MALAYSIA

MINISTRY OF AGRICULTURE
DRAINAGE AND IRRIGATION DEPARTMENT

PERAI BARRAGE GATE OPERATION STUDY IN SEBERANG PERAI PULAU PINANG

FINAL REPORT

VOLUME I

DECEMBER 1988

JAPAN INTERNATIONAL COOPERATION AGENCY

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Preface

We are pleased to submit the report on the Peral Barrage gate operation study. We hope that this report will serve for the operation and maintenance of Perai Barrage gate and contribute to the promotion of friendly relations between our two countries.

Furthermore, we should like to extend our deepest appreciation for the support and cooperation rendered espesially by those as listed herein below.

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December 1988

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Contents

Summary

The Sungai Perai river is situated in the central part of Seberang Prai, in the State of Pulau Pinang. The Perai Barrage is locatd some 8 km from the river estuary and its praimary function is to prevent ingression of saline water and to maintain the water level of upstream.

Over the last few year, the State Drainage and Irrigation Depertment have made attempt to formulate gate operation procedure so as to achive the primary function of maintainig a controlled water level upstream. The attempts to close the gates have result in the rising of the water level in downstream reach of the river, causing inundation in lowlying areas. There is thus a need to formulate the procedure of gate operation through the use of mathematical model of river behavior.

The inundation originate in the topographic condition which the high tide is higher than the ground level of lowlying areas. As the result of investigation the collected data and the simulation, the inundation was caused by the abnormal higher condition of tide which occured before and after 17 May 1984, and the gate operation which made the period of high water became longer. The water level of the tide, however, is estimated nearly same a high tide which has a peak water level of 1.48 m and occures a few days in a year. When the operation is carried out on a same order high tide, inundation will occure, since the elevation of lower area is about EL. 1.2 ~ 1.3 m and the water level near the downstream of the Barrage may be over 1.48 m.

For the safty and freely operation of the gates, the embankment should be constracted to prevent inundation of downstream lowlying area. On the operation and maintenance, the gates should be renewed, since the gate leaves were hardly damaged by corrosion and the structure of gate requires delicate operation.

I. INTRODUCTION

This report presents the result of study and survey of the gate operation of Prai Barrage. The general map of study area is presented in Fig.1-1.

The cotruction of the Barrage started in 1979, and the Barrage structure was completed in 1981. The state Drainage and Irrigation Depertment (D.I.D.) have made attempts to formulate gate operation procedures as to achive the primary function of maintaining a controlled water level upstream in 1984. The attempt to close the gates have result in rising the water level in downstream reach of the river, causing inundation lowlying areas.

Owing to the intensiive land use in recent year, the damage potential is increasing, and consequently, the residents in these area have objected strongly to further trial and error attempts formulating the gate operation procedures.
There is thus a need to formulate such procedures through the use of mathematical modeling of river behavior. D.I.D. didn't have the necessary experties to carry out this task. While D.I.D. has grouped to perform the task, the gates were damaged by rust corrosion.

The Goverment of Japan was requested for the technical assistance and experties to carry out the study of Perai Barrage gate operation by the Goverment of Malaysia. In respose to the request, the JICA study team carried out the study in January-March and October-December 1988.

The objective of the study are:

1. To analyze the unstedy flow hydraulics of Sg. Perai within its tidal reaches using mathmatical modelling approach.
2. To formulate gate operation procedures for the Perai Barrage so as to minimise inundation both upstream and downstream, while maintaining a controlled water level upstream, taking into consideration drainage and irrigation requirements of affecte d agriculturural land.
3. To propose other countermesures as are necessary, to mitigate inundation of lowlying area, and to keep proper gate opration and maintenance.

Fig.1-1 Study area

KEY MAP

II. General background

2-1 Reclamation plan of Sungai Perai
A proposal to drain the tidal swamp area was formulated under the First Five-Year Malaysia Plan(1966-1970). The feasibility study on the drainage and reclamation of Sungai(Sg.) Perai basin and the detail design of Perai Barrage had been performed under the tecnical assistance of the Japan Overseas Technical Cooperation Agency.

The feasibility study and the detail design of Prai Barrage were completed under the technical assistance of the Japan Overseas Technical Cooperation Agency. The construction of the Barrage started in 1979, and the Barrage structure was completed in 1981. The outline of this plan is as follwing to;

1) construct the tidal barrage, Perai Barrage,
2) reclaim the tidal swamp area of 670 ha along Sg. Prai,
3) improve the drainage condition of paddy field of 1,900 ha and coconut land of 520 ha,
4) impound fresh water upstream of the Barrage for industry water supply,
5) incorporate a permanent bridge in barrage to replace the pontoon bridge.

2-2 Locaion of study area
Sg. Perai river basin is situated in the central part of Seberang Prai, in the state of Pulau Pinang, north of Malaysia, and facing Penang Island. Topography of the area is generally plain and extending coastward, lowlying into western direction with mean gradient of about 1:4,000 from the foot of mountain of the eastern area, and it is sloped down toward Sg. Perai with about 1:5,000.

2-3 Climate
This area has a tropical climate with the temperature about 27 c throughout the year. Annual rainfall is approximately 2,300 mm, slightly more in September-November and April-May, and slightly less in December-March. Atmospheric pressure is ranged from 1,009 mb to 1,011 mb throughout the year, which means little fluctuation. The humidity ranges from 60 to 80 percent. There is no record of storm wind cause serious damage. According to the record of Penang Airport, the evaporation is 4 to 6 mm per day.

2-4 Agriculure

There is agricultural land, mainly paddy fields in upstream and midstream area, while in lower reaches there are rubber and coconut estates. With regard to the rice in 1988, transplant of 866 ha and harvesting of 152 ha on Junrry, and transplant of 1046 ha and harvesting of 487 ha on February were performed in Sg. Kulim irrigation scheme (K area).
However, the conversion of crop from rice to other cash crops has been planned in a part of K area.

2-5 Industrial area

Industrial area have been developed in lower reach of Sg. Perai of rigt bank area and swamp land has been reclaimed covering in with earth for industrial land.

III. Sungai Perai and Prai Barrage

3-1 Sungai Perai (Perai River)

Sg. Perai starting from the confluence of three tributeries; Sg. Kulim, Sg. Jarak and Sg. Kerah originated from mountains of the eastern area, flows into the Straits of Malacca, meandering southwestward. The length of it is about 20 km and the whole reach is in tidal area. There is frequent inundatuion of the riverine areas during high tide.

Sg. Perai river basin, included Sg. Kulim, Sg. Jarak and Sg.Kerah, has a total catchement area of 497 sq. km.

The surveying of river cross sections were carried out by D.I.D. in 1987 and 1988. The locations are shown in Fig.3-1-1. The cross sections of its and longitudinal profile are as shown in Fig.3-1-2 and Fig.3-1-3, width is 40 m at upstream, 150 m at near the Barrage, 250 m at estuary, and the water depth range from 3 to 8 m.

3-2 Perai Barrage

Perai Barrage was located some 8 km from the river estuary. The location is pointed out as the characteristic of this tidal barrage which is far from the estuary.

In 1968, under the tecnical assistance of Japan Overseas Technical Cooperation Agency, the detailed design of Prai Barrage was completed. The construction started in 1979 and the barrage structure was completed in 1981.

The barrage body is made of the rainforced concrete, and its structure is shown in Fig.3-1-4. The gate type is double stage roller gate to regulate the water level on upstream and to prevent intercept an ingression of saline water. The specification of the Barrage and its gates are as follows:

Width of Barrage : 62.2 m
Gate type: Double stage roller gate
Number of gate : Four (4) gates
Dmension of leaf:

Upper leaf	Width 14.6 m, (48 ft)	Height 2.6 m (8.5 ft)
Lower leaf	Wirth 14.6 m, (48 ft)	Height 2.7 m (9.0 ft)
	Width between piers	13.7 m (45 ft)

There are factories and residences at downstream area of right bank near the Barrage, and these areas don't have embankment. The lower elevation of the areas which inoundated on 14 May 1984 are R.L. 1.2 ~ 1.3 m. In this areas, according to the field investigation on 16 Febrary 1988 when tide table water level was R.L. 1.38 m, an ingress water flowed out from the drain ditch.

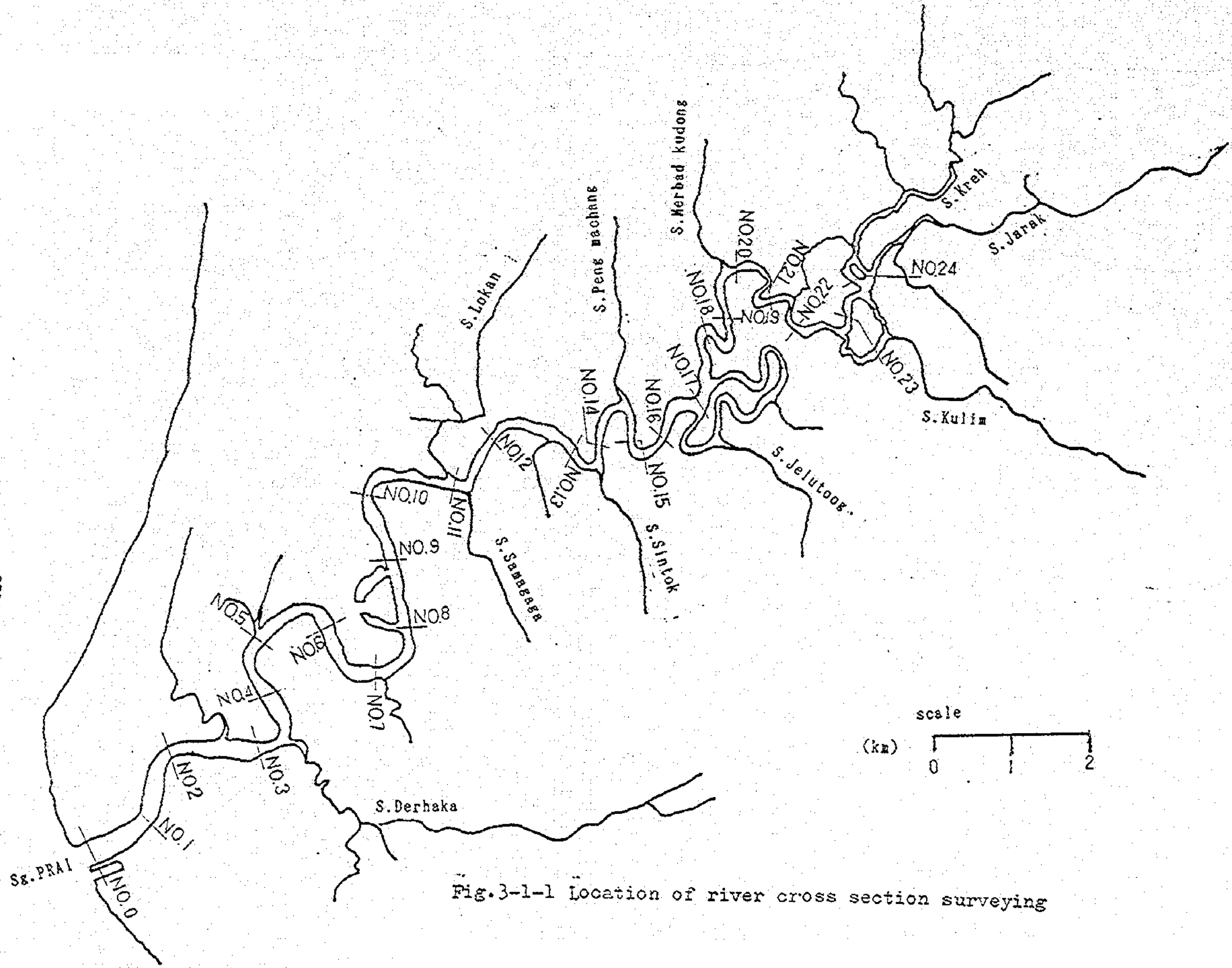

Fig.3-1-1 Location of river cross section surveying

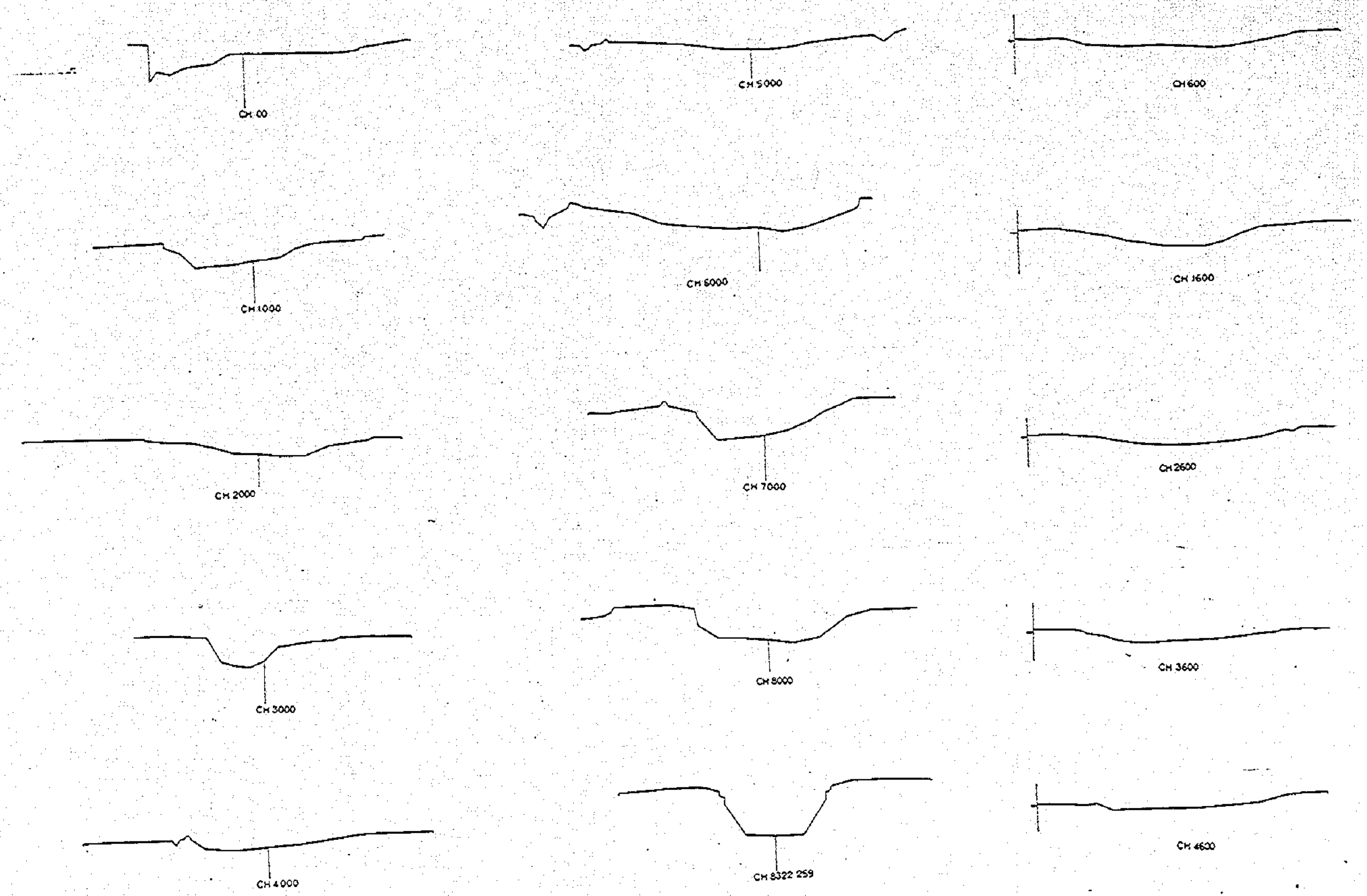

Fig.3-1-2 (1) river cross sections of Sg. Perai

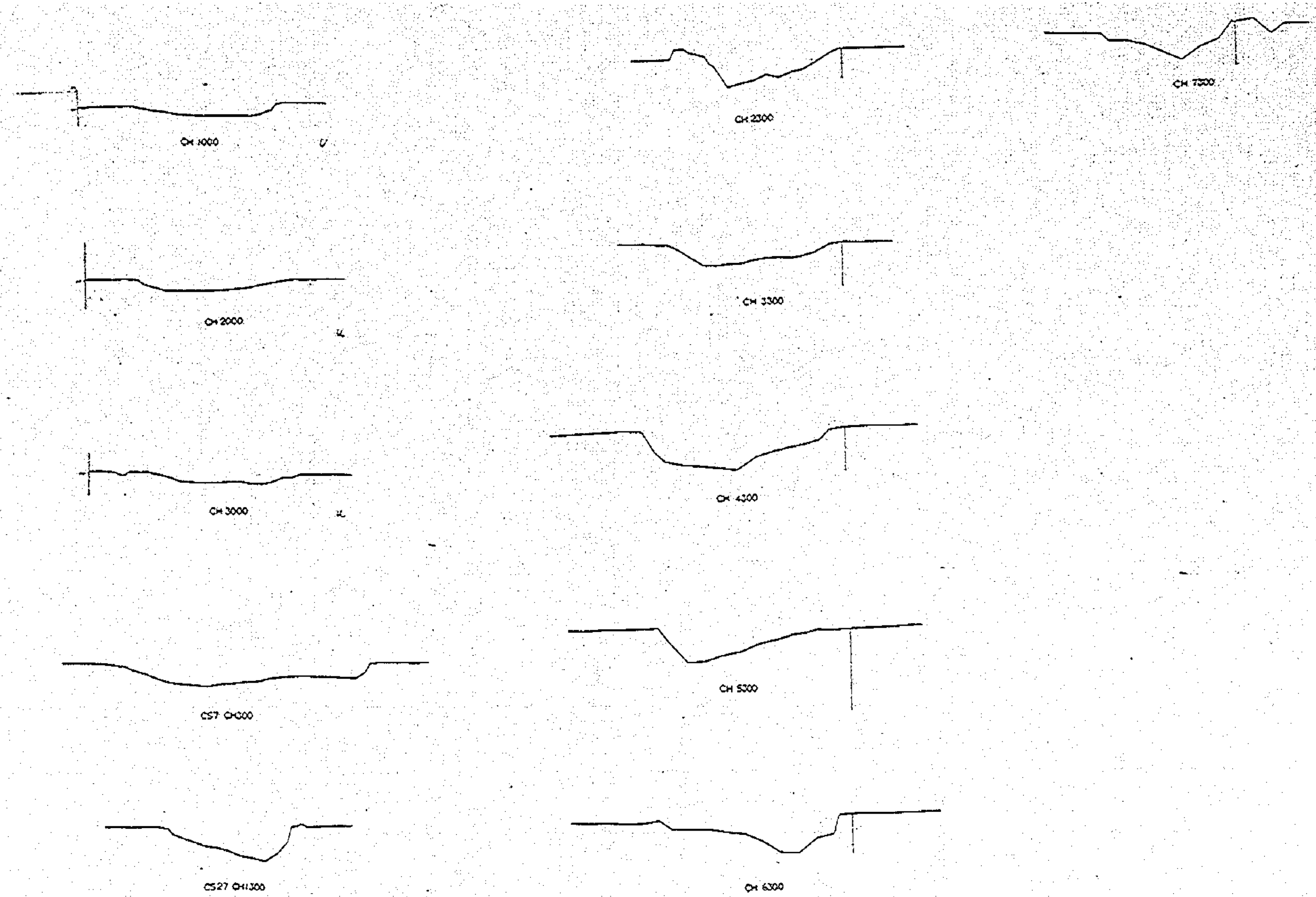

Fig.3-1-2 (2) River cross sections of Sg. Perai

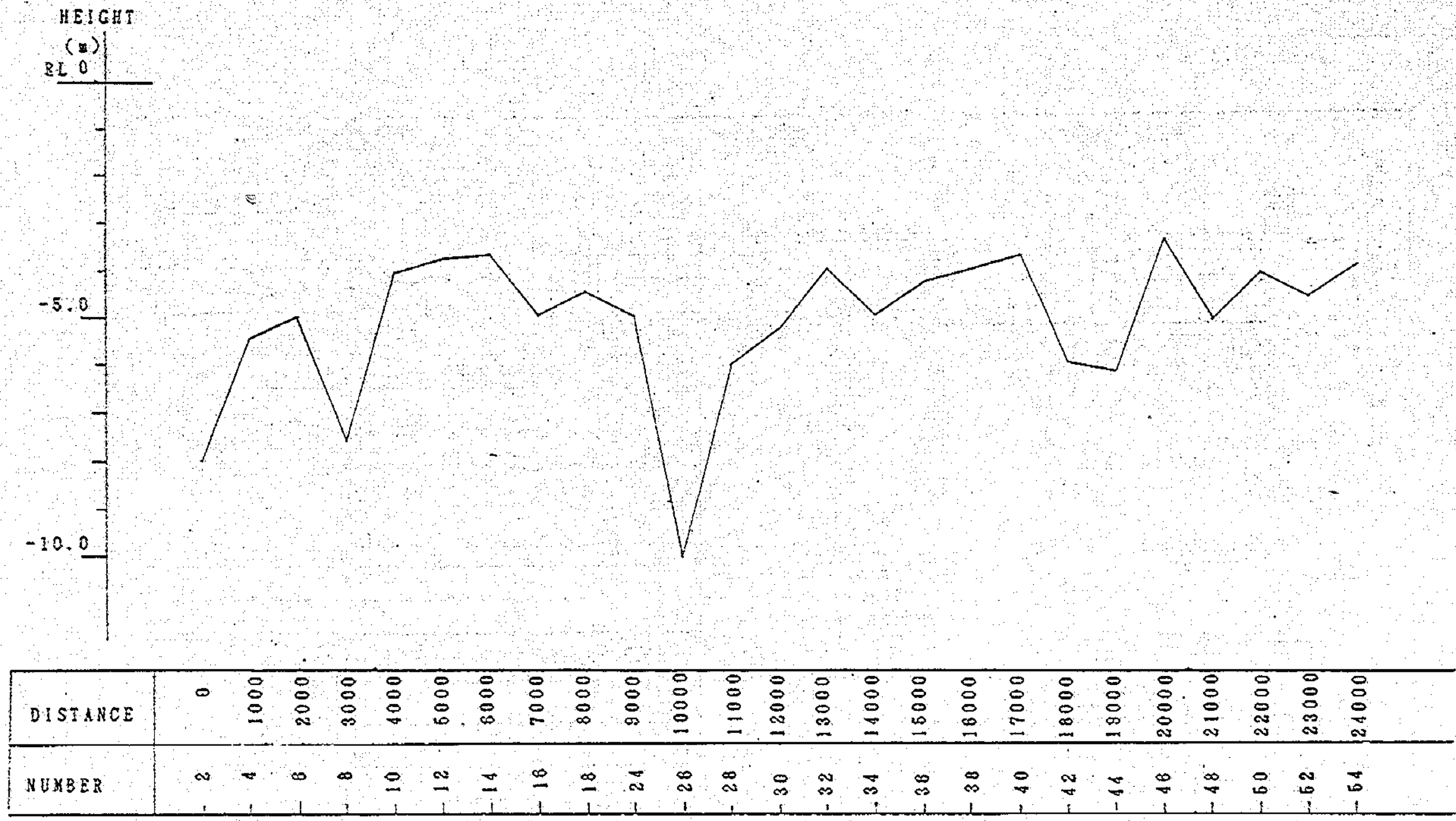

Fig.3-1-3 Longitudinal profile of Sg.Perai

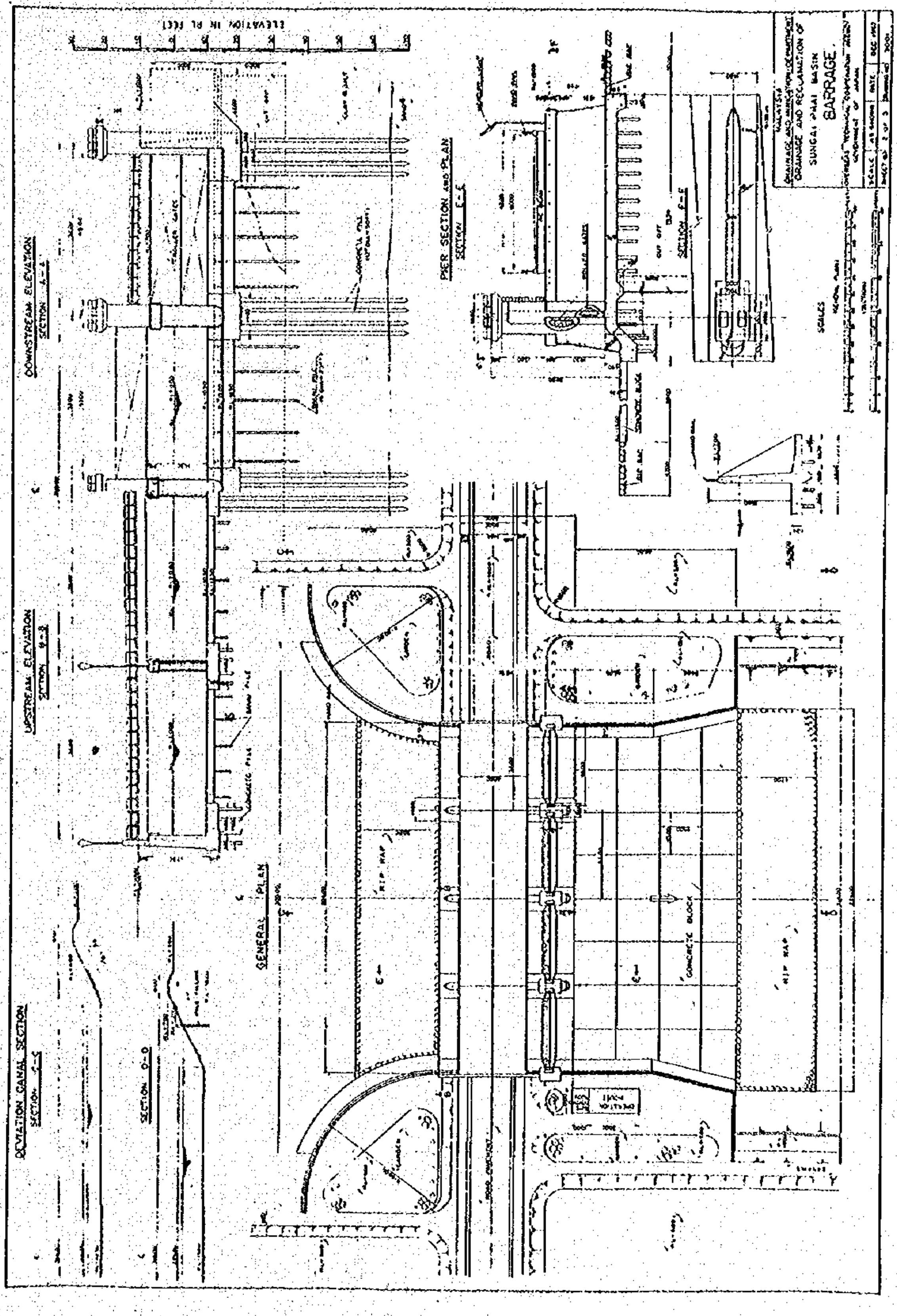

Fig.3-1-4 Profile of Perai Barrage

IV. HYDROLOGICAL ANALYSIS ON SG.PERAI BASIN

4-1 General characteristics on hydrology

4-1-1 Location and topography

Sg.Perai basin is located 5°17′~5°33′ °N latitude and 100°22′~100°38′ °E longitude in the central part of Seberang Perai and has a total catchment area of approximately 497.5 sq.km. The topography is generally flat and lowers coastward. So, there exist many meanders.

Sg.Perai starts from the confluence of three big tributaries; Sg. Kulim, Sg.Jarak and Sg.Kreh that originate from foot of the mountain in the state of Kedah. It flows into the Straits of Malacca, meandering southwesterly. The length of the river is about 20 km and its mean gradient of bed is about 1:4,000.

As for the topographic map, the paddy area at the both sides of Sg. Perai are put in order, because the irrigation schemes has been almost executed. However, for coconuts and rubber fields as well as swamp at both sides of Sg.Perai, the map are not availale. The basin of three big tributaries; Sg.Kulim, Jarak and Kereh is covered with rubber, oil palm and coconuts fields, so that its detailed contour map are not available. But, topographical information on hydrology can be roughly obtained with the maps of series 16,17,28 and 29 that show SUNGAI PETANI, KUALA KETIL, PULAU PINANG & BUTTERWORT and KULIM respectively with a scale of 1:63,360.

The whole basin can be divided as shown in Fig.4-1, and the upstream area of confluence point occupies about 77 %. From this point, it is obvious that rainfall on the basin of three tributaries has a great effect on the discharge of Sg.Perai.

Since flow capacity of rivers is relatively at a loss, poor drainage sometimes causes trouble. But from another point of view, its situation has prevented the concentration of drainage damage from the particular area.

4-1-2 Climate

This area has a tropical climate, but not belonging to tropical monsoon area. The mean air temperature is about 27.5°C throughout the year. The annual mean rainfall is about 2,400 mm. There is a little distinction between dry and rainy season. Judging from the rainfall record, it can be said that periods from December to March and from June to July are dry season, and period from September to November is rainy season. As for April and May, amount of rainfall is relatively above the average, whereas that of August is below the average.

The information of climate on hydrology is observed and recorded by several agencies including D.I.D. and published as following books:
(a)Malaysian Meteological survice;Agrometeorogical Bulletin
(b)D.I.D.,Ministry of Agriculture Malaysia;Hydrological Data, Rainfall and Evaporation Record for Malaysia

Based on book(a), rainfall, mean air temperature, mean sunshine hour and mean evaporation in Peninsular Malaysia are observed at the stations shown in Fig.4-2, and update information in the state of Penang was arranged in Table 4-1. Book(b) has more information concerning rainfall and evaporation. It has no record of evaporation in the state of Pinang, but the data on the outskirts of Pinang shown in Fig.4-3 is available and long term records were arranged in Table 4-2.

(1) Air temperature
The mean annual air temperature is about 27.5°C. It has little obvious fluctuation among months, but generally high during the period from February to June.

On the other hand, fluctuation in a day is rather large within the range of 20~36°C. It hardly falls down below 20°C.

(2) Mean atmospheric pressure
Mean atmospheric pressure is within the range from 1,006 mb to 1,012 mb throughout the year, and it has little fluctuation.

(3) Mean relative humidity
Mean relative humidity is almost more than 80%.

(4) Mean wind speed
The mean wind speed in a day is approximately 2 m/sec on an average. It almost shows a state of calm, because the atomospheric pressure does not vary so much.

(5) Mean sunshine hour
The mean monthly sunshine hour in a day changes within the range of 4~10 hours. It is larger in dry season and shorter in rainy season.

(6) Rainfall
Characteristics of rainfall is described in detail in the section 4-2-3.

(7) Evaporation
Evaporation per day has monthly fluctuation within the range of 3~7 mm/day. It is larger in the period from January to April. Evapotranspiration is considered to be 1.1~1.5 times larger than evaporation owing to the planting stage of crops and sunshine.

4-1-3 Situation of land use and agriculture
The land use on Sg.Perai basin has been progressively developed within the last 30 years not only by the development of land reclamation but also by the development of drainage and irrigation systems. The up-and midstream areas of Sg:Perai have extensive arable land of mainly paddy field and downstream area is covered with Rubber and Coconuts Estates along the industrial and residential lots. Generally, unutilized land resouces do not exist except the swamp that are covered with nipahs and mangroves.

As for the mid-and downstream of tributaries, paddy fields spread along the both sides of rivers with a small scale. Outside of the paddy field is rubber and oil palm. Upstream of Sg.Kulim is covered with Rubber and Oil palm Estates.

Present location of paddy field was drawn by D.I.D. and shown in Fig. 4-4. Detailed situation of present land use is partly shown on the map of Fig.4-5. Agriculture Master Plan of Penang has a plan of conversion from rice to other profitable crops, for example pineapples, bananas, maize, vegetables, sorhum, cocoa and so on.

In this area, the programming of agricultural development is considered with every unit of irrigation scheme shown in Fig.4-6.

Water is severely limited and has little control in off-season. Seberang Perai is almost entirely depend for its water resources on rivers with catchments within the state of Kedah which borders Penang to the north and east.

The main river, supplying much of Penang's domestic and industrial water requirements and up to about 80 % of irrigation needs, is Sg. Muda which has the reservoir formed by the Muda dam.

Most of the other rivers of which Penang makes use for irrigation, although supply is inadequate, have much smaller catchments in the hills east of Penang is the Kulim district of Kedah. These catchments are generally less steep than that of Sg.Muda and are largely covered with rubber and oil palm plantations.

According to the 'UPGRADING OF IRRIGATION & DRAINAGE SCHEMES IN BALIK PULAU AND SEBERANG PERAI, PULAU PINANG : VOLUME III' , water requirements of each crop and irrigation requirements had been examined as shown in Tables 4-3~4-5 and Figs.4-7~4-9, and they concluded as follows within some assumptions:
The double-cropping of rice can be achieved throughout nearly all of the irrigation schemes and it is not necessary therefore to consider in detail alternative cropping programes that might be introduced on a limited area, because the both peak and total water requirements of rice are higher than any other crops considered. For instance the proposed area of vegetable production could be substantially

increased in response to market demands without increasing irrigation demands.

Their programming were based on the assessment of water requirement for 10-day periods that was derived from crop coefficient as shown in eq.(4-1),

$$ETcrop = Kc * ETO \tag{4-1}$$

Where, ETcrop:consumptive use of crop evapotranspiration
Kc:crop coefficient appropriate to the stage of growth
ETO:open water-surface evaporation and grass evapotranspiration

They continued to say with the assumptions for land use in future: Fig.4-10 indicates the arrangement of the Sg.Muda to Sg.Kulim transfer system and the schemes in Seberang Perai which are supplied wholly or in part from Sg.Muda. These comprise about 94 % of the total area. In the average year there is adequate water in Sg.Kulim, Sg.Jarak and Sg.Kereh for there to be no need for water to be transferred from Sg.Muda for the schemes so connected. In the 1-in-5-year case there is adequate available total flow in the three rivers to meet total demands during the ten months from May to February, although for part of this time water would need to be transferred from Sg.Muda. Therefore there are no supply restrictions on the main crop on any of these schemes. However during March and April shortage will occur.

But under the present situation of agriculture and land use, as water is not enough, so that planting schedule and distribution of water are decided by the meeting with farmers.

The most significant topics for land use on hydrology is urban development. The conversion of forested or tree-cropped areas to urban or industrial use will have a major impact on the total quantity and the characteristics of runoff. The main influence of the destruction of vegetative cover or the construction of impermeable surfaces is to decrease the retention capacity of the catchment. Therefore, following changes can be expected as a result of urban development:
(1)the increase in the frequency and magnitude of flood discharge
(2)the reduction of base flow

The programming of urban expansion is designed in Fig.4-11.

4-1-4 Drainage and irrigation system

It is important to establish drainage and irrigation system for agricultural development. Especially, the drainage condition in this area remains unchanged except a portion of the rubber and coconut estate, because it has not been improved by the systemized drainage schemes. The complicated drainage systems had been clarified by the field investigations, and as a result of it, catchment area was classified in accordance with the river system as shown in Fig. 4-1.

The Sg. Perai is a main drainage canal in this basin. But, the drainage is generally poor because the basin has an extremely gentle slope and water level of Sg. Perai is fluctuated by tide. In such a poor condition, people chose to live in high-floor houses. At the new residences and factories, the existing ground is obliged to be levelled up more than 0.6 meter by filling land. At the rubber and coconut estates, the owners are trying to improve the drainage by means of providing bunds, drains and tidal gates.

Since a plot-to-plot irrigation has been adopted in the paddy field, there are few drains and they form a style of plot-to-plot drainage. The drainage blocks are bounded by irrigation canals or roads, in which main drains exist in a natural condition and each of them is connected with Sg. Perai.

In the vicinity of the place where three meandering tributaries of Sg. Perai meet, there are a lot of small streams, each of which forms the drainage block. However, this area is very flat as a whole so that when storm and high tide came together, the paddy field would get into temporary flood, and drainage systems could not be justified. There are a lot of rubber estates in this area with a complete drainage canal which is connected with Sg. Perai at the end, where the barrage was located.

In Sg. Perai basin, no drainage scheme has been taken up except small scaled schemes executed privately by the estate owners and others. Recently, D.I.D. is planning to improve the drainage condition in the upstream of Sg. Perai by gate operation at barrage, keeping the water level equal to be 0.3 meter above sea level.

According to the assessment of adequacy of existing internal drainage system presented by D.I.D. as Sg. Perai drainage and reclamation works, the 5-year 72-hour storm is used as the design storm. It shows that design storm volume and specific storm discharge are represented by eq. (4-2) and (4-3) respectively.

$$\text{design storm volume} = i*A*10 \quad \text{(cu.m)} \tag{4-2}$$

$$\text{specific discharge} = i*10/(72*36) \quad \text{(cu.m/sec/sq.km)} \tag{4-3}$$

Here, i : storm rainfall (170 mm)

A : catchment area of each river (ha)

Therefore, specific discharge for design storm is considered 0.656 cu.m/sec/sq.km.

For each river, the equivalent cross-sectional area and wetted perimeter at bank-full stage were determined, and the average bed slope for the respective river stretch was estimated. Using

Manning's equation and taking n=0.030, the capacities of the rivers were obtained for the purpose of comparing with the design storm discharges of the respective river sub-catchments. As the results, all the rivers except Sg.Jarak and Sg.Datuk appeared to have sufficient capacity. But in this assessment, the following concepts are not taken into account:

(1) hourly distribution of total amount of design rainfall
(2) concept of effective rainfall
(3) hydrological process of runoff
(4) dealings as unsteady flow

They are especially important when the river is influenced by water level of downstream or tide.

The relation between crops other than rice and drainage is mentioned by D.I.D. Manual as follows:
The crops to be protected are generally coconuts and rubber for which it is assumed that the ground surface may be inundated occasionally for periods up to 72 hours. As for the field crops, for example maize, the duration of flooding is allowed less than 24 hours.

As for the utilization of river water, the upstream tributaries, receiving no tidal effect, are used for irrigation and municipal water supply. On the other hand, the mid-and downstream areas of Sg. Perai depend on natural rainfall and artificial irrigation systems. Amount of irrigation water is not sufficient in both areas.

The main canal for irrigation is made by earth lining and terminal facilities are used both for drainage and irrigation. Generally the irrigation systems are more systematic and clear than the drainage systems.

The map shown in Fig.4-6 is the systems of drainage and irrigation. Out of many schemes, 'K1' area which is one of the Sg.Kulim scheme was chosen and investigated to understand representative situation of drainage and irrigation system by the collaboration with D.I.D.

Fig.4-12 shows the chain of irrigation canals and drains, and locations of Sg.Kulim headworks and terminal facilities to intake water. Intake discharge from Sg.Kulim headworks to main irrigation canal is about 2.12 cu.m/sec. Although the canal from Sg.Muda which gradient is reverse has been completed, sufficient water supply can not be expected, because upstream shemes have a priority.

Main irrigation canal is made by earth lining and sometimes crosses the drains by syphon which originate from estate fields. The terminal facilies are controled by D.I.D. based on the regulation with farmers. Almost all area of 'K1' are irrigated through Sg.Kulim headworks, but some are directly irrigated by the pumped up water from Sg.Kulim.

Drainage water from each paddy field lot usually flows through the another lot; this system sometimes called plot-to-plot drainage, and meets together to reach Sg.Kulim or Sg.Perai.

Outline of respective irrigation sheme are as follows:

(1) 'M' area

The paddy fields of 6,700 ha throughout the northern part of Sg. Perai is irrigated from Sg.Muda running through a boundary beween the states of Penang and Kedah. 46 cu.m/sec water in total is pumped up from Sg.Muda at the two stations of Pinang Tunggal and Bumbong Lima, by which the current water requirement is provided.

Maintenance and operation for these irrigation facilities is managed directly by D.I.D., under whom an adequate improvement for them will be made.

(2) 'K' area

In the southern part of Sg.Perai, the paddy field is irrigated in gravity flow from the Sg.Kulim headworks.

Since the above headworks was constructed over 50 years ago, this capacity is unable to meet an expanding tendency of double-cropping in recent years. According to the Pinang Tunggal Pumping Scheme by 'CHENDERAMATA, JABATAN PARIT DAN TAIAYER, PULAU PINANG', the plan is proceeding to increase the 2.83 cu.m/sec capacity to 4.25 cu.m/sec, for which an addtional supply is to be made available by pumping up at the Pinang Tunggal station. By the implementation of this plan, the irrigation source would have some potential. Also, the enlargement of the irrigation canals is planned for increasing demand.

(3) 'J' area

The area surrounded by the water districts of Sg.Muda and Sg.Kulim is irrigated from Sg.Jarak. Since droughty discharge of Sg.Jarak is insufficient having only 0.9 cu.m/sec, the additional supply is conducted from the Pinang Tunggal station through Sg.Kreh.

(4) other area

Almost all of other area is depending on natural rainfall and uncontrolled natural river. There are no need to irrigate for rubber, oil palm and coconuts fields.

4-2 Hydrological analysis

4-2-1 Purpose of analysis

The purpose of analysis is to estimate both ordinary discharge and flood discharge. So, at first all the data on hydrology collected untill now were arranged, then the characteristics of rainfall and runoff were clarified. The records of water level at ARA KUDA station were used for the both estimations of ordinary discharge and hydrograph in case of flood against the designed probable daily rainfall, because Q~H curve was available there. Analyzed conclusions were given as specific discharge for the following convenience, based on the assumption that the catchment is of similar nature to most of the others.

The obtained values will give the boundary conditions of discharge to the hydraulic analysis that is related in next chapter. Conceputual chart of drainage was drawn in Fig. 4-13.

4-2-2 Collected and available data

Collected and available data used in this analysis is as follows:

(1) General information of climate that is already shown in Table 4-1, Table 4-2, Fig. 4-2 and Fig. 4-3.

(2) River gauging records at ARA KUDA station which show the relation between discharge(Q) and water level(H).
(38 times data from 20 Apr. 1987 to 22 Feb. 1988)

(3) Hourly record of water level at ARA KUDA station.
(10 years' record from 1978 to 1987)

(4) Rainfall
There are many rainfall gauging stations in and in the vicinity of Sg.Perai basin as shown in Fig. 4-14. The mark which indicates respective rainfall gauging station is the same as that used in the F/S report in 1968. The period and method of observation are not uniform as shown in Table 4-6.

All of observations except the Lahar Ikan Mati (mark A) is made for daily rainfall, and the total amount of daily rainfall from 8:00 a.m. yesterday to 8:00 a.m. today is recorded as the rainfall on that day. So, time-intensity pattern can not be clear. Therefore, for studying time-intensity patterns for the designed probable rainfall, the data obtained by automatic rain gauges(daily winding type) of Lahar Ikan Mati will be used.

(5) Map
a) Locations of rainfall and water level gauging stations
b) Topographical map that is already related in 4-1-1

4-2-3 Characteristics of rainfall and runoff

1. Mean annual rainfall and its monthly change

All the rainfall records were arranged with a unit of month in Annex A-4, and mean monthly values of each station were caluculated in Table 4-7. The value of -99.9 in Annex A-4 means the data loss or no observation.

Annex A-4 and Table 4-7 show that there are quite a large differences among the years and locations of stations. Mean annual rainfall was shown in Fig. 4-15. It has more rain in mountainous areas.

In order to estimate the mean annual rainfall on Sg. Perai basin, the Thiessen's method was adopted. Table 4-8 indicates the ratio($Ai/\sum Ai$) of Thiessen's triangle area occupied by each sation. Based on Table 4-7, mean annual rainfall was estimated 2371.7 mm.

As for the monthly rainfall, period from September to November has much rain enough to be called rainy season. On the other hand, periods from December to March and from June to July have less rain than average. But, it can not be said that there is clear distinction between rainy and dry seasons, comparing from the tropical monsoon area.

2. Correlation of rainfall according to places

One of the characteristics of rainfall in this area is that its duration time is relatively short, especially during the daytime, and it is limited only within local area. So, correlation of daily rainfall among rainfall gauging stations is very low as shown in Tables 4-9~11., even in case that distance between two stations is not so far. As for the correlations of monthly rainfall, coefficients shown in Table 4-12 are almost more than 0.7, and some of them exceed 0.85. They have a tendency to be higher in case that the two stations are nearer.

Table 4-13 provides the maximum daily rainfall in each year from 1960 to 1966 and the simultaneous rainfall at the other stations. Table 4-14 provides the daily rainfall of more than 100 mm at one of any stations and the simultaneous rainfall at the other stations within the period of 1980~1987.

These tables mean that when it rains considerably much, for example more than 100 mm/day, total amount of rainfall is relatively similar within the whole basin of Sg. Perai, although there are many exceptions which indicate that even the intense rainfall has a great variance.

3. Correlogram

To examine the statistical independency of rainfall sequence, autocorrelation coefficients $R(\tau)$ can be employed. Detail theory and equations are described in Annex A-1.

The autocorrelation diagram can be drawn to facilitate the analysis of structure of time series, with $R(\tau)$ as the ordinate and τ as abscissa. This graphical representation is called a correlogram.

In runoff analysis using daily rainfall and discharge, the actual time series of rainfall must considered to be independent. So, correlograms were drawn in Fig.4-16~25, using the daily rainfall from 1981 to 1987 in each rainfall gauging station. Judging from above figures, the independency of rainfall was considered to be made sure.

4. Characteristics of runoff obtained by linear multiple regression model

Runoff from Sg.Perai basin has been obserbed since 1957 by an automatic water level recorder at the ARA KUDA Road Bridge, midstream of Sg.Kulim, from which a lot of data have been collected. Total catchment area at ARA KUDA is 139.13 sq.km, representing about 85 % of the catchment at Sg.Kulim headworks. Schematic figure and photograph of ARA KUDA water level gauging point is shown in Fig. 4-26. Fig.4-27 shows the fluctuation of water level from 1978 to 1987.

The Q(discharge)~H(water level) curve drawn in Fig.4-28 was obtained by a least squares method using 38 times observations of discharge from 20 Apr.1987 to 22 Feb.1988 and the relation between Q and H was shown in eq.(4-4).

$$Q=(1.5535*(H-5.4864)+0.3118)^2 \qquad (4-4)$$

Here, Q:discharge (cu.m/sec)

H:water level (m)

Although there are little data of discharge in high water level, eq. (4-4) is used to estimate discharge from water level in this analysis. Fig.4-29 represents the fluctuation of discharge from 1978 to 1987. The disscharge was converted from the water level record at 12:00 a.m.. Annex A-5 shows the respective annual fluctuation of discharge. Out of last 10 years' hourly record, last 7 years' record from 1981 to 1987 was used in this runoff analysis, because daily rainfall records observed within above period were available.

The theory and application of linear multiple regression model are described in Annex A-1. The data of daily rainfall used in this analysis was derived from eq.(4-5), because it could not be possible to collect the data in the upstream of Sg.Kulim.

$$r(i) = \frac{R_L(i) + R_P(i) + R_B(i-1) + R_{G-H}(i-1)}{4} \times \alpha$$

$$\alpha = \frac{(R^x_{A-K} + R^x_N)/2}{(R^x_{G-H} + R^x_L + R^x_P + R^x_B)/4} = 1.285 \qquad (4-5)$$

Here, $r(i)$:daily rainfall used in this runoff analysis

$R(i)$:daily rainfall in each gauging station

R^x :mean annual rainfall in each gauging station obtained from Table 4-7.

Following to eq.(4-5), the estimated annual rainfall became 2672.8 mm.

By calculations using computer(FACOM M340 in National Research Institute of Agricutural Engineering, Japan), conclusions were obtined in Table 4-15 and stastical unit graph were drawn in Fig. 4-30. They indicate that the process of runoff is very natual and ratio of runoff which contains base flow is within the range of 17~24 %. The reasons why the ratio of runoff is not so high are considered as follows;

(1) The existing state of land use in up-and midstream of Sg. Kulim is mainly rubber and oil palm, so that runoff from these areas considered to be quite a few, because they have much effect of water storage.

(2) The density of drains in paddy field is very small.

(3) The mean daily evapotranspiration is enough high to reach about 5 mm.

Following figures drawn in Figs. 4-31~37 show the annual comparison between actual discharge observed at ARA KUDA point and estimated discharge by linear multiple regression model with F.M.D. (fixed maximum discharge) equal to be 30 cu.m/sec. In each above figure, the thick solid line represents the former and the dotted line represents the latter. The point is record of daily rainfall. All three values;actual discharge, estimated discharge and daily rainfall, were represented with the same unit of mm/day, by converting the unit of discharge cu.m/sec to mm/day. So, ratio of discharge to rainfall indicates the ratio of runoff.

5. Probable daily rainfall

The first step for the flood estimation is to obtain the probable daily rainfall. In this analysis, Iwai-Kadoya's method was adopted which is widely used in Japan. The theory of analysis is described in Annex A-2.

Using all the data of maximum daily rainfall in each year shown in Table 4-16, the probable rainfalls in various return perids obtained by this method were given in Table 4-17. The values within ( ) are doubtful, because the number of data is at a loss.

As for the probable rainfall on whole basin, arithmetic mean of C, E, F, G-H, I, J-K, L, M, N, P and R stations was calculated out of Table 4-17, and arranged in Table 4-18, because above stations are in the catchment of Sg. Perai. Table 4-18 indicates that the probable daily rainfalls of 5, 10, 20 and 40 years return period are 127.3, 142.5, 157.0 and 171.0 mm/day respectively.

6. Time-intensity pattern for probable daily rainfall

No hourly record of rainfall is available in Sg. Perai basin, so the record of Lahar Ikan Mati(A) was used to analyze the characteristics of rainfall intensity.

Out of records, 20 series of intense rainfalls were abstracted as shown in Fig. 4-38, which had more than 50 mm in a single duration. These rainfalls may be clssified into two categories;(1)the 11 short-duration type at about 10 hours and (2)the 9 long-duration type at the range of 20~30 hours. By observing the rainfall distribution for the latter type, there are 6 cases of 2 peaks type and 3 cases of 3 peaks type. Considering the latter 2 peaks for the 3 peaks type to be one peak, these are transformed into 2 peaks type. The duration for the first peak is about 6 hours and the second peak about 11 hours, and the rainfall ratio for the first peak is 36 % and the second peak 64 %.

The curves in Fig.4-39 were obtained by plotting the following factors on log-log paper; t/24, while t is the rainfall duration in hour and the accumulative hourly rainfall Rt obtained by arranging the rainfall in the order of its intensity(large number first) and by adding these progressively. Among these curves, each rainfall is practically straight line, therefore they may be expressed by the exponential function of Sherman type. The basic formula of it is represented in eq.(4-6).

$$i_t = \frac{a}{t^n} \qquad (4\text{-}6)$$

Here, t :rainfall duration in hour

i_t :averrage rainfall intensity in mm/hr

n :constant

a :constant

Let $a=R_T/24^K$ and $n=1-K$, where R_T is the rainfall in T hours duration and K constant. Then, the above formula may be written as shown in eq.(4-7).

$$R_t = R_T \cdot \left(\frac{t}{24}\right)^K \qquad (4\text{-}7)$$

Here, R is the rainfall in t hours duration. The result of calculating K was given in Fig.4-39 and K is an index to exp ress the time distribution of the rainfall.

The mean K value is 1/4.22(n=0.763) for the short-duration type, 1/2.25(n=0.555) for the long-duration type. Above mentions were arranged in Table 4-19.

The designed daily rainfall is to be divided by 24 hours by using eq.(4-7). Table 4-20 expresses the ratio of R_t against R_T , which was already amended by taking the duration time into account. Time-intensity pattern for probable daily rainfall R_T can be derived by multipling R_T by the ratio in Table 4-20.

7. Base flow and accumulated initial loss of rainfall in case of flood

In reviewing the flood data, the discharge does not increase instantaneously when there is rainfall on basin. It usually takes a certain duration of rainfall and a certain amount of accumulated rainfall, before the discharge increases strictly. Initial loss of rainfall is considered to depend on the retention in the basin. Therefore, the index factor called accumulated initial loss of rainfall is adopted to estimate the effective rainfall.

Hourly effective rainfall is calculated by obtaining the accumulated loss of rainfall from the difference between accumulated rainfall and accumulated effective rainfall. By considering the discharge Q_e that is larger than base flow, accumulated effetive rainfall is represented as shown in eq.(4-8).

$$\Sigma R_e = (\Sigma Q_e * \Delta t) / A \qquad (4\text{-}8)$$

Here, Q_e:discharge that are obtained by substracting the base flow from observed discharge

A :catchment area

Δt:time increment

ΣR_e:accumulated effective rainfall

Q represents the direct runoff. Accumulated loss of rainfall is calculated by eq.(4-9).

$$\Sigma R_\ell = \Sigma R - \Sigma R_e \qquad (4\text{-}9)$$

Here, ΣR :accumulated rainfall

Base flow is considered to be a constant discharge before the rainfall. Above mentions are shown in Fig.4-40.

The minimum discharge out of 10 days' discharge before the intensive rainfall was adopted as base flow in this analysis.

Annex A-6 shows the rainfalls selected and used in this analysis. Selection was done by the criteria that any of daily rainfall records at L,P,R and G-H exceeded 100 mm. Annex A-7 shows not only the fluctuation of observed discharge at ARA KUDA and obtained base flow, but also the period that is used to caluculate the accumulated effective rainfall. Decision of period was done by the judgement based on previous experience of analysis, because detail hourly records of rainfall could not be collected.

Based on above procedure, each series of rainfall were analyzed and the relations between accumulated rainfall and accumulated loss of rainfall were arranged in Table 4-21. The base flow fluctuated within the range from 0.7 to 4.2 cu.m/sec. Monthly change indicates that it is generally larger in rainy season as shown in Fig.4-41.

Moreover, the relation between accumulated rainfall and accumulated loss of rainfall shown in Fig.4-42 indicates that almost more than 70 % of rainfall is lost even when accumulated rainfall exceeds 100 mm.

4-2-4 Estimation of ordinary discharge

Estimation of ordinary discharge was done by three methods. Two of three are based on simple calculations using the records of water level at ARA KUDA. The other is based on the coclusions obtained by linear multiple regression model. As the definition of ordinary discharge is not clear, several concepts are introduced.

(1) Discharge based on mean water level

Mean water level at ARA KUDA for 10 years from 1978 to 1987 was 6.157 m. It is easily converted to the discharge by Q~H curve shown in eq.(4-4), and it becomes 1.83 cu.m/sec. Therefore, specific discharge is estimated to be 0.013 cu.m/sec/sq.km, because the catchment area is 139.13 sq.km.

(2) Discharge based on most frequent water level

Frequency of water level at ARA KUDA for 10 years within the period above mentioned was arranged in Table 4-22 and Fig.4-43.

They indicate that most frequent water level is 6.005 m, which value is converted to 0.009 cu.m/sec/sq.km.

(3) Discharge based on base flow and ratio of runoff

This discharge is based on the conclusions of analysis with linear multile regression model. Considering that ordinary discharge consists of both base flow and runoff by mean annual rainfall, the discharge is represented as shown in eq.(4-10). It differs owing to F.M.D..

$$Qrb = a*r + b*Qb/A \qquad (4\text{-}10)$$

Here, Qrb:specific ordinary discharge (cu.m/sec/sq.km)

r:ratio of runoff shown in Table 4-15 (%)

Qb:base flow shown in Table 4-15 (cu.m/sec)

A:catchment area (139.13 sq.km)

a, b:constans(refer to eq.(4-5))

$$a=0.000752 \quad \left(= \frac{2371.7\times10^{-3}\times10^{6}\times10^{-2}}{365.25\times86400}\right)$$

$$b=0.887 \quad \left(= \frac{2371.7}{2672.8}\right)$$

Following to eq.(4-10), the discharge fluctuated as follows:

F.M.D.(cu.m/sec)	Qrb(cu.m/sec/sq.km)
4.0	0.013
8.0	0.016
12.0	0.017
16.0	0.018
20.0	0.018
30.0	0.018

In this study, 0.013 cu.m/sec/sq.km is adopted as ordinary discharge. It means the discharge at the barrage to be about 5.85 cu.m/sec.

4-2-5 Estimation of hydrograph under probable daily rainfall

Kinematic wave method was adopted to estimate the hydrograph under probable daily rainfall in this analysis. Detailed theory is described in Annex A-3.

This method deals with the flow as non-linear phenonena which have two processes, as shown in Fig.4-44. The former is the runoff on uniform gradient slope, while the latter is the flow in river or drain.

Before the calculation using computer, both effective rainfall and basin model must be given. Furthermore, the parameter N called equivalent roughness coefficient is prepared for some cases in advance, which will be finally decided by the comparison between the actual and calculated hydrographs.

As for the caluculation of hourly effective rainfall against the probable daily rainfall, the new parameter Rlmax which represents the maximum accumulated loss of rainfall is introduced. It indicates the boundary of accumulated rainfall, and the accumulated rainfall which exceeds its value is all accounted as effective rainfall. In this analysis, three values;100, 150 and 200 mm, are prepared as Rlmax, judging from the conclusions derived from Fig.4-42.

Relation between accumulated rainfall and accumulated loss of rainfall is shown in Fig.4-45 and eq.(4-11).

(Rlmax=100) (4-11)

$\Sigma R\ell = (1.0 - 0.0025 * \Sigma R) * \Sigma R \quad (0 < \Sigma R < Rlmax)$

$\Sigma R\ell = Rlmax \quad (Rlmax < \Sigma R)$

(Rlmax=150)

$\Sigma R\ell = (1.0 - 0.00167 * \Sigma R) * \Sigma R \quad (0 < \Sigma R < Rlmax)$

$\Sigma R\ell = Rlmax \quad (Rlmax < \Sigma R)$

(Rlmax=200)

$\Sigma R\ell = (1.0 - 0.00125 * \Sigma R) * \Sigma R \quad (0 < \Sigma R < Rlmax)$

$\Sigma R\ell = Rlmax \quad (Rlmax < \Sigma R)$

Basin model was roughly assumed as shown in Fig.4-46, from the topographical information in the up-and midstream of Sg.Kulim basin.

Caluculations were executed against 72 cases, which were due to two types of time-intensity pattern, four types of return period, three types of Rlmax and three types of equivalent roughness coefficient N that were 0.7, 1.0 and 1.5 . As for the base flow, the value of 0.03 cu.m/sec/sq.km was used in whole cases, which is derived from Fig. 4-41.

Conclusions are arranged in Figs.4-47~70, and in Annex A-8 as digital values. The values are shown as the specific discharge with a unit of cu.m/sec/ha. They indicate that the peak discharge and time-lag change according to Rlmax and N.

The parameters of Rlmax and N should be determined by the coincidence of observed and calculated hydrographs, using the actual hourly rainfall. But in this study, as the record of the actual hourly rainfall could not be collected, only the tendency of hydrograph was discussed.

On the other hand, actual discharge fluctuated as shown in Annex A-7, and annual maximum discharge and return period of peak discharge were arranged in Table 4-23 and Table 4-24 respectively, although the discharge more than 25.5 cu.m/sec or 0.0018 cu.m/sec/ha were not sure because the cross-sectional area of flow changes rapidly at this level as shown in Fig.4-26 and the relation between water level and discharge did not observed at such a high water level as shown in Fig.4-28.

Therefore, by the integrated discussion and judgement, following cases are turned in as the boundary condition of hydraulic analysis that is related in next chapter.

	Return period		
N	1/10	1/20	1/40
1.0	Rlmax=150	150	150
	(Fig.4-63)	(Fig.4-66)	(Fig.4-69)
0.7		150	100
		(Fig.4-66)	(Fig.4-68)

(Long-duration type of rainfall was selected for time-intensity pattern)

The hydrographs using N equal to be 1.0 are within the range of expectation under the existing state of land use and drainage system. The peak discharge at barrage will be about 120 cu.m/sec in case that once a 10 years return period of probable rainfall should occure on whole basin uniformly.

The hydrographs using N equal to be 0.7 are cases considering the development in future, and in the most dangeous case;once a 40 years return period, 100 mm of Rlmax and 0.7 of N, the peak discharge at barrage would exceed 550 cu.m/sec.

The more accurate analysis would be done not only by the detailed observation of rainfall and discharge but also by the detailed investigation of topography and land use , because characteristics of runoff differ due to above items. The imaginary basin model is shown in Fig.4-71.

4-3 Conclusions

The purpose of this chapter was to estimate both ordinary discharge and hydrograph of flood under probable daily rainfall.

As the preparation of analysis, general characteristcs of Sg.Perai basin on hydrology were arranged by the data collection and field investigation. They sugested that considerable changes of land use, kinds of crops and increment of drainage and irrigation system; total quantity of water supply from other basin and density of drains, have possibilites to affect the hydrological environment of this area.

The first step of analysis was to grasp characteristics of rainfall. By the arrangement of collected data and its statistical treatment, following items were clarified:

(1) mean annual rainfall and its monthly change
(2) correlation of rainfall according to places
(3) correlogram of daily and monthly rainfall
(4) prabable daily rainfalls of 5, 10, 20 and 40 years return period
(5) time-intensity pattern for above probable daily rainfall
(6) monthly change of base flow
(7) relation between accumulated rainfall and accumulated loss of rainfall

Characteristics of runoff was examined with linear multiple regression model using water level record at ARA KUDA. Its conclusion indicated that ratio of runoff would change within the range of 17~24 %, according to the fixed maximum discharge(F.M.D.). Furthermore, it can be used as material by the combination with water use to discuss water resources, because detailed characteristics of runoff were obtained. The example of its application is shown in the latter half of Annex A-1 which discusses about water balance in the area.

Ordinary discharge although its definition was not clear, was estimated to be 0.013 cu.m.sec/sq.km, by the arithmetic mean of water level, the most frequent water level and the discharge based on base flow and ratio of runoff which were derived from runoff analysis with linear multiple regression model.

As for the estimation of hydrograph under probable rainfall, kinematic wave method was adopted. The parameter of Rlmax which was used to calculate hourly effective rainfall was introduced and 72 cases of calculation were excuted. They were due to time-intensity pattern of rainfall, return period, Rlmax and equivalent roughness coefficient N. Conclusions were arranged in Figs.4-47~70. This method also can be used to estimate the situation in future by the simple change of basin model and parameters of Rlmax and N. The imaginary examples are shown in Fig4-71 and Fig.4-72.

Obtained values will be selected by integrated discussion and used as the boundary conditions of discharge for the hydraulic analysis which is related in next chapter.

Table 4-1 General information of hydrology (1987)

	Rainfall (mm/month)					Mean Air Temperature (°C)					Mean Sunshine (hours/day)					Mean Evaporation (mm/day)				
	①	②	③	④	⑤	①	②	③	④	⑤	①	②	③	④	⑤	①	②	③	④	⑤
Jan.	15	9	5	0	3	27.5	27.1	27.2	–	26.9	8.1	–	–	7.9	7.5	5.2	–	–	5.0	–
Feb.	0	0	0	0	0	28.0	27.9	27.8	–	27.8	10.4	–	–	10.2	9.8	6.6	–	–	6.6	–
Mar.	128	129	89	51	104	27.9	28.0	27.9	28.1	28.5	7.8	–	–	7.6	7.0	4.9	–	–	4.8	–
Apr.	199	125	148	235	121	28.2	28.3	28.1	28.5	28.9	7.7	–	–	8.0	7.3	4.4	–	–	4.8	–
May	286	214	313	160	191	27.6	27.6	27.7	27.9	28.2	6.3	–	–	7.0	6.3	3.5	–	–	4.2	–
Jun.	187	98	155	138	131	28.1	28.2	28.2	28.2	28.4	7.0	–	–	6.7	6.3	3.7	–	–	3.9	–
Jul.	195	104	199	116	154	27.4	27.7	27.6	27.5	27.9	8.1	–	–	7.8	7.1	4.0	–	–	4.0	–
Aug.	251	133	284	271	212	26.9	27.0	27.0	27.1	27.5	4.9	–	–	5.4	4.7	3.2	–	–	3.9	–
Sep.	348	356	341	359	322	26.6	26.7	26.7	–	27.2	4.7	–	–	5.1	4.7	2.9	–	–	3.5	–
Oct.	321	251	256	176	378	27.1	27.2	27.1	–	27.9	6.2	–	–	6.3	6.2	3.7	–	–	4.0	–
Nov.	431	312	366	474	334	26.6	26.8	26.8	27.1	27.3	4.9	–	–	5.0	4.4	2.9	–	–	3.1	–
Dec.	106	98	103	145	128	27.2	26.8	26.9	26.9	27.0	6.6	–	–	5.8	5.7	4.4	–	–	3.6	–

①～⑤ are station number shown in Fig. 4-2

Table 4-2 Monthly change of daily evaporation

(mm/day)

station no.	6108301	5903351	5003328	5005304
Jan.	6.3	6.4	4.9	5.4
Feb.	7.0	6.2	5.2	6.0
Mar.	6.8	6.5	5.3	5.6
Apr.	5.9	5.8	5.2	5.4
May	4.9	5.2	4.9	5.1
Jun.	4.7	4.9	4.9	4.8
Jul.	4.8	5.0	4.9	4.7
Aug.	4.8	5.1	4.9	4.8
Sep.	4.6	5.0	4.7	4.8
Oct.	4.4	4.6	4.6	5.1
Nov.	4.0	4.7	4.5	5.3
Dec.	4.8	5.2	4.4	4.9
Total (mm/year)	1905	1969	1778	2010

(ref.) Fig. 4-3

current equipment installed is American class A pan, Aluminium

Table 4-3 Crop factors

RICE						MAIZE	
'125-day' varieties			'140-day' varieties				
10-day period after transplanting		k_c	10-day period after transplanting		k_c	10-day period	k_c
1	(Transplanting)	1.25	1	(Transplanting)	1.25	1	0.75
2	Vegetative growth	1.25	2	Vegetative growth	1.25	2	0.75
3		1.29	3		1.28	3	0.85
4		1.36	4		1.32	4	1.05
5	Reproduction	1.40	5		1.38	5	1.10
6		1.35	6		1.39	6	1.10
7	Heading	1.27	7	Reproduction	1.33	7	1.10
8	(Irrigation discontinued)	1.19	8		1.26	8	1.03
9	Ripening	0	9	Heading	1.18		
10	(Harvesting)	0	10	(Irrig. discontinued)	1.1/0		
			11	Ripening	0		
			12	(Harvesting)	0		

Notes: (1) k_c taken as zero for last two periods of rice season as irrigation is not required.

Table 4-4 Average rice irrigation requirements

Month	Average monthly requirements Field (mm/d)	Intake (mm/d)	Intake (L/s/ha)	Total monthly requirement (m^3/ha)
January	1.00	1.67	0.19	509
February	-	-	-	-
March	5.20	8.67	1.00	2,678
April	7.43	12.38	1.43	3,707
May	4.30	7.17	0.83	2,223
June	3.56	5.93	0.69	1,788
July	1.57	2.62	0.30	804
August	3.00	5.00	0.58	1,553
September	3.06	5.11	0.59	1,529
October	1.40	2.33	0.27	723
November	2.87	4.78	0.55	1,425
December	3.23	5.39	0.62	1,660
			Total	18,599 m^3/ha

Table 4-5 Maize irrigation requirements

Schedule	March 2	March 3	April 1	April 2	April 3	May 1	May 2	May 3	June 1	June 2	June 3
						(mm/d)					
A : Crop irrigation requirement	–	0.9	1.2	1.2	2.4	1.5	0.4	2.4	3.1	–	–
B : " "	–	–	1.2	0.8	1.6	1.4	0.4	2.4	3.3	2.0	–
C : " "	–	–	–	0.8	1.2	0.7	0.3	2.4	3.3	2.2	1.6
						(l/s/ha)					
A + B : Average field required	–	0.09	0.23	0.19	0.39	0.28	0.08	0.46	0.62	0.19	–
Average intake required	–	0.13	0.34	0.28	0.57	0.41	0.12	0.68	0.91	0.28	–
B + C : Average field required	–	–	0.12	0.15	0.27	0.20	0.07	0.46	0.64	0.41	0.15
Average intake required			0.18	0.22	0.40	0.29	0.10	0.68	0.94	0.60	0.22

Notes: 1. Field application efficiency taken as 60%

2. Distribution efficiency taken as 68%

Table 4-6 List of collected data

year \ gauging station	A			B			C			D			E			F			G-H			I			J-K			L			M			N			O			P			Q			R		
	①	②	③	①	②	③	①	②	③	①	②	③	①	②	③	①	②	③	①	②	③	①	②	③	①	②	③	①	②	③	①	②	③	①	②	③	①	②	③	①	②	③	①	②	③	①	②	③
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1957												○			○			○			○			○						○			○			○												
1958						○		○	○			○		○	○		○	○		○	○			○		○	○			○			○		○	○					○							
1959						○		○	○		○	○		○	○		○	○		○	○		○	○		○	○		○	○			○		○	○					○							
1960			○			○		○	○		△	△		○	○		○	○		○	○		○	○		△	○		○	○			○		○	○					○							
1961			○			○		○	○		○	○		○	○		○	○		○	○		○	○		○	○		○	○			○		○	○					○							
1962			○			○		○	○		○	○		○	○		○	○		○	○		○	○		○	○		○	○			○		○	○					○							
1963			○			○		○	○		○	○		○	○		○	○		○	○		○	○		○	○		○	○			○		○	○					○							
1964			○			○		○	○		○	○		○	○		○	○		○	○		○	○		○	○		○	○			○		○	○					○							
1965		△	△			○		○	○		○	○		○	○		○	○		○	○		○	○		○	○		○	○			○		○	○					○							
1966		△	△			○		○	○		○	○		○	○		○	○		○	○		○	○		○	○		○	○			○		○	○					○							
1967		△						○	○		○	○								○	○		○	○		○			○	○											○							
1968		○						○	○		○	○								○	○		○	○		○			○	○											○							
1969		○						○	○		○	○								○	○		○	○		○			○	○											○							
1970		○						○	○		○	○								○	○		○	○		○			○	○											○							
1971		○	○					○	○		○	○								○	○		○	○		○	○		○	○											○	○						
1972		○	○					○	○		○	○								○	○		○	○		○	○		○	○											○	○						
1973		○	○					○	○		○	○								○	○		○	○		○	○		○	○											○	○						
1974		○	○					○	○		○	○								○	○		○	○		○	○		○	○											○	○						
1975		△	△					○	○		○	○								○	○		○	○		○	○		○	○											○	○						
1976								○	○		○	○											○	○		○	○		○	○											○	○						
1977								○	○		○	○											○	○		○	○		○	○											○	○						
1978	○	○	○					○	○		△									○	○		○	○		○	○		○	○											○	○						
1979	○	○	○					○	○		○	○								○	○	○	○	○		○	○		○	○											○	○						
1980	○	○	○					○	○		○	○								○	○	○	○	○		○	○		○	○											○	○						
1981	○	○	○	○	○	○	○	○	○				○	○	○				○	○	○							○	○	○							○	○	○	○	○	○	○	○	○	○	○	○
1982	○	○	○	○	○	○	○	○	○				○	○	○				○	○	○							○	○	○							○	○	○	○	○	○	○	○	○	○	○	○
1983	○	○	○	○	○	○	○	○	○				○	○	○				○	○	○							○	○	○							○	○	○	○	○	○	○	○	○	○	○	○
1984	○	○	○	○	○	○	○	○	○				○	○	○				○	○	○							○	○	○							○	○	○	○	○	○	○	○	○	○	○	○
1985	○	○	○	○	○	○	○	○	○				○	○	○				○	○	○							○	○	○							○	○	○	○	○	○	○	○	○	○	○	○
1986	○	○	○	○	○	○	○	○	○				○	○	○				○	○	○							○	○	○							○	○	○	○	○	○	○	○	○	○	○	○
1987	○	○	○	○	○	○	○	○	○				○	○	○				○	○	○							○	○	○							○	○	○	○	○	○	○	○	○	○	○	○

(remark)

① daily rainfall

② monthly rainfall

③ maximum daily rainfall in a year

○ : all data exist

△ : some data loss

(blank) : no data

Table 4-7 Monthly change of rainfall

(mm)

sta. no.	Jan.	Feb.	Mar.	Apr.	May	Jun.	Jul.	Aug.	Sep.	Oct.	Nov.	Dec.	Total
A	45.6	76.6	88.0	177.5	214.8	152.9	163.5	177.0	336.5	332.5	200.5	113.9	2079.4
B	30.6	34.3	49.9	130.9	112.3	50.1	63.6	86.6	205.5	141.9	115.4	36.6	1057.6
C	62.9	73.7	115.7	186.4	212.6	150.8	172.5	197.1	314.8	340.4	260.2	140.7	2234.3
D	81.6	142.6	164.1	202.2	218.9	170.8	164.1	202.3	345.2	343.8	230.7	122.7	2389.1
E	59.0	69.6	126.2	147.8	196.1	122.1	125.0	154.4	236.7	226.6	213.7	96.8	1774.0
F	126.0	68.9	136.1	188.1	215.2	128.4	160.3	171.3	244.1	312.3	282.3	190.8	2224.0
G - H	73.8	81.9	148.5	222.5	230.2	134.8	154.1	193.5	279.0	330.6	284.3	146.9	2280.2
I	64.1	63.4	108.7	170.8	164.5	139.3	143.3	185.2	262.7	332.3	213.2	137.3	1984.8
J - K	92.0	114.0	179.0	278.6	257.4	176.8	190.6	217.1	291.9	459.0	354.7	206.2	2817.3
L	74.6	74.1	131.8	180.7	194.9	96.3	125.9	143.2	250.1	329.9	247.1	142.8	1971.4
N	124.0	120.8	214.5	281.2	230.6	145.1	209.8	195.8	272.0	399.7	385.9	232.8	2812.2
O	70.3	87.4	173.3	248.2	244.0	102.5	159.9	126.6	328.6	300.1	261.1	125.6	2227.7
P	84.6	95.5	151.1	214.7	206.9	123.9	142.8	175.7	266.0	356.3	287.7	158.6	2266.8
Q	67.8	62.9	113.0	177.6	204.4	83.7	132.5	129.4	283.3	278.6	176.4	95.3	1804.9
R	51.6	94.5	161.6	236.1	213.9	108.5	117.2	195.9	330.1	313.8	280.9	127.1	2241.2

Table 4-8 Average annual rainfall

blok	average total rainfall* (mm/yr.)	ratio of area $(a_i / \Sigma a_i) \times 100$** (%)
A	2079.4	–
B	1057.6	1.06
C	2234.3	7.58
D	2389.1	3.76
E	1774.0	3.89
F	2224.0	19.95
G-H	2280.2	4.34
I	1984.8	6.25
J-K	2817.3	19.74
L	1971.4	6.89
M	–	–
N	2812.2	16.96
O	2227.7	1.18
P	2266.8	1.25
Q	1804.9	4.16
R	2241.2	2.99
mean or total	2371.7	100.00

* from Table 4-7
** ratio of area occupied by each rainfall gauging station (by Thiessen's method)

Table 4-9 Coefficient of correlation of daily rainfall

	A	B	C	E	G-H	I	L	O	P	Q	R
A	1.000 (3652)	0.780 (2556)	0.545 (2556)	0.685 (2556)	0.563 (2556)	0.626 (731)	0.127 (2556)	0.481 (2556)	0.137 (2556)	0.550 (2556)	0.611 (2556)
B		1.000 (2556)	0.538 (2556)	0.806 (2556)	0.602 (2556)	– (–)	0.111 (2556)	0.483 (2556)	0.118 (2556)	0.542 (2556)	0.626 (2556)
C			1.000 (2556)	0.580 (2556)	0.483 (2556)	– (–)	0.187 (2556)	0.409 (2556)	0.176 (2556)	0.424 (2556)	0.547 (2556)
E				1.000 (2556)	0.708 (2556)	– (–)	0.145 (2556)	0.553 (2556)	0.147 (2556)	0.615 (2556)	0.737 (2556)
G-H					1.000 (2556)	– (–)	0.176 (2556)	0.556 (2556)	0.176 (2556)	0.560 (2556)	0.818 (2556)
I						1.000 (731)	– (–)	– (–)	– (–)	– (–)	– (–)
L							1.000 (2556)	0.192 (2556)	0.835 (2556)	0.141 (2556)	0.172 (2556)
O								1.000 (2556)	0.224 (2556)	0.649 (2556)	0.537 (2556)
P									1.000 (2556)	0.154 (2556)	0.172 (2556)
Q										1.000 (2556)	0.572 (2556)
R											1.000 (2556)

(n) : the number of samples

condition of calculation : all daily data

Table 4-10 Coefficient of correlation of rainfall

	A	B	C	E	G-H	I	L	O	P	Q	R
A	1.000 (1426)	0.740 (1095)	0.440 (1148)	0.624 (1132)	0.485 (1313)	0.554 (396)	-0.005 (1423)	0.365 (1205)	0.005 (1461)	0.457 (1200)	0.530 (1221)
B		1.000 (756)	0.413 (970)	0.758 (914)	0.531 (1197)	- (-)	-0.029 (1311)	0.352 (1034)	-0.020 (1353)	0.438 (1046)	0.543 (1068)
C			1.000 (669)	0.456 (963)	0.360 (1221)	- (-)	-0.006 (1234)	0.192 (1021)	-0.020 (1275)	0.236 (1028)	0.407 (1084)
E				1.000 (756)	0.651 (1176)	- (-)	0.002 (1308)	0.427 (1017)	0.002 (1340)	0.518 (1038)	0.670 (1032)
G-H					1.000 (1059)	- (-)	0.035 (1441)	0.440 (1217)	0.029 (1460)	0.457 (1224)	0.768 (1182)
I						1.000 (338)	- (-)	- (-)	- (-)	- (-)	- (-)
L							1.000 (971)	0.007 (1297)	0.776 (1116)	-0.032 (1316)	0.013 (1370)
O								1.000 (758)	0.042 (1325)	0.525 (1031)	0.389 (1107)
P									1.000 (1032)	-0.020 (1345)	0.011 (1402)
Q										1.000 (809)	0.454 (1130)
R											1.000 (916)

(n) : the number of samples

condition of calculation : in case one or both of two gauging stations observe rainfall

Table 4-11 Coefficient of correlation of daily rainfall considering 1-day delay (L, P gauging station)

	A	B	C	E	G-H	I	L	O	P	Q	R
A	1.000 ( )	( )	( )	( )	( )	( )	0.467 (2556)	( )	0.452 (2556)	( )	( )
B		1.000 ( )	( )	( )	( )	( )	0.453 (2555)	( )	0.443 (2555)	( )	( )
C			1.000 ( )	( )	( )	( )	0.399 (2555)	( )	0.382 (2555)	( )	( )
E				1.000 ( )	( )	( )	0.515 (2555)	( )	0.501 (2555)	( )	( )
G-H					1.000 ( )	( )	0.568 (2555)	( )	0.527 (2555)	( )	( )
I						1.000 ( )	- (-)	( )	- (-)	( )	( )
L							1.000 (2556)	0.641 (2555)	0.835 (2555)	0.572 (2555)	0.560 (2555)
O								1.000 ( )	0.617 (2555)	( )	( )
P									1.000 (2556)	0.550 (2555)	0.528 (2555)
Q										1.000 ( )	( )
R											1.000 ( )

(n) : the number of samples

condition of calculation : all daily data

Table 4-12 Coefficient of monthly rainfall

	A	B	C	D	E	F	G-H	I	J-K	L	M	N	O	P	Q	R
A	1.000	0.795	0.832	0.678	0.727	0.363	0.756	0.752	0.647	0.695	-	0.390	0.722	0.634	0.775	0.803
B		1.000	0.790	-	0.897	-	0.743	-	-	0.675	-	-	0.768	0.669	0.708	0.727
C			1.000	0.753	0.791	0.859	0.827	0.828	0.754	0.767	-	0.778	0.819	0.755	0.789	0.881
D				1.000	0.776	0.631	0.708	0.747	0.580	0.609	-	0.473	-	0.595	-	-
E					1.000	0.729	0.735	0.826	0.678	0.656	-	0.634	0.787	0.705	0.788	0.833
F						1.000	0.834	0.768	0.838	0.851	-	0.798	-	0.830	-	-
G-H							1.000	0.882	0.781	0.835	-	0.759	0.841	0.805	0.807	0.952
I								1.000	0.742	0.789	-	0.748	-	0.776	-	-
J-K									1.000	0.824	-	0.803	-	0.895	-	-
L										1.000	-	0.865	0.837	0.927	0.783	0.875
M											-	-	-	-	-	-
N												1.000	-	0.839	-	-
O													1.000	0.842	0.842	0.829
P														1.000	0.783	0.859
Q															1.000	0.821
R																1.000

Table 4-13 Relation of rainfall in case that any of gauging stations observe more than 100 mm/day (mm)

date	A	C	E	F	G-H	J-K	N
Mar., 27th, 1960	77.2	4.8	140.2	0.0	0.0	0.0	15.0
Nov., 6th, 1960	0.0	10.2	31.2	27.7	6.1	38.1	116.1
Jul., 25th, 1961	0.0	45.0	17.3	104.6	0.0	93.2	29.2
Jul., 26th, 1961	20.3	0.0	0.0	0.0	119.9	0.0	0.0
Nov., 30th, 1961	0.8	1.3	15.2	0.0	0.0	2.5	109.7
Oct., 9th, 1962	12.1	45.2	42.7	25.4	2.5	138.7	81.3
Oct., 20th, 1962	99.3	154.7	136.4	71.4	10.7	117.3	114.3
Oct., 23th, 1962	104.1	130.6	141.7	113.8	17.8	110.5	97.1
Oct., 24th, 1962	40.1	41.4	35.6	25.9	128.8	29.0	40.6
Sep., 20th, 1963	37.1	93.2	102.4	49.9	0.0	20.3	14.3
Sep., 23th, 1963	1.0	5.8	0.3	0.8	105.4	6.9	1.3
Nov., 12th, 1963	25.9	34.8	38.6	0.0	46.2	16.5	132.1
Nov., 23th, 1963	39.4	122.7	61.7	44.2	10.7	7.9	17.3
Sep., 6th, 1964	51.3	90.2	54.1	160.8	7.6	84.1	114.3
Sep., 7th, 1964	17.5	28.7	15.0	45.2	5.6	123.2	71.1
Nov., 9th, 1964	162.6	167.6	155.4	82.6	157.5	69.6	88.9
Aug., 8th, 1965	8.9	62.7	88.6	73.2	104.9	36.3	10.2
Oct., 31th, 1965	129.8	78.0	0.0	47.5	38.1	39.0	50.8
Nov., 21th, 1965	57.2	80.5	74.2	57.9	134.9	31.8	61.0
Dec., 22th, 1965	14.5	17.5	14.7	32.0	11.9	130.8	0.0
Mar., 25th, 1966	119.1	7.6	10.7	0.0	5.1	0.5	0.0
Oct., 3ed, 1966	29.2	112.0	81.0	76.5	35.1	56.6	40.6
Oct., 12th, 1966	7.1	6.4	6.4	21.6	7.1	111.8	81.3
Oct., 17th, 1966	12.4	43.2	39.4	119.6	83.8	62.0	45.7
Dec., 10th, 1966	10.9	0.5	39.9	56.4	94.5	126.7	53.3

Table 4-14 Relation of rainfall in case that any of gauging stations observe more than 100 mm/day (mm)

date	A	B	C	E	G-H	I	L	O	P	Q	R
Sep., 5th, 1980	93.5	–	–	–	–	120.5	–	–	–	–	–
Apr., 29th, 1981	0.0	0.0	0.0	16.5	129.0	–	0.0	45.0	0.0	22.0	72.5
Aug., 18th, 1981	101.5	90.0	66.0	95.0	81.0	–	0.0	46.5	0.0	105.0	62.5
Sep., 6th, 1981	155.0	521.0	75.0	153.0	158.0	–	0.0	95.0	0.0	57.5	104.0
Sep., 1st, 1982	112.0	53.0	80.0	52.0	110.0	–	2.5	49.5	0.0	54.0	103.5
Sep., 2ed, 1982	132.0	32.5	27.0	41.0	42.0	–	35.0	26.5	37.5	68.0	41.5
Nov., 23th, 1982	27.5	26.0	10.0	52.0	83.0	–	20.0	0.0	50.0	17.0	108.0
Sep., 9th, 1983	94.5	66.5	90.0	80.0	104.0	–	2.5	49.0	2.5	50.0	129.0
Oct., 16th, 1983	35.0	22.5	0.0	20.5	0.0	–	0.0	29.5	0.0	135.0	15.5
Apr., 8th, 1984	22.5	15.0	36.0	4.0	108.0	–	0.0	65.0	0.0	91.0	100.0
Apr., 9th, 1984	34.0	12.0	144.0	42.0	45.0	–	35.0	53.5	35.0	40.0	33.0
Apr., 18th, 1984	111.5	108.0	104.0	93.5	106.0	–	0.0	39.0	0.0	98.0	135.0
Apr., 19th, 1984	27.5	9.5	13.0	13.0	21.0	–	50.0	196.5	70.0	32.0	20.0
Apr., 27th, 1984	116.5	29.5	115.5	33.5	54.0	–	1.5	42.5	2.5	35.5	95.0
Jul., 23th, 1985	25.0	12.5	43.0	45.5	110.0	–	46.5	35.0	50.0	20.0	87.0
Oct., 11th, 1985	132.5	78.0	48.0	108.0	146.0	–	20.0	155.0	19.0	140.0	139.0
Oct., 12th, 1985	87.0	49.0	0.0	44.0	52.0	–	122.5	75.0	100.0	27.0	58.0
Aug., 1st, 1986	95.0	47.5	44.0	53.0	89.0	–	7.5	26.0	5.0	32.0	103.5
Sep., 19th, 1986	27.0	20.0	0.0	14.0	143.0	–	6.5	52.5	11.5	10.0	136.0
Oct., 5th, 1986	250.0	112.0	55.0	19.0	81.0	–	2.5	65.0	12.5	78.0	111.0
Nov., 8th, 1987	115.0	60.0	56.0	68.0	155.0	–	7.5	78.0	12.5	65.0	131.0

Table 4-15 Characteristics of runoff

		F.M.D. (m^3/sec)					
		4.0	8.0	12.0	16.0	20.0	30.0
time-lag (days)	1	0.0124	0.0257	0.0316	0.0396	0.0464	0.0564
	2	0.0072	0.0152	0.0194	0.0218	0.0211	0.0197
	3	0.0065	0.0113	0.0140	0.0139	0.0142	0.0139
	4	0.0052	0.0105	0.0134	0.0123	0.0120	0.0122
	5	0.0041	0.0101	0.0100	0.0114	0.0114	0.0111
	6	0.0057	0.0091	0.0098	0.0110	0.0114	0.0118
	7	0.0048	0.0088	0.0102	0.0102	0.0107	0.0093
	8	0.0042	0.0084	0.0098	0.0101	0.0094	0.0094
	9	0.0051	0.0072	0.0081	0.0088	0.0098	0.0098
	10	0.0044	–	0.0078	0.0075	0.0078	0.0084
ratio of runoff (%)		5.95	10.63	13.40	14.65	15.42	16.20
base flow	Q_b	1.308	1.266	1.115	1.041	1.007	0.968
	q_b	0.0094	0.0091	0.0080	0.0075	0.0072	0.0070
	f_b	11.10	10.74	9.46	8.83	8.55	8.21

$$Q_b = Q_o \quad (m^3/sec)$$

$$q_b = Q_b/A = Q_b/139.13 \quad (m^3/sec/Km^2)$$

$$f_b = (q_b/R)\cdot\alpha = \frac{Q_b \times 86400 \times 365.25}{2672.8\times10^{-3}\times10^{6}\times139.13}\times 100 \quad (\%)$$

Table 4-16 Maximum daily rainfall in each year

	A	B	C	D	E	F	G·H	I	J-K	L	M	N	O	P	Q	R
1945								108.7								
1946					133.4		114.3	165.1								
1947					188.0	224.5	114.3	114.3								
1948					74.2	113.8	78.7	83.5								
1949					83.8	130.0	111.0	118.1								
1950					75.2	122.7	100.1	123.2								
1951					96.5	107.1	126.2	91.4								
1952					147.3	141.2	177.0	170.9		131.9						
1953					102.9	140.5	126.2	86.9		81.3						
1954					103.9	160.5	118.6	97.5		67.3						
1955				119.9	86.0	216.7	83.1	83.6		64.8						
1956				112.4	91.4	89.2	99.6	92.4		71.1						
1957				68.3	115.4	81.3	120.1	95.3		114.3	80.0					
1958		80.8	183.4	138.7	125.7	72.7	117.3	76.2	65.5	74.0	83.8	86.4				
1959		115.6	111.3	141.5	118.6	128.3	74.9	76.2	94.7	106.7	95.3	82.3				
1960	77.2	82.6	67.8	(87.1)	140.2	89.2	78.5	129.0	88.8	58.4	91.4	116.1				
1961	67.1	82.3	81.0	72.5	72.5	104.6	119.9	102.1	93.2	92.7	69.9	109.7				
1962	104.1	127.0	154.7	167.4	141.7	113.8	128.8	106.2	138.7	147.3	105.4	114.3				
1963	70.4	87.9	122.7	69.1	103.4	97.3	105.4	109.2	78.0	69.9	78.7	132.1				
1964	112.6	155.7	162.6	181.1	135.4	160.8	157.5	134.9	123.2	81.3	101.6	114.3				
1965	(129.8)	99.1	99.1	123.6	88.6	88.4	134.9	81.0	130.8	106.7	99.1	91.4				
1966	(119.1)	106.2	112.0	109.2	72.6	117.6	94.5	106.7	126.7	73.7	72.6	81.3				
1967			110.7	113.0			109.2	72.4		123.2						
1968			85.4	106.2			107.7	83.3		102.9						
1969			219.2	130.3			88.9	82.3		75.7						
1970			108.0	132.9			124.5	109.7		88.9						
1971	101.6		108.0	152.9			124.5	109.7	109.7	99.1				101.6		
1972	241.3		259.1	190.5			147.3	110.0	127.0	119.4				131.9		
1973	132.1		127.4	133.4			135.1	108.0	105.9	132.1				96.5		
1974	96.5		93.0	128.8			106.2	93.1	101.6	94.0				96.5		
1975	(181.5)		78.5	70.4			122.4	180.8	95.0	130.0				60.5		
1976			200.0	219.0				179.0	134.0	155.5				104.5		
1977			133.0	115.0				133.5	111.0	98.5				103.0		
1978	79.3		113.5				126.5	145.0	77.0	102.0				80.0		
1979	82.0		136.0	86.5			106.0	70.0	102.0	104.9				80.0		
1980	93.0		130.0	111.6			117.0	130.5	91.0	86.5				71.0		
1981	155.0	90.0	80.0		153.0		158.0			71.0			76.5	72.0	105.0	104.0
1982	132.0	53.0	80.0		52.0		110.0			70.0			86.5	75.0	78.5	103.5
1983	94.5	66.5	69.0		80.0		104.0			70.0			65.5	67.5	135.0	139.0
1984	116.5	108.0	145.5		95.5		108.0			78.5			196.5	85.0	97.0	135.0
1985	132.5	78.0	50.0		108.0		146.0			122.5			155.0	100.0	140.0	139.0
1986	250.0	112.0	55.0		64.0		143.0			87.5			98.0	70.0	78.0	136.0
1987	115.0	60.0	57.0		68.0		135.0			70.0			78.0	77.5	80.0	131.0

Table 4-17 Return period of daily rainfall

(mm)

	A	B	C	D	E	F	G-H	I	J-K	L	M	N	O	P	Q	R
the number of year	19	16	30	24	28	20	40	36	19	36	10	9	7	17	7	7
1/5	151.5	115.1	153.1	160.5	130.6	152.9	136.9	130.8	122.6	113.5	98.4	118.0	140.9	99.4	120.0	138.6
1/10	185.5	128.6	183.6	180.5	149.5	185.2	148.0	149.9	135.4	128.7	104.3	128.8	172.7	108.1	138.8	146.5
1/20	221.9	140.6	213.4	198.2	167.0	212.6	157.4	168.6	146.9	143.2	(109.4)	(138.9)	(206.3)	115.8	(158.5)	(153.5)
1/40	(261.0)	(151.7)	243.2	214.5	183.8	(240.4)	165.9	187.2	(157.7)	157.5	(114.0)	(148.6)	(242.3)	(122.9)	(179.4)	(159.7)

Table 4-18 Probable daily rainfall in Sg.Perai basin

return period	daily rainfall (mm)
1/5	127.3
1/10	142.5
1/20	157.0
1/40	171.0

mean value of C, E, F, G-H, I, J-K, L, M, N, P and R gauging stations (from Table4-17)

Table 4-19 Two types of rainfall

	type I	type II	
characteristic	short-duration type	long-duration type	
K	$1/4.22$	$1/2.25$	
n	0.763	0.555	
duration time (hr)	10	first peak	second peak
		6	11
ratio (%)	100	36	64

Table 4-20 Time-intensity pattern of daily rainfall

(%)

	type I	type II
1	0.025	0.028
2	0.030	0.036
3	0.037	0.058
4	0.053	0.163
5	0.103	0.044
6	0.579	0.031
7	0.069	0.0
8	0.044	0.0
9	0.033	0.0
10	0.027	0.0
11	0.0	0.0
12	0.0	0.0
13	0.0	0.0
14	0.0	0.028
15	0.0	0.033
16	0.0	0.038
17	0.0	0.049
18	0.0	0.079
19	0.0	0.220
20	0.0	0.060
21	0.0	0.043
22	0.0	0.034
23	0.0	0.030
24	0.0	0.026

Table 4-21 Relation between accumulate rainfall and accumulate rainfall loss

date	ΣR * (mm)	ΣRe * (mm)	ΣRℓ * (mm)	base flow (m³/sec)
1981. 4. 23 ~	74.1	14.7	59.4	1.55
1981. 5. 27 ~	70.4	18.6	51.8	2.07
1981. 8. 14 ~	98.9	1.2	97.7	0.77
1981. 9. 2 ~	172.4	24.2	148.2	0.77
1982. 4. 29 ~	87.5	18.9	68.6	1.26
1982. 8. 27 ~	122.1	7.2	114.9	0.71
1982. 11. 17 ~	131.1	27.5	103.6	4.23
1983. 1. 1 ~	74.6	24.3	50.3	2.25
1983. 9. 2 ~	104.0	20.9	83.1	0.76
1983. 10. 15 ~	103.9	15.3	88.6	1.15
1984. 4. 2 ~	124.0	33.2	90.8	1.59
1984. 4. 14 ~	165.1	21.3	143.8	3.19
1984. 4. 26 ~	73.0	19.9	53.1	3.19
1984. 11. 13 ~	82.0	34.0	48.0	1.58
1985. 2. 8 ~	133.0	24.4	108.6	1.65
1985. 3. 5 ~	132.0	20.8	111.2	2.17
1985. 7. 20 ~	77.6	5.1	72.5	1.07
1985. 10. 6 ~	211.6	27.6	184.0	1.90
1985. 10. 27 ~	185.8	55.3	130.5	2.94
1985. 11. 26 ~	48.0	8.6	39.4	3.36
1986. 7. 30 ~	120.1	6.5	113.6	0.68
1986. 9. 16 ~	163.5	17.6	145.9	1.34
1986. 10. 2 ~	78.6	5.8	72.8	3.49

remark

① base flow was determinated from minimum water level within 20 days.

② discharge above base flow was considered to be effective rainfall.

③ $\Sigma R_\ell = \Sigma R - \Sigma Re$

④ * range used by calculation was showed in Annex A-7

Table 4-22 Frequency of water level at ARA KUDA

1973 ~ 1987 (N=10 Y.)

Water level	Jan.	Feb.	Mar.	Apr.	May.	Jun.	Jul.	Aug.	Sep.	Oct.	Nov.	Dec.	Total	A-Total	%
											16		16	16	0.4
18.5 ~ 18.9	22	30	9								3		54	80	2.2
19.0 ~ 19.4	56	96	92	40	8	30	53	82	1	1	4	24	487	567	15.5
19.5 ~ 19.9	93	74	93	73	40	100	166	114	48	26	15	46	888	1455	39.8
20.0 ~ 20.4	74	24	47	50	58	94	45	44	72	61	30	40	639	2094	57.4
20.5 ~ 20.9	31	19	27	39	59	36	20	33	43	31	29	40	407	2501	68.5
21.0 ~ 21.4	18	16	14	33	55	18	11	12	36	46	15	37	318	2819	77.2
21.5 ~ 21.9	7	7	10	20	28	7	5	12	25	30	13	36	205	3024	82.8
22.0 ~ 22.4	4	8	9	14	22	7	4	6	15	30	32	28	179	3203	87.7
22.5 ~ 22.9	1	2	4	8	10	1	1	1	16	22	35	20	124	3327	91.1
23.0 ~ 23.4	2	2	1	7	9	2	1	4	14	22	21	13	98	3425	93.8
23.5 ~ 23.9	1		1	1	1	1	2		5	12	22	6	52	3477	95.2
24.0 ~ 24.4		1	1	1	4	1		1	4	6	18	8	45	3522	96.4
24.5 ~ 24.9	1	1		2	3		1		7	5	6	4	30	3552	97.3
25.0 ~ 25.4			1		5	1			3	5	9		24	3576	97.9
25.5 ~ 25.9		1		3	4	1	1		2	3	7	1	23	3599	98.5
26.0 ~ 26.4				1					4	2	5		12	3611	98.9
26.5 ~ 26.9				1					1	3	2	1	8	3619	99.1
27.0 ~ 27.4			1	1				1	1	2	5	3	12	3631	99.4
27.5 ~ 27.9					1				2	1	2	2	8	3639	99.6
28.0 ~ 28.4		1		1							4	1	7	3646	99.8
28.5 ~ 28.9									1		1		2	3648	99.9
29.0 ~ 29.4					1								1	3649	99.9
29.5 ~ 29.9															
30.0 ~ 30.4															
30.5 ~ 30.9													1	3650	99.9
31.0 ~ 31.4															
31.5 ~ 31.9					2								2	3652	100
32.0 ~ 32.4															
32.5 ~ 32.9															
(FEET)															
Total	310	282	310	300	310	300	310	310	300	310	300	310	3652		

Rem : * at 80% Water level

Rem : Above 28.0ft happened flood

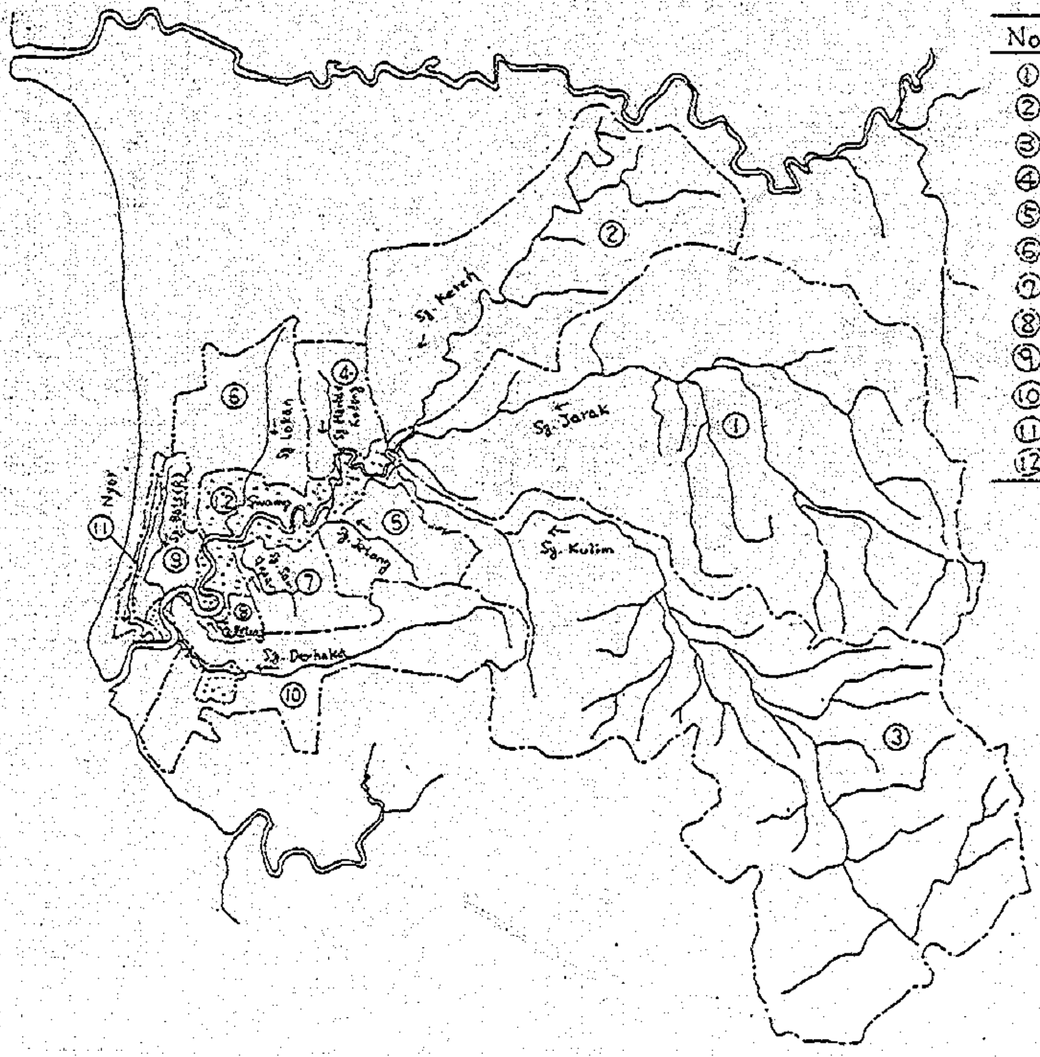
Table 4-23 Annual maximum water level and discharge at ARA KUDA

year	water level (m)	discharge * (m^3/sec)	ranking
1978	7.800	15.26	10
1979	8.535	25.48	6
1980	8.955	32.49	1
1981	8.426	23.80	9
1982	8.434	23.92	8
1983	8.666	27.58	5
1984	8.768	29.27	3
1985	8.792	29.67	2
1986	8.440	24.01	7
1987	8.717	28.41	4

* $Q = \{1.5535 \cdot (H - 5.4864) + 0.3118\}^2$

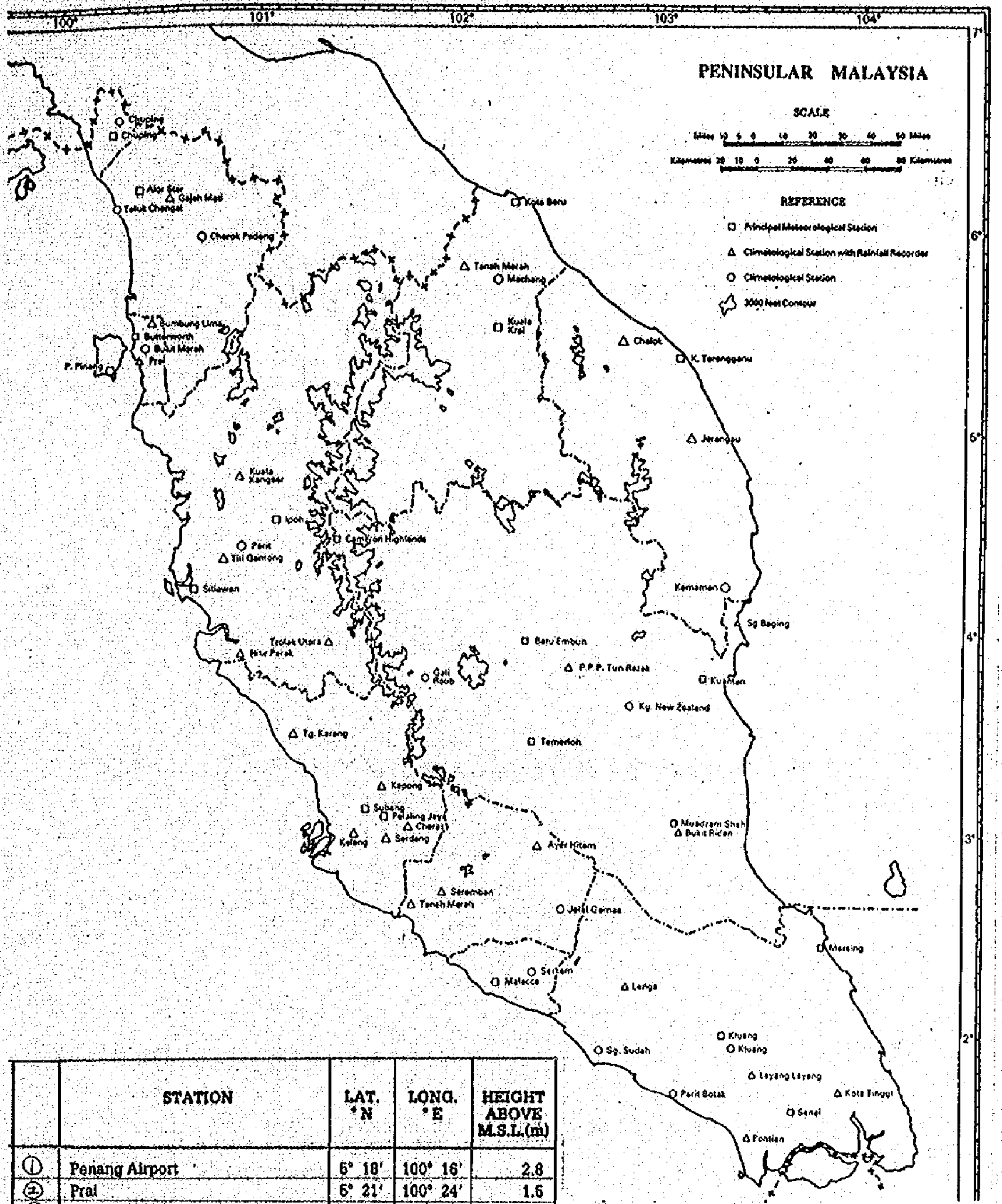
Table 4-24 Return period of peak discharge

	peak discharge (m^3/sec)
1/40	(38.48)
1/20	(36.02)
1/10	33.38
1/5	30.45



No.	Designation	Catchment Area (km²)
1	Sg. Jarak	149.01
2	Sg. Kreh	60.58
3	Sg. Kulim	174.82
4	Sg. Marbau Kudong	13.17
5	Sg. Jetong	12.61
6	Sg. Lokan	18.19
7	Sg. Sama Gagam	9.25
8	Sg. Bats (L)	2.17
9	Sg. Bats (R)	6.58
10	Sg. Derhaka	35.70
11	Sg. Nyor	3.04
12	Swamp land	12.38

Fig.4-1 Diagram of drainage block in Sg.Perai basin



	STATION	LAT. °N	LONG. °E	HEIGHT ABOVE M.S.L.(m)
①	Penang Airport	6° 18′	100° 16′	2.8
②	Prai	6° 21′	100° 24′	1.6
③	Butterworth	6° 28′	100° 23′	2.0
④	MARDI Bumbung Lima	6° 32′	100° 28′	4.0
⑤	Pertanian Bukit Merah	6° 25′	100° 26′	6.0

Fig.4-2 Location of stations

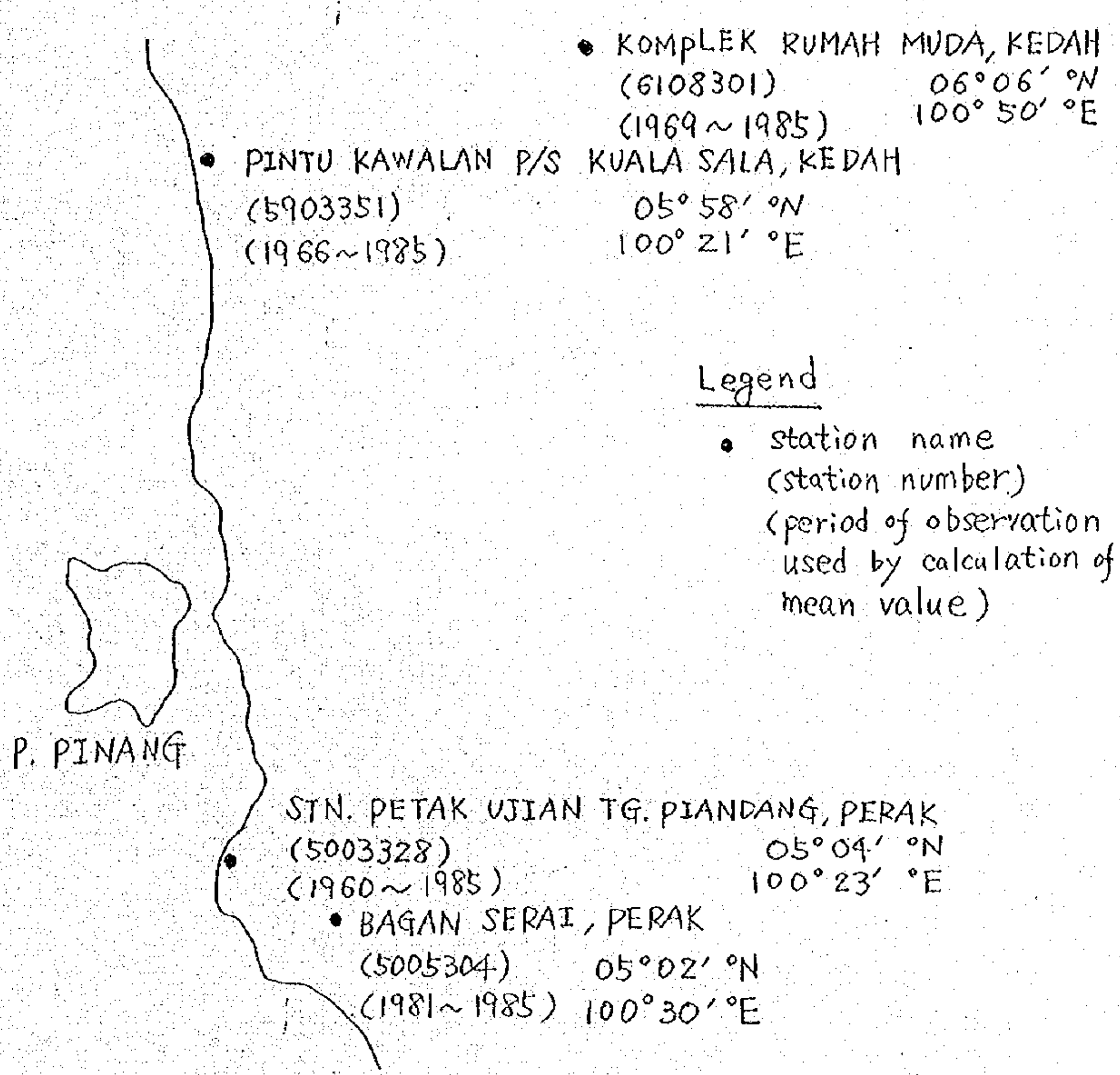


Fig.4-3 Location of evaporation stations

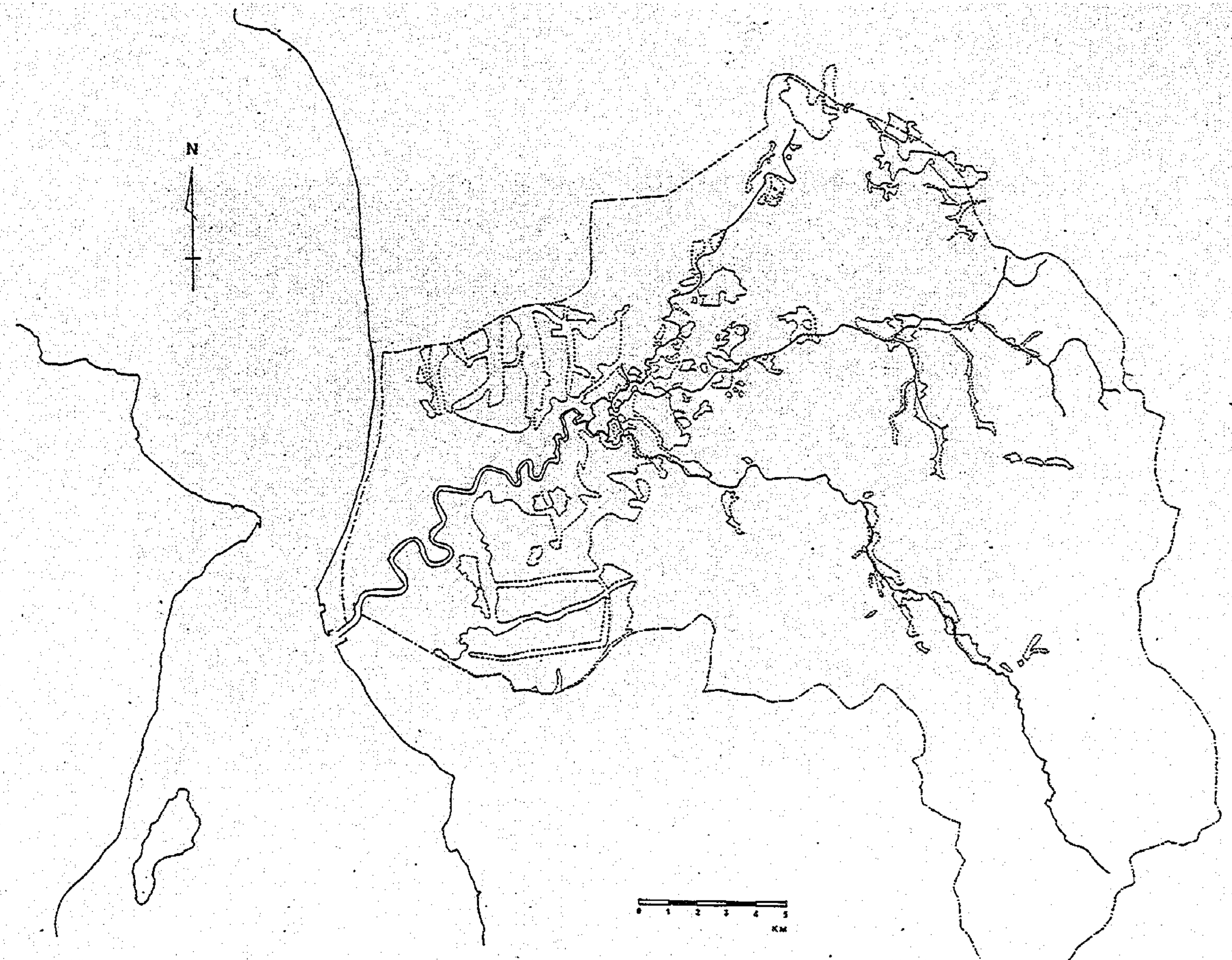


Fig.4-4 Present location of paddy field

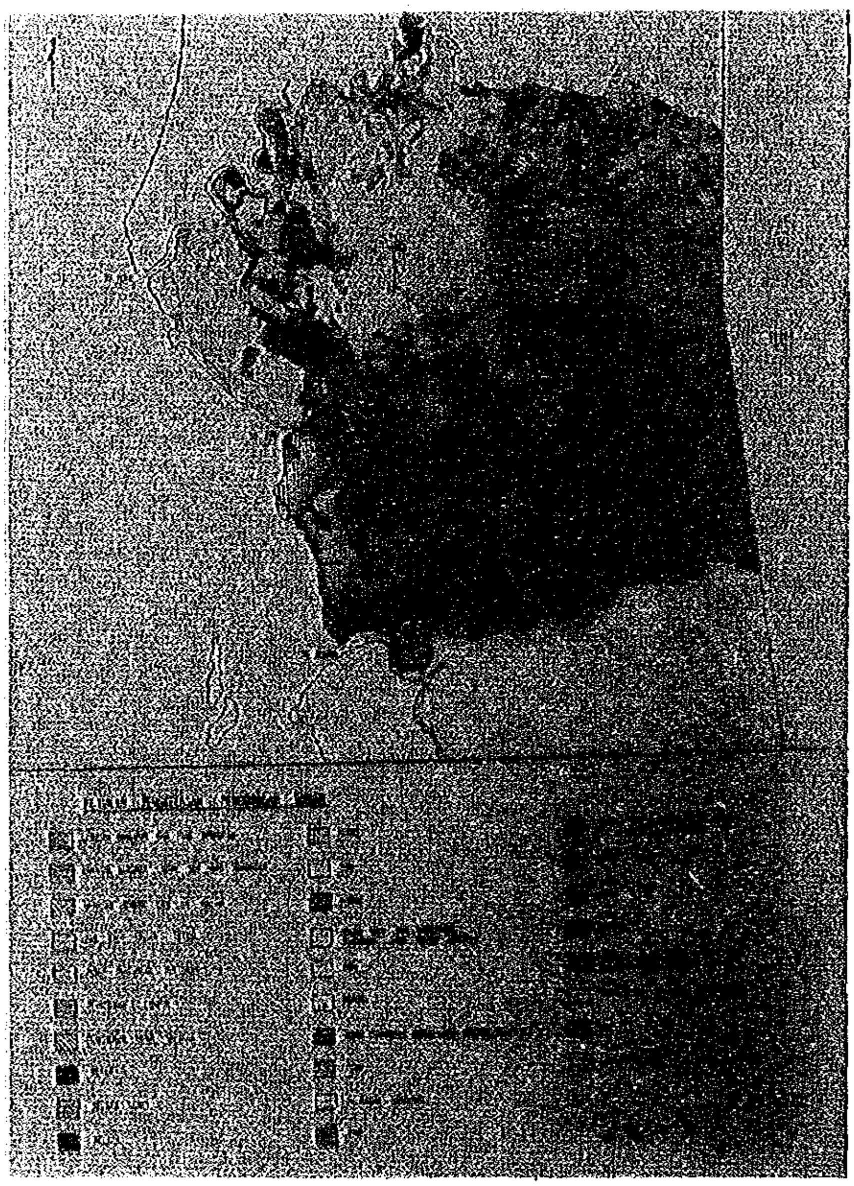


Fig.4-5 Situation of present land use

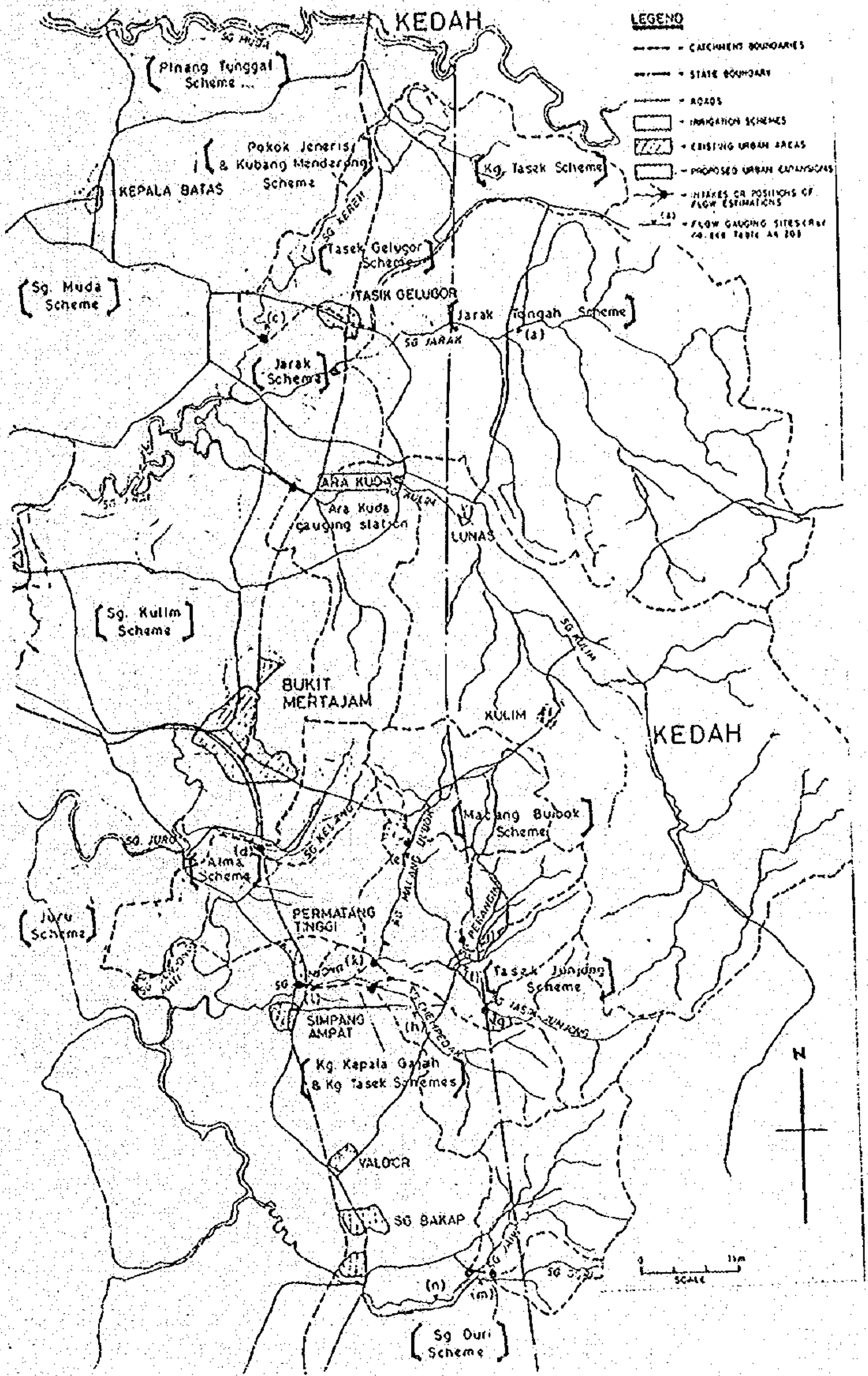
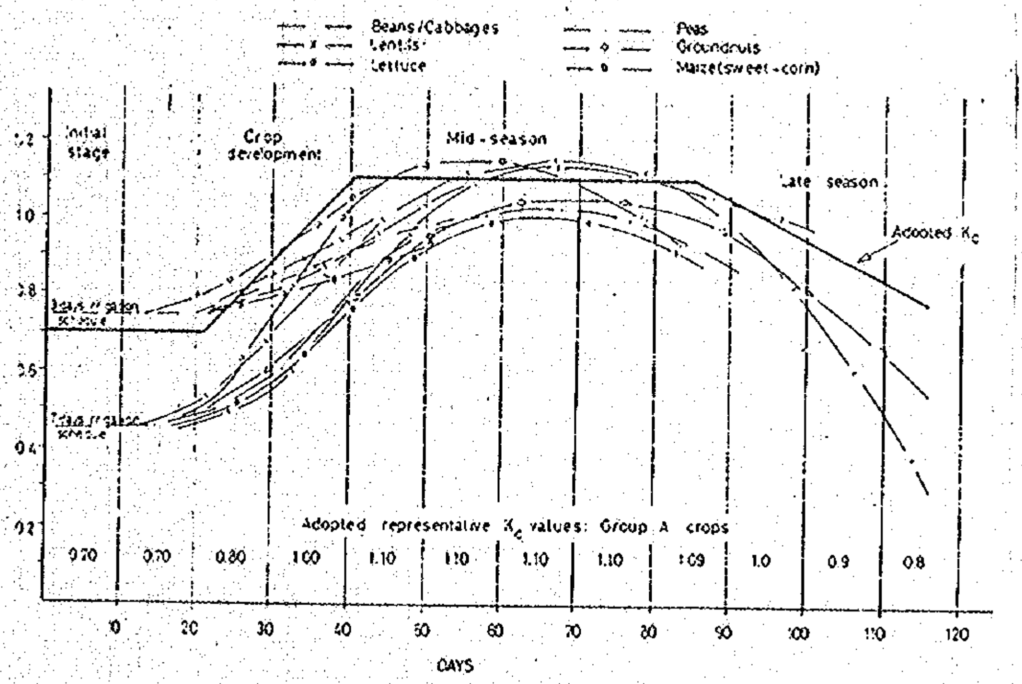
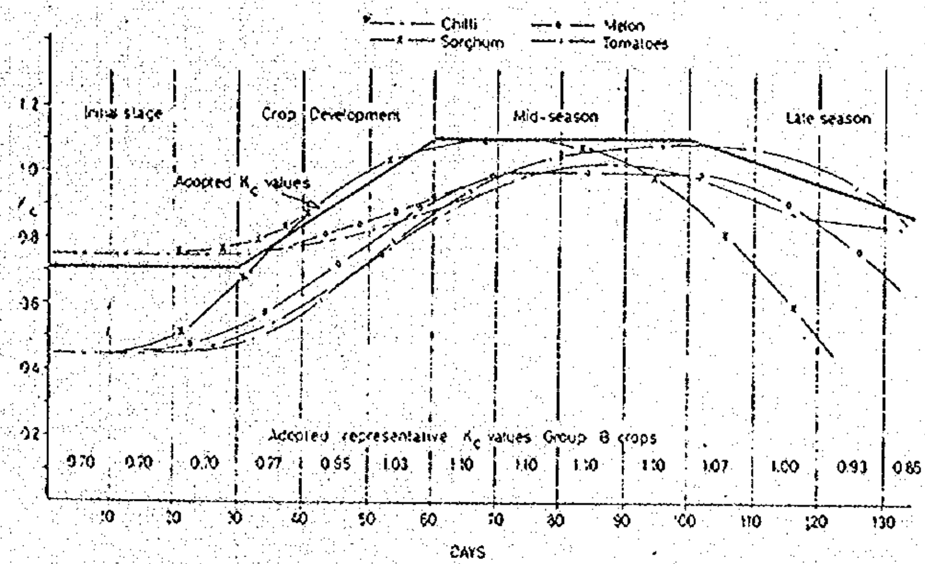


Fig.4-6 Catchment boundaries and irrigation schemes



GROUP A CROPS: LESS THAN 16 WEEKS



GROUP B CROPS GREATER THAN 16 WEEKS

Fig.4-7 Vegetables crop coefficient

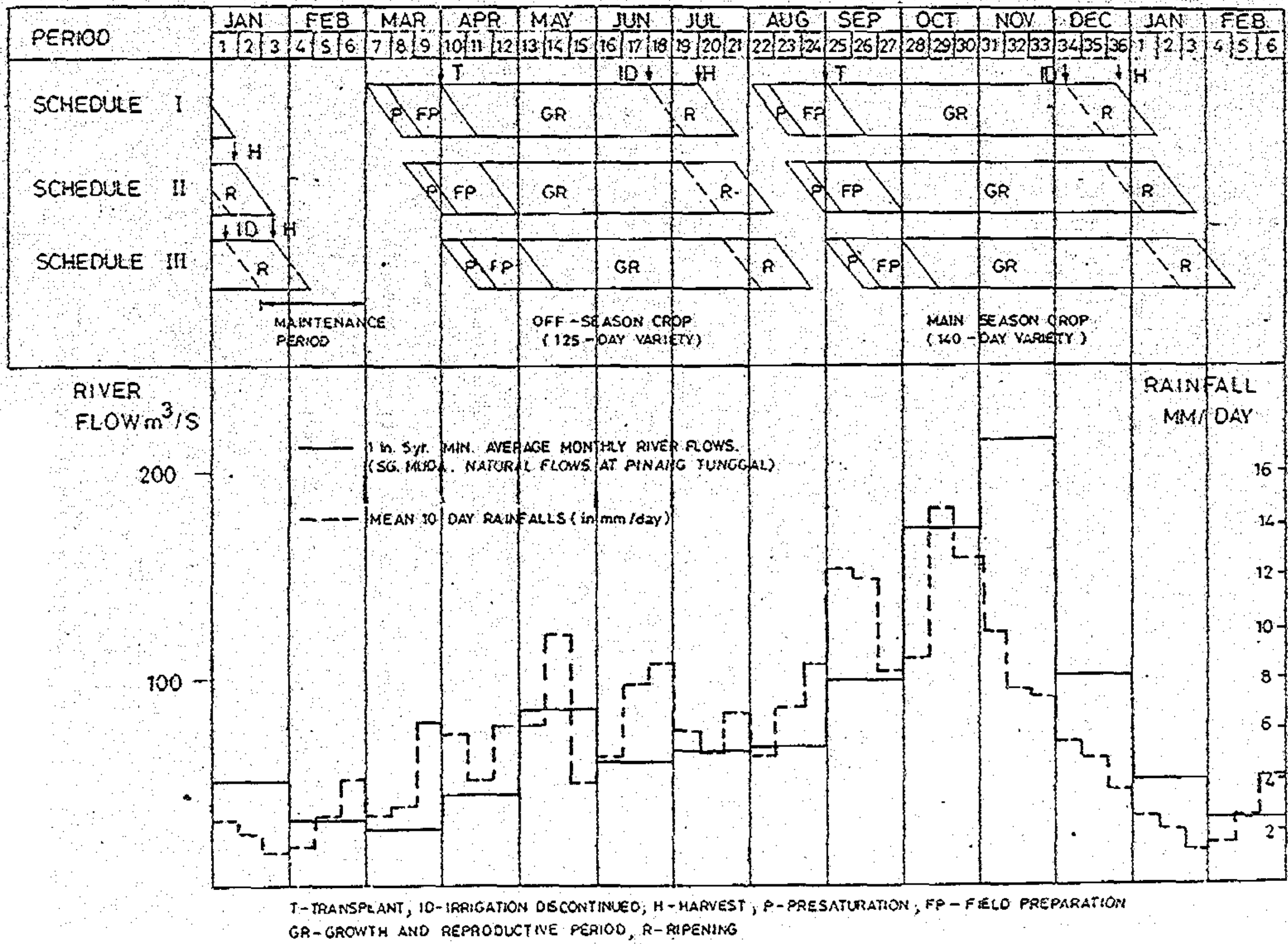


Fig.4-8 Programme for double cropping of rice

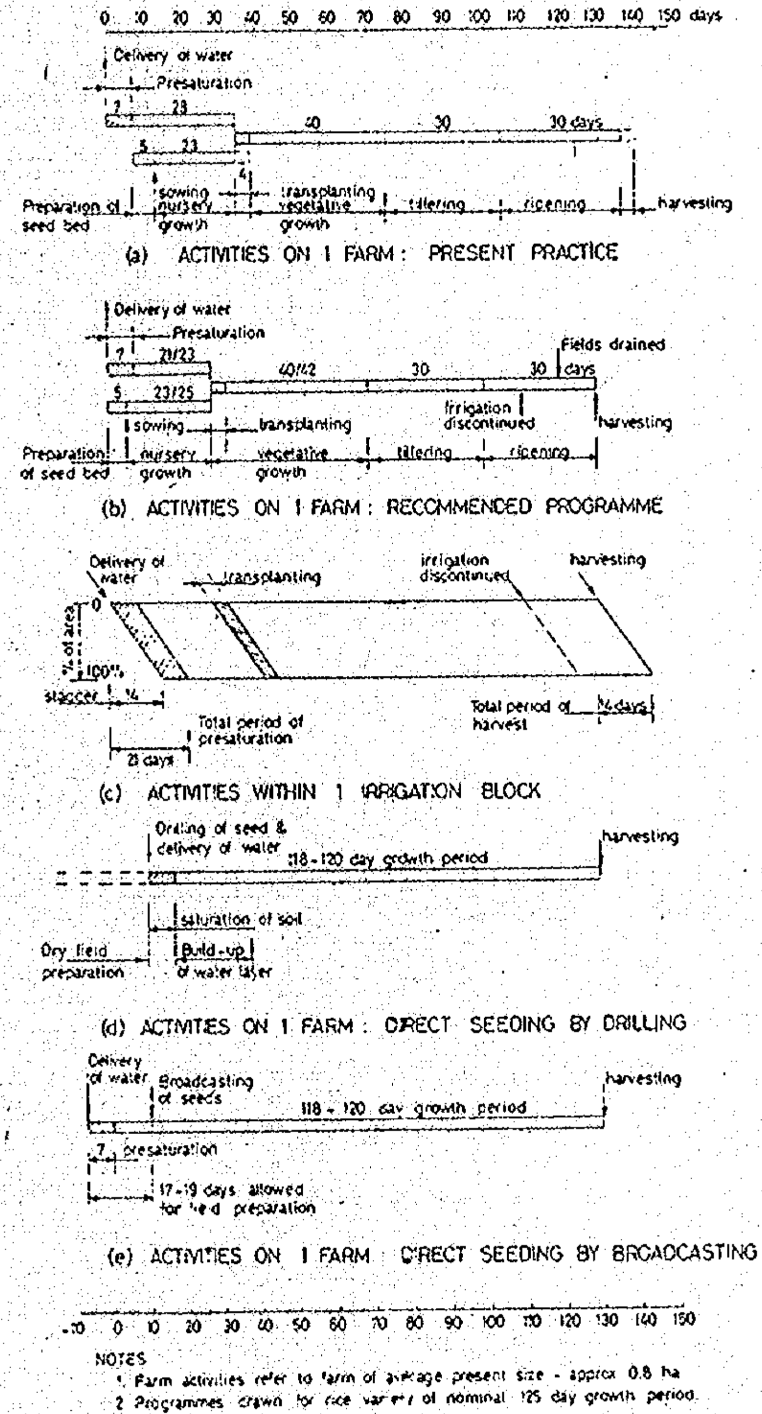


Fig.4-9 Rice cultivation farming activities

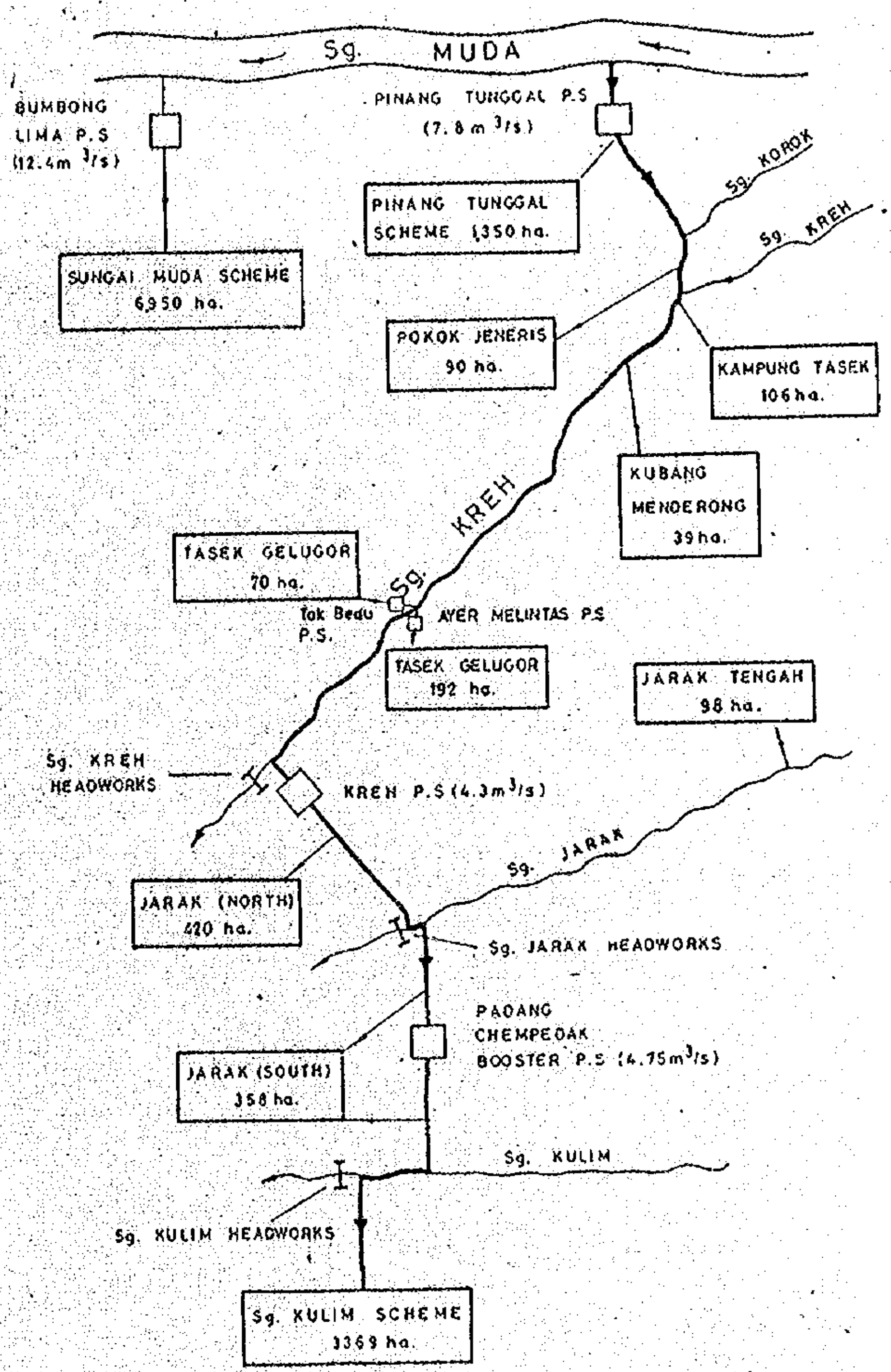


Fig.4-10 Transfer system and schemes abstracting from Sg.Muda

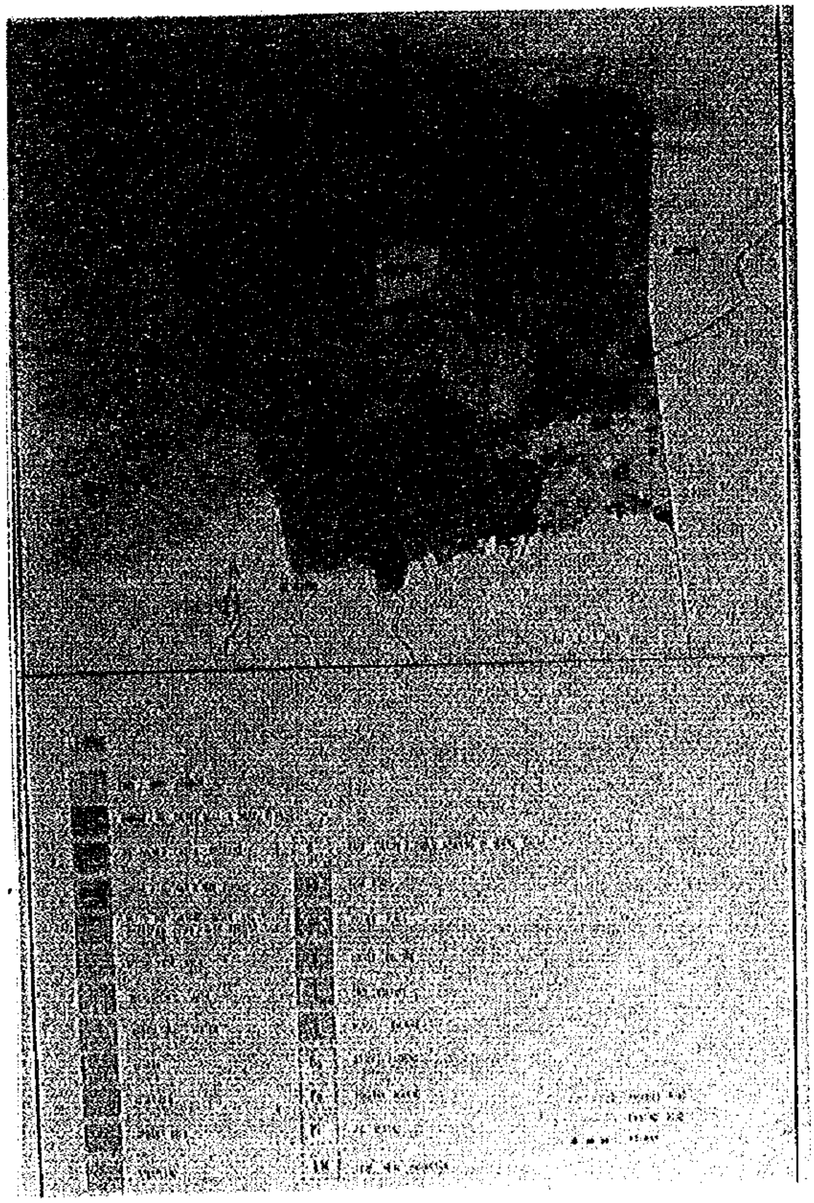


Fig.4-11 Programming of urban development

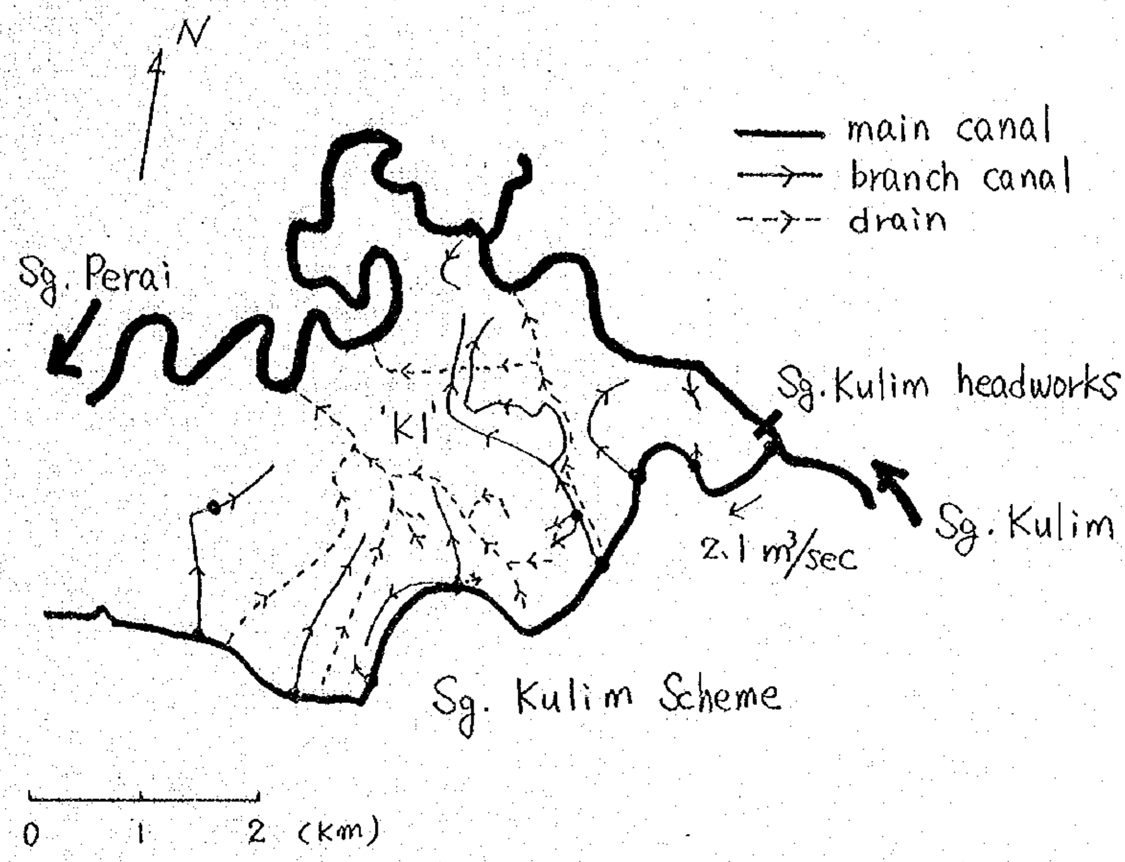


Fig.4-12 Drainage and irrigation system in 'K1' area

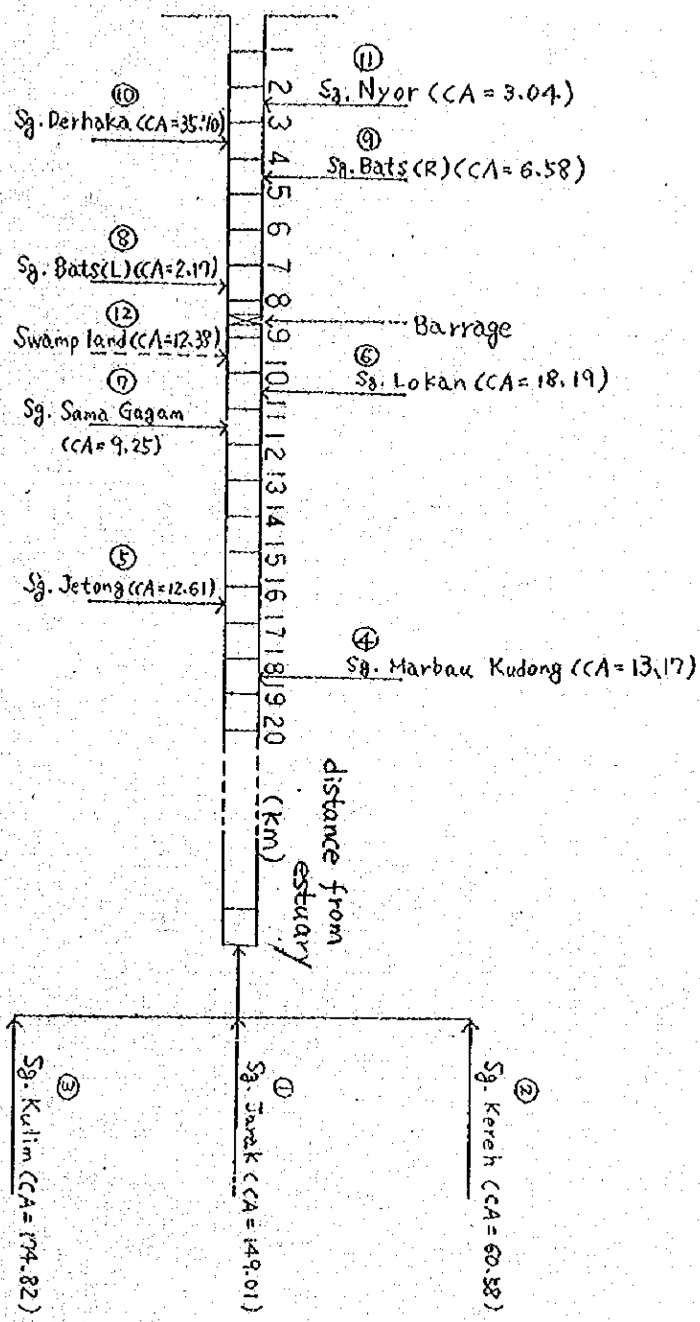


Fig.4-13 Chart of drainage system in Sg.Perai basin

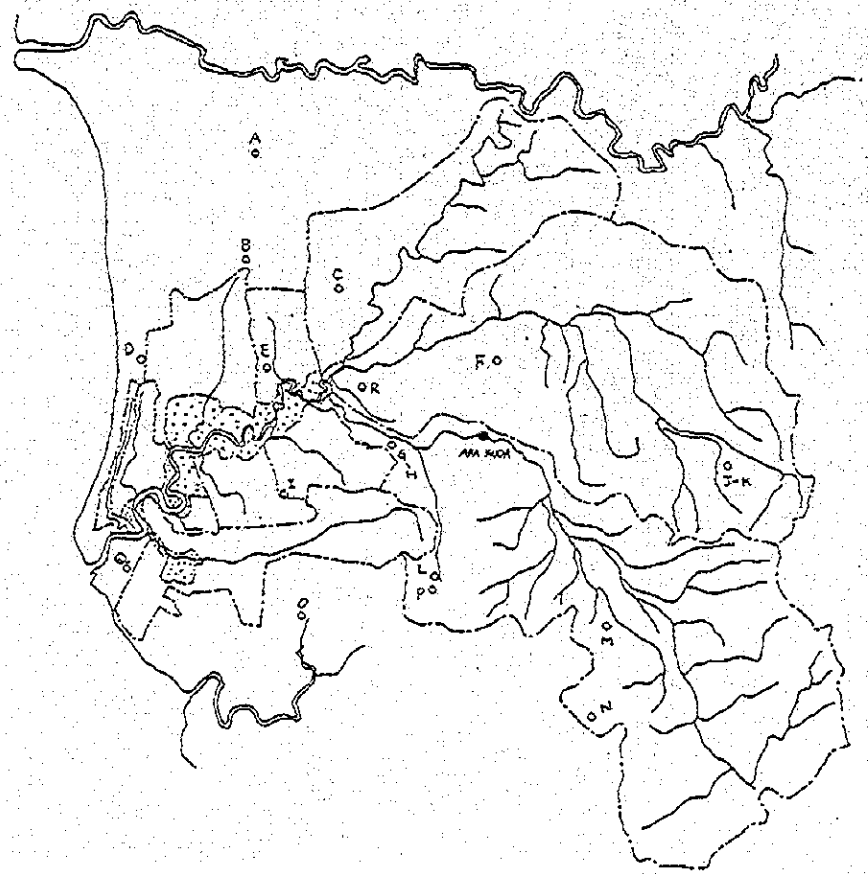


Fig.4-14 Location of gauging stations

mark	gauging station	station number
A	LAHAR IKAN MATI	5504035
B	PARIT LAGAN	—
C	MALIKOFF ESTATE	5404041
D	METELOGICAL STATION, BUTTERWORTH	5403042
E	Sg. DUA	-
F	MAYFIELD ESTATE	-
G-H	Sg. KULIM HEADWORK	5404043
I	BUKIT MERAH PADI STATION	5404044
J-K	BUKIT MERTAJAM ESTATE	5406083
L	BUKIT BERAPIT RESERVOIR	5304045
M	PWD office KULIM	-
N	BUKIT BESAR RESERVOIR	-
O	PERMATANG RAWA	5304046
P	KOLAMAIR CEROK TO'KUN	5304047
Q	KOMPLEK PERAI	5303053
R	RUMAH PAM JARAK	

(rem) mark A~N are used at F/S (1968)

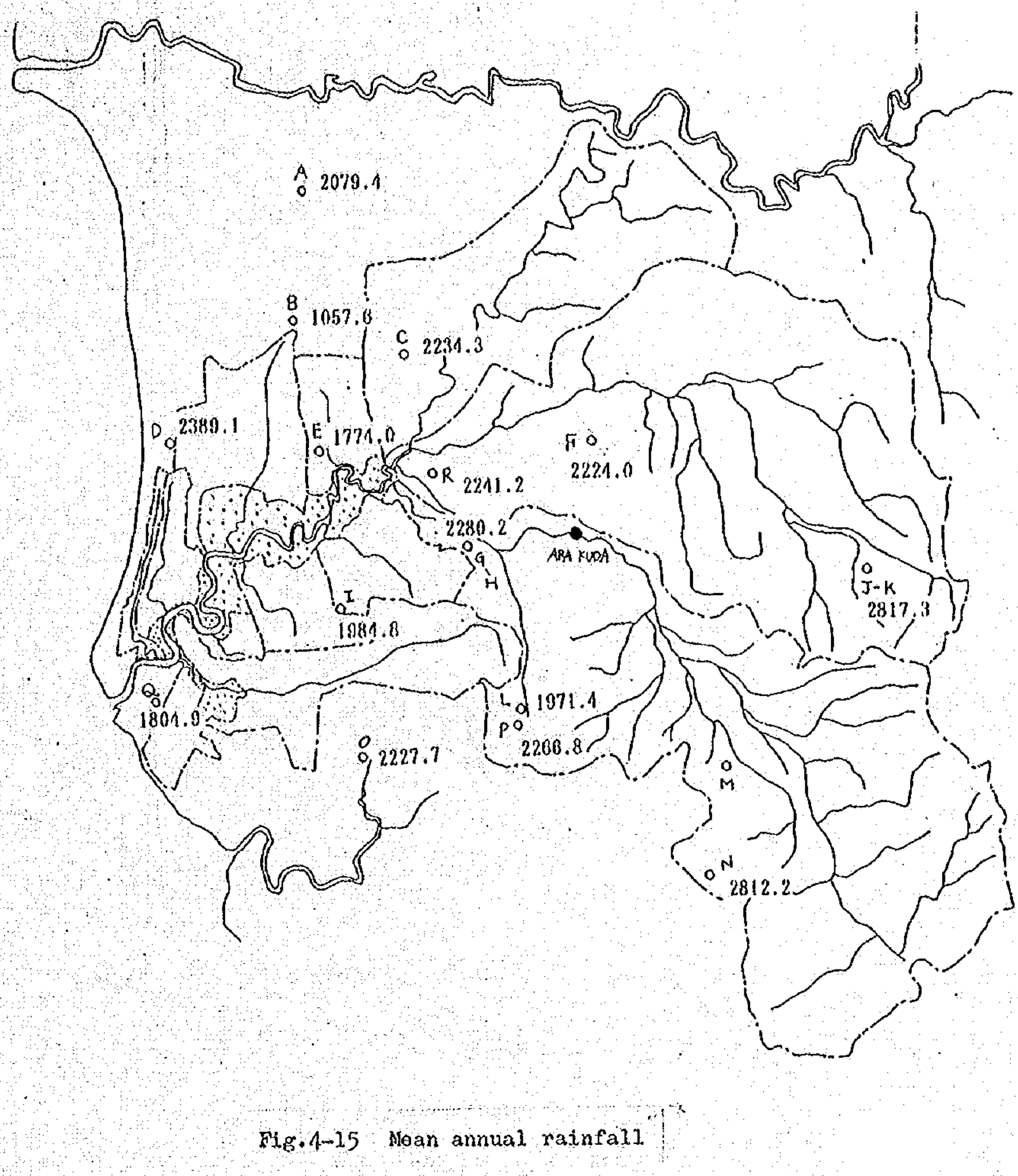


Fig.4-15 Mean annual rainfall

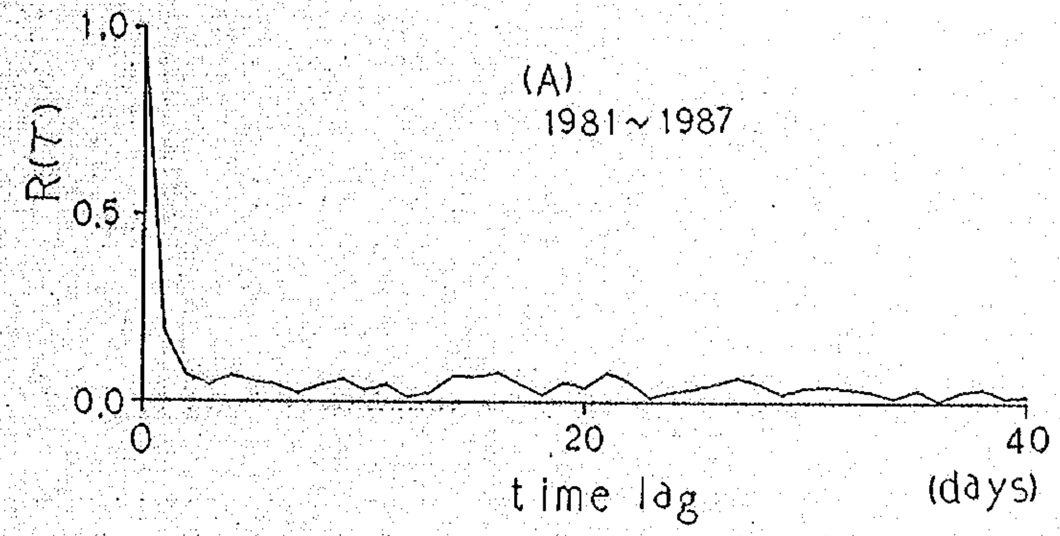


Fig. 4-16 Pattern of the correlogram

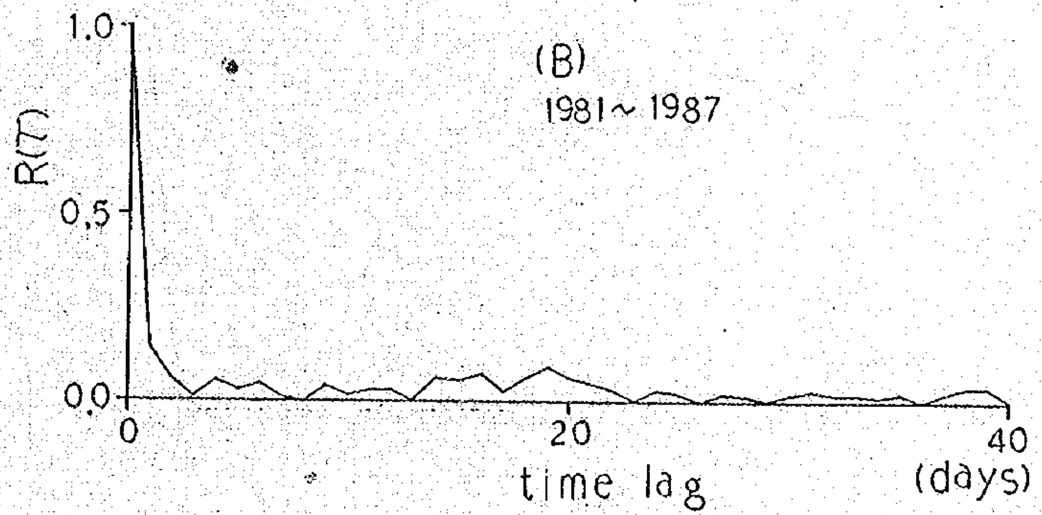


Fig. 4-17 Pattern of the correlogram

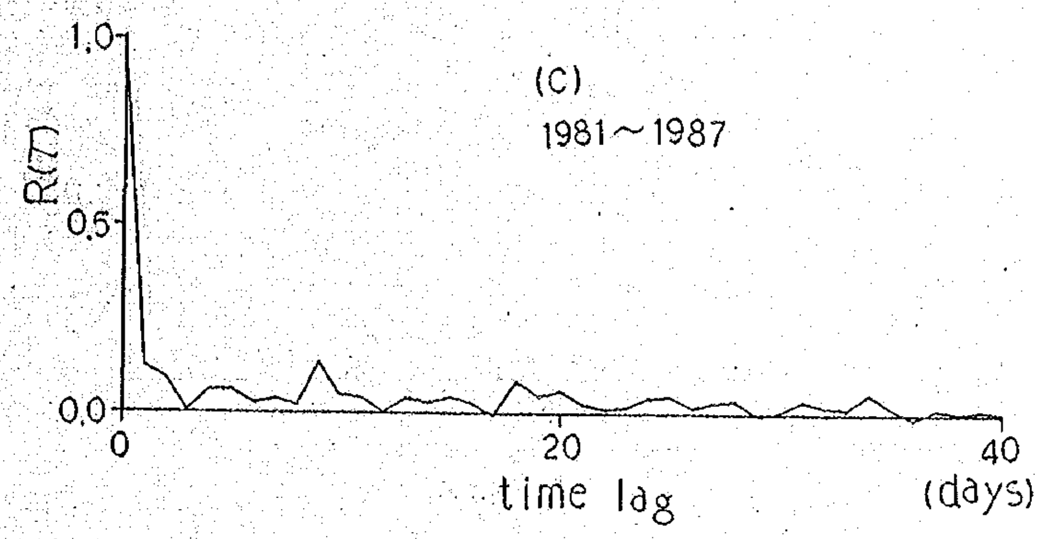


Fig. 4-18 Pattern of the correlogram

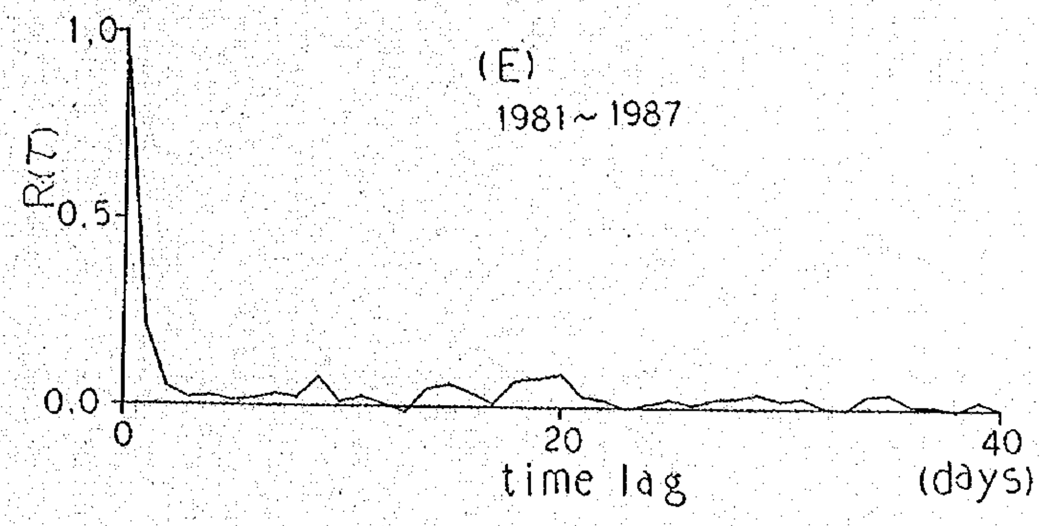


Fig. 4-19 Pattern of the correlogram

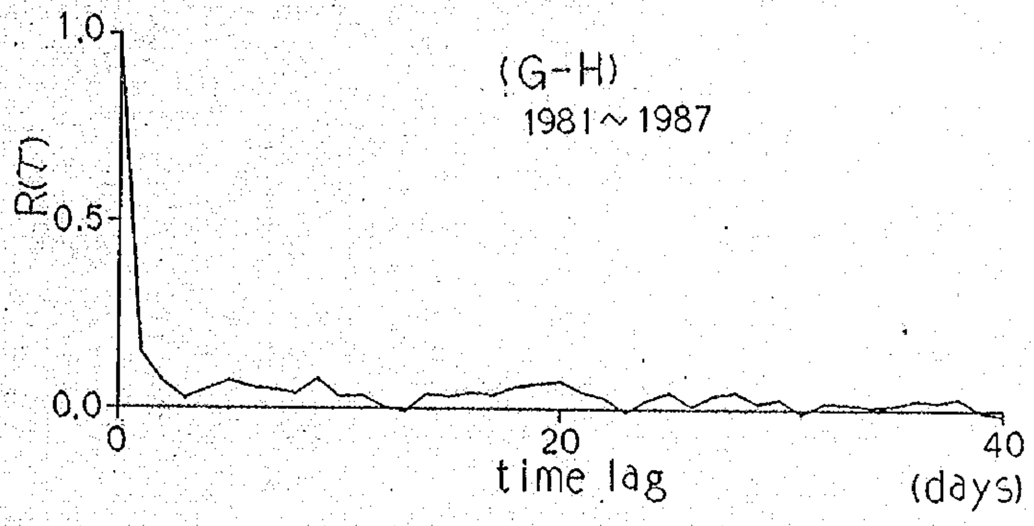


Fig. 4-20 · Pattern of the correlogram

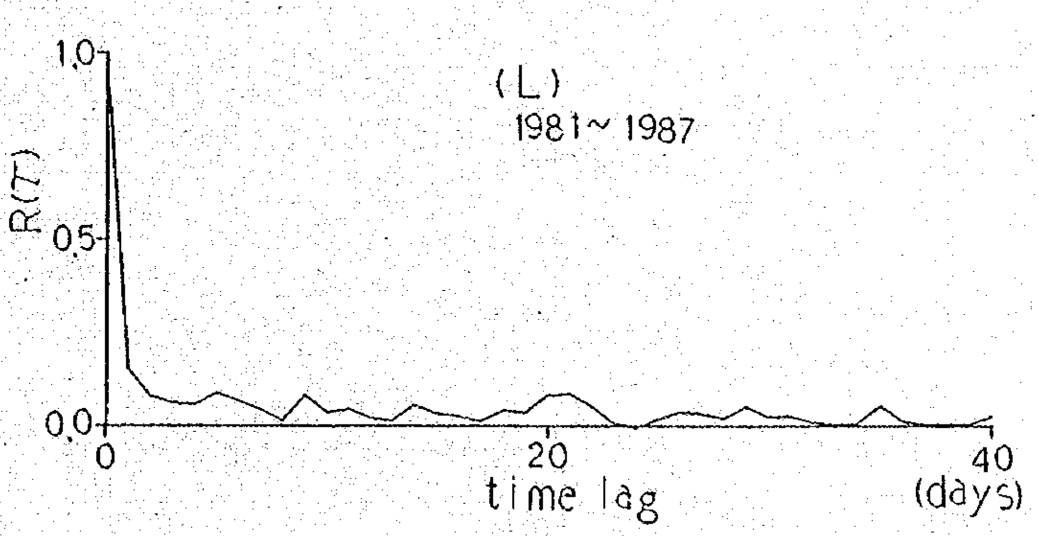


Fig. 4-21 Pattern of the correlogram

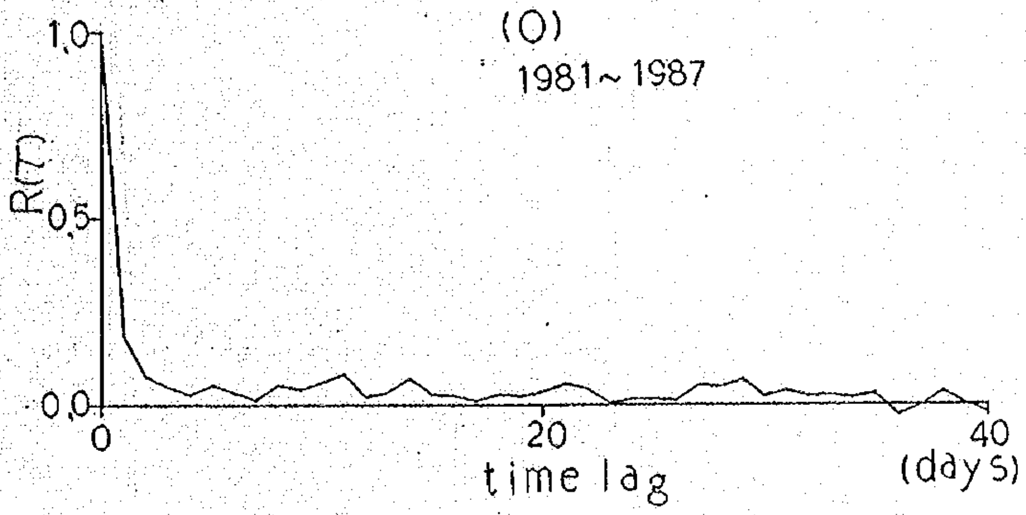


Fig. 4-22 Pattern of the correlogram

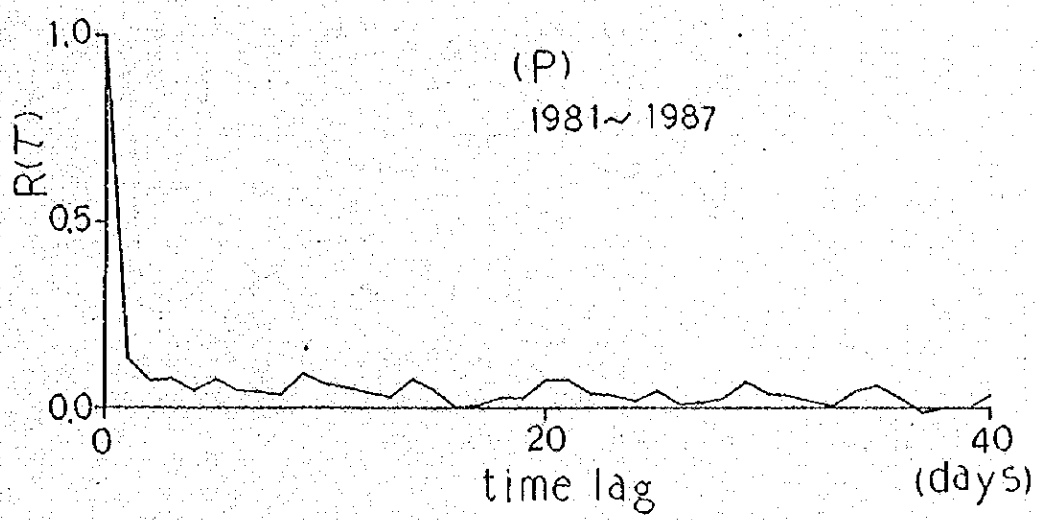


Fig. 4-23 Pattern of the correlogram

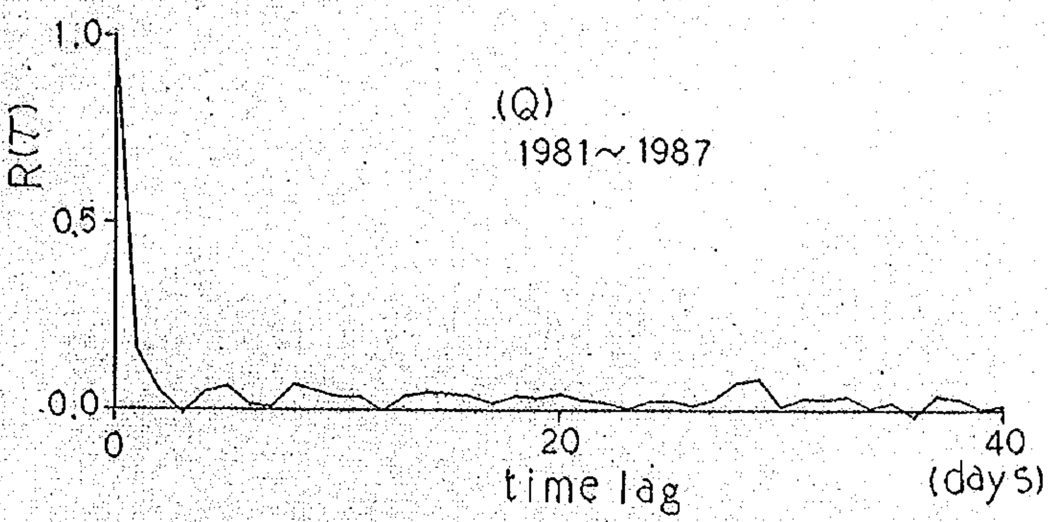


Fig. 4-24 Pattern of the correlogram

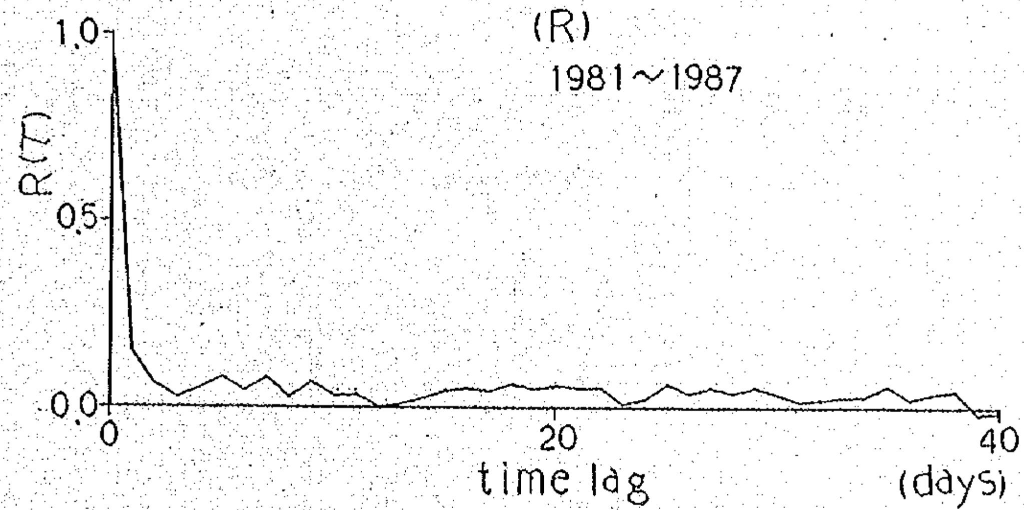


Fig. 4-25 Pattern of the correlogram

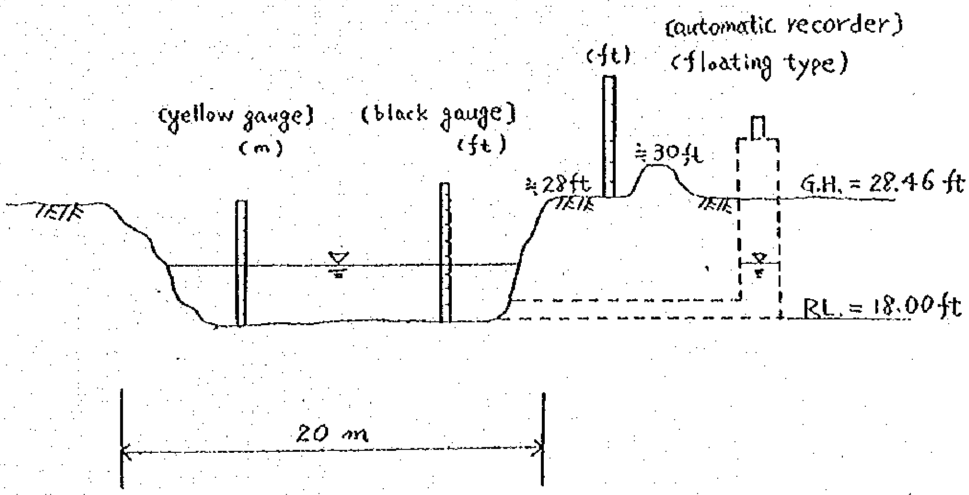


Fig.4-26 ARA KUDA water level gauging station

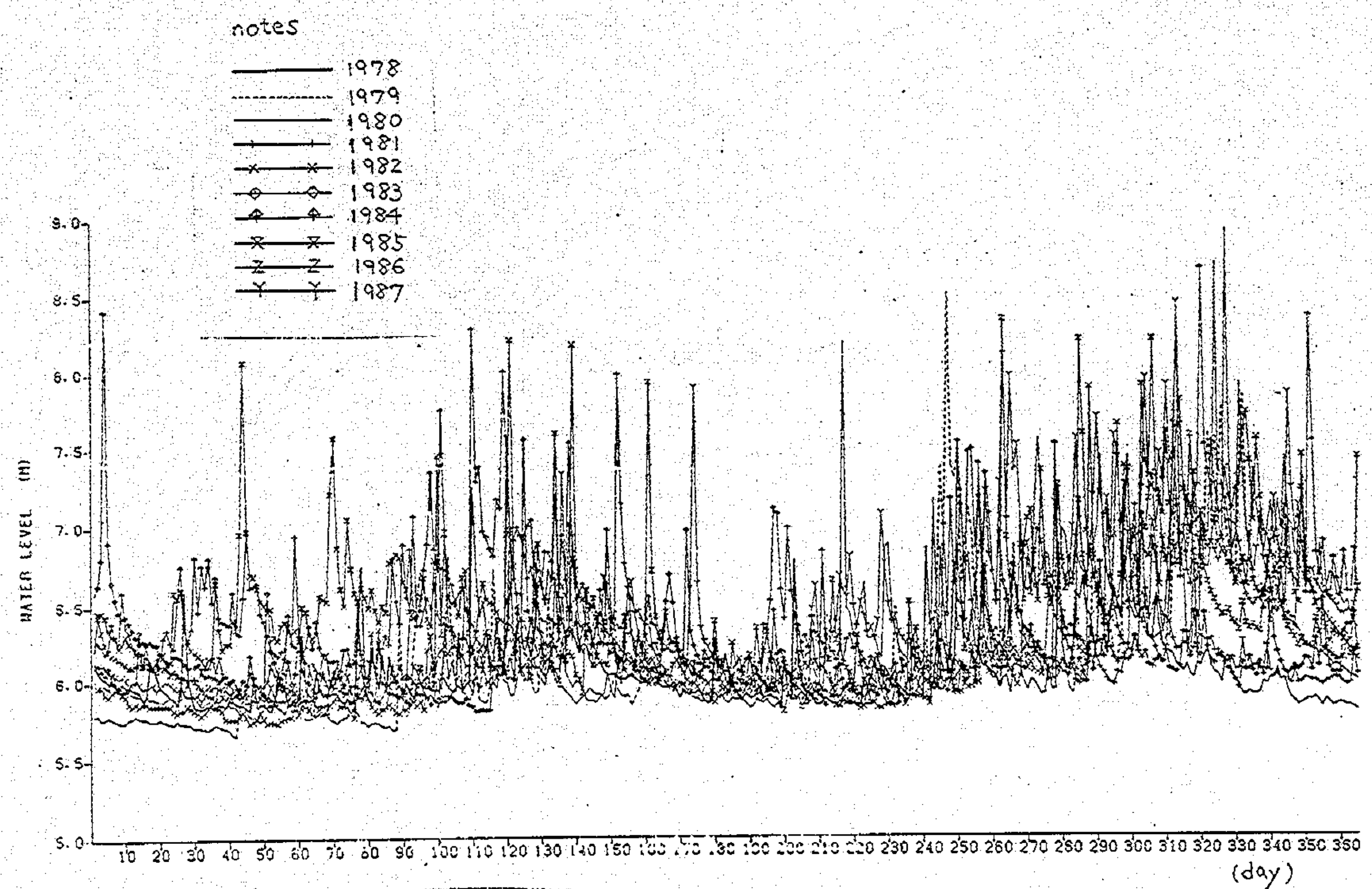


Fig.4-27 Fluctuation of water level at ARA KUDA

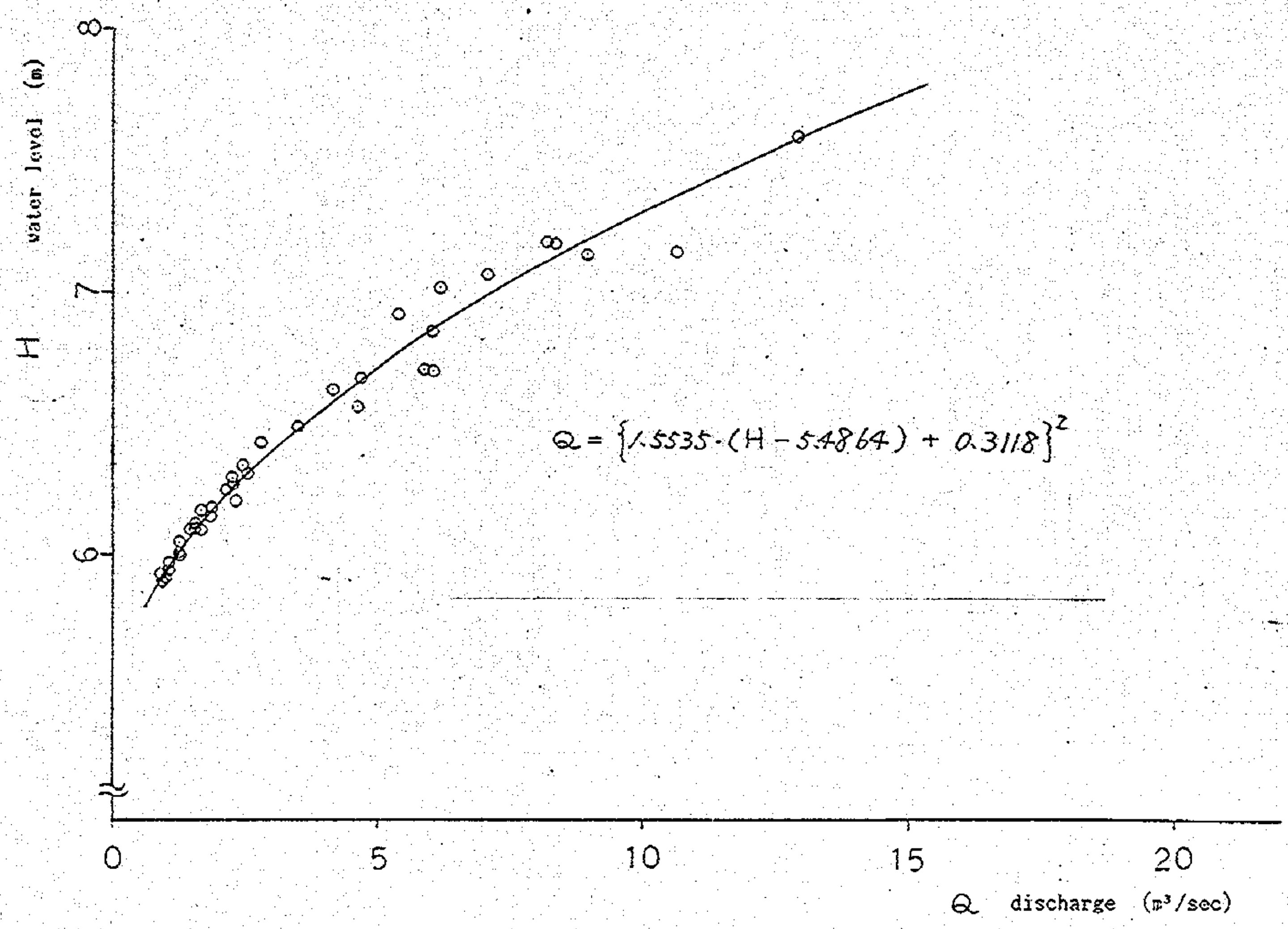


Fig.4-28 Q ~ H curve at ARA KUDA (CA = 139.13 km^2)

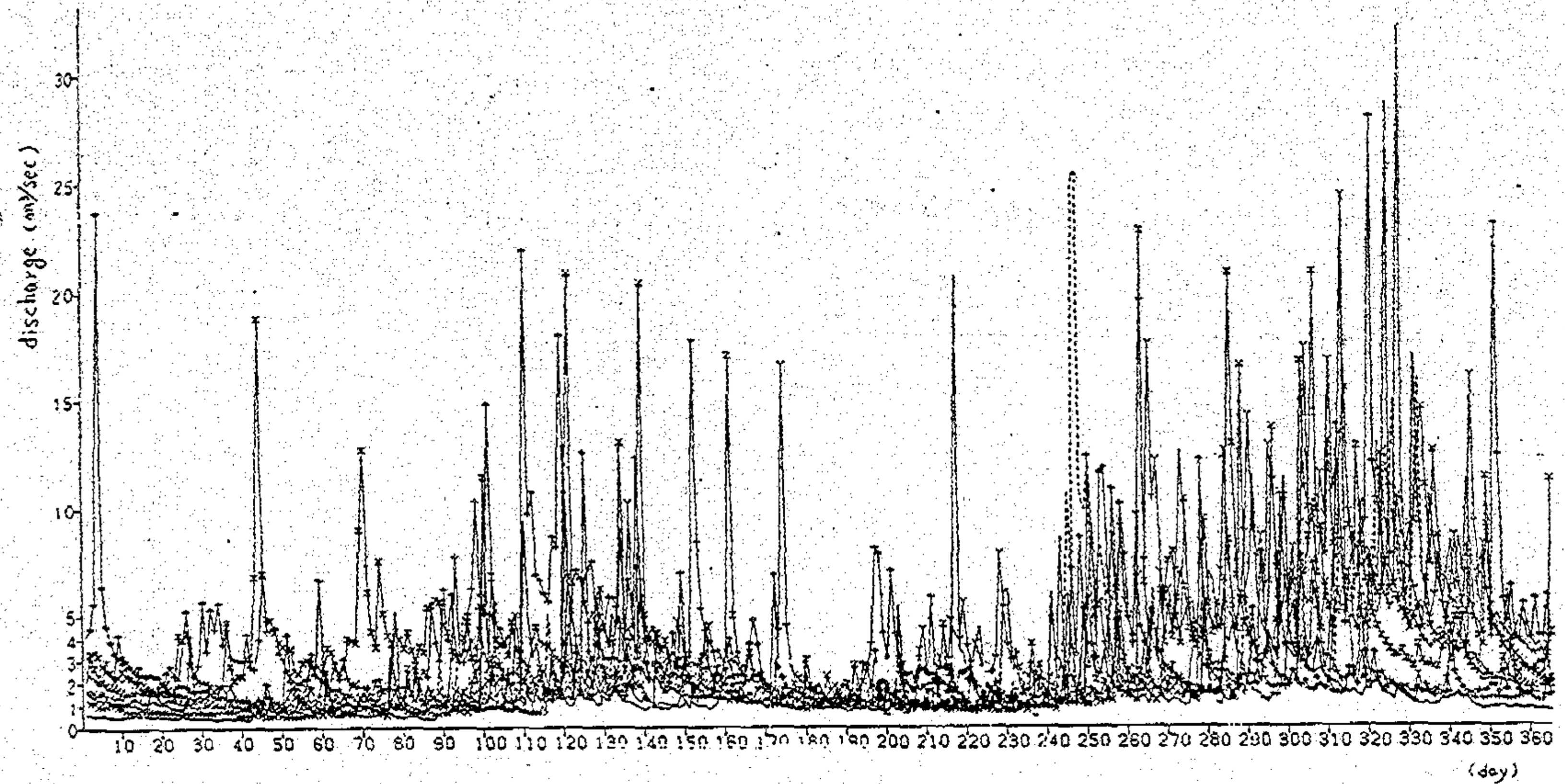


Fig.4-29 Fluctuation of discharge at ARA KUDA

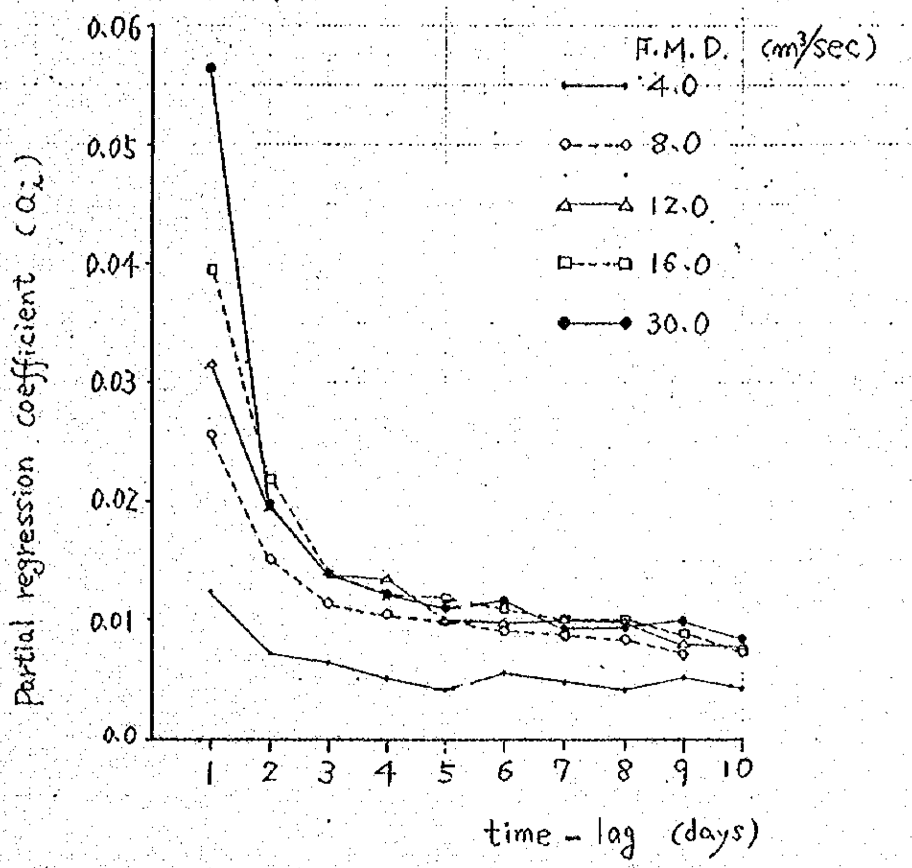


Fig.4-30 Stastical unit graph for daily rainfall

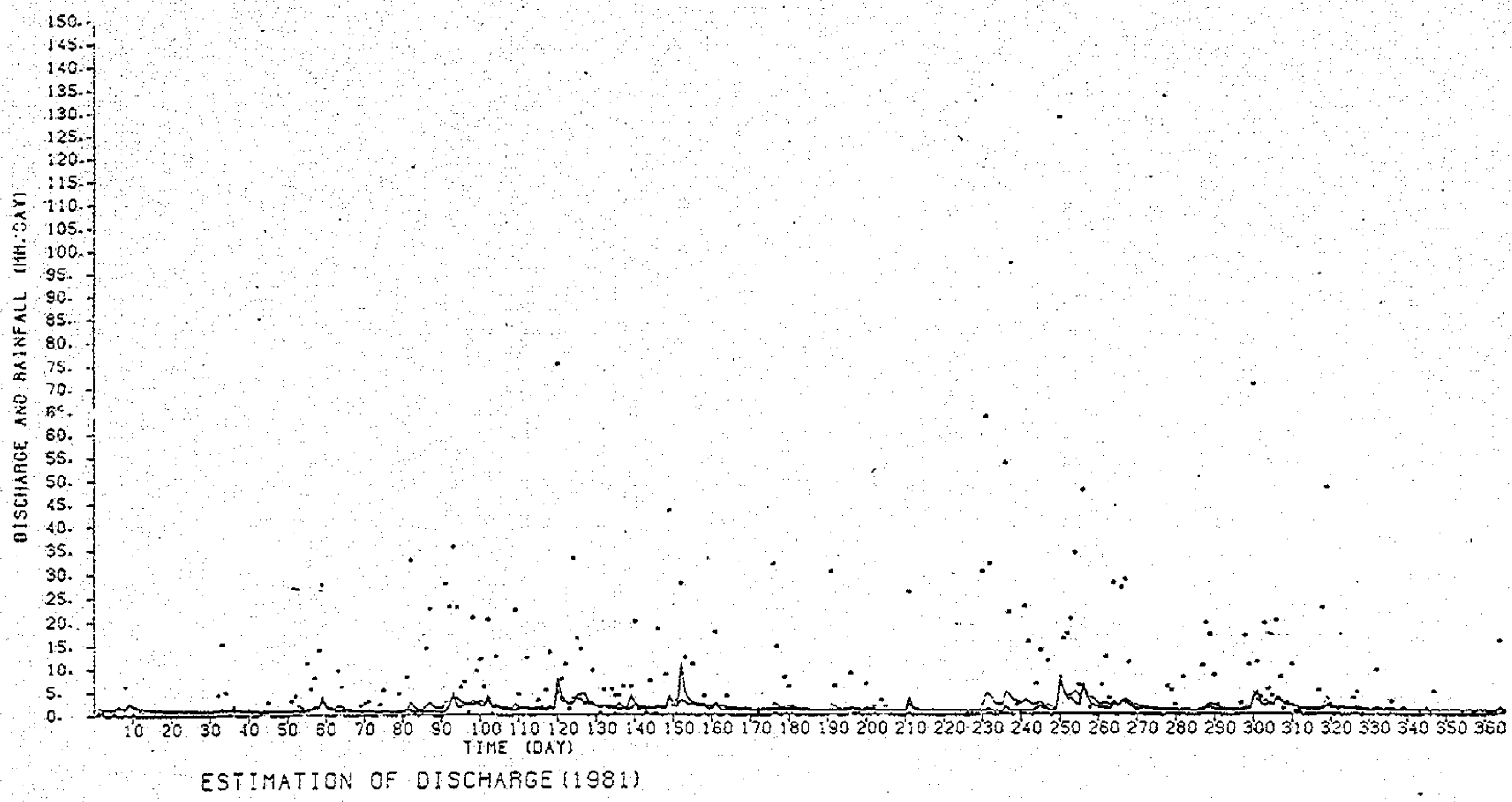
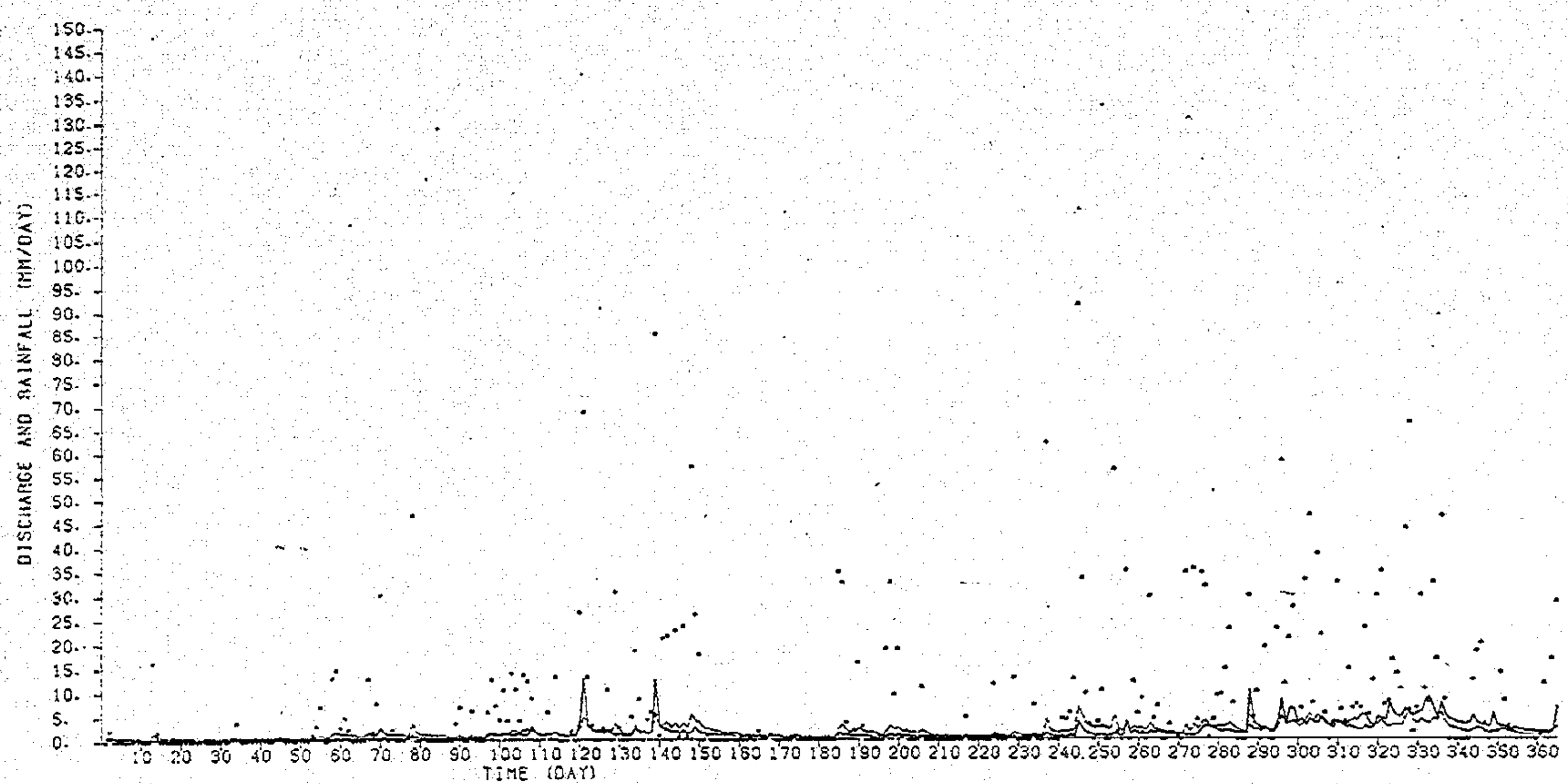


Fig.4-31 Comparison among observed and estimated discharge and daily rainfall (1981)



ESTIMATION OF DISCHARGE(1982)

Fig.4-32 Comparison among observed and estimated discharge and daily rainfall (1982)

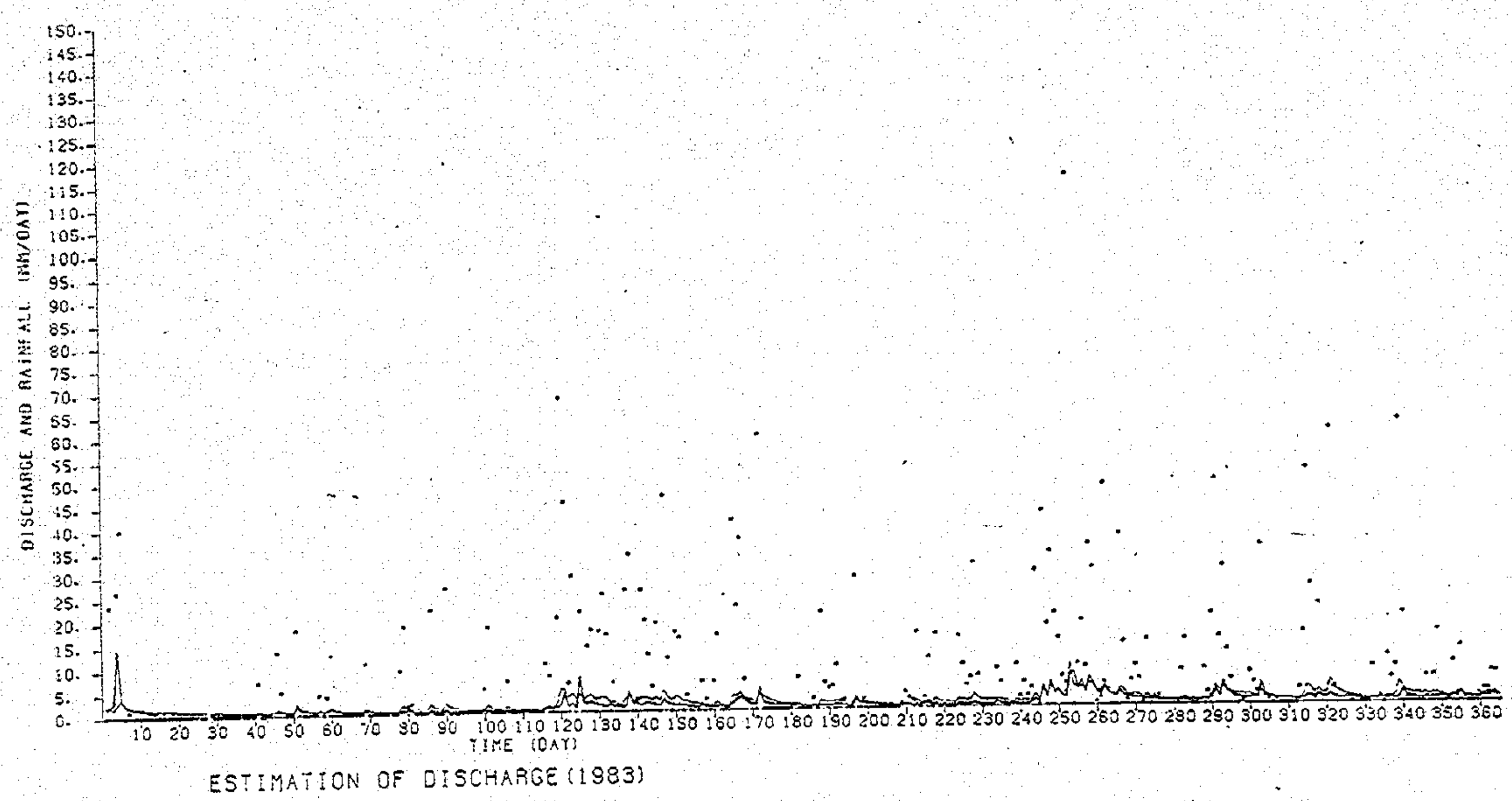


Fig.4-33 Comparison among observed and estimated discharge and daily rainfall (1983)

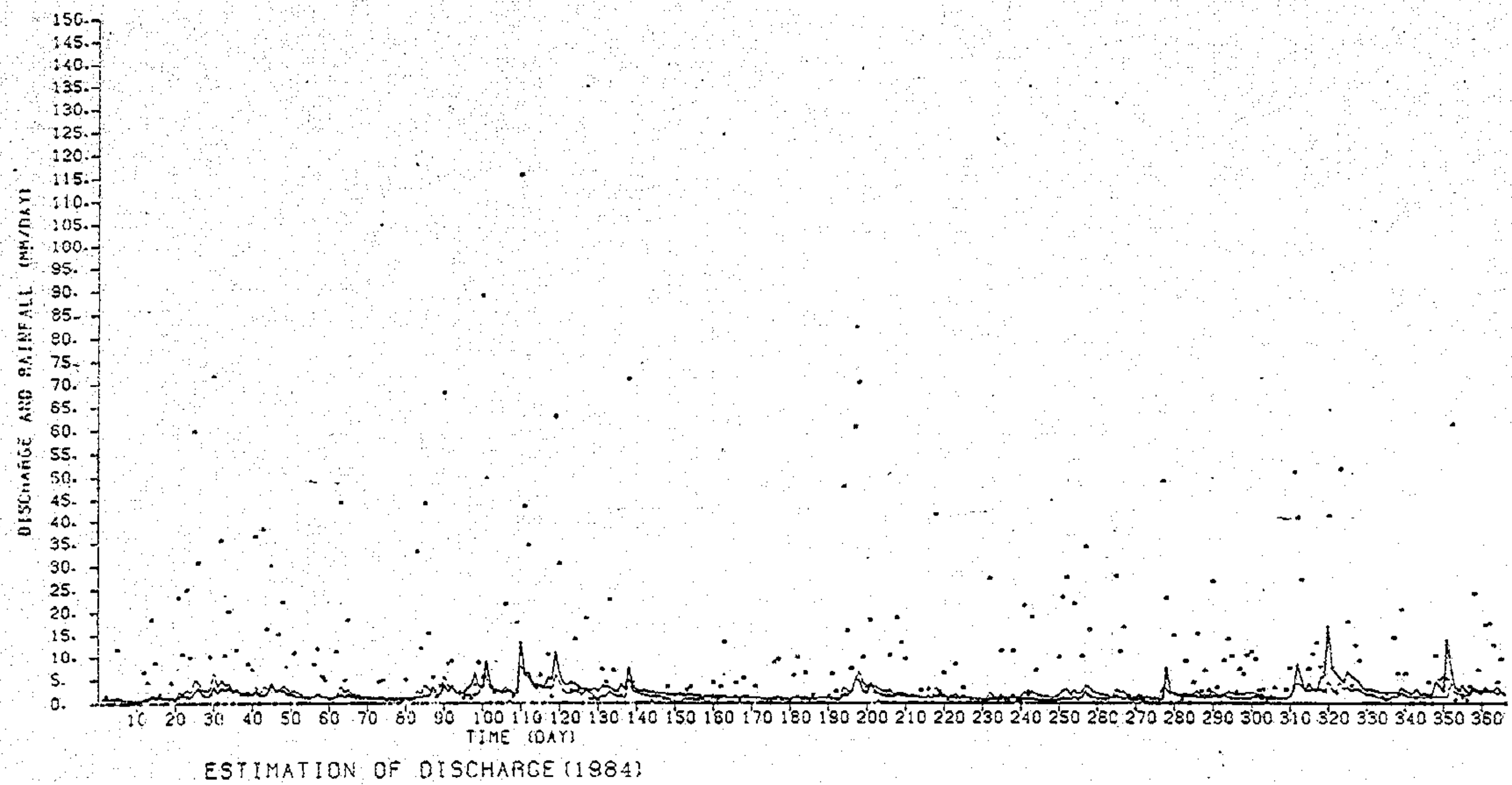


Fig.4-34 Comparison among observed and estimated discharge and daily rainfall (1984)

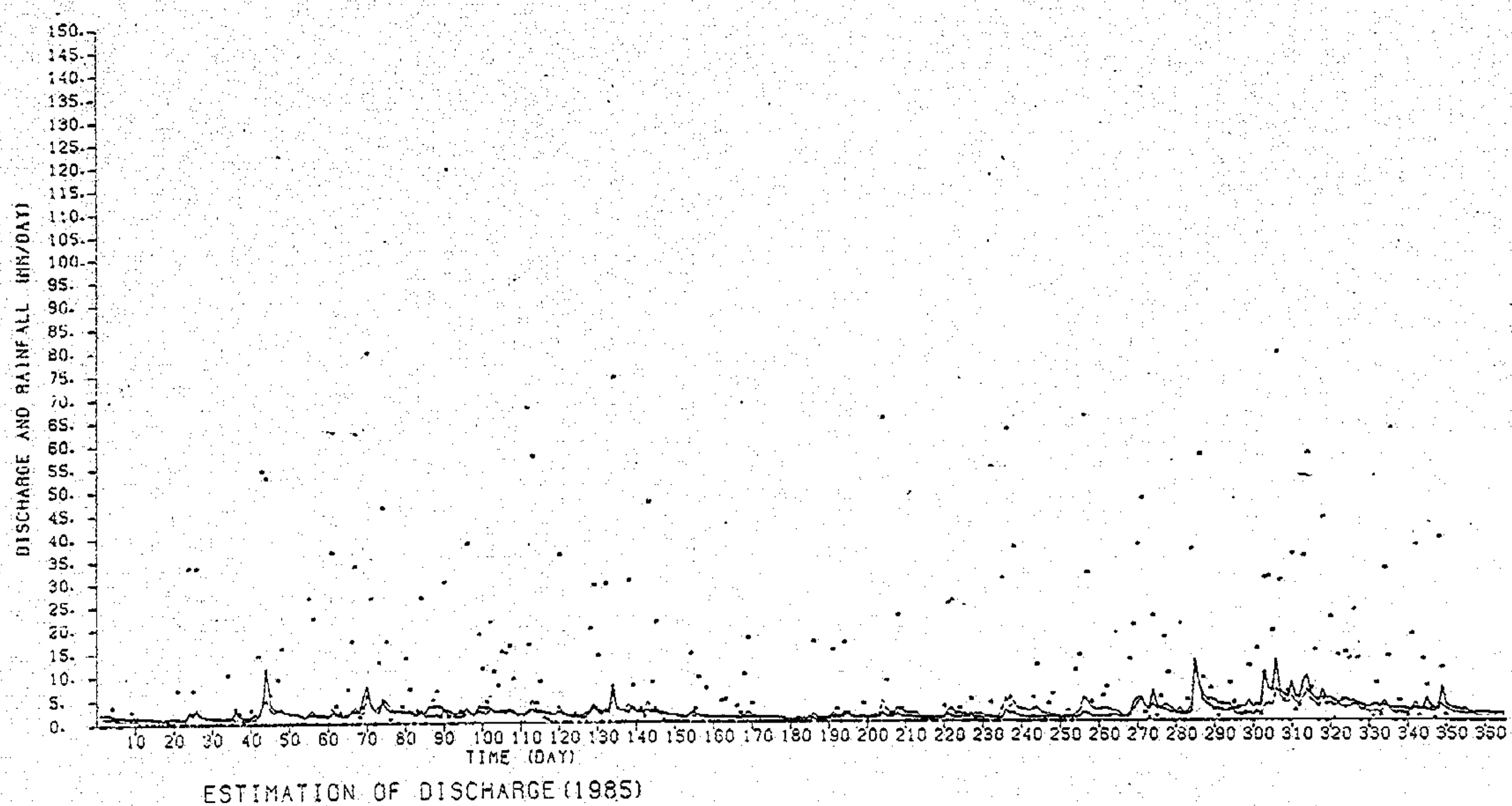


Fig.4-35 Comparison among observed and estimated discharge and daily rainfall (1985)

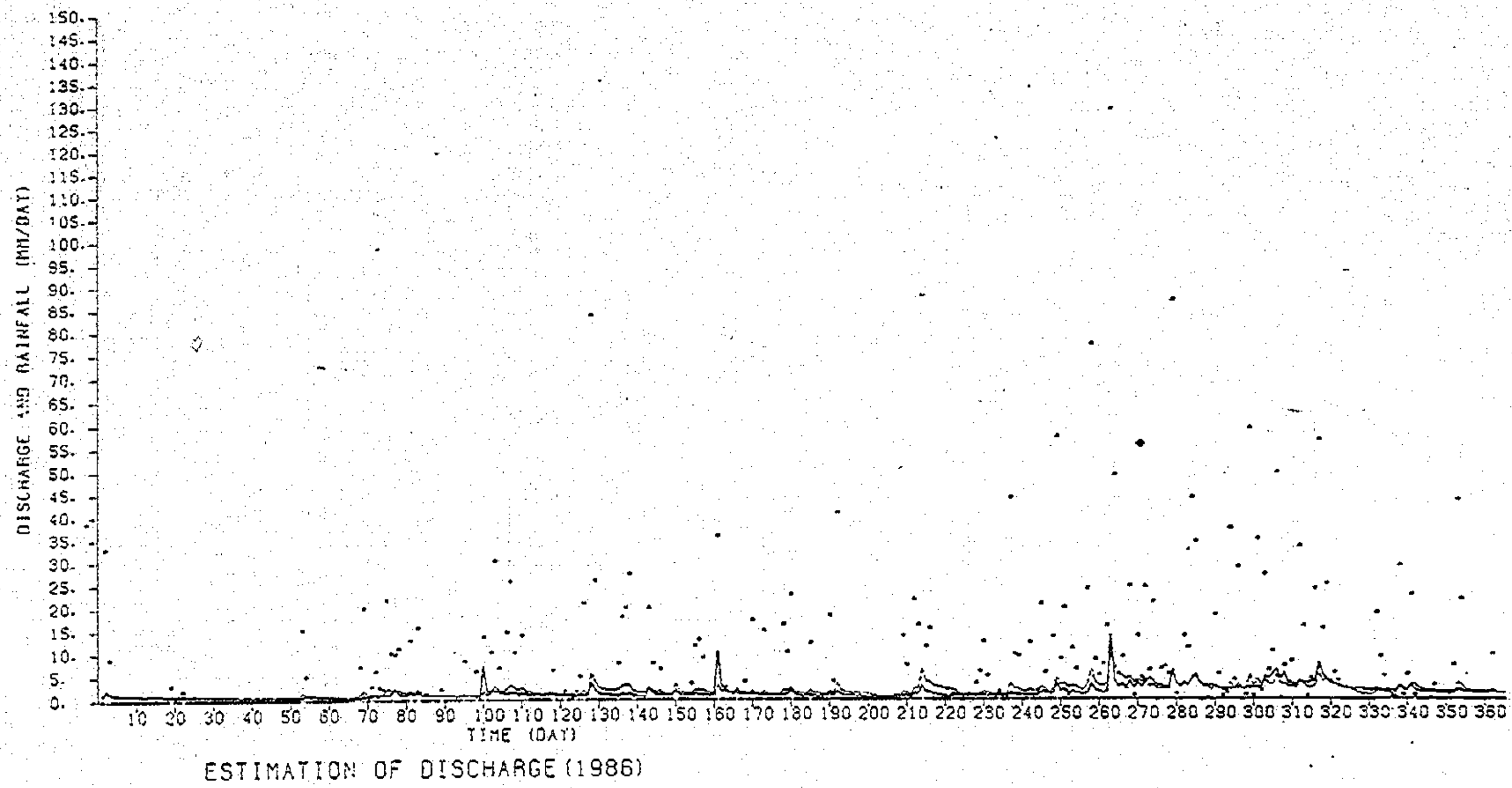


Fig.4-36 Comparison among observed and estimated discharge and daily rainfall (1986)

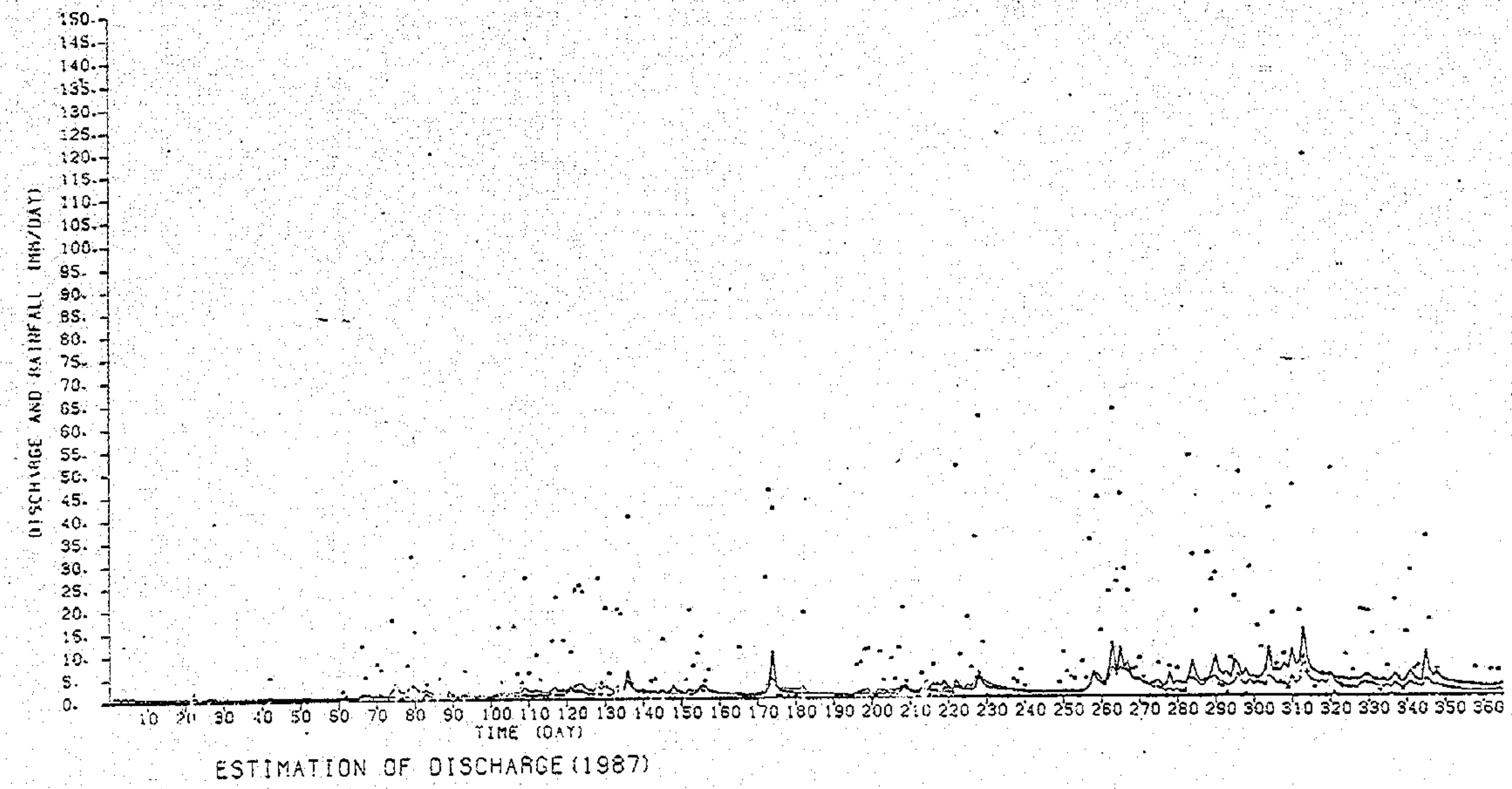


Fig.4-37 Comparison among observed and estimated discharge and daily rainfall (1987)

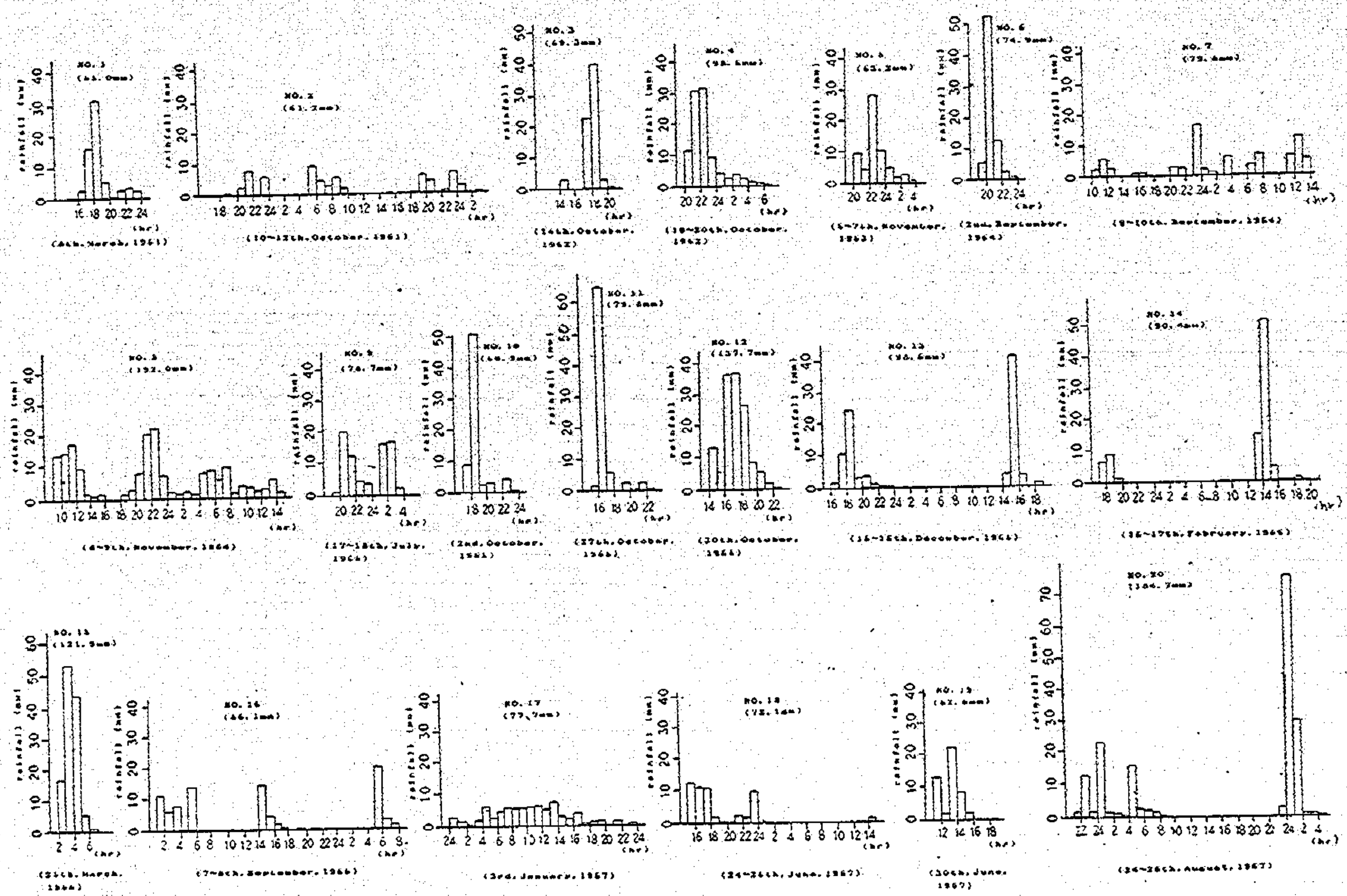


Fig.4-38 Hourly changes of rainfall at Lahar Ikan Mati

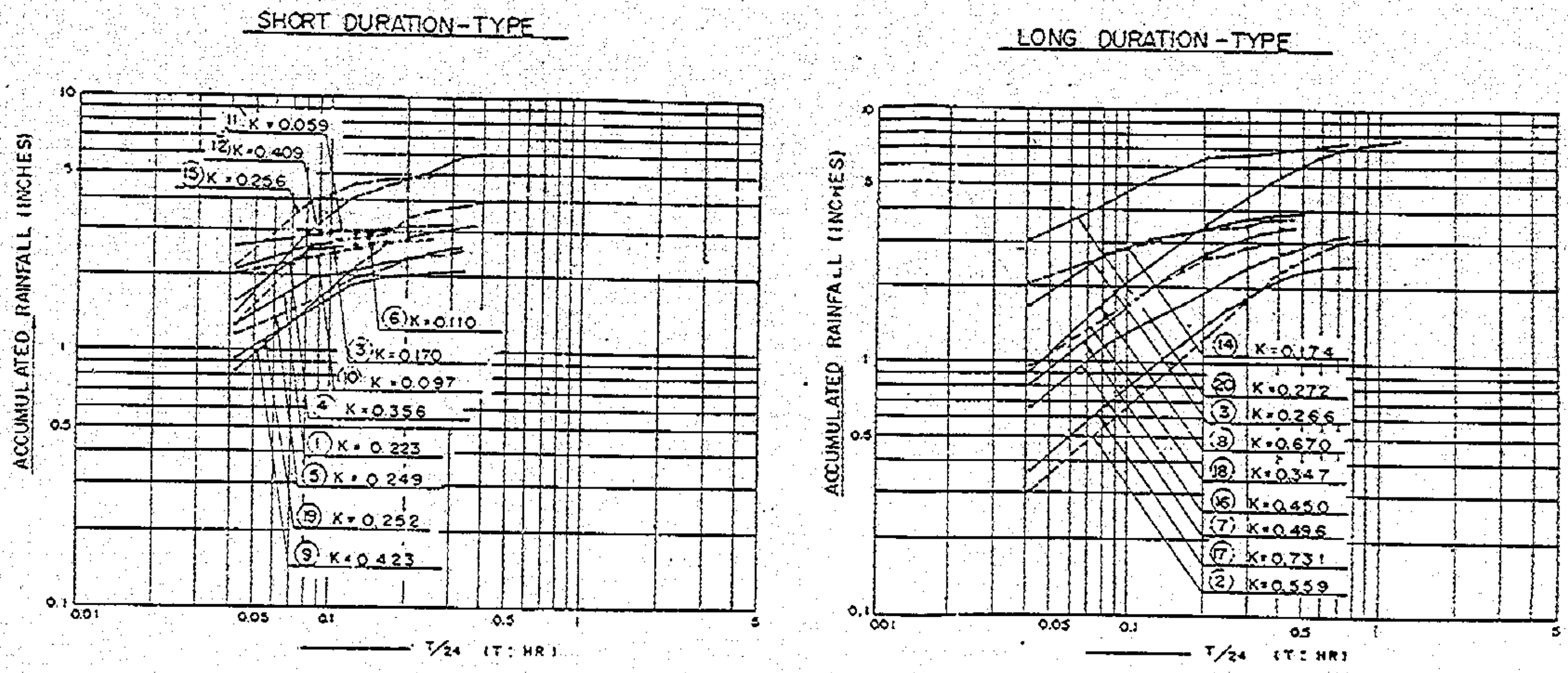


Fig.4-39 Duration – accumulated rainfall curves

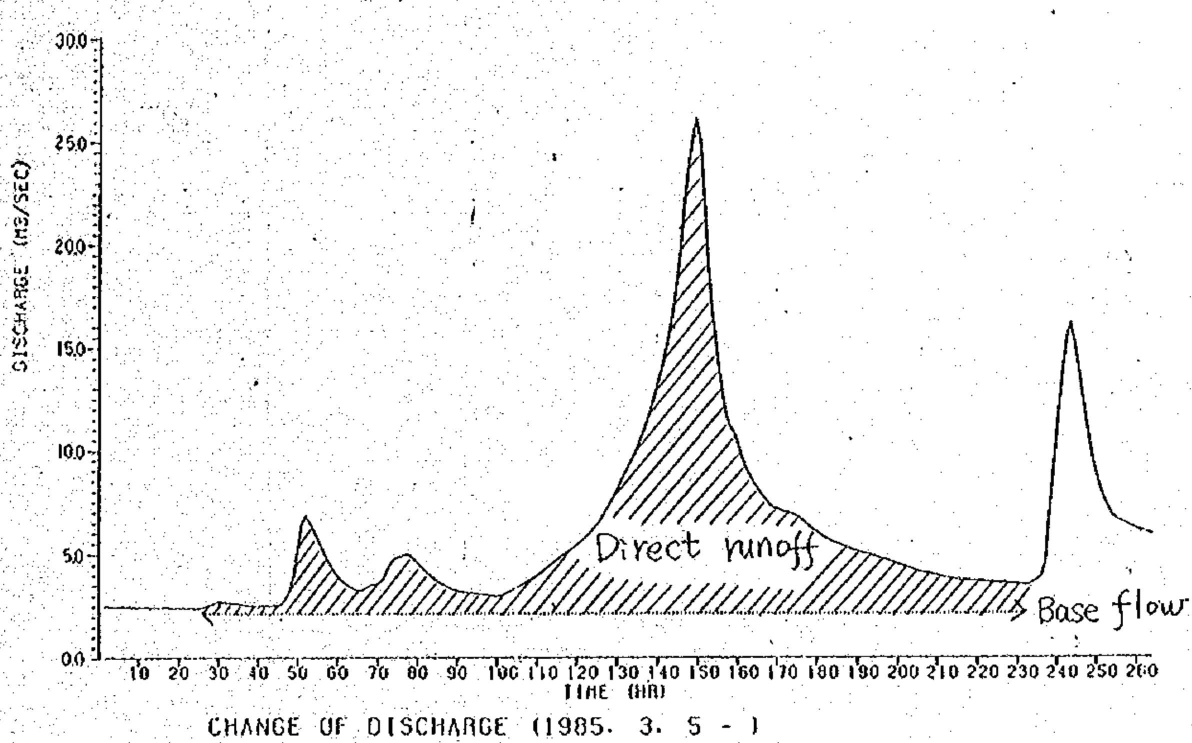


Fig.4-40 Separation of direct runoff and base flow

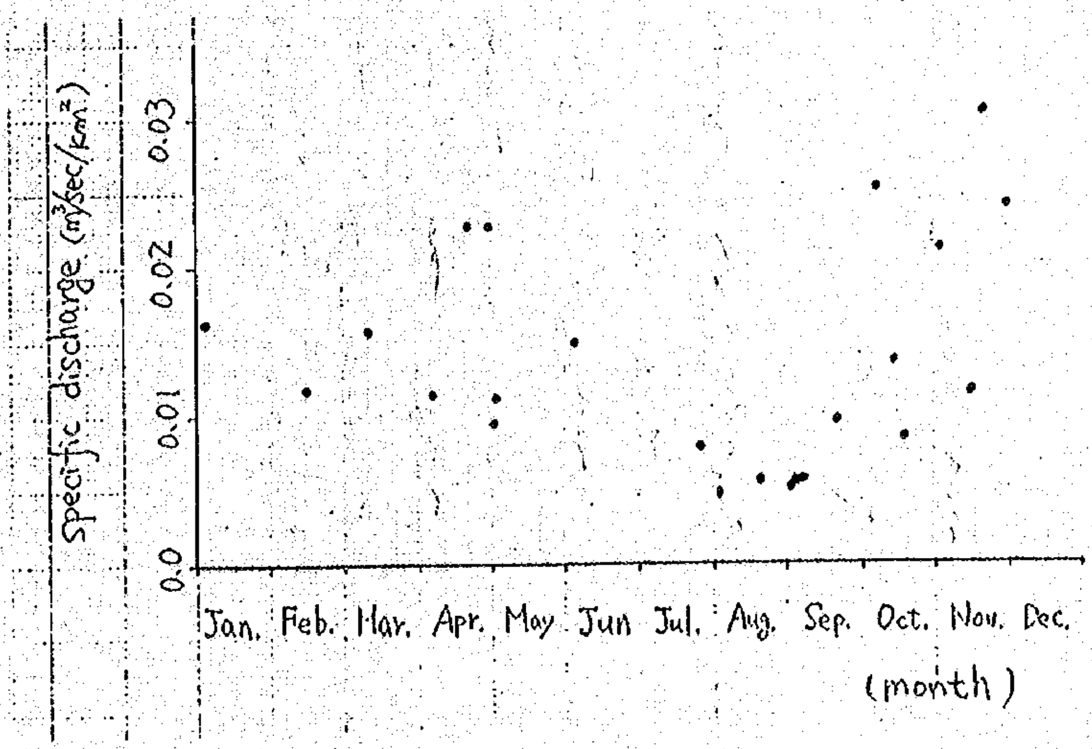


Fig.4-41 Change of specific base flow in flood

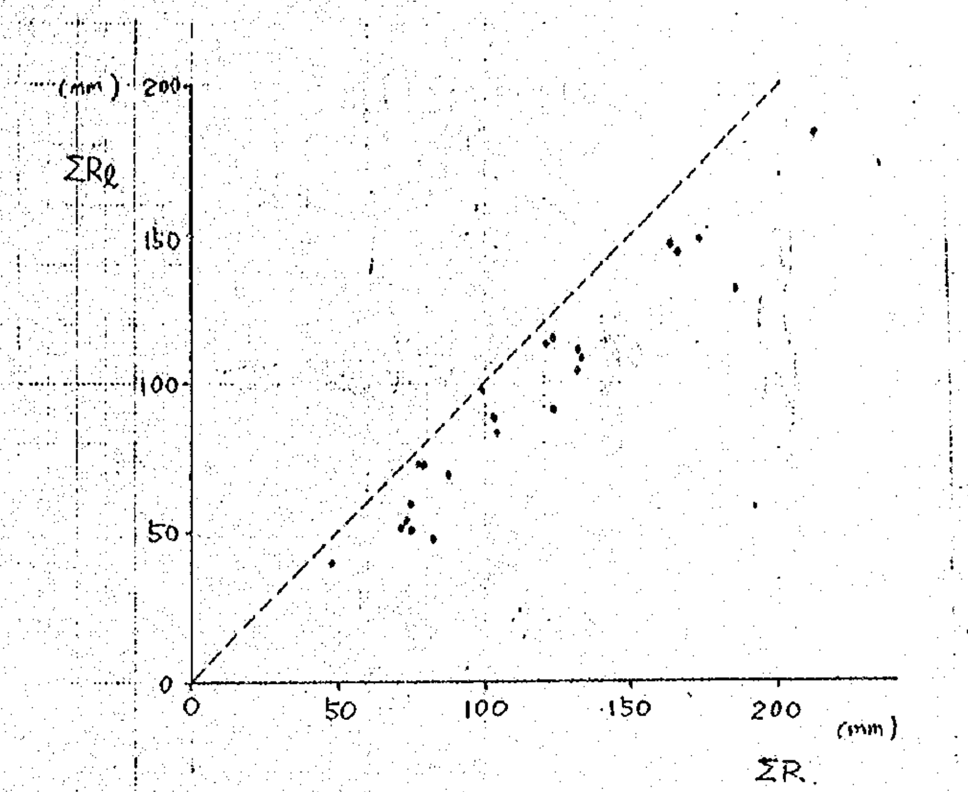


Fig.4-42 Relation between accumulated rainfall and accumulated rainfall loss

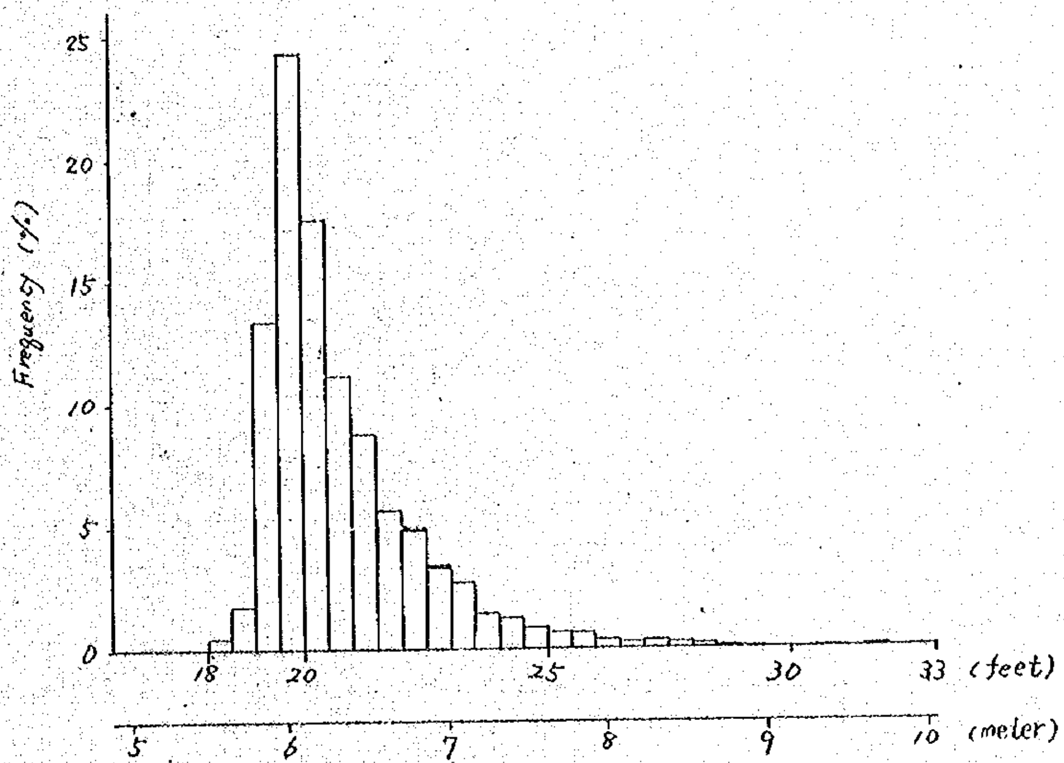
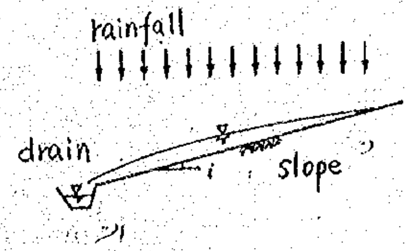
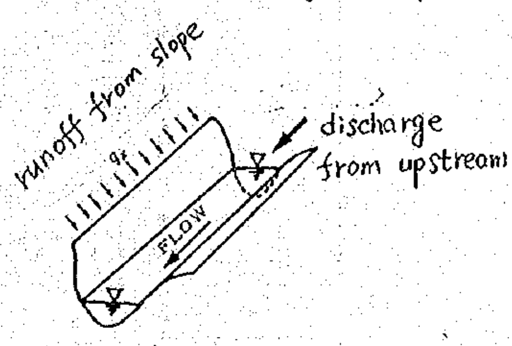


Fig.4-43 Frequency of water level at ARA KUDA (1978~1987)



Runoff model on uniform gradient slope



Drainage canal model with influent from side area

Fig.4-44 Runoff model

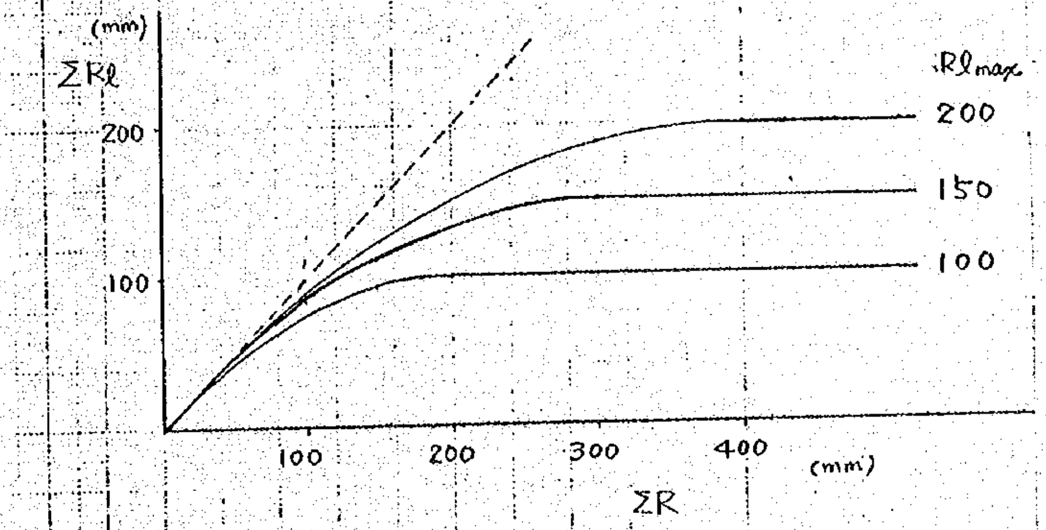


Fig.4-45 Relation between accumulated rainfall and accumulated rainfall loss

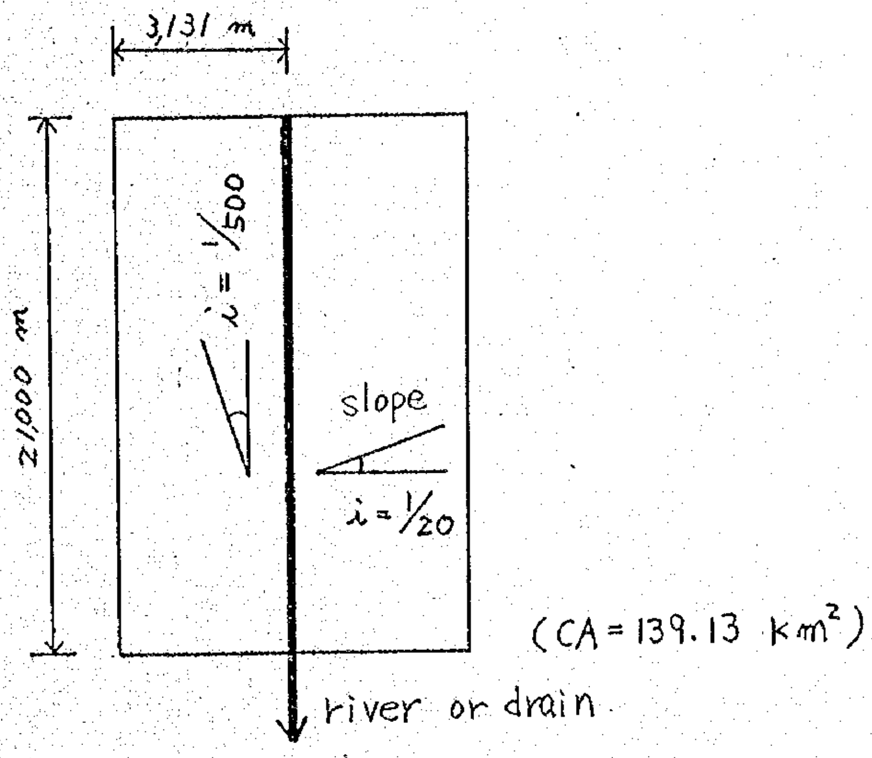


Fig.4-46 Basin model used with Kinematic Wave Method

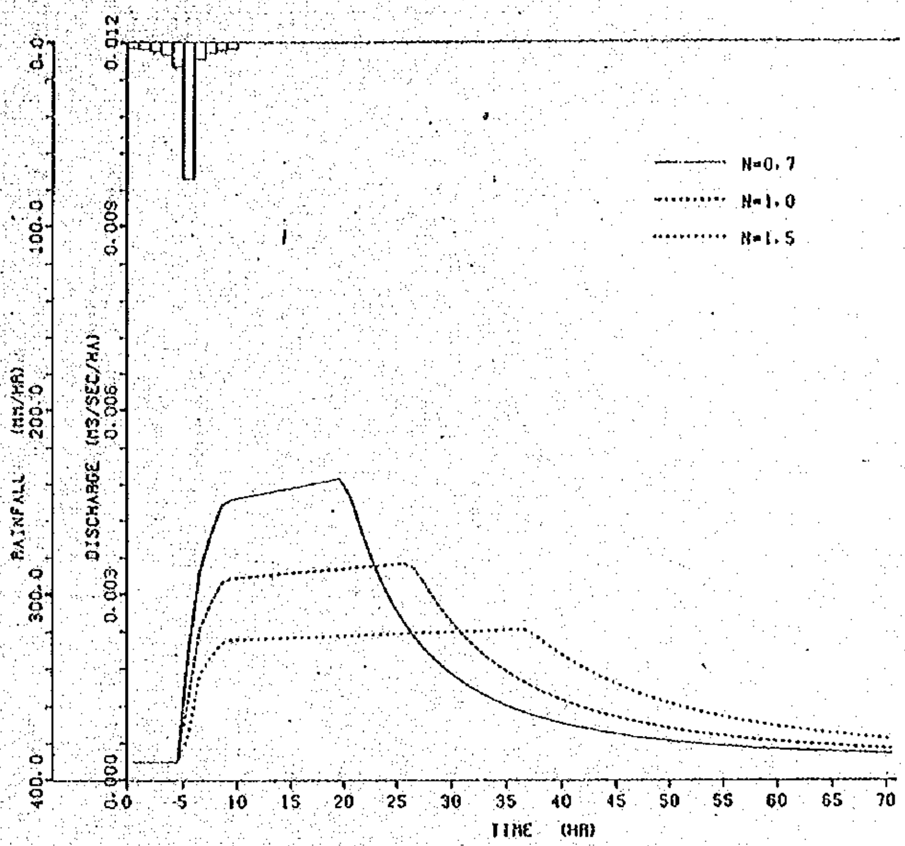


Fig.4-47 Hydrograph (T=1/5, R1max=100)

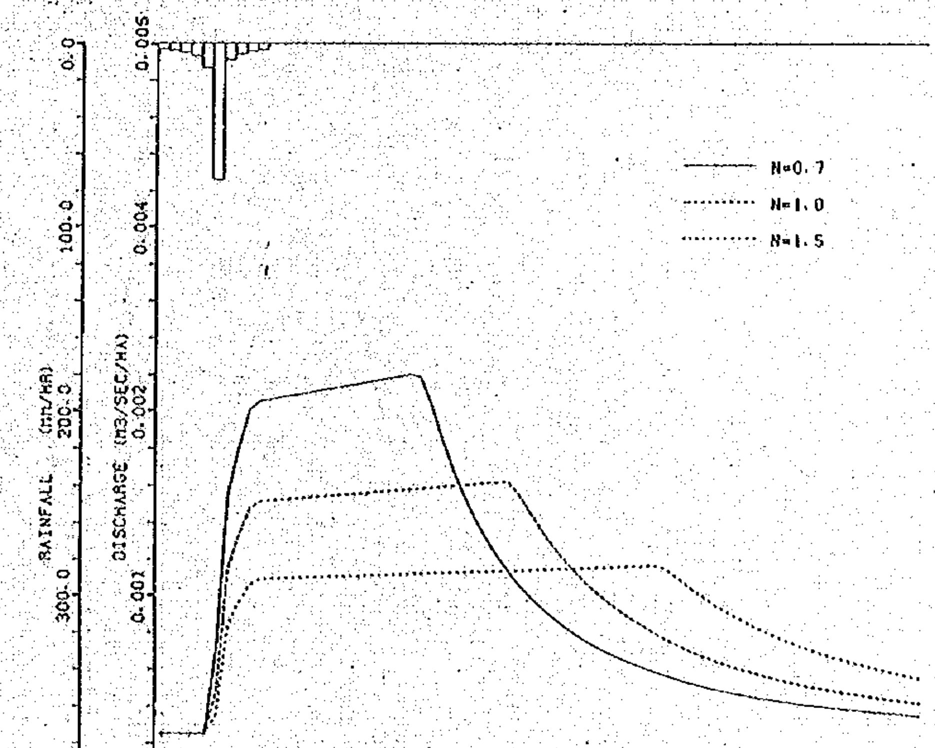


Fig.4-48 Hydrograph (T=1/5, R1max=150)

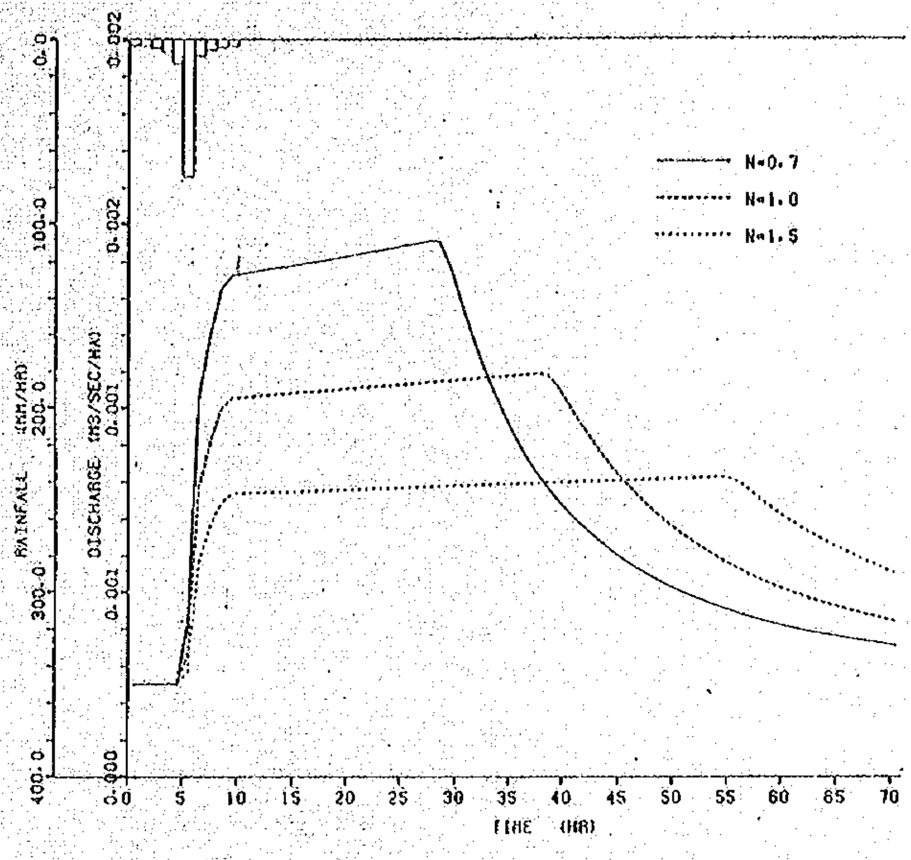


Fig.4-49 Hydrograph (T=1/5, Rlmax=200)

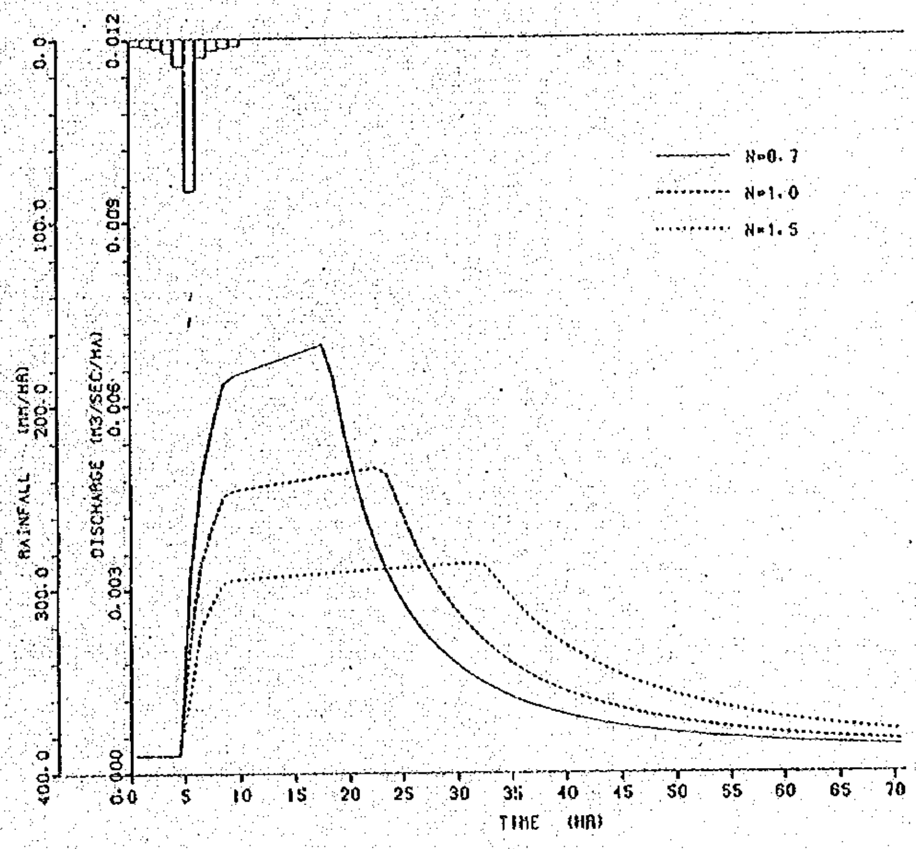


Fig.4-50 Hydrograph (T=1/10, Rlmax=100)

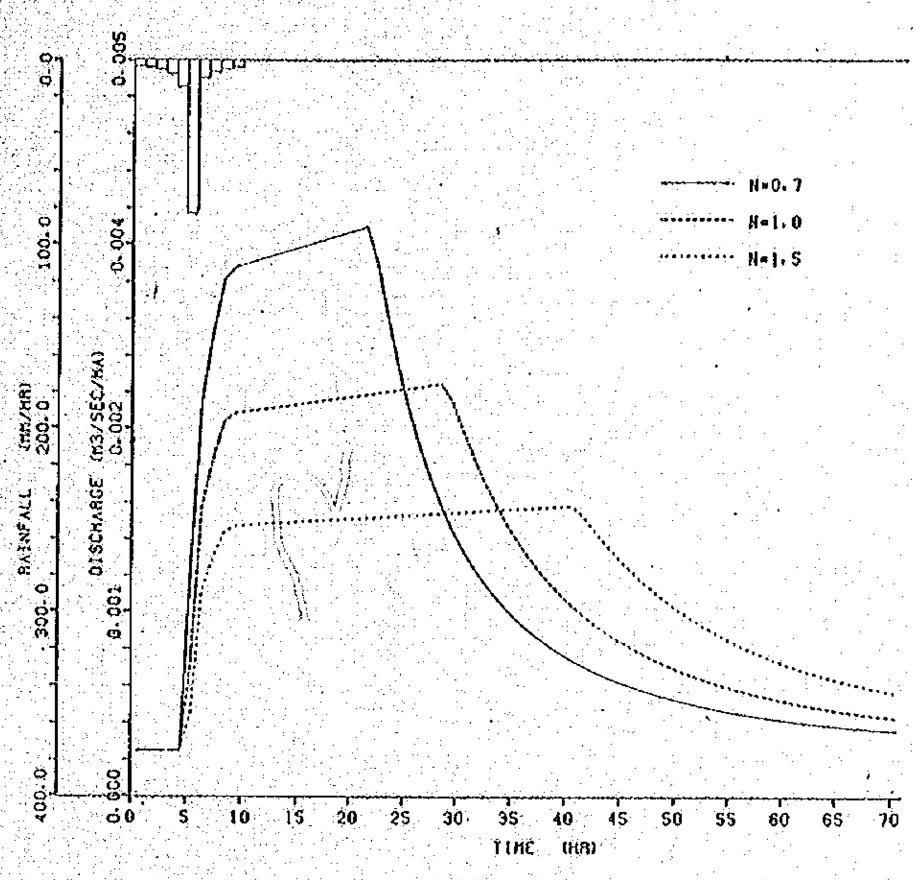


Fig.4-51 Hydrograph (T=1/10, R1max=150)

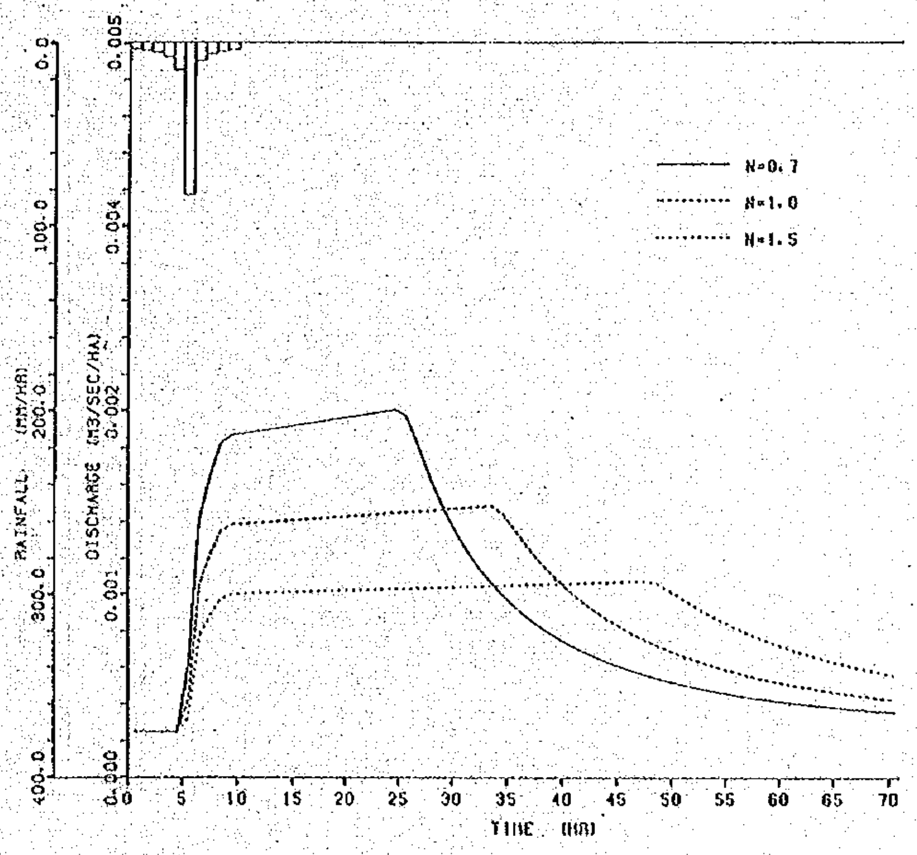


Fig.4-52 Hydrograph (T=1/10, R1max=200)

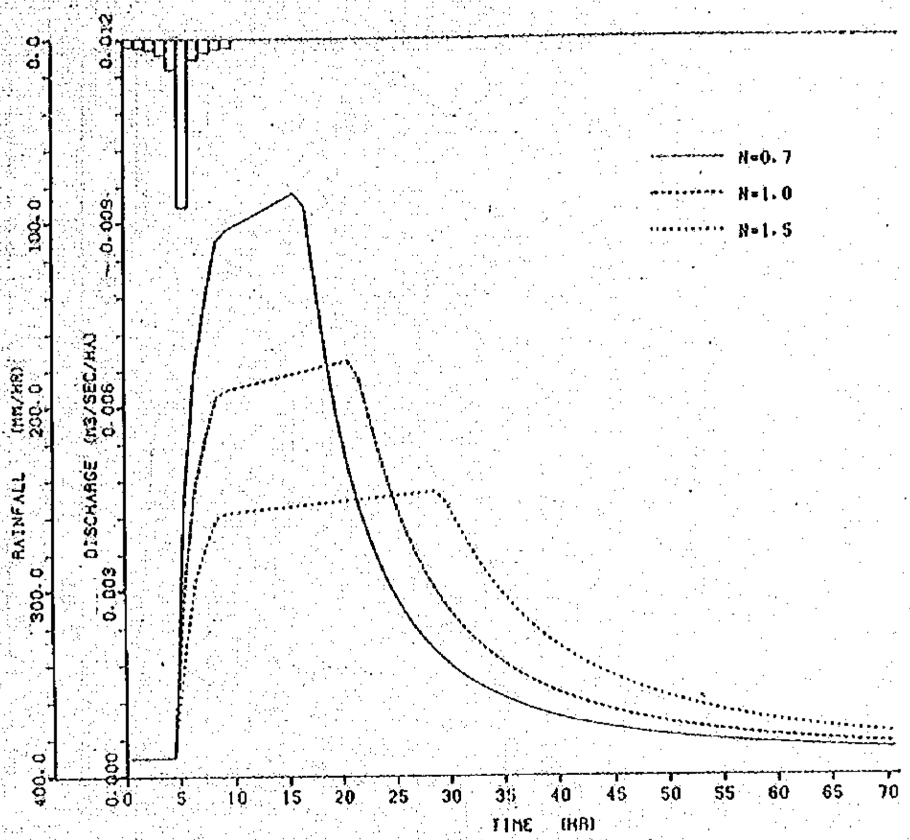


Fig.4-53 Hydrograph (T=1/20, Rlmax=100)

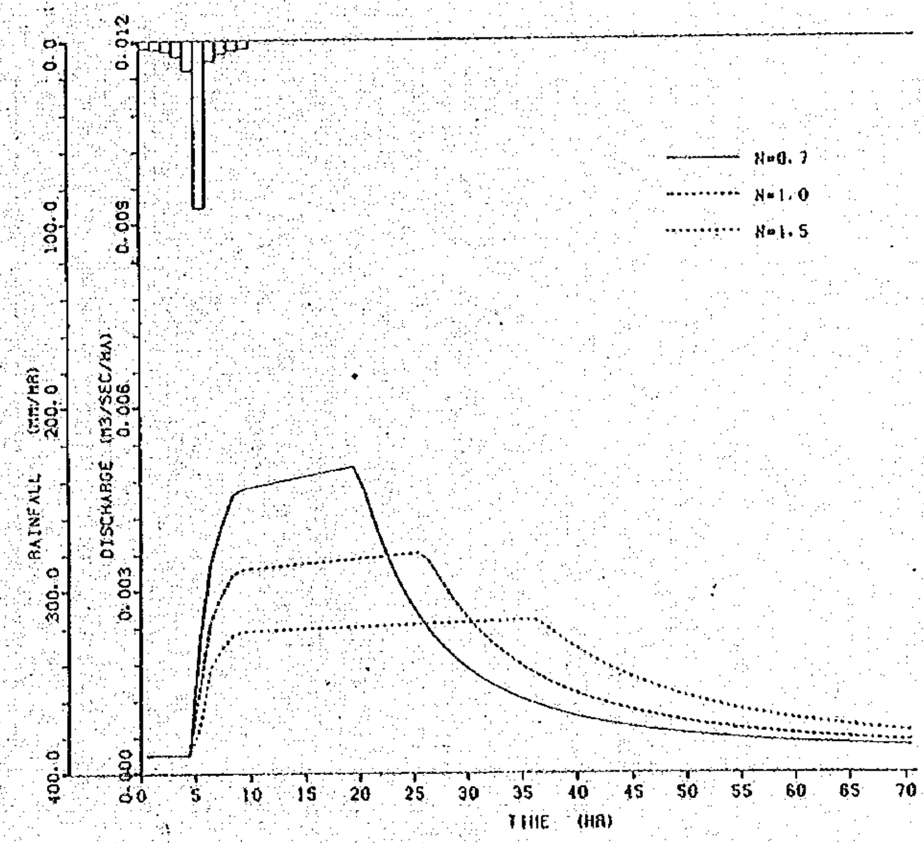


Fig.4-54 Hydrograph (T=1/20, Rlmax=150)

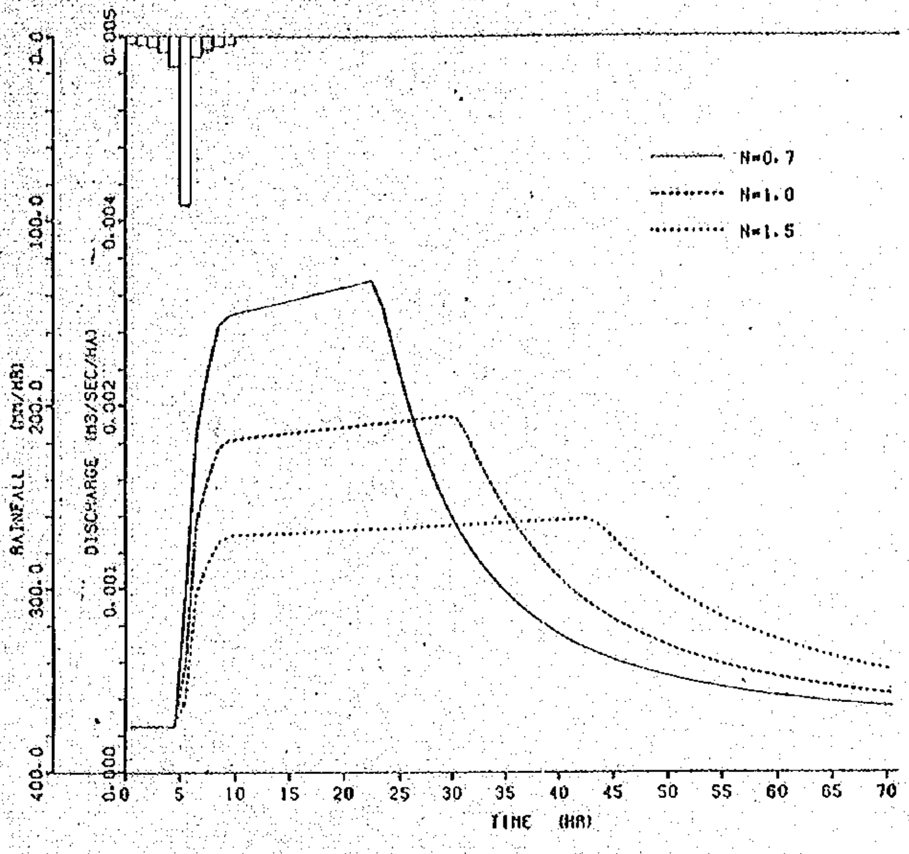


Fig.4-55 Hydrograph (T=1/20, Rlmax=200)

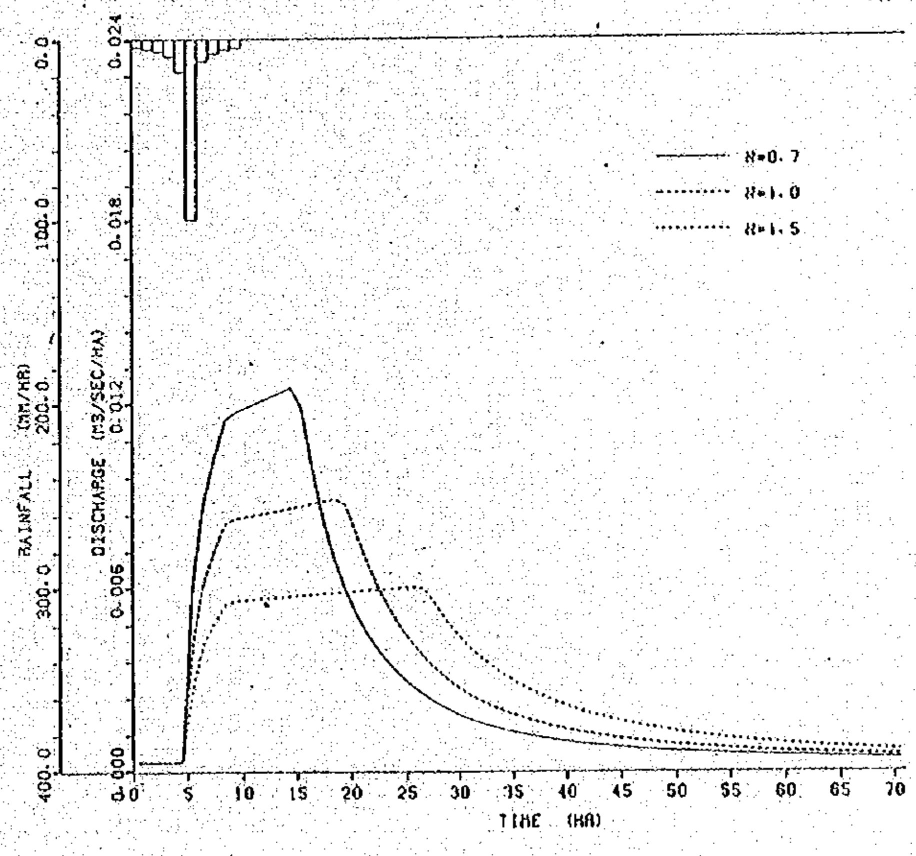


Fig.4-56 Hydrograph (T=1/40, Rlmax=100)

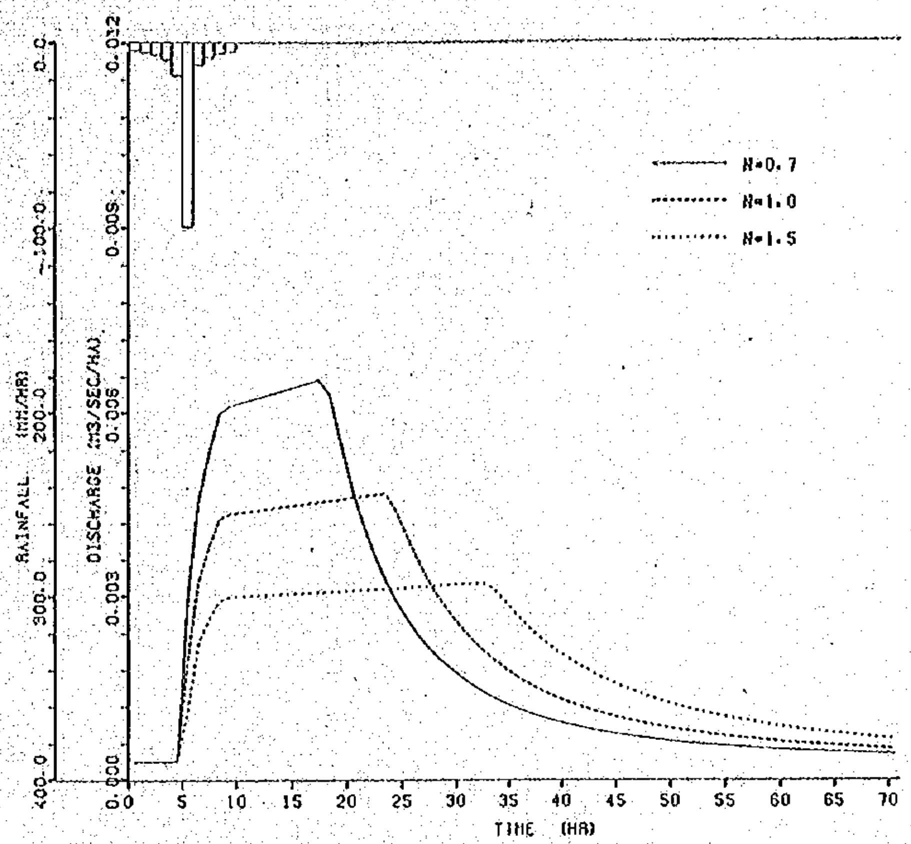


Fig.4-57 Hydrograph (T=1/40, R1max=150)

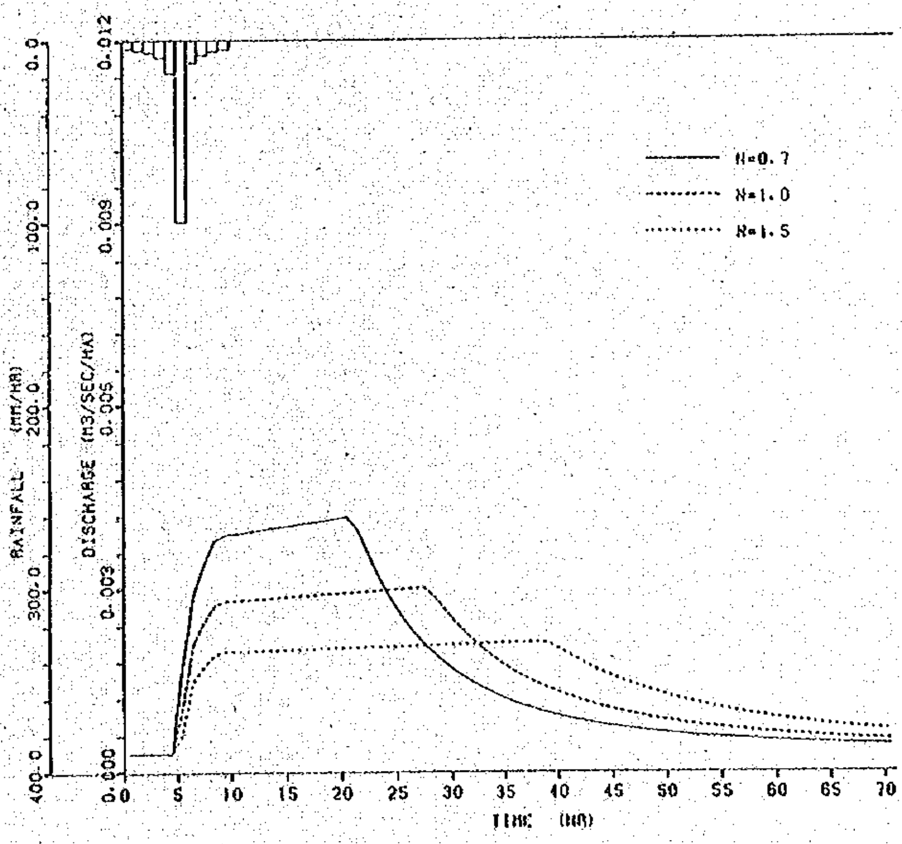


Fig.4-58 Hydrograph (T=1/40, R1max=200)

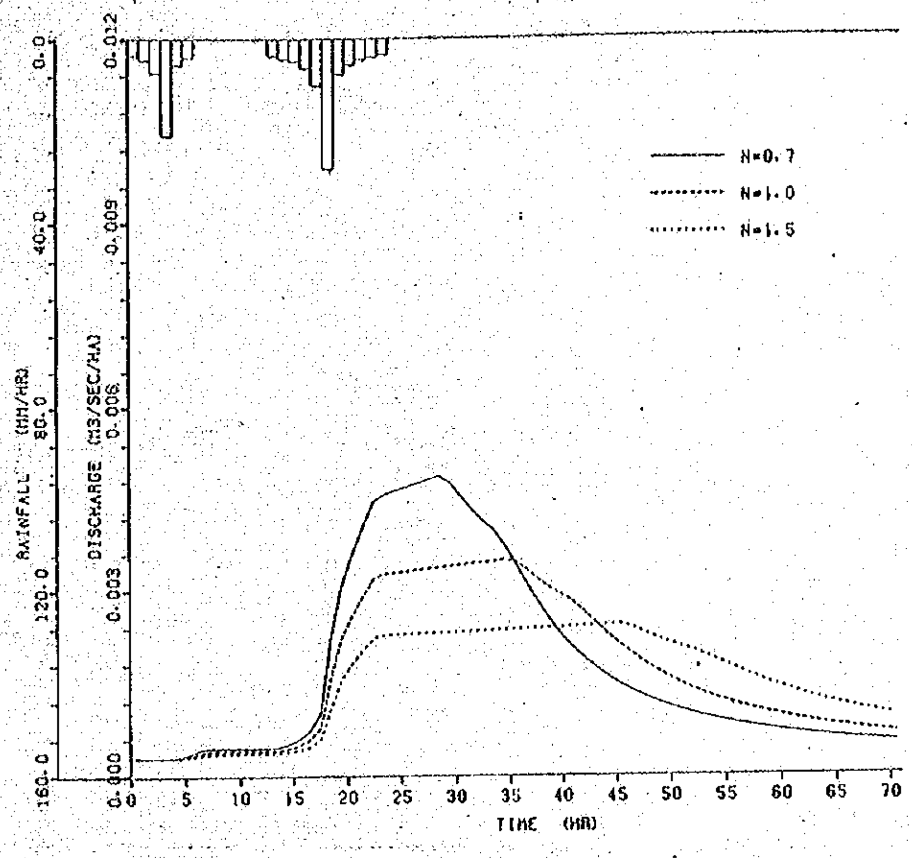


Fig.4-59 Hydrograph (T=1/5, Rlmax=100)

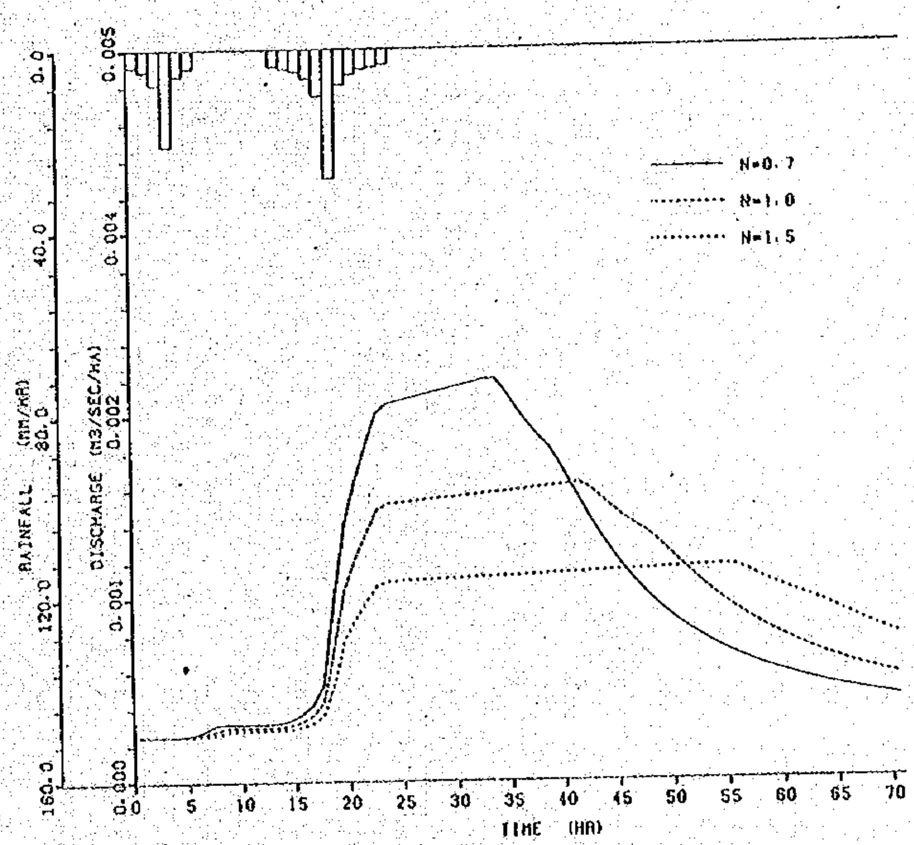


Fig.4-60 Hydrograph (T=1/5, Rlmax=150)

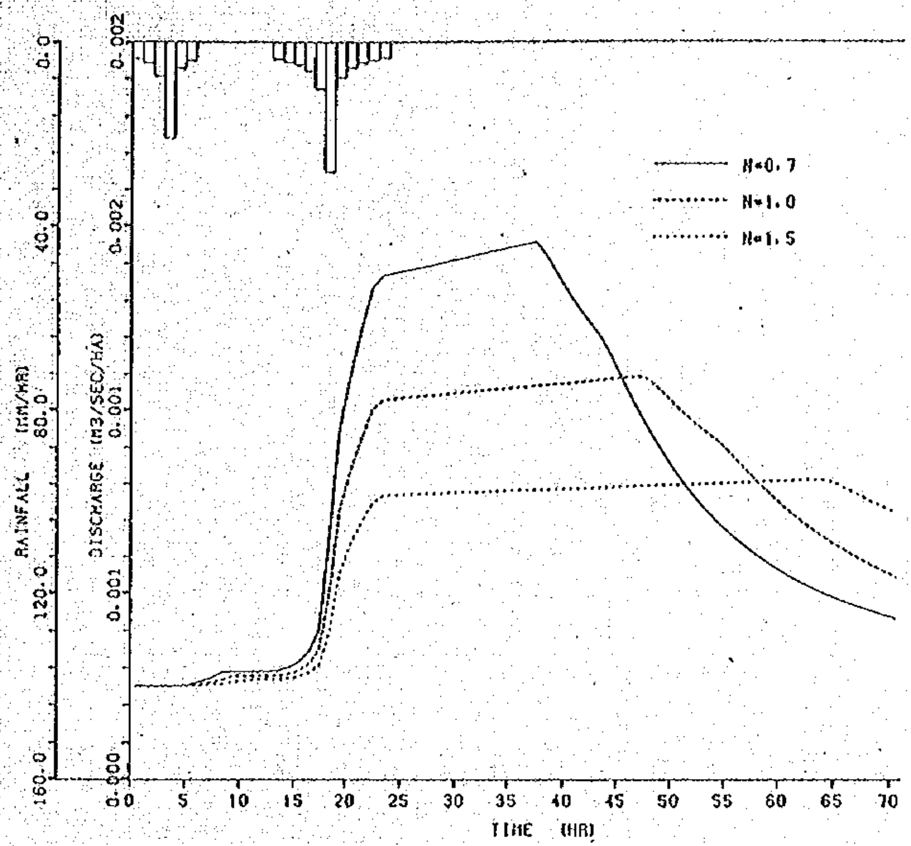


Fig.4-61 Hydrograph (T=1/5, Rlmax=200)

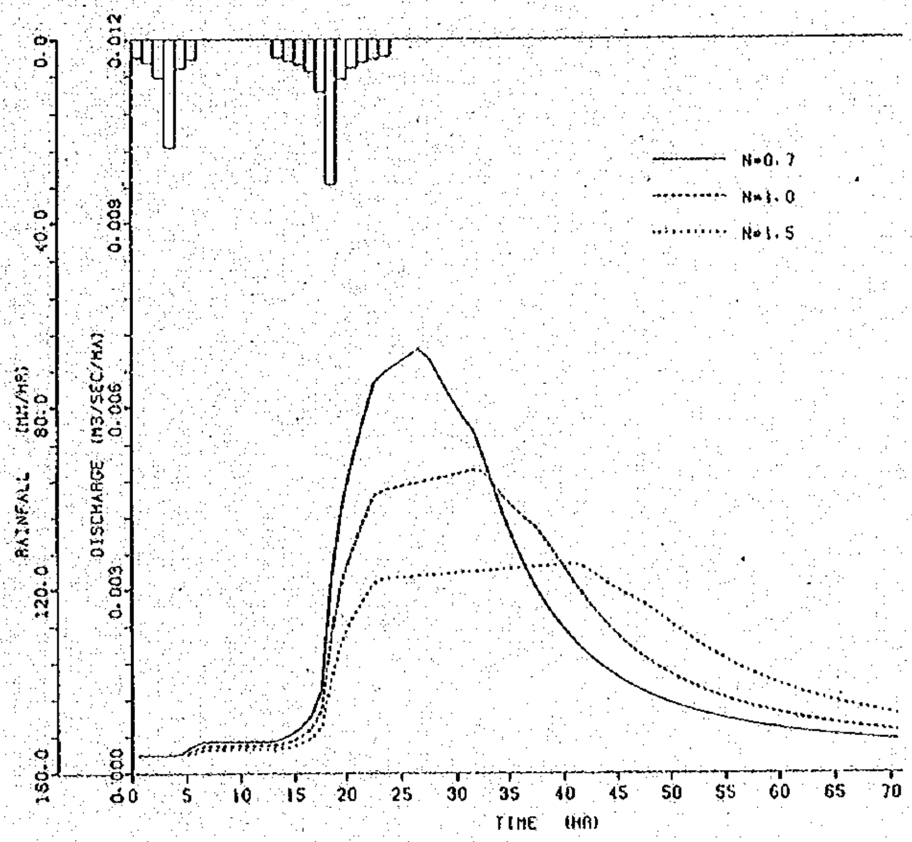


Fig.4-62 Hydrograph (T=1/10, Rlmax=100)

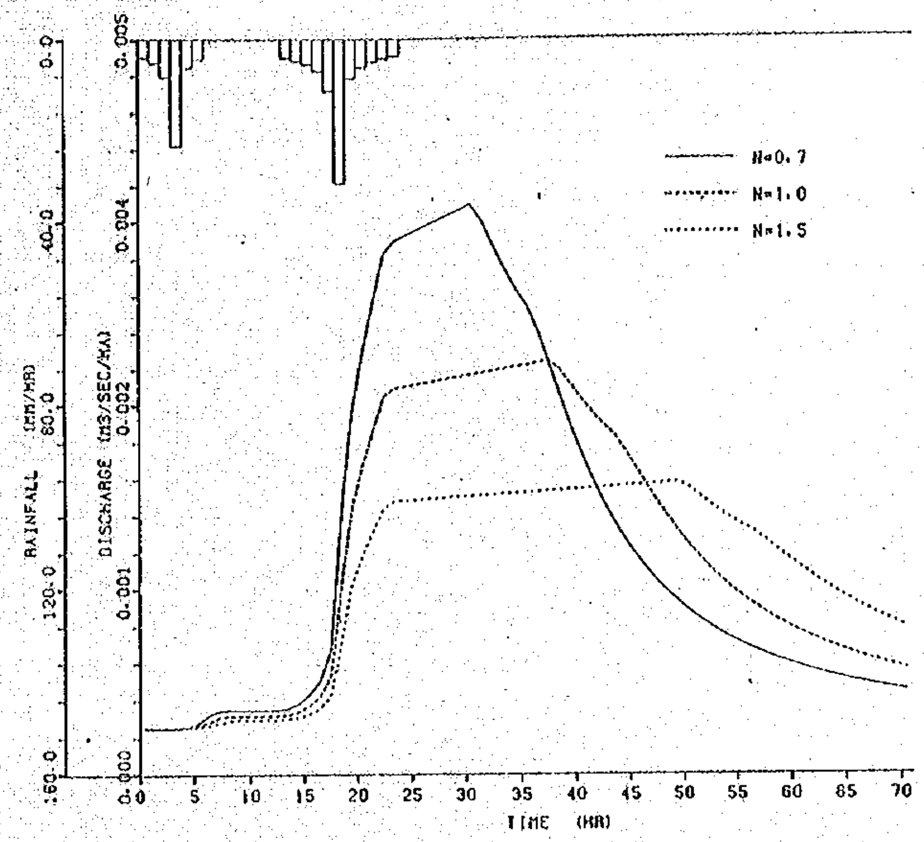


Fig.4-63 Hydrograph (T=1/10, R1max=150)

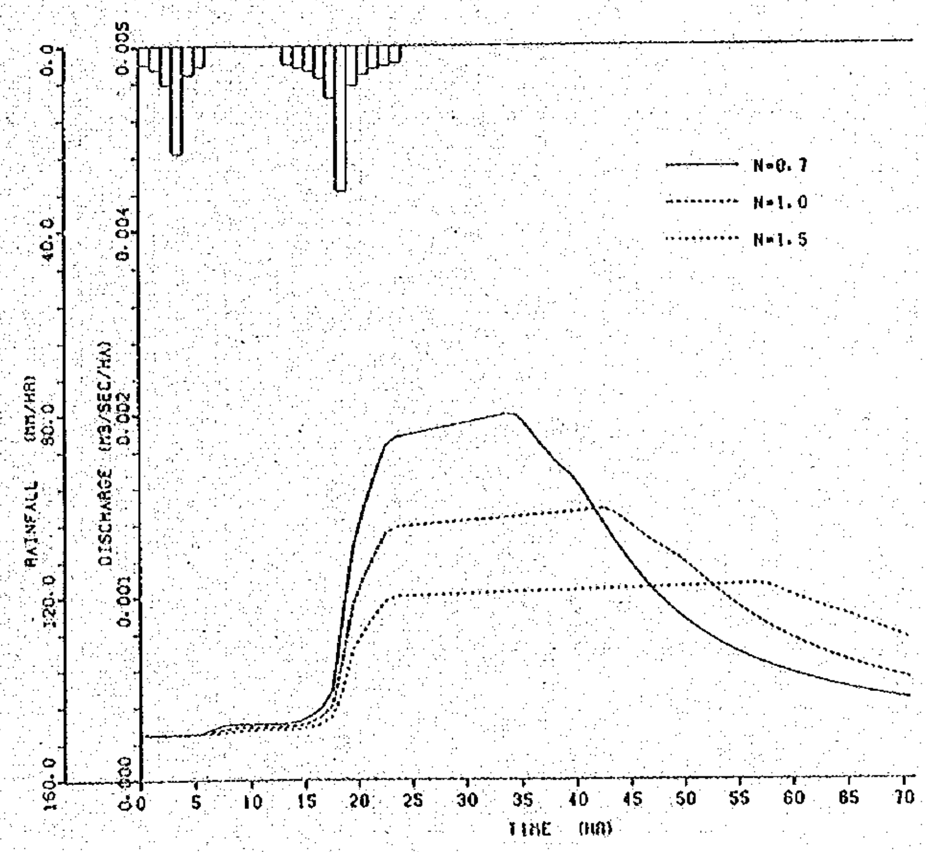


Fig.4-64 Hydrograph (T=1/10, R1max=200)

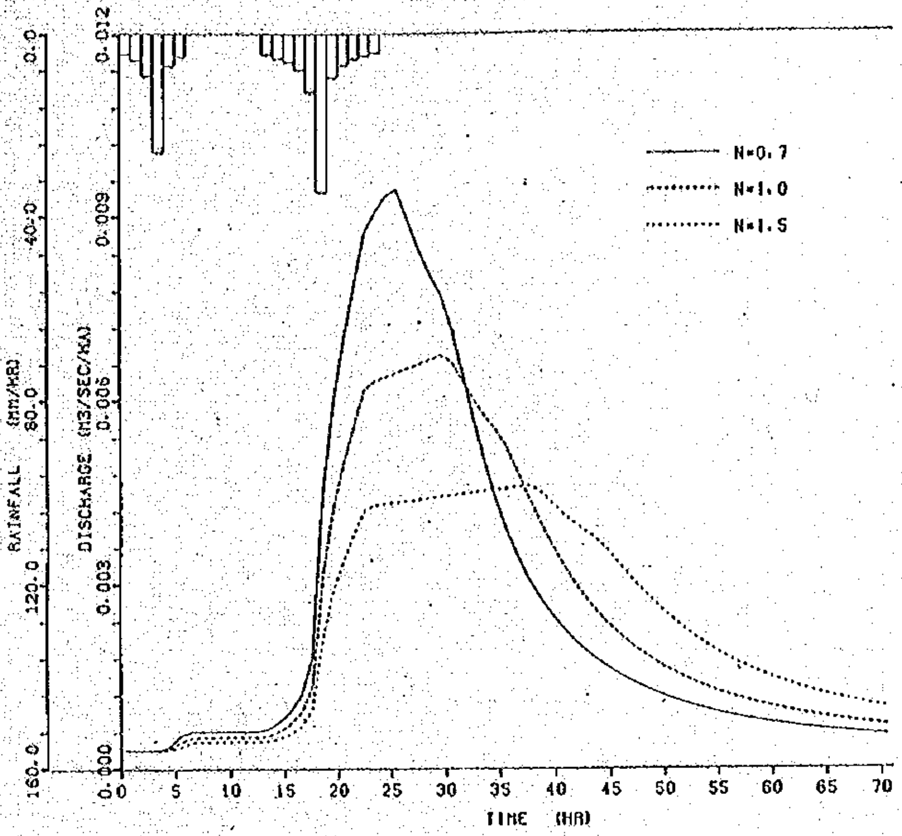


Fig.4-65 Hydrograph (T=1/20, R1max=100)

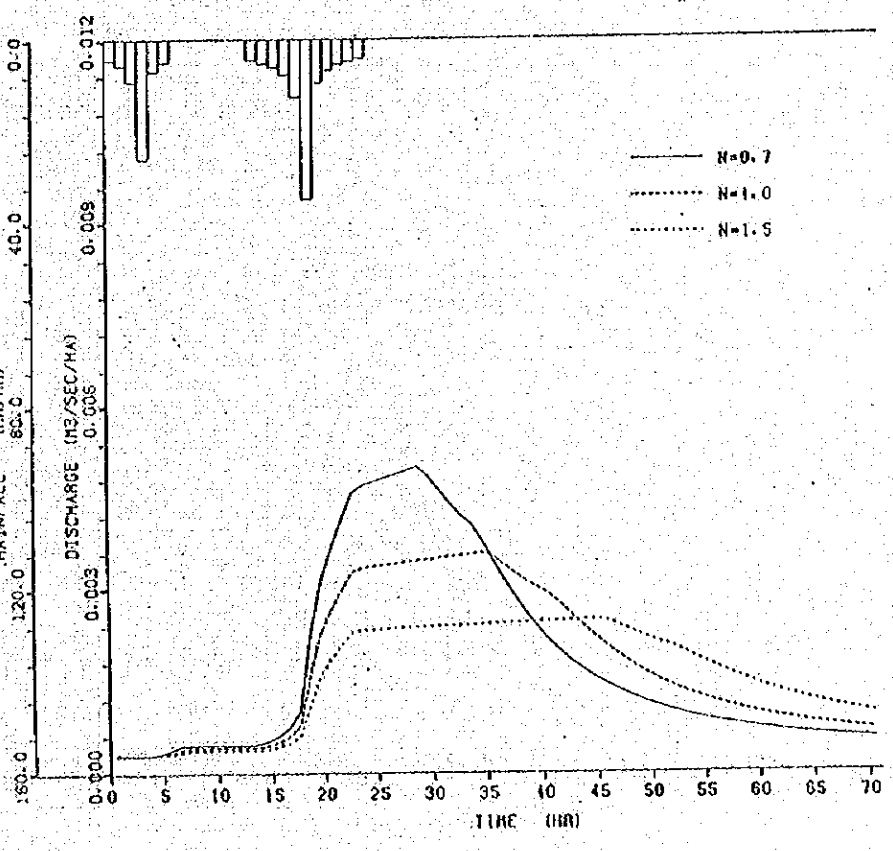


Fig.4-66 Hydrograph (T=1/20, R1max=150)

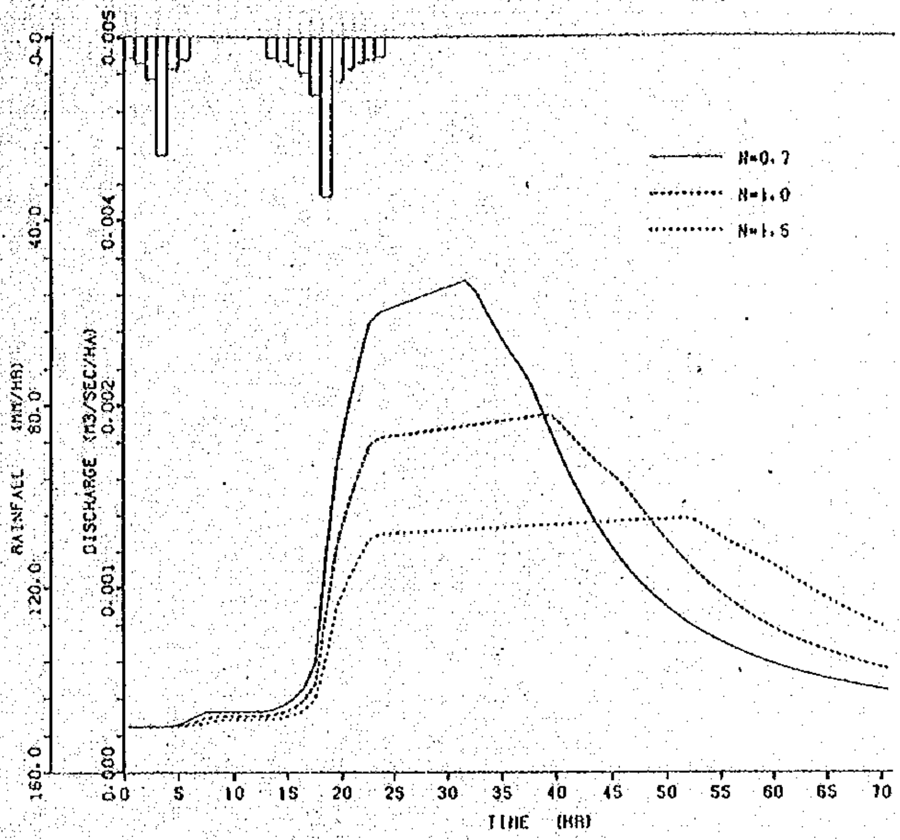


Fig.4-67 Hydrograph (T=1/20, R1max=200)

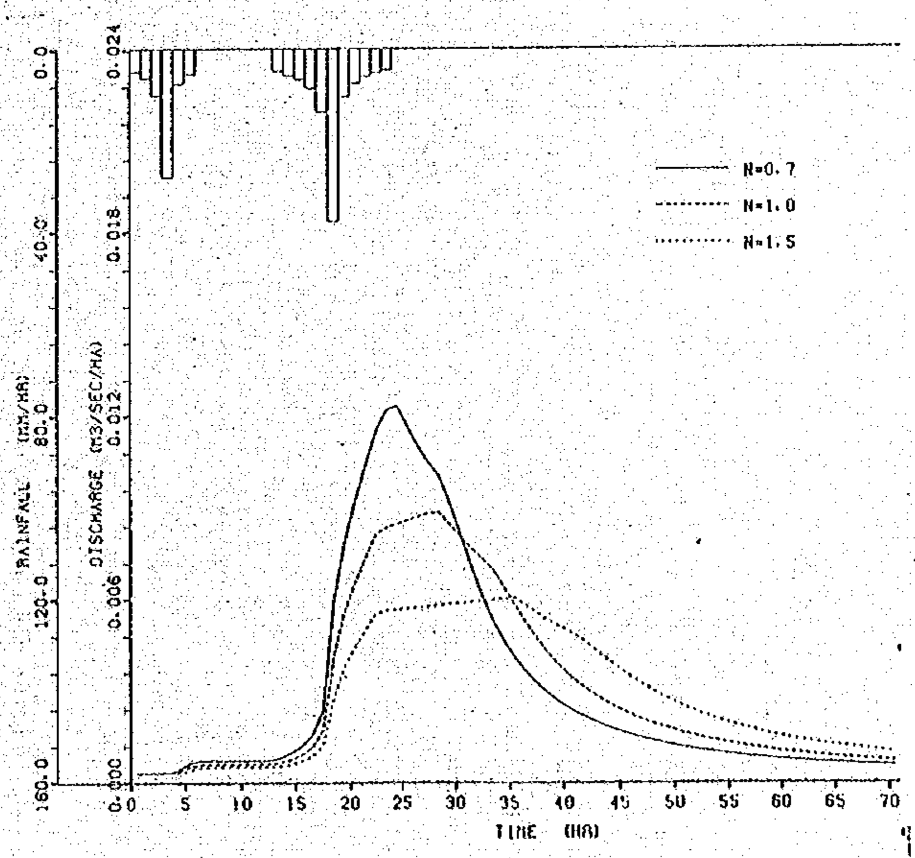


Fig.4-68 Hydrograph (T=1/40, R1max=100)

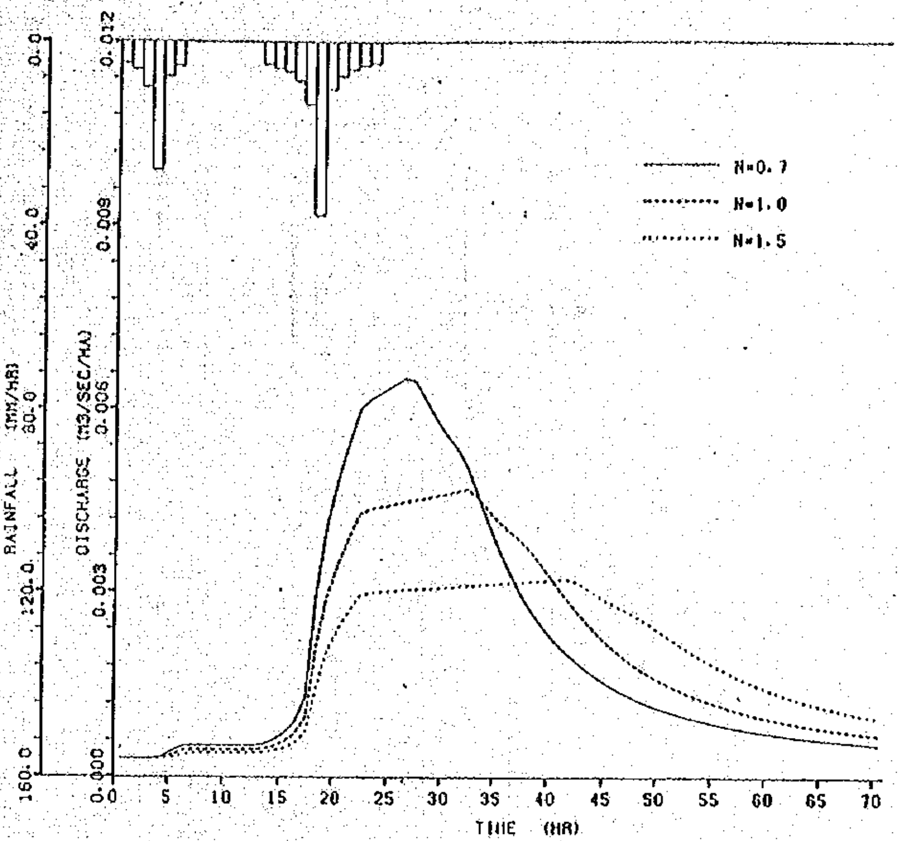


Fig.4-69 Hydrograph (T=1/40, R1max=150)

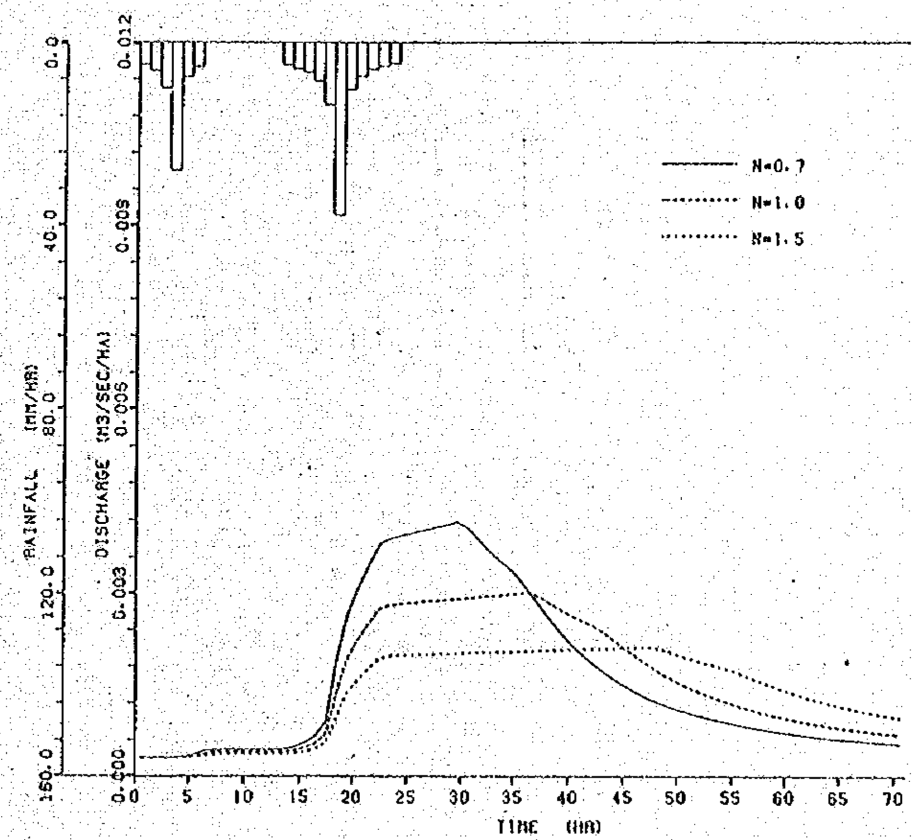


Fig.4-70 Hydrograph (T=1/40, R1max=200)

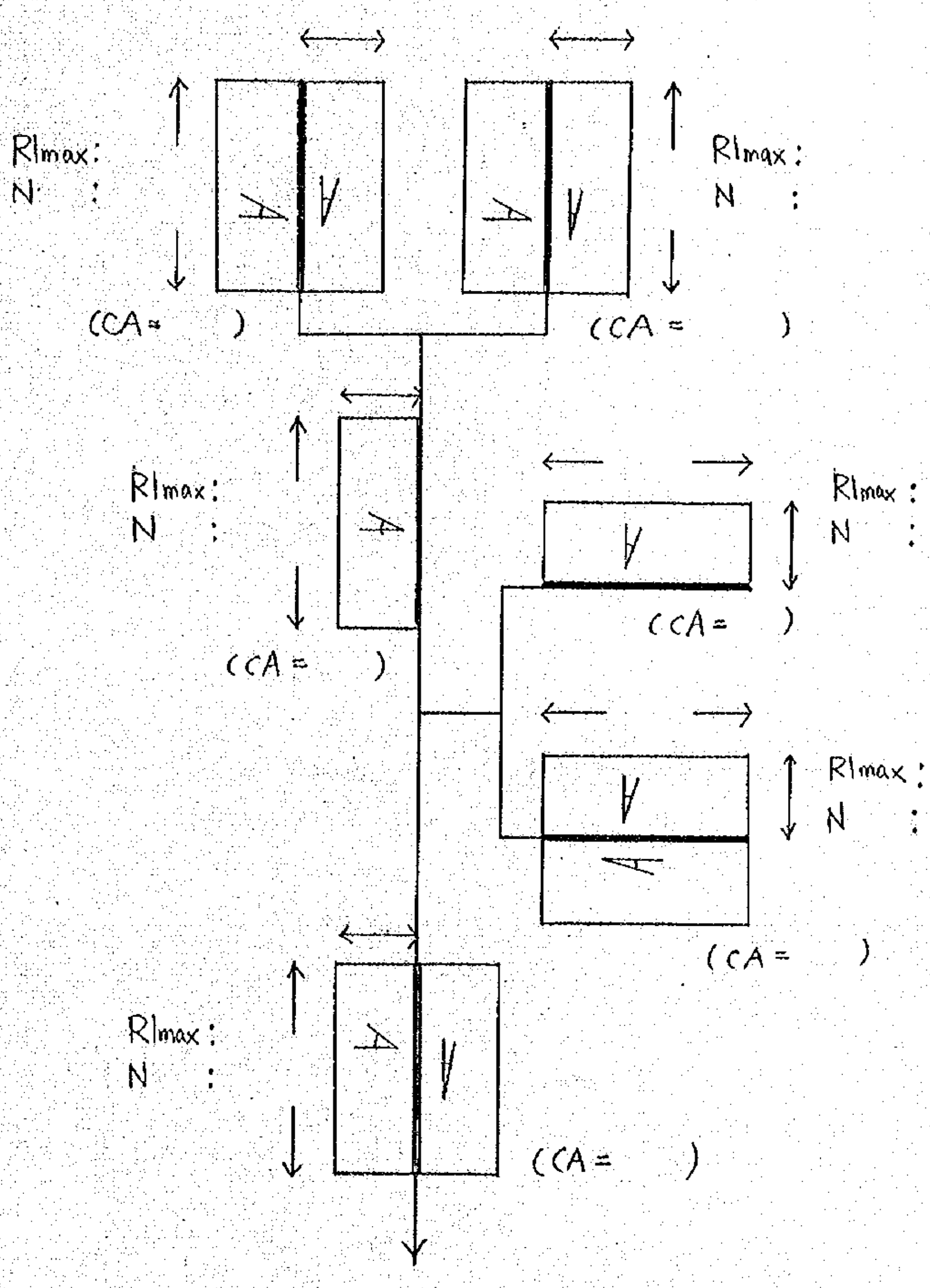
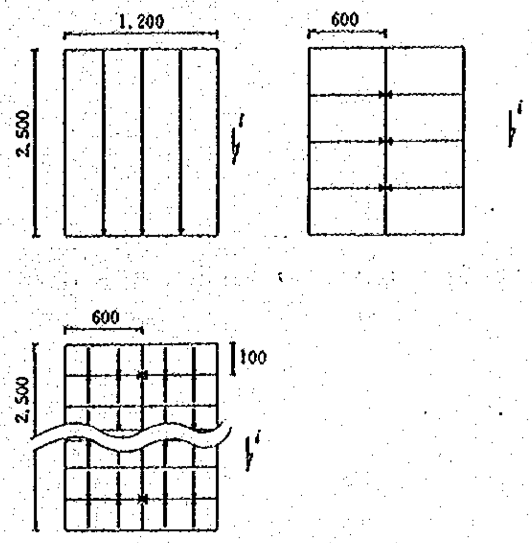


Fig.4-71 Imaginary basin model



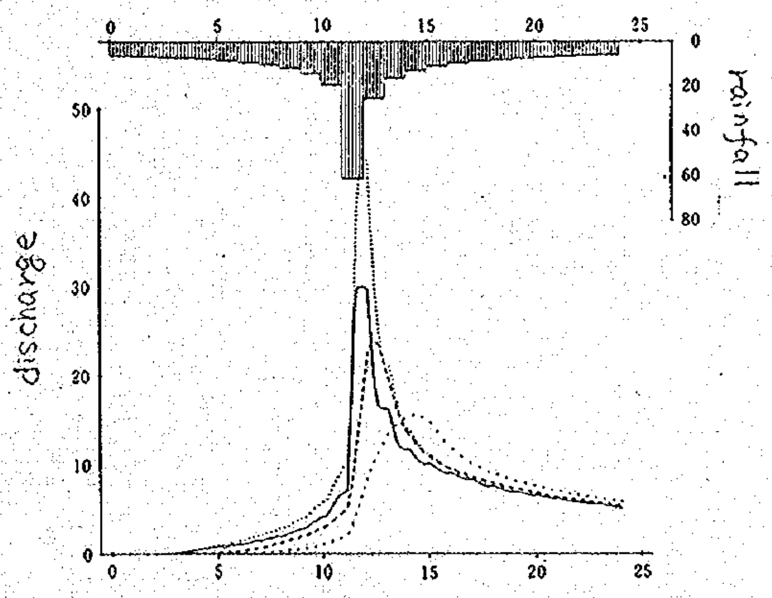


Fig.4-72 Changes of basin model and hydrograph according to the stage of development

V. Hydraulic analysis on Sg. Perai river

5-1 Mathmatical model

5-1-1 Basic equations

The hydraulic phenomenon in Sg. Perai analyzed by mean of numerical computation, mathematical simulation model. The outline of the mathmatical model are described hereafter and refer to Annex B in more detail.

The physical laws which govern an unsteady flow are the following two; that is, one is the equation of motion which controls motion of flow, an another is the equation of continuity which represents the change of water level due to the inflow and outflow.

For given initial and boundary conditions, the equations are integrated numerically with respect to time and distance by mean of mathmatical model. Hydraulic characteristics of an unsteady flow are given by simulataneous solution of both the equation and that continuity. The fundamental eqations of a one-dimensional flow with the downtream end as the origin are expressed as follws:

$$\left\{\begin{array}{ll} \frac{1}{g}\left(\frac{\partial v}{\partial t}\right) + \frac{1}{g}\frac{\partial}{\partial x}\left(\frac{v^2}{2}\right) + S + \frac{\partial h}{\partial x} + \frac{n^2|v|}{h^{4/5}}v = 0 & \text{(eq.5-1)} \\ \frac{\partial A}{\partial t} + \frac{\partial Q}{\partial x} - q = 0 & \text{(eq.5-2)} \end{array}\right.$$

Here,

- g: acceleration of gravity,
- v: velocity (positive for the upstream direction),
- s: bottom slope,
- h: water depth,
- x: distance in longitudinal directions on a horizontal datum plane,
- n: Manning's roughness coefficient,
- t: elapsed time,
- A: cross-sectional area,
- Q: discharge through a section,
- q: lateral inflow per unit length (positive for inflow, negative for outflow)

In representation of hydraulic behaviour in mathematical model, the fundamental equation eq. (5-1) and eq. 2(5-2) are converted into difference expressions. Then, for the given initial, boundary and geometric condition, the numerical integration is performed. It is desirable to constitute the efficient and economic grid system from the standpoint of computer performance. The calculation proceeds from the downstream to upstream, with distance interval Δx and time interval Δt as shown in Fig. 5-1-1, where i and n indicate distance and time, respectively.

5-1-2 Type of simulation model

Two type simulation models are prepared for this analysis. The first model is W-W model as shown in Fig. 5-1-2 which have water stage-time boundary at up and downstream end. The second is W-Q model as shown in Fig. 5-1-3 which have the water stage-time boundary at downstream end and the dischare -time boundary at upstream end.

The W-W model is used for determination of roughness coeffient of Perai river and the W-Q model is used for analysis of gate operation.

In these model water depth is calculated at even mesh and velocity is at odd mesh as shown in Fig. 5-1-4.

5-2 Basic data

5-2-1 Topographic data

The river cross section data is one of important data for river flow study. The relation between area and water depth is ploted in log-log graph as shown in Fig.5-2-1. Therefore, the cross section area in river channel may be obtain the following equation,

$$A = \alpha \cdot H^{\beta} \qquad \text{eq. (5-3)}$$

where, A:cross section area (m^2)
H:water depth (m)
α:coefficient
β:coefficient

It express the topographic condition in the mathematical model. The river cross secion surveying was carried out on upstream reach in 1987 and on downstream reach in 1988 by D.I.D. The coefficients of each section are given in Table5-2-1.

5-2-2 Water level record

The water level data is used to determine Manning's roughness coefficient, to study of the tide fluctuation and boundary condition data, and to verify the simulation model. The period of the water level record to be collected are not so long time as given in Table5-2-2. The location of the water level gauging station are shown in Fig.5-2-2.

5-2-3 Tidal regime and fluctuation at estuary (the mouth of river)

As for the tidal level outside the estuary, a tide table of Penan Island, of which summary is given in Table5-2-3 is available as a long term data. The other is the record of water level at gauging station No.11 which is adjacent to the estuary. It was observed from 11 March 1987 to 20 March 1987 by D.I.D. According to the recorded data, the water level of No.11 station have slight time lag to tide table of Penang.

Usualy tide of Penang have two peak in a day, higher and lower. The higher peak of spring tide is nearly same tide table of Penang, but the low tide have some difference. The difference between the recorded data and tide table is not so large. It is about 0.1 ~ 0.2 m as shown in Fig.5-2-3

The high tide is higher than tide table and the low tide is lower than that in neap tide as shown in Fig.5-2-4. The range of neap tide at the estuary is larger than that of tide table.

Automatic water level recorder were installed by the study team in March 1988, and water level have been observed by D.I.D.
According to this data, the higher peak is nearly same the tide table
as given in Table5-2-4.

5-2-4 Design tide
Tidal fluctuation changes every day. Since the fore condition affects the next flow situttion, the periodical tide is useful for comparing the calculated result. Then, design tide is produce for this analysis. Since the highest water level is important in inundation poblem, the higher tide is selected from tide table of Penang and arranged. The selected tide is of 31 July 1988 as shown in Fig.5-2-5, and design tide is shown in Fig.5-2-6.

Automatic water level recorder were installed by the study team in March 1988, and water level have been observed by D.I.D.
According to this data, the higher peak is nearly same the tide table
as given in Table5-2-4.

5-2-4 Design tide
Tidal fluctuation changes every day. Since the fore condition affects the next flow situttion, the periodical tide is useful for comparing the calculated result. Then, design tide is produce for this analysis. Since the highest water level is important in inundation poblem, the higher tide is selected from tide table of Penang and arranged. The selected tide is of 31 July 1988 as shown in Fig.5-2-5, and design tide is shown in Fig.5-2-6.

5-3 Roughness coefficients of Sg. Perai River and head loss of Perai Barrage

5-3-1 Roughness coefficient in simulation model

In mathematical model, hydraulic behaviour is simulated without reducing the scale of the prototype of river.
In the basic equation, however, the constant such as Manning's roughness coefficient , which is inherent to the prototype, is included. The treatment of this constant determines the similarity of the mathematical model to the prototype. The roughness coefficient is determined by comparing the calculated results with the observed hydraulic behavior.

5-3-2 Head loss of Perai Barrage

There is the Perai Barrage as a hydraulic structure in Sg.Perai. Sg. The cross section of Sg.Perai, is reduced by Perai Barrage, therefore a large head loss occures at the Barrage section. The stretch of the barrage is treated as an open channel when gate is opened and as an orifice when gate is in water as shown in Fig.5-3-1. The head loss must be considered. In the simulation model, a considered value is given for the roughness of the barrage stretch. The equivalent roughness coefficient is determined RN=0.030 from the head loss and velocity observed on 6 Feb. 1988 as given in Table5-3-1.

5-3-3 Determination of Roughness coefficient of Sg. Perai

For determination of roughness coefficient, 8 case, two groups simulations were carrid out by using W-W model which the given roughness cefficient were 0.020 to 0.035.

The first group has boundaries of No.1-A station(Mesh 34) and No.11 station(Mesh 2) and the data observed in March 1987 are used.
The second group has boundaries of Sg. Samagaga gauging station(Mesh 28) and railway bridge gauging station(Mesh 2) and the data observed by automatic recorder in 1988 are used.

The change of watel levels at Mesh 8(station No.8), Mesh 14(station No.5), Mesh 16(station No.4) and Mesh 20(Barrage)are shown in Figures 5-3-2 ~ 5-3-13. Each case simulated fluctuation is close to the observed data.

The logitudinal profile are shown in Figures 5-3-14 ~5-3-19. There are some difference between simulation result and the observed data, but as shown in Fig.5-3-20 they are as same as diffrence which occured in 10 minute time lag.

The profile, which roughness coeffient is RN=0.020, are most fit to the observed data. The roughness coefficient can be determined to be RN=0.020 by this result.

5-4 Simulation of the attempt operation on 17 May 1984

5-4-1 Simulation using tide table as estuary condition

The simulation of the operation on 17 May 1984 are carried out using W-Q model. According to the operation record (BUKU CATITAN PARAS AIR HARIAN PRAI BARRAGE), the gate operation is rougly as follows:

time	operation and remark
11:12am	tide were going up.
12:15pm	start closing of gate No.3,4,2
1:20pm	all gate closed.
2:00pm	all gate open

No.1 gte was closed by stop log.

The flow was stoped for about 40 minutes by this operation. The tide table of Penang is used as boundary condition of downstream.

In this condition, the highest water level does not reached the observed highest water level as shown in Fig.5-4-1. The peaks of tide table, there at, are compaered with the observed water level peaks at Perai Barrage from 15 to 18 May 1984. As shown in Fig.5-4-2 and Table 5-4-1, each observed peak is higher than the peak of tide table. Especially, lower peaks are more higher. The peak in spring tide of other time are shown in Figures 5-4-3 ~ 5-4-5 and Table5-4-2. The higher peak of spring tide is same order to tide table. As a result, tide of before and after that day was recognized to be in an abnormal condition. It is estimated, however, that its order is nearly same to the design tide.

5-4-2 Simulation using design tide as estuary condition

The compution result ,which the design tide is used as the boundary condition and inflow discharge is 10 cu.m/sec, is shown in Fig.5-4-6. The peak is nealy same to the observed peak water level. When inflow is large, both water level, peak and bottom, are more close. In fact, rainfall of Butterworth on 16 May 1984 recorded 54.4 mm.

5-5 Simulation for the study of gate operation

5-5-1 Computation case of gate operation

Unlimited number of gate operation could be considered. In this report the following cases of operation are studied:

case-1: condition in which all gates are closed.

case-2: the gates are operated to open when upstream water level is higher than downstream water level and upstream water level is higher than normal impounding water level(N.I.W.L.). This is to operate the gate by using the data of the barrage site only.

case-3: case in which the gates are operated on the same judgement as case-2, and gate are closed when tide flow rising and estuary water level is higher than R.L. 0.0 m on tide table. The flow chart of operation is shown in Fig.5-5-1.

case-4: case in which the gates are operated on the same judgement as case-2, and gate are started to open when water level at downstream of the Barrage(Mesh 20) become over R.L. 1.0 m.

case-5: case in which gate are operated as case 3, but one of gates is allways full-open.

These case-4 and case-5 are some compromised operation to maintain the water level lower condition and to prevent the inoundation, but this allows the ingress saline water. each case control water level to N.I.W.L. 0.61 m with 0.1 m allowance.

5-5-2 Case-1

The peak water level at downstream of the Barrage(Mesh 20) is about 1.55 m as compared with a peak of design tide 1.48 m. (Fig.5-5-2). The peak water level at down stream is higher than the peak of estuary tide and the range of fluctuation is larger than the range at estuary. The hydraulic phenomenon as this called seiche. The water level fluctuations of Mesh 2(estuary), Msh 20 and Mesh 22 are shown in Fig.5-5-3.

5-5-3 Case-2 (series H)
This operation controls easily the upstream water level of the Barrage to N.I.W.L. comparing with up and downstream water level., the gate closing operation is late aginst the rise up tide flow because it takes time to close. As the result, saline water flows into upstream passing through one or two gate as shown in Fig.5-5-4. The water level fluctuations of Mesh 2(estuary), Msh 20 and Mesh 22 are shown in Fig.5-5-4, and discharge at gate and opening heigh of gates are shown in Fig.5-5-5 andFig.5-5-6.

5-5-4 Case-3 (series F)
This operation prevents the ingress of rising flow, except the case which discharge is large. A high water level is not higher than that of case-2. The lowest water level at upstream of the barrage becomes 0.2 m lower than N.I.W.L.(2 ft= 0.61m), because gate closing is late. It takes about 20 minutes to close a gate. If all gate are opened, it takes aout 80 min.

As the result of diminising of water level by the delay, water stage remain at lower level while gate are closed. When the upstream inflow is much, over release is effective.

The water level fluctuations of Mesh 2(estuary), Mesh 20 and Mesh 22 are shown in Fig.5-5-7, and discharge at gate and opening height of gates are shown in Fig.5-5-8 andFig.5-5-9.

5-5-5 Case-4 and case-5
This operation is compromised operation in which the Perai Barrage gates are operatd to prevent the rising of tide flow, taking into consideration to prevent inundation downstream low laying area under the existing condition.

In case-4, the water level at downstream of the Barrage remain 1.4 m but upstream rise rapidly to about 1.2 m as shown Fig.5-5-10 and Fig.5-5-11. The discharge at gate and opening height of gates are shown in Fig.5-5-12 and Fig.5-5-13.

In case-5, the water level fluctuations of Mesh 2(estuary), Mesh 20 and Mesh 22 are shown in Fig.5-5-14, and discharge at gate and opening heigh of gates are shown in Fig.5-5-15 and Fig.5-5-16.

These operation does not satisfy both function, to prevent inundation and to keep upstream water level lower. It is dificult to operate the gate without inundation and poor drainage condition when tide is high.

5-5-6 flood routing

The operation in flood condition is calculated as shown in Fig.5-5-17~5-5-19. The operation procedure is case-3, flood has the peak discharge of about 550 cu.m/sec which is analysed in chapter IV, and its parameters are N=0.7, R1max=100mm, return period T=1/40.

In this case, the peak water level near the barrage is sligthly lower than the peak of tide and that of upstream reach near the confluence is over 1.7 m.

5-6 Change of flow condition the down-stream reach by Prai Barrge gate operation

There is the port in the estuary. The change of sedimentation condition is one of important problems. Gate operation may affect the flow condition of the down stream reach. Shear velocity $u_* = \sqrt{\tau / \rho}$ is one of factors on sedimentation.

In data of Feasibility Study report on Drainage and Reclamation Sg. Prai Basin in Malaysia, over 90 % of bed material have less 1 mm diameter on downstream reach. According to Iwagaki formula, critical shear velocity for 1 mm diameter sediment is as follow:

$$u_{*c}^2 = 55d$$

where, u_* :critical shear velocity (cm/sec)
d : diameter of sediment (cm)
$0.0565 < d < 0.118$

The critical shear velocity $u_{*c} = 2.3$ cm/sec is calculated by ths formula.

Shear velocity of downstream reach are shown in Fig.5-6-1~5-6-3 with gate operatin case-3, and Fig.5-6-4~5-6-6 with all gates full open , inflow of upstream is 10 cu.m/sec.

Maximum shear velocity of gate full open condtion is larger than u_{*c}, and that of gate operation condition is larger, too. As a result, sediment on downstream reach may be move by tidal flow.

The shear velocty with operation becomes smaller than gate full open condition. Because of no data on sedimentation supply, it is impossible to say that sedimentation becomes much or not by gate operation. Since shear velocit is larger than critical shear, sediment could be still moved by tidal flow.

5-7 Conclusion on simulation

The plofile of water surface of Sg. Perai without gate operation at high tide and low tide are illustrated as shown in Fig.5-7-1. The estuary water level is higher than the ground level downstream near the Barrage. When tide is higher than the ground level and gates are closed, the downstream area will be inundated.

Table 5-2-1 Coeffient of cross section

Mesh NO.	NO. in Fig.	distance	bottom height	α	β	FIGURE NO.
2	0	0	-8.01	0.63	3.28	CH 0
4	1	1000	-5.49	28.51	1.86	CH 1000
6	2	2000	-5.05	58.54	1.29	CH 2000
8	3	3000	-7.60	5.88	2.29	CH 3000
10	4	4000	-4.11	47.35	1.62	CH 4000
12	5	5000	-3.80	27.13	2.00	CH 5000
14	6	6000	-3.76	52.55	1.53	CH 6000
16	7	7000	-5.05	29.32	1.61	CH 7000
18	8	8000	-4.52	38.33	1.56	CH 8000
20			-4.57	73.40	1.04	
--	Barrage	8322				CH 8322
22			-3.20	91.50	1.04	
24	9	9000	-5.02	21.04	1.74	CH 600
26	10	10000	-10.07	8.96	1.79	CH 1600
28	11	11000	-6.02	22.86	1.56	CH 2600
30	12	12000	-5.33	15.72	1.78	CH 3600
32	13	13000	-4.05	49.06	1.26	CH 4600
34	14	14000	-5.07	10.91	1.83	CH 1000
36	15	15000	-4.30	27.62	1.44	CH 2000
38	16	16000	-4.00	26.91	1.43	CH 3000
40	17	17000	-3.76	23.08	1.74	CH 300
42	18	18000	-6.06	8.89	1.68	CH 1300
44	19	19000	-6.22	6.17	1.84	CH 2300
46	20	20000	-3.43	17.72	1.59	CH 3300
48	21	21000	-5.16	7.18	1.89	CH 4300
50	22	22000	-4.14	5.90	1.92	CH 5300
52	23	23000	-4.63	3.40	2.27	CH 6300
54	24	24000	-3.97	2.54	2.40	CH 7300

Table 5-2-2 Period of Water Level observation

gauging station	Mesh	1987 11 Mar. ~ 20 Mar.	1988 9 Mar. ~ 12 Sep.	1987 5 Feb. ~ 6Feb.
1 − A	34		xx	
3	22	xx		
3 A	20			
4	16		xx	xx
5	14			xx
8	8			xx
11	2		xx	xx
Railway Bridge	2	xx		XX
Sunyai Samagaga	28	xx		xx

xx : NO observed

Main Dimensions of Tidal Level

Mean Range of Spring Tide = 2 (Hm + Hs)
= 5.920 (ft)   1.804 m

Mean Range of Neap Tide = 2 (Hm - Hs)
= 1.532 (ft)

The mean Tidal Range = 2 Hm
= 2.726 (ft)

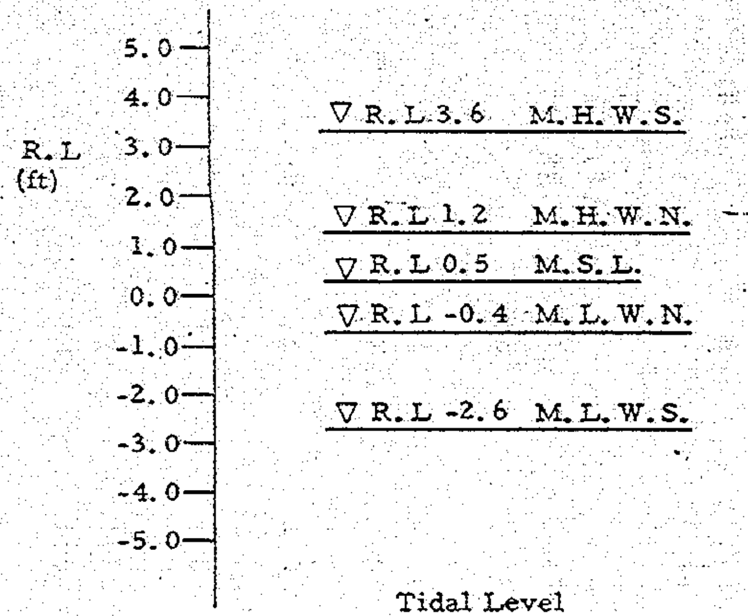
The mean duration between two successive high tides
= Km/28.98 = 12.198 (hr)

Harmonic Constants of Main Constituents

Item	H (ft)	K (deg)
M_2	1.863	353.5
S_2	1.097	034.7
K_1	0.500	339.8
O_1	0.195	262.2
P_1	0.252	352.3
N_2	0.346	355.4
K_2	0.314	030.4
Sa	0.300	127.0
Ssa	0.160	131.4
Mm	0.062	300.4
Mf	0.055	003.3

Table 5-2-3

Tidal Level



Tidal Level

Notes:

1. Tidal Table : Liverpool Observatory and Tidal Institute, Hydrographic Department, Admiralty, London
2. Duration of Harmonic Analysis:
   365 days from 1953, 7.1 to 1954, 6.30
3. Location:
   Lat. 5°25'N, Longit. 100°21'E

Tidal Characteristics in Penang Island

Table 5-2-4 Peak water Level at st.NO 11, NO 8 and tide table in April and May 1988

Date	Time	Station NO 11	Station NO 8	tide table
April/88				
26	11	0.47	0.55	0.58
	12	0.40	0.48	0.58
27	11	0.60	0.66	-
	12	0.59	0.66	0.68
28	12	0.85	0.83	0.88
29	12	1.02	0.99	0.98
30	12	1.03	1.05	1.08
	13	1.07	1.06	1.08
May				
1	12	1.06	1.15	1.08
	13	1.12	1.14	1.18
	14	1.05	1.02	1.08
9	18	0.53	0.65	0.68
	19	0.52	0.65	0.68
10	20	0.47	0.65	0.68
	21	0.50	0.65	
13	11	0.92	1.04	1.08
	12	1.00	1.09	1.08

Table5-3-1 Equivalent Roughness coeffient of Barrage

Time	6. Feb PM 2:00	PM12:50
observed velocity	1.0 m/sec	0.67 m
upstream water level	0.640 m	0.671 m
down Stream water level	0.701 m	0.700 m
Δh	-0.061 m	-0.029 m
bottom elevation	-3.2 m	-3.2 m
mean water depth	3.87 m	3.89 m
equivalent roughness coeffient n	1*1.82*0.017 =0.0309	1.5*1.83*0.012 =0.033

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

$$= \frac{1}{V}\left(\frac{B*H}{2*H+B}\right)^{2/3}\left(\frac{\Delta h}{\Delta x}\right)^{1/2}$$

Δx=200m

Table 5-4-2 Comparison with Peak WEL observed at Barrage and Tide Table on 15～18 May 1984.

DATE	Time	observed WEL at Barrage (ft)	(m)	Time	Tide Table (m)	R.L.(m)
May						
15	3:00pm	4.80	1.463	14:00	2.8	1.38
	9:00pm	-2.00	-0.610	20:00	0.6	-0.82
16	2:00am	4.30	1.311	01:00	2.3	0.88
	9:00am	-2.00	-0.610	07:00	0.8	-0.62
	2:00pm	5.00	1.524	14:00	2.8	1.38
	9:00pm	-2.20	-0.671	20:00	0.6	-0.82
17	2:00am	4.50	1.372	02:00	2.3	0.88
	11:00am	-1.8	-0.549	08:00	0.8	-0.62
	1:20pm	5.4	1.646	14:00	2.7	1.28
	10:00pm	-2.0	-0.610	21:00	0.6	-0.82
18	2:30am	4.50	1.372	02:00	2.2	0.78
	11:00am	-1.40	-0.427	09:00	0.9	-0.52
	4:00pm	4.3	1.311	14:00	2.6	1.18

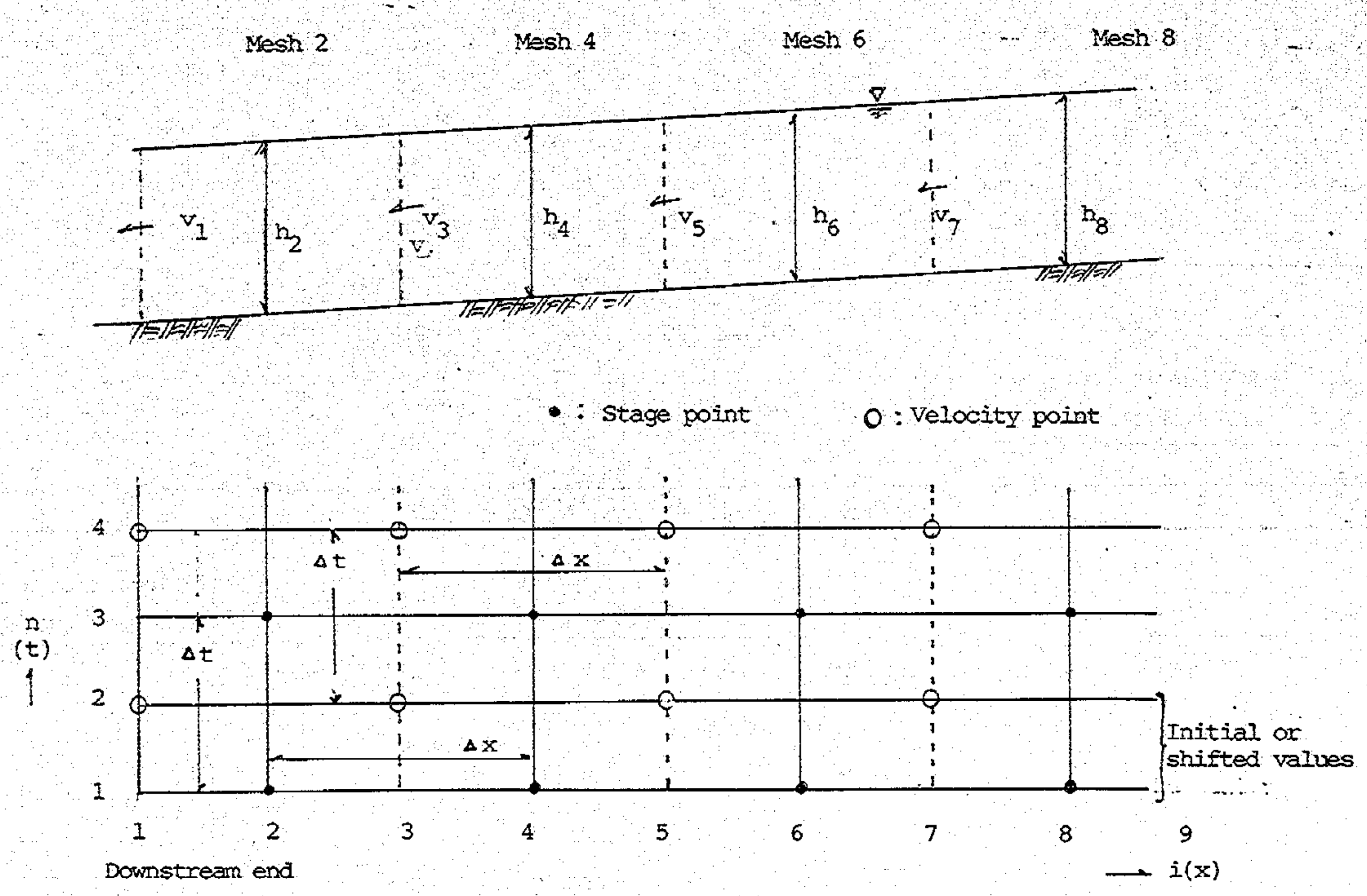


Fig.5-1-1 Operational grid system

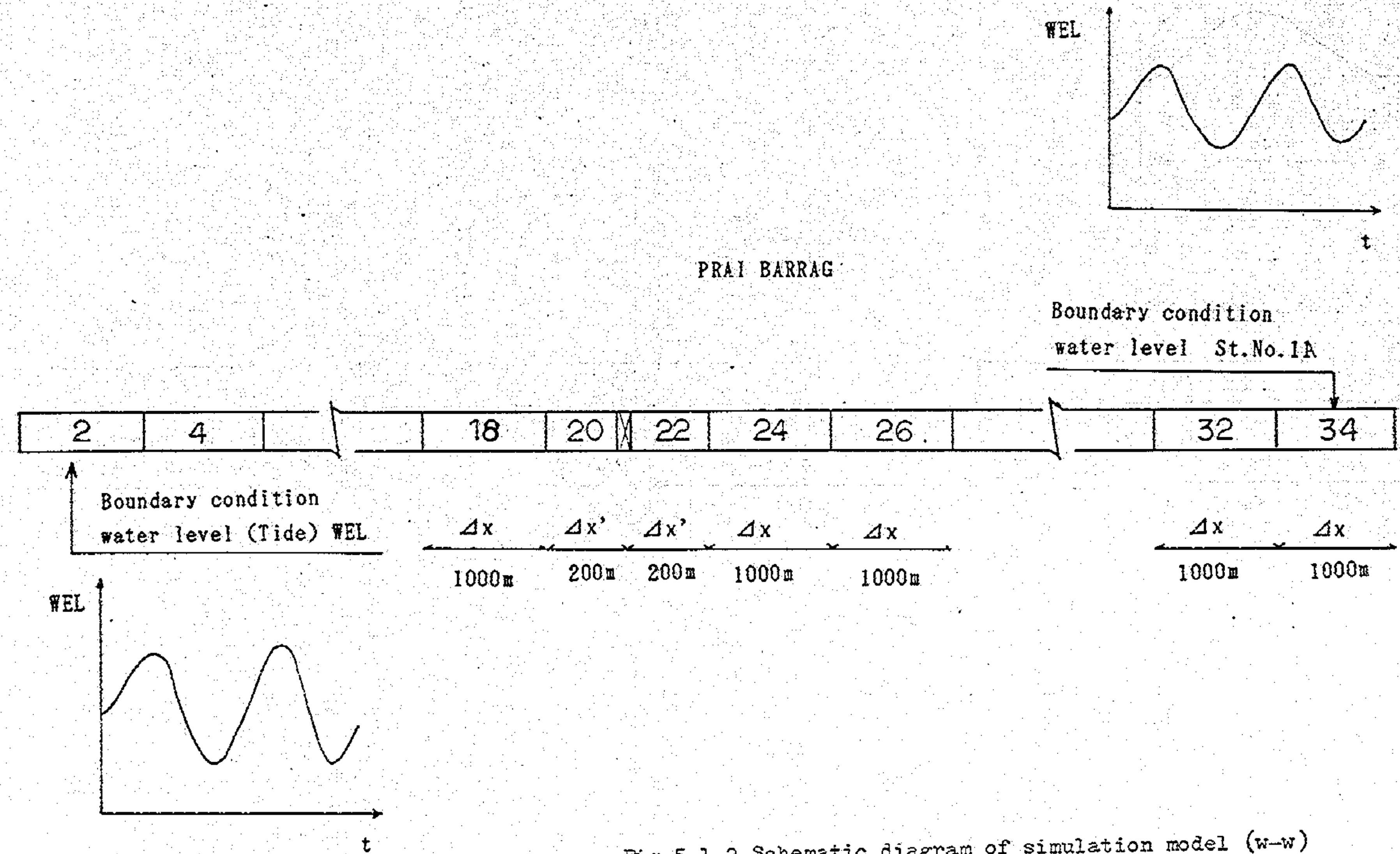


Fig.5-1-2 Schematic diagram of simulation model (w-w)