EFFECT OF URBAN DEVELOPMENT
ON FLOODS IN THE PIEDMONT PROVINCE OF NORTH CAROLINA

By

Arthur L. Putnam

Engineer, U.S. Geological Survey

First Printing 1972

Second Printing 1973

U.S. Geological Survey open-file report

Prepared in cooperation with the cities of Charlotte,

Durham, Lenoir, Morganton, and Winston-Salem, North Carolina

Raleigh, North Carolina

1972

72-304

CONTENTS

ILLUSTRATIONS

Page

TABLES

EXPLANATION OF SYMBOLS

A Drainage area, in square miles.

d A subscript indicating a developed urban condition within a basin.

I The ratio of the area of man-made impervious cover to the total drainage area, used as a decimal fraction and/or percentage.

L The length of the main watercourse, in miles.

P_2 The flood-peak discharge, in cubic feet per second, for the recurrence interval, in years, specified by the subscript.

r A subscript indicating a rural or natural condition within a basin.

S The slope, in feet per mile, of the main watercourse between points 10 and 85 percent of the distance upstream from the stream-gaging site.

T The basin lag time, defined as the average time, in hours, between the centroid of the time distribution of rainfall excess and the centroid of the time distribution of direct runoff.

EFFECT OF URBAN DEVELOPMENT ON FLOODS IN THE PIEDMONT PROVINCE OF NORTH CAROLINA

By Arthur L. Putnam

ABSTRACT

This report relates peak discharges for recurrence intervals ranging up to 100 years to drainage area, stream length, stream slope, and percent of basin covered by impervious surfaces. The relations are based on analysis of flood information for approximately 200 sites, 42 of which are in metropolitan areas of the North Carolina Piedmont province. The estimating relations are limited to providing flood discharge estimates at open-channel sites in the Piedmont province of North Carolina where runoff is unaffected by artificial storage or diversion. The estimates are most reliable for smaller size floods at sites where the drainage area ranges between 0.3 and 150 square miles, where the $L/\sqrt{S}$ ratio ranges between 0.1 and 9.0, and where impervious cover of less than 30 percent is uniformly distributed over the basin.

Changes from rural to urban conditions significantly affect flood flows. Urban development may reduce the basin lag time to one-sixteenth that of a comparable natural system. This reduction in basin lag time, along with the increased storm runoff resulting from impervious cover, increases the flood-peak discharge by a factor that ranges up to five. The increase in flood-peak discharge depends on the drainage-basin characteristics and the recurrence interval of the flood.

INTRODUCTION

Purpose and Scope

An increase in the population of an area is accompanied by development and construction that change the runoff characteristics of the area. A significant change is that caused by buildings, paved streets, parking lots, and other impervious areas which reduce the infiltration of rainfall, thus resulting in a greater volume of water available for runoff. The lag time between the rainfall and the flood peak also decreases, mainly because of the hydraulically more efficient channels--such as ditches, gutters, storm sewers, and improved stream channels--through which the storm runoff can flow. The combined effect of these two changes is to increase peak discharges.

Designs of drainage facilities or land-use planning that do not account for this increased peak discharge are inadequate, and may result in heavy damage and loss of property. In view of the expenditures involved in providing drainage facilities and in planning for optimum land use, it is imperative to consider not only the estimated runoff from the watershed in its present state, but also to consider the increase in runoff because of urbanization. A comprehensive understanding of the effect of urbanization on the runoff process is presently lacking. In many instances, the procedures that are presently used for hydrologic design of urban drainage structures have not changed since the turn of the century (Ardis and others, 1969). An understanding of the effects of urbanization on the runoff is obviously essential not only to develop better methods of analysis and procedures to be used in design and planning, but also to apply the methods and procedures in realistic design and planning of urban drainage systems.

The goal of this report is to describe the procedures used and the results obtained in an analysis of the effects of urbanization on flooding in the Piedmont province of North Carolina. The discussion and presentation of the results as mathematical and graphical relations will provide the information needed to design drainage systems and facilitate the optimum land-use planning. It is hoped that the report will assist in the understanding of the effect of urbanization on storm runoff and, perhaps, be a guide to future studies that will provide easily used procedures with a wider areal applicability.

This report covers the work performed under cooperative agreements with five cities in the Piedmont province of North Carolina. This study is the second program on the effects of urban development on floods in North Carolina. The earlier study was carried-out under a cooperative agreement with the city of Charlotte and the results of that study are applicable only to the metropolitan area of the city. Martens (1968) summarized the work done in the original study in Charlotte.

The present investigation extended the study program to include the entire Piedmont. Also data collection and analysis were extended to cover a range in drainage area from 0.27 to 178 square miles with a more accurate means of recording simultaneous data of rainfall and streamflow. The major advantages of the results of this study over the previous investigation are: (1) a more objective means of estimating flooding in basins where conditions are intermediate between completely rural and completely urban, (2) a broader geographical area of applicability, and (3) a foundation in a greater volume of data, which were collected by the most accurate means presently available.

The empirical equations presented in this report provide simple techniques of obtaining design discharges based on the most recent hydrologic data and analytical concepts. Although these equations may be modified later on, we feel they are more reliable than the empirical runoff equations which were developed 4 or 5 decades ago, and which many drainage engineers use without question. Also, the equations presented in this report provide the most accurate estimates of the change in floods resulting from urban development in the Piedmont region of North Carolina and should be used in lieu of the results described by Martens (1968). It is not suggested that the individual equations are applicable in areas other than the Piedmont section of North Carolina.

This report first offers as background information a general description of the study area, followed by an explanation of the available data and the approach used for the collection of other pertinent data. Next, the procedures used in the analysis are presented in two parts. First, the analysis and results for estimating basin lag time are presented, mainly to point out that the effect of urban development significantly changes the time distribution of storm runoff. Second, the analysis and results for estimating peak discharge are described utilizing the change in time between the distribution of rainfall and the distribution of the resulting storm runoff. Following the section of data analysis is a discussion which describes quantitative evaluations of the effects of urban development on floods in the study area. The last section contains a brief illustration of the use of the results presented in this report.

Administration and Acknowledgment

In the late 1950's and early 1960's, urban development began to encroach heavily upon the flood plains of streams in Charlotte, North Carolina. Complaints concerning flooded residences, businesses, and streets were becoming annual events. More frequent flooding was causing problems previously expected only on rare occasions. One method which was considered by the city officials for bringing this problem under control was to enact zoning regulations that restrict building below the 50-year flood elevation. During the process of enacting zoning regulations it was found that adequate data were not available to define changes in runoff characteristics caused by suburban and urban development. Hence, the city officials had no basis for enforcing the zoning restrictions or for designing adequate drainage structures in new developments.

The officials of the city of Charlotte became actively interested in the effects of urban development on floods and entered into a cooperative agreement with the U.S. Geological Survey in January 1962. This initial study was directed toward watersheds that drain areas exceeding five square miles (Martens, 1968). As a result of extensive flooding in urban areas, officials of the cities of Charlotte, Durham, Lenoir, Morganton, and Winston-Salem became concerned about floods on smaller watersheds. The study at Charlotte was expanded to include smaller watersheds, and studies at Durham, Lenoir, Morganton, and Winston-Salem were started under cooperative agreements with those cities in 1966. This report contains the results of the studies conducted in these cities and presents them in a form applicable to all urban areas in the Piedmont province of North Carolina.

This program of study was initiated under the general direction of E. B. Rice, district chief, retired, and L. A. Martens, hydraulic engineer, both of the U.S. Geological Survey's Water Resources Division.

The interest and cooperation of W. E. McIntyre, R. C. Birmingham and Clark Readling of Charlotte; W. G. Brown and H. W. Pickett, Jr., of Durham; H. L. Price of Lenoir; C. L. Brooks and J. B. Wiles of Morganton; and J. H. Berrier of Winston-Salem are gratefully Acknowledged.

This report was prepared under the general direction of Ralph C. Heath, district chief, Water Resources Division, Raleigh, North Carolina.

Description of the Area

Population.--Population and the resultant construction of buildings and other facilities are on the upswing in North Carolina. During the 1940 census, 27.3 percent of the State's total population was classified as urban, 33.7 percent in 1950, 39.5 percent in 1960, and 45.0 percent in 1970. In 1970, the Piedmont had less than 60 percent of the State's total population, but contained about 70 percent of the State's urban population. The most pronounced changes in urbanization have occurred since 1940, particularly in the thirteen counties comprising the Piedmont Crescent (pl. 1) that extends from Wake County westward through Guilford and Forsyth Counties, southward to Mecklenburg and Gaston Counties. In the Piedmont Crescent more than 45 percent of the State's total urban population is concentrated in 15 urban centers. Of these 15 centers, the smallest, Thomasville, contained 15,230 persons and the largest, Charlotte, contained 279,512 persons during the 1970 census. Projections of population growth curves indicate that the greatest period of growth probably lies ahead.

Topography and Drainage.--The Piedmont Province, which lies between the Coastal Plain and the Appalachian Mountains, includes about two-fifths of the land area of the State. The eastern boundary has an elevation of approximately 400 to 600 feet above mean sea level. The surface rises more or less irregularly at the rate of four or five feet per mile to an elevation of about 1,500 to 2,000 feet above mean sea level along the western boundary of the Piedmont at the foot of the Blue Ridge escarpment, which marks the eastern front of the Appalachians.

The topography consists of rounded hills and long, rounded ridges with a northeast-southwest trend. The bedrock formations underlying the area also have a northeast-southwest trend.

The major streams flow generally from the northwest to the southeast. Streams within the Piedmont urban areas are relatively small with well entrenched channels and sandy bottoms, and stream channels in a few places are cut into bedrock. Streams are fairly steep with main channel slopes of more than 15 feet per mile and small tributary slopes of over 100 feet per mile. In the natural state, most flood plains are covered with a dense growth of brush; however, extensive developments have taken place and are continuing in the watersheds and on the flood plains of these urban streams. As a result, problems of drainage and flooding are increasing.

Because the annual precipitation of about 45 inches is more or less uniformly distributed throughout the year, problems associated with drainage and flooding in the area can occur during any season. The major problems are caused by three types of storms. Summer thunderstorms having short-duration but high-intensity precipitation are the most frequent cause. Hurricane storms in late summer and fall may be the cause of very severe problems, and the longer-duration but lower-intensity precipitation resulting from frontal storms may occur during any season and occasionally cause flood problems.

Charlotte.--The city is located in Mecklenburg County in the south-central part of the State. The area is underlain by crystalline rocks predominately of igneous origin. The relief is low and the topography is characterized by low rounded hills and broad valleys. The metropolitan area is drained by five relatively small streams--Irwin-Sugar, Little Sugar, Briar, McMullen, and McAlpine Creeks. For the most part, flooding is caused by these five streams and their tributaries, which head in the northern and eastern part of the metropolitan area and flow southward through the city. Extensive developments have taken place and are continuing in these watersheds and in flood plains along the streams, resulting in increased problems of drainage and flooding.

Maps covering the entire city have been available since 1957 when the city was mapped to a scale of 1 inch equals 200 feet, with a contour interval of 2 feet. Details, such as buildings, parking areas, roads, fence lines, wooded areas, streams, and numerous vertical control points, are shown. The city is mapping additional areas as annexation takes place, as well as potential growth areas within the county. The maps facilitated the determination of basin parameters that were essential to this study.

Durham.--The city is located in Durham County in the northeastern part of the Piedmont, on the divide between the Neuse River basin to the northeast and the Cape Fear River to the southwest. The city and its environs are drained by five streams of which Ellerbe, Goose, and Warren Creeks are in the Neuse River basin. The south side of the city is drained by Sandy Creek and Third Fork Creek in the Cape Fear River basin. The total drainage area of the five streams is approximately 60 square miles.

The North Carolina Railroad was located on the ridge that separates the watersheds of the Neuse River and the Cape Fear River in the vicinity of Durham. Because the city first developed along the railroad, most of the residential properties and business section of Durham are on high ground above flood danger. As the city grew, the high ground near the railroad was developed and movement toward the flood plains started. Important utilities and commercial, industrial, and residential construction is already extensive in the upper reaches of the streams and is spreading downstream.

Lenoir.--The city is located in Caldwell County in the western part of the Piedmont in the foothills just east of the Blue Ridge Mountains. All of the streams in Lenoir flow in a generally southwesterly direction. Lower Creek, joined by Zacks Fork Creek, flows through residential and commercial areas of southeastern Lenoir. Much valuable property is presently located in the Lower Creek flood plain including homes, shopping centers, manufacturing plants, businesses, and other buildings. Blair Fork, joined by Long Branch north of the city limits, flows through several residential areas on the west side of the city and joins lower Creek near the southwestern city limits. The terrain within the watersheds varies from steep mountain slopes in the headwater regions to wide flat flood plains in the creek valleys downstream. Elevations in the area vary from less than 1,000 feet to more than 2,300 feet above mean sea level.

Morganton.--The city is located in Burke County in the western part of the Piedmont in the foothills of the Blue Ridge Mountains. The area is drained by the Catawba River and its tributaries including Silver Creek and Hunting Creek. The stream channels are well defined with generally wide and open flood plains, while the adjacent terrain is moderately steep to gently rolling. At present there is little residential development, but some commercial development in the flood plains. With the present rate of population growth, there will be pressures for residential as well as commercial development in the flood plains.

Winston-Salem.--The city is located in Forsyth County in the northwestern part of the Piedmont. The metropolitan area is drained to the Yadkin River by four relatively small streams--Fiddler, Salem, Silas and Muddy Creeks. The topography is rolling to steep, with sharp breaks in topography occurring along the edge of the flood plains. Generally, the flood plains are broad along the main stem of such streams as Salem and Silas Creeks. Tributary streams are numerous and have steep gradients and relatively narrow flood plains.

Encroachment into the flood plain of streams in the Winston-Salem Metropolitan Area is a practice that is gaining momentum. These encroachments include flood-plain filling with materials from grading operations for new commercial and industrial building sites, highways, and residential development adjacent to the flood plain.

DATA COLLECTION

Studies of the effect of urbanization on floods require the collection of several different kinds of data. These data include records of streamflow and rainfall, and the determination of various basin characteristics--such as size, shape, slope and impervious cover--that may influence the magnitude or timing of floods.

In planning the collection of these data, we decided that the most efficient and advantageous approach was to measure these variables in basins of various sizes and degrees of development. The alternate to this approach would have been to monitor the changes in streamflow in a few basins as they are changed from rural to urban. Such an approach would require many years to collect the necessary records, and the records obtained during any period might not be of sufficient length to define the effects of any particular level of urbanization. On the other hand, sampling many basins in various stages of development would allow us to evaluate the effects of different levels of urbanization without having to wait for developers to completely urbanize the basins in which we were collecting data.

The Geological Survey has developed a method of concurrently measuring rainfall and streamflow by use of two digital recorders that simultaneously punch data on paper tapes. One of these instruments can be used to record the time distribution of rainfall, while the other records the time distribution of the change in the elevation of the water surface in the stream. A single electric timer actuates both recorders, providing an accurate synchronization between the rainfall and water surface elevation. Discharge measurements are made to determine rating curves needed to convert the recorded elevation of the water surface to a continuous record of streamflow.

We decided to collect the dual-digital data because we felt that a rainfall-runoff model would give us the most accurate information on flood frequency from short-term records. In addition to providing data for the rainfall-runoff model, the dual-digital stations would provide the most accurate data to examine the time distribution of the streamflow hydrograph with respect to the time distribution of rainfall. The time of response of streamflow to rainfall or basin lag time, which will be discussed more fully later, is an important indicator of the effects of urbanization on flooding. Some earlier investigators have successfully used the average basin lag time as a variable for estimating the peak rate of storm runoff.

In planning the dual-digital network for data collection, we decided to use either a 5- or 15-minute recording interval depending on the expected shape and duration of the flood hydrograph. These time intervals would be sufficient for both the rainfall-runoff model and the basin lag time analyses. We used the following criteria in selecting basins suitable for the data collection program:

1. The drainage area should be relatively small for two reasons: first, virtually no streamflow data were available for small watersheds in the Piedmont urban areas and, second, the rainfall-runoff models would provide more reliable results when the point rainfall data could be considered uniformly distributed over the watershed.

2. Building activities in the basins should be minimal to insure that the data would represent stable conditions.

3. Development should be evenly distributed throughout the basin to eliminate the difficulties of trying to evaluate the effects of nonuniform urbanization on flooding.

4. The impervious cover, both in a particular basin and among the individual basins, should be representative of as many types of development as possible to reflect an average condition and not that of an extreme associated with a particular type of development.

5. Natural characteristics of the watershed should be as varied as possible from each other to insure that the data collection program would sample as broad a range of natural conditions as possible.

Streamflow

Prior to the initiation of the urban studies, the Geological Survey had collected streamflow data for Little Sugar Creek at Tyvola Road near the southern boundary of Charlotte. At this point, Little Sugar Creek carries flow from the Charlotte urban area and some of the surrounding rural area. The records for this gaging station began in 1924 and continued through the present.

During the first study of the effects of urban conditions on flooding, Martens (1968, p. 4-5) installed four additional continuous-record gaging stations and seven partial-record, crest-stage gages. These stations, established in 1962, gaged flow from drainage areas ranging from about 5 to 50 square miles, and the range in impervious cover was from less than 1 to about 22 percent.

At the start of this investigation, we established seven additional gaging stations in Charlotte, which were set to measure only the flood runoff part of the flow hydrograph. These stations extended the data collection from drainage areas of 5 square miles down to 0.27 square miles. To extend the study to other cities in the Piedmont, we also established 28 gaging stations at sites in Durham, Lenoir, Morganton, and Winston-Salem. Of these 28 gaging stations, 9 were equipped to record the entire flow hydrograph, 15 were set to record only the flood runoff part of the hydrograph, and 4 were equipped to record only the water-surface elevation of the flood crest.

These stations permitted us to sample a range of urban conditions in the Piedmont of North Carolina. Tables 1-5 list the stations that we used to determine the effects of urbanization on floods in North Carolina and give pertinent information including drainage area, land use, percent of impervious cover, period of record, and type of record. Figures 1-5 are maps of the cities showing the location of these stations.

At the most important data collection sites, the gaging stations equipped with instruments that recorded stream level and rainfall with respect to time, provided a complete record during storm periods. The crest-stage stations provided a record of the peak discharge that occurred during floods at other key locations. This data collection network provided, for this study, the most complete information that any investigator has yet obtained in North Carolina for a study of the effects of urban development on floods.

Rainfall

As another requirement for extending short-term streamflow records to a longer period of time, we needed a long-term record of the time distribution of precipitation in the Piedmont of North Carolina. Fortunately such a record is available. The National Weather Service established a rainfall station in Charlotte in 1878. Since 1901, they have obtained a continuous record of precipitation in the Charlotte area. This long-term record was of great importance in determining the expected frequency of occurrence of flooding for various basins at different stages of development.

In order that the areal variability in storm rainfall distribution could be determined more accurately and rainfall-runoff comparisons made, rain gages were installed as part of the data collection network. A total of 46 recording rain gages were included as a part of the investigation: 16 at Charlotte, 7 at Durham, 2 at Lenoir, 3 at Morganton, and 18 at Winston-Salem. Most of these rain gages were installed at stream-gaging stations as previously described.

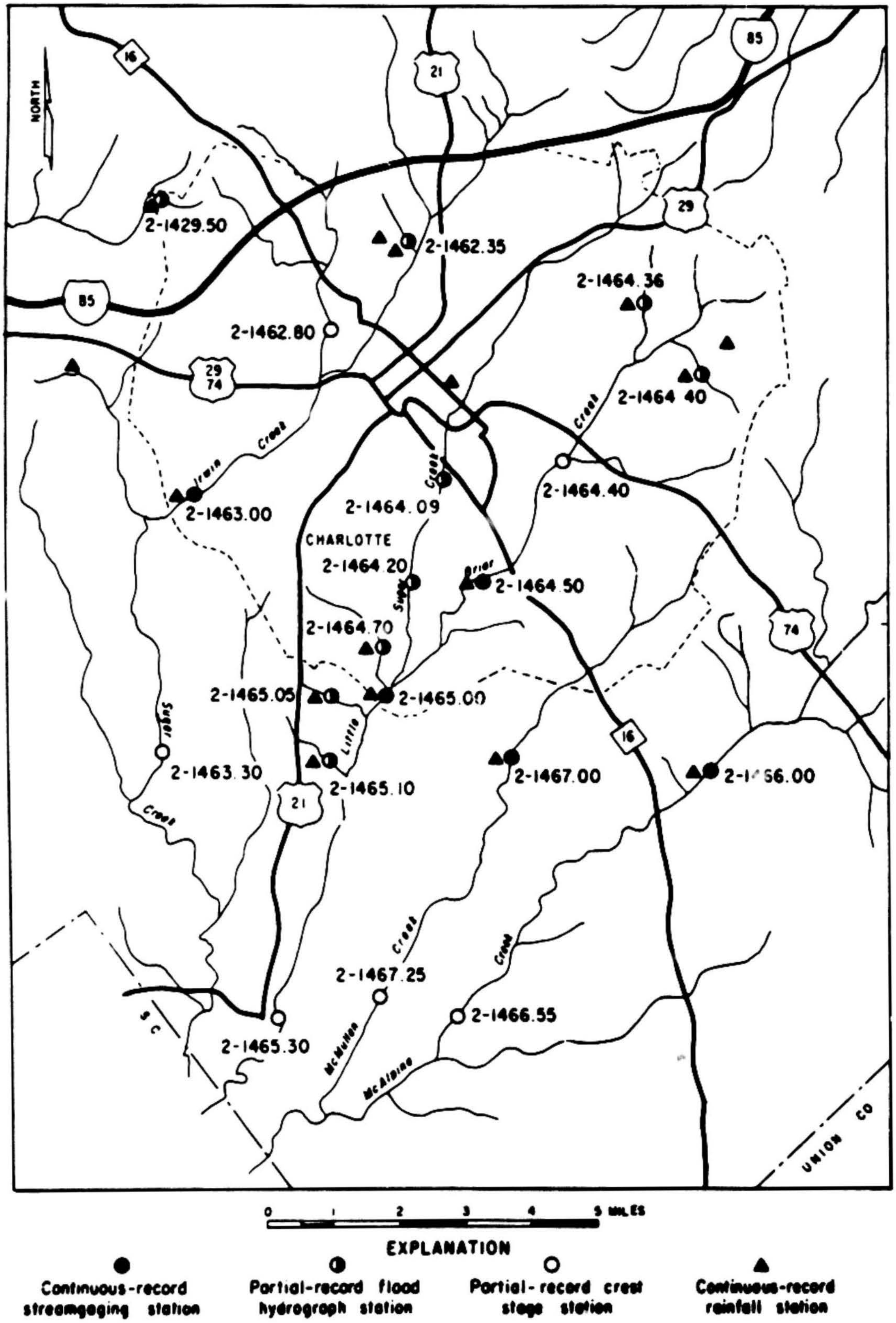


Figure 1.--Map showing data-collection points in the Charlotte area.

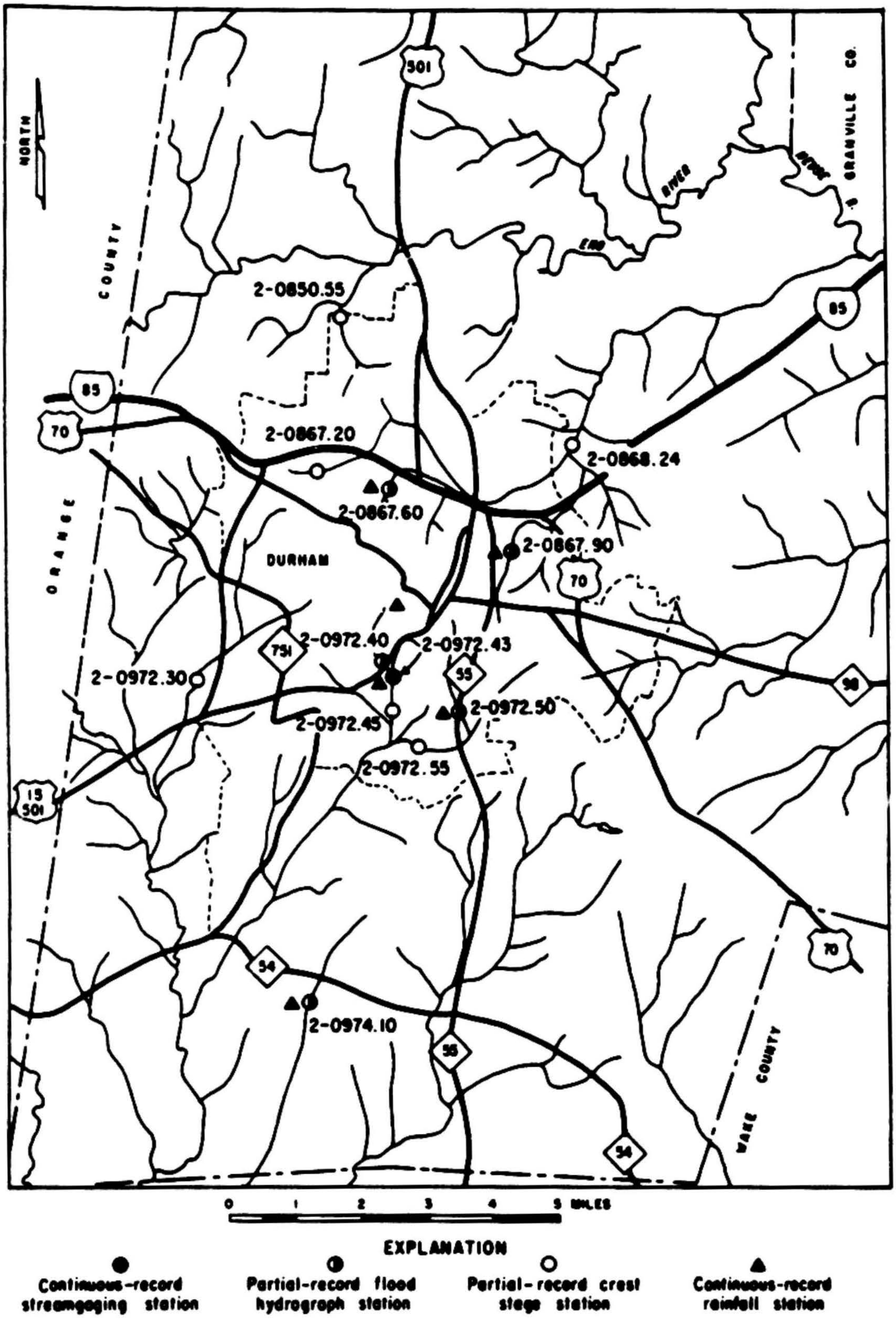


Figure 2.--Map showing data-collection points in the Durham area.

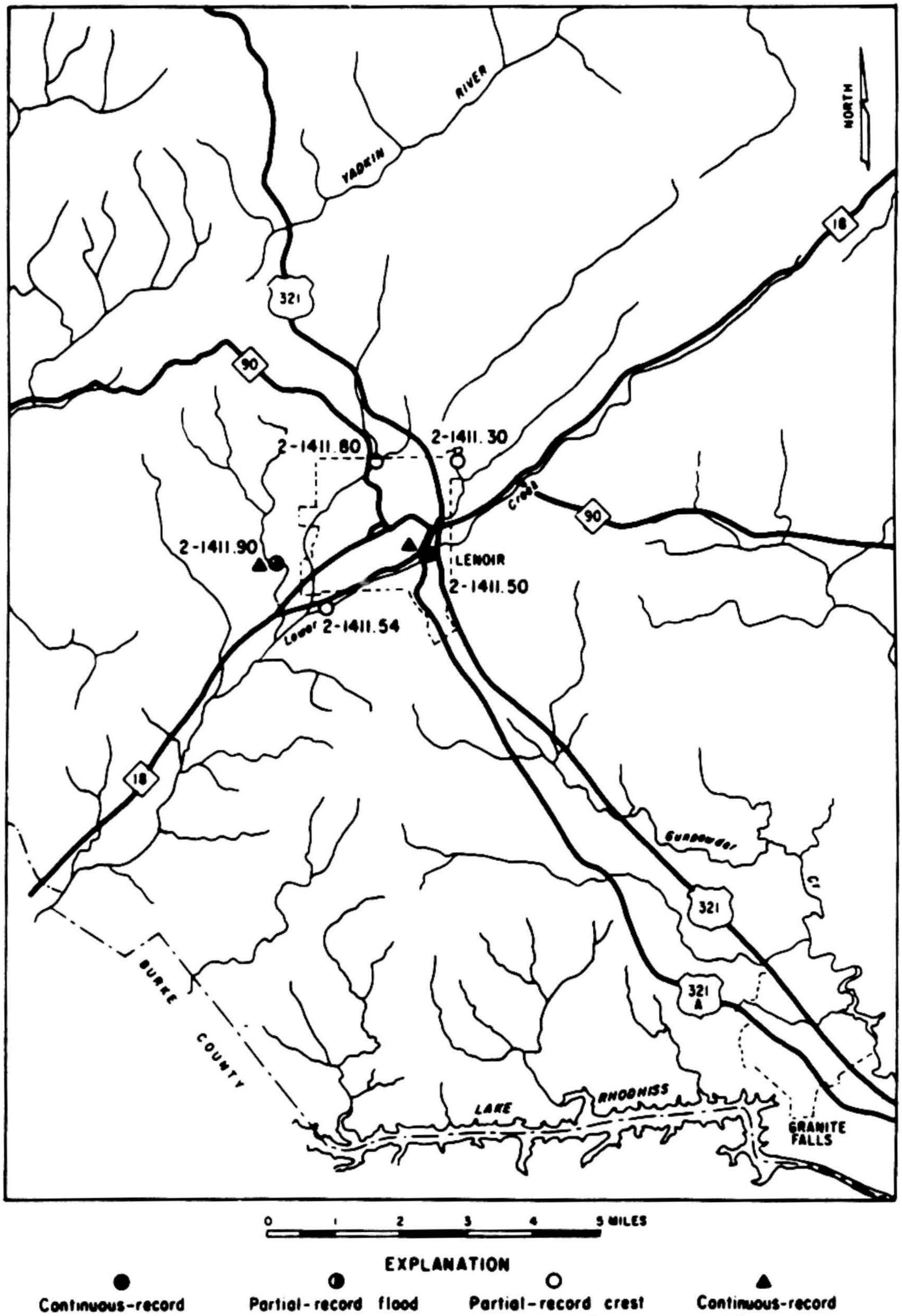


Figure 3.--Map showing data-collection points in the Lenoir area.

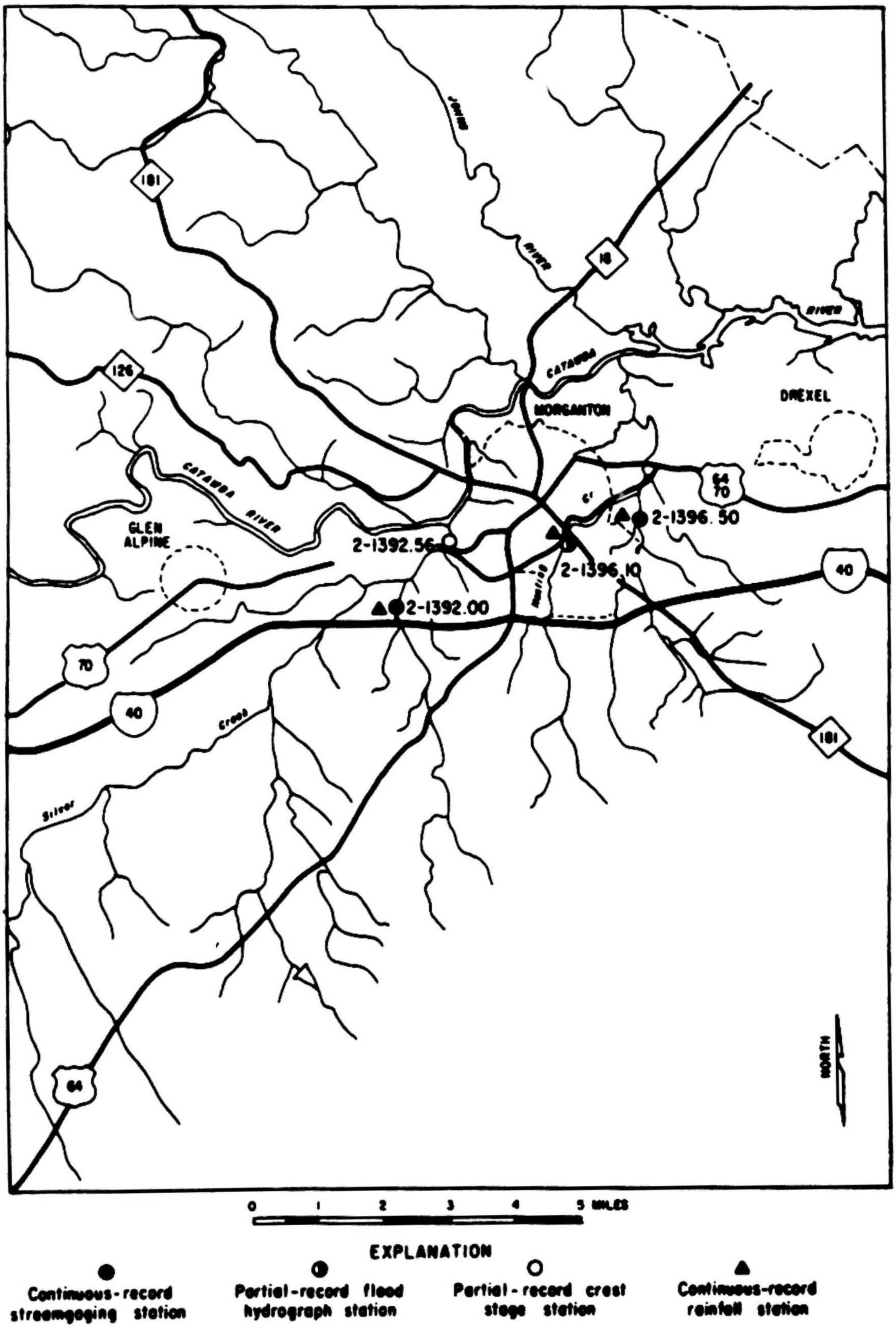


Figure 4.--Map showing data-collection points in the Morganton area.

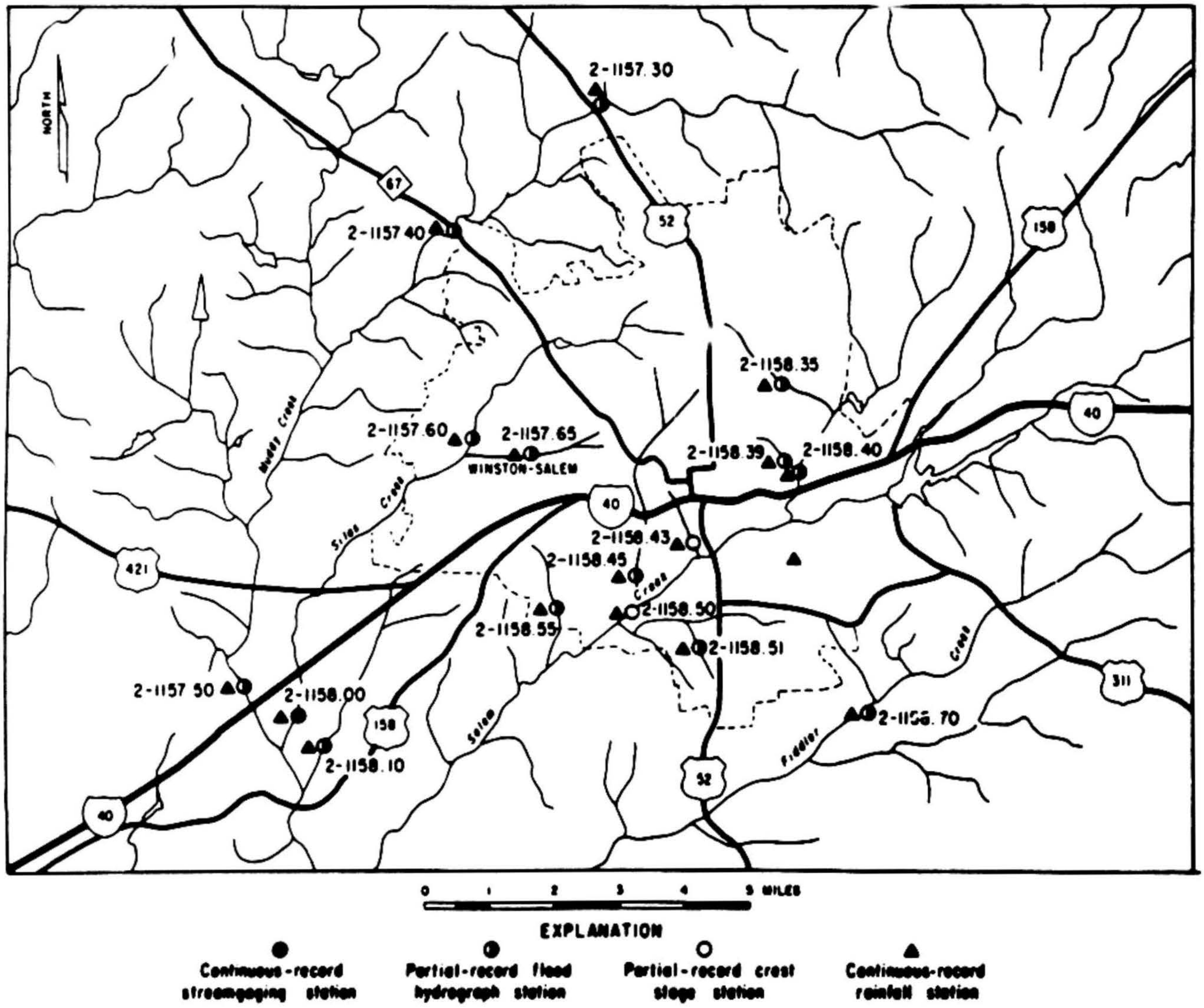


Figure 5.--Map showing data-collection points in the Winston-Salem area.

Basin Characteristics

We used three basin characteristics--drainage area, channel length, and channel slope--as indices of the physical characteristics of the watersheds included in this study. Drainage area is usually the most important factor influencing the hydrologic regime of streams. Generally speaking, for a given amount of rainfall, a larger basin will have greater flood runoff. Other factors are necessary to help explain the exceptions to this rule. We, along with other investigators (Snyder, 1958; Carter, 1961; Martens, 1968; and Anderson, 1970), chose channel length and slope as the most significant indices influencing flood runoff that could be easily measured. Length is an indication of the distance that runoff must travel, after falling as rain, to get to the point at which flow is measured. Slope, of course, is an indirect way to estimate the velocity at which the runoff will travel this distance. Slope and length are, therefore, very closely related to the lag time of the basin--the time between the center of mass of the rainfall excess and the center of mass of the resultant runoff.

For most sites we determined the drainage area by tracing the basin outline with a planimeter on detailed topographic maps, which the cooperating cities furnished. For other sites that were too large or lay outside the boundaries of the detailed topographic maps, we outlined the watershed boundaries and traced them with a planimeter on the best available maps.

Channel length, as we used it, is the distance in miles from the stream-gaging site to the basin boundary, measured along the main water course. Benson (1959, p. 4) defined the main water course above each stream junction as the stream channel draining the largest area. We measured all lengths on the most detailed topographic maps available.

Channel slope, as Benson (1959, p. 5-6) defined it, is the slope in feet per mile of the main water course between points 10 and 85 percent of the distance upstream from the stream-gaging site. We determined differences in elevation of the 10 and 85 percent points on the most detailed topographic maps available. For very small watersheds where contours crossing the stream channel are very dense or sparse, we made more accurate determinations with field surveys.

Tables 6 and 8 list drainage area, length, and slope for the stream-gaging sites that we used as a data base for this study.

Impervious Areas

Manmade structures, such as buildings, paved streets, and parking lots effectively prevent the infiltration of precipitation into the ground. In urban areas, a greater percentage of rainfall runs off the watershed and becomes streamflow than in rural areas. For this reason a given storm will produce relatively greater floods on an urban basin than on a rural basin. Thus the percentage of impervious cover on a basin is an important indicator of the hydrologic response of the basin.

Impervious cover is not only an indicator of the amount of rainfall that will become runoff, but it also is an indicator of the hydraulic improvements of the basin. For a given watershed the quicker the water runs off, the greater the flood magnitude. Impervious cover and the associated ditching, curb and gutters, drains, and storm sewers all tend to decrease the lag time of basins, and to increase peak flows. Although we made no attempt to measure storm sewers and the other man-made routes of flow that tend to speed runoff to the stream channel, we believe that impervious cover is a reliable index of this important factor in studies of urban flooding.

It is conceivable that the kind of impervious cover may affect the hydrologic response of a basin to some degree, but with the present state of knowledge of urban hydrology, it is difficult to distinguish such effects. For instance roofs in a residential community, where each house is surrounded by a lawn, may have relatively less effect than a paved parking lot that drains directly into storm sewers. In the present study, however, we assumed that paved roads, parking lots, roofs, sidewalks, and other impervious surfaces contribute equally to the hydrologic changes resulting from urbanization.

A factor probably of more significance than the kind of impervious cover is the location of the bulk of the impervious cover within the basin. If the impervious cover is concentrated far from the stream-gaging site, its effect on flood peaks, as measured at the site, may be different than if the heavy development is closer to the gaging site or is uniformly distributed throughout the watershed. Sufficient data to define the effect of location was not available and neglecting this is one reason for the scatter about the average relation derived.

For most sites in our study areas in North Carolina, we determined the impervious cover by visual inspection in the drainage basin. Survey teams counted houses in residential areas and multiplied this number by the average size of roofs in the development to obtain the roof area. They measured road mileage to the nearest one-tenth mile by car odometer and multiplied by the street width, including sidewalks (if there were any), to obtain the paved area. The teams estimated or measured the size of individual commercial and industrial buildings, the area covered by parking lots, and any other manmade impervious areas within the watersheds. Tables 1-8 list the percentage of impervious cover in all the basins used in this study, including percentages earlier investigators obtained using slightly different methods.

At 2-year intervals during the study we conducted field surveys to record changes that had occurred. We found only one watershed, that of Little Sugar Creek tributary 6 at Brookcrest Drive in Charlotte, in which the initial impervious cover changes as much as 10 percent. This change occurred immediately after installation of the gaging station, however, and consequently did not affect the records from that site, which were used in this study.

Depending on the use of the data, one may express impervious cover either as a percentage or as a ratio of impervious area to total drainage area. The computational procedures resulting from the inclusion of impervious cover in the analysis of flood runoff made it necessary to assume that rural watersheds contained at least 1 percent impervious cover. In most drainage areas of the North Carolina Piedmont, this assumption is acceptable, because nearly all basins contain paved highways, houses and small communities.

DATA ANALYSIS

Need for Improved Technique

Managers, developers, and planners need to know the magnitude-frequency relation of floods in order to make the decisions involved in the efficient planning and construction of urban developments. The primary objective of frequency analysis of hydrologic data is to determine the recurrence interval of hydrologic events such as floods of a given magnitude. In the case of floods, the average interval of time within which a flood of a certain magnitude is expected to be equaled or exceeded once is known as the recurrence interval, return period, or frequency of that event.

Hydrologists have derived many formulas for computing flood-peak discharges, but the most common method for computing a peak discharge of desired recurrence interval is the rational formula. While engineers sometimes use this formula to compute flood-peak discharge for relatively large areas, they probably should limit its use to areas of less than about 200 acres. The basic concept of the rational method may be stated as follows. For every watershed there is a period known as the time of concentration which is the time required for a particle of water to flow from the most remote part of the watershed to the site of interest. The peak discharge occurs when runoff from the whole watershed is reaching the site. The discharge relation can be expressed as follows:

$$Q = C\ i\ A$$

Where C is the coefficient of runoff; i is the intensity of the rainfall during the period equal to the time of concentration and has a recurrence interval of the desired return period; A is the drainage area; and Q is the flood-peak discharge. Q is usually expressed in cubic feet per second, i is in inches per hour, and A is in acres.

Computation of the flood-peak discharge using the rational formula involves the selection of three values; the coefficient of runoff indicating various types of watershed characteristics; the time of concentration incorporating the time required for overland flow and channel flow; and the rainfall intensity related to the time of concentration and the desired recurrence interval of the flood-peak discharge. Selection of these values requires considerable judgment, and often among experienced users the computed flood magnitude for a given watershed and recurrence interval will vary widely.

Another common method for determining the flood-peak discharge at an ungaged site is based on the ratio of the discharge of desired recurrence interval to the average annual flood. This technique is known as the index-flood method. The ratio is developed from gaging-station data within the same geographical region, and a method is devised to use the gaging-station data to predict the average annual flood for the ungaged site; then the discharge of desired recurrence interval at the ungaged site is estimated using the ratio for that recurrence interval. A typical equation takes the following form:

$$Q_i = R\, Q_{Avg} ,$$

Where Q_i is the discharge of the desired recurrence interval, Q_{Avg} is the discharge of the average annual flood, and R is the ratio of the discharge with recurrence interval i to the discharge of the average annual flood.

In North Carolina, three index-flood methods are presently in use (Speer and Gamble, 1964; Hinson, 1965; and Martens, 1968). The major problems associated with these methods are in the limitations of their use. The method described by Speer and Gamble (1964) is limited to the magnitude and frequency of floods for natural drainage areas greater than 30 square miles and for recurrence intervals of 50 years or less. Hinson (1965) describes an index-flood method for drainage areas between 1 and 150 square miles, but this method is again limited to recurrence intervals of 50 years or less, and non-natural influences on floods--such as urbanization--are not evaluated. Martens (1968) describes a method that does account for the influence of urbanization on floods, but again is limited to recurrence intervals of 50 years or less, and is geographically limited to the metropolitan area of Charlotte, North Carolina. No one has developed an index-flood method that incorporates recurrence intervals greater than 50 years, the effects of urbanization on floods, and a wide area of applicability.

Other methods exist, but will not be discussed in this report because they are rarely used in the Piedmont province of North Carolina. The above methods are discussed because of their widespread use in the study area. They are also discussed because they demonstrate the need for an improved technique to overcome the lack of data, the problems of subjective judgment, and the inability to account for urbanization.

Approach to the Problem

A number of investigators have evaluated the influence of urbanization on the shape of the surface-runoff hydrograph by establishing relationships between indices of hydrograph shape and indices of drainage basin characteristics. A logical procedure for studying the effects of urbanization would be to examine runoff under various stages of urbanization and show how the indices of the hydrograph shape are affected. It would then be possible to estimate future runoff conditions.

Urban development in watersheds, insofar as it affects hydrographs, is manifested in two ways. One way is by the reduction in rainfall losses when permeable soils are covered with impermeable roofs, roads, sidewalks, and parking areas. The other way is by the provision of hydraulically more efficient channels through which the storm runoff can flow. While the increase in total runoff is of significance in some areas, particularly in those areas where sandy or otherwise very permeable soils occur, it appears that the most significant effect of urban development is the sharp increase in the rate of peak storm runoff, resulting from the reduced time of concentration.

This effect is illustrated in figure 6. The hydrograph shown by the line consisting of long and short dashes represents a flood from an undeveloped (natural) drainage basin. If the drainage system of the basin is made hydraulically more efficient by adding storm sewers--including ditching, straightening of channels to eliminate excessive meander, and clearing the channels and flood plains of excessive obstructions to flow--the storm runoff will leave the basin faster and thereby change the hydrograph to a shape represented by the broken line. Hydraulically more efficient drainage is assumed to have negligible effect on the volume of rainfall excess which is also shown in figure 6. So the volume of storm runoff shown by the area under the broken-line hydrograph is the same as that for the natural channels and natural surface.

As the drainage basin is developed, construction of buildings, roads and parking areas effectively reduce the amount of rainfall that infiltrates into the ground and the amount of rainfall that is retained in surface depressions, thereby increasing the amount of rainfall excess or storm runoff. These changes result in the hydrograph represented by the solid line and the additional rainfall excess shown in the rainfall graph. The changes in the discharge hydrograph shown in figure 6 are based on the assumption of uniform areal distribution of development within the basin. In actual basin development, the hydraulic improvements and construction of impervious areas occur concurrently, so that there is little opportunity to observe the effects of either change independently.

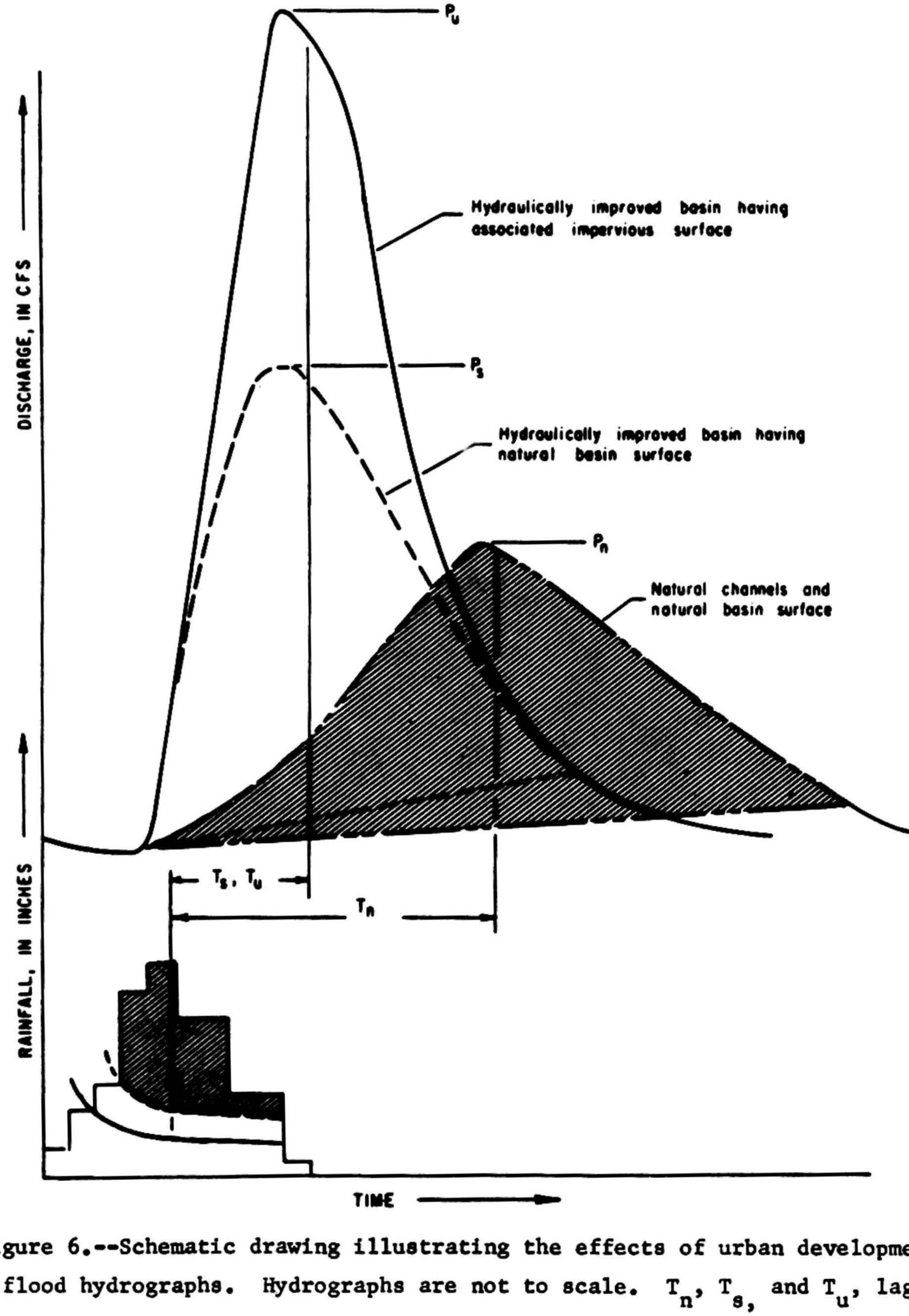


Figure 6.--Schematic drawing illustrating the effects of urban development on flood hydrographs. Hydrographs are not to scale. T_n, T_s, and T_u, lag times; P_n, P_s, and P_u flood peaks of the hydrographs for the three basin types shown.

The factors which determine the shape of the surface-runoff hydrograph depend on the distribution of the rainfall excess and on certain physical characteristics of the drainage basin. It is essentially a hydraulics problem in unsteady, spatially variable flow in a complex system of nonprismatic channels. The equations of motion for this system are so complicated that an analytical solution is possible only if broad simplifying assumptions are introduced. Even the analysis of complex hydrographs produced by continuous or closely spaced rains of varying intensities cannot be carried out with full confidence that each rain event is represented by a clearly delineated surface-runoff hydrograph. Therefore, attention must be focused on simple hydrographs produced by one relatively short period of rainfall excess.

For basins of a given area, the shape of a simple hydrograph depends on indices which represent the distribution of rainfall excess available for runoff, the length of the stream channel or distance the runoff must travel, velocity of water in the channel, and the characteristic of the area contributing surface runoff. The channel-length factor may be taken as the total length along the main water course. The velocity is a function of the slope of the stream, and the characteristic of the area contributing surface runoff can be indicated by the percent of impervious cover in the basin. Basin lag time is representative of the time distribution of the rainfall excess with respect to the time distribution of the surface-runoff hydrograph.

Basin lag time is defined as the average time interval in hours between the center of mass of the rainfall excess and the center of mass of the resultant runoff. The changes in basin lag time are also illustrated in figure 6. The hydraulic improvements within the basin shorten the storm-runoff period, thereby decreasing the basin lag time, as shown by the time difference between the centers of mass for the hydrograph representing the natural condition and the hydrograph representing the hydraulically improved condition.

As mentioned previously, it is possible to estimate future runoff conditions by examining runoff under various stages of urbanization and showing how the indices that influence the shape of the hydrograph are affected. The urban development would have little or negligible effect on the length of the main water course, and the stream slope would remain essentially unchanged. The characteristic of the area contributing to surface runoff, indicated by the percent of impervious cover, would be substantially changed, but this change can be anticipated or planned in advance. The basin lag time which is a representation of the time distribution of the rainfall excess with respect to the time distribution of the surface-runoff, would be significantly affected, and an estimate of the lag time for the degree of development would be required to estimate future flood conditions.

Many investigators have used lag time as the significant parameter in urban hydrologic studies and in studies of the effect of urbanization on runoff. Snyder (1958), Carter (1961), Viesman (1966A, 1966B and 1968), Eagleson and March (1965), Espey and others (1966), Schaake (1965), Martens (1968), Sarma and others (1969), and Anderson (1970) based their investigation on the assumption that lag time is affected mainly by watershed characteristics, and many of these investigators have proposed relationships for lag time in terms of various physical characteristics of the watershed.

Most investigators agree that the percent of impervious cover, although permanently affecting the basin hydrology, does not have the same effect on the rainfall available for runoff of all storms. As the storm magnitude increases, the percentage of rainfall that infiltrates into the ground, is trapped in surface depressions or is lost by evaporation or other means becomes less and less even for a rural basin until the amount of loss has little or no effect on the volume of rainfall available for runoff. This would tend to imply that at some point in the flood frequency distribution there is no difference between an urban and a rural flood magnitude. However, the improvements in the hydraulic efficiency of the drainage system remain in effect and continue to speed the runoff past a point on the stream. This means that the bulk of the runoff passes a point on the stream in a shorter period. The rate of flow, and consequently the peak flow, is appreciably higher than for an undeveloped condition. If we consider the percent of impervious cover within the watershed as an index to the initial increase in the volume of rainfall available for runoff and, at the same time, an easily determined index to the improved efficiency in the rate at which the runoff moves across the watershed and enters a stream, we can follow the lead of some of the earlier investigators and relate the basin lag time to the channel length, the channel slope, and the percent of impervious cover in the basin.

Then, because the basin lag time is a representation of the time distribution of the rainfall excess with respect to the time distribution of the surface runoff--in other words, an index of the improved hydraulic efficiency associated with the degree of urban development in the basin--a change in lag time affects the storm runoff of all magnitudes. Therefore, average basin lag time, which our data indicates is essentially constant through a wide range of flood magnitudes, helps explain the variations in flood-peak discharge from watershed to watershed and from one condition of development to another. We again followed the lead of some of the earlier investigators and related the flood-peak discharge to the watershed size and the basin lag time.

Definition of Relations

As indicated in the previous section the approach to data analysis for this study required examining the effect of urban development on basin lag time as it related to changes in flood-peak discharge. The most logical procedure was to investigate the changes in lag time under various degrees of urbanization and relate these to flood-peak discharge of different recurrence intervals. The analysis was, therefore, divided into two parts. In the first part, an equation was developed for predicting the effect of urban development on the basin lag time. In the second part, equations were developed for predicting flood-peak discharge for recurrence intervals ranging up to 100 years. In both parts of the data analysis, multiple regression techniques were used in the development of the equations. Regression techniques are computational procedures that permit an investigator to relate a dependent variable to one or more independent variables. This mathematical technique results in an equation to predict the dependent variable from the independent variables that can be determined for ungaged sites. Because digital computers can quickly and efficiently run complex multiple-regressions, we were able to utilize in this study a large volume of data representing a wide range of basin characteristics.

Basin lag time.--The computation of lag time is a tedious job involving the separation of rainfall excess from the total measured rain and the separation of direct runoff from the total measured runoff. As mentioned previously, attention must be focused on a simple hydrograph produced by one relatively short period of rainfall excess to insure that each rain event is represented by a clearly delineated hydrograph. We determined lag time for 6 to 20 storms for each stream-gaging station where rainfall and streamflow data were available. The selection was based on the storms that most closely approximated the following conditions:

1. Rainfall appeared to be uniformly distributed over the basin.

2. Rainfall duration was relatively short compared to the expected lag time.

3. Rainfall intensities approached a uniform distribution.

4. The amount and intensity of rainfall was sufficient to produce significant flood-peak discharge.

5. The flood hydrograph contained only a single peak. (Multiple-peak storms complicate the determination of storm runoff and the portion of the rain causing the different peaks.)

6. Basin soils were neither excessively dry nor excessively wet at the beginning of the storm.

For each selected storm, we determined the time distribution of direct runoff from the hydrograph of total measured runoff using the method described by Linsley and others (1958, p. 149-161). Then we adjusted the volume of rainfall excess to equal the volume of direct runoff. Infiltration curves similar to those described by Sherman (1940, p. 541-550) aided in defining the time distribution of the rainfall excess. The time difference between the centers of mass of these two distributions provided a measure of the basin lag time. These computations are illustrated in figure 7.

Data for the 118 drainage basins listed in table 6 defined lag time as a function of length, slope, and impervious cover. Forty-four of these basins are located in the Piedmont of North Carolina (pl. 1). Data for the remainder of the stations were obtained from a report by Anderson (1970). The percent of impervious cover ranged from less than 1 percent for undeveloped basins to 100 percent for some urban basins that drain less than an acre.

We tried several regression models and determined that the following equation, which has a standard error of estimate of $\pm$ 20%, was the best representation of the data:

$$T = 0.49 \left(L/\sqrt{S}\right)^{0.50} I^{-0.57} \tag{1}$$

where

T is the lag time in hours,

L is the length of the main water course in miles,

S is the stream bed slope of the main water course in feet per mile, and

I is the ratio of the area of impervious cover to the total drainage area.

The standard error of estimate, or standard deviation, is an indication of the effectiveness of an equation relating a dependent variable (T in equation 1) to one or more independent variables (L, S, and I in this instance). At about 68 out of 100 sites where lag time is estimated with the equation, the estimate will be accurate within one standard deviation. At approximately 95 out of 100 sites these estimates will be within two standard deviations of correct. Figure 8

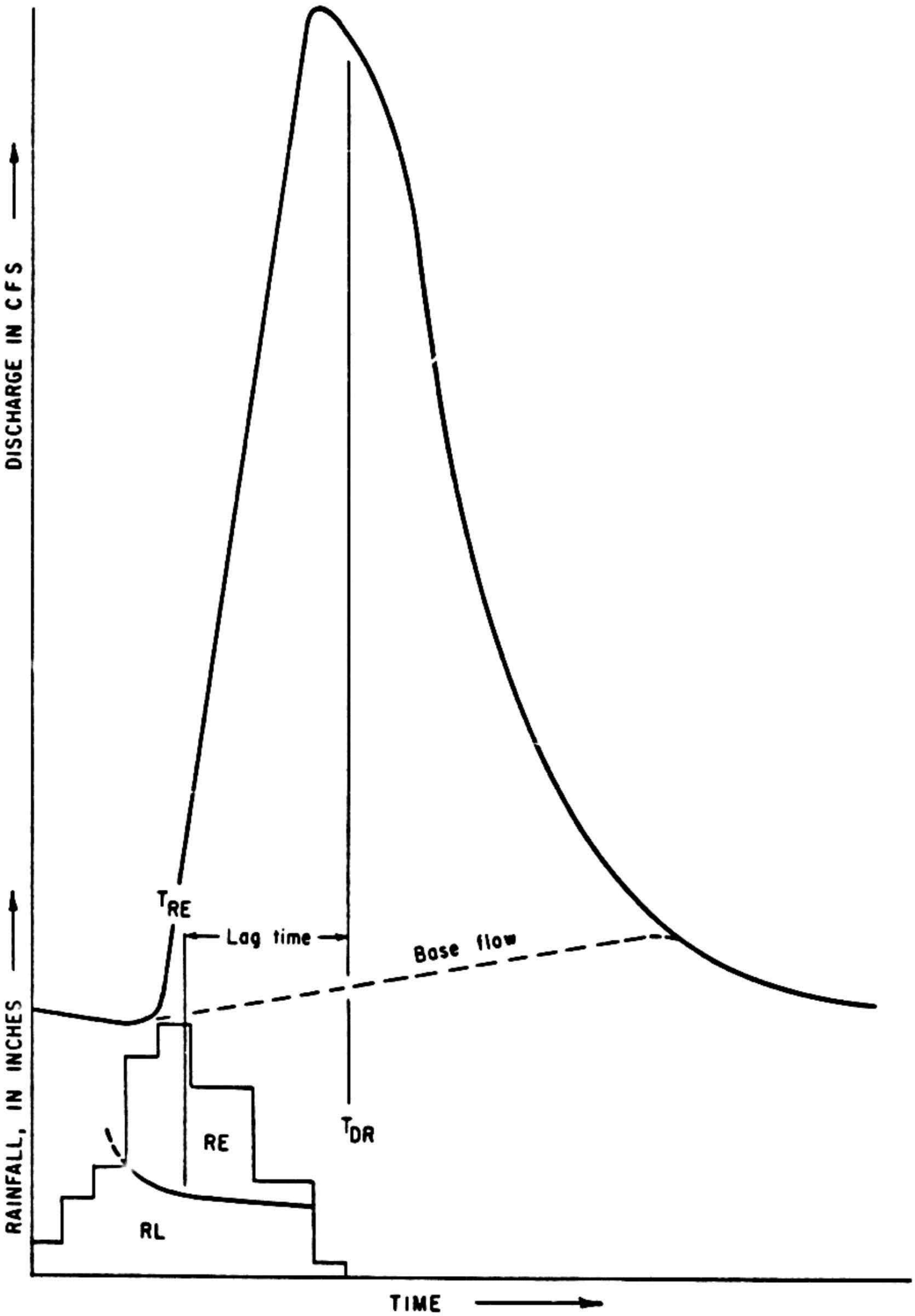


Figure 7.--Schematic drawing illustrating the computation of basin lag time. Not to scale. RE, rainfall excess; RL, rainfall losses; T_{RE}, time of the centroid of rainfall excess; T_{DR}, time of the controid of direct runoff.

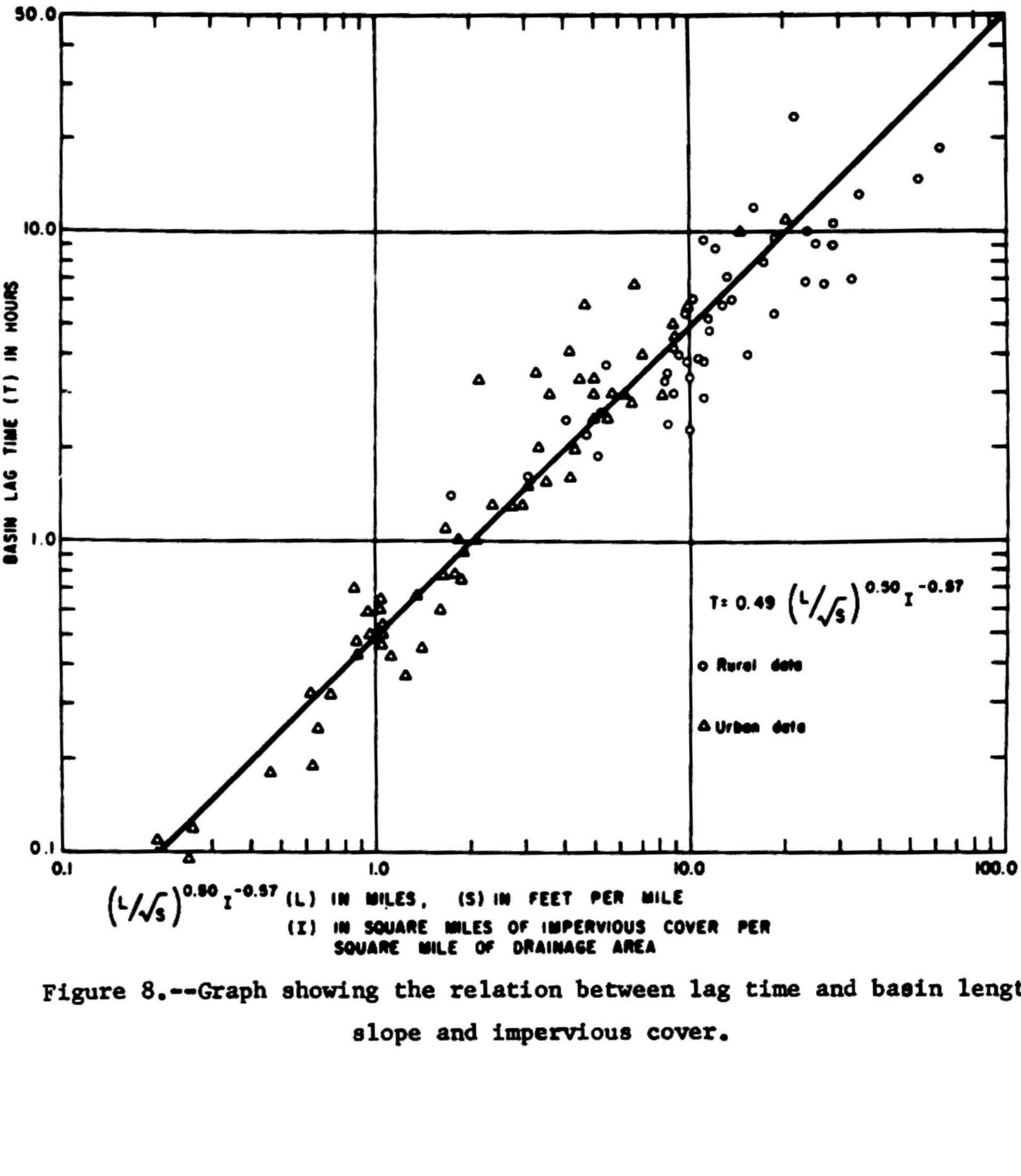


Figure 8.--Graph showing the relation between lag time and basin length, slope and impervious cover.

graphically illustrates the fit of the relationship for estimating lag time with the independent variables and the basin lag time determined from gaged streamflow and rainfall data. The plot in figure 8 indicates that equation 1 may not be the line-of-best-fit at the upper end of the urban data. When the urban data is considered separately from the rural data, equation 1 has a standard error of estimate of $\pm$ 30 percent.

Flood discharge.--The basin-to-basin variation in the magnitude of flood-discharge is related to variations in basin size and to the timing of the volume and peak of the flood runoff--the basin lag time.

From the observed data for the stream-gaging stations with more than 8 years of record, shown in table 7, we used only the largest peak in each year (annual-flood series) and constructed flood-frequency curves for each basin with the log-Pearson Type III frequency analysis as recommended by the U. S. Water Resources Council (1967). For sites with less than 8 years of record, we used all of the largest floods above a base that would average three or four floods a year to be included in the analysis. We then constructed the partial-duration series curves for sites with less than 8 years of record. Langbein (1949, p. 879-881) found that the recurrence interval of floods for the annual flood series and the partial-duration series had a definite relationship-- for example, from the annual series, the 2.00-year recurrence interval is comparable to the 1.45-year recurrence interval from the partial-duration series. While this may not always be true, analysis of long flood records in the study area indicate that Langbein's comparison is applicable. We used the relationship that Langbein (1949) presented and constructed an annual-flood series frequency curve for each of the sites with less than 8 years of record. Table 7 lists discharge data that are the result of these frequency analyses. These data were extended on logarithmic probability paper to extrapolate values up to the 25-year recurrence interval.

Our next step in the analysis was to derive equations relating these flood frequency data to the basin characteristics, drainage area and lag time. We wanted to be able to determine the magnitude of a flood of a certain recurrence interval knowing only the drainage area and basin lag time. The basin characteristics for the stations appearing in table 7 vary considerably. The impervious cover for these basins ranges from less than one percent to 32 percent. The drainage area includes a range from 0.27 to 178 square miles. We tried several multiple-regression models to fit equations to these varied data. The following equations proved to be the most reliable and the best representation of the data:

$$P_2 = 221\ A^{0.87}\ T^{-0.60} \qquad (2)$$

$$P_5 = 405\ A^{0.80}\ T^{-0.52} \qquad (3)$$

$$P_{10} = 560\ A^{0.76}\ T^{-0.48} \qquad (4)$$

$$P_{25} = 790\ A^{0.71}\ T^{-0.42} \qquad (5)$$

where

P_i is the peak discharge for the flood having the recurrence interval indicated by the subscript,

A is the drainage area in square miles, and

T is the lag time in hours.

The above regression equations, which are based on lag times that were determined from observed data, have a standard error of estimate of $\pm$ 30 percent for equations 2, 3, and 4 and $\pm$ 35 percent for equation 5. The standard error of estimate is a measure of the lack of fit of the data used to determine the equations. The standard error of prediction, however, is somewhat greater than the standard error of estimate, for it includes both the measure of the lack of fit of the data used to determine the equations and the measure of error in the data used in the application of the equations. In application, lag time would be estimated from equation 1 which has a standard error of estimate of $\pm$ 30 percent for urban conditions. The error of prediction would be approximately equal to the square root of the sum of the squares of the two separate standard errors of estimate. The standard error of prediction for urban conditions would then be $\pm$ 42 percent for equations 2, 3, and 4 and $\pm$ 46 percent for equation 5.

After using the regression analyses to develop these equations for the 2-, 5-, 10-, and 25-year flood, we plotted the coefficients of the discharge equations and the exponents of drainage area and basin lag time on logarithmic probability paper to extrapolate the coefficients and exponents for the 50- and 100-year floods. The resulting curves appear in figures 9 through 11. Using the values defined by the curves in figures 9 through 11, the equations for the 20-, 50-, and 100-year floods are:

$$P_{20} = 735\ A^{0.72}\ T^{-0.43} \qquad (6)$$

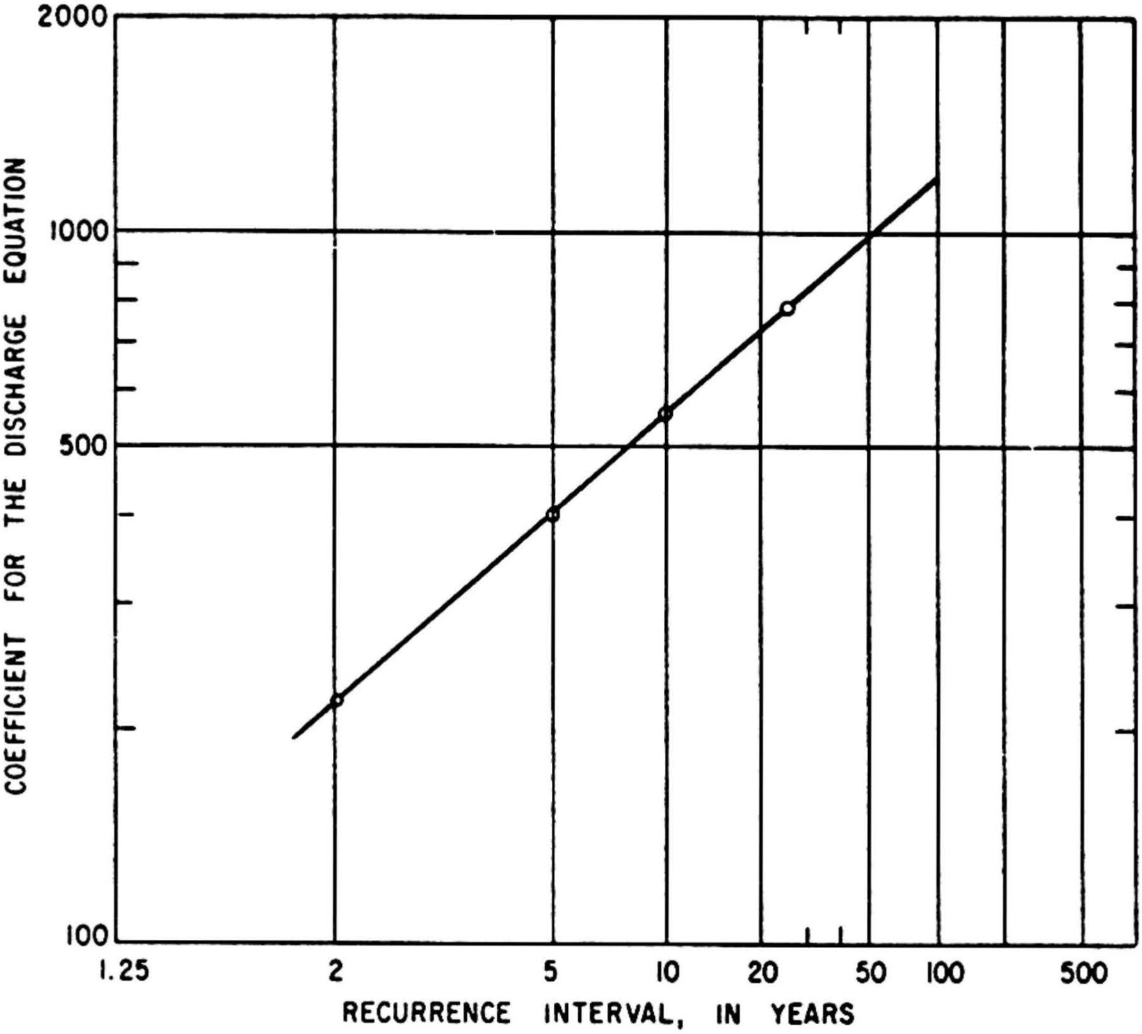


Figure 9.--Graph showing extension of the coefficient of the flood-peak discharge equations to cover the 50- and 100-year flood-peak discharges.

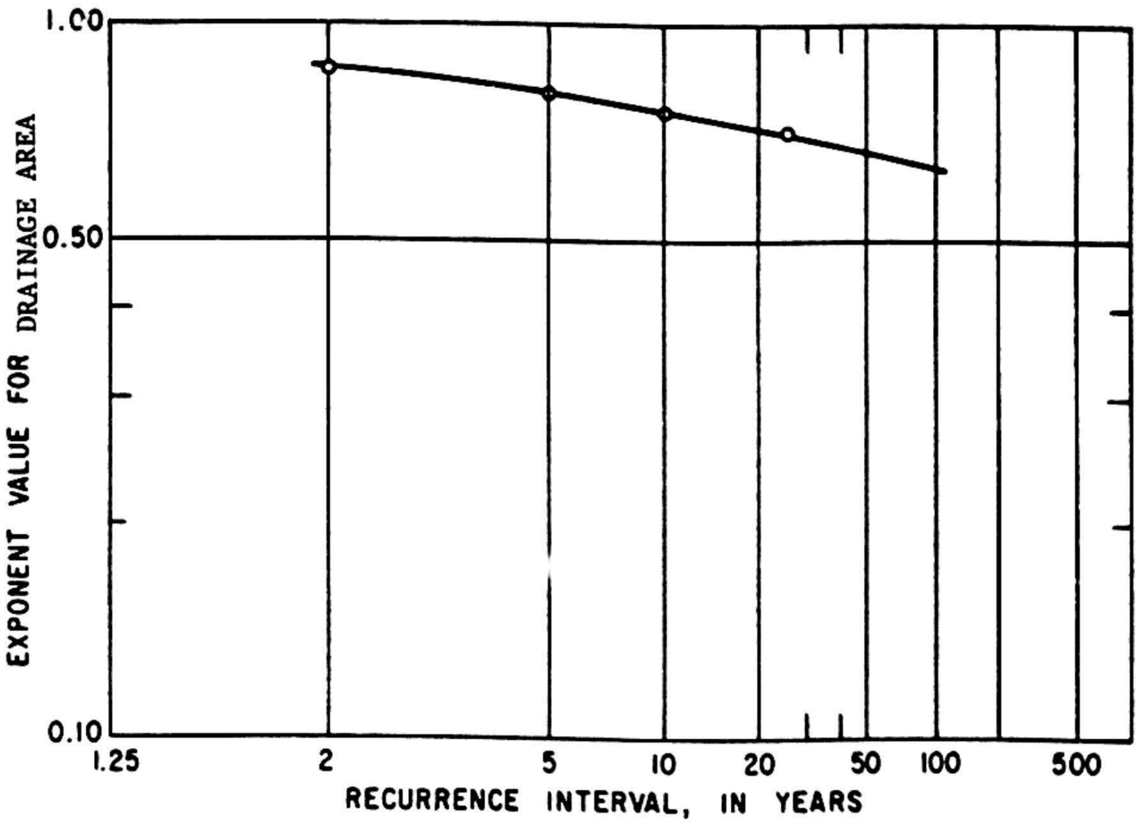


Figure 10.--Graph showing extension of the exponent for drainage area in the flood-peak discharge equations to cover the 50- and 100-year flood-peak discharges.

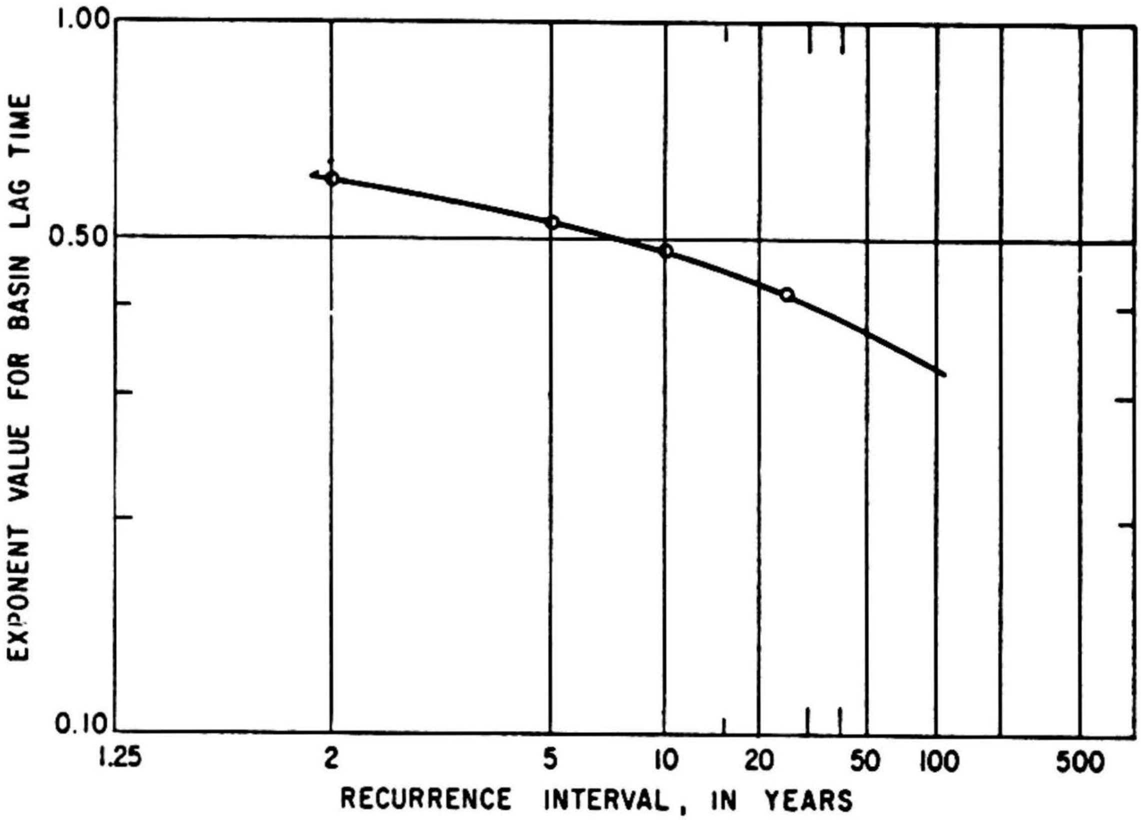


Figure 11.--Graph showing extension of the exponent for basin lag time in the flood-peak discharge equations to cover the 50- and 100-year flood-peak discharges.

$$P_{50} = 990\ A^{0.67}\ T^{-0.37} \tag{7}$$

$$P_{100} = 1200\ A^{0.63}\ T^{-0.33} \tag{8}$$

To demonstrate the reliability of equations 2-8, we have plotted the data from the station frequency curve for each of the stations in table 7 versus values computed from the equations. These points are the open triangles on figures 12-17. All data would plot on the 45 degree lines on figures 12-17, if the computed values agreed exactly with the values from the station frequency curve.

To further test the validity of the flood discharge equation, we selected 76 long-term gaging stations (including a few that also appear in table 7) located in undeveloped areas over much of the State and for which we had log-Pearson Type III flood frequency data. These stations appear in table 8 along with the values of their 2-, 5-, 10- and 25-year floods. We also included the 50- and 100-year floods for stations where the period of record is long enough. Table 8 lists the period of record, drainage area, slope, length, and percent impervious cover for each of the stations. We computed the peak flood discharges for the 2-, 5-, 10- and 25-year recurrence intervals for all of these stations, and, where we had enough records to determine the 50- and 100-year floods from the station frequency curve, we also computed these values. The computed flood discharges are plotted versus the data from the station frequency curve on figures 12-17 as open circles.

The data in tables 7 and 8 are, in general, for rural streams with relatively long records or for urban streams with relatively short records. Obviously, we needed to have some data from urban streams to further test the validity of the discharge equations for the 50- and 100-year floods. Since there are not enough urban streams with long record from which we could directly determine the 50- and 100-year floods, we selected six urban stations for extension of the short record by hydrologic modeling.

We used the rainfall-runoff model that Dawdy and others (1970) developed to simulate flood records. This simulation is done by computer and involves calibrating the mathematical model which describes the way in which runoff occurs in a basin following a rain. The calibration procedure requires several years of concurrent streamflow and rainfall records to determine the optimum parameters for use in the model. For the six urban stations, we used several years of concurrent rainfall-runoff data to adjust, or calibrate the parameters for this model. The adjustment or calibration of the model parameters is accomplished with the recorded rainfall as input to the model and simulated runoff as

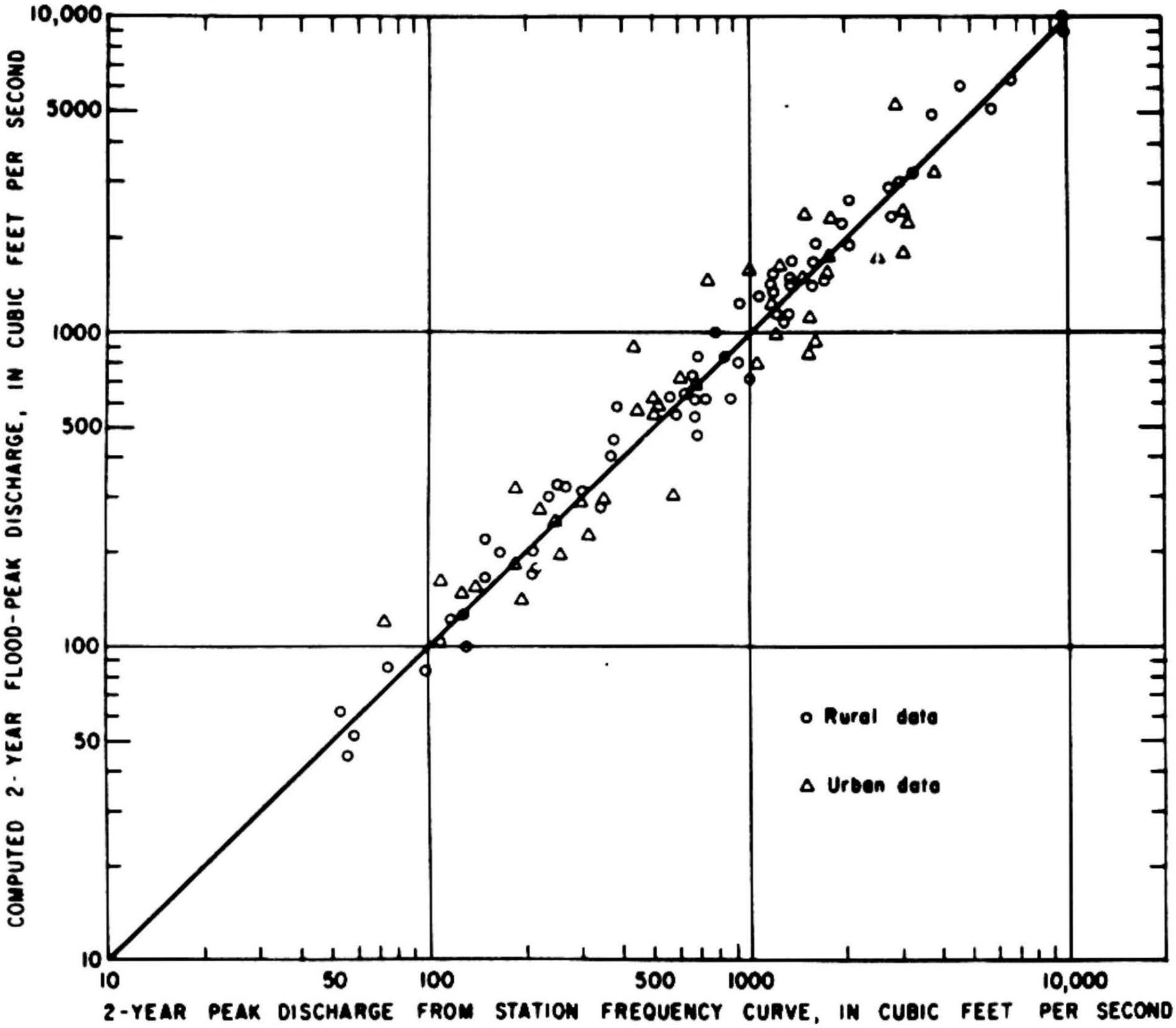


Figure 12.--Graph comparing computed 2-year flood-peak discharge base on relation, $P_2 = 221\ A^{0.87}\ T^{-0.60}$, with observed 2-year flood-peak discharge for 118 gaging stations.

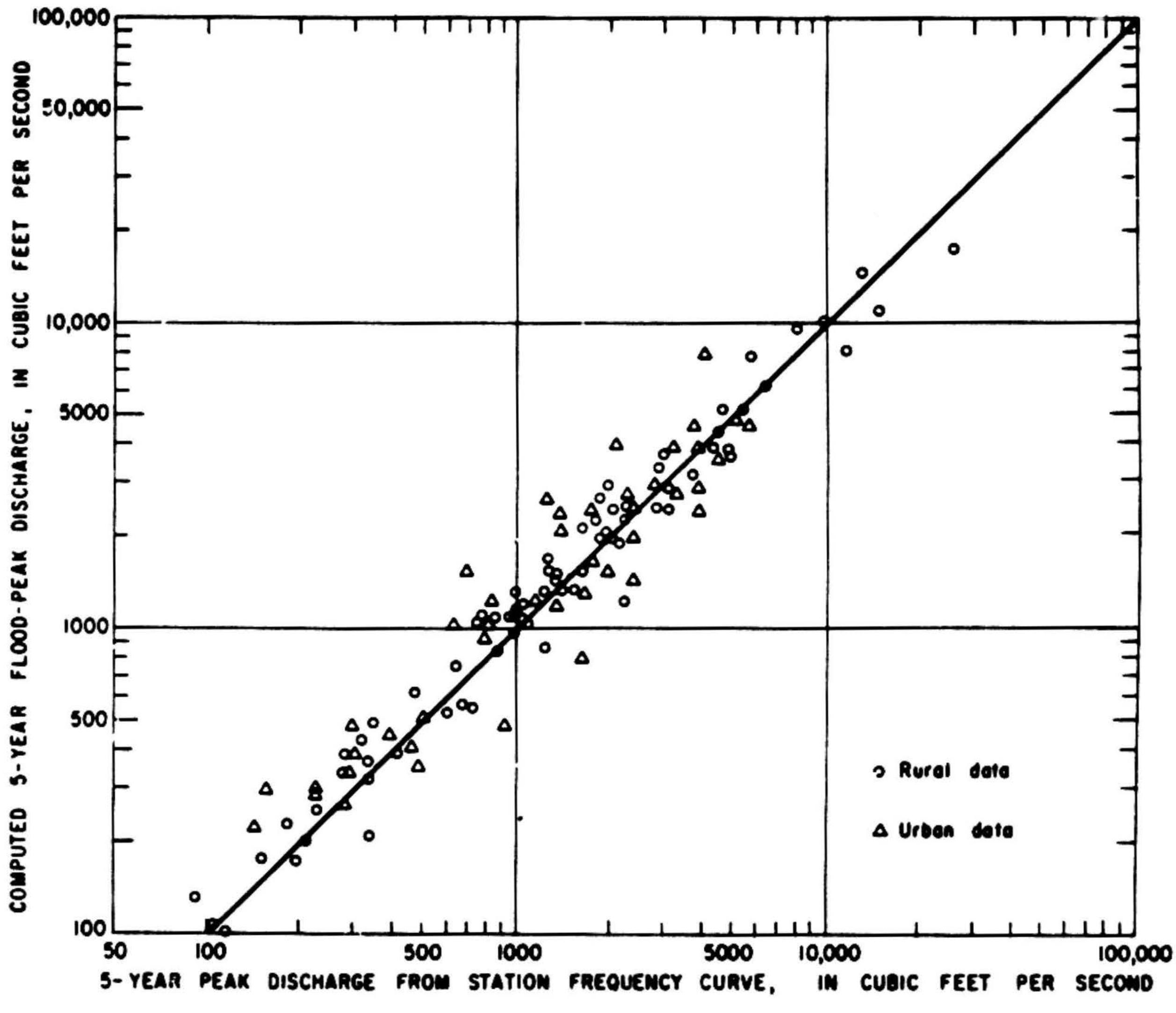


Figure 13.--Graph comparing computed 5-year flood-peak discharge based on relation, $P_5 = 405\ A^{0.80}\ T^{-0.52}$, with observed 5-year flood-peak discharge for 118 gaging stations.

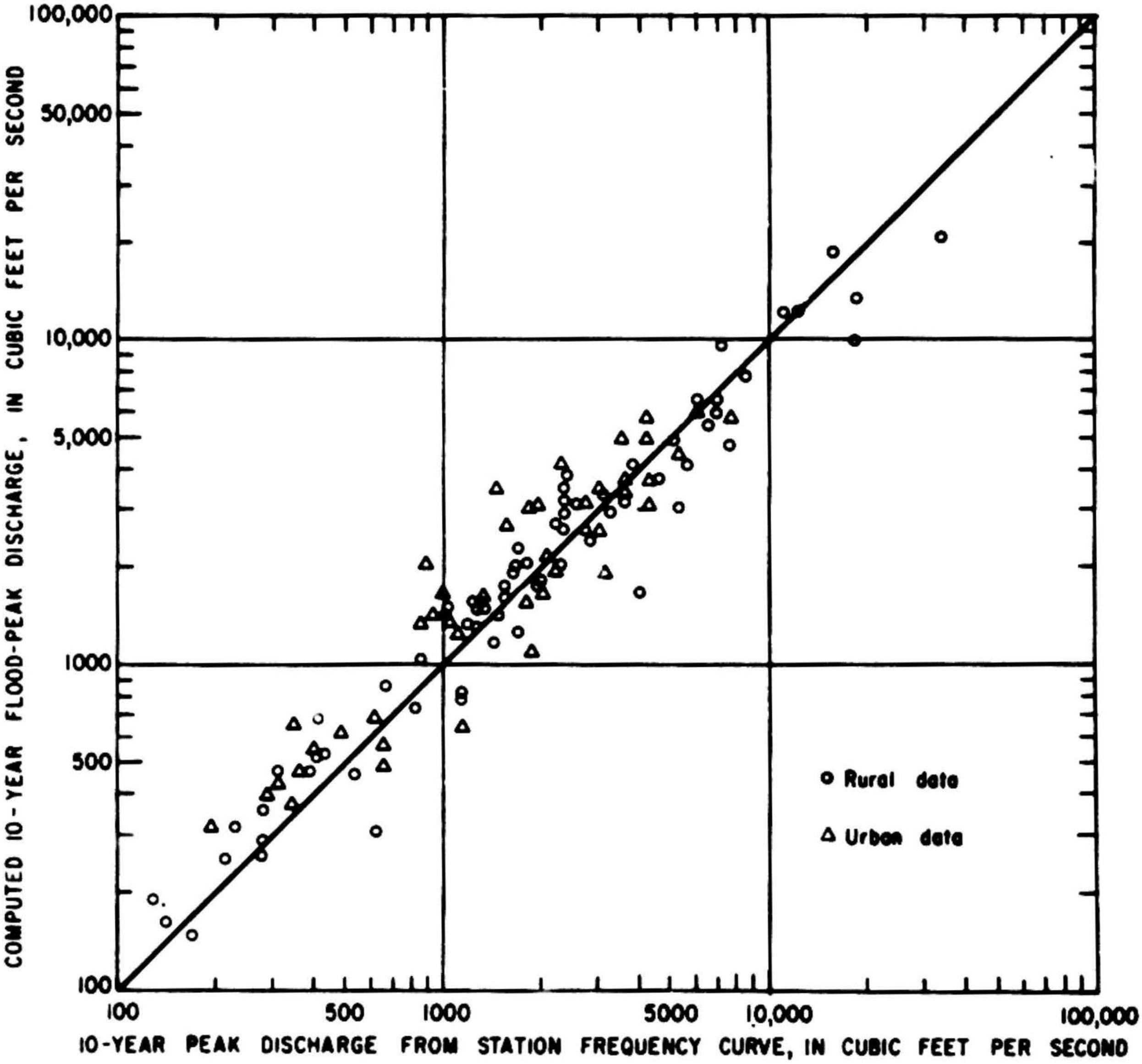


Figure 14.--Graph comparing computed 10-year flood-peak discharge based on relation, $P_{10} = 560\ A^{0.76}\ T^{-0.48}$, with observed 10-year flood-peak discharge for 118 gaging stations.

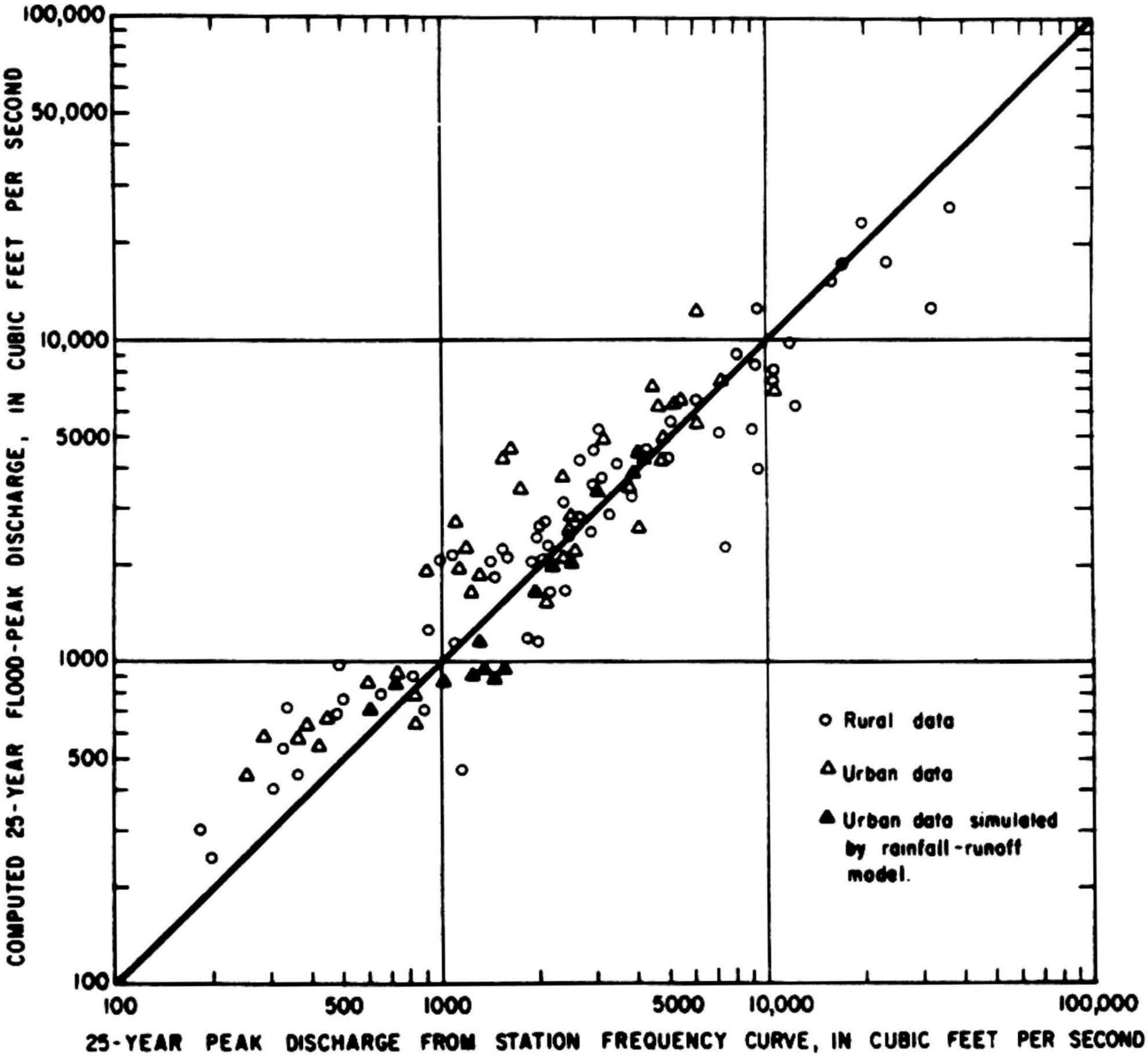


Figure 15.--Graph comparing computed 25-flood-peak discharge based on relation, $P_{25} = 790\ A^{0.71}\ T^{-0.42}$, with observed 25-year flood-peak discharge for 118 gaging stations.

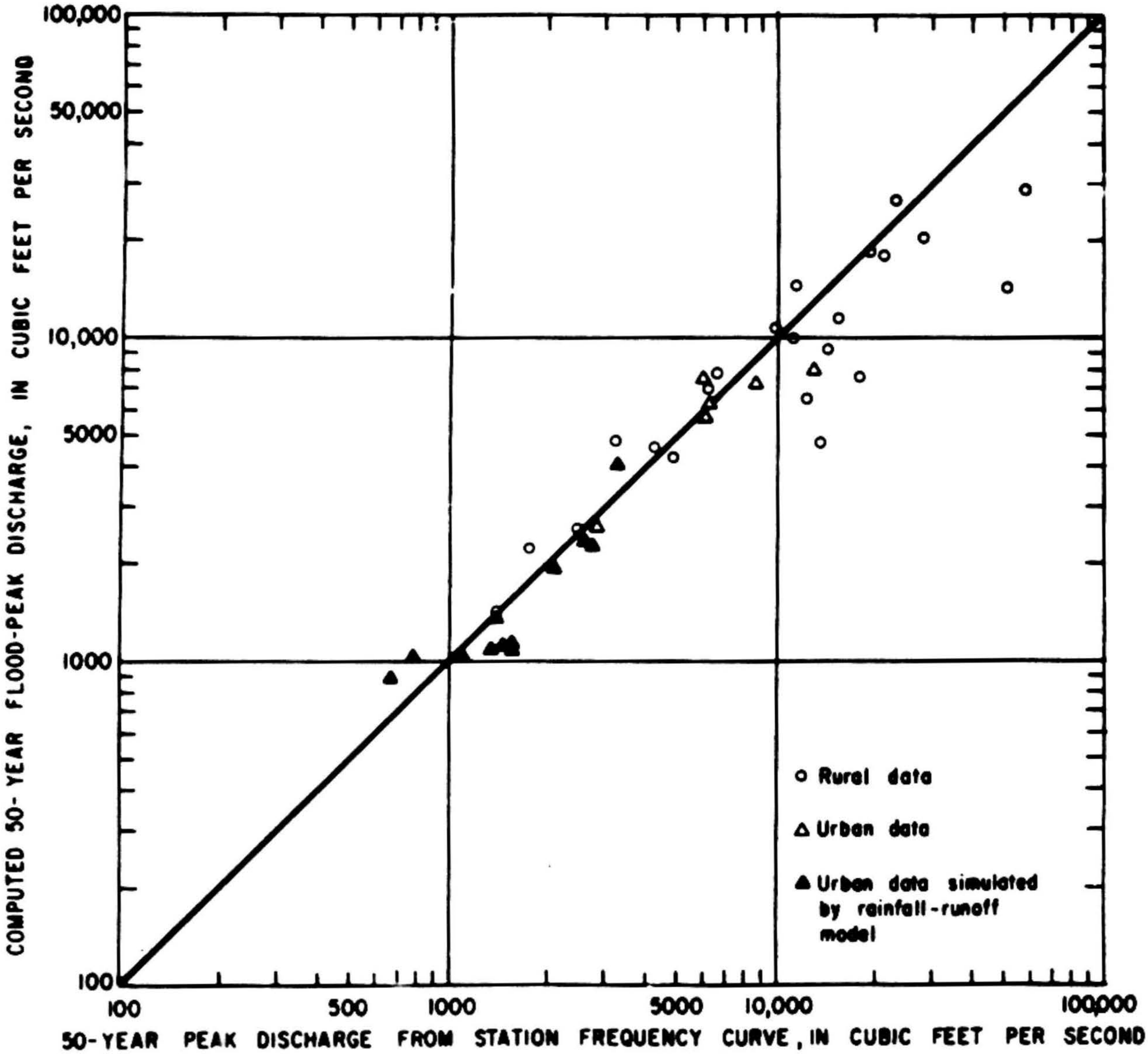


Figure 16.--Graph comparing computed 50-year flood-peak discharge based on relation, $P_{50} = 990\ A^{0.67}\ T^{-0.37}$, with observed 50-year flood peak discharge for 34 gaging stations.

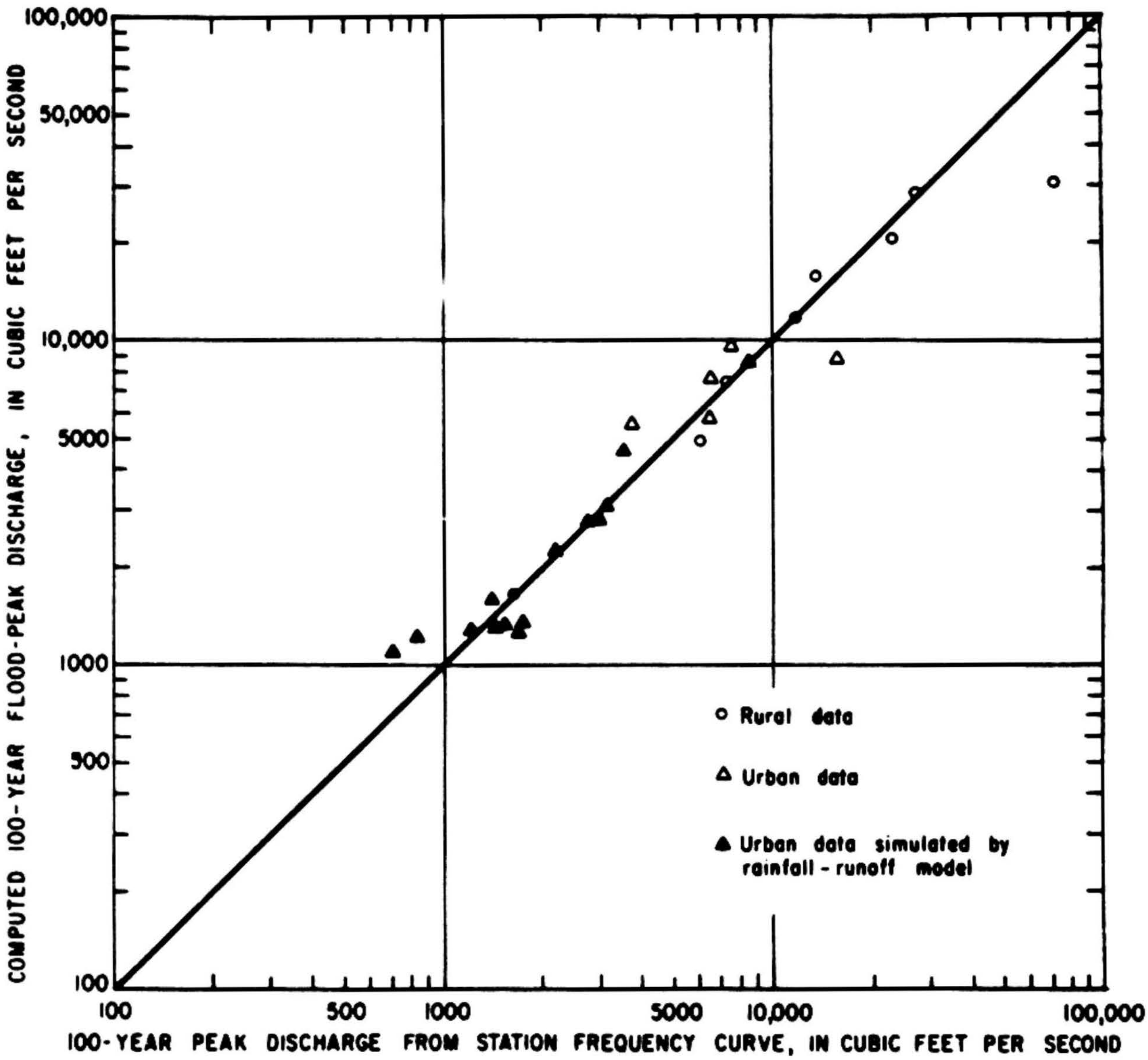


Figure 17.--Graph comparing computed 100-year flood-peak discharge based on relation, $P_{100} = 1200\ A^{0.63}\ T^{-0.33}$, with observed 100-year flood-peak discharge for 20 gaging stations.

output. When the simulated runoff agrees closely with the recorded runoff, the model parameters should be at their optimum value. We could then use a long record of rainfall to generate an equally long record of floods. In this case we used the long rainfall record collected in Charlotte and mentioned earlier in this report for all six stations. It should be noted here, we have assumed that the sample storm rainfall for Charlotte would be representative of a sample taken from an extremely long rainfall record at any location in the Piedmont of North Carolina.

In using the rainfall-runoff model, we selected six gaging stations that ranged in drainage area from 0.52 square miles to 8.26 square miles and ranged in impervious cover from 3 to 37 percent. For each of these stations, we simulated 68 years of flood record for the existing conditions at the gaging station. Then, we arbitrarily selected a projected condition of urban development of 40 percent impervious cover and using the rainfall-runoff model simulated another 68 years of flood record for each gaging station with the 40 percent impervious cover included in the model parameters. This provided us with 12 simulated, long-term urban streamflow records. We determined the flood frequency relations that defined the 25-, 50-, and 100-year floods for each of these records by fitting log-Pearson Type III curves to the simulated data. The results of this analysis appear in table 9 along with other pertinent information.

We plotted the simulated flood values from table 9 versus the flood values computed from equations 5, 7, and 8 on figures 15, 16, and 17. These points appear as solid black triangles.

The results of this exercise, which figures 12-17 depict, seem to validate the equations for computing flood discharges (equations 2-8). The closeness that the open triangles fit the 45 degree line shows that the equations are representative of the data on which they were based. The open circles indicate that the equations will also reliably predict floods for other rural basins in the region, and the solid triangles show that the equations are reliable for estimating urban flooding.

We are particularly happy that there appears to be little bias in the flood discharge equations. It would be a matter of concern if, for instance, all the urban data plotted below the 45 degree line and all the rural data plotted above, or if data for floods on streams with large drainage areas tended to plot on one side of the line while data for small drainage area streams plotted on the other. The data seems to fit well for floods of different magnitudes and recurrence intervals and for different kinds of basins.

There is some scatter around the lines of equality. Some of this is errors that are inherent in the equations. Empirical equations of this kind will not give perfect answers every time. But some of the scatter is because of the uncertainties in determining the observed flood frequency values from relatively short-term record.

EFFECTS OF URBANIZATION

Many factors influence the flood peak at a particular site on a stream. Among these factors are the amount of rainfall available for direct surface runoff, the distance that the surface runoff must travel to reach the site, and the velocity at which it moves over this distance. In this report we use basin lag time as an index to the changes in the hydraulic efficiency of a basin. Because urbanization changes the hydraulic efficiency of a basin, it follows that the basin lag time is greatly affected by urban development.

Because we needed a method to determine lag time for basins where flow is not gaged and because the computation of lag time is so time consuming for gaged basins, we developed an empirical method to predict the lag time based on natural basin characteristics and on the degree of urbanization. The method requires three basin characteristics; the length of the main water course, the slope of the stream, and the impervious cover in the basin. The length is an index of the distance surface runoff must travel to reach the point of interest, while the slope of the stream is an index of the velocity at which the surface runoff moves over this distance. The impervious cover is an index of the degree of urban development, indicating not only the improvement in hydraulic efficiency over natural or rural conditions but also the additional amount of rainfall available for flood runoff.

The change in hydraulic efficiency when a basin progresses from a rural or undeveloped condition through various degrees of urban development can be illustrated by computing lag times for a basin at various stages of development. This can be illustrated using data for Brushy Creek Tributary 2 at U.S. Highway 311 in Winston-Salem. The drainage area for this site is 0.55 square miles. The length of the main water course is 1.10 miles, and the slope of the channel is 143 feet per mile.

Before any development occurred, the basin lag time could be evaluated by equation 1 as follows:

$$T_r = 0.49\ ({}^{L}/\sqrt{S})^{0.50}\ I^{-0.57}$$

$$T_r = 0.49\ ({}^{1.10}/\sqrt{143})^{0.50}\ (0.01)^{-0.57}$$

$$T_r = 0.49\ (0.09)^{0.50}\ (13.8)$$

$$T_r = 0.49\ (0.30)\ (13.8)$$

$$T_r = 2.03 \text{ hours},$$

where the subscript r refers to a rural or undeveloped condition, and the impervious cover (I) amounts to 1 percent or less.

Now suppose that the development has progressed to a level associated with 25 percent impervious cover. Solving equation 1 with 0.25 substituted for I gives a basin lag time of 0.30 hours. This lag time reflects conditions existing at the present time, but suppose that the ultimate development of this basin will reach 50 percent impervious cover. Solving equation 1 for an I of 0.50 gives a basin lag time of 0.22 hours. Developers and planners can use this lag time to evaluate the effects of the projected ultimate development on peak flows from the basin.

Once we have determined the basin lag times, we then can compute the corresponding flood-peak discharge. For undeveloped conditions in the Brushy Creek Tributary 2 basin, equation 2 gives the 2-year flood-peak discharge as follows:

$$P_2 = 221\ A^{0.87}\ T^{-0.60}$$

$$P_2 = 221\ (0.55)^{0.87}\ (2.03)^{-0.60}$$

$$P_2 = 221\ (0.59)\ (0.65)$$

$$P_2 = 85 \text{ cubic feet per second.}$$

Substituting in equation 2 the lag time value, 0.30 hours, for the development level corresponding to 25 percent impervious cover gives a 2-year flood-peak discharge of 269 cubic feet per second. Substituting in equation 2 the lag time value, 0.22 hours, corresponding to 50 percent impervious cover, gives a 2-year flood-peak discharge of 323 cubic feet per second for the projected, ultimate condition of development in the basin.

For the rural or undeveloped condition of Brushy Creek Tributary 2 the 25-year flood-peak discharge can be computed using equation 5 as follows:

$$P_{25} = 790\ A^{0.71}\ T^{-0.42}$$

$$P_{25} = 790\ (0.55)^{0.71}\ (2.03)^{-0.42}$$

$$P_{25} = 790\ (0.65)\ (0.74)$$

$$P_{25} = 380 \text{ cubic feet per second.}$$

Following the same procedures as for the 2-year flood-peak discharge, the 25-year flood-peak discharge is 852 cubic feet per second for 25 percent impervious cover, and 970 cubic feet per second for 50 percent impervious cover.

Equation 8 gives the 100-year flood-peak discharge for undeveloped conditions of the Brushy Creek Tributary 2 basin as follows:

$$P_{100} = 1200\ A^{0.63}\ T^{-0.33}$$

$$P_{100} = 1200\ (0.55)^{0.63}\ (2.03)^{-0.33}$$

$$P_{100} = 1200\ (0.69)\ (0.79)$$

$$P_{100} = 654 \text{ cubic feet per second.}$$

Again, following the same procedures, the 100-year flood-peak discharge is 1230 cubic feet per second for 25 percent impervious cover, and 1370 cubic feet per second for the basin conditions associated with 50 percent impervious cover. The results of these calculations are summarized in the table below.

	Degree of Development (Percentage Impervious Cover)		
Flow Characteristic	None	25	50
Basin Lag time (hours)	2.03	0.30	0.22
2-year flood peak discharge (cfs)	85	269	323
25-year flood peak discharge (cfs)	380	852	970
100-year flood peak discharge (cfs)	654	1230	1370

These results show that urbanization has a significant effect on basin lag time. This basin, progressing from an undeveloped condition to that of 50 percent impervious cover, has a nine-fold decrease in lag time. This decrease in lag time has the effect of getting the bulk of runoff past a point on the stream in a shorter period. The rate of flow, and consequently the peak flow, is appreciably higher than for undeveloped conditions. In conjunction with this effect is the increase in the water that becomes runoff during a given rain. These factors combine to increase significantly the magnitude of a flood. For example the table shows, for Brushy Creek tributary 2, that a basin change from an undeveloped condition 50 percent impervious cover increased the 2-year flood by a factor of about four--85 to 323 cubic feet per second. For floods of the 100-year recurrence interval, the increase is not as great but is, nevertheless, extremely significant. The table shows a two-fold increase in the peak of the 100-year flood--654 to 1370 cubic feet per second.

We can further illustrate these changes by dividing the equations for undeveloped conditions into those for developed conditions to obtain the ratio between them. For the lag time equation the ratio is as follows:

$$\frac{T_d}{T_r} = \frac{0.49\ (L_d/\sqrt{S_d})^{0.50}\ (I_d)^{-0.57}}{0.49\ (L_r/\sqrt{S_r})^{0.50}\ (I_r)^{-0.57}} ,$$

where the subscripts d and r refer to developed and rural conditions, respectively. If the change in channel length and slope resulting from urbanization are negligible then this equation may be rewritten as:

$$\frac{T_d}{T_r} = \frac{(I_d)^{-0.57}}{(I_r)^{-0.57}} = \left(\frac{I_r}{I_d}\right)^{0.57}$$

This equation shows that the ratio of lag time after development to that before development is inversely proportional to the ratio of the percentage of impervious cover (before and after) raised to the 0.57 power. Solving this equation it is found that for a basin completely covered with an impervious surface, the lag time is about one-sixteenth that of the same area under natural conditions.

For anticipated development conditions (between 25 and 50 percent impervious cover) in the cities of Piedmont North Carolina, the change in lag time is less radical. For the conditions associated with 25 and 50 percent impervious cover, the lag time is about one-seventh and one-tenth, respectively, that for the same basin in an undeveloped condition.

Because basin lag time is an index of the hydraulic efficiency of a watershed, it follows that changes in flood-peak discharge are proportional to the changes in basin lag time. These changes can be evaluated by placing the discharge equations in a ratio form similar to the one for basin lag time. For the 2-year flood discharge (equation 2) the ratio form would be as follows:

$$\frac{P_d}{P_r} = \frac{221\ A^{0.87}\ T_d^{-0.60}}{221\ A^{0.87}\ T_r^{-0.60}} = \left(\frac{T_r}{T_d}\right)^{0.60} ,$$

where the subscripts d and r refer to developed and rural conditions, respectively. For a very small watershed that could conceivably be completely covered with an impervious surface, the flood-peak discharge would be increased by a factor of 4.84 for the 2-year flood, 3.02 for the 25-year flood, and 2.38 for the 100-year flood.

For the anticipated development conditions in the Piedmont of North Carolina, the flood-peak discharge will be increased by a factor that ranges from 1.8 to 3.8, depending upon the recurrence interval of the flood and the degree of urban development. For conditions associated with 25 percent impervious cover the flood-peak discharge is increased by a factor of 3.0 for the 2-year flood, 2.2 for the 25-year flood, and 1.8 for the 100-year flood. For conditions associated with 50 percent impervious cover the flood-peak discharge is increased by a factor of 3.8 for the 2-year flood, 2.5 for the 25-year flood, and 2.1 for the 100-year flood.

APPLICABILITY OF RELATIONS

Although the relations in this report are empirical and may be improved as more data are obtained, we feel that they have much merit by providing for simple techniques of obtaining design discharges that are based on the most recent hydrologic data and analytical concepts. The equations in this report were defined on the basis of rainfall-runoff data observed primarily in the Piedmont province of North Carolina. We are in no way suggesting that the individual equations are applicable in areas other than the Piedmont of North Carolina. The methods of data analysis, however, are general and should be applicable to other areas where the major floods result from rainfall and where estimates are desired in open channels.

The method of prediction is dependent on two relationships. One relates the average basin lag time to the length of the main watercourse, the slope of the main watercourse, and the ratio of the area of impervious cover to the total drainage area. The other relates the flood-peak discharge to the size of drainage area and the average basin lag time. The combination of these two relationships provides a means of determining a flood-peak discharge for any location on a stream, gaged or ungaged, rural or urban. We recommend using the relations for watersheds where an estimate of urban flooding is needed. The estimating relations are limited to providing flood discharge estimates at open-channel sites in the Piedmont province of North Carolina where the runoff is unaffected by artificial storage or diversion. The estimates are most reliable for smaller size floods where the drainage area ranges between 0.3 and 150 square miles, where the $L/\sqrt{S}$ ratio ranges between 0.1 and 9.0, and where impervious cover of less than 30 percent is uniformly distributed over the basin.

It should be noted that multiple regression results in empirical equations that do not provide answers that would agree exactly with the data collected at all streams locations. These equations serve to sum up all the evidence of a large number of observations in a single statement that expresses in condensed form the extent to which average differences in the dependent variable tend to be associated with the average differences in each of the independent variables. While the equations appear deceptively simple--and they are, of course, simple to use--they incorporate rather involved hydrologic techniques that cannot be feasibly applied to the planning and construction in every individual watershed. Because regression analyses define equations that are most accurate for the average of the sample data, flood-peak discharge estimates should be most accurate for basins with characteristics near the average of the sample basins.

USE OF RELATIONS

The lag time and flood-peak discharge equations have been converted to the nomographs shown in figures 18-26. These nomographs are presented for the convenience offered by a graphical solution to the equations resulting from this study. Also, on the nomograph scale representing the natural basin characteristics, two sets of measurement units are provided for the user who may be more accustomed to one set of units than the other. One may enter the $L/\sqrt{S}$ scale on the nomograph with the stream length in miles and the slope in feet per mile or with the stream length in thousands of feet and the slope in feet per foot. The drainage area can be in either square miles or acres.

As with the mathematical equations, four variables are needed to use the nomographs. The first three are natural basin characteristics consisting of drainage area, stream length, and channel slope, all of which may be measured from topographic maps. The fourth variable is the ratio of the area of man-made impervious cover to the total drainage area, which may be determined either by a basin inspection, by measurements on up-to-date areal photographs, or by measurements on large scale maps. In actual practice, it may be preferable to assume some level of future development rather than determining the level in existence at the time. In doing this, however, it is necessary to consider whether the future development will include changes in drainage area, stream length, and channel slope. If such changes are likely to occur they must be accounted for in the computations by using the anticipated values of area, length, and slope rather than the existing values. Correcting for these changes is important where large amounts of earth are being moved and where significant channel straightening and dredging is being done.

The following example illustrates the application of the nomographs in figures 18 through 26. Suppose one desires to estimate the 20-year flood-peak discharge for Little Hope Creek at Seneca Place, Charlotte. The projected future development consists of 40 percent impervious cover, with the associated improvements in the hydraulic efficiency of the drainage system. From a detailed topographic map or plans showing probable future development, measure:

A = 2.72 square miles.
I = 40 (given above).
L = 2.66 miles from the site to the rim of the drainage basin.
  = 0.27 miles from the site to a point that is 10 percent of the distance to the rim of the drainage basin.
  = 2.26 miles from the site to a point that is 85 percent of the distance to the rim of the drainage basin.

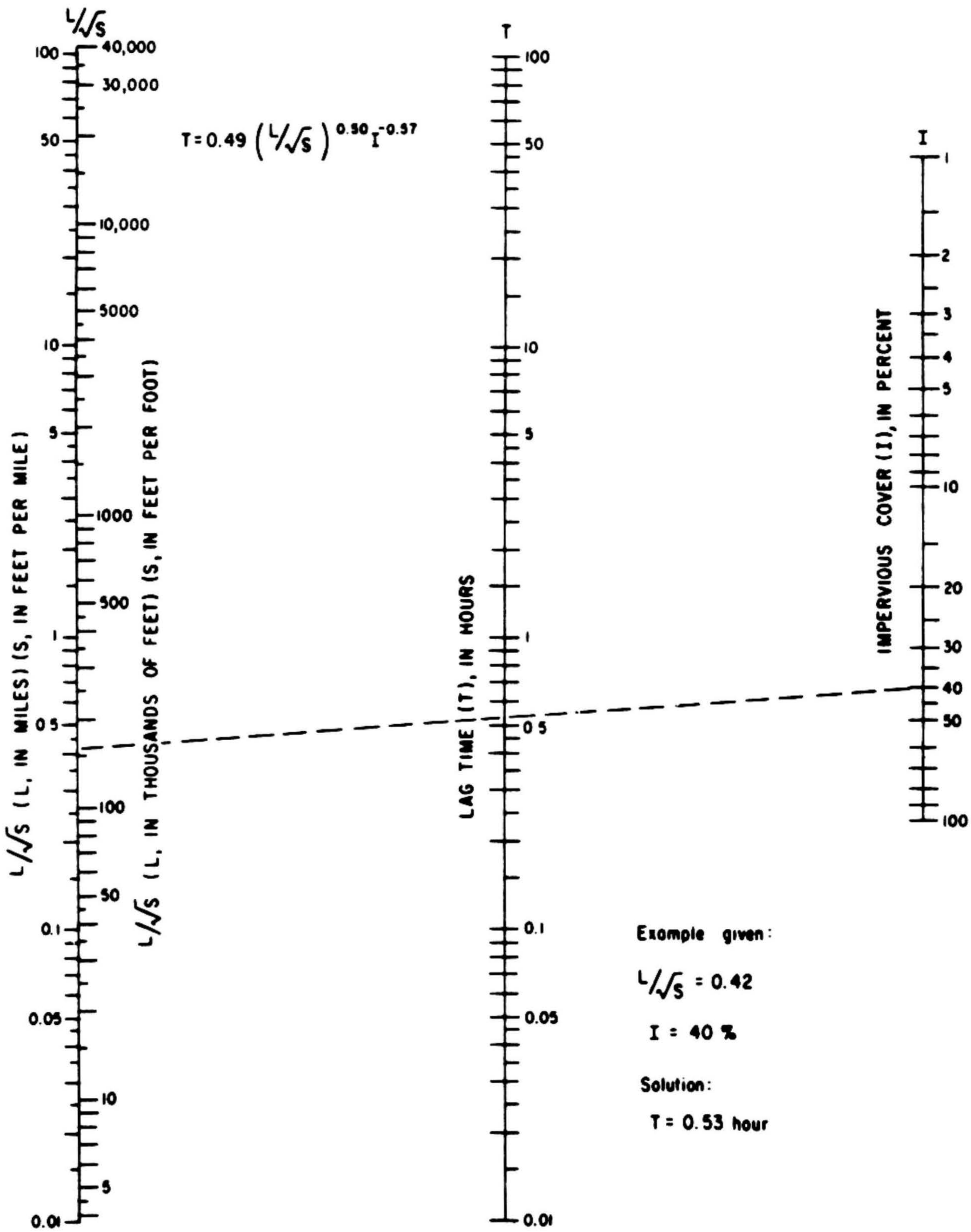


Figure 18.--Nomograph for estimating basin lag time.

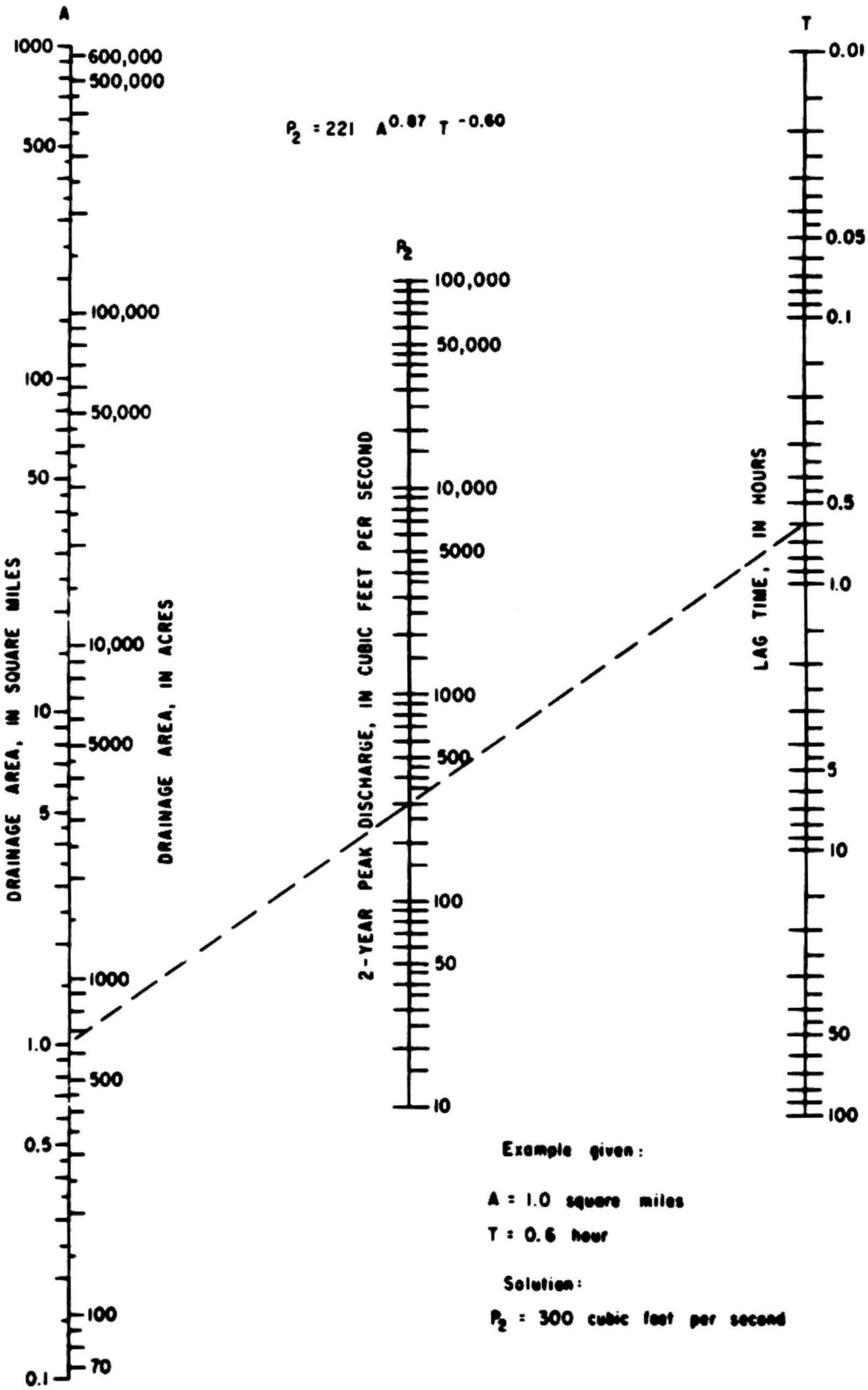


Figure 19.--Nomograph for estimating 2-year flood-peak discharge.

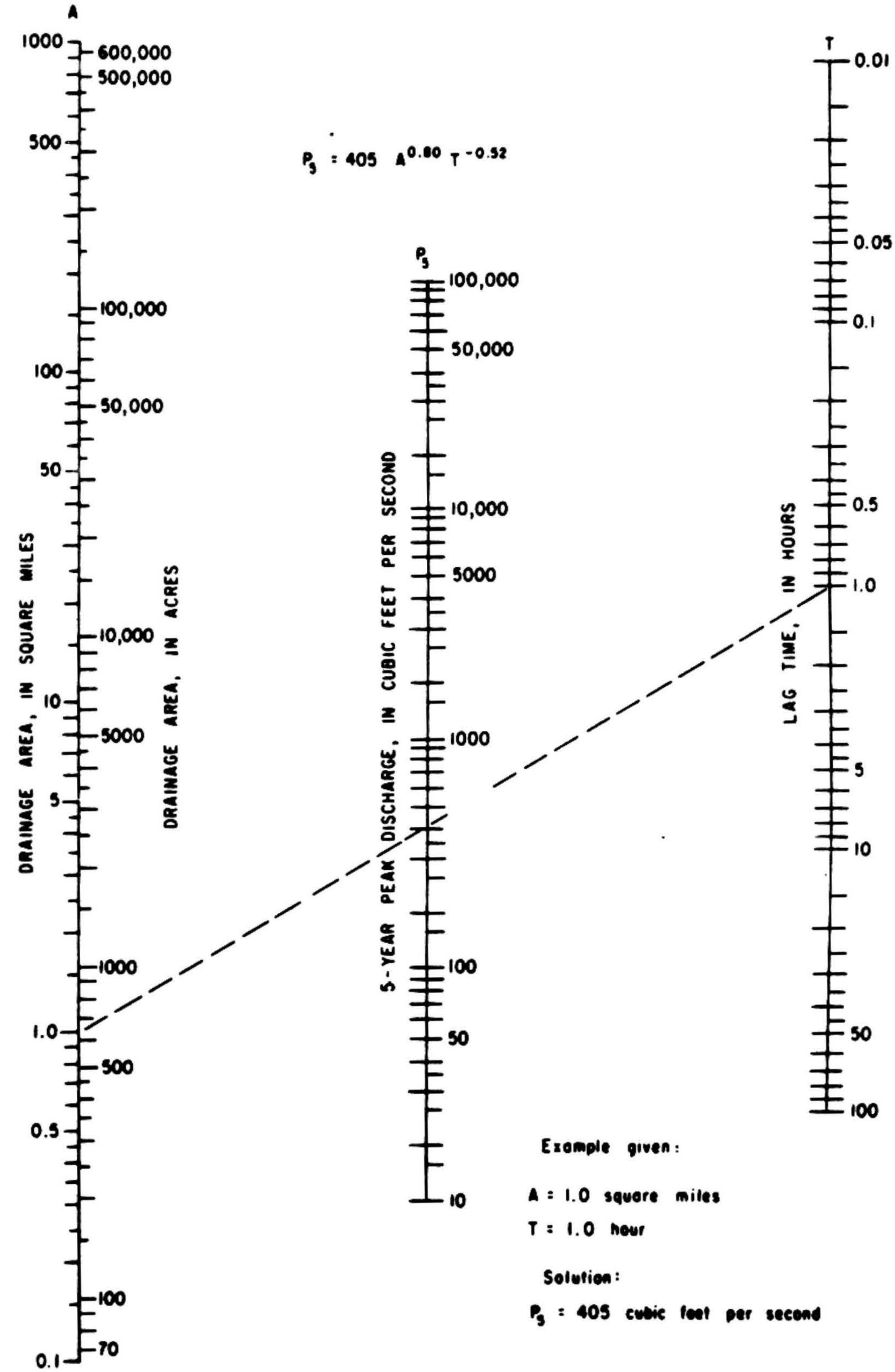


Figure 20.--Nomograph for estimating 5-year flood-peak discharge.

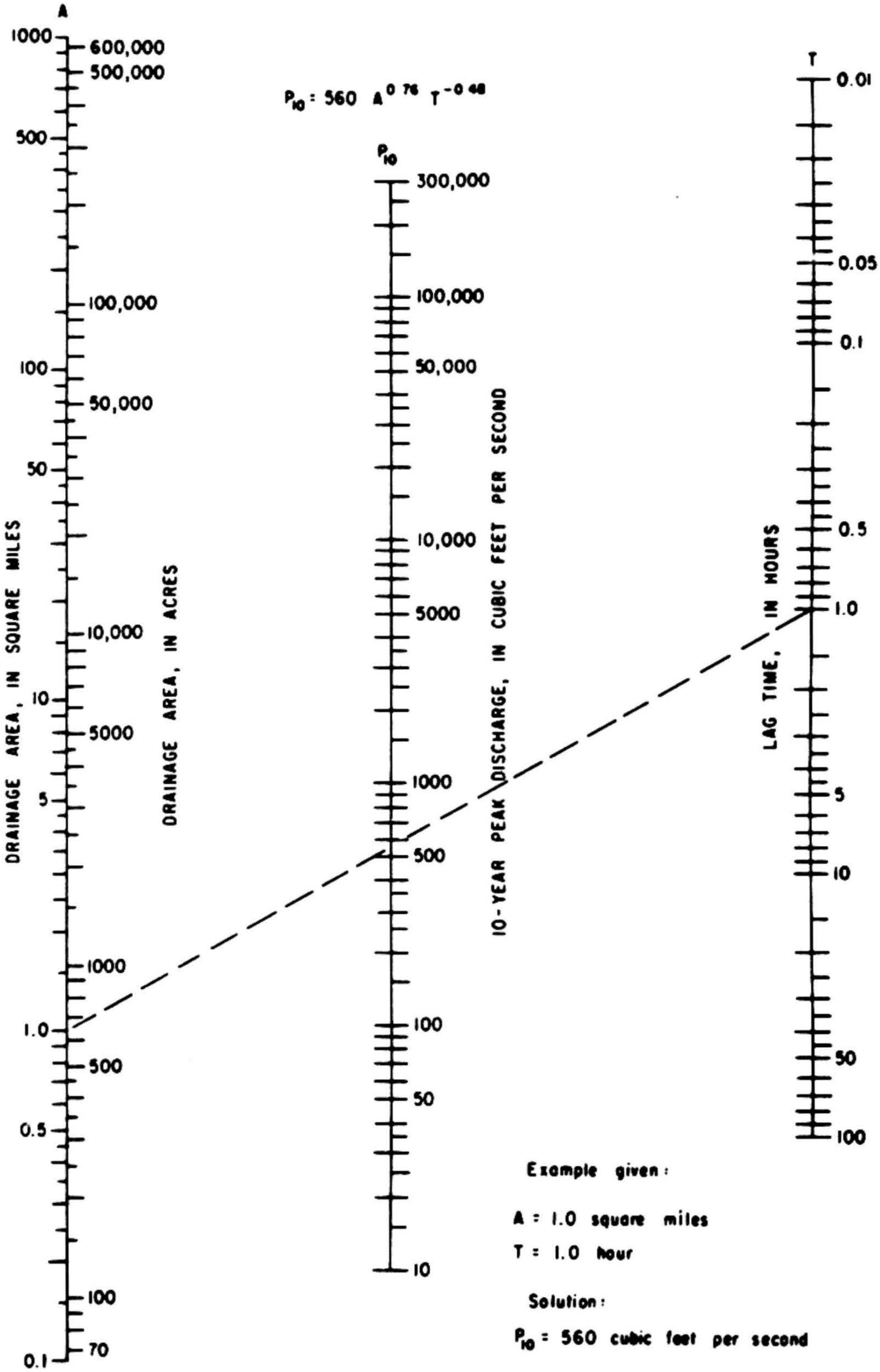


Figure 21.--Nomograph for estimating 10-year flood-peak discharge.

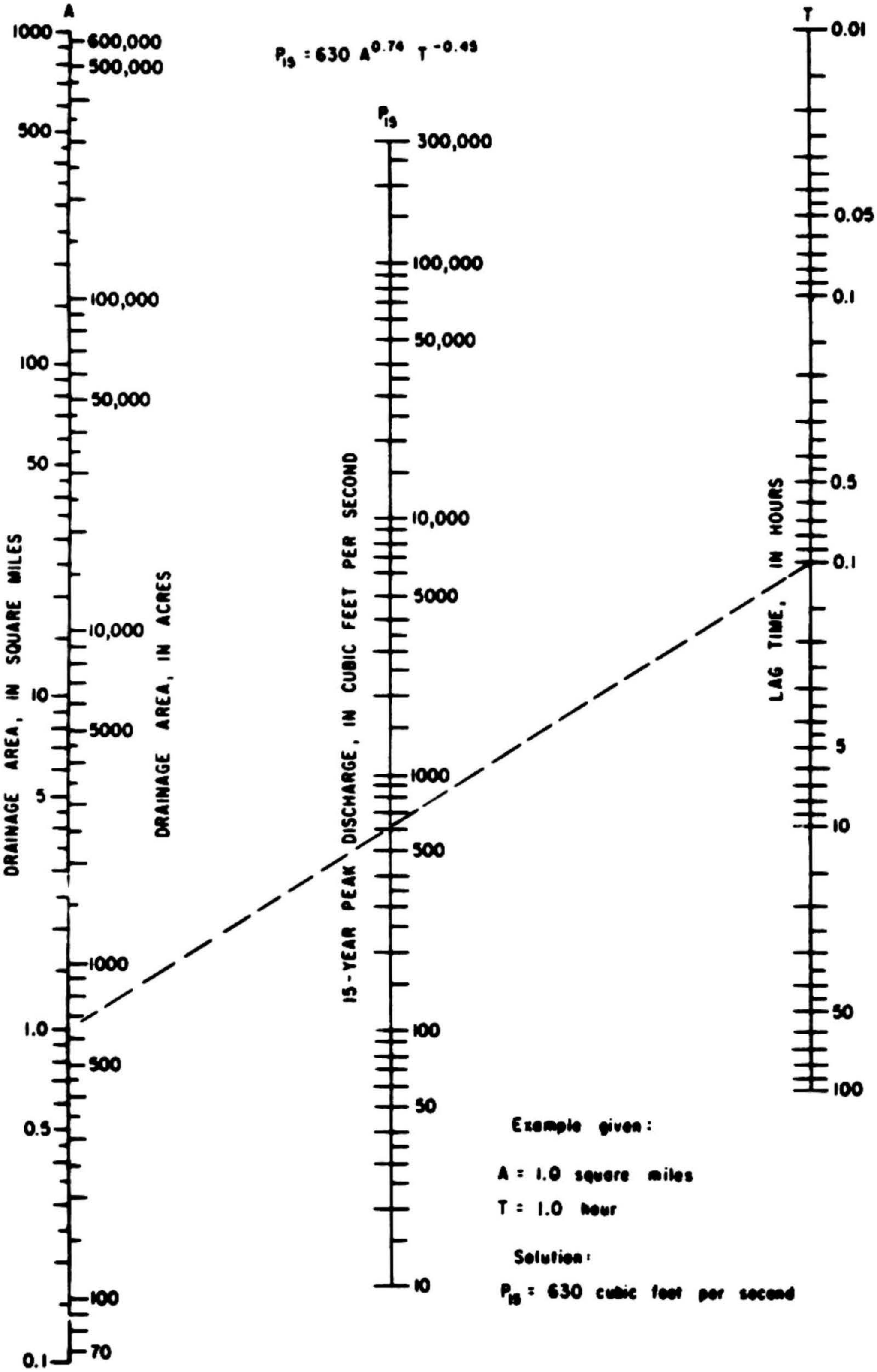


Figure 22.--Nomograph for estimating 15-year flood-peak discharge.

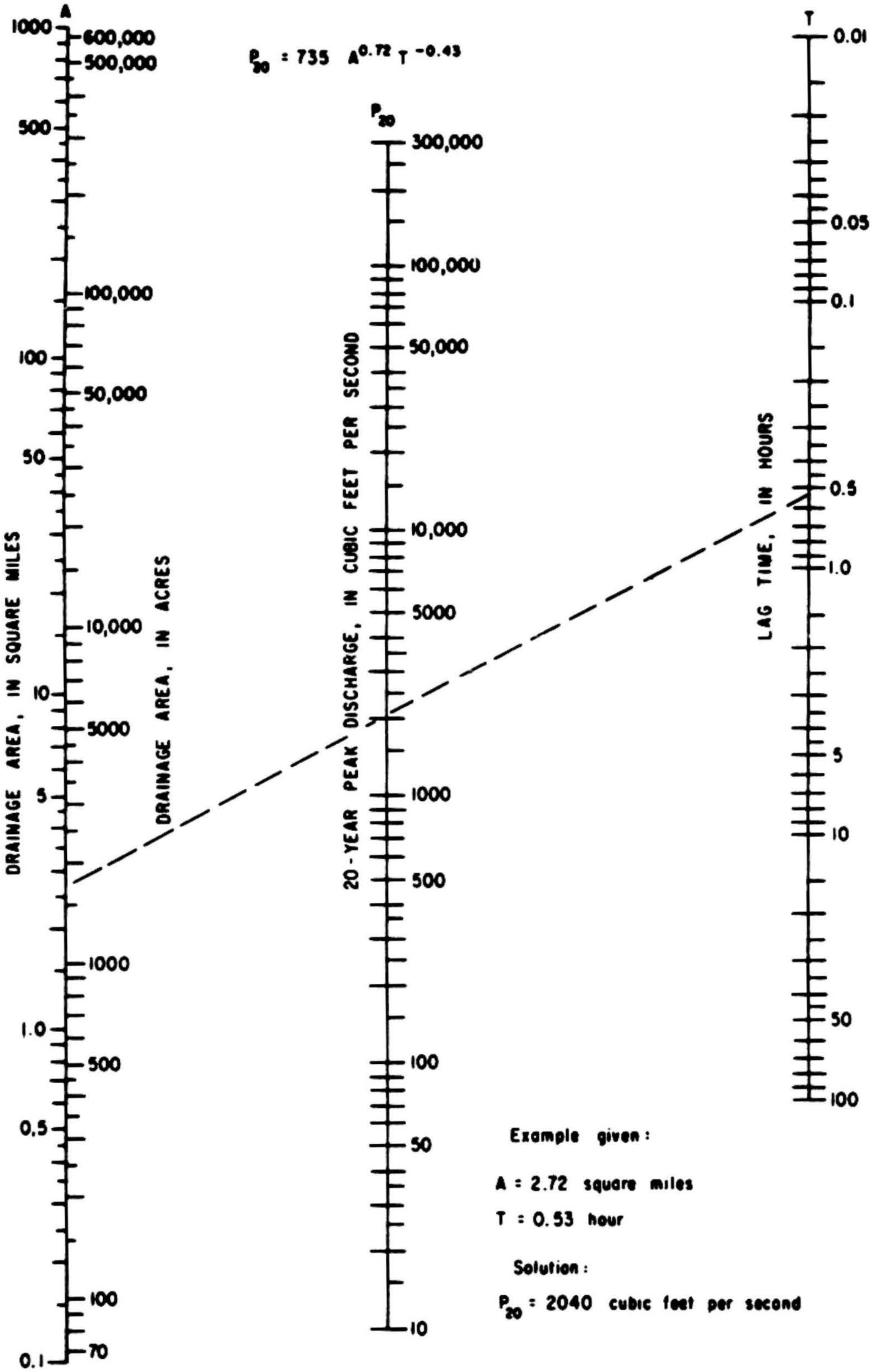


Figure 23.--Nomograph for estimating 20-year flood-peak discharge.

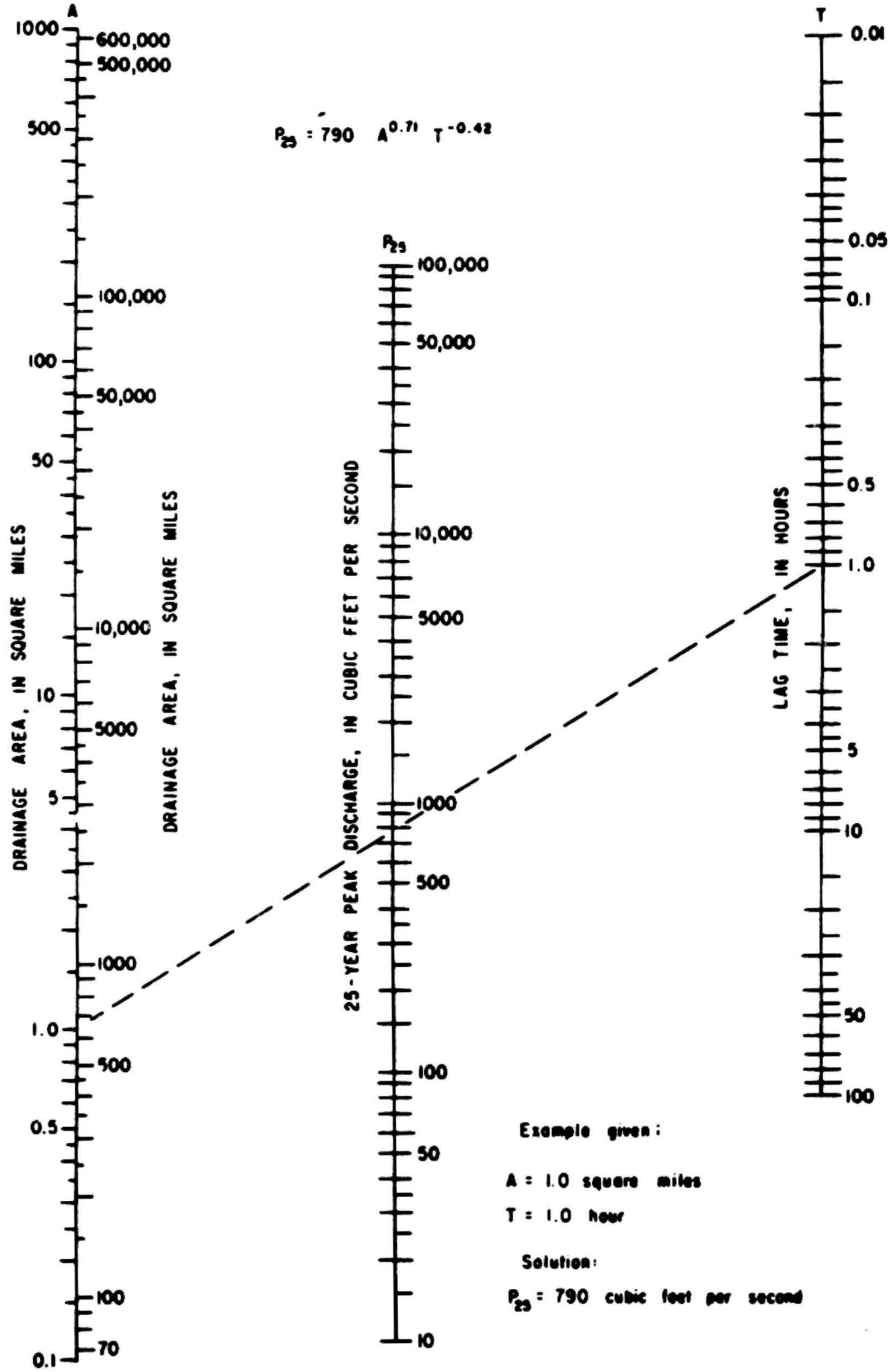


Figure 24.--Nomograph for estimating 25-year flood-peak discharge.

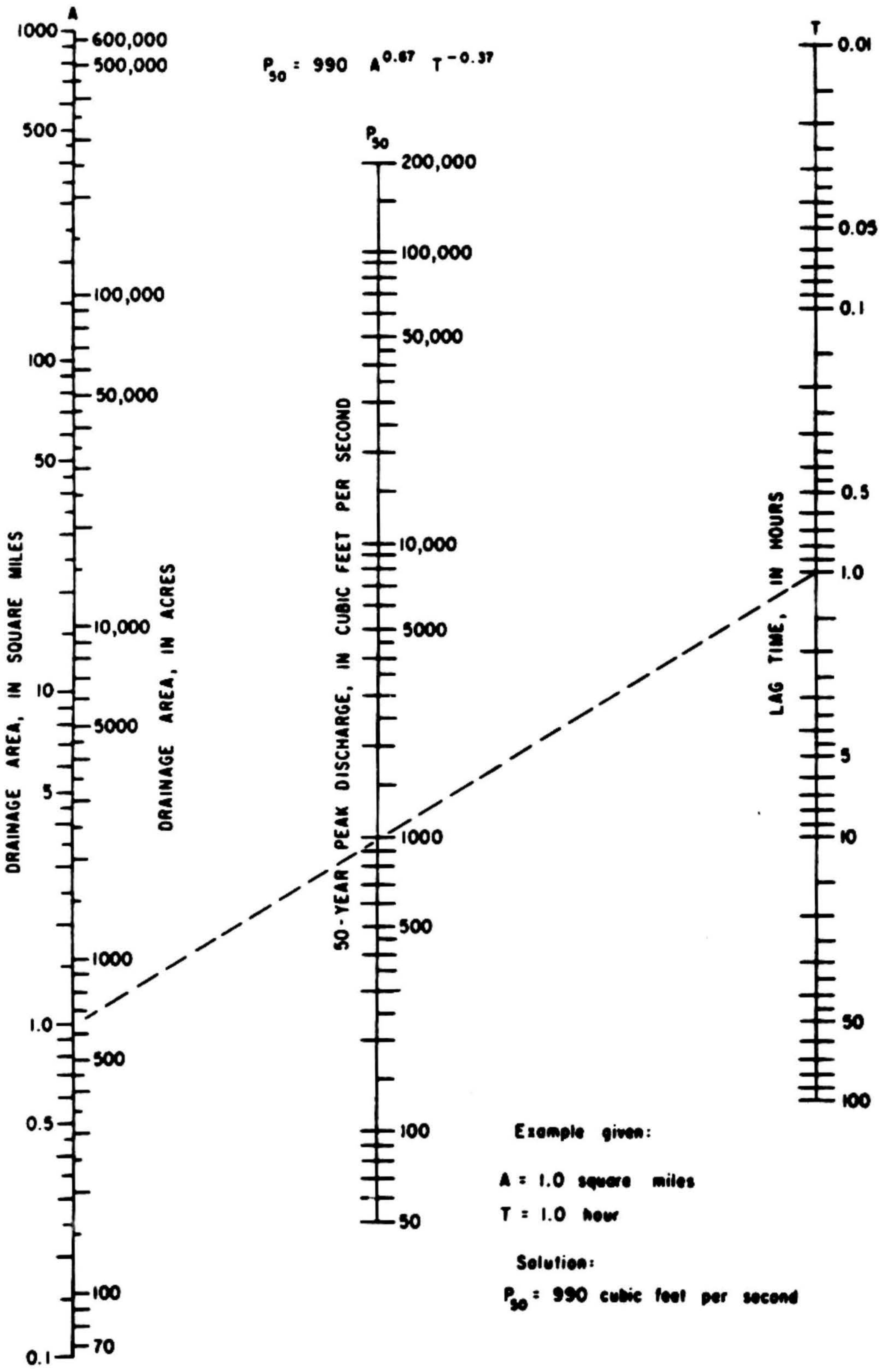


Figure 25.--Nomograph for estimating 50-year flood-peak discharge.

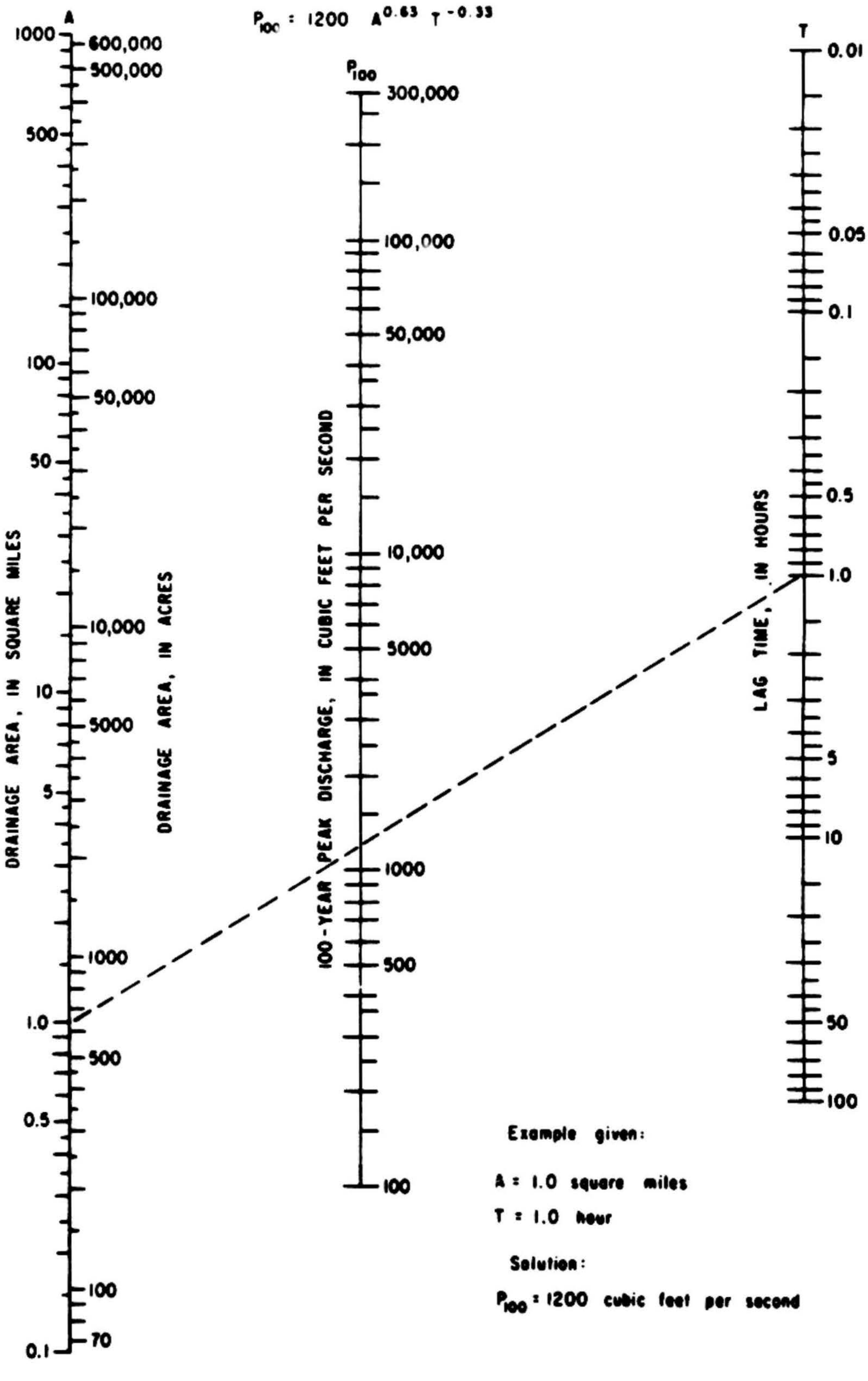


Figure 26.--Nomograph for estimating 100-year flood-peak discharge.

Elevation = 605 feet at the point that is 10 percent of the distance to the rim of the drainage basin.

= 686 feet at the point that is 85 percent of the distance to the rim of the drainage basin.

Compute slope:

$$S = \frac{686-605}{2.26-0.27} = \frac{81}{1.99} = 41.0 \text{ feet per mile}$$

Compute length-slope factors:

$$L/\sqrt{S} = 2.66/\sqrt{41.0} = 0.42$$

Determine lag time by nomograph from figure 18, plot the value of impervious cover, I = 40, on the scale at the right; then plot the value of the length-slope factor, $L/\sqrt{S} = 0.42$, on the scale at the left. Connect these two points with a straight line and read the lag time value, T = 0.53 hours, on the center scale.

Determine 20-year flood-peak discharge by nomograph from figure 23, plot the value of lag time, T = 0.53 hours, on the scale at the right; then plot the value of drainage area, A = 2.72 square miles, on the scale at the left. Connect these two points with a straight line and read the 20-year flood-peak discharge value, P_{20} = 2040 cubic feet per second, on the center scale.

Summary

This report presents equations for estimating the discharge for floods having recurrence intervals up to 100 years for drainage basins in various degrees of urban or suburban development in the Piedmont province of North Carolina. The user of these equations must determine the drainage area, stream length, and stream slope in order to apply the relations. Also information is required on the actual or estimated extent of impervious cover. In actual practice any part or all of the needed information about the basin characteristics may be available from plans showing probable future development.

The equations were developed from information available on the Piedmont area and pertinent data collected from other areas. They are applicable to basins in the Piedmont area of North Carolina. Although others may refine the equations on the basis of information subsequently obtained, they are presently the best method available for predicting flood-peak discharge for ungaged sites in the study area.

Urban development greatly changes the basin lag time. For the streams studied for this report, the lag time for an urban basin having an impervious cover of about 25 percent is one-seventh that of a comparable natural watershed, whereas the lag time for a basin completely covered with impervious surfaces is about one-sixteenth that of the same area under natural (undeveloped) conditions.

This study indicates that urban development significantly changes flood magnitudes. Urban development, including the provision of a hydraulically more efficient drainage system associated with 50 percent impervious cover, may increase the peak discharge by a factor of about 2.5 for a flood having a recurrence interval of 25-years and about 2.1 for a 100-year flood. For the type of urban development expected in the Piedmont area of North Carolina, the peak discharge can be expected to increase by a factor of about two to four depending upon the recurrence interval of the flood and the anticipated conditions of development.

REFERENCES

Anderson, D. G., 1970, Effects of urban development on floods in Northern Virginia: U.S. Geol. Survey Water-Supply Paper 2001-C, 22 p.

Ardis, C. V., Dueker, K, J., and Lenz, A. T., 1969, Storm drainage practices of thirty-two cities: Proc. Am. Soc. Civil Engineers, Jour. Hydraulic Div., v. 95, no. HY 1, p. 383-408.

Benson, M. A., 1959, Channel-slope factor in flood-frequency analysis: Am. Soc. Civil Engineers Proc., v. 85, no. HY 4, 9 p.

______1962, Factors influencing the occurrence of floods in a humid region of diverse terrain: U.S. Geol. Survey Water-Supply Paper 1580-B, 64 p.

Carter, R. W., 1961, Magnitude and frequency of floods in suburban areas, in short papers in the geologic and hydrologic sciences: U.S. Geol. Survey Prof. Paper 424-B, p. B9-B14.

Dawdy, D. R., Lichty, R. W., and Bergmann, J. M., 1970, A rainfall-runoff model for estimation of flood peaks for small drainage basins--A progress report: U.S. Geol. Survey open-file rept.

Eagleson, P. S., and March, Frederic, 1965, Approaches to the linear synthesis of urban runoff systems: Hydrodynamic Laboratory Rept. no. 85, M.I.T., Cambridge, Mass.

Espey, W. H., Jr., Morgan, C. W., and Masch, F. D., 1966, A study of some effects of urbanization on storm runoff from a small watershed: Rept. no. 23, Texas Water Devel. Board, Austin, Texas.

Hinson, H. G., 1965, Floods on small streams in North Carolina, probable magnitude and frequency: U.S. Geol. Survey Circ. 517, 7 p.

Langbein, W. B., 1949, Annual floods and the partial-duration flood series: Am. Geophys. Union Trans., v. 30, p. 879-881.

Langbein, W. B., and Iseri, K. T., 1960, General introduction and hydrologic definitions: U.S. Geol. Survey Water-Supply Paper 1541-A.

Linsley, R. K., Kohler, M.A., and Paulhus, J. L., 1958, Hydrology for engineers: New York, McGraw-Hill Book Co., Inc.

Martens, L. A., 1968, Flood inundation and effects of urbainzation in metropolitan Charlotte, North Carolina: U.S. Geol. Survey Water-Supply Paper 1591-C, 60 p.

Sarma, P. B. S., Delleur, J. W., and Rao, A. R., 1969, A program in urban hydrology: Part II. An evaluation of rainfall-runoff models for small urbanized watersheds and the effect of urbanization on runoff: Tech. Rept. no. 9, Water Resources Research Center, Purdue Univ., Lafayette, Ind., 240 p.

Schaake, J. C., Jr., 1965, Storm drainage research project: Tech. Rept. no. 3, The Johns Hopkins Univ., Baltimore, Md.

Sherman, L. K., 1940, Derivation of infiltration-capacity (f) from average loss-rates (fav): Am. Geophys. Union, v. 21, pt. 2, p. 541-550.

Snyder, F. F., 1958, Synthetic flood frequency: Proc. Am. Soc. Civil Engineers, Jour. Hydraulic Div., v. 84, no. HY 5, p. 1808-1-1808-22.

Speer, P. R., and Gamble, C. R., 1964, Magnitude and frequency of floods in the United States, Pt. 2-A. South Atlantic slope basins, James River to Savannah River: U.S. Geol. Survey Water-Supply Paper 1673, 329 p.

Stuckey, J. L., 1965, North Carolina: Its geology and mineral resources: Raleigh, N. C. State Univ. Print Shop, p. 16-19.

U.S. Water Resources Council, 1967, A uniform technique for determining flood-flow frequencies: Water Resources Council Bull. 15, 15 p.

Viessman, Warren, Jr., 1966A, The hydrology of small impervious areas: Water Resources Research, v. 2. no. 3, p. 405-412.

_____1966B, Complexities of runoff prediction for urban drainage areas: Tech. Paper no. 1, Water Resources Center, Orono, Me., Univ. of Maine.

_____1968, Runoff estimation for very small drainage areas: Water Resources Research, v. 4, no. 1, p. 87-93.

Table 1.--Stream-gaging and rainfall stations in Charlotte, North Carolina

U. S. Geological Survey station number	Stream and location	Area (sq mi)	Land use	Percent of impervious cover	Period of record	Type of record
2-1429.50	Paw Creek tributary 2 at Allenbrook Drive, Charlotte, N. C.	.62	Residential	18	1966-70	Flood hydrograph and rainfall
2-1462.35	Irwin Creek tributary at Charlotte, N. C.	.27	Residential	19	1966-70	Flood hydrograph and rainfall
2-1462.80	Stewart Creek at Charlotte, N. C.	9.40	Residential and commercial	8	1962-70	Crest stage
2-1463.00	Irwin Creek near Charlotte, N. C.	30.5	Residential, business, and commercial	11	1962-70	Continuous record streamflow and rainfall
2-1463.30	Sugar Creek near Charlotte, N. C.	43.7	Mixed urban and rural	9	1962-70	Crest stage
2-1464.09	Little Sugar Creek at Brunswick Avenue, Charlotte, N. C.	12.2	Residential and business	25	1964-66	Flood hydrograph and rainfall
2-1464.20	Little Sugar Creek at Hillside Avenue, Charlotte, N. C.	15.4	Residential and business	22	1962-70	Flood hydrograph
2-1464.35	Briar Creek tributary 6 at Sudbury Road, Charlotte, N. C.	.56	Residential	16	1966-70	Flood hydrograph and rainfall
2-1464.36	Briar Creek tributary 7 at Shamrock Drive Charlotte, N. C.	.52	Residential	20	1966-70	Flood hydrograph and rainfall
2-1464.40	Briar Creek at East Seventh Street, Charlotte, N. C.	14.5	Residential, business, and commercial	8	1962-70	Crest stage
2-1464.50	Briar Creek at Sharon Road, Charlotte, N. C.	18.5	Residential, business, and commercial	10	1962-70	Continuous record streamflow and rainfall

Table 1.--Stream-gaging and rainfall stations in Charlotte, North Carolina--Continued

U. S. Geological Survey station number	Stream and location	Area (sq mi)	Land use	Percent of impervious cover	Period of record	Type of record
2-1464.70	Little Hope Creek at Seneca Place, Charlotte, N. C.	2.72	Residential and commercial	15	1966-70	Flood hydrograph and rainfall
2-1465.00	Little Sugar Creek near Charlotte, N. C.	41.0	Residential, business, and commercial	15	1924-70	Continuous record streamflow and rainfall
2-1465.05	Little Sugar Creek tributary 7 at Burnley Road, Charlotte, N. C.	.44	Residential	14	1966-70	Flood hydrograph and rainfall
2-1465.10	Little Sugar Creek tributary 6 at Brookcrest Drive, Charlotte, N. C.	.84	Residential	21	1966-70	Flood hydrograph and rainfall
2-1465.30	Little Sugar Creek at Pineville, N. C.	48.7	----------------	-----------	1965-70	Crest stage
2-1466.00	McAlpine Creek at Sardis Road near Charlotte, N. C.	38.3	Residential	2	1962-70	Continuous record streamflow and rainfall
2-1466.55	McAlpine Creek at N. C. Highway 51 near Charlotte, N. C.	51.5	Residential	1	1962-70	Crest stage
2-1467.00	McMullen Creek at Sharon View Road near Charlotte, N. C.	6.98	Residential	6	1962-70	Continuous record streamflow and rainfall
2-1467.25	McMullen Creek near Griffith, N. C.	13.0	Residential	4	1962-70	Crest stage
---------	City Hall rain gage, Charlotte, N. C.	-------	----------------	-----------	1963-70	Rainfall
---------	Methodist Home rain gage, Charlotte, N. C.	-------	----------------	-----------	1963-70	Rainfall
---------	Vest Station rain gage, Charlotte, N. C.	-------	----------------	-----------	1963-70	Rainfall

Table 1.--Stream-gaging and rainfall stations in Charlotte, North Carolina--Continued

U. S. Geological Survey station number	Stream and location	Area (sq mi)	Land use	Percent of impervious cover	Period of record	Type of record
---------	Douglas Airport Weather Bureau rain gage, Charlotte, N. C.	------	-----------------	-----------	1901-70	Rainfall

Table 2.--Stream-gaging and rainfall stations in Durham, North Carolina

U. S. Geological Survey station number	Stream and location	Area (sq mi)	Land use	Percent of impervious cover	Period of record	Type of record
2-0850.55	Warren Creek near Mill Grove, N. C.	2.47	----------------	-----------	1967-70	Crest stage
2-0867.20	Ellerbe Creek at Hillandale Road at Durham, N. C.	2.86	Residential	5	1967-70	Crest stage
2-0867.60	Dye Creek at Guess Road, Durham, N. C.	.81	Residential and business	32	1967-70	Flood hydrograph and rainfall
2-0867.90	Goose Creek at East Geer Street, Durham, N. C.	1.48	Residential and business	25	1967-70	Flood hydrograph and rainfall
2-0868.24	Ellerbe Creek below Goose Creek near Durham, N. C.	17.0	----------------	-----------	1967-70	Crest stage
2-0972.30	Sandy Creek at Picket Road, Durham, N. C.	5.81	Residential and commercial	8	1967-70	Crest stage
2-0972.40	Third Fork Creek tributary at University Drive, Durham, N. C.	.52	Residential and business	20	1967-70	Flood hydrograph and rainfall
2-0972.43	Third Fork Creek at Durham, N. C.	1.67	Residential, business, and manufacturing	-----------	1967-70	Continuous record streamflow and rainfall
2-0972.45	Third Fork Creek at Roxboro Street, Durham, N. C.	2.09	----------------	-----------	1967-70	Crest stage
2-0972.50	Rocky Creek tributary at N. C. Highway 55, Durham, N. C.	.45	Residential, business, and manufacturing	26	1967-70	Flood hydrograph and rainfall
2-0972.55	Rocky Creek at Fayetteville Street, Durham, N. C.	3.60	Residential business, and manufacturing	12	1967-70	Crest stage
2-0974.10	Crooked Creek near Lowes Grove, N. C.	1.82	Residential	1	1967-70	Flood hydrograph and rainfall
---------	Blue Cross-Blue Shield Parking Lot rain gage, Durham, N. C.	--------	----------------	-----------	1968-70	Rainfall

Table 3.--Stream-gaging and rainfall stations in Lenoir, North Carolina

U. S. Geological Survey station number	Stream and location	Area (sq mi)	Land use	Percent of impervious cover	Period of record	Type of record
2-1411.30	Zacks Fork Creek near Lenoir, N. C.	9.14	-----------------	-----------	1966-70	Crest stage
2-1411.50	Lower Creek at Mulberry Street, Lenoir, N. C.	31.8	Mixed rural and urban	13	1966-70	Continuous record streamflow and rainfall
2-1411.54	Lower Creek at Virginia Street, Lenoir, N. C.	34.5	-----------------	-----------	1966-70	Crest stage
2-1411.80	Blair Fork near Lenoir, N. C.	5.83	-----------------	-----------	1966-70	Crest stage
2-1411.90	Greasy Creek at Lenoir, N. C.	4.40	Rural residential	2	1966-70	Flood hydrograph and rainfall

Table 4.--Stream-gaging and rainfall stations in Morganton, North Carolina

U. S. Geological Survey station number	Stream and location	Area (sq mi)	Land use	Percent of impervious cover	Period of record	Type of record
2-1392.00	Bailey Fork near Morganton, N. C.	7.86	Rural	1	1966-70	Continuous record streamflow and rainfall
2-1392.56	Silver Creek at Morganton, N. C.	68.6	----------------	-----------	1966-70	Crest stage
2-1396.10	Hunting Creek at Morganton, N. C.	8.26	Mixed rural and urban	3	1966-70	Flood hydrology and rainfall
2-1396.50	East Prong near Morganton, N. C.	8.94	Mixed rural and urban	2	1966-70	Continuous record streamflow and rainfall

Table 5.--Stream-gaging and rainfall stations in Winston-Salem, North Carolina

U. S. Geological Survey station number	Stream and location	Area (sq mi)	Land use	Percent of impervious cover	Period of record	Type of record
2-1157.30	Mill Creek near Stanleyville, N. C.	10.2	Rural residential	3	1964-70	Flood hydrograph and rainfall
2-1157.40	Mill Creek near Oldtown, N. C.	27.8	Rural residential	5	1964-70	Flood hydrograph and rainfall
2-1157.50	Muddy Creek near Lewisville, N. C.	82.8	Rural residential	2	1964-70	Continuous record streamflow and rainfall
2-1157.60	Silas Creek at Winston-Salem, N. C.	5.25	Residential and commercial	6	1968-70	Flood hydrograph and rainfall
2-1157.65	Silas Creek tributary at Pine Valley Road, Winston-Salem, N. C.	.89	Residential	12	1967-70	Flood hydrograph and rainfall
2-1158.00	Silas Creek at Clemmons, N. C.	11 8	Mixed urban and rural	6	1964-70	Continuous record streamflow and rainfall
2-1158.10	Little Creek near Clemmons, N. C.	6.81	Residential	7	1964-70	Flood hydrograph and rainfall
2-1158.35	Brushy Creek tributary at Winston-Salem, N. C.	1.88	Residential and commercial	20	1968-70	Flood hydrograph and rainfall
2-1158.39	Brushy Creek tributary 2 at U. S. Highway 311, Winston-Salem, N. C.	.55	Residential and business	37	1968-70	Flood hydrograph and rainfall
2-1158.40	Brushy Creek at Winston-Salem, N. C.	11.9	Residential, business, and commercial	9	1964-70	Flood hydrograph
2-1158.43	Tar Branch at Walnut Street, Winston-Salem, N. C.	.59	Residential and business	28	1967-70	Flood hydrograph and rainfall
2-1158.45	Peters Creek at Winston-Salem, N. C.	5.30	Residential, commercial, and business	20	1964-70	Flood hydrograph and rainfall

Table 5.--Stream-gaging and rainfall stations in Winston-Salem, North Carolina--Continued

U. S. Geological Survey station number	Stream and location	Area (sq mi)	Land use	Percent of impervious cover	Period of record	Type of record
2-1158.50	Salem Creek at Winston-Salem, N. C.	51.3	Mixed urban and rural	8	1964-70	Continuous record streamflow and rainfall
2-1158.51	Salem Creek tributary 2 at Cloister Drive, Winston-Salem, N. C.	.89	Residential	13	1967-70	Flood hydrograph and rainfall
2-1158.55	Burke Branch at Silas Creek Parkway, Winston-Salem, N. C.	1.24	Residential	20	1967-70	Flood hydrograph and rainfall
2-1158.60	Muddy Creek near Muddy Creek, N. C.	178	Mixed rural and urban	4	1964-70	Continuous record streamflow and rainfall
2-1158.70	Fiddlers Creek near Winston-Salem, N. C.	9.73	Rural residential	2	1964-70	Flood hydrograph and rainfall
2-1159.00	South Fork Muddy Creek near Clemmons, N. C.	42.3	Rural residential	2	1964-70	Continuous record streamflow and rainfall
---------	City Yard rain gage, Winston-Salem, N. C.	-------	----------------	-----------	1967-70	Rainfall

Table 6.--Stream-gaging stations used to define relation for estimating lag time

[Data source: USGS, U.S. Geological Survey; A, Anderson, 1970, U.S. Geol. Water-Supply Paper 2001-C]

U. S. Geological Survey station number	Stream and location	Period of record used	Data source	Basin characteristics				
				Area (sq mi)	Lag time (hours)	Percent of impervious cover	Length (miles)	Slope (feet per mile)
1-6440.00	Goose Creek near Leesburg, Va.	1909-12 1930-66	A A	338	14.7	<1	41.7	7.5
1-6443.00	Sugarland Run at Herndon, Va.	1965-66	A	3.36	4.2	<1	2.6	38.9
1-6450.00	Seneca Creek at Dawsonville, Md.	1930-66	A	101	7.0	<1	21.6	15.1
1-6457.00	South Fork Little Difficult Run near Fairfax, Va.	1967	A	1.59	3.0	<1	3.0	51.2
1-6458.00	Piney Branch at Vienna, Va.	1963-66	A	.29	.18	30	.5	86.5
1-6458.70	Calvin Run tributary at Reston, Va.	1963	A	1.06	1.9	<1	1.6	84.9
1-6459.00	Calvin Run at Reston, Va.	1961-66	A	5.09	3.4	<1	3.7	49.3
1-6459.50	Piney Run at Reston, Va.	1965-66	A	2.06	2.6	1	1.3	79.6
1-6460.00	Difficult Run near Great Falls, Va.	1935-66	A	58	9.2	1	13.2	16.0
1-6462.00	Scott Run near McLean, Va.	1961-66	A	4.69	1.6	5	4.2	54.0
1-6465.50	Little Falls Branch near Bethesda, Md.	1944-66	A	4.1	1.0	15	3.1	58.0
1-6466.00	Pimmit Run near Falls Church, Va.	1961-66	A	2.87	1.0	12	3.0	59.4
1-6467.00	Pimmit Run at Arlington, Va.	1961-66	A	8.12	3.0	12	7.2	38.7
1-6467.50	Little Pimmit Run tributary at Arlington, Va.	1962-66	A	.41	.32	28	.9	98.5

Table 6.--Stream-gaging stations used to define relation for estimating lag time--Continued

U. S. Geological Survey station number	Stream and location	Period of record used	Data source	Basin characteristics				
				Area (sq mi)	Lag time (hours)	Percent of impervious cover	Length (miles)	Slope (feet per mile)
1-6468.00	Little Pimmit Run at Arlington, Va.	1961-66	A	2.31	.37	20	2.2	77.4
1-6480.00	Rock Creek at Sherrill Drive at Washington, D. C.	1929-66	A	62.2	10.0	5	23.0	10.5
1-6495.00	North East Branch Anacostia River at Riverdale, Md.	1938-66	A	72.8	12.0	2	15.5	27.2
1-6500.50	North West Branch Anacostia River at Norwood, Md.	1966-67	A	2.45	2.3	3	2.0	46.7
1-6500.85	Nursery Run at Cloverly, Md.	1966-67	A	.35	2.0	<1	1.0	117
1-6501.90	Batchellors Run at Oakdale, Md.	1966-67	A	.47	2.2	<1	1.2	108
1-6505.00	North West Branch Anacostia River near Colesville, Md.	1924-66	A	21.3	4.8	3	7.4	19.1
1-6510.00	North West Branch Anacostia River near Hyattsville, Md.	1938-60	A	49.4	4.6	7	18.5	20.9
1-6524.00	Long Branch at Arlington, Va.	1961-66	A	.94	.50	30	2.1	81.[illegible]
1-6525.00	Fourmile Run at Alexandria, Va.	1961-66	A	14.4	1.3	20	7.8	42.5
1-6526.00	Holmes Run at Merrifield, Va.	1960-66	A	2.70	3.3	10	2.8	69.5
1-6526.10	Holmes Run near Annadale, Va.	1960-66	A	7.10	3.5	12	5.8	36.8
1-6526.20	Tripps Run at Falls Church, Va.	1960-66	A	1.78	.43	25	2.3	79.2
1-6526.45	Tripps Run tributary near Falls Church, Va.	1963-66	A	.50	.32	25	1.1	102

Table 6.--Stream-gaging stations used to define relation for estimating lag time--Continued

U. S. Geological Survey station number	Stream and location	Period of record used	Data source	Basin characteristics				
				Area (sq mi)	Lag time (hours)	Percent of impervious cover	Length (miles)	Slope (feet per mile)
1-6526.50	Tripps Run near Falls Church, Va.	1960-66	A	4.55	.78	25	4.1	52
1-6526.90	Holmes Run at Alexandria, Va.	1960-61	A	18.9	5.8	12	10.7	31.3
1-6527.10	Backlick Run at Springfield, Va.	1960-66	A	2.02	1.1	15	2.3	50.3
1-6528.10	Turkeycock Run at Alexandria, Va.	1960-64	A	2.26	1.3	8	2.8	78.2
1-6529.10	Backlick Run at Alexandria, Va.	1960-66	A	13.4	2.0	10	7.1	28.9
1-6530.00	Cameron Run at Alexandria, Va.	1955-66	A	33.7	4.1	15	11.1	30.9
1-6530.07	Pike Branch at Alexandria, Va.	1960-64	A	2.65	.78	12	2.5	75.4
1-6532.00	Penn Daw Outfall at Alexandria, Va.	1963-66	A	.82	.48	20	1.5	158
1-6535.00	Henson Creek near Oxon Hill, Md.	1948-60	A	16.7	5.4	3	8.4	23.4
1-6539.00	Accotink Creek at Fairfax, Va.	1961-66	A	6.8	2.0	10	4.7	35.9
1-6540.00	Accotink Creek near Annadale, Va.	1947-66	A	23.6	6.8	8	11.2	18.9
1-6545.00	Long Branch near Annadale, Va.	1950-57 1959-66	A	3.71	2.9	<1	4.3	44.6
1-6550.00	Accotink Creek near Accotink Station, Va.	1949-57 1960-61	A	37.0	9.4	5	17.1	14.9
1-6553.10	Rabbit Branch near Burke, Va.	1961-62 1964-66	A	3.81	3.8	<1	3.4	44.2

Table 6.--Stream-gaging stations used to define relation for estimating lag time--Continued

U. S. Geological Survey station number	Stream and location	Period of record used	Data source	Basin characteristics				
				Area (sq mi)	Lag time (hours)	Percent of impervious cover	Length (miles)	Slope (feet per mile)
1-6553.30	Sideburn Branch near Fairfax Station, Va.	1960-62	A	2.79	2.4	<1	2.7	49.3
1-6553.40	Pohick Creek tributary near Burke, Va.	1964-66	A	.34	1.4	3	.68	149
1-6553.50	Pohick Creek near Springfield, Va.	1961-66	A	15.0	5.4	<1	9.0	23.8
1-6553.60	Sangster Branch near Burke, Va.	1963-64	A	.15	1.6	<1	.60	151
1-6553.70	Middle Run near Lorton, Va.	1961-66	A	3.56	3.9	<1	4.0	44.5
1-6553.80	South Run near Lorton, Va.	1961-66	A	6.54	4.0	<1	6.7	29.3
1-6553.90	Pohick Creek at Lorton, Va.	1961-66	A	31.0	6.9	<1	14.0	24.0
1-6555.00	Cedar Run near Warrenton, Va.	1950-66	A	13.0	2.3	<1	4.4	67.5
1-6560.00	Cedar Run near Catlett, Va.	1950-66	A	93.5	10.6	<1	19.4	21.9
1-6565.00	Broad Run at Buckland, Va.	1950-66	A	50.3	6.8	<1	17.3	20.9
1-6568.00	Cub Run near Chantilly, Va.	1963-66	A	7.13	4.8	<1	3.4	20.1
1-6570.00	Bull Run near Manassas, Va.	1950-66	A	147	13.2	<1	20.8	10.0
1-6575.00	Occoquan Creek near Occoquan, Va.	1913-16 1920-23 1937-56	A	570	18.6	<1	52.0	6.5
1-6576.00	Sandy Run near Fairfax Station, Va.	1966	A	2.35	3.3	<1	2.9	62.2

Table 6.--Stream-gaging stations used to define relation for estimating lag time--Continued

U. S. Geological Survey station number	Stream and location	Period of record used	Data source	Basin characteristics				
				Area (sq mi)	Lag time (hours)	Percent of impervious cover	Length (miles)	Slope (feet per mile)
1-6578.00	Giles Run near Woodbridge, Va.	1965-66	A	4.54	2.8	3	5.5	50.1
1-6585.00	South Fork Quantico Creek near Independence Hill, Va.	1951-66	A	7.50	6.0	<1	4.9	24.7
1-6595.00	Middle Fork Chopawamsic Creek near Garrisonville, Va.	1951-57 1960-66	A	4.51	4.8	<1	4.7	43.2
---------	Grayhaven, Baltimore, Md.	1960-62	A	.036	.094	52	.25	67.7
---------	Hamilton Hills No. 1, Baltimore, Md.	1963-66	A	.0011	.065	56	.09	43.8
---------	Hamilton Hills No. 3, Baltimore, Md.	1963-66	A	.0029	.056	36	.11	44.8
---------	Hamilton Hills No. 4, Baltimore, Md.	1962-64	A	.00034	.051	96	.11	45.4
---------	Hamilton Hills No. 5, Baltimore, Md.	1962-64	A	.0027	.050	32	.08	111
---------	Montebello No. 3, Baltimore, Md.	1961	A	.0007	.039	57	.03	42.7
---------	Montebello No. 4, Baltimore, Md.	1961	A	.0008	.044	65	.07	41.7
---------	Newark 9-inch flume, Newark, Del.	1960-62	A	.0006	.040	100	.11	177
---------	Newark 12-inch flume, Newark, Del.	1960-62	A	.0048	.065	100	.17	35.9
---------	Northwood, Baltimore, Md.	1959-62	A	.074	.11	68	.39	213
---------	Swansea, Baltimore, Md.	1958-62	A	.074	.080	44	.41	215

Table 6.--Stream-gaging stations used to define relation for estimating lag time--Continued

U. S. Geological Survey station number	Stream and location	Period of record used	Data source	Basin characteristics				
				Area (sq mi)	Lag time (hours)	Percent of impervious cover	Length (miles)	Slope (feet per mile)
---------	Uplands, Baltimore, Md.	1951-62	A	.047	.12	52	.42	166
---------	Walker Avenue, Baltimore, Md.	1951-62	A	.24	.19	33	1.04	83.3
---------	Yorkwood South, Baltimore, Md.	1958-62	A	.017	.080	41	.20	150
2-0860.00	Dial Creek near Bahama, N. C.	1925-68	USGS	4.71	7.02	<1	5.06	30.6
2-0867.60	Dye Creek at Guess Road, Durham, N. C.	1967-70	USGS	.81	.59	32	1.70	48.0
2-0873.49	Rocky Branch at Dan Allen Drive, Raleigh, N. C.	1965-68	USGS	.56	.60	9	1.48	81.0
2-0873.50	Rocky Branch at Carmichael Gymnasium, Raleigh, N. C.	1965-68	USGS	.78	.66	14	1.72	75.0
2-0940.00	Horsepen Creek at Battle Ground, N. C.	1925-31 1934-59	USGS	15.9	9.44	<1	7.32	15.5
2-0950.00	South Buffalo Creek near Greensboro, N. C.	1928-58	USGS	33.6	23.7	<1	9.59	15.2
2-0955.00	North Buffalo Creek near Greensboro, N. C.	1928-68	USGS	37.0	8.87	5	15.3	9.92
2-0972.40	Third Fork Creek at University Drive, Durham, N. C.	1964-69	USGS	.52	.47	20	1.48	72.0
2-0974.10	Crooked Creek near Lowes Grove, N. C.	1967-69	USGS	1.82	3.99	<1	2.60	33.0
2-0990.00	East Fork Deep River near High Point, N. C.	1928-69	USGS	14.7	3.82	2	6.36	21.0

Table 6.--Stream-gaging stations used to define relation for estimating lag time--Continued

U. S. Geological Survey station number	Stream and location	Period of record used	Data source	Basin characteristics				
				Area (sq mi)	Lag time (hours)	Percent of impervious cover	Length (miles)	Slope (feet per mile)
2-1157.30	Mill Creek near Stanleyville, N. C.	1964-69	USGS	10.2	3.5	3	6.70	25.7
2-1157.40	Mill Creek near Oldtown, N. C.	1964-69	USGS	27.8	5.64	5	10.7	11.1
2-1157.50	Muddy Creek near Lewisville, N. C.	1964-69	USGS	82.8	10.0	2	22.2	12.1
2-1157.60	Silas Creek at Winston-Salem, N. C.	1964-69	USGS	5.25	3.34	6	4.50	30.4
2-1157.65	Silas Creek tributary at Pine Valley Road, Winston-Salem, N. C.	1967-69	USGS	.89	.45	12	1.62	88.0
2-1158.00	Silas Creek, at Clemmons, N. C.	1964-69	USGS	11.8	4.00	6	10.6	28.4
2-1158.10	Little Creek near Clemmons, N. C.	1964-69	USGS	6.81	2.50	7	6.70	30.5
2-1158.40	Brushy Creek at Winston-Salem, N. C.	1964-69	USGS	11.9	1.50	9	4.07	45.5
2-1158.43	Tar Branch at Walnut Street, Winston-Salem, N. C.	1968-69	USGS	.59	.25	28	1.27	156
2-1158.45	Peters Creek at Winston-Salem, N. C.	1964-69	USGS	5.30	.76	20	4.39	45.9
2-1158.50	Salem Creek at Winston-Salem, N. C.	1964-69	USGS	51.3	3.00	8	13.5	13.0
2-1158.60	Muddy Creek near Muddy Creek, N. C.	1964-69	USGS	178	11.0	4	26.6	9.27
2-1158.70	Fiddlers Creek near Winston-Salem, N. C.	1964-69	USGS	9.73	5.24	2	7.16	20.7
2-1159.00	South Fork Muddy Creek near Clemmons, N. C.	1964-69	USGS	42.2	8.00	2	12.2	13.1

Table 6.--Stream-gaging stations used to define relation for estimating lag time--Continued

U. S. Geological Survey station number	Stream and location	Period of record used	Data source	Basin characteristics				
				Area (sq mi)	Lag time (hours)	Percent of impervious cover	Length (miles)	Slope (feet per mile)
2-1396.10	Hunting Creek at Morganton, N. C.	1966-69	USGS	8.26	3.43	3	6.56	28.0
2-1429.50	Paw Creek tributary 2 at Allenbrook Drive, Charlotte, N. C.	1966-69	USGS	.62	.65	18	1.33	75.4
2-1462.35	Irwin Creek tributary at Charlotte, N. C.	1966-69	USGS	.27	.70	19	1.12	102
2-1462.80	Stewart Creek at Charlotte, N. C.	1962-69	USGS	9.40	2.00	8	5.55	28.4
2-1463.00	Irwin Creek near Charlotte, N. C.	1962-69	USGS	30.5	2.95	11	11.4	14.2
2-1463.30	Sugar Creek near Charlotte, N. C.	1962-69	USGS	43.7	5.00	9	16.6	10.7
2-1464.09	Little Sugar Creek at Brunswick Avenue, Charlotte, N. C.	1964-66	USGS	12.2	1.30	25	8.05	20.1
2-1464.20	Little Sugar Creek at Hill-side Avenue, Charlotte, N. C.	1962-69	USGS	15.4	1.54	22	9.27	18.5
2-1464.35	Briar Creek tributary 6 at Sudbury Road, Charlotte, N. C.	1966-69	USGS	.56	.54	16	1.10	59.0
2-1464.36	Briar Creek tributary at Shamrock Drive, Charlotte, N. C.	1966-69	USGS	.52	.43	20	1.07	78.1
2-1464.40	Briar Creek at East 7th Street, Charlotte, N. C.	1962-69	USGS	14.5	2.50	8	7.03	18.0
2-1464.50	Briar Creek at Sharon Road, Charlotte, N. C.	1962-69	USGS	18.5	2.99	10	9.03	14.8

Table 6.--Stream-gaging stations used to define relation for estimating lag time--Continued

U. S. Geological Survey station number	Stream and location	Period of record used	Data source	Basin characteristics				
				Area (sq mi)	Lag time (hours)	Percent of impervious cover	Length (miles)	Slope (feet per mile)
2-1464.70	Little Hope Creek at Seneca Place, Charlotte, N. C.	1966-69	USGS	2.72	.93	15	2.66	41.0
2-1465.00	Little Sugar Creek near Charlotte, N. C.	1924-69	USGS	41.0	3.39	15	11.5	16.2
2-1465.05	Little Sugar Creek tributary 7 at Burnley Road, Charlotte, N. C.	1966-69	USGS	.44	.50	14	1.05	83.0
2-1465.10	Little Sugar Creek tributary 6 at Brookcrest Dirve, Charlotte, N. C.	1966-69	USGS	.84	.60	21	1.71	92.3
2-1466.00	McAlpine Creek at Sardis Road near Charlotte, N. C.	1962-69	USGS	38.3	5.81	2	8.75	21.9
2-1466.55	McAlpine Creek at NC 51 near Charlotte, N. C.	1962-69	USGS	51.0	9.00	<1	14.1	10.8
2-1467.00	McMullen Creek at Sharon View Road near Charlotte, N. C.	1962-69	USGS	6.98	2.99	6	5.06	25.3
2-1467.25	McMullen Creek near Griffith, N. C.	1962-69	USGS	13.0	6.00	4	9.82	13.7
3-4500.00	Beetree Creek near Swannanoa, N. C.	1926-69	USGS	5.46	3.70	<1	3.89	649

Table 7.--Stream-gaging stations used to define relations for estimating flood peak discharges

USGS Station Number	Stream and Location	Period of Record Used	Area, in sq mi	Flood peak discharge, in cfs for indicated recurrence interval, in years				
				2	5	10	25	50
2-0860.00	Dial Creek near Bahama, N. C.	1925-69	4.71	347	600	810	1,130	1,410
2-0867.60	Dye Creek at Guess Road, Durham, N.C.	1967-70	.81	223	297	347	410	-----
2-0873.49	Rocky Branch at Dan Allen Drive, Raleigh, N. C.	1965-69	.56	140	225	285	365	-----
2-0873.50	Rocky Branch at Carmichael Gymnasium, Raleigh, N. C.	1965-69	.78	315	460	645	830	-----
2-0940.00	Horsepen Creek at Battle Ground, N. C.	1925-31 1934-59	15.9	654	998	1,240	1,540	1,770
2-0950.00	South Buffalo Creek near Greensboro, N. C.	1928-58	33.6	1,530	2,370	2,990	3,840	4,530
2-0955.00	North Buffalo Creek near Greensboro, N. C.	1928-69	37.0	1,750	2,790	3,660	5,000	6,190
2-0972.40	Third Fork Creek at University Drive, N. C.	1964-69	.52	185	287	357	445	------
2-0974.10	Crooked Creek near Lowes Grove, N. C.	1967-69	1.82	128	231	304	398	------
2-0990.00	East Fork Deep River near High Point, N. C.	1928-69	14.7	1,550	2,470	3,150	4,070	4,810
2-1157.30	Mill Creek near Stanleyville, N. C.	1964-69	10.2	610	840	990	1,180	------
2-1157.40	Mill Creek near Oldtown, N. C.	1964-69	27.8	640	965	1,240	1,570	------
2-1157.[illegible]0	Muddy Creek near Lewisville, N. C.	1964-69	82.8	1,800	3,200	4,200	5,400	------
2-1157.60	Silas Creek at Winston-Salem, N. C.	1964-69	5.25	510	805	1,030	1,320	------
2-1157.65	Silas Creek tributary at Pine Valley Road, Winston-Salem, N. C.	1967-69	.89	250	390	485	600	------
2-1158.00	Silas Creek at Clemmons, N. C.	1964-69	11.8	435	695	875	1,100	------
2-1158.10	Little Creek near Clemmons, N. C.	1964-69	6.81	670	1,340	1,800	2,380	------
2-1158.40	Brushy Creek at Winston-Salem, N. C.	1964-69	11.9	740	1,380	1,820	2,380	------
2-1158.43	Tar Branch at Walnut Street, Winston-Salem, N. C.	1968-69	.59	580	920	1,150	1,450	------
2-1158.45	Peters Creek at Winston-Salem, N. C.	1964-69	5.30	1,600	1,980	2,200	2,450	------
2-1158.50	Salem Creek at Winston-Salem, N. C.	1964-69	51.3	2,850	3,750	4,200	4,550	------
2-1158.60	Muddy Creek near Muddy Creek, N. C.	1964-69	178	2,900	4,050	5,050	6,250	------

Table 7.--Stream-gaging stations used to define relations for estimating flood peak discharges--Continued

USGS Station Number	Stream and Location	Period or Record Used	Area, in sq mi	Flood peak discharge, in cfs for indicated recurrence interval, in years				
				2	5	10	25	50
2-1158.70	Fiddlers Creek near Winston-Salem, N. C.	1964-69	9.73	450	625	745	895	------
2-1159.00	South Fork Muddy Creek near Clemmons, N. C.	1964-69	42.2	990	1,260	1,430	1,640	------
2-1396.10	Hunting Creek at Morganton, N. C.	1966-69	8.26	500	760	940	1,160	------
2-1429.50	Paw Creek tributary 2 at Allenbrook Drive, Charlotte, N. C.	1966-69	.62	185	300	400	525	------
2-1462.35	Irwin Creek tributary at Charlotte, N. C.	1966-69	.27	73	144	192	252	------
2-1462.80	Stewart Creek at Charlotte, N. C.	1962-69	9.40	1,220	1,740	2,080	2,520	------
2-1463.00	Irwin Creek near Charlotte, N. C.	1962-69	30.5	3,150	4,480	5,280	6,120	------
2-1463.30	Sugar Creek near Charlotte, N. C.	1962-69	43.7	3,050	3,820	4,230	4,750	------
2-1464.09	Little Sugar Creek at Brunswick Avenue, Charlotte, N. C.	1964-66	12.2	1,750	2,350	2,720	3,150	------
2-0464.20	Little Sugar Creek at Hillside Avenue, Charlotte, N. C.	1962-69	15.4	2,550	3,250	3,620	4,050	------
2-1464.35	Briar Creek tributary 6 at Sudbury Road, Charlotte, N. C.	1966-69	.56	260	485	645	845	------
2-1464.36	Briar Creek tributary at Shamrock Drive, Charlotte, N. C.	1966-69	.52	195	280	340	420	------
2-1464.40	Briar Creek at East 7 Street, Charlotte, N. C.	1962-69	14.5	1,170	1,400	1,550	1,750	------
2-1464.50	Briar Creek at Sharon Road, Charlotte, N. C.	1962-69	18.5	1,430	1,740	1,920	2,150	------
2-1464.70	Little Hope Creek at Seneca Place, Charlotte, N. C.	1966-69	2.72	500	800	1,100	1,400	------
2-1465.00	Little Sugar Creek near Charlotte, N. C.	1924-69	41.0	3,880	5,190	5,910	6,670	7,160
2-1465.05	Little Sugar Creek tributary 7 at Burnley Road, Charlotte, N. C.	1966-69	.44	110	157	230	290	------
2-1465.10	Little Sugar Creek tributary 6 at Brookcrest Drive, Charlotte, N. C.	1966-69	.84	350	500	605	735	------

Table 7.—Stream-gaging stations used to define relations for estimating flood peak discharges--Continued

USGS Station Number	Stream and Location	Period of Record Used	Area, in sq mi	Flood peak discharge, in cfs for indicated recurrence interval, in years				
				2	5	10	25	50
2-1466.00	McAlpine Creek at Sardis Road near Charlotte, N. C.	1962-69	38.3	3,050	3,850	4,320	4,900	------
2-1466.55	McAlpine Creek at NC 51 near Charlotte, N. C.	1962-69	51.0	3,050	3,880	4,280	4,780	------
2-1467.00	McMullen Creek at Sharon View Road near Charlotte, N. C.	1962-69	6.98	880	1,150	1,320	1,520	------
2-1467.25	McMullen Creek near Griffith, N. C.	1962-69	13.0	1,380	1,650	1,850	2,100	------
3-4500.00	Beetree Creek near Swannanoa, N. C.	1926-69	5.46	232	382	521	753	977

Table 8.--Stream-gaging stations used to verify relations for estimating flood peak discharge

U.S. Geological Survey station number	Stream and location	Period of record used	Basin characteristics: Area (sq mi)	Basin characteristics: Slope (feet per mile)	Basin characteristics: Length (miles)	Basin characteristics: Percent of impervious cover	Flood peak discharge, in cfs for indicated recurrence interval, in years: 2	5	10	25	50	100
2-0686.10	Hog Rock Creek near Moores Spring, N. C.	1954-68	.30	570	1.15	1	118	181	228	292	------	------
2-0690.30	Belews Creek near Kernersville, N. C.	1954-68	15	10.0	6.20	1	680	1,030	1,280	1,600	------	------
2-0708.10	Jacobs Creek near Wentworth, N. C.	1954-68	16	19.6	5.10	1	663	991	1,260	------	------	------
2-0751.60	Moon Creek near Yanceyville, N. C.	1961-68	29.9	4.80	10.5	1	841	1,630	2,300	3,300	------	------
2-0773.10	Storys Creek near Roxboro, N. C.	1954-68	2.0	38.4	2.50	1	209	277	309	338	------	------
2-0810.60	Smithwick Creek tributary near Williamston, N. C.	1953-68	.9	13.1	1.43	1	74.2	150	212	301	------	------
2-0811.10	White Oak Swamp near Windsor, N. C.	1953-68	17	7.70	7.15	1	569	1,000	1,280	------	------	------
2-0817.10	Long Creek at Kittrell, N. C.	1954-68	7.5	26.7	5.00	1	373	633	861	------	------	------
2-0818.00	Cedar Creek near Louisburg, N. C.	1956-68	47.8	15.5	12.9	1	1,350	2,010	2,340	2,65[illegible]	------	------
2-0820.00	Tar River near Nashville, N. C.	1928-68	701	3.40	99.0	1	6,680	9,890	12,400	16,200	19,400	22,900
2-0825.40	Wildcat Branch near Mapleville, N. C.	1953-68	.4	6.00	.73	1	56.6	116	170	------	------	------
2-0828.35	Fishing Creek near Warrenton, N. C.	1954-68	45	9.10	10.7	1	1,190	2,210	3,290	------	------	------
2-0834.10	Deep Creek near Scotland Neck, N. C.	1953-68	12	6.70	7.00	1	379	880	1,410	2,400	------	------
2-0838.00	Conetoe Creek near Bethel, N. C.	1956-68	78.1	2.55	16.8	1	1,190	1,850	2,330	------	------	------
2-0842.40	Collie Swamp near Everetts, N. C.	1953-68	29.0	5.60	9.50	1	682	1,270	1,790	2,620	------	------
2-0845.20	Upper Goose Creek near Yeatsville, N. C.	1953-68	1.49	1.67	2.40	1	98.0	196	275	------	------	------
2-0860.00	Dial Creek near Bahama, N. C.	1925-68	4.71	30.6	5.06	1	347	600	810	1,130	1,410	1,720
2-0870.30	Lick Creek near Durham, N. C.	1954-68	13.8	17.6	6.04	1	727	855	927	1,010	------	------
2-0871.40	Lower Barton Creek tributary near Raleigh, N. C.	1954-70	.70	95.0	1.05	1	131	211	275	367	------	------
2-0872.40	Stirrup Iron Creek tributary near Nelson, N. C.	1952 1954-70	.25	144	.60	1	59	102	140	195	------	------

Table 8.--Stream-gaging stations used to verify relations for estimating flood peak discharge--Continued

U.S. Geological Survey station number	Stream and location	Period of record used	Basin characteristics				Flood peak discharge, in cfs for indicated recurrence interval, in years					
			Area (sq mi)	Slope (feet per mile)	Length (miles)	Percent of impervious cover	2	5	10	25	50	100
2-0875.00	Neuse River near Clayton, N. C.	1927-70	1,140	4.30	101	1	9,750	13,000	15,700	19,900	23,200	27,200
2-0880.00	Middle Creek near Clayton, N. C.	1939-70	80.7	11.8	22.6	1	1,620	2,850	3,790	5,090	6,120	7,210
2-0882.10	Hannah Creek near Benson, N. C.	1953-70	2.6	12.0	3.45	1	150	335	530	888	------	------
2-0885.00	Little River near Princeton, N. C.	1930-70	229	5.60	47.4	1	2,700	4,600	6,000	8,200	9,900	11,800
2-0905.60	Lee Swamp tributary near Lucama, N. C.	1953-70	2.8	23.4	3.53	1	188	315	402	514	------	------
2-0918.10	Halfmoon Creek near Fort Barnwell, N. C.	1953-70	4.9	14.4	3.14	1	240	670	1,120	2,000	------	------
2-0921.20	Bachelor Creek near New Bern, N. C.	1953-70	34	4.40	8.80	1	788	1,260	1,680	2,370	------	------
2-0922.90	Rattlesnake Branch near Comfort, N. C.	1953-70	2.50	10.5	1.84	1	211	410	623	1,030	------	------
2-0925.00	Trent River near Trenton, N. C.	1951-70	168	3.35	30.6	1	2,050	4,400	6,600	10,500	------	------
2-0925.20	Vine Swamp near Kinston, N. C.	1953-70	6.30	15.0	4.60	1	253	477	655	910	------	------
2-0930.00	New River near Gum Branch, N. C.	1949-70	74.5	5.10	15.5	1	1,580	3,100	4,580	7,140	------	------
2-0930.40	Southwest Creek tributary near Jacksonville, N. C.	1953-70	1.00	51.2	1.06	1	129	225	278	330	------	------
2-0930.70	Southwest Creek near Jacksonville, N. C.	1953-70	27	1.24	9.61	1	632	1,130	1,540	2,140	------	------
2-0938.00	Reedy Fork near Summerfield, N. C.	1955-70	19.9	16.0	8.60	1	726	1,400	1,980	2,900	------	------
2-0940.00	Horsepen Creek at Battle Ground, N. C.	1925-31 1934-59	15.9	15.5	7.32	1	654	998	1,240	1,540	1,770	------
2-0967.40	Gun Branch near Alamance, N. C.	1954-70	5.0	27.0	4.00	1	301	717	1,130	1,850	------	------
2-0979.10	White Oak Creek near Wilsonville, N. C.	1954-70	24	14.6	11.6	1	904	1,330	1,620	1,990	------	------
2-1029.30	Crane Creek near Vass, N. C.	1954-70	32	25.0	8.00	1	930	1,640	2,210	3,060	------	------
2-1059.00	Hood Creek near Leland, N. C.	1956-70	21.6	6.71	7.95	1	595	1,090	1,480	2,030	------	------
2-1062.40	Turkey Creek near Turkey, N. C.	1953-70	16	5.80	6.52	1	389	745	1,020	1,410	------	------

Table 8.--Stream-gaging stations used to verify relations for estimating flood peak discharge--Continued

U.S. Geological Survey station number	Stream and location	Period of record used	Basin characteristics				Flood peak discharge, in cfs for indicated recurrence interval, in years					
			Area (sq mi)	Slope (feet per mile)	Length (miles)	Percent of impervious cover	2	5	10	25	50	100
2-1064.10	Stewarts Creek tributary near Warsaw, N. C.	1953-70	.46	35.2	.90	1	53	93	128	184	------	------
2-1076.00	Northeast Cape Fear River near Seven Springs, N. C.	1958-70	47.5	3.70	8.50	1	1,060	1,800	2,360	------	------	------
2-1080.00	Northeast Cape Fear River near Chinquapin, N. C.	1940-70	600	1.29	43.0	1	4,590	7,950	11,000	16,000	20,700	------
2-1085.00	Rockfish Creek near Wallace, N. C.	1955-70	63.8	4.98	14.7	1	1,330	2,320	3,100	4,240	------	------
2-1086.10	Pike Creek near Burgaw, N. C.	1953-70	1.1	14.0	1.40	1	109	340	616	1,160	------	------
2-1100.20	Mill Branch near Tabor City, N. C.	1953-70	3.80	11.8	3.61	1	150	318	495	820	------	------
2-1110.00	Yadkin River at Patterson, N. C.	1939-70	29.0	97.0	15.5	1	1,210	2,010	2,720	3,830	4,850	6,060
2-1124.10	Fisher River near Bottom, N. C.	1954-70	45	34.7	15.0	1	1,710	2,810	3,640	4,800	------	------
2-1158.30	Kerners Mill Creek near Kernersville, N. C.	1954-70	2.2	34.0	1.80	1	167	278	420	650	------	------
2-1174.10	McClelland Creek near Statesville, N. C.	1954-70	1.6	160	2.30	1	217	320	390	480	------	------
2-1175.00	Rocky Creek at Turnersburg, N. C.	1940-70	102	21.5	26.0	1	2,790	4,260	5,100	6,010	6,580	------
2-1180.00	South Yadkin River near Mocksville, N. C.	1938-70	313	11.9	41.5	1	3,730	5,640	7,180	9,460	11,400	13,600
2-1205.00	Third Creek at Cleveland, N. C.	1940-70	87.4	10.3	32.8	1	1,370	1,960	2,410	3,040	------	------
2-1225.60	Cabin Creek near Jackson Hill, N. C.	1954-70	13.7	24.4	7.00	1	870	970	1,020	1,090	------	------
2-1270.00	Brown Creek near Polkton, N. C.	1937-70	110	8.00	25.7	1	2,320	4,870	7,550	12,500	17,900	------
2-1282.60	Cheek Creek near Pekin, N. C.	1954-70	15.4	36.7	8.60	1	630	2,220	4,000	7,400	------	------
2-1294.40	South Fork Jones Creek near Morven, N. C.	1954-70	17	18.0	6.20	1	851	1,230	1,530	1,970	------	------
2-1409.80	Carroll Creek near Collettsville, N. C.	1955-70	2.3	562	3.32	1	236	344	410	486	------	------
2-1418.90	Duck Creek near Taylorville, N. C.	1954-70	19	66.7	10.0	1	894	1,360	1,670	2,070	------	------
2-1420.00	Lower Little River near All Healing Springs, N. C.	1952 1953-70	31.2	58.3	6.87	1	1,170	1,940	2,560	3,460	------	------

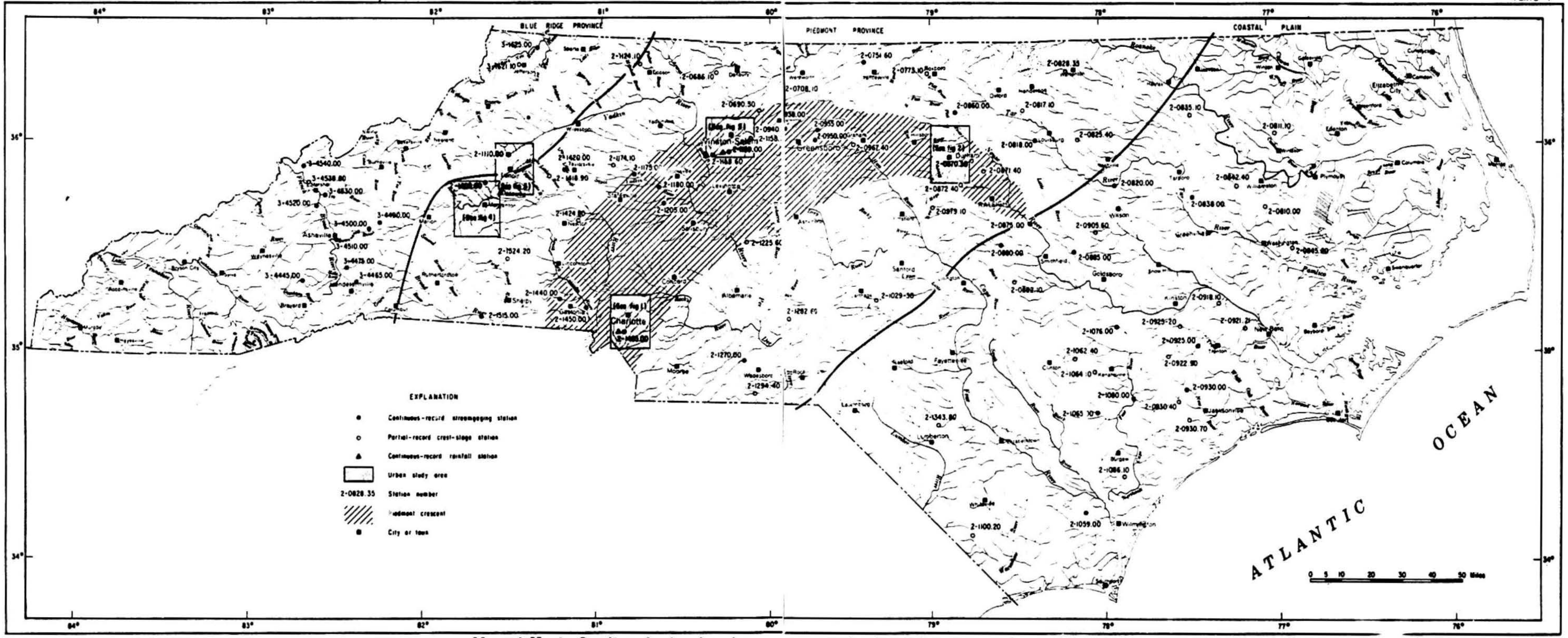
Table 8.--Stream-gaging stations used to verify relations for estimating flood peak discharge--Continued

U.S. Geological Survey station number	Stream and location	Period of record used	Basin characteristics				Flood peak discharge, in cfs for indicated recurrence interval, in years					
			Area (sq mi)	Slope (feet per mile)	Length (miles)	Percent of impervious cover	2	5	10	25	50	100
2-1424.80	Hagan Creek near Catawba, N. C.	1954-70	7.80	36.7	4.00	1	695	1,240	1,690	2,150	------	------
2-1440.00	Long Creek near Bessemer City, N. C.	1952-70	31.4	31.9	10.8	1	1,310	1,880	2,310	2,920	------	------
2-1450.00	South Fork Catawba River at Lowell, N. C.	1942-70	630	8.80	81.9	1	9,910	14,800	18,600	23,900	28,200	------
2-1515.00	Broad River near Boiling Springs, N. C.	1925-70	864	34.9	67.0	1	16,100	25,800	33,900	46,700	58,300	71,800
2-1524.20	Big Knob Creek near Fallston, N. C.	1953-70	16.4	43.1	8.35	1	1,000	1,520	1,920	2,480	------	------
3-1621.10	Buffalo Creek at Warrensville, N. C.	1955-70	23	57.0	7.00	1	1,270	2,150	2,820	3,820	------	------
3-1625.00	North Fork New River at Crumpler, N. C.	1908-16 1928-58	277	22.0	34.5	1	5,780	11,400	18,300	33,200	51,400	79,000
3-4445.00	South Fork Mills River at the Pink Beds, N. C.	1925-49 1965-70	9.99	52.8	3.79	1	580	969	1,320	1,910	2,470	------
3-4465.00	Clear Creek near Hendersonville, N. C.	1944-55	42.2	32.0	11.9	1	1,410	2,300	3,010	------	------	------
3-4475.00	Cane Creek at Fletcher, N. C.	1942-58	63.1	31.0	15.5	1	2,060	3,700	5,600	9,000	12,300	------
3-4490.00	North Fork Swannanoa River near Black Mountain, N. C.	1926-58	23.8	275	7.40	1	1,580	3,100	5,300	9,400	13,600	------
3-4510.00	Swannanoa River at Biltmore, N. C.	1920-26 1934-70	130	19.4	23.4	1	2,760	4,840	6,900	10,600	14,300	------
3-4520.00	Sandymush Creek near Alexander, N. C.	1942-55	79.5	42.2	19.0	1	1,960	3,000	3,910	5,320	------	------
3-4530.00	Ivy River near Marshall, N. C.	1933-70	158	44.7	22.6	1	3,950	6,310	8,440	11,900	15,200	------
3-4538.80	Brush Creek at Walnut, N. C.	1954-70	7.96	146	5.35	1	681	982	1,190	1,460	------	------
3-4540.00	Big Laurel Creek near Stackhouse, N. C.	1933-70	126	68.9	28.6	1	3,280	5,340	6,950	9,280	11,200	------

Table 9.--Stream-gaging stations used in the rainfall-runoff model analysis

U.S. Geological Survey station number	Stream and location	Period of rainfall record used	Basin characteristics				Flood peak discharge, in cfs for indicated recurrence interval, in years					
			Area (sq mi)	Slope (feet per mile)	Length (miles)	Percent of impervious cover	2	5	10	25	50	100
2-1157.65	Silas Creek tributary at Pine Valley Road, Winston-Salem, N. C.	1901-69	0.89	88.0	1.62	12	--------	--------	--------	1020	1100	1200
						40	--------	--------	--------	1300	1400	1450
2-1158.39	Brushy Creek tributary 2 at U.S. Highway 311, Winston-Salem, N. C.	1901-69	.55	143	1.10	37	--------	--------	--------	1250	1350	1450
						40	--------	--------	--------	1350	1450	1550
2-1158.43	Tar Branch at Walnut Street, Winston-Salem, N. C.	1901-69	.59	156	1.27	28	--------	--------	--------	1450	1550	1700
						40	--------	--------	--------	1550	1650	1780
2-1396.10	Hunting Creek at Morganton, N. C.	1901-69	8.26	28.0	6.56	3	--------	--------	--------	2200	2600	3000
						40	--------	--------	--------	3000	3250	3550
2-1464.36	Briar Creek tributary at Shamrock Drive, Charlotte, N. C.	1901-69	.52	78.1	1.07	20	--------	--------	--------	600	660	700
						40	--------	--------	--------	720	780	830
2-1464.70	Little Hope Creek at Seneca Place, Charlotte, N. C.	1901-69	2.72	41.0	2.66	15	--------	--------	--------	1950	2100	2200
						40	--------	--------	--------	2500	2700	2800

PLATE 1



Map of North Carolina showing the urban stu(areas and selected long-term gaging stations