A STUDY AND EVALUATION OF METHODS OF ESTIMATING RUNOFF FROM AGRICULTURAL WATERSHEDS

by

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INTRODUCTION

Two problems must be considered in the design and construction of many soil and water conservation structures. The first is estimating maximum or peak flow which must be provided for in all structures involving water disposal. The second is the estimation of total yield that may be expected from a given area over some period of time. The period of time, being determined by the water requirements, is often taken as one year in length. The yield estimation is used in determining sizes of watersheds and reservoirs needed to furnish a given amount of water.

The accurate estimating of maximum rates of runoff from agricultural watersheds is essential in the design and construction of such structures as terraces, farm ponds, spillways, diversion ditches, culverts, irrigation reservoirs, and flood control structures. Estimating maximum runoff is also important in determining how much water has to be removed from the surface before subsurface drainage systems can be designed. An estimated peak runoff expected once in ten, twenty-five, or fifty years is used in the design of most soil conservation structures. In cases where life may be endangered the highest peak runoff rates ever expected should be considered.

Water conservation is important at the present and will, undoubtedly, become more important in the future as irrigation, industrial, and domestic use of water increases. All of these uses depend upon either ground storage or reservoir storage. The supply is increased by conservation of water on the land as very little flood water can be used for any of these purposes. Until recent years not much attention was given to irrigation in the eastern United States; however, a marked increase in irrigation in all humid areas has occurred recently. With farming practices now being used by high producing farmers, it has been estimated that there will be insufficient water in two out of five years to maintain the level of crop yields which our soils are capable of producing. This condition has been brought about mainly by an increase in production. For instance, the increase in Virginia corn yield has been about sixty percent per acre in the past thirty years.

The U. S. Geological Survey (1) has estimated the use of irrigation water in Virginia for 1950 was 3,560,000 gallons per day. The number of irrigation systems in Virginia increased from one in 1922 to 302 in 1953 (2). The acreage irrigated by the 302 systems was 12,238 acres. A breakdown of the sources of water used showed that 33 percent of the systems pumped directly from streams, 30 percent from reservoirs fed by streams, 24 percent from irrigation reservoirs that required large amounts of storage, and 13 percent from wells.

A survey made at the end of 1955 (3) showed that assistance was given by the Soil Conservation Service on 732 systems planned to irrigate 22,730 acres. Over 800 irrigation reservoirs have been planned and built. In 1955 alone, 408 of these reservoirs were built which would indicate how rapidly the practice of irrigation is growing. Undoubtedly as irrigation increases a large portion of the systems will have irrigation reservoirs requiring long-time storage. In view of these facts it seems obvious that a need exists for a method of determining the total yield that may be expected from a watershed.

Methods of estimating total yield are also needed in determining the watershed area required to replenish the water of any farm pond regardless of the pond's intended use. There are many farms or sections of farms that do not have springs or streams. Such farms must depend on surface runoff for uses such as supplemental irrigation, orchard spraying, stock watering, farm home improvements, and fire protection. Furthermore better land use can be implemented on many farms by relocating pasture and crop land if a supply of water is made available.

OBJECTIVES

The objectives of this investigation are: (1) To study and evaluate methods of estimating runoff applicable to small watersheds; (2) To compile a bibliography of references pretaining to estimating runoff.

PROCEDURE

A large portion of the work involved in this problem consisted of library research. The literature on the subject was reviewed and an evaluation of the methods of estimating runoff was made. A complete bibliography of the references studied was compiled.

The first part of the study consisted of making a survey of the available material. Several methods of estimating peak volume and total yield were selected and the merits of each are outlined in this paper. Special consideration was given to the selection of methods that appeared to meet Virginia's conditions and methods that could be solved by the use of tables or nomography. In cases where two or more similar methods existed the simpliest method was presented. At least one example of all major methods was outlined.

The data collected on watersheds W-II and W-III, consisting of 5.44 and 19.3 acres respectively, located near Blacksburg, Virginia were analyzed. These data consist of runoff measurements from 1939 through 1950 for W-II and from 1939 through 1955 for W-III. The data were analyzed by comparing the actual peak runoff and yield with the estimated peak runoff and yield obtained by the methods studied. Recording rain gauge records from a station between the two watersheds were also analyzed along with the runoff records.

FACILITIES

The facilities for the study and evaluation of existing methods of estimating peak runoff rates and total yield consisted partly of library references found in the library on the V. P. I. campus. Assistance was obtained from the librarians.

Material for the study of existing methods of estimating peak volume of runoff and total yield was found by reference to library card files, periodical literature, ASCE Journals, ASAE Journals, Transactions ASCE, and Transactions American Geophysical Union. All ASAE Journals and all Transactions ASCE after 1915 were examined. Members of the Agricultural Engineering Department were contacted to get their ideas and references.

The records of two watersheds near Blacksburg were available in the Soil and Water Conservation Research office, V. P. I. Agricultural Engineering Department. These records were analyzed and compared with the peak runoff rates and total yield as determined by the methods of estimation outlined in the study.

METHODS OF PREDICTING PEAK RATES OF RUNOFF FROM SMALL WATERSHEDS

Many detailed phases of hydrology undoubtedly have an affect on the peak runoff rate from a given watershed. Among these are intensity of rainfall, rate of infiltration, hydraulics of flow, size and shape of watershed, cover, type of soil, and time of year.

Due to the large number of variables many different methods are used to estimate the peak rate of runoff. Studies are continually being made by various industries, government agencies, and private agencies to determine the most accurate method of estimating the peak rates. The following paragraphs outline several of these methods. They also indicate something of the merits of each for use in connection with small agricultural watersheds.

Ramser's Method: In 1927 C. M. Ramser (4) presented the formula $Q = CIA$, often called the rational formula. In this formula "Q" is the discharge in cubic feet per second, "A" is the area of the watershed in acres, "I" is the rate of rainfall in inches per hour or intensity in cubic feet per second per acre (which units happen to be approximately identical), and "C" is a coefficient which represents the portion of rainfall that forms the surface runoff (it is practically a coefficient of imperviousness).

There is some question whether "I"* or "C" in this formula is

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* A method of determining "I" can be found in Appendix C.

the most critical factor to be estimated. "C" is quite often estimated from tables computed on the basis of previous experiences. For watersheds that have different covers and different slopes a composite "C" must be determined. "C" represents many elements in runoff. Some of these factors include:

1. Infiltration (which is quite variable under different conditions even in the same watershed)
2. Surface detention
3. Channel storage and distribution
4. Slope of land and topography
5. Shape of watershed, contour, and condition of land
6. Vegetal cover

Obviously many of these factors are difficult to evaluate even by one experienced in using the formula.

"I" is the rate of rainfall in inches per hour for a period equal to the time of concentration* of runoff from the watershed. The time of concentration is defined as the length of time it takes a particle of water to arrive at the outlet from the most remote spot in the watershed. Yarnell (5) is probably the most used source of rainfall intensity data. Another source is outlined in Appendix C.

In a recent survey (6) it was estimated that 85 percent of all civil engineers designing storm sewers and small culverts used the

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* See Appendix E for time of concentration for different watershed.

rational method of estimating peak discharges.

The chief advantage of Ramser's method of estimating peak runoff is its simplicity. Also it takes less prior knowledge of the area than other methods outlined in this paper.

The disadvantages are the large number of variables covered by "C" and the difficulty of choosing the correct "C", "I", and time of concentration.

This rational method provides no direct means of evaluating the effect of ponding or surface storage on the runoff rate. This handicap can be overcome by arbitrary modification of the formula.

During the past two years the author has had all members of the senior class in Agricultural Engineering make an estimate of the maximum runoff to be expected from W-III once in 12 years by the Ramser method. The results have ranged from 19.4 to 71.4, the average being 29.7. The maximum runoff gauged from W-III has been 16.52 c.f.s. during the period 1943-1955. During straight row cultivation of corn in 1938 the peak runoff was 36.90 c.f.s.

The Unit Hydrograph: The unit hydrograph was introduced in useable form in 1932 by L. K. Sherman (7). In 1929 J. A. Folse (8) presented much the same thing with most of his derivations by the method of least squares. Due to the complexity of his calculations, Folse is given very little credit for developing this useful tool in runoff hydrology.

The basic assumptions of the unit hydrograph principal are:

1. For a watershed the duration of surface runoff is essentially constant for all uniform-intensity storms of the same length, regardless of total volume of runoff.

2. For a given watershed storms of equal length produce rates of runoff at any given time from the beginning that are proportional to the total volume of runoff.

3. The time of distribution of runoff from a given storm is independent of concurrent runoff from antecedent storms.

<u>The Development of Unit Hydrographs by the Soil Conservation Service Method</u> (9): There are several methods of developing a runoff hydrograph (10, 11). The Soil Conservation method is outlined here because it seems to more nearly meet Virginia conditions.

The development of a runoff hydrograph by the unit hydrograph principal is dependent upon records from the watershed in question. Generally, records are not available for small agricultural watersheds, therefore the dimensionless unit hydrograph (figure 9, Appendix A) has been developed by the S. C. S. for use on these watersheds.

The time of concentration (travel time) is estimated by dividing the length of the longest waterway by the estimated velocity of water in feet per second. A unit rain period is chosen whose duration is between one-third and one-fifth of the time of concentration, usually an even number of minutes for convenience. An alignment chart is used to compute "time of peak" which is a function of the lag time of concentration. Also the peak rate of runoff for a unit storm producing a net runoff of one inch from the watershed is determined from the alignment chart.

The mass rainfall curve for the particular area is plotted using the rainfall intensity frequency data of Yarnell (5). The mass infiltration curve, a straight line for a constant rate, is plotted on the same chart. The point on the mass rainfall curve where its tangent is parallel to the infiltration curve gives the maximum net rainfall or precipitation excess (see figure 8, Appendix A).

At the end of each unit rain period the rainfall and corresponding infiltration are tabulated. A unit hydrograph for one inch of runoff from the watershed is prepared from a dimensionless unit hydrograph typical of the area and the unit rain periods. These data are also tabulated in table form.

The final hydrograph is prepared by multiplying the ordinates of the unit hydrograph by the corresponding net supply rate.

This method has the advantage of applying the unit hydrograph principal and also the infiltration rate in estimating the peak flow.

It has the disadvantage of requiring a large amount of work in its solution, even for small watersheds. For some watersheds the infiltration rate may vary over the watershed and therefore be difficult to estimate.

<u>Fuller's Method</u>: The formula $q = \bar{q}\,(1 + c \log_{10} T_p)$ for determining peak flows was first suggested by Fuller (12, 13). "q" is flood c.f.s. with return period T_p, "$\bar{q}$" is average annual flood, and "c" is a coefficient based on watershed size and characteristics. "c" ranges from 0.69 to 4.5 (14).

This formula was originally designed for large watersheds but has been adapted to watersheds as small as three acres in size. It was included in this study because it considers "$\bar{q}$" the annual peak rate of runoff in estimating the maximum peak runoff expected once in a given number of years T_p. The inclusion of "$\bar{q}$" may, in a large number of cases, be a handicap because of the lack of such data from small watersheds. Its greatest disadvantage when applied to small watersheds is the difficulty in obtaining the appropriate "c".

<u>Potter's Method</u>: Potter (15) suggests the relationship of peak rate of runoff and area could be expressed by the equation of $\log q = 0.490 - 0.299 \log A$, where "q" is the ten-year peak runoff in c.f.s per acre and "A" is the area of the watershed in acres.

The relationship between the peak rate for a ten-year recurrence interval and the area was determined by computing a least squares correlation. The standard error was found to be + 40.6 percent and -28.9 percent. Potter concluded that the large standard error for the correlation of "q" and "A" could have been occasioned by differences in vegetative cover, rainfall, or topography between the 51 watersheds studied. He further concluded that if these differences could be expressed as variables which could easily be derived from available data, then the reliability of peak-rate estimates would be materially increased by the inclusion of the additional variables in a multiple correlation.

For the vegetative cover factor, the vegetative cover was divided into three classes: cropland, pasture, and woodland. It was found that many of the largest deviations above the regression value were for watersheds where the cropland was zero and the woodland was 98 percent.

Likewise, many of the largest deviations below the regression value were for watersheds where the cropland was 50 to 60 percent. For this reason Potter concluded that for the Allegheny-Cumberland Plateau area and for watersheds within the range of the experimental data, the magnitude of peak rates of runoff are little affected by vegetative cover when the recurrence interval is ten years or greater.

The rainfall factors which Potter called "P-ratio and S-ratio" were found as follows: "P-ratio" was defined as the average ratio of rainfall intensity for ten-year recurrence intervals for any particular location to similar values derived from long-term rainfall records for Columbus, Ohio; "S-ratio" was defined as the average ratio of the product of annual rainfall and number of excessive storms for 10, 25, and 50 year recurrence intervals for any particular location to similar values derived from long-term rainfall records for Columbus, Ohio.

The topographic factor selected involved the measurement of the length and slope of the principal waterway. It was designated as the "T-factor" and defined as the length of the principal waterway divided by the square root of the channel slope.

A multiple correlation was computed in which these factors were considered as independent variables with the ten-year peak rate of runoff the dependent variable. The equation was $\log q = -1.421 + 0.170 \log A - 0.554 \log T + 0.929 \log P + 0.449 \log S$.

q = peak rate of runoff in c.f.s per acre for a ten-year recurrence interval

A = area in acres

T = T-factor (topography)

P = P-factor (rainfall intensity)

S = S-factor (frequency)

The lowering of the standard error in all cases was highly significant.

Potter concluded that the reliability of peak-rate estimated on short term runoff records can be greatly increased by the introduction of carefully selected rainfall and topographic factors as independent variables in a multiple correlation.

In this method of determining maximum peak rate of runoff Potter found that the vegetative cover had no effect on the peak rate of runoff when the return period was ten years or longer. This is quite different from the findings of many other investigators (16, 17, 18). All forms of the rational formula give considerable weight to the cover of the land. In an earlier publication (19) Potter has indicated that the cover of the land had an effect on the peak rate of runoff. No explanation was given for having obtained different conclusions in the two studies. In all the references studied in this investigation, Potter's report was the only one indicating that vegetative cover had no effect on peak rate of runoff.

Izzard's Method: Izzard (20) found after analyzing data on peak rate of runoff obtained by the S. C. S. from experimental watersheds in Maryland, Ohio, Wisconsin, and Nebraska that these peaks could be approximated by a single curve of runoff. The curve suggested by Izzard is shown in figure 1. It can be seen that the curve is similar to the one developed by the author (figure 2, page 27) except for the slightly curved line instead of an envelope curve.

Izzard suggests the following formula for design Q:

Q design = RF x FF x LF x Q

Q = c.f.s from figure 1

RF = rainfall factor ranging from 1.6 on the Gulf Coast to 0.6 along the Canadian border. Virginia has a factor of 1.

FF = frequency factor

LF = land use and slope factor

Table 1

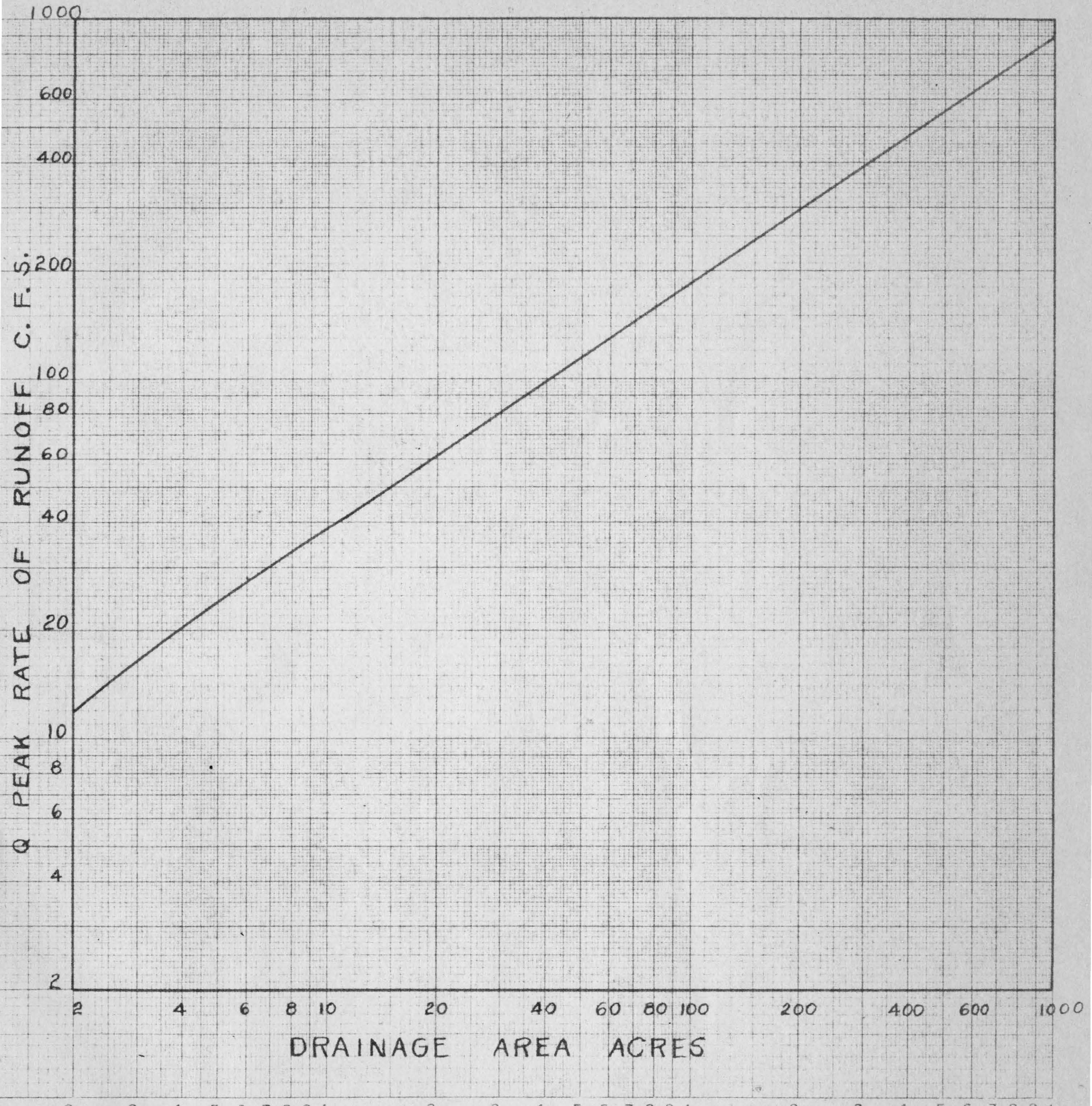
FREQUENCY FACTOR

FREQUENCY - YEARS	5	10	25	50
FACTOR	0.6	0.8	1.0	1.2

Table 2

LAND USE AND SLOPE FACTOR

LAND USE	Over 2%	Flat 0.2%	Very Flat No Ponds
100% Cultivated (row crops)	1.2	0.8	0.25
Mixed Cover	1.0	0.6	0.20
Pasture	0.6	0.4	0.10
Woods Deep Forest Litter	0.3	0.2	0.05



PEAK RATES OF RUNOFF AFTER IZZARD
MIXED COVER FREQUENCY 25 YEARS
RAINFALL FACTOR 1.0

FIGURE 1

Minshall* says in a discussion of Izzard's method that there is danger that indiscriminate application will lead to serious errors. Minshall presented tables showing estimated 25-year peaks and actual maximum measured discharges from the states of Wisconsin, Iowa, Missouri, and Illinois. Of the 14 examples given, 11 had a measured peak greater than the estimated peak. One watershed of 309 acres at Little Sioux City, Iowa had an estimated peak of 360 c.f.s. During only four years of record the estimated peak had been exceeded three times. Once the peak rate of runoff was 1750 c.f.s.

Izzard does not explain how he arrived at his land use and slope factor. The land use factor varied from 1.2 for cultivated row crops down to 0.3 for woods with deep forest litter. This is quite a contrast to the findings of Potter (page 16).

Izzard's method has the advantage of being simple to use and it has provisions for a rainfall factor, a frequency factor, and a land use and slope factor.

Army Method: The Army method (18) is a modification of the method developed by Horton (21). The basic formula for development of the graph is:

$$q = \sigma \tanh^2 \left[0.922\ t\ (\sigma/nL)^{0.50}\ S^{0.25}\right] \qquad (1)$$

q = rate of overland flow at the lower end of an elemental

* N. E. Minshall, Hydr. Engineer, Soil and Water Conservation Branch, Agri. Research Service, U.S. Dept. of Agriculture, Madison, Wisconsin.

strip in inches per hour or in cubic feet per second per acre of drainage area.

σ = rate of supply or rainfall in excess of rate of infiltration.

t = time of duration in minutes; time from the beginning of supply.

n = coefficient of roughness.

L = length of an elemental strip, overland or channel flow in feet.

S = slope or hydraulic gradient (absolute i.e. 1.0 percent = 0.01).

The factor "n" has the same meaning for overland flow as it has for channel flow, but is made up differently.

Retardence Coefficients "n" for Overland Flow

Surface	Value of "n"
Smooth pavements	0.02
Bare packed soil free of stone	0.10
Sparce grass cover or moderately rough bare surface	0.30
Average grass cover	0.40
Dense grass cover	0.80

For the solution of equation 1 the Army has designed several graphs. An example of this method can be seen in Appendix B for the solution of runoff from watershed W-III.

The Army method was tested against the rational method by the Civil Aeronautics Administration (22). There were 11 gauging stations established. Four of the 11 had a greater observed maximum runoff than the estimated maximum runoff by the rational method. Five of the 11 watersheds had a greater observed maximum runoff than the estimated maximum runoff by the Army method. The estimated peak discharge as computed by the rational method deviated from the actual record by a smaller margin in every case than the Army method.

The Army method seems to be one of the most comprehensive and best attempts to develop a better method of estimating peak discharges. It can be solved entirely by charts and graphs and the amount of labor compares reasonably well with that involved in the use of the rational formula. However, the Civil Aeronautics Administration's study indicates that the rational method is more accurate than the Army method.

Flood Peak Versus Drainage Area: Flood peak versus drainage area is another method of determining the maximum peak runoff. This method is used quite often on large watersheds, 100 square miles and larger (23). Areas are plotted as abscissas and maximum Q's are plotted as ordinates. The equation for the resulting curve which of necessity is usually an envelope curve takes the form $Q = CA^n$, where "Q" is c.f.s. maximum and "A" is area in acres. This is the equation for a straight line when plotted on log log paper. Such a curve has been plotted (figure 2) using the data from several watersheds listed in Table 3. The resulting equation in this case become $Q = 14A^{0.54}$.

It can be seen from the graph (figure 2) that for the 19 acre watershed the expected maximum peak runoff in 71.0 c.f.s. The maximum runoff based on 19 years of records was only 36.9 c.f.s.

This method of estimating does not actually take into account the return period but gives the maximum peaks direct. However, the length of record should be given some weight.

The method of flood flow versus drainage area has one outstanding characteristic. Once the equation of the curve for the area has been established the solution for peak runoff from any size watershed within the area is a simple matter. Also rainfall expectancy data doesn't have to be considered.

The big disadvantage to this method is that no consideration is given to the characteristics of the particular watershed in question. The peak runoff is determined strictly by the size of the watershed. This technique is based on an assumption that is known to be erroneous, it being a well established fact that many elements other than size are influencing factors. In view of this fact it is doubtful that this system, as presented, can be satisfactorily adapted for use on small agricultural watersheds without a system similar to Izzard's for modifications in "Q".

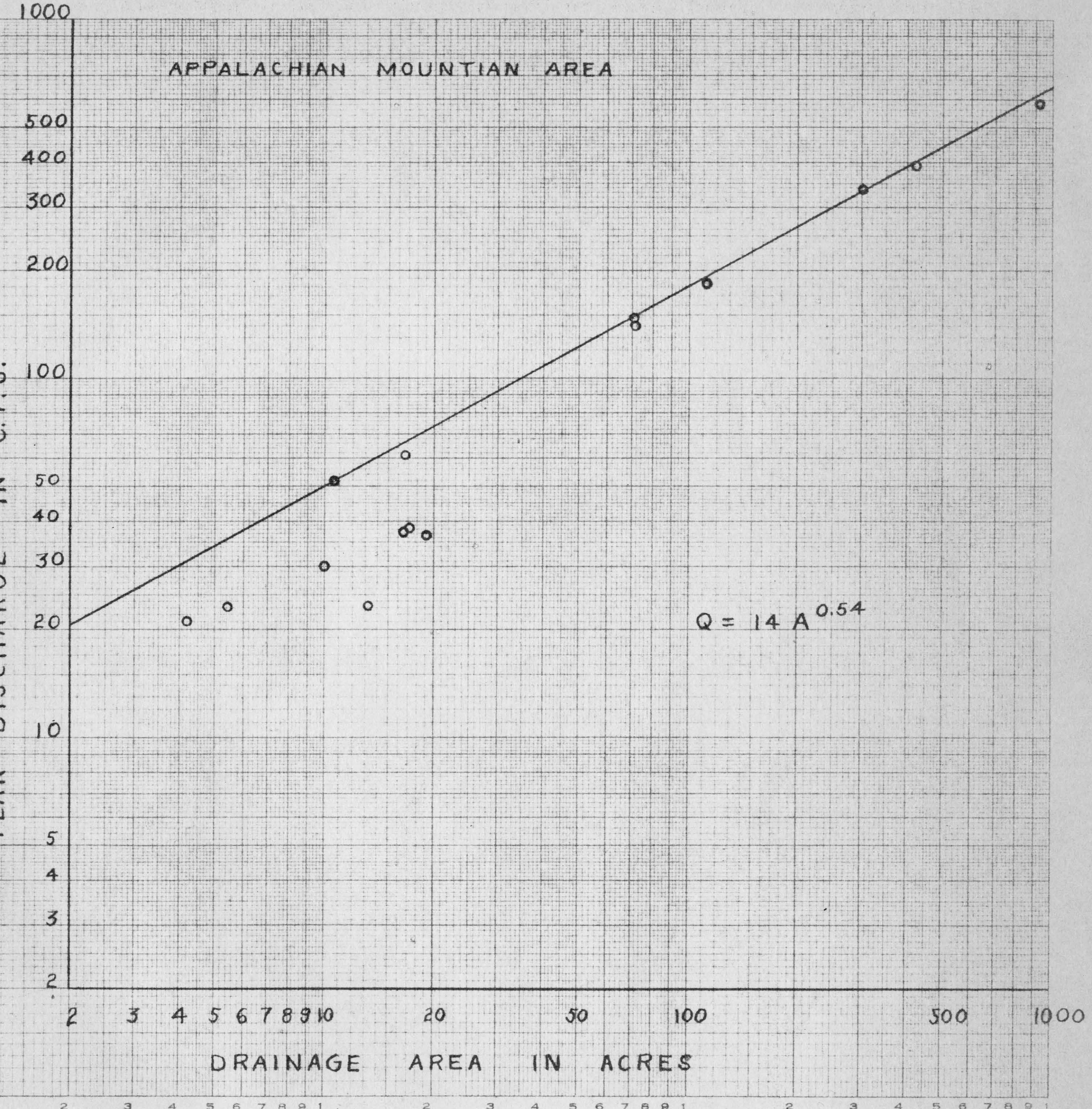
<u>Cook's Method</u>: A modification of Cook's method (24) for determining peak runoff is presented in the Virginia Handbook of Recommended Engineering Practices for Soil and Water Conservation. This method is presented in the form of tables and graphs. The results of this method when applied to watershed W-III is shown in Table 4.

Cook's method has the advantage of being relatively simple to use. In addition to the size of the area weight is given to relief, infiltration, vegetative cover, and surface storage when determining peak discharge. After a summation of the watershed characteristics has been computed, the peak rate is read directly from a chart. Provisions are also made for applying a rainfall factor to the final results.

Table 3

MAXIMUM RUNOFF FROM SELECTED WATERSHEDS

WATERSHEDS	AREA ACRES	MAXIMUM RUNOFF c.f.s	YEARS OF RECORD
Appalachian Experimental Watershed Coshocton, Ohio #183	74.2	147 ?	
#177	75.6	140 ?	
#10	122	185 ?	
#196	303	338 ?	
#20	373	390 ?	
#92	920	585 ?	
W-II Blacksburg, Virginia	5.4	23.4	19
W-III Blacksburg, Virginia	19.3	36.9	19
W-I Danville, Virginia	13.3	27.3	10
W-II Danville, Virginia	16.6	60.8	10
W-III Danville, Virginia	17.1	38.1	10
Coweeta Asheville, North Carolina	4.2	21.1	
	10.1	30.2	
Bent Creek Asheville, North Carolina	10.8	52.4	
	16.5	37.4	



FLOOD FLOW ENVELOPE CURVE
FLOOD PEAK VERSUS DRAINAGE AREA
FIGURE 2

Table 4

COMPARISON OF THE METHODS STUDIED WITH ACTUAL RUNOFF FROM W-III 19.3 ACRE WATERSHED

Blacksburg, Virginia

(All calculations for 12-year return period)

Peak runoff 18 years of record	36.90	c.f.s.
Peak runoff during 12 years of contour strip cropping	16.52	c.f.s.
ESTIMATED PEAK RATE OF RUNOFF FOR 12 YEARS		
Rational Method	35.0	c.f.s.
S.C.S. Unit Hydrograph	56.9	c.f.s.
Fuller's Method	22.0	c.f.s.
Potter's Method	24.3	c.f.s.
Izzard's Method	57.6	c.f.s.
Army Method	22.8	c.f.s.
Flood Peak Method	71.0	c.f.s.
Cook's Method	37.8	c.f.s.

SUMMARY OF PEAK RUNOFF RATES

Several methods of estimating the peak rates of runoff from small agricultural watersheds were outlined in this study. There was quite a wide variation in the maximum peak expected once in 12 years from watershed W-III by the different methods.

Table 4 gives a comparison of the methods outlined. Fuller's, Potter's, and the Army method gave an estimated peak runoff of 22.0, 24.3, and 22.8 c.f.s. respectively, which compares favorably with the 16.52 maximum c.f.s. that was actually gauged from watershed W-III during the 12 year period of the study. The rational method and Cook's method, with results of 35.0 and 37.8 c.f.s respectively, compare favorably with each other but are higher than the actual gauged peak by over 100 percent. It must be kept in mind, however, that this is only one 12 year period and may not be representative.

The S. C. S. Unit Hydrograph method gave an estimated 56.9 c.f.s. This method is designed and recommended for larger watersheds than 19.3 acres, and there is some question of its application to small watersheds. Izzard's method gave an estimated peak of 57.6 c.f.s. The chart was set up for a frequency of 25 years with frequency factors for other years given. No consideration was given in the slope factor for slopes greater than two percent.

The flood peak method gave an estimate of 71.0 c.f.s. In this method the frequency is dependent upon the years of record of all the watersheds plotted, and no consideration was given to a frequency factor or land use and slope factor. Therefore, it was expected that this result would be higher.

The rational method has an advantage of being simple to use. Simplicity is very important on small watersheds because as a rule time and funds are limited when considering small projects. The disadvantages of this method are the large number of variables covered by "C" and the difficulty in choosing the correct "C", "I", and time of concentration.

The unit hydrograph method applies the unit hydrograph principal to small watersheds. This principal has worked well on large watersheds and is widely accepted, but for small watersheds there is some question of its accuracy. It also has the disadvantage of requiring a large amount of work in its solution, even for small watersheds.

Fuller's method employs an empirical formula and is relatively easy to solve. The big disadvantage is the average annual peak runoff from the watershed under study has to be known and this information is usually not available for small watersheds.

Potter's method is of the empirical type and is relatively easy to solve once the terms in the equation are known. It gives weight to several factors that are known to have an effect on peak rates of runoff, however, no weight is given to vegetative cover a factor which most authorities believe has an effect on peak runoff.

Izzard's method employs a chart, "Flood Peak Versus Drainage Area", as a basis for determining peak runoff. Provisions are made for modifying the peak runoff by using factors for rainfall, frequency, and land use.

The Army method uses a series of charts to solve the equation developed by Horton. This method was designed primarily for airfield drainage. The work involved in the solution compares favorably with that of the rational formula. However, from studies made on the two methods the rational method seems to be more accurate.

The method of flood peak versus drainage area is recommended for the Appalachian Mountain area. It should have correction factors for land use, rainfall, and frequency before its wide spread use in that area.

Cook's method, as modified in the Virginia Handbook of Recommended Engineering Practices, meets the requirements of being easy to use and accurate. Peak flow is taken from a chart, "Peak Flow Versus Drainage Area". It has provisions for corrections in peak flow due to relief, infiltration, vegetative cover, surface storage, and frequency.

The literature is literally filled with many more formulas and charts for determining peak runoff. It is believed that the methods of estimating peak flow outlined in this paper are examples of the most important ones suitable for use on small agricultural watersheds and that they are representative of all the methods reviewed.

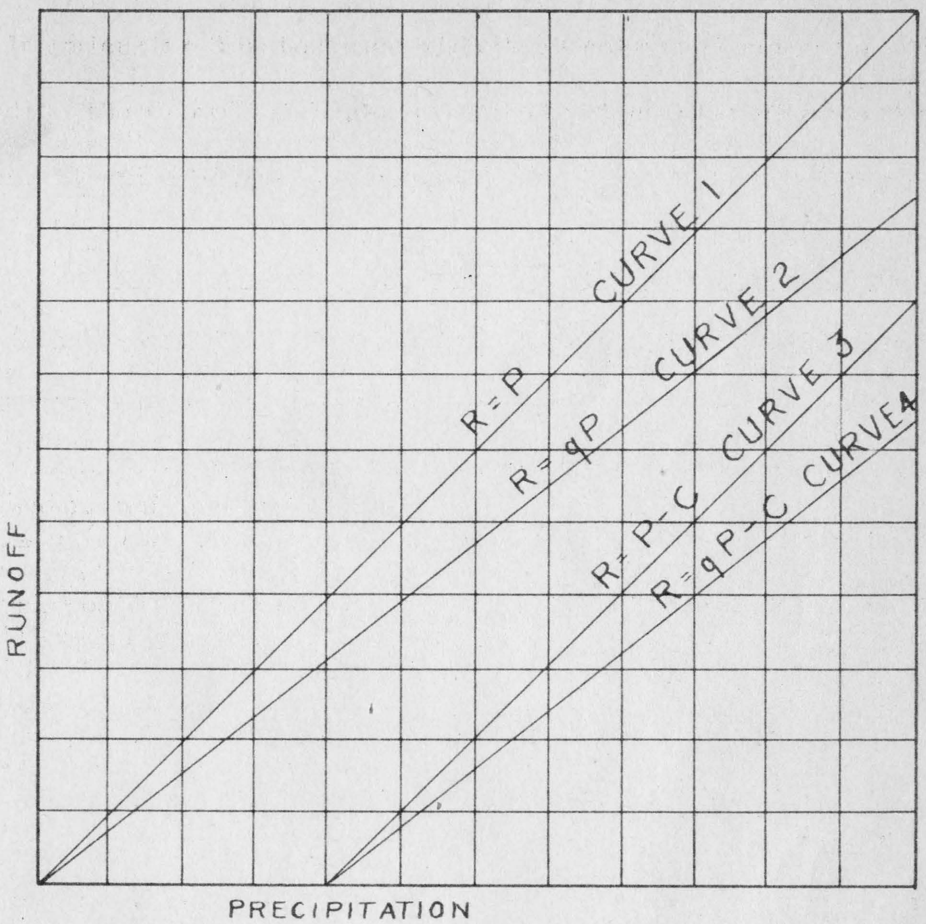
WATERSHED STUDIES TO DETERMINE THE WATER YIELD

Many phases of hydrology, undoubtedly, have an effect on the annual yield from a watershed. This study is primarily concerned with small agricultural watersheds that have no ground water to be contributed to the annual yield. One of the principal factors affecting yield is annual rainfall.

Annual Runoff as a Function of Annual Precipitation: Johnstone and Cross (25) attempted to arrive at a relationship between annual precipitation and runoff for large areas. They concluded that if the data could be represented by a straight line the most rational equation would be $R = q(P) - C$, where "q" is a percentage and "C" a constant in inches. Several possible rainfall-runoff relationships are shown in figure 3. The features claimed for curve 4, figure 3 are:

1. It passes to the right of the origin.
2. In most cases it's slope is less than that of curve 1 and 3. It also allows runoff to increase with precipitation.
3. It recognizes the tendency of plant cover to grow more luxuriantly, and therefore to transpire more in wet years than in dry ones.
4. It also recognizes the tendency toward higher ground-water levels in wet years.

The rainfall runoff relationship for large streams in southwest Virginia tend to follow curve 4, figure 3. Rainfall-runoff relationship data were plotted using data (Table 15, see Appendix D) from the James River at Cartersville, Virginia (26). The curves were fitted (figure 16) to the data by the: (1) method of averages; (2) graphical method; and



POSSIBLE RAINFALL RUNOFF RELATIONSHIPS
FIGURE 3

(3) the method of least squares.

In an attempt to determine if a similar relationship exists for small areas the data from watersheds W-II and W-III (Tables 5 and 6) were plotted (figure 4) and an attempt was made to determine the relationship between rainfall and runoff. An eye curve was drawn in. The equation of the eye curve was found to be $R = 0.032 (P) - 1.02$.

A curve was fitted to the data from the W-III watershed by the method of averages (27). In this method the best line is based upon the assumption that the location is such as to make the algebraic sum of the differences of the observed and calculated values equal to zero. Algebraically, this means $\sum (y - mx - b) = 0$.

The two constants "m" and "b" must be determined. The data were divided into two groups--odd years and even years. The resulting equation was $R = 0.121 P - 4.36$. For solution see Appendix D.

A third line was fitted to the data by the method of least squares. The position of this line is based upon the assumption that its location is such as to make the sum of the squares of the differences of the observed and calculated values a minimum. For solution see Appendix D.

The above study has been made on the runoff records of watershed W-III* Blacksburg, Virginia for the years 1944 through 1955, a total of 12 years. This watershed during the entire period was contour strip

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* W-III characteristics from Table 6.

Table 5

SUMMARY OF RUNOFF FROM WATERSHED W-II
Blacksburg, Virginia
1939 - 1950

YEAR	RAINFALL TOTAL Inches	RUNOFF TOTAL Inches	RUNOFF AS PERCENTAGE OF RAINFALL Percent	MAXIMUM DISCHARGE c.f.s.	CROP
1939*	19.50	1.962	10.06	21.80	Corn - straight row cultivation
1940	39.76	1.546	3.89	4.76	Wheat - straight row seeding
1941	28.43	0.000	0.00	0.00	Clover
1942	34.53	2.554	7.40	23.400	Corn - straight row cultivation
1943	38.24	2.641	6.91	14.140	Contour strip cropping began with corn and small grain
1944	37.19	0.505	1.35	5.72	Contour strip cropping using alternate strips of corn, small grain and clover in 3-year rotation
1945	39.36	0.025	0.06	0.462	
1946	33.80	0.1731	0.51	0.2644	
1947	41.82	0.1385	0.33	0.640	
1948	49.00	0.1674	0.34	0.598	
1949	46.29	0.0908	0.20	1.268	
1950	35.48	0.0829	0.23	3.660	
Yearly Av.**	38.01	0.847	2.23	—	

W-II Watershed Characteristics: Area = 5.44 acres
Prevailing Slopes = 6-10%
Waterway Length = 600 feet
Soil:
Dunmore Silt Loam = 5.14 A
Emory Silt Loam = 0.30 A
Form Factor = A/L^2 = 0.29

* May - December
** Based on 11 years and 8 months.

Table 6

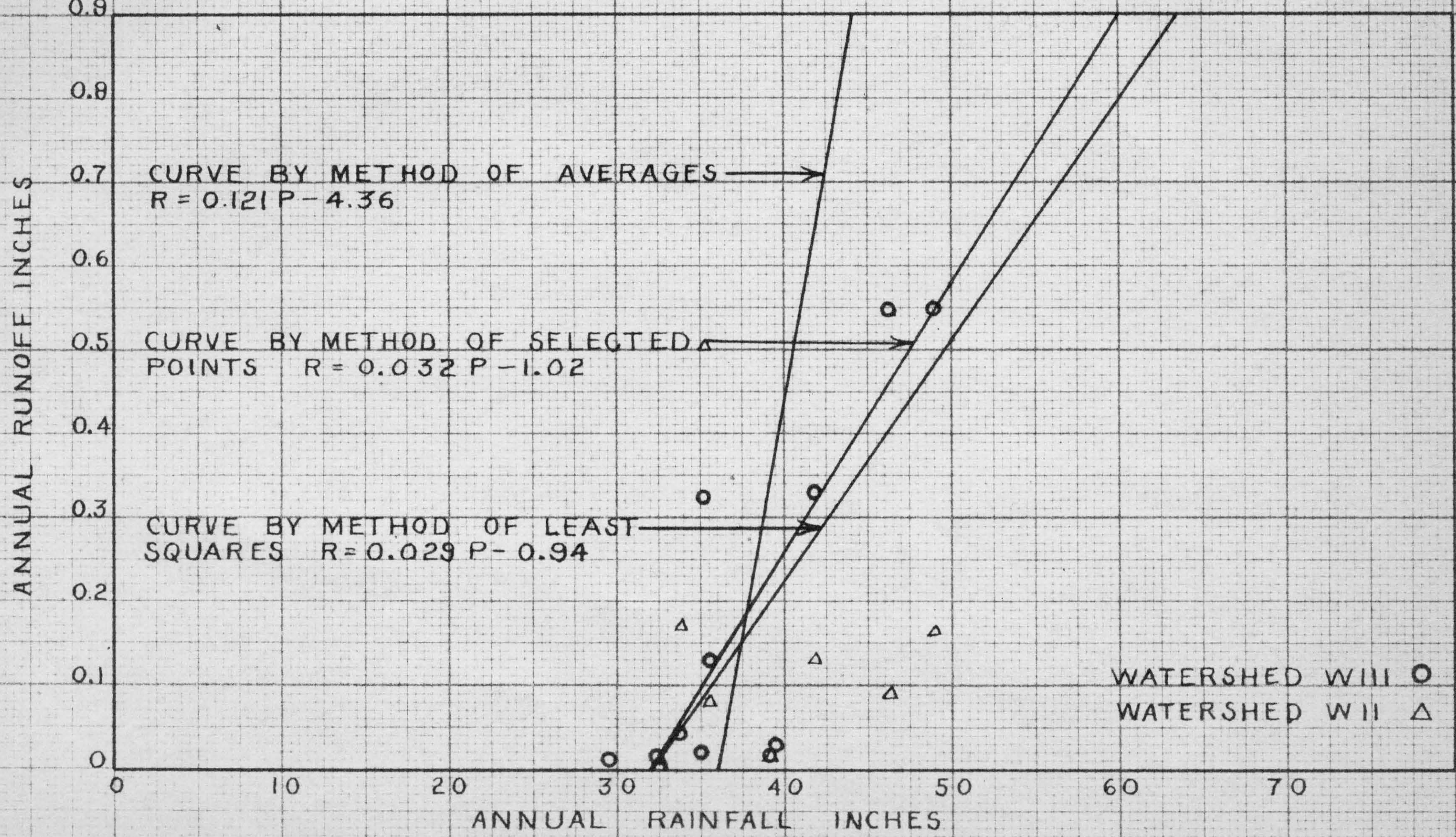
SUMMARY OF RUNOFF FROM WATERSHED W-III
Blacksburg, Virginia
1939 - 1955

YEAR	RAINFALL TOTAL Inches	RUNOFF TOTAL Inches	RUNOFF AS PERCENTAGE OF RAINFALL Percent	MAXIMUM DISCHARGE c.f.s.	CROP
1939*	19.50	1.4100	7.24	26.500	Corn - straight row cultivation
1940	39.76	0.4330	1.09	4.660	Wheat - straight row seeding
1941	28.43	0.0050	0.02	0.1696	Clover
1942	34.53	2.9350	8.50	36.900	Corn - straight row cultivation
1943	37.43	1.5990	4.27	18.600	Contour strip cropping began with corn and small grain
1944	35.24	0.3248	0.92	10.59	Contour strip cropping using alternate strips of corn, small grain, and clover in 3-year rotation,
1945	39.36	0.0166	0.04	0.383	
1946	33.80	0.0438	0.13	1.198	
1947	41.82	0.3258	0.78	5.555	
1948	49.00	0.5479	1.12	2.720	
1949	46.29	0.5477	1.18	8.179	
1950	35.48	0.1301	0.37	16.5220	
1951	35.08	0.0285	0.08	0.9628	
1952	39.52	0.0316	0.08	0.9268	
1953	32.41	0.0184	0.06	0.138	
1954	32.56	0.0035	0.01	0.068	
1955	29.54	0.0116	0.04	0.4109	
Av.**	36.89	0.4380	1.18	—	

W-III Watershed Characteristics: Area = 19.3 acres
Prevailing Slopes = 3%, 5%, 13%
Average Slope = 4.6%
Waterway Length = 1400 feet
Soil:
Dunmore Silt Loam = 17.3 A
Emory Silt Loam = 1.20 A
Dunmore Clay Loam = 0.8 A
Form Factor = A/L^2 = 0.50

* For part year (May - December)

** Part year (1939) not included.



ANNUAL RAINFALL RUNOFF RELATIONSHIP

WATERSHED WIII BLACKSBURG, VA

FIGURE 4

cropped, using alternate strips of corn, small grain, and clover in a three year rotation. The weather bureau records of rainfall in Blacksburg covers a period of 64 years (see Table 8, page 42). By combining one of the equations from figure 4 and the data from Table 8 the expected yield for any return period up to 64 years can be computed. This yield would apply only to a watershed similar to W-III.

A similar study can be made on any watershed that has runoff records available. The validity of applying curves from one watershed to another with different cropping practices or different soil conditions is highly questionable. Curves similar to those shown in figure 4 should be plotted from runoff data from watersheds with different cropping practices and soil conditions. The above curves combined with local rainfall records could be used to determine expected runoff.

Harrold (28) found that at Coshocton, Ohio the precipitation differed as much as 12.84 inches from dry to wet years, but that water used by evapo-transpiration differed only by 1.95 inches (see Table 7, line 4). He concluded that evaporation and transpiration takes its toll of precipitation in humid sections of the country almost without regard to the amount of precipitation.

If this observation of Harrold's holds true for Virginia, then it could be concluded that after a certain amount of yearly precipitation had been reached the remainder would show up as runoff similar to curve 3, figure 3. A more logical assumption would be that after a given amount of precipitation the runoff would be a function of precipitation.

It should then take the form $R = q P - c$ as found by Johnstone and Cross (page 32).

Table 7

MAXIMUM AND MINIMUM ANNUAL PRECIPITATION WATER FLOW FOR DIFFERENT SIZE WATERSHEDS

1937-1955

Coshocton, Ohio

WATERSHED SIZE	YEAR OF MAXIMUM FLOW		YEAR OF MINIMUM FLOW	
	Precipitation	Flow	Precipitation	Flow
Acres	Inches	Inches	Inches	Inches
17,540	44.78	22.19	34.02	5.15
4,580	44.78	23.34	34.02	4.37
2,570	46.86	19.00	34.02	4.21
349	46.86	18.13	34.02	4.02
122	46.86	14.22	34.02	3.50
74	44.24	18.25	28.94	2.15
29	43.53	13.92	32.12	0.94

Nixon and Schwab (29) found that for Midwest conditions the mean water yield at a given location could be predicted from the mean annual precipitation and mean annual evaporation.

Kirkpatrick (30) developed a table on the minimum runoff from small watersheds that can be expected for one, two, three, and four consecutive year periods under various cropping conditions in southwest Virginia. The information is given in Appendix F, Table 17. This table is based on observation in this area and should be valid for the area and conditions specified.

Rich (31) found that for certain drainage basins in Arizona a highly significant correlation exists between precipitation and runoff. The equation took the form $y = 1242X^{1.83}$ in which "y" is annual runoff in cubic feet and "x" is yearly rainfall in inches. This particular study was made on a watershed of 12 acres, however, he found that a similar relationship existed between annual runoff and annual precipitation on watersheds of 700 acres and 1087 acres.

Applying this formula to the Blacksburg area, an average of more than 20 inches of water would run off per year. An examination of the runoff records of watersheds W-II and W-III reveal that the highest annual runoff during the 17 years of the experiment was 2.935 inches, with an average for the two watersheds of only 0.642 inches. It seems obvious, therefore, that although an equation of the general type developed by Rich might be applicable in this area the one developed for Arizona conditions definitely is not.

In a study to determine the minimum runoff to be expected once in 25 years from large watersheds, McGauhey (26) found that a relationship existed between annual rainfall and runoff. He expressed the relationship with the aid of three curves plotted on log probability paper. He concluded that the minimum coefficient (rainfall runoff) could be read from the graph and multiplied by the minimum rainfall to obtain the minimum runoff expected.

A study similar to probability study number two of McGauhey's report (26) was made on watershed W-III (1944 to 1955 record). The rainfall data were from the U. S. Weather Bureau records of 1892 through 1955 for Blacksburg (shown in Table 8).

The data presented in Table 8, columns 4 and 5, were plotted on probability paper to give curve 3, figure 5. The data in Table 9, columns 4 and 5, were plotted on the same figure. Curve 1 was the result of this plotting.

The curve of best fit for the data in Table 9 was plotted in figure 5. This curve is designated as curve 2. Calculations for curve 2 are shown in Appendix G.

An example of predicting runoff by the use of curves in figure 5 can be found in Appendix G.

Table 8

ANNUAL RAINFALL

Blacksburg, Virginia

1892 - 1955

1	2	3	4	5
YEAR	RAINFALL	ORDER OF SIZE	LOG RAIN	PERCENT CENTER OF STRIP
1892	27.99	21.09	1.324	0.781
1893	34.49	25.95	1.414	2.344
1894	25.95	27.80	1.444	3.907
1895	35.45	27.99	1.447	5.470
1896	48.89	28.43	1.454	7.033
1897	40.11	29.54	1.470	8.596
1898	31.14	31.14	1.493	10.159
1899	45.35	32.25	1.509	11.722
1900	43.75	32.41	1.511	13.285
1901	53.46	32.56	1.513	14.848
1902	33.73	33.66	1.527	16.411
1903	44.76	33.73	1.528	17.974
1904	32.25	33.80	1.529	19.537
1905	51.94	34.18	1.534	21.100
1906	44.78	34.49	1.538	22.663
1907	43.33	34.53	1.538	24.226
1908	47.44	35.08	1.545	25.789
1909	36.27	35.24	1.547	27.352
1910	43.39	35.45	1.550	28.915
1911	45.67	35.48	1.550	30.478
1912	43.72	35.91	1.555	32.041
1913	43.95	36.27	1.560	33.604
1914	44.59	37.43	1.573	35.167
1915	41.92	37.69	1.576	36.730
1916	47.28	38.45	1.585	38.293
1917	38.90	38.90	1.590	39.856
1918	65.56	38.93	1.590	41.419
1919	47.44	39.36	1.595	42.982
1920	46.15	39.49	1.596	44.545
1921	40.69	39.52	1.597	46.108
1922	42.99	39.76	1.599	47.671
1923	42.12	40.11	1.603	49.234
1924	49.05	40.69	1.610	50.797
1925	27.80	41.82	1.621	52.360

Table 8 (continued)

1	2	3	4	5
YEAR	RAINFALL	ORDER OF SIZE	LOG RAIN	PERCENT CENTER OF STRIP
1926	38.93	41.92	1.622	53.923
1927	44.40	42.01	1.623	55.486
1928	37.69	42.12	1.624	57.049
1929	46.89	42.99	1.633	58.712
1930	21.09	43.33	1.637	60.275
1931	39.49	43.39	1.637	61.838
1932	47.50	43.69	1.640	63.401
1933	33.66	43.72	1.641	64.964
1934	38.45	43.75	1.641	66.527
1935	43.69	43.95	1.643	68.090
1936	42.01	44.40	1.647	69.653
1937	48.75	44.59	1.649	71.216
1938	35.91	44.76	1.651	72.779
1939	34.18	44.78	1.651	74.342
1940	39.76	45.35	1.657	75.905
1941	28.43	45.67	1.660	77.468
1942	34.53	46.15	1.664	79.031
1943	37.43	46.29	1.666	80.594
1944	35.24	46.89	1.671	82.157
1945	39.36	47.28	1.675	83.720
1946	33.80	47.44	1.676	85.283
1947	41.82	47.44	1.676	86.846
1948	49.00	47.50	1.677	88.409
1949	46.29	48.75	1.688	89.972
1950	35.48	48.89	1.689	91.535
1951	35.08	49.00	1.690	93.098
1952	39.52	49.05	1.691	94.661
1953	32.41	51.94	1.716	96.224
1954	32.56	53.46	1.728	97.787
1955	29.54	65.56	1.817	99.350

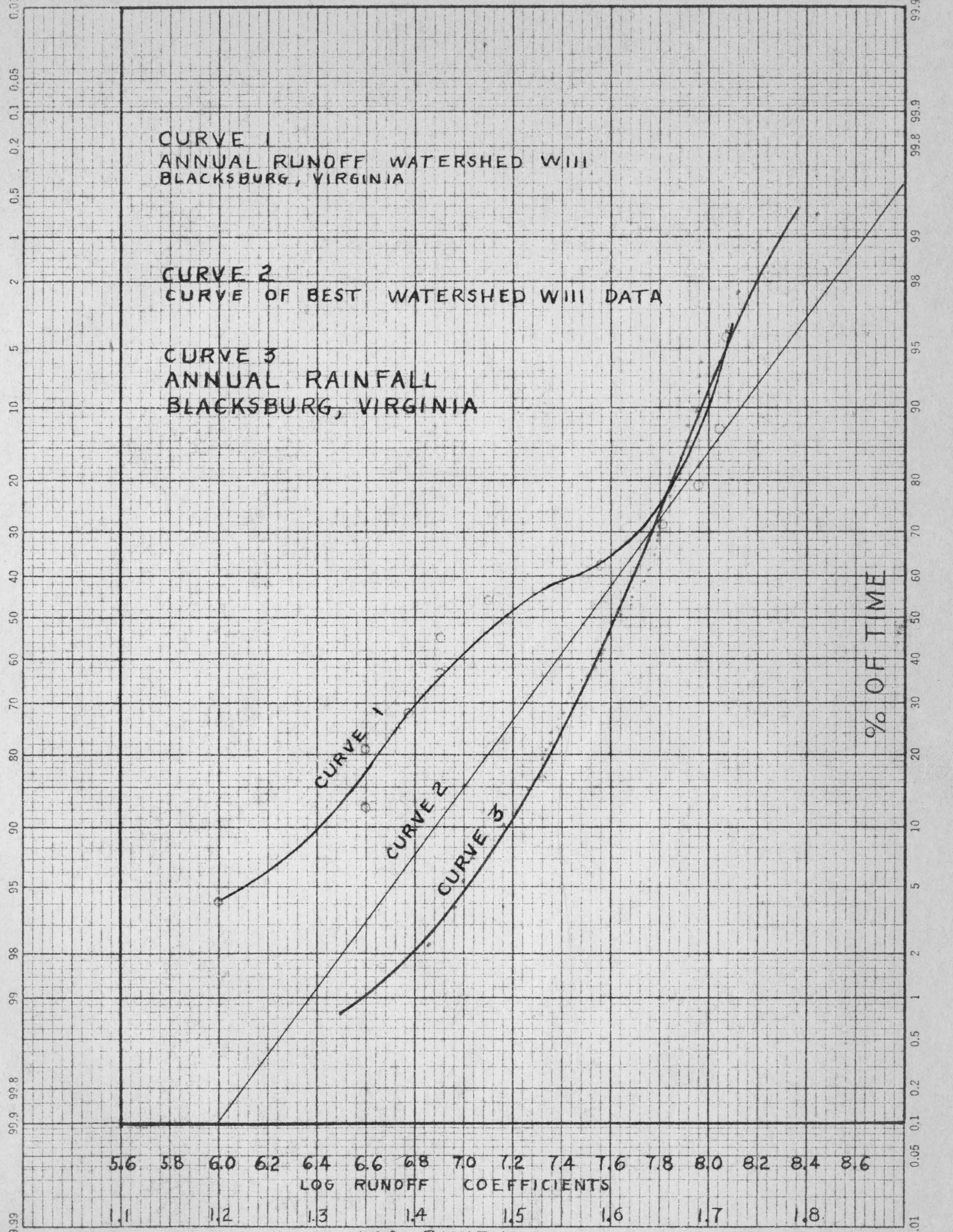
Table 9

ANNUAL RUNOFF FROM WATERSHED W-III

Blacksburg, Virginia

1944 - 1955

1	2	3	4	5
YEAR	RUNOFF PERCENT RAINFALL	COEFFICIENT ORDER OF SIZE	LOG COEFFICI-ENT	PERCENT CENTER OF STRIP
1944	0.92	0.0001	6.000-10	4.15
1945	0.04	0.0004	6.602-10	12.48
1946	0.13	0.0004	6.602-10	20.81
1947	0.78	0.0006	6.778-10	29.14
1948	1.12	0.0008	6.903-10	37.47
1949	1.18	0.0008	6.903-10	45.80
1950	0.37	0.0013	7.114-10	54.13
1951	0.08	0.0037	7.568-10	62.46
1952	0.08	0.0078	7.892-10	70.79
1953	0.06	0.0092	7.964-10	79.12
1954	0.01	0.0112	8.049-10	87.45
1955	0.04	0.0118	8.072-10	95.78



CURVE 1
ANNUAL RUNOFF WATERSHED WIII
BLACKSBURG, VIRGINIA
CURVE 2
CURVE OF BEST WATERSHED WIII DATA
CURVE 3
ANNUAL RAINFALL
BLACKSBURG, VIRGINIA
CURVE 1
CURVE 2
CURVE 3
% OF TIME
5.6 5.8 6.0 6.2 6.4 6.6 6.8 7.0 7.2 7.4 7.6 7.8 8.0 8.2 8.4 8.6
LOG RUNOFF COEFFICIENTS
1.1 1.2 1.3 1.4 1.5 1.6 1.7 1.8
LOG. RAINFALL

FIGURE 5

SUMMARY OF WATER YIELD STUDIES

Far less has been written about small watershed yields than about peak runoff. Figure 16 in Appendix D indicates that there is a definite relationship between rainfall and runoff for large watersheds. The three curves plotted to the data are reasonably close together. The same method was used in an attempt to establish a relationship between annual yield and precipitation on W-III (figure 4, page 37). Due to the very small amount of runoff during the 12 years of contour strip cropping it was difficult to determine if such a relationship exists.

Different scales had to be used for runoff and rainfall which is somewhat misleading when comparing Cartersville data with that of W-III. If the same scales had been used for rainfall and runoff on the W-III data the graph would have been meaningless. The curve from figure 4, page 37 by the method of least squares has slope and other characteristics similar to the Cartersville data for large watersheds. This line described by the equation $R = 0.029\ P - 0.94$ gives results similar to those found by Kirkpatrick as outlined in Table 17, Appendix F.

The data from W-III plotted on probability paper gave curve number 1 shown in figure 5. Runoff is plotted against percent of time, theoretically the results should appear as a straight line represented by line 2. Due to the wide deviation of line 1 from line 2, particularly in the lower area, this method is not given much weight.

Curves similar to those shown in figure 4, page 37 for watershed W-III could be plotted from runoff data from watersheds with different cropping practices and soil conditions. The equation of the above curves could then be combined with local rainfall data to determine the expected annual yield.

Most methods of determining yield from small watersheds depend on empirical formulas. There are many watersheds that have different cropping conditions and soil types which do not have sufficient data to justify the use of empirical formulas in predicting annual yield.

Table 17, Appendix F, gives expected yields for ridges and valleys section of Virginia as found by Kirkpatrick.

CONCLUSIONS

On the basis of this study the following observations, as they apply to the Blacksburg area, appear to be warranted:

1. More than 20 methods of predicting peak runoff were studied during the course of this investigation. Eight of these methods which seem to meet local requirements in Virginia were presented in this paper.

2. The rational method is widely used in predicting runoff from small agricultural watersheds. It may, in the future, be superseded by some other method, but at present other methods are too complex to be used in the design of small structures.

3. Cook's method, as modified in the Virginia Handbook of Recommended Engineering Practices, seems to give approximately the same results as the rational method. Cook's method has a systematic procedure for determining the coefficient that was not available in most other methods studied.

4. The method of estimating maximum peak runoff by the flood peak versus drainage area method has promise of meeting the requirements of accuracy and simplicity. To use the flood peak method a set of tables similar to Izzard's for modifying the results should be designed.

5. Due to the large amount of work involved in the solution of the unit hydrograph this method does not seem to be suited to small watersheds.

6. Several investigators have arrived at a system of predicting yield based on annual rainfall as expressed by the general equation $R = q P - C$. On the basis of this study, the equation for water yield appears to be $R = 0.029 P - 0.94$ for contour stripped watersheds near Blacksburg.

7. Several other methods are available for predicting yield. Among the most important of these is the method listed in Appendix F by Kirkpatrick.

RECOMMENDATIONS

The author recommends:

1. A more intensive educational program for the small area floods.

2. A study be made of runoff from springs that have relatively small watersheds and study the depletion curves of these springs and ground water storage curves to determine the relationship of surface runoff to the storage capacity of the watershed.

3. A study be made of the methods of sediment transportation in large and small streams to determine the extent of damage caused by small area floods and large area floods.

4. Test chemicals to determine the change in infiltration rate on cultivated land and what effect it has on runoff.

5. A long-time study be made of area depth of rainfall from thunderstorms to determine if and to what extent they are concentrated at a point and to determine to what extent this heavy point rainfall causes a greater soil and water loss from small areas in the summertime.

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APPENDIX

CONTENTS OF APPENDIX

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APPENDIX A

DEVELOPMENT OF UNIT HYDROGRAPH AND FINAL HYDROGRAPH FOR DESIGN STORM ON WATERSHED W-III BLACKSBURG, VIRGINIA BY THE SOIL CONSERVATION SERVICE METHOD

1. Drainage area = 19.3 acres = 0.032 square miles

   Travel distance from ridge to gauging station = 1400 feet

   Average slope = 4.6 percent

   Channel velocity estimated 1.5 feet per second (time of concentration for W-III is 15.5 minutes)

   Infiltration rate 1.20 (average of three tests with infiltrometer)

2. Time of concentration (F) 1400/1.5 = 933 seconds
   933/60 = 15.5 minutes 15.5/60 = 0.26 hr.

   Let D = unit rain period ("unit time") for use in analyzing the mass rainfall and developing the unit hydrograph. (Observation has shown that an approximate value of "D" should be about one-third to one-fifth of time of concentration "F".) Thus D = 1/5 x F = 15.5/5 = 3.1 minutes. Use D = 4 minutes for convenience.

3. Next use the alignment chart, figure 6. From the line on the right side of the page determine the "lag time" (t) corresponding to the concentration time of 0.26 hours. Thus t = 0.16 hours = 9.6 minutes. Next compute "time of peak" (a_o) as $a_o = d/2 + t = 4/2 + 9.6 = 11.6$ minutes = 11.6/60 = 0.193 hours. Now determine peak rate of runoff from the lines on the left side of the alignment chart as follows:

   Draw a straight line connecting the "a_o" value of 0.193 hours with the area value of 0.032 squares miles. It passes through the value 75 c.f.s. on the scale for R (peak rate of runoff for a unit storm producing a net runoff of one inch from the watershed). Figure 7 illustrates the meaning of "lag" (t) and its connection with the unit time (D) and the time (a_o) from beginning of net runoff to peak runoff and this explains the equation $a_o = D/2 + t$.

4. The Design Storm (see figure 8, and Table 10). From the rainfall intensity frequency data of Yarnell (5) tabulate the data on the 12 year frequency maps for Blacksburg.

Table 10

RAINFALL IN INCHES AT TIME SHOWN

TIME Minutes	5	10	15	30	60	120
RATE In. Per Hour	7.20	5.90	5.00	3.50	2.35	1.48
TOTAL RAINFALL Inches	0.60	0.98	1.25	1.75	2.35	2.96

Plot rainfall against time and connect the points by a smooth curve to give the mass rainfall curve (figure 8). The mass infiltration curve was also plotted, which is a straight line rising 1.20 inches per hour. Determine the point on the mass rainfall curve where its tangent is parallel to the infiltration curve; this gives the maximum

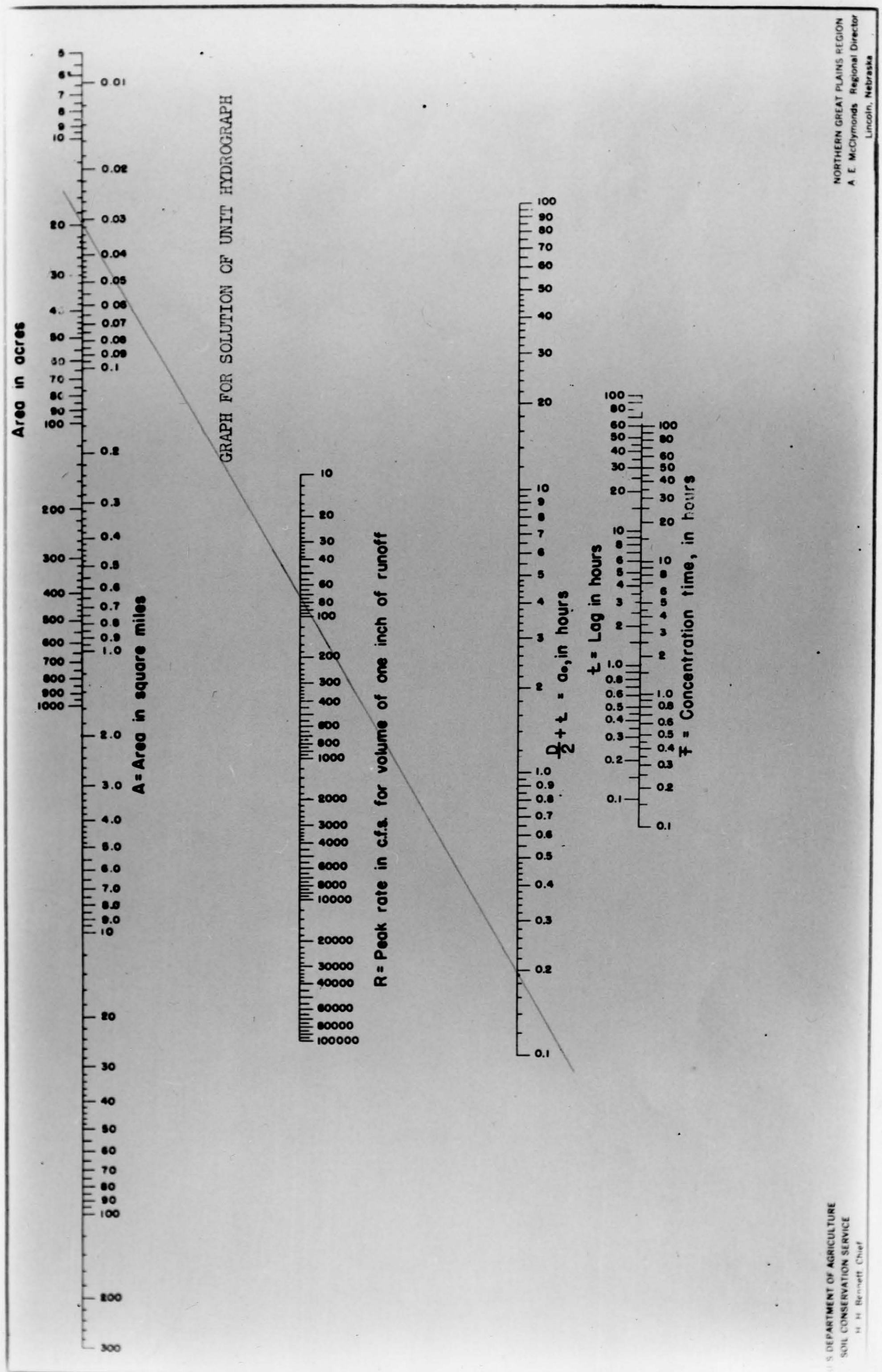
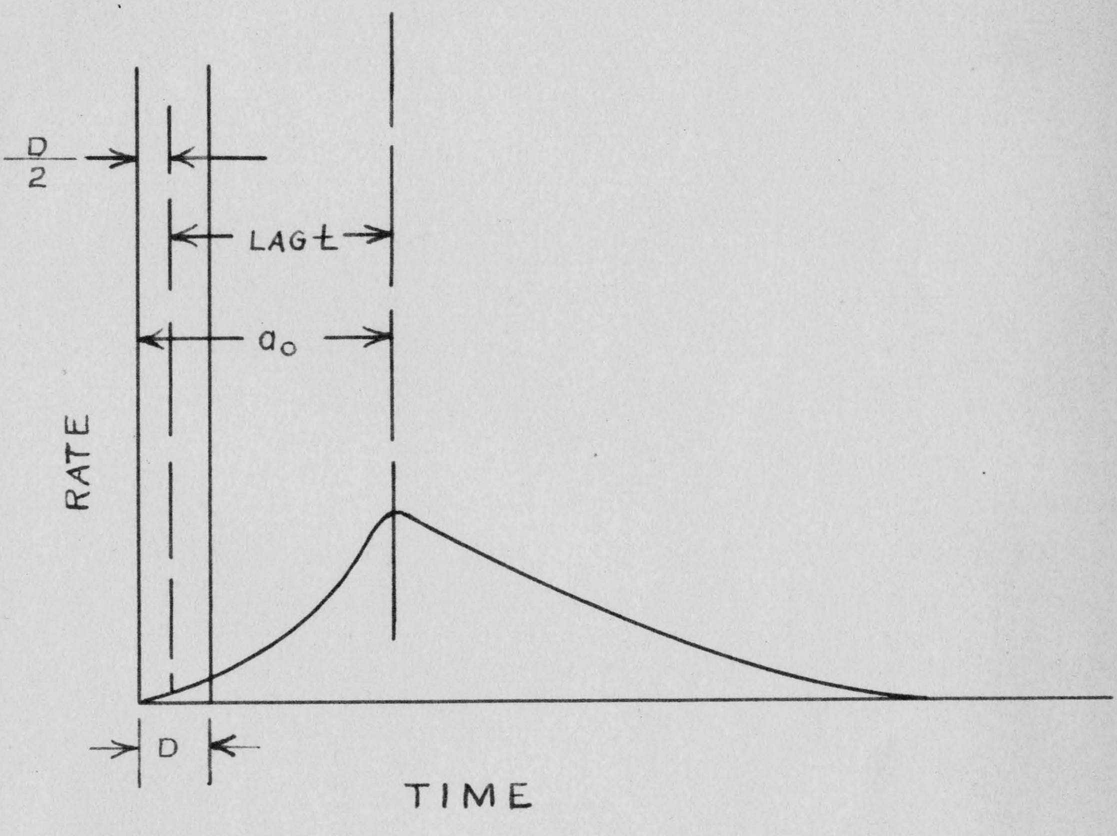
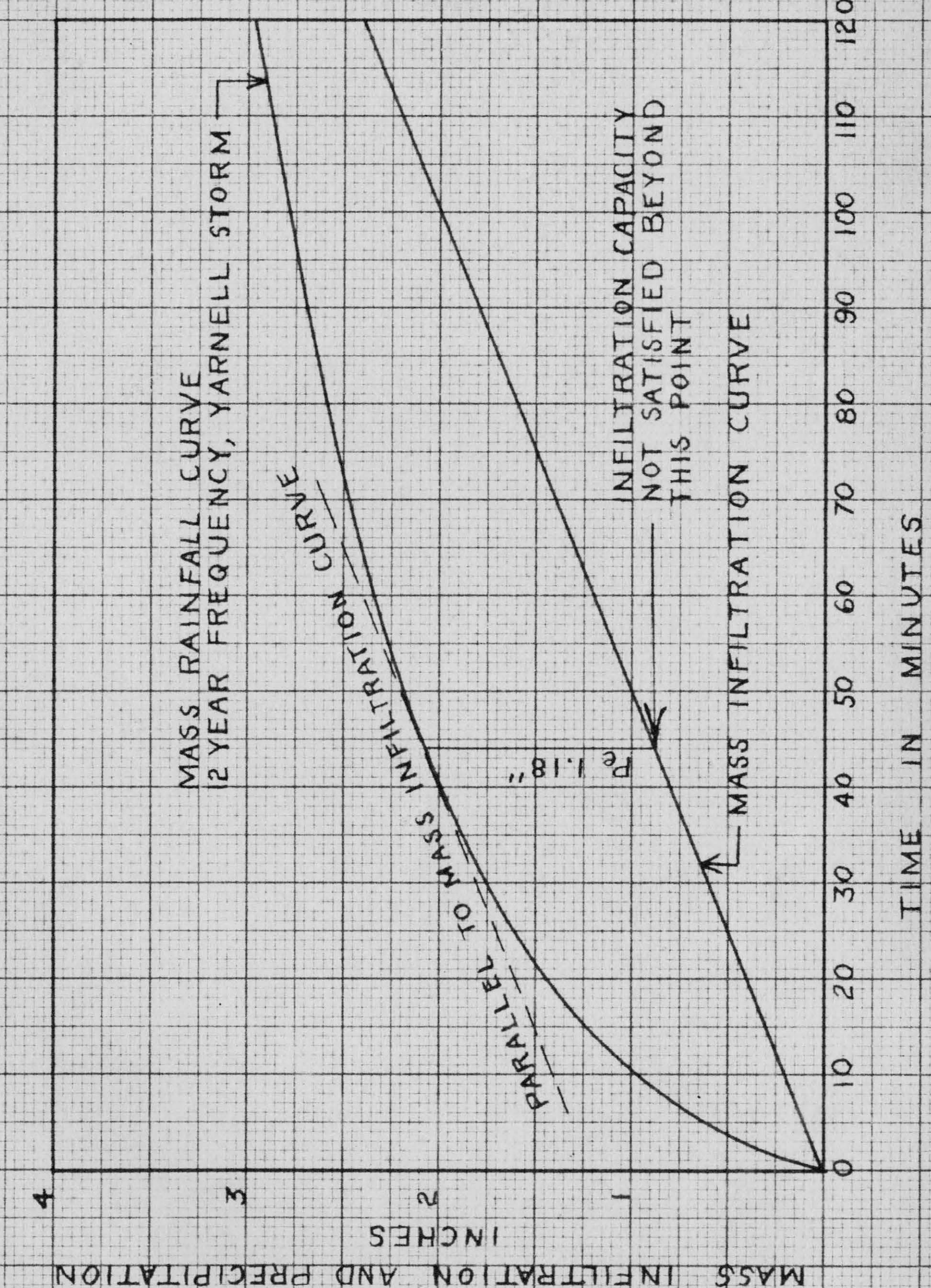

GRAPH FOR SOLUTION OF UNIT HYDROGRAPH
Area in acres
A = Area in square miles
R = Peak rate in c.f.s. for volume of one inch of runoff
D/2 + t = a_e, in hours
t = Lag in hours
T = Concentration time, in hours
NORTHERN GREAT PLAINS REGION
A. E. McClymonds Regional Director
Lincoln, Nebraska
U.S. DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE
H. H. Bennett Chief

FIGURE 6



$$a_o = LAG + \frac{D}{2}$$

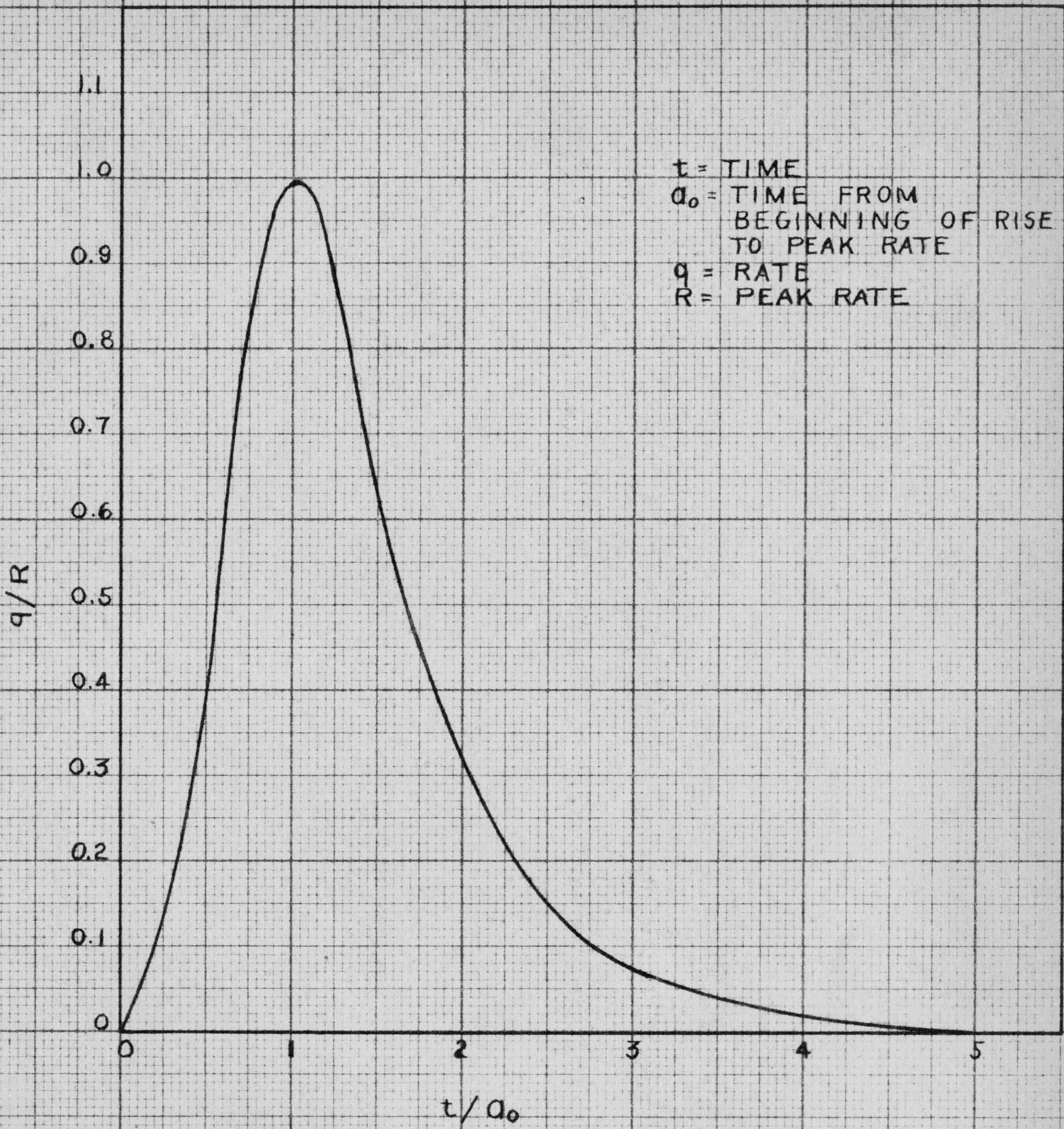
LAG AND TIME OF PEAK
FIGURE 7



MASS CURVES

RAINFALL AND INFILTRATION

FIGURE 8



DIMENSIONLESS UNIT HYDROGRAPH

FIGURE 9

net rainfall or precipitation excess P_e (rainfall minus infiltration) as 1.18 inches at 44 minutes.

5. For each 4 minute interval (i.e. at end of each unit rain period D) read corresponding rainfall and infiltration from figure 8 and tabulate in Table 11. For each such entry in the table compute rainfall minus infiltration and tabulate in the ΔP_e column Table 11.

Table 11

NET (EXCESS) RAINFALL COMPUTATION

Time in Minutes	Accumulative (Mass) Value (Inches)			ΔP_e (Excess Rainfall for each 4 min. period
	Rainfall	Infiltration	Excess Rainfall P_e	
0	0	0	0	
4	0.50	0.08	0.42	0.42
8	0.85	0.16	0.69	0.27
12	1.10	0.24	0.86	0.17
16	1.28	0.32	0.96	0.10
20	1.44	0.40	1.04	0.08
24	1.58	0.48	1.10	0.06
28	1.70	0.56	1.14	0.04
32	1.80	0.64	1.16	0.02
36	1.89	0.72	1.17	0.01
40	1.98	0.80	1.18	0.01
44	2.06	0.88	1.18	0

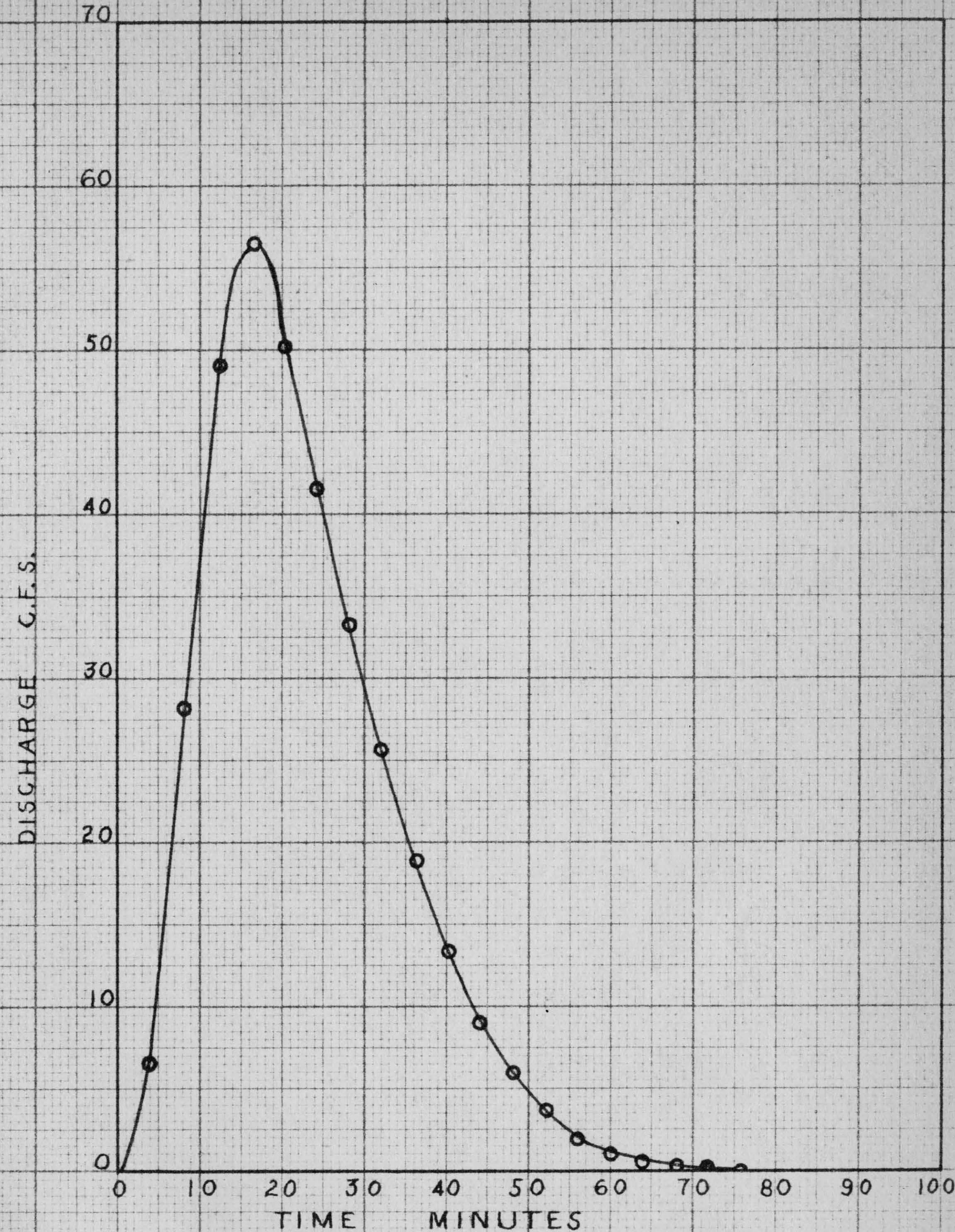
Example for findings ΔP_e: $0.42 - 0 = 0.42$
$0.69 - 0.42 = 0.27$ etc.

The ratio t/a_o was computed for each time interval in the first column of Table 12 by dividing each of the times 4, 8, 12, etc. by "a_o" or 11.6 minutes. For each of these values a corresponding q/R value from the dimensionless unit hydrograph (figure 9) was recorded in column 3, Table 12. The q/R values were multiplied by 75 from figure 6 and recorded in column 4. These values constitute the co-ordinates of points on the unit hydrograph for the W-III watershed. The ordinates of the unit hydrograph were multiplied by the corresponding net supply rate ΔP_e (from Table 11) as shown in each D Column in Table 12. The sum of each horizontal line (Table 12) of the c.f.s. values of the D columns gives the final rate c.f.s. applying to each time 4, 8, 12, etc. for the final hydrograph shown in figure 10. From figure 10 the peak runoff from W-III expected once in 12 years can be read direct as 56.87 c.f.s.

Table 12

HYDROGRAPH FOR DESIGN STORM (COMPUTATIONS)
WATERSHED W-III

"t" Time Min.	t/a_o = t/11.6	q/R from Dim. Unit Hyd.	Unit Hydrograph q (cfs) q/R x 75	Hydrograph Ordinates (cfs) Corresponding to Unit Rain Periods										Totals for Final Hydrograph
				D_1 0.42	D_2 0.27	D_3 0.17	D_4 0.10	D_5 0.08	D_6 0.06	D_7 0.04	D_8 0.02	D_9 0.01	D_{10} 0.01	
0	0	0	0											
4	0.35	0.215	16.1	6.76	0	0	0	0	0					6.76
8	0.69	0.765	57.4	24.11	4.35	0	0	0	0					28.46
12	1.04	0.995	74.6	31.33	15.50	2.74	0	0	0					49.57
16	1.38	0.770	58.0	24.36	20.14	9.76	1.61	0	0					56.87
20	1.72	0.470	35.0	14.70	15.66	12.68	5.74	1.29	0					50.07
24	2.07	0.293	22.0	9.20	9.45	9.86	7.47	4.59	0.97	0				41.53
28	2.41	0.180	13.5	5.67	5.94	5.95	5.80	5.97	3.44	0.64	0			33.41
32	2.78	0.098	7.3	3.06	3.64	3.74	3.50	4.64	4.48	2.30	0.32	0		25.68
36	3.10	0.066	4.9	2.06	1.97	2.29	2.20	2.80	3.48	2.98	1.15	0.16	0	19.09
40	3.45	0.040	3.0	1.26	1.32	1.24	1.35	1.76	2.10	2.32	1.49	0.57	0.16	13.57
44	3.79	0.023	1.7	0.71	0.81	0.83	0.73	1.08	1.32	1.40	1.16	0.75	0.57	9.36
48	4.14	0.015	1.1	0.46	0.46	0.51	0.49	0.58	0.81	0.88	0.70	0.58	0.75	6.22
52	4.48	0.010	0.7	0.29	0.30	0.29	0.30	0.39	0.44	0.54	0.44	0.35	0.58	3.92
56	4.83	0.005	0.4	0.17	0.19	0.19	0.17	0.24	0.29	0.29	0.27	0.22	0.35	2.38
60					0.11	0.12	0.11	0.14	0.18	0.20	0.14	0.14	0.22	1.36
64						0.07	0.07	0.09	0.10	0.12	0.10	0.07	0.14	0.76
68							0.04	0.06	0.07	0.07	0.06	0.05	0.07	0.42
72								0.03	0.04	0.04	0.04	0.03	0.05	0.23
76									0.02	0.03	0.02	0.02	0.03	0.12
80										0.02	0.01	0.01	0.02	0.06
84											0.01	0.01	0.01	0.03
88												0	0.01	0.01
92													0	0



FINAL HYDROGRAPH

WATERSHED W III

FIGURE 10

APPENDIX B

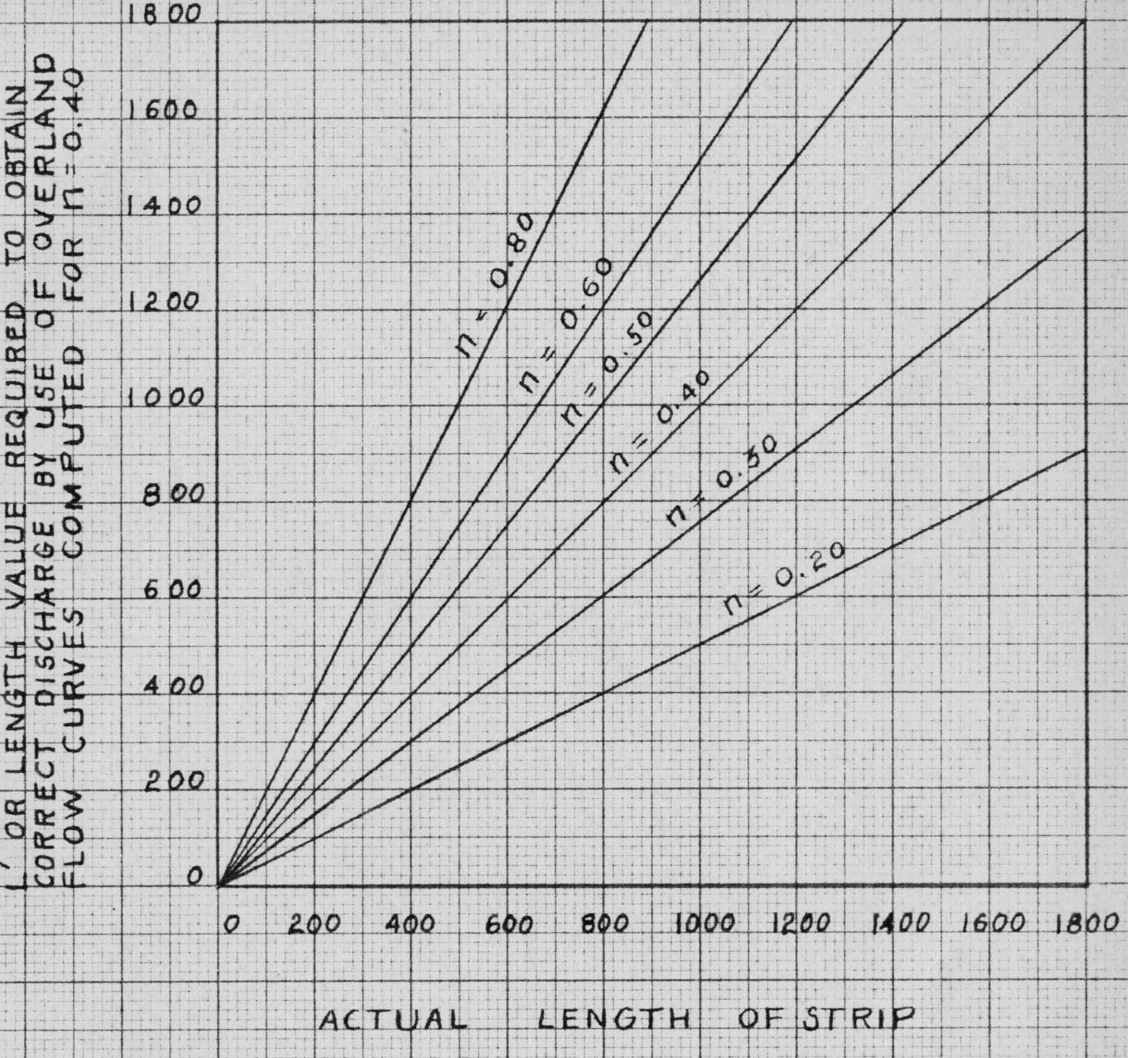
SOLUTION OF THE ARMY METHOD

The Corps of Engineers had designed four sets of charts for the solution of the equation $q = \sigma \tanh^2 \left[0.922\ t\ (\sigma/nL)^{0.50}\ S^{0.25}\right]$. The following procedure is suggested for the solution of the equation:

1. Prepare topographical maps and profiles of the waterways.
2. Determine the drainage area by use of planimeters.
3. Assign "n" values to various areas.
4. Estimate the average slope of overland flow.
5. Determine the effective length of the flow. All charts are based on "n" of 0.40 and "S" of 1 percent, therefore a chart is provided to convert all "n's" and all "S's" to the standard.
6. Design storm is determined from one hour rainfall intensity frequency data.
7. Infiltration is obtained by infiltrometer.
8. Supply is rainfall minus infiltration.
9. "q" is determined from chart figure 13. (There is a chart for 16 supply rates.)

Example Watershed W-III:

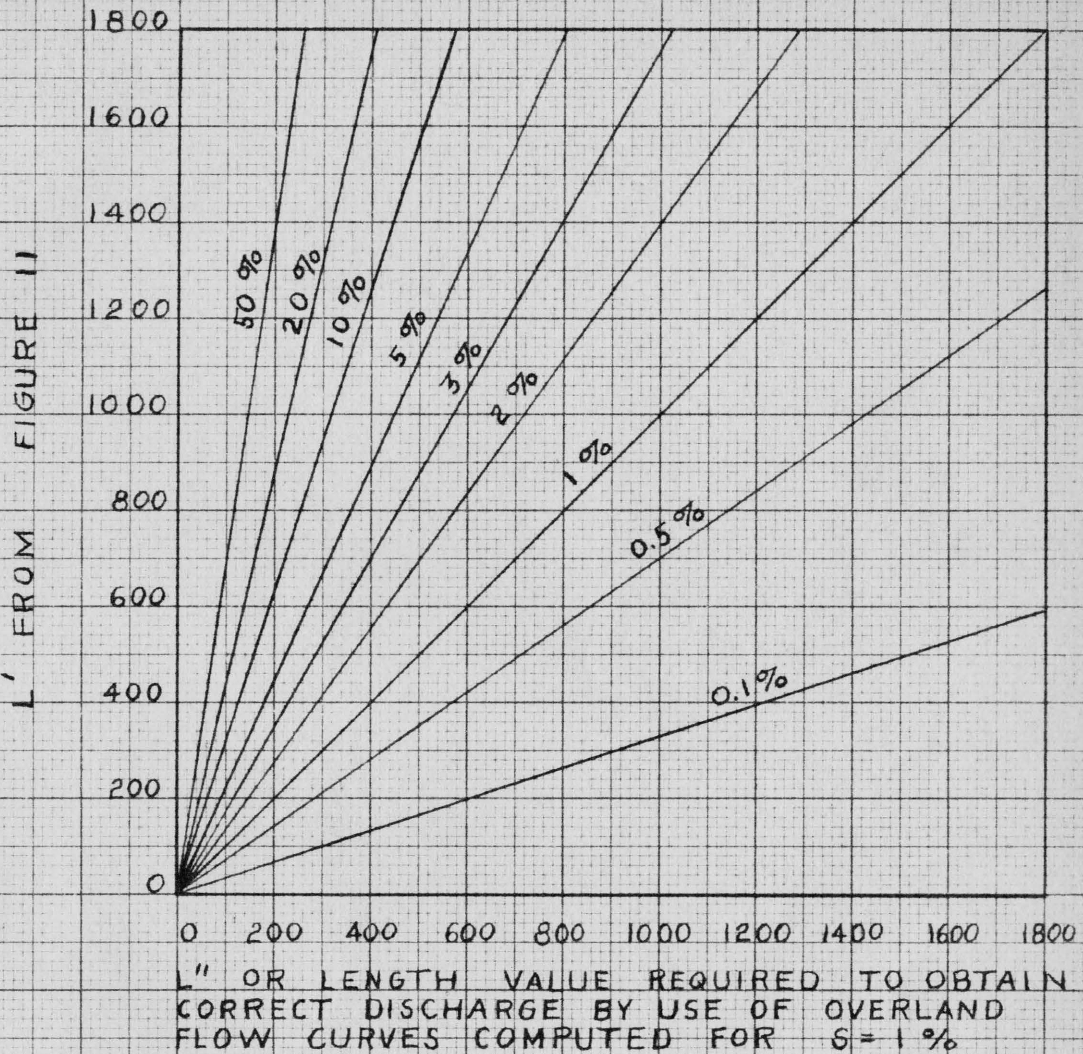
Area = 19 acres
n = 0.40
Prevailing slope = 6 percent
Length of waterway L = 1400 feet
L' = 1400 feet from figure 11
L" = 600 feet from figure 12
Rainfall 2.6" for one hour
Infiltration 1.20" per hour
Supply 2.6 - 1.20 = 1.40
Therefore use supply curve 1.4 (figure 13)
Peak rate = 1.2 x 19.3 = 22.8 c.f.s.



AIRFIELD DRAINAGE
OVERLAND FLOW RELATIONS

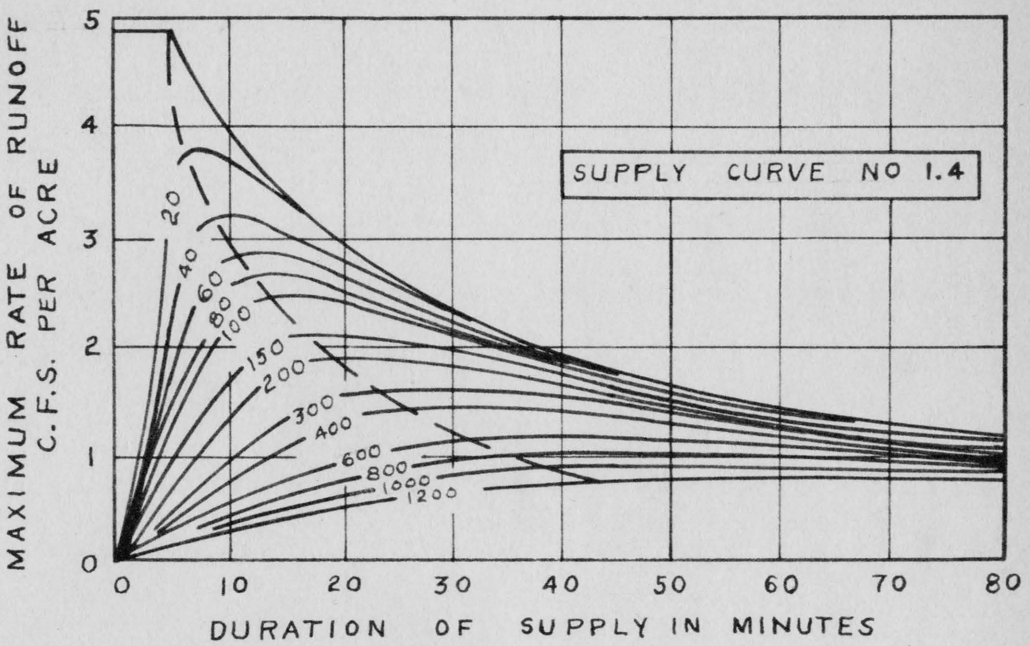
CORRECTION FOR DIFFERENCES IN RETARDENCE COEFFICIENT n

FIGURE 11



AIRFIELD DRAINAGE
OVERLAND FLOW RELATIONS
CORRECTION FOR DIFFERENCES IN SLOPE S

FIGURE 12



RATE OF OVERLAND FLOW CORRESPONDING TO STANDARD SUPPLY CURVES $n = 0.40$ $S = 1\%$

L = EFFECTIVE LENGTH IN FEET
σ = RATE OF SUPPLY DEPTH PER HOUR IN INCHES
t_c = CRITICAL TIME OF RUNOFF IN MINUTES

AIRFIELD DRAINAGE
RATE OF OVERLAND FLOW
FIGURE 13

APPENDIX C

MAXIMUM RAINFALL FOR BLACKSBURG AND WYTHEVILLE

The intensity and duration of rainfall probably has more effect on the maximum runoff than any other variable that has to be considered. It is obvious that for paved surfaces Q = I except for detention and channel storage. In most systems of predicting runoff the intensity of rainfall in inches per hour will have to be estimated. Yarnell (5) presents rainfall intensity data for the entire country. These data are almost universally used where local data are not available. An estimate of the expected intensity of rainfall can be determined by Sherman's method (32) which for any given location should be more accurate than that given by Yarnell. The data are plotted on log paper after which the estimates are read directly from the chart.

Sherman reasoned that if any given record represented a fair average for that period then the highest rainfall could be taken as the maximum rainfall for the period. The rainfall of second magnitude could be expected to be equalled or exceeded twice in the period of record, and the third to be equalled or exceeded three times in the period, etc.

It was also reasoned that the highest intensity may be the highest for a much longer period of time than the period of record. For instance, if the period is 50 years then a 100 or a 500 year record intensity may be recorded. They must fall in some 50 year period, likewise it is possible that some of the maximum intensities may be lower than the long time records might indicate. Consquently, it is obviously advisable to treat records of limited duration with due caution and skill.

The maximum rainfall intensities for the periods indicated in Table 13 were recorded from 1937 to 1955 in connection with the runoff studies on watersheds W-II and W-III. The highest intensities for each time interval were not used. The second highest were assumed to be the maximum rainfall occurring twice in 19 years or once in nine and one-half years, the fourth line maximum once in five years, and the sixth line once in three years. These data (Table 13) were plotted as curves 1, 2, and 3 (figure 14).

The empirical equation of best fit for these curves is $i = K_1/t^{0.54}$, where $K_1 = 9.1\ F^{0.29}$. Therefore $i = 9.1\ F^{0.29}/t^{0.54}$, when "i" is inches per hour, "t" is duration of rain in minutes, and "F" is frequency in years. Using the above formula one can extrapolate to any desired frequency. However, it is always dangerous to go far beyond the length of time covered by the data. As an illustration of how this may be done, the 25 year and 50 year expectancies for 5 and 30 minutes are computed below and the results plotted as curves 4 and 5 on figure 14.

<u>To Obtain the 25 Year Expectancy</u>

For 5 minutes $i = \frac{9.1(25)^{0.29}}{(5)^{0.54}} = \frac{9.1(2.54)}{2.35} = 9.82$

For 30 minutes $i = \frac{9.1(25)^{0.29}}{(30)^{0.54}} = \frac{9.1(2.54)}{6.06} = 3.80$

To Obtain the 50 Year Expectancy

For 5 minutes $i = \frac{9.1(50)^{0.29}}{(5)^{0.54}} = 11.65$

For 30 minutes $i = \frac{9.1(50)^{0.29}}{(30)^{0.54}} = 4.51$

Again one is cautioned that the 50 year return period should be used with caution because of the uncertainty connected with extrapolating from 19 to 50 years.

Sherman (32) found that the equation $i = K/(t + a)^d$ was the best fit for Boston data. By adding from 9 to 11 minutes to it he found for the Boston area the curve would result in more nearly a straight line. Numerous other investigators have produced formulas for data of a specific locality. The usual formula is of the form $i = k/t^d$, $i = K/t + a$, $i = Kt/t + a$, $i = K/(t + a)^d$, or $i = a/t + b$.

The data from Table 14, "Record of Rainfall Intensity in Inches Per Hour from Wytheville, Virginia", for a period of 52 years were treated in the same manner as the Blacksburg data with the results shown in figure 15.

The curve becomes more nearly a straight line when five is added to all "t's" resulting in an equation of the general form $i = K_1/(t + a)^d$. For the Wytheville data the equation becomes $i = 7.7\ F^{0.45}/(t + 5)^{0.65}$.

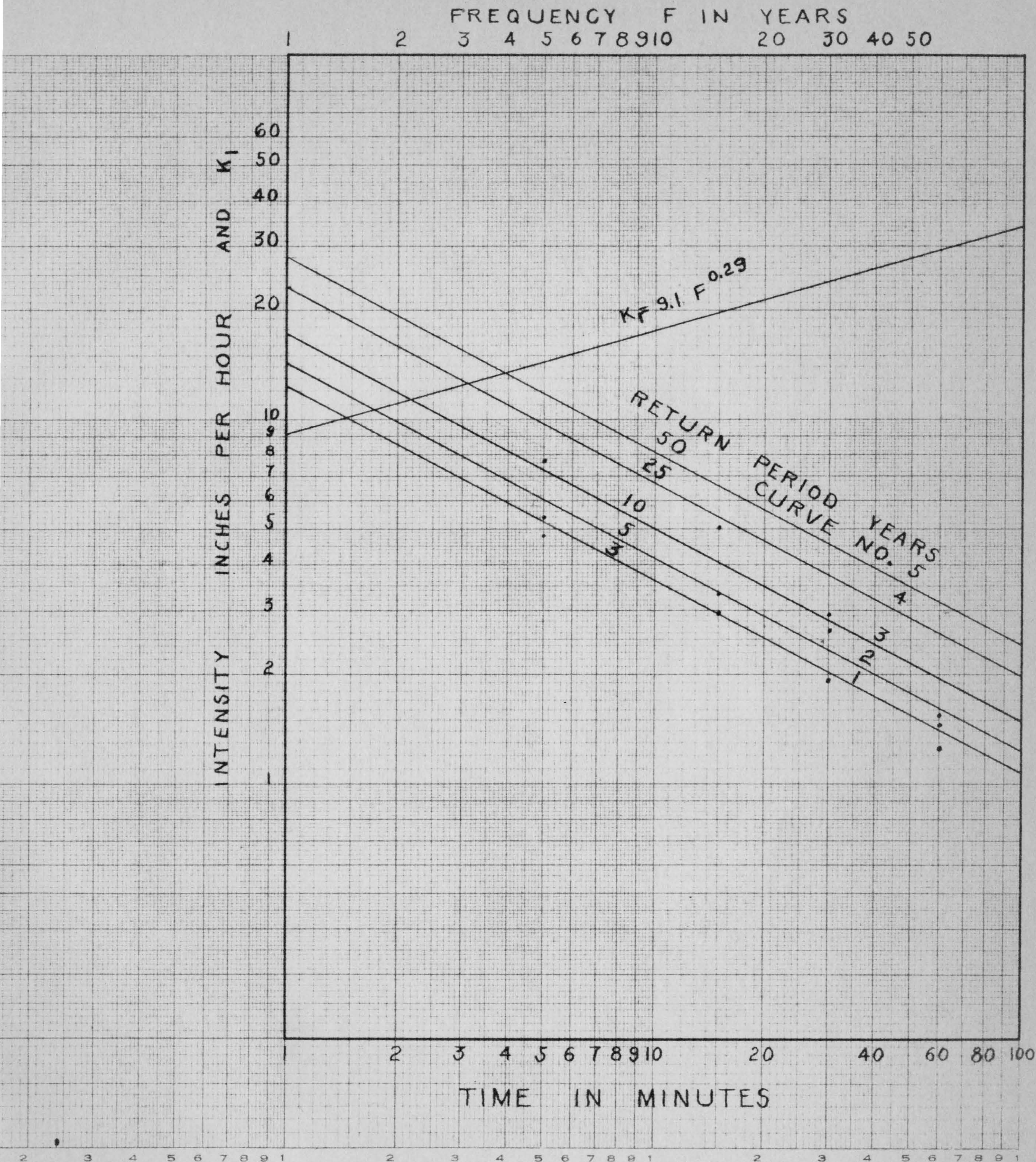
Table 13

MAXIMUM RAINFALL IN BLACKSBURG, VIRGINIA*

1937 THROUGH 1955

5 Minutes	15 Minutes	30 Minutes	60 Minutes
8.40	5.12	3.08	1.62
7.68	5.00	2.90	1.55
6.00	3.60	2.74	1.54
5.40	3.32	2.64	1.47
5.40	3.24	2.42	1.44
5.28	3.20	2.16	1.37
4.80	2.96	1.96	1.26
4.80	2.96	1.96	1.04
4.56	2.88	1.88	1.01
4.56	2.76	1.86	1.00

* From Soil Conservation Service records of Watershed W-II, Blacksburg, Virginia.



RAINFALL INTENSITY CURVES
BLACKSBURG VIRGINIA
FIGURE 14

__Rainfall Intensity Duration Curves:__ The rainfall intensities shown in figure 15 compare favorably with those listed for Wytheville in Technical Paper No. 25, "Rainfall Intensity-Duration-Frequency Curves". This paper supersedes Yarnell's rainfall intensity curves. Paper No. 25 is based on data from 1903 through 1940. The Wytheveille curves are appreciably lower than the curves derived by the author for Blacksburg. The Blacksburg data available covered the period 1937 through 1955. Some weight should be given to the longer period of the Wytheville data, but it is the opinion of the author that the intensity and duration of rainfall in Blacksburg is higher than in Wytheville as indicated by figures 14 and 15.

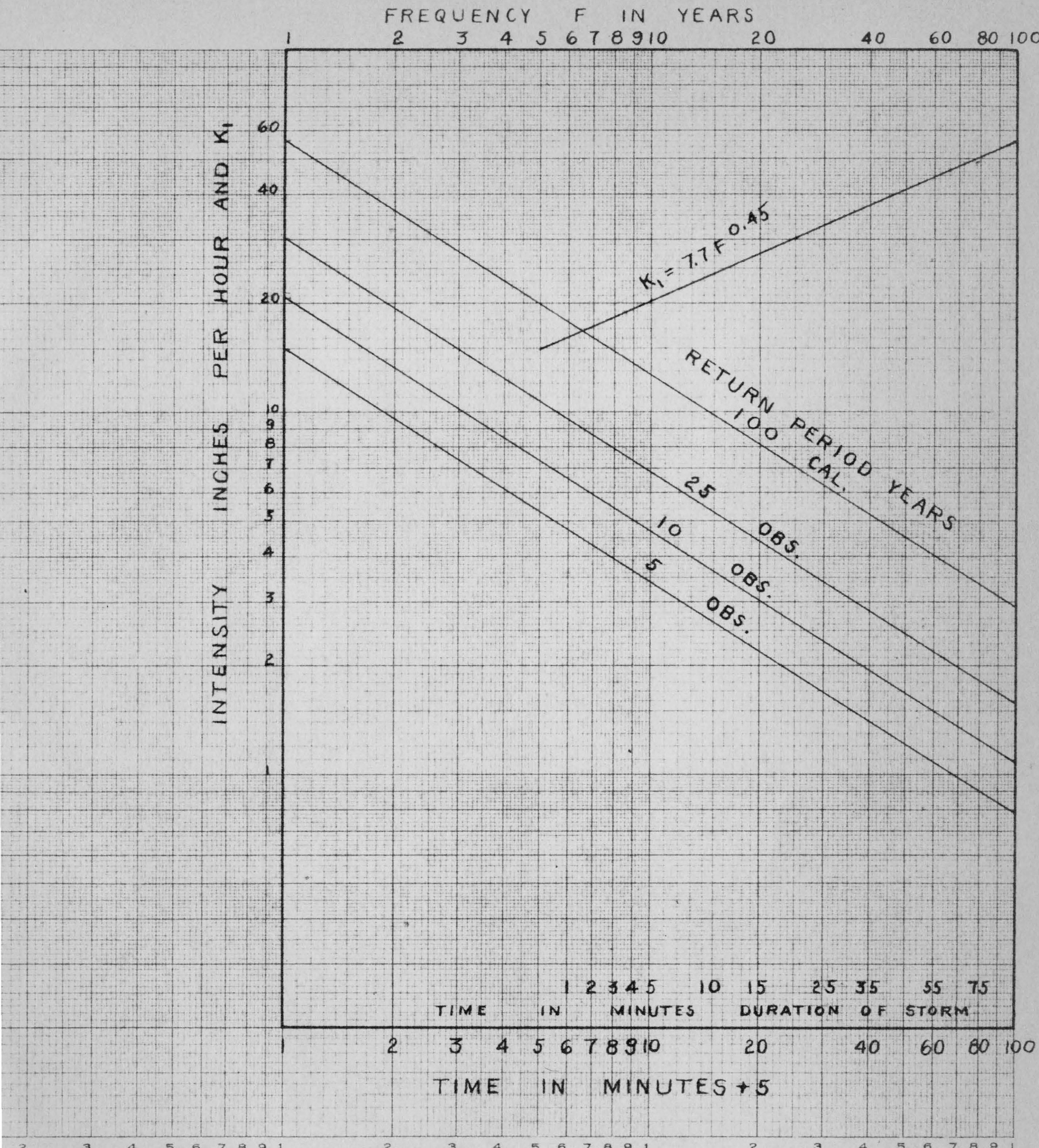
Table 14

RECORD OF RAINFALL INTENSITIES IN INCHES PER HOUR

Wytheville, Virginia

1903 - 1955

Duration of Storm in Minutes									
5	10	15	20	25	30	35	40	45	50
7.81	6.50	5.55	4.71	4.20	3.85	3.50	3.10	2.90	2.60
6.50	5.51	4.55	3.95	3.75	3.30	2.90	2.80	2.50	2.15
5.91	4.81	4.00	3.54	3.12	2.80	2.50	2.30	2.05	1.85
5.55	4.32	3.60	3.00	2.81	2.62	2.25	1.99	1.95	1.75
5.10	3.91	3.38	2.87	2.42	2.20	2.00	1.97	1.90	1.50
4.60	3.75	3.30	2.70	2.38	2.00	1.90	1.75	1.60	1.50
4.57	3.70	3.09	2.51	2.11	1.98	1.90	1.55	1.56	1.37
4.37	3.60	3.00	2.50	2.10	1.90	1.80	1.45	1.37	1.26
4.21	3.27	2.81	2.43	1.94	1.80	1.70	1.45	1.30	1.20
3.32	2.75	2.33	1.90	1.75	1.70	1.60	1.39	1.25	1.10
2.78	2.30	1.80	1.83	1.60	1.40	1.30	1.20	1.00	0.85



RAINFALL INTENSITY CURVES
WYTHEVILLE VIRGINIA
FIGURE 15

APPENDIX D

RAINFALL RUNOFF AT CARTERSVILLE, VIRGINIA

Table 15

AREAL RAINFALL AND RUNOFF OF THE JAMES RIVER
CARTERSVILLE, VIRGINIA

Year	Areal Rainfall P	Runoff R
1900	38.66	13.60
1901	54.11	24.05
1902	40.25	18.77
1903	51.91	24.70
1904	34.02	10.69
1905	41.42	13.73
1906	43.43	15.90
1907	46.13	21.74
1908	45.63	21.01
1909	37.43	18.01
1910	38.58	12.94
1911	37.16	10.81
1912	45.97	19.25
1913	42.20	15.61
1914	34.38	14.61
1915	41.63	16.90
1916	40.63	14.40
1917	36.20	13.44
1918	40.96	14.44
1919	42.06	19.20
1920	44.14	14.83
1921	33.72	13.85
1922	40.84	16.25
1923	37.50	12.04
1924	43.94	17.44
1925	27.86	12.23
1926	35.45	10.49
1927	41.40	15.01
1928	50.55	19.90
1929	33.92	16.53
1930	27.44	10.73
1931	37.73	7.84
1932	30.10	10.03
1933	47.24	19.37
1934	38.33	9.90
1935	49.82	21.41

$\Sigma P = 1452.74$

$\Sigma P^2 = 60,002.15$

$\Sigma R = 561.65$

$\Sigma R^2 = 9,370.40$

$\Sigma RP = 23,401.43$

RAINFALL RUNOFF AT CARTERSVILLE
SOLUTION BY METHOD OF AVERAGES

Year	Equation	Year	Equation
1900	38.66 = 13.60 m + b	1901	54.11 = 24.05 m + b
1902	40.25 = 18.77 m + b	1903	51.91 = 24.70 m + b
1904	34.02 = 10.69 m + b	1905	41.42 = 13.73 m + b
1906	43.43 = 15.90 m + b	1907	46.13 = 21.74 m + b
1908	45.63 = 21.01 m + b	1909	37.43 = 18.01 m + b
1910	38.58 = 12.94 m + b	1911	37.16 = 10.81 m + b
1912	45.97 = 19.25 m + b	1913	42.20 = 15.61 m + b
1914	34.38 = 14.61 m + b	1915	41.63 = 16.90 m + b
1916	40.63 = 14.40 m + b	1917	36.20 = 13.44 m + b
1918	40.96 = 14.44 m + b	1919	42.06 = 19.20 m + b
1920	44.14 = 14.83 m + b	1921	33.72 = 13.85 m + b
1922	40.84 = 16.25 m + b	1923	37.50 = 12.04 m + b
1924	43.94 = 17.44 m + b	1925	27.86 = 12.23 m + b
1926	35.45 = 10.49 m + b	1927	41.40 = 15.01 m + b
1928	50.55 = 19.90 m + b	1929	33.92 = 16.53 m + b
1930	27.44 = 10.73 m + b	1931	37.73 = 7.84 m + b
1932	30.10 = 10.03 m + b	1933	47.24 = 19.37 m + b
1934	38.33 = 9.90 m + b	1935	49.82 = 21.41 m + b
	713.30 = 265.18 m + 18b		739.44 = 296.47 m + 18b

$$\begin{array}{r} 739.44 = 296.47\ m + 18b \\ -713.30 = -265.18\ m - 18b \\ \hline 26.14 = 31.29\ m \end{array}$$

$$m = 0.835$$

$$739.44 = 296.47\ (0.835) + 18b$$

$$b = 27.25$$

$X = 0.835\ Y + 27.25$

$P = 0.835\ R + 27.25$

$R = 1.2\ P - 32.6$

When $P = 30$ $R = 3.4$

When $P = 50$ $R = 27.4$

SOLUTION BY METHOD OF LEAST SQUARES

$\Sigma P = 1452.74$

$\Sigma R = 561.65$

$\Sigma P^2 = 60{,}002.15$

$\Sigma R^2 = 9{,}970.40$

$\Sigma RP = 23{,}401.43$

$$\Sigma R = m\Sigma P + nb$$

$$\Sigma PR = m\Sigma P^2 + b\Sigma P$$

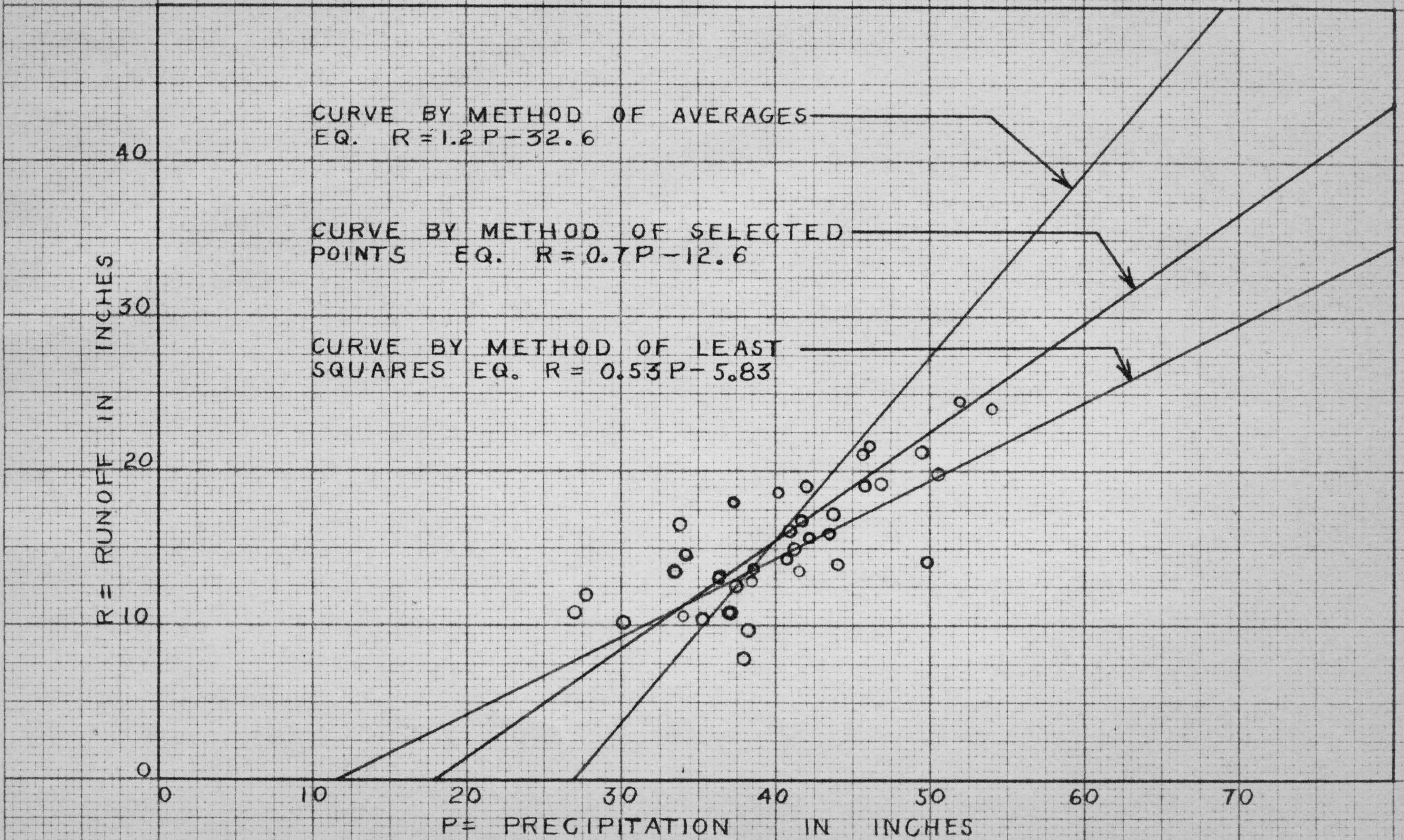
$$561.65 = m\ 1452.74 + 36b$$

$$23{,}401.43 = m\ 60002.15 + 1452.74b$$

$m = 0.53$

$b = 5.83$

Therefore $R = 0.53\ P - 5.83$



ANNUAL RAINFALL RUNOFF RELATIONSHIP
DATA FROM JAMES RIVER AT CARTERSVILLE VA.

FIGURE 16

SOLUTION OF DATA FROM WATERSHED W-III BY METHOD OF AVERAGES

Year	Equation	Year	Equation
1944	$35.24m = 0.3248 + b$	1945	$39.36m = 0.0166 + b$
1946	$33.80m = 0.0438 + b$	1947	$41.82m = 0.3258 + b$
1948	$49.00m = 0.5479 + b$	1949	$46.29m = 0.5477 + b$
1950	$35.48m = 0.1301 + b$	1951	$35.08m = 0.0285 + b$
1952	$39.52m = 0.0316 + b$	1953	$32.41m = 0.0184 + b$
1954	$32.56m = 0.0035 + b$	1955	$29.54m = 0.0116 + b$
	$225.60m = 1.0817 \quad 6b$		$224.50m = 0.9486 \quad 6b$

$$225.60m = 1.0817 + 6b$$

$$-224.50m = 0.9486 - 6b$$

$$1.10m = 0.1331$$

$$m = 0.121$$

$$b = -4.36$$

$$R = 0.121 (P) - 4.36$$

(see figure 4)

SOLUTION OF DATA FROM WATERSHED W-III BY METHOD OF LEAST SQUARES

$$\Sigma R = m \Sigma P + nb$$

$$\Sigma PR = m \Sigma P^2 + b \Sigma P$$

n = number observations = 12

$$\Sigma P = 450.1$$

$$\Sigma P^2 = 17260$$

$$\Sigma R = 2.03$$

$$\Sigma (R)^2 = 0.833$$

$$\Sigma (RP) = 87.32$$

$$2.03 = m\,450.1 + 12b$$

$$87.32 = m\,17260 + 450.1b$$

$$\begin{array}{rl} 87.32 = & m\,17260 + 450.1b \\ -77.84 = & -m\,17260 - 460.2b \\ \hline 9.48 = & \qquad\qquad -10.1\,b \end{array}$$

$$b = \frac{9.48}{10.1} = -0.94$$

$$2.03 = m\,450.1 + 12\,(-0.94)$$

$$m = \frac{2.03 + 11.28}{450.1} = \frac{13.31}{450.1}$$

$$m = 0.029$$

$$R = 0.029\,P - 0.94$$

when $R = 0$, $P = 32.4$

(see figure 4)

APPENDIX E

TIME OF CONCENTRATION AS AFFECTED BY WATERSHED SIZE

Table 16

TIME OF CONCENTRATION AS AFFECTED
BY WATERSHED SIZE (33)

Size of Watershed Acres	Time of Minimum Concentration Minutes
1	1.4
3	3.0
5	3.5
10	4.0
20	4.8
30	8.0
50	12.0
100	17.0
200	23.0
300	29.0
400	35.0
500	41.0
600	47.0
700	53.0
800	60.0
900	67.0
1000	75.0

APPENDIX F

EXPECTED WATER YIELD FROM RIDGES AND VALLEYS SECTION OF VIRGINIA

Table 17

MINIMUM RUNOFF, IN ACRE-INCHES PER ACRE OF WATERSHED, TO BE EXPECTED FOR CONSECUTIVE PERIODS UNDER VARIOUS CROPPING CONDITIONS

Watershed Cropping Conditions	Minimum Runoff Expectancy For Length of Period			
	1 yr.	2 yr.	3 yr.	4 yr.
	Acre-in. per acre	Acre-in. per acre	Acre-in. per acre	Acre-in. per acre
Pasture (natural bluegrass)				
Fertilized, moderately heavy grazed, 13% slope	0.05004	0.47004	0.78000	2.13000
Untreated, 13% slope	0.03000	0.24000	0.50004	1.61004
Fertilized, extremely heavy grazed, 23% slope	0.41004	0.84000	2.40996	5.16996
Poor, weedy, 23% slope	0.42996	1.07004	2.73996	5.55000
Corn, wheat, clover rotation				
Contour strip cropped	0.02004	0.12000	0.36000	0.72000
Single crop each year	0	0.03000	1.07004	2.45004
1/3 of watershed in each crop but not contour strip cropped	0.08004	0.51996	2.16000	3.80004

APPENDIX G

SOLUTION OF RAINFALL RUNOFF PROBABILITY CURVES

Table 18

CALCULATIONS FOR CURVE 2 FIGURE 5

X	CLASS MEAN	FREQUENCY F	FX	DEV. FROM ARB. ORIGIN Y	FY	FY^2
0 - .001	.0005	6	.0030	-4	-24	96
.001 - .002	.0015	1	.0015	-3	- 3	9
.002 - .003	.0025	0	0	-2	0	0
.003 - .004	.0035	1	.0035	-1	- 1	1
.004 - .005	.0045	0	0	0	0	0
.005 - .006	.0055	0	0	1	0	0
.006 - .007	.0065	0	0	2	0	0
.007 - .008	.0075	1	.0075	3	3	9
.008 - .009	.0085	0	0	4	0	0
.009 - .010	.0095	1	.0095	5	5	25
.010 - .011	.0105	0	0	6	0	0
.011 - .012	.0115	2	.0220	7	14	98
Total		12	.0470		- 6	238

$$\sigma = \sqrt{\frac{FY^2}{N}\, i^2 - \left(\frac{FY}{N}\right)^2 i^2 - \frac{i^2}{N}}$$

$$\sigma = \sqrt{\frac{238}{12}\,(.001)^2 - \left(\frac{-6}{12}\right)^2 (.001)^2 - \frac{(.001)^2}{12}}$$

$$\sigma = .0066$$

Line of best fit passes through mean at 50% and mean $\pm\sigma$ at 84.13% and 15.87%.

PREDICTING WATER YIELD BY USE OF FIGURE 5

To use figure 5 let it be required to determine the minimum flow once in four years. From curve 1 opposite 25 percent of time, read log. Coefficient of runoff = 6.73 - 10. From curve 3 read log rainfall = 1.55. Therefore expected annual runoff is 36.27 x 0.0006 = 0.021 inches.