FLOOD FORECASTING MODELS FOR THE CIMANUK RIVER IN WEST JAWA, INDONESIA

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By

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ABSTRACT

This thesis concerns the development and application of two methods of estimating flood discharges on a real-time basis for the Cimanuk River at the City of Jatigede in West Jawa, Indonesia.

The first method was based on a multiple regression analysis of the rainfalls at each of the four sub-basins of the Cimanuk River Basin and the runoff at the outlet of the Basin at Jatigede. This multiple regression model was developed based on the peakflows at the basin outlet as the dependent variable and the total rainfall at each of the four sub-basins as the independent variables. Both Ordinary Least Squares (OLS) and Robust regression methods were used to develop the multiple regression equation.

The second approach was based on the concept of a Transfer Hydrograph (TH), which is a transfer function that transformed total rainfall into flood hydrographs. As opposed to the conventional unit hydrograph, which is derived using the effective rainfall and direct runoff of a basin, the TH was derived using the total rainfall at each of the four sub-basins and the direct runoff at the outlet of the whole basin at Jatigede.

These two methods were validated using rainfa[illegible] off data that we[illegible] not used in deriving the models. In comparing the two methods the results showed that the TH approach gave more accurate forecasts in terms of both the magnitudes and the timing of the flood peaks.

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Ckj	Cikajang
Dym	Dy.Manggung
Wnr	Wanaraja.
Mlb	Malangbong
TR_c	Total Rainfall at the Cikajang Sub-basin.
TR_d	Total Rainfall at the Dy.Manggung Sub-basin.
TR_w	Total Rainfall at the Wanaraja Sub-basin.
TR_m	Total Rainfall at the Malangbong Sub-basin.
R_{adj}	Adjustified coefficient of determination.
R	Correlation coefficient.
HEC-1	Hydrologic Engineer Centre
HYMO	Hydrologic Model
TR-20	Technical release no.20
C	Celcius
LS	Least Squares
LMS	Least Median of Squares
RLS	Reweighted Least Squares
e	Residual

‹	Discharge
	Regression Coefficient
VIF	Variance Inflation Factor
oeff	Coefficient.

CHAPTER 1

INTRODUCTION

1.1 Background

The Cimanuk river is located in West Jawa, Indonesia, as shown in Figure 1.1. Prior to 1980, overtopping of this river caused severe flooding to areas downstream of Jatigede about 4 to 5 times a year during the wet season. The wet season is from November to May. The flooded area has typically included 55000 hectares of rice fields, about 100 villages, and the two cities of Indramayu and Kadipaten.

In 1977, the Government of Indonesia proposed a two-stage plan to protect the flooded areas prone to flooding. The first step was to protect the city of Indramayu, as well as 50000 hectares of rice fields, and most of the villages by constructing about 200 km of dykes beside the river. The areas prone to flooding and location of the dykes are shown in Figure 1.2. The dykes were designed based on flood discharges with return periods ranging from 10 to 25 years, depending on the locality. However, there are still about 5000 hectares of rice fields and many villages that are presently not protected from the annual floods. These areas are now being used as detention basins to attenuate the flood-peaks so as to reduce the effects of flooding on downstream areas.

The second step was to protect all the areas prone to flooding by means of a flood control dam at Jatigede. This dam was scheduled to be built in 1985. However, because of difficulty in funding the project, the construction of the dam has been postponed indefinitely.

At present, there are two alternatives available to minimize the impacts of flooding of these unprotected areas. The first alternative was to build dykes along the present detention basin and to raise the existing dykes downstream. However this alternative is very expensive and given the present economic situation is almost impossible to be realized. The second alternative was to establish a flood forecasting and flood warning system for the unprotected areas. This alternative is relatively inexpensive and perhaps is the most-cost effective alternative at present.

The method now used for flood forecasting and flood warning in the Cimanuk River is by means of measuring local water levels. If the water level at a given location reaches a certain elevation considered by the government to be a dangerous level, people in the areas prone to flooding are evacuated. This system is unreliable and has not worked well. This is because the maximum water level associated with the peakflow , as well as the time of the peakflow cannot be predicted using this procedure.

From the foregoing, it can be seen that in order that the impact of the flooding on the currently unprotected areas be lessened, and in view of the budgetary constraints, an inexpensive yet accurate method of forecasting floods on the river was required. The methods to be used would depend on the availability of data, which will be discussed in the next section.

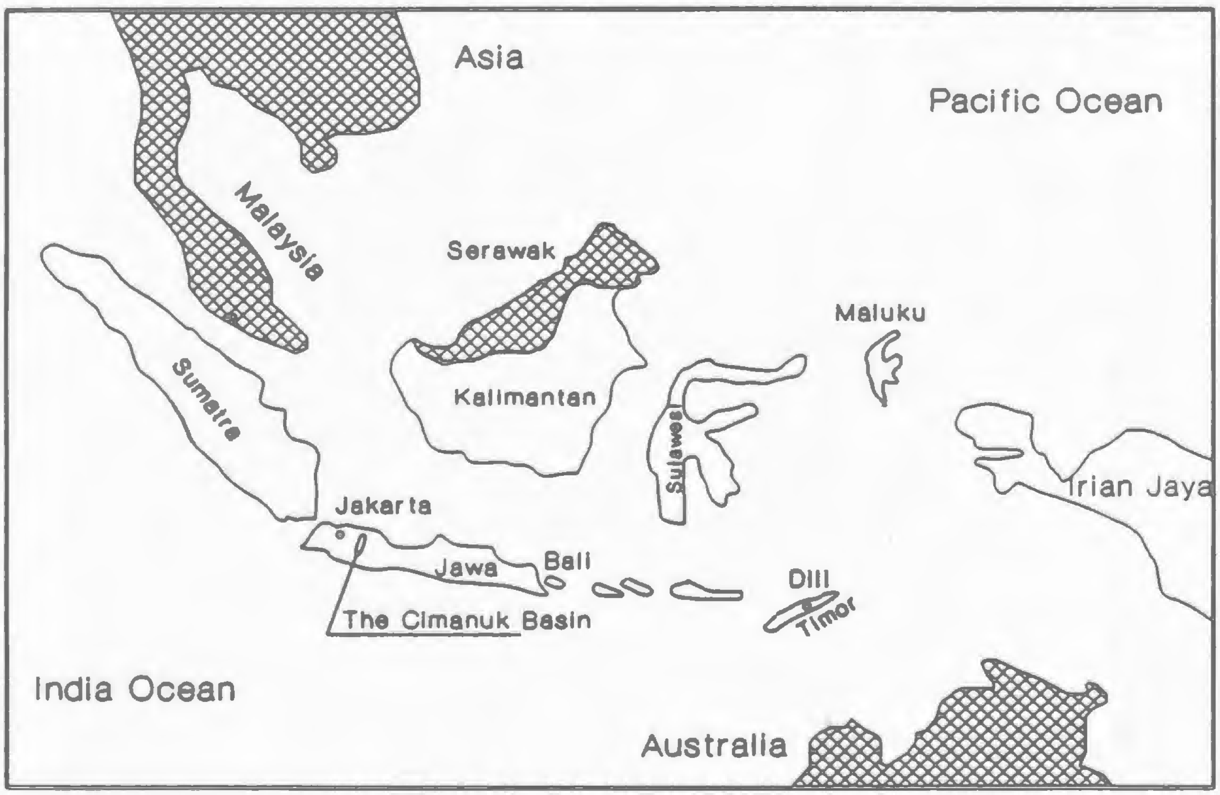


Figure 1.1 Location of Cimanuk River Basin within Indonesia.

Figure 1.2 Location of the flooded areas and the existing dyke.

1.2 Available Data

Continuous measurement of the Cimanuk River discharge at Jatigede, which is upstream of the unprotected areas, has been done since 1970. Automatic measurement of rainfalls has been available since 1978, when the Management of the Cimanuk River was taken over by a government agency known as "The Cimanuk Project". The organisation of the Cimanuk Project is shown in Figure 1.3.

Based on the topography of the area, there are four sub-basins upstream of Jatigede. Each sub-basin rainfall is presently measured by means of tipping bucket raingauges. These gauges are located at Cikajang, Dy.Manggung, Wanaraja and Malangbong. However, flow measurements upstream of the areas prone to flooding is only available at Jatigede. Therefore, any rainfall-runoff relationship can only be developed between the rainfalls at the four sub-basin gauges, and the out-flow at Jatigede.

In addition to the rainfall and runoff data some general information on land use and topography was available. However, this information is not sufficiently complete for use as input to such computer-based flow simulation models as HEC-1 (1973), HYMO (1972) or TR-20 (1973). Computer models have therefore not been used for flow simulation in the Cimanuk River Basin up to the present.

1.3 Objectives of the Research

From the previous section, the only reliable data available are the rainfall data at each of the four sub-basins and the runoff at Jatigede. Therefore, the primary objective of this thesis is to make use of this rainfall-runoff data to develop a real-time flood forecasting model for the outlet at Jatigede.

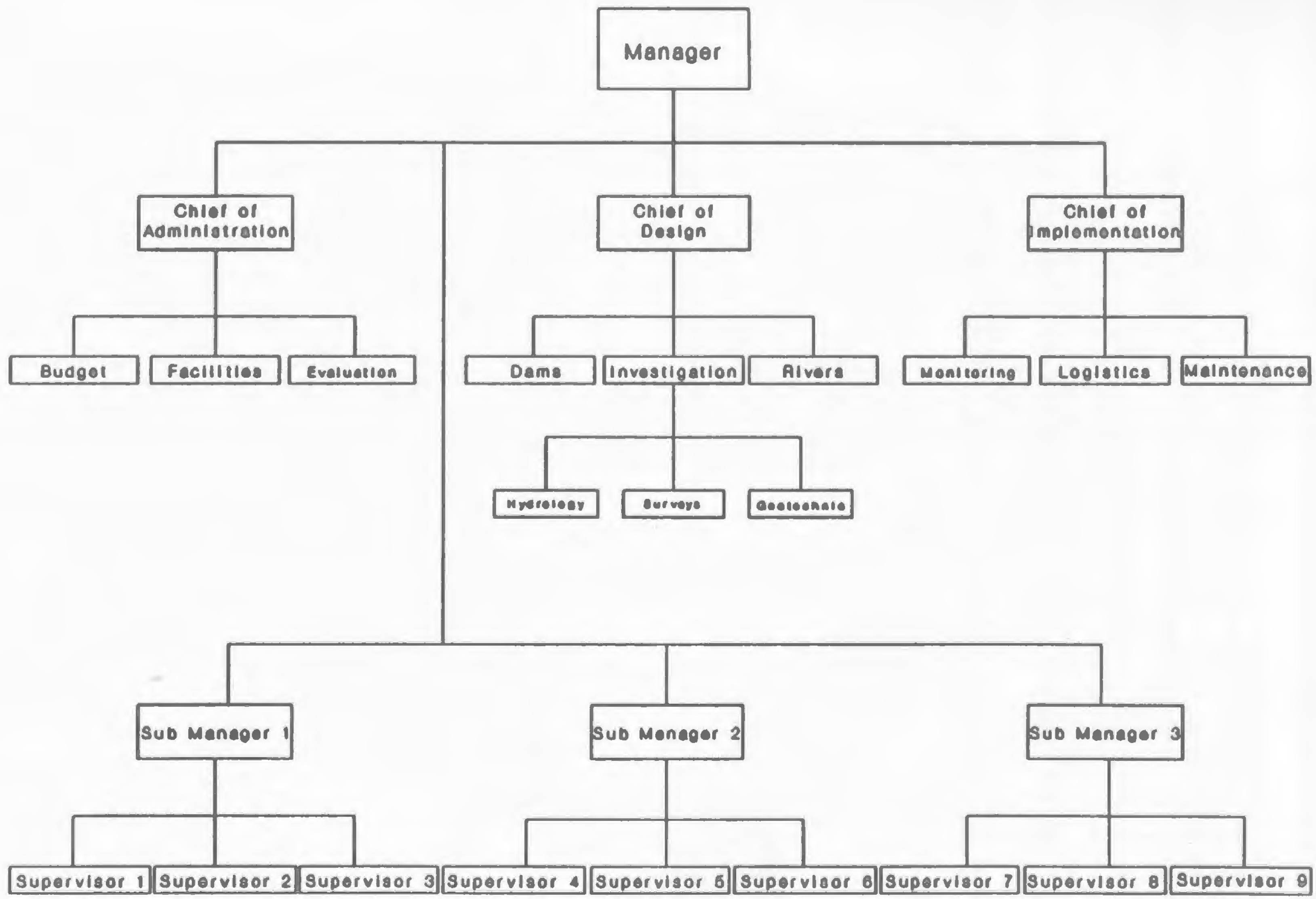


Figure 1.3. The Cimanuk Project Organisation.

Two methods were considered in this thesis. The first method was based on a multiple regression analysis between the peakflows at Jatigede as the dependent variable and the rainfalls at four sub-basins as the independent variables. The second method was based on the concept of the Transfer Hydrograph (TH). Four transfer hydrographs were derived for flood forecasting model. These four transfer hydrographs were combined to give the flood hydrograph at the outlet, Jatigede. The two methods were compared and recommendations were required with respect to the implementation of the flood forecasting models.

1.4 Outline of the Thesis

The introduction to the flooding problem on the Cimanuk River Basin downstream of Jatigede is described in this chapter. In the next chapter, a description of the study area is presented. Chapter 3 discusses the literature relevant to the present study. The procedures for developing the flood forecasting models in the Cimanuk River are discussed in Chapter 4. Chapter 5 presents and discusses the results of the study. The conclusions and recommendations of this study are presented in chapter 6. The flow charts, the listing of the computer programs, comparison between the results of the Multiple Regression model and the Transfer Hydrographs, the convoluted hydrograph and the type of events used in derivation and validation of the multiple regression and the transfer hydrographs are shown in Appendices.

CHAPTER 2

DESCRIPTION OF STUDY AREA

2.1 Location and land use

The area considered in this study is the Cimanuk River Basin, located in West Jawa, Indonesia. The region lies in the tropical zone, between 6.7° to 7.3° south latitude and between 107° to 109° east longitude. The headwaters of the river are 1200 m above sea level, whereas the outlet of the study area is 200 m above sea level. The location and area of the drainage basin and the four main sub-basins are shown on Figure 2.1 and Table 2.1. The various types of land-use are homogeneously distributed within the total drainage area. These land uses are shown in Table 2.2. The population density of the area is about 893 people per square kilometre and the majority of the population are farmers (Directorate General of Reforestation and Land Rehabilitation 1989).

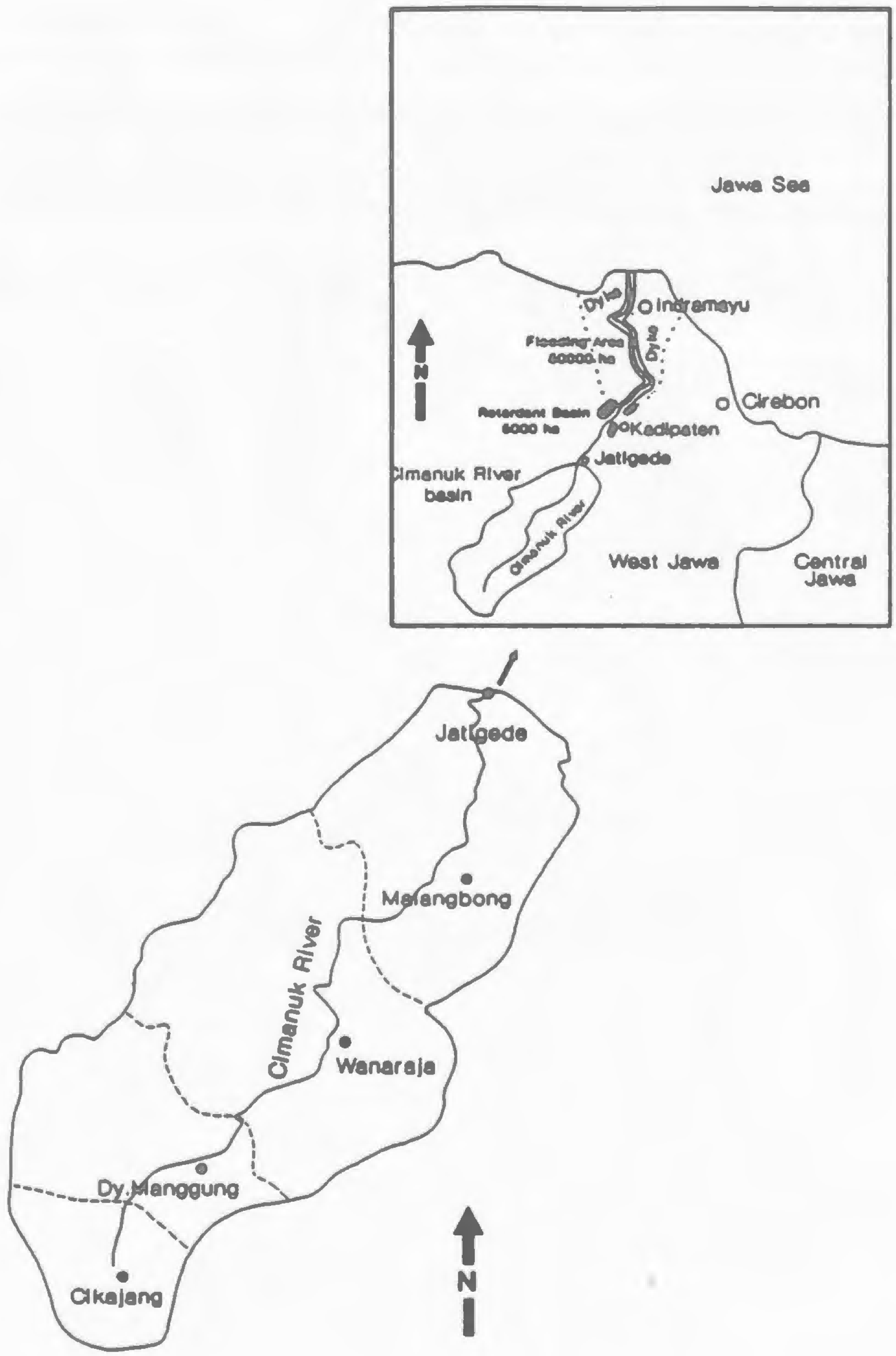


Figure 2.1 Location of the four Cimanuk Sub-basins (not to scale).

Table 2.1 The sub-basin area.

No	Sub-basin	Area (km^2)	Length of the river (km)	Average width of the river (m)
1	Cikajang	160	11	20
2	Dy.Manggung	260	11	25
3	Wanaraja	621	33	40
4	Malangbong	401	30	50

Total area of the Cimanuk drainage basin: 1442 km^2

Table 2.2 Types of land use.

No	Type of land use	Percent area
1	Horticulture	26 %
2	Rice fields and fish ponds	29 %
3	Forest	40 %
4	Others (eg. residential)	5 %

2.2 Sources of data

The rainfall, runoff, topographic and demographic data were provided by the following institutions:

a) The rainfall data were collected from the Hydrology Office of the Cimanuk

Project in Cirebon, West Jawa. The rainfall data were read directly from the rainfall chart-trace from the automatic raingauges. Twelve years (1978 to 1989) of hourly rainfall data were available for this study.

b) The runoff data were obtained from the Hydrology Office of the Cimanuk Project in Cirebon and from the Parakan Kondang Hydropower Station in Sumedang, West Jawa. A portion of the data was obtained directly from water level measurements during the flood season. These water elevations were converted into flow discharges using a rating curve, which was established by the Hydrology Office of the Cimanuk River Project.

c) The topographic map was supplied by the Geodetic Office of the Cimanuk Project in Cirebon.

d) The demographic and the land use data were obtained from a government agency known as "the Cimanuk Basin Critical Land Improvement Project" in Majalengka.

2.3 Hydrology and Climate

The Cimanuk River Basin, as mentioned before, is located in the tropical zone. A characteristic of this region is the small variation in the incoming solar energy. For this reason the temperature and the humidity are relatively constant during the year. The climate of Indonesia is mainly affected by the trade winds, which are influenced by the monsoons. The monsoons are the winds that blow over Indonesia from the northwest and southwest for about six months of the year.

The climate of the Cimanuk River basin is significantly influenced by the monsoons during the wet season, November through May, when the trade winds are strengthened by the monsoons. These develop in the cold areas of Asia due to differential atmospheric pressure. As the wind passes over the South China Sea, it picks up moisture. The direction of the monsoons then changes toward the north and eventually becomes northwesterly as they cross the equator. During the dry season, May through November, the temperature over Asia becomes warmer and causes a reversal in the direction of the air mass movement. Since the southeast monsoon does not pass over as much water area prior to reaching the island of Jawa, the rainfall during this period is significantly less.

In general, the Cimanuk River basin is characterized by high and relatively consistent temperature and humidity in both the wet and the dry seasons. The annual average humidity in the catchment area is estimated to be 84.0 % (Directorate General of Reforestation and Land Rehabilitation 1989). The seasonal temperature variation in the basin is very small, being generally less than 3° C. Therefore, it is difficult to distinguish season by temperature alone. The monthly average air temperature at the Cimanuk River near Jatigede ranges from 26.1°C to 28.0°C. The annual average air temperature at Jatigede in the Cimanuk River basin is approximately 27°C. The wind records indicate that the wind velocity in the Cimanuk River basin is nearly constant from month to month. The maximum, the minimum and the average wind velocity are respectively 2.15 m/s, 1.59 m/s, and 1.85 m/s. For these reasons the effects of the climate on the hydrology of the basin does not vary greatly over time.

The maximum, minimum and average annual flow of the Cimanuk River at Jatigede are respectively, 1470, 470 and 1009 m^3/s. The maximum annual flow on record occurred in 1978. The average annual rainfall in the Cimanuk basin varies from 2000 mm in the lower elevation of the Cimanuk Basin to 4000 mm in the most upstream portion of the Cimanuk Basin (Directorate General of Reforestation and Land Rehabilitation 1989).

2.4 Topography

The upper reaches and headwaters of the Cimanuk River are surrounded by volcanoes. Most of the volcanoes in the upper Cimanuk region are higher than those in the middle region of the Cimanuk basin. The highest peaks in the basin are Mt.Cikuray (2821 m), Mt.Papandayan (2665 m), Mt.Kendang (2608 m) and Mt.Guntur (2249 m). There are also some volcanoes in the middle of the basin; these are the Calancang (1667 m), Cakrabuana (1721 m) and Sanghiang (1632 m) peaks. The lowest elevation (200 m) is at Jatigede, where the flow is gauged. The location of the volcanoes are shown in Figure 2.2.

According to a study of the Cimanuk catchment area, which was done by Directorate General of Reforestation and Land Rehabilitation in 1989, the ground slope of the Cimanuk basin is quite uniform as shown in Table 2.3.

Figure 2.2 Location of volcanoes (not to scale).

Table 2.3 Average Sub-basin ground slopes.

No.	Sub-basin	Ground slope (%)
1	Cikajang	23
2	Dy.Manggung	27
3	Wanaraja	24
4	Malangbong	27

Although there is a large difference in elevation between the upper reaches of the river and the point at which the discharges are observed, the river bed gradient is generally not very steep. This is because of a number of waterfalls along the river. The longitudinal section of the river is shown in Figure 2.3. Recently, at the upper reaches of the Cimanuk River, a number of river bed control structures and detention basin structures have been constructed. Also, rice fields and fishponds in the Cimanuk basin may have a significant influence on the hydrology of the basin because of their storage volume (See Table 2.4).

Table 2.4 Percentage of rice field and fish pond area within each sub-basin.

No.	Sub-basin	Rice field and fish pond area (% of area)
1	Cikajang	24
2	Dy.Manggung	27
3	Wanaraja	25
4	Malangbong	27

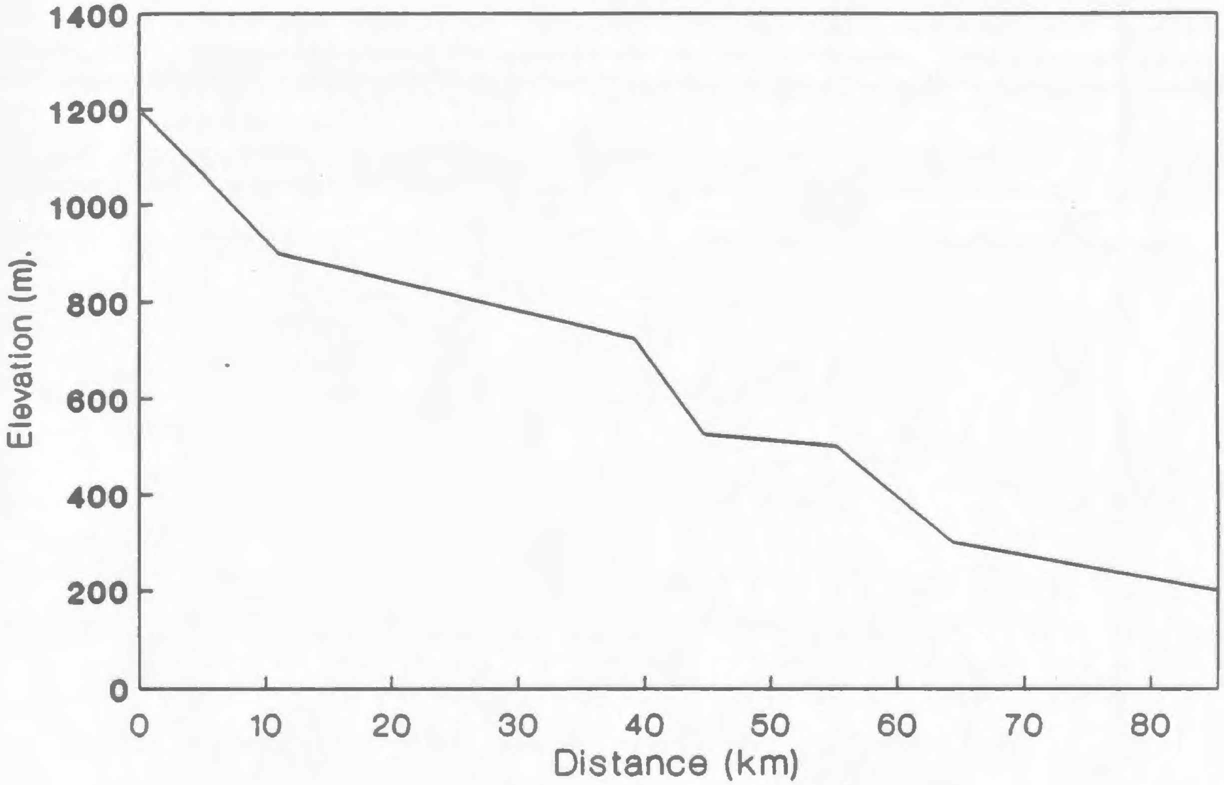


Figure 2.3 Longitudinal section of the Cimanuk River.

CHAPTER 3

REVIEW OF LITERATURE

Flood forecasting systems have been widely used to predict the magnitude of discharge so as to mitigate loss of life and property caused by flooding. Many models have been developed for flood forecasting purposes. Using a model, the flood discharge at a certain point in the river can be estimated from the rainfall data that are inputted into the model.

The method for estimating the magnitude of discharge is based on the availability of data and on whether the characteristics of the basin are well understood. Two types of basins can be distinguished, depending on the availability of data. The first type is a basin where the physical characteristics of the basin are well documented. The second type is a basin where the physical characteristics of the basin have not been well documented.

3.1 Models where the physical characteristics of basin are well documented

Many models are developed based on the assumption that almost all the physical phenomenon of nature, such as evaporation, infiltration, storage, overland flows, and channel flows, are known and can be taken into account in transforming rainfall into runoff. These models can be successfully used if the physical processes in transforming rainfall into runoff are well understood. Some examples of this type of flood forecasting models are HEC-1 (1973), HYMO (1972) and the TR-20 (1973).

For example, HEC-1 is a flood hydrograph computer package developed by the Hydrologic Engineering Centre, US Army Corps of Engineers, in Davis California. This program can be used for flood forecasting purpose. In the use of HEC-1, the basin area is divided into sub-basins. Each sub-basin, which is considered to be a part of the river network, must have rainfall and runoff data. The calculation procedure in HEC-1 starts with the uppermost basin of the river network until a confluence is reached. Before proceeding further downstream, all upper-reach flow must be computed and routed to that point. At confluences the flows are combined. This procedure is continued to the outlet of interest.

The HYMO model (Williams and Hann 1972) can also be used for flood forecasting. The model is based on unit hydrographs, approximated by a two parameter gamma function. The parameters are estimated based on basin area, slope and the ratio of length and width. Like HEC-1, HYMO needs flow hydrographs at every control point of the river, such as at the outlet of each sub-basin, at the confluence of the river, and at detention areas.

Soil Conservation Service TR-20 model (SCS TR-20) (quoted in Ponce, 1989, p.413-416) was developed by Soil Conservation Service, USA (1973). This model can also be used for flood forecasting. The unit hydrograph can be derived from rainfall-runoff data or can be synthetically derived using basin parameters. The procedure to estimate the hydrograph at the basin outlet starts from the uppermost part of the basin, as in HEC-1 and HYMO. Therefore, the flow hydrograph data at each of sub-basin must be available for estimating the downstream hydrograph.

There are several limitations of computer simulation models, such as the HEC-1, the HYMO, and the TR-20, as described by Ponce (1989). These include:

a) Misinterpretation in determining the physical characteristics of the basin;

b) The model can only be used for a specific design, but it can not determine alternative design;

c) A weak input data set can still produce an output.

3.2 Model used when the physical characteristics of the basin are not well documented

Many basins exist where the physical characteristics of the basin have not been well documented. Models that are normally used in a basin where there is a lack of data are generally black-box type models. Black-box models are used in transforming rainfall data into run-off data regardless of the physical process of the transformation. Examples of the black-box models include Multiple Regression Models and those based on the Unit Hydrograph approach.

3.2.1 Regression approach

Multiple regression has been widely used in the field of hydrology. This method is especially useful when the physical characteristics of the basin have not been completely documented. Many statistical program are available and the technique is well known.

An application of multiple regression in flood forecasting for example was discussed by Liang (1988). He developed a model for flood forecasting in the Hankou Basin in China using a multiple input, single output, linear, time-invariant regression model. The flow hydrograph at Hankou was estimated based on the flow hydrographs at the outlet of three tributaries: Hanjiang River, Changjiang River, Qingjiang River, and the spillway outlet of Dongting lake.

3.2.2 Unit hydrograph approach

The concept of a unit hydrograph (UH) was originally proposed by Sherman (1932). He defined a unit hydrograph as the graph that resulted in 1.00 inch of runoff. Also, he defined that the time base for all unit hydrographs, derived from rainfalls with the same duration, was the same, and the ordinates of the unit hydrograph were considered to be proportional to the volume of runoff.

Johnstone and Cross (1949) (quoted in Singh, 1988, p.141) further clarified the definition of the unit hydrograph, and this has formed the basis for unit hydrograph theory today. They concluded that the time-base of all hydrographs resulting from rainfalls with the same duration, was the same. The ordinates of any two hydrographs, derived from two uniform-intensity storms with the same duration, are then proportional to their intensities.

There are two types of unit hydrographs. The first type uses the recorded rainfall-runoff data in its derivation. The second type, which is known as a synthetic unit hydrograph, is empirically derived. For example, Snyder (1938) developed a synthetic unit hydrograph based on the physical geometry of the area. However, it is always preferable to *avoid* relying on synthetic UH's if non-synthetic UH's can be developed.

Laurenson and O'Donnel (1969) examined three methods of deriving unit hydrographs using rainfall-runoff data. The three methods were: harmonic analysis, Meixner polynomials, and least squares. A comparison of the results indicated that no one method was better than any other methods, from a general point of view. Dooge and Garvey (1970) studied the same four methods for the unit hydrograph derivation and found that the least squares method was the best method for good data, but the worst method for bad data and that Meixner polynomials was the best for erroneous data.

Dooge (1979) discussed four categories of methods, which can be used to derive unit hydrographs. The four categories were Direct Matrix Inversion, Optimisation, Transform System Methods, and Identification Using Conceptual Models. The first category of the methods was Matrix Inversion and this category consisted of forward substitution, backward substitution, and the Collins method of matrix inversion. The second category was Optimisation methods and these consisted of least squares, regularisation and quadratic programming methods. The third category, known as Transform Systems, included: Z-Transform, Harmonic Analysis and Meixner Analysis. The fourth category was identification of unit hydrograph based on Conceptual Models. These consisted of single reservoir, triangle, two equal reservoirs, routed triangle, routed rectangle, and n equal reservoirs method. Based on a detailed comparison of the various methods, Dooge recommended three methods: Harmonic Analysis, Meixner Analysis, and the Conceptual Model using n equal reservoirs.

Singh (1976) compared the use of Linear Programming and Least Squares Methods in the derivation of unit hydrograph. The Linear Programming method minimized the sum of absolute errors and the Least Squares Method minimized sum of the squares of the errors. The Linear Programming Technique constrained the ordinates of unit hydrograph to be positive. The results of the study showed that two of the result of the test were the same, while two others showed some deviation in the unit hydrographs. However, the deviated unit hydrographs were derived using more nonuniform and longer duration rainfall data.

Singh, Baniukiewicz, and Ram (1982) (quoted in Singh, 1988, pp.181) studied nine procedures to derive the unit hydrograph. The nine procedures were: Matrix Method (MT), Forward Substitution (FS), Successive Over-Relaxation (SOR), Least Squares (LS), Harmonic Analysis (HA), Laguerre Polynomials (LP), Meixner polynomials (MP), Time Series Method (TS) and Linear Programming Method (LPM). They concluded that LP, HA and LS were the best methods to derive unit hydrographs.

Mawdsley and Tagg (1981) discussed a Householder Transformation Technique to solve the ill-conditioned sets of equations. Several events were analyzed simultaneously to minimize the difference between the observed and the computed ordinates of the unit hydrographs. It was shown that better unit hydrographs can be obtained using the Householder Transformation in analyzing several events simultaneously. They concluded that the more the number of events analyzed the better the result will be.

Bruen and Dooge (1984) discussed an efficient and robust method for estimating unit hydrograph ordinates. They concluded that the Smoothed Least Squares is an effective method of overcoming the instability of unit hydrograph due to numerical ill-conditioning.

Wang (1986) discussed four methods of estimating the parameters of discrete linear input-output models. The four methods used for estimating of parameters were: Linear Programming, Quadratic Programming, and Least Squares estimates, and Correlation Function estimates. He found that the Quadratic Programming and the Least Squares estimates gave the best fit.

Dooge and Bruen (1989) studied the sensitivity of classical methods such as Forward Substitution, Collins Method, Least Squares Method, and the Smoothed Least Squares Method in the derivation of unit hydrograph. They concluded that for the Forward Substitution Method, the amplification of error was greater when the intensity of effective rainfall increased during a storm.

From the above discussion it can be seen that different conclusions have been reached by various hydrologists concerning the best method of deriving unit hydrographs. However, many researchers recommended the use of the Least Squares methods of deriving unit hydrographs from rainfall-runoff data.

The techniques considered in this research were restricted to Black Box approaches because of the lack of detailed information about the hydrology of the four sub-basins. Two techniques were used in developing the flood forecasting model for the Cimanuk River Basin. One method was based on multiple regression, and another method was based on the concept of the Transfer Hydrograph, which was a modification of the classical unit hydrograph because of the unavailability of runoff data at each of the three sub-basins; the Cikajang, the Dy.Manggung, and the Wanaraja. These methods would be discussed in the next chapter.

CHAPTER 4

METHODOLOGY

4.1 Introduction

This chapter discusses the two techniques that were used to develop flood forecasting models for the Cimanuk River. One technique was based on regression analysis, and the other technique was based on the Transfer Hydrograph approach. The use of the two techniques is in accordance with the availability of data in the Cimanuk River, as was discussed in the previous chapters.

The regression and the Transfer Hydrograph (TH) techniques of flood prediction may be categorized as black-box approaches. They can be represented schematically, as shown in Figure 4.1. Black box methods are usually applied to complex basins where the physical characteristics have not been well documented.

With black-box methods, the input data are transformed into output using a system operation that combines all physical characteristics of the basin. The physical characteristics of the basin include the physiography of the drainage basin, the nature of the drainage network, the land use and other hydrometeorological characteristics of the basin. A change in the physical characteristics of the basin will affect the system operation. The input data were rainfalls at the four sub-basins upstream of Jatigede, and the output was the discharge at the outlet, Jatigede.

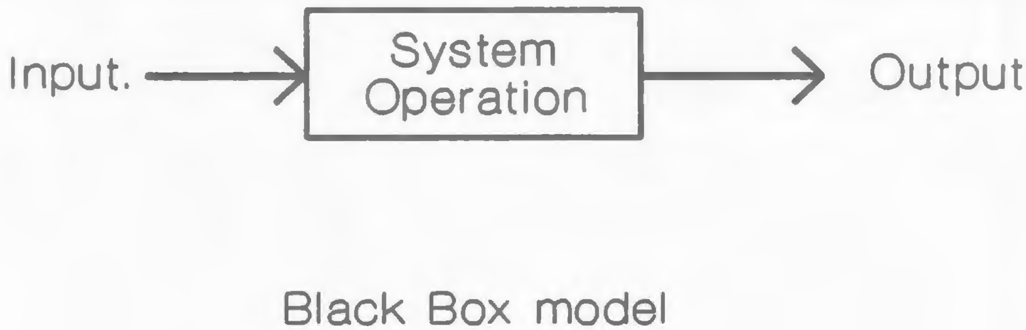


Figure 4.1 Black-box method

4.2 The Multiple Regression Approach

This section discusses the multiple regression approach that was used to develop a flood forecasting model for the Cimanuk River Basin at Jatigede. The multiple regression model, which was developed in this thesis, describes the statistical relationship between the rainfalls at the four sub-basins and the peak-flow at the outlet, Jatigede. The four sub-basins are shown in Figure 2.1. The multiple regression was derived using the rainfall events presented in Table 5.1. The regression function for peak discharge can be generally written as:

$$Q_p = f(T_c, T_d, T_w, T_m, TR_c, TR_d, TR_w, TR_m) \qquad (4.1)$$

where: T_c = time of the beginning of rainfall at Cikajang sub-basin,

T_d = time of the beginning of rainfall at Dy.Manggung sub-basin,

T_w = time of the beginning of rainfall at Wanaraja sub-basin,

T_m = time of the beginning of rainfall at Malangbong sub-basin,

TR_c = total rainfall at Cikajang sub-basin in mm,

TR_d = total rainfall at Dy.Manggung sub-basin in mm,

TR_w = total rainfall at Wanaraja sub-basin in mm,

TR_m = total rainfall at Malangbong sub-basin in mm,

Q_p = peakflow at Jatigede in m^3/s.

Without considering the time of the rainfall, the above equation can be simplified to:

$$Q_p = f(TR_c, TR_d, TR_w, TR_m) \tag{4.2}$$

Similarly, the time to peakflow (t_p) depends on the time of the beginning of the rainfall and the total rainfalls at the four sub-basins, and can be expressed as:

$$t_p = f'(T_c, T_d, T_w, T_m, TR_c, TR_d, TR_w, TR_m) \tag{4.3}$$

4.2.1 Methods used to derive the multiple regression model

Two methods were used to derive the multiple regression model. The first was based on the standard ordinary least squares (OLS), and the second method was the more recently developed robust regression technique, based on the least median of squares (LMS). These two methods of regression analyses will be discussed in the following sections.

a) The Ordinary Least Squares Method.

The OLS method has been extensively used in estimating parameters of regression equations. It is well known that the derivation of the regression equation based on the OLS method corresponds to:

$$Minimize \sum_{i=1}^{n} e_i^2 \qquad (4.4)$$

where: e_i = Y_i-$\hat{Y}$,

e = residual,

Y_i = observed peakflow,

$\hat{Y}_i$ = estimated peakflow from the regression model.

There are three advantages to using the OLS Method for data that are well-behaved (no-outliers). First, the OLS method gives unbiased estimators of the regression coefficients. Second, it gives the most efficient estimator, and the resulting regression equation produces minimum prediction error. Third, the regression equation will produce consistent estimators. However, most hydrologic data are seldom well-behaved. The OLS method lacks the ability to detect outliers. Such outliers can be detected by examining a plot of the residuals. An undesirable phenomenon can therefore occur when applying the OLS method: incorrect estimation of the response variable due to unidentified outliers in the response and predictor variables (Rousseeuw, 1990). In view of the possibility of encountering data containing outliers, a robust regression method based on the LMS method was used as a complementary form of analysis to detect possible outliers. The presence of outliers would probably mean that some events within the data set came from different populations.

b) Robust Regression Methods.

As mentioned above, a major problem in the use of the OLS regression method is that it lacks the power to detect outliers. To overcome this problem, statisticians have developed a method so called 'Robust methods' that are resistant to the effects of outliers. For example, Rousseeuw and Leroy (1987) developed the LMS method, and Huber (1973) have developed and applied a method so-called 'M Estimators' for regression analysis.

The LMS and M Estimators for regression analysis are more powerful than OLS methods in detecting the outliers as confirmed by Rousseeuw and Leroy (1987) and Rousseeuw (1990). In comparing the LMS and the M Estimator methods, the LMS method is more powerful in identifying outliers in both dependent *and* independent variables. Therefore, the LMS method of robust regression was used in this research to develop the multiple regression equation.

A computer program called "PROGRESS" (Program for Robust Regression) based on the LMS has been developed by Rousseeuw et al. (1987) and was used in this study to develop the regression equation. The regression equation based on the LMS method corresponds to:

$$\textit{Minimize median } e_i^2 \tag{4.5}$$

where: $e_i = Y_i - \hat{Y}$,

$Y_i =$ observed peakflow,

$\hat{Y}_i =$ estimated peakflow from regression model.

It should be pointed out that the derivation of a robust regression equation based on LMS can only be achieved in a reasonable time through the use of a computer.

Three regression options are available in 'PROGRESS'. Under the first option, the regression analysis is based on the OLS method. Under the second option, the regression analysis is based on the LMS method, which can be used to identify outliers. The third option is regression analysis using reweighted least squares (RLS) based on the LMS method. In developing a regression equation using the RLS based on LMS, outliers are given a weight (W_i) of 0 and for the remaining data, which are not categorized as outliers, are given weight (W_i) of 1. Indeed, for data set without outliers, the RLS and OLS will always show the same regression coefficients. This is because for a regression analysis based on RLS, all data will be given a weight of 1 and the calculation is carried out using the OLS method (ie, the first option).

A plot of the standardized residuals versus the estimated peakflows from the output of the second option was used to detect outliers in this study. The standardized residual was calculated using:

$$Standardized\ Residual = Residual/Sc$$

where: Sc (Scale estimate) is the standard deviation of residuals and is defined as:

$$Sc = \sqrt{\frac{\sum_{i=1}^{n} W_i\ e_i^2}{\sum_{i=1}^{n} W_i - p}} \qquad (4.6)$$

where: W_i = weight,

e_i = residual,

p = number of predictors.

In detecting outliers, any standardized residual larger than 2.5 was considered to be outliers as was pointed by Rousseeuw and Leroy (1987).

4.2.2 Measure of Goodness of fit

The reliability of a regression equation is indicated by the following statistical parameters:

a) The coefficient of determination.

The coefficient of determination (R^2) and the adjusted coefficient of determination (R_{adj}^2) describe the percentage of variance in the dependent variable explained by the independent variables. The adjusted R^2 takes into account the number of independent variables.

b) The estimated standard error.

The estimated standard error describes the scatter of the data about the regression line. It is the standard deviation of the differences between the observed and the calculated dependent variables. The smaller the standard error the better the fit.

c) The Variance Inflation Factor (VIF).

The VIF is a parameter calculated based on coefficient of correlation between one predictor variable and the others. The VIF is calculated using:

$$VIF = \frac{1}{1 - R^2} \tag{4.7}$$

where: R is a coefficient of determination of the relationship between one predictor and the others.

Generally, a VIF larger than 10 indicates that the regression coefficients are poorly estimated due to multicollinearity as discussed by Myers (1990).

4.2.3 Assumptions in developing the Regression Model

The multiple regression model for the rainfall-runoff relationship given by equation 4.1 was derived based on the following assumptions:

a) A *linear relationship* exists between total rainfalls at the four sub-basins and the peakflow at the outlet.

b) The rainfall data at one sub-basin was considered to be *independent* of the rainfalls at the other sub-basins.

c) The *effect* of the predictors, (that are, rainfalls at the four raingauges) on the dependent variable is *additive*.

d) The flow at the outlet was assumed to be the sum of the flows contributed by each of the four sub-basins, that is:

$$Q_p = \sum_{1}^{4}(C_i \; TR_i \; A_i) \qquad (4.8)$$

where C_i = the runoff coefficient for sub-basin (i),

TR_i = the total rainfall for sub-basin (i),

A_i = the sub-basin area for sub-basin (i).

e) The multiple regression model can therefore be written in the form:

$$Q_p = C_1TR_c + C_2TR_d + C_3TR_w + C_4TR_m + e \qquad (4.9)$$

where: Q_p = peakflow (m^3/s),

C_i = regression coefficients.

TR_c = total rainfall at the Cikajang sub-basin,

TR_d = total rainfall at the Dy.Manggung sub-basin,

TR_w = total rainfall at the Wanaraja sub-basin,

TR_m = total rainfall at the Malangbong sub-basin,

e = residual.

4.2.4 Regression Analysis Procedure

The procedure used to derive the multiple regression equation was as follows.

a) Four sub-sets of data for the purpose of outliers analysis were made up of:

- Set (i) = all 32 available rainfall-runoff events.
- Set (ii) = data with peakflows greater than 300 m^3/sec (22 events).
- Set (iii) = data with peakflows greater than 400 m^3/sec (16 events).
- Set (iv) = data with peakflows greater than 500 m^3/sec (12 events).

b) Four multiple regression equations were derived based on the above four sub-sets of data based on the LMS method. The standardized residuals of the response variable for each of the four regression equations were then calculated in order to identify the outliers.

c) Other statistical analyses were then performed, such as computation of:

- Correlation between predictors.
- Coefficient of determination.
- Variance inflation factors.

d) The best regression equation was selected, as indicated by the above statistical analyses and outliers analyses.

e) The selected regression equation was analyzed for the significance of each independent variable.

f) The final regression equation was selected such that only the significant predictors were included.

The above procedure to derive the multiple regression is also shown schematically in Figure 4.2.

4.2.5 Validation of the Multiple Regression Model

The multiple regression models were validated using the PRESS (Prediction sum of squares) statistic, which was confirmed by Myers (1990). From n rainfall-runoff events, n-1 events were first used to derive two multiple regression models based on the OLS and LMS methods. Each of the multiple regression model was then used to estimate the peakflow, the residual and the PRESS residual for that event, which was *not* used to obtain the regression coefficients. Then, another event was set aside and the remaining n-1 events were used to derive yet two other multiple regression models. Each of the OLS and LMS multiple regression models was then used to estimate the peakflow, the residual and PRESS residual for the data that was set aside. This procedure was repeated until all n candidate models based on the OLS and n candidate models based on the LMS were completed. Then PRESS statistic were calculated based on the OLS, and based on the LMS methods. The PRESS residual and the PRESS statistic were calculated using the following equations, as described by Myers (1990).

$$PRESS\ Residual = (Y_i - \hat{Y}_{i,-i}) \tag{4.10}$$

$$PRESS\ statistic = \sum_{i=1}^{n} (Y_i - \hat{Y}_{i,-i})^2 \tag{4.11}$$

where: Y_i = i^{th} observed peakflow,

$\hat{Y}_{i,-i}$ = estimated peakflow based on a regression model, which was derived with the i^{th} event set aside.

For the validation, the PRESS statistic calculated based on the OLS method was compared to the PRESS statistic calculated using the LMS method. The multiple regression model with the smallest PRESS statistic was considered to be the best model for a flood forecasting.

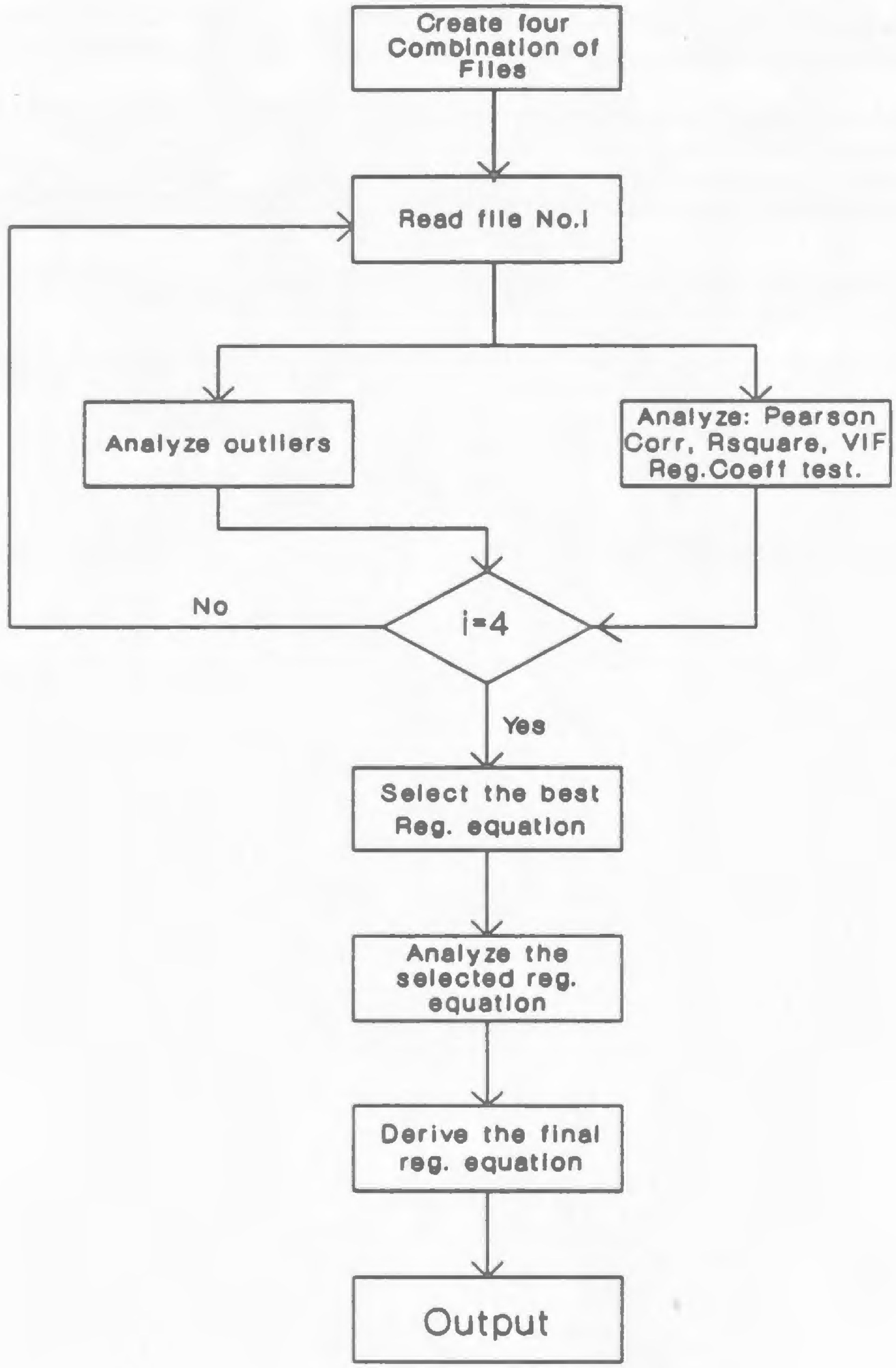


Figure 4.2 Flow-chart to calculate the Multiple Regression model.

4.3 The Transfer Hydrograph Approach

This section discusses the second method that was used to develop a flood forecasting model for the Cimanuk River. This method was based on the concept of a Transfer Hydrograph, which is a modification of the classical unit hydrograph. The Transfer Hydrograph for a sub-basin was a hydrograph at the outlet (Jatigede) caused by 1 mm total rainfall occuring in one hour at a given sub-basin, as shown in Figures 4.3. through 4.6.

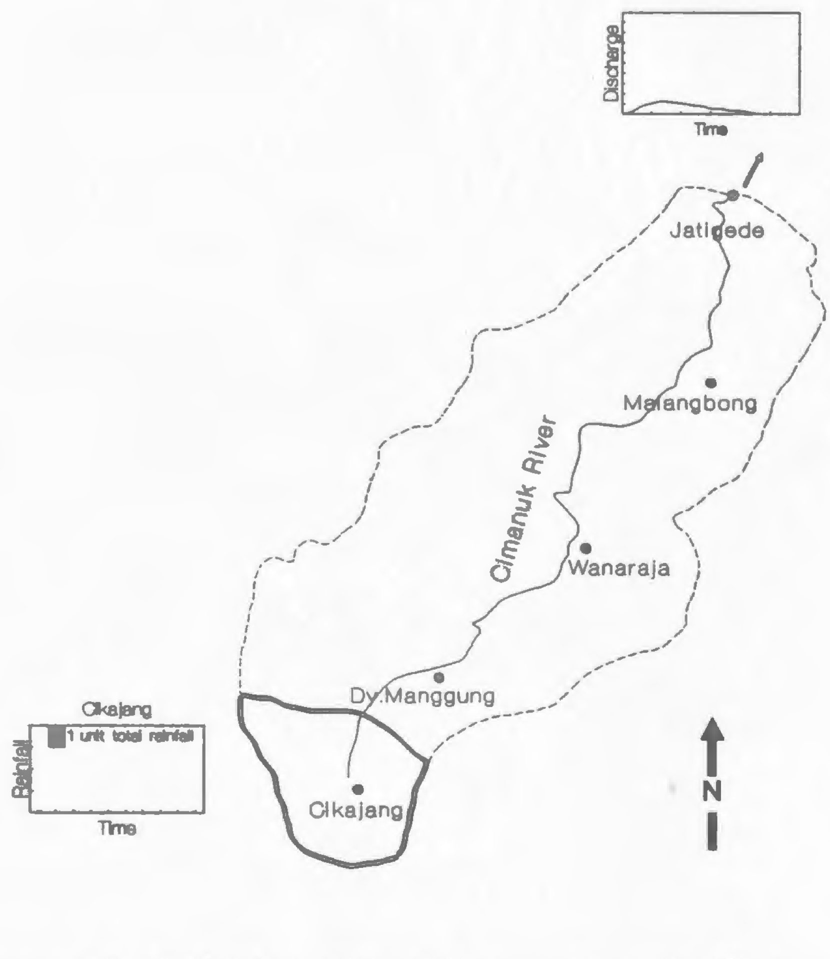


Figure 4.3 Definition of the Cikajang Transfer Hydrograph.

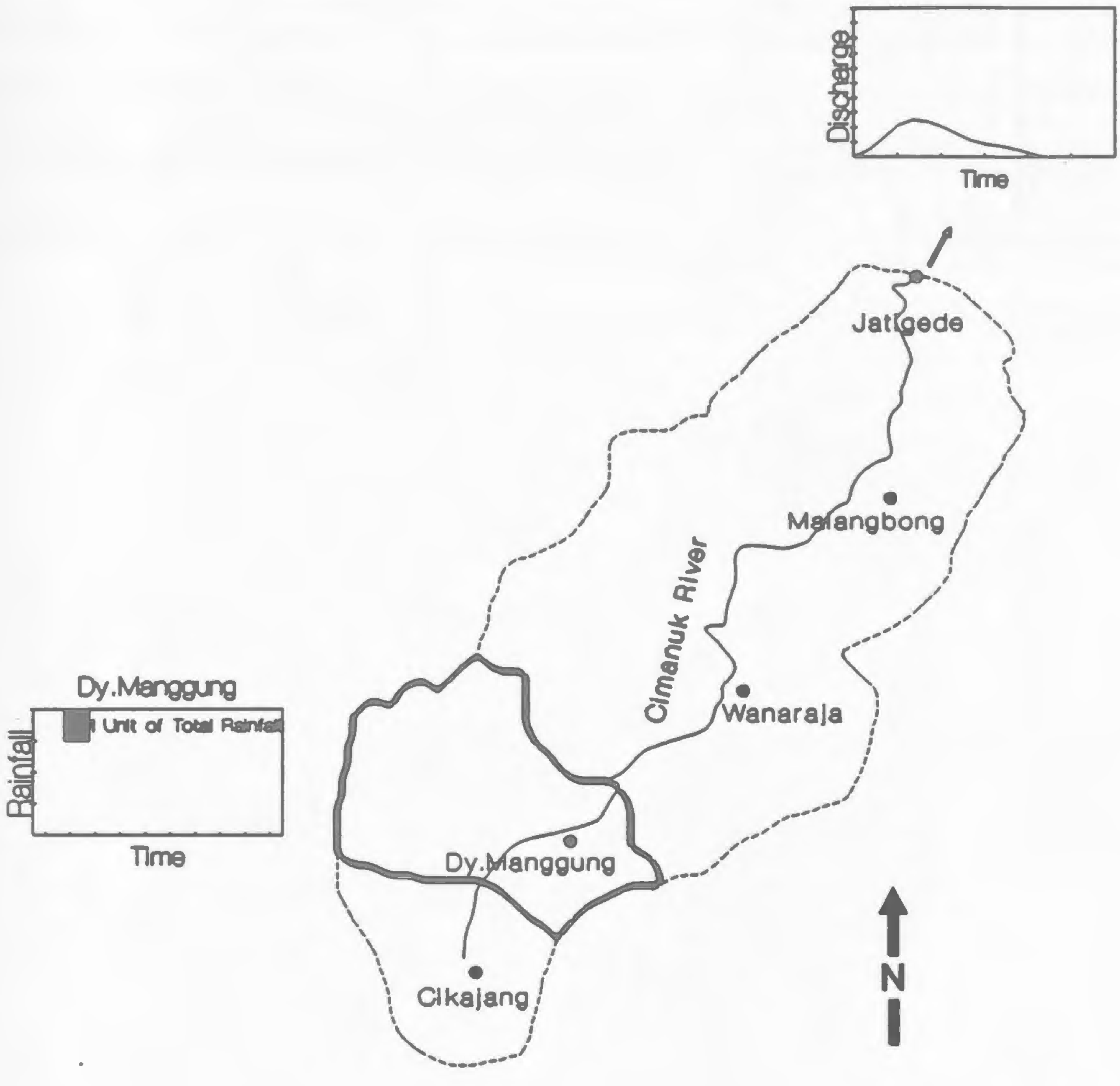


Figure 4.4 Definition of the Dy.Manggung Transfer Hydrograph.

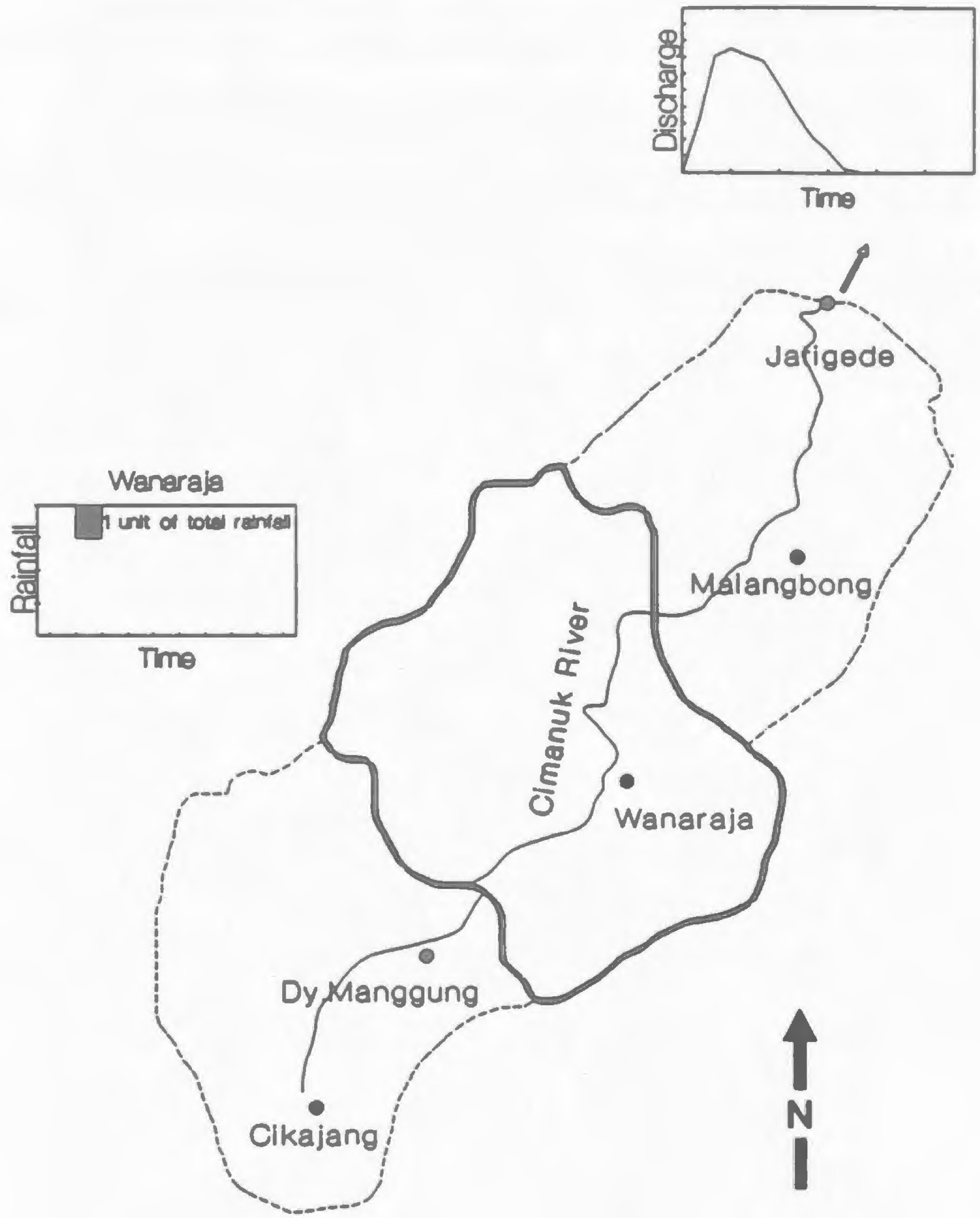


Figure 4.5 Definition of the Wanaraja Transfer Hydrograph.

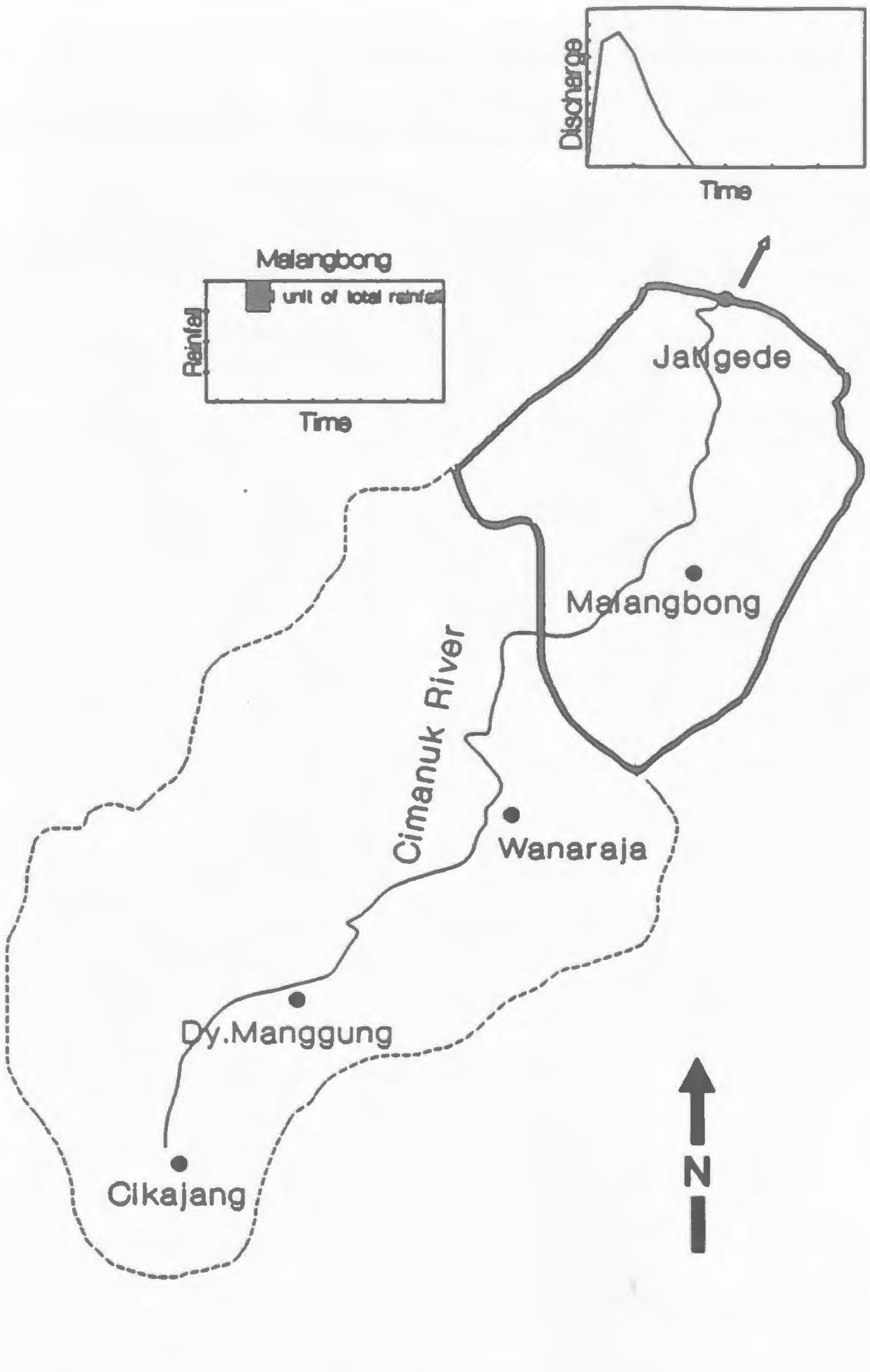


Figure 4.6 Definition of the Malangbong Transfer Hydrograph.

The normal procedure to estimate discharge at an outlet of a basin using the unit hydrograph approach is shown schematically in Figure 4.7. The estimation of discharge would begin from the uppermost sub-basin (the first sub-basin). From the first outlet A, the flow would be routed to the second outlet B. At the outlet B, the flow from the uppermost sub-basin would be added to the flow from the second sub-basin. Then from the outlet B, the flow would then be routed to outlet C. At this point the flow from outlet B would be added to the flow from the third sub-basin. Finally, the flow from outlet C would be routed to the outlet at Jatigede. The flow from outlet C would be added to the flow from the fourth sub-basin, to give the hydrograph at the last outlet D.

The procedure to calculate discharge at the last outlet (Jatigede) based on the classical UH approach described above could be readily carried out if the unit hydrograph for each sub-basin at outlets A, B, C and D had been determined based on the measurement of discharge at each of the outlets: A, B, C and D. However, in the Cimanuk River, only the discharge data at the outlet (Jatigede) was available (see Figure 4.8). For this reason the normal approach could not be used to estimate the flood discharge at the outlet, Jatigede. Consequently, the flood forecasting model for the Cimanuk River was developed based on a technique referred to herein as the Transfer Hydrograph. This technique was developed based on total rainfall and observed discharge at the outlet, Jatigede.

The differences between the classical unit hydrograph and the Transfer Hydrograph technique that was used in this thesis are as follows:

a) The classical unit hydrograph is derived based on *effective* rainfall, whereas the Transfer Hydrograph was derived based on *total* rainfall, as shown in Figure 4.9.

b) The Runoff depth as calculated based on the classical unit hydrograph is equal to 1 unit of depth (such as 1 cm), whereas the runoff depth calculated based on the Transfer Hydrograph will be less than 1 unit of depth. This is because of the losses or hydrologic abstractions in the basin. In the TH concept, the runoff depth was calculated using the following equation:

$$Runoff\ depth = \sum_{i=1}^{n} Q_i \, \Delta t \qquad (4.12)$$

where Q_i = i^{th} ordinate of the unit hydrograph or the transfer hydrograph,

Δt = time interval between ordinates of the hydrograph.

c) In the classical unit hydrograph, the outlet of each sub-basin is located at the end of each sub-basin. Under the Transfer Hydrograph method the outlet for each sub-basin was located at the basin (not sub-basin) outlet, as shown in Figure 4.10. Except for the above differences, all other unit hydrograph assumptions (such as linearity and superposition) were employed in the Transfer Hydrograph method.

In view of the similarity between the classical unit hydrograph and the Transfer Hydrograph approaches, the classical unit hydrograph will be discussed first.

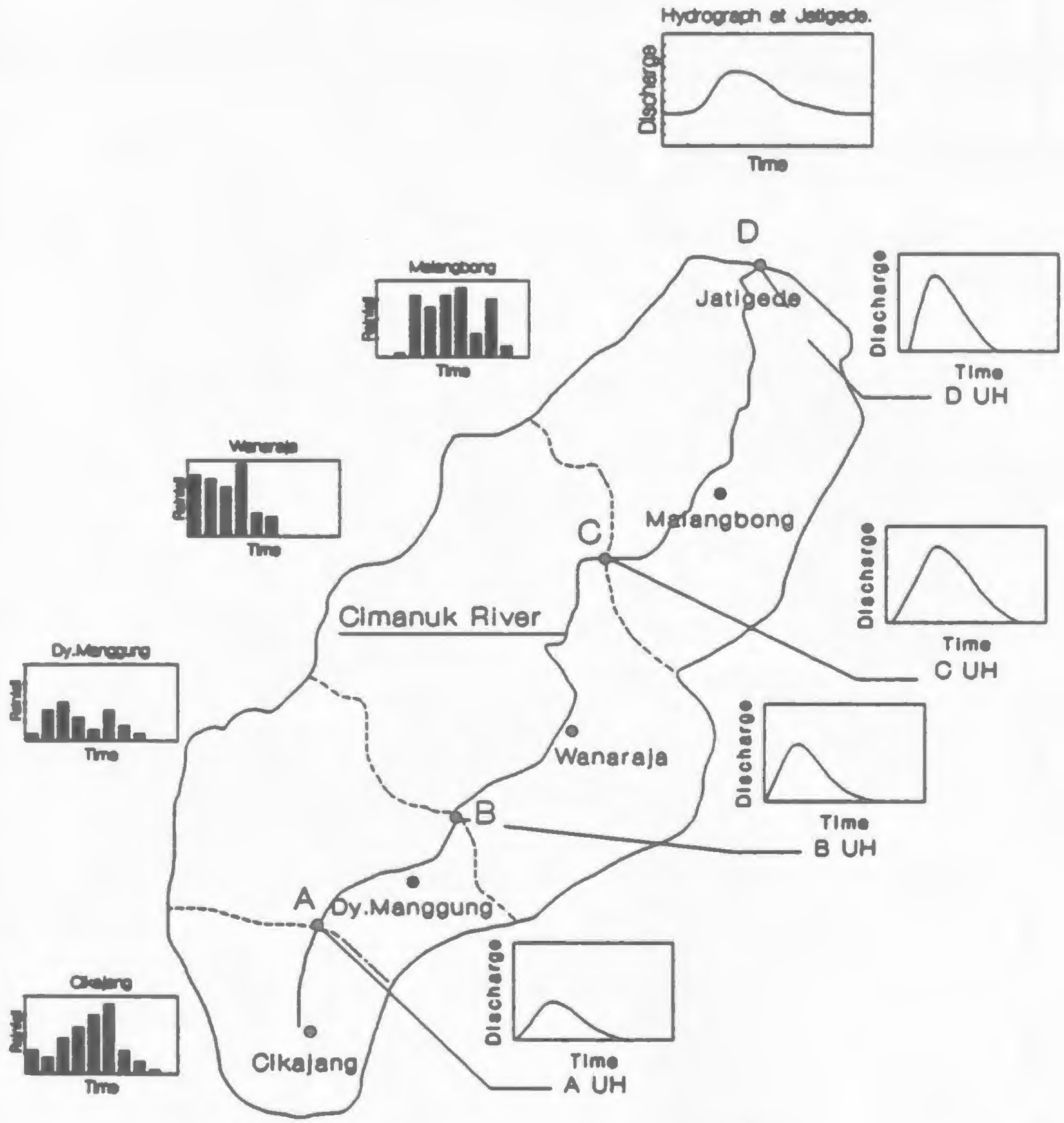


Figure 4.7 A more typical approach for estimation of flow at a basin outlet.

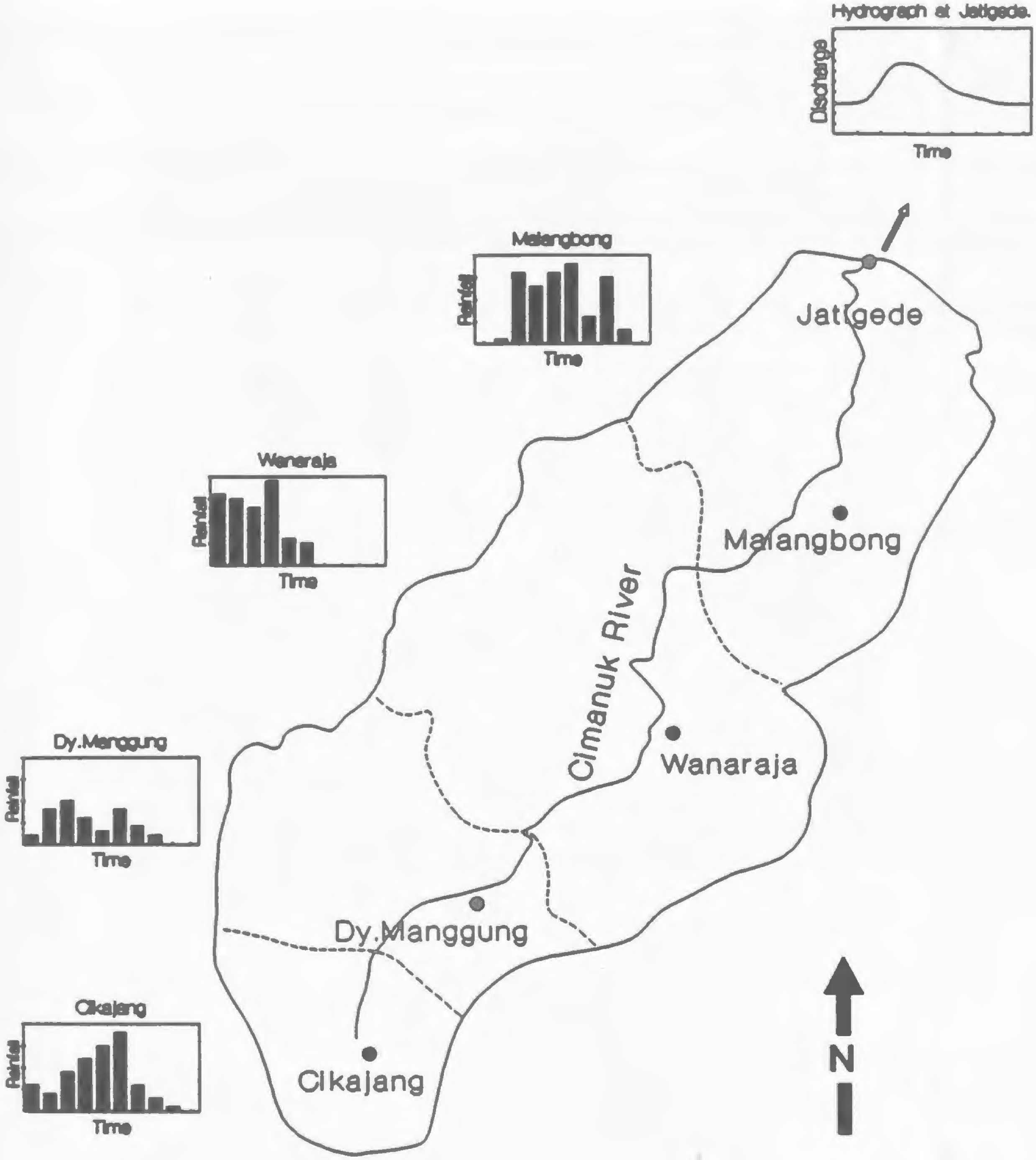
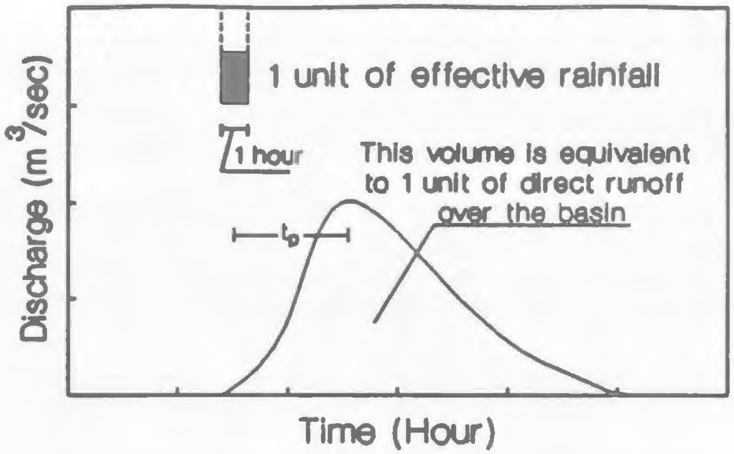
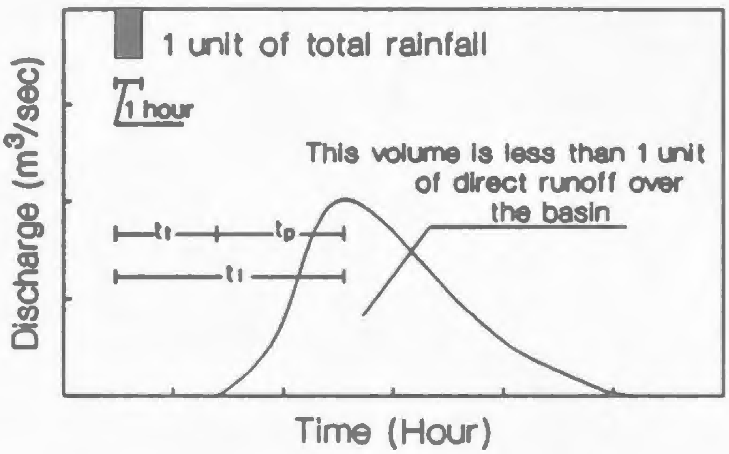


Figure 4.8 The available data in the Cimanauk River Basin.



The classical unit hydrograph



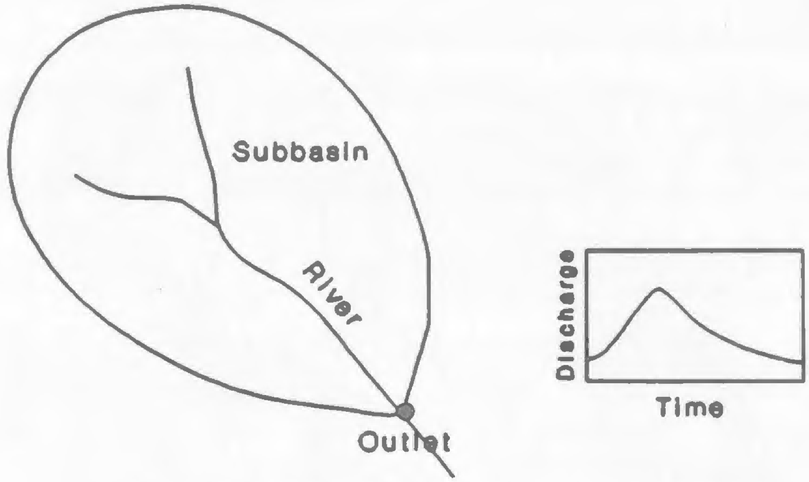
The transfer hydrograph

Where: t_t : time travel,

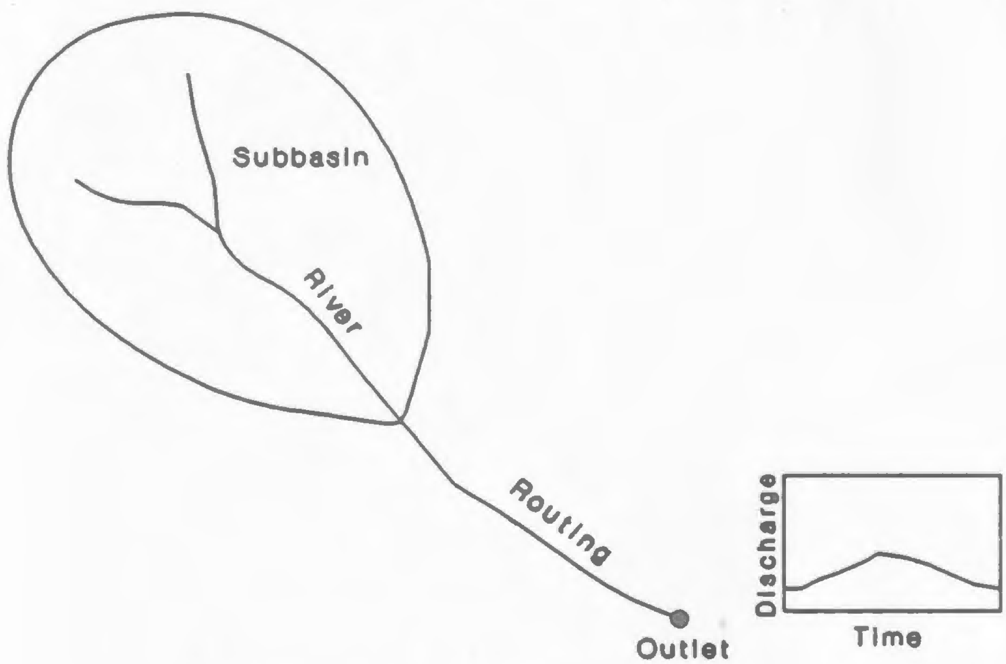
t_p : time-to-peak

t_l : time-lag

Figure 4.9 Different between the Classical UH and the TH, and the respective parameter definitions.



The Classical Unit Hydrograph



The Transfer Hydrograph

Figure 4.10 Different between the Classical UH outlet and the TH outlet.

4.3.1 The Classical Unit Hydrograph

The classical unit hydrograph method, which is a means of transforming rainfall into runoff, was briefly discussed in Chapter 3. The following are the assumptions used in deriving the classical unit hydrograph as discussed by Lye (1991).

a) Rainfall excesses with the same duration produce hydrographs with the same time bases, regardless of the rainfall intensity.

b) The ordinates of a direct runoff UH are proportional to the rainfall excess volumes.

c) The assumption of linear superposition is used when *applying* a classical unit hydrograph so as to obtain an actual *event* hydrograph. The procedure to obtain an event hydrograph is shown graphically in Figure 4.11.

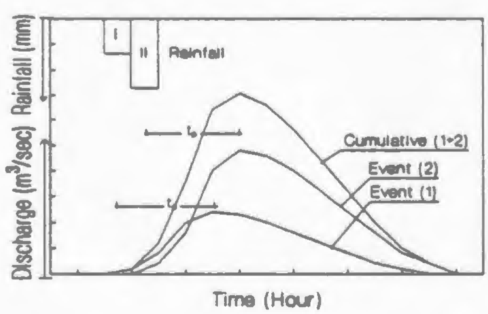


Figure 4.11 The linear superposition assumption.

d) Rainfall is distributed uniformly over the catchment-area for a given duration.

e) The time distribution of direct runoff is independent of the antecedent rainfall.

The following are the general steps used to develop a unit hydrograph for rainfall of a given duration.

a) The rainfall-runoff event is selected.

Storm should be selected so that the rainfall produces 12 mm to 50 mm of runoff depth (Lye 1991) and the rainfall duration should be approximately 10-30 % of the timelag (Viessman 1989). Bedient et al. (1988) suggested that the rainfall duration should be 25 to 30 % of the timelag.

b) The baseflow is separated.

There are many methods of separating direct runoff from the baseflow. In this study only three methods were considered and these are illustrated in Figure 4.12. The first method assumes that the baseflow decreases until the hydrograph reaches a peakflow; then the baseflow increases until the end of the runoff. The second method assumes that if the ground water affects runoff, the baseflow will decrease until the point of the peakflow. Then the baseflow will increase until the inflection point of the recessing curve and finally it will decrease until the end of the runoff. In the third method the baseflow is assumed straight line from the start to end point. A more realistic method to separate the baseflow is to consider the effect of ground water, infiltration/exfiltration so that the baseflow forms a curve (Method 2), as shown in Figure 4.12. However, this procedure for drawing the baseflow curve is quite subjective in the absence of detailed hydrologic modelling. The straight line method is simpler, gives consistent results, and was therefore used in this research.

c) The runoff depth (h) is calculated. This is based on the total volume of direct runoff; that is, the area of the direct runoff hydrograph determined according to:

$$Runoff\ Volume = \sum_{i=1}^{n} Y_i \, \Delta t \tag{4.13}$$

where: Y_i = ordinate of hydrograph,

Δt = interval of the ordinates.

$$h = \frac{runoff\ volume}{basin\ area} \tag{4.14}$$

d) The ordinates of the hydrograph are divided by the depth of runoff (h) to obtain the unit hydrograph for a given duration of rainfall excess.

e) The resulting volume under the unit hydrograph should be 1 unit (such as 1 cm) of direct runoff depth.

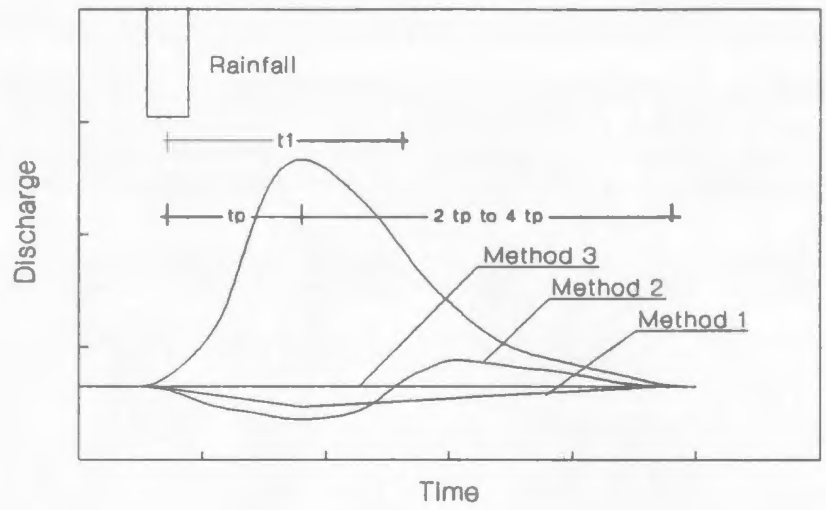


Figure 4.12 Base-flow separation.

In practice, it is commonly found that the ordinates of a unit hydrograph are not exactly proportional to the volume of water, as discussed by Linsley et al. (1982). Furthermore, it is very often found that different rainfall events, which have the same intensity and duration, produce somewhat different ordinates of the associated unit hydrographs. In deriving a unit hydrograph based on rainfall-runoff events, different *shapes* of derived unit hydrographs will often be found. The procedure to obtain a single unit hydrograph from several derived unit hydrographs of similar duration according to Linsley et al.(1982) can be summarized as follows:

a) The time to peak of the final unit hydrograph (t_p) can be calculated as the average of the several time to peaks of the derived unit hydrographs.

$$t_p = \frac{\sum_{i=1}^{n} t_{pi}}{n} \qquad (4.15)$$

where: t_{pi} = i^{th} time to peak,

n = number of the derived unit hydrographs.

b) The peak (Q_p) of the final unit hydrograph can be taken as average of the several peaks of the derived unit hydrographs.

$$Q_p = \frac{\sum_{i=1}^{n} Q_{pi}}{n} \qquad (4.16)$$

where: Q_{pi} = i^{th} peak of the derived unit hydrograph.

c) The final unit hydrograph can be sketched based on the average time to peak t_p, and the average peak-flow Q_p, such that the result corresponds to 1 unit of direct runoff (see Figure 4.13).

Many analysts increase the peakflow ordinate by 5 to 20 %, before using the unit hydrograph to estimate a peak-flow because it has been found that the bigger the flood discharge, the shorter the timelag (Linsley et al. 1982).

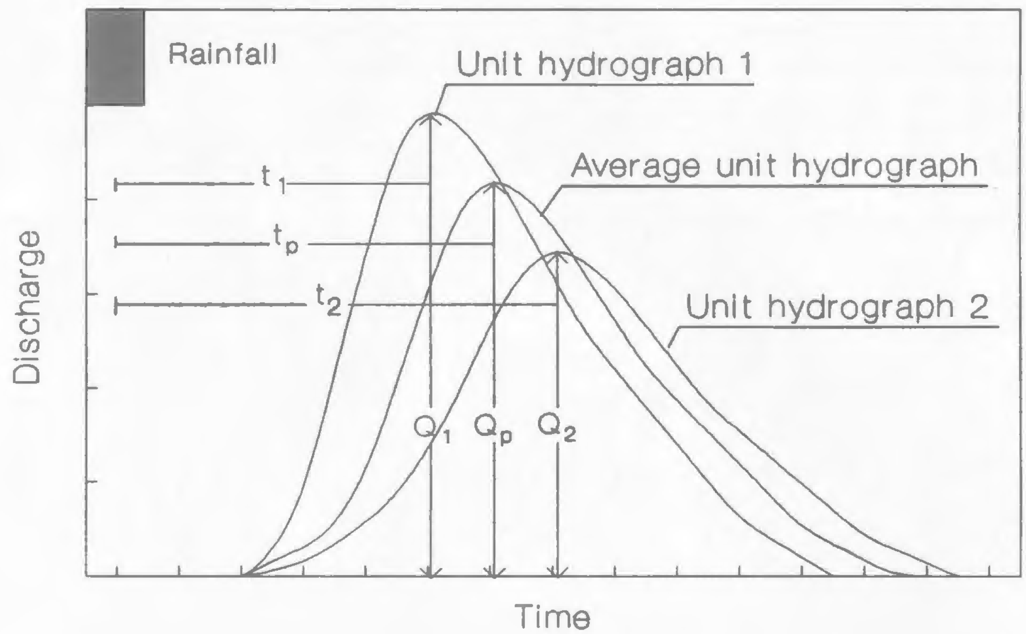


Figure 4.13 Average unit hydrograph.

In 1960, Minshall studied the unit hydrograph method and stated that there were good relationships between:

a) The rainfall intensity and the peak of the unit hydrograph.

b) The rainfall intensity and the time from the beginning of the effective rainfall.

In spite of the drawbacks of the unit hydrograph procedure, many engineers support the use of unit hydrograph in practice because the method is simple, easy to understand, and produces reasonable results.

4.3.2 The Transfer Hydrograph.

This section discusses the derivation of the four Transfer Hydrographs for the flood forecasting model of the Cimanuk River. The four Transfer Hydrographs are: the Cikajang sub-basin TH, the Dy.Manggung sub-basin TH, the Malangbong sub-basin TH, and the Wanaraja sub-basin TH. The data requirements, assumptions, and procedures used to develop these four Transfer Hydrographs will be discussed in the following subsections.

4.3.2.1 Data requirements.

Four categories of rainfall-runoff data were used in developing and validation of each TH. These categories can be distinguished from each other based on the rainfalls occurrences, as was presented in Table 4.1 and Table 5.19.

a) Rainfall that occurred in one sub-basin only was called a type A event and was used to derive the Cikajang TH.

b) Rainfall that occurred in two sub-basins were called type B events. The type B events were used to derive the Wanaraja TH.

c) Rainfall that occurred in three sub-basins were called type C1 events or type C2 events. Type C1 events were used to derive the Dy.Manggung TH, and type C2 events were used to derive the Malangbong TH.

d) Rainfall occuring at all four sub-basins were called type D events. The type D events were used in validating of the Transfer Hydrographs.

It would be generally desirable to develop a Transfer Hydrograph based on rainfall occuring in an individual sub-basin (such as event that can be categorized as type A event). However, Type B, Type C1, and Type C2 events were also used in *developing* the Cimanuk Transfer Hydrographs because of the unavailability of sub-basin runoff data. The Transfer Hydrographs were *validated* using Type D events.

The direct runoff hydrograph that were used to derive the sub-basin Transfer Hydrographs were separated from baseflow based on the straight line method (Method 3), as shown in Figure 4.12.

4.3.2.2 Assumptions.

The following assumptions were made in developing the sub-basin Transfer Hydrographs for the Cimanuk River.

a) The Cimanuk River Basin was divided into four sub-basins, as can be seen in Figure 2.1.
b) The conceptual model of the rainfall-runoff relationship that was used for the Cimanuk River Basin is shown in Figure 4.14. It is important to note that all four of the sub-basins have their outlets at the same point, Jatigede, where the discharge was observed.

c) The rainfall intensity was considered to be uniformly distributed in the same duration over a given sub-basin.

d) The flow coming from a sub-basin was assumed not to have any effect on the flow coming from other sub-basins.

e) The baseflow was determined based on the straight line method.

f) An averaging method, as was illustrated in Figure 4.13, was used to determine the final Transfer Hydrograph for a given sub-basin.

4.3.2.3 Procedure.

The Transfer Hydrographs were derived using the least squares method, as recommended by Singh (1988). The rainfall and runoff were expressed in matrix form and the least squares method was used to derive the Transfer Hydrographs. The general equation to derive the Transfer Hydrograph can be expressed as:

$$[Q]=[I][U] \tag{4.17}$$

wher: [Q] = vector of storm hydrograph ordinates.

[I] = matrix of rainfall.

[U] = vector of the TH ordinates.

The ordinates of the TH [U] were computed using:

$$[U]=[I^T\ I]^{-1}\ [I^T]\ [Q] \tag{4.18}$$

Where: $[I^T]$ = transpose matrix of matrix [I],

$[I^T\ I]^{-1}$ = inverse matrix of $[I^T\ I]$.

The relationship between the rainfall duration (i), the number of Transfer Hydrograph ordinates (j), and the number of hydrograph ordinates (n) is:

$$n = j + i - 1 \tag{4.19}$$

The Transfer Hydrograph for each sub-basin was calculated according to the type of event, as shown in Table 4.1. The derivation of the Transfer Hydrograph can be outlined in the following steps:

a) The 1 hour TH for the first sub-basin (Cikajang) was derived using the rainfall-runoff data corresponding to a type A event, as shown in Table 4.1. The procedure to calculate the TH for the Cikajang sub-basin is described in Appendix A1, and a listing of the computer program is given in Appendix B1. This 1 hour TH was calculated using the equation:

$$U_c = [I_c^T\ I_c]^{-1}\ [I_c^T]\ [Q_t] \tag{4.20}$$

where: U_c = the Cikajang transfer hydrograph,

I_c = rainfall matrix for the Cikajang sub-basin,

I_c^T = transpose matrix of the I_c matrix,

Q_t = vector of observed discharge.

b) The 1 hour TH for the second sub-basin (Wanaraja) was derived using the rainfall-runoff data corresponding to type B events, as were shown in Table 4.1. The procedure to calculate the TH for the Wanaraja sub-basin is described in Appendix A2, and a listing of the computer program is given in Appendix B2. This 1 hour TH was calculated using the equation:

$$Q_w = Q_t - [I_c]\,[U_c]_{tc} \qquad (4.21)$$

$$U_w = [I_w^T\, I_w]^{-1}\,[I_w^T]\,[Q_w] \qquad (4.22)$$

where: U_w = the Wanaraja transfer hydrograph,

I_w = rainfall matrix for the Wanaraja sub-basin,

I_w^T = transpose matrix of the I_w matrix,

Q_w = vector of Wanaraja discharge,

t_c = timelag of the flow from Cikajang sub-basin.

c) For the third sub-basin (Dy.Manggung), the TH was calculated using rainfall-runoff data corresponding to type C1 events, as shown in Table 4.1. The procedure to calculate the TH for the Dy.Manggung is shown in Appendix A3, and a listing of the computer program is given in Appendix B3. This 1 hour TH was calculated using the equation:

$$Q_d = Q_t - [I_c][U_c]_{tc} - [I_w][U_w]_{tw} \tag{4.23}$$

$$U_d = [I_d^T I_d]^{-1}[I_d^T][Q_d] \tag{4.24}$$

where: U_d = the Dy.Manggung transfer hydrograph,

I_d = rainfall matrix for the Dy.Manggung sub-basin,

I_d^T = transpose matrix of the I_d matrix,

Q_d = vector of Dy.Manggung discharge,

t_c = timelag of the flow from Cikajang sub-basin,

t_w = timelag of the flow from Wanaraja sub-basin.

d) For the fourth sub-basin (Malangbong), the TH was calculated using rainfall-runoff data corresponding to type C2 events, as shown in Table 4.1. The procedure to calculate the TH for the Malangbong sub-basin is shown in Appendix A3, and a listing of the computer program is given in Appendix B3.

The Malangbong Transfer Hydrograph was calculated, based on the following equations, depending on the availability of data. The equations were given by:

$$Q_m = Q_t - [I_c][U_c]_{tc} - [I_w][U_w]_{tw} \tag{4.25}$$

$$Q_m = Q_t - [I_c][U_c]_{tc} - [I_d][U_d]_{td} \tag{4.26}$$

$$Q_m = Q_t - [I_w][U_w]_{tw} - [I_d][U_d]_{td} \tag{4.27}$$

$$U_m = [I_m^T I_m]^{-1}[I_m^T][Q_m] \tag{4.28}$$

where: U_m = the Malangbong transfer hydrograph,

I_m = rainfall matrix for the Malangbong sub-basin,

I_m^T = transpose matrix of the I_m matrix,

Q_m = vector of Malangbong discharge,

t_c = timelag of the flow from Cikajang sub-basin.

t_w = timelag of the flow from Wanaraja sub-basin.

t_d = timelag of the flow from Dy.Manggung sub-basin.

Table 4.1 Classification of rainfall events to derive the 1 hour Transfer Hydrographs

Event No	Date	Rainfall at sub-basin				Type of Event
		Ckj	Dym	Wnr	Mlb	
1	January, 23 1987	Yes	No	No	No	A
2	November, 14 1981	Yes	No	No	Yes	B
3	October, 22 1980	Yes	No	No	Yes	B
4	March, 26 1989	Yes	Yes	No	Yes	C1
5	October, 04 1979	Yes	Yes	Yes	No	C2
6	March, 10 1988	Yes	No	No	Yes	B
7	January, 21 1985	Yes	No	Yes	Yes	C2
8	Macrh, 22 1988	Yes	Yes	No	Yes	C1
9	October, 02 1981	Yes	Yes	Yes	No	C2
10	March, 03 1984	Yes	Yes	No	Yes	C1
11	December, 29 1982	Yes	Yes	No	Yes	C1
12	December, 06 1984	Yes	Yes	No	Yes	C1
13	March, 24 1982	No	Yes	Yes	Yes	C2
14	April, 10 1982	Yes	No	No	Yes	B
15	November, 17 1983	Yes	No	No	Yes	B
16	October, 05 1978	Yes	No	Yes	Yes	C2

The above data were used for constructing the 1 hour Transfer Hydrograph for each of the four sub-basins.

4.3.3 Validation

The magnitude of peakflow and the timelags of the Transfer Hydrographs were validated based on data Type D events, as shown in Table 5.19. The validation of these two components for each of the four Transfer Hydropgraphs were carried out by calculating the hydrograph at the outlet, Jatigede, based solely on rainfall data, and then by comparing the convoluted hydrograph with the observed hydrograph for the same event.

The procedure to calculate the hydrograph at the outlet is shown in Figure 4.14, and a listing of the computer program is given in the Appendix B4.

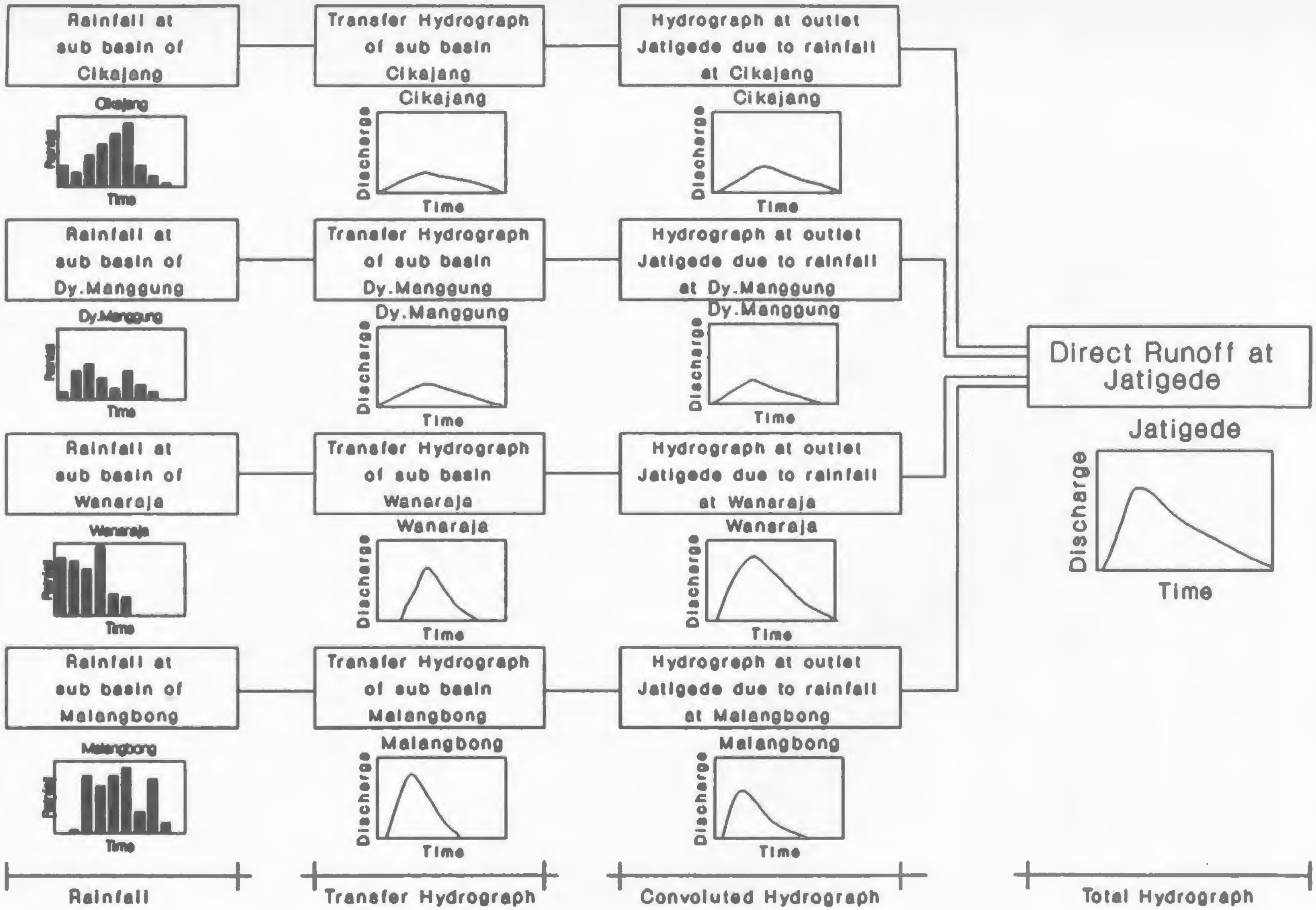


Figure 4.14 The model to transfer rainfalls on the four subbasins of the Cimanuk River to be runoff at outlet Jatigede.

CHAPTER 5

RESULTS AND DISCUSSIONS

This chapter discusses flood forecasting models developed for the Cimanuk River based on the Multiple Regression and the Transfer Hydrograph techniques described in Chapter 4. The results were divided into two parts: the first part is the results of development of the multiple regression model and the second part is the results of the development of the Transfer Hydrograph method. The two methods were then validated and compared to each other.

5.1. Multiple Regression Analysis.

This section discusses the development of a model for flood forecasting based on multiple regression approach. Two parameters should be estimated in flood forecasting, the magnitude of peak flow and the time to peak. The magnitude of the peak flow is estimated based on the multiple regression model, which describes the relationship between the total rainfalls at each of the four subbasins and the peak-flow at the outlet, Jatigede. Various combinations of rainfall-runoff data were studied to develop a model for estimation of the time to peak flow, however no satisfactory result was obtained. Therefore only the peak flow was estimated using the multiple regression approach.

Since direct runoffs were used, the multiple regression was derived without a constant term. In order to find a reasonable regression model, the rainfall-runoff events were analyzed in different combinations as follows:

Case a = All available data (32 events) were used in developing the Multiple Regression Model,

Case b = Regression model based on 22 events with peak flows greater than 300 m^3/s.

Case c = Regression model based on 16 events with peak flows greater than 400 m^3/s.

Case d = Regression model based on 12 events with peak flows greater than 500 m^3/s.

Table 5.1 Rainfall-runoff data used in the development of the multiple regression model.

Event	Peak flow (m^3/sec)	Total Rainfall at individual subbasin (mm)			
		Cikajang	Dy.Manggung	Wanaraja	Malangbong
1	95.6	9.5	7.8	4.2	5.0
2	193.1	25.3	15.1	11.0	0.4
3	197.2	2.4	2.5	9.2	8.3
4	212.1	92.0	0.0	0.9	1.2
5	226.8	26.7	0.4	18.4	0.5
6	251.6	28.1	0.2	16.5	1.0
7	261.1	8.3	17.8	11.5	17.5
8	275.5	11.5	37.8	9.5	4.5
9	284.6	21.5	32.7	8.3	24.3
10	290.7	5.2	4.6	1.3	12.1
11	300.4	13.3	23.2	12.9	14.6
12	334.7	34.3	33.3	17.9	17.3
13	339.4	40.9	62.3	17.7	3.3
14	354.1	39.0	40.8	11.2	0.0
15	356.3	44.0	26.0	3.0	36.4
16	357.8	4.9	46.8	0.0	33.3
17	453.6	29.1	0.5	31.3	0.0
18	459.9	13.6	0.0	4.7	36.6
19	461.5	23.3	69.6	15.3	20.0
20	485.4	37.9	55.1	15.8	0.0
21	509.3	22.8	12.7	0.0	48.0
22	542.3	30.9	60.4	24.6	0.0
23	599.4	14.2	58.7	23.0	0.0
24	632.8	37.0	96.9	15.8	0.0
25	689.3	0.7	4.0	24.9	40.5
26	758.9	5.1	0.0	56.9	2.3
27	764.4	71.4	44.6	19.1	38.9
28	787.9	8.9	3.0	55.3	0.0
29	803.2	11.2	42.1	3.8	47.4
30	824.3	62.8	94.2	42.3	2.3
31	831.2	31.5	13.5	10.6	50.1
32	927.4	10.6	0.6	3.1	73.4

5.1.1 Analysis of the regression model

This subsection presents the results of the regression analyses for each of the four cases listed above. The multiple regression model was selected based on five statistical and physical criteria. First, the regression model must be free of outliers. Second, the coefficient of correlations between the peak flows and total rainfalls at each of the subbasins should be positive. Third, the coefficient of determinations of the regression must be statistically significant at the 5 % level. Fourthly, the Variance Inflation Factor (VIF) must be less than 10, and finally the regression coefficients must be positive and statistically significant at the 5 % level.

Table 5.2 Results of the statistical analyses toward refinement of the regression model.

No	Number of events	Outliers	Corr. coefficients between the peak flows and the total rainfalls at				Coefficient of determinations
			Ckj	Dym	Wnr	Mlb	
1	32	Yes	0.06	0.26	0.32	0.24	0.963
2	22	Yes	-0.05	-0.11	0.35	0.33	0.971
3	16	Yes	0.02	-0.10	0.20	0.39	0.984
4	12	No	0.04	-0.26	0.12	0.39	0.992

Note: Column 4 describes the correlation between the peak flows and the total rainfall at individual sub-basin. None of these correlations were significant at a 5 % level of significant.

Table 5.3 Regression coefficients corresponding to the four categories of events.

Cases	Items	Sub-basins			
		Cikajang	Dy.Manggung	Wanaraja	Malangbong
a	Coeff.	1.371	2.442	13.043	7.446
	t-value	1.885	4.163	13.848	8.05
	Note	Ns	S	S	S
b	Coeff.	-0.531	3.143	13.494	10.523
	t-value	-0.345	3.352	10.470	11.433
	Note	Ns	S	S	S
c	Coeff.	-1.314	4.481	13.277	11.488
	t-value	-1.011	5.729	14.704	16.625
	Note	Ns	S	S	S
d	Coeff.	-2.222	4.841	13.107	11.809
	t-value	-1.521	5.513	13.871	15.748
	Note	Ns	S	S	S

Notes: Coeff. = coefficient.

Ns = not significant.

S = significant.

Case a = all 32 events.

Case b = 22 events (those with Q_p > 300 m^3/s).

Case c = 16 events (those with Q_p > 400 m^3/s).

Case d = 12 events (those with Q_p > 500 m^3/s).

$t_{8,0.05}$ = 2.306.

$t_{12,0.05}$ = 2.179.

$t_{18,0.05}$ = 2.101.

$t_{28,0.05}$ = 2.048.

Based on cases a, b, c, and d, the regression coefficients for the Cikajang rainfalls were less than $t_{n-p,\alpha}$ and were not considered significant at the 5 % level.

Table 5.4 The Variance Inflation Factors corresponding to the four categories of events.

Case	Number of events	Variance Inflation Factor			
		Cikajang	Dy.Manggung	Wanaraja	Malangbong
a	32	1.1	1.1	1.2	1.3
b	22	1.4	2.0	1.9	2.4
c	16	1.6	2.4	2.5	3.1
d	12	2.4	5.7	5.1	7.7

Based on the consideration of the outliers, correlation coefficients, coefficient of determination and the variance inflation factor as shown in Tables 5.2 through 5.4, it was found that the regression model that was derived using the twelve rainfall-runoff events was satisfactory.

5.1.2 Study of the Selected Model.

The multiple regression equation that was developed based on the twelve events as shown in Table 5.4 through Table 5.6 was considered to be satisfactory in comparison to the other three cases. The outliers analysis in Table 5.7 shows that all standard residuals are less than 2.5, which is the critical point for outliers as discussed by Rousseeuw (1987). The coefficient of correlation between peak flow and the total rainfall at individual sub-basins (Table 5.5) showed that the correlation coefficient for Dy.Manggung was negative (-0.26), however the correlation coefficient was statistically not different from zero at the 5 % level. Variance Inflation Factors for the four independent variables were less than 10 (Myers 1990) as shown in Table 5.5. It can be concluded that the multiple regression model, which was derived using the four independent variables, were not contaminated by multicollinearity.

Table 5.5 Variance Inflation Factor and correlation between the peak flow and total rainfalls at each of the four sub-basins.

Rainfall at	VIF	Correlation between peak flow and rainfall
Cikajang	2.4	0.04
Dy.Manggung	5.7	-0.26
Wanaraja	5.1	0.12
Malangbong	7.7	0.39

Table 5.6 Results of hypothesis testing of the regression coefficients computed using the Ordinary Least Squares method (at $\alpha = 0.05$).

Variable	Coefficient	Stand. error	T-value	P-value	Note
Ckj	-2.222	1.461	-1.52	0.17	Ns
Dym	4.841	0.878	5.51	0.00	S
Wnr	13.107	0.945	13.87	0.00	S
Mlb	11.809	0.750	15.75	0.00	S

Coefficient of determination (R^2) = 0.992

The F-Value = 5.116 (with 4 and 7 DF) P-Value = 0.03016

Table 5.7 Results of residual analyses based on the Least Median of Squares.

No.	Observed peak flows m^3/sec	Estimated peak flows m^3/sec	Residuals m^3/sec	Res/Sc
1	2	3	4=2-3	5
1	509.3	509.30	0	0
2	542.3	542.30	0	0
3	599.4	631.85	-32.45	-0.24
4	632.8	606.99	25.80	0.19
5	689.3	899.53	-210.23	-1.54
6	758.9	849.03	-90.13	-0.66
7	764.4	533.33	231.06	1.69
8	787.9	787.90	0	0
9	803.2	841.91	-38.71	-0.28
10	824.3	824.30	0	0
11	831.2	635.51	195.68	1.43
12	927.4	881.86	45.53	0.33

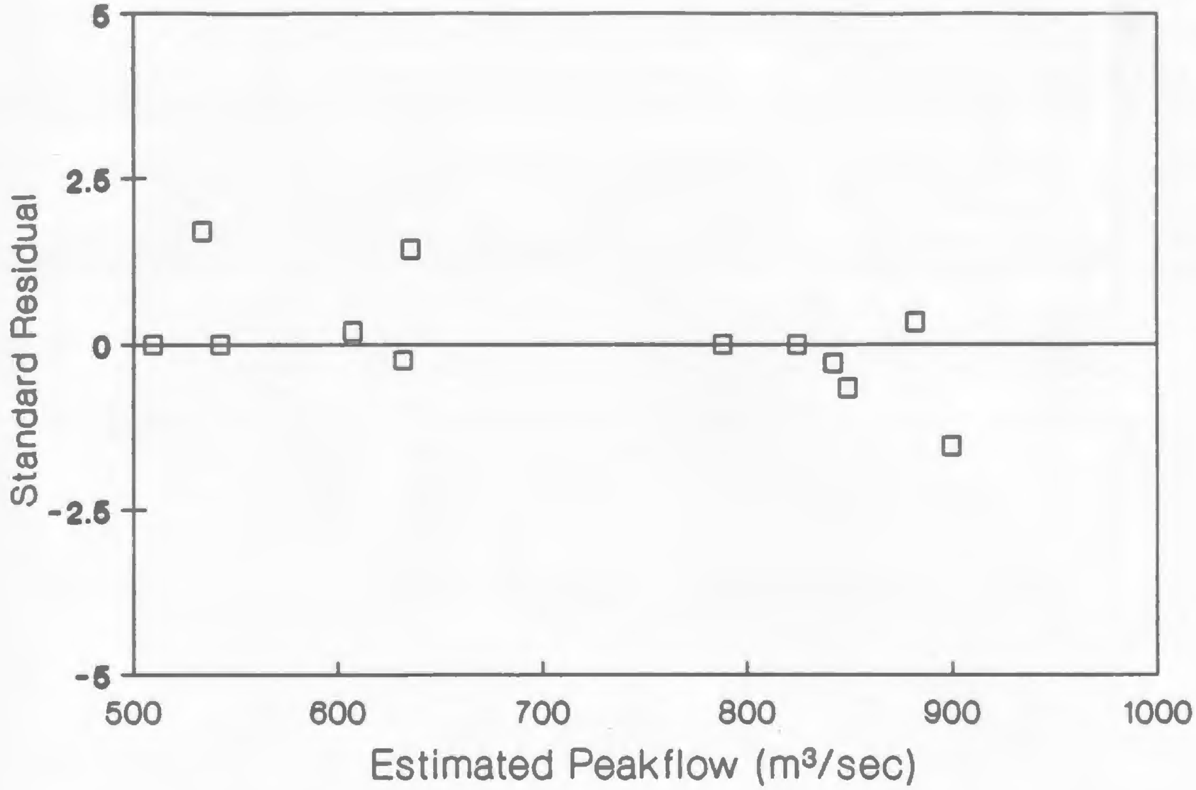


Figure 5.1. Plot of standardized residual versus estimated peakflow.

The statistical analysis for the twelve events showed that:

1) Based on LMS analysis, there were no outliers in the residual plot.

2) Based on OLS analysis, the coefficient of determination (R^2) was 0.992.

3) Based on the OLS analysis the regression coefficients are shown in Table 5.5, and it was found that the regression coefficients for Cikajang were not statistically significant, and this variable was therefore dropped from the regression model. The final regression model was developed with only three

predictors the total rainfall at Dy.Manggung, Malangbong and Wanaraja as presented in the following section.

5.1.3 The regression model with three independent variables.

From the previous section, it was found that the regression coefficient for the Cikajang was not statistically significant. This section discusses the multiple regression analysis with three variables using data as shown in Table. 5.8. The results of the study of the multiple regression using three variables are shown in Table 5.9 through Table 5.16. The regression model is as follows:

$$Q_{peakflow} = C_1 TR_d + C_2 TR_w + C_3 TR_m \tag{5.1}$$

where: Q = estimated peak flow,

TR_d = total rainfall at the Dy.Manggung sub-basin,

TR_w = total rainfall at the Wanaraja sub-basin,

TR_m = total rainfall at the Malangbong sub-basin.

Table 5.8 Data used in developing the multiple regression model with three independent variables.

No.	Total rainfall (mm)			Peak flow (m^3/sec)
	Dy.Manggung	Wanaraja	Malangbong	
1	12.7	0	48.0	509.3
2	60.4	24.6	0	542.3
3	58.7	23.0	0	599.4
4	96.9	15.8	0	632.8
5	4.0	24.9	40.5	689.3
6	0	56.9	2.3	758.9
7	44.6	19.1	38.9	764.4
8	3.0	55.3	0	787.9
9	42.1	3.8	47.4	803.2
10	94.2	42.3	2.3	824.3
11	13.5	10.6	50.1	831.2
12	0.6	3.1	73.4	927.4

Table 5.9 The standardized observations for the multiple regression model with three predictors.

No.	Standardized observations of the predictors and the dependent variable			
	Dy.Manggung	Wanaraja	Malangbong	Peak flow
1	0.3081	0	1.5716	0.4510
2	1.4654	0.7882	0	0.4802
3	1.4242	0.7370	0	0.5308
4	2.3510	0.5063	0	0.5604
5	0.0970	0.7979	1.3261	0.6104
6	0	1.8232	0.0753	0.6721
7	1.0821	0.6120	1.2737	0.6769
8	0.0728	1.7719	0	0.6977
9	1.0214	0.1218	1.5520	0.7113
10	2.2855	1.3554	0.0753	0.7300
11	0.3275	0.3396	1.6404	0.7361
12	0.0146	0.0993	2.4033	0.8213

All standardized observations in Table 5.9 were less than 2.5. This indicated that leverage points, which represent outliers in the predictor variables, were not found in the data.

Table 5.10. Variance inflation factors and coefficient correlation between peak flow and total rainfall at each sub-basin.

Independent variable	VIF	Peak flow
Dym	2.6	-0.26
Wnr	4.0	0.12
Mlb	5.4	0.39

Table 5.10 showed that the Variance Inflation Factors were less than 10. That indicated that the data were not contaminated by multicollinearity. It can be seen also that the correlation between peak flows and the total rainfalls at each sub-basin was not statistically significant. Negative correlation was found in the correlation between the peak flows and the total rainfalls at the Dy.Manggung sub-basin. However the squared correlation coefficient was less than 0.1 and was statistically not significant.

Table 5.11 Estimated regression coefficients based on Ordinary Least Squares.

Variable	Coefficient	Stand. error	T-value	P-value
Dym	3.81	0.593	6.412	0.00
Wnr	12.77	0.983	12.986	0.00
Mlb	11.24	0.697	16.120	0.00

The statistical test of the regression coefficients showed that they were significantly different from zero.

Table 5.12 showed that there were not outliers in the regression residuals.

Coefficient of determination (R Squared) = 0.99

The F-value = 302.765 (With 3 and 9 DF) P-Value = 0.00

The Least Squares regression model is therefore:

$$Q_{peakflow} = 3.81\ TR_d + 12.77\ TR_w + 11.24\ TR_m \qquad (5.2)$$

Scale estimate = 83.82

Table 5.12 Regression residuals based on Ordinary Least Squares.

No.	Observed peak flow (m^3/s)	Estimated peak flow (m^3/s)	Residual (m^3/s)	Res/Sc
1	2	3	4=2-3	5=4/Sc
1	509.3	588.0	-78.7	-0.94
2	542.3	543.9	-1.6	-0.02
3	599.4	517.0	82.3	0.98
4	632.8	570.4	62.3	0.74
5	689.3	788.6	-99.3	-1.19
6	758.9	752.5	6.3	0.08
7	764.4	851.0	-86.6	-1.03
8	787.9	717.6	70.2	0.84
9	803.2	741.7	61.4	0.73
10	824.3	924.4	-100.1	-1.20
11	831.2	750.1	81.0	0.97
12	927.4	867.2	60.1	0.72

Table 5.13 Regression Coefficients based on Least Median of Squares.

Variable	Coefficient
Dym	4.31
Wnr	13.61
Mlb	12.02

Coefficient of determination = 0.99

Scale estimate = 107.58

The LMS Regression model is:

$$Q_{peakflow} = 4.31\ TR_d + 13.61\ TR_w + 12.02\ TR_m \tag{5.3}$$

Table 5.14 Regression residuals based on Least Median of Squares.

No.	Observed peak flow (m^3/sec)	Estimated peak flow (m^3/sec)	Residual (m^3/sec)	Res/Sc
1	2	3	4=2-3	5=4/Sc
1	509.3	631.9	-122.6	-1.14
2	542.3	595.2	-52.9	-0.49
3	599.4	566	33.3	0.31
4	632.8	632.8	0	0
5	689.3	843.1	-153.8	-1.43
6	758.9	802	-43.1	-0.4
7	764.4	919.9	-155.5	-1.45
8	787.9	765.5	22.3	0.21
9	803.2	803.2	0	0
10	824.3	1009.4	-185.1	-1.72
11	831.2	804.9	26.2	0.24
12	927.4	927.4	0	0

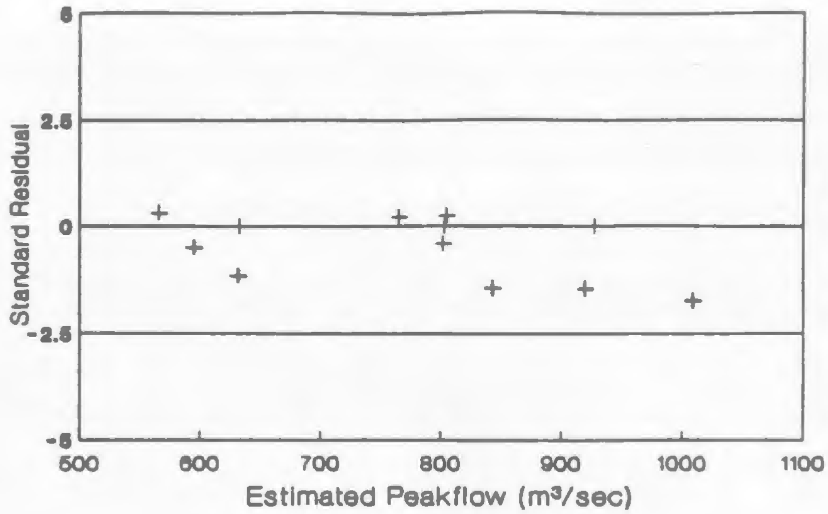


Figure 5.2. Plot of standardized residual versus estimated peakflow.

Table 5.14 and Figure 5.2 showed the standardized residuals were less than 2.5. This indicated no outliers in the residuals. Therefore the data were used in developing the final multiple regression model based on the Reweighted Least Squares.

Table 5.15 Regression coefficients based on the Reweighted Least Squares, based on the LMS.

Variable	Coefficient	Stand. error	T-value	P-value
Dym	3.80	0.59	6.41	0.00
Wnr	12.77	0.98	12.98	0.00
Mlb	11.24	0.69	16.12	0.00

Table 5.15 was the same with Table 5.11 because all data were given weight equal to 1 and the calculation was carried out based on the Least Squares method.

Coefficient of determination (R Squared) = 0.99

The F-Value = 302.765 (With 3 and 9 DF) P-Value = 0.00

There are 12 Point with non-zero weight.

Average weight = 1.00

Scale estimate = 83.82

The regression model is therefore:

$$Q_{peakflow} = 3.8\ TR_d + 12.77\ TR_w + 11.24\ TR_m + e \qquad (5.4)$$

Table 5.16 Regression residual based on Reweighted Least Squares.

No.	Observed peak flow (m^3/sec)	Estimated peak flow (m^3/sec)	Residual (m^3/sec)	Res/Sc	Weight
1	2	3	4=2-3	5=4/Sc	6
1	509.3	588.08	-78.78	-0.94	1.0
2	542.3	543.97	-1.67	-0.02	1.0
3	599.4	517.07	82.32	0.98	1.0
4	632.8	570.47	62.32	0.74	1.0
5	689.3	788.63	-99.33	-1.19	1.0
6	758.9	752.5	6.39	0.08	1.0
7	764.4	851.05	-86.65	-1.03	1.0
8	787.9	717.62	70.27	0.84	1.0
9	803.2	741.73	61.46	0.73	1.0
10	824.3	924.48	-100.18	-1.2	1.0
11	831.2	750.1	81.09	0.97	1.0
12	927.4	867.25	60.14	0.72	1.0

The final result of the regression analysis using the three methods: the Least squares, Least Median of Squares and Reweighted Least Squares based on LMS showed:

1. The final regression model is:

$$Q_{peakflow} = 3.81\ TR_d + 12.77\ TR_w + 11.24\ TR_m \qquad (5.5)$$

2. The coefficient of determination was $R^2_{adj} = 0.99$.
3. All the standardized residuals as shown in Table 5.16 were less than 2.5 (Rousseeuw et al., 1987). Hence, there were no outliers detected.
4. The variation inflation factor for the three variables were less than 10 (Myers 1990) as shown in Table 5.10 indicated that the regression model was not contaminated by multicollinearity.
5. The regression coefficient for the Wanaraja sub-basin is greater than the regression coefficient for the Malangbong because the area of the Wanaraja sub-basin is greater than the area of the Malangbong sub-basin.

5.1.4 Validation of the Multiple Regression Model

Table 5.17 and Table 5.18 respectively are the PRESS statistics calculated based on the Standard Ordinary Least Squares and based on the Least Median of Squares. It was found that the PRESS statistic calculated based on the OLS is relatively smaller than the PRESS statistic based on the LMS. This indicates that the multiple regression model based on the OLS is better than the LMS regression model in estimating the Peak flow.

Table 5.17 PRESS statistic calculated based on the OLS.

No.	Observed peak flow (m^3/sec)	Coefficient of			Estimated peak flow (m^3/sec)	Residual (m^3/sec)	PRESS residual
		Dym	Wnr	Mlb			
1	2	3	4	5	6	7=6-2	$8=7^2$
1	509.3	3.8	12.7	11.5	602.4	93.1	8673.9
2	542.3	3.8	12.8	11.2	544.3	2.0	3.9
3	599.4	3.6	12.7	11.3	504.8	-94.6	8944.6
4	632.8	3.4	13.0	11.4	533.4	-99.4	9877.0
5	689.3	3.7	13.1	11.5	808.4	119.1	4187.0
6	758.9	3.8	12.7	11.3	747.4	-11.5	133.0
7	764.4	3.9	12.8	11.5	864.3	99.9	9974.2
8	787.9	4.1	11.9	11.3	669.2	-118.7	14095.9
9	803.2	3.7	12.9	11.0	727.2	-76.0	5771.6
10	824.3	4.2	13.0	11.1	976.0	151.7	23024.4
11	831.2	3.8	12.7	10.9	734.3	-96.9	9381.5
12	927.4	3.9	12.8	10.8	831.8	-95.6	9135.0
							113201.9

PRESS statistic : 113201.9

Table 5.18 PRESS statistic calculated based on the LMS.

No.	Observed peak flow (m^3/sec)	Coefficent of regression			Estimated peak flow (m^3/sec)	Residual (m^3/sec)	PRESS residual
		Dym	Wnr	Mlb			
1	2	3	4	5	6	7=6-2	$8=7^2$
1	509.3	4.3	13.6	12.0	631.9	122.6	15039.8
2	542.3	4.3	13.6	12.0	595.1	52.8	2791.0
3	599.4	4.3	13.6	12.0	566	-33.4	1113.8
4	632.8	2.7	13.0	8.8	467.7	-165.1	27263.6
5	689.3	4.3	13.6	12.0	843.1	153.8	23667.2
6	758.9	4.3	13.6	12.0	802.1	43.2	1863.3
7	764.4	4.3	13.6	12.0	919.9	155.5	24195.6
8	787.9	4.4	12.9	12.0	723.9	-64.0	4096.6
9	803.2	4.4	13.0	8.6	642.7	-160.5	25772.1
10	824.3	4.3	13.6	12.0	1009.4	185.1	34248.1
11	831.2	4.4	13.0	8.6	628.1	-203.1	41254.1
12	927.4	4.4	13.0	8.6	674.2	-253.2	64133.0
							265438.4

PRESS statistic : 265438.4

5.2 Transfer Hydrograph

This section discusses the flood forecasting model for the Cimanuk River based on the Transfer Hydrograph as described in the previous chapter. The Transfer Hydrographs were derived using data that were categorized as follows:

a) Type A event was an event in which rainfall occurred only at the Cikajang sub-basin as shown in Appendix E1. This event was used in deriving the transfer hydrograph for the Cikajang sub-basin.

b) Type B events were events in which rainfalls occurred at two sub-basin: Cikajang and Wanaraja as shown in Appendix E2. These events were used in deriving the Wanaraja transfer hydrograph.

c) Type C1 events were events in which rainfalls occurred at three sub-basin: Cikajang, Wanaraja and Dy.Manggung as shown in Appendix E3. These events were used in deriving the Dy.Manggung transfer hydrograph.

d) Type C2 events were used in deriving the Malangbong transfer hydrograph. Type C2 events were divided into three categories. The first category was for rainfalls occuring on the Cikajang, Wanaraja and Malangbong sub-basins as shown in Appendix E4. The second category was for rainfalls occuring on the Cikajang, Dy.Manggung and Malangbong sub-basins as shown in Appendix E5. The third category was for rainfalls occuring on the Wanaraja, Dy.Manggung and Malangbong sub-basins as shown in Appendix E6.

Table 5.19 Types of rainfall-runoff events used for derivation and validation of the Transfer Hydrographs.

Event No.	Date			Type of event
1	December,	11	1978	D
2	October,	26	1980	D
3	April,	18	1989	D
4	Janauary,	23	1987	A
5	November,	14	1981	B
6	October,	22	1980	B
7	March,	13	1983	D
8	December,	13	1980	D
9	November,	14	1978	D
10	February,	1	1982	D
11	September,	27	1981	D
12	December,	6	1979	D
13	February,	28	1985	D
14	March,	26	1989	C1
15	December,	31	1986	D
16	October,	4	1979	C2
17	March,	10	1988	B
18	January,	21	1985	C2
19	November,	16	1979	D
20	March,	22	1988	C1
21	October,	2	1981	C2
22	March,	3	1984	C1
23	December,	29	1982	C1
24	December,	6	1984	C1
25	March,	24	1982	C2
26	April,	10	1982	B
27	March,	30	1985	D
28	November,	17	1983	B
29	March,	5	1989	D
30	February,	18	1983	D
31	April,	6	1986	D
32	October,	5	1978	C2

e) Type D events, where rainfalls occur at the four sub-basins, were used in validating the Transfer Hydrographs.

5.2.1 The results of the Transfer Hydrograph study

The Transfer Hydrographs were developed using the matrix method as discussed in Chapter 4. Two sets of results were found in the derivation of the transfer hydrographs. The first set of the results were ordinates of the four transfer hydrographs as presented in Table 5.22 through Table 5.25 and in Figures 5.3 through Figure 5.6. The second set of results was the timelags for the four transfer hydrographs. The timelag is the difference between the time at the beginning of the hourly rainfall and the peakflow. The timelag definition is shown in Figure 5.7.

An assumption in the transfer hydrograph similar to the assumption made in the unit hydrograph as discussed in Chapter 4, was the proportionality between the rainfall volume and the runoff volume. The rainfall volume and the runoff volume for every rainfall-runoff event of data used in developing the transfer hydrograph were compared and the results are presented in Table 5.20. The average and the standard deviation of the ratio between the rainfall volume and runoff volume were respectively 0.5 and 0.07. This ratio indicated that about fifty percent of the rainfall volume was converted into runoff volume. The standard deviation of 0.07 and coefficient of variation of 0.14 indicating that the ratio between the rainfall volume and the runoff volume were practically constant. This indicated that the data were reasonable to be used in deriving the transfer hydrographs.

Table 5.20 Comparison between rainfall volumes and runoff volumes for data used in deriving Transfer Hydrographs.

Type	Peakflow (m^3/s)	Total Rainfall (mm)				Volume (10^6 m^3)		Coeff.of Runoff
		Ckj	Dym	Wnr	Mlb	Rainfall	Runoff	
1	2	3	4	5	6	7	8	9=8/7
A	212.1	92.0	0.0	1.2	0.9	18273.1	6167.9	0.34
B	226.8	26.7	0.4	0.5	18.4	14475.5	7115.0	0.49
B	251.6	28.1	0.2	1.0	16.5	13956.5	7117.0	0.51
C1	354.1	39.0	40.8	0.0	11.2	25768.8	10792.0	0.42
C2	357.8	4.9	46.8	33.3	0.0	30461.9	13665.0	0.45
B	453.6	29.1	0.5	0.0	31.1	21023.3	11819.0	0.56
C2	459.9	13.6	0.0	36.6	4.7	21097.1	12304.0	0.58
C1	485.4	37.9	55.1	0.0	15.8	32357.8	13819.0	0.43
C2	509.3	22.8	12.7	48.0	0.0	29563.6	14302.0	0.48
C1	542.3	30.9	60.4	0.0	24.6	37072.6	16826.0	0.45
C1	599.4	14.2	58.7	0.0	23.0	32603.8	14823.0	0.45
C1	632.8	37.0	96.9	0.0	15.8	45397.4	18580.0	0.41
C2	689.3	0.7	4.0	40.5	24.9	31662.6	17936.0	0.57
B	758.9	5.1	0.0	2.3	56.9	30143.2	18212.0	0.60
B	787.9	8.9	3.0	0.0	55.3	29994.7	18107.0	0.60
C2	927.4	0.6	0.6	73.4	3.1	36233.1	18992.0	0.52

Average runoff coefficient = 0.50.

Standard deviation of the runoff coefficients = 0.07.

Coefficient of variation = 0.14.

Table 5.21 Data of Type D events that were used in validation of Transfer Hydrographs.

Event No.	Date			Observed peakflow Q (m^3/s)
1	December,	11	1978	95.6
2	October,	26	1980	193.1
3	April,	18	1989	197.2
4	March,	13	1983	261.1
5	December,	13	1980	275.5
6	November,	14	1978	284.6
7	February,	1	1982	290.7
8	September,	27	1981	300.4
9	December,	6	1979	334.7
10	February,	28	1985	339.4
11	December,	31	1986	356.3
12	November,	16	1979	461.5
13	March,	30	1985	764.4
14	March,	5	1989	803.2
15	February,	18	1983	824.3
16	April,	6	1986	831.2

5.2.1.1 The derived transfer hydrographs.

This section presents the ordinates of the four transfer hydrographs as shown in Table 5.22 through Table 5.25. It was found that 1 mm rainfall on the Cikajang sub-basin is equivalent to approximately 0.42 mm of runoff on the Cikajang sub-basin as shown in Table 5.22. Table 5.23 showed that 1 mm rainfall on the Wanaraja sub-basin is equivalent to approximately 0.51 mm of runoff on the Wanaraja sub-basin. Table 5.24 showed that 1 mm rainfall on the Dy.Manggung sub-basin is equivalent to approximately 0.44 mm of runoff on the Dy.Manggung sub-basin. Table 5.25. showed that 1 mm of rainfall on the Malangbong sub-basin is equivalent to approximately 0.56 mm of runoff on the Malangbong sub-basin.

The results of the Transfer Hydrograph study showed that the runoff depth calculated based on the volume of each the Transfer Hydrograph was not unity as in the classical unit hydrograph volume. This is because the Transfer Hydrograph were developed based on total rainfall. The runoff depth describes approximately runoff coefficient at each sub-basin. Based on the analysis of the Transfer Hydrograph volumes, it was found that the runoff coefficient for each sub-basin was relatively constant. However, it would not be possible to determine the loss rate, whether the losses were constant with the time or whether it decreases exponentially or otherwise.

Table 5.22 Ordinates of the Cikajang Transfer Hydrograph.

Time	Ordinate of the Cikajang TH (m^3/s)
1	0
2	0.57
3	1.61
4	2.19
5	2.53
6	2.33
7	2.2
8	1.71
9	1.59
10	1.11
11	1.09
12	0.71
13	0.66
14	0.29
15	0

The runoff depth can be calculated from:

$$Runoff\ depth = \frac{\sum_{i=1}^{15} Q_i\ 3600}{A} \tag{5.6}$$

where: Q_i = the transfer hydrograph ordinates,

A = the Cikajang sub-basin area (m^2).

$$Runoff\ depth\ (mm) = \frac{(18.59)\ (3600)\ (1000)}{160000000} \tag{5.7}$$

$$= 0.42\ mm.$$

The rainfall intensity of 1.0 mm/hour in the Cikajang sub-basin produced a hydrograph as shown in Figure 5.3.

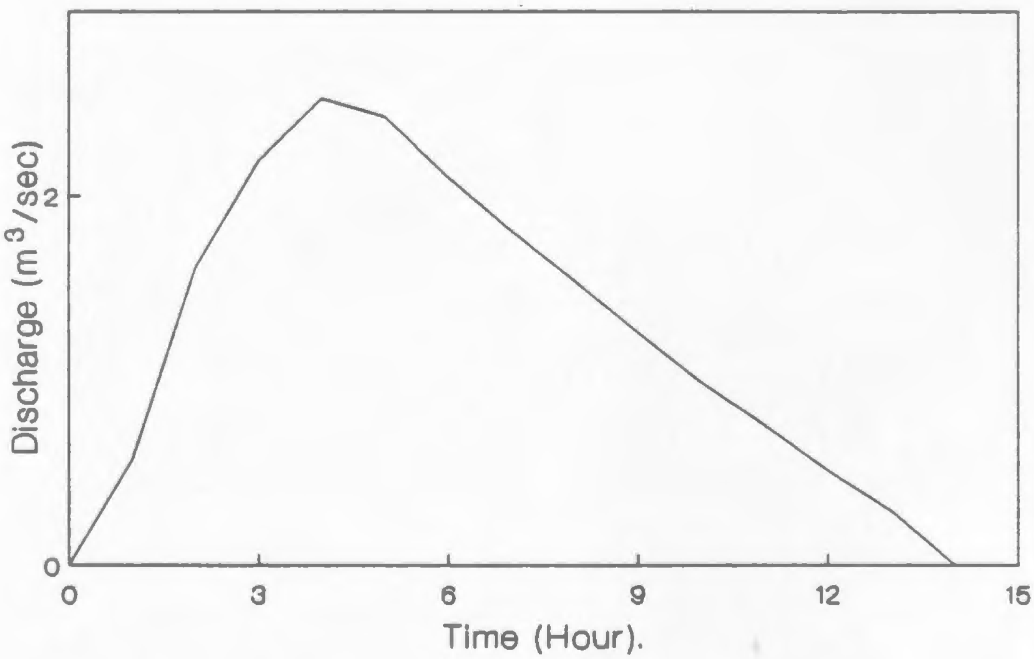


Figure 5.3 The Cikajang Transfer Hydrograph.

Table 5.23 Ordinates of the Wanaraja TH.

Time	Ordinates of Transfer Hydrograph (m^3/s)					Sketched (m^3/s)
	Event No.5	Event No.6	Event No.17	Event No.26	Event No.28	
0	0	0	0	0	0	0
1	7.02	6.07	6.03	5.93	6.11	6.23
2	14.12	13.99	14.08	14.08	14.17	14.09
3	15.15	15.21	15.12	14.33	15.13	14.99
4	14.81	14.15	14.21	12.73	14.65	14.11
5	13.88	13.38	13.36	13.16	13.74	13.5
6	10.28	9.92	10.48	11.1	10.82	10.52
7	7.33	5.19	7.54	8.5	7.65	7.24
8	4.77	3.39	4.51	4.27	4.74	4.34
9	2.35	3.87	2.42	1.82	2.6	2.61
10	0.44	1.4	0.29	0	0.31	0.49
11	0	0	0	0	0	
	90.15	86.57	88.04	85.92	89.92	88.12

$$Runoff\ depth\ (mm) = \frac{\sum_{i=1}^{11} Q_i\ (3600)\ (1000)}{621000000} \tag{5.8}$$

Runoff depth (event no.5) = 0.52 mm

Runoff depth (event no.6) = 0.50 mm

Runoff depth (event no.17) = 0.51 mm

Runoff depth (event no.26) = 0.50 mm

Runoff depth (event no.28) = 0.52 mm

Runoff depth average = 0.51 mm

The rainfall intensity of 1.0 mm/hour in the Wanaraja sub-basin produced a flow hydrograph as shown in Figure 5.4.

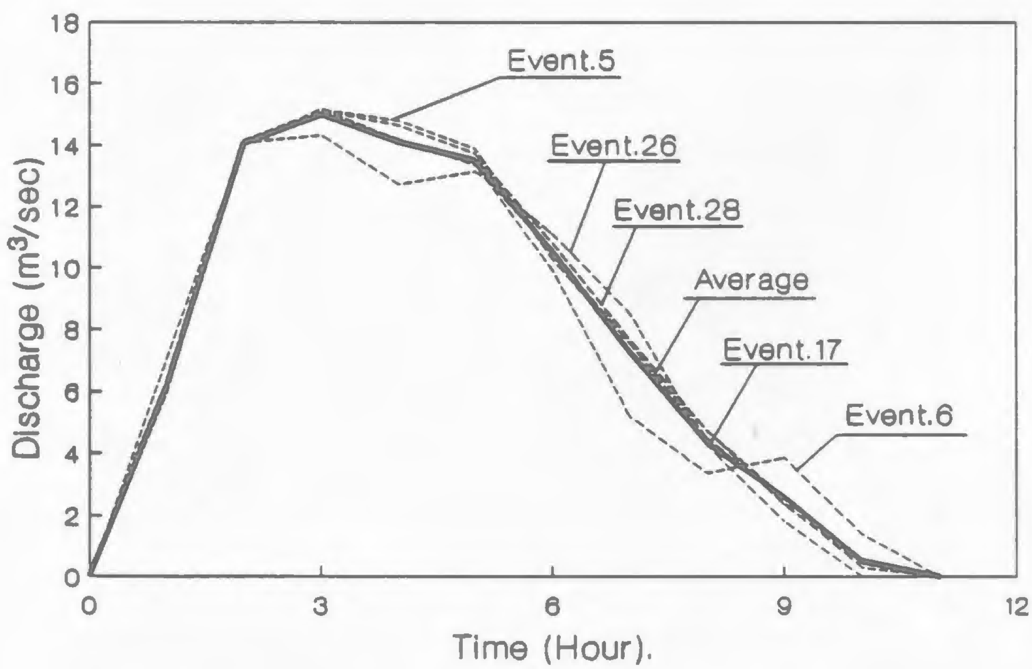


Figure 5.4 The Wanaraja Transfer Hydrograph.

Table 5.24 The ordinates of the Dy.Manggung Transfer Hydrograph.

Time	Ordinates of the Transfer Hydrographs (m^3/s)					Sketched (m^3/s)
	Event No.14	Event No.20	Event No.22	Event No.23	Event No.24	
0	0	0.03	0	0	0.01	0.01
1	1.01	0.86	0.41	1.17	1.56	1.00
2	3.04	3.00	1.10	3.07	2.23	2.49
3	4.63	4.78	2.38	4.57	4.90	4.25
4	4.86	5.00	4.72	4.93	5.28	4.96
5	4.75	4.62	5.05	4.81	4.59	4.76
6	4.03	3.93	5.06	3.95	3.57	4.11
7	2.86	2.94	3.88	2.80	3.24	3.24
8	2.07	2.18	2.64	2.16	2.19	2.45
9	1.71	1.64	2.12	1.75	1.50	1.85
10	1.51	1.40	1.74	1.39	1.40	1.39
11	1.06	0.97	1.56	0.90	1.09	0.92
12	0.46	0.51	0.91	0.52	0.41	0.46
13	0	0	0	0	0	0
	32.04	31.86	31.57	32.02	31.97	31.89

$$Runoff\ depth\ (mm) = \frac{\sum_{i=1}^{13} Q_i\ (3600)\ (1000)}{260000000} \qquad (5.9)$$

Runoff depth (event no.14) = 0.44 mm

Runoff depth (event no.20) = 0.44 mm

Runoff depth (event no.22) = 0.44 mm

Runoff depth (event no.23) = 0.44 mm

Runoff depth (event no.24) = 0.44 mm

Runoff depth average = 0.44 mm

The rainfall intensity of 1.0 mm/hour in the Dy.Manggung sub-basin produced a flow hydrograph as shown in Figure 5.5.

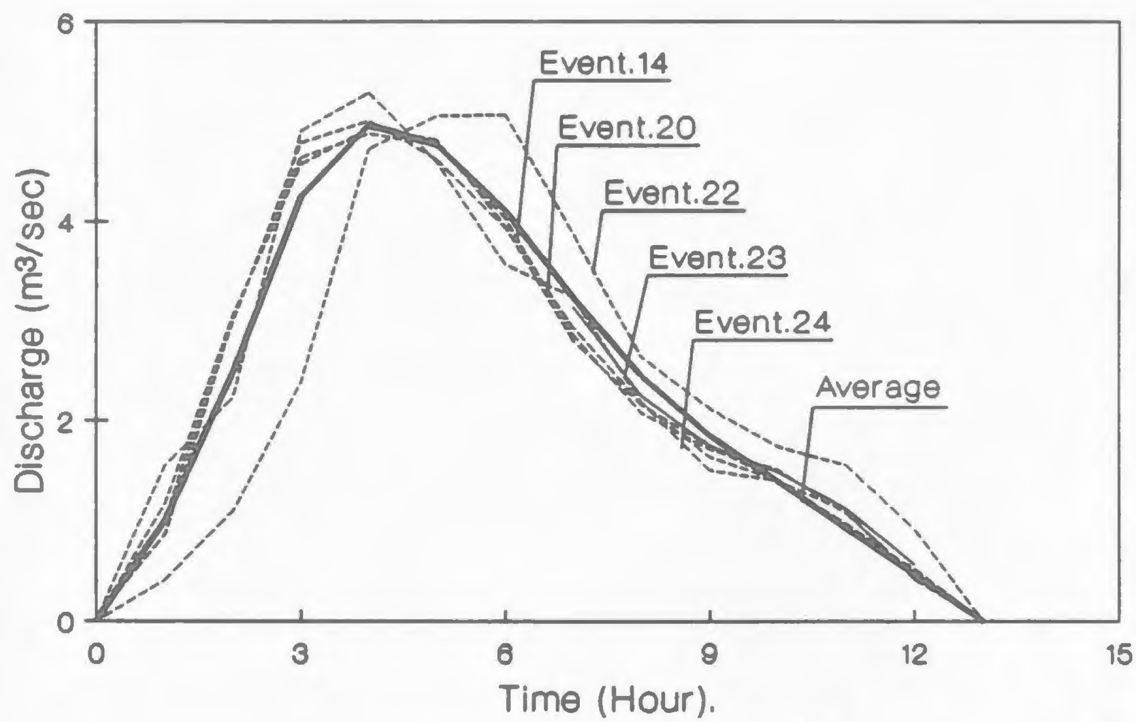


Figure 5.5 The Dy.Manggung Transfer Hydrograph.

Table 5.25 Ordinates of the Malangbong TH.

Time	Ordinates of Transfer Hydrograph (m^3/s)					Sketched (m^3/s)
	Event No.16	Event No.18	Event No.21	Event No.25	Event No.32	
0	0	0	0	0	0	0
1	15.69	15.83	16.07	16.8	15.91	16.06
2	16.12	17.05	16.89	17.32	17.01	16.88
3	11.05	14.47	13.6	14.22	14.06	13.48
4	7.63	9.36	9	9.05	9.48	8.9
5	1	5.42	4.97	5.02	5.66	4.41
6	0	2.38	2.52	3.13	3.15	2.24
7	0	0	0.16	0.16	0	
8	0	0				
	51.49	64.51	63.05	65.7	65.43	61.97

$$Runoff\ depth\ (mm) = \frac{\sum_{i=1}^{7} Q_i\ (3600)\ (1000)}{401000000} \qquad (5.10)$$

Runoff depth (event no.16) = 0.46 mm

Runoff depth (event no.18) = 0.58 mm

Runoff depth (event no.21) = 0.57 mm

Runoff depth (event no.25) = 0.59 mm

Runoff depth (event no.32) = 0.59 mm

Runoff depth average = 0.56 mm

The rainfall intensity of 1.0 mm/hour in the Dy.Manggung sub-basin produced a flow hydrograph as shown in Figure 5.6.

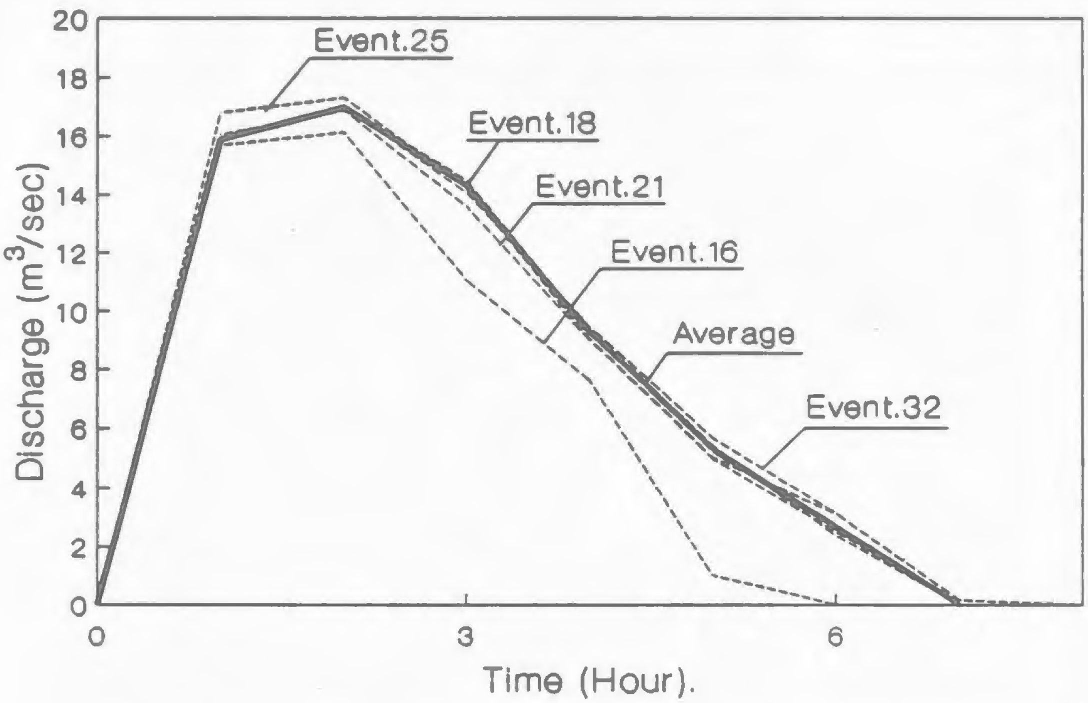


Figure 5.6 The Malangbong Transfer Hydrograph.

5.2.1.2. The timelags of the transfer hydrographs.

The second results in deriving the four transfer hydrographs were the timelags for each of the sub-basin as shown in Table 5.22.

Table 5.26. Time lag, derived based on data Type A, B, C1 and C2 events.

Sub-basin	Delay-time (hours)	Time-to-peak (hours)	Time-lag (hours)
Cikajang	7	4	11
Dy.Manggung	6	4	10
Wanaraja	4	3	7
Malangbong	1	2	3

5.2.2 Validation of the Transfer Hydrographs

The four Transfer Hydrographs were validated using data that were categorized as Type D events. Two factors were used in examining the validity of the Transfer Hydrographs. They were magnitude of the peakflows and the timelags. The timelag was divided into two parts: delay time and time to peak as shown in Figure 5.7. In the validation of the timelags, the time to peaks were assumed constant. Therefore, only delay times for the four transfer hydrographs were validated. For the purpose of the validation of the four transfer hydrographs, convoluted hydrographs at the outlet, Jatigede were derived based on Type D events.

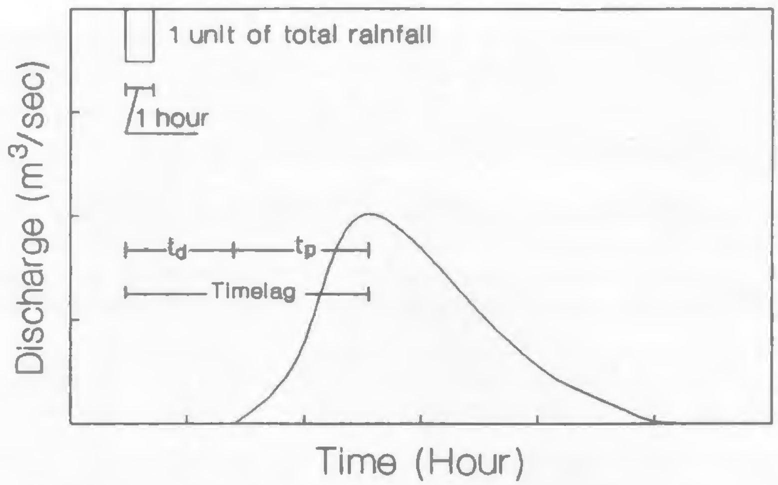


Figure 5.7 Definition of the time-lag, delay-time (t_d) and time-to-peak (t_p).

5.2.2.1. Peakflow validation.

The transfer hydrographs were validated by comparing the peakflows of the observed hydrographs and the peakflows of the convoluted hydrographs, which were calculated based on the Type D events. These convoluted hydrographs were derived based on the delay times and the time to peaks as shown in Table 5.28 and were called S14 curve, as shown in Appendices D1 through D45. These Appendices are only the convoluted hydrographs for the data no.8 through no.16 in Table 5.21. The comparison between the magnitudes of the convoluted hydrograph peakflows and the observed hydrograph peakflows are shown in Table 5.27. This table showed that the error in predicting the peakflows using the Transfer Hydrograph especially for the magnitude of discharge greater than 300.4 m^3/s is less than 12 %. For flood forecasting, this magnitude of error was perhaps acceptable.

Table 5.27 Error in peakflows calculated using Transfer Hydrographs.

No.	Observed peakflow (m^3/s)	Peakflow calculated using TH (m^3/s)	Error (%)
1	2	3	4=(3-2)/2*100%
1	95.6	112.02	17
2	193.1	226.54	17
3	197.2	229.01	16
4	261.1	285.55	9
5	275.5	304.91	11
6	284.6	401.99	41
7	290.7	334.40	15
8	300.4	373.15	24
9	334.7	345.61	3
10	339.4	327.62	-3
11	356.3	369.87	4
12	461.5	514.47	11
13	764.4	743.77	-3
14	803.2	773.63	-4
15	824.3	778.16	-6
16	831.2	808.49	-3

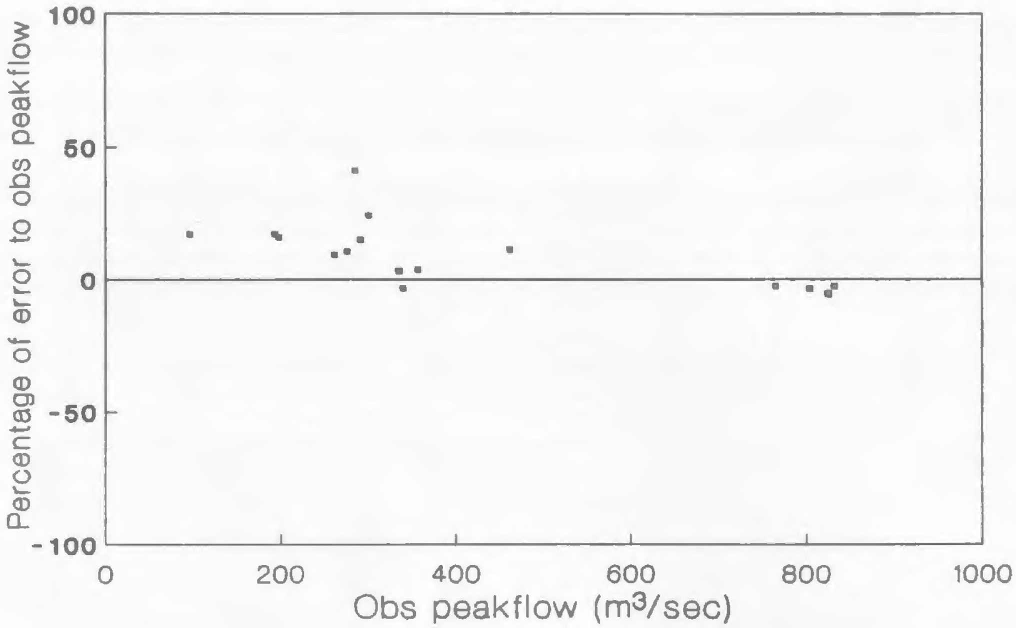


Figure 5.8 Plot of the percentage of residuals, which were residuals divided by the observed peakflows calculated using THs, versus the observed peakflows.

5.2.2.2. Delaytime validation.

For the purpose of delay time validation, convoluted hydrographs at the outlet, Jatigede were derived using Type D events based on several combinations of delay times as shown in Table 5.28. The peakflows of the convoluted hydrographs were compared to the observed peakflow. The delay times for the closest magnitude of the peakflow of the convoluted hydrograph to the magnitude of the observed peakflow were considered as the optimal delay times. The results of the delay time validations for event no.8 through event no.16 are shown in Appendix D1 through Appendix D45.

Table 5.28 Delay-time combinations for validation of time-lags.

Type of combination	Time-lag (hours)			
	Ckj	Dym	Wnr	Mlb
S1	7	5	3	0
S2	7	5	3	1
S3	7	5	3	2
S4	7	5	4	0
S5	7	5	4	1
S6	7	5	4	2
S7	7	5	5	0
S8	7	5	5	1
S9	7	5	5	2
S10	7	6	3	0
S11	7	6	3	1
S12	7	6	3	2
S13	7	6	4	0
S14	7	6	4	1
S15	7	6	4	2
S16	7	6	5	0
S17	7	6	5	1
S18	7	6	5	2
S19	7	7	3	0
S20	7	7	3	1
S21	7	7	3	2
S22	7	7	4	0
S23	7	7	4	1
S24	7	7	4	2
S25	7	7	5	0
S26	7	7	5	1
S27	7	7	5	2

The simulated delay times for all sub-basins are shown in Table 5.29, and compared to the delay times which were obtained in the derivation of the Transfer Hydrographs. It can be seen that the optimal delay times and the derived delay -times are the same except for the Wanaraja sub-basin. The Wanaraja optimal delay time based on the validation was 4 hours for peakflows greater than 600 m^3/s and 5 hours for peakflow less than 600 m^3/s.

Table 5.29 Comparison between derived delay times and simulated delay times.

Sub-basin	Delay-time (hours)		
	Derived TH	Simulated	
		$Q > 600$ m^3/s	$Q < 600$ m^3/s
1. Cikajang	7	7	7
2. Dy.Manggung	6	6	6
3. Wanaraja	4	4	5
4. Malangbong	1	1	1

5.3 Comparison between peakflows calculated based on the Multiple Regression and using the Transfer Hydrographs

A comparison between peakflows, estimated using the multiple regression model and those estimated using the Transfer Hydrograph method, are shown in Table 5.30 and Figures 5.9a and b. It can be seen from Table 5.30 and Figure 5.9a and b that:

a) The predictions of the peakflow based on the Transfer Hydrograph method were relatively more accurate than those of the multiple regression model.

b) The estimated peakflows using the Transfer Hydrograph for the magnitude of discharges greater than 600 m^3/s tend to be underestimated. The estimated peakflow for magnitude of flows less than 600 m^3/s tend to be overestimated.

c) The errors in the calculated peakflow, computed using the Transfer Hydrographs, were relatively less than the errors in the computed peakflow that were calculated using the multiple regression model.

Table 5.30 Comparison of peakflows obtained using the two modelling techniques with observed peakflows.

No.	Observed peakflow (m^3/s)	Estimated peakflow based on Regression model (m^3/s)	% Error (m^3/s)	Estimated peakflow based on TH (m^3/s)	% Error (m^3/s)
1	2	3	4	5	6
1	95.6	139.5	45.9	112.02	17.1
2	193.1	202.4	4.8	226.54	17.3
3	197.2	220.3	11.7	229.01	16.1
4	261.1	411.3	57.5	285.55	9.4
5	275.5	315.9	14.7	304.91	10.7
6	284.6	503.7	76.9	401.99	41.2
7	290.7	170.1	-41.5	334.4	15
8	300.4	417.2	38.9	373.15	24.2
9	334.7	549.9	64.3	345.61	3.2
10	339.4	500.4	47.4	327.62	3.4
11	356.3	546.5	53.3	369.87	3.8
12	461.5	685.3	48.5	514.47	11.5
13	764.4	851.1	11.3	743.77	2.7
14	803.2	741.7	7.6	773.63	3.7
15	824.3	924.9	12.2	778.16	5.6
16	831.2	749.9	9.8	808.49	2.7

Notes: Data no.13 through no.16, in Table 5.30, were used also in the derivation of the multiple regression model as discussed in section 5.1. The convoluted hydrograph, which were derived using those data no.13 through no.16, were compared graphically to the estimated peakflow based on the multiple regression model as shown in Appendices C1 and C2.

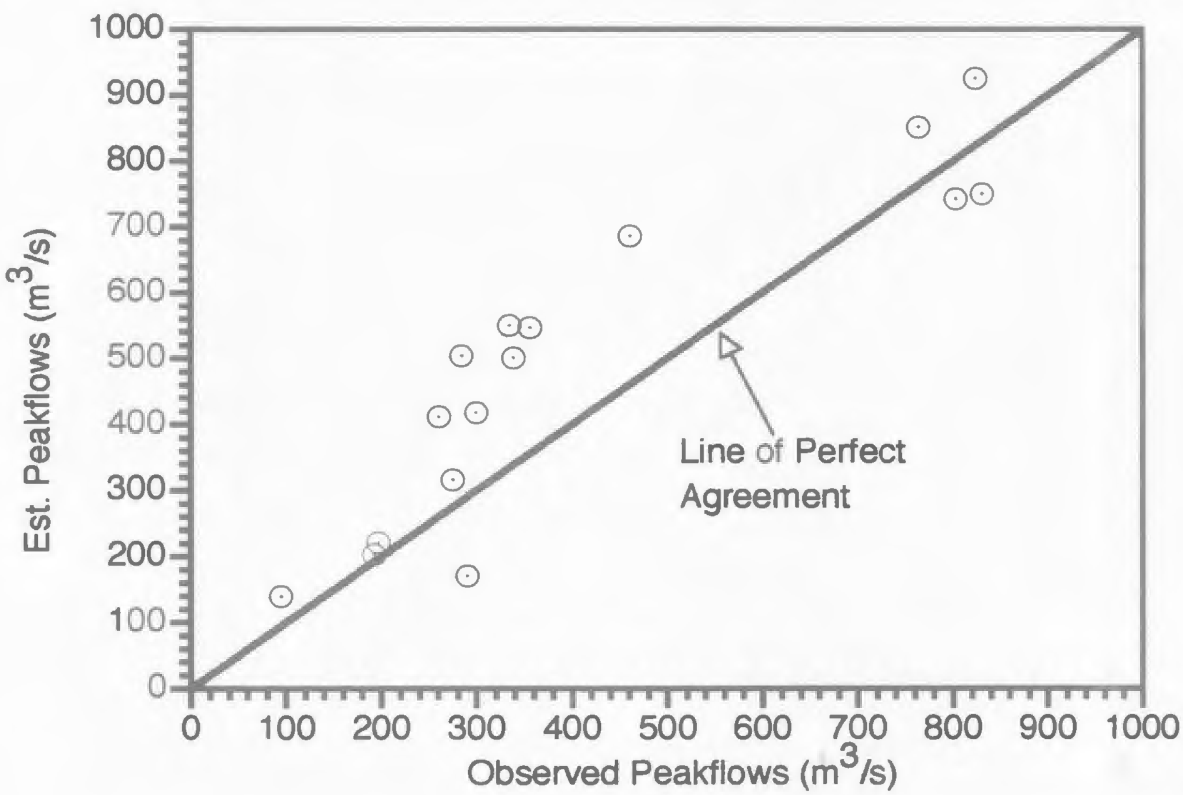


Figure 5.9a Plot of the estimated peakflows using the regression model versus the observed peakflows.

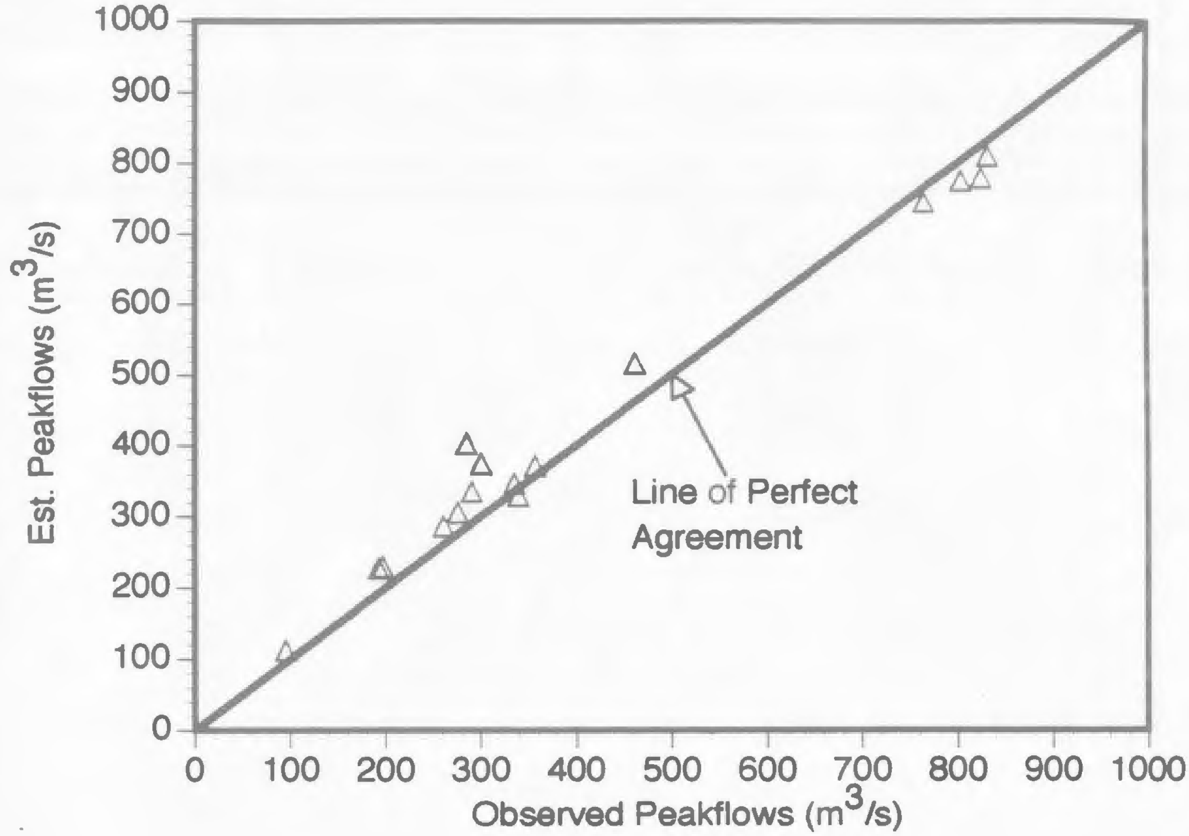


Figure 9b. Plot of the estimated peakflows using the transfer hydrograph versus the observed peakflows.

5.4 Operating Procedure in Flood Forecasting.

This section discusses the operating procedures to estimate the magnitude of flood discharge and the time to peakflow using both flood forecasting models: the multiple regression model and the flood forecasting model based on the Transfer Hydrograph concept. An example of Input data and output of the computer program for the flood forecasting is shown in Appendix F.

The procedure in the use of both methods for flood forecasting can be outlined as follows:

a) Record the total rainfall for each of the four sub-basins: Cikajang, Dy.Manggung, Wanaraja, and sub-basins.

b) Record the time of the beginning of the rainfall at each the four sub-basins.

c) Calculate the peakflow using the multiple regression model as follows:

$$Q_p = 3.81\ TR_d + 12.77\ TR_w + 11.24\ TR_m \qquad (5.11)$$

where: Q_p = peakflow (m^3/s),

TR_d = total rainfall at Dy.Manggung (mm),

TR_w = total rainfall at Wanaraja (mm),

TR_m = total rainfall at Malangbong (mm).

If the estimated peakflow calculated based on the multiple regression model is less than 600 m^3/s, the flood forecasting can be stopped, otherwise continue to point (d).

d) Calculate direct runoff hydrograph at the outlet, Jatigede using the Cimanuk Flood Forecasting Model based on the Transfer Hydrograph. The procedure to calculate the hydrograph at the outlet, Jatigede is shown in Figure 4.14 and the listing of the computer program in Appendix B4. The output of the computer program shows the direct runoff hydrograph at the outlet, Jatigede and the time of the peak. The equation to calculate the hydrograph based on rainfall at one sub-basin is as follows:

$$[Q_i] = [I_i] \, |U_i| \qquad (5.12)$$

where: $[Q_i]$ = vector of discharges, m^3/s

$[I_i]$ = matrix of the rainfall intensity (mm/hour),

$[U_i]$ = vector of the ordinates of the transfer hydrograph, m^3/s.

e) Estimate the base flow at the outlet, Jatigede based on the antecedent flow at the outlet. That is, the flow before the on set of rainin the basin.

f) Add the base flow to the convoluted direct runoff to obtain the total hydrograph at the outlet, Jatigede.

CHAPTER 6.

CONCLUSIONS AND RECOMMENDATIONS

Conclusions

Based on the study of the flood forecasting model for the Cimanuk River, described in Chapters 1 through 5, it can be concluded that:

1. A rainfall intensity of 1 mm with duration of 1 hour on the Cikajang, Dy.Manggung, Malangbong and Wanaraja sub-basins respectively yielded runoff depths of about 0.42, 0.44, 0.51 and 0.56 mm, respectively. However, these numbers cannot be considered as runoff coefficients because these numbers were calculated based on the Transfer Hydrograph at the outlet, Jatigede, where the effects of translation was included.
2. Based on the data, a heavy rainfall in the Dy.Manggung sub-basin or Cikajang sub-basin with a total rainfall up to 80 mm and a duration of 4 hours, would not cause a significant discharge unless heavy rainfalls at Wanaraja sub-basin and or Malangbong sub-basin also occurred.

3. The Multiple Regression model gave better predictions for large magnitudes of discharge because:

   a) The multiple regression model was derived based on observed peak-flows that were greater than 500 m^3/s.

   b) The larger the total rainfall, perhaps the more uniform the rainfall throughout a given sub-basin.

   c) The larger the total rainfall, the more constant the percentage of rainfall losses. This is because of the rice field and fish pond, which account for about 30 % of the basin area, and are distributed uniformly over the basin. These may conserve alot of water during a rainfall.

4. Similar to the Multiple Regression model, the Transfer Hydrograph concept gave better predictions as the magnitude of discharge increased. For this reason, the Transfer Hydrographs are not suggested to predict floods, where the magnitude of discharge is less than 600 m^3/s. However, for the flood forecasting purposes, the magnitude of a discharge should be considered to be a flood discharge when the magnitude of the discharge exceeds 600 m^3/s.

5. Time to peak can be estimated using the Transfer Hydrograph concept. It may give a reasonable prediction for magnitude of flood greater than 600 m^3/s. However, for small magnitudes of discharge (less than 600 m^3/s) the estimated time to peak tended to be longer than the observed time to peak.

6. The Transfer Hydrograph concept produced better overall predictions than the Multiple Regression model, as shown in Table 5.30 and in Figure 5.9a and b. However, the use of the multiple regression model is relatively easy for flood forecasting, and can therefore be used for preliminary forecasting.
7. The accuracy of the both methods each having errors of less than 15 %, for flood forecasting and flood warning purpose should be sufficient in view of the uncertainties involved and the quality of available data.

Recommendations

1. With error prediction of less than 15 % (as shown in Table 5.30), the multiple regression and the Transfer Hydrographs concepts may be considered as reasonable tools for the prediction of flood discharges at present.
2. The multiple regression and the Transfer Hydrograph models should be updated with the increase of the rainfall-runoff data every year.
3. A radar raingauge is recommended to be installed in the Cimanuk Basin to estimate rainfall at each of the four sub-basins, especially for the nearest sub-basin to the outlet, Jatigede. The time-lag for the Malangbong sub-basin of 3 hours is usually less than the rainfall duration. Therefore, the total rainfall at the Malangbong sub-basin must be estimated for flood forecasting at the outlet, Jatigede.

4. For further study it is suggested:

   a) To observe the flow at each outlet of the four sub-basins and to study the effect of channel routing.

   b) To study hydrologic abstractions, such as infiltration and evaporation at each sub-basin.

These studies can then be used to develop a more sound conceptual model for flood forecasting. Such a conceptual model could be used to estimate the flow at the outlet for various magnitude of discharge.

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APPENDICES

APPENDIX A

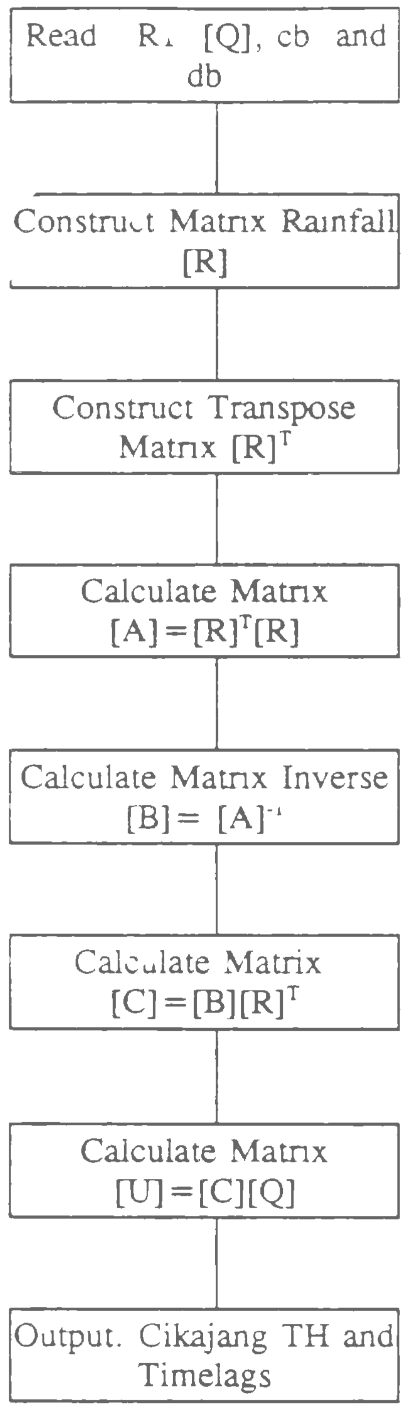


Figure 4 10 Flow-chart to calculate the Cikajang Transfer Hydrograph.

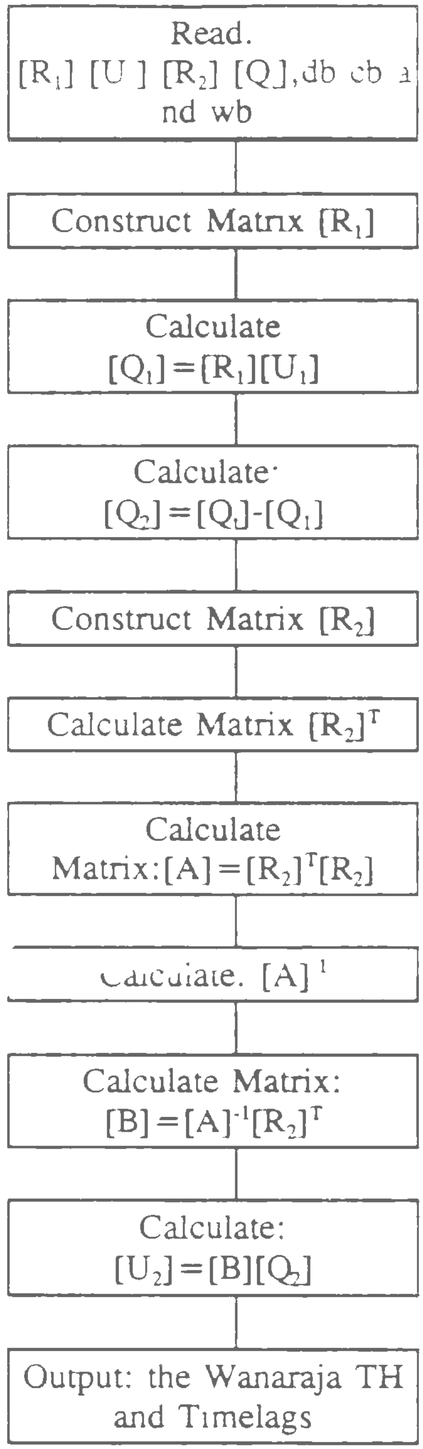


Figure 4.1.1 Flow-chart to calculate Wanaraja Transfer Hydrograph.

APPENDIX A3

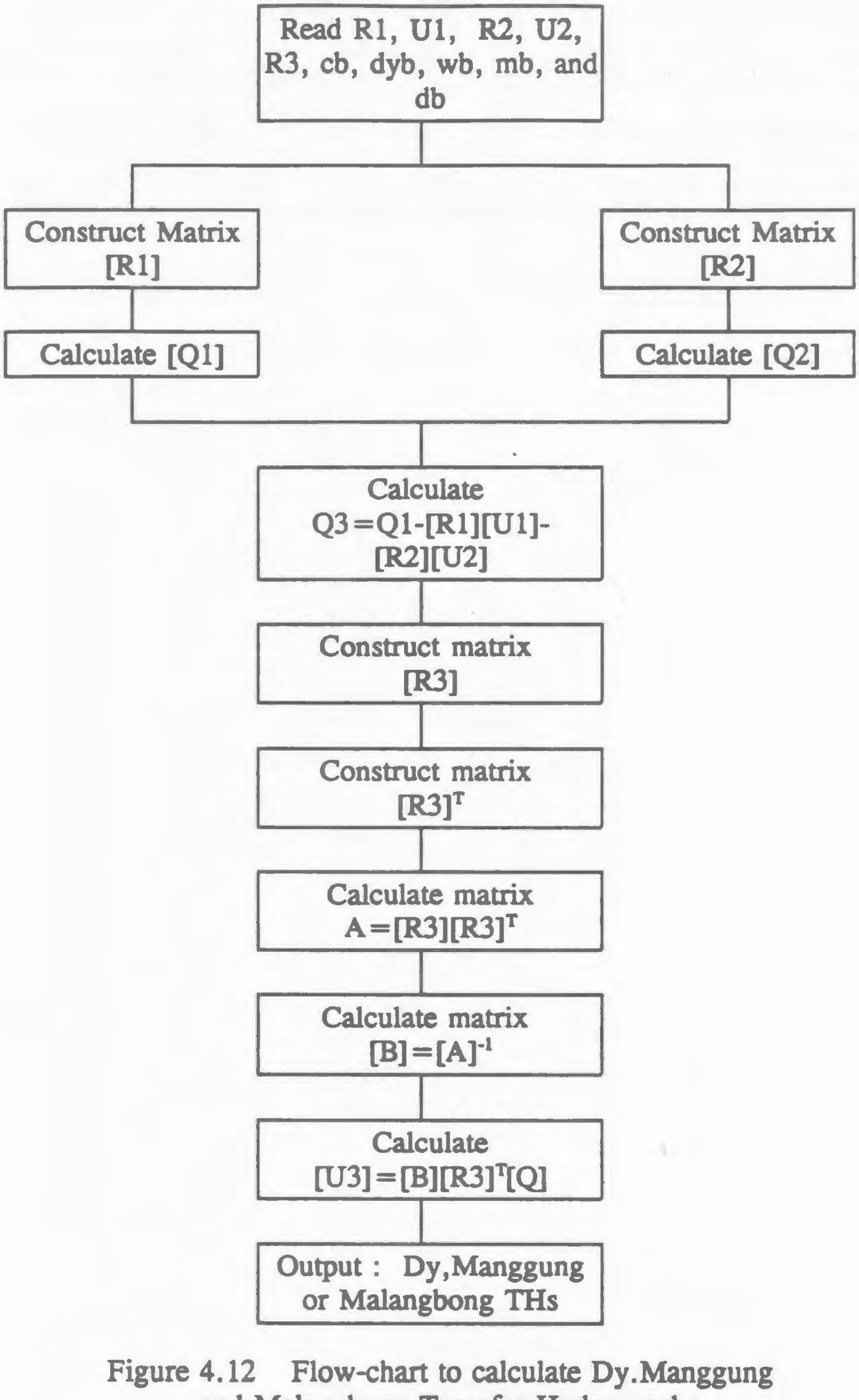


Figure 4.12 Flow-chart to calculate Dy.Manggung and Malangbong Transfer Hydrographs.

APPENDIX B1

```
' Program to calculate the Cikajang TH.
  DIM c, d, M, W, H, U, Q1, V, T, i, HINVS
  DATA number of the hydrograph ordinates.
  DATA ordinates  of the hydrograph.
  DATA duration of the rainfall data.
  DATA ordinates of the rainfall data.
' Reading the hydrograph ordinates.
    READ d1
    PRINT d1
    FOR i = 1 TO d1
        READ Q1(i)
        NEXT i
    READ d
' Matrix zero.
    c = d1 + 1 - d
    H = c + d - 1
    V = 0
    FOR e = 1 TO c
        FOR f = 1 TO H
            V(f) = 0
            M(e, f) = V(f)
        NEXT f
    NEXT e
' Reading rainfall data.
    FOR i = 1 TO d
        READ U(i)
    NEXT i
' Matrix P
    FOR j = 1 TO c
        FOR i = 1 TO d
        M(i + j - 1, j) = U(i)
        NEXT i
    NEXT j
' Matrix transpose.
    FOR x = 1 TO c
        FOR y = 1 TO H
        T(x, y) = M(y, x)
        NEXT y
    NEXT x
```

```
' Matrix P multyplied by Matrix Transpose.
    FOR i = 1 TO c
        FOR j = 1 TO H
        H(i, j) = 0
            FOR k = 1 TO H
            H(i, j) = H(i, j) + T(i, k) * M(k, j)
            NEXT k
        NEXT j
    NEXT i


'  Matrix Inverse.
50  n = c
    FOR i = 1 TO n
        FOR j = n + 1 TO 2 * n
        i(i, j) = 0
        NEXT j
    NEXT i
    FOR i = 1 TO c
        H(i, i + c) = 1
    NEXT i

'  Calculation of Inverse.
5   nt2 = 2 * n
    np1 = n + 1
    nm1 = n - 1
    nm2 = n
    FOR j = 1 TO nm2
        A = ABS(H(j, j))
        jp1 = j + 1
        FOR i = jp1 TO n
            B = ABS(H(i, j))
            FOR i = jp1 TO n
                quot = H(i, j) / H(j, j)
                FOR k = j TO nt2
                    H(i, k) = H(i, k) - quot * H(j, k)
                NEXT k
            NEXT i
        NEXT i
    NEXT j
42  k = n
11  i = k - 1
12  quot = H(i, k) / (H(k, k))
```

```
    FOR j = 1 TO 2 * c
        H(i, j) = H(i, j) - quot * H(k, j)
    NEXT j
    i = i - 1
    IF i = 0 GOTO 13
    GOTO 12
13  k = k - 1
    IF k = 1 GOTO 14
    GOTO 11
14  FOR i = 1 TO n
        FOR j = 1 TO n
            HINVS(i, j) = H(i, n + j) / H(i, i)
        NEXT j
    NEXT i

' Matrix transpose.
    FOR x = 1 TO c
        FOR y = 1 TO H
        T(x, y) = M(y, x)
        NEXT y
    NEXT x
' Matrix inverse * Matrix transpose.
    FOR i = 1 TO c
        FOR j = 1 TO H
        W(i, j) = 0
            FOR k = 1 TO H
            W(i, j) = W(i, j) + HINVS(i, k) * T(k, j)
            NEXT k
        NEXT j
    NEXT i
' Matrix inverse * PT * Ordinates of the hydrograph.
    FOR i = 1 TO c
        U(i) = 0
        FOR k = 1 TO d1
            U(i) = U(i) + W(i, k) * Q1(k)
        NEXT k
    NEXT i
    FOR y = 1 TO c: PRINT #2, USING "####.##"; U(y); : NEXT y
100 END
```

APPENDIX B2

```
' Program to calculate the Wanaraja TH.
DIM uc, cik, Qt, Qcp, Qc, Qnet, c, W, h, U, Q1, v, d, rc,
DIM i, T, HINVS, M
DATA number of the Cikajang TH.
DATA ordinates of the Cikajang TH.
DATA duration of the rainfall data at the Cikajang subbasin.
DATA ordinates of the rainfall hyetograph at the Cikajang subbasin.
DATA number of the ordinates of the hydrograph.
DATA the beginning of the rainfall at the Cikajang.
DATA the beginning of the rainfall at the Wanaraja.
DATA the beginning of the arising of the hydrograph.
DATA ordinates of the hydrograph.
DATA duration of the rainfall at the Wanaraja.
DATA ordinates of the rainfall at the Wanaraja subbasin.

'  Reading Cikajang TH.
    READ u1
    FOR i = 1 TO u1
        READ uc(i)
    NEXT i
'  Reading rainfall data at Cikajang.
    READ dc
    FOR i = 1 TO dc
        READ rc(i)
    NEXT i
'  Matrix P
    FOR i = 1 TO u1
        FOR j = 1 TO dc
        cik(i + j - 1, i) = rc(j)
        NEXT j
    NEXT i
    FOR i = 1 TO hc
        Qcp(i) = 0
        FOR k = 1 TO u1
            Qcp(i) = Qcp(i) + cik(i, k) * uc(k)
        NEXT k
    NEXT i
'  Sum of discharge.
'  Read total discharge.
```

```
  READ dt
  READ cb
  j = 1
  FOR i = cb + 7 TO cb + hc + 7
      Qc(i) = Qcp(j)
      j = j + 1
  NEXT i
  READ wb
  READ db
  dl = db + dt - 1
  FOR i = db TO dl
      READ Qt(i)
  NEXT i
' Qn : Nett Discharge.
  FOR i = db TO dl
      Qnet(i) = Qt(i) - Qc(i)
      IF Qnet(i) < 0 THEN Qnet(i) = 0
  NEXT i
  READ d
' Matrix zero.
  c = dt + 1 - d
  h = c + d - 1
  v = 0
  FOR i = 1 TO c
      FOR j = 1 TO h
          v(j) = 0
          M(i, j) = v(j)
      NEXT j
  NEXT i
' Reading the Wanaraja rainfall data.
  FOR i = 1 TO d
      READ U(i)
  NEXT i
' Matrix P
  FOR j = 1 TO c
      FOR i = 1 TO d
          M(i + j - 1, j) = U(i)
      NEXT i
  NEXT j
```

```
'  Transpose Matrix.
    FOR i = 1 TO c
        FOR j = 1 TO h
            T(i, j) = M(j, i)
        NEXT j
    NEXT i
' Matrix P multyplied by Matrix Transpose.
    FOR i = 1 TO c
        FOR j = 1 TO h
        h(i, j) = 0
            FOR k = 1 TO h
            h(i, j) = h(i, j) + T(i, k) * M(k, j)
            NEXT k
        NEXT j
    NEXT i
' Matrix Inverse.
50   n = c
    FOR i = 1 TO n
        FOR j = n + 1 TO 2 * n
        i(i, j) = 0
        NEXT j
    NEXT i
    FOR i = 1 TO c
        h(i, i + c) = 1
    NEXT i
' Calculation of Inverse.
5     nt2 = 2 * n
    np1 = n + 1
    nm1 = n - 1
    nm2 = n
' The first elimination.
    FOR j = 1 TO nm2
        A = ABS(h(j, j))
        jp1 = j + 1
        FOR i = jp1 TO n
            B = ABS(h(i, j))
            FOR i = jp1 TO n
                quot = h(i, j) / h(j, j)
                FOR k = j TO nt2
                    h(i, k) = h(i, k) - quot * h(j, k)
                NEXT k
            NEXT i
        NEXT i
   NEXT j
```

```
' The second elimination.
42  k = n
11  i = k - 1
12  quot = h(i, k) / (h(k, k))
    FOR j = 1 TO n * 2
        h(i, j) = h(i, j) - quot * h(k, j)
    NEXT j
    i = i - 1
    IF i = 0 GOTO 13
    GOTO 12
13  k = k - 1
    IF k = 1 GOTO 14
    GOTO 11
14  FOR i = 1 TO n
        FOR j = 1 TO n
            HINVS(i, j) = h(i, n + j) / h(i, i)
        NEXT j
    NEXT i

' Matrix transpose.
    FOR i = 1 TO c
        FOR j = 1 TO h
        T(i, j) = M(j, i)
        NEXT j
    NEXT i
' Matrix inverse multyplied by Matrix Transpose.
    FOR i = 1 TO c
        FOR j = 1 TO h
        W(i, j) = 0
            FOR k = 1 TO h
            W(i, j) = W(i, j) + HINVS(i, k) * T(k, j)
            NEXT k
        NEXT j
    NEXT i
    k = 1
    FOR i = db TO dl
        Qnet(k) = Qnet(i)
        k = k + 1
    NEXT i
' Unit Hydrograph.
```

```
' Matrix inverse * PT * Hydrograph ordinates.
  FOR i = 1 TO c
      U(i) = 0
      FOR k = 1 TO 23
          U(i) = U(i) + W(i, k) * Qnet(k)
      NEXT k
  NEXT i
      FOR i = 1 TO c
      IF U(i) < 0 THEN U(i) = 0
          U(i) = U(i)
      NEXT i
  FOR i = 1 TO c: PRINT USING "####.##"; U(i); : NEXT i: PRINT
100 END
```

APPENDIX B3

```
' Program  to calculate the Dy.Manggung TH and the Malangbong TH.
DIM c, cik, d, h, hinvs, i, M, Q1, Qcw, Qt, Qc, Qw, Qwn, Quot
DIM Qnet, Qcc, rw, rm, T, U, Qt1, uc, uwa, v, W, wan
DATA number of the ordinates of the Cikajang TH.
DATA ordinates of the Cikajang TH.
DATA number of the ordinates of Cikajang hyetograph.
DATA ordinates of the hyetograph at Cikajang.
DATA number of the ordinates of Wanaraja hyetograph.
DATA number of the ordinates of the Wanaraja TH.
DATA ordinates of the hyetograph at Wanaraja.
DATA ordinates of the Wanaraja TH.
DATA number of the ordinates of the hydrograph.
DATA beginning of rainfall at Cikajang.
DATA beginning of rainfall at Wanaraja.
DATA beginning of the arising of the hydrograph.
DATA ordinates of the hydrograph.
DATA number of ordinates of Dy.Manggung hyetograph.
DATA ordinates of the hyetograph at Dy.Manggung
'  Reading the Cikajang TH.
    READ u1
    FOR i = 1 TO u1
        READ uc(i)
    NEXT i
'  Reading the rainfall data.
    READ dc
    FOR i = 1 TO dc
        READ rc(i)
    NEXT i
'  Matrix P.
    FOR i = 1 TO u1
        FOR j = 1 TO dc
        cik(i + j - 1, i) = rc(j)
        NEXT j
    NEXT i
    hc = u1 + dc - 1
'  Discharge caused by the rainfall at Cikajang.
    FOR i = 1 TO hc
        Qc(i) = 0
        FOR k = 1 TO u1
            Qc(i) = Qc(i) + cik(i, k) * uc(k)
        NEXT k
    NEXT i
```

```
    READ dw
    READ uw


    FOR i = 1 TO dw
        READ rw(i)
    NEXT i
FOR i = 1 TO uw
        FOR j = 1 TO dw
            wan(i + j - 1, i) = rw(j)
        NEXT j
    NEXT i
    hw = uw + dw - 1
    FOR i = 1 TO uw
        READ uwa(i)
    NEXT i
'  Discharge came from Wanaraja.
    FOR i = 1 TO hw
        Qwn(i) = 0
        FOR k = 1 TO uw
            Qwn(i) = Qwn(i) + wan(i, k) * uwa(k)
        NEXT k
    NEXT i
    READ dt
    READ cb
    READ wb
    READ db
    j = 1
    FOR i = cb + 7 TO cb + hc + 7
        Qcc(i) = Qc(j)
        j = j + 1
    NEXT i
    j = 1
    FOR i = wb + 4 TO wb + hw + 8
        Qw(i) = Qwn(j)
        j = j + 1
    NEXT i
    dl = db + dt - 1
    FOR i = db TO dl
        Qcw(i) = Qcc(i) + Qw(i)
    NEXT i
```

```
'  Reading hydrograph.
    FOR i = 1 TO dt
        READ Qt1(i)
    NEXT i
    k = 1
    FOR i = db TO db + dt - 1
        Qt(i) = Qt1(k)
        k = k + 1
    NEXT i
'  Qn : Nett Discharge.
        FOR i = db TO db + dt - 1
            Qnet(i) = Qt(i) - Qcw(i)
            IF Qnet(i) < 0 THEN Qnet(i) = 0
        NEXT i
' Matrix for  Dy.Manggung.
' Matrix zero.
  READ d
  dtt = dt
  hm = dtt + 1 - d
  h = hm + d - 1
  v = 0
  FOR i = 1 TO hm
      FOR j = 1 TO h
          v(j) = 0
          M(i, j) = v(j)
      NEXT j
  NEXT i
' Reading the DYM rainfall data.
  FOR i = 1 TO d
      READ rm(i)
  NEXT i
' Matrix P
  FOR j = 1 TO hm
      FOR i = 1 TO d
          M(i + j - 1, j) = rm(i)
      NEXT i
  NEXT j
' Transpose of Matrix.
  FOR i = 1 TO hm
      FOR j = 1 TO h
          T(i, j) = M(j, i)
      NEXT j
  NEXT i
```

```
' Matrix P multyplied by Matrix Transpose.
    FOR i = 1 TO hm
        FOR j = 1 TO h
        h(i, j) = 0
            FOR k = 1 TO h
                h(i, j) = h(i, j) + T(i, k) * M(k, j)
            NEXT k
       NEXT j
    NEXT i
' Matrix Inverse.
50  c = hm
    n = c
    FOR i = 1 TO n
        FOR j = n + 1 TO 2 * n
            i(i, j) = 0
        NEXT j
    NEXT i
    FOR i = 1 TO c
        h(i, i + c) = 1
    NEXT i
' Inverse.
5   nt2 = 2 * n
    np1 = n + 1
    nm1 = n - 1
    nm2 = n
' The first elimination.
    FOR j = 1 TO nm2
        A = ABS(h(j, j))
        jp1 = j + 1
        FOR i = jp1 TO n
            B = ABS(h(i, j))
            FOR i = jp1 TO n
                Quot = h(i, j) / h(j, j)
                FOR k = j TO nt2
                    h(i, k) = h(i, k) - Quot * h(j, k)
                NEXT k
            NEXT i
        NEXT i
    NEXT j
' The second elimination.
42  k = n
11  i = k - 1
12  Quot = h(i, k) / (h(k, k))
```

```
    FOR j = 1 TO 2 * n
        h(i, j) = h(i, j) - Quot * h(k, j)
    NEXT j
    i = i - 1
    IF i = 0 GOTO 13
    GOTO 12
13  k = k - 1
    IF k = 1 GOTO 14
    GOTO 11
14  FOR i = 1 TO n
        FOR j = 1 + n TO 2 * n
            hinvs(i, j) = h(i, j) / h(i, i)
        NEXT j
    NEXT i
' Matrix inverse multyplied by Matrix Transpose.
    FOR i = 1 TO c
        FOR j = 1 TO h
            W(i, j) = 0
            FOR k = 1 TO h
                W(i, j) = W(i, j) + hinvs(i, k + n) * T(k, j)
            NEXT k
        NEXT j
    NEXT i
    dl = db + dt - 1
    k = 1
    FOR i = db TO dl
        Qnet(k) = Qnet(i)
        k = k + 1
    NEXT i
' Matrix inverse * PT * ordinates of hydrograph.
    FOR i = 1 TO c
        U(i) = 0
        FOR k = 1 TO dtt
            U(i) = U(i) + W(i, k) * Qnet(k)
        NEXT k
    NEXT i
        FOR i = 1 TO c
            IF U(i) < 0 THEN U(i) = 0
            U(i) = U(i)
        NEXT i
    FOR i = 1 TO c
        PRINT USING "###.###"; U(i);
    NEXT i
100 END
```

APPENDIX B4

```
'           Program to calculate hydrograph.

DIM uct, uc, cik, dct, rc, hc, Qc, Qcc, udt, ud, dym, ddt, rd
DIM hd, Qd, Qdd, uwt, uw, wnr, dwt, rw, hw, Qw, Qww, umt, um
DIM mlb, dmt, rm, hm, Qm, Qmm, Qcdwm.
DATA number of ordinates of the Cikajang TH.
DATA ordinates of the Cikajang TH.
DATA number of the ordinates of Cikajang hyetograph.
DATA ordinates of the rainfall data at Cikajang sta.
DATA The number of data of the Dy.Manggung unit hydrograph point.
DATA The Unit Hydrograph of Dy.Manggung.
DATA The number of the rainfall data at Dy.Manggunmg.
DATA The rainfall data at Dy.Manggung.
DATA The number of data of the Wanaraja unit hydrograph point.
DATA
DATA The number of the rainfall data at Wanaraja.
DATA The rainfall data at Wanaraja.
DATA The number of data of the Malangbong unit hydrograph point.
DATA The number of the rainfall data at Malangbong.
DATA The rainfall data at Malangbong.
DATA  cb
DATA  db
DATA  wb
DATA  mb
'  Reading the Cikajang discharge data.
     READ uct
     FOR i = 1 TO uct
        READ uc(i)
     NEXT i
'  Reading the rainfall data.
     READ dct
     FOR i = 1 TO dct
        READ rc(i)
     NEXT i
     PRINT uct
     PRINT dct
```

```
' Matrix P of the Cikajang rainfall data.
  FOR i = 1 TO uct
     FOR j = 1 TO dct
     cik(i + j - 1, i) = rc(j)
     NEXT j
  NEXT i


  hc = uct + dct - 1
   FOR i = 1 TO hc
   FOR j = 1 TO uct
   PRINT USING "###.###"; cik(i, j);
   NEXT j: PRINT : NEXT i: PRINT
  FOR i = 1 TO uct: PRINT USING "###.###"; uc(i); : NEXT i
  PRINT
  PRINT
' Discharge caused by a rainfall at Cikajang sta.
  FOR i = 1 TO hc
     Qc(i) = 0
     FOR k = 1 TO uct
     Qc(i) = Qc(i) + cik(i, k) * uc(k)
     NEXT k
  NEXT i
  FOR i = 1 TO hc: PRINT USING "###.###"; Qc(i); : NEXT i
  PRINT
' Reading the Dy.Manggung discharge data.
  READ udt
  FOR i = 1 TO udt
     READ ud(i)
  NEXT i
' Reading the rainfall data.
 READ ddt
  FOR i = 1 TO ddt
     READ rd(i)
  NEXT i
  PRINT udt
  PRINT ddt
' Matrix P of the Dy.Manggung rainfall data.
  FOR i = 1 TO udt
     FOR j = 1 TO ddt
     dym(i + j - 1, i) = rd(j)
     NEXT j
  NEXT i
```

```
    hd = udt + ddt - 1
     FOR i = 1 TO hd
     FOR j = 1 TO udt
     PRINT USING "###.###"; dym(i, j);
     NEXT j: PRINT : NEXT i
    FOR i = 1 TO udt: PRINT USING "###.###"; ud(i); : NEXT i
'   Discharge caused by a rainfall at Dy.Manggung.
    FOR i = 1 TO hd
       Qd(i) = 0
       FOR k = 1 TO udt
       Qd(i) = Qd(i) + dym(i, k) * ud(k)
       NEXT k
    NEXT i

    FOR i = 1 TO hd: PRINT USING "###.###"; Qd(i); : NEXT i
'   Reading the Wanaraja discharge data.
    READ uwt
    FOR i = 1 TO uwt
       READ uw(i)
    NEXT i
'   Reading the rainfall data.
    READ dwt
    FOR i = 1 TO dwt
       READ rw(i)
    NEXT i
    PRINT uwt
    PRINT dwt
'   Matrix P of the Dy.Manggung rainfall data.
    FOR i = 1 TO uwt
       FOR j = 1 TO dwt
       wnr(i + j - 1, i) = rw(j)
       NEXT j
    NEXT i
    hw = uwt + dwt - 1
     FOR i = 1 TO hw
     FOR j = 1 TO uwt
     PRINT USING "###.###"; wnr(i, j);
     NEXT j: PRINT : NEXT i: PRINT
    FOR i = 1 TO uwt: PRINT USING "###.###"; uw(i); : NEXT i
    PRINT
    PRINT
```

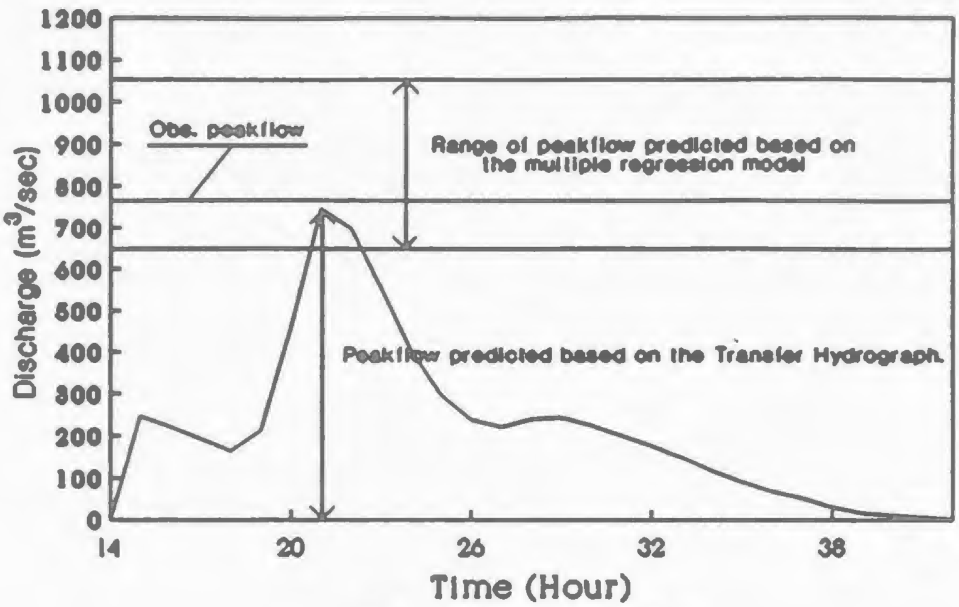
```
' Discharge caused by a rainfall at Wanaraja.
  FOR i = 1 TO hw
     Qw(i) = 0
     FOR k = 1 TO uwt
     Qw(i) = Qw(i) + wnr(i, k) * uw(k)
     NEXT k
  NEXT i
  FOR i = 1 TO hw: PRINT USING "###.###"; Qw(i); : NEXT i
  PRINT
' Reading the Malangbong discharge data.
  READ umt
   FOR i = 1 TO umt
     READ um(i)
  NEXT i
 Reading the rainfall data.
  READ dmt
  PRINT dmt
  FOR i = 1 TO dmt
     READ rm(i)
  NEXT i
' Matrix P of the Malangbong rainfall data.
  FOR i = 1 TO umt
     FOR j = 1 TO dmt
     mlb(i + j - 1, i) = rm(j)
     NEXT j
  NEXT i
  hm = umt + dmt - 1
   FOR i = 1 TO hm
   FOR j = 1 TO umt
   PRINT USING "###.###"; mlb(i, j);
   NEXT j: PRINT : NEXT i: PRINT
  FOR i = 1 TO umt: PRINT USING "###.###"; um(i); : NEXT i
  PRINT
  PRINT
' Discharge caused by a rainfall at Malangbong sta.
  FOR i = 1 TO hm
     Qm(i) = 0
     FOR k = 1 TO umt
     Qm(i) = Qm(i) + mlb(i, k) * um(k)
     NEXT k
  NEXT i
  FOR i = 1 TO hm: PRINT USING "###.###"; Qm(i); : NEXT i
  PRINT
```

```
    READ  b
    READ db
    READ wb
    READ mb
    IF cb + 7 > db + 6 GOTO 99
     F cb + 7 > wb + 4 GOTO 98
    IF cb + 7 > mb + 1 GOTO 97
    hcdwm1 = cb + 7
    GOTO 95
99   IF db + 6 > wb + 4 GOTO 98
    IF db + 6 > mb + 1 GOTO 97
    hcdwm1 = db + 6
    GOTO 95
98   IF wb + 4 > mb + 1 GOTO 97
    hcdwm1 = db + 4
    GOTO 95
97   hcdwm1 = mb + 1
    GOTO 95
95   IF cb + 7 + hc < db + 6 + hd GOTO 89
    IF cb + 7 + hc < wb + 4 + hw GOTO 88
    IF cb + 7 + hc < mb + 1 + hm GOTO 87
    hcdwm2 = cb + 7 + hc
    GOTO 85
89   IF db + 6 + hd < wb + 4 + hw GOTO 88
    IF db + 6 + hd < mb + 1 + hm GOTO 87
    hcdwm2 = db + 6 + hd
    GOTO 85
88   IF wb + 4 + hw < mb + 1 + hm GOTO 87
    hcdwm2 = wb + 4 + hw
    GOTO 85
87   hcdwm2 = mb + 1 + hm
85   k = 1
    s = 1
    TLC = 7
    FOR j = 1 TO hc
      Qcc(j + cb - 1 + TLC) = Qc(j)
    NEXT j
    FOR i = hcdwm1 TO hcdwm2: PRINT USING "###.###"; Qcc(i);
            NEXT i
    l = 1
```

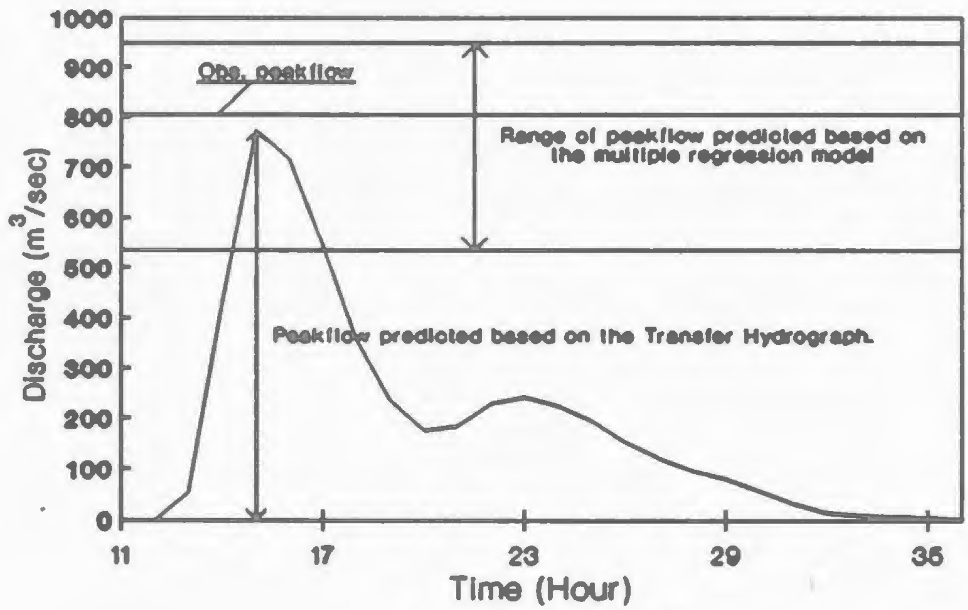
```
FOR TLD = 5 TO 7
  FOR j = 1 TO hd
  Qdd(j + db - 1 + TLD, l) = Qd(j)
  NEXT j
  PRINT
  FOR i = hcdwm1 TO hcdwm2
  PRINT USING "###.###"; Qdd(i, l); : NEXT i
  PRINT
  m = 1
  FOR TLW = 3 TO 5
    FOR j = 1 TO hw
    Qww(j + wb - 1 + TLW, m) = Qw(j)
    NEXT j
    PRINT
'   FOR i = hcdwm1 TO hcdwm2
    PRINT USING "###.###"; Qww(i, m); : NEXT i
    PRINT
    n = 1
    FOR TLM = 0 TO 2
      FOR j = 1 TO hm
      Qmm(j + mb - 1 + TLM, n) = Qm(j)
      NEXT j
      FOR i = hcdwm1 TO hcdwm2
      PRINT USING "###.###"; Qmm(i, n); : NEXT i
      PRINT
      s = s + 1
      FOR i = hcdwm1 TO hcdwm2
      Qcdwm(i, s) = Qcc(i) + Qdd(i, l) + Qww(i, m) +
      Qmm(i, n)
      NEXT i
      PRINT #1, TAB(15)
      PRINT #1, "s.... = "; : PRINT #1, s - 1
      PRINT #1, TAB(15)
      PRINT #1, "TLD.. = "; : PRINT #1, TLD
      PRINT #1, TAB(15)
      PRINT #1, "TLW.. = "; : PRINT #1, TLW
      PRINT #1, TAB(15)
      PRINT #1, "TLM  = "; : PRINT #1, TLM
      PRINT #1,
      FOR i = hcdwm1 TO hcdwm2
      PRINT #1, USING "####.##"; Qcdwm(i, s); : NEXT i
      PRINT s - 1
      PRINT l - 1
      PRINT m - 1
```

```
            PRINT n - 1
            PRINT TLD
            PRINT TLW
            PRINT TLM
            FOR i = hcdwm1 TO hcdwm2
            PRINT USING "####.##"; Qcdwm(i, s); : NEXT i
          n = n + 1
          NEXT TLM
        m = m + 1
        NEXT TLW
    l = l + 1
    NEXT TLD
    FOR i = hcdwm1 TO hcdwm2: PRINT USING "####.#"; Qcc(i); : NEXT
i
    FOR i = hcdwm1 TO hcdwm2: PRINT USING "####.#"; Qdd(i, 2); :
NEXT i
    FOR i = hcdwm1 TO hcdwm2: PRINT USING "####.#"; Qww(i, 2); :
NEXT i
    FOR i = hcdwm1 TO hcdwm2: PRINT USING "####.#"; Qmm(i, 2); :
NEXT i
    FOR i = hcdwm1 TO hcdwm2
    PRINT USING "####.##"; Qcdwm(i, 15); : NEXT i
100 END
```

APPENDIX C1



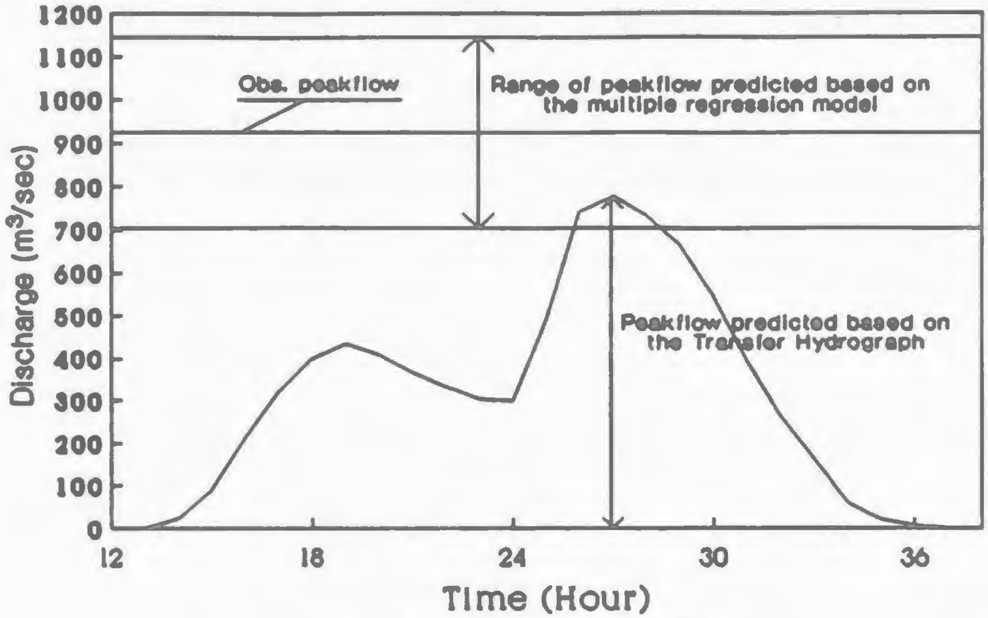
March, 30 1985



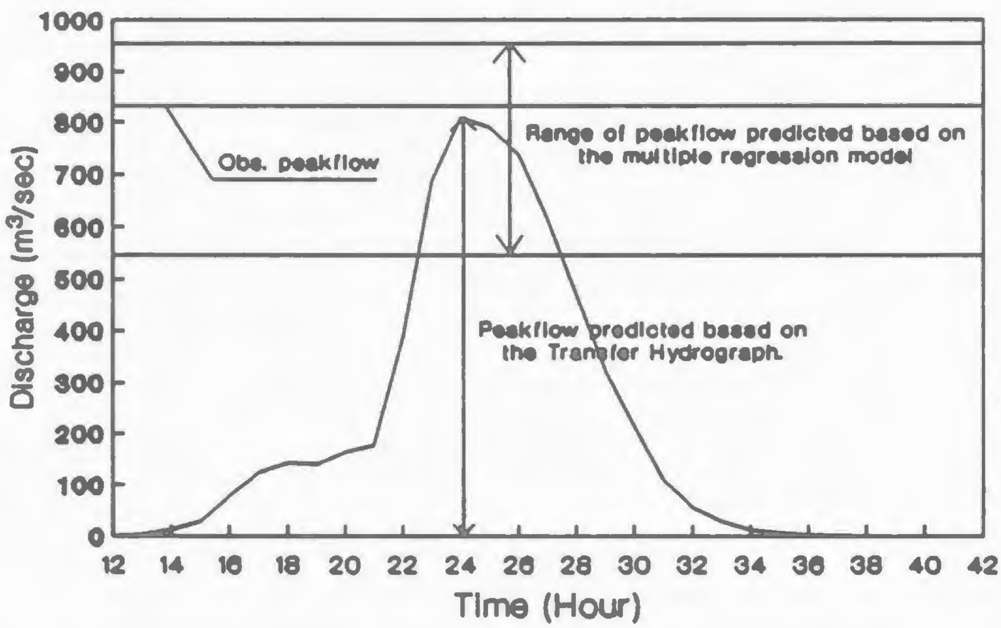
March, 5 1989

Comparison between peakflow predicted based on the multiple regression model and the Transfer Hydrograph.

APPENDIX C2



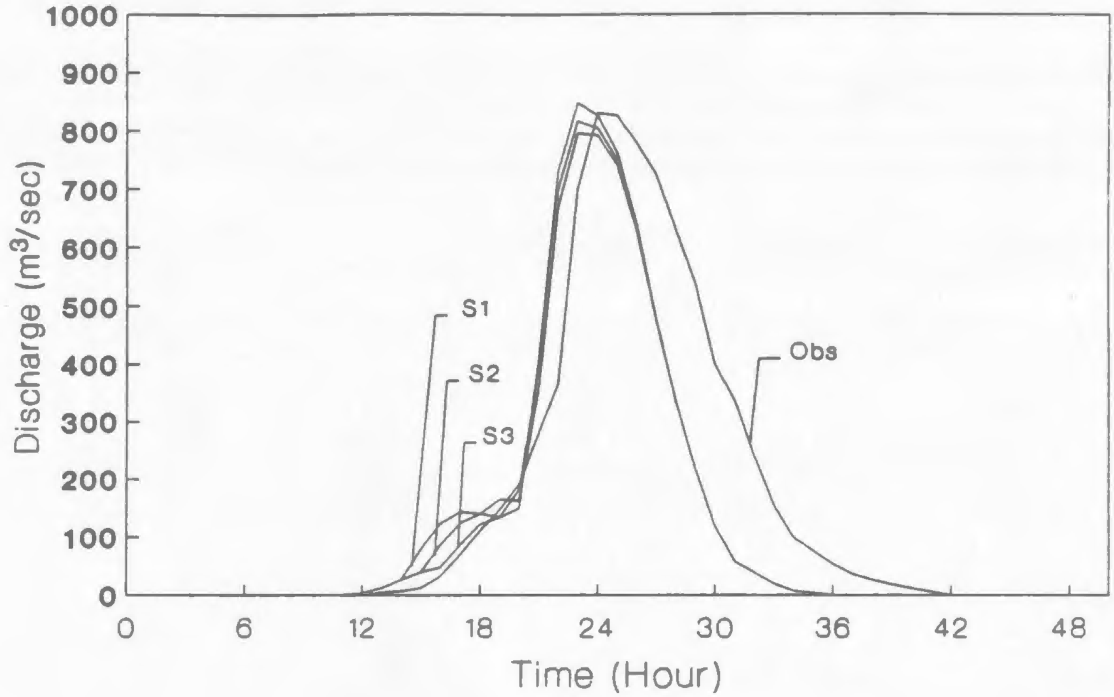
February, 18 1983.

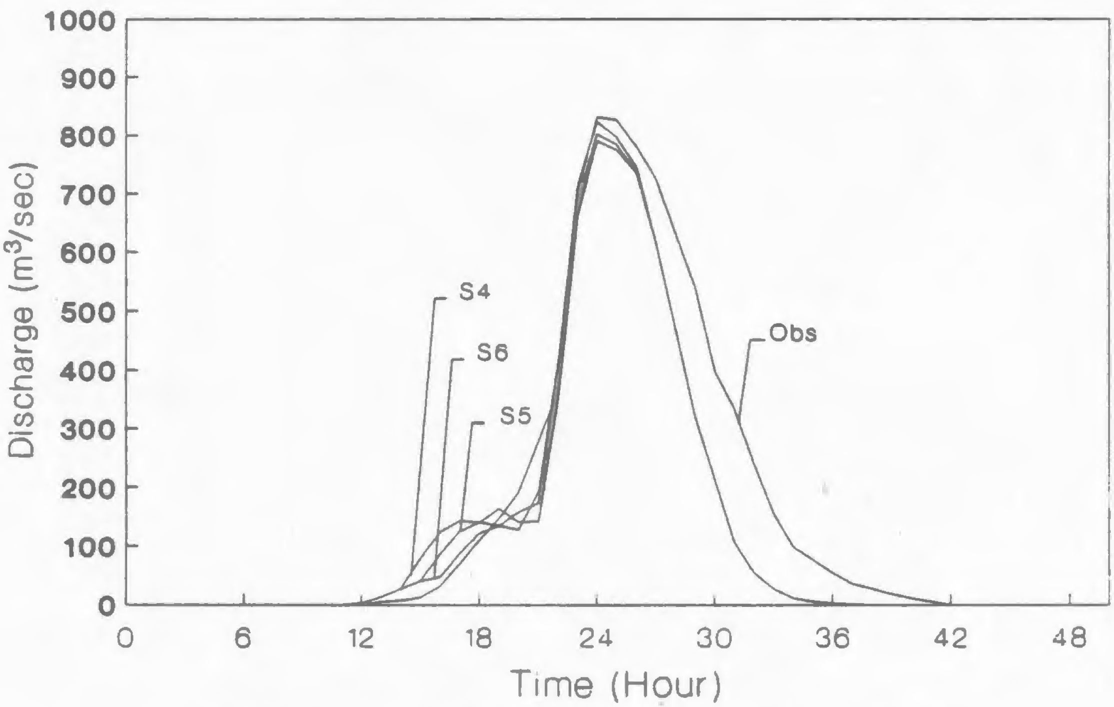


April, 6 1986.

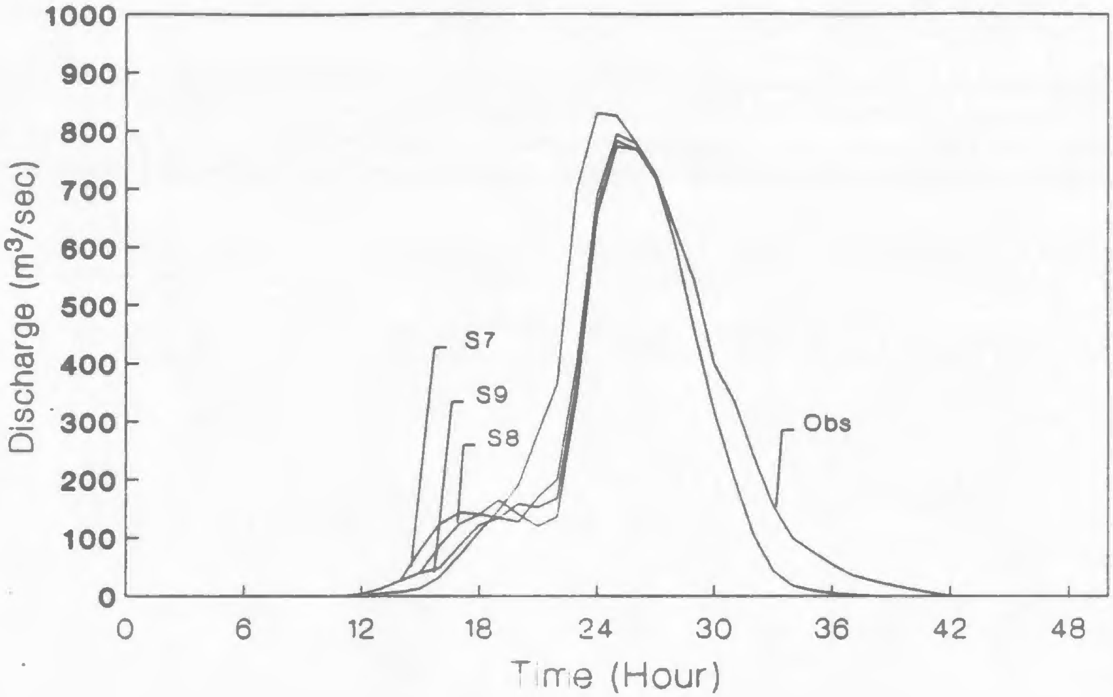
Comparison betwen peakflow predicted based on the multiple regression model and the transfer hydrograph

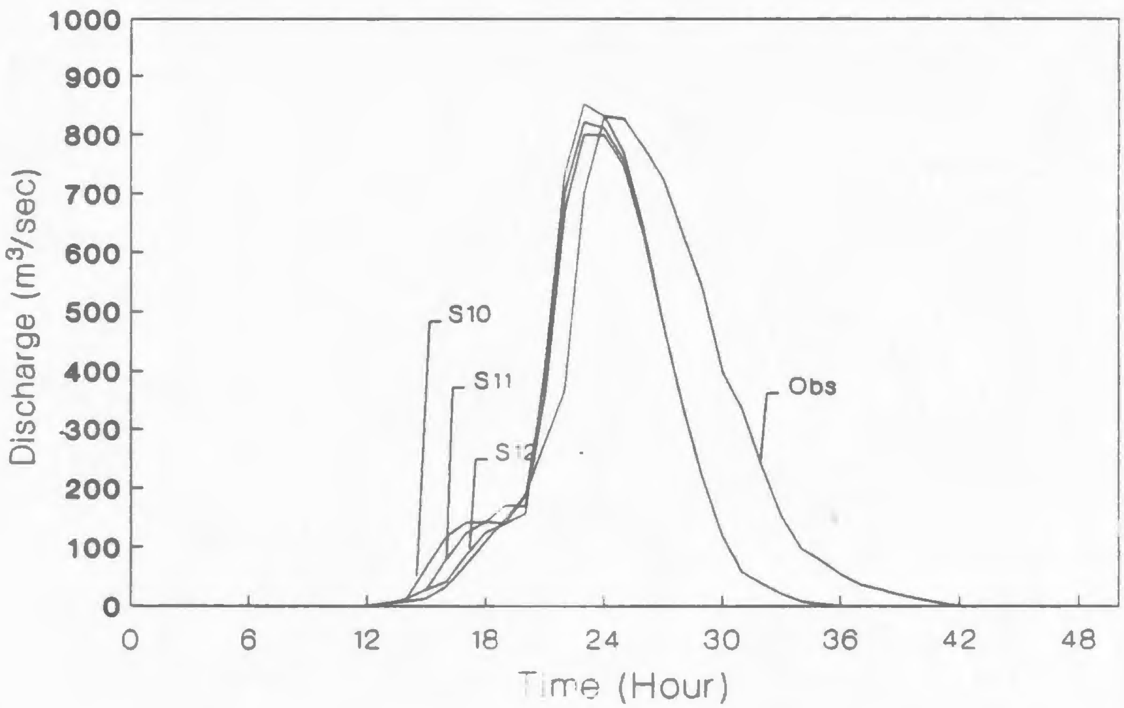
APPENDIX D1





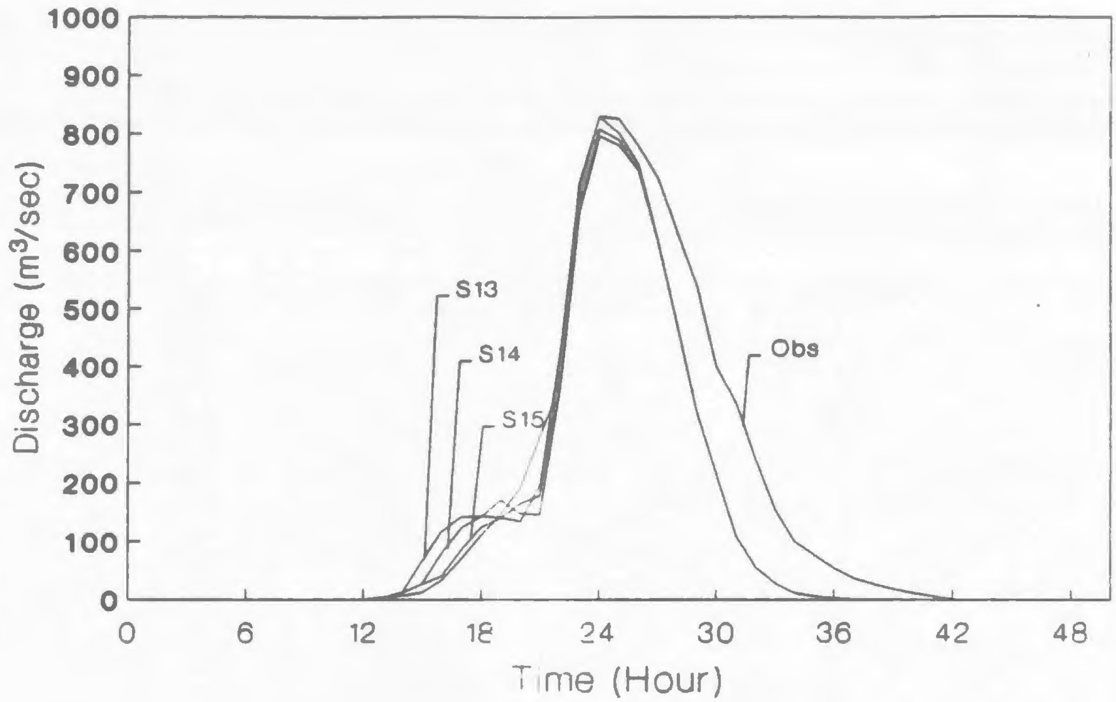
The convoluted hydrograph : April, 6 1986

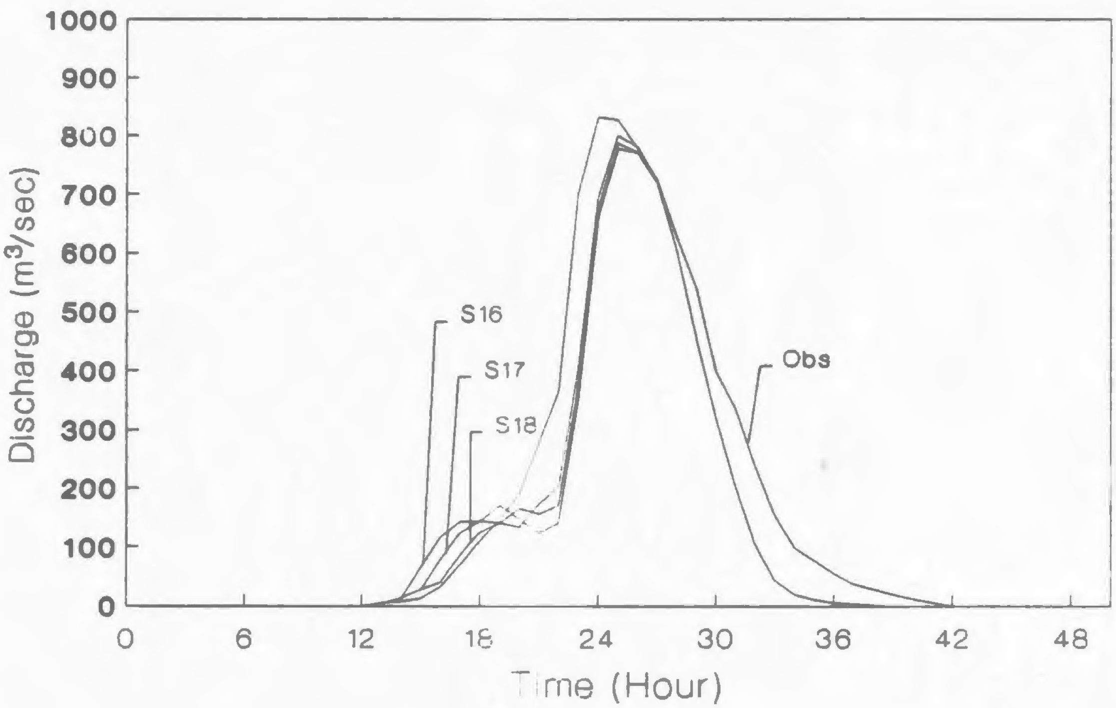




The convoluted hydrograph : April 6, 1986

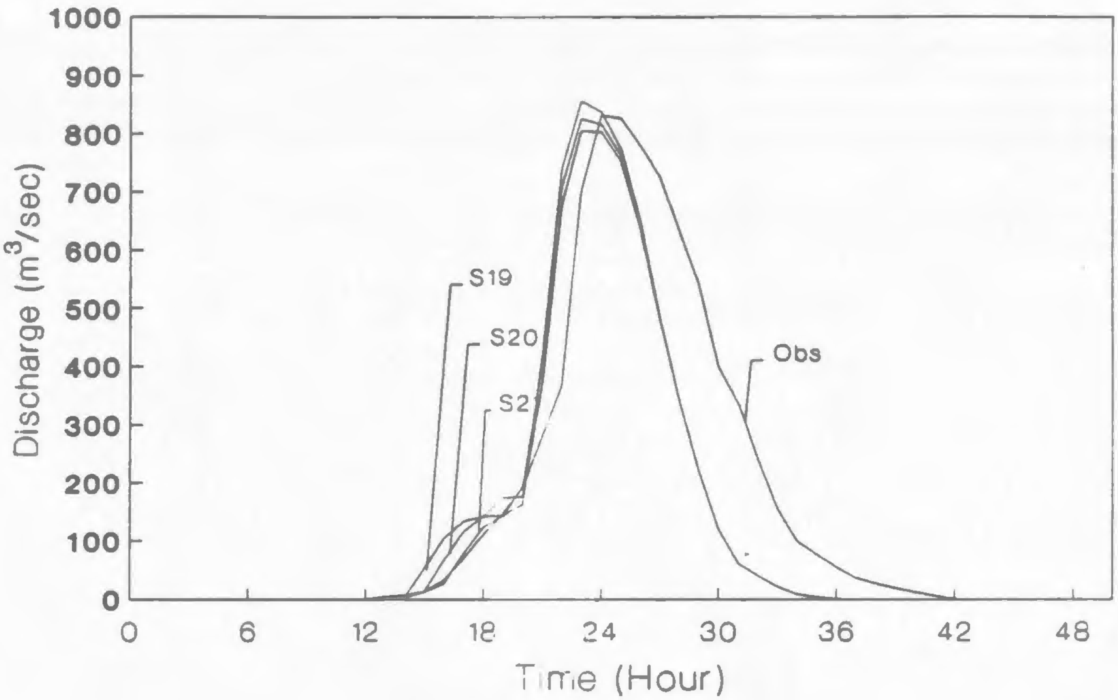
APPENDIX D3

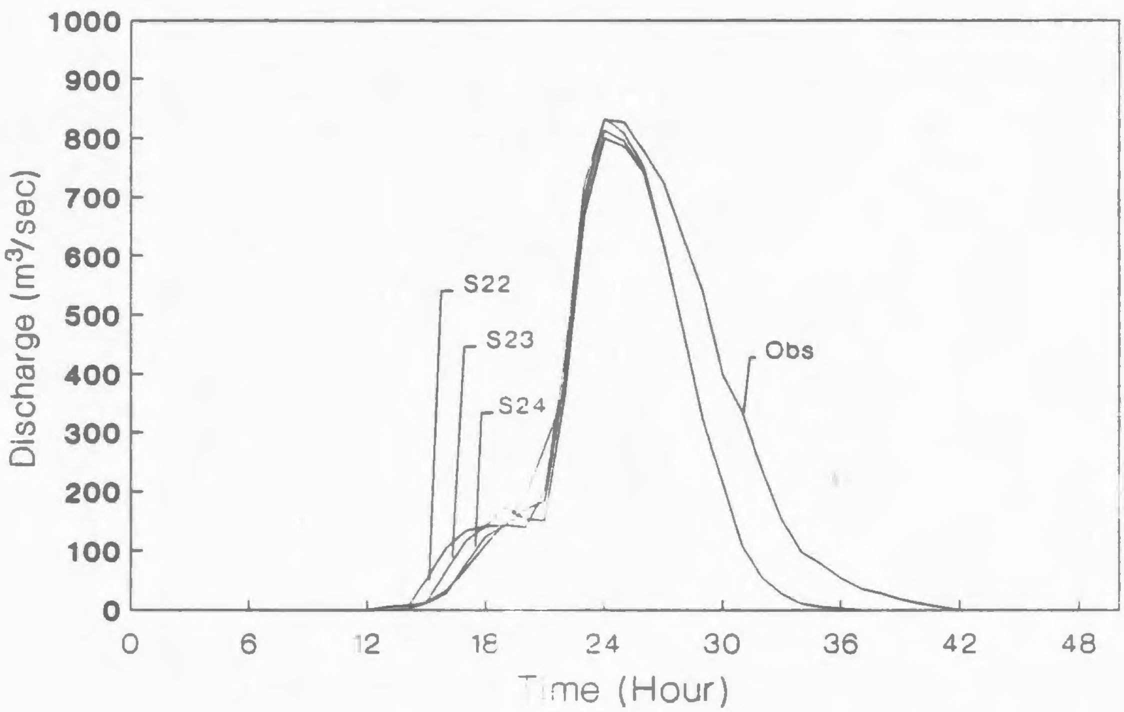




The convoluted hydrograph : April, 6 1986

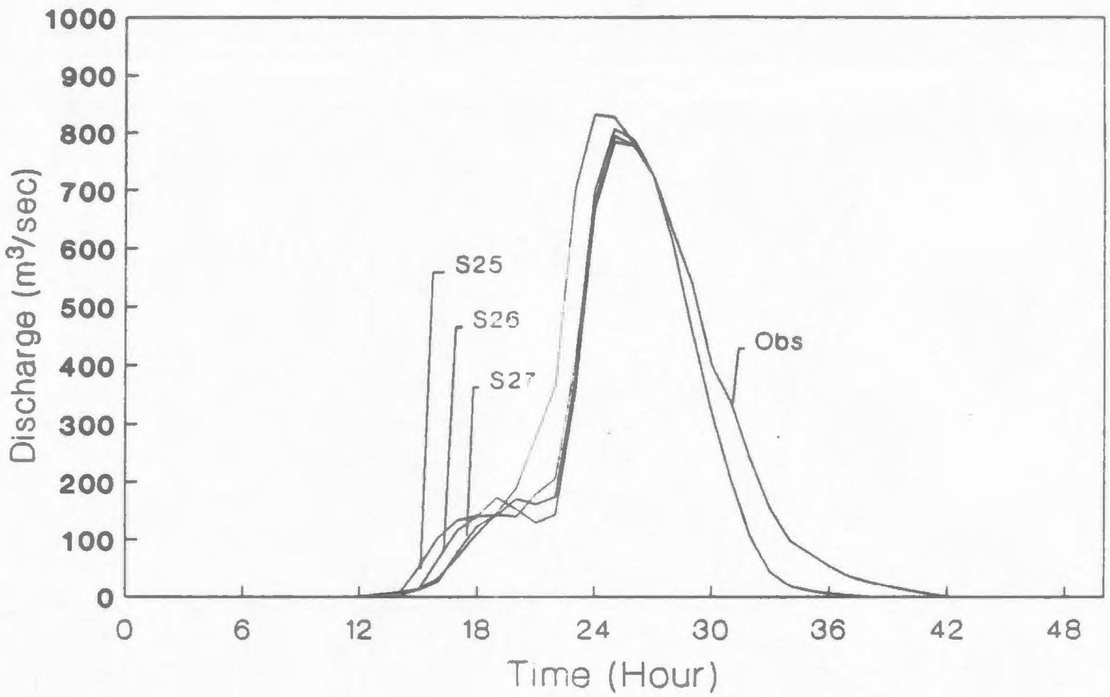
APPENDIX D4





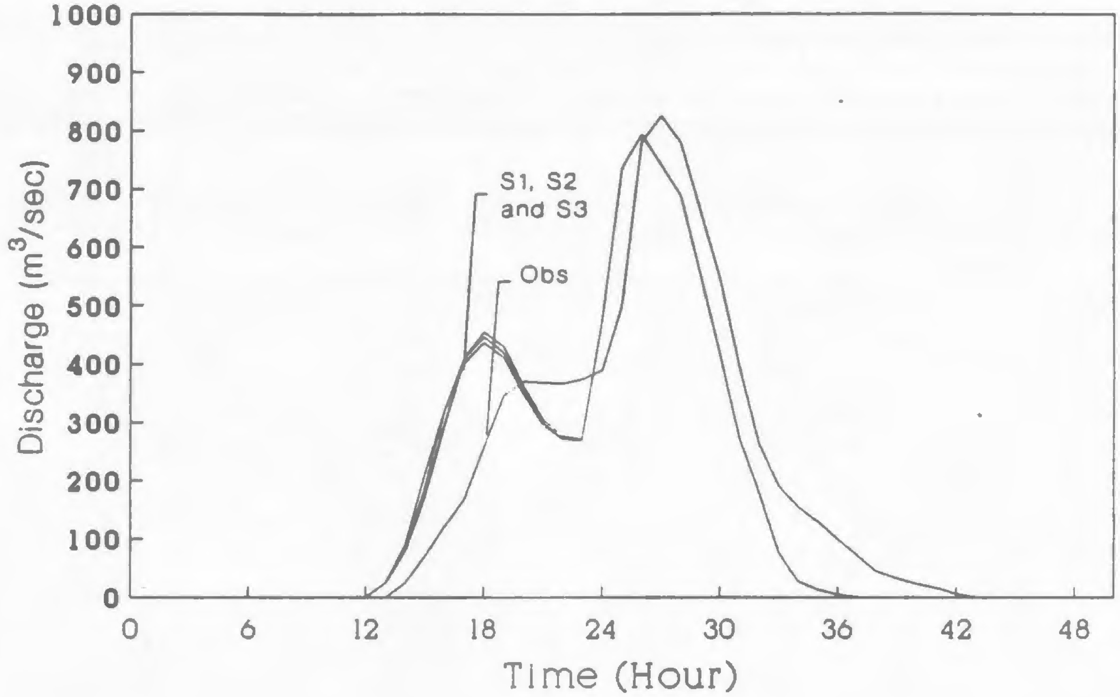
The convoluted hydrograph : April, 6 1986.

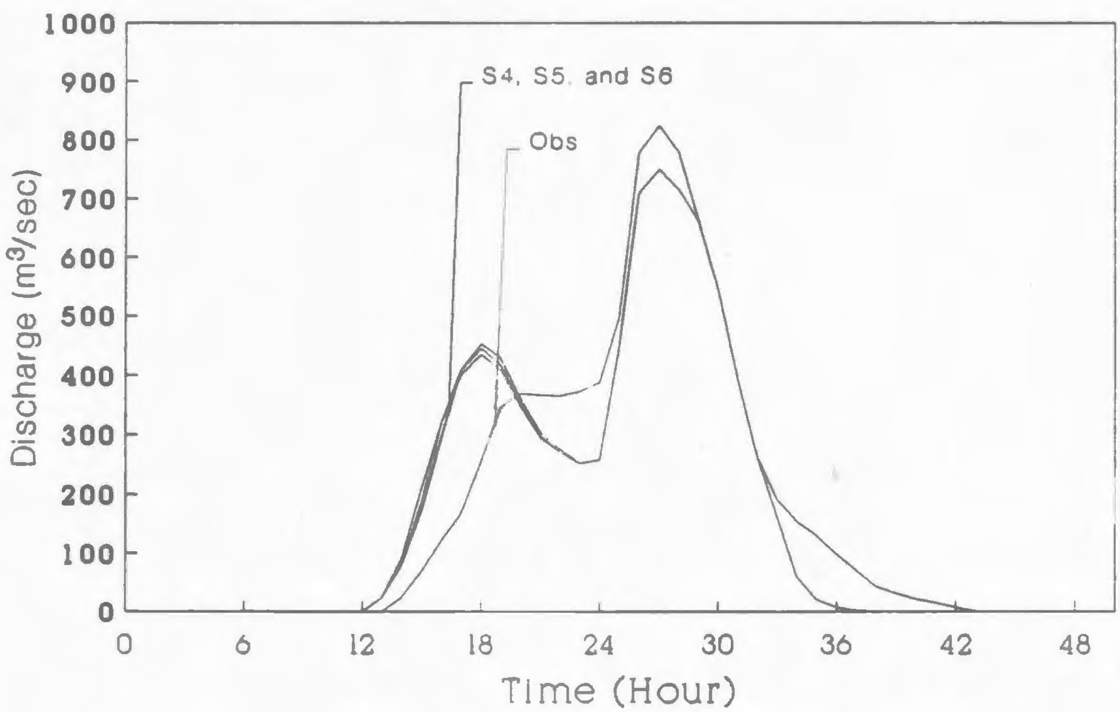
APPENDIX D5



The convoluted hydrograph : April, 6 1986.

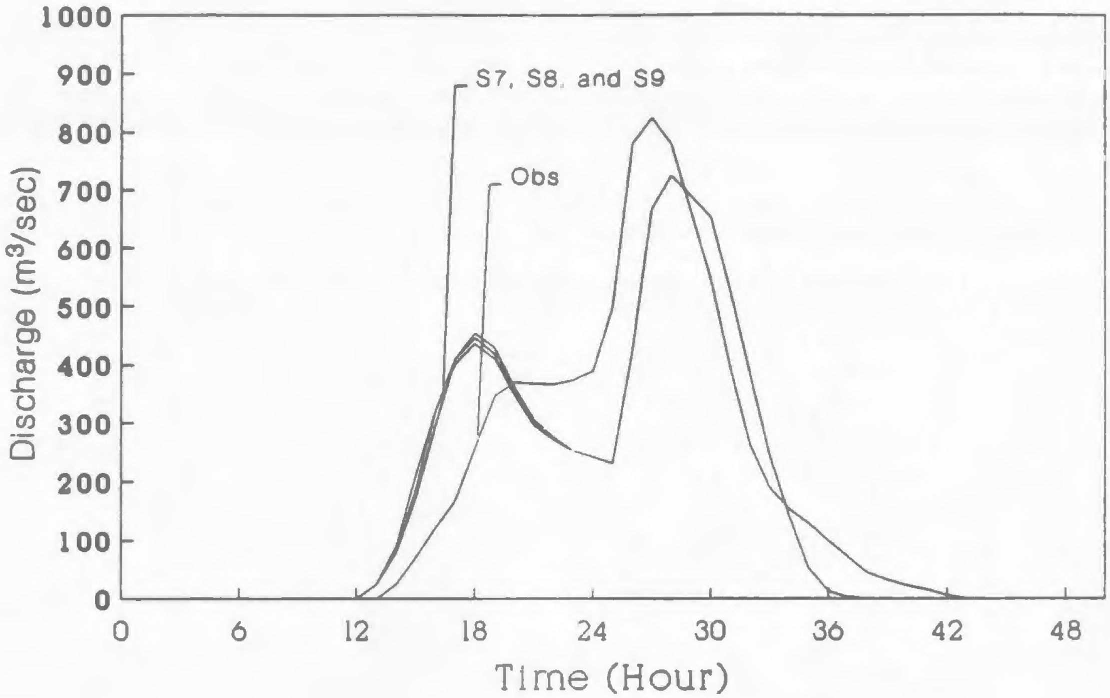
APPENDIX D6

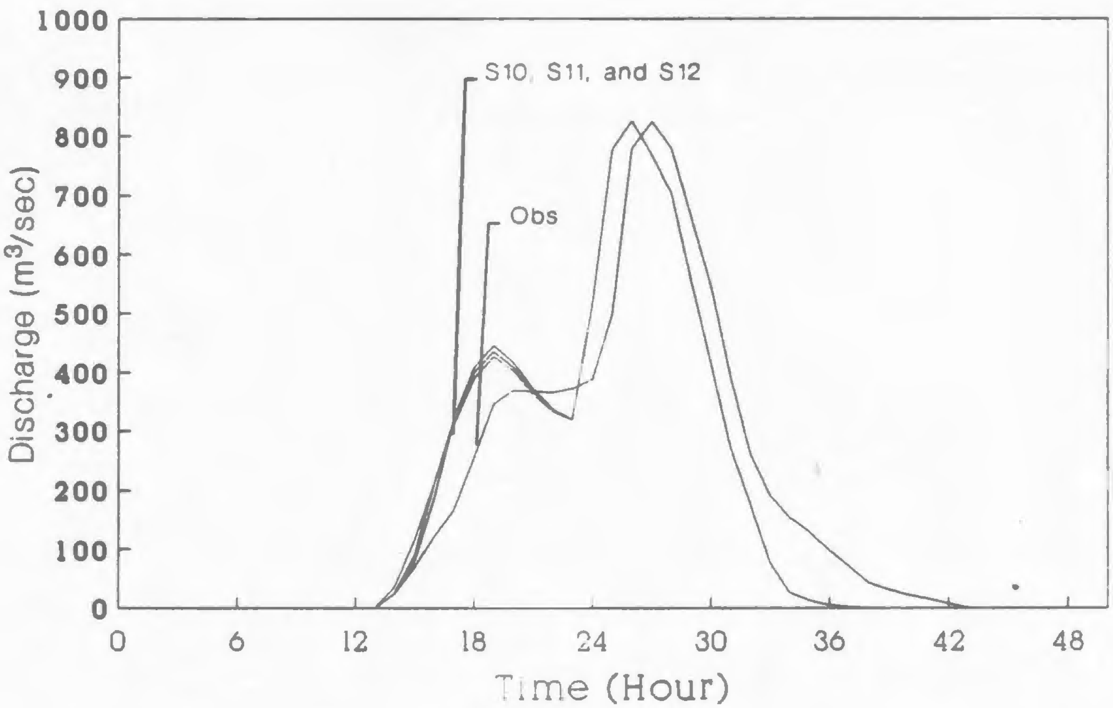




The convoluted hydrograph :February, 18 1983

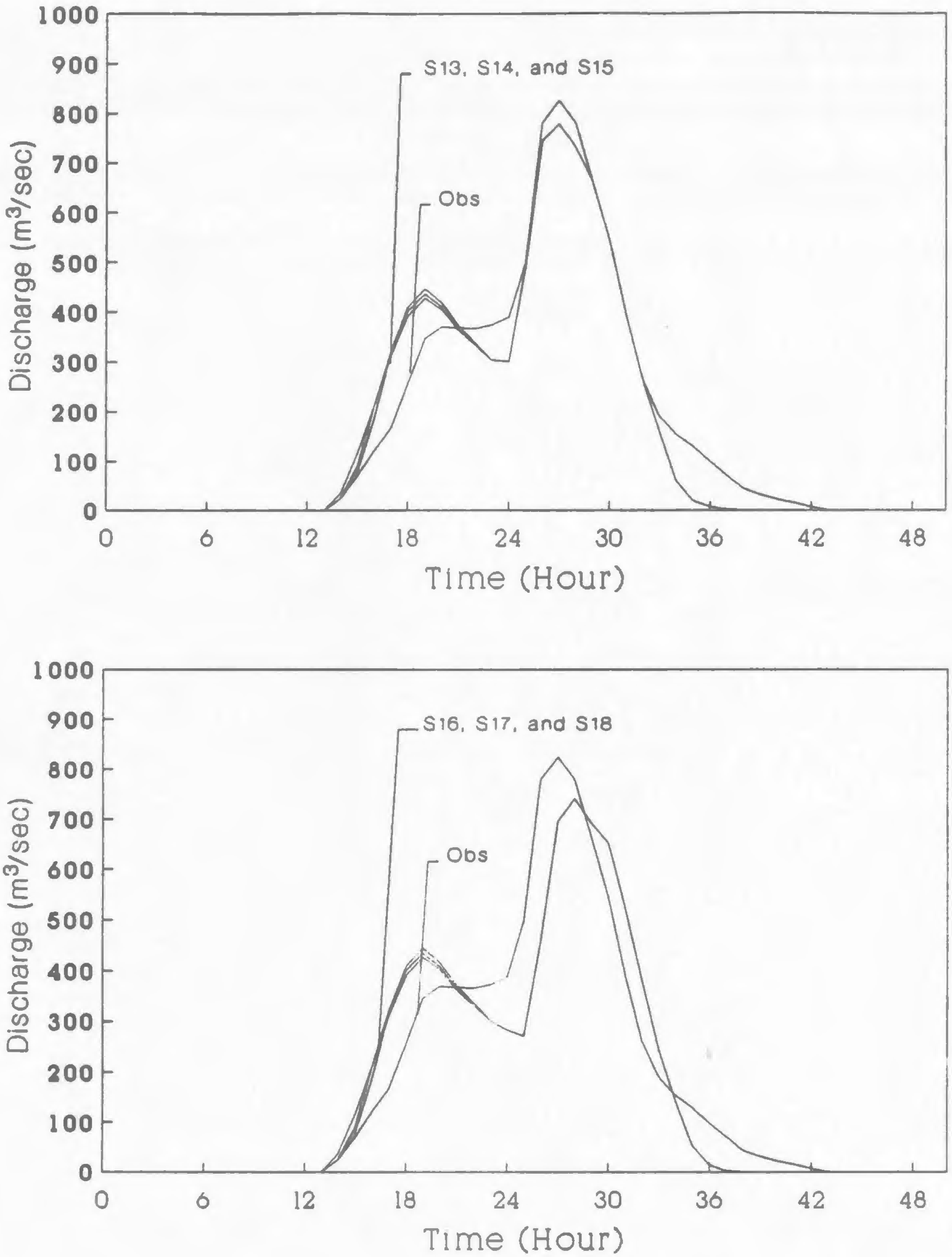
APPENDIX D7





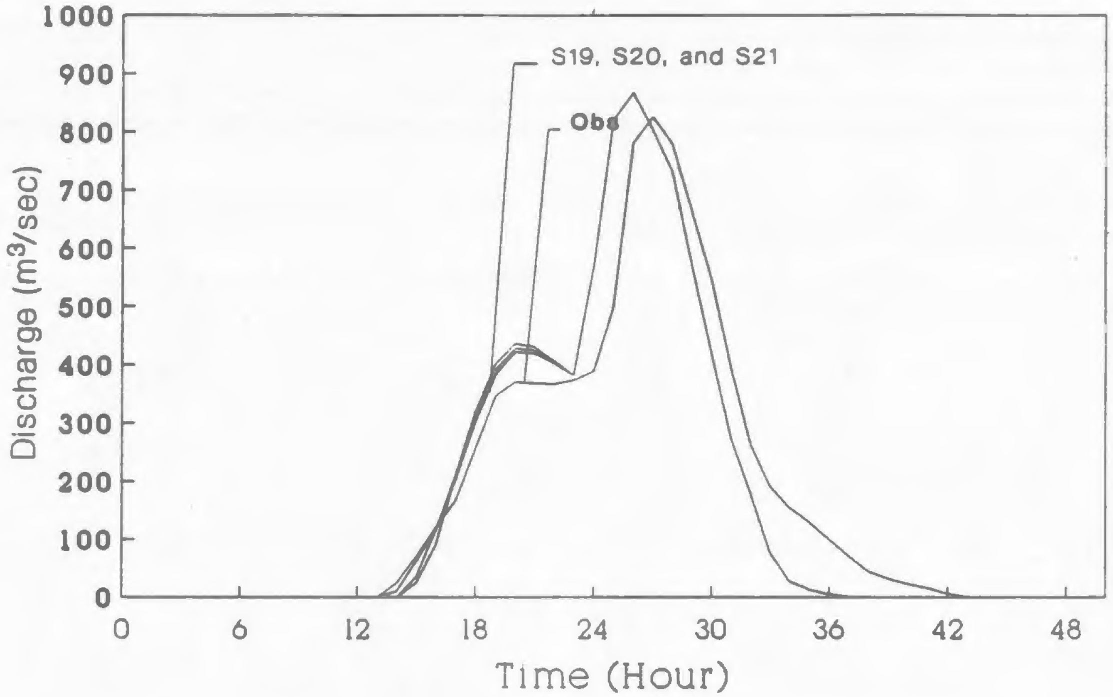
The convoluted hydrograph :February, 18 1983

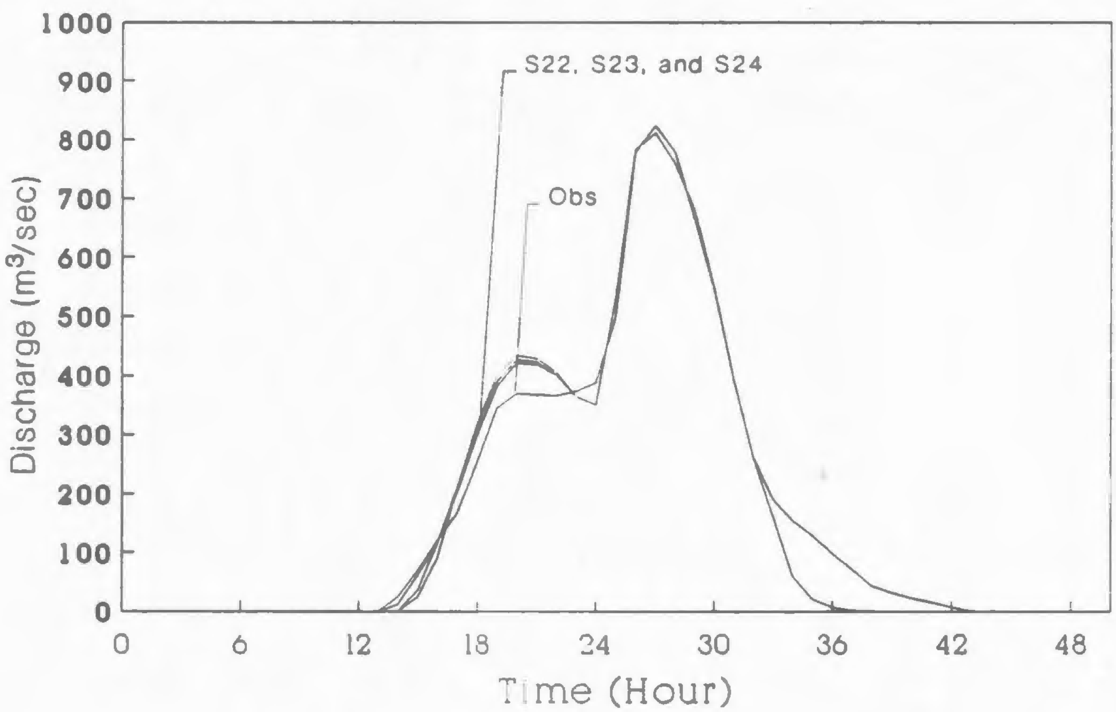
APPENDIX D8



The convoluted hydrograph : February, 18 1983.

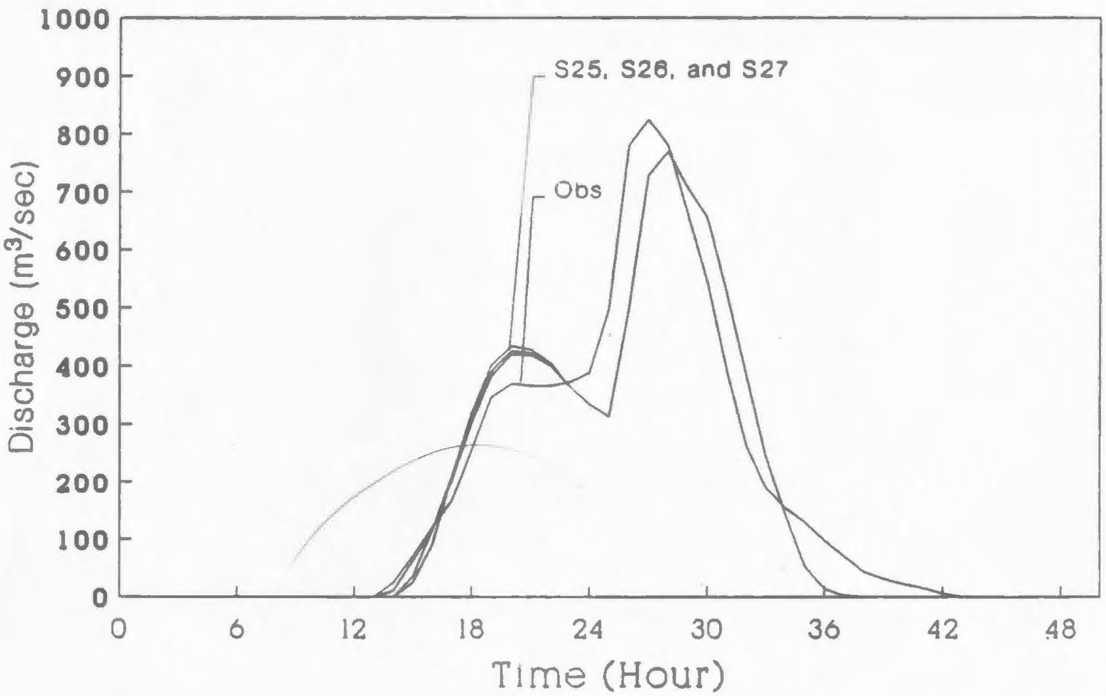
APPENDIX D9





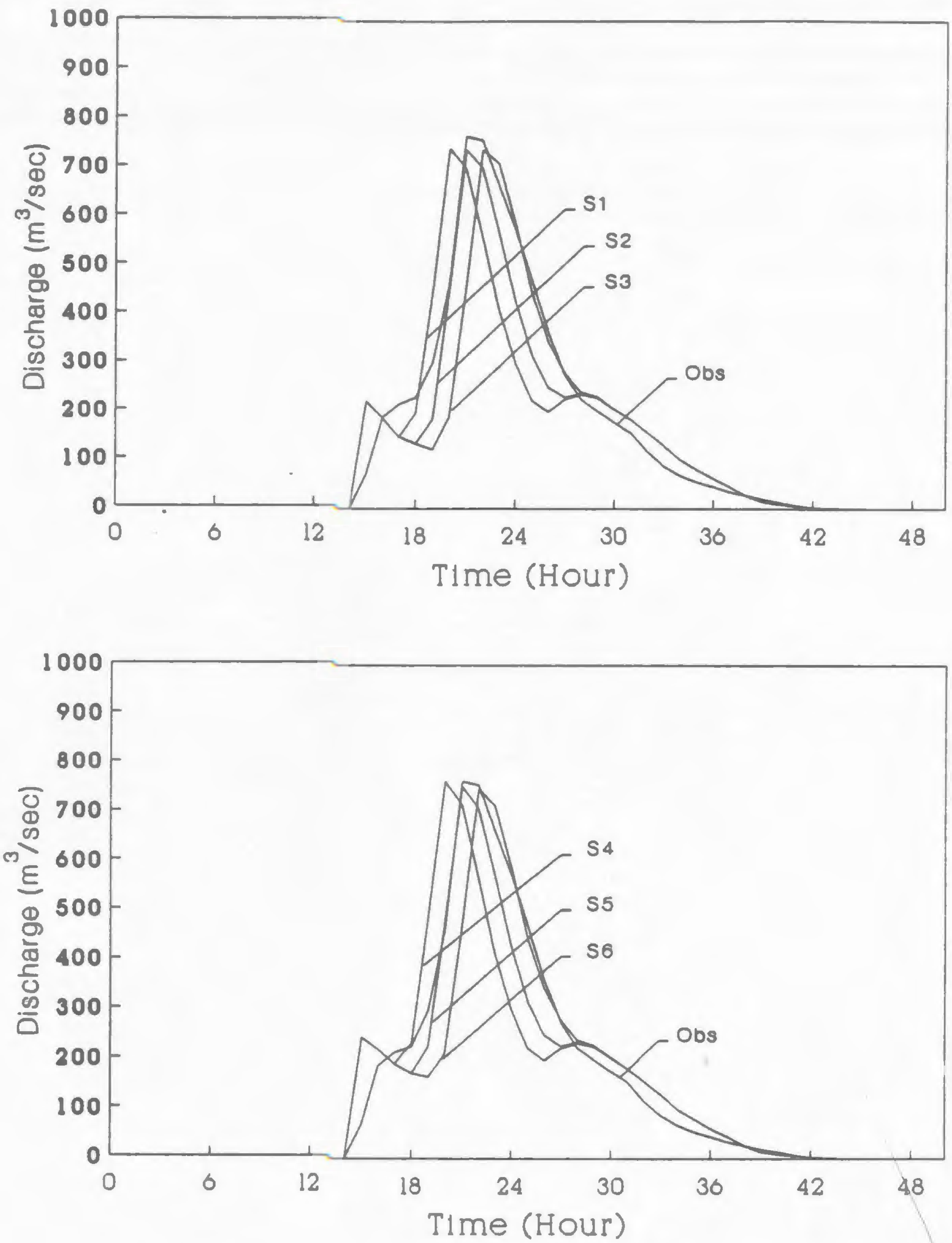
The convoluted hydrograph : February, 18 1983.

APPENDIX D10



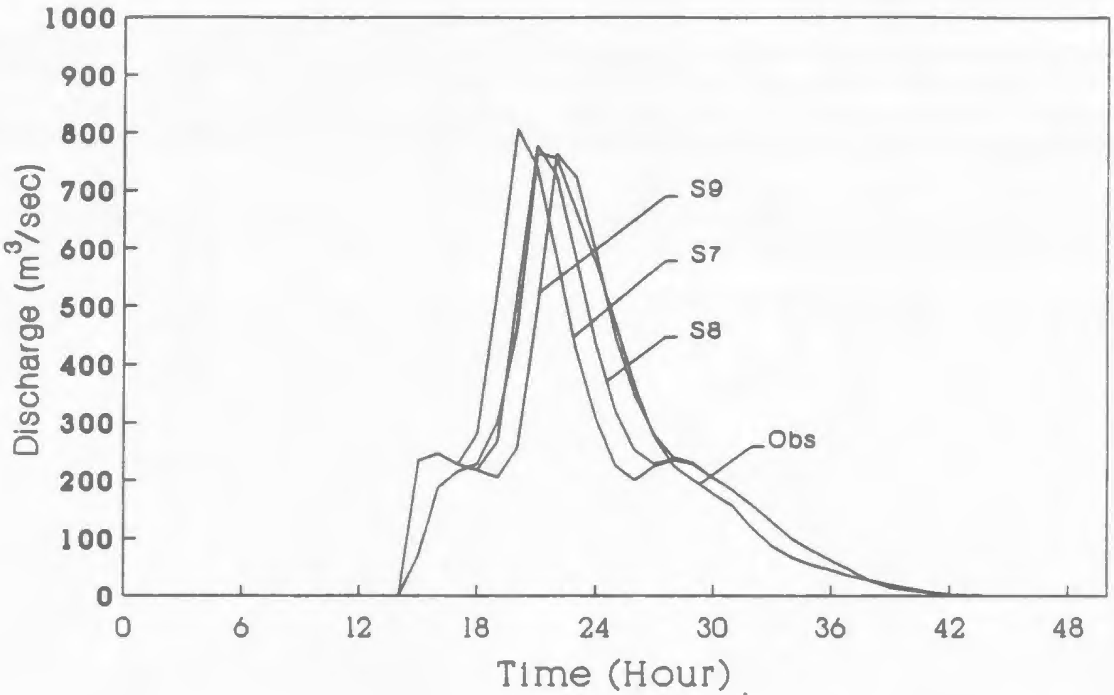
The convoluted hydrograph : February, 18 1983.

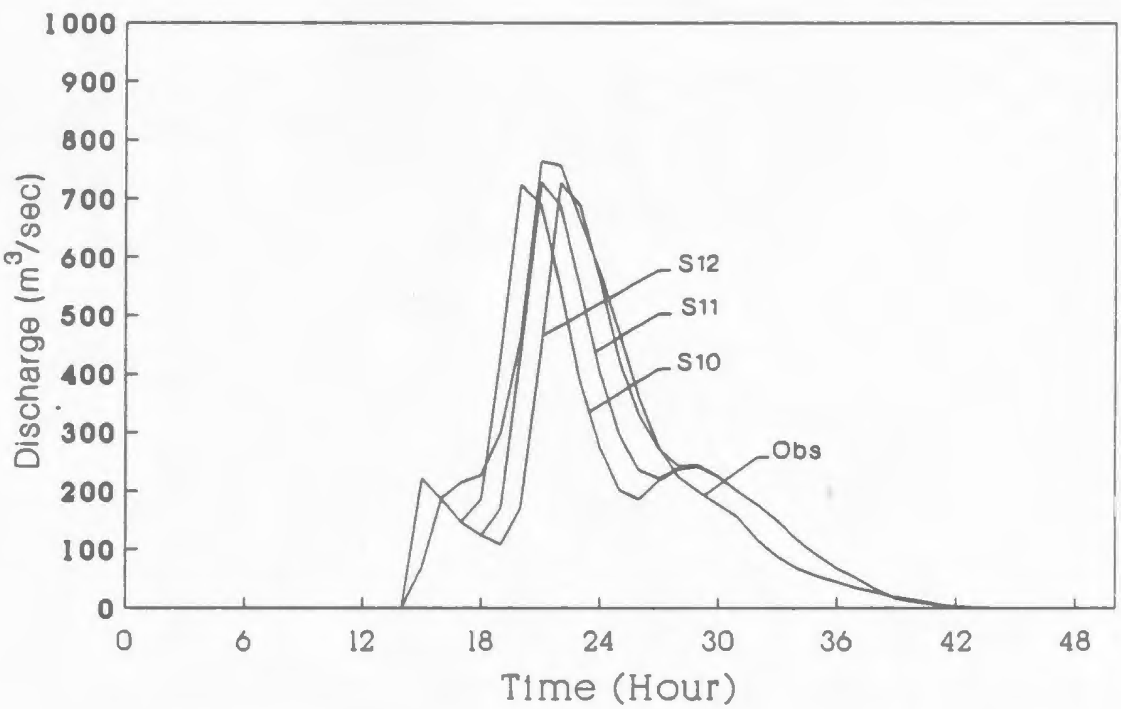
APPENDIX D11



The convoluted hydrograph : March, 30 1985.

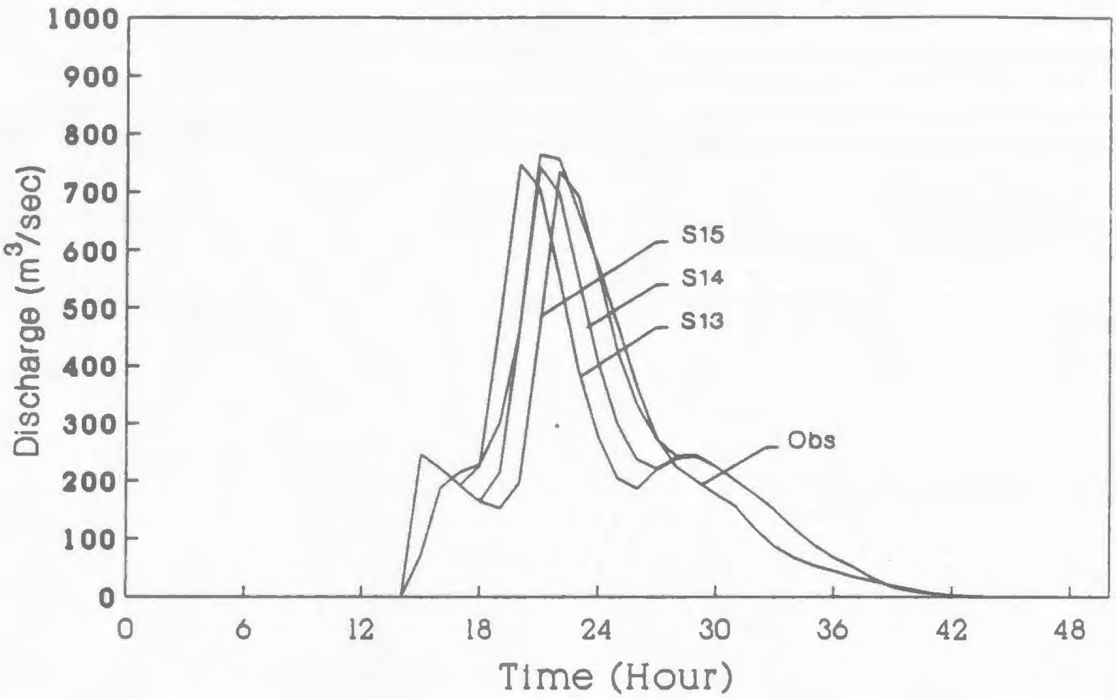
APPENDIX D12

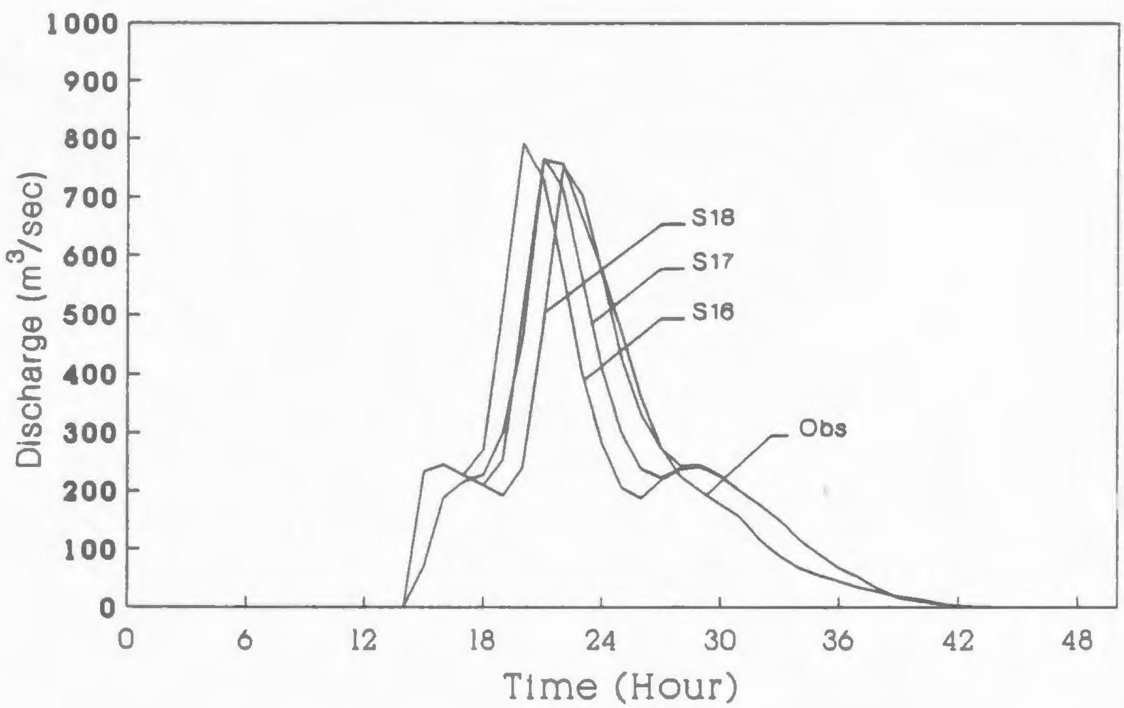




The convoluted hydrograph : March, 30 1985.

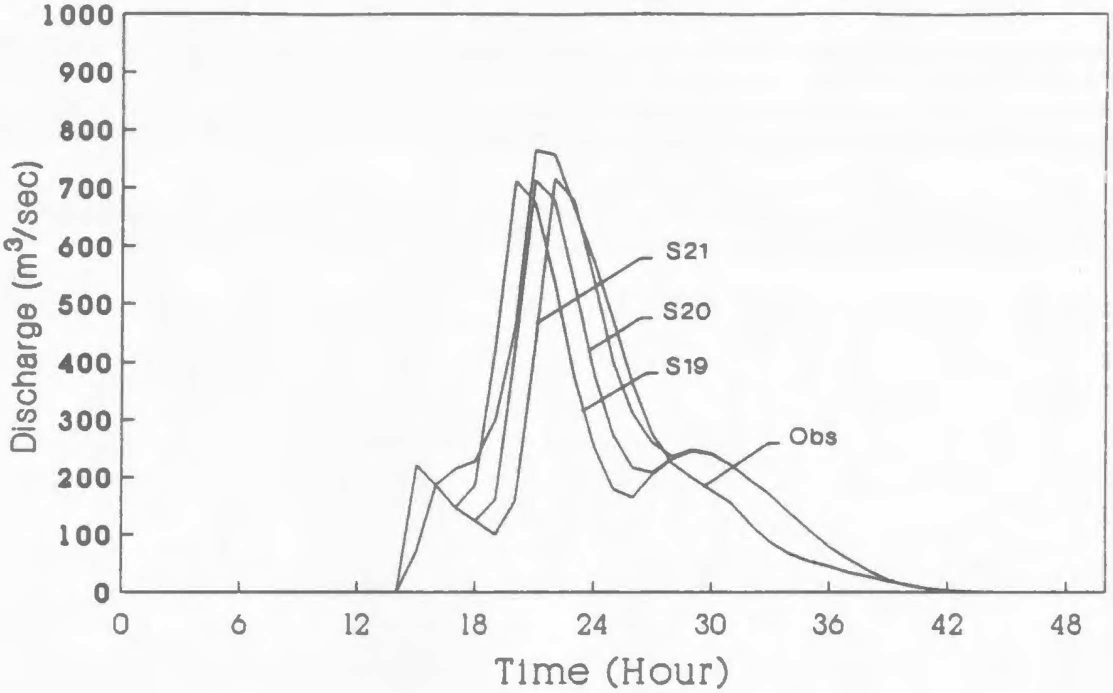
APPENDIX D13

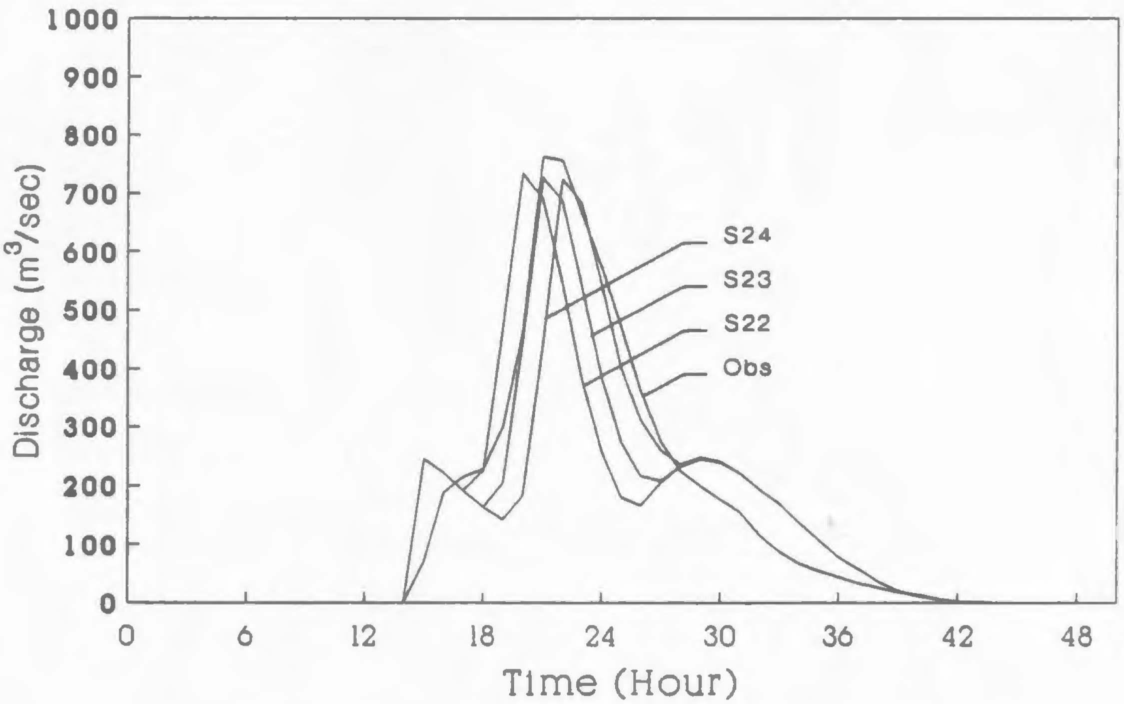




The convoluted hydrograph : March, 30 1985.

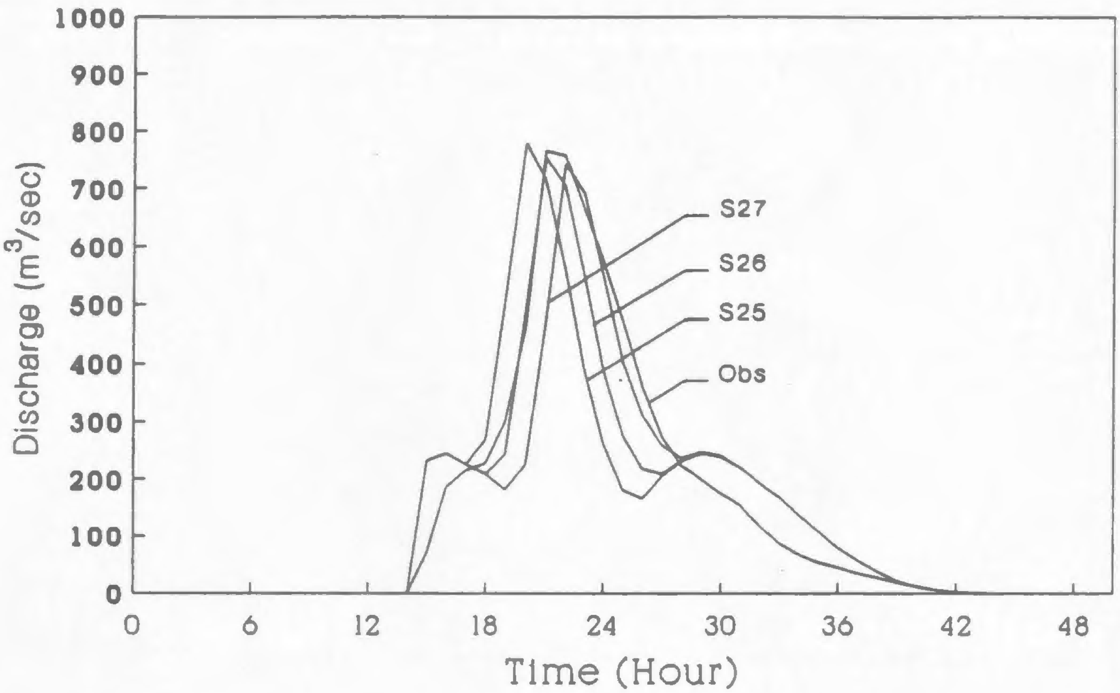
APPENDIX D14





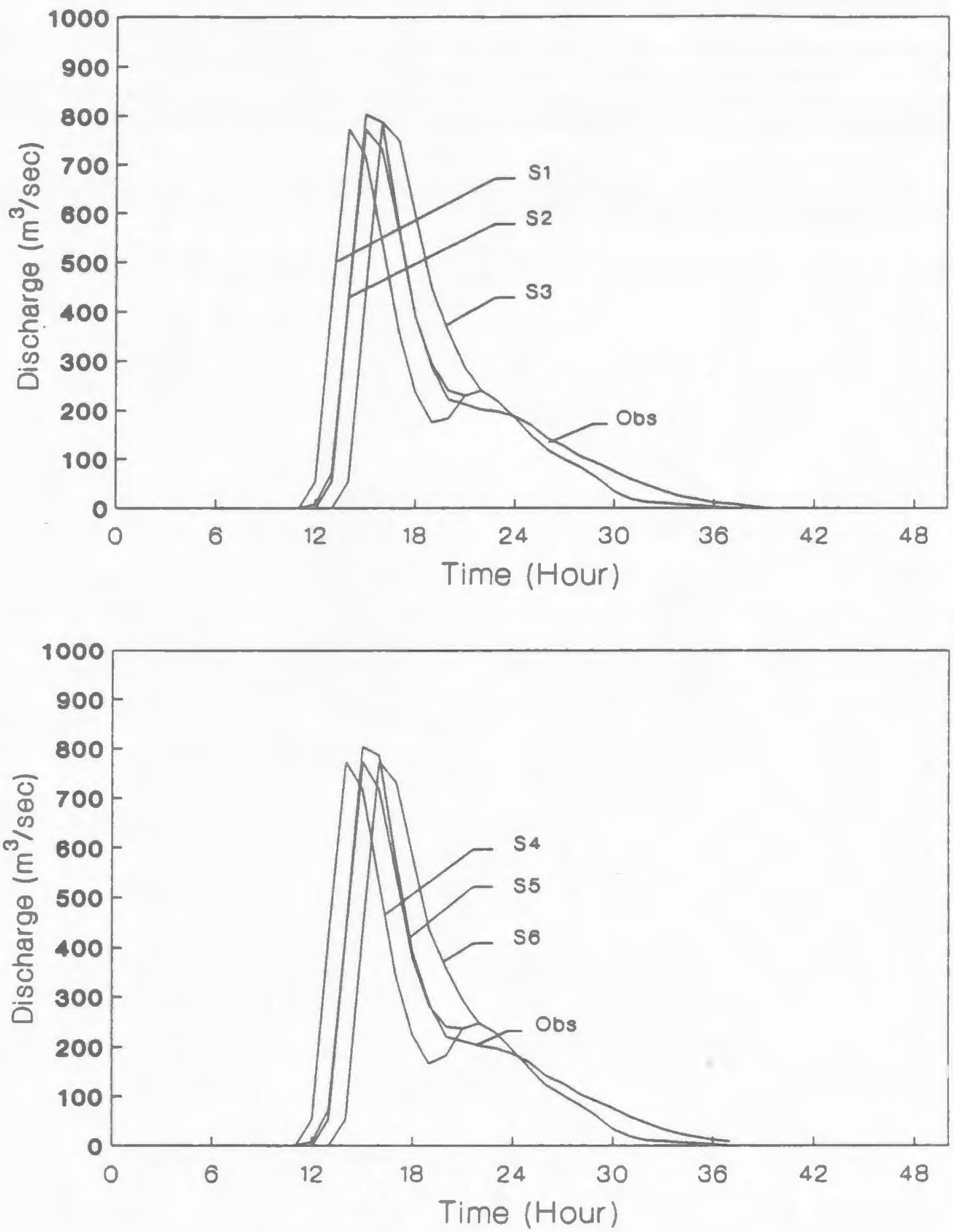
The convoluted hydrograph : March, 30 1985.

APPENDIX D15



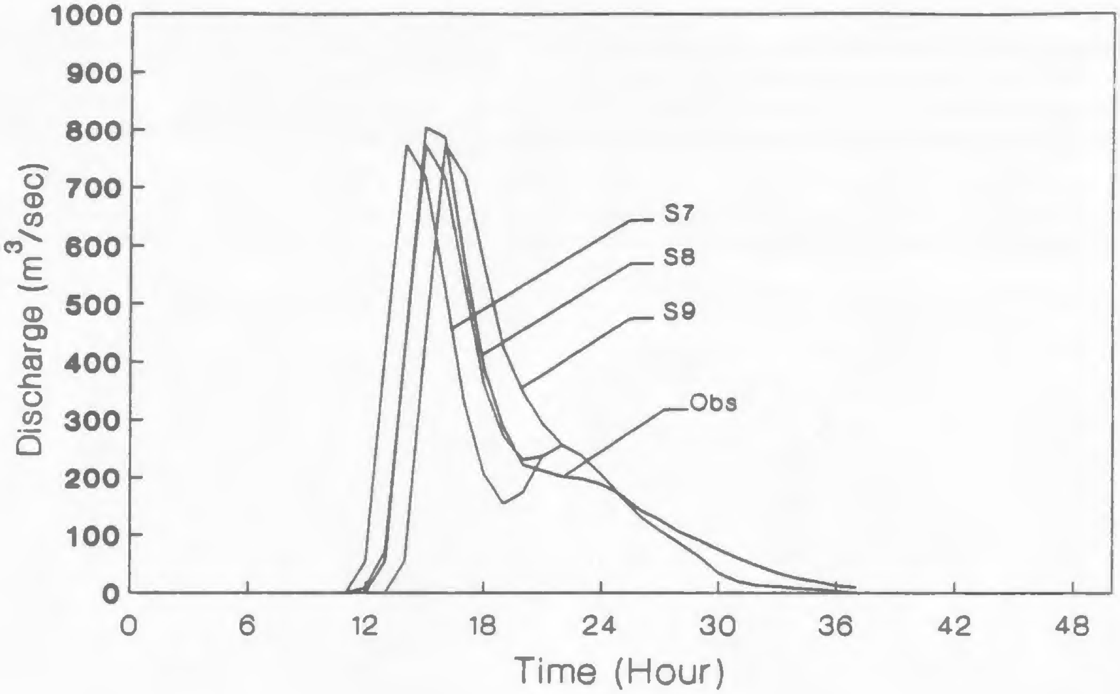
The convoluted hydrograph : March, 30 1985.

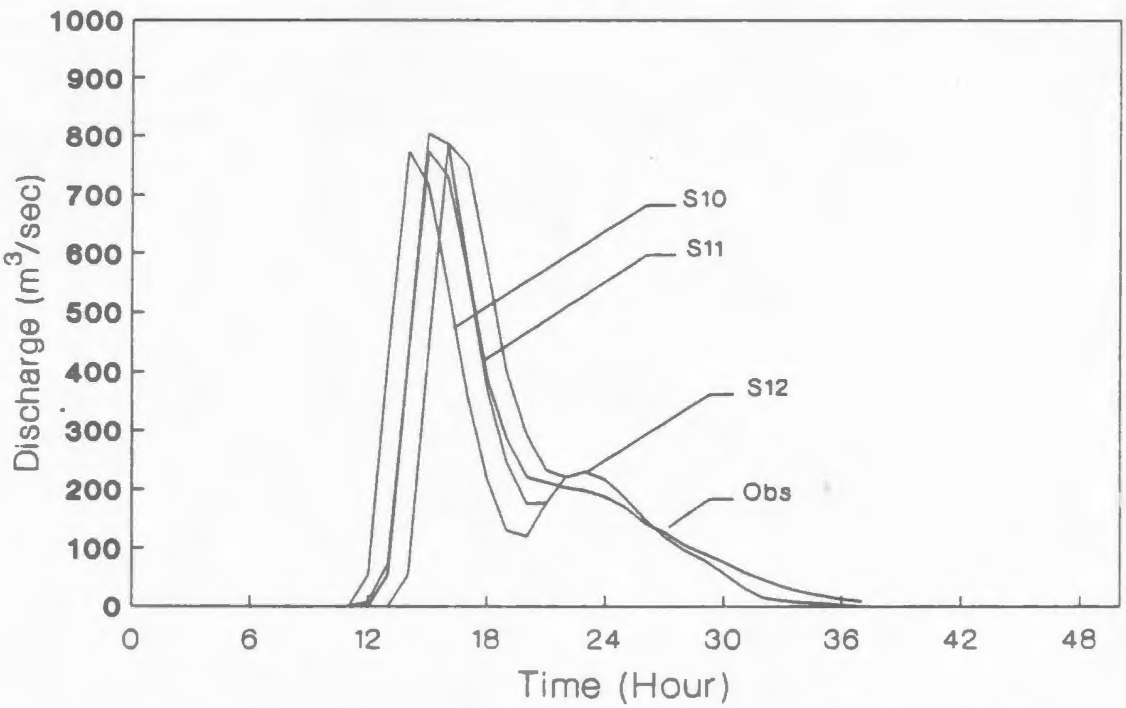
APPENDIX D16



The convoluted hydrograph : March, 5 1989.

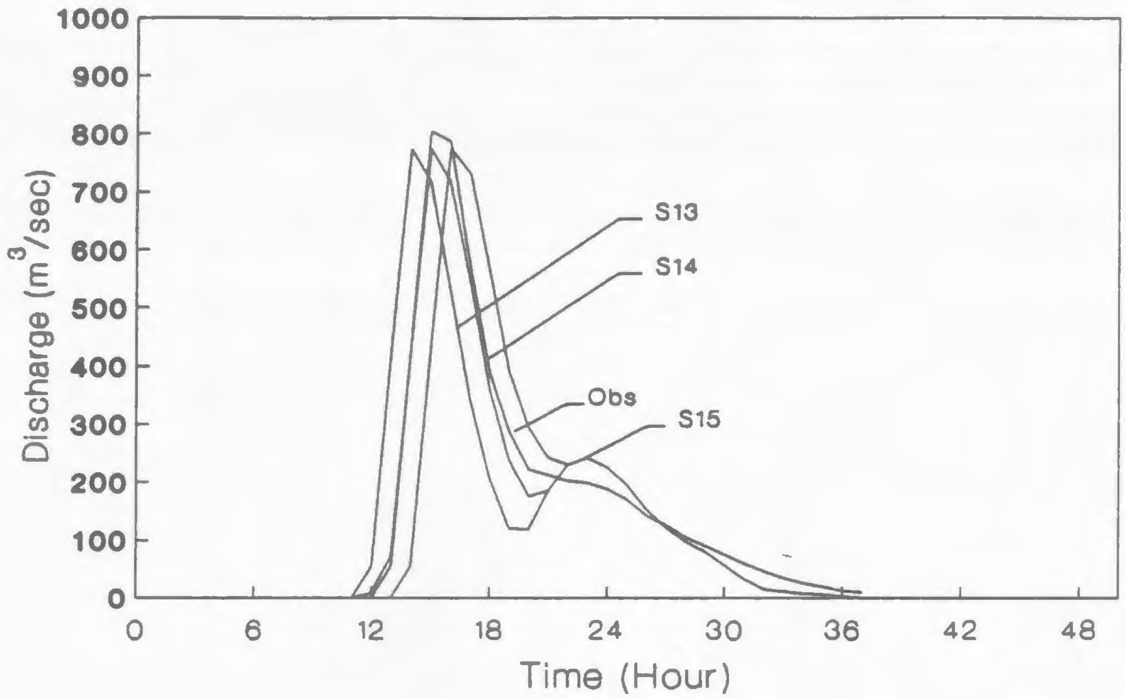
APPENDIX D17

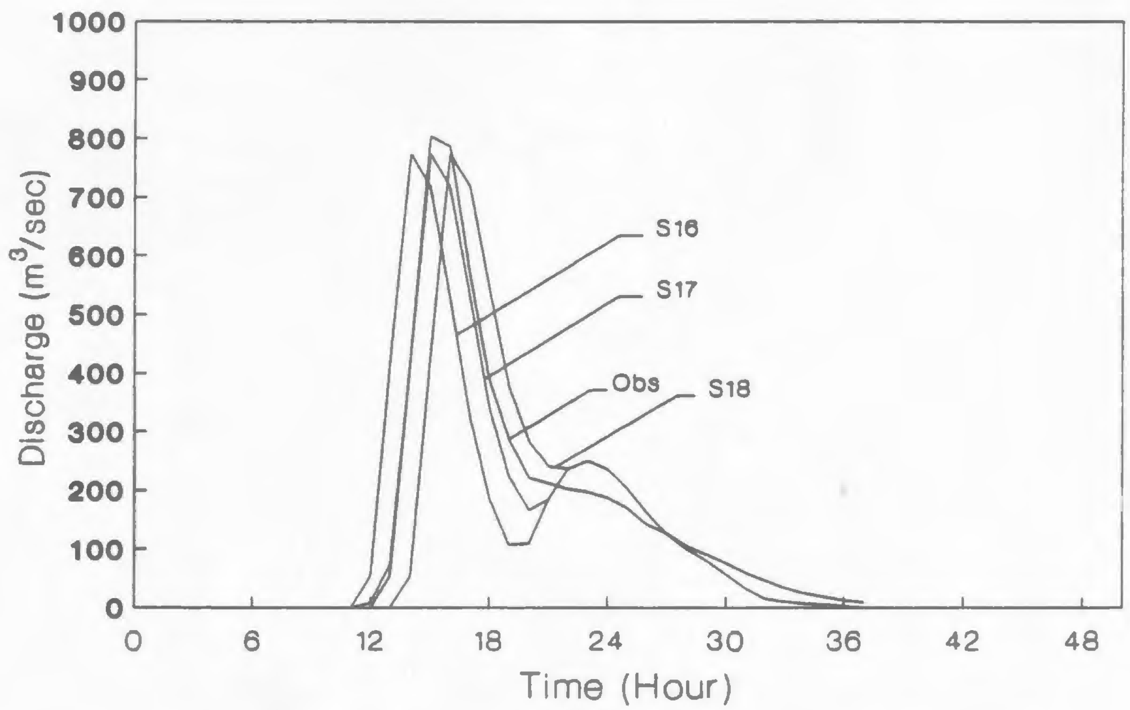




The convoluted hydrograph : March, 5 1989.

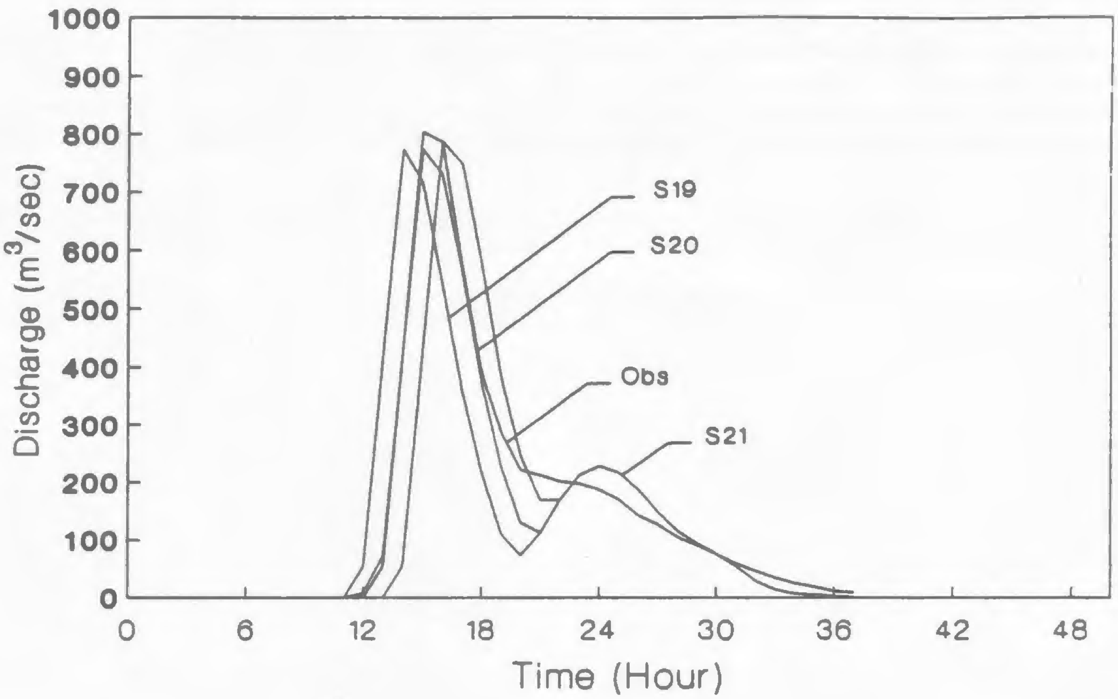
APPENDIX D18

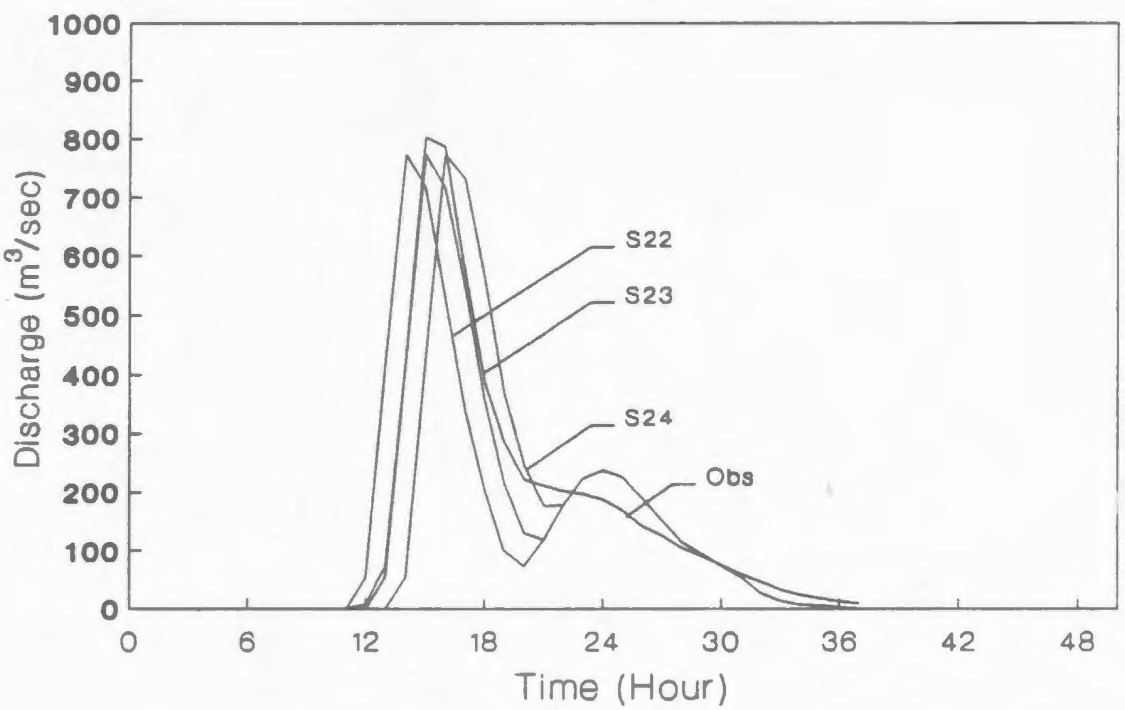




The convoluted hydrograph : March, 5 1989.

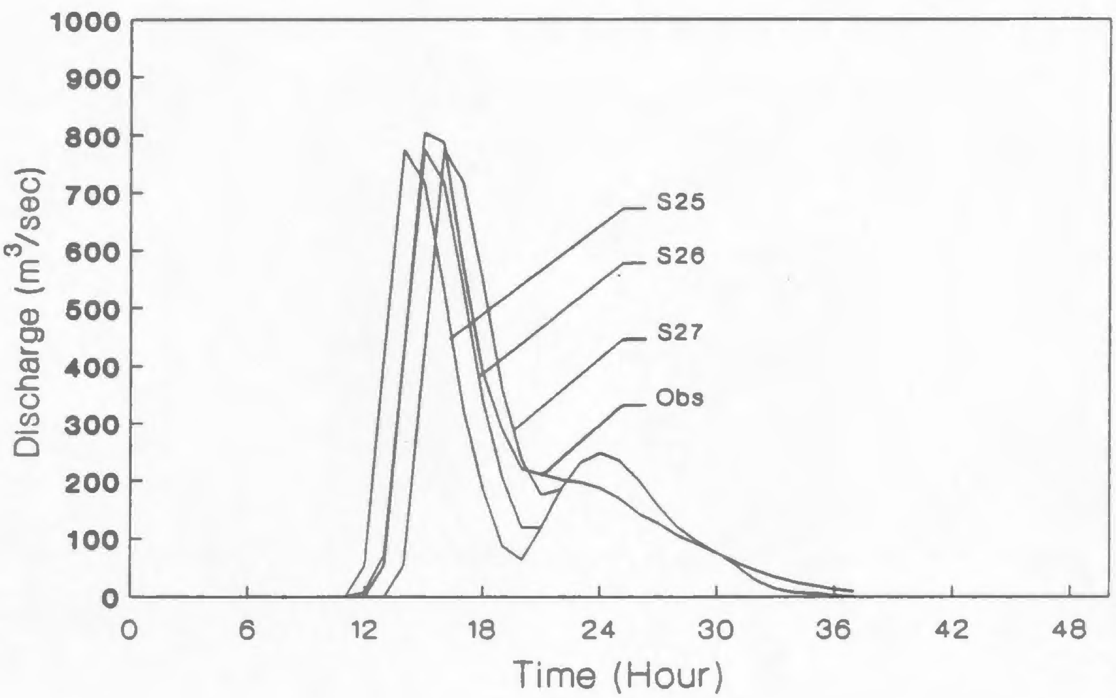
APPENDIX D19





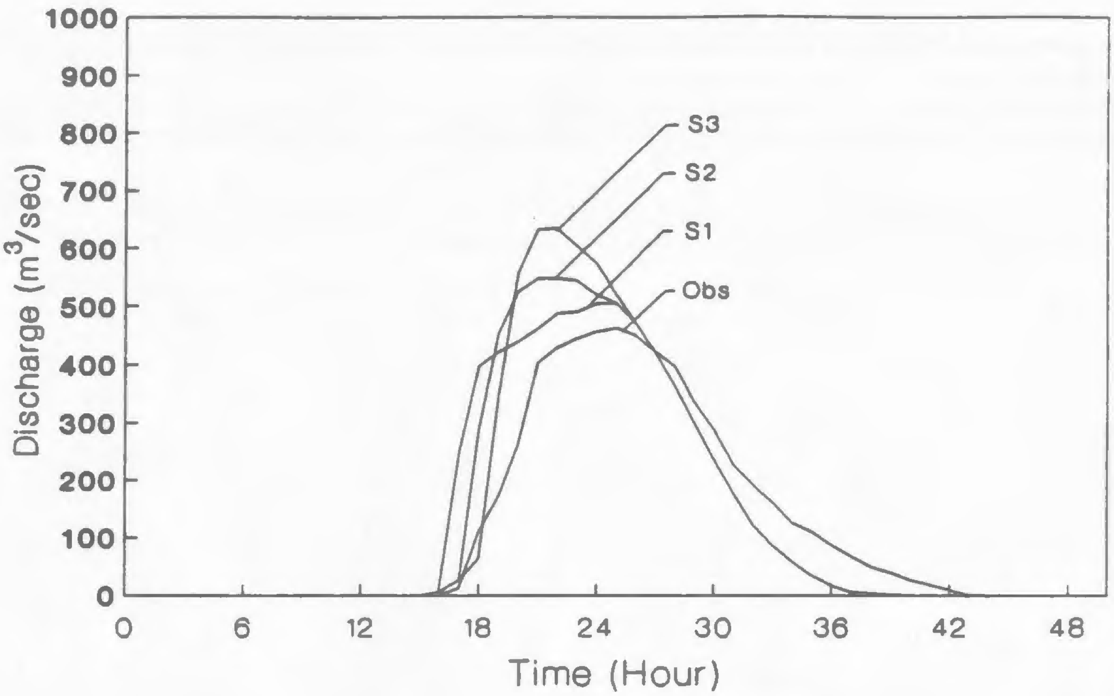
The convoluted hydrograph : March, 5 1989.

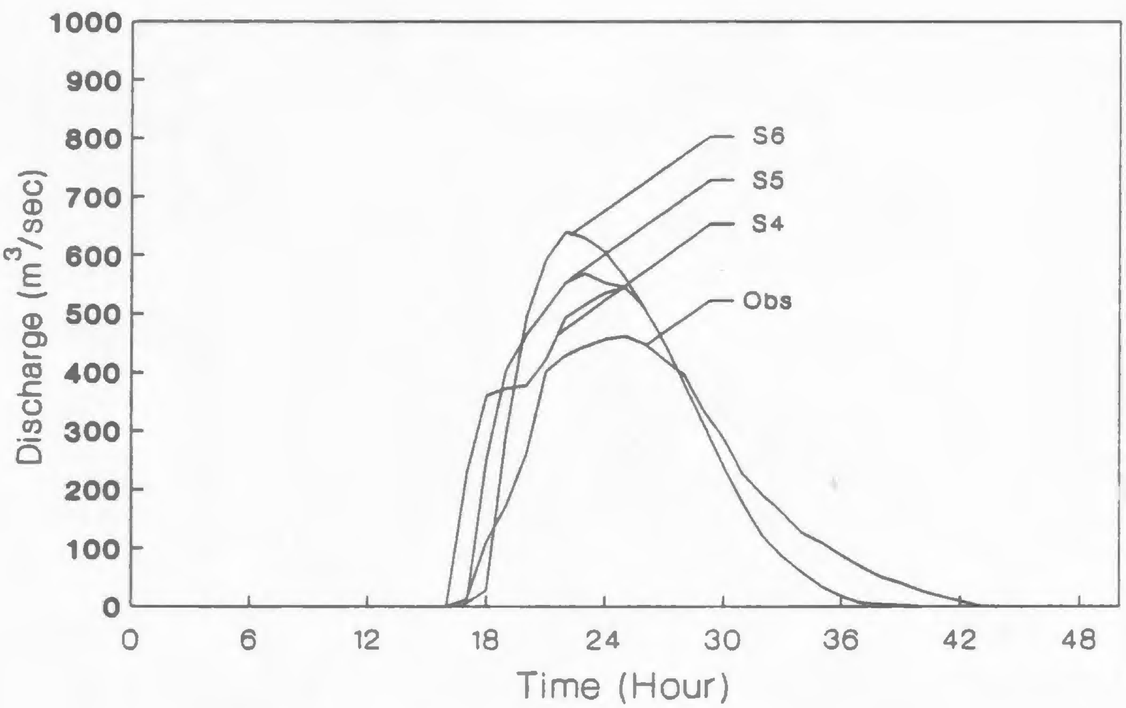
APPENDIX D20



The convoluted hydrograph : March, 5 1989.

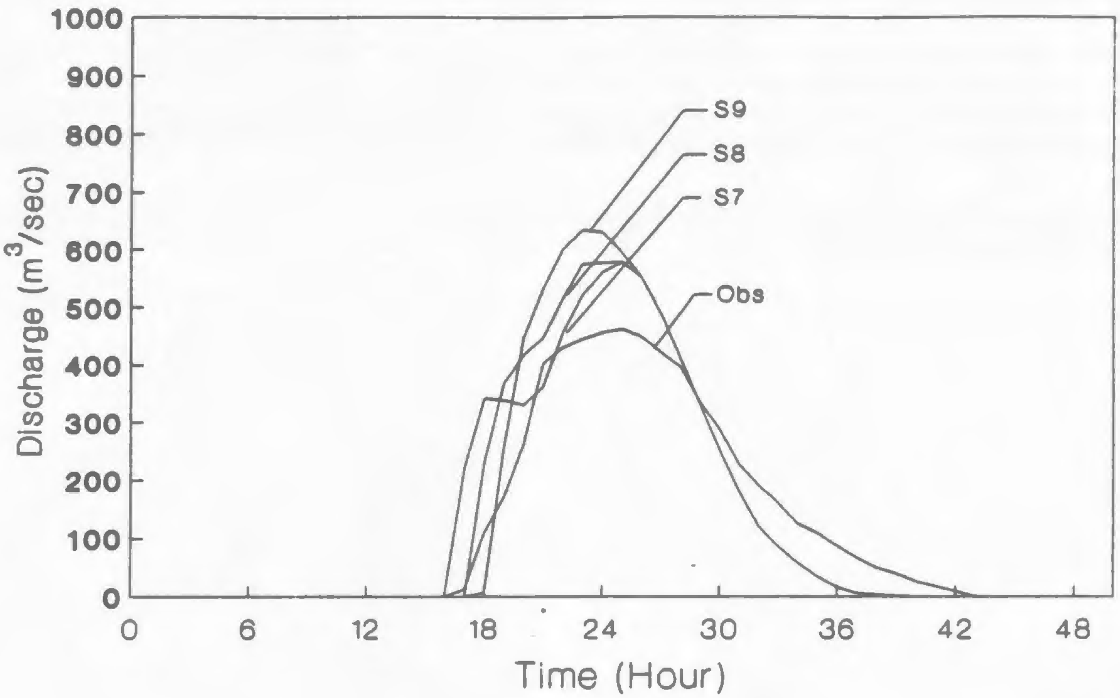
APPENDIX D21

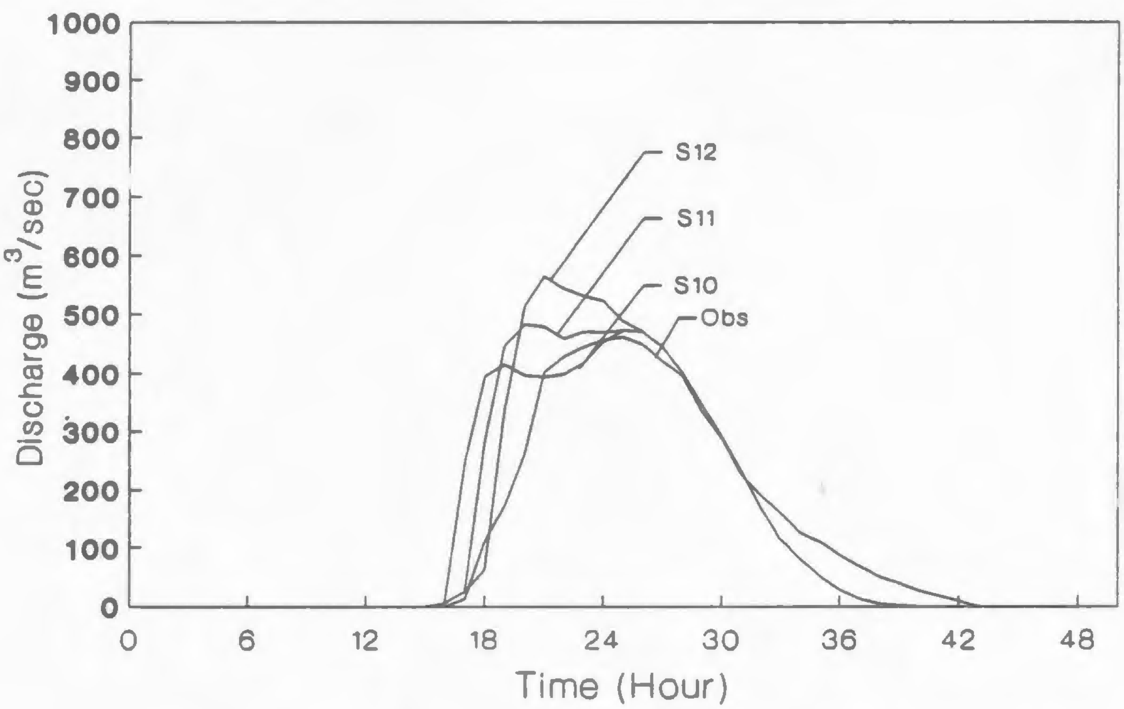




The convoluted hydrograph : November, 16 1979.

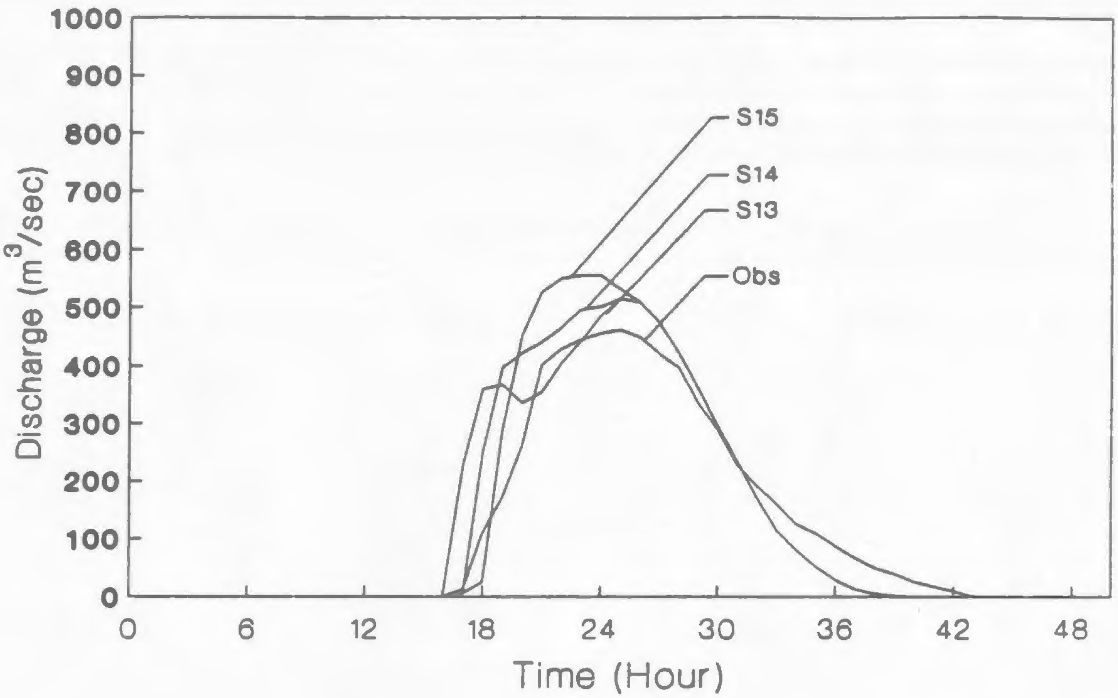
APPENDIX D22

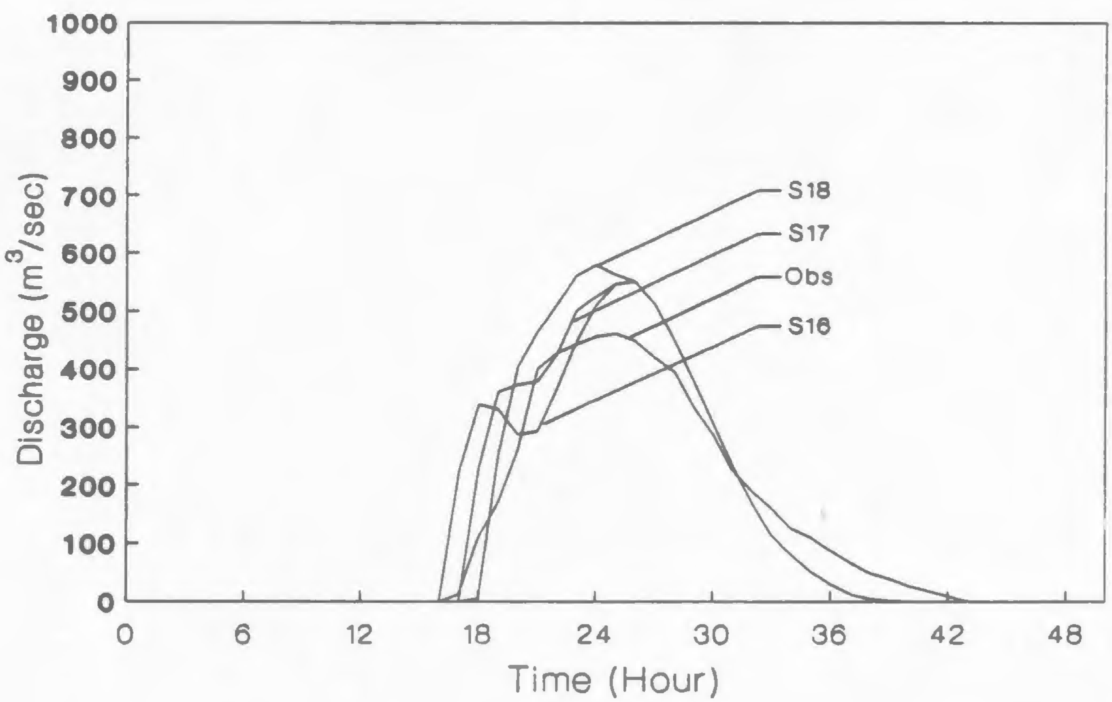




The convoluted hydrograph : November, 16 1979.

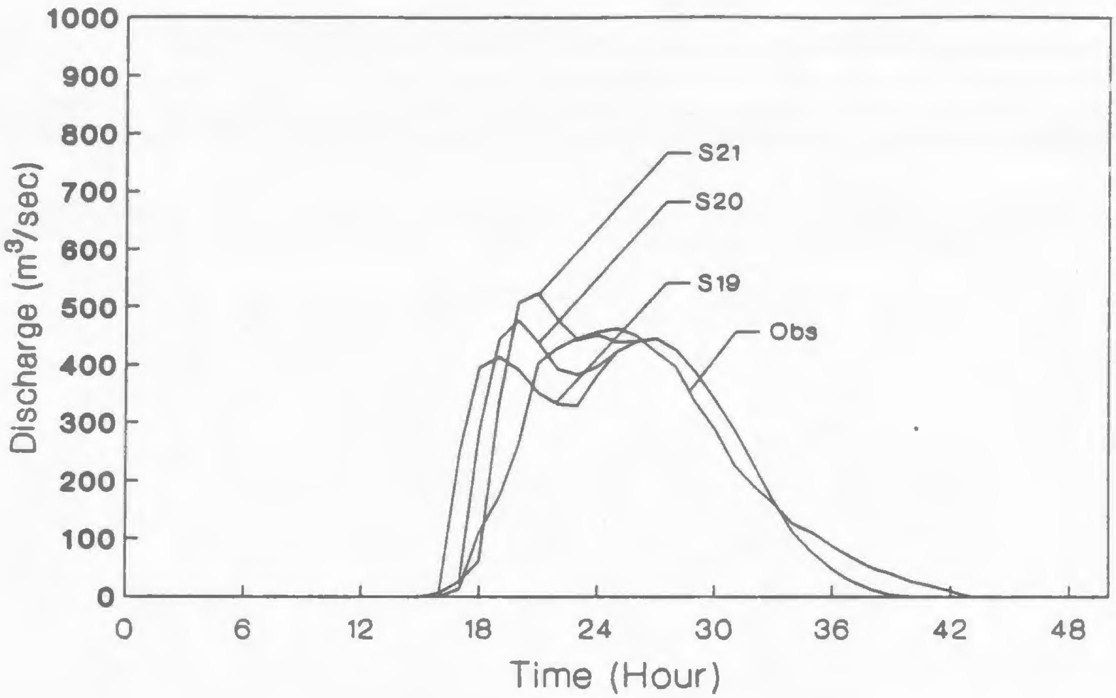
APPENDIX D23

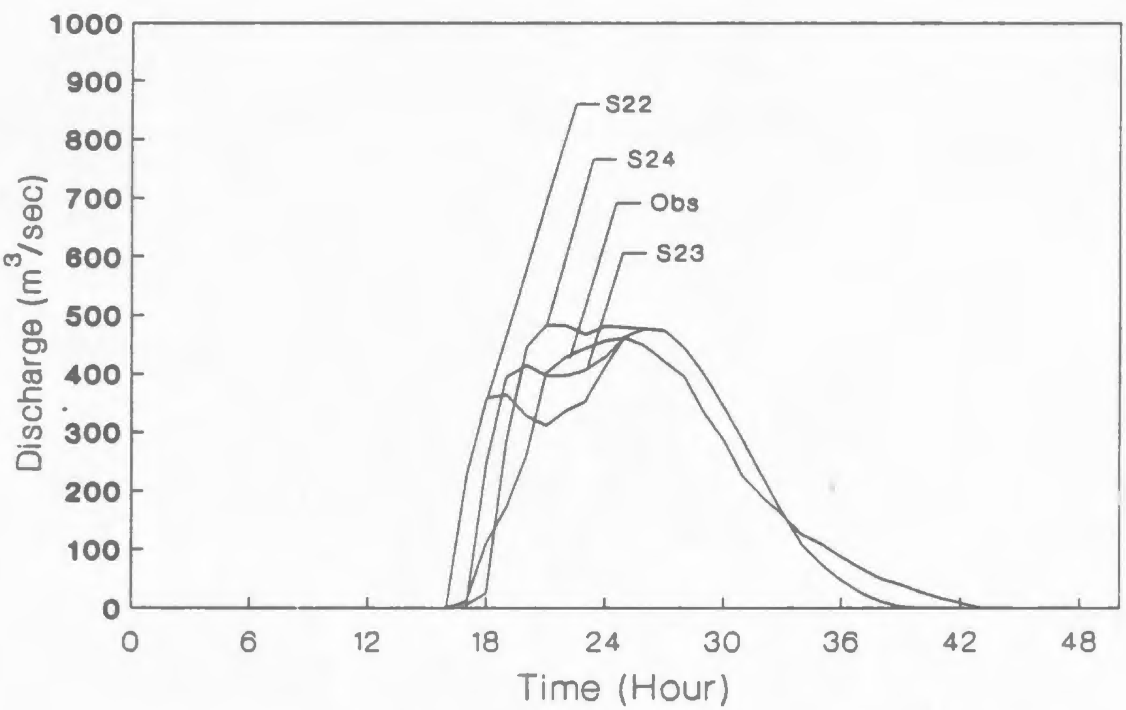




The convoluted hydrograph : November, 16 1979.

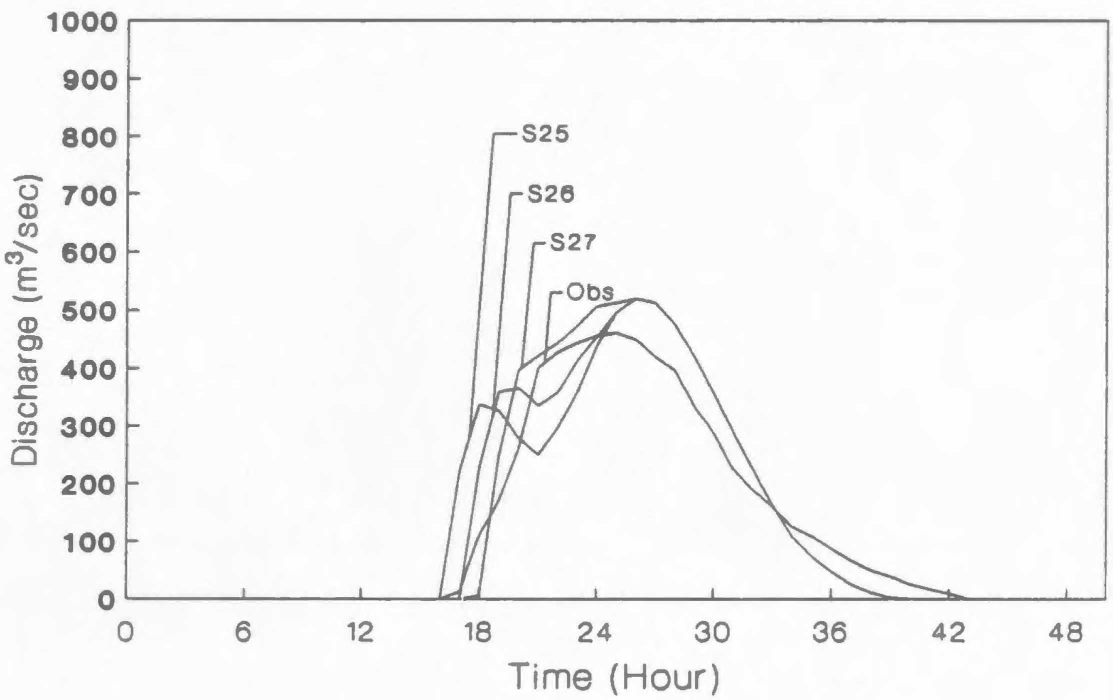
APPENDIX D24





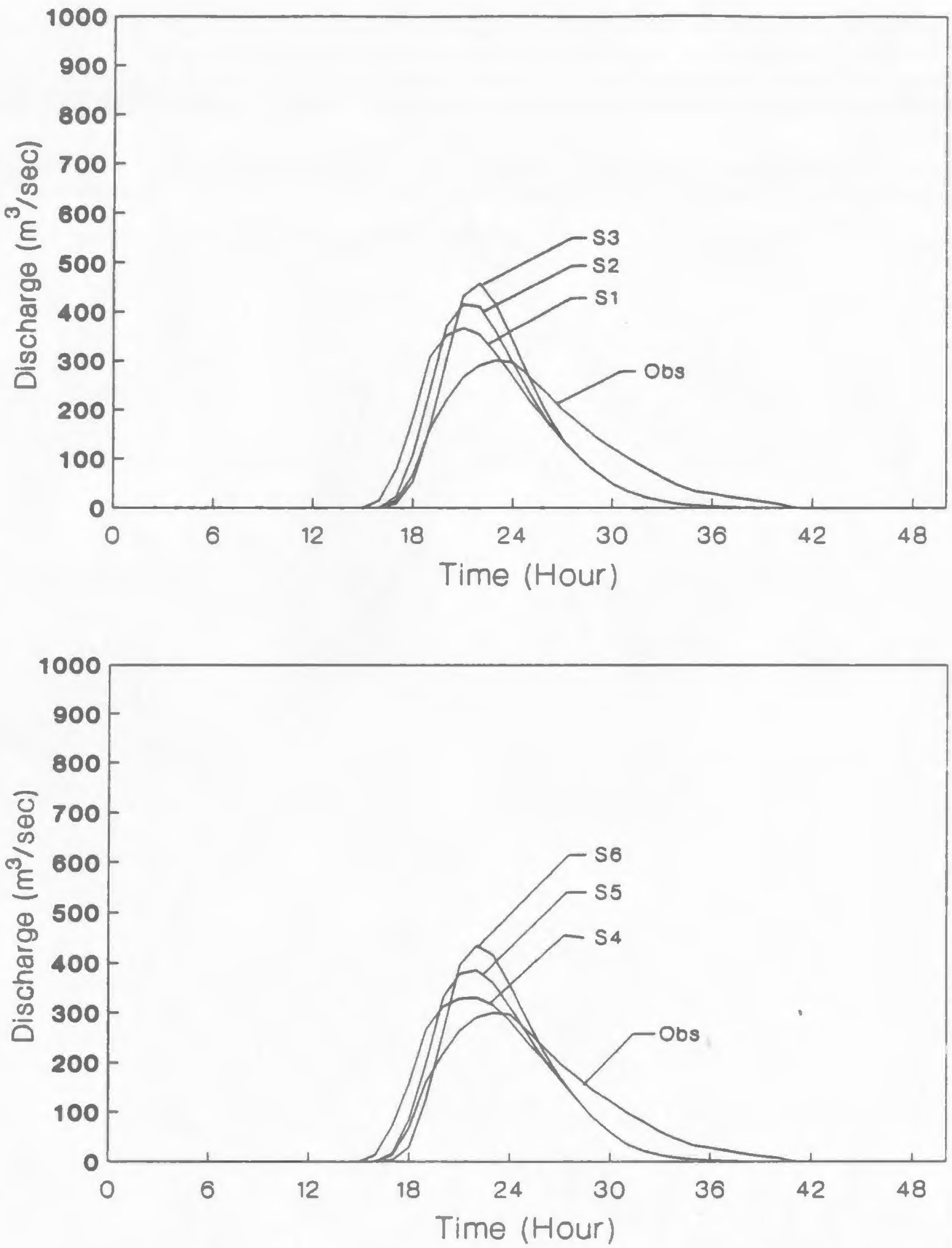
The convoluted hydrograph : November, 16 1979.

APPENDIX D25



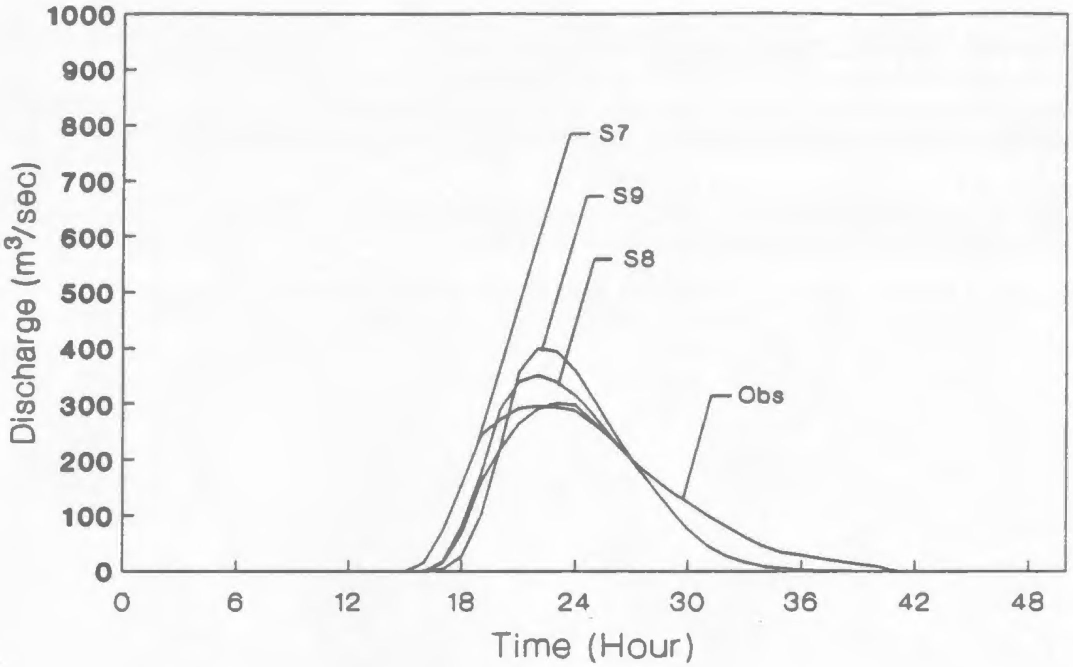
The convoluted hydrograph :November, 16 1979.

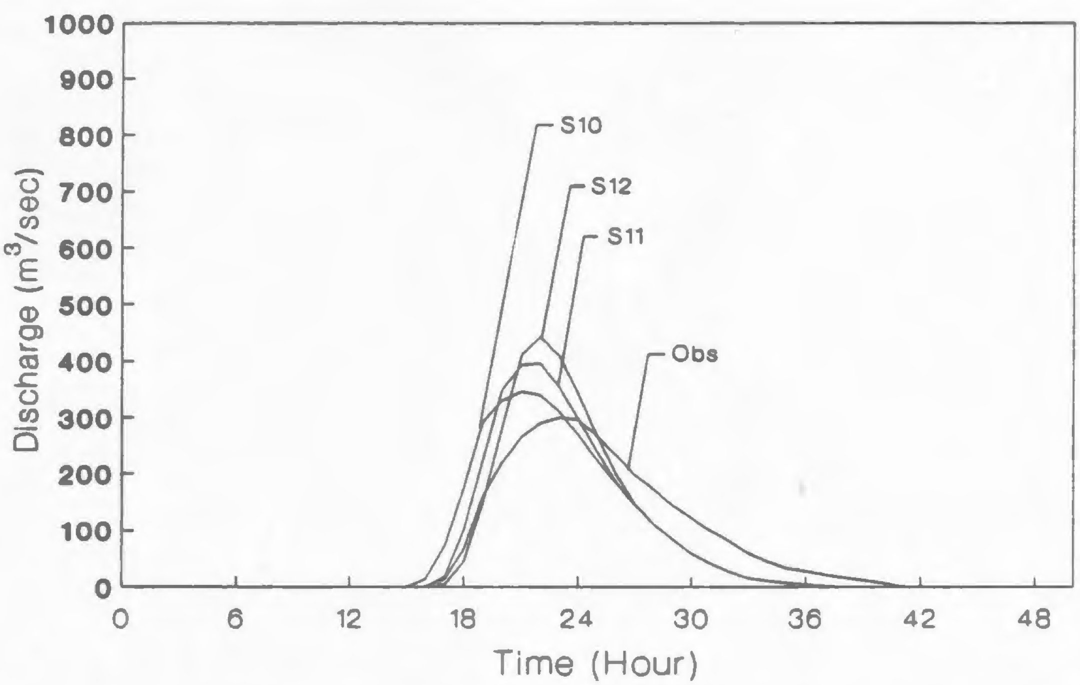
APPENDIX D26



The simulated hydrograph : September, 27 1981.

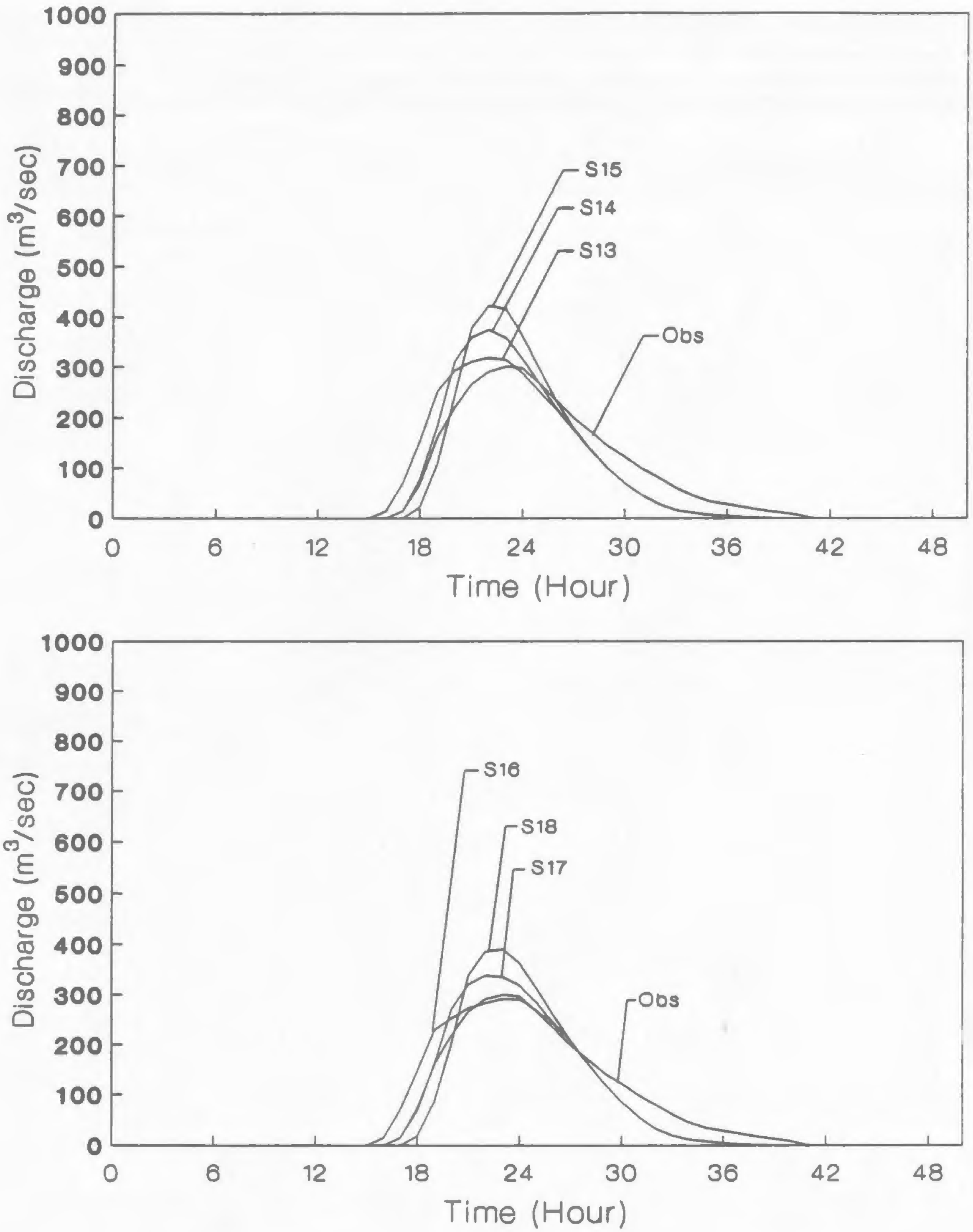
APPENDIX D27





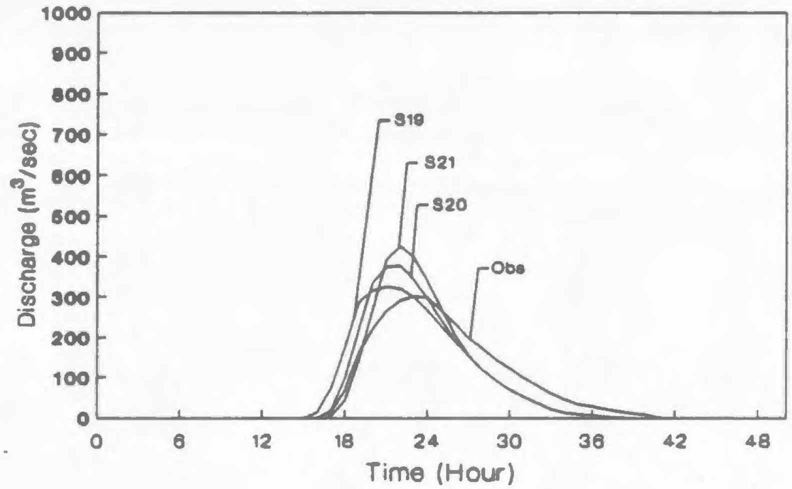
The convoluted hydrograph : September, 27 1981.

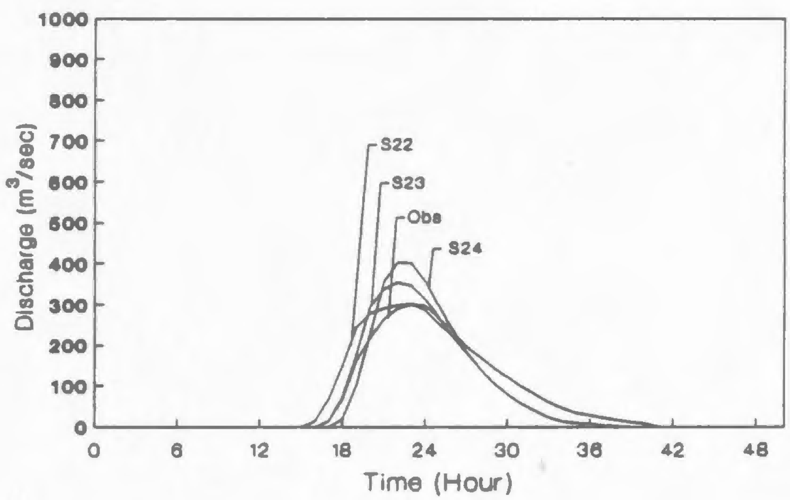
APPENDIX D28



The convoluted hydrograph : September, 27 1981.

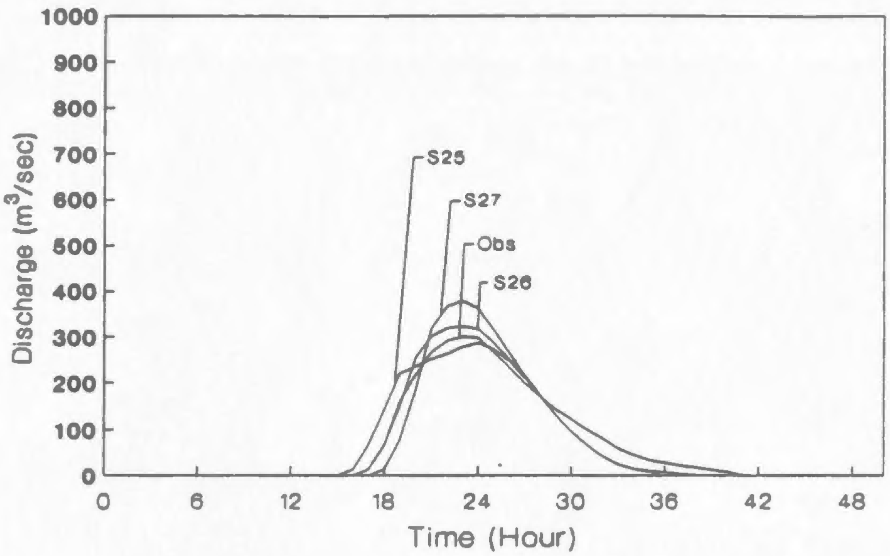
APPENDIX D29





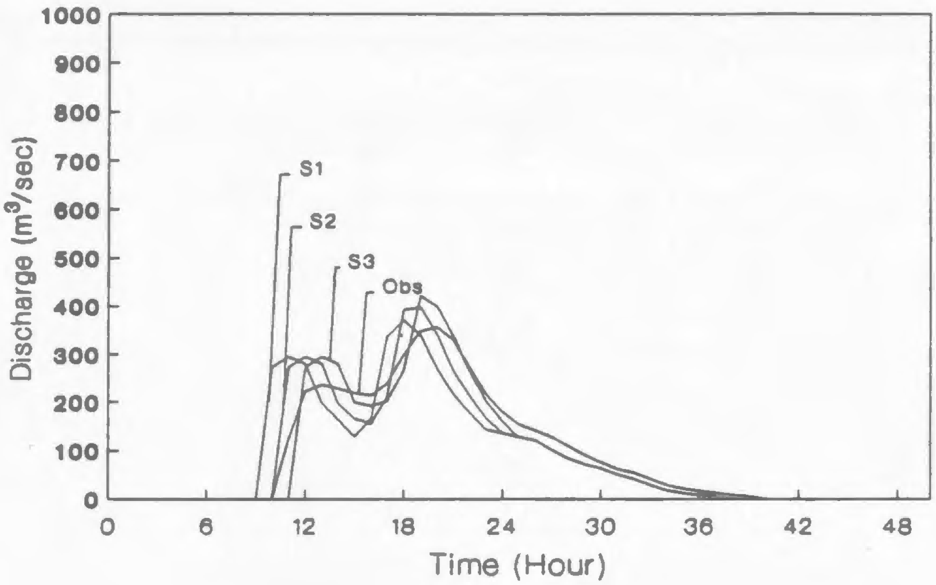
The convoluted hydrograph : September, 27 1981

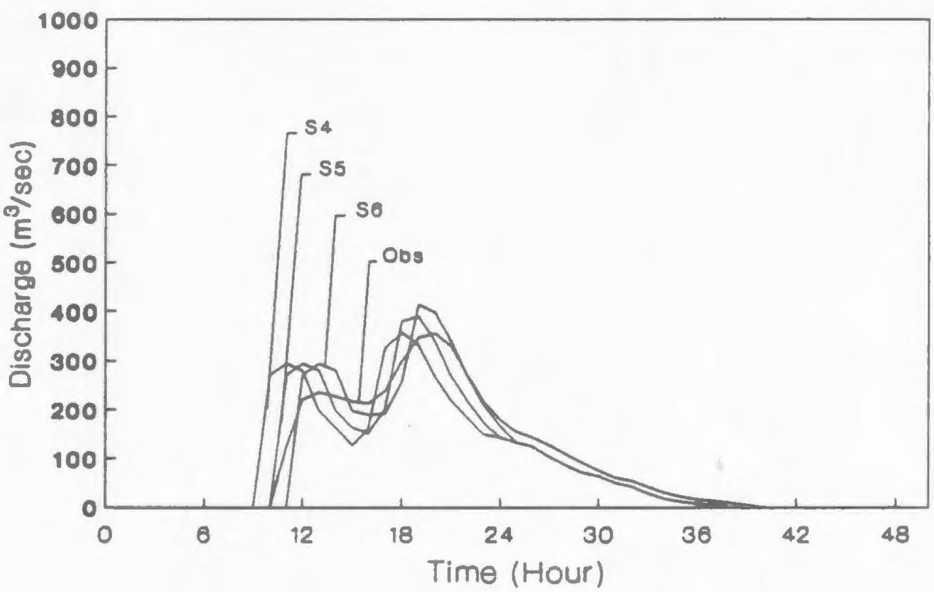
APPENDIX D30



The convoluted hydrograph : September, 27 1981

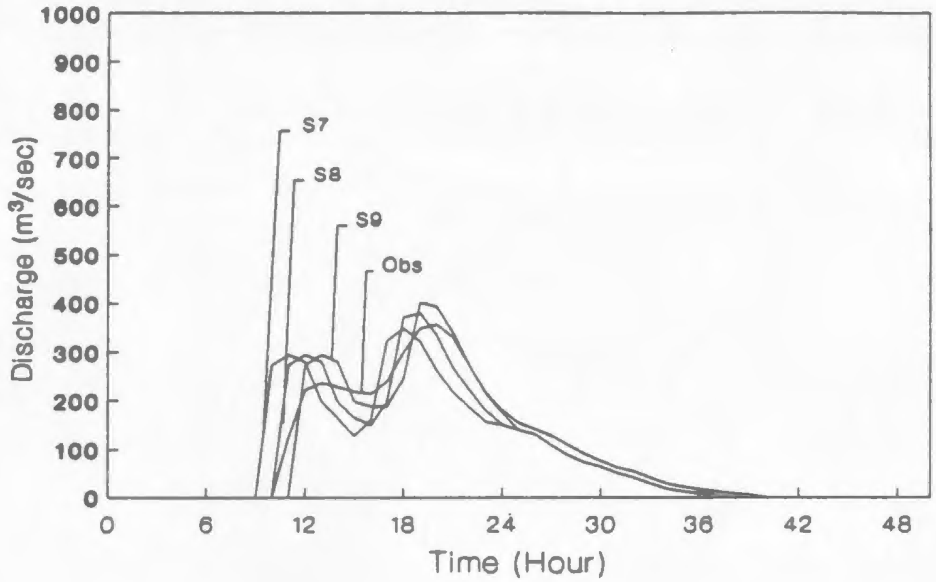
APPENDIX D31

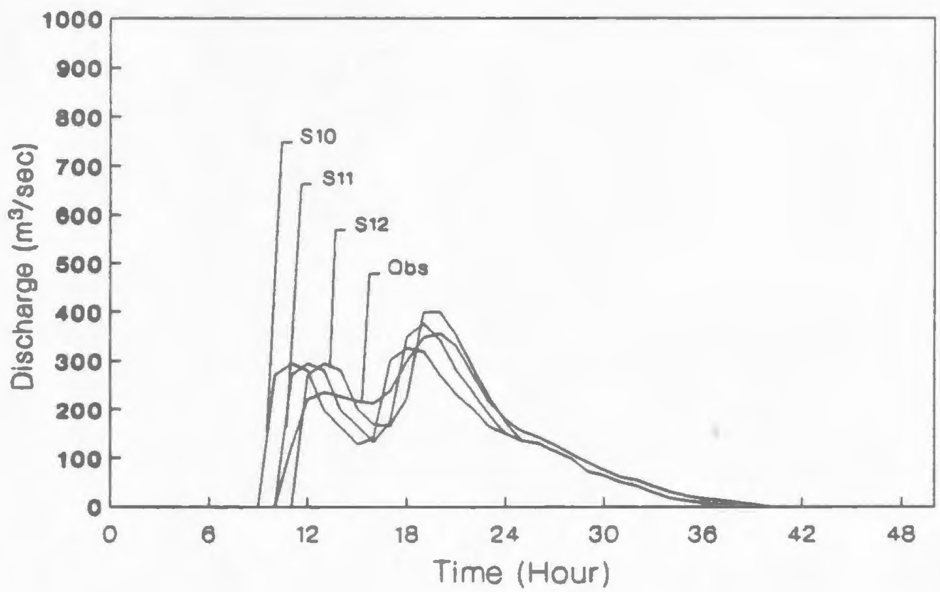




The convoluted hydrograph : December, 31 1986

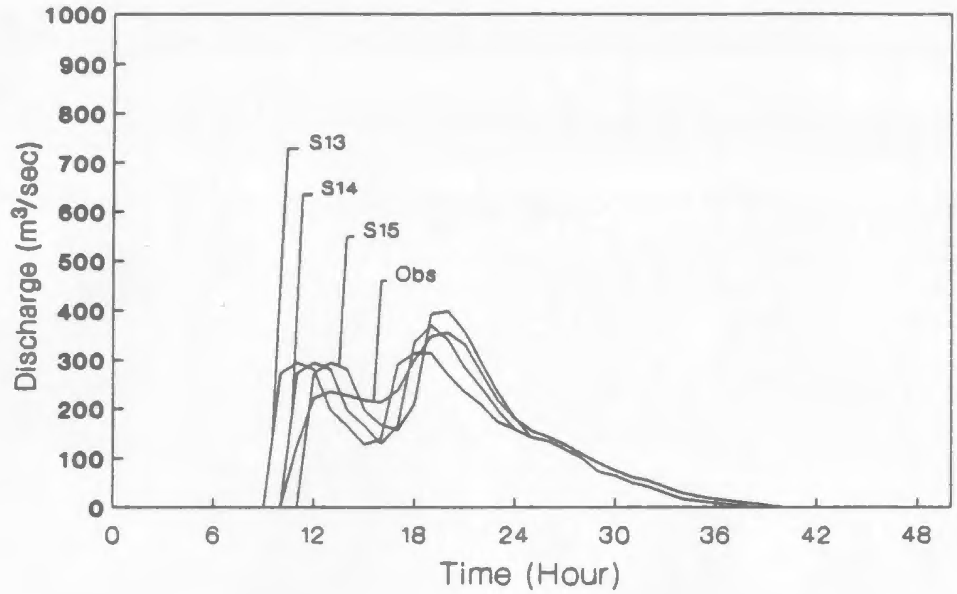
APPENDIX D32

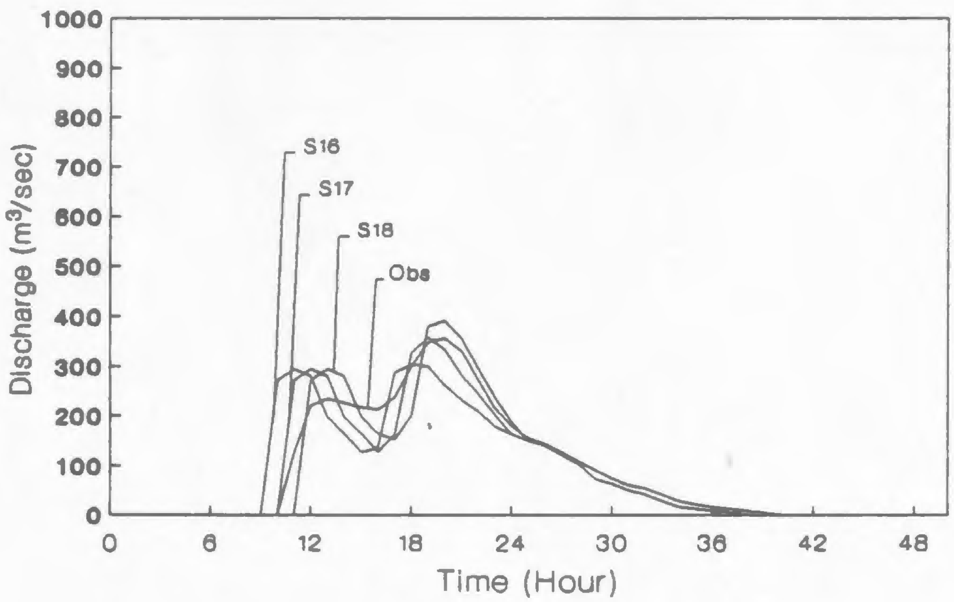




The convoluted hydrograph : December, 31 1986

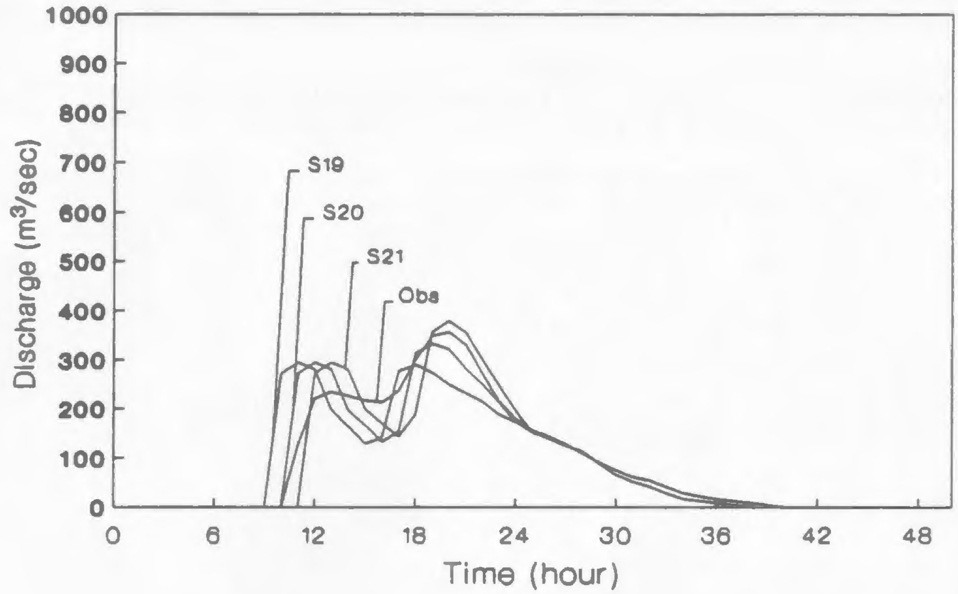
APPENDIX D33

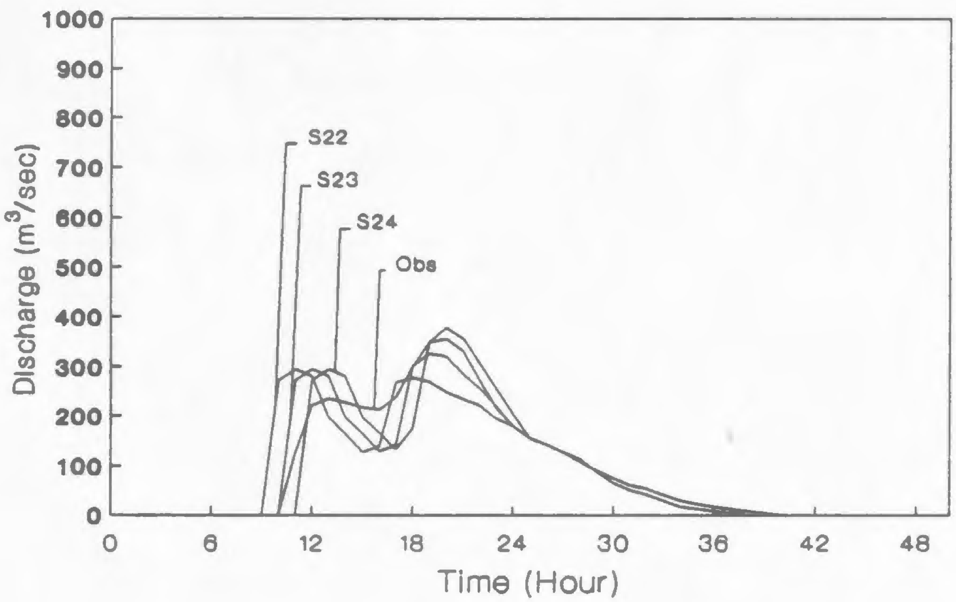




The convoluted hydrograph : December, 31 1986

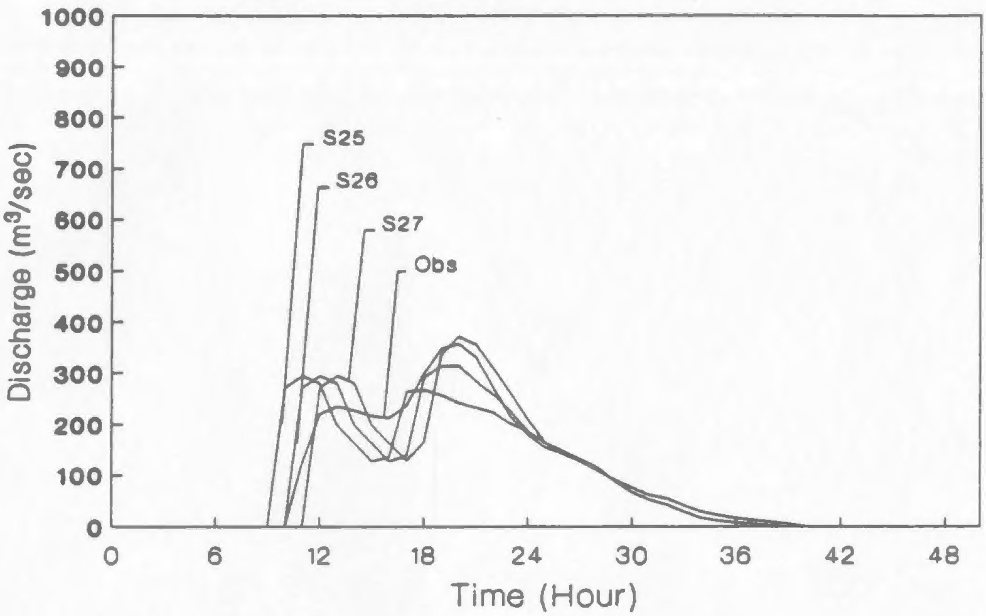
APPENDIX D34





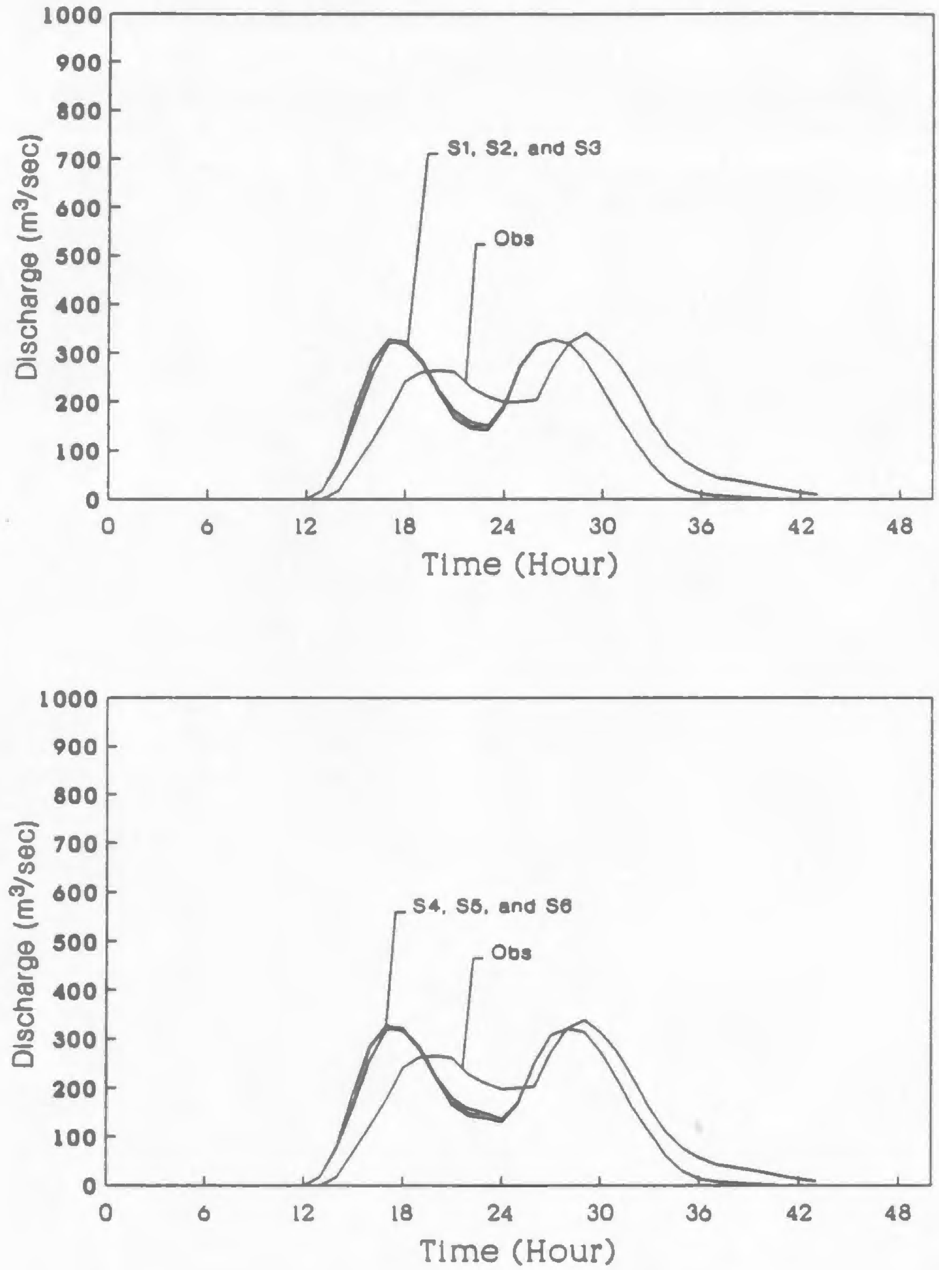
The convoluted hydrograph : December, 31 1986

APPENDIX D35



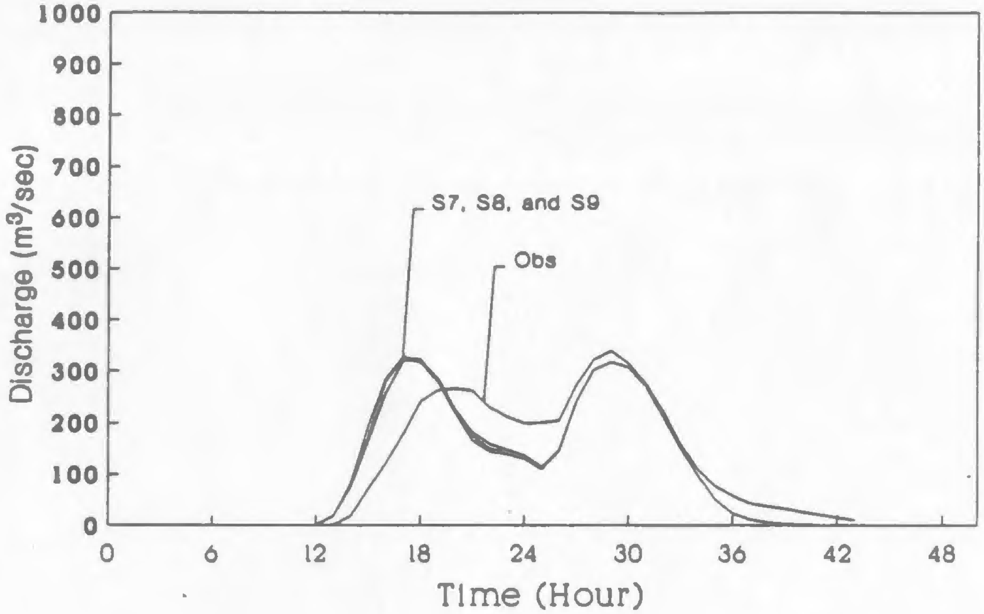
The convoluted hydrograph : December, 31 1986

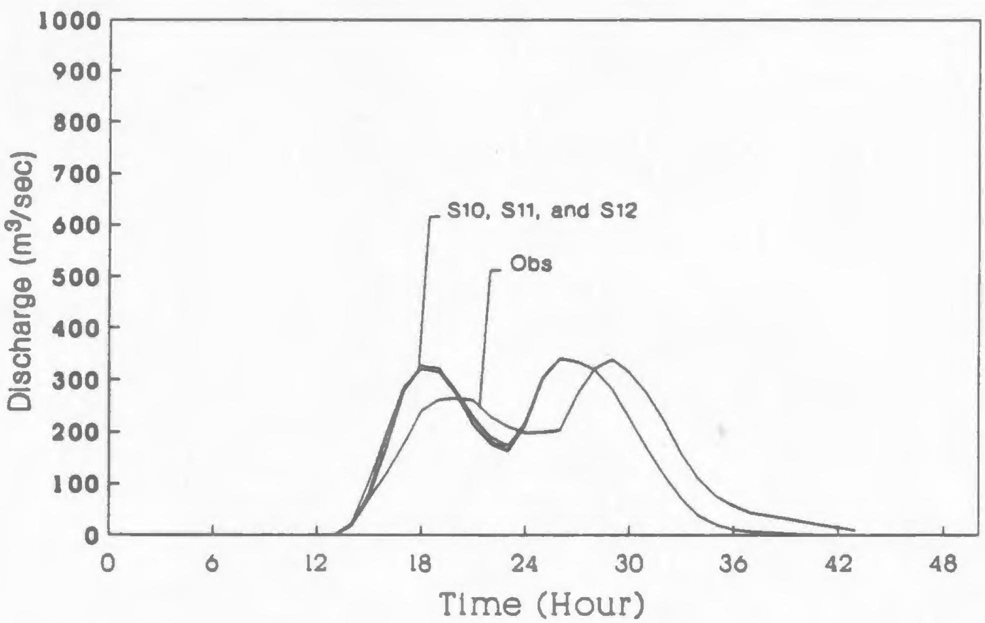
APPENDIX D36



The convoluted hydrograph : February, 28 1985

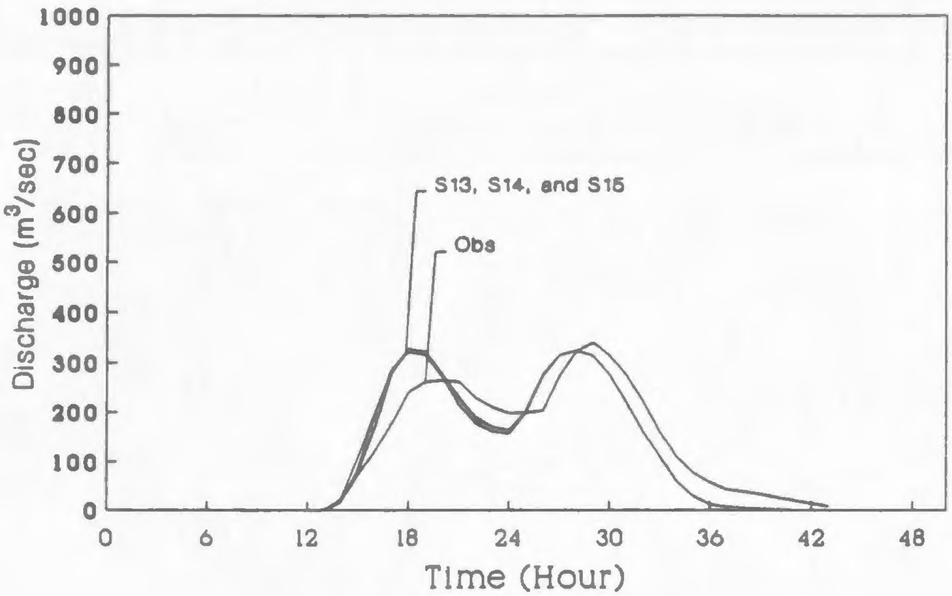
APPENDIX D37

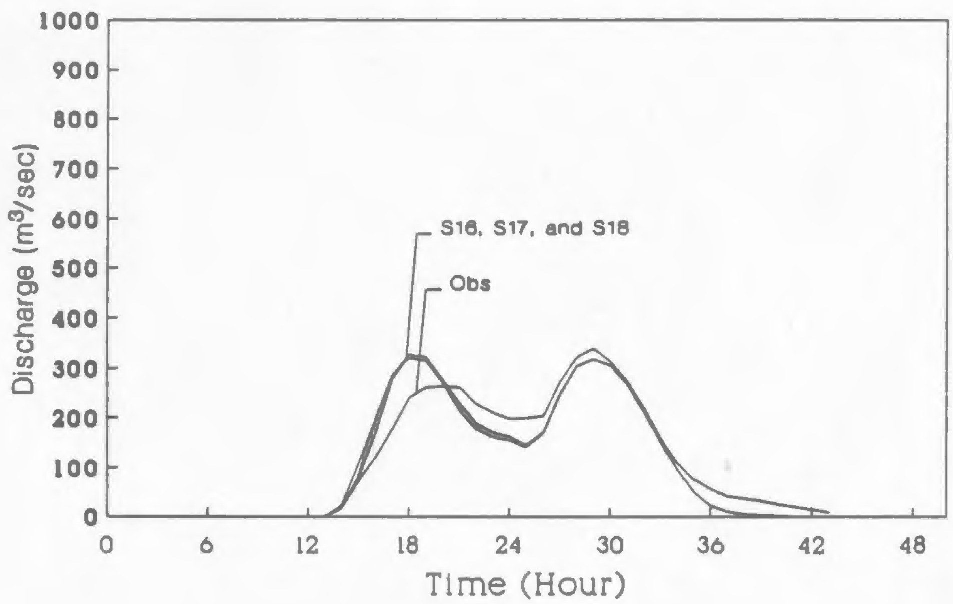




The convoluted hydrograph : February, 28 1985

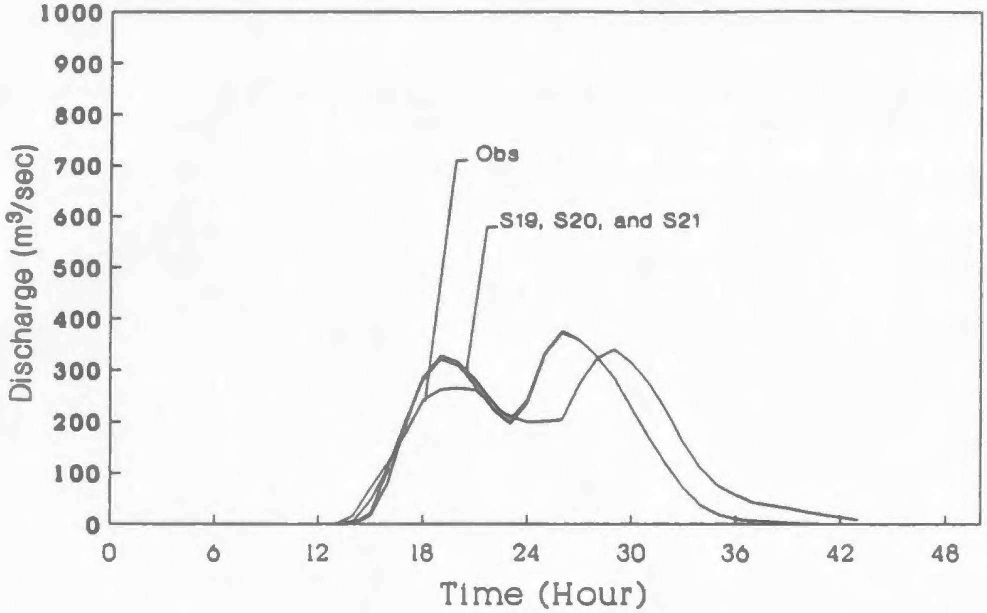
APPENDIX D38

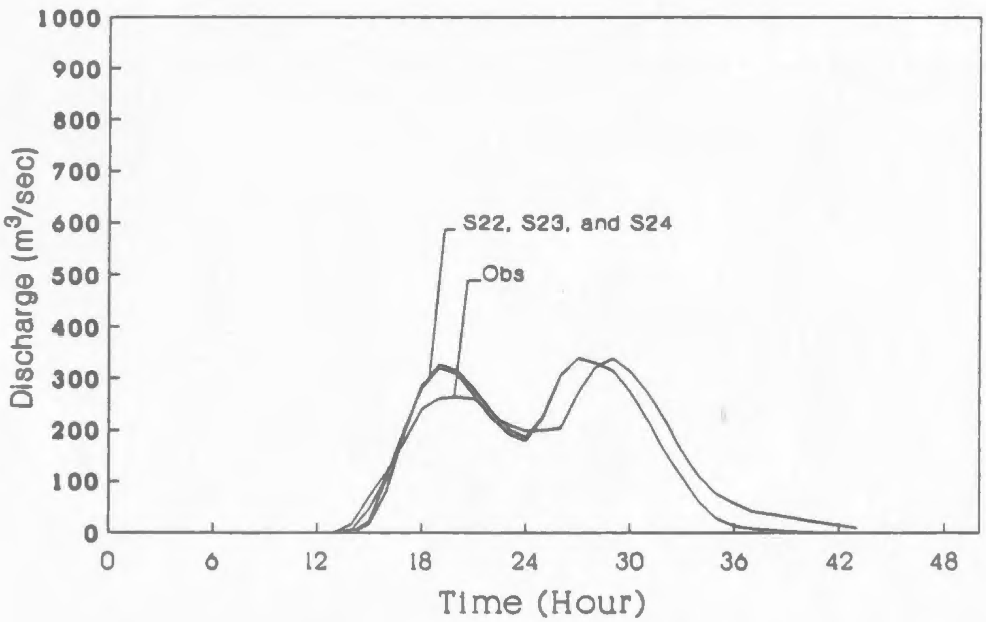




The convoluted hydrograph : February, 28 1985

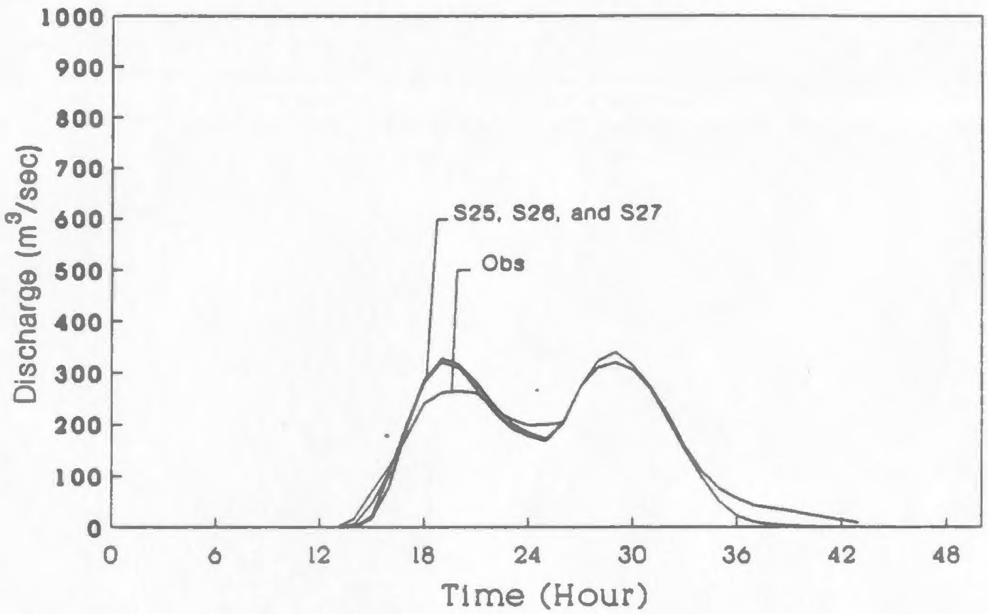
APPENDIX D39





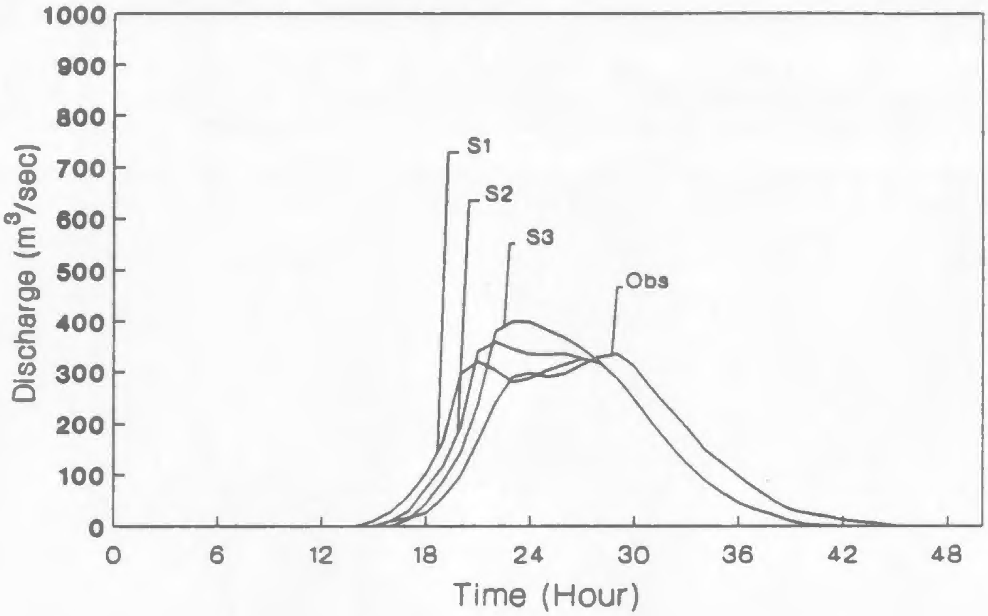
The convoluted hydrograph : February, 28 1985 .

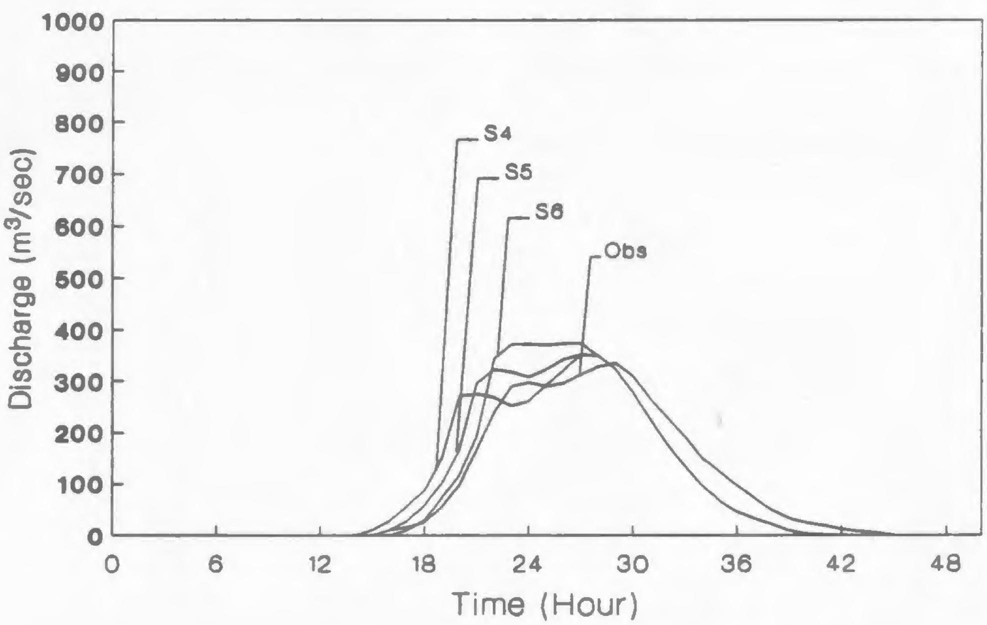
APPENDIX D40



The convoluted hydrograph : February, 28 1985

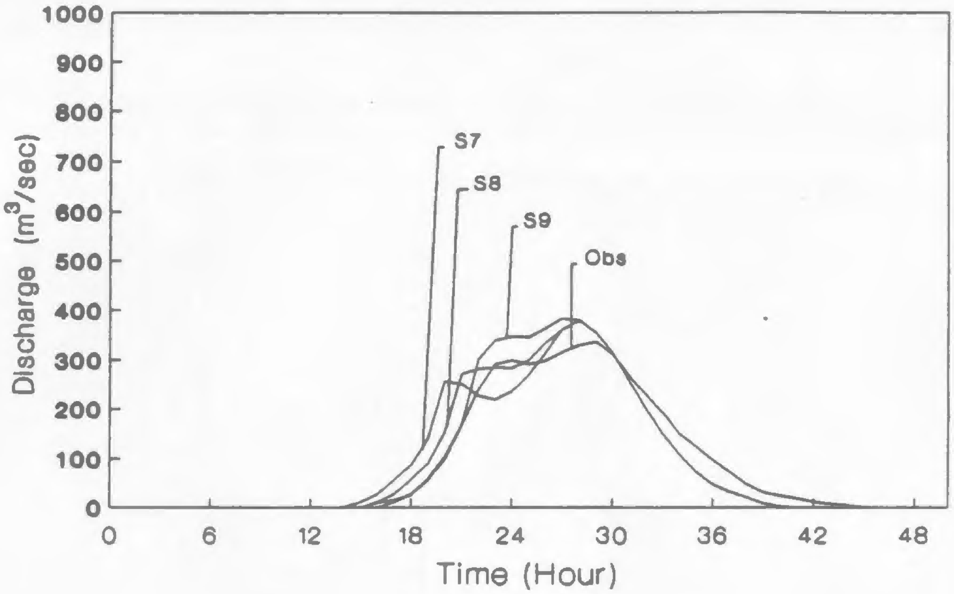
APPENDIX D41

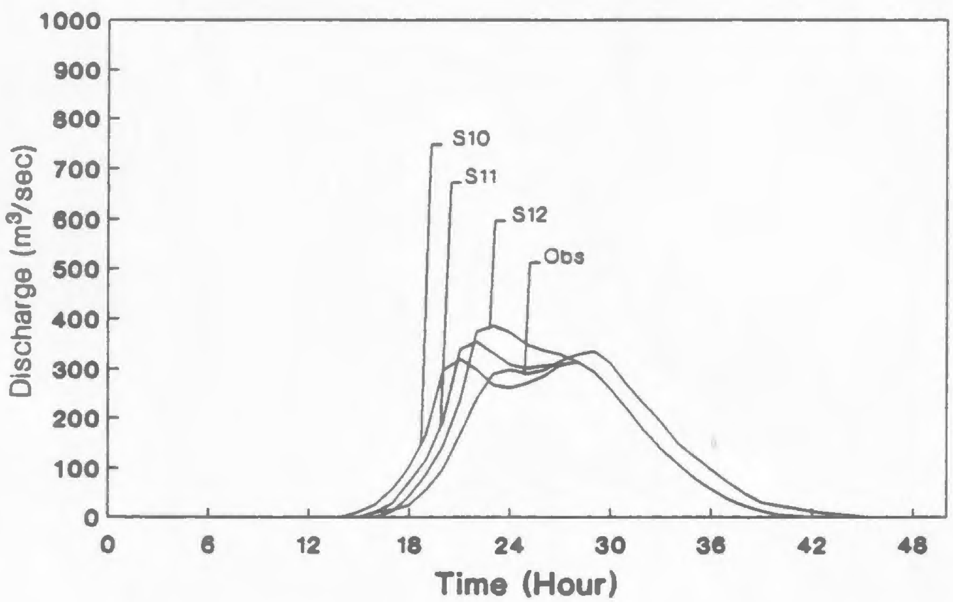




The convoluted hydrograph : December, 6 1979

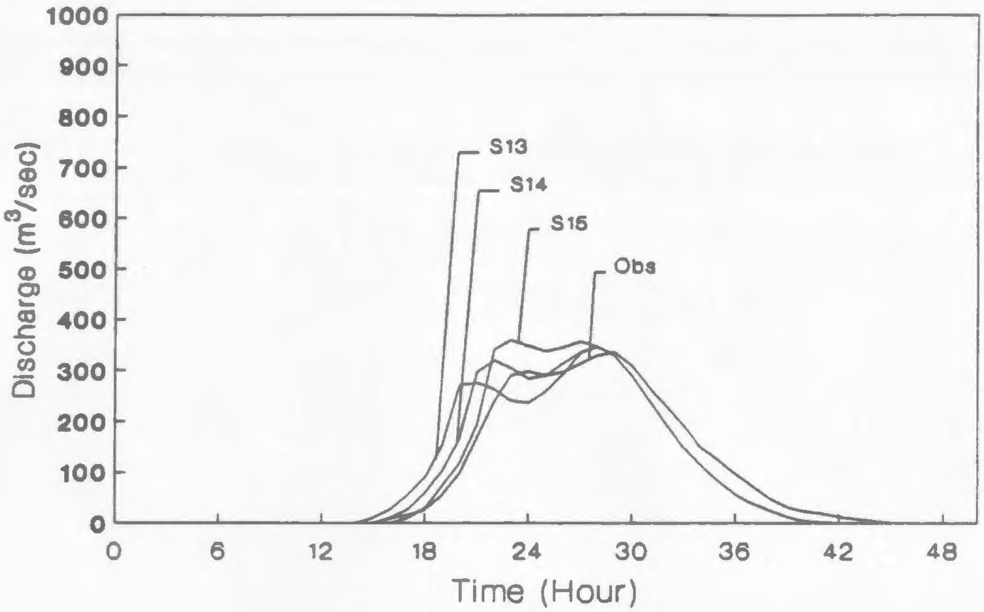
APPENDIX D42

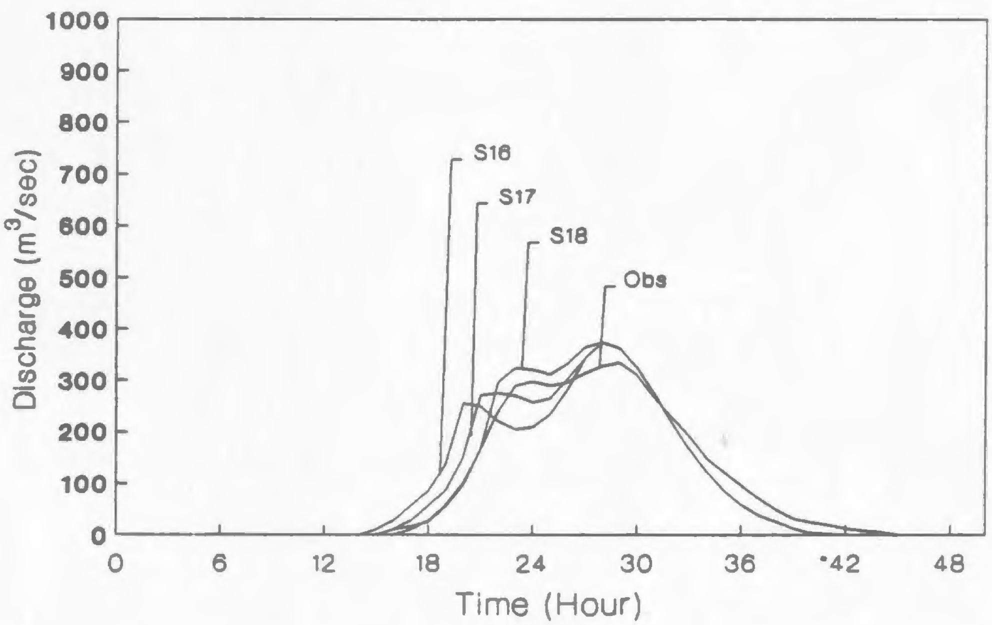




The convoluted hydrograph : December, 6 1979

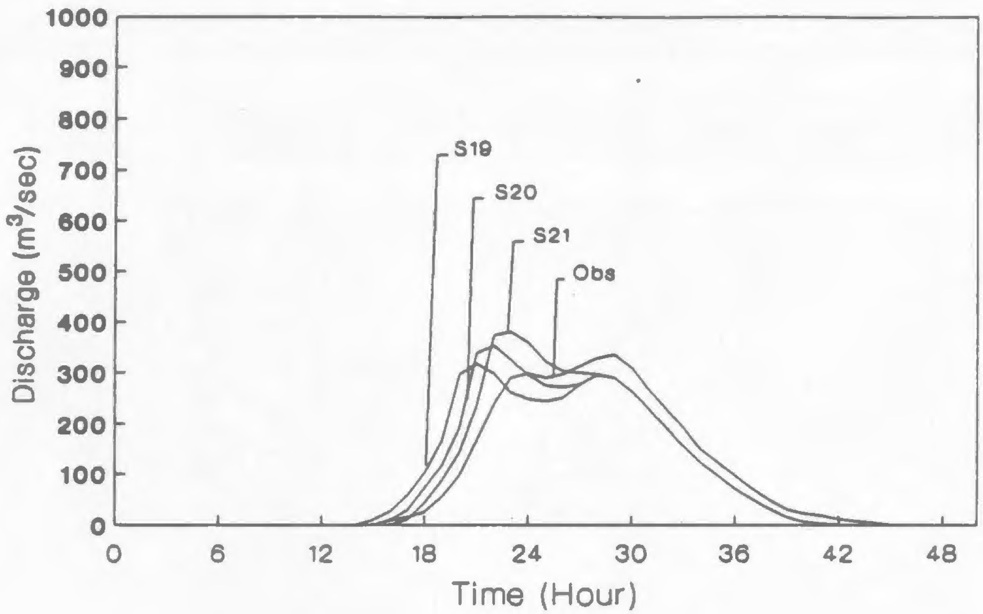
APPENDIX D43

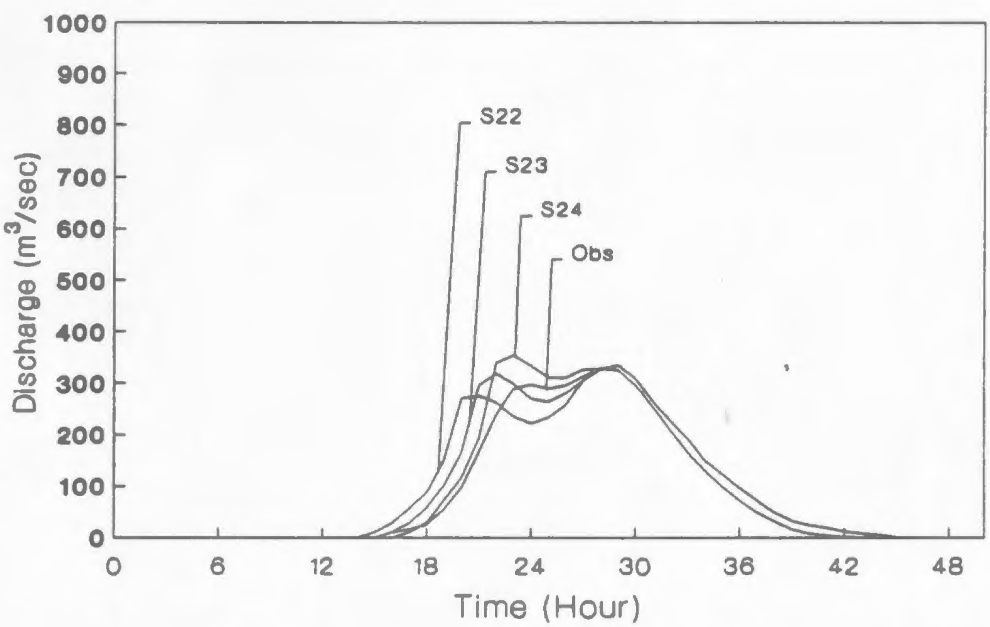




The convoluted hydrograph : December, 6 1979

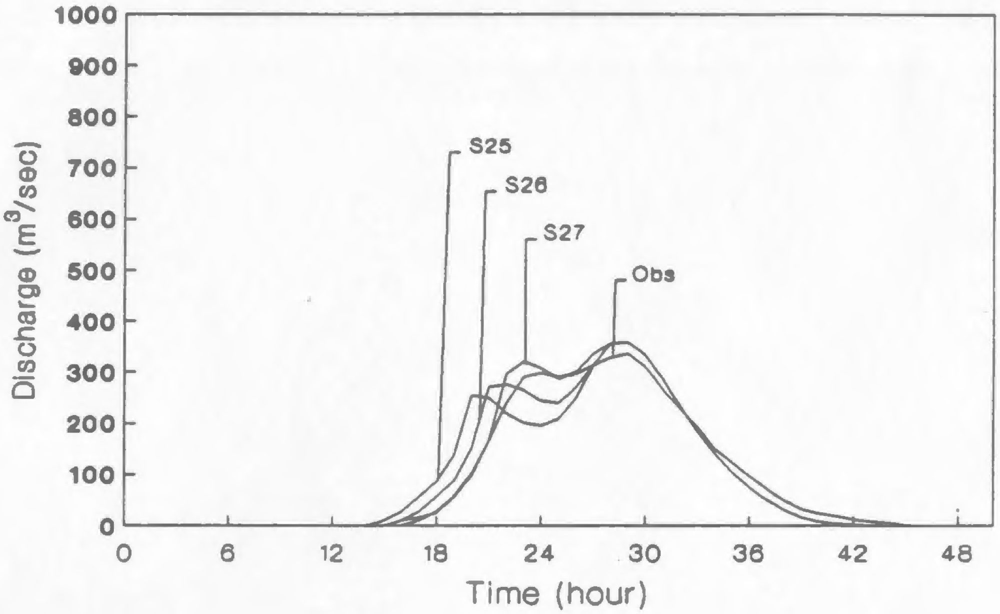
APPENDIX D44





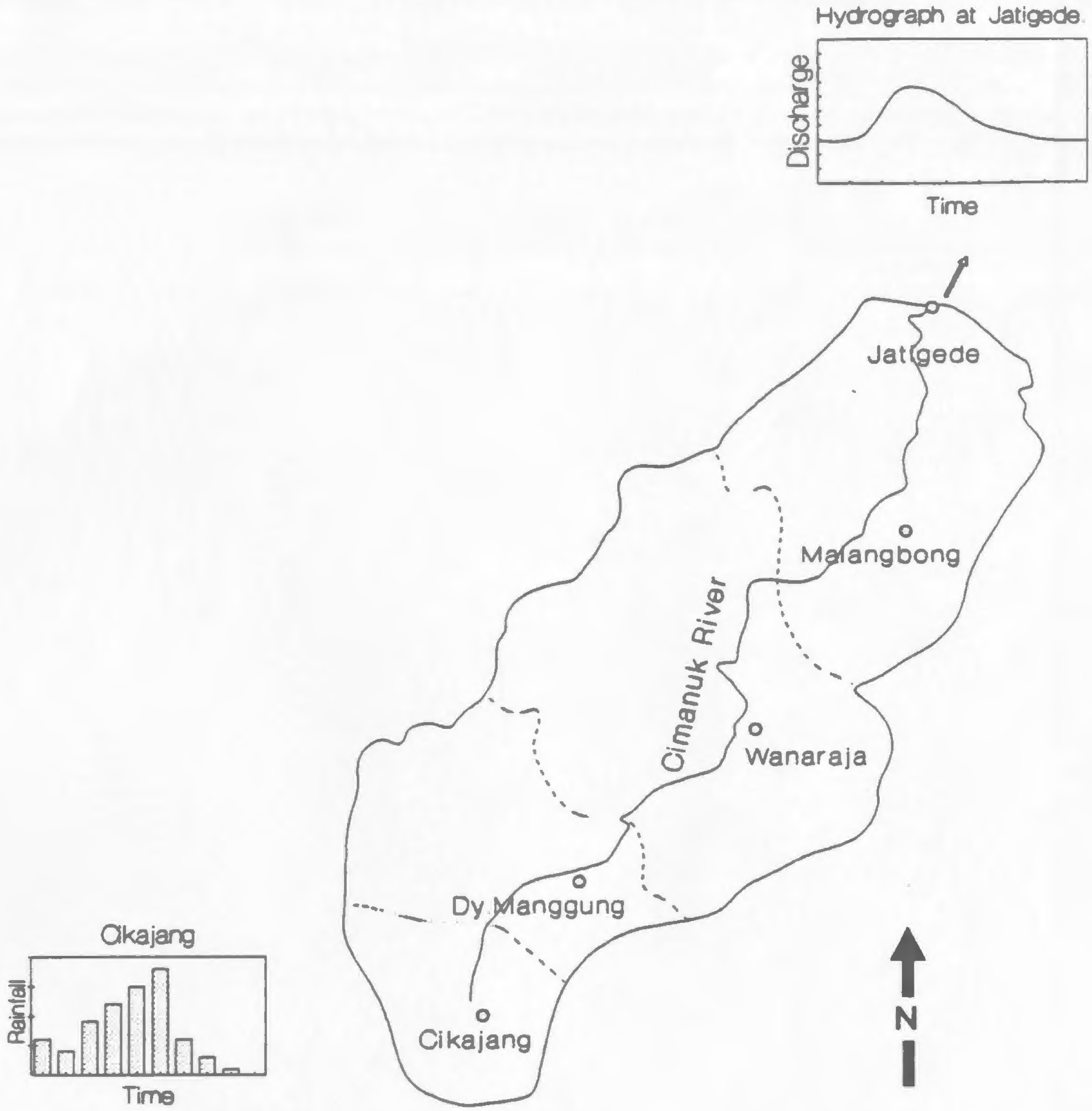
The convoluted hydrograph : December, 6 1979

APPENDIX D45



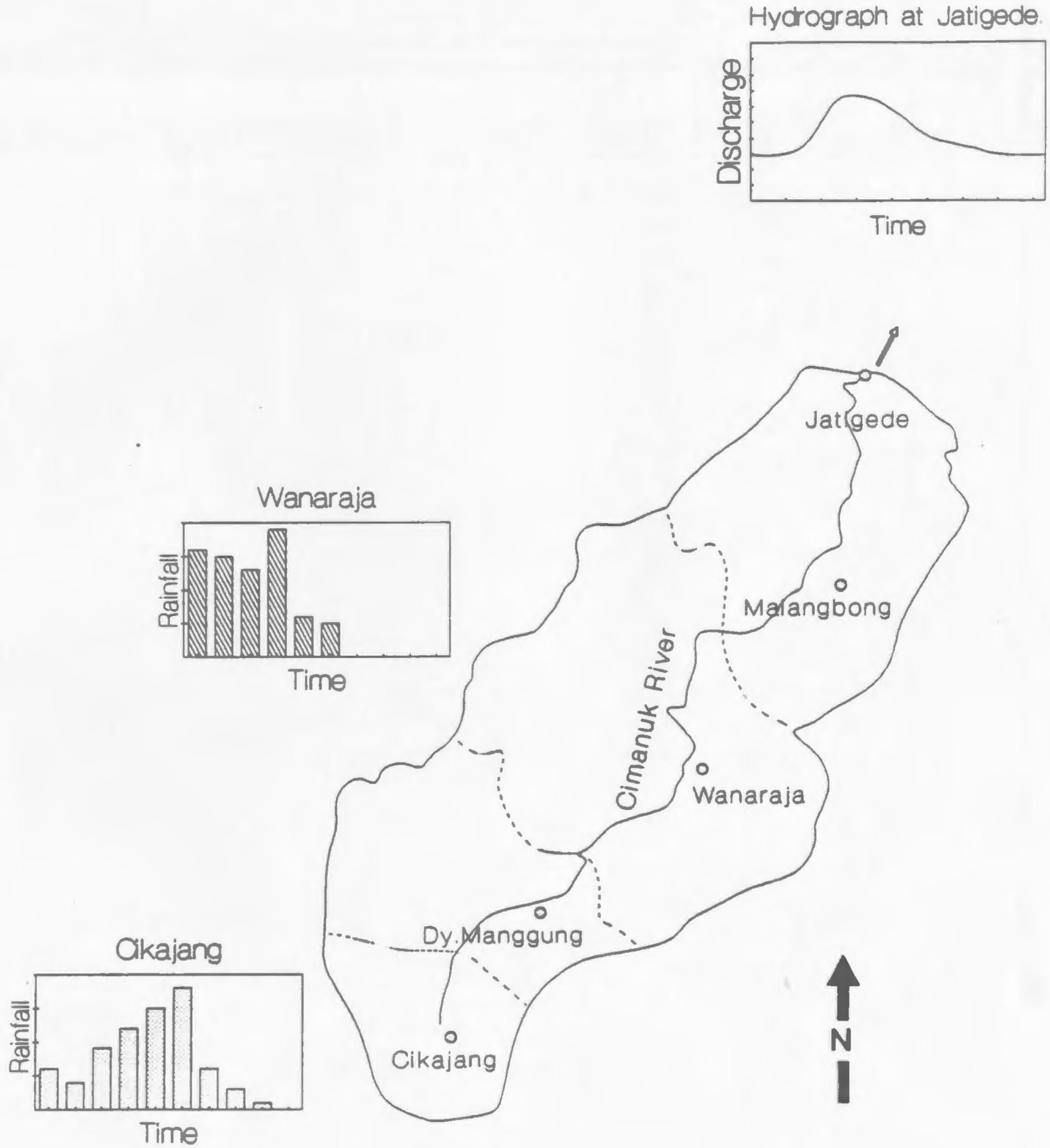
The convoluted hydrograph : December, 6 1979

APPENDIX E1



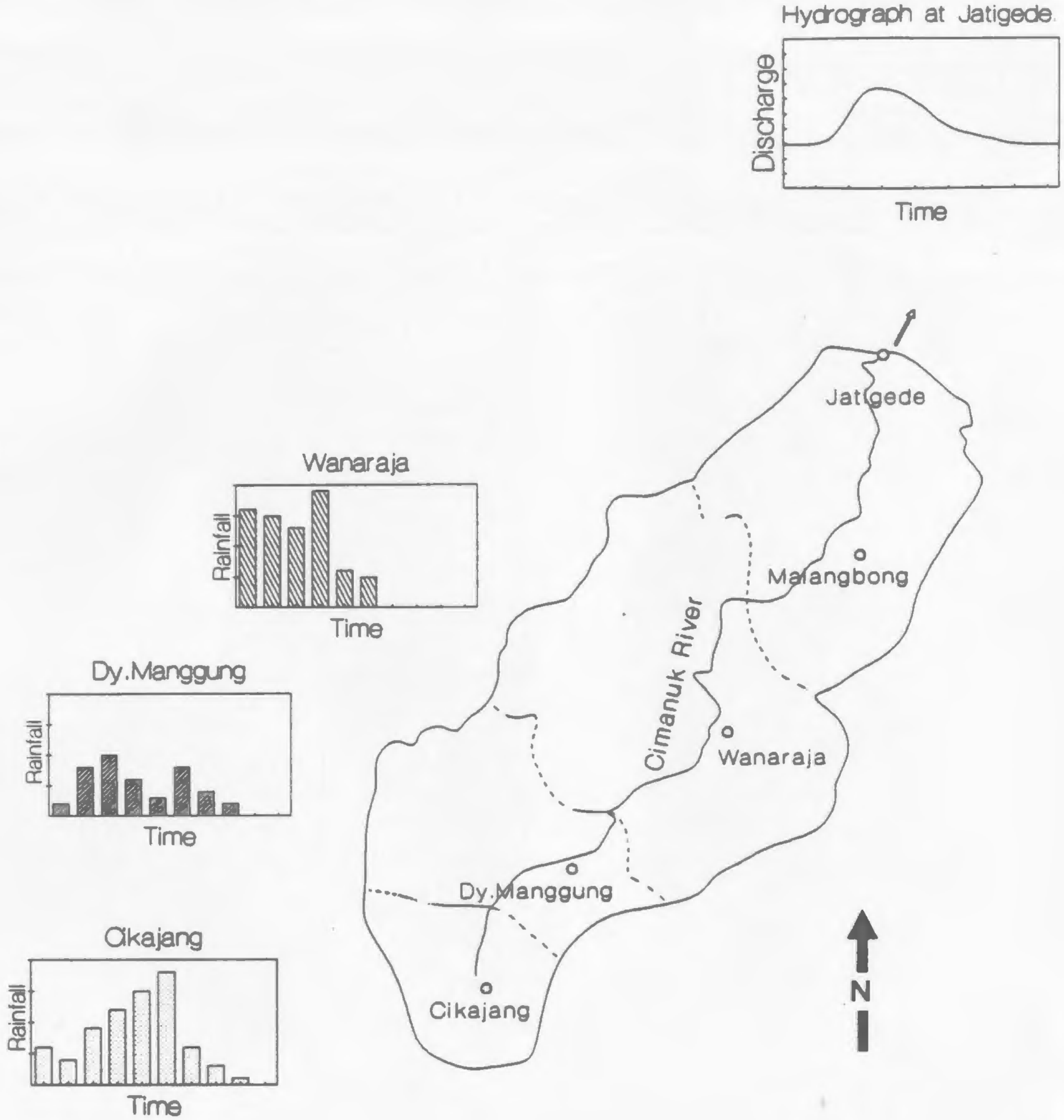
Type A event

APPENDIX E2



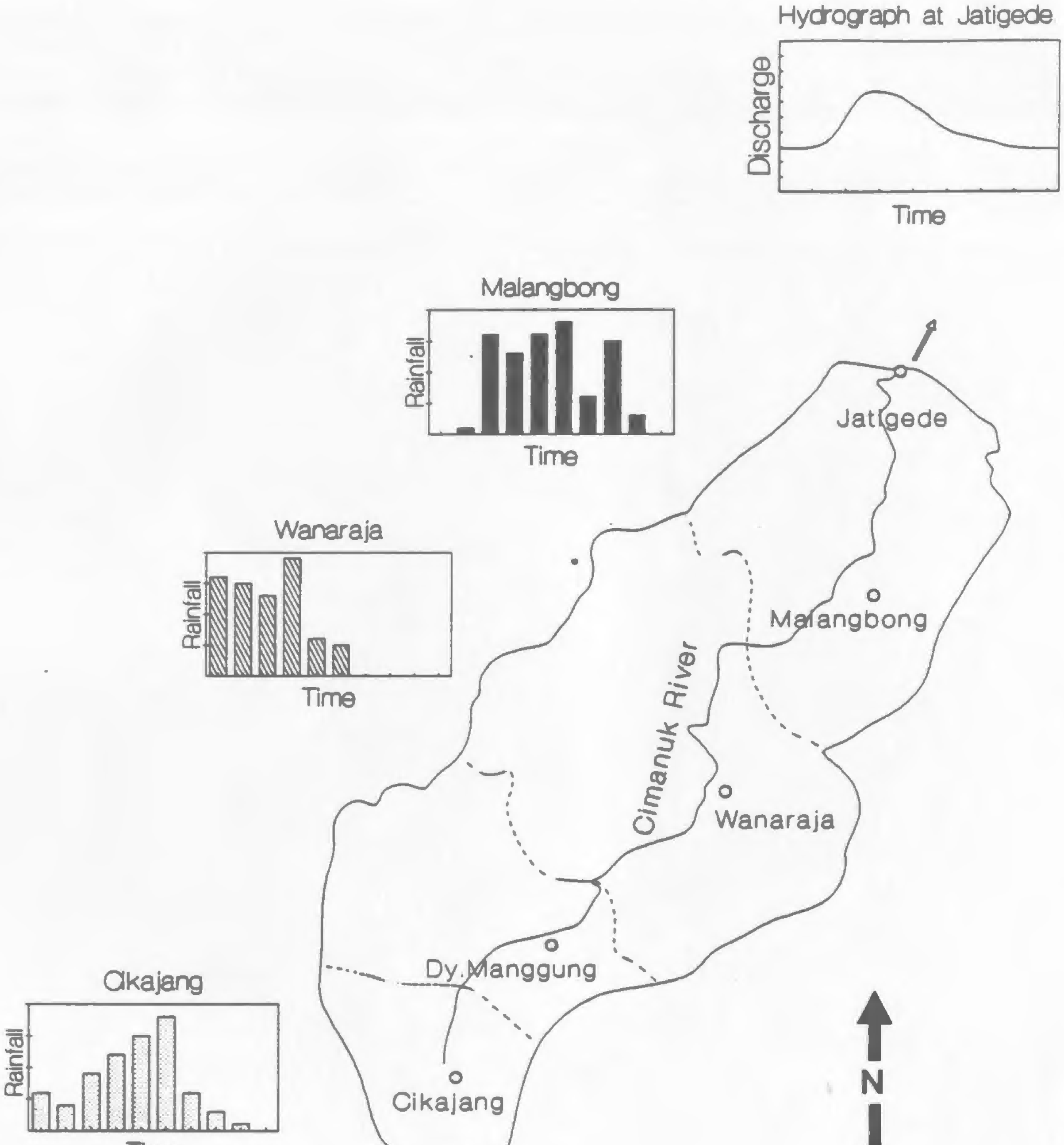
Type B event

APPENDIX E3



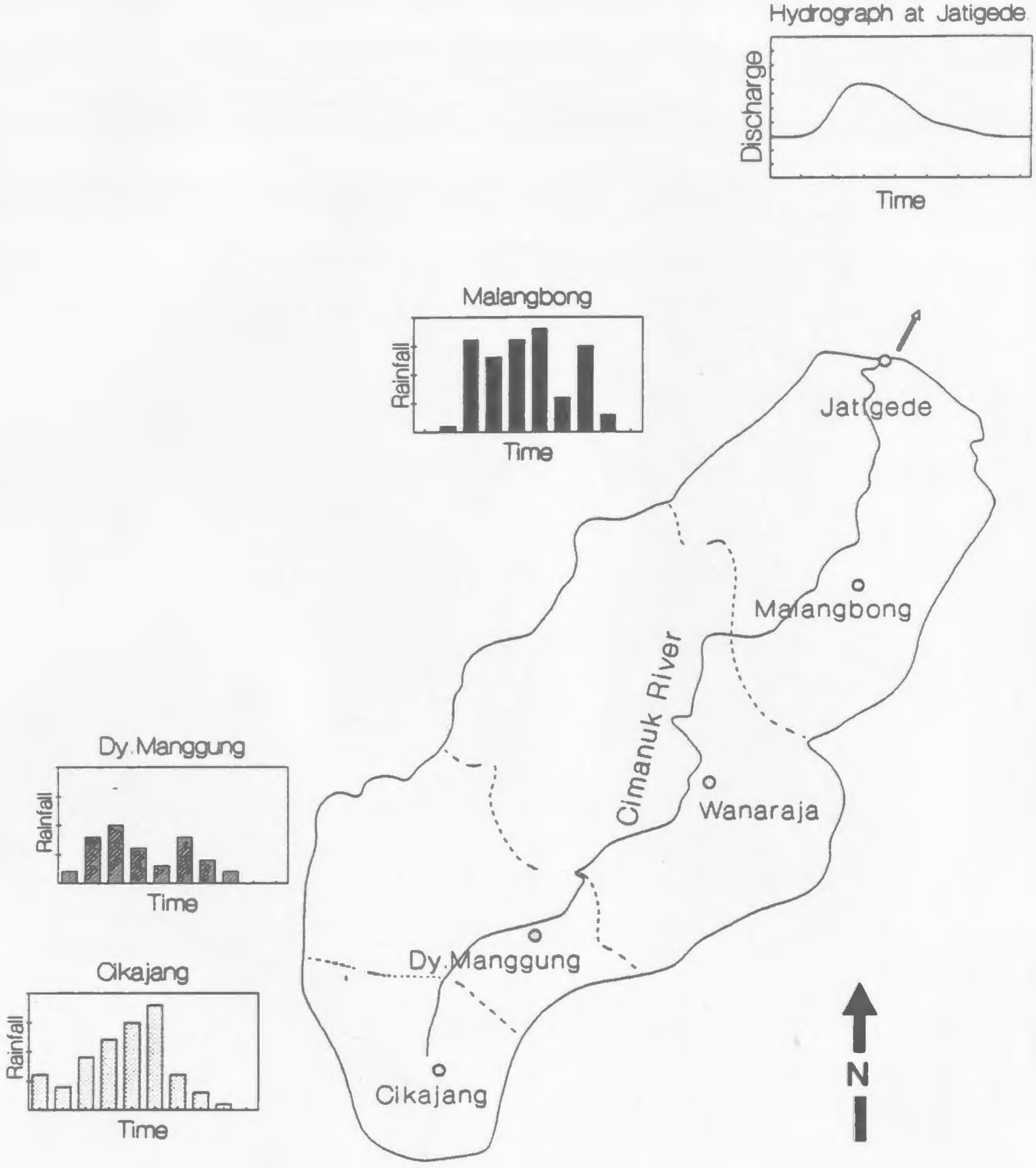
Type C1 event

APPENDIX E4



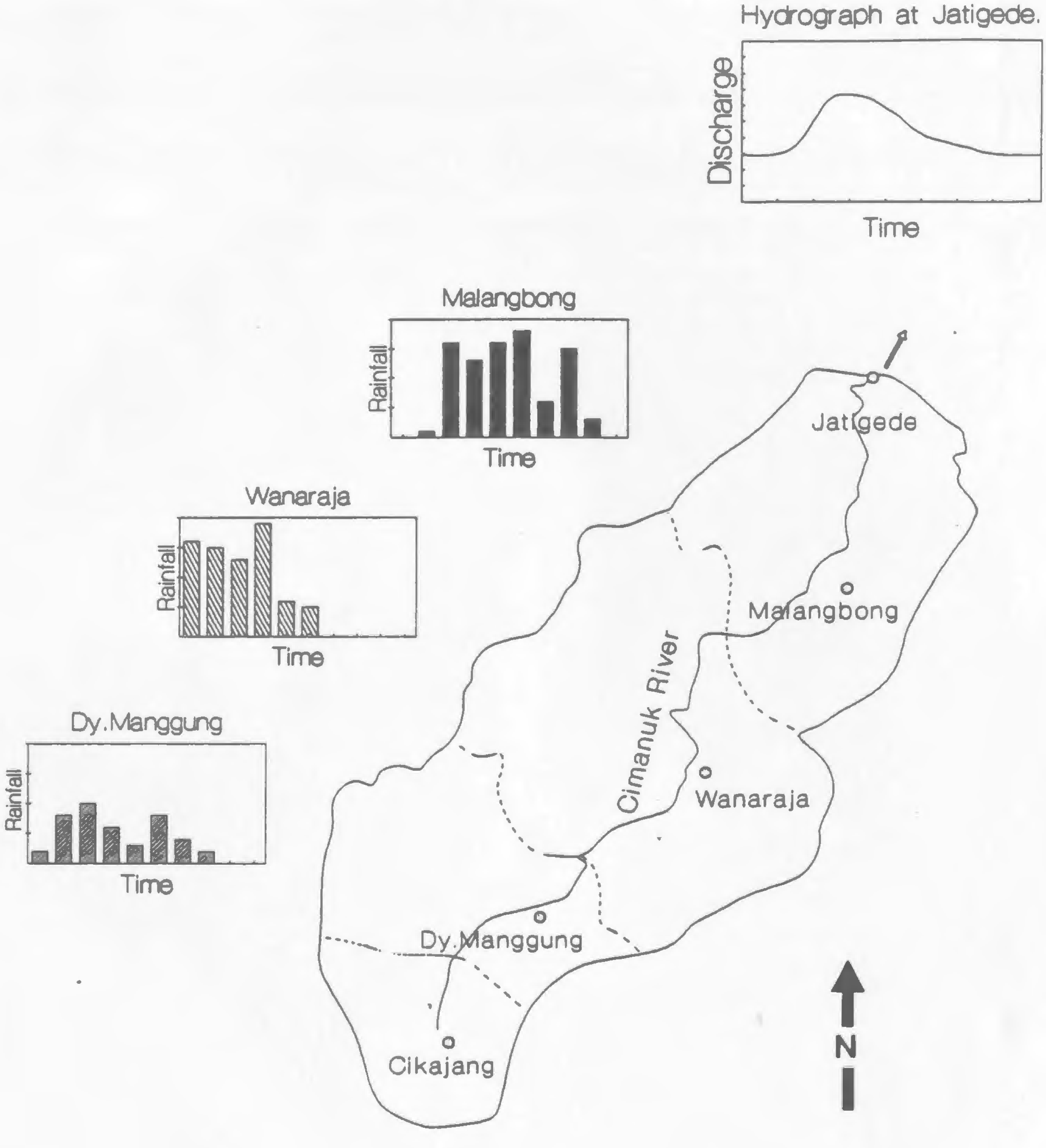
Type C2(a) event

APPENDIX E5



Type C2(b) event

APPENDIX E6



Type C2(c) event

APPENDIX F

```
REM       "INPUT DATA"
DATA      15
REM       The number of the ordinates of the Cikajang TH.
REM
DATA      0, 0.57, 1.61, 2.19, 2.53, 2.33, 2.2, 1.71, 1.59, 1.11, 1.09,
          0.71, 0.66, 0.29, 0
REM       The ordinates of the Cikajang TH.
REM
DATA      5
REM       The rainfall duration at Cikajang.
REM
DATA      5, 6, 5, 4, 5
REM       The rainfall intensity at Cikajang.
REM
DATA      14
REM       The number of the ordinates of the Dy.Manggung TH.
REM
DATA      0, 1, 2.49, 4.25, 4.96, 4.76, 4.11, 3.14, 2.25, 1.75,  1.49,
          1.12, .56, 0
REM       The ordinates of the Dy.Manggung TH.
REM
DATA      4
REM       The rainfall duration at Dy.Manggunmg.
REM
DATA      12, 10, 9, 11
REM       The rainfall intensity at Dy.Manggung.
REM
DATA      12
REM       The number of the ordinates of the Wanaraja TH.
REM
DATA      0, 6.23, 14.09, 14.99, 14.11, 13.50, 10.52, 7.24, 4.34, 2.61,
          0.52, 0
REM
DATA      6
REM       The rainfall duration at Wanaraja.
REM
```

```
DATA    11, 13, 7, 5, 5, 4
REM     The rainfall intensity at Wanaraja.
REM
DATA    8
REM     The number of the ordinates of the Malangbong TH.
REM
DATA    0, 15.9, 17, 14.3, 9.3, 5.3, 2.7, 0
REM     The ordinates of the Malangbong TH.
REM

DATA    4
REM     The rainfall duration at Malangbong.
REM

DATA    12, 13, 10, 8
REM     The rainfall intensity at Malangbong.

DATA    12
REM     The beginning of rainfall at Cikajang sub-basin.

DATA    12
REM     The beginning of rainfall at Dy.Manggung sub-basin.

DATA    16
REM     The beginning of rainfall at Wanaraja sub-basin.

DATA    14
REM     The beginning of rainfall at Malangbong sub-basin.
```

"THE OUT PUT"

The ordinates of the hydrograph.

0.00	190.80	410.70	551.60	606.70
506.23	473.60	557.37	669.55	736.84
769.08	762.53	680.91	547.77	419.09
293.96	182.48	103.58	52.50	20.68
6.54	1.45	0.00	0.00	