WRRC

Bulletin 52

Predicting Peak Flow of Small Watersheds by use of Channel Characteristics

By

Curtis L. Larson, Ronald F. Gronwald, and Alfred G. Pennell

University of Minnesota

Department of Agricultural Engineering

The work upon which this publication is based was supported in part by funds provided by the United States Department of the Interior as authorized under the Water Resources Research Act of 1964, Public Law 88-379

June 1972

Minneapolis, Minnesota

WATER RESOURCES RESEARCH CENTER

UNIVERSITY OF MINNESOTA

GRADUATE SCHOOL

CONTENTS

ILLUSTRATIONS

TABLES

FOREWORD

This Bulletin is published in furtherance of the purposes of the Water Resources Research Act of 1964. The purpose of the Act is to stimulate, sponsor, provide for, and supplement present programs for the conduct of research, investigations, experiments, and the training of scientists in the field of water and resources which affect water. The Act is promoting a more adequate national program of water resources research by furnishing financial assistance to non-Federal research.

The Act provides for establishment of Water Resources Research Centers at Universities throughout the Nation. On September 1, 1964, a Water Resources Research Center was established in the Graduate School as an interdisciplinary component of the University of Minnesota. The Center has the responsibility for unifying and stimulating University water resources research through the administration of funds covered in the Act and made available by other sources; coordinating University research with water resources programs of local, State and Federal agencies and private organizations throughout the State; and assisting in training additional scientists for work in the field of water resources through research.

This Bulletin is number 52 in a series of publications designed to present information bearing on water resources research in Minnesota and the results of some of the research sponsored by the Center. The Bulletin is concerned with the channel phase of the surface runoff process. The overall method for predicting the effects of channel characteristics, including watershed size and shape, on peak flow from small ungaged watersheds was tested. The results of this study are part of a continuing effort by the Principal Investigator and associates to develop a reliable method for predicting peak watershed runoff by the use of rainfall data and measurable watershed characteristics. Such a method is needed to improve the design of the hundreds of highway culverts, small bridges, erosion control structures, channel modifications, grass waterways, storm sewers, etc., installed each year in the United States.

This Bulletin serves as the Research Project Technical Completion Report for the following Center project:

OWRR Project No.: A-023-Minn.

Project Title: Predicting Peak Flow of Small Watersheds by Use of Channel Characteristics

Principal Investigator: C. L. Larson, Dept. of Agricultural Engineering, Univ. of Minn.

Project Began: July 1, 1970 Project Completed: June 30, 1972

FCST Research Category: II-E

Publication Abstract:

In previous studies, a method was developed for predicting the effects of channel characteristics, including watershed size and shape on peak flow from small watersheds. The method was incomplete, however, since it lacked a working method of estimating the time parameter for ungaged watersheds. Therefore, the first objective of this study was to satisfy this need. The second objective was to test the overall method as a means of predicting peak flow for small ungaged watersheds, given the runoff volume. The overall method begins with a hydrologic analysis of numerous rainfall-runoff events observed at selected experimental watersheds. This yields certain hydrologic parameters which can be evaluated only for gaged watersheds. Then, the physical characteristics of these watersheds, primarily the channel characteristics, are utilized to evaluate the same parameters by use of an hydraulic or flow approach. If this can be accomplished successfully, the same procedure can be applied to ungaged watersheds.

The following conclusions can be made based on the results of the study: A new time parameter, time to 50% of equilibrium, T_{50}, was proposed. It can be evaluated hydrologically, i.e., from observed hydrographs in many but not all cases this is essential if it is to be used in peak flow predictions for other, ungaged watersheds. T_{50} increases with watershed size, approximately as watershed area to the 1/3 power. It decreases as the mean rate of rainfall excess (q_s), increases, varying as q_s to the minus 1/2 power (roughly). The residual variability is substantial, indicating that other factors also affect T_{50} significantly. The channel characteristics, cross-sectional area and wetted perimeter can be estimated with reasonable accuracy from measurements of bankfull topwidth and depth. However, a number of complete channel cross-sections must be taken or be available in the region in order to evaluate the two coefficients needed, one for area and one for wetted perimeter. It appears likely that these coefficients can be generalized through further study, and also that relationships will have other applications in watershed engineering. Three methods used to divide the watershed into an upper and lower half hydrologically gave only slightly different results and, therefore, the simplest method (arc) appears preferable. The travel time approach to evaluating the time parameter yields values (designated T_{CH}) that are consistently and significantly lower than the true values (T_{50}). Thus, a coefficient applicable to the region is necessary to relate T_{50} to T_{CH}. The peak flow predictions by the methods of this study were quite variable, as compared to the observed values, but on the average were about the right magnitude, i.e., neither consistently high or consistently low. The combination of peak flow equation, the time parameter, T_{50}, and the relationship of C_p, the peak flow coefficient, to the ratio D/T_{50}, where D is the duration of rainfall excess, appears to provide a satisfactory but not highly accurate procedure for estimating peak runoff, given the volume of rainfall excess and its approximate time distribution.

Publication Descriptors: *Surface Runoff/ *Peak Runoff/ *Ungaged Watersheds/ Interflow/ Channel Characteristics/ Rainfall - Runoff/ Hydrographs

Publication Identifiers: *Overall Method/ *Time Parameter/ *Travel Time/ Peak Flow Equation

I. INTRODUCTION

The task of estimating peak flows for ungaged watersheds is a very difficult and elusive problem, one that has plagued hydrologists and engineers for many years. This problem is especially severe for small watersheds, arbitrarily defined here as those up to about 100 square miles in area, even though watersheds of several hundred square miles are often considered "small" also. Because the number of watersheds in this category is extremely high, a high percentage of them will never be gaged. Thus, the task of estimating peak flow for ungaged watersheds will always be with us.

Each year engineering hydrologists are called upon to estimate peak flows of various recurrence intervals for thousands of small watersheds. These estimates are needed for the design of stabilizing (gully control) structures, highway culverts and bridges, storm sewers, channel design and various types of water control structures. This work must go forward with existing methods of estimating peak flow, whether of adequate accuracy or not. The result can be either underdesign, with high risk of failure, or overdesign, at added cost. Although the amount of money "lost" in each case may be only a few thousand dollars, the total annual loss is extensive due to the large number of such structures.

Reich and Heimstra (1967) compared peak flow predictions by five methods to observed peak flows for numerous small watersheds. All of the methods were found to be either consistently high or consistently low. The 95% confidence limits varied from as low as 40% of the observed values (lower limit)

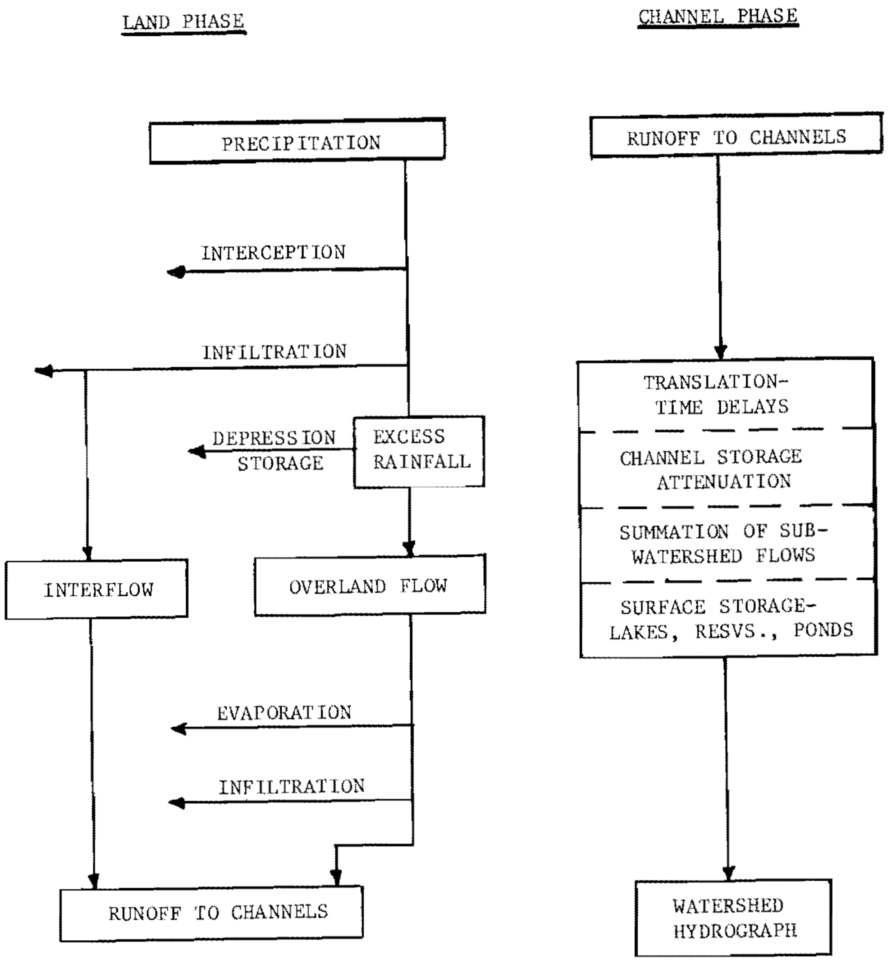


Figure 1. The Process of Surface Runoff

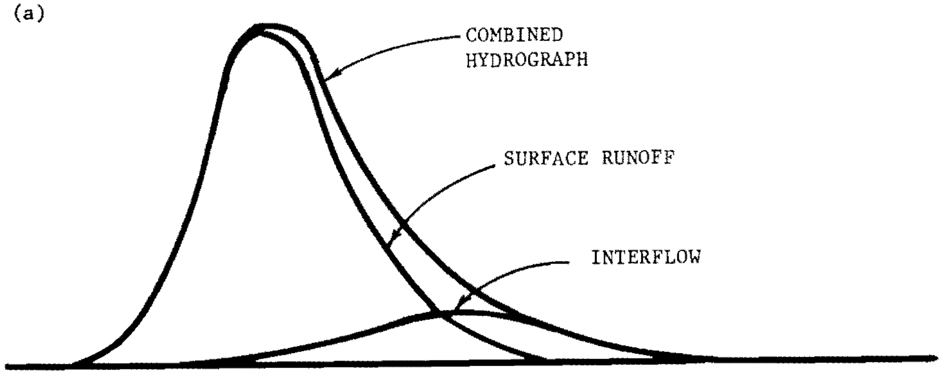
for one method to as high as 265% (upper limit) for another method. It seem evident that considerable improvement is needed in peak flow predictions for small, ungaged watersheds.

The process of surface or surface runoff can be considered ideally as having two separate phases, the land phase and the channel phase. The volume of storm runoff for a major rainfall event is determined largely in the land phase (Figure 1), which includes the processes of interception, infiltration, overland flow, soil moisture storage, interflow and evapotranspiration between and during runoff events. The channel phase is a process of unsteady flow in a network of open channels, including elementary channels, some of which are poorly defined swales or waterways that are evident only during runoff. The channel phase, to a large extent determines the hydrograph shape and the peak flow, given a certain volume of storm runoff from the land phase.

In reality, these two phases are often not fully independent or separable, and vary in their degree of independence with watershed size and land characteristics. For very small watersheds, e.g., 100 acres, it has been shown (Morgali, 1963) that the calculated overland flow hydrograph is a close approximation of the watershed hydrograph. In such cases, the channel phase effects are insignificant and the overland flow routing determines the hydrograph shape. As the watershed size increases, however, the channel phase becomes more and more the determinant of hydrograph shape and peak flow, and overland flow becomes less and less important (Golany and Larson, 1971, p. 100).

Secondly, shallow subsurface flow (interflow) complicates the process in those areas where it is a significant percentage of the storm runoff volume. With significant interflow, the storm runoff amount in inches, R, becomes

$$R = P - F + I \qquad (1)$$



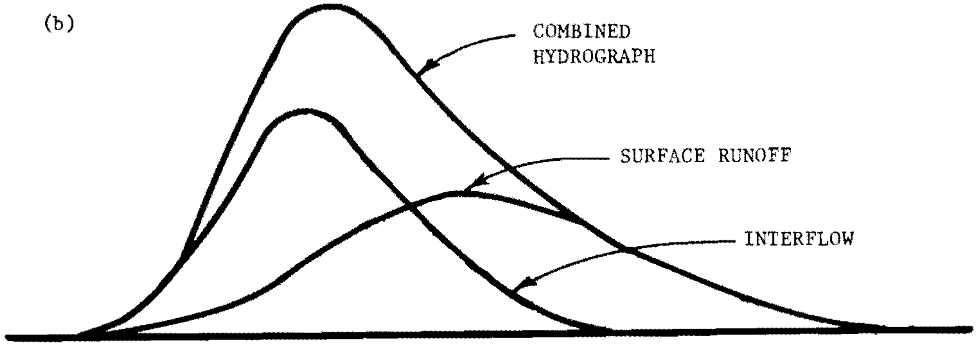


Figure 2. Sketches of Hydrographs Showing the Effect of Interflow on Surface Runoff

where P is the storm rainfall, F the cumulative infiltration and I as the interflow in inches. If there is interflow, the watershed retention is not equal to the infiltration amount, making it much more difficult to predict runoff amounts.

The interflow process is, of course, slower than surface runoff. Thus, as shown in Fig ure 2(a), a low interflow contribution in terms of volume has a small, perhaps negligible, effect on the peak runoff. A high interflow contribution, shown in Figure 2(b), adds a significant percentage to the peak flow. It also tends to enlarge the time base of the watershed hydrograph and delays the time of peak discharge.

Runoff as it is generated on the land surface is usually referred to as "excess rainfall" or "rainfall excess" (Figure 1). Without interflow, the time distribution of runoff to channels is determined by the rainfall pattern and the overland flow process. With significant interflow occurring, the runoff to channels is extended over a greater length of time. If the runoff is mostly interflow, the time of peak runoff to channels can be delayed considerably. Thus, interflow tends to increase the importance of the land phase as a factor affecting hydrograph shape and peak discharge. In general, interflow affects both the volume of runoff and its time distribution.

This study is concerned mainly with the channel phase of the runoff process. For purposes of analysis and prediction, it is advantageous to consider the channel phase independently from the land phase for watersheds of any size. The foregoing discussion, however, indicates that this may not be feasible for relatively small watersheds (up to several hundred acres), especially if interflow is significant.

In previous studies (described in the next section), a method has been developed for predicting the effects of channel characteristics, including watershed size and shape on peak flow from small watersheds (Larson, 1965; Larson and Machmeier, 1968). The method was incomplete, however, since it lacked a working method of estimating the time parameter for ungaged watersheds. Therefore, the first objective of the study reported herein was to satisfy this need insofar as possible. The second objective was to test the overall method as a means of predicting peak flow for small ungaged watersheds, given the runoff volume. The prediction of storm runoff volumes was not within the scope of this study. It is a major problem in itself.

The overall procedure intended to accomplish these objectives begins with a hydrologic analysis of numerous rainfall-runoff events observed at selected experimental watersheds. This yields certain hydrologic parameters which can be evaluated only for gaged watersheds. Then, the physical characteristics of these watersheds, primarily the channel characteristics, are utilized to evaluate the same parameters by use of an hydraulic or flow approach. If this can be accomplished successfully, the same procedure can be applied to ungaged watersheds. The final step is to test the overall method for predicting peak flows.

II. Review of Channel Phase Methods and Parameters

Most methods for estimating the peak flow for ungaged watersheds of necessity have two things in common. First, they utilize an assumed or predicted storm rainfall amount and pattern as the "input" or source of runoff. Secondly, they utilize one or more watershed characteristics to estimate the peak flow, given a certain rainfall event, but vary greatly in how this is done. A second general approach which will not be considered here is to "regionalize" existing runoff records. This is accomplished by correlating peak flows of various recurrence intervals to watershed area and key land characteristics within a given region having essentially uniform geology and soil types. Rainfall data are usually not utilized in this approach.

Returning to the first, more general approach, some of the methods lump the land phase and channel phase processes together. One example is the much used but often criticized Rational formula

$$Q_p = C\ i\ a \qquad (2)$$

in which a is the watershed area in acres and i is the mean rainfall intensity in inches per hour for a duration equal to the "time of concentration". The runoff coefficient C is related to the infiltration and topography of the watershed (Williams, 1950) and is varied from zero to one, which is desirable. The use of a duration equal to the time of concentration is presumed to account for the channel phase of the runoff process. At this duration, it is assumed that all parts of the watershed are contributing to the outflow, i.e., equilibrium has been reached. However, many hydrologists believe that existing tables and equations for time of concentration underestimate time of concentration, perhaps compensating to some extent for the fact that equilibrium flow rarely if ever occurs.

Snyder (1938) proposed one of the first channel phase-only equations. The peak flow of the unit hydrograph (one inch of runoff) is given by

$$Q_u = 640\ C_p \frac{A}{t_p} \qquad (3)$$

where A is the watershed area in square miles, t_p is the lag time in hours, and C_p is a peak flow coefficient. For various gaged watersheds in the Appalachians, Snyder found values of C_p from 0.56 to 0.69.

For the channel phase, Chow (1962) suggested the following equation for the peak flow of the unit hydrograph:

$$P = 1.008 \frac{AZ}{D} \tag{4}$$

in which A is the watershed area in sq.mi., Z is a peak flow reduction factor similar to C_p and D is the rainfall duration. The factor Z was related to duration and lag time for streams in Illinois and values from 0.08 to 0.95 were obtained for individual events.

Another channel phase, peak flow equation was proposed by Mockus for use by the Soil Conservation Service (1955). The peak discharge is given by

$$Q_p = 484 \frac{AR}{T_p} \tag{5}$$

where R is the runoff amount in inches, and T_p is the time to peak of either the Mockus dimensionless hydrograph or the corresponding triangular hydrograph. A peak flow coefficient is not included, but one sees that 484/640 x 1.008 = 0.75. Thus, Mockus has, in effect used C_p = 0.75 as the general case.

A land phase, peak flow equation quite similar to those given above was proposed by Larson (1965) (See also Larson and Machmeier, 1968). It is the method mentioned earlier which requires testing with actual runoff data and which provides the basis for the study herein reported. The equation is as follows:

$$Q_p = 646\ A\ C_p \frac{R}{D} \tag{6}$$

in which A is watershed area in sq.mi., R is the runoff amount in inches, D is the duration of the unrouted runoff (rainfall excess) and C_p is a routing or peak flow coefficient.

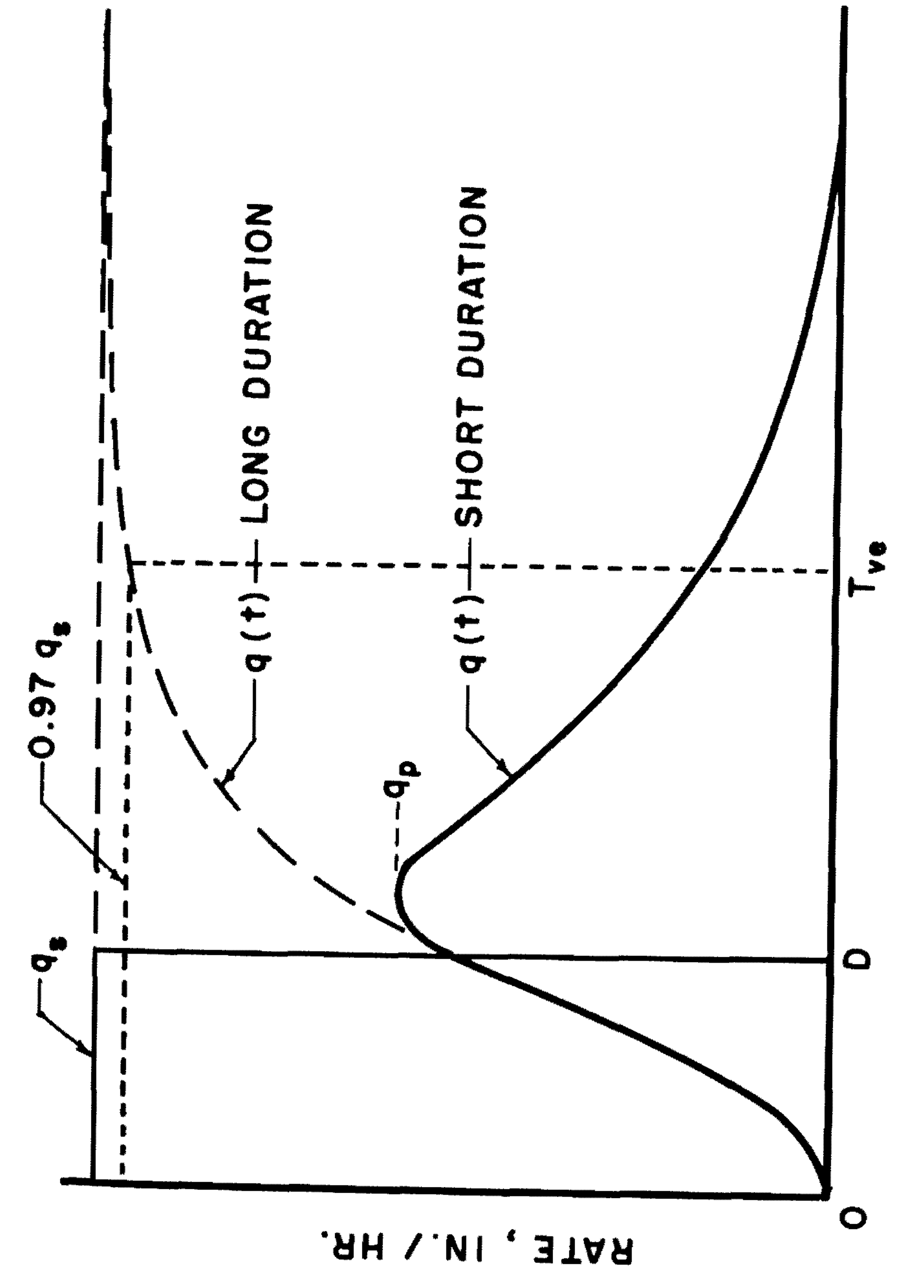


Figure 3. Sketch of Hydrograph Showing the Effect of Long and Short Duration Storms, and Showing Time to Virtual Equilibrium

In determining values for use in Eq. 6, R and D come from the land phase analysis, which is treated as a separate problem. This leaves one factor to be determined, namely C_p. From Figure 3, we see that C_p can be defined theoretically as the ratio of the peak watershed runoff in inches per hour to the mean runoff supply rate, q_s, which is equal to R/D. Thus C_p varies from zero to one and for a given supply rate, q_s, increases with the duration, D. For a duration equal to or greater than the time required to reach equilibrium flow, the value of C_p is 1.0.

As indicated in Figure 3, the watershed outflow approaches equilibrium slowly and, therefore, the time to reach equilibrium is poorly defined. Larson (1965) defined a watershed time parameter, T_{ve}, time to virtual equilibrium, as the time required for the watershed outflow to reach 97% of a constant supply rate, q_s. If the rainfall excess rate is constant and of long duration, T_{ve} is easily evaluated from the outflow hydrograph and is significantly less than the time to 100% of equilibrium flow (Figure 3). It will, of course, increase with watershed size and depend also on channel characteristics.

For a given watershed, then, it can be seen that C_p will increase from zero for D = 0 to 0.97 at D = T_{ve} and to 1.00 at equilibrium. Due to variations in speed of response between various watersheds a curve of C_p vs. D will be different for different watersheds. As a first attempt to obtain a general relation for C_p , Larson (1965) plotted C_p vs values of the ratio D/T_{ve} for several small watersheds, and obtained a single curve. However, the values of C_p and T_{ve} were obtained by use of the time-area routing procedure then used in the Stanford watershed. Since this procedure relies on the assumption of runoff linearity, which implies flow velocities independent of discharge, the resulting curve could not be assumed generally correct and applicable.

Following this, a relationship of C_p to D/T_{ve} was obtained by Machmeier (1966) using a mathematical watershed model in which the dynamic equations of unsteady flow were used to calculate flow rates throughout the channel system. The watershed area was about 21 miles. Although only one watershed was utilized, six different durations and six supply rate were used for the input to the channel system. Values of T_{ve} were determined by running the model to near equilibrium with each supply rate, showing that T_{ve} varies with supply rate, indicating non-linearity of runoff. Nevertheless, the many values of C_p obtained plotted against the ratio D/T_{ve} yielded a single curve, which is believed to be a reasonable approximation of the true relationship for small watersheds in general. It should be noted, that overland flow routing was included in the model and therefore its effect is included in the relationship for C_p.

The relationship of T_{ve} to channel characteristics was studied by Golany (Golany and Larson, 1971) for a mathematical watershed model similar to Machmeier's, but with backwater effects included at channel junctions. T_{ve} was expressed as a sum of two components, the rise time for an elementary (1st order) watershed and the main channel rise time. A method for evaluating the latter was developed but was rather complex and difficult to generalize. In addition, the parameter, T_{ve}, cannot in general be evaluated from watershed hydrologic data, since 97% of equilibrium flow is rarely if ever attained.

In a second study with a model watershed similar to the one used by Golany, the effects of watershed shape and of input time distribution were studied by Wei and Larson (1971). The study showed that elongating the watershed considerably decreases peak flow by about 20 per cent as compared to a compact waterhsed of the same area, and increased time to peak about 10 per cent. Various triangular time distributions at the input rainfall excess were utilized and compared to rectangular time patterns. The effects on peak flows are presented later, being useful in the hydrologic analysis of this study. The effects of non-uniform rainfall patterns, both in space and time, were also studied.

The above review of past work includes only the studies having direct and significant relavence to the present study. There are numerous other references which relate to the general topic, either indirectly or to a lesser degree, or are potentially useful to a somewhat different approach. Thus, only a partial review of the overall topic has been presented.

Before proceeding to the study itself, it might be well to distinguish between <u>hydrologic</u> and <u>hydraulic</u> time parameters. A hydrologic time parameter is one that is evaluated from observed hydrographs. The most common example would be lag time, which is described in most hydrology textbooks, e.g., Linsley et al (1958). As defined earlier, T_{ve} is also a hydrologic time parameter. A hydraulic, watershed time parameter is one that is evaluated by flow calculations, i.e., mean velocities and travel times for the various channels. The parameter, time of concentration, is often estimated this way, though actually defined hydrologically. Both the hydrologic and hydraulic approaches to determining watershed time parameters are used in this study.

III. SELECTION OF HYDROLOGIC DATA

Although no hydrologic observations were made as a part of the study, considerable time was spent in procuring and selecting hydrologic data for analysis and for testing the method of peak flow prediction. Both runoff hydrographs and the associated rainfall patterns were needed to carry out the study. The principal concern was with small watersheds, as stated earlier. It also appeared desirable at this stage to concentrate in a few selected areas where one could find a number of gaged watersheds having similar land characteristics and hydrologic data extending over 10 years or more.

It was evident that the research watersheds of the Agricultural Research Service (some initiated under the Soil Conservation Service) satisfy these requirements better than any other. In addition, both the runoff and rainfall data are published by individual runoff events, using short intervals of time as needed for a complete analysis. Antecedent rainfall amounts are given for each event also. Watershed topographic maps are given for each of the watersheds. In general, the authors found that the ARS data are very convenient and, in fact, invaluable in carrying out a study of this type. Furthermore, the data are available to others to analyze in their own way, perhaps leading to new and better techniques. Thus, one cannot emphasize too much the value of such observations extending over a period of years.

The sources of the hydrologic data used in the study, all published by the U.S. Department of Agriculture, are as follows:

1. Hydrologic Data, North Appalachian Experimental Watershed, Coshocton, Ohio, 1939, Soil Conservation Service, USDA, Hydrologic Bulletin No. 1.

2. Hydrologic Data, North Appalachian Experimental Watershed, Coshocton, Ohio, 1940, Soil Conservation Service, USDA, Hydrologic Bulletin No. 4.

3. Selected Runoff Events for Small Agricultural Watersheds in the U.S. (1938-1959), Agricultural Res. Service, USDA, January 1960.

4. Hydrologic Data for Experimental Agricultural Watersheds in the U.S., 1956-59, Agricultural Research Service, USDA, Misc. Publ. 945.

5. Hydrologic Data for Experimental Agricultural Watershed in the U.S., 1960-1961, Agricultural Research Service, USDA, Misc. Publ. 994.

6. Hydrologic Data for Experimental Agricultural Watershed in the U.S., 1962, USDA, Misc. Publ. 1070.

7. Hydrologic Data for Experimental Agricultural Watershed in the U.S., 1963, USDA, Misc. Publ. 1164.

8. Hydrologic Data for Experimental Agricultural Watersheds in the U.S., 1964, Agricultural Research Service, USDA, Misc. Publ. 1194.

Observations have continued since 1964, but the data are not yet available in published form. However, copies of the necessary data sheets for the years 1965, 1966 and 1967 were obtained directly from the Agricultural Research Service Hydrologic Data Center, J. R. Burford, Manager. Data for the years beyond 1967 were not yet available.

A search was made of other data sources, e.g., SCS Technical Papers, but none of these satisfied the requirements of the study outlined above. Certain small watersheds in the U.S. Geological Survey stream gaging program probably could have been used, along with local rainfall data of the Environmental Data Service, N.O.A.A. (formerly U.S. Weather Bureau). However, such watersheds are in scattered locations and thus were not especially suited to this particular study. In some cases, the length of record is relatively short and, in many cases, there is not a recording rain gage within or near the watershed.

Several possible "study areas" were delineated as a first step. At the same time, it was decided that the analyses should be carried out only on watersheds exceeding 100 acres in size, preferably more than one square mile.

The Agricultural Research Service has a continuing program of hydrologic data collection on experimental watersheds with areas over 100 acres at the following locations:

1. Coshocton, Ohio, 1939
2. Oxford, Mississippi, 1957
3. Riesel (Waco), Texas, 1941
4. Blacksburg, Virginia, 1939
5. Safford, Arizona, 1940

The list above indicates the beginning dates of these studies, although observations for a number of the individual watersheds was begun later, including many of those selected for study. Publication No. 3 (above) lists a number of other locations with experimental watersheds over 100 acres in size, e.g., Fennimore, Wisconsin, but many of these have been discontinued, probably to concentrate the effort in selected areas.

The observed hydrologic data for the period of study were tabulated and analyzed for the first four of the five areas listed above. The Arizona watersheds were not included because they present special problems to such an analysis, due to the high variability in areal distribution of rainfall in the Southwest and the high transmission losses in the channels. Data for the Ralston Creek watershed at Iowa City, Iowa, were tabulated and considered, since records are available from 1943. However, the hydraulic analyses was not carried out for Ralston Creek or the Blacksburg watersheds.

The watersheds selected for hydrologic analysis are listed in Table 1, along with their principal characteristics. The geologic topographic and soil conditions of the Coshocton watersheds have been described by McGuiness and Harrold (1962). In general, the soils are residual soils of Hydrologic Group C, i.e., having "below average infiltration after presaturation." Subsoils are generally less permeable than the plow layer. Bedrock is usually found at depths of 5 to 8 ft. The land surface is strongly rolling and the stream gradients are moderate to steep.

TABLE 1 .Data for Watersheds Selected for Hydrologic Analyses

COSHOCTON, OHIO

WTSD #	Area (Acres)	Beginning of Record (Year)	Estimated Mean Bed Slope (Feet/Ft)	No. of Runoff Events Studied
5	349	1940	.15	15
10	122	1939	.15	17
92	920	1939	.15	15
94	1520	1939	.15	20
95	2750	1939	.15	17
97	4580	1937	.15	20
196	303	1937	.14	28

OXFORD, MISSISSIPPI

WTSD #	Area (Acres)	Beginning of Record (Year)	Mean Bed Slope (Feet/Ft)	No. of Runoff Events Studied
4	2000	1957	.0040	8
5	1130	1957	.0047	14
10	5530	1957	.0034	9
12	22800	1957	.0018	12
28	1080	1957	.0035	11
32	20000	1957	.0022	11
35	7550	1957	.0022	11

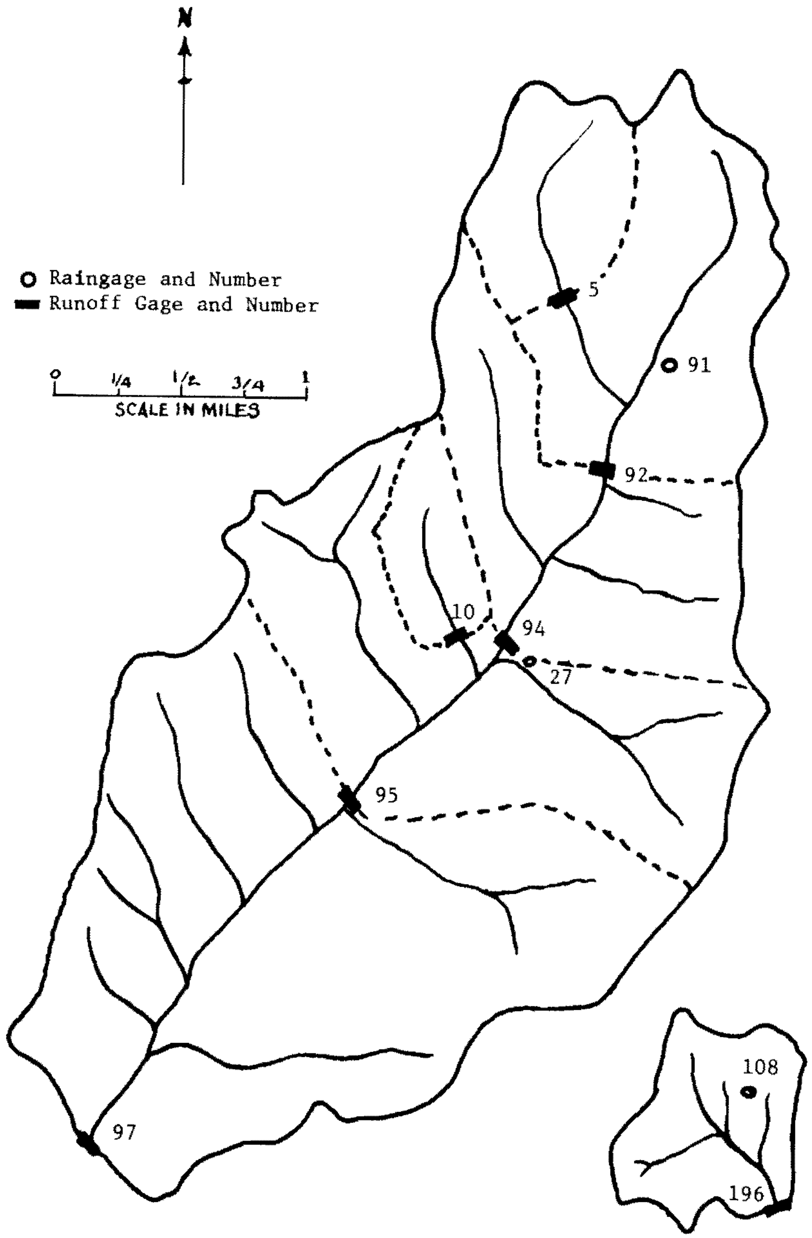


Figure 4. Coshocton, Ohio Watersheds

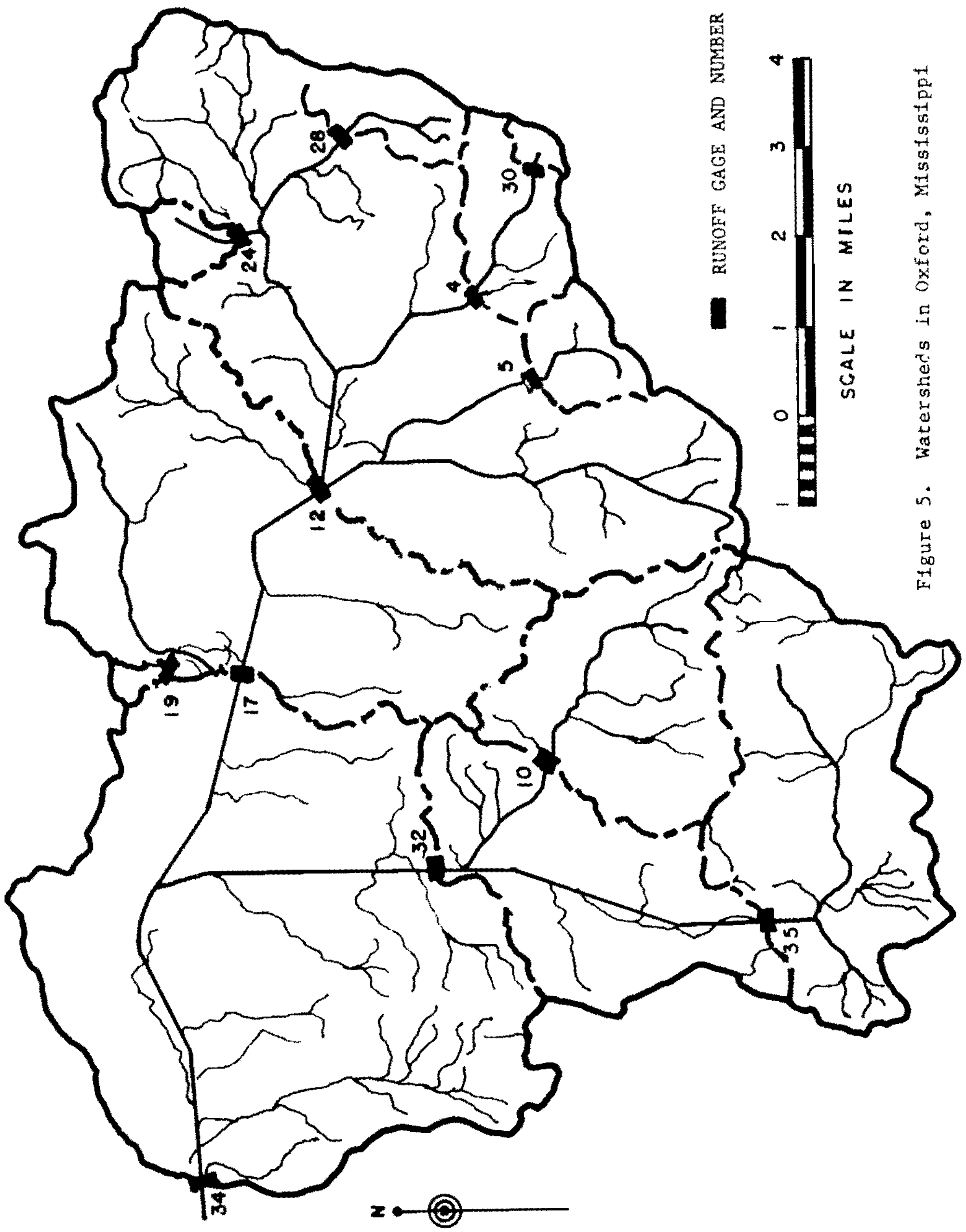


Figure 5. Watersheds in Oxford, Mississippi

The Oxford watersheds are quite different in character. Briefly, the soils are a shallow loess underlain by sand. They are, therefore, above average in infiltration capacity and without natural, restricting layers in the profile. The topography is quite rolling, as at Coshocton, and both the sheet and gully erosion rates are very high unless protection is provided.

Figures 4 and 5 show the location of the watersheds with respects to each other for the Coshocton, Ohio watersheds and for the Oxford, Mississippi watersheds, respectively. At the Coshocton location the larger watersheds are composed of several small watersheds. This is the case also in the Oxford location.

The significant runoff events are tabulated in Appendix A. Except for the base flow amounts and rates the data given in Appendix A are taken directly from the A.R.S. publications cited earlier. These are the basic data utilized in the study, along with the time distributions of both rainfall and runoff. The latter are not reproduced herein, but are available in the source publications.

IV. HYDROLOGIC ANALYSIS

The general procedure for this phase of the study was to first determine the characteristics of the rainfall excess pattern for each event, then to evaluate the hydrologic time parameter, T_{50}, for each event for further use in study. The parameter, T_{50}, time to 50 per cent of equilibrium flow is a new parameter proposed in this study. Finally, as a matter of interest, T_{50} is related to watershed area and mean runoff supply rate.

Rainfall Excess Pattern and Duration

Figure 6 gives two examples of the rainfall patterns associated with the runoff events used in the study, one rather simple, the other quite complex. In every case, one is faced with the difficult task of determining the time distribution of the excess rainfall or unrouted runoff. For the purposes of this study, it is assumed that rainfall occurring at low intensities infiltrates and is retained in the soil, while rainfall occurring at high intensities becomes runoff. The latter could move from the place of origin to the channel system as either overland flow or interflow, so far as this analysis is concerned. However, a high percentage of interflow increases the total flow time to the gaging station, tending to introduce more variability into the results.

Although the infiltration or retention rates are known to decrease with time, one cannot represent this change without applying a continuous soil moisture model and an infiltration model to each event. Although infiltration modeling is progressing, it remains a difficult job and involves some uncertainty. Considering the numerous events to be studied and the probable benefits, it was decided not to adopt such a procedure for this study. Instead, a level (constant) line of separation was used to separate rainfall excess from retention, as with the ∅-index approach frequently used in the past for infiltration. This separation is shown in the two examples (Figure 6) and the area above the line is taken as representing the rainfall excess or net runoff. The line is drawn in each case such that the area above it equals the volume of runoff observed at the gaging station adjusted for the baseflow volume, if any. This procedure is, of course, arbitrary and approximate but does provide a definite procedure that can be followed consistently and without ambiguity.

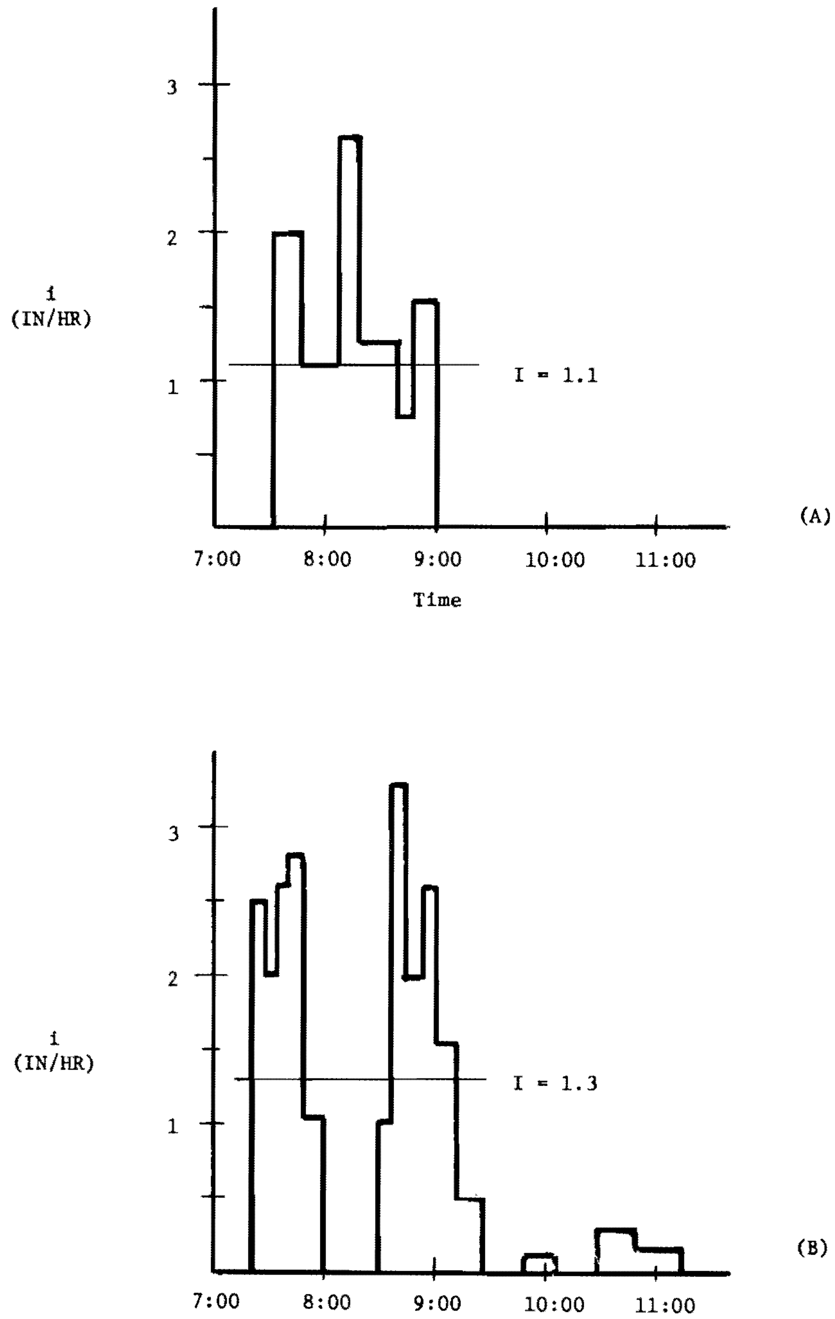


Figure 6. Two Types of Rainfall Pattern With Dividing Line Separating Excess Rainfall.

This leaves excess rainfall patterns which are sometimes quite complex, in contrast to the rectangular or triangular patterns used in the model watershed studies by Machmeier, Wei and Golany and Larson (cited earlier). For analysis, the next step required is to select a rectangular or triangular pattern which represents the actual time distribution reasonably well. This was done for each event. The results are illustrated for several events in Figure 7 as examples. In the first case, it was reasonable to use a rectangular time distribution as shown, with a duration D and a mean supply rate, $\bar{q}_s$.

In the other two cases, the rainfall excess pattern could be approximated better by use of a triangular pattern. This "fitting" was done in one of two ways, depending on which one gave the best fit. First (Figure 7b), the peak of the triangle was placed at the same level (q_s-value) as the actual rainfall excess pattern. The time for this point and the two sides of the triangle were then located so as to approximate the actual pattern reasonably well, using an asymmetrical triangle where appropriate. If the resulting fit was not good, a second method was used in which the peak of the triangle was lowered as needed (Figure 7c). This was done with only a few of the events, and proved desirable mainly where the rainfall excess was spread over a relatively long time, with a short period of high intensity rainfall excess sometime during that period.

Because the above procedure required visualization (plotting) and some use of judgment, it was carried out by hand rather than by computer. Computerization would have been possible but, without great care, could have led to illogical results that would go unnoticed. A considerable amount of time was of course required to carry out this analysis for all the events used in the study.

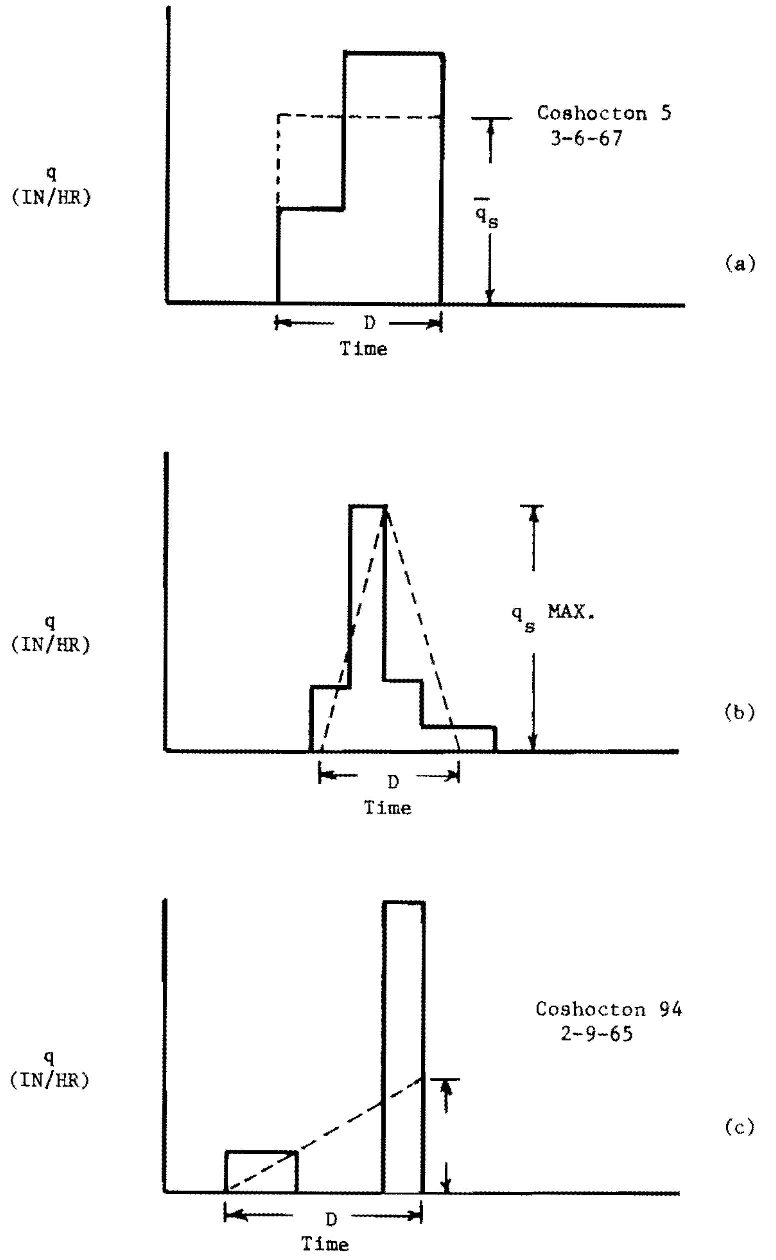


Figure 7. Excess Rainfall Patterns With Adjustments to Represent the Distribution by (a) Rectangular, (b) Triangular, and (c) Triangular With Lowered Peak.

In the following analysis and discussion which follows, the <u>rate</u> at which excess rainfall is being generated will, for convenience, be referred to as the "supply rate" and designated q_s. The units of q_s are inches per hour. The mean supply rate for a given event is designed $\overline{q}_s$, which is of course the runoff amount in inches divided by the duration in hours. Thus, rainfall excess and runoff amount are taken as being the same, even though some of the runoff may follow the path of interflow.

Supply Rates

These results are given for all events in Appendix B. The maximum q_s is the maximum supply rate of the rainfall excess pattern, prior to fitting a simplified pattern. In the next column, T1 indicates the first method of fitting a triangle described above and T2 the second method. The variable $\overline{q}_s$ is the mean rate for the pattern used, whether a rectangle or a triangle. For a rectangle, it is frequently the same as but not always equal to the maximum q_s given in the preceding column. For a triangular pattern, Type 1, $\overline{q}_s$ is one half of the maximum q_s. For a Type 2 triangle, it is one half of the triangle peak but is not equal to one half of the maximum q_s.

Using the model watershed referred to earlier, Wei and Larson (1971) found that a triangular pattern of rainfall excess increases the peak discharge as compared to a corresponding rectangular pattern. This effect was representted by a time distribution coefficient, C_t, defined as the ratio of the peak discharge for a triangular pattern to that for a rectangular pattern of the same duration and amount. The value of C_t varied with the relative location of the triangle peak within the duration, which was expressed as T_{pr}/D, where T_{pr} is the time of the peak rainfall excess. The relationships obtained for the model sub-watershed (160 acres) were averaged

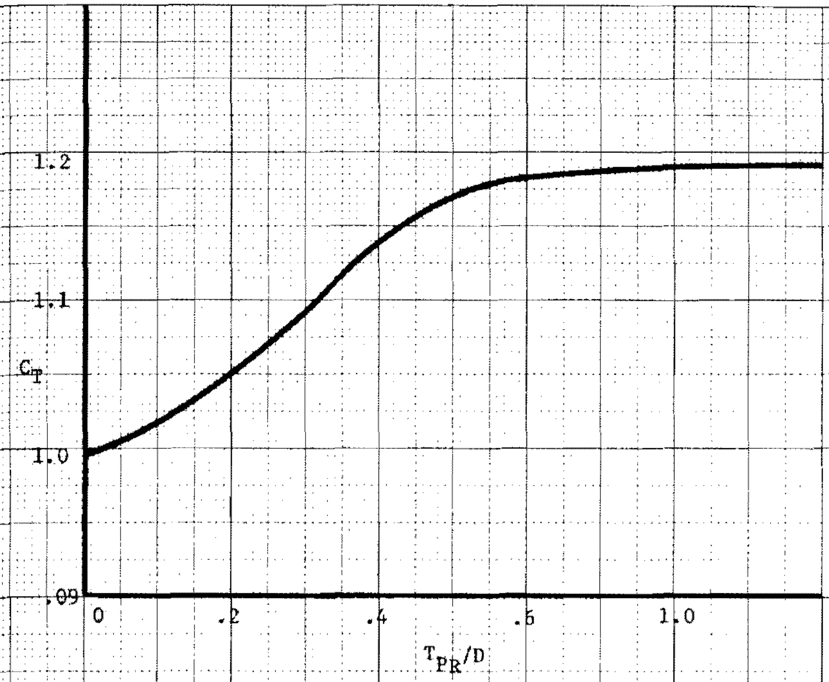


Figure 8

Plot of C_t vs. T_{PR}/D (From Wei and Larson, 1971) with those obtained the entire model watershed (935 acres) to obtain the curve given in Figure 8, which was then used in the present study. Although there is no assurance that it is applicable to watersheds of different size, it is assumed to apply equally if the rainfall excess duration is in proportion to the watershed response time. Also, since the factor itself is not large as compared to 1.0, the errors resulting from this application are not expected to be great, despite the fact that Figure 8 is not clearly applicable to watersheds of varying size and for events having various durations.

The factor C_t was then applied to each event represented by a triangular pattern to find an equivalent rectangular pattern, i.e., one that would produce the same peak discharge. The supply rate for such a pattern is given in Appendix B as $\overline{q}_s'$. It is of course slightly more than $\overline{q}_s$ and serves to put the triangular and the rectangular pattern events on the same basis for further analysis.

The Time Parameter, T_{50}

The hydrograph time parameter, T_{ve}, was used by Larson and Machmeier (1968) as an index of watershed response time. It was found to vary as supply rate, $\overline{q}_s$ to the -0.27 power for Machmeier's model watershed indicating non-linearity of surface runoff, as expected. Using a hydraulic, theoretical approach, Larson (1965) found T_{ve} to be proportional to $\overline{q}_s^{-.20}$. As a watershed time parameter, it has an advantage over such time parameters as time to peak and lag time, which depend on both supply rate and duration of input (Machmeier and Larson, 1968). However, T_{ve} has one overriding disadvantage, namely that it cannot be evaluated from observed runoff hydrographs, as indicated in Chapter II, since equilibrium flow never occurs in nature except possibly in very small watersheds.

A new time parameter, T_{50}, time to 50 per cent of equilibrium, is proposed here as a watershed time parameter which can be evaluated from runoff records. It has the same advantages as T_{ve} in that, for a given watershed, it varies only with supply rate. It has the potential for both hydrologic and hydraulic evaluation, which provides the vital linkage between gaged and ungaged watersheds needed for prediction purposes. A flow of 50 per cent of equilibrium is not reached in every runoff event, particularly those where the input supply or rainfall excess if of short duration. However, one can expect that this flow rate will be reached occasionally in each watershed, often enough to obtain a number of values of T_{50} for analysis. Knowing the effective, mean supply rate $\overline{q}_s'$ for a runoff event, T_{50} occurs at a flow rate in inches per hour equal to one half of $\overline{q}'_s$. If this flow rate is reached prior to the point of inflection on the rising limit of the hydrograph, one can simply read the value of T_{50} from the hydrograph. If it is not reached, the event cannot be used for this purpose.

Values of T_{50} determined in this way for the various runoff events are given in Appendix B in the last column. We see that it could be evaluated for only about one-half of the runoff events, more for some watersheds, less for others. A total of 46 values of T_{50} were obtained for the various Coshocton watersheds and 36 values for the Oxford watersheds. Thus, we have a hydrograph time parameter which should serve as index of the response time of a watershed, varying somewhat with supply rate but independent of duration, much the same as T_{ve}. Whether it can be evaluated hydraulically and used in predicting peak flow for an ungaged watershed remains to be determined in a later chapter of this report.

Relating T_{50} to Supply Rate and Watershed Area

The goal in this study was to develop a method applicable to ungaged watersheds, preferably a hydraulic approach. Although not required for this approach, it is of interest to study the relationships of the time parameter T_{50} to watershed area and to the mean supply rate, $\overline{q}'_s$.

In both cases trial plots indicated better results on log-log paper than on either rectangular coordinates or semi-log paper. Therefore, a straight power relationship was fitted to each data set, in the form

$$Y = a\,X^b \tag{7}$$

where X is the independent vairable and a and b are constants. This can be written as

$$\ln Y = \ln a + b \ln X \tag{8}$$

Using T_{50} as Y and $\overline{q}'_s$ or A (watershed area in acres) as X, standard linear regression techniques were used with Eq. 8 for fitting.

For T_{50} vs. A, the resulting equation, for the Coshocton watersheds, based on 46 runoff events, is

$$T_{50} = 15.0\,A^{0.33} \tag{9}$$

For the Oxford area, with 36 events, the equation is

$$T_{50} = 10.0\,A^{0.29} \tag{10}$$

We note that the two exponents are quite similar and could be brought into correspondence without greatly affecting the relationships. It is evident from values of the other constant that values of T_{50} are significantly greater for Coshocton watersheds than for Oxford watersheds. This could be caused by a variety of factors which will be discussed later.

In the study with a watershed routing model, Machmeier and Larson (1968) found T_{ve} proportional to area to the exponent 0.30. Although T_{50} is not the same as T_{ve}, they are similar in nature. In any case, the agreement between the two studies, one with a model watershed the other with field observations, is remarkably good. We should note also the simple fact that the time parameter T_{50} increases with watershed area, but at a slower rate than area, as indicated by the exponent of about 0.3.

The accuracy of Eqs. 9 and 10 as prediction equations, however, is not good. Values calculated by the equations vary widely as compared to the observed values of T_{50}. Although some T_{50} values were predicted fairly well, 10 of the 46 Coshocton values were less than half of the corresponding observed values and 10 were more than twice the observed values. For Oxford, 2 of the 36 calculated values of T_{50} were less than half of the observed values and 3 more than twice the observed value. Thus, there is less variation in the values of T_{50} at Oxford, which can be seen also by inspection of the data in Appendix B.

Repeating the procedure for T_{50} vs. mean supply rate, the resulting equations, for the Coshocton and Oxford watersheds, respectively, are:

$$T_{50} = 83.9\,(\overline{q}'_s)^{-.24} \tag{11}$$

$$T_{50} = 55.5\,(\overline{q}'_s)^{-.46} \tag{12}$$

We see that T_{50} decreases as the supply rate increases, as expected. In this case, we note a wider difference in exponents between the two locations. Again, using a $\overline{q}'s$ around 1.0 inch per hour as a basis for comparison, we see from the constants that the Coshocton watersheds have values of T_{50} about 50% higher according to Eqs. 11 and 12.

Theoretically, Larson (1965) found that T_{ve} varies as $(\bar{q}'_s)^{-.20}$. For the model watershed (Machmeier and Larson, 1968), T_{ve} varied as $(\bar{q}'_s)^{-.23}$. These exponents are similar to the ones for the Coshocton area (Equation 11).

As for the accuracy of Eqs. 11 and 12, they are no better than Eqs. 9 and 10, in which watershed areas was used instead of $\bar{q}'_s$ For the Coshocton data, 10 of the 46 calculated values were less than half of the corresponding observed values and 11 were more than twice. At Oxford, 3 values were under 50% of the observed values and 4 were more than twice. Thus, it can be concluded that neither watershed area or supply rate alone is adequate as a predictor of T_{50}.

Using a transformation similar to that employed in Eq. 8, multiple correlations were made fitting the data to an equation of the form

$$Y = a\, X_1^b\, X_2^c \qquad (13)$$

with 3 coefficients, a, b, and c, and two independent variables X_1 and X_2. Values of A and $\bar{q}'_s$ were used for X_1, and X_2, respectively, with the following result for the Coshocton watersheds:

$$T_{50} = 10.38(A)^{.33}\ (\bar{q}'_s)^{-.23} \qquad (14)$$

For the Oxford data, the resulting equation is

$$T_{50} = 12.84(A)^{.20}\ (\bar{q}'_s)^{-.36} \qquad (15)$$

The exponents in Eqs. 14 and 15 are generally similar to those obtained by relating T_{50} to A and $\bar{q}'_s$ separately. This indicates that the two input variables, A and $\bar{q}'_s$, are to a large degree independent, as assumed. The differences could easily be caused by data errors or other factors not included in the analysis. Thus the discussion given above concerning the

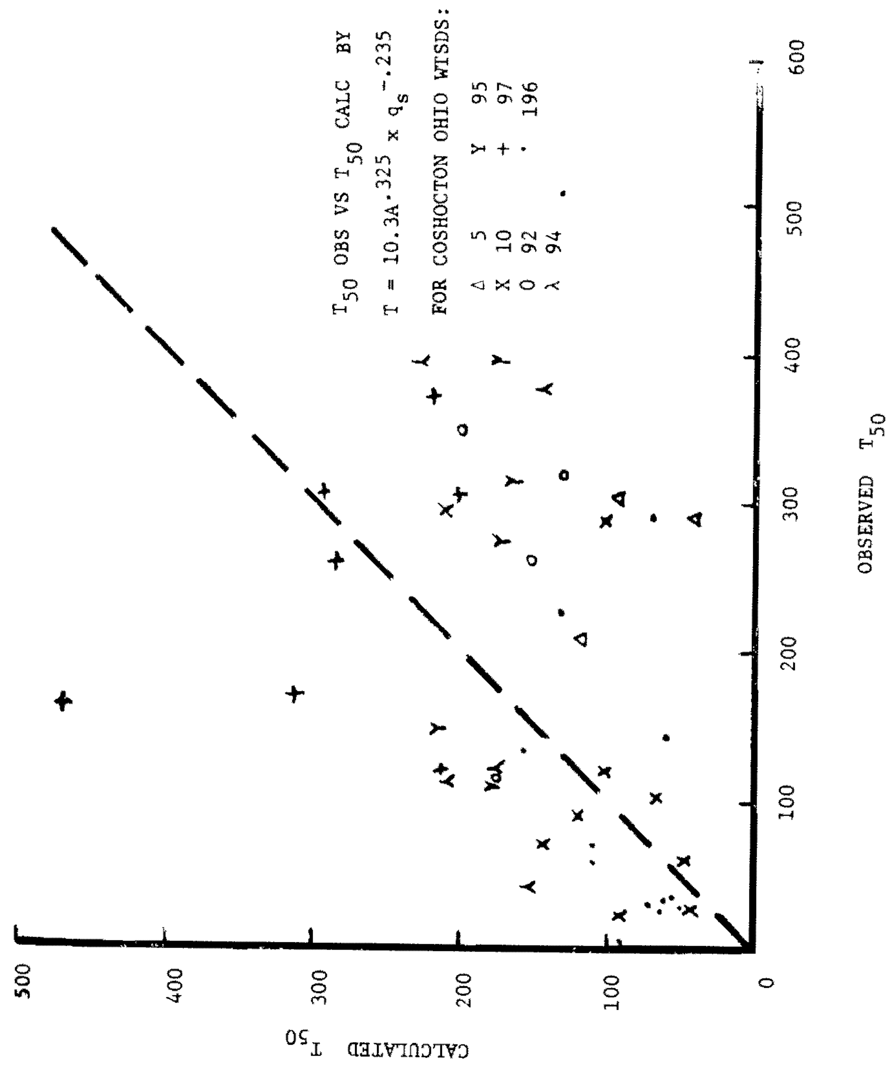


Figure 9. Plot of T_{50} Observed vs. T_{50} Calculated by $T = 10.3A^{.325} \times q_s^{-.235}$, for Coshocton, Ohio Watersheds

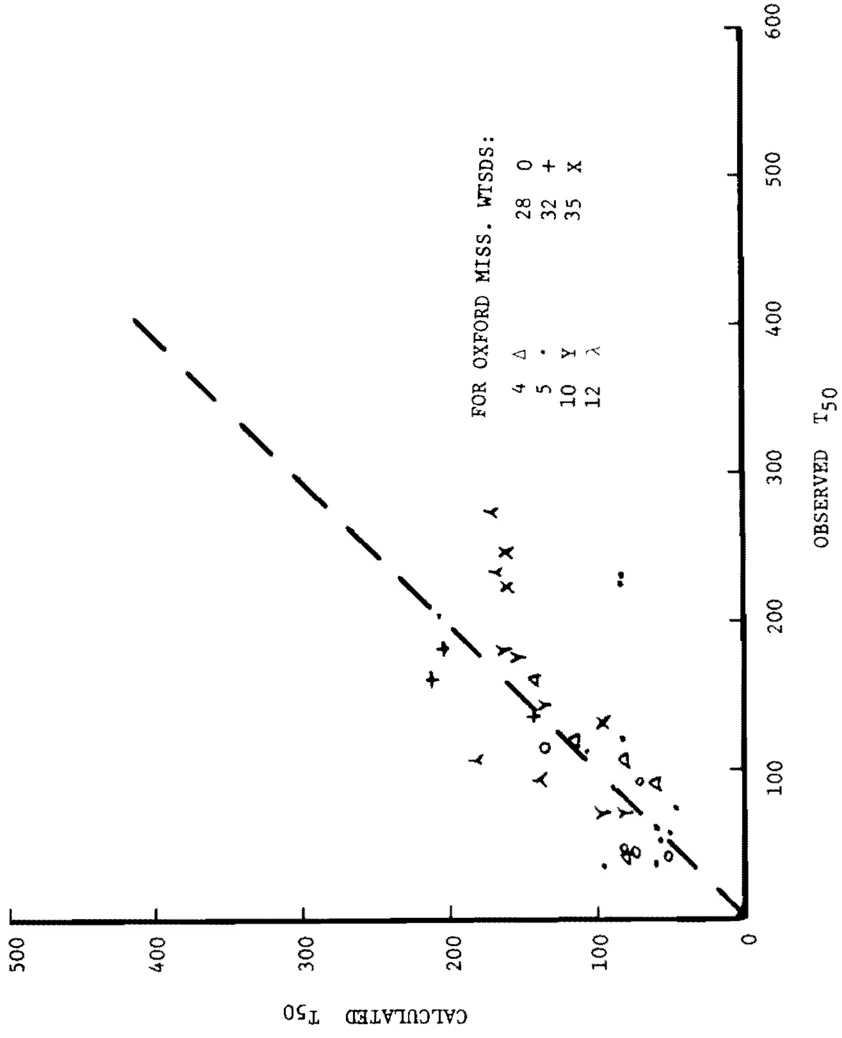


Figure 10. Plot of T_{50} Observed vs. T_{50} Calculated by $T_{50} = 12.8A^{.20} \times q_s^{-.36}$, for the Oxford, Miss. Watersheds

magnitudes of the exponents, comparing them to those obtained in other studies, would apply also to the exponents in Eqs. 14 and 15.

The differences in the exponents between the Oxford and Coshocton data are more difficult to explain. The results indicate that the supply rate has a greater effect for the Oxford watersheds. Whether or not this difference is truly significant was not determined.

The predictive accuracy of Eqs. 14 and 15 is somewhat better than those using only A or $\bar{q}'_s$, but still not good. A visual comparision for each set of data is given as Figures 9 and 10 The results for Oxford are fair but those for Coshocton are still poor indicating that other factors must be involved. However, there is no indication that any of the watersheds give values that are consistently high or low compared to the equations. Thus, it seems unlikely that the introduction of some watershed characteristic, e.g., channel or land slope, as an additional independent variable would improve the results. More likely, some undetermined characteristic of the runoff events is influencing the results.

For convenience q_s will be used to represent $\bar{q}'_s$ in the remainder of this report.

V. DATE ON CHANNEL CHARACTERISTICS

The hydraulic approach to determination of peak flow or flow parameters requires data on the physical characteristics of the watershed, especially channel characteristics. In this study, such data were needed both for development purposes and for testing proposed methods. The collection of these data are described here, their analysis in the next chapter.

For the analysis, a series of cross-sections were required for each watershed as well as channel slopes and roughness. The hydraulic approach to T_{50} is concerned primarily with the main channel and not the lateral or upstream tributaries, as described later. Thus, channel cross-section data are not required throughout the watershed. This is a helpful feature of the general procedure, especially in application.

At the Coshocton station, channel cross-sections were not available for most of the channels. Thus, it was necessary to conduct field surveys for this purpose. The second author conducted these surveys with the able assistance of a senior agricultural engineering student from Ohio State University, George Wallace, and general guidance from the ARS staff at Coshocton. Complete cross-sections were taken at five or more stations along the principal channels of the various watersheds. Channel lengths between cross-sections were measured and a common datum was used for all cross-sections, making it possible to calculate channel slopes.

For the Oxford watersheds, cross-section data were available in the files of the USDA (ARS) Sedimentation Laboratory. With the help of the staff at Oxford, the necessary data were selected from the files for machine-copying and subsequent use. For some of the channels, cross-sections have been measured on several occasions to note the progress of channel erosion. For the purposes of this study, cross-sections measurements made in or near the year 1966 were utilized. For the Reisel, Texas, watersheds, the limited cross-section data available were received by mail from the ARS staff at Reisel.

Determining the appropriate roughness coefficient or coefficients for the various channels was a difficult problem, as usual. The only way to measure the coefficient is by slope-area measurements combined with a

discharge measurements. This was not done because of the time and effort required and the fact that such a measurement would probably be applicable only to a small base flow, rather than the substantial discharge represented by major runoff events. Furthermore, the Oxford channels and some of the Coshocton channels are normally without flow.

Accordingly, estimates of the Manning roughness coefficient were made following a visual observation of the general appearance of the channels in each study area (except Reisel). A series of channel photographs along with measured channel roughness has been published (Barnes, 1967). For the most part, the channels shown are of perennial streams somewhat larger in size and drainage area than those of this study. The estimated roughness for the channels at the three locations (Manning's n) were as follows:

Coshocton - 0.045
Reisel - 0.040
Oxford - 0.040

No attempt was made to vary the roughness coefficient according to depth or discharge. Neither was there any attempt to separate the flow into channel flow and flood plan flow, with different roughness coefficients for each, if flood plain flow should occur.

For each cross-section, after plotting, a bank-full elevation was established by inspection. Typical cross-sections at Coshocton and Oxford are shown in Figure 11. The bank-full line was drawn to the point on either bank where a sharp break in the cross-section was noted. If such a break occured on both sides, the lower one was used. With the bank-full line established, the corresponding flow area, A_{BF}, could be measured from the cross-section and also the wetted perimeter, P_{BF}.

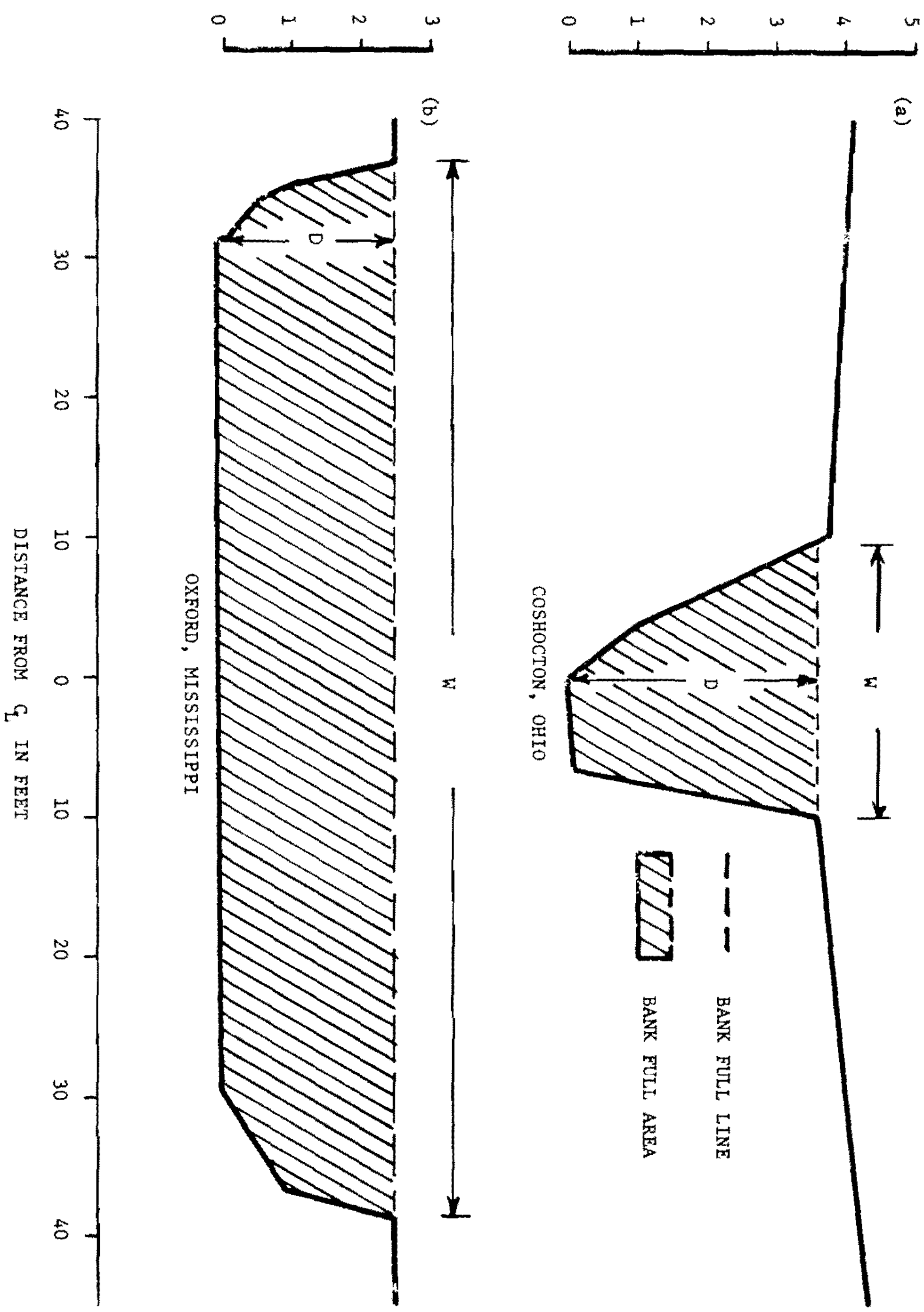


Figure 11. Sketch of Typical Cross Sections of Channels in (a) Coshocton, Ohio, (b) Oxford, Miss.

For each cross-section the bank-full width, W_{BF} (the topwidth) was measured and recorded. Also, the channel depth, D_{BF}, the depth to the low point of the cross-section (Figure 11) was determined for each.

Values for the A_{BF}, P_{BF}, W_{BF} and D_{BF} for all the cross-sections used in the study are given in Appendix C.

VI. ANALYSIS OF CHANNEL CHARACTERISTICS

For the engineering hydrologist concerned only with estimating peak flow for the design of a culvert or other small structure, measurement of a series of channel cross-sections probably would not be considered practical, considering the effort and cost of doing so. If channel characteristics are to be used in hydrologic predictions, they should be limited to characteristics that can be determined either from standard topographic maps or by simple field measurements. Thus, the purpose of the foregoing analysis is to determine whether simple measurements can be substituted for complete cross-section data.

In making hydraulic calculation in channels, one needs values of the cross-section area (A) and the hydraulic radius (R). The latter might be determined directly or instead one could determine the wetted perimeter (P) and then calculate R from A and P. The width and depth at bank-full flow (W_{BF} and D_{BF}) are easily measured characteristics that might be correlated with the required characteristics just referred to for purposes of prediction. This possibility was explored by Gronwald (unpublished) as a part of this study and is described in the following pages.

All the variables used in this analysis are for bank-full flow. Thus for convenience, the subscript BF will be omitted in this chapter and it will be understood that A, P, W, and D represent the values for bank-full.

Leopold and Maddock (1953) and Leopold and Miller (1956) showed that channel characteristics have generalized relationships and, therefore, can be related to flow at the station (with increasing depth) and also to the increasing flow from station to station. Although their analyses do not appear to be of direct use in the current study, they do provide a precedent and some encouragement.

Three types of general relationships were studied, utilizing the cross-section data for the Coshocton and the Oxford watersheds, as follows:

$$A = f\ (W,D)$$
$$R = f\ (D)$$
$$P = f\ (W)$$

As indicated above, only the first and one of the last two of these is actually needed.

Coshocton Watersheds

First it was assumed that the cross-section area, A, is a function of the product W x D for a given region. Plotting A vs. WD on linear coordinates indicated that a linear relationship would fit the data reasonably well, although there appeared to be slight curvature in the relationship. Three alternative methods were tried with the Coshocton data for Little Mill Creek: (1) a linear equation with a non-zero intercept, (2) a linear equation through the origin and (3) a power equation of the form $Y = aX^b$. Although Method No. 1 yielded slightly better correlation coefficients than Method 2, there is no theoretical basis for the non-zero intercept obtained with Method 1. The power equation improved the fit slightly by using values of the exponent b somewhat less than one (around 0.9). However, both

Methods 1 and 3 require two coefficients and Method 2 requires only one. Thus, after some testing, which will not be fully reported here, it was decided to adopt the relationship

$$A = C_1\ (WD) \qquad (16)$$

where C_1 is the coefficient which one can expect it to be between 0.5 for a triangular channel and 1.0 for a rectangular channel.

In a similar manner, the three different methods for relating hydraulic radius to depth were tried for the Little Mill Creek data at Coshocton. In this case, the scatter of the points on a plot of R vs. D was somewhat greater. The differences between the three methods was greater than for A vs. WD. However, because of the variability in the data analyzed in this way, one could not say that one method was clearly better than another. The correlation coefficients were distinctly lower than those for the case of A vs. WD. Thus, following the same reasoning as before, it seemed most appropriate to again give preference to a single coefficient, linear relationship,

$$R = C_2 D \qquad (17)$$

in which C_2 has a limiting value of 1.0.

The results obtained relating wetted perimeter to top width (in a similar fashion) were considerably better than for R vs. D. There was little difference between results by the three types of relationships given above and, therefore, a single parameter, linear relationship was adopted, as follows:

$$P = C_3 W \qquad (18)$$

In this case, one sees that the coefficient C_3 has a lower limit of 1.0.

The best-fit lines for Equations 16, 17 and 18 were obtained by imposing two conditions (1) the line must pass through the origin and (2) the sum of the squares of the deviations are minimized. The latter is accomplished by using

$$b = \frac{\Sigma XY}{\Sigma X^2} \tag{19}$$

where b is the slope of the line (C_1 C_2 or C_3) and X and Y are the independent and dependent variables, respectively, e.g., W and P in Eq. 18.

Equations 16, 17 and 18 were then fitted to all the Coshocton data (30 cross-sections) with the following results:

$$A = 0.530\ WD \tag{20}$$
$$R = 0.575\ D \tag{21}$$
$$P = 1.048\ W \tag{22}$$

Values of the coefficient of determination, r^2, for Eqs. 20, 21, and 22 were 0.86, 0.73, and 0.99, respectively. Obviously, Eq. 22 is much better than Eq. 21 and, therefore, Eq. 21 was not used further.

With a least square linear equation forced through the origin, it is evident that the large values of X and Y dominate the fit and that the small values have little influence in determining the coefficient. To explore this point, the values at Coshocton for those cross-sections less than 100 sq.ft. in area were taken separately. The results for A and P, using 11 cross-sections, are as follows:

$$A = .702\ WD \tag{23}$$

$$P = 1.131\ W \tag{24}$$

with r^2 values of 0.95 and 0.93, respectively. A separate set of equations for cross-section areas over 100 sq.ft. was not considered necessary, since they would be very similar to those already obtained for the entire set of data (Eqs. 20 and 22). Eqs. 23 and 24 were not used in the hydraulic analysis.

Oxford Watersheds

For the Oxford watersheds, a somewhat different procedure was followed in order to provide a test of the relationships derived from the Coshocton data. The relationships of Eqs. 16, 17 and 18 were assumed to be generally applicable with coefficients that might vary somewhat from one region to another. Thus, the coefficients were reevaluated for the Oxford area.

For this purpose, the Oxford watersheds were divided into two groups of five watersheds each, similarly distributed in terms of size (Table 2). Both groups have a wide range of watershed area. Group A was used for fitting Eqs. 16-18 and Group B was used for testing the resulting equations.

Table 2. Division of Oxford Watershed Into Two Groups For Fitting and Testing

Group A			Group B		
Watershed Number	Area Acres	No. of Sections	Watershed Number	Area Acres	No. of Sections
24	511	4	19	243	5
5	1130	5	28	1080	5
4	2000	5	17A	3200	6
10	5530	6	35	7550	6
32	20000	5	12	22800	5

The equations obtained by fitting to the observed data for the Group A watershed (25 cross-sections) are:

$$A = 0.852\ WD \tag{25}$$
$$P = 1.15\ W \tag{26}$$
$$R = 0.712\ D \tag{27}$$

The corresponding r^2-values were 0.99, 0.99 and 0.94, indicating good fits, especially for A vs. WD and P vs. W. As at Coshocton, the expression for P is superior to the one for R.

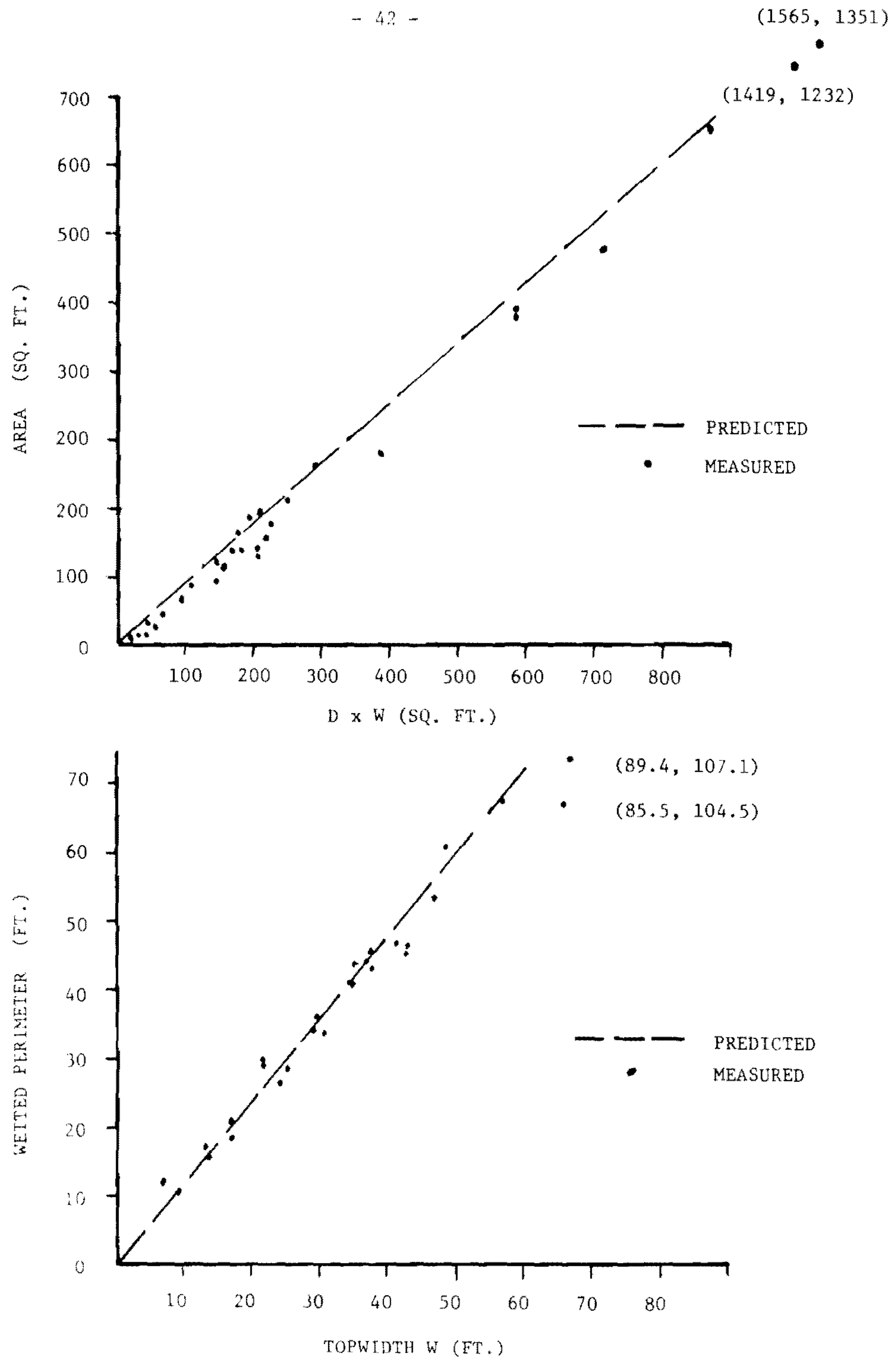


Figure 12. Plot of Cross Sectional Area of Channel vs. Depth X Width of That Channel Section

Eqs. 25 and 26 were then used to make independent predictions of area and wetted perimeter for the watersheds in Group B. Measured values of W and D for the cross-sections of these watersheds were used in Eqs. 25 and 26. The measured values of A and P are shown in Figure 12 along with the prediction equations (dashed lines) for comparison. For the Oxford watersheds Eqs. 25 and 26 were judged to be good to excellent as prediction equations, using coefficients determined from <u>other</u> watersheds in the same area. For channel area, 90% of the predicted values were within ±40% of the measured values and 50% were within ±11%. For wetted perimeter, 90% of the predicted values were within ±14% of the measured values and 50% were within ±4%.

Eqs. 25 and 26 were, therefore, adopted as prediction equations for the Oxford region. Equation 27 was not used further, being less accurate and unnecessary.

VII. HYDRAULIC ANALYSIS

To make hydraulic predictions for ungaged watersheds, the general method of this study requires <u>hydraulic</u> determination of the time parameter T_{50}. The development and testing of some method for doing this is the main objective of this chapter. In general terms, it will involve the use of channel characteristics and flow calculations.

Basis for Hydraulic Time Parameter

First, a hydraulic basis is needed for the time parameter, T_{50}. Hydrologically, T_{50} is defined as the time required for the watershed outflow to reach 50 per cent of the supply rate, $\overline{q}_s$ (assumed constant) in corresponding units. Hydraulically, it will be defined in terms of the appropriate flow or travel time. The rationale for this is similar to that used earlier for time to virtual equilibrium by Larson (1965).

We begin by visualizing the watershed area divided into an upper and lower half by a line (iso chrone) drawn such that at the time T_{50}, only the lower half is contributing to the flow at the outlet (Figure 13). Runoff is occurring from the upper half, but it has not yet reached the watershed outlet. T_{50} is then taken as the travel time along the principal channel from the 50% isochrone to the watershed outlet. This distance is designated as L_{50}. The variable T_{CH} will be defined as the travel time determined in this way, hydraulically, to distinquish it from the comparable quantity T_{50}, which is determined hydrologically. Ideally, the two would prove to be equal.

Travel times over the distance L_{50} can be calculated by reaches, as indicated in Figure 13. Thus, over the distance L_{50}

$$T_{CH} = \Sigma \frac{Li}{Vi} \tag{28}$$

where L_i and V_i are the length and the mean velocity for individual reaches. The lengths are easily defined and measured, but the mean velocity is more difficult to determine. The desired velocity is for a particular discharge associated with the rising limb of the hydrograph. To determine that discharge at various stations, we visualize a drop of water originating at time zero (the beginning of excess rainfall) at the intersection of the main channel and the 50% isochrone and traveling to the watershed outlet. At the starting point, the discharge is the base flow, which way be zero. (The same is true for all points along the channel.) However, the discharge builds up

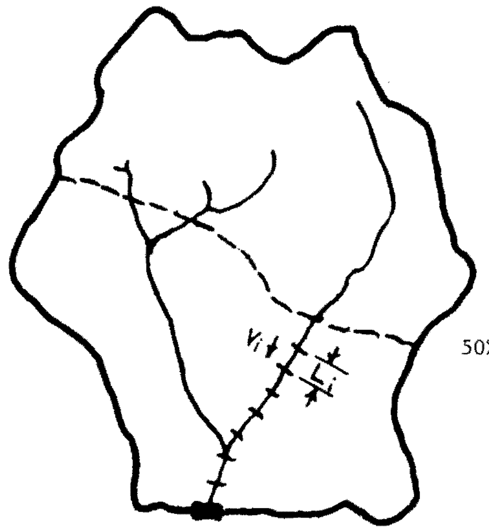


Figure 13. Diagram of Watershed Divided Into Upper and Lower Half by 50% Isochrone

as this drop moves toward the outlet. If runoff is being generated uniformly over the watershed, the discharge at a particular station when this drop arrives will be in proportion to the watershed area contributing to the flow. This will be the area tributary to the channel position being considered but below the 50% isochrone, which can be measured for each cross-section location. Knowing the mean discharge for each reach and the channel characteristics, including slope, one can calculate the mean velocity for each reach for use in Eq. 28.

Determining Mean Velocity

At this point we recall that, in the preceding chapter, prediction equations were developed for bank-full area and wetted perimeter. The flow as described above is probably much less than the bank-full flow but, for a major event occurring in a watershed with a small main channel, could be greater than the bank-full flow. There are perhaps several ways in which one could attempt to resolve this problem, one of which was selected for this study. The flow velocities for use in Eq. 28 were first calculated at each station for bank-full discharge. They were then adjusted to take into consideration the actual discharge, calculated from the tributary area as indicated above. This involves use of a generalized relationship between discharge and mean velocity, which will be described next.

Brakensiek (1963) presented log-log plots of conveyence ($AR_{2/3}$) vs. area for two cross-sections of a stream in Arkansas. For the channel portion of the plots, a straight line was obtained with slopes of 1.41 and 1.36 for the two stations. The senior author of this report has made analyses of trapezoidal channels with various bottom widths and depth (unpublished) and found that a log-log plot of discharge vs. area yields a straight line (very

nearly) with a slope of 1.37 to 1.40, agreeing very closely with Brakensiek's values for a natural channel. This can be expressed as

$$Q = K_1 A^x \tag{29}$$

where K is a constant for a given channel reach with uniform flow and x is the slope of the plots referred to above, around 1.39 for trapezoidal or similar natural channels. Substituting Q/V for A in Eq. 29 and solving for V,

$$V = K_2\, Q^{\frac{x-1}{x}} \tag{30}$$

where K_2 is another constant. Using x = 1.39, one obtains

$$V \propto Q^{0.29}$$

Leopold and Miller (1956) have analyzed a larger number of natural channel cross-sections in various regions and related topwidth, mean depth, and mean velocity at individual cross-sections to the discharge. For mean velocity, they obtained the general power equation

$$V = c\, Q^{0.34} \tag{32}$$

although there was considerable variation from this for individual values. The above relation is seen to be quite similar to Eq. 31. For the hydraulic analysis which follows, mean velocity was taken to be proportional to discharge to the 1/3 power.

Returning to the determination of T_{CH} by use of Eq. 28, the procedure for calculating bank-full discharge (Q_{BF}) and velocity (V_{BF}) have been given above. The basis for determining the discharge at each cross-section for 50 per cent of equilibrium (Q_{50}) was given also. To find the velocity corresponding to this discharge (V_{50}), the proportionality given above was used, i.e.,

$$V_{50} = V_{BF} \left(\frac{Q_{50}}{Q_{BF}}\right)^{1/3} \tag{33}$$

In most cases, Q_{50} was less than Q_{BF}. Wherever Q_{50} was found to be greater than Q_{BF}, the bank-full velocity was used arbitrarily for V_{50} instead of Eq. 33. If the discharge is greater than at bank-full, it was reasoned, flood plain flow would be occurring and the mean velocity of the total flow is probably close that at bank-full, unless the discharge is quite high.

These calculations were carried out by computer for each runoff event, using the mean supply in inches per hour as determined in the hydrologic analysis of Chapter IV. This gave Q_{50} for each cross-section for use in Eq. 33 and the values of V_{50} thus calculated were averaged over each reach and used in the summation of travel times by Eq. 28.

Locating the 50% Isochrone

One aspect of the analysis not yet described was the method of locating the isochrone which divides the watershed into two equal parts. Actually, three different methods were explored, all of which are approximate. A highly accurate procedure would be very difficult to establish. Moreover, it was not deemed necessary for this analysis.

The three methods investigated are the arc method, the angle method, and the distance-slope method. The simplest and most definitive of these, though not necessarily the best, is the arc method (Figure 14a). By trial, using the watershed outlet as the center, an arc is drawn across the watershed such that it is divided into two equal areas. For the angle method (Figure 14b), a 60-degree angle was used on both sides of the principal channel, measured from the general direction of the channel. This method is relatively simple to apply but does leave some room for judgment, which has both advantages and disadvantages. Where the main channel branched into two more or less equal parts before reaching the 50% line, a double angle was used (Figure 14c). Whether a single or double (120-degree) angle was used, the position of the lines were located by trial and error to obtain equal areas above and below.

The distance-slope method involves an approximation based on the slope as well as distance. Applying the Manning formula and ignoring the variation in hydraulic radius from starting point to the watershed outlet, one obtains travel time proportional to $L^{1.5}/H^{0.5}$, where L is the distance and H the difference in elevation. A mean slope H/L was used in this. Using the Leopold and Maddock (1953) generalized relationship that mean depth increases along a channel with $Q^{0.40}$, subsituting D for R and using Q proportional to L, one obtains travel time proportional to $L^{1.23}/H^{0.50}$.

Another approximation is the Kirpich equation (1940) for time of concentration, also a travel time, which is given as proportional to $L^{.77}/S^{.38}$ or $L^{1.15}/H^{.38}$.

Considering each of these and also the time required for calculation, the parameter $L/H^{.50}$ was selected as a basis for the distance-slope method. A 50% isochrone was drawn as a line having equal values of $L/H^{.50}$. (Figure 14d). This necessitated calculation of this quantity for numerous points, interpolation and trial positions of the line. Thus, it proved to be a laborous process as compared to the arc and angle methods.

Calculations for T_{CH} were carried out by all three methods for the Coshocton watersheds. These results are given in Table 3 for each of the four largest events on each of the seven watersheds. Using the distance-slope method as a base, the mean ratio of T_{CH} values by the arc method to the distance-slope values was 1.18. For the same events, the mean ratio of T_{CH} values by the angle method to the distance-slopevalues was 1.17. Thus, the arc and angle methods gave slightly higher values, on the average, than the distance-slope method. The differences were small, however, and there was no way of determining which method is most nearly correct. Thus, there was little basis for choosing between the three methods. Considering the simplicity and ease of application with the arc method, it was selected for further use in the study. An additional advantage of the arc method is the fact that it requires no judgment and therefore gives consistent results.

For the Oxford watersheds, only the arc and angle methods were used. The results are given in Table 4 for the four largest events on each of the five watersheds used in the study. Comparing the two by ratios, we see that these two methods give very similar results, on the average. The mean ratio of T_{CH} by the arc method to T_{CH} by the angle method was 0.89.

For the single watershed suitable for this purpose at Reisel, Texas (Watershed G), there were only three runoff events available and seven

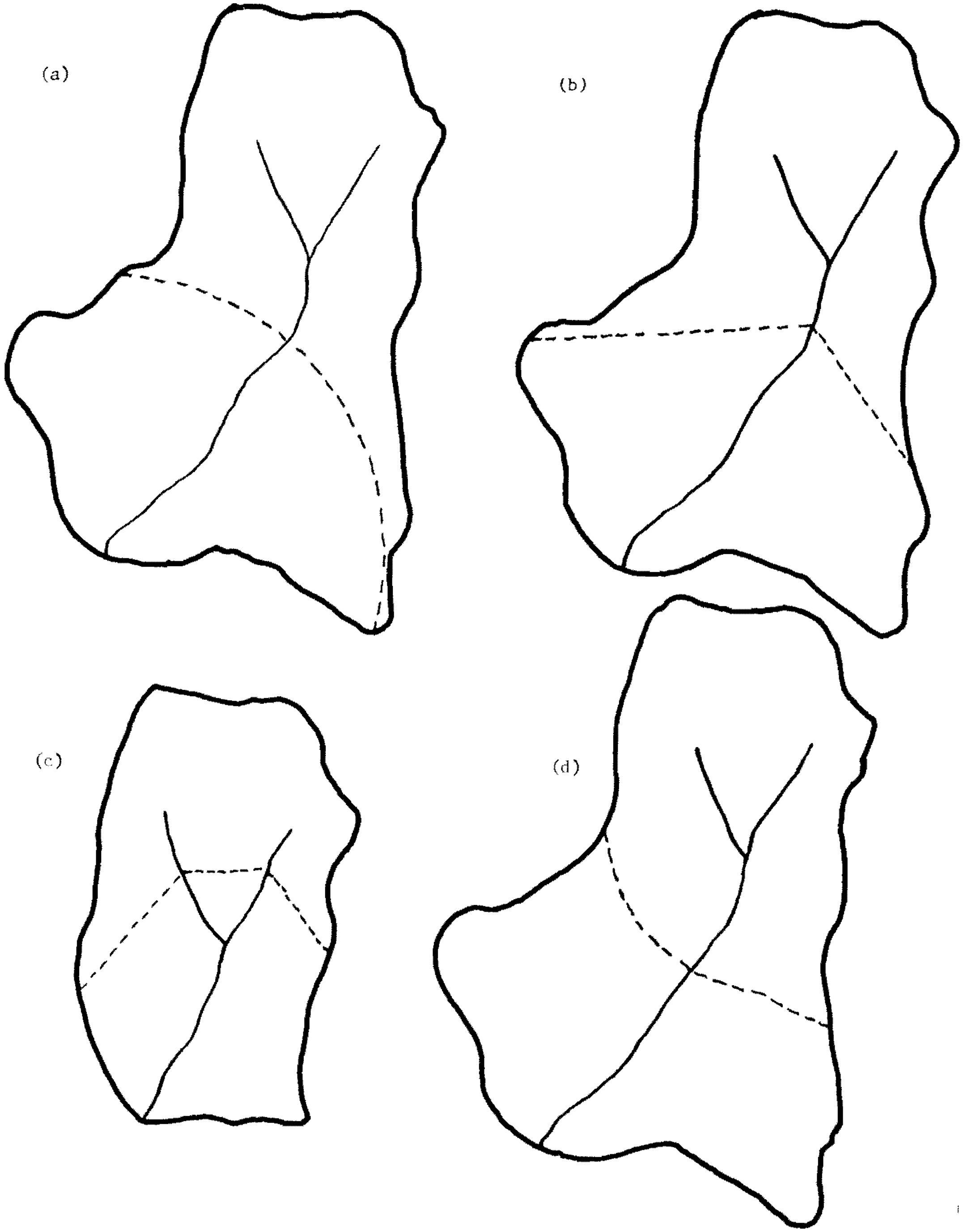


Figure 14. Various Methods Used for Locating the 50% Isochrone: (a) Arc, (b) Angle, (c) Doubel Angle and (d) Distance-Slope Methods.

cross-sections. The channel cross-section for this watershed appears to be rather small. Nevertheless, the arc and angle methods were applied to the data. For the events of 3-29-65, 6-23-59, and 11-4-59, values of T_{CH} by the arc method were 133.4, 136.2 and 157.2 minutes, respectively. By the angle method, they were 146.7, 153.5 and 187.6 minutes, respectively. The corresponding values of T_{50} were earlier found to be 95, 151, and 205 minutes.

T_{CH} as a Predictor of T_{50}

T_{CH} is intended as a predictor of T_{50}, which is determined from runoff hydrographs (Chapter IV). Ideally, the two would be the equal, so that T_{CH} could be used to predict T_{50} directly, without modification. This did not prove to be the case, as seen in the following paragraphs.

Values of T_{CH} by the arc method and the corresponding values of T_{50} were compared by plotting one against the other. This was done on a log paper due to the wide range of values and the high variability in values of T_{50}. For this, all events for which T_{50} could be evaluated (Chapter IV) down to $\bar{q}_s$ = 0.05 in/hr. were used.

The resulting plot for the Coshocton watersheds is given as Figure 15. Considering that the scales are logarithmic, the amount of scatter is indeed large. The probable reasons for this are included in the discussion section, Chapter IX. Despite this high variability, a line was drawn to provide a means of predicting T_{50} from T_{CH} for the Coshocton region. A simple linear relationship beginning at T_{50} = 0 for T_{CH} = 0 was desired, and appeared to be virtually as good as a line drawn without such constraints. Accordingly, the line was drawn with a slope of 1.0, at a position such that the sums of the squares of the deviations was minimized. Actually, this was done by computation, arriving at the "best" value of the constant (thus defined) by

TABLE 3 Results of Calculations for T_{CH} for Coshocton Watersheds

Coshocton WTSD	$\bar{q}$'s (IN/HR)	T_{50} (MIN)	T_{ch} Dist-Slope Method (MIN)	T_{CH} Angle Method (MIN)	T_{CH} ARC Method (MIN)	Ratio: Angle / Dist. Slope	Ratio: ARC / Dist. Slope
5	.25	303	9.5	11.1	10.9	1.17	1.15
5	.10	209	12.4	13.8	13.7	1.11	1.10
5	.05	291	16.2	17.3	17.4	1.07	1.07
10	1.84	28	8.4	8.5	8.5	1.01	1.01
10	1.11	62	10.0	10.1	10.1	1.01	1.01
10	.26	103	16.5	16.6	16.6	1.01	1.01
10	.07	24	25.9	26.2	26.1	1.01	1.01
92	.25	318	17.1	26.5	23.4	1.55	1.37
92	.13	263	20.5	32.1	28.3	1.57	1.38
92	.07	115	25.2	39.5	34.8	1.56	1.38
92	.04	350	30.3	47.4	41.8	1.56	1.38
94	.35	377	38.2	33.8	36.4	.88	.95
94	.27	44	39.0	34.1	37.2	.87	.95
94	.15	121	41.3	35.3	39.1	.85	.95
94	.07	113	47.0	38.5	44.2	.82	.94
95	.43	315	30.4	31.1	37.7	1.02	1.24
95	.35	275	30.8	31.8	38.1	1.03	1.24
95	.31	112	31.0	32.3	38.4	1.04	1.24
95	.14	144	35.4	37.8	43.0	1.07	1.21
97	.38	305	59.1	66.2	62.0	1.12	1.05
97	.31	120	61.1	69.3	64.7	1.13	1.06
97	.27	372	63.3	72.0	67.1	1.14	1.06
97	.08	263	92.5	105.9	98.7	1.14	1.07
196	3.88	30	4.6	6.9	7.4	1.50	1.60
196	2.29	37	5.1	7.4	7.9	1.45	1.55
196	1.68	35	5.5	7.8	8.2	1.42	1.49
196	1.55	145	5.7	7.9	8.4	1.39	1.47

TABLE 4 Results of Calculations for T_{CH} for Oxford Watersheds

Oxford WTSD.	$\overline{q}$'s (In/Hr)	T_{50} (Min)	T_{ch} Angle Method (Min)	T_{CH} ARC Method (Min)	Ratio: ARC / Angle
4	.92	88	26.8	24.6	.92
4	.40	104	35.5	32.6	.92
44	.15	119	49.6	45.5	.92
44	.08	160	62.4	57.2	.92
5	1.42	72	20.4	18.9	.93
5	1.03	57	22.7	21.0	.93
5	.71	50	25.8	23.8	.92
5	.65	35	26.6	24.6	.92
10	.66	70	48.5	30.7	.63
10	.42	70	56.6	35.8	.63
10	.17	144	77.5	49.0	.63
10	.12	175	87.5	55.3	.63
12	.34	91	67.3	59.2	.88
12	.20	233	80.2	70.4	.88
12	.19	273	81.2	71.2	.88
12	.16	107	86.5	75.9	.88
28	.94	40	18.6	17.3	.93
28	.37	91	25.5	23.8	.93
28	.35	43	26.0	24.2	.93
28	.27	41	28.3	26.4	.93
32	.29	135	101.6	93.2	.92
32	.10	183	142.7	130.9	.92
32	.09	161	148.7	136.4	.92
35	.53	130	61.5	47.0	.76
35	.12	246	102.5	78.3	.76
35	.12	222	102.8	78.6	.76

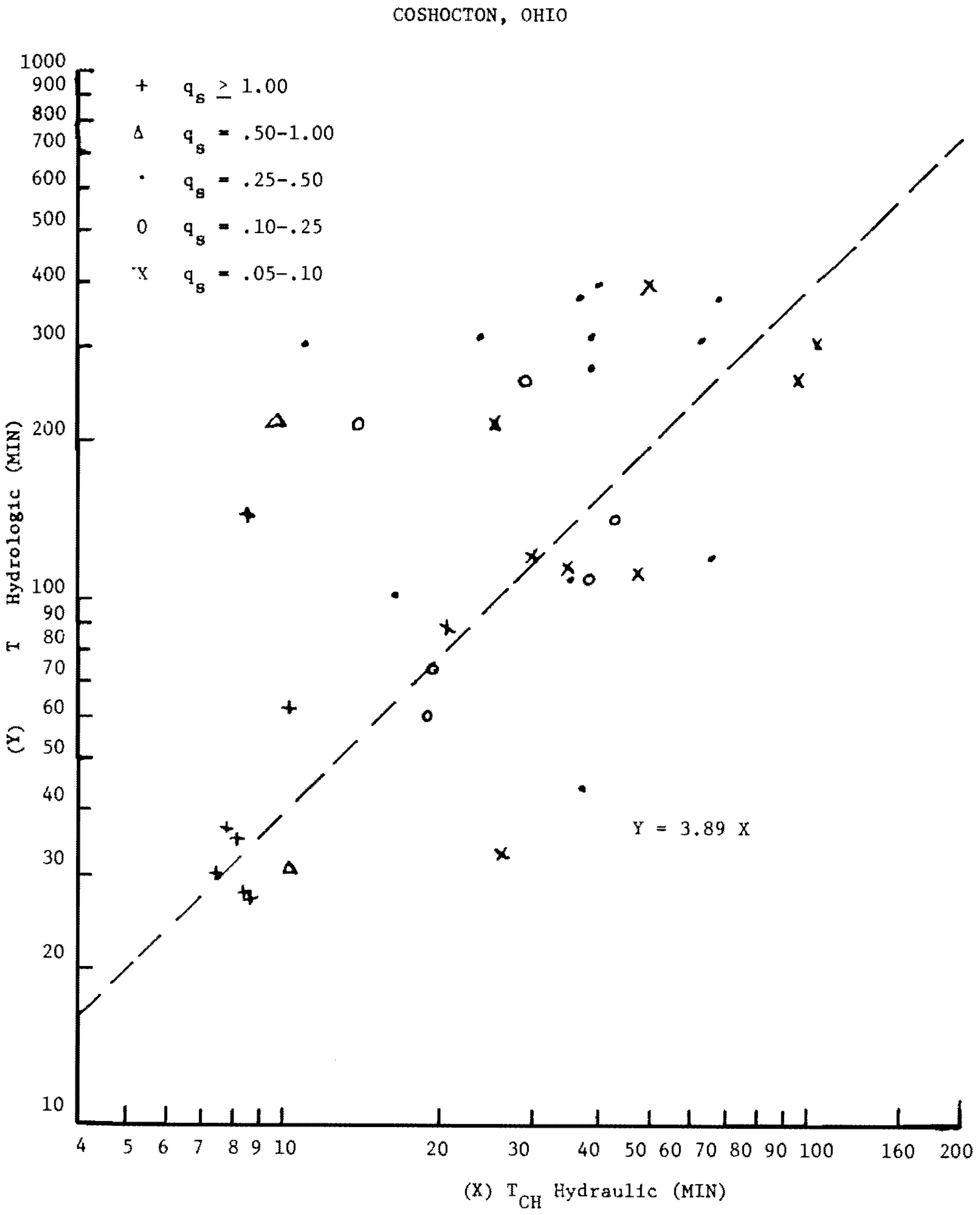


Figure 15. Log-lot Plot of Hydrologic T_{50} vs. Hydraulic T_{CH} for Coshocton, Ohio

trial. For the Coshocton data, the resulting equation is

$$T_{50} = 3.89\ T_{CH} \tag{34}$$

If the values of T_{50} determined earlier are correct, the T_{50}-values predicted by this equation have larger errors. Most likely, it is the high variability of the T_{50}-values previously determined (Chapter IV) that causes the large scatter of points in Figure 15, making it impossible to obtain a good fit with any type of equation relating T_{50} to T_{CH}.

In Figure 15, values for different ranges of q_s are indicated separately. This was done in view of the possibility that the variability evident here was partly due to the wide range in q_s value, and that the theory used in the various analyses described earlier might "break down" for small events, indicated by values of q_s. However, inspection of Figure 15 does not indicate clearly, if at all, that this is true. Values for the larger q_s-values also have considerable scatter. Likewise, those for the lower q_s-events are not consistently found below or above the line, as one would expect if the variability were explainable in this way.

The Oxford data were plotted in a similar manner (Figure 16). The results are somewhat better but considerable variability is evident here also. Using the same technique as for the Coshocton watersheds, the relationship obtained is

$$T_{50} = 2.21\ T_{CH} \tag{35}$$

A slightly non-linear relation, i.e., a line in Figure 16 with a little less slope might improve the fit somewhat. However, there is no assurance that such a relation would actually be better, considering the variability of the data and the probabilities associated with this fact. Thus, Eq. 35 was

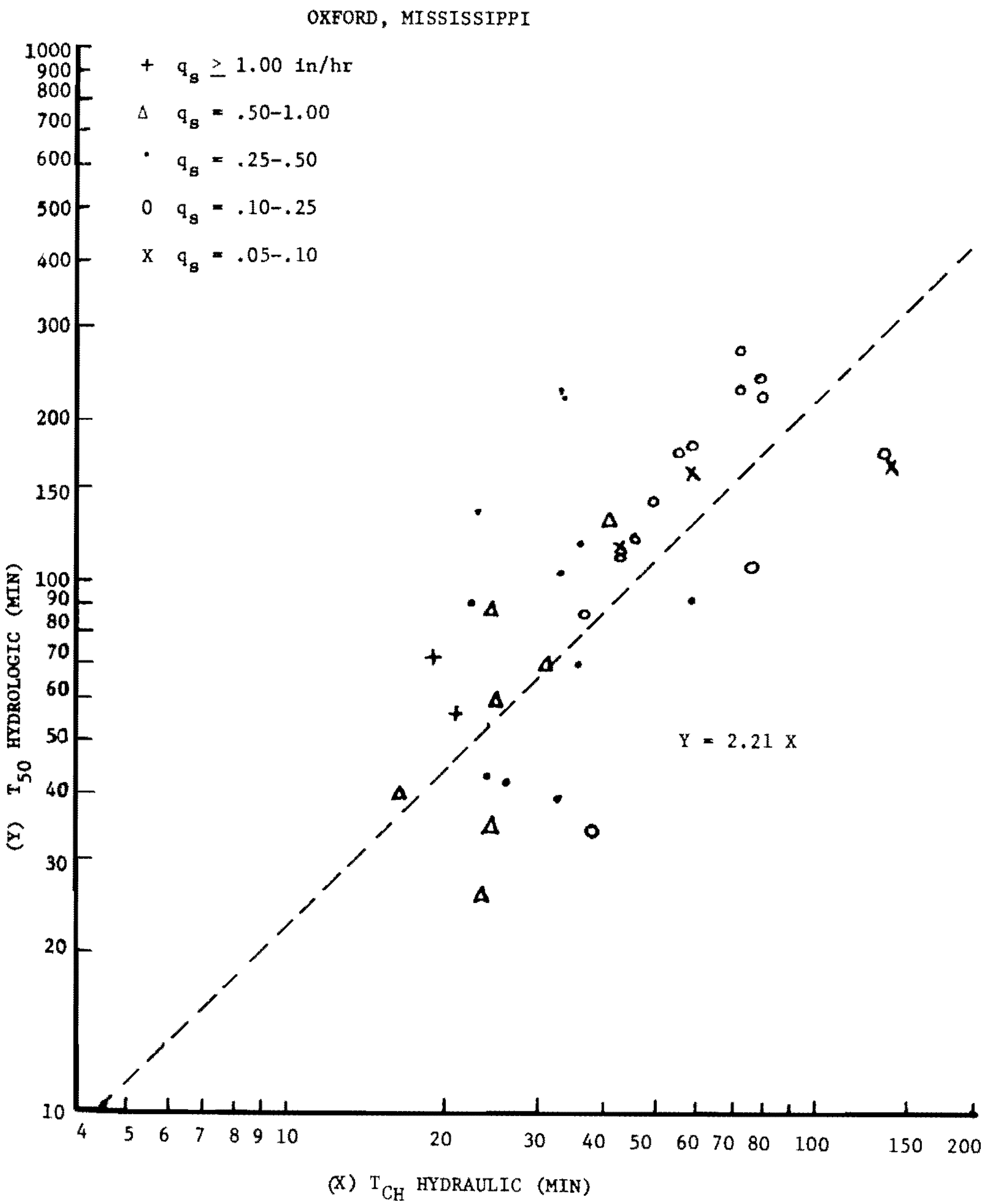


Figure 16. Log-log Plot of Hydrologic T_{50} vs. Hydraulic T_{CH} for Oxford, Miss.

adopted for further use for the Oxford region, at the same time recognizing that it is an approximate relationship.

For a comparision only, the constants for relationships similar to Eq. 34 and 35, using the angle and distance-slope methods were determined also. The three values of the constant obtained for Coshocton watershed data are:

Arc method: 3.89
Angle method: 3.73
Distance-slope: 3.68

For the Oxford data, by the two methods used, the constants were found to be:

Arc method: 2.21
Angle method: 1.89

Thus, there is some difference in the constants but probably not a significant one, considering the variability in the data.

Using the limited data for the Reisel, Texas, watershed, the mean ratio of T_{50} to T_{CH} by the arc method was found to be 1.07, and for the angle method, it was 0.94. These values are, of course, much lower than those at Coshocton and Oxford, and also close to the ideal value of 1.0. Due to the relatively few data points, however, one cannot draw any conclusions from these results. One can only wonder if they are meaningful. Further analysis and discussion are therefore limited to the Coshocton and Oxford regions.

An explanation or discussion of why the values of the coefficients in Eq. 34 and 35 are significantly different from each other and from 1.0 is needed. However, since this involves both the hydrologic and hydraulic processes and the corresponding methods used to represent them, it will be deferred until the discussion section, Chapter IX.

VIII. PEAK FLOW PREDICTIONS

The final part of the study and one of the major objectives was to test the peak flow equation (Eq. 6) with the parameters and relationships developed in this study and described in the preceding chapters. This equation represents the routing or channel phase effects only, i.e., it is intended to predict peak flow given the volume and time distribution of the excess rainfall. Other techniques must be used for predicting the excess rainfall.

The data for testing the overall method are chosen from the entire group of runoff events for the two main study areas, as given in Appendices A and B. A minimum of six runoff events was used for each watershed, as shown in Table 5. Were possible, only those events not used in the preceding analyses were used for testing, in order to make the testing as independent as possible. For a few of the watersheds, however, it was necessary to use several of the same events in order to get six events for this purpose. This was deemed preferable to using very small events or reducing the number below six for some watersheds. For the Coshocton data, only one of the 51 events was used "both ways". For Oxford, 7 out of 47 events were used for fitting as well as testing.

Peak Flow Coefficient

In addition to the relationships developed in earlier chapters, a relationship for the peak flow coefficient, C_p, in Eq. 6 is needed. In prior work with the equation, C_p has been related to the ratio D/T_{ve}. Since T_{50} is to be substitute for T_{ve}, a modified relationship for C_p as a function of D/T_{50} is required.

Table 5. Summary of Observed Runoff Events Used for Testing the Peak Flow Equation

Watershed Number	Number of Events	No. Used in Fitting	Range of q_s
Coshocton			
196	7	0	0.47-2.26
5	7	0	0.92-3.33
10	6	1	0.71-3.25
92	7	0	1.13-3.05
94	9	0	0.45-3.25
95	8	0	0.45-2.58
97	7	0	1.23-2.21
TOTAL	51	1	
Oxford			
4	6	1	0.26-0.92
5	6	3	0.71-2.20
10	6	2	0.31-0.66
12	7	0	0.25-0.86
28	6	1	0.16-0.94
32	8	0	0.41-0.80
35	8	0	0.20-1.82
TOTAL	47	7	

The new relationship for C_p comes from a concurrent study by James, Wei and Larson (report in preparation). The data were obtained by use of the mathematical model watershed developed by Wei (Wei and Larson, 1971) in collaboration with Golany (Golany and Larson, 1971). The model watershed had an area of 1.46 sq. mi. (935 acres). A number of simulated events with different supply rates and durations were routed to the watershed outlet by use of the method of characteristics to obtain watershed hydrographs. Knowing the peak discharges as well as other quantities in Eq. 6, values of C_p could be determined for each simulated event.

The resulting curve is given in Figure 17 along with the individual values on which it is based. We see that C_p approaches 1.0 at values of D/T_{50} of about 1.5 and above instead of at $D/T_{ve} \geq 1.0$ for the earlier relationship. We note also that $D = T_{50}$ produces a peak discharge well above 0.50 q_s, as it should. It might be added that the values obtained by Machmeier (1966) (see also Larson and Machmeier, 1968) for a 21.35 sq.mi. watershed agree fairly well when related to the new time parameter, T_{50}. The small differences may be due in part to differences in routing techniques used in the two studies. Since the watersheds used in this study are for the most part the same size range as the model watersheds of Wei and Machmeier, this relationship (Figure 17) was taken as being suitable for application to the Coshocton and Oxford watersheds.

Procedure

For an ungaged watershed the steps required to estimate peak discharge fall into two categories. First are those that need to be done only once, involving watershed characteristics only. They are as follows:

1. On the watershed map, draw an arc dividing the watershed area into an upper and a lower half. (Chapter VII)
2. In the field, beginning at the point where the arc intersects the principal channel and proceding to the outlet, choose five or more representative channel locations. At each one, measure the topwidth (W), maximum depth (D) and approximate channel distance. Estimate Manning's n for the channel.
3. By field or map measurements and calculations, determine the channel slope for each reach or station by use of the Manning equation.

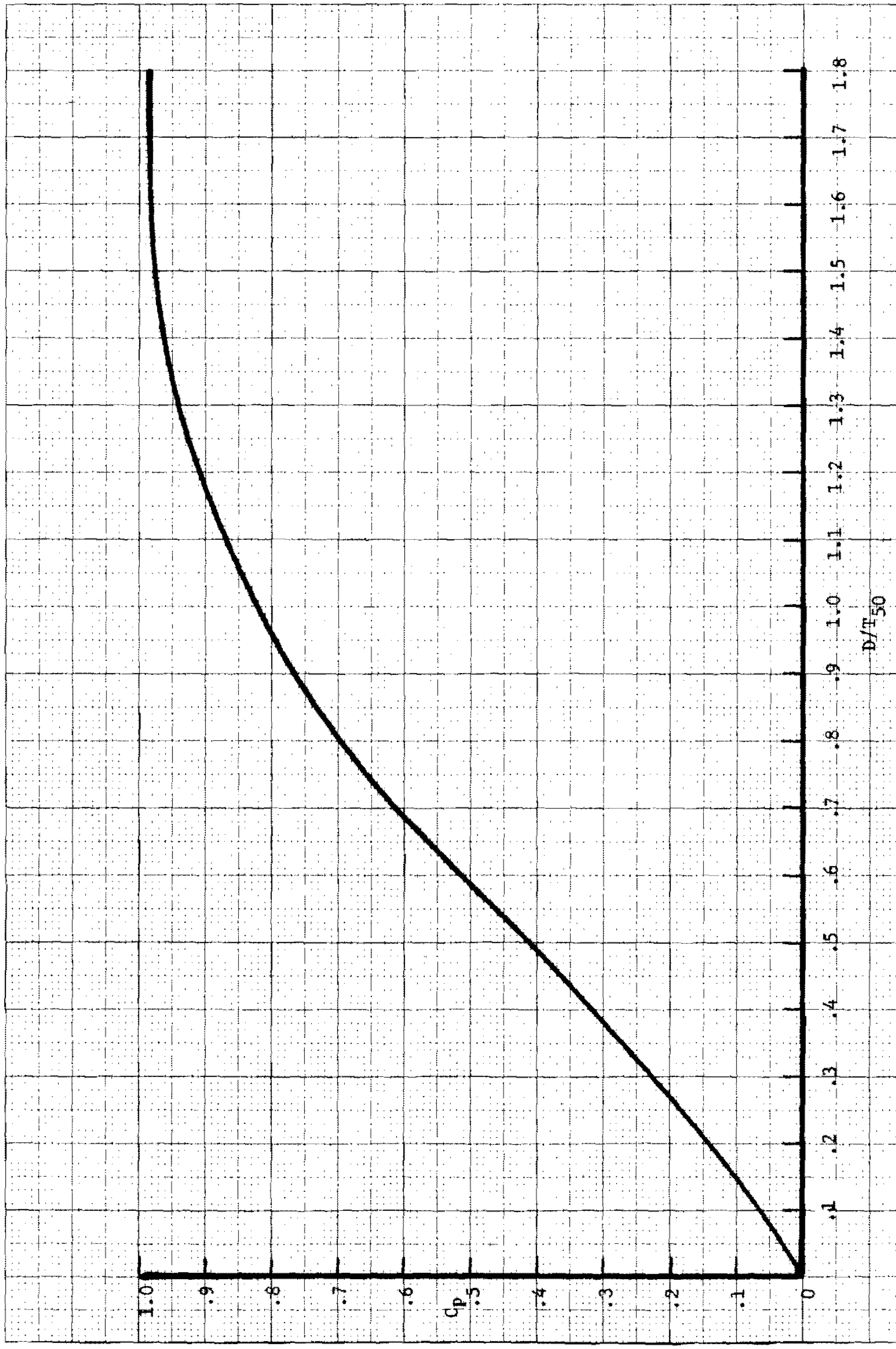


Figure 17. Plot of C_p vs. D/T_{50} (From James, Wei and Larson, 1972).

4. For each channel location using methods described in Chapter VI, calculate bankfull area, bankfull wetted perimeter, bankfull velocity and bankfull discharge.

All of these steps were used in the testing procedures.

The second part of the procedure includes the steps needed for each event, i.e., peak flow calculation. They are as follows:

1. Choose the design storm (or storms) to be used in determining the design runoff.
2. By methods not covered herein, determine for each storm the rainfall excess, including its duration and average rate.
3. If not a rectangular pattern approximate the actual time distribution of rainfall excess by a triangle (Chapter VI). Multiply the average rate $\overline{q}_s$ (one half the peak) by the appropriate value of C_t to obtain $\overline{q}_s'$, the supply rate for an equivalent rectangular pattern.
4. Calculate T_{ch} using the methods described in Chapter VII.
5. Calculate T_{50} for each event from T_{ch}, which requires a relationship such as Equation 34 or 35, applicable to a given area.
6. Calculate D/T_{50} for each event.
7. Determine C_p by use of Figure 17.
8. Calculate peak discharge by use of the peak flow equation (Eq. 6).
9. Add expected base flow, if any.

These steps were followed in testing the peak flow method, except for Steps 1 and 2. Step 2 is recognized as a difficult one which also has a great effect on the final result when estimating peak runoff from storm rainfall. The amount of rainfall excess and its time distribution are, in this case, determined by analysis of observed runoff hydrographs.

Results

The calculated peak flows for the watersheds and events selected earlier (Table 5) for the Coshocton and Oxford areas are given in Appendix D. The values of the necessary parameters and coefficients for each event are given also. For comparison, the observed peak discharges are given in the last column. A one-by-one comparison shows that the calculated (predicted) values are sometimes reasonably close, sometimes considerably high and sometimes too low.

The predicted and the observed peak flows for the two regions are compared graphically in Figures 18 and 19. This was done on log paper because of the individual values varied from well under 100 cfs. to several thousand cu. ft. per sec., nearly 10,000 cfs for the larger Oxford watersheds. It must be recognized that this tends to give a more favorable impression of the results, unless one takes time to take a careful look at the values.

The Coshocton data, in particular, have a rather broad scatter both above and below the line of equality. For the Oxford area, the discrepancies are noticeably less, but still substantial in many cases. The dashed lines are drawn to indicate calculated peak discharge of twice and one-half the observed peak discharges. We see that, for the Oxford watersheds, only a few of the events outside this band. For the Coshocton region, a larger number of events fall outside this band, a few of them far outside. Thus, it appears that predicted values by this method may be up to twice the true values, and occasionally more, or as little as one-half the true values, and even less at times.

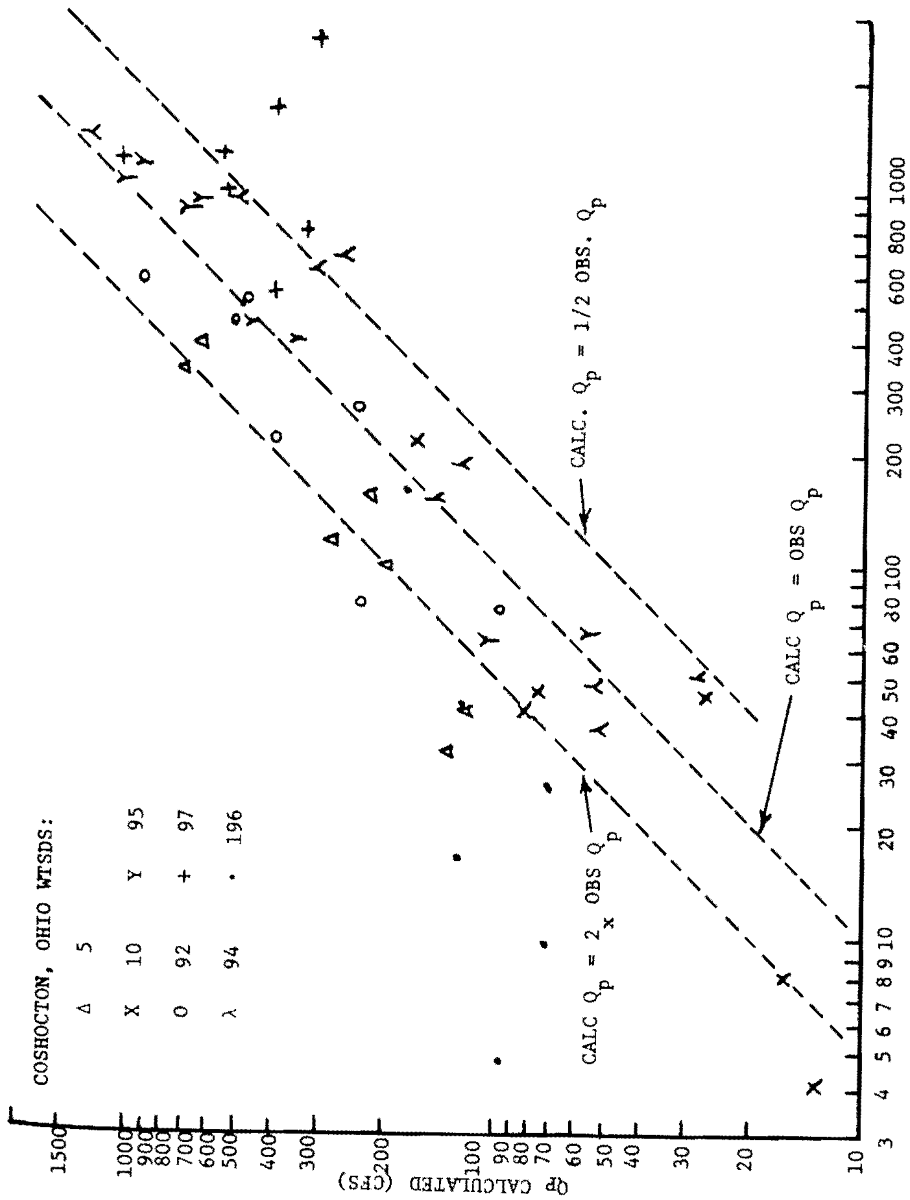


Figure 18. Plot of Calculated Peak Flow vs. Observed Peak Flow for Coshocton, Ohio Watersheds

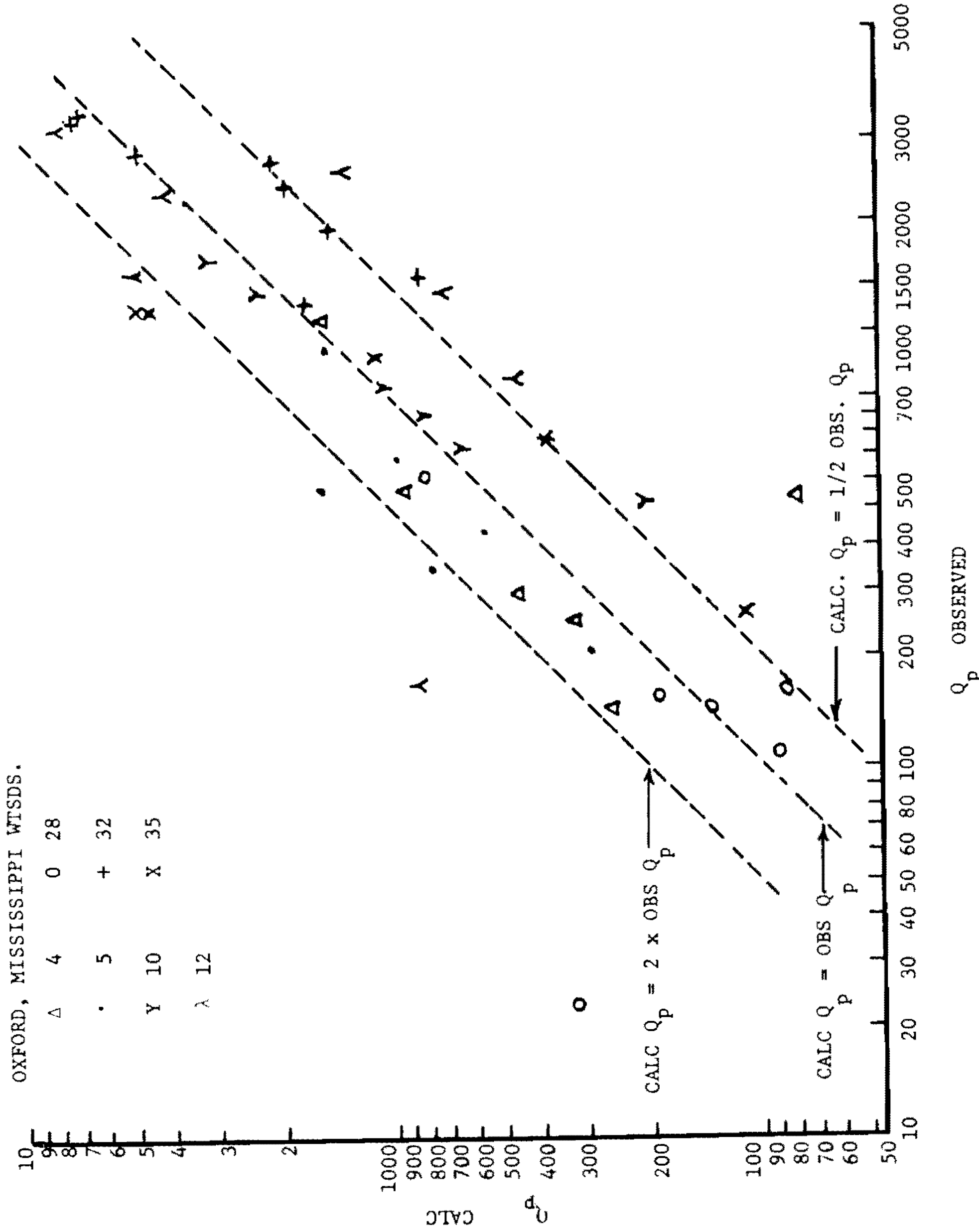


Figure 19. Plot of Calculated Peak Flow Vs. Observed Peak Flow for Oxford, Miss. Watersheds

IX. DISCUSSION

The study reported in the preceding pages is basically an hydraulic approach to the channel or routing phase of runoff. The prediction of runoff volume is considered to be a separate and equally important step, as stated earlier. The hydraulic approach to the routing phase is necessary for predicting peak flows of ungaged watersheds, where records cannot be used as the basis for flow predictions.

Now that all phases of the study have been presented, an overall discussion and evaluation of the results would be appropriate. Is the overall producedure promising or not? Are all parts of the procedure satisfactory, or are some parts in need of improvement? What are the possibilities for using such a procedure in practice? What are the limitations? Are there reasonable alternatives? Is further study along these lines likely to be productive?

The Time Parameter, T_{50}

First, we shall consider the hydrologic analysis (Chapter IV) in which the time parameter, T_{50}, was introduced and evaluated for various watersheds and various events. Of primary importance was the fact that the parameter T_{50} could be evaluated for many (about 50%) of the recorded runoff events. It is independent of duration for a given watershed, whereas most hydrologic time parameters vary with both the intensity and duration of rainfall excess. Time to virtual (97% of) equilibrium has this advantage also, but in general cannot be evaluated from observed runoff events. Thus, the new parameter appears to be preferable to other time parameters, if it can be evaluated with equal accuracy.

The process of evaluating T_{50} involves representing the rainfall excess time distribution by either a rectangular or triangular pattern. This is a somewhat subjective process, since some degree of judgment is used in choosing and fitting one of these patterns. Also, the process of dividing the rainfall hyetograph into two parts, retention and rainfall excess, is arbitrary, doubtful and, at best, approximate. Nevertheless the resulting errors do not necessarily affect the time parameter T_{50} a corresponding amount. It can be noted, also, that arbitrary and subjective procedures are required in the evaluation of other hydrologic time parameters, for example, lag time. In this respect, therefore, the parameter T_{50} is neither better than or worse than existing hydrologic parameters.

Once a rainfall excess pattern that has been selected for a given event, it is a relatively simple matter to determine the mean value for the event. In the case of a triangular pattern, an adjusted value of the rainfall excess rate is used, one that produces the same peak discharge as a uniform rate (rectangular pattern) of the same mean rate. This adjustment is made on a somewhat limited basis (Wei and Larson, 1972). However, this adjustment is not a major one, being less than 20% in all cases. Thus, if the adjustment factors eventually prove to be less than accurate, the effect on the overall result is expected to be relatively small.

A careful inspection of the values obtained for T_{50} (Appendix B) indicates that they have a great deal of variation between watersheds and between events. For a given watershed, one can find events that appear to be similar but which have quite different values of T_{50}. The question which

naturally follows is "Can the variation of T_{50} be explained by other factors, such as characteristics of the watersheds or of the runoff events?"

In Eqs. 9 and 10, T_{50} was related to watershed area for the Coshocton and Oxford Watersheds, respectively. The relative effect of area, as indicated by the exponent, was about the same for both regions. The exponent agrees well with a previous study; thus, there is good evidence that this time parameter and similar ones vary as drainage area to a power of about 0.30.

The absolute values of T_{50}, however, were significantly higher at Coshocton than for the Oxford watershed. What are the reasons for this? Significant amounts of interflow have been observed in the Coshocton area and are caused by the relatively shallow soils and low permeability of the subsoil. At Coshocton, interflow is probably greater during the winter months when soil moisture conditions remain at a high level most of the time. It can be noted (Appendix B) that a number of the runoff events occurred during the winter. In the Oxford area, the soils are deep and the subsoil is more permeable than the topsoil. These are conditions which do not encourage shallow subsurface flow. A significant percentage of interflow tends to delay the runoff, as studied earlier. Thus, to a large extent, the higher values of T_{50} for the Coshocton are probably caused by the interflow which is known to occur there. The fact that the Coshocton-values of T_{50} are more variable than those at Oxford is, most likely, also due to the occurence of interflow, which varies considerably with soil moisture conditions as well as with fixed conditions.

Eqs. 11 and 12 show that T_{50} decreases as the rainfall excess rate increases, as expected. The rate of change (exponent) for the Coshocton watersheds agrees well with previous studies, whereas the exponent for the

Oxford area is larger. No explanation is offered for this difference or for the larger exponent for the Oxford watershed. It would be readily explained if the Oxford exponent were the smaller of the two and if it agreed with the values obtained theoretically.

Multiple correlation of T_{50} to watershed area and average rainfall excess rate reduced the residual error somewhat. The effect of area remained about the same and the gap between the exponents for $\bar{q}$'s for the two areas was reduced. A considerable amount of variation remains unexplained (Figures 9 and 10). Other variables might be included to improve these relationships. Fortunately, however, neither the simple or multiple regression equations are needed or utilized in any way in predicting peak runoff by the methods proposed in this study.

Channel Characteristics

In general, bankfull channel area correlated well with the product of topwidth and bankfull depth (Eqs. 20 and 25). The correlation of wetted perimeter with bankful width (Eqs. 22 and 26) for these two regions is excellent. Using Eqs. 22 and 26 as prediction equations at Oxford, good results were obtained (Figure 12), especially for wetted perimeter.

Although only two regions have been studied, the possibility of developing a general procedure for predicting bankfull area from W x D and wetted perimeter from W appear to be quite good. It would of course require more study with data on channel characteristics from additional regions. Such a procedure is not only needed for the overall method proposed in this study but could also have a number of other uses in hydrologic and hydraulic studies of natural channels. The measurement of complete channel cross-sections is quite time consuming and therefore costly. Such a procedure, if proven to be adequate, could reduce this cost considerably.

Some comparisons of the channel characteristics of the Oxford watersheds to those of the Coshocton watershed would be of interest. This can be done by comparing the coefficients in Eqs. 25 and 27 to those of Eqs. 20 and 22, respectively, and also the r^2-values.

In general, the channel cross-section parameters were more consistent and therefore more predictable at Oxford, as indicated by the r^2-values. At Coshocton, the channels are rather small at some locations but not at others. At Oxford the channels appear larger (in relation to watershed size) and seem to have ample capacity for all but extreme flows.

It might be added that, in some cases, the channels of the Coshocton watershed, are not as clearly defined as at Oxford, i.e., the bankfull elevatins are not always easy to identify.

Comparing coefficients in the equations, we note first that the ratio A/WD for the Coshocton area is 0.530 and is 0.852 for the Oxford watersheds, Group A. This indicates that the Coshocton watersheds have channels that are roughly triangular in shape rather than trapezoidal, except for the flood plain sections. At Oxford, the channels are more nearly trapezoidal or semi-circular in shape. Comparing the ratio P/W for the two regions, the value for Oxford indicates deeper channels and/or wider channel bottoms as compared to the topwidth W. This is evident also from the values of the coefficient in the equation relating R to D.
From this, we see that channel characteristics vary from one region to another and that procedures for generalizing channel characteristics should be flexible to allow for such variations.

Hydraulic Approach to T_{50}

The time parameter corresponding to T_{50} but evaluated hydraulically has been designated T_{CH}. It was evaluated for various watersheds and various events by summing calculated travel times from the 50% isochrone to the watershed outlet. Ideally, T_{CH} would be found equal to T_{50}. In general, however, it was found to be considerably less than T_{50}. Some further discussion of the possible reasons for this difference the techniques utilized in calculating T_{CH} was given in the following paragraphs.

For the Coshocton and the Oxford watershed, respectively, values of T were, on the average, found to be 3.89 and 2.21 times the values of T_{CH}. Thus, the difference at Coshocton was much greater than at Oxford. Why this difference between the two locations? Again, the frequent occurence of interflow in the Coshocton area appears to be the most logical explanation, although there is no proof that it is the principal factor. Interflow does delay the runoff process, as compared to direct, surface flow tending to increase T_{50}. In the method utilized herein for calculating T_{CH}, no consideration is given to the possibility of interflow, i.e., only direct runoff is considered. Thus, it is not surprising, for the Coshocton area, at least, that T_{CH} values are considerably less than the values of T_{50}.

If interflow seldom occurs in the Oxford area or if it occurs in small quantities only, which is assumed to be the case, why are the Oxford values of T_{CH} less than half of the T_{50}-values, on the average? Only speculative answers can be given to this questions, but will be given nevertheless. One possibility is that ground water flow contributes to the runoff hydrograph, in much the same manner as interflow. The subsoils at Oxford are sandy, allowing infiltration water to percolate beyond the root zone with relative

ease. However, due to the sandy subsoils, it can move at a fairly rapid rate and could enter the channels in time to contribute to the hydrograph. Although this is a possible explanation, it does not appear to be a likely one in view of the more than 2 to 1 ratio of T_{50} to T_{CH}.

Another possible explanation is simply that the technique used for calculating T_{CH} does not adequately represent the process itself. In view of the complexities of the process and the approximations utilized in calulating T_{CH}, this is quite possible and perhaps probable. The process of overland flow, for example, is not represented or considered in the method. Overland flow does add a time delay of 15 to 30 minutes in most cases. On the other hand, overland flow from areas close to the channels enters the channels in a relatively short time. Thus, it is difficult to know whether an overland flow component should be added to the value of T_{CH} as calculated herein. Since the flow times are calculated along the main channel only, perhaps there would be some logic in adding such a component.

Those thoughts might also apply to the process of interflow, if occurring in significant amount. Interflow introduces a somewhat greater time delay, as studied earlier. Channel flow will certainly begin or begin to rise in response to overland flow. However, if the interflow contribution is high, it is possible that 50% of equilibrium flow is not reached without the interflow contribution. It might be noted too that, under certain conditions, interflow can constitute the major portion of the runoff hydrograph. So, here also, it is difficult to know whether an interflow time delay should be inclued for certain conditions and events.

In addition to the question of whether or not to add time delays for overland flow or interflow, or both, there remains the question of whether the channel flow process is represented adequately. Here there is less doubt, since the process was represented directly, even though some approximations were utilized. Of course, the estimation of the roughness coefficient is always difficult, and a possible source of significant error, probably up to 20%, perhaps even more. Then too, if flood plain flow occurs before T_{50} is reached, as it might if the channel is small, there is combined flow in the channel and flood plain. Existing methods for representing this type of flow are only approximate. Furthermore, no effort was made to represent the combined flow in this study, since it was assumed that the channels would be able to handle 50% of the equilibrium flow in most cases.

Three methods were used for locating the 50% isochrone. The arc method was chosen because of its simplicity, even though the other two methods gave values of T_{CH} somewhat closer to T_{50}. The benefit one would have by using one of the other methods does not appear to be worth the effort. It is possible that a considerably more refined method for doing this would give significant improvement, but this is doubtful also.

Overall Results

The overall results of the study are, of course, represented by the peak flow predictions. These predictions constitute tests of not only the methods developed in this study but also of the peak flow equation which was proposed as a result of earlier studies. The tests were nearly independent of the analysis portions of the study in terms of events, the events used for testing, the predictions were different from those used in evaluating coefficients. Of course, only the routing phase of runoff predictions was involved.

From Figures 18 and 19, we note first that the predicted peak discharges for both the Coshocton and Oxford watersheds fall more or less equally above and below the observed peak discharges. This suggests that neither the peak flow equation or the technique utilized with it are seriously deficient, i.e., the predictions are neither consistently high or consistently low. This provides some encouragement. It must be recognized, however, that the method does depend on runoff data within the region in order to determine the ratio of T_{50} to T_{CH}.

It is also evident from Figures 18 and 19 that for individual test events, there are substantial descrepancies between the predicted and observed peak discharges. These differences are probably caused by various inadequacies of the prediction method as compared to the actual processes. These probable inadequacies have been discussed earlier, in some detail, which will not be repeated here, since they apply equally to the overall method and to its individual parts.

It is appropriate to note once more that the actual processes occurring during runoff are quite complex and very difficult, perhaps impossible, to represent adequately by anything less than a continuous, complete mathematical watershed model. At present such models are dependent on observed runoff data for evaluating coefficients also. In other words, the objectives adopted for this study were rather ambitious ones, particularly if one expects to achieve good accuracy. For the reasons stated above, it appears that good accuracy on individual peak flow predictions is unlikely to be attained in the near future. Furthermore, perhaps this type of accuracy is not really necessary.

The more important type of accuracy, in the opinion of the authors, is that represented by the <u>average</u> result. In other words, the more important criterion is the consistency of the results. A more serious deficiency exists if the results are either consistently high or consistenly low by a significant amount. The results reported here satisfy this criterion for the watersheds of the region taken as a group. By inspection of Figures 18 and 19, they appear to satisfy this requirement for the individual watersheds also. The data for each watershed, however, are too limited to state this conclusively.

The overall method of this study cannot at present be applied to an ungaged watershed without some analysis of runoff events from other watersheds in the same region, which is necessary to determine the ratio of T_{50} to T_{CH} (the coefficient in Eqs. 34 and 35). The only alternative would be to make an estimate of this ratio on the basis of watershed characteristics and the values of this ratio obtained for the Coshocton and Oxford area. Obviously, the experience gained in this study is not sufficient to recommend such a procedure. However, further experience with the method could lead to a set of generalized values, each of which would be applicable to a region having certain general watershed characteristics.

X. CONCLUSIONS

The overall procedure utilized and/or developed in this study is outlined in Chapter VII, as it would be applied to an ungaged watershed. A more detailed description is given in earlier chapters. The specifics of procedure must be considered tentative, for reasons given and disucssed in the preceding chapter. Nevertheless, some important observations and conclusions can be made and are given below. Some of them, however, should be

considered tentative, in view of the complexity of the problem, the variability in the results, and the fact that only two different regions were studied intensively.

1. A new time parameter, time to 50% of equilibrium, T_{50}, was proposed. It can be evaluated hydrologically, i.e., from observed hydrographs in many but not all cases this is essential if it is to be used in peak flow predictions for other, ungaged watersheds.
2. T_{50} increases with watershed size, approximately as watershed area to the 1/3 power. It decreases as the mean rate of rainfall excess ($\bar{q}_s$), increases, varying as $\bar{q}_s$ to the minus 1/3 power (roughly). The residual variability is substantial, indicating that other factors also affect T_{50} significantly,
3. The channel characteristics, cross-sectional area and wetted perimeter can be estimated with reasonable accuracy from measurements of bankfull topwidth and depth. However, a number of complete channel cross-sections must be taken or be available in the region in order to evaluate the two coefficients needed, one for area and one for wetted perimeter. It appears likely that these coefficients can be generalized through further study, and also that relationships will have other applications in watershed engineering.
4. The three methods used to divided the watershed into an upper and lower half hydrologically gave only slightly different results and, therefore, the simplest method (arc) appears preferable.
5. The travel time approach to evaluating the time parameter yields values (designated T_{CH}) that are consistently and significantly lower than the true values (T_{50}). The possible reasons have been discussed in the preceding chapter. Thus, a coefficient applicable to the region is necessary to relate T_{50} to T_{CH}.

6. The peak flow predictions by the methods of this study were quite variable, as compared to the observed values, but on the average were about the right magnitude, i.e., neither consistently high or consistently low.
7. The combination of peak flow equation (developed earlier) the time parameter, T_{50}, and the relationship of C_p, the peak flow coefficient, to the ratio D/T_{50}, where D is the duration of rainfall excess, appears to provide a satisfactory but not highly accurate procedure for estimating peak runoff, given the volume of rainfall excess and its approximate time distribution.
8. It appears likely that further study of all parts of the procedure would lead to further improvements and improved accuracy in predicting peak flows for small watersheds.

REFERENCES

1. Barnes, H. H. Jr., Roughness Characteristics of Natural Channels, Geological Survey Water-Supply Paper 1849, U.S. Govt. Printing Office, Washington, 1967.
2. Brakensiek, D.L., Estimating Coefficients for Storage Flood Routing, Journal of Geophysical Research, Vol. 68, No. 24, December 1963.
3. Chow, V.T., Hydrologic Determination of Waterway Area for the Design of Drainage Structures in Illinois, University of Illinois Engineering Experiment Station Bulletin No. 463, 1962.
4. Golany, P. and Larson, C. L., Effects of Channel Characteristics on Time Parameters for Small Watershed Runoff Hydrographs, Water Resources Research Center Bulletin 31, University of Minnesota, Graduate School, May 1971.
5. Kirpich, P. A., Time of Concentration of Small Agricultural Watersheds, Civil Engineering, October, 1940.
6. Larson, C. L., "A Two Phase Approach to the Prediction of Peak Rate and Frequency of Runoff From Small Watersheds, Tech. Paper No. 53, Stanford University, Dept. of C.E., June 1965.
7. Larson C.L. and Machmeier, R.E., Peak Flow and Critical Duration for Small Watersheds, Trans. of ASAE, 1968.
8. Leopold, L.B. and Maddock, T. Jr., The Hydraulic Geometry of Stream Channels and Some Physiographic Implications, Geographical Survey Professional Paper 252, U.S. Govt. Printing Office, Washington, 1953.
9. Leopold, L.B. and Miller, J.P., Ephemeral Streams--Hydraulic Factors and Their RElation to the Drainage Net, Geological Survey Professional Paper 282-A, U.S. Govt. Printing Office, Washington, 1956.
10. Linsley, R.K., Kohler, M.A. and Paulhus, J.L.H., Hydrology for Engineers, McGraw-Hill, New York, 1958.
11. McGuinness, J. L., and Harrold, L. L., Seasonal and Areal Effects on Small Watershed Streamflow, Journal of Geophysical Research, Volume 67, No. 11, October 1962.
12. Machmeier, R.E., The Effect of Runoff Supply Rate and Duration on Runoff Time Parameters and Peak Flow Rates, Ph.D. Thesis, University of Minnesota, 1966.
13. Machmeier, R.E. and Larson, C.L., Runoff Hydrographs for Mathematical Watershed Model, Journal of the Hydraulics Division, November, 1968.
14. Mockus, V. et al, Hydrology Soil Conservation Service National Engineering Handbook, Section 4, 1966.

15. Morgali, J.R. Hydraulic Behavior of Small Drainage Basins, Tech Paper No. 30, Stanford University, Dept. of C.E., October 1963.

16. Reich, B.M. and Hiemstra, L.A.V., Purpose and Performance of Peak Predictions, Proceedings, the International Hydrology Symposium, Fort Collins, Colorado, September 1967.

17. Snyder, F.F., Synthetic Unit-Graphs, Transactions, American Geophysical Union, Vol. 19, Part 1, 1938.

18. Wei, T.C., and Larson, C.L., Effects of Areal and Time Distribution of Rainfall on Small Watershed Runoff Hydrographs, Water Resources Research Center, Bulletin 30, Univ. of Minn., Graduate School, March 1971.

19. Williams, G.R., Hydrology, Ch. IV., Engineering Hydraulics, John Wiley and Sons, 1950.

APPENDIX A

COSHOCTON OHIO WATERSHEDS

WTSD	Date of Event	Peak Flow (CFS)	Peak Flow (In/Hr)	Rain-fall (In)	Antecedent Soil Condition	Run-off (In)	Base Flow (In)	Base Flow at Q_p (In/Hr)	Rainfall Duration (Min)
5	6-18-40	39.1	.111	.91	Relatively Wet	.217	.026	.003	68
5	6-28,29-40	29.8	.0847	1.10	Dry	.244	.091	.004	300
5	9-23-45	113.0	.321	2.21	Dry	.459	0	0	400
5	6-12-57	152.0	.432	2.54	Dry	.398	0	0	170
5	6-28-57	383.6	1.09	2.96	Dry	1.54	0	0	467
5	1-21-59b	90.1	.249	1.17	Snow Cover	2.16	.140	.012	504
5	8-21-60	330.8	.940	3.91	Wet	1.25	.153	.020	344
5	4-25-61	86.2	.245	1.27	Wet	.515	.214	.003	150
5	3-9-64	40.8	.116	1.39	Wet	1.05	-	-	631
5	2-9-65	15.4	.0439	.450	Rel Dry	.044	.098	.006	300
5	2-24-65	14.0	.040	1.36	Normal	.209	.186	.007	764
5	3-23-65	4.2	.012	.530	Rel Dry	.043	.052	.005	413
5	4-27-66	20.8	.059	1.02	Wet	.167	.138	.012	357
5	3-5,6-67	9.1	0.26	.430	Wet	.101	.177	.011	431
5	5-11-67	8.7	.025	.800	Rel Wet	.077	.094	.006	330
10	6-18-40	45.0	.366	1.14	Rel Wet	.388	.012	.005	79
10	6-28,29-40b	4.3	.035	.75	Rel Dry	.180	.0575	.004	60
10	9-23-45a	211.6	1.72	1.61	Dry	.96	0	0	156
10	6-12-57	40.4	.329	2.54	Dry	.457	0	0	170
10	1-21-59b	29.0	.236	.920	Snow Cover	.653	-	-	192
10	8-21-60b	34.8	.283	.70	Wet	.169	.218	.04	35
10	4-25-61	103.6	.842	1.46	Wet	.63	.276	.04	105
10	2-24,25-65b	6.3	.0514	.30	Wet	.187	-	-	335
10	8-27-65	4.6	.0371	.81	Dry	.011	0	0	285
10	1-2-66	4.1	.033	.90	Wet	.105	0	0	388
10	2-13-66b	2.7	.022	.19	Rel Dry	.049	0	.011	240
10	4-30-66	3.4	.028	.67	Dry	.049	-	.006	221
10	3-5,6-67a	3.0	.024	.38	Wet	.174	-	-	172
10	3-20,21-67	3.4	.028	.70	Normal	.222	-	-	600
10	3-28-67a	2.2	.018	.79	Dry	.141	0	0	513
10	3-28-67b	0.7	.006	.16	Rel Wet	.010	0	0	73
10	5-11-67	3.2	.026	.83	Rel Dry	.087	-	-	488
10	7-2-67a	0.9	.007	.76	Dry	.0131	0	0	57

WTSD	Date of Event	Peak Flow (CFS)	Peak Flow (In/Hr)	Rain-fall (In)	Antecedent Soil Condition	Run-off (In)	Base Flow (In)	Base Flow at Q_p (In/Hr)	Rainfall Duration (Min)
92	6-18-40	75.1	.081	.91	Rel Wet	.303	.0240	.004	70
92	6-28,29-40	71.4	.077	1.10	Dry	.266	.085	.004	365
92	9-23-45	212.4	.229	2.21	Dry	.508	0	0	400
92	6-12-57	261.6	.282	2.54	Dry	.408	0	0	170
92	6-28-57b	558.5	.602	2.08	Rel Dry	1.183	0	0	250
92	1-21-59b	213.4	.230	1.15	Snow Cover	.708	.260	.011	240
92	8-21-60	447.1	.482	3.91	Wet	.66	.313	.06	342
92	4-25-61	395.2	.425	1.27	Wet	.58	.6342	.045	90
92	3-9-64	180.9	.195	1.39	Rel Wet	1.39	-	-	631
92	2-24-65	33.4	.036	1.36	Wet	.289	.208	.008	837
92	2-25-65	36.2	.039	.36	Wet	.087	.208	.013	200
92	4-24-66	50.1	.054	.670	Wet	.054	.093	.008	362
92	4-27-66	36.2	.039	1.02	Wet	.244	.167	.011	577
92	3-5-67	27.8	.030	.430	Wet	.149	.220	.011	311
92	5-11-67	23.2	.025	.800	Wet	.083	.122	.008	655
94	6-18-40	243.7	.159	.91	Wet	.341	.021	.006	70
94	6-28,29-40	131.8	.086	1.55	Dry	.310	.086	.004	260
94	9-23-45a	608.5	.397	1.41	Rel Dry	.62	0	0	90
94	6-12-57	669.8	.437	2.54	Dry	.692	0	0	170
94	6-28-67	1407.0	.918	2.96	Wet	1.94	0	0	247
94	1-21-59b	493.5	.322	1.17	Wet	.76	.283	.012	250
94	8-21-60	898.1	.586	3.91	Wet	1.02	.313	.040	374
94	4-25-61	720.4	.470	1.27	Wet	.74	.383	.030	150
94	3-9-64	378.6	.247	1.39	Wet	1.39	-	-	631
94	2-9-65	30.7	.020	.45	Wet	.060	.096	.007	300
94	2-11-65	29.1	.019	.55	Wet	.105	.111	.006	336
94	2-24-65	58.2	.038	1.36	Rel Wet	.243	.208	.008	765
94	2-25-65	53.6	.035	.36	Wet	.243	.208	.010	200
94	9-1-65	26.1	.017	1.63	Wet	.066	-	-	496
94	4-24-66	35.8	.024	.67	Wet	.118	.075	.006	362
94	4-27-66	151.7	.099	1.02	Wet	.274	.230	.011	577
94	4-30-66	42.9	.028	.63	Wet	.076	.046	.005	209
94	3-5-67	55.2	.036	.43	Wet	.163	.304	.011	551
94	5-11-67	41.4	.027	.80	Wet	.148	.112	.005	655
94	7-19-67	29.1	.019	1.33	Dry	.031	0	0	125
95	6-18-40	454.8	.164	.91	Wet	.369	.025	.003	70

WTSD	Date of Event	Peak Flow (CFS)	Peak Flow (In/Hr)	Rain-fall (In)	Antecedent Soil Condition	Run-off (In)	Base Flow (In)	Base Flow at Q_p (In/Hr)	Rainfall Duration (Min)
95	6-28-,29-40	415.9	.150	1.55	Wet	.363	.120	.004	150
95	9-23-45a	1003.8	.362	1.33	Dry	.72	0	0	90
95	6-12-57	959.4	.346	2.54	Normal	.79	0	0	170
95	1-21-59a	970.5	.350	1.19	Snow Cover	1.17	-	-	489
95	1-21-59b	854.1	.308	.92	Snow Cover	.92	-	-	190
95	1-21-59c	740.4	.267	.68	Snow Cover	.66	-	-	375
95	8-21,22-60	1103.6	.398	3.4	Dry	1.00	.119	.01	335
95	4-25-61	1167.4	.421	1.46	Wet	.757	.241	.03	105
95	3-9-64	582.3	.210	1.39	Wet	1.39	-	-	631
95	2-25-65	66.5	.024	.24	Wet	.057	.174	.024	200
95	4-24-66	49.9	.018	.67	Wet	.160	.064	.018	362
95	4-27-66	227.4	.082	1.02	Wet	.251	.294	.012	577
95	4-30-66	69.3	.025	.63	Normal	.074	.032	.004	209
95	3-5-67	77.6	.028	.59	Wet	.278	0	0	304
95	5-11-67	63.8	.023	.80	Wet	.103	.129	.005	330
95	7-19-67	30.5	.011	1.33	Dry	.025	0	0	125
97	6-18-40	521.9	.113	1.14	Wet	.434	.017	.003	100
97	6-28-40	928.8	.201	1.55	Wet	.614	.126	.005	154
97	6-4-41	1662.5	.360	1.13	Wet	.697	-	-	82
97	7-11-46	974.4	.211	2.78	Dry	.501	0	0	144
97	6-12-57	12007	.260	2.54	Dry	.690	0	0	180
97	1-21-59a	1625.6	.352	1.19	Snow Cover	1.19	-	-	485
97	1-21-59b	1630.2	.353	.92	Snow Cover	.92	-	-	192
97	8-21,22-60	1256.1	.272	3.40	Dry	.76	0	0	337
97	4-25-61	2383.0	.516	1.46	Wet	1.11	.26	.040	315
97	3-9-,10-64	859.0	.186	1.47	Wet	1.47	-	-	498
97	2-25-65b	2263	.049	.30	Wet	.139	-	-	215
97	10-21-65	60.0	.013	.700	Normal	.068	-	-	460
97	1-2-66	184.7	.040	.900	Dry	.141	-	-	390
97	1-6-66	184.7	.040	.700	Dry	.177	-	-	246
97	2-13-66	175.5	.038	.95	Dry	.199	-	-	603
97	2-28-66	50.8	.011	.62	Dry	.051	-	-	282
97	3-5,6-67a	115.5	.025	.80	Normal	.316	-	-	688
97	3-20-67	101.6	.022	.70	Normal	.21	-	-	720
97	3-28-67b	27.7	.006	.16	Wet	.010	-	-	80
97	5-11-67	110.8	.024	.83	Dry	.176	-	-	484

WTSD	Date of Event	Peak Flow (CFS)	Peak Flow (In/Hr)	Rain-fall (In)	Antecedent Soil Condition	Run-off (In)	Base Flow (In)	Base Flow at Q_p (In/Hr)	Rainfall Duration (Min)
196	6-18-40	35.1	.115	1.06	Dry	.176	.089	.005	40
196	6-28-40a	52.9	.173	.80	Wet	.270	.081	.007	242
196	9-23-45b	140.5	.460	.73	Wet	.240	-	-	160
196	6-16-46	580.5	1.90	3.38	Wet	1.44	-	-	196
196	8-16-47	179.0	.586	1.24	Rela. Wet	.249	0	0	38
196	8-1-50	540.8	1.77	4.42	Rel Wet	1.78	0	0	238
196	6-12-57	1136.5	3.72	3.25	Rel Wet	1.47	-	-	134
196	6-28-57	424.7	1.39	2.80	Dry	1.63	0	0	427
196	1-21-59a	151.5	.496	1.54	Snow Cover	1.61	.20	.008	240
196	1-21-59b	66.9	.219	.570	Snow Cover	.66	.20	.008	308
196	8-21-60	41.8	.137	1.81	Wet	.098	.064	.0003	306
196	4-25-61	320.8	1.05	1.83	Wet	1.26	.400	.027	101
196	5-13-64	156.1	.511	1.07	Rel Wet	.336	0	0	219
196	2-25-65b	13.1	.043	.270	Wet	.136	0	0	298
196	3-23-65	7.6	.025	.72	Normal	.171	-	-	689
196	10-22-65	5.5	.018	.23	Wet	.111	0	0	120
196	3-5-67a	15.9	.052	.350	Wet	.307	-	-	312
196	3-5,6-67	22.3	.073	.400	Wet	.317	-	-	360
196	3-12-67	11.0	.036	.46	Normal	.193	0	0	18
196	3-17-67	9.8	.032	.60	Rel Dry	.203	0	0	94
196	3-20-67	11.6	.038	.61	Wet	.531	0	0	843
196	3-28-67b	6.1	.020	.26	Wet	.050	-	-	295
196	4-13-67	4.6	.015	.72	Dry	.181	0	0	420
196	5-6-67	16.5	.054	.61	Wet	.263	-	-	540
196	5-11-67	19.2	.063	.78	Rel Dry	.352	0	0	308
196	5-29-67	8.2	.027	.61	Dry	.088	0	0	135
196	7-28-67	10.4	.034	.86	Wet	.140	-	-	483
196	10-21-67	6.7	.022	1.27	Rel Dry	.112	0	0	720

OXFORD MISSISSIPPI WATERSHEDS

WTSD	Date of Event	Peak Flow (CFS)	Peak Flow (In/Hr)	Rain-fall (In)	Antecedent Soil Condition	Run-off (In)	Base Flow (In)	Base Flow at Q_p (In/Hr)	Rainfall Duration (Min)
4	5-22-57	494.1	.245	1.35	Dry	.424	0	0	140
4	4-3-58	292.4	.145	.550	Wet	.305	-	.01	50
4	9-9-59	586.8	.291	2.15	Rel Dry	.294	0	.01	75
4	1-17-60	133.1	.066	.980	Dry	.156	0	0	225
4	8-31-61	94.8	.047	1.09	Dry	.054	0	0	105
4	9-4-62	290.4	.144	2.04	Dry	.187	0	0	132
4	8-29-63	568.7	.282	3.15	Dry	.459	0	0	165
4	3-4-64	1706.1	.846	2.38	Normal	1.59	0	0	285
5	1-22-57	184.8	.151	.560	Wet	.395	0	0	135

WTSD	Date of Event	Peak Flow (CFS)	Peak Flow (In/Hr)	Rain-fall (In)	Antecedent Soil Condition	Run-off (In)	Base Flow (In)	Base Flow at Q_p (In/Hr)	Rainfall Duration (Min)
5	12-6-57	344.0	.281	2.11	Dry	.669	0	0	1020
5	4-3-58	375.8	.307	.690	Wet	.501	.120	.018	50
5	6-10-59	743.0	.609	1.30	Wet	.713	.034	-	50
5	6-11-59	610.8	.499	1.31	Wet	.846	.044	-	145
5	1-17-60	155.5	.127	.930	Normal	.410	0	0	345
5	8-31-61	415.0	.339	1.81	Dry	.419	0	0	180
5	9-4-62	222.8	.182	1.95	Dry	.250	0	0	154
5	8-29-63	564.3	.461	3.26	Normal	1.05	0	0	165
5	3-4-64	1456.7	1.19	3.00	Rel Wet	2.13	0	0	135
5	3-1-65	252.2	.206	1.18	Normal	.749	0	0	465
5	3-3-66a	189.1	.166	1.13	Normal	.512	0	0	180
5	5-24-66	115.1	.094	1.57	Dry	.260	0	0	330
5	5-31-67	465.2	.380	1.52	Dry	.461	0	0	[illegible]
10	5-22-59	524.4	.094	1.25	Rel Dry	.091	0	0	135
10	1-17-60	474.0	.085	1.03	Wet	.264	0	0	225
10	8-31-61	2414.4	.433	2.40	Normal	.668	0	0	120
10	9-4-62	730.5	.131	1.74	Rel Dry	.186	0	0	225
10	8-29-63	1957.2	.351	2.85	Dry	.653	0	0	150
10	3-4-64a	1070.6	.192	.80	Rel Wet	.248	.099	.014	165
10	3-1-65a	239.8	.043	.63	Wet	.180	-	-	195
10	12-28-66	652.4	.117	1.47	Rel Dry	.473	0	0	420
10	5-31-67	908.9	.163	1.33	Dry	.277	0	0	120
12	5-22-57	5701.5	.248	2.84	Dry	.691	0	0	240
12	11-13-57a	4184.2	.182	2.15	Dry	.790	0	0	795
12	11-13-57	758.7	.033	.181	Wet	.181	-	-	130
12	4-3-58	1931.2	.084	.690	Rel Wet	.121	-	-	50
12	3-2-60	2482.9	.108	1.65	Rel Wet	.560	0	0	1080
12	8-31-61	1241.5	.054	1.68	Normal	.126	0	0	180
12	9-4-62	1563.3	.068	1.81	Dry	.125	0	0	270
12	8-29-63	3632.4	.158	3.21	Dry	.432	0	0	345
12	3-4-64	5402.6	.235	2.01	Rel Wet	.725	.154	.001	300
12	3-1-65	1586.3	.069	.97	Normal	.397	0	0	480
12	5-24-66	2482.9	.108	1.74	Normal	.454	0	0	345
12	5-31-67	1701.3	.074	1.32	Wet	.140	.017	.013	120
28	6-30-57	144.8	.133	.83	Dry	.105	0	0	75
28	7-22-58	263.5	.242	1.39	Rel Wet	.230	0	0	85

WTSD	Date of Event	Peak Flow (CFS)	Peak Flow (In/Hr)	Rain-fall (In)	Antecedent Soil Condition	Run-off (In)	Base Flow (In)	Base Flow at Q_p (In/Hr)	Rainfall Duration (Min)
28	9-9-59	610.9	.561	2.76	Normal	.656	0	0	120
28	1-17-60	51.2	.047	.950	Wet	.083	0	0	270
28	11-15-61	159.0	.146	1.56	Wet	.134	0	0	165
28	9-4-62	108.9	.100	2.12	Dry	.115	0	0	150
28	8-29-63	153.5	.141	3.21	Dry	.229	0	0	150
28	3-4-64	302.7	.278	2.17	Rel Wet	.466	0	0	345
28	3-1-65	34.8	.032	1.12	Normal	.124	–	–	465
28	5-24-66	22.9	.021	1.31	Normal	.039	0	0	240
28	5-31-67	132.9	.122	1.23	Dry	.120	0	0	120
32	11-18-57	6131.5	.283	1.27	Wet	.920	–	–	345
32	4-14-58	1776.6	.082	1.34	Dry	.380	0	0	435
32	5-22-59	1928.3	.089	1.11	Dry	.175	0	0	150
32	3-2-60	4636.5	.214	1.81	Wet	1.15	0	0	1080
32	8-31-61	4658.2	.215	1.91	Dry	.420	0	0	135
32	9-4-62	2274.9	.105	1.70	Dry	.195	0	0	240
32	8-29-63	4116.5	.190	2.28	Dry	.431	0	0	165
32	3-4-64	4983.2	.230	1.26	Rel Wet	.694	0	.013	270
32	3-1-65	1841.6	.085	.63	Normal	.33	0	0	405
32	5-24-66	6348.1	.293	2.11	Normal	.912	0	0	345
35	11-18-57	1773.8	.233	1.18	Wet	.803	0	0	190
32	5-31-67	3054.9	.141	1.24	Dry	.303	0	0	135
35	4-14-58	867.9	.114	1.30	Dry	.483	0	0	390
35	5-22-59	1301.8	.171	1.81	Wet	.304	–	–	125
35	3-2-60	1773.8	.233	1.87	Wet	1.20	0	0	1080
35	8-31-61	258.8	.034	1.29	Normal	.060	0	0	135
35	9-4-62	837.4	.110	1.63	Dry	.156	0	0	195
35	8-29-63	228.4	.030	1.52	Dry	.070	0	0	165
35	3-4-64	3007.1	.395	1.52	Rel Wet	1.08	.209	.030	270
35	3-1-65	609.0	.080	.620	Normal	.439			240
35	5-24-66	1515.0	.199	1.90	Normal	.509	0	0	270
35	5-31-67	1591.1	.209	1.43	Dry	.477	0	0	120

A P P E N D I X B

COSHOCTON, OHIO

WTSD	Date of Event	Rainfall Excess Peak (Hr/In)	Pattern Used R,T_1,T_2	Max qs (In/Hr)	Mean qs (In/Hr)	OBS D_1 (Min)	CALC D_2 (Min)	Mean $\bar{q}$'s (In/Hr)	T50 (Min)
5	6-18-40	2.23	T1	2.23	1.171	8	11	1.377	
5	6-28,29-40	1.60	T1	1.60	.400	86	37	.4684	
5	9-23-45	1.90	T1	1.90	.835	126	33	.985	
5	6-12-57	3.17	T1	3.17	1.26	17	19	1.32	
5	6-28-57	3.43	T1	3.43	1.58	248	54	1.87	
5	1-21-59b	.420	T1	.420	.210	504	257	.249	303
5	8-21-60	5.94	T1	5.94	2.97	140	22	3.33	
5	4-25-61	1.62	T1	1.62	.811	60	22	.916	
5	3-9-64	.56	T1	.560	.280	448	224	.327	
5	2-9-65	.182	T1	.182	.091	43	29	.108	
5	2-24-65	.076	T1	.076	.038	366	329	.045	291
5	3-23-65	.095	T1	.095	.048	65	54	.056	
5	4-27-66	.349	T2	.170	.0835	120	–	.098	209
5	3-5,6-67	.104	R	.075	.076	80	–	.070	
5	5-11-67	.164	T1	.164	.082	63	58	.096	
10	6-18-40	3.30	T1	3.30	1.65	17	14	1.932	
10	6-28-,29-40b	1.40	T1	1.40	.699	11	5	1.776	
10	9-23-45a	3.10	T1	3.10	1.55	32	30	1.85	28
10	6-12-57	3.39	T1	3.39	1.70	17	16	1.80	
10	1-21-59b	.641	T1	.641	.321	192	173	.257	103
10	8-21-60b	1.27	T1	1.27	.634	15	16	.706	
10	4-25-61	1.87	T1	1.87	.935	63	40	1.11	62
10	8-27-65	.0684	R	.068	.068	10	–	.068	24
10	2-24,25-65b	.199	T1	.199	.099	185	113	.104	
10	1-2-66	.081	T1	.081	.0405	98	155	.047	122
10	2-13-66b	.036	T1	1036	.018	157	70	.021	90
10	4-30-66	3.25	R	3.25	3.25	2	–	3.250	
10	3-5,6-67a	.300	T1	.300	.150	172	70	.165	
10	3-20,21-67	.08	T1	.08	.04	462	333	.042	288
10	3-28-67a	.06	T1	.06	.03	170	282	.031	195
10	3-28-67b	.01	R	.01	.01	60	–	.010	72
10	5-11-67	.10	T1	.10	.05	220	107	.058	
10	7-2-67a	.04	R	.040	.040	6	–	.640	

WTSD	Date of Event	Rainfall Excess Peak (Hr/In)	Pattern Used R,T1,T2	Max qs (In/Hr)	Mean qs (In/Hr)	OBS D_1 (Min)	CALC D_2 (Min)	Mean q's (In/Hr)	T_{50} (Min)
92	9-23-45	1.89	T1	1.89	.944	95	32	1.12	
92	6-18-40	2.42	T1	2.42	1.21	10	15	1.42	
92	6-28,29-40	2.00	T1	2.00	1.00	80	16	1.18	
92	6-12-57	3.22	T1	3.22	1.61	19	15	1.71	
92	6-28-57b	3.34	T1	3.34	.949	340	75	1.13	
92	1-21-59b	.418	T1	.418	.209	500	203	.248	318
92	8-21-60	5.29	T1	5.29	2.65	141	15	3.05	
92	4-25-61	1.98	T1	1.98	.990	84	35	1.16	
92	3-9-64	.600	T1	.600	.300	448	278	.351	
92	2-24-65	.07	T1	.07	.042	455	346	.042	350
92	2-25-65	.12	T1	.12	.06	70	87	.072	115
92	4-24-66	.643	T1	.643	.322	28	10	.382	
92	4-27-66	.226	T2	.135	.113	130	130	.132	263
92	3-5-67	.135	T1	.135	.068	106	132	.080	
92	5-11-67	.170	T1	.170	.085	63	59	.099	
94	6-18-40	2.72	T1	2.72	1.36	10	15	1.595	
94	6-28-,29-40	3.30	T1	3.30	1.65	90	11	1.947	
94	9-23-45	2.29	T1	2.29	1.15	67	32	1.36	
94	6-12-57	4.20	T1	4.20	2.10	17	20	2.19	
94	6-28-57	3.40	T1	3.40	1.70	185	68	2.01	
94	1-21-59b	.45	T1	.45	.23	188	203	.266	44
94	8-21-60	5.82	T1	5.82	2.91	135	21	3.27	
94	4-25-61	2.12	T1	2.12	1.06	117	35	.118	
94	3-9-64	.600	T1	.300	.300	448	278	.351	377
94	2-9-65	.208	T1	.208	.104	95	35	.124	
94	2-11-65	.810	T1	.810	.405	30	16	.481	
94	2-24-65	.080	T1	.080	.040	545	364	.047	396
94	2-25-65	.119	T2	.066	.059	88	0	.070	113
94	9-1-65	.456	T1	.456	.228	27	18	.267	
94	4-24-66	.813	T1	.813	.407	28	17	.483	
94	4-27-66	.420	T2	.260	.130	126	-	.147	121
94	4-30-66	.90	R	.90	.45	10	-	.450	
94	3-5-67	.142	R	.142	.122	80	-	.122	
94	5-11-67	.220	T1	.220	.110	136	80	.128	
94	7-19-67	.390	R	.390	.185	10	-	.185	
95	6-18-40	2.92	T1	2.92	2.13	28	15	2.50	
95	6-28-,29-40	3.20	T1	3.20	1.60	90	14	1.89	

WTSD	Date of Event	Rainfall Excess Peak (Hr/In)	Pattern Used R,T1,T2	Max qs (In/Hr)	Mean qs (In/Hr)	OBS D_1 (Min)	CAIC D_2 (Min)	Mean q's (In/Hr)	T_{50} (Min)
196	8-16-47	2.29	T2	2.29	1.15	13	-	1.16	27
196	8-1-50	2.87	T1	2.87	1.43	80	74	1.68	35
196	6-12-57	6.54	T1	6.54	3.27	33	27	3.88	30
196	6-28-57	2.63	T1	2.63	1.31	187	74	1.55	145
196	1-21-59a	1.36	T1	1.36	.682	240	144	.795	291
196	1-21-59b	.44	T1	.44	.22	308	180	.242	7
196	8-21-60	.12	R	.12	.12	5	-	.120	60
196	4-25-61	3.84	T1	3.84	1.92	101	39	2.26	
196	5-13-64	2.14	T1	2.14	1.07	15	19	1.25	
196	2-25-65b	.097	T1	.097	.048	250	168	.051	277
196	3-23-65	.108	T1	.108	.054	372	190	.064	
196	10-22-65	.360	T1	.360	.180	74	37	.213	
196	3-5-67a	.710	T1	.710	.355	145	52	.422	
196	3-6-67	.196	T1	.196	.098	60	194	.115	72
196	3-12-67	2.25	T1	2.25	1.13	7	10	1.34	
196	3-17-67	.90	T1	.90	.45	24	27	.472	
196	3-20-67	.087	T1	.087	.043	843	736	.049	510
196	3-28-67b	.040	T1	.040	.020	210	149	.024	135
196	4-13-67	.250	T1	.250	.125	66	87	.145	
196	5-6-67	1.74	T1	1.74	.870	93	18	1.03	
196	5-11-67	.500	T1	.500	.250	213	84	.296	
196	5-29-67	.310	T1	.310	.155	40	34	.183	
196	7-28-67	.210	T1	.210	.105	347	80	.123	
196	10-21-67	.300	T1	.300	.150	525	45	.176	
				Oxford Mississippi Watersheds					
4	5-22-57	.781	T2	.679	.339	90	-	.404	104
4	4-3-58	1.39	T1	1.39	.695	50	28	.664	
4	9-9-59	.680	T1	.680	.340	45	52	.403	54
4	1-17-60	.130	T1	.130	.065	90	144	.077	160
4	8-31-61	.161	R	.161	.161	20	-	.161	
4	9-4-62	.260	T1	.260	.130	80	86	.151	119
4	8-29-63	1.20	T1	1.20	.599	45	46	.713	
4	3-4-64	1.57	T1	1.57	.780	120	123	.902	88
5	1-22-57	.347	T1	.347	.173	420	164	.275	228
5	12-6-57	.490	T1	.490	.245	120	107	1.42	72
5	4-3-58	2.10	T1	2.10	1.05	45	29	1.24	
5	6-10-59	1.76	T1	1.76	.880	40	48	1.03	57

WTSD	Date of Event	Rainfall Excess Peak (Hr/In)	Pattern Used R,T1,T2	Max qs (In/Hr)	Mean qs (In/Hr)	OBS D_1 (Min)	CAlC D_2 (Min)	Mean q's (In/Hr)	T_{50} (Min)
95	9-23-45a	2.55	T1	2.55	1.27	32	29	1.52	
95	6-12-57	4.37	T1	4.37	2.19	50	22	2.28	
95	1-21-59a	.730	T1	.730	.370	489	192	.431	315
95	1-21-59b	.530	T1	.530	.270	190	209	.312	112
95	1-21-59c	.600	T1	.600	.300	375	132	.354	275
95	8-21,22-60	4.43	T1	4.43	2.22	14	27	2.58	
95	4-25-61	1.90	T1	1.90	1.44	63	31	1.71	
95	3-9-64	.600	T1	.600	.300	448	278	.351	397
95	2-25-65	.109	T1	.109	.054	70	63	.065	
95	4-24-66	.833	T1	.833	.417	28	18	.495	
95	4-27-66	.404	T2	.404	.202	126	0	.135	144
95	4-30-66	.440	R	.440	.447	10	-	.447	
95	3-5-67	.345	T2	.345	.173	80	-	.073	
95	5-11-67	.193	T1	.193	.097	130	64	.112	
95	7-19-67	.150	R	.150	.156	10	-	.156	
97	6-18-40	3.80	T1	3.80	1.90	25	14	2.21	
97	6-28-40	3.60	T1	3.60	1.80	88	20	2.13	
97	6-4-41	2.11	T1	2.11	1.06	52	40	1.22	
97	7-11-46	2.88	T1	2.88	1.44	43	21	1.55	
97	6-12-57	4.20	T1	4.20	2.10	19	20	2.19	
97	1-21-59a	.730	T1	.730	.365	485	196	.384	305
97	1-21-59b	.530	T1	.530	.265	192	208	.310	120
97	8-21,22-60	3.43	T1	3.43	1.71	155	27	1.997	
97	4-25-61	2.20	T1	2.20	1.10	183	61	1.31	
97	3-9,10-64	.450	T1	.450	.225	498	392	.267	372
97	2-25-65b	.160	T1	.160	.080	185	105	.084	263
97	10-21-65	.090	T1	.090	.045	107	90	.053	
97	1-2-66	.100	T1	.100	.050	98	169	.058	172
97	1-6-66	.130	T1	.130	.065	182	164	.075	306
97	2-13-66	.170	T1	.170	.085	535	140	.101	
97	2-28-66	.070	T1	.070	.035	63	88	.042	
97	3-5,6-67a	.360	T1	.360	.180	466	105	.200	
97	3-20-67	.090	T1	.090	.045	468	281	.048	
97	3-28-67b	.010	R	.010	.010	60	-	.010	165
97	5-11-67	.150	T1	.150	.075	313	141	.088	
196	6-18-40	1.50	T1	1.50	.750	14	14	.892	
196	6-28-40a	3.50	T1	3.50	1.75	74	9	2.037	
196	9-23-45b	.790	R	.790	.720	20	-	.720	31
196	6-16-46	3.92	T1	3.92	1.96	57	44	2.29	37

WTSD	Date of Event	Rainfall Excess Peak (Hr/In)	Pattern Used R,T1,T2	Max qs (In/Hr)	Mean qs (In/Hr)	OBS D_1 (Min)	CAIC D_2 (Min)	Mean q's (In/Hr)	T_{50} (Min)
5	6-11-59	3.73	T1	3.73	1.87	25	27	2.20	
5	1-17-60	.225	T1	.225	.113	150	218	.132	112
5	8-31-61	.58	R	.58	.58	40	-	.628	60
5	9-4-62	1.68	T1	1.68	.840	35	18	.979	
5	8-29-63	1.20	T2	1.20	.601	105	0	.648	35
5	3-4-64	2.40	T1	2.40	1.20	135	137	.202	87
5	3-1-65	.455	T1	.455	.228	270	198	.269	225
5	3-3-66a	.512	T2	.512	.256	120	120	.262	120
5	5-24-66	.520	T2	.347	.174	165	-	.178	34
5	5-31-67	1.38	T1	1.38	.691	30	40	.714	50
10	5-22-59	.360	R	.360	.360	15	-	.360	
10	1-17-60	.166	T1	.166	.083	210	190	.098	180
10	8-31-61	1.14	T1	1.14	.570	60	70	.662	70
10	9-4-62	.410	T1	.410	.205	45	54	.244	
10	8-29-63	.720	T1	.720	.360	120	109	.421	70
10	3-4-64a	.744	T1	.744	.372	45	40	.428	
10	3-1-65a	.203	T1	.203	.102	180	106	.117	175
10	12-28-66	.320	T1	.320	.160	275	177	.167	144
10	5-31-67	.534	T1	.534	.267	30	51	.312	
12	5-22-57	.590	T1	.590	.295	65	141	.347	91
12	11-13-57a	.330	T1	.330	.165	225	287	.195	233
12	11-13-57	.830	T1	.830	.415	25	26	.493	
12	4-3-58	1.455	T1	1.445	.723	37	10	.855	
12	3-2-60	.470	T1	.470	.235	150	143	.277	
12	8-31-61	.290	R	.290	.252	30	-	.252	
12	9-4-62	.240	T1	.240	.120	45	62	.139	
12	8-29-63	.820	T1	.820	.410	45	63	.467	
12	3-4-64	.950	T1	.950	.475	105	91	.563	
12	3-1-65	.199	T2	.154	.099	255	240	.118	273
12	5-24-66	.303	T2	.303	.152	180	180	.156	107
12	5-31-67	.120	R	.120	.120	30	-	.280	
28	6-30-57	.42	R	.42	.42	15	-	.700	
28	7-22-58	.610	T1	.610	.305	35	45	.347	43
28	9-9-59	1.60	T1	1.60	.800	30	49	.935	40
28	1-17-60	.066	R	.066	.066	75	-	.066	114
28	11-15-61	.270	R	.270	.270	30	-	.268	41
28	9-4-62	.350	T1	.350	.175	30	39	.208	
28	8-29-63	.624	R	.624	.624	22	-	.624	
28	3-4-64	1.00	T2	.621	.311	90	-	.368	91

WTSD	Date of Event	Rainfall Excess Peak (Hr/In)	Pattern Used R,T1,T2	Max qs (In/Hr)	Mean $\overline{q}s$ (In/Hr)	OBS D_1 (Min)	CAIC D_2 (Min)	Mean $\overline{q}$'s (In/Hr)	T_{50} (Min)
28	3-1-65	.120	T1	.120	.060	75	124	.070	
28	5-4-66	.155	R	.155	.155	15	15	.155	
28	5-31-67	.480	R	.480	.480	15	15	.480	
32	11-18-57	1.36	T1	1.36	.680	90	81	.801	
32	4-14-58	.160	T1	.160	.080	330	285	.093	161
32	5-22-59	.700	R	.700	.700	30	30	.700	
32	3-2-60	.816	T2	.512	.256	270	-	.285	135
32	8-31-61	.560	R	.560	.560	45	-	.560	
32	9-4-62	.690	T1	.690	.345	45	34	.410	
32	8-29-63	.680	R	.680	.680	45	-	.520	
32	3-4-64	.834	T1	.834	.417	165	100	.495	
32	3-1-65	.189	T1	.189	.095	240	211	.105	183
32	5-24-66	1.70	T1	1.70	.850	60	64	.992	
32	5-31-67	.686	T1	.686	.343	30	53	.400	
35	11-18-57	2.09	T1	2.09	1.05	90	46	1.23	
35	4-14-58	.900	T1	.900	.450	240	270	.116	222
35	5-24-59	1.82	R	1.82	1.82	10	-	1.82	
35	3-2-60	1.14	T1	1.14	.570	270	126	.673	
35	8-31-61	.240	R	.240	.240	15	-	.240	
35	9-4-62	.620	R	.620	.620	15	-	.62	
35	8-29-63	.20	R	.20	.20	15	-	.202	
35	3-4-64	1.35	T2	.962	.481	135	-	.526	130
35	3-1-65	.213	T1	.213	.107	405	247	.117	246
35	5-24-66	1.24	T1	1.24	.620	30	49	.725	
35	5-31-67	1.23	T1	1.23	.617	30	46	.722	

APPENDIX C

COSHOCTON, OHIO

WTSD	X-Sec Label	Dist Up-stream From Gaging Station	Depth	Top Width	Cross Sectional Area	Predicted Cross Sectional Radius	Actual Hydraulic Radius	Predicted Hydraulic Radius	Actual Wetted Perimeter	Predicted Wetted Perimter	Elev of Lowest Portion X-Sec
97	B	1000	5.8	81.0	165.8	248.8	2.0	2.9	84.2	84.9	813.5
97	C	2000	6.0	37.3	125.5	118.5	3.2	3.0	39.2	39.1	816.5
97	D	3000	2.0	27.5	29.8	29.1	1.0	1.0	29.7	28.8	822.0
97	F	5500	3.9	48.5	138.0	100.1	2.7	2.0	51.7	50.8	831.1
97	H	7500	4.6	29.0	104.5	71.0	3.4	2.3	30.9	30.4	840.4
97	J	9500	4.4	29.5	79.1	68.7	2.4	2.2	32.5	30.9	852.7
97,95	L	1100	2.4	19.5	33.8	24.9	1.6	1.2	20.7	20.4	865.5
97,95	M	2000	4.6	85.0	250.3	209.1	3.0	2.4	84.6	88.0	873.0
97,95	N	3000	3.6	19.5	56.1	37.6	2.5	1.8	22.7	20.6	879.1
97,95,94	Q	800	1.7	9.0	11.9	7.5	1.2	0.9	10.3	8.7	900.3
97,95,94	R	1600	1.4	11.0	13.2	8.2	1.1	0.7	12.4	11.5	906.8
97,95,94	T	3000	2.1	8.3	14.1	9.2	1.1	1.1	12.7	8.7	924.1
97,95,94	T+500	3500	1.5	14.9	18.9	11.8	1.2	0.8	16.1	15.6	929.5
97,95,94,92	V	800	2.1	10.5	12.4	11.3	1.1	1.1	11.5	10.7	940.8
97,95,94,92	W	1600	2.6	8.0	13.5	12.4	1.2	1.3	11.8	9.4	953.6
97,95,94,92	W+500	2100	2.8	14.0	16.5	17.9	1.0	1.2	15.9	14.8	962.3
97,95,94,92	Y	3200	3.0	17.7	34.6	28.1	1.8	1.5	18.8	18.6	988.2
97,95,94,92	Z	4000	2.9	33.0	72.2	50.7	2.1	1.5	33.8	34.6	998.2
97,95,94,92	Z1	4800	1.8	10.2	15.2	97.7	1.4	0.9	11.0	10.7	1022.2
194	A-194	389	2.5	14.0	25.1	17.8	1.7	0.8	21.3	22.0	957.0
194	B-194	505	2.6	26.5	74.0	53.3	1.7	3.0	18.7	17.8	958.6

WTSD	X-Sec Label	Dist Up-stream From Gaging Station	Depth	Top Width	Cross Sectional Area	Predicted Cross Sectional Radius	Actual Hydraulic Radius	Predicted Hydraulic Radius	Actual Wetted Perimeter	Predicted Wetted Perimeter	Elev of Lowest Portion X-Sec
194	C194	927	1.2	10.5	14.6	12.1	1.3	1.1	11.2	11.0	983.3
196	A196	665	2.4	14.0	25.1	17.8	1.6	1.2	15.3	14.7	922.0
196	B196	540	3.8	26.5	74.0	53.3	2.6	1.9	28.8	27.8	929.1
196	C196	770	1.7	13.5	16.3	12.1	1.1	0.9	14.2	14.1	937.1
97,95,94,92,5	A5	-1800	.5	3.5	1.8	0.9	0.5	0.2	3.7	3.7	954.9
97,95,94,92,5	B5	-1000	1.5	6.5	5.3	5.2	0.7	0.8	7.8	6.8	966.2
97,95,94,92,5	C5	-200	1.7	7.5	8.5	6.8	0.9	0.9	9.7	7.9	976.1
97,96,94,92,5	D5	+600	3.1	8.5	16.9	14.1	1.6	1.6	10.8	8.9	988.6
97,96,94,92,5	E5	+1400	1.1	1.5	1.2	0.9	0.4	0.6	3.0	1.6	1005.5
17,95,10	10A	525	1.6	13.4	12.5	11.4	0.7	0.8	18.1	14.0	-
97,95,10	10B	1500	1.4	6.3	2.7	4.6	0.4	0.7	7.0	6.6	-
					OXFORD, MISSISSIPPI						
4	85+00	0	5.4	61.0	288.3	280.8	4.4	4.0	65.7	70.2	452.3
4	70+00	1500	5.1	33.4	151.6	145.2	4.2	3.8	36.3	38.4	459.0
4	40+00	4500	6.0	34.1	186.1	174.4	4.6	4.4	40.2	39.2	470.6
4	19+76	6524	8.5	39.0	273.1	282.6	5.9	6.3	46.1	44.9	479.5
4	-14+50	9950	11.2	39.0	349.6	372.3	7.1	8.3	49.1	44.9	499.0
5	50+00	0	6.2	54.0	227.6	285.4	3.6	4.6	63.1	62.1	452.7
5	40+00	1000	5.7	63.5	263.3	308.5	3.9	4.2	66.8	73.1	457.3
5	30+00	2000	4.2	38.1	125.1	136.4	2.8	3.1	44.1	43.8	461.3
5	10+00	4000	4.6	27.7	100.7	108.6	3.3	3.4	30.7	31.9	470.5
5	0+00	5000	4.0	36.5	92.5	114.2	2.5	3.0	36.5	38.6	478.5
10	437+45	0	11.9	109.3	1074.7	1108.7	8.2	8.8	130.5	125.8	395.7
10	410+00	2745	10.6	112.0	943.5	1012.0	7.4	7.9	127.8	128.9	405.7
10	380+00	5745	9.5	39.0	310.1	315.8	6.0	7.0	51.4	44.9	414.5
10	335+00	10245	8.8	46.5	360.7	348.8	6.1	6.5	59.3	53.5	432.9
10	310+10	12735	8.5	48.8	301.0	353.6	5.5	6.3	55.1	56.2	443.6

WTSD	X-Sec Label	Dist Up-stream From Gaging Station	Depth	Top Width	Cross Sectional Area	Predicted Cross Sectional Radius	Actual Hydraulic Radius	Predicted Hydraulic Radius	Actual Wetted Perimeter	Predicted Wetted Perimeter	Elev of Lowest Portion X-Sec
10	295+00	14245	9.1	28.8	201.3	223.4	4.9	6.7	40.7	33.1	449.0
12	320+00	0	8.8	66.5	478.3	498.8	6.5	6.5	73.5	76.5	386.0
12	245+00	7500	7.9	37.3	260.7	251.2	5.6	5.9	46.4	42.9	407.0
12	200+00	12000	4.7	37.5	162.3	150.2	3.8	3.5	43.0	43.1	424.0
12	145+00	17500	4.3	42.3	128.6	155.0	2.9	3.2	45.1	48.7	441.0
12	100+00	22000	5.8	33.8	139.0	143.4	4.1	4.3	33.8	33.4	455.0
12	60+00	26000	4.8	33.6	89.6	124.8	2.7	3.6	33.6	35.1	472.7
17A	44+85	2995	5.0	41.0	139.3	174.7	3.0	3.7	46.9	47.2	370.7
17A	100+15	8525	5.3	42.6	173.1	192.5	3.7	3.9	46.3	49.0	390.2
17A	130+00	11510	3.2	65.3	191.9	178.1	2.9	2.4	66.9	75.1	404.7
17A	185+00	17010	5.5	35.0	186.1	164.1	4.3	4.1	43.7	40.3	429.0
17A	204+75	18985	5.9	34.9	144.2	175.5	3.5	4.4	40.7	40.2	440.0
17A	223+50	20860	6.8	36.7	211.2	212.7	4.9	5.0	43.3	42.2	450.8
19	5+00	3903	4.0	13.4	28.2	45.7	1.8	2.9	15.6	15.4	362.7
19	25+00	1903	2.2	17.0	17.5	31.9	1.0	1.6	18.2	19.6	371.5
19	30+00	1403	3.2	13.1	32.5	35.7	1.9	2.4	17.4	15.1	373.4
19	40+00	403	3.7	8.0	14.9	25.2	1.2	2.7	12.2	9.2	377.5
19	45+00	-97	1.8	9.4	11.7	14.4	1.1	1.3	10.5	10.8	381.6
24	1+15	1950	4.8	25.7	85.7	105.2	2.8	3.6	30.3	29.6	441.5
24	15+00	5650	4.5	29.5	87.7	113.2	2.7	3.3	32.5	33.9	450.3
24	25+00	-435	6.0	40.0	158.1	179.0	4.0	4.5	40.0	40.3	455.0
24	30+00	-935	5.7	23.8	104.6	115.6	3.7	4.2	28.3	27.4	456.7
28	65+00	0	6.7	21.5	121.8	122.8	4.1	5.0	29.5	24.7	491.3
28	55+00	1000	7.2	21.5	117.5	132.0	4.0	5.3	29.1	24.7	496.0
28	40+00	2500	4.3	25.2	88.3	92.4	3.0	3.2	28.7	29.0	506.0
28	19+75	4525	4.0	24.0	69.5	81.8	2.7	3.0	26.2	27.6	520.0
28	5+20	5980	3.7	17.3	47.1	54.6	2.3	2.7	20.4	19.9	533.0
32	545+00	477	11.2	105.0	1096.1	1002.5	9.3	8.3	117.3	120.8	367.0
32	480+15	6932	11.5	77.0	794.5	754.8	9.1	8.5	87.5	88.6	377.0
32	425+00	12447	12.2	84.2	937.7	875.6	9.7	9.0	97.0	96.9	387.5

WTSD	X-Sec Label	Dist Up-stream From Gaging Station	Depth	Top Width	Cross Sectional Area	Predicted Cross Sectional Radius	Actual Hydraulic Radius	Predicted Hydraulic Radius	Actual Wetted Perimeter	Predicted Wetted Perimeter	Elev of Lowest Portion X-Sec
32	365+00	18447	13.3	91.5	1096.8	1037.4	10.5	9.9	104.4	105.3	397.0
32	320+00	22947	16.5	86.7	1170.0	1219.4	11.7	12.2	99.7	99.8	405.0
35	305+00	1000	17.5	89.4	1352.0	1333.6	12.6	13.0	107.1	102.9	407.7
35	290+00	2500	16.6	85.5	1232.1	1209.8	11.8	12.3	104.5	98.4	410.8
35	175+00	14000	12.6	56.6	577.3	607.9	8.6	9.3	67.2	65.1	440.3
35	160+00	15500	12.2	48.0	490.9	499.2	8.0	8.2	61.0	55.2	446.0
35	80+00	23500	8.3	46.5	280.8	329.0	5.3	6.1	53.2	53.5	475.4
35	70+00	24500	7.4	29.5	152.3	186.1	4.2	5.5	35.8	33.9	478.5

A P P E N D I X D

COSHOCTON, OHIO

WTSD	Date of Event	OBS Q_p (CFS)	$\overline{q}'$ (In/Hr)	CALC D_2 (Min)	$\overline{q}s$ (Min)	T_{CH} (Min)	T_{50} (Min)	D/T_{50}	T_{PR}/D	C_p	C_t	CALC Q_p (CFS)	CALC Q_p + Base Flow(CFS)
5	6-18-40	40	1.38	11	1.17	8.65	33.6	.33	.72	.24	1.19	118	119
5	6-29-40	31	.468	37	.400	9.86	38.4	.96	.64	.80	1.19	134	135
5	9-23-45	113	.985	33	.835	8.86	34.5	.97	.78	.81	1.19	283	283
5	6-12-57	152	1.32	19	1.26	8.61	33.5	.57	.22	.48	1.06	226	226
5	6-28-57	384	1.87	54	1.58	8.49	33.0	1.64	.83	.99	1.19	655	655
5	8-21-60	338	3.33	22	2.97	8.43	32.8	.67	.40	.59	1.14	703	724
5	4-25-61	97	.92	22	.81	8.88	34.5	.64	.43	.56	1.15	184	199
10	6-18-40	45	1.93	14	1.65	8.81	34.3	.41	.64	.32	1.19	77	77
10	6-29-40b	8	1.78	5	1.50	9.10	35.4	.14	.82	.075	1.19	16	16
10	9-23-45a	212	1.85	30	1.55	9.00	35.0	.86	1.00	.74	1.19	168	168
10	6-12-57	40	1.80	16	1.70	8.72	33.9	.47	.24	.38	1.06	84	84
10	8-21-60b	45	.706	16	.634	12.20	47.5	.34	.38	.25	1.13	22	27
10	4-30-66	4	3.25	2	3.25	7.94	30.9	.06	-	.03	-	12	13
92	6-18-40	78	1.42	15	1.21	16.64	64.7	.23	.66	.15	1.19	200	235
92	6-28-40	75	1.18	16	1.00	17.01	66.2	.24	.79	.15	1.19	165	199
92	9-23-45	212	1.12	32	.944	17.13	66.6	.48	.80	.39	1.19	406	406
92	6-12-57	262	1.71	15	1.61	16.13	62.7	.24	.26	.15	1.07	240	240
92	6-28-57b	578	1.13	75	.949	17.12	66.6	1.13	.95	.89	1.19	932	932
92	8-21-60	502	3.05	15	2.65	15.82	61.5	.24	.47	.15	1.16	428	483
92	4-25-61	436	1.16	35	.990	17.03	66.2	.53	.61	.44	1.18	477	518
94	6-18-40	248	1.60	15	1.36	34.69	135	.11	.66	.06	1.19	149	149
94	6-29-40	184	1.95	11	1.65	34.57	134	.08	.77	.04	1.19	120	126
94	9-23-45a	608	1.36	32	1.15	34.80	135	.24	.87	.15	1.19	314	314
94	6-12-57	670	2.19	20	2.10	34.44	134	.15	.20	.08	1.05	270	270

WTSD	Date of Event	OBS Q_p (CFS)	$\overline{q}'$ (In/Hr)	CALC D_2 (Min)	$\overline{q}s$ (Min)	T_{CH} (Min)	T_{50} (Min)	D/T_{50}	T_{PR}/D	C_p	C_t	CALC Q_p (CFS)	CALC Q_p + Base Flow(CFS)
94	6-28-57	1407	2.01	68	1.70	34.56	134	.51	.93	.42	1.19	1302	1302
94	8-21-60	958	3.27	21	2.91	34.27	133	.16	.41	.09	1.14	457	518
94	2-11-65	36	.481	16	.405	36.15	141	.11	.98	.06	1.19	44	53
94	4-24-66	47	.483	17	.406	36.15	141	.12	1.00	.06	1.19	44	54
94	4-30-66	50	.450	10	.450	35.98	140	.07	-	.03	-	21	28
95	6-18-40	435	2.50	15	2.13	28.40	110.5	.14	.69	.07	1.19	459	465
95	6-28-40	396	1.89	14	1.60	28.40	110.5	.13	.84	.07	1.19	345	355
95	9-23-45a	938	1.52	29	1.27	28.60	111.3	.26	1.00	.17	1.19	668	668
95	6-12-57	897	2.28	22	2.19	28.40	110.5	.20	.18	.12	1.04	708	708
95	8-22-60	1065	2.58	27	2.22	28.40	110.5	.24	.51	.15	1.17	1009	1034
95	4-25-61	1182	1.71	32	1.44	28.40	110.5	.29	1.00	.20	1.19	888	953
95	4-24-66	62	.495	18	.417	31.30	121.8	.15	1.00	.08	1.19	103	105
95	4-30-66	65	.447	10	.447	31.07	120.9	.08	-	.04	-	46	57
97	6-18-40	776	2.21	14	1.90	58.00	226	.06	.54	.03	1.18	310	333
97	6-28-40	536	2.13	20	1.80	58.00	226	.09	.82	.04	1.19	396	410
97	6-4-41	1256	1.23	40	1.06	58.28	227	.18	.50	.10	1.17	570	570
97	7-11-46	2531	1.55	21	1.44	58.16	226	.09	.29	.04	1.09	288	315
97	6-12-57	1663	2.19	20	2.10	58.00	226	.09	.20	.04	1.05	407	407
97	8-22-60	970	2.00	27	1.71	58.00	226	.12	.53	.06	1.17	554	554
97	4-25-61	1201	1.31	61	1.10	58.26	227	.27	1.00	.18	1.19	1088	1088
196	6-18-40	26	.892	14	.750	10.04	39.1	.36	1.00	.26	1.19	71	71
196	6-28-40a	42	2.04	9	1.75	8.20	31.9	.28	.53	.19	1.17	119	122
196	4-25-61	339	2-26	39	1.92	8.10	31.5	1.24	.70	.93	1.19	649	667
196	5-13-64	156	1.25	19	1.07	8.90	34.6	.55	.55	.46	1.18	177	177
196	3-12-67	4.6	1.34	10	1.13	8.11	31.5	.32	1.00	.23	1.19	94	94
196	3-17-67	9.6	.472	27	.450	11.94	46.4	.58	.22	.49	1.06	71	71
196	5-7-67	16	1.03	18	.870	9.54	37.1	.49	.96	.40	1.19	126	126

OXFORD, MISSISSIPPI

WTSD	Date of Event	OBS Q_p (CFS)	$\overline{q}'$ (In/Hr)	CALC D_2 (Min)	$\overline{q}s$ (Min)	T_{CH} (Min)	T_{50} (Min)	D/T_{50}	T_{PR}/D	C_p	C_t	CALC Q_p (CFS)	CALC Q_p + Base Flow(CFS)
4	4-3-58	293	.644	28	.560	29.14	64.4	.43	.91	.34	1.19	457	457
4	8-29-63	568	.713	46	.599	28.48	62.9	.73	1.00	.64	1.19	920	920
4	3-4-64	1662	.920	123	.780	26.03	57.5	2.14	.83	.99	1.19	1853	1853
4	3-3-66	146	.346	40	.295	36.91	81.6	.49	.333	.39	1.11	258	258
4	5-24-66	532	.264	15	.334	40.47	89.4	.17	1.00	.10	1.19	80	80
4	5-31-67	249	.691	35	.287	28.82	63.7	.55	1.00	.46	1.19	317	317
5	4-3-58	350	1.24	29	1.05	20.92	46.2	.63	.85	.55	1.19	783	783
5	6-10-59	692	1.03	48	.88	22.21	49.1	.98	.70	.81	1.19	966	966
5	6-11-59	569	2.20	27	1.87	17.20	38.0	.71	.74	.62	1.19	1568	1568
5	9-4-62	208	.979	18	.840	22.56	49.9	.36	.56	.26	1.18	294	294
5	3-4-64	1355	1.42	107	1.12	20.46	45.2	2.37	.89	.99	1.19	1503	1503
5	5-31-67	432	.714	40	.691	24.11	53.3	.75	.30	.66	1.09	566	566
10	5-22-59	525	.360	15	.360	37.72	83.4	.18	-	.10	-	201	201
10	8-31-61	2415	.662	70	.570	32.26	71.3	.98	.51	.81	1.17	3012	3012
10	9-4-62	730	.244	54	.205	43.05	95.1	.57	1.00	.48	1.19	653	653
10	8-29-63	1960	.421	109	.360	37.72	83.4	1.31	.63	.95	1.18	2250	2250
10	3-4-64a	1070	.428	40	.372	37.30	82.4	.49	.48	.40	1.17	967	1045
10	5-31-67	906	.312	51	.267	41.75	92.3	.55	.61	.46	1.18	808	808
12	11-13-57a	4180	.493	26	.415	57.13	126	.21	1.00	.13	1.19	1346	1346
12	4-3-58	1920	.855	10	.723	54.37	120	.08	.83	.04	1.19	722	722
12	3-2-60	2274	.277	143	.235	62.46	138	1.04	.742	.85	1.19	4985	4985
12	8-1-61	1134	.252	30	.145	64.49	143	.21	.576	.13	1.18	467	467
12	8-29-63	3631	.467	63	.410	57.20	126	.50	.46	.41	1.16	4090	4090
12	3-4-64	5413	.563	91	.475	56.30	124	.73	.89	.64	1.19	7587	7901
12	5-31-67	1702	.280	30	.280	62.30	138	.22	-	.14	-	822	852
28	6-30-57	145	.700	15	.700	25.03	55.3	.27	-	.18	-	137	137
28	6-30-57	145	.700	15	.700	25.03	55.3	.27	-	.18	-	137	137
28	9-9-59	611	.935	49	.800	23.92	52.9	.93	.61	.78	1.18	801	801
28	11-15-61	159	.268	30	.268	34.69	76.7	.39	-	.29	-	85	85
28	9-4-62	109	.208	39	.175	35.77	79.1	.49	1.00	.40	1.19	91	91

WTSD	Date of Event	OBS Q_p (CFS)	$\overline{q}'$ (In/Hr)	CALC D2 (Min)	$\overline{q}s$ (Min)	T_{CH} (Min)	T_{50} (Min)	D/T_{50}	T_{PR}/D	C_p	C_t	CALC Q_p (CFS)	CALC Q_p + Base Flow(CFS)
28	8-29-63	154	.624	22	.624	26.03	57.5	.38	-	.28	-	190	190
28	5-24-66	23	.155	15	.155	39.53	87.4	.17	-	.10	-	325	325
32	11-18-57	5699	.801	81	.680	69.89	154	.53	.74	.44	1.19	7180	7180
32	5-22-59	1799	.700	30	.700	69.38	153	.20	-	.12	-	1693	1693
32	8-31-61	4336	.560	45	.560	74.07	164	.27	-	.18	-	2033	2033
32	9-4-62	2120	.410	34	.345	87.33	193	.18	1.00	.10	1.19	828	828
32	8-29-63	3821	.520	45	.520	75.96	168	.27	-	.18	-	1888	1888
32	3-4-64	4640	.495	100	.417	81.88	181	.55	.92	.46	1.19	4603	4875
32	5-24-66	5900	.992	64	.850	66.14	146	.44	.59	.35	1.18	7079	7079
32	5-31-67	2850	.400	53	.343	87.50	193	.27	.57	.18	1.18	1469	1469
35	11-18-57	1770	1.23	46	1.05	37.20	82.2	.56	.65	.47	1.19	4449	4449
35	5-22-59	1300	1.82	10	1.82	30.81	68.1	.15	-	.08	-	1108	1108
35	3-2-60	1774	.673	126	.57	45.72	101.0	1.25	.80	.93	1.19	4802	4802
35	8-31-61	260	.240	15	.240	61.34	135.6	.11	-	.06	-	110	110
35	9-4-62	837	.620	15	.620	44.43	98.2	.15	-	.08	-	377	377
35	8-29-63	258	.202	15	.100	65.05	143.8	.10	-	.05	-	38	38
35	5-24-66	1511	.725	49	.620	44.43	98.2	.50	.61	.41	1.18	2284	2284
35	5-31-67	1588	.722	46	.620	44.43	98.2	.47	.65	.38	1.19	2134	2134