PROGRESS REPORT NO. 1

STUDY OF RUNOFF
FROM SMALL WATERSHEDS
FOR HIGHWAY DRAINAGE DESIGN
IN INDIANA

MAY 1961

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Research
Project

PURDUE UNIVERSITY
LAFAYETTE INDIANA

by

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Progress Report No. 1.

STUDY OF RUNOFF FROM SMALL WATERSHEDS
FOR HIGHWAY DRAINAGE DESIGN IN INDIANA

TO: K. B. Woods, Director
Joint Highway Research Project

May 18, 1961

FROM: H. L. Michael, Assistant Director
Joint Highway Research Project

File: 9-8-1
Project: C-36-62A

Attached is Progress Report No. 1 on "Study of Runoff from Small Watersheds for Highway Drainage Design in Indiana". The report has been prepared by I. P. Wu and J. W. Delleur of our staff.

The report includes a statistical analysis of existing data on small watersheds and a geomorphological study of selected watersheds.

The report is presented for the record.

Respectfully submitted,

Harold L. Michael
Harold L. Michael
Secretary

HLM:kmc

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Progress Report No. 1

Study of Runoff from Small Watersheds
for Highway Drainage Design
in Indiana

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File No. 9-8-1

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May, 1961

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Abstract

The purpose of this research is to study the hydrology of watersheds less than 200 square miles throughout the State of Indiana and to propose methods of estimating the runoff from these watersheds. The study will include four parts: a statistical analysis of existing data, a geomorphological study of selected watersheds, a study of small watershed hydrographs, and an analysis of overland flow. The present report summarizes the work done from September, 1959, to date on the first two parts.

The statistical analysis has been made by means of extreme values method. The 25 year-instantaneous peak flow has been determined for all the gaging stations with a drainage of less than 200 square miles for which the U.S.G.S. publishes records. Among these watersheds, for all of which sufficient topographic data were available, five geomorphological parameters were calculated: the watershed area, the mean relief, the main stream slope, the stream density, and the shape factor. A multiple correlation was made between these variables and the instantaneous 25 year peak flow. A chart is given to obtain graphically the peak flow knowing the five geomorphological parameters.

I. INTRODUCTION

In the design of hydraulic structures or in the study of many hydraulic projects, a great and difficult problem to the designing engineer is to estimate accurately the amount of runoff that can be expected. In the case of highway drainage structures, the designer is interested, mainly, in the maximum runoff resulting from a storm. The ideal highway drainage structure should have enough capacity to drain the flood runoff safely without causing any damage to the structure itself or to the surrounding areas.

The determination of the required waterway area of a bridge or the selection of the size of a culvert are problems which require an accurate estimate of the flood discharge that will pass through the structure. The determination of this discharge is more difficult for small watersheds because the majority of them are ungaged. There are very few gaged small watersheds on which to base an estimate. Particularly in the state of Indiana, there is very little information on watersheds less than 200 square miles. There are only twelve watersheds of less than 100 square miles, and seventeen watersheds with an area between 100 and 200 square miles in the state of Indiana for which the U.S.G.S. is currently reporting flows. The purpose of this research is to obtain reliable methods for estimating the runoff for a safe and economic design of highway drainage structures, serving watersheds of less than 200 square miles.

The existing methods are empirical and fail to take into account the factors upon which the runoff depends. Runoff is the water flowing on the ground, the amount of which varies greatly with the storm characteristics such as the storm duration, intensity, distribution; and watershed characteristics such as area, slope, shape, soil physical properties, and channel geometry.

Unfortunately, due to the lack of data, most of these variables are difficult to evaluate. The designers have therefore used some empirical formulas for estimating runoff. The most famous is Talbot's formula, which estimates the waterway area from the watershed area. The formula is:

$$A = c M^{\frac{3}{4}}$$

Where: A = waterway area in square feet

M = area of watershed in acres

c = a coefficient varying between 1/5 and 1 depending on the slope and character of the watershed. The selection of the coefficient depends, among other things, on the experience of the designer.

Due to the various factors that affect the runoff other than the watershed area, the value of the coefficient c cannot be accurately determined to represent all the watershed characteristics. Talbot's formula is unsatisfactory for a safe design of a hydraulic structure. Therefore, it seems desirable to develop more accurate methods of computation that will include most of the factors that can affect the runoff.

In this study the variables that affect the runoff are evaluated; and improved methods are proposed for determining the runoff from small, ungaged watersheds, within the state of Indiana for highway drainage design. For the purpose of this report, small watersheds are defined as those of less than 200 square miles, that is those for which there is very little information in the state of Indiana, and larger than 1,000 acres so that the land use is not a predominant factor.

II. METHODS OF APPROACH

Four methods of approach will be considered:

1. Statistical analysis
2. Geomorphological analysis
3. Hydrograph study
4. Overland flow hydraulics

In the statistical analysis runoff records are examined, and the theoretical instantaneous 25-year flow is calculated by means of Gumbel's extreme value theory for thirty-two watersheds for which records are available. This is based on the idea that the best way of estimating design discharges is from discharge records themselves. This has the advantage of eliminating the need of relating rainfall and runoff by means of a number of variables difficult to evaluate. The use of 25-year frequency is taken as a reference frequency which approximates the expected life of highway drainage structures, (1).

A study is made to evaluate the geomorphological factors that can be related to the stream flow in order to estimate the runoff from ungaged watersheds. If the watersheds are not too far from one another and the storm patterns are assumed constant over such areas, there must be a definite relationship between the geomorphological factors of watersheds and the stream flow. Multiple correlations are used to find the relations between the geomorphological characteristics and the 25-year maximum instantaneous flow. The assumptions made in the above approach are not absolutely true because the storm patterns are not constant and uniform over the gaged and ungaged watersheds. The error will depend on the deviation of storm patterns among the several watersheds.

The hydrograph analysis includes the evaluation of the storm runoff hydrograph from the several watersheds and checking the assumptions made in the geomorphological study. The infiltration and detention losses of water before the runoff process occurs will be evaluated. Unit hydrographs and instantaneous hydrographs will be obtained from which synthetic hydrographs for ungaged watersheds will be established.

The last approach is the study of overland flow hydraulics which is based on the hydrodynamics of sheet flow with lateral inflow. The differential equations for this kind of flow resulting from rainfall on smooth plane surface were derived from the general equations of motion and continuity. With the appropriate initial and boundary conditions, they can be solved mathematically as a propagation problem by the methods of characteristics. The resulting overland flow hydrograph at any particular point on the plane can then be obtained. It is hoped that with certain simplifying assumptions that this kind of approach can be used to estimate runoff from a small watershed for highway drainage design.

The present progress report is concerned with the work done up to date which includes the first two approaches.

III. REVIEW OF LITERATURE

The literature review of the statistical and geomorphological study of runoff determination can be divided as three parts:

- (1) Statistical theory of flood frequency analysis--flood forecasting
- (2) Geomorphological factors which effect the amount of runoff
- (3) Empirical formulas for estimating runoff--relating to the basin characteristics

1. Statistical Theory of Flood Frequency Analysis

In 1940, Geyer (2) derived a mathematical expression relating the flood magnitude to the exceedance interval. The expression derived is as follows:

$$Y = LR^{-t^K}$$

Where: Y = flood magnitude in sec-ft. that has an exceedance interval of (t) .

t = exceedance-interval in years

L, R, K = constants for any stream

Expressing the above formula in the logarithm's form:

$$\log Y = \log L - t^K \log R$$

$$\log L - \log Y = t^K \log R$$

$$\log(\log L - \log Y) = K \log t + \log \log R$$

It is of the form $Y = mx + b$, which is the equation of a straight line when $\log(\log L - \log Y)$ is plotted against $\log t$. If the appropriate value for L is assigned, then the plot of $\log(\log L - \log Y)$ versus $\log t$ should result in a straight line. However, the value of L has to be assigned by trial and error procedure.

In 1941, Gumbel (3, 4, 5, 6, 7, 8, 9, 10) developed the extreme value theory. A special probability paper was designed for flood frequency analysis. According to Gumbel, a theoretical straight line can be plotted on the special probability paper with the observed data and can be used for future flood forecasting. This will be discussed in detail under the section, "Extreme Value Theory".

In 1950, Benson (11) made use of historical data in flood-frequency analysis. The study was made on the basis of annual peak discharges, and data were plotted on probability paper based on Gumbel's extreme value theory.

In 1951, Chow (12) derived a general formula for hydrological frequency analysis. It was presented to simplify many existing methods of hydrological frequency analysis. The frequency factor K depends upon the law of occurrence of a particular hydrological event under consideration.

If

$$\begin{aligned}\Delta Y &= \sigma K \\ Y &= \bar{Y} + \Delta Y \\ Y &= \bar{Y} + \sigma K \\ \frac{Y}{\bar{Y}} &= 1 + \frac{\sigma}{\bar{Y}} K \\ &= 1 + C_v K\end{aligned}$$

where

ΔY = deviation from mean

σ = standard deviation

Y = magnitude of hydrological data

Then the general formula is $\frac{Y}{\bar{Y}} = 1 + C_v K$ where K is related to recurrence interval T .

In 1957, Whisler and Smith (13) derived a method of estimating the frequency of rare floods. The drawbacks of using the record of annual

peak flows which may result in errors of large magnitude in the determination of the recurrence interval of the rarest floods were pointed out. This makes the use of statistical methods for such studies completely inapplicable. Therefore, the technique of using actual monthly peak flows as the basis for determining the distribution of annual peak flows was introduced.

In the same year Rowe et'al (14) made flood frequency analysis by regional synthesis for determining 30--100 years flood for highway design in California. The procedure recommended was as follows:

1. Define a region of homogeneity.
2. From this region, select gaging stations likely to combine with un-biased analogy.
3. Construct a frequency graph for each station.
4. Determine the five-years flood for each station.
5. For each station, express all floods as multiples of the 5-years flood.

$$Q_i = R_i Q_5$$

6. Assemble the several sets of R in one synthetic set and arrange in descending order of magnitude.
7. Construct a frequency graph for this synthetic record as though it were observed in a period of N year where N is the sum of the periods of record of the combined stations.
8. Determine visually the best-fit line for the well-defined part of the graph.
9. Transfer the slope of this line to each of the separate station graphs.

2. Geomorphological Factors

In 1947, Langbein et al (15) indicated that the river floods were the results of many causes. One of the primary objectives of scientific hydrology is the segregation and evaluation of the causative factors. The climatic factor and the soil-vegetation complex are variables that exercise their principal influence on the volume of the runoff. The topography of drainage basins is a sensibly permanent characteristic which influences mainly the concentration or time distribution of the discharge from a drainage basin. The topographic characteristics of drainage basin cited were: area of basin, drainage density, area-distance distribution, length of basin, land slope, area-altitude distribution, and area of water surfaces.

In 1952, Strahler (16) expressed the area-altitude relation by a hypsometric analysis. The hypsometric curve relates horizontal cross-section area of a drainage basin to relative elevation above basin mouth. By the use of dimensionless parameters, the curves can be described and compared irrespective of true scale. The area under the curve can be used to find total land mass and the mean relief of watersheds.

In 1957, Strahler (17) made a quantitative analysis of watershed geomorphology and showed that the linear scale measurements include length of stream channels of given order, drainage density and relief. Surface and cross-sectional areas of basins are length products. Dimensionless properties include stream order numbers, stream length and bifurcation ratio, ~~maximum~~ valley-side slopes, mean slopes of watershed surfaces, channel gradients, relief ratio and hypsometric curve properties.

In 1959, Benson (18) cited the following basin characteristics: drainage area, channel slope, land slope, tributary channel slope, watershed shape factor, mean elevation, percentage of lakes, swamps and reservoirs, and drainage density. The shape factor of the watershed was represented by:

L^2/A , equivalent of basin length divided by basin width.

L/A , basin length divided by drainage area.

$\sum ab$, the summation of small subdivisions of the drainage area, each multiplied by the distance of travel to the gaging point.

3. Empirical Formulas for Estimating Runoff--Relating to the Basin Characteristics

In 1946, Kinnison (19) compiled a list of all the flood formulas which included drainage-basin characteristics; there are the following:

(a) Formula for extreme or high flood

$$Q = C M^n$$

Where: Q = discharge in cfs.

C = constant

M = area in sq. mi.

$n = 0.5 \sim 0.8$

There are many such formulas listed in engineering handbooks. However, because of their limited significance and questionable value, they are not enumerated.

(b) Numerous formulas for floods if an implied frequency were proposed, such as: the modified Myer formula, the Kuichling, Craig, and Burge formula.

(1) Modified Myer formula

$$Q = 1000 P \sqrt{M}$$

Where: Q = peak discharge in cfs.

P = Myer percentage rating

M = drainage area in sq. miles.

(ii) Kuichling formula

$$Q = \frac{44000}{M+170} + 20 \quad \text{-----frequent flood}$$

$$Q = \frac{127000}{M+370} + 7.4 \quad \text{-----rare flood}$$

Where: Q = discharge in sec-ft/sq.mi.

M = drainage area in sq. miles

(iii) Craig formula

$$Q = 440 n B \left(\text{hyperbolic log } \frac{8L^2}{B} \right)$$

Where: Q = peak discharge cfs

L = length of drainage basin in miles

B = average width of basin in miles

n = a coefficient ranging from less than one to more

than two depending on the rainfall and topography

This formula, published in 1868, was an early effort on the part of engineers to introduce the physical characteristic of drainage basin into the flood formula.

(iv) Burge formula

$$Q = 1300 \frac{M}{L^{1/2}}$$

Where: Q = peak discharge in cfs

M = area in sq. miles

L = length of basin in miles

(c) Formulas for culverts and storm drains

(i) Burhli-Ziegler formula

$$Q = RMC \sqrt[3]{SM}$$

Where: Q = discharge in cfs
 R = max. rate of rainfall in in./hr.
 S = average ground slope in 1'/1000'
 C = a coefficient varying from 0.25 - 0.75
 depending on the type of land being
 drained

(ii) Talbot formula

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

Where: A = the area of water way in sq. feet
 C = a factor depending on the topography and varies
 from 1/5 to 1.
 M = area in acres

(iii) Manning formula

$$A = 0.23 \sqrt{M^5}$$

Where: A = area of water way in sq. ft
 M = area in acres

(d) Formulas including rainfall

(1) Grunsky formula

$$Q = \frac{3200 a R}{t_c^2} \quad \text{WHERE} \quad a = \frac{600}{60 + C^3 \sqrt{t_c}}$$

Q = discharge in cms

t_c = the critical time in minutes during the continuance
 of a rainstorm for the area under consideration
 within which the rain will produce the max. rate
 of runoff.

R = max. rainfall for one hour

C = a coefficient varies from 0.1 to 2.0

This formula was published in 1922 and is applicable to California streams.

(ii) Lillie formula

$$Q = V A \lambda \Sigma (OL)$$

Where: Q = peak discharge

V = standard mean velocity of river in flood

$$R = 2 + \frac{\text{annual rainfall}}{15}$$

λ = a factor based on the length of basin and is proportional to the width of the basin.

(iii) Pettis formula

$$Q = C P W^{1.25}$$

Where: Q = peak flood in cfs

P = rainfall coefficient

W = average width of basin in miles

C = a coefficient of drainage basin characteristics

(iv) Bresson formula

$$Q_m = P_m T G_m A^x$$

Where: Q_m = max. flood

P_m = max. precipitation during flood period

G_m = factor representing influence of ground condition

T = factor representing topograph

A = drainage area

x = an exponent

(v) Switzer and Miller formula

$$Q = P_c C W^n$$

Where: Q = 24 hr. average flood cfs.

P_c = rainfall factor

W = width of basin in miles

C and n = empirical constant

(vi) Formula including total runoff

American Society of Civil Engineers' Formula

$$Q = C_f R \sqrt{A}$$

Where: Q = peak flow cfs.

C_f = coefficient based on drainage basin characteristics

R = total flood runoff in inch

A = drainage area in sq. mi.

(vii) Formulas with frequency relation expressed

(1) H. E. Miller (1914) formula:

$$Q_{ave} = C A^{.65}$$

$$Q = Q_{ave} (1 + 0.6 \log_{10} T)$$

$$Q_{max} = Q (1 + 2A^{-.23})$$

Where: Q_{ave} = average of the annual 24-hr. floods in cfs.

Q = max. 1 day flood in cfs.

Q_{max} = peak discharge in cfs.

A = drainage area in sq. miles

T = number of years in period considered

C = coefficient constant

(ii) R. E. Horton formula (1914)

$$Q = 4021.5 \left(\frac{T^{0.25}}{A} \right)$$

Where: Q = flood "equalled or exceeded" in the average interval of T years

A = drainage area in sq. miles

(iii) E. W. Lane (1926) formula

$$Q = K(\log I + B)$$

Where: I = recurrence interval

K and B = constant for the station in question

(iv) W. P. Creager formula

$$Q = CA^{0.5} \left[\frac{2 - C^{0.4} A^{0.5}}{3} \left(1 - \frac{\log 0.1 T}{3} \right) + \frac{\log 0.1 T}{3} \right]$$

T = frequency in years

C = coefficient depending on characteristics

(g) Flood formula including basin characteristics

$$Q = f(S, M, L)$$

for max. floods

$$Q = (0.123 S^{1.6} + 1500) \frac{M^{0.7}}{L^{0.7}}$$

Where: Q = flood in cfs.

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

M = drainage area in sq. mi.

L = average distance in miles which water from runoff over the basin must travel to the outlet.

In 1950, Yule (20) developed a formula for the waterway area of bridges to be used in Indiana, which expressed the waterway area as a function of the $2/3$ power of drainage area. It is:

$$A = Ca^{2/3}$$

Where: A = waterway area in square feet

a = drainage area in square miles

C = a general coefficient which varies with the watershed topography

C = 0.3 to 0.7--flat land

C = 0.7 to 1.3--rolling land

C = 1.3 to 2.2--hilly land

In 1955, Bigwood and Thomas (21) developed a flood-flow formula for Connecticut. The method of analysis was based on Gumbel's extreme value theory to develop a state-wide flood frequency relation; then, the effects due to the important physical characteristics of each drainage basin were evaluated. The formula has the following form:

$$Q = R(C_B A_s)$$

Where: R = design ratio from the frequency curve

$C_B A_s$ = mean annual flood based on the watershed characteristics

In 1957, Dooge (22) discussed the rational method for determining the size of trunk sewers, surface drainage channels, highway culverts, and bridge waterways. The essential feature of the rational method is that the runoff is calculated for a storm, the duration of which is equal to the time of concentration of the watershed. The time of concentration is that time taken for the first drops of rain falling on the most remote part of the

watershed to travel to the outlet. The rational formula has the form

$$Q = CIA$$

Where: Q = design peak runoff rate in cfs.

A = area of watershed in acres

I = the rainfall intensity in inches per hour for the design recurrence interval and for a duration equal to the time of concentration of the watershed

C = runoff factor

This formula was derived originally by Thomas Mulvaney in 1851 for small watersheds less than 5 square miles.

In 1959, Benson (18) found that the mainstream slope is the next in importance to drainage area size which affects the runoff. The slope for that part of the mainstream between 85 and 10 percent of the total distance above the gaging point provides the best correlation with flood magnitudes. The following empirical formula was found for the New England region:

$$Q = a A^b S^c$$

Where: Q = peak discharge in cfs.

A = drainage area in square miles

S = 85-10% slope of mainstream

a, b, c = regression coefficient varying with recurrence interval of flood

IV. STATISTICAL ANALYSIS

Gumbel's Extreme Value Statistics

The aim of a statistical theory of extreme value is to analyze observed extremes and to forecast further extremes. The flood problem is a problem of extreme. However, no matter how big a flood gets, there will always be a bigger one to come.

A Extreme value theory

1. The return period and the "expected" extremes value

Let $f(x)$ = initial probability density function of original data

$F(x)$ = cumulative frequency distribution function, that is, the probability that the value of the variate is less than x

n = number of observations

U_n = a certain expected largest value

Since $1-F(x)$ is the probability that the value of the variate is equal or larger than x , then its reciprocal

$$T(x) = \frac{1}{1-F(x)} \quad 4-(1)$$

is called the return period. It is the number of observations such that, on the average, there is one observation equalling or exceeding x . From given $f(x)$ and hence $F(x)$, then knowing n , we can find U_n such that the average return period is n . Its probability $F(U_n)$ is given by

$$\begin{aligned} T(x) &= \frac{1}{1-F(x)} \\ n &= \frac{1}{1-F(U_n)} \\ 1-F(U_n) &= \frac{1}{n} \\ F(U_n) &= 1 - \frac{1}{n} \end{aligned} \quad 4-(2)$$

2. Exact distribution of extremes

Since $F(x)$ is the probability that the value of the variate is less than a certain x , then the probability that n independent observations all fall short of x evidently is $F^n(x)$. This is the probability for x to be the largest among n independent observations. In other words, the probability $\Phi_n(x_n)$ that the largest value fall short of x_n is

$$\Phi_n(x_n) = F^n(x_n) \quad 4-(3)$$

and the derivative

$$\begin{aligned} \phi_n(x_n) &= n F^{n-1}(x_n) F'(x_n) \\ &= n F^{n-1}(x_n) f(x_n) \end{aligned} \quad \text{--- 4-(4)}$$

where $f(x_n) = F'(x_n)$ the initial frequency distribution is the distribution of the largest among n independent observations.

This formula constitutes the basis of the whole exact theory of extreme values; all conclusions are ultimately based on it. If the initial distribution $f(x)$ is known, then the distribution of the extreme values can be calculated.

3. Extremal parameters

By recalling the expected extremes U_n defined as equation 4-(2) and introducing $\mathcal{Q}_n$ which is defined by the equation

$$\mathcal{Q}_n = n f(u_n) \quad \text{4-(5)}$$

we can write

$$F(u_n) = 1 - \frac{1}{n}$$

$$\frac{1}{n} = 1 - F(u_n)$$

$$n = \frac{1}{1 - F(u_n)}$$

$$\mathcal{Q}_n = \frac{f(u_n)}{1 - F(u_n)} \quad \text{or}$$

$$\mathcal{Q}_n du_n = \frac{f(u_n) du_n}{1 - F(u_n)} \quad \text{4-(6)}$$

The reason for introducing the quantity $\mathcal{Q}_n$ is to see how the expected largest value U_n increases with n . Differentiation of equation 4-(2) with

respect to n leads to

$$f(u_n) du_n = \frac{1}{n^2} dn$$

$$f(u_n) = \frac{1}{n^2}$$

$$\text{Since } Q_n = n f(u_n) \quad \text{or} \quad n = \frac{Q_n}{f(u_n)}$$

$$f(u_n) du_n = \frac{dn}{n} \frac{f(u_n)}{Q_n}$$

$$\frac{du_n}{d \log n} = \frac{1}{Q_n} \quad 4-(7)$$

Thus $\frac{1}{Q_n}$ measures the increase of the expected largest value with the logarithm of sample size.

4. Asymptotic solution

Considering varieties which are unlimited, the probability function $F(x)$ converges with increasing x toward unity at least as quickly as an exponential type. In order to obtain the asymptotic distribution of the largest value for the exponential type, we expand the initial probability function about the expected largest value U_n by Taylor's expansion.

$$F(x) = F(u_n) + \frac{x-u_n}{1} f(u_n) + \frac{(x-u_n)^2}{2!} f'(u_n) + \dots$$

$$\dots + \frac{(x-u_n)^v}{v!} f^{(v)}(u_n) + \dots$$

$$\text{Since} \quad F(u_n) = 1 - \frac{1}{n}$$

$$F(x) = 1 - \frac{1}{n} + \frac{1}{n} (x-u_n) [n f(u_n)] + \frac{1}{n} \frac{(x-u_n)^2}{2!} [n f'(u_n)] + \dots$$

$$\dots + \frac{1}{n} \frac{(x-u_n)^v}{v!} [n f^{(v)}(u_n)] + \dots$$

$$F(x) = 1 - \frac{1}{n} \left(1 - \sum_{i=1}^v \frac{(x-u_n)^i}{i!} n f^{(i)}(u_n) \right) \quad 4-(8)$$

By using L'Hopital's rule for large positive values of the variate x , the probability density of $f(x)$ and probabilities $1 - F(x)$ are small; the derivative $f'(x)$ are also small and negative. Then, since $f(x) = F'(x)$

$$\lim_{x \rightarrow \infty} \frac{f(x)}{1 - F(x)} = - \lim_{x \rightarrow \infty} \frac{f'(x)}{f(x)}$$

become indeterminate for large values

$$\text{Then, } \frac{f(x)}{1 - F(x)} = - \frac{f'(x)}{f(x)} \quad 4-(9)$$

will be approximately valid for very large values of the variate, since

$x = U_n$. The equation 4-(9) can be written

$$f'(U_n) = - \frac{f^2(U_n)}{1 - F(U_n)} \quad 4-(10)$$

from equation 4-(5), and equation 4-(2)

$$Q_n = n f(U_n) = \frac{f(U_n)}{1 - F(U_n)} \quad 4-(11)$$

using the relation in equation 4-(10)

$$Q_n = - \frac{f'(U_n)}{f(U_n)} \quad 4-(12)$$

Multiplying the last two equations:

$$n f'(U_n) = - Q_n^2 \quad 4-(13)$$

Taking the derivative of equation 4-(10), we have

$$f''(U_n) = \frac{-[1 - F(U_n)] \cdot 2 f(U_n) f'(U_n) + f^2(U_n) [-f'(U_n)]}{[1 - F(U_n)]^2}$$

$$f''(U_n) = \frac{2 f(U_n) f'(U_n)}{-[1 - F(U_n)] \cdot f(U_n)} - \frac{f^2(U_n)}{[1 - F(U_n)]^2}$$

$$= 2 Q_n^2 - Q_n^2 = Q_n^2$$

by 4-(11) and 4-(12).

$$n f''(u_n) = 2_n^3 \quad 4-(14)$$

The relations 4-(13) and 4-(14) may be generalized:

if

$$n f^{(v)}(u_n) = (-1)^v 2_n^{(v+1)} \quad 4-(15)$$

holds for a certain v , it also holds for $v + 1$.

For the proof, we start from

$$f^{(v)}(u_n) = (-1)^v 2_n^v \cdot \frac{1}{n} 2_n = (-1)^v 2_n^v f(u_n) = (-1)^v \frac{f^{(v+1)}(u_n)}{[1 - F(u_n)]^v}$$

For the next derivative, we have

$$f^{(v+1)}(u_n) = \frac{(-1)^v [1 + F(u_n)]^v \cdot (v+1) f^{(v)}(u_n) f'(u_n) - f^{(v+1)}(u_n) \cdot v [1 - F(u_n)]^{v-1} [-f(u_n)]}{[1 - F(u_n)]^{2v}}$$

$$\begin{aligned} \frac{f^{(v+1)}(u_n)}{f(u_n)} &= \frac{(-1)^v (v+1) f^{(v)}(u_n) f'(u_n)}{[1 - F(u_n)]^v f(u_n)} + \frac{(-1)^v f^{(v+1)}(u_n) \cdot v \cdot f(u_n)}{[1 - F(u_n)]^{v+1} f(u_n)} \\ &= (v+1)(-1)^{v+1} 2_n^{(v+1)} - (-1)^{v+1} \cdot v \cdot 2_n^{(v+1)} \\ &= (-1)^{v+1} 2_n^{(v+1)} \\ n f^{(v+1)}(u_n) &= (-1)^{v+1} 2_n^{(v+2)} \end{aligned} \quad 4-(16)$$

which proves the statement. If we write $v - 1$ instead of $v + 1$ and introduce

$$n f^{(v-1)}(u_n) = (-1)^{v-1} 2_n^v \quad 4-(17)$$

into equation 4-(8), the expansion of the initial probability $F(x)$

$$F(x) = 1 - \frac{1}{n} \left[1 - \frac{2}{1} \frac{(x - u_n)^v}{v!} [(-1)^{v-1} 2_n^v] \right]$$

We introduce the reduced largest value

$$Y = \alpha_n (X - \alpha_n) \quad 4-(18)$$

Then,

$$\begin{aligned} F(x) &= 1 - \frac{1}{n} \left[1 - \sum_{i=1}^{\infty} \frac{Y^i}{i!} (-1)^{i-1} \right] \\ &= 1 - \frac{1}{n} e^{-Y} \end{aligned} \quad 4-(19)$$

This approximation holds for any initial probability function of the exponential type for large x .

The initial distribution $f(x)$

$$f(x) = \frac{1}{n} \alpha_n e^{-Y} \quad 4-(20)$$

equations 4-(19) and 4-(20) lead immediately to the asymptotic probability.

$$\begin{aligned} \Phi(x) &= F^n(x) \\ &= \left(1 - \frac{e^{-Y}}{n} \right)^n \\ &= 1 - n \frac{e^{-Y}}{n} + \frac{n(n-1)}{2!} \left(\frac{e^{-Y}}{n} \right)^2 \end{aligned}$$

as $n \rightarrow \infty$

$$\Phi(x) = e^{-e^{-Y}} = \exp(-e^{-Y}) \quad 4-(21)$$

$$\begin{aligned} \phi(x) &= \frac{d\Phi(x)}{dx} \\ &= e^{-e^{-Y}} (-e^{-Y}) (-\alpha_n) \\ &= \alpha_n e^{-Y - e^{-Y}} \end{aligned} \quad 4-(22)$$

The probability $\Phi(x)$ and the distribution $\phi(x)$ as function of y . The inverse function $Y = -\rho_n(-\ln \Phi(x))$ and the distribution

$\phi(x) = -\Phi(x) \ln \Phi(x)$ as a function of $\Phi(x)$ have been tabulated by the National Bureau of Standards. (9) These tables are indispensable

for all uses of extreme values.

5. Probability paper

The probability paper is used to forecast the most probable largest values to be obtained within a certain time and gives limits within which these values may be expected to lie with a certain probability. It is based on extreme value theory and the derived formulas:

$$\bar{F}(x) = e^{-e^{-y}}$$

$$y = \alpha_n(x - u_n)$$

A probability paper designed for extreme value was proposed by Power.

The observed variate x is traced on the ordinate, the reduced largest value (y) is traced on the abscissa, both in linear scales, the value of the probabilities $\bar{F}(x) = (e^{-e^{-y}})$ and the return period $T = \left(\frac{1}{1 - \bar{F}(x)}\right)$ are laid off on auxiliary horizontal scales. These, of course, are not linear scales.

From the equation relating the observed variate x to the reduced variate y , $x = \frac{1}{\alpha_n} y + u_n$, it appears that there exists a linear relationship between x and y . Therefore, theoretically, the plot of x vs. y should be a straight line on probability paper, where $\frac{1}{\alpha_n}$ is the slope of the line and u_n is the intercept.

B. Application of Gumbel's extreme value theory (9, 10)

Gumbel's extreme value theory shows clearly that there exists a linear relationship between the reduced largest value (y) and the observed extremes (x) on the special probability paper. By extending this linear relationship, it is possible to predict the future flood with any desired frequency. This will hold as long as the runoff is kept below that resulting from the "maximum probable storm" as determined from hydrometeorological studies.

In applying Gumbel's extreme value theory to flood frequency analysis, the following steps are followed:

1. Collecting the past flood data

To collect the past flood data from a watershed and select the instantaneous annual peak flood as the extreme value of each year, list them in increasing order according to the magnitude of each flood; the smallest flood will be ranked as 1 and the largest one will be ranked as n , which is the total number of floods.

2. Plotting position

The plotting position on the probability paper is defined by the two coordinates: the discharge x and the probability of occurrence $\bar{\Phi}$ of a maximum.

Gumbel's formula for plotting position is given as follows:

$$\bar{\Phi} = \frac{m}{n+1} \quad 4-(23)$$

where $\bar{\Phi}$ = probability to be maximum

n = number of observations

m = rank of observation in increasing order ($m = 1$,
the smallest one)

This seems to be a suitable and convenient formula for plotting position, because:

- i. All observations can be plotted.
- ii. It has an intuitive meaning and does not involve complicated calculations.
- iii. It is analytically simple and general.
- iv. The return period of the largest value is simply $n + 1$ instead of n ; the difference is insignificant.

3. Calculating the reduced value Y

From Gumbel's extreme value, it can be shown that the relation between x and Y is

$$Y = -\log_e (-\log_e \Phi)$$

Since Y is plotted on a linear scale, it is convenient for plotting a straight line fitting by the least square method.

A complete table listing Φ and its corresponding Y was given in Table 2, p. 19 of "Probability Tables for the Analysis of Extreme-Value Data." (9)

4. Fitting straight line

After having all n observations plotted on the probability paper, the next step is to fit the best straight line through the plotted points. The probability paper is so constructed that x and Y are both linear scales. The statistical method of least square may be applied to get a theoretically fitting straight line with the least deviations.

Since the reduced value Y and the observations x have the relation $Y = Q(x-u)$, which was shown in Gumbel's extreme value theory, it can be rewritten as $X = u + \frac{1}{\alpha} Y$, which is the equation of straight line, and the u

and $\frac{1}{\alpha}$ are two constants which can be determined by the least square method:

$$\frac{1}{\alpha} = \frac{n \sum XY - \sum X \sum Y}{\sum Y^2 - (\sum Y)^2}$$

$$u = \bar{X} - \frac{1}{\alpha} \bar{Y}$$

A more simple way introduced by Gumbel is as follows:

$$X_i = u + \frac{1}{\alpha} Y_i \quad i = 1, 2, 3, 4, 5, \dots, n$$

$$X_i^2 = u^2 + 2 \frac{1}{\alpha} u Y_i + \left(\frac{1}{\alpha} Y_i\right)^2$$

$$\sum_{i=1}^n X_i^2 = nu^2 + \frac{2u}{\alpha} \sum_{i=1}^n Y_i + \left(\frac{1}{\alpha}\right)^2 \sum_{i=1}^n Y_i^2$$

$$\bar{X}^2 = u^2 + 2u \frac{1}{\alpha} \bar{Y} + \left(\frac{1}{\alpha}\right)^2 \bar{Y}^2$$

4-(24)

$$\text{where} \quad \bar{X}^2 = \frac{\sum X^2}{n} \quad \bar{Y}^2 = \frac{\sum Y^2}{n}$$

$$\bar{X} = u + \frac{1}{\alpha} \bar{Y}$$

$$\bar{X}^2 = u^2 + 2u \frac{1}{\alpha} \bar{Y} + \left(\frac{1}{\alpha}\right)^2 (\bar{Y})^2 \quad 4-(25)$$

$$\text{equation 4-(24) - equation 4-(25)} \quad \bar{X}^2 - (\bar{X})^2 = \left[\left(\frac{1}{\alpha}\right)^2 (\bar{Y}^2 - (\bar{Y})^2)\right]$$

$$\text{we have} \quad \frac{1}{\alpha} = \sqrt{\frac{\bar{X}^2 - (\bar{X})^2}{\bar{Y}^2 - (\bar{Y})^2}} \quad 4-(26)$$

and

$$u = \bar{X} - \frac{1}{\alpha} \bar{Y} \quad 4-(27)$$

5. Prediction of future flood

The future flood with a certain frequency can be predicted simply by extending the straight line on the probability paper. From the intersection point of the extended line and the line of equal recurrence interval,

the predicted flood can be read from the ordinate of the probability paper. An example is given with complete procedures to show the application of Gumbel's extreme value in flood frequency analysis.

Example:

Watershed No. S-1

Gaging Station - Bean Blossom Creek at Dolan, Indiana

Year	Instantaneous Annual Peak Discharge Q cfs.	Rank	Q cfs.	Φ	y
1946	1830	1	361	0.0665	- 0.99716
1947	9420	2	1320	0.133	- 0.70181
1948	2110	3	1390	0.200	- 0.47588
1949	9060	4	1740	0.266	- 0.28085
1950	8740	5	1830	0.333	- 0.09496
1951	3700	6	2110	0.400	+ 0.08742
1952	5100	7	3040	0.466	+ 0.26975
1953	1320	8	3700	0.532	0.46027
1954	361	9	4270	0.600	0.67173
1955	1390	10	5100	0.665	0.89657
1956	1740	11	5480	0.733	1.16922
1957	4270	12	8740	0.800	1.49994
1958	3040	13	9060	0.866	1.93884
1959	5480	14	9420	0.932	2.65324

The discharge Q and the reduced value y were then plotted in probability paper as shown in page (39). A straight line was obtained by the least square method.

C. Flood frequency analysis for small watersheds in Indiana

1. The location of watersheds

Thirty-two watersheds distributed throughout the whole state were selected to study for flood frequency analysis. Fig. (1) is a map of Indiana showing the thirty-two watersheds, and Table (1) lists the names of the watersheds and their assigned number. For convenience, these numbers will be used in this report instead of the name of the watersheds.

The purpose of this study is to determine the runoff from small watersheds. Unfortunately, there are only three watersheds, the area of which is less than 50 square miles, 12 watersheds under 100 square miles, and 29 watersheds under 200 square miles for which flow records are available from the U.S.G.S. In order to have a good distribution of the watersheds over the whole state, three additional larger watersheds were included for the frequency analysis. A bar diagram was plotted in Fig. (2) to show the time period of records for each watershed.

2. The instantaneous peak flood

Instantaneous annual peak flow was used for the extreme value analysis instead of the maximum mean daily flow. The reason for this is that the hydrographs for small watersheds have much sharper peaks and the instantaneous peak is the most critical discharge for hydraulic structures. Data for this instantaneous annual peak were obtained from the U.S.G.S. office in Indianapolis, Indiana. The data are analyzed on a water year basis---that is, from September to August.

Figure 1

Location of Watersheds for Flood Records Studied

LOCATION OF WATER-
SHEDS FOR FLOOD
RECORDS STUDIED

Figure 1

Table I - List of Watersheds, Their Area and Assigned Number

Watershed No.	Gaging Station	Watershed Area (sq mi)
S-1	Bean Blossom Creek at Dolan, Ind.	100
S-2	Clifty Creek at Hartsville, Ind.	88.8
S-3	North Fork Vernon Fork near Butlerville, Ind.	87.3
S-4	Hart Ditch at Munster, Ind.	69.2
S-5	Salt Creek near McCool, Ind.	78.7
S-6	Little Calumet River at Porter, Ind.	62.9
S-7	Cedar Creek at Auburn, Ind.	93.0
S-8	West Creek near Schneider, Ind.	46.3
S-9	Iroquois River at Rosebud, Ind.	30.3
S-10	Bice Ditch near South Marion, Ind.	22.6
S-11	Big Slough Creek near Collegeville, Ind.	84.1
S-12	Carpenter Creek at Egypt, Ind.	48.1
1	Tippecanoe River at Oswego, Ind.	115
2	Mississinewa River near Ridgeville, Ind.	130
3	Wildcat Creek at Greentown, Ind.	162
4	Cicero Creek near Arcadia, Ind.	131
5	Fall Creek near Fortville, Ind.	172
6	Eagle Creek at Indianapolis, Ind.	179
7	Young Creek near Edinburg, Ind.	109
8	Blue River at Carthage, Ind.	187
9	Sand Creek near Brewersville, Ind.	156
10	North Fork Salt Creek near Belmont, Ind.	120
11	Patoka River at Jasper, Ind.	257
12	Busseron Creek near Carlisle, Ind.	228

(continued)

Table I - List of Watersheds, Their Area and Assigned Number (continued)

Watershed No.	Gaging Station	Watershed Area (sq mi)
13	East Fork White Water River at Richmond, Ind.	123
14	Silver Creek near Sellersburg, Ind.	188
15	Big Indiana Creek near Corydon, Ind.	129
16	Kankakee River near North Liberty, Ind.	152
17	Singleton Ditch at Schneider, Ind.	122
18	Deep River at Lake George Outlet at Hobart, Ind.	125
19	Pigeon Creek at Hogback Lake Outlet near Angola, Ind.	102
20	Laughery Creek near Farmers Retreat, Ind.	248

/

Figure 2

Period of Record of Instantaneous Annual Peaks at Gaging Stations

Fig. 2 PERIOD OF RECORD OF INSTANTANEOUS
ANNUAL PEAKS AT GAGING STATIONS.

Watershed NO.	PERIOD OF RECORD										* n
	1900	1910	1920	1930	1940	1945	1950	1955	1960		
S-1											14
S-2											12
S-3											18
S-4											17
S-5											15
S-6											15
S-7											17
S-8											9
S-9											11
S-10											11
S-11											10
S-12											10
1											11
2											13
3											16
4											5
5											18
6											21
7											17
8											10
9											12
10											14
11											12
12											16
13											10
14											5
15											17
16											9
17											11
18											13
19											14
20											19

* n = Length of Record in Years

3. Examine the data

In examining the data, the man-made changes, such as new reservoirs, soil and watervation works, and changes of gaging site will be carefully studied because they can change the natural conditions of stream flow and affect the peak discharge. There are four kinds of man-made changes to be studied for those thirty-two watersheds. They are discussed as follows:

i. Changing of gaging site (23)

There are four watershed gaging sites which were changed.

Watershed	Site Change	Watershed Area	Change of Area
Hobart	400' upstream	125	0
Brewersville	1.7 miles upstream	153	-3
Angola	1.5 miles downstream	105	+3
Greentown	2.0 miles downstream	172	+10

Table II

(The watershed area changes due to the change of gaging site.)

The above table shows the four watersheds whose gaging sites were changed, causing the changes of watershed area. Since the gaging station did not move far away up or downstream from the old gaging site and the change of area is small, its effect on the peak flow can be ignored.

ii. Reservoir

A reservoir was built upstream of each of the gaging stations of watersheds (S-1) and (11). The U.S.G.S. reported that their effect on the peak flow was insignificant. (23)

iii. Over bank storage

The backwater caused by overbank storage may influence the storage-discharge relation of the watershed (16). Since the storage-discharge relation was corrected for this effect, the overbank flow will not affect the flow record at all.

iv. Dredging

Some dredging has taken place in channels of watersheds (17) and (4). This mainly affects the stage-discharge relation; however, there seem to be no serious influence on the peak discharge measurement.

From a close study of these four man-made changes, it can be concluded that their influence on peak flood is insignificant and can be ignored.

4. The historical flood (11, 24)

In plotting the observed data on the probability paper, a problem arises when using the historical data. The historical flood is important to the flood frequency analysis. It gives more information and makes the frequency analysis more reliable, especially in cases where short time records are available. It is questionable whether or not the probability curves derived from short periods of records, say less than 25 years, can be extrapolated to cover the historical data.

The plotting position of historical floods may be computed for the following two cases:

1. Where there is one historical flood, higher than any for the period of record. In this case the recurrence interval of the historical flood should be recorded as equal to one plus the period for which it is the greatest. An example of this is the 1913 historical

flood in Potaka River at Jasper, Indiana. The flood was 16,000 cfs. The period for which it is the largest is 47 years.

According to the formula

$$\Phi = \frac{m}{n+1} = \frac{47}{48}$$

The recurrence interval is

$$T = \frac{1}{1-\Phi} = 48 \text{ YEARS.}$$

- ii. The second case is where there is one historical flood known to be the highest until a greater flood occurs during the period of record. An example of this is the 1913 historical flood in Eagle Creek at Indianapolis, Indiana. It is 19,000 cfs. And the 1957 flood is 28,800 cfs which is higher than the historical one. The plotting position of these cases are shown in the following Table (3):

Floods	n	Φ	T
28800 cfs. max. (1913--1959)	47	47/48	48
19000 cfs. 2nd flood (1913--1959)	47	46/48	24

Table III - Calculation of Historical Floods

5. Straight line fitting for future prediction

According to Gumbel's theory, the observed extremes should plot along a straight line on the probability paper. The statistical method of least square is used to obtain best fit line with the least deviation. An important question arises when the record of flood data is short and needs to be extrapolated to longer periods. The historical flood data can help to give a better frequency-flood relation. But, there will still be a question as to the reliability and recurrence interval of the historical flood data itself.

Based on the above discussions, it is questionable whether the least square method for straightline fitting can be applied, since the lower points in the probability paper play a more important role than the upper larger flow points in the true flood-frequency relation. Therefore, these lower points should be weighed more in the process of line fitting. The line fitting in this report is fitted in two ways:

- i. For those which the plotted points have already formed a straight line pattern, the least square method is used in order to get a best fitting.
- ii. For those which some of upper points are too far away the normal straight line pattern, then, the line is fitted simply by inspection in order to give the lower points more weight in the fitted line.

Results

Following are the results from the frequency analysis for each selected watershed. The instantaneous annual peak discharge against the reduced value (Y) were plotted in probability paper, and a straight line was fitted by the least square method. By extending this line, the future floods of different frequencies were predicted and shown in Table IV.

(S-1) Bean Blossom Creek at Dolan, Ind. (23)

Location.--Lat $39^{\circ}14'30''$, long $86^{\circ}29'57''$, in SW $\frac{1}{4}$ sec. 2, T. 9 N., R. 1 W., on downstream side of right pier of highway bridge at Dolan, 17.5 miles upstream from mouth.

Drainage area.--100 sq mi.

Gage.--Nonrecording gage Apr. 3, 1946, to Sept. 27, 1951; recording gage thereafter. Datum of gage is 576.41 ft above mean sea level, unadjusted.

Stage-discharge relation.--Defined by current-meter measurements. Discharge adjusted for rate of change of stage above 5 ft. Only annual maximums adjusted prior to installation of recording gage.

Flood stage.--15 ft.

Remarks.--Flow regulated since April 1953 by Bloomington Reservoir (capacity, 4,640,000,000 gallons) $7\frac{1}{2}$ miles upstream; peak discharges probably not materially affected.

Peak Stages and Instantaneous Annual Peak Discharge

Date	Gage Height	Discharge cfs	Water Year	Date	Gage Height	Discharge cfs
May 16, 1946	13.0	1,830	1953	Mar. 4, 1953	11.07	1,320
June 2, 1947	17.8	9,420	1954	May 2, 1954	5.45	361
Mar. 27, 1948	13.5	2,110	1955	Apr. 13, 1955	11.63	1,390
Jan. 5, 1949	17.9	9,060	1956	May 28, 1956	12.93	1,740
Jan. 4, 1950	17.75	8,740	1957	May 22, 1957	15.78	4,270
Jan. 21, 1951	15.50	3,700	1958	June 14, 1958		3,040
May 24, 1952	16.12	5,100	1959	Jan. 21, 1959		5,480

BEAN BLOSSOM CREEK AT DOLAN, INDIANA

Return Period (years)

Probability - percent

Reduced Variate (Y)

(S-2) Clifty Creek at Hartsville, Ind.

Location.--Lat $39^{\circ}16'25''$, long $85^{\circ}42'10''$, in NW $\frac{1}{4}$ sec. 36, T. 10 N., R. 7 E., at downstream side of left abutment of highway bridge, a quarter of a mile north of Hartsville, and 5 miles upstream from Duck Creek.

Drainage area.--83.8 sq mi.

Gage.--Nonrecording gage Feb. 12, 1948, to Sept. 23, 1952; recording gage thereafter. Datum of gage is 677.34 ft above mean sea level, datum of 1929.

Stage-discharge relation.--Defined by current-meter measurements below 6,000 cfs.

Historical data.--Flood of 1913 on Clifty Creek reached a stage of about 3 ft higher than the McKinley (1897) flood according to a report in the Evening Republican of Columbus, Ind. dated Mar. 25, 1913. (The preceding statement was apparently for the Petersville area, about 6 miles downstream from Hartsville).

Peak Stages and Instantaneous Annual Peak Discharge

Water Year	Date	Gage Height	Discharge cfs	Water Year	Date	Gage Height	Discharge cfs
1913	Mar. 25, 1913	25.1		1954	May 27, 1954	4.17	635
1948	Mar. 27, 1948	8.48	3,710	1955	July 8, 1955	6.24	1,760
1949	Jan. 5, 1949	13.4	8,100	1956	June 22, 1956	11.10	5,890
1950	Jan. 4, 1950	11.8	6,520	1957	July 4, 1957	9.28	4,270
1951	Nov. 20, 1950	8.9	3,910	1958	May 6, 1958		2,700
1952	Jan. 26, 1952	11.5	6,250	1959	Jan. 21, 1959		11,300
1953	Mar. 4, 1953	5.57	1,370				

CLIFTY CREEK AT HARTSVILLE, INDIANA

Return Period (years)

(S-3) North Fork of Vernon Fork near Butlerville, Ind.

Location.--Lat $39^{\circ}02'55''$, long $85^{\circ}32'40''$, in SE $\frac{1}{4}$ sec. 17, T. 7 N., R. 9 E., on left bank, 0.3 mile downstream from Muscatatuck State School dam, $1\frac{1}{4}$ miles downstream from Brush Creek, and 2 miles northwest of Butlerville.

Drainage area.--87.3 sq mi.

Gage.--Nonrecording gage Feb. 16, 1942, to Aug. 18, 1942; recording gage thereafter.
Datum of gage is 669.40 ft above mean sea level, datum of 1929.

Stage-discharge relation.--Defined by current-meter measurements.

Flood stage.--11 ft.

Peak Stages and Instantaneous Annual Peak Discharge

Year	Date	Gage Height	Discharge cfs	Water Year	Date	Gage Height	Discharge cfs
1942	Apr. 9, 1942	8.94	2,560	1951	Nov. 20, 1950	15.98	8,030
1943	Mar. 16, 1943	17.79	9,910	1952	Jan. 26, 1952	13.18	5,300
1944	Apr. 11, 1944	12.63	4,780	1953	Mar. 4, 1953	10.34	3,260
1945	Mar. 6, 1945	18.72	10,900	1954	Jan. 1, 1954	5.58	840
1946	Feb. 13, 1946	15.95	8,030	1955	Feb. 27, 1955	12.05	4,300
1947	June 2, 1947	14.30	6,330	1956	May 28, 1956	16.23	8,330
1948	Mar. 27, 1948	15.12	7,130	1957	May 22, 1957	17.04	9,080
1949	Jan. 24, 1949	18.73	10,900	1958	July 22, 1958		7,730
1950	Jan. 4, 1950	17.90	10,000	1959	Jan. 21, 1959		26,200

(S-4) Hart ditch at Munster, Ind.

Location.--Lat $41^{\circ}33'40''$, long $87^{\circ}28'50''$, in N $1/2$ sec. 20, T. 36 N., R. 9 W., on left bank at city limits of Munster, a quarter of a mile downstream from U. S. Highway 41, and 0.4 mile upstream from mouth.

Drainage area.--69.2 sq mi.

Gage.--Recording. Datum of gage is 591.21 ft above mean sea level, datum of 1929.

Stage discharge relation.--Defined by current-meter measurements. Dredging operations assumed to have occurred between April 1944 and April 1945, and subsequent filling have affected high-water rating. Backwater from Little Calumet River and possibly from overbank return affects stage at gage at times during periods of extremely high flow.

Flood Stage.--7 ft.

Remarks.--Hart ditch is tributary to Little Calumet River. At this point low flow of Little Calumet River runs west into Calumet Sag Channel or into Lake Michigan through Grand Calumet River; floodflow at times runs east into channel storage or through Burns ditch to Lake Michigan.

Peak Stages and Instantaneous Annual Peak Discharge

Water Year	Date	Gage Height	Discharge cfs	Water Year	Date	Gage Height	Discharge cfs
1943	Mar. 16, 1943	6.95	2,280	1952	June 14, 1952	4.39	1,190
1944	Mar. 15, 1944	7.23	2,420	1953	Mar. 15, 1953	3.84	960
1945	May 8, 1945	3.73	1,270	1954	Mar. 25, 1954	4.25	1,110
1946	Jan. 6, 1946	2.88	780	1955	Oct. 11, 1954	7.83	2,600
1947	Apr. 6, 1947	6.17	2,490	1956	May 11, 1956	5.27	1,550
1948	May 11, 1948	5.60	1,950	1957	July 14, 1957	7.60	2,060
1949	Feb. 13, 1949	3.00	850	1958	June 10, 1958		960
1950	Dec. 22, 1949	4.83	1,570	1959	Apr. 28, 1959		2,670
1951	May 11, 1951	5.01	1,430				

HART DITCH AT MUNSTER, INDIANA

Return Period (years)

(S-5) Salt Creek near McCool, Ind.

Location.--Lat $41^{\circ}35'48''$, long $87^{\circ}08'40''$, in SE $\frac{1}{4}$ sec. 6, T 36 N., R 6 W, on left bank on downstream side of highway bridge, 50 ft downstream from New York Central Railroad bridge, $1\frac{1}{4}$ miles north of McCool, and 1.5 miles upstream from Little Calumet River.

Drainage area.--78.7 sq mi.

Gage.--Nonrecording gage May 5, 1945, to July 24, 1955; recording gage thereafter. Datum of gage is 594.10 ft above mean sea level, datum of 1929 (levels by Indiana Flood Control and Water Resources Commission).

Stage-discharge relation.--Defined by current-meter measurements below 2,300 cfs.

Flood stage.--10 ft.

Peak Stages and Instantaneous Annual Peak Discharge

Year	Date	Gage Height	Discharge cfs	Water Year	Date	Gage Height	Discharge cfs
45	June 29, 1945	10.83	990	1953	Mar 16, 1953	8.16	454
46	June 13, 1946	11.27	1,280	1954	Mar. 26, 1954	10.48	910
47	Apr. 5, 1947	11.83	1,580	1955	Oct. 11, 1954	14.12	3,180
48	May 11, 1948	12.3	1,910	1956	Apr. 29, 1956	11.26	1,280
49	Feb. 14, 1949	9.28	525	1957	Apr. 27, 1957	9.81	725
50	Dec. 22, 1949	12.02	1,700	1958	Nov. 15, 1957		456
51	May 11, 1951	10.78	970	1959	Apr. 28, 1959		1,200
52	Nov. 14, 1951	10.63	912				

SALT CREEK NEAR MCCOOL, INDIANA

Return Period (years)

(S-6) Little Calumet River at Porter, Ind.

Location.--Lat $41^{\circ}37'18''$, long $87^{\circ}05'13''$, in NE $1/4$ sec. 34, T. 37 N., R. 6 W., near center of span of downstream side of highway bridge, three-quarters of a mile northwest of Porter, and 4.5 miles upstream from Salt Creek.

Drainage area.--62.9 sq mi.

Gage --Nonrecording gage May 5, 1945, to June 25, 1952; recording gage thereafter.
Datum of gage is 603.48 ft above mean sea level, datum of 1929.

Stage-discharge relation.--Defined by current-meter measurements below 2,500 cfs.
Rating subject to changes throughout range of stage.

Flood stage.--7 ft.

Peak Stages and Instantaneous Annual Peak Discharge

er r	Date	Gage Height	Discharge cfs	Water Year	Date	Gage Height	Discharge cfs
5	June 28, 1945	9.88	2,440	1953	May 23, 1953	6.64	521
6	June 13, 1946	6.99	715	1954	Apr. 26, 1954	8.32	1,170
7	Apr. 5, 1947	9.42	2,140	1955	Oct. 10, 1954	11.66	3,110
8	May 11, 1948	9.10	1,960	1956	Apr. 29, 1956	8.67	1,370
9	May 20, 1949	6.88	690	1957	Apr. 27, 1957	7.65	848
0	Dec. 22, 1949	8.72	1,720	1958	Feb. 28, 1958		490
1	May 11, 1951	8.11	1,360	1959	Apr. 28, 1959		1,420
2	Nov. 14, 1951	7.92	1,060				

LITTLE CALUMET RIVER AT PORTER, INDIANA

Return Period (years)

(S-7) Cedar Creek at Auburn, Ind.

Location.--Lat $41^{\circ}21'$, long $85^{\circ}03'$, in SW $1/4$ sec. 29, T. 34 N., R. 13 E., near center of span on upstream side of Ninth Street Bridge in Auburn and 2 miles upstream from Peckhart ditch.

Drainage area.--93 sq mi., approximately.

Gage.--Nonrecording gage July 30, 1943, to Sept. 30, 1953; recording gage thereafter. Datum of gage is 847.14 ft above mean sea level (city of Auburn bench mark).

Stage-discharge relation.--Defined by current-meter measurements.

Flood stage.--8 ft.

Peak Stages and Instantaneous Annual Peak Discharge

Year	Date	Gage Height	Discharge cfs	Water Year	Date	Gage Height	Discharge cfs
43	May 1943	9.8	1,470	1952	Mar. 11, 1952	8.45	900
44	Apr. 12, 1944	9.3	1,230	1953	Mar. 4, 1953	5.80	471
45	May 18, 1945	9.13	1,150	1954	Mar. 25, 1954	7.57	707
46	June 13, 1946	8.58	916	1955	Jan. 6, 1955	7.61	707
47	Apr. 21, 1947	9.02	983	1956	Apr. 30, 1956	8.85	1,050
48	Feb. 28, 1948	8.58	905	1957	Apr. 6, 1957	6.89	651
49	Feb. 16, 1949	9.21	995	1958	Dec. 20, 1957		540
50	Apr. 5, 1950	9.90	1,520	1959	Feb. 14, 1959		890
51	Feb. 22, 1951	8.65	940				

CEDAR CREEK AT AUBURN, INDIANA

Return Period (years)

(S-8) West Creek near Schneider, Ind.

Location.--Lat $41^{\circ}12'52''$, long $87^{\circ}29'36''$, in NW $1/4$ NE $1/4$ sec. 19, T. 32 N., R. 9 W., on left bank at downstream side of county highway bridge, 1.2 miles upstream from Singleton ditch and $2\frac{3}{4}$ miles northwest of Schneider.

Drainage area.--54.5 sq mi.

Gage.--Nonrecording gage July 29, 1948, to Dec. 31, 1951, and Jan. 1, 1954, to June 10, 1956; recording gage since June 11, 1956. Datum of gage is 627.86 ft above mean sea level, datum of 1929 (levels by Soil Conservation Service).

Stage-discharge relation.--Defined by current-meter measurements.

Flood stage.--7 ft.

Peak Stages and Instantaneous Annual Peak Discharge

Date	Gage Height	Discharge cfs	Water Year	Date	Gage Height	Discharge cfs
Feb. 13, 1949	4.58	504	1956	Feb. 25, 1956	5.42	710
Dec. 22, 1949	6.56	1,050	1957	July 13, 1957	7.02	1,250
Feb. 19, 1951	5.52	738	1958	June 9, 1958		794
Mar. 25, 1954	6.10	1,000	1959	Apr. 28, 1959		1,200
Oct. 10, 1954	8.06	1,840				

Return Period (years)

(S-9) Iroquois River at Rosebud, Ind.

Location.--Lat $41^{\circ}02'$, long $87^{\circ}11'$, in SW $1/4$ sec. 24, T. 30 N., R. 7 W., 100 ft downstream from bridge on county road, half a mile north of Rosebud, half a mile downstream from confluence of Swain and Dexter ditches, 1.5 miles upstream from Davidson ditch, and 2 miles east of Farr.

Drainage area.--30.3 sq mi.

Gage.--Nonrecording gage July 22, 1948, to Sept. 30, 1953; recording gage thereafter. Datum of gage is 661.47 ft above mean sea level, datum of 1929.

Stage-discharge relation.--Defined by current-meter measurements below 330 cfs.

Flood stage.--10 ft

Peak Stages and Instantaneous Annual Peak Discharge

er r	Date	Gage Height	Discharge cfs	Water Year	Date	Gage Height	Discharge cfs
9	Feb 15, 1949	6.15	254	1955	Jan 6, 1955	4.84	126
0	Apr. 4, 1950	8.3	422	1956	Apr 29, 1956	6.65	225
1	July 9, 1951	7.2	235	1957	Apr. 28, 1957	7.90	290
2	Apr. 23, 1952	7.3	263	1958	June 10, 1958		308
3	Mar 15, 1953	5.75	185	1959	Feb. 10, 1959		343
4	Mar 25, 1954	4.59	130				

MISSISSIPPI RIVER AT ROSEBUD, INDIANA

Return Period (years)

(S-10) Bice ditch near South Marion, Ind.

Location.--Lat $40^{\circ}52'$, long $87^{\circ}06'$, on line between secs. 15 and 22, T. 28 N., R. 6W., on left bank at upstream side of bridge on State Highway 16, 2 miles upstream from Big Slough Creek, 3 miles southeast of South Marion, and 5 miles southeast of Rensselaer.

Drainage area.--22.6 sq mi.

Gage.--Nonrecording gage Dec. 31, 1948, to Aug. 4, 1955; recording gage thereafter. Datum of gage is 653.30 ft above mean sea level, datum of 1929.

Stage-discharge relation.--Defined by current-meter measurements.

Peak Stages and Instantaneous Annual Peak Discharge

Year	Date	Gage Height	Discharge cfs	Water Year	Date	Gage Height	Discharge cfs
49	Feb. 15, 1949	9.09	410	1955	June 11, 1955	10.16	353
50	July 19, 1950	10.06	490	1956	Apr. 29, 1956	10.75	504
51	July 9, 1951	11.43	610	1957	July 13, 1957	10.36	458
52	June 14, 1952	10.80	556	1958	June 13, 1958		780
53	July 5, 1953	8.65	374	1959	Feb. 10, 1959		480
54	June 22, 1954	8.12	339				

... SOUTH ... IANION , INDIANA

(S-11) Big Slough Creek near Collegeville, Ind.

Location.--Lat $40^{\circ}53'$, long $87^{\circ}09'$, in SW $\frac{1}{4}$ NW $\frac{1}{4}$ sec. 7, T. 28 N., R. 6 W., on right bank on downstream side of bridge on State Highway 53, $1\frac{1}{2}$ miles south of Collegeville, $2\frac{1}{2}$ miles upstream from mouth, and $2\frac{3}{4}$ miles downstream from Bice ditch.

Drainage area.--84.1 sq mi.

Gage.--Nonrecording gage July 28, 1948, to Dec. 31, 1951, and Oct. 1, 1952, to Aug. 4, 1955; recording gage since Aug. 5, 1955. Datum of gage is 637.75 ft above mean sea level, datum of 1929.

Stage-discharge relation.--Defined by current-meter measurements.

Peak Stages and Instantaneous Annual Peak Discharge

Year	Date	Gage Height	Discharge cfs	Water Year	Date	Gage Height	Discharge cfs
1913	March	13.7		1954	June 22, 1954	8.84	390
1927		12.5		1955	June 11, 1955	12.4	1,100
1949	Feb. 15, 1949	11.26	880	1956	Apr. 29, 1956	13.0	1,470
1950	Apr. 4, 1950	12.3	1,180	1957	July 14, 1957	12.96	1,470
1951	July 10, 1951	13.22	1,450	1958	June 13, 1958		2,030
1953	Apr. 1, 1953	10.50	655	1959	Apr. 28, 1959		1,030

(S-12) Carpenter Creek at Egypt, Ind.

Location.--Lat $40^{\circ}52'$, long $87^{\circ}12'$, on line between SW $\frac{1}{4}$ sec. 15 and NW $\frac{1}{4}$ sec. 22, T. 28 N., R. 7 W., on left bank on downstream side of bridge on State Highway 16, 2 $\frac{3}{4}$ miles upstream from mouth, and 4 miles southwest of Collegeville.

Drainage Area.--48.1 sq mi.

Gage.--Nonrecording gage July 26, 1948, to Dec. 31, 1951, and Oct. 1, 1952, to Sept. 5, 1955; recording gage since Sept. 6, 1955. Datum of gage is 640.37 above mean sea level, datum of 1929.

Stage-discharge relation.--Defined by current-meter measurements.

Peak Stages and Instantaneous Annual Peak Discharge

Year	Date	Gage Height	Discharge cfs	Water Year	Date	Gage Height	Discharge cfs
1949	Feb 15, 1949	10.14	1,160	1955	June 8, 1955	9.80	984
1950	Apr. 4, 1950	10.3	1,300	1956	Apr. 29, 1956	9.93	1,040
1951	July 9, 1951	10.92	1,790	1957	July 13, 1957	9.42	810
1953	July 6, 1953	9.21	735	1958	June 10, 1958		3,720
1954	June 22, 1954	8.95	655	1959	Feb. 10, 1959		2,690

(1) Tippecanoe River at Oswego, Ind.

Location.--Lat $41^{\circ}19'14''$, long $85^{\circ}47'21''$, in NE $\frac{1}{4}$, NE $\frac{1}{4}$ sec. 14, T. 33 N., R 6 E., on left bank 10 ft downstream from dam at Tippecanoe Lake outlet in Oswego, 3 miles east of Leesburg.

Drainage area.--115 sq mi.

Gage.--Nonrecording gage Oct. 1, 1949, to Aug. 11, 1953; recording gage thereafter. Datum of gage is 830.00 ft above mean sea level, datum of 1929.

Stage-discharge relation.--Defined by current-meter measurements below 680 cfs and extended to 1,050 cfs by logarithmic plotting.

Remarks.--Peak discharges affected by natural storage in numerous lakes upstream.

Peak Stages and Instantaneous Annual Peak Discharge

Year	Date	Gage Height	Discharge cfs	Water Year	Date	Gage Height	Discharge cfs
1943	May 21,	9.4	1,050	1955	Oct. 17, 1954	8.65	700
1950	Apr. 8--10, 1950	8.62	630	1956	May 5, 6, 1956	8.08	450
1951	Feb. 27, 28, 1951		430	1957	Apr. 17--22, 1957	7.59	315
1952	Jan. 30, 1952		370	1958	Sept. 19--23, 1957		383
1953	Mar. 22, 23, 1953	6.76	179	1959	Feb. 18, 1959		548
1953	Apr. 29, 30, 1953	7.60	322

(2) Mississinewa River near Ridgeville, Ind

Location.--Lat $40^{\circ}17'$, long $85^{\circ}00'$, in SW $1/4$ sec. 8, T. 19 N., R. 14 E., on right bank 10 ft downstream from highway bridge, 0.8 mile downstream from Mud Creek, and 2 miles east of Ridgeville.

Drainage area.--130 sq mi.

Gage --Nonrecording gage Aug. 30, 1946, to Oct. 3, 1950; recording gage thereafter.
Datum of gage is 965.23 ft above mean sea level, datum of 1929.

Stage-discharge relation.--Defined by current-meter measurements below 3,400 cfs.

Flood stage.--10 ft.

Historical data.--Local residents stated that the 1913 flood was secondary to a flood in the early 1930's when the river reached an estimated stage of 15.0 ft.

Peak Stages and Instantaneous Annual Peak Discharge

Year	Date	Gage Height	Discharge cfs	Water Year	Date	Gage Height	Discharge cfs
1947	Jan. 30, 1947	11.16	2,490	1954	Mar. 30, 1954	7.80	1,020
1948	Jan. 1, 1948	12.2	3,480	1955	Jan. 6, 1955	11.16	2,490
1949	Jan. 5, 1949	13.1	4,560	1956	Nov. 16, 1955	12.79	4,200
1950	Feb. 14, 1950	13.4	4,920	1957	June 28, 1957	14.57	8,830
1951	Feb. 21, 1951	12.75	4,200	1958	June 10, 1958		13,900
1952	Jan. 26, 1952	11.99	3,250	1959	Jan. 21, 1959		9,120
1953	Mar. 4, 1953	12.00	3,250				

MUSKOGEE RIVER NEAR MOOREVILLE, INDIANA

(3) Wildcat Creek at Greentown, Ind.

Location.--Lat $40^{\circ}27'$, long $85^{\circ}57'$, on line between secs. 9 and 10, T. 23 N., R. 5 E., on left bank at downstream side of bridge on State Highway 213, 1.5 miles south of Greentown.

Drainage area.--162 sq mi; 172 sq mi prior to June 5, 1954.

Gage.--Nonrecording gage Feb. 20, 1945, to June 4, 1954; recording gage thereafter. Prior to June 5, 1954, at site 2.0 miles downstream at datum 5.34 ft lower than present datum. Datum of present gage is 809.33 ft above mean sea level, datum of 1929.

Stage-discharge relation.--Defined by current-meter measurements.

Flood stage.--11 ft at both sites.

Historical data.--The following statements appear in old newspapers for Kokomo, about 9 miles downstream. September 1866: "Wildcat Creek raging; no train for 3 days." 1904: "Greatest flood in Kokomo history."

Flood of August 1871 reached a stage 3 inches below that of the 1913 flood at a bridge $1\frac{1}{2}$ miles downstream from present site according to information by local resident on the basis of remembered high-water marks made on the same tree.

Peak Stages and Instantaneous Annual Peak Discharge

er r	Date	Gage Height	Discharge cfs	Water Year	Date	Gage Height	Discharge cfs
3	May, 1943	15.0	5,960	1952	Mar. 11, 1952	11.52	2,580
5	Apr. 1, 1945	10.04	1,680	1953	Mar. 4, 1953	10.34	1,810
6	Oct. 2, 1945	9.94	1,640	1954	Apr. 12, 1954	5.63	450
7	Apr. 30, 1947	10.94	2,140	1955	Jan. 7, 1955	10.00	1,650
8	Mar. 22, 1948	11.63	2,670	1956	May 28, 1956	9.97	1,650
9	Jan. 19, 1949	13.19	4,110	1957	June 29, 1957	12.37	2,260
0	Jan. 4, 1950	15.3	6,320	1958	June 10, 1958		4,900
1	Feb. 21, 1951	10.7	2,020	1959	Feb. 10, 1959		5,390

WILDCAT RIVER NEAR GREENTOWN, INDIANA

Return Period (years)

67.

Probability - percent

Reduced Variate (Y)

(4) Cicero Creek near Arcadia

Location.--Lat $40^{\circ}11'$, long $86^{\circ}00'$, on line between secs. 18 and 19, T. 20 N, R. 5E., on left bank on downstream side of county bridge, $1\frac{1}{2}$ miles east of Arcadia, Hamilton County, and 5 miles upstream from little Cicero Creek.

Drainage area.--131 sq. mi.

Gage.-- Water-stage recorder. Datum of gage is 815.12 ft above mean sea level Datum of 1929. Prior to Dec. 7, 1955, wire-weight gage at same site and datum.

Peak Stages and Instantaneous Annual Peak Discharge

Water Year	Date	Gage Height	Discharge cfs	Water Year	Date	Gage Height	Discharge cfs
1955	July 16, 1955		1,280	1958	June 15, 1958		2,740
1956	July 21, 1956		1,540	1959	Feb. 11, 1959		2,170
1957	June 29, 1957		6,720

(5) Fall Creek near Fortville, Ind.

Location.--Lat $39^{\circ}57'15''$, long $85^{\circ}52'05''$, in sec 5, T. 17 N., R. 6 E., on right bank at downstream side of bridge on State Highway 238, 1 mile downstream from Lick Creek and 2 miles northwest of Fortville.

Drainage area.--172 sq mi.

Gage.--Nonrecording gage July 1, 1941, to June 26, 1942; recording gage thereafter
Datum of gage is 787.43 ft above mean sea level, datum of 1929 (levels by Indianapolis Water Co.)

Stage-discharge relation.--Defined by current-meter measurements.

Flood stage.--6 ft.

Historical data.--Flood of 1913 reached a stage of about 12 feet according to information from local resident.

Peak Stages and Instantaneous Annual Peak Discharge

Water Year	Date	Gage Height	Discharge cfs	Water Year	Date	Gage Height	Discharge cfs
1942	Mar. 17, 1942	7.39	2,460	1951	Feb. 22, 1951	8.36	4,250
1943	May 18, 1943	9.77	8,240	1952	Jan. 27, 1952	6.85	2,000
1944	Apr. 12, 1944	8.79	5,000	1953	July 6, 1953	8.14	3,850
1945	June 17, 1945	6.86	1,840	1954	Mar. 30, 1954	5.50	1,140
1946	Oct. 2, 1945	6.87	1,840	1955	Jan. 6, 1955	5.76	1,260
1947	July 15, 1947	7.25	2,250	1956	Feb. 28, 1956	7.93	3,130
1948	Mar. 24, 1948	7.97	3,500	1957	June 29, 1957	8.12	3,430
1949	Jan. 19, 1949	7.19	2,350	1958	June 14, 1958		5,040
1950	Jan. 4, 1950	8.85	5,150	1959	Feb. 10, 1959		3,010

Return Period (years)

(6) Eagle Creek at Indianapolis, Ind

Location.--Lat $39^{\circ}46'40''$, long $86^{\circ}15'02''$, in NW $\frac{1}{4}$ sec. 6, T. 15 N., R. 3 E., on right bank at downstream side of bridge on Lynnhurst Drive, 3.0 miles upstream from Little Eagle Creek, 5.0 miles west of Monument Circle in Indianapolis, and 6.7 miles upstream from mouth.

Drainage area--179 sq mi.

Gage.--Nonrecording gage Nov. 18, 1938, to Mar. 19, 1939; recording gage thereafter
Datum of gage is 706.21 ft above mean sea level, datum of 1929.

Stage-discharge relation.--Defined by current-meter measurements below 9,000 cfs and extended above on basis of a combined current-meter measurement and slope-area measurement. High-water relation has shown a tendency to shift. Discharge shown for the 1913 flood is an approximate value based on logarithmic extension of an early rating curve above 9,000 cfs.

Historical data.--The following information was obtained from a report on "Eagle Creek at Indianapolis, Channel Improvements for Flood Control" by Indiana Flood Control and Water Resources Commission, dated February 1956: "Investigations on past flooding by searching newspaper files, interrogation of local persons, examination of old surveys and reports, indicate that flooding occurred along Eagle Creek in 1875, 1904, 1913, 1915, 1926, 1933, 1943, and 1950. Newspaper accounts of flooding on other streams in the Indianapolis area indicate that flooding probably also occurred in 1847, 1858, 1866, 1882 and 1883."

"It is probable that the floods of 1875 and 1904 were among the greater floods on the stream. Newspaper accounts and weather records indicate that the flood of July 1915 was nearly as great as that of March 1913."

Peak Stages and Instantaneous Annual Peak Discharge

Water Year	Date	Gage Height	Discharge cfs	Water Year	Date	Gage Height	Discharge cfs
1913	March 1913	16.0	19,000	1947	Jan. 30, 1947	8.47	3,370
1938	April 1938	14.5	-	1948	Apr. 6, 1948	12.26	9,550
1939	Mar. 12, 1939	10.6	6,610	1949	Jan. 19, 1949	11.86	7,250
1940	Mar. 3, 1940	6.30	1,860	1950	Jan. 4, 1950	13.03	8,670
1941	June 12, 1941	5.77	1,470	1951	Feb. 21, 1951	8.47	3,950
1942	Feb. 7, 1942	8.66	4,120	1952	Jan. 27, 1952	9.88	5,520
1943	May 11, 1943	12.17	9,660	1953	Mar. 5, 1953	9.34	4,920
1944	Apr. 11, 1944	10.61	6,610	1954	Apr. 6, 1954	6.41	2,250
1945	Mar. 31, 1945	8.80	4,230	1955	July 16, 1955	6.91	2,650
1946	May 17, 1946	8.99	3,860	1956	May 28, 1956	13.62	9,920
				1957	June 28, 1957	16.38	28,800
				1958	Aug. 8, 1958		8,560
				1959	Jan. 22, 1959		6,290

EAGLE CREEK AT INDIANAPOLIS, INDIANA

Return Period (years)

Probability - percent

Reduced Variate (Y)

(7) Youngs Creek near Edinburg, Ind

Location.--Lat $39^{\circ}25'08''$, long $86^{\circ}00'18''$, in SW $1/4$ sec 5, T. 11 N., R. 5 E., on left bank, on upstream side of highway bridge half a mile southwest of Amity, 2 miles upstream from mouth, and 5 miles northwest of Edinburg.

Drainage area.--109 sq mi

Gage.--Nonrecording gage Dec. 7, 1942, to June 29, 1955; recording gage thereafter
Datum of gage is 670.20 ft above mean sea level, datum of 1929.

Stage-discharge relation.--Defined by current-meter measurements below 7,000 cfs and by contracted-opening measurement at 10,700 cfs.

Flood stage.--7 ft.

Peak Stages and Instantaneous Annual Peak Discharge

Year	Date	Gage Height	Discharge cfs	Year	Date	Gage Height	Discharge cfs
43	Mar 19, 1943	10.40	3,700	1952	Jan. 27, 1952	13.4	10,700
44	Apr 11, 1944	11.00	4,290	1953	Mar. 4, 1953	8.37	2,080
45	Mar. 6, 1945	11.00	4,290	1954	Jan. 27, 1954	3.27	443
46	May 16, 1946	9.0	2,510	1955	May 28, 1955	6.2	1,110
47	June 2, 1947	11.12	4,390	1956	Nov. 16, 1955	12.20	7,790
48	Mar. 27, 1948	7.68	1,650	1957	July 5, 1957	11.62	6,510
49	Jan. 5, 1949	11.9	5,190	1958	June 11, 1958		4,350
50	Jan. 4, 1950	10.8	4,090	1959	Jan. 21, 1959		6,270
51	Jan. 15, 1951	9.15	2,730				

Return Period (years)

Probability - percent

Reduced Variate (Y)

(8) Blue River at Carthage, Ind.

Location.--Lat $39^{\circ}46'$, long $85^{\circ}34'$, in sec. 18, T 15 N., R 9 E, on right bank 500 ft upstream from highway bridge, half a mile west of Carthage, and $2\frac{1}{4}$ miles downstream from Three Mile Creek.

Drainage area.--187 sq mi.

Gage.--Nonrecording gage Oct. 11, 1950, to July 18, 1951; recording gage thereafter Prior to July 19, 1951, at bridge 500 ft downstream. Datum of gage is 859.33 ft above mean sea level, datum of 1929.

Stage-discharge relation.--Defined by current-meter measurements.

Flood stage.--7 ft.

Peak Stages and Instantaneous Annual Peak Discharge

Water Year	Date	Gage Height	Discharge cfs	Water Year	Date	Gage Height	Discharge cfs
1949	Jan. 5, 1949	10.6	5,750	1955	Jan. 6, 1955	5.97	1,290
1951	Feb. 21, 1951	11.2	6,650	1956	Nov. 16, 1955	11.52	5,800
1952	Jan. 27, 1952	11.02	6,350	1957	June 18, 1957	9.77	3,900
1953	Mar. 4, 1953	9.17	3,580	1958	June 14, 1958		7,020
1954	Apr. 5, 1954	10.02	4,850	1959	Jan. 21, 1959		8,340

BLUE RIVER AT CARTHAGE, INDIANA

Return Period (years)

(9) Sand Creek near Brewersville, Ind.

Location.--Lat $39^{\circ}05'05''$, long $85^{\circ}39'30''$, in NW $\frac{1}{4}$ sec 5, T. 7 N., R. 8 E., on left bank at downstream side of county highway bridge, $2\frac{1}{2}$ miles west of Brewersville, and 5.2 miles upstream from Bear Creek.

Drainage area.--156 sq mi; 153 sq mi prior to Oct. 6, 1952.

Gage.--Nonrecording gage Feb. 11, 1948, to Oct. 5, 1952, at bridge 1.7 miles upstream at datum approximately 8 ft higher. Recording gage since Oct. 6, 1952, at present site. Altitude of present gage is 630 ft (by altimeter).

Stage-discharge relation.--Defined by current-meter measurements at former site and by gage-height relationship with former site at present location.

Peak Stages and Instantaneous Annual Peak Discharge

Water Year	Date	Gage Height	Discharge cfs	Water Year	Date	Gage Height	Discharge cfs
1948	Mar. 27, 1948	17.5	9,980	1954	June 1, 1954	5.75	1,240
1949	Jan. 5, 1949	19.0	12,100	1955	Feb. 27, 1955	11.42	4,300
1950	Jan. 4, 1950	19.2	12,400	1956	May 28, 1956	15.45	7,560
1951	Nov. 20, 1950	18.4	11,100	1957	Apr. 4, 1957	16.33	8,480
1952	Jan. 26, 1952	13.4	5,780	1958	July 22, 1958		7,150
1953	Mar. 4, 1953	10.19	3,460	1959	Jan. 21, 1959		19,900

(10) North Fork Salt Creek near Belmont, Ind.

Location.--Lat $39^{\circ}09'00''$, long $86^{\circ}20'14''$, in NW 1/4 sec. 5, T. 8 N., R. 2 E., on right bank 15 ft downstream from bridge on State Highway 46, 100 ft upstream from Schooner Creek, 0.7 mile northeast of Belmont, 6 1/2 miles upstream from Brummett Creek, and 20 miles upstream from mouth.

Drainage area.--120 sq mi, includes that of Schooner Creek.

Gage.--Nonrecording gage Apr. 4, 1946, to Oct. 8, 1951; recording gage thereafter. Altitude of gage is 546 ft (from topographic map).

Stage-discharge relation.--Defined by current-meter measurements below 9,800 cfs. Discharge adjusted for rate of change of stage above 7 ft. Only annual maximums adjusted prior to installation of recording gage.

Flood stage.--16 ft

Peak Stages and Instantaneous Annual Peak Discharge

Year	Date	Gage Height	Discharge cfs	Water Year	Date	Gage Height	Discharge cfs
1913	March	25.7	-	1953	Mar. 4, 1953	17.76	2,780
1946	May 16, 1946	20.3	5,910	1954	Jan. 27, 1954	9.38	825
1947	June 2, 1947	21.2	10,100	1955	Mar. 21, 1955	15.91	2,220
1948	Mar. 27, 1948	18.0	3,010	1956	May 28, 1956	18.12	3,030
1949	Jan. 5, 1949	22.2	13,300	1957	Apr. 4, 1957	19.92	6,340
1950	Jan. 4, 1950	21.7	11,600	1958	June 14, 1958		5,920
1951	Feb. 21, 1951	19.53	5,100	1959	Jan. 21, 1959		12,000
1952	May 24, 1952	22.55	15,200

(11) Patoka River at Jasper, Ind

Location.--Lat $38^{\circ}24'49''$, long $86^{\circ}52'36''$, in SE $\frac{1}{4}$ sec 20, T. 1 S., R. 4 W., on left bank, 0.3 mile upstream from unnamed outlet of Jasper Lake, 1.0 mile downstream from Coon Seitz bridge, 1.2 miles downstream from Beaver Creek, and 3.3 miles northeast of Jasper.

Drainage area.--257 sq mi; 270 sq mi at former site

Gage --Nonrecording gage Nov. 20, 1947, to Sept. 17, 1956; recording gage thereafter Prior to Sept. 18, 1956, at site 5.6 miles downstream at datum 0.34 ft lower; datum of present gage is 446.19 ft above mean sea level, datum of 1929.

Stage-discharge relation.--Defined by current-meter measurements below 5,000 cfs at former site and below 1,100 cfs for present site.

Flood stage.--14 ft; 9 ft at former site.

Historical data.--Flood of March 1913 is maximum stage known. Maximum stage at present site for period 1925-57, 20 ft in 1925 (information from local resident).

Remarks.--Flow slightly regulated by Beaver Creek Reservoir, whose outlet enters the Patoka River 1.2 miles upstream from the gage; peak discharges not materially affected.

Peak Stages and Instantaneous Annual Peak Discharge

er r								
	Date	Gage Height	Discharge cfs	Water Year	Date	Gage Height	Discharge cfs	
3	March 1913	15.9	16,000	1953	Mar. 8, 1953	7.90	1,640	
7	January 1937	14.8	12,100	1954	Mar. 2, 1954	-	950	
8	Apr. 15, 1948	11.57	4,920	1955	Mar. 3, 1955	9.80	2,940	
9	Jan. 28, 1949	11.13	4,220	1956	Feb. 29, 1956	9.98	3,100	
0	Jan. 7, 1950	12.37	6,300	1957	May 25, 1957	17.87	6,900	
1	Mar. 21, 1951	11.46	4,760	1958	Dec. 22, 1957		4,250	
2	Mar. 14, 1952	10.78	3,880	1959	Jan. 24, 1959		9,150	

POTOKA RIVER AT JASPER, INDIANA

Return Period (years)

(12) Busseron Creek near Carlisle, Ind.

Location.--Lat $38^{\circ}58'30''$, long $87^{\circ}25'35''$, in $W\frac{1}{4}$ sec. 17, T. 6 N., R. 9 W., on right bank 10 ft downstream from bridge on State Highway 58, $1\frac{1}{2}$ miles northwest of Carlisle, and $6\frac{3}{4}$ miles upstream from mouth.

Drainage area.--228 sq mi.

Gage.--Nonrecording gage Oct. 15, 1943, to Nov. 7, 1950; recording gage thereafter. Datum of gage is 425.36 ft above mean sea level (State Highway Department of Indiana bench mark).

Stage-discharge relation.--Defined by current-meter measurements below 4,500 cfs and extended above by logarithmic plotting.

Flood stage.--12 ft.

Peak Stages and Instantaneous Annual Peak Discharge

Water Year	Date	Gage Height	Discharge cfs	Water Year	Date	Gage Height	Discharge cfs
1944	Apr. 12, 1944	16.96	4,700	1952	Mar. 11, 1952	16.17	4,070
1945	Apr. 2, 1945	17.60	5,500	1953	Mar. 4, 1953	16.04	3,890
1946	May 20, 1946	14.90	2,900	1954	Aug. 4, 1954	6.31	430
1947	June 2, 1947	14.60	2,720	1955	Apr. 13, 1955	13.13	2,040
1948	Jan. 3, 1948	15.15	3,100	1956	June 22, 1956	16.12	3,980
1949	Jan. 20, 1949	16.3	4,200	1957	May 23, 1957	17.61	5,200
1950	Jan. 5, 1950	20.05	8,800	1958	Dec 21, 1957		5,400
1951	Feb. 21, 1951	14.75	2,900	1959	Jan. 22, 1959		3,100

Return Period (years)

(13) East Fork Whitewater River at Richmond, Ind.

Location.--Lat $39^{\circ}48'24''$, long $84^{\circ}54'26''$, in SE $1/4$ sec. 7, T. 13 N., R. 1 W., on left bank 50 ft downstream from highway bridge, three-quarters of a mile south of Richmond, and 2 miles upstream from Short Creek.

Drainage area.--123 sq mi.

Gage.--Nonrecording gage Apr. 27, 1949, to July 26, 1949; recording gage thereafter. Datum of gage is 854.01 ft above mean sea level, datum of 1929 (levels by Indiana Flood Control and Water Resources Commission).

Stage-discharge relation.--Defined by current-meter measurements below 5,100 cfs and by slope-area measurement at 13,500 cfs.

Flood stage.--10 ft.

Historical data.--Flood of September 1866 was reported by the Indianapolis Journal to be higher than ever before known. Flood of March 1913 is the maximum stage known according to information by local residents.

Peak Stages and Instantaneous Annual Peak Discharge

Water Year	Date	Gage Height	Discharge cfs	Water Year	Date	Gage Height	Discharge cfs
1913	March 1913	15.0	-	1955	Feb. 21, 1955	6.07	2,540
1950	Jan. 15, 1950	12.49	13,500	1956	Nov. 16, 1955	10.70	8,200
1951	Nov. 20, 1950	10.82	9,250	1957	June 28, 1957	10.54	7,800
1952	Jan. 26, 1952	10.66	9,250	1958	Aug. 2, 1953		10,400
1953	May 22, 1953	6.53	3,160	1959	Jan. 21, 1959		14,100
1954	Mar. 30, 1954	3.86	1,160				

WATERSHED, INDIANA

(14) Silver Creek near Sellersburg

Location.--Lat $38^{\circ}22'15''$, long $85^{\circ}43'35''$, in SW $\frac{1}{4}$ Lot 68, Clark Military Grant, on upstream side of Straws Mill Bridge on Watson Road, 0.3 mile downstream from Pleasant Run, 2.4 miles southeast of Sellersburg, and 11.9 miles upstream from mouth.

Drainage area.--183 sq. mi.

Gage.--Wire-weight gage read twice daily.

Peak Stages and Instantaneous Annual Peak Discharge

Water Year	Date	Gage Height	Discharge cfs	Water Year	Date	Gage Height	Discharge cfs
1955	Feb. 22, 1955		4,220	1953	Nov. 19, 1957		5,080
1956	Feb. 2, 1956		5,250	1959	Jan 22, 1959		19,600
1957	May 23, 1957		6,250				

Return Period (years)

Reduced Variate (Y)

(15) Big Indian Creek near Corydon, Ind

Location.--Lat $33^{\circ}16'35''$, long $86^{\circ}06'35''$, in SE $1/4$ sec. 6, T. 3 S., R. 4 E., on upstream side of bridge on State Highway 335, 0.6 mile upstream from Raccoon Branch and $4\frac{1}{2}$ miles north of Corydon.

Drainage area.--129 sq mi.

Gage.--Nonrecording gage Oct. 12, 1943, to Dec. 8, 1948; recording gage thereafter
Datum of gage is 577.12 ft above mean sea level, datum of 1929.

Stage-discharge relation.--Defined by current-meter measurements below 6,600 cfs and extended above by logarithmic plotting.

Historical data.--Flood of Mar. 19, 1943, is the maximum known at Corydon since beginning of knowledge in 1815.

Peak Stages and Instantaneous Annual Peak Discharge

Water Year	Date	Gage Height	Discharge cfs	Water Year	Date	Gage Height	Discharge cfs
1943	Mar. 19, 1943	22.4	17,000	1952	Mar. 11, 1952	18.76	10,400
1944	Apr. 11, 1944	14.0	4,800	1953	Mar. 3, 1953	14.57	5,400
1945	Mar. 6, 1945	19.2	11,000	1954	Sept. 20, 1954	14.04	4,800
1946	Feb. 14, 1946	15.0	5,910	1955	Mar. 15, 1955	12.98	3,900
1947	June 2, 1947	14.3	5,160	1956	Feb. 25, 1956	16.25	7,180
1948	Apr. 12, 1948	19.3	11,100	1957	Feb. 10, 1957	16.32	7,300
1949	June 15, 1949	16.20	7,180	1958	Nov. 19, 1957		6,590
1950	Feb. 9, 1950	16.77	7,900	1959	Jan. 21, 1959		23,800
1951	Jan. 14, 1951	15.9	6,820				

BIG INDIANA CREEK NEAR LORYDON , INDIANA

Return Period (years)

(16) Kankakee River near North Liberty, Ind.

Location.--Lat $41^{\circ}33'50''$, long $86^{\circ}29'50''$, on line between secs 11 and 23, T. 36 N., R. 1 W., on left bank at downstream side of bridge on St. Joseph County highway named "New Road," 4 miles northwest of North Liberty.

Drainage area.--152 sq mi.

Gage.--Nonrecording gage Jan. 12, 1951, to June 25, 1956; recording gage thereafter. Datum of gage is 680.04 ft above mean sea level, datum of 1929 (levels by Indiana Flood Control and Water Resources Commission).

Stage-discharge relation.--Relation affected by varying amounts of backwater caused by return flow from overbank storage. Frequent current-meter measurements necessary to define relationship during this period.

Peak Stages and Instantaneous Annual Peak Discharge

Water Year	Date	Gage Height	Discharge cfs	Water Year	Date	Gage Height	Discharge cfs
1951	May 11, 1951	6.15	520	1956	Apr. 30, 1956	6.92	660
1952	Nov. 14, 1951	6.97	680	1957	Apr. 27, 1957	6.90	660
1953	Mar. 16, 1953	4.42	260	1958	Dec. 20, 1957		560
1954	Apr 26, 1954		800	1959	Mar. 27, 1959		560
1955	Oct. 10, 1954	8.64	686				

KANKAKEE RIVER NEAR NORTH LIBERTY, INDIANA

Return Period (years)

Probability - percent

(17) Singleton ditch at Schneider, Ind.

Location.--Lat $41^{\circ}12'44''$, long $87^{\circ}26'44''$, on line between NE $1/4$ sec. 21 and NW $1/4$ sec. 22, T. 32 N., R. 9 W., on left bank 15 ft upstream from bridge on U. S. Highway 41, half a mile upstream from Bruce ditch, $1\frac{1}{2}$ miles downstream from Cedar Creek, and $1\frac{2}{3}$ miles north of Schneider.

Drainage area.--122 sq mi.

Gage.--Nonrecording gage July 28, 1948, to Aug. 13, 1951; recording gage thereafter. Prior to Oct. 1, 1949, at datum 2.00 ft higher. Datum of present gage is 623.67 ft above mean sea level, datum of 1929.

Stage-discharge relation.--Defined by current-meter measurements. Dredging in 1950 and subsequent floods and channel deterioration have materially affected the stage-discharge relation.

Peak Stages and Instantaneous Annual Peak Discharge

Water Year	Date	Gage Height	Discharge cfs	Water Year	Date	Gage Height	Discharge cfs
1949	Feb. 15, 1949	-	550	1955	Oct. 11, 1954	10.10	953
1950	Apr. 10, 1950	-	1,100	1956	Feb. 25, 1956	9.62	888
1951	Feb. 19, 1951	8.50	841	1957	Apr. 28, 1957	10.27	979
1952	June 15, 1952	9.82	1,010	1958			714
1953		8.39	812	1959	Feb. 14, 1959		992
1954	Mar. 25, 1954	9.04	810				

SINGLETON DITCH AT SCHNEIDER, INDIANA

Return Period (years)

Probability - percent

Reduced Variate (Y)

(18) Deep River at Lake George Outlet at Hobart, Ind.

Location.--Lat $41^{\circ}32'10''$, long $87^{\circ}15'25''$, in NW $1/4$ sec. 32, T. 36 N., R. 7 W., on left bank at upstream side of highway bridge, 300 ft upstream from Duck Creek, and 400 ft downstream from Lake George Dam.

Drainage area.--125 sq mi.

Gage.--Nonrecording gage Apr. 4, 1947, to July 29, 1952; recording gage thereafter. Prior to July 21, 1955, at site 400 ft upstream at datum 11.80 ft higher than present datum. Datum of present gage is 588.17 ft above mean sea level, datum of 1929 (levels by Indiana Flood Control and Water Resources Commission).

Stage=discharge relation.--Defined by current-meter measurements below 3,300 cfs.

Peak Stages and Instantaneous Annual Peak Discharge

Water Year	Date	Gage Height	Discharge cfs	Water Year	Date	Gage Height	Discharge cfs
1947	Apr. 6, 1947	5.41	2,410	1954	Mar. 26, 1954	4.55	1,440
1948	May 11, 1948	5.86	2,740	1955	Oct. 11, 1954	7.68	3,880
1949	Feb. 14, 1949	3.50	620	1956	May 11, 1956	11.15	1,320
1950	Dec. 22, 1949	5.35	2,390	1957	July 14, 1957	12.35	1,650
1951	May 11, 1951	4.52	1,440	1958	June 10, 1958		720
1952	Nov. 14, 1951	4.41	1,340	1959	July 24, 1959		1,970
1953	Mar. 16, 1953	3.86	912				

DEEP RIVER AT LAKE GEORGE OUTLET AT HOBART, IND

Return Period (years)

(19) Pigeon Creek at Hogback Lake Outlet, near Angola, Ind.

Location.--Lat $41^{\circ}37'24''$, long $85^{\circ}05'44''$, in NE $1/4$ NW $1/4$ sec. 36, T. 37 N., R. 12 E., on right bank 200 ft north of lake outlet, 2 miles southeast of Flint, and 5.1 miles west of Angola.

Drainage area.--102 sq mi, 105 sq mi prior to October 1947.

Gage.--Nonrecording gage Oct. 16, 1945, to Aug. 3, 1953; recording gage thereafter. Prior to Oct. 1, 1947, at site $1\frac{1}{2}$ miles downstream at different datum. Oct. 1, 1947, to Aug. 3, 1953, at site 600 ft downstream at present datum. Datum of present gage is 940.00 ft above mean sea level, datum of 1929.

Stage-discharge relation.--Defined by current-meter measurements below 240 cfs at former site and by current-meter measurements at present site.

Peak Stages and Instantaneous Annual Peak Discharge

Water Year	Date	Gage Height	Discharge cfs	Water Year	Date	Gage Height	Discharge cfs
1946	Feb. 19, 1946	-	220	1953	Mar. 19, 1953	9.30	122
1947	Apr. 24, 1947	10.71	458	1954	Mar. 30, 1954	11.31	317
1948	Mar. 2, 1948	11.79	355	1955	Oct. 17, 1954	11.54	339
1949	Feb. 19, 1949	11.93	366	1956	May 4, 1956	13.39	548
1950	Apr. 8, 1950	14.95	744	1957	Apr. 14, 1957	11.29	317
1951	Feb. 24, 1951	12.50	448	1958	Sept. 21, 1957		274
1952	Jan. 21, 1952	11.85	370	1959	Feb. 17-19, 1959		442

PIGEON CREEK AT HAGBACK LAKE OUTLET NEAR ANGOLA, INDIANA

Return Period (years)

(20) Laughery Creek near Farmers Retreat, Ind.

Location.--Lat $38^{\circ}57'05''$, long $85^{\circ}04'22''$, in sec. 2, T. 4 N., R. 3 W., on right bank 2 miles southeast of Farmers Retreat and $3\frac{3}{4}$ miles downstream from Bear Creek.

Drainage area.--248 sq mi.

Gage.--Nonrecording gage Oct. 3, 1940, to Apr. 15, 1941; recording gage thereafter. Altitude of gage is 526 ft (by barometer).

Stage-discharge relation.--Defined by current-meter measurements below 14,000 cfs.

Flood stage.--13 ft.

Historical data.--Flood of 1897 reached a stage of about 18 feet and is the highest known flood, from information by local residents.

Peak Stages and Instantaneous Annual Peak Discharge

Water Year	Date	Gage Height	Discharge cfs	Water Year	Date	Gage Height	Discharge cfs
1941	June 9 or 10, 1941	9.62	3,960	1951	Jan. 3, 1951	13.50	9,660
1942	Apr. 9, 1942	13.91	10,800	1952	Mar. 10, 1952	13.46	9,660
1943	Mar. 19, 1943	14.50	12,900	1953	May 17, 1953	9.44	3,960
1944	Apr. 11, 1944	13.16	8,880	1954	May 3, 1954	3.99	640
1945	Mar. 6, 1945	15.54	17,000	1955	Mar. 21, 1955	13.88	10,800
1946	Feb. 13, 1946	12.78	7,980	1956	May 28, 1956	14.45	12,500
1947	May 25, 1947	14.62	13,300	1957	July 5, 1957	16.15	20,200
1948	Apr. 12, 1948	13.01	8,410	1958	July 22, 1958		17,000
1949	Jan. 24, 1949	15.23	15,700	1959	Jan. 21, 1959		47,800
1950	Feb. 3, 1950	15.03	14,800

Table IV

The Predicted Annual Instantaneous Peak Discharge From
Flood Frequency Analysis

Watershed No.	Predicting Annual Instantaneous Peak Discharge			
	25-yrs cfs.	50-yrs cfs.	75-yrs cfs.	100-yrs cfs.
S-1	11800	13800	14700	15900
S-2	12900	15000	16000	17200
S-3	20500	24000	25400	27000
S-4	3300	3750	3940	4200
S-5	2950	3420	3600	3860
S-6	3300	3800	4000	4300
S-7	1630	1800	1880	1980
S-8	2100	2350	2450	2600
S-9	465	520	540	570
S-10	835	925	960	1010
S-11	2440	2770	2900	3080
S-12	4160	4800	5100	5500
1	880	1000	1040	1110
2	14000	16500	17400	18800
3	7400	8600	9000	9700
4	6800	7800	8300	8900
5	7700	8900	9300	10000
6	17000	19700	21000	22300
7	10500	12400	13000	14000
8	10800	12200	12800	13600
9	21500	24000	25200	26800
10	19000	22000	23300	25200
11	13500	15600	16500	17700
12	8700	9500	10300	11000
13	19800	22800	24200	26000
14	13300	15100	15900	17000
15	22300	25700	27300	29400
16	1020	1130	1170	1230
17	1270	1370	1410	1470
18	4800	5500	5700	6200
19	760	860	900	960
20	28600	33000	35000	37300

V. GEOMORPHOLOGICAL STUDY

The Purpose of Geomorphological Study

After a study of the flood frequency analysis and obtaining the 25 years flood for small gaged watersheds, a study was made to estimate the flood from ungaged watersheds. This study investigates the geomorphological factors of each watershed and attempts to correlate them to the flood's flows. If a relationship can be obtained between the geomorphological factors describing the watershed and its corresponding 25 year flood, then the flood discharge can be estimated by simply measuring the geomorphological factors from a topographic map.

The geomorphological study is based on the assumption that region is homogeneous. That is to say that the storm pattern above and the geological conditions below, are homogeneous. It is assumed that the geomorphological factors are the main factors that control the rate and amount of runoff. The first geomorphological factor put into consideration is the drainage area. It is the projected area of watershed, also called the catchment area. Obviously, the bigger the area of catchment the larger the amount of runoff. However, the rate of runoff is largely dependant on the slope of the land, the drainage density, and the slope of the streams. A steep land slope with high drainage density and larger slope of stream will give a higher rate of runoff than those which have smaller slope and lower drainage density. The shape of the watershed is also a factor affecting the runoff. An elongated watershed will have a longer time of concentration than those with round shapes, and thus less discharge will take place.

With a three-dimensional concept in mind, the geomorphological factors

which can affect the amount of runoff are listed below:

- 1) The drainage area
- 2) The drainage density
- 3) The land slope
- 4) The main stream slope
- 5) The watershed shape

Evaluation of Geomorphological Factors

1) Drainage Area

Area of the watershed is directly measured from topographic maps with a planimeter. It is expressed in square miles.

2) Drainage Density (17)

Drainage density is defined as the total length of streams in the watershed divided by its total area, that is the length of streams per unit area of the watershed. It can be expressed as follows:

Fig. (3)

Drainage Density

$$D = \frac{L}{A}$$

Where

L = total length of streams

A = drainage area

The length of stream can be measured in miles by map measurement along the streams plotted in the topographic map. Since the area is expressed in square miles, the drainage density can be expressed as miles per square miles.

3) Land Slope (17)

Since the land slope is changing from place to place in a watershed, it is hard to find a quantitative value to represent the land slope of a

whole watershed. A new parameter introduced here to replace the land slope is the mean relief of land. The mean relief is defined as the total volume of land mass above the gaging station of a watershed divided by its projected area. This can be evaluated quantitatively by using the so-called hypsometric analysis. The hypsometric curve developed by Langbein and others (15) gives a dimensionless relationship between the horizontal cross-sectional drainage basin area and the elevation. The total volume of land mass can be calculated, and the mean relief also determined.

Fig. (4)

Figure (4) shows the elevation and its related cross-sectional drainage basin area. The elevation can be read from topographic maps, and its cross-sectional area can be measured by using the planimeter. By obtaining the H , h , A , a , from the topographic map, a hypsometric curve can be plotted in a dimensionless form as shown in Figure (5).

Fig. (5)

From the above hypsometric curve, the total volume of land mass and the mean relief can be determined. Since the area under the hypsometric curve can be easily measured, this can be expressed as,

$$\int_0^1 \frac{a}{A} d\left(\frac{h}{H'}\right) = \alpha$$

This can be rewritten as

$$\frac{1}{AH'} \int_0^1 a dh = \alpha$$

The total volume of land mass

$$V = \int_0^1 a dh = \alpha AH'$$

The mean relief

$$H = \frac{V}{A} = \alpha H'$$

4) Main Stream Slope

The slope of main stream can be obtained from the topographic map, but it is not the same throughout the whole stream. Usually, the upper streams are steeper, and the downstream reaches are flatter. The problem is how to get a mean slope to represent the main stream slope. It would not be sufficient just to find the slope of a straight line which connects the upper stream and the downstream. A method introduced by Taylor and Schwary (25) for determining mean slope of stream is shown as follows:

- (1) Divide stream into many sections, for each increment of stream length

$$t_i = \frac{L_i}{V_i}$$

Where

t_1 = travel time of flow through the increment of length

L_1 = increment of length

V_1 = average velocity of flow through the increment.

- (2) From Manning's formula

$$V = \frac{1.486}{n} R^{2/3} S^{1/2}$$

and for each length of increment $\frac{1.486}{n} R^{2/3}$ is determined and equal K ,

thus,

$$V_i = K_i S^{1/2}$$

- (3) total travel time

$$\begin{aligned} T &= t_1 + t_2 + t_3 + t_4 + \dots + t_n \\ &= \frac{L_1}{V_1} + \frac{L_2}{V_2} + \frac{L_3}{V_3} + \frac{L_4}{V_4} + \dots + \frac{L_n}{V_n} \\ &= \frac{L_1}{K_1 S_1^{1/2}} + \frac{L_2}{K_2 S_2^{1/2}} + \frac{L_3}{K_3 S_3^{1/2}} + \dots + \frac{L_n}{K_n S_n^{1/2}} \end{aligned}$$

- (4) If equal increments of length are used

$$L_1 = L_2 = L_3 \dots \dots = L_n = \frac{L}{n}$$

then,

$$T = \frac{L}{n} \left(\frac{1}{K_1 S_1^{1/2}} + \frac{1}{K_2 S_2^{1/2}} + \dots + \frac{1}{K_n S_n^{1/2}} \right)$$

- (5) Assuming a uniform channel having hydraulic elements equivalent to the average for the prototype channel, the travel time would be

$$T = \frac{L}{V}$$

and the velocity

$$V = K S^{1/2}$$

$$T = \frac{L}{K S^{1/2}}$$

where S = bed slope

Substitute this relation to

$$T = \frac{L}{n} \left(\frac{1}{K_1 S_1^{1/2}} + \frac{1}{K_2 S_2^{1/2}} + \dots + \frac{1}{K_n S_n^{1/2}} \right)$$

$$\frac{L}{K S^{1/2}} = \frac{L}{n} \left(\frac{1}{K_1 S_1^{1/2}} + \frac{1}{K_2 S_2^{1/2}} + \dots + \frac{1}{K_n S_n^{1/2}} \right)$$

$$S^{1/2} = \frac{n}{K \sqrt{\frac{1}{K_1 S_1^{1/2}} + \frac{1}{K_2 S_2^{1/2}} + \dots + \frac{1}{K_n S_n^{1/2}}}}$$

$$\text{if } \frac{K}{K_1} = \frac{K}{K_2} = \frac{K}{K_3} = \dots = \frac{K}{K_n} = 1$$

$$S = \left[\frac{n}{\left(\frac{1}{S_1^{1/2}} + \frac{1}{S_2^{1/2}} + \frac{1}{S_3^{1/2}} + \dots + \frac{1}{S_n^{1/2}} \right)} \right]^2$$

5) Watershed shape factor

The watershed shape factor is the factor that transforms the irregular shape of a watershed to a standard one which is circular. The length of longest waterway is used as parameter to determine the shape factor.

Fig. (6)

Figure (6) shows two watersheds with the same area but different shape. The former is an actual watershed, and the latter is an imaginary one.

AO is measured along the main stream, and A'O' is calculated from the known area of the actual watershed

$$A'O' = 2 \sqrt{\frac{\text{AREA OF ACTUAL WATERSHED}}{\pi}}$$

by assuming that this watershed is a circular one and A'O' is the diameter of the circle.

The shape factor is determined as $f = \frac{AO}{A'O'}$

Geomorphological Factors of Small Watersheds in Indiana

The following table (V) shows quantitatively the geomorphological factors of 16 small watersheds in Indiana.

Table V

Watershed Characteristics and 25-Year Annual Instantaneous
Peak Runoff of 16 Watersheds in Indiana

Watershed No.	25-Year Annual Instantaneous Peak Runoff Q cfs.	Watershed Characteristics				
		Area A sq. mi.	Mean Relief H ft.	Drainage Density D mi./sq. mi.	Shape Factor f	Main Stream Slope, $S \times 10^{-4}$
S-1	11,800	100	216	10.66	2.63	9.84×10^{-4}
S-2	12,900	88.8	270	7.38	3.00	20.83
S-5	2,950	78.7	101	6.57	1.75	9.05
S-6	3,300	62.9	110	8.00	1.12	21.10
S-7	1,630	93.0	79	5.10	1.47	8.29
1	880	115	65.4	3.35	1.41	2.64
6	17,000	179	195.2	7.88	2.12	13.40
7	10,500	109	86	7.02	1.94	10.39
9	21,500	156	250	9.76	2.85	10.68
10	19,000	120	237	11.20	2.18	9.90
11	13,500	257	181.5	13.95	2.87	2.95
12	8,700	228	99.8	13.20	1.93	5.43
14	13,300	188	195.8	10.47	1.35	6.21
15	22,300	129	231	8.70	2.56	10.16
18	4,800	125	84.7	4.50	1.91	6.05
19	760	102	66.1	3.16	1.93	7.93

The following is an example to show a complete calculation of geomorphological factors studied in this section:

Watershed S-1

(1) Area $A = 100$ sq. mi.

(2) Mean relief H

(i) Hypsometric curve

$A = 100$ sq. mi.

$H' = 400$ ft.

Fig. (7)

Area under the hypsometric curve

$$\mathcal{L} = \int_0^1 \frac{a}{A} d\left(\frac{h}{H'}\right) = 0.54$$

(ii) The total volume of land mass

$$V = \int_0^1 a \cdot H' = \mathcal{L} A H'$$

(iii) The mean relief

$$H = \frac{V}{A}$$

$$= \frac{\mathcal{L} A H'}{A} = \mathcal{L} H' = 0.54 \times 400 = 216'$$

(3) Drainage Density D

$A = 100$ sq. mi.

$$\sum L = 1066 \text{ mi}$$

$D = \text{Drainage Density}$

$$= \frac{\sum L}{A} = \frac{1066}{100} = 10.66 \text{ mi./sq. mi.}$$

(4) Main stream slope S

Divide the main stream into 16 equal sections, and calculate the slope of each section. Then, a weighted main stream slope can be computed. The calculation is shown in the following table.

Table VI

	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16
$S_i \times 5000$	9	12	6	10	10	7	6	5	5	5	5	8	4	2	2	2
$\frac{1}{S_i}$	556	416	832	500	500	715	832	1000	1000	1000	1000	626	1250	2500	2500	2500
$(\frac{1}{S_i})^{1/2}$	23.6	20.4	28.9	22.4	22.4	26.7	28.9	31.6	31.6	31.6	31.6	25.0	35.4	50.0	50.0	50.0

$$\sum \left(\frac{1}{S_i} \right)^{1/2} = 510.1$$

$$n = 16$$

$$\begin{aligned}
 S &= \left[\frac{n}{\sum \left(\frac{1}{S_i} \right)^{1/2}} \right]^2 \\
 &= \left(\frac{n}{\sum \left(\frac{1}{S_i} \right)^{1/2}} \right)^2 = \left(\frac{16}{510.1} \right)^2 \\
 &= 9.84 \times 10^{-4}
 \end{aligned}$$

(5) The shape factor, f

$$A = 100 \text{ sq. mi.}$$

$$A' O' = 11.24 \text{ mi.}$$

$$A O = 29.6 \text{ mi.}$$

$$f = \frac{A O}{A' O'} = \frac{29.6}{11.24} = 2.63$$

VI. MULTIPLE CORRELATION

Multiple correlation is a statistical method to find the relationship among one dependent variable and a number of independent variables. If a linear relationship exists, then the method of fitting is to make the sum of the squares of the deviations of actual observations from the theoretical linear relation to be a minimum. This is so-called the method of least square.

A. General Formula for Multiple Correlation (26)

If y_i = i 'th observed value of y

$\hat{y}_i$ = estimated y_i

$$= a + b_1 x_{1i} + b_2 x_{2i} + b_3 x_{3i} + \dots + b_k x_{ki}$$

k = number of independent variables

n = number of observations

$a, b_1, b_2, b_3, b_4, \dots, b_k$ are constants obtained by minimizing the sum of squares.

Then

$$Q = \sum (y_i - \hat{y}_i)^2$$

$$= \sum (y_i - a - b_1 x_{1i} - b_2 x_{2i} - b_3 x_{3i} - \dots - b_k x_{ki})^2$$

Taking the derivative of Q with respect to $a, b_1, b_2, b_3, \dots, b_k$, and equating them to zero.

$$\frac{\partial Q}{\partial a} = -2(\sum y_i - na - b_1 \sum x_{1i} - b_2 \sum x_{2i} - \dots - b_k \sum x_{ki})$$

$$\frac{\partial Q}{\partial b_1} = -2(\sum x_{1i} y_i - a \sum x_{1i} - b_1 \sum x_{1i}^2 - b_2 \sum x_{1i} x_{2i} - \dots - b_k \sum x_{1i} x_{ki}) = 0$$

$$\frac{\partial Q}{\partial b_k} = -2(\sum x_{ki} y_i - a \sum x_{ki} - b_1 \sum x_{ki} x_{1i} - \dots - b_k \sum x_{ki}^2) = 0$$

Then, we have,

$$a + b_1 \sum x_{1i} + b_2 \sum x_{2i} + \dots + b_k \sum x_{ki} = \sum y_i$$

$$\begin{array}{rcl} a \sum x_{1i} + b_1 \sum x_{1i}^2 + b_2 \sum x_{1i} x_{2i} + \dots + b_k \sum x_{1i} x_{ki} & = & \sum x_{1i} y_i \\ \vdots & & \vdots \\ a \sum x_{ki} + b_1 \sum x_{ki} x_{1i} + b_2 \sum x_{ki} x_{2i} + \dots + b_k \sum x_{ki}^2 & = & \sum x_{ki} y_i \end{array}$$

The first equation can be written as

$$a = \bar{y} - b_1 \bar{x}_1 - b_2 \bar{x}_2 - b_3 \bar{x}_3 - \dots - b_k \bar{x}_k$$

substituting into other equations, we obtain:

$$(\bar{y} - b_1 \bar{x}_1 - b_2 \bar{x}_2 - \dots - b_k \bar{x}_k) \sum x_{1i} + b_1 \sum x_{1i}^2 + b_2 \sum x_{1i} x_{2i} + \dots + b_k \sum x_{1i} x_{ki} = \sum x_{1i} y_i$$

$$(\bar{y} - b_1 \bar{x}_1 - b_2 \bar{x}_2 - \dots - b_k \bar{x}_k) \sum x_{2i} + b_1 \sum x_{2i} x_{1i} + b_2 \sum x_{2i}^2 + \dots + b_k \sum x_{2i} x_{ki} = \sum x_{2i} y_i$$

$$(\bar{y} - b_1 \bar{x}_1 - b_2 \bar{x}_2 - \dots - b_k \bar{x}_k) \sum x_{ki} + b_1 \sum x_{ki} x_{1i} + b_2 \sum x_{ki} x_{2i} + \dots + b_k \sum x_{ki}^2 = \sum x_{ki} y_i$$

By rearranging, these equations become:

$$b_1 \sum x_{1i} (x_{1i} - \bar{x}_1) + b_2 \sum x_{1i} (x_{2i} - \bar{x}_2) + \dots + b_k \sum x_{1i} (x_{ki} - \bar{x}_k) = \sum x_{1i} (y_i - \bar{y})$$

$$\begin{aligned}
 b_1 \sum X_{2i} (X_{1i} - \bar{X}_1) + b_2 \sum X_{2i} (X_{2i} - \bar{X}_2) + \dots + b_k \sum X_{2i} (X_{ki} - \bar{X}_k) &= \sum X_{2i} (y_i - \bar{y}) \\
 \vdots &\vdots \\
 b_1 \sum X_{ki} (X_{1i} - \bar{X}_1) + b_2 \sum X_{ki} (X_{2i} - \bar{X}_2) + \dots + b_k \sum X_{ki} (X_{ki} - \bar{X}_k) &= \sum X_{ki} (y_i - \bar{y})
 \end{aligned}$$

by introducing new symbols

$$S(X_1^2) = \sum X_{1i}^2 - \frac{(\sum X_{1i})^2}{n} = \sum X_{1i} (X_{1i} - \bar{X}_1)$$

$$\begin{aligned}
 S(X_1 X_2) &= \sum X_{1i} X_{2i} - \frac{\sum X_{1i} \sum X_{2i}}{n} = \sum X_{1i} (X_{2i} - \bar{X}_2) \\
 &= \sum X_{2i} (X_{1i} - \bar{X}_1)
 \end{aligned}$$

$$\begin{aligned}
 S(X_1 X_k) &= \sum X_{1i} X_{ki} - \frac{\sum X_{1i} \sum X_{ki}}{n} = \sum X_{1i} (X_{ki} - \bar{X}_k) \\
 &= \sum X_{ki} (X_{1i} - \bar{X}_1)
 \end{aligned}$$

Then, the general equations of multiple correlations are as follows:

$$a = \bar{y} - b_1 \bar{X}_1 - b_2 \bar{X}_2 - b_3 \bar{X}_3 - \dots - b_k \bar{X}_k$$

$$b_1 S(X_1^2) + b_2 S(X_1 X_2) + b_3 S(X_1 X_3) + \dots + b_k S(X_1 X_k) = S(X_1 y)$$

$$b_1 S(X_1 X_2) + b_2 S(X_2^2) + b_3 S(X_2 X_3) + \dots + b_k S(X_2 X_k) = S(X_2 y)$$

$$b_1 S(X_3 X_1) + b_2 S(X_3 X_2) + b_3 S(X_3^2) + \dots + b_k S(X_3 X_k) = S(X_3 y)$$

$$b_1 S(X_k X_1) + b_2 S(X_k X_2) + b_3 S(X_k X_3) + \dots + b_k S(X_k^2) = S(X_k y)$$

B. General Formula for Estimating of Variance

The estimate of the variance

$$S_{y_1, x_1, x_2, x_3, \dots, x_n} = \frac{\sum (y_i - \hat{y}_i)^2}{n - (k+1)}$$

where $n - (k+1)$ = degree of freedom.

The sum of the square of the deviations

$$\begin{aligned} \sum (y_i - \hat{y}_i)^2 &= \sum (y_i - a - b_1 x_{1i} - b_2 x_{2i} - \dots - b_k x_{ki})^2 \\ &= \sum y_i (y_i - a - b_1 x_{1i} - b_2 x_{2i} - b_3 x_{3i} - \dots - b_k x_{ki}) \\ &\quad - a \sum (y_i - a - b_1 x_{1i} - b_2 x_{2i} - b_3 x_{3i} - \dots - b_k x_{ki}) \\ &\quad - b_1 \sum x_{1i} (y_i - a - b_1 x_{1i} - b_2 x_{2i} - b_3 x_{3i} - \dots - b_k x_{ki}) \\ &\quad \vdots \\ &\quad - b_k \sum x_{ki} (y_i - a - b_1 x_{1i} - b_2 x_{2i} - b_3 x_{3i} - \dots - b_k x_{ki}) \end{aligned}$$

All terms, except the first one, in the right side are equal to zero since

$$\frac{\partial Q}{\partial a}, \frac{\partial Q}{\partial b_1}, \frac{\partial Q}{\partial b_2}, \dots, \frac{\partial Q}{\partial b_n} = 0$$

Then, the sum of square of the deviations can be expressed as

$$\sum (y_i - \hat{y}_i)^2 = \sum y_i (y_i - a - b_1 x_{1i} - b_2 x_{2i} - \dots - b_k x_{ki})$$

Substitute the relation

$$a = \bar{y} - b_1 \bar{x}_1 - b_2 \bar{x}_2 - \dots - b_K \bar{x}_K$$

Thus,

$$\begin{aligned} \sum (y_i - \hat{y}_i)^2 &= \sum y_i \left[(y_i - \bar{y}) - b_1 (x_{1i} - \bar{x}_1) - b_2 (x_{2i} - \bar{x}_2) - \right. \\ &\quad \left. \dots - b_K (x_{Ki} - \bar{x}_K) \right] \\ &= S(y^2) - b_1 S(x_1 y) - b_2 S(x_2 y) - \dots - b_K S(x_K y). \end{aligned}$$

Hence, the general formula for estimating of variance

$$S_{y, x_1, x_2, \dots, x_K} = \frac{S(y^2) - b_1 S(x_1 y) - b_2 S(x_2 y) - \dots - b_K S(x_K y)}{n - (K+1)}$$

C. Results of Correlations between Peak Runoff and Geomorphological Factors of Small Watersheds in Indiana

The following Table VII shows the results of correlation between the 25-years instantaneous peak runoff and the geomorphological factors of small watersheds in Indiana.

Table VII - Multiple Correlation between Predicted 25-Years
Instantaneous Peak Runoff and Geomorphological Factors

Runoff	Geomorphological Factors	Regression Formula	*Standard Deviation
Q	A f	$Q = 30.29 A^{0.8471} f^{1.9245}$	0.394
	A S	$Q = 0.0005716 A^{2.6792} S^{1.5829}$	0.305
	A D	$Q = 47.23 A^{0.1994} D^{1.9847}$	0.296
	A H	$Q = 0.03993 A^{0.7229} H^{1.7396}$	0.234
	A D S	$Q = 16.85 A^{1.4179} D^{1.3292} S^{0.9985}$	0.2375
	A D H	$Q = 0.3432 A^{0.3925} H^{1.2732} D^{0.8601}$	0.213
	A D f H	$Q = 0.8164 A^{0.3036} H^{1.0936} D^{0.9409} S^{0.4098}$	0.218
	A H D S f	$Q = 0.05363 A^{0.9715} H^{0.7844} D^{0.8234} f^{0.4160} S^{0.5901}$	0.190

* in Log-Unit

It is quite interesting to see from the table that the larger the number of geomorphological factors put into multiple correlation, the smaller the standard deviation. That is, the more geomorphological factors we considered, the more accurate predicted runoff can be expected.

The last regression formula in Table VII

$$Q = 0.05363 A^{0.9715} H^{0.7844} D^{0.8234} f^{0.4160} S^{0.5901}$$

with the least deviation is of course the best expression obtained from multiple correlation. If all the geomorphological factors with their

powers together are combined as a basin characteristic, B ,

$$B^x = A^{0.9715} H^{0.7344} D^{0.8234} F^{0.4160} S^{0.5901}$$

then, the regression formula can be expressed as

$$Q = C B^x \quad C = \text{constant}$$

which is the equation of a straight line in log-log paper. Therefore, this regression formula was plotted, and a theoretical straight line was fitted by the least square method.

Figure (8) shows the result of plotting the basin characteristic against the 25-years instantaneous annual peak runoff. It seems that the deviation of actual observation to the theoretical line is small, and a straight line can be fitted quite well. A 95% confidence limit (26, 27) was calculated and plotted. That means the probability of any observation falling between these limits is 95%.

D. Construction of Correlation Chart for Predicting Future Flood

The regression formula obtained from multiple correlation can be used to calculate the flood discharge if the geomorphological factors are known, and also the Figure (8) is a means that can be used to find the flood discharge. This requires knowing the different power of each geomorphological factor before the flood discharge can be obtained.

There is a somewhat more convenient way for the design engineer to convert the regression formula to a chart from which the predicting flood discharge can be read directly simply by entering the known geomorphological factors into it.

The way to convert the regression formula to a chart is as follows:

- 1) Since the regression formula has the form

$$Q = K A^a B^b C^c D^d E^e$$

Figure 8
Regression Line for Theoretical 25 Years Instantaneous Peak
Discharge Against Basin Characteristics

FIG. 8

2) It is assumed that

$$M_1 = A^a B^b$$

$$M_2 = A^a B^b C^c = M_1 C^c$$

$$M_3 = A^a B^b C^c D^d = M_2 D^d$$

$$Q = K A^a B^b C^c D^d E^e = K M_3 E^e$$

3) The above equations can be plotted when B, C, D, E, are used as parameters. They are shown as follows:

Fig. (9)

4) The above figures can be combined as shown below:

Fig. (10)

As shown in Figures (10), the A^B is in arithmetic scale. This can be transformed to a corresponding A^B scale such that the area A can be introduced directly into the chart and join with other factors to find the flood discharge.

According to the above procedures the regression formula,

$$Q = 0.05363 A^{0.9715} H^{0.7844} D^{0.8234} P^{0.4160} S^{0.5901}$$

can be plotted as a chart for the convenient use of the design engineer to predict the 25 years instantaneous peak flood discharge for highway drainage structure design in Indiana. There are two charts shown in

Figure (11) and Figure (12). Figure (11) is prepared for the watershed less than 100 square miles and Figure (12) is prepared for the watershed less than 300 square miles.

Two examples are presented as to show how to use the chart to obtain the 25 year instantaneous flood discharge.

a) Watershed No.		S-6
Watershed area (A)		62.9 sq. mi.
Mean relief (H)		110 feet
Drainage density (D)		6 mi./sq.mi.
Shape factor (f)		1.12
Mean stream slope (S)		21.10×10^{-4}

It is shown as the dotted line in Figure (11); the flood discharge is read from the chart directly as 3,750 cfs., while the flood predicted by the frequency study is 3,300 cfs.

b) Watershed No.		S-1
Watershed area (A)		100 sq. mi.
Mean relief (H)		216 feet
Drainage density (D)		10.66 mi./sq.mi.
Shape factor (f)		2.63
Main stream slope (S)		9.84×10^{-4}

This is shown as the dotted line in Figure (12); the flood discharge is read from the chart directly as 12,450 cfs., while the flood predicted by the frequency study is 11,800 cfs.

Figure 11
25 Years Instantaneous Annual Peak Runoff for
Watersheds Less Than 100 Square Miles in Indiana

FIG. II

Figure 12

25 Years Instantaneous Annual Peak Runoff for
Watersheds Less Than 300 Square Miles in Indiana

FIG. 12

VII. CONCLUSIONS AND DISCUSSION

1. Table IV gives the predicted flood of 25, 50, 75, 100 years frequency for 32 watersheds in Indiana.

2. Figure (8) is a plot of the regression formula

$$Q = 0.05363 A^{0.9715} H^{0.7844} D^{0.3234} f^{0.4160} S^{0.5901}.$$

This can be used to find the 25-year peak discharge if all the geomorphological factors are known and to have the basin characteristic calculated.

3. Figure (11) and Figure (12) are the charts used to find the 25-year peak discharge for the watershed less than 100 square miles and 300 square miles respectively. From the two examples shown in the last section, it is learned that the discharge read from the chart is very close to the discharge predicted from statistical frequency analysis.

4. The upper and lower confidence limits in Figure (8) mean that the probability of any observation falling into this interval is 95%. It may be safer to use the upper limit line as a base for design. Then, it will have 97.5% confidence; that is, there is only 2.5% probability that the flood will be greater than the predicted one within a certain frequency of 25 years.

5. Therefore, to use the upper limit line provides safer design.

The procedure to find the safe design discharge is as follows:

- a. Find the watershed characteristics: A, H, D, f, S.
- b. From Figure (11) or Figure (12), read the 25-years instantaneous annual peak discharge, Q, in cfs.

- c. Go back to Figure (8); point out the discharge in theoretical line, and go straight up to the point of upper limit; then, read the corresponding discharge.
6. This report contains only the first two approaches on runoff study, the statistical and geomorphological study. It is based on the assumption that the rainfall pattern is homogeneous over this area, and the geomorphological factor turns out to be the only factor affecting the amount of runoff.
7. The hydrograph study will consider the different rainfall patterns and their relations to their producing runoff. The theoretical instantaneous unit graph and the way to get the synthetic hydrograph are the main purposes for the next step in the study.

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