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UNIVERSITY OF ILLINOIS ENGINEERING EXPERIMENT STATION BULLETIN NO. 462

HYDROLOGIC DETERMINATION OF WATERWAY AREAS FOR THE DESIGN OF DRAINAGE STRUCTURES IN SMALL DRAINAGE BASINS

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ABSTRACT

This study contains a scientific, simple, and practical method to determine the peak discharge of flow from small rural drainage basins for the design of waterway openings of minor drainage structures such as culverts and small bridges. For practical applications of the method, a design chart for climatic and physiographic conditions in Illinois is presented.

Major phases of the study include a historical review of engineering studies and methods of waterway area determination, a survey of design practice in different state highway agencies in the United States, a collection and analysis of available hydrologic data for the State of Illinois, the development of a method for waterway area determination, a simplification of the developed method, a compilation of formulas for waterway area determination, and an annotated supplementary bibliography.

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On the part of the University, the work covered by this report was carried out under the general administrative supervision of W. L. Everitt, Dean of the College of Engineering, Ross J. Martin, Director of the Engineering Experiment Station, N. M. Newmark, Head of the Department of Civil Engineering and Professor of Civil Engineering, and Ellis Danner, Director of the Illinois Cooperative Highway Research and Professor of Highway Engineering.

On the part of the Division of Highways of the State of Illinois, the work was under the administrative direction of R. R. Bartelsmeyer, Chief Highway Engineer, Theodore F. Morf, Engineer of Research and Planning, and W. E. Chastain, Sr., Engineer of Physical Research.

Technical advice was provided by a Project Advisory Committee consisting of the following personnel:

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NOMENCLATURE

(Notation used in Appendix I not included)

A = Waterway area in square feet; or drainage area in acres or in square miles.

a = Waterway area in square feet.

$a_1, a_2, a_3, \ldots$ = Subdivided drainage areas in acres.

b = A constant.

C = A coefficient; a runoff coefficient depending on characteristics of drainage areas.

D = drainage area in acres or in square miles; time interval in hours.

$D.D.$ = Drainage density, or total length of visible channels per unit drainage area, in feet per acre.

F = A frequency factor.

FF = A frequency factor.

H = Fall of the drainage basin from the farthest point on the watershed to the outlet of runoff, in feet.

I = Rainfall intensity in inches per hour.

K = A coefficient; a lag-time factor equal to $L/\sqrt{S}$.

K_o = A lag-time factor equal to $A^{0.3}/S_o\sqrt{D.D.}$

k = A physiographic factor.

L = Length of stream or of drainage basin in miles or in feet.

L_o = Total length of visible channels in a drainage basin in feet.

LF = A land use and slope factor.

M = Drainage area in square miles.

m = An exponent.

N = Runoff number or hydrologic soil-cover complex number.

n = An exponent.

P = Peak discharge in cubic feet per second (c.f.s.); peak discharge of a unit hydrograph.

p = Numerical percentage rating on the Myers scale.

Q = Discharge in c.f.s.; 10-year discharge in c.f.s.; direct runoff in inches.

Q_b = Base flow in c.f.s.

Q_d = Computed design peak discharge in c.f.s.

Q_{max} = Maximum peak discharge in c.f.s.

q = Discharge in c.f.s. per acre in the Bürkli-Ziegler formula.

q_p = Peak discharge in c.f.s.

R = Rainfall or rainfall excess in inches; a rainfall factor.

R_e = Rainfall excess or direct runoff in inches.

R_{eu} = Rainfall excess in inches at Urbana, Illinois.

R_u = Rainfall in inches at Urbana, Illinois.

RF = A rainfall factor.

S = Average ground or channel slope in feet per 1,000 feet, in feet per foot, or in per cent.

S_o = Average land slope of watershed in per cent.

T = A slope factor.

T_b = Base time of a triangular hydrograph in hours.

T_c = Time of concentration in hours.

T_p = Time from beginning of direct runoff to peak flow in hours.

t = Duration of rainfall or rainfall excess in minutes or in hours.

t_c = Time of concentration in hours.

t_c' = Time of concentration in hours for sub-areas.

t_h = Time in hours since rainfall excess began.

t_o = Lag or the time interval in hours from center of mass of rainfall excess to center of mass of runoff.

t_p = Lag time in hours; the time interval from center of mass of rainfall to the resulting runoff peak; the time of rise of the peak flow in an instantaneous unit hydrograph.

V = Total volume of runoff in acre-feet.

W = A rainfall factor; weight.

X = A runoff factor equal to R_{eu}/t; difference in elevation in feet of a stream-bed at the culvert site and at $0.7L$ upstream, in which L is the length of the stream.

x = Part of length of drainage basin in miles for sub-areas.

Y = A climatic factor equal to $1.008R/R_u$; difference in elevation in feet of a stream-bed at $0.7L$ upstream and at the headwater, in which L is the length of the stream.

Z = A peak-reduction factor equal to $Pt/1.008A$.

I. INTRODUCTION

A. SIGNIFICANCE AND PURPOSE OF THE STUDY

It is estimated that over 15% of the total cost of the development of modern highways across the country is spent on the construction and maintenance of minor drainage structures such as culverts and small bridges. In view of this high percentage of expenditure there is great need for improvement in the design method for economical determinations of the water-carrying capacity of these structures.

Current methods employed by most highway engineers for the determination of waterway areas involve the use of empirical approaches such as the Talbot formula. Such approaches do not generally encourage consideration of the many significant factors involved in a given problem, but cause these factors to be treated in a lump, usually by means of a coefficient. In the use of an empirical approach, moreover, there is danger that the limitations are often overlooked or ignored. Some judgment and experience are therefore necessary for the application of the empirical method, particularly in the selection of the proper coefficients. Also, an inexperienced designer using the empirical approach has a tendency to overdesign the structure.

The purpose of the present study is to develop a simple but scientific procedure for the use of engineers in establishing economical and adequate waterway areas of small drainage structures. The procedure would not rely so heavily on the judgment of the designer as does an empirical approach. It could therefore be used easily by relatively inexperienced designers. The method developed can be applied to any area for which hydrologic data are available. The procedure presented is based on data for Illinois and is therefore especially applicable to this state.

The method thus developed should be useful not only to highway engineers, but also to railroad and agricultural engineers who design drainage structures, to practicing hydraulic engineers, and to hydrologists dealing with small rural drainage basins.

B. DEVELOPMENT OF THE STUDY

This study was started in the fall of 1952. Engineers of the Illinois Division of Highways were not satisfied with existing methods (based on empirical formulas) of designing the waterway openings of culverts and other drainage structures. They recommended a thorough analytical investigation of the problem by the Department of Civil Engineering as one phase of the Illinois Cooperative Highway Research Program at the University of Illinois. By 1957 the Bureau of Public Roads had become interested in the study and was participating in the project.

In the beginning, a compilation was made of existing formulas and available literature related to the subject. It was followed by a nation-wide survey of drainage structure design practices adopted by different state highway agencies. A critical review of existing methods for the determination of waterway areas was then made. In the meantime work was also continued in two directions: one was the collection of hydrologic data and their analysis and the other was the exploration of available analytical methods for hydrologic analysis. The outcome of this investigation, which considered all significant hydrologic factors involved in the problem, was the development of a procedure for the determination of design peak discharge of small drainage basins for the design of waterway openings. This procedure was proposed as a practical solution to the problem; it was later reconsidered for further improvement and then modified and simplified. For the use of practicing engineers, a design chart was prepared for the proposed method for the conditions in the State of Illinois.

As a result of the intensive study, seven preliminary reports were produced and submitted to the members of the Project Advisory Committee and to the sponsors of the project. After reviewing these reports, the Committee recommended that the proposed method should be made available to students of hydraulic engineering and to hydraulic engineers. At its meeting of December 8, 1960, the Committee recommended the preparation and publication of this final report summarizing the previous preliminary reports and presenting a comprehensive picture of the work done on the project.

C. SCOPE OF THE STUDY

This study deals with the determination of peak discharge from small rural drainage basins in Illinois, because such a determination is required for an economical design of waterway areas of highway culverts and small bridges.

This study consists of the following major phases:

(1) A compilation of existing formulas for waterway area determination.

(2) An extensive review of available literature on the subject and compilation of an annotated bibliography.

(3) A historical review of engineering studies and methods of waterway area determination.

(4) A survey by questionnaire of design practice employed by different state highway agencies in the United States.

(5) Collection and analysis of available hydrologic data for small rural drainage basins.

(6) Development of a scientific, simple, and practical method for the determination of waterway areas and its simplification for practical design purposes.

II. A HISTORICAL REVIEW

A. GENERAL

A survey of the literature reveals that the engineering studies on the problem of waterway area determination started as early as a century ago when the surveyor of London, John Roe, prepared a drainage table for sewer sizes and slopes in 1852. Records show that the studies in the United States began about three-quarters of a century ago, when the problem was first recognized by sewerage engineers. About 25 years later the railroad engineers began to be interested in it and continued to be so during the subsequent 30 years. Then there was a short period of recess before the highway engineers and the water and soil conservation workers started to investigate the problem. These investigations have continued over the last 40 years.

A list of historical events in the chronological development of engineering studies on waterway area determination is presented later in Section II-J. The detailed review and discussion in the following articles are more or less in close chronological order with the listed events.

B. THE MYERS FORMULA

Major E. T. C. Myers, Chief Engineer of the Richmond, Fredericksburg, and Potomac Railway shortly after the Civil War, is believed to be the first American railroad engineer to propose the use of a formula as a guide for determining waterway areas. His formula was first presented by Cleemann[1]* in a paper before the Engineers' Club of Philadelphia and was published in the Club's Proceedings in 1879.

The Myers formula appears in the following form:

$$A = C\sqrt{D} \tag{1}$$

in which A = area of waterway in square feet

D = drainage area in acres

C = coefficient recommended to be 1.0 as a minimum for flat country, 1.6 for hilly compact ground, 4.0 as a maximum for mountainous and rocky country, and higher values in exceptional cases.

* Superscript numbers in parentheses refer to "Appendix II — References Cited."

Cleemann suggested that the coefficient C be derived from careful and judicious gagings at characteristic points within the region under treatment and be applied liberally. Also, the formula should be applied only to small structures, probably because the formula results in openings which are too small for large drainage areas. The formula was found satisfactory for regions adjacent to the line of the Richmond, Fredericksburg, and Potomac Railroad in the State of Virginia. Hence, it was used widely by railroad engineers in the New England states and generally in the eastern part of the United States.

The Myers formula received many comments after its publication. Among the comments, Wellington's editorial[2] is typical:

> It is natural for fallible man to wish to reduce everything to rule, even if it be only a rule of thumb. The responsibility of the individual is much diminished if he has something of that kind to lean on, and in so doubtful a matter as the proper size of culverts, this is especially natural. It is well, however, to be certain that we are not simply making a rule where there is no rule, and so laying the foundation of future trouble, and we must confess to doubts as to whether this is not the case with the various formulas for proportioning the waterway for culverts . . . when in addition the probable variations in maximum rainfall and possible future changes in the condition of the surfaces are considered, we cannot but regard the proportioning of culverts by a formula as entirely futile. Even in the much simpler, because more regular and determinable, problem of proportioning the size of city sewers, many engineers claim that safety can only be assured by comparison with experience with as many similarly situated sewers as possible and then taking care not to overload the sewer after it is built; and with much reason. For culverts, if we were called upon to suggest a formula, we could do no better than this:
>
> Estimate the necessary area as carefully as possible by existing evidences of maximum flow, which let equal A. Then will $\sqrt[3]{8A}$ equal the proper area for the culvert. In more popular language: "Guess at the proper size and double it." We apprehend that this formula will

give far more satisfactory and trustworthy results than that which our correspondent quotes (Myers' formula), or any other which purports to be of general application to a problem subject to such extremely diverse conditions.

It is apparent that the Myers formula was not rational enough to be considered as a rigid rule, and furthermore there was the danger of its being abused rather than used intelligently and properly. However, this formula has had two significant consequences: to stimulate the development of other formulas, and to inspire the later use of a device called the "Myers scale" by C. S. Jarvis in connection with the study of floods at the U. S. Bureau of Public Roads and U. S. Geological Survey.

In 1926, Jarvis[3] modified the Myers formula and broadened its use by the introduction of the Myers scale. The Modified Myers formula is written

$$Q = 100\ p\sqrt{M} \tag{2}$$

in which Q = discharge in c.f.s.
M = drainage area in square miles
p = numerical percentage rating on the Myers scale.

The advantage of the Myers scale is that it furnishes a standard by which the flood flow characteristics in different streams can be roughly compared. In order to assist in visualizing the flood potentialities of the various regions within the United States, maximum flood flows have been expressed in per cent on Myers scale as experienced at widely scattered stream gaging stations in this country.[4] The use of the Myers scale is ingenious, but it was soon found to be too simple to be an index representing the complicated nature of flood flow.

C. PROFESSOR TALBOT'S RENOWNED FORMULA

A year after Wellington's comment on the Myers formula, Professor A. N. Talbot[5] of the University of Illinois published (1887) his well-known formula for determining the waterway area of culverts, which has since been very widely adopted in the United States. In deriving his formula, Professor Talbot made use of the Bürkli-Ziegler formula.[6] The latter is a storm water runoff formula published by the Swiss engineer Bürkli-Ziegler in 1880 and then introduced into American practice by Hering[7] in 1881.

The Bürkli-Ziegler formula for discharge q in c.f.s. per acre is written as

$$q = CI\sqrt[4]{\frac{S}{A}} \tag{3}$$

in which A = drainage area in acres
S = average slope of ground in feet per 1,000 feet
I = average rate of rainfall in inches per hour during the heaviest storm
C = coefficient depending on nature or relative imperviousness of ground surface, equal to 0.31 for an average condition, 0.20 for rural sections, 0.25 for farm country, 0.30 for village with lawns and macadam streets, 0.65 for ordinary city streets, and 0.75 for paved streets and built-up business blocks.

Professor Talbot derived his formula as follows:

Since by this formula (Eq. 3) the quantity of discharge per acre varies inversely as the fourth power of the area drained, the volume of discharge from the whole area will vary as $A\sqrt[4]{\frac{1}{A}}$, or $A^{3/4}$; and, assuming the same velocity through the culvert as in the stream above, the opening will vary likewise. This assumption will be true when the grade of the culverts is the same as that of the stream above and when the smaller coefficient of friction in the culvert over that of the channel itself is counteracted by the resistance to entering the culvert. We may then write

$$a = C\sqrt[4]{A^3}, \text{ or} \tag{4}$$

Area of water-way in sq. ft. =

$$C\sqrt[4]{(\text{Drainage area in acres})^3}$$

for which the coefficient C must be determined.

By comparison with the formula of Bürkli-Ziegler and with the flood flow of streams up to several of 77 square miles area, I conclude that for rolling agricultural country subject to floods at time of melting of snow, and with the length of valley three or four times the width, ⅓ is the proper value of C. If the stream is longer in proportion to the area, decrease C. In districts not affected by accumulated snow, and where the length of the valley is several times the width, ⅕ or ⅙ or even less may be used. C should be increased for steep side slopes, especially if the upper part of the valley has a much greater fall than the channel at the culvert.

In any case, the judgment must be the main dependence, the formula being a guide to it. On a road already constructed the C may be determined for the character of surface along that line by comparing the formula with the high-water mark of a known drainage area. Experience and observation on similar water-courses is the most valuable guide. A knowledge of the action of streams of similar situations in floods and of the effects of peculiar formations and slopes is of far more value than any extended formula.

In a subsequent discussion of his paper, Professor Talbot proposed that "for steep and rocky ground, C varies from $\frac{2}{3}$ to 1."

Concerning the difficulty of developing a rational formula to determine waterway areas, Professor Talbot listed the following considerations:

(1) The variation of the rate of rainfall in different localities.

(2) Paucity of data, since records are generally given as so much per day and rarely per hour, while the duration of the severe storm is not recorded.

(3) The melting of snow with a heavy rain.

(4) The permeability of the surface of the ground, depending upon the kind of soil, condition of vegetation and cultivation, etc.

(5) The degree of saturation of the ground and the amount of evaporation.

(6) The character and inclination of the surface to the point where the water accumulates in the water-course proper.

(7) The inclination or slope of the water-course to the point considered.

(8) The shape of the area drained and the position of the feeders.

He emphasized that "any formula will be approximate, that the estimation of the values of the different conditions entering into the subject will be almost wholly a matter of judgment, so that the formula must be considered more as a guide to the judgment than as a working rule."

For estimating the discharge of a stream flow in large drainage areas, Professor Talbot recommended the Chézy formula for waterway area determination.

An investigation of the Talbot formula reveals the following points of interest: The formula was derived with special reference to areas under 77 sq. mi. in size, although it has been applied to an area as large as 400 sq. mi. Generally, the results are much too high for large areas. Since this formula was based on the runoff data of a large number of observations in the Midwest, it does not take into account the variation in intensity of rainfall, velocity of flow, and frequency factor when applied to other localities. Studies on results obtained by this formula indicate that the maximum rainfall for these observations was probably about 4 in. per hr., and the velocity in the observed cases was variable but less than 10 ft. per sec.

Because of its simplicity the Tablot formula has been widely used either in its original form or with modified coefficients to meet local conditions. The formula has been presented by charts[8, 9, 10] and tables,[11] and slide rules for solving it have been developed.[12]

From the modern hydrologic and hydraulic viewpoint, the Talbot formula gives only a very crude answer to the problem. The formula assumes that the waterway area is directly proportional to the discharge which varies with the $\frac{3}{4}$-power of the drainage area. This is not accurate for a reliable design of numerous drainage structures being built nowadays. The relationship between the waterway area and the drainage area is far more complex than the $\frac{3}{4}$-power law; it depends on many physical characteristics of the drainage basin, as well as on the various hydrologic and hydraulic factors involved in a given problem.

D. EARLY CONTRIBUTIONS BY SEWERAGE ENGINEERS

Sewerage engineers are interested in waterway area determination primarily for the purpose of designing storm sewers. The Bürkli-Ziegler formula mentioned in the previous article is one of the earliest contributions by sewerage engineers.

One of the well-known contributions by sewerage engineers is the *rational formula*, which was developed primarily for estimating rates of runoff from urban areas. The origin of this formula is somewhat obscure. In American literature, the formula was first mentioned in 1889 by Emil Kuichling.[13] The runoff coefficient in the formula was derived by him from measurements of rainfall and of the flow in the sewers of Rochester, New York, during the period from 1877 to 1888. According to Dooge,[14] the principles of the method were explicit in the work of Mulvaney[15] in 1851. In England it is often referred to as the *Lloyd-Davis method* and hence by implication ascribed to his paper of 1906.[16]

The rational formula is

$$Q = CIA \tag{5}$$

in which Q = discharge in c.f.s.

C = runoff coefficient depending on characteristics of the drainage basin

I = rainfall intensity in inches per hr.

A = drainage area in acres

Many formulas have been proposed for estimating the rainfall intensity in the rational formula (see Appendix I). The general form may be written as

$$I = \frac{KF^m}{(t + b)^n} \tag{6}$$

in which t = duration of rainfall in minutes
F = frequency factor indicating the frequency of occurrence of the rainfall
K, b and m, n = coefficient, constant, and exponents, respectively, depending on conditions which affect the rainfall intensity

When using the rational formula, one assumes that the maximum rate of flow, due to a certain rainfall intensity over the drainage area, is produced by that rainfall which is maintained for a time equal to the period of concentration of flow at the point under consideration. This is the time required for the surface runoff from the remotest part of the drainage basin to reach the point being considered. In other words, the critical duration of rainfall t in the rainfall intensity formula (Eq. 6) should be equal to the time of concentration.

Values of C commonly recommended for design purposes are as follows:

Type of Drainage Area	Runoff Coefficient, C
Business:	
Downtown areas	0.70–0.95
Neighborhood areas	0.50–0.70
Residential:	
Single-family areas	0.30–0.50
Multi units, detached	0.40–0.60
Multi units, attached	0.60–0.75
Suburban	0.25–0.40
Apartment dwelling areas	0.50–0.70
Industrial:	
Light areas	0.50–0.80
Heavy areas	0.60–0.90
Parks, cemeteries	0.10–0.25
Playgrounds	0.20–0.35
Railroad yard areas	0.20–0.40
Unimproved areas	0.10–0.30
Streets:	
Asphaltic	0.70–0.95
Concrete	0.80–0.95
Brick	0.70–0.85
Drives and walks	0.75–0.85
Roofs	0.75–0.95
Lawns:	
Sandy soil, flat, 2%	0.05–0.10
Sandy soil, average, 2–7%	0.10–0.15
Sandy soil, steep, 7%	0.15–0.20
Heavy soil, flat, 2%	0.13–0.17
Heavy soil, average, 2–7%	0.18–0.22
Heavy soil, steep, 7%	0.25–0.35

The above values were reported by a joint committee of the American Society of Civil Engineers and the Water Pollution Control Federation in "Design and Construction of Sanitary and Storm Sewers," ASCE Manuals of Engineering Practice No. 37 and WPCF Manual of Practice No. 9, 1960. They are applicable for storms of 5-year to 10-year frequencies. Less frequent higher-intensity storms will require the use of higher coefficients because infiltration and other losses have a proportionally smaller effect on runoff.

From the data of the American Railway Engineering Association, Dr. J. A. L. Waddell[11] has compiled values of C for different watershed sizes as follows:

Drainage Area sq. mi.	Value of C	Drainage Area sq. mi.	Value of C	Drainage Area sq. mi.	Value of C
1,000	0.95	8,000	0.49	15,000	0.32
2,000	0.82	9,000	0.46	16,000	0.30
3,000	0.74	10,000	0.43	17,000	0.29
4,000	0.66	11,000	0.40	18,000	0.28
5,000	0.61	12,000	0.38	19,000	0.26
6,000	0.56	13,000	0.36	20,000	0.26
7,000	0.52	14,000	0.34	21,000	0.25

These values are for average drainage conditions in the United States and do not apply to drainage basins which have exceptional runoffs. The latter cases must receive individual consideration.

The rational formula is based on a number of assumptions. According to Krimgold,[17] the assumptions are:

(1) The rate of runoff resulting from any rainfall intensity is a maximum when this rainfall intensity lasts as long or longer than the time of concentration.

(2) The maximum runoff resulting from a rainfall intensity, with a duration equal to or greater than the time of concentration, is a simple fraction of such rainfall intensity; that is, it assumes a straight line relation between Q and I, and $Q = O$ when $I = O$.

(3) The frequency of peak discharges is the same as that of the rainfall intensity for the given time of concentration.

(4) The relationship between peak discharges and size of drainage area is the same as the relationship between duration and intensity of rainfall.

(5) The coefficient of runoff is the same for storms of various frequencies.

(6) The coefficient of runoff is the same for all storms on a given watershed.

It is believed that these assumptions might conceivably hold for paved areas with gutters and sewers of fixed dimensions and hydraulic characteristics. The formula has thus been rather popular for the design of drainage systems in urban areas and airports. The exactness and satisfaction of these assumptions in application to other drainage basins, however, have been questioned. In fact, many hydrologists[18, 19] have called attention to the inadequacy of this method. Bernard[20] had attempted to modify the rational formula, but his solution is hardly practicable for design purposes. Another study by Gregory and Arnold[21] resulted in a general rational formula, taking into account such factors as basin shape and slope, the pattern of the stream system, and the elements of channel flow. However, the complexity of the method hinders its wide application.

Other well-known formulas developed by sewerage engineers includes the Hawksley formula of 1857, the Adams formula of 1880, the McMath formula of 1887, the Hering formula of 1889, the Parmely formula of 1898, and the Gregory formula of 1907. These formulas and many others are listed in Appendix I.

E. DRAINAGE TABLES OF RAILROAD ENGINEERS

Various drainage tables have been developed for the determination of waterway areas. These tables, generally prepared from the actual stream flow data, give the size of waterway for a given drainage area. The most popular and frequently quoted drainage table is the *Dun waterway table* or *Dun drainage table* (Table 1) prepared by James Dun, former Chief Engineer of the Santa Fe railroad system. The table was first published in 1906,[22] but Professor W. D. Pence[23] of the University of Wisconsin pointed out that the forerunner of this table was a drainage table of a somewhat smaller range issued in 1897 in blueprint form.

According to Dun,[22]

> The accompanying table has been in use on the Santa Fe System for the past 15 years for proportioning waterways. In general, we have found this table to be sufficient, and particularly up to drainage areas of 5 square miles. In 1893, however, we noticed some floods in Central Kansas which exceeded the tables from 200 to 300 per cent. Also in the year 1905 we had a series of floods in the vicinity of Fort Madison, Iowa, that far exceeded our tables. In one case, where the drainage area is about 150 square miles, the area of waterway was about 12,000 square feet, and the current was so swift as to scour out the stream to a depth of 40 feet. I believe, however, that these floods are rare exceptions and that it would not pay a railway company or anyone else to undertake to provide for them.
>
> The table referred to is based upon observations taken by me and others under my jurisdiction on floods in Missouri, Kansas, Indian Territory and Texas. The section of waterway at the contracted part of different streams was accurately measured from time to time as floods occurred and the table was made up from these data. Wherever possible, cross-sections were taken in the larger streams at points where rock bluffs came in on both sides and where the stream has a rock bottom, thus eliminating the question of scour. This, however, was not practicable in every case.

The Dun table was prepared from observations taken along the line of the Santa Fe railroad system in Missouri, Arkansas, Kansas, and Indian Territory. This region in general is composed of steep rocky slopes, and percolation is a small percentage of the total rainfall. For other sections of the Santa Fe line, Dun gave in his table coefficients to be applied to the basic values listed. The use of the table can apparently be extended to other regions of comparable conditions.

Dun was of the opinion that his data could not be expressed by a formula for practical use. However, Purdon[24] has derived an approximate waterway area formula based on Dun's data:

$$a = A\,(240 - 12\sqrt[6]{A}) \tag{7}$$

in which a = waterway area in square feet
A = drainage area in square miles

This formula applies to drainage areas of more than 16 sq. mi. For small areas the waterway areas should be increased to allow for drift, etc.

The Dun data have also been expressed[23] by a curve of logarithmic plotting. Two curious breaks can be found on the curve at drainage areas of about 1 sq. mi. and 4 sq. mi., respectively. They are probably due to the abrupt change in nature of the rainfall intensity or in basin characteristics, or to some other unknown reasons.

Other notable drainage tables developed in early days and used in railroad engineering practice are:

(1) The "Table of Minimum Cross-Section Areas for Waterways of Culverts and Small Bridges" of the Pittsburg and Lake Erie Railroad. In this table the runoff was calculated by the

Table 1

The Dun Drainage Table
Atchison, Topeka & Santa Fe Railway System (1906)

Areas Drained in Square Miles	Areas of Waterway: Missouri and Kansas	Areas of Waterway: Cast Pipe. Banks over 15 Ft. Use 80 Per Cent	Areas of Waterway: Box and Arch Culverts. 1st Fig. Diam. 2d. Fig. Bench	Percentage of Column 2: Illinois	Percentage of Column 2: Indian Territory	Percentage of Column 2: Texas	Percentage of Column 2: New Mexico
1	2	3	4	5	6	7	8
.01	2.0	1-24 in.	2 x1 B				
.02	4.0	1-24 "	2 x2 "				
.03	6.0	1-30 "	2 x3 "				
.04	7.5	1-36 "	2½x3 "				
.05	9.0	1-42 "	3 x3 "	East of	South of	Use	Use
.06	10.5	1-42 "	3½x3 "	Streator	Purcell	Column	Column
.07	12.0	1-48 "	3 x4 "	use 60	use Texas	2	2
.08	13.5	2-36 "	2½x3 "	per cent	Column		
.09	15	2-36 "	2½x3 "				
.10	16	2-36 "	3 x3 "				
.15	25	2-48 "	3 x4 "				
.20	32	3-42 "	6 x4 A				
.25	38	3-48 "	6 x5 "				
.30	44	......	6 x5½ "				
.35	51	......	8 x4½ "				
.40	56	......	8 x5 "				
.45	62	......	8 x6 "				
.50	66	......	8 x6 "				
.55	70	......	8 x6½ "				
.60	74	......	10 x4½ "				
.65	78	......	10 x5 "				
.70	81	......	10 x5½ "				
.75	85	......	10 x6 "				
.80	88	......	10 x6½ "				
.85	91	......	10 x6½ "				
.90	94	......	10 x6½ "				
.95	97	......	12 x5 "				
1.0	100	......	12 x5 "				98½
1.1	110	......	12 x6 "			105	98½
1.2	120	......	12 x7 "			105	98½
1.3	130	......	12 x8 "			105	98½
1.4	140	......	14 x6½ "	West of	North of	105	98½
1.5	150	......	14 x7 "	Streator	Purcell	105	98½
1.6	160	......	16 x6½ "	use 80	use Col-	105	98½
1.7	170	......	16 x7 "	per cent	umn 2	105	98½
1.8	180	......	16 x7½ "			105	98½
1.9	190	......	16 x8 "			105	98½
2.0	200	......	18 x7 "			105	98½
2.2	220	......	18 x8 "			105	98½
2.4	240	......	18 x9½ "			105	98½
2.6	260	......	20 x8 "			105	98½
2.8	280	......	20 x9 "			105	98½
3.0	300	......	20 x9½ "			105	98½
3.2	321	......	22 x8½ "			105	98½
3.4	340	......	22 x9 "			105	98½
3.6	357	......	24 x8½ "			105	98½
3.8	373	......	24 x9 "			105	98½
4.0	388	......	28 x7 "			105	97
4.2	403	......	28 x7½ "			105	97
4.4	417	......	28 x8 "			105	97
4.6	430	......	28 x8½ "			105	97
4.8	443	......	28 x9 "			105	97
5.0	455	......	28 x9½ "	West of	North of	105	97
5.5	483	......	28 x10 "	Streator	Purcell	105	97
6.0	509	......	32 x7½ "	use 80	use Col-	105	97
6.5	533	......	32 x8 "	per cent	umn 2	105	97
7.0	556	......	32 x9 "			105	97
7.5	579	......	32 x10 "	East of	South of	105	97
8.0	601	......	32 x11 "	Streator	Purcell	105	97
8.5	622	......	32 x11½ "	use 60	use Texas	105	97
9.0	641	......	32 x12 "	per cent	Column	105	93½
9.5	660	......	32 x12½ "			105	93½
10	679	......	32 x13 "			105	93½
11	710	......				105	93½
12	740	......				105	93½
13	775	......	Bridges de-			105	93½
14	805	......	signed to			105	93½
15	835	......	provide area			105	93½
16	865	......	according to			105	94
17	890	......	circumstances			105	94
18	920	......				105	94
19	945	......				105	94
20	970	......				105	94
22	1,015	......				105	94

Areas Drained in Square Miles	Areas of Waterway: Missouri and Kansas	Percentage of Column 2: Illinois	Percentage of Column 2: Indian Territory	Percentage of Column 2: Texas	Percentage of Column 2: New Mexico
1	2	5	6	7	8
24	1,060			110	94
26	1,100			110	92
28	1,140			110	92
30	1,180			110	92
32	1,220	East of	South of	110	92
34	1,255	Streator	Purcell	110	92
36	1,290	use 60	use Texas	110	91
38	1,320	per cent	Column	110	91
40	1,350			110	91
45	1,435			110	91
50	1,510			110	89½
55	1,580			115	89½
60	1,650			115	89½
65	1,720			115	88
70	1,780			115	88
75	1,840			115	88
80	1,900			115	86½
85	1,960			115	86½
90	2,015			115	86½
95	2,065			115	86½
100	2,120			120	85
110	2,220			120	85
120	2,315			120	85
130	2,405			125	83½
140	2,500			125	83½
150	2,580			130	82
160	2,665			130	82
170	2,745			130	80½
180	2,820			130	80½
190	2,900			130	79
200	2,970			130	79
220	3,115	West of	North of	130	77½
240	3,245	Streator	Purcell	130	77½
260	3,370	use 80	use Col-	130	76
280	3,495	per cent	umn 2	130	76
300	3,615			130	74½
325	3,770			130	74½
350	3,900			130	73
375	4,035			130	73
400	4,165			130	71½
450	4,385			130	70
500	4,610			130	68½
550	4,825			130	67
600	5,030			130	65½
650	5,230			130	64
700	5,420			130	62½
750	5,610			130	61
800	5,800			130	59½
850	5,890			130	58
900	6,080			130	56½
950	6,230			130	....
1,000	6,380			130	....
1,100	6,705	West of	North of	130	....
1,200	6,960	Streator	Purcell	130	....
1,300	7,230	use 80	use Col-	130	....
1,400	7,480	per cent	umn 2	130	....
1,500	7,725			130	....
1,600	7,960	East of	South of	130	....
1,700	8,195	Streator	Purcell	130	....
1,800	8,390	use 60	use Texas	130	....
1,900	8,625	per cent	Column	130	....
2,000	8,820			130	....
2,200	9,240			130	....
2,400	9,605			130	....
2,600	9,970			130	....
2,800	10,320			130	....
3,000	10,640			130	....
3,500	11,445			130	....
4,000	12,160			130	....
4,500	12,825			130	....
5,000	13,500			130	....
5,500	14,080			130	....
6,000	14,520			130	....
6,500	15,140			130	....

The above classification by states is for convenience only, and merely denotes the general characteristics of topography and rainfall.

Column 2 in this table is prepared from observations of streams in Southwest Missouri, Eastern Kansas, Western Arkansas and the southeastern portions of the Indian Territory. In all this region steep, rocky slopes prevail and the soil absorbs but a small percentage of the rainfalls. It indicates larger waterways than are required in Western Kansas and level portions of Missouri, Colorado, New Mexico and Western Texas.

Bürkli-Ziegler and McMath formulas (Appendix I) with maximum rainfall equal to 3 in. per hr. and a runoff coefficient of 0.3. The assumed maximum velocity for culvert running full is 6 ft. per sec., that is, the maximum discharge in c.f.s. is equal to 6 times the opening in sq. ft.

(2) The "Drainage Table of El Paso and South Western Railway" compiled under the direction of James Dun:

Table 2

Drainage Table of El Paso and South Western Railway

Drainage Area sq. mi.	Waterways sq. ft.	Drainage Area sq. mi.	Waterways sq. ft.
0.12	18.8	4.6	353
0.22	31.4	5.8	428
0.40	51.0	7.2	508
0.70	74.0	9.5	628
1.15	102.0	12.4	756
1.55	134.0	15.8	878
2.00	170.0	19.9	978
2.50	209.0	24.6	1,088
3.50	280.0	30.2	1,199

In this table, the waterway area is that which is supposed to be adequate on steep rocky slopes where very little rainfall is absorbed. The area should be multiplied by a coefficient greater than 1 for exceedingly mountainous country and less than 1 for comparatively level country. The coefficients may vary from 0.50 for flat country with porous soil to 1.50 for rocky mountain gorges.

(3) The table of "Data for Concrete Arches and Waterway Areas, 1908" for Missouri, Kansas and Texas Railway. This table was constructed on the basis of the Talbot formula, using $C = 1.1$ for steep, 0.85 for medium, and 0.60 for flat lands.

(4) The "Table of Areas Drained by Culverts and Bridges" by Mississippi River and Bonne Terre Railway, prepared under the direction of James Dun for use in the Boston Mountain country of Northwest Arkansas. This table was constructed more or less on the basis of the original Dun drainage table. Where the land is not so rugged and water collects more slowly, the size of opening should be reduced somewhat, as judgment directs. The maximum velocity was assumed to be 7 ft. per sec.

(5) The table for "Dimensions of Pipe and Culvert Openings" by Mobile and Ohio Railroad. This table was derived from the Talbot formula in which the coefficient C varies from 0.2 for level land, 0.4 for rolling land, and 0.6 for hilly land to 0.8 for mountainous regions. It was recommended that no drain less than 24-in. diameter be used and that all drains over 4 feet in diameter be of concrete.

F. DEVELOPMENT OF NUMEROUS OTHER FORMULAS

Since the problem of waterway area determination was of interest to the sewerage and railroad engineers, a great number of methods involving the use of formulas, tables, and charts were developed and proposed for design purposes. This development almost reached its climax during the time the American Railway Engineering and Maintenance of Way Association held its Twelfth Annual Convention at the Congress Hotel, Chicago, Illinois, on March 21 to 23, 1909. During the convention, the Sub-Committee on Formulas for Waterways presented a report[26] which contains a description of formulas for waterways. In Appendix A of this report, "Waterway for Culverts" by W. D. Pence, a brief historical account, compilation and comparison of formulas, permissible velocity, and other features are given. Appendix B lists data on the maximum flood flow of stream in various sections of the United States. Appendix C contains an index to literature on the subject of waterways for culverts and allied topics. This report was published in 1911 together with an earlier report presented at the Tenth Annual Convention of the Association. This report[23] contains important discussions and reviews of current practice with reference to the methods of dimensioning waterways that had appeared in publications[25, 22] of two representative technical societies. Together they contain a very comprehensive survey of the methods and practices current at the time, as well as many authoritative comments on the subject.

The problem of waterway area determination has also been investigated in other countries. Besides the well-known Bürkli-Ziegler formula developed in Switzerland, there were the Chamier formula of 1898 in London, the Possenti formula of 1881 in Vienna, the Lillie formula of 1924 in London, the Lauterberg formula of 1887 in Germany, the Craig formula of 1868 in England, the Wood formula of 1917 in New Zealand, and the Kresnik formula of 1886 in Vienna. Even recently (1951) the Russian Scientific Academy formula was published in U.S.S.R. and the Ribeiro formula was published in Brazil. These formulas and many others are listed in Appendix I.

An early study of waterways for culverts, which included a compilation of formulas, was carried out as a thesis investigation by A. F. Gilman and G. W. Chamberlain in 1909-1910, under the direction of

Professor W. D. Pence, at the University of Wisconsin. A digest of the thesis material was incorporated in a report[23] in 1909.

At a later date (1934), T. M. Munson[27] listed 36 formulas and 4 sets of curves used at the time of his study in determining runoff and sizes of highway and railroad drainage structures.

Many of the published formulas, such as the Bürkli-Ziegler formula and the Talbot formula, have been very popular for many years. Based on local experience, various coefficients were developed to satisfy the local conditions. However, most engineers have never been satisfied with the wide range of accuracy provided by these formulas.

G. EFFORTS OF AGRICULTURAL ENGINEERS

Since the formation of the Soil Erosion Service, the forerunner of the Soil Conservation Service, in 1933, and the establishment of the CCC (Civilian Conservation Corps) camps in the following year, the soil and water conservation engineers began to work actively in the construction of many drainage structures on small drainage basins. This stimulated the need for a reliable and practical method to determine design discharges for these structures.

One of the early efforts made to satisfy this need consisted of the well-known *Ramser curves.*[28] It is believed that these curves paved the way for the development of the refined procedure of waterway area determination later used by the Soil Conservation Service and the Bureau of Public Roads.

Ramser presented three sets of curves, giving runoff in c.f.s. for drainage basins of different characteristics and of sizes up to 30 acres for 10-year rainfall frequency and from 30 to 1,000 acres for 10- and 50-year rainfall frequencies. The discharge shown by the curves is for the region covering Pennsylvania, Ohio, Indiana, Illinois, Wisconsin, Minnesota, South Dakota, Nebraska and parts of their neighboring states. For other regions the value may be multiplied by a factor of rainfall intensity ratio.

The Ramser curves were computed by the rational method with values of C and of times of concentration based largely on the results of measurements made in 1918 on six small drainage basins, 1.25 to 112 acres, near Jackson, Tennessee.[29] Rainfall intensities for various durations and the 10- and 50-year frequencies were taken from Meyer's book on hydrology.[30] The curves were extended beyond Meyer's ranges after the Yarnell rainfall intensity frequency data were published in 1935.[31]

However, actual experience showed that Ramser curves could not be applied over a wide range of conditions as are encountered in the United States. The curves were later gradually abandoned because more information was needed on values of C, and also because the estimation of the time of concentration is uncertain and hence would affect the accuracy of the results.

In seeking a reliable coefficient of runoff to be used in the rational method, the Soil Conservation Service established runoff and experimental drainage basin studies[32] in widely separated locations as early as 1938. The studies covered a broad range in topography, soils, vegetal cover, and tillage practices. The results obtained indicated that the coefficient of runoff and the time of concentration could be employed very reliably in the rational method. However, the assumptions underlying the method were found to be inadequate and inapplicable to small rural drainage basins. Characteristics and conditions of such drainage areas, as well as of the channels, are greatly affected, not only by the amounts and intensities of rainfall, but also by other climatic factors and by land use, tillage, and cropping practices. In the meantime, Krimgold[17] made a study on the relation of peak rates of runoff to rainfall intensities for drainage basins of various sizes and locations, but failed to show the significance of such a relation. He also pointed out that the frequency of runoff cannot be the same as the frequency of rainfall intensity which is assumed in the rational method (Section II-D). He suggested a frequency study of the recorded peak discharges and then derived the frequency curves for the Claypan Prairies[33] and other areas.

The Soil Conservation Service later developed a procedure known as the *Cook method*[34] after Howard L. Cook, in which the working curves are based on the results of runoff studies undertaken by the Bureau of Agricultural Engineering in its central district and on representative formulas of flood flow and runoff coefficients then in use in the North Central States. The method has been modified by M. M. Culp and others.[35] In this procedure, the probable maximum peak discharge from a given drainage basin is computed as the product of three factors, namely, the peak flow P in c.f.s., the rainfall factor R, and the frequency factor F; or

$$Q = P \times R \times F \qquad (8)$$

Table 3

Runoff Producing Characteristics of Drainage Basins with Corresponding Weights W
(The weights are shown in brackets)

Designation of Basin Characteristics	Runoff Producing Characteristics			
	(100) Extreme	(75) High	(50) Normal	(25) Low
Relief	(40) Steep, rugged, terrain, with average slopes generally above 30 per cent	(30) Hilly, with average slopes of 10 to 30 per cent	(20) Rolling with average slopes of 5 to 10 per cent	(10) Relatively flat land, with average slopes of 0 to 5 per cent
Soil Infiltration	(20) No effective soil cover; either rock or thin soil mantle of negligible infiltration capacity.	(15) Slow to take up water; clay or other soil of low infiltration capacity, such as heavy gumbo.	(10) Normal, deep loam with infiltration about equal to that of typical prairie soil.	(5) High; deep sand or other soil that takes up water readily and rapidly.
Vegetal Cover	(20) No effective plant cover; bare except for very sparse cover.	(15) Poor to fair; clean-cultivated crops or poor natural cover; less than 10 per cent of drainage area under good cover.	(10) Fair to good; about 50 per cent of drainage area in good grassland, woodland, or equivalent cover; not more than 50 per cent of area in clean-cultivated crops.	(5) Good to excellent, about 90 per cent of drainage area in good grassland, woodland, or equivalent cover.
Surface Storage	(20) Negligible; surface depressions are few and shallow; drainage-ways steep and small; no ponds or marshes.	(15) Low; well-defined system of small drainage-ways; no ponds or marshes.	(10) Normal; considerable surface-depression storage; drainage system similar to that of typical prairie lands; lakes, ponds and marshes less than 2 per cent of drainage area.	(5) High; surface-depression storage high; drainage system not sharply defined; large flood-plain storage or a large number of lakes, ponds or marshes.

The peak flow P is estimated from a chart in Figure 1. It depends on runoff producing characteristics which are measured by the summation of weights as shown in Table 3. For a given drainage area, the runoff producing characteristics are evaluated by the sum of the weights, ΣW, counted for different conditions. The peak discharge P is then determined with this value of ΣW from the chart. The rainfall factor R varies with location as indicated in the map of Figure 1. The frequency factor F is 1.00 for 50-year frequency, 0.83 for 25-year frequency, and 0.71 for 10-year frequency. For example, the area of a drainage basin in Pike County, Illinois, is 440 acres. The runoff producing characteristics of the drainage basin are evaluated as follows:

Basin Characteristics	*Weights*, W
Relief: slightly rolling with average slopes of 5 to 16%	20
Soil Infiltration: normal	10
Vegetal Cover: fair	10
Surface Storage: normal	10
	$\Sigma W = 50$

From the chart, with $\Sigma W = 50$ and drainage area of 440 acres, P is found to be 750 c.f.s. From the rainfall factor map, $R = 0.9$. For a frequency of 25 years, $F = 0.83$. Therefore the peak discharge $Q = 750 \times 0.90 \times 0.83 = 560$ c.f.s. In applying this method to drainage areas larger than 600 acres, however, other methods of runoff determination should be used to aid the judgment in arriving at proper peak rates.

In recent years the U. S. Agricultural Research Service has developed a *method of hydrograph synthesis* for estimating flow characteristics from the physiographic features of small drainage areas.[36]

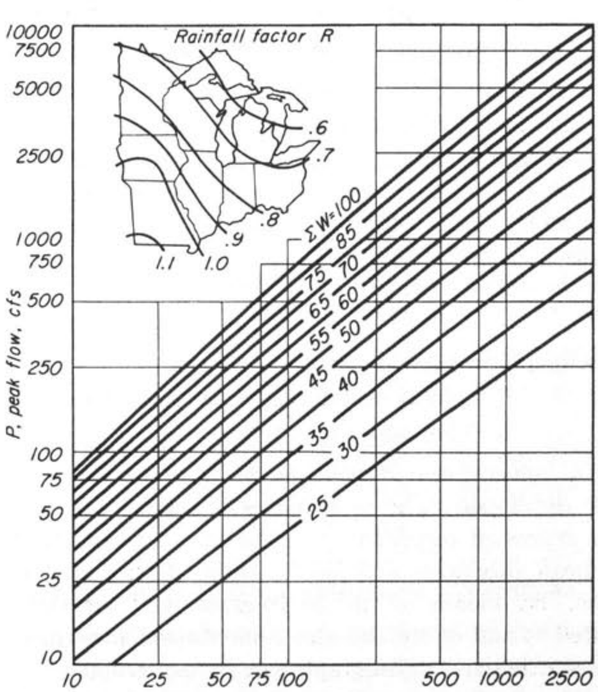


Figure 1. Chart and map for peak flow determination by the Cook method

The method involves (1) estimation of a characteristic lag time from readily determined basin parameters, (2) use of the basin lag time to predict the hydrograph peak rate for an assumed total volume of runoff, and (3) synthesizing the entire hydrograph using the lag time, the estimated peak rate, and a standard dimensionless hydrograph.

In developing the method of hydrograph synthesis, multiple correlations of lag time with various combinations of basin and channel slopes and lengths, drainage density, shape, and size were made. Of some 50 such multiple correlations a formula for lag time was derived. The lag time t_p is defined as the time interval measured from the center of mass of a block of intense rainfall to the resulting peak of the hydrograph. The formula for t_p in hours proposed by the Agricultural Research Service is

$$t_p = 1.75K_o^{0.61} \quad (9)$$

where K_o is a lag-time factor or

$$K_o = \frac{A^{0.3}}{S_o\sqrt{D.D.}} \quad (10)$$

in which A = drainage area in acres

S_o = average land slope of the drainage basin in percentage

D.D. = drainage density, this is total length L_o of visible channels divided by drainage area A, expressed in feet per acre.

The lag time was found to be a major determinant of the hydrograph shape, and hence a correlation of the lag time with the ratio of the peak rate of runoff to the total runoff volume was obtained. Thus,

$$q_p = 90.8\frac{V}{t_p} \quad (11)$$

in which q_p = peak discharge in c.f.s.

V = total runoff volume in acre-feet

t_p = lag time in hours

A generalized dimensionless hydrograph was also developed. The coordinates of the hydrograph are expressed respectively in the ratio of discharge to peak discharge and in the ratio of time to lag time. By means of this hydrograph and the estimated t_p and q_p by the above-mentioned formulas, the synthesized hydrograph can be constructed.

The development of the above method is based on the analysis of rainfall and runoff records for 14 experimental drainage basins in Arizona, New Mexico, and Colorado. The results were found to be satisfactory for these regions. For other regions, new correlation formulas may be necessary. The method generalizes the rainfall pattern, soil condition, land use, and other factors. It ignores the frequency factor which should be considered in the computation of the total runoff volume.

The U. S. Soil Conservation Service has proposed another method of hydrograph synthesis for developing design hydrographs.[37] In brief, this method involves the following steps:

(1) Take a maximum probable 6-hour point rainfall amount for the appropriate geographical location of the structure.

(2) Modify the 6-hour point rainfall amount to account for size of drainage area above the structure in accordance with a given synthetic rainfall depth-area relationship.

(3) Develop a rainfall hyetograph for the modified 6-hour point rainfall in accordance with a given synthetic hyetograph distribution pattern.

(4) Determine the *hydrologic soil-cover complex number* of the drainage basin above the structure. The number shows the relative value of the hydrologic soil-cover complexes as direct runoff producers. The higher the number, the greater the amount of direct runoff to be expected from a storm. The numbers for various land uses, crop treatments, hydrologic condition, and hydrologic soil groups were prepared using data from gaged drainage basins with known soils and cover.

The determination of the soil-cover complex number is done with reference to both soil cover and soil type.

The soil cover, as described from the hydrologic point of view, is given as either good, fair, or poor, depending on the infiltration capacity. A soil cover of high, medium, or low infiltration capacity is described as being of good, fair, or poor condition respectively.

The soil types are classified on the basis of intake of water at the end of long-duration storms occurring after prior wetting and opportunity for swelling, and without the protective effects of vegetation. The major hydrologic soil groups are:

Type A (lowest runoff potential) includes deep sands with very little silt and clay; also deep, rapidly permeable loess.

Type B includes mostly sandy soils less deep than type A, and loess less deep or less aggregated than type A, but the group as a whole has above-average infiltration after thorough wetting.

Type C comprises shallow soils and soils containing considerable clay and colloid, though less than those of type D. This type has below-average infiltration after pre-saturation.

Type D (highest runoff potential) includes mostly clays of high swelling per cent, but the type also includes some shallow soils with nearly impermeable subhorizons near the surface.

A classification of about 2,000 major soils of continental United States into the above four types was made available by the Service.[37]

(5) Determine the direct runoff Q in inches by the following formula:

$$Q = \frac{\left(R - \frac{200}{N} + 2\right)^2}{R + \frac{800}{N} - 8} \qquad (12)$$

in which R = rainfall in inches from step (3)
N = hydrologic soil-cover complex number from step (4)

Apparently, Equation 12 becomes invalid when $R < (200/N\text{-}2)$. For a value of R equal to $200/N\text{-}2$, Q will be zero. For R values greater than this quantity, the Q versus R relationship is good. For R values less than this quantity, Q has a positive value, even when R equals zero. Obviously, the equation must be modified so that when $R < (200/N\text{-}2)$, Q is taken as zero. In the practical application, the relationship would probably never be applied in this lower range. Therefore, the equation is not valid for these lower values.

The above equation was derived by plotting storm rainfall versus direct runoff for observed floods and correlating the results with the field hydrologic soil-cover complex numbers for an average antecedent moisture condition.

(6) Correct the direct runoff values obtained in the previous step for high or low antecedent conditions if the design criterion is not for an average antecedent condition.

(7) Obtain the direct runoff from the previous step for uniform time intervals in the synthetic hyetograph.

(8) Compute the time to peak (T_p), base time (T_b), and peak discharge (q_p) of a triangular hydrograph for the direct runoff in each time interval of the hyetograph by the following equations:

$$T_p = \frac{D}{2} + 0.6T_c \qquad (13)$$

in which T_p = time from beginning of direct runoff to peak in hours
D = time interval of effective rainfall in hours
T_c = time of concentration in hours

$$T_b = 2.67T_p \qquad (14)$$

in which T_b = base time of the triangular hydrograph

and
$$q_p = \frac{484AQ}{T_p} \qquad (15)$$

in which q_p = peak rate of flow in c.f.s.
A = drainage area in square miles
Q = direct runoff in inches

(9) Add all the triangular hydrographs and thus obtain a composite hydrograph. The latter is the design outflow hydrograph for the drainage basin.

The Soil Conservation Service method is too complicated. It appears that certain details in this method can be simplified or modified in order to arrive at a simple method that can be used for practical design of waterway areas.

Another activity of agricultural engineers has been the provision of basic hydrologic information for use in the design of small drainage structures. The U. S. Agricultural Research Service has recently published valuable hydrologic data for small agricultural drainage basins:

"Monthly Precipitation and Runoff for Small Agricultural Watersheds in the United States," since July, 1957.

"Annual Maximum Flows from Small Agricultural Watersheds in the United States," since June, 1958.

"Selected Runoff Events for Small Agricultural Watersheds in the United States," since January, 1960.

These data should be of great use to the future development of methods for waterway area determination.

H. DEVELOPMENTS BY HIGHWAY ENGINEERS

Since the beginning of the period of rapid expansion of highway constructions at about 1920, highway engineers have been much interested in the unavoidable problem of waterway area determination for their drainage structures. At the early

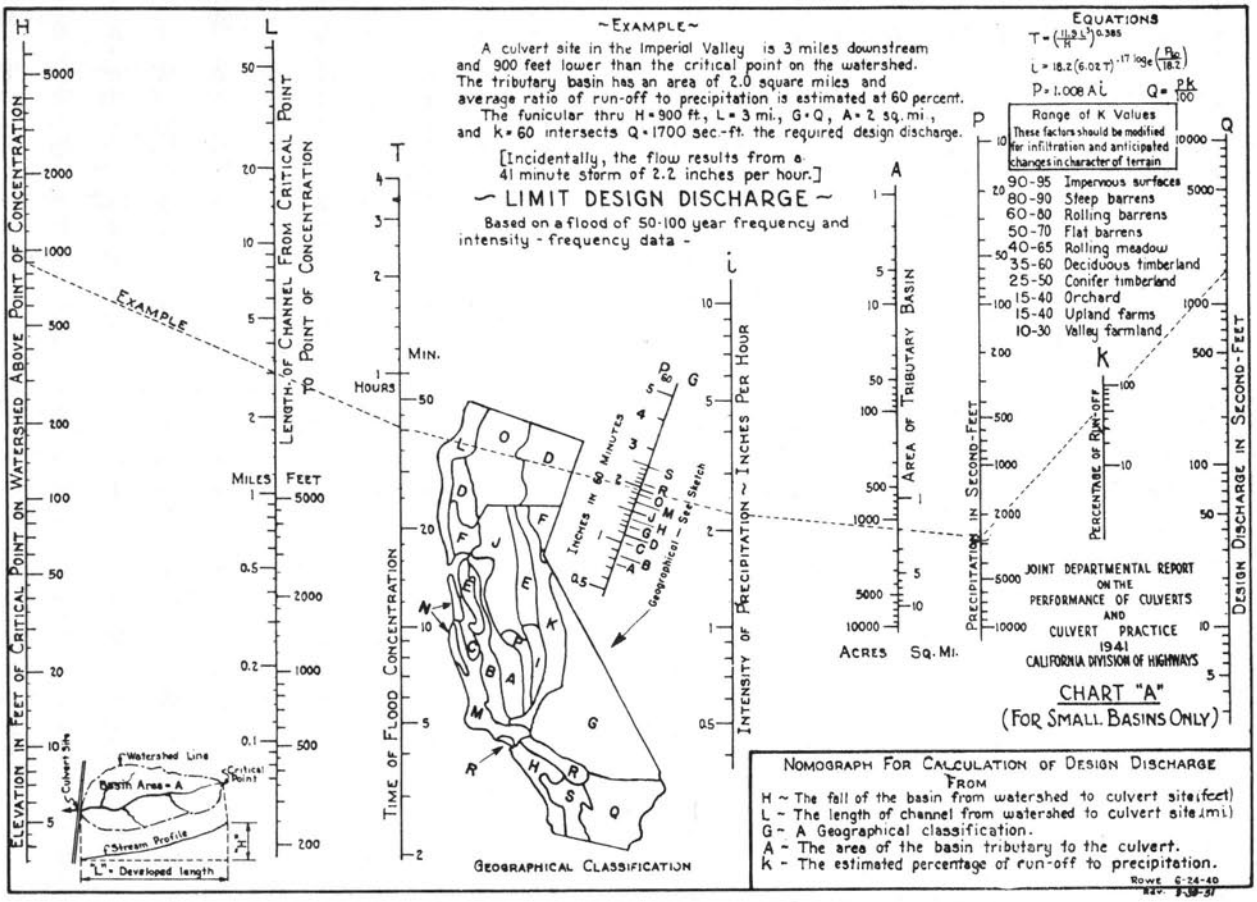


Figure 2. Chart for computation of design discharge by the California method
Source: California Division of Highways

stage, the highway practice in designing drainage structures was based almost entirely on the experience of railroad engineers and some sewerage engineers. Even at the present time, many highway engineers are still using the Talbot formula and the Dun table, which were developed primarily for railroad engineers, or the rational formula, which was proposed for sewerage engineers.

The interest in waterway area determination by highway engineers is reflected through the many papers published by highway agencies and presented in highway engineering conferences. Most of them recommend the use of formulas, charts, and modified rational methods. One of the first comprehensive studies was made by the Oregon State Highway Commission in 1934.[11] This bulletin contains a general description of the economic design of waterway areas and the results of research by the Oregon Highway Department during the 15 years preceding the time of publication. It treats both hydrologic and hydraulic aspects of the problem.

Rowe and Thomas of California State Department of Highways presented a new formula for design discharge in the form of a nomograph in 1942.[38] This study was later expanded and revised under the supervision of Rowe and published in a bulletin entitled "California Culvert Practice."[39] The nomograph is shown in Figure 2. In computing a design discharge from the nomograph, five factors are considered: channel slope and length, rainfall intensity-frequency, drainage area, and basin texture. This California method is based on the rational formula and hence is subject to the same criticisms as that formula.

Other early papers on the subject of waterway areas for highway engineers in the hydrology field include those by Houk[40] in 1922, by Springer[41] presented to Road School at Purdue University in 1931, by Greve[42] in 1943, by Mavis[43] in 1946,

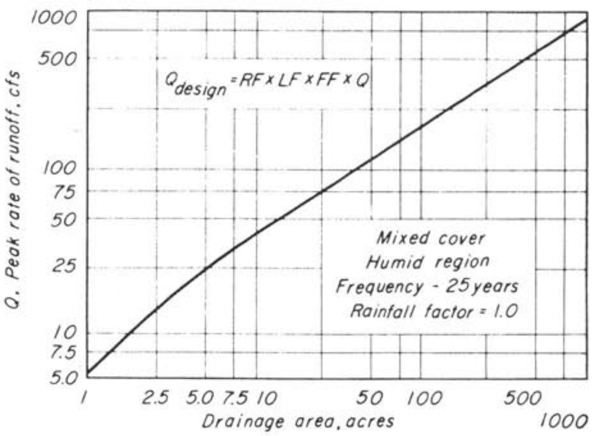


Figure 3. Peak rates of runoff for drainage basins under 1,000 acres (from Bureau of Public Roads Manual, August, 1951)

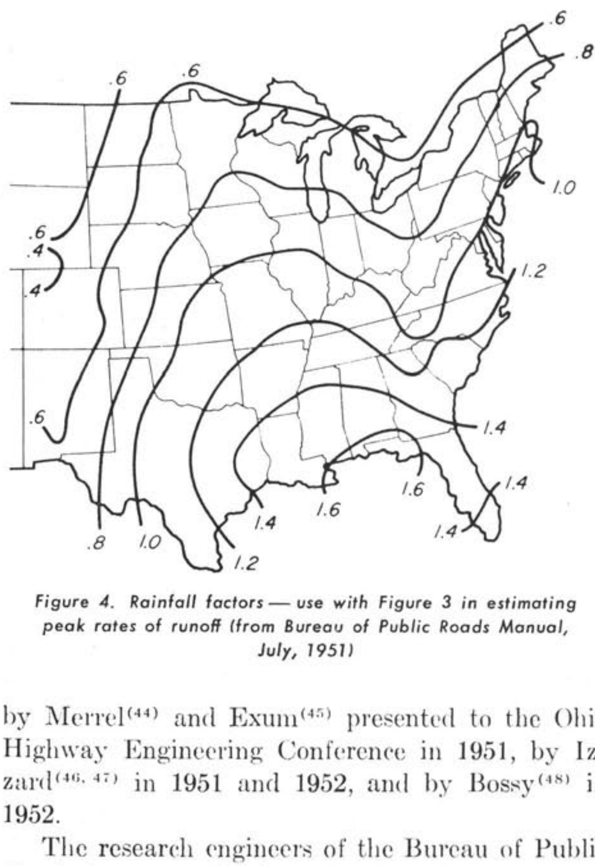


Figure 4. Rainfall factors — use with Figure 3 in estimating peak rates of runoff (from Bureau of Public Roads Manual, July, 1951)

by Merrel[44] and Exum[45] presented to the Ohio Highway Engineering Conference in 1951, by Izzard[46, 47] in 1951 and 1952, and by Bossy[48] in 1952.

The research engineers of the Bureau of Public Roads have recognized the fact that the determination of waterway areas should be considered in two steps. The first is to estimate the peak discharge of a given frequency and the second is to determine the physical dimensions relating to the culvert site and thus to find the size of culvert required. In developing the research work, the Bureau had the cooperation of the U. S. Soil Conservation Service and the U. S. Geological Survey. The experimental drainage basins established by the Soil Conservation Service supplied most of the runoff data[19] for small agricultural drainage basins of less than 1,000 acres with different types of land use in the humid region of the United States, ranging from Maryland to Nebraska and south to Texas. Based on the statistical analysis of these peak rates of runoff and the Yarnell rainfall intensity data, the Bureau developed a method, the *BPR method,* which consists of the use of two charts as shown in Figure 3 and Figure 4.[49, 50]

The procedure involved in the BPR method is similar to the Cook method of the Soil Conservation Service described previously. In this method, the design peak discharge from a given drainage basin is computed as the product of four factors; namely, the rainfall factor RF, the land use and slope factor LF, the frequency factor FF, and the peak rate of runoff Q for mixed cover in humid regions with a frequency of 25 years and rainfall factor of unity, or

$$Q_{\text{design}} = RF \times LF \times FF \times Q \qquad (16)$$

The rainfall factor is obtained from Figure 4. The land use and slope factor and the frequency factor are to be selected from the table in Figure 3. The discharge Q is to be selected from the curve in Figure 3 corresponding to the given drainage area in acres. This curve is applicable to localities where the 25-year frequency of one-hour rainfall has an intensity of approximately 2.75 in. per hr. For other localities the peak rate of runoff should be increased or decreased in proportion to the one-hour rainfall intensity for this frequency. This correction is to be applied by means of the rainfall factor.

It is understood that the land use factors given in Figure 3 are most reliable for steep land since all of the observed data were for steeply sloping land. The factors for flat and very flat land are estimated from limited data on the effect of slope on rates of runoff from test plots under simulated rainfall with allowance for increased channel storage. It is therefore noted that land use and slope factors for flat and very flat land slopes are subject to revision when more observed data become available.

Like the Cook method the BPR method appears to be practical and simple. However, the land use and slope factor is still subject to a certain amount of personal judgment. Furthermore, since the method is based on the data of different geographic locations, its application to a specific region will give only a general estimate of the design discharge.

In order to consider the climatic and topographic features of a special region, the multiple correlation method has also been used to determine discharge from drainage areas over 100 acres. Potter[51, 52] of the Bureau of Public Roads applied the method to the analysis of runoff data from 51 drainage basins in the Allegheny-Cumberland Plateau ranging from 100 to 350,000 acres in size. A correlation of the runoff data to topography of the drainage basins as well as to rainfall data from 89 stations in the region resulted in a correlation formula:

$$Q = 0.038\, A^{1.170}\, T^{-0.554}\, W \tag{17}$$

in which Q = 10-year discharge in c.f.s.
A = drainage area in acres
W = a given rainfall factor
T = slope factor given by

$$T = \frac{0.7L}{\sqrt{X/0.7L}} + \frac{0.3L}{\sqrt{Y/0.3L}} \tag{18}$$

in which L = length of stream in miles
X = difference in elevation in feet of the streambed at the culvert site and at $0.7L$ upstream
Y = difference in elevation in feet of the streambed at $0.7L$ and at the headwater

The problem of waterway area determination is continually of keen interest to highway engineers. This is indicated by various research projects on this subject sponsored by many state highway agencies either with or without the cooperation of the Federal highway agency. For example, a study made by Kentucky Department of Highways is reported by E. M. West and W. H. Sammons.[53] Many states are installing stream gages at the culvert sites for collecting data for future analysis and use in the determination of culvert areas.

I. CLASSIFICATION OF EXISTING METHODS

From a comprehensive study of available literature in this investigation, the existing methods of hydrologic determination of waterway areas may be classified into the following categories:

(1) *Method of Judgment.* By this method, the determination of waterway areas is dependent on practical experience and individual judgment. The judgment developed by the engineer is invariably guided by personal observation and general information collected on the ground such as flood height, size of channel, openings in the vicinity carrying the same stream, etc. This may be a satisfactory method if the judgment is good. However, the disadvantage lies in the fact that no judgment can be perfect because the conditions vary so greatly from problem to problem. Also, the method is not valid for beginners or for those who have little practical experience.

(2) *Method of Classification and Diagnosis.* By this method, drainage areas are classified and prescribed for different sizes and kinds of openings, the limits for each opening allowing variations to be made according to the local conditions, topography, slopes, soil, rainfall, etc. This method has some advantages if a table is prepared with reference to a specific territory so that due allowance can be made for the variable rainfall conditions and the prevailing regional characteristics of the territory. A glance at the table serves to indicate the general class of opening required. The final determination, however, will still be dependent on individual judgment and a personal examination of the area.

(3) *Method of Empirical Rules.* An empirical rule of thumb is usually developed to replace judgment. Such methods were used frequently in the early days, but have now become almost obsolete because of their crudeness and the development of better methods.

(4) *Method of Formulas.* By this method, a formula is developed to determine the waterway area. In Appendix I, a compilation of a large number of formulas is presented. The formulas range from simple to complex ones; many like the Talbot formula[5] are still very popular in engineering practice. The use of a formula may generally cast a certain amount of scientific glamour on the method. The greatest merit of formulas is their function of serving as a guide to determine quickly the general range of the probable minimum, maximum, and average values. The method can also be considered as practicable and serviceable for rough calculations. However, the disadvantage of the method is the uncertainty involved in the selection of the proper coefficient in most formulas in order to meet closely the conditions of the problem under consideration.

(5) *Method of Tables and Curves.* Instead of formulas, tables and curves are sometimes prepared to serve the same purpose. The Dun table[22] is a

prominent example. The simplicity of this method is its chief advantage. However, the table or curve is too simple and does not usually include the consideration of the many variables involved in the problem.

(6) *Method of Direct Observations.* This method involves making careful field surveys of drainage area and stream characteristics and then making a precise hydrologic analysis and hydraulic study. Finally, it is used to arrive at the required size and shape of the waterway which will carry off the water quickly and without causing either scouring or deposition in the channel.

(7) *The Rational Method.* This is a method which is based on the rational formula, such as the California method[39] or the Gregory and Arnold method.[21]

(8) *Method of Correlation Analyses.* This method involves the correlation of important hydrologic factors by statistical analysis. The result may be represented by a formula or nomograph for practical applications. The Cook method,[34] the BPR method,[49, 50] and the Potter's multiple correlation method[51, 52] are examples.

(9) *Method of Hydrograph Syntheses.* By this method, the hydrograph theory is used to derive a synthetic hydrograph for design purposes. The U. S. Agricultural Research Service[36] and Soil Conservation Service[37] have developed such methods.

J. CHRONOLOGICAL DEVELOPMENT

From a historical review of the literature and the study of existing methods, it appears that the development of engineering studies of hydrologic determination of waterway areas could be described by a series of significant events. A list of such events is:

1852 Preparation of a table expressing the relation between the diameter and slope of a circular outlet sewer and the size of its drainage area by John Roe, Surveyor of the Holborn and Finsbury Sewers, London, after numerous observations of the storm discharges.

1857 Presentation of the Hawksley formula in a "Report of Commission on Metropolitan Drainage, London." The original formula seems to have been established at some time between 1853 and 1856.

1879 Presentation of the Myers formula by T. M. Cleemann in a paper before the Engineers' Club of Philadelphia. The formula was then published in "Railroad Engineers' Practice, Discussion on Formulas" in the *Proceedings* of the Club.[1] This is the first known waterway formula by an American author, Major E. T. D. Myers, Chief Engineer of the Richmond, Fredericksburg, and Potomac Railway in Virginia.

1880 Publication of the Bürkli-Ziegler formula in a report[6] by the Swiss hydraulic engineer, A. Bürkli-Ziegler, Switzerland.

1881 Introduction of the Bürkli-Ziegler formula to the American technical literature by Rudolph Hering in a report on "Sewerage Works in Europe," to the National Board of Health.[7]

1886 Discussion of the Myers formula and the use of waterway formulas in general by Wellington in an "Editorial" in the *Railroad Gazette*.[2]

1887 Publication of the Talbot formula by Professor A. N. Talbot of the University of Illinois.[5]

1889 Publication of the rational method for estimating rates of runoff from urban areas by Emil Kuichling.[13]

1897 Publication of a report of Committee on Waterway for Culverts by W. G. Berg.[25]

1897 Issuance of a drainage table for the Atchison, Topeka, and Santa Fe Railway in blueprint form. This table covers a range of drainage areas from 0.1 to 1,000 sq. mi., and was later expanded and appeared as the well-known Dun Waterway Table in 1906.

1898 Publication of a discharge formula by George Chamier.[54]

1906 Publication of the Dun Waterway Table by James Dun, Chief Engineer of the Santa Fe System.[22]

1909 Publication of Reports of Committee No. I on Roadway in the *Proceedings of American Railway Engineering and Maintenance of Way Association*,[23] including "The Best Method for Determining the Size of Waterways," pp. 967-978, and a "Digest of Current Practice" collected from the members of the Association, pp. 978-1022.

1910 Completion of a thesis investigation on waterway for culverts by A. F. Gilman and

G. W. Chamberlain in 1909-1910,[23] under the direction of Professor W. D. Pence, University of Wisconsin.

1911 Publication of a Report of Sub-Committee on Formulas for Waterways of Committee No. 1 — On Roadway, in the *Proceedings of American Railway Engineering and Maintenance of Way Association,*[26] in which various formulas for waterways were discussed.

1922 Publication of a paper by Ivan E. Houk,[40] emphasizing that each opening requires individual design and formulas are makeshifts.

1926 Introduction of the Myers scale and the modified Myers formula by C. S. Jarvis.[3, 55]

1931 Publication of a paper by G. P. Springer on waterways for culverts and bridges.[41]

1933 Recommendation of Ramser's Curves for the design of soil and water conservation works.[28]

1934 Publication of a comprehensive discussion on design of waterway areas for bridges and culverts by C. B. McCullough,[11] describing economic design of waterway areas and results of research by the Oregon State Highway Department in the preceding 15 years, including both hydrologic and hydraulic studies.

Listing of 36 formulas and 4 sets of curves used at that time for determining runoff and sizes of highway and railroad drainage structures by T. A. Munson.[27]

1940 Publication of the Cook method by the U. S. Department of Agriculture.[34]

1943 Publication of a paper by F. William Greve on bridge and culvert flow areas.[42]

1944 Publication of the first edition of *California Culvert Practice* by the State of California, Department of Public Works, Division of Highways, describing the California method of culvert opening design.[39]

1946 Publication of a paper by D. B. Krimgold on the hydrology of culverts, introducing the U. S. Soil Conservation Service's practice to highway drainage problems.[17]

1949 Publication of the BPR method in *Highway Practice in the United States of America* by Public Roads Administration.[49]

1951 Publication of a paper by Carl F. Izzard,[46] outlining some ideas about how the latest knowledge of hydrology and hydraulics could be applied to the design of highway drainage structures.

Publication of a paper by J. F. Exum on waterway areas for culverts and small bridges.[45]

Revision of the discharge-area curve in the BPR method in Figure 1 of Hydraulic Information, *Circular No. 1,*[50] by the U. S. Bureau of Public Roads. The revision was based on a statistical analysis of actual records of runoff on small agricultural drainage basins.

1952 Presentation of a paper by Tate Dalrymple[56] on hydrology in design of bridge waterways, describing the U. S. Geological Survey's approach to the design of highway bridge waterways. The paper was first presented to Raleigh Engineers' Club, Raleigh, North Carolina, on February 11, 1952.

Presentation of a paper by Carl F. Izzard[47] to the American Society of Civil Engineers at New Orleans Convention on March 5-7, 1952. The paper describes the latest practice of estimating peak discharges for the design of highway bridges and culverts.

Presentation of a paper by Herbert G. Bossy[48] on simple methods for hydraulic design of culverts at the 1952 annual meeting of the Southeastern Association of Highway Officials.

1955 Publication of a report by E. M. West and W. H. Sammons[53] of the Kentucky Highway Department on the study of runoff from small drainage areas and the openings in attendant drainage structures.

1957 Publication of *Monthly Precipitation and Runoff for Small Agricultural Watersheds in the United States* by the U. S. Department of Agriculture, Agricultural Research Service.[57]

1958 Publication of *Annual Maximum Flows from Small Agricultural Watersheds in the United States* by the U. S. Department of Agriculture, Agricultural Research Service.[58]

Publication of a paper by Franklin F. Snyder[59] describing the method developed by U. S. Army Corps of Engineers for the determination of peak discharges from small drainage basins of a given frequency.

1959 Publication of a paper by R. B. Hickok,

R. V. Keppel, and B. R. Rafferty,[36] describing hydrograph synthesis for small drainage basins.

1960 Publication of *Selected Runoff Events for Small Agricultural Watersheds in the United States* by the U. S. Department of Agriculture, Agricultural Research Service.[60]

Publication of a paper by Neal E. Minshall[61] on a synthetic method of predicting runoff from small experimental drainage basins.

Publication of a pamphlet by W. D. Potter,[62] describing a correlation analysis of peak discharges from small drainage basins.

III. A SURVEY OF DESIGN PRACTICE

A. THE QUESTIONNAIRE

In order to assess the current methods for the determination of waterway areas as an important reference to the present investigation, a nation-wide survey was conducted by sending a questionnaire to all State Highway Departments in the United States in June 1953. Replies were received from 43 of the 48 states.

As supplements to this survey, additional information was obtained from publications or other indirect sources, and reference was also made to the report of an earlier survey conducted in 1943 by Tilton and Rowe[63] of the California Division of Highways.

In the questionnaire used in the 1953 survey, the following major items concerning the design of waterway areas were requested:

(1) The recommended design frequency of drainage structures.

(2) The methods of calculation of the design discharge and the determination of the size of culvert and bridge openings.

(3) The consideration of the computation of culvert slope and sections.

(4) The consideration of the computation of head losses through culverts.

(5) The consideration of the hydraulic effects of bridge piers and approach conditions, such as the backwater effects.

The first two items are related to the hydrologic design of drainage structures and the other items to the hydraulic design. Since the present investigation is concerned primarily with the hydrologic design of the determination of waterway areas, the main objective of the survey was essentially covered by the first two items.

It is obvious that the results obtained from the survey cannot be considered as conclusive because the information received was in general incomplete and inaccurate and it represents only the practice in 1953. Nevertheless, the findings of the survey have furnished general knowledge about the design practice at that time and such knowledge was found valuable in the present investigation.

B. THE DESIGN FREQUENCY

From the replies to the questionnaire regarding the policy of design frequency, the following major findings were obtained:

(1) The design frequency and its use varied in different states. There was no definite rule for determining the design frequency. Generally speaking, the design frequency depended mainly on the size, type, importance, and location of the structure. However, in most cases, the importance of the structure depended on economic and social factors.

(2) In several states, the design of important drainage structures was based on historical floods or independent investigations of the structures, but not on design frequency. The historical floods were usually obtainable from the records of the U. S. Geological Survey. If the flood record was not available at the site of the structure under consideration, independent investigation became necessary for a proper determination of the design flood.

(3) For culverts, small bridges, and the drainage structures in secondary highway systems, the design frequency varied widely from 5 to 100 years. The design frequency most commonly used was 25 years.

(4) For bridges, large culverts, and the drainage structures in primary highway systems, the design frequency varied from 5 to 200 years. The design frequency most commonly used was 50 years.

(5) Most replies do not specify clearly whether the frequency referred to is the rainfall frequency or the runoff frequency. It is known in hydrology that the two frequencies are not identical. However, it may be generally assumed that many cases imply the rainfall frequency rather than the runoff frequency, because even at the present time runoff data in most small drainage basins are not enough to assure any reliable determination of the runoff frequency.

Table 4

Design Frequency for Culverts, Small Bridges, and Drainage Structures in Secondary Highway Systems

No.	State	Frequency of Design Flood										Historical	Independent Investigation	Type of Structures
		5	10	15	20	25	30	35	40	50	100			
1	Alabama				x									C
2	Arizona					x								C
3	Arkansas													
4	California		x*								x**			C
5	Colorado		x									x		G
6	Connecticut		x									x		G
7	Delaware					x	- -	- -	- -	x				C
8	Florida					x	up							C
9	Georgia					x	- -	- -	- -	x				B, S
10	Idaho		x	- -	x									S
11	Illinois	x	x									x		C
12	Indiana				x	- -	- -	- -	- -	x				G
13	Iowa					x								S
14	Kansas					x								G
15	Kentucky												x	G
16	Louisiana				x									C
17	Maine													
18	Maryland					x								G
19	Massachusetts									x				C
20	Michigan	x	x											C
21	Minnesota					x								C
22	Mississippi					x								S
23	Missouri			x	- -	x								S
24	Montana													
25	Nebraska					x								S
26	Nevada		x			x	- -	- -	- -	x				G
27	New Hampshire												x	C
28	New Jersey	x	x	x										C
29	New Mexico	x	- -	- -	- -	- -	- -	- -	- -	x				S
30	New York										x		x	G
31	North Carolina												x	C
32	North Dakota		x	- -	- -	x								G
33	Ohio		x	- -	- -	x								C
34	Oklahoma					x								C
35	Oregon					x								G
36	Pennsylvania											x		C
37	Rhode Island		x											C
38	South Carolina				x									C
39	South Dakota												x	G
40	Tennessee									x				C
41	Texas		x			x	- -	- -	- -	x				C, B
42	Utah											x		C
43	Vermont					x								C
44	Virginia		x	- -	- -	- -	- -	- -	- -	x				C, B
45	Washington												x	G
46	West Virginia		x											G
47	Wisconsin									x				S
48	Wyoming												x	G

Legend: C = Culverts.
B = Small bridges.
S = Drainage structures in secondary highway system.
G = Minor drainage structures in general.
* = No head on crown of the culvert.
** = Balanced design which is defined as that combination of conduit section, shape, texture, and gradient with entrance and outlet appurtenances which will just pass a 100-yr. flood without interruption of traffic and without serious damage to structure, embankment or abutting property.

A summary of the reported design frequencies adopted in different state highway agencies is given in Tables 4 and 5.

C. HYDROLOGIC DESIGN PRACTICE

From the replies to the questionnaire regarding the hydrologic design of waterway areas, the major findings were as follows:

(1) Of the 43 states that supplied information, 25 states, or 58%, used the Talbot formula directly or with modification. Ten states, or 23%, used the rational formula; 8 states, or 19%, used the Bürkli-Ziegler formula; 3 states, or 7%, used their own formulas; and 9 states, or 21%, used miscellaneous formulas.

(2) Of the 43 states, 5 states, or 12%, used the Dun table; 8 states, or 19%, used the Bureau of Public Roads charts; and 5 states, or 12%, used their own charts or tables.

(3) Of the 43 states, 11 states, or 26%, used U. S. Geological Survey's flood data or information for the design.

(4) Most states which did not have a formula, table, or chart prepared for their own design purpose used several existing methods in order to obtain the most appropriate design. For the design of large or important structures, technical aid from other agencies, such as the U. S. Geological Survey, were frequently sought, and special independent investigations were usually made.

Table 5

Design Frequency for Bridges, Large Culverts, and Drainage Structures in Primary Highway Systems

No.	State	Frequency of Design Flood											Historical Flood	Independent Investigation	Type of Structures
		5	10	15	20	25	30	35	40	50	100	200			
1	Alabama														
2	Arizona														
3	Arkansas														
4	California						x								B*
5	Colorado									x					P
6	Connecticut		x												G
7	Delaware									x					B
8	Florida					x -- up									C, B
9	Georgia										x	x			B, P
10	Idaho									x					P
11	Illinois		x ---							--- x					B
12	Indiana												x		B
13	Iowa									x					P
14	Kansas					x -- up								x	G
15	Kentucky													x	B
16	Louisiana														
17	Maine														
18	Maryland						x								G
19	Massachusetts													x	
20	Michigan	x	x	x											G
21	Minnesota					x ---				--- x					B
22	Mississippi									x					P
23	Missouri									x	x				P
24	Montana														
25	Nebraska									x					B, P
26	Nevada					x ---				--- x					G
27	New Hampshire													x	
28	New Jersey			x											B
29	New Mexico					x ---					--- x				P
30	New York													x	B
31	North Carolina													x	B, P
32	North Dakota		x ---			--- x									G
33	Ohio									x				x	P
34	Oklahoma													x	B
35	Oregon		x			x				x					G
36	Pennsylvania													x	G
37	Rhode Island														
38	South Carolina													x	G
39	South Dakota													x	G
40	Tennessee														
41	Texas									x	x				B
42	Utah														
43	Vermont														
44	Virginia														
45	Washington														
46	West Virginia					x									P
47	Wisconsin									x					P
48	Wyoming									x					P

Legend: C = Large culverts.
B = Bridges.
P = Drainage structures in primary system.
G = Drainage structures in general.
* = Plus freeboard for drift.

(5) Many states were engaged in independent studies for developing better methods of design. At least ten states had developed either charts, formulas, or manuals for design purposes. Three states, Illinois, Kentucky, and Missouri, were active in continuous investigation for an adequate solution of the problem.

A summary of the reported hydrologic design practice is given in Table 6.

D. HYDRAULIC DESIGN PRACTICE

From the replies to the questionnaire regarding the hydraulic design practice, the major findings were as follows:

(1) Based on the information available, 25 states indicated that they considered the slope in the culvert design. Most states which considered the slope tried to place the invert of the culvert along the natural slope of the stream or ground. In those states where the land is flat, the slope of the floor of the culvert was generally set to achieve a safe velocity of flow through the culvert.

(2) Based on the information available, 16 states indicated that they considered the head loss in the design and 12 states ignored it. In certain cases the head loss was considered but minimized or avoided in the design.

(3) Based on the information available, 13 states indicated that they considered the backwater and approach conditions, 7 states ignored them, and 4 states avoided them.

(4) The design references used by most states were:

Table 6

Summary of Hydrologic Design Practice

No.	State	Formula					Table or Chart			U.S.G.S. Data	Independent Investigation	No Information
		Talbot	Rational	Bürkli-Ziegler	State's Own	Miscellaneous	Dun	B.P.R.	State's Own			
1	Alabama											x
2	Arizona					x						
3	Arkansas											x
4	California				x				x			
5	Colorado						x					
6	Connecticut	x		x		Meyer						
7	Delaware	x		x								
8	Florida	x								x		
9	Georgia	x								x		
10	Idaho	x	x									
11	Illinois	x	x					x		x	x	
12	Indiana	x										
13	Iowa	x					x	x				
14	Kansas	x					x			x		
15	Kentucky	x								x		
16	Louisiana								x			
17	Maine	x										
18	Maryland									x		
19	Massachusetts	x	x	x	x							
20	Michigan	x										
21	Minnesota		x					x				
22	Mississippi	x				x				x		
23	Missouri	x	x			Pettis	x					
24	Montana											x
25	Nebraska	x						x		x		
26	Nevada		x	x		McMath, Fuller						
27	New Hampshire	x										
28	New Jersey								x			
29	New Mexico			x		McMath				x		
30	New York	x		x								
31	North Carolina	x				Wentworth						
32	North Dakota	x					x					
33	Ohio							x	x			
34	Oklahoma	x										
35	Oregon					California						
36	Pennsylvania								x			
37	Rhode Island											x
38	South Carolina	x								x		
39	South Dakota	x						x				
40	Tennessee											x
41	Texas		x									
42	Utah		x									
43	Vermont							x				
44	Virginia	x	x	x								
45	Washington			x				x		x		
46	West Virginia	x										
47	Wisconsin		x			Meyer					x	
48	Wyoming				x							
	Total	25	10	8	3	9	5	8	5	11	2	5
	% of 43 States	58	23	19	7	21	12	19	12	26	5	

(a) "Basic Principles of Highway Drainage," *Hydraulic Information Circular No. 1,* August, 1951, and "Special Problems in Drainage," *Hydraulic Information Circular No. 2,* September 1, 1951, both prepared by the U. S. Bureau of Public Roads.

(b) *Hydraulics of Steady Flow in Open Channels,* by Sherman M. Woodward and Chesley J. Posey. New York: John Wiley and Sons, Inc., 1941.

(c) Information or aid given by the U. S. Geological Survey.

A summary of the reported hydraulic design practice is given in Table 7.

Table 7

Summary of Hydraulic Design Practice

No.	State	Slope	Head loss	Backwater and Approach Condition	Remarks
1	Alabama				No information
2	Arizona				No information
3	Arkansas				No information
4	California	Considered	Considered	Considered	Use Nagler formula for backwater computation
5	Colorado		Usually ignored	Usually ignored	Use Manning formula for discharge Most streams are small
6	Connecticut	0.5 to 1%	Ignored		
7	Delaware	Natural			Use B.P.R. Manual
8	Florida		Variable	Variable	Use Kutter or Manning formula
9	Georgia			Considered	Aid from U.S.G.S.
10	Idaho	Computed	Considered		
11	Illinois	Natural or ignored	Ignored	Considered	Use Manning formula
12	Indiana	Natural	Ignored	Ignored	
13	Iowa			Considered	Studies under way
14	Kansas	Natural	Considered	Considered	
15	Kentucky				No information
16	Louisiana				No information
17	Maine	Natural			
18	Maryland		Ignored		Use B.P.R. Manual
19	Massachusetts			Considered	
20	Michigan	Considered	Considered		Use Kutter formula
21	Minnesota	Considered		Avoided	Use Manning formula
22	Mississippi	Natural	Usually ignored	Usually ignored	
23	Missouri	Ignored	Ignored	Ignored	Use Portland Cement Association formula
24	Montana				No information
25	Nebraska	Considered	Considered		
26	Nevada	Considered			Use Manning formula and B.P.R. charts
27	New Hampshire	Considered	Considered	Considered	Use standard publications
28	New Jersey	Considered	Considered	Considered	
29	New Mexico	Considered	Considered	Considered	Use Manning formula, and Univ. of Iowa, Univ. of Minnesota, and B.P.R. formulas
30	New York		Avoided	Avoided	
31	North Carolina				No information
32	North Dakota	Considered	Ignored	Usually ignored	
33	Ohio	Natural slope	Considered	Considered	Use B.P.R. Charts
34	Oklahoma	Natural slope	Ignored	Ignored	
35	Oregon		Considered	Considered	New studies made
36	Pennsylvania	Natural slope		Avoided	
37	Rhode Island				No information
38	South Carolina				Aided by U.S.G.S.
39	South Dakota	No strict design	Ignored	Considered	
40	Tennessee				No information
41	Texas		Considered		Use own manual
42	Utah				No information
43	Vermont	Considered	Considered	Ignored	Use B.P.R. Manual
44	Virginia	Considered	Considered	Considered	Use of B.P.R. Charts and nomographs
45	Washington	Overcome friction	Considered		Use own chart
46	West Virginia	Natural slope	Ignored		
47	Wisconsin	No less than 0.5%	Ignored	Avoided	
48	Wyoming		Minimized		Use Manning and other formulas
	Total	25 Considered	14 Considered 12 Ignored 2 Avoided or Minimized	13 Considered 7 Ignored 4 Avoided	

IV. HYDROLOGIC PRINCIPLES, DATA, AND ANALYSES

A. HYDROLOGY OF A DRAINAGE BASIN

For the design of small drainage structures, the peak discharges under consideration are those of runoff from small drainage basins.

Hydrologic investigations have shown that there is a significant difference between the small and the large drainage basin.[64, 65] For a small basin the rates and amounts of runoff are dominantly influenced by the physical condition of soil and cover over which man has some control. Thus, more attention in hydrologic study is given to the basin itself. For a large basin the channel storage effect becomes very pronounced and more attention is given to the hydrology of the stream. In the hydrologic study of large basins, direct measurements of runoff at individual stream locations are generally used, and often extrapolated and extended. For small basins, on the other hand, it has been necessary to use a statistical "sampling" procedure because the task of gaging every small drainage basin where runoff data might be required would be impracticable. In this procedure reliance is placed largely on sampling of upstream areas with extension of the findings to other areas by means of rainfall-runoff relations, and also on evaluation of the climatic and physiographic factors influencing runoff.

Strictly speaking, it is difficult to distinguish a small drainage basin from a large drainage basin using only the size of the watershed as a criterion. Frequently two basins of the same size may behave entirely differently from the hydrologic viewpoint. One drainage basin may show prominent channel storage effects, like most basins of large size, while the other of the same size may manifest strong influence of the land use, like most basins of small size. In other words, a distinct characteristic of the small basin is the fact that the effect of overland flow rather than the effect of channel flow is a dominating factor affecting the peak runoff. Consequently, a small basin is very sensitive both to high-intensity rainfalls of short duration and to land use. On large basins, the effect of channel storage is so pronounced that such sensitivities are greatly suppressed. Therefore, *a small drainage basin may be defined as one that is so small that its sensitivity to high-intensity rainfalls of short durations and to land use is not suppressed by the channel characteristics.* By this definition, the size of a small basin may be from a few acres to 1,000 acres, or even up to 50 sq. mi. The upper limit of the area depends on the condition at which the above-mentioned sensitivity becomes practically lost due to the overwhelming channel-storage effect. For the present study, however, a limit of 6,000 acres was adopted as a criterion of small drainage basins for practical purposes.

From the hydrologic point of view, the runoff from a drainage basin can be considered as a product in the hydrologic cycle, which is influenced by two major groups of factors: climatic factors and physiographic factors. Climatic factors include mainly the effects of rain, snow, and evapotranspiration, all of which exhibit seasonal changes in accordance with the climatic environment. Physiographic factors may be further classified into two kinds: basin characteristics and channel characteristics. Basin characteristics include such factors as size, shape, and slope of drainage area, permeability and capacity of ground water reservoirs, presence of lakes and swamps, land use, etc. Channel characteristics are related mostly to hydraulic properties of the channel which govern the movement and configuration of flood waves and develop the storage capacity. It should be noted that the above classification of factors is by no means exact because many factors are interdependent to a certain extent. For clarity, the following is a list of the major factors:

Climatic factors

(1) Rainfall
- (a) Intensity
- (b) Duration
- (c) Time distribution
- (d) Areal distribution
- (e) Frequency
- (f) Geographic location

(2) Snow

(3) Evapotranspiration

Physiographic factors

(1) Basin characteristics
- (a) Geometric factors
  1. Drainage area
  2. Shape
  3. Slope
  4. Stream density
- (b) Physical factors
  1. Land use or cover
  2. Surface infiltration condition
  3. Soil type
  4. Geological condition, such as the permeability and capacity of ground water reservoir
  5. Topographical condition, such as the presence of lakes and swamps

(2) Channel characteristics
- (a) Carrying capacity, considering size and shape of cross section, slope, and roughness
- (b) Storage capacity

In midwest areas, high peak discharges from small drainage basins are usually caused by rainfalls of short duration largely due to thunderstorms in summer months. A part of the precipitation is lost through the process of interception, evapotranspiration, and infiltration. The remaining portion which eventually becomes runoff is known as the rainfall excess and it is usually expressed in inches. The proportion of the rainfall excess to the total precipitation depends on climatic factors such as the rainfall as well as on physiographic factors such as antecedent moisture condition of the ground, type of surface soil and subsoil, and vegetation. These factors vary largely with seasons.

As mentioned previously, land use plays an important role in runoff phenomenon on a small drainage basin because the flow on the watershed is mostly of the overland type. Theoretically, the variables governing overland flow are the same as those governing ordinary hydraulic flow of the same type. For turbulent overland flow, which is often the case in nature, these variables are the depth of water, the slope of the ground, and the surface roughness coefficient. The depth of overland flow at a given time and at a given point in a drainage basin is further governed by the length of overland flow, duration of excess rainfall, rainfall intensity during the time of rainfall excess, volume of depression storage, infiltration capacity, and initial detention. All of these variables control the behavior of surface runoff and hence also the magnitude of the maximum runoff. Many analytical and experimental investigations have been undertaken to develop laws and formulas for the determination of overland flow. However, these laws and formulas are found less than satisfactory when applied to natural drainage basins because of the complexity, uncertainty, and variability of basin characteristics. It seems at present that the best approach to evaluation of the variables controlling the overland flow, as well as other hydrologic phenomena of a drainage basin, is the use of statistical and hydrologic analysis of all factual data that have been collected from the watersheds.

It is generally considered that the runoff from a given drainage basin is composed of two parts: a base flow and a direct runoff which is produced by the rainfall excess. The runoff, influenced by all the physiographic and climatic factors described above, is not steady but varies with time, reaching a maximum or peak discharge and then falling off. The curve of the discharge versus time is defined as the hydrograph for the drainage basin under investigation at its outlet, where the runoff is measured. The hydrograph can be regarded as an integral expression of the physiographic and climatic characteristics that govern the relations between rainfall and runoff of the particular basin. It shows the time distribution of runoff, defining the complexities of the basin characteristics by a single empirical curve. By separating the base flow, the remaining hydrograph is the direct runoff hydrograph. The latter is to be discussed below.

For the purpose of hydrologic analysis, Sherman[66] has introduced the idea of a *unit hydrograph* or *unitgraph*. By definition, the unit hydrograph of a drainage basin is a hydrograph of direct runoff resulting from 1 inch of rainfall excess generated uniformly over the basin area at a uniform rate during a specified period of time. In applying the method of unit hydrograph according to this definition, it should be noticed that the following major assumptions are implied.

(1) The rainfall excess is uniformly distributed within its duration or specified period of time.

(2) The rainfall excess is uniformly distributed throughout the whole area of the drainage basin.

(3) The ordinates of the hydrographs of a common time base are directly proportional to the total amount of runoff represented by each hydrograph.

Under the natural condition of rainfall and drainage basins, these assumptions can hardly be satisfied perfectly. However, the results obtained by using unit hydrograph analysis have been found acceptable for practical purposes. Although the unit hydrograph was originally developed for large drainage basins, many investigators[67, 36] have subsequently shown that it is applicable also to small drainage basins as well. It is known that there are some exceptional cases which do not support the use of unit hydrographs for small drainage basins, but in the majority of the cases the results obtained are acceptable within practical limits of error. The method of unit hydrographs is therefore considered practical also for small drainage basins.

The hydrologic quantities can be treated as statistical variables, and in their study reference should be made to the frequency of their occurrence. The frequency is defined here as the average time interval for such a quantity to be equalled or exceeded. In a hydrologic study of runoff from small drainage basins, the determination of the runoff frequency is important. For numerous small drainage structures accommodating floods from small drainage basins along highways and on farm lands, the frequency determination offers a logical basis for establishing a design policy and makes feasible a rational economic analysis for the design purpose. It should be noted that the nature of these structures is different from that of large drainage structures designed for large floods, such as the spillway of a dam, flood walls, etc. The failure of small structures does not usually involve loss of human life, nor does it cause the catastrophe often connected with the failure of large structures. The design of small structures can therefore be justifiably made on the basis of a calculated risk determined by a frequency analysis.

B. SOURCES OF HYDROLOGIC DATA

The data used in the present study include the rainfall, runoff, and other hydrologic data that are available for the State of Illinois and its neighborhood. The sources of these data are as follows:

1. Rainfall

In the State of Illinois, rainfall records of long periods are published by the U. S. Weather Bureau for first-order stations at Chicago, Cairo, Peoria, Moline, and Springfield. The excessive precipitation data for short durations at these places for the period 1881-1896 were published in the annual reports of the Chief of the Weather Bureau 1896-1897. Data for the years 1897 through 1934 were published in the annual reports of the Chief of the Weather Bureau. For the years 1935 through 1949 the data were published in the issues of the United States Meteorological Yearbook. For 1950 and succeeding years excessive precipitation data are presented in the annual issues of the *Climatological Data, National Summary*.

The data for the intensity-duration-frequency of the excessive precipitation were prepared by Yarnell.[68] A revision of the data was published recently by the U. S. Weather Bureau.[69, 70] Detailed analyses of the Chicago and Urbana data[71, 72] are also available. However, the results obtained from these later analyses indicate that they are in general much lower in value than the Yarnell data. As valuable references to this investigation other rainfall frequency data[73, 74, 75, 76, 77] for Illinois are also available.

A comprehensive compilation of maximum precipitation point data was published by the Weather Bureau.[78, 79] The probable maximum precipitations based on a hydrometeorological analysis were published by the Hydrometeorological Section of the Weather Bureau.[80, 81] Unofficial data and data from Weather Bureau Cooperative Stations are also available for comparative studies.

2. Runoff

The runoff data from large drainage basins in Illinois have been studied and analyzed by the U. S. Geological Survey. The unit hydrographs[82] and flood frequencies[83] for basins larger than 10 sq. mi. have been published.

The runoff data from small drainage basins in Illinois are limited. The U. S. Agricultural Research Service (abbreviated as ARS, formerly combined with the Soil Conservation Service or SCS) has maintained experimental plots and small single-crop drainage basins at Urbana, Dixon Springs, and Joliet. For securing information on the drainage properties of the soil and cover, precipitation, runoff, and soil loss, data from plots at Urbana and Dixon Springs were obtained from the ARS project at the Agricultural Engineering Department, Purdue University, Lafayette, Ind. No records for Joliet were included, however, because these plots are not equipped to measure rates of runoff, and moreover, very little runoff has occurred since the initiation of the plot study.

Table 8

Maximum Recorded Point Rainfalls in Inches in Illinois and Vicinity

	Minutes					Hours				
	5	10	15	30	60	2	3	6	12	24
Cairo, Ill.	0.63	1.01	1.33	1.89	3.15	3.67	3.86	5.40	5.40	5.69
Month/Day	7/7	7/1	7/30	8/12	6/28	6/28	6/28	3/13	3/13	10/3
Year	1915	1934	1913	1935	1905	1905	1905	1938	1938	1910
Chicago, Ill.	0.64	1.11	1.31	2.03	2.81	3.67				6.19
Month/Day	7/15	9/13	9/13	7/7	7/6	7/6				8/2
Year	1906	1936	1936	1921	1943	1943				1885
Davenport, Iowa	0.64	1.11	1.43	1.88	2.71	2.74	3.57	5.14	5.18	5.18
Month/Day	7/17	7/17	7/17	7/7	7/7	7/7	6/4	7/13	7/13	7/13
Year	1939	1939	1939	1915	1915	1915	1914	1889	1889	1889
Dubuque, Iowa	0.80	1.20	1.54	2.23	2.84	3.24	4.55	4.55	5.48	5.48
Month/Day	7/9	7/9	7/9	7/9	6/14	6/14	7/4	7/4	9/8	9/8
Year	1919	1919	1919	1919	1938	1938	1876	1876	1927	1927
Evansville, Ind.	0.51	0.87	1.19	1.96	2.79	3.11	3.41	4.40	4.82	6.94
Month/Day	9/15	8/10	8/10	6/26	7/20	7/20	9/20	9/19	10/5	10/5
Year	1934	1908	1908	1943	1916	1916	1924	1924	1910	1910
Hannibal, Mo.	0.56	0.99	1.39	1.91	2.95	4.67	4.79	4.98	5.19	5.83
Month/Day	7/7	8/18	8/18	6/19	6/19	6/19	6/19	6/19	6/19	9/3
Year	1915	1906	1906	1930	1930	1930	1930	1930	1930	1926
Keokuk, Iowa	0.69	1.05	1.33	1.95	2.56	3.11	3.33	4.39	4.99	5.88
Month/Day	5/22	8/1	8/1	8/1	8/1	8/1	6/29	6/28	6/28	6/28
Year	1899	1932	1932	1932	1932	1932	1933	1933	1933	1933
Madison, Wis.	0.60	1.08	1.41	2.15	3.67	4.91	4.96	4.96	5.16	5.31
Month/Day	9/1	8/8	8/8	8/8	8/8	8/8	8/8	8/8	9/12	9/7
Year	1937	1906	1906	1906	1906	1906	1906	1906	1915	1941
Milwaukee, Wis.	0.79	1.11	1.34	1.86	2.22	3.24	4.20	4.71	5.45	5.76
Month/Day	8/29	8/29	8/6	6/24	6/24	6/23	6/23	6/22	6/22	6/22
Year	1939	1939	1942	1904	1904	1917	1917	1917	1917	1917
Peoria, Ill.	0.73	0.95	1.26	2.10	2.60	3.18	3.32	4.33	4.33	5.52
Month/Day	8/17	8/17	7/2	7/2	7/2	9/10	9/10	5/18	5/18	5/18
Year	1925	1925	1931	1931	1931	1911	1911	1927	1927	1927
Royal Center, Ind.	0.65	0.94	1.11	1.56	2.27	2.40				3.23
Month/Day	8/9	7/9	7/9	8/4	7/9	7/9				5/18
Year	1930	1925	1925	1923	1925	1925				1927
St. Louis, Mo.	0.60	1.04	1.39	2.56	3.47	3.68	3.68	3.72	5.38	8.78
Month/Day	7/9	8/8	8/8	8/8	7/23	7/23	7/23	7/23	8/19	8/15
Year	1942	1923	1923	1923	1933	1933	1933	1933	1915	1946
Springfield, Ill.	0.66	1.23	1.41	2.12	2.75	3.01	3.20	3.93	5.94	5.94
Month/Day	7/23	7/23	7/23	7/23	7/6	9/8	9/8	6/4	6/4	6/4
Year	1917	1917	1917	1917	1912	1926	1926	1917	1917	1917
Terre Haute, Ind.	1.15	1.32	1.38	1.82	2.81	3.28	3.87	4.89	5.60	5.60
Month/Day	7/7	7/7	7/7	8/5	8/5	8/5	9/8	9/8	9/14	9/14
Year	1915	1915	1915	1938	1946	1946	1926	1926	1931	1931
Urbana, Ill.	0.62	1.08	1.39	1.90	2.48	2.55	2.55	3.22	4.10	4.61
Month/Day	7/8	6/22	6/22	6/22	6/30	6/30	6/30	11/2	5/25	5/25
Year	1942	1931	1931	1931	1931	1931	1931	1936	1921	1921
Summarized max. in Illinois	0.73	1.23	1.41	2.12	3.15	3.67	3.86	5.40	5.94	6.19
Max. in Ill.[a]					3.25	4.67	4.97	5.92	8.35	11.47
Max. in Indiana[a]					3.20	3.85	4.31	6.55	6.80	6.94
Max. in Ohio[a]					3.65	6.07	6.07	6.07	6.07	6.35
Max. in Ill. and vicinity	1.15	1.32	1.54	2.56	3.67	6.07	6.07	6.55	8.35	11.47
Probable max. (PMP) at Urbana, Ill.								24.0	28.2	30.5

[a] Reported in U.S.W.B. Technical Paper No. 15.(79)

For small natural drainage basins, the ARS has the projects at Alhambra, Edwardsville, Elmwood, and Monticello. However, only the data at Monticello are continuous and suitable for analyses. Such data were obtained from the Department of Agricultural Engineering, University of Illinois.

At the final stage of the present investigation, observed data for runoff from small agricultural drainage basins in the United States(57, 58, 60) became available, and such data were used to great advantage.

3. Hydrologic Soil Types

The University of Illinois Agricultural Experiment Station has made a comprehensive study on the types of soil in Illinois. From this information(84) the hydrologic properties of different soil types in Illinois were identified. The State Office of the U. S. Soil Conservation Service in Champaign, Illinois also supplied the data on hydrologic soil groups for Illinois.

C. RAINFALL ANALYSIS

The frequency data of rainfall intensity at Urbana, Illinois are used as a basis for this study. The data were developed mainly by the analysis in an earlier study at the Department of Civil Engineering of the University of Illinois.(72) Data for rainfall of long durations were adjusted by the maximum daily precipitation reported by the Illinois State Water Survey.(85) The rainfall amount frequency data at Urbana for frequencies up to 100 years is shown in Figure 5.

The maximum recorded rainfalls of various durations in Illinois and its vicinity were also studied. They are listed in Table 8. The maximum recorded rainfalls at four rainfall stations in Illinois and ten close-by stations in the neighboring states were obtained from the U. S. Weather Bureau *Technical Report* No. 2.(78) The data at Urbana were obtained from the Illinois State Water Survey.(73) For durations of 1, 2, 3, 6, 12, and 24 hours, the maximum recorded rainfalls in Illinois, Indiana, and Ohio were obtained from the U. S. Weather Bureau *Technical Report* No. 15.(79) From these data, the maximum recorded rainfalls for all listed durations in Illinois and in Illinois and its vicinity were selected.

Table 8 also lists the probable maximum precipitations (PMP) at Urbana which were interpolated from the all-season envelope for areas of 10 sq. mi.(81) These were derived by a theoretical hydrometeorological method which involves an analysis of air-mass properties (effective precipitable water, depth of inflow layer, temperature variation, winds, etc.), synoptic conditions prevailing during the recorded storms in the region, topographical features, season of occurrence, and location of the respective areas involved. However, the rainfall amounts thus derived are extremely high, and therefore are somewhat unrealistic and impractical for design purposes. They are listed only for the purpose of comparison.

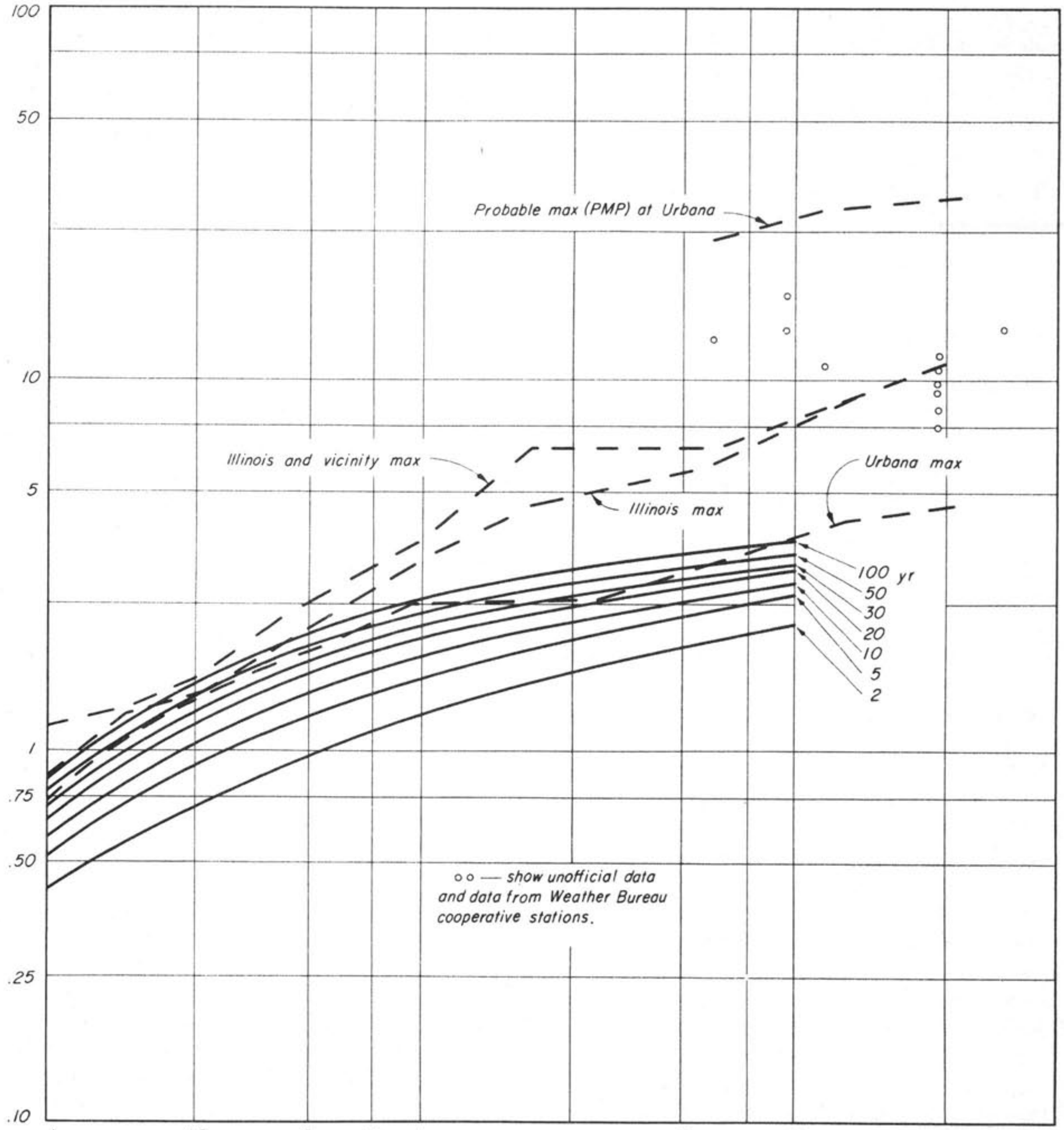


Figure 5. Rainfalls of different frequencies and maximum rainfalls

There are some unofficial data in Illinois which are higher than the official maximum recorded rainfalls given in Table 8. These data, supplied by Mr. F. A. Huff of the Illinois State Water Survey in a letter dated June 24, 1957, are as follows:

(1) Field observation values:

(a) 16.54 in. at East St. Louis and 13.75 in. at Belleville for 9.5 hr. duration on June 14-15, 1957 (recorded by a station of the East Side Levee and Sanitary District).

(b) 12.5 in. at Ocoya for 6 hr. duration on July 8, 1951 (unofficial).

(c) 12.5 in. at Belvidere for 6 hr. duration on July 18, 1952 (unofficial).

(d) 9.75 in. at Aurora and 11.45 in. at Waterman for 24 hr. duration on October 9-10, 1954 (recorded by the State Water Survey gage).

(e) 11.00 in. at Bellflower for 12 hr. duration and 14.0 in. total in 36 hr. on May 26-27, 1956 (unofficial).

(2) Maximum daily amounts at Weather Bureau Cooperative Stations:

(a) 9.15 in. at Galva, August 20, 1924.

(b) 10.48 in. at Aurora, October 9-10, 1954.

(c) 8.41 in. at Rockford, July 18-19, 1952.

(d) 8.24 in. at Mascoutah, August 16, 1946.

(e) 7.50 in. at Grafton, July 9, 1942

The fact that some high values of unofficial observation have not been registered by the long-range Weather Bureau first-order stations indicates that the probability of occurrence of these values at a particular location is small. For the purpose of designing minor drainage structures, the official data should constitute a practical and reasonable upper limit.

The above data are all plotted in Figure 5. Comparing these data with the frequency curves described previously as shown in the figure, the frequency of the Urbana maxima is approximately in the order of 50 years, and the Illinois maxima may be in the order of 100 years for durations less than 30 minutes and of about 1,000 years or more for higher durations.

In a study made by the Illinois State Water Survey,[86] it was found that the State of Illinois may be divided into four geographical sections of generally similar precipitation climate. Annual and seasonal maximum precipitation data used in the study were from 39 stations for the 40-year period 1916-55. By several methods of frequency analysis isohyetal maps were developed for a 50-year recurrence of one-day precipitation. This isohyetal pattern and the isohyetals of mean annual precipitation indicated that it is climatologically satisfactory to divide the entire state into four sections: northwest, north central, south central, and southeast. In each section the average value of 50-year one-day precipitation was computed as shown in Figure 6. The sectional values in extreme northwestern and southern Illinois are high. These areas coincide with the Rock River Hills and the Shawnee Hills, the most prominent hill regions in the state, which may have some augmenting effect on storm precipitation.

In the present investigation the 50-year one-day

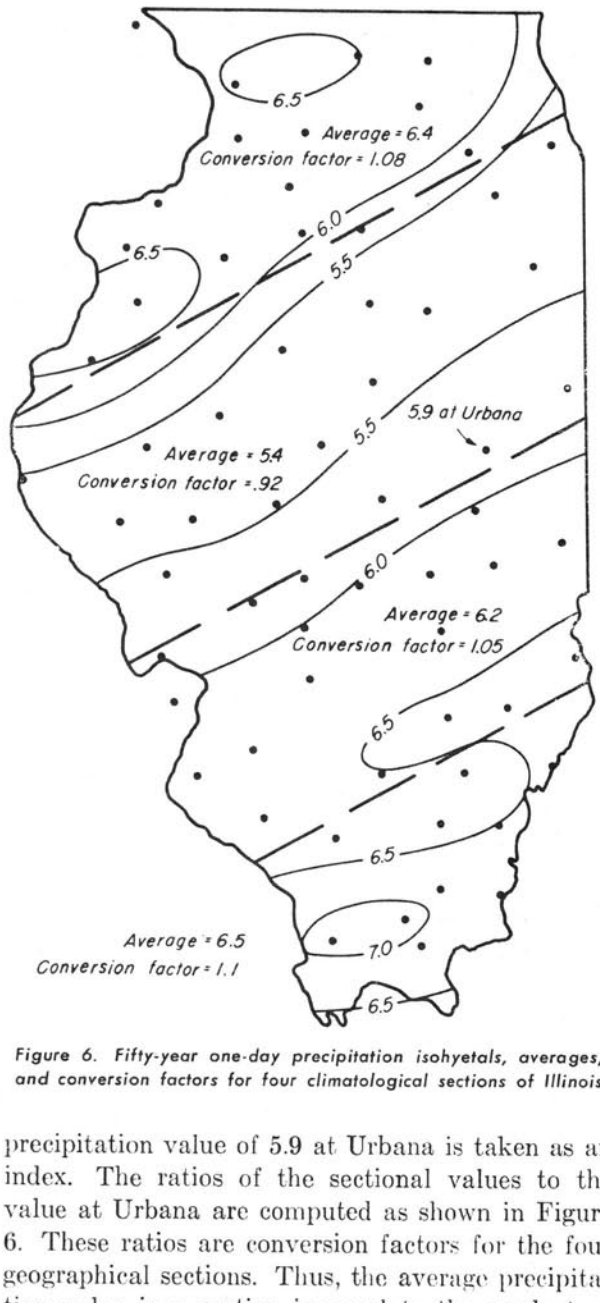


Figure 6. Fifty-year one-day precipitation isohyetals, averages, and conversion factors for four climatological sections of Illinois

precipitation value of 5.9 at Urbana is taken as an index. The ratios of the sectional values to the value at Urbana are computed as shown in Figure 6. These ratios are conversion factors for the four geographical sections. Thus, the average precipitation value in a section is equal to the product of the Urbana value and the conversion factor for the section. As mentioned before, the Urbana maxima fit the 50-year frequency agreeably, and all frequency curves in Figure 5 appear to have a similar trend of variation. Therefore, the conversion factor may be considered applicable to all Urbana frequency curves.

Should more reliable rainfall data become available, greater accuracy may be gained by expressing the climatic factor by isocontours. With the limited information available at present, however, the average sectional values are believed to provide more realistic values than isocontour values. The study made by the Illinois State Water Survey indicated that the sensitivity of isohyetal patterns to sampling variation of data is remarkably large. With insufficient data the orientation and distribution of the isohyets thus determined would be therefore very uncertain.

It may be noted in Figure 6 that Urbana is located in the north central region. The conversion factor for this region is 0.92 while the theoretical conversion factor should be 1.00. Because of the uncertain sampling variation described above, such difference may be ignored for practical purposes. However, the regional conversion factors are averages. For any location away from the middle zone of the region, an interpolated value of the conversion factor may be obtained if so desired.

The rainfall data at Urbana were observed at the site of the rain gage and are therefore referred to as "point-data." The maximum rainfall occurs at a point or over a small area at the center or centers of a storm. Outside the center the rainfall amount decreases so that the average amount over a larger area becomes less with increase of this area. The pattern of rainfall distribution in a storm is very irregular and indefinite. For small drainage basins under consideration (less than 6,000 acres), the rainfall reduction is small. For practical purposes, it may be assumed that the average rainfall over the basin area is equal to the point value. Therefore, to be on the safe side, no correction for areal distribution is made.

D. DEVELOPMENT OF UNIT HYDROGRAPHS

As mentioned previously the derivation of unit hydrographs was originally based directly on actually measured hydrographs fulfilling as nearly as possible the assumptions of the method. There are however other methods for the determination of unit hydrographs which have been developed in later years. Three methods for the development of a unit hydrograph will be discussed. They are (1) direct derivation from an observed hydrograph or hydrographs, (2) synthesizing observed hydrographs from a number of drainage basins, and (3) building-up of hydrographs on the basis of theory.

1. The Method of Direct Derivation

A unit hydrograph can be developed by direct

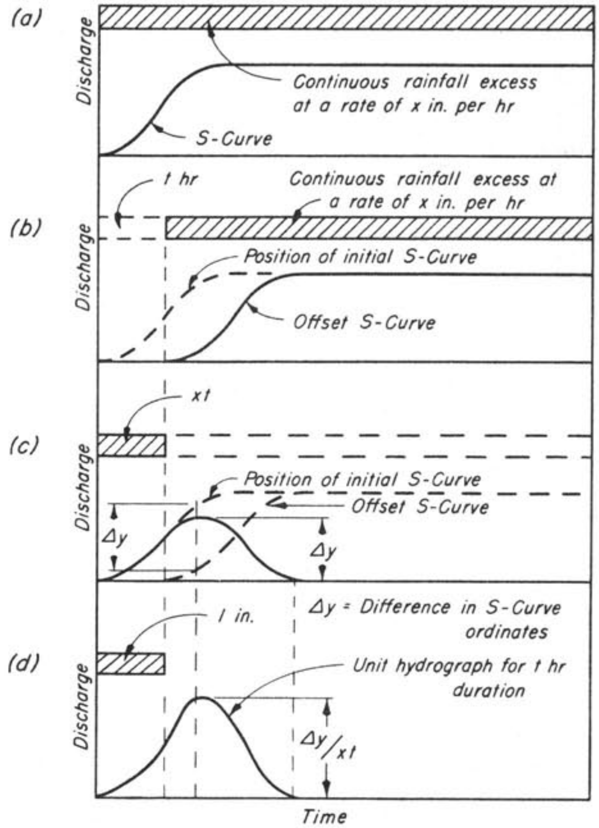


Figure 7. Derivation of unit hydrograph by the S-curve method

derivation from an observed hydrograph or hydrographs. This method is described in most hydrology textbooks. In selecting a hydrograph, care should be taken that assumptions involved in the unit hydrograph theory should be satisfied as closely as possible. A hydrograph resulting from an isolated storm of nearly uniform distribution in space and time is the most desirable. When a hydrograph exhibits several closely related peaks as a result of the occurrence of multiple storms, single-peaked hydrographs should be first segregated in the unit hydrograph analysis. A method of hydrograph segregation is described by Mitchell.[82]

From the hydrograph of a single storm, a unit hydrograph can be derived. The duration of this unit hydrograph is defined as being equal to the duration of the rainfall excess of the storm. From the same data, unit hydrographs of other durations may be derived by the so-called *S-curve method* first suggested by Morgan and Hullinghors.[87] The theoretical S-curve is a hydrograph that is produced by a continuous rainfall excess at a constant rate for an indefinite period. The curve assumes a

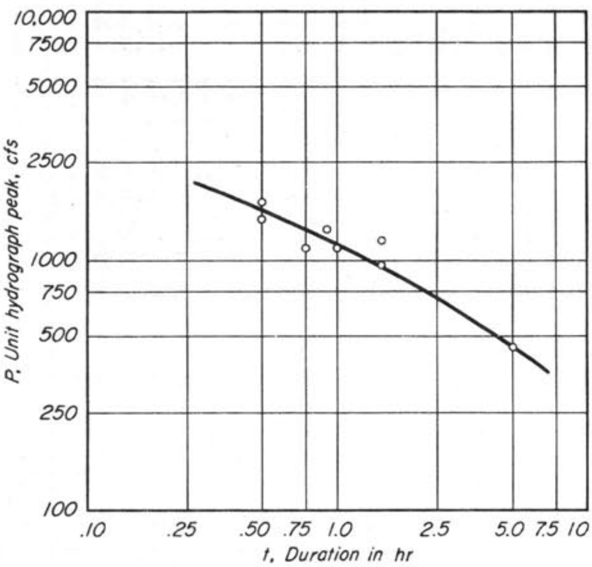


Figure 8. Relationship between the unit hydrograph peak and its duration for the Boneyard drainage basin, Champaign-Urbana, Illinois

deformed S-shape and its ordinates ultimately approach the rate of rainfall excess as a limit. The S-curve can be constructed graphically by summing up a series of identical unit hydrographs spaced at intervals equal to the duration of the rainfall excess from which they are derived. The procedure and a numerical example of the S-curve method are described in a report by Chow.(72)

After the S-curve is constructed, the unit hydrograph of a given duration may be derived as follows (Figure 7): Assume that the S-curve is produced by a continuous rainfall excess at a constant rate of x in. per hr. Then advance or offset the position of the S-curve for a period equal to the desired duration in t hr. and call this S-curve as an offset S-curve. The difference between the ordinates of the original S-curve and the offset S-curve, divided by xt, should result in the desired unit hydrograph.

When a number of unit hydrographs of various durations are obtained, a curve of unit hydrograph peak discharge against its duration can be plotted in a logarithmic scale. Experience has shown the use of actual data in plotting such a curve is better than derivation by theory, which assumes the condition of linearity. Therefore, when a number of observed hydrographs for various durations are available, it is advisable to derive separate unit hydrographs from these hydrographs and then plot their peak discharges against the corresponding durations. For example, the unit hydrograph peak discharges for a number of observed hydrographs for the Boneyard Creek drainage area at Champaign-Urbana, Illinois, are plotted against the duration as shown in Figure 8. The resulting curve, despite the scattering of the plotted data, usually represents a better relationship than that obtained by the S-curve method from a single hydrograph.

When a curve showing the relationship between the unit-hydrograph peak and the duration is available, the value of a unit hydrograph peak discharge P can be interpolated from this curve for any given duration within the range of the plot.

2. The Method of Synthesizing

A method of developing unit hydrographs for ungaged drainage basins by synthesizing a number of representative unit hydrographs in a given region was first proposed by Snyder.(88) However, his study was made primarily for the Appalachian highlands. In Illinois, a synthetic study of 58 unit hydrographs for large drainage basins has been made by Mitchell,(82) resulting in three synthetic S-curves (Table 9) for basin areas of 80, 500, and 1,200 sq. mi. respectively. The ordinates of these curves are expressed in per cents. For an equilibrium runoff the rate is equal to the rate of rainfall excess. Considering a rate of rainfall excess equal to 1 in. per t hours and the drainage area as A acres, the equilibrium runoff is equal to $1.008A/t$ c.f.s. The abscissas of these curves are expressed in lags. The *lag* t_o is equal to the time interval in hours from the center of mass of rainfall excess to the center of mass of runoff.

In a frequency study of floods in Illinois by Mitchell,(83) an expression for t_o was found as

$$t_o = k\,D^{0.6} \tag{19}$$

in which k is a physiographic factor varying from 0.60 to 1.82 with an average value of 1.05 and D is the watershed area in square miles.

Applying the S-curve method described previously, the unit hydrograph peak discharges for various durations for drainage basins of the three sizes mentioned above can be computed from the synthetic S-curves. The ratio of the unit hydrograph peak discharge P to $1.008A/t$, or $Pt/1.008A$, is defined as the *peak-reduction factor* Z, or

$$Z = \frac{Pt}{1.008A} \tag{20}$$

To illustrate the effect of the duration on the

Table 9

Ordinates of Synthetic S-Curves in Per Cent of Equilibrium Runoff for Drainage Basins of 80, 500, and 1,200 sq. mi. in Illinois

Time in Terms of Lag t_o	Basin Area, sq. mi. 80	500	1,200	Time in Terms of Lag t_o	Basin Area, sq. mi. 80	500	1,200	Time in Terms of Lag t_o	Basin Area, sq. mi. 80	500	1,200	Time in Terms of Lag t_o	Basin Area, sq. mi. 80	500	1,200
0.05	0.12	0.23	0.66	1.05	70.38	66.10	63.85	2.05	92.44	95.75	94.84	3.05	98.51	99.58	99.45
0.10	0.72	0.83	1.62	1.10	72.57	69.02	66.84	2.10	92.95	96.19	95.36	3.10	98.68	99.63	99.51
0.15	2.02	1.80	2.88	1.15	74.57	71.76	69.61	2.15	93.43	96.59	95.84	3.15	98.84	99.68	99.57
0.20	4.17	3.17	4.49	1.20	76.39	74.33	72.18	2.20	93.88	96.95	96.26	3.20	98.99	99.72	99.62
0.25	7.27	4.95	6.54	1.25	78.04	76.73	74.57	2.25	94.30	97.27	96.65	3.25	99.13	99.76	99.66
0.30	11.10	7.16	9.14	1.30	79.52	78.96	76.78	2.30	94.69	97.55	96.99	3.30	99.26	99.79	99.70
0.35	15.48	9.86	12.14	1.35	80.88	81.02	78.81	2.35	95.05	97.81	97.30	3.35	99.38	99.82	99.74
0.40	20.41	12.95	15.49	1.40	82.11	82.91	80.68	2.40	95.38	98.04	97.58	3.40	99.49	99.85	99.78
0.45	25.64	16.43	19.09	1.45	83.24	84.63	82.41	2.45	95.69	98.25	97.83	3.45	99.58	99.87	99.81
0.50	30.89	20.26	22.83	1.50	84.29	86.18	84.00	2.50	95.98	98.44	98.05	3.50	99.66	99.89	99.84
0.55	35.83	24.46	26.71	1.55	85.27	87.58	85.47	2.55	96.26	98.61	98.25	3.55	99.73	99.91	99.86
0.60	40.47	28.97	30.64	1.60	86.19	88.84	86.83	2.60	96.53	98.76	98.43	3.60	99.79	99.92	99.88
0.65	44.82	33.65	34.61	1.65	87.06	89.97	88.08	2.65	96.79	98.89	98.59	3.65	99.84	99.93	99.90
0.70	48.89	38.45	38.57	1.70	87.88	90.99	89.22	2.70	97.04	99.01	98.74	3.70	99.88	99.94	99.92
0.75	52.69	43.12	42.48	1.75	88.65	91.91	90.27	2.75	97.28	99.12	98.88	3.75	99.91	99.95	99.94
0.80	56.23	47.64	46.33	1.80	89.38	92.73	91.23	2.80	97.51	99.22	99.00	3.80	99.94	99.96	99.96
0.85	59.52	51.91	50.10	1.85	90.07	93.47	92.10	2.85	97.73	99.31	99.11	3.85	99.96	99.97	99.97
0.90	62.57	55.89	53.76	1.90	90.72	94.13	92.89	2.90	97.94	99.39	99.21	3.90	99.98	99.98	99.98
0.95	65.39	59.58	57.29	1.95	91.33	94.73	93.61	2.95	98.14	99.46	99.30	3.95	99.99	99.99	99.99
1.00	67.99	62.97	60.66	2.00	91.90	95.27	95.26	3.00	98.33	99.52	99.38	4.00	100.00	100.00	100.00

peak discharge, the peak-reduction factor can be plotted as the dimensionless ratio of t/t_o. The results obtained for the synthetic unit hydrographs of the three areas are presented in Figure 9.

A study of the curves in Figure 9 indicates that they appear to converge to a limiting position as the drainage area decreases. For small drainage basins, therefore, it can be assumed that the corresponding curve would lie somewhere above the curve for 80 sq. mi.

3. The Method of Building-up

This method derives a unit hydrograph by breaking up the drainage area into a number of segments and calculating the contributing flow of each segment. The method will be described with reference to a hypothetical circular drainage basin.

The fictitious circular basin (Figure 10) is assumed to have a length L, which is measured along the stream, and to be exposed to a uniform and continuous rainfall excess of intensity of 1 in. per t hours. The slope from the upstream end at A to the point of concentration O is S.

As the rain starts to fall over the whole basin, the water reaching O will be in the beginning only that falling in its immediate vicinity, say in area

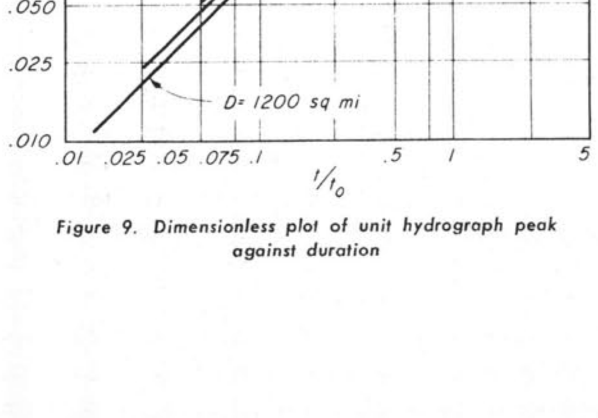


Figure 9. Dimensionless plot of unit hydrograph peak against duration

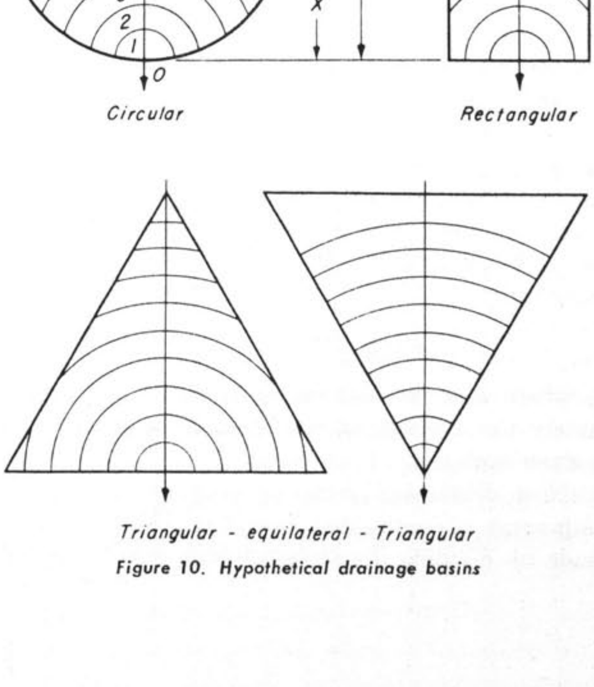


Figure 10. Hypothetical drainage basins

Table 10

Computation of the Dimensionless S-Curve for a Hypothetical Circular Drainage Basin

x/L (1)	t_c'/t_c (2)	a (3)	$a/11.95$ (4)	Σ (5)
0.1	0.17	0.24	0.021	0.021
0.2	0.29	0.77	0.065	0.086
0.3	0.40	0.96	0.080	0.166
0.4	0.50	1.26	0.105	0.271
0.5	0.59	1.48	0.124	0.395
0.6	0.68	1.54	0.129	0.524
0.7	0.76	1.74	0.145	0.669
0.8	0.85	1.68	0.141	0.810
0.9	0.92	1.38	0.115	0.925
1.0	1.00	0.90	0.075	1.000
		11.95	1.000	

a_1. It will, however, gradually increase as the water from upstream points on the basin arrives. Eventually, when water from A reaches O in some time t_c after the commencement of the storm, the discharge at O will be maximum.

For small drainage basins, Ramser[29] has determined the time of concentration by noting the time required for the water in the channel at the gaging station to rise from the low to the maximum stage as recorded by the water-stage recorders. Based on these and other data, R. R. Rowe (from a private correspondence dated March 22, 1957) obtained an equation for the time of concentration t_c in hours as follows:

$$t_c = (11.9L^3/H)^{0.385} \tag{21}$$

in which L is the length of basin area in miles, measured along the watercourse from the gaging station and in a direct line from the upper end of the watercourse to the farthest point on the drainage basin, and H is the fall in feet of the basin from the farthest point on the basin to the outlet of runoff. The equation was developed by empirical compromise of data from many sources. The lower values of t_c obtained by Equation 21 closely follow Ramser's data of 1927 as reduced by Kirpich.[89] Above 30-min. concentrations, use was made of a summary of many unit hydrograph computations for bridge and culvert design by several engineers from the Bridge Department of the California Division of Highways. Equation 21 can be modified and expressed as

$$t_c = 0.00013\,(L/\sqrt{S})^{0.77} \tag{22}$$

in which L is in feet and $S = H/L$ or approximately the average slope of the drainage basin in feet per foot.

Now divide the circular basin into ten parts with areas a_1, a_2, a_3, etc., in acres. The division is made by dividing the stream length into ten equal

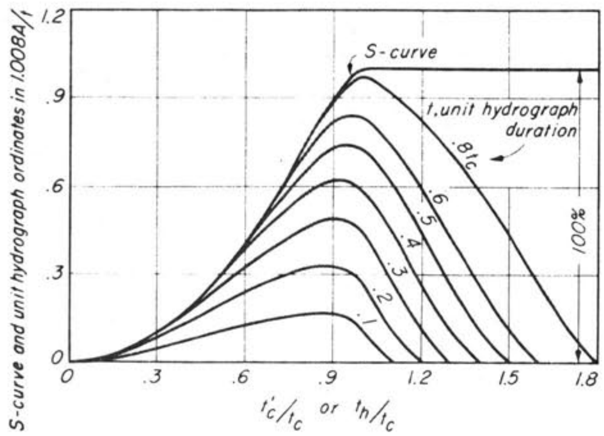


Figure 11. S-curve and unit hydrographs for a hypothetical circular drainage basin

parts, and arcs passing through the divided points are drawn with their common center at the outlet O. In case of a real drainage basin, division should be made in accordance with the topographic condition.

Let t_c' be the time of concentration for any subarea a and x be the length of the subdivided basin, then Equation 22 gives

$$t_c' = 0.00013\,(x/\sqrt{S})^{0.77} \tag{23}$$

Dividing Equation 23 by Equation 22, and assuming the slope of the sub-areas equal to the slope of the area,

$$\frac{t_c'}{t_c} = \left(\frac{x}{L}\right)^{0.77} \tag{24}$$

The computation for an S-curve of this basin is given in Table 10. Column 1 of this table gives values of x/L. Column 2 gives values of t_c'/t_c as computed by Equation 24. Column 3 gives the measured subdivided areas. Column 4 gives the proportioned sub-areas so that the sum of the areas is equal to unity. A value in this column is equal to the value in column 3 divided by the sum of the values in column 3, or 11.95. Column 5 gives the cumulative values of column 4. Since the uniform rainfall excess is assumed to be of an intensity of 1 in. per t hr., the runoffs in c.f.s. from the sub-areas are $1.008a_1/t$, $1.008a_2/t$, $1.008a_3/t$, etc. These are the rates that runoff is being generated on the upland parts of the sub-areas. Their summation is predicated on those rates combining in phase as they progress downstream to an equilibrium value of $1.008A/t$. To attain this equilibrium the duration of rainfall excess must continue at the constant rate for a period equal to or greater than t_c for A.

Table 11

Computation of the Built-Up Unit Hydrograph for a Hypothetical Circular Drainage Basin

Duration $t = 0.1t_c$

t/t_c (1)	S-curve (2)	Offset S-curve (3)	Unit Hydrograph Ordinates (4)	Moment (5)
0.1	0.008	...	0.008	0.0008
0.2	0.025	0.008	0.017	0.0034
0.3	0.088	0.025	0.063	0.0189
0.4	0.166	0.088	0.078	0.0312
0.5	0.270	0.166	0.104	0.0520
0.6	0.405	0.270	0.135	0.0810
0.7	0.561	0.405	0.156	0.1092
0.8	0.725	0.561	0.164	0.1313
0.9	0.895	0.725	0.170	0.1530
1.0	1.000	0.895	0.105	0.1050
		1.000	0.000	0.0000
			1.000	$0.6858t_c$
			$0.5 \times 0.1t_c =$	$0.0500t_c$
			Lag $= t_o =$	$0.6358t_c$

Therefore, the values in column 5 are the cumulative runoffs expressed in fractions of $1.008A/t$. By plotting these cumulative runoffs against the values in column 2 and smoothing the plot, a dimensionless S-curve can be obtained as shown in Figure 11. The times of concentration t_c' marked along the abscissa of the curve are actually equal to the times since the beginning of the rainfall excess, when the flows arrive at the gaging station. It is possible therefore to replace them by a continuous time unit t_h which is independent of the mode of subdivision of the area.

After the S-curve is constructed, a unit hydrograph of duration equal to t may be constructed by the method described previously. For simplicity of computation, the S-curve ordinates are read from the S-curve in Figure 11 for every tenth of t_h/t_c, and the values are listed in columns 1 and 2 in Table 11.

Considering a duration of the unit hydrograph equal to $t = 0.1\,t_c$, the computation of the unit hydrograph is shown in columns 1 to 5 of Table 11. Column 1 gives values of t_h/t_c in tenths. Column 2 gives values of S-curve ordinates in fractions of $1.008A/t$ from Figure 11. Column 3 is the offset values of S-curve ordinates for a time interval equal to the unit hydrograph duration t. Column 4 gives the difference between values in column 2 and column 3. This column gives the ordinates for the unit hydrograph. According to the definition of a unit hydrograph, the sum of these ordinates should be equal to a corresponding rainfall excess of 1 inch. Since the intensity of rainfall excess is $1/t$ and the duration is t, the amount of rainfall excess is equal to 1. The ordinates of the unit hydrograph are therefore expressed in fractions of $1.008A/t$.

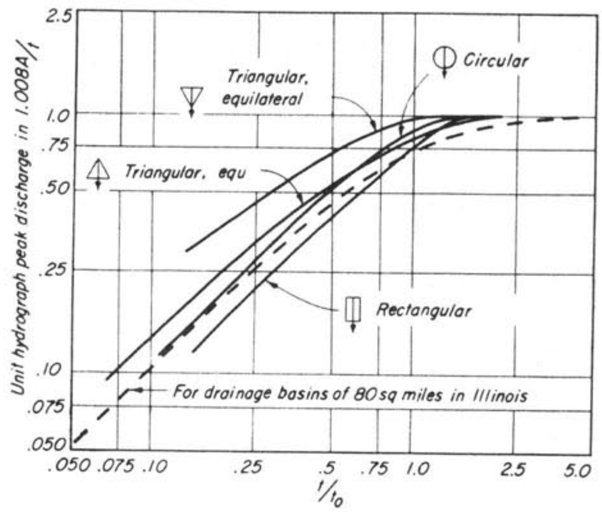


Figure 12. Dimensionless plot of unit hydrograph peak against duration for hypothetical drainage basins

This unit is the same as that used for the S-curve. The peak discharge of the unit hydrograph is found to be 0.170 of $1.008A/t$. The unit hydrograph is plotted as shown in Figure 11.

The lag t_o of the unit hydrograph is equal to the time interval between the center of mass of rainfall excess to the center of mass of runoff. The center of mass of rainfall excess is equal to $0.5t = 0.5 \times 0.1t_c = 0.05t_c$ from the beginning of the rainfall excess. The center of mass of runoff may be computed as shown in column 5 of Table 11. The values in this column are the moments of unit hydrograph ordinates about $t_h = 0$, or equal to the products of values in column 1 and the corresponding values in column 4. The sum of the values is equal to $0.6858t_c$. The moment arm is equal to $0.6858t_c/1.000 = 0.6858t_c$. Thus, the lag should be equal to $t_o = 0.6858t_c - 0.05t_c = 0.6358t_c$. Hence, $t/t_o = 0.1t_c/0.6358t_c = 0.157$. This ratio corresponds to the abscissa of the plots in Figure 9.

From the above computation for a unit hydrograph duration equal to $0.1t_c$, a point with coordinates of $Z = 1.008A/Pt = 0.170$ and $t/t_o = 0.157$ can be plotted in Figure 12. Points with coordinates for other durations can also be computed and plotted, resulting in a dimensionless curve of unit hydrograph peak versus duration. The unit hydrographs for the hypothetical circular basin for other durations are shown in Figure 11.

Similarly, such plots for hypothetical rectangular and triangular basins (Figure 10) have also been computed as shown in Figure 12. A comparison of these curves indicates the effect of the shape

Table 12

Computation of the Unit Hydrograph Ordinates for a Storm of 6-Hour Duration Having a Uniform Rainfall Distribution

t_h in hr. (1)	t_h/t_o (2)	S-curve Ordinates (3)	S-curve Offset (4)	Unit Hydrograph Ordinates (5)
0.5	0.05	0.12		0.12
1.0	0.10	0.72		0.72
1.5	0.15	2.02		2.02
2.0	0.20	4.17		4.17
2.5	0.25	7.27		7.27
3.0	0.30	11.10		11.10
3.5	0.35	15.48		15.48
4.0	0.40	20.41		20.41
4.5	0.45	25.64		25.64
5.0	0.50	30.89		30.89
5.5	0.55	35.83		35.83
6.0	0.60	40.47		40.47
6.5	0.65	44.82	0.12	44.70
7.0	0.70	48.89	0.72	48.17
7.5	0.75	52.69	2.02	50.67
8.0	0.80	56.23	4.17	52.06
8.5	0.85	59.52	7.27	**52.26**
9.0	0.90	62.57	11.10	51.47
9.5	0.95	65.39	15.48	49.91
10.0	1.00	67.99	20.41	47.58
etc.	etc.	etc.	etc.	etc.

Note: Values in columns 3, 4, and 5 are in units of per cent of $1.008A/t$.

of a basin. Apparently, the rectangular shape with its width narrower than the length has a high retention effect and hence shows low unit hydrograph peaks. On the other hand, the triangular shapes produce higher peaks and the circular shape gives average peak values.

The unit hydrograph peak vs. duration curve developed in Figure 9 for drainage basins in Illinois of 80 sq. mi. in size is also plotted in Figure 12 for comparison. This curve fits in a position between the circular and rectangular shapes.

It should be noted that the built-up method is based on a number of hypotheses which are unrealistic in actual drainage basins. Therefore, the results of computation by this method will have more qualitative than quantitative significance. Because of the hypotheses the theoretical results may deviate considerably from the actual values.

E. VARIATION OF RAINFALL INTENSITY IN A STORM

In the derivation of a unit hydrograph, a constant value of rainfall intensity is assumed within the period of the rainfall excess. The actual distribution of rainfall intensity, however, is rarely uniform in a storm. Based on a study of several typical rainfall distributions, the U. S. Soil Conservation Service has recommended an average pattern of rainfall distribution for use in general cases.[37] This pattern as given in the top part of Table 13 is based on the maximum amount of rainfall experienced for durations of up to 6 hours.

This SCS pattern of rainfall distribution and the synthetic S-curves in Illinois (Table 9) by the U. S. Geological Survey will be used in the following analysis for the study of the effect of the time variation of rainfall intensity upon runoff peak discharges. Since the pattern of infiltration loss is unknown, it will be assumed that the SCS pattern of rainfall distribution is approximately equal to that of the rainfall excess. It should be noted therefore that the analysis will be made only on a hypothetical basis, assuming $t/t_o = 0.60$ and $t = 6$ hr., then $t_o = 10$ hr. Using an average value of $k = 1.05$, Equation 19 gives $D = 43$ sq. mi. The analysis is thus applicable to the watershed of a size equal to 43 sq. mi., and it is given below.

Based on the USGS data, the hydrograph ordinates for a storm of 6-hr. duration, having uniform rainfall distribution, are computed as shown in Table 12. In the table, column 1 gives the time t_h in hours, column 2 gives t_h/t_o, column 3 gives S-curve ordinates from the USGS data (Table 9), column 4 gives the offset S-curve ordinates, and column 5 gives the unit hydrograph ordinates which are the differences between the values in columns 3 and 4. The unit hydrograph ordinates are expressed in per cent of $1.008A/t$. It can be seen that for $t = 6$ hr. the peak discharge is 52.3% of $1.008A/t$.

Table 13 shows the computation of the hydrograph ordinates for a storm of 6-hr. duration having a pattern of rainfall distribution recommended by the SCS. In the table, column 1 gives the time t_h in hours, column 2 gives t_h/t_o, column 3 gives the ordinates of unit hydrograph for a duration of $t = 0.5$ hr. as derived from the USGS data in a similar way to that used in Table 12, columns 4 to 15 give runoff contributed by different rainfall depths in the storm, and column 16 gives the resulting hydrograph. The peak discharge is found to be 4.6129% of $1.008A/t$, in which $t = 0.5$ hr. This is equivalent to $4.6129 \times 12 = 55.4\%$ of $1.008A/t$ if $t = 6$ hr.

From the above computation, the peak discharge for uniform distribution of rainfall intensity was found to be 52.3% of $1.008A/t$ with $t = 6$ hr., and for the non-uniform distribution recommended by the SCS it is 55.4% of $1.008A/t$ with $t = 6$ hr. Therefore, the effect of the non-uniform distribution according to the pattern recommended by the SCS is to increase the peak discharge about 6.0%. It should be reminded that this analysis is based on an average condition of $t/t_o = 0.6$ and $D = 43$

Table 13

Computation of the Hydrograph Ordinates in Per Cent of Equilibrium Runoff for a Storm of 6-Hour Duration Having a Rainfall Distribution Pattern Developed by the Soil Conservation Service

t_k in hr.	t_k/t_o	Ordinates of 0.5 hr. Unit Hydrograph	Rainfall Distribution, Expressed as Ratio to Total Rainfall During a 6-Hour Period, for Half-hour Time Increments .035	.045	.055	.095	.370	.105	.075	.055	.050	.040	.040	.035	Σ
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)
0.5	.05	0.12	.0042												.0042
1.0	.10	0.60	.0210	.0054											.0264
1.5	.15	1.30	.0455	.0270	.0066										.0791
2.0	.20	2.15	.0753	.0585	.0330	.0114									.1782
2.5	.25	3.10	.1086	.0966	.0715	.0570	.0444								.3781
3.0	.30	3.83	.1340	.1395	.0726	.1235	.2220	.0126							.7042
3.5	.35	4.38	.1534	.1724	.1706	.2150	.4810	.0630	.0090						1.2644
4.0	.40	4.93	.1726	.1972	.2107	.2950	.7960	.1365	.0450	.0066					1.8596
4.5	.45	5.23	.1833	.2220	.2410	.3640	1.148	.2260	.0975	.0330	.0060				2.5593
5.0	.50	5.25	.1838	.2356	.2710	.4160	1.417	.3255	.1613	.0715	.0300	.0048			3.1165
5.5	.55	4.94	.1730	.2362	.2880	.4690	1.620	.4020	.2325	.1183	.0650	.0240	.0048		3.6328
6.0	.60	4.64	.1623	.2220	.2890	.4976	1.824	.4600	.2875	.1705	.1075	.0520	.0240	.0042	4.1006
6.5	.65	4.35	.1522	.2090	.2720	.4990	1.937	.5176	.3290	.2108	.1550	.0860	.0520	.0210	4.4406
7.0	.70	4.07	.1424	.1960	.2550	.4695	1.942	.5500	.3700	.2410	.1915	.1240	.0860	.0455	**4.6129**
7.5	.75	3.80	.1330	.1831	.2395	.4410	1.824	.5510	.3927	.2716	.2190	.1533	.1240	.0753	4.6075
8.0	.80	3.54	.1240	.1710	.2240	.4130	1.717	.5190	.3940	.2880	.2465	.1752	.1533	.1086	4.5336
8.5	.85	3.29	.1152	.1594	.2090	.3870	1.610	.4870	.3700	.2890	.2615	.1973	.1752	.1340	4.3946
9.0	.90	3.05	.1068	.1480	.1948	.3610	1.505	.4570	.3480	.2720	.2625	.2095	.1973	.1534	4.2153
9.5	.95	2.82	.0988	.1372	.1810	.3363	1.405	.4276	.3262	.2550	.2470	.2100	.2095	.1726	4.0062
10.0	1.00	2.60	.0910	.1270	.1680	.3130	1.310	.3990	.3060	.2395	.2320	.1978	.2100	.1833	'3.7766
10.5	1.05	2.39	.0838	.1170	.1551	.2900	1.218	.3720	.2850	.2240	.2175	.1856	.1978	.1838	3.5296
11.0	1.10	2.19	.0766	.1075	.1430	.2680	1.130	.3460	.2656	.2090	.2035	.1740	.1856	.1730	3.2818
11.5	1.15	2.00	.0700	.0985	.1315	.2470	1.044	.3200	.2465	.1950	.1900	.1630	.1740	.1623	3.0418
12.0	1.20	1.82	.0636	.0900	.1204	.2270	0.961	.2960	.2290	.1810	.1770	.1521	.1630	.1522	2.8123
12.5	1.25	1.65	.0578	.0820	.1100	.2080	0.885	.2730	.2115	.1680	.1645	.1418	.1521	.1424	2.5961
etc.	etc.	etc.	etc.	etc.	etc.	etc.	etc.	etc.	etc.	etc.	etc.	etc.	etc.	etc.	etc.

sq. mi. By assuming other values of t/t_o, different results would be obtained. However, it is believed that the effect of an average non-uniform rainfall distribution in most cases will not increase the peak discharge by more than 10%.

In order to support the above analysis, a theoretical study by Richards[90] may be quoted. He assumed two extreme cases of linear rainfall distribution: one with a heavy intensity at the beginning and zero at the end, and one with zero intensity at the beginning and a maximum at the end. The results of the study are shown in Table 14. It will be noted that with the increase of the duration t (or the time of concentration for maximum runoff as implied in Richards' study), the effect of the time variation of the rainfall intensity upon runoff peaks is nearly constant. For small drainage basins, the duration is generally less than 6 hr. The corresponding effects due to the two extreme cases are about +13% and −20% respectively. These cases, however, occur infrequently.

For a more comprehensive understanding on the effect of the rainfall intensity distribution, it would be necessary to analyze all available storm data in the Midwest of various durations and intensities, as well as the infiltration data. The work would not only be laborious but also unjustified, because the data vary in a wide range, and the magnitude of the effect is small. In the present study, it is therefore assumed that the average effect of non-uniform distribution of rainfall intensity increases the peak discharge due to uniform distribution by about 6.0%. This is, of course, more or less arbitrary but it is considered to be a reasonable value. In order to include this effect of non-uniform distribution of rainfall intensity in the determination of a peak discharge, the rainfall intensity used in the computation assuming a uniform distribution may be increased by the same per cent.

Table 14

Effect of Rainfall Distribution Pattern on Peak Discharge (Q_{max}) According to Richards[90]

t hrs.	Rainfall Distribution Pattern Delayed $\%Q_{max}$	Uniform $\%Q_{max}$	Advanced $\%Q_{max}$
1	88.5	100.0	106.5
3	81.9	100.0	112.0
5	79.7	100.0	113.2
7	78.5	100.0	113.9
10	77.6	100.0	115.2
25	76.8	100.0	116.1
50	76.0	100.0	116.7
100	75.6	100.0	117.5

V. DEVELOPMENT OF THE NEW METHOD

A. THE PROPOSED FORMULA

An intensive study of the many existing methods and a survey of current practice reveals that a desirable method for the determination of peak discharge for waterway opening design must satisfy the following requirements:

(1) It should consider the major climatic conditions in the area of the given drainage basin.

(2) It should consider the major physiographic conditions in the area of the given drainage basin.

(3) It should consider either a definite design frequency or a limiting design condition.

(4) It should be based on sound and simple hydrologic principles so that the practicing engineer can use it with confidence and understanding.

(5) It should be simple and practical so that a beginner can use it readily with ease.

(6) It should depend less on personal judgment and more on logical procedure so that the result will be relatively consistent among determinations by different individuals.

In order to satisfy the above requirements, a method has been derived which utilizes the concept of unit hydrographs and is based on unit hydrograph synthesis. It also makes use of principles and concepts discussed in Section IV and in Sections II-G and II-H. The development of the method is as follows:

The direct peak runoff from a drainage basin may be computed as a product of the rainfall excess and the peak discharge of a unit hydrograph, or

$$Q = R_e P \tag{25}$$

in which R_e = rainfall excess in inches for a given duration of t hrs.

P = unit hydrograph peak in c.f.s. per inch of direct runoff for the duration t hr. of rainfall excess

From Equation 20, using the concept of peak-reduction factor Z:

$$P = \frac{1.008AZ}{t} \tag{26}$$

Then, substituting Equation 26 in Equation 25,

$$Q = \frac{1.008R_eAZ}{t} \tag{27}$$

In this expression the factor $1.008R_e/t$ may be replaced by the product of two factors: X and Y. The factor X is a *runoff factor*, expressed by

$$X = \frac{R_{eu}}{t} \tag{28}$$

in which R_{eu} = rainfall excess at Urbana, Illinois, increased by 6.0% to allow for the effect of variable rainfall distribution in the duration t

The factor Y is a *climatic factor*. Assuming $R_e/R_{eu} = R/R_u$, this factor represents

$$Y = \frac{1.008R}{R_u} \tag{29}$$

in which R_u = rainfall in inches at Urbana, Illinois

R = rainfall in inches at other location

R/R_u = conversion factor for converting the rainfall at Urbana to that at other places in Illinois

Consequently, Equation 25 may be written as

$$Q = AXYZ \tag{30}$$

If the base flow at the time of the peak discharge is Q_b, then the design peak discharge is

$$Q_d = Q + Q_b \tag{31}$$

The factors involved in the proposed formula (Equation 30) will be discussed in the following articles.

B. FACTORS AFFECTING RUNOFF

The factors affecting runoff considered in the proposed method can be divided into two groups. One group affects directly the amount of rainfall excess or direct runoff and it consists mainly of land use, surface condition, and soil type, and the amount and duration of rainfall. The other group affects the distribution of direct runoff and it in-

Table 15

Selection of Runoff Number *N*

Land Use or Cover	Surface Condition	Soil Type A	Soil Type B	Soil Type C	Soil Type D
Fallow	Straight row	77	86	91	94
Row crops	Straight row	70	80	87	90
	Contoured	67	77	83	87
	Contoured and terraced	64	73	79	82
Small grains	Straight row	64	76	84	88
	Contoured	62	74	82	85
	Contoured and terraced	60	71	79	82
Legumes (closed-drilled or broadcast) or rotation meadow	Straight row	62	75	83	87
	Contoured	60	72	81	84
	Contoured and terraced	57	70	78	82
Pasture or range	Poor	68	79	86	89
	Normal	49	69	79	84
	Good	39	61	74	80
	Contoured, poor	47	67	81	88
	Contoured, normal	25	59	75	83
	Contoured, good	6	35	70	79
Meadow (permanent)	Normal	30	58	71	78
Woods (farm wood lots)	Sparse or low transpiration	45	66	77	83
	Normal	36	60	73	79
	Dense or high transpiration	25	55	70	77
Farmsteads	Normal	59	74	82	86
Roads	Dirt	72	82	87	89
	Hard surface	74	84	90	92
Forest	Very sparse or low transpiration	56	75	86	91
	Sparse or low transpiration	46	68	78	84
	Normal	36	60	70	76
	Dense or high transpiration	26	52	62	69
	Very dense or high transpiration	15	44	54	61
Impervious surface		100	100	100	100

Table 16

Hydrologic Soil Groups in Illinois

Group A — Soil Series			
Perks			
Plainfield			
Morocco			
Chute			
Kilbourne			
Group B — Soil Series			
Landes	Port Byron	Sable	Carmi
Sharon	Joy	Atterbery	Brenton
Bold	Biggsville	Dresden	Capron
Maumee	Mt. Carroll	St. Charles	Miami
Biggs	Fall	Clinton	Birkbeck
Billett	Alvin	Iona	Ringwood
Worthen	Metea	Selma	Tama
Tallula	Decorra	Drummer	Saybrook
Hagener	Hopper	Lisbon	Sidell
Littleton	Timula	Beaver	Proctor
Disco	Seaton	Clary	Muscatine
Lorenzo	Allison	Elco	Flanagan
Sumner	Sawmill (Illinois)[a]	Warsaw	Ellison
			Fayette
Group C — Soil Series			
Petrolia	Edgington	Ashkum	Virgil
Ambraw	Oconee	Virden	Fay
Beaucoup	O'Fallon	Varna	Westville
Elliott	Richview	Bluford	Starks
Millbrook	Berwick	Stoy	Grantsburg
Shiloh	Hosmer	Bogota	Andres
Herrick	Rankin	Ava	
Group D — Soil Series			
Jacob	Brooklyn	Lukin	Schapville (Illinois)[a]
Darwin	Milroy	Rantoul	Clarence
Denny	Niota	Bryce	Denrock
Osceola	Mones	DeSoto	Huey
Okaw	Weir	Whitson	Racoon
Wynoose	Henry	Flora	
Rinard	Colp		Cisne

[a] This is where Illinois places these soils. Iowa has grouped Sawmill and Schapville differently.

cludes the size and shape of the drainage basin, the land slope, and a time measure of detention effect such as the lag time. This distribution of direct runoff is expressed in terms of the unit hydrograph.

There may be a certain interdependence existing between the two groups of factors described above. This interdependence is however unknown and for practical purposes it may be assumed that it does not affect the relationship between the direct runoff and rainfall excess. This assumption forms the basis upon which Equation **25** is established.

After an extensive search for information pertinent to the study of the runoff condition in small drainage basins, it was found that the data used in the method of hydrograph synthesis by the U. S. Soil Conservation Service (Section II-G) can be used for the evaluation of the rainfall excess or direct runoff. These data, however, should be modified in order to meet the present purpose.

For the present investigation, an average hydrologic condition of drainage basins was assumed. This condition represents the average conditions of antecedent moisture content and groundwater influence during the optimum time of the year.

The hydrologic soil-cover complex numbers used by SCS to describe different land uses, surface conditions, and soil types were modified with the aid of other data and renamed the *runoff number*, *N*. Table **15** lists the runoff numbers thus obtained. The soil types are classified by SCS in accordance with the runoff characteristics of the material into four hydrologic soil groups A, B, C, and D as defined in Section II-G.

For the convenience of identifying the soil types in Illinois, a map (Figure 13) was prepared in accordance with a study made at the University of Illinois.[84] On this map, different hydrologic soil types are clearly indicated. In certain parts of the State where the soil has a variable textured, stratified alluvium, the hydrologic soil type is described as "variable" and its runoff number may be taken as the average of the runoff numbers of the types A, B, C, and D.

For a more specific identification of the soil type, a list of "Hydrologic Soil Groups for Illinois" has been prepared by the State Office of the U. S. Soil Conservation Service. This list is given in Table 16, showing various names of soils found in Illinois grouped under the four soil types.

For a more or less homogeneous runoff condition, the runoff number can be obtained directly from Table 15 with the aid of the soil type map (Figure 13) or the soil type list (Table 16). For a composite runoff condition, a weighted runoff number should be determined. For example, when a

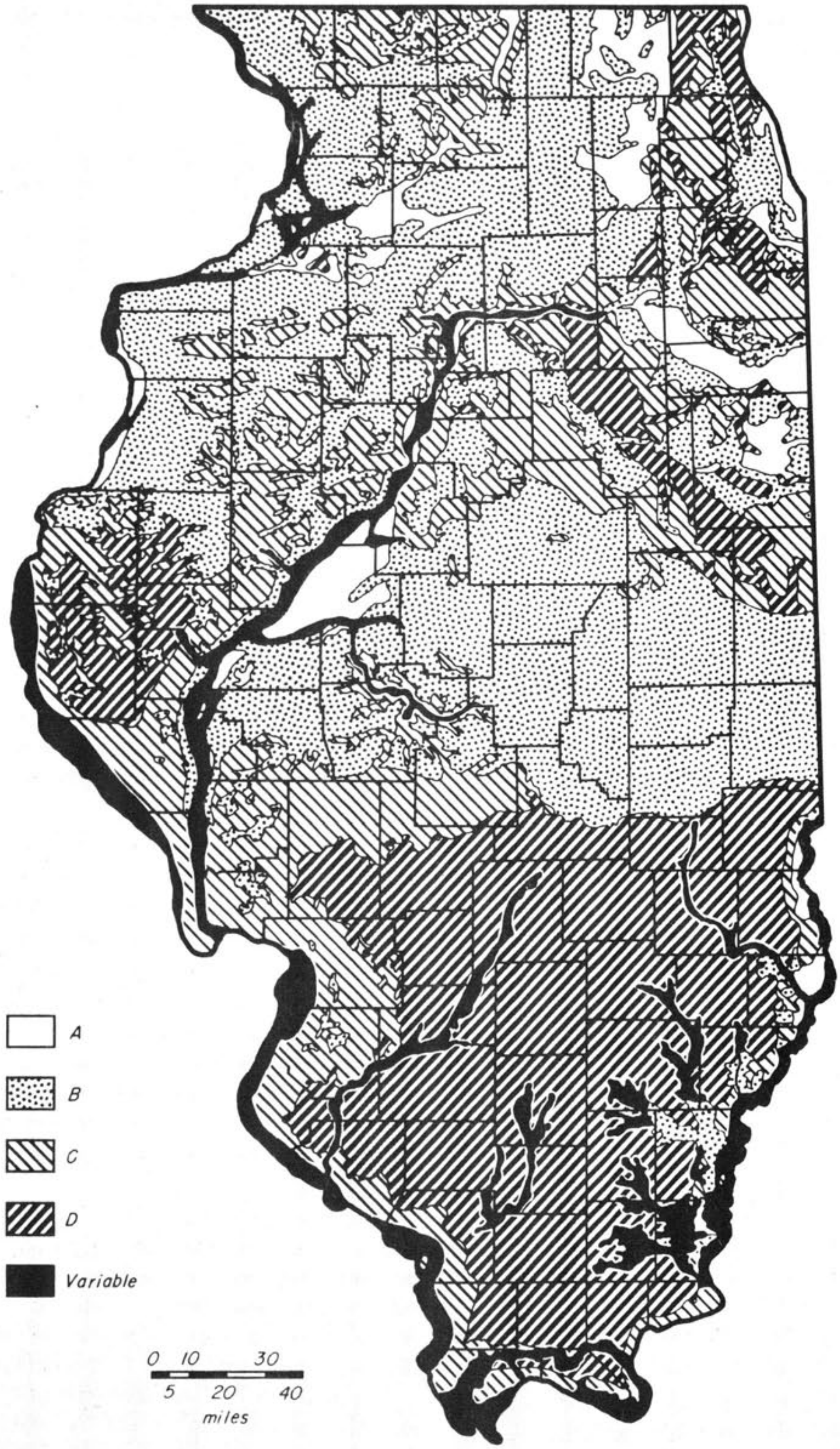


Figure 13. Soil types in Illinois

drainage basin contains 37.4% of impervious area and the remaining area of meadow, the weighted runoff number is computed as follows:

Cover	Percentage		Runoff No.		Product
Impervious Surface	37.4	×	100	=	37.4
Meadow	62.6	×	58	=	36.3
			Sum	=	73.7

The weighted runoff number is 73.7.

After the runoff number is determined, the value of R_e or rainfall excess for a given rainfall depth can be computed or found directly from the chart in Figure 14 developed by the U. S. Soil Conservation Service.[37] The chart shows the relationship between the direct runoff (R_e) and the rainfall (R) for different runoff numbers (N). The curves in this chart were computed by Equation 12 which is rewritten using the present notation as:

$$R_e = \frac{\left(R - \frac{200}{N} + 2\right)^2}{R + \frac{800}{N} - 8} \qquad (32)$$

For example, with $N = 73.7$ and $R = 3$ in., the direct runoff from either the chart or Equation 32 is $R_e = 0.89$ in.

C. DETERMINATION OF RUNOFF FACTOR X

When the runoff number is determined, the rainfall excess or direct runoff for a given rainfall can be either computed by Equation 32 or determined from the curves in Figure 14. Then, the runoff factor X for duration t of the rainfall can be computed by Equation 28.

For rainfalls of 5-year frequency at Urbana, Illinois, the computation for the runoff factor X is given in Table 17. In this table, column 1 gives the assigned rainfall duration t in hours. Column 2 gives the corresponding rainfall in inches obtained from the rainfall frequency curves in Figure 5, with an increase of 6.0% to account for the effect of nonuniform distribution of rainfall in an average storm. Columns 3 to 11 give the runoff factor X for runoff numbers $N = 100$ to 60. The value of X is computed by Equation 28. For example, when $t = 0.1$ hr., the rainfall for 5-year frequency from Figure 5

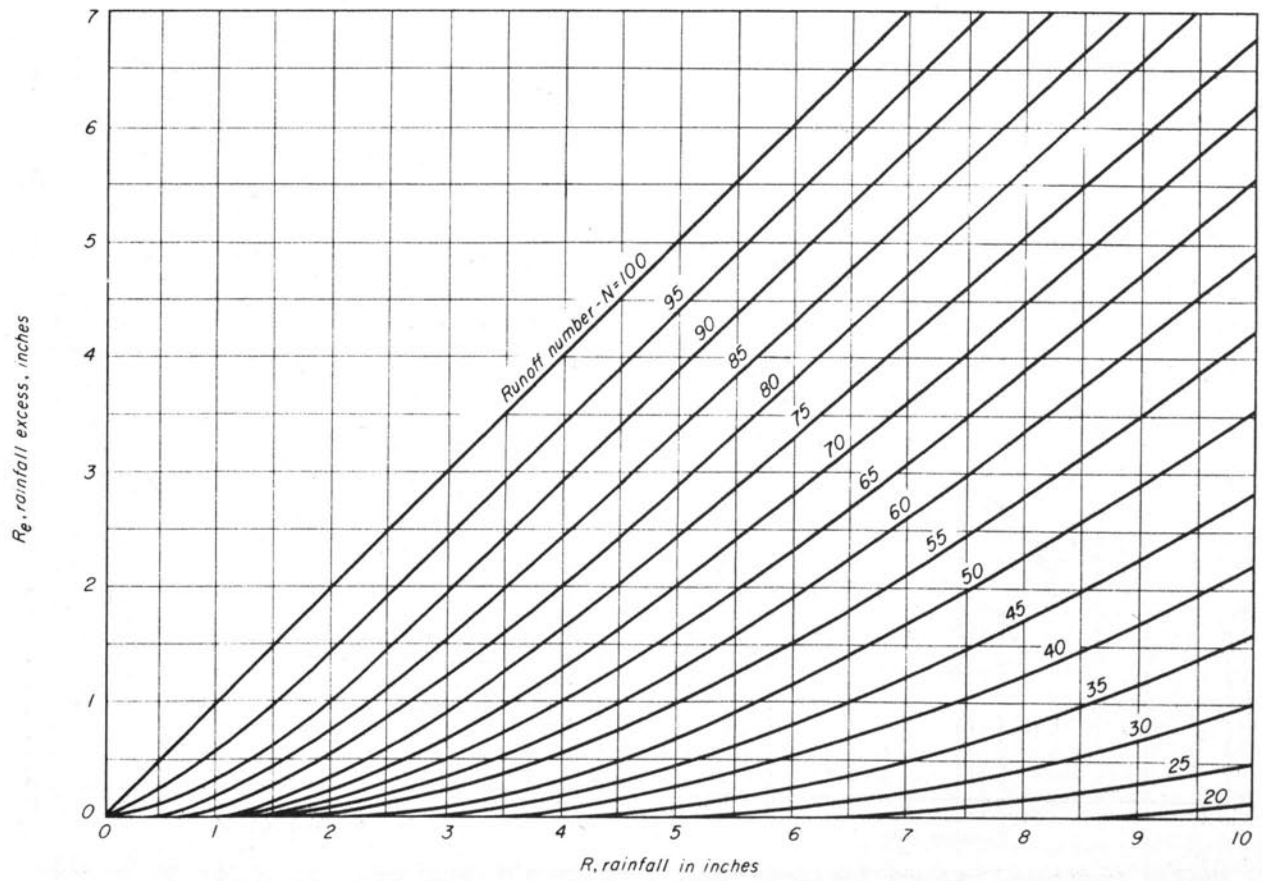


Figure 14. Relationship between rainfall and rainfall excess for various runoff numbers

is 0.52 inches. With an increase of 6.0%, the rainfall is 0.55 inches. For $N = 95$, the rainfall excess from Equation 32 or Figure 14 is 0.30. By Equation 28, $X = 3.00$.

Similarly, computations of runoff factor X for frequencies of 10, 25, 50, and 100 years are given in Tables 18, 19, 20, and 21, respectively. The computed values are represented graphically in Figures 15, 16, and 17. The value of runoff factor X for given frequency, runoff number, and duration can be readily interpolated from the curves in these graphs.

D. DETERMINATION OF CLIMATIC FACTOR Y

The climatic factor Y is represented by Equation 29. This is equal to the conversion factor

Table 17

Computation of Runoff Factor X for 5-Year Frequency

Duration in hrs. (1)	Rainfall in in. (2)	Runoff Factor X for N equal to 100 (3)	95 (4)	90 (5)	85 (6)	80 (7)	75 (8)	70 (9)	65 (10)	60 (11)
0.10	0.55	5.50	3.00	1.50	0.40	0.00	0.00	0.00	0.00	...
0.20	0.85	4.25	2.50	1.30	0.70	0.25	0.00	0.00	0.00	...
0.30	1.07	3.67	2.20	1.20	0.77	0.37	0.07	0.00	0.00	...
0.40	1.19	2.98	1.88	1.15	0.73	0.40	0.13	0.03	0.00	...
0.50	1.31	2.62	1.72	1.08	0.70	0.42	0.18	0.08	0.00	...
0.75	1.53	2.04	1.39	0.93	0.63	0.41	0.23	0.13	0.04	...
1.00	1.68	1.68	1.18	0.82	0.57	0.38	0.23	0.14	0.07	...
1.25	1.78	1.43	1.02	0.72	0.50	0.35	0.21	0.14	0.07	...
1.50	1.87	1.25	0.90	0.65	0.45	0.32	0.20	0.13	0.07	...
2.00	2.02	1.01	0.76	0.55	0.40	0.29	0.20	0.13	0.08	...
2.50	2.11	0.84	0.64	0.46	0.35	0.26	0.17	0.11	0.08	...
3.00	2.20	0.73	0.57	0.42	0.32	0.23	0.16	0.11	0.07	...
4.00	2.35	0.59	0.46	0.35	0.26	0.20	0.14	0.10	0.07	...
5.00	2.46	0.49	0.40	0.30	0.23	0.17	0.12	0.09	0.06	...
6.00	2.56	0.42	0.34	0.26	0.20	0.16	0.11	0.08	0.06	...
7.00	2.67	0.38	0.31	0.24	0.18	0.14	0.10	0.08	0.07	...
8.00	2.72	0.34	0.27	0.22	0.17	0.13	0.09	0.07	0.05	...

Table 18

Computation of Runoff Factor X for 10-Year Frequency

Duration in hrs. (1)	Rainfall in in. (2)	Runoff Factor X for N equal to 100 (3)	95 (4)	90 (5)	85 (6)	80 (7)	75 (8)	70 (9)	65 (10)	60 (11)
0.10	0.63	6.30	3.50	1.60	0.60	0.00	0.00	0.00	0.00	0.00
0.20	0.99	4.95	3.00	1.70	1.00	0.50	0.00	0.00	0.00	0.00
0.30	1.22	4.07	2.57	1.53	1.00	0.57	0.17	0.07	0.00	0.00
0.40	1.40	3.50	2.35	1.50	1.00	0.63	0.30	0.13	0.00	0.00
0.50	1.63	3.26	2.26	1.54	1.06	0.70	0.40	0.24	0.10	0.00
0.75	1.76	2.35	1.67	1.16	0.83	0.56	0.35	0.22	0.11	0.00
1.00	1.92	1.92	1.40	1.00	0.75	0.52	0.33	0.21	0.13	0.07
1.25	2.04	1.63	1.22	0.88	0.66	0.46	0.32	0.20	0.13	0.06
1.50	2.14	1.43	1.09	0.80	0.60	0.44	0.30	0.20	0.13	0.07
2.00	2.26	1.13	0.87	0.65	0.49	0.38	0.25	0.18	0.12	0.07
2.50	2.37	0.95	0.75	0.56	0.42	0.32	0.22	0.16	0.10	0.06
3.00	2.46	0.82	0.65	0.50	0.38	0.29	0.21	0.14	0.10	0.06
4.00	2.61	0.65	0.52	0.41	0.31	0.24	0.18	0.13	0.09	0.06
5.00	2.72	0.54	0.44	0.38	0.27	0.21	0.15	0.11	0.08	0.05
6.00	2.82	0.47	0.38	0.30	0.23	0.19	0.14	0.10	0.08	0.04
7.00	2.90	0.41	0.34	0.27	0.21	0.17	0.12	0.09	0.07	0.04
8.00	2.99	0.37	0.31	0.25	0.19	0.16	0.12	0.08	0.07	0.04

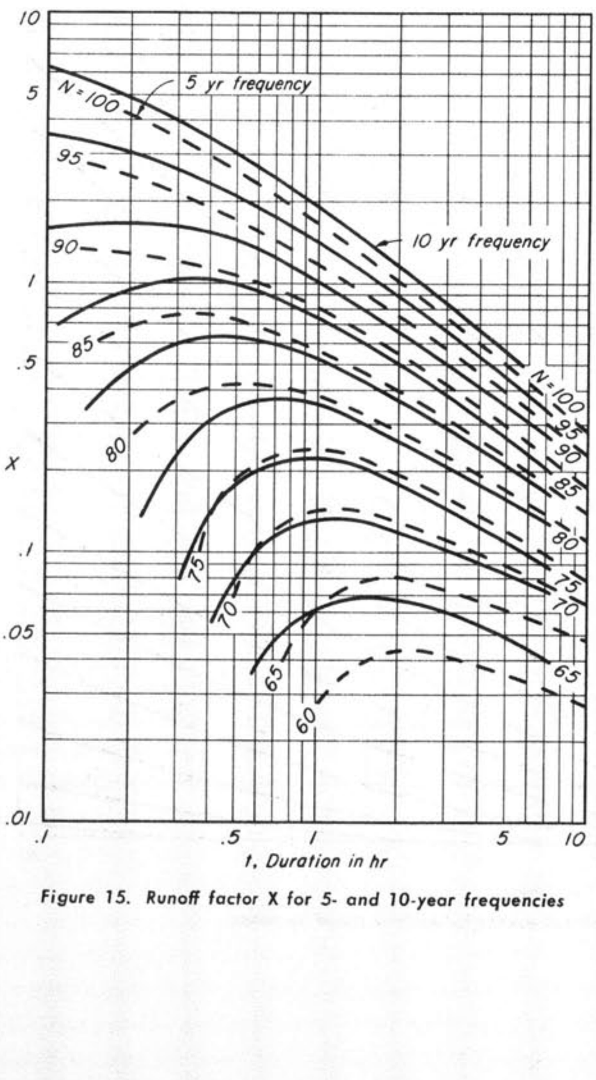


Figure 15. Runoff factor X for 5- and 10-year frequencies

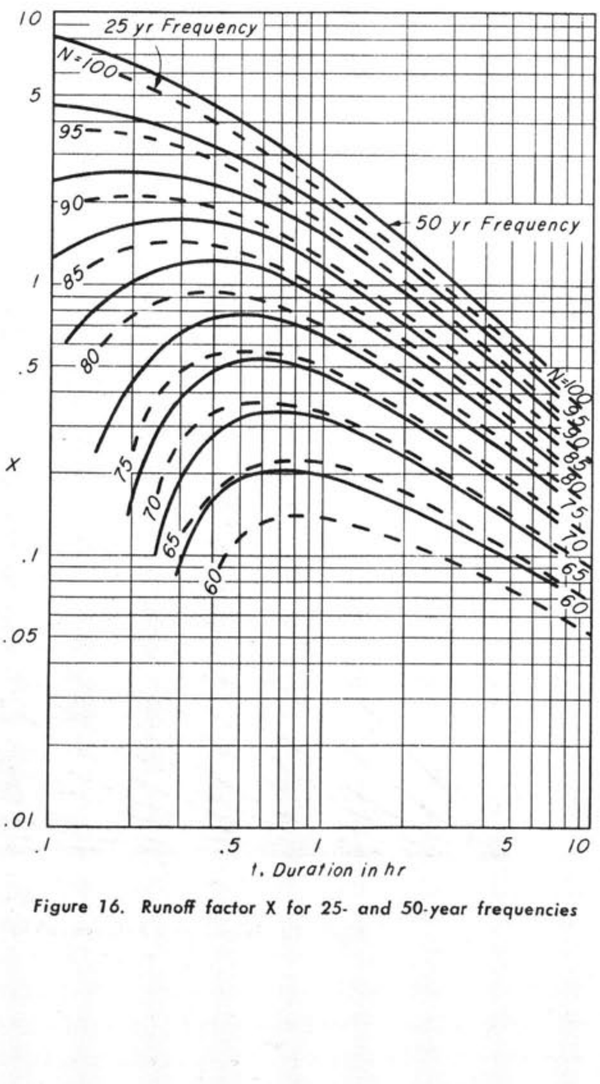


Figure 16. Runoff factor X for 25- and 50-year frequencies

shown in Figure 6 multiplied by 1.008. Figure 18 is a map which shows boundaries of sections and the average value of the climatic factor for each section.

E. DETERMINATION OF PEAK REDUCTION FACTOR Z

The peak-reduction factor Z represented by Equation 20 is equal to the ratio between the peak discharge of a unit hydrograph due to the rainfall of a given duration t and the equilibrium runoff or the runoff of the same rainfall intensity continuing indefinitely. In Figure 9, this ratio is shown to be a function of the ratio between duration t and lag t_o. For the purpose of developing the proposed method, however, the value of Z is to be represented by a function of the ratio between duration t and

Table 19

Computation of Runoff Factor X for 25-Year Frequency

Duration in hrs.	Rainfall in in.	Runoff Factor X for N equal to 100	95	90	85	80	75	70	65	60
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)
0.10	0.73	7.30	4.10	2.10	1.00	0.20	0.00	0.00	0.00	0.00
0.20	1.17	5.85	3.70	2.20	1.40	0.80	0.20	0.00	0.00	0.00
0.30	1.47	4.90	3.33	2.17	1.47	0.93	0.50	0.27	0.03	0.00
0.40	1.66	4.15	2.90	2.00	1.38	0.95	0.55	0.35	0.15	0.08
0.50	1.81	3.62	2.66	1.84	1.32	0.92	0.56	0.36	0.20	0.12
0.75	2.08	2.77	2.11	1.53	1.14	0.83	0.57	0.36	0.23	0.17
1.00	2.24	2.24	1.73	1.28	0.97	0.72	0.50	0.34	0.22	0.13
1.25	2.36	1.89	1.48	1.11	0.84	0.64	0.44	0.31	0.21	0.12
1.50	2.45	1.63	1.29	0.98	0.75	0.57	0.41	0.29	0.20	0.12
2.00	2.61	1.31	1.02	0.78	0.61	0.47	0.34	0.24	0.17	0.10
2.50	2.72	1.09	0.88	0.68	0.53	0.42	0.30	0.22	0.16	0.10
3.00	2.77	0.92	0.75	0.59	0.45	0.36	0.27	0.19	0.14	0.09
4.00	2.93	0.73	0.60	0.48	0.38	0.30	0.22	0.16	0.12	0.08
5.00	3.04	0.61	0.50	0.41	0.32	0.25	0.20	0.15	0.11	0.07
6.00	3.14	0.52	0.43	0.35	0.29	0.22	0.18	0.13	0.09	0.08
7.00	3.22	0.46	0.38	0.31	0.25	0.20	0.15	0.11	0.09	0.06
8.00	3.31	0.41	0.34	0.28	0.23	0.19	0.14	0.11	0.08	0.06

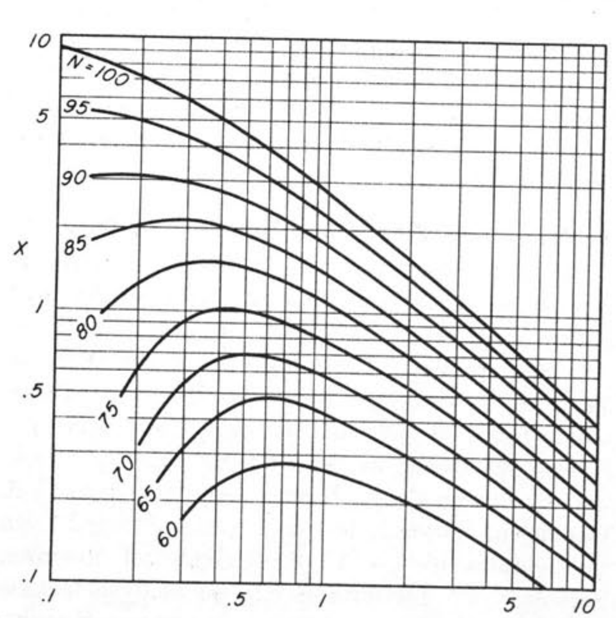


Figure 17. Runoff factor X for 100-year frequency

Table 20

Computation of Runoff Factor X for 50-Year Frequency

Duration in hrs.	Rainfall in in.	Runoff Factor X for N equal to 100	95	90	85	80	75	70	65	60
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)
0.10	0.83	8.30	4.80	2.50	1.30	0.50	0.00	0.00	0.00	0.00
0.20	1.31	6.55	4.25	2.65	1.65	1.05	0.40	0.15	0.00	0.00
0.30	1.64	5.47	3.76	2.57	1.77	1.20	0.67	0.40	0.17	0.00
0.40	1.87	4.68	3.37	2.40	1.75	1.20	0.75	0.50	0.25	0.18
0.50	2.05	4.10	3.05	2.24	1.66	1.30	0.80	0.54	0.34	0.20
0.75	2.34	3.12	2.44	1.83	1.39	1.07	0.73	0.51	0.35	0.20
1.00	2.54	2.54	2.02	1.55	1.18	0.92	0.66	0.47	0.33	0.20
1.25	2.66	2.13	1.71	1.34	1.03	0.80	0.58	0.42	0.30	0.18
1.50	2.77	1.85	1.49	1.18	0.92	0.71	0.53	0.39	0.28	0.18
2.00	2.90	1.45	1.18	0.95	0.87	0.58	0.44	0.33	0.24	0.15
2.50	3.02	1.21	1.00	0.81	0.64	0.51	0.39	0.29	0.21	0.14
3.00	3.09	1.03	0.85	0.69	0.55	0.44	0.34	0.25	0.20	0.13
4.00	3.24	0.81	0.68	0.55	0.45	0.36	0.28	0.21	0.16	0.11
5.00	3.36	0.67	0.56	0.46	0.37	0.31	0.24	0.18	0.14	0.10
6.00	3.46	0.58	0.49	0.40	0.32	0.27	0.21	0.16	0.12	0.09
7.00	3.53	0.51	0.43	0.36	0.29	0.24	0.18	0.15	0.11	0.08
8.00	3.62	0.45	0.39	0.32	0.26	0.21	0.17	0.13	0.10	0.08

Figure 18. Climatic factor Y for climatological sections in Illinois

Table 21

Computation of Runoff Factor X for 100-Year Frequency

Duration in hrs.	Rainfall in in.	Runoff Factor X for N equal to 100	95	90	85	80	75	70	65	60
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)
0.10	0.91	9.10	5.50	2.90	1.60	0.60	0.00	0.00	0.00	0.00
0.20	1.42	7.10	4.75	3.00	2.00	1.25	0.60	0.30	0.00	0.00
0.30	1.80	6.00	4.33	3.03	2.17	1.50	0.93	0.57	0.33	0.17
0.40	2.03	5.08	3.83	2.75	2.03	1.50	1.00	0.63	0.40	0.23
0.50	2.24	4.48	3.48	2.60	1.94	1.66	1.00	0.70	0.46	0.26
0.75	2.56	3.41	2.72	2.11	1.63	1.27	0.89	0.65	0.47	0.28
1.00	2.77	2.77	2.24	1.77	1.38	1.07	0.80	0.58	0.42	0.27
1.25	2.89	2.31	1.89	1.50	1.18	0.94	0.70	0.52	0.38	0.25
1.50	3.01	2.01	1.65	1.33	1.05	0.83	0.65	0.47	0.35	0.23
2.00	3.20	1.60	1.33	1.09	0.88	0.70	0.55	0.41	0.31	0.21
2.50	3.28	1.31	1.10	0.91	0.73	0.59	0.46	0.34	0.26	0.18
3.00	3.41	1.14	0.95	0.79	0.64	0.52	0.41	0.31	0.24	0.17
4.00	3.58	0.90	0.76	0.64	0.52	0.43	0.34	0.26	0.20	0.15
5.00	3.66	0.73	0.62	0.52	0.43	0.35	0.28	0.22	0.17	0.12
6.00	3.79	0.63	0.54	0.46	0.38	0.31	0.25	0.20	0.15	0.11
7.00	3.86	0.55	0.47	0.40	0.33	0.27	0.22	0.17	0.14	0.10
8.00	3.95	0.49	0.42	0.36	0.30	0.25	0.20	0.16	0.13	0.09

lag time t_p. The relationship between lag and lag time will be discussed later in this article.

1. Time Measure of Detention Effect

As stated in Section V-B, one of the physiographic characteristics defining the hydrograph of a drainage basin is a time measure of detention effect. This time element may be the time of concentration (t_c) in the rational method (Section II-D), the lag time (t_p) used in the method of hydrograph synthesis by the U. S. Agricultural Service as defined by Equation 9, or the lag (t_o) used by the U. S. Geological Survey as defined in Equation 19 for conditions in Illinois.

In search for a suitable time measure of detention effect in the proposed method, the lag time t_p was adopted. This lag time has been defined before. For an instantaneous unit hydrograph this lag time is equal to the time of rise from the beginning of runoff to the runoff peak. The instantaneous unit hydrograph is a hypothetical unit hydrograph whose duration of rainfall excess approaches zero as a limit, while maintaining a fixed amount of rainfall excess equal to 1 inch.

It may be pointed out that the lag time so defined may not exactly correspond to the classical concept of "time of concentration." For natural drainage basins of large size and of complex drainage pattern, runoff water originating from the most remote portion may and usually does arrive at the outlet too late to contribute to the peak flow. Accordingly, the lag time will generally be less than the time of concentration for a given basin. For small drainage basins with simple drainage patterns, lag time may be very close to the time of concentration. Moreover, critical peak flows from small basins are usually caused by rainfall due to thunderstorms of short duration. The durations are relatively so short that the mass of rainfall excess is practically concentrated near the beginning of the rise of hydrograph. Thus, the resulting unit hydrograph may approach an instantaneous unit hydrograph.

For small agricultural drainage basins, as mentioned previously in Section IV-D3, Ramser[29] has determined the time of concentration by noting the time required for the water in the channel at the gaging station to rise from the low to the maximum stage as recorded by the water-stage recorders. Thus, Ramser's time of concentration may be very close to the lag time, or $t_c = t_p$. Thus, Equation 22 becomes

$$t_p = 0.00013\ K^{0.77} \tag{33}$$

in which $K = L/\sqrt{S}$ is a lag-time factor, where L in feet measured up to the basin boundary and S in feet per foot are defined in Equation 22.

The lag time used in the method of hydrograph synthesis by the U. S. Agricultural Research Service, as described previously in Section II-G by Equation 9, is measured from the center of mass rainfall block to the peak discharge. For small drainage basins, however, this may be close to the rise of the instantaneous unit hydrograph, that is to the lag time defined in this study. This conclusion is supported by the results of a study of small drainage basins in arid-land.[36] In that study, it was found that the lag time depends mainly on the hydrograph shape and its relation to basin physiography and it is practically independent of the rainfall duration. In other words, the lag times of hydrographs of different durations for a given basin are practically all the same because theoretically they belong to the same instantaneous unit hydrograph of the basin.

2. Verification of Lag-Time Equations by Data

In order to verify the lag-time relationships represented by Equations 9 and 33, the data from a number of small drainage basins in the midwestern area are used. These data are shown in Table 22 covering drainage basins in Illinois, Indiana, Ohio, Iowa, Wisconsin, Missouri, and Nebraska. Except for basins at West Salem, Madden Creek, and Hurricane Creek, Illinois (obtained from U. S. Geological Survey), the data were obtained from publications by the U. S. Agricultural Research Service.[57, 60] The data used in the analysis include 20 drainage basins and 53 storms or 60 runoff peaks.

For each storm and basin, a unit hydrograph and the corresponding instantaneous unit hydrograph were derived; and thus, the duration, lag time, and unit hydrograph peak were determined. Most of the storms selected for analyses have a well-defined single peak of runoff. In cases of multiple peak runoff, the peaks of the hydrograph were first separated before making a unit hydrograph analysis.

For the derivation of an instantaneous unit hydrograph, the methods developed by O'Kelly,[91] Nash,[92, 93] or Dooge[94] may be used. As it can be shown that the integration of an area covered by an instantaneous unit hydrograph is proportional to the ordinate of an S-curve which is a hydrograph due to a uniform rainfall intensity of continuous duration. In other words, the instantaneous unit hydrograph represents the slope of the S-curve. The time at the point of inflection on the S-curve corresponds to the time of the peak flow of the instantaneous unit hydrograph. Therefore, an approximate procedure to determine the lag time is to construct the S-curve of a given direct-runoff hydrograph and then to locate its point of inflection.

Table 22 shows the computed values of lag-time factors K_o by Equation 10 and K, defined in Equation 33, and the average lag time for each drainage basin. The average lag time is taken from the computation in Table 23.

It should be noted that the slope S_o used for computing the lag-time factor K_o is the average land slope. The original land slopes given in the publications of the U. S. Agricultural Research Service[57, 61] are shown in percentage of the drainage area lying in each slope class. For example, the slopes of Watershed W-I at Edwardsville, Illinois, are given as "63% in 0-1.5% class; 21% in 1.5-4%; 9% in 4-7%; and 7% in 7-12%." The average land slope is therefore 2.21%.

The slope S used for computing the lag-time factor K may be determined by many methods.[95] Generally, the length of the channel is measured beyond the upper end of the clearly discernible stream channel to the drainage divide and then the difference in elevation between this point on the ridge line and the outflow point of interest in the channel is divided by the length. In the present determination, the profile of the stream was plotted using the elevation as the ordinate and the distance up the principal channel as the abscissa. Then, a straight line through the gaging point was fitted to the stream profile so that the area between the straight line and the stream profile lying below the line was equal to that lying above it. The slope of this "straight line of best fit" was taken as the channel slope. As a comparative study, broken lines of best fit were also tried, and the weighted average of the slopes was taken as the channel slope, but the results showed no significant differences. The single straight line of best fit was therefore used owing to its simplicity.

The computed lag-time factors are plotted against the lag time as shown in Figures 19 and 20. In Figure 19, Equation 9 is plotted as a straight line. Similarly, in Figure 20, Equation 33 is plotted as a straight line. It appears that the data do not satisfactorily fit either Equation 9 or Equation 33.

Table 22

Computation of Lag-Time Factors

No.	State	Drainage Basin	Size A acres	Visible Channel Length ft.	Drainage Density ft/ac	Land Slope S_o%	$K_o = \frac{A^{0.3}}{S_o\sqrt{D.D.}}$	Channel Slope S%	Channel Length L, ft.	$K = \frac{L}{\sqrt{S}}$	Av. Lag Time t_p in hr.
1	2	3	4	5	6	7	8	9	10	11	12
1	Illinois	W-1, Edwardsville	27.22	1,650	61.0	2.21	0.156	1.51	1,461	1.19×10^4	0.315
2		W-4, Edwardsville	289.8	19,800	68.32	5.37	0.128	1.29	5,765	5.08×10^4	0.443
3		W-1-A, Monticello	82.0	3,530	43.05	1.56	0.366	0.66	2,120	2.62×10^4	0.576
4		W-1-B, Monticello	45.5	2,053	45.13	1.15	0.407	0.54	2,750	3.75×10^4	0.436
5		West Salem	969	6,600	6.59	0.60	5.07	0.51	9,926	13.82×10^4	1.872
6		Madden Creek, Edwards County	992	6,440	6.49	1.87	1.664	0.58	8,848	10.69×10^4	1.809
7		Hurricane Creek Trib., Witt	92.2	3,500	38.0	2.00	0.315	0.60	1,900	2.45×10^4	0.625
8	Ohio	W-97, Coshocton	4,580	52,800	11.52	17.21	0.215	0.53	26,100	35.8×10^4	1.900
9		W-183 Coshocton	74.2	3,400	45.8	15.86	0.034	0.65	3,180	1.22×10^4	0.240
10		W-196, Coshocton	303	9,000	29.7	16.20	0.063	3.70	4,460	2.32×10^4	0.310
11		W-1, Hamilton	187	5,390	28.8	4.59	0.195	1.15	5,000	4.66×10^4	0.286
12	Missouri	W-3, Bethany	4.85 4.48	1,660 1,660	340 370	7.72	0.011 (av.)	6.25	675	0.27×10^4	0.117
13	Wisconsin	W-1, Fennimore	330	8,200	24.8	5.97	0.191	2.03	5,780	4.05×10^4	0.419
14		W-2, Fennimore	22.8	560	24.6	5.92	0.087	4.75	1,000	0.458×10^4	0.116
15		W-4, Fennimore	171	4,000	23.4	4.96	0.191	2.17	3,270	2.15×10^4	0.267
16		W-1, Colby	345	4,490	13.0	2.47	0.648	0.76	6,575	7.55×10^4	0.417
17	Indiana	W-5, Lafayette	2.87	280	97.6	1.93	0.072	1.41	566	0.48×10^4	0.139
18		W-6, Lafayette	2.79	240	86.0	2.26	0.065	1.48	585	0.48×10^4	0.165
19	Iowa	Ralston Creek, Iowa City	1,926	64,240	33.4	10.25	0.163	0.60	21,600	27.9×10^4	1.422
20	Nebraska	W-3, Hastings	481	30,500	63.4	5.30	0.151	0.55	9,000	12.1×10^4	0.636

Table 23

Computation of Lag Time and Peak-Reduction Factor Z

No.	State	Drainage Basin	Size A acres	Date of Storm	Duration t hours	Lag Time t_p hours	$\frac{t}{t_p}$	Unit Hydrograph Peak P c.f.s.	Z	Points for Plotting	Remarks
1	2	3	4	5	6	7	8	9	10	11	12
1	Illinois	W-1, Edwardsville	27.22	May 27, 1938	0.150	0.167	0.900	121.2	0.662	1(a)	
				June 21, 1942	0.267	0.517	0.516	35.74	0.348	1(b)	
				March 31, 1952	0.550	0.386	1.590	39.80	0.797	1(c)	Multiple Peaks
				March 31, 1952	0.383	0.229	1.675	56.16	0.785	1(d)	Do.
				July 2, 1952	0.317	0.277	1.145	62.05	0.717	1(e)	
					av.	0.315					
2		W-4, Edwardsville	289.8	May 27, 1938	0.533	0.413	1.292	508.1	0.927	2(a)	
				June 21, 1942	0.417	0.453	0.920	490.7	0.702	2(b)	
				March 31, 1952	0.550	0.467	1.179	333.8	0.629	2(c)	Multiple Peaks
				March 31, 1952	0.450	0.567	0.794	449.6	0.693	2(d)	Do.
				July 2, 1952	0.367	0.313	1.173	491.9	0.619	2(e)	
					av.	0.443					
3		W-1-A, Monticello	82.0	Oct. 21, 1949	0.167	0.595	0.280	92.5	0.187	3(a)	
				July 9, 1951	0.217	0.709	0.306	139.7	0.367	3(b)	Multiple Peaks
				July 9, 1951	0.200	0.423	0.473	67.1	0.162	3(c)	Do.
					av.	0.576					
4		W-1-B, Monticello	45.5	Oct. 21, 1949	0.150	0.436	0.344	51.5	0.168	4	
					av.	0.436					
5		West Salem	969	July 18, 1958	0.333	1.920	0.174	454.1	0.155	5(a)	
				July 19, 1958	0.167	1.825	0.091	503.8	0.086	5(b)	
					av.	1.872					
6		Madden Creek, Edwards County	992	July 18, 1958	0.333	1.867	0.179	455.0	0.152	6(a)	
				July 19, 1958	0.167	1.750	0.095	496.0	0.083	6(b)	
					av.	1.809					
7		Hurricane Creek Tributary, Witt.	92.2	April 5, 1958	0.233	0.583	0.400	113.8	0.286	7(a)	
				May 9,10, 1959	0.167	0.633	0.263	158.2	0.284	7(b)	
				Aug. 6, 1959	0.500	0.733	0.682	111.8	0.603	7(c)	Multiple Peaks
				Aug. 6, 1959	0.250	0.550	0.455	116.8	0.314	7(d)	Do.
					av.	0.625					
8	Ohio	W-97, Coshocton	4,580	June 4, 1941	0.167	2.250	0.047	3,005	0.108	8(a)	Multiple Peaks
				June 28, 1957	1.683	1.550	1.087	2,030	0.738	8(b)	Do.
					av.	1.900					
9		W-183 Coshocton	74.2	June 16, 1946	0.533	0.267	2.000	122.7	0.875	9(a)	Multiple Peaks
				Aug. 16, 1947	0.200	0.250	0.800	185.8	0.496	9(b)	
				Sept. 1, 1950	0.733	0.233	3.143	87.0	0.854	9(c)	Multiple Peaks
				Sept. 1, 1950	0.500	0.217	2.308	129.2	0.864	9(d)	Do.
				June 12, 1957	0.350	0.233	1.500	154.2	0.722	9(e)	Do.
					av.	0.240					
10		W-196 Coshocton	303	June 16, 1946	0.667	0.323	2.060	393	0.858	10(a)	Multiple Peaks
				Aug. 16, 1947	0.217	0.341	0.635	673	0.478	10(b)	
				Sept. 1, 1950	0.667	0.310	2.150	360	0.785	10(c)	Multiple Peaks
				Sept. 1, 1950	0.367	0.310	1.180	532	0.639	10(d)	Do.
				June 12, 1957	0.317	0.267	1.187	697	0.723	10(e)	Do.
					av.	0.310					
11		W-1, Hamilton	187	July 4, 1939	0.200	0.317	0.632	477	0.450	11(a)	Multiple Peaks
				July 7, 1943	0.167	0.255	0.654	370	0.328	11(b)	
					av.	0.286					
12	Missouri	W-3, Bethany	4.85	May 21, 1933	0.067	0.142	0.471	53.8	0.738	12(a)	
			4.48	Oct. 19, 1934	0.183	0.083	2.200	19.5	0.790	12(b)	
				June 17, 1935	0.083	0.125	0.667	43.4	0.797	12(c)	
					av.	0.117					
13	Wisconsin	W-1, Fennimore	330	Aug. 12, 1943	0.150	0.375	0.400	678	0.305	13(a)	
				July 1, 1944	0.167	0.545	0.306	491	0.246	13(b)	
				June 28, 1945	0.133	0.338	0.394	748	0.299	13(c)	
					av.	0.419					
14		W-2, Fennimore	22.8	June 28, 1945	0.100	0.082	1.224	120.8	0.525	14(a)	
				June 24, 1949	0.250	0.150	1.667	72.1	0.785	14(b)	
					av.	0.116					
15		W-4, Fennimore	171	July 11, 1944	0.083	0.375	0.222	318	0.153	15(a)	
				June 28, 1945	0.133	0.183	0.727	483	0.372	15(b)	
				June 24, 1949	0.300	0.242	1.242	376	0.654	15(c)	
					av.	0.267					
16		W-1, Colby	345	July 28, 1949	0.233	0.417	0.538	342	0.229	16	Multiple Peaks
					av.	0.417					
17	Indiana	W-5, Lafayette	2.87	July 5, 1943	0.067	0.100	0.667	30.4	0.704	17(a)	
				June 19, 1946	0.033	0.200	0.167	12.6	0.144	17(b)	
				June 24, 1950	0.133	0.117	1.143	16.0	0.735	17(c)	
					av.	0.139					
18		W-6, Lafayette	2.79	July 5, 1943	0.067	0.167	0.400	16.1	0.383	18(a)	
				June 19, 1946	0.117	0.208	0.560	13.4	0.557	18(b)	
				June 24, 1950	0.133	0.120	1.111	11.1	0.525	18(c)	
					av.	0.165					
19	Iowa	Ralston Creek, Iowa City	1,926	June 1, 1943	0.250	1.367	0.183	1,710	0.220	19(a)	Multiple Peaks
				July 1, 1950	1.617	1.400	1.155	978	0.816	19(b)	Do.
				July 18, 1956	0.333	1.500	0.222	1,778	0.305	19(c)	
					av.	1.422					
20	Nebraska	W-3, Hastings	481	June 20, 1939	0.250	0.625	0.400	612	0.315	20(a)	Multiple Peaks
				June 7, 1953	0.717	0.650	1.102	396	0.585	20(b)	
				July 15, 1957	0.217	0.633	0.342	770	0.344	20(c)	Multiple Peaks
					av.	0.636					

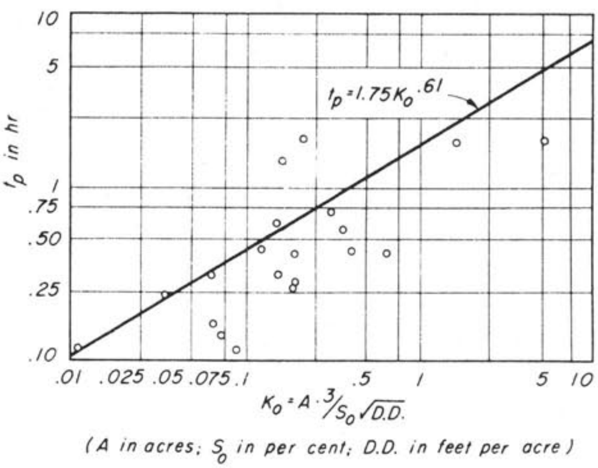


Figure 19. Verification of t_p-K_o relationship

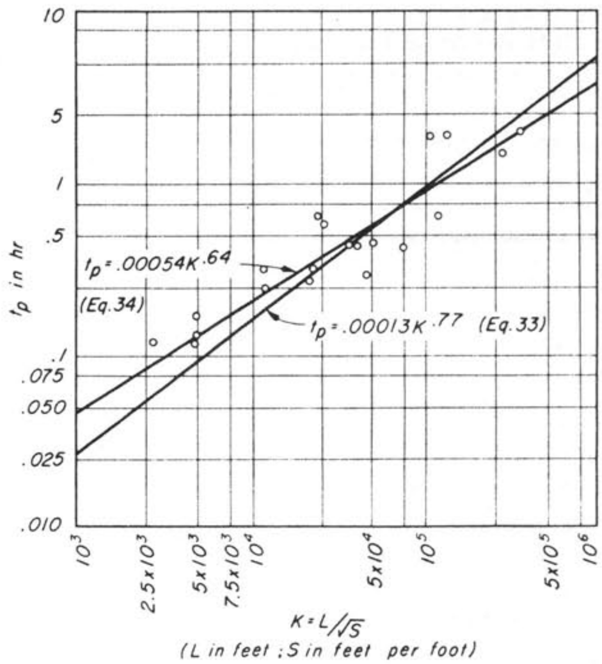


Figure 20. Verification of t_p-K relationship

In Figure 20, however, the plotted points do not scatter as broadly as they do in Figure 19, and they indicate a well-defined trend. Therefore, a line of best fit was drawn and its equation was found to be

$$t_p = 0.00054\ (L/\sqrt{S})^{0.64} \qquad (34)$$

When S is expressed in per cent, the above equation becomes

$$t_p = 0.00236\ (L/\sqrt{S})^{0.64} \qquad (35)$$

where L is in feet.

In deriving the line of best fit the method of least squares was used to compute two regression lines, one taking abscissas as independent variables and the other taking ordinates as independent variables. The line of best fit is the mean of the two regression lines; that is, its slope is equal to the mean slope of the regression lines and it passes through the centroid of the plotted data. For practical applications, Equation 35 is shown by a chart in Figure 21.

3. Conversion of Z Factor

In Figure 9 the factor Z is shown as a function of t/t_o where t is the duration of rainfall excess in hours and t_o is the lag. The lag can be converted to lag time by establishing a relationship between the lag t_o and lag time t_p. This relationship can be established theoretically by using the concept of instantaneous unit hydrograph.

Figure 22b shows the sketch of a unit hydrograph for the duration t of rainfall excess, with the lag t_o and the lag time t_p marked on it. This unit hydrograph was developed from an S-curve and the same S-curve in an offset position for a delay of rainfall excess equal to t as shown in Figure 22a. The principle and procedure for the development of the unit hydrograph have been described in Section IV-D. Figure 22c shows the instantaneous unit hydrograph derived from the same S-curve. The ordinates of the instantaneous unit hydrograph are proportional to the slope of the S-curve as mentioned previously. It can be seen from Figure 22c that the lag time t_p for instantaneous unit hydrographs is equal to the time of rise to the peak.

In the present discussion, the S-curve derived by the U. S. Geological Survey from the data for drainage basins of about 80 sq. mi. in Illinois was used.[82] This is plotted in Figure 23 with the abscissa expressed in terms of the lag t_o. The instantaneous unit hydrograph was derived from the slope of the S-curve. It can be seen from Figure 23 that the relationship obtained between t_o and t_p is as follows:

$$t_p = 0.47\ t_o \qquad (36)$$

From the above relationship, the curve for factor Z in Figure 9 can be readily modified to produce a new curve of factor Z versus the ratio t/t_p. Both the original and new curves are shown in Figure 24.

4. Verification of Factor Z by Data

In order to check the corrected curve for factor Z, the data for the verification of the lag time were used. The computation for factor Z by Equation 20 is shown in Table 23.

The computed values of Z are plotted against t/t_p as shown in Figure 25. The converted curve based on the unit hydrograph for drainage basins of an average size of 80 sq. mi. in Illinois is plotted in dashed line. Obviously, the plotted data do not fit the converted curve but are shown above the latter. This is reasonable, because the drainage basins under consideration are much smaller than 80 sq. mi. Figure 9 shows that the curve for smaller basins has a higher unit hydrograph peak or a higher value for factor Z.

To fit the plotted data, a curve in full line was constructed in Figure 25. It should be noted that the fitted curve is the converted curve displaced horizontally except its upper portion where Z approaches unity. Theoretically, Figure 22 indicates that t should not be greater than $2t_p$. Otherwise, the peak discharge would occur before the end of rainfall excess. At $t = 2t_p$ and greater, the unit hydrograph should reach and maintain a maximum value. In other words, $Z = 1$ at $t/t_p \geqq 2$. The upper portion of the curve was therefore drawn to satisfy this condition.

It is believed that the general relationship between Z and t/t_p should follow the trend of the converted curve, whereas the horizontal shifting of the data is due to the size of basin areas under consideration. In addition to the basin size effect,

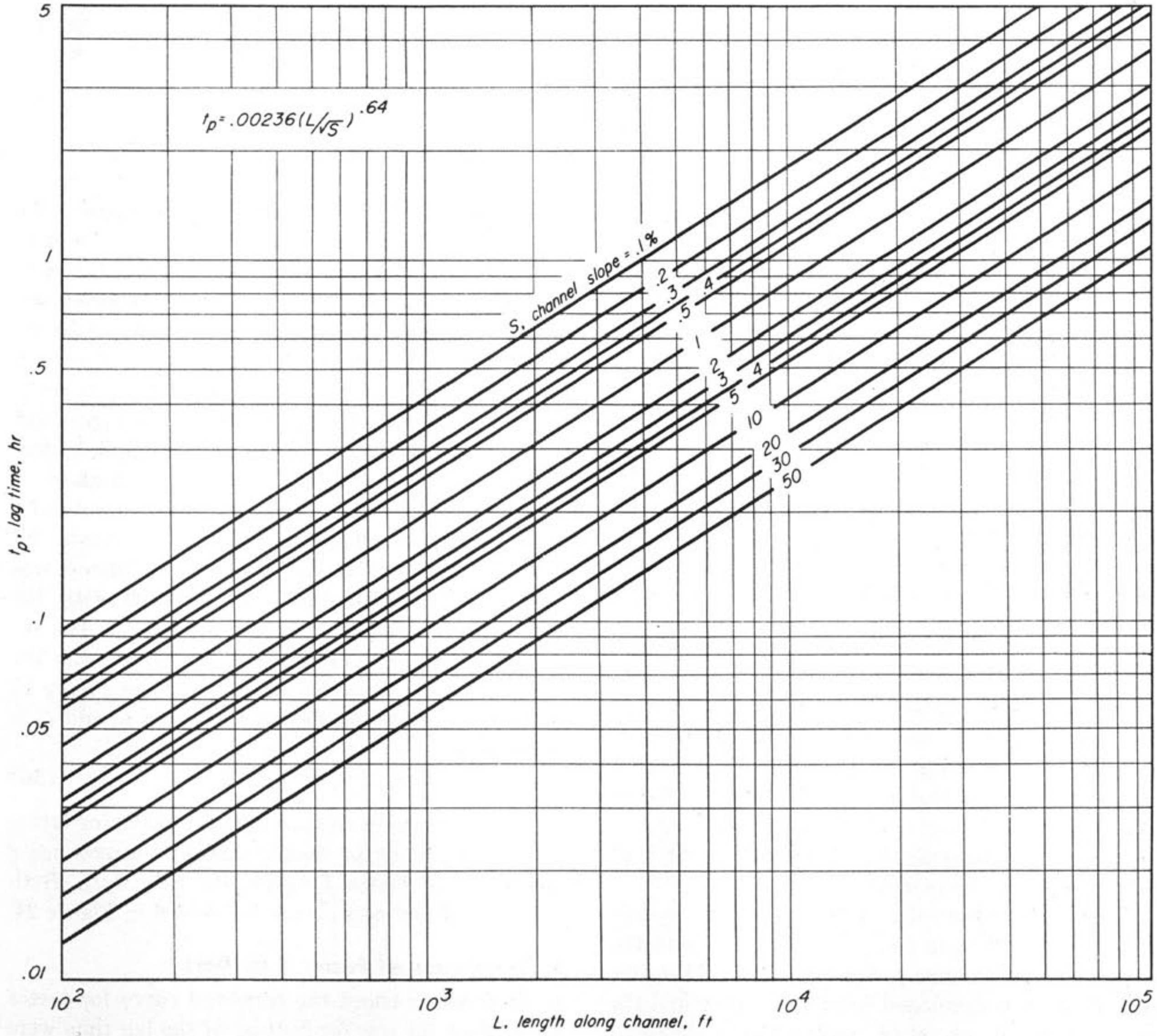


Figure 21. Determination of lag time by Equation 35

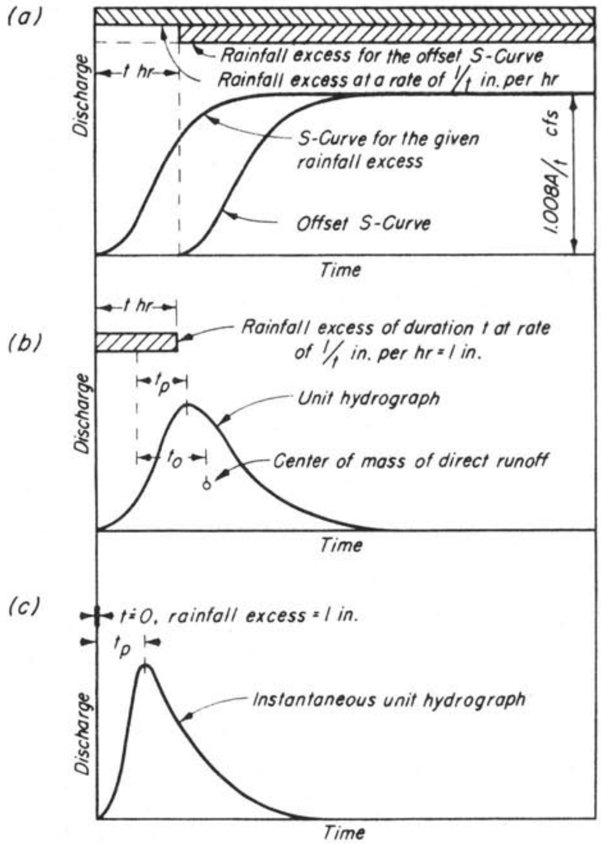


Figure 22. Schematic relation between the duration t, lag t_o, and lag time t_p

the scattering of the plotted data may be mainly due to the errors involved in the accuracy of the data as well as in the processing of the data, such as the uncertain flow separation in the case of multiple-peak hydrographs. In four cases: 3c, 8a, 12a, and 16 of Tables 22 and 23, there are strong evidences of inconsistency due to either the inaccuracy of the data or the way it was processed. Hence, these cases were excluded from the plotting. In all other cases, the errors involved are believed to be compensative. Therefore, despite the scattering, fitting with an average curve is justifiable.

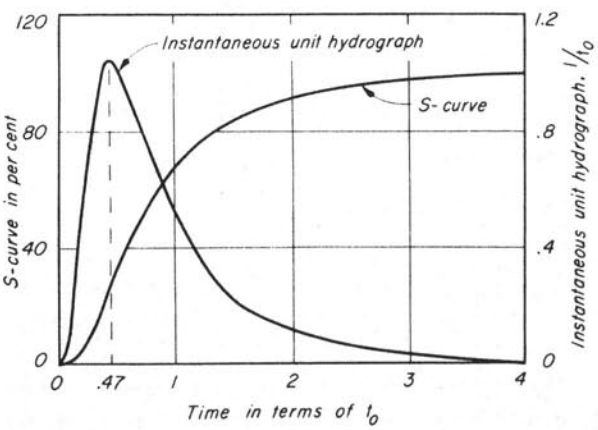


Figure 23. The S-curve and instantaneous unit hydrograph for drainage basins of an average size of 80 sq. mi. in Illinois based on U.S.G.S. data

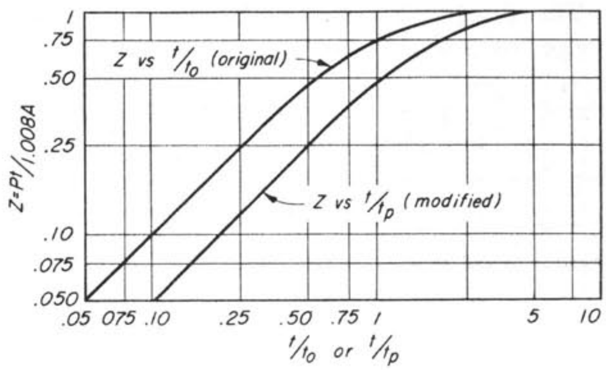


Figure 24. Modification of peak-reduction factor Z

F. DESIGN CRITERIA

From the nation-wide survey of design practice on waterway area determination in different state highway agencies, described in Section III, it was found that there have been no definite criteria concerning the climatic and physiographic conditions of drainage basins forming the basis for the design of waterway openings of drainage structures. In the proposed method it is suggested that adequate design criteria be considered and established as a first step towards the design of the structures. For this purpose, a definition of the design discharge to be used by the proposed method is given as follows:

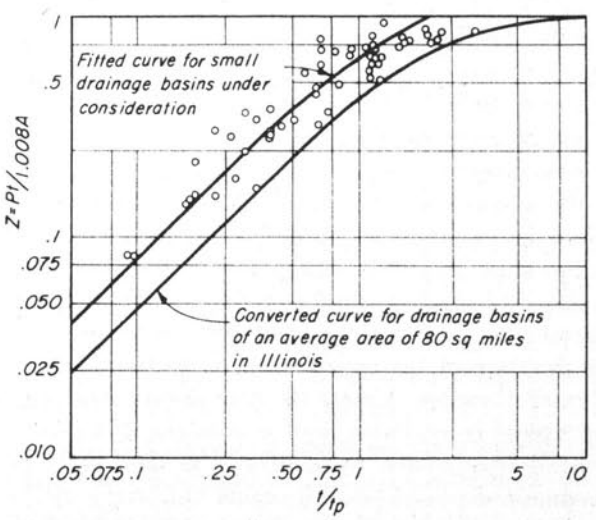


Figure 25. Relationship between Z and t/t_p

The design discharge is the maximum discharge that would occur under an average physiographic condition of the drainage basin due to rainfall of a given frequency and various durations and due to base flow.

In Illinois the design of culverts for highway drainage structures is handled in the highway districts. The design of bridges is handled in the Bridge Office at the Bureau of Design. The districts, with the exception of District 10, have been using the Talbot formula almost exclusively in the determination of the required waterway openings for the design of culverts. In District 10, with the District office at Chicago, there are two designing agencies; the regular District design office and the Expressway design office. Both offices reported making use of both the Talbot formula and the rational formula in culvert design. The regular District design office reported that when the rational formula is applied, 5-year and 10-year frequencies are used. The use of a 10-year frequency is limited to depressed sections where flooding is possible. In view of the lack of a universal criterion for the design frequency, values of runoff factor X for frequencies of 5, 10, 25, 50, and 100 years were prepared for use in the proposed method. The choice of a certain frequency in a design remains to be a matter of policy. From the rainfall analysis described in Section IV-C, the maximum recorded rainfall at Urbana, Illinois, was found to have a frequency of the order of 50 years. It may appear reasonable to set the 50-year frequency as a practical upper limit for the design of culverts.

The definition of the design discharge signifies it as the maximum discharge due to rainfall of various durations and due to base flow. Accordingly, different durations must be assigned in the computation by the proposed method until a maximum value of discharge is obtained. This value is then taken as the design discharge.

It is important to point out that the duration referred to in the determination of runoff factor X is the time interval within which the maximum depth of rainfall of a given frequency would occur. Therefore, this duration is not necessarily equal to the duration of the entire storm or to the duration of rainfall excess. Unless the true duration of rainfall excess is available and used in the analysis of rainfall frequencies, we can only assume that the maximum depth of rainfall occurs uniformly within the designated duration, and thus for this block of uniform rainfall the duration is equal to the duration of rainfall excess. Since thunderstorms of high intensities and short durations usually are the cause of the peak flows from small drainage basins, this assumption is reasonably justified. If the true duration of rainfall excess were considered, the rainfall amount for a given frequency would be reduced. Therefore, the assumption is on the conservative side.

G. DESIGN PROCEDURE BY THE PROPOSED METHOD

To facilitate the application of the proposed method, all information necessary for the computation is shown in a design chart, Figure 26. For a drainage basin of given size, runoff condition, and location in Illinois, the procedure of computing the design discharge is as follows:

(1) From the soil type map in the design chart (or Figure 13), determine the soil type.

(2) From the runoff number table (or Table 15), determine the runoff number for the soil type and the given cover and surface condition. If the drainage basin has composite soil types and cover and surface conditions, a weighted runoff number should be computed.

(3) Assign a certain rainfall duration t.

(4) From the curves for runoff factor X (or Figures 15 to 17), determine the value of X for the assigned duration, the given frequency and the runoff number.

(5) From the chart for climatic factor (or Figure 18), determine the value of Y.

(6) From the chart for lag time (or Figure 21), determine the value of t_p for the given length and slope of the stream.

(7) Compute the ratio t/t_p.

(8) From the curve for factor Z (or Figure 25) determine the value of Z for the computed t/t_p.

(9) Compute the discharge by Equation 30 or $Q = AXYZ$.

(10) Repeat the steps for other assigned durations.

(11) Plot the computed discharges against assigned durations. The largest computed discharge is the design discharge Q_{max}.

(12) If the stream is perennial, the base flow Q_b should be estimated and added to the discharge determined in step 10. The design discharge is then $Q_d = Q_{max} + Q_b$.

The above procedure is illustrated by the following example:

Example. Determine the design discharge for a highway culvert from the following data:

(a) Location:	FA Route 14, Section IX-1, Station 802 + 25 Williamson County, Illinois
(b) Land use:	Cover — 3% pasture or range; 33% row crop; and 64% small grain. Surface condition—Normal, straight row
(c) Basin Area:	30.0 acres
(d) Channel length:	1,550 ft.
(e) Channel slope:	2.6%
(f) Design frequency:	50 years
(g) Base flow:	None

Solution. From the given location the following are obtained from the design chart:

Soil type: D

Climatic factor: $Y = 1.10$

The runoff number for the given land use is found from the runoff number table in the design chart. The weighted runoff number is computed as follows:

Land use	*Percentage*		*Runoff number*		*Product*
Pasture	3.0%	×	84	=	2.5
Row crop	33.0%	×	90	=	29.7
Small grain	64.0%	×	88	=	56.4
			Sum	=	88.6
			Say, N	=	89

For $L = 1{,}550$ ft. and $S = 2.6\%$, the lag-time curves give $t_p = 0.19$ hr.

Now assign $t = 0.10$ hr. Since the design frequency is 50 years and the runoff number is 89, the curve for runoff factor X gives $X = 2.30$.

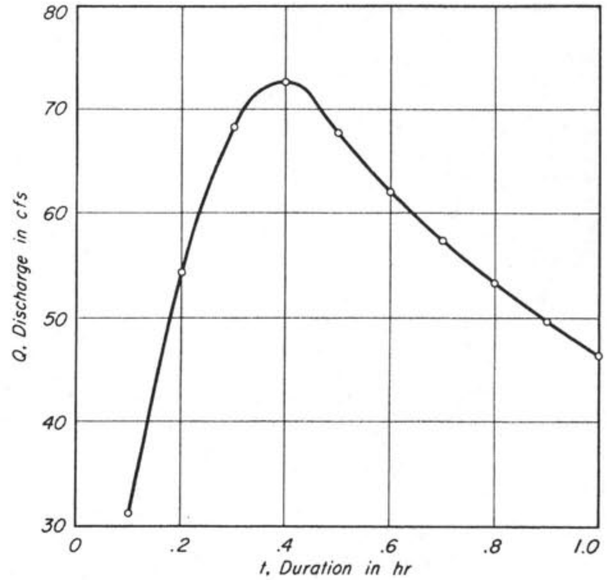
Since $t = 0.10$ hr. and $t_p = 0.19$ hr., $t/t_p = 0.53$. The curve for peak-reduction factor Z gives $Z = 0.41$. Thus, the discharge is $30.0 \times 2.30 \times 1.10 \times 0.41 = 31.1$ c.f.s.

Similarly, the discharges for other assigned values of t can be computed. The whole computation is shown in Table 24. A plot of the discharge against the assigned t is then constructed as shown in the accompanying figure. The maximum discharge $Q_{max} = 72.6$ c.f.s. In practical applications, the value of Q_{max} can be easily estimated from the column of Q values for the assigned durations, and hence a plot of Q vs. t is not necessary. Also, after some practice, fewer assigned values of t would be necessary for the determination of Q_{max}.

Table 24

Computation of a Design Discharge

Given Data	Computation				
Location: FA Route 14, Sec. IX-1	t hr.	t/t_p	X	Z	Q
Sta. 802 + 25					
Williamson County, Ill.					
Land use: 3% Pasture	0.1	0.53	2.30	0.41	31.1
33% Row Crop	0.2	1.05	2.50	0.66	54.4
64% Small Grain	0.3	1.58	2.38	0.87	68.3
Normal, Straight Row	0.4	2.11	2.20	1.00	72.6
Basin Area, A = 30.0 Acres	0.5	2.64	2.05	1.00	67.7
Channel Length, L = 1,550 ft.	0.6	3.16	1.88	1.00	62.0
Channel Slope, S = 2.6%	0.7	3.69	1.75	1.00	57.8
Design Frequency: 50 yrs.	0.8	4.21	1.62	1.00	53.5
Base Flow, Q_b = 0 c.f.s.	0.9	4.74	1.51	1.00	49.8
Chart Values	1.0	5.27	1.41	1.00	46.5
Soil Type: D					
Runoff Number, N = 89	$Q_d = 72.6 + 0 = 72.6$ c.f.s.				
Climatic Factor, Y = 1.10					
Lag Time, t_p = 0.19 hr.					
$Q_{max} = AXYZ$; $Q_d = Q_{max} + Q_b$					



Since the base flow is zero, the design discharge is $72.6 + 0 = 72.6$ c.f.s.

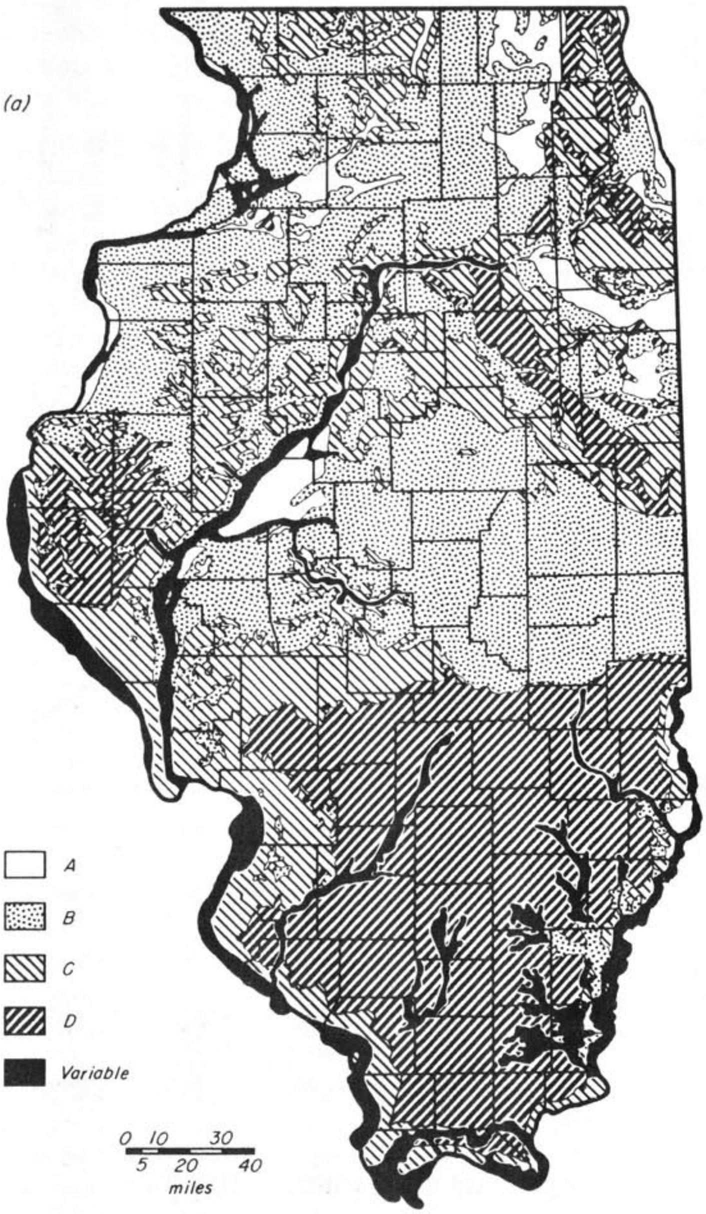
H. MERITS OF THE PROPOSED METHOD

There are several major merits that can be stated for the proposed method. They are as follows:

(1) The method has an analytical basis since it is developed from sound hydrologic principles. Therefore, the method is rational and the user, if he wishes, can follow through the procedure of the development and thus develop an insight to the underlying hydrologic principles.

(2) The method is based on the available data which are adequate to the local conditions under consideration.

(a)



N, runoff number

Cover	Surface condition	Soil type A	Soil type B	Soil type C	Soil type D
Fallow	Straight row	77	86	91	94
Row crops	Straight row	70	80	87	90
	Contoured	67	77	83	87
	Contoured & terraced	64	73	79	82
Small grains	Straight row	64	76	84	88
	Contoured	62	74	82	85
	Contoured & terraced	60	71	79	82
Legumes or rotation meadow	Straight row	62	75	83	87
	Contoured	60	72	81	84
	Contoured & terraced	57	70	78	82
Pasture or range	Poor	68	79	86	89
	Normal	49	69	79	84
	Good	39	61	74	80
	Contoured, poor	47	67	81	88
	Contoured, normal	25	59	75	83
	Contoured, good	6	35	70	79
Meadow, permanent	Normal	30	58	71	78
Woods, farm wood lot	Sparse	45	66	77	83
	Normal	36	60	73	79
	Dense	25	55	70	77
Farmsteads	Normal	59	74	82	86
Roads	Dirt	72	82	87	89
	Hard surface	74	84	90	92
Forest	Very sparse	56	75	86	91
	Sparse	46	68	78	84
	Normal	36	60	70	76
	Dense	26	52	62	69
	Very dense	15	44	54	61
Impervious surface		100	100	100	100

(b)

(e)

Step	Procedure
1	Read soil type from (a)
2	With cover, surface condition, and soil type read N, runoff number, from (b)
3	With N, frequency, and assigned t, read factor X from (c) or (d)
4	Read Y, climatic factor, from (e)
5	With drainage basin data read t_p from (f)
6	Compute the ratio t/t_p
7	With t/t_p read factor Z from (g)
8	Compute discharge by Eq $Q = AXYZ$ (Q = discharge in cfs) (A = drainage area in acres)

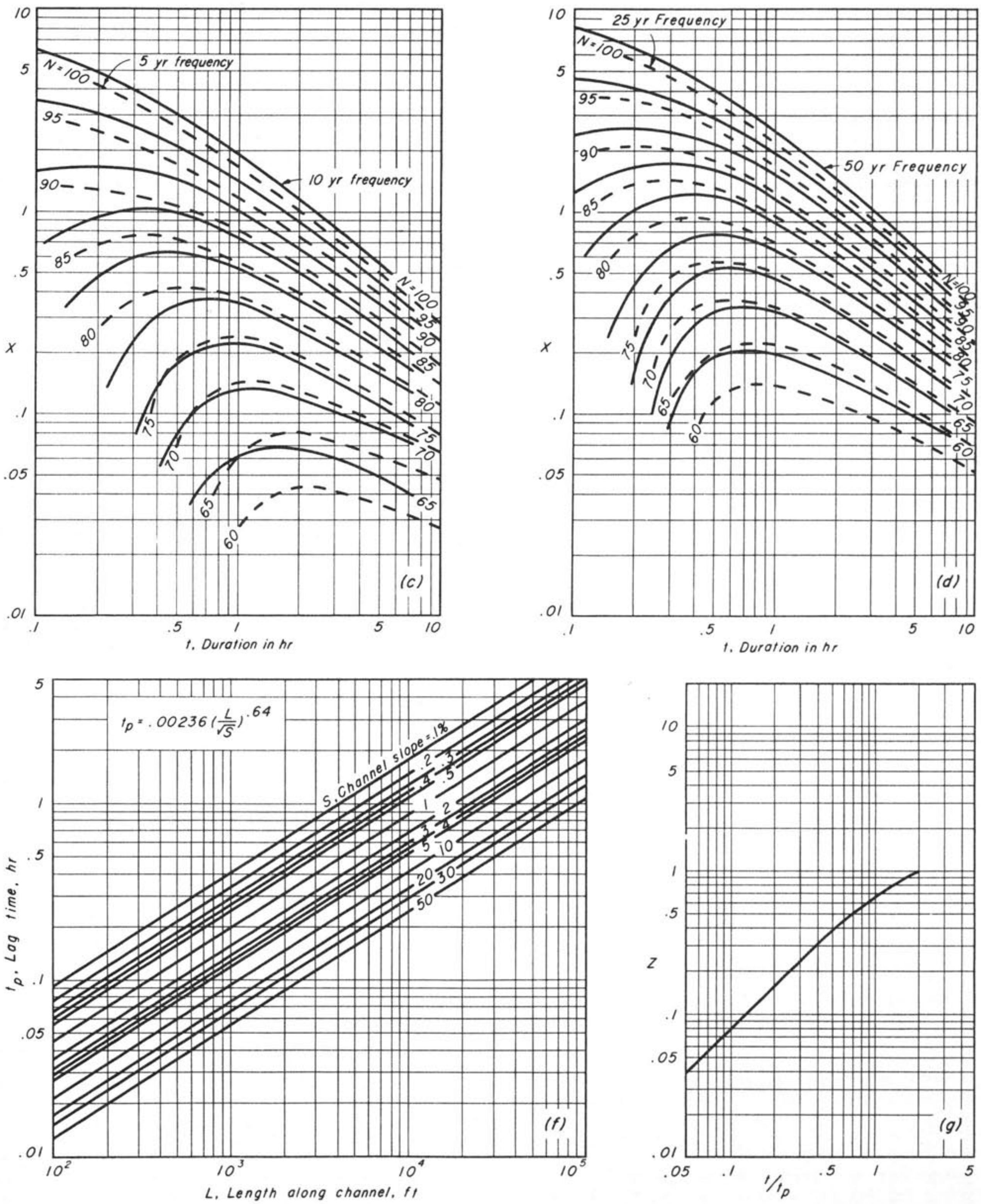


Figure 26. Chart for the determination of design discharge for small rural drainage basins in Illinois

(3) The criterion for the determination of a design discharge is clearly defined.

(4) Although the result obtained by the method may still require some professional supervision or review for final adoption in a design, the method will provide a unique solution or produce close answers by different individuals.

(5) The procedure of the method is simplified by a design chart so that practicing engineers can use it with great ease.

(6) The method can be readily improved by further verification with the accumulation of rainfall and runoff data and the field experience. Since the method is based on analytical principles, the improvement will not change the basic scheme, but will involve only the modification of the curves and charts which depend on the quantitative part of the data.

Like most methods in scientific and engineering works, the proposed method has disadvantages as well as advantages. The major disadvantage is the fact that the design discharge thus determined is based on a given frequency of rainfall instead of runoff. This shortcoming is entirely due to the lack of suitable data of runoff frequency for small drainage basins. The analysis of a large quantity of suitable runoff data may overcome this problem in the future.

VI. FUTURE STUDIES

The present investigation has led to the development of a practical procedure for the determination of peak discharges for the design of drainage structures on small rural drainage areas. Although the procedure illustrated in this report is prepared for design conditions in Illinois, the concept of the method is universally applicable to other states provided adequate data in these states are available for similar analysis and development.

Generally speaking, hydrologic data for small drainage basins are meager as compared to the data for large basins. In the present study, however, the data were available for the development of the proposed method, but they were not of sufficient quantity to justify a refinement of the method by correlation analyses of the data. When a substantial amount of data becomes available in the future, correlation analysis should be applied to the data in order to refine the functional relationships between various factors considered in the method. As data become abundant, a program of computation by digital computers should be developed for the selection and analysis of the data to be used in the formulation of the design charts. Simplifying the tedious processing and derivation of instantaneous unit hydrographs will be of great value. In connection with this future study, a comprehensive investigation on various theories that have been proposed for the concept of instantaneous unit hydrographs is necessary. From such an investigation it will be possible to select the best theory suitable to the present purpose. For a physical understanding and interpretation of some assumptions underlying the unit hydrograph principle, a hydraulic model of drainage basins designed for different geometric elements and physiographic features should prove most useful. Such a model should be capable of testing, for instance, the principle of superposition that basically constitutes the conventional method of unit hydrographs.

When suitable rainfall data become available, a further study on rainfall variables should be in order for an improvement of the proposed method. The rainfall variables include such items as the frequency range of variation of the effect of antecedent moisture condition upon runoff, the relationship between the durations of gross rainfall and rainfall excess (Section V-F), and the areal and time distributions of rainfall. Similarly, sufficient quantitative information on physiographic conditions of drainage basins will enable an investigator to perform a frequency analysis of the runoff number used in the proposed method. At present, the runoff number based on an average condition of runoff is assumed.

Because of the shortage of runoff data pertaining to small drainage basins, the design frequency used in the proposed method is based on the data of rainfall intensities (Section V-F). When more runoff data for small drainage basins are accumulated in the future, there will be a need to perform a frequency analysis of the runoff data and to adopt a design criterion on the basis of runoff frequencies. The family of curves for the runoff factor will have to be modified accordingly, while the general concept of the proposed method may remain the same.

VII. APPENDICES

APPENDIX I. A COMPILATION OF FORMULAS FOR WATERWAY AREA DETERMINATION

A. GENERAL

In engineering literature there are numerous empirical and semi-empirical formulas which have been developed to determine the waterway areas of culverts and bridges for various hydrologic and geographic conditions. A number of these formulas give an answer to the required waterway area directly; while others provide the maximum or design values of rainfall and runoff by which the desired waterway area may be determined.

In this appendix 102 such formulas are presented. Informative remarks and references for each formula are given, furnishing further information concerning the original development of the formula and its background. However, this compilation of formulas is by no means complete, as the listed formulas are only those which have been popular primarily in English-speaking countries.

Formulas are attractive to practicing engineers because they are simple in form and easy to use. However, the user may often ignore the conditions for which the formula was developed, thus producing erroneous answers. It is apparent that formulas are so many in number and vary so much in results that it is impossible to reconcile the widely divergent answers which would be obtained by applying a number of them to any particular problem. Each formula has its particular purpose, but none is suitable for general application. For example, the popular Talbot formula which is based on data obtained from Middle Western United States is often applied to other parts of the country having entirely different hydrologic conditions. Such indiscriminate uses of the formulas should be avoided.

Despite the fact that most of these formulas were not intended for general application, attempts are usually made to broaden the application by means of radical changes in coefficients. It is therefore necessary in each case to select a proper coefficient for the problem under consideration. Such a step requires practical experience and sound judgment. A good selection depends on a thorough knowledge of the conditions for which the formula was derived and also on a correct understanding of the limitations to which the formula should be subjected. Unfortunately, the limited applicability of such formulas is not always well understood, and the personal factors involved in selecting proper coefficients vary greatly. Consequently, the attractiveness of the simplicity of such formulas often leads to much abuse and results in errors in design.

Furthermore, modern hydrology discusses numerous factors which govern the rainfall on any drainage basin and the runoff from it. Some of the empirical formulas achieve their simplicity by neglecting a number of factors which may be vital to the establishment of a correct answer. Other formulas are complicated and contain certain factors which may be insignificant and difficult to evaluate. Therefore, a study of these formulas and the factors involved in them is desirable in evaluating appropriate factors which must be considered in the development of a rational procedure for waterway area determination.

B. CLASSIFICATION OF FORMULAS

The 102 formulas collected in this appendix are classified into five groups as follows:

Group 1. **Waterway Area Formulas.** In these formulas the waterway area is expressed directly in terms of the drainage area. A coefficient is generally used to take care of the variation in conditions which affect the waterway area. The general form of the formulas is

$$a = C\,F(A)$$

in which a is the waterway area, C is the coefficient, and $F(A)$ is a function of the drainage area A. The most common form of $F(A)$ is

$$F(A) = A^n$$

in which n is an exponent ranging from 0.5 to 1.0. The value of C varies from 0.2 to 4.0.

A collection of 12 formulas is given in this group.

Group 2. **Simple Flood Formulas.** In these formulas the flood discharge is expressed directly in terms of the drainage area. The waterway area is then obtained by dividing the computed discharge by an assigned desirable or safe value of the velocity of flow V through the waterway opening, giving

$$a = \frac{Q}{V}$$

The general form of the formulas is

$$Q = C\ F(A)$$

in which Q is the discharge, C is a coefficient depending on conditions which affect the discharge, and $F(A)$ is a function of the drainage area. The most common form of $F(A)$ is again $F(A) = A^n$. The value of C and n varies widely.

A collection of 30 formulas is given in this group.

Group 3. **Rainfall Intensity Formulas.** These formulas are used to compute the rainfall intensity. The discharge is computed by the rational formula or

$$Q = C\ I\ A$$

in which Q = discharge in c.f.s.

C = percentage of runoff depending on characteristics of the drainage basin

I = rainfall intensity in inches per hour

A = drainage area in acres

The general form for rainfall intensity formulas is

$$I = \frac{K\ F^{n_1}}{(t+b)^n}$$

in which

K, b, and n, n_1 = respectively coefficients and exponents depending on conditions which affect the rainfall intensity.

F = frequency factor indicating the frequency of occurrence of the rainfall

t = duration of the storm in minutes which is equal to the time of concentration

The forms of rainfall intensity formulas are generally of the following three types:

$$\text{Type I:}\quad I = \frac{K}{t+b}$$

$$\text{Type II:}\quad I = \frac{K}{t^n}$$

$$\text{Type III:}\quad I = \frac{K}{\sqrt{t}+b}$$

A collection of 24 formulas is given in this group.

Group 4. **Frequency Formulas.** These formulas express the discharge in terms of basin characteristic parameters and the frequency of occurrence. The formulas are generally developed by means of a frequency analysis of the flood data. The general form of the formulas is

$$Q = a + b\ F(T)$$

in which a and b are constants, and $F(T)$ is a function of the recurrence interval T in years. The recurrence interval is defined as the average interval of time within which the magnitude of the flood will be equalled or exceeded once on the average.

The waterway area for a given design frequency is obtained by dividing the computed discharge of the corresponding frequency by an assigned desirable or safe velocity of flow through the drainage structure.

A collection of 5 formulas is given in this group.

Group 5. **Elaborate Discharge Formulas.** These formulas express the discharge in terms of a number of factors indicating climatic variations and characteristics of drainage basin. These formulas are generally developed by the rational formula or by the method of multiple correlation. The general form of these formulas is

$$A = F\ (D, W, L, S, F\ .\ .\ .\ )$$

in which D, W, L, S, F . . . are factors under consideration.

A collection of 31 formulas is given in this group.

C. GROUPING OF FORMULAS

The following formulas are compiled under various groups; within each group the formulas are listed alphabetically.

Group 1. **Waterway Area Formulas**

1-1. The C. B. and Q. formula
1-2. The Fanning formula
1-3. The Hawksley formula
1-4. The Modified McMath formula
1-5. The Myers formula
1-6. The Peck formula
1-7. The Purdon-Dun formula
1-8. The Ramser formula
1-9. The Talbot formula
1-10. The Tidewater (Virginia) Railway formula
1-11. The Wentworth formula
1-12. The Yule formula

Group 2. **Simple Flood Formulas**

2-1. The Beale formula
2-2. The Bahadur formula
2-3. The C. B. and Q. Railroad formula
2-4. The Cooley formula
2-5. The Dickens formula
2-6. The El Paso and S. W. Railway formula
2-7. The Elliot formulas
2-8. The Fanning formula
2-9. The Frizell formula
2-10. The Ganguillet formula
2-11. The Gray formula
2-12. The Horn formula
2-13. The Inglis formula
2-14. The Italian formulas
2-15. The Kresnik formula
2-16. The Kuichling formulas
2-17. The Lauterberg formula
2-18. The McCrory and Others formula
2-19. The Metcalf and Eddy formula
2-20. The Meyer formula
2-21. The Morgan Engineering Co. formulas
2-22. The Modified Myers formula
2-23. The Murphy and Others formula
2-24. The Nagler formula
2-25. The New Kuichling formula
2-26. The O'Connell formula
2-27. The Ryves formula
2-28. The Schnackenberg formula
2-29. The U.S.G.S. formulas
2-30. The Williams formula

Group 3. **Rainfall Intensity Formulas**

3-1. The Allen-Babbitt formula
3-2. The Bleich formulas
3-3. The Brackenbury formula
3-4. The de Bruyn-Kops formula
3-5. The Bureau of Survey formula
3-6. The Clarks formula
3-7. The Dorr formula
3-8. The Gregory formula
3-9. The Hendrick formula
3-10. The Hill formula
3-11. The Horner formula
3-12. The Institution of Civil Engineers' formula
3-13. The Kuichling formula
3-14. The L. J. Le Conte formula
3-15. The Metcalf and Eddy formula
3-16. The Meyer formula
3-17. The Nipher formula
3-18. The Sherman formula
3-19. The Schafmayer formula
3-20. The Steel formula
3-21. The Talbot formula
3-22. The Webster formula
3-23. The Whiney formula
3-24. The Williams formula

Group 4. **Frequency Formulas**

4-1. The Creager formula
4-2. The Fuller formula
4-3. The Horton formula-A
4-4. The Horton formula-B
4-5. The Lane formula

Group 5. **Elaborate Discharge Formulas**

5-1. The Adams formula
5-2. The Desson formula
5-3. The Boston Society of Civil Engineers' formula
5-4. The Burge formula (See: The Dredge formula)
5-5. The Bürkli-Ziegler formula
5-6. The Chamier formula
5-7. The Craig formula
5-8. The Cramer formula-A
5-9. The Cramer formula-B
5-10. The Dredge or Burge formula
5-11. The Gregory formulas
5-12. The Gregory and Arnold formula
5-13. The Gregory and Hering formula

D. NOTATION

The following is a list of notation used in the formulas which may differ from those used in the original presentation of the formulas. When the units are different from those given in the list, they will be specified under each individual case.

a = Waterway area in square feet.
b, b_1, b_2, b_3 = Coefficient or parameter.
c, c_1, c_2, c_3 = Coefficient or parameter, depending on basin characteristics, hydrologic condition, or other factors.
A, A_1, A_2 = Drainage area in acres.
A_k = Drainage area in square kilometers.
A'_k = Forested part of the drainage area in square kilometers.
D, D_1 = Drainage area in square miles.
d = Diameter of a circular sewer in inches.
F = Frequency factor indicating the occurrence of a storm or flood.
G = Geographic factor.
H = Average water content of snow in millimeters of depth.
I = Rainfall intensity in inches per hour.
I_d = Design rainfall intensity in centimeters per minute.
i = Permeability of soils in centimeters per minute.
K = Coefficient or parameter in rainfall intensity formula.
k = Climatic factor.
L = A length on the drainage basin in miles, feet, or kilometers measured along the main stream or nearly so; it may be the greatest length of the drainage basin, the straight line distance from point of discharge to center of gravity of basin, the length of the outlet channel from the edge of the drainage area to the outlet, the length of path of raindrops from farthest point of drainage area to point where discharge is considered, the length of the stream from its source to the point of observation, the length of sectors of drainage area, or the average distance which water travels to the outlet.
m = A dividing factor.
N = Length of sewer in feet per foot of fall.
n, n_1, n_2, n_3 = Exponents.
P = Proportion of impervious surface.
Q = Discharge in c.f.s.
Q_m = Maximum discharge in cubic meters per second.
Q_{max} = Maximum discharge in c.f.s.
Q_{av} = Average discharge in c.f.s.
Q_r = Recorded discharge in c.f.s.
q, q_1 = Discharge in c.f.s. per square mile.
q_{max} = Maximum discharge in c.f.s. per square mile.
R = Rainfall depth in inches or meters; it may be the mean, normal or maximum value in an hour, a day, a year or in record; or direct runoff in inches.
R_m = Maximum one-day rainfall in inches.
R_r = Recorded one-day rainfall in inches.
r = Parameter for forestation.
S, S_1, S_2 = Slope or fall of drainage area or stream expressed in ratio or in feet per thousand feet or in feet per mile.
s, s_1, s_2 = Median altitude of the drainage area above a datum or the outlet, in feet.
T = Recurrence interval in years.
T_e = Exceedance interval in years.
t = Duration of the rainfall in minutes, or the time of concentration in minutes when the rational formula is used, or time for absolute maximum.

t_e = Shortest time for infiltration during a 24-hr. intensive rainfall.

V = The average velocity of flow in feet per second or in meters per second when specifically stated.

W = Average width of the drainage basin in miles.

θ = Angle of sectors of drainage area in degrees.

E. GROUP 1: WATERWAY AREA FORMULAS

1-1. The C. B. and Q. formula

$$a = \frac{0.46875\ A}{3 + 0.079\sqrt{A}}$$

or

$$a = \frac{300\ D}{3 + 2\sqrt{D}}$$

Remark: This formula was derived from the Chicago, Burlington and Quincy Railroad formula (2-3) for flood discharge, assuming a velocity of flow equal to 10 ft. per sec.

Reference: Thaddeus Merriman and T. H. Wiggin, *American Civil Engineers' Handbook* (5th ed.; New York: John Wiley and Sons, 1930), p. 2010.

1-2. The Fanning formula

(1) Assuming a velocity of 4 ft. per sec.,

$$a = 0.23\ A^{5/6}$$

or

$$a = 50\ D^{5/6}$$

(2) Assuming a velocity of 10 ft. per sec.,

$$a = 0.09\ A^{5/6}$$

or

$$a = 20\ D^{5/6}$$

Remark: The formulas were derived from the Fanning formula (2-8) for flood discharge.

References: Thaddeus Merriman and T. H. Wiggin, *American Civil Engineers' Handbook* (5th ed.; New York: John Wiley and Sons, 1930), p. 2009.

George H. Bremner, "Areas of Waterways for Railroad Culverts and Bridges," *Journal, Western Society of Engineers,* Vol. 11, No. 2 (April, 1906), p. 139.

1-3. The Hawksley formula

$$a = \frac{\pi d^2}{4}$$

in which $\log d = \dfrac{3 \log A + \log N + 6.8}{10}$

where d = diameter of circular sewer in inches

N = length of sewer in feet per foot of fall

Remarks: (1) This formula was first published in a *Report of Commission of Metropolitan Drainage, 1857,* London.

(2) The formula is believed to have been established between 1853 and 1856. It was developed to express analytically the relationship between the diameter and slope of a circular outlet sewer and the size of its drainage area. This relationship was found from numerous observations of storm discharges in sewers and then represented by a tabular form in 1852 by John Roe, Surveyor of the Holborn and Finsbury sewers, London.

(3) A maximum rainfall intensity of 1 in. per hr. was probably assumed.

Reference: Emil Kuichling, "Storm Water in Town Sewerage," *Transactions, Association of Civil Engineers,* Cornell University, Vol. 1 (1893), p. 46.

1-4. The Modified McMath formula

$$a = 0.5908A^{0.8}$$

Remark: This was derived from the McMath formula (5-22) for conditions at St. Louis, Missouri, with $C = 0.75$, $I = 2.75$, $S = 15$, and a velocity of flow of 6 ft. per sec.

Reference: Thaddeus Merriman and T. H. Wiggin, *American Civil Engineers' Handbook* (5th ed.; New York: John Wiley and Sons, 1930), p. 2010.

1-5. The Myers formula

$$a = C\ A^{0.5}$$

in which C = 1 as a minimum for flat country
= 1.6 for hilly compact ground
= 4.0 as a maximum for mountainous and rocky country

Remarks: (1) The formula was applied satisfactorily to the water courses mostly on the line of the Richmond, Fredericksburg and Potomac Railroad (State of Virginia).

(2) It was generally found that the formula gives values too large for drainage areas less than about 1 sq. mi. in flat country or 0.5 sq. mi. in mountainous regions.

(3) The formula was first published in a paper read by Cleemann before the Engineers' Club of Philadelphia in 1879.

Reference: T. M. Cleemann, "Railroad Engineers' Practice, Discussion of Formulas," *Proceedings, Engineers Club of Philadelphia,* Vol. I (April 5, 1879), p. 146.

1-6. The Peck formula

$$a = \frac{A}{C}$$

in which $C = 4$ for very mountainous country, where slopes of hills and mountains are steep and abrupt
$= 6$ for ordinary flat rolling country, such as in most agricultural regions

Remarks: (1) This formula was introduced by R. M. Peck for design of structures on the Missouri Pacific Railway and St. Louis, Iron Mountain and Southern Railway.

(2) This formula appears to have no great merit.

Reference: W. G. Berg, "How to Determine Size and Capacity of Openings for Waterways," *Committee Reports for 1896-97*, presented at the 7th Annual Convention of the Association in Denver, Colorado, October, 1897, and published in *Proceedings, Association of Railway Superintendents, Bridges and Buildings*, Vol. 7 (1897), pp. 86-100.

1-7. The Purdon-Dun formula

$$a = \sqrt{\frac{A}{640}}\left(240 - 12\sqrt[6]{\frac{A}{640}}\right)$$

Remarks: (1) This formula is based on James Dun's waterway area data and the data collected from the land adjacent to the Santa Fe Railway.

(2) Purdon found that the formula fitted the Dun data for drainage areas of more than 16 sq. mi. For small areas an arbitrary addition was made to allow for drift, etc.

Reference: C. D. Purdon, "Discussion on Flood Flow Characteristics," *Transactions, American Society of Civil Engineers*, Vol. 89 (1926), p. 1090.

1-8. The Ramser formula

$$a = C\left(130 + \frac{77{,}000}{A + 600}\right)$$

in which $C = 1.4$ for cultivated hilly land
$= 1.0$ for cultivated rolling land
$= 0.8$ for pasture in hilly land
$= 0.6$ for pasture in rolling land
$= 0.4$ for woods in hilly land
$= 0.3$ for woods in rolling land

Remarks: (1) This formula was recommended for areas not larger than 800 acres.

(2) For fan- or square-shaped drainage basins the results should be multiplied by 1.25.

(3) The formula was designed primarily for drop-inlet culverts. The values computed by the formula are used for vertical drops through culverts up to 5 feet. The values should be multiplied by 0.71 for drops through culverts up to 10 feet and by 0.58 for drops through culverts up to 15 feet.

Reference: Q. C. Ayres, *Recommendations for the Control and Reclamation of Gullies*, Bulletin 121 (Ames, Iowa: Iowa State College Engineering Experiment Station, 1935).

1-9. The Talbot formula

$$a = C\,A^{0.75}$$

in which $C = 1.00$ for mountainous region
$= 0.66$ for very hilly country
$= 0.50$ for hilly country
$= 0.33$ for rolling country
$= 0.25$ for gently rolling country
$= 0.20$ for flat land

Remark: The values listed above are commonly employed. For originally recommended values and discussion of the formula see Section II-C.

Reference: A. N. Talbot, "The Determination of Water-Way for Bridges and Culverts," *Selected Papers of the Civil Engineers' Club, Technograph No. 2*, University of Illinois (1887-88), pp. 14-22.

1-10. The Tidewater (Virginia) Railway formula

$$a = 0.62\,A^{0.7}$$

Remark: The computed area may be increased 30% for streams having a flat fall, and 20% for double openings.

References: *Tables and Data for Estimates and Comparisons* (blue print), Tidewater Railway, Roanoke, Virginia. January, 1905.

"Report of Sub-Committee of Roadway Committee No. 1, Bulletin 131," *Proceedings, American Railway Engineering and Maintenance of Way Association*, Vol. 12, Pt. 3 (March, 1911), p. 499.

1-11. The Wentworth formula

$$a = A^{3/4}$$

Remarks: (1) This formula was derived for design on the Norfolk and Western Railway. It was found to fit the conditions along that line satisfactorily. It has also been used in the southeastern states.

(2) For very flat ground with less rainfall, the results obtained by this formula are generally too large. Wentworth observed that 60% of the computed area may be taken as the lower limit.

Reference: Letter to the Editor, *Railroad Gazette*, 1903, p. 57.

1-12. The Yule formula (Also known as the Indiana State Highway Department formula)

$$a = C\,A^{2/3}$$

in which C = 0.3 to 0.7 for flat country
= 0.7 to 1.3 for rolling country
= 1.3 to 2.2 for hilly country

Remarks: (1) This formula was given only for conditions of topography and rainfall similar to those in Indiana. It was not proposed for arid regions, for areas having large average annual rainfall, or for mountainous country.

(2) The formula was developed from a study by the Indiana State Highway Department since 1927.

Reference: Robert B. Yule, "Bridge Waterway Area Formula Developed for Indiana," *Civil Engineering*, Vol. 20, No. 10 (October, 1950), pp. 26 and 73.

F. GROUP 2: SIMPLE FLOOD FORMULAS

2-1. The Beale formula

$$Q = C\,A^{0.75}$$

in which C = 1,600 for unforested area
= 1,400 to 1,000 for forested area in the central provinces of India

Remark: This is an adaptation of the Dickens formula (2-3) to suit the conditions of the Western Ghats in the Bombay Presidency from the observed discharges on the Nira Canal.

Reference: G. R. Hearn, "The Effect of Shape of Catchment on Flood Discharge," *Proceedings, The Institution of Civil Engineers*, Vol. 217 (1923-24), p. 289.

2-2. The Bahadur formula

$$Q = C\,D^{[0.92-(1/14)\log D]}$$

in which C = 1,600 to 2,000

Remark: This formula was proposed by Nawab Jung Bahadur for drainage basins in Hyderabad-Deccan area in India.

Reference: V. B. Priyani, *The Fundamental Principles of Irrigation Engineering* (Anand, India: Charotar Book Stall, 1957), p. 48.

2-3. The C. B. and Q. Railroad formula

$$Q = \frac{59.2\,A}{37.9 + \sqrt{A}}$$

or

$$Q = \frac{3{,}000\,D}{3 + 2\sqrt{D}}$$

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

Reference: G. H. Bremner, "Areas of Waterways for Railroad Culverts and Bridges," *Journal, Western Society of Engineers*, Vol. 11, No. 2 (April, 1906), p. 139.

2-4. The Cooley formula

$$Q = 2.43\,A^{2/3} = 180\,D^{2/3}$$

and

$$Q = 2.70\,A^{2/3} = 200\,D^{2/3}$$

Reference: G. H. Bremner, "Areas of Waterways for Railroad Culverts and Bridges," *Journal, Western Society of Engineers*, Vol. 11, No. 2 (April, 1906), pp. 139 and 170.

2-5. The Dickens formula

$$Q = CA^{0.75}$$

or

$$Q = C_1 D^{0.75}$$

in which C = 1.56 or C_1 = 200 for Madras Presidency, India
= 3.91 or C_1 = 500 for Central Provinces, India
C = 6.45 or C_1 = 825 for Bengal and Bihar, India
C = 9.37 or C_1 = 1,200 for Upper Kaveri, India
C = 17.2 or C_1 = 2,200 for Gadamatti, India
C = 6.6 or C_1 = 850 for average conditions

Remark: Other values of C and C_1 suggested by S. K. Gurtu in *Proceedings, The Institution of Civil Engineers*, Vol. 217 (1923-24), p. 386, are as follows:

C = 11.0-15.6 and C_1 = 1,400-2,000 for bare drainage basins, covered with precipitous hills (Class I)
C = 7.8-9.4 and C_1 = 1,000-1,200 for basins with hills on the skirts, with undulating country below up to the outfall (Class II)
C = 6.3-7.8 and C_1 = 800-1,000 for undulating country, with hard indurated clay soil (Class III)
C = 1.6-4.7 and C_1 = 200-600 for flat, sandy, absorbent, or cultivated plains (Class IV)

References: C. H. Dickens, "Flood Discharge of Rivers," *Professional Papers on Indian Engineer-*

ing, Thomason College Press, Roorkee, India, Vol. II (1865), pp. 133-136.

G. R. Hearn, "The Effect of Shape of Catchment on Flood-Discharge," *Proceedings, The Institution of Civil Engineers*, Vol. 217 (1924), p. 288.

S. K. Gurtu, "Correspondence on Flood-Discharge," *Proceedings, The Institution of Civil Engineers*, Vol. 217 (1924), p. 386.

2-6. The El Paso and S. W. Railway formula

$$Q = 60\,A^{0.5}$$

Remark: This is practically the same formula developed by Joseph P. Frizell, which he obtained from records of flow over the Holyoke Dam, Massachusetts, for a period of 50 years, and published in his book on hydraulics.

Reference: "Report of Sub-Committee of Roadway Committee No. 1, Bulletin 131," *Proceedings, American Railway Engineering and Maintenance of Way Association*, Vol. 12, Pt. 3 (March, 1911), p. 499.

2-7. The Elliot formulas

(1) For swamps and wet lands in Northeastern Arkansas

$$Q = \left(\frac{24}{\sqrt{D}} + 6\right) D$$

or

$$Q = \left(\frac{0.948}{\sqrt{A}} + 0.00937\right) A$$

(2) For swamps and other wet lands of the Upper Mississippi Valley

$$Q = \left(\frac{20}{\sqrt{D}} + 3.63\right) D$$

or

$$Q = \left(\frac{0.792}{\sqrt{A}} + 0.00568\right) A$$

(3) For satisfactory drainage areas in North Central Illinois

$$Q = \left(\frac{673}{19.2 + \sqrt{D}} - 11.3\right) D$$

or

$$Q = \left(\frac{26.6}{486 + \sqrt{A}} - 0.0177\right) A$$

Remarks: (1) The first formula was used to compute the discharge from the low flat alluvial lands in the preliminary drainage investigation in Northeastern Arkansas. The results may be increased 50% for the more rolling and less sandy land in the east part of Mississippi County, 100% for the clay soils east of Crowleys Ridge, and 200% for the slopes of Crowleys Ridge.

(2) The second formula specifies that the soils are absorptive and easily drained.

(3) The third formula was given to areas of 200 sq. mi. and less.

(4) These formulas were used for rough approximations. The results should be checked for local conditions.

References: C. G. Elliot, *Engineering for Land Drainages* (New York: John Wiley and Sons, Inc., 1919), pp. 198-199.

A. E. Morgan, "A Preliminary Report on the St. Francis Valley Drainage Project in Northeastern Arkansas," *U. S. Department of Agriculture Circular 86*, Office of Experimental Stations, 1919, p. 20 (for the first formula).

2-8. The Fanning formula

$$Q = 0.92\,A^{5/6}$$

or

$$Q = 200\,D^{5/6}$$

Remark: The formula was derived from a relatively small number of observations on American rivers.

Reference: J. T. Fanning, *Practical Treatise on Water Supply Engineering* (New York: D. Van Nostrand Co., Inc., 1878), p. 66.

2-9. The Frizell formula

$$Q = 61.3\,A^{0.5}$$

or

$$Q = 1{,}550\,D^{0.5}$$

Remark: This formula is converted from the original form $q = 17.35\sqrt{8006/D}$ for maximum flood rate in c.f.s. per sq. mi. on the Connecticut River. The general form is $q = q_1\sqrt{D_1/D}$ where q_1 is the observed maximum flood rate in c.f.s. per sq. mi. and D_1 is the corresponding drainage area in sq. mi.

Reference: J. P. Frizell, *Water Power* (3rd ed.; New York: John Wiley and Sons, Inc., 1905), p. 41.

2-10. The Ganguillet-Kutter formula

$$Q = \frac{56.2\,A}{78.7 + \sqrt{A}}$$

or

$$Q = \frac{1{,}421\,D}{3.11 + \sqrt{D}}$$

Remark: This formula was developed for Swiss streams in 1869.

Reference: C. S. Jarvis, "Hydrology," Section 2 of

Handbook of Applied Hydraulics, edited by C. V. Davis (1st ed.; New York: McGraw-Hill Book Co., Inc., 1942), p. 109.

2-11. The Gray formula

$$Q = 0.049\,A^{1.75}$$

or

$$Q = 3{,}770\,D^{1.75}$$

Remark: The original form is $Q = 5.89\,D^{5/6}$, where Q is the discharge in c.f.s. per acre and D is the drainage area in sq. mi.

Reference: "Report of Sub-Committee of Roadway Committee No. 1, Bulletin 131," *Proceedings, American Railway Engineering and Maintenance of Way Association*, Vol. 12, Pt. 3 (March, 1911), p. 499.

2-12. The Horn formula

$$Q = A(10.45 - \ln A)$$

or

$$Q = 640\,D(4 - \ln D)$$

Remark: This formula was proposed for maximum discharge in c.f.s. from drainage areas less than 20 sq. mi.

Reference: G. R. Hearn, "The Effect of Shape of Catchment on Flood-Discharge," *Proceedings, the Institution of Civil Engineers*, Vol. 217 (1924), pp. 279-280.

2-13. The Inglis formula

$$Q = \frac{7{,}000\,D}{\sqrt{D + 4}}$$

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

Reference: V. B. Priyani, *The Fundamental Principles of Irrigation Engineering* (Anand, India: Charotar Book Stall, 1957), p. 48.

2-14. The Italian formulas

$$(1)\quad Q = \frac{71.8\,A}{7.87 + \sqrt{A}}$$

or

$$Q = \frac{1{,}819\,D}{0.311 + \sqrt{D}}$$

$$(2)\quad Q = \frac{103.0\,A}{7.87 + \sqrt{A}}$$

or

$$Q = \frac{2{,}600\,D}{0.311 + \sqrt{D}}$$

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

Reference: Emil Kuichling, *Annual Report on the Barge Canal*, New York, 1901.

2-15. The Kresnik formula

$$Q = C\,\frac{71\,A}{7.84 + \sqrt{A}}$$

in which C = a maximum of 3.07 for the Pliessnitz River near Bertschodorf, Germany
= up to 6.0 for rivers outside of Europe

Remark: Tables and charts have been prepared for the values of C for a number of rivers. They may be found in *Hydraulic Structures* by A. Schoklitsch [English translation: (New York: The American Society of Mechanical Engineers, 1937), Vol. I, pp. 58-60].

Reference: "Allgemeine Berechnung der Wasserprofile und Gefällsverhältnisse für Flüsse und Kanäle," *Technische Vorträge und Abhandlungen*, No. 8, Vienna, 1886.

2-16. The Kuichling formulas

$$Q = \left(\frac{44{,}000}{A + 108{,}800} + \frac{1}{42}\right)A \qquad \text{For frequent floods}$$

$$Q = \left(\frac{127{,}000}{A + 23{,}700} + \frac{1}{86.5}\right)A \qquad \text{For rare floods}$$

Remarks: (1) The formulas were developed for floods on streams similar to Mohawk River in New York State. Kuichling noted that the formulas are applicable to hilly or mountainous regions, such as are found in New England, Middle and North Atlantic States, and are probably also applicable to a rolling country having a clayey surface soil.

(2) The original forms of the above formulas are

$$q = \frac{44{,}000}{D + 170} + 20$$

and

$$q = \frac{127{,}000}{D + 370} + 7.4$$

in which q = discharge in c.f.s. per sq. mi. and D = drainage area in sq. mi.

(3) These formulas apply to drainage areas larger than 100 sq. mi. For drainage areas less than 100 sq. mi., the corresponding formulas are

$$q = \frac{25{,}000}{D + 125} + 15$$

and

$$q = \frac{35{,}000}{D + 32} + 10$$

as published in *American Sewerage Practice* by Leonard Metcalf and Harrison P. Eddy, (Vol. I;

New York: McGraw-Hill Book Co., Inc., 1914), p. 250.

Reference: Emil Kuichling, *Annual Report on the Barge Canal, New York,* 1901, p. 848.

2-17. The Lauterberg formula

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

or

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

Remark: The formula was developed from floods due to continuous heavy rain of 3 to 4 days of duration at an average rate of 2 in. per day.

Reference: Emil Kuichling, *Annual Report on the Barge Canal, New York,* 1901.

2-18. The McCrory and Others formulas

$$(1)\quad Q = 0.159\,A^{5/6}$$

$$(2)\quad Q = \left(\frac{90}{A} + \frac{1}{64}\right)A$$

$$(3)\quad Q = \left(\frac{3{,}400}{A + 32{,}000} + \frac{1}{128}\right)A$$

Remarks: (1) These formulas were prepared for swamps and wet lands at particular locations.

(2) The first formula was given for Cypress Creek Drainage District, Arkansas.

(3) The second formula was given tentatively for certain Louisiana districts with no considerable storage in bayous and ditches.

(4) The third formula was proposed tentatively for certain districts in the Everglades of Florida.

Reference: *Drainage Investigation,* U. S. Department of Agriculture Bulletin 198, Publications of the Office of Experimental Station, Professional Papers of U. S. Department of Agriculture, 1915.

2-19. The Metcalf and Eddy formula

$$Q = 3.95\,A^{0.73}$$

or

$$Q = 440\,D^{0.73}$$

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

Reference: L. Metcalf and H. P. Eddy, *American Sewerage Practice* (Vol. I; New York: McGraw-Hill Book Co., Inc., 1941), p. 251.

2-20. The Meyer formula (also known as the Minnesota Flood Flow formula)

$$A = 100\,C\,F\,D^{0.6}$$

in which C = a coefficient depending on different slope, character of soil, and topography. Recommended values of C are as follows:

Character of Drainage Basin	Sandy Soil	Loam	Clayey Soil
1. Very flat agricultural or timber land with some marshes and swamps.	0.35	0.40	0.50
2. Relatively flat agricultural or timber land with some marshes and ponds.	0.45	0.50	0.60
3. Gently rolling agricultural or timber land full of lakes, ponds and marshes connected by poorly defined water courses.	0.50	0.60	0.75
4. Relatively flat agricultural or timber land of fairly uniform slope, without lakes and ponds.	0.60	0.70	0.85
5. Slightly undulating agricultural or timber land without lakes or ponds; or distinctly rolling to hilly agricultural or timber land, with lakes and ponds.	0.70	0.80	1.00
6. Gently rolling agricultural or timber land without lakes or ponds.	0.85	1.00	1.25
7. Distinctly rolling to hilly agricultural or timber land without lakes and ponds; or hilly agricultural or timber lands with steep slopes and lakes, ponds and marshes in valleys.	1.10	1.50	2.00

Character of Drainage Basin	Sandy Soil	Loam	Clayey Soil
8. Hilly agricultural or timber land with steep slopes barely admitting of cultivation; without lakes, ponds or marshes.	2.25	3.00	4.00
9. Very hilly timber or brush-covered land, slopes too steep for cultivation; ravines and gullies with occasional small ponds or marshes.	3.50	4.50	6.00
10. Very hilly timber or brush-covered land with much rock outcropping; ravines and gullies, occasional small ponds and marshes.	5.00	6.00	8.00
11. Very hilly to rugged country with much rock outcropping; scattered timber; occasional small ponds and marshes.	9.00	10.00	12.00
12. Rugged to precipitous rocky country with practically no soil cover; small timber and brush; ravines and gullies; no lakes, ponds, or marshes to retard runoff.	...	15.00	...

F = frequency factor recommended as follows:

For a flood to be expected	Factor F
Once in 10 years	0.85
Once in 25 years	1.00
Once in 100 years	1.40

Remarks: (1) This formula was developed by A. F. Meyer with the aid of C. M. Halseth in the Department of Drainage and Waters, State of Minnesota, particularly for Minnesota conditions.

(2) The formula was found to apply fairly well to floods which have occurred at many stations on the Mississippi and Ohio rivers.

References: E. V. Willard, *Drainage Areas of Minnesota Streams and Method of Estimating Probable Flood Flows*, Commissioner, Department of Drainage and Waters, State of Minnesota, October, 1922.

Adolph F. Meyer, *Elements of Hydrology* (New York: John Wiley and Sons, Inc., 1928), pp. 369-371.

2-21. The Morgan Engineering Co. formulas

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

or

$$Q = \left(\frac{380}{\sqrt{D}} + 8.0\right)D$$

$$(2)\quad Q = \left(\frac{1.1}{\sqrt{A}} + \frac{1}{88.8}\right)A$$

or

$$Q = \left(\frac{2.80}{\sqrt{D}} + 7.2\right)D$$

Remarks: (1) The first formula was used for the Cache River Drainage District.

(2) The second formula was used for Mississippi County, Arkansas.

(3) These formulas were used by the Morgan Engineering Company of Memphis, Tennessee, in their design of most drainage structures.

(4) These formulas were given for swamps and wet lands.

Reference: Daniel W. Mead, *Hydrology* (New York: McGraw-Hill Book Co., Inc., 1950), p. 666.

2-22. The Modified Myers formula

$$Q = 10{,}000\, C\, A^{0.5}$$

in which C = coefficient representing variable percentage expressing the ratio of maximum flood to an assumed extreme maximum flood.

= 5% for the Colorado River below the Gila Junction at Yuma
= 4% for the Nile
= 50% for the Amazon
= 21% for the Mississippi at Cairo, Illinois
= 32% for the Ohio at Paducah, Kentucky
= 10% for the Gila both at Yuma and Florence, Arizona
= 27% for the Salt River, before reservoir controls at the mouth and at Roosevelt, Arizona
= 117% for Salado Creek, Texas

$= 50\%$ for Miami, Dayton, Ohio
$= 40\%$ for Otay, Lower Otay Dam, California

Remark: The value of $100C$ is equivalent to the so-called "Myers scale" (Section II-B). Values of the Myers scale for major floods in the United States may be found in *Applied Hydrology* by R. K. Linsley, Jr., M. A. Kohler, and J. L. H. Paulhus (New York: McGraw-Hill Book Co., Inc., 1949), pp. 227-242.

Reference: C. S. Jarvis, "Flood Flow Characteristics," *Transactions, American Society of Civil Engineers*, Vol. 89 (1926), p. 994.

2-23. The Murphy and Others formula

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

or

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

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

Reference: E. C. Murphy and others, "Destructive Floods in the United States in 1904," *U. S. Geological Survey Water Supply and Irrigation Paper No. 147* (1905), p. 189.

2-24. The Nagler formula

$$Q = 2.84\, A^{2/3}$$

or

$$Q = 210\, D^{2/3}$$

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

Reference: F. A. Nagler, "A Survey of Iowa Floods," *Proceedings, Iowa Engineering Society*, 1928, pp. 48-68.

2-25. The New Kuichling formula

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

or

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

in which $Q =$ maximum discharge

Remark: Kuichling indicated that this formula applies to river basins in the Southern Atlantic States, and it is based on the greatest observed discharges of the Potomac River at Point of Rocks, Md., New River at Radford, Va., the Catawba River at Rock Hill, N. C., the Little Tennessee River at Judson, N. C., Can Creek at Bakersville, N. C., and numerous other streams which exhibit somewhat smaller rates of discharge than the preceding ones. It may be regarded as applicable to mountainous and hilly drainage basins having areas of not more than 10,000 sq. mi. in the part of the country indicated.

Reference: Emil Kuichling, "Discussion on Flood Flows," *Transactions, American Society of Civil Engineers*, Vol. 77 (1914), p. 649.

2-26. The O'Connell formula

$$Q = \sqrt{458\ (4.58 + A)}\ - 45.8$$

Remark: This formula was said to have been based on studies of rivers in Europe, India, and America, and to be best adapted to small drainage basins.

Reference: P. O. L. O'Connell, "Relation of the Freshwater Floods of Rivers to Areas and Physical Features of Their Basins; and on a Method of Classifying Rivers and Streams, with Reference to the Magnitude of Their Floods, Proposed as a Means of Facilitating the Investigation of the Laws of Drainage," *Proceedings, Minutes of the Institution of Civil Engineers*, Vol. 27 (1868), pp. 206-210.

2-27. The Ryves formula

$$Q = C\, A^{2/3}$$

in which $C =$ local coefficient depending on the rainfall, soil and slope of the district
$= 9.1$ for upper India

Remarks: (1) In the original form the drainage area is in sq. mi. and the values of C in India are as follows:

$C = 450$ within 15 miles of the coast
$C = 560$ within 15 to 100 miles inland
$C = 675$ for a limited area near the hills

(2) This formula is used extensively in India.

Reference: K. R. Sharma, *Irrigation Engineering* (Punjab, India: Rama Krishna and Sons, 1944), p. 575.

2-28. The Schnackenberg formula

$$Q = 20{,}000\, A^{0.5}$$

Remark: This formula was developed for extremely high runoffs for the worst New Zealand conditions.

Reference: E. C. Schnackenberg, *Extreme Flood Discharges*, New Zealand Institution of Engineers, 1949.

2-29. The U.S.G.S. formulas

No.	Region	Formula
1.	North Atlantic Slope	$Q = 190\ A^{0.5}$
2.	South Atlantic and Eastern Gulf of Mexico drainage	$Q = 250\ A^{0.5}$
3.	Ohio River Basin	$Q = 230\ A^{0.5}$
4.	St. Lawrence River Basin	$Q = 1{,}020\ A^{0.35}$
5.	Hudson Bay and Upper Mississippi drainage	$Q = 230\ A^{0.43}$
6.	Missouri River Basin	$Q = 130\ A^{0.5}$
7.	Lower Mississippi River Basin	$Q = 250\ A^{0.5}$
8.	Western Gulf of Mexico drainage	$Q = 34.5\ A^{0.77}$ (below 2,550,000 acres) $Q = 104{,}000\ A^{0.13}$ (above 2,550,000 acres)
9.	Colorado River Basin	$Q = 99\ A^{0.5}$
10.	The Great Basin	$Q = 26\ A^{0.5}$ (below 256,000 acres) $Q = 15{,}000$ (above 256,000 acres)
11.	Pacific Slope Basin in California	$Q = 200A^{0.5}$
12.	Pacific Slope Basins in Washington and Upper Columbia River Basin	$Q = 180\ A^{0.5}$
13.	Snake River Basin	$Q = 0.51\ A^{0.83}$
14.	Pacific Slope Basins in Oregon and Lower Columbia River Basin	$Q = 229\ A^{0.5}$

Remark: These formulas were developed from separate enveloping curves of peak streamflow for each of the 14 regions used by the U. S. Geological Survey for publication of streamflow data.

Reference: R. K. Linsley, Jr., M. A. Kohler, and J. L. H. Paulhus, *Applied Hydrology* (New York: McGraw-Hill Book Co., Inc., 1949), p. 580.

2-30. The Williams formula

$$Q = \frac{C}{D^n}$$

in which the coefficients C and n are as follows:

Locality	Coefficients	Drainage area Less than 10 sq. mi.	Drainage area 10-20,000 sq. mi.
North-East United States	C	1,480	2,400
	n	0.75	0.54
Mississippi Valley	C	2,500	4,800
	n	0.75	0.47
Rocky Mountains	C	1,900	3,600
	n	0.75	0.45
Pacific Coast, U.S.A.	C	1,625	2,700
	n	0.75	0.53
Uplands in British Isles	C	800	1,200
	n	0.75	0.54
Western India	C	2,700	4,600
	n	0.75	0.52
North-East India	C	1,400	1,700
	n	0.75	0.65

Remark: The coefficients for the United States are based on flood records listed in the paper "Flood Flow Characteristics" by C. S. Jarvis, *Transactions, American Society of Civil Engineers,* Vol. 89 (1926), pp. 985-1032. For the British Isles, they are based on flood records reported by a Committee on Floods of the Institution of Civil Engineers. For Western India, they are based on records of floods in the Bombay Presidency. For North-East India, they are based on papers presented before the Institution of Civil Engineers by Sir Gordon Hearn, G. L. Lillie, and E. L. Glass and the papers by W. A. Buyers before the Institution of Engineers, India.

Reference: G. B. Williams, *Storage Reservoirs* (London: Chapman and Hall, 1937), p. 71.

G. GROUP 3: RAINFALL INTENSITY FORMULAS

3-1. The Allen-Babbitt formula

$$I = \frac{200}{t + 20}$$

Remark: This formula was derived by H. E. Babbitt from Kenneth Allen's 25-year frequency curve based on a 51-year record observed at Central Park, New York City. See "The Prediction of Probable Rainfall Intensities" by Kenneth Allen, *Engineering News Record,* Vol. 86, No. 14 (April 7, 1921), p. 588.

Reference: *Handbook of Culvert and Drainage Practice* (Middletown, Ohio: Armco Drainage Products Association, 1945), p. 180.

3-2. The Bleich formulas

The reciprocal formula: $I = \dfrac{C_1}{t + b_1}$

The exponential formula: $I = \dfrac{C_2}{t^{n_2}}$

The modified exponential formula:

$$I = \frac{C_3}{(t + b_3)^{n_3}}$$

in which the parameters are as follows:

Storm	C_1	b_1	C_2	n_2	C_3	b_3	n_3
1-year	76.4	14	10.11	0.529	38.85	10	0.842
2-year	96.3	15	12.38	0.530	41.62	9	0.813
5-year	128.0	15	18.10	0.576	60.53	10	0.835
10-year	168.0	20	18.10	0.530	63.75	12	0.795
25-year	261.0	25	24.80	0.526	1,468	40	1.342
50-year	291.0	25	28.00	0.530	4,201	50	1.513

Remark: These formulas are said to be applicable to New York City.

Reference: S. D. Bleich, "Rainfall Studies for New York," *Transactions, American Society of Civil Engineers,* Vol. 100 (1935), pp. 618-619 and 621.

3-3. The Brackenbury formula

$$I = \frac{23.95}{t + 2.15} + 0.154$$

Remark: This formula was derived for Spokane, Washington.

Reference: R. A. Brackenbury, "Construction of a Large Sewer in Spokane," *Engineering Record,* Vol. 66, No. 6 (August, 1912), p. 156.

3-4. The de Bruyn-Kops formula

$$I = \frac{K}{t + b}$$

in which $K = 191$ and $b = 19$ for maximum storms
$K = 163$ and $b = 27$ for storms occurring once in 2 years
$K = 141$ and $b = 27$ for storms occurring once a year

Remark: This formula was developed for Savannah, Georgia, in 1908, and based on U. S. Weather Bureau Records, 1899-1906.

Reference: J. de Bruyn-Kops, "Notes on Rainfall at Savannah, Georgia," *Transactions, American Society of Civil Engineers,* Vol. 60 (1908), pp. 248-257.

3-5. The Bureau of Survey formula

$$I = \frac{K}{t^{0.5}}$$

in which $K = 27$ for a maximum storm
$K = 18$ for rainfalls of high intensity
$K = 9$ for ordinary storms

Remarks: (1) This formula was developed for Philadelphia, Pennsylvania as given in the Annual Report of the Bureau of Survey, 1911.

(2) It is based on a rainfall record of 25 years.

Reference: Leonard Metcalf and Harrison P. Eddy, *American Sewerage Practice* (Vol. I; New York: McGraw-Hill Book Co., Inc., 1914), p. 22.

3-6. The Clarke formula

$$I = \sqrt{\frac{K}{t}}$$

in which $K = 54$ for storms to be expected each year
$K = 162$ for storms to be exceeded once in 8 years
$K = 324$ for storms to be exceeded once in 15 years

Remark: The first two values of K were good representations of the New York rainfall records.

Reference: E. W. Clarke, "Storm Flows from City Areas, and Their Calculation," *Engineering News,* Vol. 48, No. 19 (1892), p. 388.

3-7. The Dorr formula

$$I = \frac{150}{t + 30}$$

Remarks: (1) This formula was developed for basis of storm sewer design at Boston, Massachusetts, in 1892.

(2) Coefficients for the rational formula to be used with this formula vary from 0.15 to 0.90, which were selected to suit the conditions the districts may attain in 25 or 30 years.

Reference: C. W. Sherman, "Maximum Rates of Rainfall at Boston," *Transactions, American Society of Civil Engineers,* Vol. 54 (1905), p. 179.

3-8. The Gregory formula

$$I = \frac{K}{t^n}$$

in which $K = 12$ and $n = 0.5$ for ordinary severe storms
$K = 6$ and $n = 0.5$ for winter storms
$K = 32$ and $n = 0.8$ for the maximum storm

Reference: C. E. Gregory, "Rainfall and Runoff in Storm Water Sewers," *Transactions, American Society of Civil Engineers,* Vol. 58 (1907), p. 475.

3-9. The Hendrick formula

$$I = \frac{K}{t + b}$$

in which $K = 300$ and $b = 25$ for heavy storms
$K = 105$ and $b = 10$ for basis of design

Remark: This formula was proposed for Baltimore, Maryland.

Reference: Calvin W. Hendrick, "Design and Construction of the Baltimore Sewerage System," *Engineering and Contracting,* Vol. 36, No. 6 (August, 1911), p. 160.

3-10. The Hill formula

$$I = \frac{120}{t + 15}$$

Remark: This formula was developed for Chicago, Illinois in about 1907 and based on the U. S. Weather Bureau observations on important storms, 1889-1910, inclusive.

Reference: Leonard Metcalf and H. P. Eddy, *American Sewerage Practice* (Vol. I; New York: McGraw-Hill Book Co., Inc., 1914), p. 224.

3-11. The Horner formula

$$I = \frac{56}{(t + 5)^{0.85}}$$

Remark: This formula was developed for St. Louis, Missouri, and based on the Weather Bureau record of excessive rains from 1873-1909.

Reference: W. W. Horner, "Modern Procedure in District Sewer Design," *Engineering News,* Vol. 64, No. 13 (September, 1910), p. 329.

3-12. The Institution of Civil Engineers' formula

$$I = \frac{8}{t + I}$$

in which I = extreme rainfall intensity in inches per hour likely to cause "catastrophe"
t = duration of storms in hours

Remark: This was developed for the British Isles, and adopted by the Committee of the Institution of Civil Engineers on "Floods in Relation to Reservoir Practice." For rainfalls likely to cause normal maximum floods, one-half the figure arrived by this formula was considered sufficient.

Reference: G. B. Williams, *Storage Reservoirs* (London: Chapman and Hall, 1937), p. 20.

3-13. The Kuichling formula

$$I = \frac{K}{t + b}$$

in which $K = 120$ and $b = 20$
$K = 106$ and $b = 13$

Remarks: (1) This formula was developed for the basis of design at Boston, Massachusetts, in 1905.

(2) This formula was found suitable for heavy rainfalls near New York City.

References: Emil Kuichling, "Discussion on Maximum Rates of Rainfall," *Transactions, American Society of Civil Engineers,* Vol. 54 (1905), p. 195.

C. E. Gregory, "Rainfall and Runoff in Storm Water Sewers," *Transactions, American Society of Civil Engineers,* Vol. 58 (1907), p. 475.

3-14. The L. J. Le Conte formula

$$I = \frac{7}{t^{0.5}}$$

Remark: This was found suitable for San Francisco, California.

Reference: L. J. Le Conte, "Discussion on Maximum Rates of Rainfall," *Transactions, American Society of Civil Engineers,* Vol. 54 (1905), p. 198.

3-15. The Metcalf and Eddy formula

$$I = \frac{K}{t^{0.5} + b}$$

in which $K = 15.5$ and $b = 0$ for Boston, Massachusetts.
$K = 14$ and $b = 0$ for Louisville, Kentucky.
$K = 19$ and $b = 0$ for New Orleans, Louisiana.
$K = 84$ and $b = 4$ for Denver, Colorado.

Remark: This formula was proposed in 1911 as a basis of design for maximum rainfalls.

Reference: L. Metcalf and H. P. Eddy, *American Sewerage Practice* (Vol. I; New York: McGraw-Hill Book Co., Inc., 1914), pp. 222, 224, 227-228.

3-16. The Meyer formula

$$I = \frac{K}{t + b}$$

in which the coefficients K and b are listed as follows:

Regional Group	Coefficients	Storm Frequency, Years 1	2	5	10	25	50	100
1	K	145	180	220	276	355	450	600
	b	23	24.5	27	32	40	50	65
2	K	100	131	171	214	252	289	325
	b	18	21	23.5	26	28	30	32
3	K	72	96	122	150	181	216	256
	b	13	16	18	19.5	21	23	25
4	K	60	84	108	132	160	186	210
	b	15	16	17.5	19	20	21	22
5	K	60	75	90	105	126	152	180
	b	13	13	13	13	14	16	18

Remark: The rainfall stations used by Meyer as basis for his formula cover the cities which are grouped in five regions as follows:

Group 1 ..... Galveston, New Orleans, Jacksonville

Group 2 ..... New York, Philadelphia, Washington, D. C., Norfolk, Raleigh, Savannah, Atlanta, Little Rock, Fort Worth, Abilene, Bentonville, St. Louis, Kansas City, Lincoln, Des Moines.

Group 3 ..... Boston, Albany, Pittsburgh, Elkins, Asheville, Knoxville, Memphis, Cairo, Indianapolis, Cincinnati, Cleveland, Detroit, Grand Haven, Chicago, Madison, St. Paul, Moorhead, Yanton, Dodge.

Group 4 ..... Duluth, Escanaba, Buffalo, Rochester.

Group 5 ..... Denver, Bismark.

Reference: Adolph F. Meyer, *Elements of Hydrology* (2nd ed.; New York: John Wiley and Sons, Inc., 1928), pp. 191-200.

3-17. The Nipher formula

$$I = \frac{360}{t}$$

Remark: This formula was based on a study of the rainfall intensity record of St. Louis, Missouri for a period of 47 years.

Reference: Leonard Metcalf and H. P. Eddy, *American Sewerage Practice* (Vol. I; New York: McGraw-Hill Book Co., Inc., 1914), p. 220.

3-18. The Sherman formula

$$I = \frac{K}{t^n}$$

in which $K = 38.64$ and $n = 0.687$ for maximum storms

$K = 25.14$ and $n = 0.687$ for basis of design of maximum storms

$K = 18$ and $n = 0.5$ for extraordinary conditions

Remark: This formula was applied to Chestnut Hill, Boston, Massachusetts.

Reference: C. W. Sherman, "Maximum Rates of Rainfall at Boston," *Transactions, American Society of Civil Engineers,* Vol. 54 (1905), pp. 178-179.

3-19. The Schafmayer formula

$$I = \frac{K}{t + b}$$

in which the coefficients are as follows:

Storm frequency, years	K	b
2	102	16
5	138	19
10	166	21
(15)*	(182)	(21)
20	193	22
(25)*	(203)	(22)

* Interpolated from Schafmayer's data.

Remarks: (1) This formula was developed for the Chicago area in 1938.

(2) This formula was intended for applications to storms not exceeding 120-minute duration.

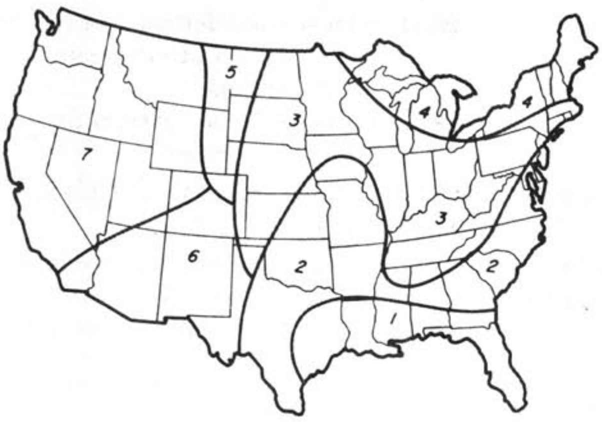
Reference: A. J. Schafmayer and B. E. Grant, "Rainfall Intensities and Frequencies," *Transactions, American Society of Civil Engineers,* Vol. 103 (1938), p. 355.

3-20. The Steel formula

$$I = \frac{K}{t + b}$$

in which K and b are coefficients depending on regions as shown in the following map (Figure 27) and table:

Frequency, Years	Coefficients	Regions 1	2	3	4	5	6	7
2	K	206	140	106	70	70	68	32
	b	30	21	17	13	16	14	11
5	K	247	190	131	97	81	75	48
	b	29	25	19	16	13	12	12
10	K	300	230	170	111	111	122	60
	b	36	29	23	16	17	23	13
25	K	327	260	230	170	130	155	67
	b	33	32	30	27	17	26	10
50	K	315	350	250	187	187	160	65
	b	28	38	27	24	25	21	8
100	K	367	375	290	220	240	210	77
	b	33	36	31	28	29	26	10



Remarks: (1) These formulas were prepared from Yarnell's data under the direction of Professor E. W. Steel.

(2) The formulas give the average maximum precipitation rates for durations up to 2 hours to be expected for the particular area and frequency.

Reference: E. W. Steel, *Water Supply and Sewerage* (New York: McGraw-Hill Book Co., Inc., 1947), pp. 350-352.

3-21. The Talbot formula

$$I = \frac{K}{t + b}$$

in which $K = 105$ and $b = 15$ for ordinary maximum
$K = 180$ and $b = 30$ for maximum occurring once in about fifteen years
$K = 360$ and $b = 30$ for maximum exceeded two or three times per century

Remark: These formulas were developed for areas east of the Rocky Mountains.

Reference: A. N. Talbot, "Rates of Maximum Rainfall," *Technograph No. 5,* University of Illinois (1891-92), p. 103.

3-22. The Webster formula

$$I = \frac{12}{t^{0.6}}$$

Remark: This formula was said to be applicable to ordinary conditions for Philadelphia, Pennsylvania.

Reference: Harold E. Babbitt, *Sewerage and Sewage Treatment* (New York: John Wiley and Sons, Inc., 1940), p. 40.

3-23. The Whiney formula

$$I = \frac{0.6}{(t + 4)^{2/3}}$$

Remark: This formula is based on excessive precipitation data at nine cities in Northeastern United States.

Reference: S. Whiney, "Discussion on Maximum Rates of Rainfall," *Transactions, American Society of Civil Engineers,* Vol. 54 (1905), p. 203.

3-24. The Williams formula

$$I = \frac{K}{t + b}$$

in which t = duration of the rainstorm in hours
$b = 0.75$
$K = 3$ for maximum probable in short periods
$K = 6$ for maximum probable in 100 years
$K = 9$ for maximum probable in long periods of time

Remark: These formulas were developed for Eastern India. However, the results are probably on the low side and would certainly be so for some parts of India, and still more so for Ceylon.

Reference: G. B. Williams, *Storage Reservoirs* (London: Chapman and Hall, 1937), p. 20.

H. GROUP 4: FREQUENCY FORMULAS

4-1. The Creager formula

$$Q = C\,D^{0.5}\left[\frac{2 - e^{-0.04D^{-0.3}}}{3}\left(1 - \frac{\ln 0.1T}{3}\right) + \frac{\ln 0.1T}{3}\right]$$

in which C = coefficient depending upon the characteristics of the drainage area, equal to 6,000 for areas most favorable to large floods

Remark: The coefficient C has to be determined by the judgment of the engineer.

Reference: W. P. Creager and J. D. Justin, *Hydroelectric Handbook* (New York: John Wiley and Sons, Inc., 1927), p. 55.

4-2. The Fuller formula

$$Q_{\max} = Q\,(1 + 2D^{-0.3})$$

in which $Q = Q_{av}(1 + 0.8 \log T)$
$Q_{av} = C D^{0.8}$

Remarks: (1) In this formula, Q_{av} is the average of the annual 24-hour flood discharge, Q is the probable greatest average discharge for 24 consecutive hours during a period of T years, or the maximum 1-day flood, and Q_{max} is the maximum flood discharge; all quantities are in c.f.s.

(2) The expression for Q_{max} is approximate as it was based on only 26 available records of the maximum flood and the 24-hour flood. The maximum flood does not necessarily occur on the same day as the 24-hour flood.

(3) This formula was developed primarily for eastern streams of the United States.

Reference: W. E. Fuller, "Flood Flows," *Transactions, American Society of Civil Engineers,* Vol. 77 (1914), p. 567.

4-3. The Horton formula — A

$$Q_{av} = Q_{max}\left(1 - e^{-bT_e^n}\right)$$

in which Q_{max} = maximum possible flood
Q_{av} = average magnitude of flood
T_e = average exceedance interval of an event of magnitude Q
b and n = factors varying with locality

Remarks: (1) The value of Q_{max} is to be assumed, and Q_{av}, b, and n are to be determined from observed frequency data.

(2) From data of flood records for 70 years for the Connecticut River at Hartford, Connecticut, (1843-1917), $Q_{max} = 1.82Q_{av}$, $b = 0.255$, and $n = 0.54$.

Reference: R. E. Horton, "Discussion of Flood Flow Characteristics," *Transactions, American Society of Civil Engineers,* Vol. 89 (1926), p. 1086.

4-4. The Horton formula — B

$$Q = 4{,}021.5T^{0.25}$$

in which Q = flood discharge equalled or exceeded in an average interval of T years

Remarks: (1) This formula was developed for streams in Eastern Pennsylvania.

(2) Special cases were also developed as follows:

$Q = 30T^{0.25}D$ for the Neshaming ($D = 1{,}393$).
$Q = 30\,T^{0.27}D$ for the Perkiomen ($D = 152.0$).
$Q = 40\,T^{0.25}D$ for the Tohickon ($D = 102.2$).

Reference: R. E. Horton, "Discussion of Flood Flows," *Transactions, American Society of Civil Engineers,* Vol. 77 (1914), p. 665.

4-5. The Lane formula

$$Q = C \log(T + b)$$

in which T = period in years in which a given Q will be equalled or exceeded
C and b = constants

Remark: This formula was developed for New England streams.

Reference: C. S. Jarvis, "Hydrology," Section 2 of *Handbook of Applied Hydraulics,* edited by C. V. Davis (New York: McGraw-Hill Book Co., Inc., 1942), p. 111.

I. GROUP 5: ELABORATE DISCHARGE FORMULAS

5-1. The Adams formula

$$Q = CAI\sqrt[12]{\frac{S}{A^2 I^2}}$$

in which $C = 1.035$
I = 1.0 or maximum intensity of rainfall in inches per hour
S = slope in feet per thousand feet

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

References: J. W. Adams, *Sewers and Drains for Populous Districts* (New York: D. Van Nostrand, 1880).

Emil Kuichling, "Storm Water in Town Sewerage," *Transactions, Association of Civil Engineers,* Cornell University, Vol. I (1893-94), p. 41.

5-2. The Besson formula

$$Q = CA^n = R\,T\,G\,A^n$$

For any drainage area,

$$Q_{max} = Q_r \frac{R_m C_1}{R_r C_2}$$

in which Q_{max} = maximum discharge
Q_r = recorded discharge
R_m = maximum one-day rainfall
R_r = recorded one-day rainfall
C = coefficient equal to the product of the precipitation
R in inches, the topographic factor

T, and a factor G for ground surface conditions. C_1 is for maximum condition and C_2 for recorded condition.

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

Reference: F. S. Besson, "Maximum Flood Flow Prediction," *The Military Engineer*, Vol. 25, No. 143 (September and October, 1933), p. 424.

5-3. The Boston Society of Civil Engineers' formula

$$Q = C_1 R \sqrt{A}$$

or

$$Q = C_2 R \sqrt{D}$$

in which $C_1 = \frac{\sqrt{A}}{t}$ and $C_2 = \frac{1{,}290\sqrt{D}}{t}$, where t is the time in hours of the flood period

$C_1 = 2.4$ to 4 and $C_2 = 60$ to 100 for flat streams with relatively large channel pondage

$C_1 = 4$ to 24 and $C_2 = 100$ to 500 for ordinary conditions

$C_1 = 20$ to 40 and $C_2 = 500$ to 1,000 for mountainous regions

R = total flood runoff, inches on drainage area

= 3 in. for occasional floods in New England (25 to 75 yr. frequency)

= 6 in. for rare floods in New England (50 to 200 yr. frequency)

= not over 8 in. for maximum floods in New England

Remarks: (1) This formula gives the total runoff and is based on floods in New England.

(2) This formula is based upon a concept that peak flows tend to vary directly with the total volume of flood runoff.

Reference: "Report of the Committee on Floods, March 1920," *Journal, Boston Society of Civil Engineers,* Vol. 130 (September, 1930), p. 297.

5-4. The Burge formula (see The Dredge formula)

5-5. The Bürkli-Ziegler formula

$$Q = C\,A\,I\left(\frac{S}{A}\right)^{0.25}$$

in which C is a coefficient depending upon the character of the ground surface as follows:

Description of Area	Values of C
For densely built up area where streets, walks and yards are paved and the remaining area is practically all roof area as in downtown districts	0.75
For areas adjacent to downtown district where streets and alleys are paved and yards small	0.70
For densely built-up residence districts where streets are paved and houses are close together	0.65
For ordinary residential areas	0.55–0.65
For areas having small yards and a medium density of population	0.45–0.55
For sparsely built-up areas or those having large yards	0.35–0.45
For suburbs with gardens and lawns and macadamized streets	0.30
For parks, golf courses, etc. covered with sod and having no pavements	0.20

I = average rate of rainfall in inches per hour during the heaviest storm, varying from 1.0 to 3.0 and commonly using 2.75 in the Middle West

Remarks: (1) This formula was first published in "The Greatest Discharge of Municipal Sewers," (Grösste Abflussmengen bei Städtischen Abzugskanäle) by the Swiss hydraulic engineer, A. Bürkli-Ziegler, in Zurich in 1880. In 1881, it was introduced to American technical literature in a report on "Sewerage Works in Europe" by Rudolph Hering to the National Board of Health.

(2) It is based on the expression of Hawksley's formula.

(3) In deriving the formula, observations extended up to slopes of 10 feet per 100, but were limited to small areas of less than 50 acres. The formula was found especially applicable to sewer design.

Reference: Rudolph Hering, "Sewerage Systems," *Transactions, American Society of Civil Engineers,* Vol. 10 (1881), p. 362.

5-6. The Chamier formula

$$Q = 5\,C\,I\,A^{0.75}$$

or

$$Q = 640\,C\,I\,D^{0.75}$$

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

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

Remark: This formula was tested by Chamier on streams in New South Wales along the Cootamundra-Gundagai Railway having drainage areas of from 20 acres to 400 sq. mi.

Reference: George Chamier, "Capacities Required for Culverts and Flood-Openings," *Proceedings, Institution of Civil Engineers*, Vol. 134 (1898), pp. 313-323 (p. 319).

5-7. The Craig formula

$$Q = 440\,C\,W \ln\left(\frac{8L^2}{W}\right)$$

in which L = mean length of the drainage area in miles

W = mean width of the drainage area in miles

$C = C_1 V R$

where C_1 = coefficient of discharge

V = velocity towards the culvert in feet per second

R = depth of rainfall in inches

Remarks: (1) Generally, $C = 0.68$ to 1.95, varying with rainfall and topography.

(2) The formula is based on Indian records.

Reference: J. Craig, "Maximum Flood-Discharge from Catchment Areas, with Special Reference to India," *Minutes of Proceedings, Institution of Civil Engineers*, Vol. 80 (1884-1885), p. 201.

5-8. The Cramer formula — A

$$Q = \frac{C\,C_1\,R\,A\,S^{1/3}}{57{,}600 + (27{,}000{,}000\,C_1\,R\,A)^{1/3}}$$

in which C = 186 for rough, natural basins of rivers.

C = 697 for smooth, comparatively level, and impervious areas such as well-built cities.

R = mean annual rainfall in inches.

C_1 = a coefficient depending on the total area A in acres, the flat area A_1 in acres, which is likely to be inundated in freshets, and the mean annual rainfall R, such relation being expressed by:

$C_1 = 1 - \sin\left(-\tan^{-1}\frac{709\,A_1}{AR}\right)$ for the simple case when A_1 is distributed in an approximately uniform manner throughout the whole basin, and

$C_1 = 1 - \sin\left(-\tan^{-1}\frac{1{,}418\,A_1}{AR}\right)$ if A_1 is concentrated only at the lower end of the basin.

S = the mean slope and declivity of the whole basin in feet per thousand; or $(s_1 + s_2)/2L$, where s_1 is the average altitude in feet, s_2 the altitude of the point of discharge, and L the average distance in feet traveled by the water from the boundaries of the watershed to the point of measurement.

Remarks: (1) The original form of the formula is

$$Q = \frac{C_3\,R_3\,m\,A\,(S_2)^{1/3}}{9 + (0.0658\,m\,R_3\,A)^{1/3}}$$

in which C_3, R_3, m, A and S_2 correspond to C, R, C_1, A, and S respectively, and A is expressed in square miles.

(2) The original form was reduced by Kuichling, for conditions on Mohawk River, to

$$Q = \frac{80.6}{1 + 0.1347\,(A)^{1/3}}\,A$$

Reference: "Report of Sub-Committee of Roadway Committee No. 1, Bulletin 131," *Proceedings, American Railway Engineering and Maintenance of Way Association*, Vol. 12, Pt. 3 (March, 1911), p. 498.

5-9. The Cramer formula — B

$$Q = 640\,R\,C\,A^{3/4}$$

Reference: Emil Kuichling, *Annual Report on the Barge Canal, New York*, 1901.

5-10. The Dredge or Burge formula

$$Q = \frac{2.03\,A}{L^{2/3}}$$

or

$$Q = \frac{1{,}300\,D}{L^{2/3}}$$

Remarks: (1) This formula, based on Indian flood records, was used for the Madras Railway, India.

(2) If the area is taken as a rectangle having dimensions of L by $(\frac{1}{2})L$ in miles, then the formula reduces to $Q = 1{,}030\,D^{2/3}$.

Reference: J. T. Fanning, *A Practical Treatise on Water-Supply Engineering* (New York: D. Van Nostrand Co., Inc., 1878), p. 66.

5-11. The Gregory formulas

$$Q = C\,I\,S^{0.186}\,A^{\,0.86}$$

in which $CI = 2.8$ for impervious surface; and

$$Q = 105\,C\,(84\sqrt[5]{AS^2} + 25)$$

in which $C = 0.10$ to 0.54

Remarks: (1) This formula was developed for use in New York, 1907.

(2) The formula was based on the rational formula.

Reference: C. E. Gregory, "Rainfall and Runoff in Storm Water Sewers," *Transactions, American Society of Civil Engineers,* Vol. 58 (1907), p. 474.

5-12. The Gregory and Arnold formula

$$Q = \frac{(3{,}600\,t)^{4nn_1}}{(1{,}000)^{2nn_1}}\left(\frac{C_1}{L}\right)^{4nn_1}(CAI)^{4c}\,(C_2)^{\,8nn_1}\,(S)^{1.5nn_1}$$

in which C = coefficient representing the ratio of the rate of runoff to the rate of rainfall

C_1 = constant, depending on shape of drainage area and its manner of concentration. $C_1 = v/V$, in which v = average velocity of water in feet per second in traversing the length L and V = velocity of Q at outlet of drainage area, based on average values of C_2 and S for the area

C_2 = constant, depending on shape and condition of main channel of flow

I = average rainfall intensity, in inches per hour, for a period of t hours

L = length, in feet, which water must traverse in running from the most remote portion of the drainage area to its outlet.

S = fall, in feet per 1,000 feet of main channel of flow

n = a positive fractional exponent in rainfall intensity formula

t = time of concentration in hours

$n_1 = 1/(4 - n)$

Remark: This formula was derived by the rational formula with several important basin factors being considered.

Reference: R. L. Gregory and C. E. Arnold, "Runoff — Rational Runoff Formulas," *Transactions, American Society of Civil Engineers,* Vol. 96 (1932), pp. 1038-1099.

5-13. The Gregory and Hering formula

$$Q = C\,I\,A^{0.833}\,S^{0.27}$$

Remarks: (1) This formula was deduced by Charles E. Gregory in 1907 from diagrams of runoff to be expected in New York City prepared in 1889 by Rudolph Hering.

(2) The value of $CI = 1.02$ for suburban areas to 1.64 for metropolitan areas.

References: C. E. Gregory, "Rainfall and Runoff in Storm Water Sewers," *Transactions, American Society of Civil Engineers,* Vol. 58 (1907), p. 458.

Leonard Metcalf and H. P. Eddy, *American Sewerage Practice* (Vol. I; New York: McGraw-Hill Book Co., Inc., 1914), p. 235.

5-14. The Grunsky formula — A

For maximum urban storm-water flow:

$$Q = \frac{5\,C\,I\,A}{\sqrt{t}}$$

For maximum stream flow from large areas:

$$Q = \frac{3{,}200\,C\,I\,A}{\sqrt{t}}$$

For general applications:

$$Q = \frac{C_2\,I\,A}{t^n}$$

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

I = maximum rainfall in 1 hour, based on the California record

t = critical time in minutes for continuance of rainfall

$C_1 = 0.5$ for impervious areas

$C_1 = 5.0$ for mountainous areas

$C_1 = 20.0$ for rolling country

$C_1 = 50.0$ for flat country

$C_1 = 250.0$ for sandy regions

$C_2 = 3{,}500$ and $n = 0.5$ for impervious areas

$C_2 = 3{,}300$ and $n = 0.6$ for mountainous areas

$C_2 = 3{,}000$ and $n = 0.7$ for rolling country

$C_2 = 2{,}100$ and $n = 0.75$ for flat country

$C_2 = 600$ and $n = 0.8$ for sandy regions

Remark: This formula was based on California records.

Reference: C. E. Grunsky, "Rainfall and Runoff Studies," *Transactions, American Society of Civil Engineers,* Vol. 85 (1922), p. 67.

5-15. The Grunsky formula — B

$$Q_{\max} = \frac{C\,C_1\,I\,A}{t^n}$$

in which $Q_{\max}$ = maximum rate of discharge

t = time of concentration in hours

$C = 0.586$ and $n = \frac{3}{4}$ for less than 0.33 hr.

$= 0.782$ and $n = \frac{1}{2}$ for t greater than 0.33 hr. and less than 64 hrs.

$= 1.562$ and $n = \frac{2}{3}$ for t greater than 64 hrs.

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

$C_2 = 0.013$ for impervious areas

$= 0.25$ for mountains

$= 0.40$ for rolling country

$= 1.3$ for flat country (ordinary soil)

$= 6.5$ for sandy regions

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

Reference: R. L. Gregory and C. E. Arnold, "Runoff — Rational Runoff Formulas," *Transactions, American Society of Civil Engineers,* Vol. 96 (1932), p. 1146.

5-16. The Hawksley formula

$$Q = CAI\sqrt[4]{\frac{S}{AI}}$$

in which $C = 0.7$

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

Remark: This formula was derived from the Hawksley original formula (1-3).

Reference: Emil Kuichling, "Storm Water in Town Sewerage," *Transactions, Association of Civil Engineers,* Cornell University, Vol. I (1892-93), p. 47.

5-17. The Hering formula

$$Q = \frac{RVA}{640\,L} \text{ or } \frac{RVA}{640\,t}$$

in which R = total runoff in inches during a storm

V = mean velocity of the stream

L = length of the river

t = time of concentration

Remark: In the original form of the formula, the drainage area is expressed in square miles.

Reference: T. M. Cleemann, "Railroad Engineers' Practice, Discussion on Formulas," *Proceedings, Engineers' Club of Philadelphia,* Vol. I (1879), p. 146.

5-18. The Iszkowski formula

$$Q_m = (0.022\,C_1 + b\,C_2)\,R\,D$$

in which Q_m = probable maximum flood discharge in cubic meters per second

R = mean annual depth of rainfall in meters

D = drainage area in square kilometers

$C_1 = 0.20$ for very flat, sandy, or swampy areas to 0.65 for high mountainous areas

$C_2 = 0.035$ for very permeable land covered with vegetation to 0.70 for impervious rocky or frozen land, without active vegetation, and covered with snow which will increase the runoff by melting

$b = 7.88$ for $D = 10$ square kilometers to 0.65 for $D = 100{,}000$ square kilometers as set forth in a table from which Kuichling has deduced the approximate relation:

$$b = \frac{0.59\,(11{,}150 + D)}{818 + D}$$

Remarks: (1) This formula was converted to English units by Kuichling as follows:

For a hilly territory with slightly permeable soil and sparse vegetation:

$$Q_{\max} = \frac{0.568\,R\,(4{,}129.5 + D)}{315.8 + D}$$

For mountainous territory with rocky or frozen soil:

$$Q_{\max} = \frac{0.848\,R\,(4{,}147.4 + D)}{315.8 + D}$$

where $Q_{\max}$ = maximum discharge in cubic feet per second

R = mean annual depth of rainfall in inches

D = drainage area in square miles

(2) This formula was proposed by R. Iszkowski, Chief Engineer of the Austrian Ministry of Public Works in 1884. It was called an "induction formula for estimating the normal and flood discharges, based on the characteristics of the watershed."

Reference: Emil Kuichling, "Discussion of Flood Flows," *Transactions, American Society of Civil Engineers,* Vol. 77 (1914), p. 648.

5-19. The Kinnison and Colby formulas

$$Q = (0.000036\, s^{2.4} + 124)\, \frac{D^{0.95}}{P^{0.04}\, L^{0.7}} \quad \text{for minor floods}$$

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

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

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

in which Q = the peak discharge in c.f.s.
s = the median altitude of the drainage basin in feet above the outlet
P = the percentage that lake, pond, and reservoir surface is to the total drainage area
L = average distance in miles which water from runoff uniformly distributed over the basin must travel to the outlet

Remark: These formulas were developed by the U. S. Geological Survey for the Commonwealth of Massachusetts.

Reference: H. B. Kinnison and B. R. Colby, "Flood Formulas Based on Drainage Basin Characteristics," *Transactions, American Society of Civil Engineers,* Vol. 110 (1945), p. 871.

5-20. The Lauterberg formula

$$Q = 0.0001285\, C\, R\, A$$

in which Q = average discharge of a river in c.f.s.
R = total annual rainfall in inches
C = 0.20 for marshy soil
= 0.25 for level plains
= 0.30 for rolling land
= 0.35 for low hills
= 0.45 for hilly country like the Ardennes, the Odenwald and the Eifel
= 0.55 for the Black Forest and the Vosges
= 0.70 for high rocky mountains

Remarks: (1) This formula is based upon data collected in Switzerland.

(2) In the original form, the drainage area is in square kilometers and a coefficient of 0.03171 is used.

Reference: *Abstract in the Minutes of Proceedings, Institution of Civil Engineers,* Vol. 149 (1887), p. 392, from Lauterberg's *Schweizerische Strömabflussingen* (Bern: Huber and Co., 1876).

5-21. The Lillie formula

$$Q = V\, R\, C\, \Sigma(\theta\, L)$$

in which Q = discharge in c.f.s. at the moment of peak flood
V = standard mean velocity in feet per second
$R = 2 +$ annual rainfall/15
$C = 1.1 + \log L$
L = length of sectors of drainage area in miles
θ = angle in degrees, at the discharge point, of the sections into which the catchment is divided. The sections are in fan shape having a common center meeting at the discharge point.

Remark: This formula was developed with reference to rivers in India.

Reference: G. E. Lillie, "Discharge from Catchment-Areas in India, as Affecting the Waterways of Bridges," *Proceedings, Institution of Civil Engineers,* Vol. 217 (1924), p. 309.

5-22. The McMath formula

$$Q = C A I \sqrt[5]{\frac{S}{A}}$$

in which C = 0.20 for rural sections
= 0.30 for macadamized streets
= 0.75 for paved streets
= 0.75 for St. Louis, Missouri
I = 1.9 to 2.75 for maximum intensity of rainfall in inches per hour; the latter value was used for St. Louis.
S = slope of the ground surface in feet per thousand; a value of 15 being recommended for St. Louis

Remark: This formula was proposed for St. Louis, Missouri.

Reference: R. E. McMath, "Determination of the Size of Sewers," *Transactions, American Society of Civil Engineers,* Vol. 16 (1887) p. 183.

5-23. The Parmley formula

$$Q = C\, I\, S^{1/4} A^{5/6}$$

in which $C = 0$ to 1

$I =$ rainfall intensity in in. per hr. for a period of 8 to 10 minutes for the Walworth Run River, Cleveland. Use $I = 4$ in. per hr. for the most violent storms and also for the further damage caused by the prevailing direction of the storms.

Remark: This formula was used for the Walworth Sewer of Cleveland, Ohio.

Reference: W. C. Parmley, "The Walworth Sewer, Cleveland, Ohio," *Transactions, American Society of Civil Engineers,* Vol. 55 (1905), p. 345.

5-24. The Pettis formula

$$Q = C\ (R\, W)^{1.25}$$

in which $Q =$ approximately the 100-year flood peak in c.f.s.

$C =$ 310 for the humid regions east of the Mississippi and along the Pacific Coast

$=$ 100 for level prairie region of Illinois

$=$ 200 for the semi-arid Rocky Mountain region such as Colorado

$=$ 40 in desert region

$R =$ 100-year 1-day rainfall in inches

$W =$ average width of the basin in miles as determined by dividing the area of the basin by the length of the stream, neglecting minor sinuosities

Remarks: (1) This formula, also known as the "width formula," is designed for use on unregulated basins having areas greater than 100 but less than 40,000 sq. mi.

(2) It was found that the formula was applied successfully to basins of the areas stated, ranging from twice as long as they are wide to others which are about nine times as long as they are wide.

(3) This formula was first given as

$$Q = 328\ (R\, W)^{1.25}$$

in which R is the maximum rainfall in inches to be expected on the basin within a period of six days, on the average of once in 100 years. In 1934, the formula was revised to use a general coefficient C in place of 328 as shown above.

References: C. R. Pettis, Major, *A New Theory of River Flow,* Corps of Engineers, U. S. Army, Baltimore, Maryland, 1927.

C. R. Pettis, "Flood Probability Formula Modified to Simplify Application," *Engineering News-Record,* Vol. 112 (June, 1934), pp. 804-805.

C. R. Pettis, "Relation of Rainfall to Flood Run-off," *Military Engineers,* Vol. 28, No. 158 (1936), pp. 94-98.

5-25. The Possenti formula

$$Q = C\frac{R}{L}\left(A_2 + \frac{A_1}{3}\right)$$

in which $C =$ coefficient with an average of 1.72

$A_1 =$ flat area in acres

$A_2 =$ hilly area in acres

$R =$ depth of 24-hr. rainfall in inches

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

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

Reference: W. E. Fuller, "Flood Flows," *Transactions, American Society of Civil Engineers,* Vol. 77 (1914), pp. 564-617.

5-26. The Protodiakonov formula

$$Q_m = 16.67\ (I_d k - i)\ A_k$$

in which $I_d =$ design rainfall intensity in centimeters per minute

$k =$ climatic factor equal to the ratio of the maximum rainfall intensity at the given watershed to that at the center of European Soviet Union

$i =$ permeability of soil in centimeters per minute to be determined experimentally

The design rainfall intensity is computed by

$$I_d = \frac{5}{1 + 0.06t}$$

in which $t =$ duration of rainfall in minutes which is equal to the time of concentration or

$$t = 16.67\left(\frac{L_1}{V_1} + \frac{L_2}{V_2}\right)$$

in which $L_1 =$ length of the channel in kilometers

$L_2 =$ half width of the drainage area in kilometers

$V_1 =$ velocity of flow in the channel in meters per second

V_2 = velocity of overland flow in meters per second

Remarks: (1) This formula was developed by M. M. Protodiakonov and recommended for the determination of runoff due to intense storms on defrozen soils in summer as stated in "The Specifications for Normal Runoff" of the U.S.S.R. Public Transportation Committee in 1931 and Transportation Design Committee in 1938 and 1939.

(2) The value of I_d is for a frequency of 500 years. For the design of culverts and small bridges, the recurrence interval is in the order of 50 to 100 years. The corresponding design discharge should be equal to the computed discharge divided by a factor *m*, or

$$Q_{50\text{-}100} = \frac{Q_{500}}{m}$$

the values of m are as follows:

All types of small bridges	1.25
Culverts, inverted siphons, siphons, embankments, drainage ditches with possible ponding or overflowing	1.50
Temporary structures	1.75
Drainage ditches without ponding	2.00

Reference: V. V. Lebedev, *Hydrology and Hydrometry Manual* (Soviet Hydrometeorological Publication, 1955).

5-27. The Rhind formula

$$Q = \frac{C\,S\,R\,D^n}{L}$$

in which C = coefficient depending on R/L

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

R = greatest annual rainfall

L = greatest length of drainage basin

n = a variable index

Remark: The formula appears to be founded on observation of rivers with drainage areas exceeding 41 sq. mi.

Reference: G. R. Hearn, "The Effect of Shape of Catchment on Flood-Discharge," *Proceedings, Institution of Civil Engineers,* Vol. 217 (1924), p. 289.

5-28. The Ribeiro formula

$$Q_m = \frac{17\,F\,C\,C_1\,A_k\sqrt{R}}{\sqrt{A_k+10}}$$

in which $C_1 = 1.28 - 0.07\,\dfrac{L^2}{A_k}$

A_k = drainage area considered, in square kilometers (total or partial watershed)

L = length of path of raindrop in kilometers, from farthest point of the watershed to point where flood is to be computed

Q_m = maximum flood discharge in cubic meters per second

F = frequency factor, equal to 1.00 for a maximum storm

R = mean (weighted) average annual rainfall in inches over the watershed

C = 1.00 for pastures of cultivated ground with vegetation

= 1.25 for barren or rocky slopes

= 0.75 for highly wooded ground

Reference: George Ribeiro, "Formula Presented for Estimating Flood Discharges," *Civil Engineering,* Vol. 21, No. 11 (November, 1951), p. 661.

5-29. The U.S.S.R. NTK-NKPS formula

$$Q_m = G\,C\,A_k$$

in which G = geographical factor, being equal to the maximum design rainfall intensity in centimeters per minute at the site of the drainage basin, divided by 16.67

C = coefficient depending on permeability of soils, runoff conditions, and channel length and slope

Remarks: (1) This formula applies to drainage areas less than $4G$ sq. km. when $G > 15$ and less than 60 sq. km. when $G < 15$.

(2) For unfrozen porous soils (less than 1 meter thick of sand, gravels, or limestone with crevices) the computed discharge should be reduced by 50%. When there is influence of forest on the drainage basin, the reduction should not be greater than 20% for drainage areas greater than 20 sq. km. When the soils are impervious (compact clay or crystalline rocks with no crevices) or frozen, the computed discharge may be increased 20%.

(3) Experience has shown that the discharge computed by the formula is usually too high. According to the All-Soviet Standards (GOST), the discharge thus computed should be multiplied by 0.7.

(4) This formula was proposed by the Technical Research Committee of the U.S.S.R. Public Communication Committee (NTK-NKPS) in 1928. It

has been used by the Design Division of the U.S.S.R. Department of Interior, Bureau of Highway Administration, for the design of culverts and small bridges.

Reference: V. V. Lebedev, *Hydrology and Hydrometry Manual* (Soviet Hydrometeorological Publication, 1955).

5-30. The U.S.S.R. Scientific Academy formula

$$Q_m = \frac{C\, H^{5/4}\, r^{5/4}\, C_1^{3/4}\, I^{3/17}\, W^{1/4}}{3\, t_c^{5/4}\, L^{3/4}} A_k$$

in which Q_m = Maximum discharge in cubic meters per second

C = coefficient for maximum discharge (for full maximum discharge or Q_{100}, $C = 1$)

H = average water-content of snow in millimeters before melting and becoming runoff as indicated by a long period observation (to be obtained from a snow isohyetal map)

t_c = shortest time for infiltration during a 24-hour intensive rainfall (to be obtained from an isohyetal map)

r = parameter for forestation, computed by the formula $\frac{1}{1 + A_k'/A_k}$, where A_k' in square kilometers is the part of the area A_k that is forested and A_k is the sub-drainage area under consideration

C_1 = roughness coefficient, equal to 6.5 for areas without forest and 5.0 for forested areas

S = average slope of the main channel counting from the upstream edge of the drainage area to the culvert

W = average width of the drainage area or A_k/L

L = length of the outlet channel from the edge of the drainage area to the outlet

A_k = total drainage area in square kilometers

Reference: M. F. Sribnyi, "Method of Determining Maximum Flood Discharge from Its Relation to the Area of the Watershed," *Russian Scientific Academy News* (Isv. AN SSSR otd. tekh nauk) No. 1 (January, 1952), p. 151.

5-31. The Switzer and Miller formula

$$Q = R\, C\, W^n$$

in which Q = 24-hr. flood in c.f.s.

R = rainfall in inches

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

C = 80

n = 1.5

Remarks: (1) The formula is based on a study of 47 rivers in the United States.

(2) When Q is expressed for peak flows in c.f.s., then $C = 135$ and $n = 1.4$.

Reference: F. G. Switzer and H. G. Miller, *Floods* (Engineering Experiment Station Bulletin No. 13), Cornell University, December, 1929, p. 11.

5-23. The Walker formula

$$Q = \frac{C\, R\, D}{L^{5/6}}$$

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

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

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

Reference: Thomas Walker, "Flood Discharge Formulas," *American Railway Engineering Association Bulletin*, Vol. 24, No. 248 (August, 1922), pp. 23-26.

APPENDIX II. REFERENCES CITED

(References marked * are also annotated in Appendix III)

1*. T. M. Cleemann, "Proper Amount of Water-Way for Culverts," *Proceedings, Engineers'*

Club of Philadelphia, Vol. 1 (April 5, 1879), pp. 146-149.

2*. A. M. Wellington, "Culvert Proportions," Editorial, *Railroad Gazette,* Vol. 18 (September, 1886), pp. 629-630.

3. C. S. Jarvis, "Flood Flow Characteristics," *Transactions, American Society of Civil Engineers,* Vol. 89 (1926), pp. 985-1032.

4. C. S. Jarvis, "Average and Maximum River Discharges," Appendix I in *Low Dams,* National Resources Committee, 1938, pp. 393-426.

5*. A. N. Talbot, "The Determination of Water-Way for Bridges and Culverts," Selected Papers of the Civil Engineers' Club, *Technograph No. 2,* University of Illinois, 1887-8, pp. 14-22.

6. A. Bürkli-Ziegler. *The Greatest Discharge of Municipal Sewers (Grösste Abflussmenge bei Städtischen Abzugkanälen).* Zurich: Orell, Füssli & Co., 1880.

7. Rudolph Hering, "Sewerage Works in Europe," *Report to the National Board of Health,* 1881, and "Sewerage Systems," *Transactions, American Society of Civil Engineers,* Vol. 10 (1881), pp. 361-384.

8. W. C. Curd, "Discussion of the Best Method for Determining the Size of Waterways," Appendix B of Report of Committee No. 1 — on Roadway, *Proceedings, American Railway Engineering and Maintenance of Way Association,* Vol. 10, Pt. II (1909), pp. 991-993. The chart entitled "Logarithmic Diagram for Area of Waterway Required by Talbot's Formula" is between pp. 992 and 993.

9*. Bill Turner, "Graph for Talbot's Formula," *Engineering News Record,* Vol. 126, No. 25 (June 19, 1941), p. 971.

10. "Diagram for Solution of Talbot Formula for Culvert Sizes," *Handbook of Drainage and Construction Products,* Armco Drainage and Metal Products Inc., 1958, p. 227.

11. C. B. McCullough, "Design of Waterway Areas for Bridges and Culverts," *Oregon State Highway Commission Bulletin No. 4,* July, 1934.

12*. C. B. Coe, "Talbot's Formula by Slide Rule," *Civil Engineering,* Vol. 9, No. 8 (August, 1939), pp. 496-497.

13. Emil Kuichling, "The Relation Between the Rainfall and the Discharge of Sewers in Populous Districts," *Transactions, American Society of Civil Engineers,* Vol. 20 (1889), pp. 1-56.

14*. J. C. I. Dooge, "The Rational Method for Estimating Flood Peaks," *Engineering,* (London), Vol. 184 (September 6 and 20, 1957), pp. 311-313, 374-377.

15. T. J. Mulvaney, "On the Use of Self-Registering Rain and Flood Gauges in Making Observations of the Relations of Rainfall and of Flood Discharges in a Given Catchment," *Transactions, Institution of Civil Engineers of Ireland,* Vol. 4, Pt. 2 (1850-51), p. 18.

16. D. E. Lloyd-Davies, "The Elimination of Storm Water from Sewerage Systems," *Minutes of Proceedings, Institution of Civil Engineers,* Vol. 164 (1906), p. 41.

17*. D. B. Krimgold, "On the Hydrology of Culverts," *Proceedings, 26th Annual Meeting, Highway Research Board,* Vol. 26 (1946), pp. 214-226.

18. W. W. Horner and S. W. Jens, "Surface Runoff Determination from Rainfall Without Using Coefficients," *Transactions, American Society of Civil Engineers,* Vol. 107 (1942), pp. 1039-1075.

19*. W. D. Potter, "Surface Runoff from Agricultural Watersheds," *Surface Drainage, Highway Research Board Report No. 11-B,* 1950, pp. 21-35.

20. Merrill Bernard, "Modified Rational Method of Estimating Flood Flows," Appendix A in *Low Dams,* National Resources Committee, 1938, pp. 209-233.

21. R. L. Gregory and C. E. Arnold, "Runoff — Rational Runoff Formulas," *Transactions, American Society of Civil Engineers,* Vol. 96 (1932), pp. 1038-1099.

22. G. H. Bremner, James Dun, J. W. Alvord, A. N. Talbot, and others. "Symposium on Methods of Determining the Size of Waterway for Bridges and Culverts," *Journal, Western Society of Civil Engineers,* Vol. 11 (April, 1906), pp. 137-190.

23*. W. D. Pence, "The Best Method of Determining the Size of Waterways" (Appendix B of Report of Committee No. 1 on Roadway), *Proceedings, American Railway Engineering and Maintenance of Way Association,* Bulletin 108, Vol. 10, Pt. II (1909), pp. 967-1022.

24. C. D. Purdon, "Discussion on Flood Flow Characteristics," *Transactions, American Society of Civil Engineers,* Vol. 89 (1926), p. 1090.

25*. W. G. Berg, "How to Determine Size and Capacity of Openings for Waterways," *Report of the 7th Convention, American Railway Bridge and Building Association*, 1897, pp. 86-100.

26. "Report of Sub-Committee of Roadway Committee No. 1, Bulletin 131," *Proceedings, American Railway Engineering and Maintenance of Way Association*, Vol. 12, Pt. 3 (March, 1911), pp. 481-528.

27*. T. M. Munson, "Formulas and Methods in General Use by Engineers to Determine Runoff from Watershed Areas," *Texas Engineer*, Vol. 4, No. 4 (April, 1934), p. 7.

28*. C. E. Ramser, "Brief Instruction on Methods of Gully Control," U. S. Department of Agricultural Engineering (mimeographed), August, 1933, pp. 16, 21-23, 26-27.

29*. C. E. Ramser, "Runoff from Small Agricultural Areas," *Journal of Agricultural Research*, Vol. 34, No. 9 (May 1, 1927), pp. 797-823.

30. A. F. Meyer. *Elements of Hydrology* (1st ed.). New York: John Wiley and Sons, Inc., 1917. Table 15.

31. D. L. Yarnell, "Rainfall Intensity Frequency Data," *U. S. Department of Agriculture Miscellaneous Publication No. 204*, August, 1935.

32*. D. B. Krimgold, "Runoff from Small Drainage Basins," *Agricultural Engineering*, Vol. 19, No. 10 (October, 1938), pp. 439-446.

33*. D. B. Krimgold and N. E. Minshall, "Hydrologic Design of Farm Ponds and Rates of Runoff for Design of Conservation Structures in the Claypan Prairies," *U. S. Department of Agriculture, Soil Conservation Service Research Report SCS-TP-56*, May, 1945.

34. C. L. Hamilton and H. G. Jepson, "Stock-Water Development: Wells, Springs, and Ponds," *Farmers' Bulletin No. 1859*, U. S. Department of Agriculture, 1940, p. 39.

35. "Engineering Handbook," U. S. Soil Conservation Service, Milwaukee Office, 1942.

36*. R. B. Hickok, R. V. Keppel, and B. R. Rafferty, "Hydrograph Synthesis for Small Aridland Watersheds," *Agricultural Engineering*, Vol. 40, No. 10 (October, 1959), pp. 608-611, 615.

37. "Hydrology," Supplement A, Section 4 of *Engineering Handbook*, U. S. Department of Agriculture, Soil Conservation Service, 1957.

38*. R. R. Rowe and R. L. Thomas, "Comparative Hydrology Pertinent to California Culvert Practice," *California Highways and Public Works*, Vol. 20 (September, 1942), pp. 6-11.

39. *California Culvert Practice*. Division of Highways, Department of Public Works, State of California, 1st ed., 1944, 2nd ed., 1953.

40*. I. E. Houk, "Hydraulic Design of Bridge Waterways," *Engineering News Record*, Vol. 88, No. 26 (June 29, 1922), pp. 1071-1075.

41*. G. P. Springer, "Adequate Waterways for Culverts and Bridges," *Proceedings, 17th Annual Road School at Purdue University, January 19-23, 1931*, Purdue University Engineering Bulletin, Vol. 15, No. 2 (March, 1936).

42*. F. W. Greve, "Hydrological Factors in the Design of Culverts and Small Bridges," *Roads and Bridges*, Vol. 81, No. 4 (April, 1943), pp. 32, 66, and 68.

43*. F. T. Mavis, "Reducing Unknowns in Small Culvert Design," *Engineering News Record*, Vol. 137, No. 2 (July 11, 1946), pp. 51-52.

44*. J. C. Merrell. "Hydrology for Highway Structures," *Proceedings of the Ohio Highway Engineering Conference, April 3-5, 1951* (Engineering Experiment Station Bulletin No. 145). Columbus, Ohio: The Ohio State University, 1951. Pp. 64-77.

45*. J. F. Exum. "Waterway Area for Culverts and Small Bridges," *Proceedings of the Ohio Highway Engineering Conference, April 3-5, 1951* (Engineering Experiment Station Bulletin No. 145). Columbus, Ohio: The Ohio State University, 1951. Pp. 61-63.

46*. C. F. Izzard, "Importance of Drainage Design in Highway Construction," *Proceedings of the 3rd California Conference on Street and Highway Problems*, Institute of Transportation and Traffic Engineering, University of California, 1951.

47*. C. F. Izzard, "Peak Discharge for Highway Drainage Design," *Transactions, American Society of Civil Engineers*, Vol. 119 (1954), pp. 1005-1015.

48*. H. G. Bossy, "Simple Methods for Hydraulic Design of Culverts," *Proceedings, Southeastern Association of Highway Officials*, October, 1952, pp. 34-46.

49. "Highway Practice in the United States of

America," Public Roads Administration, 1949, pp. 136-137.

50*. "Basic Principles of Highway Drainage," *Hydraulic Information Circular No. 1,* U. S. Bureau of Public Roads, 1951.

51*. W. D. Potter, "Rainfall and Topographic Factors that Affect Runoff," *Transactions, American Geophysical Union,* Vol. 34 (1953), pp. 68-73.

52*. W. D. Potter, "Use of Indices in Estimating Peak Rates of Runoff," *Public Roads,* Vol. 28, No. 1 (April, 1954), pp. 1-8.

53. E. M. West and W. H. Sammons, "A Study of Runoff from Small Drainage Areas and the Openings in Attendant Drainage Structures," *University of Kentucky Highway Research Laboratory Report No. 2,* July, 1955.

54*. George Chamier, "Capacities Required for Culverts and Flood Openings," *Proceedings, Institution of Civil Engineers,* Vol. 134 (1898), pp. 313-323.

55*. C. S. Jarvis, "General Formula for Waterways," *Engineering World,* Vol. 29 (August, 1926), pp. 79-83.

56*. Tate Dalrymple, "Hydrology in Design of Bridge Waterways," *North Carolina Engineer,* July, 1952.

57. "Monthly Precipitation and Runoff for Small Agricultural Watersheds in the United States," U. S. Department of Agriculture, Agricultural Research Service, since July, 1957.

58. "Annual Maximum Flows from Small Agricultural Watersheds in the United States," U. S. Department of Agriculture, Agricultural Research Service, June, 1958.

59*. F. F. Snyder, "Synthetic Flood Frequency," *Proceedings of the American Society of Civil Engineers, Journal of the Hydraulics Division,* Vol. 84, No. HY5, Pt. 1 (October, 1958), pp. 1-22.

60. "Selected Runoff Events for Small Agricultural Watersheds in the United States," U. S. Department of Agriculture, Agricultural Research Service, January, 1960.

61*. N. E. Minshall, "Predicting Storm Runoff on Small Experimental Watersheds," *Proceedings of the American Society of Civil Engineers, Journal of Hydraulics Division,* Vol. 86, No. HY8, Pt. 1 (August, 1960), pp. 17-38.

62*. W. D. Potter, "Peak Rates of Runoff from Small Watersheds," *Hydraulic Design Series No. 2,* U. S. Department of Commerce, Bureau of Public Roads, U. S. Government Printing Office, April, 1961.

63*. G. A. Tilton and R. R. Rowe, "Culvert Practice in California," *Proceedings, Twenty-third Annual Meeting, Highway Research Board,* Vol. 23 (1943), p. 205.

64. V. T. Chow, (Chairman) and others, "Report of the Committee on Runoff, 1955-56," *Transactions, American Geophysical Union,* Vol. 38, No. 3 (June, 1957), pp. 379-384.

65. V. T. Chow, "Hydrologic Studies of Floods in the United States," in *Floods,* Vol. III of *Symposia Darcy, International Association of Scientific Hydrology, Publication No. 42,* 1956, pp. 134-170.

66. L. K. Sherman, "Stream Flow From Rainfall by the Unit-Graph Method," *Engineering News-Record,* Vol. 108 (April 7, 1932), pp. 501-505.

67*. E. F. Brater, "The Unit Hydrograph Principle Applied to Small Watersheds," *Transactions, American Society of Civil Engineers,* Vol. 105 (1940), pp. 1154-1178.

68. D. L. Yarnell, "Rainfall Intensity-Frequency Data," U. S. Department of Agriculture Miscellaneous Publication 204, 1936.

69. "Rainfall Intensity-Duration-Frequency Curves for Selected Stations in the United States, Alaska, Hawaiian Islands, and Puerto Rico," *U. S. Weather Bureau Technical Paper No. 25,* December, 1955.

70. "Rainfall Intensity-Frequency Regime, Part 4 — The Ohio Valley," *U. S. Weather Bureau Technical Report No. 29,* 1957.

71. V. T. Chow. *Frequency Analysis of Hydrologic Data with Special Application to Rainfall Intensities* (Engineering Experiment Station Bulletin No. 414). Urbana, Ill.: University of Illinois College of Engineering, 1953.

72. V. T. Chow, "Hydrologic Studies of Urban Watersheds, Rainfall and Runoff of Boneyard Creek, Champaign-Urbana, Illinois," *University of Illinois Civil Engineering Studies, Hydraulic Engineering Series No. 2,* November, 1952.

73. F. A. Huff, "Rainfall Intensity-Frequency Data for Champaign-Urbana, Illinois," *Illinois State Water Survey Division Circular No. 28,* 1949.

74. F. A. Huff and J. C. Neil, "Frequency Relations for Storm Rainfall in Illinois," *Illinois State Water Survey Division Bulletin 46*, 1959.

75. "Rainfall Frequencies," *Illinois State Water Survey Division Technical Letter No. 1*, 1959.

76. "Rainfall Frequencies for 5 to 60 Minutes," *Illinois State Water Survey Division Technical Letter No. 4*, 1960.

77. R. H. Shaw, G. L. Barger, and R. F. Dale, "Precipitation Probabilities in North Central States," *North Central Region Publication No. 115*, Agricultural Experimental Stations of Illinois, Indiana, Iowa, Kansas, Michigan, Minnesota, Missouri, Nebraska, North Dakota, Ohio, South Dakota, Wisconsin and the U. S. Department of Agriculture cooperating, University of Missouri Agricultural Experiment Station, June, 1960.

78. "Maximum Recorded United States Point Rainfall for 5 Minutes to 24 Hours at 207 First Order Stations," *U. S. Weather Bureau Technical Paper No. 2*, April, 1947.

79. "Maximum Station Precipitation for 1, 2, 3, 6, 12, and 24 Hours," *U. S. Weather Bureau Technical Paper No. 15*, Part XX: Indiana, (1956); Part XXI: Illinois, (1958); and Part XXII: Ohio, (1958).

80. "Maximum 24-Hour Precipitation in the United States," *U. S. Weather Bureau Technical Paper No. 16*, January, 1952.

81. J. T. Riedel, J. F. Appleby, and R. W. Schloemer, "Seasonal Variation of the Probable Maximum Precipitation East of the 105th Meridian for Areas from 10 to 1,000 Square Miles and Durations of 6, 12, 24, and 48 Hours," *U. S. Weather Bureau and Corps of Engineers Hydrometeorological Report No. 33*, April, 1956.

82. W. D. Mitchell, "Unit Hydrographs in Illinois," U. S. Geological Survey and Illinois Division of Waterways, 1948.

83. W. D. Mitchell, "Floods in Illinois: Magnitude and Frequency," U. S. Geological Survey and Illinois Division of Waterways, 1954.

84. H. L. Wascher, J. B. Fehrenbacher, R. T. Odell, and P. T. Veale, "Illinois Soil Type Descriptions," AG-1443, Department of Agronomy, Agricultural Experiment Station, University of Illinois, 1950.

85. S. A. Changnon, Jr., "Second Progress Report, Illinois Cooperative Project in Climatology, 1 July 1955 through 30 June 1956," *Illinois Water Survey Circular No. 57*, 1956.

86. F. A. Huff and J. C. Neill, "Frequency Relations for Storm Rainfall in Illinois," *Illinois State Water Survey Bulletin 46*, 1959.

87. R. Morgan and D. W. Hullinghors, "Unit Hydrographs for Gaged and Ungaged Watersheds" (Unpublished manuscript), U. S. Engineers Office, Binghamton, New York, July, 1939.

88. F. F. Snyder, "Synthetic Unit-Graphs," *Transactions, American Geophysical Union*, Vol. 19 (1938), pp. 447-454.

89*. Z. P. Kirpich, "Time of Concentration of Small Agricultural Watersheds," *Civil Engineering*, Vol. 10, No. 6 (June, 1940), p. 362.

90. B. D. Richards. *Flood Estimating and Control.* London: Chapman and Hall, 1955.

91. J. J. O'Kelly, "The Employment of Unit Hydrographs to Determine the Flows of Irish Arterial Drainage Channels," *Proceedings, Institution of Civil Engineers*, Vol. 4, Pt. III (August, 1955), pp. 365-412.

92. J. E. Nash, "Frequency of Discharges from Ungaged Catchments," *Transactions, American Geophysical Union*, Vol. 37, No. 6 (December, 1956), pp. 719-725.

93. J. E. Nash, "The Form of the Instantaneous Unit Hydrograph," *Comptes-Rendus et Rapports, Assemblée de Toronto 1957*, International Association of Scientific Hydrology, Vol. 3, Publication No. 45 (1958), pp. 114-121.

94. J. C. I. Dooge, "A General Theory of the Unit Hydrograph," *Journal of Geophysical Research*, Vol. 64, No. 2 (February, 1959), pp. 241-256.

95. B. L. Golding and D. E. Low, "Physical Characteristics of Drainage Basins," *Proceedings, American Society of Civil Engineers, Journal of Hydraulics Division, Paper 2409*, Vol. 86, No. HY3 (March, 1960), pp. 1-11.

APPENDIX III. ANNOTATED SUPPLEMENTARY BIBLIOGRAPHY

Allan, A. G., "Some Railway Construction in Oklahoma," *Transactions, American Society of Civil Engineers,* Vol. 51 (1903), pp. 424-442.

Presents a method of determining bridge and culvert opening using drainage table by James Dun. Shows approximate areas of waterways for drainage areas of 0.1 to 6,000 sq. mi.

Allis, J. A., "Runoff from Conservation and Nonconservation Watersheds," *Agricultural Engineering,* Vol. 34 (1953), pp. 766-768.

Small experimental drainage basins at Hastings, Nebraska, are compared to illustrate the methods of analysis and the effects of conservation methods.

"Basic Principles of Highway Drainage," *Hydraulic Information Circular No. 1, U. S. Bureau of Public Roads,* 1951. [50]*

Contains methods of determination of peak rates of runoff, hydraulic design of culverts, principle of flow in open channels, location and slope of culverts, types of culverts, bridge waterways, roadway drainage, and design of ditches and gutters.

Berg, W. G., "How to Determine Size and Capacity of Openings for Waterways," *Report of the 7th Convention, American Railway Bridge and Building Association,* 1897, pp. 86-100. [25]

Committee report of methods used on railroads to obtain quick and practical results.

Bigwood, B. L., and M. P. Thomas, "Flood-Flow Formula for Connecticut," *U. S. Geological Survey Circular No. 365,* 1955.

Describes a study of the frequency and flood magnitude within the Connecticut area and the development of regional flood frequency relationship and formula for mean annual flood for ungaged areas.

Bodhaine, G. L., "Flood-Frequency Relationships in the Pacific Northwest," *Proceedings, American Society of Civil Engineers, Journal of the Hydraulics Division,* Vol. 86, No. HY9 (1960), Paper No. 2644, pp. 1-10.

Describes a method of determining the magnitude and frequency of expected floods that is applicable to the Pacific Northwest of the United States.

Bolling, A., "Maximum Probable Storm Runoff from Small Watersheds (1-100 sq km) in Norway," *Teknisk Ukeblad,* Vol. 75, No. 18 (1928), pp. 181-183.

Presents records of observations, formulas, diagrams and tables for computing probable maximum volumes of floods and storm runoff, and also chart and formula giving intensity of runoff as a function of drainage-basin area.

Bossy, H. G., "Simple Methods for Hydraulic Design of Culverts," *Proceedings, Southeastern Association of Highway Officials,* October, 1952, pp. 34-46. [48]

Discusses an engineering method practised by the Bureau of Public Roads for design of culverts, including a brief description and comment on early design methods, determination of runoff, hydraulic design of culverts flowing full and part-full, and containing charts for determining peak rates of runoff and for estimating headwater of culverts flowing full and part-full.

Brater, E. F., "The Unit Hydrograph Principle Applied to Small Watersheds," *Transactions, American Society of Civil Engineers,* Vol. 105 (1940), pp. 1154-1178. [67]

Application of the unit-hydrograph principle to drainage-basin areas varying from 4.24 acres to 1,877 acres. The principle was found sound and applicable.

Bremner, G. H., "Areas of Waterways for Railroad Culverts and Bridges," *Journal, Western Society of Engineers,* Vol. II, No. 2 (1906), pp. 137-146, discussion, pp. 146-190.

Publication of the Dun waterway table and other information for the determination of culvert and bridge openings.

Bunch, C. M., "Discharge Records and Their Value in Design of Bridge Waterways," *Georgia Geological Survey Bulletin 60,* 1953, pp. 157-161.

Describes briefly the need for considering flow conditions in bridge design.

California Culvert Practice, State of California, Department of Public Works, Division of Highways, 1st ed., 1944; 2nd ed., 1953.

Describes the culvert design practice at California Highway Public Works, including determination of design discharge, hydraulic design, and structural design of culverts of all sizes. Includes a nomogram relating physical features of a drainage area to peak flow.

* Numbers in brackets are cross-references to "Appendix II — References Cited."

Carreker, J. R., and A. P. Barnett. *Rainfall and Runoff Characteristics on a Small Watershed in the Southern Piedmont.* U. S. Soil Conservation Service, SCS-TP-114, 1953.

Reports observations over a 12-year period on a 19-acre drainage basin near Watkinsville, Georgia. Effect of cropping on runoff is evaluated, probable peak flows are estimated, and the extent of periods without runoff is summarized for design of farm ponds.

Carreker, J. R., "The Effects of Rainfall, Land Slope, and Cropping Practices on Runoff and Soil Losses," *Journal, Soil Water Conservation,* Vol. 9 (1954), pp. 115-119.

A report of plot tests at Watkinsville, Georgia, over a 12-year period.

Chamier, George, "Capacities Required for Culverts and Flood Openings," *Proceedings, Institution of Civil Engineers,* Vol. 134 (1898), pp. 313-323. [54]

Describes basic elements for determination of culvert openings, including catchment area, rainfall, amount of surface-discharge and diminution in proportionate flood-discharge due to area. Presents a formula and its practical applications.

Chamier, George, "Flood Discharges," *Engineering Record,* Vol. 39, No. 8 (1899), pp. 163-165.

An elaborate review of runoff from different areas resulting in a new formula for estimating it.

Chamier, George, "Capacities for Culverts and Flood Discharge," *Engineering News,* Vol. 41 (January, 1899), p. 61.

Gives a formula proposed for average cases and the results of tests.

Chow, V. T., "Frequency Analysis in Small Watershed Hydrology," *Agricultural Engineering,* Vol. 39 (April, 1958), pp. 222-225.

Since a significant difference in hydrology exists between small and large drainage basins, the author discusses the application of frequency analysis as it pertains only to small drainage basins or for upstream regions which comprise most agricultural areas.

Cleemann, T. M., "Proper Amount of Water-Way for Culverts," *Proceedings, Engineers' Club of Philadelphia,* Vol. 1 (April 5, 1879), pp. 146-149. [1]

Presentation of the Myers formula for computation of waterway areas.

Coe, C. B., "Talbot's Formula by Slide Rule," *Civil Engineering,* Vol. 9, No. 8 (August, 1939), pp. 496-497. [12]

Slide rule settings for solution of Talbot's formula for dimensioning of culvert openings.

Cornelius, M. G., A. C. Kyser, and S. P. Gilbert, "Rational Design of Culverts and Bridges," Third reprint of *District No. 12 Manual,* Texas Highway Department, 1946.

Describes hydrologic and hydraulic methods of designing culverts and bridges. Determination of runoff is made by the rational method.

"Curves for Determining Areas of Openings for Road Culverts and Bridges," Illinois Society of Engineers and Surveyors, Drainage Committee, excerpt published in *Engineering and Contracting,* Vol. 39, No. 10 (1913), pp. 255-256.

Compares results using 13 different formulas for waterway areas.

Dalrymple, Tate, "Use of Stream Flow Records in Design of Bridge Waterways," *26th Annual Meeting Proceedings, Highway Research Board 1946,* 1947, pp. 163-179.

Discusses measurement of the physical features of the stream channel, determination of the design flood by the Gumbel method of frequency analysis, and the hydraulic computation of water level at the bridge site and extent of backwater upstream.

Dalrymple, Tate. "Discharge Records as Used in Design of Bridge Waterways," *Proceedings of Ohio Highway Engineering Conference 1949* (Engineering Experiment Station Bulletin No. 136). Columbus, Ohio: The Ohio State University, 1949. Pp. 152-168.

Describes the computation of magnitude and frequency of floods and recommends design procedure for single-span and multiple-span bridges and culverts.

Dalrymple, Tate, "Cooperative U.S.G.S. Program Improves Design of Bridge Waterway Areas," *Civil Engineering,* Vol. 21, No. 3 (1951), p. 50.

Outlines the limitations involved in the use of empirical flood flow formulas. Demonstrates application of published hydrologic data to bridge waterway design.

Dalrymple, Tate, "Hydrology in Design of Bridge

Waterways," *North Carolina Engineer,* July, 1952. [56]

Discusses the function of a bridge as a grade separation structure for both the vehicular traffic as well as the hydraulic traffic, requirement of magnitude and frequency of flood discharges, stage-discharge relationship, and physical and hydraulic characteristics of channel for a proper calculation of the size of a bridge opening; flow through single and multiple spans, factors considered in design; and factors of safety.

Davison, A. H., "Methods of General Application Developed for Finding Peak Flood Flow," *Civil Engineering,* Vol. 22, No. 6 (1952), pp. 418-420.

A series of charts, diagrams, and formulas which the author uses to develop an empirical method of computing peak flow. The method developed is basically a modification of the Manning formula.

"Design of Waterway Areas for Bridges and Culverts," *Oregon State Highway Commission — Highway Department Technical Bulletin No. 4,* 1934.

Compilation on economic design of waterway areas and results of research by Oregon State Highway Department in past 15 years, proportioning waterway areas directly from formulas, drainage tables, or from flood discharge; relieving backwater head, deepening of channel due to construction; theory of scour; current stresses; and ice effect, drift impact.

Donovan, D. E., "Chart for Drainage Calculations," *Engineering News Record,* Vol. 135, No. 8 (1945), pp. 250-251.

Gives a formula for determining the time of concentration necessary in calculating storm flows.

Dooge, J. C. I., "Rational Method for Estimating Flood Peaks," *Engineering,* (London) Vol. 184, No. 4774 (September 6, 1957), pp. 311-313, and No. 4776 (September 20, 1957), pp. 374-377. [14]

Method used in determining size of trunk sewers, surface drainage channels, highway culverts and bridge waterways, attempts of group of Irish engineers, working more than 100 years ago, to develop a satisfactory method of predicting peak rate of runoff from small drainage basins; various steps from state of virtual ignorance to fully developed rational method being clearly traced.

Enger, M. L., "The Relation of Waterways to Drainage Areas," *Engineering Record,* Vol. 57 (1908), p. 138, reprinted from a paper read at the Annual Meeting of the Illinois Society of Engineers and Surveyors.

Brief notes on the size of waterway for culverts and bridges with particular reference to Minnesota and Dakota conditions.

Exum, J. F. "Waterway Area for Culverts and Small Bridges," *Proceedings of the Ohio Highway Engineering Conference, April 3-5, 1951* (Engineering Experiment Station Bulletin No. 145). Columbus, Ohio: The Ohio State University, 1951. Pp. 61-63. [45]

Emphasizes that changing conditions have brought about the necessity for more precise and accurate methods of designing the waterway area than the old method of the application of a formula such as Talbot formula. Suggests two parts of study: (1) ascertaining the quantity of water to be carried through the structure in a unit of time, and (2) selecting the optimum size, shape, and position of the structure. The rational method is recommended.

Garstka, W. V., "Hydrology of Small Watersheds Under Winter Conditions of Snow Cover and Frozen Soil," *Transactions, American Geophysical Union,* 1944, pp. 838-1060.

Measurements of winter runoff and precipitation on 3 small Michigan drainage basins.

Greve, F. W., "Hydrological Factors in the Design of Culverts and Small Bridges," *Roads and Bridges,* Vol. 81, No. 4 (1943), pp. 32, 66, 68. [42]

Various methods of estimating peak flow are reviewed, including comparison with other streams, empirical formulas, and actual flow measurement.

Greve, F. W., "Bridge and Culvert Flow Areas — Culvert Practice in Determining Runoff," *Civil Engineering,* Vol. 13, No. 8 (1943), pp. 381-382.

Same as Greve's paper in *Roads and Bridges.*

Griffith, J. R., "Submerged Outlet Culvert Discharge — Construction Design," *Western Construction News,* Vol. 14, No. 3 (1939), p. 102.

Gives alignment charts for the solution of an exponential formula for culvert discharge.

Gutmann, I., "New Cloudburst Flood Formula," *Engineering News Record,* Vol. 117, Nos. 14, 19, and 26 (1936), pp. 474-475, 655, 905.

Presents an Italian formula for phenomenal cloudburst discharges modified to fit unusually high runoff records in the United States and abroad. The formula applies to drainage basins of 1 to 70 sq. mi. in size.

Harrison, E. A., "Determination of the Area of Waterway Required," *Canadian Engineer,* June 5, 1908.

Discusses various methods of computing culvert areas.

Harrold, L. L., "Flow from Drainage Basins Determined by Short-Term Records," *Transactions, American Society of Civil Engineers,* Vol. 111 (1946), pp. 597-608, discussion by Samuel Roberts and the author, pp. 609-612.

Runoff data from small agricultural drainage basins of Soil Conservation Experiment Station at La Crosse, Wisconsin, are used to derive values of intervals. Suggests procedure for adjusting short period to a normal by comparing the rainfall frequency curve for the period to a nearby long-record frequency curve.

Harrold, L. L., "Hydrologic Design of Farm Ponds and Rates of Runoff for Design of Conservation Structures in North Appalachian Region," *U. S. Department of Agriculture, Soil Conservation Service — Research SCS-TP 64,* Washington 25, D. C., 1947, 18 pp.

Presents the hydrologic design of farm ponds, gives rates of runoff for use in the design of conservation structures on agricultural areas up to 10,000 acres, and lists data on sedimentation.

Hickok, R. B., R. V. Keppel, and B. R. Rafferty, "Hydrograph Synthesis for Small Arid-Land Watersheds," *Agricultural Engineering,* Vol. 40, No. 10 (October, 1959), pp. 608-611, 615. [36]

Method is based on analysis of rainfall and runoff records for 14 experimental drainage basins in Arizona, New Mexico and Colorado; it involves estimation of characteristic lag time from basin parameters, use of basin lag time to predict hydrograph peak rate of assumed total volume of runoff, synthesizing entire hydrograph using lag time, estimating peak rate, and standard dimensionless hydrograph.

Hoad, W. C., "Waterway Required at Culverts for Drainage Areas in Eastern Kansas," *Engineering and Contracting,* Vol. 37, No. 17 (1912), pp. 481-482.

Presentation of a formula for waterway area applicable to the unusual combination of slope, soil, and cover of eastern Kansas and Missouri.

Horner, W. W., "A Rational Culvert Formula," *Engineering News,* Vol. 69, No. 18 (1913), pp. 912-913.

Conversion of a rainfall curve for the St. Louis area to a formula for the discharge of a small stream.

Horner, W. W., "The Analysis of Hydrologic Data for Small Watersheds," *Soil Conservation Service Technical Publication No. 30,* 1940.

Analysis of data from several small drainage basins by "losses" method.

Houk, I. E., "Hydraulic Design of Bridge Waterways," *Engineering News Record,* Vol. 88, No. 26 (June, 1922), pp. 1071-1075. [40]

Discusses individual design required for each opening, makeshift nature of formulas, cloudburst data, applying 1913 flood figures, scour velocities, and other considerations.

"Instruction for Bridge and Culvert Surveys," Kentucky State Highway Department, no date.

Describes arrangement of data, drainage areas, culverts and bridge survey items, and methods of determining sizes of openings for culverts and small bridges in Kentucky, the rational method being employed.

Izzard, C. F., "Importance of Drainage Design in Highway Construction," *Proceedings of the 3rd California Conference on Street and Highway Problems,* Institute of Transportation and Traffic Engineering, University of California, 1951. [46]

An outline of some ideas as to how the knowledge of hydraulics can be applied to the design of highway drainage structures, including the use of charts and diagrams for investigating the hydraulic properties of culverts, the entrance and outlet controls, the hydraulic jump, the determination of peak run-off rates for small drainage basins, and the estimation of backwater caused by bridges.

Izzard, C. F., "Empirical Formulas for Bridge Waterways Questioned," *Civil Engineering,* Vol. 46 (1951), p. 54.

A rebuttal to an October, 1950 article on empirical design formulas, pointing out limitations of empirical equations.

Izzard, C. F., "Peak Discharge for Highway Drainage Design," *Transactions, American Society of Civil Engineers*, Vol. 119 (1954), pp. 1005-1015. [47]

Illustrates the use of regional stream-flow frequency curves as a basis for estimating design flow for bridges and culverts on streams without records. Describes procedure using statistical correlation of rainfall, topographic data, and drainage area to estimate peak rates of runoff from areas under 1,000 acres.

Jarocki, Walenty. *Calculation of Bridge Outlets and Culverts (Obliczanie Otworów Mostów i Prizepustów)*. Wydawnictwa Komunikacyjne, Warsaw, 1955. English translation for U. S. Department of Interior OTS 60-21565, Office of Technical Service, U. S. Department of Commerce, 1961.

This is a treatise on various formulas and methods for the computation of the waterway areas of small bridges and culverts.

Jarvis, C. S., "General Formula for Waterways," *Engineering World*, Vol. 29 (August, 1926), pp. 79-83. [55]

Gives graphic representation of selected runoff formulas, including original and modified Myers formulas, and maximum runoff of representative streams.

Kirby, W. S., "Time of Concentration for Overland Flow," *Civil Engineering*, Vol. 29 (March, 1959), p. 60.

A nomograph for computing time of concentration as a function of length, slope, and surface retardance of flow of the given drainage area.

Kirpich, Z. P., "Time of Concentration of Small Agricultural Watersheds," *Civil Engineering*, Vol. 10, No. 6 (June, 1940), p. 362. [89]

Gives relation between time of concentration and two factors depending on length of travel and slope, which holds for areas of less than 200 acres which were studied; discussion by Langbein in No. 8 (1940), p. 533.

Krimgold, D. B., "Runoff from Small Drainage Basins," *Agricultural Engineering*, Vol. 19, No. 10 (October, 1938), pp. 439-446. [32]

Use of synchronized rain gages and water stage recorders to obtain runoff data from agricultural drainage basins.

Krimgold, D. B., and N. E. Minshall, "Hydrologic Design of Farm Ponds and Rates of Runoff for Design of Conservation Structures in the Claypan Prairies," *U. S. Department of Agriculture, Soil Conservation Service Research Report SCS-TP-56*, May, 1945. [33]

The principles involved in planning farm ponds are outlined and data on evaporation minus precipitation and runoff are presented with examples of how to use the data in the design of ponds.

Krimgold, D. B., "On the Hydrology of Culverts," *Proceedings, Highway Research Board*, Vol. 26 (1946), pp. 214-226. [17]

Describes the studies of Soil Conservation Service on the relation between rainfall and peak rates of runoff, emphasizing the failure of the rational method to conform to the data. Presents typical curves of ten-year flood peaks as a function of drainage area for specific physiographic provinces, soil types, and cover conditions. Urges that more complete flow data be collected using existing bridges and culverts as controls.

Krimgold, D. B., "Rates of Runoff from Small Drainage Basins," *Agricultural Engineering*, Vol. 28, No. 1 (1947), pp. 25-28.

Discussion of factors affecting the rate of runoff from small drainage basins.

Lillie, G. E., "Discharge from Catchment-Areas in India, as Affecting the Waterways of Bridges," *Proceedings, The Institution of Civil Engineers*, Vol. 217 (1923-24), pp. 295-332.

Proposes a formula for the discharge of peak flood. The formula contains factors of standard velocity, annual rainfall, length of the catchment above the discharging point, angle at the discharge point of the sectors into which the catchment is divided, etc. Comparison of various formulas is presented.

Luten, D. B., "A Comparison of Bridge Waterways," *Proceedings, The 26th Annual Meeting of the Indiana Engineering Society*, Vol. 29 (1909), discussions, pp. 53-56.

Describes the shape of bridge openings with respect to the efficiency of passing flood discharges. Arch opening is suggested as an efficient structure. Several examples are given.

Marsh, F. B., "Jarvis-Myers Formula Recommended for Bridge Waterway Areas," *Civil Engineering*, Vol. 21, No. 2 (1951), p. 47.

A discussion of the Jarvis-Myers empirical formula for design flood for highway and culvert design.

Mavis, F. T., "Reducing Unknowns in Small Culvert Design," *Engineering News Record,* Vol. 137, No. 2 (July 11, 1946), pp. 51-52. [43]

Development of rating chart to be used for determination of discharge through culvert; schematic analysis of rainfall and runoff records shows how time of concentration for drainage basin and intensity of rainfall can be found easily.

Merrell, J. C., "Hydrology for Highway Structures," *Proceedings of the Ohio Highway Engineering Conference, April 3-5, 1951,* (Engineering Experiment Station Bulletin No. 145). Columbus, Ohio: The Ohio State University, 1951. Pp. 64-77. [44]

Recommends aids in obtaining data and three design factors pertaining to hydrology which should be considered in designing a structure. Describes hydraulic conditions of channel changes and overflows which are encountered but often ignored in practical problems.

Miller, J. F., and J. L. H. Paulhus, "Rainfall-Runoff Relations for Small Basins," *Transactions, American Geophysical Union,* Vol. 38, No. 2 (1957), pp. 216-218.

Procedures for computing storm runoff from rainfall for large basins are not adaptable to small ones. A derivation of rainfall-runoff relation for basin of 4.1 sq. mi. and its applications are given.

Minshall, N. E., "Rates of Runoff for the Design of Conservation Structures in the Upper Mississippi Valley Upland Loessial Areas," *U. S. Department of Agriculture, Soil Conservation Service — Research, SCS-TP-73,* Washington 25, D. C., 1949, 30 pp.

Presents a brief history of the work and a detailed description of the drainage basins, instrumentation, and procedures employed in the collection and compilation of data from runoff studies near Fennimore, Wisconsin. Outlines briefly the methods of analysis and presents, in both tabular and graphical form, recommended rates of runoff, from small agricultural drainage basins, to be used in the design of conservation structures.

Minshall, N. E., "Predicting Storm Runoff on Small Experimental Watersheds," *Proceedings, American Society of Civil Engineers, Journal of Hydraulics Division,* Vol. 86, No. HY8, Pt. 1 (August, 1960), pp. 17-38. [61]

A method is presented for extending the period of runoff records based on analysis of existing short-term rainfall-runoff records for the watershed and a longer record of the rainfall alone. The method involves: (1) estimating runoff volumes from the rainfall pattern and antecedent rainfall, and (2) distributing this runoff by the unit hydrograph principle. A method is also presented for developing synthetic unit hydrographs for ungaged areas.

Munson, T. M., "Formulas and Methods in General Use by Engineers to Determine Runoff from Watershed Areas," *Texas Engineer,* Vol. 4, No. 4 (April, 1934), p. 7. [27]

Lists 36 formulas and 4 sets of curves in determining runoff and sizes of highway and railroad drainage structures.

Murphy, E. C., "Method of Computing Cross-Section Area of Waterways," *Water Supply and Irrigation, U. S. Geological Survey Paper No. 147,* 1905, pp. 182-193.

Describes factors determining maximum rate of discharge, data on maximum discharges in Northeastern United States, and formulas for maximum discharge and area of cross-section.

Ordon, C. J., "A Modified Rational Formula for Storm-Water Runoff," *Water Sewage Works,* Vol. 101 (1954), pp. 275-277.

Discusses the rational formula proposing new values of the runoff coefficient. Suggests an area factor to modify (reduce) computed flows for areas over 500 acres.

Pence, W. D., "The Best Method of Determining the Size of Waterways" (Appendix B of Report of Committee No. 1 — On Roadway), *Proceedings, American Railway Engineering and Maintenance of Way Association Bulletin 108,* Vol. 10, Pt. II (1909), pp. 967-1022. [23]

Describes the need for the best method and gives a comprehensive presentation of several methods in general use. Includes a digest of current practice and items covered by a circular of inquiry to 27 writers on the subject of waterway areas.

Pence, W. D., "Waterway for Culverts," *Proceedings, American Railway Engineering and Maintenance of Way Association,* Vol. 12, Pt. 3 (1911), pp. 481-528.

A brief historical account, compilation and comparison of formulas, permissible velocity, and other features.

Pettis, C. R., "A New Theory of River Flood Flow," Corps of Engineers, U. S. Army, Baltimore, Maryland, 1927.

Flood probability formula modified to improve it both practically and theoretically; rainfall index for the entire United States developed from runoff records; limits of applicability.

Potter, W. D., "Surface Runoff from Agricultural Watersheds," *Surface Drainage, Highway Research Board Report No. 11-B,* 1950, pp. 21-35. [19]

Describes unsatisfactory attempts to supplement short-time runoff records with long-time rainfall records for small watersheds by use of: (1) direct relationships between rainfall intensity and rates of runoff, (2) rational method, (3) unit hydrograph and (4) infiltration theory. Suggests the Gumbel method for use in probability studies of skewed distributions as those of rainfall and runoff data. The method of computing peak rates based on the probability curve is described. Peak rates summarized for five of seven physiographic areas are recommended for use in the design of conservation structures.

Potter, W. D., "Rainfall and Topographic Factors that Affect Runoff," *Transactions, American Geophysical Union,* Vol. 34 (1953), pp. 68-73. [51]

Describes a procedure in which easily determined rainfall and topographic factors are used to compensate for the lack of long-term runoff records and to increase the reliability of estimates of peak rates from small drainage basins in Allegheny-Cumberland Plateau area. Discussion by R. W. Powell, Vol. 34 (1953), p. 952.

Potter, W. D., "Use of Indices in Estimating Peak Rates of Runoff," *Public Roads,* Vol. 28, No. 1 (April, 1954), pp. 1-8. [52]

Presents results of a sample study for estimating peak rates of runoff in the Allegheny-Cumberland Plateau and the glaciated sandstone and shale areas of New York, Pennsylvania, and Ohio.

Potter, W. D., "Upper and Lower Frequency Curves for Peak Rates of Runoffs," *Transactions, American Geophysical Union,* Vol. 39, No. 1 (February, 1958), pp. 100-105.

Potter, W. D., "Peak Rates of Runoff from Small Watersheds," *Hydraulic Design Series No. 2,* U. S. Department of Commerce, Bureau of Public Roads, U. S. Government Printing Office, April, 1961. [62]

This investigation deals with drainage basins of 25 sq. mi. or less in size located east of the 105th meridian. First, a correlation analysis of the ungaged basin sample is established between a topographic index T, a precipitation index P, and the area of the basin A. The result is applied to the gaged watershed sample for developing satisfactory correlations between the peak discharge of 10-year frequency, Q_{10}, and the indices A, T and P. These correlations are used to correct the errors in Q_{10} for those gaged samples which do not show satisfactory correlations but discrepancies of varying degrees.

Powell, R. W., "Insurance Concept Balances Economic Factors in Culvert Design," *Civil Engineering,* Vol. 23 (1953), p. 64.

Presents an analytical solution to the problem of adequacy in culvert design, assuming that annual cost of capital recovery plus cost of insurance against damage is least for the correct design.

Ramser, C. E., "Runoff from Small Agricultural Areas," *Journal of Agricultural Research,* Vol. 34, No. 9 (May 1, 1927), pp. 797-823. [29]

Description of tests of 6 small drainage basins in Tennessee to determine runoff coefficient for various types of cover, soil, and topography.

Ramser, C. E., "Brief Instructions on Methods of Gully Control," U. S. Department of Agricultural Engineering (mimeographed), August, 1933, pp. 16, 21-23, 26-27. [28]

The so-called "Ramser Curves" are presented. These curves for 10- and 50-year expectancies were computed by the rational method with values of "*C*" and of "times of concentration" based largely on the results of measurements made in 1918 on 6 small drainage basins (1.25 to 112 acres) near Jackson, Tennessee.

Rowe, R. R., and R. L. Thomas, "Comparative Hydrology Pertinent to California Culvert Practice," *California Highways and Public Works,* Vol. 20 (September, 1942), pp. 6-11. [38]

Rainfall and flood frequency data are presented for California and compared with those for other areas. A new formula for design discharge is presented in form of nomograph.

Smith, W. E., "The Hydrologic Behavior of Agricultural Watersheds," *Transactions, American Geophysical Union*, 1944, pp. 1048-59.

Gives runoff data from several small drainage basins in Ohio and discusses the influences of type of cover.

Snyder, F. F., "Synthetic Flood Frequency," *Proceedings of the American Society of Civil Engineers, Journal of the Hydraulics Division*, Vol. 84, No. HY5, Pt. 1 (October, 1958), pp. 1-22. [59]

A procedure is developed for computing the flood discharge probability associated with a given rainfall-duration-frequency pattern on natural drainage basins, non-channelized overland flow areas and areas with storm sewer drainage utilizing basin runoff-producing characteristics of area, length, slope, friction, and shape.

Spindler, W. H., "How to Design and Install Culverts," *Public Works*, Vol. 87, No. 1 (1956), pp. 83-90.

Discusses waterway area determination by inspection, Talbot formula, and Jarvis-Myers formula; and also outlet conditions, alignment problems, lengths required, and installation procedures.

Springer, G. P., "Adequate Waterways for Culverts and Bridges," *Proceedings, 17th Annual Road School at Purdue University, January 19-23, 1931*, Purdue University Engineering Bulletin, Vol. 15, No. 2 (March, 1936). [41]

Describes highway drainage in general, capacity of side ditches, and its calculation, information for design office, determination of drainage opening, and study of culverts and small bridges.

Springer, G. P., "Waterways for Culverts and Bridges," *Canadian Engineering*, Vol. 60, No. 17 (1931), pp. 25, 61-62, 65.

Same as Springer's paper in *Proceedings, 17th Annual Road School at Purdue University, January 19-23, 1931*.

Sribnyi, M. F., "Method of Determining Maximum Flood Discharge from its Relation to the Area of the Watershed," *Russian Scientific Academy News*, (Izv. AN SSSR otd. teck nauk), No. 1 (1952), pp. 140-152.

Describes the brief history and the trend of the development of hydrology in USSR. At the present time, methods involving basin characteristics and climatological effects are used in place of the old Kochrin approach. Suggests an organization to study the problems of flood discharge on a large scale. A formula developed by the Russian Scientific Academy is presented.

Talbot, A. N., "The Determination of Water-Way for Bridges and Culverts," Selected Papers of the Civil Engineers' Club, *Technograph No. 2*, University of Illinois, 1887-8, pp. 14-22. [5]

Proposes a formula for the determination of waterway area for culverts and a method for the calculation of bridge openings. The formula is based on the Bürkli-Ziegler formula and on two assumptions that area of flow in the stream is directly proportional to the discharge, and that velocity in the culvert would be the same as in the stream above. For bridge waterways, the use of Chézy formula is suggested. It is stated that the judgment must be the main dependence, the formula being a guide to it.

Tasker, E., "Some Observations on the Carrying of Main Roads over Small Watercourses (Culverts, Bridges, etc.)," *Surveyor (Land)*, Vol. 73, No. 1876 (1928), pp. 15-18.

Discusses waterway areas of small culverts; types of structures; steel construction; concrete culverts; flat-slab reinforced concrete culverts; materials, parapets, approaches; tests.

Tholin, A. L., and C. J. Keifer, "Hydrology of Urban Runoff," *Proceedings, American Society of Civil Engineers, Journal of Sanitary Engineering Division Paper No. 1984*, Vol. 85 (1959), pp. 47-106.

A study of rainfall-runoff relationships in urban areas based upon a design storm on different types of uniform land use is presented. Design graphs and the effects of variations in areal distribution of rainfall are given.

Tilton, G. A., Jr., and R. R. Rowe, "Culvert Design in California," *Proceedings, Highway Research Board, 1943*, Vol. 23 (1944), pp. 165-206. [63]

Reviews hydrologic and hydraulic principles used in design of California culverts. Flood frequency in California is compared with eastern states and nomograph for determination of design discharge is presented.

Turner, Bill, "Graph for Talbot's Formula," *Engineering News Record*, Vol. 126, No. 25 (June 19, 1941), p. 971. [9]

Presentation of logarithmic charts giving area of opening by Talbot's formula for drainage basins of from 1 to 10,000 acres in flat, rolling, hilly, and mountainous country.

Waddell, J. A. L. "Determination of Waterways," *Bridge Engineering* (Vol. 2). New York: John Wiley and Sons, Inc., 1925. Pp. 1109-1136.

Describes data useful in determining areas of waterways, formulas and examples for computing peak discharges.

Walker, Thomas, "Flood Discharge Formulae," *American Railway Engineering Association Bulletin,* Vol. 24, No. 248 (1922), pp. 23-26.

Presents a formula for estimating flood discharges.

"Waterway Design Specifications for Highway Structures," State of Ohio, Department of Highways, 1951, pp. 9-12.

Outlines the general procedure and requirements of waterway design. Includes the determination of waterway area, ordinary flood and extreme floods, legal and navigational requirements for channel opening, ice floes, debris, drift and impinging current, pier spacing and location, and channel change.

Weeks, D. P., "Application of the Law of Variables to the Design of Drainage Outlets," *Agricultural Engineering,* Vol. 2, No. 4 (1921), pp. 81-84.

Summary of present methods of selecting runoff factor for drainage of lands in humid sections and proposed plan for promoting investigation.

Wellington, A. M., "Culvert Proportions," Editorial, *Railroad Gazette,* Vol. 18 (September, 1886), pp. 629-630. [2]

Discusses use of empirical formula, such as the Myers formula, for proportioning culvert openings.

Whinery, S., "Determining the Size of Railroad Culverts," *Railway Gazette,* 1902, p. 976.

Discusses the elements that must be considered in the computation, and method of arriving at the proper capacity.

Wickline, G. G., "Design of Drainage Structures and Bridges," *Good Roads,* Vol. 58, No. 18 (1920), pp. 231-232, 240.

Talbot's formula is recommended for the waterway area of culverts, but several factors must be considered in selection of the coefficient.

Yarnell, D. L., "Determining Flood Discharges from Small Watersheds," *Agricultural Engineering,* Vol. 18, No. 1 (1937), pp. 13-14.

Drawing flood frequency curve from meager runoff data by plotting on log paper parallel to rainfall-intensity-frequency curve.

Yule, R. B., "Bridge Waterway Area Formula Developed for Indiana," *Civil Engineering,* Vol. 20, No. 10 (1950), p. 26.

Development of a diagram for adequate waterway area under bridges by plotting data of waterway area against drainage area in logarithmic scales.