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PREDICTION OF THE MAGNITUDES AND FREQUENCIES OF FLOODS IN MICHIGAN

Ernest F. (Ernest Frederick)

E. F. Brater
Professor of Hydraulic Engineering

J. D. Sherrill
Research Assistant

Department of Civil Engineering
University of Michigan

Ann Arbor, Michigan

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ABSTRACT

This research program was undertaken to gain a better understanding of the surface runoff process and to derive an accurate and practical method of predicting the magnitudes of flood flows of any frequency on drainage basins of various sizes and degrees of urbanization. The support of the Michigan Department of State Highways and the Federal Highway Administration was used to provide a continuing emphasis on applications to Michigan and particularly to Southeastern Michigan. The other principal sponsor was the U. S. Environmental Protection Agency.

The basis for the research was the analysis of rainfall and snow melt and the corresponding flood runoff which occurred in drainage basins varying in size from .02 to 734 square miles and in population density from 100 to 13,000 persons per square mile.

All known methods of predicting floods from precipitation were investigated and the infiltration capacity-unit hydrograph procedure was selected as the most suitable method for practical application because it combined a high degree of accuracy with simplicity of application. This method requires that information on infiltration capacity and hydrograph shape be obtained from the analysis of rainfall and surface runoff events. The infiltration capacities apply specifically to the areas where they were obtained. The infiltration values shown in the report and used in deriving the design curves were

obtained from 16 drainage basins located in Southeastern Michigan. They may be used elsewhere if the soil and vegetative cover are similar. The shape of the unit hydrograph depends on the physical characteristics of the drainage basin and the degree of urbanization. The research showed that the two most important parameters are the area of the drainage basin and the population density.

Curves were derived relating peak unit hydrograph discharge to area and population density.

A frequency curve of rainfall plus snow melt was prepared from the analysis of 535 station years of records for Southeastern Michigan. With this information a set of design curves for flood magnitudes was derived for Southeastern Michigan for frequencies of 10, 20, 30, 50 and 100 years and for population densities varying from 100 to 10,000 persons per square mile. It is believed that these provide the most accurate procedure for designing storm sewers, culverts and bridge openings in this area and that they could also be applied elsewhere in Michigan with great confidence except where the soil is very sandy.

The report is divided into two parts, Part I, pages 1-46 provides a description of the research and detailed information on the results and applications. Part II, pages 47-52 gives the practical design curves.

PART I

The Research Program

Synopsis

This research grant was oriented toward the development of a workable procedure for predicting flood magnitudes of various frequencies for basins in Southeastern Michigan. Such a procedure was developed and is presented in Part II of this report. However, accomplishing this objective required such a tremendous amount of preliminary work as well as related basic research that a brief summary of the total research program as it has progressed to this date is also presented.

The first portion of the report provides a brief historical review of the total research effort related to surface runoff processes. This is followed by an outline of the present state of the art including a brief discussion of other methods of approaching the problem. The amount of effort required to gather data from small urbanized areas is discussed and the procedures developed to analyze rainfall hyetographs and runoff hydrographs to determine the surface runoff hydrograph and infiltration capacity are described. Methods of computing infiltration capacity and its variations with time and location are discussed. Progress in learning about the factors which influence surface runoff hydrograph shape is presented in some detail. The development and use of a mathematical watershed model is also referred to in the discussion. Current and future investigations which are being planned to improve the new techniques are described briefly. As mentioned above the final section of this report presents the working method including the most recent inputs of new data.

Historical Resume

Research on surface runoff processes was started with a small grant from the University of Michigan Graduate School in 1964. In 1965 a grant was obtained from the National Institute of Health (continued by FWQA and the Environmental Protection Agency to the present time) for fundamental research on runoff processes. This grant was later re-oriented to put major emphasis on the effects of urbanization on small drainage basins. Participation of the Michigan State Highway Department, in cooperation with the U. S. Bureau of Public Roads, began in 1966. The grant from the Highway Department was made on the basis that the portion of the effort supported by this grant would be directed toward developing better practical procedures for predicting floods of various frequencies for the design of drainage structures in Southeastern Michigan. During the last several years there have also been major financial inputs to the total research program by two consulting engineering firms which have also provided emphais on applications. Over the total period of research the financial input of the Highway Department grant amounted to between a third and one fourth of the total funds. The Highway Department grant provided funds for one graduate student as a half time research assistant plus some additional student help as well as for miscellaneous expenses related to obtaining rainfall and runoff data, and for computer time.

The ultimate goal of this total research program is the development of dependable flood prediction procedures and is therefore little different than that of the Highway Department grant itself. However, the Highway Department grant provided funds and incentive for maintaining a continuing emphasis on practical application. On the other hand the analysis of hundreds of storm events, the fundamental research and the development of a mathematical model made possible by the other related grants were also essential to the attainment of the practical objectives. In other words, the last portion of this report which is a specific answer to the Highway Department needs could not have been produced without the total research program.

The State of the Art

An intensive study of procedures and literature dealing with runoff processes was carried out at the beginning of the research. Most all of this was completed before the first Highway Department grant was started. In addition to studying the literature, the principal investigator visited centers and researchers working on this phase of hydrology in the U. S. and Europe. Rather complete outlines of this phase of the work have been presented.[1,2,3]

The review of the subject included a re-evaluation of methods which had been used in the past as well as those in current usage. In a general way, these procedures can be separated into the following categories; statistical methods, procedures utilizing empirical equations or curves, storage routing procedures and unit hydrograph procedures. The later two categories refer to the way in which the surface runoff hydrograph is formed, the total volume of runoff being usually determined by the use of the infiltration capacity concept.

Statistical methods utilize flood records on a particular basin. Sometimes they are applied regionally by assuming similar rainfall, snow melt and drainage basin characteristics. These methods are usually limited in accuracy because of lack of sufficient records to make a significant statistical analysis. They lack the flexibility to determine the effect of urbanization or other watershed changes. The various empirical methods also require a fairly long period of records to determine the necessary constants and again they do not lend themselves to modification for changing conditions.

Many storage routing methods have been developed. In most such methods, the total volume of rainfall excess or surface runoff is determined by applying the infiltration capacity to the rain intensity. The rainfall excess may then be routed one or more times through simulated basin storage to obtain runoff or the hydrograph of outflow from the land surface may be determined first and then this outflow is treated as inflow to the channel storage system. In many such procedures a time-area graph is developed to simulate travel time in the river system and contributions for various successive time increments are combined by a convolution process prior to routing through storage. Storage methods have one striking similarity to unit hydrograph methods in that the entire process is assumed to be linear. This means that discharge is assumed to be directly proportional to storage and that the total discharge at any time is equal to the sum of the responses from several increments of input. The results obtained from experimenting with several of these methods indicated that they are no more accurate than the unit hydrograph procedure and that they are more difficult to use especially for engineers who predict floods only occasionally and don't have time to become expert hydrologists. The conclusion was reached that with the present state of the art no other procedure provides the accuracy along with simplicity and flexibility for changes in basin characteristics that has been demonstrated for the unit hydrograph method.

Hydrograph Analysis

It was clearly recognized from the beginning of this research that any effort that would provide a better understanding of surface runoff processes and eventually lead to a dependable method of predicting runoff from rainfall would require the analysis of many rainfall and/or snow melt and runoff events from many different sizes and types of drainage basins. Hydrograph analysis on perennial streams for the purpose of determining the surface runoff, infiltration capacity and the form of the surface runoff hydrograph requires the separation of surface runoff from ground water discharge or base flow. The method conventionally used is based on some judgment. Therefore one of the initial goals of this project was to develop an objective method of carrying out this operation. Furthermore, it had been common practice to neglect initial retention (the portion of the rain which is intercepted by vegetation or the ground surface and never becomes infiltration or surface runoff) and the effect of runoff from impervious areas on the computations of infiltration capacity. Once a satisfactory solution to these problems was attained[1] computer programs were written for the portion of this operation for which they were helpful and the hydrograph analysis was carried out in an intensive manner. A great deal of effort was devoted to searching for available runoff and rainfall records. It became obvious early in the work that the effect of urbanization would be one of the most important factors in the relations dealing with hydrograph shape. Many runoff records from urbanized areas are unpublished and must be obtained by copying from the original hydrographs where they are stored. Even the records for runoff from Southeastern Michigan Streams were not published in sufficient detail and therefore it was necessary to work from the original gage charts or tapes in the U.S.G.S. office to obtain the records. In the case of the Red Run drainage basin it was also necessary to combine the runoff from the gaging station with the discharge diverted into the sewer system. Another part of the analysis consisted of the determination of the weighted hourly precipitation for each event and the computation of snow melt when it occurred. Each event was then plotted as shown in Fig. 1. It is estimated that the collection of data along with the hydrograph analysis mentioned above required about 70 per cent of the total effort to date.

The number of drainage basins studied was 58, varying in size from 0.02 to 734 square miles with 42 basins having areas of less than 20 square miles. Sixteen basins were located in Southeastern Michigan, 31 in urban areas in Texas, five in Louisville, Kentucky and six at other locations in the United States. Population densities vary from less than 100 to more than 13,000 persons per square mile. More than 1,300 flood runoff hydrographs were analyzed.

The Unit Hydrograph-Infiltration Capacity Procedure

Extensive reference material to literature on this topic has been presented in publications resulting from this research.[1,2,3] This method was first developed early in the 1930's. Many refinements were developed during the 1930's and 1940's. During the last ten years there has again been a large amount of research and writing on this subject.

Infiltration Capacity. The determination of infiltration capacity for a runoff event may be described with reference to Fig. 1. The amount of surface runoff is computed first, then the infiltration capacity is computed by finding the value which makes the precipitation excess equal to the surface runoff. Sketching this value of infiltration capacity on the hyetograph as shown in Fig. 1 then establishes the duration of precipitation excess which becomes an important parameter in the formation of the surface runoff hydrograph. Infiltration capacity decreases during rain storms as illustrated by the two rains in Fig. 1 and it also varies seasonally as shown in Fig. 2. The only method of determining the average infiltration capacity of a particular drainage basin is by means of hydrograph analysis. Each of the points in Fig. 2 was determined in that manner. Infiltration capacity also varies from place to place depending primarily on soil type. The values shown in Fig. 2 are from fourteen drainage basins located in the Detroit Metropolitan area. There may be large differences between these basins and those in other parts of Michigan. The computed value of infiltration capacity depends to some extent on whether retention and runoff from impermeable areas are included in the computations. The latter factor becomes particularly important for highly urbanized areas where the impermeable area becomes large. In the analysis, the total surface runoff (SRO) is taken to be equal to the surface runoff from impermeable area (SRO_i) plus that from the permeable area (SRO_p) and these latter values are defined by the following equations

$$SRO_i = P - R_i$$

$$SRO_p = P - F - R_p$$

where P is the average precipitation, R_i is the retention on the impermeable area, F is the total infiltration and R_p is the retention on the permeable area. A detailed description of the determination of the impermeable area and the retention has been presented.[1]

The computations for determining the infiltration capacity for the first stream rise in Fig. 1 are presented here to illustrate the procedure.

Area of Drainage Basin, $A = 22.9$ sq. mi.

Impermeable area, $A_i = .02A$

Retention on impermeable area, $R_i = 0.02$in.

Retention on permeable area, $R_p = 0.10$in.

Weighted average precipitation, $P = 1.35$in.

The first step is the computation of the surface runoff. This requires that a line separating surface runoff from groundwater discharge be drawn such as b_1b_2 in Fig. 1. The procedure used to do this was referred to earlier in this report and is presented in detail elsewhere.[1] In the example shown in Fig. 1, it was also necessary to draw the line xy to separate the hydrograph produced by the first rain from the one resulting from the second rain. This is done by sketching a line having the same form as S b_2 which represents the same release of storage as would have occurred at the end of the first hydrograph. Ordinates from the first hydrograph were tabulated for two hour intervals as shown in columns 1 and 2, Table 1. The surface runoff is then computed from the summation of column 2 by multiplying by the number of seconds in two hours and dividing by the area of the basin as follows.

$$SRO = \frac{3456 \times 2 \times 3600}{22.9 \times 5280 \times 5280} \times 12 = .468\text{in.}$$

The surface runoff from the impermeable area is computed using the equation shown above and is then converted to inches on the entire basin by multiplying by the ratio of the impermeable area to the total area as follows

Table 1

Computations of Surface Runoff and Unit Hydrograph Ordinates for Rain of April 1, 1959, on Plum Brook. Drainage Area = 22.9 Sq. Mi.

No. of two Hour Intervals	Avg. Rate of Surface Runoff in cfs	Unit Hydrograph in cfs per sq. mi. per in.
1	28	2.6
2	116	10.8
3	230	21.5
4	294	27.4
5	308	28.8
6	320	29.9
	(324)	(30.3) (peak)
7	320	29.9
8	308	28.8
9	280	26.1
10	240	22.4
11	190	17.7
12	148	13.8
13	120	13.8
14	96	9.0
15	80	7.5
16	66	6.2
17	56	5.2
18	48	4.5
19	40	3.7
20	36	3.4
21	32	3.0
22	26	2.4
23	22	2.1
24	18	1.7
25	14	1.3
26	10	.9
27	7	.7
28	3	.3
	3456	

$$SRO_i = \frac{A_i}{A} (P - R_i)$$

$$= .02 \ (1.35 - .02) = .027\text{in. on } A$$

The surface runoff from the permeable area is then the difference between the total surface runoff and SRO_i and is computed as follows

$$SRO_p = \frac{A}{A_p} (.468 - .027) = \frac{100}{98} \times .441 = .452\text{in. on } A_p$$

The total precipitation excess on the permeable area is the surface runoff from this area, computed above, plus the retention.

$$P_{ep} = SRO_p + R_p$$

$$= .452 + .10 = .552\text{in. on } A_p$$

The infiltration capacity of the permeable area is then computed by trial. The final trial is shown in Table 2 in which hourly precipitation is given in column 2, the assumed infiltration capacity in column 3 and the precipitation excess in column 4. The total of column 4 multiplied by 60/60 to convert from inches per hour to inches is 0.55 which agrees with the value of P_{ep} computed above. Therefore the infiltration capacity of the permeable portion of the basin is 0.124 inches per hour. This procedure applies to simple hydrographs which can readily be assigned to a particular rain. For more complex storms the method involves an application of the unit hydrograph and will be described later.

Table 2

Computation of Infiltration Capacity

Hours	Precipitation Intensity in. per hr.	Infiltration Capacity in. per hr.	Precipitation Excess in. per hr.
17	.00	.124	-
18	.07	.124	-
19	.26	.124	.136
20	.16	.124	.036
21	.18	.124	.056
22	.32	.124	.196
23	.25	.124	.126
24	.03	.124	-
25	.04	.124	-
26	.01	.124	-
27	.03	.124	-
28	.00	.124	-
			.550

Unit Hydrographs. By far the major portion of the research effort was devoted to studying unit-hydrograph characteristics. This procedure in its simplest form is based on the assumption that there is a characteristic form of surface runoff hydrograph for any basin which is constant if the duration of precipitation excess is less than some critical value, that the ordinates of this hydrograph vary linearly with the magnitude of rainfall excess and that various complex rainfall or snow melt inputs can be transformed into a complete hydrograph by a linear additive convolution process.

The unit hydrograph is obtained from a surface runoff hydrograph produced by a precipitation excess having a duration less than some critical duration to be defined later by taking the ordinates for average discharge for selected time intervals and converting them to cfs per square mile per inch of rainfall excess. This is done by dividing the ordinates in cfs by the area of the drainage basin in square miles and by the rainfall excess in inches.* An example is shown in Table 1 for the first of the two hydrographs of Fig. 1. In Fig. 3 are shown five unit hydrographs from the same basin. These five were selected to show typical variations for a basin. The average unit hydrograph is also shown.

The critical duration has been found to be about equal to the lag time for basins up to 80 square miles in area and about half the lag time for larger basins. The lag is defined as the time from the center of gravity of the rainfall excess to the hydrograph peak as shown in Fig. 1.

The most important characteristic of a unit hydrograph is its peak because this is the value used to predict peak flows. However, in order to construct a complete flood hydrograph a complete set of unit hydrograph ordinates and abscissas must be known such as shown in columns 1 and 3 in Table 1 or in Fig. 3. The ultimate test of the accuracy of the unit hydrograph procedure is obtained by applying the unit hydrograph for a drainage basin to a complex rain storm in which the contributions from various portions of the rainfall excess must be added taking into account the time of the periods of rainfall excess and comparing the computed hydrograph with the actual hydrograph. This has been done many times and a typical example is shown in Fig. 4. When a unit hydrograph is applied to successive portions of a

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*The ordinates of a unit hydrograph could also be expressed in percentage of total surface runoff. In this form the graph is often referred to as a distribution graph. For the sake of simplicity only the ordinates mentioned above (cfs per sq. mi. per in.) will be used in this report.

long complex series of rains the infiltration capacity is computed for each separate stream rise and adjusted by trial for overlapping hydrographs to provide information on the variations of infiltration capacity with time during the series of rains.

It became apparent early in the work that an analytical approach to the runoff problem would be an exceedingly valuable tool if it could be developed well enough to simulate natural watershed responses. Consequently a mathematical model of a watershed was developed which seems to fulfill these needs up to a certain point (the next step is to incorporate more complex drainage networks into the model). After developing the analytical model the study of the natural basins and the model basin could be carried along together so that each effort supplemented the other.

The results from both natural watersheds and the analytical model agree that as previously stated that one important time characteristic is the lag and that the shape of the unit hydrograph is relatively independent of the duration of precipitation excess as long as this duration is less than the lag for small basins. Since linearity has been an important inherent characteristic of the unit hydrograph concept, this aspect was also studied intensively. A degree of non-linearity due to the magnitude of the precipitation excess was found under some conditions. This non-linearity could be reduced in the model by changing the stream cross-section so that the rate of change of channel storage with respect to depth was increased thus giving a clue to the cause of non-linearity.

It was previously discovered that for a group of watersheds from within the same large watershed system there was a relatively consistant relationship between unit hydrograph peaks and periods of rise and the areas of the drainage basins.[4] This research provided an opportunity to determine if such relationships exist for a wide variety of watersheds from different regions. The relations between unit hydrograph peak and area and between period of rise and area were found to be quite consistant for all watersheds if the degree of urbanization was accounted for. At this stage in the research the population density seems to be a very significant factor in expressing the degree of urbanization. Satisfactory practical relationships were developed for varying population densities. This factor appears to be much more important than such factors as watershed shape, channel slope or roughness. This is illustrated in Fig. 5 which shows unit hydrograph peaks for all of the watersheds except for those from Dallas and Austin, Texas. Preliminary studies indicated that the Dallas and Austin basins were being influenced by other factors which are not yet understood. The upper points in Figure 5 are for population densities of 6,400 or more persons per square mile with an average of 11,800 persons per square mile and the lower points for population densities of 1,200 or less with an average of 550 persons per square mile. The lines through these two groups were derived by a least squares optimization for a best fit. It is of interest to note that the slopes of lines determined in this manner (-0.41 and -0.36 respectively) agree generally with similar sets of points determined analytically which showed this slope to be -0.4[3] and also those determined from other groups of natural basins.[4] The center or intermediate group of points in Fig. 5 with an average population density of 2,450, when fitted by least squares showed a steeper slope (-.53). One explanation for this is that the development of the drainage system is probably not always a gradual process related directly to gradual changes in population density but it may lag behind or jump ahead of population increases thus creating anomolous situations. Because of the fact that there is so much evidence that the slope of this line should be about -0.4, the least squares method was applied again to find the best fit with a slope of -0.4. The five curves for which this was done, shown in Fig. 6, include the three groups of points from Fig. 5 plus a fourth and fifth line for the low population basins (average population density is 610) and intermediate population basins (average population density is 2,038) in Southeastern Michigan. Another approach to the correlations shown in Figures 5 and 6 is to develop a new parameter q_{pA_o} which is defined by the following equation

$$q_{pA_o} = \left(\frac{A}{A_o}\right)^{.40} q_{pA}$$

in which q_{pA} is the unit hydrograph peak for any area A in cfs per sq. mi. per in. and q_{pA_o} is the corresponding unit hydrograph peak for a selected base area A_o. The value q_{pA_o} is obtained for any basin from its value of q_{pA} by following along a line such as those in Fig. 6 to be base area A_o. In this case A_o was chosen as 10 sq. mi. but any other size of basin could be selected for this purpose. The resulting points have the area parameter effectively eliminated and may then be plotted against population density as shown in Fig. 7. It will be seen that the trend is quite clear and that the basins from Southeastern Michigan follow a slightly different trend than those from other areas. Also shown by x's in Figure 7 are the low and intermediate population density points for Michigan determined from Figure 6. The high population point determined from Figure 6 is also shown. Fig. 7 was used to locate intermediate lines such as those in Fig.[15] for the practical application of this information shown in the final section.

Similar analyses were made for the time parameters such as period of rise and lag. Only the final results are shown in the last section of this report.

It is recognized that population density alone cannot completely express the degree of urbanization. Industrial areas, for example, act as urbanized areas, but do not have a high population. Work presently underway suggests that an important parameter related to urbanization is the nature of the drainage network. This has top priority for research in the immediate future. The fact that two groups of watersheds (Dallas and Austin) have different characteristics than the others studied may provide a valuable insight into other important factors affecting the unit hydrograph. It is fully expected that as the basic research progresses it will be possible to make further refinements in the procedure.

A Flood Prediction Procedure

The unit hydrograph-infiltration capacity method is based on the idea that a rain of any selected frequency can be transposed onto a drainage basin and the storm hydrograph that would result from that rain can be constructed. A brief outline of the steps necessary to accomplish this will be presented prior to presenting the final curves for predicting flood peaks in Southeastern Michigan. Background information closely related to this outline is presented in the previous portion of this report.

Rainfall Frequency and Snow Melt

Because few rain gages have records for periods longer than 70 years, records from a single rain gage cannot be used to predict rains of rare frequency such as 25, 50, 100 or 200 years with acceptable accuracy. However, for meteorologically homogeneous areas the station-year method[4] of combining records from a number of rain gages can be used to effectively extend the period of records. For Southeastern Michigan about 2000 station years of records have been analyzed. As shown by Fig. 2 the infiltration capacity varies so much from winter to summer that by treating the two seasons separately a better degree of accuracy is obtained. The summer season contains the months of June through September while the months of October through May are called the winter season. The precipitation frequency curves were also developed separately for the two seasons as shown in Fig. 8.

The precipitation records given by the U. S. Weather

Bureau do not differentiate between snow fall and rainfall. This makes the winter values in error in so far as flood prediction is concerned. Furthermore, the recorded rainfall values do not include the snow melt that often takes place concurrently with winter rains. It was therefore decided that it would be essential to the accuracy of flood prediction to study all winter precipitation results and eliminate snow fall from the list of values and include snow melt whenever it occurred either separately or in conjunction with a rain. It is believed that no such analysis has ever been made up to this time. As a result, the winter curve in Fig. 8 is that station-year curve that includes not only all rains of record but all snow melt events either as separate events or combined with rain that occurred at the same time. The frequency studies described above are for 24 hour rainfall plus snow melt. For non-recording stations the values were corrected to include additional rain recorded on adjacent days but which were probably part of the maximum 24 hour rainfall.[4]

The frequency studies were all made with 24 hour rainfall because many more 24 hour rainfall data are available than for shorter durations. For the smaller basins rainfall of much shorter duration was needed. Use was made of U. S. Weather Bureau studies of shorter duration rains. It was found that the ratio of precipitation occurring during any shorter duration such as one hour to the 24 hour precipitation of the same frequency was relatively constant. It was also found that the ratio did not depend upon the rainfall season.[5,6,7,8] Fig. 9 shows the curve derived in this manner. On this basis it was possible to derive rains of any desired frequency for shorter durations from the 24 hour values determined in the manner previously described.

For small drainage basins it was necessary to derive time intensity patterns or hyetographs or typical rain storms of various frequencies broken down into time intervals as small as 30 minutes. The order in which the portions of the rain are arranged is based on the analysis of many rains. The most intense portion is placed before the middle of the total duration. A detailed study has been carried out by Tholin and Keiffer[9] and their typical pattern is showin in Fig 10. Typical rain storms of selected frequencies were developed in as much detail as necessary by making use of Figures 9 and 10. The rain used as an example in Fig. 10 is a 3 inch 24 hour rain. It has the required characteristic that all portions of it, as for example, the maximum 30 minutes or the maximum two hours, have the same frequency.

All rainfall frequency data discussed so far are obtained for point rainfall, that is, from individual rain gages. For flood prediction average precipitations on the drainage basin is needed. Therefore it is necessary to relate point rainfall to average rainfall on various areas of various sizes for the same frequency. The U. S. Weather Bureau[5] has derived ratios

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(5) U.S. Weather Bureau, No. 29, Part 4, (6) U.S. Weather Bureau, No. 40, (7) U.S. Weather Bureau, No. 25, (8) U.S. Weather Bureau, Tech. Paper No. 2 (revised 1963).

between point rainfall and average rainfall of the same frequency for basins up to 400 square miles. These have been used to derive the curves for small areas shown in Fig. 11 and extended to larger areas using area-depth curves as a guide.[4]

Infiltration Capacity

This has been discussed earlier in this report when Fig. 2 was presented and also in the previous section on precipitation frequency. Generally the values for the two seasons shown in Fig. 2 can be used with reasonable accuracy at least in the region south of Flint and east of Ann Arbor. The values in Fig. 2 were based on the analysis of about 800 stream rises from this region. Whenever time and money and streamflow records are available it would be better to derive values of infiltration capacity for the river or area in which floods are being predicted because there are some variations even in Southeastern Michigan. An orderly program of rainfall and runoff analysis for determining infiltration capacities for areas throughout the state selected on the basis of soil or geologic type would extend the use of this method to all of Michigan.

Retention

The retention is the portion of the rain which is permanently retained as interception by vegetation or in depressions and is eventually evaporated or infiltrated. A method of estimating this quantity was developed[1] and for Southeastern Michigan the average values were found to be 0.09 inches in winter and 0.15 inches in summer.

Impermeable Areas

A method of estimating the hydrologically significant impermeable area (HSIA) from rainfall and runoff analyses was developed.[1] This area is that portion of the drainage basin which always contributes 100 per cent of its precipitation, less retention, to surface runoff. It varies with population density. The following equation relates HSIA expressed in per cent of the total drainage to population density in 1,000's per square mile for Southeastern Michigan.

$$\text{HSIA } (\%) = 1.38\ P_d \text{ (1,000's of persons per sq. mi.)}$$

Base Flow

After predicting the surface runoff hydrograph or the peak rate of surface runoff discharge of such a hydrograph the groundwater discharge which is expected at that time must be added to obtain the total discharge. This is illustrated in Fig. 1 where a line b_1b_2, called the base line, which separates the groundwater discharge from surface runoff is shown. The base flow is usually small, perhaps only 5 per cent of the total peak flow during a large storm. Therefore an error in estimating the groundwater's contribution does not introduce a serious error in the predicted flood peak.

The average base flows for basins in Southeastern Michigan were determined and plotted against area. It was found that base flow is a linear function of area for this region and that the relation can be expressed quite well by the following equation.

$$Q_{base} \text{ (cfs)} = 0.6\ A \text{(Sq.mi.)}$$

Unit Hydrograph

After determining the volume of surface runoff from the rainfall-infiltration studies this volume is distributed in time by the unit hydrograph. The assumptions used in deriving and applying the unit hydrograph to a rainfall or a series of complex rains has been stated in the first section of this report. It was found that the unit hydrograph shape may be described by the following parameters which are defined in Fig. 14; * q_{PA}, t_r, T_r, W_{75}, W_{50}, W_{25} and W_0. Because t_r was found to be a close approximation of lag, for basins as large as 80 square miles it can be used as the lag of the basin. The lag was defined earlier but its definition may also be seen in Fig. 14. Therefore t_r also becomes the critical duration of precipitation excess which by definition produces a hydrograph shape independent of duration of precipitation excess as long as this duration is less than the critical duration.

The working curves for the above unit hydrograph parameters were derived from a similar set of analyses as was described earlier in this report for the curves for the unit hydrograph peaks. These final curves are presented in Figures 15, 16 , 17 , 18, 19, 20 and 21.

With the information described above, a flood peak magnitude and its corresponding frequency can now be determined. A direct determination of a flood of a given frequency cannot be made for reasons which will become apparent. The procedure used was as follows :

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* There are no figures numbered 12 & 13.

1.) For any given magnitude of 24 hour rainfall plus snowmelt a typical hyetograph was developed by the use of Figures 9, 10, and 11.

2.) The average infiltration capacity was then superimposed on the hyetograph as illustrated in Fig. 10 where the summer infiltration capacity is used as an example. The hatched area then becomes the precipitation excess from the pervious portion of the drainage basin. Also shown on this figure is the summer retention of 0.15 inches. (The winter retention was 0.09 inches.)

3.) The total area under the hyetograph less a retention of 0.05 inches is the surface runoff from the impermeable area (HSIA). The value of HSIA is taken from the equation on p. 22.

4.) For large basins the lag, or t_r, is usually greater than the total period of rainfall excess as shown in Fig. 10. For smaller basins the lag may be smaller than the total period of rainfall excess and the period of excess must be divided into a series of time increments equal to the lag. Then the rainfall excess is determined for each selected time increment.

5.) The average unit hydrograph characteristics are obtained from curves 15, 16, 17, 18, 19, 20 and 21. After sketching the unit hydrograph a final check should be made on the volume. If the volume was not equal to 1.0 inch a slight adjustment was made. Terms used in the graphs are defined in Fig. 14.

6.) The unit hydrograph is then applied to successive increments of precipitation excess as determined in (4) and the hydrographs are combined. This process is referred to as convolution.

7.) Finally the total runoff hydrograph is obtained by adding the typical groundwater flow from the equation for Q_{base} on p. 22.

8.) This procedure is carried out for rains of various sizes and frequencies and a curve relating peak discharge to frequency is plotted as shown in Fig. 22. Figure 22 shows both a winter and a summer curve.

9.) A curve for total frequency is then obtained by combining the summer and winter curves and is shown in Fig. 22. This is done by adding the probabilities for floods of selected magnitudes. The probabilities are obtained by taking the reciprocals of the return periods or frequencies. The operation is shown by the following equation

$$\frac{1}{T_T} = \frac{1}{T_W} + \frac{1}{T_S}$$

and

$$T_T = \frac{1}{\frac{1}{T_W} + \frac{1}{T_S}}$$

in which T_W, T_S and T_T are the winter, summer and total return intervals respectively.

10.) The final step was to graph the peak discharge versus area for any given frequency and population density. Thus these final design curves appear as Figures 23, 24, 25, 26 and 27. The method of using these curves is described in Part II.

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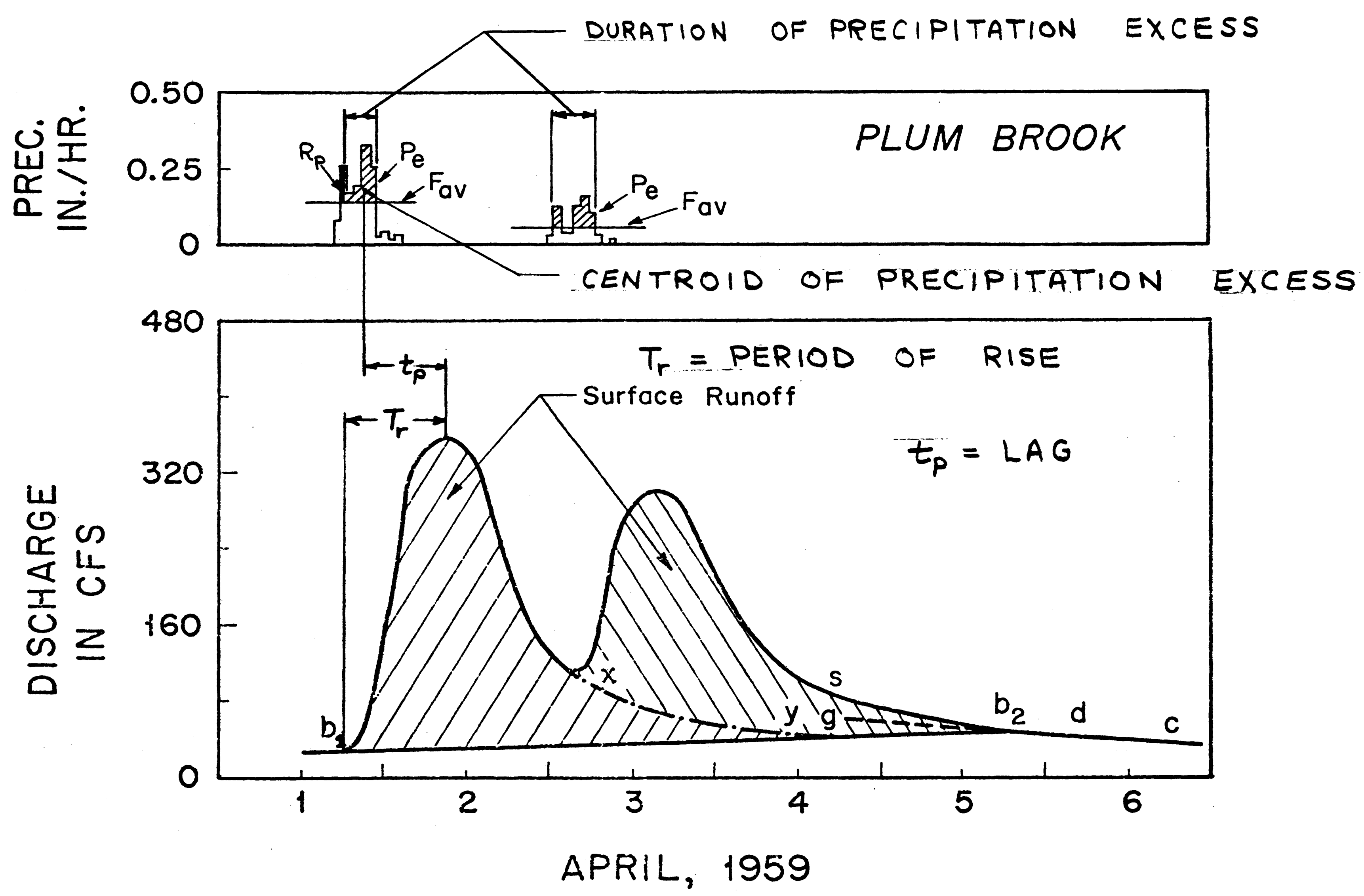


DURATION OF PRECIPITATION EXCESS
CENTROID OF PRECIPITATION EXCESS
PLUM BROOK
PREC. IN./HR.
0.50
0.25
0
R_R
P_e
F_{av}
P_e
F_{av}
DISCHARGE IN CFS
480
320
160
0
T_r = PERIOD OF RISE
Surface Runoff
t_p = LAG
t_p
T_r
b_1
x
y
g
s
b_2
d
c
1
2
3
4
5
6
APRIL, 1959

FIGURE 1

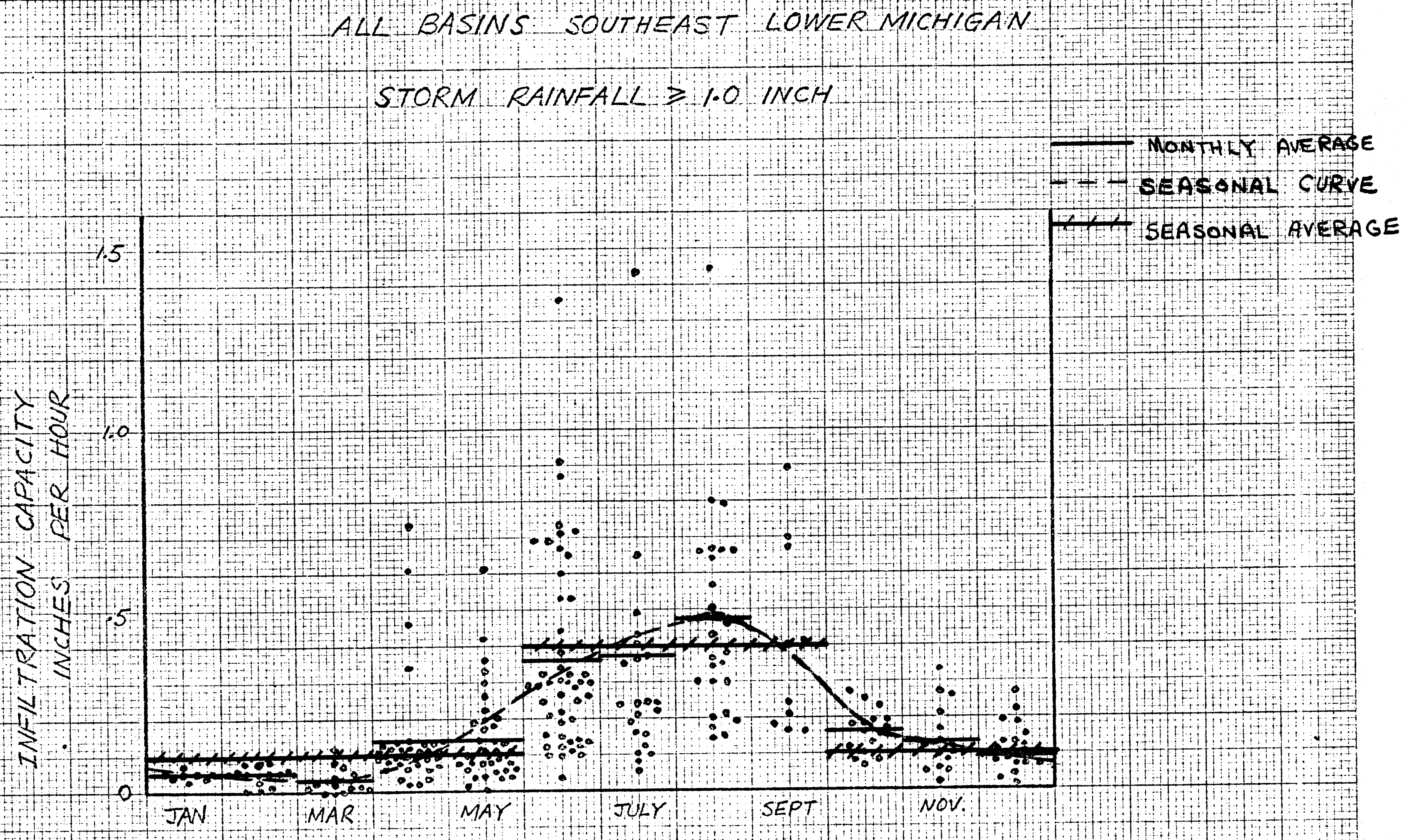

ALL BASINS SOUTHEAST LOWER MICHIGAN
STORM RAINFALL ≥ 1.0 INCH
MONTHLY AVERAGE
SEASONAL CURVE
SEASONAL AVERAGE
INFILTRATION CAPACITY
INCHES PER HOUR
1.5
1.0
.5
0
JAN
MAR
MAY
JULY
SEPT
NOV.

FIGURE 2

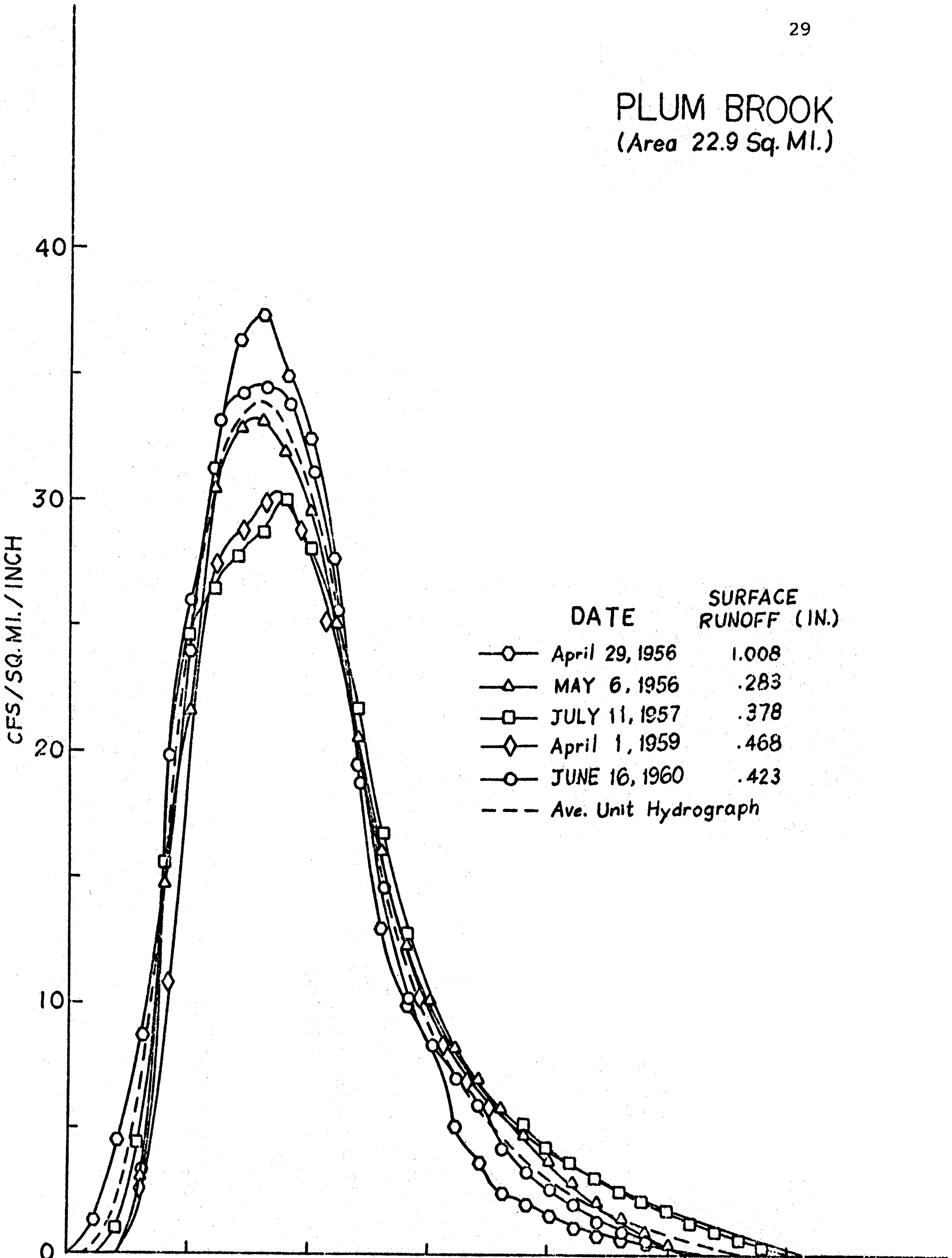
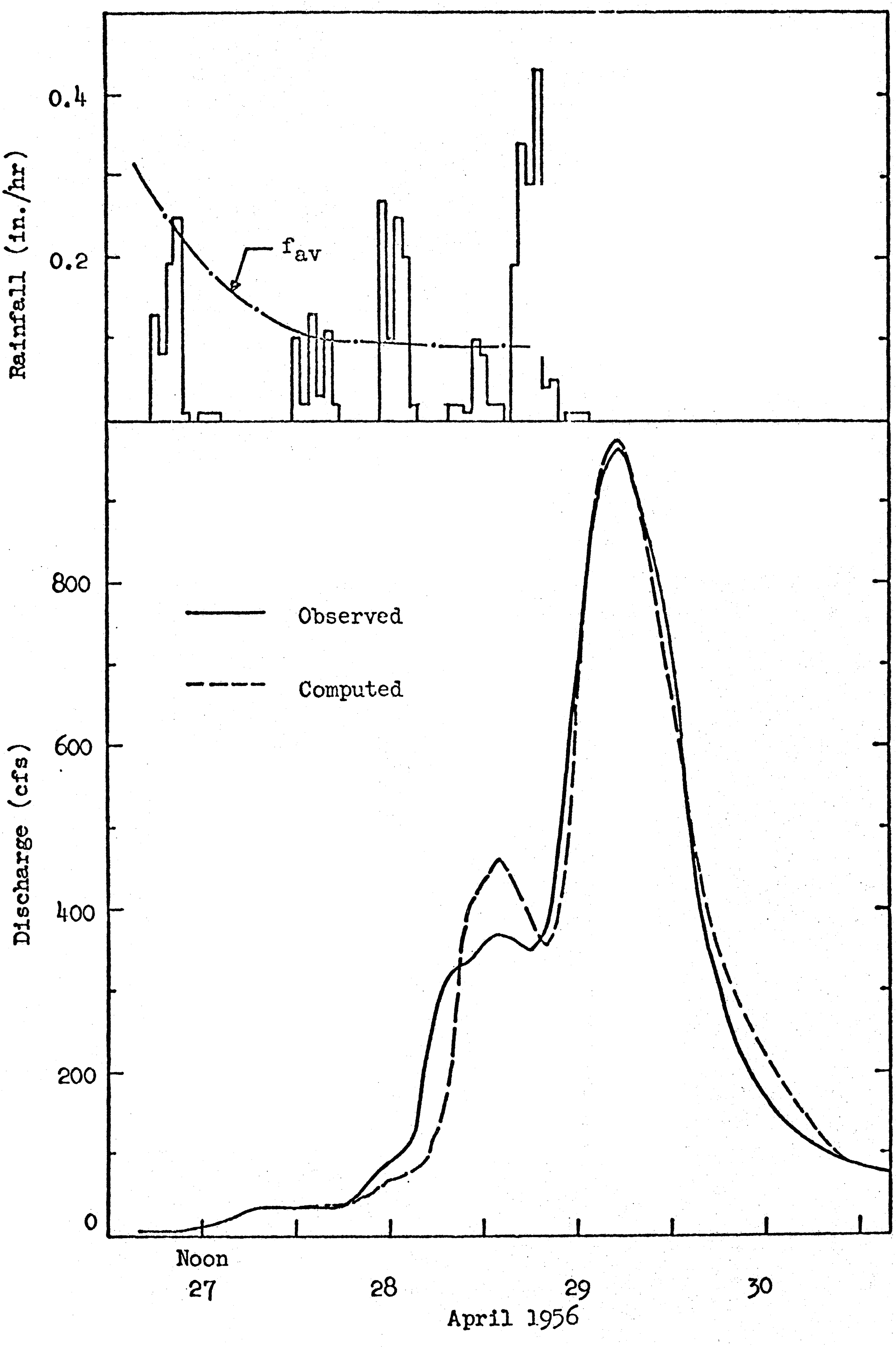
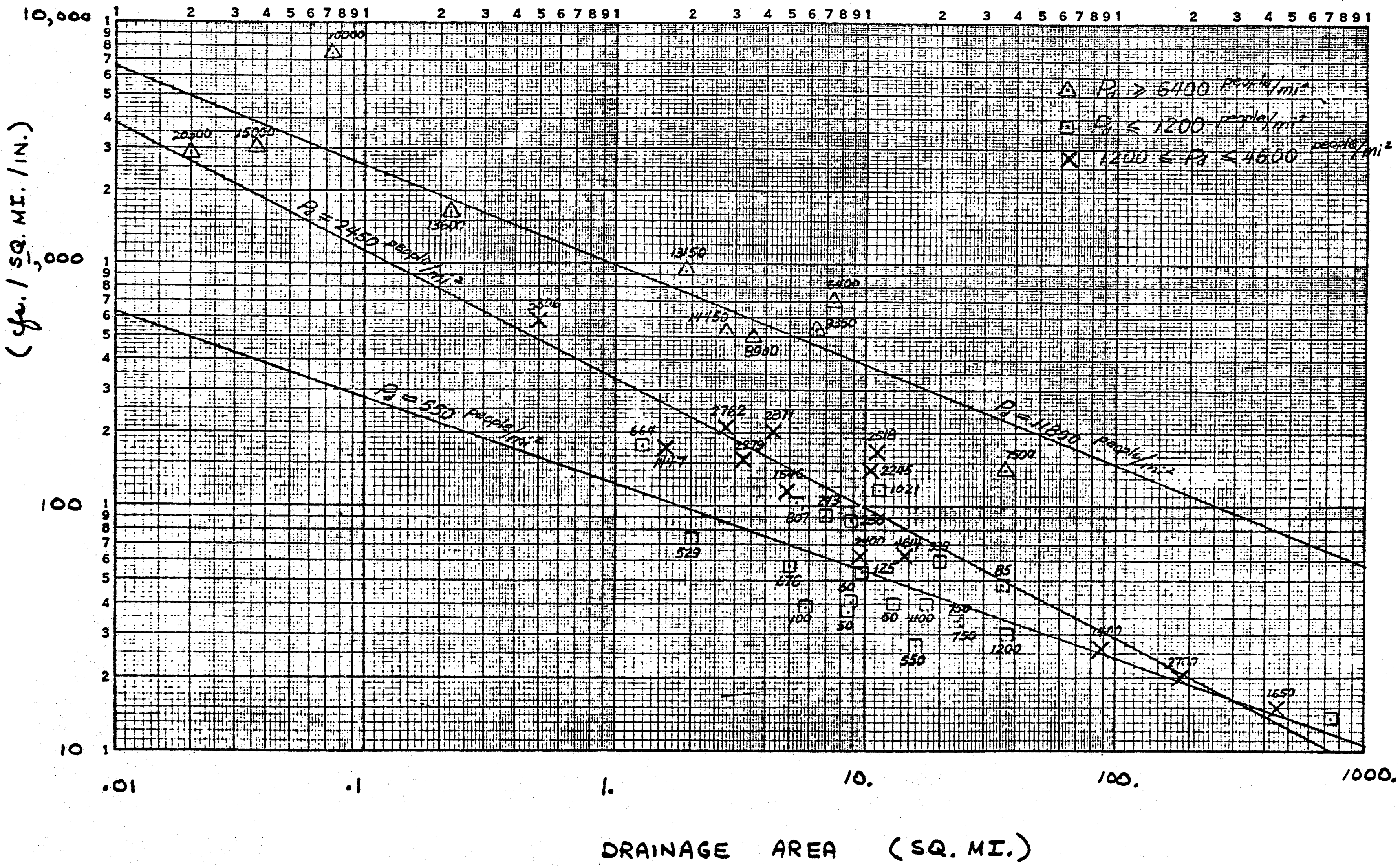

PLUM BROOK
(Area 22.9 Sq. MI.)
CFS/SQ. MI./INCH
0
10
20
30
40
DATE
SURFACE RUNOFF (IN.)
April 29, 1956 1.008
MAY 6, 1956 .283
JULY 11, 1957 .378
April 1, 1959 .468
JUNE 16, 1960 .423
Ave. Unit Hydrograph
10
20
30
40
50
60
70
TIME (Hrs.)

FIGURE 3



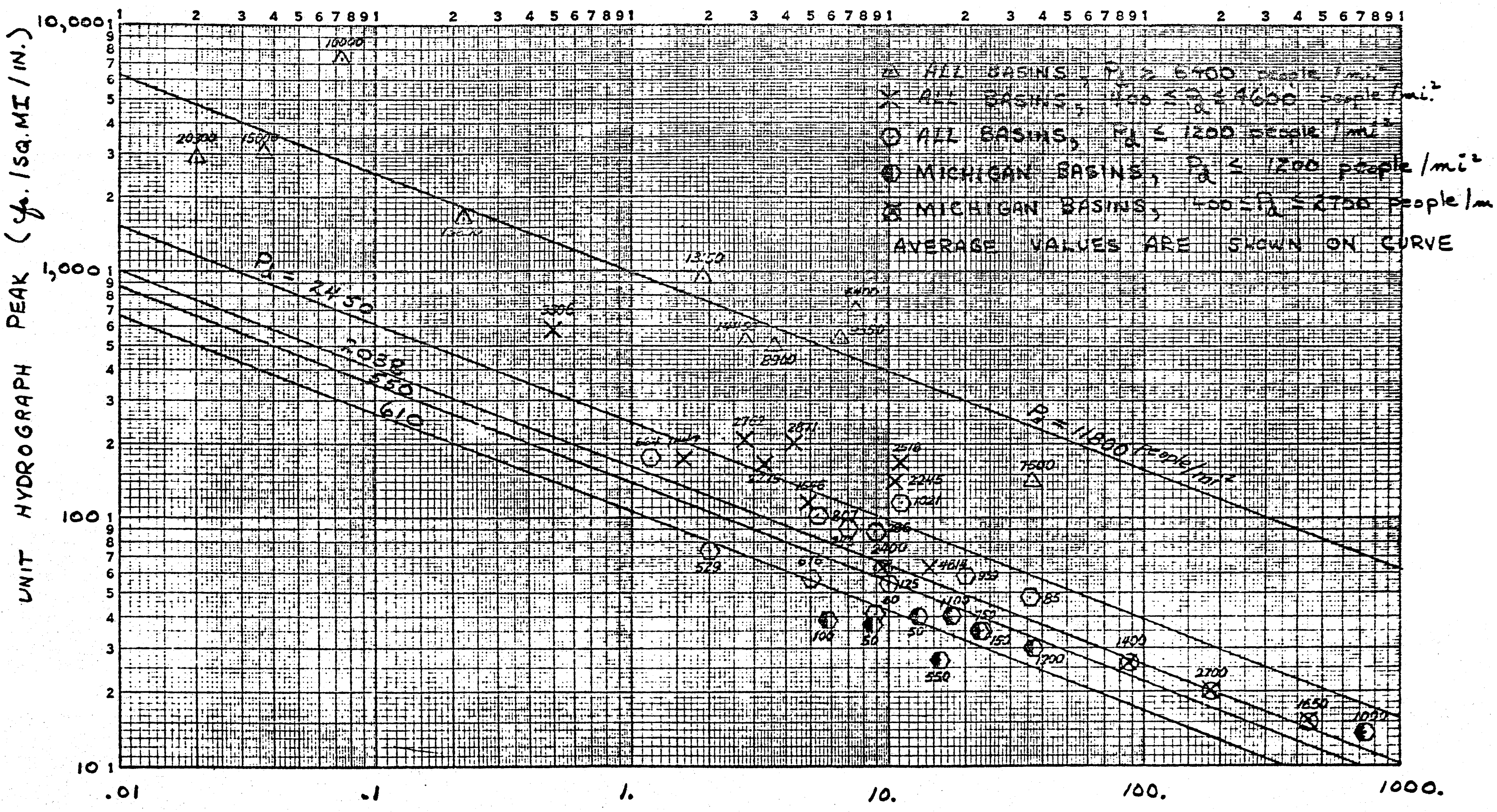
Reproduction of flood of April 27-30, 1956 for Plum Brook at Utica (A = 22.9 sq mi).

FIGURE 4



P_d shown by points

FIGURE 5



P_d shown by points

FIGURE 6

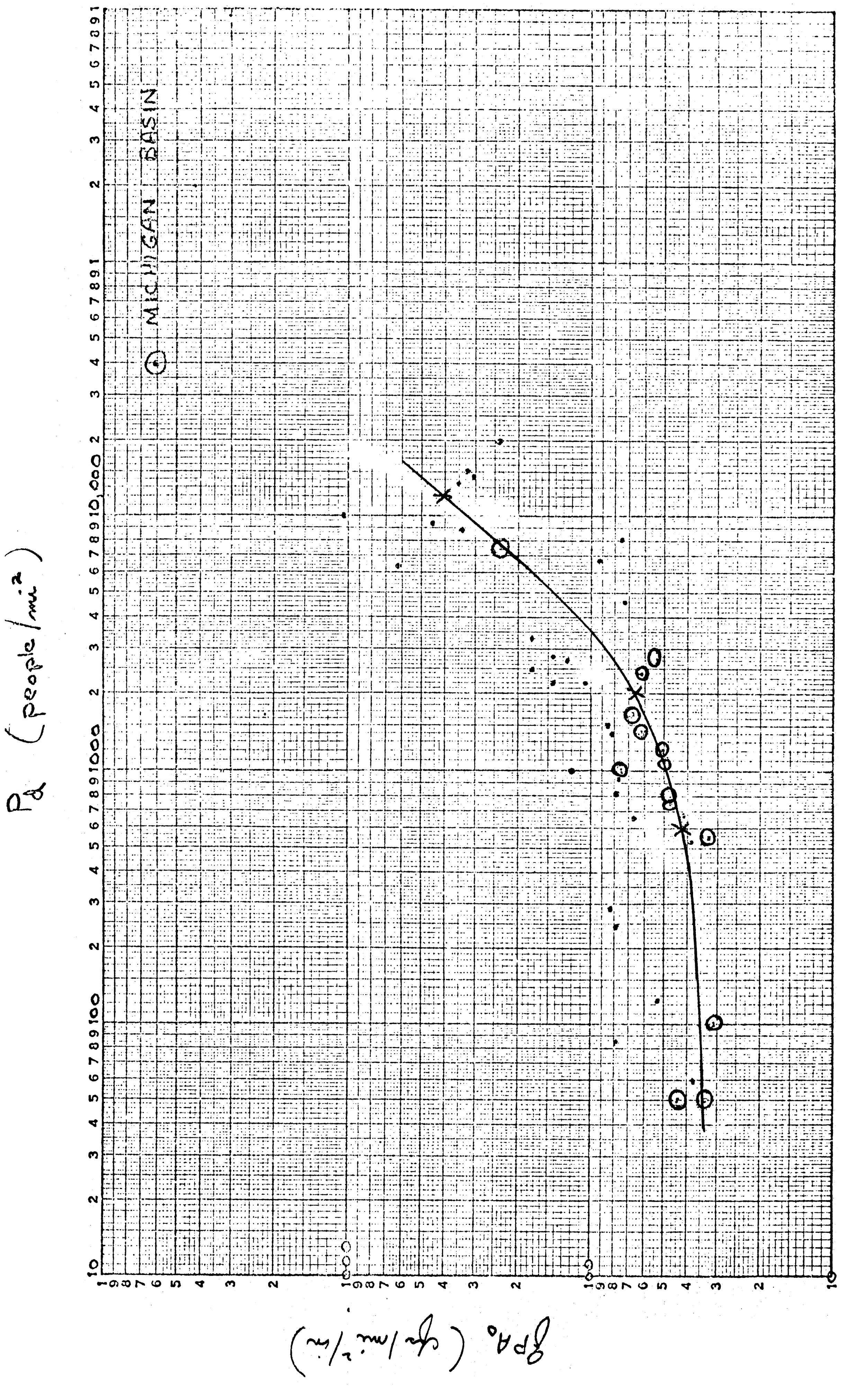


FIGURE 7

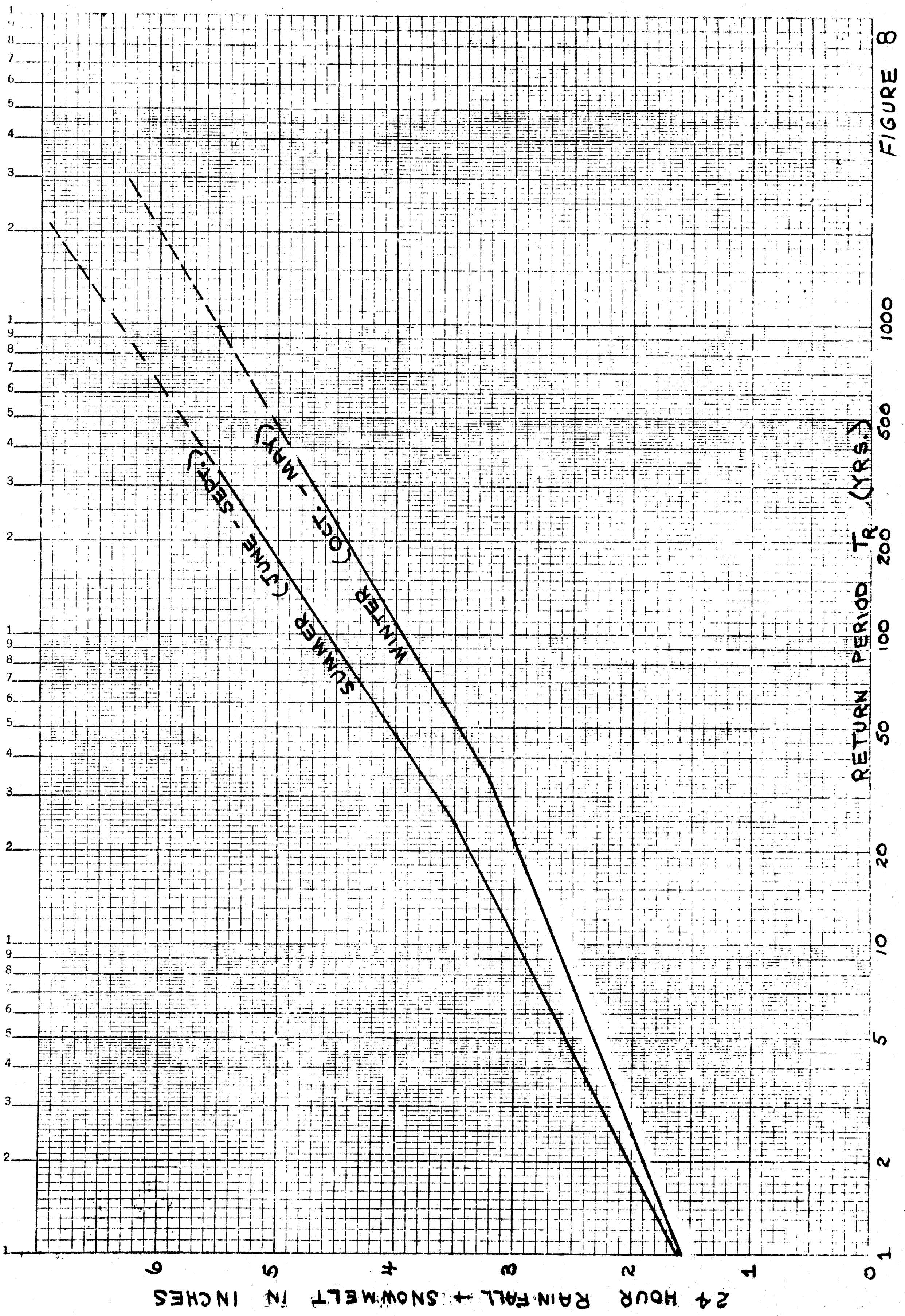


FIGURE 8
RETURN PERIOD T_R (YRS.)
1
2
5
10
20
50
100
200
500
1000
24 HOUR RAINFALL + SNOWMELT IN INCHES
0
1
2
3
4
5
6
WINTER (OCT. - MAY)
SUMMER (JUNE - SEPT.)

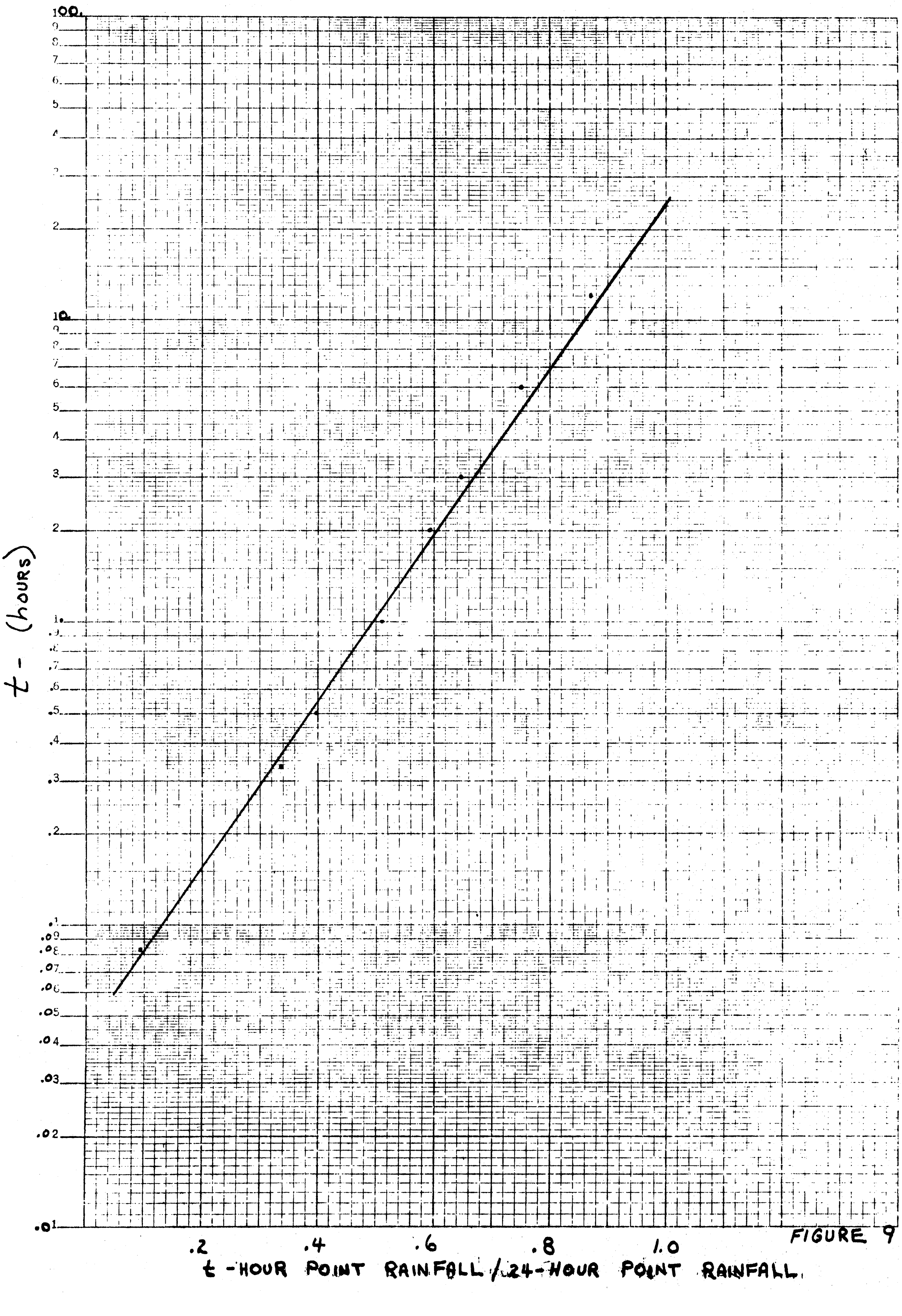
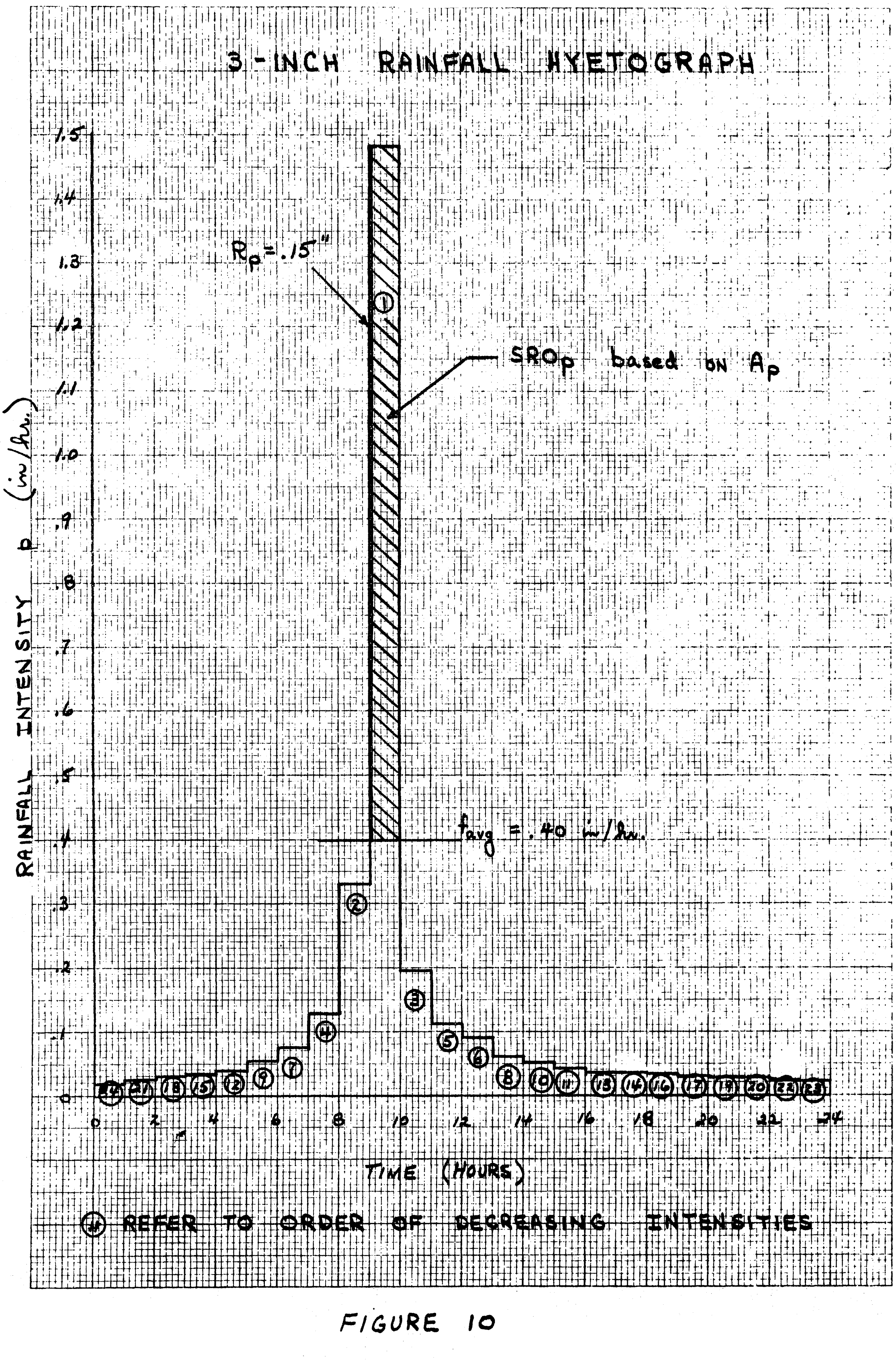

t - (hours)
t -HOUR POINT RAINFALL / 24-HOUR POINT RAINFALL
.2
.4
.6
.8
1.0
100.
10.
1.
.1
.01

FIGURE 9



3-INCH RAINFALL HYETOGRAPH
$R_p = .15''$
SRO_p based on A_p
$f_{avg} = .40$ in/hr.
RAINFALL INTENSITY p (in/hr.)
TIME (HOURS)
(#) REFER TO ORDER OF DECREASING INTENSITIES

FIGURE 10

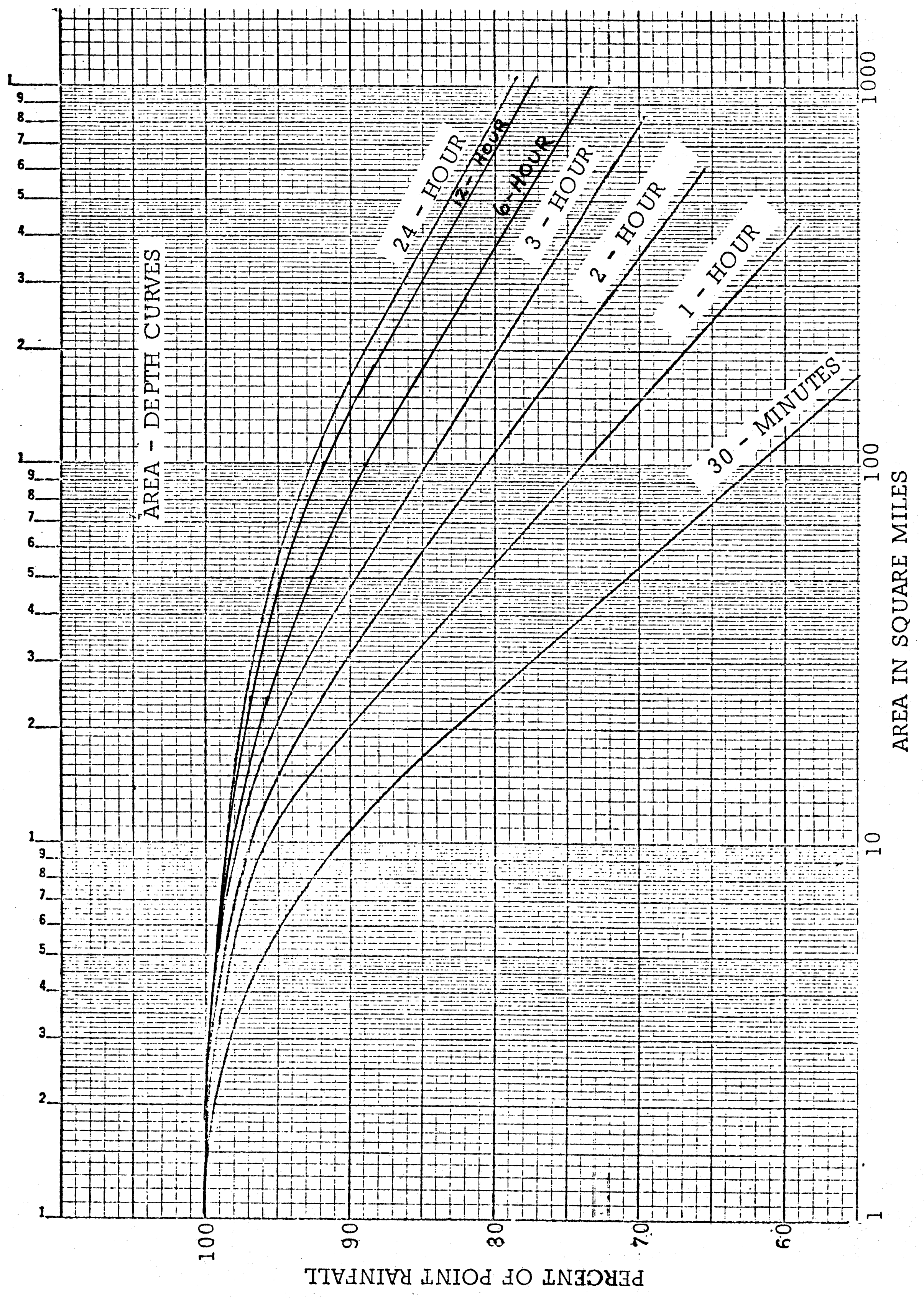

AREA - DEPTH CURVES
24 - HOUR
12 - HOUR
6-HOUR
3 - HOUR
2 - HOUR
1 - HOUR
30 - MINUTES
AREA IN SQUARE MILES
1
10
100
1000
PERCENT OF POINT RAINFALL
100
90
80
70
60

FIGURE 11

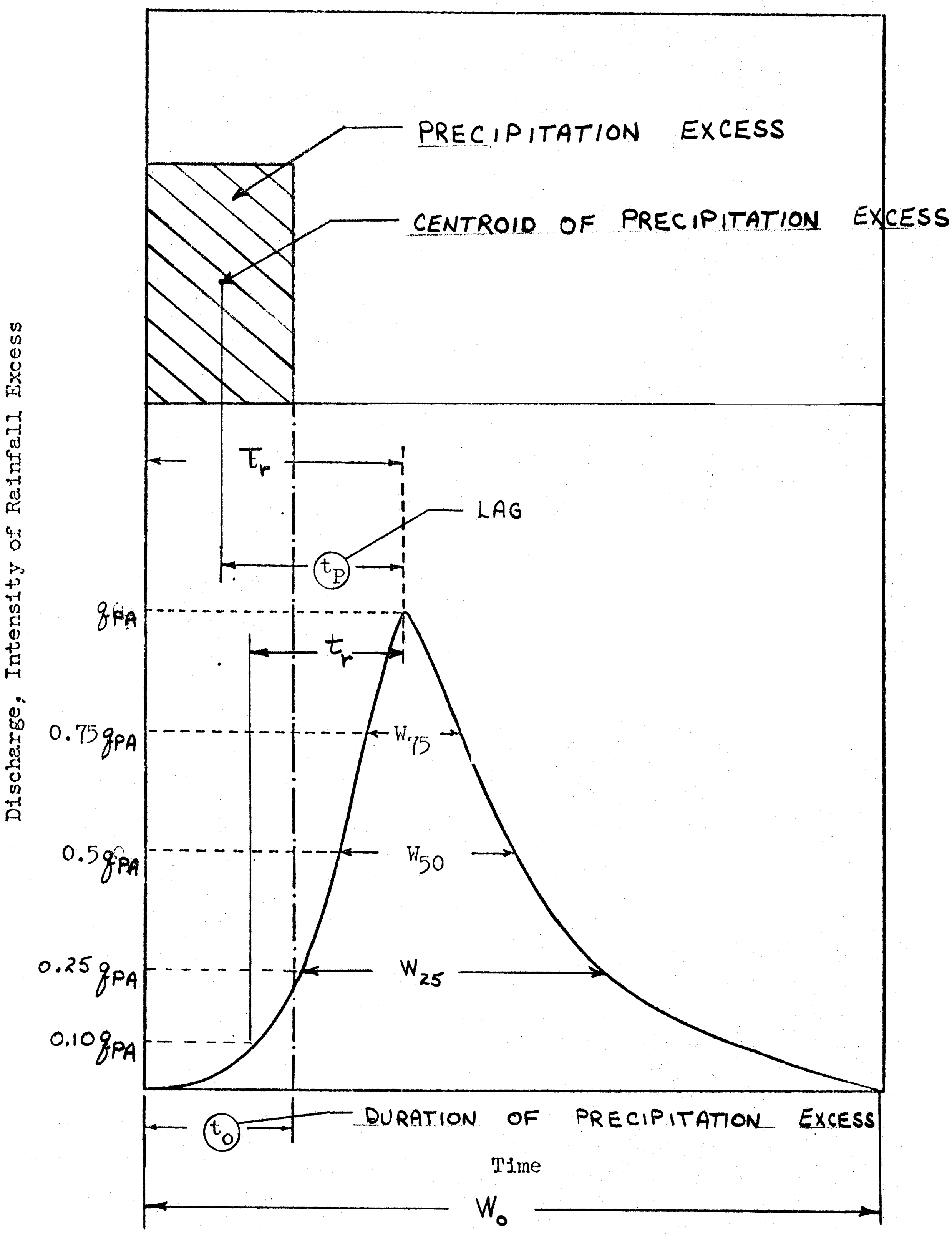
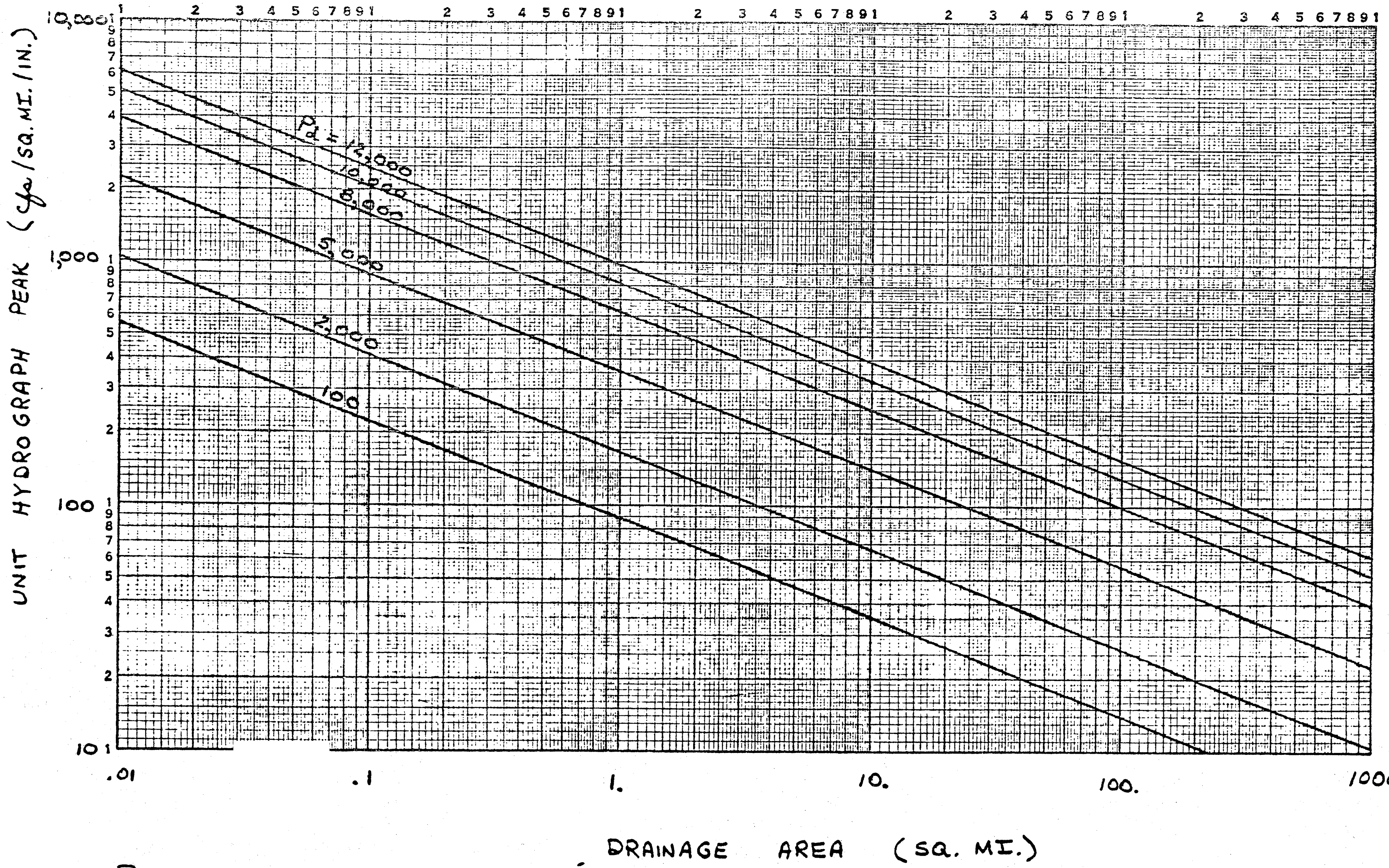
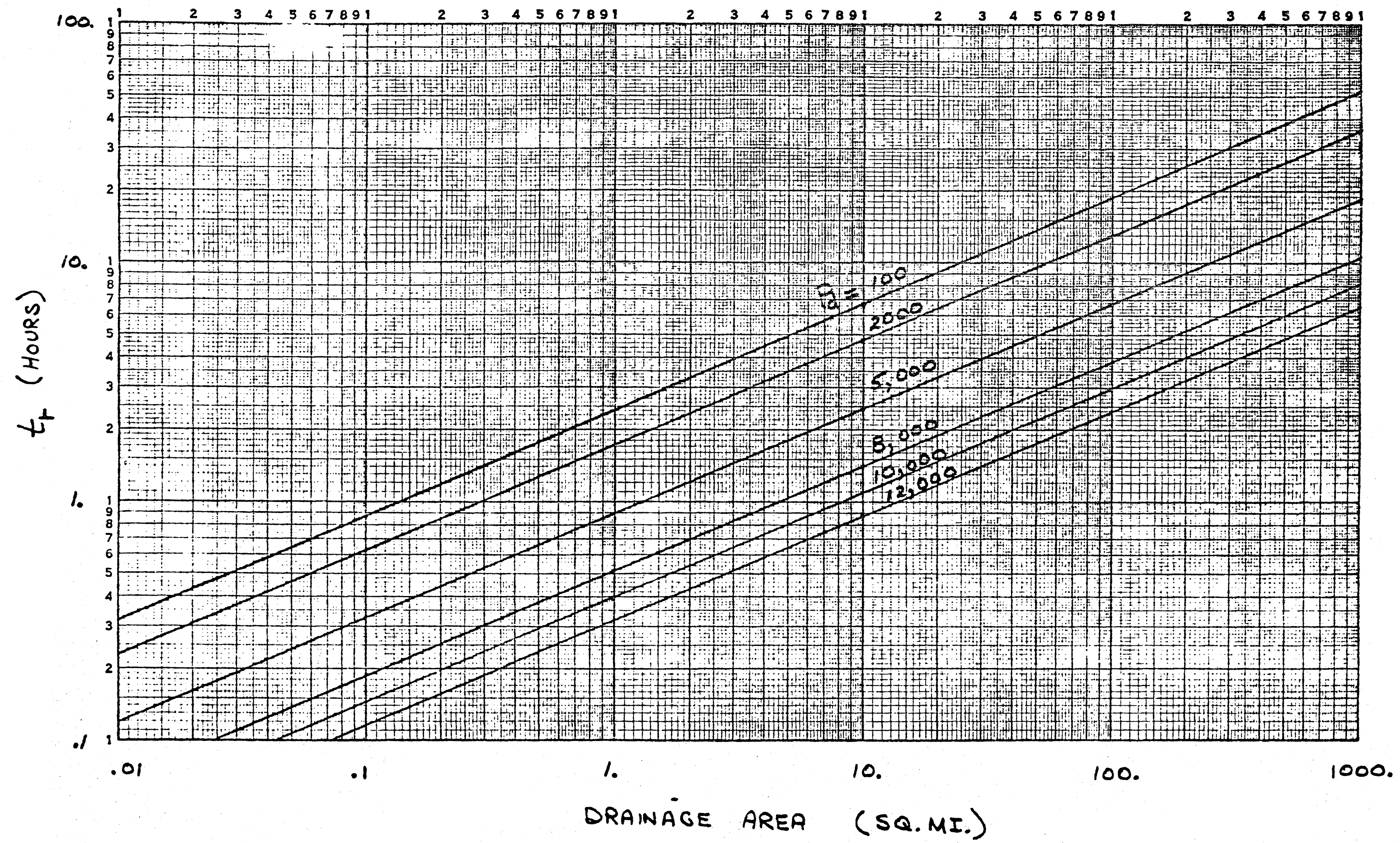

PRECIPITATION EXCESS
CENTROID OF PRECIPITATION EXCESS
Discharge, Intensity of Rainfall Excess
LAG
t_P
q_{PA}
t_r
0.75 q_{PA}
W_{75}
0.5 q_{PA}
W_{50}
0.25 q_{PA}
W_{25}
0.10 q_{PA}
t_o
DURATION OF PRECIPITATION EXCESS
Time
W_o

FIGURE 14



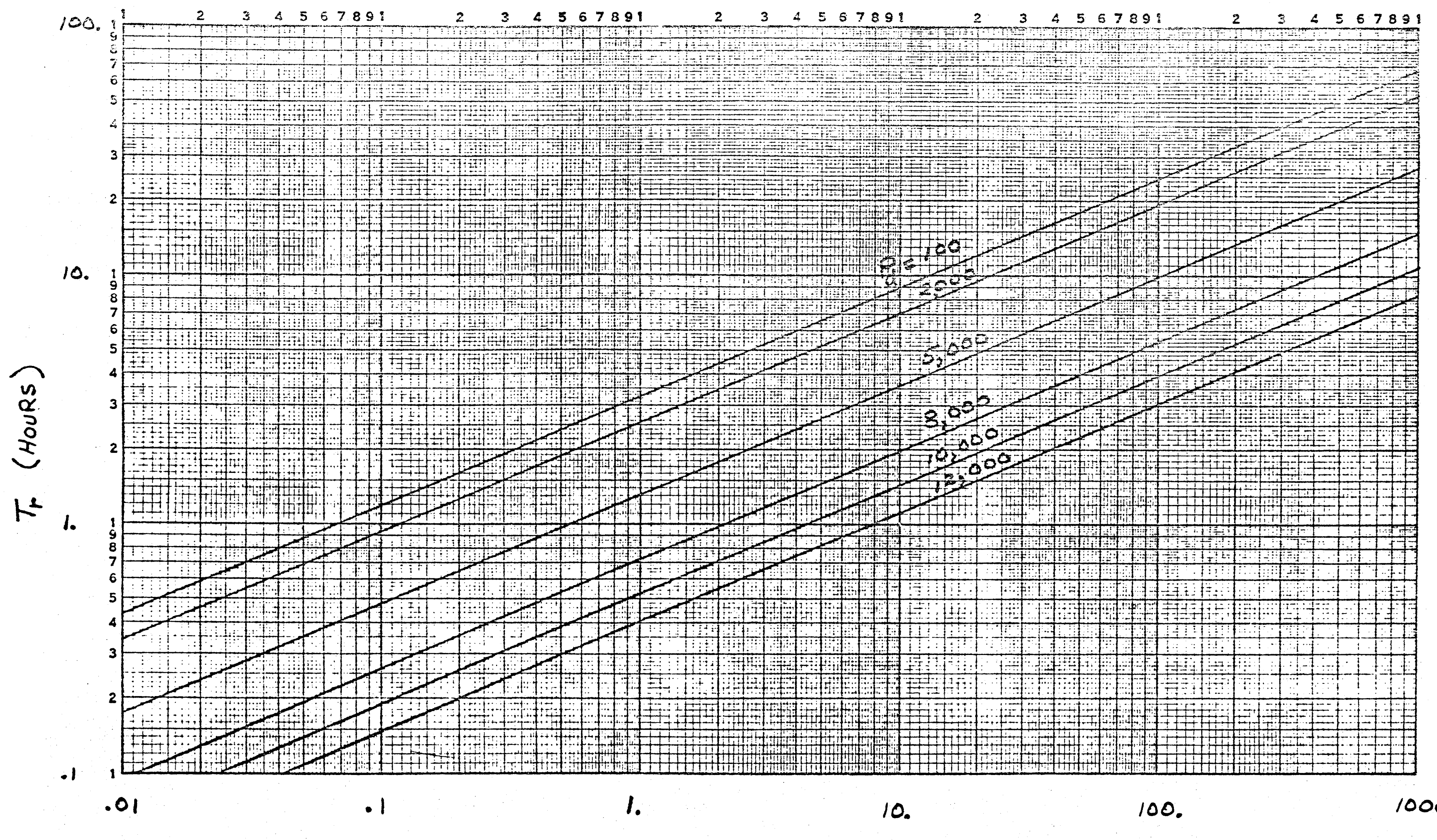
P_d = Population Density (people/SQ.MI.)

FIGURE 15



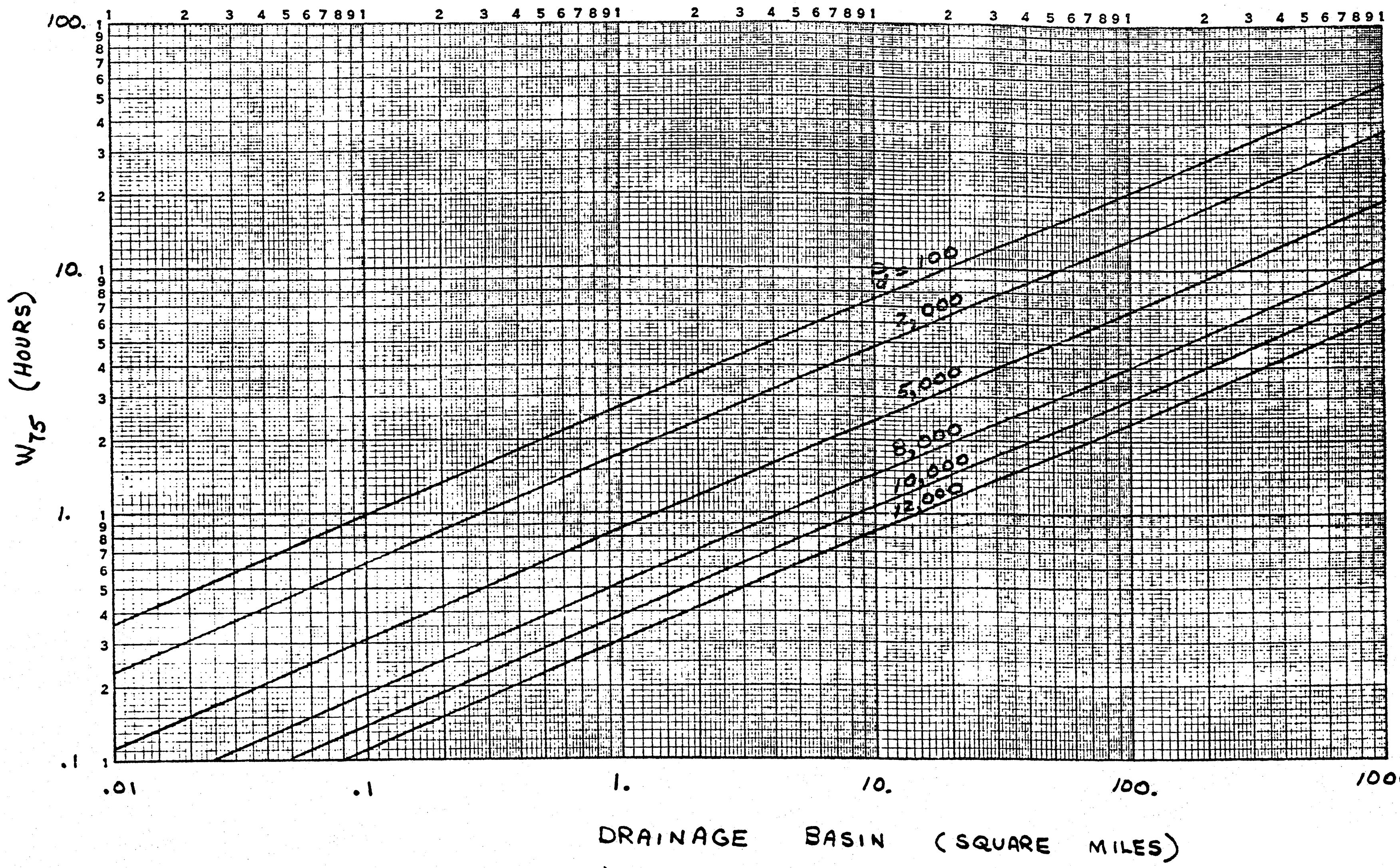
P_d = population density (people / mi^2)

FIGURE 16



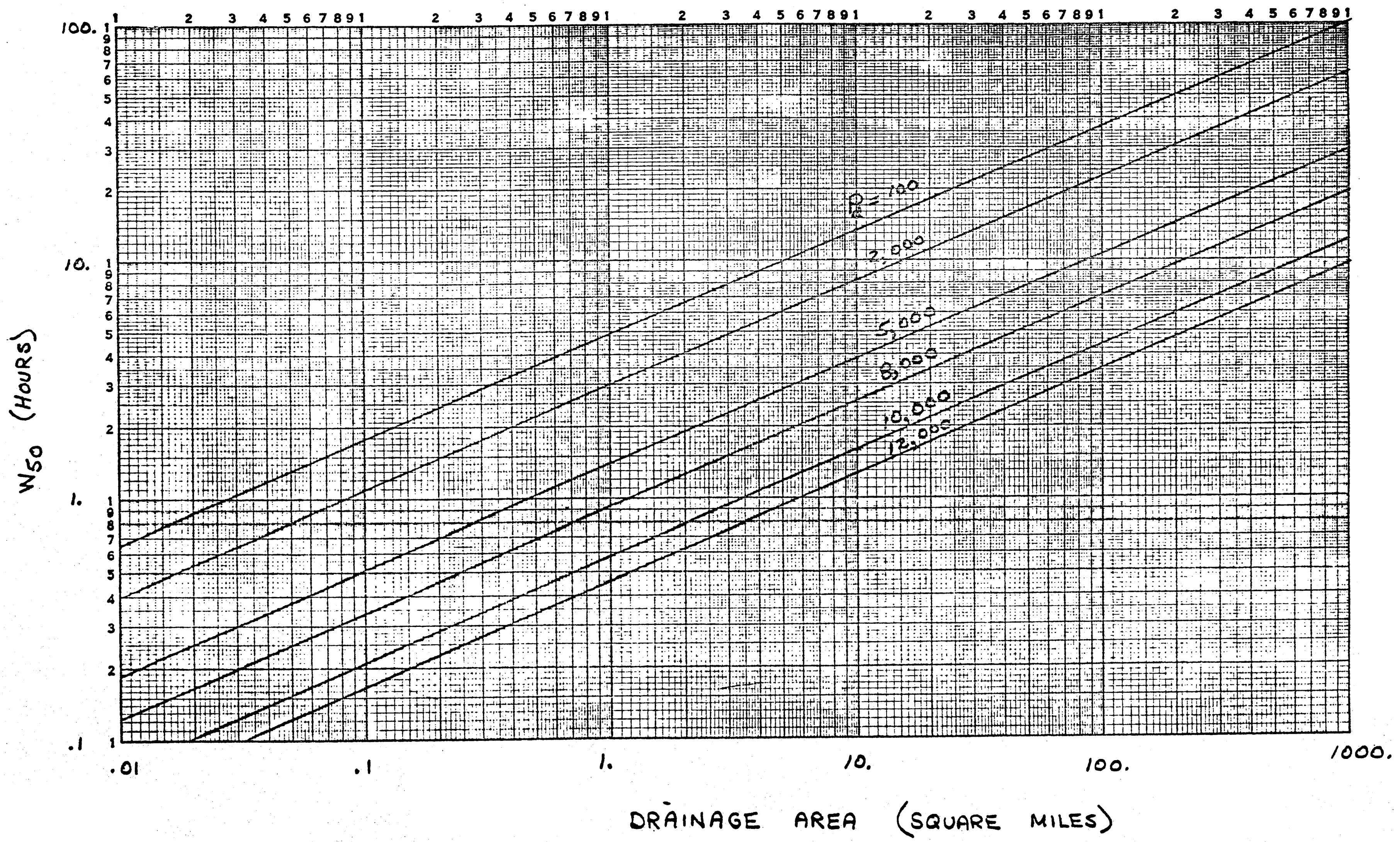
P_d = population density (people / mi^2)

FIGURE 17



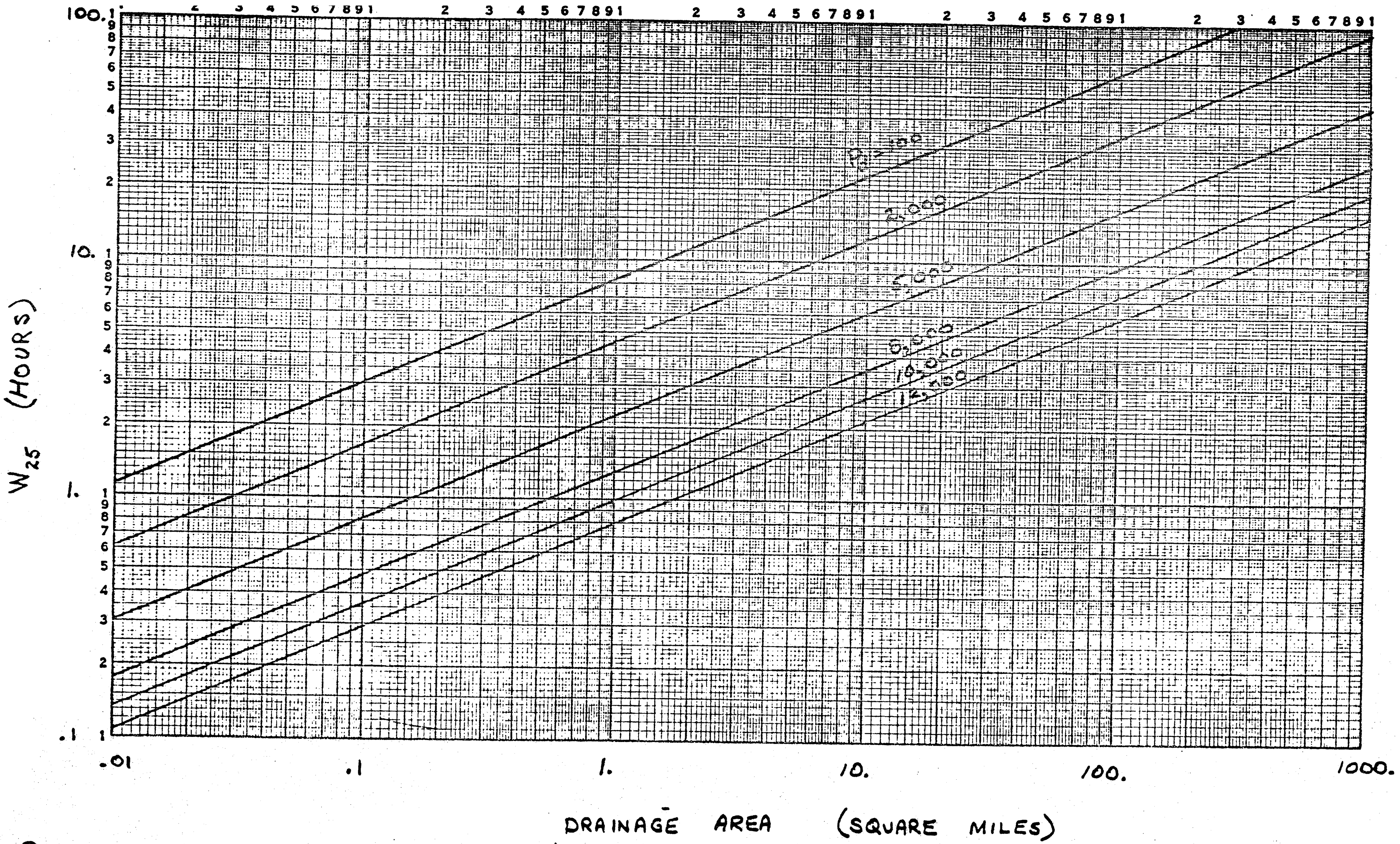
P_d = Population Density (people / mi^2)

FIGURE 18



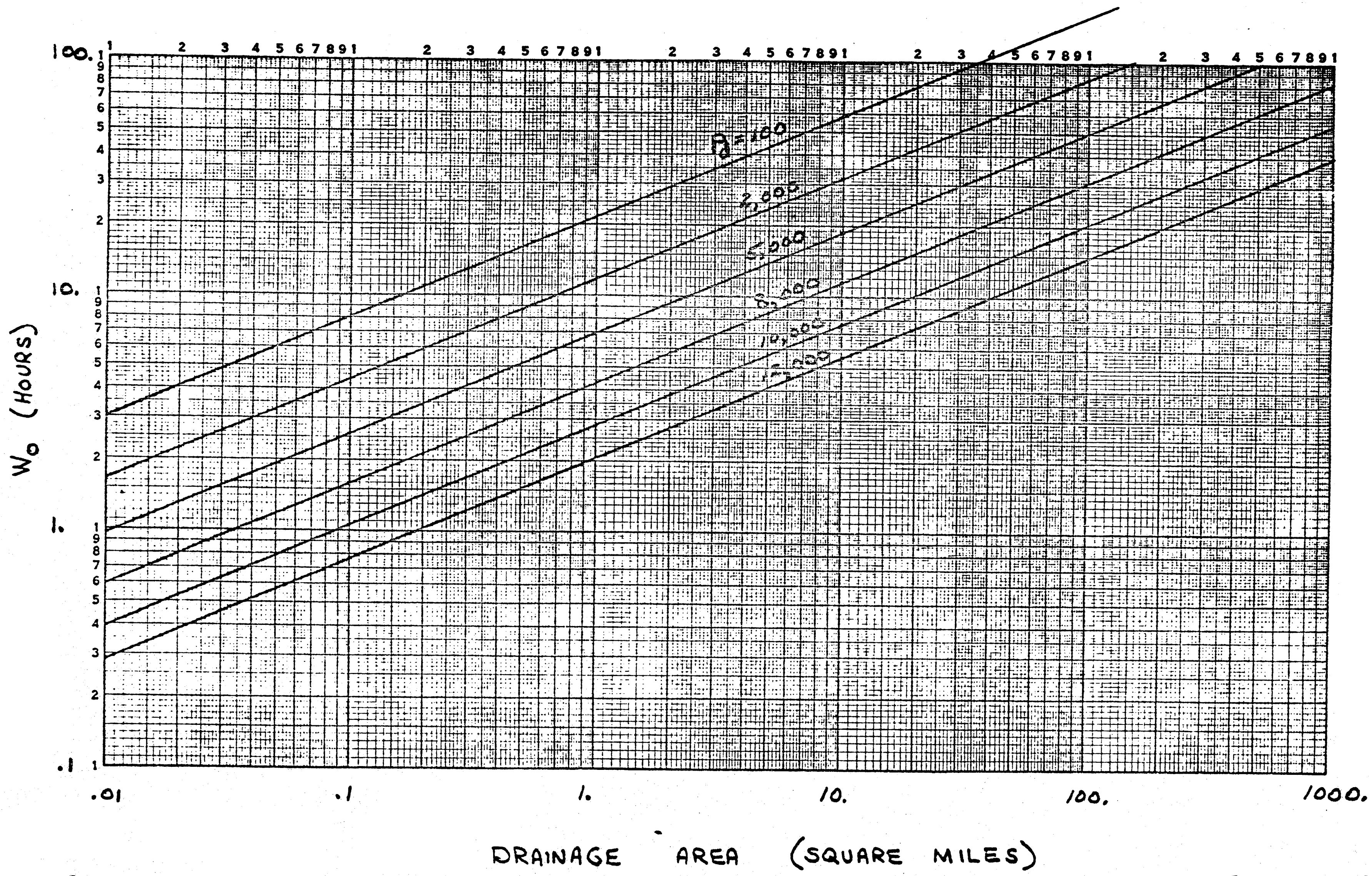
P_d = Population Density (people / mi^2)

FIGURE 19



P_d = Population Density (people/mi^2)

FIGURE 20



Pd = Population Density (people /mi²)

FIGURE 21

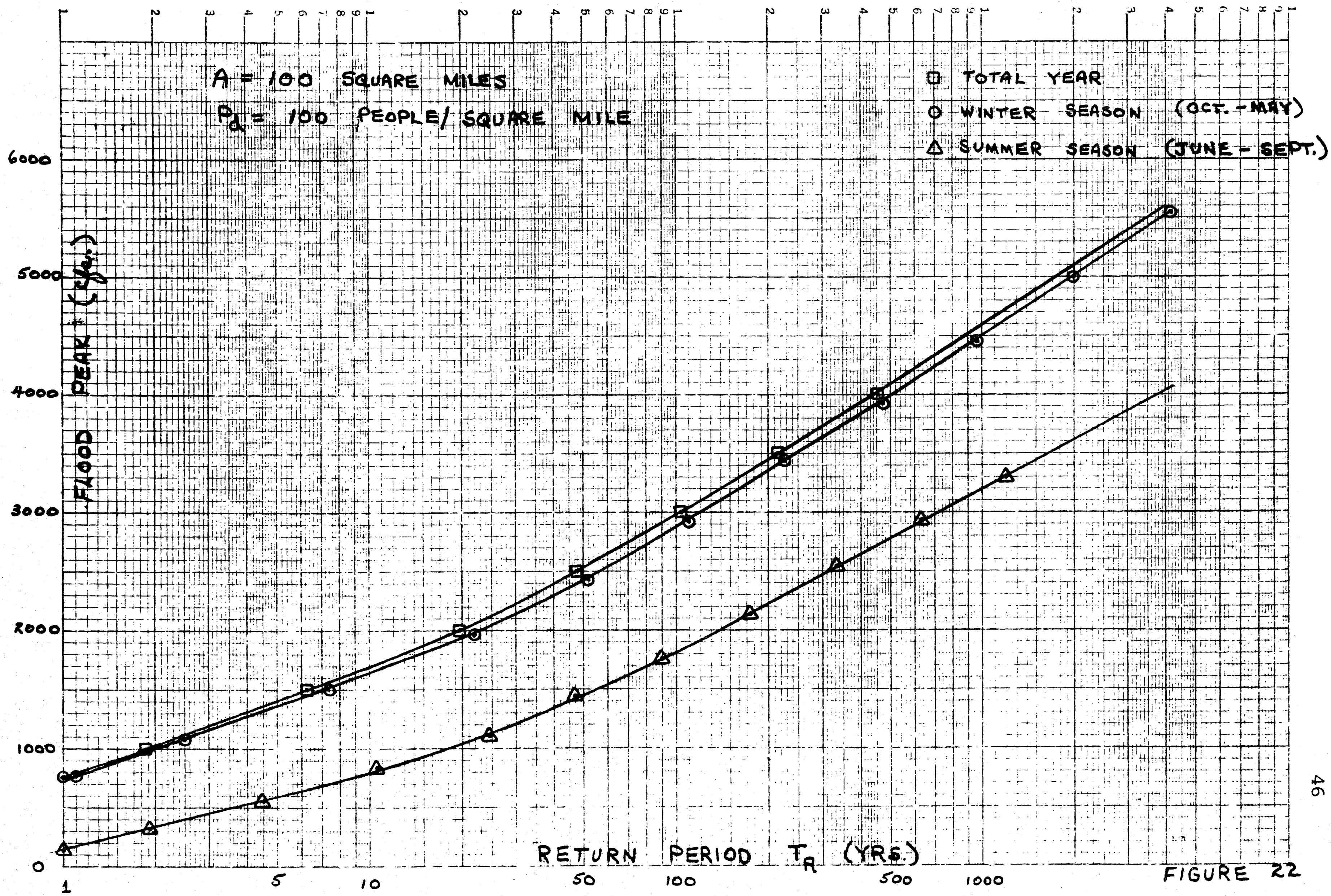

A = 100 SQUARE MILES
P_d = 100 PEOPLE/ SQUARE MILE
□ TOTAL YEAR
○ WINTER SEASON (OCT. - MAY)
△ SUMMER SEASON (JUNE - SEPT.)
FLOOD PEAK (cfs.)
6000
5000
4000
3000
2000
1000
0
RETURN PERIOD T_R (YRS.)
1
5
10
50
100
500
1000

FIGURE 22

PART II

Practical Design Curves

The sets of curves shown in Figures 23 thru 27 include all the information and procedures described in the first part of this report. They have been carefully checked and found to give results in agreement with measured flood records. In order to use the curves it is only necessary to determine the area of the drainage basin above the site of the proposed structure and estimate the probable population density during the life of the structure.

The size of the drainage area for many basins can be determined from U.S.G.S. topographic maps. However, for urban areas it is usually also necessary to obtain maps showing the storm sewer network because the natural drainage areas are sometimes modified artificially. It is also worthwhile to determine if any major changes of this type are expected in the future.

Population densities both present and projected can often be obtained from city or county planning commissions or from public utilities. In the Detroit Metropolitan area the following sources have provided very good information. The Detroit Metropolitan Area Regional Planning Commission, 800 Cadillac Square Bldg., Detroit, Michigan and the Planning Division, Southeast Michigan Council of Governments, 1248 Washington Blvd., Detroit, Michigan.

Example: Suppose the drainage basin area is 10 square miles and the expected population density is 6,000 persons per square mile. Then the 10-year flood peak discharge is read from Figure 23 as 2,500 cfs, the 20-year flood is 3,000 cfs (Fig. 24), the 30-year flood is 3,300 cfs (Fig. 25) the 50-year flood is 3,600 and the 100-year flood is 4,100 cfs (Figs. 26 & 27).

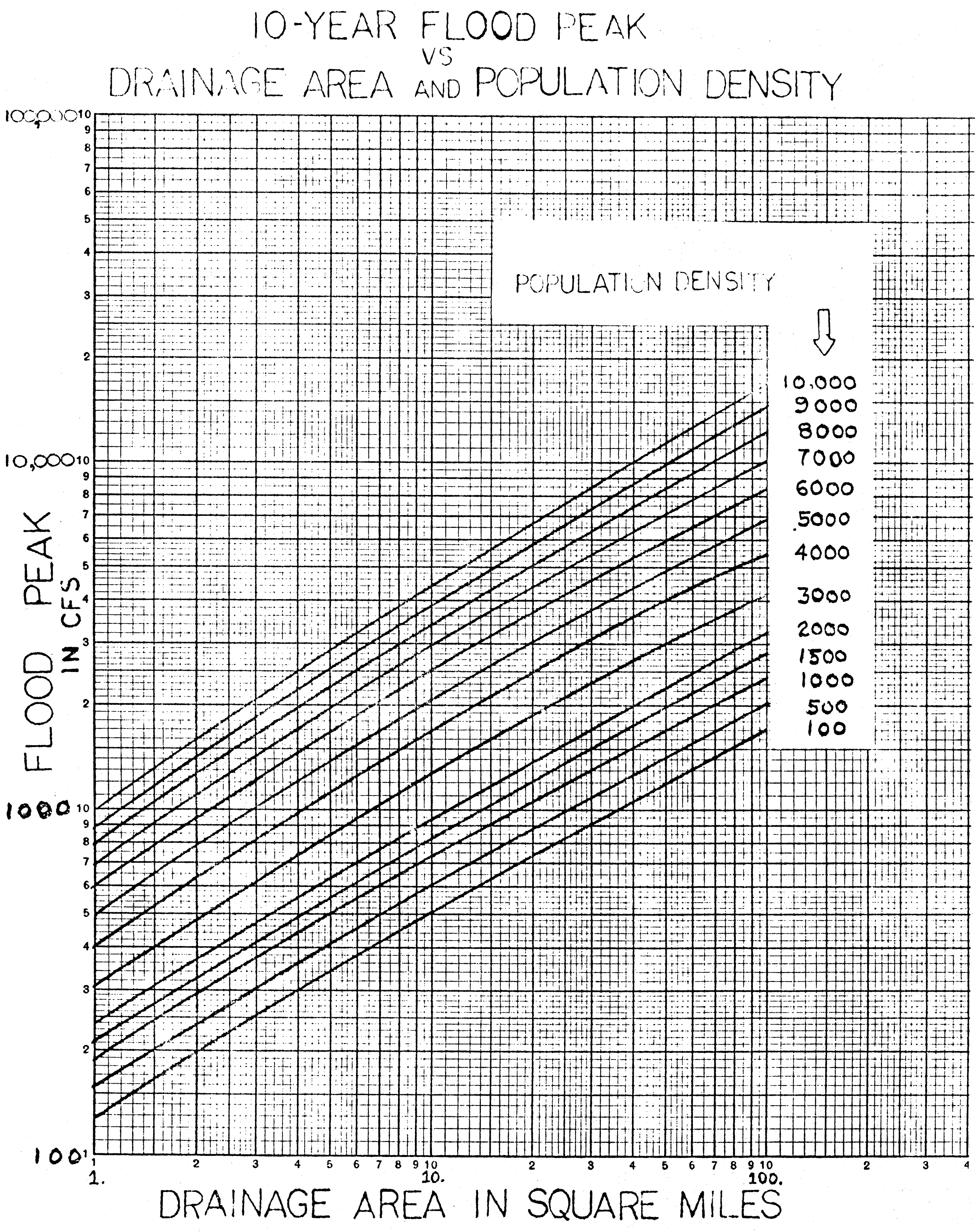

10-YEAR FLOOD PEAK
VS
DRAINAGE AREA AND POPULATION DENSITY
POPULATION DENSITY
10,000
9000
8000
7000
6000
5000
4000
3000
2000
1500
1000
500
100
FLOOD PEAK IN CFS
100,000
10,000
1000
100
1.
10.
100.
DRAINAGE AREA IN SQUARE MILES

FIGURE 23

20-YEAR FLOOD PEAK
VS
DRAINAGE AREA AND POPULATION DENSITY

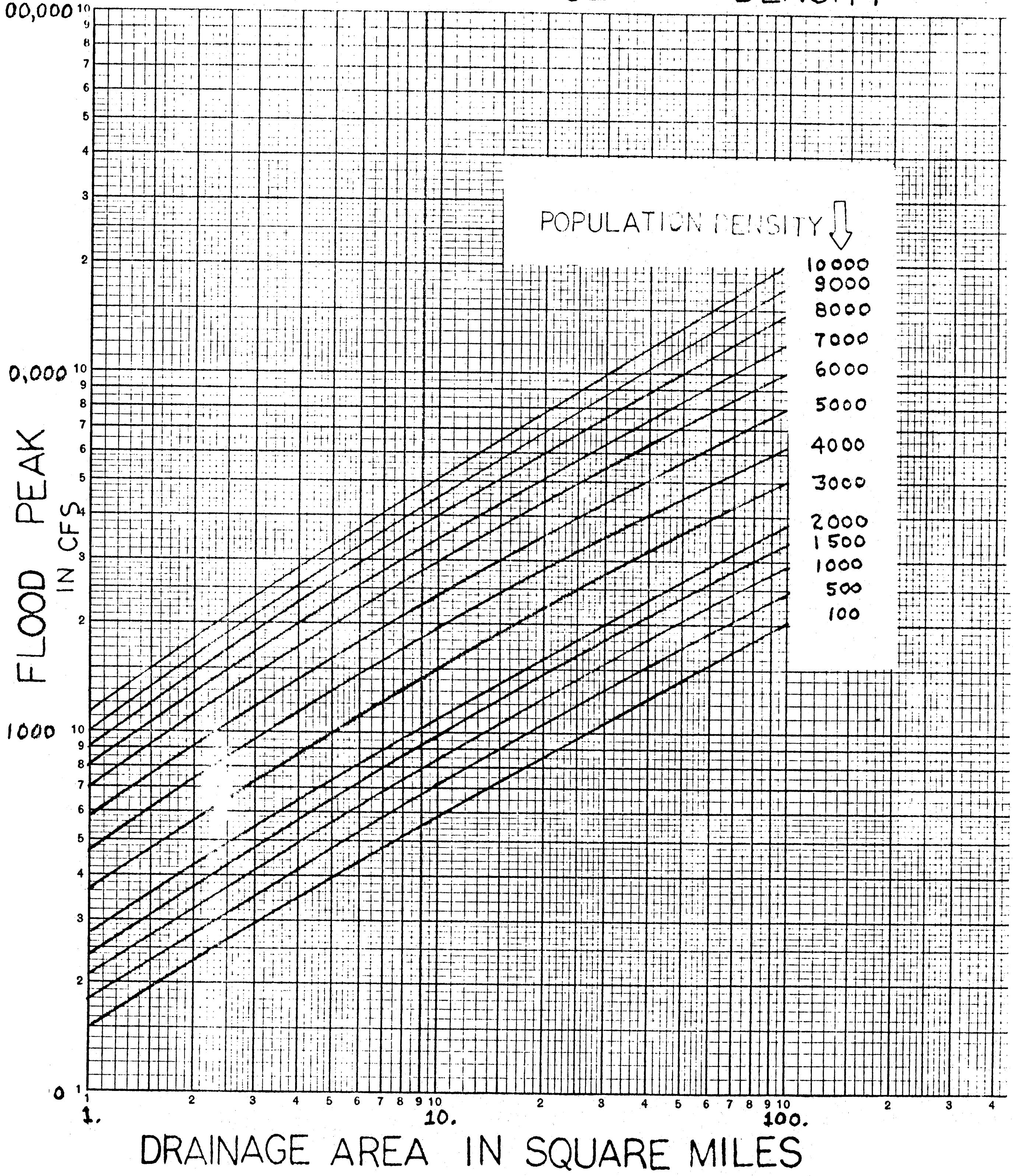


FIGURE 24

30-YEAR FLOOD PEAK VS DRAINAGE AREA AND POPULATION DENSITY

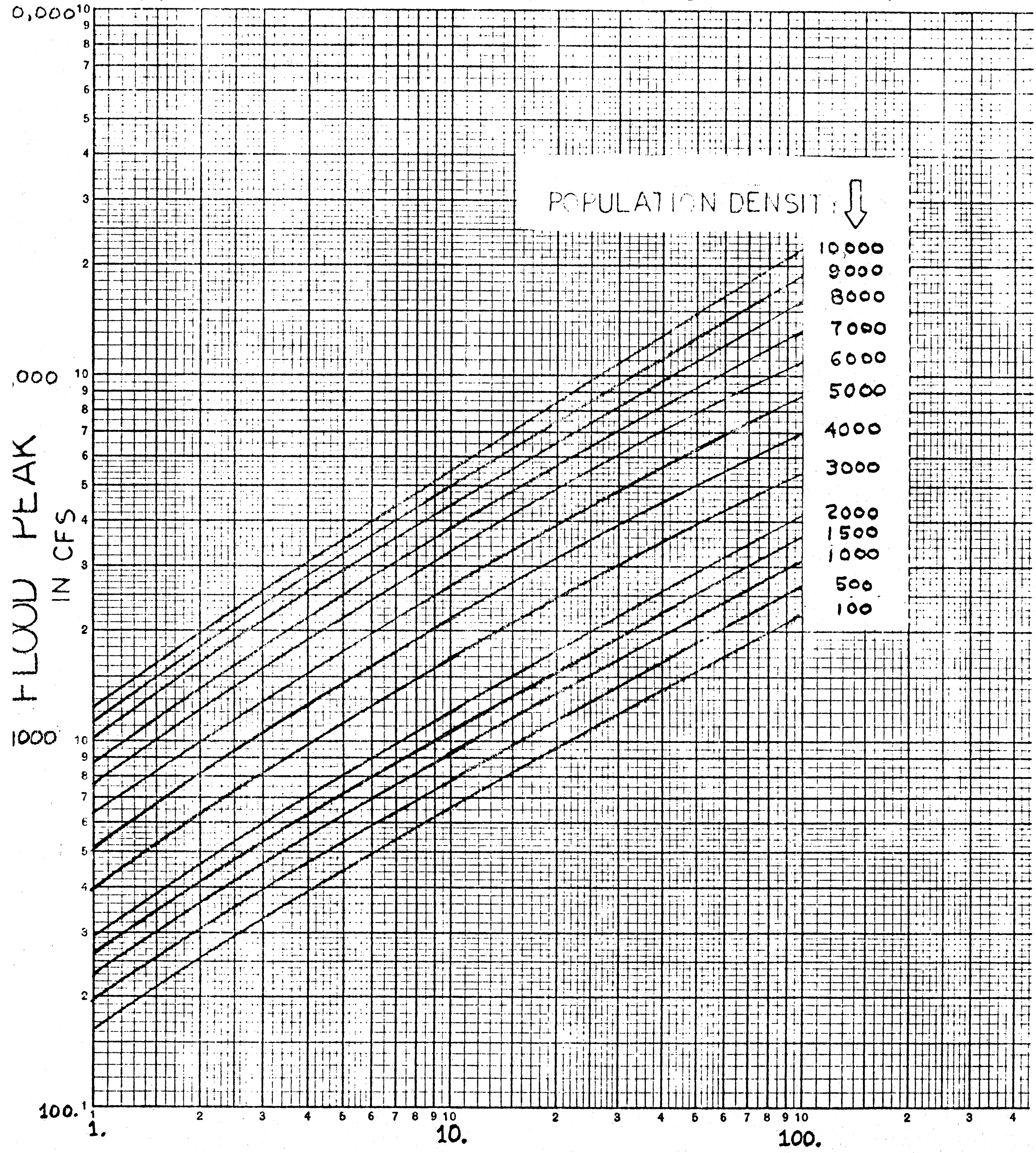


FIGURE 25

50-YEAR FLOOD PEAK VS DRAINAGE AREA AND POPULATION DENSITY

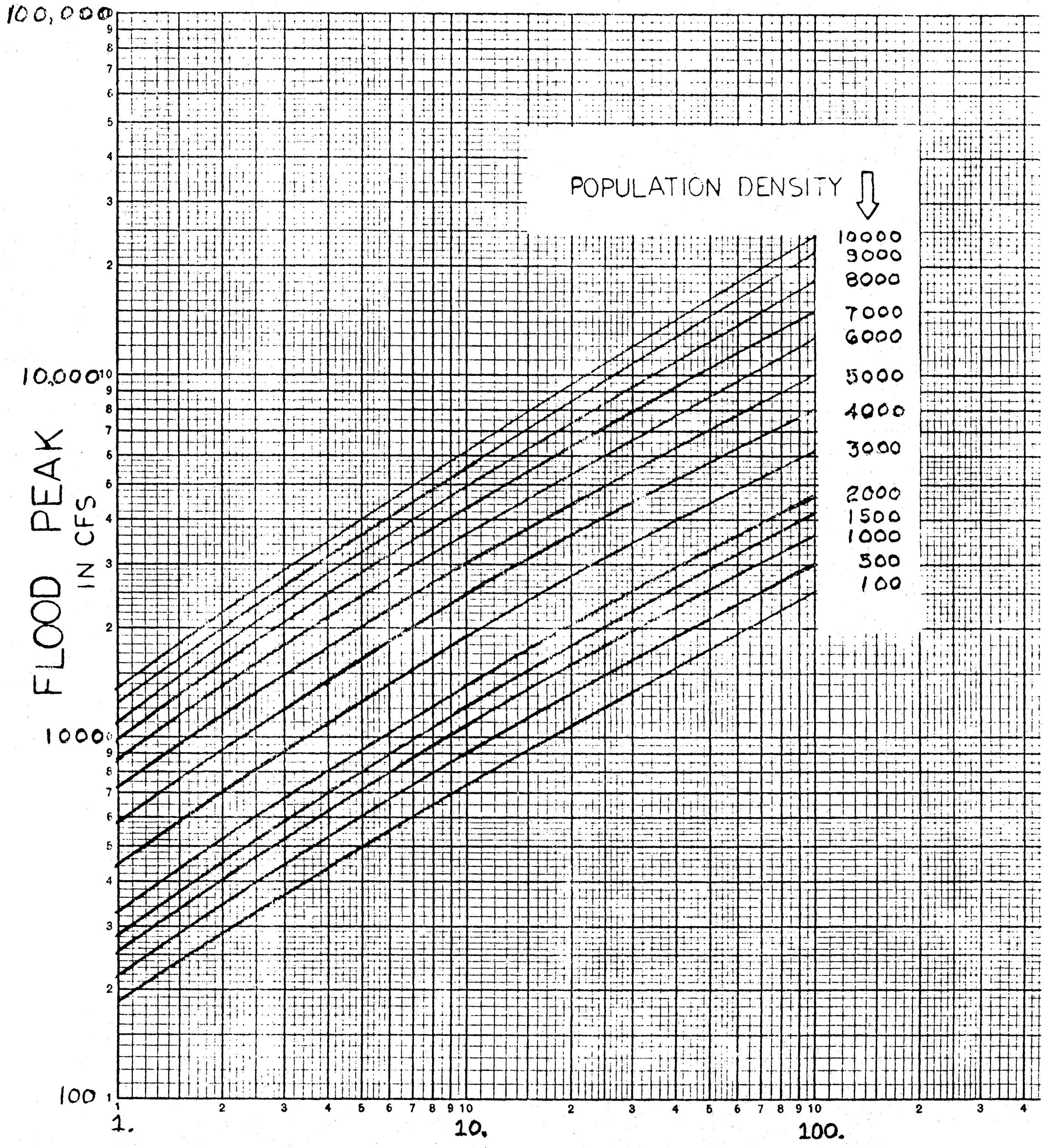


FIGURE 26

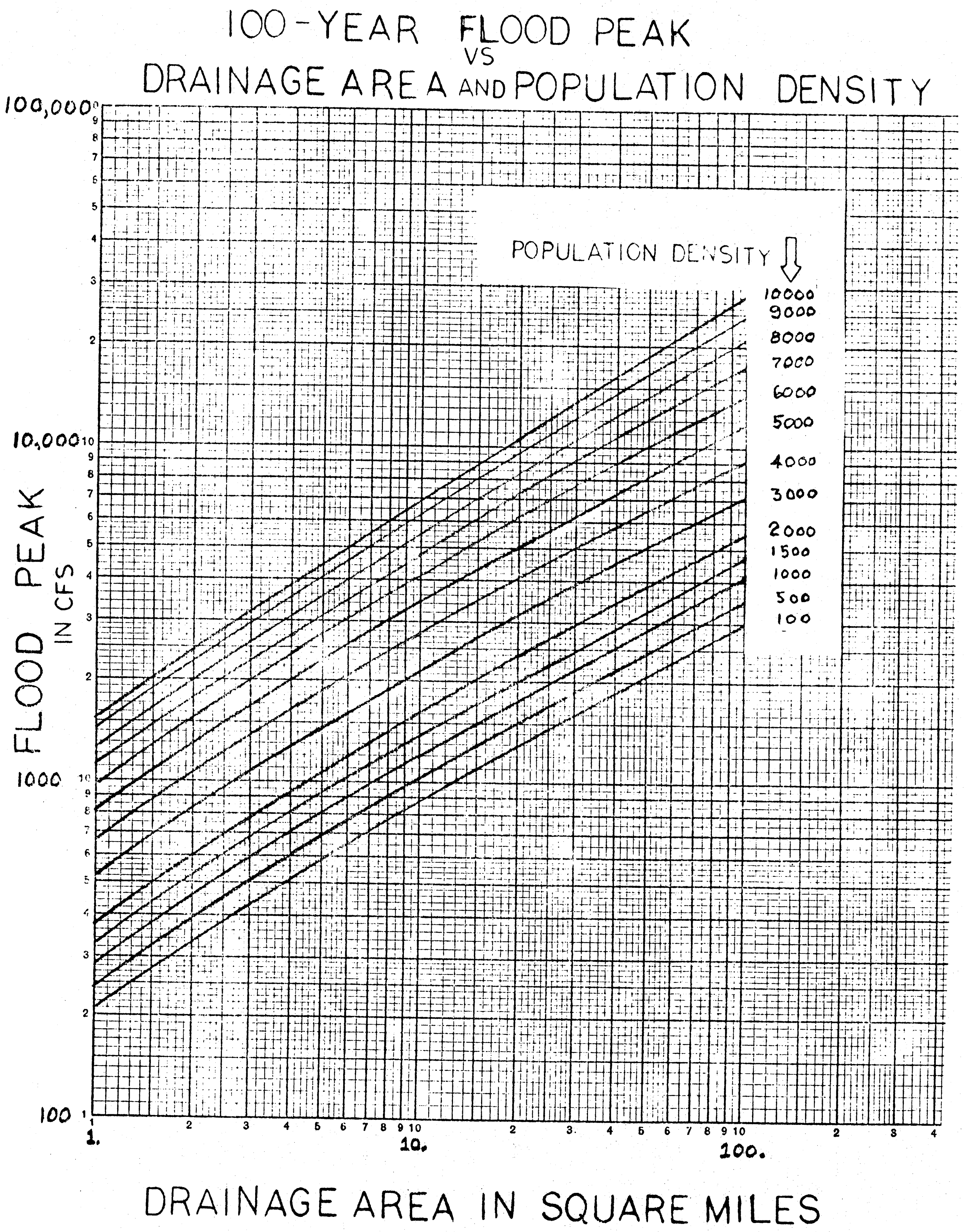

100-YEAR FLOOD PEAK
VS
DRAINAGE AREA AND POPULATION DENSITY
POPULATION DENSITY
10000
9000
8000
7000
6000
5000
4000
3000
2000
1500
1000
500
100
FLOOD PEAK IN CFS
100,000
10,000
1000
100
1.
10.
100.
DRAINAGE AREA IN SQUARE MILES

FIGURE 27