TEXAS
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BOARD

REPORT 23

Study of Some Effects of Urbanization on Storm Runoff From A Small Watershed

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TEXAS WATER DEVELOPMENT BOARD

REPORT 23

A STUDY OF SOME EFFECTS OF URBANIZATION ON STORM RUNOFF FROM A SMALL WATERSHED

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Prepared in the Center for Research in Water Resources at
The University of Texas
in cooperation with the
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FOREWORD

On September 1, 1965, the Texas Water Commission (before February 1962, the State Board of Water Engineers) experienced a far-reaching realignment of functions and personnel, directed toward the increased emphasis for planning and developing Texas' water resources and for administering water rights.

Realigned and concentrated in the Texas Water Development Board were the investigative, planning, development, research, financing, and supporting functions, including the reports review and publication functions. The name Texas Water Commission was changed to Texas Water Rights Commission, and the responsibility for functions relating to water-rights administration was vested therein.

The then Texas Water Commission in 1964 and 1965 supported cooperatively with the Bureau of Engineering Research and the Center for Research in Water Resources at The University of Texas the studies reported herein. This report is based on the doctoral dissertation of William H. Espey, Jr., BSCE, MSCE, presented in August 1965 to the University of Texas Graduate School in partial fulfillment of the Doctor of Philosophy degree requirements.

Dr. Espey has planned a related study to develop procedures and nomographs for use with selected storm and runoff criteria which can provide flood hydrographs to be utilized in the design of flood-control and floodwater-conveyance facilities (sensitive to various degrees of urban development within watersheds having drainage areas of 10 square miles and less). These relationships would be derived from basic data and analytical material for 11 rural and 22 urban watersheds assembled and developed for this report.

The Texas Water Development Board thanks the authors for providing valuable data and analyses important to water resource planning.

TEXAS WATER DEVELOPMENT BOARD

John J. Vandertulip

John J. Vandertulip
Chief Engineer

PREFACE

During the past several decades the American society has changed from that of a predominantly rural society to that of the complex urban society of today. Centralization of large populations into relatively small areas has given rise to many complex problems of an economic, social, political or physical nature. Because of urban development, increased demands are made on man's surrounding environment, and consequently the problems of urbanization cover a broad spectrum. The research reported herein is concerned with only one of the many urban problems, and that is, with the effects of rubanization on the runoff characteristics of a small watershed.

The University of Texas has had a long interest in the effects of urbanization on the hydrologic characteristics of small watersheds. In 1963, the Bureau of Engineering Research at The University of Texas provided funds to evaluate the effects of urbanization on the Waller Creek watershed in Austin, Texas. This study was under the directorship of Dr. Carl W. Morgan, Associate Professor of Civil Engineering, The University of Texas. In 1964, the Texas Water Commission also desiring further knowledge of the effects of urbanization on the runoff characteristics of small watersheds entered into an Inter-Agency Contract with the Center for Research in Water Resources at The University of Texas to continue the study on Waller Creek. This project has been under the direction of Dr. Frank D. Masch, Associate Professor of Civil Engineering, The University of Texas.

The authors wish to express their appreciation to Drs. J. J. McKetta and W. A. Cunningham of the Bureau of Engineering Research at The University of Texas and to Messrs. John J. Vandertulip and Louis L. McDaniels of the Texas Water Commission for their support of this study. Special acknowledgements are due to Professors W. L. Moore,

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This report also has been given distribution as a Technical Report through the Center for Research in Water Resources and the Hydraulic Engineering Laboratory at The University of Texas.

ABSTRACT

The evaluation of the effects of urbanization on the runoff characteristics of a small watershed is a problem that can be studied by either a short-range or a long-range investigation. Because the long-range type of investigation would require several years for hydrologic data accumulation, it cannot provide any immediate information on the changes in watershed behavior arising as a result of urbanization. A short-range investigation, however, based on synthetic evaluation of present data would provide immediate answers. It is in the realm of this short-range objective that this study of a small urban watershed is directed.

This study was made to evaluate the various effects of urbanization on the hydrologic characteristics of a small urban watershed located within Austin, Texas. A linear regression analysis of data from twenty-four urban and eleven rural watersheds was used to derive equations which would evaluate the past rural conditions and predict future urban conditions for the Waller Creek watershed. The Waller Creek watershed contains two streamflow stations. One is located at 38th Street and the other at 23rd Street, gaging areas of 2.31 square miles and 4.13 square miles respectively. The watershed above 38th Street is relatively undeveloped when compared to the lower portion of the watershed located between the two stations. The lower portion has extensive residential development and some channel improvement. Results indicate that urban development in the Waller Creek watershed has caused extensive changes in the discharge hydrograph and runoff yield for the watershed. Prediction of the effects of future development indicate the same trend. The time sequence of the discharge hydrograph will be shortened, the peak discharge will be increased and the unit yield (in/mi^2) will be increased.

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Chapter I

INTRODUCTION

During the past several years there has been an increased need for engineering data on the hydrology of small urban watersheds. The importance of this need is emphasized by the constantly increasing cost of, and demand for, adequate urban drainage facilities. Millions of dollars are being spent annually by federal, state, and local agencies, yet the engineering designs for these expenditures are often of necessity based on meager hydrologic data. The annual losses can range from such temporary inconveniences as travel delays, power failures, and minor flooding, to extensive damage of highly valuable property from inundation. Storm drainage facilities are often expensive because of the large capacity required. For example, Los Angeles County has spent 179 million dollars on storm drains to relieve local flooding, and now finds that it needs additional storm drains costing about a billion dollars to provide adequate relief from local floods and to protect as yet undeveloped areas (Engineering News-Record, 1958). In Tacoma, Washington the lack of adequate storm drainage systems has resulted in limited development of the western part of the city (Brown and Caldwell, 1957). The floods of April-June, 1952, in Salt Lake City, Utah again exemplify the problem of drainage design in urban areas. In this case, the major trans-city tributaries of the Jordan River had been piped underground. The smallest streams were eliminated entirely, and the flows of the next larger streams were placed in small culverts. The culverts were inadequate to pass the storm runoff and extensive flooding occurred resulting in serious property losses. Development high on the hills surrounding Caracas, Venezuela is still another example where urbanization has greatly increased the runoff from rains resulting in a high flood potential. To meet this problem it was necessary to build a new type of channel cross-section to pass the Rio Guaire through the city (Civil Engineering, September 1962).

In order to focus attention on the problems of urban hydrology, the American Society of Civil Engineers has established a Task Force (1964) on the "Effects of Urban Development on Flood Discharge." This Task Force has as a part of its purpose

> to seek out information pertaining to changes in runoff characteristics of watersheds due to urban development and to the effects of such changes on the concentration of flood waters in stream channels, ...

As noted in the Progress Report of this Task Force, the urban population of the United States may represent three-fourths of the total population by 1980, and possibly as much as four-fifths of the population by the year 2000. The 1960 urban population of 12.5 million occupied an area of 21.4 million acres. Urban populations estimated at 193 million for 1980 and 219 million for 2000 will occupy urban land areas of 32 million and 45 million respectively. Therefore the problem of urbanization appears to be a localized problem when viewed from the standpoint of the national land area (year 2000, urban land area only 2.4%); but as stated by the Task Force,

> It is in this limited area that some 80 percent of our population will live and where the bulk of our economic wealth will be situated. Recognizing that it is in the realm of protecting life and property that the flood control program operates, it is obvious that it is in this same limited land area that most flood control development will occur. Hence the need for greater insight and understanding of the effects of urban development on the flood flows against which protection must be provided.

The Committee on Surface Drainage of Highways of the Highway Research Board (1962) also considers the hydrology of small rural and urban watersheds one of the major problems in highway drainage. Because the lack of basic data on small watersheds is nationwide and because the aggregate cost of small drainage structures is about equal to the aggregate cost of all bridges, the committee classified the hydrology small watersheds as one of their drainage problems most in need of research.

The U. S. Geological Survey, recognizing the need for more basic data on the hydrology of urban watersheds, has established several cooperative programs throughout the country. Summarized in Table 1 are some of these current programs.

City of Philadelphia, Pennsylvania
City of Houston, Texas
City of Dallas, Texas (Gilbert, 1963)*
City of Alexandria, County of Fairfax, Virginia
City of Nashville, County of Davidson, Tennessee
City of Champaign-Urbana, Illinois(Chow,1952;Schmidt,1950)
City of Baton Rouge, Louisiana (Cox, 1940)
County of Nassau, Nassau County Department Public Works, New York (Sawyer, 1961)
County of Maricopa, Flood Control District, Arizona
County of Macomb, Southeastern Oakland Sewage Disposal District, Michigan (Wiitala, 1961)
Menlo Park District, California

TABLE 1. Some Current Urban Hydrology Programs - U.S.G.S.

The U. S. Bureau of Public Roads anticipates initiation of research on urban runoff relative to storm drain design in the Fiscal Year 1965. The Indiana Flood Control and Water Resources Commission - Purdue University research study on "Urban Hydrology for Selected Sites in Indiana" was scheduled to begin in September 1964.

Since 1949 a storm drainage research project at Johns Hopkins University has been in progress sponsored jointly by Baltimore City, Baltimore County, the State of Maryland and the U. S. Bureau of Public Roads. This study is primarily concerned with

* References are listed when available.

the design aspects of urban hydrology.

Other urban hydrology projects are in progress at the Taft Sanitary Engineering Center, Cincinnati, Ohio and at the University of New South Wales, Sydney, Australia.

The Engineering Foundation in cooperation with the American Society of Civil Engineers' Research Council on Urban Hydrology is sponsoring a conference on "Urban Hydrology Research" to be held at Proctor Academy, Andover, New Hampshire, during the week of August 9-13, 1965. The Conference will discuss the need for research on the variety of problems inherent in providing storm drainage facilities in areas of fast growing urban concentrations.

A. LITERATURE REVIEW

The extent to which urbanization alters the hydrologic performance of a watershed is difficult to evaluate because runoff data are usually not available before the encroachment of urbanization. Because of this lack of data on watersheds prior to urbanization, two general types of studies have resulted. The first involves the use of synthetic methods to predict the hydrologic conditions of the watershed prior to urban development. The second involves a direct comparison between an existing urban and a rural watershed which are assumed to be hydrologically similar except for the effects of urbanization.

Both the synthetic method and the direct comparison of different watersheds require that certain hydrologic properties be selected as a basis for evaluating urban effects. Most of the previous investigations have been concerned with the effects of urbanization on hydrograph characteristics such as the lag time or the peak discharge.

The peak discharge has been defined in terms of the unit hydrograph or the mean annual flood. Previous research on the effects of urbanization on these hydrologic properties are discussed separately in order to simplify their presentation.

1. Lag Time. Most investigators have used lag time as a measure of the effects of urbanization on the time characteristics of runoff. Carter (1961) presented the first comprehensive study of the effects of urbanization on lag time in which he defined the lag time, T_3*, as the time from the center of mass of rainfall excess to the center of mass of runoff. By determining the lag time for 22 streams in the Washington, D. C. area, Carter found lag time to be a function of the ratio, $L/\sqrt{s}$, where L is the total length of the main channel to the rim of the basin, in miles, and s is the weighted slope of the main stream channel expressed in feet per mile. Curves presented by Carter are shown in Figure 1. The upper curve represents the relation for natural undeveloped areas in the Piedmont Province near Washington; the middle curve represents the relation for basins that are partially sewered but with principal stream channels maintained in their natural condition; and the lower curve represents the relation for basins that are completely sewered with all natural channels eliminated. Based on the natural basin curve, when a watershed becomes partially sewered the lag time is reduced approximately 60 percent and when it becomes completely sewered the lag time is reduced approximately 80 percent.

Wiitala (1961) further studied the relationships derived by Carter for two small watersheds near Detroit, Michigan. One watershed was rural, Plum Brook (22.9 square miles); and the other was urban, Red Run (36.5 square miles), completely

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* Lag times are subscripted because of the numerous definitions used throughout the literature. The various definitions of lag time are summarized in detail in Table 6, page 39.

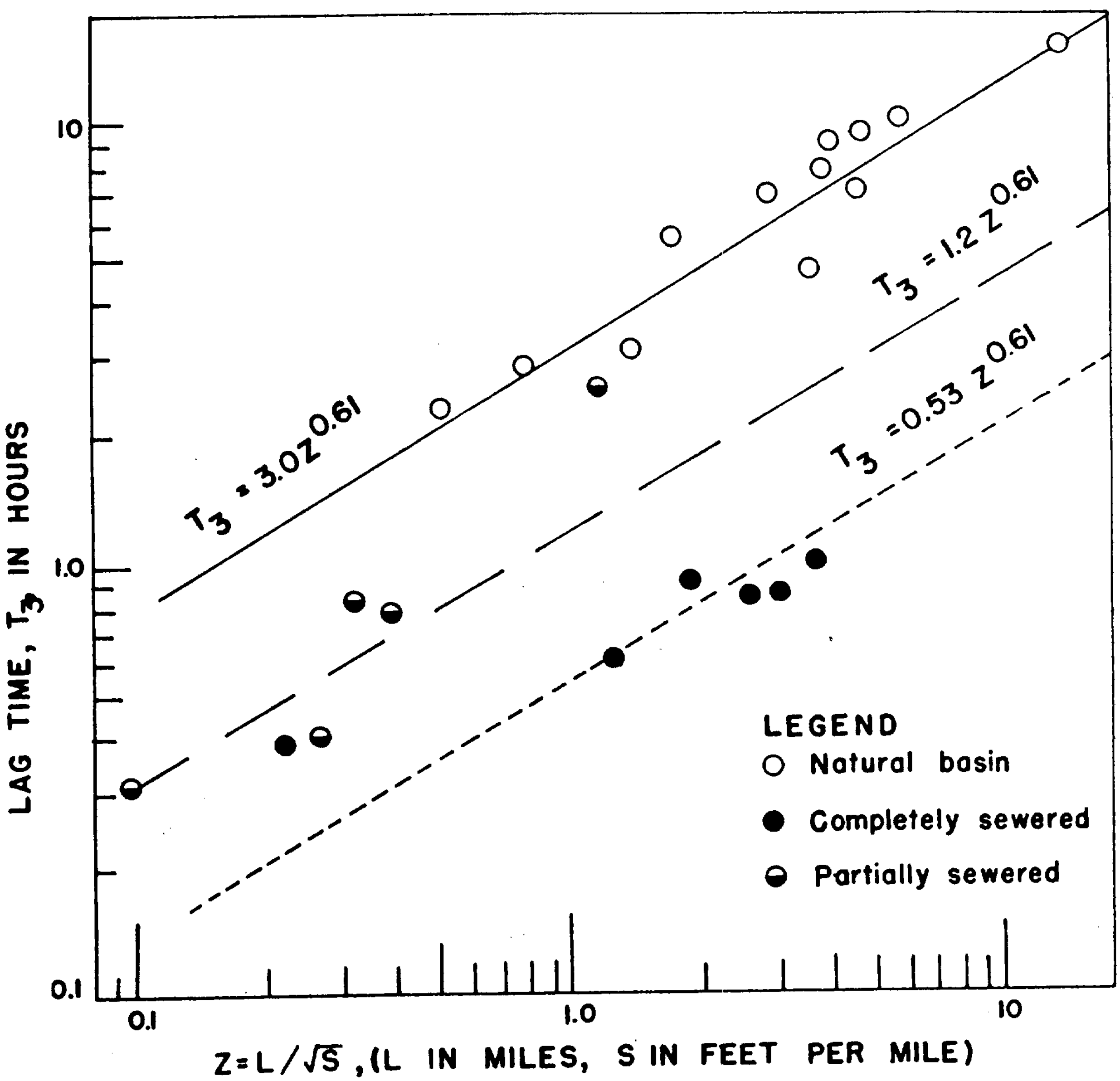


FIGURE 1. LAG TIME, T_3, VS $L/\sqrt{S}$ (after Carter, 1961)

sewered and contained approximately 25 percent impervious cover. Wiitala found that the lag time for Red Run was reduced approximately 70 percent because of urbanization.

Linsley, Kohler and Paulhus (1958) presented a correlation of the lag time, T_1, in terms of a geometrical parameter of the watershed having the form $LL_{ca}/\sqrt{s}$, where L and s are the same as previously defined, the lag time is T_1, the time from beginning of rainfall to the centroid of runoff, and L_{ca} is defined as the distance, in feet, measured along the main drainage channel from the point of interest to a point opposite the computed centroid of the drainage area. Curves having the same slope are given for natural drainage areas in mountainous terrain, in foothills, and in valleys of California. Eagleson (1962) extended these curves to include five small urban watersheds in Louisville, Kentucky. From Figure 2, it is seen that urbanization causes reductions in lag time of 86 percent, 78 percent and 49 percent when compared to the lag times of mountainous, foothill, and valley watersheds respectively. Eagleson's urban relationship is based on data from watersheds having impervious cover greater than 30 percent and having fully developed sewer systems with no natural channels.

Van Sickle (1962) in a study in Houston, Texas further subdivided Eagleson's urban classification into the following four general classes (Figure 3): (1) Cultivated, some urban, no storm sewers; (2) More urban, some storm sewers, no channel improvement; (3) Extensive urban, storm sewers, no channel improvement; and (4) Extensive urban storm sewers, considerable channel improvement. These class descriptions are taken directly from Van Sickle's report. Urbanization of a rural watershed classified as undeveloped pasture is seen to decrease the lag time 67 percent for Class 1; 75 percent for Class 2; 83 percent for Class 3; and 92 percent for Class 4. Van Sickle concluded that because of urbanization, watersheds in the Houston, Texas area could experience as much as a 90 percent reduction in lag time.

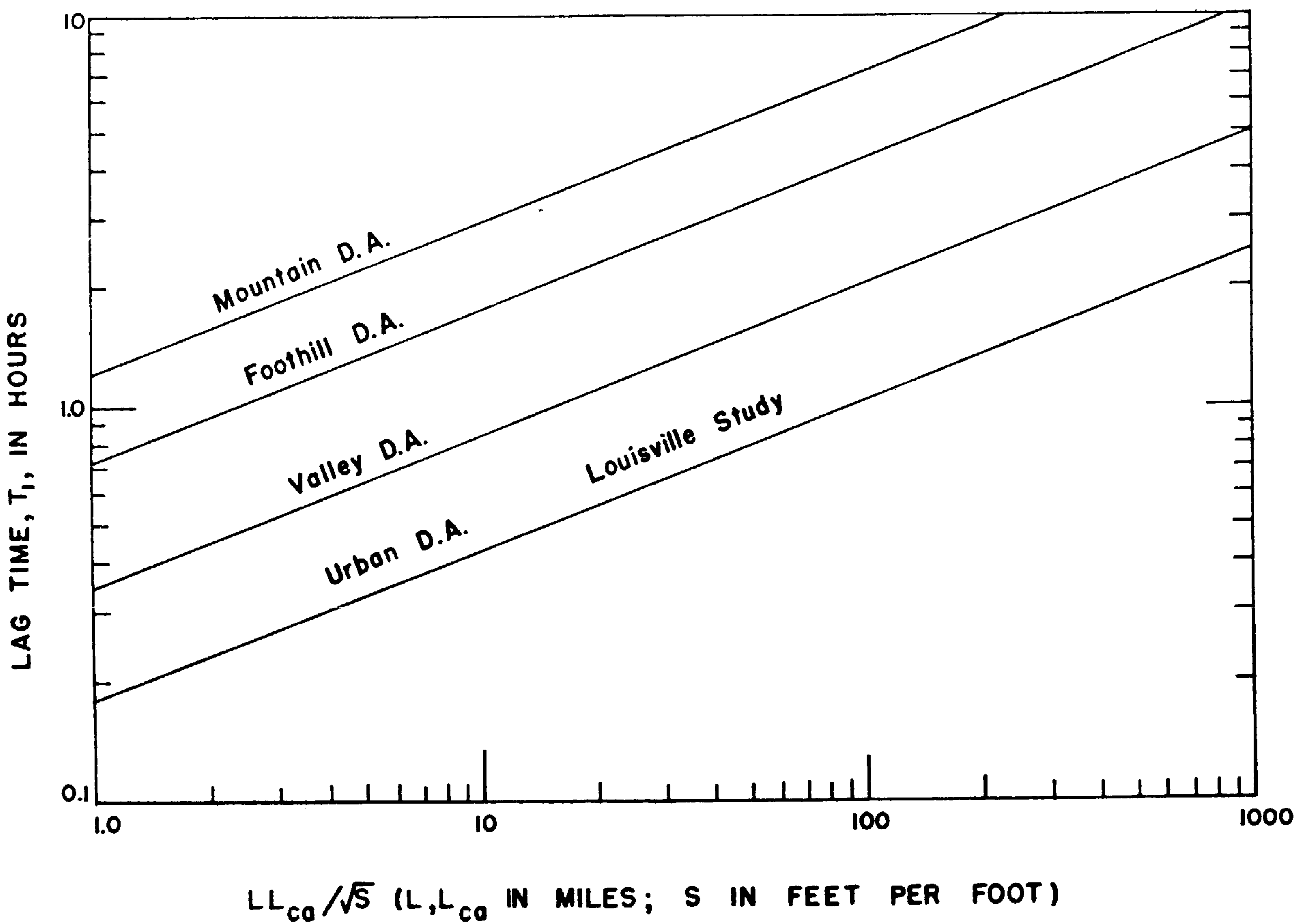


FIGURE 2. LAG TIME, T_l, VS. $LL_{ca}/\sqrt{S}$ (after Eagleson, 1962)

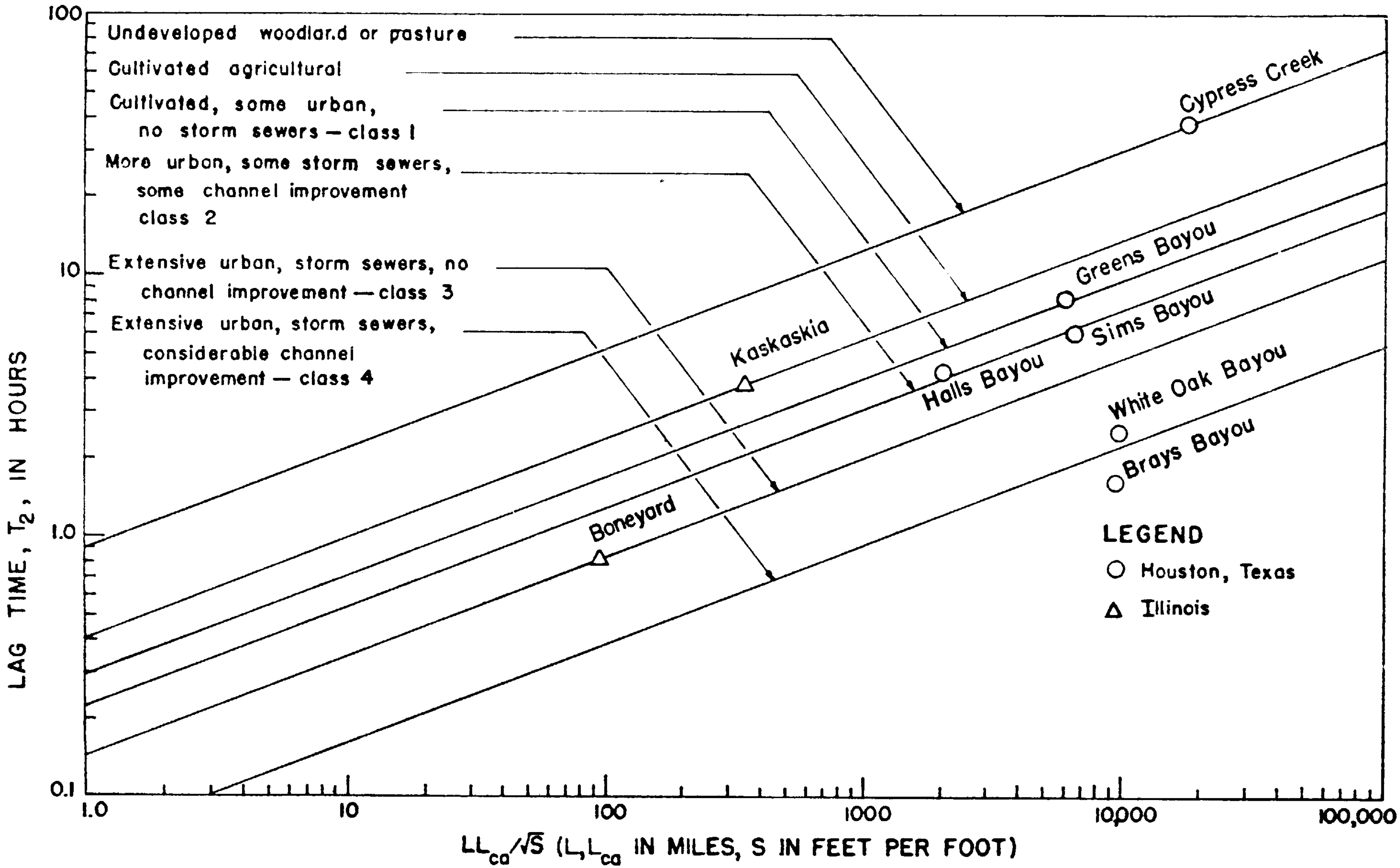


FIGURE 3. LAG TIME, T_2, VS. $LL_{ca}/\sqrt{S}$ (after Van Sickle, 1962)

2. Peak Discharge. Carter (1961) developed an empirical equation relating the mean annual flood to the lag time, drainage area and percentage of impervious cover to determine the effect of urbanization on the mean annual flood in the vicinity of Washington, D. C. This equation is

$$\overline{Q} = 223\ K\ A^{0.85}\ T_3^{-0.45} \quad \dots\dots\dots\dots\dots\dots \quad (1)$$

where $\overline{Q}$ is the mean annual flood in cubic feet per second and is equivalent to the flood having a recurrence interval of 2.33 years, A is the drainage area in square miles, T_3 is the same as previously defined and is expressed in hours, and K is an adjustment factor based upon the degree of imperviousness of the area. The factor K is expressed as,

$$K = \frac{0.30 + 0.0045\ I}{0.30} \quad \dots\dots\dots\dots\dots\dots\dots \quad (2)$$

where I is the percent of impervious cover.

Wiitala (1961) also used Carter's equations to evaluate the effects of urbanization on the mean annual flood for the Red Run watershed in Michigan. Results indicated "that for areas near Detroit comparable in size and degree of development to Red Run, the natural mean annual flood is more than doubled by urbanization." Wiitala also compared the mean annual flood derived from recent flood-frequency studies covering southeastern Michigan to evaluate the effect of urbanization. The measured mean annual flood for Red Run was found to be three times as large as that indicated from a flood frequency study for a natural drainage basin of comparable size.

Van Sickle (1962) used the unit hydrograph as a means to detect the effects of urbanization on peak discharge in Houston, Texas. Of the watersheds studied, eight had continuous water-stage records. Brays Bayou, the watershed with the most urban development, had a period of record of twenty-seven years. During this period, the watershed had changed from undeveloped farm land to an extensively urbanized area. The unit hydrographs of Figure 4 readily show the changes in runoff characteristics for Brays Bayou during this period. Van Sickle concluded "that urban development of a watershed in Harris County can be expected to produce peak discharge rates of from two to five times those which would occur on the same watershed for undeveloped rural conditions."

3. Runoff Yield. Other investigators have studied the effects of urbanization on the runoff yield from a watershed. Sawyer (1961) reported "that the increased urbanization has altered the characteristics and regimen of many of the streams on Long Island, ..." No quantitative information regarding the increase in runoff yield as a result of urbanization was presented in Sawyer's study. Recently a study by Harris and Rantz (1964) of a small watershed in Santa Clara County, California also indicated that "a substantial increase in the volume of storm runoff coincided with the period of major urban development." Again no general conclusion could be made regarding the effects of urbanization on the runoff yield from a watershed.

B. PURPOSE AND SCOPE

The evaluation of the effects of urbanization on the runoff characteristics of a small watershed is a problem that can be studied by either a short-range or a long-range investigation. The long-range investigation would involve a program of expanded data collection carefully planned to provide measurements of rainfall and runoff from watersheds both before and after urbanization. Because this type of investigation would

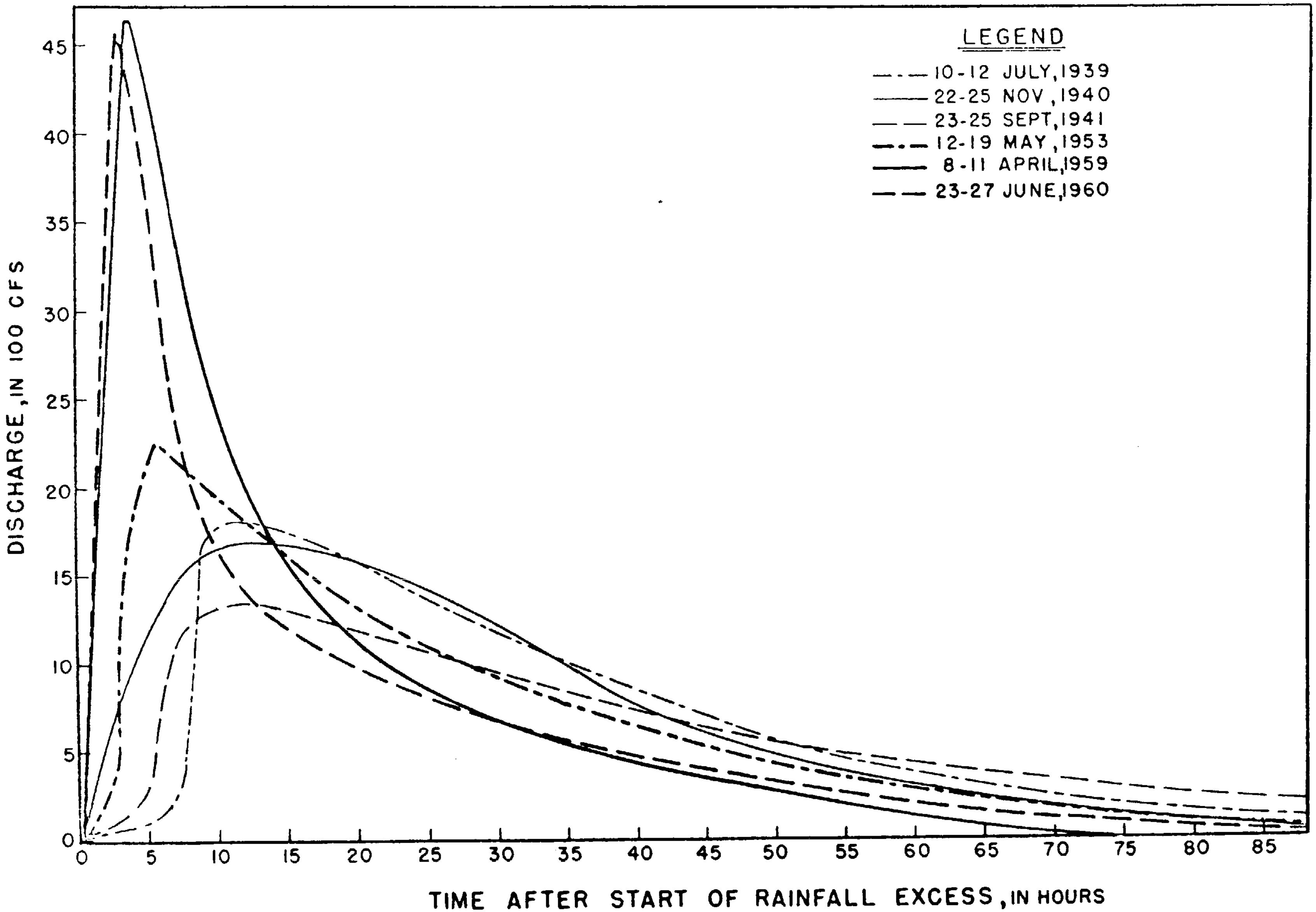


FIGURE 4. BRAYS BAYOU UNIT HYDROGRAPHS (AFTER VAN SICKLE)

require several years for hydrologic data accumulation, it cannot provide any immediate information on the changes in watershed behavior arising as a result of urbanization. A short-range investigation, however, based on synthetic evaluation of present data would provide answers now. It is in the realm of this short-range objective that this study of a small urban watershed is directed.

In this investigation both the effects of the existing and future urbanization on the discharge hydrograph and runoff yield from the Waller Creek watershed located in Austin, Texas will be studied. Because no hydrologic data is available for Waller Creek before urbanization, empirical relations are derived from data on eleven rural watersheds to describe the Waller Creek discharge hydrograph before urban development. This empirically derived hydrograph is then compared with the hydrograph as it exists today to evaluate the effect of existing urbanization on the hydrologic characteristics of the Waller Creek watershed. In a similar manner, empirical equations are derived based on data from 22 urban watersheds to describe the Waller Creek discharge hydrograph during urban development. The empirically derived hydrograph is then compared both with the hydrograph as it exists today and the empirically derived rural hydrograph to evaluate the effects of future urban development on the discharge hydrograph of Waller Creek. Based on selected storm data, a rainfall-runoff relationship is also derived with impervious cover as one of the independent variables. This equation is then used to evaluate the effects of increasing impervious cover on the runoff yield from the Waller Creek watershed.

Chapter II

DEVELOPMENT OF EMPIRICAL EQUATIONS

Presented in this chapter is the development of the empirical equations which will be used to determine rural and future urban unit hydrographs and to evaluate the effects of both existing and future urbanization on the runoff characteristics of the Waller Creek watershed. These equations are determined from storm and runoff data from several rural and urban watersheds. Both the method employed to analyze the storm and hydrograph data and the statistical procedure used to derive the empirical equations are presented. The statistical significance of the derived equations and a comparison with the results from other studies is also presented.

A. ANALYSIS OF HYDROGRAPH DATA

In order to develop empirical relationships describing the hydrologic characteristics of a number of watersheds, it was necessary to reduce all the hydrograph data to a common basis for direct comparison. This was done by reducing all the runoff data for each watershed to a common duration unit hydrograph. A 30 minute duration of rainfall excess was selected as the basis of comparison because most of the storms on the watersheds studied were approximately 30 minutes in duration.

1. Unit Hydrograph. The basic theory of the unit hydrograph appears to have been suggested first by Folse (1929). The Boston Society of Civil Engineers (1930) stated, "the base of the flood hydrograph appears to be approximately constant for different floods, and peak flows tend to vary directly with the total volume of runoff." Three years later, Sherman (1932) formulated the popular unit hydrograph theory. The unit

hydrograph defined by Sherman was the hydrograph representing one-inch of runoff from a 24-hour rainfall. Hoyt (1936) defined the unit hydrograph as "a hydrograph of surface runoff resulting from rainfall within a unit of time, as a day or an hour." Brater (1940) successfully applied the unit hydrograph theory to small watersheds varying in size from 4.24 acres to 1,876.7 acres. Brater also introduced the concept of the unit hydrograph resulting from a "unit storm." A unit storm was defined as "an isolated rainfall falling at an intensity greater than the infiltration capacity and having a duration equal to or less than the period of rise." Wisler and Brater (1959) stated that a unit storm is defined as a rain whose duration is such that the period of surface runoff is not appreciably less for any rain of shorter duration.

The important difference between Hoyt's and Brater's approach to the unit hydrograph is the effect of the duration of rainfall excess. Brater states that if the unit storm duration is less than the period of rise then the same shaped hydrograph will be generated from different storms. Linsley, Kohler and Paulhus (1958) define the unit hydrograph as the hydrograph of one-inch of direct runoff from a storm of specified duration. Hydrographs for different durations can be obtained by means of the S-curve technique which assumes linearity of the system.

The criteria selected in this study for the unit hydrograph analysis is a combination of both approaches and can be summarized as follows:

1. The rainfall duration, D_T, must be either equal to or less than the period of rise, T_R.

2. The rainfall intensity must be approximately constant and uniform throughout the watershed.

3. The beginning and end of rainfall must be approximately the same

at every point in the watershed.

4. The storm period must have occupied a place of comparative isolation in the record.

5. The hydrograph for any duration of rainfall excess can be obtained by the S-curve procedure from a hydrograph of known rainfall excess duration.

Of approximately 435 storms on Waller Creek that occurred during the period of record, only 18 approximately satisfied the unit hydrograph requirements; a satisfactory time record for both rainfall and runoff was available for only 13 of these. Some of the 13 storms studied did not completely meet all the storm requirements. The following variations in the unit hydrograph criteria were allowed by noting that these variations at different rain gaging stations resulted in no significant change in the discharge hydrographs:

1. Up to a 15 minute variation in the initiation of rainfall,

2. Up to a 30 percent variation in the total amount of rainfall.

In general the rainfall studied was the result of convective storms. As a result for the small Waller Creek watershed, 4.13 square miles, the time intensity pattern was practically uniform. For many of the other watersheds studied the conversion of a hydrograph of a given duration to one of a different duration by the S-curve technique resulted in a relatively small change in the peak discharge.

a. Method of Analysis. The S-curve characteristics of each individual storm were analyzed and reduced to a common S-curve representing one-inch per hour of rainfall excess (Chow, 1964). In most cases sufficient data were available for the analysis of at least three separate storms. The derivation of an S-curve requires that the duration of rainfall excess be known. In most cases a good estimate can be made from rainfall data. Based on a suggestion by Linsley, Kohler and Paulhus (1958), the correct duration of rainfall excess would result in the minimum amount of S-curve fluctuation. Subsequent analysis indicated that a second criterion was necessary to determine the correct duration of rainfall excess. This second criterion was satisfaction of the theoretical equilibrium discharge, q_c, defined by the equation

$$q_c = \frac{645.6\ A}{D} \qquad (3)$$

where A is the drainage area in square miles, D is the duration of rainfall excess in hours (Linsley, Kohler and Paulhus, 1958). A computer program was developed which allowed the duration of rainfall excess to be varied. When the final S-curve for each storm was determined, the reduction of each S-curve to a common base of one-inch per hour allowed for a direct comparison. An average S-curve was then graphically drawn by eye to best fit the data. In most cases the resulting S-curves were in close agreement with one another. A smooth S-curve was found to always result when the input discharge hydrograph time increment was equal to the assumed duration of rainfall excess.

Meier (1964) made a similar study of S-curve characteristics of small rural watersheds in Texas. Meier's study consisted of a more sophisticated statistical method of determining the best S-curve. A polynomial of the tenth order was used to define the S-curve.

b. Discussion of the Unit Hydrograph. The unit hydrograph was selected as the means of measuring the effects of urbanization on the flood potential of a small watershed. Since the introduction of the basic unit hydrograph theory by Sherman (1932), considerable hydrologic analysis has been made assuming that the hydrograph results from a linear system. A linear system may be defined as one which relates the dependent variables to a weighted sum of independent variables (Shen, 1963). Stated mathematically, a drainage basin system is linear if the differential equation of the input and output relationship is linear (Chow, 1964). For a linear system the principle of superposition can be used. Recent work by other investigators has called attention to the non-linear nature of hydrologic systems. The non-linear system approach attempts to take into account the interaction of the other variables with one another. The work of Amorocho (1961), Harder (1962), Liggett (1959) and Ishihare (1956) are examples of the non-linear hydrologic approach.

The application of the unit hydrograph method to small watersheds varying in size from approximately 4 acres to 10 square miles has been shown by Brater (1940). Tippetts-Abbett-McCarthy and Stratton, Consulting Engineers, successfully applied the unit hydrograph to runoff calculations for the city of Philadelphia in 1947 (Eagleson, 1962). The study conducted by Watkins (1963) in England found "for 3 urban areas that the unit hydrograph agreed with the recorded hydrographs but could only be obtained accurately from the observed hydrographs." Watkins concluded that "although the unit hydrograph method is satisfactory for calculating runoff for existing systems, it is not suitable for use as a basis for a sewer design method." The recent work by Willeke (1962 and 1964) for small urban watersheds found no significant indication of non-linearity and concluded that the system could be treated as a single linear storage system whose characteristics can be represented by the constants in the Muskingum routing equations. The assumption is thus made in this study that the unit hydrograph can be used to describe the hydrologic system of both an urban and rural watershed. The unit hydrograph was used to develop and S-curve for an intensity of one-inch per hour of rainfall excess which was in turn used to develop a 30 minute unit hydrograph for each watershed.

2. Watersheds Studied. Physiographic and storm or unit hydrograph data were available for 24 urban and 11 rural watersheds. The following information is listed in Tables 2 and 3 for each watershed:

1. An identifying number or letter,
2. Name of watershed,
3. Availability of storm data,
4. Availability of unit hydrograph data, and
5. Sources of data.

To distinguish between urban and rural watersheds studied, the following coding system was adopted:

1. Urban watersheds are indicated by number,
2. Rural watersheds are indicated by letters.

Complete hydrologic data for the 24 urban and 11 rural watersheds will be published by the Texas Water Commission in their Bulletin Series in the near future. Additional data concerning the lag time characteristics of 43 urban watersheds were also available and are given in Appendix C.

B. MULTIPLE LINEAR REGRESSION ANALYSIS

Frequently problems have arisen where an observed variable is known or is suspected to be dependent upon one or more other variables, although the exact form of the true relationship is unknown. Such relationships are often determined by the method of regression analysis. This method involves hypothesizing the relation between the

No.	Watershed	Storm Data	Unit Hydrograph Data	Sources of Data
1	Anacostia, N.W., Illinois		X	U. S. Corps of Engrs. (1954)
2	Anacostia, N.E., Illinois		X	U. S. Corps of Engrs. (1954)
3	Boneyard, Illinois		X	Chow (1952)
4	Brays Bayou, Texas	X	X	Van Sickle (1964)
5	Greens Bayou, Texas	X	X	Van Sickle (1964)
6	Halls Bayou, Texas	X	X	Van Sickle (1964)
7	Sims Bayou, Texas	X	X	Van Sickle (1964)
8	White Oak Bayou, Texas	X	X	Van Sickle (1964)
9	Red Run, Michigan	X		Wiitala (1963)
10	Waller Creek at 38th Street, Texas	X		U. S. Geological Survey*
11	Waller Creek at 23rd Street, Texas	X		U. S. Geological Survey*
12	Salt Fork, West Branch, Illinois		X	Mitchell (1948)
13	Louisville, 17th Street, Kentucky		X	U. S. Corps of Engrs. (1949), Snyder (1958), Eagleson (1962)
14	Louisville, N. W. Trunk, Kentucky		X	U. S. Corps of Engrs. (1949), Snyder (1958), Eagleson (1962)
15	Louisville, Western Outfall, Kentucky		X	U. S. Corps of Engrs. (1949), Snyder (1958), Eagleson (1962)
16	Louisville, Southern Outfall, Kentucky		X	U. S. Corps of Engrs. (1949), Snyder (1958), Eagleson (1962)
17	Louisville, S. W. Outfall, Kentucky		X	U. S. Corps of Engrs. (1949), Snyder (1958), Eagleson (1962)
18	Freeman, A, Indiana	X	X	U. S. Corps of Engrs. (1947)
19	Freeman, B + A, Indiana	X	X	U. S. Corps of Engrs. (1947)
20	Freeman, B + T, Indiana	X	X	U. S. Corps of Engrs. (1947)
21	Lockbourne, 2, Ohio**	X	X	U. S. Corps of Engrs. (1947)
22	Lockbourne, 3T, Ohio**	X	X	U. S. Corps of Engrs. (1947)
23	St. Anne, 1, Indiana	X	X	U. S. Corps of Engrs. (1947)
24	Godman, 1, Kentucky	X	X	U. S. Corps of Engrs. (1947)

TABLE 2. Data on Urban Watersheds.

* Data furnished by Austin District.

** Only used for lag time and general relationships.

No.	Watershed	Storm Data	Unit Hydrograph Data	Sources of Data
A	Calaveras, Tex.	X		U. S. Geological Survey*
B	Deep Creek No. 3, Tex.	X		U. S. Geological Survey*
C	Deep Creek No. 8, Tex.	X		U. S. Geological Survey*
D	Escondido No. 1, Tex.	X		U. S. Geological Survey*
E	Honey Creek No. 11, Tex.	X		U. S. Geological Survey*
F	Honey Creek No. 12, Tex.	X		U. S. Geological Survey*
G	Cow Bayou, No. 4, Tex.	X		U. S. Geological Survey*
H	Albuquerque, N.M.	X		Agricultural Res. Ser. (1960)
I	Bentonville, Okla.	X		Agricultural Res. Ser. (1960)
J	Guthrie, Okla.	X		Agricultural Res. Ser. (1960)
K	Stillwater, Okla.	X		Agricultural Res. Ser. (1960)
L	Freeman Field, D, Ind.**	X	X	U. S. Corps of Engrs. (1947)
M	St. Anne, 2, Ind.**	X	X	U. S. Corps of Engrs. (1947)

TABLE 3. Data on Rural Watersheds.

* Data furnished by Austin District.

** Only used for lag time and general relationships.

dependent and independent variables, to determine the coefficients that provide the best fit of the data, and then to test the validity or accuracy of the results. Based on the results of other investigations (Sribnyi, 1952; Chow, 1962; Ryono and Goltz, 1963), an equation of the form

$$\Theta = P\, X^{a}\, Y^{b}\, G^{c}\, R^{d} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (4)$$

was used in this study to describe hydrologic properties as functions of various physiographic parameters. In equation 4, Θ is the dependent variable, X, Y, G and R are independent variables, and P, a, b, c and d are regression coefficients.

Equation 4 can be reduced to the following convenient logarithmic form,

$$\text{Log}\ \Theta = \log P + a \log X + b \log Y + c \log G + d \log R \quad \ldots\ldots \quad (5)$$

and the method of least squares can be used to evaluate the regression coefficients. The values of these coefficients in the equation are so computed that the sum of the squares of the deviations of the observed values from values computed from the resulting equation is a minimum.

A variable may be said to be independent of another if knowledge of a particular value of one is of no help in estimating the corresponding value of the other. In such a case, the correlation coefficient of the two variables would approximate zero. If two independent variables are not really independent of each other, then the inclusion of any two such variables in a multiple regression equation results in the numerical values of the variables in the prediction equation being affected, each by the inclusion of the other variable (Tennessee Valley Authority, 1962).

In addition, it is recognized (Ezekiel, Mordecai, 1941) that the exponents on the independent variables:

ascribe to any particular independent variable not only the variation in the dependent variable which is directly due to that independent variable but also the variation which is due to such other independent variables correlated with it as have not been separately considered in the study.

Correlation coefficients between each pair of selected "independent variables" are given in Appendix A.

In order to describe the statistical significance of the derived equations the following statistical parameters will be given for each equation.

1. Regression Correlation Coefficient -- Comparative measure of association, defined as

$$r = \frac{\sum x\, w}{\sqrt{\sum x^2}\sqrt{\sum w^2}}$$

where x denotes the measured value and w denotes the predicted value from the regression equations.

2. Standard Error of Estimate -- Measure of the degree of association between series. The larger the value of the standard error of estimate the greater the scatter about the line of regression and, of course, the poorer the relationship. The standard error of estimate is defined as

$$Se = \sqrt{\sum (x - w)^2 / N}$$

where x and w are the same as defined previously and N represents the number of data points.

3. Significance of the Correlation Coefficient -- When the correlation coefficient is calculated from a large number of pairs, one can use the standard error of the correlation coefficient, σ_r, as a test of significance:

$$\sigma_r = \frac{1 - r^2}{\sqrt{N - 1}}$$

Where $r > 2\sigma_r$ there is a 95 percent chance that r is significant (Fisher, 1958);

Where $r > 3\sigma_r$ there is only one chance in a hundred that $r \neq 0$ could have happened by chance.

4. Explained Variance -- A measure of the proportion of the variation in the predicted variable explained by the derived equation. The explained variance can be stated in terms of the ratio of the predicted variance (σ_p^2) to the observed variance (σ_o^2) and can be expressed as a percent in the convenient form

$$100(\sigma_p^2/\sigma_o^2) = r^2$$

C. EQUATIONS FOR HYDROGRAPH PROPERTIES

In order to develop a procedure to describe the characteristics of the unit hydrograph, empirical equations were derived for the following hydrograph properties: (1) Time of rise (T_R); (2) Peak discharge (Q); (3) Time base (T_B); and (4) Hydrograph widths at 50 percent (W_{50}) and 75 percent (W_{75}) of the peak discharge. These hydrograph properties are illustrated in Figure 5.

Equations were derived for each hydrograph property based on data from eleven rural watersheds. These equations are hereafter referred to as rural equations. Similarly, equations were also derived from data on 22 urban watersheds and are hereafter referred to as urban equations.

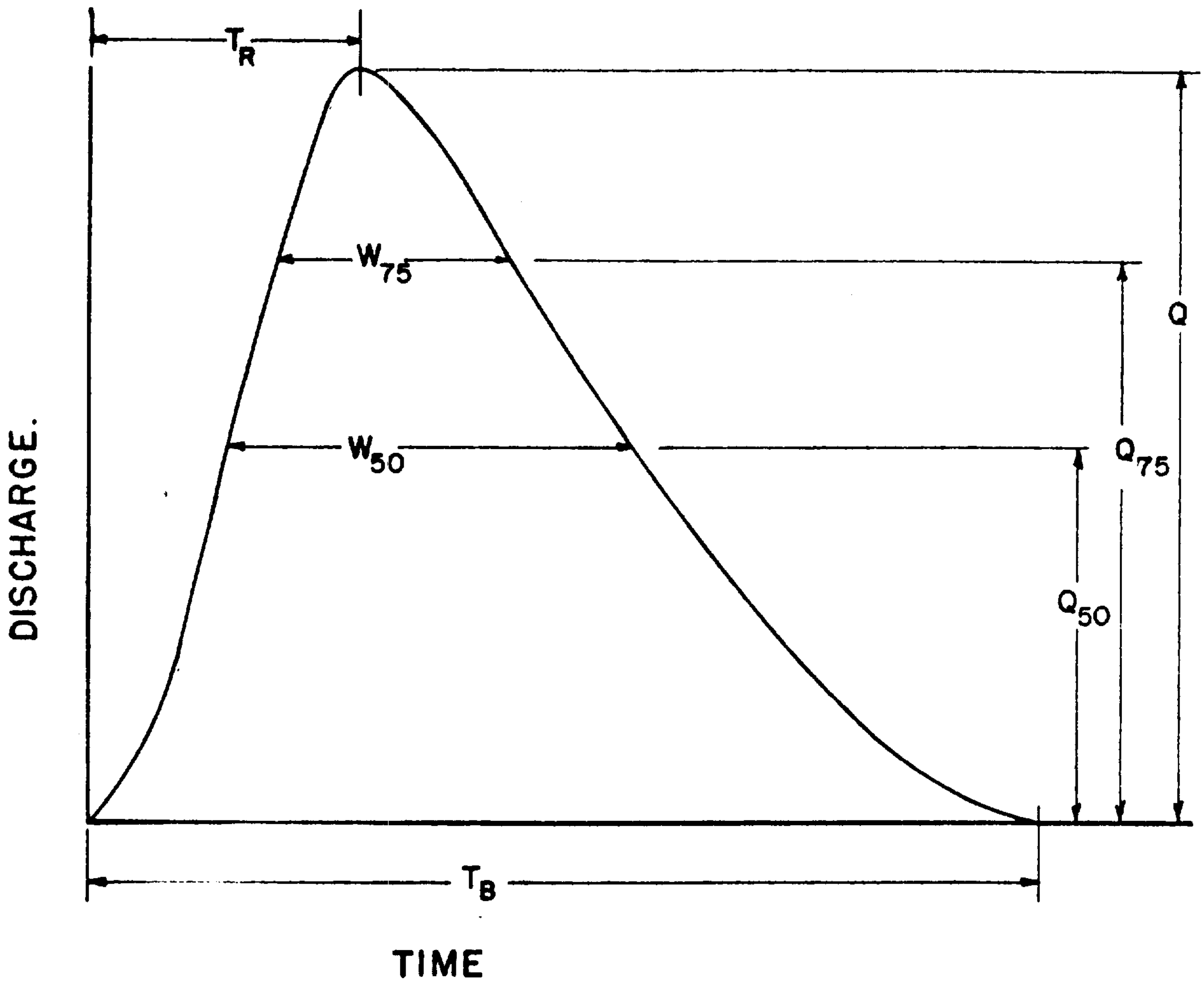


FIGURE 5. DEFINITION OF HYDROGRAPH PROPERTIES.

1. Time of Rise. Some hydrologists, Ramser (1918), Kirpich (1940), Gray (1961), and Wu (1963) have used the time of rise, T_R, defined as the time (minutes) required for the water in the channel at the gaging station to rise from the low to the maximum stage (Figure 5) as a significant time parameter for rural watersheds. Wu (1963) in his study of 21 small rural watersheds (2.86 to 100 square miles) indicated that the time of rise did not vary significantly for different storms and therefore could be used as a hydrograph parameter.

In general, the time of rise of the unit hydrograph for a small watershed can be considered a function of two primary groups of factors: (1) Hydraulic characteristics of the watershed, and (2) Storm characteristics, and can be expressed in the following form:

$$T_R = f\ (\text{Hydraulic characteristics, storm characteristics}) \ldots\ldots \quad (6)$$

or

$$T_R = f\ (\text{H.C., S.C.}) \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (7)$$

The hydraulic characteristics can be divided into two main groups: (1) Surface properties, and (2) Geometry of the watershed. Surface properties can be further subdivided into percentage of impervious cover, channel characteristics, type and extent of cultivation, soil moisture and geology. Watershed geometry includes such factors as area, length, slope and shape. By selection of storms having essentially the same characteristics, the time of rise can be considered a function of only the hydraulic characteristics. Substituting surface properties and geometry of the watershed for hydraulic characteristics, equation 7 reduces to

$$T_R = \beta\ (\text{Surface properties, geometry}) \ldots\ldots\ldots\ldots \quad (8)$$

or $$T_R = \beta(\text{S.P., G.W.}) \qquad (9)$$

If the surface properties can be considered as constant for rural watersheds then equation 9 reduces to

$$T_{RR} = \delta\,(\text{G.W.}) \qquad (10)$$

where T_{RR} is the time of rise of the unit hydrograph for a rural watershed. Gently rolling terrain, pastures and little cultivation characterize the surface properties of the eleven rural watersheds used in this study; therefore the assumption of constant surface properties appears reasonable. Similarly, if surface properties of an urban watershed may also be considered essentially constant, equation 9 reduces to

$$T_{RU} = \delta'\,(\text{G.W.}) \qquad (11)$$

where T_{RU} is the time of rise of the unit hydrograph for an urban watershed. Subsequent analysis indicated that surface properties could not be considered constant.

a. Rural Conditions. Multiple regression equations were derived to express the functional relationship of the time of rise with various geometric characteristics of the watershed as suggested by equation 10. This analysis is based on data compiled from eleven watersheds located in Texas, New Mexico and Oklahoma (Table 3 and Appendix C). One functional form of regression equation

$$T_{RR} = V\,L^e\,S^f \qquad (12)$$

was found to have a high degree of reliability in estimating the time of rise. The reliability of this relationship could not be significantly improved by the addition of other basin parameters. The resulting multiple linear regression equation for the functional relationship expressed by equation 12 is

$$T_{RR} = 2.65\,L^{0.12}\,S^{-0.52} \qquad (13)$$

where L and S are the same as previously defined. The correlation coefficient is 0.972 which is significant at the one percent level. Approximately 95 percent of the variance of the time of rise is explained by equation 13. The standard error of estimate is 18 minutes. Previous investigators (Kirpich, 1940; Chow, 1963) have found it convenient for plotting purposes to restrict the functional form of equation 12 to

$$T_{RR} = M\,(L/\sqrt{s})^{g} \quad \ldots\ldots\ldots\ldots\ldots \quad (14)$$

The resulting linear regression equation for the eleven rural watersheds expressed by equation 14 is

$$T_{RR} = 1.24\,(L/\sqrt{s})^{0.36} \quad \ldots\ldots\ldots\ldots\ldots \quad (15)$$

with a correlation coefficient of 0.956, significant at the one percent level, and a standard error of estimate of 23 minutes (Figure 6). Approximately 92 percent of the variance of the time of rise is explained by equation 15. Both equations 13 and 15 are based on the following range of fairly uniformly distributed data: (1) L (3,250 ft. to 25,300 ft.); (2) S (0.00793 ft/ft to 0.146 ft/ft); and (3) T_{RR} (30 minutes to 150 minutes).

b. Urban Conditions. Statistical analysis indicated that the time of rise for urban watersheds could be best expressed as a function of the length, slope and impervious cover. The resulting equation based on 22 urban watersheds is

$$T_{RU} = 20.8\,L^{0.29}\,S^{-0.11}\,I^{-0.61} \quad \ldots\ldots\ldots\ldots\ldots \quad (16)$$

with a correlation coefficient of 0.954, significant at the one percent level, and a standard error of estimate of 102 minutes. Approximately 91 percent of the variance

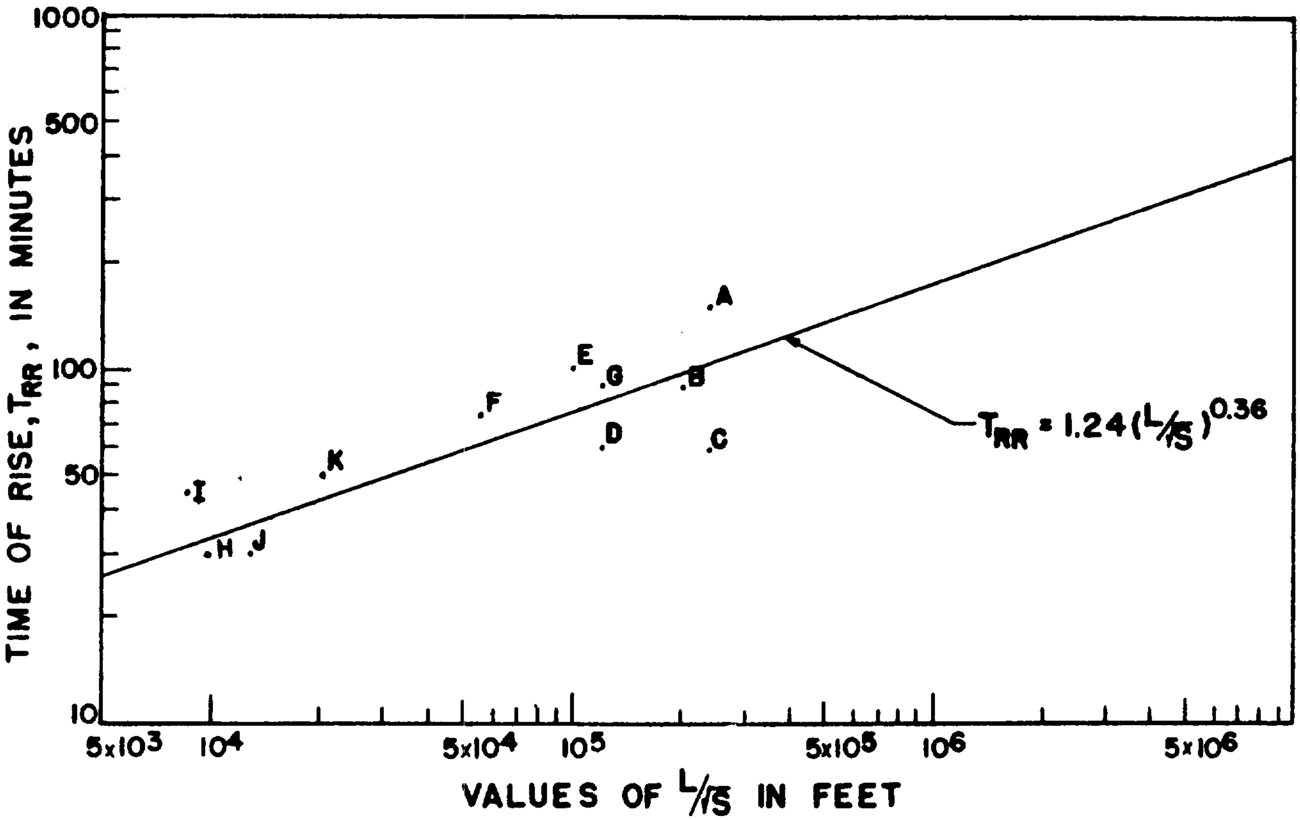


FIGURE 6. RELATIONSHIP BETWEEN T_{RR} AND $L/\sqrt{S}$ — RURAL.

in the time of rise is explained by equation 16. Equation 16 is based on the following range of fairly uniformly distributed data: (1) L (200 ft. to 54,800 ft.); (2) S (.0064 ft/ft to .0104 ft/ft); (3) I (2.7 percent to 100 percent); and (4) T_{RU} (30 minutes to 720 minutes). The introduction of the impervious cover as an index of urbanization was found not to be sufficient to adequately describe the characteristics of some urban watersheds. In most cases where the channel had been improved or a system of storm sewers was present the predicted values of the time of rise were high compared to the measured values (Table 4). A new urban factor, Φ, was therefore introduced to account for the reduction in the time of rise that was due to channel improvements or addition of storm sewers. Values of Φ were first determined for each watershed so that the predicted and given values of T_{RU} would be equal. The physical conditions of the drainage systems of each of the urban watersheds were then studied. Values of Φ were then rounded off and grouped into three classifications (Table 5). The first classification, $\Phi = 1.0$, represents natural condition, no urban development. The second classification represents watersheds that have undergone some urban development with partially sewered drainage systems and some channel improvement. The third classification represents watersheds that have extensive urban development, fully developed storm sewer and extensive channel improvement. Equation 16 can now be expressed in the form

$$T_{RU} = 20.8\,\Phi\,L^{0.29}\,S^{-0.11}\,I^{-0.61} \quad \ldots\ldots\ldots\ldots\ldots \quad (17)$$

2. <u>Peak Discharge</u>. From a design standpoint the most important hydrograph property is the peak discharge. As a result most of the empirically derived equations describing hydrograph properties have been concerned with prediction of the peak discharge. Following the same theoretical development as for the time of rise, T_R, the peak discharge

Watersheds	Urban Equations: Equation 16 $T_R = 20.8 \frac{L^{.29}}{S^{.11} I^{.61}}$ Percent Difference	Selected Values of Φ	Equation 17 $T_R = 20.8 \Phi \frac{L^{.29}}{S^{.11} I^{.61}}$ Percent Difference
Red Run+	50	0.6	-11
Boneyard	-10	1.0	-10
Waller Cr. - 23rd St.*	42	0.8	+14
Waller Cr. - 38th St.*	69	0.8	+35
Louisville - S. Out.+	52	0.6	-8
Louisville - 17th St.+	-6	0.6	-43
Louisville - W. Out.+	65	0.6	0
Louisville - N.W. Trunk+	80	0.6	+8
Louisville -S.W. Out.+	278	0.6	+100
Freeman Field A	-18	1.0	-18
Godman Field 1	-15	1.0	-15
St. Anne Field 1	94	1.0	+94
Freeman B & Taxi	-36	1.0	-36
Freeman B & Apron	-50	1.0	-50
Salt Fork W. Branch	-37	1.0	-37
Anacostia, N.E. Branch	-14	1.0	-14
Anacostia, N.W. Branch	-13	1.0	-13
Brays Bayou*	32	0.8	-26
Greens Bayou	-35	1.0	-35
Halls Bayou	-44	1.0	-44
White Oak Bayou*	-62	0.8	-70
Sims Bayou	-46	1.0	-46

TABLE 4. Percent Difference Based on Given Value of the Time of Rise - Urban.

* Some channel improvement.

\+ Extensive channel improvement.

of the unit hydrograph can also be considered as only a function of the geometry of the watershed and can be expressed as

$$Q_R = \gamma(G.W.) \quad \ldots \quad (18)$$

and

$$Q_U = \gamma'(G.W.) \quad \ldots \quad (19)$$

where Q_R and Q_U are the peak discharge of the unit hydrograph for a rural and urban watershed respectively.

Φ	CLASSIFICATION
0.6	Extensive channel improvement and storm sewer system, closed conduit channel system.
0.8	Some channel improvement and storm sewers; mainly cleaning and enlargement of existing channel.
1.0	Natural channel conditions.

TABLE 5. Φ Classifications.

a. Rural Conditions. Again multiple regression analysis was used to derive empirical equations expressing the functional relationship of equations 18 and 19. The introduction of the time of rise which represents the integrated effects of the geometric characteristics of the watershed was found to considerably improve the statistical fit of the regression equations. One functional form of regression equation based on the eleven rural watersheds

$$Q_R = M\,A^h\,T_{RR}^{\,i} \quad \ldots \quad (20)$$

was found to have a high degree of reliability in estimating the peak discharge. The reliability of this relationship could not be significantly improved by the addition of other basin parameters. The resulting multiple regression equation for the functional relationship expressed by equation 20 is

$$Q_R = 1.70 \times 10^3 A^{0.88} T_{RR}^{-0.30} \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (21)$$

where A and T_{RR} are the same as previously defined. The correlation coefficient is 0.931 (significant at the one percent level) and the standard error of estimate is 470 cfs for equation 21. Approximately 87 percent of the variance in the peak discharge is explained by equation 21. Equation 21 is based on the following range of fairly uniformly distributed data: (1) Q_R (96 cfs to 3,000 cfs); (2) A (0.134 square miles to 7.01 square miles); and (3) T_{RR} (30 minutes to 150 minutes).

b. Urban Conditions. Statistical analysis indicated that the peak discharge for the urban watersheds also could be expressed as a function of the drainage area, A, and the time of rise, T_{RU}. The resulting equation based on the 22 urban watersheds is

$$Q_U = 1.93 \times 10^4 A^{0.91} T_{RU}^{-0.94} \quad \ldots\ldots\ldots\ldots\ldots \quad (22)$$

with a correlation coefficient of 0.811, significant at the one percent level, and a standard error of estimate of 2,220 cfs. Approximately 66 percent of the variance in peak discharge is explained by equation 22. Equation 22 is based on the following range of data: (1) Q_U (8.1 cfs to 13,200 cfs), fairly uniformly distributed below 4,500 cfs with one value at 13,200 cfs; (2) A (0.0128 square miles to 92 square miles), fairly uniformly distributed; and (3) T_{RU} (30 minutes to 720 minutes), fairly uniformly distributed.

3. Hydrograph Widths. To aid in the construction of the unit hydrograph, equations were derived describing the hydrograph widths at 0, 50 and 75 percent of the peak discharge (Figure 5). Based on the same theoretical development as for T_R and Q the hydrograph widths T_B, W_{50} and W_{75} can also be considered a function of the geometry and type of watershed and can be expressed as

$$\text{Hydrograph widths, rural} = \gamma\,(\text{G.W.}) \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (23)$$

and

$$\text{Hydrograph widths, urban} = \gamma'\,(\text{G.W.}) \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (24)$$

a. Rural Conditions. Multiple regression equations were derived to express the functional relationship of the hydrograph widths with various geometric characteristics of the watersheds as indicated by equations 23 and 24. One functional form of the regression equation

$$W_{50R},\ W_{75R},\ T_{BR} = E\,A^{i}\,Q_R^{\,k} \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (25)$$

was found to have a high degree of reliability in determining the hydrograph widths. The resulting multiple linear regression equations for the functional relationship expressed by equation 25 are

$$T_{BR} = 7.41 \times 10^{3}\,A^{0.64}\,Q_R^{\,-0.53} \quad \ldots\ldots\ldots\ldots\ldots \quad (26)$$

(correlation coefficient = 0.976, 95 percent of the variance explained, significant at the one percent level, standard error of estimate 72 minutes).

$$W_{50R} = 7.37 \times 10^{4}\,A^{1.11}\,Q_R^{\,-1.13} \quad \ldots\ldots\ldots\ldots\ldots \quad (27)$$

(correlation coefficient = 0.950, 95 percent of the variance explained, significant at the one percent level, standard error of estimate 9 minutes).

$$W_{75R} = 4.46 \times 10^4 A^{1.06} Q_R^{-1.13} \quad \ldots\ldots\ldots\ldots \quad (28)$$

(correlation coefficient = 0.973, 95 percent of the variance explained, significant at the one percent level, standard error of estimate 13 minutes).

where T_{BR}, W_{50R} and W_{75R} are expressed in minutes and A and Q_R are the same as previously defined. Equations 26, 27 and 28 are based on the following range of fairly uniformly distributed data: (1) T_{BR} (100 minutes to 550 minutes); (2) W_{50R} (31 minutes to 170 minutes); (3) W_{75R} (20 minutes to 123 minutes); (4) A (0.134 square miles to 7.01 square miles); and (5) Q_R (91 cfs to 3,000 cfs).

b. Urban Conditions. Statistical analysis also indicated that the hydrograph widths for urban watersheds could best be expressed as a function of the drainage area and peak discharge. The resulting equations based on 22 urban watersheds are

$$T_{BU} = 4.44 \times 10^5 A^{1.17} Q_U^{-1.19} \quad \ldots\ldots\ldots\ldots \quad (29)$$

(correlation coefficient = 0.945, 89 percent of the variance explained, significant at the one percent level, standard error of estimate 1,060 minutes).

$$W_{50U} = 4.14 \times 10^4 A^{1.03} Q_U^{-1.04} \quad \ldots\ldots\ldots\ldots \quad (30)$$

(correlation coefficient = 0.977, 96 percent of the variance explained, significant at the one percent level, standard error of estimate 120 minutes).

$$W_{75U} = 1.34 \times 10^4 A^{0.92} Q_U^{-0.94} \quad \ldots\ldots\ldots\ldots \quad (31)$$

(correlation coefficient = 0.964, 93 percent of the variance explained, significant at the one percent level, standard error of estimate 73 minutes).

Equations 29, 30 and 31 are based on the following range of data: (1) T_{BU} (70 minutes to 7,000 minutes), fairly uniformly distributed; (2) W_{50U} (31 minutes to 1,350 minutes), fairly uniformly distributed; (3) W_{75U} (25 minutes to 650 minutes), fairly uniformly distributed; (4) Q_U (8.1 cfs to 13,200 cfs), fairly uniformly distributed below 4,500 cfs with one value of 13,200 cfs; and (5) A (0.0128 square miles to 92.0 square miles), fairly uniformly distributed.

4. Lag Time. Probably the first comprehensive study of urban hydrology was made by Horner and Flynt (1942) but because of the limitations of the instrumentation, no reliable relationships were developed. One basic conclusion was that "the comparatively wide range in the lag at each location led to the inference that the lag was a variable, its value being determined more by rainfall characteristics than by the characteristics of the drainage area."

Two basic approaches have evolved for analyzing and evaluating lag time. One method is to determine the lag time properties of a watershed under various storm conditions, Linsley (1943), Viessman (1961), Landreth (1963), Viessman and Abdel-Razag (1964). The second approach is to analyze only hydrographs generated by certain types of storms, Wiitala (1961), Eagleson (1962), Van Sickle (1962), thus reducing some of the unknown storm variables. The lag time is then correlated with some physiographic features of the watershed. Because storms of various types were not available for some watersheds used in this study and because the storms selected for this study had nearly constant characteristics, the second approach was adopted. As in the case of the time of rise, the lag time can also be considered a function of the surface properties and geometry of the watershed and can be stated in the form

$$\text{Lag Time} = \Theta \text{ (Surface properties, geometry of watershed)} \quad \ldots\ldots \quad (32)$$

Carter (1961) simplified equation 32 by classifying the watersheds according to the following three groups: (1) Natural; (2) Partially sewered; and (3) Completely sewered. Shown in Figure 7 is the relationship between T_3 and $L/\sqrt{s}$ as determined by

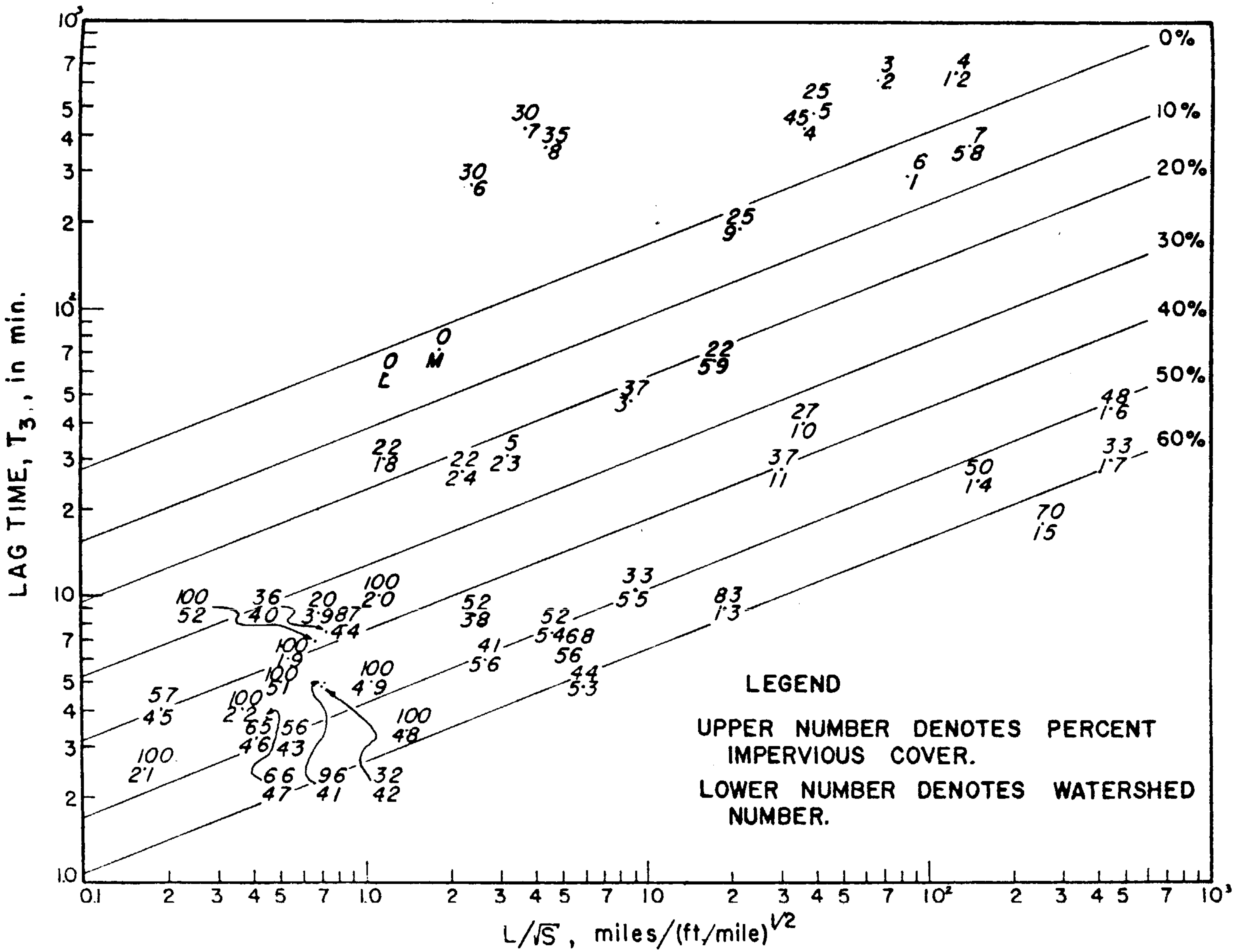


FIGURE 7. LAG TIME, T_3, VS. $L/\sqrt{S}$

Carter but with the surface properties represented by the percentage of impervious cover (I). The lines of constant impervious cover were constructed by interpolation assuming they would form a family of parallel lines. The equation expressing the relationship shown in Figure 7 for zero impervious cover is

$$T_3 = C\,(L/\sqrt{s})^{0.39} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (33)$$

with C varying from 67 for zero percent impervious cover to 2.6 for 60 percent impervious cover. Because of the limited physiographic data that was available for some of the urban watersheds the statistical analysis was limited to the following two forms:

Lag Time = R (Area, slope, impervious cover) ..... (34)

and

Lag Time = R' (Length, slope, impervious cover)... (35)

Two linear regression equations for various groupings of the urban data for the two definitions of lag time were determined.

The relation for the lag time, T_4, (Table 6) as a function of A, I and S for 40 urban watersheds was found to be

$$T_4 = 30.1\,A^{0.21}\,I^{-0.51}\,S^{-0.26} \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (36)$$

Equation 36 gives a correlation coefficient of 0.972, a standard error of estimate of 23 minutes, and predicts two-thirds of the values of T_4 within $\pm$ 36 percent and explains 94 percent of the variance of T_4. Equation 36 is based on the following range of fairly uniformly distributed data: (1) T_4 (3.1 minutes to 300 minutes); (2) A (0.00062 square miles to 4.13 square miles); (3) I (1.9 percent to 100 percent); (4) S (0.0056 ft/ft to 0.0610 ft/ft). Similarly the lag time, T_4, was also considered to be a function

T_1	=	Time from beginning of rainfall to the centroid of runoff. Linsley, Kohler and Paulhus (1958).
T_2	=	Time from center of mass of rainfall excess to the peak discharge. Snyder (1938), Eagleson (1962), Morgan and Johnson(1962), Gray (1961), U. S. Corps of Engineers (1963), Taylor and Schwartz (1952).
T_3	=	Time from center of mass of the rainfall excess to the center of mass of the runoff. Mitchell (1948).
T_4	=	Time from centroid of rainfall to centroid of runoff.
T_5	=	Time from beginning of rainfall to the peak discharge.
T_6	=	Time from cessation of effective rainfall to the inflection point of the recession side of the resulting runoff hydrograph, $T_6 \cong T_c$. (Snyder, 1958).
T_7	=	Time from centroid of rainfall to the peak discharge.
T_R	=	Time required for the water in the channel at the gaging station to rise from the low to the maximum stage. Ramser (1918), Kirpich (1940), Gray (1961), Wu (1963).
T_c	=	Time required for a drop of water to travel from the most remote point in the watershed to the gaging point.

TABLE 6. Summary of Lag Time Definitions.

of L, I and S for 28 urban watersheds and was found to be

$$T_4 = 1.90\, L^{0.35}\, I^{-0.35}\, S^{-0.26} \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (37)$$

Equation 37 gave a correlation coefficient of 0.873, a standard error of estimate of 8.2 minutes, and predicts two-thirds of the values of T_4 within $\pm$ 27 percent and explains 76 percent of the variance of T_4. Equation 37 is based on the following range of fairly uniformly distributed data: (1) T_4 (3.1 minutes to 45 minutes); (2) L (153 ft. to 27,560 ft.); (3) S (0.009 ft/ft to 0.0610 ft/ft); and (4) I (8.7 percent to 100 percent).

D. DISCUSSION AND COMPARISON OF DERIVED EQUATIONS

The derived equations for both rural and urban watersheds are summarized and discussed in this section. Comparison is made between the derived equations and the results published by other investigators.

1. Time of Rise. For both the urban and rural data the length and slope of the main channel were found to be significant parameters. Because of urban development two additional parameters, impervious cover and the factor Φ were introduced in the urban equation. The derived equation 13 for the time of rise on a rural watershed is

$$T_{RR} = 2.65\, L^{0.12}\, S^{-0.52}$$

Equation 13 predicts the values of T_{RR} within $\pm$ 12 percent for two-thirds of the 11 rural watersheds.

The derived equation 17 for the time of rise on an urban watershed is

$$T_{RU} = 20.8\ \Phi\ L^{0.29}\ S^{-0.11}\ I^{-0.61}$$

which predicts the value of T_{RU} within $\pm$ 35 percent for two-thirds of the 22 urban watersheds.

The functional forms of equation 13 has also been studied by Kirpich (1940) for small rural watersheds located in the far West, and by Chow (1962) for watersheds in the midwest. Ramser (1918) analyzed data collected over a six month period by the U. S. Department of Agriculture for six small watersheds (1.25 to 112 acres) located in California. The storms analyzed were not uniform and followed no particular pattern. Only high intensity storms were considered in the study. Approximately 10 storms were studied for each watershed. Kirpich (1940) applied the results of Ramser's study to other small watersheds. Kirpich developed a relationship between the time of rise and a geometric factor, $L/\sqrt{s}$ (Figure 8)

$$T_{RR} = 0.0078\ (L/\sqrt{s})^{0.77} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (38)$$

where L is the length of basin area in feet, measured along the water course from the gaging station, and s is defined as H/L, where H is the fall in feet of the basin from the farthest point on the basin to the gaging station. The relationship was extended further by the California Department of Public Works (1944) on the basis of studies by the U. S. Soil Conservation Service in California. The upper end of Kirpich's curve in Figure 8 is defined by these studies. Chow (1962) derived a relationship between lag time, T_2, as defined in Table 6 and shown in Figure 9, and the same geometric factor, $L/\sqrt{s}$, for twenty small watersheds (2.79 to 4,580 acres) located in the mid-west, which is

$$T_2 = 0.0324\ (L/\sqrt{s})^{0.64} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (39)$$

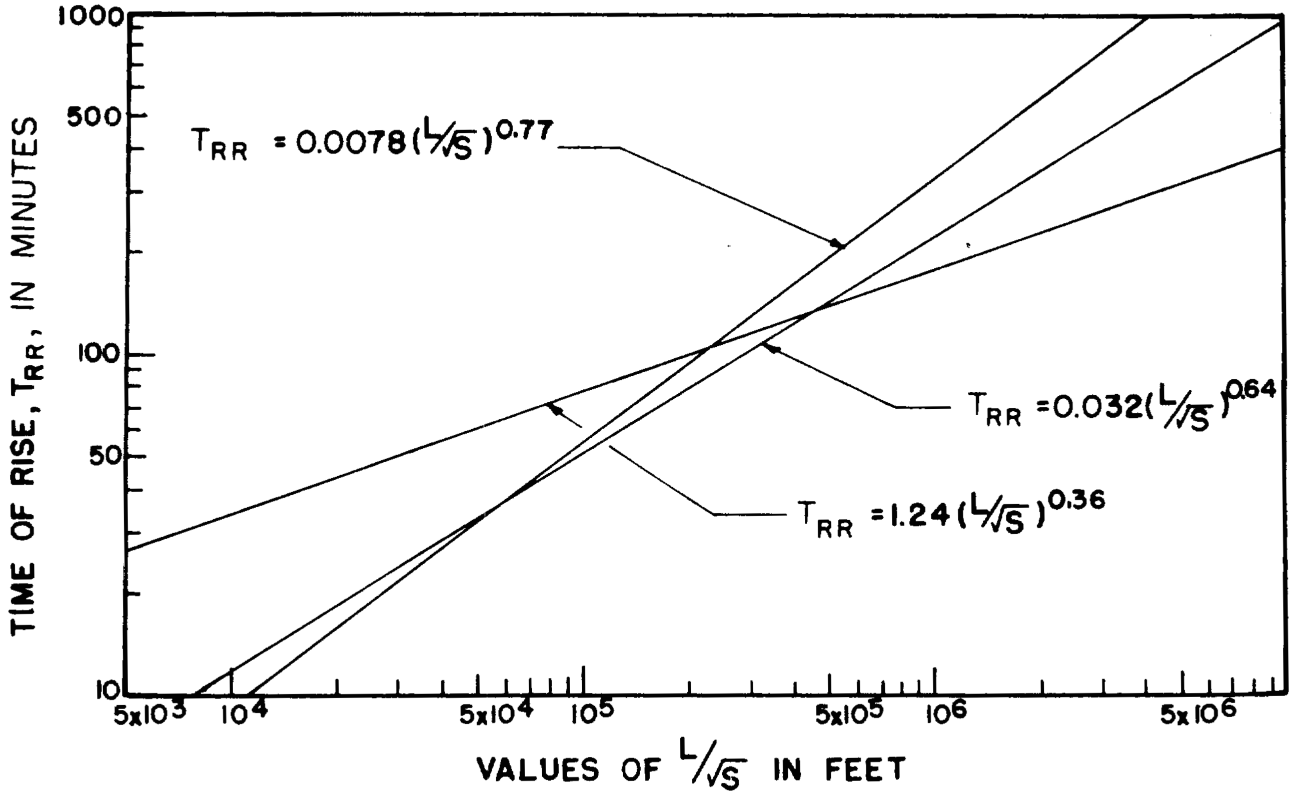


FIGURE 8. RURAL RELATIONSHIPS BETWEEN T_{RR} AND $L/\sqrt{S}$

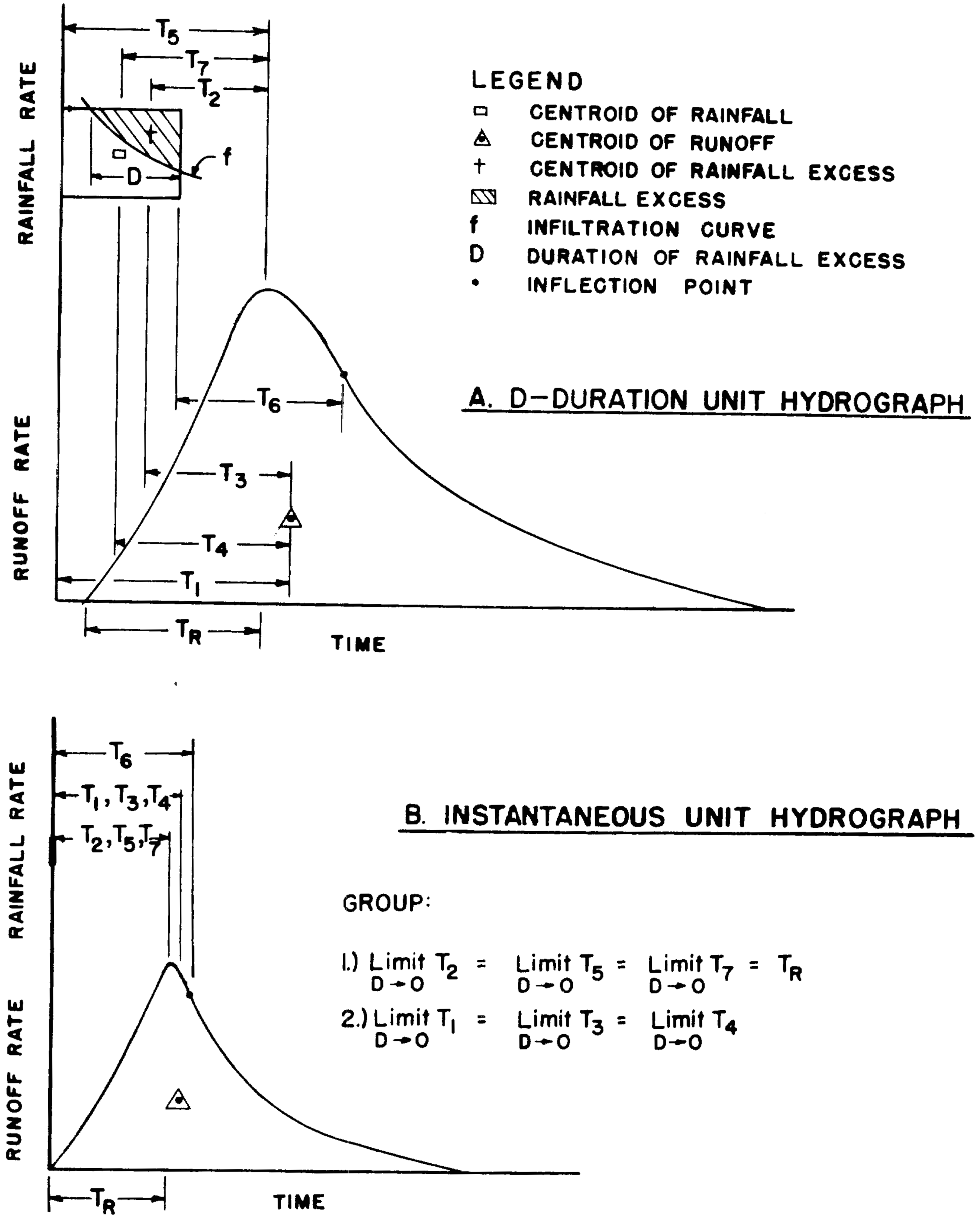


FIGURE 9. COMPARISON OF LAG TIMES FOR A D-DURATION AND INSTANTANEOUS UNIT HYDROGRAPH.

Equation 39 is also shown in Figure 8. Chow stated that for small watersheds the lag time, T_2, is approximately equal to the time of rise. Short duration, high intensity thunderstorms are usually found to be the type of storm that best satisfies the unit storm criterion for the study of unit hydrograph characteristics of small watersheds. The duration of these storms are short so that the total amount of rainfall excess is concentrated near the beginning of the period of rise. Therefore the resulting hydrograph may approach the instantaneous hydrograph (Chow, 1962) resulting in the following two groups of lag time: (1) $T_2 = T_5 = T_7 = T_R$, and (2) $T_1 = T_3 = T_4 \cong T_6$, while T_6 differs in definition from the other lag times in Group 2, its value will be approximately equal to the lag times in this group. Thus based on Chow's argument a direct comparison can be made between equations 38 and 39 (Figure 8) for small drainage areas. Also presented in Figure 8 is equation 15

$$T_{RR} = 1.24\ (L/\sqrt{s})^{0.36}$$

derived for the 11 rural watersheds considered in this study and located in Texas, Oklahoma and New Mexico. The relationships derived by Chow and Kirpich were found to be in fairly good agreement with one another, while the relationship developed from the Texas, Oklahoma and New Mexico data indicates a longer time of rise in the lower range of $L/\sqrt{s}$. Analysis of the data used to derive Chow's and Kirpich's equations indicated that the majority of watersheds studied were less than 400 acres; whereas of the watersheds located in Texas, Oklahoma and New Mexico, six are greater than 2,100 acres with one 807 acres, three between 90 and 100 acres and the smallest 22 acres. Direct comparison of these derived relationships is based on the assumption that watersheds are so small that the discharge hydrographs approach the instantaneous hydrograph. This assumption was not satisfied for the Texas, New Mexico and Oklahoma watersheds because of their size. Therefore some differences between the derived expression equation 15 and Chow's and Kirpich's equations may be expected.

2. Peak Discharge. The peak discharge characteristics for both the urban and rural data were found to be best expressed as functions of the drainage area and time of rise. The resulting multiple regression equation 21 for the rural data is

$$Q_R = 1.70 \times 10^3 A^{0.88} T_{RR}^{-0.30}$$

which predicts values of the peak discharge within $\pm$ 25 percent for two-thirds of the rural watersheds. For the urban data the multiple regression equation 22 is

$$Q_U = 1.93 \times 10^4 A^{0.91} T_{RU}^{-0.94}$$

which predicts values of the peak discharge within $\pm$ 33% for two-thirds of the urban data. The functional form of equations 21 and 22 indicated that perhaps a general relationship could be derived for both the urban and rural watersheds in which the time of rise would reflect the various differences in geometry and surface properties of the urban and rural watersheds. For convenience in plotting, an equation is proposed of the form

$$Q/A = W\, T_R^{\,m} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (40)$$

The resulting regression equation based on all watershed data is

$$Q/A = 4.09 \times 10^4\, T_R^{-1.11} \quad \ldots\ldots\ldots\ldots\ldots \quad (41)$$

with a correlation coefficient of 0.947 and a standard error of estimate of 187 cfs/mi^2. The equation, which is illustrated in Figure 10, predicts values of Q/A within $\pm$ 34 percent for two-thirds of the combined urban and rural watersheds. Approximately 90

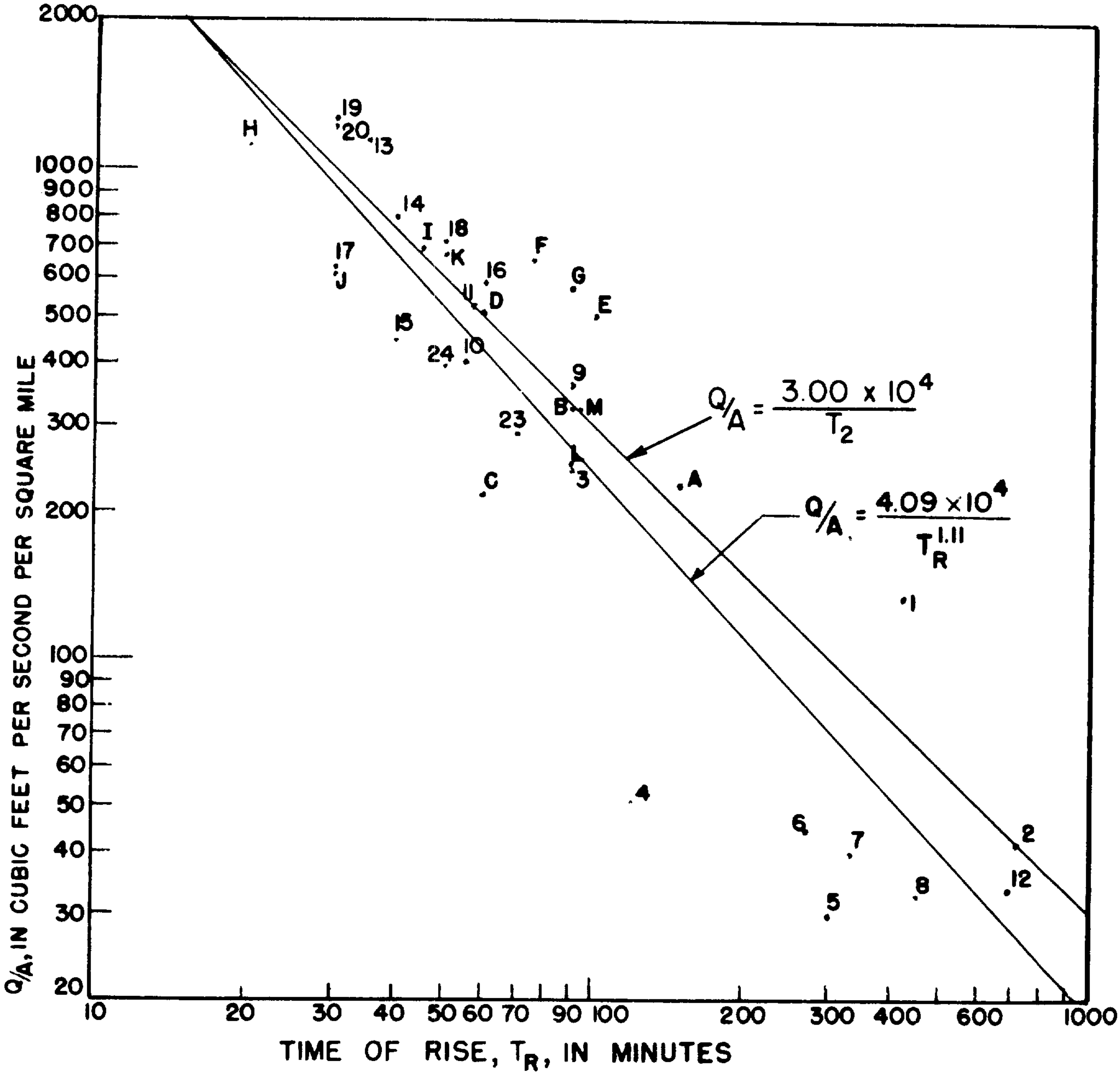


FIGURE 10. RELATIONSHIP BETWEEN Q/A AND T_R – GENERAL FORM.

percent of the variance of Q/A was explained by equation 41.

The forms of equations 21, 22 and 41 are all similar to the equation developed by Snyder (1938). Snyder found a relationship between the peak discharge, Q, area, A, and T_2 in hours which could be expressed as

$$Q = J\, A^{1.0}\, T_2^{-1.0} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (42)$$

for storms in which the duration, D, was given by

$$D = T_2 / 5.5 \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (43)$$

Snyder found values of the constant, J, to vary between 360 and 440 for the Appalachian Highlands. Coefficients determined for watersheds in Texas by the U. S. Army District, Fort Worth, Texas (Meier, 1964) were found to vary from 310 for the Neches River above Dam B to 600 for the Hords Creek above Hords Creek Dam. Therefore, the constant J in equation 42 for T_2 expressed in minutes would vary in Texas from 2.16×10^4 to 3.6×10^4 compared to the values of 1.70×10^3, 1.93×10^4 and 4.09×10^4 in equations 21, 22 and 41 respectively. Using an average value of 3.0×10^4 for J in Texas, Snyder's equation 42 is also presented in Figure 10. Variation in the values of J is probably due in part to the quantity and range of data used to derive the equations. The rural equation was derived based on eleven watersheds whereas the urban and general equations were based on 22 and 33 watersheds respectively. The low value of the exponent on the time of rise for the rural equation is believed to be the result of the small sample of data used to derive the expression. Variations in the size of watersheds, geographical location, and storm duration, all probably contribute to some of the variation in the J values.

3. Hydrograph Widths. The hydrograph width characteristics for both urban and rural data were found to be best expressed by an equation of the form

$$\text{Hydrograph Widths} = C\,A^m\,Q^n \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (44)$$

The relationship for the hydrograph widths was also found to considerable improved by the introduction of the peak discharge. Peak discharge also reflects the integrated effects of the complex physical system of the watershed on the storm runoff. The resulting multiple regression equations for the three different hydrograph widths for the rural data are

$$T_{BR} = 7.41 \times 10^3\,A^{0.64}\,Q_R^{-0.53}$$

equation 26 which predicts values within $\pm$ 21 percent for two-thirds of the rural watersheds and explains 95 percent of the variance of T_{BR};

$$W_{50R} = 7.37 \times 10^4\,A^{1.11}\,Q_R^{-1.13} \quad ,$$

equation 27 which predicts values within $\pm$ 10 percent for two-thirds of the rural watersheds and explains 99 percent of the variance of W_{50R}; and

$$W_{75R} = 4.46 \times 10^4\,A^{1.06}\,Q_R^{-1.13} \quad ,$$

equation 28 which predicts values within $\pm$ 20 percent for two-thirds of the rural watersheds and explains 95 percent of the variance of W_{75R}.

The urban data was found to be best described by the equations which are

$$T_{BU} = 4.44 \times 10^5\,A^{1.17}\,Q_U^{-1.19} \quad ,$$

equation 29 which predicts values within $\pm$ 39 percent for two-thirds of the urban watersheds and explains 89 percent of the variance of T_{BU};

$$W_{50U} = 4.14 \times 10^4 A^{1.03} Q_U^{-1.04} \quad ,$$

equation 30 which predicts values within $\pm$ 16 percent for two-thirds of the urban watersheds and explains 95 percent of the variance of W_{50U}; and

$$W_{75U} = 1.34 \times 10^4 A^{0.92} Q_U^{-0.94} \quad ,$$

equation 31 which predicts values within $\pm$ 25 percent for two-thirds of the urban watersheds and explains 93 percent of the variance of W_{75U}. With the introduction of a hydrologic parameter, Q/A, into equation 44, the urban and rural data were combined to derive three general relationships. For convenience in plotting an equation is proposed of the form

$$\text{Hydrograph Widths} = Y\,(Q/A)^P \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (45)$$

The resulting equation for the time base expressed in minutes is

$$T_B = 3.18 \times 10^4 / (Q/A)^{1.13} \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (46)$$

with a correlation coefficient of 0.974, significant at the one percent level, and a standard error of estimate of 846 minutes. Equation 46 predicts values for two-thirds of the urban and rural watersheds within $\pm$ 28 percent and explains 90 percent of the variance (Figure 11). Examination of Figure 11 suggests that equation 46 derived from both urban and rural data can be used to describe the time base characteristics of both urban and rural watersheds. For the hydrograph width at 50 percent the equation is

$$W_{50} = 3.88 \times 10^4 / (Q/A)^{1.025} \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (47)$$

where W_{50} is expressed in minutes or

$$W_{50} = 647 / (Q/A)^{1.025} \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (48)$$

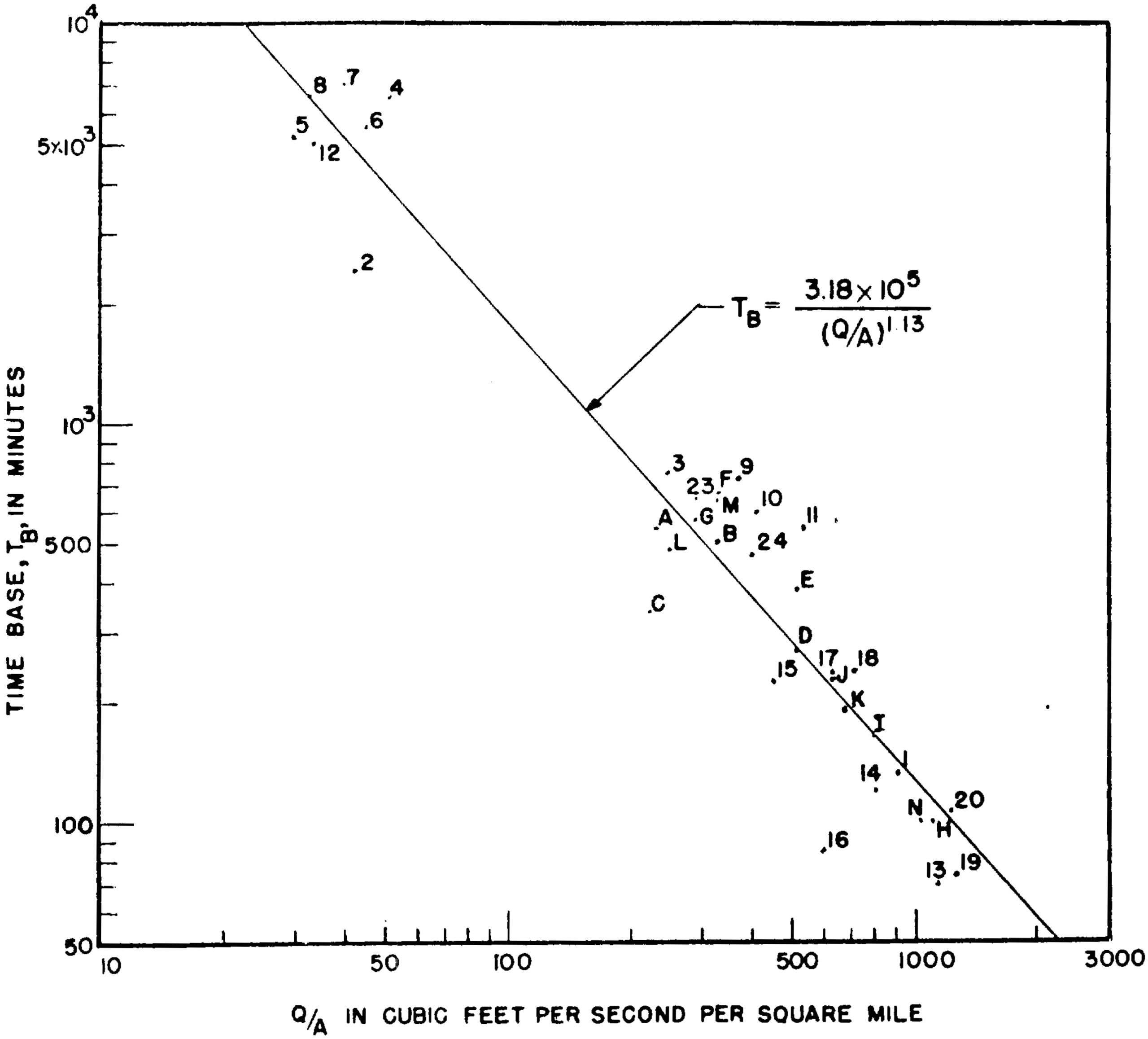


FIGURE 11. RELATIONSHIP BETWEEN Q/A AND T_B — GENERAL FORM.

when W_{50} is expressed in hours, with a correlation coefficient of 0.977, significant at the one percent level, a standard error of estimate of 98 minutes. Equation 47 predicts values for two-thirds of the urban and rural watersheds within $\pm$ 24 percent and the equation also explains 94 percent of the variance (Figure 12). Similarly the equation for the hydrograph width at 75 percent is

$$W_{75} = 1.00 \times 10^4/(Q/A)^{0.89} \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (49)$$

where W_{75} is expressed in minutes or

$$W_{75} = 167/(Q/A)^{0.89} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (50)$$

where W_{75} is expressed in hours, with a correlation coefficient of 0.968, significant at the one percent level, and a standard error of estimate of 60 minutes. Equation 49 predicts values for two-thirds of the urban and rural data within $\pm$ 14 percent and the equation explains 96 percent of the variance (Figure 13).

The U. S. Army Corps of Engineers (1959) made a study of unit hydrographs for a large number of drainage basins which indicated a relationship between the unit hydrograph peak discharge per square mile and W_{50} and W_{75}. Curves for W_{50} and W_{75} (Figures 12 and 13) were drawn by the Corps to envelop the majority of values of the unit hydrograph widths. The data were obtained from a study of a large number of unit hydrographs for drainage basins of various configurations and runoff characteristics. The equations for the relationships expressed by Figures 12 and 13 (Chow, 1964) are

$$W_{50} = 4.64 \times 10^4\ (A/Q)^{1.08} \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (51)$$

and

$$W_{75} = 2.64 \times 10^4\ (A/Q)^{1.08} \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (52)$$

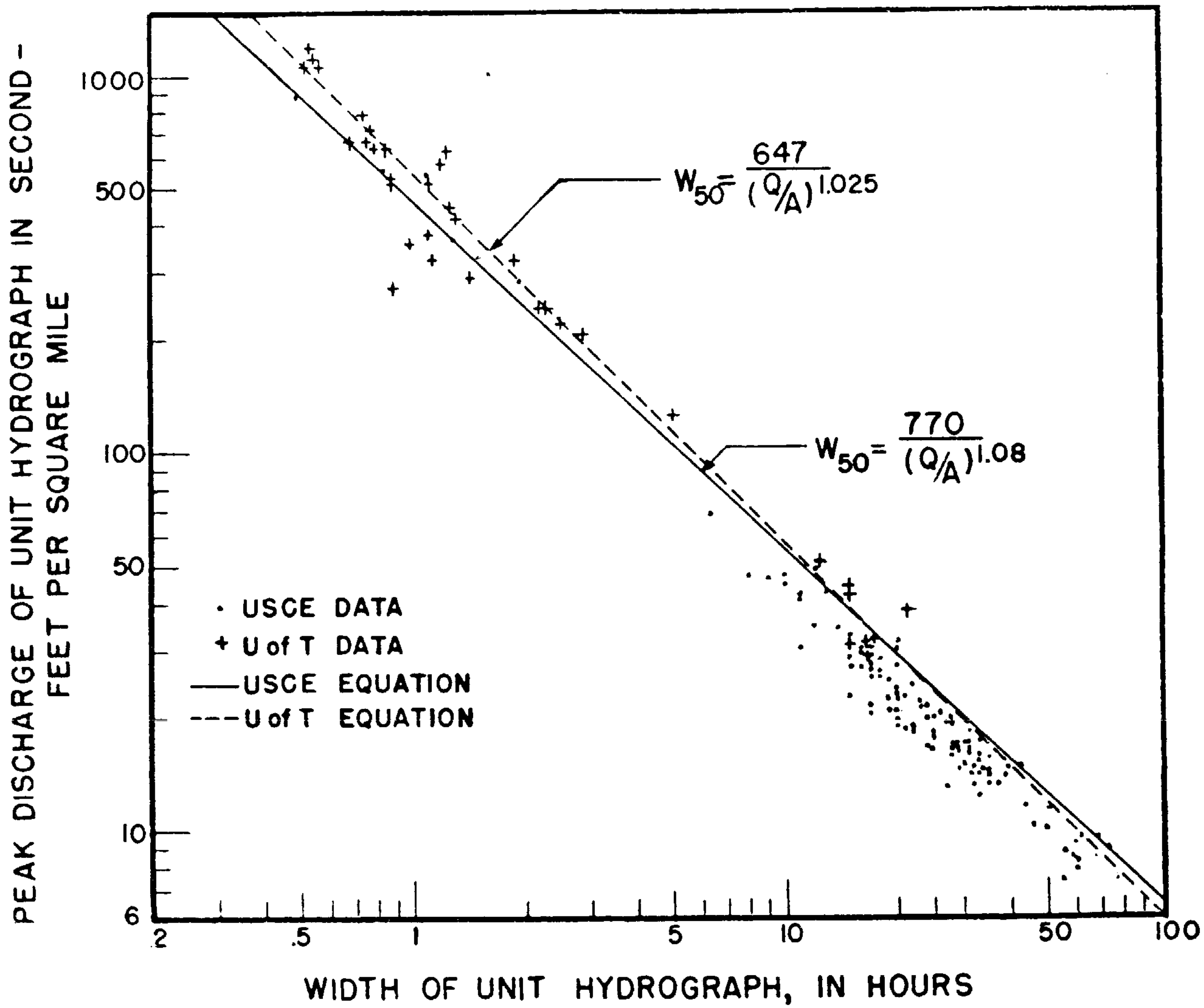


FIGURE 12. RELATIONSHIP BETWEEN Q/A AND W_{50}—GENERAL FORM

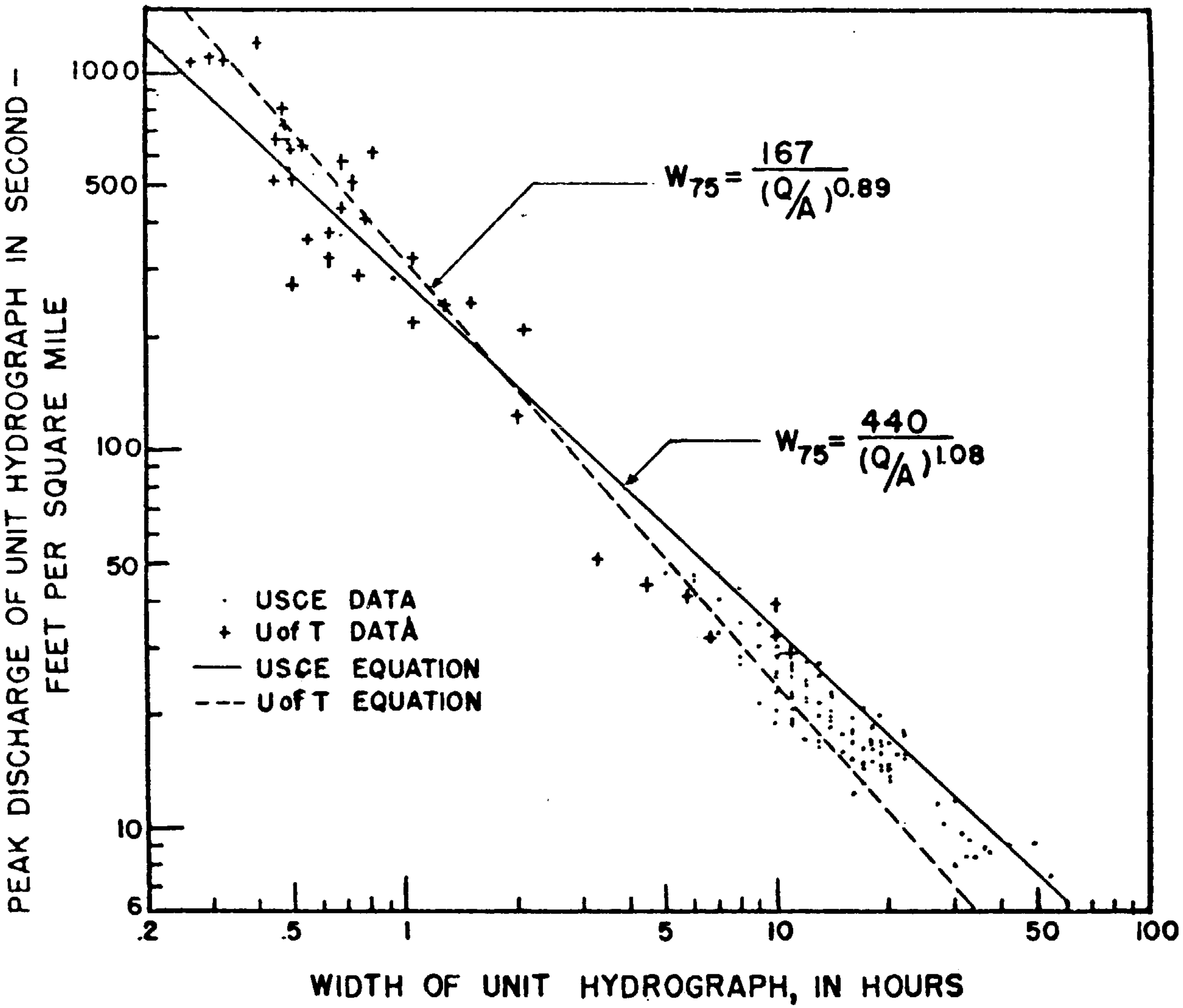


FIGURE 13. RELATIONSHIP BETWEEN Q/A AND W_{75}—GENERAL FORM

where W_{50} and W_{75} are in minutes or

$$W_{50} = 770 \times 10^4 \, (A/Q)^{1.08} \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (53)$$

and

$$W_{75} = 440 \times 10^4 \, (A/Q)^{1.08} \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (54)$$

where W_{50} and W_{75} are expressed in hours. The general relationship shown in Figures 11, 12 and 13 indicates that the functional form of equations 46, 48 and 50 can be used for all types of watersheds. The comparison of the derived relationships for W_{50} and W_{75} with the expressions developed by the U. S. Army Corps of Engineers for a large and varied group of unit hydrograph data indicates that derived equations 48 and 50 (Figures 12 and 13) can be used for any type of watershed and for any storm duration provided the peak flow is known for that duration.

4. Lag Time. Scharake, Geyer and Knapp (1964) studied the lag time, T_4, characteristics of 14 urban watersheds, tabulated in Appendix C. The lag time, T_4, was found to fall within a range of durations for which the best correlation between peak runoff rates and average rainfall intensities could be obtained. Therefore, this lag time, which is a measure of the time required for the runoff to flow through the drainage area, was selected as the value of T_c to be used in their study. A multiple regression analysis of their data was made to obtain an equation for T_4. The resulting equation is

$$T_4 = 2.13 \, L^{0.27} \, S^{-0.13} \, I^{-0.38} \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (55)$$

Equation 36 which is based on 28 watersheds is

$$T_4 = 1.90 \, L^{0.35} \, S^{-0.26} \, I^{-0.35}$$

Probably part of the reason for the variation in the regression coefficient in equations 55 and 37 is the number of watersheds used to derive the equations. Equation 55 was based on 14 watersheds while equation 37 was based on 28 watersheds. Part of the reason for the variation in the regression coefficients in equations 55 and 37 may be the difference in the range of watershed lengths used to derive each equation. Equation 55 is based on a range of L from 290 feet to 2,264 feet; whereas the derived equation 37 is based on a range of L from 280 feet to 27,500 feet.

5. <u>Comparison of Derived Rural Hydrograph with Commons' and Mockus' Hydrographs.</u> In 1959 Diehl made a study of the peak discharge for the Waller Creek watershed using the methods of Commons (1942) and Mockus (1955). His results are shown in Figure 14.

To compare the rural hydrograph that can be obtained from the statistical equations derived in section C of this chapter a unit storm duration of 15 minutes was selected. Based on rural equations the estimated peak discharge for Waller Creek at 23rd Street under rural conditions is 1,460 cfs compared to 1,390 cfs based on Commons' method and 1,410 based on Mockus' method (Figure 14). Also presented in Figure 14 is the derived rural 30 minute unit hydrograph derived from the rural equations 13, 21, 26, 27 and 28, and adjusted to represent one inch of runoff. For all practical considerations the peak discharge determined by the Commons method, Mockus method and the rural equation is the same.

6. <u>Application of Urban Equations to Beargrass Creek Watershed.</u> The derived urban equations were applied to a small urban watershed which had not been used in the study in order to further test their accuracy. Unit hydrograph data was available for a small urban watershed which was part of the Louisville, Kentucky flood control program (Figure 15). The Beargrass Creek basin contains two types of areas with totally different

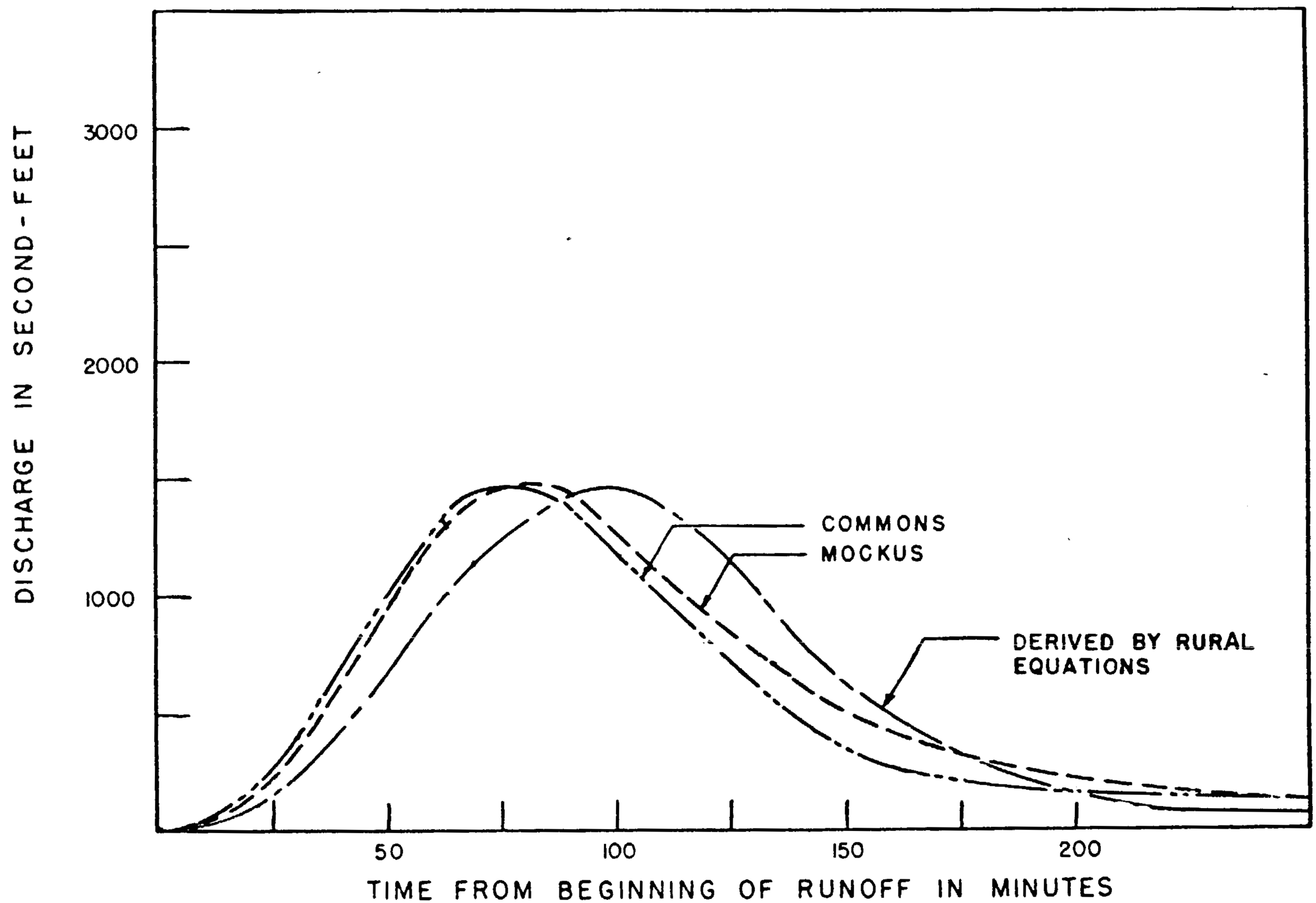
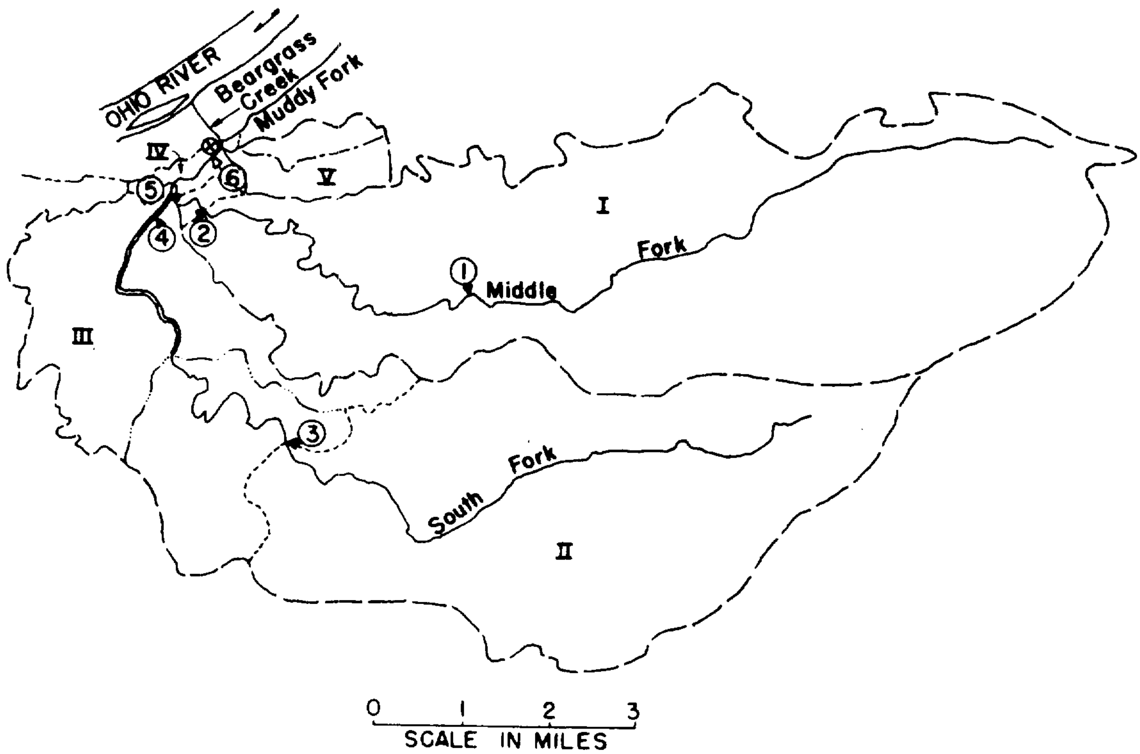


FIGURE 14. COMPARISON OF COMMONS', MOCKUS', AND DERIVED UNIT HYDROGRAPH .AT 23 rd STREET.



GAGING STATIONS

1. Middle Fork at Cannons Lane
2. Middle Fork at Payne Street
3. South Fork at Trevillian Way
4. South Fork at Baxter Avenue
5. Beargrass Creek at Main Street
6. Beargrass Creek at Brownsboro Road

LEGEND

—— Drainage Divide
------ Drainage Sub-divide
—··— Upstream Limit of Sewered Area
—·— Brownsboro Road Sewer
——— Creek (Paved Section)
—··— Line of Protection
⊗ Proposed Pumping Plant
▼ Recording Stream Gage
▽ Non-recording Stream Gage

DRAINAGE AREAS

I Middle Fork above Payne Street (25.1 Sq. Mi.)
II South Fork above Trevillian Way (17.6 Sq. Mi.)
III Area above Main Street and below Payne Street and Trevillian Way (9.7 Sq. Mi.)
IV Area between Pumping Plant and Main Street (0.3 Sq. Mi.)
V Brownsboro Road Trunk Sewer (1.2 Sq. Mi.)

NOTE:
Unit Hydrographs have been developed from stream-flow records for areas I, II, and III.

FIGURE 15. BEARGRASS CREEK DRAINAGE AREA (AFTER, USCE, 1946)

runoff characteristics: (1) The rural and suburban areas in the upstream portions of the basins of South Fork and Middle Fork; (2) Urban area in the downstream portion of South Fork basin, along the main channel, and in the downstream portion of the Middle Fork basin. The Beargrass Creek basin above Main Street was divided into three areas for the development of unit hydrographs: (1) South Fork basin above Trevillian Way gaging station; (2) Middle Fork above Payne Street gaging station; (3) Area upstream from Main Street and downstream from the gaging station at Trevillian Way and Payne Street. The rapid runoff characteristics of the areas downstream from the Payne Street and Trevillian Way gaging stations and upstream from the Main Street gaging station allowed the separation of the discharge hydrograph for the urban area between Main Street and Trevillian Way (Figure 15). This area contains 9.7 square miles, of which 6.3 square miles is intensively developed urban area with a well-developed storm sewer system. Table 7 contains various physical factors which were used in the derived equations to predict the 30 minute unit hydrograph. An average value of 0.7 was selected

FACTOR	VALUE
Area, A	9.7 mi^2
Length, L	5.6 mi
Slope, S	0.0012
Impervious*	70 %

TABLE 7. Physical Characteristics of the Beargrass Creek Watershed.

* Estimate based on description of area and Eagleson's work on similar data in Louisville.

for Φ because of the extensive channel improvement in the lower part of the watershed and sewer lines located throughout the watershed. The predicted 30 minute unit hydrograph for Beargrass Creek based on the derived equations, 17, 22, 29, 30, 31, and adjusted to represent one inch of runoff, is compared with the measured unit hydrograph in Table 8 and Figure 16. The predicted and measured unit hydrographs were found to be in good agreement. The assumption is therefore made that the urban equations can be used to describe the unit hydrograph characteristics of urban watersheds.

Hydrologic Characteristics	Measured Values	Predicted Values	Difference % of Measured Value
Q, cfs	4,700	4,400	- 6
T_R, min	62	43	- 31
T_B, min	350	294	- 19
W_{50}, min	68	70	+ 3
W_{75}, min	40	42	+ 5

TABLE 8. Comparison of Predicted and Measured Unit Hydrograph Characteristics for Beargrass Creek Watershed.

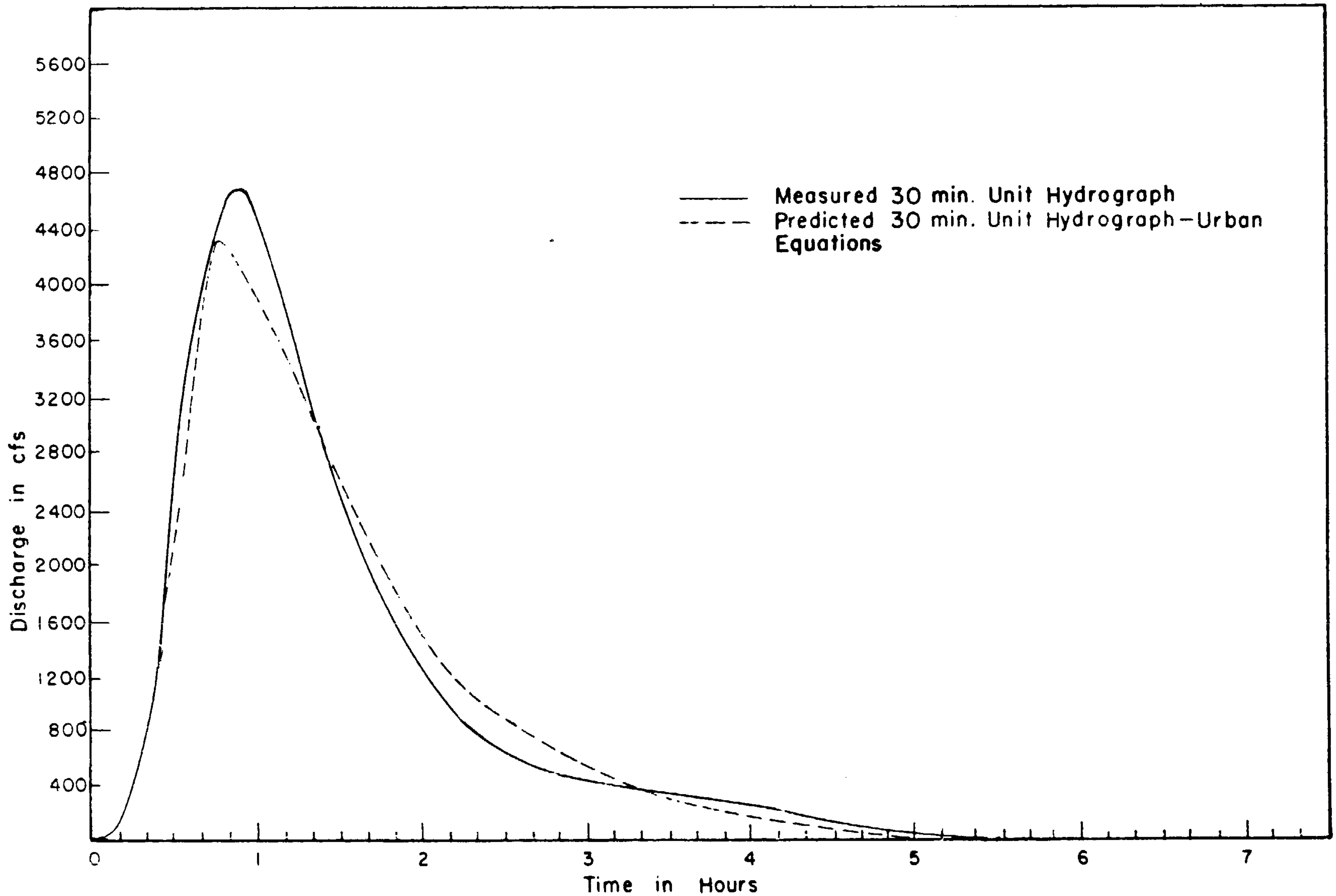


FIGURE 16. COMPARSION OF PREDICTED AND MEASURED UNIT HYDROGRAPH FOR BEARGRASS CREEK ABOVE MAIN STREET AND BELOW PAYNE STREET AND TREVILLIAN STREET.

Chapter III

APPLICATION OF EMPIRICAL EQUATIONS TO THE WALLER CREEK WATERSHED

The U. S. Geological Survey has initiated collection of hydrologic data from several urban watersheds located throughout the United States. As a part of this program the Austin District of the U. S. Geological Survey has had a hydrologic data collection program since December 1954 for the Waller Creek watershed located in Austin, Texas. There were four primary reasons for the selection of the Waller Creek watershed: (1) Cooperation and support from the University of Texas, Texas Water Commission, U. S. Weather Bureau and City of Austin; (2) Favorable gaging station sites in the watershed; (3) Unique opportunity to compare the runoff properties of the more urbanized lower portion with the less developed upper portion of the watershed: and (4) To provide basic data for research in urban hydrology.

Because no data was obtained before urbanization and only nine years of record was available, the derived rural and urban equations of Chapter II are used to evaluate the effect of urbanization on the hydrologic characteristics of the Waller Creek watershed.

A. DESCRIPTION OF STUDY AREA

Waller Creek, a tributary of the Colorado River (Figure 17), lies entirely within the city limits of Austin, Texas. Located in the northern part of the city (Figure 18), the drainage area above the 23rd Street station is 4.13 square miles. The watershed centerline lies approximately in a northeast direction (Figure 18).

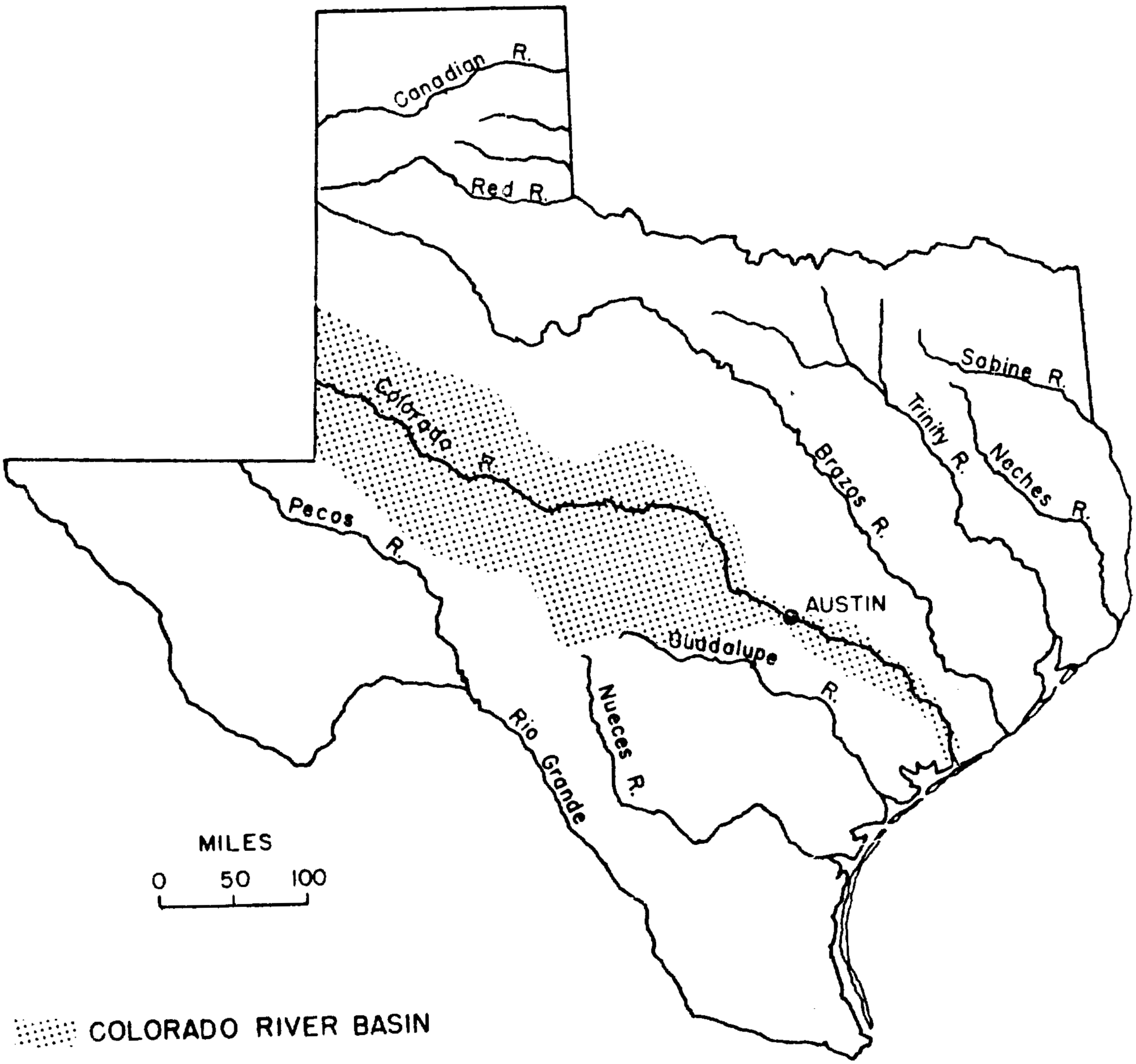


FIGURE 17. MAP OF TEXAS SHOWING LOCATION OF AUSTIN.

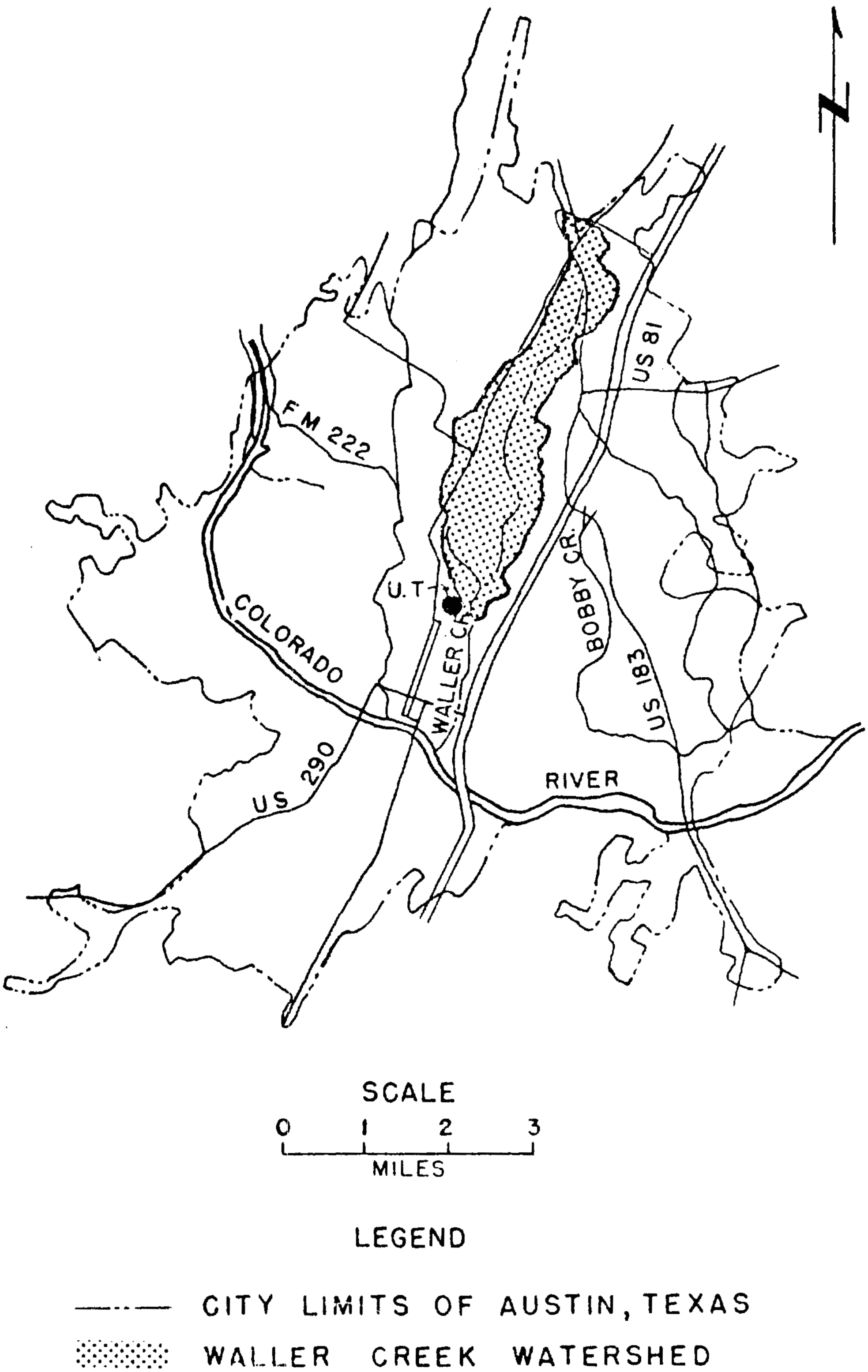


FIGURE 18. MAP OF AUSTIN, TEXAS SHOWING LOCATION OF WALLER CREEK WATERSHED.

1. Climate. The climate is generally mild and semihumid. March through October is the period of warm weather. Sudden changes of temperature are common. The ground seldom freezes, and then only at the surface. The mean annual precipitation at the U. S. Weather Bureau Airport Station, approximately one-half mile east of the watershed, is 33.36 inches. The rainfall is generally fairly well distributed throughout the year; however, individual rainfalls of excessive amounts occur at irregular intervals. The area is subjected to occasional intense precipitation that usually results from tropical or semi-tropical storms.

2. Geology. The Waller Creek watershed is located in the West Gulf Coastal Plain and is underlain by two bedrock formations and a thin alluvial formation. Eagle Ford Shale underlies the extreme northwestern part of the watershed, comprising most of the area west of Guadalupe Street (Figure 19). The majority of the remaining area is underlain by Austin Chalk. The Austin Chalk weathers to a very heavy black clay soil, which has a very low permeability. The bedrock formations are covered in the southern part by an alluvial terrace of the ancient Colorado River. The soil formed by the terrace is sandy material and has a high permeability. The terrace is well defined in the area east of Guadalupe, north of 32nd and south of 45th Streets (Figure 19).

3. Topography. The area consists of gently rolling, hilly land and is characterized by glaring white outcrops of limestone on the slopes and in the bluffs of the creek. The area is relatively long and narrow with a maximum width of 2.6 miles at 45th Street to 0.9 miles in the vicinity of Denson Drive (Figure 19). The average slope, S, of the main channel is 0.009 ft/ft and is fairly constant (Diehl, 1959). The average slope is defined as H/L where L is the maximum length of travel, in feet, and H is the difference in elevation, in feet, between the most remote point and the outlet. Based on the U.S.

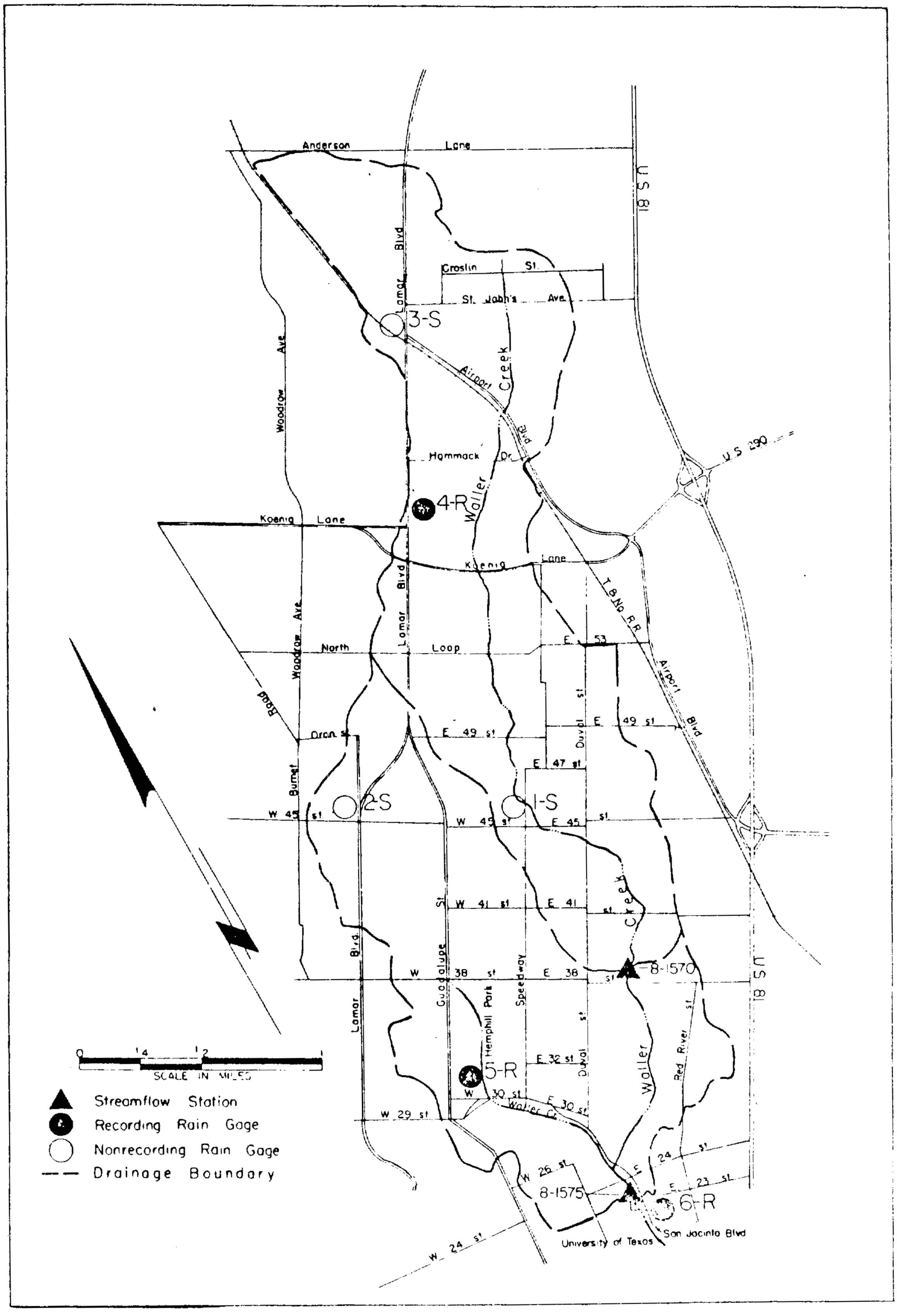


FIGURE 19. INSTRUMENTATION OF WALLER CREEK WATERSHED.

Corps of Engineers method, Eagleson (1962), the mean basin slope, $\bar{S}$, was found to be approximately the same as the average slope (Table 9). The mean basin slope, $\bar{S}$, is defined as $\bar{H}/\bar{L}$; $\bar{H}$ denotes the mean rise of watershed as given by the area under the area-rise (hypsometric) curve divided by the total basin area. Area-height curve represents the relationship of area above each elevation (elevation established by contours) plotted against the height above channel bottom at gaging station, and $\bar{L}$ is defined as the area under the area-distance curve divided by the total basin area. The area-distance curve represents the relationship of the basin area between the gaging station and each line of equal travel distance from the base gage, plotted against the corresponding distance of this line above the gaging station.

Station	$\bar{S}$ (ft/ft)	$\bar{H}$ (ft)	$\bar{L}$ (ft)	L_{ca} (ft)	S = H/L (ft/ft)	A (mi^2)	L (ft)
23rd Street	0.0124	134	10,800	10,500	0.009	4.13	27,600
38th Street	0.0126	125	9,940	13,000	0.009	2.31	23,080
Δ A*	0.0159	93.8	5,900	5,700	0.009	1.82	13,500

TABLE 9. Geometric Factors for Waller Creek Watershed.

4. Instrumentation. The watershed's basic instrumentation consists of two streamflow stations and five rain gages (2 non-recording and 3 recording) (Figure 19). The location of each gaging station and rain gage is shown in Figure 19. The two streamflow stations

* Area between 23rd and 38th Street stations in square miles.

are equipped with concrete Ellenville controls. The 23rd and 38th Street stations are equipped with standard A-10 Stevens recorders. Recently installed at the 23rd Street station is a locally designed float type rain gage and a Fischer and Porter Analog-to-Digital recorder (Figure 20). Both streamflow stations have a 9.6 inch per day time scale.

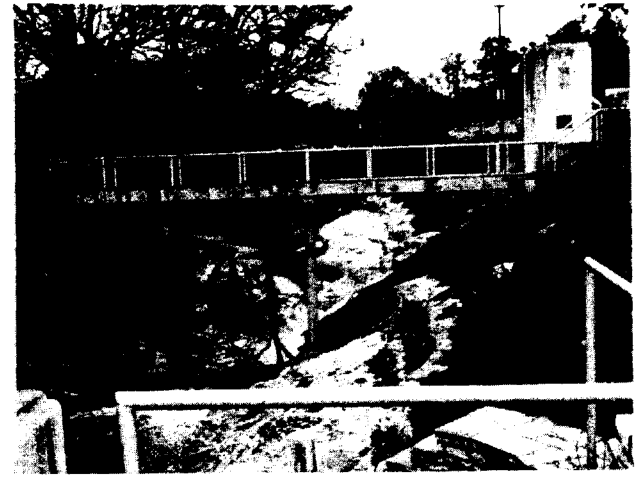
5. Drainage Conditions. The headwaters of Waller Creek are located south of Anderson Lane, in the northern part of the city. The main channel has been extended by excavation to the natural divide just north of Croslin Street (Figure 19 and Figure 21). A drainage ditch joins the main channel just south of where the main channel crosses Airport Boulevard. The drainage ditch was formed by the Texas and New Orleans Railroad track. The T & NO drainage ditch was found to contribute additional runoff from an area of 0.3 square miles which would normally drain into Shoal Creek to the west.

A second branch, called West Branch, originating in the general area of West 45th Street and Lamar, joins the main channel just west of San Jacinto Boulevard approximately two blocks above the 23rd Street stream gaging station. Beginning in the Hemphill Park area this second branch is a rock-lined channel varying in cross-section from trapezoidal to rectangular in shape between 32nd Street (Figure 21) and just south of West 30th Street where the rock lining ends.

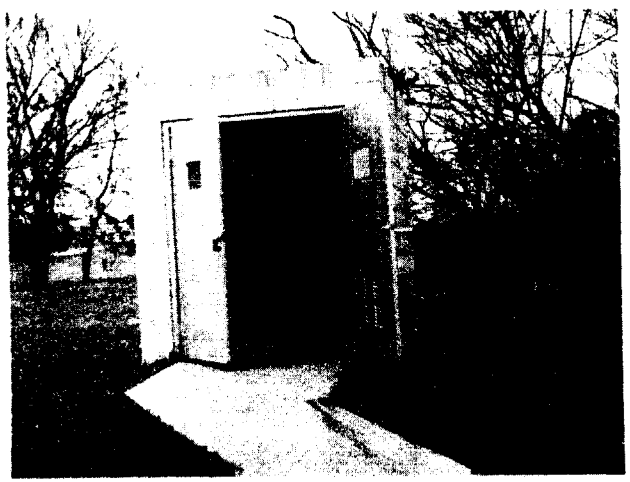
Based on field observations and studies of aerial photographs, it is estimated that approximately one-third of the basin is undeveloped with the remaining two-thirds classified as new business, old and new residential (Diehl, 1959). Many small diversions within the natural basin caused by storm sewers and embankments are present. As a result of these man-made diversions the drainage area corresponding to these diversions was determined by University of Texas surveying classes by observations in the field.

6. Impervious Cover. An extensive study was made of the impervious cover of the

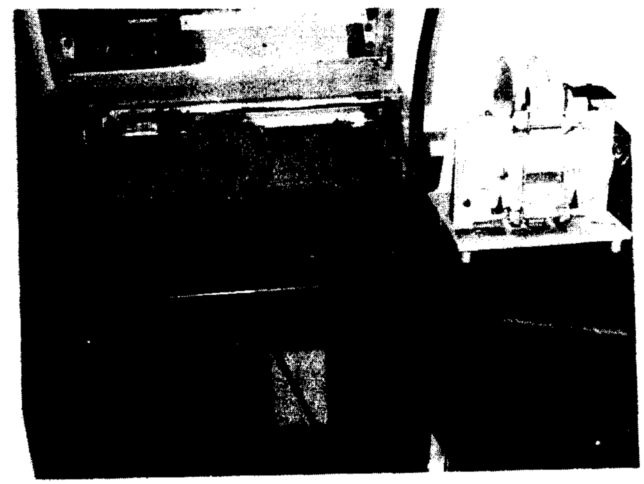
23rd STREET STATION



23rd STREET STATION

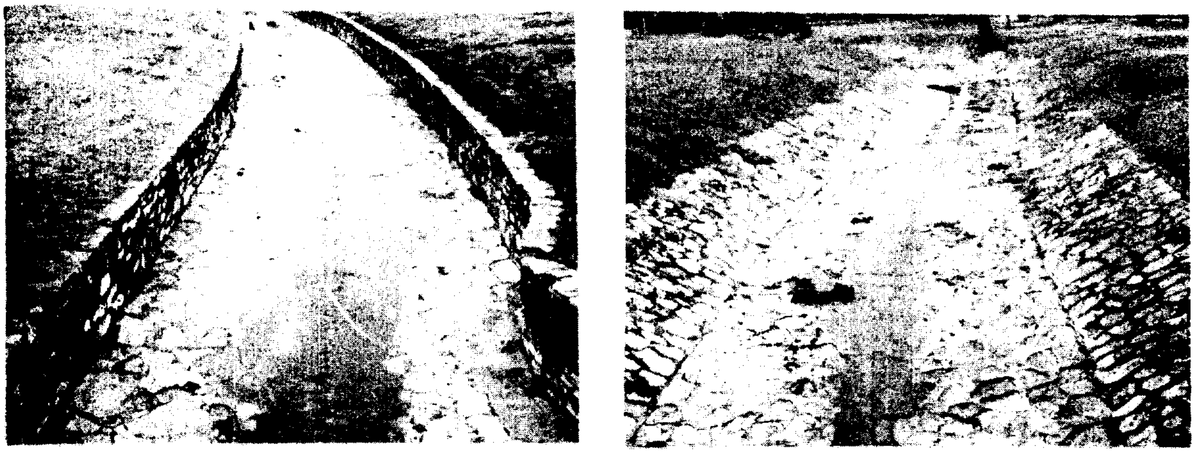


MEASURING EQUIPMENT SHELTER 38th STREET

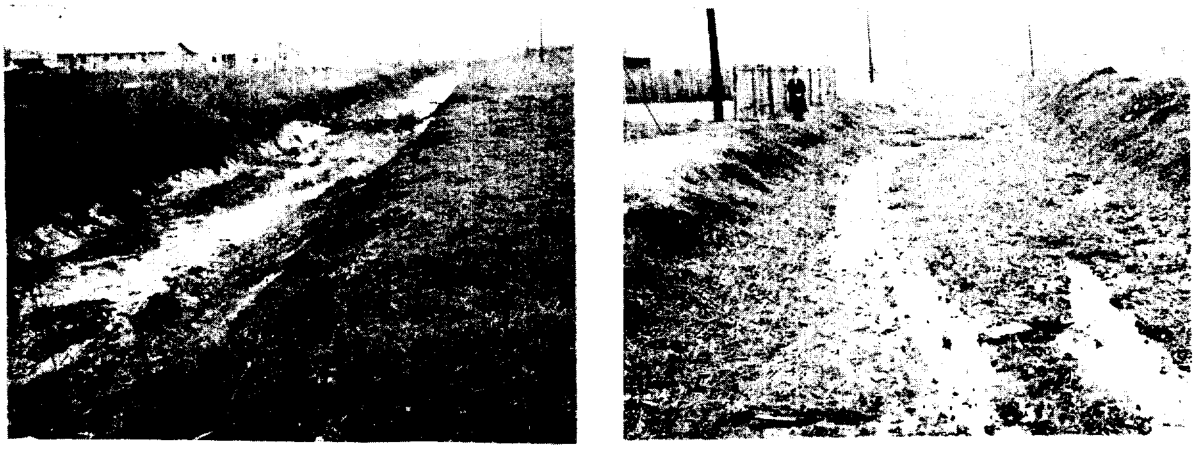


RECORDING EQUIPMENT 23rd STREET

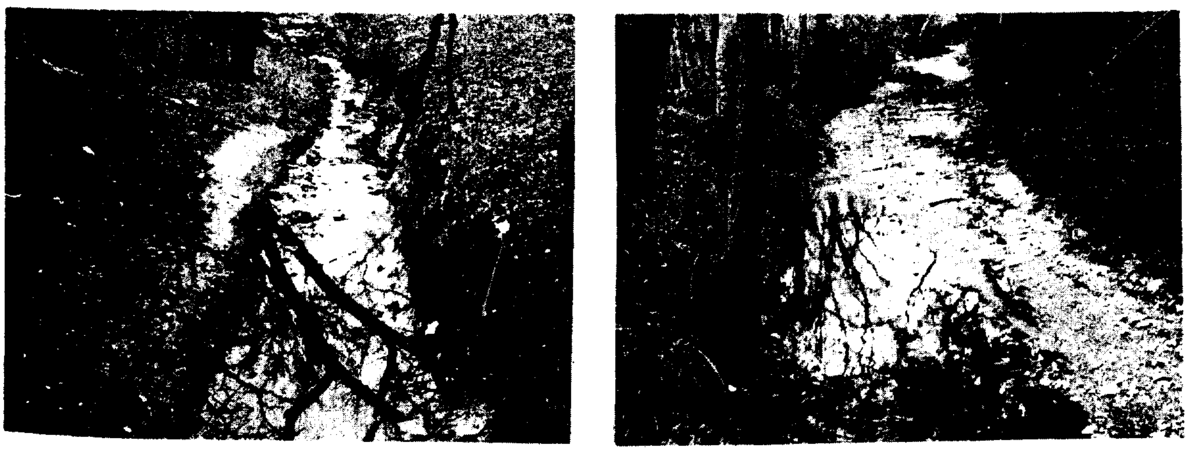
FIGURE 20. GAGING STATIONS



ROCK-LINED CHANNEL WEST BRANCH - HEMPHILL PARK



EXCAVATED CHANNEL ABOVE AIRPORT BOULEVARD



NATURAL CHANNEL

FIGURE 21. CHANNEL CONDITIONS.

Waller Creek watershed. Aerial photographs used were made in March 1954 and March 1962. Photo enlargements, 1" = 800' scale, were available for the 1954 flight; for the 1962 flight chronapague and chronaflex positive, enlarged 1" = 200', were also used. Records of the City Building Inspector, the Texas State Hospital, the University of Texas, the project files of the Department of Public Works, and two sets of aerial photographs were used to determine the chronological urban development of the watershed. The man-made impervious cover was measured directly from the 1962 flight photographs, 1" = 200'. Figure 22 summarizes the chronological development of the impervious cover for 23rd, 38th and the area located between the 23rd and 38th stations, ΔA, drainage areas.

B. EFFECTS OF URBANIZATION

In order to evaluate the effects of the present urban development, the rural equations 13, 21, 26, 27 and 28 were used to determine the unit hydrograph characteristics of the Waller Creek watershed as they might have existed under undeveloped conditions. The effects of future development were evaluated for different values of impervious cover by application of the derived urban equations 17, 22, 29, 30 and 31. The results from both the urban and rural equations are then combined and presented graphically. These results are computed for both the 23rd Street and 38th Street gaging station locations.

1. 23rd Street Station. The results of the application of the rural and urban equations to Waller Creek watershed at 23rd Street are shown in Figure 23. The effects of projected future development on the time of rise, T_{RU}, were introduced by increasing the impervious cover, I, in equation 17, from its present value of 27 percent to 50 percent. The value of Φ of 0.8 is assumed constant during this development. Comparison of the

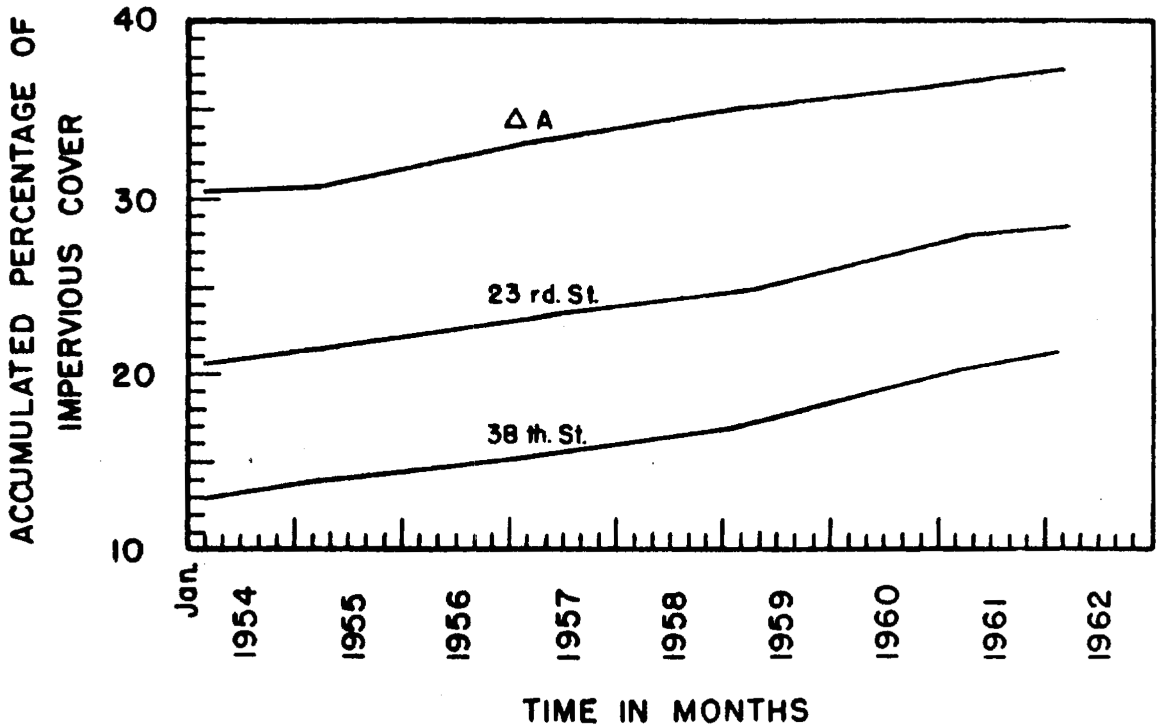


FIGURE 22. IMPERVIOUS COVER OF WALLER CREEK WATERSHED.

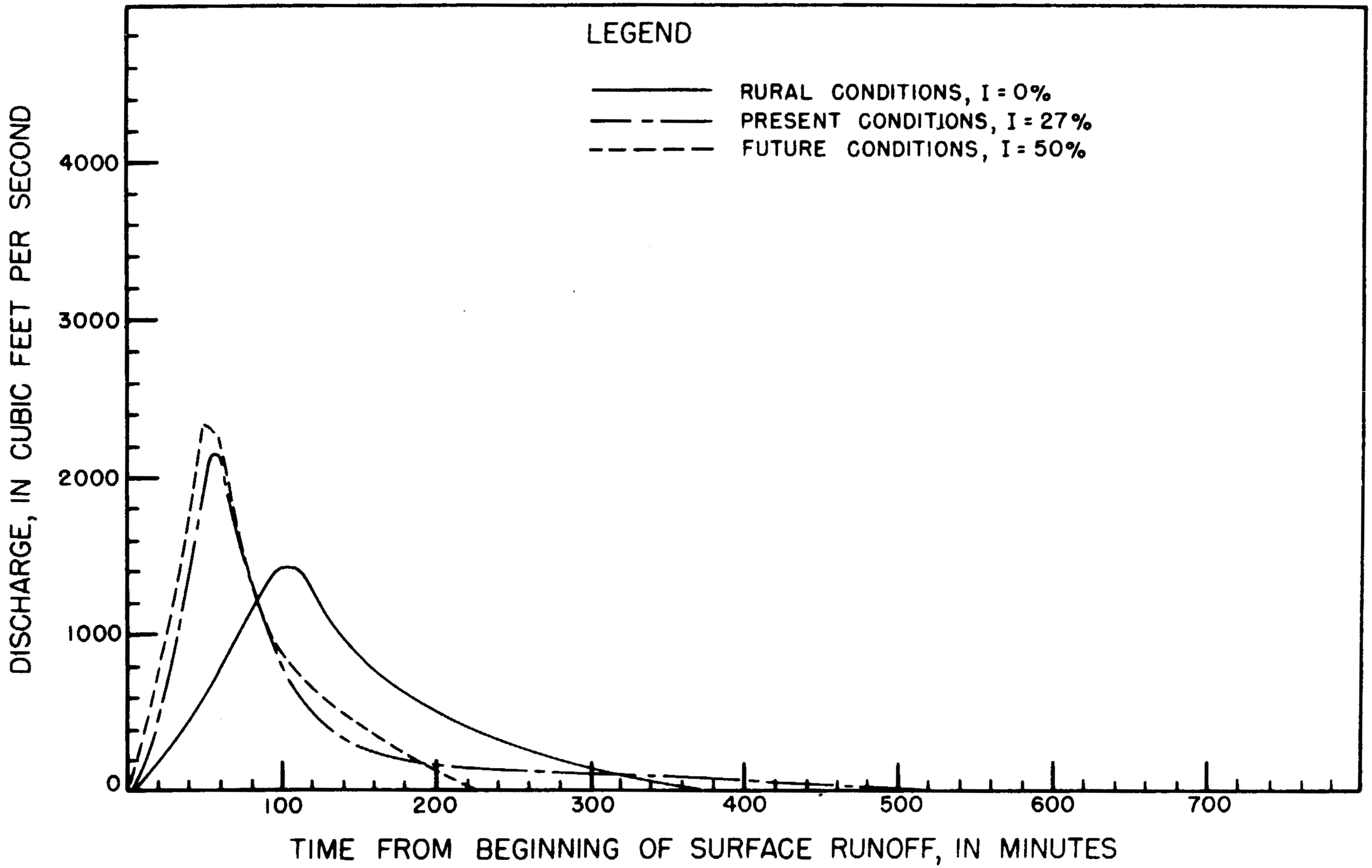


FIGURE 23. EFFECTS OF FUTURE URBAN DEVELOPMENT ON THE UNIT HYDROGRAPH AT THE 23rd STREET GAGING STATION.

present unit hydrograph and the rural hydrograph indicates that the peak discharge has increased approximately 51 percent and the time of rise has decreased 46 percent due to present urbanization as compared with rural conditions. The effect of future development will continue this trend of increased peak discharge and reduced time of rise resulting in an increase in the peak discharge of 62 percent and a reduction in the time of rise of 52 percent at 50 percent impervious cover. The results of the effects of urbanization on the hydrograph characteristics of the Waller Creek watershed at 23rd Street are summarized in Table 10 and shown in Figure 23.

Stage of Development	Time of Rise (minutes)	Percent Difference Based on Rural Values	Peak Discharge (cfs)	Percent Difference Based on Rural Values
Rural I = 0 %	105	0	1,460	0
Present I = 27 %	57	- 46 %	2,200	+ 51 %
Future I = 50 %	50	- 52 %	2,360	+ 62 %

TABLE 10. Summary of Some Effects of Present and Future Urban Development on the Waller Creek Watershed at 23rd Street.

2. 38th Street Station. Similarly the rural and urban equations were applied to the less developed upper part of the watershed. Again values of the impervious cover were

increased from the present value of 21 percent to 50 percent. The value of Φ of 0.8 was again assumed constant during the development. Comparison of the measured and rural unit hydrographs indicated that the peak discharge has only been increased 6 percent while the time of rise has been reduced 47 percent due to present conditions of urbanization. Future development would continue this trend resulting in a 54 percent reduction in the time of rise and 66 percent increase in the peak discharge at 50 percent impervious cover (Figure 24). The effects of urbanization on the hydrologic characteristics of the 38th Street watershed are summarized in Table 11.

Stage of Development	Time of Rise (minutes)	Percent Difference Based on Rural Values	Peak Discharge (cfs)	Percent Difference Based on Rural Values
Rural $I = 0\ \%$	103	0	880	0
Present $I = 21\ \%$	55	- 47 %	930	+ 6 %
Future $I = 50\ \%$	47	- 54 %	1,460	+ 66 %

TABLE 11. Summary of Some Effects of Present and Future Urban Development on the Waller Creek Watershed at 38th Street.

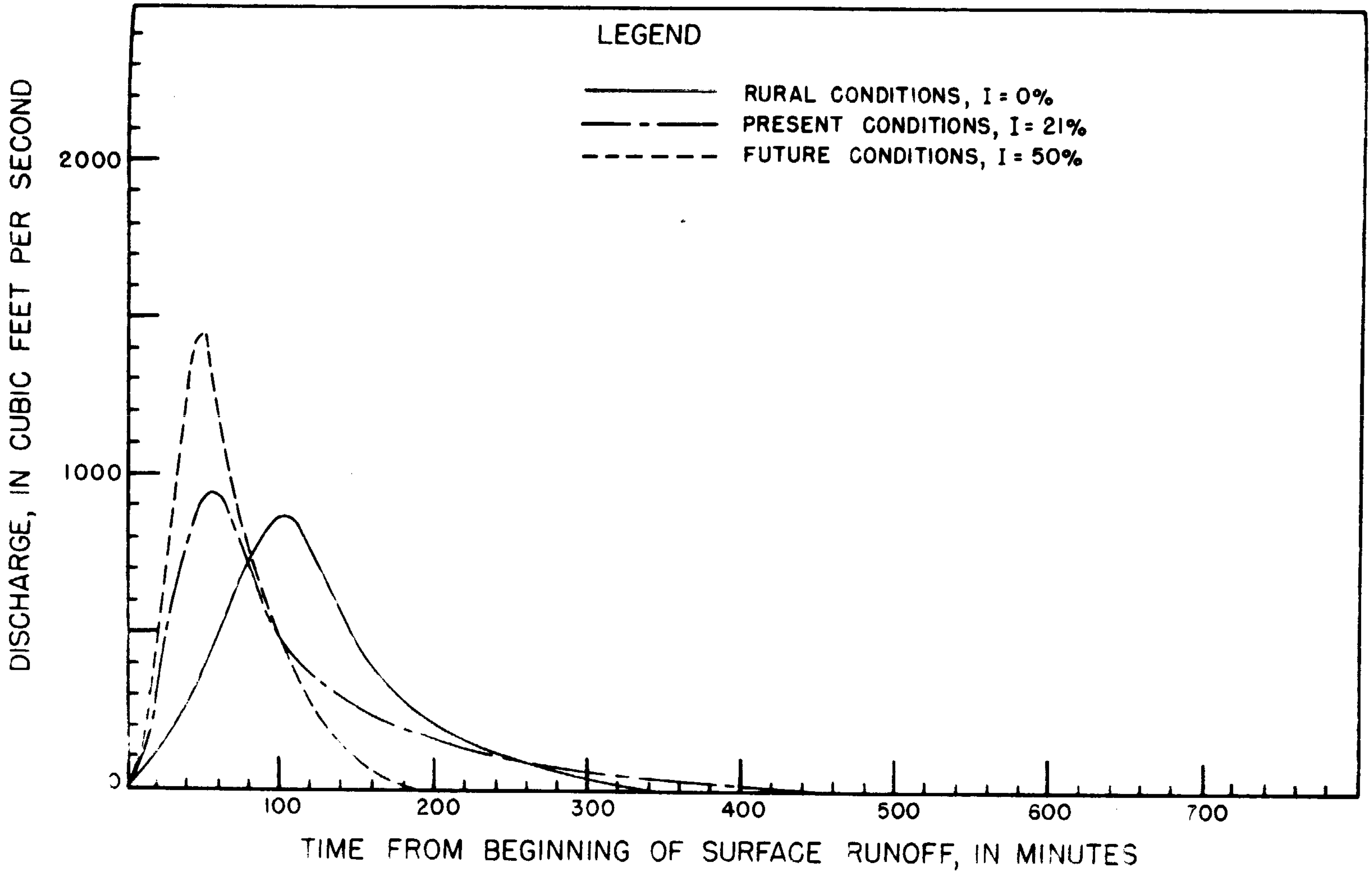


FIGURE 24. EFFECTS OF FUTURE URBAN DEVELOPMENT ON THE UNIT HYDROGRAPH AT THE 38th STREET GAGING STATION.

C. EFFECTS OF URBANIZATION ON LAG TIME

Equation 37 was used to predict the lag times for the Waller Creek watershed under future development characterized by 50 percent impervious cover. Based on equation 37 the lag time will be reduced 35 percent and 70 percent for the 23rd and 38th Street stations respectively for 50 percent impervious cover. Summarized in Table 12 are the predicted values of T_4 for the 23rd and 38th Street stations at 50 percent impervious cover.

Watershed	Equation 37 $T_4 = 40.3\ A^{.21}\ I^{-.51}\ S^{-.26}$ Predicted Values (minutes)
23rd Street 50 %	20
38th Street 50 %	14
Present values: 23rd Street 27 %	44
38th Street 21 %	31

TABLE 12. Predicted Lag Time Values.

Chapter IV

RUNOFF YIELD STUDY

As a watershed becomes urbanized, more of the basin surface is covered by impervious cover, thus reducing the infiltration characteristics of the area and increasing the amount of runoff. Sawyer (1961) reported "that the increased urbanization has altered the characteristics and regimen of many of the streams on Long Island, while those streams not affected by urbanization remained unchanged." The 1952 station analysis stated, "...Nassau County has developed so greatly that it is evident that under these changing conditions a greater part of the precipitation enters the stream than previously. ...The base flow* for this station accordingly has been raised from 75 cfs to 100 cfs." And again in the 1959 station analysis, "...The base flow for peak discharge computed from 22 years of record is 100 cfs, whereas that computed from the past 8 years of record is 170 cfs." A similar study was conducted by Harris and Rantz (1964) of a small watershed of 5.12 square miles located in Santa Clara County, California. They found that a substantial increase in the volume of storm runoff coincided with the period of major urban development. "As a result of the impervious surface in the project area increasing from about 4 percent of the total area in 1945 to 19 percent in 1958, the ratio of outflow from the area (including channel seepage) to inflow increased from 1.18 to 1.70."

In order to evaluate the effects of urbanization on the unit runoff on Waller Creek a detailed unit storm analysis was made. By analyzing only unit storms the unit yield

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* The base flow is generally selected as equal to the lowest annual flood so that at least one flood in each year is included; however, in a long period, the base is generally raised so that on the average only 3 or 4 floods a year are included.

(in/sq. mile) from the area between the 23rd and 38th Street stations, $\triangle A$, could be compared directly to the unit yield from other parts of the study area. A greater unit yield of approximately 100 percent was found for $\triangle A$ as compared with the 38th Street watershed (Figure 25). The relationship $R_{\triangle A} = 2.0 R_{38}$ is indicated in Figure 25 merely for reference. A greater unit yield of approximately 50 percent for 23rd Street was also found when compared to 38th Street (Figure 26). The relationship $R_{23} = 1.5\, R_{38}$ is indicated also in Figure 26 for reference. If the assumption is made that the 38th Street and $\triangle A$ watersheds are geometrically and physiographically similar, the increased runoff can be attributed to the difference of impervious cover between the areas compared. The difference in impervious cover is approximately 16 percent between $\triangle$ A and 38th Street and 8 percent between 23rd and 38th Streets for the period of record (Figure 22, page 71).

In order to evaluate the effects of increasing impervious cover on the runoff yield from the Waller Creek watershed a rainfall runoff relationship was derived with impervious cover as one of the assumed independent variables. Other assumed independent variables used in the analysis included the following: antecedent-precipitation index (API_t), amount of rainfall (WMR), duration of rainfall (D_T). The antecedent-precipitation index is a measure of the soil moisture and is assumed to decrease logarithmically with time during a period of no precipitation. It can be expressed as

$$API_t = API_o \, c^t \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (56)$$

where API_o is the initial value of the antecedent-precipitation index, API_t is the reduced value t days later, and c is a recession factor ranging normally between 0.85 and 0.98. Based on Nation's (1959) and Sauer's (1963) studies of watersheds in Texas,

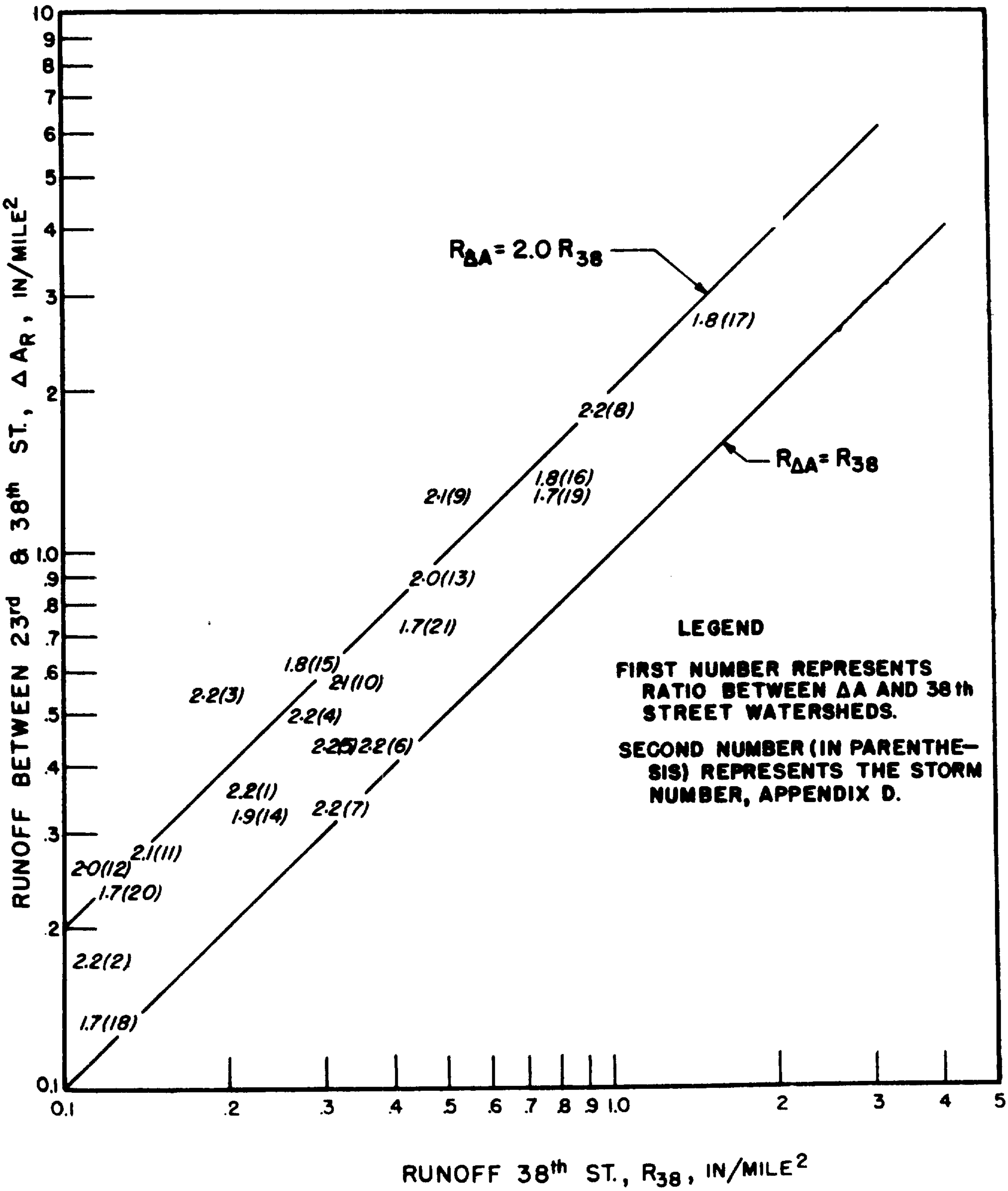

FIGURE 25. RUNOFF VOLUME FROM ΔA COMPARED WITH RUNOFF VOLUME AT 38th STREET STATION.

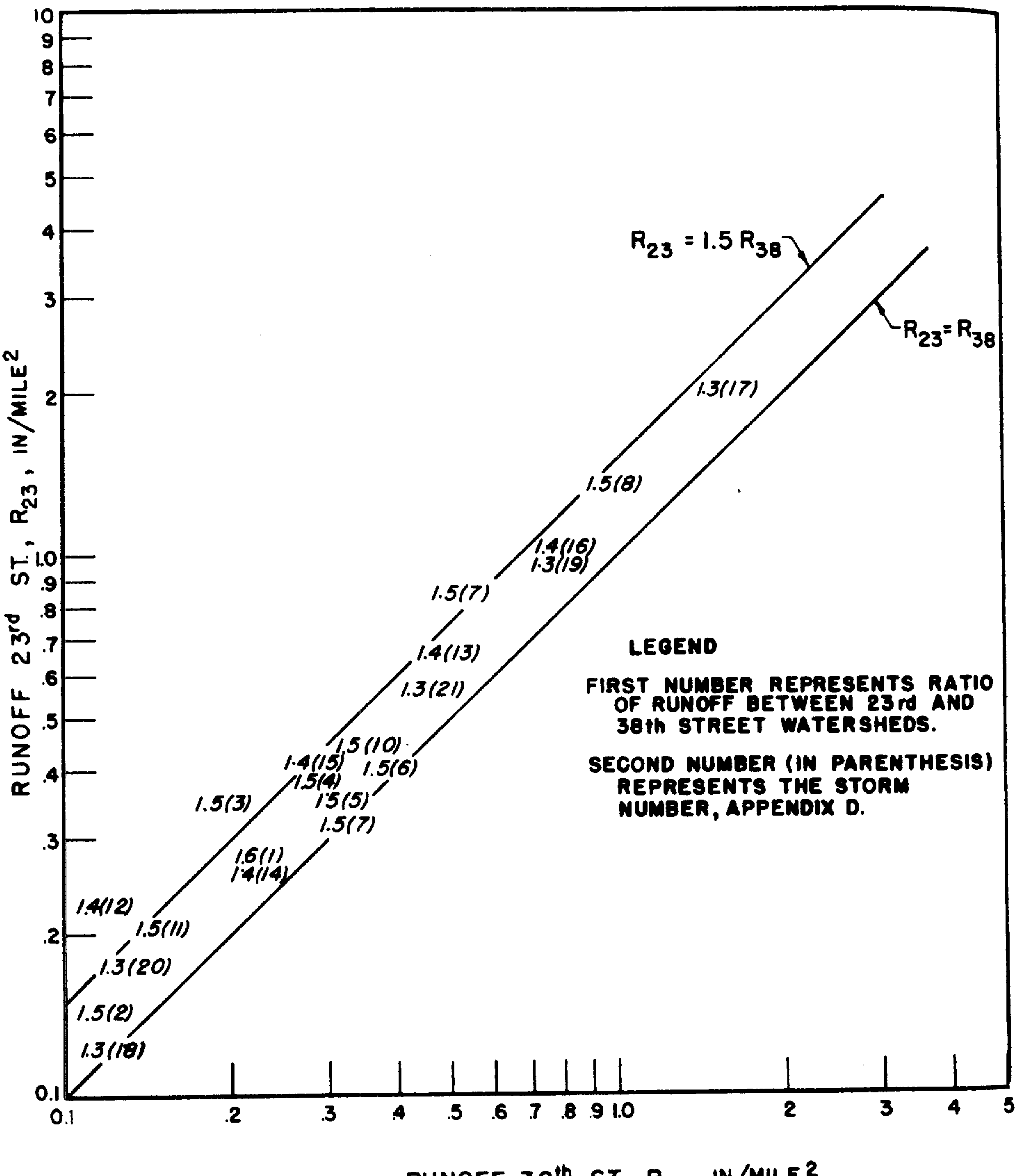

FIGURE 26. COMPARISON OF RUNOFF VOLUME AT 23rd AND 38th STREET STATIONS.

a value for c of 0.80 was assumed for the Waller Creek watershed. The runoff can be stated in equation form as

$$\text{Runoff} = f' \left(API_t,\ WMR,\ D_T,\ S,\ I + 1\right) \quad \ldots\ldots\ldots\ldots\ldots\ldots (57)$$

The impervious cover was introduced in the form I + 1 in order to determine the runoff from a storm under conditions of zero impervious cover. The resulting equation for the twenty-four storms is

$$R = \frac{(I + 1)^{0.339}\,(WMR)^{1.72}\,(API_t)^{0.0741}}{D_T^{\,0.0973}} \quad \ldots\ldots\ldots\ldots\ldots (58)$$

Equation 58 gives a correlation coefficient of 0.983, significant at the one percent level, a standard error of estimate of 0.11, and it predicts two-thirds of the values of the runoff with $\pm$ 21 percent and explains 96 percent of the variance. Equation 58 is based on the following range of fairly uniformly distributed data: (1) WMR (0.80 to 2.74 inches); (2) I (21.6 to 28.7 percent); (3) API_t (0.08 to 2.31 inches); (4) D_T (30 to 600 minutes); and (5) R (0.14 to 1.31 inches per square mile). Therefore based on equation 58 the increase in runoff resulting from an increase in impervious cover can be determined by evaluation of the runoff factor, $(I + 1)^{0.339}$, as compared to that produced by conditions of zero impervious cover. The percent increase in runoff based on rural conditions is presented in Figure 27.

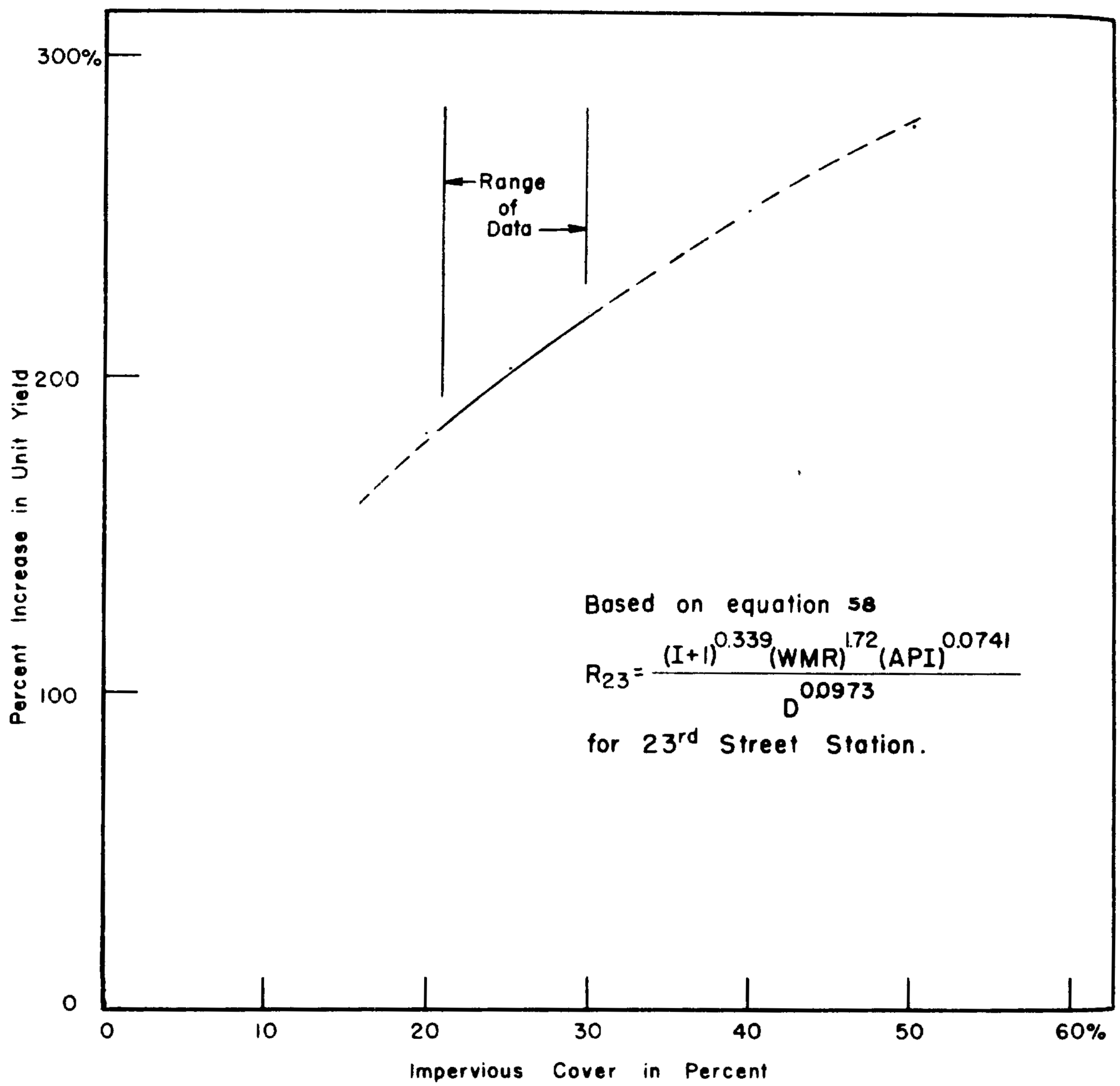

FIGURE 27. PERCENT INCREASE IN UNIT YIELD AS A FUNCTION OF IMPERVIOUS COVER (23rd STREET STATION).

Chapter V

DISCUSSION OF RESULTS

In general the results from this study indicate that urbanization has caused extensive changes to the hydrologic performance of the Waller Creek watershed. These results are based on rural conditions determined by application of the derived rural equations 13, 21, 26, 27 and 28 to the Waller Creek watershed. The changes have resulted in increased unit runoff volume, increased peak discharge and decreased time characteristics of runoff. These three effects have combined to increase the flood potential of the Waller Creek watershed. Analysis based on other more developed urban watersheds indicates that this same trend of increased flood potential will continue as the Waller Creek watershed develops.

A. TIME CHARACTERISTICS OF RUNOFF

The results from this study indicate that urbanization can result in considerable reduction in the time characteristics of runoff. The effect of urbanization on the time characteristics of runoff can best be illustrated by noting the time of rise. Analysis has indicated that the time of rise was a significant runoff characteristic and could be used as one factor to determine the discharge hydrograph. For the two streamflow stations, 23rd and 38th Streets, the reduction in the time of rise under present urban conditions was more pronounced on the 23rd Street watershed. The time of rise was reduced 57 percent for the 23rd Street watershed while only 6 percent for the 38th Street watershed as a result of the present urban development. The larger reduction in the time of rise for the 23rd Street watershed under present urban development is due to the higher concentration of urbanization located in the lower portion of the watershed, Δ A (Figure 22). This more developed urban area, ΔA, with the channel improvement in the West Branch, impervious cover, and extensive storm sewers has resulted in a reduced time of concentration thereby resulting in a rapid runoff condition. Waller Creek as a result has taken on the appearance of a typical flashy mountain stream. Future development will continue the trend of decreasing the time of rise. At stages of urban development represented by 50 percent impervious cover the time of rise will be reduced approximately 53 percent for both the 23rd and 38th street gaging stations.

B. PEAK DISCHARGE

The reduction in the time characteristics of the hydrograph, in particular the time of rise, has resulted in an increase in the peak discharge of the unit hydrograph. In other words, since the unit hydrograph represents one-inch of runoff with a reduction in the time characteristics of the hydrograph the peak discharge must increase in order to yield the same volume of runoff. The peak discharge of the unit hydrograph has increased 51 and 6 percent for the 23rd and 38th Street stations respectively under the present conditions of urbanization. The small increase in peak discharge for the 38th Street watershed can be attributed to the relatively small amount of urban development on the upper portion of the Waller Creek watershed. As urban development continues to increase, resulting in a decrease in the time characteristics of runoff, the peak discharge will continue to increase. At stages of urban development of 50 percent the peak discharge for the 23rd and 38th Street watersheds will have both been increased over rural conditions a total of 330 percent.

C. RUNOFF YIELD

Analysis of unit storm data indicated that the runoff yield from the lower portion, ΔA, of the Waller Creek watershed was greater than the runoff yield from both the 23rd Street and 38th Street watersheds. Runoff yield from ΔA was approximately 100 percent greater than the runoff yield from the 38th Street watershed; whereas the runoff yield from the 23rd Street watershed was only 50 percent greater than the runoff yield from the 38th Street watershed. Development of a rainfall-runoff relationship based on this same unit storm data enabled the extrapolation of the existing storm data to rural conditions. In other words, for a given set of storm characteristics, the runoff yield can be predicted by the rainfall-runoff relationship under both rural and future urban conditions. Based on the rainfall-runoff relationship, equation 58 (Figure 27), runoff has increased 240, 210 and 190 percent for ΔA, the 23rd Street station and the 38th Street station respectively under present urban development. As impervious cover continues to increase, the runoff yield from all stations on the Waller Creek watershed will also continue to increase. These results in general can be expected to apply for most urban watersheds. Exceptions to these results will probably be due to man-made storage facilities constructed in the watersheds for various purposes. For example, in Long Island, New York large surface pits have been constructed into which excess runoff is diverted for the purpose of ground water recharge (Sawyer, 1961).

When extreme storm events are considered, however, the effects of increased runoff caused by impervious cover are probably not as significant. This is due to the fact that during these extreme storm events the soil is saturated and for all practical purposes is absorbing very little runoff; thus the impervious cover and the soil surface are yielding approximately the same amount of runoff.

Chapter VI

CONCLUSIONS

Based on analysis of low frequency storm data from 11 rural and 22 urban watersheds the following conclusions can be drawn:

1. The 30 minute unit hydrograph for rural watersheds can be determined by the equations 13, 21, 26, 27 and 28.

$$T_{RR} = 2.65\, L^{0.12}\, S^{-0.52} \quad ,$$

$$Q_R = 1.70 \times 10^3\, A^{0.88}\, T_R^{-0.30} \quad ,$$

$$T_{BR} = 7.41 \times 10^3\, A^{0.64}\, Q^{-0.53} \quad ,$$

$$W_{50R} = 7.37 \times 10^4\, A^{1.11}\, Q^{-1.13} \quad ,$$

and

$$W_{75R} = 4.46 \times 10^4\, A^{1.06}\, Q^{-1.13} \quad .$$

These equations are based on data from 11 rural watersheds in Texas, Oklahoma and New Mexico and will predict hydrograph characteristics within $\pm$ 20 percent two-thirds of the time.

2. The 30 minute unit hydrograph for urban watersheds can be determined by the equations 17, 22, 29, 30 and 31.

$$T_{RU} = 20.8\, \Phi\, L^{0.29}\, S^{-0.11}\, I^{-0.61} \quad ,$$

$$Q_U = 1.93 \times 10^4\, A^{0.91}\, T_{RU}^{-0.94} \quad ,$$

$$T_{BU} = 4.44 \times 10^5 A^{1.17} Q^{-1.19} \quad ,$$

$$W_{50U} = 4.14 \times 10^4 A^{1.03} Q^{-1.04} \quad ,$$

and

$$W_{75U} = 1.34 \times 10^4 A^{0.92} Q^{-0.94} \quad .$$

These equations based on data from 22 urban watersheds throughout the United States will predict hydrograph characteristics within $\pm$ 30 percent two-thirds of the time.

3. The general equations 41, 46, 47 and 49:

$$Q/A = 4.09 \times 10^4 T_R^{-1.11} \quad ,$$

$$T_B = 3.18 \times 10^4 (Q/A)^{-1.13} \quad ,$$

$$W_{50} = 3.88 \times 10^4 (Q/A)^{-1.025} \quad ,$$

and

$$W_{75} = 1.00 \times 10^4 (Q/A)^{-0.89} \quad ,$$

were derived for any duration unit hydrograph and for any type of watershed. These relationships used in conjunction with either the rural or urban time of rise equation will predict hydrograph characteristics within $\pm$ 35 percent two-thirds of the time.

4. Except for the time of rise, each regression equation evaluated utilized the drainage area. It was found that the drainage area was the dominant basin parameter and that its deletion led to statistically unreliable results.

5. Extensive channel improvement can result in a reduction in the time of rise of 40 percent and an increase in the peak discharge of 60 percent.

With the aid of the derived urban and rural equations the following conclusions can be drawn regarding the effects of urbanization on the hydrologic characteristics of the Waller Creek watershed in Austin, Texas.

1. The hydrograph characteristics, time of rise and hydrograph widths at 0, 50 and 75 percent of the peak discharge have been decreased because of the present urbanization.

2. Present urbanization has also resulted in an increase in the unit yield.

3. The combined effects of decreased time characteristics and increased volume of runoff have resulted in an increase in the peak discharge of the unit hydrograph at the 23rd and 38th Street gaging stations of 260 and 200 percent respectively over rural conditions.

4. Future urban development will continue this same trend. At the stage of urban development of 50 percent impervious cover, the peak discharge for both the 23rd and 38th Street stations will increase a total of 330 percent over rural conditions.

In practice it is generally assumed that a storm of a certain frequency will produce a flood of that same frequency. That is, a 25 year storm is considered to produce a 25 year flood. Assume a given storm falls on an undeveloped watershed and produces minor flooding. As this watershed is urbanized this same storm will give the appearance of producing a flood of considerably greater frequency. The results from this study indicate that urbanization in a watershed will produce floods with peak discharges from 100 to 300 percent greater than on the undeveloped watershed. Therefore urban development, both current and future, is an important factor that cannot be overlooked in any flood frequency analysis of any watershed.

Chapter VII

RECOMMENDATIONS FOR FUTURE RESEARCH

Following is a list of various aspects of this study that warrant future study:

1. After additional data has been obtained on the Waller Creek watershed for various stages of urban development the results of this study should be examined and evaluated in light of this new data.

2. The recently established gaging station on the rural Wilbarger watershed is located approximately 10 miles northeast of the Waller Creek watershed, in a similar geologic and topographic region. The Wilbarger Creek watershed, as soon as sufficient data has been collected, provides an excellent opportunity to further evaluate the derived rural relationships and directly compare the hydrology of both the Wilbarger and Waller Creek watersheds.

3. As additional urban data is collected in the Houston, Texas urban hydrology program, the derived urban equations should be examined and evaluated in light of this new data.

4. The possibility of a general relationship applicable to both urban and rural watersheds should be further investigated.

5. Other statistical procedures such as modern factor analysis and multivariant analysis should also be studied as possible better ways of analyzing the urban and rural data.

6. Forms of solution for the hydrograph characteristics other than the exponential form used in this study should also be investigated.

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APPENDIX A

Correlation Coefficients - Rural and Urban Watersheds

	A	L	L_{ca}	S	Wid.	Q	T_R	W_{50}	W_{75}	T_B
A										
L	.957									
L_{ca}	.966	.981								
S	.840	.897	.910							
Wid.	.953	.935	.963	.926						
Q	.916	.904	.934	.893	.909					
T_R	.940	.895	.937	.887	.964	.881				
W_{50}	.896	.913	.893	.797	.882	.704	.862			
W_{75}	.896	.964	.929	.848	.880	.780	.823	.960		
T_B	.877	.906	.921	.966	.940	.844	.948	.885	.879	
Teq.	.925	.911	.925	.911	.954	.826	.973	.932	.889	.979

Correlation Coefficients - Rural Watersheds

	A	L	L_{ca}	S	Wid.	Imp.	Q	L_T	T_R	W_{50}	W_{75}	T_B
A												
L	.938											
L_{ca}	.927	.991										
S	.274	.367	.394									
Wid.	.863	.836	.825	.299								
Imp.	.274	.354	.327	.569	.390							
Q	.591	.626	.643	.264	.752	.325						
L_T	.940	.927	.943	.357	.747	.236	.531					
T_R	.930	.939	.935	.341	.768	.282	.461	.958				
W_{50}	.850	.851	.797	.200	.703	.314	.399	.815	.895			
W_{75}	.836	.826	.774	.206	.722	.325	.398	.772	.899	.957		
T_B	.926	.892	.861	.282	.852	.304	.435	.855	.904	.908	.872	
Teq.	.942	.910	.889	.304	.866	.298	.483	.896	.925	.904	.858	.992

Correlation Coefficients - Urban Data

APPENDIX B

Urban and Rural Watershed Physiographic Data

Urban Watersheds

No	Watershed	A sq. miles	I percent	S ft./ft. $\times 10^{-3}$	L feet	L_{ca} feet
1	Anacostia - N. W.	21.30	6.3	4.35	91,100	45,000
2	Anacostia - N. E.	72.80	2.7	3.20	85,400	40,000
3	Boneyard	4.45	37.4	1.80	15,000	7,000
4	Brays Bayou, Houston	88.40	40.0	0.77	123,000	54,800
5	Greens Bayou, Houston	67.50	25.0	1.26	114,000	52,800
6	Halls Bayou, Houston	26.20	30.0	1.34	71,300	30,100
7	Sims Bayou, Houston	63.00	30.0	0.64	95,100	51,300
8	White Oak Bayou, Houston	92.00	35.0	0.95	122,000	67,600
9	Red Run	36.50	25	1.73	37,000	15,000
10	Waller Creek at 38th St. at Austin	2.31	27	9.00	23,030	8,750
11	Waller Creek at 23rd St. at Austin	4.13	37	9.00	27,560	10,800
12	Salt Fork - West Branch	71.40	4	10.40	83,600	51,400
13	Louisville - 17th St.	0.22	83	3.80	4,900	1,640
14	Louisville - N. TK.	1.90	50	1.20	16,000	5,960
15	Louisville - W. Out	2.77	70	0.89	22,000	9,860
16	Louisville - S. Out	6.43	48	1.37	31,000	13,330
17	Louisville - S. W. Out	7.52	33	1.47	34,200	14,150
18	Freeman - A	0.0150	21.6	9.00	900	200
19	Freeman - B + A	0.0128	100	5.80	555	240
20	Freeman - B + T	0.0140	100	4.00	1,200	240
21	Lockbourne, 2	0.00089	100	7.50	140	70
22	Lockbourne, 3T	0.0011	100	6.00	350	150
23	St. Anne, No. 1	0.1140	5.1	8.00	2,600	1,100
24	Godman, No. 1	0.0205	22.1	15.00	1,300	350
25	Oxhey, Hertfordshire, England	0.0018	100.0	1.19		
26	Stevenage, Hertfordshire, England	0.0054	43.0	2.22		
27	Kidbrooke, Kent, England	0.0132	68.2	1.32		
28	Stevenage, Newtown, England	0.0184	30.5	1.00		
29	Blackpool, Lancashire, England	0.0186	42.0	3.33		
30	Doncaster, Yorkshire, England	0.0198	29.9	0.83		
31	Stevenage, Newtown, Hertfordshire, England	0.0308	19.8	2.22		
32	Leicester, Leicestershire, England	0.2297	36.1	1.89		
33	Brichouse, Yorkshire, England	0.3125	21.5	5.26		
34	Kensington, London, England	0.3125	94.5	0.56		
35	Oxhey S, Hertfordshire, England	0.9547	19.8	3.13		
36	Harlow Newtown, Essey, England	8.23	1.9	1.43		
37	Hamilton Hills, Baltimore, Maryland	0.0011	55.6	0.83	460	
38	Gray Haven, Maryland	0.0395	52.0	0.91	1,868	

Urban Watersheds

No	Watershed	A	I	S	L	L_{ca}
39	Hamilton Hill 2, Maryland	0.0015	20.1	0.98	505	
40	Hamilton Hill 3, Maryland	0.0029	36.4	0.85	583	
41	Hamilton Hill 4, Maryland	0.0003	96.3	0.86	583	
42	Hamilton Hill 5, Maryland	0.0027	31.8	2.10	360	
43	Midwood 5, Maryland	0.0020	56.0	6.10	166	
44	Montebello 2, Maryland	0.0024	8.7	1.73	470	
45	Montebello 3, Maryland	0.0007	57.1	0.81	153	
46	Montebello 4, Maryland	0.0008	64.8	0.79	352	
47	Montebello 5, Maryland	0.0008	65.9	0.85	352	
48	Newark 9, Maryland	0.0010	100.0	3.35	575	
49	Newark 12, Maryland	0.0015	100.0	0.68	917	
50	Northwood, Maryland	0.0741	68.0	2.87	2264	
51	SPL 1, Baltimore, Maryland	0.0006	100.0	1.71	280	
52	SPL 2, Baltimore, Maryland	0.0007	100.0	2.16	332	
53	Swansea, Baltimore, Maryland	0.0739	44.0	3.06	2500	
54	Uplands, Baltimore, Maryland	0.0470	52	0.0256	2080	
55	Walker Ave, Baltimore, Maryland	0.2397	33	0.0142	5620	
56	Yorkwood, Baltimore, Maryland	0.0162	41	0.0351	1040	
57	Montebello 1, Baltimore, Maryland	0.00033	64.6	0.0176	470	

Rural Watersheds

No	Watershed	A sq. miles	I percent	S ft./ft.$\times 10^{-3}$	L feet	L_{ca} feet
A	Calaveras	7.01	0	7.93	18,500	8,970
B	Deep Creek No. 3	3.42	0	19.0	14,200	4,750
C	Deep Creek No. 8	4.32	0	12.0	25,300	9,000
D	Escondido No. 1	3.29	0	13.3	11,700	7,000
E	Honey Creek No. 11	2.14	0	11.0	9,700	5,500
F	Honey Creek No. 12	1.26	0	15.0	8,100	4,800
G	Cow Bayou No. 4	5.25	0	16.7	21,700	8,970
H	Albuquerque	0.152	0	146.4	3,900	1,730
I	Bentonville	0.34	0	20.0	1,200	600
J	Guthrie	0.148	0	64.0	3,250	1,600
K	Stillwater	0.143	0	33.0	3,500	1,500
L	*Freeman Field D	0.015	0	50.0	1,250	1,100
M	*St. Anne No. 2	0.060	0	50.0	1,900	880

*Only used for lag time relationships.

APPENDIX C

Rural and Urban Watersheds Hydrologic Data

Urban Watersheds

No	Watershed	Peak (30 min) cfs.	T_R min.	T_1 min.	T_2 min.	T_3 min.	T_4 min.	T_5 min.	T_B min.	W_{50} min.	W_{75} min.	T_c min.
1	Anacostia - N. W.	2,800	420			278			900	300	120	360
2	Anacostia - N. E.	3,000	720			720			2,400	900	350	840
3	Boneyard	1,090	90			48			750	136	90	
4	Brays Bayou	4,500	120			420			6,500	750	200	
5	Greens Bayou	1,980	300			480			5,200	1,000	650	
6	Halls Bayou	1,160	270			270			5,430	900	270	
7	Sims Bayou	2,500	330			420			7,000	1,350	600	
8	White Oak Bayou	3,000	450			360			6,660	900	600	
9	Red Run	13,200	90			189			720	59	30	
10	Waller Cr. at 38th St.	930	55	75	15	40	44	49	600	78	46	
11	Waller Cr. at 23rd St.	2,180	57	64	19	27	31	55	542	53	30	
12	Salt Fork-West Branch	2,370	690		660				5,000	35	20	
13	Louisville-17th St.	250	35		9	23			70	33	21	23
14	Louisville-N. TK.	1,520	40		23	36			120	44	28	36
15	Louisville-W. Out	1,240	40		18	50			225	78	41	50
16	Louisville-S. Out	3,280	60		44	66			85	70	41	66
17	Louisville-S. W. Out	4,720	30		30	50'			225	51	29	50
18	Freeman-A	10.60	50	52			38		240	46	28	38
19	Freeman-B + A	16.20	30			6.0	6.5		74	31	24	6.5
20	Freeman-B + T	17.00	30				9.0		108	31	25	9.0
21	Lockbourne - 2	1.14	8				2.5		38	35	32	2.5
22	Lockbourne - 3T	1.40	20				4.0		50	31	27	4.0
23	St. Anne - No. 1	32.90	70		32				650	88	46	70
24	Godman - No. 1	8.10	50		32	45	45		470	71	38	45

Urban Watersheds

No	Watershed	Pk 30 min	T_R	LT_1	LT_2	LT_3	LT_4	LT_5	T_B	W_{50}	W_{75}	Teq. meas.	Teq. theok	Tc (min)
25	Oxhey, Hertfordshire													
26	Stevenage, Hertfordshire						(5.8)							5.8
27	Kidbrooke, Kent						(4.6)							4.6
28	Stevenage, Newtown						(9.0)							9.0
29	Blackpool, Lancashire						(8.3)							8.3
30	Doncaster, Yorkshire						(6.5)							6.5
31	Stevenage, Newtown, 5						(7.3)							7.3
32	Leicester, Leicestesshire						(7.0)							7.0
33	Brighouse, Yorkshire						(17.6)							17.6
34	Kensington, London						(15.0)							15.0
35	Oxhey 5, Hertfordshire						(26.5)							26.5
36	Harlow Newtown, Essex						(20.0)							20.0
37	Hamilton Hills 1						(300)							300.0
38	Gray Haven					6.0	5.9							5.9
39	Hamilton Hills 2					8.0	8.5							8.5
40	Hamilton Hills 3					6.9	8.7							8.7
41	Hamilton Hills 4					8.1	7.4							7.4
42	Hamilton Hills 5					3.8	4.9							4.9
43	Midwood 5					5.9	4.8							4.8
44	Montebello 2					4.0	3.1							3.1
45	Montebello 3						8.0							8.0
46	Montebello 4						4.0							4.0
47	Montebello 5						3.3							3.3
48	Neward 9						3.7							3.7
49	Newark 12					4.8	3.4							3.4
50	Northwood					5.7	5.0							5.0
51	SPL 1					8.6	6.5							6.5
52	SPL 2					5.8	4.7							4.7
53	Swansea					6.0	6.9							6.9
54	Uplands					5.0	4.8							4.8
55	Walker Ave					6.7	7.4							7.4
56	Yorkwood						11.5							11.5
57	Montebello 1						5.9							5.9

Rural Watersheds

No	Watershed	Peak (30 min) cfs	T_R min.	T_1 min.	T_2 min.	T_3 min.	T_4 min.	T_5 min.	T_B min.	W_{50} min.	W_{75} min.	Tc min.
A	Calaveras	1,600	150						550	150	65	
B	Deep Creek No. 3	1,100	90						500	112	63	
C	Deep Creek No. 8	940	60						340	170	123	
D	Escondido No. 1	1,680	60						270	68	43	
E	Honey Creek No. 11	1,080	100						380	52	27	
F	Honey Creek No. 12	814	75						330	47	31	
G	Cow Bayou No. 4	3,000	90						285	53	30	
H	Albuquerque	168	30						100	31	20	
I	Bentonville	234	30						165	40	29	
J	Guthrie	91	45						230	77	50	
K	Stillwater	96	50						190	45	27	
L	Freeman Field D	3.70	90		58		68		480	135	78	
M	St. Anne No. 2	19.30	90		72		66		740	70	37	

APPENDIX D

Storm Data - Waller Creek

No	Storm Date	Cal. yr	Storm Duration (min)	WMR 23rd	WMR 38th	Run-off 23rd	Run-off 38th	Run-off ΔA	API	Peak Discharge 23rd	Peak Discharge 38th	Impervious Cover 23rd	Impervious Cover 38th	Impervious Cover ΔA	Imp. R. ΔA/38	Imp. R. 23/38	23 Pk/A23	38 Pk/A38	ΔPk/ΔA
1	May 17-18	1955				0.27	0.21	0.35		1720	186	21.6	14.0	31.0	2.21	1.5	416	80.5	842
2	May 18-19	1955				0.14	0.11	0.17		762	147	21.6	14.0	31.0	2.21	1.5	184	63.6	337
3	March 20-21	1957	30	1.46	1.41	0.34	0.18	0.54	0.39	1200	111	23.3	15.2	33.1	2.18	1.5	290	45.0	596
4	April 22	1957	70	1.60	1.56	0.37	0.27	0.49	0.68	625	163	23.4	15.3	33.3	2.17	1.5	151	70.5	253
5	April 24	1957	135	1.24	1.16	0.36	0.30	0.43	2.37	645	223	23.4	15.3	33.3	2.17	1.5	156	96.5	231
6	April 28-29	1957	60	1.22	1.23	0.39	0.36	0.43	3.30	810	251	23.4	15.3	33.3	2.17	1.5	196	108.5	307
7	May 18	1957	540	1.72	1.88	0.31	0.30	0.33	0.30	176	96	23.5	15.4	33.4	2.17	1.5	42.6	41.5	43.9
8	June 12	1957	220	2.74	2.60	1.31	0.91	1.81	0.36	2050	500	23.5	15.4	33.5	2.17	1.5	496	216	850
9	April 26	1958	65	2.24	2.36	0.83	0.48	1.26	0.38	2055	320	24.3	16.1	34.3	2.13	1.5	497	139	1000
10	July 6-7	1958	55	1.97	2.08	0.43	0.32	0.57	0.08	1340	500	24.5	16.4	34.7	2.11	1.5	324	217	460
11	Sept. 30	1958	45	1.27	1.27	0.20	0.14	0.27	0.44	515	117	24.6	16.7	34.9	2.09	1.5	125	50.6	219
12	May 22-23	1959	40	1.22	1.33	0.22	0.11	0.26	0.03	555	132	25.3	17.5	35.2	2.01	1.4	134	57.2	232
13	Sept. 23	1959	60	2.33	2.33	0.64	0.45	0.88	0.22	1910	468	25.8	18.0	35.6	1.97	1.4	462	203	792
14	Feb. 3	1960	165	1.22	1.20	0.26	0.21	0.32	0.10	239	87	26.3	18.9	35.9	1.92	1.4	57.8	37.6	83.4
15	Oct. 16	1960	150	1.99	1.86	0.41	0.26	0.60	0.34	1020	230	27.4	19.9	36.3	1.82	1.4	247	100	434
16	Feb. 16	1961	300	2.56	2.66	1.00	0.74	1.33	0.39	872	355	27.7	20.3	36.6	1.80	1.4	216	155	229
17	July 9-10	1961	600	4.70	4.68	1.96	1.43	2.64	0.08	2170	690	28.0	20.8	36.9	1.77	1.3	525	299	869
18	June 1	1962	45	0.80	0.80	0.12	0.11	0.13		286	114	28.6	21.2	37.1	1.75	1.3	69.2	49.4	94.4
19	June 3-4	1962	55	2.35	2.40	0.95	0.72	1.25		2270	795	28.6	21.2	37.1	1.75	1.3	550	344	810
20	Sept. 6	1962		0.88	0.85	0.17	0.12	0.23		655	174	28.7	21.2	37.4	1.77	1.3	158	75.5	265
21	Sept. 7-8	1962		2.05	2.10	0.55	0.42	0.71		1010	361	28.7	21.2	37.4	1.77	1.3	244	156	355