempted in the next chapters.

6. Conclusions from the Analysis

The variability of rise times within watersheds needs to be analysed with a large amount of data. If it can be shown that a general distribution of values applies to most watersheds, this would provide a basis for the following additional studies:

1. Selection of a parameter or representative value for comparing rise times between watersheds.
2. Derivation of a formula for estimating the representative rise time from watershed characteristics.
3. Evaluation of other methods of estimating rise time and similar characteristic times.
4. Possible use of rise time for estimating design floods.

These will be attempted in the remainder of the investigation.

CHAPTER IV

PRELIMINARY EXAMINATION OF DATA

1. The Data Available

This study is based on hydrologic data assembled under the auspices of the "Small Watershed Program" at Colorado State University. All watersheds with five or more events were selected, thus giving a total of 47 watersheds and 407 flood events. The distribution of watersheds according to their geographic locations by state is shown in table 1. It is noted that most of the watersheds are from 12 western and mid-western states representing diversified regions of the drier parts of the United States.

A total of 22 watershed characteristics for each watershed was taken out of reference (48). The method of computing these watershed characteristics as defined by Laurenson et al., (31) is shown in Appendix A.

Six major rainfall and runoff parameters are recorded in table B-2 of Appendix B. These are:

(a) antecedent precipitation index, API,

(b) flood peak, q,

(c) flood frequency, P_F,

TABLE 1. DISTRIBUTION OF WATERSHEDS AND FLOOD EVENTS BY STATES

Number	States	Number of Watersheds	Number of Events
1	Arizona	8	49
2	California	6	33
3	Colorado	1	5
4	Illinois	1	5
5	Iowa	1	5
6	Mississippi	6	30
7	Nebraska	3	17
8	New Mexico	1	7
9	Ohio	7	44
10	Oklahoma	2	14
11	Pennsylvania	1	140
12	Texas	8	47
13	Wisconsin	2	11
	TOTAL	47	407

(d) total precipitation, P_s,

(e) storm duration, D,

(f) rainfall excess duration, D_e.

2. Definitions of Rise Time

Two definitions were selected (figures 13 and 14):

1. T_{R1} is the time from lowest discharge q_i to peak q_o of the runoff hydrograph.

2. T_{R2} is the time from discharge $0.05\,(q_o - q_i) + q_i$ to peak q_o of the runoff hydrograph.

T_{R1} is usually implied by the term "rise time" but sometimes this value is a doubtful representation of the main period of rise e.g., when there is a long hydrograph segment of approximately constant or slightly increasing discharge. In such cases, it is also difficult to decide when the rise commences.

T_{R2} is intended to avoid the above difficulties. It is essentially a measure of the significant rise time. Both values have been determined from the runoff data and are recorded in table B-2 of Appendix B.

3. Correlation Between the Two Measures of Rise Time

Figure 15 shows the plots of T_{R1} against T_{R2}. For measuring the degree of association of the two rise times, the correlation coefficient was computed. It is equal to 0.90.

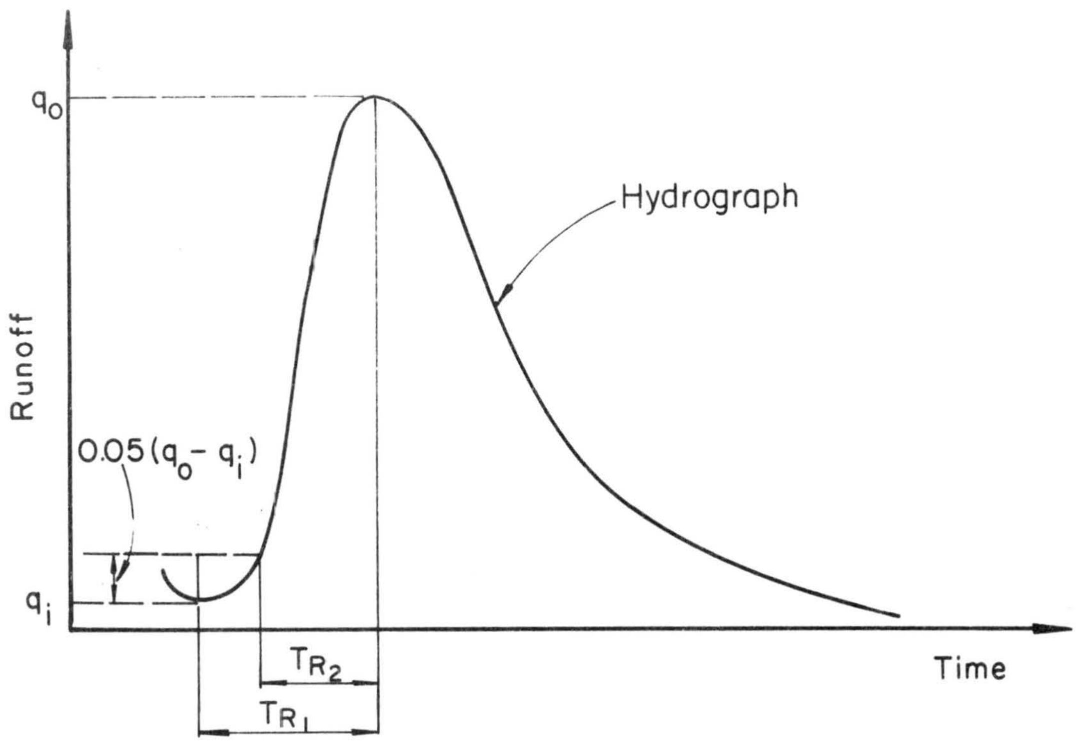


Figure 13. Rise time definitions sketch no. 1

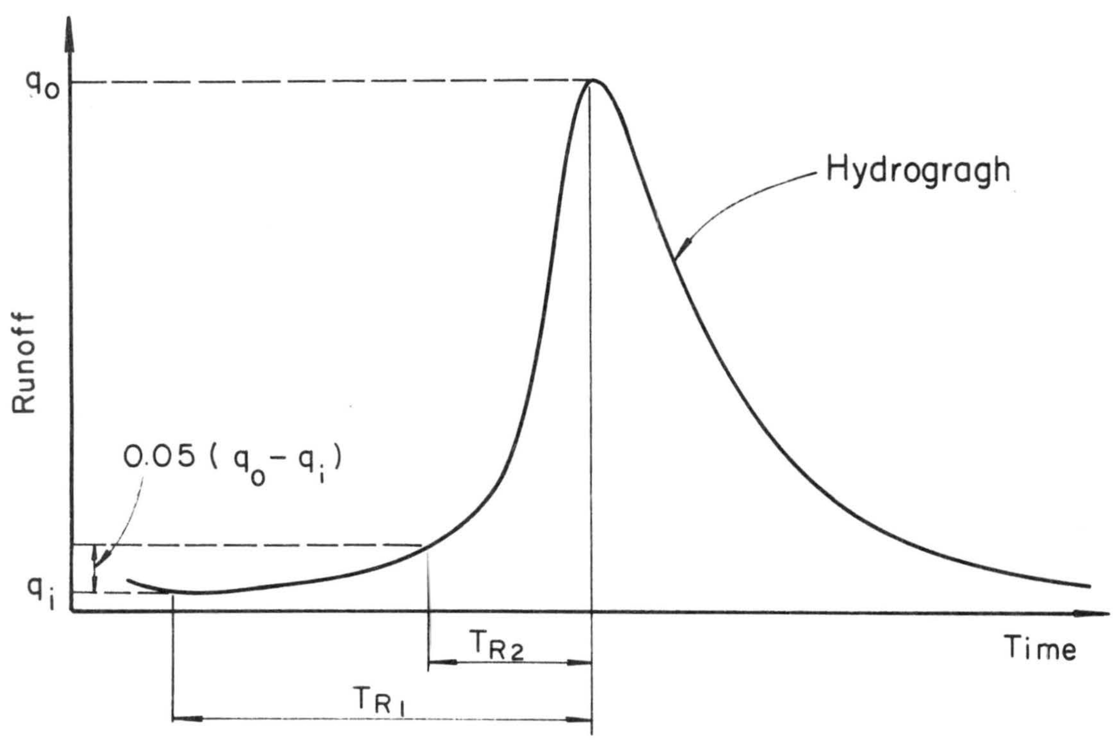


Figure 14. Rise time definitions sketch no. 2

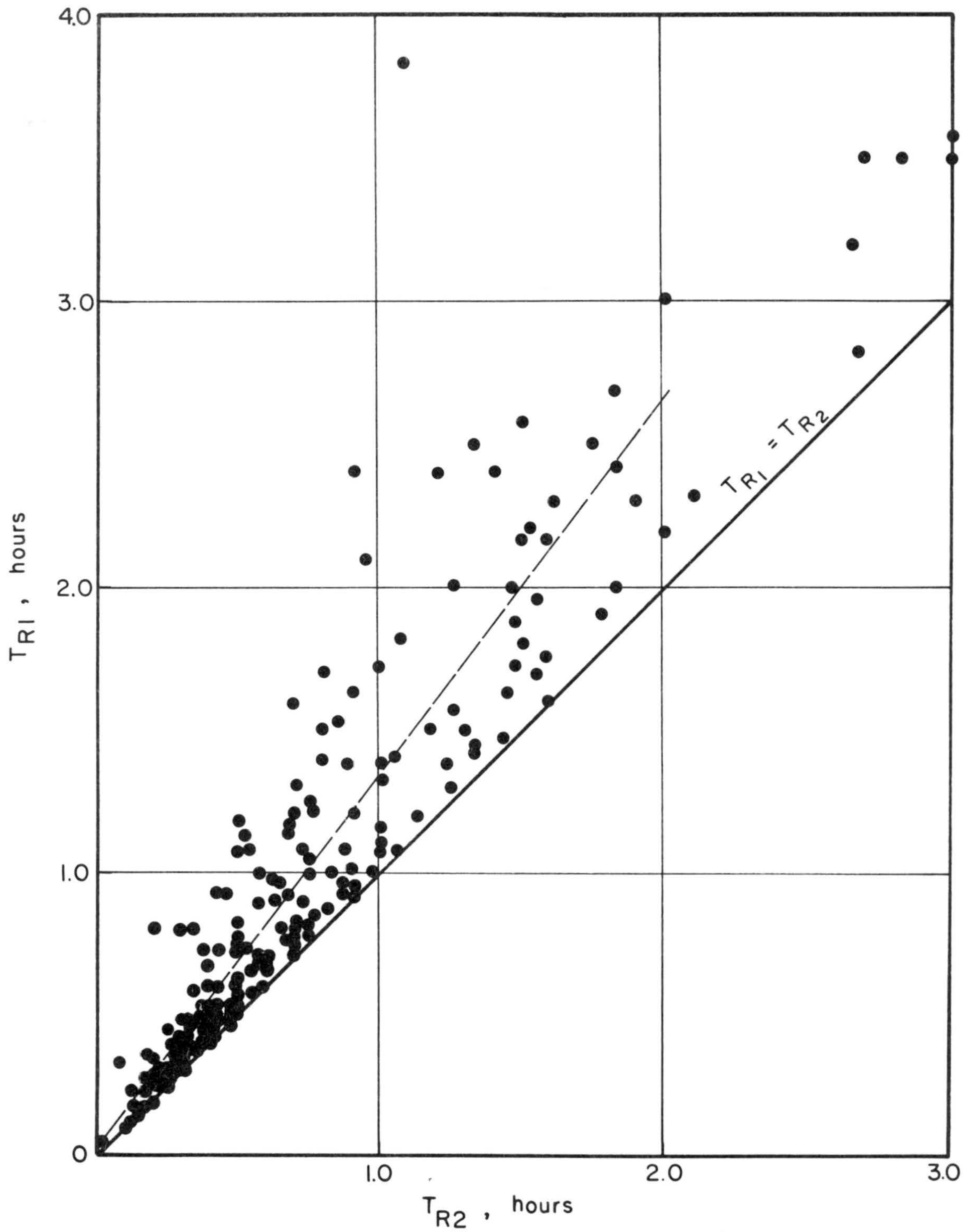


Figure 15. Correlation between T_{R1} and T_{R2}

4. Correlations of Rise Time with Rainfall and Runoff Parameters

A preliminary analysis of the data was made with simple graphical correlations of rise time with rainfall and runoff parameters.

For each watershed, T_{R1} and T_{R2} were plotted against several types of API, q, P_s, and D. No significant relationships were obtained (figure 16). Most of the watersheds have only five events, hence, most of the graphs have only five points.

With an attempt to combine all events together, the rise time was converted to the ratio of the actual rise time to the mean rise time of each watershed.

The ratios $T_{R1}/\overline{T}_{R1}$ and $T_{R2}/\overline{T}_{R2}$ were plotted against API. The drainage basin area was used as a basis to group the watersheds together in the plotting. No significant relationship was obtained (figure 17).

5. Conclusions from the Preliminary Study

T_{R1} and T_{R2} are highly correlated with each other. Therefore, T_{R1} will be the only measure of rise time considered in the next chapter.

Due to a lack of sufficient storm data pertaining to individual watersheds, the variability of rise times within a watershed cannot be formulated in complete detail.

In an effort to overcome this difficulty and thereby obtain a general picture of rise time variability, it was decided that further attempts should be made to combine the data from different watersheds.

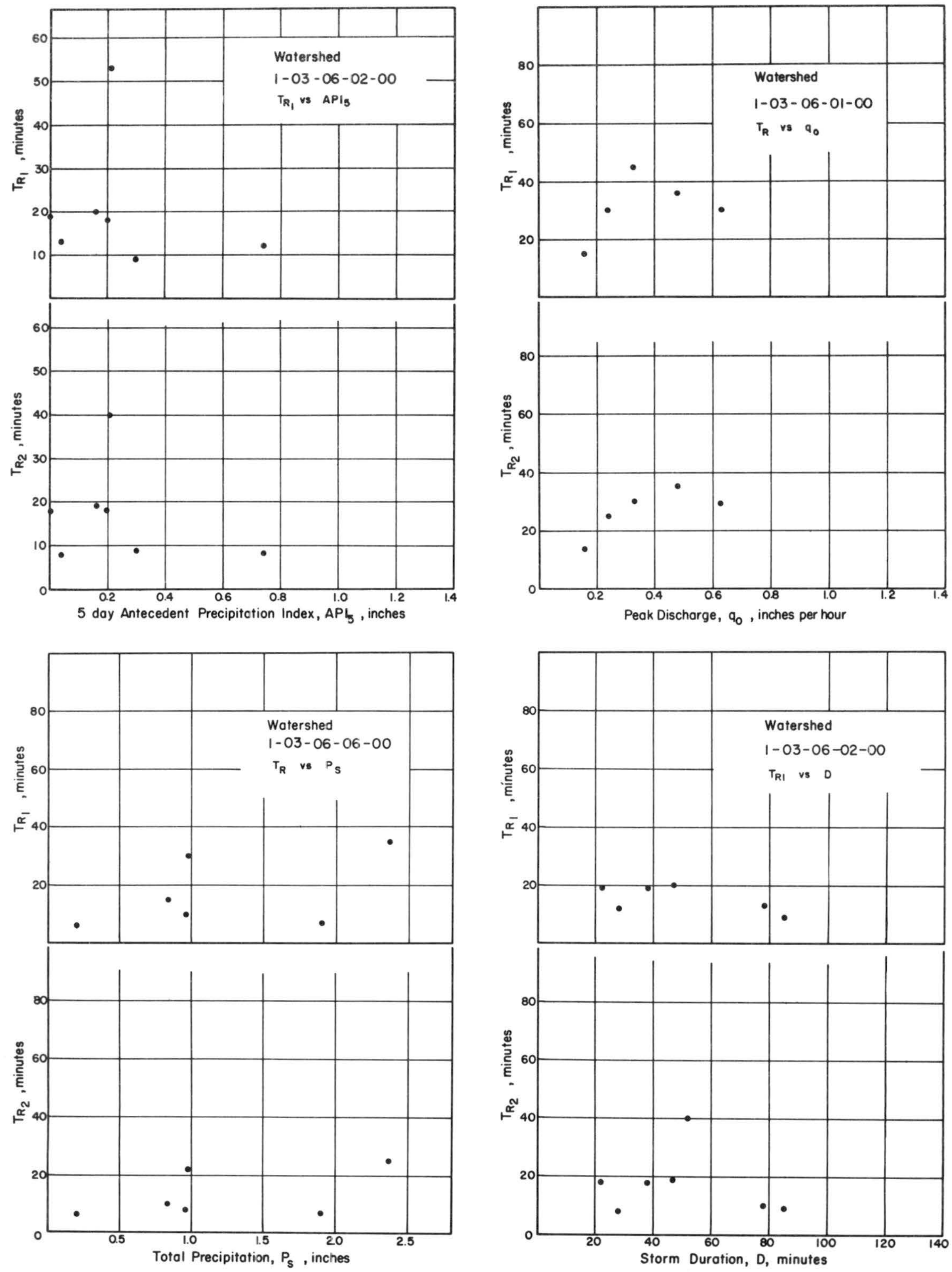


Figure 16. Correlation of rise time with API, q_o, P_s and D

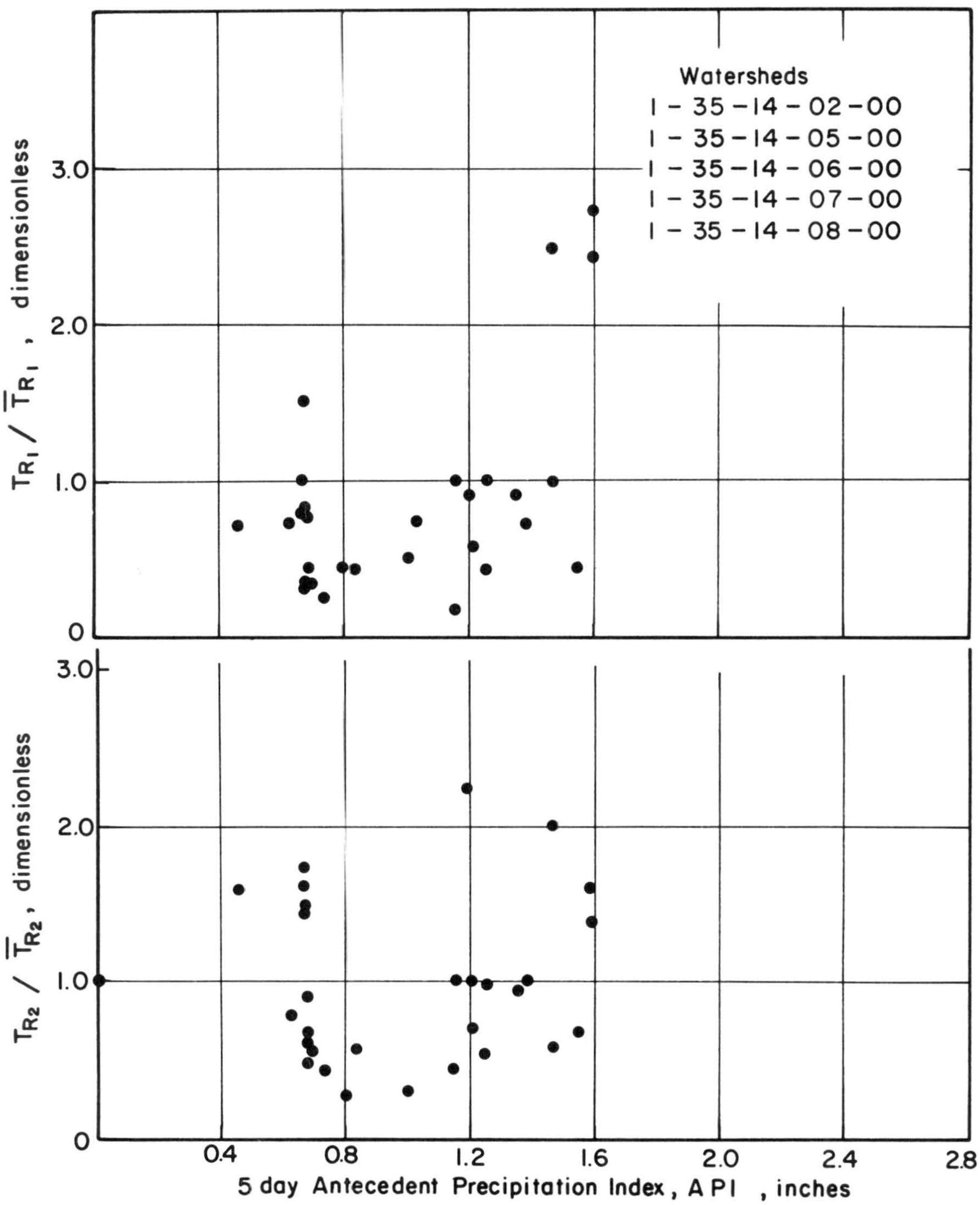


Figure 17. Five-day API versus $T_R / \overline{T}_R$

CHAPTER V

THE CONCEPT OF RELATIVE RISE TIME AND ITS DISTRIBUTION

The foregoing chapter showed a need for further attempts to combine data of the preliminary study to get a better picture of variability of rise times within watersheds. Before this can be done, the data has to be transformed so that all watershed values are comparable and may be considered as samples of a single population of values.

1. Average Rise Time

First attempt at doing this was to estimate the average rise time for all watersheds with five or more events by calculating the arithmetic mean and express actual rise times as ratios to the average, thus putting them in dimensionless form.

It was also considered that the arithmetic average rise time would be a good representative value of rise time to enable analyses of variability between watersheds.

Much of the data was analysed according to these ideas, but it soon became apparent that the arithmetic average rise time could not be accurately estimated from the small samples of some watersheds because it was unduly affected by extreme values. An attempt was made to reduce this effect by initially eliminating rise time ratios less than 0.5 and greater than 2.0 and recalculating the means. The ratios of values were then recalculated using these revised estimates of the average. This problem was later referred to Dr. Siddiqui (32), and he recommended that the median be used rather than the arithmetic mean. In small samples the median is less affected by occasional extreme values.

2. Median Rise Time

Median rise times were calculated for all watersheds as shown in table B-1 of Appendix B. This parameter was adopted as the representative value of rise time for each watershed and also as a base value for converting the actual rise times to dimensionless relative rise times.

3. Relative Rise Times

"Relative Rise Time" is defined here as the ratio of actual rise time to median rise time. The relative rise times were calculated for all events as in tables B-2 and B-3 of Appendix B, and combined to give frequency distributions for:

(a) all watersheds,

(b) a single watershed (Shaver Creek),

(c) watersheds smaller than one square mile, and

(d) watersheds larger than one square mile.

These are plotted in figure 18 which indicates similar distributions with positive skew for each.

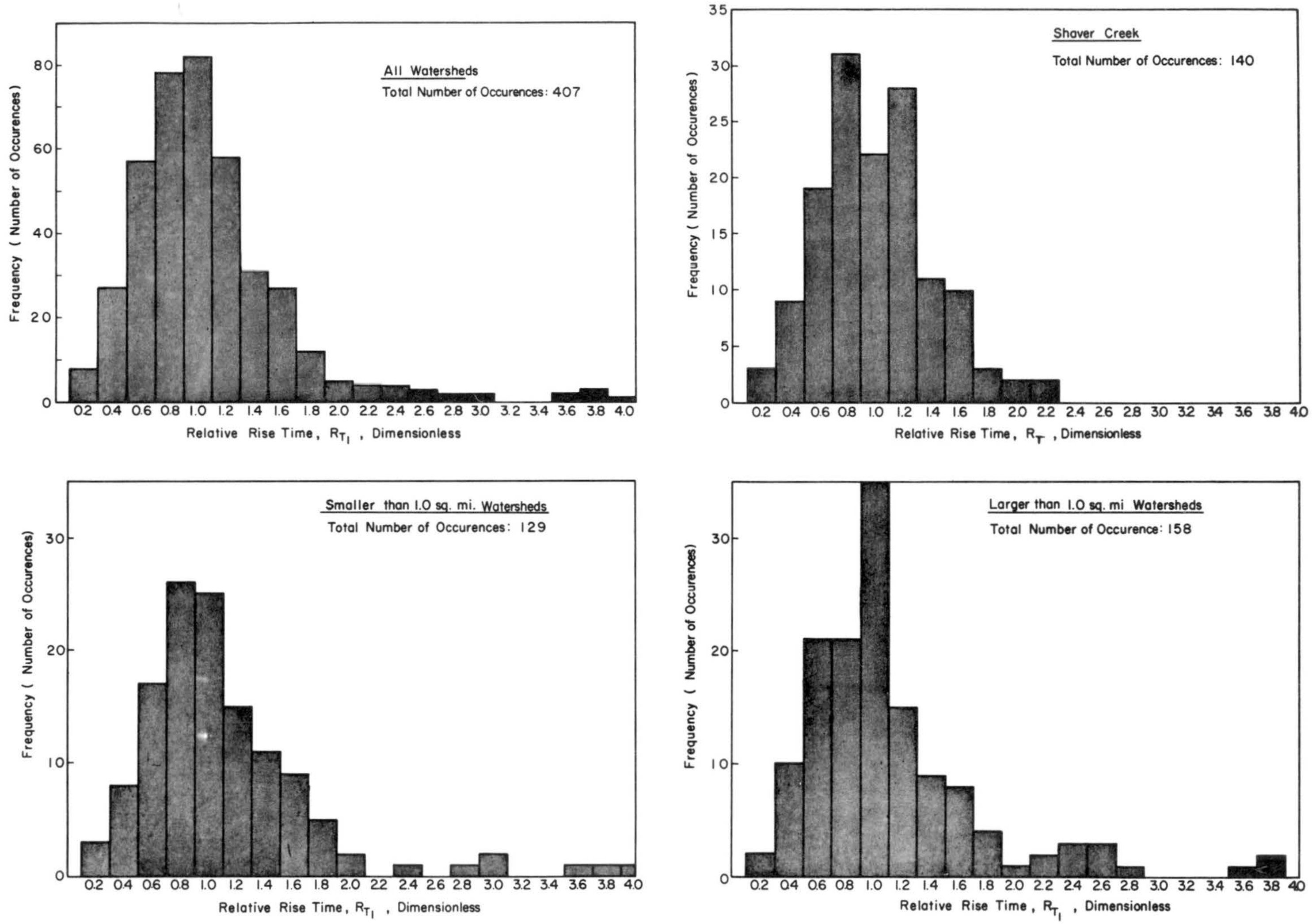


Figure 18. Histograms for frequency distribution of relative rise times

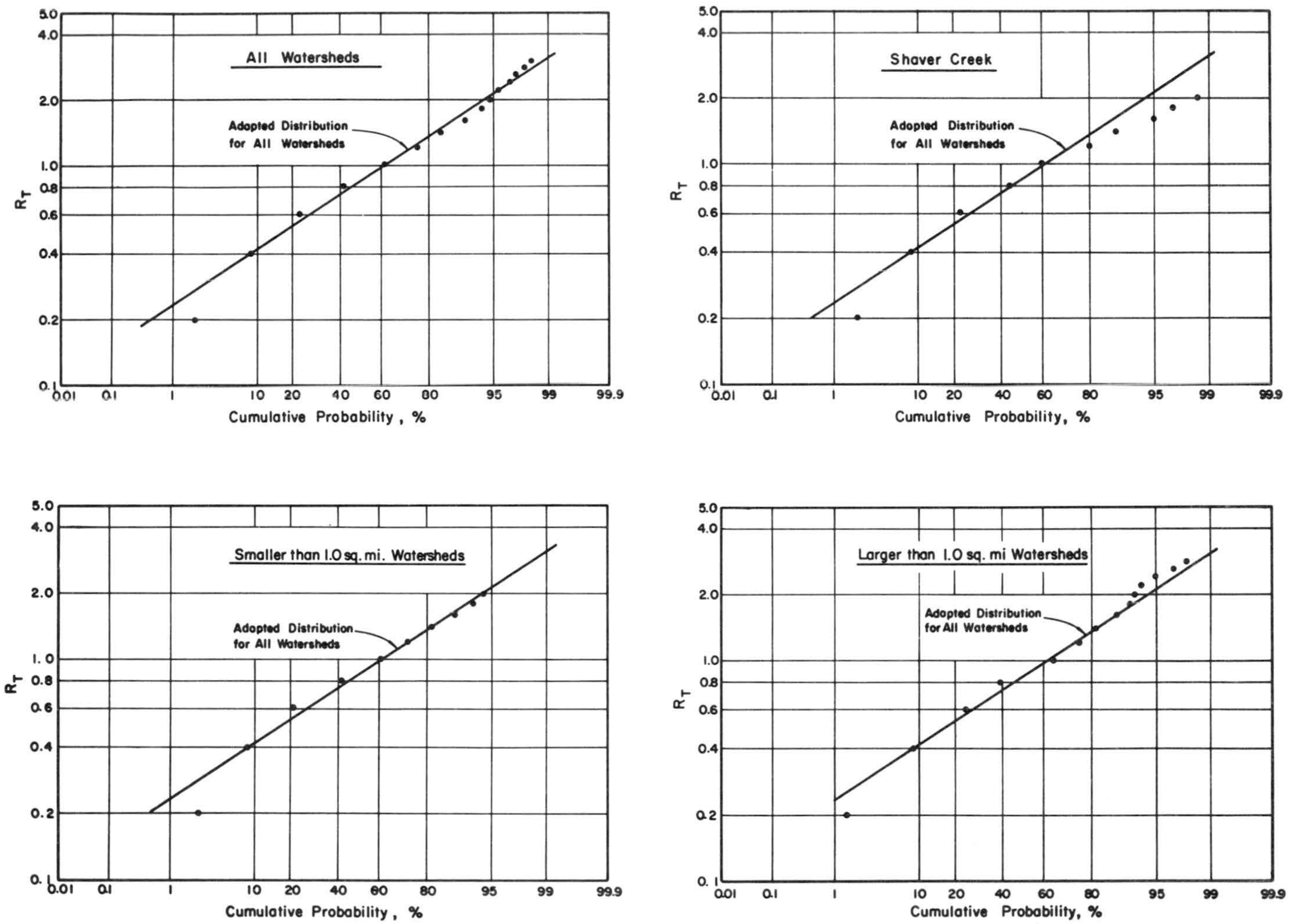


Figure 19. Cumulative probability distribution of relative rise times (separate plottings of each sample)

The cumulative frequency values were found to be closely linear when plotted on logarithmic probability paper, indicating that a logarithmic transformation is suitable for normalizing the distributions (figure 19).

To test the significance of differences between the distributions, a Chi-square (χ^2) test was applied with an adopted level of significance of five per cent. This is described in section 4 and implies the following hypothesis:

Hypothesis - The frequency distribution obtained from the combined records of all watersheds expresses the variability of rise times for any single watershed.

4. Chi-square Test of Hypothesis

The Chi-square test is summarized as follows: The total range of sample observations is divided into k class intervals, each having the observed class frequency O_j and corresponding expected class probability E_j ($j = 1, 2, \ldots, k$). The measure of total discrepancy between observations and expected values, X^2, becomes

$$X^2 = \sum_{j=1}^{k} \frac{\left(O_j - E_j\right)^2}{E_j}$$

This statistic is distributed asymptotically as Chi-square with ($k-1$) degrees of freedom (33).

a. First test of hypothesis: single watershed

The distribution of relative rise times of a single watershed, Shaver Creek, was tested against the distribution of relative rise times obtained from the combined records of all 47 watersheds. The computations are shown in table B-5 of Appendix B.

Calculated $X^2 = 12.48$.

For ten degrees of freedom: $X^2_{0.95} = 18.3$ and $X^2_{0.05} = 3.94$. Therefore, the difference is not significant.

b. Second test of hypothesis: smaller than one square mile watersheds

The 46 watersheds, excluding Shaver Creek, were separated into two groups of 23 watersheds by the size of area for further testing of the hypothesis. In this section, the first group, smaller than one square mile watersheds, is tested. The computations are shown in table B-6 of Appendix B.

Calculated $X^2 = 13.75$.

For 20 degrees of freedom: $X^2_{0.95} = 31.4$ and $X^2_{0.05} = 10.9$. Therefore, the difference is not significant.

c. Third test of hypothesis: larger than one square mile watersheds

The second group, larger than one square mile watersheds, is tested here. The computations are shown in table B-7 of Appendix B.

Calculated $X^2 = 21.53$.

For 18 degrees of freedom: $X^2_{0.95} = 28.9$ and $X^2_{0.05} = 9.39$. Therefore, the difference is not significant.

5. <u>Theoretical Distribution of Relative Rise Times in a Watershed</u>

Since a logarithmic transformation has been found suitable for normalizing the distribution, it is advantageous to derive its specific mathematical form.

The expression for the log-normal density function with two parameters is of the form (33):

$$p(x) = \frac{1}{x\,\sigma_n \sqrt{2\pi}}\, e^{-\frac{(\ln x - \mu_n)^2}{2\sigma_n^2}}$$

where

$$\mu_n = \frac{\sum_{i=1}^{N} \ln x_i}{N}$$

and

$$\sigma_n = \sqrt{\frac{\sum_{i=1}^{N} (\ln x_i - \mu_n)^2}{N}}$$

Using the graphical method, figure 20, $\overline{X}_n$ and S_n were obtained:

$$\overline{X}_n \approx \mu_n = \ln 0.85 = -0.16$$

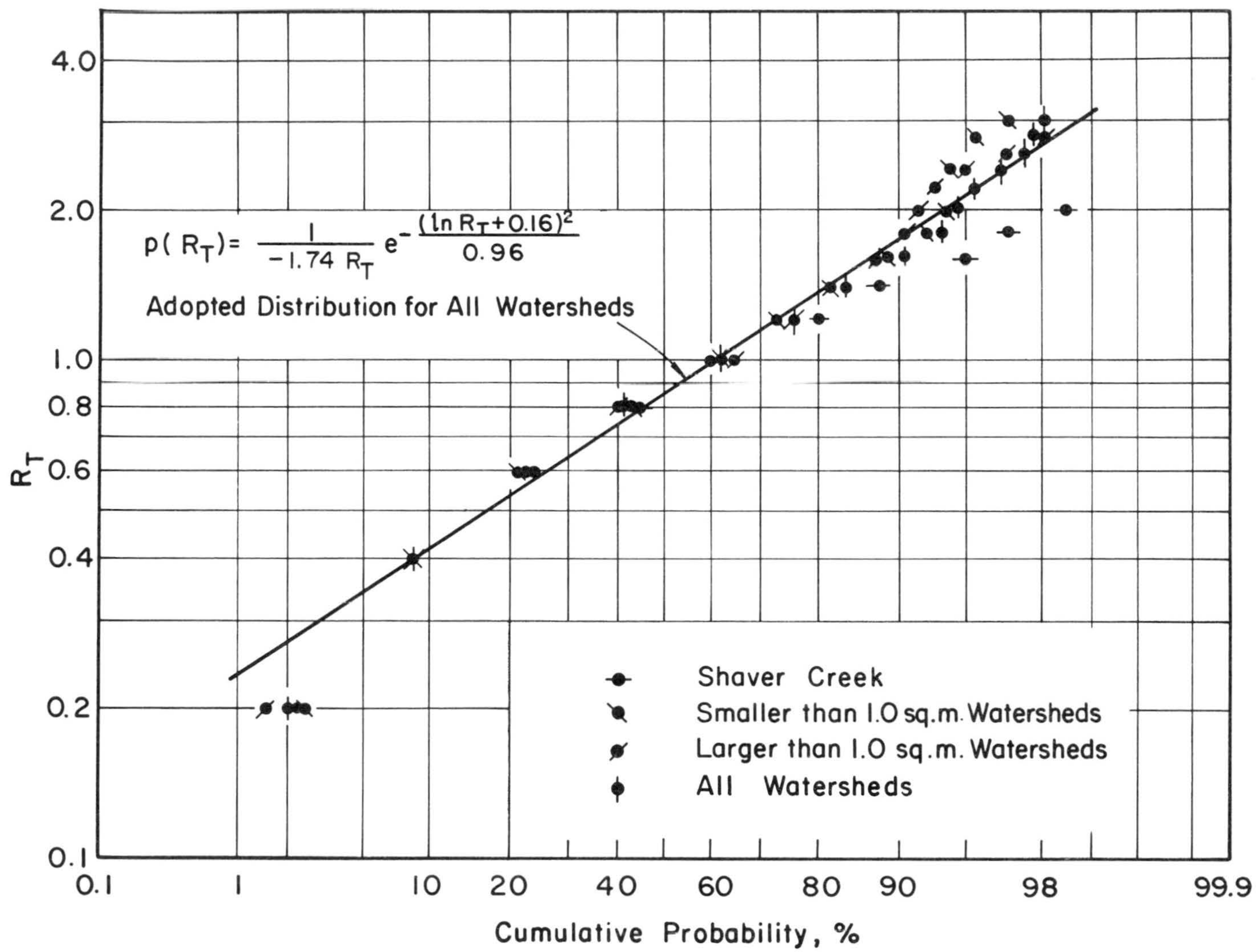


Figure 20. Cumulative probability distribution of relative rise times (combined plottings of the four samples)

$$S_n \approx \sigma_n = \ln 0.50 = -0.69$$

which gives the following equation:

$$p(R_T) = \frac{1}{-1.74\, R_T}\, e^{-\frac{(\ln R_T + 0.16)^2}{0.96}}$$

This is the theoretical distribution of relative rise times for all the watersheds studied.

6. Conclusions and Discussion of the Concept

Rise time within a watershed, far from being constant, varies from about 0.3 to 3 times the median or more.

A single distribution of relative rise times applies to all watersheds investigated. This has the following mathematical form:

$$p(R_T) = \frac{1}{-1.74\, R_T}\, e^{-\frac{(\ln R_T + 0.16)^2}{0.96}}$$

where R_T is the relative rise time.

From the hydrologic point of view, the rise time or time to peak of the hydrograph of runoff from a drainage basin is influenced by two major groups of factors:

(a) factors that are relatively constant for a particular watershed such as area, slope and drainage density;

(b) factors that vary with time for a particular watershed such as precipitation intensity, precipitation duration and antecedent moisture.

The distribution of relative rise times reflects variations due to group (b). These factors exhibit seasonal changes in accordance with the climatic environment and also random differences from storm to storm. However, their effects on the rise time appear to be consistant for a wide range of conditions when expressed in the form of the distribution of relative rise times.

There may be systematic differences in the distribution due to geographical location, climatic factors and watershed characteristics but this study has not been able to detect any such differences with the available sample of data.

The median rise time represents the effects of the relatively constant factors in group (a) and will be studied in the next chapter.

CHAPTER VI

PREDICTION OF MEDIAN RISE TIME

Several methods have been suggested for predicting rise time or time to peak, as cited in the literature review. The best known and most widely used in American engineering practice is the Soil Conservation Service Method (9). So, it is considered worthwhile to examine the prediction by this method before deriving other formulas expressing the relationship between rise time and watershed characteristics.

1. Evaluation of SCS Method for Computing Rise Time

The SCS mehtod for computing rise time or time to peak was summarized in section 3 of Chapter II. Estimated time of concentration, T_C, and rise time, T_R, for each watershed are shown in table B-1 of Appendix B.

a. Time to peak (see tables B-8 and B-9 - Appendix B)

Times to peak were plotted against median rise times (figure 21). A regression line of best fit by least squares was obtained and has an equation as follows:

$$T_M = 0.656 + 0.142\ T_R$$

where T_M is the observed median rise time and T_R time to peak by SCS method. The coefficient of determination, R^2, is 0.088. The

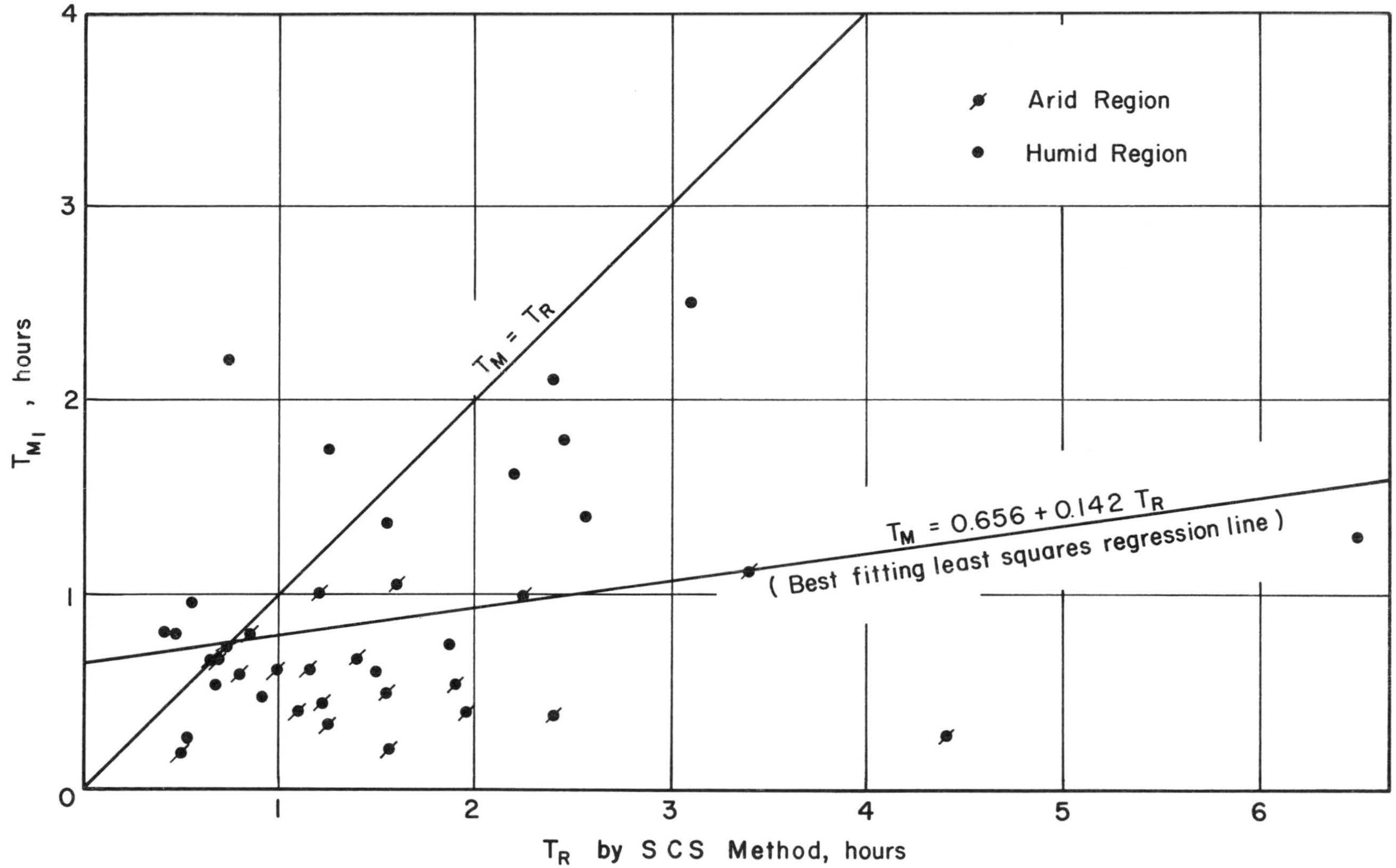


Figure 21. Correlation between median rise time and time to peak of SCS method

average error of estimate is 145 per cent. This seems to agree also with the finding from the tests done by the Hydrological Research Unit of the University of Witwatersrand, Republic of South Africa (46).

b. Time of concentration (see tables B-8 and B-9 - Appendix B)

Times of concentration were also plotted against median rise times (figure 22). The average error of estimate of median rise times by times of concentration by SCS method is 103 per cent. It is an improvement compared to the estimation of median rise time by time to peak.

c. Discussion

From the literature review it was noted that the SCS method is based on the study of Kirpirch who used the data collected by Ramser on six agricultural watersheds ranging in area from 1.25 to 112.0 acres, near Jackson, Tennessee.

The improvement of estimation of the median rise time by the time of concentration is understandable because Ramser actually measured the time of concentration by the rise time which is the time required for the water in the channel at the gauging station to rise from the low to the maximum stage as recorded by the water-stage recorders. The time of concentration calculated by SCS method is, therefore, the average rise time although this does not appear to be realized by the SCS and other users of the method.

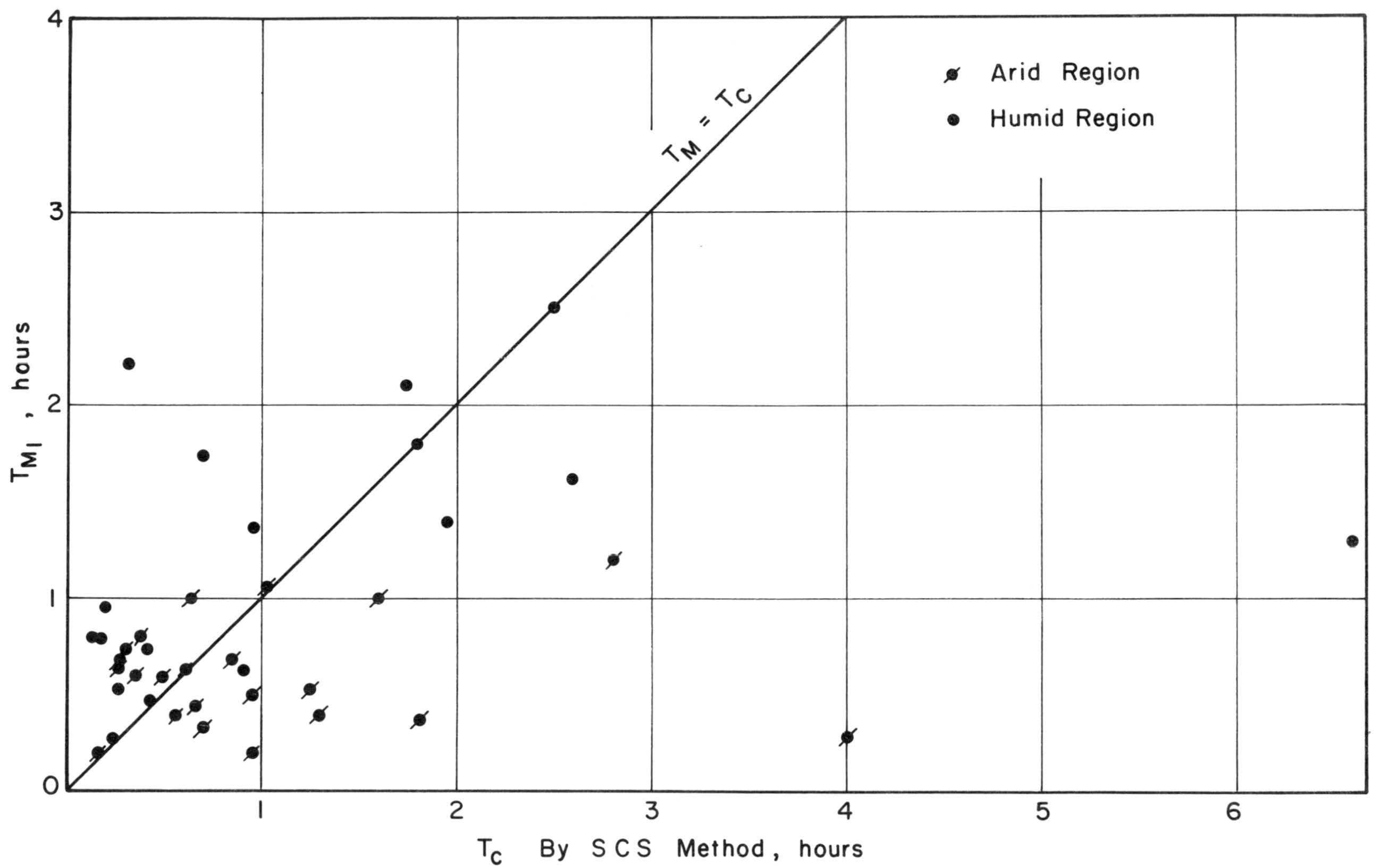


Figure 22. Correlation between median rise time and the time of concentration of SCS method

It is concluded that there is room for improving the prediction of rise time. Multiple regression analysis will be used to derive formulas for estimating the median rise time.

2. Regression Analysis

The regression and correlation analysis is one of the oldest statistical tools used in hydrology. It was first used for filling missing data and extending short records at one hydrologic station by relating the available data at this station with those at adjacent stations. Now its application has been broadened to cover the study of the relationship between two or more hydrologic variables and also the investigation of dependence between the successive values of a series of hydrologic data (34).

a. Multiple linear regression

The association of three or more variables can be investigated by multiple regression and correlation analysis (34 and 36).

The multiple regression relation may be expressed in the form

$$x_1 = (x_2, x_3, \ldots, x_m)$$

where x_1, x_2, ..., x_m are m variables. This equation gives the estimates of x_1 for given values of all other variables. If the relationship is linear, the regression is called multiple linear regression and the association is multiple linear correlation. Variables of non-linear relations in hydrology are often transformed to linear relations for multiple regression analysis because linear equations are easier to treat than non-linear multiple relations.

If there are m variables to correlate, including one dependent and m-1 externally independent, the general equation for multiple linear regression is

$$x_1 = B_1 + B_2x_2 + \dots + B_ix_i + \dots + B_mx_m$$

where B_1 is the intercept and B_i is the multiple regression coefficient of the dependent variable x_1 on the independent variable x_i, with all other variables kept constant. By the method of least squares, m linear equations may be obtained enabling one to determine the m unknown coefficients as follows:

$$B_2\,\Sigma(\Delta x_2)^2 + B_3\,\Sigma(\Delta x_2\Delta x_3) + \dots + B_m\,\Sigma(\Delta x_2\Delta x_m) = \Sigma(\Delta x_1\Delta x_2)$$

$$B_2\,\Sigma(\Delta x_2\Delta x_3) + B_3\,\Sigma(\Delta x_3)^2 + \dots + B_m\,\Sigma(\Delta x_3\Delta x_m) = \Sigma\,(\Delta x_1\Delta x_3)$$

$$\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots$$

$$B_2\,\Sigma(\Delta x_2\Delta x_m) + B_3\,\Sigma(\Delta x_3\Delta x_m) + \dots + B_m\,\Sigma(\Delta x_m)^2 = \Sigma(\Delta x_1\Delta x_m)$$

$$B_1 = \bar{x}_1 - B_2\bar{x}_2 - B_3\bar{x}_3 - \dots - B_m\bar{x}_m$$

where $\bar{x}_i = \frac{\sum x_i}{N}$ and $\Delta x_i = x_i - \bar{x}_i$, with i taken from 1 to m, and N is the sample size. These equations enable the determination of m parameters. The computational work increases with the increase of either the number m of the variables or the sample size N.

If the relationship is a curvilinear one of the form

$$x_1 = b_1 x_2^{b_2} x_3^{b_3} x_4^{b_4} \ldots x_m^{b_m}$$

a logarithmic transformation will yield a linear equation of the form

$$\log x_1 = \log b_1 + b_2 \log x_2 + b_3 \log x_3 + \ldots + b_m \log x_m$$

Inherent in the application of multiple regression methods of analysis to hydrologic problems are four assumptions (37):

1. there are no errors in the independent variables; errors occur only in the dependent variable;

2. the variance of the dependent variable does not depend on the values of the independent variables;

3. the observed values of the dependent variable are serially uncorrelated random variables;

4. the independent variables are not correlated with each other.

The multiple regression and correlation analysis is used a great deal at present because the tedious numerical computations can be executed very quickly by the aid of electronic digital computers.

b. Computer program of stepwise regression analysis

A stepwise multiple linear regression program (35), available at the Engineering Research Center of Colorado State University, was utilized to derive an equation for predicting a median rise time of a watershed. This program was originated by the Health Sciences

Computing Facility at U. C. L. A. and was modified to run on CDC 6600 computer.

This program computes a sequence of multiple linear regression equations in a stepwise manner. At each step one variable is added to the regression equation. The variable added is the one which makes the greatest reduction in the error sum of squares. Equivalently, it is the variable which has highest partial correlation with the dependent variable partial on the variables which have already been added; and, equivalently, it is the variable which, if it were added, would have the highest F value. The control of the variable entered at each step is by F-value test, the significance level being selected by the operator. The F-values for entry and deletion of variables can also be selected at any desired level of significance.

3. Application of Regression Analysis

a. General

It is worthwhile at this point to note some of the difficulties of the multiple regression technique when applied to hydrologic problems. The assumptions, cited previously, upon which the multiple regression approach is based are often violated in hydrologic problems.

The first assumption, that no errors occur in the independent variables, is violated in hydrology. In this investigation, none of the variables can be exact measurements. However, it is expected that the dependent variable contains relatively more error than any one of the independent variables. The second assumption is frequently suspect in hydrologic studies, and efforts should be made to check its significance in applications of regression analysis. In many hydrologic studies assumption three is violated, but in this investigation the values of rise times are evidently uncorrelated random variables. With regard to the fourth assumption, that the independent variables are not correlated with each other, the stepwise regression procedure usually results in the first few variables being almost independent (38).

The multiple correlation coefficient, R, is the primary statistical parameter used to appraise the prediction equations of this study. The square of R, called the multiple coefficient of determination, is a good indicator of the utility of a derived equation. It expresses the fraction of variance of the dependent variable attributable to the variation of independent variables contained in the regression equation.

b. Estimating median rise time by watershed characteristics

In this part of the study, only watershed characteristics were used in the stepwise regression analysis of median rise times by computer. Both values of rise time measurements were used.

First, covariance and correlation matrices of all variables listed in table 2 were obtained; then the variables were reselected according to their degree of correlation with the median rise time and

TABLE 2. VARIABLES* USED IN THE REGRESSION ANALYSIS

No.	Symbols	Names	Units
1	A	Area	Square miles
2	C	Compactness coefficient	Dimensionless
3	D_d	Drainage density	Miles per sq. mi.
4	F	Form factor	Dimensionless
5	H	Total fall	Feet
6	L	Length of main stream	Miles
7	L_c	Distance to centroid of area	Miles
8	L_m	Dimensionless mean travel distance	Dimensionless
9	L_s	Total length of extended streams	Miles
10	L_t	Average travel distance	Miles
11	P	Perimeter of catchment	Miles
12	R_1	Overland slope (1st definition)	Feet per mile
13	R_2	Overland slope (2nd definition)	Feet per mile
14	R_3	Overland slope (3rd definition)	Feet per mile
15	R_4	Overland slope (4th definition)	Feet per mile
16	R_5	Overland slope (5th definition)	Feet per mile
17	R_6	Overland slope (6th definition)	Dimensionless
18	S_1	Stream slope (1st definition)	Feet per mile
19	S_2	Stream slope (2nd definition)	Feet per mile
20	S_3	Stream slope (3rd definition)	Feet per mile
21	S_4	Stream slope (4th definition)	Feet per mile
22	T_{M1}	Median rise time (1st definition)	Hours
23	T_{M2}	Median rise time (2nd definition)	Hours
24	W	Average width of catchment	Miles

* The definitions of these variables are in Appendix A.

their degree of interdependency between themselves. Since any variable can be forced into the equation in the computer program as mentioned earlier, only variables best correlated with the median rise time and practically independent of each other were used to obtain the prediction equation.

The variables ennumerated in the equations to follow are in descending order of significance. Only statistically significant variables are included. The F-test at a ten per cent level was used, as described in the previous section on computer program, to determine whether an independent variable is significant or not. The physical meaning of each contribution was considered because physical comprehension and interpretation are necessary for deciding the limitations and suitability of the equations in practical application. A summary of the equations discussed in the ensuing paragraphs is presented in table 3.

c. Equations for all watersheds

Forty-six watersheds and 22 catchment characteristics listed in table 2 were used. The results showed the highest positive correlations of the median rise times with the stream slopes and overland slopes. The equations obtained are as follows:

$$T_{M1} = -1.27 + 0.15\, H \times 10^{-2} + 0.57\, R_5 \times 10^{-2} \qquad [1]$$

and

$$T_{M2} = -0.77 + 0.08\, H \times 10^{-2} + 0.40\, R_5 \times 10^{-2} \qquad [2]$$

where H is the total fall in feet and R_5 is the overland slope in feet per mile. T_{M1} and T_{M2} are the two measures of median rise time in hours. The coefficient of determination is 0.75 and 0.74 respectively for equations 1 and 2.

The positive correlations of the median rise times with the slopes is contrary to physical reasoning. Hydraulic considerations suggest that the median rise time should tend to decrease with increases in slope. Graphical plottings of the median rise times against the stream slope and the overland slopes, figure 23, show that the relationship is mainly determined by the six California watersheds which differ greatly from the other watersheds in the following respects:

1) the flood-producing storms are of the long duration storms (ten hours and longer) rather than the local convective thunderstorm type (three hours and shorter);

2) Extremely steep topography.

The second assumption of regression analysis is, therefore, violated here because the variance of median rise times does depend on the values of the independent variables. It is an unfortunate coincidence that these two extreme conditions, 1 and 2, should occur on the same watersheds as there appears to be no physical relationship between them. It seems that the long rise times are

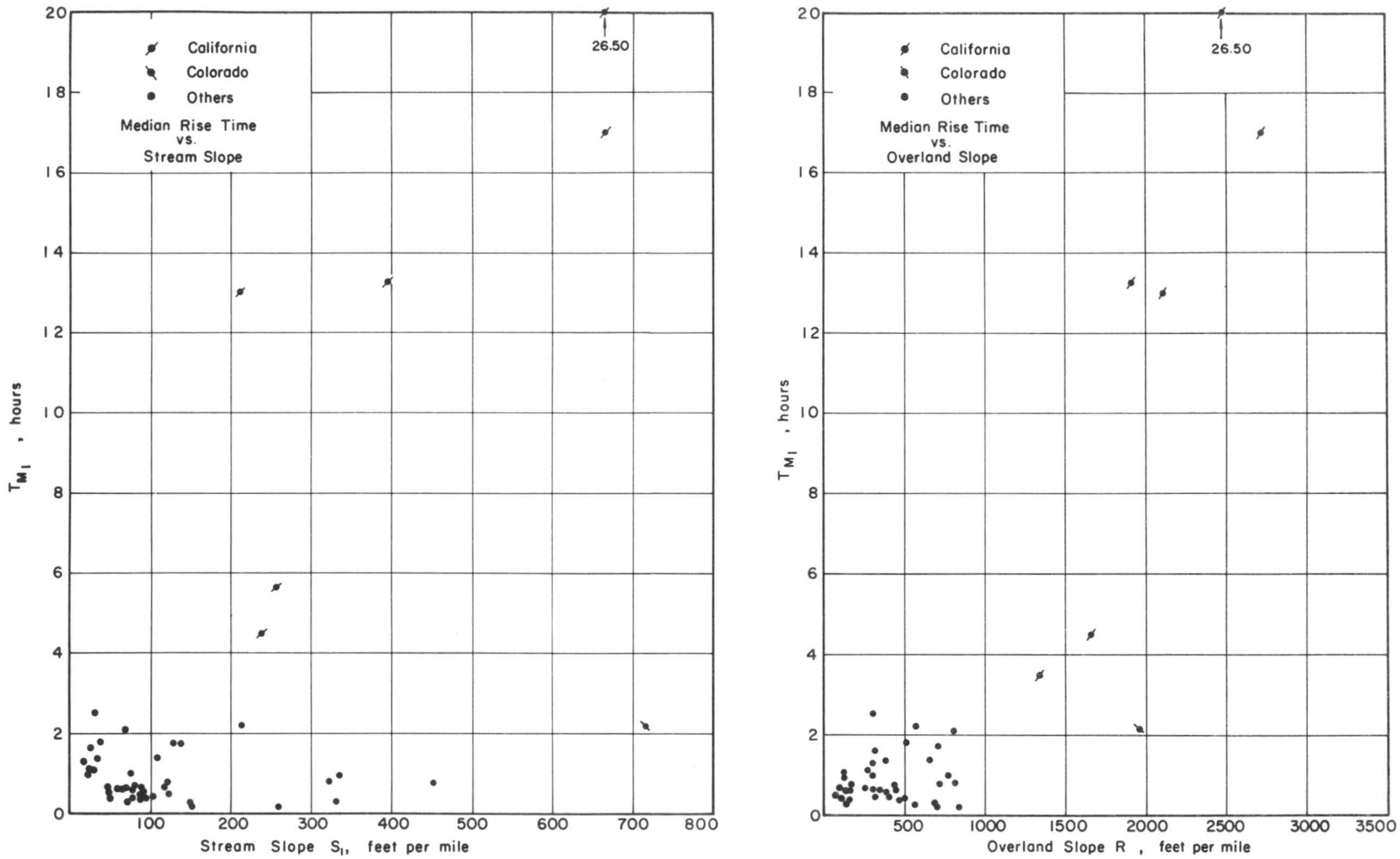


Figure 23. Median rise times versus stream slopes and overland slopes

due to (1) rather than (2) but no parameters expressing (1) were included in the regression analysis and so the correlation is with (2).

This idea may be tested by examining the correlation when the California watersheds are removed from the analysis, as will be done in the next section.

These findings suggest some possible classification of watersheds according to the types of flood-producing storms, i.e., long or short duration storms (39). Detailed investigations of rise times for long duration storms are not justified here because there are only six watersheds from California which fall within this category.

d. Watersheds subject to short duration storms only

This grouping only eliminates the California watersheds leaving 40 watersheds. The correlation coefficient of rise times with slopes becomes almost zero. The resulting equations are as follows:

$$T_{M1} = 0.42 - 0.03\,D_d + 0.48\,C \qquad [3]$$

and

$$T_{M2} = 0.34 - 0.02\,D_d + 0.34\,C \qquad [4]$$

where D_d is the drainage density in miles per square mile and C is the dimensionless compactness coefficient. T_{M1} and T_{M2} are the two measures of median rise times in hours. The coefficient of determination is 0.21 and 0.24 respectively for equations 3 and 4.

This indicates that only about 20 per cent of the variation in specific rise time has been explained.

Logarithmic transformation was also used and the equations became:

$$\log T_{M1} = 0.16 - 0.44 \log D_d \qquad [5]$$

and

$$\log T_{M2} = 0.04 - 0.44 \log D_d \qquad [6]$$

where D_d is the drainage density in miles per square mile. T_{M1} and T_{M2} are the two measures of median rise times in hours. The coefficient of determination is still low 0.25 and 0.28 respectively for equations 5 and 6.

In both cases the coefficient of determination is too low for reliable estimates and further subdivision appears necessary. Examination of hydrographs suggests that there are significant differences between hydrograph rises in humid areas and arid areas. The watersheds are grouped in two regions: humid and sub-humid region, arid and semi-arid region as specified in reference 40.

e. Humid and sub-humid watersheds - short duration storms

For the 18 humid and sub-humid watersheds the equations obtained are as follows:

$$T_{M1} = 0.81 + 0.36 L_c \qquad [7]$$

and

$$T_{M2} = 0.86 + 0.25\, L_c - 0.65\, F \qquad [8]$$

where L_c is the distance from the outlet to centroid of area in miles and F is the dimensionless form factor (ratio of the area to the square of the length of main stream, A/L^2). T_{M1} and T_{M2} are the two measures of median rise times in hours. The coefficient of determination is 0.38 and 0.69 respectively for equations 7 and 8.

Logarithmic transformation was also used:

$$\text{Log } T_{M1} = 0.06 + 0.47 \log L_c \qquad [9]$$

and

$$\text{Log } T_{M2} = -0.08 + 0.51 \log L_c \qquad [10]$$

where L_c is the distance from the outlet to the centroid of area in miles. T_{M1} and T_{M2} are the two measures of median rise times in hours. The coefficient of determination is 0.55 and 0.68 respectively for equations 9 and 10.

Sixty-nine per cent of the variation of the median rise time measurement by T_{M2} can be explained by equation 8, but only 38 per cent can be explained if the measurement was made by T_{M1}. This suggests that T_{M2} is a better measurement of median rise time than T_{M1}. The result may be explained by the data which shows that 83 per cent of the hydrographs in the humid and sub-humid region have relatively long rising limb, but the initial part of these limbs is often a small part of the hydrograph and not closely related to the main rise.

f. Arid and semi-arid watersheds - short duration storms

The equations obtained for 22 arid and semi-arid watersheds are as follows:

$$T_{M1} = 1.33 - 0.86\, S_2^{\frac{1}{2}} \times 10^{-1} - 0.93\, A \times 10^{-2} \qquad [11]$$

and

$$T_{M2} = 1.05 - 0.68\, S_2^{\frac{1}{2}} \times 10^{-1} - 0.62\, A \times 10^{-2} \qquad [12]$$

where S_2 is the stream slope in feet per mile and A is the area in miles. T_{M1} and T_{M2} are the two measures of median rise time in hours. The coefficient of determination is 0.68 and 0.58 respectively for equations 11 and 12.

Using logarithmic transformation, the equations become:

$$\text{Log } T_{M1} = 0.91 - 0.60 \log S_3 - 0.14 \log L_s \qquad [13]$$

and

$$\text{Log } T_{M2} = 0.80 - 0.60 \log S_3 - 0.11 \log L_s \qquad [14]$$

where S_3 is the stream slope in feet per mile and L_s is the total length of extended streams in miles. T_{M1} and T_{M2} are the two measures of median rise times in hours. The coefficient of determination is 0.73 and 0.69 respectively for equations 13 and 14.

The above equations show that rise time decreases as area increases in arid regions. This agrees with the findings of Keppel and Renard (16), as cited in the literature review, on their hydrograph analysis for ephemeral streams in Walnut Gulch, Arizona. Keppel and Renard claim that this is due to the high transmission loss and the translatory wave effect.

For humid and sub-humid regions, the correlation matrix shows the opposite effect, i.e., rise times tend to increase with increasing areas and these opposing effects are probably responsible for the low coefficients of determination of equations 3, 4, 5 and 6 which include all watersheds.

The above results show also that T_{M1} is a better measurement of median rise time than T_{M2}, but the difference is not as distinct as in the humid and sub-humid regions. This is reasonable because all the available hydrographs of arid or semi-arid watersheds have short rising limbs.

g. Correlation between the two measures of median rise times

It has been shown in the preliminary study that T_{R1} is highly correlated with T_{R2}. T_{M1} and T_{M2} are also highly correlated and their relationship is as follows:

$$T_{M1} = 0.12 + 1.50\ T_{M2}$$

TABLE 3. RISE TIMES PREDICTION EQUATIONS

Region	Sample Size	Equation Number	EQUATIONS	R^2	F
All Watersheds	46	1	$T_{M1} = -1.27 + 0.15\ H \times 10^{-2} + 0.57\ R_5 \times 10^{-2}$	0.75	63.6
		2	$T_{M2} = -0.77 + 0.08\ H \times 10^{-2} + 0.40\ R_5 \times 10^{-2}$	0.74	60.3
			$T_{M1} = -0.12 + 1.50\ T_{M2}$	0.99	4068
All Watersheds Short Duration Storms	40	3	$T_{M1} = 0.42 - 0.03\ D_d + 0.48\ C$	0.21	5.0
		4	$T_{M2} = 0.34 - 0.02\ D_d + 0.34\ C$	0.24	5.8
		5	$\log T_{M1} = 0.16 - 0.44 \log D_d$	0.25	12.5
		6	$\log T_{M2} = 0.04 - 0.44 \log D_d$	0.28	14.9
			$T_{M1} = 0.01 + 1.31\ T_{M2}$	0.83	188
Humid and Sub-Humid Watersheds	18	7	$T_{M1} = 0.81 + 0.36\ L_c$	0.38	9.6
		8	$T_{M2} = 0.86 + 0.25\ L_c - 0.65\ F$	0.69	16.4
		9	$\log T_{M1} = 0.06 + 0.47 \log L_c$	0.55	19.2
		10	$\log T_{M2} = -0.08 + 0.51 \log L_c$	0.68	33.4
			$T_{M1} = 0.14 + 1.27\ T_{M2}$	0.75	46.7
Arid and Semi-Arid Watersheds	22	11	$T_{M1} = 1.33 - 0.86\ S_2^{\frac{1}{2}} \times 10^{-1} - 0.93\ A \times 10^{-2}$	0.68	20.6
		12	$T_{M2} = 1.05 - 0.68\ S_2^{\frac{1}{2}} \times 10^{-1} - 0.62\ A \times 10^{-2}$	0.58	13.0
		13	$\log T_{M1} = 0.91 - 0.60 \log S_3 - 0.14 \log L_s$	0.73	25.4
		14	$\log T_{M2} = 0.80 - 0.60 \log S_3 - 0.11 \log L_s$	0.69	21.1
			$T_{M1} = 0.06 + 1.12\ T_{M2}$	0.88	153

for all watersheds,

$$T_{M1} = 0.01 + 1.31\ T_{M2}$$

for all watersheds except California watersheds,

$$T_{M1} = 0.14 + 1.27\ T_{M2}$$

for humid and sub-humid watersheds, and

$$T_{M1} = 0.06 + 1.12\ T_{M2}$$

for arid and semi-arid watersheds. T_{M1} and T_{M2} are the two measures of median rise times in hours.

The coefficients of determination are .75 to .99 as shown in table 3.

4. <u>Conclusions on Prediction of Median Rise Time</u>

The following conclusions can be made from this study of rise time by catchment characteristics:

(a). There are a number of difficulties in applying the multiple regression method to hydrologic data and a high coefficient of determination does not necessarily mean that an equation is suitable for practical applications. The equations derived from the small watersheds data are summarized in table 3.

(b). Equations derived for all watersheds were determined mainly by the six California samples which are subject to long duration storms. These were not suitable for other watersheds subject to short duration storms.

(c). For watersheds subject to short duration storms, the rise time tends to increase with area in humid regions and decrease with area in arid regions. These opposing effects are probably responsible for low coefficients of determination of equations 3, 4, 5 and 6 which include all watersheds.

(d). Reasonable estimates can be obtained with equations 8 and 10, which are limited to humid and sub-humid watersheds subject to short duration storms, and equations 11 and 13, which are limited to arid and semi-arid watersheds subject to short duration storms. The results suggest that these equations give better estimates than the widely used SCS method, although it has not been possible to thoroughly check this with independent data.

(e). T_{M1} and T_{M2} are highly correlated and the regression equations are summarized in table 3. T_{M2} is a better measurement of rise times than T_{M1} in humid regions. However, in the arid regions the two measurements are almost the same.

CHAPTER VII

APPLICATIONS OF MEDIAN AND RELATIVE RISE TIMES

For estimation of the design flood hydrograph it is often desirable to know what rise time is appropriate for floods of various frequencies. There could possibly be a tendency for lower rise times to coincide with larger floods and vice versa.

There are various hydrologic techniques that require estimates of median rise time, critical duration and similar watershed characteristics. Some of these will be discussed in this chapter.

1. Correlation of Rise Time with Flood Frequency

If there were many observations from one watershed, it would be desirable to investigate the variation in time of rise with magnitude of flood by correlating:

- rise time vs. flood peak,
- rise time vs. volume of discharge,
- rise time vs. total rainfall,

etc...

If some relationship could be established between, say, rise time and flood peak for one watershed, it may be of the form shown by curve A of figure 24.

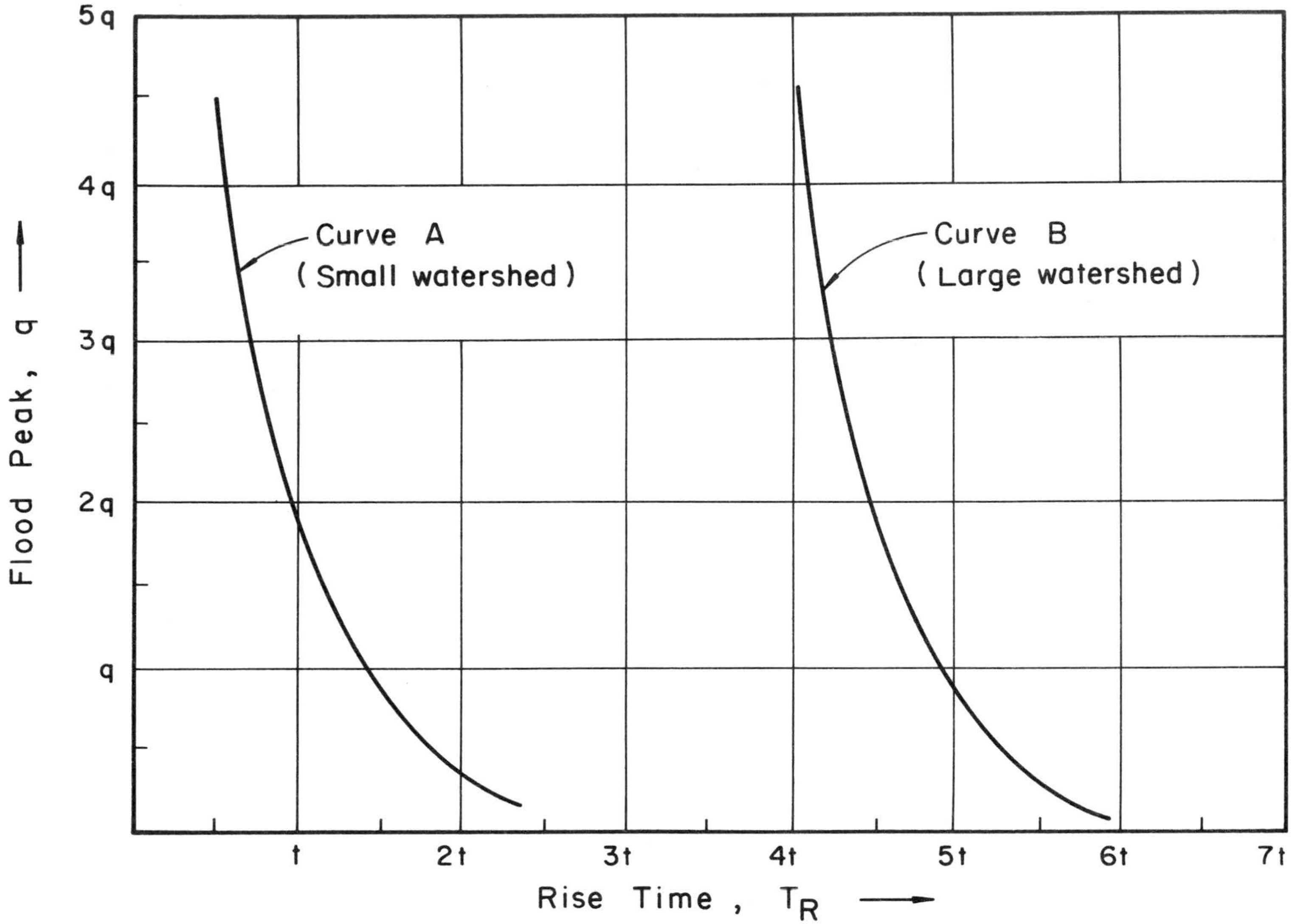


Figure 24. Correlation between rise time and flood peak

For a larger watershed, the relationship would be expected to be different because rise times tend to be larger. Curve B of figure 24 might be typical of a larger watershed.

With the small watershed data there are a few observations from many watersheds and, if one were to lump them all together to try and get a curve like A, one would be unsuccessful because of the variations due to watershed size,etc...

To remove the above variations, relative rise time is substituted for rise time and peak frequency for flood peak. Now all observations are, essentially, measures of the deviation from the average condition and can be added together to give one large homogeneous sample instead of a number of small heterogeneous samples.

Figure 25 (a) shows plottings of relative rise times, R_{T1}, against the corresponding flood frequencies. It was concluded that there is no relationship between the relative rise times and the flood frequencies. This result was also checked with the plottings of the relative rise times versus flood frequencies of 140 events from one single watershed (Shaver Creek, figure 25 (b)).

Since there is no correlation between relative rise times and flood frequencies, one can use the median rise time to estimate the design hydrograph for any flood frequency.

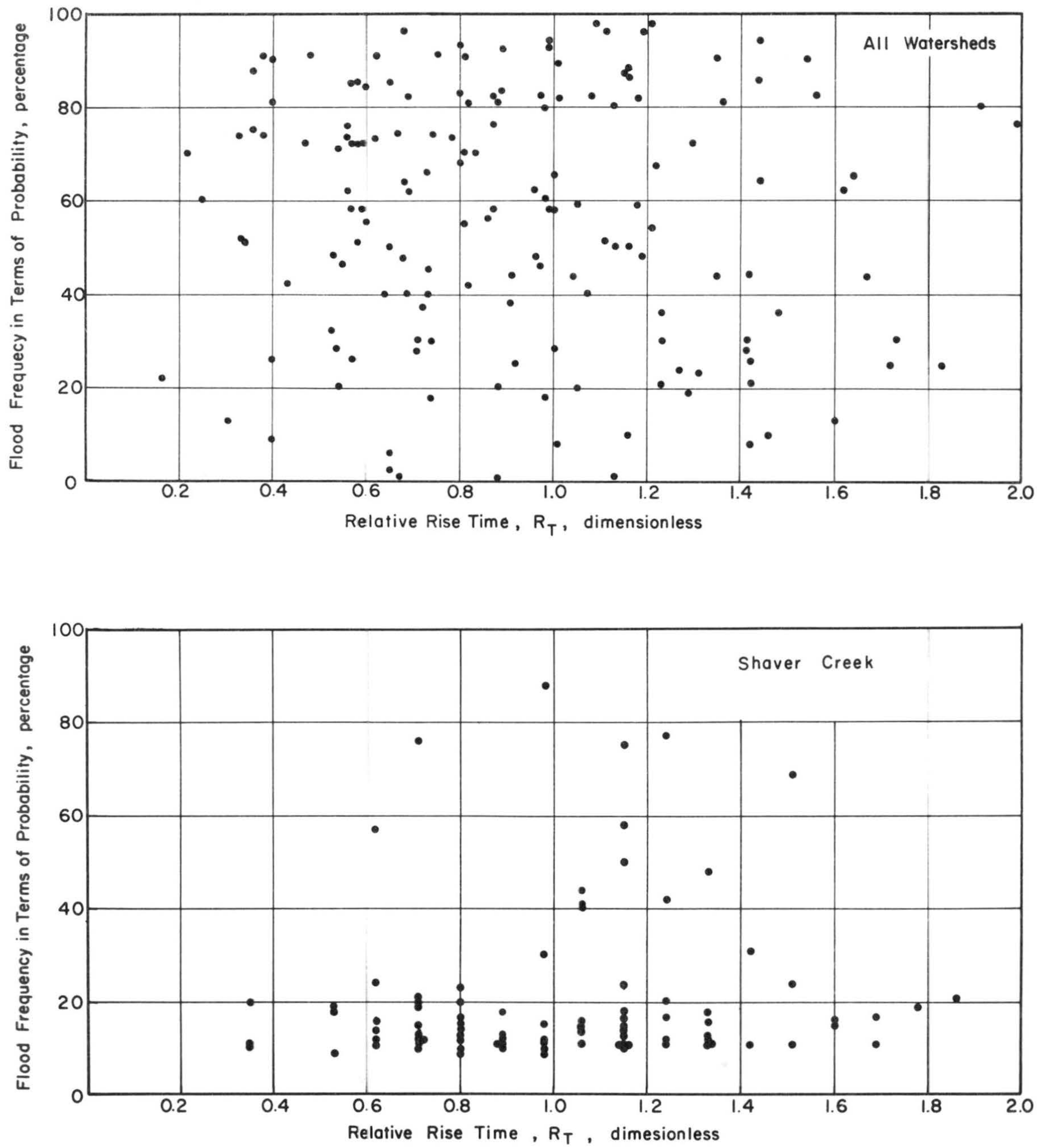


Figure 25. Correlation between relative rise time and flood frequency

2. Use of Rise Time in Hydrologic Techniques

a. Rational formula

The rational formula is:

$$Q = CIA$$

where Q is the peak discharge in cubic feet per second, C is a runoff coefficient depending on characteristics of the drainage basin, I is the rainfall intensity in inch per hour, and A is the drainage area in acres (8, 21, 47).

When using the rational formula, one must assume that the maximum rate of flow, owing to a certain rainfall intensity over the drainage area, is produced by that rainfall which is maintained for a certain effective duration. Up to the present, this duration is assumed to be equal to the period of concentration of flow at the point under consideration and referred to as time of concentration. Ramser, who is responsible for the empirical formula for the time of concentration later made known by Kirpich, has determined the time of concentration by rise time or time to peak (see Chapter II). The findings of this study suggest that rise time should be used instead of time of concentration for selecting the rainfall intensity I in the rational formula.

b. Regional analysis

The analysis of hydrologic data from relatively homogeneous areas to obtain general relationships between various hydrologic factors for the region covering the areas is sometimes referred to as regional analysis.

Where data are available to develop flood-frequency curves for small watersheds on a regional analysis, the peaks for selected frequencies (usually 2-, 10-, and 100-year frequencies) can be plotted versus A/T_R, where A is the drainage area and T_R is the rise time or the time from the beginning of rise to the peak rate of flow. The resulting plotting will show a rather consistent relationship if the physical characteristics of the watersheds are not too variable (41). The most appropriate rise time for these estimates is the median. The U.S. Geological Survey publishes circulars (on a state basis) which gives the results of regional analyses of watersheds in specific areas.

c. Synthetic hydrographs

A synthetic hydrograph is prepared using the data from a number of watersheds to develop a dimensionless unit hydrograph applicable to ungaged watersheds (3).

1) Dimensionless hydrographs - The curvilinear hydrograph shown in figure 26 is a dimensionless unit hydrograph prepared by Victor Mockus (42) from the unit hydrographs of a variety of watersheds. Various relationships between the hydrograph components can be determined, the most useful of which is the equation for the peak rate of flow:

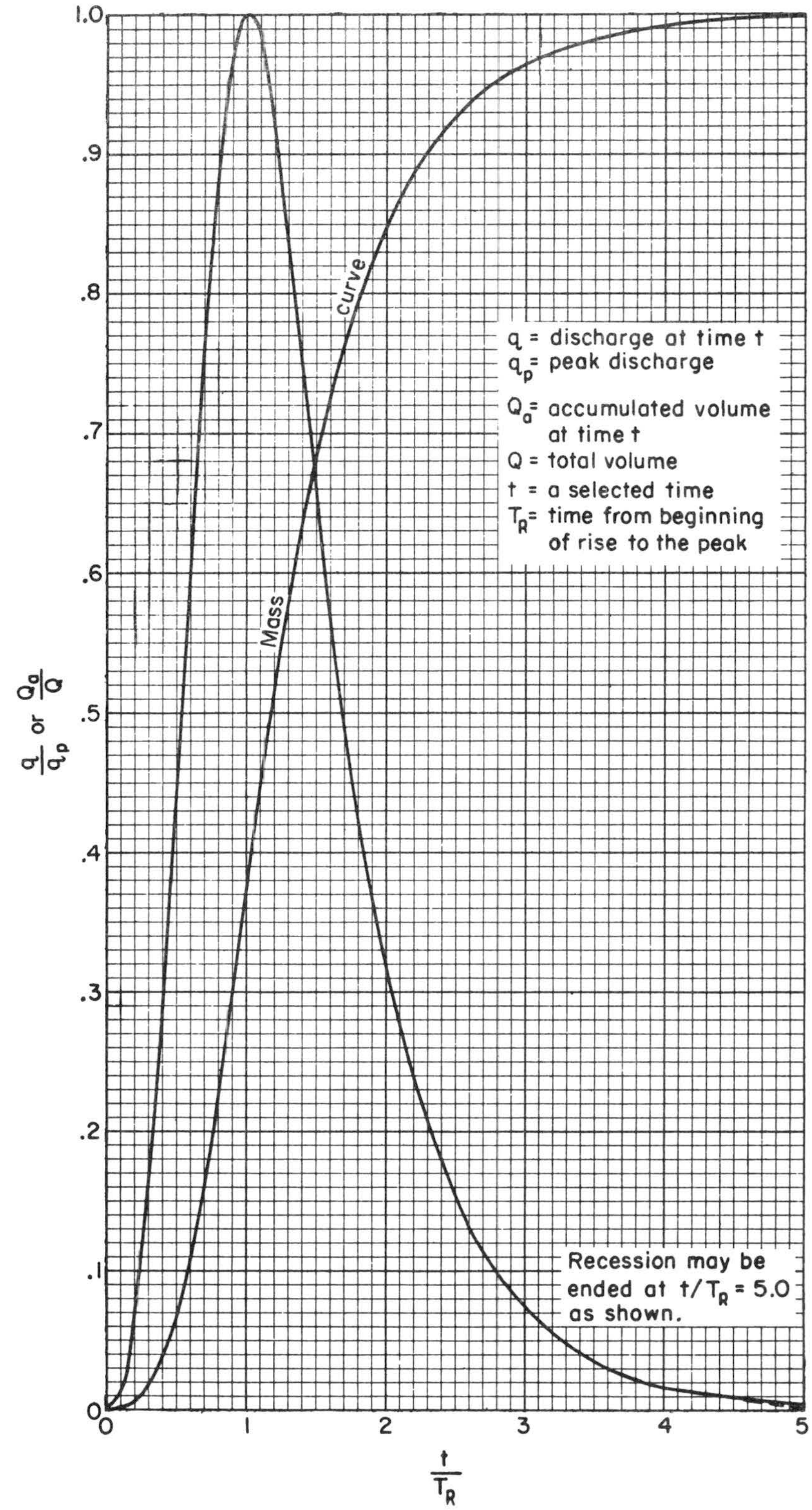


Figure 26. Dimensionless hydrograph and mass curve (after V. Mockus, 1957)

$$q_p = \frac{KAQ}{T_R}$$

where q_p is the peak rate of flow in cubic feet per second, K is a constant dependent on the shape of the hydrograph, A is the drainage area in square miles, Q is the volume of runoff in inches and T_R is the rise time which is the time in hours from start of rise to peak rate. A nomogram for the solution of this equation is given in figure 27 for K equal to 484 as suggested by Mockus. Single-peaked hydrographs can be constructed when any three of the four variables in the above equation are known with the use of figure 26.

Again, the median rise time would be the most appropriate value of T_R to use for a particular watershed.

2) <u>Triangular hydrographs</u> - The curvilinear hydrograph of figure 26 can often be replaced by an equivalent triangular hydrograph which is more easily constructed and, for routing through reservoirs or stream channels, gives results about as accurate as those obtained using the curvilinear hydrograph (41). Figure 28 shows a triangular hydrograph based on the curvilinear hydrograph of figure 26 where

$$T_b = T_R + T_r .$$

Since $T_r = 1.67\ T_R$, by construction, then

$$T_b = 2.67\ T_R .$$

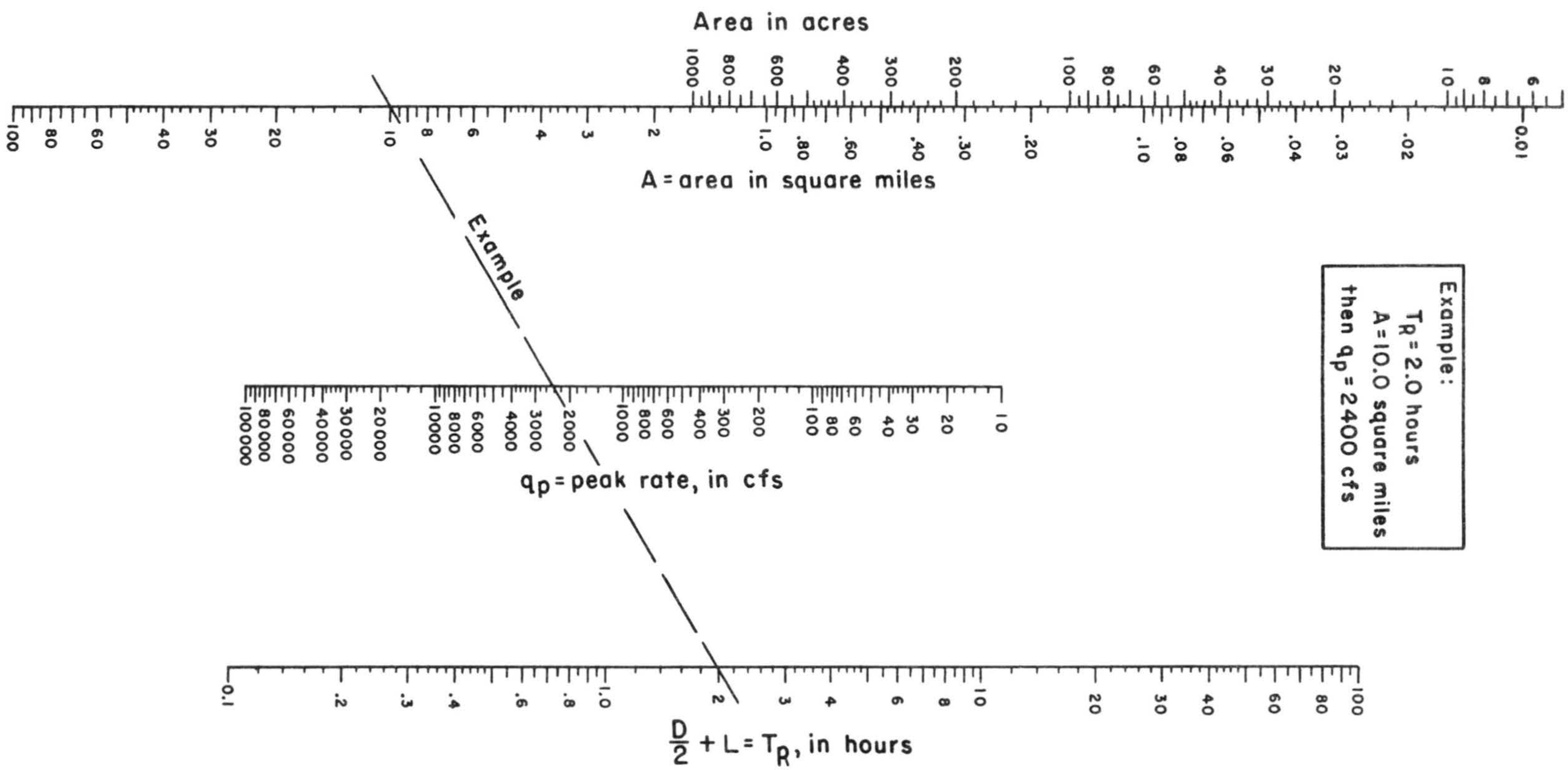
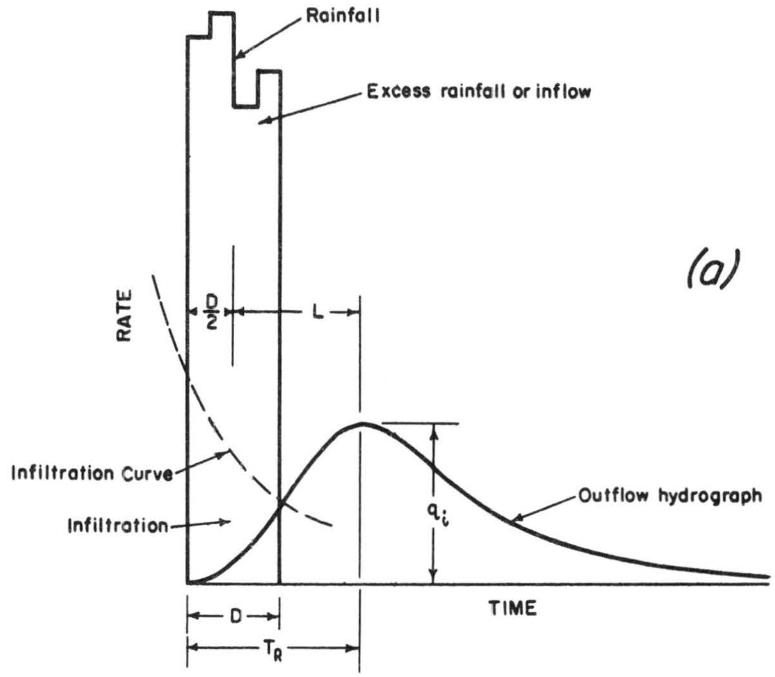


Figure 27. Peak rates when Q = 1 inch (after V. Mockus, 1957)



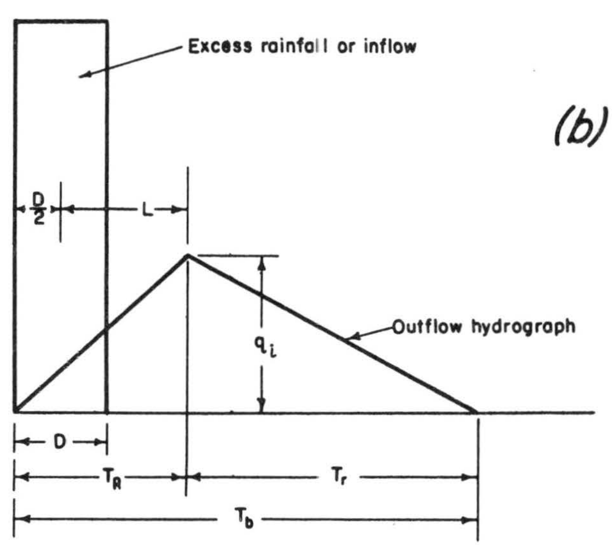


Figure 28. Triangular hydrographs (after SCS-USDA, 1957)

T_R may be the median rise time or other specific rise times, depending on the purpose of the study.

d. Design hydrograph shape

The shape of a design hydrograph is sometimes obtained by the use of the dimensionless unit hydrograph (43, 44). This idea is that the usual hydrograph for storms of short duration (45),

$$Q = f_1 \text{ (t, storm intensity, watershed properties)}$$

can be written in dimensionless form:

$$Q/Q_c = f_2 (t/t_c) .$$

The latter expression will apply to any watershed, Q being the discharge rate and Q_c the characteristic of the storm intensity - usually the peak rate of discharge following the storm: t_c , sometimes called the "time lag", is a characteristic of the particular watershed. Usually t_c measures the breadth of the hydrograph; it might, for example, be the rise time or time to peak, T_R, which is the time elapsed between the incidence of the runoff and the maximum discharge.

3. Conclusions on Applications of Median and Relative Rise Times

Relative rise time is not correlated with flood frequency; therefore, the median rise time is a suitable representative value for any frequency flood. As a time characteristic of a particular watershed, rise time is very usefull in hydrologic techniques of flood peak estimation and design hydrograph prediction.

CHAPTER VIII

SUMMARY AND CONCLUSIONS

1. Statement of Problem

In many instances, the hydrologist is requested to determine the outflow hydrograph from a basin. Hence, he is concerned not only with the maximum discharge and the total volume but also the runoff distribution with respect to time.

The time of occurrence of the maximum discharge of the outflow hydrograph from small watersheds (less than 40 square miles) is the subject of this study and is referred to as hydrograph rise time or time to peak.

2. Method of Study

This investigation uses 407 flood events of 47 watersheds in 13 states. The main objective is to study the variability of rise times and to try and relate this with climatic factors and physiographic factors. It is shown that a general distribution of values applied to most watersheds will explain the change of rise times caused by climatic factors such as antecedent moisture conditions of the watershed, storm patterns, etc. The median rise time is then a time characteristic of a particular watershed which can be estimated by the physiographic watershed characteristics. Before deriving prediction equations of rise times by using stepwise regression analysis, an evaluation of an existing popular method (Soil Conservation Service) is made.

3. Results

The complexity of small watershed responses is due to the heterogeneity between individual small watersheds with respect to climate, geometric characteristics and other basin factors. The relatively sparse field data from many small watersheds must be pooled and analysed as a large sample to yield maximum information on floods and their causative factors. This approach has been used in this study by converting rise times to dimensionless relative rise times (ratio of individual rise time to the median rise time). It is shown that the same log-normal distribution may be adopted for relative rise times in all the watersheds studied.

In the stepwise regression analysis, it has been found necessary to distinguish the watersheds subject to long duration storms (ten hours and longer) and short duration storms or thunderstorms (three hours and shorter). Furthermore, the heterogeniety between individual small watersheds does not permit a single general equation for estimating rise times within the short duration group. It is, therefore, advisable to substratify the watersheds further into humid regions and arid regions. Two different equations have been obtained for these regions with about 70 per cent of the variation explained.

4. Conclusions

Rise times within a watershed, far from being constant, vary from about 0.3 to 3 times the median or more. A single distribution of relative rise times R_T, applicable to all watersheds studied, has the following mathematical form:

$p(R_T)$ = probability density of R_T (figure 20)

$$p(R_T) = \frac{1}{-1.74\, R_T}\, e^{-\frac{\left(\ln R_T + 0.16\right)^2}{0.96}}$$

The median rise time in hours for a humid watershed subject to short duration storms can be estimated reasonably from the following equation:

$F = \frac{A}{L^2}$ L = total L of stream
L_c = L to centroid

$$T_M = 0.86 + 0.25\, L_c - 0.65\, F$$

where L_c is the distance to centroid of area in miles and F is the dimensionless form factor.

The median rise time in hours for an arid watershed can be estimated reasonably from the following equation:

$$T_M = 1.33 - 0.86\, S_2^{\frac{1}{2}} \times 10^{-1} - 0.093\, A \times 10^{-2}$$

where S_2 is the stream slope in feet per mile and A is the area in square miles.

The median rise time can be used as a time characteristic in hydrologic techniques of flood peak estimation and design hydrograph prediction.

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APPENDIX A

METHOD OF COMPUTING WATERSHED CHARACTERISTICS

This has been extracted from an unpublished report entitled: "Explanation of Symbols and Method for Filling in the Watershed Card" in the Research Data Assembly for Small Watershed Floods at Colorado State University, prepared by R. Markovic, R. Downer and M. Siddiqi, May 1964.

LIST OF SYMBOLS

A	Area	Square miles
c	Contour interval	Feet
C	Compactness coefficient	Dimensionless
D_d	Drainage density	Miles per sq. mi.
F	Form factor	Dimensionless
Δh	Difference in Elevation	Feet
h	Elevation above outlet	Feet
H	Total fall	Feet
K	Hourly depletion ratio	Dimensionless
l_i	Length of reach	Miles
l_t	Distance from grid intersection to outlet	Miles
L	Length of main stream	Miles
$\overline{L}$	Average length of two successive contour lines	Miles
L_c	Distance to centroid of area	Miles
L_{con}	Length of a contour line	Miles
L_g	Length of the grid lines within the catchment	Miles
L_m	Dimensionless mean travel distance	Dimensionless
L_s	Total length of extended streams	Miles

L_t	Average travel distance	Miles
n	Number of points	
N_i	Number of intersections of the grid lines with a contour line	
P	Perimeter of catchment	Miles
P_A	Average annual precipitation	Inches
q_A	Areal average discharge	Cfs per sq. mi.
Q_a	Average discharge	Cfs
R_1	Overland slope	Feet per mile
R_2	Overland slope	Feet per mile
R_3	Overland slope	Feet per mile
R_4	Overland slope	Feet per mile
R_5	Overland slope	Feet per mile
R_6	Overland slope	Dimensionless
S_1	Stream slope	Feet per mile
S_2	Stream slope	Feet per mile
S_3	Stream slope	Feet per mile
S_4	Stream slope	Feet per mile
s_t	Standard deviation	Miles
S_d	Dimensionless standard deviation	Dimensionless
W	Average width of catchment	Miles
X	Latitude of gaging station	Degrees
X_c	Latitude of centroid	Degrees

Y	Longitude of gaging station	Degrees
Y_c	Longitude of centroid of area	Degrees
Z	Elevation of gaging station	Feet-mean sea level
Z_1	Average elevation of a reach	Feet

CATCHMENT CHARACTERISTICS

Scale

Since all dimensions are to be expressed in feet or miles, it will be advantageous to convert the map scale to feet and miles and draw it on the margin of the map.

Area, A

The area will usually be given, but if not, it will be necessary to delineate the catchment boundary and planimeter the area. Record on the map in some convenient place (e. g. below the name of watershed) the area in square inches and the area in square miles to three decimal places. If the area is given, one needs only record the area in square miles on the map. For the present, areas less than 64 acres and greater than 25, 600 acres will be omitted.

Length of Main Stream, L

Extend all marked stream systems up to the watershed boundaries in accordance with the contours. Exceptions to this rule will be streams which appear to originate in springs or swamps. The "main stream" is defined as that stream draining the greatest area. Using the paper strip method, mark off the length of the main stream and at the same time mark and label points where the main stream crosses a contour line. Calculate the distances in miles to two decimal places between successive contours and record on the paper strip. The summation of these distances is the length of the main stream, L. Save the strip of paper for use in plotting the main stream profile, and calculation of slope.

<u>Total Length of Extended Streams,</u> L_s

Using the paper strip method, measure the total length of all extended streams. It is suggested to first measure all the tributaries on one side of the main stream, then the other side. The task can be made easier by summing distances from key stream forks rather than measuring all distances to the outlet.

<u>Length to Center of Area,</u> L_c

The centroid of the catchment can be found quickly and easily and with a fairly high degree of accuracy by centering over a map of the watershed a clear plastic overlay with a system of lines drawn on it at 45^0 angles to form a star-shaped design. L_c is the distance along the main stream from the outlet, to a point adjacent to the center of area projected to the main stream. This distance can be found most easily by using the paper strip originally used to measure the length of the main stream, L. Determine L_c in miles to two decimal places.

<u>Mean Travel Distance,</u> L_t

The average travel distance is determined by measuring the travel distance to the outlet along the stream system from each intersection of a square grid placed over a map of the catchment. The grid should be oriented in a North-South, East-West orientation. The grid should be of such a size that between 30 and 50 intersections fall within the catchment. It is suggested to number each of the intersections which fall within the catchment to aid in accounting for every intersection. The distances should be measured by the paper strip method and recorded in miles in tabular form so that L_t can easily be determined from $\frac{\Sigma l_t}{n}$, where Σl_t is the sum of individual travel distances and n is the number of intersections within the catchment. By noting and recording on the map the distances to key points or stream forks, the measurement of distances to other points can be greatly facilitated.

Standard Deviation of Travel Distance, s_t

The standard deviation of the travel distance will be computed as the square root of

$$\left\{\frac{1}{n}\Sigma l_t^2 - (\Sigma l_t)^2\right\}$$

This form lends itself to easy computation since the second term is equal to the average travel distance, L_t, squared. The first term. Σl_t^2, can be determined on the desk calculator by using accumulative multiply.

Perimeter, P

The perimeter is the distance around the catchment measured along the watershed boundary. Using the paper strip method, determine the perimeter in miles to two decimal places, starting at the gage and proceeding around the area and back to the gage to form a closed circuit.

Total Fall, H

Using the strip of paper with the main stream marked off on it as the abscissas, plot a graph of distance vs. elevation along the main stream on 20 x 20 to the inch graph paper. After the profile is plotted, extrapolate each of the ends of the profile. Determine the minimum and maximum elevations of the main stream from these extended slopes. The total fall can now be determined to the nearest foot. Record the distances in miles between successive contour lines on the profile.

Stream Slope, S_1

S_1 is calculated by dividing the total fall, H , by the Length of the Main Stream in miles, L . Calculate to the nearest foot per mile.

Stream Slope, S_2

$$S_2 = \frac{2\Sigma l_i z_i}{\Sigma l_i^2} = \frac{2\Sigma l_i z_i}{L^2} \text{ , feet/mile}$$

where l_i = distance along the main stream between successive contours. The individual l_i can be easily determined from the plotted profile of the main stream or from the paper strip used to measure the length of the main stream. z_i = the average elevation above the outlet for each reach of length, l_i .

<u>Stream Slope,</u> S_3

$$S_3 = \frac{\Sigma l_i^{\,2}}{\Sigma \dfrac{(l_i)^{3/2}}{(\Delta h)^{1/2}}}$$

where l_i = length of a reach in miles and Δh = the change in elevation in a reach. Calculate S_3 to the nearest foot per mile.

<u>Stream Slope,</u> S_4

$$S_4 = \frac{\text{elevation at 85\% of stream length-elevation at 10\% of stream length}}{\text{75\% of stream length}}$$

The elevation at 85% and 10% of the stream length can most easily be found by drawing vertical lines on the main stream profile at 85% and 10% of the length and noting the elevation where these lines cut the profile.

<u>Overland Slope,</u> R_1

$$R_1 = \frac{cL_{con}}{A}, \quad \text{feet/mile}$$

where c = the contour interval, L_{con} = the total length of contour lines on a map of the catchment, and A = the catchment area. L_{con} can best be determined by using the paper strip method. Label each strip and mark the length of each contour line on it for later use.

$$R_2 = \frac{1.57\ cN}{L_g}$$

where c = the contour interval, N = the number of intersections of the grid lines with contour line, and L_g = the total length of grid lines within the catchment measured in both the North-South and East-West directions. L_g can be determined using the paper strip method. N can be determined by starting at the catchment boundary and following each contour line and counting the number of times each crosses a grid line.

Overland Slope, R_3

$$R_3 = \frac{\Sigma(\Delta h \times \overline{L})}{A} ,$$

where Δh = the difference in elevation between contours, $\overline{L}$ = the average length of two successive contours, and A = the catchment area. $\overline{L}$ can be calculated from L_{con} found while calculating R_1. Consider the outlet and extreme boundary point of the watershed as having contour lines of zero length when computing values of $\overline{L}$.

Overland Slope, R_4

R_4 is the mean overland slope. The overland slope is calculated by dividing the contour interval by the perpendicular distance between contours at each intersection of the square grid.

$$R_4 = \frac{\Sigma s}{n} = \frac{\Sigma \frac{c}{l}}{n} = \frac{c}{\text{mean } l} , \quad \text{feet/mile}$$

where c = contour interval, l = the distance between contours at each grid intersection and n = the number of grid intersections for which l was computed.

<u>Overland Slope,</u> R_5

R_5 is the median of the overland slopes computed above. This can be found by arranging the distances between contours in descending order. The median is the distance which evenly splits this descending array, with one half of the distances above it and one half of the distances below it. When the median falls between two distances, the average of those distances is the median.

<u>Relief Ratio,</u> R_6

The longest dimension of the basin is the longest distance between two parallel lines touching the boundaries of the catchment.

$$R_6 = \frac{\text{Total Fall, } H}{\text{Longest dimension of the basin}}, \quad \text{dimensionless}$$

Calculate to 4 decimal places.

<u>Drainage Density,</u> D_d

The Drainage Density, D_d, equals the total length of Extended Streams, L_s, divided by the Catchment Area, A.

$$D_d = \frac{L_s}{A}, \quad \text{miles/square miles}$$

<u>Average Width of Catchment, W</u>

The Average Width of the Catchment, W, equals the Catchment Area, A, divided by the Length of the Main Stream, L.

$$W = A/L \quad , \quad \text{miles}$$

Form Factor, F

The Form Factor, F , equals the Average Width of the Catchment, W , divided by the Length of the Main Stream, L .

$$F = \frac{W}{L} = \frac{A}{L^2} \quad , \quad \text{dimensionless}$$

Compactness Coefficient, C

The Compactness Coefficient, C , equals 0.28 times the Perimeter, P , divided by the square root of the Catchment Area, A .

$$C = \frac{0.28\ P}{\sqrt{A}} \quad , \quad \text{dimensionless}$$

Dimensionless Mean Travel Distance, L_m

The Dimensionless Mean Travel, L_m , equals the Mean Travel Distance, L_t , divided by the square root of the Catchment Area, A .

$$L_m = \frac{L_t}{\sqrt{A}} \quad , \quad \text{dimensionless}$$

Dimensionless Standard Deviation, S_d

The Dimensionless Standard Deviation, S_d , equals the Standard Deviation, S_t , divided by the square root of the Catchment Area, A .

$$S_d = \frac{S_t}{\sqrt{A}} \quad , \quad \text{dimensionless}$$

Average Hourly Depletion Ratio, K

Plot the recession of all available flood hydrographs on semi-logarithmic paper, discharge (log-scale) vs. time (arithmetic scale).

Fit the straight line by eye to each recession line and read the discharges at the end (Q_2) and the beginning (Q_1) of time increment (Δt) equals one hour in upper part of each recession. Compute the ratio Q_2/Q_1 for each of these and then average them. Use the semi-logarithmic paper 2 cycles x 70 divisions (No. 358-61; K + Σ).

APPENDIX B

DATA AND COMPUTATIONS

TABLE B-1. TIME PARAMETERS IN HOURS

Watershed No.	Region	T_{M1} hr.	T_{M2} hr.	T_C *	T_R *
1-03-06-01	Arid	0.50	0.42	0.95	1.55
1-03-06-02	Arid	0.32	0.30	0.70	1.25
1-03-06-03	Arid	0.38	0.34	1.80	2.40
1-03-06-04	Arid	0.44	0.41	0.67	1.22
1-03-06-05	Arid	0.54	0.40	1.25	1.90
1-03-06-06	Arid	0.21	0.15	0.96	1.56
1-03-06-18	Arid	0.40	0.35	1.30	1.95
1-03-06-19	Arid	0.29	0.29	4.00	4.40
1-05-01-01	Humid	13.25	10.50	0.55	1.09
1-05-02-02	Humid	4.50	3.00	NC	NC
1-05-02-05	Humid	26.50	16.50	NC	NC
1-05-02-06	Humid	17.00	11.50	NC	NC
1-05-03-01	Humid	13.00	9.00	NC	NC
1-05-03-10	Humid	5.67	3.50	NC	NC
1-06-06-105	Humid	2.17	1.58	NC	NC
1-13-11-03	Humid	0.53	0.48	0.27	0.68
1-15-11-01	Humid	1.80	1.50	1.80	2.45
1-24-12-03	Humid	1.40	0.70	1.95	2.55
1-24-12-04	Humid	2.50	1.33	2.50	3.10
1-24-12-05	Humid	0.75	0.67	0.41	0.88
1-24-12-07	Humid	0.62	0.50	0.90	1.50
1-24-12-09	Humid	1.30	1.25	6.60	6.50
1-24-12-10	Humid	1.62	1.45	2.60	2.20

NC = Not Computed * = Computed by SCS Method

Watershed No.	Region	T_{M1} hr.	T_{M2} hr.	T_C *	T_R *
1-27-07-01	Semi-Arid	0.68	0.60	0.85	1.40
1-27-07-02	Semi-Arid	1.00	0.88	0.64	1.20
1-27-07-03	Semi-Arid	1.12	0.76	2.80	3.40
1-31-09-01	Arid	0.20	0.15	0.16	0.50
1-35-14-02	Humid	2.20	0.90	0.31	0.74
1-35-14-05	Humid	1.73	1.30	0.70	1.25
1-35-14-06	Humid	1.37	1.00	0.95	1.55
1-35-14-07	Humid	2.10	1.60	1.75	2.40
1-35-14-08	Humid	0.95	0.50	0.19	0.55
1-35-14-09	Humid	0.80	0.58	0.12	0.42
1-35-14-32	Humid	0.80	0.58	0.14	0.46
1-36-08-01	Semi-Arid	0.67	0.46	0.26	0.66
1-36-08-02	Semi-Arid	0.63	0.36	0.62	1.15
1-38-18-17	Humid	NC	NC	1.20	1.80
1-43-08-01	Arid	0.80	0.73	0.38	0.85
1-43-09-01	Semi-Arid	1.06	1.01	1.03	1.60
1-43-09-02	Semi-Arid	1.00	0.80	1.60	2.25
1-43-09-05	Semi-Arid	0.40	0.33	0.57	1.10
1-43-09-06	Semi-Arid	0.73	0.47	0.31	0.73
1-43-09-07	Semi-Arid	0.61	0.53	0.49	0.99
1-43-09-08	Semi-Arid	0.59	0.41	0.35	0.80
1-43-09-09	Semi-Arid	0.66	0.46	0.27	0.68
1-49-11-01	Humid	0.48	0.38	0.43	0.92
1-49-11-02	Humid	0.28	0.18	0.24	0.63

TABLE B-2. RAINFALL AND RUNOFF DATA

Order	Watershed No.	Event No.	API_5 in.	P_s in.	D min.	q in./hr.	P_F %	T_{R1} hrs.	T_{R2} hrs.	R_{T1}	R_{T2}	Area Class
1	1-03-06-01	1	0.59	0.50	26	0.1560	0.40	0.25	0.23	0.50	0.55	S
2		2	0.54	0.85	30	0.3266	0.67	0.75	0.50	1.50	1.19	
3		3	0.72	1.01	40	0.2426	0.55	0.50	0.42	1.00	1.00	
4		4	0	1.29	52	0.4813	0.82	0.60	0.58	1.20	1.38	
5		5	0	2.20	60	0.6284	0.91	0.50	0.48	1.00	1.14	
6	1-03-06-02	1	0	0.98	24	1.0100	0.72	0.32	0.30	1.00	1.00	L
7		2	0.75	1.56	28	1.4500	0.90	0.33	0.27	1.03	0.90	
8		3	0.08	1.20	25	0.8480	0.65	0.22	0.17	0.69	0.57	
9		4	0.27	1.01	22	1.0000	0.74	0.32	0.30	1.00	1.00	
10		5	0.60	0.71	40	0.4118	0.30	0.15	0.15	0.47	0.50	
11		6	0.23	1.47	26	1.2035	0.83	0.33	0.32	1.03	1.07	
12		7	0.47	1.30	55	0.4452	0.34	0.80	0.67	2.50	2.23	
13	1-03-06-03	1	NA	NA	NA	NA	NA	0.53	0.50	1.39	1.47	L
14		2						0.52	0.50	1.37	1.47	
15		3						0.30	0.23	0.73	0.68	
16		4						0.43	0.38	1.13	1.12	
17		5						0.32	0.30	0.84	0.88	
18		6						0.30	0.28	0.79	0.82	

NA - Not available

S = small < 1.0 sq. m.
L = large > 1.0 sq. m.

TABLE B-2. RAINFALL AND RUNOFF DATA - Continued

Order	Watershed No.	Event No.	API_5 in.	P_s in.	D min.	q in./hr.	P_F %	T_{R1} hrs.	T_{R2} hrs.	R_{T1}	R_{T2}	Area Class
19	1-03-06-04	1	NA	NA	NA	0.8443	0.64	0.50	0.48	1.14	1.17	L
20		2				2.4795	0.93	0.77	0.73	1.75	1.78	
21		3				0.3523	0.67	0.37	0.35	0.84	0.85	
22		4				0.2301	0.42	0.27	0.20	0.61	0.49	
23		5				0.3151	0.44	0.17	0.15	0.39	0.37	
24		6				0.6284	0.60	0.50	0.47	1.14	1.15	
25	1-03-06-05	1	NA	NA	NA	NA	NA	0.42	0.40	0.78	1.00	L
26		2						0.50	0.45	0.93	1.12	
27		3						0.80	0.33	1.48	0.82	
28		4						0.57	0.55	1.06	1.37	
29		5						0.72	0.70	1.33	1.75	
30		6						0.17	0.12	0.31	0.30	
31		7						0.27	0.22	0.50	0.55	
32	1-03-06-06	1	0	0.20	12	0.3603	0.74	0.10	0.10	0.48	0.67	L
33		2	0.32	1.90	32	0.3603	0.74	0.12	0.12	0.57	0.80	
34		3	0.12	2.37	39	0.4096	0.80	0.58	0.42	2.76	2.80	
35		4	0	0.96	22	0.1713	0.46	0.17	0.13	0.81	0.87	
36		5	3.71	0.98	32	0.2904	0.65	0.50	0.37	2.38	2.47	
37		6	0	0.84	30	0.1548	0.42	0.25	0.17	1.19	1.13	

TABLE B-2. RAINFALL AND RUNOFF DATA - Continued

Order	Watershed No.	Event No.	API_5 in.	P_s in.	D min.	q in./hr.	P_F %	T_{R1} hrs.	T_{R2} hrs.	R_{T1}	R_{T2}	Area Class
38	1-03-06-18	1	NA	NA	NA	NA	NA	0.75	0.70	1.88	2.00	L
39		2						0.22	0.17	0.55	0.49	
40		3						0.25	0.20	0.63	0.57	
41		4						0.50	0.48	1.25	1.37	
42		5						1.08	1.05	2.70	3.00	
43		6						0.30	0.22	0.75	0.63	
44	1-03-06-19	1	NA	NA	NA	0.0918	NA	0.25	0.25	0.86	0.86	L
45		2				0.1853		0.17	0.17	0.59	0.59	
46		3				0.0376		0.18	0.18	0.62	0.62	
47		4				0.0516		0.33	0.33	1.14	1.14	
48		5				0.1298		1.08	0.53	3.72	1.83	
49		6				0.0795		0.47	0.33	1.62	1.14	
50	1-05-01-01	1	NA	NA	NA	0.2099	NA	7.33	1.33	0.55	0.13	S
51		2				0.1834		13.25	8.50	1.00	0.81	
52		4				0.0816		22.00	17.00	1.66	1.62	
53		5				0.0550		12.00	10.50	0.91	1.00	
54		6				0.1066		21.00	20.00	1.58	1.90	
55	1-05-02-02	1	NA	1.50	1110	0.0163	NA	16.00	8.50	3.56	2.83	L
56		2		0.70	135	0.0147		1.67	0.83	0.37	0.28	
57		3		0.70	330	0.0246		5.00	3.00	1.11	1.00	

TABLE B-2. RAINFALL AND RUNOFF DATA - Continued

Order	Watershed No.	Event No.	API_5 in.	P_s in.	D min.	q in./hr.	P_F %F	T_{R1} hrs.	T_{R2} hrs.	R_{T1}	R_{T2}	Area Class
58		4	NA	1.00	390	0.0429	NA	4.50	3.83	1.00	1.28	
59		5		1.20	630	0.0517		3.50	3.00	0.78	1.00	
60	1-05-02-05	1	6.30	2.40	780	0.0282	NA	7.50	5.00	0.28	0.30	L
61		2	2.40	4.60	1650	0.0794		23.00	14.50	0.87	0.88	
62		3	0.40	5.38	3090	0.0434		14.00	12.00	0.53	0.73	
63		4	2.60	4.60	2040	0.0406		31.00	21.50	1.17	1.30	
64		5	1.70	3.00	2670	0.0298		30.00	21.00	1.13	1.27	
65		6	NA	6.20	3930	0.9001		33.00	18.50	1.24	1.12	
66	1-05-02-06	2	1.10	2.10	405	0.0371	NA	17.00	9.00	1.00	0.78	L
67		3	0.70	4.80	1620	0.0709		24.00	20.33	1.41	1.78	
68		4	3.30	2.80	1800	0.0103		28.00	25.00	1.65	2.17	
69		5	2.60	2.50	1350	0.0157		12.00	11.50	0.71	1.00	
70		6	3.60	3.20	720	0.0783		12.00	11.50	0.71	1.00	
71	1-05-03-01	1	0	5.10	1980	0.0012	NA	24.33	15.00	1.87	1.67	L
72		2	5.12	5.50	1505	0.2232		9.33	8.67	0.72	0.96	
73		3	1.60	4.00	1365	0.1440		16.33	10.83	1.26	1.20	
74		4	2.30	1.90	600	0.0960		7.00	5.67	0.54	0.63	
75		5	1.00	2.70	920	0.0480		8.00	5.42	0.62	0.60	
76		6	2.70	1.50	2425	0.0227		16.50	12.50	1.27	1.39	
77		7	1.10	3.60	1215	0.1972		13.00	9.00	1.00	1.00	

TABLE B-2. RAINFALL AND RUNOFF DATA - Continued

Order	Watershed No.	Event No.	API_5 in.	P_s in.	D min.	q in./hr.	P_F %	T_{R1} hrs.	T_{R2} hrs.	R_{T1}	R_{T2}	Area Class
78	1-05-03-10	1	NA	NA	NA	0.1240	NA	13.50	3.50	2.38	1.00	L
79		2				0.0826		5.50	3.00	0.97	0.86	
80		3				0.0751		7.50	5.50	1.32	0.97	
81		4				0.0396		3.00	2.00	0.53	0.35	
82		5				0.0258		5.67	5.00	1.00	0.88	
83	1-06-06-105	1	NA	NA	NA	0.0085	0.81	2.17	1.58	1.00	1.00	L
84		2				0.0095	0.85	1.42	1.33	0.65	0.84	
85		3				0.0099	0.86	2.83	2.67	1.30	1.69	
86		4				0.0085	0.81	2.00	1.83	0.92	1.16	
87		5				0.0086	0.82	3.83	1.08	1.76	0.68	
88	1-13-11-03	1	2.05	2.23	82	0.7000	0.94	0.53	0.48	1.00	1.00	S
89		2	0.69	1.83	160	0.4960	0.81	0.73	0.70	1.38	1.46	
90		3	0.25	2.85	82	0.2490	0.58	0.53	0.50	1.00	1.04	
91		4	1.65	0.90	40	0.2370	0.56	0.47	0.43	0.83	0.89	
92		5	0.70	1.73	100	0.3400	0.68	0.43	0.37	0.81	0.77	
93	1-15-11-01	1	0.74	1.96	61	0.4890	NA	1.50	0.80	0.83	0.53	L
94		2	0.09	1.85	120	0.3395		1.80	1.50	1.00	1.00	
95		3	0	3.23	216	0.6490		1.70	1.55	0.94	1.03	
96		4	0.46	2.94	125	0.8580		0.65	0.55	0.36	0.37	
97		5	0.30	1.89	780	0.1290		6.67	5.00	3.71	3.33	

TABLE B-2. RAINFALL AND RUNOFF DATA - Continued

Order	Watershed No.	Event No.	API_5 in.	P_s in.	D min.	q in./hr.	P_F %	T_{R1} hrs.	T_{R2} hrs.	R_{T1}	R_{T2}	Area Class
98	1-24-12-03	1	0.99	1.04	180	0.4824	0.59	1.60	0.70	1.14	1.00	L
99		2	0	1.20	NA	0.1354	0.25	1.40	0.80	1.00	1.14	
100		3	0.17	1.15	150	0.0941	0.22	0.25	0.17	0.18	0.24	
101		4	0.53	1.03	225	0.0845	0.21	2.17	1.50	1.55	2.14	
102		5	0.12	2.40	120	0.4331	0.55	0.92	0.45	0.66	0.64	
103	1-24-12-04	1	1.33	2.00	180	0.2475	0.84	1.50	1.17	0.60	0.88	L
104		2	0.86	0.59	210	0.1818	0.64	3.58	3.00	1.43	2.26	
105		3	0.99	0.49	75	0.0835	0.20	1.33	1.00	0.53	0.75	
106		4	0.32	NA	NA	0.1084	0.30	3.50	2.83	1.40	2.16	
107		5	0	1.68	180	0.0541	0.08	2.50	1.33	1.00	1.00	
108	1-24-12-05	1	NA	NA	NA	0.2734	0.40	0.75	0.67	1.00	1.00	S
109		2				0.0898	0.23	1.42	1.33	1.89	1.98	
110		3				0.1469	0.28	0.58	0.50	0.77	0.75	
111		4				0.0347	NA	2.67	1.83	3.56	0.27	
112		5				0.3017	0.42	0.47	0.30	0.63	0.45	
113	1-24-12-07	1	1.12	0.79	37	0.1331	0.26	0.28	0.20	0.45	0.40	L
114		2	2.10	1.25	88	0.2415	0.44	1.00	0.58	1.61	1.16	
115		3	1.00	2.76	80	0.5610	0.82	0.62	0.50	1.00	1.00	
116		4	0.60	0.99	240	0.0468	0.15	1.58	1.25	2.55	2.50	
117		5	2.26	1.42	75	0.1456	0.28	0.50	0.45	0.81	0.90	

TABLE B-2. RAINFALL AND RUNOFF DATA - Continued

Order	Watershed No.	Event No.	API_5 in.	P_s in.	D min.	q in./hr.	P_F %	T_{R1} hrs.	T_{R2} hrs.	R_{T1}	R_{T2}	Area Class
118	1-24-12-09	1	4.33	1.23	285	0.2826	0.50	2.00	1.25	1.54	1.00	L
119		2	0.07	1.34	435	0.0823	0.08	2.50	1.75	1.92	1.40	
120		3	0.20	1.11	150	0.0892	0.09	0.70	0.58	0.54	0.46	
121		4	0.43	1.55	540	0.2142	0.30	1.30	1.25	1.00	1.00	
122		5	0.07	1.91	135	0.2150	0.30	1.25	0.75	0.96	0.60	
123	1-24-12-10	1	4.07	1.21	195	0.2325	0.19	2.00	1.47	1.23	1.01	L
124		2	0.98	1.30	375	0.1135	0.01	1.75	1.58	1.08	1.09	
125		3	0.30	1.46	150	0.1708	0.02	1.00	0.83	0.62	0.57	
126		4	0.46	1.57	540	0.2330	0.20	1.62	1.45	1.00	1.00	
127		5	0.05	1.29	135	0.0342	0.01	1.37	0.87	0.85	0.60	
128	1-27-07-01	1	0.60	1.39	54	1.1500	0.74	0.52	0.42	0.76	0.70	S
129		2	0	2.70	294	1.7400	0.93	0.68	0.60	1.00	1.00	
130		3	0.68	1.56	97	0.7180	0.60	0.68	0.60	1.00	1.00	
131		4	1.64	1.95	112	1.8200	0.94	1.00	0.75	1.47	1.25	
132		5	0	2.26	104	0.9320	0.82	0.82	1.70	1.20	1.16	
133		6	0	1.70	85	0.1340	0.01	0.47	0.33	0.69	0.55	
134	1-27-07-02	1	1.47	1.36	66	0.2700	0.21	1.48	1.42	1.48	1.61	S
135		2	0	2.18	82	0.3230	0.37	0.87	0.82	0.87	0.93	
136		3	1.46	2.10	66	1.1500	0.93	0.97	0.88	0.97	1.00	
137		4	0	2.17	112	0.6440	0.70	1.00	0.88	1.00	1.00	
138		5	0.19	2.54	265	0.2490	0.26	1.72	1.48	1.72	1.68	

TABLE B-2. RAINFALL AND RUNOFF DATA - Continued

Order	Watershed No.	Event No.	API_5 in.	P_s in.	D min.	q in. / hr.	P_F %	T_{R1} hrs.	T_{R2} hrs.	R_{T1}	R_{T2}	Area Class
139	1-27-07-03	1	0.36	1.75	114	0.164	0.45	1.08	0.50	0.96	0.66	L
140		2	0.01	2.77	277	0.352	0.85	0.85	0.77	0.76	1.01	
141		3	0.67	1.71	99	0.264	0.73	1.17	0.67	1.04	0.88	
142		4	0.10	2.38	680	0.217	0.62	2.42	1.82	2.16	2.39	
143		5	0	2.13	61	0.266	0.73	0.83	0.75	0.74	0.99	
144		6	0.32	2.09	324	0.096	0.30	2.58	1.50	2.30	1.97	
145	1-31-09-01	1	NA	NA	NA	1.580	0.92	0.20	0.15	1.00	1.00	S
146		2				0.636	0.58	0.13	0.12	0.65	0.80	
147		3				0.652	0.59	0.27	0.23	1.35	1.53	
148		4				2.710	0.99	0.17	0.13	0.85	0.87	
149		5				0.871	0.70	0.05	0.03	0.25	0.20	
150		6				0.551	0.51	0.25	0.23	1.25	1.53	
151		7				0.324	0.36	0.35	0.17	1.75	1.13	
152	1-35-14-02	1	1.81	1.86	330	0.321	0.60	0.50	0.40	0.23	0.44	S
153		2	1.18	2.54	170	0.432	0.72	0.93	0.43	0.42	0.48	
154		3	2.72	2.08	249	1.090	0.97	2.40	1.20	1.09	1.33	
155		4	0.69	3.74	232	0.960	0.96	2.20	1.53	1.00	1.70	
156		5	1.74	1.25	87	0.275	0.54	2.40	0.90	1.09	1.00	
157	1-35-14-05	1	1.46	1.69	160	0.397	0.72	1.00	0.97	0.58	0.75	L
158		2	1.18	2.52	94	0.437	0.76	1.50	1.30	0.87	1.00	
159		3	2.72	2.12	249	0.918	0.97	1.87	1.47	1.08	1.13	

TABLE B-2. RAINFALL AND RUNOFF DATA - Continued

Order	Watershed No.	Event No.	API_5 in.	P_s in.	D min.	q in./ hr.	P_F %	T_{R1} hrs.	T_{R2} hrs.	R_{T1}	R_{T2}	Area Class
160		4	0.69	NA	NA	0.625	0.90	2.33	2.10	1.35	1.62	
161		5	1.74	↓	↓	0.503	0.82	1.73	1.00	1.00	0.77	
162	1-35-14-06	1	1.92	1.65	170	0.362	0.81	0.47	0.37	0.34	0.37	L
163		2	1.18	2.52	94	2.520	NA	1.37	1.23	1.00	1.23	
164		3	2.60	2.11	280	0.614	0.96	0.80	0.70	0.58	0.70	
165		4	0.70	3.57	337	0.411	0.85	3.50	2.67	2.55	2.67	
166		5	1.80	1.35	105	0.456	0.88	1.37	1.00	1.00	1.00	
167	1-35-14-07	1	1.48	1.19	82	0.360	NA	2.30	1.90	1.10	1.19	L
168		2	2.20	1.94	250	0.323	↓	1.95	1.55	0.93	0.97	
169		3	0	2.72	144	0.211		1.90	1.77	0.90	1.11	
170		4	2.60	2.95	467	0.724		2.10	0.95	1.00	0.59	
171		5	1.18	2.40	170	0.260		1.10	1.00	0.52	0.62	
172		6	0.70	3.11	337	0.272		3.20	2.65	1.52	1.64	
173		7	1.80	1.51	435	0.548		2.30	1.60	1.10	1.00	
174	1-35-14-08	1	1.42	3.27	100	2.580	0.91	0.57	0.35	0.60	0.70	S
175		2	0.61	4.37	144	1.760	0.80	1.17	1.00	1.23	2.00	
176		3	1.36	2.86	135	2.500	0.91	0.73	0.37	0.77	0.74	
177		4	2.12	2.25	230	1.300	0.70	6.97	1.40	7.33	2.80	
178		5	0.68	1.33	137	0.0373	0.28	1.20	0.50	1.26	1.00	
179		6	2.01	1.82	262	1.140	0.65	1.20	0.70	1.26	1.40	
180		7	2.32	1.62	82	1.410	0.73	0.73	0.43	0.77	0.86	

TABLE B-2. RAINFALL AND RUNOFF DATA - Continued

Order	Watershed No.	Event No.	API_5 in.	P_s in.	D min.	q in. /hr.	P_F %	T_{R1} hrs.	T_{R2} hrs.	R_{T1}	R_{T2}	Area Class
181		8	1.51	0.81	226	0.710	0.51	0.40	0.27	0.42	0.54	
182	1-35-14-09	1	1.60	3.20	196	1.90	0.90	0.37	0.33	0.46	0.57	S
183		2	0.82	4.39	144	1.77	0.87	1.07	1.00	1.34	1.72	
184		3	1.23	3.27	134	3.72	0.91	0.35	0.30	0.44	0.52	
185		4	1.90	1.46	75	1.06	0.71	0.50	0.45	0.63	0.78	
186		5	2.68	2.17	240	1.39	0.80	2.40	1.40	3.00	2.41	
187		6	0.61	1.57	154	0.145	0.28	1.30	0.70	1.63	1.21	
188		7	2.11	1.79	263	1.110	0.72	1.20	0.90	1.50	1.55	
189		8	0	1.23	38	0.586	0.51	0.53	0.37	0.66	0.64	
190	1-35-14-32	1	NA	2.05	157	0.165	NA	0.60	0.50	0.75	0.86	S
191		2		1.63	101	1.04		0.80	0.65	1.00	1.12	
192		3		1.18	66	0.721		0.95	0.65	1.19	1.12	
193		4		0.63	148	0.348		0.50	0.40	0.63	0.63	
194		5		3.35	98	3.14		0.80	0.20	1.00	0.34	
195		6		2.00	262	1.18		4.50	2.00	5.63	3.45	
196	1-36-08-01	1	4.56	1.30	55	0.936	0.38	0.67	0.43	1.00	0.93	S
197		2	0	3.91	260	4.520	0.90	1.13	0.67	1.69	1.46	
198		3	0.57	1.30	125	0.859	0.36	0.90	0.63	1.34	1.37	
199		4	3.45	1.01	25	0.934	0.40	0.53	0.40	0.79	0.87	
200		5	1.97	2.91	285	1.749	0.55	1.83	1.08	2.73	2.35	
201		6	4.92	1.84	190	1.240	0.44	0.67	0.50	1.00	1.09	

TABLE B-2. RAINFALL AND RUNOFF DATA - Continued

Order	Watershed No.	Event No.	API_5 in.	P_s in.	D min.	q in./hr.	P_F %	T_{R1} hrs.	T_{R2} hrs.	R_{T1}	R_{T2}	Area Class
202		7	1.14	2.51	340	1.4168	0.48	0.50	0.40	0.75	0.87	
203		8	0.48	2.29	205	1.8575	0.58	0.43	0.32	0.63	0.69	
204	1-36-08-02	1	0.40	2.79	140	2.79	0.96	0.92	0.67	1.46	1.86	S
205		2	3.49	0.92	25	0.0927	0.06	0.50	0.45	0.79	1.25	
206		3	1.77	3.15	360	1.633	0.76	1.53	0.86	2.43	2.39	
207		4	5.12	2.14	145	0.939	0.46	0.75	0.50	1.19	1.39	
208		5	1.00	2.28	190	0.998	0.50	0.50	0.42	0.79	1.17	
209		6	0.44	1.97	55	1.2552	0.62	0.43	0.30	0.68	0.83	
210	1-43-08-01	1	NA	NA	NA	1.45	0.88	0.30	0.25	0.38	0.34	S
211		2				1.17	0.82	0.90	0.73	1.13	1.00	
212		3				0.611	0.62	0.80	0.73	1.00	1.00	
213		4				1.01	0.75	0.30	0.25	0.38	0.34	
214		5				0.446	0.48	0.80	0.73	1.00	1.00	
215	1-43-09-01	1	NA	NA	NA	0.868	0.82	0.93	0.90	0.88	0.89	S
216		2				0.112	0.10	1.60	1.60	1.51	1.58	
217		3				0.566	0.58	1.20	1.13	1.13	1.12	
218		4				0.625	0.64	0.93	0.90	0.88	0.89	
219		6				0.498	0.52	0.45	0.45	0.42	0.44	
220		7				0.149	0.13	2.20	2.00	2.08	1.98	
221	1-43-09-02	1	2.07	1.65	85	0.747	NA	1.00	0.90	1.00	1.12	L

TABLE B-2. RAINFALL AND RUNOFF DATA - Continued

Order	Watershed No.	Event No.	API_5 in.	P_s in.	D min.	q in./hr.	P_F %	T_{R1} hrs.	T_{R2} hrs.	R_{T1}	R_{T2}	Area Class
222		2	3.68	1.01	85	0.322	NA	1.00	0.90	1.00	1.12	
223		3	2.46	1.81	131	0.536		1.70	0.80	1.70	1.00	
224		4	1.03	1.72	203	0.797		0.50	0.47	0.50	0.59	
225		5	2.05	0.55	62	0.670		0.53	0.43	0.53	0.54	
226		6	1.96	2.64	214	0.604		0.97	0.87	0.97	1.09	
227		7	0.12	1.60	261	0.164		1.63	0.90	1.63	1.12	
228		8	0.10	0.94	349	0.0459		0.87	0.80	0.87	1.00	
229	1-43-09-05	1	NA	NA	NA	3.40	NA	0.67	0.40	1.68	1.21	S
230		2				0.926		0.33	0.20	0.83	0.61	
231		3				2.20		0.40	0.33	1.00	1.00	
232		4				0.270		0.27	0.25	0.68	0.76	
233		5				0.132		0.73	0.63	1.83	1.91	
234	1-43-09-06	1	9.64	1.97	58	2.04	0.72	0.42	0.42	0.60	0.85	S
235		2	4.91	1.65	134	1.54	0.66	0.53	0.47	0.73	1.00	
236		3	1.84	2.13	62	1.42	0.58	0.73	0.47	1.00	1.00	
237		4	0	1.90	54	0.0459	0.10	1.07	0.73	1.47	1.55	
238		5	0.04	1.39	60	0.201	0.30	0.90	0.57	1.23	1.21	
239	1-43-09-07	1	1.72	0.51	131	0.150	0.18	0.53	0.47	0.87	0.89	S
240		2	9.72	1.76	509	1.810	0.85	0.47	0.40	0.77	0.75	
241		3	1.63	1.85	70	1.430	0.76	0.40	0.27	0.66	0.51	
242		4	1.94	1.99	59	0.661	0.44	0.97	0.63	1.59	1.19	

TABLE B-2. RAINFALL AND RUNOFF DATA - Continued

Order	Watershed No.	Event No.	API_5 in.	P_s in.	D min.	q in./hr.	P_F %	T_{R1} hrs.	T_{R2} hrs.	R_{T1}	R_{T2}	Area Class
243		5	0.02	1.46	98	0.205	0.25	1.23	0.77	2.02	1.45	
244		6	0.42	1.23	620	0.0598	0.18	0.70	0.60	1.15	1.13	
245	1-43-09-08	1	9.90	1.79	88	1.68	0.74	0.45	0.42	0.76	1.02	S
246		2	4.73	1.57	136	1.24	0.62	0.47	0.40	0.80	0.97	
247		3	1.54	1.88	38	1.79	0.76	0.40	0.30	0.68	0.73	
248		4	1.14	2.83	115	0.796	0.48	0.80	0.30	1.36	0.73	
249		5	0.03	1.46	202	0.253	0.25	1.23	0.77	0.21	1.88	
250		6	0.38	1.15	169	0.721	0.44	0.70	0.57	1.19	1.39	
251	1-43-09-09	1	10.1	1.77	78	1.61	NA	0.40	0.33	0.61	0.72	S
252		2	4.70	1.46	90	1.14		0.50	0.43	0.76	0.93	
253		3	1.48	1.85	72	1.59		0.43	0.33	0.65	0.72	
254		4	1.11	2.94	118	0.789		0.83	0.50	1.26	1.09	
255		5	0.03	1.47	73	0.325		1.13	0.53	1.71	1.15	
256		6	0.36	1.18	146	0.0622		1.07	0.87	1.62	1.89	
257	1-49-11-01	1	1.25	2.15	26	0.906	0.80	0.48	0.38	1.00	1.00	S
258		2	2.16	1.09	66	1.010	0.83	0.38	0.37	0.79	0.97	
259		3	2.90	2.16	92	0.723	0.70	0.35	0.25	0.73	0.66	
260		4	NA	NA	NA	0.438	0.50	0.50	0.45	1.04	1.18	
261		5	NA	6.98	249	1.690	0.96	1.40	1.05	2.92	2.76	

TABLE B-2. RAINFALL AND RUNOFF DATA - Continued

Order	Watershed No.	Event No.	API_5 in.	P_s in.	D min.	q in./hr.	P_F %	T_{R1} hrs.	T_{R2} hrs.	R_{T1}	R_{T2}	Area Class
262	1-49-11-02	1	1.23	2.05	25	1.21	0.89	0.32	0.08	1.14	0.44	S
263		2	0.02	1.94	54	0.362	0.48	0.17	0.15	0.61	0.83	
264		3	2.13	1.08	66	1.310	0.91	0.23	0.12	0.82	0.67	
265		4	2.87	2.39	285	1.00	0.83	0.25	0.22	0.89	1.22	
266		5	NA	NA	NA	1.06	0.86	0.45	0.25	1.61	1.39	
267		6	0.55	6.73	240	1.76	0.96	1.05	0.75	3.75	4.17	

TABLE B-3. DATA FOR SHAVER CREEK

3.75 sq. mi.

Central Pennsylvania

Storm No.	Date	Rise Time T_{R1}, hrs.	$R = \frac{T_{R1}}{\bar{\bar{T}}_{R1}}$	Peak (cfs)	Frequency %
1	3/10/43	13.0	1.15	42.90	0.24
2	5/ 6 /44	15.0	1.33	87.00	0.48
3	7/12/44	7.0	0.62	1.70	NA
6	4/17/45	13.0	1.15	11.50	0.10
7	5/17/45	7.0	0.62	103.00	0.57
8	5/28/45	7.0	0.62	22.40	0.14
9	7/ 5 /45	4.0	0.36	3.13	NA
10	9/17/45	23.0	2.04	17.00	0.12
11	11/21/45	10.0	0.89	33.00	0.18
12	11/28/45	14.0	1.24	75.00	0.42
13	3/ 9 /46	8.0	0.71	35.40	0.20
14	3/26/46	7.0	0.62	15.40	0.11
15	5/26/46	17.0	1.51	130.00	0.69
16	6/ 2 /46	12.0	1.06	72.50	0.41
17	7/ 1 /46	9.0	0.80	23.00	0.14
18	9/29/46	7.0	0.62	4.96	NA
19	10/12/46	9.0	0.80	13.50	0.10
20	10/20/46	13.0	1.15	7.20	NA
21	4/16/47	12.0	1.06	22.40	0.14
23	7/27/47	11.0	0.98	8.70	NA
24	11/ 8 /47	10.0	0.89	10.50	0.11
25	11/11/47	9.0	0.80	19.10	0.13
26	4/24/48	8.0	0.71	20.00	0.13
27	6/12/48	9.0	0.80	5.70	NA
28	7/13/48	10.0	0.89	7.30	
29	8/ 5 /48	12.0	1.06	3.25	
30	7/18/48	7.0	0.62	7.50	
31	11/ 6 /48	8.0	0.71	19.00	0.12
32	11/19/48	24.0	2.13	16.70	0.11
33	12/ 6 /48	10.0	0.89	19.60	0.12
34	1/ 5 /49	20.0	1.78	33.70	0.19
35	2/19/49	13.0	1.15	18.60	0.12
36	3/23/49	15.0	1.33	7.75	0.16
37	4/13/49	13.0	1.15	29.00	0.17
38	5/ 2/49	11.0	0.98	12.40	0.10

TABLE B-3. DATA FOR SHAVER CREEK - Continued

Storm No.	Date	Rise Time T_{R1}, hrs.	$R = \frac{T_{R1}}{\bar{\bar{T}}_{R1}}$	Peak (cfs)	Frequency %
39	5/22/49	7.0	0.62	19.40	0.12
40	7/13/49	11.0	0.98	11.85	0.10
41	7/17/49	7.0	0.62	28.70	0.16
42	8/29/49	12.0	1.06	4.80	NA
43	10/30/49	13.0	1.15	7.45	NA
44	5/18/50	19.0	1.69	12.30	0.11
45	6/ 4 /50	15.0	1.33	15.50	0.11
46	7/ 5 /50	4.0	0.35	10.20	0.10
47	7/10/50	7.0	0.62	6.05	NA
48	8/ 2 /50	4.0	0.35	7.30	NA
50	10/28/50	10.0	0.89	16.30	0.11
51	11/26/50	25.0	2.22	397.00	0.99
52	12/ 4 /50	12.0	1.06	71.30	0.40
53	3/30/51	8.0	0.71	152.00	0.76
54	4/22/51	15.0	1.33	32.10	0.18
55	4/30/51	9.0	0.80	25.00	0.15
56	9/ 6 /51	7.0	0.62	3.75	NA
57	11/14/51	6.0	0.53	1.50	
58	11/16/51	8.0	0.71	1.96	
59	12/ 5 /51	13.0	1.15	15.10	0.11
60	2/ 3 /52	18.0	1.60	26.00	0.15
61	3/11/52	14.0	1.24	150.00	0.76
62	4/ 5 /52	19.0	1.69	29.60	0.17
63	4/25/52	13.0	1.15	19.10	0.12
64	5/12/52	12.0	1.06	15.20	0.11
65	7/18/52	3.0	0.27	4.70	NA
66	8/18/52	4.0	0.35	3.50	NA
67	12/10/52	16.0	1.42	56.30	0.31
68	3/ 4 /53	10.0	0.89	13.40	0.11
69	3/15/53	8.0	0.71	15.30	0.11
70	3/24/53	14.0	1.24	155.00	0.77
71	4/ 7 /53	13.0	1.15	15.50	0.11
72	4/18/53	15.0	1.33	15.60	0.11
73	11/22/53	6.0	0.53	4.96	NA
74	12/ 7 /53	17.0	1.51	5.59	NA
75	2/20/54	13.0	1.15	15.10	0.11
76	3/ 1 /54	10.0	0.89	170.00	0.13
77	6/ 4 /54	7.0	0.62	19.60	NA

TABLE B-3. DATA FOR SHAVER CREEK - Continued

Storm No.	Date	Rise Time T_{R1}, hrs.	$R = \frac{T_{R1}}{\overline{T}_{R1}}$	Peak (cfs)	Frequency %
78	7/ 4 /54	7.0	0.62	1.62	NA
79	10/15/54	13.0	1.15	155.00	0.77
80	12/14/54	13.0	1.15	22.40	0.15
81	4/21/54	8.0	0.71	22.90	0.15
82	6/ 8 /55	17.0	1.51	14.00	0.11
83	7/ 1 /55	5.0	0.44	2.30	NA
85	8/13/55	17.0	1.51	43.50	0.24
86	8/18/55	14.0	1.24	5.90	NA
88	9/23/55	12.0	1.06	2.20	NA
89	10/14/55	13.0	1.15	90.00	0.50
90	11/16/55	9.0	0.80	34.10	0.20
91	2/25/56	13.0	1.15	30.80	0.18
92	5/ 2 /56	11.0	0.98	18.30	0.12
93	5/ 6 /56	6.0	0.53	31.40	0.18
94	6/23/56	7.0	0.62	43.70	0.24
95	7/ 2 /56	13.0	1.15	31.90	0.14
98	9/ 6 /56	9.0	0.80	20.00	0.13
99	10/ 4 /56	13.0	1.15	4.46	NA
100	10/22/56	14.0	1.24	49.60	0.17
101	11/ 2 /56	14.0	1.24	18.70	0.12
102	11/22/56	10.0	0.89	8.20	NA
103	12/14/56	18.0	1.60	24.00	0.15
104	4/ 2 /57	13.0	1.15	28.40	0.16
105	4/26/57	4.0	0.35	36.10	0.20
106	10/24/57	11.0	0.98	2.00	NA
107	12/26/57	6.0	0.53	32.70	0.19
109	6/ 1 /58	10.0	0.89	5.82	NA
110	7/22/58	21.0	1.86	3.00	NA
111	8/ 3 /58	6.0	0.53	10.05	0.09
112	9/21/58	17.0	1.51	4.65	NA
113	3/ 6 /59	11.0	0.98	12.50	0.10
114	4/10/59	8.0	0.71	15.00	0.11
115	8/ 5 /59	5.0	0.44	3.10	NA
116	10/ 1 /59	13.0	1.15	4.90	NA
118	11/ 6 /59	9.0	0.80	29.00	0.17
119	11/27/59	12.0	1.06	80.00	0.44
120	12/12/59	21.0	1.86	38.00	0.21
121	1/ 2 /60	12.0	1.06	24.00	0.15

TABLE B-3. DATA FOR SHAVER CREEK - Continued

Storm No.	Date	Rise Time T_{R1}, hrs.	$R = \frac{T_{R1}}{\overline{T_{R1}}}$	Peak (cfs)	Frequency %
122	11/ 1 /60	13.0	1.15	5.40	NA
123	11/29/60	17.0	1.51	4.90	NA
124	2/26/61	11.0	0.98	209.00	0.88
125	4/16/61	8.0	0.71	37.00	0.21
126	4/22/61	8.0	0.71	33.00	0.19
127	5/16/61	8.0	0.71	16.70	0.12
128	7/13/61	2.0	0.18	3.50	NA
129	7/15/61	4.0	0.35	11.00	0.10
132	11/16/61	16.0	1.42	6.00	NA
133	11/23/61	16.0	1.42	15.00	0.11
134	4/ 7 /62	23.0	2.04	73.00	0.41
135	8/ 9 /62	3.0	0.27	3.70	NA
137	4/18/63	12.0	1.06	8.50	NA
138	4/29/63	14.0	1.24	13.60	0.11
139	5/18/63	14.0	1.24	44.00	0.20
140	7/ 2 /63	4.0	0.35	2.70	NA
141	8/ 4 /63	4.0	0.35	4.30	↓
142	11/ 6 /63	13.0	1.15	7.50	
143	11/23/63	11.0	0.98	2.10	
144	11/29/63	15.0	1.33	17.60	0.12
145	12/ 8 /63	15.0	1.33	4.10	NA
146	3/10/64	13.0	1.15	105.00	0.58
147	8/ 3 /64	9.0	0.80	10.00	0.09
148	5/21/46	9.0	0.80	35.70	0.15
150	12/12/49	9.0	0.80	42.00	0.23
151	11/20/50	15.0	1.33	20.10	0.13
152	12/ 7 /50	11.0	0.98	54.00	0.30
153	5/ 8 /54	11.0	0.98	23.00	0.15
154	4/17/54	12.0	1.06	25.00	0.15

$\Sigma = 1576.0$

Total Number of Storms = 140

$$\overline{T_{R1}} = \frac{1576.0}{140} = 11.26 \text{ hrs.}$$

NA - Not available

TABLE B-4. DATA FOR COMPUTATION OF RELATIVE RISE TIMES DISTRIBUTION

All Watersheds

Order Number	Class Interval	Observed Frequency (0)	Cumulative Frequency	Cumulative Probability
1	0.11-0.30	8	8	0.020
2	0.31-0.50	27	35	0.086
3	0.51-0.70	57	92	0.226
4	0.71-0.90	78	168	0.413
5	0.91-1.10	82	252	0.619
6	1.11-1.30	58	310	0.762
7	1.31-1.50	31	341	0.838
8	1.51-1.70	27	368	0.904
9	1.71-1.90	12	380	0.934
10	1.91-2.10	5	385	0.946
11	2.11-2.30	4	389	0.956
12	2.31-2.50	4	393	0.966
13	2.51-2.70	3	396	0.973
14	2.71-2.90	2	398	0.978
15	2.91-3.10	2	400	0.983
16	3.11-3.30	0		
17	3.31-3.50	0		
18	3.51-3.70	2	402	0.988
19	3.71-3.90	3	405	0.995
20	5.51-5.70	1	406	0.998
21	7.31-7.50	1	407	1.000
		407		

TABLE B-5. DATA FOR COMPUTATION OF RELATIVE RISE TIMES DISTRIBUTION

Shaver Creek - 3.75 Sq. Mi.

Order Number	Class Interval	Observed Frequency (0)	Cumulative Frequency	Cumulative Probability	Expected Frequency (E)	(0-E)	$(0-E)^2$	$\frac{(0-E)^2}{E}$
1	0.11-0.30	3	3	0.021	1	2	4	4.00
2	0.31-0.50	9	12	0.086	12	3	9	0.75
3	0.51-0.70	19	31	0.221	25	6	36	1.44
4	0.71-0.90	31	62	0.443	28	3	9	0.32
5	0.91-1.10	22	84	0.600	22	0	0	0
6	1.11-1.30	28	112	0.800	20	8	64	3.20
7	1.31-1.50	11	123	0.879	12	1	1	0.08
8	1.51-1.70	10	133	0.950	9	1	1	1.11
9	1.71-1.90	3	136	0.971	4	1	1	0.25
10	1.91-2.10	2	138	0.986	4	2	4	1.00
11	2.11-2.30	2	140	1.000	3	1	1	0.33
		140			140			12.48

TABLE B-6. DATA FOR COMPUTATION OF RELATIVE RISE TIMES DISTRIBUTION

Watersheds Smaller Than 1.0 Sq. Mi.

Order Number	Class Interval	Observed Frequency (0)	Cumulative Frequency	Cumulative Probability	Expected Frequency (E)	(0-E)	$(0-E)^2$	$\frac{(0-E)^2}{E}$
1	0.11-0.30	3	3	0.023	1	2	4	4.00
2	0.31-0.50	8	11	0.085	10	2	4	0.40
3	0.51-0.70	17	28	0.217	22	5	25	1.13
4	0.71-0.90	26	54	0.419	25	1	1	0.004
5	0.91-1.10	25	79	0.612	20	5	25	1.25
6	1.11-1.30	15	94	0.729	18	3	9	0.50
7	1.31-1.50	11	105	0.814	10	1	1	0.10
8	1.51-1.70	9	114	0.884	8	1	1	0.12
9	1.71-1.90	5	119	0.922	4	1	1	0.25
10	1.91-2.10	2	121	0.938	4	2	4	1.00
11	2.11-2.30	0			3	3	9	3.00
12	2.31-2.50	1	122	0.946	1	0	0	0
13	2.51-2.70	0			1	1	1	1.00
14	2.71-2.90	1	123	0.953	1	0	0	0
15	2.91-3.10	2	125	0.969	1	1	1	1.00
16	3.11-3.30	0			0	0	0	
17	3.31-3.50	0			0	0	0	
18	3.51-3.70	1	126	0.977	0	1	1	
19	3.71-3.90	1	127	0.984	0	1	1	
20	5.51-5.70	1	128	0.992	0	1	1	
21	7.31-7.50	1	129	1.000	0	1	1	
		$\Sigma =$ 129			129			13.754

TABLE B-7. DATA FOR COMPUTATION OF
RELATIVE RISE TIMES DISTRIBUTION

Watersheds Larger Than 1.0 Sq. Mi.

Order Number	Class Interval	Observed Frequency (0)	Cumulative Frequency	Cumulative Probability	Expected Frequency (E)	(0-E)	$(0-E)^2$	$\frac{(0-E)^2}{E}$
1	0.11-0.30	2	2	0.015	1	1	1	1.00
2	0.31-0.50	10	12	0.087	11	1	1	0.09
3	0.51-0.70	21	33	0.239	24	3	9	0.37
4	0.71-0.90	21	54	0.391	26	5	25	0.96
5	0.91-1.10	35	89	0.645	21	14	196	9.33
6	1.11-1.30	15	104	0.754	19	4	16	0.84
7	1.31-1.50	9	113	0.819	11	2	4	0.36
8	1.51-1.70	8	121	0.877	8	0	0	0
9	1.71-1.90	4	125	0.906	4	0	0	0
10	1.91-2.10	1	126	0.913	4	3	9	2.25
11	2.11-2.30	2	128	0.928	3	1	1	0.33
12	2.31-2.50	3	131	0.949	1	2	4	4.00
13	2.51-2.70	3	134	0.971	1	2	4	4.00
14	2.71-2.90	1	135	0.978	1	0	0	0
15	2.91-3.10	0			1	1	1	1.00
16	3.11-3.30	0			0	0	0	
17	3.31-3.50	0			0	0	0	
18	3.51-3.70	1	136	0.985	0	1	1	
19	3.71-3.90	2	138	1.000	0	2	4	
		138			138			21.53

TABLE B-8. DATA FOR EVALUATION OF SCS METHOD

Order Number	Region	Observed T_M, Hrs. (y)	Estimated T_R, Hrs. (x)	T_R^2 (x^2)	$T_M T_R$ (x y)	$0.142\ T_R$	$T'_M = 0.656 + 0.142\ T_R$	T_M^2	$\lvert T_M - T_R \rvert$	$\frac{\lvert T_M - T_R \rvert}{T_M}$	T_C	$T_M - T_C$	$\frac{\lvert T_M - T_C \rvert}{T_M}$
1	A *	0.50	1.55	2.40	0.77	0.220	0.876	0.250	1.05	2.10	0.95	0.45	0.90
2	A	0.32	1.25	1.56	0.40	0.177	0.833	0.102	0.93	2.91	0.70	0.38	1.19
3	A	0.38	2.40	5.76	0.91	0.341	0.997	0.144	2.02	5.32	1.80	1.42	3.74
4	A	0.44	1.22	1.49	0.54	0.173	0.829	0.194	0.78	1.77	0.67	0.23	0.52
5	A	0.54	1.90	3.61	1.03	0.270	0.926	0.292	1.46	2.70	1.25	0.71	1.31
6	A	0.21	1.56	2.43	0.33	0.221	0.877	0.044	1.35	1.12	0.96	0.75	3.57
7	A	0.40	1.95	3.80	0.78	0.277	0.933	0.160	1.55	3.87	1.30	0.90	2.25
8	A	0.20	4.40	19.36	1.28	0.625	1.281	0.084	4.11	14.17	4.00	3.71	12.79
9	H **	0.53	0.68	0.46	0.36	0.097	0.753	0.281	0.15	0.28	0.27	0.26	0.49
10	H	1.80	2.45	6.00	4.41	0.348	1.004	3.240	0.65	0.36	1.80	0.00	0.00
11	H	1.40	2.55	6.50	3.57	0.362	1.018	1.960	1.15	0.82	1.95	0.55	0.39
12	H	2.50	3.10	9.61	7.75	0.440	1.096	6.250	0.60	0.24	2.50	0.00	0.00
13	H	0.75	0.88	0.77	0.66	0.125	0.781	0.562	0.13	0.17	0.41	0.34	0.45
14	H	0.62	1.50	2.25	0.93	0.213	0.869	0.384	0.88	1.42	0.90	0.28	0.45
15	H	1.30	6.50	42.25	8.45	0.923	1.579	1.690	5.20	4.00	6.60	0.70	0.54
16	H	1.62	2.20	4.84	3.56	0.312	0.968	2.624	0.58	0.36	2.60	0.98	0.60
17	SA ***	0.68	1.40	1.96	0.95	0.199	0.855	0.462	0.72	1.06	0.85	0.17	0.25
18	SA	1.00	1.20	1.44	1.20	0.170	0.826	1.000	0.20	0.20	0.64	0.36	0.36
19	SA	1.12	3.40	11.56	3.81	0.483	1.139	1.254	2.28	2.04	2.80	1.68	1.50
20	A	0.20	0.50	0.25	0.10	0.071	0.827	0.040	0.30	1.50	0.16	0.04	0.20
21	H	2.20	0.74	0.55	1.63	0.105	0.761	4.840	1.46	0.66	0.31	1.89	0.86
22	H	1.73	1.25	1.56	2.16	0.177	0.833	2.993	0.48	0.28	0.70	1.03	0.59
23	H	1.37	1.55	2.40	2.12	0.220	0.876	1.877	0.18	0.13	0.95	0.42	0.31
24	H	2.10	2.40	5.76	5.04	0.338	0.994	4.410	0.30	0.14	1.75	0.35	0.17
25	H	0.95	0.55	0.30	0.52	0.078	0.734	0.902	0.40	0.42	0.19	0.76	0.80
26	H	0.80	0.42	0.18	0.34	0.060	0.716	0.640	0.38	0.47	0.12	0.68	0.85
27	H	0.80	0.46	0.21	0.37	0.065	0.721	0.640	0.34	0.42	0.14	0.66	0.83
28	SA	0.67	0.66	0.44	0.44	0.094	0.750	0.449	0.01	0.01	0.26	0.41	0.61
29	SA	0.63	1.15	1.32	0.72	0.163	0.819	0.397	0.52	0.82	0.62	0.01	0.01
30	A	0.80	0.85	0.72	0.68	0.121	0.777	0.640	0.05	0.06	0.38	0.42	0.53
31	SA	1.06	1.60	2.56	1.70	0.227	0.883	1.124	0.54	0.51	1.03	0.03	0.03
32	SA	1.00	2.25	5.06	2.25	0.319	1.075	1.000	1.25	1.25	1.60	0.60	0.60
33	SA	0.40	1.10	1.21	0.44	0.156	0.812	0.160	0.70	1.75	0.57	0.17	0.43
34	SA	0.73	0.73	0.53	0.53	0.104	0.760	0.533	0.00	0.00	0.31	0.42	0.58
35	SA	0.61	0.99	0.98	0.60	0.141	0.797	0.372	0.38	0.62	0.49	0.12	0.20
36	SA	0.59	0.80	0.64	0.47	0.114	0.770	0.348	0.21	0.36	0.35	0.24	0.41
37	SA	0.66	0.68	0.46	0.45	0.097	0.653	0.436	0.02	0.03	0.27	0.39	0.59
38	H	0.48	0.92	0.85	0.44	0.131	0.787	0.230	0.44	0.92	0.43	0.05	0.10
39	H	0.28	0.63	0.40	0.18	0.089	0.745	0.078	0.35	1.25	0.24	0.04	0.14
		34.32	62.32	154.43	62.87			43.086		56.51			40.14

* Arid   ** Humid   *** Semi-Arid

$$\bar{T}_M = \frac{34.46}{39} = 0.883$$

TABLE B-9. DATA FOR EVALUATION OF SCS METHOD

$$\overline{T}_M = \frac{34.46}{39} = 0.883$$

$$a_o = \frac{(\Sigma y)(\Sigma x^2) - (\Sigma x)(\Sigma xy)}{N\Sigma x^2 - (\Sigma x)^2} = \frac{(34.46)(154.43) - (62.32)(62.87)}{39(154.43) - (62.32)^2} = 0.656$$

$$a_1 = \frac{N\Sigma xy - (\Sigma x)(\Sigma y)}{N\Sigma x^2 - (\Sigma x)^2} = \frac{39(62.87) - 62.32(34.46)}{39(154.43) - (62.32)^2} = 0.1423$$

$$T'_M = a_o + a_1 T_R = 0.656 + 0.142 T_R$$

Average Error of Estimate Rise Times by SCS Method

$$\text{Error} = \frac{56.51}{39} \approx 145\,\%$$

Average Error of Estimate Rise Times by T_C, SCS Method

$$\text{Error} = \frac{40.14}{39} \approx 103\,\%$$

Coefficient of Determination:

$$R^2 = \frac{[N\Sigma xy - (\Sigma x)(\Sigma y)]^2}{[N\Sigma x^2 - (\Sigma x)^2][N\Sigma y^2 - (\Sigma y)^2]}$$

$$R^2 = \frac{[39(62.87) - (62.32)(34.46)]^2}{[39(154.43) - (62.32)^2][39(43.086) - (34.46)^2]} = 0.0879$$