Technical Report

ESTIMATING DESIGN FLOODS FROM EXTREME RAINFALL
(With Special Reference to SmallWatersheds in Western U.S.A.)

by

Frederick C. Bell

Prepared for

Bureau of Land Management
U.S. Department of the Interior
as one of the reports
on investigations conducted under
Contract No. 14-11-0008-0590-62

Colorado State University
Civil Engineering Department
Fort Collins, Colorado

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SUMMARY

There are distinct differences between the estimation of specific floods from data on specific rainfall events and the estimation of design or representative floods from rainfall statistics. The latter should be regarded as a more generalized procedure in which high accuracy cannot be expected. Many of the physical details of specific events are irrelevant for the estimation of representative events.

It is shown that a single parameter is sufficient to express the time-distributing characteristics of a watershed for design purposes. The suggested parameter is the representative lag which is closely related to the volume/peak ratio.

For small watersheds in western U.S.A., it is demonstrated that the same return period may be assigned to the design flood and the corresponding extreme rainfall. This finding is not expected to apply to all climatic situations but it may be a reasonable assumption in the absence of any other information.

The rational-loss rate method is suggested for estimating extreme floods from extreme rainfall because of its simplicity,

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flexibility and consistency with the requirements and limitations of the problem. However, it does not give very satisfactory reproductions of the 10 year floods on the test watersheds and cannot be strongly supported by this performance. The estimation of median loss rates is the weakest aspect of the rational-loss rate method and further investigation of this particular topic seems justified.

ACKNOWLEDGEMENTS

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1. INTRODUCTION

When possible, design floods should be estimated directly from streamflow data or from a combination of streamflow and rainfall data. Neither of these is usually possible for small watersheds (less than, say, 50 square miles) because only a small percentage of such watersheds has been gaged. In most cases it is necessary to estimate small watershed design floods either from extreme rainfall data or from regional studies of the type suggested by the U.S. Geological Survey.[1]

There are numerous methods available for estimating design floods from extreme rainfall data, for example, the traditional "rational" formula, the U.S. Soil Conservation Procedure[2], the hydrograph-loss rate procedure (see section 5.1) and the TMP method[3]. Some of these have been reviewed and compared by Chow[4], and also by Hiemstra and Reich[5] whose findings suggest that no available method is very reliable.

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1. Dalrymple, Tate, Flood Frequency Analyses, U.S. Geological Survey Water Supply Paper, 1543-A, 1960.
2. U.S. Department of Agriculture, National Engineering Handbook, Hydrology, Washington D.C., 1956.
3. Reich, B.M. and L.A.V. Hiemstra, Tacitly Maximized Small Watershed Flood Estimates, Journal of the Hydraulics Division, ASCE, Vol. 91, No. HY3, Proc. Paper 4339, May 1965 and Vol. 92, HY4, July 1966.
4. Chow, V.T., Hydrologic Determination of Waterway Areas for the Design of Drainage Structures, University of Illinois Engineering Experiment Station Bulletin No. 462, 1962.
5. Hiemstra, L.A.V. and B.M. Reich, Engineering Judgment and Small Area Flood Peaks, Hydrology Paper No. 19, Colorado State University, Fort Collins, Colorado, April 1967.

It is the purpose of this study to examine various aspects of design flood estimation, using theoretical physical considerations coupled with analyses of data from sample watersheds in western U.S.A. It is hoped that the results of the study will contribute towards the development of better procedures than are available at present.

2. THE GENERAL PROBLEM

There is a common failure to discriminate between the estimation of specific flood events and the estimation of design or representative flood events. Although these are closely related in some aspects they are rather different problems, each with its own special features.

2.1 Specific Floods and Design Floods

Short-term flood forecasting is a typical example of the estimation of specific flood events. In this problem it may be necessary to forecast the peak flood levels and times of occurrence resulting from heavy rain that has just fallen, perhaps for the purpose of evacuating a threatened community or for the operation of a major reservoir. Successful estimates usually involve detailed physical considerations of the prevailing conditions such as the rainfall intensities, soil moisture and other factors that may influence the particular flood. Statistical or probabilistic considerations do not play a major role in these procedures although they are quite useful in the efficient specification of some highly variable factors and also in assessing the likely errors in the estimates.

Design floods are hypothetical or typical events that represent rare occurrences. The degrees of "rareness" of these occurrences may be expressed by their probabilities or return periods, which seems necessary if they are to be given any quantitative significance.

Design floods need not correspond with any specific events nor any specific times as they are essentially average or maximum values that may be expected over very long periods. The estimation of design floods should therefore be regarded as a statistical or probabilistic procedure, in contrast to the estimation of specific floods which is mainly deterministic.

Recorded specific floods are sometimes adopted for design purposes, usually with modifications such as arbitrary increases in magnitude. Relatively elaborate techniques for estimating specific floods are also used to estimate design floods by assuming critical patterns and quantities of rainfall, minimum infiltration rates and so on. In most of these procedures the calculated design floods have unknown return periods because no considerations are given to the probabilities and joint probabilities of the adopted conditions. The results consequently have little quantitative significance and under such circumstances it is difficult to justify much computational complexity.

More generalized methods are preferable for estimating design floods corresponding with given return periods. These methods should be concerned with the broad hydrologic conditions appropriate for the return periods, rather than with the physical details of specific events.

The above points may seem fairly obvious but much recent work in this field suggests that they are not widely appreciated. For example, the suitability of the rational formula for design flood estimation is commonly criticized because it fails to account for detailed differences between individual floods (such as effective rainfall durations). However, it is this very characteristic of generality that makes the rational formula more suitable for design flood estimation than most of the other recommended methods. Similar examples may be seen in recent evaluations of design flood procedures by their "errors" in reproducing observed specific floods without regard to their return periods. Such evaluations are misleading because these "errors" and the associated standard deviations are virtually meaningless unless both the estimated and observed floods can be linked to the same return periods.

2.2 Is it Possible to Estimate Return Periods Accurately?

Even when long records of streamflow data are available for a given watershed it is not possible to estimate accurately the return periods of extreme floods. This may be shown by table 1 which gives the 68% confidence intervals for estimates of various return periods when 25 years and 100 years of records are available.

The values of table 1 may be calculated from data presented in U.S. Geological Survey Water Supply Paper 1543-A[1], assuming that the flood peaks conform with the probability distribution suggested by Gumbel[6]. Even if this assumption is in error (up to a moderate degree) the general order of accuracy indicated by table 1 should still apply.

It may be seen, therefore, that the "true" return period of an estimated 100 year flood is likely to be as low as 40 years or as high as 250 years when the record of streamflow is particularly long, viz. 100 years. The situation may be considerably worse when greater return periods and shorter records are involved. In the case of small watersheds it is not often that one obtains records longer than about 25 years, and under these circumstances the assigning of return periods to rare floods is usually no more than a very rough approximation.

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6. Gumbel, E. J., Statistics of Extremes, Columbia University Press, New York, 1960.

The above refers to the direct estimation of extreme floods from streamflow records but similar results may be expected for the estimation of extreme rainfall from rainfall records. It seems impossible, therefore, to obtain good estimates of design floods from rainfall data (within our present technology) because all the methods attempting this involve sources of error that are additional to those already mentioned. However, it is better to have a rough idea of the flood corresponding to the required return period and possibly make allowances for the wide margin of uncertainty, than to base one's design on an arbitrary flood of completely unknown frequency.

Systematic allowances for the margin of uncertainty in design criteria would be a good topic for further investigation. The "risk of failure" concept is already well established but this only takes into account the return period of the design flood and the desired "life" of the structure (see Gilman[7], page 9-59). Additional risks are incurred by the uncertainties in estimating the design flood and it should be fairly straight-forward matter to develop simple allowances for this factor.

2.3 The Search for an Efficient Method

Although it seems impossible to obtain very reliable estimates of design floods from extreme rainfall it is necessary to use such methods because no better alternatives are available (in the absence of long streamflow records). Large errors are likely to arise from the sampling limitations of the rainfall data (as discussed previously), and, relative to these, some of the possible computational refinements would make insignificant differences to the required estimates. From

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7. Gilman, C. S., Rainfall, Section 9 in Applied Hydrology by V. T. Chow, McGraw-Hill, 1964.

the point of view of estimation efficiency, therefore, only simple, generalized relationships between rainfall and floods are worth considering. Fortunately, this is compatible with the previous contention that generalized relationships are desirable for estimating representative events, as compared with the detailed relationships required for estimating specific events.

Keeping the above issues in mind, the problem of estimating the 100 year flood (for example) from extreme rainfall data, may be resolved into the following three parts:

(a) What is the most appropriate or typical frequency, depth and duration of rainfall associated with the 100 year flood?

(b) What are the most appropriate abstractions from this rainfall to account for infiltration and similar "loss" factors?

(c) What is the most appropriate hydrograph or time distribution of runoff associated with (a) and (b)?

The answers to questions (a) and (b) depend to a certain extent on the time-distributing characteristics of the watershed which are the subject of question (c). It is therefore advantageous to examine (c) first, as will be done in section 4. Before this, however, the sample watersheds and floods will be described briefly.

3. THE SAMPLE WATERSHEDS AND FLOODS

A large amount of flood data is being assembled at Colorado State University as part of the Small Watershed Hydrology Program. This provided the main source of data for the analyses to be described. Additional information was obtained from publications of the Agricultural

Research Service[8], the U. S. Geological Survey[9] and California Department of Water Resources[10], as summarized in tables 2 and 3 which list the relevant particulars of all watersheds studied.

It was decided to give special attention to western U.S.A. where there are certain flood-producing conditions that have proved troublesome in other studies[5, 11, 12].

3.1 Flood-Producing Conditions for the Sample Watersheds

The climatic factors associated with floods in western U.S.A. are so variable and complex that their individual effects cannot be readily identified or separated when one attempts to analyse data from the area as a whole. A first step towards overcoming this difficulty is an appropriate grouping of watersheds so that conditions within each group are not too heterogeneous. For this purpose, four different types of flood-producing conditions may be distinguished in the area of interest, viz:

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8. U.S. Department of Agriculture, Hydrologic Data for Experimental Agricultural Watersheds, Miscellaneous Publication No. 945, 1963.

9. U.S. Geological Survey, Magnitude and Frequency of Floods in the United States, Parts 6A to 14, Water Supply Papers 1679-1694, 1966.

10. Ray, H. A., Floods From Small Drainage Areas in California, Report of U. S. Geological Survey 2nd California Department of Water Resources, May 1965.

11. Om Kar, Songthara, Hydrograph Rise Times, Colorado State University Publication, CET 66-67S033, Fort Collins, Colorado, June 1967.

12. Voytik, Andrew, Runoff Predictions from Arid Regions, Colorado State University Publication, CET 66-67AV30, Fort Collins, Colorado, June 1967.

(a) Extreme floods caused mainly by winter storms of relatively long duration (12 hours to 6 days). The term "extreme" is intended to apply to events with return periods exceeding 10 years.

(b) Extreme floods caused mainly by summer thunderstorms of short duration (1/2 to 6 hours).

(c) Extreme floods caused by storms of long duration and short duration, seasonal effects being less pronounced than for (a) and (b).

(d) Extreme floods that usually include large volumes of snowmelt.

The above are referred to as "flood groups" and their approximate geographical distributions are shown in figure 1 which is based on analyses of the available data and various references[8, 9, 13]. It was decided to exclude the snowmelt floods from the study because these require different treatments and data to (a), (b), and (c) which will be called "winter", "summer" and "mixed" flood groups respectively.

The flood groups differ not only in rainfall characteristics but also in vegetation and topography. Most watersheds in (a) have good pasture or forest covers and tend to have moderate to steep topography. They should also have relatively small storm loss rates because the floods occur in winter when evapotranspiration is low and soil moisture is high.

The summer flood group, (b), includes most of the arid and semi-arid parts of U.S.A. Watersheds in these regions have poorer vegetation, flatter slopes and higher storm loss rates. Watersheds in the mixed flood group are generally somewhere between (a) and (b) in most of these factors.

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13. Hoyt, W. G. and W. B. Langbein, <u>Floods</u>, Princeton University Press, New Jersey, 1955, 469 pages.

3.2 Watersheds Used for Flood Analyses

The initial selection involved all watersheds in the Colorado State University data collection that fulfill the following conditions:

(a) located within the three main flood groups of figure 1,

(b) having at least 5 flood events with complete rainfall and streamflow data.

As this did not provide a sufficiently large sample, some additional watersheds having only 3 and 4 flood events were added from the Colorado State University data collection and the other sources previously mentioned[8, 9, 10]. The locations of the complete selection are shown in figure 2 and their general particulars are listed in table 2. It should be noted that each flood group has about the same number of watersheds and a similar distribution of watershed sizes.

3.3 Watersheds Used for Testing Conclusions

A different set of watersheds was selected for testing the conclusions of the studies. The main requirement in this selection was a long period of streamflow data so that reasonable, direct estimates of extreme floods could be obtained. In this regard, only 20 watersheds under 50 square miles could be found with more than 20 years of records. Their particulars are listed in table 3 and their locations are shown in figure 2.

The test watersheds are not completely comparable with the watersheds selected for the main analyses because they are generally larger and not so evenly distributed amongst the three flood groups. These differences do not seem likely to be an important source of bias in testing the conclusions.

3.4 Difficulties with Small Samples

The recorded flood events for each selected watershed may be regarded as a sample of the flood characteristics of that watershed.

The parameters derived from the samples provide estimates of the required flood characteristics, and the mean values of the various factors would normally be the main parameters involved. However, when the samples are very small and there are high degrees of variability, the mean is often a poor representative value because it is affected considerably by erratic "outliers". Under these conditions the median is a more stable statistic and it will therefore be used instead of the mean for a number of aspects of this study.

The small size of each sample of floods should also be regarded as an important contributor to the deviations that may be expected in the relationships to be derived.

3.5 Other Data Limitations

In section 2.2 it was shown that hydrologic frequency of statistical data has a low accuracy due to sampling limitations, particularly for extremes. The situation is not much better for the basic records of specific hydrologic events.

There are several sources of error in the measurement of streamflows and these are particularly significant in large floods. Errors of the order of $\pm$ 10% would not be surprising for many of the flood peaks used in this analysis.

Most of the sample data has come from recording instruments operated by clockwork mechanisms that are attended weekly. Gains or losses of 10 minutes per week are considered quite reasonable for such instruments and time errors of this magnitude may be expected, especially when relating rainfall times to runoff times.

The least accurate part of the basic data, however, is the watershed rainfall. In most cases the volume of rainfall over the entire watershed must be estimated from one or two station records which represent a minute sample of the total area. For small watersheds the resulting errors may vary from a few percent in steady, uniformly distributed rain up to 50% or more in "cloudburst" or local convective rain which is characterized by its space-time concentrations. The latter type of rain is particularly important in these studies, even when the flood-producing storms are long-duration, winter occurrences.

The parameters of any individual flood must be regarded as very approximate if their derivation is strongly dependent on the calculated watershed rainfall.

4. TIME-DISTRIBUTING CHARACTERISTICS OF WATERSHEDS

The fundamental questions to be answered in this section are:

(a) How many parameters are needed to efficiently describe the time-distributing characteristics of a watershed?

(b) Are the same parameters appropriate for both common and rare floods? If not, what is their relationship?

(c) What is the best way of estimating these parameters for a watershed with no streamflow records?

4.1 Hydrograph Analysis

The analysis of hydrograph shapes has commanded an enormous amount of attention from engineers and mathematically-oriented hydrologists over the past few decades. Unfortunately most of the emphasis has been on the mathematics of the data rather than on the physics of the phenomena and consequently there have been few results of real hydrological significance.

The concept of the unit hydrograph continues to play an important role in practical hydrology because it is readily understood and seems reasonable from the physical point of view. Applications of the concept are fairly straight-forward, particularly with high-speed computer techniques, and the calculations provide a satisfying, professional-type procedure. For estimates on ungaged watersheds it is often possible to use synthetic and dimensionless unit hydrographs that are described in the standard textbooks.

If required, a greater measure of sophistication appears to be available in the more advanced instantaneous unit hydrograph concept which can be regarded as a convolution integral or kernal function. This provides a wide scope for many interesting and erudite mathematical exercises.

Despite all the above developments, any estimates based on the unit hydrograph idea can be no better than rough approximations, whether convolution integrals and high-speed computers are used or not. Unit hydrographs do not represent a physically sound relationship between rainfall and runoff, as may be demonstrated by elementary hydraulic principles[14], although the approximation may be close enough in many circumstances. Nevertheless, some real refinements are possible by allowing for the "non-linearity" in various ways. Two practical examples of this are (a) the use of different unit hydrographs for different classes of storms and (b) making systematic adjustment to the estimated peak values, based on "trend"[15].

There are several methods of relating unit hydrographs to the inflow-outflow functions of idealized storage systems. These are supposed to demonstrate the physical significance of the unit hydrograph principle but most of the proposed storage systems are too complicated or artificial to relate to measurable watershed parameters in practical situations. There appear to be some advantages, however, in simulating watersheds with simple storage systems that represent different stages of the flow, such as the slower flow of the "land

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14. Johnson, D. and W. P. Cross, Elements of Applied Hydrology, Ronald Press Company, New York, 275 pages, 1949.

15. Body, D. N., Significance of Rainfall Intensity in Applications of the Unitgraph Method, Journal of Institution of Engineers, Australia, Vol. 34, No. 1-2, January-February 1962.

phase" and the faster flow of the "channel phase". These model watersheds have the following features:

(a) Fewer parameters than unit hydrographs.

(b) The parameters are easier to derive from streamflow data than those of unit hydrographs.

(c) The parameters may be directly associated with physical aspects of real watersheds. They therefore have good prospects of being related to measurable physical characteristics with a minimum of empiricism[16].

(d) No restrictions are imposed on the mathematic form of the supply or inflow function, (e.g., unit hydrographs imply a constant inflow over the unit period).

(e) Applications involve only simple calculations that do not normally require high-speed computers.

A typical example of a simple watershed storage model has been described recently by Ho[17]. This has 3 parameters representing the "delay times" of (a) direct or surface runoff in the land phase, (b) indirect or subsurface runoff in the land phase, and (c) channel flow.

Somewhat similar to the storage approach are mathematical models of watersheds derived by assuming various hydraulic mechanisms

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16. Bell, F. C., Improved Techniques for Estimating Runoff with Brief Records, Water Research Laboratory Report, University of N.S.W., 1967, (in press).

17. Ho, Yu Bing, Hydrograph Recessions, Colorado State University Publication, CET 66-67YH39, Fort Collins, Colorado, June 1967.

of runoff[18, 19]. These could become very useful and logical methods if simplified or streamlined for practical problems.

Many other approaches have been proposed with different types of mathematical functions to describe the general hydrograph shape. Most of these are highly empirical and consequently difficult to relate satisfactorily to rainfall and watershed characteristics, but some have significant advantages for particular purposes.

While the mathematics of hydrographs continue to be a large and attractive topic for investigation, two important problems in this field remain virtually untouched. These are:

(a) The estimation of the supply or inflow function from rainfall data, which can only be done, in general, when the loss rates approach zero. This seems to be necessary for the proper testing of methods of hydrograph analysis but the issue is usually obscured by an emphasis on other factors.

(b) The separation of hydrographs into components of surface runoff, interflow, base flow, etc., which is usually considered necessary for flood analyses. Contrary to textbook assurances, different methods of separation can make quite large differences in estimated flood values[20].

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18. Wooding, R. A., The Catchment-Stream Problem, Journal of Hydrology, Vol. III, No. 3/4, 1965.

19. Machmeier, R. E., The Effects of Runoff Supply Rate and Duration on Hydrographs for a Mathematical Watershed Model, Paper presented at ASCE Hydraulics Division Annual Conference, Madison, Wisconsin, August 1966.

20. Rangana, G., Methods of Base Flow Separation, M. Tech. Thesis, University of N.S.W., Australia, 1961.

Some recent attempts to deal with these problems have been described elsewhere by the author[16, 21].

The above survey has been concerned with details of hydrograph analysis that are important in estimating specific flood events. Before returning to the generalized conditions of design floods it is desirable to examine some of the details more closely.

4.2 Effects of Supply Rate on Hydrograph Peaks

The term "supply rate" is applied to the net rainfall rate after abstractions have been made for infiltration, interception and similar losses. It is convenient to examine the ratio of the hydrograph peak (q) to the average supply rate (i) assuming, initially, that the supply rate is approximately uniform. According to the linearity principle of unitgraph theory[22], the ratio q/i should be a constant for a given duration of supply, D, i.e. it should be independent of the magnitude of i.

In a very thorough analysis[19], Machmeier has derived a theoretical relationship between the ratio q/i and i based on hydraulic considerations. This shows the ratio increasing rapidly with i at small values of i and increasing slowly at large values of i. The magnitudes of these effects appear to agree fairly well with the field data analysed by Body[15], Sugawara[23] and the author[16].

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21. Bell, F. C., An Alternative Physical Approach to Watershed Analysis and Streamflow Estimation, Paper to be presented I.A.S.H. International Hydrology Symposium, Fort Collins, Colorado, Steptember 1967.

22. Chow, V. T., Runoff, Section 14 in Applied Hydrology by V. T. Chow, McGraw-Hill, 1964

23. Sugawara, M., On Analysis of Runoff Structure of Japanese Rivers, Japanese Journal of Geophysics, Vol. 2, No. 4, March 1961.

4.3 Effects of Supply Duration on Hydrograph Peaks

Machmeier's studies show that q/i increases with increasing D, the relationship being almost linear up to a point where q/i is approximately 0.80. Further increases in D beyond this point result in slower increases in q/i until the ratio becomes almost constant near a value of 1.0. Similar results would be expected if this analysis was based on unit hydrograph theory or on a watershed storage model.

4.4 Combined Effects of Supply Rate and Duration

The "time of equilibrium" has been used in various hydrological studies, especially for estimates involving overland flow. It is denoted by T_e and may be defined as the time for the flow to increase from 0 to 0.95 i, where i is a constant supply rate of indefinite duration.

In Machmeier's work, T_e is shown to vary inversely with i, i.e. smaller values of i have larger values of T_e. For high values of i, however, T_e is almost constant. These conclusions are apparently supported in studies of real watershed data by Pilgrim[24] and Laurenson[25].

When the supply duration D, is converted to a ratio of T_e, the combined effects of supply rate and duration may be expressed by the dimensionless relationship of figure 3, according to the analyses of Machmeier. Similar dimensionless relationships have been proposed by Chow[4], using "lag" instead of T_e, and also by

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24. Pilgrim, D. H., "Measurement of Time of Concentration and Hydrograph Characteristics of Small Rural Catchments Using Radioactive Tracers", Water Research Foundation of Australia, 7th Annual Report, June 1962.

25. Laurenson, E. M., "A Catchment Storage Model for Runoff Routing", Journal of Hydrology, Vol. 2, pages 141-163, 1964.

Henderson[26], using the base width of the instantaneous unit hydrograph instead of T_e. These relationships are general and should apply to most watersheds, provided the supply rate is reasonably constant.

The above ideas have been tested with the sample of watershed data from western U.S.A. Figure 4 shows q/i plotted against D/K for the 185 flood events, where K is the lag, defined as the time between the center of the supply hyetograph and the center of the resulting hydrograph. T_e could not be used because it is impossible to estimate this value directly from rainfall and streamflow data. An easy method of estimating K is demonstrated in figure 6.

Other time parameters considered possible alternatives to T_e, were the rise time, hydrograph base width and the time between center of supply and hydrograph peak. All of these were found to be more variable than K.

The supply hyetographs were calculated for each event with the aid of a computer by assuming a constant loss rate and subtracting this from the estimated watershed rainfall. The value of the loss rate was selected so that the supply volume was equal to the surface runoff volume. D and i were both determined from the supply hyetograph, ignoring any very small rates at the beginning or end (less than 5% i).

Figure 4 shows the theoretical relationship between q/i and D/K according to Machmeier's analysis, (in which K is regarded as a function of i and D). The q/i ratio for the largest flood of each watershed is plotted against D/K in figure 5. K_r is the "representative lag" which will be described in the next section. It is

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26. Henderson, F. M., Some Properties of the Unit Hydrograph, Journal of Geophysical Research, Vol. 68, No. 16, August 1963.

approximately 10% shorter than the median value of lag, as derived in figure 6.

Figures 4 and 5 suggest the following:

(a) The general trend agrees fairly well with Machmeier's theoretical relationship, considering that much of the scatter may be attributed to inaccurate estimates of i and D, and also to the effects of non-uniform supply rates.

(b) Figure 5 is of greatest interest for design floods. In this there are no significant differences between the flood groups, as far as the general relationship is concerned.

(c) For "average" conditions associated with large floods, the hydrograph peak is related to the supply rate and duration by:

$$q/i = \begin{cases} \dfrac{0.9D}{K_r} & \text{if } \dfrac{D}{K_r} < 1.1 \\[2ex] 1.00 & \text{if } \dfrac{D}{K_r} \geq 1.1 \end{cases} \qquad \ldots\ldots\ldots\ldots(1)$$

Equation (1) expresses the main time-distributing characteristics required for design flood purposes, using only one watershed parameter, K_r.

4.5 Variability of Lag

"Relative lag" is defined as the ratio of the actual lag to the median lag for the particular watershed. This is a dimensionless flood value that may be pooled with those of other watersheds to make up a large sample. In figure 7 the relative lag of each flood event has been plotted against the probability of the associated peak.

Figure 7 agrees with other studies which show that lag decreases with increasing flood magnitudes, tending towards a constant minimum value[19, 24, 25]. The "representative lag" is close to this minimum value and may be defined as the average lag for extreme floods, i. e. with return periods exceeding ten years. The regression line of figure 7 was used for estimating the representative lag of each watershed from the median lag and the median flood probability, (see table 2).

Figure 8 shows the frequency distribution of relative lags, using the values from all flood events. It may be concluded that lags vary from about 60% to 140% of the median value, which is considerably less variation than other hydrograph time parameters that can be obtained directly from the data.

4. 6 Estimating Representative Lag

A method must be provided for estimating the representative lag from physical characteristics of watersheds. In other approaches the "time of concentration" is used for similar purposes and this may be estimated from the slope and length of the main channel[2, 3]. Unfortunately these factors were found to be useless for estimating the representative lag.

This point may be demonstrated by figure 9 in which some attempt was made to relate time of concentration to lag. A similar result was obtained by Om Kar[11], working with hydrograph rise times, and it seems that something is amiss with the time of concentration concept, at least as far as small western watersheds are concerned.

For large streams, Hoyt and Langbein[13] show lag as a function of area, viz:

$$\text{lag (hours)} = M \times (\text{area in sq. miles})^{0.4} \quad \ldots\ldots\ldots\ldots(2)$$

where M varies from 1.0 to 3.0 depending largely on the channel storage characteristics.

The following formula is better for the small sample watersheds:

$$\text{representative lag (hours)} = M \times (\text{area in sq. miles})^{0.33} \quad \ldots.(3)$$

where M varies from 0.5 to about 3.0.

No significant correlations of M could be found with any of the watershed parameters used in the Colorado State University data program. These parameters include channel slope, overland slope, drainage density, shape factors and various precipitation parameters. The only factor that seems closely related to M is the vegetation cover as shown in figure 10.

The values of M do not vary greatly within each of the adopted cover groups and the following mean values may be used for estimation purposes:

	Cover group	Mean M
A	Forest and good woodland	2.05
B	Good pasture and poor to fair woodland	1.50
C	Crops and poor to fair pasture	1.15
D	Very poor pasture and desert vegetation	0.60

The terms "good", "fair" and "poor" have the standard definitions recommended for the S.C.S. classification of vegetation[2].

The above is not intended to suggest that slopes, etc. are always unimportant in estimating lag, because these factors should have very significant effects under some circumstances. It is merely stated that the study was unable to associate lag with any factors other than area and vegetation for the sample of conditions considered.

4.7 Is a Single Parameter Adequate?

Henderson[26] and Lienhardt[27] both present data to support the contention that only one major parameter is normally needed to

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27. Lienhardt, J. H., A Statistical Mechanical Prediction of the Dimensionless Unit Hydrograph, Journal of Geophysical Research, Vol. 69, No. 24, December 15, 1964.

specify the time-distributing characteristics of a watershed. Further support is given by the dimensionless hydrograph concept that has been developed by several different investigators[2, 22, 27, 28]. These independently derived hydrographs are all very similar in shape and provide a means of estimating the design hydrograph given the volume of runoff and a single watershed time parameter, i.e. the rise time or the hydrograph base time. This approach may also be used with equation (1) for providing complete design hydrographs. After the peak has been estimated by equation (1), its time of occurrence and any other ordinates may be calculated from the appropriate ordinates of one of the recognized dimensionless hydrographs.

Equation (1) on page 18 may be manipulated as follows:

$$q/i = \frac{0.9D}{K_r} \quad \text{if} \quad \frac{D}{K_r} \angle 1.1$$

$$\text{i.e.,} \quad \frac{qD}{Q} = \frac{0.9D}{K_r} \quad \text{if} \quad \frac{D}{K_r} \angle 1.1$$

where Q = total volume of surface runoff = Di

$$\therefore \quad \frac{Q}{q} = 1.1\,K_r \quad \text{if} \quad D \angle 1.1\,K_r \; \ldots\ldots\ldots\ldots (4)$$

$\frac{Q}{q}$ is called the volume/peak ratio and has the physical dimensions of time. Equation (2) shows that it is closely related to the representative lag and should be approximately equal to the median lag (see figure 7). The ratio may therefore be used as an alternative watershed time parameter if it is easier to derive than the lag. Although this idea is not completely consistent with the conventional unit hydrograph theory, it is supported elsewhere, notably in the S. C. S. handbook[2] which also suggests that the ratio is a constant for certain conditions.

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28. Hickock, R. B., R. V. Keppel and B. R. Rafferty, Hydrograph Synthesis for Small Arid-Land Watersheds, Agricultural Engineering, October 1959.

There are several interesting conclusions that appear to follow from the above. These are:

(a) Flood peaks are dependent on the supply volume and the duration is relatively unimportant, provided it is less than the volume/peak ratio.

(b) Variability of supply rate should not be of major importance if the duration is less than the volume/peak ratio.

Chow[4] and Henderson[26] both discuss the effects of variability of supply rate on the resulting flood peaks. Their conclusions are that highly variable rates may cause flood peaks up to 15% greater than peaks caused by uniform supply rates. It is likely that these effects occur in the sample floods and they are probably largely responsible for the difference between equation (1) and the theoretical relationship of Machmeier's (figure 5). There is consequently no need to make special allowances for supply variability if equation (1) is used for design purposes.

5. THE DESIGN RAINFALL

An extreme flood is expressed by a single value, i.e. the flood peak corresponding with a particular return period. Extreme rainfall, however, is expressed by two values, i.e. the volume (or depth) and the duration corresponding with a particular return period. In estimating design floods from extreme rainfall it is necessary to decide what durations and frequencies are most appropriate and these two variables are then used to determine the required design rainfall volume.

5.1 Theoretical Physical Considerations

The average supply duration for large floods on a particular watershed should depend on:

(a) The rainfall "burst characteristics" of the flood-producing storms.

(b) The loss rates which determine how much of a particular burst becomes supply.

(c) The watershed lag. Large watersheds are expected to have longer supply durations than small watersheds.

The intense rainfall bursts in long duration winter storms and short duration summer storms are both associated with local convective cell activity[29]. Those in the winter storms are a little longer, have lower intensities and are not as distinctly different from non-burst rainfall as those in the summer storms.

The loss rates in the long-duration winter storms are relatively low. Therefore most of the burst rainfall and some of the non-burst rainfall may both contribute to the supply hyetograph. On the other hand the loss rates in the summer storms are high and usually only the short, very intense bursts contribute to the supply hyetograph.

Watershed lags are larger for the winter flood group than for the summer flood group, apparently because of the differences in vegetation. This factor should influence the effective grouping of individual bursts of rainfall. For example, two bursts one hour apart would cause two distinct hydrographs in a watershed with a lag of only 20 minutes. Two distinct hydrographs would not be expected, however, if the watershed lag were as high as 10 hours because in this case, each individual burst would be distributed over a longer time and they would have about the same effect on the hydrograph as if they were grouped into a single burst.

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29. Bell, F. C., Extreme Rainfall of Short Duration, submitted to ASCE for possible publication in Journal of Hydraulics Division, 1967.

From the above considerations one would expect shorter supply durations in the summer flood group than in the winter flood group, with the mixed group being somewhere in between. For design purposes it is necessary to work initially from extreme rainfall data and this is essentially gross rainfall rather than supply. It is therefore more relevant to consider the gross rainfall duration rather than the supply duration.

The findings of section 4.8 suggest that the volume/peak ratio (or the median lag) may be the most appropriate duration of design rainfall. The essential quantity is the volume of supply occurring within this period and the actual duration of the main supply burst is of secondary importance. Effective durations much greater than the volume/peak ratio are relatively inefficient as producers of flood peaks and are not likely to be typical for extreme floods.

5.2 Effective Durations for the Sample Watersheds

The supply duration is not readily obtained from streamflow and rainfall data. The method adopted in this study involved a trial and error technique on a digital computer, assuming a constant loss rate. For ordinary purposes with small watersheds the supply duration is sometimes considered to be approximately equal to the rise time and this is tested graphically in figure 11. It is concluded that the supply duration is only 75% of the rise time, on the average, with a standard error of 15%.

Figure 12 is intended to show whether the supply duration changes systematically with the flood magnitude. It indicates that larger floods have shorter durations in the winter flood group and longer durations in the summer flood group, although the latter is not very marked.

In figure 13 the supply durations for the largest floods are plotted against the median lags, demonstrating that:

(a) Supply durations do not usually exceed the median lag in large floods.

(b) Supply durations are not strongly correlated with median lags for any of the flood groups. Their mean values are 0.3 hours for the summer group, 0.6 hours for the mixed flood group and 0.9 hours for the winter group.

The above refers to supply durations but in previous sections it was argued that gross rainfall durations are more relevant for estimating design floods from extreme rainfall data. However, for small watersheds it is unreasonable to use the entire storm duration for a hydrograph caused mainly by one short burst and there does not appear to be any satisfactory method of determining what part of the gross rainfall should be separated for this purpose. It is essential that the selected duration be at least as long as the supply duration but it can also be considerably longer. This extra period of rain would have no effect on the flood estimates, provided all the "non-supply" rainfall is included in the loss.

In section 5.1 it was postulated that the median lag or volume/peak ratio may be the best "effective duration" for design purposes but figures 11, 12 and 13 suggest that durations of this magnitude are somewhat longer than necessary. As a compromise it is proposed that the representative lag be used for the effective duration of gross rainfall because it is shorter than the volume/peak ratio but larger than most of the supply durations in extreme floods. It is also conveniently estimated from figure 10. The combined effects of loss rates and rainfall variability account for cases in which the supply duration tends to be much shorter than the representative lag and these should present no special difficulties either in the estimation procedures or in physical interpretation.

5.3 The Design Rainfall

After adopting the representative lag as an appropriate design duration of rainfall, the question of return period should be examined more closely. The same return period is often assumed for the rainfall and the associated flood but this is not necessarily correct. The matter may be settled for the small sample watersheds by figure 14 which shows the probabilities of the gross rainfall plotted against the estimated probabilities of the associated floods for the available sample of events. The gross rainfalls in a period equal to the representative lag were used, except when the supply duration exceeded the representative lag. In these cases the supply durations were assumed equal to the durations of gross rainfall.

Although the scatter in figure 14 is very broad the essential issue is that approximately the same number of points fall on each side of the 45^0 line for the full range of values, indicating that, on the average, the same return period applies to both rainfall and the associated floods. The average 100 year flood, for example, corresponds with the average 100 year rainfall for the watersheds considered.

It is not suggested that the above conclusion applies to all small watersheds, in fact, there is evidence to show that it is not true in certain climatic situations where the highest rainfall intensities (and return periods) occur in brief summer thunderstorms on dry watersheds but cause only minor or moderate floods[30]. In these cases the extreme floods are caused by rainfall of lower intensity in long duration storms after watersheds have become very wet. Under such conditions the return periods of the extreme floods would tend to be higher than the return periods of the associated rainfall.

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30. Pilgrim, D. H., Flood Producing Storms on Small Rural Catchments with Special Reference to New South Wales, Civil Engineering Transactions, Institution of Engineers, Australia, CE8, No. 1, April 1966.

A plot similar to figure 14 has been presented by Hiemstra and Reich[5], using a different set of flood events from other small watersheds in U. S. A. The conclusions that may be drawn from this plot are the same as those from figure 14.

When the return period and effective duration of the design rainfall are known it is usually a relatively simple matter to obtain the required rainfall volume from published data such as U. S. Weather Bureau Technical Paper No. 40[31]. This procedure will be discussed in greater detail in section 7.

6. THE DESIGN LOSS FACTOR

A relatively large part of the rainfall does not become runoff, even under extreme flood conditions. This water is usually referred to by engineers as "loss", although the suitability of such a term is often questioned, especially by soil hydrologists.

The physical phenomena associated with losses are rather complex, involving infiltration, interception, evapotranspiration and similar processes. Most of these phenomena are now well understood from the physical point of view[32] but the treatment of losses is still a weak link in the estimation of both specific and design floods from rainfall.

6.1 Should Runoff be Regarded as a Residual or Percentage of Rainfall?

In order to answer this question, one should consider how the relevant physical processes are best described in mathematical terms. Before the 1940's, the "runoff coefficient" approach was widely

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31. Hershfield, D. M., Rainfall Frequency Atlas of the United States, Weather Bureau Technical Paper No. 40, 1961.

32. Bell, F. C., A Survey of Recent Developments in Rainfall-Runoff Estimation, Journal of Institution of Engineers, Australia, March 1966.

accepted by engineers perhaps mainly for computational reasons. This approach implies that runoff is a percentage of rainfall, which was regarded as illogical by proponents of the so-called "infiltration theory."

Infiltration theory treats runoff as the residual when deductions are made for infiltration and this concept has become widely accepted as a fundamental interpretation of rainfall-runoff phenomena. However, the theory has a number of deficiencies as described elsewhere by the author[16, 21]. Flood estimation techniques involving "loss rates" or "phi-indices" are recommended by the standard textbooks and these are practical applications of infiltration theory.

During recent years some studies have suggested that there are conditions in which runoff is more appropriately treated as a percentage of rainfall. In these studies the impervious and "runoff-producing" parts of a watershed have been given special emphasis[33, 34].

In developing a complete rainfall-runoff model for small watersheds in Australia, the author found that runoff is better expressed as a percentage when it is only a small fraction of the rainfall (less than 10%)[16]. In other cases, however, it is better expressed as a residual. For estimating design floods the residual approach appears preferable because design floods are generally expected to comprise a large part of the rainfall.

6.2 Applications of Design Loss Rates

For large watersheds design floods are often estimated by the "unit hydrograph-loss rate" method which involves the following steps:

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33. Betson, R. P., What is Watershed Runoff?, Journal Geophysical Research, Vol. 69, No. 8, April 1964, pages 1541-1552.

34. Crawford, N. H. and R. K. Linsley, "Conceptual Model of the Hydrologic Cycle," Publication No. 62, International Association of Scientific Hydrology, 1963, pages 573-587.

(a) Adoption of a unit hydrograph for the particular watershed. This is obtained either from streamflow data or by synthesis from watershed parameters (e.g. by the Clark-Johnston or Taylor-Swarz methods[35]).

(b) Selection of a "design loss rate" which is usually between .01 and .10 inches per hour[36, 37].

(c) Selection of a typical pattern of gross rainfall, i.e. either early-peaking, late-peaking or uniform.

(d) Selection of a number of appropriate rainfall durations and the calculation of corresponding supply hyetographs by using (b), (c) and the required return period.

(e) Application of the unit hydrograph to the supply hyetographs of (d). The resulting flood hydrograph with the highest peak flow is adopted as the design hydrograph.

The selection of suitable design loss rates (i.e. step (b)) has been discussed very thoroughly by Laurenson and Pilgrim[36] and Pilgrim[37] who derived the general distribution of loss-rates shown in figure 15. The same distribution was found to apply to Australia, U.S.A., and New Zealand for floods on large watersheds. It does not apply to small watersheds in U.S.A., however, as shown by distributions B and C in figure 15. It may be seen that these watersheds tend to have higher loss rates, especially in western U.S.A.

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35. Linsley, R. K., M. A. Kohler and J. L. H. Paulhus, 1958 Hydrology for Engineers, McGraw-Hill, New York.

36. Laurenson, E. M., and Pilgrim, D. H., "Loss Rates for Australian Catchments and Their Significance," Journal Institution of Engineers, Australia, Vol. 35, No. 1-2, January-February 1963, pages 9-24.

37. Pilgrim, D. H., "Storm Loss Rates for Regions with Limited Data," Journal of the Hydraulic Division, ASCE, Vol. 92, No. HY2, Proc. Paper 4728, March 1966, pages 193-206.

Distribution B was derived from the Colorado State University data assembly, using the records of about 200 watersheds less than 50 square miles from the entire U.S.A. Distribution C was derived from the data of the 38 sample watersheds in western U.S.A. as described in 3.2. A digital computer was used for calculating the loss rates for distributions B and C but the method was essentially the same as Laurenson and Pilgrim's.

The above loss rates are calculated for small increments of time and may be regarded as average "instantaneous" loss rates during the supply period. If the representative lag is adopted as the effective duration for design floods it is more convenient to deal with loss rates averaged over this period rather than the supply period.

Figure 16 shows the distributions of loss rates averaged over the effective durations for the sample watersheds in western U.S.A. A very wide range of values is indicated, with larger values in the summer and mixed flood groups than in the winter flood groups. Figure 17 shows the relationship between instantaneous loss rates and loss rates averaged over the effective duration.

6.3 Do Larger Floods have Smaller Loss Rates?

In figure 18 the ratios of loss rate/median loss rate for each event are plotted against the probabilities of the associated floods. No relationship is indicated and it may be concluded that the median loss rate is the typical value associated with extreme floods, as well as common floods, in the area of interest.

The above conclusion is compatible with that of section 5.3, (i.e. that rainfall and floods have same average return period) and it should be possible to deduce one of these conclusions from the other. Each deviation from the 45^0 line in figure 14 may be related to the corresponding loss rate and the randomness of these deviations is merely repeated by figure 18.

6.4 Estimating Design Loss Rates

Adopting the median loss rate (averaged over effective duration) as the most appropriate design loss rate, the next problem is to estimate this value for ungaged watersheds. Figure 19 shows an attempt to relate the median loss rates of the sample watersheds to the S.C.S. "average curve number" or CN value[2]. The CN value is a very logical index of the watershed loss potential and is calculated from vegetation cover and soil characteristics[8, 38, 39]. Unfortunately it did not seem to explain any of the variability in the correlations between median loss rate and most of the other available watershed parameters were investigated but no significant conclusions could be drawn from any of these. If the watersheds are grouped into the classes shown in table 4, the variability is considerably reduced, giving a rather unsatisfying guide to the values that may be expected under various conditions.

Table 4 shows the mean of the median loss rates for each class of watersheds and this value could possibly be used for design purposes in the absence of any other relevant data. The topic requires further investigation with more detailed data on soils and vegetation, and a larger number of flood events than were available for this study.

7. COMPLETE FLOOD ESTIMATION

It is possible to integrate the various findings of this study to provide a fairly simple method of estimating extreme flood peaks from extreme rainfall data for small watersheds in western U.S.A.

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38. Soils of the Western United States, Joint Regional Publication by the Agricultural Experiment Stations of the Western States and Land-Grant Universities and Colleges with Cooperative Assistance by the Soil Conservation Service of the United States Department of Agriculture, September 1964.

39. Hunt, C. B., Physiography of the United States, W. H. Freeman and Company, San Francisco and London, 1967, 480 pages.

7.1 The Rational-Loss Rate Method

In section 4.8 it was shown that the flood peak (q) may be related to the supply volume (Q) by:

$$\frac{Q}{q} = 1.1 \; K_r \qquad \text{if} \qquad D < 1.1 \; K_r \quad \ldots\ldots\ldots (4)$$

i.e. $$q = \frac{0.9Q}{K_r} \qquad \text{if} \qquad D < 1.1 \; K_r$$

$$= \frac{0.9}{K_r} (P-R) \qquad \text{if} \qquad D < 1.1 \; K_r$$

where P = gross rainfall in period K_r

R = total loss in period K_r

other symbols are as defined in section 4.8.

In the design situation it may be assumed that $D < 1.1 \; K_r$ and the effective duration of rainfall is K_r. The above equation may therefore be written:

$$q_y = \frac{0.9}{K_r} \left(P_y - R_y \right)$$

i.e. $$q_y = 0.9 \left(F_y P_1 - \breve{r} \right) \quad \ldots\ldots\ldots\ldots\ldots\ldots (5)$$

where q_y = flood peak with return period y

P_y = volume of rainfall in duration K_r with return period y

F_y = coefficient corresponding with y and K_r, obtained from figure 20

P_1 = 10 year, 1 hour rainfall which is used as an index value

$\breve{r}$ = median loss rate averaged over K_r.

Equation (5) may be called the "rational-loss rate" formula because it combines some of the features of the old rational formula with the loss rate principle.

The value of K_r may be estimated from figure 10 and the median loss rate from table 4 if no other data are available on these factors.

The "frequency-duration" coefficient F_y is obtained from figure 20 which was derived from the general frequency-duration function proposed by the author for extreme rainfall of short duration[29]. The use of this coefficient speeds up the calculation of the design rainfall and requires only one basic rainfall frequency map showing the 10 year 1 hour rainfall. Alternatively, the design rainfall volume P_y may be extracted directly from one of the standard sources[31] and $F_y P_1$ calculated from $F_y P_1 = \frac{P_y}{K_r}$.

The above procedure is consistent with the requirements and limitations of estimating design floods from extreme rainfall data. It is extremely simple and has the considerable advantage that the user is readily aware of the significance of each component of the calculation. This also gives the method a high degree of flexibility, enabling it to be easily modified for special circumstances, e.g. when extra information is available.

7.2 Some Complications in the Rational-Loss Rate Method

The method was derived from analyses of simple hydrographs separated from base flow and, in some cases, other hydrographs. Peak flows calculated as above do not include this "supplementary flow" which may be important in some design situations.

Supplementary flow was investigated in the sample flood events and was found to be insignificant in the summer flood group. In the other groups it was found to be roughly proportional to the associated peak flow, having an average value of 7% of the peak flow for the winter flood group and 2% for the mixed flood group. No significant differences in the percentages could be attributed to the magnitude of the flood, i.e. the percentages were no larger in larger floods.

It is convenient to allow for these effects by adopting coefficients higher than 0.9 in equation (3).

Other complications could occur in areas where different return periods apply to the design rainfall and the associated floods, as mentioned in section 5. 3.

A more general expression of the rational-loss rate formula may be postulated to cover some of the above difficulties, i.e.

$$q_x = C\,(F_y P - r_z) \qquad \ldots\ldots\ldots\ldots\ldots\ldots\ldots (6)$$

where q_x = design flood peak with return period x

C = a coefficient that is generally 0.9 but may be increased to allow for supplementary flow if necessary

F_y = duration-frequency coefficient for y and K_r

y is the rainfall return period corresponding with the flood return period x

r_z = mean loss rate corresponding with x and y .

7. 3 Testing the Method

Equation (5) was used to estimate the 10 year floods on the test watersheds described in section 3. 3. The results are listed in table 5 where they may be compared with the 10 year floods derived from streamflow data.

7. 4 Discussion of Results

Table 5 shows that the rational-loss rate method does not give very accurate estimates of the 10 year flood, as may be expected for the reasons outlined earlier. The main source of trouble is evidently in the estimation of the median loss rate which was done by means of table 4. In six of the larger watersheds the estimated median loss rate was greater than the rainfall factor $(F_{10}P_1)$ resulting in calculated negative values for the required flood peak. In cases such as these, where the flood runoff is very small compared with the rainfall, it may be better not to use the residual approach, as discussed in section 6. 1.

It seems that the method cannot be strongly recommended for practical applications unless the median loss rate can be estimated with greater confidence. Nevertheless the results shown in table 5 are no worse than would be expected with most other methods, as indicated by the recent study of Hiemstra and Reich[5]. It is doubtful that any other method accounts for the loss factor in a more satisfactory manner than equation (6) except, perhaps, when the run-off rates are small compared with the gross rainfall rates. Unfortunately these conditions may be common in arid and semi-arid regions, particularly for watersheds larger than 10 square miles, as shown by the test watersheds.

8. CONCLUSIONS AND SUMMARY

The main conclusions of this study may be summarized by:

(a) The estimation of design floods should be regarded as a more generalized procedure than the estimation of specific floods.

(b) High accuracy cannot be expected in estimating extreme floods from extreme rainfall.

(c) For design flood estimation a single parameter is sufficient to express the time-distributing characteristics of a watershed. The suggested parameter is the representative lag which is closely related to the volume/peak ratio.

(d) For small watersheds in western U.S.A. the same return period may be assigned to the design flood and the corresponding extreme rainfall.

(e) The rational-loss rate method is suggested for estimating extreme floods from extreme rainfall because of its simplicity, flexibility, and consistency with the requirements and limitations of the problem. However, it cannot be recommended strongly on the basis of its reproductions of the 10 year floods on the test watersheds.

(f) The estimation of median loss rates is the major source of error in the rational-loss rate method and if this could be improved the method would probably be very satisfactory.

TABLES

TABLE 1 68% CONFIDENCE LIMITS FOR ESTIMATING RETURN PERIOD

	50 Yr Return Period	Estimated 100 Yr Return Period	Estimated 500 Yr Return Period
25 Years of Records	12 to 220 Yrs	15 to 400 Yrs	16 to 2200 Yrs
100 Years of Records	25 to 100 Yrs	40 to 250 Yrs	60 to 1500 Yrs

TABLE 2 WATERSHEDS USED FOR FLOOD ANALYSES

	Area Sq. Miles	Mean Annual Precipitation Ins.	Flood-Soil Class	Vegetation Cover	Rep. Lag.
Winter Flood Group					
05-02-01	0.16	30	W - B	B	0.75
05-02-02	7.05	18	W - C	A	3.3
05-02-07	4.80	15	W - A	B	2.8
05-02-14	23.8	19	W - C	A	4.5
05-03-05	0.87	24	W - B	A	2.7
12-04-01	0.11	13	W - C	B	.63
12-04-03	0.23	22	W - B	C	.90
12-04-04	0.28	22	W - A	C	.80
47-04-04	1.19	20	W - A	C	1.50
Eagle Lake, Cal.[10]	0.91	15	W - C	A	2.70
Newberg, Or.[8]	0.02	40	W - B	C	.40
Placerville, Cal.[8]	0.06	37	W - C	C	.64
Summer Flood Group					
05-05-28	0.94	8	S - C	D	.64
Colorado Springs, Colo.[8]	0.06	14	S - B	D	.25
03-06-01	0.81	11	S - B	D	.65
03-06-02	1.07	11	S - B	D	.40
03-06-04	0.88	12	S - B	D	.59
03-06-06	1.13	11	S - B	D	.62
03-06-19	43.9	14	S - C	D	1.44
31-06-01	0.95	11	S - C	D	0.58
31-06-03	33.0	10	S - C	D	1.55
31-09-01	0.15	8	S - C	D	.25
31-09-04	0.22	14	S - B	D	0.36
Santa Rosa, N. M.[8]	67.0	13	S - B	D	2.60
43-08-01	0.15	19	S - C	C	.40
36-08-15	0.62	21	S - B	D	.40
Mixed Flood Group					
36-08-01	0.14	31	M - C	C	.58
36-08-02	0.32	31	M - C	C	.58
36-08-03	0.15	31	M - B	C	1.84
43-09-01	0.90	32	M - C	C	1.25
43-09-07	0.48	32	M - B	C	.67
43-09-09	0.12	32	M - C	C	.66
43-09-23	7.01	28	M - C	B	3.30
43-09-28	1.26	39	M - C	B	1.50
15-11-01	3.01	32	M - B	C	1.45
27-07-01	0.74	23	M - B	C	1.05
27-07-02	0.64	23	M - B	C	.84
27-07-03	3.26	23	M - B	C	1.70

In the above flood-soil classes W-B, for example, refers to winter flood group and soil group B. Soil groups are as defined by S.C.S. in ref. 2. Vegetation groups are defined in Figure 10.

TABLE 3 WATERSHEDS USED FOR TESTING RESULTS

Watershed
3 - 6 - 18
5 - 2 - 55
5 - 2 - 66
5 - 5 - 19
Devils Ck., Idaho
27 - 07 - 04
31 - 09 - 39
Tularosa Trib., N. M.
43 - 09 - 31
43 - 09 - 05
44 - 05 - 09
31 - 09 - 02
44 - 06 - 30
Cosgrove Ck., Cal.
Lost Ck., Idaho
Lamoille Ck., Nevada
Katzer Drain, Neb.
Madera Canyon, Texas
37 - 04 - 03
Granite Creek Az.

TABLE 4 MEDIAN LOSS RATES ($\tilde{r}$) FOR SAMPLE WATERSHEDS

	Soil Groups A and B		Soil Groups C and D	
	Mean $\tilde{r}$	Stand. Devn.	Mean $\tilde{r}$	Stand. Devn.
Winter Floods	.26	.15	.14	.07
Mixed Floods	1.06	.36	.59	.18
Summer Floods	1.20	.33	.92	.47

TABLE 5 TESTING OF RATIONAL - LOSS RATE METHOD

Watershed	Area Sq. M.	Vegetation Cover	Reptve. Lag.	Flood Soil Class	F_{10} P_1	Estimated $\tilde{r}$	Estimated q_{10}	Observed q_{10}
. 3 - 6 - 18	1.19	C	1.2	S - B	1.42	1.20	.20	.25
5 - 2 - 55	2.39	B	2.0	W - B	0.62	.26	.36	.20
5 - 2 - 66	0.16	B	0.8	W - B	1.16	.26	.90	.20
5 - 5 - 19	18.7	B	5.1	W - B	0.28	.26	.02	.02
Devils Ck., Idaho	13.0	B	3.5	W - B	0.30	.26	.04	.02
27 - 07 - 04	5.43	D	1.0	M - B	2.48	1.06	1.42	.32
31 - 09 - 39	11.6	C	2.6	S - C	0.69	0.92	0	.06
Tularosa Trib, N.M.	13.8	C	2.7	S - C	1.38	0.92	.46	.27
43 - 09 - 31	8.31	C	2.2	M - B	1.56	1.06	.50	.23
43 - 09 - 05	0.28	D	0.4	M - B	5.20	1.06	4.14	3.20
44 - 05 - 09	18.0	B	3.9	S - C	0.30	0.92	0	.01
31 - 09 - 02	0.28	C	0.7	S - B	1.62	1.20	.42	.60
44 - 06 - 30	21.4	C	3.2	M - C	0.47	.59	0	.02
Cosgrove Ck., Cal.	20.6	A	5.6	W - C	0.29	.14	.15	.21
Lost Ck., Idaho	29.4	B	4.6	W - B	0.23	.26	0	.03
Lamoille Ck., Nevada	25.0	D	1.7	S - C	0.43	.92	0	.04
Katzer Drain, Neb.	45.9	D	2.1	S - C	1.02	.92	.10	.04
Madera Canyon, Texas	53.8	D	2.3	M - B	1.12	1.06	.06	.12
37 - 04 - 03	29.6	B	4.6	W - B	0.29	.26	.03	.04
Granite Creek Az.	39.6	D	2.0	S - C	0.87	.92	0	.11

FIGURES

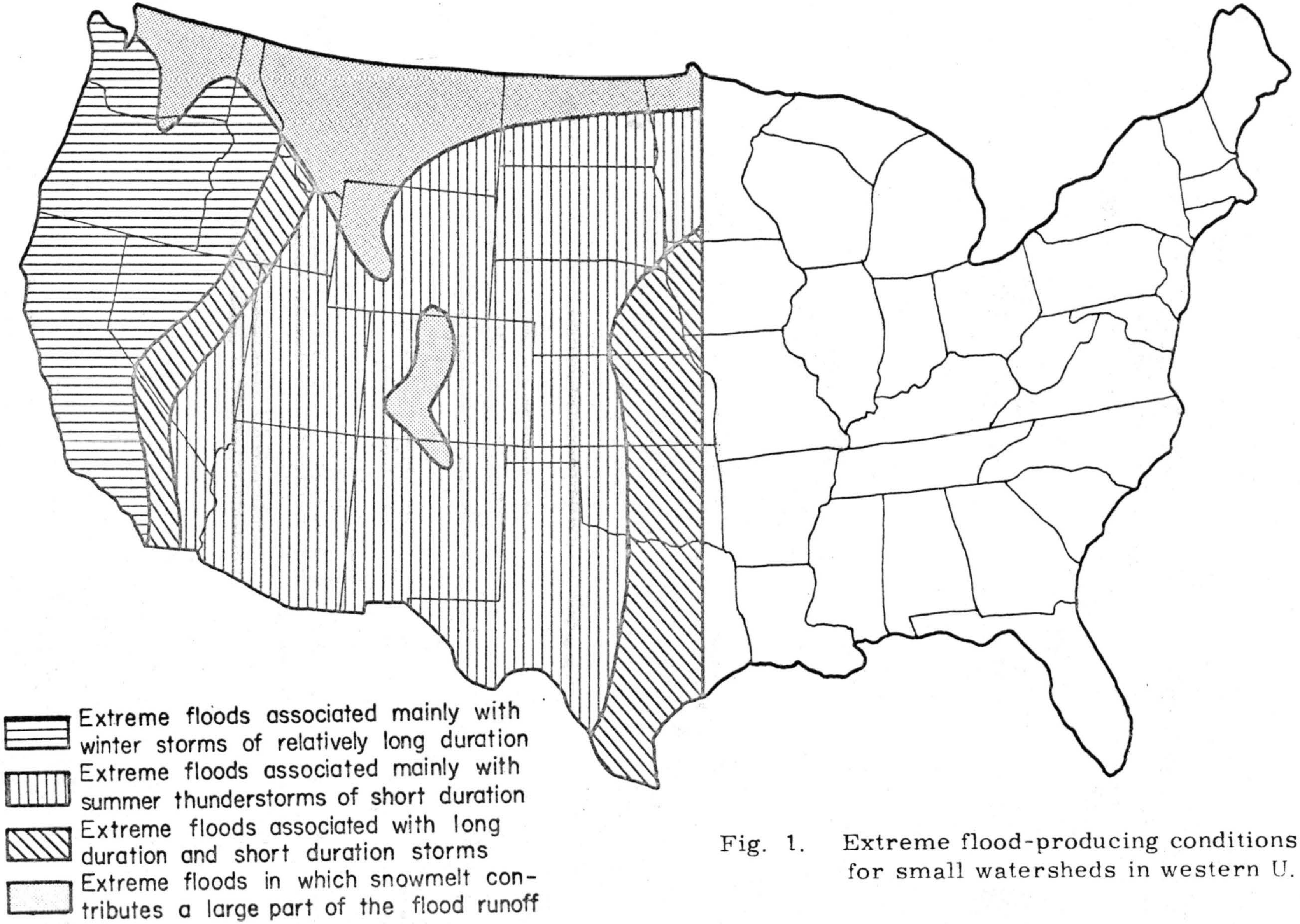

Fig. 1. Extreme flood-producing conditions for small watersheds in western U. S. A.

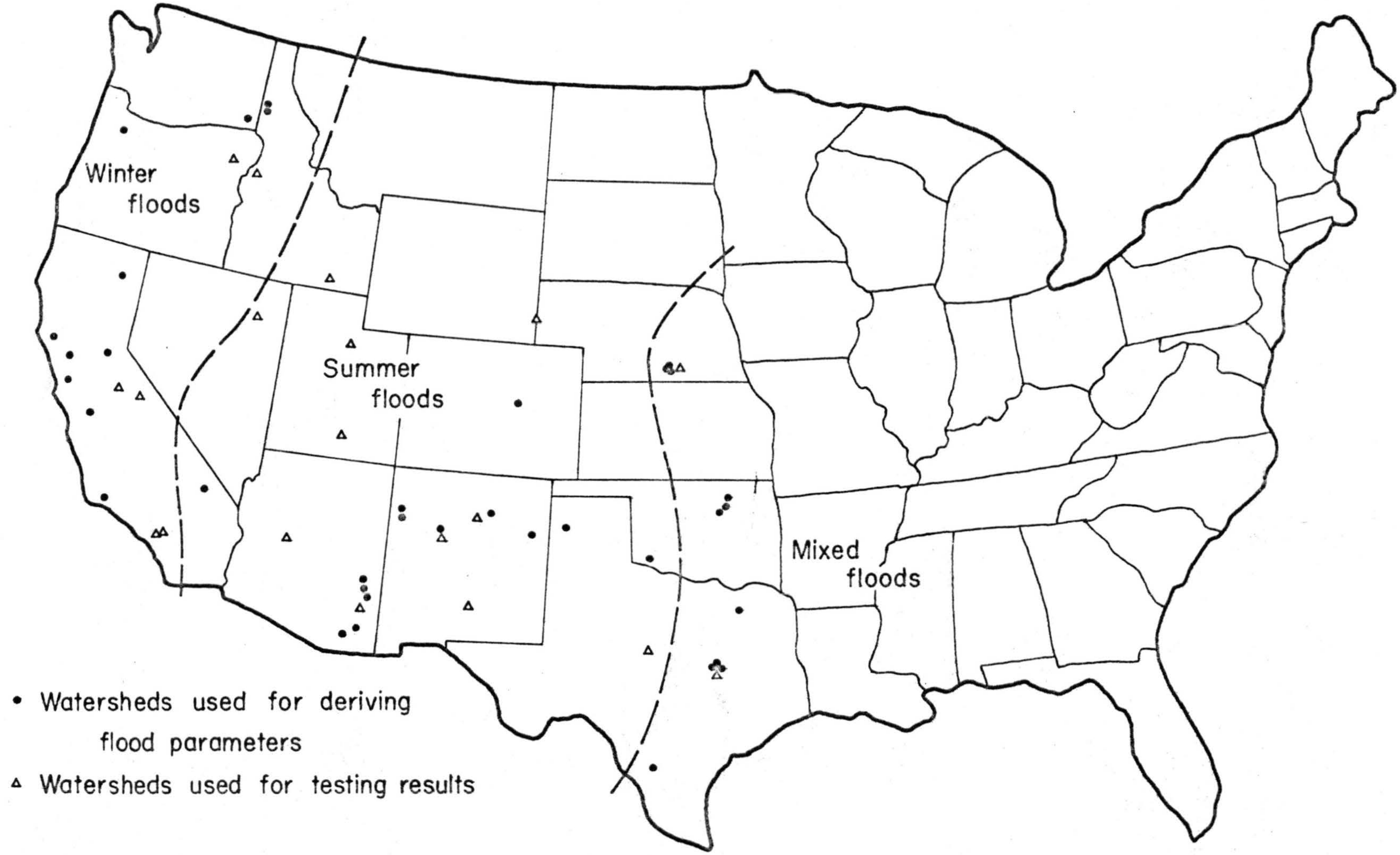

Fig. 2. Locations of watersheds used in study

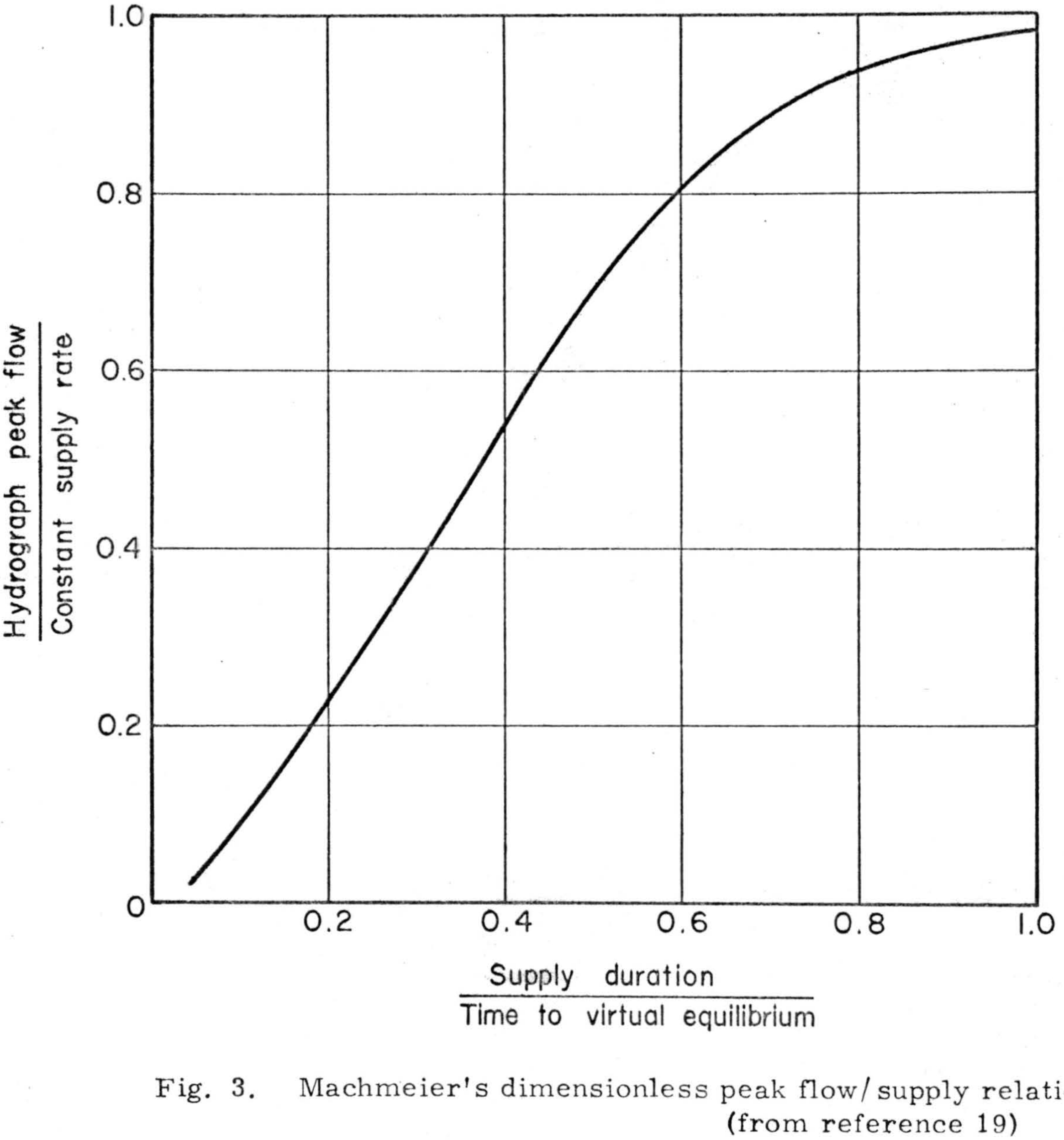

Fig. 3. Machmeier's dimensionless peak flow/supply relationship (from reference 19)

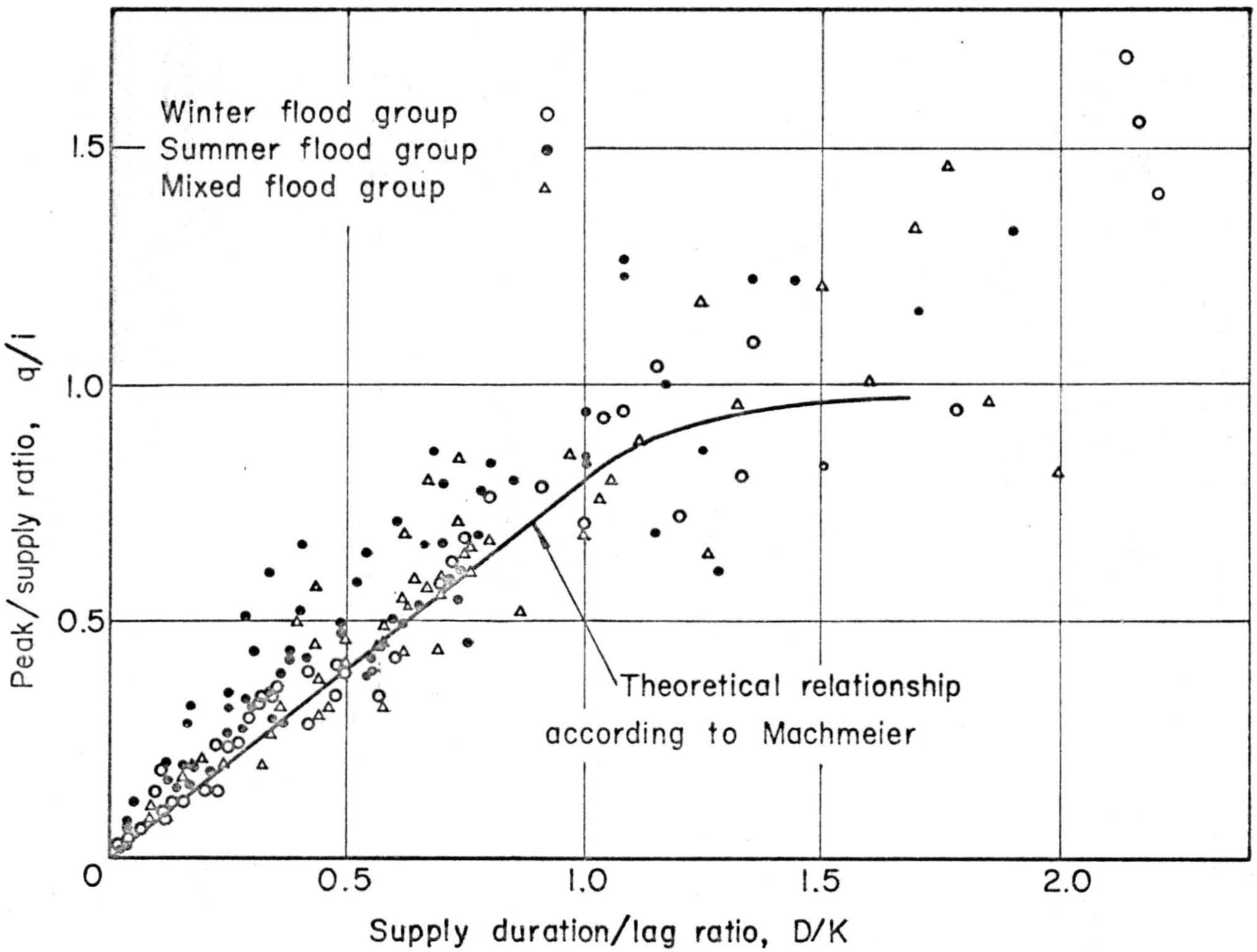

Fig. 4. Dimensionless peak/supply relationship for all floods

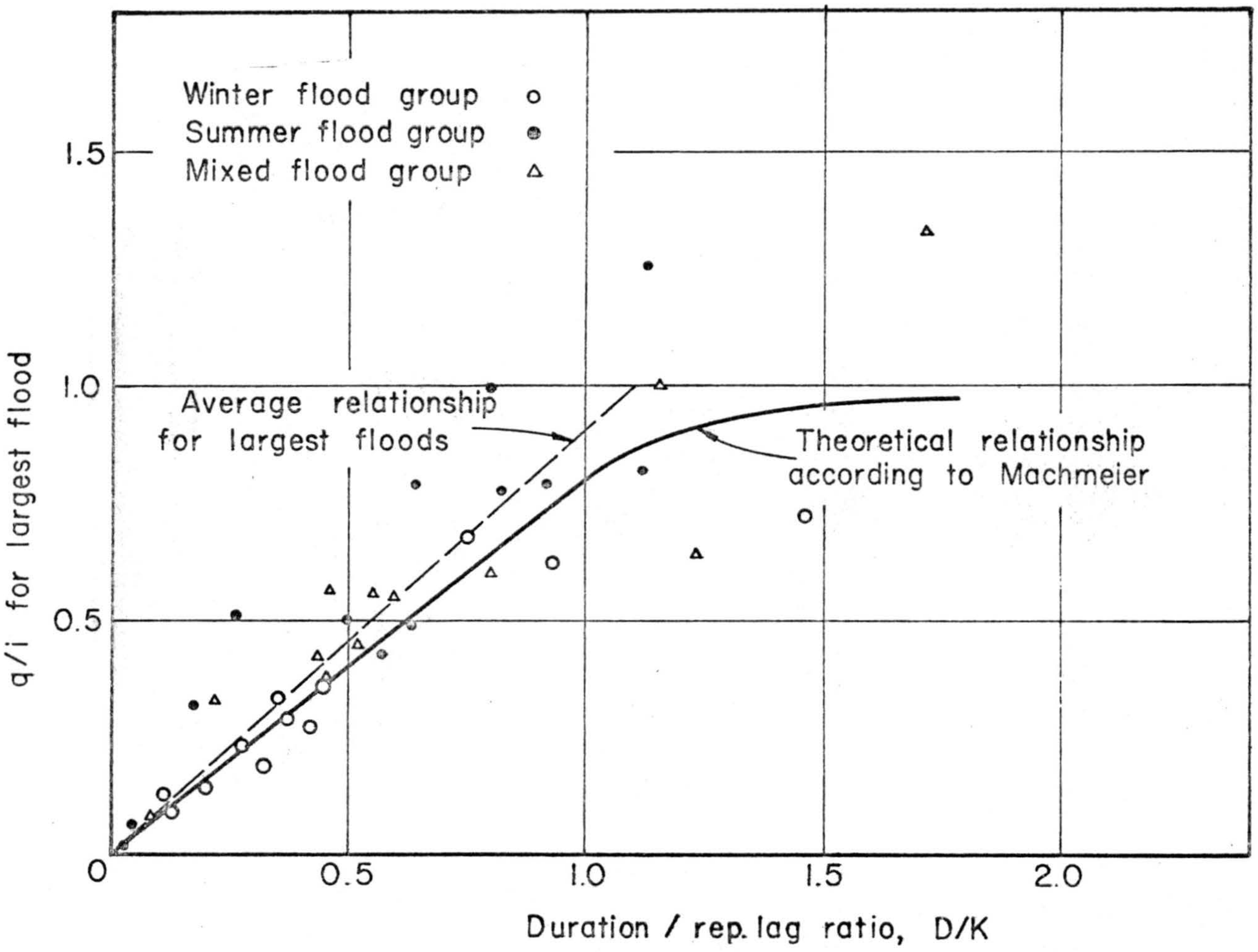

Fig. 5. Dimensionless peak/supply relationship for largest flood on each watershed

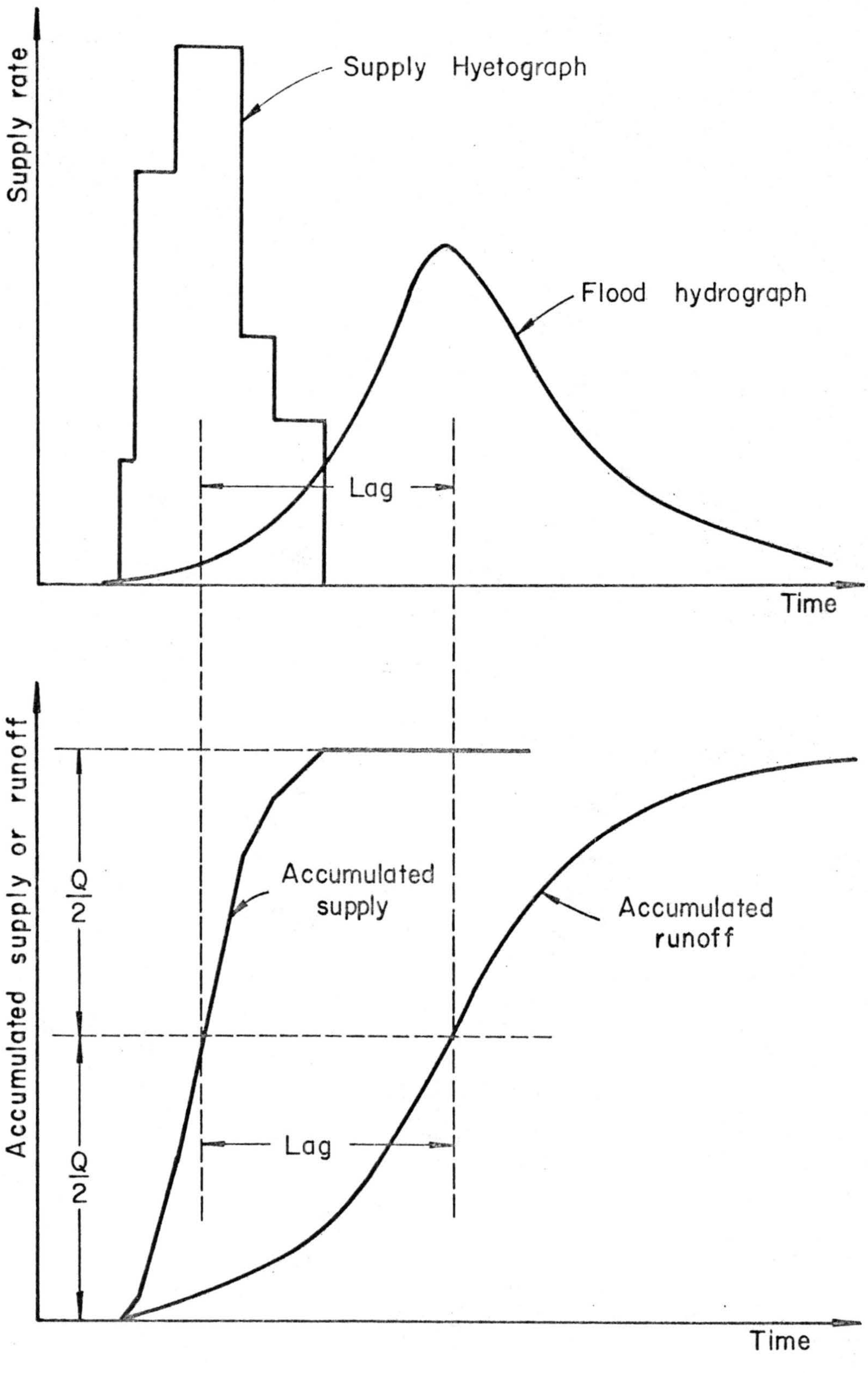

Fig. 6. Method of estimating lag from flood data

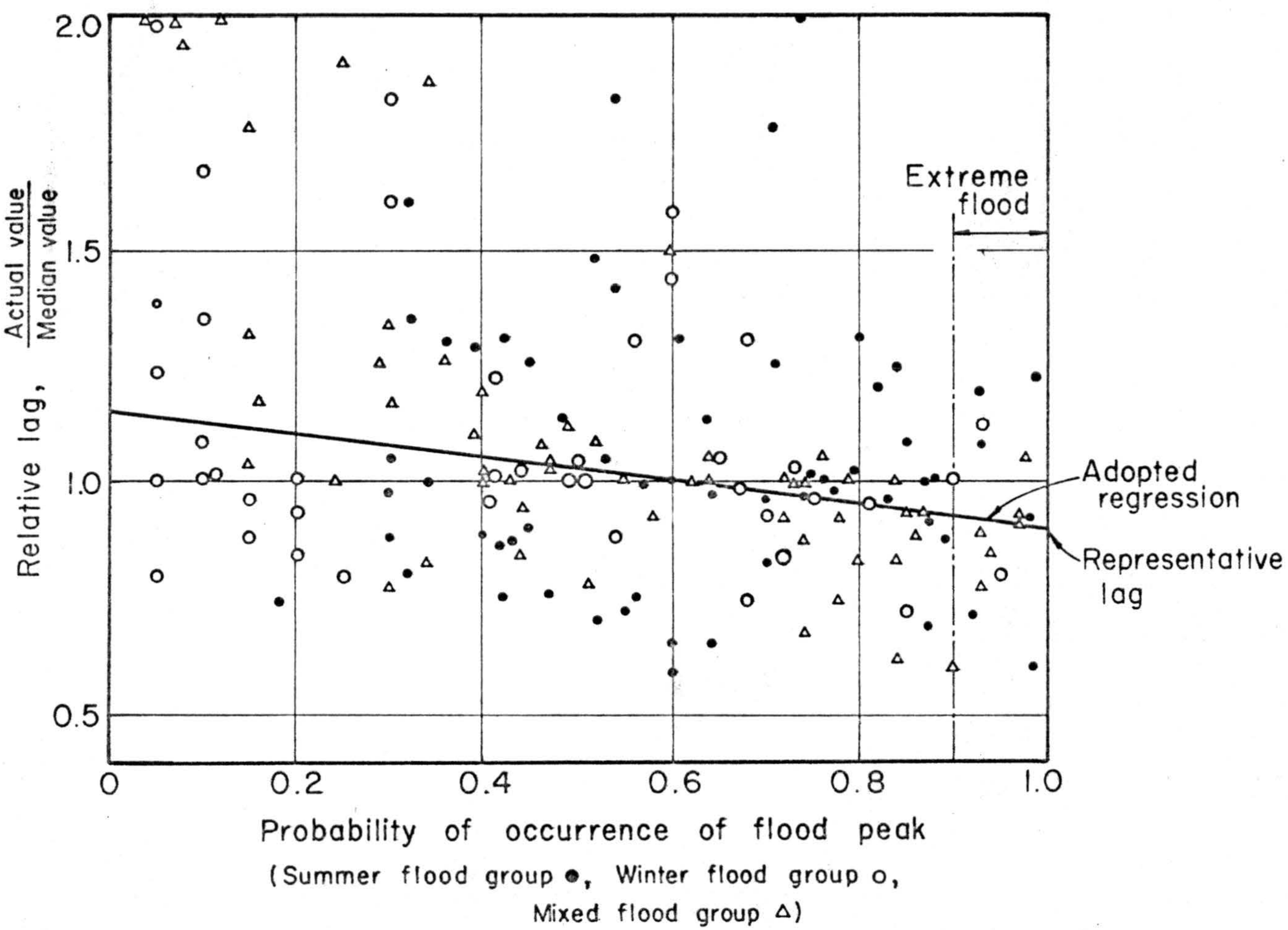

Fig. 7. Lag - probability relationship

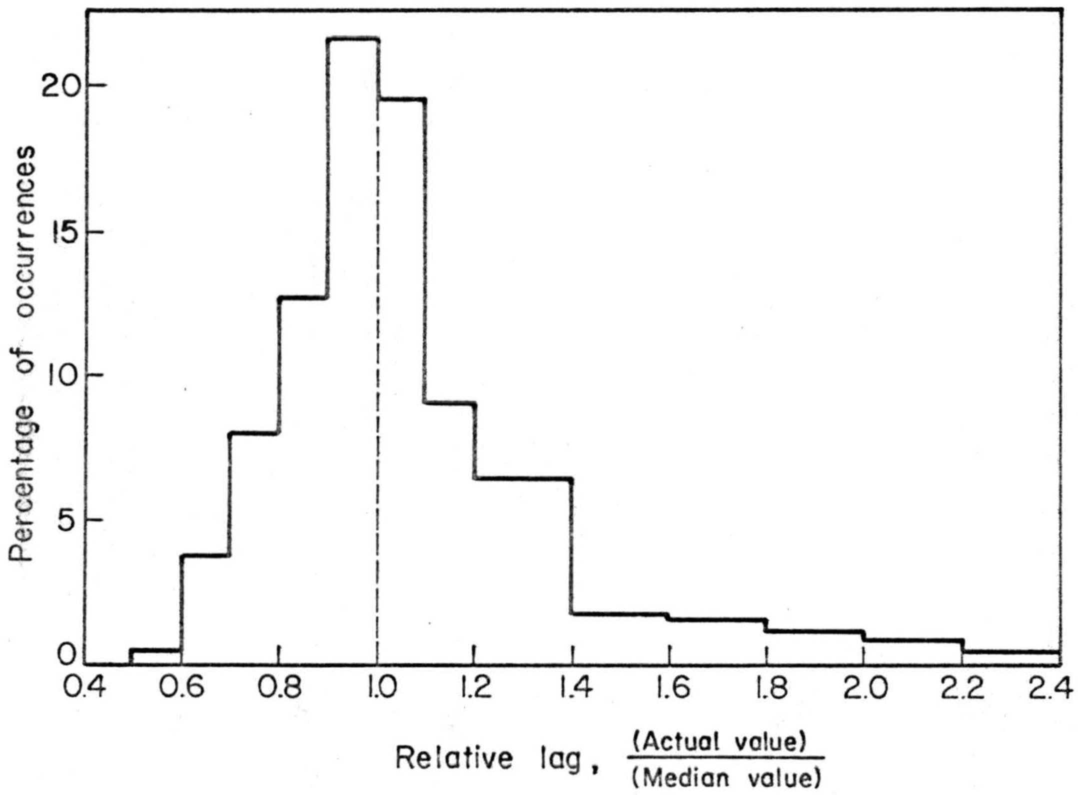

Fig. 8. Distribution of relative lag

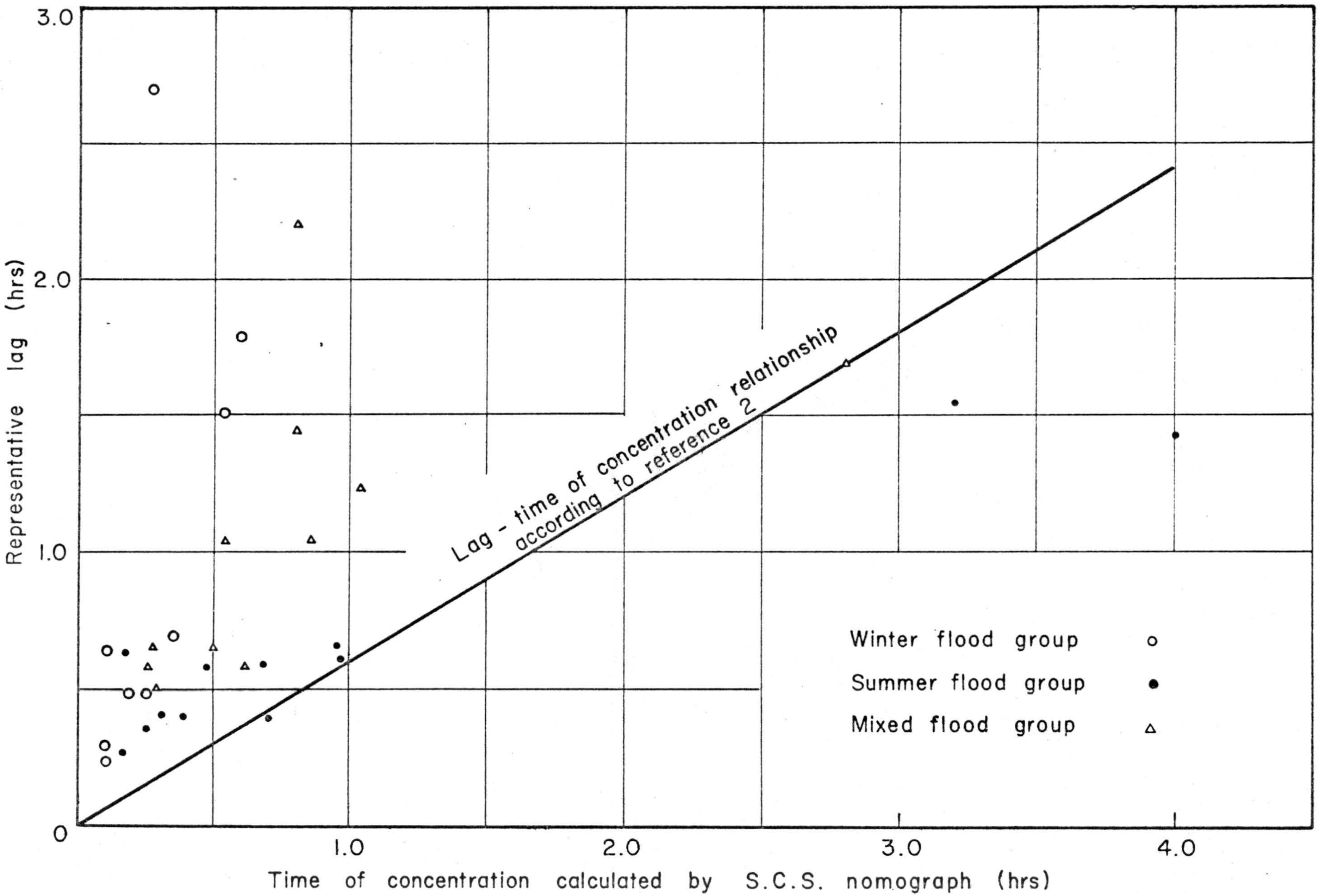

Fig. 9. Lag - time of concentration relationship

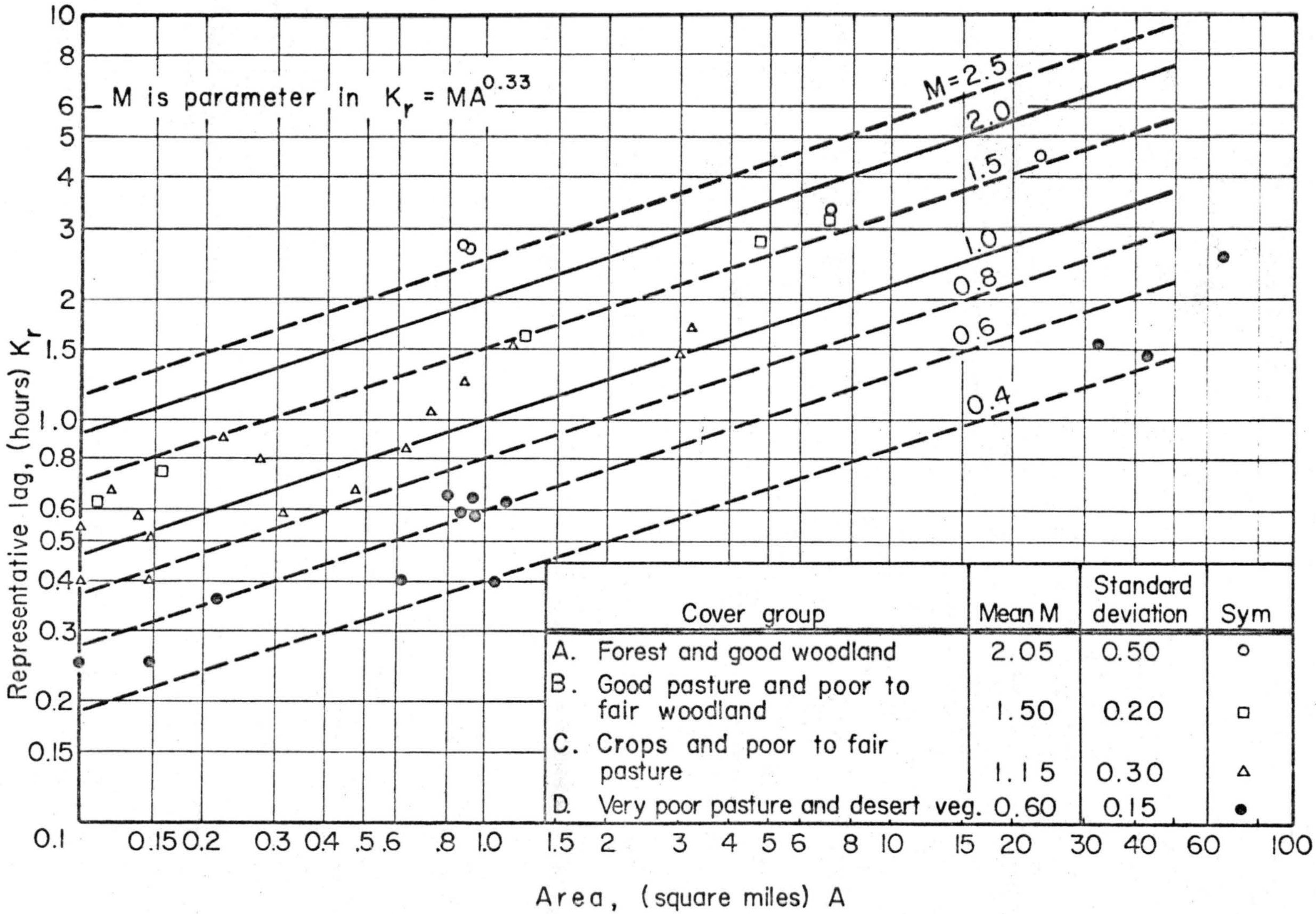

Fig. 10. Area - lag relationship

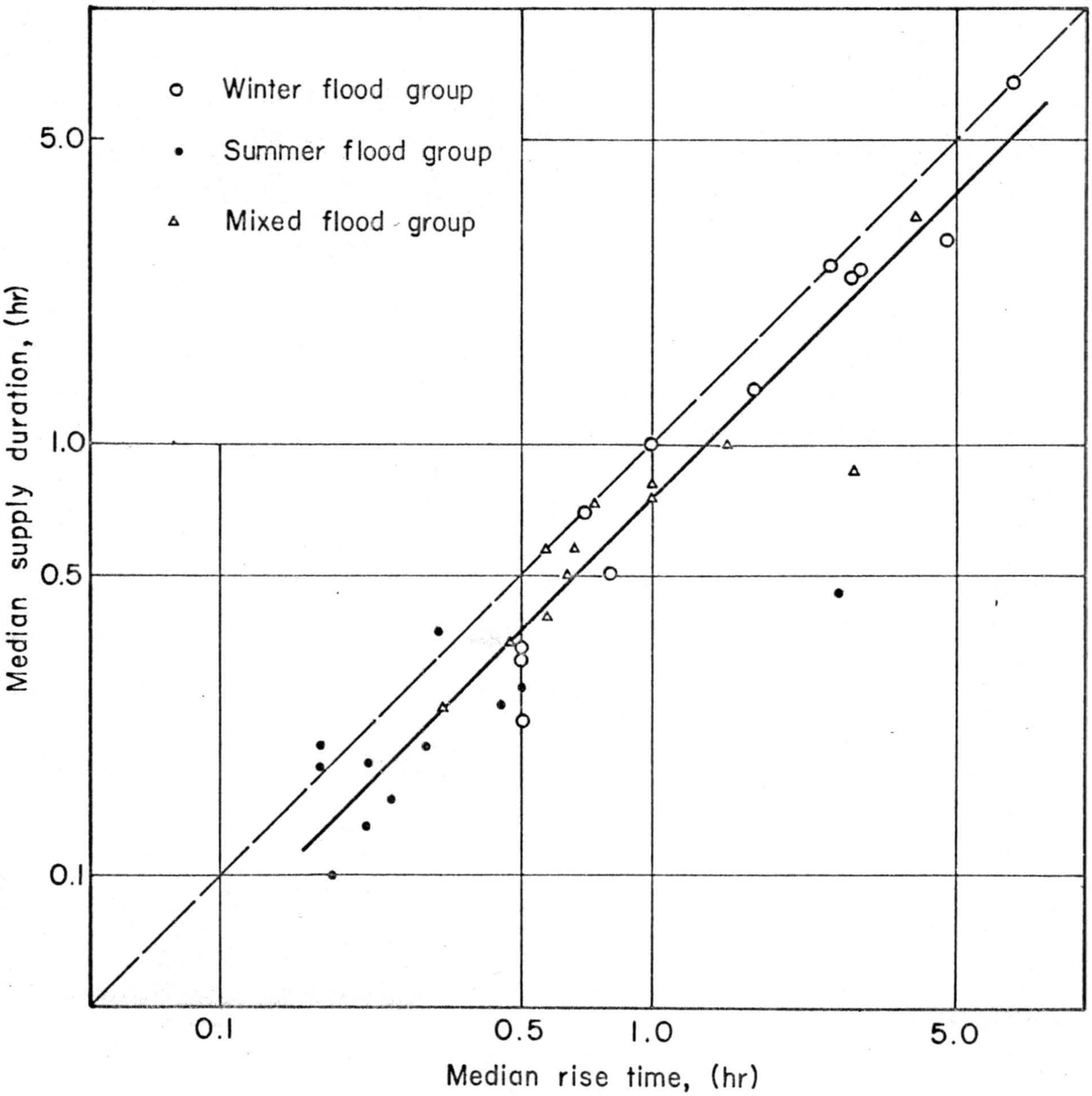

Fig. 11. Rise time - supply duration relationship

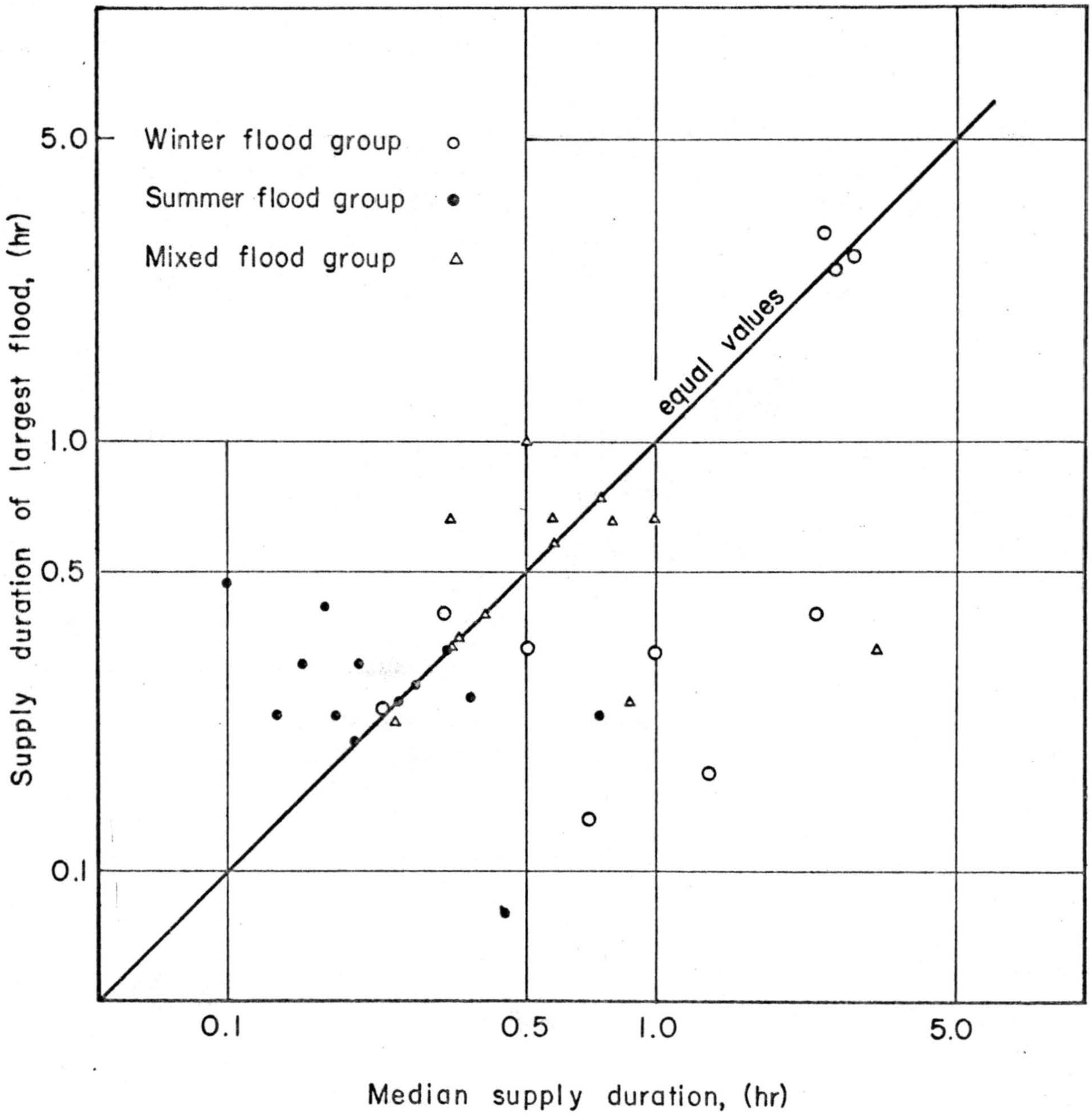

Fig. 12. Supply duration relationship

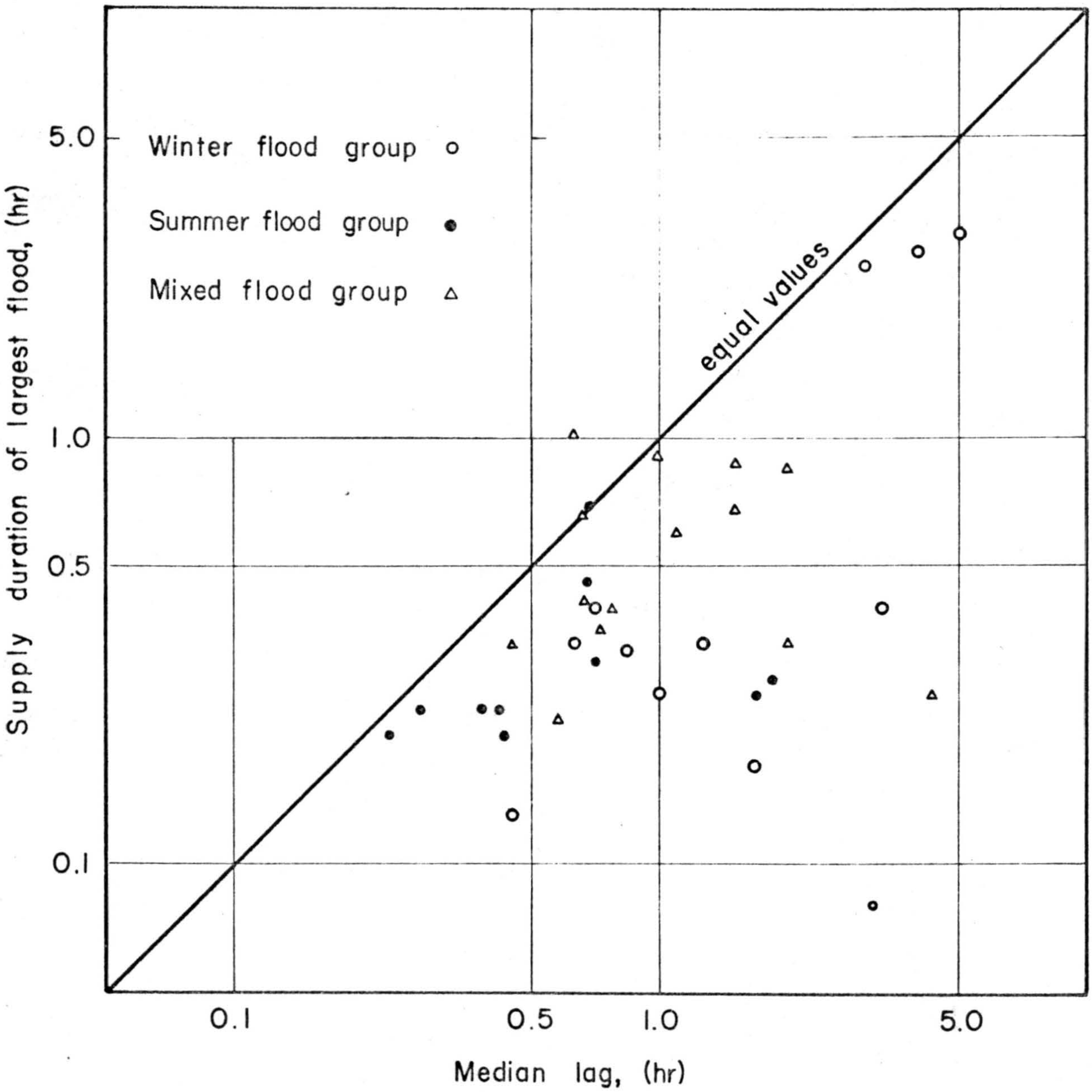

Fig. 13. Lag - duration correlation

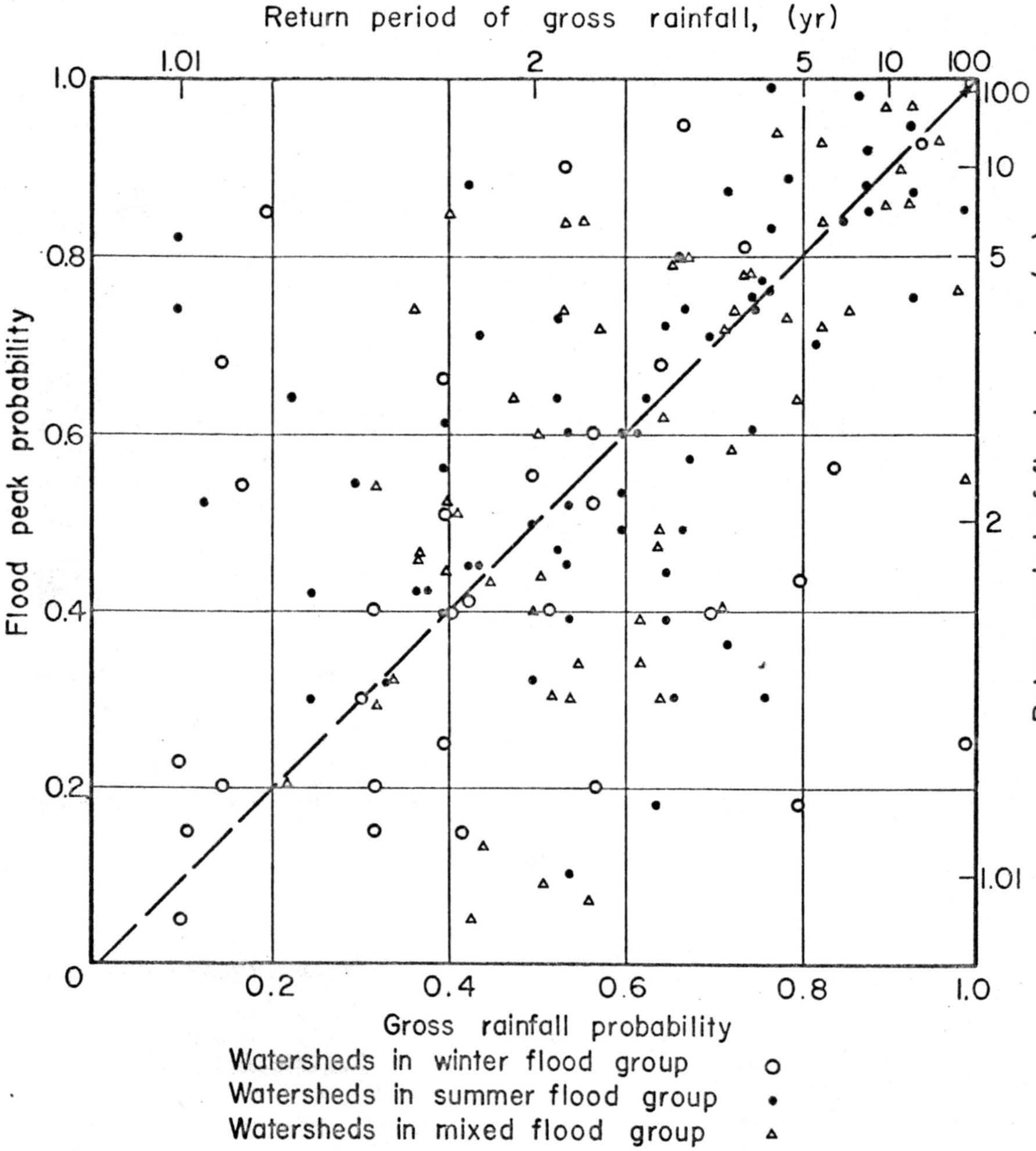

Fig. 14. Probability of rainfall and probability of associated flood peak

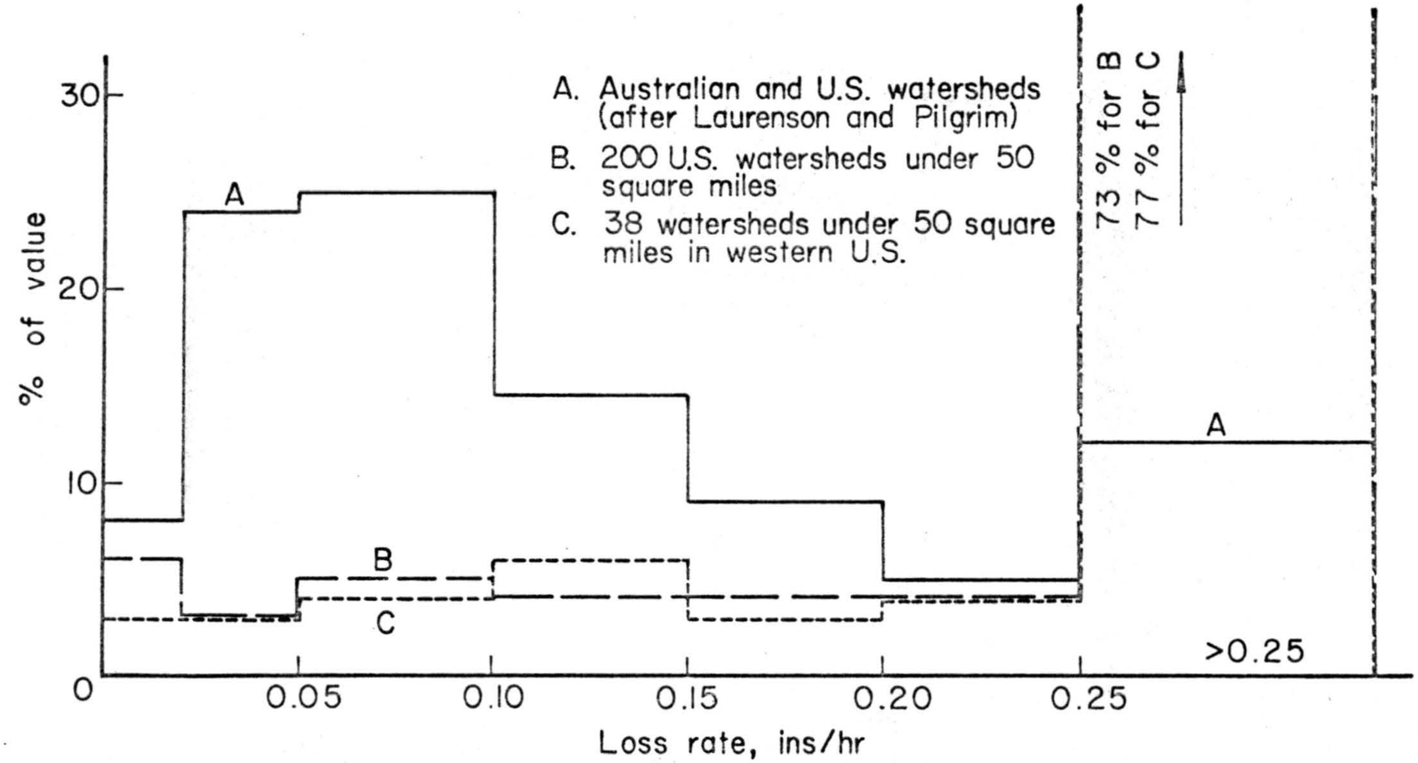

Fig. 15. Comparison of loss rate distributions (using same class intervals as refs. 36 & 37)

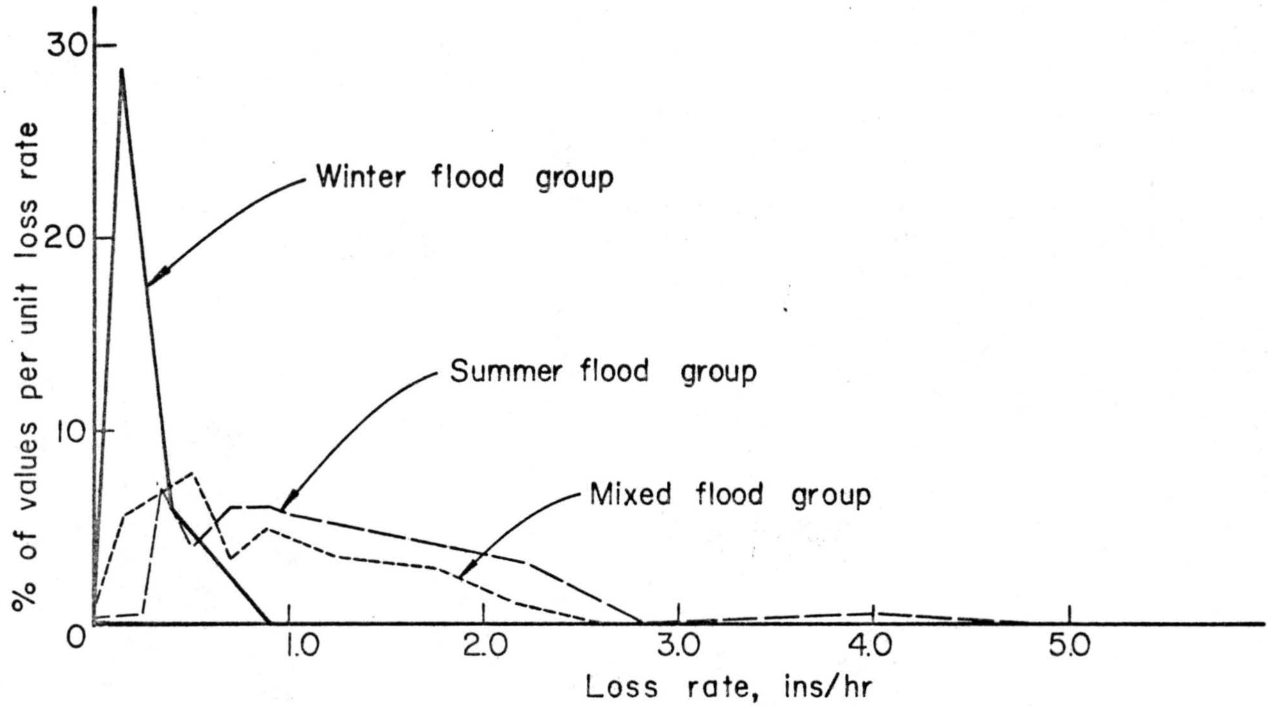

Fig. 16. Loss rate distributions for sample watersheds

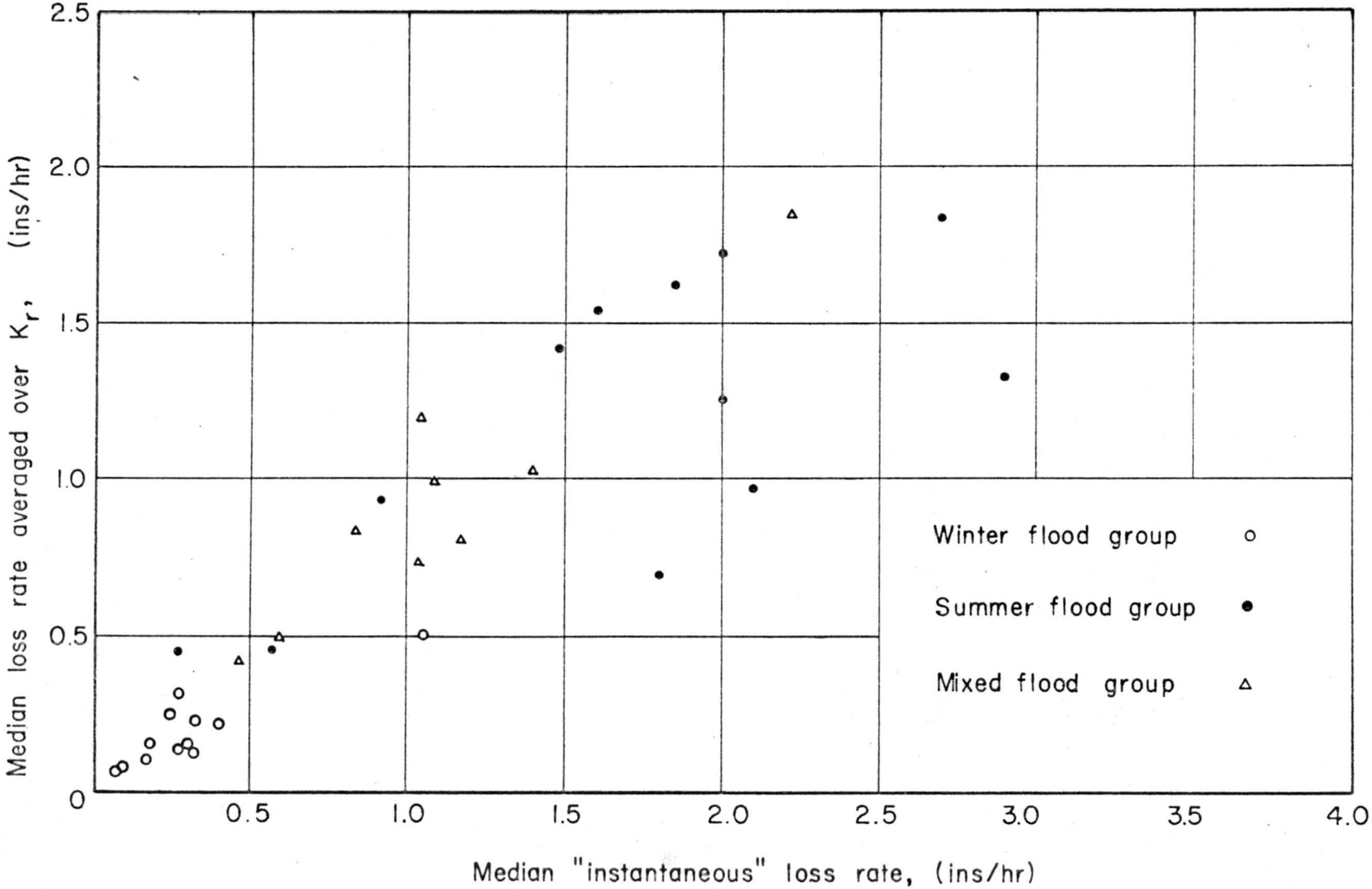

Fig. 17. Loss rate relationships

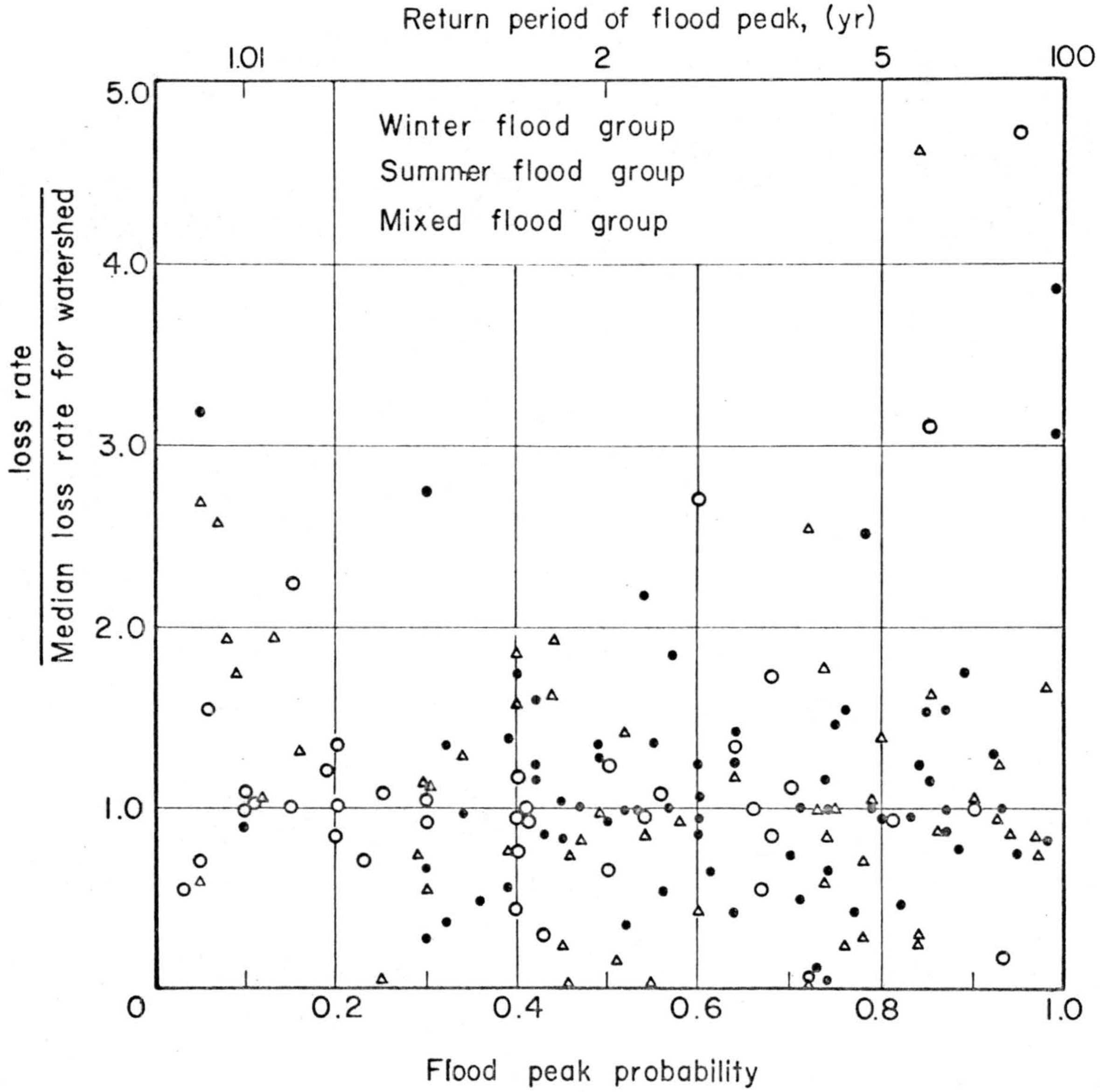

Fig. 18. Relationship between loss rates and flood magnitudes

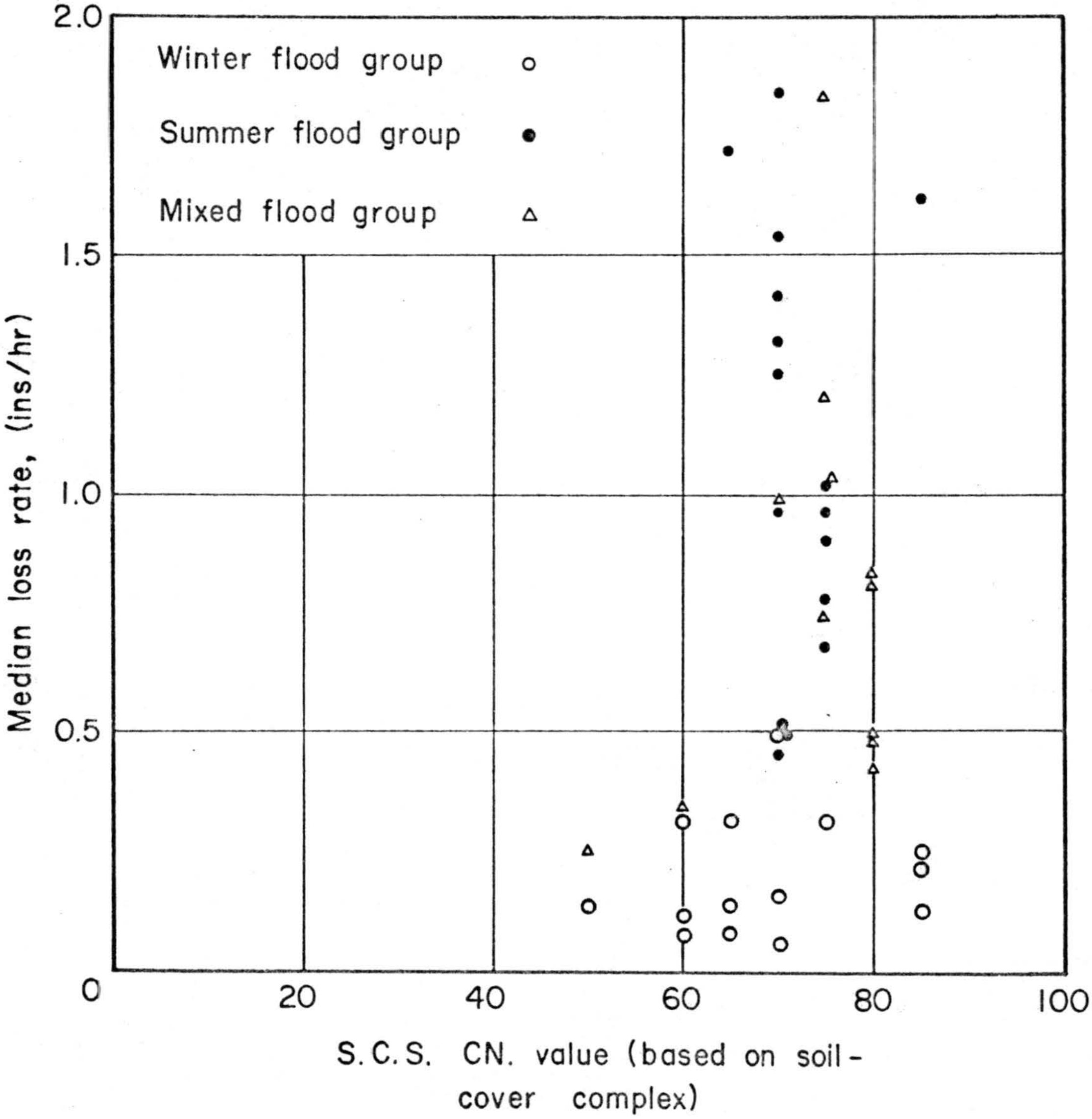

Fig. 19. CN versus median loss rate

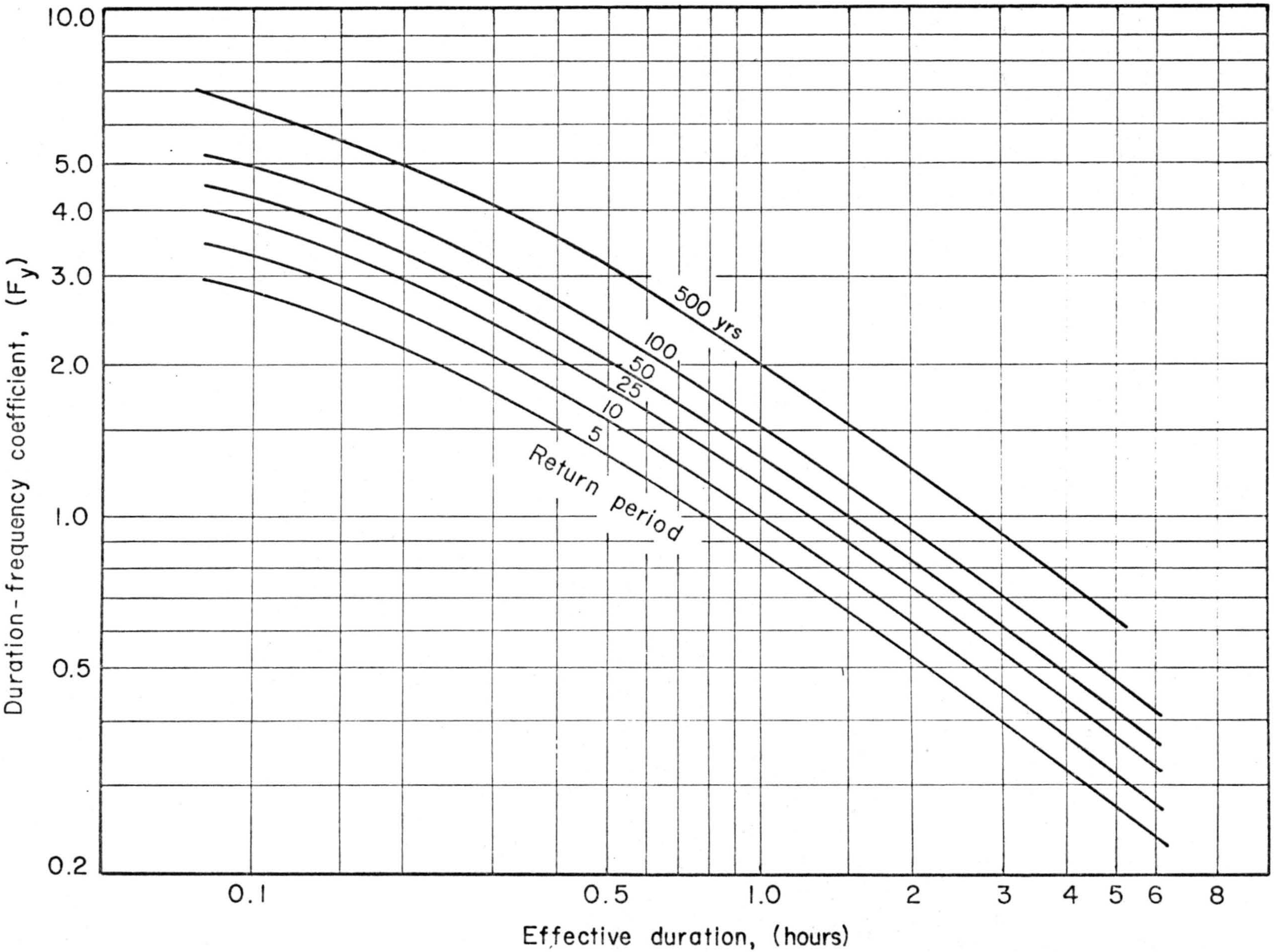

Fig. 20. Duration frequency coefficient