HYDROLOGICAL PROCEDURE NO. 11

DESIGN FLOOD HYDROGRAPH ESTIMATION FOR RURAL CATCHMENTS IN PENINSULAR MALAYSIA

JABATAN PENGAIRAN DAN SALIRAN
KEMENTERIAN PERTANIAN MALAYSIA

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CONTENTS

SYNOPSIS

The design of many water control structures requires a reliable and realistic estimate of the design floodhydrograph. When no streamflow records are available, as is often the case, the design flood hydrograph may be derived from a design storm.

This procedure presents a deterministic method of estimating the design flood hydrograph for ungauged rural catchments in Peninsular Malaysia. The procedure is based on the development of three components: a design storm, a rainfall-runoff relationship, and a triangular hydrograph. The procedure was tested on 12 gauged catchments and gave average results. The limitations of the procedure are discussed and a number of worked examples illustrating the use of the procedure are given.

DESIGN FLOOD HYDROGRAPH ESTIMATION FOR RURAL CATCHMENTS IN PENINSULAR MALAYSIA

1. INTRODUCTION

A problem commonly encountered in the engineering field is the determination of the design flood. The design flood may be defined as the flood adopted for the design of a water control structure after consideration of the hydrologic and economic factors.

Design flood estimation using established methodology is relatively simple when records of streamflow and rainfall are available for the catchment concerned. The difficulties arise when no such records are available in which case the designer is faced with two alternatives:

(i) To instrument the catchment for the period required to collect the hydrological data necessary to derive the design flood.

(ii) To estimate the design flood using a flood estimation procedure.

The former approach is generally time consuming and therefore expensive and is generally only warranted on projects involving major capital expenditure. The latter approach is undoubtedly subject to a greater degree of uncertainty, but nevertheless has to be used in the absence of any hydrological data. Design flood estimates made using a flood estimation procedure should therefore be interpreted sensibly within the limitations of the method, and where possible checked using any alternative flood estimation methods available. Aitken (1973) has reviewed existing methods of flood estimation in common usage.

Two procedures for estimating the design flood from ungauged rural catchments have recently been adopted for use in Peninsular Malaysia — the Rational Method (Heiler 1974 $\not{A}$) and the Regional Flood Frequency Method (Heiler and Chew 1974). Both procedures have been compiled from flood studies on Malaysian catchments.

The Rational Method and the Regional Flood Frequency Method provide a means of estimating the design flood peak only. Although this is often sufficient, the design of many engineering works requires a consideration of storage upstream of the structure e.g. dam spillways, culverts with upstream ponding etc. The complete design flood hydrograph is therefore necessary for the determination of the design discharge at the point of interest.

This procedure complements the two procedures mentioned previously in providing a method of estimating the total flood hydrograph for ungauged rural catchments. The procedure is not applicable to urban catchments.

2. SPECIFICATION FOR PROCEDURE

The requirements considered necessary for this procedure were that it should:

(i) Estimate the peak flow, the volume and time distribution of runoff for various recurrence intervals.

(ii) Account for the significant differences in the catchment characteristics that affect floods.

(iii) Utilize catchment data that can be readily determined from topographical maps.

(iv) Be simple and relatively fast to apply.

3. DEVELOPMENT OF PROCEDURE

3.1 General

Most synthetic procedures for estimating design flood hydrographs are deterministic in that the design flood is derived from a hypothetical design storm. A review of some of the more widely used procedures for estimating design flood hydrographs has been made by Cordery et al (1970). Three basic steps are common to this methodology of flood estimation:

(i) The specification of the design storm of which the important characteristics are usually the recurrence interval, the total rainfall volume, the areal distribution of rainfall over the catchment, the temporal distribution of rainfall, and the duration of rainfall.

(ii) The estimation of the runoff volume resulting from the design storm.

(iii) The estimation of the time distribution of runoff from the catchment.

These three main components were studied in the development of this procedure.

Over recent years there have been numerous and diverse techniques developed for estimating all of the above components. However it is considered that the main limitation in the development of a design flood hydrograph estimation procedure lies in the availability of rainfall and streamflow data, rather than any inherent limitations in the techniques used to develop the procedure. In this respect the problem is that there are very few major floods for which reliable rainfall and streamflow data are available, particularly on small catchments. Any relationships developed are therefore based on data from relatively small storms, and hence the flood estimates are made from extrapolated relationships.

The techniques used in the development of this procedure have been adopted primarily to retain a degree of simplicity commensurate with the data available.

3.2 The Design Storm

3.2.1 Recurrence Interval

As with most deterministic flood estimation procedures it is assumed that the recurrence interval of the design flood equals the design storm recurrence interval.

The choice of design recurrence interval reflects the severity of the potential damage in the event of the design flood being exceeded. On large schemes the design recurrence interval is usually based on a cost benefit analysis. On smaller schemes it is too difficult to quantify the costs pertaining to flood damage and the benefits derived from flood alleviation, and the design recurrence interval is usually selected somewhat arbitrarily. The design flood estimation procedure developed in this investigation is intended to apply to smaller schemes where the latter approach holds.

Heiler and Tan (1974) have recommended design recurrence intervals for different types of water control structures in Malaysia. Although these recommendations are tentative they serve as a useful *guide* to the selection of design recurrence intervals for small schemes.

In cases where there is considerable risk of major damage and loss of life in the event of the design flood being exceeded, it is usual practice to calculate the upper limit of the flood regime. This upper limit is the probable maximum flood and it is derived from the probable maximum storm. The probable maximum storm represents the upper limit of the design storm, for which the recurrence interval is not defined.

The techniques for estimating the probable maximum storm are beyond the scope of this procedure since it is considered that the degree of security involved warrants a fairly detailed hydrological analysis specific to the project under consideration.

3.2.2 Point Rainfall Depth and Frequency

A comprehensive depth-duration-frequency study of storm rainfall throughout Peninsular Malaysia has been compiled in the form of a hydrological design procedure by Heiler (1973). This procedure can be used to estimate the depth of rainfall for a specified duration and frequency for any point location in Peninsular Malaysia. In the procedure the short duration rainfall refers to intense bursts of rain within a storm and not the total rainfall resulting from the storm.

The procedure is based on relatively short term data (5–13 years) from 59 recording raingauges and 32 daily raingauges distributed throughout Peninsular Malaysia. It is considered that the procedure gives a reasonably reliable estimate of point rainfall depths, although it should be realized that local variations are likely to occur due to the relatively thin population of available raingauges used to develop the procedure.

3.2.3 Areal Distribution

During a storm, rainfall is usually distributed unevenly over the catchment area with rainfall depths decreasing with distance from the storm center. Large variations in rainfall depth can occur over relatively short distances, particularly in areas where thunderstorm activity is common. This characteristic of rainfall in Malaysia has been noted by Dale (1959), although no studies on the areal patterns of storm rainfall for small catchments in Malaysia have been found in the literature. A uniformly distributed areal pattern has been adopted for this procedure using an areal reduction factor to reduce the point rainfall estimate to an areal average.

In Table 8 (pg. 13) of his procedure, Heiler gives factors for the reduction of point rainfall to areal mean rainfall for a range of rainfall durations and catchment areas. These factors are based on the U.S. Weather Bureau recommendations, since only a limited study of Malaysian storms has been done by the D.I.D. (1970).

During the study reported on here, 92 storms on 18 different catchments were analysed to compute the ratio of catchment mean rainfall to the maximum recorded point rainfall. The catchment mean rainfall was calculated by the Thiessen weighting method. Because most of the study catchments were equipped with only 1 recording raingauge, only 24 hour durations could be investigated using records from the daily raingauges on the catchment.

The areal reduction factors for the study data are shown in Fig. 1 together with the U.S. Weather Bureau recommendations. The plotted points define a typical rainfall depth-area trend with the con-

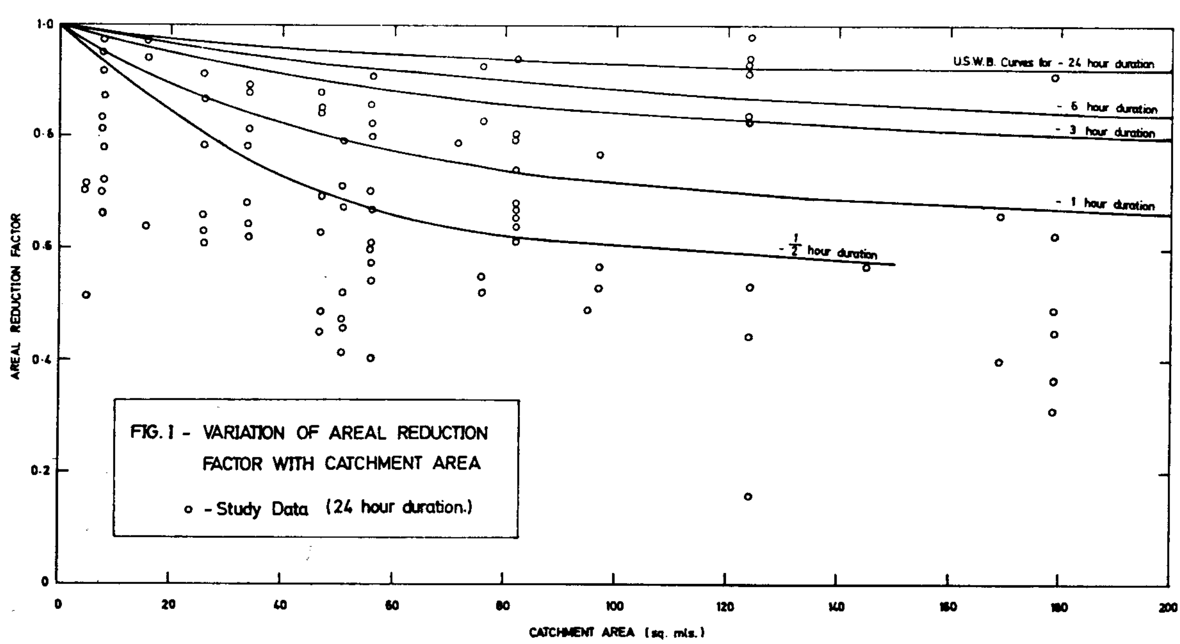

FIG. 1 - VARIATION OF AREAL REDUCTION FACTOR WITH CATCHMENT AREA

o - Study Data (24 hour duration)

siderable scatter in the points reflecting different storm rainfall characteristics and recurrence intervals. The U.S. Weather Bureau curve for 24-hour duration rainfall forms an upper envelope containing most of the study data. It is considered that this curve represents the likely upper limit of the variation of the areal reduction factor with catchment area for 24-hour rainfall typical of the more severe flood-producing storms. The variation of the aeral reduction factor with catchment area for short duration rainfall as recommended by the U.S. Weather Bureau is also shown in Fig. 1. At present there is insufficient data to deny the validity of these factors in Malaysia and they are considered satisfactory for use in this procedure.

3.2.4 Temporal Distribution

The temporal distribution of the design storm is usually adopted from a study of the temporal distribution of recorded storms, as that pattern which produces the maximum discharge from the catchment (Chow 1964 pg. 25–29). The D.I.D. (unpublished data) has compiled the temporal pattern for a large number of storms of varying durations recorded at a number of rainfall stations in Malaysia. The results indicate that the median temporal pattern is when 80% of the total rainfall falls in the first 50% of the storm duration. A design storm temporal pattern of 80% of the rainfall in the first 60% of the storm duration has been recommended for general flood estimation in Singapore by Chang (1969/70). He derived this pattern as the mean pattern from 377 storms recorded in Singapore. No relationship was found between the temporal pattern and the duration and depth of rainfall.

The temporal pattern indicated by the D.I.D. data for Malaysia is not necessarily typical of the storms producing large floods, since it is based on point sampling of small and large storms whose potential flood-producing characteristics are unknown. The pattern also refers to the total rainfall during the storm, not the intense bursts of rain within the storm on which the procedure by Heiler is based.

It is shown later (section 5.2) that quite large errors are possible in predicting the volume of rainfall excess from the storm rainfall. In view of this limitation and the lack of data on the temporal pattern of large flood-producing storms, the refinement of a varying design storm temporal pattern was considered to be not justified. The flood estimation procedure developed in this investigation therefore does not include the temporal pattern as a design storm parameter.

3.2.5 Duration

The design storm duration is usually adopted as that duration which gives the maximum discharge. This critical duration is found by trial and error by calculating the design flood for a range of storm durations. A similar practice has been adopted in this procedure.

3.2.6 Design Storm Recommendations

The following procedure is recommended for estimating the design storm:

(i) The design recurrence interval is selected on the basis of the guidelines suggested by Heiler and Tan ("Hydrological Design Return Periods", Provisional Hydrological Procedure, Drainage and Irrigation Department, Malaysia 1974).

(ii) The design storm depth for the required recurrence interval is calculated from the procedure by Heiler ("Estimation of the Design Rainstorm", D.I.D. Hydrological Procedure No. 1, Ministry of Agriculture and Fisheries, Malaysia).

(iii) The areal reduction factors recommended by Heiler are used to convert point rainfall to catchment mean rainfall.

(iv) The temporal pattern of the design storm is not considered.

(v) The design storm duration is taken as that duration which gives the highest peak discharge.

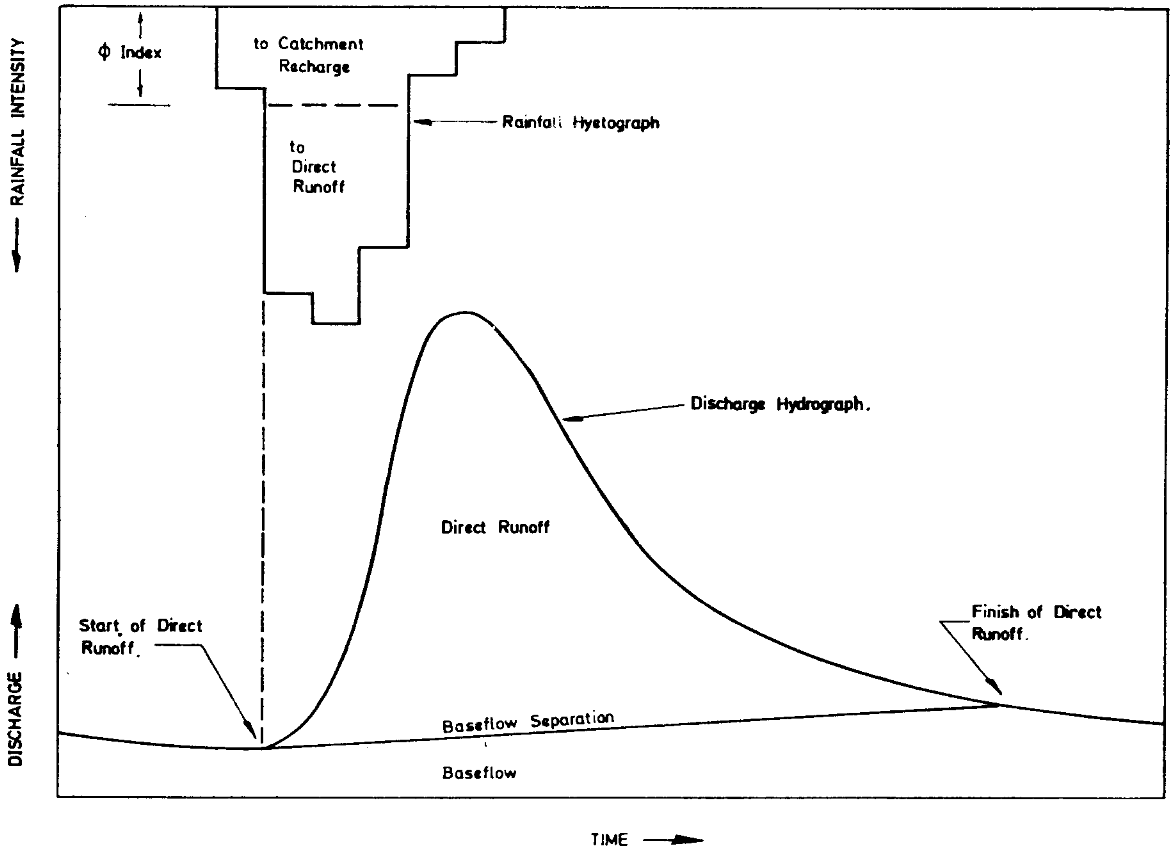

FIG. 2 - HYDROGRAPH SEPARATION

3.3 The Estimation of Runoff from Rainfall

3.3.1 Rainfall-Runoff Analysis

The purpose of this section is to briefly describe the general principles of rainfall-runoff analysis. Since there are considerable differences in the interpretation of common hydrological terms, it is important to define the meaning of the terms as used in this study.

Referring to Fig. 2 the discharge hydrograph is a graph of instantaneous discharge at the catchment outlet versus time. The area under the hydrograph over any specified time interval is runoff. The discharge hydrograph can be separated into two components — direct runoff and baseflow. Direct runoff refers to that volume of the runoff which reaches the catchment outlet soon after the causative rainfall. The residual flow is baseflow.

Since direct runoff and baseflow are empirical concepts, the method of baseflow separation is largely empirical . In this study the baseflow was separated by drawing a straight line from the start of rise of the hydrograph to an arbitrary point on the recession of the hydrograph. The arbitrary point was determined as the point after which the recession curve plotted as a straight line on log-artihmetic paper. The log-arithmetic plot was done for 1 or 2 hydrographs for a catchment, and the remaining hydrographs were separated by inspection, keeping the length of the baseflow separation line more or less constant.

The rainfall hyetograph (Fig. 2) is a plot, in discrete form, of the rainfall intensity over the catchment versus time. The point rainfall intensity was extracted at hourly intervals from the recording raingauge data and then adjusted linearly so that the area under the hyetograph equalled the catchment mean rainfall over the same period. As mentioned previously the catchment mean rainfall was determined as the Thiessen weighted mean of the 24 hour rainfall totals recorded by the daily raingauges.

The rainfall hyetograph can also be divided into several empirical components. The ϕ index or loss rate is defined as the rainfall intensity above which the volume of rainfall or rainfall excess equals the volume of direct runoff. The volume of rainfall below this intensity goes to catchment recharge.

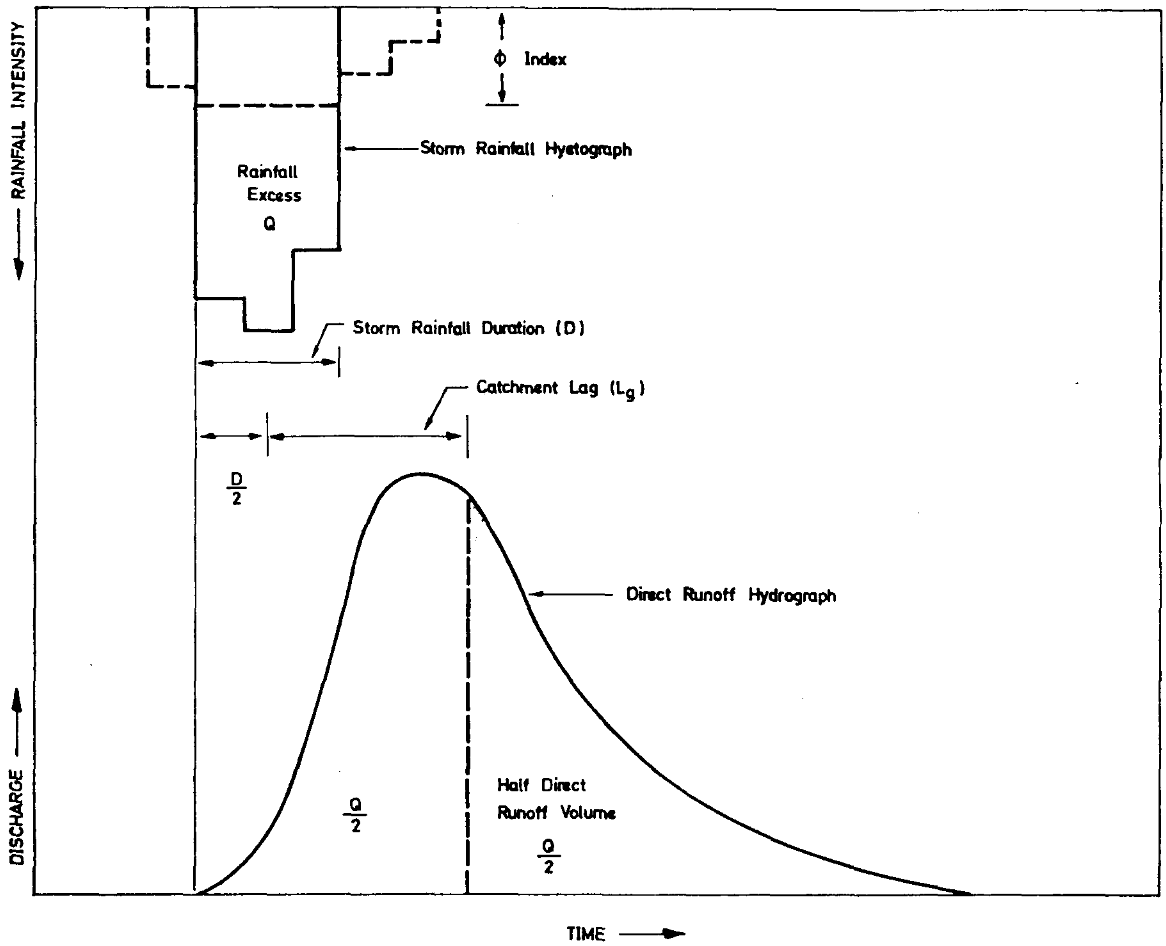

FIG. 3 - ANALYSIS OF DIRECT RUNOFF HYDROGRAPH

In flood studies the portion of the discharge hydrograph of major interest is the direct runoff hydrograph, and the portion of the rainfall hyetograph of interest is the storm rainfall (Fig. 3). The storm rainfall is that portion of the rainfall hyetograph for which the intensity exceeds the ϕ index. It is assumed that rainfall of intensity less than the ϕ index does not contribute to direct runoff.

The time distribution of direct runoff can be measured objectively by the lag parameter L_g. As shown in Fig. 3, L_g is the time from half the duration of rainfall excess to half the volume of direct runoff.

The concepts described above are largely empirical and obviously over-simplify the very complex relationship between rainfall and runoff. Nevertheless they are fundamental to hydrological analysis and are considered satisfactory for the purposes of this study.

3.3.2 The Rainfall-Runoff Relationship

In practice there are two methods of deriving the volume of runoff from the volume of rainfall. The first method is the loss rate approach in which an initial loss prior to the onset of direct runoff and a continuing loss during the storm are abstracted from the design storm. The selection of a continuing loss or ϕ index for the design storm is usually the most important factor in this approach.

In the second approach a rainfall-runoff relationship is developed from which the volume of runoff can be estimated for any design storm volume. This approach was adopted for this procedure since it is felt that the use of loss rates tends to obscure the difficulties associated with estimating volumes of runoff for design storms which are usually outside the range of the observed data.

It is important for the rainfall-runoff relationship to be compatible with the design storm as estimated from the procedure described in section 3.2. This procedure is based on a frequency analysis of intense bursts of rain within a storm. Since only the intense rainfall is assumed to contribute to direct runoff, it is necessary to use storm rainfall as defined previously and not total rainfall in the development of the rainfall-runoff relationship.

In this study 175 storms from 38 catchments in Peninsular Malaysia were analysed. For each storm the volume of direct runoff, the volume of storm rainfall and the ϕ index were computed using the methods described in section 3.3.1. Of the data analysed 97 storms from 19 catchments were used to develop the rainfall-runoff relationship. These data are listed in Appendix C. On the remaining catchments the sparse areal coverage of raingauges did not permit reliable estimates of the storm rainfall and these data were therefore not used.

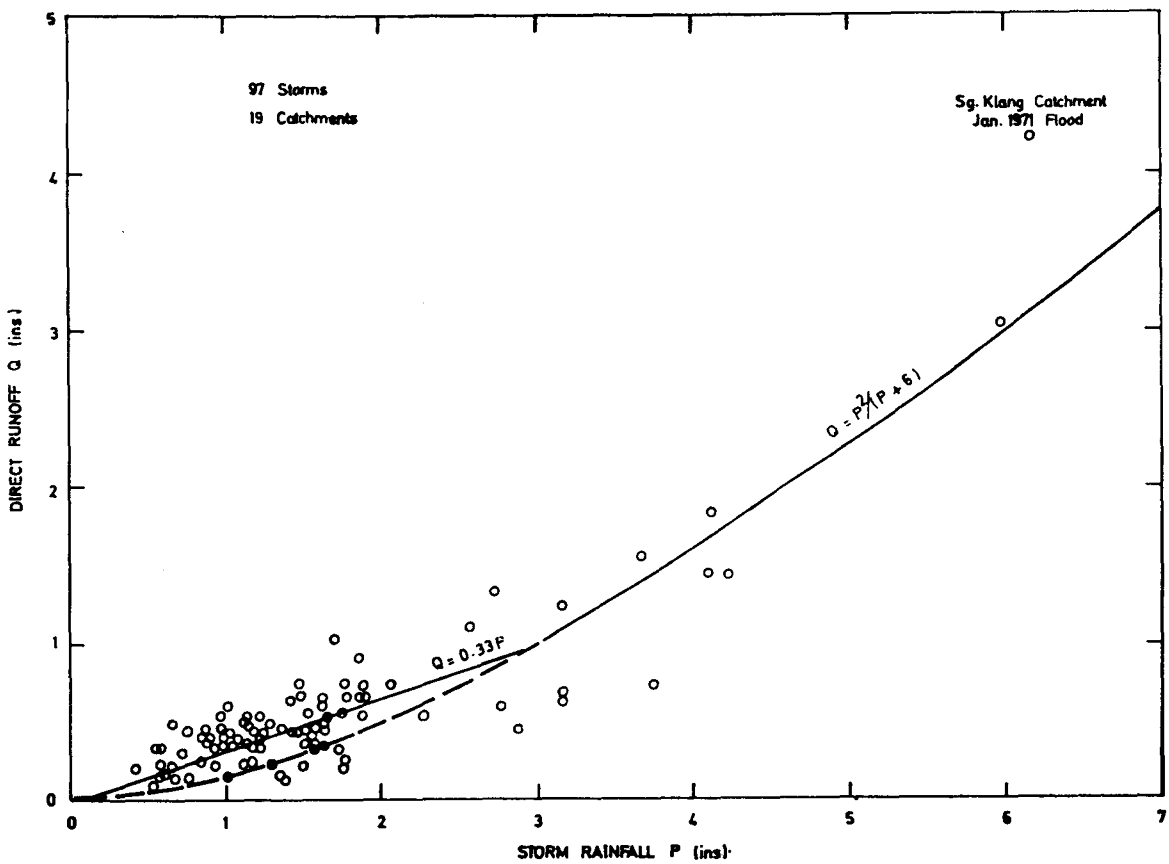

FIG. 4 - RAINFALL - RUNOFF RELATIONSHIP

The direct runoff plotted against storm rainfall is shown in Fig. 4. The scatter of the points is to be expected since the volume of runoff varies with other factors in addition to rainfall amount, such as the catchment moisture status prior to the storm, the surface cover, the soil type and the intensity of rainfall. To account for some of these additional factors, multiple correlation methods have been used as for example by Linsley et al (1958 pg. 173–178).

In this study an attempt was made to include the catchment antecedent moisture status in the rainfall-runoff relationship. A 5–day antecedent rainfall index and a baseflow index were tried. Although the latter index accounted for some of the scatter in Fig. 4 the results were not conclusive enough to justify including an index of catchment antecedent moisture status in the rainfall-runoff relationship.

For practical purposes it is useful to express the rainfall-runoff relationship as a mathematical equation rather than graphically. Several forms of equation have been reported by Boughton (1966) who used a hyperbolic tangent function and Chow (1964 pg. 21–28) who describes an empirical equation developed by the U.S. Soil Conservation Service and of the form:

$$Q = \frac{P_e^2}{(P_e + I)}$$

where Q = direct runoff (ins.)
P_e = total rainfall during the storm minus the initial loss (ins.)
I = potential infiltration (ins.)

The value of I is dependent on the soil type, ground cover and the antecedant moisture conditions.

A similar form of equation has been adopted for this procedure except that the storm rainfall P is used in place of P_e, and a single value of I is used. The choice of equation was somewhat arbitrary since there were insufficient data to indicate the suitability of any one type of equation. However the above equation is attractive in that the proportion of estimated runoff relative to rainfall increases as the storm rainfall increases. This is logical since the catchment recharge is high in the early stages of a storm and decreases to a more or less constant rate as the storm duration increases.

In Fig. 4 the equation was fitted to the observed data by eye giving emphasis to the relatively few points representing the larger floods analysed. The fitted curve does not match the observed data for the smaller storms, and for storm rainfall below 3 inches the linear relationship shown in Fig. 4 is recommended. The equations for estimating direct runoff Q from storm rainfall P are:

for storm rainfall less than 3 inches,

$$Q = 0.33P \tag{3.1}$$

for storm rainfall greater than 3 inches,

$$Q = \frac{P^2}{(P+6)} \tag{3.2}$$

Because of the scarcity of data for major storms, the uncertainties associated with predicting runoff volumes from rainfall are clearly evident in the relationship shown in Fig. 4. Comparison with other rainfall-runoff studies in Malaysia by Goh (1972), Tan (1967) and Charlton (1964) indicate that the relationship developed is reasonable within the range of data available.

A comparison with overseas data is available from the review made by Pilgrim (1966) of loss rates for catchments in the U.S.A., Australia and New Zealand. Frequency diagrams of loss rates for each country are shown in Fig. 5, together with a frequency diagram of loss rates computed in this study. Since the methods used to derive the loss rates are empirical and therefore to some extent subjective, only general comparisons can be drawn from Fig. 5. However it is obvious that loss rates for Malaysian catchments are substantially higher than loss rates observed in the U.S., Australia and New Zealand. In terms of the rainfall-runoff relationship this means that the volume of direct runoff as a proportion of storm rainfall is considerably less for Malaysian catchments than for catchments in the U.S., Australia and New Zealand.

3.4 The Time Distribution of Runoff

3.4.1 General

There are several methods of distributing the direct runoff volume with time of which the best known is probably the unit hydrograph. Numerous studies of the unit hydrograph have been made and despite some criticism of its theoretical validity, the unit hydrograph is widely used in design flood estimation. One advantage of the unit hydrograph is that it can be used to distribute runoff from storms of varying temporal pattern. The unit hydrograph also preserves the curvilinear shape characteristic of observed hydrographs. A practical disadvantage of the unit hydrograph is that it is fairly tedious to apply.

In this procedure a triangular distribution of direct runoff is adopted. The triangular hydrograph is very simple to apply and reproduces the hydrograph shape sufficiently accurately for design purposes. It is developed from the dimensionless hydrograph described in the following section.

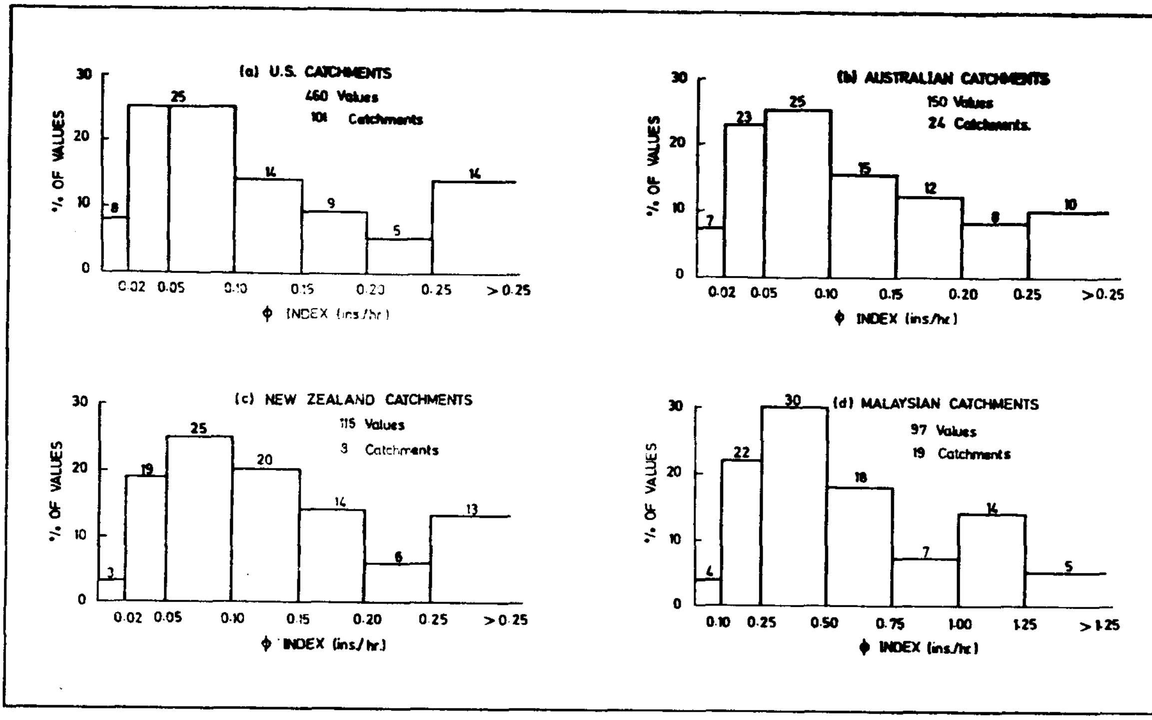

FIG. 5 - FREQUENCY DIAGRAMS OF LOSS RATES
(a,b,c from Pilgrim 1966)

3.4.2 The Dimensionless Hydrograph

The basic assumption underlying the dimensionless hydrograph method as described in the U.S. Bureau of Reclamation Manual ("Design of small Dams" pg. 39–41) is that the shape of the direct runoff hydrograph reflects the influence of all the catchment characteristics on the flood-producing rainfall. Conversion of the direct runoff hydrograph to a dimensionless form eliminates most of the effects of the catchment characteristics and enables the comparison of the essential features of the direct runoff hydrograph for different catchments. The ordinate and abscissa of the dimensionless hydrograph are defined as:

$$\text{dimensionless ordinate} = \frac{q(t) \times (L_g + D/2)}{Q \times A \times 640} \tag{3.3}$$

$$\text{dimensionless abscissa} = \frac{t \times 100}{(L_g \times D/2)} \tag{3.4}$$

where q(t) = discharge ordinate of the direct runoff hydrograph (cusecs)
t = time (hrs)
L_g = catchment lag (hrs)
D = duration of storm rainfall (hrs)
A = catchment area (sq. mls.)
Q = direct runoff (ins.)

Equations (3.3) and (3.4) describe a curvilinear dimensionless hydrograph. Because the dimensionless form largely eliminates the effects of catchment characteristics, the dimensionless hydrographs for catchments having similar flood-producing features show certain similarities. A representative dimensionless shape can be derived which is characteristic of any group of hydrologically similar catchments. It is assumed that this dimensionless hydrograph can be applied to similar ungauged catchments.

3.4.3 Triangular Hydrograph

For practical purposes the important shape parameters of the direct runoff hydrograph are the peak flow and some measure of the time distribution of which L_g is a convenient index. The direct runoff hydrograph can thus be represented by a triangular hydrograph as shown in Fig. 6.

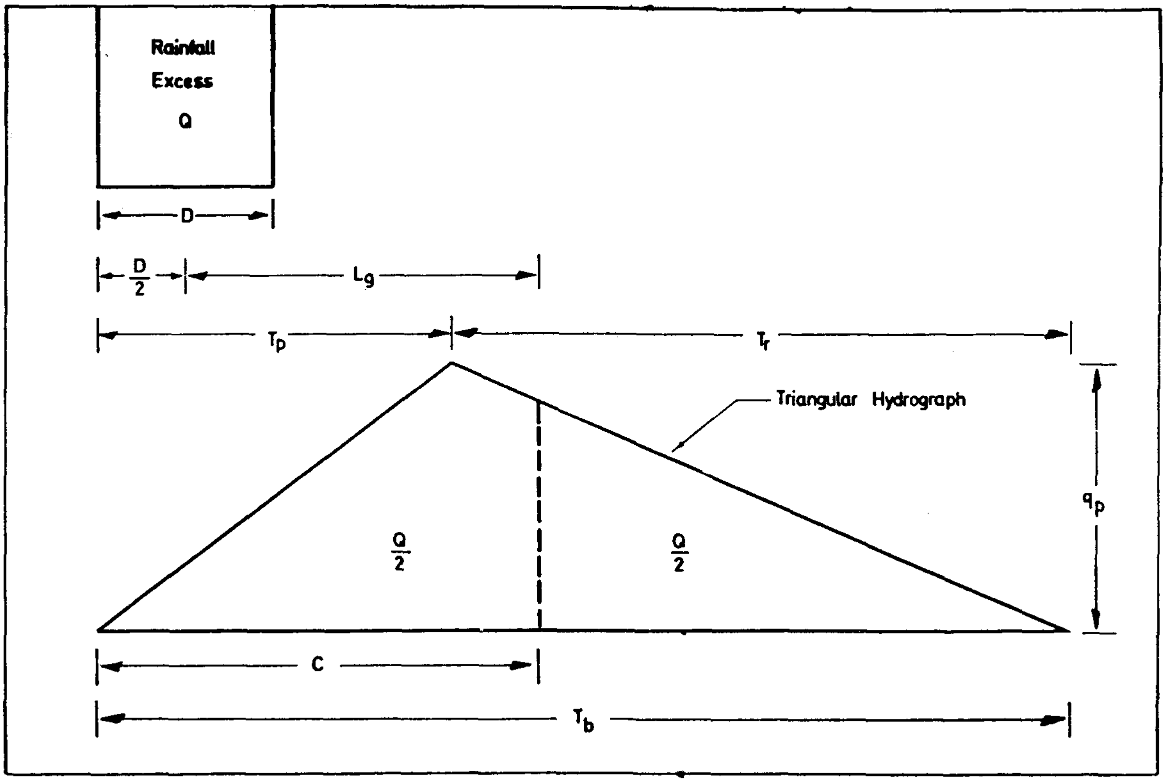

FIG. 6 - TRIANGULAR REPRESENTATION OF DIRECT RUNOFF HYDROGRAPH

The peak discharge of the triangular hydrograph is derived from equation (3.3) as

$$q_p = \frac{D_p \times A \times 640 \times Q}{(L_g + D/2)} \tag{3.5}$$

where q_p = peak discharge of the triangular hydrograph (cusecs).
D_p = peak ordinate of the dimensionless hydrograph that is characteristic of the catchment.

Catchment lag time is a function of the catchment characteristics and D_p is typically constant for any group of hydrologically similar catchments. Therefore if D_p is known and L_g can be estimated for an ungauged catchment, the peak discharge can be estimated and the triangular hydrograph constructed for any storm. The assumption made here is that the lag time is constant for a given catchment and is independent of the storm characteristics.

3.4.4 **Triangular Hydrograph Geometry**

For a given runoff volume the shape of the triangular hydrograph is fully defined by the two parameters q_p and L_g. However for ease of constructing the triangular hydrograph, the base time T_b and the time of rise T_p of the triangular hydrograph can be derived as follows:

Starting with equation (3.5) put $(L_g + D/2) = C$

$$\text{then } q_p = \frac{D_p \times Q \times A \times 640}{C} \tag{3.6}$$

The volume of the triangular hydrograph (Fig. 6) is

$$Q = \frac{\frac{1}{2} \times T_b \times q_p}{640 \times A} \quad q_p \tag{3.7}$$

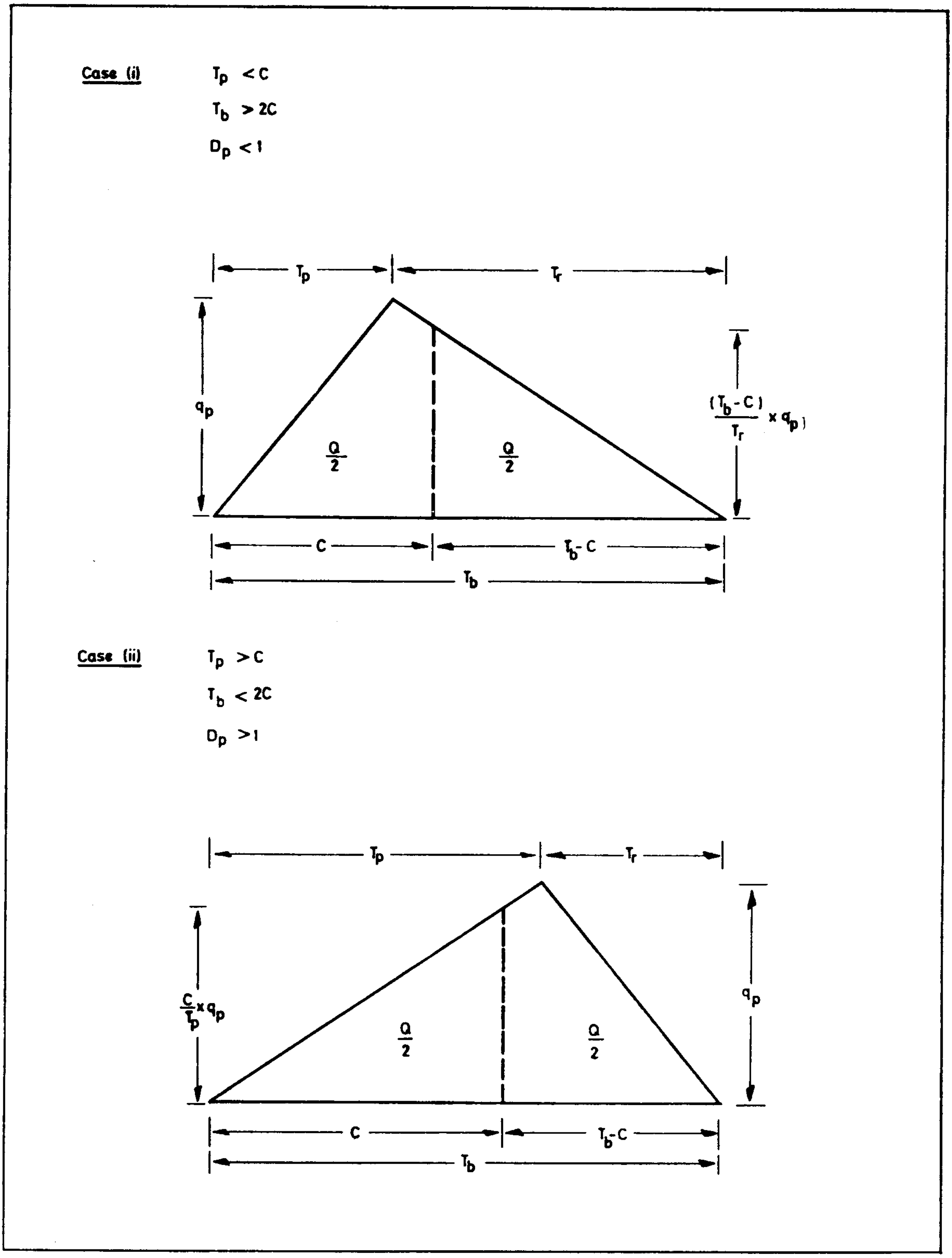

FIG. 7 - TRIANGULAR HYDROGRAPH GEOMETRY

Substituting for q_p from equation (3.6) gives

$$Q = \frac{\frac{1}{2} \times T_b \times D_p \times Q \times A \times 640}{640 \times A \times C}$$

$$\therefore T_b = \frac{2C}{D_p} \qquad (3.8)$$

With the base of the triangular hydrograph fixed there are the two possibilities as shown in Fig. 7 that T_p is either less than C [case (i)] or greater than C [case (ii)].

(a) *Case(i)*

Considering the recession limb of the triangular hydrograph:

$$\tfrac{1}{2}\times Q = \frac{\tfrac{1}{2}\times(T_b-C)\times q_p\times(T_b-C)}{T_r\times 640\times A}$$

$$\therefore\ T_r = \frac{(T_b-C)^2\times q_p}{640\times A\times Q}$$

Substituting for Q from equation (3.7) gives

$$T_r = \frac{2\times(T_b-C)^2}{T_b} \qquad (3.9)$$

$$\text{now } T_p = T_b-T_r$$

$$= T_b-2\times\frac{(T_b-C)^2}{T_b}$$

$$= \frac{(4CT_b-2C^2-T_b{}^2}{T_b} \qquad (3.10)$$

Equations (3.9) and (3.10) are only valid if T_p is less than C.
Therefore from equation (3.10)

$$(4CT_b-2C^2-T_b{}^2)<CT_b$$

$$T_b{}^2-3CT_b+2C^2>0$$

$$(T_b-2C)\times(T_b-C)>0$$

So T_b must be greater than 2C.
From equation (3.8)

$$\frac{2C}{D_p}>2C$$

$$\therefore\ D_p<1$$

So when the dimensionless peak is less than 1, T_b is greater than 2C and T_p is less than C. This corresponds to the normally observed shape of the direct runoff hydrograph where the recession limb is longer than the rising limb.

(b) *Case (ii)*

If $T_b<2C$ then from equation (3.8)

$$\frac{2C}{D_p}<2C$$

$$\text{i.e. } D_p>1$$

This implies a hydrograph shape where the recession limb is shorter than the rising limb. The equation for T_p is derived from Fig. 7 as follows:

$$\tfrac{1}{2}\times Q = \tfrac{1}{2}\times\frac{C}{T_p}\times\frac{q_p}{640\times A}\times C$$

Substituting for Q from equation (3.7) gives

$$T_p = \frac{2C^2}{T_b} \qquad (3.11)$$

The triangular hydrograph can be easily constructed from the three parameters q_p, T_b and T_p as estimated from equations (3.5), (3.8) and (3.10) or (3.11) depending on the value of D_p. Assuming that the design storm is fully defined, the only unknowns in these equations are L_g and D_p. Methods for estimating these parameters for ungauged catchments are described in the following sections. It should be noted that for a given value of D_p the ratio of T_p to T_b is constant.

3.5 The Estimation of L_g for Ungauged Catchments

Values of catchment lag were derived for 38 catchments in Peninsular Malaysia. For each catchment a number of storms (1—11) were analysed by the methods described in section 3.3.1, resulting in a range of L_g values for each catchment.

This variation in lag time for Australian catchments has been studied by Askew (1970), who found that the lag time decreased as the mean discharge rate of the flood hydrograph increased. The variation in catchment lag times for different storms in Malaysia has been discussed in the Pahang River Basin Study (1974). The physical explanation proposed is that subsurface flow is a major component of the flood hydrograph in humid tropical areas where most of the rain that contributes to direct runoff enters the soil. Subsurface flow has a highly variable response to rainfall depending mainly on the antecedent wetness of the various soil layers.

The variation in catchment lag values derived in this study is considered to be partly due to the variable subsurface response and partly due to a partial area storm effect. Depending on the spatial distribution of rainfall and antecedent soil moisture, only part of the catchment may contribute to direct runoff during a storm. Lag times derived from partial area storms reflect the lag characteristics of the runoff-generating part of the catchment only. The median of the lag times derived for each catchment was taken to be the representative value for that catchment. The median lag values for the study catchments are given in Appendix B.

Catchment lag is a quantitative measure of the influence of catchment storage in modifying the shape of the rainfall excess hyetograph. It is to be expected therefore that catchment lag should be related to those physical characteristics that determine the storage behaviour of the catchment. A number of studies relating some measure of catchment lag to catchment characteristics have been carried out previously. Examples are by Snyder (1938), Taylor and Schwarz (1952), McSparran (1968), Cordery and Webb (1974). The relationship adopted for this procedure is of a similar form to that proposed by Linsley et al (1958 pg. 207) as

$$L_g = C_t \times \left(\frac{LL_c}{\sqrt{S}}\right)^n \qquad (3.12)$$

where L = main stream length from the outlet to the catchment boundary (miles)
L_c = main stream length from the outlet to the catchment centroid (miles)
S = weighted mean stream slope (ft/ml.)
C_t, n are constants.

The product LL_c is a measure of the size and shape of the catchment, and S is a measure of catchment topography.

The 38 catchments studied were arranged into three representative hydrological groups. The groups were selected partly on the basis of similar topographical characteristics and partly on a regional basis. For each group, values of L_g were plotted against $LL_c/\sqrt{S}$ as shown in Fig. 8. Initially the parameters C_t and n were computed by least squares, but it was considered that for design flood estimation more emphasis should be given to the lower range of lag times observed in each group. Therefore the values of C_t and n adopted for design flood estimation were determined graphically. The values are given in Table 1.

TABLE 1

VALUES OF C_t AND n FOR EQUATION (3.12)

Catchment Type	C_t	n
Group 1 — Whole catchment very steep and covered in virgin jungle.	2.0	0.35
Group 2 — Upper catchment very steep and jungle covered, lower catchment reaches hilly and covered predominantly with rubber.	4.0	0.35
Group 3 — Whole catchment undulating with variable vegetation including jungle, rubber and agricultural development.	8.0	0.35

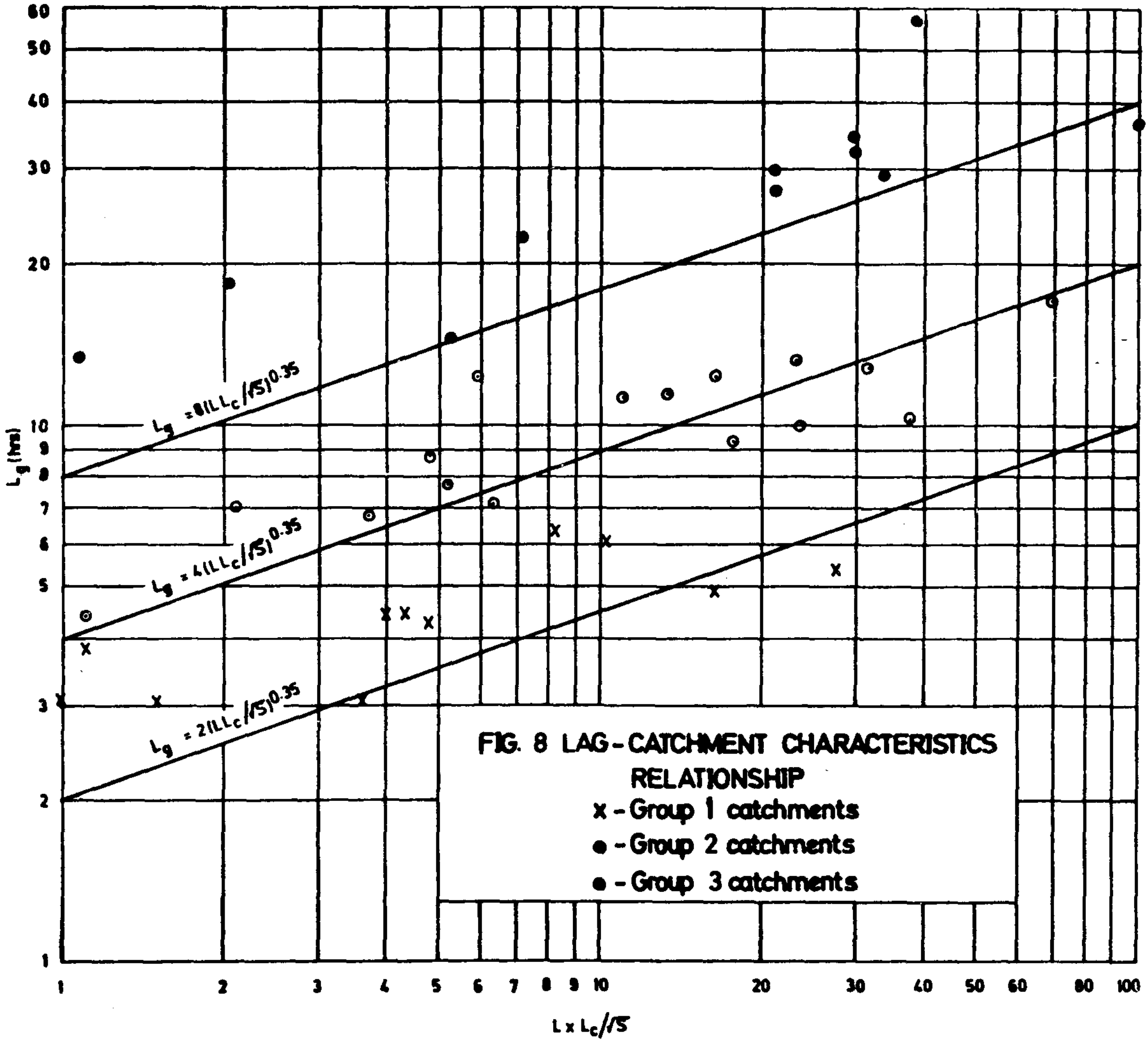

FIG. 8 LAG-CATCHMENT CHARACTERISTICS RELATIONSHIP

Equation (3.12) provides an objective means of computing the lag time of an ungauged catchment. The parameters L, L_c and S are readily determined from a topographical map. The difficulty is to identify which hydrological group the ungauged catchment conforms with. A brief qualitative description of each group is given in Table 1. However it is recommended that the selection of the group be based on a comparison of the topographical similarities of the ungauged catchment with those of the study catchments. This can be done by inspection from a topographical map. It is felt that catchment slope is probably the most important topographical characteristic affecting catchment lag time. The stream slope factor S does give some indication of the catchment slope particularly on the small catchments, but it is not sufficiently representative to provide an objective means of classifying the three hydrological groups.

The lag relationships derived in this study are significantly different from the relationships developed for comparative catchments overseas. Linsley et. al. (1958 pg. 207) reports values for C_t of 0.35 to 1.2 and n equal to 0.38 for some U.S. catchments. The longer lag times observed on Malaysian catchments is again indicative of the high proportion of subsurface flow in the flood hydrographs. The travel time of runoff from where it is deposited as rainfall to where it reaches the stream channel is longer for subsurface flow than for overland flow. This delaying effect is most significant on small catchments where the travel time of runoff in the stream channel is only a small proportion of the total lag time. As the catchment size increases the travel time through the soil becomes less significant than the travel time in the stream channel.

3.6 The Estimation of D_p, T_b, and T_p for Ungauged Catchments

As with L_g, the values of D_p derived from different storms on the catchment varied, and the median value was adopted as being representative of the catchment. The median D_p values for the study catch-

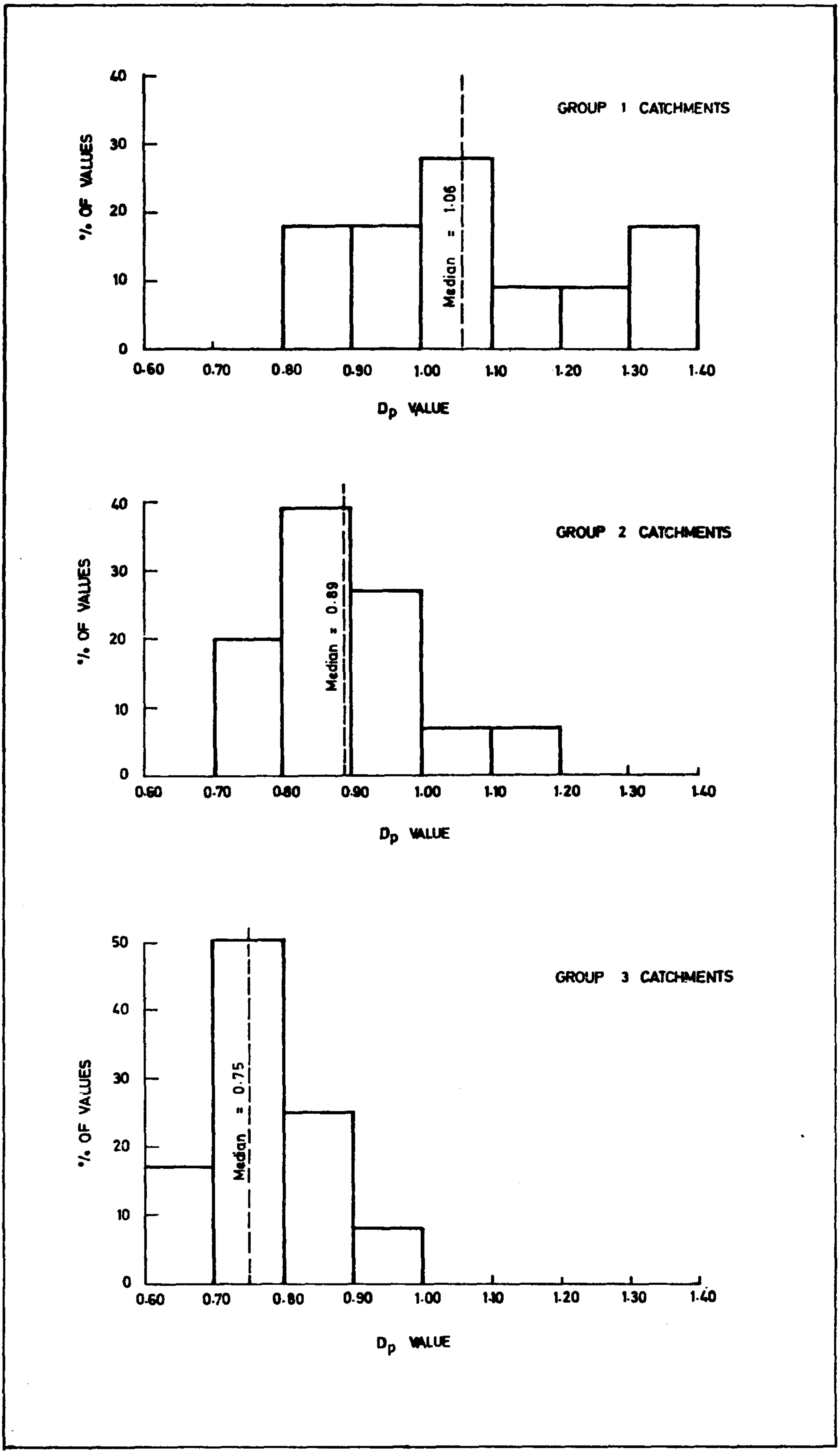

FIG. 9 - FREQUENCY DIAGRAMS OF D_p VALUES

ments are given in Appendix B. The differences observed in the D_p values between catchments were consistent with the three hydrological groups defined previously. Catchments with steep slopes tended to have higher values of D_p than the flat catchments. The distribution of D_p values within each hydrological group is shown in Fig. 9. For design flood estimation the median values of D_p for each group are used (Table 2).

TABLE 2

VALUES OF D_p, T_b AND T_p

Catchment Type	D_p	T_b	T_p	T_p/T_b
Group 1	1.06	1.89C	0.94C*	0.50*
Group 2	0.89	2.24C	0.87C	0.39
Group 3	0.75	2.67C	0.58C	0.22

* Adopted for design flood estimation.

Having fixed the value of D_p, the additional shape parameters T_b and T_p were estimated using equations (3.8), (3.10) and (3.11). For hydrological Group 1 where D_p exceeds 1.0, equation (3.11) gives T_p greater than $T_b/2$. This is contrary to observed hydrograph shapes and arises from the triangular approximation of the curvilinear hydrograph for short, peaky hydrographs. For design purposes it is recommended that T_p be made equal to $T_b/2$ for catchments conforming with Group 1.

The values of D_p, T_b and T_p for the three hydrological groups are shown in Table 2. These values may be used for design flood estimation for ungauged catchments in Peninsular Malaysia.

3.7 The Design Baseflow

The baseflow component of the hydrograph reflects the antecedent rainfall activity over the catchment. It is therefore very difficult to predict the statistical variation of baseflow prior to major floods. Fortunately baseflow is usually a relatively small quantity compared to direct runoff. It is recommended that 5 cusecs per square mile is a satisfactory figure for design baseflow on catchment less than 200 square miles in area. This figure represents moderately wet antecedent conditions as observed on the study catchments.

4. SUMMARY OF PROCEDURE

The steps for estimating the design flood hydrograph are as follows:

1. From a topographical map, compare the topography of the catchment with similar catchments studied in this investigation and select the appropriate hydrological group. Compute L, L_c, A and S for the catchment.

   The measurement of L and A is quite simple. To measure L_c the location of the catchment centroid must be known. This is determined by suspending a cutout of the catchment from three different points, and finding the intersection of plumb lines from each point. L_c is then the stream length from the point of interest to the intersection of the perpendicular from the centroid to the stream alignment.

   The stream slope S is measured as the weighted sum of the incremental slopes between successive stream contours.

$$S = \left[\frac{\sum_{i=1}^{m} l_i \times \sqrt{s_i}}{\sum_{i=1}^{m} l_i} \right]^2$$

where l_i = incremental stream length
S_i = incremental slope

The number of increments m should be sufficient to define the stream profile.

2. Calculate L_g for the catchment using equation (3.12) with n equal to 0.35, and C_t for the group from Table 1.

3. Calculate the design storm for the catchment using D.I.D. Hydrological Procedure No. 1 (Heiler 1973). The design storm should be calculated for a range of durations. Experience suggests that the critical duration giving the highest peak discharge is often similar to the catchment lag time.

4. Calculate Q from equation (3.1) or (3.2).

5. Calculate q_p from equation (3.5). If the total hydrograph is required calculate T_b and T_p from the appropriate values in Table 2.

6. Add the design baseflow component of 5 cusecs per square mile.

7. If the design structure involves storage upstream of the structure, the inflow hydrographs for varying storm durations should be routed through the storage, and the critical outflow hydrograph determined by trial and error.

Some worked examples illustrating the use of this procedure are given in Appendix A.

5. ACCURACY OF PROCEDURE

5.1 General

Two tests were made to assess the accuracy of the procedure. The first test compared recorded flood hydrographs with hydrographs estimated from recorded rainfall. This test indicated how closely the triangular hydrograph represents the time distribution of runoff. The second test compared estimates of the 20-year flood with the 20-year flood determined from a frequency analysis of recorded peak discharges. This test indicated how well the procedure estimates statistical floods. None of the test catchments were used to develop the procedure.

5.2. Test one — Comparison with Recorded Hydrographs

Only one catchment equipped with rainfall and streamflow recorders was available for this test. Details of the test catchment (station number 3118447) are given in Appendix D. Two annual maximum flood

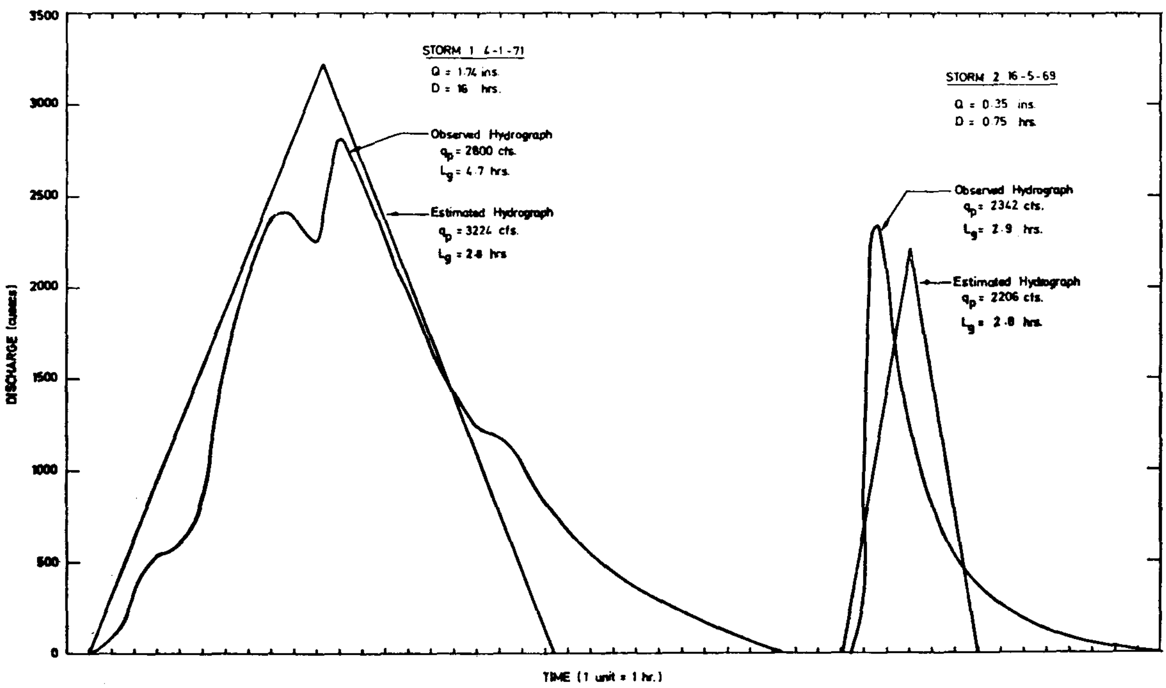

FIG. 10 - COMPARISON OF ESTIMATED AND RECORDED HYDROGRAPHS.

events were analysed to determine the direct runoff hydrograph and the volume and duration of rainfall excess. The triangular hydrographs were then estimated from the recorded rainfall excess using equation (3.5) and (3.12) to estimate L_g and q_p respectively. Values of T_p, T_b and D_p were selected from Table 2 assuming that the catchment is typical of Group 1.

Fig. 10 shows the recorded and estimated direct runoff hydrographs. The first flood is caused by long duration and relatively low intensity rainfall, and the second flood is caused by short duration and relatively high intensity rainfall. For both floods the estimated peak discharges compare reasonably well with the observed peak discharges. The triangular hydrographs represent the rising limb of the observed hydrographs reasonably well, but not the latter part of the recessions. However the tail end of the inflow hydrograph is seldom significant in the design of storage structures and for practical purposes the triangular hydrograph appears to be satisfactory.

For the same two storms the volume of runoff was estimated from the recorded storm rainfall using equations (3.1) and (3.2). A comparison with the recorded runoff volumes as shown in Table 3 demonstrates the possible inaccuracies in estimating volumes of runoff from storm rainfall.

TABLE 3

OBSERVED AND ESTIMATED RUNOFF VOLUMES

Date of Storm	Storm Rainfall (ins)	Direct Runoff (ins) Observed	Direct Runoff (ins) Estimated	Percent Difference
4.1.71	6.14	1.74	3.11	+79
16.5.69	1.80	0.35	0.42	+20

Although the analysis of only 2 storms is not sufficient to draw any firm conclusions, the test does indicate that the estimation of the runoff volume is likely to be one of the largest sources of inaccuracy in estimating the design flood.

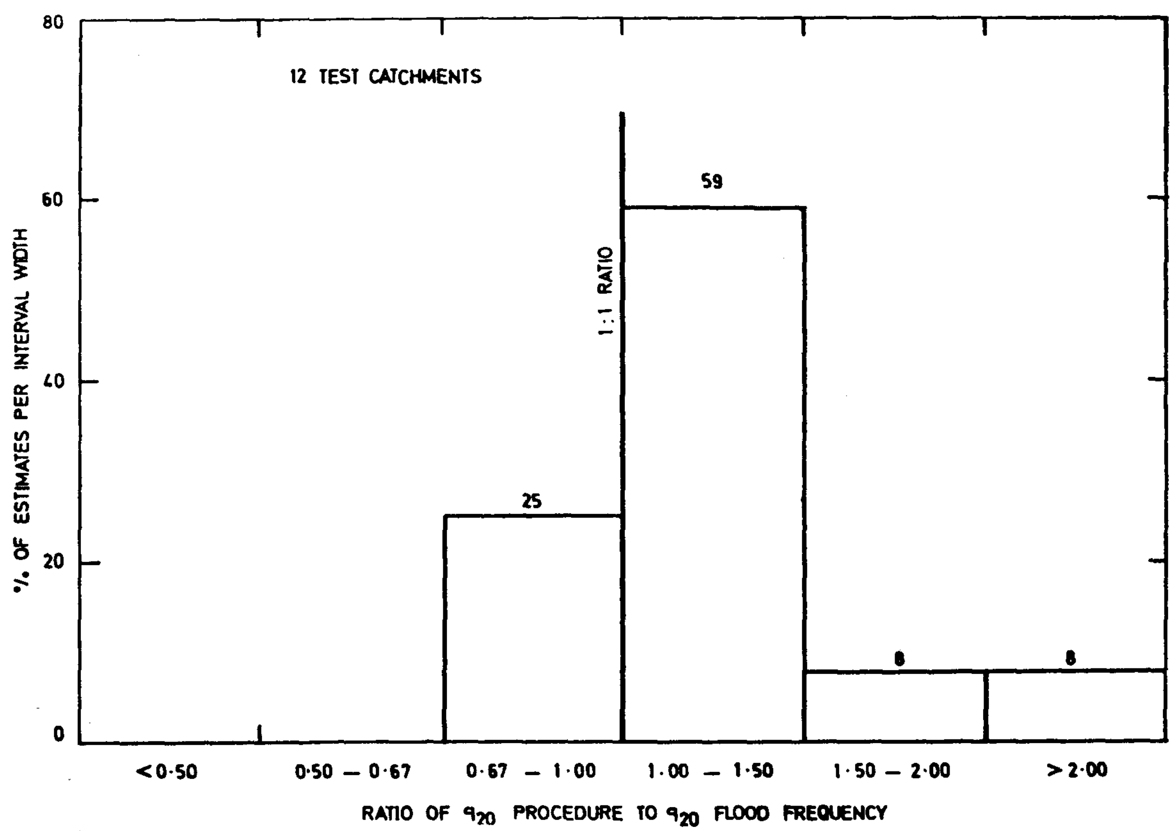

FIG. 11 - FREQUENCY DIAGRAM INDICATING RELIABILITY OF PROCEDURE.

5.3 Test Two — Comparison with Flood Frequency

Frequency analysis of historical flood records is generally recognised as one of the most reliable methods of determining the magnitude of statistical floods. A frequency analysis of annual maximum discharges was done for 12 test catchments. None of the test catchments were used in the development of the flood estimation procedure because, apart from station 3118447, they were not equipped with rainfall and streamflow recorders.

For each catchment, the annual maximum discharge series was fitted to a Type I extreme value distribution by least squares. The 20-year peak discharge determined from the frequency distribution was then compared to the 20-year flood calculated using the flood estimation procedure. The results are listed in Appendix D and shown graphically in Fig. 11.

Fig. 11 indicates that the procedure tends to somewhat over-estimate the 20-year peak discharge as determined from the frequency analysis. A possible mitigating factor is that the peak discharge data used in the frequency analysis are based on 12 -hourly staff gauge readings for most of the test catchments.Since the actual flood peak may occur between 12-hourly readings, it is possible that the 20-year floods estimated from the frequency analysis are somewhat low. However there were no major discrepancies apparent in the flood frequency curves for the 12 test catchments and it is considered that they provide a reasonably reliable basis for the test. Fig. 11 shows that 84 percent of the flood estimates made using the procedure are within the range 2/3 to $1\frac{1}{2}$ of the flood frequency estimates, which is good enough to justify the use of the procedure for design flood estimation.

6. LIMITATIONS OF PROCEDURE

This procedure has been prepared primarily to assist engineers in the selection of sensible and realistic design floods in cases where hydrological data for the catchment is sparse or nonexistent. To this end it is important that the limitations of the flood estimation methodology are clearly understood. The main limiting assumption inherent in the method is that the flood of T-year recurrence interval is actually caused by the storm of the same recurrence interval. There are three main reasons why the validity of this assumption is questionable.

Firstly, the prior moisture status of the catchment affects both the volume and time distribution of runoff. Generally speaking the proportion of direct runoff relative to rainfall is greater when the catchment antecedent moisture status is high. In the derivation of the rainfall-runoff relationship for this procedure a single curve representing average conditions has been drawn through the observed data. The variation in the time distribution of runoff is evident in the catchment lag variation observed on the study catchments. The physical causes of this non-linearity have been discussed in section 3.5. In this procedure the median lag values were taken as being representative of catchment lag time. Although there is little theoretical justification for using median values of catchment lag and average proportions of runoff to rainfall, it is a practical alternative to introducing the statistical variation of catchment moisture status as an additional variable. This latter approach has been explored briefly by Nash (1958).

The second reason is due to the areal variability of catchment rainfall during a storm. This has several effects. It contributes to the variation in lag times observed for different storms on a catchment, and also makes the assumption of uniform areal distribution of the design storm invalid. These effects become more restrictive as the catchment size increases.

The third reason why the T-year flood may not be caused by the T-year storm is because of the possibility that the peak discharge may result from a complex storm due to successive bursts of rain in the catchment with the progression of runoff. Chow (1964 pg. 25–31) states that this effect is usually typical of thunderstorm situations over large catchments. On the West Coast of Malaysia, where large areal variations in thunderstorm activity can occur over very short distances, this possibility is very real, even on small catchments. On the East Coast most of the major flooding occurs during the north-east monsoon when the rainfall tends to be more widespread. Major flood results from complex storms were observed on a number of the catchments used in this study. A good example of this type of event is the flood recorded at station number 3118447 on the 4th January 1971. This flood, which was analysed in section 5.2, was caused by successive bursts of low intensity rainfall occuring over several days. The recurrence interval of this flood as estimated from the frequency analysis is about 20 years. By comparison the 20-year design storm calculated using the procedure is a short duration, high intensity storm. This clearly emphasizes the difficulties associated with estimating statistical floods from theoretical design storms.

For the reasons mentioned above, it is recommended that the flood estimation procedure summarized in section 4 should:

a. Not be used on catchments larger than 200 square miles.

b. Not be used as a design basis when serious consequences such as major damage and loss of life would result from the design flood being exceeded. In this case the probable maximum storm should be used in preference to the design storm, with some consideration being given to the possible combination of areal and temporal pattern giving the highest peak discharge. The temporal and areal pattern of the probable maximum storm should be determined from an analysis of extreme flood producing storms in the area. The probable maximum precipitation may be estimated from meteorological data using the standard techniques given by Weisner (1970).

To handle a varying storm temporal pattern the unit hydrograph must be used. In cases where no streamflow data are available the triangular unit hydrograph may be estimated as shown in the following section.

7. THE TRIANGULAR UNIT HYDROGRAPH

In some cases the unit hydrograph may be required to estimate the design flood from a storm of varying temporal pattern. The results presented in this study may be used to derive triangular unit hydrographs for ungauged rural catchments. The procedure is as follows:

1. Select the appropriate hydrological group and compute L_g from equation (3.12) as before.

2. From equation (3.5) put $Q = 1$ and $D = U$,

$$\text{then } q_u = \frac{D_p \times A \times 640}{(L_g + U/2)}$$

where q_u = peak discharge of the triangular unit hydrograph
U = period of the triangular unit hydrograph.

3. Calculate T_b and T_p from the expressions in Table 2 as before. In this case T_b and T_p refer to the base time and time to peak of the triangular unit hydrograph respectively.

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LIST OF SYMBOLS

A	Catchment area (sq. mls.)
C	Lag time plus half the duration of rainfall excess (hrs)
C_t	Constant in equation (3.12)
D	Duration of rainfall excess (hrs)
D_p	Peak ordinate of dimensionless hydrograph
I	Potential infiltration index
L	Main stream length from outlet to catchment boundary (mls)
L_c	Length of main stream from outlet to catchment centroid (mls.)
L_g	Catchment lag (hrs.)
l_i	Incremental stream length between successive contours (mls.)
n	Constant in equation (3.12)
P	Storm rainfall (ins.)
P_e	Total rainfall minus initial loss (ins.)
Q	Volume of direct runoff or rainfall excess (ins.)
q_p	Peak discharge of triangular hydrograph (cusecs)
q	Discharge ordinate of direct runoff hydrograph (cusecs)
q_u	Peak discharge of triangular unit hydrograph (cusecs)
S	Weighted mean slope of main stream (ft. per mile)
S_i	Incremental stream slope between successive contours (ft. per mile).
T	Recurrence interval (yrs.)
T_b	Base time of triangular hydrograph (hrs.)
T_p	Time to peak of triangular hydrograph (hrs.)
T_r	Recession time of triangular hydrograph (hrs.)
t	Time
U	Period of the triangular unit hydrograph (hrs.)

WORKED EXAMPLES

Example 1

Calculate the design flood for a catchment having the following characteristics:

$$\begin{aligned} \text{Area (A)} &= 20 \text{ sq. miles.} \\ \text{Stream length (L)} &= 10 \text{ miles.} \\ \text{Length to centroid } (L_c) &= 5 \text{ miles} \\ \text{Stream slope (S)} &= 400 \text{ ft./mile} \end{aligned}$$

Catchment Location 4° N, 100° E

In this case only the peak discharge is required for a 20-year recurrence interval. The catchment is very steep and covered in jungle i.e. a Group 1 catchment.

STEP 1

Calculate catchment lag from equation (3.12)

$$L_g = 2.0 \times \left(\frac{10 \times 5}{\sqrt{400}}\right)^{0.35}$$
$$= 2.8 \text{ hrs.}$$

STEP 2

From D.I.D. Hydrological Procedure No. 1, calculate the design storm for durations of 2, 3 and 6 hours.

2-hr storm	$P = 3.8$ ins.
3-hr storm	$P = 4.2$ ins.
6-hr storm	$P = 4.7$ ins.

STEP 3

Calculate the direct runoff volume from equation (3.2)

$$\text{2-hr storm } Q = \frac{3.8^2}{(3.8+6)} = 1.5 \text{ ins.}$$

$$\text{3-hr storm } Q = \frac{4.2^2}{(4.2+6)} = 1.7 \text{ ins.}$$

$$\text{6-hr storm } Q = \frac{4.7^2}{(4.7+6)} = 2.1 \text{ ins.}$$

STEP 4

Calculate the 20-year peak discharge from equation (3.5). For a Group 1 catchment $D_p = 1.06$.

$$\text{2-hr storm } q_p = \frac{1.06 \times 20 \times 640 \times 1.5}{(2.8+1)} = 5356 \text{ cusecs}$$

$$\text{3-hr storm } q_p = \frac{1.06 \times 20 \times 640 \times 1.7}{(2.8+1.5)} = 5364 \text{ cusecs}$$

$$\text{6-hr storm } q_p = \frac{1.06 \times 20 \times 640 \times 2.1}{(2.8+3)} = 4913 \text{ cusecs}$$

STEP 5

Calculate design baseflow $= 5 \times 20$

$= 100$ cusecs.

$\therefore$ design flood peak $= 5364 + 100$

$= 5464$

say 5500 cusecs.

Example 2

It is proposed to construct a small dam across a river draining a 7 square mile catchment. Assuming a 50-year recurrence interval, calculate the design flood for the dam spillway given that:

Catchment is typical of Group 2
Catchment location 2°40'N, 103°40'E
Stream slope (S) = 45.5 ft/mile
Stream length (L) = 4.4 miles
Length to Centroid (L_c) = 2.0 miles
Dam storage area = 200 acres
Width of spillway (W) = 60 ft.

The spillway is an uncontrolled overflow type with discharge characteristics of:

$$\text{discharge} = 3.97 \times W \times H^{1.5}$$

Where H is the spillway head (ft). It can be assumed that the dam storage area does not change significantly with H.

The approach here is to compute the inflow hydrographs for a range of storm durations and calculate the outflow hydrographs at the spillway by storage routing. The outflow hydrograph with the highest peak discharge is the design flood.

STEP 1
Calculate catchment lag from equation (3.12)

$$L_g = 4 \times \left(\frac{4.4 \times 2}{\sqrt{45.5}}\right)^{0.35}$$
$$= 4.4 \text{ hrs.}$$

STEP 2
From D.I.D. Hydrological Procedure No. 1, calculate the design storm for durations of 3, 6, 12, 24 and 48 hours.

3-hr storm	P = 8.1 ins.
6-hr storm	P = 10.1 ins.
12-hr storm	P = 12.5 ins.
24-hr storm	P = 17.8 ins.
48-hr storm	P = 23.4 ins.

STEP 3
Calculate the direct runoff volume from equation (3.2)

$$\text{3-hr storm } Q = \frac{8.1^2}{(8.1+6)} = 4.7 \text{ ins.}$$
$$\text{6-hr storm } Q = \frac{10.1^2}{(10.1+6)} = 6.3 \text{ ins.}$$
$$\text{12-hr storm } Q = \frac{12.5^2}{(12.5+6)} = 8.4 \text{ ins.}$$
$$\text{24-hr storm } Q = \frac{17.8^2}{(17.8+6)} = 13.3 \text{ ins.}$$
$$\text{48-hr storm } Q = \frac{23.4^2}{(23.4+6)} = 18.6 \text{ ins.}$$

STEP 4
Calculate the triangular hydrograph for each storm using equation (3.5) to estimate q_p, and the values of T_p and T_b as given in Table 2.

$$\text{3-hr storm } q_p = \frac{0.89 \times 7 \times 640 \times 4.7}{(4.4+1.5)} = 3176 \text{ cusecs}$$
$$T_p = 0.87 \times (4.4+1.5) = 5.1 \text{ hrs}$$
$$T_b = 2.24 \times (4.4+1.5) = 13.2 \text{ hrs}$$

$$6\text{-hr storm } q_p = \frac{0.89\times7\times640\times6.3}{(4.4+3.0)} = 3394 \text{ cusecs}$$
$$T_p = 0.87\times(4.4+3) = 6.4 \text{ hrs}$$
$$T_b = 2.24\times(4.4+3) = 16.6 \text{ hrs}$$
$$12\text{-hr storm } q_p = \frac{0.89\times7\times640\times8.4}{(4.4+6.0)} = 3220 \text{ cusecs}$$
$$T_p = 0.87\times(4.4+6.0) = 9.0 \text{ hrs}$$
$$T_b = 2.24\times(4.4+6.0) = 23.3 \text{ hrs}$$
$$24\text{-hr storm } q_p = \frac{0.89\times7\times640\times13.3}{(4.4+12)} = 3234 \text{ cusecs}$$
$$T_p = 0.87\times(4.4+12) = 14.3 \text{ hrs}$$
$$T_b = 2.24\times(4.4+12) = 36.7 \text{ hrs}$$
$$48\text{-hr storm } q_p = \frac{0.98\times7\times640\times18.6}{(4.4+24)} = 2611 \text{ cusecs}$$
$$T_p = 0.87\times(4.4+24) = 24.7 \text{ hrs}$$
$$T_b = 2.24\times(4.4+24) = 63.6 \text{ hrs.}$$

Add a baseflow component of $7\times5 = 35$ cusecs to each hydrograph. The inflow hydrographs are shown in Fig. 12.

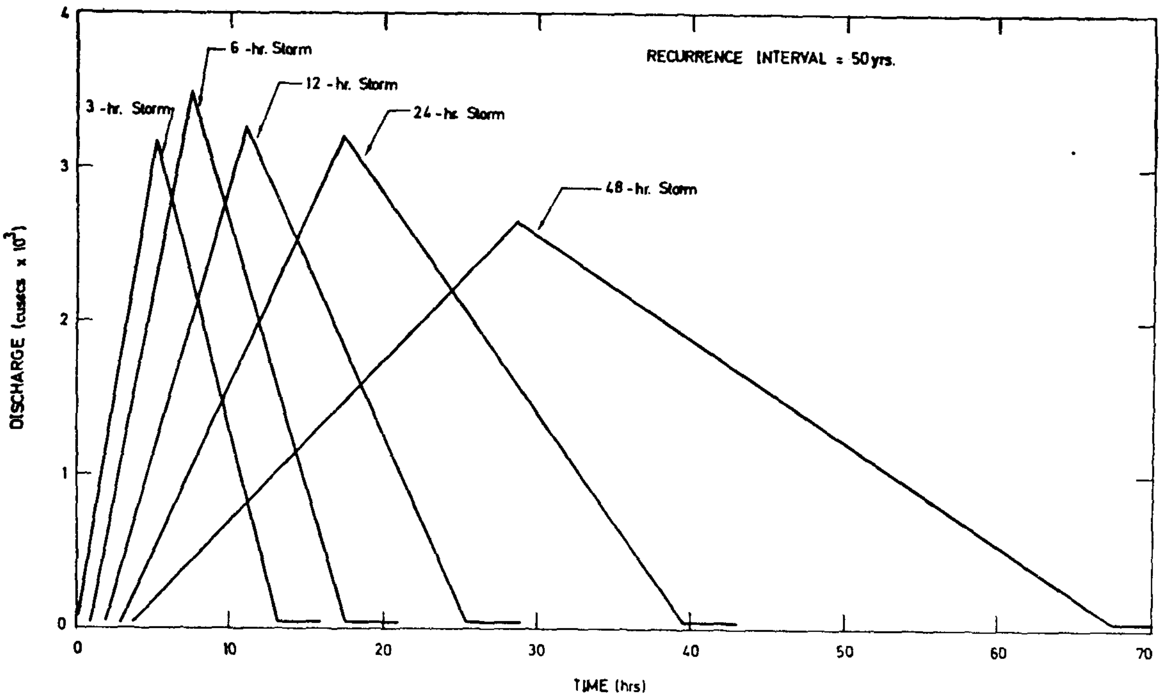

FIG. 12 - INFLOW HYDROGRAPHS FOR EXAMPLE 2

STEP 5

Route the inflow hydrographs for each storm duration through the dam storage. Any standard routing technique may be used. The method used in this example is given by Henderson (1966). For brevity the calculations are not given here, but the routed outflow hydrographs are shown in Fig. 13.

Comments

a. The design flood for the particular spillway is 2900 cusecs.

b. The outflow hydrograph having the highest peak discharge is caused by a 24-hr storm, whereas the inflow hydrograph having the highest peak discharge is caused by a 6-hr storm. This illustrates the point that for structures having a relatively large storage capacity, the short intense storms are usually less critical than longer duration storms producing large runoff volumes.

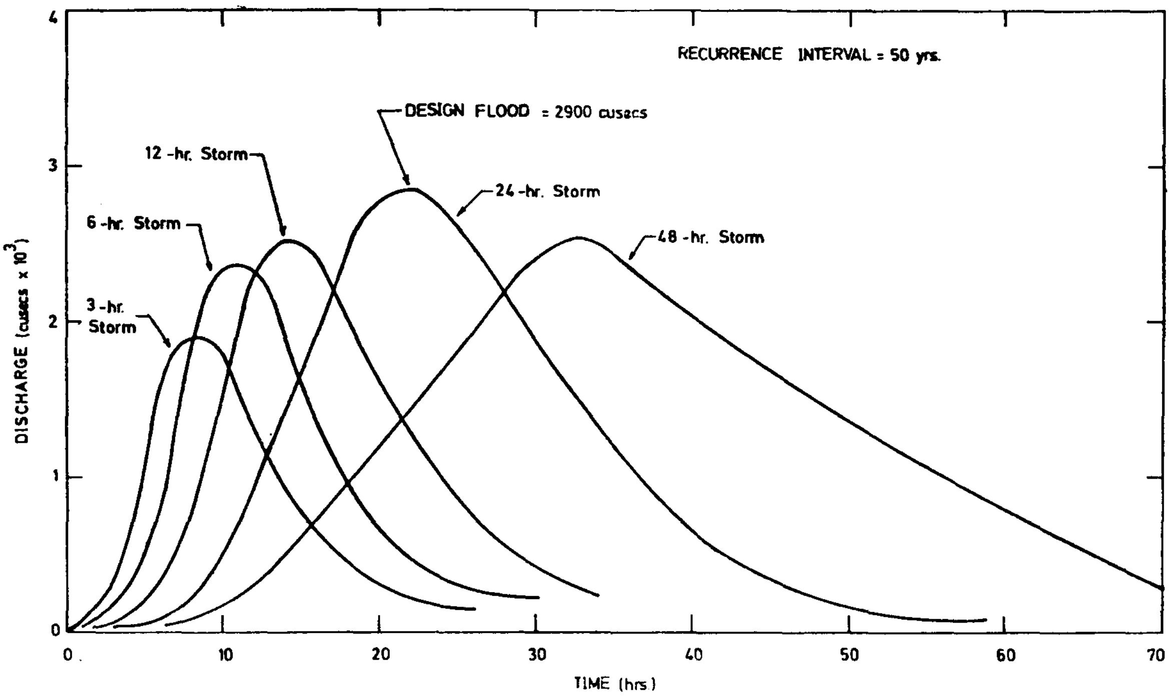

FIG. 13 - OUTFLOW HYDROGRAPHS FOR EXAMPLE 2

Example 3

The probable maximum flood is required for the design of a river bund protecting a small town. The probable maximum flood is to be estimated using the unit hydrograph and a probable maximum storm. Calculate the 1-hr triangular unit hydrograph for the catchment given that:

Catchment area (A)	=	180 sq.miles.
Stream Length (L)	=	22 miles.
Length to Centroid (L_c)	=	12 miles.
Stream slope (S)	=	150 ft./miles.

Assume the catchment is typical of Group 2.

STEP 1

Calculate catchment lag from equation (3.12)

$$L_g = 4\times\left(\frac{22\times 12}{\sqrt{150}}\right)^{0.35}$$
$$= 11.7 \text{ hrs.}$$

STEP 2

Calculate the peak discharge of the triangular unit hydrograph using equation (3.5)

$$q_p = \frac{0.89\times 180\times 640\times 1}{(11.7+0.5)}$$
$$= 8404 \text{ cusecs}$$

Calculate T_p and T_b from the values given in Table 2

$$T_p = 0.87\times(11.7+0.5) = 10.6 \text{ hrs}$$
$$T_b = 2.24\times(11.7+0.5) = 27.3 \text{ hrs}$$

APPENDIX B

DETAILS OF STUDY CATCHMENTS

	Station Number		Station Name	Operating Authority	A (Sq.mls.)	L (mls.)	L_c (mls.)	S (ft/ml.)	Median Value of		No. of Storms Analysed
	OLD	NEW							L_g (hrs.)	D_p	
GROUP 1 CATCHMENTS	3454	4112454	Sg. Bidor @ Bidor	D.I.D.	31.8	10.0	6.6	233.5	4.50	0.81	1
	3456	4112456	Sg. Batang Padang @ Tapah	"	145.0	25.7	13.2	153.0	4.92	1.06	1
	3459	4112459	Sg. Gedong @ Bidor	"	40.0	15.0	10.3	227.4	6.17	0.90	7
	3463	4611463	Sg. Kinta @ Tanjong Rambutan	"	92.5	20.0	11.8	206.8	4.92	1.35	2
	4423	3517423	Sg. Selangor @ km7 Frasers Hill Road	J.K.R.	76.0	13.1	4.3	241.21	3.08	1.06	5
	4426	3616426	Sg. Kerling @ 6000 Ulu Kuala	L.L.N.	22.8	11.2	6.4	324.0	4.50	1.36	1
	4436	3317436	Sg. Gombak @ km21 Gombak	J.K.R.	15.8	7.4	2.6	420.6	3.09	1.17	2
	8418	4413418	Sg. Bertam @ Robinson Falls Intake	L.L.N.	8.2	6.5	2.5	212.9	3.83	0.84	10
	8427	3518427	Sg. Perting @ Bentong	"	40.4	13.7	6.4	330.9	4.33	1.08	4
	8429	3518429	Sg. Benus @ Janda Baik	"	27.5	8.5	2.8	248.7	3.09	0.90	4
	8467	3817467	Sg. Liang @ Pimco Estate Road	"	75.9	15.5	8.8	270.6	6.42	1.27	3
GROUP 2 CATCHMENTS	2421	5405421	Sg. Kulim @ Ara Kuda	D.I.D.	50.7	17.3	8.8	44.5	13.17	0.90	7
	2422	5204422	Sg. Jawi @ Jem. Jalan Raya	"	16.0	7.9	4.3	50.3	8.08	0.89	4
	3458	3913458	Sg. Sungkai @ Sungkai	"	110.7	25.5	13.8	124.6	12.92	0.95	5
	3464	4311464	Sg. Kampar @ Kg. Lanjut	"	169.0	31.0	17.9	64.5	17.09	0.83	2
	3465	4410465	Sg. Raia @ Ladang Kinta Kellas	"	97.0	28.5	16.2	153.5	10.25	0.88	3
	4412	3615412	Sg. Bernam @ Tanjong Malim	"	72.0	12.8	6.5	259.9	7.83	0.82	7
	4422	3516422	Sg. Selangor @ Rasa	"	124.0	23.5	11.4	129.5	10.00	0.77	9
	4432	3116432	Sg. Klang @ Leboh Pasar, Kuala Lumpur	"	179.0	17.1	9.6	102.8	12.25	0.90	5
	4433	3116433	Sg. Gombak @ Jln. Pekeliling	"	47.0	18.2	7.5	105.7	11.08	0.91	8
	4434	3116434	Sg. Batu @ Sentul	"	56.0	18.0	6.9	128.1	11.33	0.74	11
	4443	2918443	Sg. Semenyih @ Semenyih	"	82.0	18.5	8.1	72.1	9.59	0.87	10
	4445	3118445	Sg. Lui @ Kg. Lui	"	26.3	9.5	4.8	152.0	6.83	0.82	6
	5424	2619424	Sg. Mantau @ Mantau	"	4.9	4.3	2.3	80.0	4.42	0.73	3
	8426	3519426	Sg. Bentong @ Jam. Kuala Marong	"	93.0	14.5	5.0	134.3	7.21	1.05	8
	8417	4514417	Sg. Telom @ U/S Telom Intake	L.L.N.	33.7	8.4	2.8	129.3	7.00	1.14	9

	Station Number		Station Name	Operating Authority	A (Sq.mls.)	L (mls.)	L_c (mls.)	S (ft/ml.)	Median Value of		No. of Storms Analysed
	OLD	NEW							L_g (hrs.)	D_p	
GROUP 3 CATCHMENTS	3466	4610466	Sg. Pari @ Jalan Silibin, Ipoh	D.I.D.	94.5	20.5	9.5	85.8	27.58	0.96	1
	4437	3115437	Sg. Damansara @ Subang	J.K.R.	37.8	10.0	2.0	11.5	12.46	0.65	4
	6415	2322415	Sg. Durian Tunggal @ Mile 11 Air Resam	D.I.D.	28.0	9.5	3.8	47.6	14.58	0.75	5
	6432	2224432	Sg. Kesang @ Chin Chin	"	62.0	19.5	7.6	19.8	29.63	0.76	4
	7423	1931423	Sg. Sembrong @ Jambatan Brizay	"	72.0	17.0	6.6	8.4	56.50	0.81	2
	7452	1836452	Sg. Sayong @ Hilir Sayong	"	169.0	25.5	10.7	7.2	36.75	0.74	2
	7453	1834453	Sg. Sayong @ Layang Layang	"	38.0	12.4	6.3	13.3	30.17	0.73	1
	7456	1739456	Sg. Sebol @ Jambatan	"	8.6	6.7	3.6	11.4	22.42	0.71	1
	7457	1739457	Sg. Permandi @ Batu 27, Johor Bahru/Mersing	"	9.1	4.7	1.3	32.2	13.67	0.72	4
	7462	1839462	Sg. Mupor @ Batu 32, Johor Bahru/Mersing	"	8.4	6.5	1.8	33.5	18.42	0.67	3
	7471	2237471	Sg. Lenggor @ Batu 42, Kluang Mersing	"	80.0	17.2	6.2	12.8	32.67	0.81	7
	0421	6022421	Sg. Kemasin @ Peringat	"	18.5	10.9	5.0	3.4	34.33	0.80	4

DATA USED TO DERIVE RAINFALL-RUNOFF RELATIONSHIP

Station-Number OLD	Station-Number NEW	A (Sq. mls.)	Date of Peak Discharge	P (ins.)	Q (ins.)	D (hrs.)	ϕ Index (ins./hr.)	Areal Reduction Factor	No. of Rain Gauges
2421	5405421	50.7	27- 5-73	0.94	0.23	1.0	0.71	0.46	6
			20- 4-73	1.76	0.26	2.0	0.75	0.52	6
			10-11-72	1.49	0.22	2.0	0.64	0.67	6
			30-10-72	0.94	0.32	4.0	0.16	0.47	6
			23-10-72	0.86	0.40	1.0	0.46	0.41	6
			13- 9-72	2.88	0.48	2.0	1.20	0.79	6
			18- 9-71*	3.16	0.64	8.0	0.54	0.71	6
2422	5204422	16.0	8-11-70	1.70	1.04	3.0	0.22	0.97	2
			21-10-71	2.73	1.35	3.0	0.19	0.97	2
			22-10-73	1.63	0.63	1.0	1.00	0.94	2
			27- 5-73*	4.21	1.45	5.0	0.60	0.64	2
3456	4112456	145.0	19-11-72	0.88	0.39	2.0	0.13	0.57	4
3464	4311464	169.0	23-10-69	1.49	0.74	2.0	0.32	0.37	5
			28-12-70	1.21	0.54	3.0	0.22	0.45	5
3465	4410465	97.0	23-10-69	0.79	0.15	2.0	0.32	0.53	5
			28-10-69	1.01	0.17	3.0	0.42	0.77	5
			19- 5-68	1.35	0.17	1.0	1.18	0.57	5
3466	4610466	94.5	29-11-67*	1.86	0.91	5.0	0.20	0.49	5
4422	3516422	124.0	25- 2-69	1.12	0.48	3.0	0.32	0.94	2
			19- 7-68	1.19	0.36	3.0	0.28	0.53	2
			11- 5-68	0.68	0.21	1.0	0.47	0.80	2
			6- 4-66	1.30	0.22	1.0	1.08	0.92	2
			10-12-65	0.78	0.46	2.0	0.16	0.44	2
			23-11-72	1.50	0.69	2.0	0.40	0.84	2
			13-10-72	3.17	0.65	1.0	2.52	0.83	2
			6-11-71	1.86	0.56	2.0	0.65	0.93	2
			5- 1-71*	4.11	1.84	16.0	0.15	0.98	2
4423	3517423	76.0	21- 5-65	1.57	0.48	1.0	1.09	0.83	2
			15- 6-67	1.23	0.33	1.0	0.90	–	1
			29- 5-68	0.60	0.17	1.0	0.43	0.55	2
			7-10-69	0.60	0.22	1.0	0.38	0.93	2
			26-11-70	0.54	0.33	2.0	0.10	0.52	2
4432	3316432	179.0	30-10-65	1.00	0.34	1.0	0.66	0.31	11
			12- 7-68*	2.07	0.77	19.0	0.24	0.49	10
			5- 1-71	6.16	4.26	19.0	0.11	0.91	4
			17-11-72	2.56	1.11	3.0	0.48	0.62	8
4433	3116433	47.0	14- 9-64	2.76	0.60	3.0	0.72	0.84	3
			18- 6-67	0.87	0.48	1.0	0.39	0.69	3
			17- 9-68	1.70	0.32	2.0	0.69	0.88	3
			20- 1-64*	1.03	0.61	10.0	0.05	0.49	3
			31- 3-69	1.05	0.34	1.0	0.71	0.85	3
			29-12-68	1.15	0.51	3.0	0.13	0.45	3
			7-12-73	0.99	0.54	2.0	0.23	0.63	3
4434	3116434	56.0	17- 6-67	1.41	0.65	1.0	0.76	0.80	3
			17- 9-68	1.48	0.43	1.0	1.05	0.91	3
			29- 3-68	1.44	0.43	1.0	1.01	0.82	3
			18-11-68*	1.00	0.42	3.0	0.24	0.67	3

* Complex Storm.

Station Number OLD	Station Number NEW	A (Sq. mls.)	Date of Peak Discharge	P (ins.)	Q (ins.)	D (hrs.)	ϕ Index (ins./hr.)	Areal Reduction Factor	No. of Rain Gauges
4434	3116434	56.0	28- 6-67	1.30	0.50	1.0	0.80	0.57	3
			13- 1-67*	1.52	0.56	3.0	0.38	0.54	3
			6- 5-74	1.63	0.60	1.0	1.03	0.86	3
			5- 4-73	1.86	0.65	3.0	0.40	0.70	3
			9- 9-71	1.64	0.48	1.0	0.48	0.40	3
			5- 1-71*	3.68	1.55	14.0	0.16	0.61	3
			27-12-69	1.38	0.49	1.0	0.89	0.60	3
4436	3317436	15.8	22- 5-69	1.11	0.24	1.0	0.87	–	1
			30- 7-70	1.38	0.14	1.0	1.24	–	1
4443	2918443	82.0	16- 6-70	1.21	0.40	2.0	0.41	0.64	4
			25- 5-70	1.57	0.32	1.0	1.25	0.79	4
			6- 1-70*	0.71	0.30	8.0	0.12	0.61	4
			13-12-69	1.74	0.22	1.0	1.52	0.68	4
			13-10-69	0.65	0.24	1.0	0.41	0.67	4
			25- 8-69	1.16	0.37	2.0	0.40	0.80	4
			28-12-68*	1.57	0.38	6.0	0.25	0.80	4
			6- 5-68	0.43	0.21	1.0	0.22	0.74	4
			18- 3-71	1.52	0.38	2.0	0.19	0.94	4
			1- 6-73	1.90	0.69	2.0	0.60	0.65	4
4445	3118445	26.3	12- 2-65	1.17	0.25	1.0	0.92	0.91	4
			10- 7-65	0.68	0.13	1.0	0.55	0.61	4
			5- 9-65	3.74	0.74	2.0	1.36	0.78	4
			9- 9-65	1.86	0.44	1.0	1.42	0.87	4
			17-11-68	1.89	0.77	1.0	1.12	0.66	4
			1- 4-70	1.79	0.69	2.0	0.55	0.63	4
5424	2619424	4.9	7-11-69	1.55	0.41	2.0	0.57	0.72	4
			27- 4-69	1.08	0.39	1.0	0.69	0.51	3
			7- 1-69	0.85	0.27	2.0	0.59	0.70	4
			23-10-68	1.13	0.50	1.0	0.63	0.72	4
7423	1931423	72.0	29- 2-64*	5.98	3.05	14.0	0.43	0.79	3
7452	1836452	169.0	13-10-73	0.63	0.14	1.0	0.49	0.66	5
			13- 3-72	0.53	0.10	1.0	0.43	0.40	5
8417	4514417	33.7	25-10-72*	1.73	0.27	6.0	0.43	0.68	4
			6- 1-70*	4.10	1.47	19.0	0.29	0.78	4
			27-12-70*	3.17	1.25	13.0	0.29	0.64	4
			20- 5-69	1.76	0.73	3.0	0.34	0.88	4
			14- 6-68	0.57	0.34	3.0	0.08	0.62	4
			26-10-67*	1.77	0.58	8.0	0.22	0.89	4
			9- 9-65	0.67	0.49	3.0	0.06	0.81	4
			24- 9-65	1.18	0.47	3.0	0.24	0.64	4
8418	4413818	8.2	8- 9-65	0.98	0.53	2.0	0.23	0.66	3
			10- 9-67	1.22	0.44	2.0	0.39	0.95	3
			13- 5-68	1.67	0.52	1.0	1.15	0.78	3
			10- 6-68	1.63	0.36	1.0	1.27	0.83	3
			30- 4-69	1.62	0.49	1.0	1.13	0.97	2
			2- 1-69	1.49	0.47	1.0	1.02	0.92	2
			19-12-70	1.00	0.40	2.0	0.30	0.72	2
			24- 2-72	0.89	0.40	1.0	0.49	0.87	3
			1-11-73	1.51	0.42	3.0	0.36	0.81	3
			3-10-66*	2.26	0.54	10.0	0.44	0.70	3

* Complex Storm.

Appendix D

DETAILS OF TEST CATCHMENTS

Station Number		Station Name	Operating Authority	A ((Sq.mls.)	L (mls)	L_c (mls.)	S (ft/ml.)	Catchment Group	20 Yr. Flood Estimate		Max. Flood on record (cusecs)	No. Years Record
OLD	NEW								Procedure (cusecs)	Flood Frequency (cusecs)		
2416	5506416	Sg. Sedim @ Merbau Pulas	D.I.D.	170.0	30.0	10.7	72.4	3	7350	6800	9090	23
3414	3813414	Sg. Trolak @ Trolak	D.I.D.	23.0	11.0	5.2	140.4	2	3650	4100	4050	23
3415	3814415	Sg. Bil @ Tg. Malim-Slim Road	"	16.0	9.0	4.8	563.6	1	4700	3500	3350	23
3423	5007423	Sg. Ara @ Mile 20 Taiping-Ijok Road	"	54.0	13.5	4.1	49.3	2	4900	5900	5400	24
3433	5106433	Sg. Ijok @ Titi Ijok	"	83.0	19.0	13.3	67.9	3	2900	3600	3700	24
3452	4012452	Sg. Bidor @ Mile 18 Anson-Kampar Road	"	133.0	24.1	13.4	107.5	3	6250	4150	4150	18
3462	4501062	Sg. Kinta @ Ipoh	"	121.0	29.4	20.6	118.4	2	7050	5900	6250	16
4442	2917442	Sg. Langat @ Kajang	"	148.0	28.0	17.5	58.1	2	10400	9200	11000	17
4447	3118447	Sg. Langat @ Ulu Langat Mile 20	J.K.R.	29.5	8.4	4.7	235.6	1	8750	3900	3800	8
5413	2722413	Sg. Muar @ Kuala Pilah	D.I.D.	143.0	20.8	11.6	76.7	2	11500	7850	6250	11
5421	2519421	Sg. Linggi @ Sua Bentong	"	202.0	33.0	18.8	51.4	2	10800	7900	11800	24
5422	2719422	Sg. Linggi @ Rahang	"	73.0	14.7	6.9	129.0	2	7850	4300	5000	21

PROCEDURES PREVIOUSLY PUBLISHED