REPUBLIC OF INDONESIA

REPORT
ON
BRANTAS RIVER BASIN DEVELOPMENT

(TECHNICAL STUDIES)

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PREFACE

This report presents the technical background of the "Main Report". Each chapter contains the description of the findings in a certain field.

An effort was made in preparing this report to extrapolate the available data to the range where the data is lacking but necessary for the investigation. In the same time the limitation of the results of analysis was defined for each study.

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1. GENERAL DESCRIPTIONS

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1. GENERAL DESCRIPTIONS

1-1 Indonesia

Indonesia consists of more than 3,000 islands. Main islands are Java, Sumatra, Kalimantan and West Irian. The total area is 1.9 million square kilometers. The population of Indonesia is about 120 million in 1970 and the annual population growth during 1965-1970 period is 2.5 percent.

Economy of Indonesia during the 1968-1970 period has been favorably progressed. The annual rate of growth in GDP (Gross Domestic Product) is about seven percent in 1960 prices during the 1968-1969 period, compared with an annual average growth rate of two percent during the 1961-1967 period. Notwithstanding of substantial growth in recent year, the per capita GDP was 56 dollars/1 in 1969 which is in the lowest side among the developing countries in Asia.

According to statistics, the agricultural and the industrial productions and their growth rates during the 1969-1970 fiscal years are as shown in Table 1-1. The table shows that both agricultural and industrial productions growth were generally satisfactory.

Indonesian economy is characterized by the export of primary products and the import of consumable goods as well as capital goods. According to the "Statistics of Indonesia", the export mainly comprising of that for oil sector increased by 20 percent in 1970/71. However, the import expanded at higher rate than export. As the consequence, the overall trade deficit has increased by six percent in 1970/71.

The Five-Year Plan started from 1969 emphasizes the rice production for domestic consumption. The Government had disbursed about 25 percent of the total government development budget for agricultural and irrigation development. The plan aims at increasing the annual rice production from 10.5 million tons in 1968/69 to 15.4 million tons in 1973/74.

/1 Exchange rate Rp. 400 per one US dollar is used.

The actual rice production was 10.8 million tons in 1969 and 11.6 million tons in 1970, which are almost on target level. However, the rice of about 600,000 tons in 1969 and 800,000 tons in 1970 had to be imported. Table 1-2 shows the milled rice consumption during the 1958-1970 period. This table shows that per capita consumption of rice was about 96 kilograms in 1970 which is still low level compared with other rice-consuming countries.

1-2 Java

Java lies between $5^{o}52'$ and $8^{o}47'$ south latitude, and $105^{o}13'$ and $114^{o}37'$ east longitude. It is separated from Sumatra on the west by the Sunda Strait, 20 to 80 kilometers wide and from Bali on the east by Bali Strait, 2.5 kilometers wide. Java is 970 kilometers long and 203 kilometers wide at the maximum dimensions and its area including Madura island is about 132,000 square kilometers.

Population of Java is estimated to be 80 million in 1971 which constitutes about 65 percent of the country's population. The population density is around 600 persons per square kilometer.

Geology of Java is characterized by eolian and sedimentary formation of the Neogene Tertiary and andestic and basaltic rocks of volcanic origin except a small pre-tertiary area in the West Java. In the Tertiary formation, there are two belts of the elevated coral reef formation. One of these belts is extending along the south coast of the island. Some of old coral limestone are found at an elevation of 800 meters above sea level.

The average air temperature in Java is 25.5^{o} to $26.5^{o}C$. The relative humidity is high through the island, ranging from 65 percent to 90 percent.

The paddy field in Java is estimated to be 4.3 million hectares in 1971. This is 52 percent of the total paddy field in the country. The production of the milled rice in Java is estimated to be 6.7 million tons in 1970. However, the food deficit is more conspicuous in Java than the other region. The area of Java is only ten percent of the country's area but it supports about 65 percent of the country's population. The island is far beyond of the food self-sufficiency. It has no expansion of cultivated land for long period of time and experienced a decrease of the average farm size year by year. The living standard of the rural people is relatively low.

1-3 Brantas River Basin

The Brantas River Basin lies between $110^{\circ}30'$ and $112^{\circ}55'$ east longitude and $7^{\circ}10'$ and $8^{\circ}15'$ south latitude. The basin is bounded by Mt. Sumeru in the east and Mt. Wilis in the west. A series of low hills, 300 to 500 meters high, separates the River Basin from the Indian Ocean on the south. Many tributaries originating in the volcanoes run down the alluvial cones and flow into main stream of the Brantas River with much sand eroded. The catchment area of the Brantas River Basin is about 12,000 square kilometers.

The Brantas River, originating in the southeastern slope of the Ardjuno volcano, flows approximately to the south. Striking the southern mountains of the oldest geological unit in Java, it makes a bend and turns to the right. The Brantas River turns to the north at the middle of this stretch. There the Ngrowo River joins from the south. After joining with the Widas River which draines the northeastern slope of Mt. Wilis, the Brantas River turns northward and finally empties itself into Java Sea through the Porong and the Surabaja Rivers, branching off near Modjokerto city. Total length of the Brantas River is approximately 320 kilometers.

Average air temperature in the Brantas River Basin is not extremely high ranging from 22.5° to $24.5^{\circ}C$. The relative humidity is high throughout the year, ranging from 74 percent to 87 percent. The west monsoon prevails from November through April, bringing much rain and causes much rainfall concentrically at higher mountainous area. The east monsoon prevails from May to October bringing some rain. Average annual rainfall in the basin amounts to about 2,000 millimeters.

The population of the basin is estimated to be about ten million in 1971 which constitute about 13 percent of the population of Java or eight percent of the country's total.

Table 1-3 shows the total population, the population density in 1971 and annual population growth during the 1961-1971 period in the basin. The population of the basin comprises about eight million in the rural area and two million in the urban area. More than 60 percent of the urban population is occupied by the peoples lived in Surabaja city. The estimated average population density is 847 persons per square kilometer and the average population growth is 1.81 percent. Detailed calculation of the population estimate is compiled in "Data Book". Figure 1-1 shows the distribution of the population density in the basin. The figure shows that the population concentrates mainly in Malang, Modjokerto and Surabaja cities which are the commercial centers of the basin.

Approximately 70 percent of the population of the basin is engaged in agriculture and the remaining 30 percent belongs to industry and the service business.

Total farmland in the basin is estimated to be about 730,000 hectares which is about 60 percent of the basin area. The irrigation area is about 300,000 hectares. The area irrigated by the water of the Brantas River is approximately 77,000 hectares.

Agricultural product in the basin comprises mainly rice, sugar cane, soybean and maize. The total production of the milled rice in the basin is estimated to be 600,000 tons in 1971 equivalent to about ten percent of the production in Java or five percent in the country's total. The basin produces about 230,000 tons of sugar annually. It is about 30 percent of the total production in the country.

Surabaja, capital of East Java Province is located along the Surabaja River. It is an important trade and industrial center with a well-equipped harbor which is the second-largest sea port next to Tandjung Priok in Djakarta. The Surabaja harbor has prospered as a center of inter-insular trade between Java, Kalimantan, Sulawesi and other islands, as well as the foreign trade between many countries.

1-4 Two Large Dams in the Brantas River Basin

The Soeloredjo and Karangkates Dam Projects have been constructed in the Brantas River Basin. The principal features of these dams are listed in Tables 1-4 and 1-5.

The Soeloredjo dam is located in the upper reach of the Konto River. The construction works have started in 1964 and completed in 1970. The power station is under construction and its operation will be started in 1973.

There are run-of-river power stations forming a series of cascades immediately below the Soeloredjo damsite. They are the Mendalan (23 MW) and Siman (10.8 MW) power stations. The lower end of the cascades is connected with an irrigation canal. There are about 23,000 hectares of farmland which depend on the water in the Konto River. The discharge regulated by the Soeloredjo reservoir generates 4.8 Mega Watt of power at the dam and increases the energy output at the two power stations. After that, it is conveyed to the irrigation areas.

The Karangkates dam is located in the upper reach of the Brantas River about one kilometer upstream of the Pohgadjih gauge. The principal purposes are the control of flood discharge which is high in the upper stretch, increase of the discharge for irrigation in the dry season and power generation.

The Karangkates dam project has a second stage development plan. That is the construction of a dam in the Lahor River which joins the Brantas River just downstream of the Karangkates damsite and the construction of an afterbay dam at about 30 kilometers downstream of the Karangkates dam. The regulated flow in the Lahor reservoir will be diverted into the Karangkates Reservoir.

Table 1-1 Agricultural and Industrial Production

Item	1969	1970	Growth rate (%)
Milled rice (million tons)	10.8	12.0	11
Rubber (thousand tons)	785	812	3.4
Palm oil (thousand tons)	189	214	13.2
Crude oil (million barrels)	271	312	15
Textiles (million meters)	471	594	26

Table 1-2 Milled Rice Consumption

Year	Gross production (10^6 tons)	Net production/1 (10^6 tons)	Imports (10^6 tons)	Total available for consumption (10^6 tons)	Population (10^6)	Per capita consumption (kg)
1958	7.98	7.50	0.71	8.21	91.1	90.1
1959	8.29	7.79	0.61	8.40	93.2	90.1
1960	8.76	8.23	0.89	9.12	95.3	95.7
1961	8.27	7.77	1.01	8.78	97.4	90.1
1962	8.89	8.36	1.01	9.37	99.5	94.2
1963	7.93	7.45	1.07	8.52	101.7	83.8
1964	8.42	7.91	1.02	8.93	104.0	85.9
1965	8.84	8.31	0.14	8.45	106.7	79.2
1966	9.14	8.59	0.24	8.83	109.4	80.7
1967	9.05	8.51	0.35	8.86	112.2	79.5
1968	10.17	9.56	0.63	10.19	115.1	83.8
1969	10.80	10.15	0.60	10.65	118.1	93.1
1970	11.59	10.89	0.76	11.65	121.1	96.2

/1 Estimate at 94 percent of gross production

Source: Central Bureau of Statistics, Ministry of Agriculture

Table 1-3 Total Population, Population Density and Annual Rate of Growth in Population in Brantas River Basin

	Area (km^2)	Population (1,000 persons)	Population density ($persons/km^2$)	Annual increase ratio (1961 - 1971) (%)
1) Urban area				
Surabaja	192	1,410	7,352	3.01
Modjokerto	7	60	8,271	1.49
Kediri	63	180	2,846	1.27
Blitar	16	68	4,158	0.74
Malang	69	415	6,010	1.97
Sub-total/mean	347	2,133	6,148	2.52
2) Rural area				
Surabaja	205	121	593	1.78
Sidoardjo	592	668	1,129	2.12
Modjokerto	761	506	665	1.87
Djombang	1,160	812	700	1.70
Madiun	176	58	332	1.35
Kediri	963	1,082	1,123	1.66
Ngandjuk	13	775	655	1.37
Blitar	1,183	903	595	1.24
T'agung	1,518	722	736	1.19
Trenggalek	981	495	548	1.74
Malang	904	1,491	515	1.76
Pasuruan	2,775	261	991	1.76
Sub-total/mean	11,494	7,894	687	1.63
Total	11,841	10,027	847	1.81

Source: Population Census, (Sep. 25, 1971)

Table 1-4 Soeloredjo Dam Project

Catchment area	236 km^2
Annual inflow	320 x $10^6 m^3$
Reservoir storage capacity :	
Gross	62.3 x $10^6 m^3$
Active	54.6 x $10^6 m^3$
Reservoir high water level	EL. 622.0 m
Reservoir surcharge	0.6 m
Reservoir drawdown	24.0 m
Reservoir surface area	4 km^2
Dam height (max)	46 m
Crest length of dam	410 m
Dam type	Zone-fill
Dam volume	2.0 x $10^6 m^3$
Power installed capacity	1 x 4.8 MW
Rated water head	37.1 m
Annual energy output	23 GWH

Table 1-5 Karangkates Dam Project

Catchment area	2,050 km^2
Annual inflow	2,130 x $10^6 m^3$
Reservoir storage capacity :	
Gross	343 x $10^6 m^3$
Active	253 x $10^6 m^3$
Reservoir high water level	EL. 272.5 m
Reservoir surcharge	4.5 m
Reservoir drawdown	26.5 m
Reservoir surface area	15 km^2
Dam height (max)	100 m
Crest length of dam	750 m
Dam type	Rockfill
Dam volume	6.3 x $10^6 m^3$
Power installed capacity	2 x 35 MW
Rated water head	78.0 m
Annual energy output	417 GWH

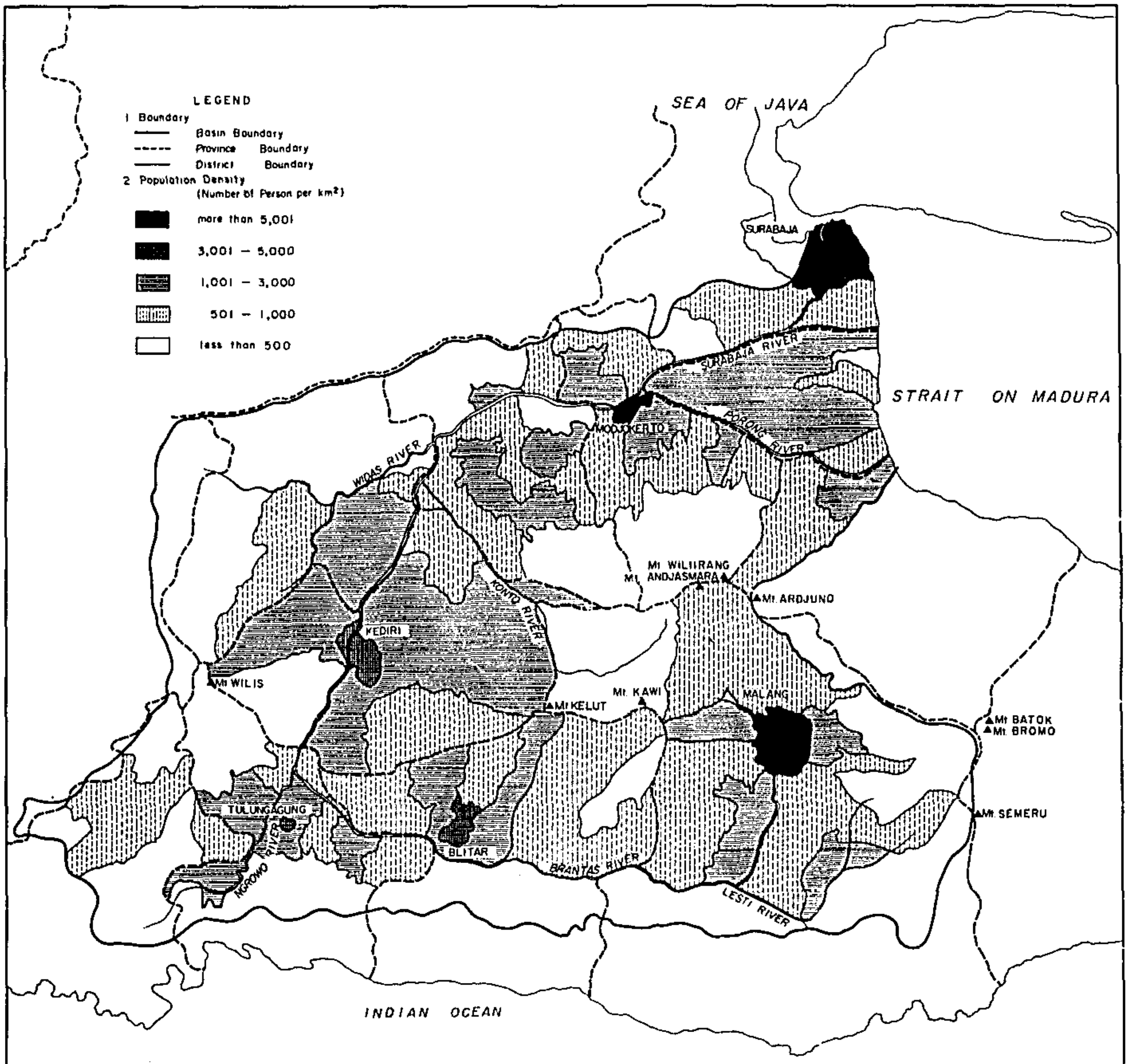

Fig. I-1 POPULATION DENSITY MAP OF THE BRANTAS RIVER BASIN

(SEPT. 1971)

2. PRESENT IRRIGATION SITUATION

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2. PRESENT IRRIGATION SITUATION

2-1 Introduction

This chapter presents the outline of the present situation of irrigation agriculture in the Brantas River Basin.

2-2 Meteorological Condition

The Brantas River Basin is affected by monsoon. The east monsoon usally lasts from April to November and the west monsoon which brings much rain prevails from December to the next March. The west monsoon causes much rainfall at the area around Mt. Kelut and Mt. Butak.

Based on the daily rainfall data recorded at 52 gauges during the period of 20 years since 1951 in Brantas River Basin, the average monthly basin rainfall at the area upstream from Terusan site[/1] is estimated as shown in Table 2-1. The yearly basin mean rainfall amounts to 2,030 millimeters, of which 80 percent occurs in six months from November to April.

There are very little variation in temperature throughout a year. The average monthly mean temperature during the period of 1928 to 1947 at Malang (445 meters above sea level) is as shwon in Table 2-2. The yearly mean temperature is 23.7^{o}C.

The relative humidity is high. The yearly mean relative humidity varies from 80 to 89 percents between inland and coastal district. The average monthly humidity during the period of 1928 to 1947 at Malang is as shown in Table 2-3.

The average monthly evaporation during the period of 1938 to 1947 at Malang is as shown in Table 2-4. This table shows that the annual evaporation is about 700 millimeters. This figure looks too low. Table 2-5 is a reproduction of data in a Japanese book[/2]. This table shows the average evaporation of 1,110 - 1,150 millimeters.

/1 Refer to Fig. 4-1, Chapter 4.

/2 "Climate in the South-East Asia" Kokon Shoin, Tokyo, 1964.

2-3 Soils

The soils in the Brantas River Basin are generally fertile because of volcanic deposit of Mt. Kelut. They are classified into nine great soil groups, i.e., Alluvials, Mediterranean Soils, Lithosols, Regosols, Andosols, Grumusols, Humus Gley Soils, Latosols and Brown Forest Soils. The distribution of these soils is shown in Fig. 2-1 and Table 2-6. Alluvials and Regosols are prevailing (53.8% of the basin) while Grumusols, Humus Gley Soils and Brown Forest Soils are few. Herein the characteristics of major soils are explained.

(1) Alluvials

These soils exist along the Brantas River and its tributaries. They are the most important agricultural soils in this basin. They are extensively cultivated, mainly for rice, unless poorly drained. They occupy 347,000 hectares, or 29.4 percent of the basin. The effective soil depth is very deep and inherent soil fertility is rich. The productivity of these soils can be greatly increased by appropriate fertilizer application under proper management of irrigation and drainage.

(2) Mediterranean Soils

These soils develop over the piedmont area of Mt. Wilis, Lima and Butak which exist between Alluvials and Latosols. They occupy 129,000 hectares, or 10.9 percent of the basin. They are medium in texture and subject to erosion. The areas are mainly upland farms and forest lands.

(3) Lithosols

These soils occupy the southern hilly area of the Brantas River Basin covering 95,000 hectares, or 8.1 percent of the basin. The agricultural potential is very low due to very shallow effective soil depth and topographical limitation. These soils are found mostly in the forests.

(4) Regosols

These soils extend on the middle slopes of mountains such as Mt. Kawi, Kelut, Ardjuno, being adjacent to Alluvials. They occupy 288,000 hectares, or 24.4 percent of the basin. They are light in soil texture and high in soil permeability. Sometimes they are affected by drought in the dry season due to low water holding capacity. Generally these soils have low agricultural potential and are not suitable for paddy cultivation. They permit an intensive farming with groundnut and manioc as main crops. It is necessary to prevent erosion of these soils, because sand deposit in the Brantas River is the efflux of these Regosols.

(5) Andosols

These soils are volcanic ash soils having humus and medium texture. They are found near the summits of Mt. Wilis, Lamas, Lima, Butak and occupy 93,000 hectares, or 7.9 percent of the basin. They are rich in soil fertility and have high water holding capacity. However, they subject to erosion. The land is used as forest land and for upland crop cultivation. Sometimes minor elements are required for certain crops.

(6) Latosols

These soils are so called "Laterite". They occur between Andosols and Mediterranean Soils. The Latosols occupies 185,000 hectares, or 15.7 percent of the basin. They are productive soils due to their excellent physical properties, deep soil depth and tolerance against soil erosion. However, the inherent soil fertility is low. They are suitable to grow food crops such as peanut, sweet potatoes, beans, manioc as well as industrial crops and fruits. They respond well to nitrogen and phosphorus. Minor elements such as boron, copper, nikel, cloride are sometimes necessary for certain crops. The present vegetations on these soils are forest and upland crops.

2-4 Land Use

The present land use in the Brantas River Basin is shown in Fig. 2-2 and Table 2-7. There are 321,000 hectares of paddy lands, 247,000 hectares of upland farms and 46,000 hectares of estates and orchards in the basin. Considering that about 60% of the settlement areas are also cultivated for agriculture, total farmland area is estimated to be 727,000 hectares, or 62 percent of the basin area. The forest areas are 279,000 hectares.

The paddy fields are located along the Brantas River and its tributaries where lands are flat and water is available. The upland crop areas are prevailing on the lower slopes of Mt. Kelut and Mt. Kawi. The estates and orchards are cocoa, coffee and rubber plantations, being located on the western slopes of Mt. Kelut and Northern slopes of Mt. Kawi. Forests are located on the mountain slopes above elevation 300 to 400 meters. The settlement areas are scattered over the basin.

2-5 Irrigation Area

There are about 321,000 hectares of the irrigation area. They are managed by 13 Sections of the Irrigation Service, East Java. Figure 2-3 shows the distribution of the irrigation areas by river system with indication of the boundary of the management by eaoh Seetion.

The irrigation areas are classified into the technical, semi-technical and non-technical irrigation areas, depending on the degree of the management of the Irrigation Service of East Java. In the technical irrigation area, the whole irrigation facilities are managed by the Irrigation Service. In the semi-technical irrigation area, the Irrigation Service manages only the main diversion facilities. In the non-technical irrigation area, all the facilities are left to the farmers' management.

Table 2-8 shows the classification of the irrigation areas. The technical irrigation areas are about 208,000 hectares which are mostly located along the Brantas River and large tributaries. The other irrigation areas total about 113,000 hectares and located mainly along small tributaries.

The irrigation areas depending on the Brantas main stream are about 77,000 hectares. Their majority are the technical irrigation areas as listed in Table 2-9. These areas and their intakes are shown in Fig. 2-4.

2-6 Cropping Pattern

Among 321,000 hectares of the irrigation areas, 276,000 hectares are paddy fields, 32,000 hectares are sugar cane plantation and the remaining 13,000 hectares are upland or fallow.

The prevailing cropping pattern is paddy in the rainy season and fallow in the dry season. Upland crops are grown in some areas in the intermediate period. Paddy is grown also in the dry season in areas where water is available.

The seeding period of the wet monsoon paddy is from mid-November to mid-February, the initial 30 - 50 days are the nursery period. The harvesting period is from mid-April to mid-July. In the case of the dry monsoon paddy, seeding period is between mid-March and mid-June and the harvesting period is mid-July to mid-October.

The data pertaining the crop areas in the irrigation areas were collected from each Section of the Irrigation Service, East Java, and the cropping intensities in the rainy and dry seasons were estimated as shown in Table 2-10. This table shows that about 86 percent of the irrigation area is cultivated for rice in the rainy season, while it reduces to 24 percent in the dry season. The rice cropping intensity in the dry season is very low in the areas belonging to the Kediri, Ngandjuk, Pare, Djombang, Modjoagung and Modjokerto Sections which are located in the middle stretch of the Brantas River and the tributary areas.

2-7 Irrigation Water Supply

The gravity irrigation depending on the river is the typical irrigation method in the Brantas River Basin. There are some intake weirs and in very few cases pump irrigation is practiced. The length of the main irrigation canals total about 1,800 kilometers.

Table 2-11 shows results of a water analysis in the Brantas River by the Agricultural Institute in Bogor. The water quality is normal and suitable for irrigation. However, the suspended matters and muddiness show quite high values due to the large sediment concentration.

The irrigation intakes in the Brantas River face the river directly, except the Voor canal which is connected with the Lengkong Dam. The control of inflow into the irrigation canal is attained by adjusting the stop logs at the intake. Proper adjustment of inflow is difficult by this method, under the condition that the river water level largely changes. Naturally the inflow would be larger than the required volume in the rainy season. The sediment-bearing water in the rainy season reduces the canal capacity by depositing sediment. The discharge capacity of the intake also decreases, because of sediment transported by every flood. The intakes and canals in the middle stretch of the Brantas River are especially affected by the sediment deposit. This fact may explain the low intensity of the dry season crop in the middle stretch. The same in the tributary areas would be the lack of river water in the dry season.

The inflow discharge records at the major intakes in the Brantas River were collected from the Irrigation Service. They were summarized as mean monthly discharge as shown in Table 2-12.

2-8 Individual Irrigation Area

Herein explained are the outline of the present condition of major irrigation areas which are:

1. Molek Irrigation Area: Located in the upstream reaches of the Brantas River. Substantially free from the eruption of Mt. Kelut. Rainfall 2,000 mm.

2. Warudjaeng: Located in the middle stretch of the Brantas River. Largely affected by sediment. Rainfall 1,500 mm.

3. Turi-Tunggorono: Located in the lower stretch of the Brantas River. Largely affected by sediment. Rainfall 2,000 mm.

4. Sidoardjo: Estuary delta. Largely affected by sediment. Rainfall 1,500 mm.

(1) Molek Irrigation Area (4,275 ha)

This area is located on the right bank of the Brantas River around Kepandjen. The Brantas water is taken at the Blobo intake located two kilometers northwest of Kepandjen. The "Molek Canal", the main canal is 17.9 kilometers long and mostly pitched with masonry. The main canal is well maintained but the intake is affected by silting.

The crop intensity of paddy is the highest among those in the whole basin. The area is fully planted in the rainy season. The dry monsoon paddy is grown on 60 percent of the area.

The mean monthly intake discharge in Table 2-12 can be expressed as the rate of diverted water in depth on the crop areas in the rainy season and the dry season. The maximum of the mean monthly intake discharge is eight cubic meters occurring in March and the minimum is 4.2 cubic meters occurring in October. These discharges correspond to the rate of diverted water in depth 16 millimeters per day in the rainy season and 14 millimeters per day in the dry season. According to a report[/1], the potential evapo-transpiration on farm per day is four millimeters. The effective rainfall in the dry season is regarded to be nil. Assuming that the deep percolation is one millimeter and the overall irrigation efficiency is 50 percent, the daily diversion requirement in the dry season is estimated to be ten millimeters. The rainfall in the rainy season is plenty and evenly distributed. Then, the daily diversion requirement in the rainy season will be very small. Comparing the diverted waters in depth with the daily diversion water requirement, it is judged that the rainy season discharge is more than enough because of inconvenience of the intake operation. Even with the presently diverted water, it would be possible to increase the dry season crop intensity by improving the efficiency of the irrigation system.

(2) Warudjaeng Irrigation Area (13,363 ha)

The Warudjaeng area is a plain extending on the left bank of the main stream between Kediri and the mouth of the Widas River. The area near the confluence of the Brantas and Widas Rivers is swampy.

/1 Report on Sempor Dam and Irrigation Project

The Mritjan intake, which is the largest among the intakes in the whole basin, irrigates most of the area through a 3.5-kilometer-long main canal. The other intakes are located at Bandjarsari, Besuk, Redungkudi and Pengkol. The irrigation system is seriously affected by sediment. The canals are heavily silted. The intake operation is difficult due to the sediment deposit. It is said that the main canal capacity was 16 cubic meters per second before the World War II, but it is 12 cubic meters per second at present.

Sugar is grown in about 1,900 hectares, and the wet monsoon paddy is grown in all the other areas. However, the cropping area of the dry monsoon paddy is only about 700 hectares.

The rate of the diverted water in depth in the crop areas both for the paddy and sugar cane is calculated to be seven millimeters per day in the rainy season and 37 millimeters per day in the dry season, respectively. The same explanation as in the Molek area would be applicable for the rainy season discharge. The rate of the diverted water in depth in the dry season looks too much in the dry season. This would be a result of the deterioration of the irrigation facilities in this area. Rehabilitation of the facilities would enable more production in the dry season.

(3) Turi-Tunggorono Irrigation Area (9,626 ha)

This area is located on the right bank of the main stream between the mouth of the Widas River and the junction of the main stream and the Porong River. The intakes are located at Turi and Tunggorono and the length of the main canal is about 60 kilometers. The irrigation canals are seriously silted.

The wet monsoon paddy is grown on 90 percent of the area and the rest of the area is sugar cane plantation. The rice-growing area reduces to about 40 percent in the dry season.

The rate of the diverted water in depth is six millimeters per day in the rainy season and eight millimeters per day in the dry season. The rate is high in the rainy season, but it is little less than the requirement of the present crops in the dry season. Since the rate represents an average figure, there must be a certain yield reduction in some dry season. Improvement of the intake condition would be needed.

(4) Sidoardjo Delta Irrigation Area (33,417 ha)

The Sidoardjo is the estuary delta of the Brantas River, located between the Surabaja and Porong Rivers. The Lengkong Dam, located in the Porong River just below the junction, supplies water to the Voor canal on the delta. The Voor canal branches into the Mangetan and Porong canals. The Mangetan canal has a length of 23 kilometers and irrigates the northern half of the delta. The Porong canal, 28 kilometers long, supplies the southern half. The original discharge capacity was 45 cubic meters per second in the Mangetan canal and 35 cubic meters per second in the Porong canal respectively, but it has reduced to about 80 percent.

The Sidoardjo area has been cultivated for sugar cane before the World War II. Presently, sugar is grown on only 15 percent of the area. Rest of the area has been turned to paddy field. The crop intensity of the dry monsoon paddy is about 38 percent in the dry season.

The rate of the diverted water in depth in the crop areas both for the paddy and sugar cane is calculated to be 18 millimeters per day in the rainy season and 16 millimeters per day in the dry season respectively. Increase of the dry monsoon paddy with the presently diverted water would be possible by improvement of the irrigation facilities.

The above descriptions may be summarized that the intake discharge is largely affected by the river flow and the sediment deposit. Large amount of water is taken while rainfall is plenty in the rainy season. This fact involves a drainage problem which is one of the constraint for yield increase. The intake discharge is more than enough to feed the present crops in many cases in the dry season. This fact suggests that more production is possible in the dry season, if the irrigation efficiency is improved by rehabilitation of the irrigation canals.

2-9 River Flow Reduction by Irrigation

It has been seen, in Section 2-8, that the intake discharge in the dry season is generally enough for the present cropping. Herein the balance of river water and the irrigation water is investigated. In this case, the intake discharge does not represent the irrigation water consumption because certain water returns from the farm to the river.

Part of rainfall is lost to the air as the evapo-transpiration and the rest appears in the river. The evapo-transpiration in the irrigation area comprises the evapo-transpiration on farm and evaporation in the irrigation facilities. It is normally much higher than in the rainfed land especially in the dry season. The river flow is reduced in a certain extent by the irrigation in the catchment area.

The irrigation area in the catchment area of the Djabon water gauge is about 220,000 hectares. The average crop intensity in the area is about ten percent in the dry season. Assuming that the evapo-transpiration/1 in the irrigation area is five millimeters per day, the river flow reduction due to irrigation is calculated to be 13 cubic meters per second:

$$\frac{220{,}000 \text{ ha} \times 0.1 \times 5 \text{ mm}}{86{,}400 \text{ sec}} \doteqdot 13 \text{ m}^3/\text{sec}$$

The minimum of the average September discharge at Djabon is about 51 cubic meters per second. Therefore, the natural flow without irrigation is estimated to be 64 cubic meters per second.

In the irrigation areas of 220,000 hectares, about 40,000 hectares depend on the Brantas River and the remaining 180,000 hectares are located in the tributary areas. If the crop intensity in the dry season would be 60 percent along the main stream and ten percent in the tributary areas, respectively, in the future, the river flow reduction due to irrigation will be:

$$\frac{(40{,}000 \text{ ha} \times 0.6 + 180{,}000 \text{ ha} \times 0.1) \times 5 \text{ mm}}{86{,}400 \text{ sec}} \doteqdot 24 \text{ m}^3/\text{sec}$$

/1 Evapo-transpiration on farm plus minor losses.

Then, the dry season discharge will reduce to about 40 cubic meters per second. A comparison of this discharge with the intake discharge in the Sidoardjo delta irrigation area (September discharge is 11.6 cubic meters per second at the Porong intake and 20.3 cubic meters per second at the Mangetan intake as shown in Table 2-12) shows that the balance is only 10 cubic meters per second. The analysis in Chapter 4 showed that the monthly mean discharge may often reduce to 20 - 30 cubic meters per second. Furthermore, more water will be needed in the future for the pollution control, municipal and industrial water supply in the downstream area. The critical water shortage will often be experienced, unless the irrigation development would be supported by a reservoir which regulates the river flow.

2-10 Crop Yield

The average unit yield of the dry stalk paddy/1 was estimated for the area of each Section, based on the 1960-1969 production data of the Irrigation Service. The average unit yield of the dry stalk paddy estimated for the area in the whole basin is 3.4 tons per hectare for the wet monsoon paddy and 3.1 tons per hectare for the dry monsoon paddy. The average yield in each area is calculated by multiplying the unit yield to the crop area as shown in Table 2-13. The yield in the whole basin thus calculated is about 1.2 million dry stalk paddy tons. This is about 10 - 12 percent of the rice production in Java.

The data of the sugar production in the Brantas River Basin are summarized in Table 2-14. The total production of sugar is about 230,000 tons which is about 30 percent of the country's total.

2-11 Conclusions

(1) The annual rainfall in the Brantas River Basin is about 2,000 millimeters of which about 80 percent occurs in November - April. The air temperature is about 24 degrees on an average. It is a little high at the beginning and end of the rainy season and a little low in August.

/1 "Stalk paddy" consists of the rice panicle, cut off by hand, with about 15 - 20 centimeters of the stalk attached. When dry, it converts to about 50 percent milled rice by weight.

(2) The soils in the Brantas River Basin are generally fertile, because of volcanic deposit of Mt. Kelut. The major soil groups are Alluvials, Mediterranean Soils, Lithosols, Regosols, Andosols and Latosols.

(3) The total basin area is 1,180,000 hectares and about 62 percent is farmland. The paddy land is about 321,000 hectares located along the rivers. The upland farms count about 247,000 hectares prevailing on the lower slopes of the mountains.

(4) The irrigation areas total about 321,000 hectares. About 208,000 hectares are located along the Brantas River and large tributaries. The remaining 113,000 hectares are depending on the small tributaries.

(5) In the irrigation area, about 276,000 hectares are paddy field, 32,000 hectares are sugar cane plantation and rests are upland or fallow. The crop area of paddy in the dry season reduces by about 72 percent.

(6) The irrigation systems along the Brantas River generally take too much water in the rainy season when rainfall is plenty. This may be explained from the condition of the intake structure of which the capacity can not be easily adjusted. The crop intensity in the dry season is very low. This is because the irrigation system is largely deteriorated by heavy silting. The intake discharge data show that the most of the intakes are taking more than enough to feed the present crops in the dry season. The crop intensity in the dry season can be rised, if the irrigation canals are rehabilitated. It seems that another reason of too much water in the rainy season and too low crop intensity in the dry season rests on the conventional custom.

(7) An estimate showed that critical water shortage will often occur in the lower reaches of the Brantas River in the future, unless the irrigation would be supported by a river regulation by the reservoir.

(8) The average unit yield of the dry stalk paddy estimated for the area in the whole basin is 3.4 tons per hectare for the wet monsoon paddy and 3.1 tons per hectare for the dry monsoon paddy. The rice yield in the Brantas River Basin is about 1.2 million tons (dry stalk paddy) being 10 - 12 percent of the production in Java. The sugar production is about 230,000 tons which corresponds to 30 percent of the national production.

(9) Rehabilitation and proper management of the irrigation system is necessary for not only maintaining but also increasing the present agricultural production. Prevention of intrusion of the sediment-bearing flood water by improvement of the intake facilities is important for maintenance of the irrigation system.

(10) The operation of the Karangkates dam will provide a large room for increased water use in the main stream. The tributaries still have the water shortage problem especially in the dry season. Construction of irrigation reservoirs in the tributary will help higher production.

Table 2-1 Average Monthly Basin Rainfall at Terusan (1951 - 1970)

(mm)

Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sep.	Oct.	Nov.	Dec.	Total
326	301	308	195	131	66	65	29	24	79	183	323	2,030

Table 2-2 Average Monthly Temperature at Malang (1928 - 1947)

(Degree in centigrade)

Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sep.	Oct.	Nov.	Dec.	Mean
23.9	23.9	23.9	24.1	23.9	23.3	22.5	22.9	23.7	24.5	24.5	23.9	23.7

Table 2-3 Average Monthly Relative Humidity at Malang (1928 - 1947)

(%)

Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sep.	Oct.	Nov.	Dec.	Mean
86	87	87	86	84	82	79	76	74	75	81	86	82

Table 2-4 Average Monthly Evaporation at Malang (1938 - 1947)

(mm)

	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sep.	Oct.	Nov.	Dec.	Total or mean
Daily mean	1.46	1.44	1.48	1.54	1.63	1.79	2.03	2.42	2.82	2.69	1.99	1.53	1.90
Monthly mean	45.3	40.3	45.9	46.2	50.5	53.7	62.9	75.0	84.6	83.4	59.7	47.4	695

Table 2-5 Average Monthly Evaporation at Pasuruan and Djember in East Java

(mm)

	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sep.	Oct.	Nov.	Dec.	Total
(1) Pasuruan (1901 - 1938)													
Monthly mean	62	50	50	60	74	87	115	133	147	152	108	74	1,112
(2) Djember (1913 - 1936)													
Monthly mean	81	70	78	84	93	93	105	118	132	121	90	84	1,149

Table 2-6 Great Soil Groups

Soil Group	Area (ha)	Percentage
Alluvials	347,000	29.4
Mediterranean Soils	129,000	10.9
Lighosols	95,000	8.1
Regosols	288,000	24.4
Andosols	93,000	7.9
Grumsols	31,000	2.6
Humus Gley Soils	5,000	0.4
Latosols	185,000	15.7
Brown Forest Soils	7,000	0.6
Total	1,180,000	100

Source: Compiled based on 1:250,000 soil map, Bogor Soil Institute.

Table 2-7 Land Use Classification

Land division	Area (ha)	Percentage
Paddy Field	321,000	27.3
Upland Farm	247,000	20.9
Forest	279,000	23.4
Settlement Area (Village & Urban Areas)	189,000	16.0
Fish Pond	15,000	1.4
Estate & Orchard	46,000	3.9
Waste Land & Naked Land	62,000	5.3
Others	21,000	1.8
Total	1,180,000	100

Source: Plan of land use in the lower reaches of Brantas River and office file in Provincial Land Use Office in East Java.

Table 2-8 Irrigation Areas in the Whole Basin

(Unit: ha)

Name of[/1] Section	Technical Irrigation	Semi-technical Irrigation	Non-technical Irrigation	Total
Kasri	9,257	800	3,643	13,700
Malang	7,820	2,559	7,665	18,044
Kepandjen	15,282	2,411	7,970	25,663
Blitar	15,530	2,392	11,649	29,571
Tulungagung	11,145	7,680	6,982	25,807
Kediri	13,592	5,829	9,720	29,141
Ngandjuk	32,424	1,376	5,064	38,864
Pare	14,239	0	5,373	19,612
Djombang	19,554	0	4,827	24,381
Modjoagung	20,807	0	2,392	23,199
Modjokerto	13,375	10,635	8,422	32,432
Sidoardjo	32,360	1,057	0	33,417
Wonokromo	2,559	4,170	0	6,729
Total	207,944	38,909	73,707	320,560

Remarks: /1 Name of the Section of the Irrigation Service. The Kepandjen, Blitar and Tulungagung Sections manage some areas outside of the basin in addition to the areas in this Table.

Table 2-9 Irrigation Areas Depending on the Main Stream

(Unit: ha)

Name of Intake	Name of Section	Technical Irrigation	Semi-technical Irrigation	Non-technical Irrigation	Total
Malang/1	Malang	1,515	0	2,672	4,187
Molek	Kepandjen	4,275	0	0	4,275
Mritjan	Ngandjuk	12,819	0	0	12,819
Besuk	"	544	0	0	544
Turi-Tunggorono	Djombang	9,305	0	321	9,626
Bunder	"	97	0	173	270
Djt. Mlerek	"	564	0	227	791
Gottan	"	188	0	492	680
Bebekan	"	229	0	0	229
Keboan	"	12	0	0	12
Djatikulon	Modjokerto	11	489	144	644
Ngares	"	1,815	0	0	1,815
Watespinggir	"	347	0	0	347
Losari	"	665	0	0	665
Lengkong dam/2	Sidoardjo	32,360	0	0	32,360
Others in Sidoardjo	"	0	1,057	0	1,057
Wonokromo	Wonokromo	2,559	4,170	0	6,729
Total		67,305	5,716	4,029	77,050

Remarks: /1 Senkaling intake and small intakes.

/2 Lengkong dam provides irrigation water to Voor, Mangetan and Porong canal.

Table 2-10 Cropping Intensity in Irrigation Area

(Unit: %)

	Rainy Season			Dry Season		
	Paddy	Sugar Cane	Upland or Fallow	Paddy	Sugar Cane	Upland or Fallow
Kasri (13,700 ha)	97	-	3	30	-	70
Malang (18,044)	89	5	6	50	5	45
Kepandjen (25,663)	79	20	1	48	20	32
Blitar (29,571)	94	6	-	29	6	65
Tulungagung (25,807)	82	6	12	30	6	64
Kediri (29,141)	80	14	6	12	14	74
Ngandjuk (38,864)	87	7	6	9	7	84
Pere (19,612)	90	10	-	12	10	78
Djombang (24,381)	83	17	-	13	17	70
Modjoagung (23,199)	85	10	5	10	10	80
Modjokerto (32,432)	92	6	2	16	6	78
Sidoardjo (33,417)	80	15	5	38	15	53
Wonokromo (6,729)	92	-	8	50	-	50
Average	86	10	4	24	10	66

Remarks: This table is compiled based on the recent records collected from the Sections of the Irrigation Service, East Java.

Table 2-11 Brantas Water Quality

		Oct. 10, 1959	Nov. 24, 1959	Dec. 14, 1959
Ca	mg/l	29.9	17.1	20.2
Mg	mg/l	10.3	4.8	5.1
Na	mg/l	11.6	8.5	9.1
K	mg/l	4.28	2.57	2.86
HCO_3	mg/l	168.6	108.7	107.6
SO_4	mg/l	6.5	6.1	8.9
Cl	mg/l	6.2	4.8	2.5
SiO_2	mg/l	60.8	41.9	39.0
Fe	mg/l	0.03	0.10	0.03
PO_4	mg/l	0.04	0	0
NO_3	mg/l	0.13	0.04	0
NH_4	mg/l	0.01	0.03	0.01
Album-N	mg/l	0.06	0.84	0.49
$KMnO_4$ Consumption	mg/l	3.3	14.6	10.4
Residuum	mg/l	219	163	155
Suspended Matter	mg/l	227.2	2,383.3	1,816.7
Muddiness	mg/l	42.6	1,920.0	3,432.0
Electric Conductivity	mho/cm	296.1	191.8	197.2
pH	-	6.8	6.7	7.0

Source: Agricultural Institute in Bogor

Table 2-12 <u>Intake Discharge</u>

(Unit: m^3/sec)

Intake	Blobo	Mritjan	Besuk	Turi-Tunggorono
Area (ha)	4,275	12,819	544	9,626
Period	1965 - 70	1950 - 70	1960 - 69	1965 - 70
Jan.	6.9	10.5	0.2	6.0
Feb.	7.4	10.7	0.2	6.4
Mar.	8.0	10.9	0.2	6.7
Apr.	7.6	10.2	0.2	6.5
May	7.3	7.9	0.2	5.5
June	6.1	7.3	0.2	5.2
July	5.3	7.0	0.2	4.7
Aug.	5.1	6.5	0.2	4.0
Sept.	4.9	5.9	0.2	3.6
Oct.	4.2	6.5	0.2	3.6
Nov.	4.5	7.3	0.2	4.5
Dec.	5.9	9.2	0.2	5.5
Total	73.2	99.9	2.4	62.2

Intake	Djatikulon	Porong	Mangetan
Area (ha)	644	...... 33,417	
Period	1960 - 70	1958 - 70	1958 - 70
Jan.	0.5	25.8	40.0
Feb.	0.7	26.9	40.9
Mar.	0.6	26.4	41.1
Apr.	0.4	23.6	36.5
May	0.3	22.1	34.8
June	0.5	21.6	36.1
July	0.6	19.0	32.3
Aug.	0.6	15.0	25.8
Sept.	0.6	11.6	20.3
Oct.	0.5	11.9	20.2
Nov.	0.4	15.6	26.1
Dec.	0.5	22.4	36.4
Total	6.2	241.9	390.5

Table 2-13 Rice Yield

Name of Section	Wet Monsoon Paddy Unit Yield (ton/ha)	Wet Monsoon Paddy Crop Area (ha)	Wet Monsoon Paddy Yield (ton)	Dry Monsoon Paddy Unit Yield (ton/ha)	Dry Monsoon Paddy Crop Area (ha)	Dry Monsoon Paddy Yield (ton)	Total Yield (ton)
Kasri	4.04	13,300	53,700	3.69	4,110	15,200	68,900
Malang	4.04	16,059	64,900	3.69	9,022	33,300	98,200
Kepandjen	4.04	20,274	81,900	3.69	12,318	45,500	127,400
Blitar	3.59	27,797	99,800	3.09	8,576	26,500	126,300
Tulungagung	3.54	21,162	74,900	2.66	7,742	20,600	95,500
Kediri	3.28	23,313	76,500	2.33	3,497	8,200	84,700
Ngandjuk	3.17	33,812	107,200	2.42	3,498	8,500	115,700
Pare	3.28	17,651	57,900	2.33	2,353	5,500	63,400
Djombang	2.81	20,236	56,900	2.22	3,170	7,000	63,900
Modjoagung	2.81	19,719	55,400	2.22	2,320	5,200	60,600
Modjokerto	3.15	29,837	94,000	3.85	5,189	20,000	11,400
Sidoardjo	3.52	26,734	94,100	2.98	12,690	37,800	131,900
Wonokromo	2.55	6,191	15,800	1.88	3,365	6,300	22,100
Total/mean	3.4	276,085	933,000	3.1	77,859	239,600	1,172,600

Remarks: Dry stalk paddy yield.

Table 2-14 Sugar Production

Name of Factory	Factory Farm Crop Area (ha)	Factory Farm Cane (ton)	Factory Farm Sugar (ton)	Non-Factory Farm Crop Area (ha)	Non-Factory Farm Cane (ton)	Non-Factory Farm Sugar (ton)	Total Sugar (ton)
Malang							
Kebonagung	981	104,400	12,400	1,852	134,000	12,700	25,100
Krebet Baru	0	0	0	2,570	254,400	25,000	25,000
Sidoardjo							
Tjandi	996	100,200	9,400	9	800	0	9,400
Krembung	1,003	111,500	10,100	1	0	0	10,100
Tulangan	1,052	115,200	11,300	30	2,500	200	11,500
Watululis	1,355	151,600	13,500	3	0	0	13,500
Krian	1,078	116,300	10,800	0	0	0	10,800
Modjokerto							
Gempolkerep	1,501	164,800	16,200	9	800	0	16,200
Djombang							
Tjukir	1,335	150,000	13,800	96	7,100	500	14,300
Djombang Baru	1,004	106,800	9,500	109	7,100	500	10,000
Kediri							
Modjopanggung	861	75,500	7,700	2,064	68,500	5,200	12,900
Ngadiredjo	2,028	169,700	14,800	1,107	71,200	6,400	21,200
Pesantren	1,167	120,500	11,500	1,859	79,200	7,400	18,900
Ngandjuk							
Mritjan	943	91,600	8,200	1,100	53,100	4,400	12,600
Lestari	1,359	125,400	12,800	443	17,700	1,400	14,200
Total			162,000			63,700	225,700

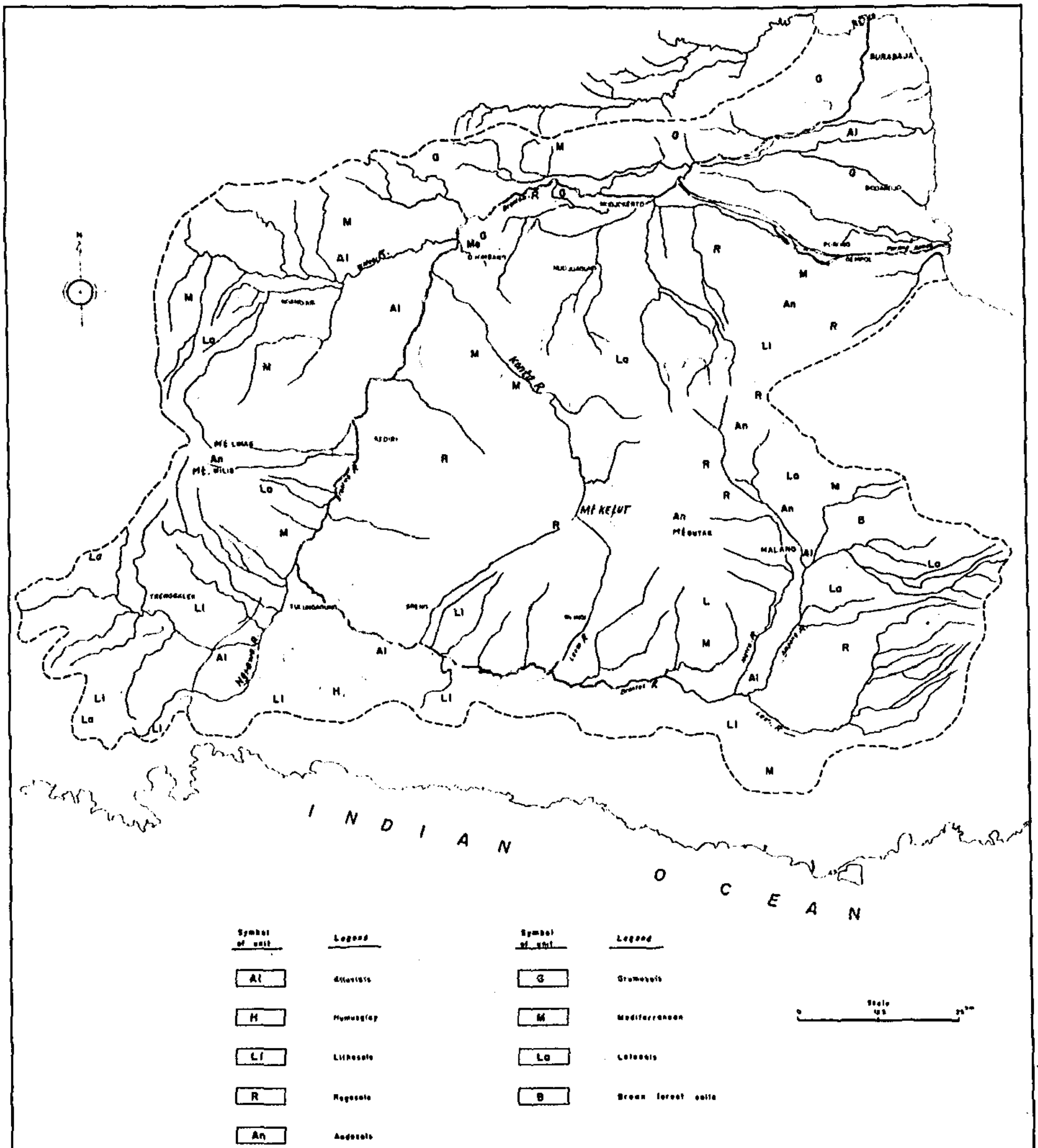


Fig. 2-1 Soil Map in the Brantas Basin

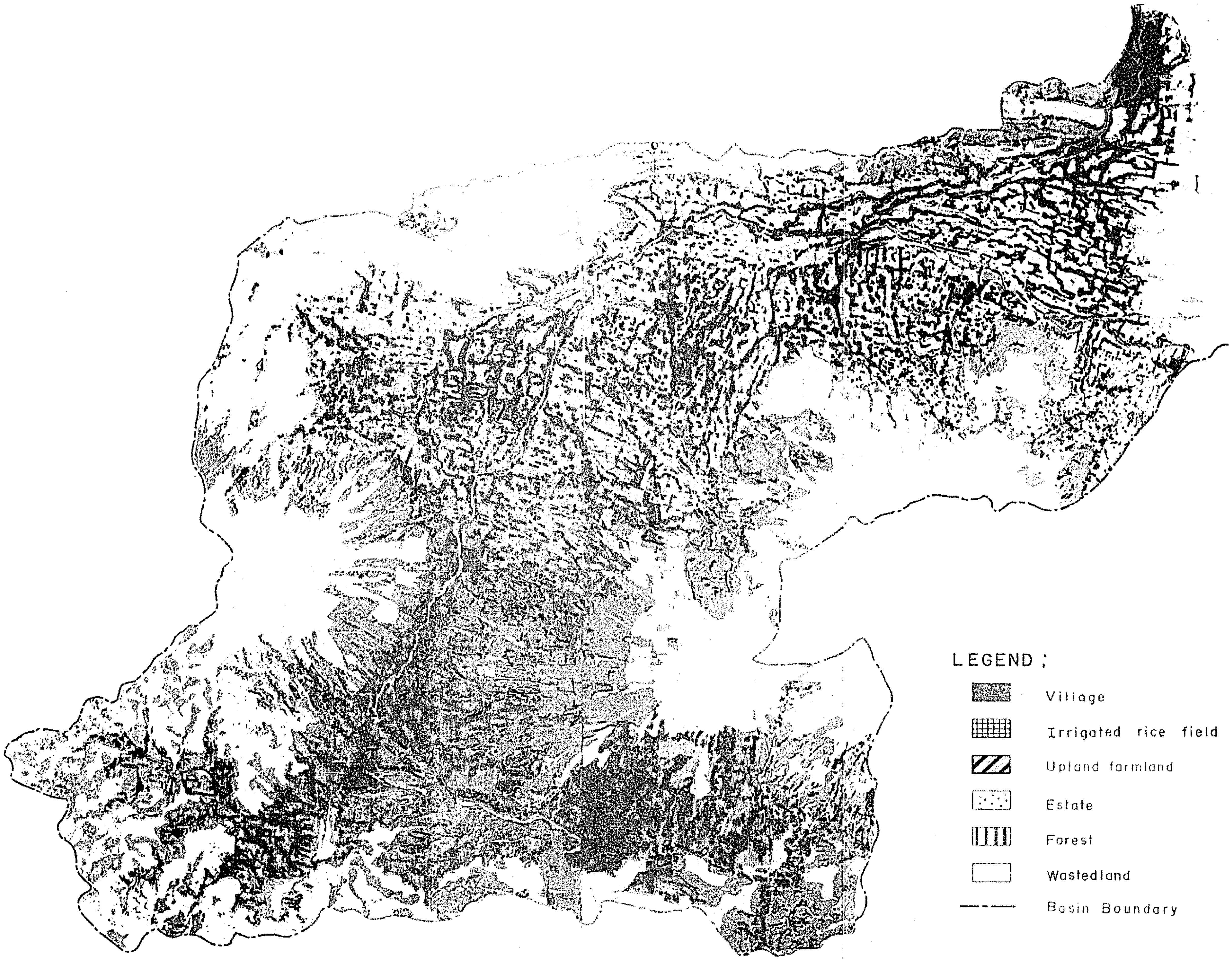
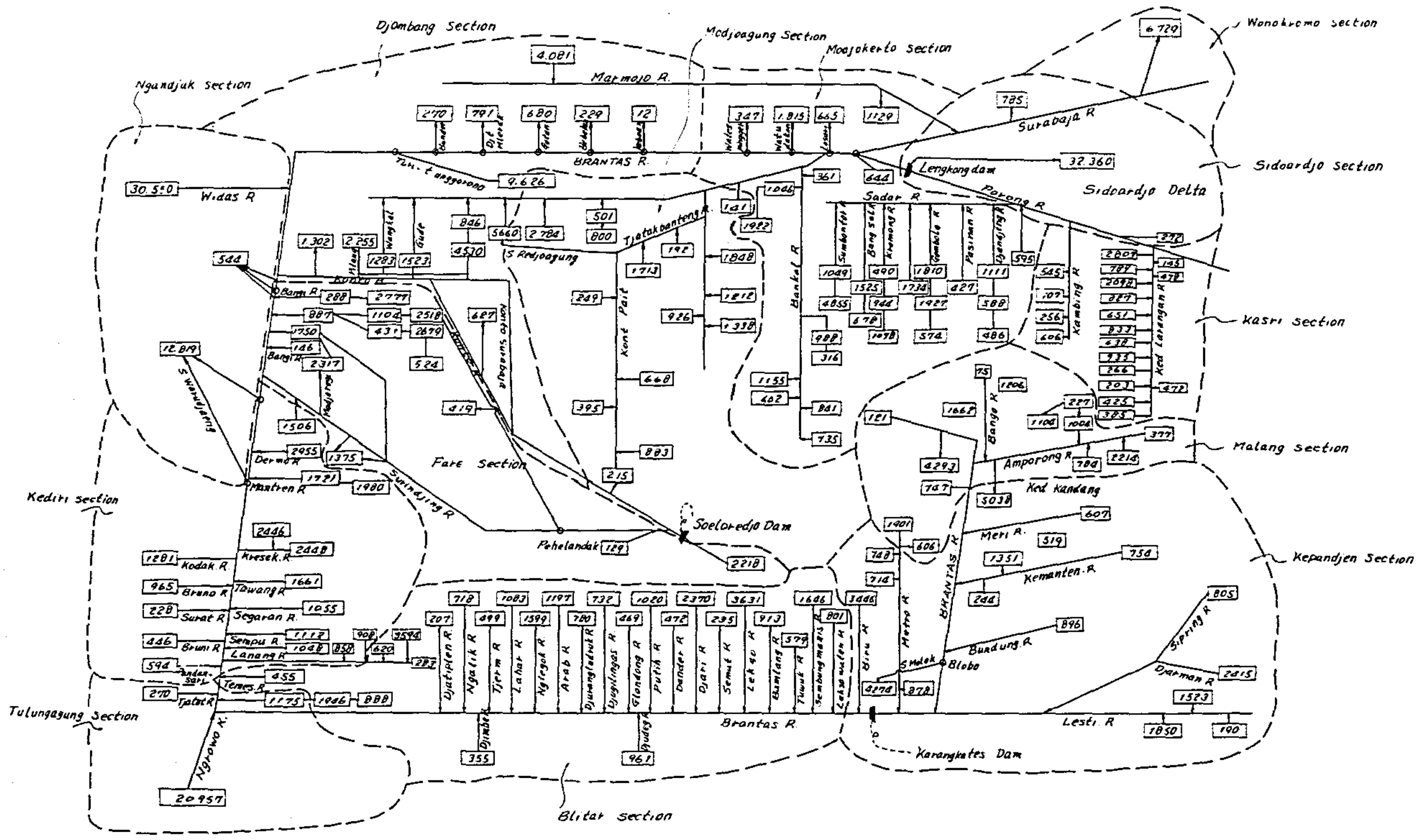
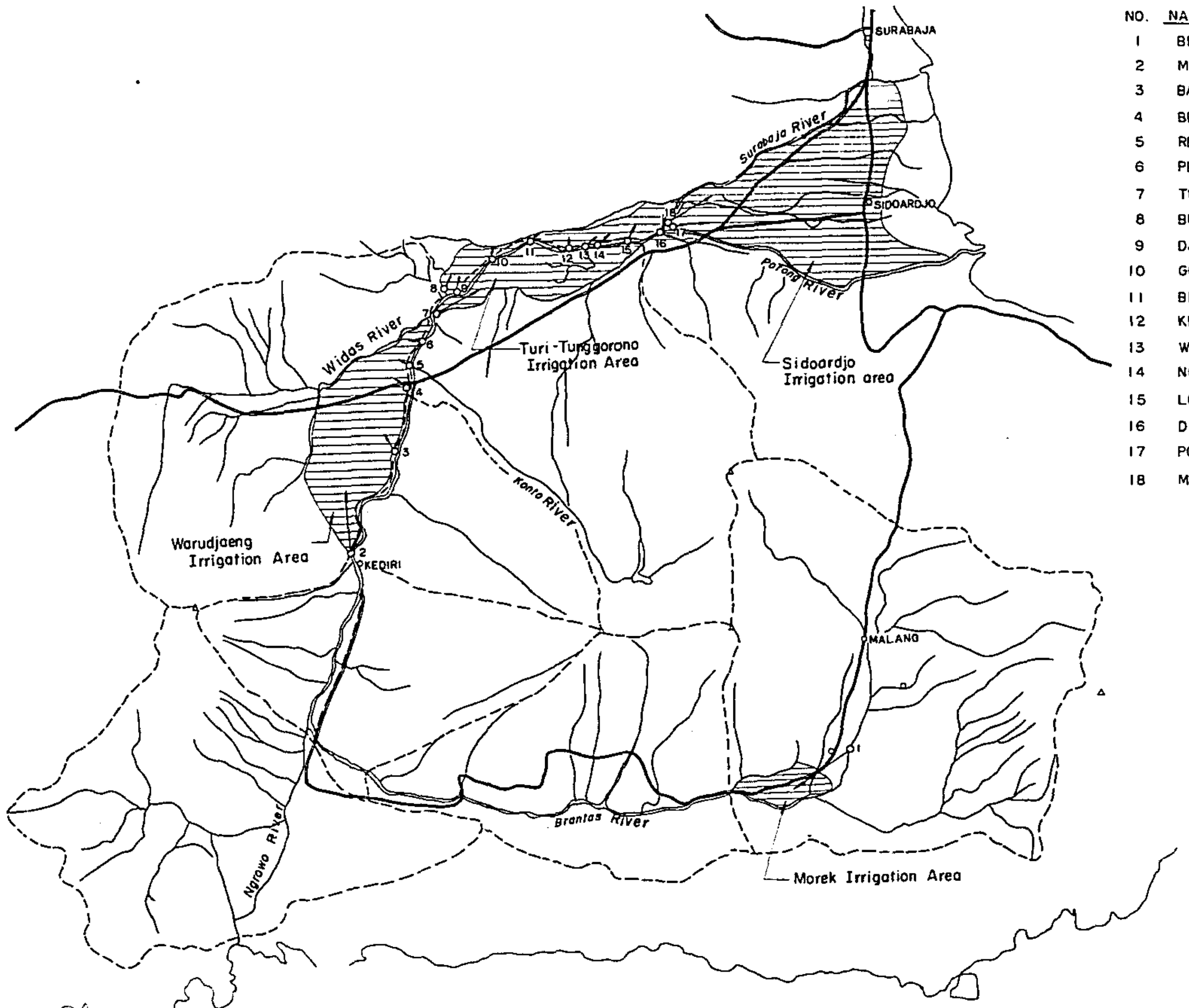


Fig. 2-2 Land Use Map



Remark : The figure of "1197 Arab R." for example means that areas of 1.197 hectares are irrigated by the discharge of Arab River and its tributaries.
; -------- means boundary Section
⊖ Intake along the Brantas main Stem.

Fig. 2-3 Schematic Distribution Map of Irrigated Area in the Brantas River Basin



NO.	NAME OF INTAKE
1	BLOBO
2	MRITJAN
3	BANDJARSARI
4	BESUK
5	REDUNGKUDI
6	PENGKOL
7	TURI-TUNGGORONO
8	BUNDER
9	DJATIMLEREK
10	GOTAN
11	BEBEKAN
12	KEBOAN
13	WATESPINGGER
14	NGARES (KEDUNGSARI)
15	LOSARI
16	DJATIKULON
17	PORONG
18	MANGETAN

Fig. 2-4 Location Map of Intake Structure and Irrigation Area along Brantas River Course

3. RAINFALL CHARACTERISTICS

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3. RAINFALL CHARACTERISTICS

3-1 Introduction

Herein presented is the study on rainfall characteristics in the Brantas River Basin. The rainfall data used are those at 52 gauges during the 1951-1970 period. The areal rainfall is estimated by the Tiessen method. The rainfall characteristics are explained from three aspects, i.e., seasonal rainfall, regional rainfall and storm rainfall.

3-2 Seasonal Rainfall

The daily rainfall is measured at 250 gauges in the Brantas River Basin. Among them the records at 52 gauges are incorporated in this study. The location map of the 52 gauges is as shown in Fig. 3-1, of which other details are explained later. The daily rainfall records at 52 gauges cover the 1951-1970 period with some interruption. The average rainfall during the 1951-1970 period at the 52 gauges is compiled in "Data Book".

The rainfall records were converted into the average rainfall depth by the Tiessen method.

"The Tiessen method assumes that the amount at any station can be applied halfway to the next station in any direction. It is applied by constructing a Tiessen polygon network, the polygons being formed by the perpendicular bisectors of the lines joining nearby stations. The area of each polygon is determind and is used to weight the rainfall amount of the station in the center of the polygon."/1

In this report, the average rainfall depth in an area, or in short the areal rainfall, is estimated by the Tiessen method.

The areal rainfall in the catchment area of Terusan discharge gauge was calculated by the Tiessen method. The result showed that the 1951-1970 average rainfall is 2,031 millimeters. The year-to-year variation is

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/1 Ven Te Chow "Handbook of Applied Hydrology" Mc Grow-Hill Book Company, New York, 1964.

rather little, i.e., the minimum during the 1950-1970 period is 1,375 millimeters, the maximum is 2,633 millimeters and the standard deviation is 375 millimeters.

Table 3-1 shows the minimum, mean and maximum of the average monthly rainfall during 1951-1970 period calculated for the catchment area of Terusan gauge. This table shows that a single rainy season and a single dry season are clearly defined, though there are some extraordinary cases. The rainy season is the six-month November - April, in which the rainfall totals 1,636 millimeters, or 80 percent of the annual. The dry season is the six-month May - October and its rainfall is 395 millimeters, or 20 percent.

Table 3-2 shows the average number of rainy days in each month estimated from rainfall records of 52 gauges. This table shows that it rains almost every day in six months from November to April.

3-3 Regional Rainfall

For the purpose of the regional analysis, the catchment area of the Brantas River Basin upstream from Terusan gauge was divided into seven sub-basins as shown in Fig. 3-1.

The sub-basin I is the area upstream from Pohgadjih gauge. The sub-basin II is the area between Pohgadjih and Pakel gauges and involves the southern area of Mt. Kelut and Mt. Butak. The sub-basin III is the Ngrowo River Basin. The sub-basin IV is the area between Pakel and Kediri gauges, except the Ngrowo River Basin. The sub-basin V is the western slope of Mt. Kelut between Kediri and Kertosono gauges. The sub-basin VI is the Widas River Basin. The sub-basin VII is the north-western slope of Mt. Kelut and Mt. Ardjuno between Kertosono and Terusan gauges.

The isohyetes in Fig. 3-1 were drawn taking into account of the orthogonal distribution of rainfall. The raingauges in the Brantas River Basin are mostly located in the low-lying areas, while the rainfall on the elevated land is very high. The Tiessen method may show too low rainfall under this condition. The rainfall in each sub-basin was calculated by both the Tiessen method and the isohyetal method. The resulted annual rainfall is shown in Table 3-3. Generally the Tiessen method shows lower rainfall than the isohyetal method but the difference is minor. The Tiessen method is exclusively used in this report because it is convenient especially in keeping the consistency of the estimate.

The calculation showed that the annual rainfall is high on the southern and western slopes of Mt. Kelut and relatively low in the downstream reaches of the Brantas River.

The sub-basin rainfall in each month during the 1951-1970 period is compiled in the "Data Book".

3-4 Storm Rainfall

The rainfall occurrence in a day is regular. Table 3-4 shows the hourly rainfall occurred simultaneously at Karangkates and Lodojo gauges. The table shows that the rainfall usually begins between two o'clock and seven o'clock in the afternoon. The duration time of one continuous rainfall ranges from three hours to 15 hours.

For use of the subsequent analysis, the probability of the annual maximum of the one-day rainfall was investigated for the catchment area of each water gauge. The 1951-1970 data were used and the Gumbel distribution was assumed. The resulted rainfall for selected return period is shown in Table 3-5.

Table 3-1 Monthly Rainfall at Terusan Gauge

(Unit: mm)

	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sep.	Oct.	Nov.	Dec.
Min.	192	203	186	81	21	0	0	0	1	5	43	195
Mean	326	301	308	195	132	66	65	29	24	79	183	323
Max.	446	368	427	319	265	199	325	113	83	381	414	448

.... Dry Season ....

Remarks: The average rainfall depth in the catchment area of the Terusan gauge by the Tiessen method based on 1951-1970 daily rainfall records at 52 gauges. The through mean is 169 millimeters per month.

Table 3-2 Number of Rainy Days

Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sep.	Oct.	Nov.	Dec.	Total
30	28	30	25	21	13	9	6	7	13	23	29	234

Table 3-3 Comparison of Calculated Annual Rainfall

(Unit: mm)

Sub-basin	(1) Tiessen method	(2) Isohyetal method	(1)/(2)
Sub-basin I (Upstream of Pohgadjih)	2,074	2,334	0.89
Sub-basin II (Pohgadjih - Pakel)	2,322	2,322	1.00
Sub-basin III(Ngrowo River Basin)	2,003	1,979	1.01
Sub-basin IV (Pakel - Kediri)	2,245	2,265	0.99
Sub-basin V (Kediri - Kertosono)	2,077	1,940	1.07
Sub-basin VI (Widas River Basin)	1,821	1,915	0.95
Sub-basin VII(Kertosono - Terusan)	1,808	2,163	0.84
Upstream of Terusan	2,041	2,112.1	0.97

Remarks: Annual rainfall is estimated from the records during the 1951-1970 period water year.

Table 3-4 Hourly Rainfall Occurred Simultaneously at Karangkates and Lodojo Gauges

	Lodojo				Karangkates			
	Beginning time of rainfall (o'clock)	Peaking time of rainfall (o'clock)	Duration time (hr)	Total rainfall (mm)	Beginning time of rainfall (o'clock)	Peaking time of rainfall (o'clock)	Duration time (hr)	Total rainfall (mm)
1965 Feb. 3	18	19	5	70	18	23	6	53
Apr. 6	14	15	3	72	13	15	5	35
May 28	19	20	5	55	20	22	4	19
1966 Jan. 8	19	19	5	14	17	17	7	43
Feb.14	13	14	6	21	13	13	15	47
Feb.24	14	20	9	47	14	20	11	31
Mar. 8	18	21	7	40	15	21	10	27
Mar. 9	14	14	4	46	15	15	5	21
Mar.18	18	19	8	83	19	19	6	38
Mar.27	12	14	8	22	14	14	7	24
Dec. 5	15	17	8	34	14	17	8	21

Table 3-5 One-day Rainfall

(Unit: mm)

Return period	Pohgadjih	Pakel	Kediri	Terusan
2 years	51	49	43	39
5 years	69	67	57	47
10 years	81	78	66	53
20 years	92	89	74	58
50 years	107	103	85	64
100 years	117	113	94	69

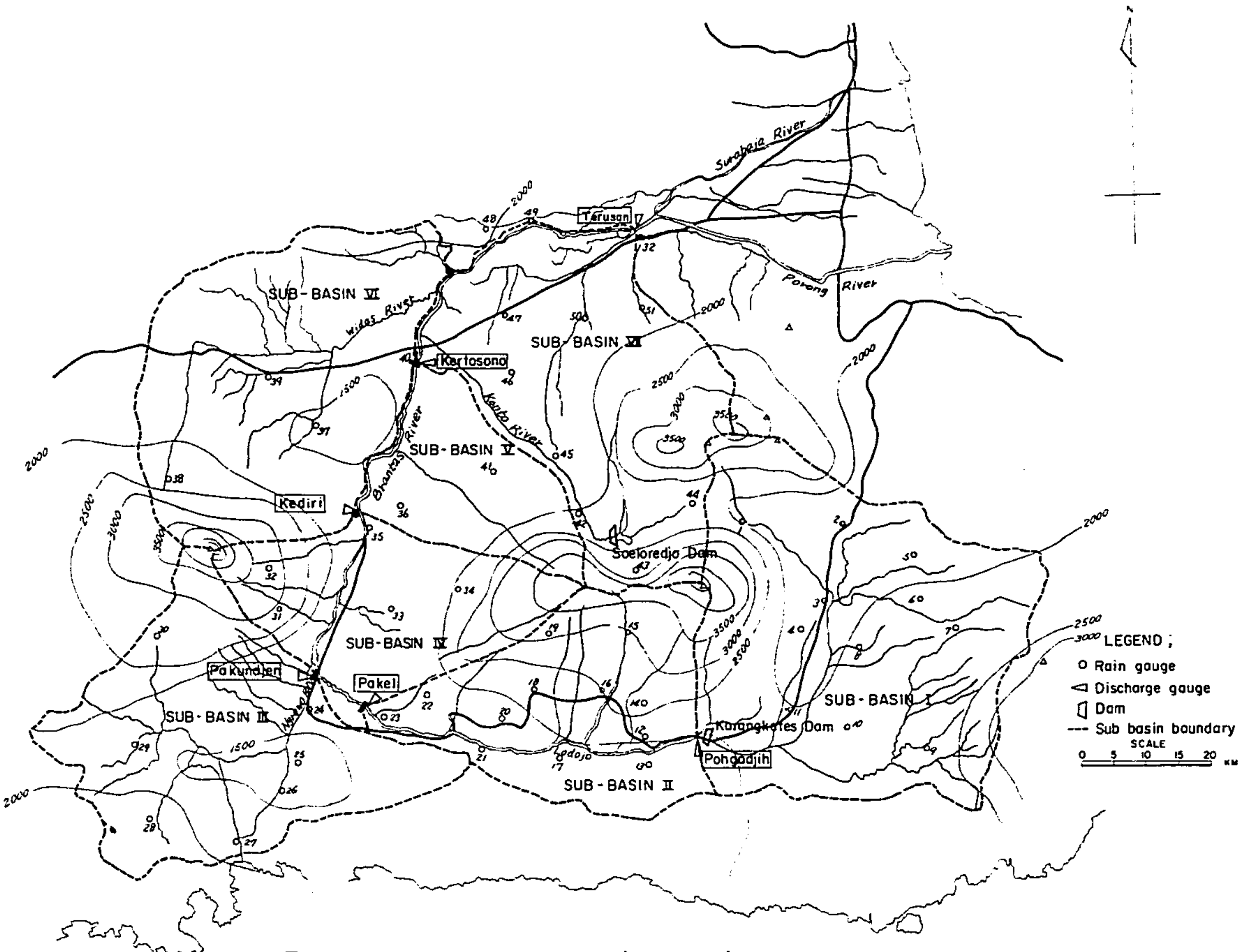

Fig. 3-1 Isohyetal Map (Annual)

4. RIVER FLOW ANALYSIS

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4. RIVER FLOW ANALYSIS

4-1 Introduction

The river flow measurement in the Brantas River Basin has long been carried out. The method of measurement and the accuracy of the records are largely constrained by the movement of the river bed.

Herein presented are explanation of hydrological data, an evaluation and adjustment of the discharge records and study on the river flow regulation by the Soeloredjo and Karangkates reservoirs.

The discharge records at seven water gauges in the Brantas River are evaluated by the results of a check measurement and the rainfall-discharge relationship. Some records are adjusted based on the evaluation.

The operation of the Karangkates and Soeloredjo reservoirs are studied by use of the flow-mass curves at the damsites. This study results in the estimate of the effects of the river regulation on the duration curves of the downstream river flow.

4-2 Available Data

4-2-1 Daily Water Level and Discharge Record

The water level and the discharge data/1 at 18 gauges along the Brantas River and its tributaries are available. The gauges are Karangkates, Pohgadjih, Kaulon, Pakel, Djeli, Djombiru, Kertosono and Djabon on the Brantas River, Wadi, Bendo, Parit Raja, Tulungagung and Pakundjen on the Ngrowo River, Soeloredjo on the Konto River, Widas on the Widas River, Mlirip and Perning on the Surabaja River and Kepadjaran on the Porong River. The location map of 18 gauges is shown in Fig. 4-1. The measurement duration period of the water level and the discharge data is shown in Fig. 4-2. The daily water level and the discharge data at eight gauges on the Brantas River, Mlirip and Perning gauges on the Surabaja River and Kepadjaran gauge on the Porong River cover mostly the 1951-1970 period. The monthly mean discharge calculated based on the these data is compiled in "Data Book".

/1 Data source: Provincial Irrigation Services in East Java.

4-2-2 Hourly Water Level Record

The hourly water level data/1 at eight gauges along the Brantas River and its tributaries are available. The gauges are Pohgadjih, Pakel, Kediri, Terusan on the Brantas River, Soeloredjo on the Konto River, Mlirip and Perning on the Surabaja River and Porong bridge on the Porong River. The locations of eight gauges are shown in Fig. 4-1. The measurement duration period of the hourly water level data is 20 years at the longest at Pakel gauge and about three years at the shortest at Pohgadjih gauge as shown in Fig. 4-2.

4-3 Discharge Record

The discharge records at seven gauges between the Pohgadjih and Djabon along the Brantas River are shown in Tables 4-1 to 4-7 as the mean discharge in each month.

The monthly discharge generally becomes the maximum in February/March and the minimum in September. Comparing this fact with the rainfall at Djabon gauge where the monthly rainfall is the highest in December and the lowest in September, it may be said that a large part of rainfall is retained as the basin storage especially at the earlier part of the rainy season, but not much water is carried over to the subsequent year.

4-4 Check Measurement

A flow measurement by a current meter was carried out at the Pakel, Djombiru, Kertosono and Djabon gauges from the middle of July to early August, 1971, to check up the accuracy of the regular measurement. The results are presented in Table 4-8 together with those in the regular measurements. The results of the regular measurements were very close to those of the check measurement at the Pakel and Djabon gauges, but they were quite different at Djombiru and Kertosono gauges.

4-5 Rainfall-Discharge Relationship

The water year instead of calender year is used in the following analysis. The water year usually starts when the basin storage is minimum. Herein the water year is determined as October - September.

/1 Source; Provincial Irrigation service in East Java.

The annual discharge was calculated from Tables 4-1 to 4-7. The data were plotted in Fig. 4-3 against the corresponding rainfall calculated by the Tiessen method explained in Chapter 3. The figure labeled on each plotting indicates the water year. For instance, the figure 51 means the data in the water year from October 1951 to September 1952.

The graph of the Djabon gauge, which is located the lowest among the gauges, shows a strong linear correlation, in which data are concentrating along a 45^{o} line.

The rainfall-discharge correlation at the Kertosono gauge is the weakest among those along the main stream. The rainfall and discharge data at the other gauges show good correlations, though weaker than that of the Djabon gauge.

4-6 Djabon Record Evaluation

The preceding analysis indicates that the discharge record at the Djabon gauge is the most reliable. The Djabon gauge is located 1.3 kilometers upstream of the junction of the Surabaja and Porong Rivers. The Mlirip gauge in the Surabaja River just downstream of the junction has the 1951-1970 discharge record. The Lengkong dam is located in the Porong River 200 meters downstream of the junction. It diverts the Prong water to the Voor irrigation canal. The Kepadjaran gauge is located just downstream of the Lengkong Dam. The 1951-1970 discharge records are available at both the head of the Voor canal and the Kepadjaran gauge.

To cross-check the Djabon record, the discharge record at the Djabon gauge was compared with the records at the Mlirip, Kepadjaran gauges and Voor canal. The result is shown in Fig. 4-4. The abscissa shows the monthly mean discharge at the Djabon gauge and the ordinate shows the total of the monthly mean dischargs at the other three gauges. Most of the data are very close to the 45^{o} line.

The results of the check measurement, rainfall-discharge analysis and this cross-check show that the discharge record at the Djabon gauge is reliable.

It is noted that the Gedek flood way located upstream of the Djabon gauge diverts a part of the flood discharge to the Surabaja River, and the Watudakon irrigation canal on the right bank of the Brantas River diverts water to the areas downstream of the gauge, crossing the Brantas River by siphone. Water through the flood way and the canal never flows the Djabon gauge site. Consequently, the Djabon record must show lower discharge than that flowing down from the catchment area of the gauge in the rainy season. The total discharge diverted through the flood way was estimated to be 100 $\times 10^6 m^3$ based on the 1962 hourly water level record at the inlet of the Gedek flood way. The discharge through the irrigation canal is minor. The year-to-year difference in the water diverted may be regarded little because of the good rainfall-discharge correlation in the preceding section.

4-7 Record Adjustment

Figure 4-5 shows the discharge hydrographs based on Tables 4-1 to 4-7. This fugire shows that there are disagreements between the records at different gauges, i.e., some records at a gauge are larger than the concurrent records at a gauge located downstream. This fact is partly explained from the inundation and water use between the gauges, but the major reason of this disagreement must be the inaccuracy of the flow measurements.

To facilitate for the analysis and planning pertaining the Brantas River, an adjustment was made on the prevailing discrepancies. In this adjustment the Djabon record is regarded to be accurate.

The 1964-1970 record at the Kertosono gauge appears too high in comparison with the others and the Kertosono record itself before 1964. Figure 4-6 shows a correlogram of the monthly discharge records at the Djabon and Kertosono gauges. The abscissa shows the monthly discharge record at the Djabon gauge and the ordinate shows that at the Kertosono gauge. Plottings in 1964-1970 period are indicated by crosses and those in the previous period are shown by dotts. Most of the 1964-1970 records show that the Kertosono discharge is larger than the Djabon discharge. A curve showing the average relationship between the Djabon and Kertosono discharges was drawn as shown in this graph, disregarding the 1964-1970 data. By use of this curve, the Djabon discharge during the 1964-1970 period was converted into the Kertosono discharge.

The same method was applied for the adjustment of the 1964-1970 Djombiru record of which measurement has a large discrepancy as explained in Section 4-4. The Djabon-Djombiru record correlogram is shown in Fig. 4-7.

Tables 4-9 and 4-10 show the adjusted records at Kertosono and Djonbiru gauges.

4-8 Water Balance

Table 4-11 shows a water balance in the 1951-1970 period at seven gauges along the Brantas River. The discharge data used are the adjusted records. Column 2 shows the average annual rainfall in the catchment area of each gauge and Column 3 shows the average annual runoff volume divided by the catchment area. The evapo-transpiration which is the difference between the rainfall and discharge is shown in Column 4. The evapo-transpiration increases with increase of cultivated land toward the downstream reaches, while it is relatively low in the elevated forest which is predominant in the upstream reaches. The proportion of discharge in the rainfall is called the runoff coefficient. It ranges between 58 percent and 42 percent.

4-9 Effect of Reservoirs

4-9-1 Effect of Karangkates Reservoir

The Karangkates dam is being constructed at about three kilometers upstream from Pohgadjih gauge.

The catchment area at Karangkates damsite is 2,050 square kilometers, and the active storage capacity of the reservoir is about 253 million cubic meters. The monthly discharge record at the damsite during the 1951-1970 period is available as shown in Table 4-12. The average discharge at the damsite during the 1951-1970 period is estimated to be 67.6 cubic meters per second. Based on the monthly discharge record at Karangkates damsite, the increase and the decrease of the monthly discharge after regulation by the reservoir are estimated as shown in Table 4-13, assuming the reservoir operation as shown below:

(1) The reservoir water surface reaches to high water surface elevation of 272.5 meters by the end of May.

(2) The reservoir water surface reaches to low water surface elevation of 246 meters by the end of November.

(3) The water in the reservoir during the period of June to November is constantly released to the downstream reaches through the generater.

Table 4-13 shows that the average increase of discharge during the dry season from May to October is about 13.5 cubic meters per second.

The discharge duration curves at Djombiru, Kertosono and Djabon gauges drawn on the assumption that the water released from Karangkates reservoir flows to the downstream reaches without any loss are shown in Figs 4-8, 4-9 and 4-10.

4-9-2 Effect of Soeloredjo Reservoir

The Soeloredjo dam was already constructed at the upstream reaches of the Konto River.

The catchment area at Soeloredjo damsite is 236 square kilometers, and the active storage capacity of the reservoir is 55 million cubic meters. The monthly discharge record at the damsite during the 1951-1970 period is available as shown in Table 4-14. The average discharge at the damsite during the 1951-1970 period is estimated to be 10.1 cubic meters per second.

In accordance with the operation rule in a MPWP report,[/1] the increase and the decrease of the monthly discharge after regulation by the reservoir are estimated as shown in Table 4-15.

Table 4-15 shows that the average increase of discharge during the dry season from May to October is about 1.7 cubic meters per second.

4-10 Conclusions

(1) The 1951-1970 discharge records were evaluated for seven water gauges in the Brantas River; the Pohgdjih, Kaulon, Pakel, Djeli, Djombiru, Kertosono and Djabon gauges. The results of a check measurement, analysis of the rainfall-discharge relationship and comparison of records showed that the Djabon record is the most reliable and the 1964-1970 records at the Djombiru and Kertosono gauges are unreliable. The unreliable records were adjusted based on the correlations between the discharges at the assigned gauge and the Djabon gauge. Part of the Brantas flow is diverted from upstream of the Djabon gauge to the Surabaja River by a flood way. The annual volume of the diverted flow was estimated based on a water level record at the inlet of the flood way and added to the measured discharge at the Djabon gauge in estimating the average annual discharge. The water balance after these adjustment is summarized in Table 4-11.

(2) The annual discharge in the Brantas River Basin is estimated to be 858 millimeters (8.3 billion cubic meters) at the Djabon gauge where the catchment area is 88 percent of the whole basin area. Comparing this figure with the annual rainfall 2,041 millimeters, the annual evapo-transpiration is estimated to be 1,183 millimeters.

(3) The monthly mean discharge becomes the maximum in February/March and the minimum in September, respectively. About 83 percent of the annual discharge volume occurs in seven-month period from November to May, and 17 percent runs in the remaining five-month period.

(4) The Karangkates dam is being constructed at the upstream reaches of the Brantas River. The Soeloredjo dam was already constructed at

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/1 MPWP "Regulations for Operation of Soeloredjo Dam of Soeloredjo Project" Nippon Koei Co., Ltd., Tokyo, 1970.

the upstream reaches of Konto River. The effect of these reservoirs is studied in this chapter. Average discharge increased by the regulation of the reservoir during the dry season from May to October is estimated to be 13.5 cubic meters per second at Karangkates damsite and 1.7 cubic meters per second at Soeloredjo damsite. The discharge duration curve before and after the regulation of Karangkates reservoir at the Djombiru, Kertosono and Djabon gauges are shown in Figs. 4-8, 4-9 and 4-10, respectively.

(5) The discharge records were adjusted to such an extent that remarkable discrepancy could be eliminated and the whole records could be used for preliminary planning of water use and control. Some discrepancies of the records remains even after the adjustment. The Djeli record especially disagrees with the Djombiru record. However, this fact was not studied, because the Djeli record was not incorporated into the flood and sedimentation analysis. Notwithstanding these discrepancies, the records presented here must be the best estimate so far based on the presently available data.

(6) The problems of flood and drought will become more serious with more intense land and water uses in the future. In such stages, discharge records with high accuracy must be needed over a long period. The discharge in the Brantas River is largely affected by the inundation, groundwater movement and water use. This suggests difficulties in extraporation of records over both area and period. The accuracy of the actual measurement is particularly important. Evidences of the inaccuracy of the present measurement has been discribed in this chapter. It is recommended that the regular measurement should be periodically checked up by a current meter measurement to correct the inaccuracy.

Table 4-1 Monthly Mean Discharge Record at Pohgadjih

(Unit: m^3/sec)

Year	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sep.	Oct.	Nov.	Dec.	Total	Mean
1951	135.9	167.8	85.3	61.0	51.9	57.3	37.3	30.6	27.7	21.4	22.3	66.9	765.4	63.8
1952	105.7	140.6	153.4	75.2	52.3	38.4	27.4	25.5	22.9	34.1	89.0	74.1	838.6	69.9
1953	71.6	69.2	73.5	77.9	93.9	51.1	41.7	27.0	23.1	21.4	35.3	56.6	642.3	53.5
1954	97.1	94.5	78.9	81.5	89.7	68.9	44.9	46.5	32.1	39.3	118.7	161.2	953.3	79.4
1955	118.7	128.3	104.5	100.4	97.0	95.3	132.9	90.2	73.3	82.3	151.3	130.2	1,304.4	108.7
1956	151.0	122.6	94.0	96.8	83.0	85.6	80.7	67.8	51.3	51.8	58.4	99.3	1,042.3	86.9
1957	88.1	121.4	173.6	106.4	82.0	52.3	92.5	60.5	42.2	36.7	44.0	77.6	977.3	81.4
1958	69.6	83.4	102.7	108.7	81.5	77.1	70.7	57.6	45.1	50.5	53.2	118.2	918.3	76.5
1959	108.7	126.7	126.8	89.1	92.1	75.5	55.2	46.1	46.2	39.4	51.6	115.2	972.6	81.0
1960	96.6	103.6	121.7	114.5	122.2	79.3	70.3	56.5	50.5	46.5	70.6	60.1	992.4	82.7
1961	122.7	86.6	67.1	73.6	74.3	52.7	52.7	47.7	41.2	33.2	44.2	-	696.0	63.3
1962	104.3	90.9	87.8	93.9	68.5	60.0	49.5	48.7	36.0	42.1	63.1	114.7	859.5	71.6
1963	104.3	90.9	87.8	93.9	68.5	60.0	49.5	48.1	36.0	42.1	63.1	114.7	858.9	71.6
1964	58.8	62.0	93.6	77.5	74.5	62.5	47.5	42.5	46.2	107.7	89.8	80.7	843.3	70.3
1965	114.3	129.3	106.5	93.4	70.3	61.2	52.9	47.3	40.4	39.8	44.9	86.1	886.4	73.9
1966	88.2	124.0	135.5	112.3	91.3	74.2	55.8	46.5	43.0	52.5	71.0	132.7	1,027.0	85.6
1967	162.1	139.2	103.1	99.6	67.4	58.2	51.0	46.2	42.1	41.8	48.7	125.2	984.6	82.0
1968	146.0	243.3	110.2	111.8	175.7	157.9	155.6	100.3	66.8	88.2	92.5	144.6	1,592.9	132.7
1969	164.9	163.0	207.2	193.9	101.0	81.0	60.0	49.1	45.5	38.3	48.7	67.3	1,219.9	101.7
1970	91.9	107.4	120.6	94.4	93.0	61.8	50.4	37.5	38.4	42.5	59.4	83.9	881.2	73.4
Total	2200.5	2394.7	2233.8	1955.8	1730.1	1410.3	1278.5	1022.2	850.0	951.6	1319.8	1909.3	19,256.6	80.6
Mean	110.0	119.7	111.7	97.8	86.5	70.5	63.9	51.1	42.5	47.6	66.0	100.5		

Table 4-2 Monthly Mean Discharge Record at Kaulon

(Unit: m^3/sec)

Year	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sep.	Oct.	Nov.	Dec.	Total	Mean
1951	166.9	175.2	125.9	97.3	87.0	91.3	62.3	53.2	46.0	42.3	39.1	90.8	1,077.3	89.8
1952	155.3	193.4	169.3	109.6	84.5	62.7	54.3	45.0	39.4	53.4	128.7	110.1	1,205.7	100.5
1953	108.4	103.2	123.6	111.5	141.4	75.8	78.1	48.8	45.2	40.7	54.8	81.2	1,012.7	84.4
1954	154.8	157.2	125.2	118.4	131.2	115.1	78.1	72.0	60.3	61.8	155.0	198.6	1,427.7	119.0
1955	122.8	141.2	108.7	126.3	118.7	120.7	152.6	125.5	89.7	98.1	161.5	101.3	1,467.1	122.3
1956	166.9	143.9	119.7	93.1	99.0	99.1	95.9	80.8	61.5	63.3	80.6	114.0	1,217.8	101.5
1957	115.5	141.8	189.3	123.4	92.7	70.3	143.4	83.7	51.7	45.9	51.8	109.4	1,218.9	101.6
1958	91.2	121.3	137.5	138.3	107.9	83.8	99.7	61.8	46.2	64.6	74.8	164.6	1,191.7	99.3
1959	146.7	132.0	148.5	112.8	108.3	90.5	66.3	46.2	44.2	38.0	51.2	158.6	1,143.3	95.3
1960	135.0	118.2	140.8	141.0	141.9	92.3	83.1	52.7	42.5	38.8	82.6	85.7	1,154.6	96.2
1961	142.5	121.8	93.5	107.8	99.1	70.8	52.5	40.4	33.8	32.7	54.9	82.6	932.4	77.7
1962	130.2	123.8	111.9	118.9	88.4	71.8	61.4	67.9	60.6	58.0	93.2	114.8	1,100.9	91.7
1963	122.2	121.8	143.0	149.5	90.0	87.3	63.5	54.4	52.9	57.7	61.9	102.5	1,106.7	92.2
1964	104.8	117.7	150.4	121.9	144.5	124.5	87.1	73.3	72.5	172.0	144.3	118.7	1,431.7	119.3
1965	156.1	154.5	117.3	105.0	57.3	56.7	46.6	38.9	34.0	31.9	39.9	71.7	909.9	75.8
1966	75.1	141.4	171.5	118.1	102.6	89.7	72.6	71.3	43.5	67.7	92.6	167.8	1,213.9	101.2
1967	146.2	123.7	134.7	158.1	105.0	91.4	77.5	47.6	37.4	52.9	96.6	137.7	1,208.8	100.7
1968	192.1	112.3	113.9	106.5	129.1	93.6	-	-	54.6	66.8	86.9	121.4	1,077.2	107.7
1969	144.3	100.3	184.3	184.2	121.1	123.6	59.5	48.5	59.2	57.1	63.8	93.5	1,239.4	103.3
1970	131.3	126.0	148.4	165.7	117.3	110.1	103.2	74.4	90.0	75.8	108.0	158.2	1,408.4	117.4
Total	2708.3	2670.7	2757.4	2507.4	2167.0	1821.1	1537.7	1186.4	1065.2	1219.5	1722.2	2383.2	23,746.1	99.8
Mean	135.4	133.5	137.9	125.4	108.3	91.1	80.9	62.4	53.3	61.0	86.1	119.2		

Table 4-3 Monthly Mean Discharge Record at Pakel

(Unit: m^3/sec)

Year	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sep.	Oct.	Nov.	Dec.	Total	Mean
1951	212.4	296.0	194.6	155.1	117.1	127.9	93.3	69.6	56.7	49.8	47.5	140.8	1,560.8	130.1
1952	195.3	236.9	303.2	180.5	110.1	100.0	68.3	37.7	57.3	59.0	197.6	157.7	1,703.6	142.0
1953	146.0	156.6	173.1	182.6	227.5	104.6	79.6	56.5	50.3	42.0	59.4	90.9	1,369.1	114.1
1954	132.1	179.6	181.4	215.7	223.8	156.0	104.9	98.0	72.9	73.0	179.3	287.7	1,904.4	158.7
1955	226.9	230.6	210.1	242.3	209.5	194.4	228.9	168.3	129.0	94.3	210.7	184.6	2,329.6	194.1
1956	216.7	198.6	164.8	138.7	153.8	173.6	160.8	142.3	100.4	111.0	123.3	186.8	1,870.8	155.9
1957	177.7	252.0	241.4	186.7	158.8	94.0	162.4	96.5	60.9	42.3	55.3	119.9	1,647.9	137.3
1958	101.6	181.4	213.2	217.3	151.5	122.2	141.6	94.6	61.4	69.7	98.5	224.7	1,677.7	139.8
1959	226.1	229.7	260.9	145.5	186.2	133.5	79.1	58.4	51.2	45.8	65.2	191.6	1,673.2	139.4
1960	170.3	220.8	245.4	200.7	233.2	123.8	108.3	72.5	58.5	54.4	117.3	98.7	1,703.9	142.0
1961	194.1	156.8	136.0	156.5	137.5	81.6	60.3	45.5	38.4	36.5	50.9	102.1	1,196.2	99.7
1962	202.5	174.6	176.0	214.8	155.3	98.7	77.8	64.6	48.8	45.8	98.2	212.8	1,569.9	130.8
1963	232.1	213.5	254.3	170.8	94.1	68.8	63.2	53.5	44.4	40.2	44.1	75.7	1,354.7	112.9
1964	68.7	96.0	165.1	162.7	195.5	159.2	62.0	58.4	64.1	117.8	111.0	127.3	1,387.8	115.6
1965	214.4	221.5	196.6	169.0	84.5	73.4	63.0	57.1	42.9	39.3	64.9	121.6	1,348.2	112.3
1966	134.9	169.4	210.2	183.0	133.9	84.2	67.2	62.1	51.2	65.7	91.0	132.5	1,385.3	115.4
1967	229.8	178.1	176.2	195.8	111.8	72.0	43.3	47.1	46.0	44.3	61.9	160.1	1,366.4	113.9
1968	168.6	161.1	174.3	170.0	232.6	238.6	208.1	111.3	98.6	124.7	176.8	212.3	2,077.0	173.1
1969	217.2	182.0	281.0	221.0	117.7	115.5	76.0	71.8	62.5	62.0	108.7	148.0	1,663.4	138.6
1970	187.2	177.5	226.3	195.3	147.7	114.3	75.4	53.4	60.4	73.2	132.0	156.3	1,599.0	133.2
Total	3654.6	3912.7	4184.1	3704.0	3182.1	2436.3	2023.5	1519.2	1255.9	1290.8	2093.6	3132.1	32,388.9	135.0
Mean	182.7	195.6	209.2	185.2	159.1	121.8	101.2	76.0	62.8	64.5	104.7	156.6		

Table 4-4 Monthly Mean Discharge Record at Djeli

(Unit: m^3/sec)

Year	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sep.	Oct.	Nov.	Dec.	Total	Mean
1951	301.6	365.7	277.9	171.0	117.7	189.4	114.4	80.6	76.7	56.6	59.2	178.5	1,989.3	165.8
1952	331.0	449.2	504.9	241.9	141.0	109.8	72.4	65.3	59.3	84.1	342.2	268.5	2,669.6	222.5
1953	190.0	251.0	229.9	225.3	271.4	115.8	90.8	57.6	45.4	39.2	61.8	117.0	1,695.2	141.3
1954	216.9	240.6	199.5	224.3	306.5	195.6	121.9	108.7	85.1	74.8	353.8	401.4	2,529.1	210.8
1955	307.5	320.3	239.6	362.0	285.2	249.0	447.0	292.8	193.7	218.0	473.8	277.6	3,666.5	305.5
1956	372.7	286.6	243.0	181.3	211.4	215.9	212.6	177.2	114.8	126.0	153.9	348.4	2,643.8	220.3
1957	255.8	338.0	376.0	284.6	184.0	102.6	343.8	137.7	76.1	57.2	66.7	130.1	2,352.6	196.0
1958	146.4	266.1	304.0	296.3	195.5	145.5	188.8	130.8	77.1	109.1	147.9	293.5	2,301.0	191.7
1959	450.8	433.9	448.8	264.5	297.5	220.1	120.1	63.1	68.7	54.4	95.1	306.5	2,823.5	235.3
1960	303.0	293.3	356.6	267.9	344.0	193.4	134.5	88.1	65.2	57.2	158.3	145.9	2,397.4	199.8
1961	294.6	240.1	202.8	206.7	192.3	92.1	59.3	51.6	40.7	41.8	62.5	126.7	1,611.2	134.3
1962	286.8	258.8	242.0	328.7	209.4	106.9	90.5	76.4	50.4	60.5	192.0	250.6	2,253.0	187.7
1963	432.4	349.0	522.5	321.0	139.8	85.0	68.1	54.4	45.6	46.4	55.0	112.0	2,231.2	185.9
1964	102.5	122.8	297.3	302.2	-	157.6	64.3	55.4	84.6	366.8	207.1	156.8	1,917.4	174.3
1965	285.8	357.1	269.1	204.2	113.8	81.8	61.0	53.0	49.7	49.7	65.7	-	1,590.9	144.6
1966	171.8	235.0	383.5	344.2	174.9	126.2	75.0	68.2	55.4	80.1	100.7	257.3	2,072.3	172.7
1967	244.5	-	186.8	191.0	134.4	86.8	61.8	54.5	44.6	43.5	65.0	157.8	1,270.7	115.5
1968	209.9	188.2	288.3	260.2	306.6	302.0	295.0	179.3	112.1	162.8	179.6	301.9	2,785.9	232.2
1969	255.6	242.6	351.4	279.0	182.1	143.0	84.0	67.2	58.3	67.2	104.9	155.5	1,990.8	165.9
1970	-	-	-	-	-	-	-	-	-	-	-	-	0	0
Total	5159.6	5238.3	5923.9	4956.3	3807.5	2908.5	2705.3	1861.9	1403.5	1795.4	2945.2	4086.0	42,791.4	190.2
Mean	271.6	291.0	311.8	260.9	211.5	153.1	142.4	98.0	73.9	94.5	155.0	227.0		

Table 4-5 Monthly Mean Discharge Record at Djombiru

(Unit: m^3/sec)

Year	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sep.	Oct.	Nov.	Dec.	Total	Mean
1951	290.1	334.6	229.1	129.0	103.2	155.7	96.9	73.9	60.4	43.9	39.9	153.2	1,709.9	142.5
1952	267.2	366.5	419.0	203.5	129.2	96.1	41.5	49.7	46.1	73.4	281.3	236.9	2,210.4	184.2
1953	171.4	213.9	262.4	211.4	336.0	119.3	82.2	52.9	41.8	35.4	58.5	120.6	1,705.8	142.1
1954	257.9	257.4	225.1	236.9	291.8	178.6	117.3	116.7	81.6	76.0	335.0	436.3	2,610.6	217.5
1955	279.3	287.6	210.2	266.5	221.2	218.9	325.8	230.2	149.7	169.4	393.9	214.4	2,967.1	247.3
1956	253.8	235.3	189.8	151.5	156.6	170.6	188.0	152.0	93.1	117.9	132.8	252.2	2,093.6	174.5
1957	197.2	282.6	452.9	248.5	167.1	100.1	295.3	123.1	77.6	65.5	73.5	185.5	2,268.9	189.1
1958	152.3	248.5	267.7	280.9	190.6	147.7	170.7	123.4	75.6	114.0	159.2	280.0	2,210.6	184.2
1959	343.9	318.7	374.7	228.5	283.4	225.6	153.6	96.3	96.2	81.5	128.9	289.8	2,621.1	218.4
1960	281.1	312.2	322.9	293.3	394.5	201.7	152.9	105.7	89.8	85.0	185.0	168.7	2,592.8	216.1
1961	280.8	237.1	208.3	224.9	201.3	109.7	90.6	75.5	70.1	77.7	73.3	137.6	1,786.9	148.9
1962	261.1	248.7	247.6	280.5	195.0	131.0	114.4	91.2	71.8	86.6	158.3	354.7	2,240.9	186.7
1963	346.1	351.2	459.7	307.1	150.3	128.2	100.0	82.0	74.8	72.5	84.4	130.9	2,287.2	190.6
1964	119.2	142.4	233.6	230.8	230.3	189.7	94.6	78.9	100.8	306.0	179.1	160.9	2,066.3	172.2
1965	278.4	368.0	269.3	195.3	130.9	108.8	95.0	94.5	79.2	69.8	97.0	176.6	1,962.8	163.6
1966	207.5	301.6	339.2	313.0	188.5	153.3	110.5	103.7	81.1	114.3	167.6	330.5	2,410.8	200.9
1967	328.1	283.1	218.2	227.6	106.1	74.1	72.9	79.2	72.4	77.3	96.1	209.4	1,844.5	153.7
1968	232.5	235.3	396.5	284.4	363.5	319.7	373.6	212.9	141.1	186.5	234.1	355.6	3,335.7	278.0
1969	325.4	294.7	384.4	390.0	184.1	175.3	114.8	108.1	111.4	113.2	152.4	217.5	2,571.3	214.3
1970	273.7	299.1	343.1	319.9	302.4	222.7	140.7	120.0	129.5	133.1	216.8	290.2	2,791.2	232.6
Total	5147.0	5618.5	6053.7	5023.5	4326.0	3226.8	2931.3	2169.9	1744.1	2099.0	3247.1	4701.5	46.288.4	192.9
Mean	257.3	280.9	302.7	251.2	216.3	161.3	146.6	108.5	87.2	104.9	162.4	235.1		

Table 4-6 Monthly Mean Discharge Record at Kertosono

(Unit: m^3/sec)

Year	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sep.	Oct.	Nov.	Dec.	Total	Mean
1951	323.9	424.8	289.5	148.6	109.9	157.1	106.1	74.7	51.3	36.9	35.8	136.5	1,895.1	157.9
1952	284.4	447.1	510.8	231.1	134.5	117.7	48.9	27.9	34.1	57.7	295.5	223.5	2,413.2	201.1
1953	209.6	263.2	319.3	272.0	377.0	116.6	85.3	52.9	36.1	32.8	53.9	102.4	1,921.1	160.1
1954	269.8	286.5	227.1	226.6	299.4	193.2	125.9	125.9	69.8	61.9	337.3	479.4	2,702.8	225.2
1955	334.4	345.7	295.3	304.9	267.1	241.5	390.2	270.4	177.0	186.0	424.9	258.8	3,496.2	291.3
1956	325.0	311.6	238.0	163.1	185.4	217.9	234.2	184.4	114.7	129.6	162.6	330.8	2,597.3	216.4
1957	252.1	301.1	487.4	308.1	225.8	117.5	338.3	191.8	93.3	67.7	87.0	250.6	2,720.7	226.7
1958	160.0	294.5	302.6	293.7	210.6	154.7	148.9	122.5	82.7	120.3	160.6	294.3	2,345.4	195.4
1959	447.0	385.5	455.5	323.7	392.0	350.9	227.5	137.7	96.8	63.5	91.7	297.0	3,268.8	272.4
1960	363.0	213.5	352.4	293.9	395.3	210.4	199.3	117.6	84.8	70.3	200.8	168.1	2,669.4	222.4
1961	292.4	269.5	269.1	261.5	212.7	130.1	108.6	82.0	60.8	52.9	58.8	132.1	1,930.5	160.9
1962	292.3	301.1	289.2	376.9	288.8	152.1	111.6	69.4	50.5	61.3	168.0	359.9	2,521.1	210.1
1963	444.5	425.8	506.6	341.8	201.8	122.6	100.5	89.4	67.2	77.8	82.2	136.6	2,596.8	216.4
1964	181.0	213.5	414.3	350.6	309.4	299.0	140.6	105.6	121.4	384.4	301.4	387.2	3,208.4	267.4
1965	552.4	693.4	473.8	362.0	184.2	167.5	125.5	102.0	96.1	85.0	143.4	274.9	3,260.2	271.7
1966	315.9	441.2	-	-	-	-	-	-	-	-	-	-	757.1	378.5
1967	353.1	279.1	248.4	300.7	163.7	130.7	108.1	108.4	112.6	110.0	164.3	378.5	2,457.6	204.8
1968	358.0	398.6	718.7	544.7	591.0	503.6	574.7	437.3	180.4	242.6	271.3	524.8	5,345.7	445.5
1969	612.2	534.1	573.8	676.7	244.0	214.8	159.9	126.8	123.0	122.6	187.8	294.1	3,869.8	322.5
1970	326.8	394.0	444.7	461.0	435.2	362.4	215.2	147.1	141.6	141.5	208.1	283.3	3,560.9	296.7
Total	6697.8	7223.8	7416.5	6241.6	5227.8	3960.3	3549.3	2573.8	1794.2	2104.8	3435.4	5312.8	55,538.1	241.5
Mean	334.9	361.2	390.3	328.5	275.1	208.4	186.8	135.5	94.4	110.8	180.8	279.6		

Table 4-7 Monthly Mean Discharge Record at Djabon

(Unit: m^3/sec)

Year	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sep.	Oct.	Nov.	Dec.	Total	Mean
1951	494.1	683.6	474.8	157.9	117.1	212.4	101.9	67.1	49.6	36.6	32.4	226.6	2,654.1	221.2
1952	505.7	654.9	776.0	366.0	153.0	119.0	55.3	40.0	39.8	57.4	425.4	463.2	3,655.7	304.6
1953	370.0	395.5	458.4	498.0	660.6	137.1	106.4	45.3	35.2	31.8	54.5	194.5	2,987.3	248.9
1954	507.4	496.4	400.9	365.0	434.0	232.9	108.8	121.8	65.6	69.2	502.4	774.7	4,079.1	339.9
1955	551.9	553.4	519.4	532.6	354.9	318.1	570.8	347.0	176.2	193.7	604.2	333.7	5,055.9	421.3
1956	524.0	505.4	359.3	147.9	184.9	341.9	210.8	164.0	87.1	105.1	139.7	413.6	3,183.7	265.3
1957	345.3	546.2	866.1	351.1	186.2	88.8	359.0	139.6	52.9	38.9	51.9	302.9	3,328.9	277.4
1958	233.0	591.0	569.1	597.2	327.1	141.7	201.3	110.9	51.5	81.5	129.5	392.1	3,425.9	285.5
1959	792.3	715.3	667.2	391.6	423.4	288.3	135.6	56.0	45.1	36.3	69.4	436.3	4,056.8	338.1
1960	536.2	297.2	645.2	460.9	545.4	150.3	132.1	60.7	41.0	38.0	151.7	173.9	3,232.6	269.4
1961	418.3	447.3	322.1	296.3	246.7	87.3	50.0	37.8	31.3	25.6	37.1	109.0	2,108.8	175.7
1962	474.8	499.0	388.7	543.8	306.9	128.3	90.4	56.9	32.7	40.8	147.5	377.8	3,087.6	257.3
1963	597.1	727.4	847.8	494.8	163.2	101.3	56.3	44.2	33.0	30.4	33.4	122.9	3,251.8	271.0
1964	129.8	183.4	473.1	278.6	224.4	240.1	62.1	41.2	44.0	375.9	212.1	155.2	2,419.9	201.7
1965	393.4	479.3	352.1	197.7	75.0	70.4	44.6	32.6	27.0	23.5	33.7	156.8	1,886.1	157.2
1966	263.0	525.6	585.3	411.6	168.4	105.2	54.9	33.8	25.8	45.6	72.1	304.5	2,595.8	216.3
1967	460.0	420.9	372.5	381.4	144.9	59.5	40.9	32.7	27.5	22.6	26.9	164.9	2,154.7	179.6
1968	219.6	404.5	580.0	476.9	473.0	336.9	386.3	196.6	90.9	103.1	168.5	410.2	3,846.5	320.5
1969	385.9	480.7	474.3	482.1	132.8	123.9	53.3	35.7	29.7	33.2	53.5	105.4	2,390.5	199.2
1970	275.6	449.0	302.9	374.8	321.2	145.5	82.1	36.6	42.0	40.3	141.2	163.8	2,375.0	197.9
Total	8477.4	10056.0	10435.2	7806.2	5643.1	3428.9	2902.9	1700.5	1027.9	1429.5	3087.1	5782.0	61,776.7	257.4
Mean	423.9	502.8	521.8	390.3	282.2	171.4	145.1	85.0	51.4	71.5	154.4	289.1		

Table 4-8 Check Measurement Results

Gauge	Date	(1) Check measurement (m^3/s)	(2) Regular measurement (m^3/s)	(3) (1)/(2) (%)
Pakel	July 14	83.2	83.1	100
	Aug. 2	60.5	58.0	104
Djombiru	July 16	98.3	149.5	66
	Aug. 4	69.2	133.5	52
Kertosono	July 15	97.4	204.0	48
	Aug. 4	70.8	161.7	44
Djabon	July 16	81.5	76.5	106
	Aug. 5	49.5	54.4	91

Table 4-9 Adjusted Record at Kertosono

(Unit: m^3/sec)

Year	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sep.	Oct.	Nov.	Dec.	Total	Mean
1951	323.9	424.8	289.5	148.6	109.9	157.1	106.1	74.7	51.3	36.9	35.8	136.5	1,895.1	157.9
1952	284.4	447.1	510.8	231.1	134.5	117.7	48.9	27.9	34.1	57.7	295.5	223.5	2,413.2	201.1
1953	209.6	263.2	319.3	272.0	377.0	116.6	85.3	52.9	36.1	32.8	53.9	102.4	1,921.1	160.1
1954	269.8	286.5	227.1	226.6	299.4	193.2	125.9	125.9	69.8	61.9	337.3	479.4	2,702.8	225.2
1955	334.4	345.7	295.3	304.9	267.1	241.5	390.2	270.4	177.0	186.0	424.9	258.8	3,496.2	291.3
1956	325.0	311.6	238.0	163.1	185.4	217.9	234.2	184.4	114.7	129.6	162.6	330.8	2,597.3	216.4
1957	252.1	301.1	487.4	308.1	225.8	117.5	338.3	191.8	93.3	67.7	87.0	250.6	2,720.7	226.7
1958	160.0	294.5	302.6	293.7	210.6	154.7	148.9	122.5	82.7	120.3	160.6	294.3	2,345.4	195.4
1959	447.0	385.5	455.5	323.7	392.0	350.9	227.5	137.7	96.8	63.5	91.7	297.0	3,268.8	272.4
1960	363.0	-	352.4	293.9	395.3	210.4	199.3	117.6	84.8	70.3	200.8	168.1	2,455.9	223.3
1961	292.4	269.5	269.1	261.5	212.7	130.1	108.6	82.0	60.8	52.9	58.8	132.1	1,930.5	160.9
1962	292.3	301.1	289.2	376.9	288.8	152.1	111.6	69.4	50.5	61.3	168.0	359.9	2,521.1	210.1
1963	444.5	425.8	506.6	341.8	201.8	122.6	100.5	89.4	67.2	77.8	82.2	136.6	2,596.8	216.4
1964	130.5	157.1	300.8	204.3	177.4	185.2	90.9	60.3	64.4	252.5	171.3	143.1	1,937.8	161.5
1965	261.2	303.8	240.7	164.2	103.3	101.1	65.3	47.7	39.5	34.4	49.3	143.9	1,554.4	129.5
1966	196.5	326.8	356.4	270.3	149.6	118.3	80.3	49.5	37.7	66.7	101.9	217.1	1,971.1	164.3
1967	294.3	274.9	250.9	255.3	138.0	87.1	59.8	47.8	40.2	33.1	39.4	147.9	1,668.7	139.1
1968	175.0	266.7	353.8	302.6	300.7	233.2	257.7	163.6	111.2	117.2	149.7	269.6	2,701.0	225.1
1969	257.5	304.5	301.4	305.2	132.0	127.6	78.0	52.2	43.5	48.6	78.3	118.4	1,847.2	153.9
1970	202.8	288.8	216.3	252.0	225.4	138.3	106.8	53.5	61.5	59.0	136.1	147.3	1,887.8	157.3
Total	5516.2	5979.0	6563.1	5299.8	4526.7	3273.1	2964.1	2021.2	1417.1	1630.2	2885.1	4357.3	46,432.9	194.3
Mean	275.8	314.7	328.2	265.0	226.3	163.7	148.2	101.1	70.9	81.5	144.3	217.9		

Table 4-10 Adjusted Record at Djombiru

(Unit: m^3/sec)

Year	Jan.	Geb.	Mar.	Apr.	May	June	July	Aug.	Sep.	Oct.	Nov.	Dec.	Total	Mean
1951	290.1	334.6	229.1	129.0	103.2	155.7	96.9	73.9	60.4	43.9	39.9	153.2	1,709.9	142.5
1952	267.2	366.5	419.0	203.5	129.2	96.1	41.5	49.7	46.1	73.4	281.3	236.9	2,210.4	184.2
1953	171.4	213.9	262.4	211.4	336.0	119.3	82.2	52.9	41.8	35.4	58.5	120.6	1,705.8	142.1
1954	257.9	257.4	225.1	236.9	291.8	178.6	117.3	116.7	81.6	76.0	335.0	436.3	2,610.6	217.5
1955	279.3	287.6	210.2	266.5	221.2	218.9	325.8	230.2	149.7	169.4	393.9	214.4	2,967.1	247.3
1956	253.8	235.3	189.8	151.5	156.6	170.6	188.0	152.0	93.1	117.9	132.8	252.2	2,093.6	174.5
1957	197.2	282.6	452.9	248.5	167.1	100.1	295.3	123.1	77.6	65.5	73.5	185.5	2,268.9	189.1
1958	152.3	248.5	267.7	280.9	190.6	147.7	170.7	123.4	75.6	114.0	159.2	280.0	2,210.6	184.2
1959	343.9	318.7	374.7	228.5	283.4	225.6	153.6	96.3	96.2	81.5	128.9	289.8	2,621.1	218.4
1960	281.1	312.2	322.9	293.3	394.5	201.7	152.9	105.7	89.8	85.0	185.0	168.7	2,592.8	216.1
1961	280.8	237.1	208.3	224.9	201.3	109.7	90.6	75.5	70.1	77.7	73.3	137.6	1,786.9	148.9
1962	261.1	248.7	247.6	280.5	195.0	131.0	114.4	91.2	71.8	86.6	158.3	354.7	2,240.9	186.7
1963	346.1	351.2	459.7	307.1	150.3	128.2	100.0	82.0	74.8	72.5	84.4	130.9	2,287.2	190.6
1964	127.9	149.3	264.8	187.2	165.6	171.9	72.0	47.8	51.0	226.0	160.7	138.0	1,762.2	146.8
1965	233.0	267.3	216.5	155.0	87.0	81.7	51.7	37.8	31.3	27.3	39.1	138.7	1,366.4	113.9
1966	180.0	285.7	309.5	240.3	143.3	118.1	53.7	39.2	29.9	52.9	83.6	197.6	1,744.8	145.4
1967	259.6	244.0	224.7	228.2	133.9	69.0	47.4	37.9	31.9	26.2	31.2	141.9	1,475.9	123.0
1968	163.7	237.4	307.4	266.3	264.8	210.5	230.2	154.5	105.4	117.2	143.3	239.7	2,440.4	203.4
1969	230.0	267.8	265.3	268.4	129.1	125.5	61.8	41.4	34.5	38.5	62.1	118.2	1,642.6	136.9
1970	186.0	255.2	196.9	225.6	204.2	134.1	95.2	42.5	48.7	46.7	132.4	141.4	1,708.9	142.4
Total	4763.4	5401.0	5654.5	4633.5	3948.1	2894.0	2551.2	1773.7	1361.3	1633.6	2756.4	4076.3	41,447.0	172.7
Mean	238.2	270.0	282.7	231.7	197.4	144.7	127.6	88.7	68.1	81.7	137.8	203.8		

Table 4-11 Water Balance

Gauge name	(1) Catchment area (km^2)	(2) Rainfall (mm)	(3) Discharge (mm)	(4) (2) - (3) (mm)	(5) (3)/(2) (%)
Pohgadjih	2,230	2,074	1,157	917	56
Kaulon	2,620	2,092	1,192	900	57
Pakel	3,410	2,156	1,244	912	58
Djeli	5,011	2,192	1,263	929	58
Djombiru	5,910	2,146	927	1,219	43
Kertosono	6,494	2,150	949	1,201	44
Djabon	9,675	2,041	858	1,183	42

Remarks; (1) This table is prepared based on the adjusted average monthly discharge record from September 1951 to October 1970.

(2) Average annual rainfall in 1951-1970 (water year) calculated by isohyetal method.

(3) Average annual discharge in 1951-1970. For the Djabon record, flood way discharge of 100 x $10^6 m^3$ is added.

(4) Average evapo-transpiration in 1951-1970.

(5) Runoff coeffieient.

Table 4-12 Monthly Mean Discharge Record at Karangkates Dam Site

(Unit: m^3/sec)

Year	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sep.	Oct.	Nov.	Dec.	Total	Mean
1951	123.1	153.1	66.1	47.2	40.1	44.5	32.4	27.0	25.3	18.5	20.6	63.7	661.6	55.1
1952	79.7	117.1	143.0	65.6	44.4	31.2	21.7	21.6	19.3	29.8	78.1	62.0	713.5	59.5
1953	62.0	58.8	56.1	65.9	80.6	46.8	36.5	21.7	17.9	17.2	28.8	47.6	539.9	45.0
1954	83.0	66.1	65.6	67.6	80.7	55.0	29.7	41.2	23.7	27.0	90.5	127.7	757.8	63.1
1955	98.9	114.1	90.6	83.6	84.8	86.2	124.3	78.8	62.7	77.6	129.3	121.8	1,152.7	96.1
1956	137.4	109.0	82.5	59.0	61.2	73.7	66.8	61.7	35.6	41.8	58.0	102.4	889.1	74.1
1957	73.8	116.2	168.7	88.1	70.4	42.6	73.1	44.3	28.6	22.9	27.7	73.5	829.9	69.2
1958	55.8	78.6	101.7	99.5	66.6	63.8	47.1	46.0	27.8	31.4	33.2	84.2	735.7	61.3
1959	84.3	121.4	119.4	88.0	83.0	63.8	49.6	35.0	27.9	26.0	43.4	113.3	855.1	71.3
1960	80.9	80.0	95.4	96.4	98.4	68.7	61.1	51.5	47.1	43.3	60.5	51.2	834.5	69.5
1961	107.4	71.2	52.0	56.0	58.1	45.3	43.5	38.5	30.8	30.8	41.2	62.4	637.2	53.1
1962	84.4	73.1	69.4	74.7	49.1	50.4	42.1	42.1	31.6	39.6	54.6	95.3	706.4	58.9
1963	84.6	100.9	110.7	80.3	56.6	52.2	45.4	47.3	43.0	39.1	42.4	65.9	768.4	64.0
1964	54.4	53.1	78.1	59.6	48.7	42.0	42.7	40.7	41.6	74.4	58.0	59.0	652.3	54.4
1965	93.9	110.8	87.7	77.2	65.1	57.0	54.5	44.0	38.6	38.5	40.2	77.8	785.3	65.4
1966	58.5	100.5	94.0	75.8	65.7	66.4	49.7	35.9	36.6	43.4	58.8	89.5	774.8	64.6
1967	146.0	130.1	87.3	79.8	56.6	53.3	48.1	44.1	39.5	39.4	44.4	115.5	884.1	73.7
1968	139.7	143.3	73.3	82.5	141.1	112.1	127.2	93.5	47.9	67.1	47.2	107.5	1,182.4	98.5
1969	153.4	156.6	198.6	188.4	94.0	71.6	55.0	45.3	38.3	32.9	35.9	65.1	1,135.1	94.6
1970	72.1	88.9	106.7	80.8	70.9	52.0	43.5	31.8	32.3	35.3	47.3	66.7	728.4	60.7
Total	1873.3	2042.9	1946.9	1616.0	1416.1	1178.6	1094.0	892.0	696.1	776.1	1040.1	1652.1	16,224.2	67.6
Mean	93.7	102.1	97.3	80.8	70.8	58.9	54.7	44.6	34.8	38.8	52.0	82.6		

Table 4-13 Increase and Decrease of Monthly Mean Discharge after regulation by Karangkates Reservoir

(Unit: m^3/sec)

Year	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sep.	Oct.	Nov.	Dec.	Total	Mean
1951	-33.1	-63.1	0.0	0.0	3.4	0.0	11.1	16.5	18.2	25.0	22.9	0.0	0.9	0.1
1952	0.0	-35.6	-61.5	0.0	0.0	12.9	22.4	22.5	24.8	14.3	0.0	-20.4	-20.6	-1.7
1953	-20.4	-17.2	-14.5	- 24.3	0.0	0.0	7.3	22.1	25.9	26.6	15.0	-1.0	19.5	1.6
1954	-36.4	-19.5	-19.0	-21.0	0.0	0.0	24.9	13.4	30.9	27.6	0.0	-44.1	-43.2	-3.6
1955	-15.3	-30.5	-7.0	0.0	6.2	4.8	-11.0	12.2	28.3	13.4	-38.3	-31.4	-68.6	-5.7
1956	-47.0	-18.6	0.0	0.0	9.1	0.0	3.5	8.6	34.7	28.5	12.3	-11.4	19.7	1.6
1957	0.0	-25.2	-60.4	0.0	0.0	10.0	0.0	8.3	24.0	29.7	24.9	-9.4	1.9	0.2
1958	0.0	-14.5	-37.6	-35.4	0.0	0.0	9.4	10.5	28.7	25.1	23.3	-4.1	5.4	0.5
1959	-4.2	-41.3	-39.3	-7.9	0.0	0.0	6.2	20.8	27.9	29.8	12.4	-39.5	-35.1	-2.9
1960	-7.1	-6.2	-21.6	-22.6	0.0	2.8	10.4	20.0	24.4	28.2	11.0	-3.0	36.3	3.0
1961	-59.2	-23.0	-3.8	-7.8	0.0	9.2	11.0	16.0	23.7	23.7	13.3	-9.0	-5.9	-0.5
1962	-31.0	-19.7	-16.0	-21.3	9.0	7.7	16.0	16.0	26.5	18.5	3.5	-20.3	-11.1	-0.9
1963	-9.6	-25.9	-35.7	-5.3	3.8	8.2	15.0	13.1	17.4	21.3	18.0	-23.1	-2.8	-0.2
1964	-11.6	-10.3	-35.3	-16.8	13.1	19.8	19.1	21.1	20.2	0.0	3.8	0.0	23.1	1.9
1965	-25.8	-42.7	-19.6	-9.1	0.0	4.6	7.1	17.6	23.0	23.1	21.4	-15.0	-15.4	-1.3
1966	4.3	-42.0	-31.2	-13.0	0.0	0.0	14.6	28.4	27.7	20.9	5.5	0.0	15.2	1.3
1967	-56.5	-40.6	0.0	0.0	3.7	7.0	12.2	16.2	20.8	20.9	15.9	-24.5	-24.9	-2.1
1968	-48.7	-23.8	0.0	0.0	0.0	0.0	0.0	0.0	38.5	19.3	39.2	-16.5	8.0	0.7
1969	-62.4	-18.1	0.0	0.0	0.0	0.0	8.3	18.0	25.0	30.4	35.0	-8.6	27.6	2.3
1970	-11.2	-28.0	-45.8	-19.9	-10.0	11.3	13.0	24.7	24.2	21.1	9.2	-10.2	-21.6	-1.8
Total	-475.2	-545.8	-448.3	-204.4	38.3	98.3	200.5	326.0	514.8	447.4	248.3	-291.5	-91.6	-0.4
Mean	-23.8	-27.3	-22.4	-10.2	1.9	4.9	10.0	16.3	25.7	22.4	12.4	-14.6		

Remarks; negative figures show the decrease

Table 4-14 Monthly Mean Discharge Record at Soeloredjo Dam Site

(Unit: m^3/sec)

Year	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sep.	Oct.	Nov.	Dec.	Total	Mean
1951	12.3	16.6	12.6	9.8	6.8	8.0	6.4	4.4	5.1	4.7	6.3	10.9	103.9	8.7
1952	13.7	17.2	14.6	11.5	9.1	7.3	5.2	4.3	4.4	5.3	13.2	13.3	119.1	9.9
1953	11.6	13.9	15.9	12.9	11.5	7.7	6.5	4.6	4.6	3.3	7.2	11.5	111.2	9.3
1954	17.6	14.8	14.0	14.9	12.3	7.8	6.5	6.9	5.8	6.9	13.2	14.3	135.0	11.2
1955	14.3	16.3	15.9	13.5	10.5	9.4	11.2	7.0	5.9	7.3	13.2	11.6	136.1	11.3
1956	13.1	18.1	13.8	11.1	9.6	10.0	6.6	6.6	5.9	7.5	9.7	11.5	123.5	10.3
1957	9.8	15.8	27.5	14.2	11.0	8.8	10.3	8.3	6.8	5.9	16.0	11.9	146.3	12.2
1958	8.0	11.7	11.4	12.1	9.8	8.3	8.3	6.6	5.2	5.9	6.5	10.4	104.2	8.7
1959	22.1	22.6	25.6	14.0	11.7	9.6	8.5	6.9	5.4	4.7	5.7	12.3	149.1	12.4
1960	19.9	33.1	19.2	20.6	15.5	9.1	9.0	7.8	6.6	6.3	7.9	7.3	162.3	13.5
1961	13.9	10.5	11.8	7.8	10.3	8.3	7.3	6.5	5.5	5.2	5.7	5.8	98.6	8.2
1962	16.6	23.7	12.9	15.7	10.7	9.3	9.5	9.2	7.0	7.7	10.3	13.6	146.2	12.2
1963	27.4	25.0	17.8	13.6	10.6	9.4	9.1	7.6	6.7	6.6	6.6	7.9	148.3	12.4
1964	8.0	7.5	9.4	9.5	8.8	6.6	4.7	3.9	4.2	13.0	5.3	6.1	87.0	7.2
1965	15.7	14.9	18.3	10.0	7.5	6.8	6.0	5.2	3.6	3.1	3.3	6.0	100.4	8.4
1966	6.5	9.4	13.0	8.3	6.1	6.2	5.0	4.6	4.2	4.4	5.7	7.6	81.0	6.7
1967	14.4	17.2	9.3	7.7	6.5	4.8	4.3	3.8	4.3	4.4	4.8	18.9	100.4	8.4
1968	14.0	23.4	22.4	15.9	14.3	11.1	10.7	7.2	5.9	6.2	11.0	12.4	154.5	12.9
1969	12.4	13.6	12.8	10.7	7.4	6.0	5.0	3.9	3.7	4.9	7.7	8.7	96.8	8.1
1970	11.5	16.5	16.1	12.2	10.9	8.8	6.7	4.5	5.0	5.7	11.4	11.3	120.6	10.0
Total	282.8	341.8	314.3	246.0	200.9	163.3	146.8	119.8	105.8	119.0	170.7	213.3	2,424.5	10.1
Mean	14.1	17.1	15.7	12.3	10.0	8.2	7.3	6.0	5.3	5.9	8.5	10.7		

Table 4-15 Increase and Decrease of Monthly Mean Discharge after regulation by Soeloredjo Reservoir

(Unit: m^3/sec)

Year	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sep.	Oct.	Nov.	Dec.	Total	Mean
1951	-2.7	-5.2	-0.2	-0.2	-0.7	1.0	2.1	4.6	3.9	4.8	3.0	-5.3	5.1	0.4
1952	-4.1	-9.4	-0.2	-0.2	-2.0	0.7	3.3	4.2	4.5	3.2	-4.7	-6.3	-11.0	-0.9
1953	-1.5	-3.6	-0.2	-0.2	-3.3	1.3	2.0	4.4	4.4	6.2	0.2	-5.9	3.8	0.3
1954	-8.0	-3.3	-0.2	-0.2	-3.3	1.2	2.0	2.1	4.4	3.3	-3.2	-6.3	-11.5	-1.0
1955	-2.0	-0.2	-0.2	-0.2	-3.0	-0.4	-0.2	3.2	4.3	2.9	-3.0	-2.6	-1.4	-0.1
1956	-2.6	-0.5	-0.2	-0.2	-2.1	-1.4	2.9	2.4	4.3	2.7	0.5	-4.5	1.3	0.1
1957	-0.2	-7.1	-0.2	-0.2	-3.2	0.2	-0.6	2.1	3.4	4.3	5.2	-5.6	-1.9	-0.2
1958	1.6	-4.4	-4.7	-0.2	-2.3	-0.3	-0.2	2.9	5.0	4.4	3.4	-4.0	1.2	0.1
1959	-11.0	-0.2	-0.2	-0.2	-3.2	0.1	0.5	2.6	4.8	5.5	4.3	-6.0	-3.0	-0.3
1960	-9.6	-0.2	-0.2	-0.2	-3.4	-0.1	0.0	2.4	3.7	3.9	2.3	-0.6	-2.0	-0.2
1961	-4.2	-1.6	-4.6	-0.2	-2.8	-0.3	1.7	3.1	4.7	5.0	4.2	-0.4	4.6	0.4
1962	-6.6	-10.3	0.4	-0.8	-3.3	-0.2	0.0	0.7	3.2	2.5	-0.1	-3.7	-18.2	-1.5
1963	-0.9	-0.2	-0.2	-0.2	-2.8	-0.7	0.4	2.3	3.5	3.5	3.7	-1.3	7.1	0.6
1964	1.8	-0.3	-3.6	-4.8	-4.1	-0.1	3.3	4.1	4.8	-5.5	5.0	0.4	1.0	0.1
1965	-6.2	-7.2	-1.2	-0.1	-0.9	0.7	2.0	3.3	5.4	5.4	0.1	-0.8	0.5	0.0
1966	0.5	-4.6	-7.2	-3.3	-1.4	-0.6	2.4	3.5	4.8	3.1	0.6	-2.1	-4.3	-0.4
1967	-4.7	-10.4	-1.4	-0.7	-0.4	1.8	3.1	4.1	4.6	1.4	0.7	-13.6	-15.5	-1.3
1968	2.1	-4.0	-3.1	-0.2	-2.3	1.2	-0.2	5.0	4.3	4.0	-0.8	-4.4	1.6	0.1
1969	-2.3	-2.1	-0.2	-0.2	0.1	0.5	0.3	4.1	5.2	2.2	-0.2	-2.5	4.9	0.4
1970	-1.9	-9.5	-4.1	-0.2	-3.2	0.2	2.3	4.5	4.5	3.8	-1.4	-4.8	-9.8	-0.8
Total	-62.5	-84.3	-31.7	-12.7	-47.6	4.8	27.1	65.6	87.7	66.6	19.8	-80.3	-47.7	-0.2
Mean	-3.1	-4.2	-1.6	-0.6	-2.3	0.3	1.4	3.3	4.4	3.3	1.0	-4.0		

Remarks; negative figures show the decrease

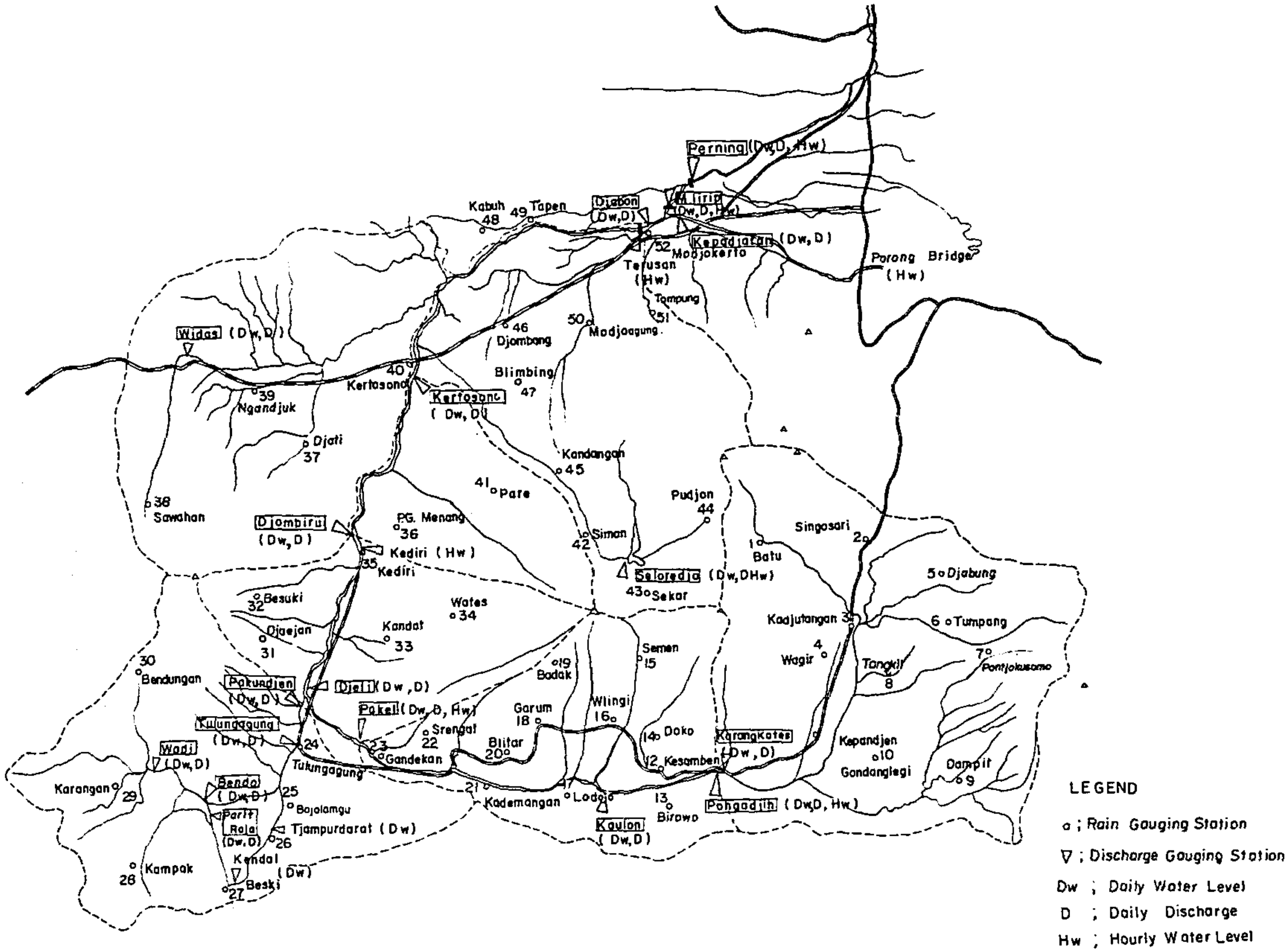


Fig. 4-1 Location Map of Rain and Discharge Gauge

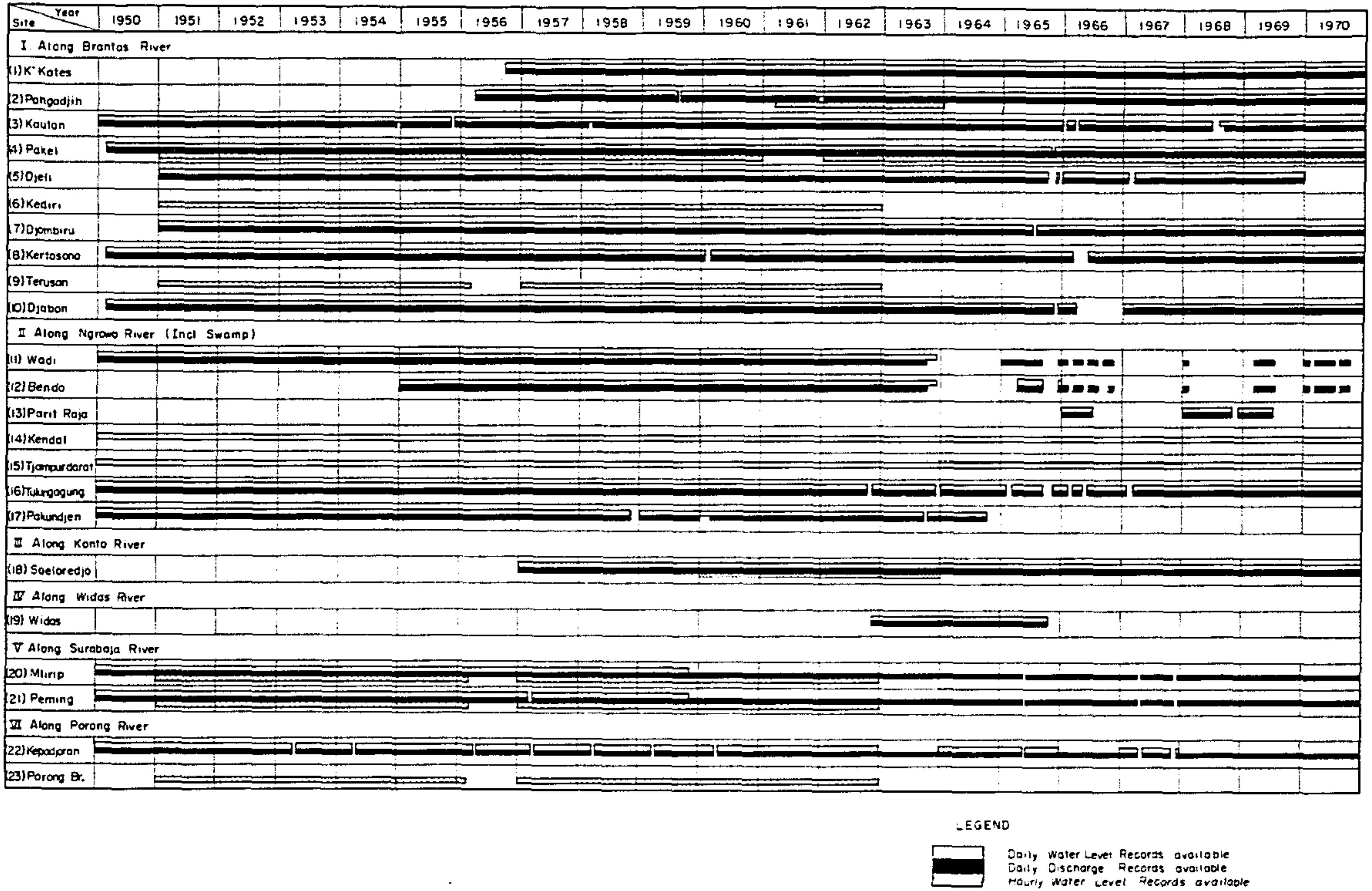


Fig 4-2 Measurement Duration Period of Daily Water Level, Discharge and Hourly Water Level

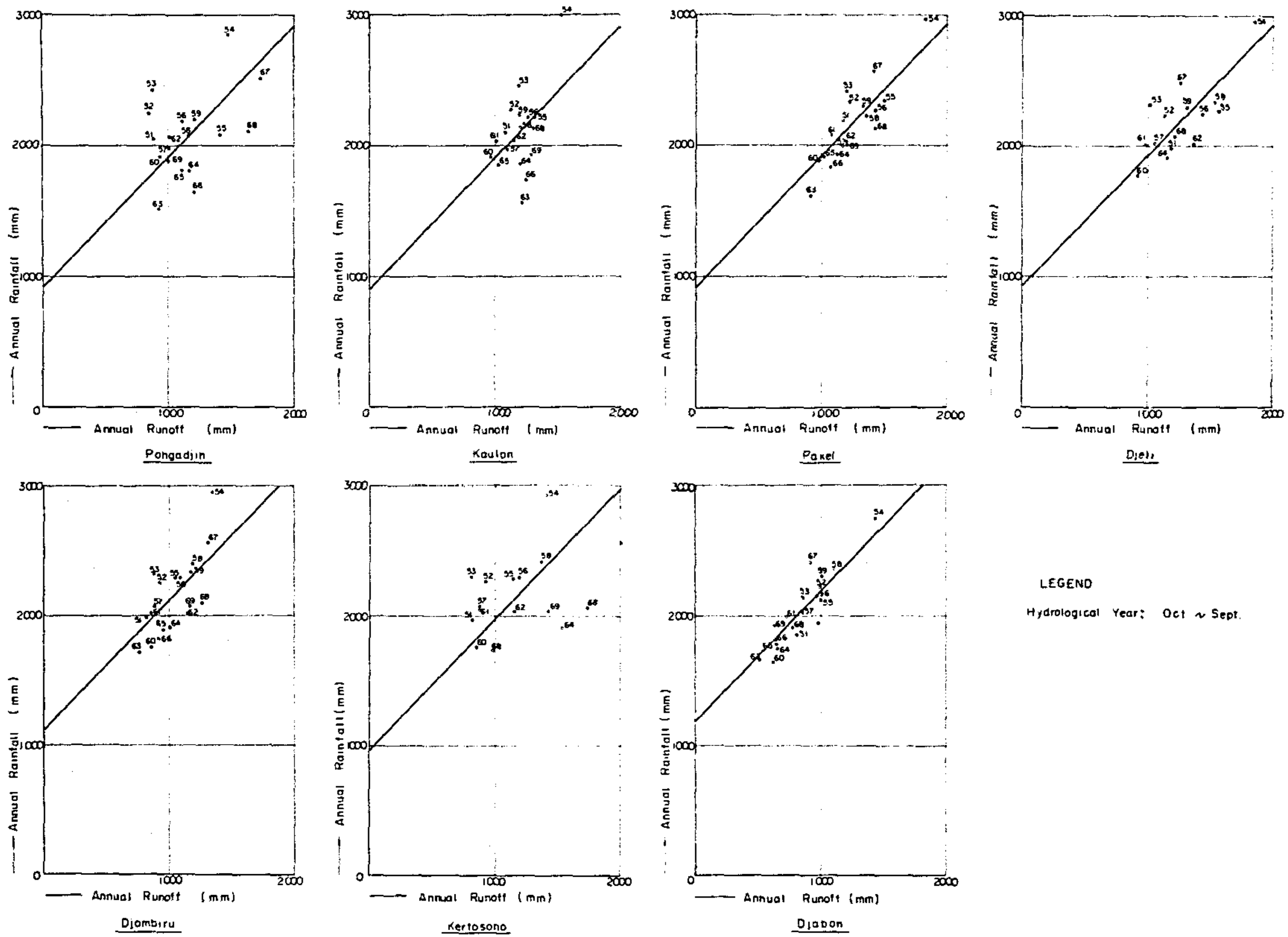


Fig. 4 - 3 Rainfall - Discharge Relationship

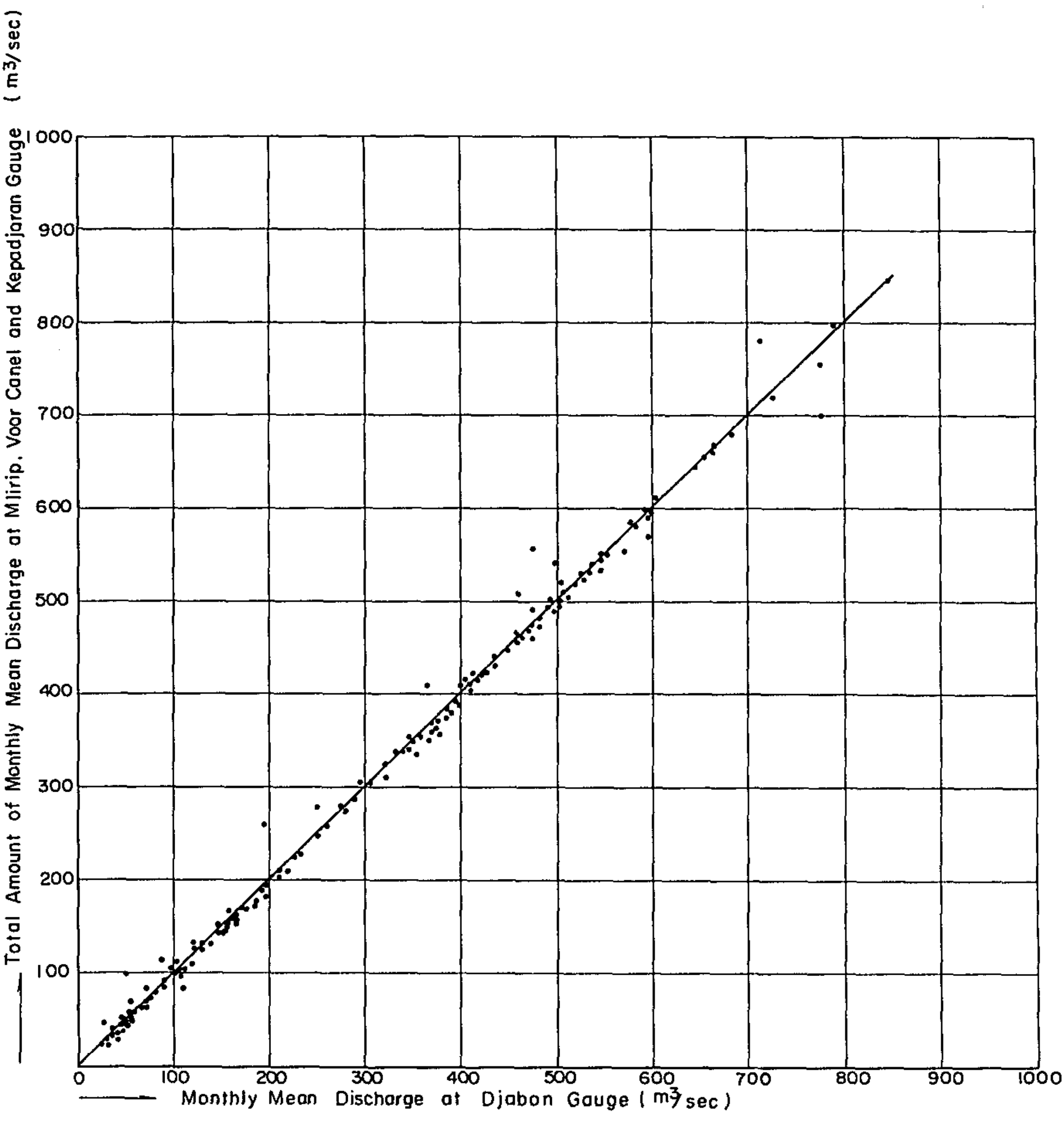


Fig. 4-4 Correlation between Monthly Mean Discharge at Djabon Gauge and Total Amount of Monthly Mean Discharge at Milrip Voor Canal and Kepadjaran Gauge.

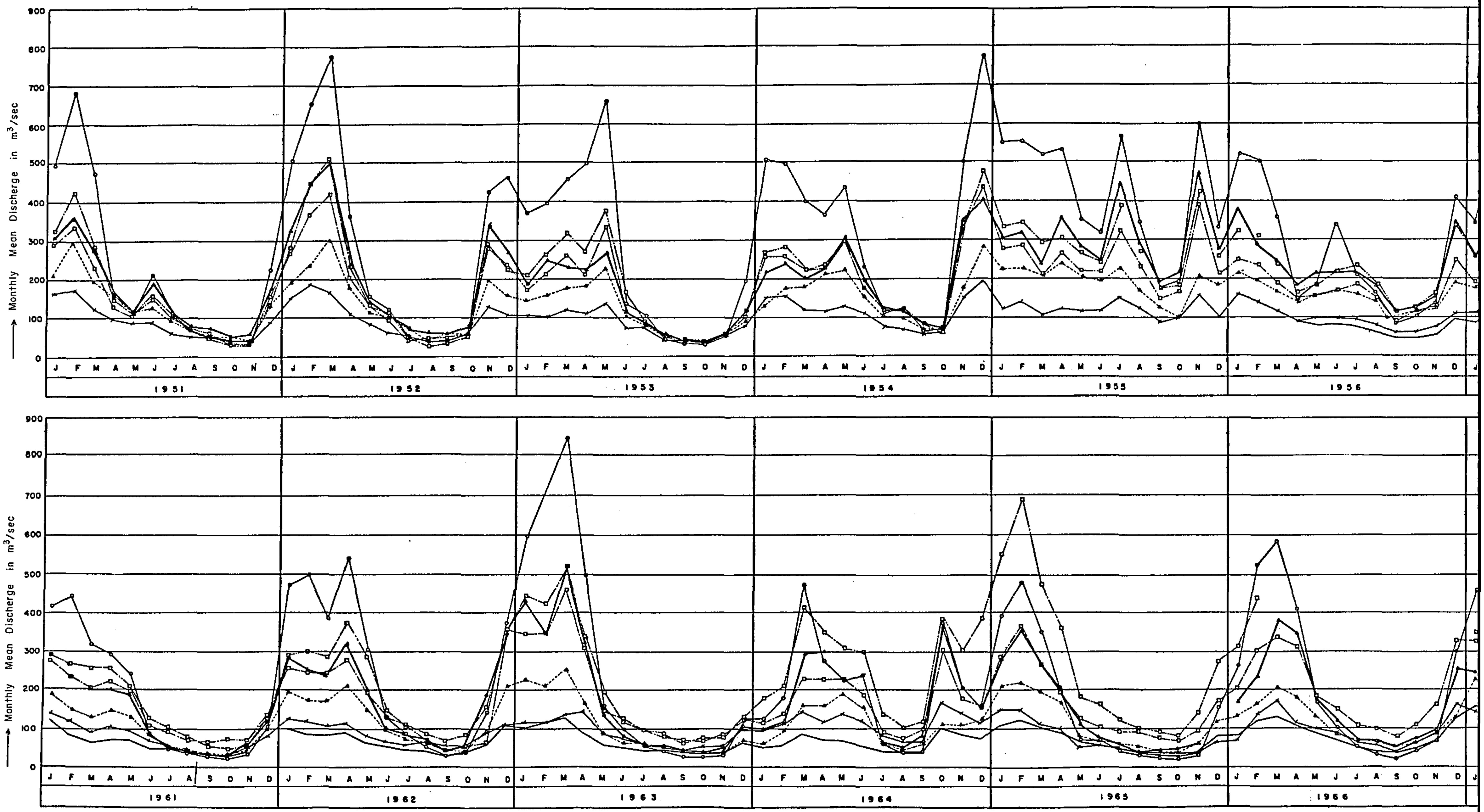
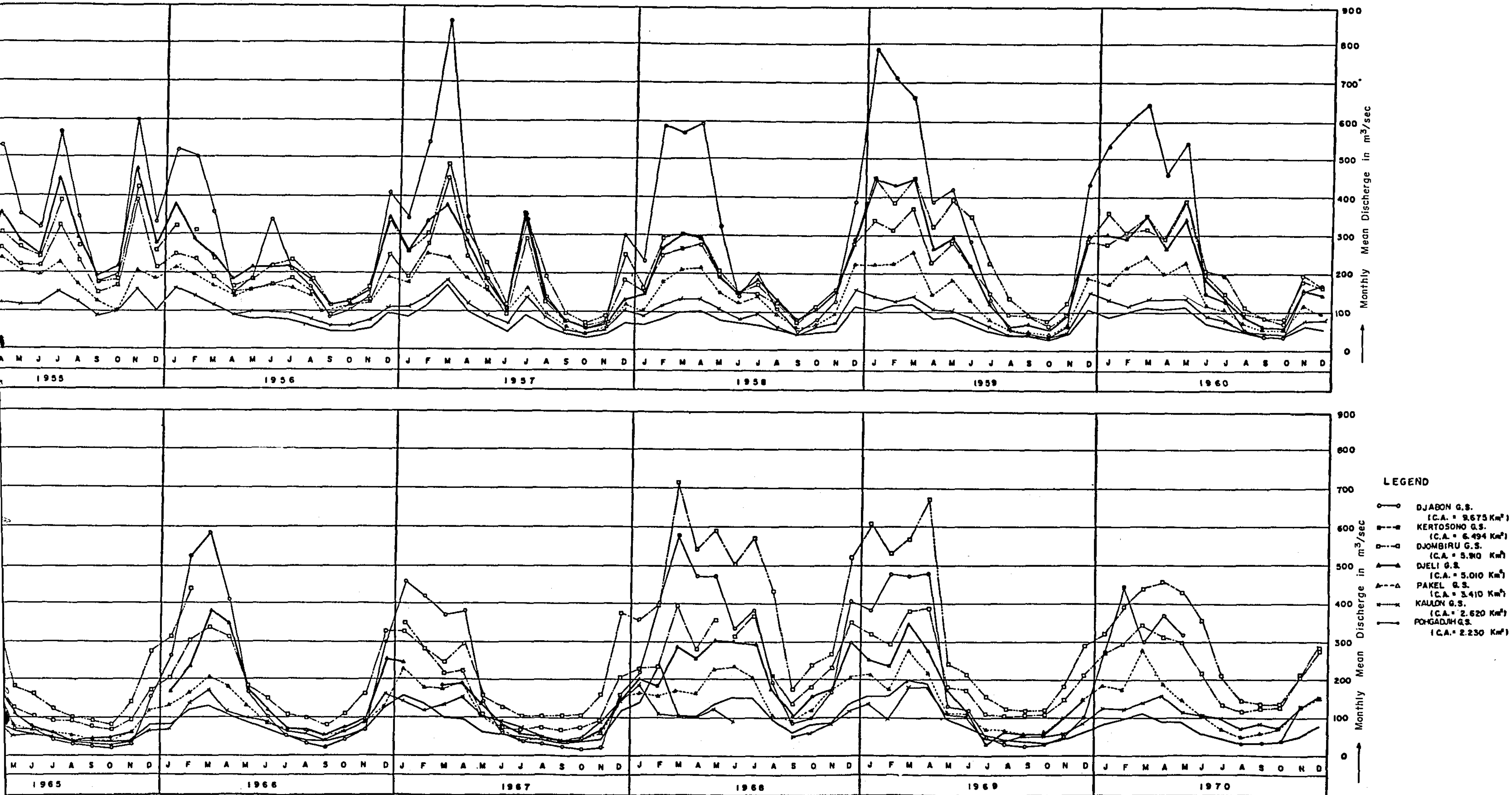


Fig. 4-5 Hydrograph of Original Data Recorded by Indon



Hydrograph of Original Data Recorded by Indonesian Government

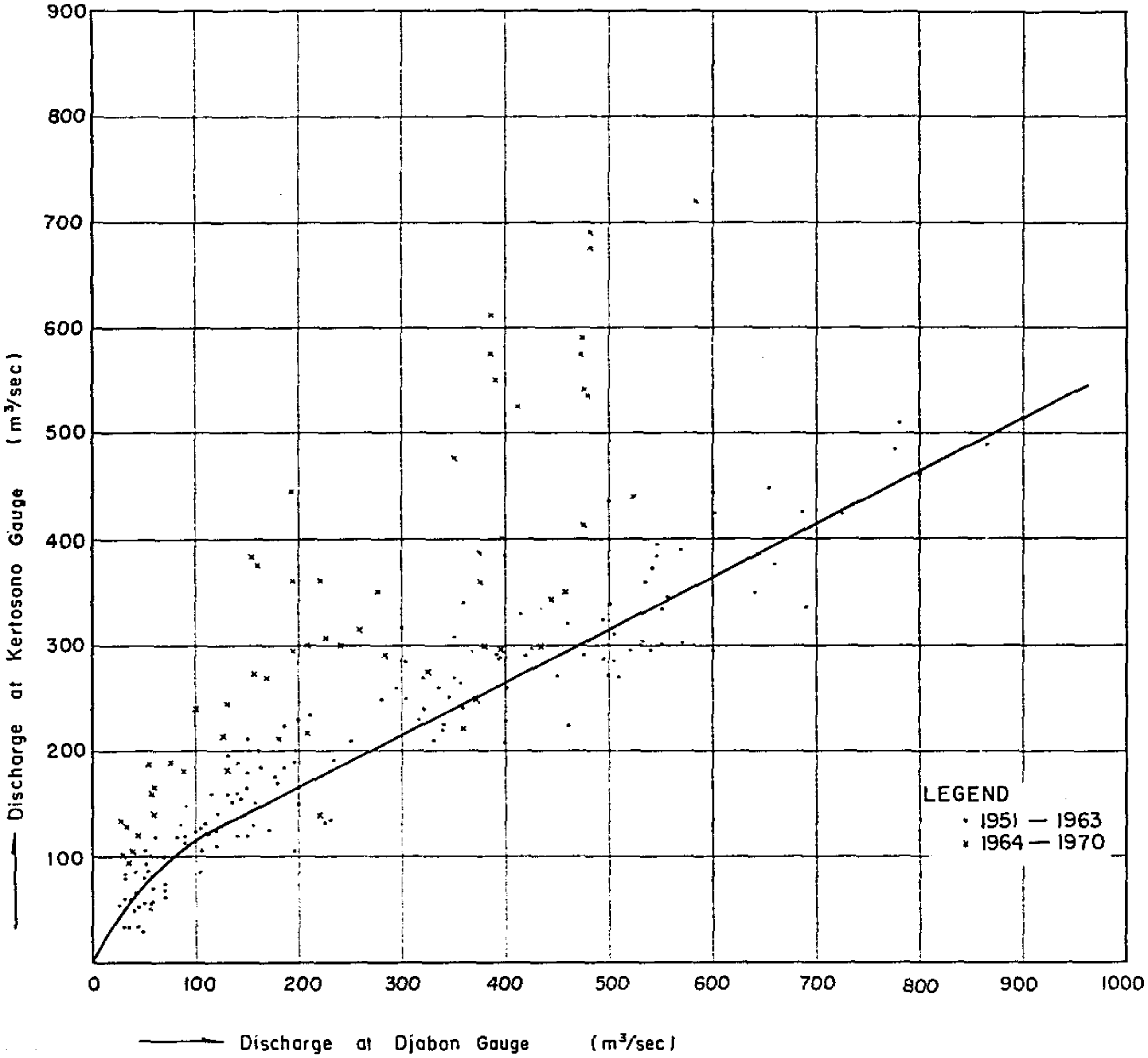
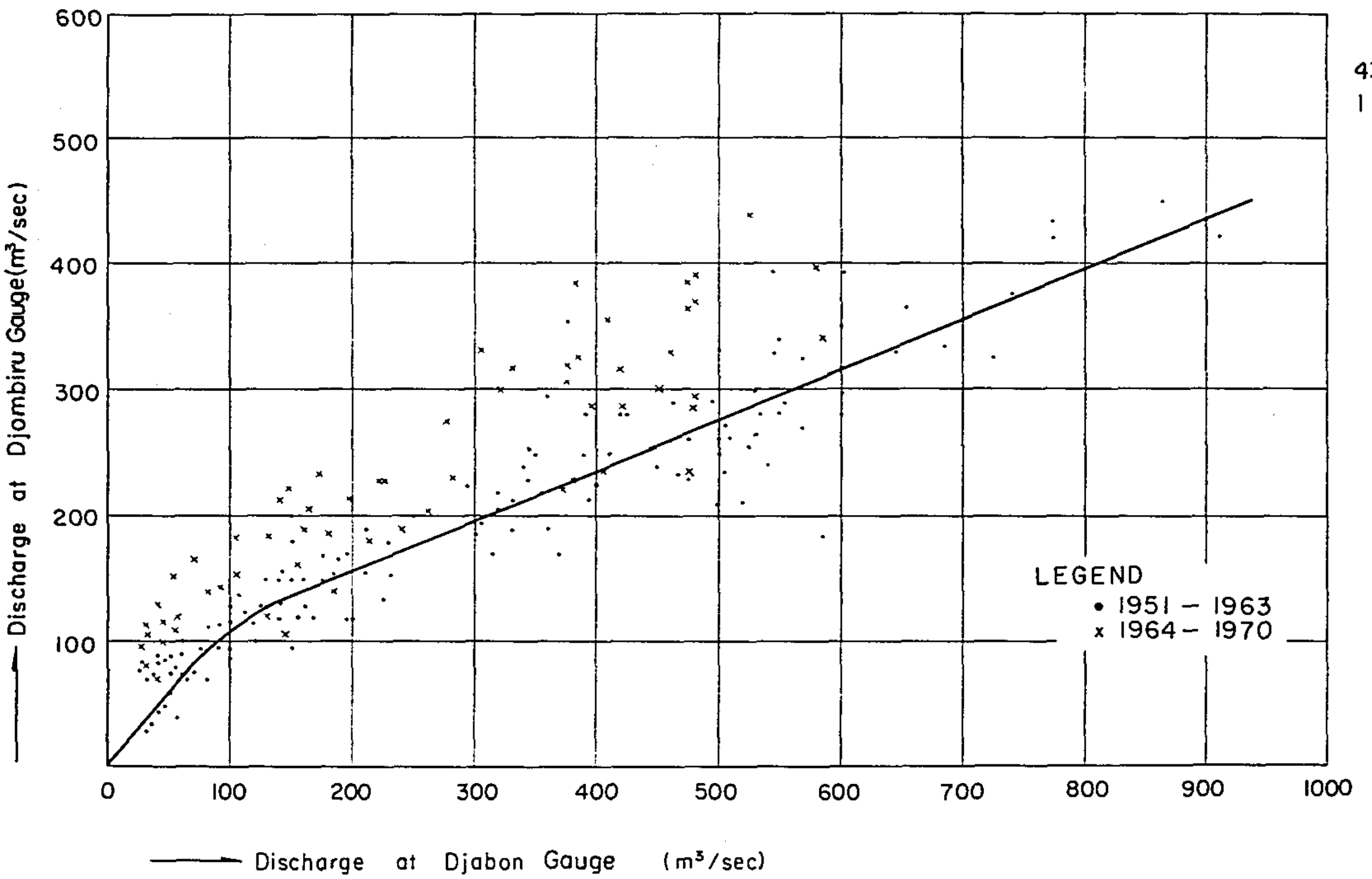


Fig. 4-6 Correlation of Discharge between Kertosono and Djabon from 1951 to 1970



$435 = a \times 900 + b$
$116 = a \times 100 + b$
$a = 0.3988$
$b = 76.12$

Fig. 4-7 Correlation of Discharge between Djombiru and Djabon from 1951 to 1970

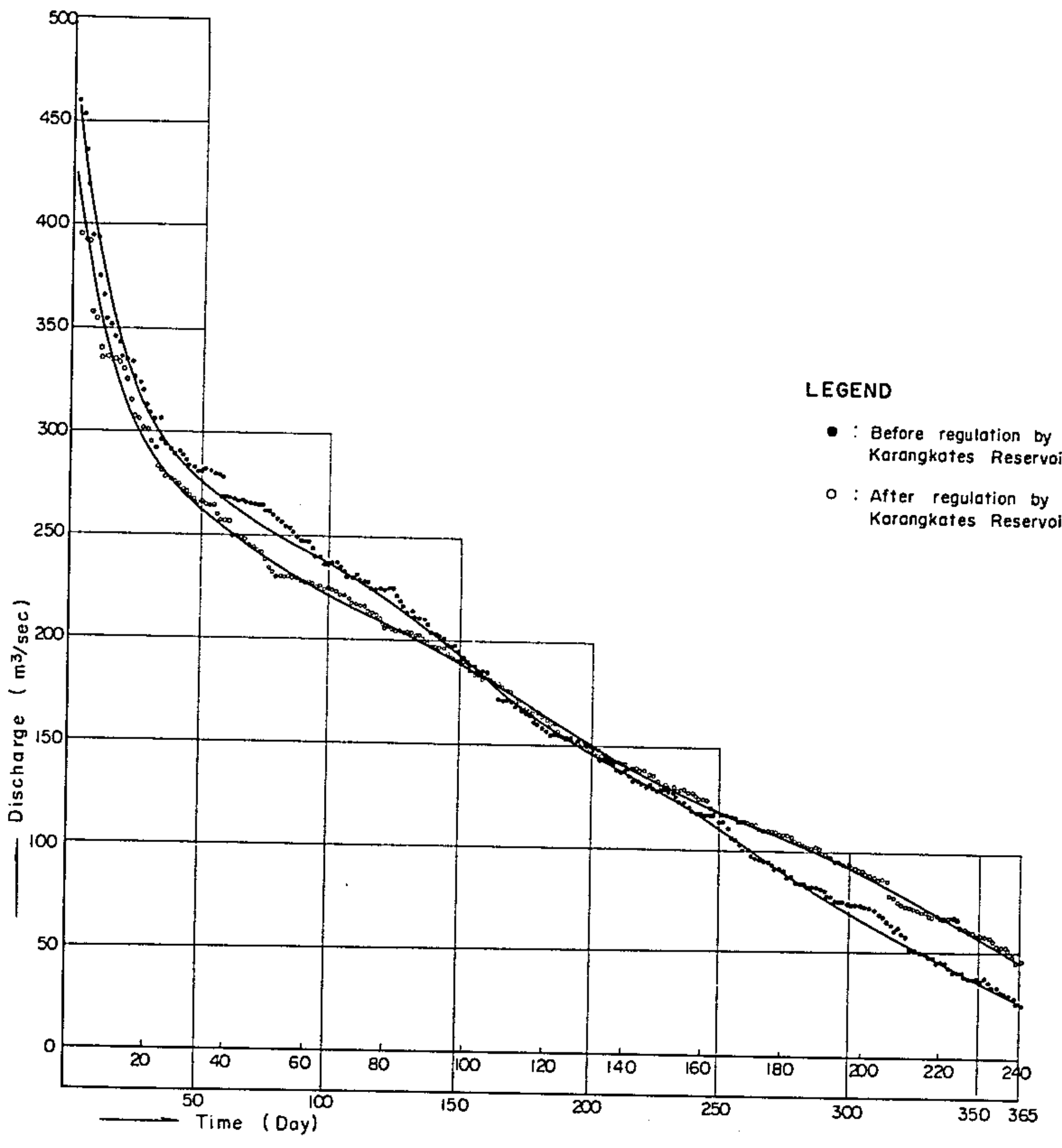


Fig. 4-8 Discharge Duration Curve at Djombiru Gauge

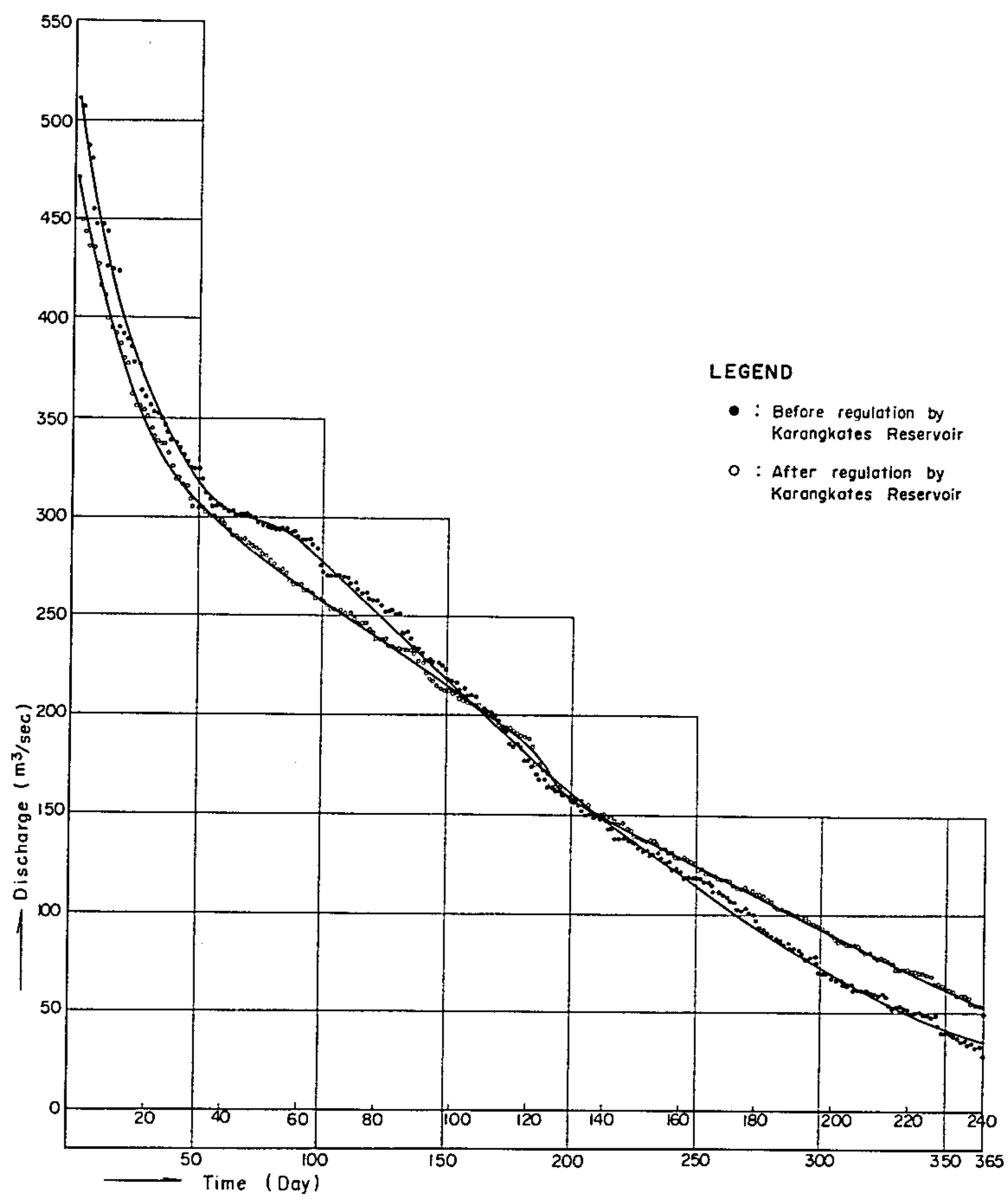


Fig. 4-9 Discharge Duration Curve at Kertosono Gauge

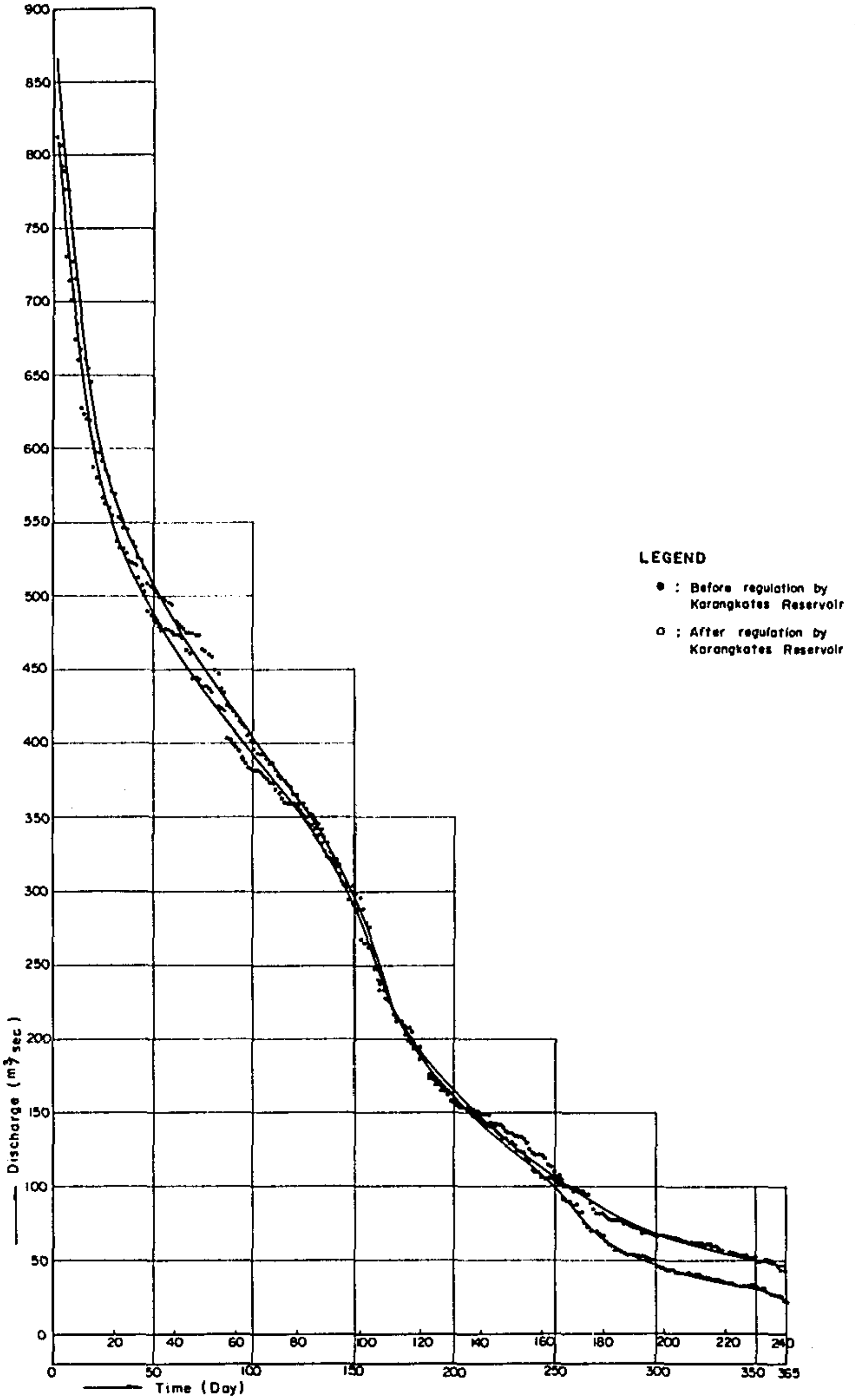


Fig. 4-10 Discharge Duration Curve at Djabon Gauge

5. SEDIMENTATION AND ITS CONTROL

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5. SEDIMENTATION AND ITS CONTROL

5-1 Introduction

The present bed of the Brantas River is highly silted up due to the eruptions of the Mt. Kelut. The river channel capacity has become too small to discharge floods. The sediment in the Brantas River has deposited around the intakes and canals of the irrigation systems. Thus, difficulty arises in protecting the land from floods in the rainy season and taking irrigation water in the dry season.

The Nippon Koei Co., Ltd. suggested the terrain control and the river channel improvement, together with the flood control by dams in 1961[/1]. The terrain control called the "debris control work" has been carried out on the southern and western slopes of Mt. Kelut by the Government since the 1966 eruption.

In this chapter, the sedimentation problem is looked in two aspects, i.e., the aggradation of the river and the sediment transportation in the river channel. These studies result in a estimate of the sediment yield after the eruption of Mt. Kelut. The river bed movement from now on and the capacity requirement of the lahar pocket are important for planning the sediment control in the Brantas River. Based on the evaluation of these factors, important points for the planning are described.

The volcanic product practically causes the aggradation in the main stream between Kaulon and Djabon, and the Porong River. These river stretches are divided into four in this study (for location of sites, see Fig. 4-1):

I.	Kaulon - Djombiru	89.8 km
II.	Djombiru - Kertosono	31.5 km
III.	Kertosono - Djabon	53.7 km
IV.	Porong River	31.5 km
	Total	206.5 km

/1 MPWP "Comprehensive Report on the Kali Brantas Overall Project" Nippon Koei Co., Ltd., Tokyo, 1961.

This chapter begins with a brief description of the eruption of Mt. Kelut and the present situation of the debris control work.

5-2 Volcanic Activity of Mt. Kelut

Mt. Kelut is an active volcano. The eruption has occurred in intervals of 3 - 37 years in recent one and a half centuries, as can be seen in Table 5-1.

The crater of Mt. Kelut usually retains water in it. The volcanic efflux rush down the crater slopes and ravines as hot and muddy masses being mixed with the crater water, when an eruption occurs. This is called the "primary lahar". The primary lahar is very destructive due to its high velocity. It took toll of 5,000 peoples in 1919 eruption. A tunnel was excavated in 1923-1926 through the crater wall and then the lake water was decreased from 40 million cubic meters to two million cubic meters. The tunnel was damaged in the 1951 eruption, but no primary lahar was produced owing to this measure. The tunnel excavation has been carried out seven times altogether until 1972 and now the crater water is about four million cubic meters.

Apart from the primary lahar, the volcanic efflux such as ash, sand, lapillis and volvanic bombs deposit on the hill slopes. They travel down as a mud flow in the ravines when rain comes. This mud flow is called the "secondary lahar". The secondary lahar is also destructive due to its high gravity. It damages bridges and other structures on its course.

Most of the primary and secondary lahar settle on the slopes as a loose deposit which is readily eroded by subsequent rains and travels toward the river. Thus, heavy sediment is loaded on the Brantas River. The sediment yield/1 on the slopes is initially very high and reduces with the pass of time.

/1 The sediment yield in an area herein defined as the total amount of eroded material which is transported from the areas to river channel.

Table 5-2 shows the distribution of the volcanic product deposited on the hill slopes during the 1966 eruption. This table shows that roughly a half of the product deposited around the summit and another half distributed on the southern and western slopes of Mt. Kelut. The total volume of the deposit is estimated to be 90 million cubic meters. The same estimated for the 1951 eruption is 192 million cubic meters/1.

5-3 Debris Control Work

The details of the debris control work are described in a MPWP report/2. This work is featured by the construction of check dams and dikes to trap and settle the lahar deposit on the mountain slopes. The check dam is a masonry dam which is constructed in a ravine in higher elevation and checks the main stream of the primary lahar. The dike is constructed surrounding the area where large amount of the secondary lahar deposits. It is usually located on the middle slope of the mountain. The area surrounded by the dike is called the "lahar pocket". The MPWP report states the purpose of the lahar pocket as follows:

(1) To localize the spreading of material derived from the secondary lahar.

(2) To safeguard the menaced areas at the foot of the volcano, especially the areas on the south, southwest and west.

(3) To prevent the lahar materials from entering the Brantas River, thus lessening the rate of silting in it.

The lahar pockets and check dam have been constructed since the 1966 eruption. The location of the structures completed by the end of 1970 is shown in Fig. 5-1.

/1 Data source; Report on "Volcanic Activity and Its Implication on Surface Drainage" prepared by the Government in 1968.

/2 MPWP "Mt. Kelut Volcanic Debris Control Project (Feasibility Report)" Director General of Water-Resources Development, Djakarta, 1969.

The check dams have small capacities and most are already filled up by deposits. It is presumed that the check dams do not reduce the sediment in the Brantas River, but dissipate the energy of the lahar or change the direction of the lahar flow.

Table 5-3 shows the principal feature of the lahar pockets constructed until 1970. The total pocket capacity is 36.2 million cubic meters and mainly faces to the Kaulon-Djombiru stretch of the Brantas River. The construction of the major lahar pockets has started in 1967/68. The lahar deposit of 19 million cubic meters has been trapped until 1970. The construction cost was 173.9 million Rupiah in total and it ranged between 3.3 and 16.2 Rupiah per cubic meter of the pocket capacity, the weighted mean being 4.8 Rupiah per cubic meter. It is noted that this low cost was resulted from low man-power cost and no compensation cost on the land.

5-4 River Bed Movement

The river bed in the investigated stretch is covered with thick sandy deposit. The river bed elevation has been measured by the Irrigation Service, East Java, since 1950. There are 22 measurement sites in the river length of 220 kilometers between Ngambul near Wlingi village and the estuary of the Porong River. The location of the measurement sites is shown in Fig.5-2. The results of the measurement collected from the relevant Sections of the Irrigation Service are shown in Table 5-4.

Figures 5-3 and 5-4 were prepared based on the data in Table 5-4. Figure 5-3 shows the river profile and channel width with indication of the measurement sites. Figure 5-4 is a plotting of the river bed elevations against year.

The river bed has risen throughout the measured river stretch, during the 1950-1970 period. The aggradation is remarkable for several years after the eruption, and lessens after that. Small up and down in the bed elevation may partly be explained from the year-by-year difference in the water discharge. Some records show a continuous aggradation and some indicate a degradation after an aggradation. This fact suggests a complex movement of the river bed.

The volume increase in the river deposit between the adjacent measurement sites in each year was estimated from Table 5-4 and summarized in Table 5-5. As the general tendency, the river deposit largely increases in about five years after the eruption and the increasing rate reduces later on. The total deposit in 1951-1970 is about 48 million cubic meters. The deposit increase in five years after the eruption is compared with the estimated deposit on the slopes of Mt. Kelut as follows:

(Unit: $10^6 m^3$)

	Mountain Deposit	5-yr. River Deposit	Percentage
1951 eruption	192	26.86	14
1966 eruption	90	16.40	18

5-5 River Sediment Load

The river sediment may be classified into two, by the movement characteristics: Coaser materials move on or near the river bed. This group of material is herein called the bed load. The bed load material is found in both the river bed and water. Its movement causes either aggradation or degradation depending on the discharge at given site. Finer materials are transported in suspension in the river. This group of material is named here the wash load. The majority of the wash load material is found in water and presents to negligible degree in the river bed.

A change in the deposit volume in a river stretch is resulted from an unbalance among the bed load yield in the areas along the stretch and the bed load entering through the upstream cross section of the river channel and flowing out through the downstream cross section. Not the wash load but the bed load is usually the cause of the river bed movement.

A lahar pocket traps both the bed and wash loads. Therefore, not only the bed load yield but also the wash load yield should be taken into account in measuring the effect of the lahar pocket on the river bed movement.

Herein, the bed and wash load discharges are estimated for river cross sections at Djombiru, Kertosono, Djabon, Kepadjaran and Porong for the 1950-1970 period.

Initially, the sediment discharges were estimated for certain water discharges based on data taken in 1959/60. This estimate determined a relationship between the sediment discharge and a hydraulic parameter of the river cross section. By applying this relationship to the monthly mean discharge data described in Chapter 4, the sediment load in each year was obtained. In this calculation, the river bed elevation in 1970 was assumed.

5-5-1 Bed Load Estimate

Table 5-6 shows results of water sampling at Djombiru, Kertosono and Djabon in 1959/1960 and 1971. The sediment concentration in this table comprises the bed and wash loads. The bed load only was calculated by the modified Einstein method/1.

In the modified Einstein method, the bed load per unit width of the river channel can be obtained when the water depth, water velocity, hydraulic gradient are known and the specific gravity and the grading of the bed material are given. The water depth and velocity are taken from Table 5-6. The results of a sieve analysis in 1959/60 in Table 5-7 show the grading of the bed material. The other parameters were assumed as shown in Table 5-8.

The results obtained are plotted on a logarithmic paper in Fig. 5-5. This graph is a non-dimensional expression of the relationship between the tractive force and bed load, in which the abscissa and ordinate, respectively, shows the following variable:

Abscissa: $X = U_*^2/(s - 1)g\ d_{50}$ .... (1)

Ordinate: $Y = q_B/U_*\ d_{50}$ .... (2)

/1 "Hydraulic Formulae" Japanese Society of Civil Engineers, Tokyo, 1963.

where, U_*: Friction velocity: $\sqrt{gIR}$

g : Acceleration of gravity

I : Hydraulic gradient

R : Hydraulic mean depth

s : Specific gravity of bed material

d_{50}: Average perticle size of bed material expressed in sieve size

q_B: Bed load discharge per unit width of channel

Assuming a straight line relationship in Fig. 5-5, the following equations were obtained. The Kertosono data showed too large bed load to fit the total sediment load estimated based on Table 5-6. Therefore, these data were not used and it was assumed that the Kertosono line passes between the Djombiru and Djabon lines.

Djombiru : $q_B = 56.3\ U_*^{5.2}$ .... (3)

Kertosono : $q_B = 59.5\ U_*^{5.2}$ .... (4)

Djabon : $q_B = 96.0\ U_*^{5.2}$ .... (5)

(q_B in m^3/sec/m and U_* m/sec)

The results of the water sampling at the Porong bridge in April, 1964 was analyzed in a DPW report/1 and the following equation was proposed for estimate of the bed load.

Porong : $q_B = 6.62 \times 10^{-2}\ (H\ U_*)^{1.48}$ ..... (6)

where, H : Water depth (m)

This equation was used for estimate of the bed load in the Porong River.

The monthly mean discharge at Djombiru, Kertosono and Djabon are shown in Tables 4-7, 4-9 and 4-10. The same data for Kepadjaran in the Porong River is listed in the "Data Book". This was extrapolated to the Porong bridge site based on the relationship between the water level

/1 DPW "Design Report on Kali Prong Project" Nippon Koei Co., Ltd., Tokyo, 1966.

records at the two sites. The friction velocity U_*, water depth H and the average width of the channel corresponding to the monthly discharge was calculated by use of the cross section, water level-discharge relationship and an assumed-hydraulic mean depth. Thus the bed load at each site was calculated from Eqs. 3 to 6.

Table 5-9 shows a summary of the estimated annual sediment load at Djombiru, Kertosono, Djabon, Kepadjaran and Porong bridge from 1951 to 1970. In this table, the bed load is expressed in the bulk volume instead of the absolute volume, assuming a void ratio of 0.4.

The bed load discharge in the Brantas River is about one million cubic meters per annum, so far estimated at the three cross sections. The sediment carrying capacity is approximately equal throughout the Kaulon - Djabon stretch of the Brantas River.

The bed load at Mlirip, a little below the upstream end of the Surabaja River was estimated in the same way as explained in this section applying Eq. 6. It is about 0.2 million cubic meters per annum. Nearly the same amount of the bed load may flow into the Voor canal. Then, the bed load at the junction of the Prong and Surabaja Rivers balances as follows:

Djabon (Main stream)	1.12 x $10^6 m^3$/year	
Kepajaran (Porong)	-0.73	"
Mlirip (Surabaja)	-0.20	"
Voor Canal	-0.20	"
Total	-0.01	"

The Porong bridge data and Kepadjaran data may represent the lower and upper stretch of the Porong River, respectively. The bed load at the Porong bridge is estimated to be 0.48 million cubic meters per annum, whereas that is about 0.73 million cubic meters per annum at Kepadjaran, i.e., about 0.25 million cubic meters of sediment must deposit in the Porong River every year.

5-5-2 Wash Load Estimate

The total of the bed and wash loads can be calculated from Table 5-6. Then the wash load can be obtained by deducting the calculated bed load from the total load. The wash load for the Brantas River was thus estimated. The relation between the discharge and the wash load in the Porong River was determined based on the results of the 1964 water sampling in which the wash load is defined as particles less than 0.1 millimeter in size and expressed in the wash load discharge per unit width of river. These data were converted into the wash load discharge through the full discharge section. The resulted wash load data are listed in Table 5-10.

The wash load is usually regarded to be proportional to the water discharge. Based on the data in Table 5-10, the following equations were obtained:

Djombiru : $Q_W = 0.203 \times 10^{-6}\ Q^{2.4}$ .... (7)

Kertosono: $Q_W = 0.112 \times 10^{-6}\ Q^{2.4}$ .... (8)

Djabon : $Q_W = 0.059 \times 10^{-6}\ Q^{2.4}$ .... (9)

Porong : $Q_W = 0.031 \times 10^{-6}\ Q^{0.59}$ .... (10)

where, Q_W: Wash load in bulk volume (m^3/s)

Q : Water discharge (m^3/s)

These equations were applied to the monthly discharge data and the annual wash load was obtained as shown in Table 5-11. A comparison of this table and Table 5-9 shows that the wash load is about four times of the bed load.

5-5-3 Effect of Reservoir

The Karangkates dam will be completed in the upstream reaches of the Brantas River. This dam will trap the sediment load from its catchment area of about 2,050 square kilometers.

Results of a water sampling at Karangkates in 1959/1960 show that the sediment concentration is 330 to 420 milligrams per litre in the rainy season and 200 milligrams per litre in the dry season, respectively. An application of these figures to the river discharge data at Karangkates damsite resulted in an annual sediment yield of about 370,000 cubic meters, or 180 cubic meters per square kilometer. Assuming that 25 percent of this volume is the bed load material, it is calculated that the Karangkates dam traps about 90,000 cubic meters of bed load.

The river discharge with operation of the Karangkates reservoir was calculated as explained in Chapter 4. Equations 3 to 6 were applied to this discharge and the bed load was estimated as shown in Table 5-12. A comparison of this table with Table 5-9 shows that the annual bed load discharge decreases by 20,000 - 100,000 cubic meters.

The trapping of the bed load material will result in a reduction in the river bed deposit, while the regulation by the reservoir decreases the high flow. The decrease of the high flow reduces the tractive force, which causes an increase in the river bed deposit in the downstream reaches of dam. These two effects will counterbalance each other in this case. In any case, the Karangkates will insignificantly affect the river bed movement in comparison with the large influence of the eruption product.

5-6 Sediment Yield

The sediment yield in a catchment area can be calculated as a sum of the increase of the river deposit and the sediment load lost to the outside of the area through the river. In the case of the Brantas River Basin, a part of the sediment is lost to the Surabaja River and Voor canal between Djabon and Kepadjaran. This loss was calculated as a difference between the sediment discharges through Djabon section and the Kepadjaran section. Another loss is the sediment load flowing into the sea. This volume was regarded to be equal to the sediment load through Porong bridge site. Then, the total sediment yield for the whole basin can be calculated, applying the above consideration to the bed load.

As explained in Section 5-2, the volcanic deposit distributes on the southern and western slopes of Mt. Kelut, i.e., mostly on the right bank of the Brantas River between Kaulon and Kertosono. This area is called here the affected area and the rest of the catchment area is named the unaffected area. The catchment area of each river stretch upstream of Djabon is classified as shown in Table 5-13. The affected and unaffected areas are 2,340 and 7,335 square kilometers respectively.

The annual sediment yield in the unaffected area is assumed to be 180 cubic meters per square kilometers, of which the bed load material is 25 percent and the wash load material is 75 percent, respectively (see Section 5-5-3). The sediment yield in the affected area is obtained by deducted the yield in the unaffected area from the total yield.

Table 5-14 shows an estimate of the sediment yield in the affected area. The yield is estimated to be 56.6 million cubic meters in 1951-1955, 43.6 million cubic meters in 1956-1965 and 27.6 million cubic meters in 1966-1970, respectively. The yield in each period by the total of eruption product (see Section 5-2) is as follows.

	Initial Five-Year	Subsequent Ten-Year
1951 eruption ($192 x 10^6 m^3$)	30 %	23 %
1966 eruption ($90 x 10^6 m^3$)	31 %	

The sediment yield in five years after the eruption is about 30 percent of the total eruption product on the affected area in both cases of the 1951 and 1966 eruptions. The yield rate in the subsequent period largely reduces.

5-7 River Bed Movement After 1970

The 1971-80 sediment yield was assumed to be 47 percent of the 1956-65 yield. This percentage is the ratio of the estimated volcanic products in the 1966 and 1951 eruptions ($90 x 10^6 m^3 / 192 x 10^6 m^3$, see Section 5-2), i.e., the bed load yield in the affected area in 1971-1980 was estimated to be 4.82 million cubic meters ($0.47 x 10.27 x 10^6 m^3$). As this volume approximately corresponds to the increase in the river bed deposit in 1956-1965, it can be said that the river bed deposit in the investigated stretch as a whole will neither increase nor decrease in the 1971-1980 period.

Table 5-15 was prepared to investigate the river bed movement in each stretch in 1971-1980. The bed load yield in the areas along a river stretch and the bed load inflow through the upstream channel cross section of the stretch work to increase the river deposit, and the bed load outflow through the downstream cross section counteract to them. Therefore the increase in the river bed deposit is obtained as a balance of these three factors. As has been seen in Table 5-2, the majority of the eruption product is distributed in the areas along the Kaulon - Djombiru stretch. Then the affected area yield in 1971-80 was calculated on an assumption that the sediment yield in the affected area of the Kaulon - Djombiru stretch is still high, but those in all the other areas has reduced to 180 cubic meters per year per square kilometer which is equal to the unaffected area yield. The effect of the Karangkates reservoir to the sediment was disregarded for simplicity because it is minor as explained in Section 5-5-3. To make the picture clear, the resulted increase was converted into the average river bed rise in each stretch by dividing by the length of the stretch and the average width of the river channel in Table 5-15. This table shows that a degradation occurs in the upper stretches while an aggradation occurs in the lower stretches. However, this prediction is only very preliminary and is based on many assumption. In reality, judging from the past river movement, it may be said that the river bed deposit in the Kaulon - Djabon stretch as whole will neither increase nor decrease in the 1971 - 1980 period.

5-8 Future Lahar Pocket

Table 5-14 shows that the total sediment yield in the affected area was about 100 million cubic meters during 15-year period after the 1951 eruption. About 32 million cubic meters remained on the river bed, among the bed load yield about 46 million cubic meters in the same period. It will be necessary to intercept 70 percent ($100 \times 32 \times 10^6 m^3 / 46 \times 10^6 m^3$) of the bed load material on the mountain slope for preventing aggradation if an eruption would occur in the future in the same scale as in 1951.

The sediment material on the mountain slope is a mixture of the bed and wash load materials. The lahar pocket inevitably traps not only the bed load material but also the wash load material. Then, the lahar pocket should be capable of about 70 million cubic meters (70% x 100 x $10^6 m^3$) of sediment material, for intercepting 70 percent of the bed load material. The full capacity of the lahar pocket may not always be utilized. Considering a certain allowance for that, it is estimated that the required pocket capacity is in the order of 100 million cubic meters for a future eruption in the same scale as 1951's.

According to the Government, 10,000 - 15,000 hectares of areas could be made available on the southern and western slopes of Mt. Kelut. These areas may correspond to 100 million - 300 million cubic meters of the pocket capacity. In addition, the existing lahar pockets may be increased in their capacity to some extents by heightening of the levees. Then, it is concluded that the lahar pocket can be constructed with a capacity enough to prevent the aggradation caused by one to three times of eruption in the future. Based on the unit cost in Table 5-3, it is estimated that the construction of 100 million cubic meters of the lahar pocket will cost one million to 1.5 million dollars.

5-9 River Improvement

It was estimated from Table 5-5, that the river bed deposit between Kaulon and Djabon has increased by 33 million cubic meters during the 1951-1970 period. It will take about 30 years for the present bed load transporting capacity of about 1.1 million cubic meters per annum to reduce the river bed elevation to the 1950 level even without any sediment yield in the catchment area.

The estimate in Section 5-7 showed that the supply and loss of the sediment load in the investigated stretch as a whole will approximately be balanced. However, it has been pointed out in Section 5-5-1 that the river bed material must increase in a rate of 0.25 million cubic meters per annum in the Porong River due to unbalanced sediment transportation capacity in the upper and lower stretches. The sediment transporation capacity in the lower stretch of the Porong River should be increased by river cannel improvement even if the sediment in the Brantas River would perfectly be controlled.

5-10 Conclusions

(1) The Mt. Kelut is an active volcano. The volcanic product of 192 million cubic meters in the 1951 eruption and 90 million cubic meters in the 1966 eruption deposited on the southern and western slopes of the mountain. The river bed on the Brantas and Porong Rivers has largely aggraded because the volcanic deposit has flowed into the Brantas River.

(2) The river deposit increased by about 33 million cubic meters in the 175 kilometers long stretch between Kaulon and Djabon in the Brantas River and by about 15 million cubic meters in the Porong River, respectively during the 1951-1970 period. This increase is especially remarkable for several years after the eruption.

(3) The sediment load through certain river cross section was estimated. The results showed that the annual bed and wash load is 1 - 1.1 million cubic meters and 4 - 4.5 million cubic meters, respectively, being approximately equal throughout the Kaulon - Djabon stretch of the Brantas River.

(4) The sediment yield from the areas affected by the volcanic product is about 100 million cubic meters during the 15-year period after the 1951 equption. The same volume during the five-year period after the 1966 eruption is about 28 million cubic meters. In both cases the sediment yield is about 30 percent of the total eruption product on the affected area in the initial-five-year.

(5) The sediment yield in 1971-1980 will approximately balance with the tractive force of the Brantas River as a whole. But a degradation will occur in the Porong River due to the unbalanced tractive force in the river channel. The tractive force in this river should be increased by river channel improvement.

(6) It is estimated that 70 million cubic meters of excessive sediment from the affected area will cause an aggradation of river, if an eruption would occur in the same scale as the 1951 eruption. On the other hand, there is a possibility to provide the lahar pockets capable of 100 million - 300 million cubic meters or more. This capacity can control the sediment yield in one to three times of the future eruption. The construction cost of the lahar pockets capable of 100 million cubic meters will be one million - 1.5 million dollars.

(7) The study in this chapter has certain limitations because of the availability of data. For instance the estimate of the sediment discharge based only on the 1959/60 data can not explain the large deposit shortly after the eruption in the lower stretches, along which little increase in the sediment yield is expected.

(8) Basic observations such as a periodical water and bed material sampling, improved discharge and river bed elevation measurements, geomorphological investigation will enable more detailed analysis and interpretation.

Table 5-1 Eruption of Mt. Kelut

Year and date of eruption	Number of victims	Property loss	Amount of crater water expelled ($10^6 m^3$)
1000	No Record		
1311	No Record		
1334	No Record		
1376	No Record		
1385	No Record		
1395	No Record		
1411	No Record		
1451	No Record		
1462	No Record		
1481	No Record		
1586	10,000		
1752 1, May			
1771 10, Jan.	No Record		
1811 5, June			
1826 13, Oct.		65 villages	
1835	No Record		
1848 16, May	21	11 villages 100,000 coffee trees	
1851 24, Jan.	No Record		
1864 3, Jan.	Many	Handreds of houses	
1901 22, May	"		
1919 19, May	5,110	104 villages 900 houses, 1571 head of cattle, 135 km^2 of arable land	40
1951 31, Aug.	7	70 km^2 of arable land	1.8
1966 26, Apr.	212 (died) 74 (lost) 89 (wounded)	116 " "	20

Source; MPWP "Mt. Kelut Volcanic Debris Control Project (Feasibility Report)", Director General of Water-Resources Development, Djakarta, 1969.

Table 5-2 Distribution of Volcanic Product in 1966 Eruption

(Unit: $10^6 m^3$)

Around Summit	
Destroyed Area	27.5
Scorched Area	12.2
Sub-total	39.7
In Tributaries	
Badak	6.1
Putih	4.8
Semut	2.9
Ngobo	1.4
Konto	1.1
Rivers in Kediri, Blitar & Malang	1.6
Sub- total	17.9
Other Areas	31.8
Total	89.4

Source: MPWP "Mt. Kelut Volcanic Debris Control Project (Feasibility Report)" Director General of Water-Resources Development Djakarta, 1969.

Table 5-3 Lahar Pocket

	Area occupied (ha)	Planned pocket ($10^6 m^3$)	Trapped upto 1970 ($10^6 m^3$)	Construction cost 10^6 Rp	Construction cost Rp/m^3
Kaulon - Djombiru					
Putih - I	300	14.2	3.0	66.9	4.7
Djatilangger	440	6.5	6.0	21.3	3.3
Salam	600	9.0	4.0	39.1	4.3
Sub-total	1,340	29.7	13.0	127.3	4.3
Djombiru - Kertosono					
Sukoredjo	100	0.5	0.5	8.1	16.2
Pulo	400	6.0	5.5	38.5	6.4
Sub-total	500	6.5	6.0	46.6	7.2
Total	1,840	36.2	19.0	173.9	4.8

Table 5-4 River Bed Movement Observation

(Unit: EL. - m)

River km channel width stretch	Ngambul	Kaulon	Glongdong	Boro	Radjotangan	Pakal	Djeli	Kediri	Mritjan	Minggiran	Papar	Purwasri	Kertosono
1950	159.60	149.90	124.70	112.00	107.50	90.50	76.60	59.40	55.95				39.30
1951	159.75	150.20	125.00	112.15	107.80	90.55	76.80	59.50	56.05				39.55
52	159.90	150.92	126.52	113.51	108.30	90.90	77.30	59.90	56.35				40.10
53	160.50	150.22	126.87	114.23	108.50	91.40	77.27	60.00	56.80				40.36
54	160.56	150.92	126.40	115.35	108.80	91.26	77.32	60.10	56.66				40.40
55	160.52	151.05	126.11	115.21	108.50	91.40	77.20	59.91	56.68				40.64
56	160.60	150.92	126.50	116.41	109.00	91.70	76.90	59.76	56.63				40.57
57	160.35	150.91	126.31	115.91	109.20	91.48	76.74	59.76	56.56	49.41	47.74	43.93	40.40
58	160.70	150.58	126.30	116.22	109.40	91.74	77.04	59.95	56.56	49.65	47.69	43.97	40.87
59	160.57	150.60	126.18	116.31	109.60	91.61	77.10	60.00	56.71	49.40	47.63	43.95	40.97
60	160.62	150.51	125.99		109.80	91.64	77.40	60.40	56.70	49.61	47.73	44.02	41.25
61	160.62	150.64	125.68		109.70	91.65	77.27	60.44	56.94	49.80	47.70	44.03	41.04
62	160.52	150.50	126.20		109.60	91.60	77.16	60.38	56.64	49.77	47.86	43.94	40.46
63		150.50	126.10		109.60	91.69	77.17	60.10	56.81	49.67	47.85	44.04	41.25
64		150.30	126.21		109.40	91.42	77.10	60.38	56.95	49.92	48.10	43.92	40.45
65		150.18	126.56		109.88	91.20	77.31	60.31	57.23	50.00	48.22	43.97	41.10
66		150.37	127.88		109.60	91.37	77.48	60.36	56.81	49.90	48.02	44.0	41.46
67		150.50	128.52		109.70	91.57	77.31	60.66	57.14	49.95	48.32	43.82	40.66
68		150.90	127.81		110.00	91.45	77.72	60.47	57.12	50.56	48.55	44.12	41.30
69		150.55	128.25		109.61	91.53	78.22	60.58	57.31	50.05	48.35	44.13	42.18
70		150.21	128.24		110.08	91.66	78.25	60.60	57.39	50.10	48.00	44.02	42.16

	Turi	Bunder	Ploso	Tapen	Kesamben	Gampolkerep	Terusan	Ngrame	Porong
1950			26.90			18.70			0.60
1951			27.00			18.80			0.80
52			27.20			19.10			1.79
53			27.40			19.00			1.89
54			27.60			19.10			2.15
55			27.90			19.00			2.50
56			27.80			18.90			2.60
57	34.48	33.37	27.61	24.70	21.25	19.00	16.61	10.11	2.62
58	34.62	33.65	27.54	24.61	21.37	19.10	16.60	10.15	2.69
59	34.32	33.77	27.37	24.47	21.15	19.10	16.64	10.23	2.60
60	34.22	33.70	27.21	24.72	21.03		16.64		2.63
61	34.17	33.61	27.00	24.74	21.21				2.71
62	34.10	33.67	27.42	25.25	20.91				2.75
63	34.04	33.74	27.42	24.67	21.24				2.80
64	34.14	33.50	27.31	24.40	21.44				2.81
65	34.06	33.50	27.55	24.68	21.08				2.90
66	34.53	33.71	27.72	25.00	21.98	19.03	16.91	10.33	2.95
67	34.39	33.56	27.89	25.34	21.93	19.02	16.68	10.47	3.10
68	34.69	33.85	28.64	26.02	21.55	18.89	16.92	11.05	3.50
69	34.60	34.02	28.30	25.83	22.28	18.79	16.96	10.96	3.79
70	34.70	33.68	28.12	25.15	21.93	18.60	16.76	11.20	3.96

Data source; Provincial Irrigation Service in East Java.

Table 5-5 Annual Deposit Increase

(Unit: 10^6 m^3)

	Kaulon-Djombiru	Djombiru Kertosono	Kertosono-Djabon	Djabon-Porong Estuary	Total
1951 (Eruption)	1.54	0.93	1.11	0.57	4.15
1952	4.68	2.31	2.55	4.08	13.62
1953	1.48	1.92	1.04	0.41	4.85
1954	0.05	-0.35	1.05	1.07	1.82
1955	-1.01	0.61	1.38	1.44	2.42
Subtotal	6.74	5.42	7.13	7.57	26.86
1956	0.60	-0.35	-0.75	0.41	-0.10
1957	-0.43	-0.62	-0.88	0.08	-1.85
1958	1.84	0.62	0.82	0.35	3.63
1959	0.22	-0.12	-0.73	-0.11	-0.74
1960	1.66	0.67	-0.11	0.12	2.34
Subtotal	3.89	0.20	-1.65	0.85	3.29
1961	-0.15	0.20	-0.37	0.33	0.01
1962	-0.30	-0.71	0.35	0.17	-0.49
1963	-0.34	0.30	0.49	0.21	0.66
1964	-0.29	0.29	-1.25	0.04	-1.21
1965	0.98	0.96	0.57	0.37	2.88
Subtotal	-0.10	1.04	-0.21	1.12	1.85
1966 (Eruption)	0.81	0.37	3.04	0.21	4.43
1967	1.28	-0.08	-0.34	0.82	1.68
1968	0.83	1.60	2.90	2.65	7.98
1969	1.23	0.14	0.25	0.77	2.39
1970	1.05	-0.35	-1.90	1.12	-0.08
Subtotal	5.20	1.68	3.95	5.57	16.40
Total	15.73	8.34	9.22	15.11	48.40

Remarks: This table shows the annual volume increase of the river bed deposit calculated based on the difference between the annual average river bed elevations in the assigned and the previous years. Mt. Kelut erupted in 1951 and 1966.

Table 5-6 <u>Water Sampling in the Brantas River</u>

Date	Discharge (m^3/s)	Water depth (m)	Velocity (m/s)	Sediment concentration (kg/m^3)
Djombiru				
Nov. 24, 1959	196	1.44	1.12	1.62
Dec. 30	138	1.66	0.97	0.656
Jan. 14, 1960	274	1.76	1.28	2.175
Apr. 13	204	1.47	1.14	1.349
Apr. 20	230	1.58	1.20	0.756
Apr. 27	216	1.52	1.17	1.152
Aug. 4, 1971	69.2	0.75	0.93	0.112
Kertosono				
Dec. 14, 1959	263	1.16	1.39	1.412
Jan. 28, 1960	287	1.23	1.44	0.714
Apr. 15	230	1.07	1.32	1,068
Apr. 25	192	0.97	1.23	0.649
May 2	400	1.50	1.65	1.847
Aug. 4, 1971	70.8	0.54	0.90	0.080
Djabon				
Dec. 23, 1959	660	3.70	0.70	2.290
Dec. 29	221	2.30	0.47	0.454
Jan. 7, 1960	380	2.85	0.59	1.789
Apr. 14	486	3.18	0.67	1.217
Apr. 22	411	2.95	0.61	0.892
Apr. 27	356	2.77	0.58	0.893
Aug. 5, 1971	49.8	1.25	0.22	0.029

Source; MPWP "Comprehensive Report on the Kali·Brantas Overall Project"
Nippon Koei Co., Ltd., Tokyo, 1961.
The 1971 sampling was carried out by the OTCA team.

Table 5-7 Bed Material Grading in the Brantas River

Unit: % (Retained)

Sieve size (mm)	Djombiru	Kertosono	Djabon
4.75	0.3	1.9	-
2.00	1.3	7.2	-
0.84	8.3	26.8	0.4
0.42	38.1	67.5	7.8
0.25	82.5	92.1	46.4
0.105	99.7	99.4	96.4
0.074	100	99.9	99.2
d 50 (mm)	0.36	0.56	0.23
d 10 (mm)	0.20	0.27	0.13
Specific Gravity	2.804	2.772	2.871

Source; MPWP "Comprehensive Report on the Kali Brantas Overall Project" Nippon Koei Co., Ltd. Tokyo, 1961

Table 5-8 Assumed Parameters

	I	s	d 50
Djombiru	1/1730	2.80	0.36 mm
Kertosono	1/2190	2.77	0.56 mm
Djabon	Variable	2.87	0.23 mm

Remarks;

I: Hydraulic Gradient. For Djabon, a curve showing the relationship between the water level and the hydraulic gradient was drawn based on the river cross section and the relationship between the water level and discharge. The coefficient of roughness 0.03 was assumed.

s: Specific Gravity of Bed Material

d_{50}: Average Particle Size of Bed Material

Table 5-9 Annual Bed Load Transported

(Unit: 10^6 m^3)

	Djombiru	Kertosono	Djabon	Kepadjaran	Porong
1951	0.69	0.73	0.93	0.52	0.35
52	1.12	1.12	1.35	0.92	0.70
53	0.70	0.74	1.06	0.71	0.48
54	1.32	1.18	1.82	0.97	0.65
55	1.49	1.62	1.84	1.34	0.92
Sub-total	5.32	5.39	7.00	4.46	3.10
56	0.83	1.03	1.11	0.77	0.45
57	1.11	1.20	1.21	0.74	0.55
58	0.93	0.89	1.20	0.84	0.54
59	1.31	1.60	1.52	0.93	0.70
60	1.29	1.22	1.30	0.80	0.55
Sub-total	5.47	5.94	6.34	4.08	2.79
61	0.68	0.68	0.69	0.41	0.22
62	1.00	1.07	1.07	0.67	0.40
63	1.19	1.22	1.24	0.74	0.59
64	0.81	0.89	0.81	0.51	0.27
65	0.57	0.58	0.61	0.43	0.24
Sub-total	4.25	4.44	4.42	2.76	1.72
66	0.93	0.97	0.83	0.60	0.39
67	0.64	0.68	0.71	0.48	0.28
68	1.62	1.55	1.36	1.02	0.66
69	0.80	0.81	0.81	0.58	0.37
70	0.95	0.97	0.85	0.58	0.34
Sub-total	4.94	4.92	4.56	3.26	2.04
Total	19.98	20.75	22.32	14.56	9.65
Mean	1.00	1.04	1.12	0.73	0.48

Table 5-10 Wash Load Data

(Unit m^3/sec)

Date	Discharge	Wash load	Date	Discharge	Wash load
Djombiru			Porong Bridge		
Nov. 24, 1959	196	0.100	Apr. 1, 1964	224	0.179
Dec. 30	138	0.022	2	153	0.384
Jan. 14, 1960	274	0.200	3	275	0.792
Apr. 13	204	0.084	4	174	0.231
Apr. 20	230	0.044	6	209	0.119
Apr. 27	216	0.073	7	161	0.069
			8	104	0.068
Kertosono			9	137	0.130
Dec. 14, 1959	263	0.125	10	73	0.224
Jan. 28, 1960	287	0.061	11	69	0.047
Apr. 15	230	0.066	13	174	0.358
Apr. 25	192	1.687	14	118	0.279
May 2	400	16.864	15	347	0.996
			16	375	1.478
Djabon			17	322	0.638
Dec. 23, 1959	660	33.725	18	416	0.892
Dec. 29	221	1,860	20	375	0.802
Jan. 7, 1960	380	16.560	21	266	0.432
Apr. 14	486	15.274	22	240	0.245
Apr. 22	411	7.119	23	168	0.125
Apr. 27	356	7.842	24	159	0.047
			27	97	0.135
			May 4	176	0.386

Table 5-11 Annual Wash Load Transported

(Unit: 10^6 m^3)

	Djombiru	Kertosono	Djabon	Kepadjaran	Porong
1951	2.56	3.04	3.51	2.03	2.60
52	4.93	5.55	6.49	4.93	6.30
53	2.61	3.06	4.12	3.00	3.78
54	5.73	5.23	8.76	3.95	5.00
55	6.24	7.35	7.81	5.55	7.02
Sub-total	22.07	24.23	30.69	19.46	24.70
56	2.79	3.91	3.31	2.07	2.67
57	4.73	5.41	5.61	3.83	5.05
58	3.38	3.24	4.42	3.00	3.83
59	5.61	8.05	8.01	5.03	6.45
60	5.47	5.36	5.68	3.27	4.17
Sub-total	21.98	25.97	27.03	17.20	22.17
61	2.36	2.42	1.82	0.86	1.15
62	3.93	4.57	3.71	1.97	2.52
63	5.43	6.31	7.18	4.47	5.72
64	2.79	3.30	1.85	0.83	1.14
65	2.07	2.17	1.66	1.06	1.37
Sub-total	16.58	18.77	16.22	9.19	11.90
66	3.69	4.04	2.95	2.17	2.78
67	2.34	2.64	2.11	1.30	1.66
68	7.11	7.11	4.69	3.60	4.60
69	3.04	3.21	2.70	1.97	2.48
70	3.60	3.77	2.01	1.64	2.10
Sub-total	19.78	20.77	14.46	10.68	13.62
Total	80.41	89.74	88.40	56.53	72.39
Mean	4.01	4.49	4.42	2.83	3.62

Table 5-12 Annual Bed Load Transported with Karangkates Dam

(Unit: 10^6 m^3)

	Djombiru	Kertosono	Djabon	Kepadjaran	Porong
1951	0.63	0.67	0.85	0.48	0.30
52	1.01	1.02	1.22	0.47	0.62
53	0.67	0.73	0.96	0.69	0.46
54	1.21	1.09	1.64	0.91	0.58
55	1.45	1.60	1.67	1.31	0.88
Sub-total	4.97	5.11	6.34	3.86	2.84
56	0.85	1.05	1.03	0.84	0.42
57	1.01	1.13	1.09	0.70	0.48
58	0.89	0.85	1.08	0.79	0.50
59	1.25	1.55	1.39	0.89	0.65
60	1.21	1.16	1.16	0.76	0.50
Sub-total	5.21	5.74	5.75	3.98	2.55
61	1.21	0.66	0.64	0.39	0.20
62	0.94	1.01	0.97	0.63	0.36
63	1.12	1.17	1.12	0.70	0.54
64	0.79	0.87	0.75	0.49	0.25
65	0.53	0.54	0.57	0.40	0.21
Sub-total	4.59	4.25	4.05	2.61	1.56
66	0.82	0.87	0.72	0.53	0.32
67	0.59	0.63	0.65	0.45	0.24
68	1.60	1.55	1.26	0.98	0.61
69	0.84	0.85	0.80	0.55	0.32
70	0.91	0.92	0.77	0.54	0.30
Sub-total	4.76	4.82	4.20	3.05	1.79
Total	19.53	19.92	20.34	13.50	8.74
Mean	0.98	1.00	1.02	0.68	0.44

Table 5-13 Catchment Area

(Unit: km^2)

River stretch	Affected	Unaffected	Total
Upstream of Kaulon	-	2,620	2,620
Kaulon - Djombiru	1,340	1,950	3,290
Djombiru - Kertosono	563	21	584
Kertosono - Djabon	437	2,744	3,181
Total	2,340	7,335	9,675

Table 5-14 Estimate of Sediment Yield in Affected Area

(Unit: $10^6 m^3$)

	1951-1955	1956-1965	1966-1970
Bed Load Material			
1. Deposit Increase (Table 5-5)	26.86	5.14	16.40
2. Lost between Djabon & Kepadjaran (Table 5-9)	2.54	3.92	1.30
3. Lost through Porong Estuary (Table 5-9)	3.10	4.51	2.04
4. Total Bed Load Yield: (1)+(2)+(3)	32.50	13.57	19.74
5. Unaffected Area Yield (0.25 x 180 $m^3/yr/km^2$)	1.65	3.30	1.65
6. Affected Area Yield: (4)-(5)	30.85	10.27	18.09
Wash Load Material			
7. Through Djabon (Table 5-11)	30.69	43.25	14.46
8. Unaffected Area Yield (0.75 x 180 $m^3/yr/km^2$)	4.95	9.90	4.95
9. Affected Area Yield: (7)-(8)	25.74	33.35	9.51
10. Affected Area Yield Total: (6)+(9)	56.59	43.62	27.60

Table 5-15 Estimate of 1971-1980 River Bed Movement

(Unit: $10^6 m^3$)

	Kaulon-Djombiru	Djombiru-Kertosono	Kertosono-Djabon	Kepadjaran-Porong Estuary
1. Affected Area Yield	3.11	0.25	1.44	-
2. Unaffected Area Yield	0.87	-	1.24	-
3. Total Bed Load Yield: (1)+(2)	3.98	0.25	2.68	-
4. Bed Load Inflow	1.18	9.72	10.38	6.84
5. Bed Load Outflow	9.72	10.38	10.76	4.51
6. Deposit Increase: (3)+(4)-(5)	-4.56	-0.41	2.30	2.33
7. Length of Stretch (km)	89.8	31.5	53.7	31.5
8. Average Channel Width (m)	100	160	157	136
9. Average Aggradation: (6)/(7)x(8) (m)	-0.51	-0.08	0.27	0.56

Remarks; (1) Total $4.80 \times 10^6 m^3$.

Djombiru - Kertosono stretch; $0.25 \times 10^6 m^3 = 0.25 \times 180 m^3/yr/km^2 \times 584 km^2 \times 10$ years

Kertosono - Djabon stretch; $1.44 \times 10^6 m^3 = 0.25 \times 180 m^3/yr/km^2 \times 3181 km^2 \times 10$ years

Kaulon - Djombiru stretch; $3.11 \times 10^6 m^3 = 4.80 \times 10^6 m^3 - (0.25+1.44) \times 10^6 m^3$

(2) $0.25 \times 180\ m^3/yr$.

(4) Bed load discharge through upstream channel cross section.

(5) Bed load discharge through downstream channel cross section.

(9) Average river bed rise during 1971-1980.

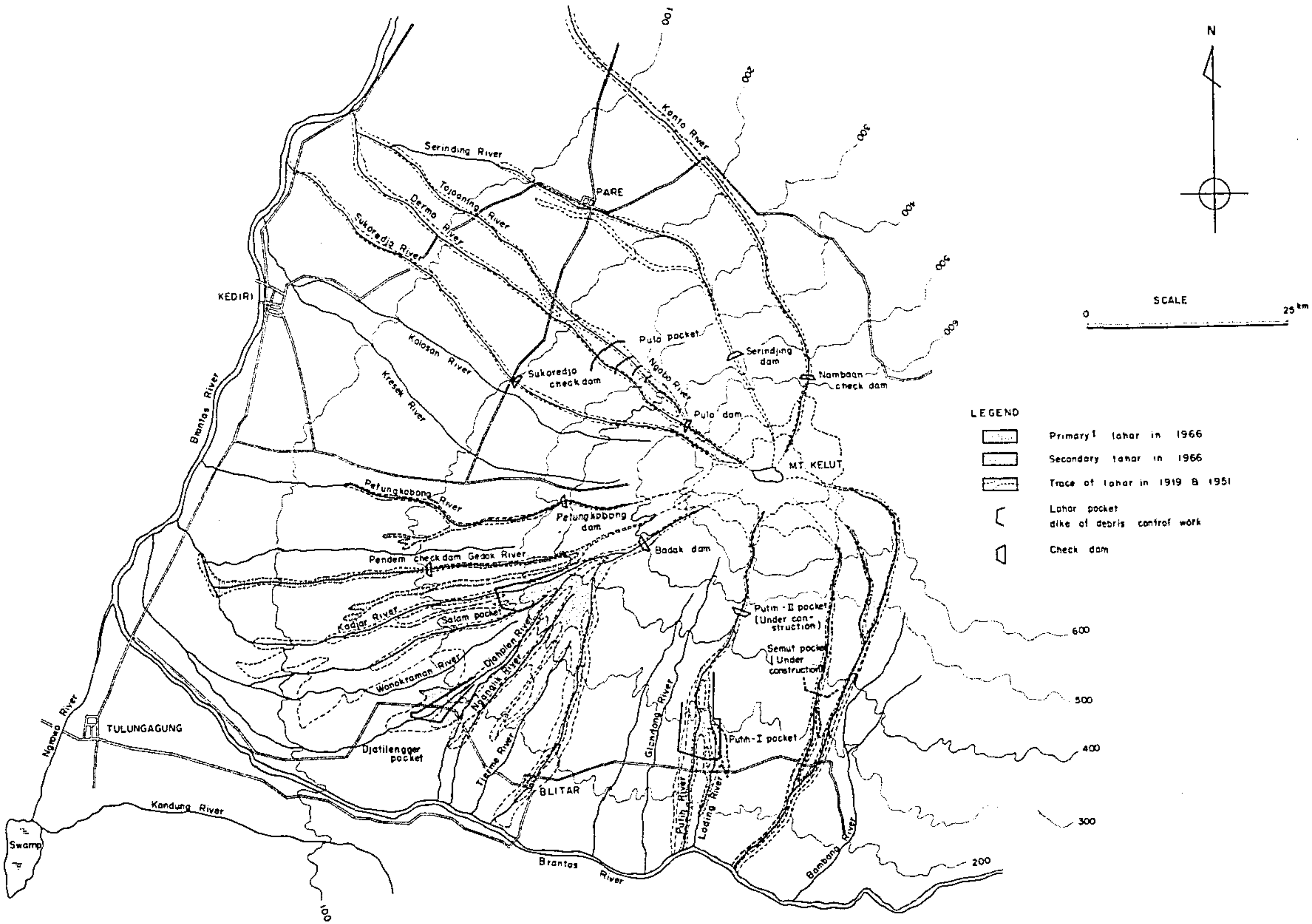

Fig. 5-1 Map Showing Debris Control Work

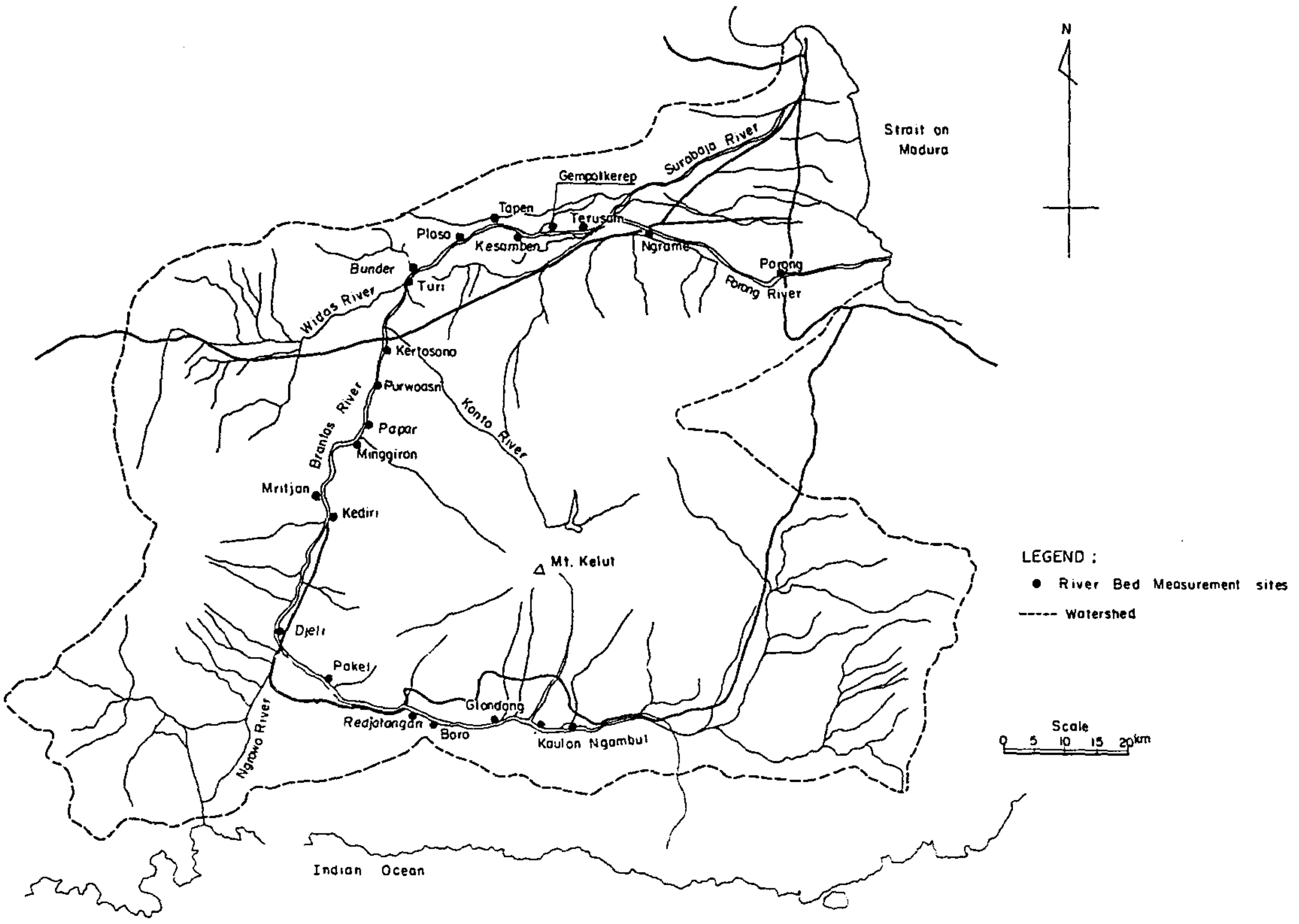

Fig. 5-2 Location Map of River Bed Measurement Site

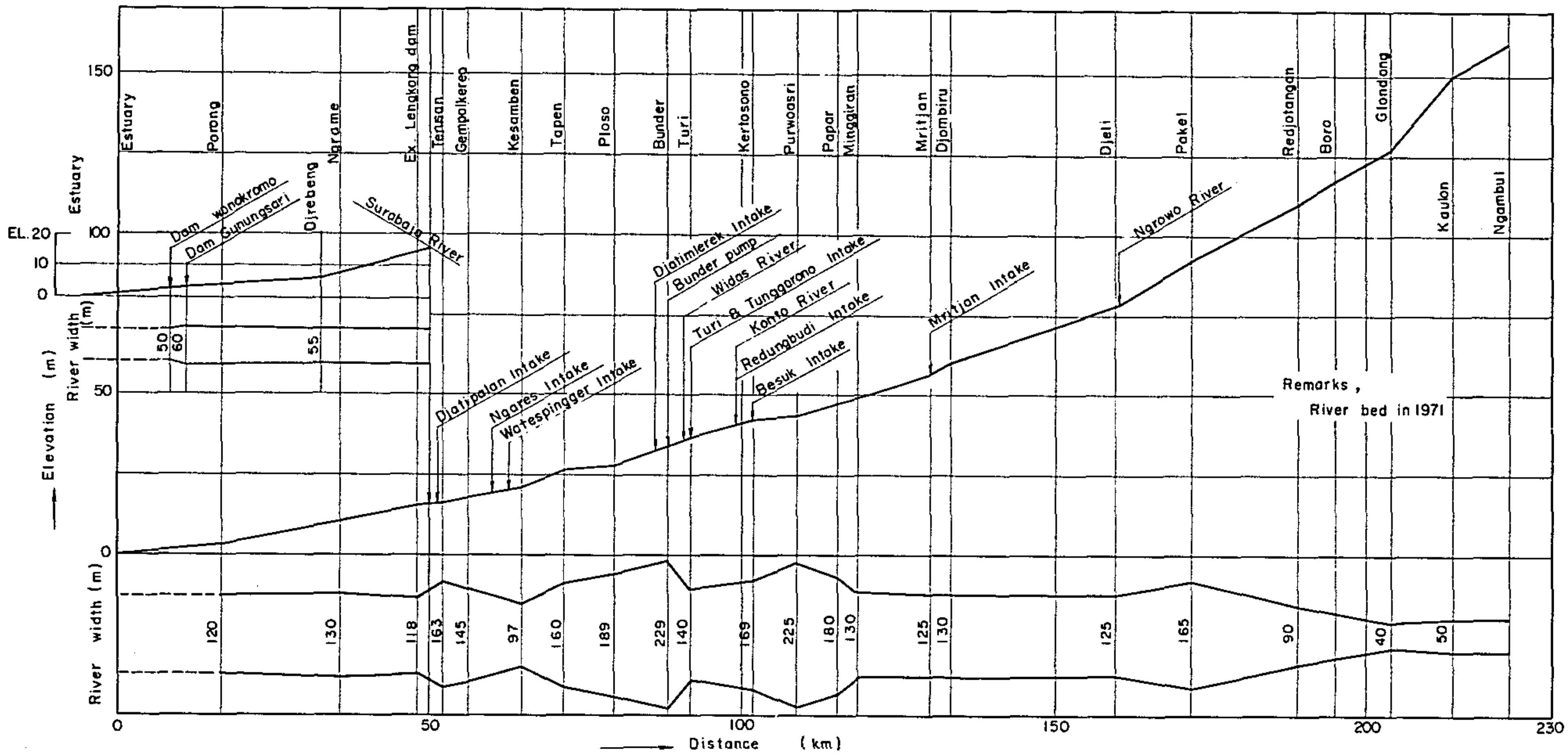

Fig. 5-3 Longitudinal Section of the Brantas River

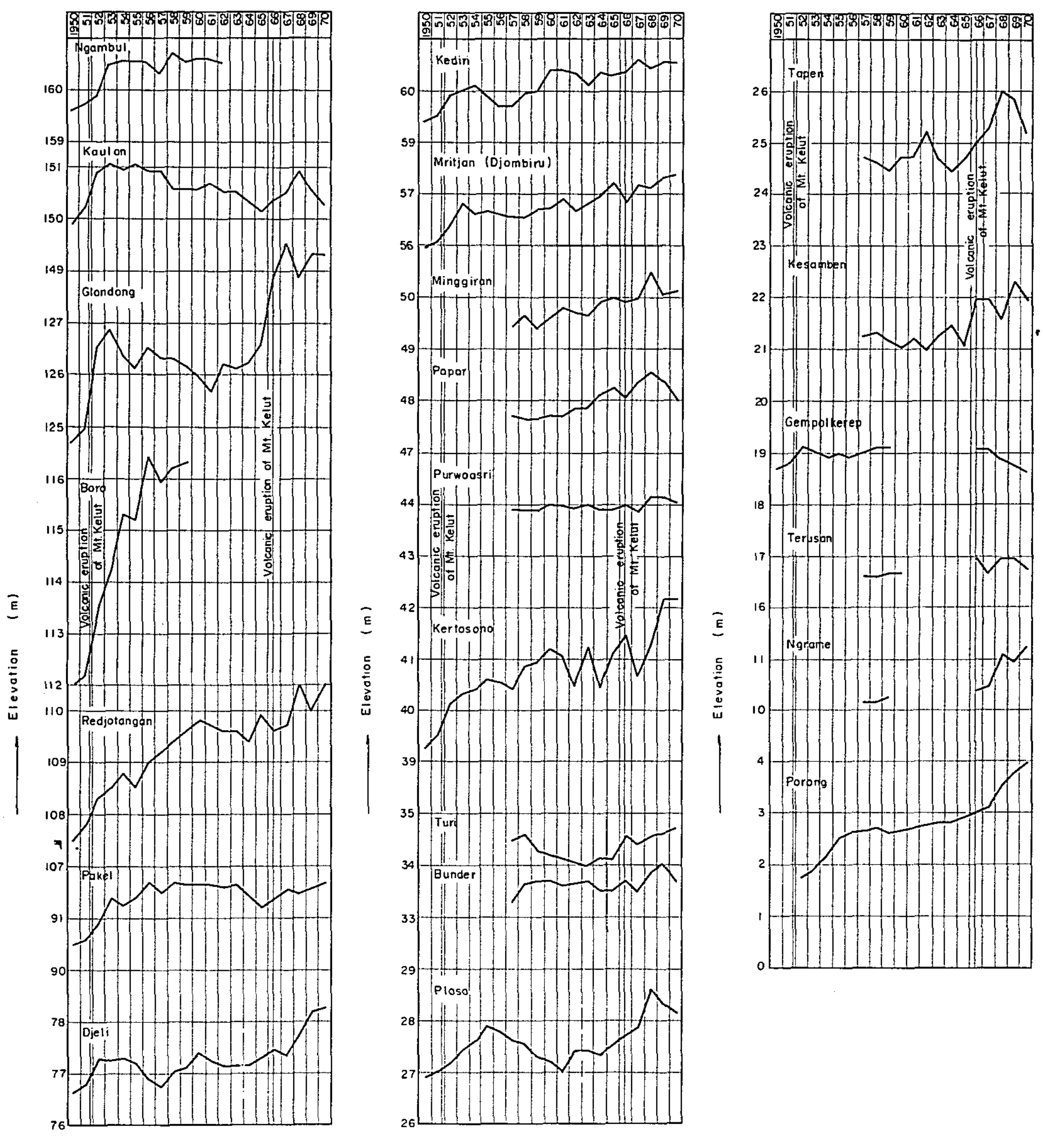

Fig. 5-4 River Bed Movement

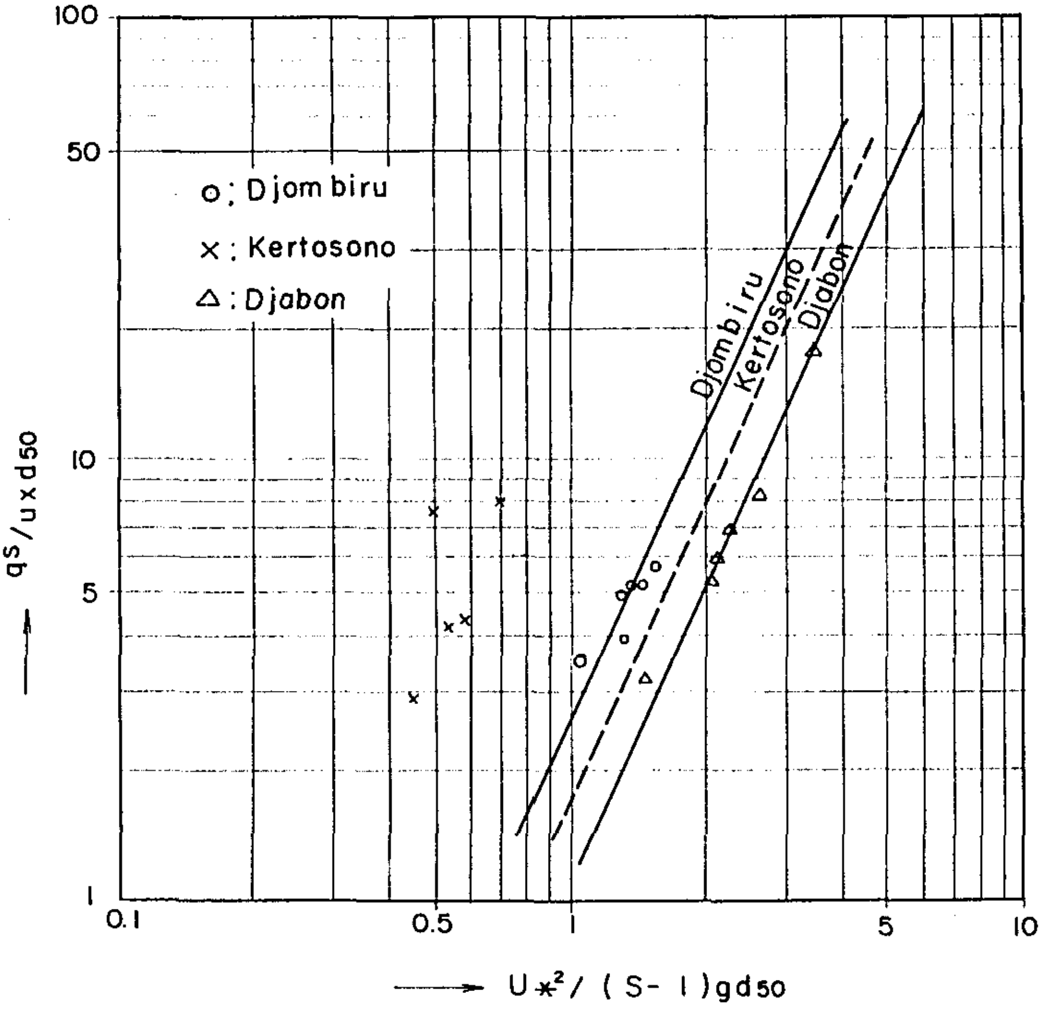

Fig 5-5 Tractive Force – Bed Load Relationship

6. FLOOD ANALYSIS

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6. FLOOD ANALYSIS

6-1 Introduction

This chapter presents an analysis of flood in the Brantas River between the Karangkates damsite and the Terusan gauge, for the purpose of providing data for distribution of flood discharge and planning of the river channel improvement.

Section 6-2 outlines the features of the river stretches divided by the location of main water gauges. For obtaining the flood discharge hydrograph, the relationship between the water stage and discharge is investigated. Then the discharge hydrographs are compared and remarkable characteristics of the flood are explained. In this discussion, it is seen that the flood hydrograph becomes flat and long in the downstream reaches by the retardation effect of the storage in the course.

In Section 6-3, the storage function is determined. The storage function represents the water volume retained in a river stretch during the travel of flood. The theory of the storage function is explained. The storage functions for the Pohgadjih - Pakel and Pakel - Kediri stretches are determined. They are used for the subsequent studies.

In Section 6-4, the flood peak discharge is investigated in connection with the probability of exceedence. The peak discharge is usually the most important variable of the flood for the engineering purpose. The statistical analysis of the peak discharge may give necessary informations for planning a river structure, if the data has a sufficiently long duration. However, it is not the case in the Brantas River. Bias of the data due to the short duration of the records resulted in an unreasonable estimate in a statistical analysis of the peak discharge record. Therefore the rainfall data is incorporated in the probability analysis of the peak discharge. The probable peak discharge at the Pakel gauge is determined by the rainfall-discharge relationship. This discharge is extrapolated to the other gauges by either the storage function method or the peak discharge correlation depending on the data available.

The flood peaks thus obtained are regarded as those having a certain probability. This estimate is theoretically weak but it should be accepted as an approximation, under the present condition of the data.

In Section 6-5, the probable flood is investigated assuming the operation of the Karangkates reservoir. The flood discharge in the lower reaches is estimated by taking into account an unfavorable case for the reservoir flood reduction. The results obtained in this section show the present condition of the flood in the Brantas River.

Section 6-6 deals with the flood reduction by the Soeloredjo reservoir.

In Section 6-7, the discharge capacity of the present river channel is estimated. Comparison with the probable flood obtained in Section 6-5 shows a shortage of the discharge capacity.

Section 6-8 is a study on the retardation effect of the retained water. Some indication of the quantitive relationship between the water volume and flood peak discharge is obtained.

In Section 6-9, an estimate of the flood damage is explained in very preliminary level.

6-2 Provisional Analysis

6-2-1 River Stretch for Analysis

The records of four water gauges were incorporated in the flood analysis in the Brantas River, because the hourly observation of the flood water levels has been carried out at these gauges. The gauges are the Pohgadjih, Pakel, Kediri and Terusan gauges (for locations, see Fig. 4-1). Among them, the hourly water level record only is available at the Kediri and Terusan gauges. Then the water stage discharge relationship at each of them was estimated based on the closely-located Djombiru and Djabon records respectively. The Karangkates gauge record, which involves the hourly observation, was not used, because the gauge is located very close to the Pohgadjih gauge and the river channel is much more irregular at the former than the latter. The duration of the records are illustrated in Fig. 4-2.

The Brantas River was divided into four stretches for analysis at the abovementioned gauges. The outline of each stretch is explained below;

(1) Upstream of Pohgadjih

This is the uppermost stretch of the Brantas River. The Pohgadjih gauge is located at 235 river kilometers from the estuary of the Porong River. The length of river stretch is 80 kilometers. The left bank is the western slopes of about 600 meters high hills. The right bank is the eastern slope of Mt. Butak. The catchment area is 2,210 square kilometers. The average rainfall is 2,070 millimeters. The catchment area is little affected by the recent eruptions of Mt. Kelut. The Karangkates Dam with a flood control space of 50 million cubic meters will be completed in 1973 at 2.5 kilometer upstream of the Pohgadjih gauge. The analysis in this chapter involves a study on the effect of this dam on floods.

(2) Pohgadjih - Pakel

The Pakel gauge is located at 170 river kilometers from the estuary of the Porong River. The river flows westward in this stretch of 65 kilometers and the river slope is 1/800, on an average. The river channel is rather wide and deep. The Brantas River scarcely inundates the lands along this stretch, though the river bed has risen by the eruption of Mt. Kelut. The left bank is the northern slope of an east-west-trending ridge. The right bank is the southern slope of Mt. Kelut. The slope is steep and much affected by the eruption of Mt. Kelut. It is featured by many tributaries which bear heavy sediment load. The catchment area of this stretch is 1,200 square kilometers, on which the annual rainfall is 2,300 millimeters.

(3) Pakel - Kediri

The Kediri gauge is located at 135 river kilometers from the estuary of the Porong River. The length of the river stretch is 35 kilometers and the slope is 1/1,200. The river cross section is relatively narrow and shallow especially in the lower half of this stretch. The river stretch upstream of Kediri is unleveed. The tributaries are silted up. There are many small depressions retarding water in the rainy season. The Brantas River turns to the north at the middle of this stretch. There, the Ngrowo River joins from the sourth. The left bank of the Brantas River is mostly the Ngrowo River Basin and small tributaries drain rest of the area into the lower half of the Brantas River in this stretch. The right bank is the western slope of Mt. Kelut. This slope is affected by the eruption of Mt. Kelut. The catchment area is 2,500 square kilometers. The rainfall is 2,200 millimeters.

(4) Kediri - Terusan

The Terusan gauge is located at 50 river kilometers from the estuary of the Porong River. The length of the river stretch is 85 kilometers and the slope is 1/1,900. The river is silted up. It is leveed for the entire stretch. The left bank is mostly the Widas River Basin. The Widas River Basin is rather flat. The Widas River collects many tributaries in the upper region and its long channel in the lower stretch frequently meanders. This river inundates and develops a large swamp area along its course in the rainy season. The area between the main stream and the Widas River is intensively cultivated. There many back swamps develop. The lower end of this area is a large swamp which is flooded every rainy season. The right bank is the northwestern slope of Mt. Kelut, but is not much affected by the eruption. There are back swamps at the end of this slope. A remarkable tributary on this slope is the Konto River, which has been utilized for irrigation. The Soeloredjo dam was completed in 1971. It is expected that the operation of this dam will substancially reduce the inundation of the Konto River. The Gedek gate is located at five kilometers upstream of the Terusan gauge. This gate diverts a part of the flood discharge to the Marumojo river.

The river length, average slope and catchment area of each stretch are summarized in Table 6-1.

6-2-2 Stage-Discharge Relationship

The discharge hydrographs of the recorded flood were constructed based on the hourly records of the flood water level assuming a relationship between the water stage and discharge.

The daily discharge in 1951-1970 was plotted against the corresponding water stage for the Pohgadjih, Pakel, Djombiru and Djabon gauges. The data shows that the stage-discharge relationship frequently changes because of the large movement of the river bed. Then, a stage-discharge curve was drawn based on the November - October data in each year and it was used for converting the hourly water level record of the particular year to the hourly discharge record.

The concurrent water levels at the Djombiru and Kediri gauges were plotted and the correlation of the water levels was determined. By use of this correlation the stage-discharge curves at the Djombiru gauge were converted to those at the Kediri gauge. The same method was applied for converting the stage-discharge curves at the Djabon gauge to those at the Terusan gauge. The stage-discharge curves used are shown in Figs 6-1 to 6-4.

6-2-3 Flood Hydrographs

The discharge hydrographs were drawn for the case of the largest flood in each year during 1951-1970 period. Those for the largest-four floods are shown in Figs. 6-5 to 6-8.

The flood hydrograph at the Pakel site shows a sharp rise and recession. The later half of the recession is slow and long.

The Kediri hydrograph is very flat and has a very long duration in comparison with the Pakel hydrograph. The reasons are as follows;

(1) The river course is relatively wide in the upper half of the Pakel - Kediri stretch and it becomes narrow and shallow in the lower half. The flood flow in this river stretch is largely retarded by the channel storage in the upper half and reduces by inundating the land in the lower half.

(2) The major part of the catchment area of the Pakel - Kediri stretch is the Ngrowo River Basin, 1,550 square kilometers. The low land in the Ngrowo River Basin is a back swamp formed by the natural levee of the Brantas River. The Ngrowo River drains its basin by cutting this natural levee. Naturally, its channel capacity is very little and most of the flood in the Ngrowo River Basin has been retarded before entering the Brantas River.

The hydrograph at Terusan is further flat and has an elongated pattern. Since the river stretch between the Kediri and Terusan gauges is long, the flood flow occurring at Kediri must be retarded in a certain extent by the river storage. As explained in Section 6-2-1, the catchment area of this stretch has many swamps caused by the discharge from the tributaries. Therefore, the inflow from the tributaries must be retarded before it reaches to the main stream. Sharp peak appears only occasionally.

The Porong hydrograph has the similar tendency as the Terusan hydrograph, but delays about ten hours.

A typical flood hydrograph has the approaching, rising and recession limbs. The approaching limb is the groundwater outflow caused by the antecedent rainfall. It is normally low and relatively constant. The river flow increases when the rainfall is higher than the groundwater storage capacity. This forms the rising limb. The shape of the rising limb depends upon the distribution of the rainfall. The recession limb represents the withdrawal of water from the groundwater storage. Therefore, its shape depends upon the characteristics of the groundwater storage. The discharge in the recession limb can be expressed by the following equation:

$$Q_t = Q_o K^t \qquad (1)$$

where Q_t is the discharge at any time t after Q_o occurs, and K is called the <u>recession constant</u> which is usually less than one.

The typical hydrograph may be observed only when a concentrated storm rainfall occurs. In the actual case, the flood hydrograph is a complex depending on the distribution of rainfall. The approaching limb may not always be the groundwater outflow only, but it may contain substantial part of the antecedent flood discharge. The rising and the recession limb may also be a resultant of many hydrographs. The complex hydrograph has more complicated shape than the typical one. For instance, it may be double-peaked. However, in the sence of average, the approaching, rising and recession limbs can be defined for the complex hydrograph.

Pohgadjih Hydrograph

Figure 6-9 shows the flood hydrographs at the Pohgadjih gauge during the observation period of 1962/63. The discharge in the approaching limb is almost equal for all the hydrographs. This discharge is herein called the base flow. The peak discharge and the base flow are listed in Table 6-2. The base flow is 100 cubic meters on an average.

Figure 6-10 is prepared to show the characteristics of the rising and recession limbs. The abscissa shows the duration starting from the peak time. The ordinate shows a relative discharge to the peak discharge. The discharge is reduced by the base flow in calculating the relative discharge. The rising and recession limbs shown in the dashed lines are proposed for the average hydrograph. The rising limb is assumed to be a straight line having six-hour duration at the top of the base flow. The recession limb is expressed by Eq. 1, where the recession constant K is determined to be 0.87.

Pakel Hydrograph

The hydrographs of the annual maximum flood during 1951-1970 period are shown in Fig. 6-11. The base flow, rising limb and recession limb were determined in the same procedure as explained for Pohgadjih. The peak and the base flow are listed in Table 6-3. The average base flow is 260 cubic meters per second. The proposed hydrograph has a S-shaped rising limb, flat peak of one-hour duration and the recession constant 0.91 as shown in Fig. 6-12.

Kediri Hydrograph

The peak and the base flow estimated from the annual maximum during 1951-1962 are listed in Table 6-4. The base flow is 410 cubic meters per second on an average. The rising and recession limbs can no longer be defined because the observed hydrographs are mixed up each other.

Terusan Hydrograph

The peak and base flow of the annual maximum flood in 1951-1962 are estimated as shown in Table 6-5. The base flow averages 850 cubic meters per second.

6-3 Storage Function

6-3-1 Storage Function Method

A flood hydrograph becomes flat and elongated during the course of travel. This is called the retardation of flood and is caused by the water storage in the river channel. The retardation is an important aspect of the flood in the Brantas River. Due to this, the flood peak in the downstream reaches is considerably reduced.

The method employed here for investigation of the retardation is the storage function method/1 which has been proposed by T. Kimura. The gist of the theoretical background is explained herein.

The law of continuity in a river stretch may be written as follows;

$$I - O = \frac{dS}{dt} \qquad (2)$$

where, I : Inflow through the upstream cross section of the river stretch.

O : Outflow through the downstream cross section of the river stretch.

S : Water volume in the river stretch

t : Time

/1 T. Kimura: Flood Runoff Analysis by Storage Function, Public Works Research Institute, Ministry of Construction, Tokyo, 1961.

Kimura semi-theoretically reasoned that the water volume S at an arbitrary time is an one-valued function of a discharge appearing in a certain section in the stretch. Then Eq. 2 can be written as follows;

$$I - O = \frac{dS}{dQ}\frac{dQ}{dt} \qquad (3)$$

where, Q : A discharge in a certain section in the river stretch.

It was assumed that the discharge Q appears in the downstream cross section T time unit later.

$$Q(t) = O_T = O(t + T) \qquad (4)$$

where, Q (T) : Q at the time t

O_T : O (t + T) : O at the time T + t

then,

$$O_T - O \doteqdot T\frac{dO_T}{dt} \qquad (5)$$

Deducting Eq. 5 from Eq. 3;

$$\begin{aligned} I - O_T &\doteqdot \frac{dS}{dQ}\frac{dQ}{dt} - T\frac{dO_T}{dt} \\ &= \left(\frac{dS}{dO_T} - T\right)\frac{dO_T}{dt} \\ &= \frac{d(S - T\,O_T)}{dt} \qquad (6) \end{aligned}$$

Equation 6 is in the same form as Eq. 2 and shows the relationship between the inflow and the outlfow a time T later. The function $S - T\,O_T$ in Eq. 6 corresponds to the channel storage S in Eq. 2, and called the storage function of unsteady flow;

$$S_l = S - T\,O_T \qquad (7)$$

The storage function S_l can be regarded as an one-valued function of O_T for an appropriate time interval T.

More general expression of the relationship in Eq. 6 is as follows;

$$\Sigma fI - O_T = \frac{dS_l}{dt} \qquad (8)$$

where, $\sum$: Total for main stream and main tributaries

f : Adjustment factor for inflow from minor tributaries.

The storage function is determined as explained below: Initially the values of f and T are assumed. The difference in the left of Eq. 8 is calculated in constant time intervals by use of the observed hydrograph at the up- and downstream end of the river stretch. The accumulative of the difference multiplied by the time interval is plotted against the corresponding value of O_T. The curve thus drawn generally loops. Then the same calculation is repeated for different values of f and T, until the loop substantially narrows. A curve passing the middle of the narrowed loop is regarded as the storage function. The storage function is often expressed in the following form.

$$S_1 = K\,O_T^{\,p} \qquad (9)$$

Once the storage function is determined, the discharge in the downstream cross section for an arbitrary flood inflow condition can be calculated by use of Eq. 8.

This method was applied only for the Pohgadjh - Pakel and Pakel - Kediri stretches, because a large error must be involved, if applied to such a long stretch as the Kediri - Tensan stretch.

6-3-2 Pohgadjih - Pakel Storage Function

The storage function in the Pohgadjih - Pakel stretch was calculated based on five flood data in 1962/63. The inflow from the catchment area of the stretch was considered in the adjustment factor f. The f-value varied between 1.2 and 1.6 depending on the flood characteristics of each flood. The time T was in the range of 1 - 6 hours. Table 6-6 shows the main feature of the incorporated flood and the assumed f- and T-values. The results of the calculation are shown in Fig. 6-13. The dashed line shows a proposed storage function. Its mathematical expression is as follows;

$$S_1 = 1{,}260\;O_T^{\,0.4} \qquad (10)$$

where, S_1: $(10^3 m^3)$
O_T: (m^3/sec)

This equation shows a normal channel condition and indicates that the river flow did not inundate any large area.

Figure 6-14 shows the results of routing by Eq. 10 assuming the f-value is 1.4. The calculated hydrograph at the Pakel site can be compared with the recorded one in these graphs.

6-3-3 Pakel - Kediri Storage Function

The annual maximum floods during 1951-1956 were incorporated in the estimate of the storage function in the Pakel - Kediri stretch. The basic data and assumed T-value are presented in Table 6-7. The results are shown in Fig. 6-15.

Good approximation could usually be obtained when f-value is unity and the inflow from the catchment area of the river stretch is constant. This fact means that the inflow from the catchment area of the stretch is largely retarded before entering the main stream.

The discharge-storage function relationship in Fig. 6-15 shows different tendencies for the small and large discharges. Therefore, two curves were drawn to determine the average relationship.

$$\text{For } O_T < 500 \text{ m}^3/\text{sec} : \quad S_1 = 79.2\, O_T^{0.92} \qquad (11)$$

$$\text{For } O_T > 500 \text{ m}^3/\text{sec} : \quad S_1 = 1.44\, O_T^{1.58} \qquad (12)$$

where, S_1 : $(10^3 m^3)$
O_T : (m^3/sec)

The results of routing by Eqs. 11 and 12 assuming the constant inflow in Table 6-7 are shown in Fig. 6-16.

6-4 Probable Flood

Pakel Gauge

The Pakel gauge has the longest coverage period of the hourly water level observation among the gauges in the Brantas River, and the flood peak there can be clearly identified.

Figure 6-17 shows a plotting of the flood peak discharge against the corresponding one-day rainfall. The peak discharge in this diagram is the observed maximum discharge deducted by the observed base flow. A line showing the rainfall-discharge relationship of four-largest discharges is approximately straight. This line is regarded as indicating the general relationship between the rainfall and peak discharge. The plottings show that many events may occur above this line and a few may occur below the line. This means that most of the events will be less critical than those determined by the abovementioned line, though a few events may be more critical. The same was studied also based on the two-day and three-day rainfall, but no paticular difference was found between the results.

The peak discharges for selected return periods were calculated applying the straight line relationship in Fig. 6-17 to the probable rainfall in Table 3-5. Giving these peak discharges to the average hydrograph determined in Section 6-2-3, the flood hydrographs having the selected return periods were obtained as shown in Fig. 6-18 in which the base flow was assumed to be 300 cubic meters per second. The probable peak discharges are shown in Table 6-8.

Kediri Gauge

The Pakel hydrographs having the selected probability were routed by the storage function determined in Section 6-3-2.

The southern 1,120 square kilometers of the Ngrowo River Basin has been drained through the Nejama Tunnel to the Indian Ocean since 1962 as will be explained in Chapter 8. This area no longer contributes to the flood discharge in the Brantas River. The previous catchment area 2,500 square kilometers has reduced to 1,380 square kilometers.

The base flow was estimated to be 260 cubic meters per second at Pakel and 410 cubic meters per second at Kediri on an average respectively in Section 6-2-3. The balance 150 cubic meters per second was the base flow coming from the catchment area, 2,500 square kilometers of the Pakel - Kediri stretch. Therefore the base flow from the catchment area of 1,380 square kilometers is estimated to be 80 cubic meters per second;

$$150 \ m^3/s \times \left(\frac{1,380 \ km^2}{2,500 \ km^2} \right) \fallingdotseq 80 \ m^3/s$$

The routed peak discharge for the selected return period are listed in Table 6-9. This is regarded as the probable peak discharge at the Kediri gauge.

Terusan Gauge

The catchment area of the Kediri - Terusan stretch, especially in the Widas River Basin, has many swamps as explained in Section 6-2-1. It is presumed that the flood inflow from this area is largely retarded, i.e., the inflow hydrograph has a flat and elongated pattern.

The peak discharges and base flows of large floods at the Kediri and Terusan gauges are compared in Table 6-10. This table shows that the difference in the peak discharges at the two gauges is about 470 cubic meters per second and this value is independent on the magnitude of flood. The same applies to the defference in the base flow of which the average is about 400 cubic meter per second.

This fact is interpreted as follows: The inflow from the catchment area of the Kediri - Terusan stretch adds 400 cubic meters of base flow to the hydrograph in the main stream. The inflow discharge may increase in a certain extent during the flood time. On the other hand, the peak discharge in the main stream tends to reduce due to the storage in the river channel. These increase and decrease approximately cancel out each other. Consequently, the peak discharge at the Kediri gauge increases by only 470 cubic meters per second until it reaches to the Terusan gauge.

The peak discharge having a return period at the Kediri gauge was increased by 470 cubic meters, and this increased discharge was regarded the peak discharge having the same return period at the Terusan gauge. The results are shown in Table 6-11.

6-5 Effect of the Karangkates Reservoir

6-5-1 Peak Discharge at Karangkates

The Karangkates dam is located at 2.5 kilometer upstream of the Pohgadjih gauge. The discharge at the damsite is calculated by multiplying the ratio of the catchment area to the Pohgadjih discharge through the following study;

Karangkates :	2,050 km^2
Pohgadjih :	2,210 km^2
Area Ratio :	0.93

It is the matter of interest how much the peak discharge having a given probability in the downstream is reduced by the reservoir. Generally, there is no functional relationship between the peak discharges in the up-and downstream reaches,though they are correlated each other. Therefore, many different peak discharges upstream of a reservoir are conceivable for a given discharge in the downstream reaches. The peak discharge at the Pakel gauge was plotted against that at the Pohgadjih gauge in Fig. 6-19. Two lines were drawn on the graph. The upper line shows the average ratio of the Pohgadjih discharge to the Pakel discharge, 0.83. This represents the most probable relationship between the two discharges. The lower line has a gradient of 0.70. This line approximately shows an extreme case in which a small Pohgadjih discharge corresponds to a large Pakel discharge. The reduction of the discharge by the reservoir is small, when the inflow discharge is small. Therefore the ratio 0.7 is used as representing an unfavorable case for the reservoir flood reduction.

The peak discharge flowing into the Karangkates dam corresponding to the peak discharge having a selected return period at the Pakel gauge was calculated by multiplying the area ratio 0.93 and the discharge ratio 0.7 to the latter discharge and listed in Table 6-12.

Karangkates Discharge = 0.93 x 0.7 x Pakel Discharge

6-5-2 Discharge Reduction at Karangkates

The Karangkates reservoir has a flood control space of 50 million cubic meters between the elevations 272.5 meters and 275.5 meters.

The flood hydrograph at the Karangkates damsite corresponding to a selected return period of the peak discharge at the Pakel gauge was determined, based on the average hydrograph at Pohgadjih in Section 6-2-3 and the peak discharge in Table 6-12. A base flow of 90 cubic meters per second was assumed. This hydrograph was routed by the Puls method /1 assuming that the above flood control space in the Karangkates reservoir is available.

The Puls method assumes that the difference between the inflow and the instantaneous outflow of a reservoir is equal to the change in the storage in a given time;

$$I - O = \frac{dS}{dt} \quad (13)$$

where, I : Inflow

O : Outflow

S : Storage

t : Time

The equation of motion is given by the stage-discharge curve of the spillway. The relationship between the storage and water level of the reservoir is obtained from the storage curve.

Figure 6-20 shows the results of calculation for selected return period. The difference between the inflow and outflow at any time is the reduction of the discharge by the reservoir. The outflow hydrograph intersects the recession limb of the inflow hydrograph and there it shows the peak discharge. After that the outflow is larger than the inflow.

/1 Ven Te Chow, ; "Handbook of Applied Hydrology", McGraw-Hill Book Company, New York, 1963.

6-5-3 Downstream Effect

Pakel Gauge

The inflow and outflow of the Karangkates reservoir were routed by the storage function to the Pakel gauge, assuming that no discharge is added to the river channel between the Karangkates damsite and the Pakel gauge, i.e., the f-value was assumed to be unity. The difference between the routed inflow and outlfow was regarded as the discharge reduction at the Pakel gauge by the reservoir. Then the Pakel hydrograph with the Karangkates reservoir operation was calculated by deducting the discharge reduction from the original Pakel hydrograph which has been shown in Fig. 6-18. It was assumed that the peak of the routed Karangkates inflow coincides with the peak of the original Pakel hydrograph. Both the original hydrograph and the hydrograph with the Karangkates reservoir operation are shown for selected return period in Fig. 6-21.

Kediri Gauge

The Pakel hydrograph with the Karangkates reservoir operation in Fig. 6-21 was routed by the storage function to the Kediri gauge. The assumptions about the base flow are the same as in Section 6-4. The selected hydrograph for selected return period is shown in Fig. 6-22 together with the hydrograph obtained in Section 6-4.

Terusan Gauge

The peak discharge at the Terusan gauge with the Karangkates reservoir operation was calculated by simply adding 470 cubic meters to that at the Kediri gauge.

The peak discharge calculated assuming the Karangkates reservoir operation is compared with the peak discharge without the Karangkates reservoir for selected return period in Table 6-13.

6-6 Effect of the Soeloredjo Reservoir

The flood discharge reduction by the Soeloredjo reservoir was estimated in the same way as in the case of the Karangkates reservoir. The flood hydrographs are shown in Fig. 6-23. Based on these hydrographs, it is estimated that the base flow is 20 cubic meters per second, the rising limb is a straight line with the duration of three hours, and the recession constant is 0.82. The relationship between the one-day rainfall and the peak discharge was determined based on the plotting in Fig. 6-24. The probable peak discharge was obtained by applying the above relationship to the probable rainfall and is shown in Table 6-14 together with the probable one-day rainfall. The results of the flood routing by the Puls method are shown in Fig. 6-25. The peak discharge with reservoir operation is shown in Table 6-15 for selected return period.

6-7 River Discharge Capacity

A river cross section survey has been carried out in interval of five kilometers in the Porong River in 1970 and in the Brantas River up to the Pakel gauge in 1971 by the Government. The discharge capacity of the Porong and the Brantas River was estimated based on the results of this survey.

It was assumed, in calculating the discharge capacity at a river cross section, that the river bank downstream from the assigned cross section is high enough to carry the discharge equal to the capacity of the assigned cross section, i.e., a vertical wall was assumed on the top of the existing levee where the levee is too low to carry the same discharge as in the assigned cross section.

The water surface was calculated assuming a non-uniform steady flow. The calculation formula is as follows.

$$-i + \frac{dH}{dx} + \frac{1.1\ Q^2}{2g}\frac{d}{dx}\left(\frac{1}{A^2}\right) + \frac{Q^2\ n^2}{A^2\ R^{4/3}} = 0 \qquad (14)$$

where,
- i: Slope of water surface
- H: Water depth (m)
- x: River length between the estuary and the assigned cross section (m)
- Q: Discharge (m^3/sec)
- A: Flow area (m^2)
- n: Coefficient of roughness
- R: Hydraulic mean depth (m)

The coefficient of roughness was determined as shown in Table 6-16 to fit the stage-discharge relationship in Figs. 6-1 to 6-4.

The water surface profile was determined for the discharges of 100, 500, 1,000 and 1,500 cubic meters per second. Then the stage-discharge curve was drawn for each cross section of the river (for the individual curve, see "Data Book"). Then the channel capacity of each cross section was obtained by comparing the curve and the elevation of the river bank.

The calculated discharge capacity is illustrated in Fig. 6-26. The thick line shows the discharge capacity for the water surface just coinciding the ground surface of the river bank, or the levee top. The dashed line shows the capacity for the water surface one meter below the levee top. In the leveed section, the levee will be broken by some discharge between the two lines.

The river stretch upstream of Kediri is unleveed as explained in Section 6-2-1. The discharge capacity at Pakel is 1,370 cubic meters per second. This corresponds to a flood peak of 30 - 50-year return period (see Table 6-13). The capacity further upstream was not calculated, because the river cross section has not been surveyed recently. However, the morphological feature in the upstream reaches shows that the channel capacity is large enough to carry normal floods. The channel capacity in the Pakel - Kediri stretch was calculated to be very low. It is presumed that the inundation occurs very often.

The river stretch downstream of Kediri is leveed. The flood peak is largely reduced in the Pakel - Kediri stretch and flows down to the confluence of the main stream and the Widas River without much inflow from the tributary valley. The flood discharge substantially increases at the confluence. The discharge capacity of the river generally represents these discharge conditions; The levees are low between Kediri and the Widas mouth, and it is high further downstream. The full capacity 690 cubic meters per second at Kediri corresponds to 5 - 10-year return period of flood. This indicates that the levees in the Kediri - Widas stretch are quite liable to damage. The discharge capacity at Terusan is 1,080 cubic meters per second for the water level one meter below the levee top. This capacity corresponds to one or two-year return period of flood. Even the levee in this vicinity does not have enough safety to the flood.

The poor discharge capacity of the Brantas River explained above is a result of the aggradation due to the recent eruption of Mt. Kelut.

6-8 Retardation Effect

It has been seen in this chapter that the retardation is important aspect in determining the flood discharge in the Brantas River. In this type of river, protection of land by levee in the upstream reaches may increase the flood discharge in the downstream reaches.

Herein presented are some study on the retardation in the Pakel - Kediri stretch and Kediri - Terusan stretch of the Brantas River. There is a flood way upstream of the Terusan gauge. The discharge in this flood way is also explained because of the same importance for the river improvement planning.

Retardation Between Pakel and Kediri

The channel of the Brantas River narrows just after joining with the Ngrowo River. Naturally the discharge is disturbed to flow down. Then a certain amount of water is retained in the river course upstream of the narrow. Since the river stretch is unleveed, part of flood water will inundate the land. The retained water in the river and the inundating water cause a large retardation. Therefore the storage function for larger discharge in the Pakel - Kediri stretch in Fig. 6-15 shows a very large storage.

The Pakel flood was routed to the Kediri gauge, assuming that the storage function of the smaller discharge is applicable for all the discharge. The other assumptions are the same as the estimate of the probable flood under the influence of the Karangkates reservoir obtained in

Section 6-5-3. The resulted hydrograph at the Kediri gauge is compared with that assuming the two storage functions in Fig. 6-27. This diagram shows that the flood peak would be much magnified, if the stretch is prevented from retaining water.

The area between the curves assuming the storage function of low discharge for entire range of the flood discharge and the curve assuming the two different storage functions defined for the low and high discharges in Fig. 6-27 represents the water volume retained in the river stretch during the flood time at the present condition. This volume ranges between 15 million - 30 million cubic meters depending on the magnitude of flood. The flood peak at the present condition, flood peak without retardation and the water volume retained are summarized for selected return periods in Table 6-17. This table suggests that the prevention of inundation by means of levee will largely magnify the flood peak in the downstream reaches. Reduction of water retained in the Pakel - Kediri stretch by one million cubic meters would result in an increase in the flood peak by roughly 10 cubic meters per second.

Retardation between Kediri and Terusan

The flood inflow from the area along the Kediri - Terusan stretch has been regarded to be almost constant, because the flood from this area is retarded by many swamps. Most of this area is the Widas River Basin.

The result of the rough estimate shows that the large swamp located near the confluence of the Brantas and Widas Rivers retains water of 30 million - 40 million cubic meters in the rainy season. It is presumed that the flood peak will be increased by 300 - 400 cubic meters per second, if the swamp would be drained.

The hydrologic observation in the Widas River Basin has not regularly carried out yet. The morphological feature of the Widas River is different from that of the Brantas main stream. Under these conditions, no more detailed study was contemplated.

Gedek Flood Way

Presently, part of the Brantas flood is diverted to the Marumojo River by a flood way for the purpose of reducing the flood discharge in the Porong

River. The inlet of the flood way is the Gedek gate located five kilometers upstream of the Terusan gauge on the left bank of the Brantas River. The flood water enters into the Gedek canal and released to the Marumojo River which joins with the lower channel of the Surabaja River. It seems that the Gedek canal sometimes inundates the lands.

The hourly water level record during the 1951-1962 period at the end of the stilling pool of the Gedek gate is available. A stage-discharge curve at this site was estimated by a hydraulic calculation as shown in Fig. 6-28. Based on these data, the discharge in the Gedek canal was calculated. Table 6-18 shows the Gedek discharge when the annual maximum flood occurred. The discharge is 80 cubic meters per second on an average.

6-9 Flood Damage Estimate

A rough estimate of the flood damage was carried out for the Kediri-Terusan stretch based on the results of the river cross section survey in 1971 and 1:50,000 map. These river cross sections are compiled in "Data Book". The location of the cross sections is shown in Fig.6-29.

6-9-1 Hydraulic Assumptions

The flood flow will inundate the land by either overtopping or breaking the levee at the weakest portion. The water diverted to the land flows towards the lower portion of the ground. Due to the relief of the land, the water stands here and there. The levees along the tributaries may be obstacles of the overland flow. Thus whether a land is flooded or not is not directly related to the absolute elevation but rather depends on the height relative to the local average.

A result of the cross section survey involves the ground lines of the lowland as well as the river cross section in intervals of five river kilometers. A horizontal line drawn on the cross section is assumed as a water surface. The length of the water surface at a given height from the lowest ground level of the lowland in a cross section was averaged with that in the adjacent cross section. Multiplying the distance between the two cross sections to the average water surface length results in a water surface area. Then, a height-area relationship can be obtained. The height means the height of the water surface measured from the lowest ground level and the area means the water surface area between the two cross sections. The area was summed up from the sections 1 to 18 for each height and a height-area curve for the whole stretch was obtained.

An integration of the area by the height gives a height-volume curve, in which the water volume retained on the land is expressed as a function of the water surface height above the lowest ground level. The height-volume curve is shown in Fig. 6-30.

Based on the discharge capacity diagram in Fig. 6-26, it is assumed that the inundation occurs in the vicinity of Kediri when the river discharge exceeds over 700 cubic meters per second. Then the water below 700 cubic meters per second of the flood hydrograph will remain in the river and the upper portion will run to the land as illustrated in Fig. 6-31. There may occur further inundation in the downstream and part of the retained water may be drained to the river again. These effects were disregarded. Then the water volume retained on the land at a given time is the accumulative discharge inundated until the time, i.e., the volume can be calculated as the hatched area bounded by the hydrograph, the horizontal line showing the constant discharge 700 cubic meters per second and the vertical line passing the given time t in Fig. 6-31. Thus a time-volume curve can be drawn for the period in which the flood discharge is more than 700 cubic meters.

When the inundation ends, the water retained on the land decreases depending on the discharge capacity of the drainage canals and tributaries. This capacity is assumed to be 200 cubic meters per second. Then the later half of the time-volume curve becomes a straight line starting from the top of the previously-obtained curve and declining to the zero volume with a gradient of 200 cubic meters per second.

6-9-2 Flooding Depth and Duration

The time-volume curve was converted to a time-height curve by use of the previously-obtained height-volume curve. The time-height curve thus obtained was further converted into a height-duration curve as follows: In the time-height curve, a horizontal line showing an arbitrary height was drawn. This line intersects twice with the time-height curve. The intercept of the constant-height-line between the two intersection points is the duration of flooding at the assigned height. Then the duration was plotted in the abscissa corresponding to the height in the ordinate. The height-duration curve above a horizontal line showing a given height is the depth-duration curve of retained water on the ground having the given height from the lowest ground.

A height-duration curve is co-axially drawn with the height area curve in Fig. 6-32. The left abscissa shows the duration and the right abscissa is the area. The ordinate shows the height. In this graph, the maximum height of the retained water surface above the lowest ground is H. Given the height Zo of a ground, the duration in which the ground is flooded is obtained as To. The maximum water depth on this ground is H - Zo and the minimum is zero. Therefore the average water depth during the period To is assumed to be (H - Zo)/2. The area Ao corresponds to Zo in the right graph. Both the flooding duration and average water depth thus obtained are minimum of those in the area Ao which is located below the ground level Zo.

6-9-3 Unit Damageable Value

The 1:50,000 map shows that the farmland and the residential area is about 85 percent and 15 percent of the total flood vulnerable land, respectively. The average number of houses is estimated to be 1.5 houses per hectare in the farmland area and 9.3 houses per hectare in the residential area, respectively, based on data collected from the Kediri, Djombang and Modjokerto Sections of the Irrigation Service. Assuming the value of the house and its relevant facilities to be 300 dollars in the farmland area and 450 dollars in the residentail area, respectively, the average value of house in an unit flooded area is culculated as follows;

Farmland	0.85 ha x 1.5/ha x 300 \$ =	384 \$
Residential area	0.15 ha x 9.3/ha x 450 \$ =	630 \$
Total		1,014 \$

The crop value of rice was estimated, assuming that the total farmland is paddy field. The unit yield assumed is 3.8 tons of dry stalk paddy per hectare. The unit value is estimated to be 47.5 dollars per stalk ton based on the 1969 market price. Then the unit rice crop value is estimated as follows;

0.85 ha x 3.8 t/ha x 47.5 \$/t = 154 \$

The unit value of the upland crop was estimated to be 86 dollars per hectare of the crop area assuming the proportion of the crop area is as shown in Table 6-19. Therefore, the unit upland crop value per flooded area, assuming the whole farms are the upland farm, is:

0.85 ha x 86 \$/ha = 73 \$

The proportion of the paddy and upland crop are taken into consideration in Section 6-9-4.

6-9-4 Damage Percentage

The degree of the flood damage depends on various characteristics of the flood. Herein it is assumed that the prevailing factor determining the degree of the house damage is the maximum water depth. Figure 6-33 shows the assumed relationship between the maximum water depth and the damage degree of a house.

The crop damage degree is normally expressed as the relationship between the duration of flooding and the yield reduction percentage for different water depth and growing stage. The muddiness is also an important factor for determining the yield reduction. This relationship is obtained based on the on-the-spot survey supported by experiments. Such data in Japan is incorporated in the study of the crop damage percentage.

The occurring chance of a flood at the Kediri gauge in each month was estimated from the hourly water level record. The crop area of the paddy and upland crop were obtained based on data collected from Kediri Irrigation Service. The damage degree in each growing stage was reduced by multiplying the occurring chance and the crop area proportion to the damage degree. The reduced damage degree were summed up for a year for an arbitrary duration. Then the duration-damage degree curve was obtained. This curve was prepared for each of selected water depth ranges as shown in Figs. 6-34 and 6-35. The calculation details are compiled in the "Data Book".

6-9-5 Probable Damage

The estimate of the flood damage was carried out for three cases; 5-, 20- and 100-year return periods. The duration-height curve was drawn for each return period, based on the probable hydrograph at the Kediri gauge obtained in Fig. 6-22. By use of this curve, the duration of flooding, corresponding maximum and average water depth were obtained. Applying these data to the damage degree curve, average damage degree is obtained for the land at a given height. Actually the damage degree was obtained for a certain range of the depth which determined the relevant area on the height-area curve. The damage of each damageable item was then obtained as the product of the area, unit value and damage degree as shown in Table 6-20.

The estimated flood damage is about 3.9 million dollars for 100-year return period, 1.8 million dollars for 20-year return period and 0.7 million dollars for 5-year return period, respectively. The probability-damage curve is shown in Fig. 6-36. From this graph, the annual flood damage is estimated to be about 0.4 million dollars.

6-10 Conclusions

(1) The flood characteristics in the Brantas River was investigated to obtain necessary data for planning purposes. Data used are the hourly water level records, from which the discharge-hydrographs were derived.

(2) The morphology of the Brantas River is characterized by wide and deep river channel between the mountain slopes in the upstream reaches and narrow and silted channel accompanied by many back swamps in the downstream reaches. These stretches are bounded at the confluence between the Brantas and Ngrowo Rivers where the discharge is largely retarded by being retained in the upper river channel. The lower reaches may further be divided into the unleveed stretch and the leveed stretch at Kediri. The former often causes inundation and the latter develops many swamps along the river.

(3) The flood in the upstream reaches shows a sharply-shaped hydrograph with a high peak discharge reflecting the young topography. The retardation of the flood by the channel storage largely flattens the flood hydrograph downstream. The tributaries from their large catchment areas bring a large amount of water to the Brantas River. However, the inflow from the tributaries does not show any high peak, because the flood discharge from the silted-up tributaries is once retained in the back swamp and gradually drained. Consequently, the flood peak discharge in the downstream reaches of the Brantas River is very low compared with the drainage area.

(4) The knowledge of the flood peak discharge with a certain return period is always the basis for the engineering of a river. The southern half of the Ngrowo River Basin is drained to the Indian Ocean since the completion of the Nejama tunnel in 1963 (see Chapter 8). The Karangkates Dam will be in operation in the upstream reaches of the Brantas River from 1973. The probable flood peak in the Brantas River was estimated for two cases; with and without the Karangkates Dam operation, but without the Nejama tunnel in both cases. The results of the estimate are summarized in Table 6-13.

(5) The Brantas River has been largely aggraded due to the recent eruptions of Mt. Kelut. The discharge capacity in the downstream of the Pakel gauge, located ten kilometers upstream of the Ngrowo mouth, is in short of the capacity.

(6) The storage function method is one of the routing method of a flood. It is useful for the river having a large channel storage. The value of the storage function represents the water volume retained in a river stretch during the flood. The functions were estimated in two stretches up- and downstream of the Ngrowo mouth. The upstream function shows a normal channel storage. The downstream function shows very high values. This fact is explained from the large channel storage and much retarded tributary inflow. It is estimated that the present peak discharge reduction by retardation in the Pakel - Kediri stretch and the swamp storage at the Widas mouth is several hundred cubic meters per second.

(7) The small discharge capacity of the Brantas River should be considered together with the silted-up tributaries and swamps. Any measure for either increasing the discharge capacity of the tributaries or draining the swamps will magnify the flood in the Brantas River.

(8) The reduction of the flood peak by operation of the Soeloredjo Reservoir is estimated as shown in Table 6-15. The flood discharge diverted through the Gedek canal is estimated to be 80 cubic meters per second on an average.

(9) The annual flood damage between Kediri and Terusan along the Brantas River is estimated to be in the order of 0.4 million dollars.

(10) The probable flood obtained in this chapter contains many assumptions. The methods applied for the estimate are not necessarily consistent for all the gauges, depending on the data available. The figures obtained should be checked up in the future when more data would become available, though the figures are judged to be reasonable.

(11) The estimate of the flood damage is only very preliminary. This should be regarded as a relative figure. Detailed survey in the flood vulnerable area only can provide necessary informations for the estimate accurate enough for investigating economic aspect of the flood problem.

Table 6-1 River Stretches

Stretch	River length (km)	Average slope	Catchment area (km^2)
Upstream of Pohgadjih	80		2,210
Pohgadjih - Pakel	65	1/800	1,200
Pakel - Kediri	35	1/1,200	2,500
Kediri - Terusan	85	1/1,900	3,765

Table 6-2 Base flow and Peak Discharge (Pohgadjih)

Unit:m^3/sec

Date	Base flow	Peak
1962 Dec. 17	80	655
1962 Dec. 22	80	590
1963 Jan. 17	100	375
1963 Jan. 29	110	445
1963 Feb. 3	100	645
1963 Mar. 4	90	545
1963 Mar. 15	110	510
1963 Mar. 23	100	410
Mean	100	

Table 6-3 <u>Base Flow and Peak Discharge (Pakel)</u>

Unit: m^3/sec

Date		Base Flow	Peak
1951	Feb. 18	330	850
1952	Feb. 17	390	800
1953	May 5	260	730
1954	Dec. 17	240	1,310
1955	Nov. 12	150	1,740
1956	Dec. 5	330	770
1957	Mar. 5	95	620
1958	Dec. 28	270	550
1959	Dec. 21	200	630
1960	Mar. 3	310	580
1961	-	-	-
1962	Dec. 3	95	620
1963	Mar. 5	310	710
1964	Oct. 5	280	500
1965	Jan. 21	440	560
1966	Feb. 25	220	720
1967	Jan. 3	220	580
1968	Mar. 16	130	610
1969	Jan. 26	320	680
1970	Apr. 26	320	690
Mean		260	

Remarks: Annual maximum flood data

Table 6-4 Base Flow and Peak Discharge (Kediri)

Unit: m^3/sec

Date	Base flow	Peak
1951 Feb. 18	260	670
1952 Feb. 17	490	750
1953 May 5	420	630
1954 Dec. 17	450	760
1955 Nov. 13	530	900
1956 Dec. 6	270	630
1957 Mar. 6	580	700
1958 Mar. 9	210	540
1959 Apr. 3	290	690
1960 Mar. 3	490	650
1961 Feb. 18	470	580
1962 Apr. 30	420	650
Mean	410	

Remarks: Annual maximum flood data

Table 6-5 Base Flow and Peak Discharge (Terusan)

Unit: m^3/sec

Date	Base flow	Peak
1951 Feb. 24	1,050	1,240
1952 Mar. 17	880	1,230
1953 May 6	980	1,180
1954 Dec. 12	1,000	1,320
1955 Feb. 10	930	1,150
1956 Feb. 9	600	970
1957 Mar. 7	1,010	1,360
1958 Dec. 29	610	1,030
1959 Jan. 7	670	1,390
1960 Mar. 3	840	1,130
1961 Feb. 18	830	970
1962 Jan. 23	740	1,210
Mean	850	

Remarks: Annual maximum flood data

Table 6-6 Data for Storage Function Determination (Pohgadjih - Pakel)

Date	Peak Discharge Pohgadjih (m^3/sec)	Peak Discharge Pakel (m^3/sec)	T (hr)	f
1962 Jan. 22	460	450	2	1.24
1962 Dec. 17	640	540	4	1.54
1963 Jan. 15	410	380	4	1.19
1963 Mar. 3	540	540	6	1.60
1963 Mar. 14	510	490	1	1.39

Table 6-7 Data for Storage Function Determination (Pakel - Kediri)

Date	Peak Discharge Pakel (m^3/sec)	Peak Discharge Kediri (m^3/sec)	T (hr)	Constant inflow (m^3/sec)
1951 Feb. 17	850	660	6	130
1952 Feb. 18	750	740	3	150
1953 May 4	720	630	4	150
1954 Dec. 16	1,310	760	4	270
1955 Nov. 11	1,740	900	4	60
1956 Dec. 5	760	620	7	90

Table 6-8 Probable Peak Discharge at Pakel

Return Period (yr.)	5	10	20	30	50	100
Discharge (m^3/sec)	1,520	1,750	2,000	2,150	2,300	2,500

Table 6-9 Probable Peak Discharge at Kediri

Return Period (yr.)	5	10	20	30	50	100
Discharge (m^3/sec)	750	810	880	920	960	1,020

Table 6-10 Comparison of Kediri and Terusan Flood

Unit: m^3/sec

Date		Peak discharge			Base flow discharge		
		Kediri	Terusan	Difference	Kediri	Terusan	Difference
1951	Feb. 18	670	1200	530	260	1050	790
1952	Feb. 18	740	1020	280	490	790	300
1953	May 5	630	1180	530	420	980	560
1954	Dec. 17	760	1070	310	450	790	340
1955	Nov. 13	890	1100	210	530	700	170
1956	Jan. 1	600	970	370	390	620	230
1957	Mar. 6	700	1370	670	580	1010	430
1958	Dec. 28	520	1030	510	340	610	270
1959	Jan. 7	670	1390	720	260	670	410
1960	Mar. 3	650	1130	480	490	840	350
1961	Feb. 18	580	960	380	300	830	530
1962	Jan. 22	580	1200	620	350	740	390
	Mean			470			400

Table 6-11 Probable Peak Discharge at Terusan

Return Period (yr.)	5	10	20	30	50	100
Discharge (m^3/sec)	1220	1280	1350	1390	1430	1490

Table 6-12 Probable Peak Discharge at Karangkates

Return Period (yr.)	5	10	20	30	50	100
Discharge (m^3/sec)	990	1140	1300	1400	1490	1620

Table 6-13 Estimated Probable Peak Discharge

Unit: m^3/sec.

Return Period	Without Karangkates Reservoir				With Karangkates Reservoir			
(year)	Karangkates	Pakel	Kediri	Terusan	Karangkates	Pakel	Kediri	Terusan
5	990	1520	750	1220	400	1000	660	1130
10	1140	1750	810	1280	470	1090	720	1190
20	1300	2000	880	1350	490	1250	790	1260
30	1400	2150	920	1390	500	1350	820	1290
50	1490	2300	960	1430	530	1440	860	1330
100	1620	2500	1020	1490	560	1560	910	1390

Table 6-14 Soeloredjo Probable Peak Discharge

Return period (year)	Rainfall (mm)	Peak Discharge (m^3/sec)
10	108	390
20	123	440
50	142	505
100	156	555

Table 6-15 Soeloredjo Probable Peak Discharge with Reservoir Operation

Return period (year)	Peak Discharge (m^3/sec)
10	115
20	135
50	160
100	185

Table 6-16 Coefficient of Roughness

Porong River	0.025
Terusan - Confluence of Widas River	0.030
Confluence of Widas River - Kertosons	0.032
Kertosono - Kediri	0.035
Kediri - Pakel	0.037

Table 6 -17 Retardation Effect in the Stretch between Pakel and Kediri

Return period (year)	Kediri Peak discharge		Retained Water Volume ($10^6 m^3$)
	Present Condition (m^3/sec)	Without Retardation (m^3/sec)	
5	660	800	15
10	720	880	17
20	790	990	21
50	860	1120	28
100	910	1210	32

Table 6-18 *Discharge Released from Gedek Gate*

Date		Terusan Peak discharge (m^3/sec)	Gedek Water level (m)	Gedek Peak discharge (m^3/sec)
1951	Feb. 20	1220	22.05	120
1952	Mar. 17	1230	21.81	90
1953	May 6	1180	21.66	80
1954	Dec. 12	1320	21.89	100
1955	Feb. 10	1150	21.16	60
1956	Jan. 1	970	20.03	20
1957	Mar. 7	1360	21.67	90
1958	Dec. 29	1030	20.51	30
1959	Jan. 7	1390	21.64	80
1960	Mar. 3	1130	22.10	120
1961	Feb. 18	970	21.61	80
1962	Jan. 23	1210	21.59	80
	Mean			80

Remarks: Water Level, above S.H.V.P. (Surabaja Harbour Flood Pile)

Table 6-19 Proportion of Crop Area

	Unit Yield (ton/ha)	Market Price ($/ton)	Planted Area (%)	Total Value ($/ha)
Maize	0.71	55	43.5	17
Cassava	7.92	18	3.4	4.8
Potato	10.13	17.7	6.7	12
Peanut	0.84	125	12.1	12.7
Soybean	0.65	177	34.3	39.5
Total			100	86

Table 6-20 Probable Damage Estimate

Return period	100-Year			20-Year		5-Year	
Maximum Depth (m)	0-1.0	1.0-1.3	Total	0-0.8	Total	0-0.4	Total
Average Depth (m)	0-0.5	0.5-0.65		0-0.4		0-0.2	
Average Duration (day)	4.8	5.8		4.0		2.3	
Area (ha)	13,800	20,000	33,800	27,000	27,000	21,600	21,600
House							
Damageable Value (10^3\$)	13,993	20,280		27,378		21,902	
Damage Degree (%)	6	9.4		4.2		1.5	
Damage (10^3\$)	840	1,906	2,746	1,150	1,150	329	329
Paddy Crop							
Damageable Value (10^3\$)	2,125	3,080		4,158		3,326	
Damage Degree (%)	7.2	16.5		6.4		4.2	
Damage (10^3\$)	153	508	661	266	266	140	140
Upland Crop							
Damageable Value (10^3\$)	1,008	1,460		1,971		1,577	
Damage Degree (%)	19.0	19.5		18.0		13.8	
Damage (10^3/\$)	192	285	477	355	355	218	218
Total Damage (10^3\$)	1,185	2,699	3,884	1,771	1,771	687	687

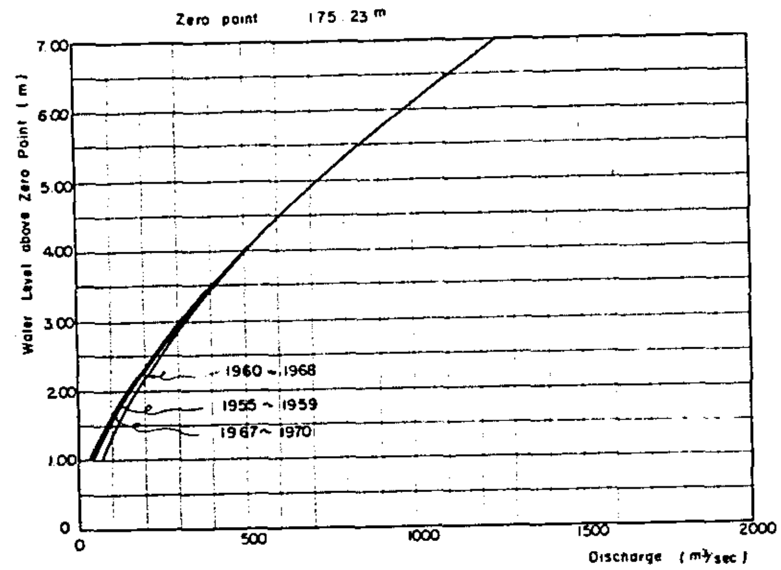


Fig. 6-1 Stage - Discharge Curve at Pohgadjih Gauge

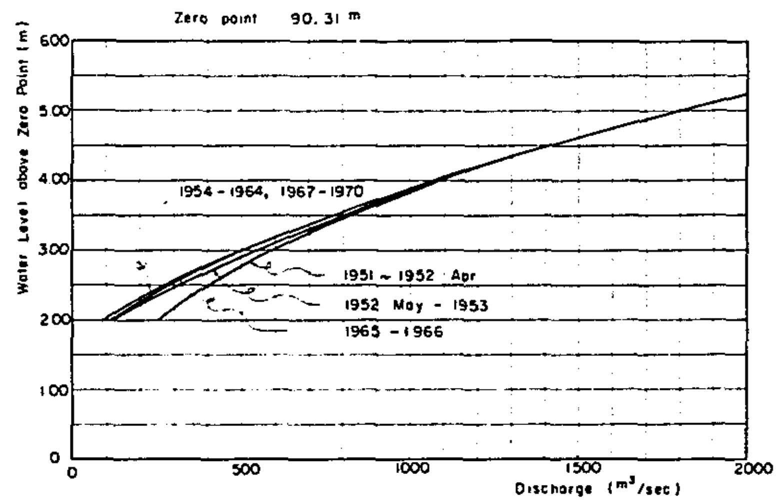


Fig. 6-2 Stage - Discharge Curve at Pakel Gauge

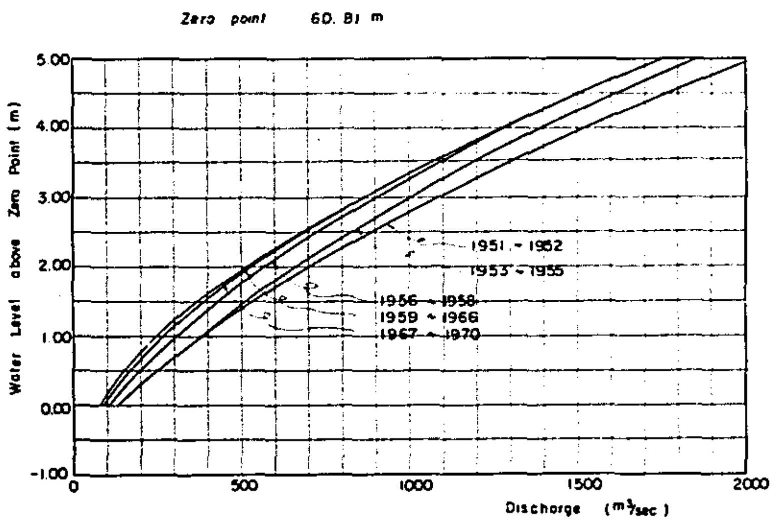


Fig. 6-3 Stage - Discharge Curve at Kediri Gauge

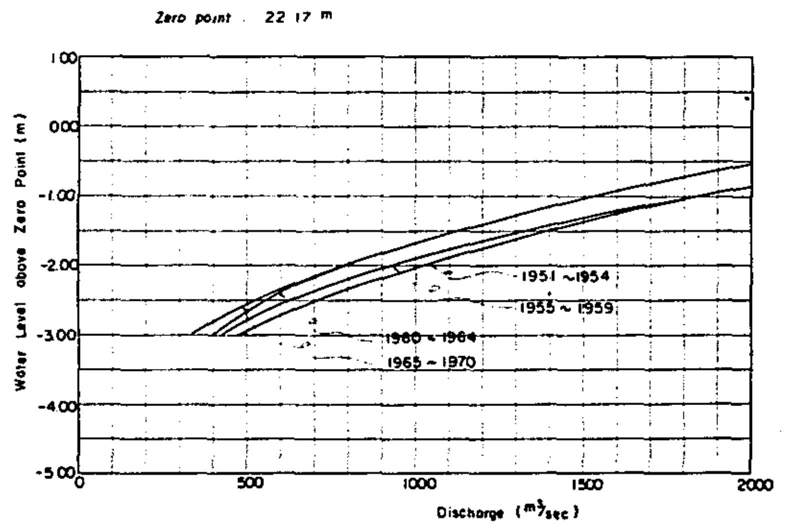


Fig. 6-4 Stage - Discharge Curve at Terusan Gauge

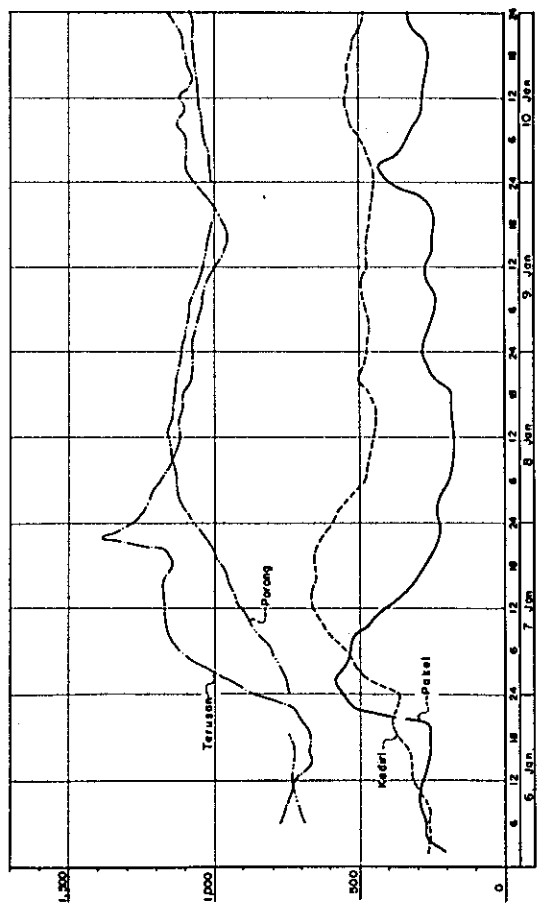


Fig. 6-5 Observed Flood Hydrograph in 1953

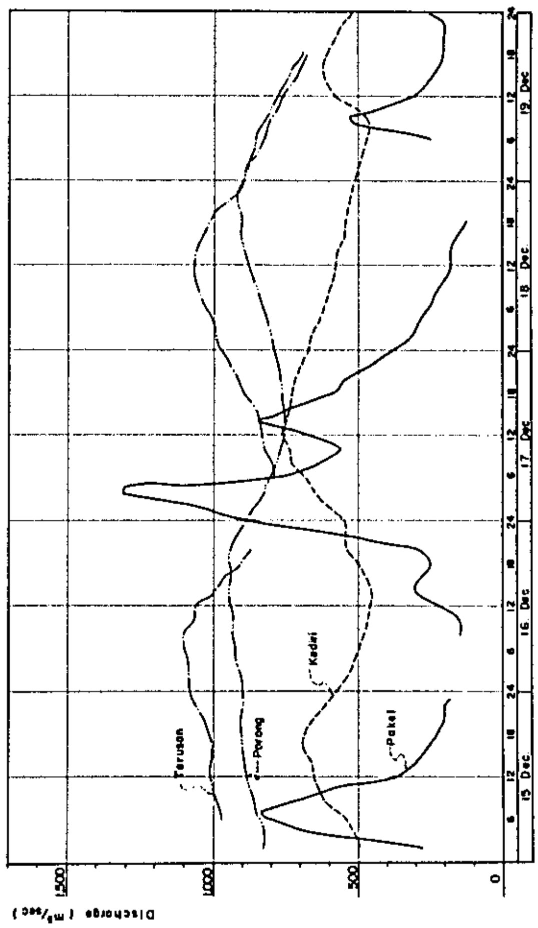


Fig. 6-6 Observed Flood Hydrograph in 1954

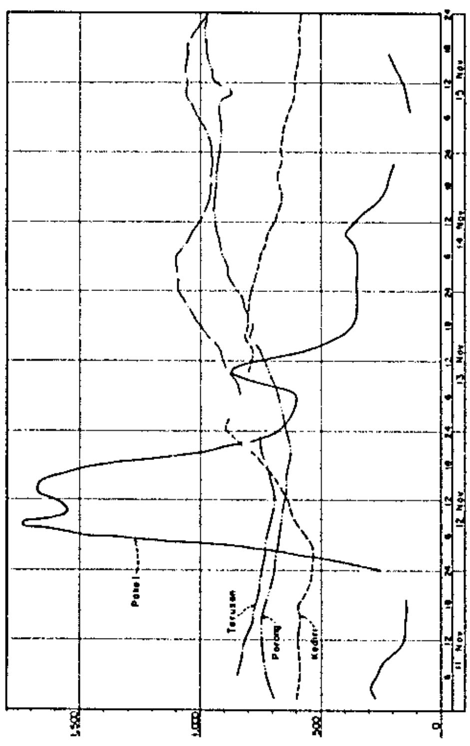


Fig. 6-7 Observed Flood Hydrograph in 1955

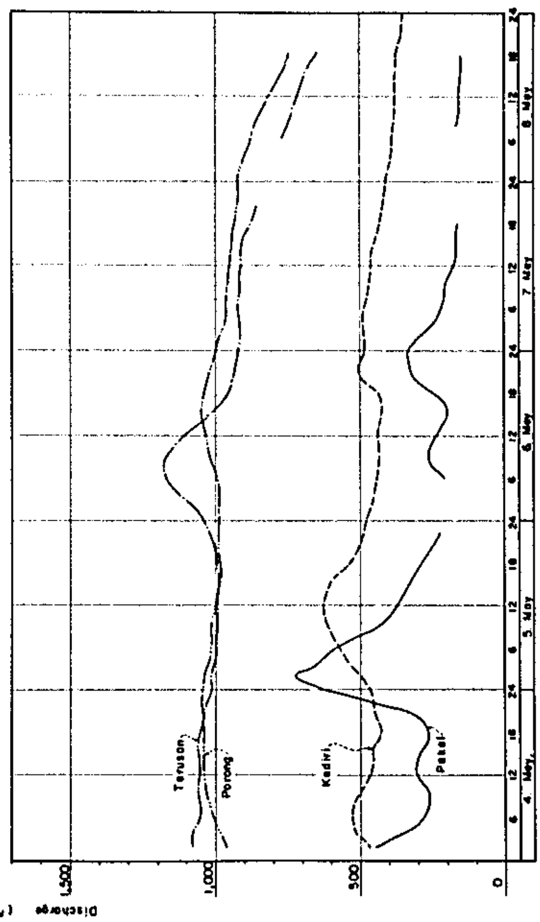


Fig. 6-8 Observed Flood Hydrograph in 1959

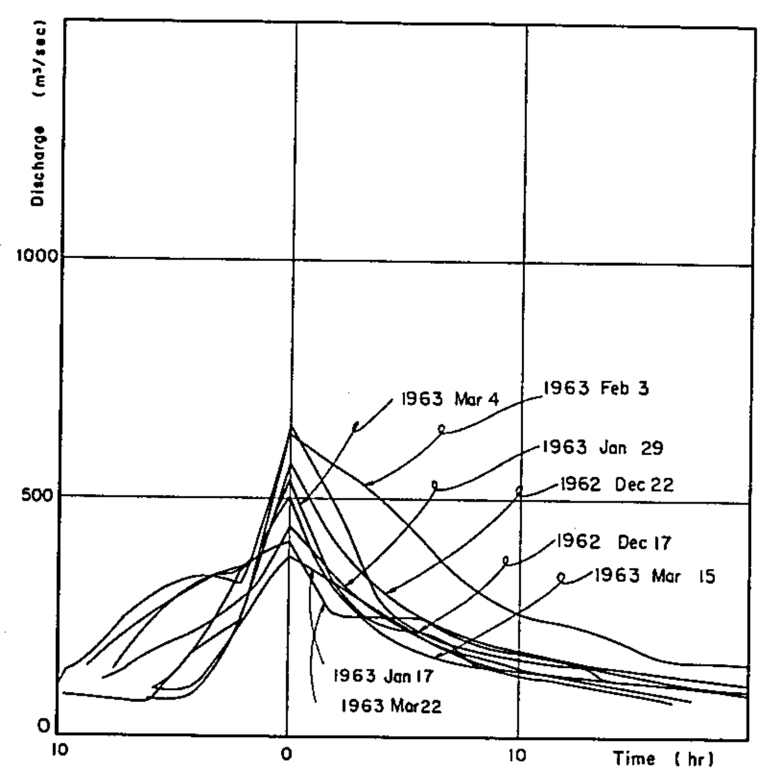


Fig 6–9 Observed Flood Hydrograph at Pohgadjih

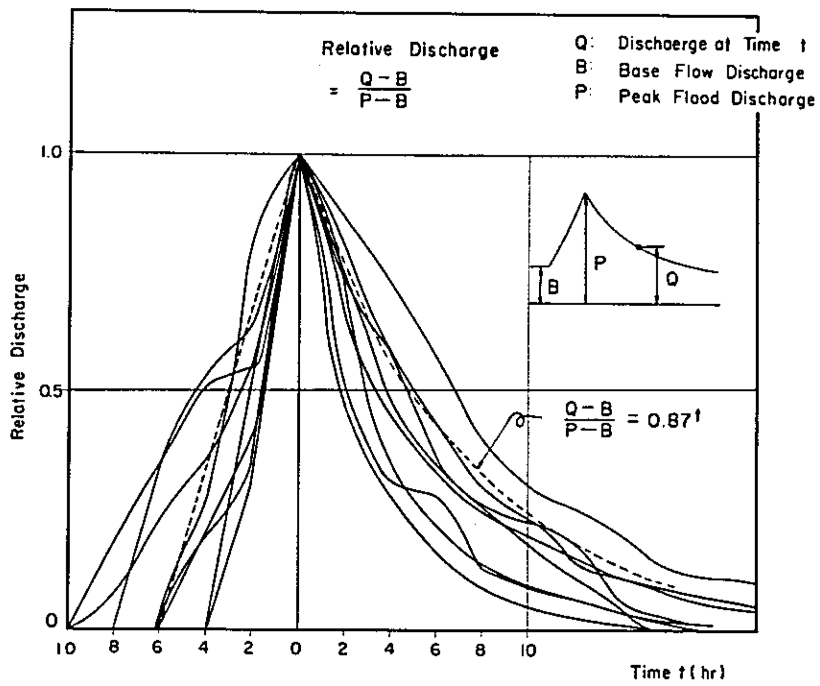


Fig 6–10 Duration — Relative Discharge Curve at Pohgadjih

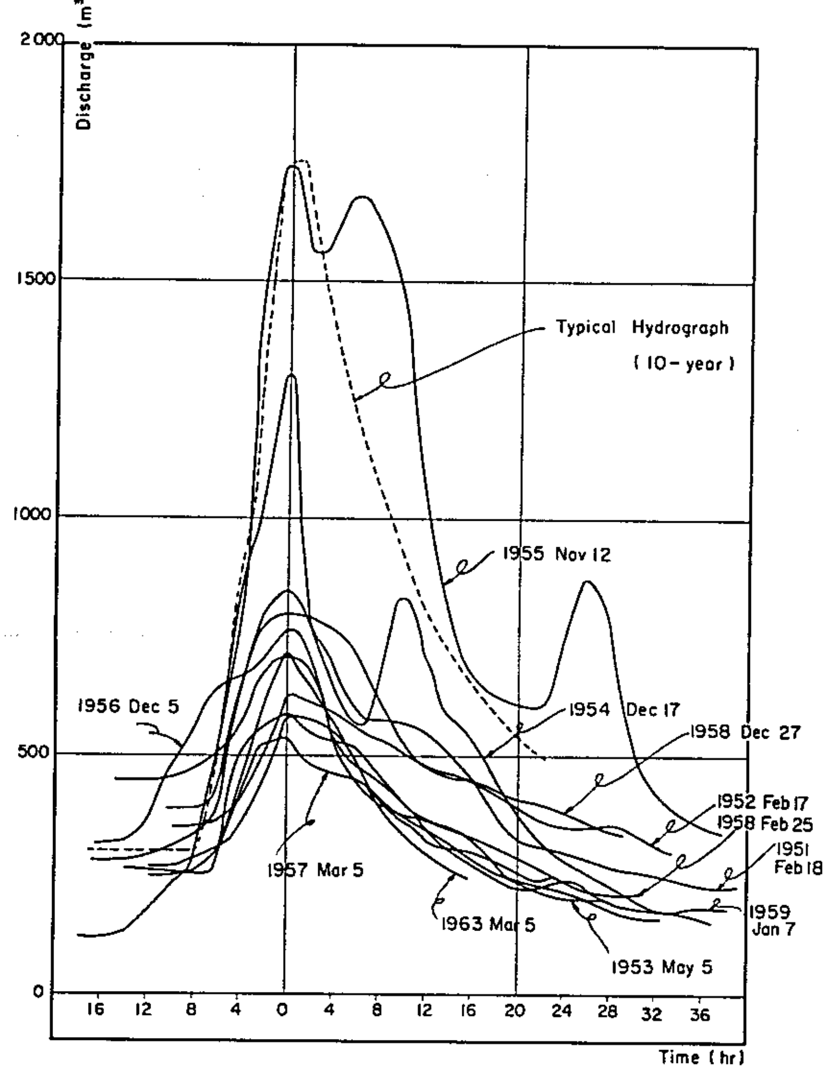


Fig 6–11 Observed Flood Hydrograph at Pakel

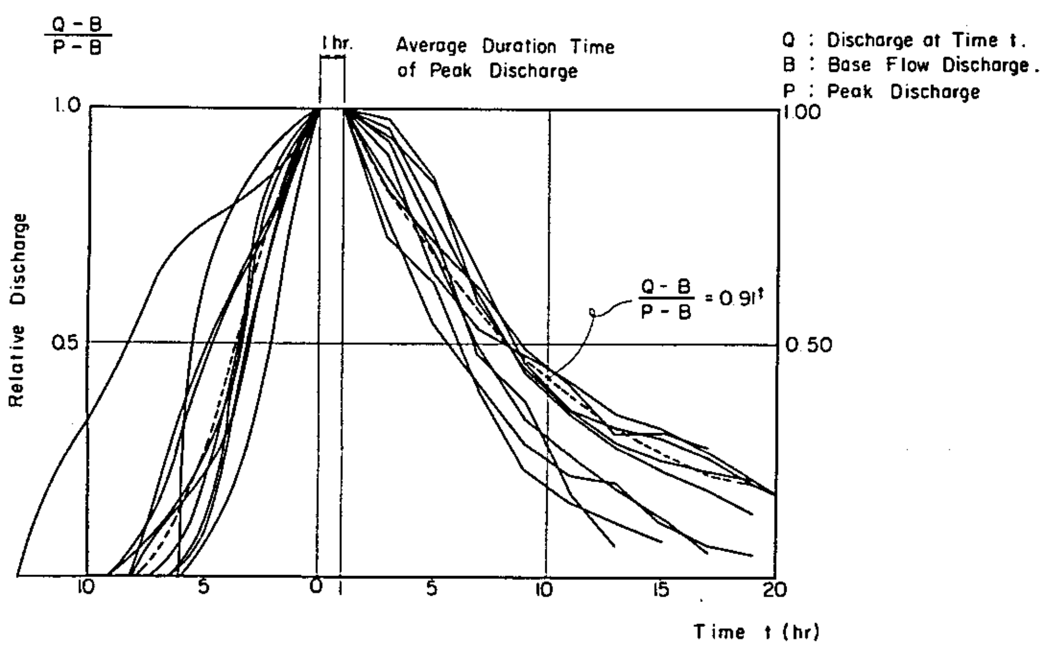


Fig. 6-12 Duration - Relative Discharge Curve at Pakel

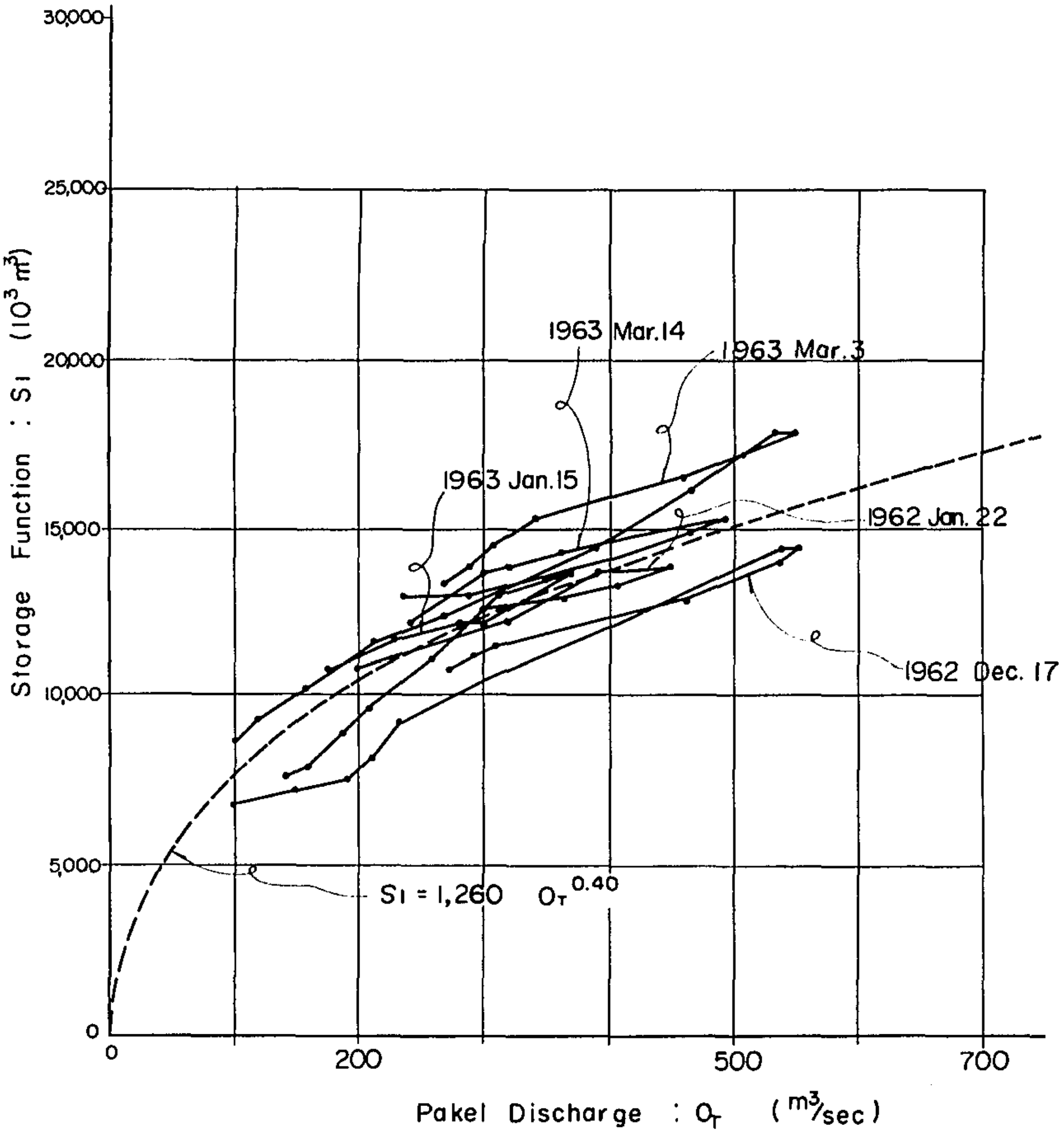


Fig. 6-13 Storage Function (Pohgadjih - Pakel)

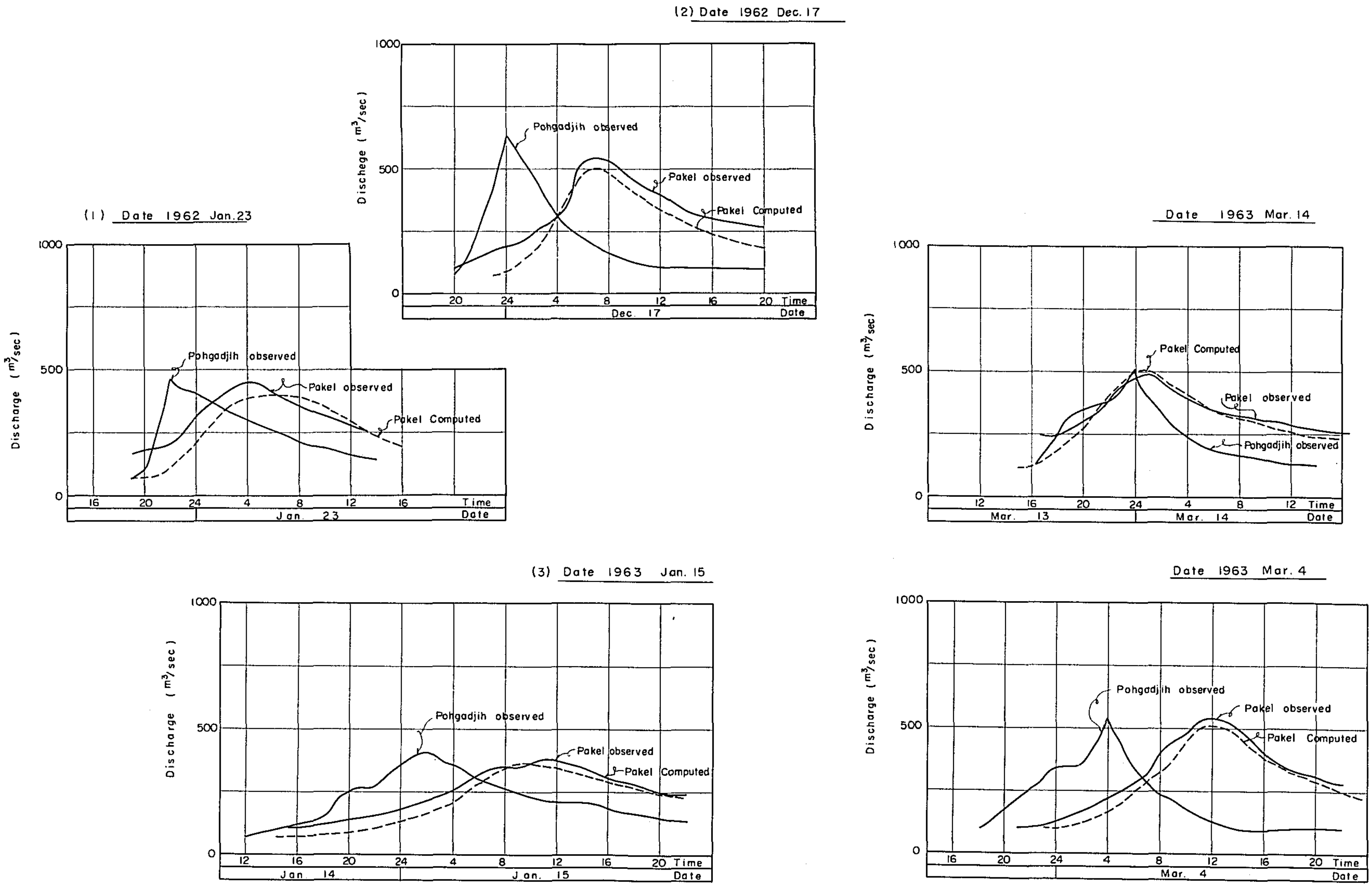


Fig. 6-14 Flood Routing in the Stretch between Pohgadjih and Pakel

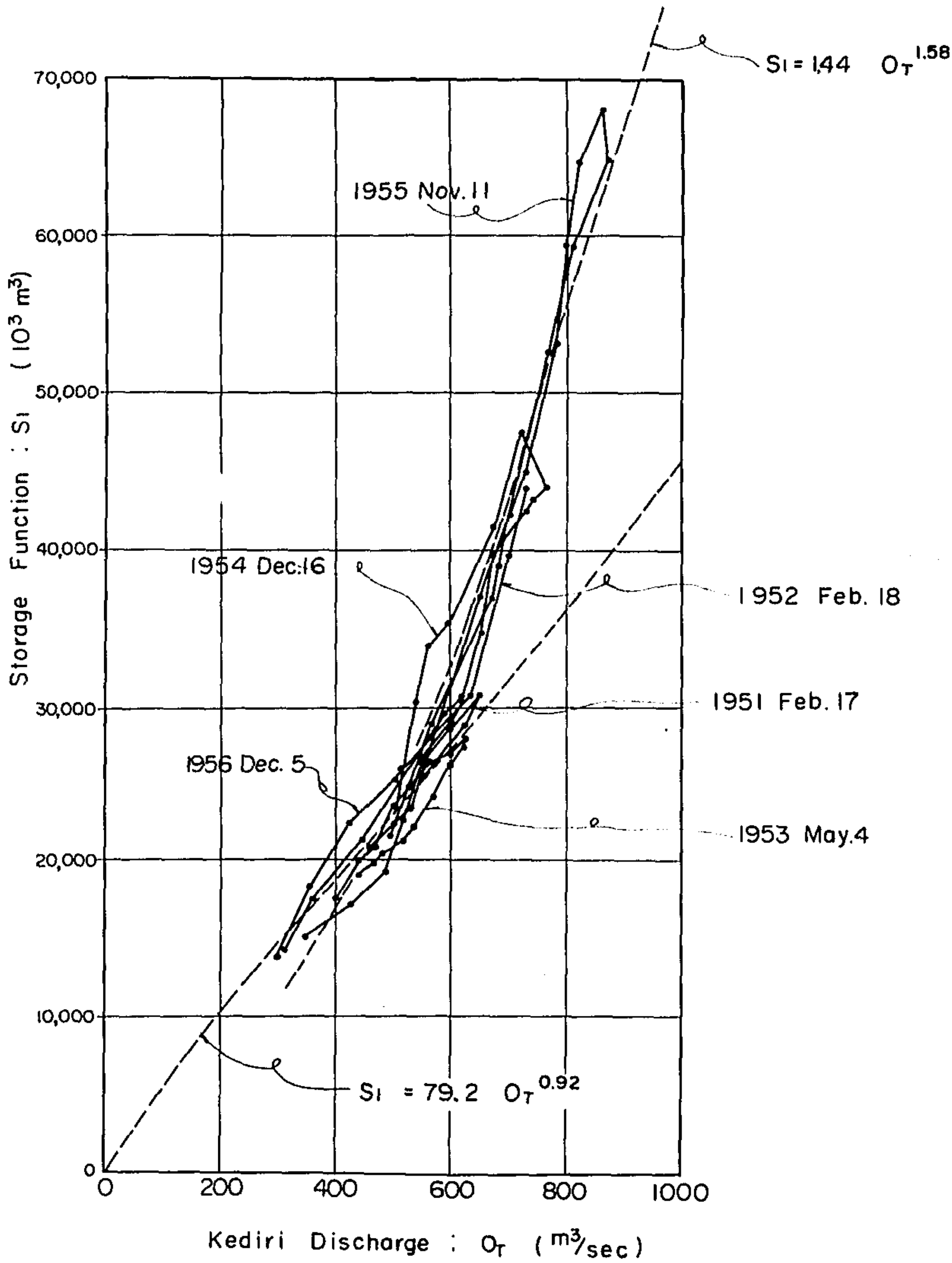


Fig. 6-15 Storage Function (Pakel - Kediri)

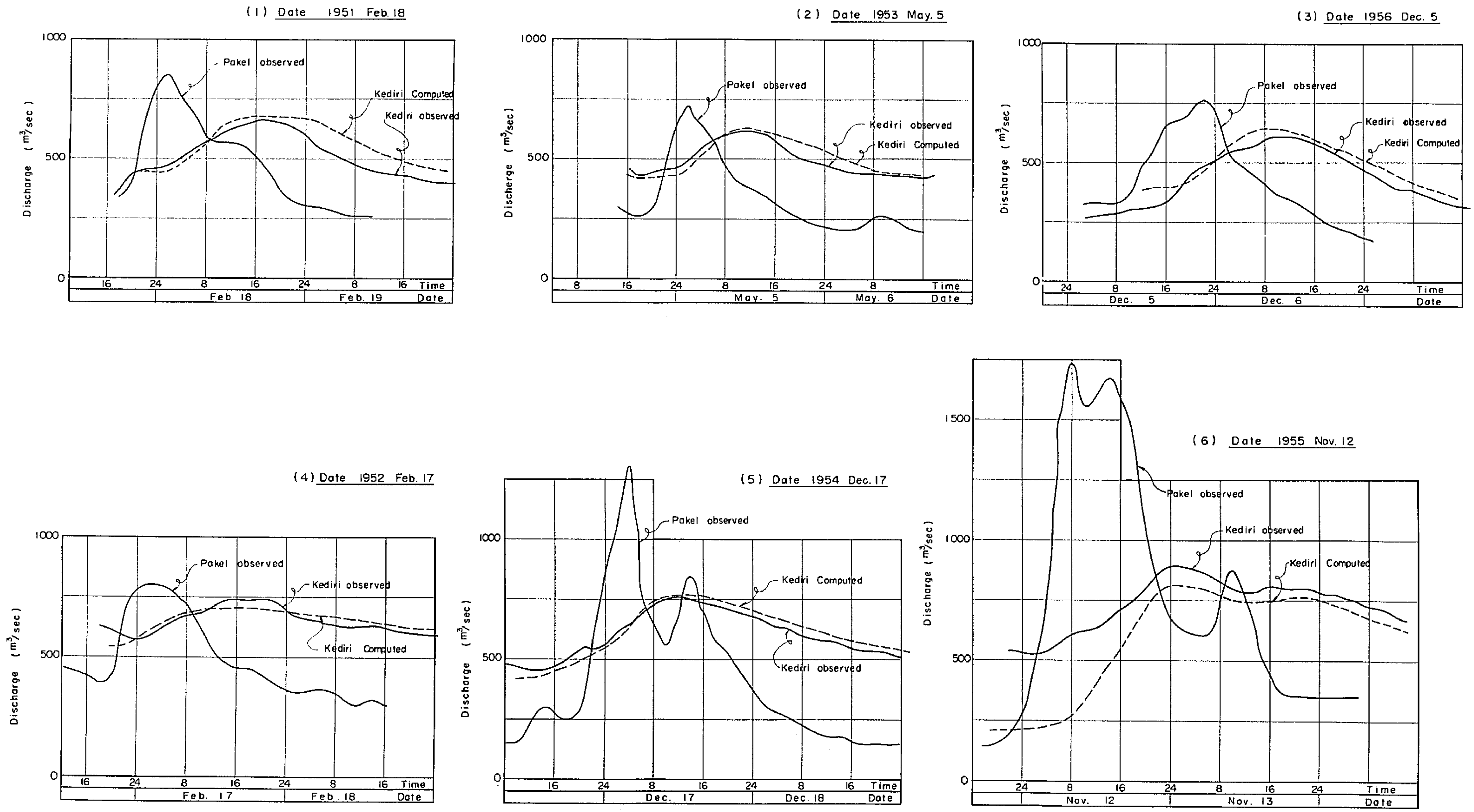


Fig. 6-16 Flood Routing in the Stretch between Pakel and Kediri

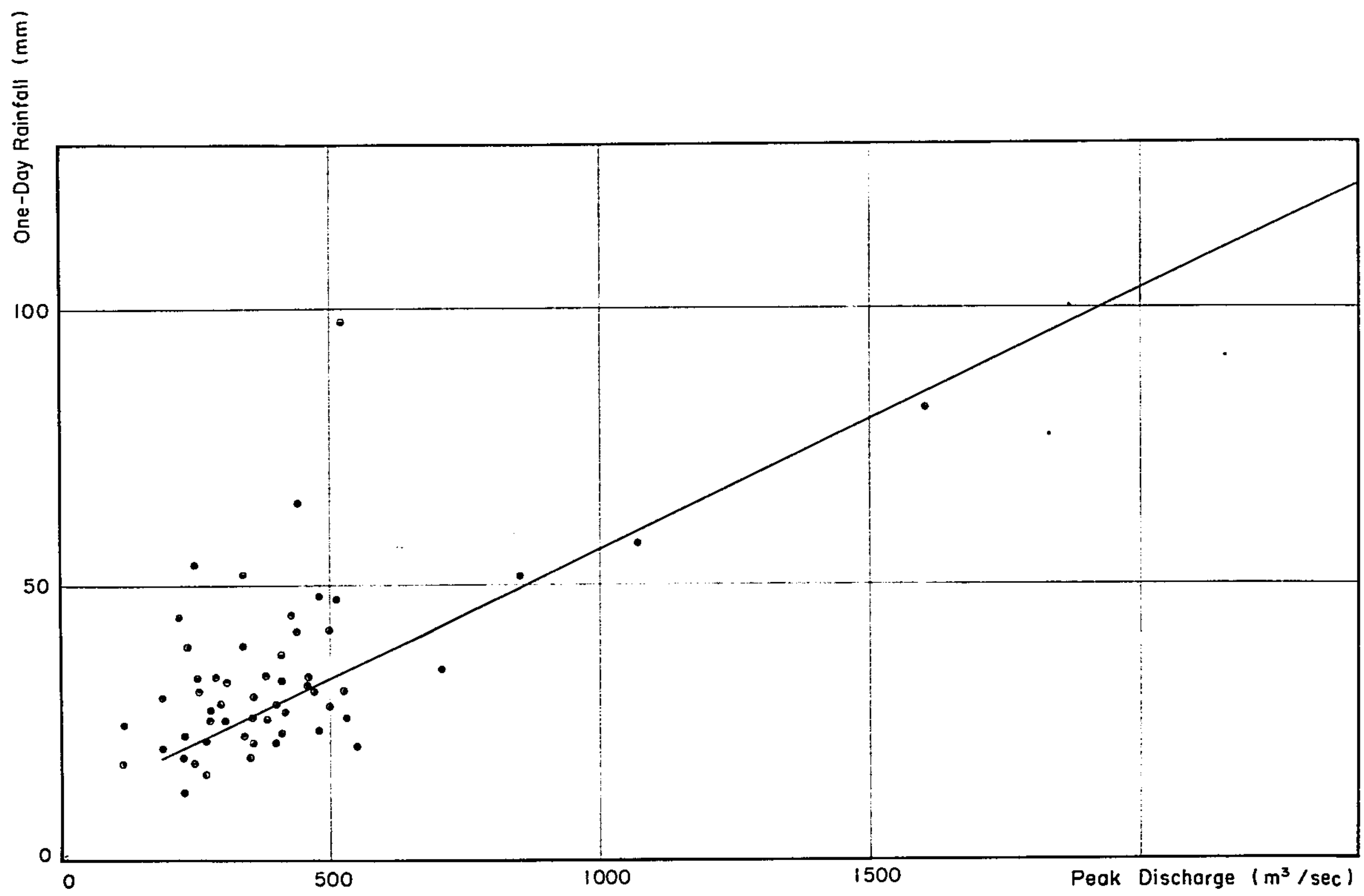


Fig 6－17 Rainfall — Peak Discharge Relationship at Pakel

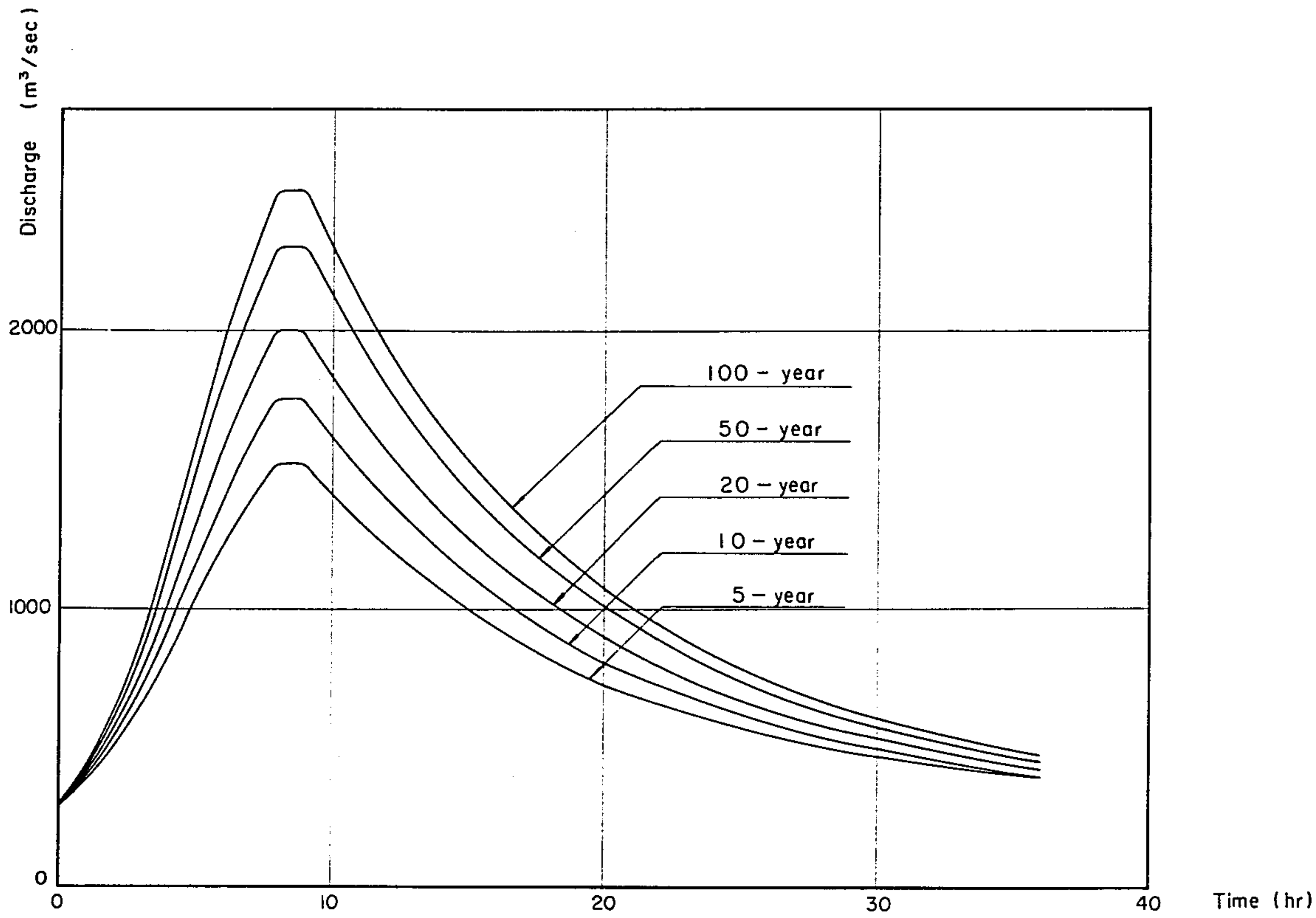


Fig 6—18 Probable Flood Hydrograph at Pakel

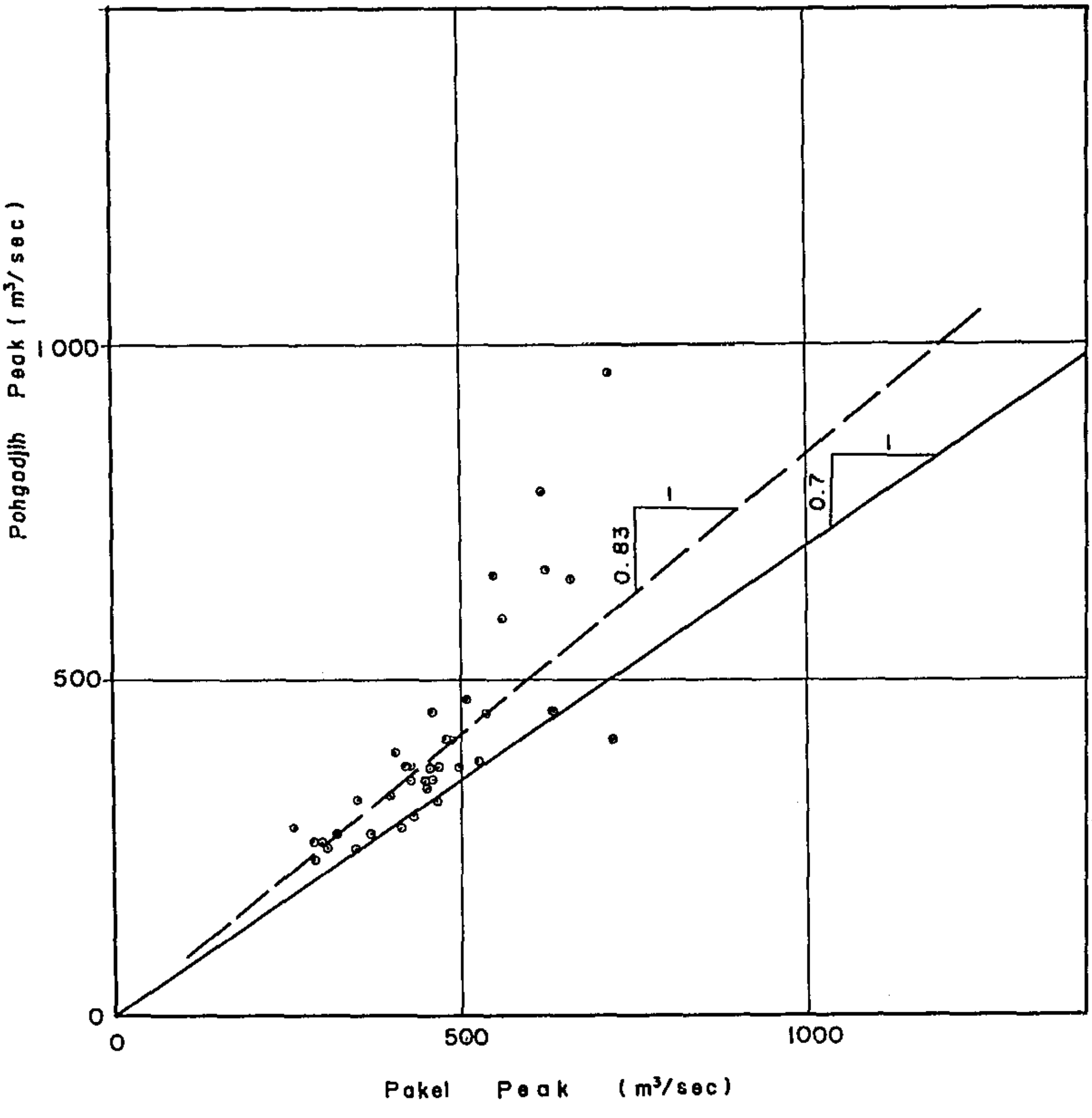


Fig 6—19 Pakel — Pohgadjih Flood Peak Correlation

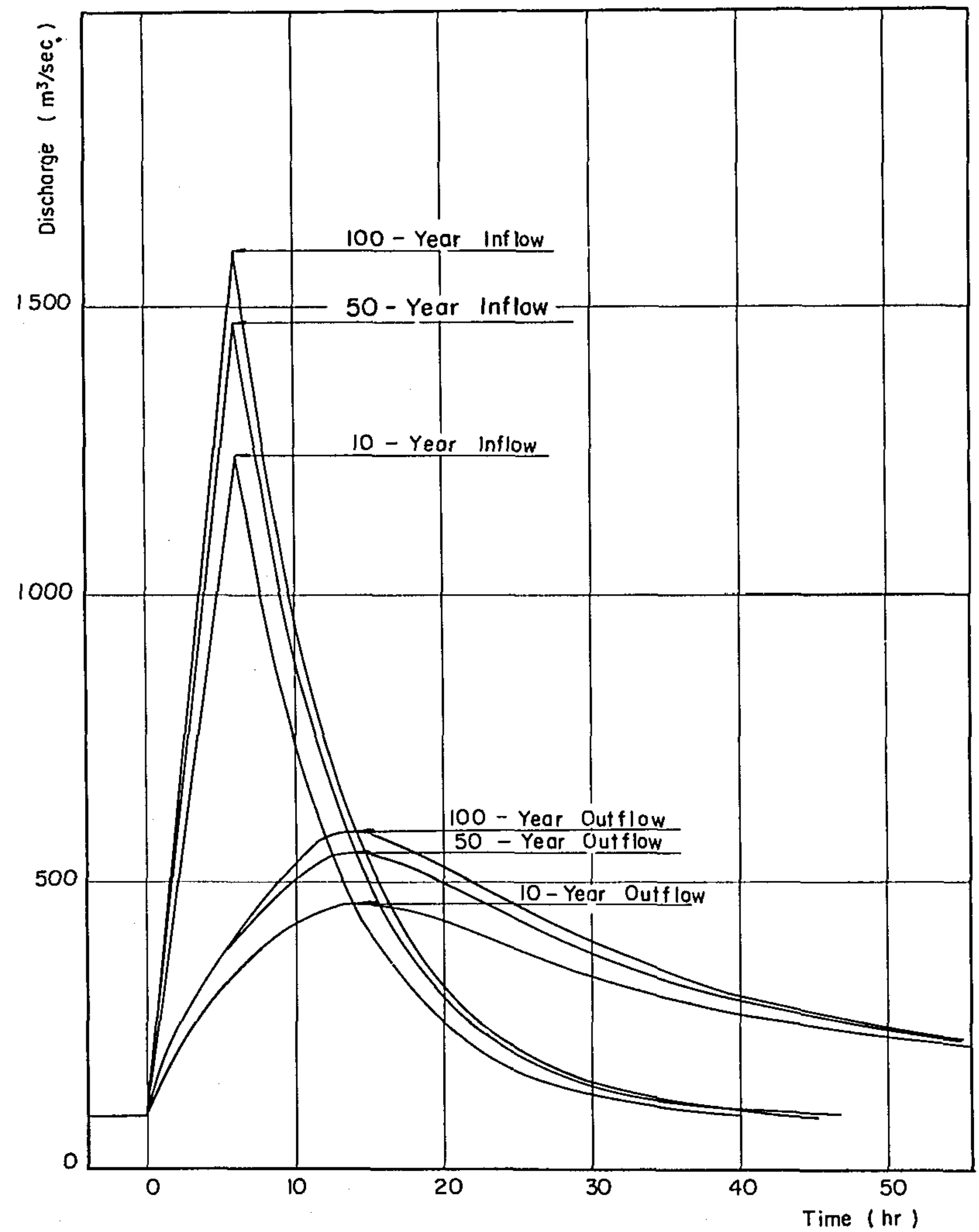


Fig 6-20 Discharge Reduction at Karangkates

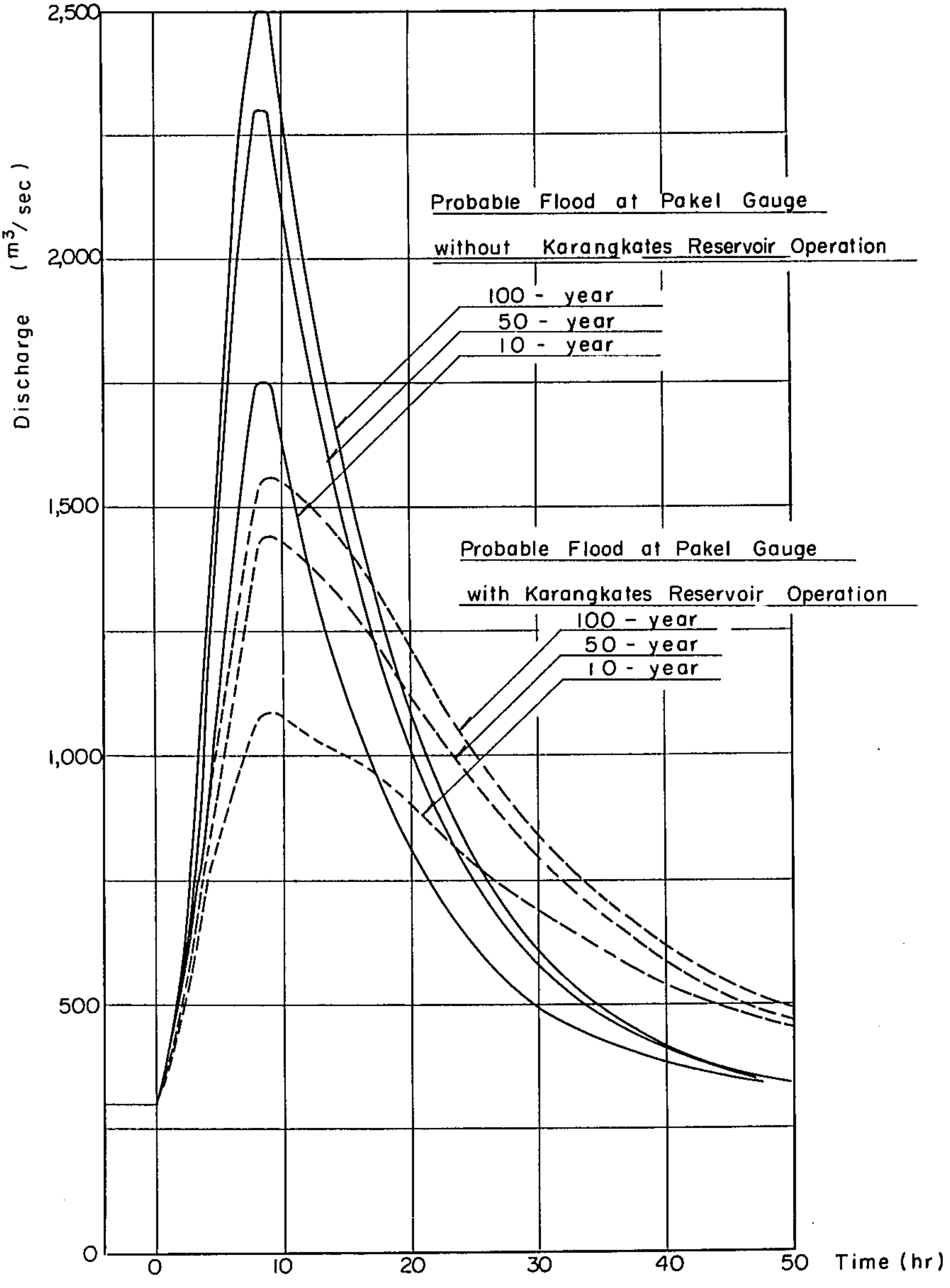


Fig. 6-21 Discharge Reduction at Pakel Gauge

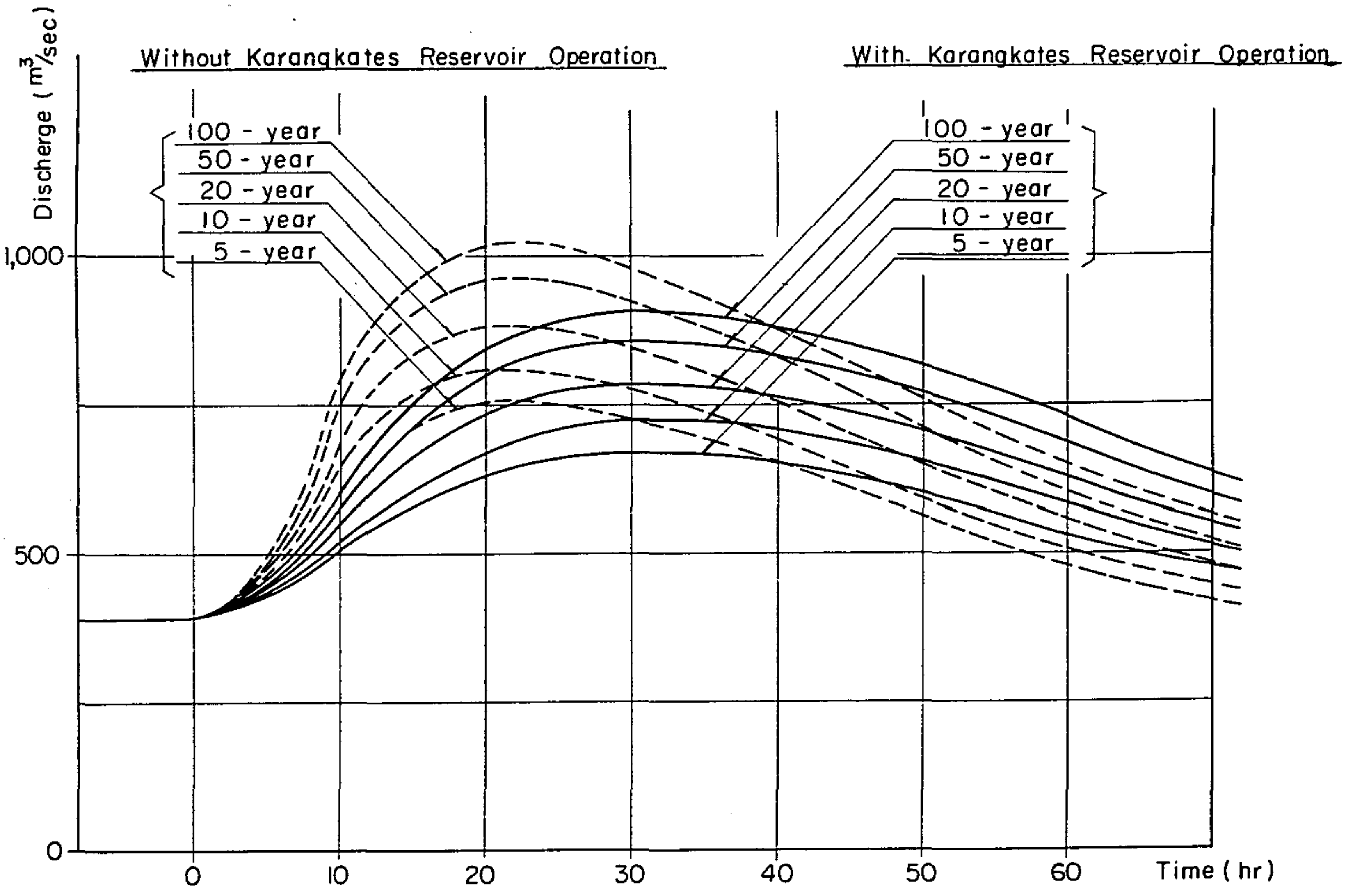


Fig. 6-22 Probable Flood Hydrograph at Kediri Gauge

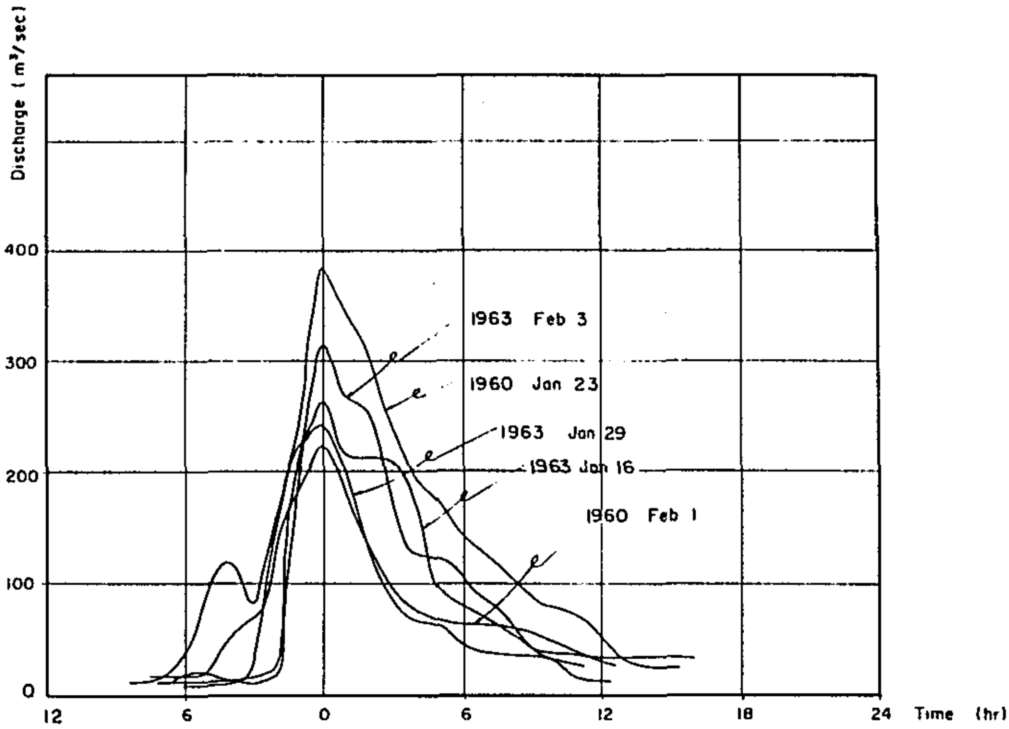


Fig 6—23 Observed Flood Hydrograph at Soeloredjo

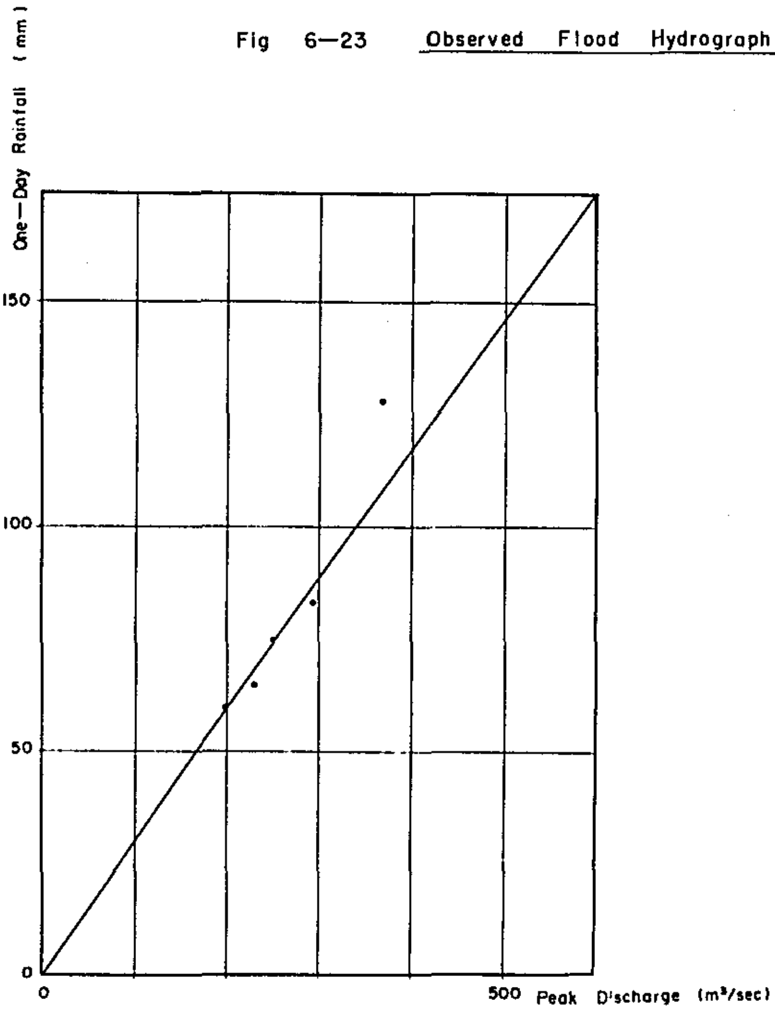


Fig 6-24 Rainfall — Peak Discharge Relationship at Soeloredjo

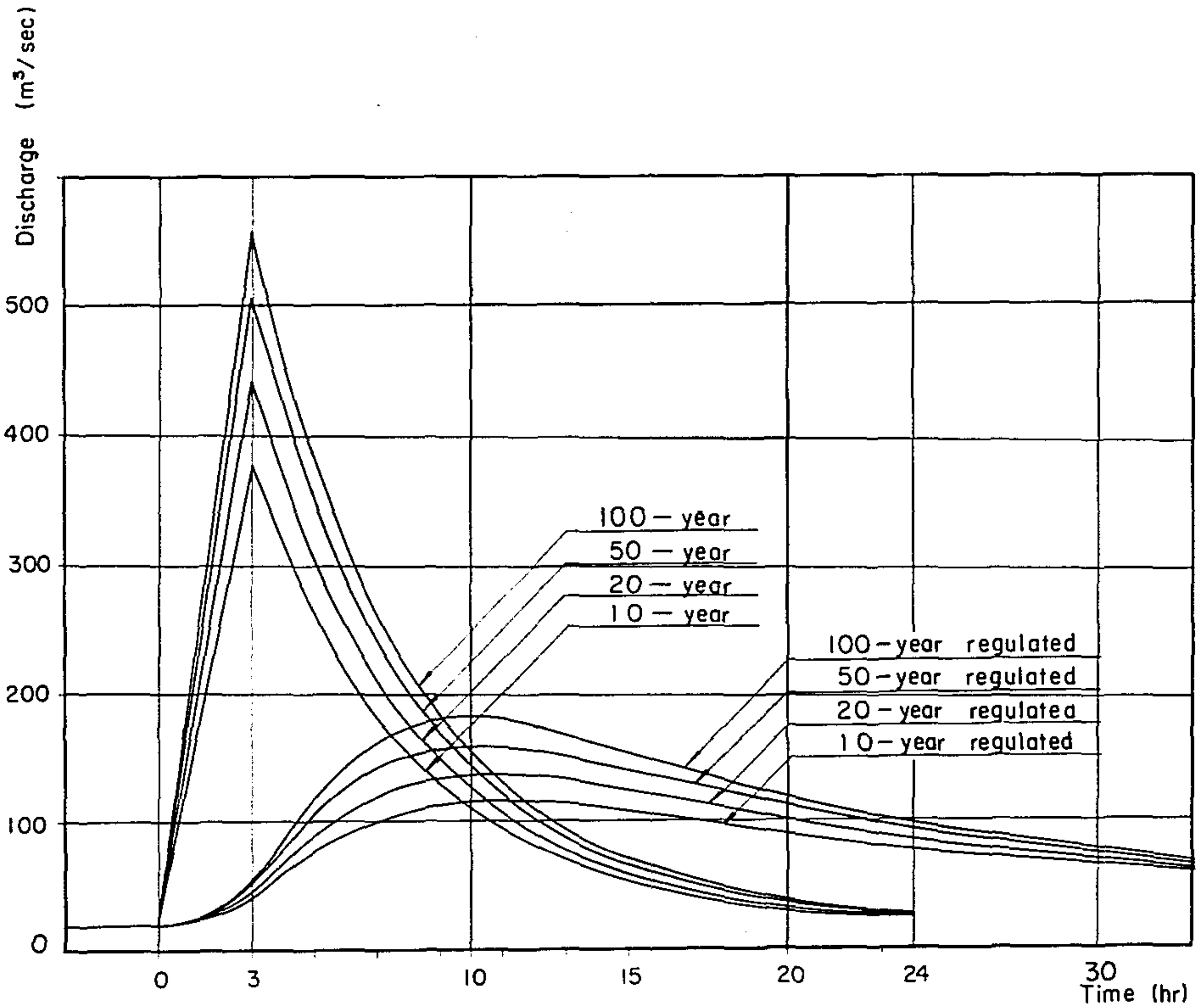


Fig. 6—25 Probable Flood Hydrograph at Soeloredjo

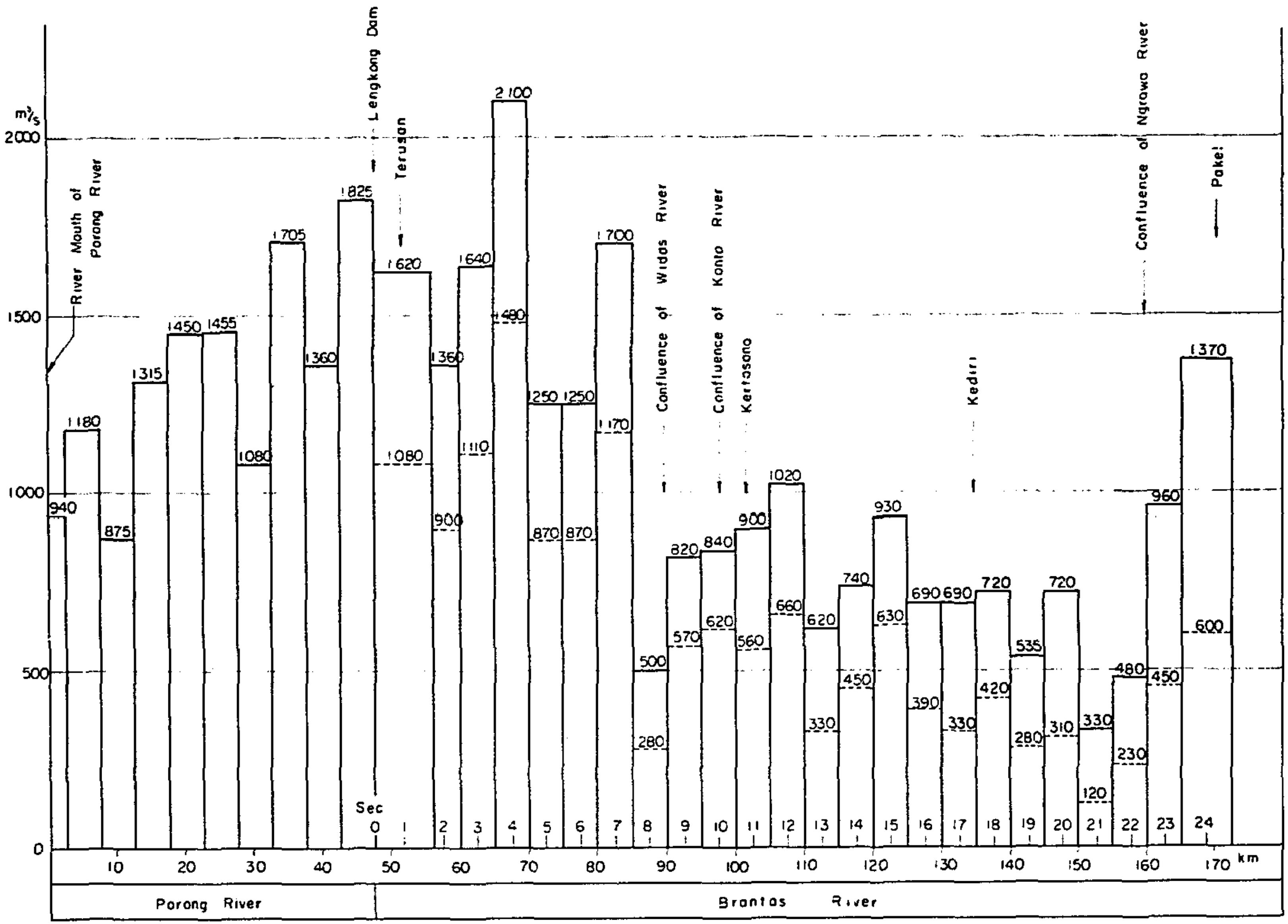


Fig. 6-26 Discharge Capacity of Brantas and Porong River

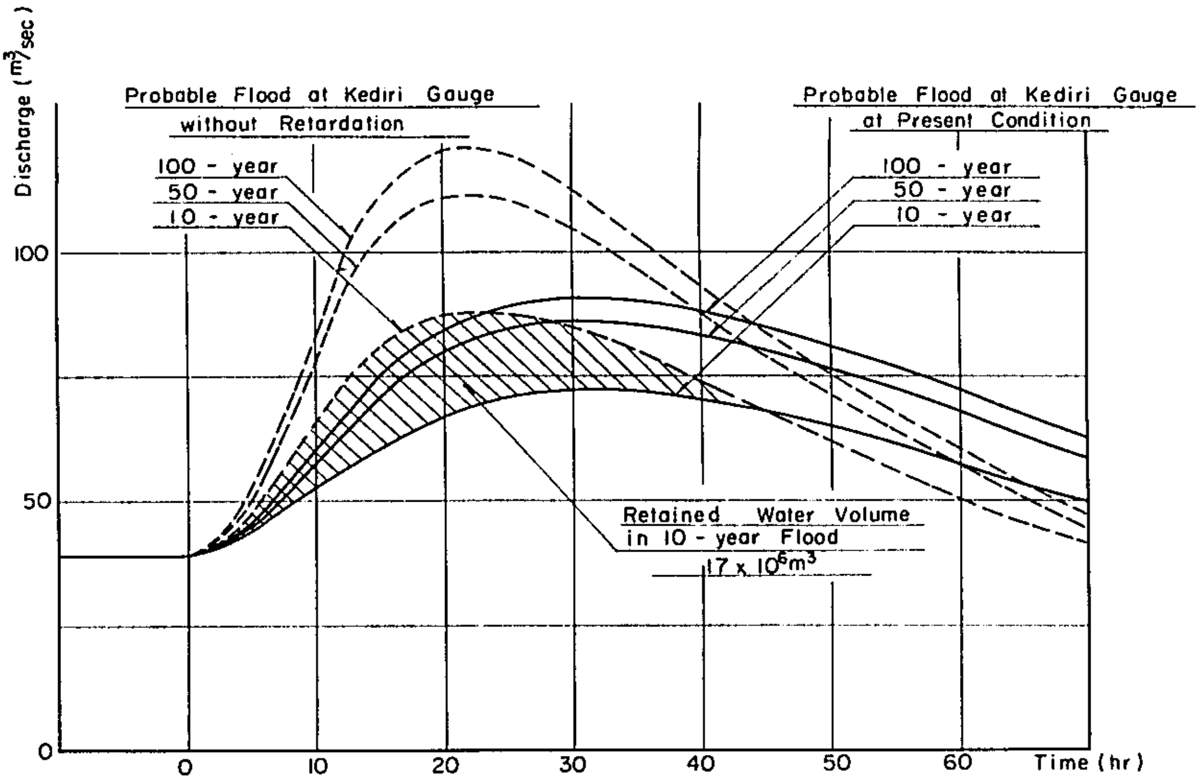
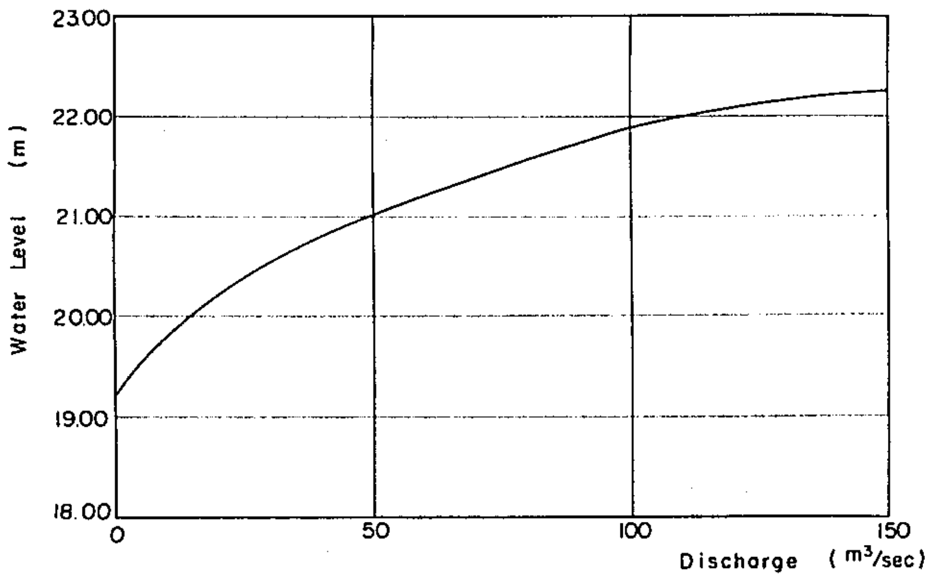


Fig. 6-27 Retardation Effect in the Stretch between Pakel and Kediri



Remarks:

This Stage Discharge Curve is estimated by calculation of non-uniform flow in the Gedek Canal using 0.030 of the Roughness Coefficient

Fig. 6-28 Stage Discharge Curve at Gedek Gate in Gedek Canal

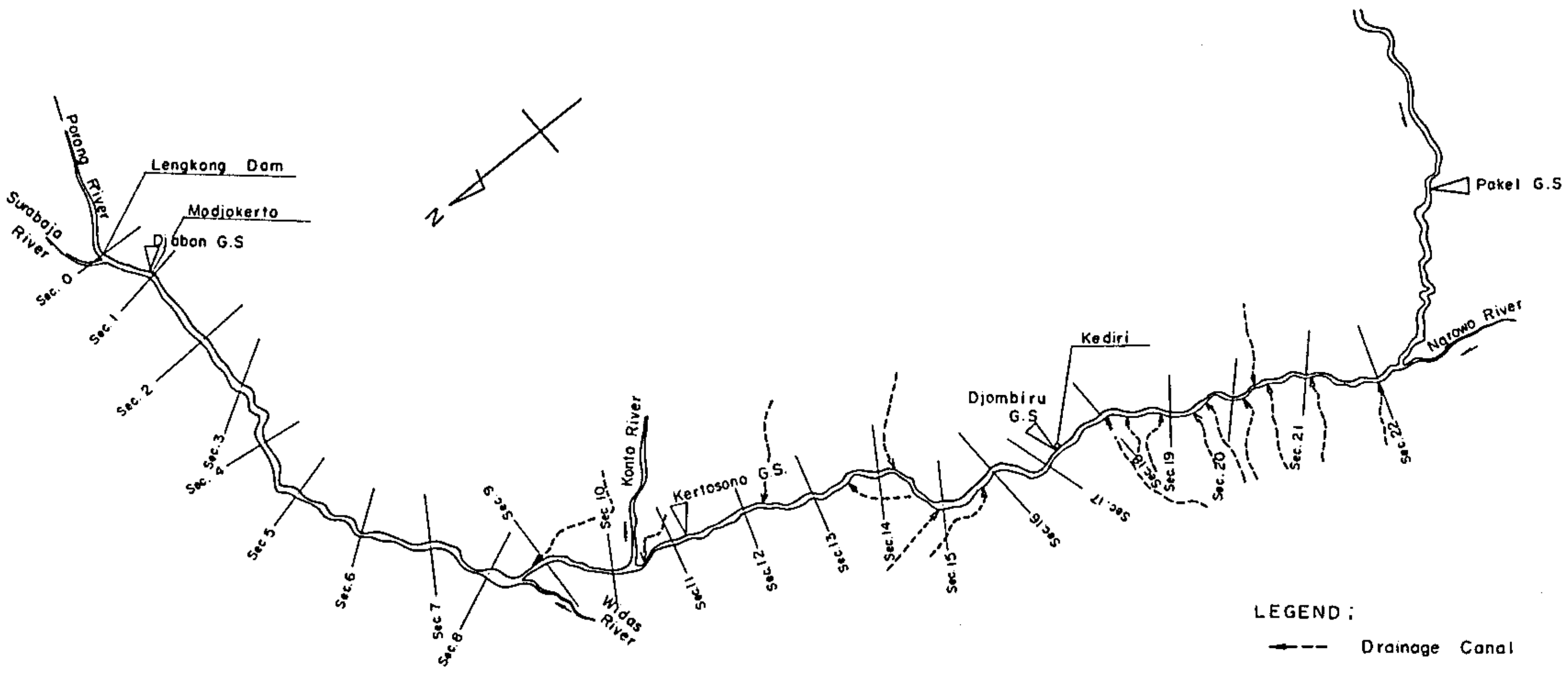


Fig. 6-29 Location of Cross Section

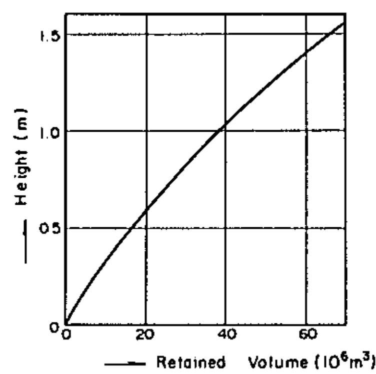


Fig. 6-30 Height - Volume Curve

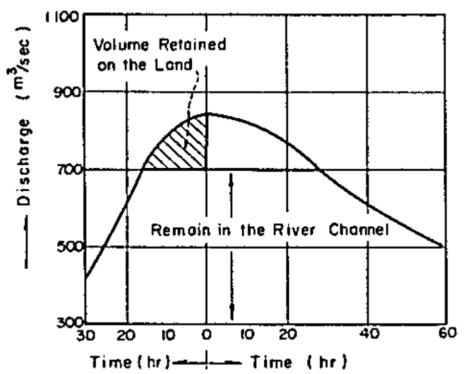


Fig. 6-31 Hydrograph

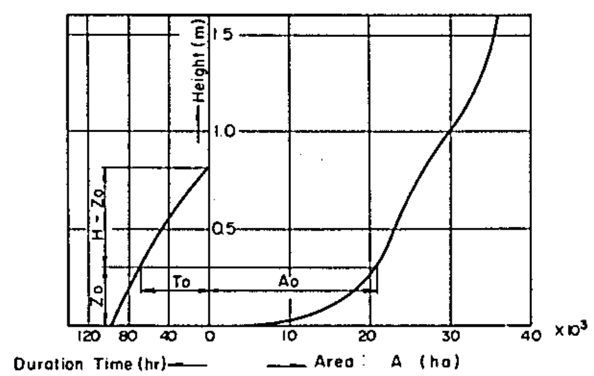


Fig. 6-32 Height - Duration Time - Area Curve

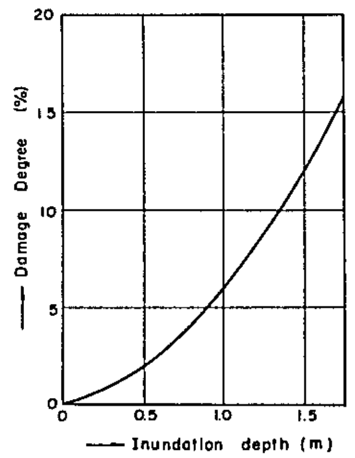


Fig. 6-33 Damage Degree of House

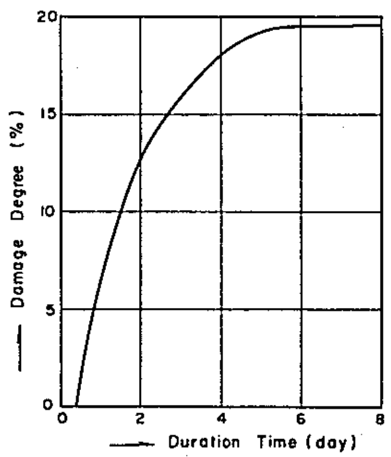


Fig. 6-34 Damage Degree of Upland Crop

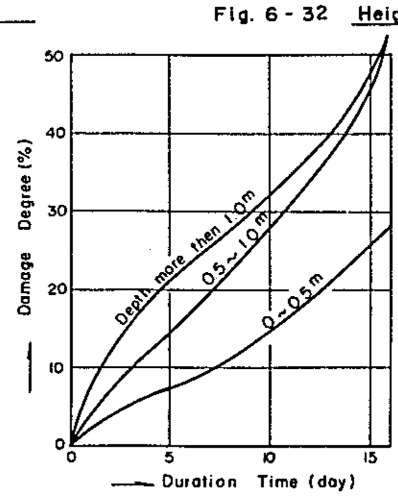


Fig. 6-35 Damage Degree of Paddy Crop

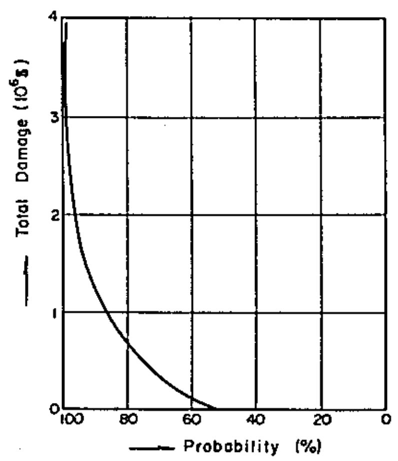


Fig. 6-36 Probability - Damage Curve

7. RIVER IMPROVEMENT PLANNING

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7. RIVER IMPROVEMENT PLANNING

7-1 Introduction

It is quite natural that the improvement of a river system proceeds from the main stream especially from its downstream reaches to the upstream tributaries depending on the intensity of the human activities. However, it has in many cases experienced that more important areas in the downstream reaches is compelled to repeat the reconstruction of river structures due to the endless increase of flood discharge arised from the progressing improvement in the upstream rivers. A systematic construction of the river structure should be based on a well-balanced allocation of flood discharge.

Herein proposed flood discharge distribution plan is prepared based on the results of the flood analysis in Chapter 6. Based on this, any of the river improvement and drainage projects in the Brantas River should be evaluated.

The river improvement plans are proposed to meet the flood discharge distribution plan. A preliminary design and cost estimate for these plans were presented based on the river cross section in intervals of five kilometers.

An alternative plan is also proposed, but its recommendability is still in question due to the limited data of the sediment transportation.

7-2 Flood Discharge Distribution

The distribution of the 50-year flood peak in the Brantas River is drawn for four different cases in Fig. 7-1, based on the estimate in Chapter 6. It is noted that the tributary discharge in this diagram includes the discharge in the minor tributaries in the vicinity.

(1) Present Condition

As the present condition, it is assumed that the southern 1,120 square kilometers of the Ngrowo River Basin is drained to the Indian Ocean through the Nejama tunnel. No reservoir operation is considered.

The results of the flood routing calculation for this case in Chapter 6 show that the flood 1,490 cubic meters per second at the Karangkates damsite increases to 2,300 cubic meters per second collecting the tributary discharges, until it reaches to the Pakel gauge.

From the Ngrowo River, 80 cubic meters of constant discharge joins. On the other hand, the flood discharge in the Brantas River is largely retarded because 28 million cubic meters of water is retained in the vicinity of the Ngrowo mouth. Then the peak discharge at the Kediri gauge becomes 960 cubic meters per second.

The Konto River inundates a considerable area. Other tributaries on the northwestern slope of Mt. Kelut are minor but they develop swamps. The flood discharge from the Konto River and the other tributaries herein is estimated to be 100 cubic meters per second. The 50-year flood peak discharge at the Soeloredjo damsite has been estimated to be 510 cubic meters per second in Chapter 6. Then the balance about 410 cubic meter is regarded to be retained on the land.

The Gedek flood way diverts the flood discharge of 80 cubic meters per second at upper vicinity of the Terusan gauge. The 50-year flood discharge at the Terusan gauge was estimated to be 1,430 cubic meters per second (960 m^3/sec+470 m^3/sec) in Chapter 6. When a flood peak discharge occurs in the Brantas River, the Widas River discharge was regarded as almost constant, because 30 million - 40 million cubic meters of water is retained in the swamp at the river mouth. The Widas discharge is calculated to be 450 cubic meters per second as follows;

1,430 m^3/s	+	80 m^3/s	-	960 m^3/s	-	100 m^3/s	=	450 m^3/s
Terusan		Gedek		Kediri		Konto		Widas

A comparison of the discharge in the main stream with the channel capacity in Fig. 6-26 shows that the entire stretch is in short of the capacity to carry the 50-year flood except some reaches downstream of the Widas mouth.

(2) Reservoir-Operational Condition

The flood control space is 50 million cubic meters in the Karangkates reservoir and ten million cubic meters in the Soeloredjo reservoir respectively. The effect of the operation of these reservoirs was studied in Chapter 6.

The inflow 1,490 cubic meters per second is reduced to 530 cubic meters per second by the Karangkates reservoir. According to the flood routing calculation, this discharge becomes 1,440 cubic meters per second at the Pakel gauge and 860 cubic meters per second at the Kediri gauge, respectively. The Ngrowo discharge and the storage near the Ngrowo mouth are the same as in the previous case.

The peak discharge of 510 cubic meters per second is reduced to 160 cubic meters per second at Soeloredjo damsite. This reduction would not much affect to the main stream but it would largely reduce the inundated area along the Konto River. It is assumed that the inflow from the Konto River is 100 cubic meters like in the previous case and the inundating discharge reduces from 410 cubic meters per second to 60 cubic meters per second. The condition in the left bank remains the same as in the previous case and the Terusan discharge becomes 1,330 cubic meters per second.

The peak discharge is reduced remarkably in the upstream reaches. Its reduction in the lower stretch is about 100 cubic meters per second.

(3) Substantially Drained Condition

The Ngrowo, the Konto and the Widas Rivers and the Gedek canal are the tributaries involving major flood problem areas. It is assumed that these areas are drained. The large retardation in the Pakel - Kediri stretch can not be explained from only the channel storage. Certain areas must be inundated when a big flood occurs. The elimination of the storage on these lands is also assumed in this case.

The peak discharge down to the Pakel gauge is the same as in the reservoir-operational condition. The inflow from the Ngrowo River Basin will increase by 80 - 130 cubic meters per second by construction of a cut-off canal which will be discussed in Chapter 8. This increased flood flow will be so little in volume that it will be retarded by a small channel storage. The increased flow will be reduced to almost a constant flow in the downstream reaches. It is assumed that the increase in the Kediri discharge due to the cut-off canal construction is 30 cubic meters per second. The Kediri discharge with a reduced storage in the Pakel - Kediri stretch was estimated to be 260 cubic meters per second more than that with the present storage. Then the Kediri discharge 860 cubic meters per second will be magnified to 1,150 cubic meters per second.

By improvement of the downstream reaches of the Konto River, the discharge 60 cubic meters per second to the inundated area will directly flow into the Brantas River. Then the inflow from the Konto River becomes 160 cubic meters per second. The storage of 30 million - 40 million cubic meters in the swamp at the Widas mouth will be lost by drainage of this area. Then the discharge from the Widas River will become 750 - 850 cubic meters per second. The shut down of the Gedek canal for prevention of the inundation along the canal will result in an increase in the Terusan discharge by 80 cubic meters per second. Consequently, the Terusan discharge will be 2,060 - 2,160 cubic meters per second.

The flood discharge downstream of Kediri will be increased far beyond of the present channel capacity by the improvement assumed in this case.

The Brantas River has aggraded due to the 1951 and 1966 eruptions of Mt. Kelut. The river bed rise in the 1951-1970 period was about 1.5 meters. The critical situation of the river channel seen under the present condition has arised from this aggradation.

The operation of the Karangkates Reservoir remarkably reduces the flood discharge in the upstream reaches. The Soeloredjo dam can substantially solve the flood problem along the Konto River. However, even with these, the flood danger arised from the recent aggradation still remains. An increase of the channel capacity by dredging is necessary.

The distribution of the flood discharge for the substantially drained condition showed that the drainage in the tributary area as well as prevention of the main stream inundation will cause a increase of the main stream flow. If all the tributary's flood would be perfectly drained to the Brantas River, the main stream discharge will be uncontrollably magnified.

The estimate for the substantially drained condition suggests that both the reservoir and the natural storage are quite effective to reduce the flood discharge. The improvement of the drainage condition in the tributary area should involve a construction of a reservoir in the upstream reaches where possible. Certain swamps and inundation areas should be regarded as retardation basin which works to reduce the flood discharge in the main stream. The distribution of the flood discharge in the following condition is proposed based on the above-derived criteria.

(4) Proposed Distribution

The retardations near the Ngrowo mouth and the Widas mouth are left as they are now. The drainage along the Ngrowo River by a cut-off canal is included, because it will increase the Brantas flood only a little. The major part of the flood discharge in the Konto River is substantially controlled by the Soeloredjo dam. Therefore, the Konto River is improved to an extent that no inundation occurs. The Gedek flood way is closed off to protect the land along the Gedek canal. The peak discharge at the water gauges in this plan is as follows;

Karangkates	530 m^3/s
Pakel	1,440 m^3/s
Kediri	890 m^3/s
Terusan	1,500 m^3/s

The proposed channel capacity in each stretch is shown in the flood discharge distribution diagram in Fig. 7-1. The Mlirip gate, which is located at 3.2 kilometers downstream of Terusan gauge, should be also closed off to protect the urban area of Surabaja. Therefore, the flood discharge to the Porong River is the same as that at Terusan gauge.

7-3 Overall Plan

The river improvement plan of the Brantas River was prepared based on the flood discharge distribution proposed in Section 7-2. The plan involves the increase of the channel cross section of the main stream between the Ngrowo mouth and the Terusan gauge, construction of a cut-off canal in the Ngrowo River Basin, improvement of the downstream channel of the Konto River, and shut down of the Gedek flood way and Mlirip gate.

The proposed design of the Brantas River and Konto River is shown in Figs. 7-2 and 7-3, respectively. These drawings also show the design of the first stage construction, which is described in Section 7-4. The improvement plan of the Ngrowo River Basin is discussed in Chapter 8.

In determining the river cross section, the river width was confined within the present width. The bottom elevation was determined taking into account the elevations of the existing irrigation intakes, bridges and other structures. The free board is one meter. The bottom width of the low water channel is 70 meters upstream of the Widas mouth and 100 meters downstream of the Widas mouth respectively.

The total construction cost for the overall plan is estimated to be 25.9 million dollars as shown in Table 7-1. This estimate is based on the earth work quantity calculated from the river cross section in the intervals of five kilometers. This estimate may involve some error. Assuming that the error in the estimate is ±25 percent, the construction cost will be between 19 million and 33 million dollars.

The cost estimate did not include the cost for stabilization of the river bed in the upper stretch. It was predicted in Chapter 5 that the upstream degradation and downstream aggradation will continue under the natural condition. Certain cost will be required for construction of ground sill, submerged groyne dam, etc. in the upper stretch.

7-4 First Stage Construction

In the overall plan, the dredging volume 15 million cubic meters will control the construction schedule and require substantial amount of cost.

For the purpose of saving the initial cost and time for construction, it is proposed to initiate the construction with the first stage in which the structure will be constructed with the minimum required safety, i.e., 80-centimeter free board against a ten-year flood instead of one-meter free board against a 50-year flood.

The flood discharge distribution for the first stage construction is shown in Fig. 7-4.

The river channel is shallow in the first stage as shown in the profile in Fig. 7-2. The width of the low water channel is also less than that of the overall plan.

The dredging volume in the first stage is estimated to be about a half as much as that in the overall plan. The construction period will be six years, and the construction cost is estimated to be 16.9 million dollars as shown in Table 7-2. With the same assumption as in Section 7-3, the actual construction cost will be in the range of 13 million and 21 million dollars.

7-5 Diversion Canal Scheme

Herein outlined is an alternative plan of the overall plan. This plan contemplates to reduce the discharge in the downstream reaches by diverting the total discharge in the upstream reaches of the Brantas River into the Indian Ocean. By this plan, the drainage of the areas between Pakel and Kediri, improvement of the Konto River, and closure of the Gedek flood way are made possible, without changing the present cross section of the Brantas River. A weir is constructed across the Brantas River three kilometers downstream of the Pakel gauge. The gate installed on the weir is closed in the rainy season and opened in the dry season. A 30-kilometers-long diversion canal is constructed from upstream of the weir to the Nejama site where diversion tunnel is constructed close to the present Nejama tunnel (see Chapter 8). The alignment of the diversion canal is shown in Fig. 7-6.

The water level in the downstream of the weir will be lowered, and therefore, the areas along the Brantas River for several ten kilometers downstream of the weir will be drained well. The river stretch between the weir site and Kediri is 32 kilometers in length. The catchment area of this stretch is 1,380 square kilometers, excluding the southern 1,100 square kilometers of the Ngrowo River Basin which has been drained to the Indian Ocean. This area may yield about 700 cubic meters per second of flood discharge, under well-drained condition. A rough estimate showed that this discharge is reduced to 500 cubic meters per second at Kediri, assuming the channel storage between the weir site and Kediri is ten million cubic meters.

The flood discharge distribution diagram for the alternative plan, assuming the Kediri discharge 500 cubic meters per second is shown in Fig. 7-5. The discharge in the downstream stretch is calculated to be 660 cubic meters per second between the Konto and Widas mouths, and 1,110 cubic meters per second in further downstream.

A comparison of these results with the channel capacity in Fig. 6-26 shows that the flood discharge in the alternative plan can be safely carried by the present river channel.

The inlet discharge of the new diversion canal is determined by the Pakel discharge 1,440 cubic meters per second. Since the canal can collect the discharge in the Ngrowo River Basin, the southern 1,100 square kilometers will be more effectively drained. The total outflow from this area to the Nejama site is roughly estimated to be 600 - 800 cubic meters per second. Then the total discharge capacity of the existing and new tunnels is determined to be 2,040 - 2,240 cubic meters per second.

The construction cost of this scheme will amount to 40 million - 50 million dollars.

The construction for improvement of the Porong River is going on. The size of the structures is determined based on the design flood discharge 1,500 cubic meters per second at the Terusan gauge. Within this limitation, the drainage of the large swamp at the Widas mouth will be made possible, if this alternative scheme would be implemented.

The sediment problem in the middle and the lower streches of the Brantas River will be lightened in a great extent by this alternative scheme. On the other hand, the majority of the sediment load in the Brantas River will be shifted to the new diversion canal especially when an eruption of Mt. Kelut would occur in the future. The river bed movement in the Brantas River, which is highly affected by the volcanic efflux, is not fully made clear yet. The river bed movement analysis in Chapter 5 showed some evidence that the sediment transportation shortly after the eruption is so large as being beyond of the theoretical interpretation. The decision on this scheme has to be left for the further investigations.

7-6 Conclusions

(1) A flood discharge distribution plan was proposed based on the results of the flood analysis in Chapter 6. In view of saving the cost for improvement of the river, of which the discharge capacity is too small, certain swamps are proposed to be left undrained because of their great retardation effects. The proposed flood discharge in the main stream is 1,200 cubic meters per second between the Ngrowo mouth and Kediri, 900 cubic meters per second between Kediri and the Konto mouth, 1,100 cubic meters per second between the Konto and Widas mouth, and 1,500 cubic meters per second between the Widas mouth and Terusan. This plan was formulated against the 50-year flood.

(2) The overall improvement plan comprises the increase of the discharge capacity of the Brantas River downstream of the Ngrowo mouth by dredging and levee heightening, drainage of the areas along the Ngrowo River by construction of a cut-off canal, improvement of the lower stretch of the Konto River and closure of the Gedek flood way and Mlirip gate. The inundation area near the Ngrowo mouth and the swamp at the Widas mouth are regarded as the retardation basins. The construction cost will be 19 million - 33 million dollars.

(3) The first stage development of the river improvement plan was proposed with a construction cost 13 million - 21 million dollars. The design flood for the first stage is ten-year flood as an intermediate target.

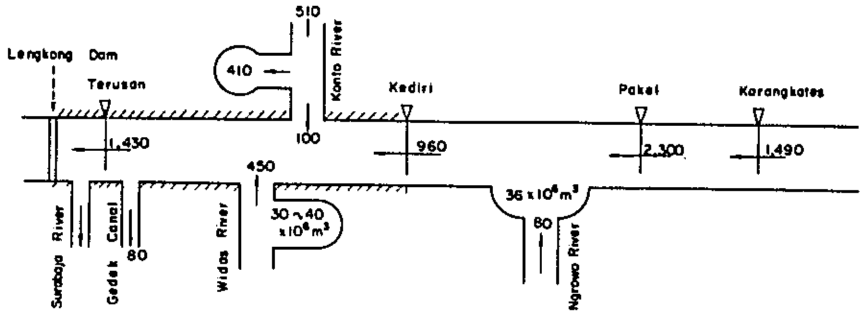
(4) An alternative plan was proposed. This is the diversion of the upstream reaches of the Brantas River to the Indian Ocean, by constructing a gated weir near the Pakel gauge, a 30-kilometer-long diversion canal and tunnels. The total construction cost will be 40 million - 50 million dollars. By this plan made possible are the drainage of the areas upstream of Kediri and improvement of the Konto River without changing the present channel cross section of the Brantas River. The drainage of the swamp at the Widas mouth is also possible without any harmful influence to the downstream rivers. The sediment problem in the lower stretch will be much reduced by this scheme, while the same arises in the new diversion system. Since the sediment transportation mechanism in the volcano-affected Brantas River is not fully clear yet, this scheme has to be left to the future study. The catchment area of the diversion canal is about a half of the whole basin area. This means that the implementation of this scheme is a great change in the hydrological condition of the Brantas River. Careful study is necessary for preventing unforeseen problems.

Table 7-1 Construction Cost Estimate for Overall Plan

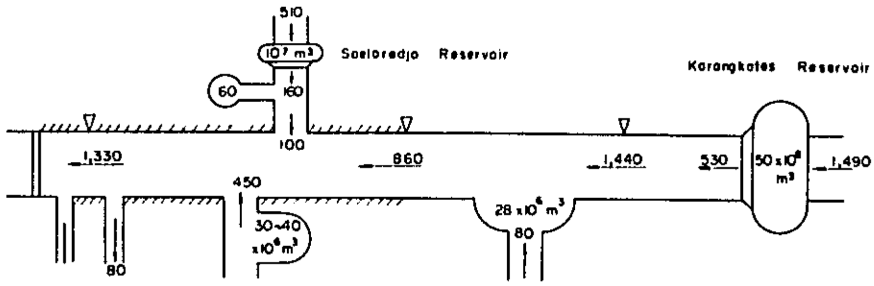
	Quantity ($10^6 m^3$)	Unit Cost ($\$/m^3$)	Amount ($10^6 \$$)
1. Main Stream Improvement			
Dredging	15.0	0.5	7.50
Embankment	7.0	0.8	5.60
Revetment, Groyne & Other Structures			3.90
Compensation			0.80
Engineering & Administration			1.20
Sub-total			19.00
Interest during Construction (6 % interest, 10 years)			5.70
Total			24.70
2. Konto River Improvment			
Dredging	0.3	0.5	0.15
Others			0.05
Total			0.20
3. Ngrowo River Improvement			
(See Chapter 8)			1.00
Grand Total			25.90

Table 7-2 Construction Cost Estimate for First Stage Construction

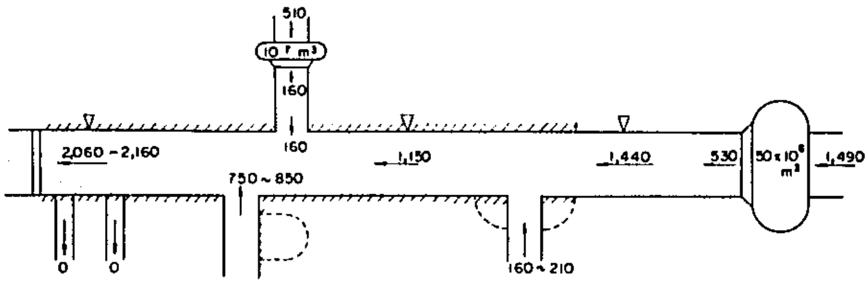
	Quantity ($10^6 m^3$)	Unit Cost ($\$/m^3$)	Amount ($10^6 \$$)
1. Main Stream Improvement			
Dredging	7.0	0.5	3.5
Embankment	7.0	0.8	5.6
Other Structures			2.7
Compensation			0.6
Engineering & Administration			0.9
Sub-total			13.3
Interest during Construction (6 %-interest, 6 years)			2.4
Total			15.7
2. Konto River Improvement			
(Table 7-1)			0.2
3. Ngrowo River Improvement			
(Table 7-1)			1.0
Grand Total			16.9



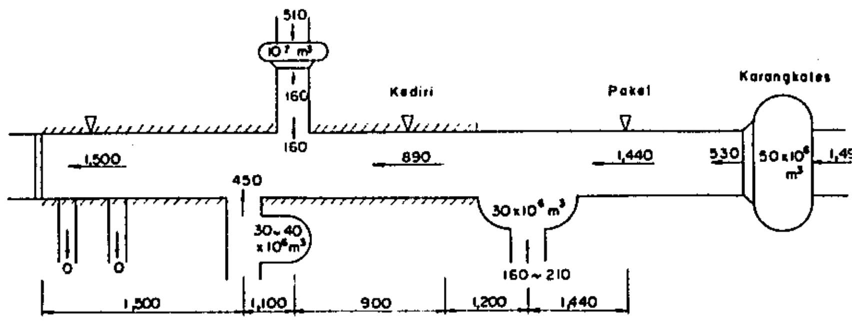
(1) Present Condition



(2) Reservoir - Operational Condition

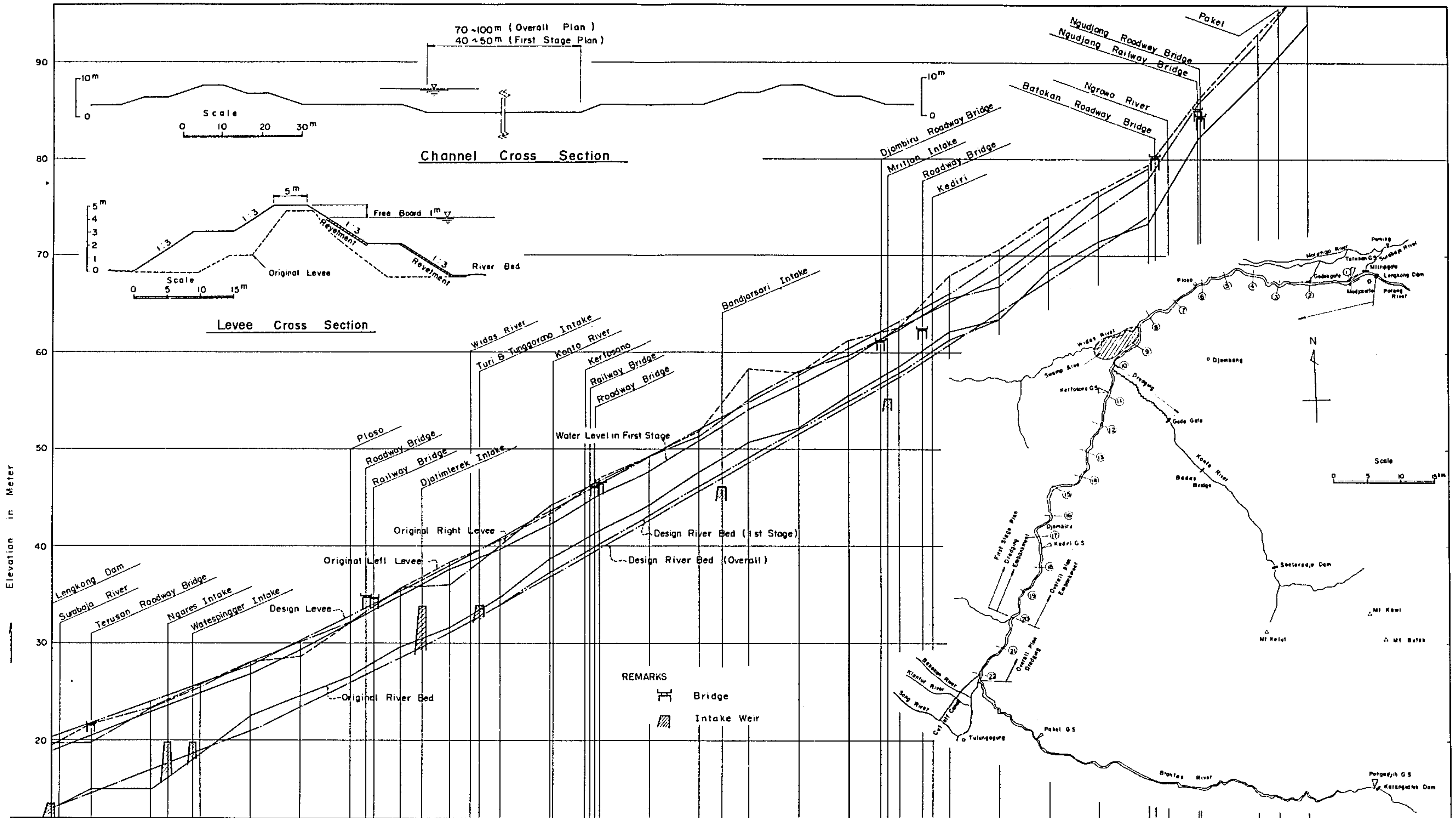


(3) Substantially Drained Condition



(4) Proposed Distribution

Fig. 7 - 1 Flood Discharge Distribution Diagram



Section No.	0	1	2	3	4	5	6	7	8	9	10	11	12
River Kilometer	47.40	51.40	57.40	62.40	67.40	72.40	77.40	82.40	87.40	92.40	97.40	102.40	107.40
Planned Elevation: Levee (m)	20.30	21.80	24.00	25.90	27.70	30.20	32.80	35.40	38.00	40.80	43.60	46.40	49.20
Planned Elevation: River Bed, First Stage (m)	13.00	14.60	17.00	19.00	21.00	23.50	26.00	28.50	31.00	33.90	37.30	40.30	43.20
Planned Elevation: River Bed, Overall (m)	13.00	14.60	17.00	19.00	21.00	23.50	26.00	28.50	31.00	33.90	36.80	39.80	42.70

Section No.	13	14	15	16	17	18	19	20	21	22	23	24	25
River Kilometer	112.40	117.40	122.40	127.40	132.40	137.40	142.40	147.40	152.40	157.40	162.40	168.40	173.40
Planned Elevation: Levee (m)	52.00	54.80	57.70	60.40	63.20	66.00	69.50	73.00					
Planned Elevation: River Bed, First Stage (m)	46.20	49.10	52.00	55.00	57.90	61.30	63.40	68.50	71.50	73.40			
Planned Elevation: River Bed, Overall (m)	45.70	48.60	51.50	54.50	57.40	60.80	64.10	67.40	70.70	74.10			

Fig. 7-2 River Improvement Plan (Brantas River)

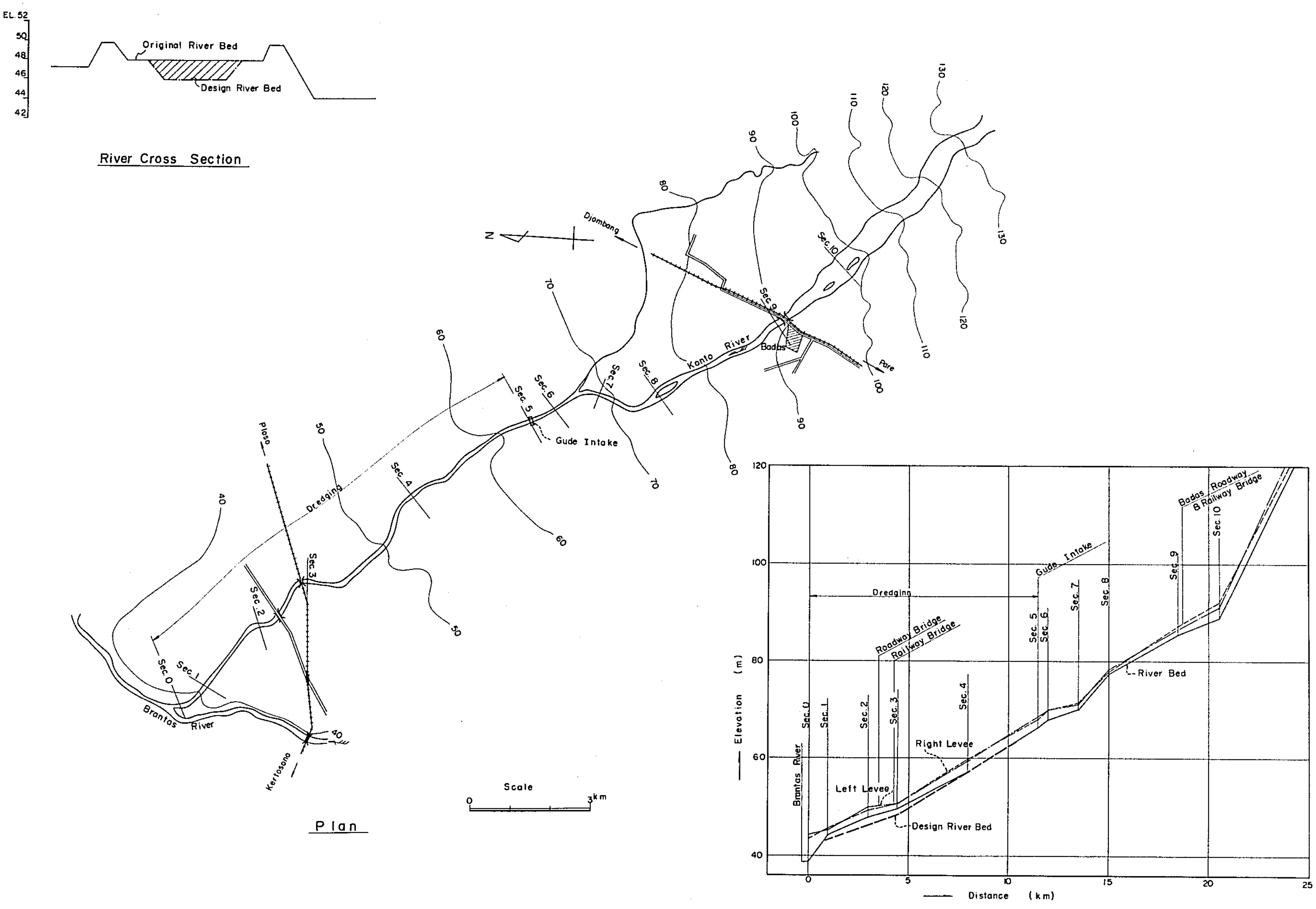


Fig. 7-3 River Improvement Plan (Konto River)

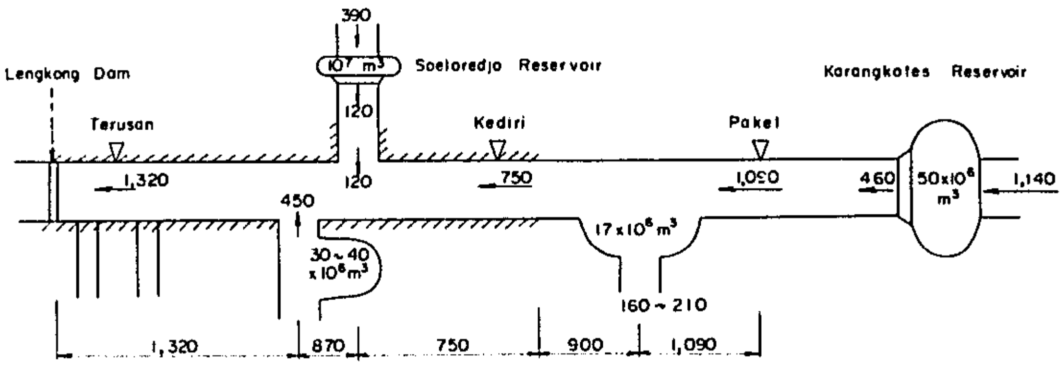


Fig. 7-4 Flood Discharge Distribution for First Stage Construction

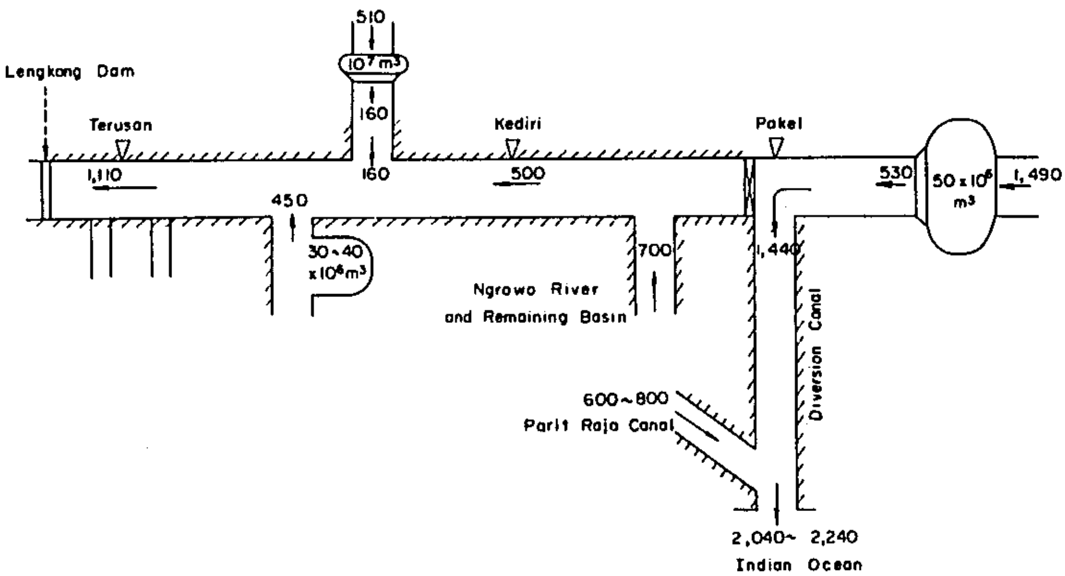


Fig. 7-5 Flood Discharge Distribution for Diversion Canal Plan

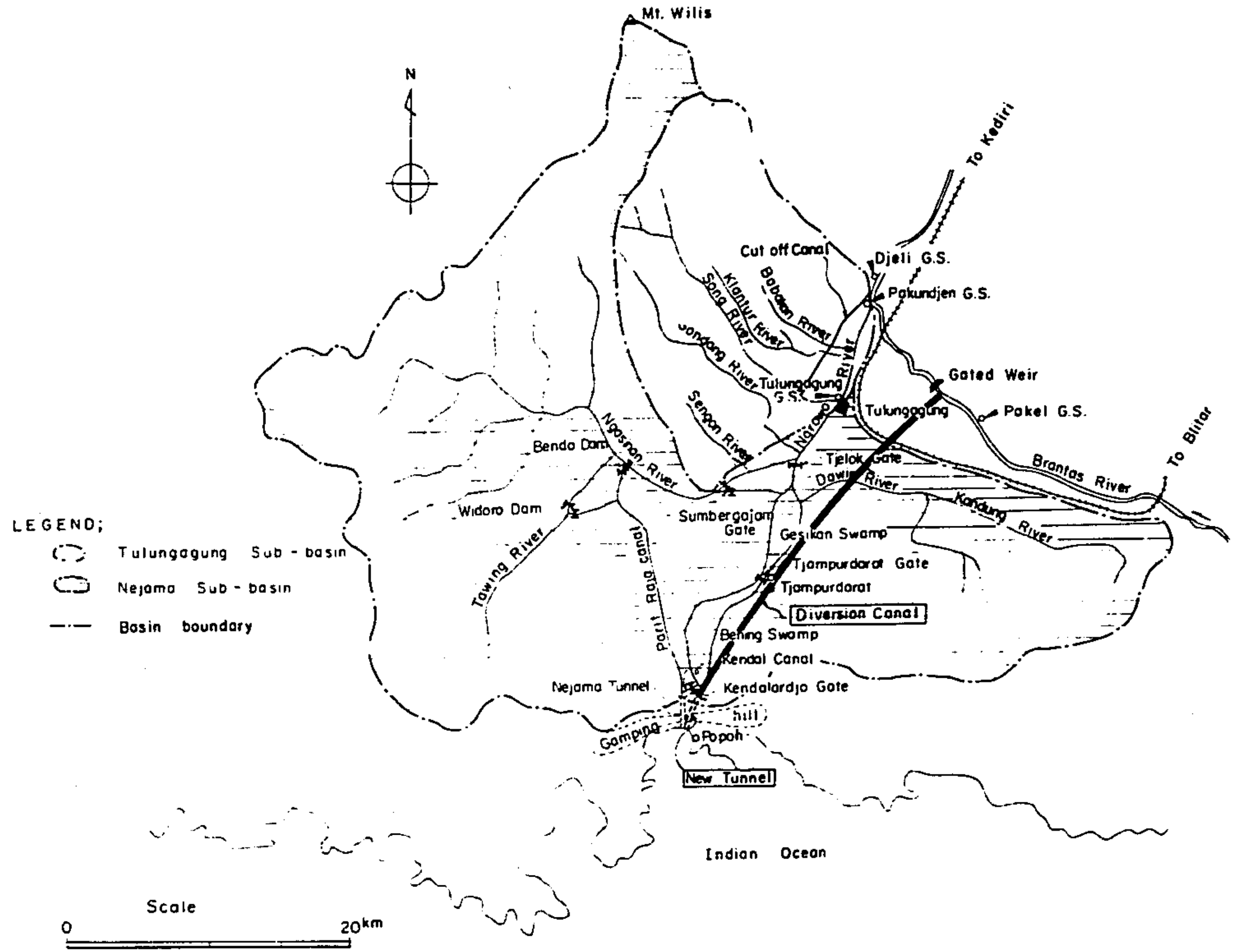


Fig. 7-6 Diversion Canal Plan

8. FLOOD PROBLEM ALONG THE NGROWO RIVER AND ITS BASIC SOLUTION

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8. FLOOD PROBLEM ALONG THE NGROWO RIVER AND ITS BASIC SOLUTION

8-1 Introduction

The Ngrowo River Basin is located in the southwest of the Brantas River Basin. There are remarkable swamps in the Ngrowo River Basin. Effort has long been paied for drainage and utilization of these swamps, because the surrounding plain is intensively cultivated.

The problem recently has risen is the flood control in the main stem of the Ngrowo River. Tulungagung city is located on the right bank of the Ngrowo River about eight kilometers upstream of the mouth. The lowland near this city is ill-drained because the Ngrowo River and its tributaries have been silted up.

Herein explained is a study of the flood problem along the Ngrowo River. Some possible solutions are evaluated on a reconnaissance level and the direction of the basic solution is indicated.

8-2 Ngrowo River System

8-2-1 Natural Condition

The Ngrowo River Basin is located in the southwest of the Brantas River Basin. Its southern watershed is Gamping hill which is a narrow and east-west trending ridge facing the Indian Ocean. The western boundary is featured by Mt. Sumber, Mt. Bulu and Mt. Gepeng. The northern part is the slope of Mt. Wilis (EL. 2,563 m). The basin neighbours with those of small tributaries of the Brantas River in the northeast. The basin area is 1,554 square kilometers. The geological formation is Miocene sedimentary rock. The central part is a northeast-southwest trending alluvial plain.

Fig. 8-1 shows the Ngrowo River System. The Ngrowo River cutts the central plain and joins the Brantas River running northeastwards. The Babakan, Klantur, Song and Gondang Rivers join the Ngrowo River from the slopes of Mt. Wilis. The Ngasinan River originating from Mt. Gepeng in the west runs roughly eastwards collecting many tributaries and joins the Ngrowo River. The main stem of the Ngrowo River is about 14 kilometers in length and further southwestern part becomes the Gesikan and the Bening Swamps. The Kandung River from the east flows into the Gesikan swamp. The Tawing River from the west has been flowing into the Bening swamp before construction of the Parit Raja canal.

8-2-2 Flood Control Structure

In 1939, a plan was contemplated to construct the following structures in the Ngrowo River Basin.

(1) Sumbergajam gate
(2) Widoro dam
(3) Ngasinan-Ngrowo canal
(4) Ngasinan floodway
(5) Tjelok gate

As can be seen in Fig. 8-2, the aims of this plan were (1) to retain the flood flow from the Ngasinan River by diverting it to the Gesikan swamp through the new Ngasinan floodway for reduction of flood peak discharge in the Brantas River, and (2) to divert the Tawing River flow to the Ngrowo River for irrigation in the downstream reaches. However, the diversion of the Ngasinan flood largely silted up the Gesikan swamp, and it could not be continued. The Tjelok gate has become unmoveable due to the sediment and since then the direct connection of the Gesikan swamp and the Ngrowo River has lost.

The lowland around the Gesikan and Bening swamps is intensively cultivated. Therefore, too high water surface in the swamps causes a crop damage and too low water surface disturbes irrigation. It was desirable to keep the water surface elevation below 82.2 meters to minimize the flood damage but above 81.5 meters at the end of rainy season for irrigation purpose.

The Japanese army constructed a diversion tunnel in Gamping hill to drain the swamps to the Indian Ocean, after a serious flood damage in November 1942. However, this tunnel has lost its function due to the lack of the maintenance.

The 1951 eruption of Mt. Kelut caused a large aggradation of the Brantas River. Accordingly, the discharge capacity of the Ngrowo River has decreased to such an extent that the lowland near Tulungagung city is often inundated.

A diversion plan to the Indian Ocean has emerged again in a different form. The plan involved the construction of the following structures.

(1) Parit Raja canal
(2) Kendal canal
(3) Diversion canal through Gamping hill
(4) Kendalardjo gate
(5) Tjampurdarat gate

The purposes of this plan were (1) to lessen the flood discharge by diverting the Ngasinan and Tawing Rivers to the Indian Ocean through the Parit Raja canal and diversion canal, and (2) to control the water level in the swamps by the Kendal canal, Kendalardjo gate and Tjampurdarat gate. In implementing this plan, the Nejama diversion tunnel was excavated instead of the diversion canal. This plan is illustrated in Fig. 8-3.

With the abovementioned structures, the flood control is carried out as follows: The flood discharge in the Ngasinan River is diverted to the Indian Ocean through the Parit Raja canal and Nejama tunnel by operating the Bendo gate as far as the Parit Raja canal is capable of the discharge. For larger discharge, part of the flood flow is diverted to the Gesikan swamp through the Bendo and Sumbergajam gates, and released to the Nejama tunnel through the Gesikan swamp, Tjampurdarat gate, Bening swamp and Kendalardjo gate on the Kendal canal.

In the dry season, the Bendo and Sumbergajam gates are operated to convey water to the irrigation areas along the middle stretch of the Ngasinan River, Ngasinan-Ngrowo canal, and Ngrowo River. The conveyed water may reach to the Brantas River. It will serve for irrigation and sediment transportation in the Brantas River in certain extents.

The abovementioned operation implies that the Ngrowo River system is divided into two in the rainy season. The rivers on the southeastern slope of Mt. Wilis and small tributaries on the right bank drain into the Brantas River through the Ngrowo River, but the remaining tributaries such as the Ngasinan, Tawing and Dawir Rivers and other tributaries drain into the Indian Ocean through the Nejama tunnel.

In this chapter, the catchment area draining to the Brantas River is called the Tulungagung sub-basin and that draining to the Indian Ocean is named the Nejama sub-basin as shown in Fig. 8-4. The catchment area of tributaries in each sub-basin is shown in Table 8-1.

8-3 Problem Area

The Ngrowo River is 14 kilometers in length and 1:2,500 in the average bed slope between its mouth and the Tjelok gate. Further upstream channel is closed off at the Tjelok gate. The six-kilometer-long Ngasikan-Ngrowo canal joins with the Ngrowo River at just downstream of the gate. The gate of the Sumbergajam dam is closed in the rainy season.

The Ngrowo River is silted up and develops the natural levee on its bank. Fig. 8-5 is a profile of the Ngrowo River and the Ngasinan-Ngrowo canal showing the river bed, top of the natural levee and the lowest ground on the river bank, based on a MPWP map/1. The river bed is higher than the riparian area in the upper stretch of the Ngrowo River. The natural levee is 2 - 4 meters high above the river bed.

/1 Brantas Multipurpose Project "Topographic map of inundated area of Kali Ngrowo at Tulungagung", 1:25,000 Directorate General of Water Resources Development.

The left bank of the Ngrowo River is characterized by the tributaries such as the Song River, Klantur River, Babakan River, etc. running down on the steep slopes of Mt. Wilis. The tributaries meander on the plain at the foot of the mountain. Here the tributaries are silted up due to the sediment transported from the upstream reaches. The lands along the tributaries are generally elevated because of the repeated inundation and meandering of the tributary. When a tributary inundates the land, water is retained in the depression between the natural levee of the Ngrowo River and the hill slope. The swamp thus created can not be drained for a long time because the rivers are elevated. This problem generally arises in the lowland between the mouth and the Gondang River, especially in the depressions which are located on both banks of the Song River. The lowest ground surface is at about 80 meters in elevation.

The right bank of the Ngrowo River is generally higher than the left bank in the lower river stretch. Tulungagung city is located at about ten river kilometers from the mouth. There is a remarkable depression upstream of the Tulungagung city. The lowest ground elevation is about 79 meters being about two meters below the Ngrowo river bed.

There are two water level gauges in the Ngrowo River, i.e., the Tulungagung gauge at about eight river kilometers from the mouth and the Pakundjen gauge at the river mouth. Fig. 8-6 shows the river water level in 1968 and 1969 recorded at these gauges.

It is seen in the 1969 dry season record that the water levels at the two stations are almost equal each other. This fact shows that the discharge from the Ngrowo River Basin is so little that the Ngrowo river water surface is determined by the water level in the Brantas River.

The water level at the Pakendjen gauge rises depending on the discharge in the Brantas River in the rainy season. It often rises to elevations close to the top of the natural levee. The water level at the Tulungagung gauge becomes 1 - 1.5 meters higher than that of the Pakendjen gauge, because the discharge from the tributaries flows in the Ngrowo River. The Tulungagung water level usually rises to elevations between

81 - 82 meters. This is also a little below the top of the natural levee. High water stage in the Ngrowo River works to reduce the discharge capacity in the tributaries coming in, by flattening the hydraulic gradient in the lower reaches of the tributaries. Thus the tributaries inundate the land.

8-4 Flood Water Balance

All the flood water does not enter into the Ngrowo River, because part of it inundates the lands before that. Thus the Ngrowo River itself scarcely inundates the lands.

The cross section of the Ngrowo River approximately has a width of 20 meters at the bottom and its depth between the top of the natural levee and the bottom is less than three meters. Assuming about one meter per second of the water velocity, it is estimated that the maximum discharge capacity is less than 70 cubic meters per second, or less than six million cubic meters per day. The discharge capacity may reduce when a large flood occurs in the Brantas River.

The probability distibution of the one-day rainfall in the Ngrowo River Basin was calculated based on the records at seven raingauges inside and six raingauges outside of the basin. The flood discharge from the Tulungagung sub-basin of 434 square kilometers was calculated for each probable rainfall assuming a runoff coefficient of 0.3. The water volume inundating the land can be calculated as a balance between the discharge from the sub-basin and the discharge in the Ngrowo River, if the time lag in discharge is disregarded. This calculation is shown in Table 8-2. The resulted water volume inundating the land can be extrapolated to an arbitrary probability, because the probable rainfall was calculated assuming the Gumbel's distribution. The mean of the water volume is about four million cubic meters, or 400 hectares with an average water depth of one meter.

Generally the residential areas are located on the relatively elevated lands and the low-lying areas are cultivated for paddy. Therefore, the inundated areas are mostly the paddy field. Once a low-lying land is inundated, it can not be drained for a long time. Assuming a complete loss of the paddy crop in the inundated area, the potential loss of the average production is calculated to be about 70 thousand dollars as below (for unit yield and unit value of crop, see Chapter 6);

400 ha x 3.8 t/ha x 47.5 \$/t = 72,000 \$

8-5 Possible Solution

A lowering of the river bed in the Ngrowo River and improvement of the tributary river channel may reduce the flood problem in a certain extent. A comparison of the discharge from the Tulungagung sub-basin and the discharge capacity of the Ngrowo River shows that the discharge capacity of the Ngrowo River has to be doubled, if only this method is relied on.

As has been seen in Table 8-1, the Song River is the largest among the tributaries flowing into the Ngrowo River. Its catchment area is about 31 percent of the Tulungagung sub-basin. The Gondang, Blendis, Sengan and Gador Rivers entering into the Ngasinan-Ngrowo canal have the total catchment area about 28 percent of the sub-basin area. The inundated area will be largely reduced, if the Song River and Ngasinan-Ngrowo canal are diverted to the Gesikan and Bening swamps. These swamps have about 40 square kilometers of the surface area. Therefore, the flood peak would be substancially reduced and could be safely released to the Indian Ocean through the Nejama tunnel together with the discharge from the Nejama sub-basin. The works required for this purpose are as follows;

1. Construction of levee across the Ngasinan-Ngrowo channel near the Tjelok gate to separate the Ngasinan-Ngrowo canal from the Ngrowo River (about 2.5 m high and 25 m long).

2. Improvement of the river channel between the Tjelok gate and the Gesikan swamp (about 4 km long).

3. Construction of a canal connecting the Song River to the Ngasinan-Ngrowo canal (about 4 km long).

4. Improvement of the Kendal canal and the downstream end of the Parit Raja canal (about 4 km long).

The total construction cost will be one million - 1.6 million dollars.

The weakness of this plan will be silting in the Gesikan Swamp and deterioration of the upper stretch of the Ngrowo River.

Another possible plan is a diversion of the Song, Klantur and Babakan Rivers to the Brantas River by construction of a cut-off canal of which length is about ten kilometers. The construction cost will be 0.8 million - 1.2 million dollars. This plan seems to be the best among the possible ones.

8-6 Conclusions

(1) The Ngrowo River Basin has a catchment area of 1,554 square kilometers. The Ngrowo River System in the rainy season has been separated into two sub-basins by the Sumbergajam and Tjelok gates. The southern part of the basin is drained into the Indian Ocean through the Nejama Tunnel. The northern 434 square kilometers is drained into the Brantas River through the Ngrowo River.

(2) The Ngrowo River is a silted-up river developing the natural levee on the bank. The tributaries on the slope of Mt. Wilis join with the Ngrowo River. They are also silted up. When a flood occurs, the rised water level in the Ngrowo River reduces the discharge capacity in the lower stretch of the tributaries. Then, the tributaries inundate the low-lying areas along the Ngrowo River. The water flooding on the land can not be drained for a long time, because the low-lying lands are below the water surface in the river.

(3) It is estimated that the flood discharge from all the tributaries joining with the Ngrowo River is about 11 million cubic meters per

day with a probability of 20 percent and 20 million cubic meters per day with a probability of one percent. On the other hand, the discharge capacity in the Ngrowo River is less than six million cubic meters. The balance remains in the inundated area.

(4) The inundated areas along the Ngrowo River are mostly cultivated for paddy agriculture. The potential loss of the average annual crop is estimated to be in the order of 70,000 dollars.

(5) A plan increasing the discharge capacity of the Ngrowo River and its tributaries will not be workable because the present capacity of the Ngrowo River is too small.

(6) One of the possible solution is a diversion of the Song River and the Ngasinan-Ngrowo canal to the Indian Ocean through the Gesikan and Bening Swamps by constructing diversion canals. The construction cost will be one million - 1.6 million dollars. The problems of silting in the swamps and deterioration of the upper stretch of the Ngrowo River will emerge.

(7) The best solution will be the construction of a cut-off canal which directly divert the Song, Klantur and Babakan Rivers to the Brantas River. The construction cost will be 0.8 million - 1.2 million dollars.

(8) The study in this chapter was made on the basis of the 1:50,000 map. It could reach to the basic solution of the problem. More detailed survey and planning are necessary for economic evaluation and design.

Table 8-1 Catchment Area

(Unit: km^2)

Tulungagung sub-basin	
Babakan River	72
Klantur River	63
Song River	133
Gondang River	63
Blendis & Sengan Rivers	31
Gador River	26
Right Bank of Ngrowo River	46
Sub-total	434
Nejama sub-basin	
Ngasinan River	423
Tawing River	181
Karangtuwo River	70
Keboireng River	37
Dawir River	231
Others & Swamps	178
Sub-total	1,120
Total	1,554

Table 8-2 Flood Water Balance

Probability		1	2	5	10	20
1. One-day Rainfall	(mm)	153	138	117	102	84
2. Sub-basin Rainfall: (1) x 434 km^2	($10^6 m^3$)	66	60	51	44	36
3. Sub-basin Discharge: (2) x 0.3	($10^6 m^3$)	20	18	15	13	11
4. Ngrowo River Discharge	($10^6 m^3$)	6	6	6	6	6
5. Water inundating land: (3) - (4)	($10^6 m^3$)	14	12	9	7	5

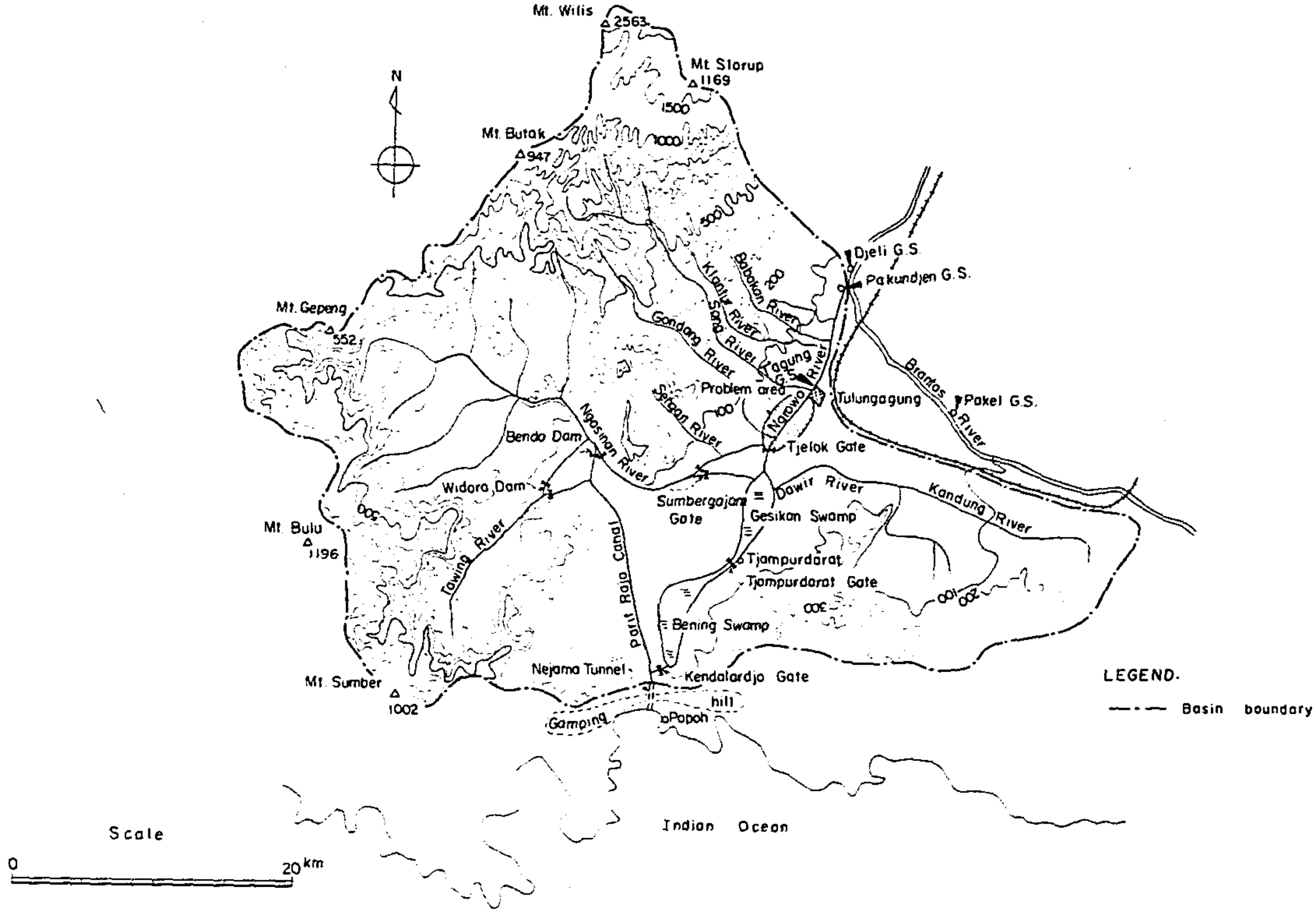

Fig. 8-1 Ngrowo River System

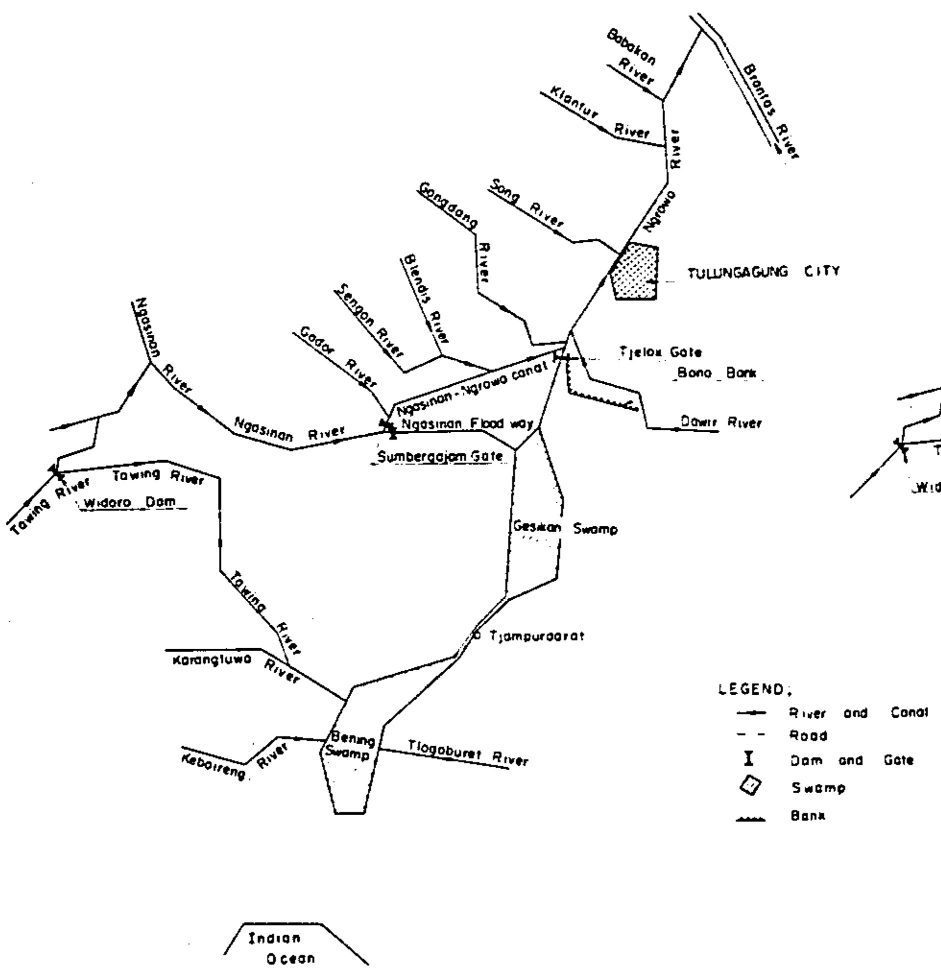

Fig. 8-2 1939 Flood Control Project

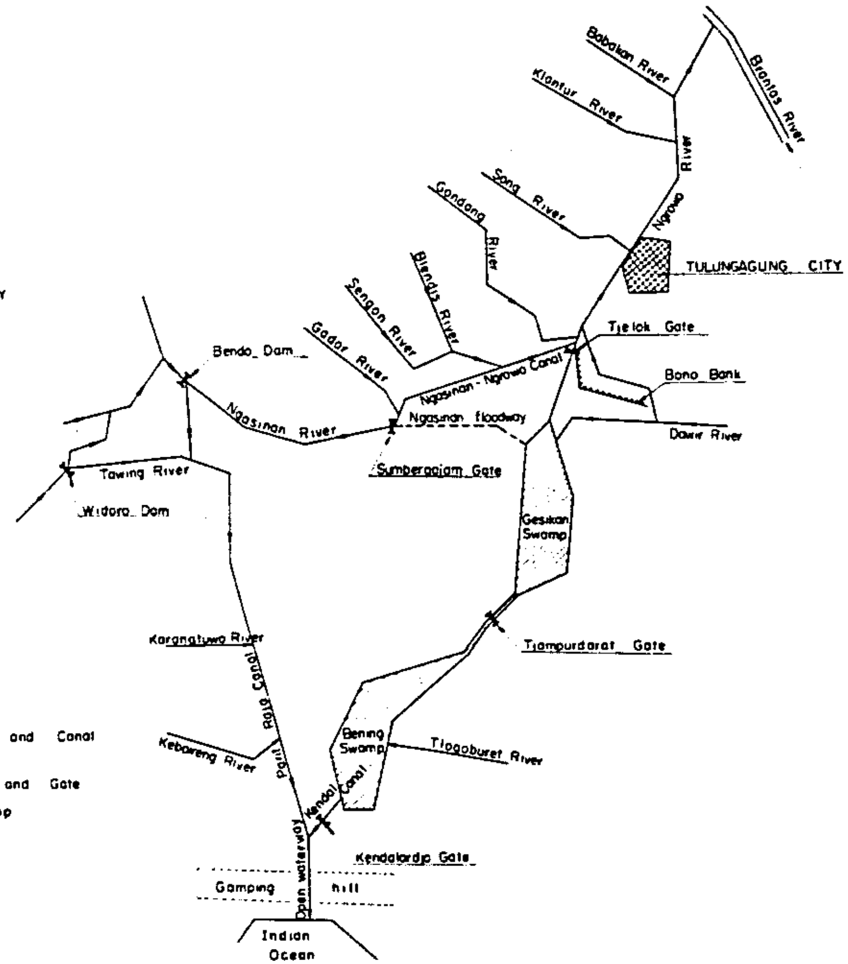

Fig. 8-3 1951 Flood Control Project

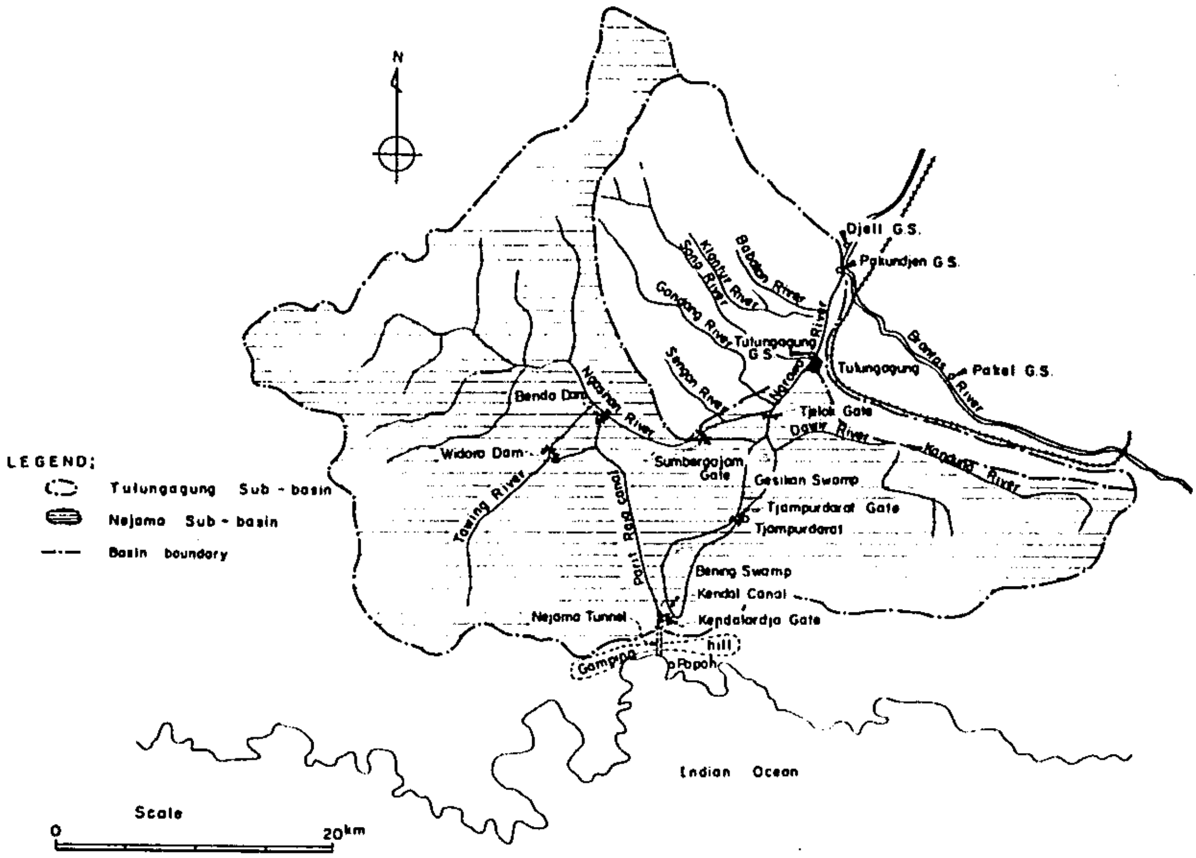

Fig. 8-4 Sub - Basins

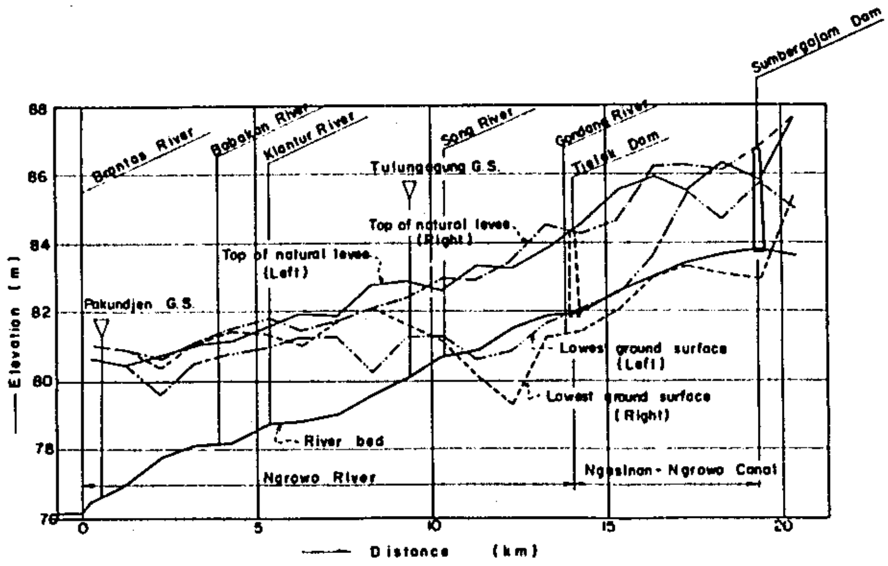

Fig. 8-5 Profile of the Ngrowo River and Ngasinan-Ngrowo Canal

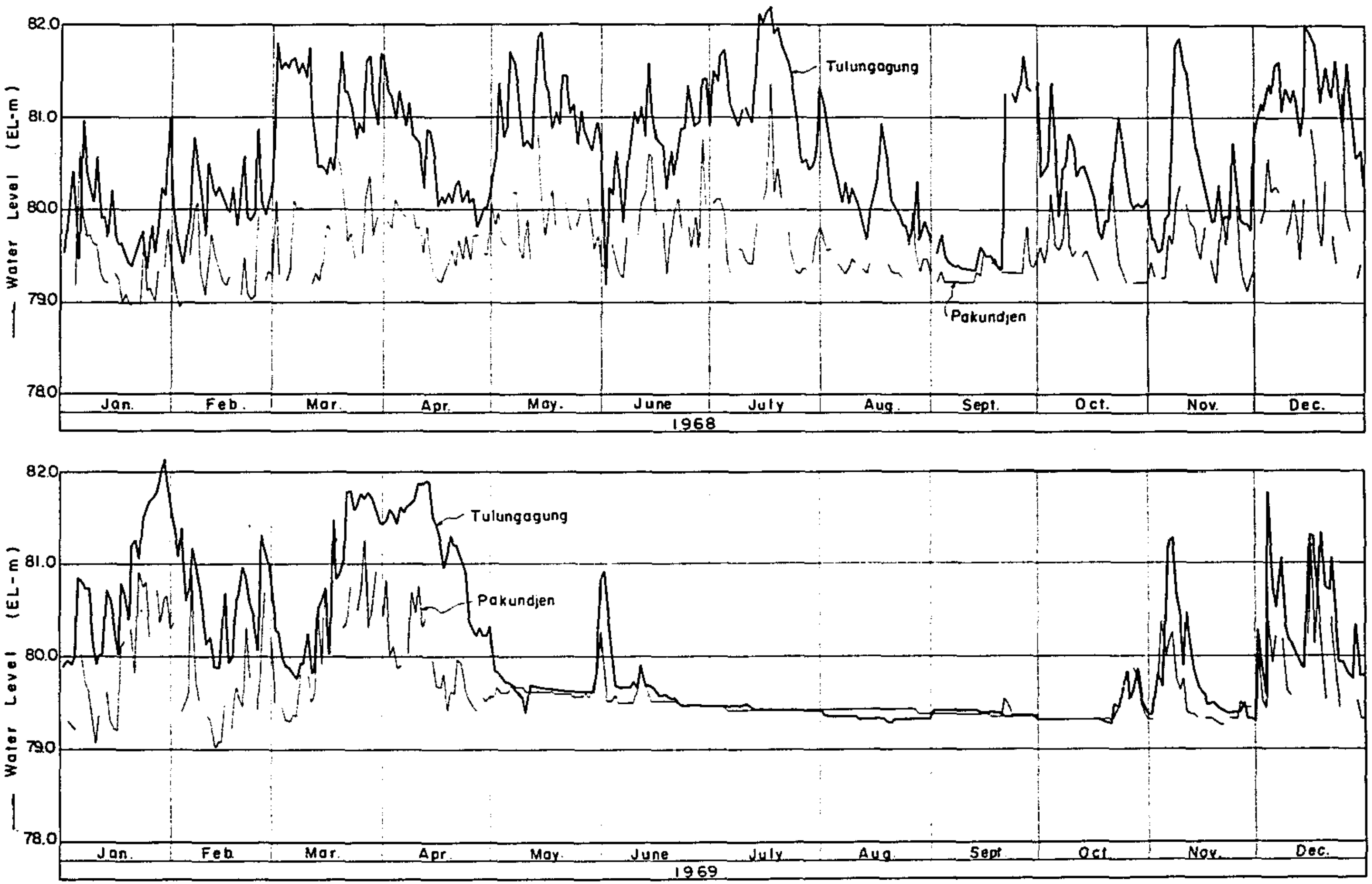

Fig. 8-6 Water Level Record