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Statistical Relationships Between Storm and Urban Watershed Characteristics

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by

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ABSTRACT

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Because of the rapid urban development in recent years, hydrologic problems associated with urban watersheds have gained importance. Large sums of money are being spent for the design of urban drainage systems based upon inadequate procedures for predicting peak runoff rates.

In this report a procedure is proposed for predicting peak runoff rates from small urban and rural watersheds based upon measurable storm and watershed characteristics. The technique was tested for a number of runoff events on the Boneyard Creek watershed at Urbana, Illinois, and the results of this test are included. The procedure will be particularly useful for estimating runoff rates from small ungaged drainage areas, and thus will be directly applicable to both design and water management problems.

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M. Akbar Sial
J. Paul Riley
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TABLE OF CONTENTS

LIST OF FIGURES

LIST OF TABLES

INTRODUCTION

Studies at Utah State University (Narayana, et al., 1969) have demonstrated that computer simulation is a useful technique for predicting realistic changes in runoff characteristics which might result from various levels of urban development on watersheds. However, because simulation requires some precipitation and runoff information for model calibration and testing, it is not possible to apply this technique directly to ungaged watersheds. The basic objective of this study was, therefore, to develop a satisfactory procedure for predicting sufficient output information from ungaged watersheds (both urban and rural) for verification of the simulation model. A predictive technique of this nature will permit the application of simulation models to problems of storm drainage and other studies involving the hydrologic systems of ungaged watersheds.

Objectives

The objectives of this study were:

1. To derive equations for predicting the peak rate and volume of runoff from rural and urban watersheds using multiple regression analysis techniques.
2. To evaluate the relative effects of various storm and watershed characteristics on the peak rate and total volume of runoff.
3. To develop concurrency charts between the storm and watershed characteristics and peak rate and volume of runoff.

Review of Past Work

A survey of literature reveals that engineering study on the problem of predicting runoff began as early as a century ago. The problem was first recognized by engineers in the design of sewage systems. E. T. C. Myers (Chow, 1962) was the first American to present a specific formula. His work received much attention, but the formula was not sufficiently rational for general application. Myers' formula was later modified by Jarvis (1926) to read as follows:

$$Q = 100\, p\, M \quad \ldots\ldots\ldots \quad (1)$$

in which

Q = discharge in cfs

M = drainage area in square miles

p = numerical percentage rating on the Myers scale

An advantage of the Myers scale is that it provides a standard by which flood flow characteristics in different streams can be roughly compared. The use of a scale of this nature is ingenious, but it was soon found to be too simple an index to suitably represent the complicated nature of flood flow.

A well-known contribution by sewerage engineers is the rational formula for estimating rates of runoff from urban areas. In American literature, the formula was first mentioned by Emil Kuichling (1889), but its origin is somewhat obscure. The rational formula is given as:

$$Q = CIA \quad \ldots\ldots\ldots \quad (2)$$

in which

Q = discharge in cfs

C = runoff coefficient depending upon the characteristics of the drainage basin

I = rainfall intensity in inches per hour

A = drainage area in acres

The rational formula assumes that the maximum runoff rate due to a certain rainfall intensity over the drainage area is produced by that rainfall which is maintained for a period equal to the time of concentration of flow at the point under consideration. This is the time required for the surface runoff from the most remote part of the drainage basin to reach the runoff point being considered. The Joint Committee of American Society of Civil Engineers (Chow, 1962) and the Water Pollution Control Federation reported values of C as given by Table 1.

Many studies have been undertaken during the past 60 years which deal with the problem of predicting runoff for various types of watersheds. A number of formulas, in addition to those already cited, were developed before 1957, and they are presented in Appendix A. In the past 10 years, however, the general problem of runoff prediction has gradually developed into that of synthesizing the runoff hydrograph for the present and future

Table 1. Values of C in rational formula reported by a Joint Committee of American Society of Civil Engineers and the Water Pollution Control Federation in 1960, (Chow, 1962).

Type of drainage area	Runoff Coefficient, C
Business.	
Downtown areas	0.70 - 0.95
Neighborhood areas	0.50 - 0.70
Residential.	
Single-family areas	0.30 - 0.50
Multi-units, detached	0.40 - 0.60
Multi-units, attached	0.60 - 0.75
Suburban	0.25 - 0.40
Apartment dwelling areas	0.50 - 0.70
Industrial:	
Light areas	0.50 - 0.80
Heavy areas	0.60 - 0.90
Parks, cemeteries	0.10 - 0.25
Playgrounds	0.20 - 0.35
Railroad yard areas	0.20 - 0.40
Unimproved areas	0.10 - 0.30
Streets:	
Asphaltic	0.70 - 0.95
Concrete	0.80 - 0.95
Brick	0.70 - 0.85

design of flood control systems in urban areas. A number of quantitative evaluations of the effects of urbanization on flood flow entailed the use of the "rational formula" and the "unit hydrograph method of analysis" in the design of drainage structures. Boch (1958) reported a study of flows into storm drains and inlets in the city of Baltimore. In his "inlet method" of predicting runoff, Boch considered the degree of imperviousness and magnitude of the intense part of thunderstorms as the independent variables. Benson (1959) showed, as sug- by Nash (1958) and others, that after three or four independent meteorologic and physiographic variables have been used, additional variables do not appreciably decrease the standard error in estimating floods. Benson's analysis eliminates the effect of individual storms since flood peaks of specified return periods, obtained from a frequency analysis of annual maxima, were used as his dependent variable. The main channel slope was found to be next in importance to drainage area size. Benson's study has little application to small watersheds, however, since only three of the 170 New England drainage areas included in his study possessed areas of less than 10 square miles.

Hickok et al. (1959) made a significant contribution to hydrograph synthesis. They studied about 130 hydrographs and hyetographs from 14 watersheds ranging in size from 11 to 790 acres in the arid southwest. Lag time was related to watershed area, average land slope, and drainage density. The estimated lag time was used to predict the hydrograph peak rate for an assumed total volume of runoff. Finally, the entire synthesized hydrograph was obtained from a generalized hydrograph expressed non-dimensionally in terms of lag time and peak rate. Their dimensionless hydrograph appeared to be independent of rainfall pattern or of soil and cover condition. It is likely, however, that this simplification resulted, at least in part, from the very similar climatic and cover conditions within the four research locations. No consideration was given to urbanization in this study. Sawyer (1961) studied the effects of urbanization on the runoff yield from watersheds, and reported that the characteristics of many streams on Long Island were changed by increased urbanization. No quantitative information regarding the increase in runoff volume as a result of urbanization was presented in Sawyer's study.

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

Manuel A. Benson (1962) developed relations between flood peaks and hydrologic factors in a humid region with limited climatic variation but diversity of terrain. He applied statistical multiple-regression techniques to hydrologic data from New England. His equations related peak discharges of 1.2 to 300-year recurrence intervals to 6 hydrologic variables. His equation for the 25 year recurrence interval is:

$$Q = 2.08\ AS^{0.5} St^{-0.3} I^{0.5} t^{0.4} 0^{1.1}. \quad (3)$$

in which

- Q = peak discharge in cubic feet per second for 25-year recurrence interval
- A = drainage area, in square miles
- S = main channel slope, in feet per mile
- St = percent of surface storage area plus 0.5 percent
- I = 25-year, 24-hour rainfall intensity, in inches
- t = average January degrees below freezing, in degrees Fahrenheit
- 0 = orographic factors

Because of lack of data, urbanization effects were not examined in Benson's study.

Chow (1962) presented a method for determining peak discharges from rural watersheds smaller than 6,000 acres in area. By a trial and error technique, the method determines the duration of rainfall excess giving the maximum rate of runoff, and estimates the latter by applying four charts. The method involves runoff curve numbers and relationships presented by the U. S. Soil Conservation Service. Although the charts presented are applicable only to Illinois, the first two phases of the method are general in nature and can be applied to data from other watersheds. To complete the procedure, it is necessary to express the peak reduction factor as a function of the ratio of the duration of rainfall excess to lag time. The lag time must also be estimated from watershed characteristics. Chow obtained these two relationships from 53 storms covering 20 small watersheds in the midwest. Until similar relationships are available for other climatic and topographic areas, the method is regionally restricted.

R. W. Cruff and S. E. Rantz (1965) examined several methods of analyzing flood frequencies on a regional basis, and evaluated the relative reliability of these methods. The areas selected for study were the sub-humid San Diego area in southwestern California and the humid coastal area of northwestern California. Six methods of analysis were studied, namely, index flood, multiple correlation, logarithmic normal distribution, extreme value probability distribution (Gumbel method), Pearson Type IV distribution, and gamma distribution. Where applicable, basin and climatological characteristics were used in developing additional statistical relations. Three general conclusions were reached: (i) results are more reliable in humid regions where stream flow is less variable, (ii) the multiple-correlation method is preferred if historical data are available, and (iii) the Pearson Type IV is more desirable for distribution analysis where the period of record is used.

John R. Crippen (1965), from a study of Sharon Creek basin near Palo Alto, California, concluded that peak discharge rates from a particular storm type increased from 180 cfs in 1960 to 250 cfs in 1963 due to the growth of urbanization accompanied by the construction of paving and drainage facilities. Van Sickle (1965) applied the unit hydrograph method to determine the effects of urbanization on peak discharge in Houston, Texas. Continuous stage records were available for eight of the watersheds which he studied. Records for Brays Bayou, the watershed within his study containing the greatest urban development, were available for the 27-year period 1939 and 1961. During this period, the watershed changed from undeveloped farmland to an extensively urbanized area. Van Sickle divided this period into six stages of urbanization ranging from low to very high. Peak flow unit hydrographs corresponding to each of the six urbanization stages are shown by Figure 1. Van Sickle concluded that urban development of a watershed in Harris County can be expected to produce peak discharge rates of from two to five times those which would occur on the same watershed under rural conditions.

Linear regression analysis was used by Espey et al. (1965) to analyze 11 rural and 24 urban watersheds. The independent variables considered in his study were area, mean slope, percentage impervious cover, and length of the main channel in the watershed. The expressions developed by Espey describe the characteristics of the 30-minute unit hydrograph. He applied his equations to the Waller Creek watershed at Austin, Texas, and indicated that the peak discharge would approximately double as the watershed changed from rural (0 percent of impervious cover) to highly urbanized conditions (50 percent impervious cover).

The studies cited in this section indicated several storm and watershed characteristics which are important in determining hydrograph characteristics for ungaged areas. This information was of great value to the investigation reported herein, in which an attempt was made to develop a model capable of realistically estimating peak discharge rates and total runoff volumes corresponding to particular storm events on ungaged watersheds.

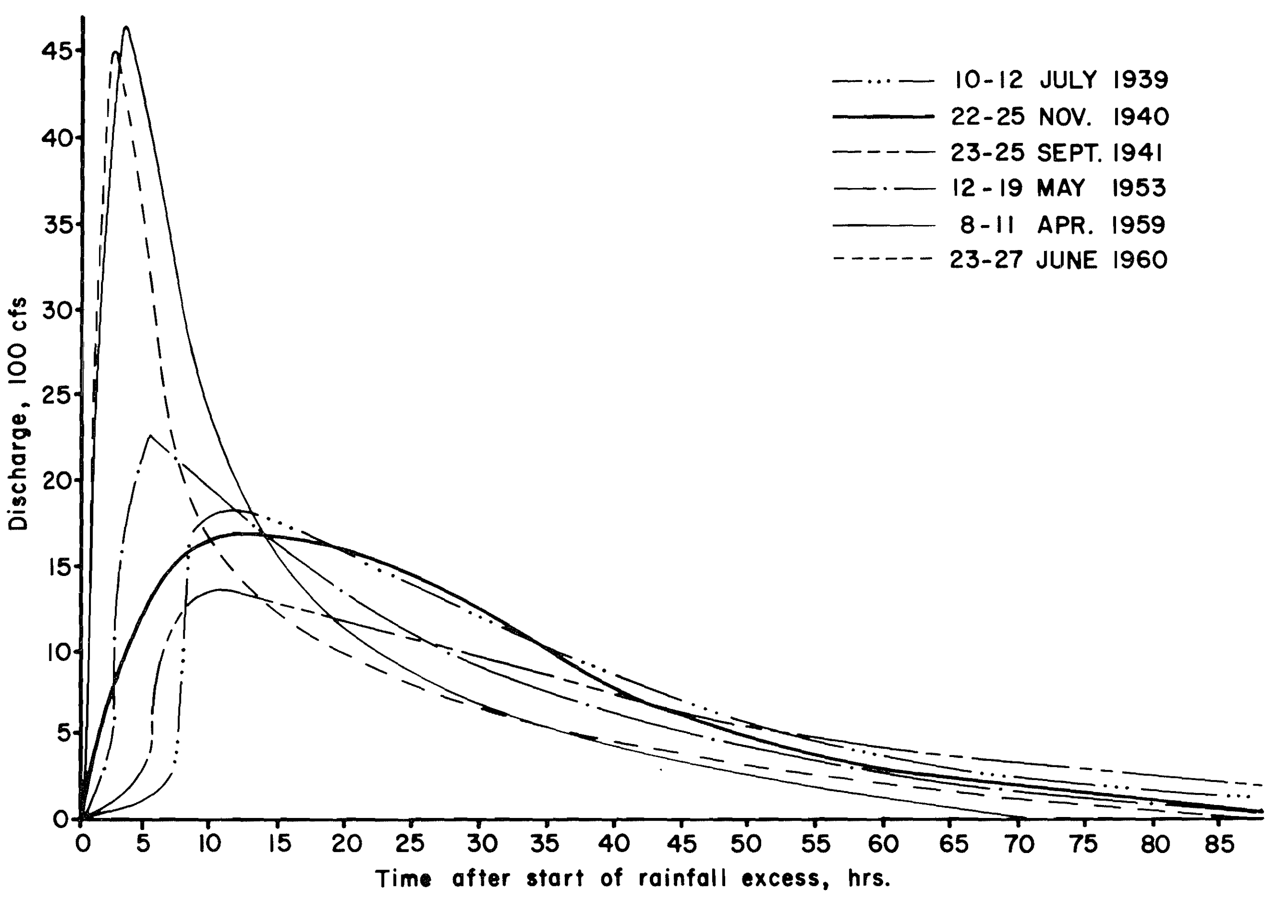

Figure 1. Brays Bayou unit hydrographs (after Van Sickle).

SELECTION OF MODEL VARIABLES

Ordinarily floods are caused by runoff from rainfall and snowmelt and less frequently by dam failures or high tides. Many factors influence the rate and the total volume of runoff after the precipitation reaches the ground surface. Meteorologic factors such as temperature, dewpoint, radiation, wind, and cloud cover influence the amount of precipitation and evaporation and thus affect runoff. After runoff begins, the pattern is controlled by the topographic characteristics of the watershed. This is especially true when precipitation is in the form of rain. Watershed characteristics may be either surface or underground features. Most of the geologic features of a watershed, such as drainage area and land slope (aspect and degree), are relatively stable; but other variables, such as percentage impervious cover in case of urban watersheds and the land use in the case of rural watersheds, change with time. Within a watershed, the variable parameters account to some extent for the variation in the magnitude of the flood peak and volume of runoff from year to year.

The first step in developing a statistical runoff model is to select those parameters which are significant in describing the system to be modeled. The second step is to break those parameters selected into their simplest components, to evaluate them on the basis of hydrologic and hydraulic principles, and to choose those factors having the least interdependence. Finally, statistical methods are applied in developing relationships between runoff and storm and watershed characteristics.

As previously indicated, multiple correlation techniques were employed to relate a number of storm and watershed characteristics (considered as independent variables) to certain characteristics of the runoff hydrograph (considered as dependent variables). The various independent and dependent variables used in this study will be discussed in the following paragraphs.

Independent Variables

The proper selection of the independent variables is critical, because, if the explanatory variables are highly correlated with one another, it becomes difficult to distinguish their separate influences and obtain a reasonable estimate of their relative effects. In fact, there are few variables in a hydrologic system which are completely independent, and so in developing a statistical model of the runoff process it becomes a problem of selecting those variables with the least degree of dependence. Previous research has indicated that a highly important variable affecting runoff is the size of the drainage area. The larger the area, the larger the volume of rain that may fall on it and, in general, the larger the total runoff volume and rate. With the drainage area selected as an indèpendent variable, most of the remaining factors that may be chosen as variables have some degree of interdependence. The general magnitude of rainfall is virtually independent being a climatic factor, yet rainfall intensity varies with size of the drainage area, and rainfall distribution varies with directional or orographic characteristics of the basin. Soil, cover, and slope may be effected by the quantity of annual rainfall. Thus, topographic and meteorologic variables are not independent. The precipitation falling on a basin flows initially by an overland route to small channels, then to progressively larger tributaries through a complex drainage pattern to the principal stream and the gaging point. Therefore, the slopes of land surfaces and drainage channel slopes are important independent variables. The ground cover, the channel bed materials, and channel form roughness retard the flow of runoff at various stages and should be considered, if adequate data are available. Since runoff occurs by both surface and underground routes, the type of soil and geology may also be considered. The drainage pattern influences the timing of the flood peak and should be included possibly as a basin shape factor. Attitude or orientation of the basin with respect to storm pattern may influence the amount and timing of rainfall and merits consideration. The amount of storage in lakes, ponds, reservoirs, swamps, or within river channels may reduce the flood peaks and, if pertinent, should be considered as an independent characteristic.

Because of their interdependence, many of the topographic characteristics cited above were not included in the final equations developed under this study. Thus, it is possible to explain much variance in the system by including only one of many interrelated factors. For example, a study of the precipitation data used in this investigation revealed the following average levels of correlation between accumulated rainfall occurring in 15-minute, 30-minute, and 60-minute periods, respectively.

1. Correlation between 15-minute rainfall and 30-minute rainfall - 0.96
2. Correlation between 15-minute rainfall and 60-minute rainfall - 0.87
3. Correlation between 30-minute rainfall and 60-minute rainfall - 0.94

It was therefore decided to use the 30-minute rainfall as a characteristic of the precipitation, and to delete the 15-minute and 60-minute quantities as independent variables in this study. Thus, considerable latitude exists in the method of defining variables for a statistical model, and simplicity is a highly desirable feature of any method.

In this study the following storm and watershed characteristics were selected initially as independent variables.

Storm characteristics

1. Duration of the storm, D.
2. Total rainfall, P_T.
3. Maximum rainfall in an interval of 15-minutes, P_{15}.
4. Maximum rainfall occurring in an interval of 30-minutes, P_{30}, during a storm event.
5. Maximum rainfall occurring in an interval of 60-minutes, P_{60}, during a storm event.

Watershed characteristics

1. Watershed area, A.
2. Mean slope, S.
3. Main channel length, L.
4. Impervious cover factor, c_f, where $c_f = 1 - R_i$, and R_i is the ratio of paved surfaces (roofs, roadways) to unpaved surfaces. For rural watersheds, $c_f = 1$.
5. Degree of channelization Φ. Classification of Φ is given in Table 2.

Table 2. Classification of the degree of channelization (Johnson, 1966).

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

Dependent Variables

The dependent parameters adopted in this study were the peak rate of runoff, Q_p, and the total volume of runoff, Q_T. Through multiple regression techniques relationships were developed between these characteristics of the runoff hydrograph and those parameters listed as independent variables.

SOURCES OF DATA

In this study, a total of 393 storms occurring on 70 different watersheds were considered. Of the 70 watersheds 50 were rural and 20 represented various degrees of urban development. Records for 200 runoff events were taken from the rural watersheds, while the remaining 193 events occurred on urban watersheds. All watersheds were equipped with at least one recording rain gage and a stream gaging station.

Rural Watersheds

Data from the rural watersheds were collected from the following publications:

1. Hydrologic Data for Experimental Agricultural Watersheds in the United States, 1956-59. Miscellaneous Publication No. 945. Agriculture Research Service, United States Department of Agriculture.

2. Hydrologic Data for Experimental Agricultural Watersheds in the United States, 1960-61. Miscellaneous Publication No. 994. Agriculture Research Service, United States Department of Agriculture.

3. Hydrologic Data for Experimental Agricultural Watersheds in the United States, 1962. Miscellaneous Publication No. 1070. Agriculture Research Service, United States Department of Agriculture.

Names of the watersheds and the state wherein they lie are given in Table 3.

Urban Watersheds

Hydrologic data for urban watersheds are relatively scarce, but records were available for the 20 drainage basins listed by Table 4. The first 16 watersheds given by Table 4 lie within the

Table 3. List of rural watersheds analyzed.

1	Oxford, Mississippi, W-4
2	Oxford, Mississippi, W-5
3	Oxford, Mississippi, W-10
4	Oxford, Mississippi, W-12
5	Oxford, Mississippi, W-17
6	Oxford, Mississippi, W-19
7	Oxford, Mississippi, W-24
8	Oxford, Mississippi, W-28
9	Oxford, Mississippi, W-30
10	Oxford, Mississippi, W-32
11	Oxford, Mississippi, W-34
12	Oxford, Mississippi, W-35
13	Fennimore, Wisconsin, W-1
14	Fennimore, Wisconsin, W-2
15	Hastings, Nebraska, W-3
16	Hastings, Nebraska, W-5
17	Hastings, Nebraska, W-8
18	Hastings, Nebraska, W-11
19	Safford, Arizona, W-1
20	Safford, Arizona, W-2
21	Safford, Arizona, W-4
22	Safford, Arizona, W-5
23	Albuquerque, New Mexico, W-1
24	Watkinsville, Georgia, W-1
25	High Point, North Carolina, West Ford Deep River Watershed
26	Blacksburg, Virginia, W-3
27	Blacksburg, Va., Thorne Creek Watershed, W-1
28	Blacksburg, Virginia, Brush Creek Watershed, W-1
29	Blacksburg, Va., Powells Creek Watershed, W-1
30	Blacksburg, Virginia, Rocky Run Branch Watershed, W-1
31	Blacksburg, Va., Pony Mountain Branch, W-1
32	Blacksburg, Virginia, Fosters Creek, W-1
33	Blacksburg, Virginia, Chestnut Branch, W-1
34	Coshcocton, Ohio, W-10
35	Coshcocton, Ohio, W-5
36	Coshcocton, Ohio, W-92
37	Coshcocton, Ohio, W-94
38	Coshcocton, Ohio, W-95
39	Coshcocton, Ohio, W-97
40	Coshcocton, Ohio, W-994
41	Cherokee, Oklahoma, W-1
42	Cherokee, Oklahoma, W-2
43	Cherokee, Oklahoma, W-3
44	Cherokee, Oklahoma, W-5
45	Cherokee, Oklahoma, W-9
46	Reisel (WACO), Texas, W-C
47	Reisel (WACO), Texas, W-1
48	Reisel (WACO), Texas, W-2
49	Reisel (WACO), Texas, W-8
50	Reisel (WACO), Texas, W-10

Table 4. List of urban watersheds.

1	Bering Ditch at Woodway, Houston, Texas
2	Bering Bayou at Forest Oaks, Houston, Texas
3	Berry Creek at Galveston, Houston, Texas
4	Berry Bayou at Gilpin, Houston, Texas
5	Berry Bayou Tributary at Globe, Houston, Texas
6	Brickhouse Gully at Costarica, Houston, Texas
7	Hunting Bayou at Calvacade, Houston, Texas
8	Hunting Bayou at Falls Street, Houston, Texas
9	Hunting Bayou at U. S. 90A, Houston, Texas
10	Willow Waterhole Bayou at Landsdowne, Houston, Texas
11	Brickhouse Gully at Clarblak, Houston, Texas
12	Colecreek at John Road, Houston, Texas
13	Halls Bayou at Deer Trail, Houston, Texas
14	Keegans Bayou at Keegans Road, Houston, Texas
15	Keegans Bayou at Roak Road, Houston, Texas
16	Sims Bayou at Carlsbad, Houston, Texas
17	Waller Creek, 23rd Street, Austin, Texas
18	Waller Creek, 38th Street, Austin, Texas
19	Northwood, Maryland
20	Gray Haven, Maryland

boundaries of Houston, Texas, and data pertaining to runoff events for these watersheds have been compiled by the U. S. Geological Survey in the following reports: (1) Urban Hydrology of the Houston, Texas, Metropolitan Area. (2) Compilation of Basic Data, April, 1964, to September, 1965, by S. L. Johnson and R. E. Smith.

Data on the drainage areas within the City of Austin, Texas, were taken from the following report: Compilation of Hydrologic Data, Waller Creek, Colorado River Basin, Texas, 1963, 1964, 1965. U. S. Geological Survey, Water Resources Division.

Data for the two watersheds within the City of Baltimore, Maryland were taken from the following publications:

1. Northwood Gaging Installation, Baltimore, Instrumentation and Data, ASCE Urban Water Resources Research Program, Technical Memorandum No. 1, by L. S. Tucker, August 1968.

2. Availability of Rainfall-Runoff Data for Sewered Drainage Catchments, ASCE Urban Water Resources Research Program Memorandum No. 8, by L. S. Tucker, March 1969.

Data Reduction

In general, the rain data required some processing in order to convert it to the proper format for input to the computer program. The various parameters and corresponding dimensions required for the computer analysis are given in the following list:

1. Watershed area in acres.
2. Mean slope in percent.
3. Main channel length in miles.
4. Impervious cover factor in dimensionless decimal.
5. Degree of channelization in dimensionless decimal.
6. Length of roads in miles.
7. Storm duration in hours.
8. Total rainfall in inches.
9. Maximum 15-minute rainfall in inches per 15-minutes.
10. Maximum 30-minute rainfall in inches per 30-minutes.
11. Maximum 60-minute rainfall in inches per 60-minutes.
12. Peak rate of runoff in cubic feet per second.
13. Total volume of runoff in acre feet.

A computer program, available at the Utah Water Research Laboratory, was used to compute the equal interval rainfall for the 200 rural events. The percent impervious cover was converted to decimal form so that rural watersheds could be included and would have a value of one.

ANALYSIS PROCEDURE

Multiple Linear Regression

The technique of multiple linear regression analysis establishes a functional relationship which predicts the dependent variable from a number of independent variables. An anticipated relationship is established and the least squares criteria is applied to empirical observations of both dependent and independent variables solved simultaneously for the coefficients of each term. Since there is one equation for each variable, the computations become cumbersome and require a digital computer. A linear mathematical model is presented as an example.

$$\hat{Y} = b_o + b_1x_1 + b_2x_2 + b_3x_3 \ldots b_nx_n \quad . \quad . \quad . \quad . \quad . \quad . \quad (4)$$

in which

$\hat{Y}$ = dependent variable

$x_1, x_2 \ldots x_n$ = independent variables

$b_1, b_2 \ldots b_n$ = regression coefficients

b_o = the regression constant

In the case of two independent variables, the b coefficients are evaluated by the solution of the following simultaneous equations:

$$b_1\ \Sigma(x_1) + b_2\ \Sigma(x_1x_2) = \Sigma(yx_1) \quad . \quad . \quad (5)$$

$$b_1\ \Sigma(x_1x_2) + b_2\ \Sigma(x_2)^2 = \Sigma(yx_2) \quad . \quad (6)$$

Considering the case of three independent variables, coefficients can be computed by the solution of the following simultaneous equations:

$$b_1\ \Sigma(x_1)^2 + b_2\ \Sigma(x_1x_2) + b_3\ \Sigma(x_1x_3) = \Sigma(yx_1) \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (7)$$

$$b_1\ \Sigma(x_1x_2) + b_2\ \Sigma(x_2)^2 + b_3\ \Sigma(x_2x_3) = \Sigma(yx_2) \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (8)$$

$$b_1\ \Sigma(x_1x_3) + b_2\ \Sigma(x_2x_3) + b_3\ \Sigma(x_3)^2 = \Sigma(yx_3) \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (9)$$

When more than three independent variables are involved, the appropriate number of simultaneous equations is constructed in a manner similar to that illustrated previously.

The regression constant b_o is determined as follows:

$$b_o = \overline{Y} - b_1\overline{X}_1 - b_2\overline{X}_2 \ldots -b_n\overline{X}_n \quad . \quad (10)$$

in which

$\overline{Y}$ = the mean of the dependent variable

$\overline{X}_1, \overline{X}_2 \ldots \overline{X}_n$ = the respective means of the independent variables

In Equations (5) through (9), the quantities $\Sigma(x)^2$, $\Sigma(x_1x_2)$, and $\Sigma(yx_1)$ are evaluated as follows:

$$\Sigma(x)^2 = \Sigma(X - \overline{X})^2 = \Sigma(X^2) - (\Sigma X)^2/N \quad . \quad (11)$$

$$\Sigma(x_1x_2) = \Sigma(X_1\overline{X}_1)\ (X_2 - \overline{X}_2) = \Sigma(X_1X_2) - \Sigma X_1 \Sigma X_2/N \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (12)$$

$$\Sigma(yx) = \Sigma(Y - \overline{Y})(X - \overline{X}) = \Sigma(YX) - \Sigma Y \Sigma X/N \quad . \quad . \quad . \quad (13)$$

Computer Programs

A multiple regression analysis involves numerous computations and the use of a digital computer is indispensable. In this study, use was made of two library programs written by Dr. Rex L. Hurst for the Digital Computer Center at Utah State University. The important phases of the two programs are briefly described as follows.

Multivariate data collection revised

This program, abreviated MDCR, was written to serve as a basic data collection program for a wide variety of multivariate analysis. It computes means, standard deviations, corrected sum of squares and products, and corrections among the variables. In addition, several kinds of transformations can be performed on the input data. These transformations include products, square roots, logarithmic, exponents, sums of variables, arc sin, and trignometric. A listing of the program and a sample output is given in Appendix B.

Stepwise multiple regression revised

The stepwise multiple regression revised (SMRR) program was written to perform a multiple regression analysis, either stepwise or non-stepwise, from any possible group of variables used in the MDCR program. The two computer programs, MDCR and SMRR, therefore, were used together to perform the multiple regression analysis of this study.

The SMRR program initially includes all of the independent variables in the model and then deletes the least significant variables one at a time. The first deleted variable is that which contributes the least to the model sum-of-squares. Once a variable is deleted, a new model is formed, an analysis performed, and a second variable is deleted as before. Once a variable is deleted from a model, the variable is not reconsidered. A sample of the listing and output of the SMRR program is included in Appendix B.

Statistical Regression Models

The following empirical models were tested by multiple regression analysis.

Model A

$$Y = b_o + b_1 X_1 + b_2 X_2 + \ldots + b_n X_n \quad . \quad . \quad . \quad (14)$$

Model B

$$Y = b_o X_1^{b_1} X_2^{b_2} X_3^{b_3} \ldots X_n^{b_n} \quad . \quad . \quad . \quad (15)$$

in which

Y	=	the dependent variable
X_i, $i=1, \ldots n$	=	independent variables
b_o and b_i, $i=1 \quad n$	=	regression coefficients

In the case of Model A, non-logarithmic relations were developed. However, for Model B data were transformed into logarithms and the model was expressed in the following linear form.

$$\ln Y = \ln b_o + b_1 \ln X_1 + b_2 \ln X_2 + \ldots \; b_n \ln X_n \quad . \quad . \quad . \quad (16)$$

Model C

A third model was also tested in which eight independent variables were grouped to form three independent variables as follows:

1. Watershed factor, $W = A\,S^{1/2}L^{0.3}$
2. Storm factor, $St = d^{0.3}\,P_T P_{30}^{\;0.3}$
3. Urbanization factor, $U = \phi/c_f$

in which all variables have been previously defined. A regression analysis was then performed including the preceding three independent variables and the two dependent variables of peak discharge rate and total runoff volume. The following model was assumed.

$$Y = b_o W^{b_1} St^{b_2} U^{b_3} \quad . \quad . \quad . \quad . \quad . \quad . \quad (17)$$

Equation development and testing

For each of the three models described previously multiple regression analysis were performed for 193 storms on urban watersheds and 200 storms on rural drainage areas. Equations were developed and tested for both urban and rural conditions. The possibility of developing general relationships which would apply to both urban and rural conditions was investigated by repeating the analysis using pooled data from the urban and rural areas. Finally, co-axial curves were plotted by assuming various values for the independent variables.

RESULTS AND DISCUSSION

The regression analysis of this study included eight independent and two dependent variables. Independent variables:

$A = x_1$ = area in acres

$S = x_2$ = slope in percentage

$L = x_3$ = length of the main channel in miles

$D = x_4$ = duration of storm in hours

$p = x_5$ = total rainfall in inches

$P_{30} = x_6$ = maximum 30-minute rainfall in inches

$c_f = x_7$ = impervious cover factor

$\phi = x_8$ = degree of channelization

Dependent variables:

$Q_p = Y_1$ = peak runoff in cfs

$Q_T = Y_2$ = total volume of runoff in acre feet

Each of the three mathematical models presented in the previous section was used to analyze the urban and rural storm data to form equations for predicting peak discharge rate, Q_p, and total runoff volume, Q_T, for urban, rural, and general conditions of watershed cover. The following equations are those derived from the three models for the cases indicated.

Model A

Rural

$$Q_p = -404.55 + 0.025A + 5.9S + 187.35L + 40.77D + 163.34p - 58.62P_{30} \quad . . . . . . (18)$$

$$Q_T = -150.41 + 0.0341A - 0.0945S + 28.05L + 45.67D - 6.64P + 4.32P_{30} \quad . . . . (19)$$

General

$$Q_p = 55.40 + 0.04A + 30.88S + 133.96L - 21.51D + 256.52p - 47.36P_{30} - 1.19c_f - 499.15\phi \quad . . . . (20)$$

$$Q_T = -186.20 + 0.039A + 4.59S + 16.46L + 0.72D + 104.27p - 82.45P_{30} - 0.765c_f + 110.83\phi \quad . . . . (21)$$

Deletion of the equation for the urban model is due to a slight anomoly which appeared after the computer work was finished. A rerun was not made because the model change would not have been significant enough to change the rank of the urban Model A.

Model B

Urban

$$Q_p = \frac{0.143A^{0.9855}S^{0.225}p^{1.17}P_{30}^{0.32}}{L^{0.285}D^{0.351}c_f^{1.45}\phi^{1.49}} \quad . . . . (22)$$

$$Q_T = \frac{0.00104A^{1.24}p^{1.323}\phi^{0.612}}{S^{0.33}L^{0.233}D^{0.094}P_{30}^{0.049}c_f^{4.23}} \quad . . . . (23)$$

Rural

$$Q_p = \frac{3.936A^{0.553}L^{0.356}p^{0.906}}{S^{0.175}A^{0.065}P_{30}^{0.039}} \quad . (24)$$

$$Q_T = \frac{0.048A^{0.909}L^{0.181}D^{0.099}p^{1.219}}{S^{0.342}P_{30}^{0.358}} \quad . . . . (25)$$

General

$$Q_p = \frac{0.777A^{0.738}S^{0.204}{}_{p}^{1.016}P_{30}^{0.179}}{L^{0.042}D^{0.26}c_f^{0.797}\phi^{1.23}} \quad . \; . \; . \; . \; . \; . \; . \quad (26)$$

$$Q_T = \frac{0.777A^{0.738}S^{0.036}L_p^{1.248}\phi^{1.164}}{L^{0.272}D^{0.076}P_{30}^{0.187}c_f^{2.209}} \quad . \; . \; . \; . \; . \; . \; . \quad (27)$$

Model C

Urban

$$Q_p = 1.607W^{0.664}St^{0.53}U^{0.55} \quad . \; . \quad (28)$$

$$Q_T = 0.0595W^{0.937}St^{0.868}U^{1.04} \quad . \quad (29)$$

Rural

$$Q_p = 0.752W^{0.723}St^{0.589} \quad . \; . \; . \; . \quad (30)$$

$$Q_T = 0.007W^{1.019}St^{0.75} \quad . \; . \; . \quad (31)$$

General

$$Q_p = 0.734W^{0.706}St^{0.615}U^{1.91} \quad . \quad (32)$$

$$Q_T = 0.012W^{0.975}St^{1.027}U^{4.63} \quad . \quad (33)$$

Model Selection

The coefficient of determination, R^2, was used as a test to determine which model most completely explained the runoff prediction variance. Table 5 shows the relative R and R^2 values for the models. Model B gave the highest R^2 value, so it was used as the best model in construction of the concurrency charts. Comparing the rural and general cases, Model A was the poorest model which fact also influenced the decision to not make a rerun of the urban case. Tables 6 through 11 are tables of variance analysis for Q_p and Q_T resulting from the application of Model B to the three watershed cases. The level of significance shown in the tables is calculated from the following equation and condition:

$$\sigma_R = \frac{1 - R^2}{\sqrt{N - 1}} \quad . \; . \; . \; . \; . \; . \; . \; . \; . \quad (34)$$

99 percent significant if $R > 3\sigma_R$

in which

N = number of events considered

Table 5. Coefficients of correlation and determination for the three models.

Case / Model	Rural R	Rural R^2	Urban R	Urban R^2	General R	General R^2	Dependent Parameter
A	0.765	0.586			0.740	0.548	Q_p
	0.826	0.687			0.794	0.629	Q_T
B	0.890	0.795	0.914	0.834	0.885	0.784	Q_p
	0.945	0.891	0.920	0.850	0.935	0.876	Q_T
C	0.866	0.752	0.809	0.665	0.844	0.712	Q_p
	0.915	0.829	0.880	0.774	0.868	0.774	Q_T

Table 6. Analysis of variance for peak runoff (urban), Model B.

Source	DF	Mean square	F
Total	192	1.8015	119.017**
A	1	37.1453	4.866*
S	1	1.5188	4.389*
L	1	1.3701	22.99 **
D	1	7.1774	94.20 **
p	1	29.4000	10.06 **
P_{30}	1	3.1410	8.429**
c_f	1	2.6307	16.729**
ϕ	1	5.2214	115.53 **
Model	8	36.0595	
Error	184	0.3121	

*significant at 0.95 level

**significant at 0.99 level

Table 7. Analysis of variance for total runoff (urban), Model B.

Source	DF	Mean square	F
Total	192	3.4031	
A	1	59.3306	111.71 *
S	1	3.2754	6.166*
L	1	0.9150	1.722*
D	1	0.5126	0.9651*
p	1	37.3562	70.33 **
P_{30}	1	0.0751	0.1414
c_f	1	22.4527	42.27 **
ϕ	1	0.8753	1.648
Model	8	69.4564	130.77 **
Error	184	0.53118	

*significant at 95 percent level

**significant at 99 percent level

Table 8. Analysis of variance for peak runoff (rural), Model B.

Source	DF	Mean square	F
Total	199	4.0678	
A	1	18.5842	21.65 **
S	1	2.5270	2.944*
L	1	2.3543	2.743*
D	1	0.2570	0.2994
p	1	16.4239	19.13 **
P_{30}	1	0.0436	0.0508
Model	6	107.3132	125.05 **
Error	193	0.8581	

*significant at 95 percent level

**significant at 99 percent level

Table 9. Analysis of variance for total runoff (rural) Model B.

Source	DF	Mean square	F
Total	199	7.0899	
A	1	50.3512	63.35 **
S	1	9.5628	12.03 **
L	1	0.6106	0.768
D	1	0.6083	0.765
p	1	29.7757	37.46 **
P_{30}	1	3.6657	4.612*
Model	6	209.5877	263.73 **
Error	193	0.7947	

*significant at 95 percent level

**significant at 99 percent level

Table 10. Analysis of variance for peak runoff (general), Model B.

Source	DF	Mean square	F
Total	392	2.9803	
A	1	78.7227	119.32 **
S	1	33.0614	50.32 **
L	1	0.0830	.126
D	1	8.5261	12.97 **
p	1	44.0216	67.00 **
P_{30}	1	1.9368	2.94
c_f	1	4.6128	7.021**
ϕ	1	4.0319	6.13
Model	8	114.4937	174.26 **
Error	384	0.6570	

*significant at 95 percent level
**significant at 99 percent level

Table 11. Analysis of variance for total runoff (general), Model B.

Source	DF	Mean square	F
Total	392	5.7751	
A	1	177.8101	244. **
S	1	1.0341	1.421
L	1	3.5326	4.85 *
D	1	0.7291	1.001
p	1	66.4329	91.3 **
P_{30}	1	2.1820	2.99
c_f	1	35.4442	48.7 **
ϕ	1	3.6012	4.95 *
Model	8	248.0279	341.0 **
Error	384	0.7281	

*significant at 95 percent level
**significant at 99 percent level

Classification on an Area Basis

The watersheds were separated into three groups based on area: Group I, 0-100 acres; Group II, 101-1000 acres; Group III, greater than 1000 acres.

The multiple regression analysis program was run assuming Model B for each group. It was noticed that R^2 decreased greatly in each case compared to the R^2 obtained when all watersheds were combined. Therefore, all the observations combined explained more variability than segregating on an area basis.

Coding

The magnitude of some of the independent variables, like area, was large compared to the other variables, such as total rainfall. The possibility that the variables with large numbers might affect or dominate the variables with small numbers was suspect. Therefore, a coding process was implemented by dividing each variable by a multiple of ten so that the coded values had the same order of magnitude as the smaller variables. The multiple regression program was run, but coding did not improve the R^2. Therefore, the variables in the original form were used in the final equations.

Co-axial Curves

The expressions for the peak runoff and total volume of runoff were developed using 393 storms, both for urban and rural watersheds. Co-axial curves are developed based on Equations 26 and 27.

The eight independent variables in Equation 26 were divided into three groups as follows:

$$W_1 = \frac{A^{0.738} S^{0.206}}{L^{0.042}} \quad . \; . \; . \; . \; . \; . \quad (35)$$

$$S_1 = \frac{P^{1.016} P_{30}^{0.179}}{D^{0.26}} \quad . \; . \; . \; . \; . \; . \quad (36)$$

$$U_1 = \frac{1}{c_f^{0.797} \phi^{1.28}} \quad . \; . \; . \; . \; . \; . \quad (37)$$

The dependent variable, Q_p, then took the form:

$$Q_p = 0.777 \; W_1 S_1 U_1 \quad . \; . \; . \; . \; . \; . \quad (38)$$

The value of Q_p can be found from Figures 2, 3, 4, and 5. The use of these figures is illustrated by Table 12.

Similarly, the eight independent variables in Equation 27 were grouped as follows:

$$W_2 = \frac{A^{1.109} S^{0.036}}{L^{0.272}} \quad . \; . \; . \; . \; . \; . \quad (39)$$

$$S_2 = \frac{P^{1.248}}{D^{0.076} P_{30}^{0.187}} \quad . \; . \; . \; . \; . \; . \quad (40)$$

$$U_2 = \frac{\phi^{1.164}}{c_f^{2.209}} \quad . \; . \; . \; . \; . \; . \; . \; . \quad (41)$$

The dependent variable, Q_T, then took the form:

$$Q_T = 0.00882 \; W_2 S_2 U_2 \quad . \; . \; . \; . \; . \; . \quad (42)$$

Co-axial curves for Equations 39, 40, and 41 are plotted in Figures 6, 7, and 8, respectively. By following the arrows shown in these figures, it is possible to find the value of the independent parameters required for the solution of Equation 42 (Figure 9). Using the same example as previously cited, the values of $W_2 = 4000$, $S_2 = 3.5$, and $U_2 = 1.35$ are obtained. Now, entering Figure 9 with these values, the value of Q_T is found to be 170 acre feet.

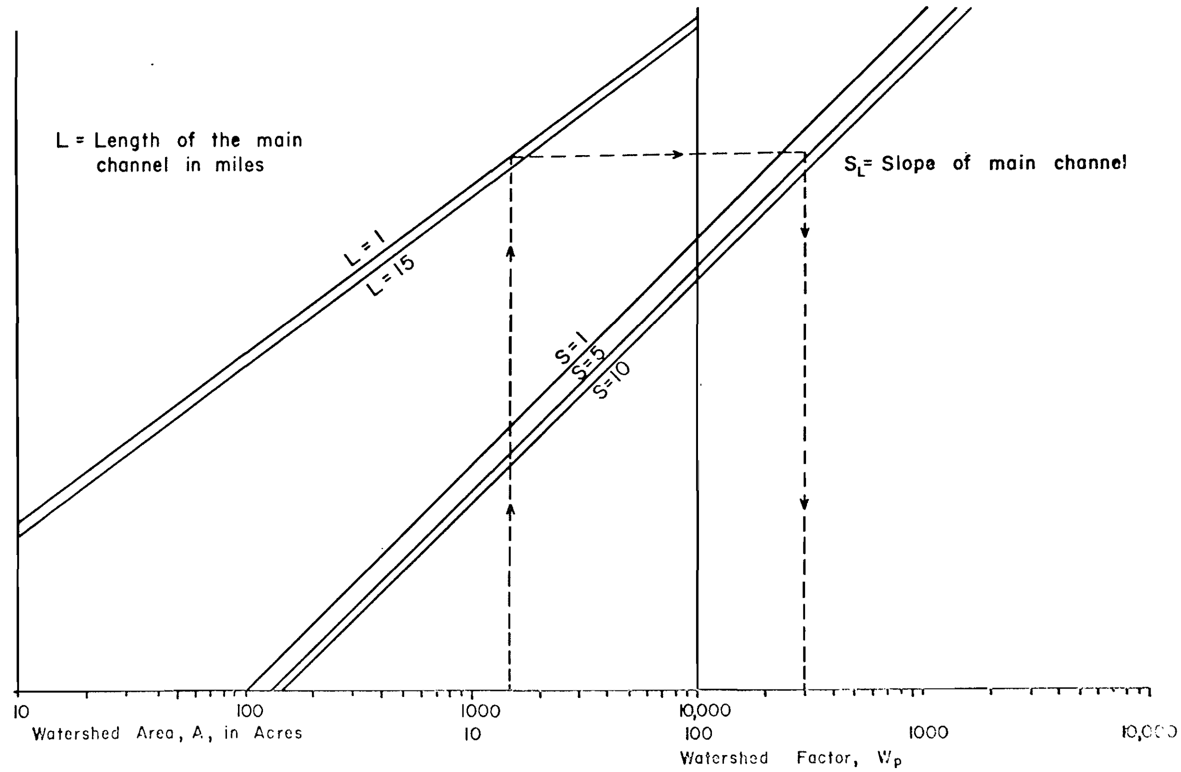

Figure 2. Nomograph solution of Equation 35 for estimating peak discharge rates.

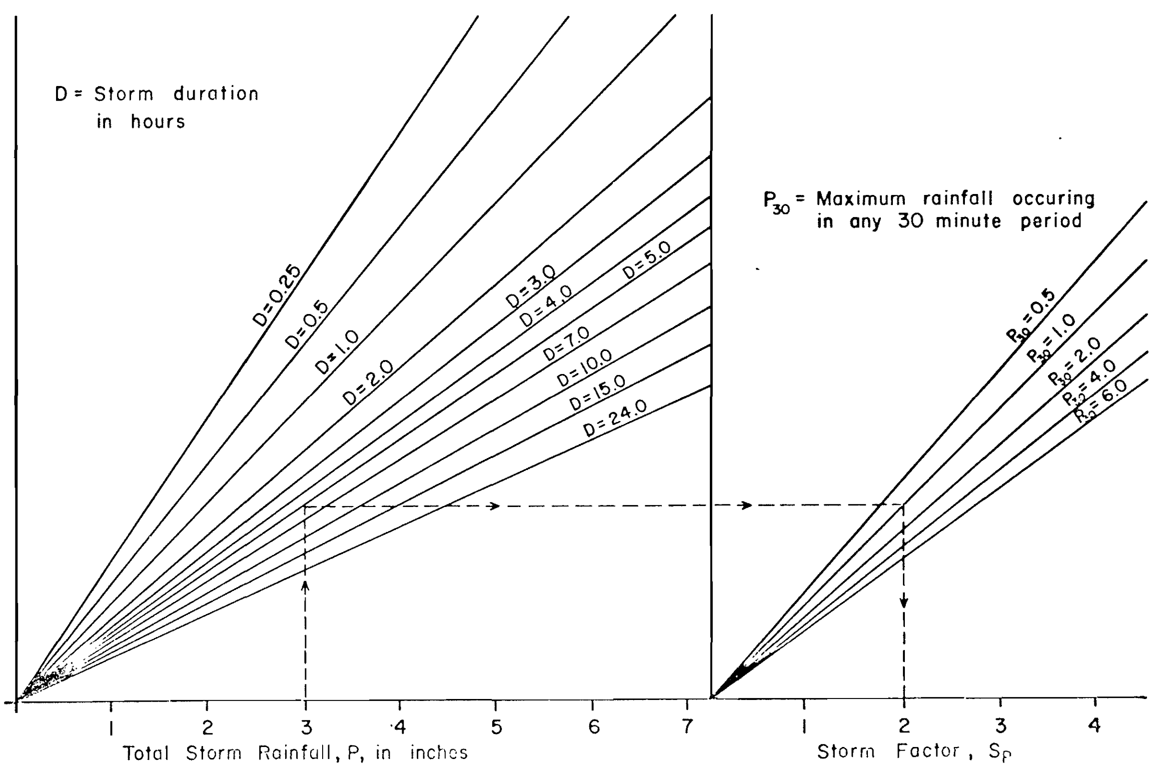

Figure 3. Nomograph solution of Equation 36 for estimating peak discharge rates.

Table 12. Sample computation of peak discharge using nomograph charts.

Figure No.		
Figure 2	(1) watershed area, A = 1500 acres	
	(2) main channel length, L = 2 miles	
	(3) average slope of main channel, S = 5 percent	W_1 = 300
Figure 3	(1) total storm precipitation, P = 3 inches	
	(2) total storm duration, D = 5 hours	
	(3) accumulated precipitation 30-minutes from beginning of storm P_{30} = 1 inch	S_1 = 2.0
Figure 4	(1) impervious cover factor, c_f = 0.80	
	(2) watershed channelization factor, ϕ = 0.85 (See Table 2)	U_1 = 1.5
Figure 5	(1) W_1 = 300	
	U_1 = 1.5	
	S_1 = 2.0	Q_p = 700 cfs

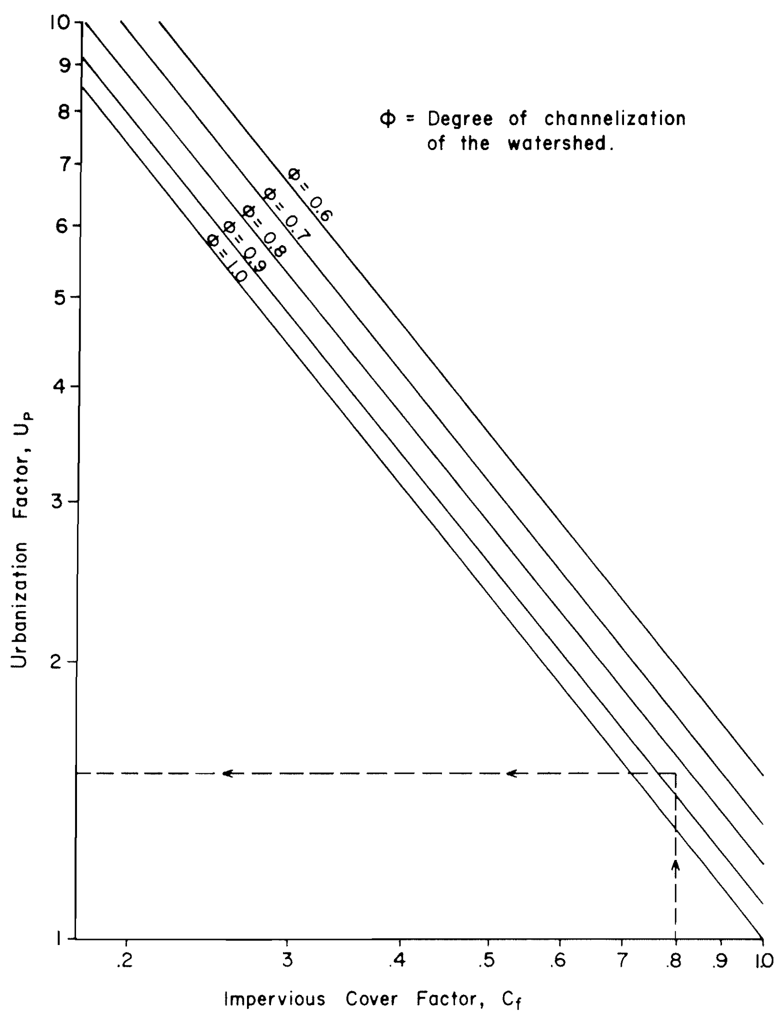

Figure 4. Nomograph solution of Equation 37 for estimating peak discharge.

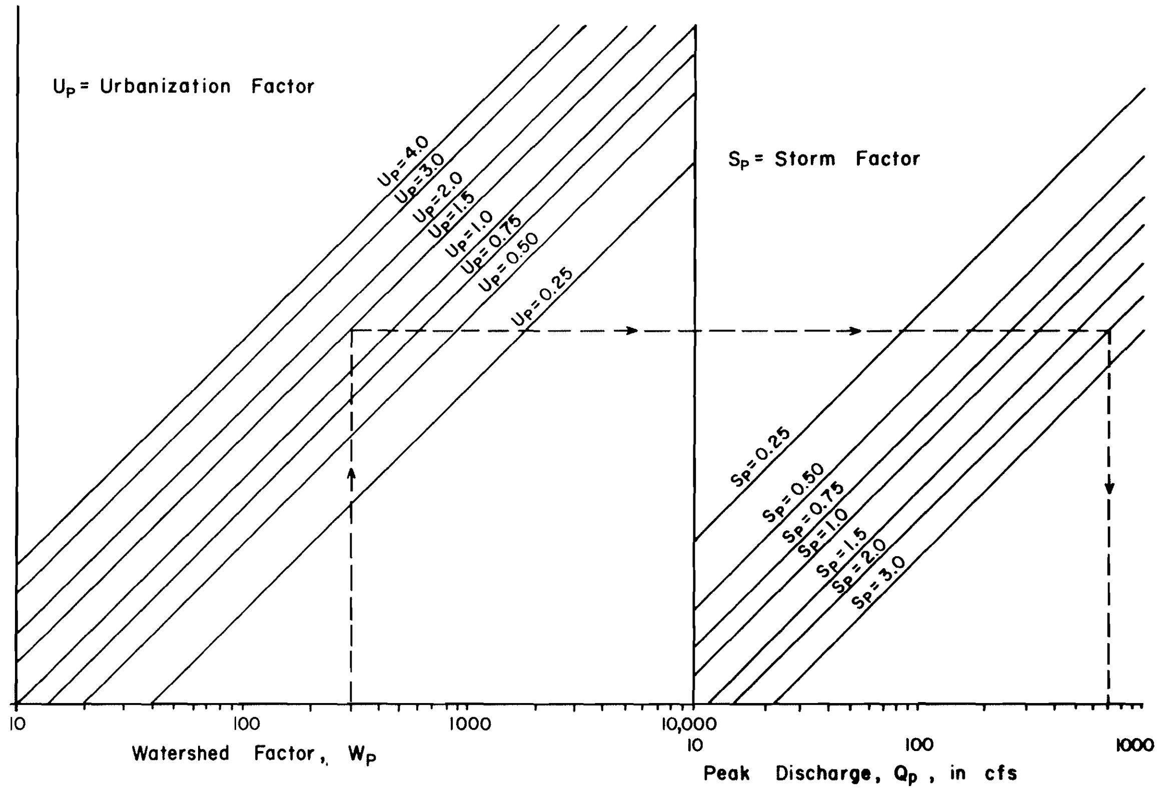

Figure 5. Nomograph solution of Equation 38 for estimating peak discharge rates.

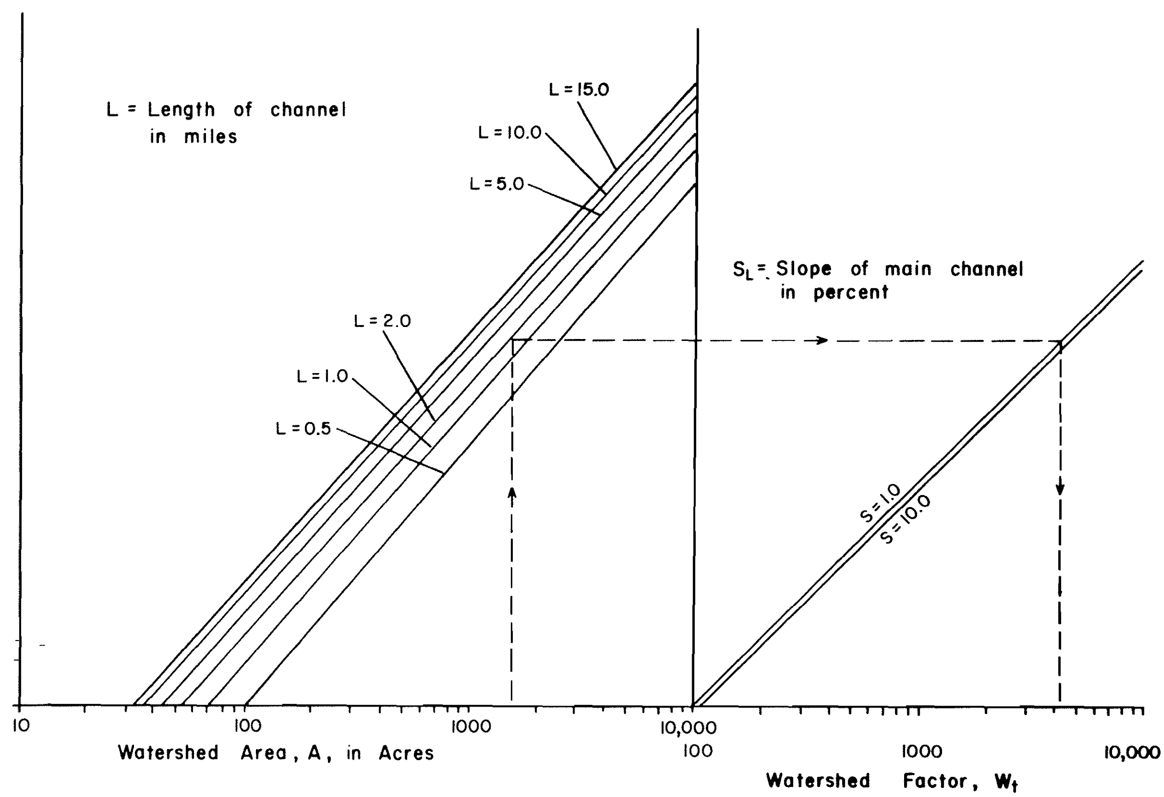

Figure 6. Nomograph solution of Equation 39 for estimating total runoff volume.

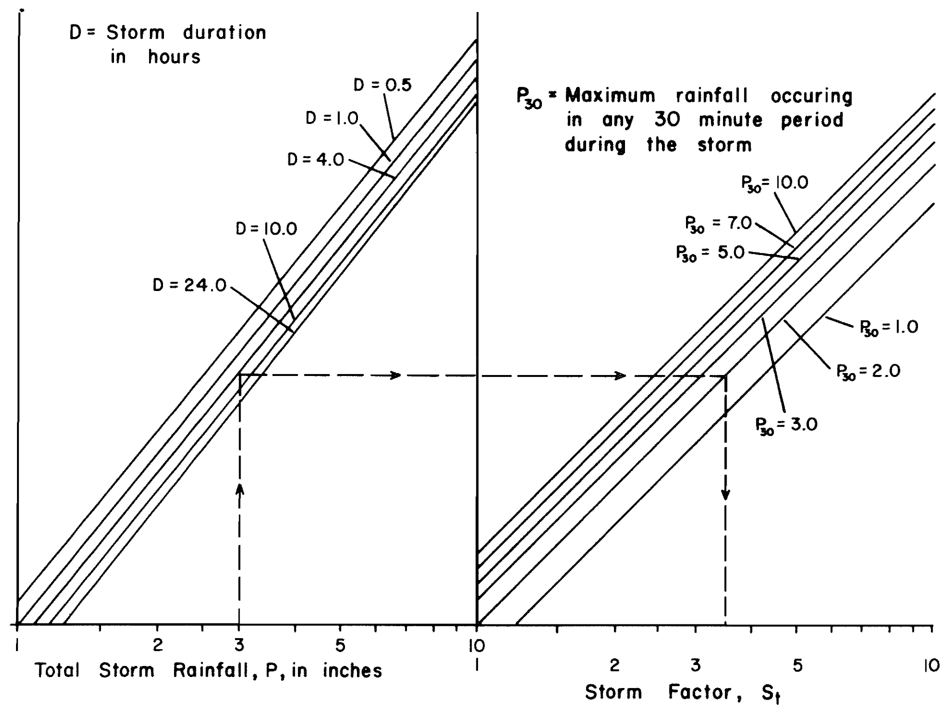

Figure 7. Nomograph solution of Equation 40 for estimating total runoff volume.

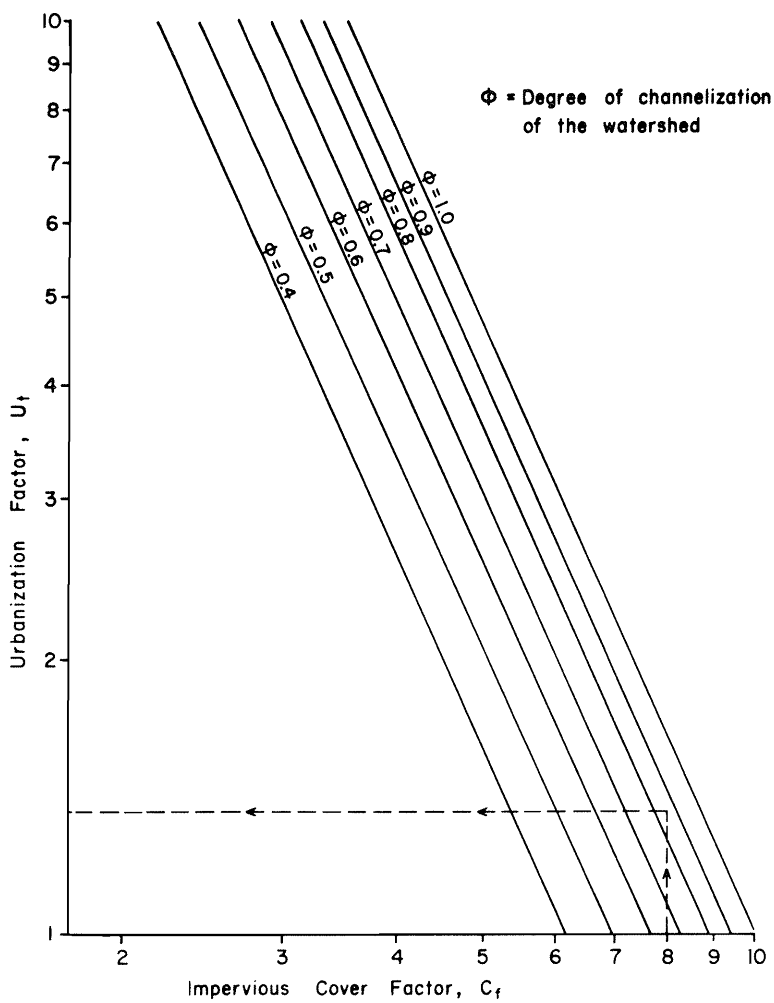

Figure 8. Nomograph solution of Equation 41 for estimating total runoff volume.

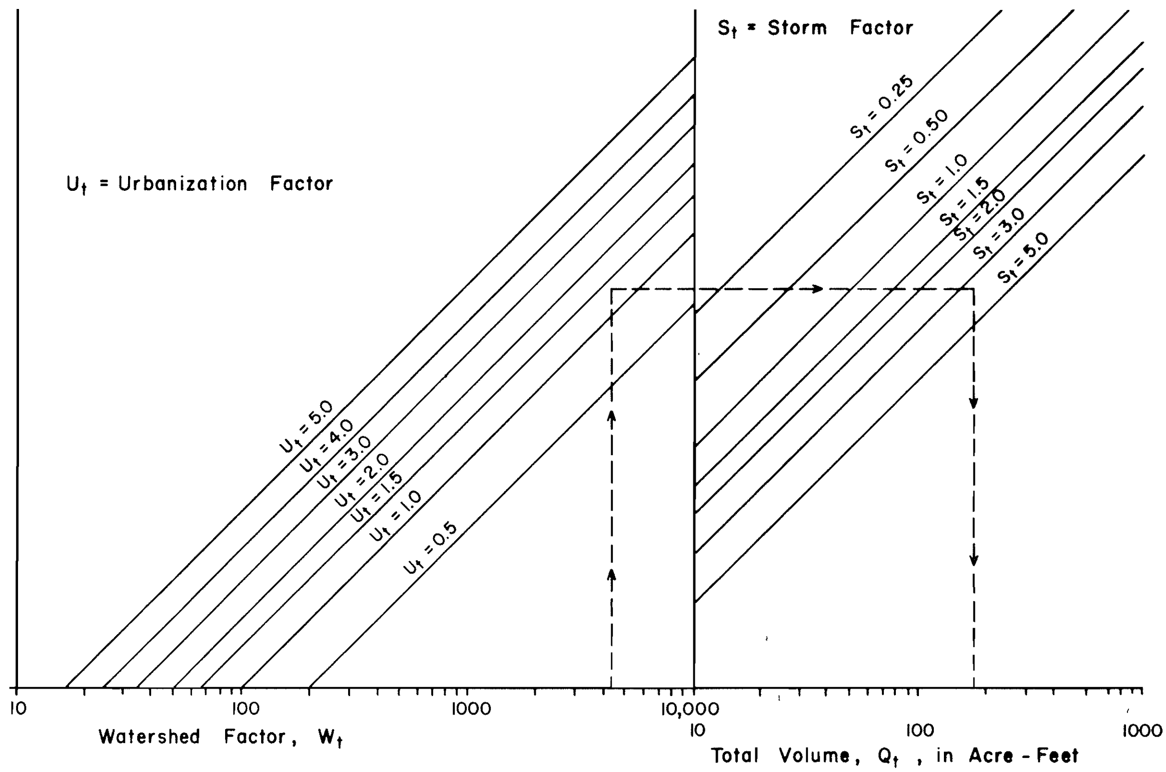

Figure 9. Nomograph solution of Equation 42 for estimating total runoff volume.

Verifications of Equations on Runoff from Boneyard Creek Watershed Urbana, Illinois

Rainfall-runoff data were collected for 29 storms occurring in Boneyard Creek Watershed from 1956 to 1966. Data on accumulated rainfall in inches were given with time for each storm. Total rainfall, P, in inches, maximum 30-minute rainfall, P_{30}, in inches, and duration, D, in hours were calculated for each storm. Area, slope, and length of the main channel were measured from the map provided by the U. S. Geological Survey, Washington, D. C. This watershed is comprised mainly of the city of Champaign, Illinois, which is highly urbanized with 48 percent impervious cover. In the developed equations, the factor $c_f = (1 - .48) = 0.52$. As the watershed has extensive channel improvement and storm sewer systems, the value of ¢ was taken as 0.6. Data on runoff were available in the form of gage height with time at an interval of 10-minutes to 30-minutes. A rating table was provided by the U. S. Geological Survey, Washington, D. C., which gives the discharge in cfs with gage height. Discharge in cfs was compared to time for each storm, and peak discharge in cfs was recorded. Total volume of runoff in acre feet was calculated for each storm. All data collected and reduced are presented in Table 13.

Peak discharge prediction

A computer program was written to solve Equation 26. All the independent variables are presented in Table 13 for each storm. The predicted values of Q_p in cfs are reproduced in Table 14. The relationship between the observed and predicted values is shown in Figure 10. A simple regression analysis was made between Q_p (predicted) versus Q_p (observed). A correlation coefficient of 0.9179 was found. A linear relation was found with the following equation:

$$\hat{Y} = -7.675 + 0.9726X \quad . \quad . \quad . \quad . \quad (43)$$

in which

$\hat{Y} = Q_p$ (predicted)

$X = Q_p$ (observed)

Total volume of runoff prediction

The computer program was used to solve Equation 27, giving value of Q_T for each storm. The values of independent parameters for each storm are given in Table 13. Table 15 compares the values of Q_T (predicted) and Q_T (observed). Figure 11 shows a linear regression relationship between the predicted and observed values of the total volume of runoff:

$$Y = -6.638 + 1.0224X \quad . \quad . \quad . \quad . \quad (44)$$

in which

$Y = Q_T$ (predicted)

$X = Q_T$ (observed)

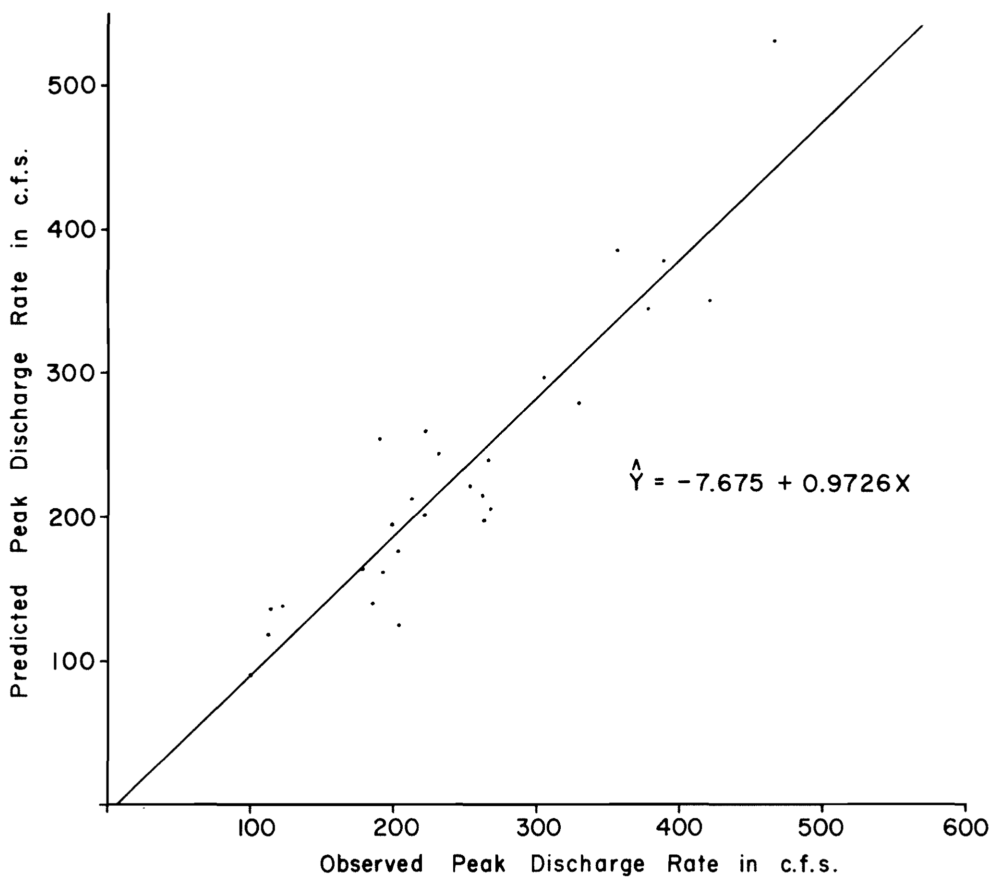

Figure 10. A comparison between observed and predicted peak discharge rates from the Boneyard Creek watershed, Urbana, Illinois.

Table 13. Data from Boneyard Creek watershed.

Date	D (hrs)	p (in)	P_{30} (in)	Q_p (cfs)	Q_T(ac-ft)
10-26-60	2.80	0.65	0.35	178	18.47
11-15-60	3.00	0.84	0.28	223	35.39
11-28-60	0.60	0.36	0.35	115	14.76
03-04-61	0.70	0.65	0.60	223	30.63
06-06-61	2.03	2.08	1.30	477	86.35
09-23-61	1.40	0.39	0.35	115	14.09
05-10-62	2.60	0.64	0.50	204	32.41
05-26-62	1.60	0.47	0.41	185	12.64
05-27-62	1.56	0.47	0.34	122	17.08
07-11-62	3.30	0.64	0.45	192	25.92
07-13-62	2.60	0.86	0.52	266	39.15
08-21-62	2.00	0.71	0.69	253	25.72
09-03-62	2.30	0.63	0.35	219	25.75
06-10-63	1.70	0.86	0.65	329	37.06
07-19-63	1.56	1.12	0.07	388	49.29
08-28-63	1.40	1.08	0.82	355	51.85
03-08-64	1.10	0.65	0.40	261	30.05
04-18-64	1.20	0.40	0.35	204	20.89
04-19-64	7.40	1.12	0.27	211	63.99
04-19-64	1.30	0.61	0.39	263	33.91
04-20-64	1.30	2.81	0.55	465	269.08
06-14-64	1.50	0.60	0.57	199	17.53
05-25-65	1.40	1.04	0.56	377	56.87
07-02-65	1.10	1.91	0.91	460	95.95
08-25-65	2.70	2.10	1.24	600	78.43
09-14-65	2.10	0.74	0.37	266	43.20
04-20-66	3.70	1.15	0.55	304	57.56
06-27-66	2.60	0.90	0.45	231	29.81
08-18-66	3.20	4.27	0.65	420	58.49

Area, A, = 2100.0 acres

Slope, S, = 0.30 percent

Length of the main channel, L, = 1.93 miles

Degree of channelization, ϕ, = 0.60

(1 - percentage impervious cover), c_f, = .52

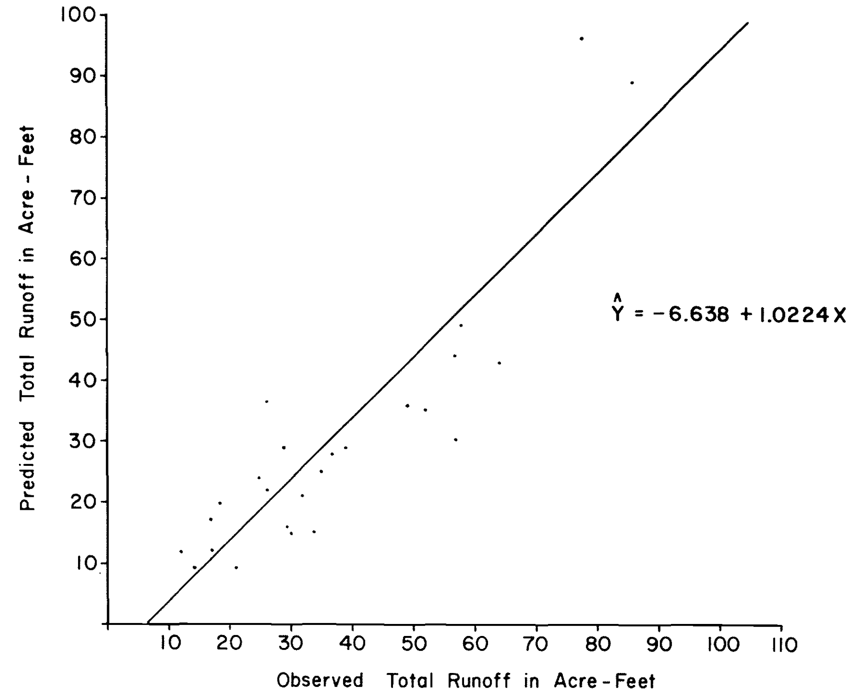

Figure 11. Comparison between observed and predicted total runoff from the Boneyard Creek watershed, Urbana, Illinois.

Table 14. A comparison between predicted and observed peak runoff rates from Boneyard Creek watershed, Urbana, Illinois.

Date	Q_p (predicted) (cfs)	Q_p (observed) (cfs)
10-26-60	164.60	178.00
11-15-60	201.58	223.00
11-28-60	134.79	115.00
03-04-61	259.94	223.00
09-23-61	117.30	115.00
05-10-62	176.07	204.00
05-26-62	140.88	185.00
05-27-62	137.14	122.00
07-11-62	162.40	192.00
07-13-62	239.39	266.00
08-21-62	221.90	253.00
09-03-62	253.69	190.00
06-10-63	278.25	329.00
07-19-63	377.10	388.00
08-28-63	384.53	355.00
03-08-64	214.94	261.00
04-18-64	125.28	204.00
04-19-64	212.13	211.00
04-19-64	197.07	263.00
04-20-64	529.88	465.00
06-14-64	194.77	199.00
05-25-65	345.65	377.00
09-14-65	204.39	266.00
04-20-66	296.38	304.00
06-27-66	244.31	321.00
08-18-66	350.78	420.00

Table 15. A comparison between predicted and observed total runoff volumes from Boneyard Creek watershed, Urbana, Illinois.

Date	Q_t (predicted) (ac-ft)	Q_t (observed) (ac-ft)
10-26-60	19.89	18.47
11-15-60	24.69	35.39
03-04-61	15.32	30.63
06-06-61	89.14	86.35
09-23-61	9.51	14.09
05-10-62	21.36	32.41
05-26-62	12.60	12.64
05-27-62	11.80	17.08
07-11-62	22.26	25.92
07-13-62	29.29	39.15
08-21-62	24.21	25.72
09-03-62	36.93	25.75
06-10-63	27.52	37.06
07-19-63	35.97	49.29
08-28-63	35.19	51.85
03-08-64	15.54	30.05
04-18-64	9.34	20.89
04-19-64	43.32	63.99
04-19-64	15.21	33.91
06-14-64	17.57	17.53
05-25-65	30.10	56.87
07-02-65	60.58	95.95
08-25-65	96.87	78.43
04-20-66	44.77	57.56
06-27-66	29.35	29.81
08-18-66	49 91	58.49

CONCLUSIONS AND RECOMMENDATIONS

Conclusions

1. Multiple regression equations are developed for peak rate of runoff and total volume of runoff. The expressions can be applied both to the urban and rural watersheds.

2. Area of the watershed explains the maximum variability in the model. Next in importance is the total amount of rainfall.

3. Co-axial curves present easy solution of the equations developed.

4. Grouping of observations on an area basis did not improve the model.

Recommendations

1. That further studies be undertaken involving other independent parameters, such as the following: (i) soil type, (ii) antecedent rainfall and snowfall, (iii) length of storm drains and sewers, and (iv) diameter of sewers and width of drains.

2. That the model be further generalized by testing it with data from widely diverse regions of this country and from other parts of the world.

3. That other mathematical models be studied to test their ability to represent runoff characteristics of prototype watersheds. As indicated by the study reported herein, it is possible to group all independent parameters into one of three general categories, namely watershed factors, storm factors, and urbanization factors. This approach simplifies the multi-variate analysis and facilitates the testing of a wide range of models.

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APPENDIXES

Appendix A
Flood Formulas

A list of notations used in the following formulas which may differ from those used in original presentation is given. Information concerning the original development of the formula and its background may be obtained from references. When the units are different than given in the list, they will be specified under each individual case.

A = drainage area in acres

D = drainage area in square miles

S = slope of drainage area in feet per thousand feet

G = geographical factor

Simple flood formulas

1. Kuichling formulas (1901)

$$q = \frac{44,000}{D + 170} + 20 \text{ --- for frequent floods} \quad \ldots \quad (A1)$$

$$q = \frac{127,000}{D + 370} + 7.4 \text{ --- for rare floods} \quad \ldots \quad (A2)$$

in which

q = discharge in cfs per square mile

D = drainage area in square miles

These formulas apply to drainage areas larger than 100 square miles. For drainage areas less than 100 square miles, the corresponding formulas are

$$q = \frac{25,000}{D + 125} + 15 \text{ ---} \quad \ldots \quad (A3)$$

and

$$q = \frac{35,000}{D + 32} + 10 \quad \ldots \quad (A4)$$

2. Lauterberg Formula (Kuichling, 1901)

$$Q = A\left(\frac{0.96}{6 + 0.0000039A} + 0.0008275\right) . \quad (A5)$$

or

$$Q = D\left(\frac{615}{6 + 0.0025D} + 0.53\right) \quad \ldots \quad (A6)$$

The formula was developed from floods due to continuous heavy rain of three to four days duration at an average rate of two inches per day.

3. Italian Formulas (Kuichling, 1901)

$$(a)\ Q = \frac{71.8A}{7.87 + \sqrt{A}} \quad \ldots \quad (A7)$$

or

$$Q = \frac{1.8.9D}{0.311 + \sqrt{D}} \quad \ldots \quad (A8)$$

$$(b)\ Q = \frac{103.0A}{7.87 + \sqrt{A}} \quad \ldots \quad (A9)$$

or

$$Q = \frac{2,600D}{0.311 + \sqrt{D}} \quad \ldots \quad (A10)$$

The first formula was developed for northern Italy and the second formula for small brooks in the same region.

4. The Murphy and others formula (1905)

$$Q = \left(\frac{46,790}{A + 205,000} + \frac{1}{42.7}\right) A \quad \ldots \quad (A11)$$

or

$$Q = \left(\frac{46,700}{D + 320} + 15\right) D \quad \ldots \quad (A12)$$

This formula was developed for streams of the northeastern United States from which Murphy had collected the data.

5. The Frizell formula (1905)

$$Q = 61.3\,A^{0.5} \quad \ldots\ldots\ldots \quad (A13)$$

or

$$Q = 1,550\,D^{0.5} \quad \ldots\ldots\ldots \quad (A14)$$

This formula is converted from the original for $q = 17.35\sqrt{8006/D}$ for maximum flood rate in cfs per square mile on the Connecticut River. The general form is

$$q = q_1 \sqrt{D_1/D} \quad \ldots\ldots\ldots \quad (A15)$$

in which q_1 is the observed maximum flood rate in cfs per square mile and D_1 is the corresponding drainage area in square miles.

6. C. B. and Q. Railroad formula (Bremner, 1906)

$$Q = \frac{59.2\,A}{37.9 + \sqrt{A}} \quad \ldots\ldots\ldots \quad (A16)$$

or

$$Q = \frac{3,000\,D}{3 + 2\sqrt{D}} \quad \ldots\ldots\ldots \quad (A17)$$

This formula was used for culvert design by the Chicago, Burlington, and Quincy Railway Company.

7. The Cooley formula (Bremner, 1906)

$$Q = 2.43\,A^{2/3} \quad 180\,D^{2/3} \quad \ldots \quad (A18)$$

8. The El Paso and S. W. Railway formula (Report, 1901)

$$Q = 60A^{0.5} \quad \ldots\ldots\ldots \quad (A19)$$

This is practically the same formula developed by Frizell.

9. The Gray formula (Report, 1911)

$$Q = 0.049A^{1.75} \quad \ldots\ldots\ldots \quad (A20)$$

or

$$Q = 3,770D^{1.75} \quad \ldots\ldots\ldots \quad (A21)$$

The original form is $Q = 5.89D^{3/4}$, where

Q = discharge in cfs per acre and

D = drainage area in square miles.

10. The New Kuichling formula (1914)

$$Q = \frac{0.065A\,(396,800 + A)}{15,360 + A} \quad \ldots \quad (A22)$$

or

$$Q = \frac{41.6D\,(620 + D)}{24 + D} \quad \ldots \quad (A23)$$

in which

Q = maximum discharge

Kuichling said that this new formula applies to river basins in the Southern Atlantic States, and it is based on the greatest observed discharges of the Potomac River at Point of Rocks, Maryland, New River at Radford, Virginia, the Catawba River at Rock Hill, North Carolina, Can Creek at Bakersville, North Carolina, and numerous other streams which exhibit somewhat smaller rates of discharge. It may be regarded as applicable to mountainous and hilly drainage basins having areas of not more than 10,000 square miles in that part of the country.

11. The Elliot formulas (1919)

(a). For swamps and wet lands in Northeastern Arkansas:

$$Q = (\frac{24}{\sqrt{D}} + 6)\ D \quad . . . . . . (A24)$$

or

$$Q = (\frac{0.948}{\sqrt{A}} + 0.00937)\ A \quad . . . (A25)$$

(b). For swamps and other wet lands of the Upper Mississippi Valley:

$$Q = (\frac{20}{\sqrt{D}} + 3.63)\ D \quad . . . . . . (A26)$$

or

$$Q = (\frac{0.792}{\sqrt{A}} + 0.00568)\ A \quad . . . (A27)$$

(c). For satisfactory drainage areas in North Central Illinois:

$$Q = (\frac{673}{19.2 + \sqrt{D}} - 11.3)\ D \quad . . . (A28)$$

or

$$Q = (\frac{26.6}{468 + \sqrt{A}} - 0.0177)\ A \quad . . . (A29)$$

These formulas were used for rough approximations. The results should be checked for local conditions. The first formula was used to compute the discharge from the low flat alluvial lands in preliminary drainage investigation in Northeastern Arkansas. The results may be increased 50 percent for the more rolling and less sandy land in the east part of the Mississippi County, 100 percent for the clay soils east of Crowleys Ridge and 200 percent for the slopes of Crowleys Ridge. The second formula specifies that soils are absorptive and easily drained. The third formula was given to areas of 200 square miles or less.

12. Dickens formula (Gurtu, 1923-1924)

$$Q = c\ A^{0.75} \quad . . . . (A30)$$

$$Q = C_1\ D^{0.75} \quad . . . . (A31)$$

in which

C = 1.56 or C_1 = 200 for Madras Presidency, India

C = 3.91 or C_1 = 500 for Central Province, India

C = 6.45 or C_1 = 825 for Bengal and Bihar, India

C = 9.37 or C_1 = 1,200 for Upper Kaveri, India

C = 17.2 or C_1 = 2,200 for Gadamatti, India

C = 6.6 or C_1 = 850 for average conditions

13. The Beale formula (Hearn, 1923-1924)

$$Q = C\ A^{0.75} \quad . . . . (A32)$$

in which

C = 1,600 for unforested area

C = 1,400 to 1,000 for forested area in the central provinces of India

This is an adaptation of the Dickens formula to suit the conditions of the Western Ghates in the Bombay Presidency from the observed discharges on the Nira Canal.

14. The Nagler formula (1928)

$$Q \quad 2.84\ A^{2/3} \quad . . . . (A33)$$

or

$$Q \quad 210\ D^{2/3} \quad . . . . (A34)$$

This formula was developed for the 50-year flood to be expected in Iowa streams.

15. The Williams formula (Williams, 1937)

$$Q = \frac{C}{D_n} \quad \text{. . . . . (A35)}$$

in which the coefficients C and n are as follows:

Locality	Co-efficients	Drainage area: Less than 10 square miles	Drainage area: 10 - 20,000 square miles
Northeast U.S.	C	1,480	2.400
	n	0.75	0.54
Mississippi Valley	C	2,500	4,800
	n	0.75	0.47
Rocky Mountains	C	1,900	3,600
	n	0.75	0.45
Pacific Coast, USA	C	1,625	2,700
	n	0.75	0.53
Western India	C	2,700	4,600
	n	0.75	0.52
North-East India	C	1,400	1,700
	n	0.75	0.05

The coefficients for the United States are based on flood records listed in the paper "Flood Flow Characteristics" by C. S. Harvis (1926). For Western India, they are based on records of floods in the Bombay Presidency. For Northeast India, they are based on papers presented before the Institution of Civil Engineers by Sir Gordon Hearn (1923).

16. The Metcalf and Eddy formula (1941)

$$Q = 3.95\, A^{0.73} \quad \text{. . . . . . . (A36)}$$

$$Q = 440\, D^{0.73} \quad \text{. . . . . . . (A37)}$$

This formula was developed to suit drainage areas of 6,400 to 160,000 acres near Louisville, Kentucky, in connection with studies for the flood water discharge of Beargrass Creek, Louisville, Kentucky.

17. The Ryves formula (Sharma, 1944)

$$Q = C\, A^{2/3} \quad \text{. . . . . (A38)}$$

in which

C = local coefficients depending upon the rainfall, soil, and slope of the district
= 9.1 for Upper India

This formula is used extensively in India.

18. The USGS Formulas (Linsley et al. 1949)

The following formulas were developed from separate enveloping curves of peakflow for each of the 14 regions used by the U. S. Geological Survey for publication of stream flow data.

No.	Region	Formula
1	North Atlantic Slope	$Q = 190\, A^{0.5}$
2	South Atlantic and Eastern Gulf of Mexico Drainage	$Q = 250\, A^{0.5}$
3	Ohio River Basin	$Q = 230\, A^{0.5}$
4	St. Lawrence River Basin	$Q = 1,020\, A^{0.35}$
5	Hudson Bay and Upper Mississippi Drainage	$Q = 230\, A^{0.43}$
6	Missouri River Basin	$Q = 130\, A^{0.5}$
7	Lower Mississippi River Basin	$Q = 250\, A^{0.5}$
8	Western Gulf of Mexico Drainage	$Q = 34.5\, A^{0.77}$ (below 2,550,000 acres) $Q = 104,000\, A^{0.13}$ (above 2,550,000 acres)

No.	Region	Formula
9	Colorado River Basin	$Q = 99\ A^{0.5}$
10	The Great Basin	$Q = 26\ A^{0.6}$
11	Pacific Slope Basin in California	$Q = 200\ A^{0.5}$
12	Pacific Slope Basin in Washington and Upper Columbia River Basin	$Q = 180\ A^{0.5}$
13	Snake River Basin	$Q = 0.51\ A^{0.83}$
14	Pacific Slope Basins in Oregon and Lower Columbia River Basin	$Q = 229\ A^{0.5}$

19. The Morgan Engineering Company formulas (Mead, 1950)

$$Q = \left(\frac{1.8}{\sqrt{A}} + \frac{1}{80}\right) A \quad . . . . . \text{(A39)}$$

$$Q = \left(\frac{1.1}{\sqrt{A}} + \frac{1}{88.8}\right) A \quad . . . . . \text{(A40)}$$

The first formula was used for the Cache River Drainage District. The second formula was used for Mississippi County, Arkansas. Morgan Engineering Company of Memphis, Tennessee, used these formulas in their design of most drainage structures.

20. The Bahadur formula (Priyani, 1957)

$$Q = C\ D^{(0.92 - (1/14) \log D)} \quad . . . \text{(A41)}$$

in which

C = 1,600 to 2,000

The formula was developed by Sir C. C. Inglis for fan-shaped drainage basins in Bombay State, India.

Complicated discharge formulas

1. The Adams formula (1880)

$$Q = C\ A\ I\ \sqrt[12]{\frac{S}{A^2 I^2}} \quad . . . . . \text{(A42)}$$

in which

C = 1.035

I = 1.0 or maximum intensity of rainfall in inches per hour

S = slope in feet per thousand feet

This formula was developed from the fundamental expression for a circular conduit flowing full, and the assumption that one-half of the precipitation in inches per hour will reach the sewer at the time of maximum discharge.

2. The Craig formula (1884-1885)

$$Q = 440\ C\ W\ \ln\left(\frac{8L^2}{W}\right) \quad . . . . . \text{(A43)}$$

in which

L = mean length of the drainage area in miles

W = mean width of drainage area in miles

$C = C_1\ V\ R$

in which

C_1 = coefficient of discharge

V = velocity towards the culvert in feet per second

R = depth of rainfall in inches

This formula is based on Indian records and value of C generally varies from 0.68 to 1.95.

3. The McMath formula (1887)

$$Q = C\ A\ I\ \sqrt[5]{\frac{S}{A}} \quad . . . . \text{(A44)}$$

in which

C = 0.20 for rural sections
= 0.30 for macadamized streets
= 0.75 for paved streets
= 0.75 for St. Louis, Missouri

I = 1.9 to 2.75 for maximum intensity of rainfall in inches per hour. The latter value was used for St. Louis

S = slope of ground surface in feet per thousand. A value of .015 is recommended for St. Louis.

This formula was proposed for St. Louis, Missouri.

4. The Hawksley formula (Kuichling, 1892-1893)

$$Q = C A I \sqrt[4]{\frac{S}{AI}} \quad . . . . . . (A45)$$

in which

C = 0.7

I = 1.0 or maximum intensity of rainfall in inches per hour.

5. The Chamler formula (1898)

$$Q = 5 C I A^{0.75} \quad . . . . (A46)$$

in which

C = coefficient of surface drainage, giving the proportion of rainfall that may be expected to flow off the surface

I = anticipated greatest rainfall intensity in inches per hour for a duration equal to the time of concentration

This formula was tested by Chamler on streams in New South Wales along the Cootamundra-Gundagai Railway having drainage areas of from 200 acres to 400 square miles.

6. The Gregory formulas (1907)

$$Q = C I S^{0.186} A^{0.86} \quad . . . . . (A47)$$

in which

CI = 2.8 for impervious surface

$$Q = 105 C L 84 (5 \sqrt{AS^2} + 2S) \quad . . . (A48)$$

in which

C = 0.10 to 0.54

This formula was developed for use in New York in 1907.

7. The Gregory and Hering formula (1907)

$$Q = CI A^{0.833} S^{0.27} \quad . . . . . (A49)$$

This formula was deduced by Charles E. Gregory in 1907 from diagrams of runoff to be expected in New York City prepared in 1889 by Rundolph Hering. The value of CI = 1.02 for suburban areas to 1.64 for metropolitan areas.

8. The Possenti formula (Fuller, 1914)

$$Q = C \frac{R}{L} (A_2 + \frac{A_1}{3}) \quad . . . . . . (A50)$$

in which

C = coefficient with an average of 1.72

A_1 = flat areas in acres

A_2 = hilly areas in acres

R = depth of 24-hour rainfall in inches

L = length of stream from its source to the point of observation in miles

This formula was found satisfactory for mountain streams of moderate size in the Appennines.

9. The Grunsky formula -- A (1922)

For maximum urban storm-water flow

$$Q = \frac{S C I A}{\sqrt{t}} \quad . . . . (A51)$$

For maximum stream flow from large areas

$$Q = \frac{3,200 C I A}{\sqrt{t}} \quad . . . . . . (A52)$$

For general applications

$$Q = \frac{C_2 IA}{t^n} \quad . . . . (A53)$$

in which

C = coefficient as function of time = $60/(60 + C_1 \sqrt[3]{t})$

I = maximum rainfall in one hour based on California record

t = critical time in minutes for continuance of rainfall

C_1 = 0.5 for impervious areas

C_1 = 5.0 for mountainous areas

C_1 = 20.0 for rolling country

C_1 = 50.0 for flat country

C_1 = 250.0 for sandy regions

C_2 = 3,500 and n = 0.5 for impervious areas

C_2 = 3,300 and n = 0.6 for mountainous areas

C_2 = 3,000 and n = 0.7 for rolling country

C_2 = 2,100 and n = 0.75 for flat country

C_2 = 600 and n = 0.80 for sandy regions

This formula was based on California records.

10. The Walker formula (1922)

$$Q = \frac{C\ R\ D}{L\ 5/6} \qquad \text{(A54)}$$

in which

C = 4 to 30, being a maximum for drainage basins having impervious surfaces, little storage, steep slopes, little vegetation, direct alignment of waterways, etc., and minimum for pervious surfaces, much storage, flat areas, much vegetation, and waterways with irregular and meandering alignment. Most values of C range between 8 to 20 for average conditions. A general average of C is about 12.

R = mean, or normal, annual rainfall in inches over the entire basin

L = straight line distance in miles from point of discharge to center of gravity of the basin

11. The Lillie formula (1924)

$$Q = V\ R\ C\ \Sigma(OL) \qquad \text{(A55)}$$

in which

Q = discharge in cfs at the moment of peak flood

V = mean velocity in feet per second

R = 24 annual rainfall/15

C = 1.1 + log L

L = length of sectors of drainage area in miles

O = angle in degrees, at the discharge point, of the sections into which the catchment is divided. The sections are in fan shape having a common center meeting at the discharge point.

This formula was developed with reference to rivers in India.

12. Rhind formula (Hearn, 1924)

$$Q = \frac{C\ S\ R\ D^n}{L} \qquad \text{(A56)}$$

in which

C = coefficient depending on R/L

S = average fall in feet per mile of bed in a length of 3 miles above the point of discharge

R = greatest annual rainfall

L = greatest length of drainage basins

n = a variable index

13. The Switzer and Miller formula (1929)

$$Q = R\ C\ W^n \qquad \text{(A57)}$$

in which

Q = 24-hour flood in cfs

R = rainfall in inches

W = mean width of drainage basin in miles, obtained by dividing the area of drainage basin in square miles by the length of the main stream in miles

C = 80

n = 1.5

The formula is based on a study of 47 rivers in the United States. When Q is expressed for peak flows in cfs, then C = 135 and n = 1.4.

14. The Boston Society of Civil Engineers' formula (1930)

$$Q = C_1 R \sqrt{A} \qquad \text{(A58)}$$

in which

$C_1 = \sqrt{\frac{A}{t}}$ where t is the time in hours of the flood period

C_1 = 2.4 to 4 for flat streams with relatively large channel pondage

C_1 = 4 to 24 for ordinary conditions

C_1 = 20 to 40 for mountainous regions

R = total flood runoff, inches on drainage area

This formula gives the total runoff and is based on floods in New England. This formula is based upon a concept that peak flows tend to vary directly with the total volume of flood runoff.

15. Besson formula (1933)

$$Q = C A^n = R T G A^n \quad . . . . . (A59)$$

For any drainage area

$$Q_{max} = Q_r \frac{R_m C_1}{R_r C_2} \quad . . . . . . (A60)$$

in which

Q_{max} = maximum discharge

Q_r = recorded discharge

R_r = recorded one-day rainfall

C = coefficient equal to the product of the precipitation R in inches, to topographic factor T, and a factor G for ground surface conditions. C_1 is for maximum conditions and C_2 for recorded conditions.

n = exponent which has been given values varying from 0.5 to 0.83.

16. The Grunsky formula (1932)

$$Q_{max} = \frac{C C_1 I A}{t^n} \quad . . . . . . (A61)$$

in which

Q_{max} = maximum rate of discharge

t = time of concentration in hours

C = 0.586 and n = 3/4 for less than 0.33 hours

C = 0.782 and n = 1/2 for t greater than 0.33 hours and less than 64 hours

C = 1.562 and n = -2/3 for t greater than 64 hours

C_1 = $1/(1 + C_2\sqrt{t})$, where C_2 is a factor dependent on the surface conditions of the discharge basin

C_2 = 0.013 for impervious areas
= 0.25 for mountains
= 0.40 for rolling country
= 1.3 for flat country (ordinary soil)
= 6.5 for sandy regions

The values of C_1 were suggested for ordinary conditions in a temperate climate. They should be increased in localities where the ground may be frozen or waterlogged or where the maximum runoff occurs when heavy rain falls on snow.

17. The Kinnison and Colby formulas (1945)

$$Q = (0.000036\, s^{2.4} + 124) \frac{D^{0.95}}{p^{0.4} L^{0.7}}$$

for minor floods . . . . (A62)

$$Q = (0.0344\, s^{1.5} + 200) \frac{D^{0.85}}{L^{0.5}}$$

for major floods . . . . (A63)

$$Q = (0.0595\, s^{1.5} + 342) \frac{D^{0.95}}{L^{0.7}}$$

for rare floods . . . . (A64)

$$Q = (0.128\, s^{1.6} + 1,800) \frac{D^{0.90}}{L^{0.7}}$$

for maximum floods . . . . (A65)

in which

Q = the peak discharge in cfs

s = the median altitude of the drainage basin in feet above the outlet

P = the percentage that lake, pond, and reservoir surface is to the total drainage area

L = the average distance in miles in which runoff uniformly distributed over the basin must travel to the outlet

Their formulas were developed by USGS for Commonwealth of Massachusetts.

Appendix B

Computer Listings

Listing of MDCR (Multivariate Data Collection Revised) Program

```
// JOB MDCR          000120,M005
// OPTION CATAL
 PHASE MDCR,*
// EXEC FORTRAN
// ETC RCD
C   MULTIVARIATE DATA COLLECTION      REVISED
C   REX L. HURST
C   UTAH STATE UNIVERSITY
C   @@@@@@@@@CONTROL CARD@@@@@@@@@@@@@
C   NVI= NUMBER OF VARIABLES IN READ LIST
C   NVO= NUMBER OF VARIABLES AFTER MAKING TRANSFORMATIONS
C.  NT= NUMBER OF TRANSFORMATIONS TO MAKE
C   NCI= NUMBER OF CARDS TO CONTAIN DATA FORMAT
C   IN= INPUT DEVICE CODE       1= CARD READER
C   @@@@@@@@TRANSFORMATION DESCRIPTOR CARD@@@@@@@@
C   IT= TRANSFORMATION CODE
C   JJ= POSITION INTO WHICH THE RESULTANT TRANSFORMATION IS TO BE PUT
C   IA= SUBSCRIPT OF FIRST VARIABLE TO BE USED IN TRANSFORMATION
C   IB= SUBSCRIPT OF SECOND VARIABLE TO BE USED IN TRANSFORMATION
C   IC= SUBSCRIPT OF THIRD VARIABLE TO BE USED IN TRANSFORMATION
C   ID= SUBSCRIPT OF FOURTH VARIABLE TO BE USED IN TRANSFORMATION
C   CA= FORST CONSTANT USED IN TRANSFORMATION
C   CB= SECOND CONSTANT USED IN TRANSFORMATION
C   CC= THIRD CONSTANT USED IN TRANSFORMATION
      DOUBLE PRECISION A,Y,SUM,Z
      DIMENSIONA(70,70),X(70),Y(70),SUM(70),Z(70)
      NI=70
      IPR=3
      LUA=12
      LUB=13
      CALL TRNV(NVAR,NOBS,LUB)
      CALL DTAC(A,Y,SUM,X,NVAR,NOBS,LUB,LUA,NI)
      WRITE (IPR,100)
  100 FORMAT(//@ CORRELATION MATRIX@  )
      CALL DCORPL(A,1,NVAR,X,Z,NI)
      CALL EXIT
      END
      SUBROUTINE TRNV(NVO,NOBS,IO)
      INTEGER*2 IT,JJ,IA,IB,IC,ID
      DIMENSION IT(90),JJ(90),IA(90),IB(90),IC(90),ID(90),CA(90),CB(90),
     1IAA(20),NL(20),IZ(20,20),LA(32),LB(32),IFRST(12),ISCND(12),
     1CC(90),X(100),FMI(60),Z(8)
      IRD=1
      IPP=3
      IPH=2
      READ  (IRD,100) NVI,NVO,NT,NCI,IN,NF,NINT,(FMI(I),I=1,10)
      WRITE (IPR,101) NVI,NVO,NT,NCI,IN,NF,NINT,(FMI(I),I=1,10)
  101 FORMAT(1H1,7I3,19X,10A4)
  100 FORMAT(     7I3,19X,10A4)
      TPI=8.0*ATAN(1.0)
      IF(NF.EQ.0) GO TO 45
      DO 50 I=1,NF
      READ  (IRD,109) LA(I),IAA(I),KK,(IZ(I,J),J=1,KK)
      WRITE (IPR,110) LA(I),IAA(I),KK,(IZ(I,J),J=1,KK)
  110 FORMAT(4X,22I3)
  109 FORMAT(3X,22I2)
      LB(I)=LA(I)+KK-2
   50 NL(I)=KK
```

```
      IF(NINT.EQ.0) GO TO 45
      DO 51 I=1,NINT
      II=I+NF
      READ  (IRD,109)  LA(II),IFRST(I),ISCND(I)
      WRITE (IPR,110)  LA(II),IFRST(I),ISCND(I)
      N1=IFRST(I)
      N2=ISCND(I)
   51 LB(II)=LA(II)+(LB(N1)-LA(N1)+1)*(LB(N2)-LA(N2)+1)-1
   45 NCI=NCI*20
      IF(NT.EQ.0) GO TO 59
      DO 49 I=1,NT
      READ  (IRD,102) IT(I),JJ(I),IA(I),IB(I),IC(I),ID(I),CA(I),CB(I),
     1CC(I),(Z(J),J=1,8)
   49 WRITE (IPR,103) IT(I),JJ(I),IA(I),IB(I),IC(I),ID(I),CA(I),CB(I),
     1CC(I),(Z(J),J=1,8)
  103 FORMAT(1H ,6I3,3F11.4,2X,7A4,A2)
  102 FORMAT(     6I3,3F10.4,2X,7A4,A2)
   59 READ  (IRD,104)  (FMI(I),I=1,NCI)
      WRITE (IPR,105)  (FMI(I),I=1,NCI)
  105 FORMAT(1H ,20A4)
  104 FORMAT(     20A4)
      WRITE (IPR,108)
  108 FORMAT(/2 FIRST THREE OBSERVATIONS AFTER TRANSFORMATIONS )
      NOBS=0
      IF(IN.NE.IRD.AND.IN.NE.IPR.AND.IN.NE.IPH)  REWIND  IN
      IF(IO.EQ.IPR.OR.IO.EQ.IPH.OR.IO.EQ.IRD) GO TO 42
      REWIND IO
   42 READ  (IN,FMI,END=43)  (X(I),I=1,NVI)
      NOBS=NOBS+1
      IF(NF.EQ.0) GO TO 61
      DO 52 I=1,NF
      L=LA(I)
      N2=NL(I)
      N1=N2-1
      KA=IAA(I)
      IF(IFIX(X(KA)).NE.IZ(I,N2)) GO TO 53
      DO 55 K=1,N1
      Y(L)=-1
   55 L=L+1
      GO TO 52
   53 DO 56 K=1,N1
      IF(IFIX(X(KA)).EQ.IZ(I,K)) GO TO 57
      Y(L)=0
      GO TO 58
   57 Y(L)=1
   58 L=L+1
   52 CONTINUE
      IF(NINT.EQ.0) GO TO 61
      DO 60 I=1,NINT
      II=I+NF
      L=LB(II)
      N1=IFRST(I)
      N2=ISCND(I)
      N3=LA(N1)
      N4=LB(N1)
      N5=LA(N2)
      N6=LB(N2)
      DO 60 J=N3,N4

      DO 60 K=N5,N6
      Y(L)=X(J)*X(K)
   60 L=L+1
   61 IF(NT.EQ.0) GO TO 41
      DO 41 I=1,NT
      LX=IT(I)
      J=JJ(I)
      KA=IA(I)
      KB=IB(I)
      KC=IC(I)
      KD=ID(I)
```

```
      GO TO(1,2,3,4,5,6,7,8,9,10,11,12,13,14,15,16,17,18,19,20,21,22,23,
     124,25,26),LX
    1 X(J)=X(KA)
      GO TO 41
    2 X(J)=X(KA)*X(KB)
      GO TO 41
    3 X(J)=X(KA)*X(KB)*X(KC)
      GO TO 41
    4 X(J)=X(KA)*X(KB)*X(KC)*X(KD)
      GO TO 41
    5 X(J)=SQRT(X(KA))
      GO TO 41
    6 X(J)=ALOG(X(KA))
      GO TO 41
    7 X(J)=1.0/X(KA)
      GO TO 41
    8 X(J)=X(KA)-X(KB)
      GO TO 41
    9 X(J)=X(KA)/X(KB)
      GO TO 41
   10 X(J)=COS (TPI*CC(I)*(X(KA)-CA(I))/CB(I))
      GO TO 41
   11 X(J)=SIN (TPI*CC(I)*(X(KA)-CA(I))/CB(I))
      GO TO 41
   12 IF(X(KA).GT.0.0) GO TO 47
      X(J)=0.0
      GO TO 41
   47 IF(X(KA).LT.1.0) GO TO 48
      X(J)=90.0
      GO TO 41
   48 X(J)=57.29578*ATAN(SQRT(X(KA)/(1.0-X(KA))))
      GO TO 41
   13 X(J)=ALOG(X(KA)+1.0)
      GO TO 41
   14 X(J)=.5*ALOG((1.0+X(KA))/(1.0-X(KA)))
      GO TO 41
   15 X(J)=ALOG(X(KA)/(1.0+X(KA)))
      GO TO 41
   16 X(J)=SQRT(X(KA)+.5)
      GO TO 41
   17 X(J)=X(KA)+CA(I)
      GO TO 41
   18 X(J)=X(KA)+X(KB)
      GO TO 41
   19 X(J)=EXP (X(KA))
      GO TO 41
   20 X(J)=ABS(X(KA))
      GO TO 41
   21 X(J)=X(KA)**CA(I)
      GO TO 41
   22 IF(X(KA).GT.CA(I)) GO TO 46
      X(J)=0.0
      GO TO 41
   46 X(J)=X(KA)-CA(I)
      GO TO 41
   23 X(J)=X(KA)+X(KB)+X(KC)
      GO TO 41
   24 X(J)=X(KA)+X(KB)+X(KC)+X(KD)
      GO TO 41
   25 X(J)=X(KA)*CA(I)
      GO TO 41
   26 X(J)=X(KA)/CA(I)
   41 CONTINUE
      IF(NOBS.LE.3) WRITE (IPR,107) NOBS,(X(I),I=1,NVO)
  107 FORMAT(         I5,10F11.4/(5X,10F11.4))
      WRITE  (IO) (X(I),I=1,NVO)
      GO TO 42
   43 IF(IN.NE.IRD.AND.IN.NE.IPR.AND.IN.NE.IPH)  REWIND  IN
      IF(IO.EQ.IPR.OR.IO.EQ.IPH.OR.IO.EQ.IRD) GO TO 44
      END FILE IO
      REWIND IO
```

```
   44 WRITE (IPR,106) NOBS,NVO,IO
  106 FORMAT(10H THERE ARE ,I5,8H OBSN OF  ,I3,11H VAR ON LU ,I3)
      RETURN
      END
      SUBROUTINE DTAC(A,Y,SUM,X,NVAR,NOBS,IN,IO,NI)
      DOUBLE PRECISION A,Y,SUM,AVE,SD
      DIMENSION A(NI,NI),X(1),Y(1),SUM(1)
      IPR=3
      REWIND IN
      REWIND IO
      WRITE (IO) NVAR,NOBS
      DO 4 I=1,NVAR
      SUM(I)=0.0
      DO 4 J=I,NVAR
    4 A(I,J)=0.0
      DO 5 K=1,NOBS
      READ (IN) (X(I),I=1,NVAR)
      DO 5 I=1,NVAR
      SUM(I)=SUM(I)+X(I)
      DO 5 J=I,NVAR
    5 A(I,J)=A(I,J)+X(I)*Y(J)
      WRITE (IPR,109)
  109 FORMAT(/@ MEANS AND S.D.@ )
      DO 12 I=1,NVAR
      DO 6 J=I,NVAR
    6 A(I,J)=A(I,J)-SUM(I)/FLOAT(NOBS)*SUM(J)
      SD=DSQRT(A(I,I)/(FLOAT(NOBS)-1.0))
      AVE=SUM(I)/FLOAT(NOBS)
      Y(I)=AVE
   12 WRITE (IPR,100) I,AVE,SD
  100 FORMAT(I5,7F15.7/(5X,7F15.7))
      WRITE (IPR,102)
  102 FORMAT(/@ CORRECTED SS AND SP@ )
      WRITE (IO) (Y(I),I=1,NVAR)
      DO 16 I=1,NVAR
      WRITE (IO) (A(I,J),J=I,NVAR)
   16 WRITE (IPR,100) I,(A(I,J),J=I,NVAR)
      RETURN
      END
/*
// EXEC LNKEDT
/+
```

Listing of SMRR (Stepwise Multiple Regression Revised)

```
// JOB SMPR          000120,M020
// OPTION CATAL
 PHASE SMPP,*
// EXEC FORTRAN
// ETC BCD
C   STEPWISE MULTIPLE REGRESSION   REVISED
C   REX L. HURST
C   UTAH STATE UNIVERSITY
C   NX= NUMBER OF INDEPENDENT VARIABLES TO SELECT FROM MDCR
C   NY= NUMBER OF DEPENDENT VARIABLES TO SELECT FROM MDCP
C   IDY= Y TO USE FOR STEPWISE CONTROL
C   IX=1  STEPWISE MODE
C   IA=1 PRINT ORIGINAL INVERSE
C   IA=2 PUNCH OUT INVERSE MATRIX  (I5,5E15.7/(5X,5E15.7))
C   IB=1 PUNCH OUT REGRESSION COEFFICIENTS  (I5,5E15.7/(5X,5E15.7))
C   IC=1 PRINTS SUCCESSIVE INVERSES
C   IG=1 COMPUTES PREDICTED VALUES
C   IG=2 OUTPUTS Y,YP,DEV,SE,SD ON LOGICAL UNIT 3 (2I5,5F11.4)
C    NS= NUMBER OF SUBSETS OF  COEFFICIENTS TO COMPUTE
      DIMENSION A(70,70),AVE(70),ID(70),FMT(20),X(70),Z(70),W(70)
      DOUBLE PRECISION A,AVE,X,DET,Z
      NI=70
      IRD=1
      IPR=3
      IPH=2
      LUA=12
      LUB=13
      LUC=14
      READ  (IRD,100) NX,NY,IDY,IX,IA,IB,IC,IG,NS,(FMT(I),I=1,10)
      WRITE (IPR,101) NX,NY,IDY,IX,IA,IB,IC,IG,NS,(FMT(I),I=1,10)
  101 FORMAT(1H1,3I3,5I2,I3,18X,10A4)
  100 FORMAT(    3I3,5I2,I3,18X,10A4)
      IF(IG.EQ.2) REWIND LUC
      NK=NX+NY
      READ  (IRD,102) (ID(I),I=1,NK)
      WRITE (IPR,103) (ID(I),I=1,NK)
  103 FORMAT(1H ,20I4)
  102 FORMAT(    20I4)
      REWIND LUA
      READ  (LUA) NOV,NOBS
      READ  (LUA) (X(I),I=1,NOV)
      DO 60 I=1,NK
      J=ID(I)
   60 AVE(I)=X(J)
      DO 50 I=1,NOV
      READ (LUA) (X(J),J=I,NOV)
      DO 51 K=1,NK
      IF(I.EQ.ID(K)) GO TO 52
   51 CONTINUE
      GO TO 50
   52 DO 53 J=1,NK
      L=ID(J)
      IF(L.LT.I) GO TO 53
      A(K,J)=X(L)
      A(J,K)=X(L)
   53 CONTINUE
   50 CONTINUE
```

```
      NXP=NX+1
      CALL DMATIV(A,1,NX,NXP,NY,DET,NI)
      DO 3 I=1,NX
      DO 3 J=I,NX
    3 A(J,I)=A(I,J)
      IF(IA.EQ.0) GO TO 1
      WRITE (IPR,104)
  104 FORMAT(/@ INVERSE MATRIX@  )
      DO 2 I=1,NX
      IF(IA.EQ.2) WRITE (IPH,106) ID(I),(A(I,J),J=I,NX)
  106 FORMAT(I5,5E15.7/(5X,5E15.7))
    2 WRITE (IPR,105) ID(I),(A(I,J),J=I,NX)
  105 FORMAT(I5,7E15.7/(5X,7E15.7))
    1 CALL ANVR(A,AVE,X,Z,DET,ID,NX,NXP,NY,IDY,IB,NOBS,KZ,IG,LUB,NOV,W,
     1NI)
      IF(NX.EQ.1.OR.IX.EQ.0) GO TO 5
      CALL DLTE(A,AVE,DET,ID,NX,NXP,NY,IC,KZ,NI)
      GO TO 1
    5 IF(NS.EQ.0) GO TO 6
      IF(IX.EQ.1) GO TO 6
      CALL SBST(A,ID,NX,NY,NS,NI)
    6 IF(IG.NE.2) GO TO 7
      END FILE LUC
      REWIND LUC
    7 CALL EXIT
      END
```

Output of MDCR and SMRR

Model C (393 storms)

```
10  5 15  1  1  0  0
 1  1  1  0  0  0     0.0          0.0          0.0
21  2  2  0  0  0     0.5000       0.0          0.0
21  3  3  0  0  0     0.3000       0.0          0.0
 7  3  3  0  0  0     0.0          0.0          0.0
 3  1  1  2  3  0     0.0          0.0          0.0
21  4  4  0  0  0     0.3000       0.0          0.0
 1  5  5  0  0  0     0.0          0.0          0.0
21  6  6  0  0  0     0.3000       0.0          0.0
 3  2  4  5  6  0     0.0          0.0          0.0
 9  3  8  7  0  0     0.0          0.0          0.0
 6  1  1  0  0  0     0.0          0.0          0.0
 6  2  2  0  0  0     0.0          0.0          0.0
 6  3  3  0  0  0     0.0          0.0          0.0
 6  4  9  0  0  0     0.0          0.0          0.0
 6  5 10  0  0  0     0.0          0.0          0.0
"11X,F7.2,5F4.2,F4.4,F4.2,2F7.2<

FIRST THREE OBSERVATIONS AFTER TRANSFORMATION
   1 0.7331E 01 0.8755E 00 0.1545E 00 0.6422E 01 0.4478E 01
   2 0.7331E 01-0.2595E 00 0.1545E 00 0.4942E 01 0.4797E 01
   3 0.7331E 01-0.1907E 01 0.1665E 00 0.2890E 01 0.2399E 01
THERE ARE  393 OBSN OF  5 VAR ON LU  13

MEANS AND S.D.
   1  0.6847847D 01   0.1779186D 01
   2  0.7079684D 00   0.9512597D 00
   3  0.1082075D 00   0.1640496D 00
   4  0.5169305D 01   0.1726346D 01
   5  0.3490177D 01   0.2403140D 01

CORRECTED SS AND SP
   1  0.1240877D 04   0.1693859D 03 -0.3138759D 02   0.9206576D 03   0.1239442D 04
   2  0.3547189D 03 -0.9972666D 01   0.3201569D 03   0.4882380D 03
   3  0.1054961D 02 -0.7536460D 01   0.8995828D 01
   4  0.1168266D 04   0.1435719D 04
   5  0.2263832D 04

CORRELATION MATRIX
   1   0.10000000D 01    0.2553117D 00  -0.2743313D 00    0.7646504D 00    0.7395025D

   2   0.10000000D 01   -0.1466767D 00    0.4973359D 00    0.5448379D 00
   3   0.10000000D 01   -0.6798569D-01    0.5821042D-01
   4   0.10000000D 01    0.8828285D 00
   5   0.10000000D 01
```

```
  3  2  1 1 0 0 0 0  0
   1   2   3   4   5

 REGRESSION ANALYSIS OF VARIABLE    4

 SOURCE    DF     MEAN SQUARE     VAR   COEFFICIENT       AVE
 TOTAL    392   0.2980270D 01 B% 0< -0.3095996D 00  0.5169305D 01
 VAR  1     1   0.5434708D 03 B% 1<  0.7064932D 00  0.6847847D 01
 VAR  2     1   0.1239583D 03 B% 2<  0.6134978D 00  0.7079684D 00
 VAR  3     1   0.3532546D 02 B% 3<  0.1909394D 01  0.1082075D 00
 MODEL      3   0.2774879D 03    RSQ#  0.7125636D 00
 ERROR    389   0.8632447D 00    DET#  0.3986903D 07

 REGRESSION ANALYSIS OF VARIABLE    5

 SOURCE    DF     MEAN SQUARE     VAR   COEFFICIENT       AVE
 TOTAL    392   0.5775082D 01 B% 0< -0.4419555D 01  0.3490177D 01
 VAR  1     1   0.1036500D 04 B% 1<  0.9756734D 00  0.6847847D 01
 VAR  2     1   0.3477808D 03 B% 2<  0.1027609D 01  0.7079684D 00
 VAR  3     1   0.2076727D 03 B% 3<  0.4629578D 01  0.1082075D 00
 MODEL      3   0.5842183D 03    RSQ#  0.7741982D 00
 ERROR    389   0.1314080D 01    DET#  0.3986903D 07

 VARIABLE  3 WILL NOW BE DELETED


 REGRESSION ANALYSIS OF VARIABLE    4

 SOURCE    DF     MEAN SQUARE     VAR   COEFFICIENT       AVE
 TOTAL    392   0.2980270D 01 B% 0<  0.2216227D 00  0.5169305D 01
 VAR  1     1   0.5081759D 03 B% 1<  0.6618806D 00  0.6847847D 01
 VAR  2     1   0.1140645D 03 B% 2<  0.5865028D 00  0.7079684D 00
 MODEL      2   0.3985692D 03    RSQ#  0.6823261D 00
 ERROR    390   0.9516094D 00    DET#  0.4114708D 06

 REGRESSION ANALYSIS OF VARIABLE    5

 SOURCE    DF     MEAN SQUARE     VAR   COEFFICIENT       AVE
 TOTAL    392   0.5775082D 01 B% 0< -0.3131536D 01  0.3490177D 01
 VAR  1     1   0.8729674D 03 B% 1<  0.8675043D 00  0.6847847D 01
 VAR  2     1   0.3069740D 03 B% 2<  0.9621563D 00  0.7079684D 00
 MODEL      2   0.7724911D 03    RSQ#  0.6824632D 00
 ERROR    390   0.1843205D 01    DET#  0.4114708D 06

 VARIABLE  2 WILL NOW BE DELETED


 REGRESSION ANALYSIS OF VARIABLE    4

 SOURCE    DF     MEAN SQUARE     VAR   COEFFICIENT       AVE
 TOTAL    392   0.2980270D 01 B% 0<  0.8860565D-01  0.5169305D 01
 VAR  1     1   0.6830738D 03 B% 1<  0.7419412D 00  0.6847847D 01
 MODEL      1   0.6830738D 03    RSQ#  0.5846903D 00
 ERROR    391   0.1240901D 01    DET#  0.1240877D 04

 REGRESSION ANALYSIS OF VARIABLE    5

 SOURCE    DF     MEAN SQUARE     VAR   COEFFICIENT       AVE
 TOTAL    392   0.5775082D 01 B% 0< -0.3349750D 01  0.3490177D 01
 VAR  1     1   0.1238008D 04 B% 1<  0.9988434D 00  0.6847847D 01
 MODEL      1   0.1238008D 04    RSQ#  0.5468639D 00
 ERROR    391   0.2623591D 01    DET#  0.1240877D 04
```

Output of SMRR with predicted values
Model B (393 storms)

REGRESSION ANALYSIS OF VARIABLE 10

SOURCE	DF	MEAN SQUARE	VAR	COEFFICIENT	AVE
TOTAL	392	0.5775082D 01	B% 0<	-0.4731619D 01	0.3490177D 01
VAR 1	1	0.1778101D 03	B% 1<	0.1109574D 01	0.6773735D 01
VAR 2	1	0.1034163D 01	B% 2<	0.3599253D-01	0.5738903D 00
R 3	1	0.3532597D 01	B% 3<	-0.2726247D 00	0.7094439D 00
VAR 4	1	0.7291317D 00	B% 4<	-0.7613813D-01	0.1179231D 01
VAR 5	1	0.6643290D 02	B% 5<	0.1248097D 01	0.4669138D 00
VAR 6	1	0.2118203D 01	B% 6<	-0.1874050D 00	-0.3757145D 00
VAR 7	1	0.3544428D 02	B% 7<	-0.2208767D 01	-0.1810769D 00
VAR 8	1	0.3601253D 01	B% 8<	0.1162967D 01	-0.7286830D-01
MODEL	8	0.2480279D 03	RSQ#	0.8764888D 00	
ERROR	384	0.7281475D 00	DET#	0.1151058D 17	

N,Y,OBS,PRED,DEV,SE,SD

N	Y	OBS	PRED	DEV	SE	SD
1	10	0.4478E 01	0.4951E 01	-0.4726E 00	0.8251E-01	0.8573E 00
2	10	0.4797E 01	0.3775E 01	0.1022E 01	0.1190E 00	0.8616E 00
3	10	0.2399E 01	0.2566E 01	-0.1671E 00	0.1834E 00	0.8728E 00
4	10	0.4478E 01	0.4306E 01	0.1723E 00	0.1261E 00	0.8626E 00
5	10	0.5626E 01	0.5426E 01	0.1995E 00	0.1161E 00	0.8612E 00
6	10	0.4479E 01	0.4979E 01	-0.5005E 00	0.9089E-01	0.8581E 00
7	10	0.5499E 01	0.4399E 01	0.1100E 01	0.8398E-01	0.8579E 00
8	10	0.5687E 01	0.5314E 01	0.3737E 00	0.9287E-01	0.8584E 00
9	10	0.4979E 01	0.5735E 01	-0.7556E 00	0.1287E 00	0.8630E 00
10	10	0.6137E 01	0.6235E 01	-0.9874E-01	0.1433E 00	0.8653E 00
11	10	0.6633E 01	0.5213E 01	0.1421E 01	0.8874E-01	0.8579E 00
12	10	0.3508E 01	0.4349E 01	-0.8409E 00	0.1335E 00	0.8637E 00
13	10	0.6493E 01	0.6307E 01	0.1865E 00	0.2008E 00	0.8766E 00
14	10	0.5278E 01	0.5097E 01	0.1809E 00	0.1056E 00	0.8598E 00
15	10	0.4797E 01	0.4441E 01	0.3560E 00	0.8578E-01	0.8576E 00
16	10	0.5444E 01	0.4318E 01	0.1126E 01	0.9740E-01	0.8589E 00
17	10	0.4797E 01	0.4829E 01	-0.3205E-01	0.9820E-01	0.8589E 00
18	10	0.4797E 01	0.5051E 01	-0.2539E 00	0.7630E-01	0.8567E 00
19	10	0.4478E 01	0.4725E 01	-0.2468E 00	0.9326E-01	0.8584E 00
20	10	0.2399E 01	0.3378E 01	-0.9788E 00	0.1284E 00	0.8629E 00
21	10	0.4255E 01	0.4831E 01	-0.5753E 00	0.1068E 00	0.8600E 00
22	10	0.4964E 01	0.5106E 01	-0.1422E 00	0.8807E-01	0.8579E 00
23	10	0.3274E 01	0.3828E 01	-0.5535E 00	0.1037E 00	0.8602E 00
24	10	0.5766E 01	0.5620E 01	0.1460E 00	0.9733E-01	0.8589E 00
25	10	0.4191E 01	0.5358E 01	-0.1167E 01	0.1065E 00	0.8599E 00
26	10	0.5038E 01	0.4698E 01	0.3397E 00	0.9480E-01	0.8586E 00
27	10	0.4551E 01	0.5097E 01	-0.5359E 00	0.7409E-01	0.8565E 00
28	10	0.6963E 01	0.6599E 01	0.3648E 00	0.1611E 00	0.8684E 00
29	10	0.5395E 01	0.5336E 01	0.5896E-01	0.8400E-01	0.8574E 00
30	10	0.6088E 01	0.6009E 01	0.7905E-01	0.1112E 00	0.8605E 00
31	10	0.3354E 01	0.4016E 01	-0.6616E 00	0.1006E 00	0.8592E 00
32	10	0.3315E 01	0.3788E 01	-0.4732E 00	0.1225E 00	0.8621E 00
33	10	0.5343E 01	0.5005E 01	0.3388E 00	0.1138E 00	0.8609E 00
34	10	0.5343E 01	0.5610E 01	-0.2669E 00	0.1055E 00	0.8598E 00
35	10	0.5467E 01	0.5638E 01	-0.1713E 00	0.1077E 00	0.8601E 00
36	10	0.3498E 01	0.4053E 01	-0.5550E 00	0.1071E 00	[illegible]
37	10	0.4661E 01	0.5105E 01	-0.4443E 00	0.9379E-01	0.8585E 00
38	10	0.4122E 01	0.4212E 01	-0.8996E-01	0.1099E 00	0.8604E 00
39	10	0.4478E 01	0.4880E 01	-0.4010E 00	[illegible]	0.8567E 00
40	10	0.4345E 01	0.4831E 01	-0.4861E 00	[illegible]	0.8574E 00
41	10	0.3881E 01	0.4670E 01	-0.7892E 00	0.9574E-01	0.8587E 00
42	10	0.5759E 01	[illegible]	-0.4850E-01	[illegible]	0.8593E 00
43	10	0.3498E 01	0.4447E 01	[illegible]	[illegible]	0.8576E 00
44	10	0.4917E 01	0.5015E 01	-0.9838E-01	0.9923E-01	0.8591E 00
45	10	0.5865E 01	0.5910E 01	-0.4523E-01	0.1029E 00	0.8595E 00
46	10	0.5343E 01	0.5938E 01	-0.5948E 00	0.1242E 00	0.8623E 00
47	10	0.5107E 01	0.5347E 01	-0.2395E 00	0.1028E 00	0.8595E 00
48	10	0.5517E 01	0.5500E 01	0.1676E-01	0.1523E 00	0.8664E 00
49	10	0.3153E 01	0.4263E 01	-0.1100E 01	0.9673E-01	0.8598E 0

8 2 0 0 0 0 0 1 0
1 2 3 4 5 6 7 8 9 10

REGRESSION ANALYSIS OF VARIABLE 9

SOURCE	DF	MEAN SQUARE	VAR	COEFFICIENT	AVI
TOTAL	392	0.2980270D 01	B(0)	-0.2528041D 00	0.5149305D 01
VAR 1	1	0.7872267D 02	B(1)	0.7382917D 00	0.6773735D 01
VAR 2	1	0.3306149D 02	B(2)	0.2035069D 00	0.5733903D 00
VAR 3	1	0.8303806D-01	B(3)	-0.4179812D-01	0.7094439D 00
VAR 4	1	0.8526144D 01	B(4)	-0.2603608D 00	0.1179231D 01
VAR 5	1	0.4402164D 02	B(5)	0.1015991D 01	0.4669138D 00
VAR 6	1	0.1936847D 01	B(6)	0.1792029D 00	-0.3757145D 00
VAR 7	1	0.4612826D 01	B(7)	-0.7968202D 00	-0.1815769D 00
VAR 8	1	0.4031975D 01	B(8)	-0.1230550D 01	-0.7266880D-01
MODEL	9	0.1144937D 03	RSQ#	0.7840246D 00	
ERROR	384	0.6570748D 00	DET#	0.1151058D 17	

N,Y,OBS,PRED,DEV,SE,SD

1	9	0.6422E 01	0.6108E 01	0.3136E 00	0.7838E-01	0.8144E 00
2	9	0.4942E 01	0.4813E 01	0.1287E 00	0.1131E 00	0.8184E 00
3	9	0.2890E 01	0.3716E 01	-0.8256E 00	0.1742E 00	0.8291E 00
4	9	0.7090E 01	0.5831E 01	0.1259E 01	0.1198E 00	0.8194E 00
5	9	0.7056E 01	0.6267E 01	0.7888E 00	0.1103E 00	0.8181E 00
6	9	0.6994E 01	0.6263E 01	0.7307E 00	0.8634E-01	0.8152E 00
7	9	0.7365E 01	0.5868E 01	0.1498E 01	0.8453E-01	0.8150E 00
8	9	0.7626E 01	0.6690E 01	0.9358E 00	0.8822E-01	0.8154E 00
9	9	0.5858E 01	0.6564E 01	-0.7057E 00	0.1223E 00	0.8193E 00
10	9	0.7021E 01	0.6894E 01	0.1271E 00	0.1361E 00	0.8219E 00
11	9	0.6755E 01	0.6178E 01	0.5766E 00	0.8430E-01	0.8159E 00
12	9	0.4836E 01	0.5023E 01	-0.1870E 00	0.1268E 00	0.8205E 00
13	9	0.6485E 01	0.6473E 01	0.1163E-01	0.1907E 00	0.8327E 00
14	9	0.7438E 01	0.6732E 01	0.7062E 00	0.1004E 00	0.8163E 00
15	9	0.6918E 01	0.6030E 01	0.8880E 00	0.8149E-01	0.8147E 00
16	9	0.6004E 01	0.5560E 01	0.4435E 00	0.9252E-01	0.8159E 00
17	9	0.7200E 01	0.6249E 01	0.9517E 00	0.9328E-01	0.8160E 00
18	9	0.6985E 01	0.6245E 01	0.7399E 00	0.7248E-01	0.8138E 00
19	9	0.5252E 01	0.5630E 01	-0.3774E 00	0.8859E-01	0.8154E 00
20	9	0.5106E 01	0.4651E 01	0.4550E 00	0.1220E 00	0.8197E 00
21	9	0.5403E 01	0.5604E 01	-0.2017E 00	0.1015E 00	0.8169E 00
22	9	0.7555E 01	0.6598E 01	0.9564E 00	0.8366E-01	0.8149E 00
23	9	0.5844E 01	0.5584E 01	0.2596E 00	0.1033E 00	0.8172E 00
24	9	0.6586E 01	0.6746E 01	-0.1603E 00	0.9246E-01	0.8159E 00
25	9	0.5252E 01	0.6122E 01	-0.8695E 00	0.1012E 00	0.8169E 00
26	9	0.5886E 01	0.5704E 01	0.1821E 00	0.9906E-01	0.8156E 00
27	9	0.6916E 01	0.6220E 01	0.6955E 00	0.7038E-01	0.8137E 00
28	9	0.8219E 01	0.7810E 01	0.4089E 00	0.1530E 00	0.8249E 00
29	9	0.6772E 01	0.6592E 01	0.1797E 00	0.7980E-01	0.8145E 00
30	9	0.7685E 01	0.7021E 01	0.6637E 00	0.1056E 00	0.8175E 00
31	9	0.5635E 01	0.5636E 01	-0.1583E-02	0.9553E-01	0.8162E 00
32	9	0.5529E 01	0.5488E 01	0.4172E-01	0.1163E 00	[illegible]
33	9	0.7728E 01	0.6296E 01	0.1432E 01	[illegible]	0.8178E 00
34	9	0.7438E 01	0.6931E 01	0.5177E 00	[illegible]	0.8163E 00
35	9	0.7591E 01	0.6941E 01	[illegible]	[illegible]	[illegible]
36	9	0.6153E 01	0.5776E 01	0.3774E 00	[illegible]	[illegible]
37	9	0.6638E 01	0.6527E 01	0.1617E 00	[illegible]	0.8155E 00
38	9	0.6835E 01	0.5982E 01	[illegible]	[illegible]	0.8172E 00
39	9	0.6380E 01	0.6075E 01	0.3052E 00	0.7258E-01	0.8138E 00
40	9	0.6613E 01	0.6212E 01	0.4018E 00	0.7988E-01	0.8145E 00
41	9	0.5043E 01	0.5521E 01	-0.4775E 00	0.7094E-01	0.8157E 00
42	9	0.7115E 01	0.6939E 01	0.1763E 00	[illegible]	0.8143E 00
43	9	0.4942E 01	0.5518E 01	-0.5768E 00	0.8137E-01	0.8147E 00
44	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]