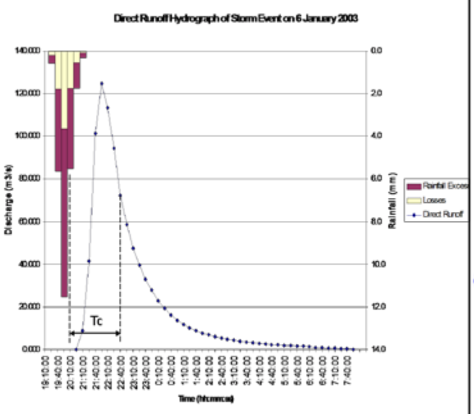
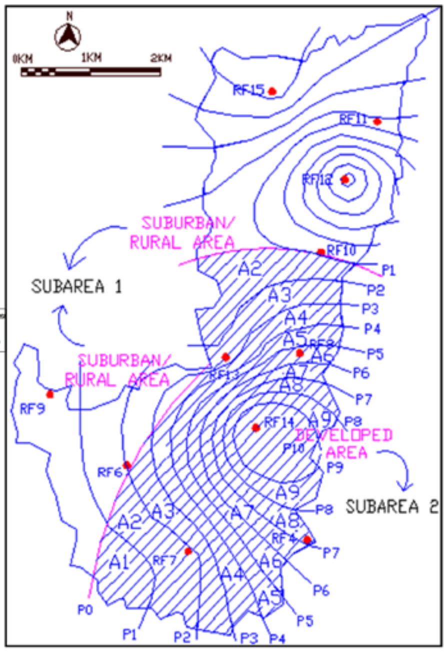
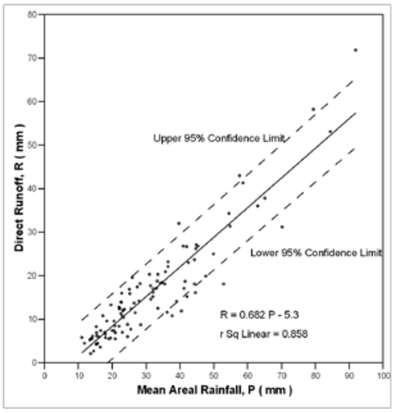
GOVERNMENT OF MALAYSIA
DEPARTMENT OF IRRIGATION
AND DRAINAGE

HYDROLOGICAL PROCEDURE NO. 5
RATIONAL METHOD OF FLOOD ESTIMATION FOR RURAL CATCHMENTS IN PENINSULAR MALAYSIA







2010

HYDROLOGICAL PROCEDURE NO. 5

RATIONAL METHOD OF FLOOD ESTIMATION FOR RURAL CATCHMENTS IN PENINSULAR MALAYSIA

DIVISION OF WATER RESOURCES MANAGEMENT AND HYDROLOGY
DEPARTMENT OF IRRIGATION AND DRAINAGE
MINISTRY OF NATURAL RESOURCES AND ENVIRONMENT

2010

DISCLAIMER

Although every effort and care has been taken in selecting the methods and proposing the recommendations that are appropriate to Malaysian conditions, the user is wholly responsible to make use of this hydrological procedure. The use of this procedure requires professional interpretation and judgment to suit the particular circumstances under consideration.

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1 INTRODUCTION

This hydrological procedure presents the results of a study on the applicability of a Rational Method for flood estimation in small rural catchments in Peninsular Malaysia.

The Rational Method can be traced back to the mid-nineteenth century. The use of the Rational Method in the urban environment has worked reasonably well in many countries. For rural catchments, the use of the Rational Method has received much criticism. Overseas researchers who have studied the method as a deterministic model and tested it with observed data have found that the method offers low accuracy when individual storms and resulting peak discharges are considered. However, studies by French et al. (1974), who examined the validity of the method, have shown that, statistically, the method serves the purpose of engineering practice, where peak discharges of a given frequency are linked with the rainfall intensities of the same frequency.

Given that the annual total expenditures on many small hydraulic structures, such as bridges, culverts, diversion works, and so on, involve significant expense, the need for a procedure to guide practitioners in arriving at a more reasonable design of such structures is both urgent and important. For this purpose, the Department of Irrigation and Drainage (DID) Hydrological Procedure No. 5 (HP5) is used as a basis for the design of the above structures by most practitioners.

There are two earlier editions of HP5. The first edition was published in 1974 (HP5:1974) (Heiler, 1974), whereas the second was released in 1989 (HP5:1989) (Azmi and Zahari, 1989). HP5:2010, the third edition, incorporates 18 rural catchments in four hydrological regions. It uses Hydrological Procedure No. 1 (Fadhillah et al., 1982) (HP1:1982) for the estimation of a design rainstorm.

2 THE RATIONAL METHOD AND FREQUENCY ANALYSIS

2.1 General

Hydraulic designs in engineering are composed of two main aspects: flood estimation and channel sizing. The Rational Method is used for flood estimation. This method is still widely used because of its simplicity; however, criticisms have been raised regarding its use, and methods that are more advanced are available.

The Rational Method is typically used for peak runoff computation. According to Ojha et al. (2008), the main considerations of the Rational Method are as follows:

- The peak runoff rate is a function of the average rainfall rate during the time of concentration; and
- Rainfall intensity is constant during rainfall.

Based on the Rational Method, the main concept of HP5 lies in the statistical link between the frequency distribution of the design rainfall and the design flood.

The Rational Method is known for its simplicity in the computation of design discharge. Its inability to simulate individual storms and its unsuitability to observe peak discharge have been criticized. However, in this case, the usefulness of the Rational Method should be viewed from a different perspective: the statistical link between the frequency of peak discharge and the design rainfall of the same frequency.

The statistical concept is the main idea of HP5, and is established in the relationship between the peak discharge and the design rainfall of the same frequency. This differs from other deterministic models because its judgment is based on the ability to simulate flood events.

The Statistical Rational Method for the estimation of peak discharge is written as

$$Q_T = 0.278 \; C_T \; I_T \; A \qquad \text{Equation 1}$$

where Q_T : Design peak discharge in m³/s, with return period of T years selected based on recommended design return period

C_T : Dimensionless runoff coefficient as a function of catchment characteristics, design rainfall, and return period

I_T : Average rainfall intensity of design rainfall in mm/h, with return period of T years and with rainfall duration being equal to the time of concentration

A : Catchment area in km²

Source: HP5:1989 (Azmi and Zahari, 1989)

2.2 Features of the Rational Method

The Rational Method is generally considered to be one of the best available flood estimation procedures for small urban and rural catchment areas. However, there has been much confusion concerning the principles underlying the method.

The Rational Method has been criticized because of its inability to reproduce particular flood events when actual rainfall is used as the input. Such criticism implies an assumption that the method is deterministic and does not represent the physical operation on the rainfall–runoff process. This is not the intended use of the method. The most realistic way to use the Rational Method is to consider it as a statistical link between the frequency distribution of rainfall and runoff. As such, it provides a means of estimating the design flood of a certain return period, with the rainfall duration equal to the time of concentration.

Rewriting Equation 1,

$$C_T = 0.278\frac{Q_T}{I_T A}$$ Equation 2

$$C_T = \frac{q_T}{I_T}$$ Equation 3

where C_T : Dimensionless runoff coefficient, which represents the statistical link between the frequency of peak discharge (q_T m³/s per km²) and the mean intensities of the design rainfall (I_T mm/h) with return period of T years

q_T : 0.278 Q_T/A is the peak discharge (q_T m³/s per km²) and the mean intensities of the design rainfall (I_T mm/h) with return period of T years. Peak runoff rate of return period T years is derived from the frequency analysis of the observed flood.

I_T : Average rainfall intensity of the design rainfall in mm/h, with return period of T years and with rainfall duration equal to T_c, which is derived from frequency analysis of the recorded rainfall

A : Catchment area in km²

Source: HP5:1989 (Azmi and Zahari, 1989)

2.3 Hydrologic Frequency Analysis

Hydrologic frequency analysis involves the analysis of hydrologic data assumed to be independent and identically distributed with the hydrologic system. These data are considered to be stochastic, as well as space- and time-independent.

The aim of hydrologic frequency analysis is to relate the magnitude of extreme events to their frequency of occurrence through probability distribution. In this case, the magnitude of the extreme event is inversely related to its frequency of occurrence (Chow et al., 1988).

Among various types of hydrologic data, the annual maximum discharge data are utilized for frequency analysis in the current study. The outcome of frequency analysis is the return period. In this case, the return period of an event of a given magnitude is defined as the average recurrence interval between events equal to or exceeding a specified magnitude (Chow et al., 1988). This information is then utilized in the design of various hydraulic structures, such as bridges and culverts for road crossings, detention and retention basins, and others.

3 THE INVESTIGATION

3.1 General

This section, rather than describing in detail the development of the procedure, outlines the general methodology employed by the general user. The frequency analyses of the annual maximum flood data from the catchment are not covered; interested readers should thus refer to DID Hydrological Procedure Nos. 1 and 4 for details.

3.2 Methodology of the Investigation

3.2.1 Design Sequence

In using the Rational Method for flood estimation, the usual design sequence is as follows:

a) Estimate the critical duration of the design storm (made equal to T_c);

b) Compute the various values of mean intensity (I_T) for duration equal to T_c;

c) Estimate the values of C_T from storm- and flood-frequency regions; and

d) Compute the peak discharge (Q_T) for various values of T_c using Equation 1.

3.2.2 Estimation of Time of Concentration

An essential part of the Rational Method is the estimation of the time of concentration T_c. Although this is an unrealistic physical concept in a natural catchment, little doubt that a characteristic time, which is critical for a particular catchment, exists.

T_c cannot be defined precisely, and likely varies from season to season and from storm to storm. Various practical methods have been proposed by researchers to estimate T_c. The current study adopts the method used in HP5:1989 (Equation 4) to estimate T_c.

$$T_c = \frac{1.286L}{A^{0.223}S^{0.263}} \qquad \text{Equation 4}$$

where T_c : Time of concentration (h)

L : Length of main river (km)

S : Slope measured along the main river, from the catchment boundary intersection to the design point (%)

Source: HP5:1989 (Azmi and Zahari, 1989)

3.2.3 Estimation of the Average Intensity of the Design Storm

The method of estimating the design rainstorm contained in Hydrological Procedure No. 1 (Fadhillah et al., 1982) (HP1:1982) is used in the current study for computing the characteristics of the design storm for each of the study catchments. The inputs in using HP1:1982 are the duration of the storm, which is equal to T_c and found in Equation 4, and the geographical location of the design point. The design intensity should be adjusted to account for the reduction in storm intensity, with the catchment area based on Table 6 of HP1:1982 (page 12).

3.2.4 Estimation of Runoff Coefficient C

The runoff coefficient C in the Rational Method is affected by various factors and processes, such as infiltration losses, variations in rainfall intensities, catchment storage, antecedent wetness, and physical characteristics of the catchment.

Numerous approaches have been made available to present the values of runoff coefficient C in tabular selection tables, graphical relations, and simple recommended values. Most of these approaches are based on engineering judgments and experiences, rather than derived from observed flood data. In this procedure, the runoff coefficient C was derived for various return periods from frequency analyses of observed flood data and design rainfall intensities in 18 small rural catchments in Peninsular Malaysia.

Based on the relation shown in Equation 3, a consistent increase is evident in the values of runoff coefficient C_T, with an increase in return period. Table 1 shows the C_T values derived from computations using Equation 3 for the 18 rural catchments in Peninsular Malaysia.

Table 1. Dimensionless Runoff Coefficient C_T for 18 Rural Catchments in this Study

Item	Catchment	Dimensionless runoff coefficient, C_T (q_T/i_T)				
		C_2	C_5	C_{10}	C_{20}	C_{50}
1	Sg Arau	0.1777	0.2181	0.2292	0.2356	0.2430
2	Sg Buluh	0.5279	0.6309	0.6554	0.6684	0.7056
3	Sg Tasoh	0.0338	0.0408	0.0430	0.0443	0.0455
4	Sg Pelarit	0.1096	0.1156	0.1164	0.1164	0.1166
5	Sg Kulim	0.1653	0.3122	0.3595	0.4198	0.4624
6	Sg Chenderiang	0.0516	0.0698	0.0781	0.0871	0.0958
7	Sg Bernam	0.1307	0.2034	0.2311	0.2495	0.2763
8	Sg Lui	0.0661	0.1071	0.1263	0.1398	0.1527
9	Sg Kepis	0.4829	0.5981	0.6514	0.6546	0.6827
10	Sg Gemencheh	0.1046	0.1925	0.2467	0.2670	0.2967
11	Sg Pedas	0.0926	0.1106	0.1184	0.1313	0.1327
12	Sg Durian Tunggal	0.0597	0.0826	0.0957	0.1053	0.1132
13	Sg Kesang	0.0493	0.0601	0.0610	0.0612	0.0637
14	Sg Kemasin	0.5138	0.5792	0.6302	0.6646	0.6843
15	Sg Lanas	0.3126	0.4026	0.4432	0.4715	0.5146
16	Sg Chalok	0.3672	0.4708	0.5256	0.5644	0.5922
17	Sg Telemong	0.4770	0.4779	0.4842	0.4845	0.4857
18	Sg Penchala	0.3608	0.5771	0.6802	0.7763	0.8634

3.2.5 Regional Runoff Coefficient C based on Flood Frequency Regions

For the application of this procedure, Peninsular Malaysia was divided into four Flood Frequency Regions (Figure 1) based on DID HP4:1987 (Ong, 1987) and HP5:1989 (Azmi and Zahari, 1989). The frequency distribution of flood peaks and flood-producing rainstorms does not significantly affect the runoff coefficient C within the same region.

The mean values of the runoff coefficient for return periods of 2, 5, 10, 20, and 50 years were computed for each region. These values of C_2, C_5, C_{10}, C_{20}, and C_{50} are presented in Table 2. The relation of mean frequency factor C_Y/C_{10} for different regions is shown in Figure 2.

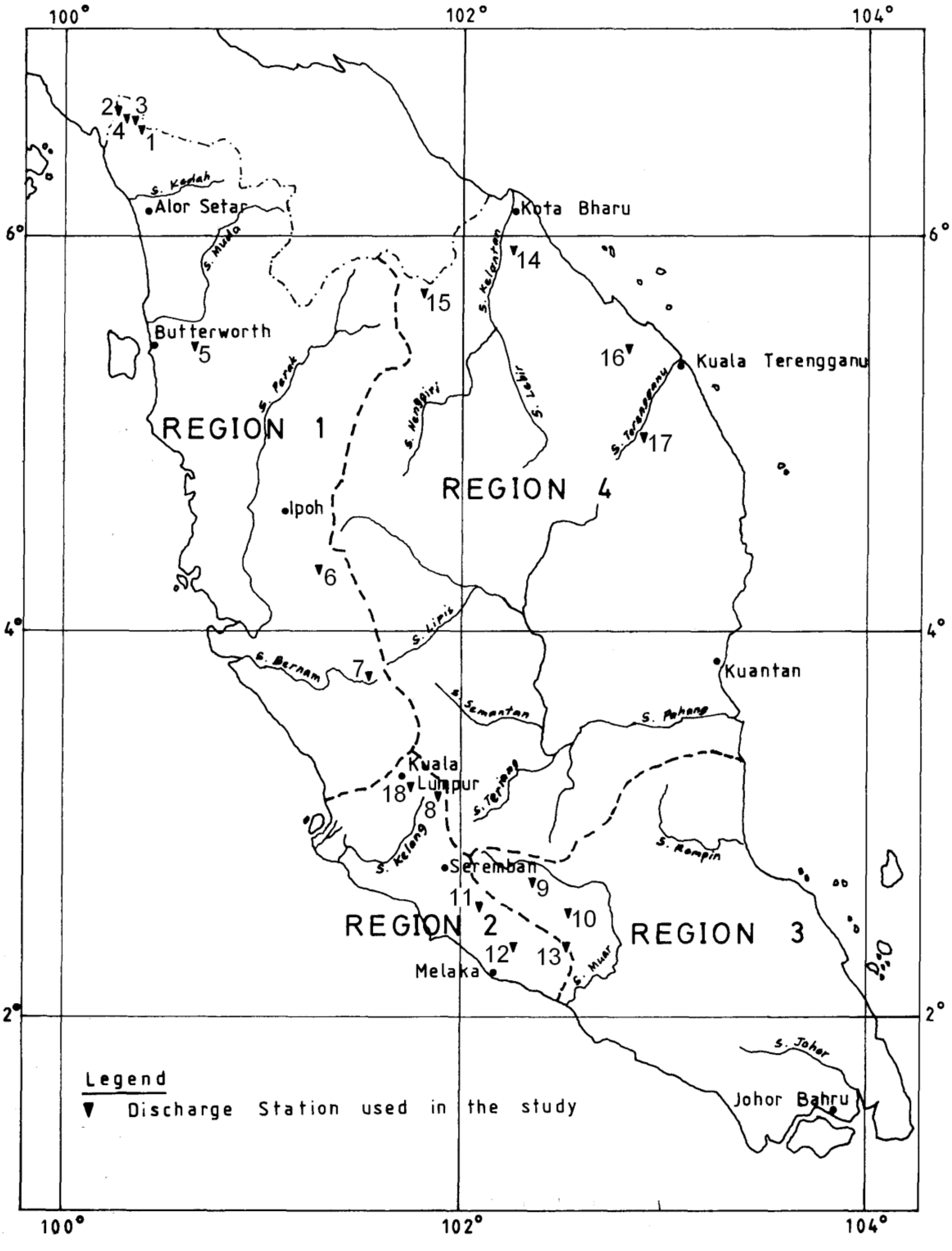

Figure 1. Hydrological Regions in Peninsular Malaysia

Table 2. Regional Runoff Coefficient C_T for Four Hydrological Regions

Hydrological Regions	Dimensionless runoff coefficient, C_T (q_T/i_T)				
	C_2	C_5	C_{10}	C_{20}	C_{50}
Region 1	0.1554	0.2055	0.2224	0.2373	0.2534
Region 2	0.1233	0.1837	0.2118	0.2360	0.2602
Region 3	0.2948	0.4113	0.4395	0.4719	0.5036
Region 4	0.3955	0.4684	0.4928	0.5193	0.5421

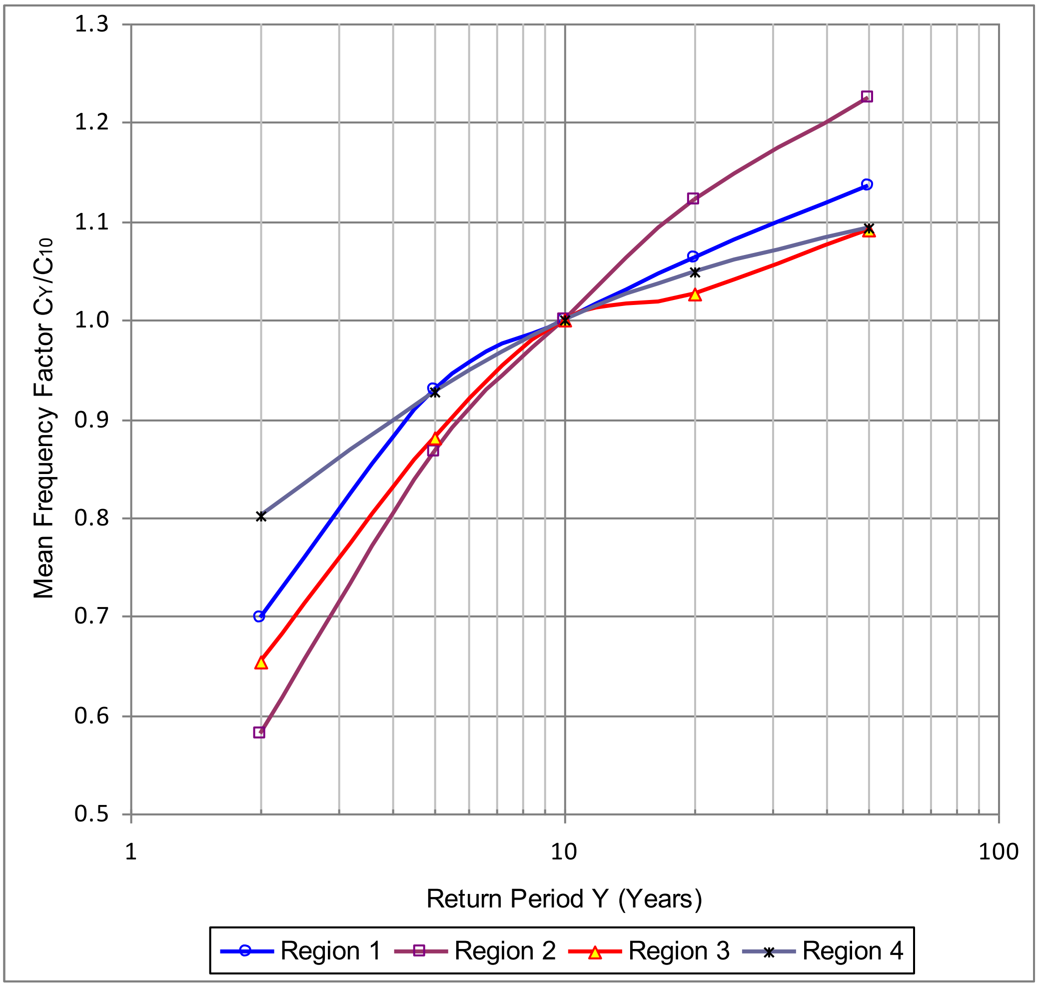

Figure 2. Relation of Mean Frequency Factor C_Y/C_{10} for Different Regions

4 ACCURACY OF THE PROCEDURE

Two methods were employed to evaluate the accuracy of the new procedure:

i. The first method compared 10-year design peak discharges estimated using the new procedure with 10-year peak discharges of observed data derived using single-station frequency analysis.

ii. The second method compared design peak discharges derived using the new procedure with design peak discharges derived using HP5:1989.

4.1 Comparison with Observed Data

The result of the comparison between 10-year design peak discharges estimated using this procedure (based on Regional C_{10}) with 10-year peak discharges of observed data is presented as a scatter diagram. Figure 3 presents the scatter diagram for Regions 1, 2, 3, and 4.

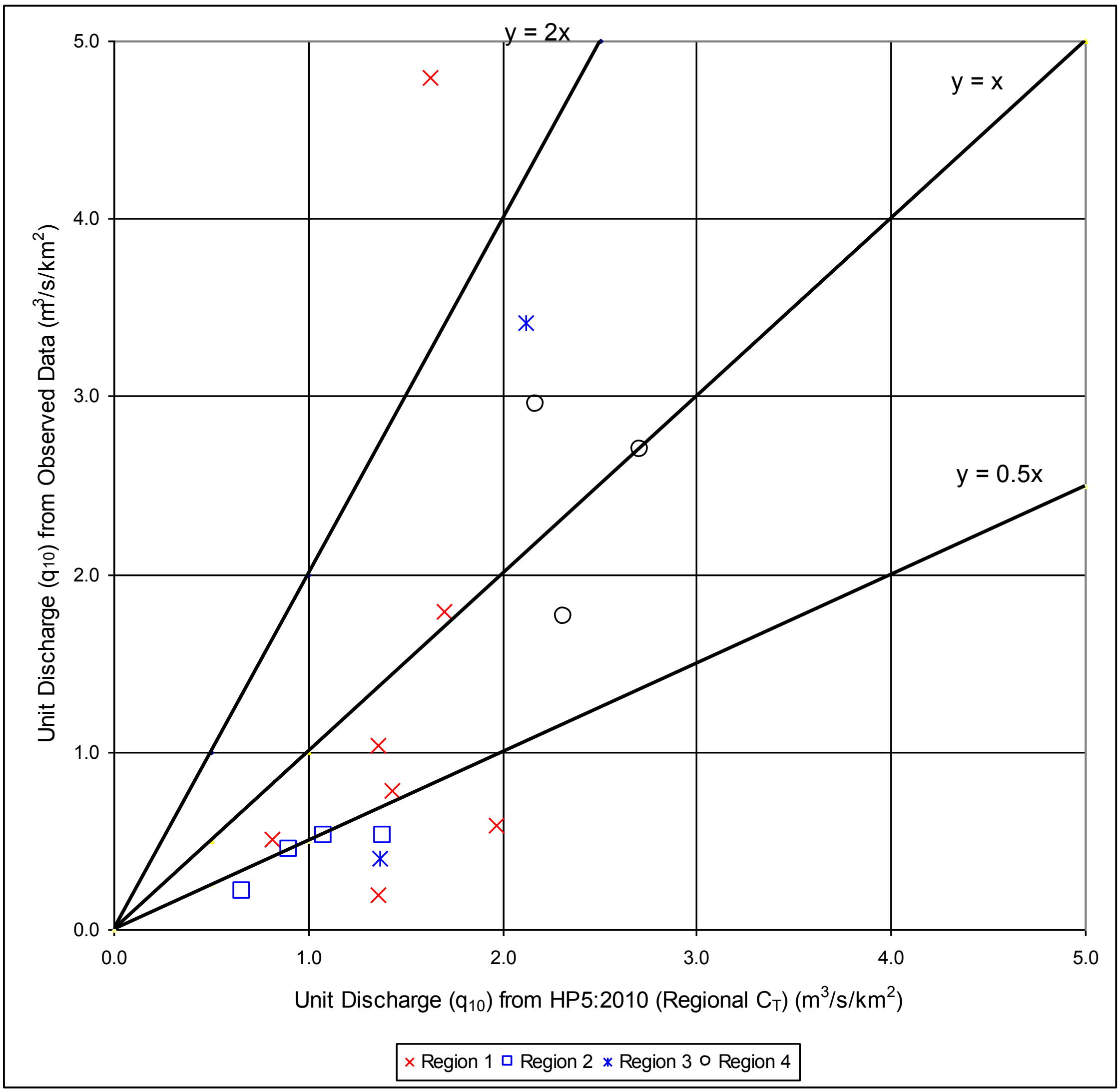

Figure 3. Scatter Diagram Comparing q_{10} Values Obtained from HP5:2010 and q_{pro} and q_{10} Values Obtained from Observed Data q_{obs}

4.2 Comparison with HP5:1989

Table 3 compares the results obtained from HP5:1989 and the frequency analysis of observed data from Sg. Telemong at Paya Rapat.

Table 3. Comparison of Results (Q_{10}) of Various Methods

Method	Q_{10} (m^3/s)
HP5:1989	316
HP5:2010	433
Frequency analysis of observed data	432

5 LIMITATIONS OF THE PROCEDURE

From the theoretical basis of the Rational Method, two important factors are neglected: (1) the effects of channel storage and (2) the temporal and spatial variations of rainfall intensities. As a result of such limitations and because the procedure was derived utilizing data from rural catchments with areas ranging from 3.9 to 186 km^2, the use of the procedure in estimating runoff for larger areas is not recommended. A multiplying factor that considers catchment development (Appendix A) should serve as a general guide to arrive at a reasonable estimate. However, there has been no study to substantiate this recommendation. The procedure may not be reliable in estimating runoff in areas with steep slopes. As a general guide, the slope value limit, as developed by this procedure, is approximately 0.1%–5%. Similar to any other flood estimation procedures, the design flood obtained from this procedure should be checked with other available procedures, and the decision to adopt the estimated design values should be complemented by sound engineering judgment.

6 USE OF THE PROCEDURE

6.1 Components of the Procedure

The following items are required to use this flood estimation procedure:

i. Figure 1. The general deposition of four regions proposed for application;

ii. Table 2. The regional runoff coefficient for four hydrological regions; and

iii. DID Hydrological Procedure No. 1 (Fadhillah et al., 1982) (HP1:1982), "Estimation of the Design Rainfall in Peninsular Malaysia."

6.2 Work Sequence

The detailed design sequence for using this procedure is as follows:

Step 1 Select a return period for the design (T_1).

Step 2 Based on catchment characteristics, estimate the value of T_C using Equation 4.

Step 3 Compute the depth of the design storm of duration T_C for return periods of 2, 10, and 20 years. Estimates for return periods of 2 and 20 years are required to estimate the confidence limit of the design storm.

Step 4 Compute the confidence limit of $X(T_1, T_C)$, which is estimated based on $X(2, T_C)$ and $X(20, T_C)$ values. Refer to HP1:1982 for a complete explanation.

Step 5 Compute the depth of the design storm of duration T_C for return period T_1.

Step 6 Estimate the value of C from Table 2.

Step 7 Compute the design discharge Q_T using Equation 1 and compute the confidence limits of the design discharge.

Step 8 Adjust Q_T based on a future land use scenario using factor F, which is listed in Appendix A.

6.3 Worked Examples

Example 1

Flood estimation is required for a rural catchment with the following data:

Location	=	Lat. 4° 00' N; Long. 102° 00' E (near Kg. Budu, Lipis)
Area	=	25.9 km^2
Slope	=	3%
River length	=	6.44 km
Development from jungle	=	40%

Solutions

Step 1 Choose return period T = 10 years.

Step 2 Compute T_c

$$T_c = \frac{1.286 \times L}{A^{0.223} \times S^{0.263}} = \frac{1.286 \times 6.44}{25.9^{0.223} \times 3^{0.263}} = 3 \ hours$$

Step 3 From HP1:1982,

X(2,3)	=	78 mm
X(10,3)	=	122 mm
X(20,3)	=	140 mm

Step 4 Confidence interval = 0.43 D

$$\begin{aligned} D &= X(20) - X(2) \\ &= 140 - 78 \\ &= 62 \ mm \end{aligned}$$

$$\begin{aligned} 0.43D &= 0.43 \times 62 \\ &= 26 \ mm \end{aligned}$$

Step 5

$$\begin{aligned} i_{10} &= \frac{X(10)}{3} \\ &= \frac{122 \pm 26.7}{3} \\ &= 40.7 \pm 8.9 \ mm/hr \end{aligned}$$

Step 6 From Table 2, C_{10} = 0.4928 for Region 4.

Step 7

$$\begin{aligned} Q_{10} &= 0.278 \times C_{10} \times i_{10} \times A \\ &= 144.41 \pm 31.58 \ m^3/s \end{aligned}$$

Step 8 From Appendix A, multiplying factor F for development = 1.05.

$$\begin{aligned} Q_{10} &= (144.41 \times 1.05) \pm (31.58 \times 1.05) \\ &= 151.63 \pm 33.16 \ m^3/s \end{aligned}$$

Example 2

A flood estimate is required for a rural catchment. The following data relate to the catchment:

Location	=	Lat. 4° 00' N; Long. 102° 00' E (near Kg. Budu, Lipis)
Area	=	25.9 km^2
Slope	=	1%
River length	=	6.44 km
Development from jungle	=	40%

Solutions

Step 1 Choose return period T = 10 years.

Step 2 Compute T_c

$$T_c = \frac{1.286 \times L}{A^{0.223} \times S^{0.263}} = \frac{1.286 \times 6.44}{25.9^{0.223} \times 1^{0.263}} = 4 \ hours$$

Step 3 From HP1:1982,

X(2,4) = 80 mm

X(10,4) = 98 mm

X(20,4) = 105 mm

Step 4 Confidence interval = 0.43 D

$$D = X(20) - X(2)$$
$$= 105 - 80$$
$$= 25 \ mm$$
$$0.43D = 0.43 \times 25$$
$$= 10.7 \ mm$$

Step 5

$$i_{10} = \frac{X(10)}{4}$$
$$= \frac{98 \pm 10.7}{4}$$
$$= 24.5 \pm 2.7 \ mm/hr$$

Step 6 From Table 2, C_{10} = 0.4928 for Region 4.

Step 7

$$Q_{10} = 0.278 \times C_{10} \times I_{10} \times A$$
$$= 86.93 \pm 9.58 \ m^3/s$$

Step 8 From Appendix A, multiplying factor F for development = 1.05.

$$Q_{10} = (86.93 \times 1.05) \pm (9.58 \times 1.05)$$
$$= 91.28 \pm 10.06 \ m^3/s$$

Example 3

Obtain a flood estimate for a culvert on a main trunk road. The following data relate to the culvert and the catchment:

Location	=	Lat. 5° 00’ N; Long. 103° 00’ E (near Kg. Pelandan, Hulu Terengganu)
Catchment area	=	5.18 km^2
Catchment slope	=	5%
River length	=	2.41 km
Development from jungle	=	0%

Solutions

Step 1 Choose return period T = 20 years.

Step 2 Compute T_c

$$T_c = \frac{1.286 \times L}{A^{0.223} \times S^{0.263}} = \frac{1.286 \times 2.44}{5.18^{0.223} \times 5^{0.263}} = 1.406 \ hours$$

Step 3 From HP1:1982,

X(2, 1.406)	=	75 mm
X(10, 1.406)	=	118 mm
X(20, 1.406)	=	135 mm

Step 4 Confidence interval = 0.43 D

$$\begin{aligned} D &= X(20) - X(2) \\ &= 135 - 75 \\ &= 60 \ mm \end{aligned}$$

$$\begin{aligned} 0.43D &= 0.43 \times 60 \\ &= 25.8 \ mm \end{aligned}$$

Step 5

$$i_{10} = \frac{X(10)}{1.4} = \frac{118 \pm 25.8}{1.4} = 83.0 \pm 18.4 \ mm/hr$$

$$i_{20} = \frac{X(20)}{1.4} = \frac{135 \pm 25.8}{1.4} = 96.0 \pm 18.4 \ mm/hr$$

Step 6 From Table 2, for Region 4,

C_{10} = 0.4928

C_{20} = 0.5193

Step 7

$$\begin{aligned} Q_{10} &= 0.278 \times C_{10} \times i_{10} \times A \\ &= 0.278 \times 0.4928 \times (83 \pm 18.4) \times 5.18 = 58.9 \pm 13.06 \ m^3/s \end{aligned}$$

$$\begin{aligned} Q_{20} &= 0.278 \times C_{20} \times i_{20} \times A \\ &= 0.278 \times 0.5193 \times (96 \pm 18.4) \times 5.18 = 71.8 \pm 13.76 \ m^3/s \end{aligned}$$

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<u>Appendix A.</u> Multiplying Factors that Account for Catchment Development

Development to Agriculture from Jungle X (%)	Multiplying Factor (F)
$0 < X \leq 25$	**1.00**
$25 < X \leq 50$	**1.05**
$50 < X \leq 75$	**1.15**
$75 < X \leq 100$	**1.20**

Note: Multiply Q_T estimates from undeveloped area by Factor F.

Source: DID HP5 (1974)

Appendix G: Multiplying Factors to take Account of Catchment Development (Extracted from Appendix A, HP5:2010).

Development to Agriculture from Jungle X (%)	Multiplying Factor (F)
$0 < X \leq 25$	1.00
$25 < X \leq 50$	1.05
$50 < X \leq 75$	1.15
$75 < X \leq 100$	1.20

Note: Multiply Q_T estimates from undeveloped area by factor F.

Source: DID HP5 (1974).