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MINISTRY OF PUBLIC WORKS AND ELECTRIC POWER
DIRECTORATE GENERAL OF WATER RESOURCES
DEVELOPMENT

FEASIBILITY REPORT ON THE WONOGIRI IRRIGATION AND UPPER SALA RIVER IMPROVEMENT PROJECT

APPENDIX II RIVER IMPROVEMENT

JAPAN INTERNATIONAL COOPERATION AGENCY

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1. OUTLINE OF THE BASIN

In this chapter, descriptions are made on the Jurug-Wonogiri basin of the Sala river, the topography, and conditions of rivers in its surrounding areas.

1.1 THE SUBJECT-ZONE AND OBJECT OF THE INVESTIGATIONS

1.1.1 The Subject-Zone

The Sala river basin is located along the eastern and central districts of Java covering an area of 16,100 km^2. The overall area of the Sala river is shown in Fig. 1.1.1. The Bengawan Sala is largely divided into 3 areas of which the Upper-Sala basin is subdivided into 3, namely, the Ngawi-Jurug, Jurug-Wonogiri and Upper Wonogiri basins.

The subject-area of the present investigations is the Jurug-Wonogiri section (1,870 km^2) of the Upper-Sala basin. By the way, Ngawi is the name of a town situated at the confluence of the Madiun river and the Sala river. Jurug is the name of a road bridge, right below Surakarta (Sala) city with a population of 400,000 situated on the left bank of the Sala river. Wonogiri is also the name of another town, some 30 km south of Surakarta city.

1.1.2 Object of the Investigations

The Sala river investigation was initiated by the O.T.C.A. survey team of Japan in 1972 and the results were condensed into the Master Plan Report in 1974. The said Report suggested that in order to carry out the integrated development of the Sala river, the top priority was to be given to the work of constructing a dam at the Wonogiri Site for the supply of irrigation water and electricity, etc. to the lower-stream districts, along with the improvements of the Sala river between Jurug and Nguter for prevention of flood disasters.

Thereafter in 1975, the pre-studies to assess the feasibility of the abovementioned works were conducted by a J.I.C.A. survey team. This team was not able to obtain satisfactory results because the topographical maps made available were not correct.

For the renewed feasibility study, and to carry out more accurate investigations, topographical maps of a scale of 1/5,000 published lately were used.

The present supporting report consists of the results of pre-feasibility investigations carried out for the sectors of hydrology and river improvement. The outline is as follows.

The hydrological investigation undertaken this time was based on newly obtained data in addition to the master plan and the results of the pre-feasibility study. Thus, sufficient information for the river improvement planning was made available.

The river improvement investigation was carried out to facilitate the planning, based on the findings of the said hydrological investigation, of the improvement works of the Sala river and its tributaries between Ngawi and Jurug from the view-points of water control and economic development of the area. The data obtained by this investigation have been used to calculate the volume and the cost of the proposed works, and the final execution plan will be formulated on the basis of the results of such calculation.

1.2 TOPOGRAPHY AND LAND-USE OF THE BASIN

1.2.1 Topography

The Jurug-Wonogiri basin (1,870 km^2) is shown in Fig. 1.2.1 The Jurug-Wonogiri basin is bordered by Mt. Lawu (3,265 m) on the east, by Mt. Merapi (2,911 m) and Mt. Merbabu on the west and by their lower ranges and hills on south and north. The basin is almost a rectangle, about 75 km longitudinally and 25 km latitudinally.

The Sala river collects the storm water of the Upper Wonogiri basin (1,350 km^2) and, after traversing between the low mountain ranges and hills around Wonogiri, flows down into the Jurug-Wonogiri basin where it turns its course northward to pass through the hilly areas near Surakarta city; thence, it meanders on around the skirt of Mt. Lawu and flows down through the Jurug-Ngawi plain before it empties itself into the Java Sea.

Although the Jurug-Wonogiri basin is bordered by mountain ranges and hills on all side as stated above, its central part is made up of a flat land, with the gradient pitch of about 1/2,000 latitudinally and about 1/1,500 longitudinally.

This flat area is the flood plane of the Sala river and of K. Dengkeng, the largest of the tributaries in the Jurug-Wonogiri basins. At the time of the large flood of 1966, a vast area of some 200 km^2 was inundated.

1.2.2 Land use

The flat land in the basin is mostly used for farming; about 80% of the cultivated land is occupied by paddy and 20% by Polowijo. The planted area spreads up to an elevation of about 800 m above the sea level, and the forestry land within the basin is extremely limited.

1.3 DIVISION OF THE BASIN

1.3.1 Tributary Basin

Division of the Jurug-Wonogiri basin by its tributaries is shown in Fig. 1.2.1. with the river system modelling in Fig.1.3.1. The outlines of the tributary basins are indicated in Table 1.3.1.

There are nine major tributaries with a total area of 1,563.1 km^2 which corresponds to about 85% of the entire Jurug-Wonogiri basin (1,870 km^2), as follows:

No.	Name	Area	Confluence
1	K. Walikan	56.6 km^2	right bank
2	K. Jlantah	75.0 km^2	right bank
3	K. Dengkeng	833.0 km^2	left bank
4	K. Pusur	42.5 "	"
5	K. Buntungan	20.0 "	"
6	K. Brambang	125.4 "	"
7	K. Kembangan	38.4 "	"
8	K. Samin	304.7 "	right bank
9	K. Wingko	67.5 "	left "
	Total area	1,563.1 km^2	

1.3.2 Features of the Basins

The feature of the tributary is described by the index A/L^2, as shown in Fig. 1.3.2.

A stands for an area of the tributary and L, the length of the main tributary. Thus the A/L^2 represents the strip of the basin and the mean A/L^2 - ratio is about 10. The tributary basin can accordingly be classified as the so-called braided basin.

Fig. 1.1.1 GENERAL MAP of RIVER BASIN

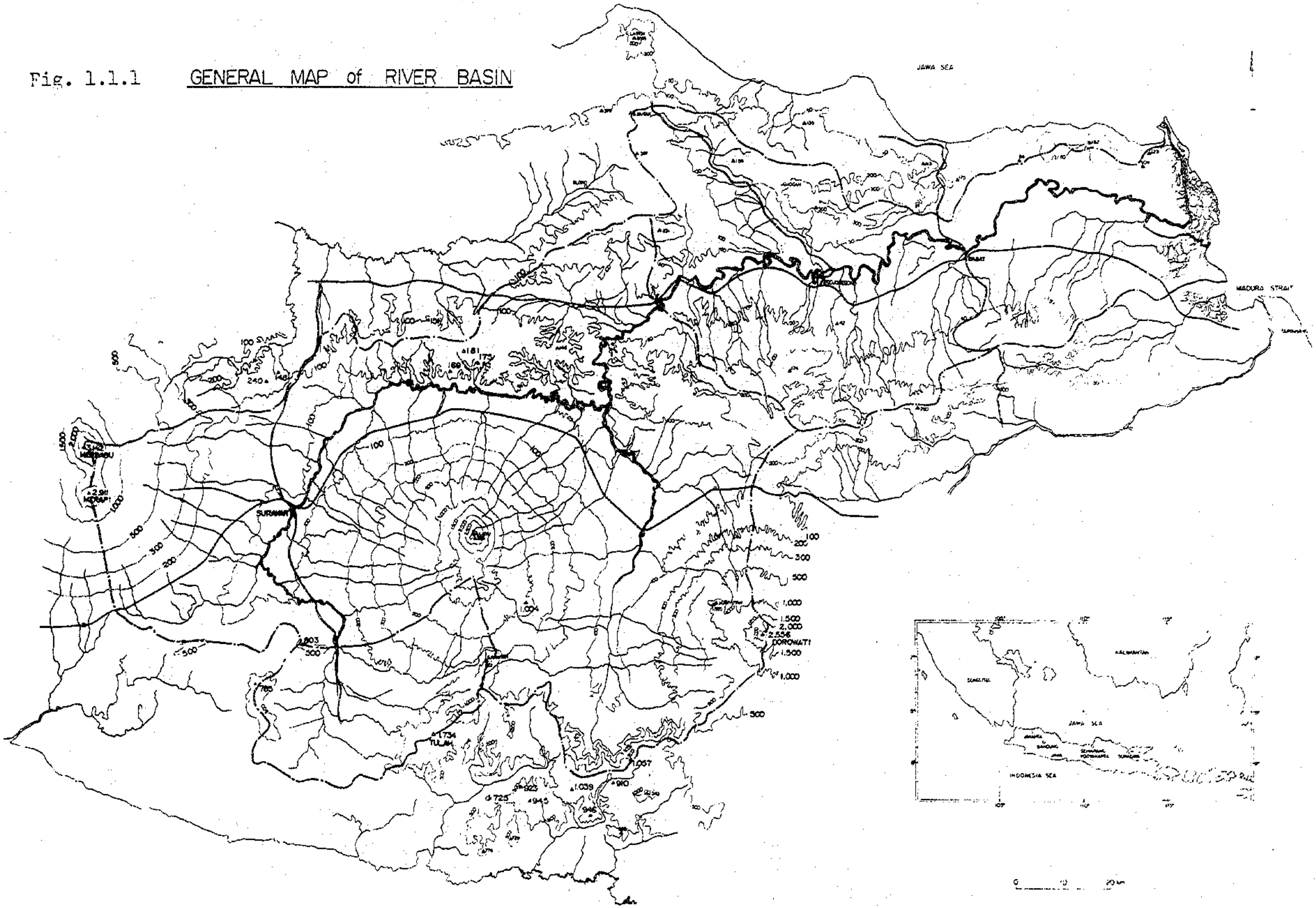


Fig. 1.2.1 UPPER SALA BASIN

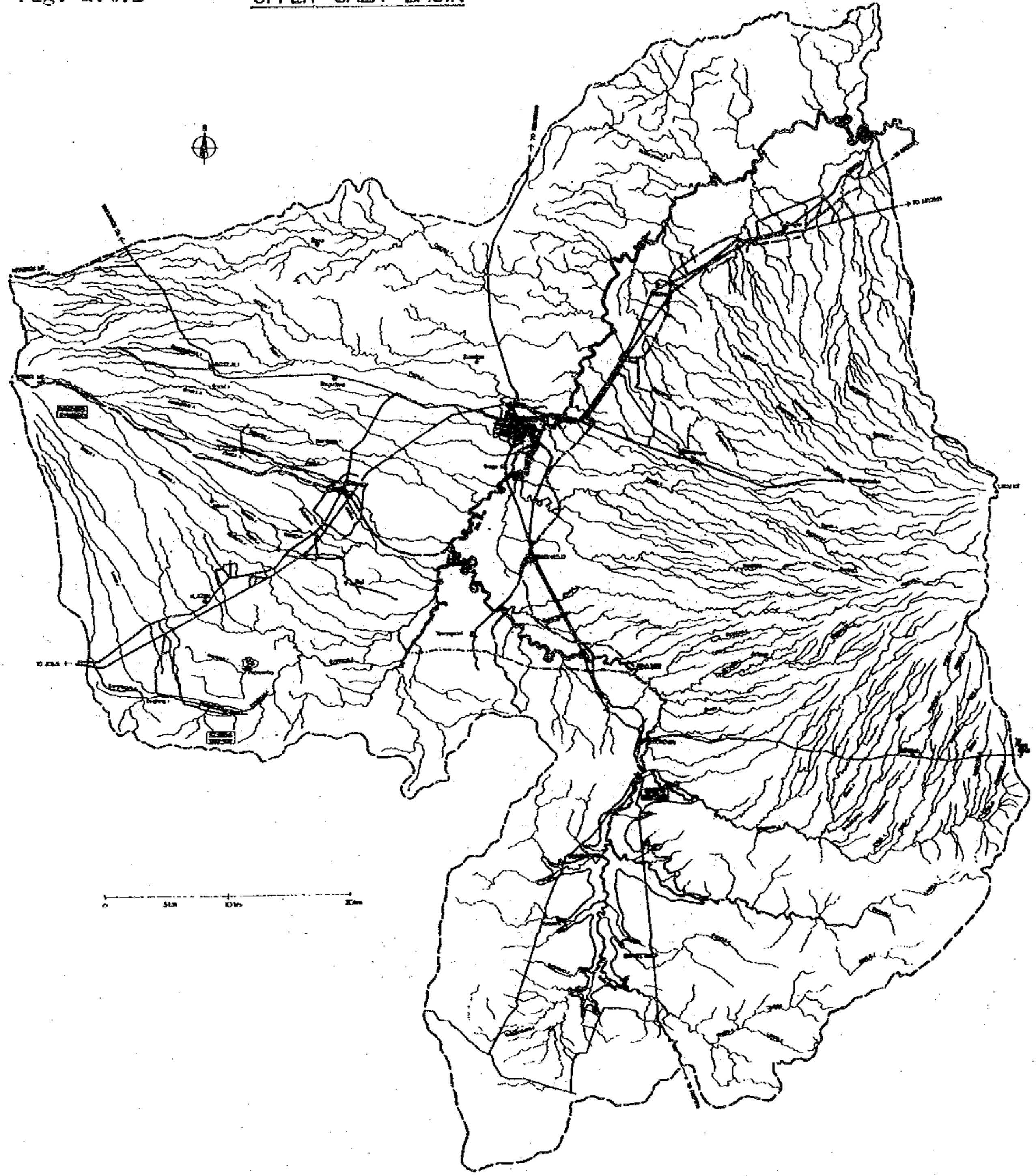
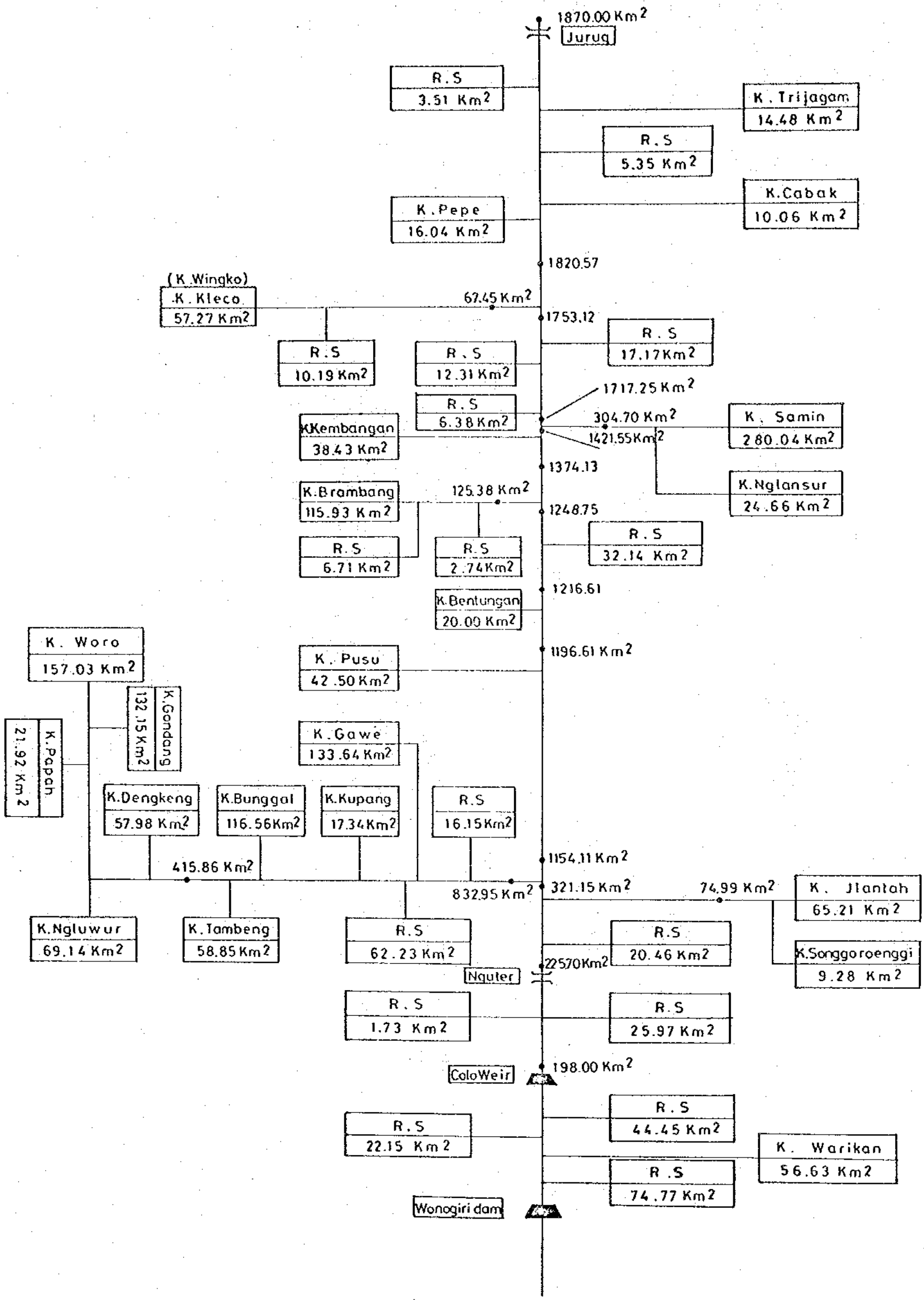


Fig. 1.3.1 River system in Wonogiri - Jurug Area



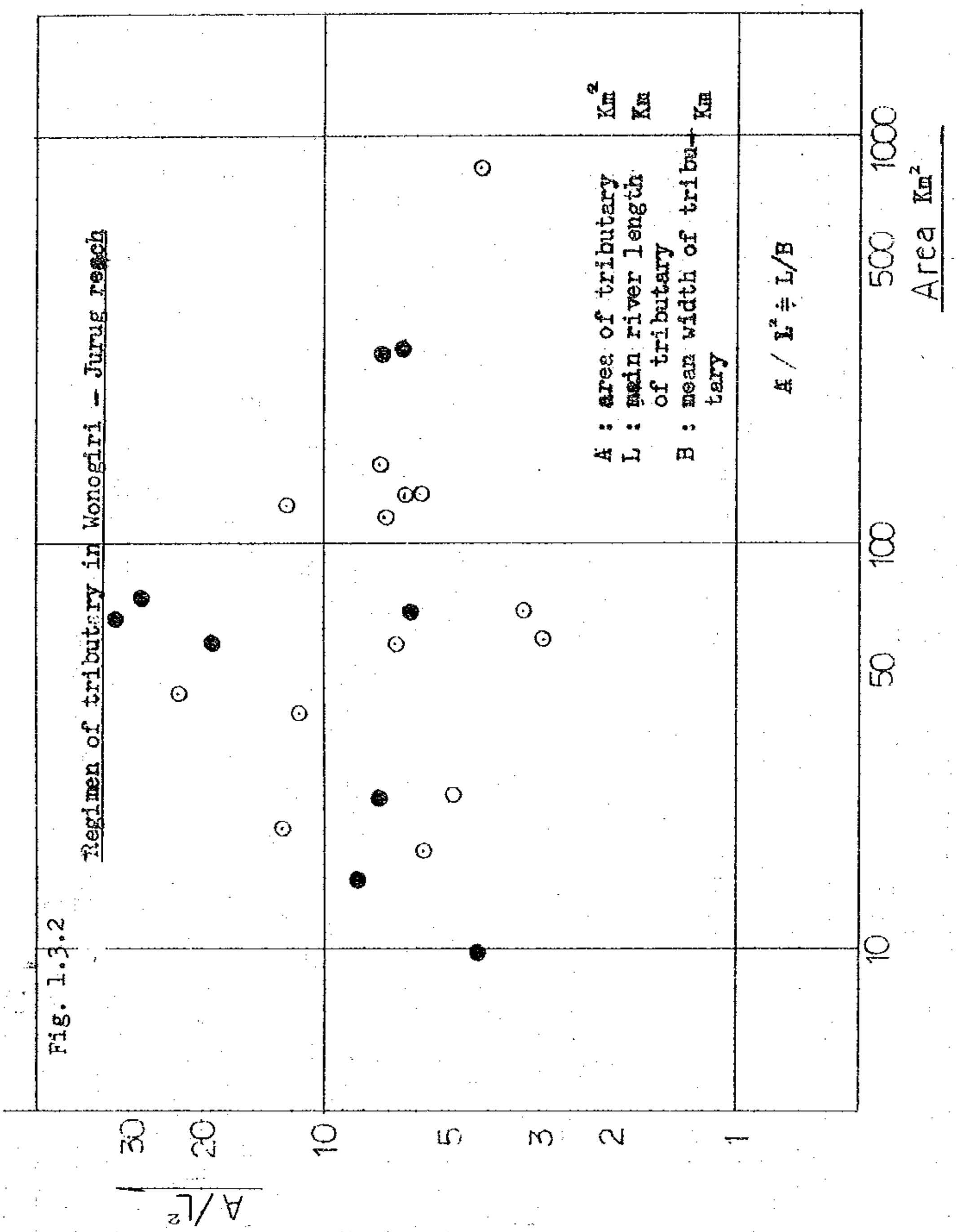


Fig. 1.3.2 Regimen of tributary in Wonogiri – Jurug reach

Table 1.3.1 Characteristics of Main Tributary

No.	Name of river: First-order tributary	Name of river: Second-order tributary	Area A km^2	Location (L or R)	Length of river L km	B = A/L B km	A/L^2	Note
1.	K. Walikan		56.63	R	33.0	1.72	19.1	
2.	K. Jlantah		74.99	R	46.0	1.63	28.2	Location
		K.Jlantah	65.21	R	46.0	1.42	32.3	R: Right side of B. Sala
		K.Sanggaroemgi	9.78	R	6.5	1.50	4.3	L: Left side of B. Sala
3.	K. Dengkeng		832.95	L	59.0	14.11	4.2	
		K. Gawe	133.64	"	28.0	4.77	5.9	
		K. Kupang	17.34	"	10.0	1.73	5.8	
		K. Bunggal	116.52	"	29.0	4.02	7.2	
		K. Tan bang	58.85	"	13.0	4.53	2.9	
		K. Ngluwur	69.14	"	15.0	4.61	3.3	
		K. Denkeng	57.98	"	20.0	2.90	6.9	
		K. Gondang	132.15	"	29.0	4.56	6.4	
		K Woro	157.03	"	34.0	4.62	7.4	
4.	K. Pusur		42.50	L	31.5	1.35	23.3	
5.	K. Bumtungam		20.00	L	16.0	1.25	12.8	
6.	K. Brambang		125.38	L	39.0	3.21	12.2	
7.	K. Kembamgan		38.43	L	21.0	1.83	11.5	
8.	K. Samin		304.70	R	45.0	6.77	6.5	
		K. Samin	280.04	"	45.0	6.23	7.2	
		K. Nglamsur	24.66	"	14.5	1.70	8.5	
9.	K. Wingko		67.45	L	20.5	3.31	6.2	
10.	K. Cabak		10.06	R	11.5	0.87	13.3	
11.	K. Trijagan		14.48	R	11.0	1.32	8.3	

2. HYDROLOGY

In the sector of hydrology, investigations were made on the meteorological factors, rainfall run-off, sand discharge and flood. The flood probability and the discharge distribution have been identified through the examination of the data collected at the hydrological investigations.

2.1 METEOROLOGICAL DATA

The meteorological investigations covered wind, sunshine, humidity, temperature, rainfall and evaporation. Relevant data have been made available by the meteorological observatories in the Jurug-Wonogiri basin, as well as Panasan airport and P.B.S. office (Pabelan). With regard to the weather conditions and others, data obtained from the observatories along the Madiun river and in the Lower Sala basin have been used for estimation.

2.1.1 Wind

Generally speaking, the Sala river area; two seasons prevail in the one is dry and the other is rainy. The dry season lasts from May to October and the rainy season starts in November and ends in April of the following year. The wind directions in Southeast Asia are as shown in Fig. 2.1.1. During the rainy season, the general direction of the wind is from the mid-Pacific toward the equator, and vice-versa in the dry season, in the Bengawan Sala river area. For this reason, in the vicinity of the Java Island, the westerly wind is predominant during the rainy season and it turns easterly during the dry season; and yet, subject to the landform the blowing directions change locally.

Fig. 2.1.1 shows a comparison of the wind distribution between Panasan and Iswahydi (Madiun) basins. In the Upper Sala basin the easterly wind is predominant during the rainy season and the southernly wind in the dry season, whereas in the Madiun basin the southerly wind is predominant throughout the year. The average wind velocity in the Upper Sala basin is 1.5 m/s during the rainy season and 2.3 m/s in the dry season; although the latter shows larger velocity than the former, it seldom develops into a gale.

2.1.2 Sunshine

As to sunshine, there are the records for about 10 years at Panasan and only for 1 year at Pabelan. The sunshine observatory records are given in Fig. 2.1.2.

According to Fig. 2-2, the values recorded at Pabelan are quite similar to those in other basins but those at Panasan are abnormally lower than those in other areas. But considering from the fact that the meteorological factors, temperature, humidity and rainfall depth related to sunshine in the Upper Sala basin are quite identical with those of other areas, the Panasan values do not seem to represent those of the Upper Sala basin.

Accordingly, the 1-year record of Pabelan was adopted for the present investigations as the sunshine ratio for the Upper Sala basin. The sunshine ratio at Pabelan is 78% in the rainy season and 88% in the dry season.

2.1.3 Temperature

The temperature was measured both at Pabelan and Panasan, and the results are as shown in Fig. 2.1.3. The temperature remains almost even all the year round, marking 31.4°C for the highest and 23.9°C for the lowest, with the normal seasonal variations of less than 2°C. The local variations are extremely small.

2.1.4 Humidity

Fig. 2.1.4 carries the observatory results of humidity at Panasan. As is evident from the figure, along with seasonal variations there are local variations also. Nevertheless, the relative humidity reaches the highest in February and March during the rainy season and the lowest in September or October during the dry season.

In the Upper Sala basin, the mean records of humidity in the rainy and dry seasons are 74.2% and 61.6%, respectively. As compared with other basins, the humidity in the Upper Sala basin is higher than in Madiun by about 2% but lower than in the Lower Sala basin by 10%.

2.1.5 Rainfall

A detailed explanation on the rainfall given later in Chapter 2-2 in which are taker up the features of the annual precipitation in the Sala river basin as compared with other areas.

Generally speaking, not only in the Sala river basin, the rainfall in the tropics assumes the form of the so-called shower which mostly takes place from noon through the evening, and its rainfall . intensity is remarkably high; the duration of continuous falling is extremely short, mostly less than 6 hours. Furthermore, as a feature of the tropical shower the rainfall area is very much limited; the rainfall expanse in Indonesia is said to be several to several tens of km^2, although the scale of the hyetal region in the Sala river basin is till unknown. The number of the observatories in the Sala river basin where the referential data being orderly arranged, and their distribution density are as follows:

The number of observatories in the entire basin whose data have been utilized for the analysis counts 40, and the area under one observatory is approximately 400 km^2 which is much larger than the size of the hyetal region. As to the areal rainfall, there is a good possibility that the actual rainfall differs considerably from the values observed.

Given in Table 2.1.1 is the mean monthly rainfall in the Upper Sala basin for a period from 1952 through 1973 which is based on the ordinary rainfall data from the 40 observatories. In Fig. 2.2.6 are shown the secular changes of the yearly total of rainfall; the rainfalls during the rainy and dry seasons are quoted in Table 2.1.2.

According to the result of calculations, the mean annual total of rainfalls during the past 22 years is about 2,100 mm/year throughout all basins as a whole. From the secular changes of mean annual total of rainfalls, it has become clear that the 6 years centering around 1955 and the latest 3 years (1972 - 74) were the wet years, and the drought years continued from 1960 to 1967. The wet years registered the rainfall of 3,000 mm/year whereas the drought years only 1,600 mm/year showing a big variation in the rainfall.

2.1.6 Evaporation and evapotranspiration

The evaporation in the Sala river basin was observed at 8 points, but only at a single place in Pabelan in the Upper Sala area. Moreover, the observation at Pabelan was started only in 1972 and besides, the data after 1974 are of poor reliability.

The mean monthly evaporation recorded at of the 8 observatories inclusive of Pabelan is indicated in Fig. 2.1.5. According to this figure, the annual evaporation at Pabelan is 876 mm/year which is equivalent to only 50% (1,752 mm/year) of the Madiun basin and 80% (1,095 mm/year) of the Lower Sala basin.

As to the evapotranspiration, no observation has been conducted within the Sala river basin. Normally, evapotranspiration is less than the volume measure with an evaporimeter. As trees and plants in the basin are apparently maintaining almost the same condition throughout the year, it may be taken for granted that the evapotranspiration corresponds to the mean monthly evaporation measured at a fixed value. In the master plan report, 0.8 was adopted for the fixed value.

2.2 RAINFALL

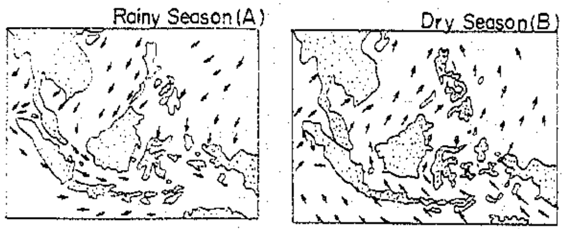
The explanation given under 2.1.5 was only about the annual volume of rainfall. Thereafter more detailed limited to the explanations are given on the rainfall characteristic, limiting the areas to the Upper Sala basin and the Jurug-Wonogiri basin.

2.2.1 Rain-gauge station

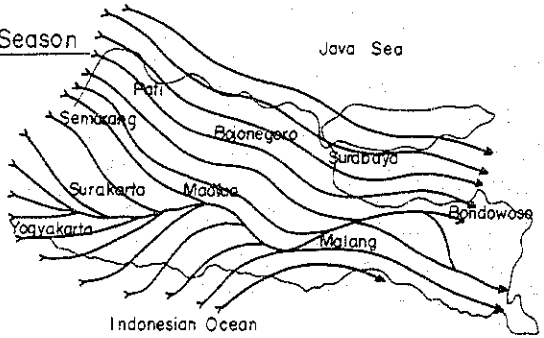
There are 208 ordinary rain-gauge stations and 10 automatic rain-gauge stations in the Upper Sala basin.

The number of the stations where the data have so far been put in order is 17 for the ordinary rain-gauge station and 10 for the automatic rain-gauge station, totalling to 27. As to the date in order, the oldest available are those for 1952, and accordingly, the data analysed covered only a comparatively short period.

WIND DIRECTION IN CENTRAL AND EAST JAVA



Rainy Season



Dry Season

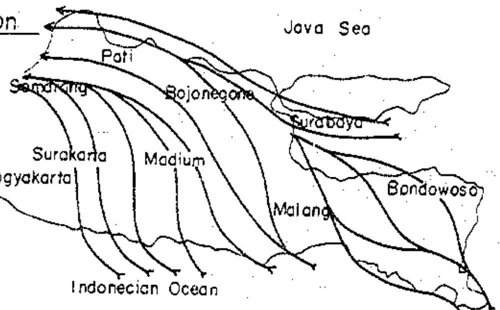
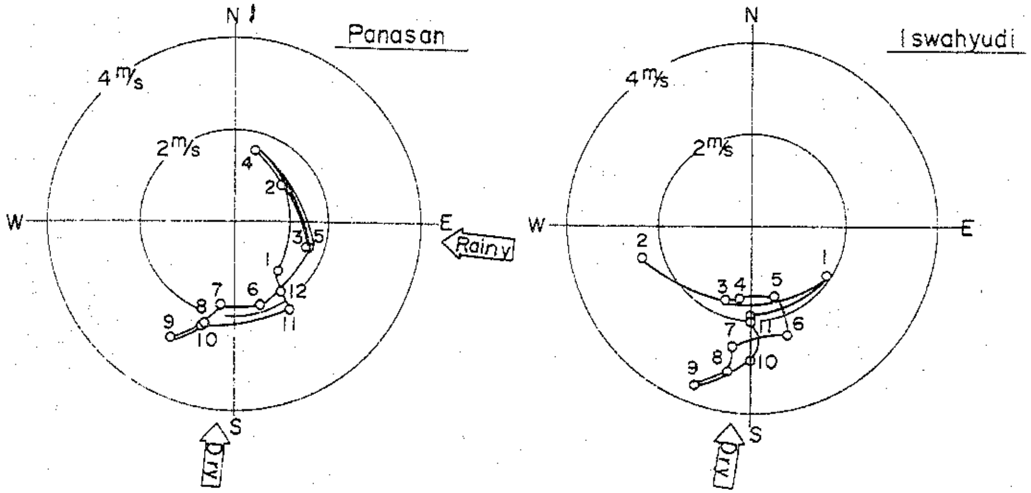


Fig.2.1.1 Wind Data

Wind Vector

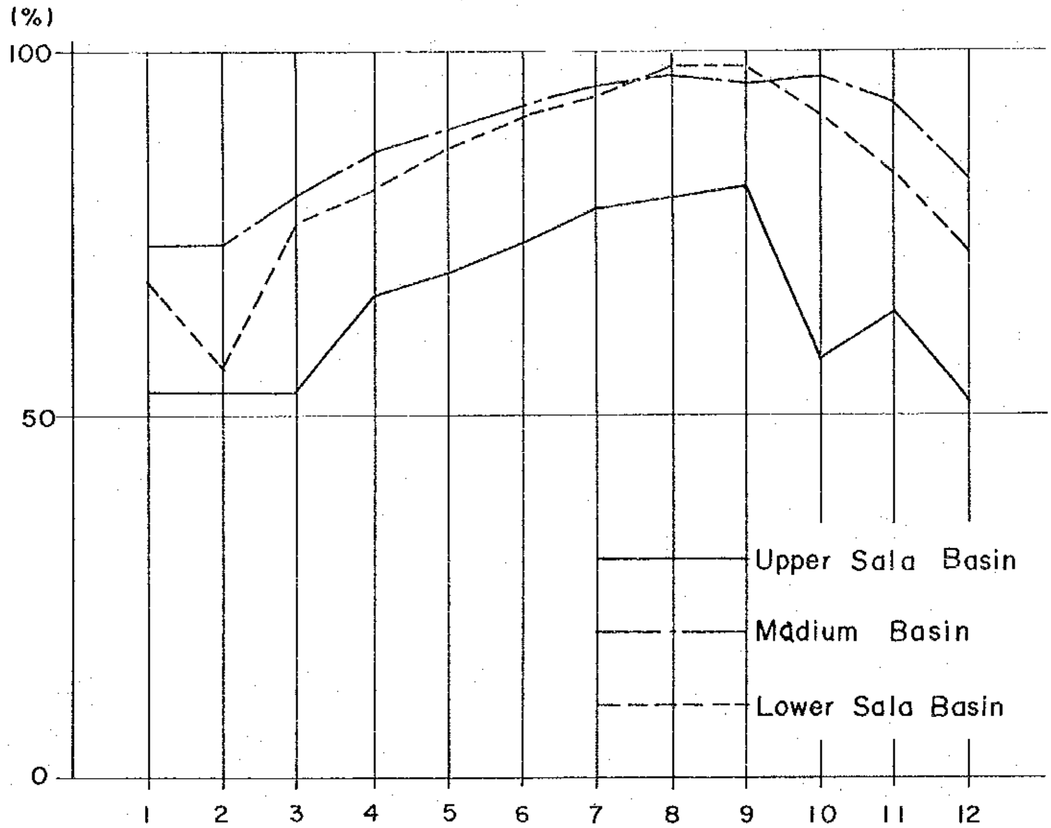


Average Monthly Wind-Velocity

Unit : m/sec

Location	Station	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sep.	Oct.	Nov.	Dec.	Mean		
Upper Sala	Panasan	1.4	1.3	1.6	1.4	1.6	1.9	1.8	2.3	2.9	2.4	2.2	1.8	2.9	1.5	2.3
Basin	Pabelan	1.8	1.0	1.5	1.3	1.3	1.5	2.2	1.9	2.1	2.0	2.6	1.8	1.7	1.5	2.1
Madium	Iswahyudi	1.9	2.4	1.7	1.6	1.7	2.4	2.6	3.1	3.6	2.9	2.0	1.9	2.3	1.9	2.8
Lower Sala	Kening	2.2	2.2	1.6	1.6	1.6	1.6	1.9	2.5	2.8	2.5	1.8	1.8	2.0	1.6	2.4
Basin	Perak	2.2	2.2	1.7	0.8	0.8	1.1	1.4	1.7	1.9	1.1	1.1	1.1	1.7	1.5	1.4

Fig. 2.1.2 Average Monthly Sun-Shine Ratio

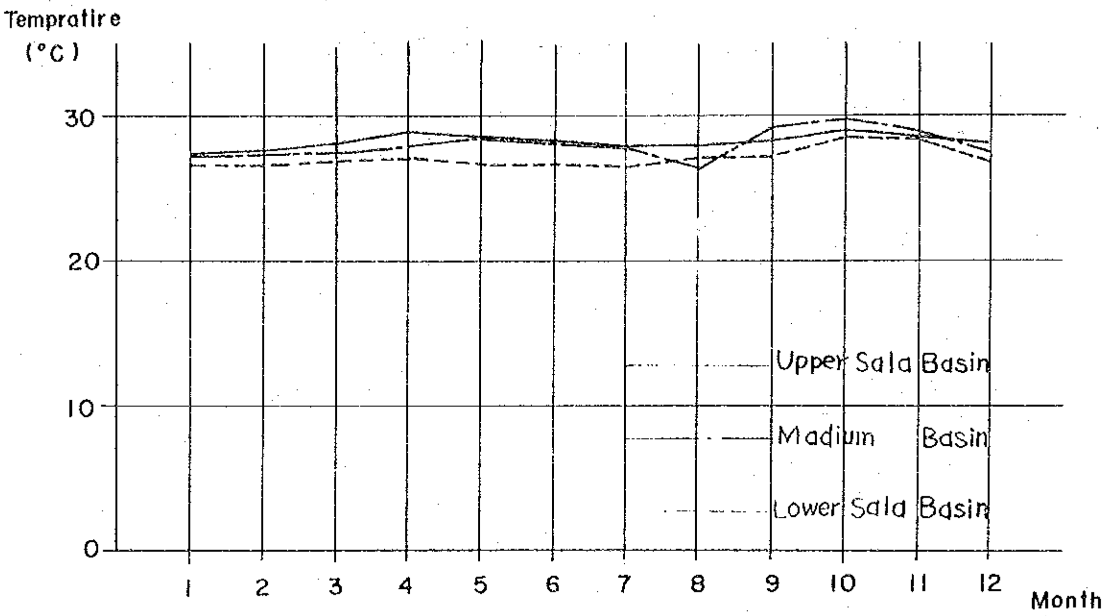


Average Monthly Sun-Shine Ratio

Unit : %

Location	Station	Jan.	Feb.	Mar.	Apr.	May.	June.	July.	Aug.	Sep.	Oct.	Nov.	Dec.	Mean
Upper Sala Basin	Panasan	30.0	31.1	36.6	45.6	51.1	63.3	70.0	72.5	75.0	32.5	48.8	33.8	49.7
	Pabelar	77.0	75.0	76.0	87.0	88.0	84.0	87.0	87.0	88.0	82.0	80.0	70.0	82.0
Madium Basin	Iswahyudi	73.7	73.7	80.1	86.2	89.4	92.4	95.4	96.9	95.3	96.6	93.3	82.5	88.0
Lower Sala Basin	Kening	59.2	38.9	73.4	81.6	82.0	86.6	89.5	96.6	95.4	90.8	88.9	67.9	81.4
	Perak	77.1	74.0	78.7	80.4	90.6	95.4	98.1	98.6	98.2	92.1	78.1	76.6	86.5

Fig 2.1.3 Average Monthly Temperature (At 9 O'clock)



Average Monthly Temproture (At 9 O'clock)

Unit : °C

Location	Station	Jan.	Feb.	Mar.	Apr.	May.	June.	July.	Aug.	Sep.	Oct.	Nov.	Dec.	Mean
UpperSala	Panasan	27.3	27.4	27.8	28.3	28.5	28.0	27.5	28.0	29.1	29.6	28.8	28.1	28.2
Basin	Pabelar	27.3	27.9	28.2	29.4	28.6	28.5	28.1	27.8	27.4	28.5	28.6	28.1	28.2
MadiumBas	Iswahyudi	27.2	27.2	27.4	28.0	28.4	28.2	27.8	26.2	29.3	29.8	28.9	27.7	28.2
LowerSala	Kening	26.4	26.6	26.6	27.3	26.7	26.5	26.5	27.4	26.9	28.6	27.9	27.1	27.0
Basin	Perak	26.8	26.5	27.0	26.9	26.7	26.9	26.2	26.5	27.3	28.3	28.6	26.9	27.0

Average Maximum Monthly Temperature

Unit : °C

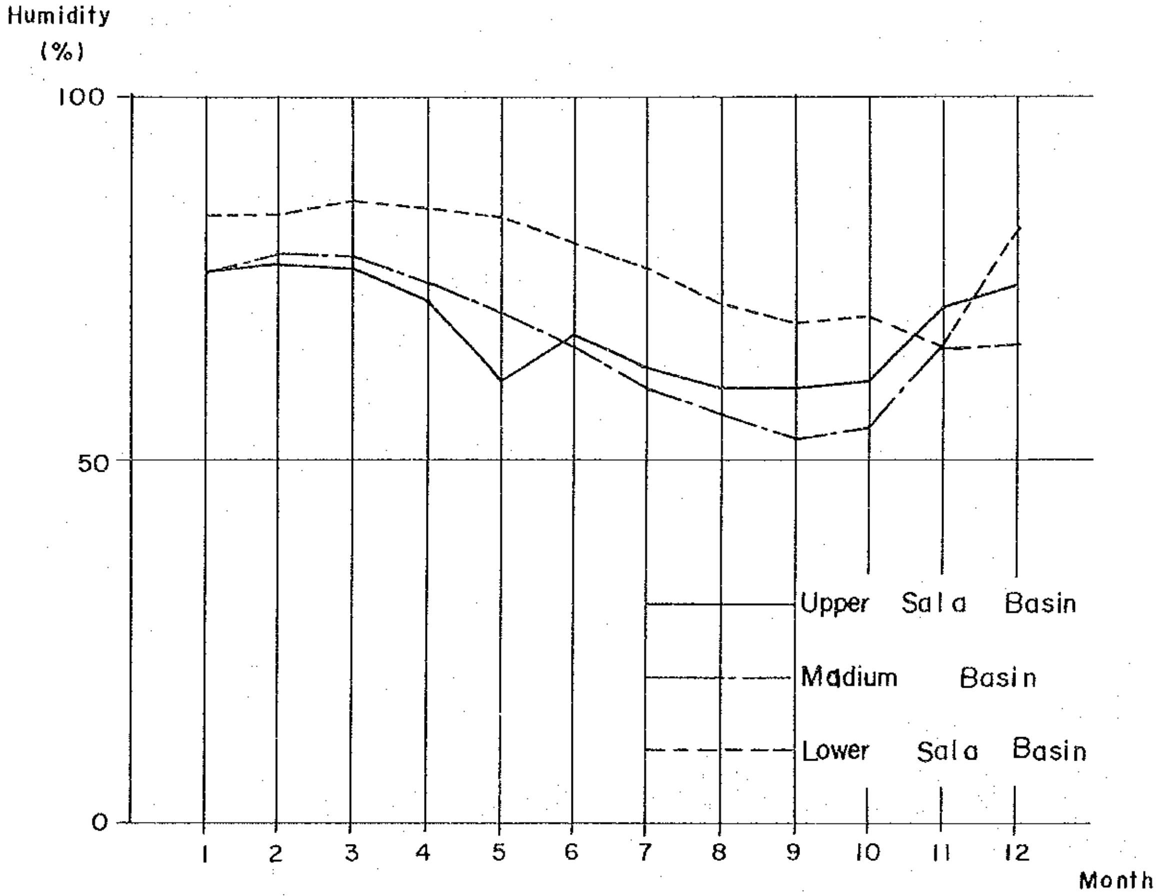
Location	Station	Jan.	Feb.	Mar	Apr	May	June.	July	Aug.	Sep.	Oct.	Nov.	Dec.	Mean
UpperSala	Panasan													
Basin	Pabelar	30.4	31.2	31.4	32.8	32.2	32.4	31.6	32.6	31.9	32.3	30.9	30.5	31.7
Madium	Iswahyudi	30.3	29.6	31.1	31.6	31.9	32.0	31.8	32.8	33.8	34.3	33.0	31.2	32.0
LowerSala	Kening	31.2	29.7	30.9	29.1	29.3	38.5	28.4	29.8	30.5	31.7	31.2	29.9	30.0
Basin	Perak	30.6	30.4	31.3	30.5	29.7	30.2	30.1	31.1	30.9	32.5	31.8	30.6	30.7

Average Minimum Monthly Temperature

Unit : °C

Location	Station	Jan.	Feb.	Mar.	Apr.	May.	June.	July.	Aug.	Sep.	Oct.	Nov.	Dec.	Mean
UpperSala	Panasan													
Basin	Pabelar	23.5	23.7	23.7	24.3	24.7	23.5	23.6	23.2	22.8	23.5	23.9	23.0	23.6
Madium	Iswahyudi	23.0	23.1	22.9	23.0	23.0	22.0	22.0	22.0	23.1	23.9	24.2	23.2	23.0
LowerSala	Kening	23.0	23.8	23.6	23.8	23.9	22.3	23.5	25.0	25.0	23.6	23.7	23.7	23.7
Basin	Perak	24.2	22.9	23.9	24.3	23.3	23.2	22.0	22.0	23.1	23.0	24.5	25.8	23.5

Fig 2.1.4 Average Monthly Relative Humidity

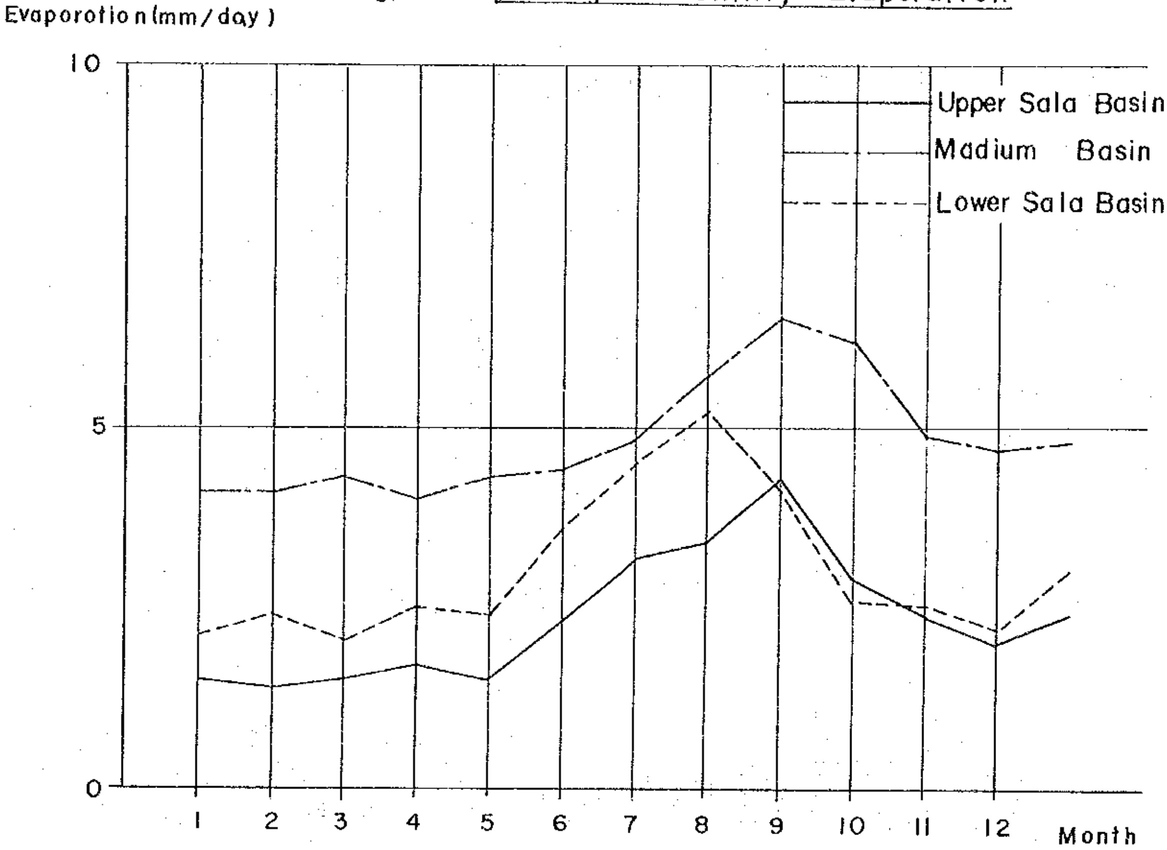


Average Monthly Relative Humidity

Unit : %

Location	Station	Jan.	Feb.	Mar.	Apr.	May.	June.	July.	Aug.	Sep.	Oct.	Nov.	Dec.	Mean
UpperSala Basin	Panasan Pabelun	75.8	76.9	76.4	72.0	60.4	66.6	62.7	59.4	59.5	60.8	70.5	74.0	68.7
Madium	Iswahyudi	75.8	78.5	77.8	74.4	70.2	65.2	59.8	56.0	53.0	54.1	65.2	65.7	66.5
LowerSala Basin	Kening	84.2	85.8	87.6	85.0	85.4	79.2	74.2	69.2	64.8	69.6	77.3	82.8	78.7
	Perak	82.7	82.4	83.1	84.0	81.1	80.0	78.3	73.3	72.6	69.1	74.4	80.8	78.4

Fig. 2.1.5 Average Monthly Evaporation



Average Monthly Evaporation

Unit : mm/day

Location	Station	Jan.	Feb.	Mar.	Apr.	May.	June.	July.	Aug.	Sept.	Oct.	Nov.	Dec.	Mean
Upper Sala Basin	Pabelan	1.5	1.4	1.5	1.7	1.5	2.3	3.2	3.4	4.3	2.9	2.4	2.0	2.4
Madium River Basin	Medium	4.1	4.2	4.5	4.6	4.9	5.0	4.7	5.6	6.7	6.6	4.8	4.8	5.0
	Dowhan	5.8	5.6	6.2	4.0	5.0	5.6	6.2	7.1	7.6	8.0	7.2	5.2	6.1
	Dung bendo	3.1	2.7	3.3	3.4	4.3	3.0	3.7	4.3	5.8	4.4	4.7	5.6	4.0
	Saradan	4.6	4.5	4.4	4.3	4.5	4.6	5.2	6.2	6.9	7.4	4.7	5.3	5.3
	Notopuro	4.2	4.3	4.2	4.7	4.4	5.2	5.8	7.1	7.6	6.9	4.9	4.6	5.3
	Ngebel	2.6	3.3	3.0	2.8	2.6	3.2	3.1	4.0	4.3	3.9	2.8	2.4	3.2
Lower Sala Basin	Kening	2.1	2.4	2.0	2.5	2.4	3.6	4.5	5.2	5.2	2.6	2.5	2.2	3.0

Table 2.1.1 (a) Monthly Rainfall in Upper Wonogiri Basin

(Unit: mm)

Year	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sep.	Oct.	Nov.	Dec.	Total
1952	380	307	257	81	99	6	4	61	55	170	328	186	1,936
1953	145	344	315	192	168	23	29	2	0	0	108	302	1,628
1954	364	473	261	297	269	61	52	53	48	105	439	250	2,672
1955	243	241	321	207	165	181	200	73	59	143	345	264	2,442
1956	312	224	109	146	136	189	136	65	53	77	143	198	1,788
1957	219	318	296	89	43	15	153	127	0	12	144	373	1,789
1958	258	281	387	222	124	121	82	24	58	148	154	473	2,332
1959	387	336	349	154	143	51	49	0	31	11	206	352	2,069
1960	431	411	257	246	314	16	21	0	0	15	330	205	2,246
1961	297	257	251	133	69	0	0	0	3	23	110	278	1,421
1962	346	279	333	345	105	18	19	18	6	63	133	366	2,026
1963	444	345	331	101	7	26	0	0	0	5	43	213	1,515
1964	237	184	242	249	152	29	4	13	17	184	139	224	1,674
1965	415	393	254	90	30	10	2	0	0	1	127	244	1,566
1966	275	296	496	244	25	86	0	0	7	174	152	269	2,024
1967	638	322	224	253	38	0	0	0	0	6	164	214	1,859
1968	505	415	506	166	268	209	202	58	29	137	199	503	3,197
1969	400	478	255	282	17	34	0	0	0	96	115	183	1,860
1970	322	360	247	262	267	49	8	0	48	128	437	435	2,563
1971	432	505	564	107	219	69	17	17	5	290	306	342	2,864
1972	431	362	433	185	95	0	0	5	0	14	183	426	2,134
1973	504	447	475	302	389	207	33	29	131	104	261	196	3,078
Mean	363	344	326	198	143	64	46	25	25	86	207	295	2,122

Table 2.1.1 (b) Monthly Rainfall in Upper Jurug Basin

(Unit: mm)

Year	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sep.	Oct.	Nov.	Dec.	Total
1952	394	317	303	98	99	12	4	82	45	192	336	259	2,141
1953	170	329	364	234	175	15	29	1	4	2	177	278	1,778
1954	393	386	225	308	259	72	62	80	45	138	431	229	2,628
1955	309	240	341	253	150	147	207	92	54	153	345	270	2,561
1956	344	306	115	154	173	187	176	134	49	82	171	213	2,104
1957	251	349	373	169	65	23	178	96	1	22	186	354	2,067
1958	272	327	412	255	156	99	133	56	85	144	179	481	2,599
1959	354	339	375	156	133	75	66	1	39	21	198	324	2,081
1960	422	429	273	244	301	21	29	5	11	47	344	187	2,313
1961	333	293	254	176	100	2	14	0	1	40	122	284	1,619
1962	394	299	330	354	78	59	27	42	7	77	182	362	2,211
1963	389	368	381	165	23	18	0	0	0	6	54	225	1,629
1964	215	221	306	250	173	73	5	18	29	257	138	277	1,940
1965	341	362	268	106	47	17	7	0	2	14	157	277	1,598
1966	312	328	451	211	46	82	0	0	12	168	195	293	2,098
1967	536	330	243	204	188	0	0	0	2	11	148	223	1,885
1968	399	261	487	157	306	203	171	101	45	130	239	401	3,000
1969	360	374	271	344	41	21	0	0	2	97	123	234	1,867
1970	356	295	310	231	277	43	18	0	64	107	379	395	2,475
1971	364	401	490	134	189	66	30	7	12	258	230	317	2,498
1972	365	264	432	175	139	0	0	10	0	7	191	360	1,943
1973	485	417	434	264	348	178	37	33	140	124	214	222	2,896
Mean	352	334	338	211	158	64	54	35	29	95	216	293	2,179

Table 2.1.1 (c) Monthly Rainfall Upper Sala Basin (Unit: mm)

Year	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sep.	Oct.	Nov.	Dec.	Total
1952	373	342	306	115	99	11	7	93	56	193	340	254	2,189
1953	203	334	327	274	203	15	29	0	1	4	191	282	1,863
1954	395	364	215	275	223	90	68	67	43	166	428	213	2,547
1955	406	249	342	229	177	145	243	93	57	204	317	255	2,717
1956	355	313	164	183	178	194	128	127	56	95	167	225	2,185
1957	259	294	418	206	80	27	156	62	3	36	193	340	2,074
1958	272	388	488	283	191	71	158	71	121	187	189	492	2,871
1959	340	339	387	181	165	75	73	7	40	42	218	323	2,190
1960	335	434	267	255	254	31	33	7	10	56	372	201	2,255
1961	331	257	263	158	137	2	16	0	0	41	166	254	1,625
1962	389	265	339	387	59	111	52	55	4	84	190	406	2,341
1963	355	371	387	185	19	26	0	0	0	27	91	257	1,718
1964	235	223	310	276	195	90	17	21	29	297	138	223	2,054
1965	340	312	284	138	50	17	17	0	1	18	160	270	1,607
1966	318	324	426	217	80	76	1	1	18	164	209	226	2,060
1967	501	320	221	173	30	0	0	0	11	38	160	245	1,689
1968	299	345	440	221	296	186	210	78	37	119	279	362	2,872
1966	333	290	267	303	40	11	2	0	1	91	143	259	1,740
1970	307	314	335	213	263	55	33	1	94	85	342	322	2,364
1971	326	387	400	104	187	113	30	6	24	299	263	310	2,449
1972	181	238	453	217	161	0	0	19	6	20	193	355	1,843
1973	413	399	417	265	348	139	55	61	153	125	235	230	2,840
Mean	331	323	337	221	157	67	61	34	34	109	227	286	2,187

Table 2.1.2 Seasonal Rainfall

Year	Upper Sala Basin (A = 6,072 km^2)			Madiun River Basin (A = 3,755 km^2)			Lower Sala Basin (A = 6,273 km^2)			Whole Sala Basin (A = 16,100 km^2)		
	Rainy	Dry	Annual	Rainy	Dry	Annual	Rainy	Dry	Annual	Rainy	Dry	Annual
1952	1730	459	2189	1687	285	1972	1522	463	1985	1639	420	2059
1953	1611	252	1863	1645	356	2001	1478	315	1793	1567	301	1868
1954	1890	657	2547	1799	602	2401	1693	682	2375	1792	654	2446
1955	1798	919	2717	3065	1089	4154	1692	701	2393	2052	874	2926
1956	1407	778	2185	2058	1045	3103	1148	525	1673	1458	742	2200
1957	1710	364	2074	1451	292	1743	1516	296	1812	1574	321	1895
1958	2072	799	2871	1883	455	2338	1520	558	2078	1813	625	2438
1959	1788	402	2190	1352	325	1677	1554	188	1742	1595	301	1896
1960	1864	391	2255	1413	267	1680	1482	290	1772	1610	323	1933
1961	1429	146	1625	1237	184	1421	1216	169	1385	1301	183	1484
1962	1976	365	2341	1776	271	2047	1449	287	1736	1724	313	2037
1963	1646	72	1718	1372	66	1438	1161	103	1264	1393	83	1476
1964	1405	649	2054	1367	545	1912	1160	588	1748	1301	601	1902
1965	1504	103	1607	1390	97	1487	1931	78	2009	1644	92	1736
1966	1720	340	2060	1917	293	2210	1502	275	1777	1681	304	1985
1967	1610	79	1689	1274	29	1303	3613	85	3698	2312	70	2382
1968	1946	926	2872	1807	849	2656	1634	1423	3057	1792	1102	2894
1969	1595	145	1740	1577	187	1764	1455	232	1687	1536	189	1725
1970	1833	531	2364	2060	517	2577	1551	380	1931	1776	469	2245
1971	1790	659	2449	1777	624	2401	2005	636	2641	1871	642	2513
1972	1637	206	1843	1408	153	1561	1224	186	1410	1458	186	1644
1973	1959	881	2840	1659	626	2285	1806	789	2595	1829	786	2615
Mean	1725	762	2187	1681	416	2097	1605	420	2025	1669	436	2105

2.2.2 Point rainfall

As regards the point rainfall in the Sala river basin, its characteristics are known only qualitatively, but not quantitatively. On that account, with the automatic stations located in and around the Jurug-Wonogiri basin as the base stations, rainfall characteristics were examined on the basis of the relevant records of 1975.

The locations of the base stations are shown in Fig. 2.2.1. A brief mention of the results is as follows:

1) The period of time, in which occurs rainfall of over 10 mm an hour, lasts from noon to 6.00 p.m and the rainfall during these 6 hours consists of about 70% of the daily total; heavy rainfall takes place in the afternoon. (See Fig. 2.2.1).

2) The correlation of the hourly rainfall among the stations was notably weak when the distance between the 2 stations was short - the shortest of which was about 15 km - meaning that the hyetal region may be very much limited. (See Fig. 2.2.1).

3) The correlation between the rainfall and its duration was very weak, especially in the plane. The mass curve shows that the rainfall stops in less than 6 hours. (See Fig. 2.2.2 and 2.2.3).

4) Therefore, the daily rainfall of the N-years retun period might correspond to the total rainfall in the 6-hour duration, but the daily rainfall of the N-years at each station did not show much difference. As estimated, 187 mm/day for 100-year and 104 mm/day for 2-year were the respective mean values for the 4 stations. (See Fig. 2.2.4).

5) The point rainfall was estimated from the hyetograph according to the mass curve and probability of precipitation. (See Fig. 2.2.5).
The hyetograph exhibited a more rainfall in the first one hour which shared about 60% of the total rainfall. The character of hyetograph for point rainfall however, shows that the specific discharge in a small basin might be very large: to quote for reference, a basin with a 1-hour concentration must have 15.0 of the specific discharge for the 100-year rainfall in rational formula if the coefficient of runoff is taken as 0.5. But this big discharge might not be transferred to the main river in its present condition because of the reasons connected with the regimen.

6) As the density of the network of stations for measuring the hourly rainfall is very low, it is difficult to make a through study on areal hourly rainfall.

2.2.3 Areal rainfall

With regard to the areal rainfall in the Upper Sala basin, the monthly, seasonal and annual rainfall studied by use of the records at 27 rainfall station mentioned earlier. As to the daily rainfall the study was made with reference to the flood which has the maximum discharge every year.

1) Rainfall - monthly, seasonal and annual.
The results of calculations on the monthly, seasonal and annual rainfall are shown in Tables 2.3 and 2.4;
The rainfall characteristics being as follows.
   a) Periodic variations have been observed with the annual rainfall during the 4/6 of the said period: a number of years centering around 1955 and 1960 were the wet years and, from 1960 through 1968, the rainfalls during the dry seasons were particularly less and, thus, drought years continued. (See Fig. 2.2.6).
   b) Judging from the rainfall ratio in the 3 areas of Upper Sala on the yearly and monthly basis, the rainfall in the said 3 areas remained almost identical and was seldom maldistributed. (See Fig. 2.2.7).
   c) When the rainfall variations are observed on the monthly basis during the rainy season extending from November through April, it occupies about 65% of the annual rainfall on an average, and 35% during the dry season. But in the dry season of a drought year, it happend that rainlessdays continued for 2 or 3 months, in 1967 completely rainless days continued for 3 months in the Upper Sala basin. (See Fig. 2.2.8).

2) Daily rainfall
The results of the study on the maximum daily rainfall in a year are indicated in Table 2.2.1. This rainfall is mostly observed in the rainy season of March.
In the basin of the Upper Jurug, the heaviest of the rainfalls was registered at the time of the 1966 flood, the largest on the record, when it rained 172 mm/3-days and 62 mm/day. At the time of the same flood, the rainfall in the Upper Wonogiri basin was even heavier, marking 215 mm in 3 days and 158 mm a day. Of other floods, the particularly large one was that of 1968.

2.2.4 Probable rainfall

The study about the probability of rainfall was made on the 3-day and one-day rainfall.

The results of the calculations are given in Fig. 2.2.9.

According to the figures, the 3-day rainfall in the 1966 flood corresponds tothe 50-year return period in the Upper-Wonogiri basin and to the 40-year return period in Upper Jurug, but in the Jurug-Wonogiri it turned to be the 15-year return period only.

2.3 DISCHARGE

Dealt with herein are the characteristics of the monthly and annual discharges in the Jurug-Wonogiri basin. As to the flood, expanations will be presented later under Item 2.5.

2.3.1 Water-gauge station

Along the Sala river in the Jurug-Wonogiri basin, the automatic water-gauges are established at Jurug and Juranggempal, and at Jurum along K. Dengkeng. In the proximities of the Nguter and Bacem Bridges and of Mojo, the staff gauges are set up but it is only at Mojo where the stage records are kept. Also the discharge observation was conducted a number of times at the Terum[2] site on K. Samin without the staff gauge.

The location of the water-gauge stations and their relevant informations are given in Fig. 2.3.1.

2.3.2 Rating curve

The rating curves for Juranggempal, Nguter, Bacem, Jurug, Jarum and the Terum[2] Site are shown in Fig. 2.3.2.

The stage-records over a long period are kept at the stations at Juranggempal (11 years) and Jurug (8 years); the reasonability of the stage records was examined in relation to the lowest water stage initiated in the dry season. (See Fig. 2.3.3.)

The conclusion arrived at after such examination was that, at the Juranggempal site, the water level during

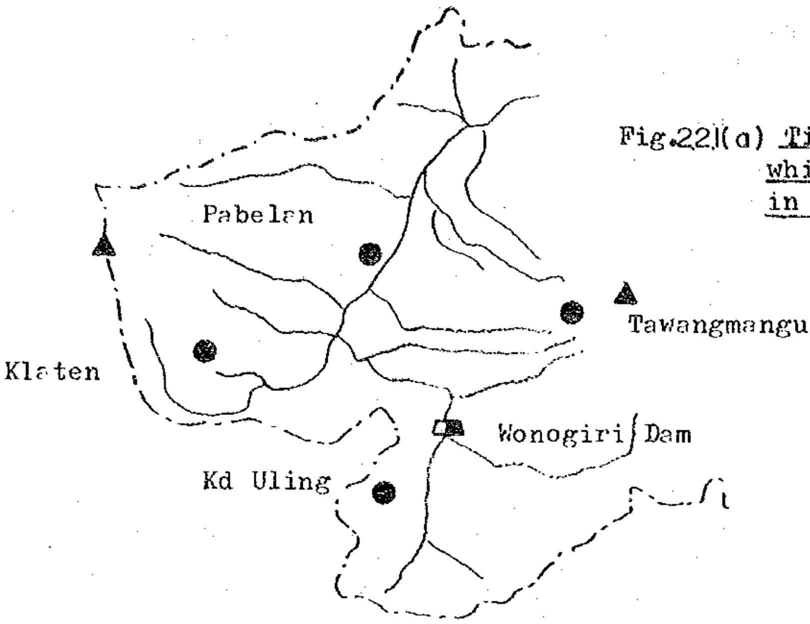


Fig.22(a) Time-zone distribution which occur over 10 mm in an hour.

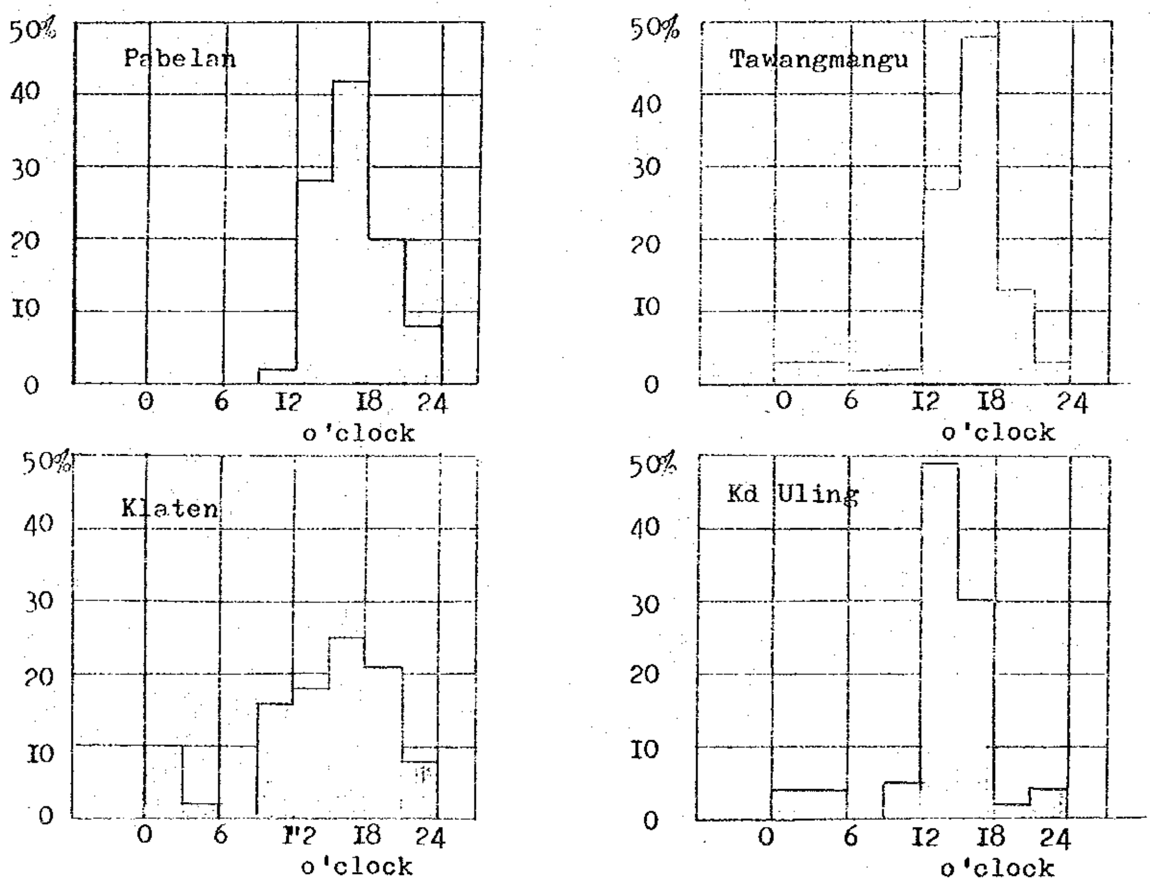


Fig.2.21(b) Correlation of Hourly Rainfall
(based on 1975 record)

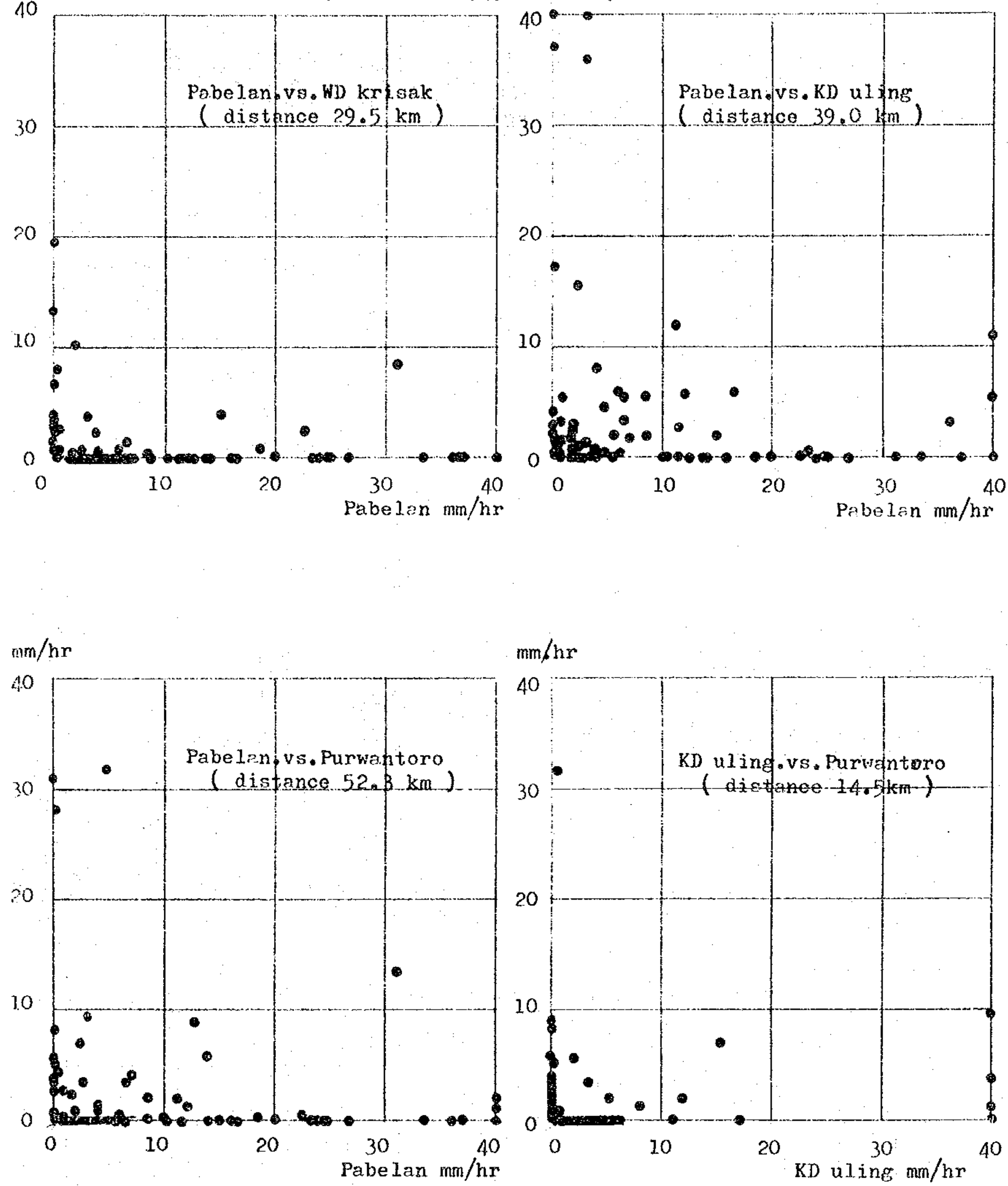


Fig.2.22 Mass curve of Rainfall (point rainfall)

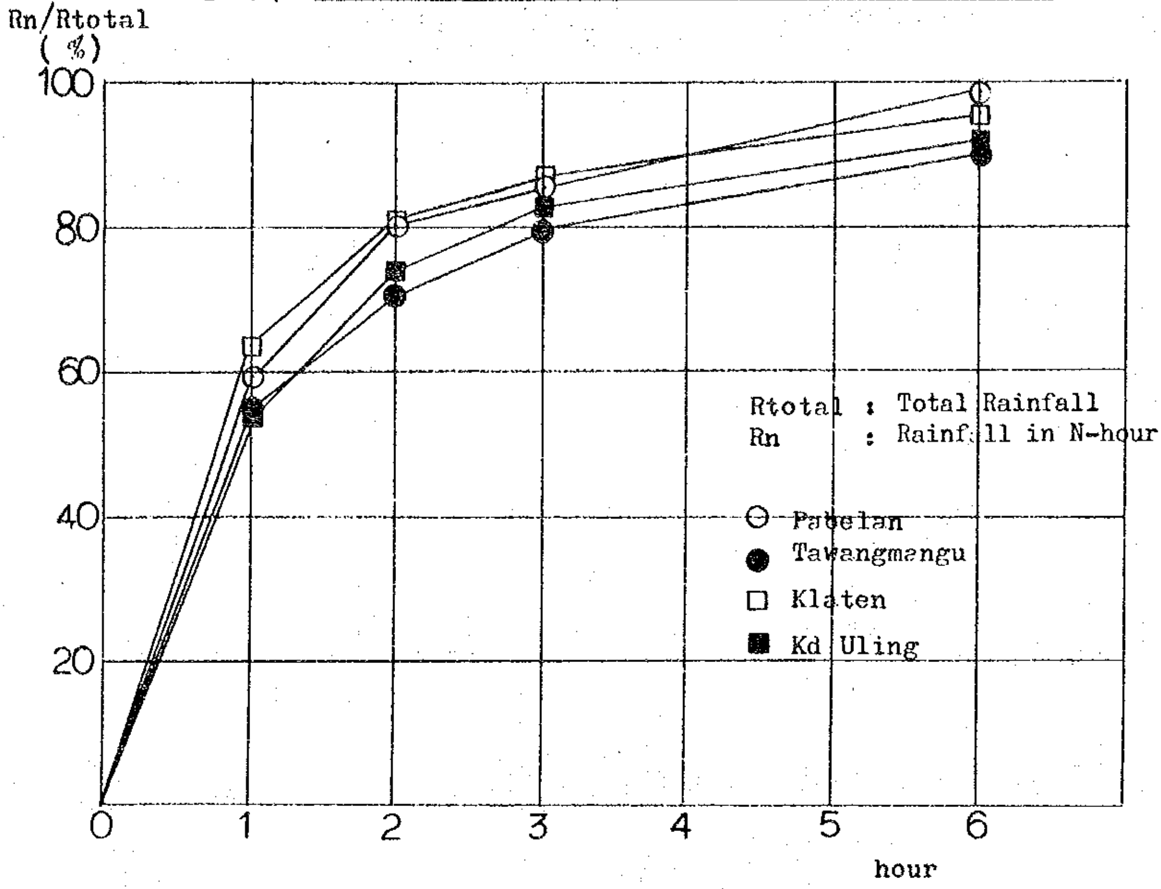


Fig. Distribution of rainfall duration

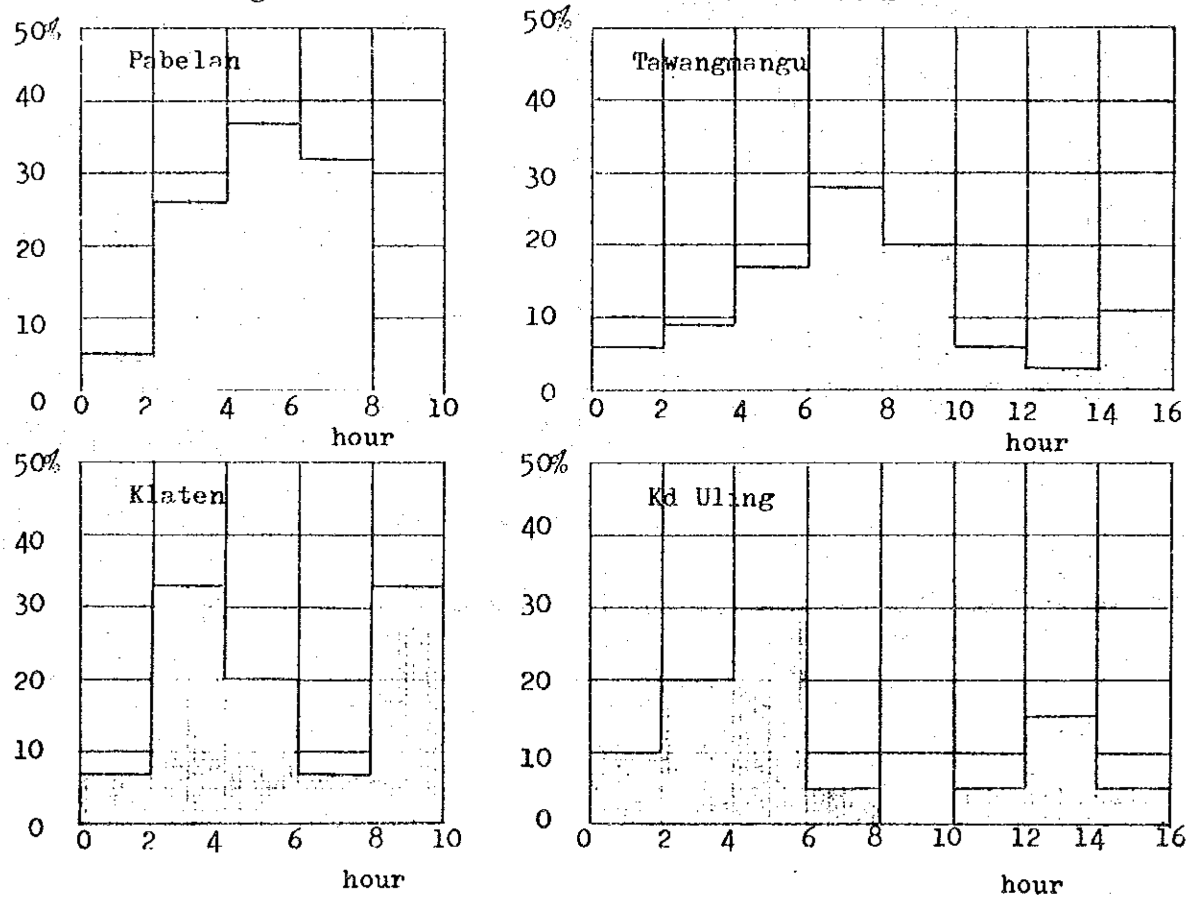


Fig.2.2.3 Mean distribution of Hyetograph Haveing six-hour rainfall duration

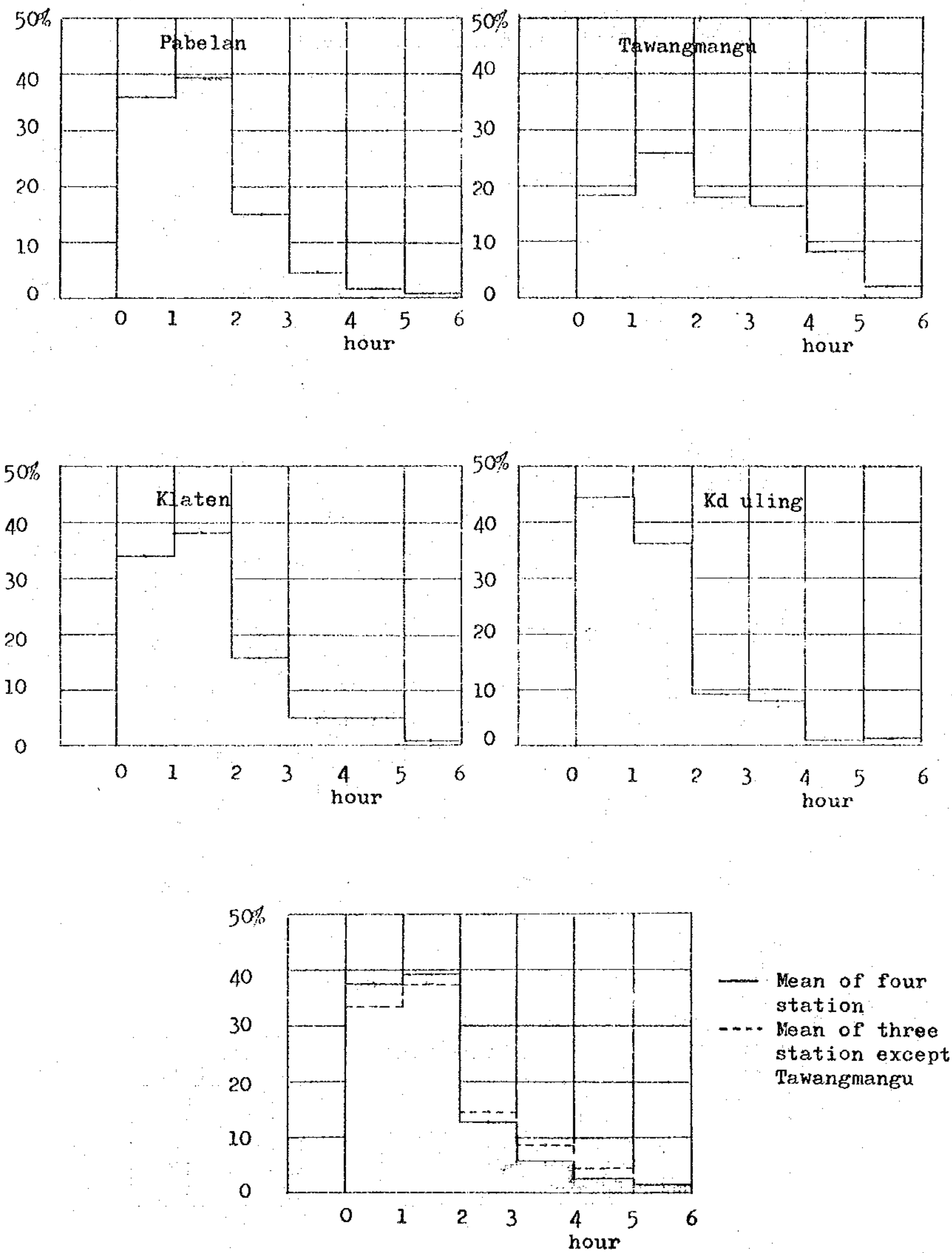
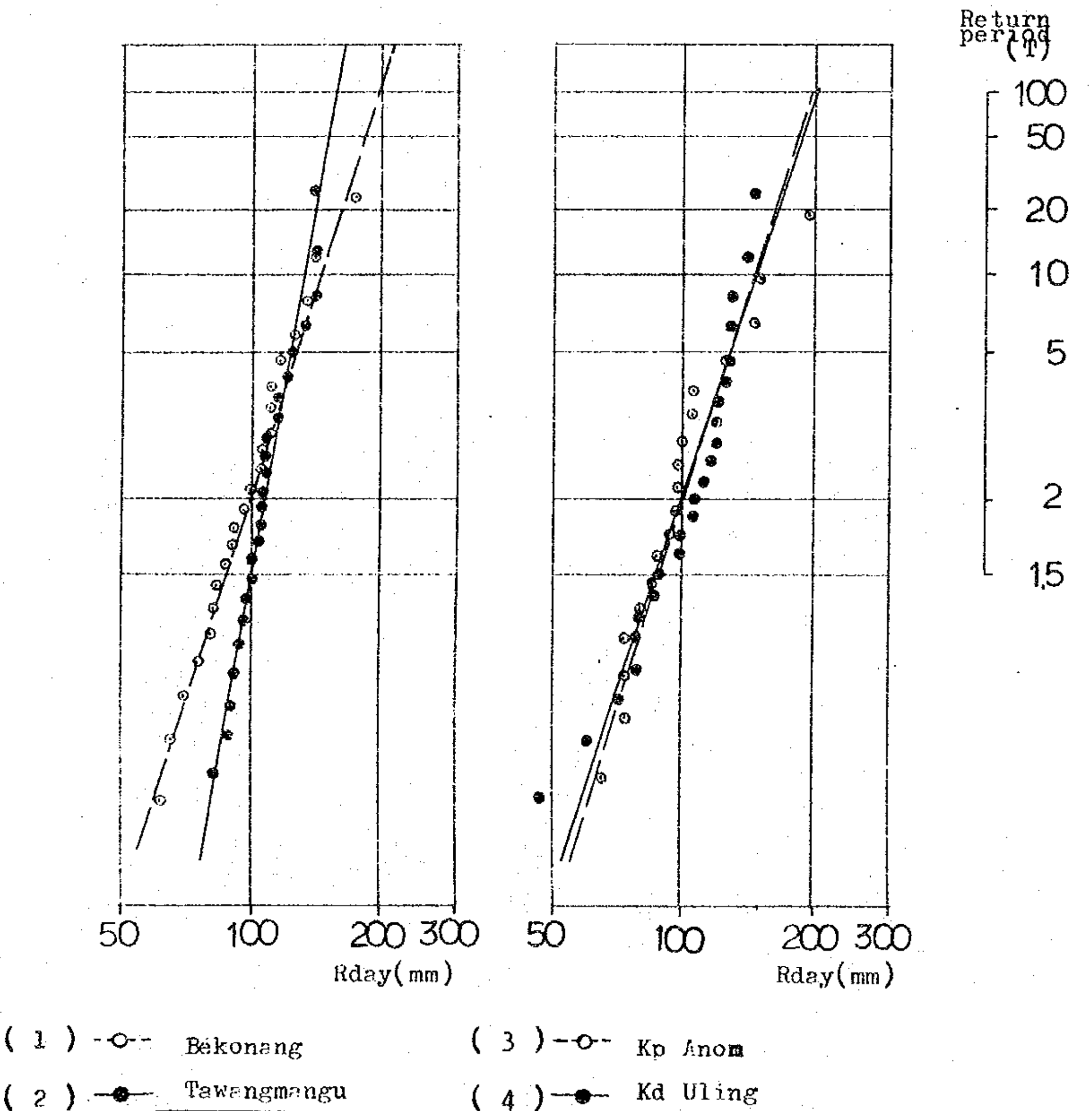


Fig. 2.2.4 Probable Daily Rainfall (Point Rainfall)



Return period	Station No.				
	(1)	(2)	(3)	(4)	Mean
100	198	155	195	201	187
50	181	150	180	183	174
10	143	130	143	145	140
5	125	121	128	128	126
2	100	104	103	103	104

Fig. 2.2.5 Estimated mass curve of Rainfall and Hyetograph for T-year rainfall

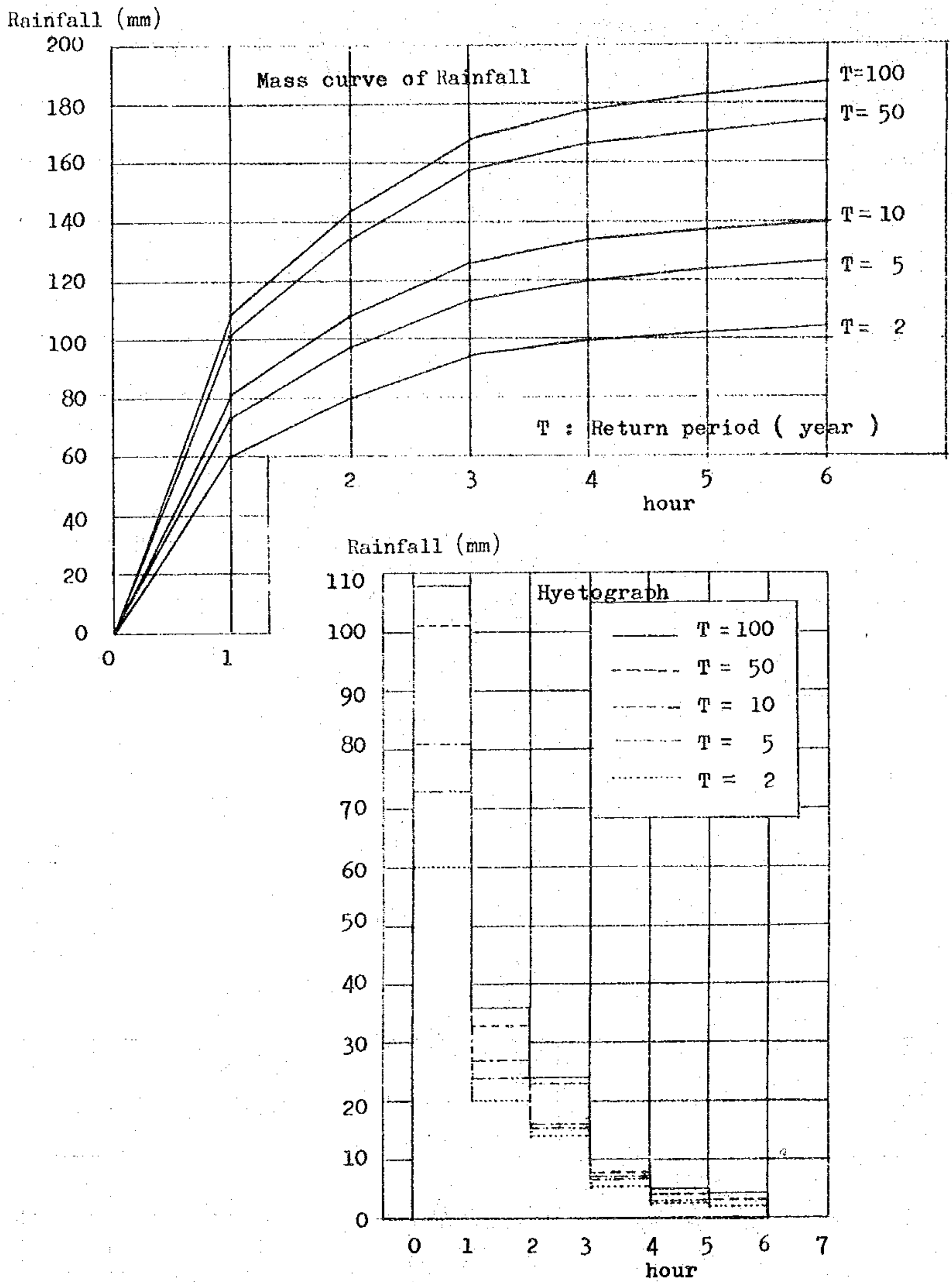


Fig. 2.2.6 Annual tendency of Rainfall

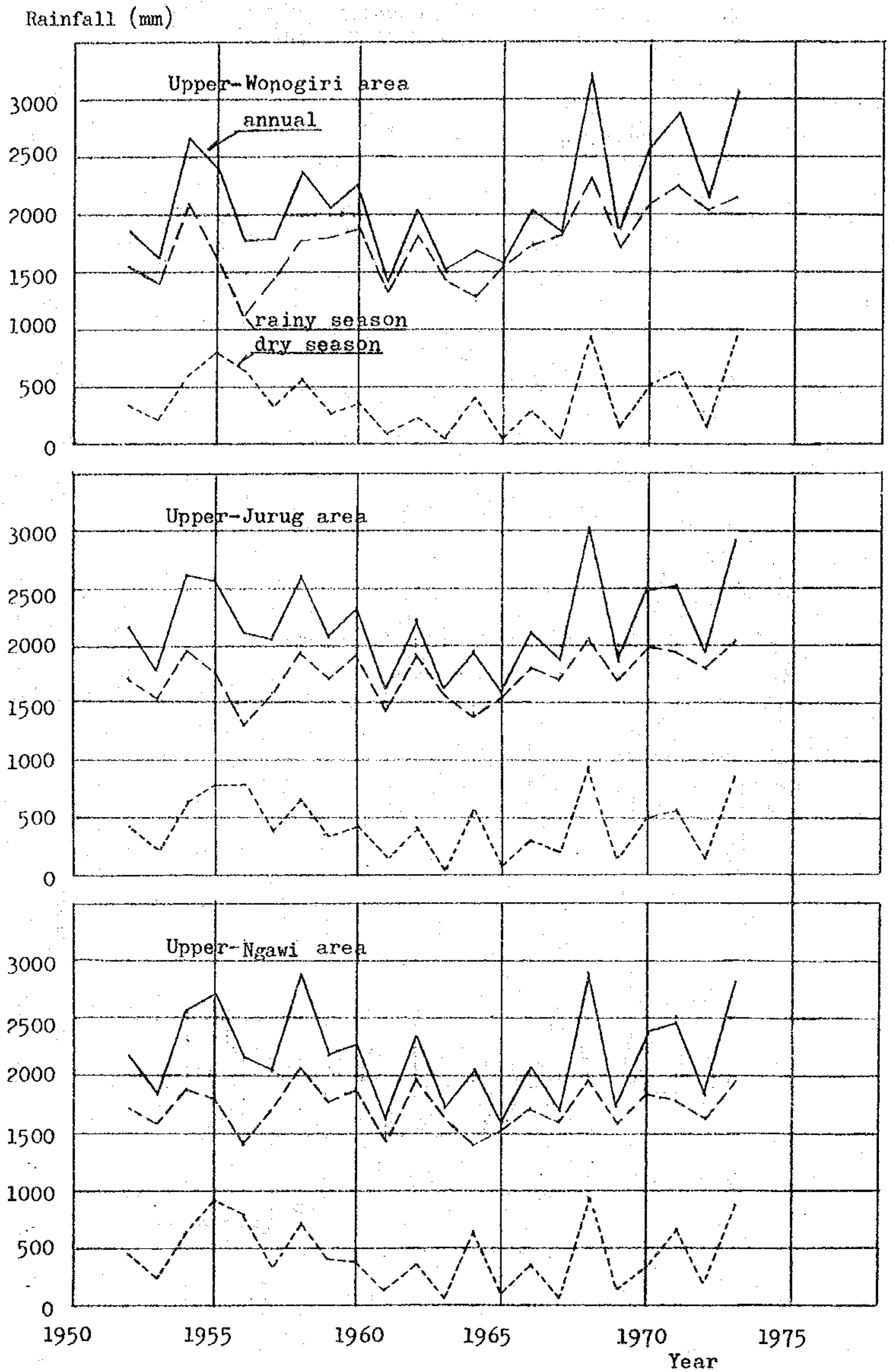
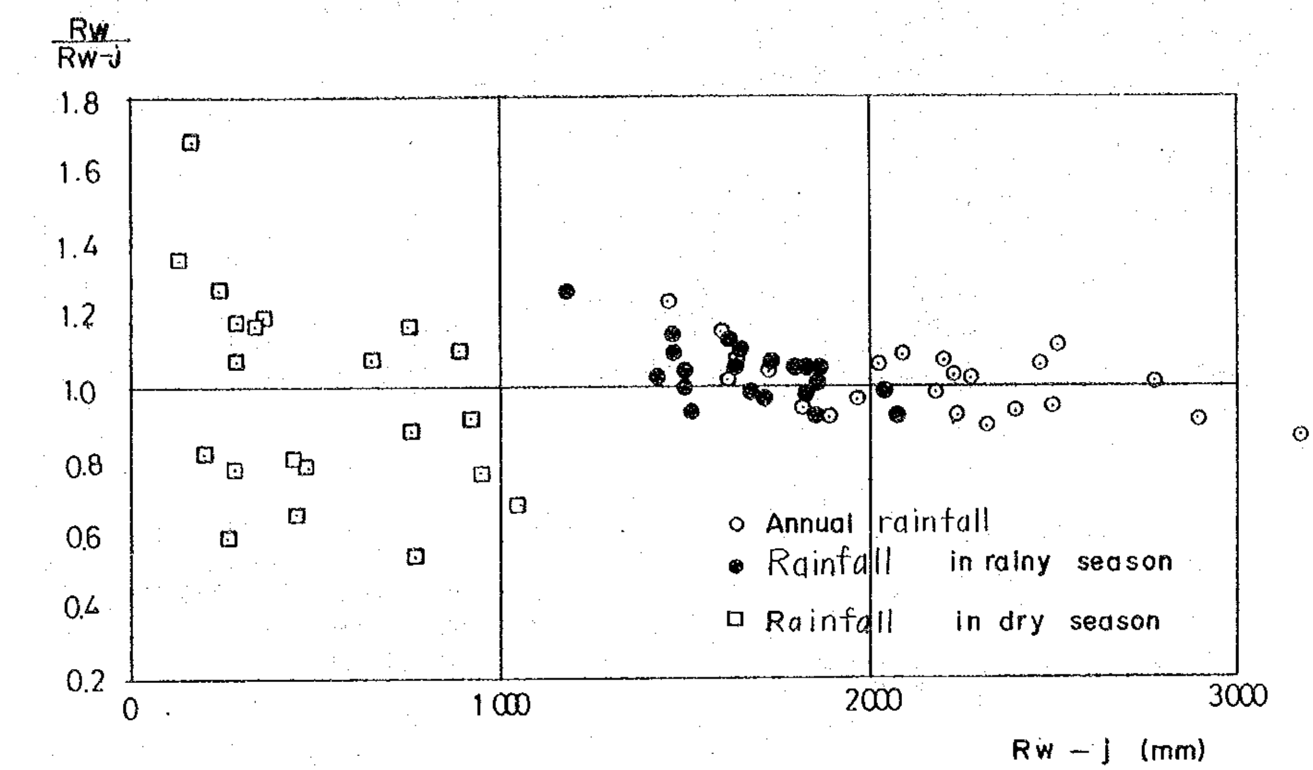
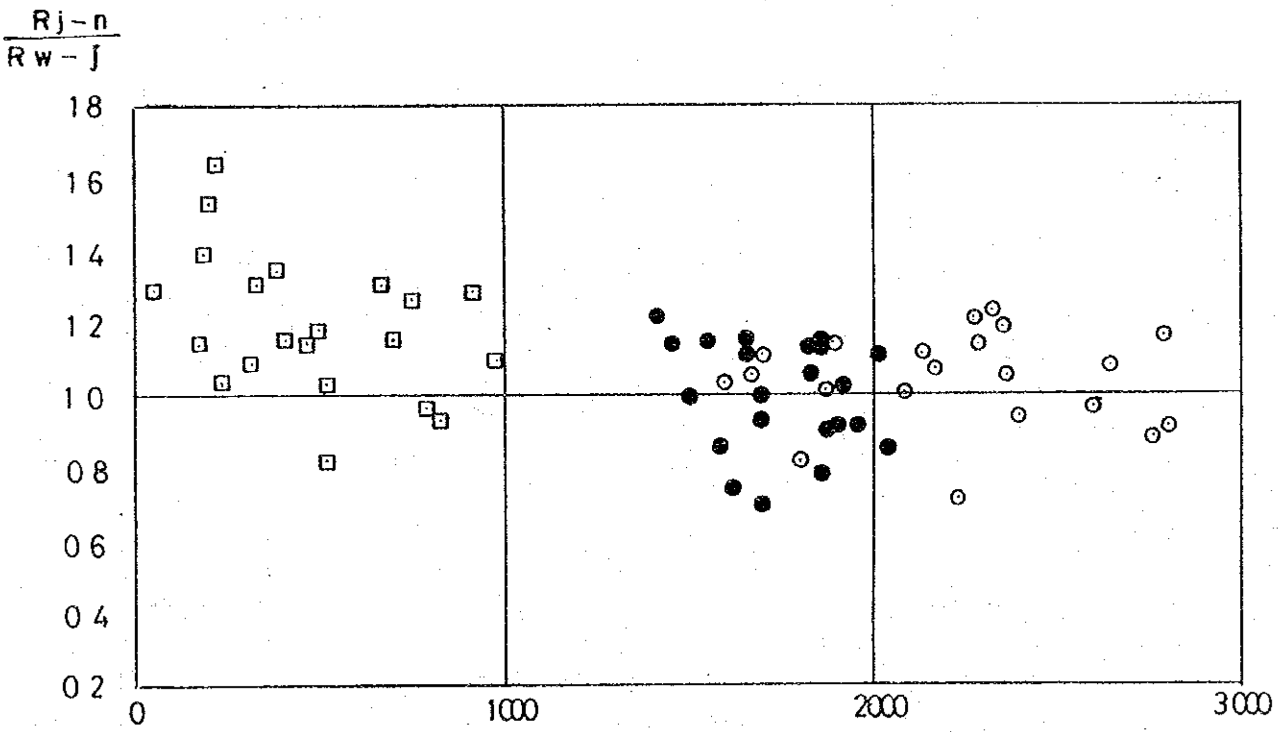


Fig. 2.2.7 Localty of Rainfall





Note; Rw-J = Rainfall in Wonogiri-Jurug area.
Rw = Rainfall in Upper-Wonogiri area.
Rj-n = Rainfall in Jurug-Ngawi area.

Fig.2.28 Correlation of Rainfall

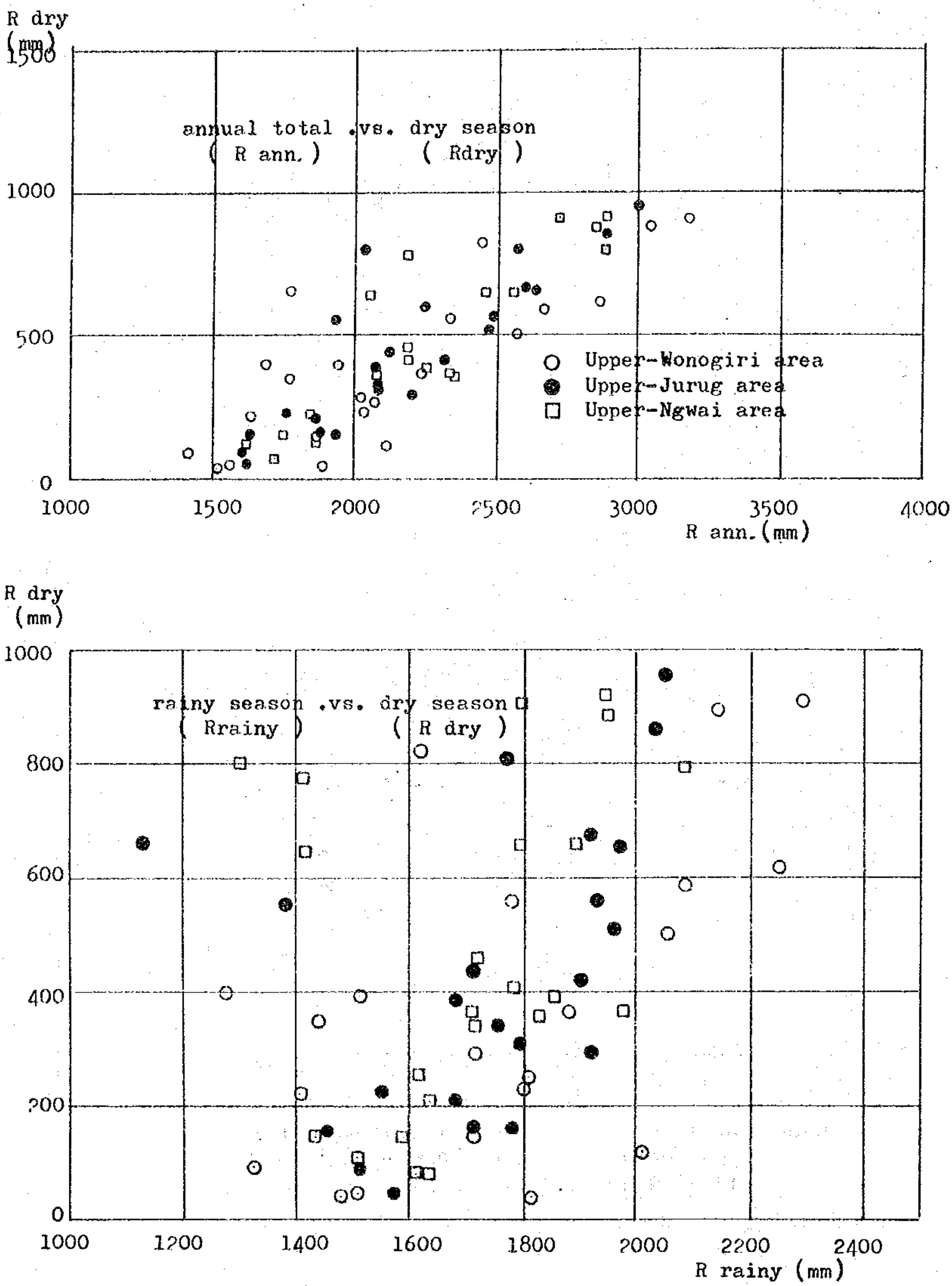
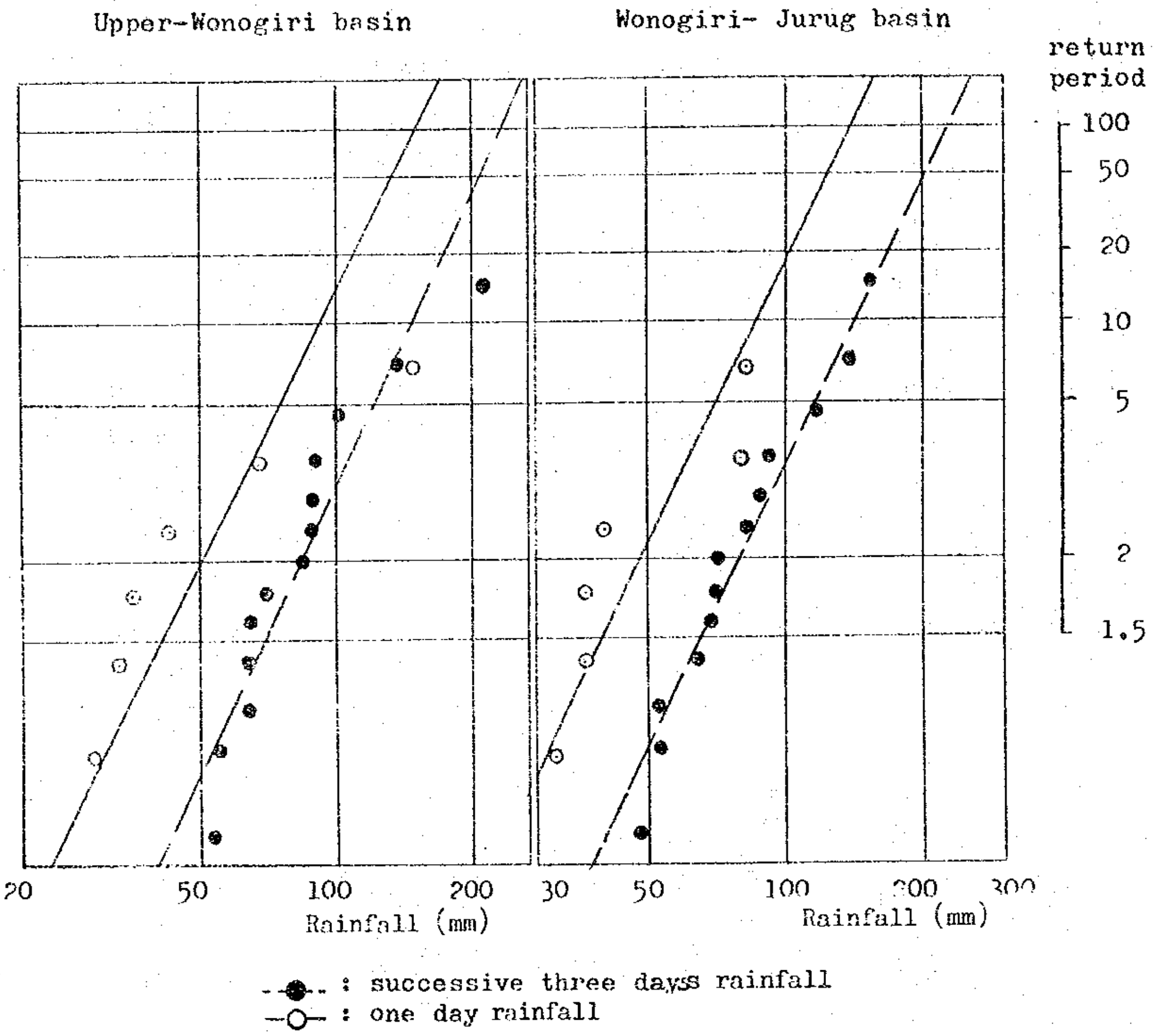


Fig. 2.2.9 Probable Rainfall in Upper-Wonogiri Basin and in Wonogiri-Jurug Basin



return period	UPPER-WONOGIRI BASIN		WONOGIRI-JURUG BASIN	
	3 days	1 day	3 days	1 day
100	230	150	215	135
50	210	130	195	120
20	170	110	160	100
10	145	90	130	85
5	120	75	115	70
2	85	50	80	50

Table 2.2.1 Maximum Annual Rainfall

(1) Successive three days.

No.	Year	Upper Wonogiri basin	Wonogiri-Jurug basin	Upper Jurug basin
1	1956	55.0	72.2	65.0
2	1958	139.0	89.1	110.0
3	1960	84.0	84.0	84.0
4	1961	89.0	118.3	106.0
5	1962	88.0	70.8	78.0
6	1963	53.0	49.6	51.0
7	1966	215.0	140.6	172.0
8	1967	101.0	70.0	83.0
9	1968	90.0	150.3	125.0
10	1970	64.0	53.7	58.0
11	1971	64.0	93.3	81.0
12	1972	64.0	65.7	65.0
13	1973	69.0	53.5	60.0

(2) One day

No.	Year	Upper Wonogiri basin	Wonogiri-Jurug basin	Upper Jurug basin
1	1966	158.0	83.0	62.0
2	1967	43.0	39.4	41.0
3	1968	68.0	81.0	75.5
4	1971	35.6	37.4	35.0
5	1972	29.1	34.6	28.2
6	1973	33.3	33.2	32.4

of January 1965 through July, 1968 was measured from a point 0.5 m above the gauge zero and, after September of 1972, 1.0 m above the zero. As to Jurug, it was assumed that the water level for the duration of January to July, 1969 was measured from a point 0.5 m below the gauge zero.

At Jarum on K. Dengkeng, the stage equipment is set in such a way as to make it unserviceable when the water depth falls to the level 1 m above the gauge zero and, therefore, the Jarum records are not reliable save for the rainy season.

2.3.3 Daily mean discharge

The daily mean discharges calculated from the rating curve are exhibited in Figs. 2.3.4 to 2.3.6.

The discharge, in the master plan and prefeasibility study reports, is assumed on the monthly unit basis at the Karangnomgko further down steam from the confluence of the Madium river and the Sala river.

The comparison of the newly assumed discharge with the previous one is indicated in Fig. 2.3.7 and, judging from the said figure, both of them are in conformity with each other in comparatively good accuracy. Accordingly, the previous results may well be usable as the monthly mean discharge prior to the commencement of the observation.

2.3.4 Character of discharge

Based on the daily mean discharge, the characteristics of discharge were examined with the findings as follows:

1) The daily discharge is to accord with the $\sqrt{Q}$-distribution. (See Fig. 2.3.8 and Fig. 2.3.9).

2) The monthly mean discharge is larger in the rainy season and smaller in the dry season. In the wet year the discharge is considerably large even in the dry season but it becomes practically nil in the drought year, presenting a rather notable fluctuation inbetween. (See Fig. 2.3.10).

3) A comparison of the monthly mean discharge between Juranggempal and Jurug indicates that the discharge is increased at about the same ratio of the areal increase. This is quite convincing even from the investigation result that the annual rainfall shows not much areal difference as already mentioned before. (See Fig.2.3.11)

4) However, when the daily mean discharges at these two points are compared it seems that:
When the discharge is smaller than 250 m^3/s at the Juranggempal site, the ratios of discharges at the two points are almost equal to the areal ratios. When the discharge is larger, the discharge at Juranggempal is considerably larger than at Jurug point. This suggests that a flood may occur at the river reach between Jurug and Juranggempal. (See Fig. 2.3.11).

5) The annual rainfall loss showed about the identical value at both Jurug and Juraggempal, the mean value being approximately 1,600 mm/year. (See Fig. 2.3.12).

2-4 SEDIMENT

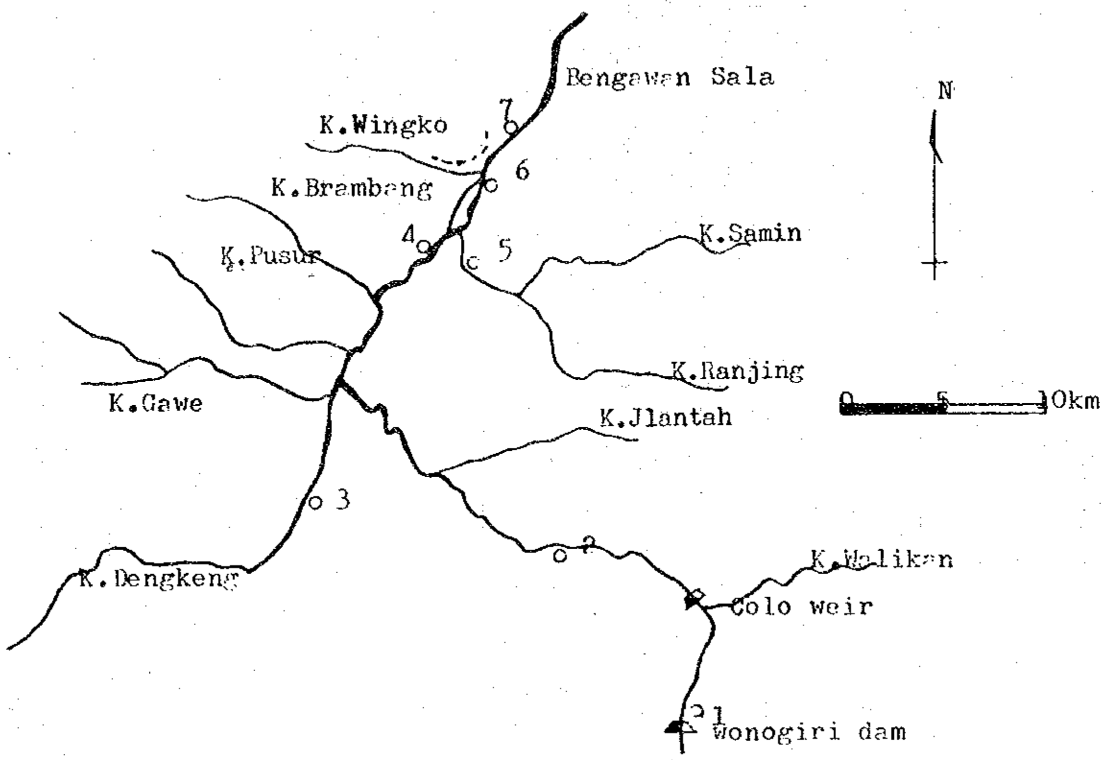
The reddish laterite extensively distributed over its basin, flows along with the storm sewage into the Sala river and, therefore, the river water is constantly in a reddish color. For this reason, it has hitherto been considered that the river bed of the Sala river was formed of the silt of extremely fine grain size.
In the present investigations, the bed materials have been collected from the main channel and tributaries for the field surveys conducted with regard to the sediment.

2.4.1 River bed materials

The positions of the 15 sampling points wherefrom the bed materials have been collected to data are shown in Fig. 2.4.1 with the sample of the grain size distribution in Table 2.4.1 and the profile of the mean grain size in Fig. 2.4.2. It is indicated in figures that the grain size of the main river bed is in good conformity with that of the tributary.

In the Jurug-Colo reach, the mean grain size (d50) is generally uniform at 0.5 m, but the material of the Wonogiri vicinity is quite coarse under the influence of the coarse sediment entering into the main stream. On its upper stream,

Fig.2.3.1 Location of Water Level Station



NO	Name	start of record	observer	river	Catchment area	Instrument	gauge zero +S.H.V P (m)
1	Jurang-gempal	1965	P.B.S D.P.M.A	Main-Sala "	1350km	Ordi. Auto.	106.473
2	Nguter	-	P.B.S	"	1571	-	-
3	Jarum	1974	P.B.S	K.Dengkeng	475	Auto.	93.668
4	Bacem	-	Irrigation office	Main-Sala	2778	Ordi.	83.516
5	Terun?	-	P.B.S	K.Samin	305	-	-
6	Mojo	-	Irrigation office	Main-Sala	3617	Ordi.	83.516
7	Jurug	1968	P.B.S	Main Sala	3220	Auto.	78.309

Fig 232(a) Rating Curve at Juranggemapal Gauge

Zero point = 107.473m S.H.U.P.

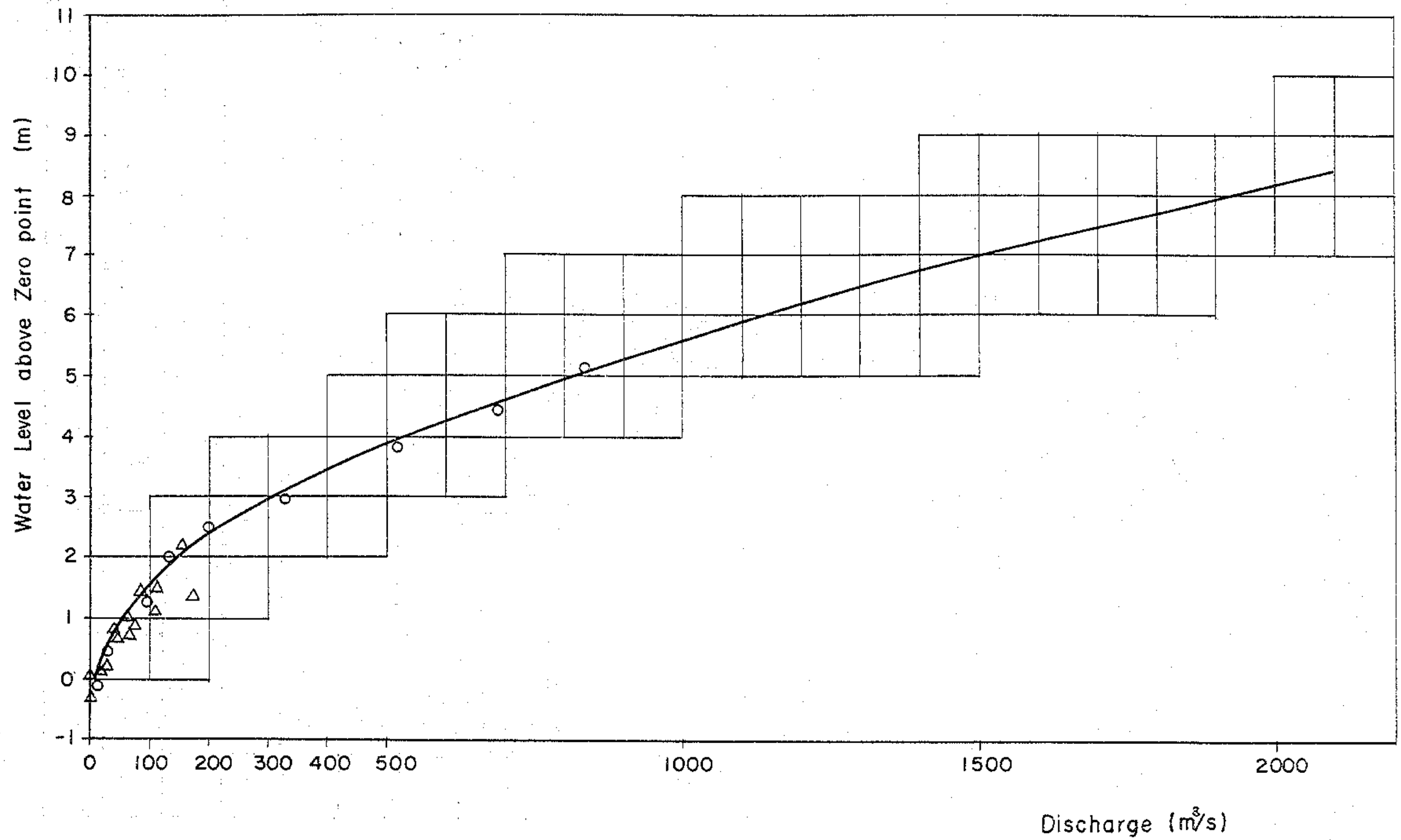


Fig.232(b) Rating Curve at Ngutur Bridge site

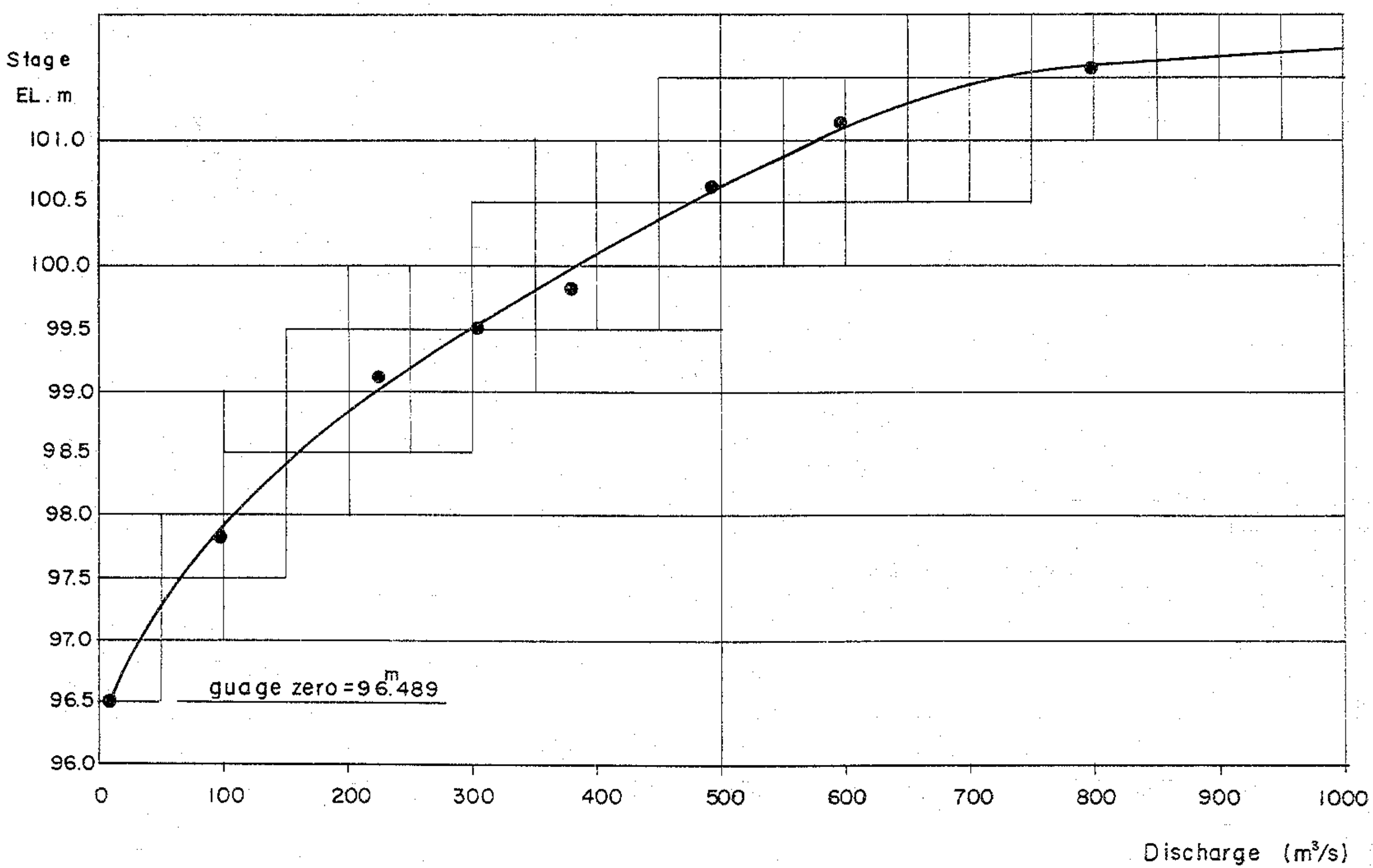


Fig.232(c) Rating curve at Jarum site in K. Denkeng

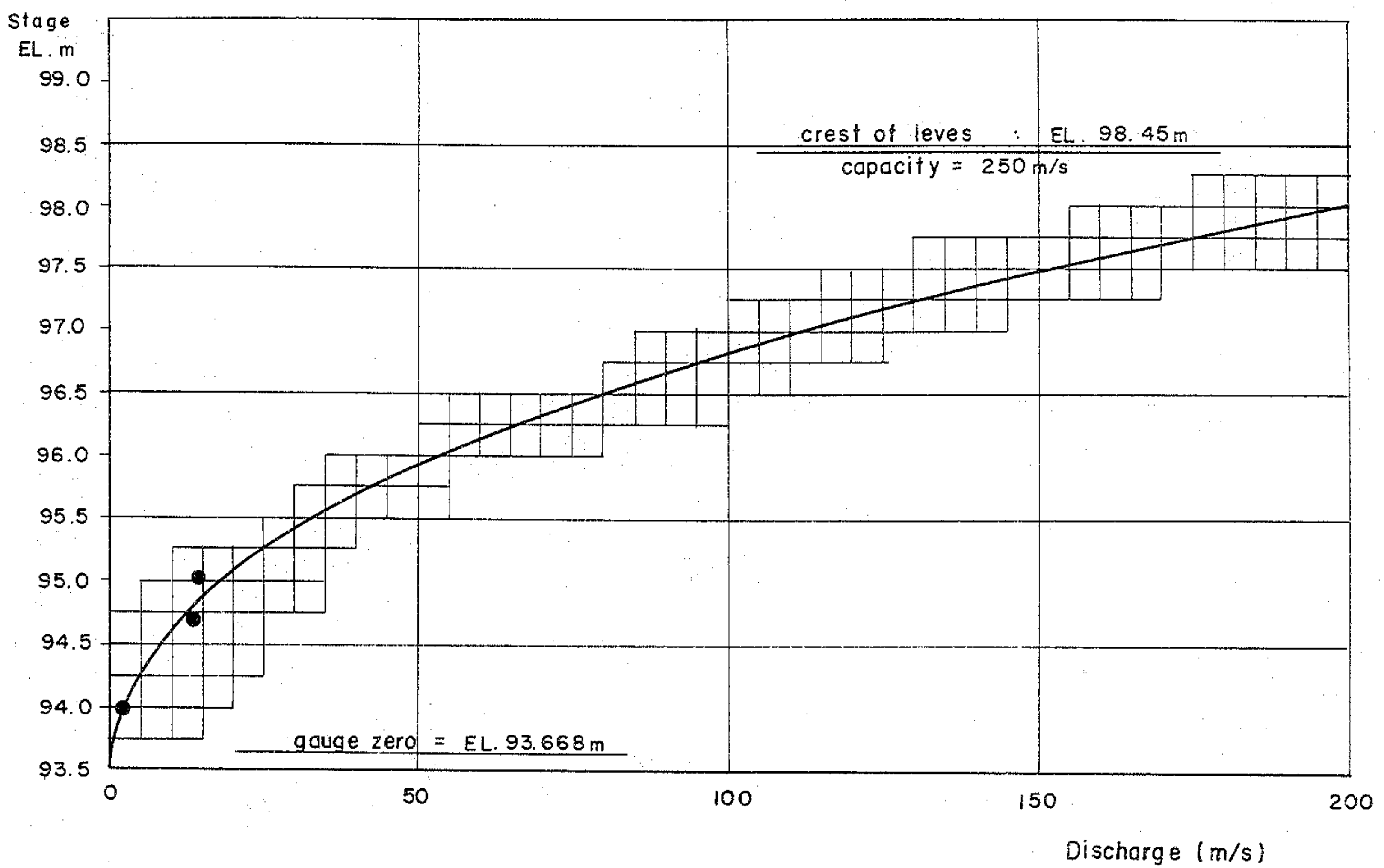


Fig 2.3(d) Rating Curve at Bacam Bridge site

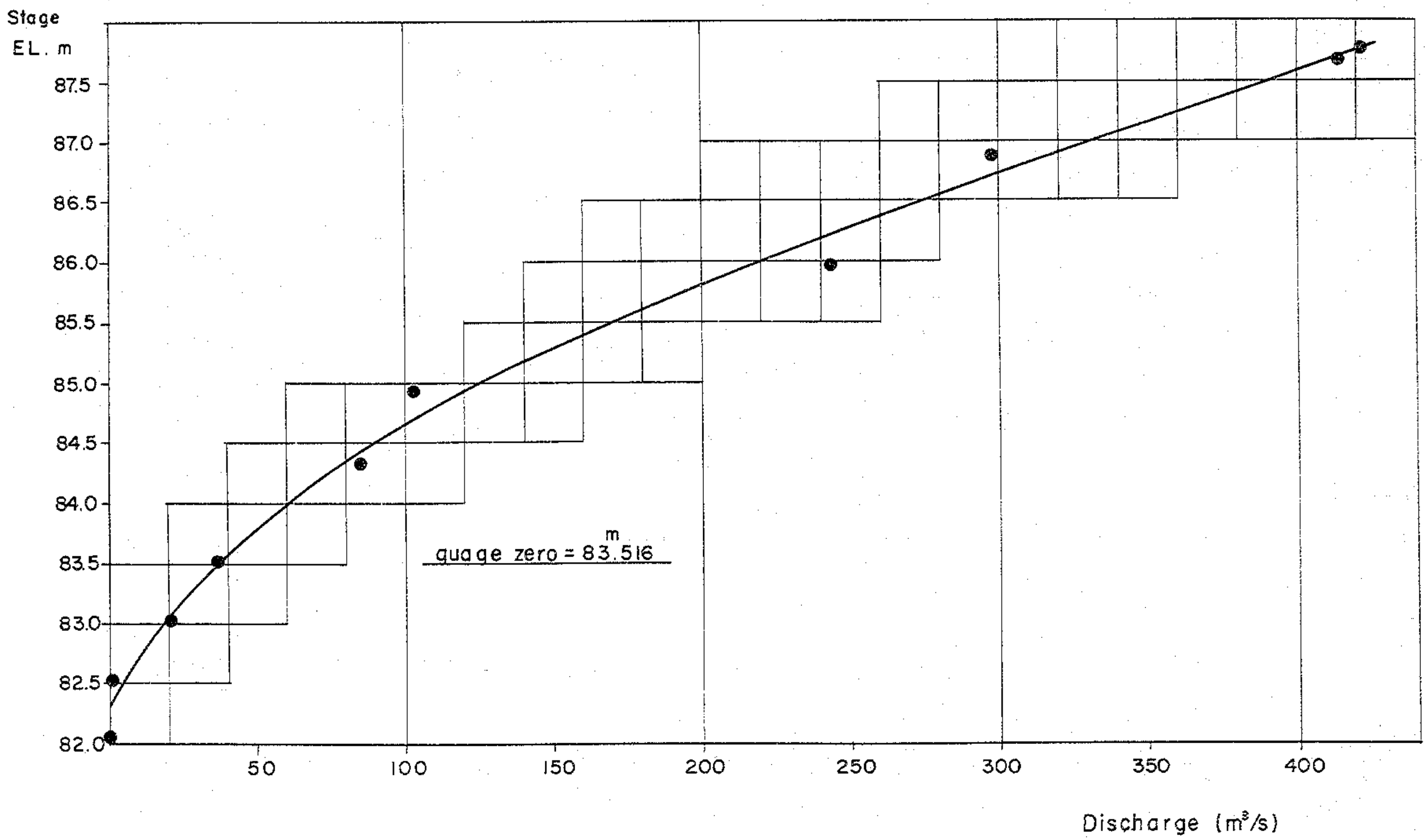


Fig.2.3.2(e) Rating Curve in K.Samin

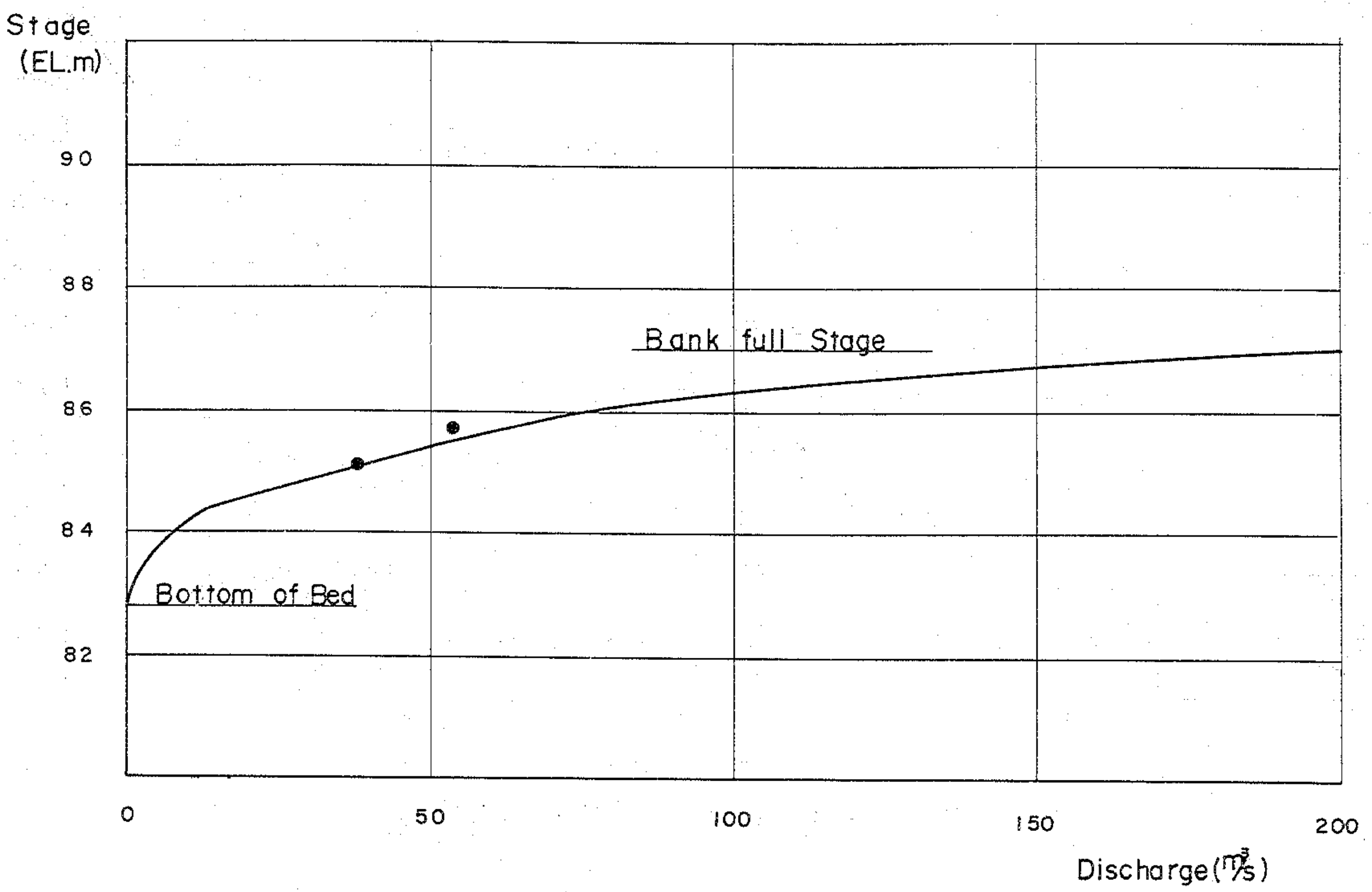


Fig. 2.32(f) Rating Curve at Jurug Gauge

Zero point = 78.309 m S.H.V.P

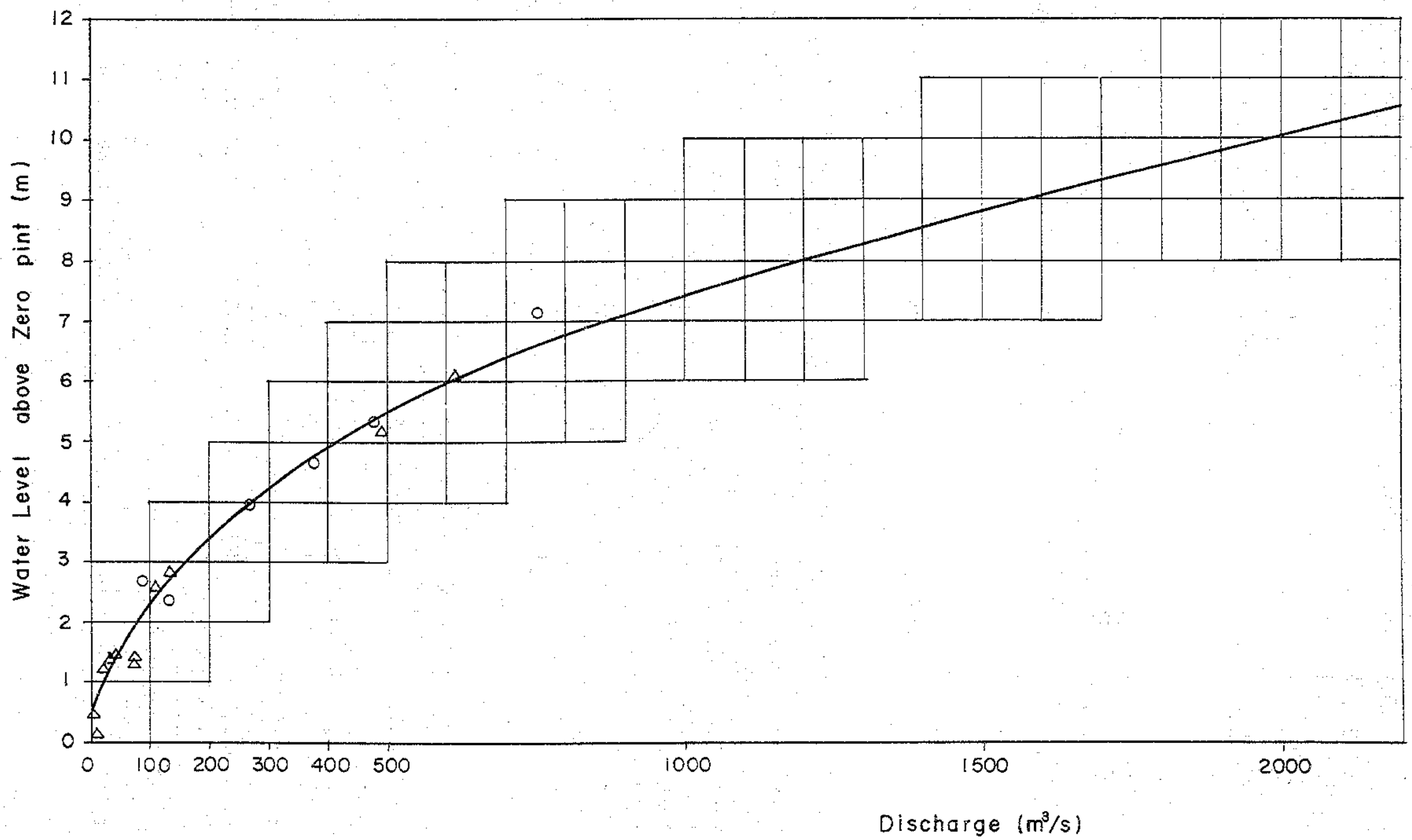
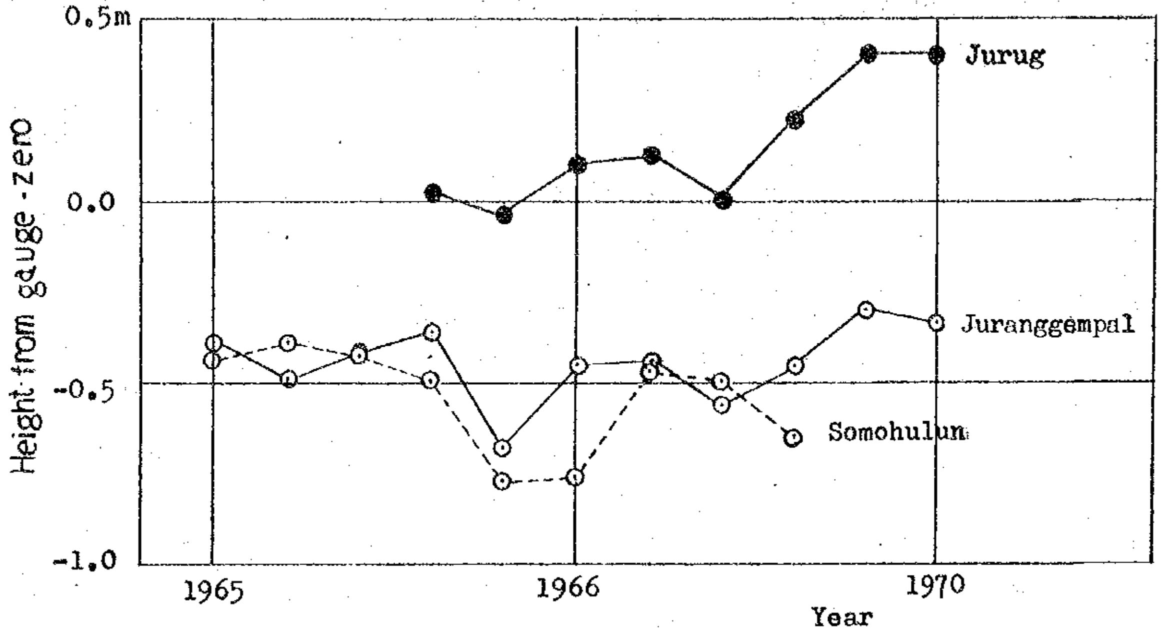


Fig. 2.3.3 Annual tendancy of the lowest water stage



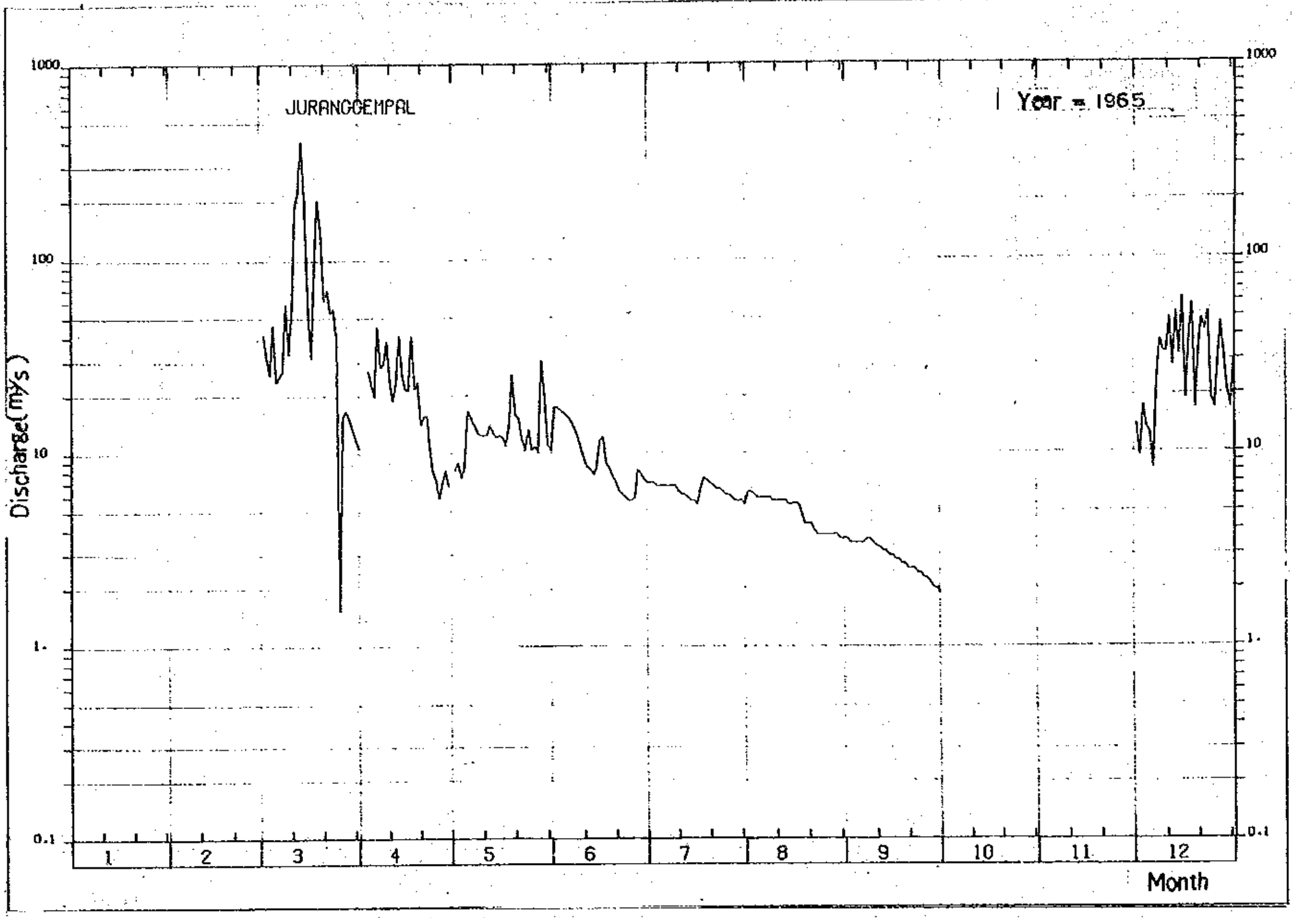
The lowest-water stage in Jun. ,in Aug. and in Sep. (raw data)

Year	Somohulun Jul.	Somohulun Aug.	Somohulun Sep.	Juraggempal Jul.	Juraggempal Aug.	Juraggempal Sep.	Jurug Jul.	Jurug Aug.	Jurug Sep.	Jarum Jul.	Jarum Aug.	Jarum Sep.
1965	0.35	0.20	0.07	0.33	0.23	0.11	----	----	----	----	----	----
1966	0.18	0.12	0.11	0.05	0.01	0.01	----	----	----	----	----	----
1967	0.20	0.14	0.07	0.20	0.14	0.07	----	----	----	----	----	----
1968	0.07	0.00	0.00	0.37	-0.36	-0.35	0.70	0.20	0.02	----	----	----
1969	-0.72	-0.76	-0.77	-0.60	-0.65	-0.68	0.21	0.29	-0.04	----	----	----
1970	-0.76	-0.39	-0.40	-0.36	-0.45	-0.43	0.15	0.10	0.10	----	----	----
1971	-0.38	-0.45	-0.47	-0.18	-0.32	-0.43	0.14	0.18	0.13	----	----	----
1972	----	-0.39	-0.50	-0.49	-0.56	0.44	0.18	0.12	0.02	----	----	----
1973	0.37	0.33	0.35	0.95	0.65	0.55	0.48	0.35	0.22	----	----	----
1974	----	----	----	0.68	0.69	0.70	0.42	0.54	0.40	0.47	0.47	0.54
1975	----	----	----	0.81	0.66	----	0.50	0.40	0.40	----	----	----

The loweat water stage in Jun. ,in Aug. and in Sep. (corrected)

Year	Somohulun Jul.	Somohulun Aug.	Somohulun Sep.	Juranggempal Jul.	Juranggempal Aug.	Juranggempal Sep.	Jurug Jul.	Jurug Aug.	Jurug Sep.	Jarum Jul.	Jarum Aug.	Jarum Sep.
1965	-0.15	-0.30	-0.43	-0.17	-0.27	-0.39	----	----	----	----	----	----
1966	-0.32	-0.38	-0.39	-0.45	-0.49	-0.49	----	----	----	----	----	----
1967	-0.30	-0.36	-0.43	-0.30	-0.36	-0.43	----	----	----	----	----	----
1968	-0.43	-0.50	-0.50	-0.37	-0.36	-0.35	0.70	0.20	0.02	----	----	----
1969	-0.72	-0.76	-0.77	-0.60	-0.65	-0.68	0.21	0.29	-0.04	----	----	----
1970	-0.76	-0.39	-0.40	-0.36	-0.45	-0.43	0.15	0.10	0.10	----	----	----
1971	-0.38	-0.45	-0.47	-0.18	-0.32	-0.43	0.14	0.18	0.13	----	----	----
1972	----	-0.39	-0.50	-0.49	-0.56	-0.56	0.18	0.12	0.02	----	----	----
1973	-0.63	-0.67	-0.65	-0.05	-0.35	-0.45	0.48	0.35	0.22	----	----	----
1974	----	----	----	-0.31	-0.31	-0.30	0.42	0.54	0.40	0.47	0.47	0.54
1975	----	----	----	-0.19	-0.34	----	0.50	0.40	0.40	----	----	----

Fig. 2.3.4 Daily mean discharge



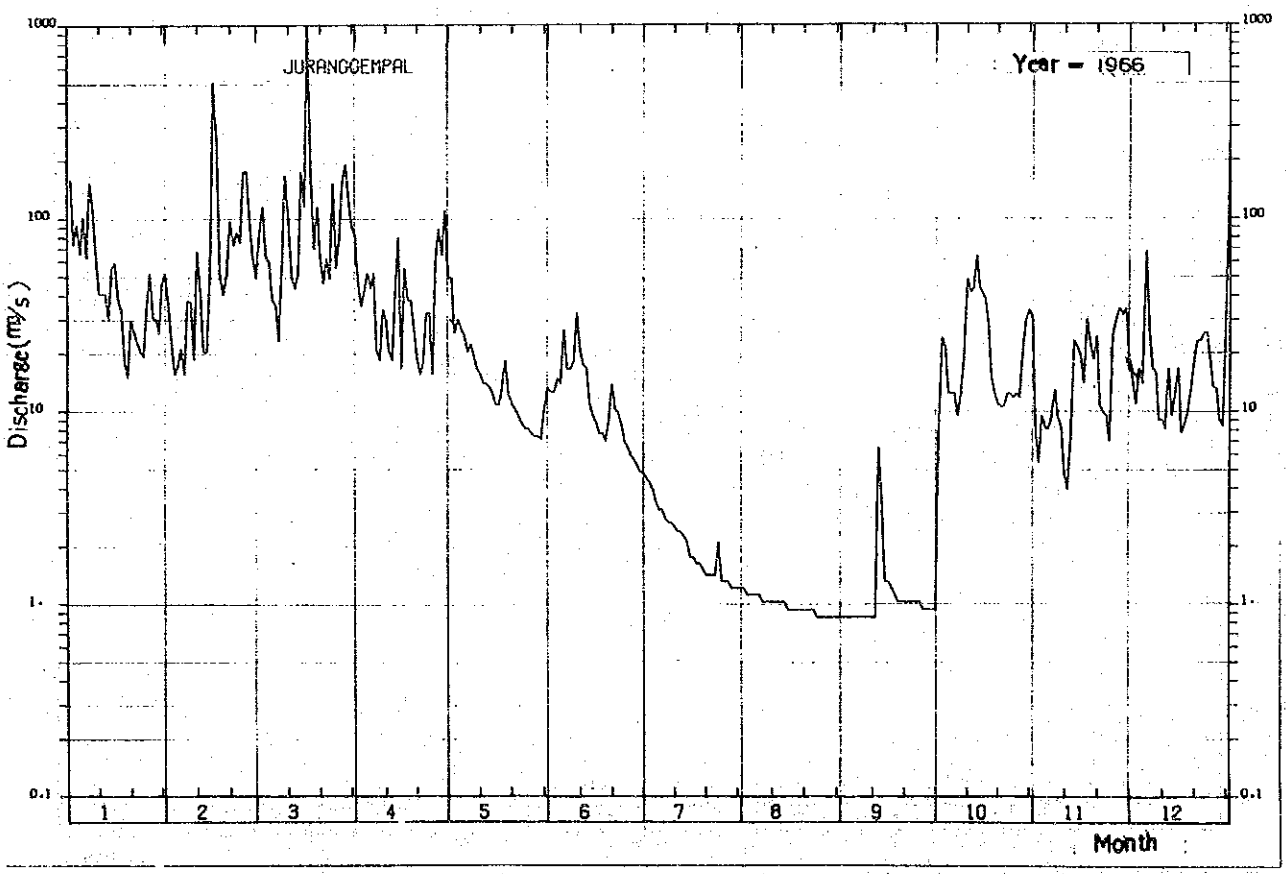
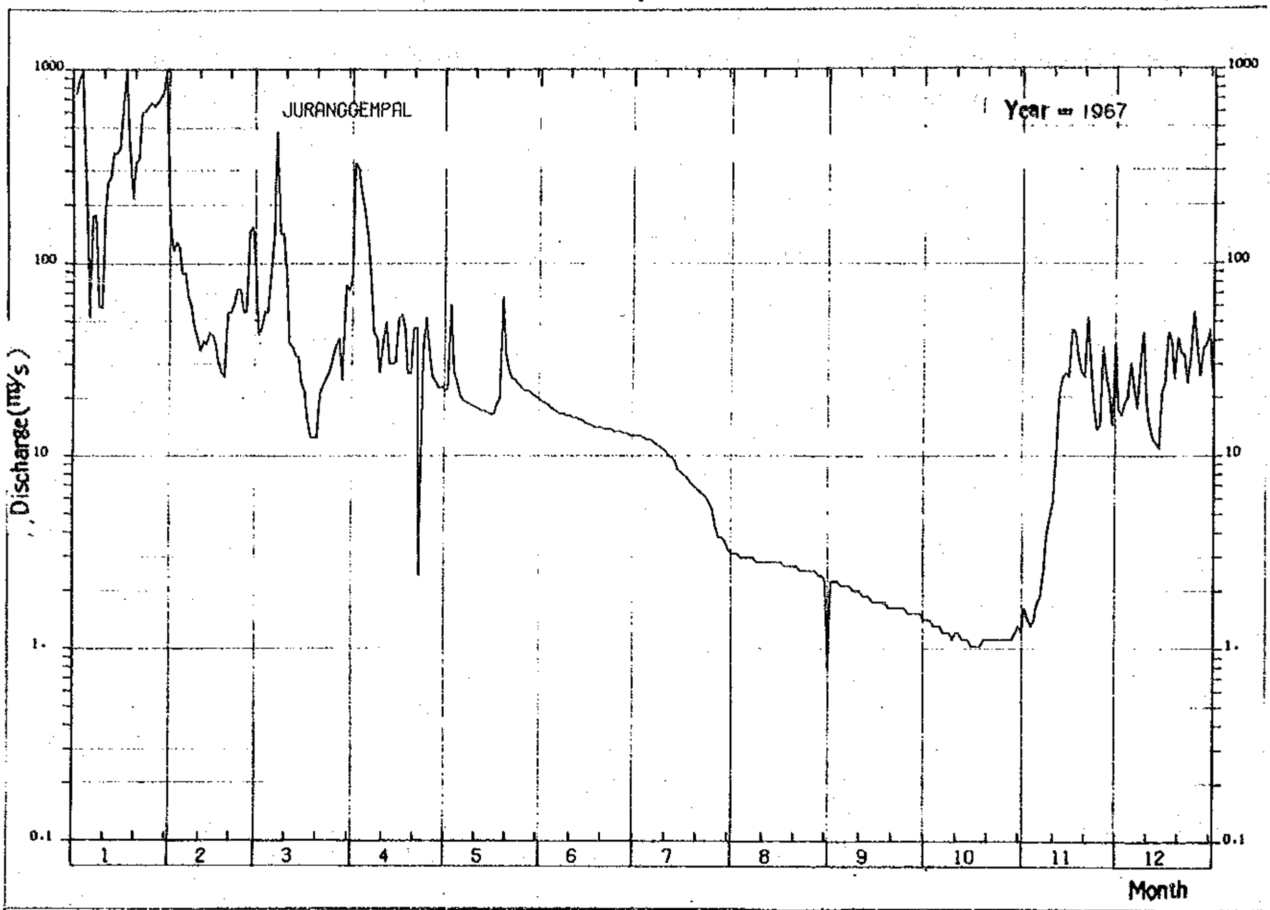


Fig. 2.3.4 Daily mean discharge



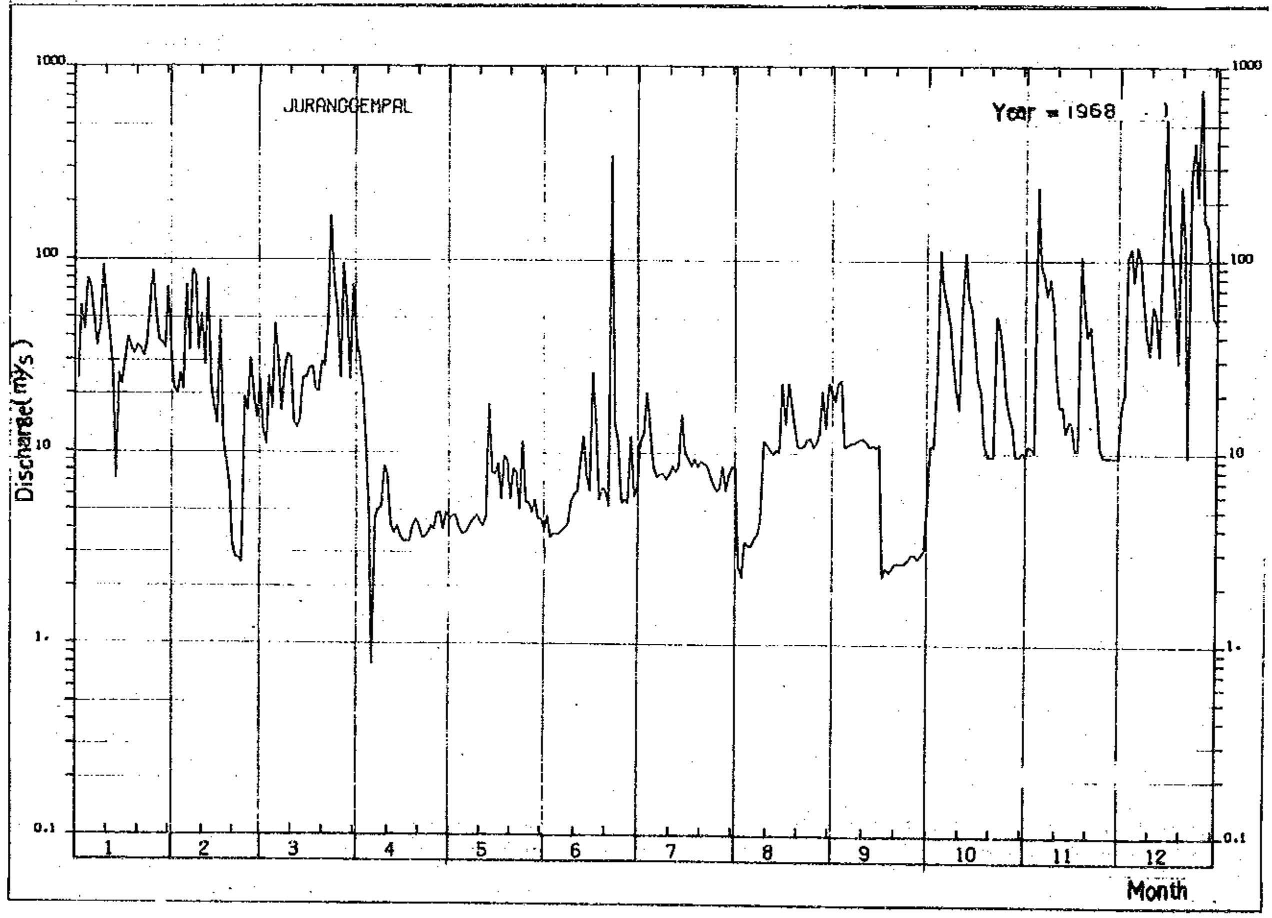
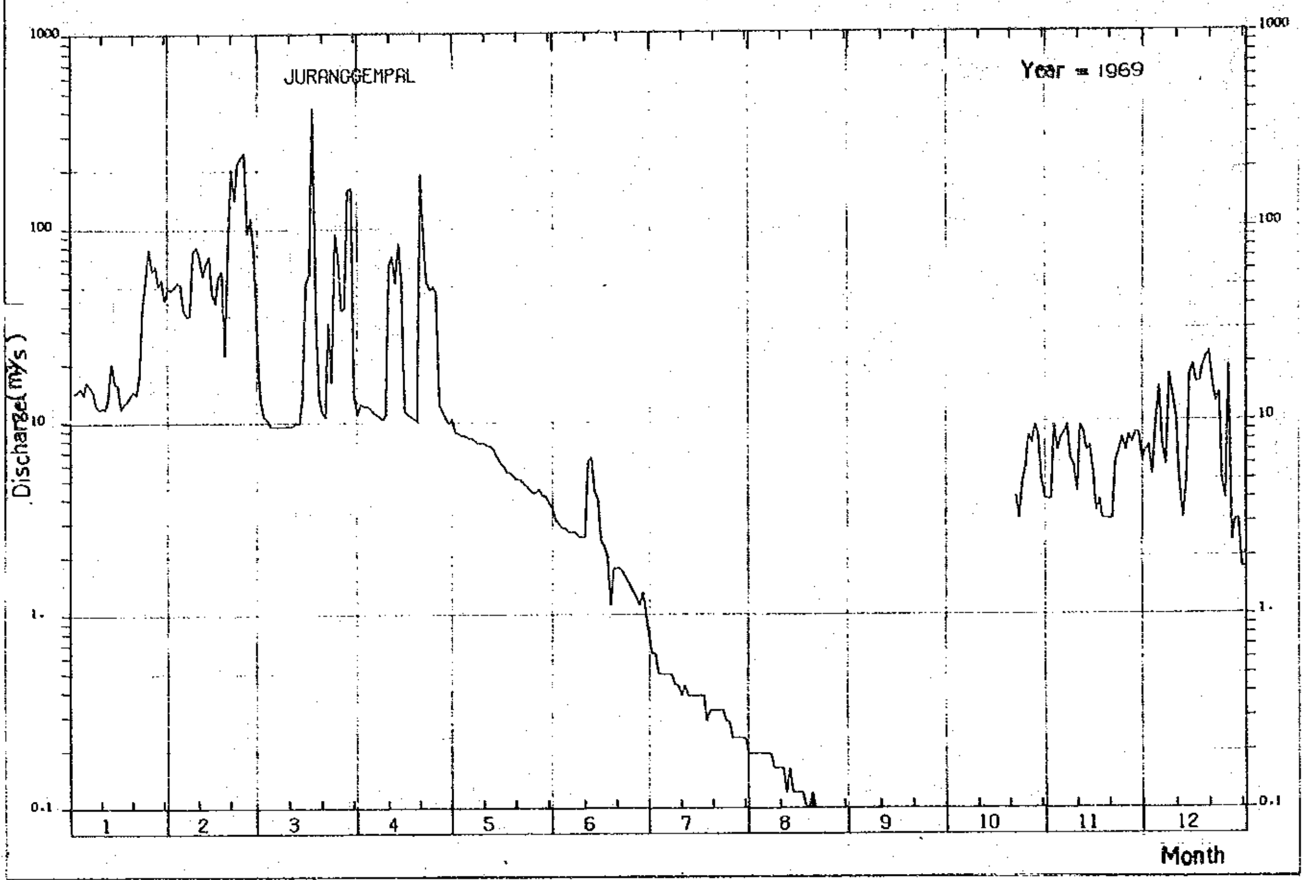


Fig. 2.3.4 Daily mean discharge



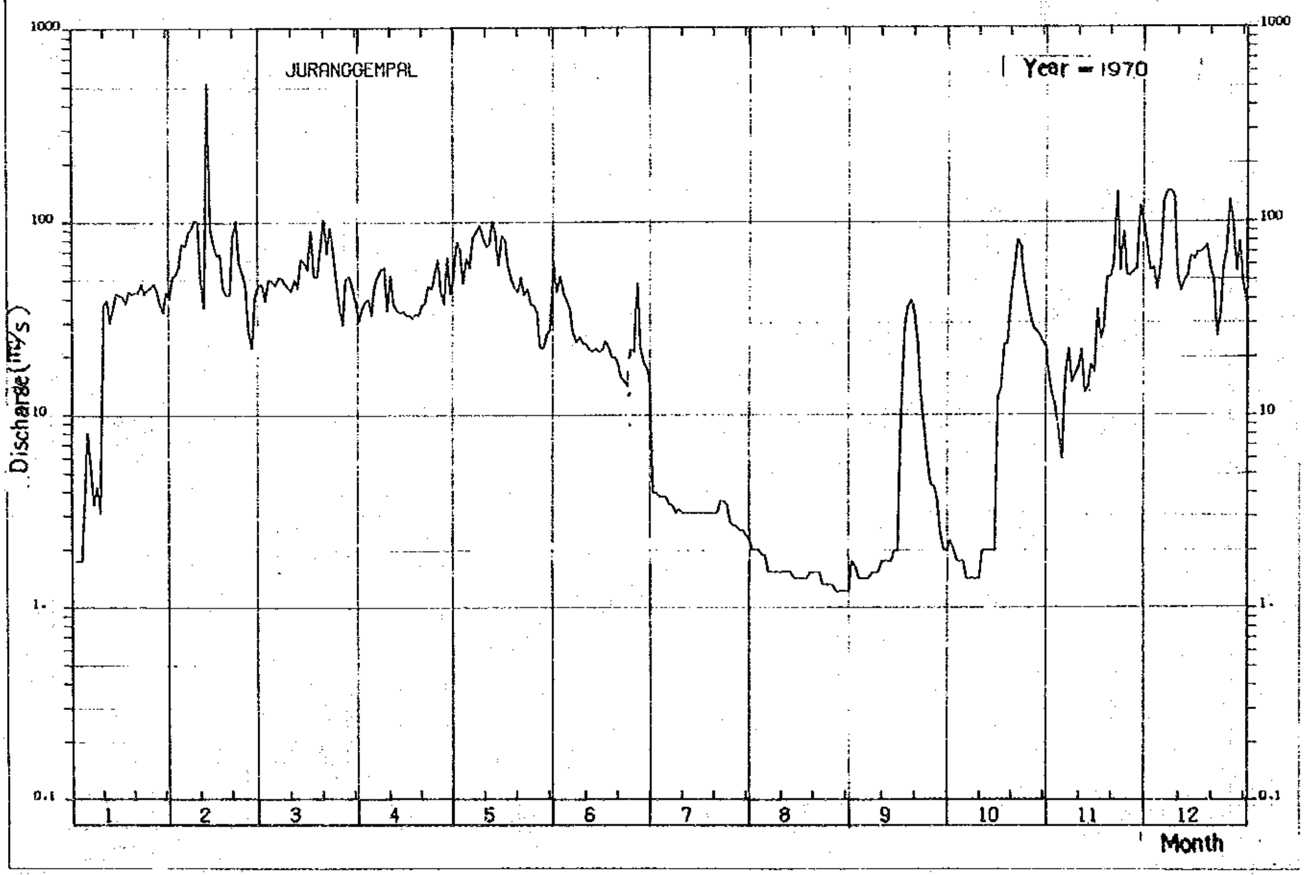
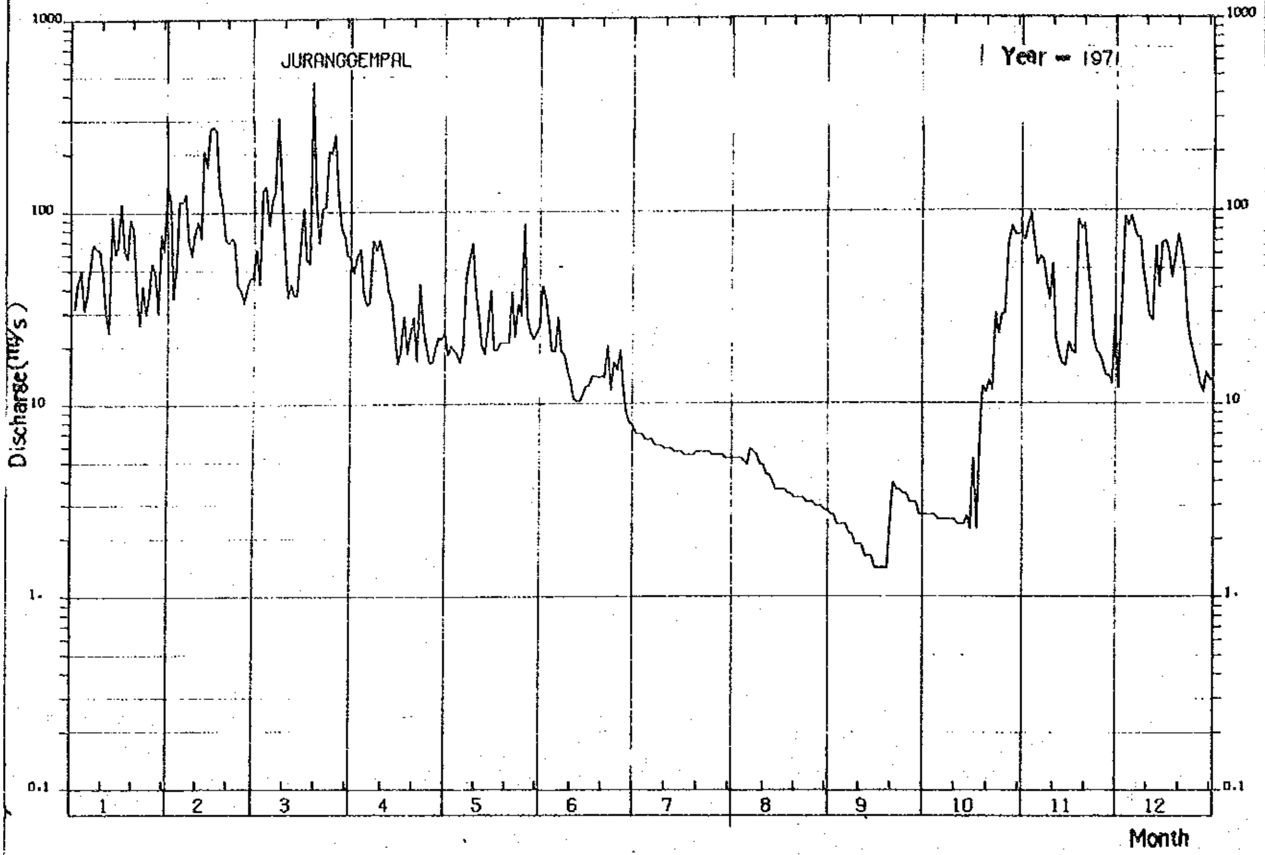


Fig. 2.3.4 Daily mean discharge



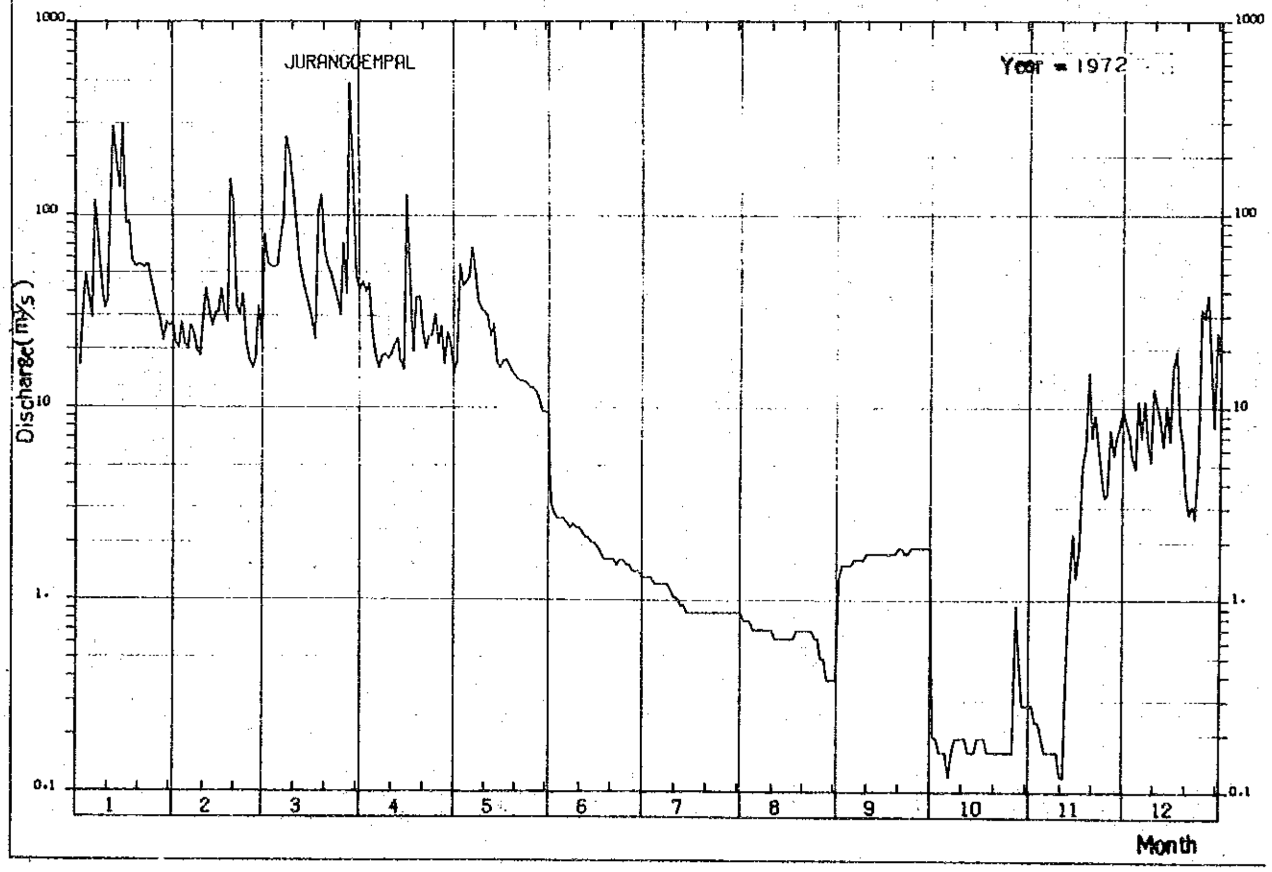
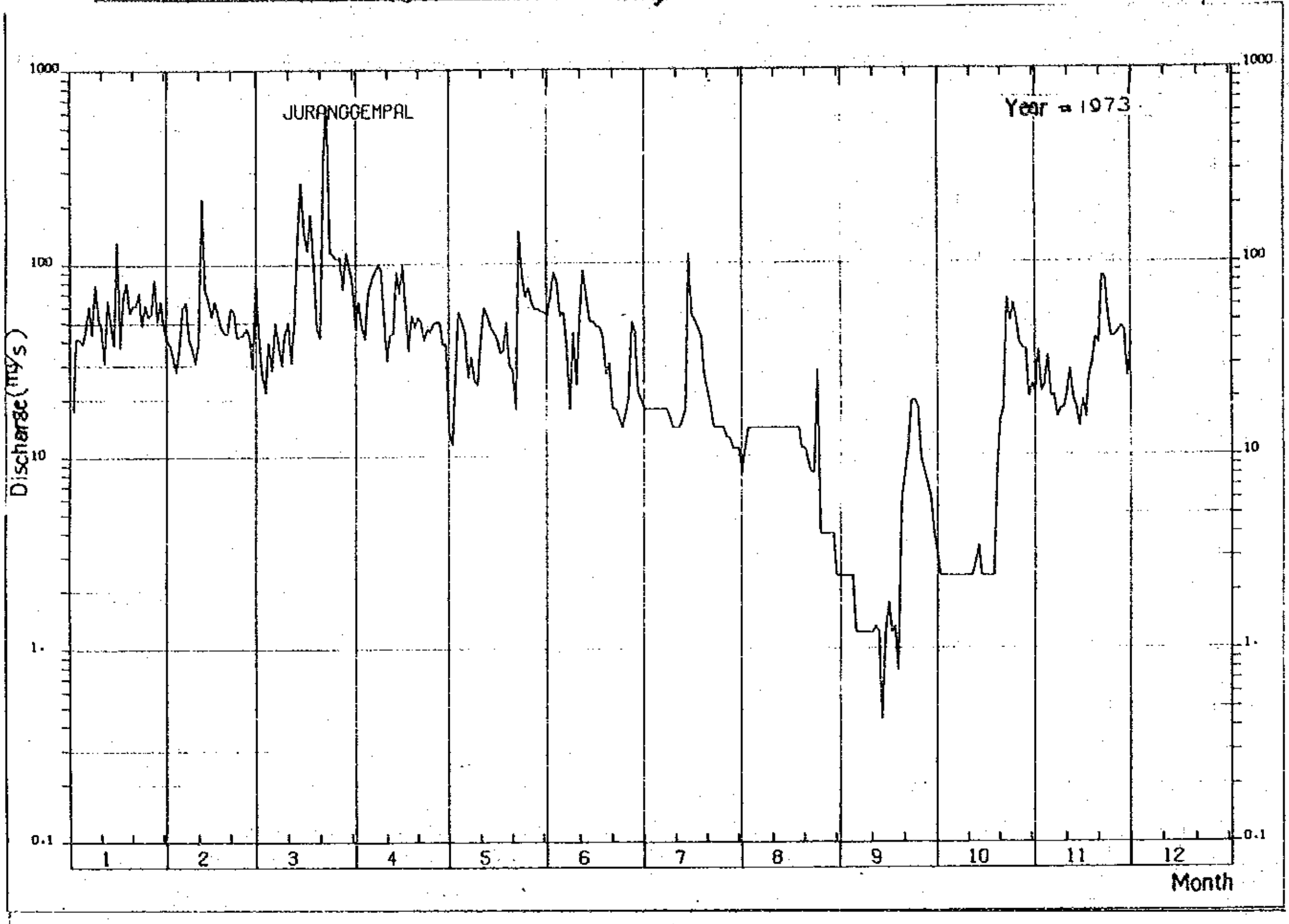


Fig. 2.3.4 Daily mean discharge



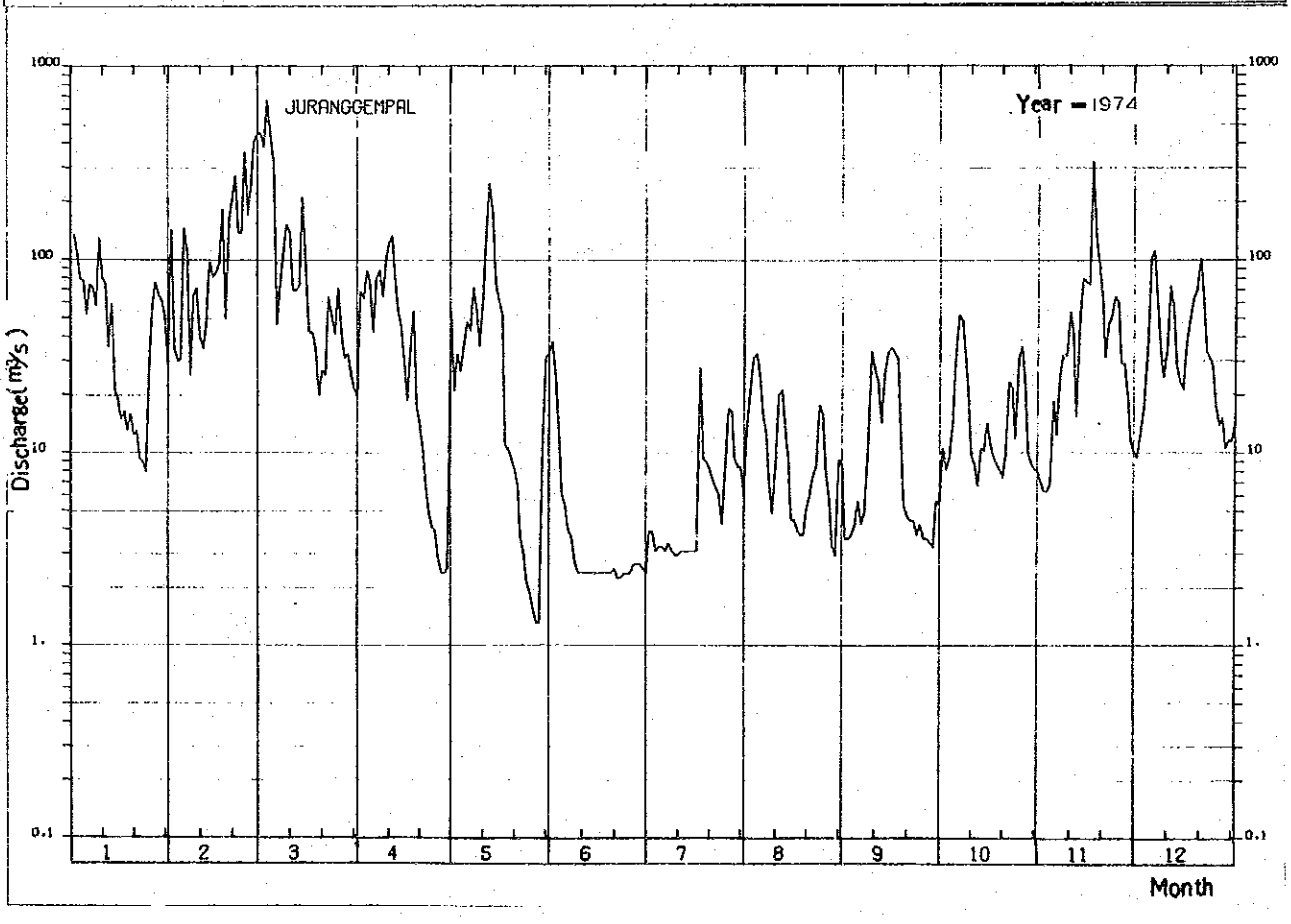


Fig. 2.3.4 Daily mean discharge

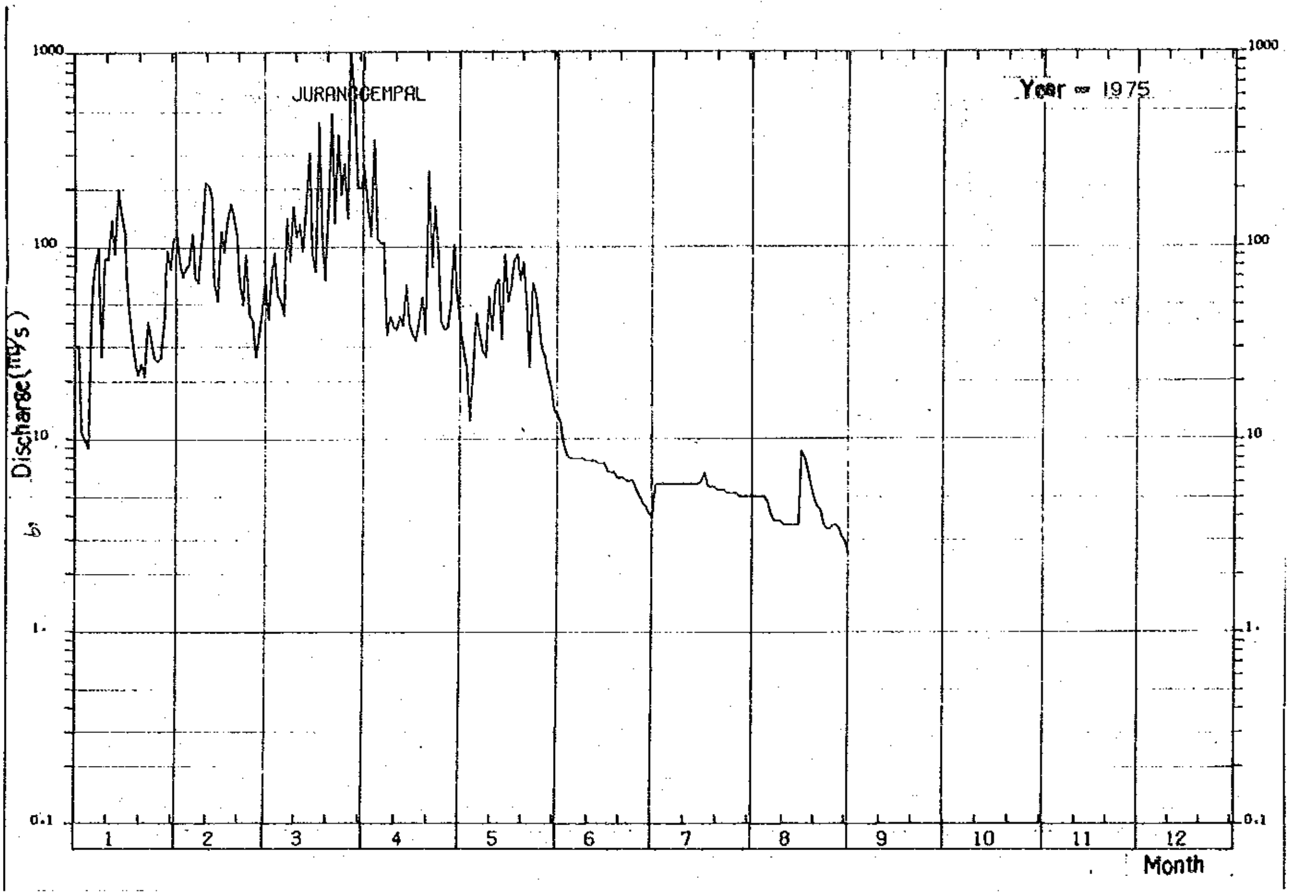


Fig. 2.3.5 Daily mean discharge

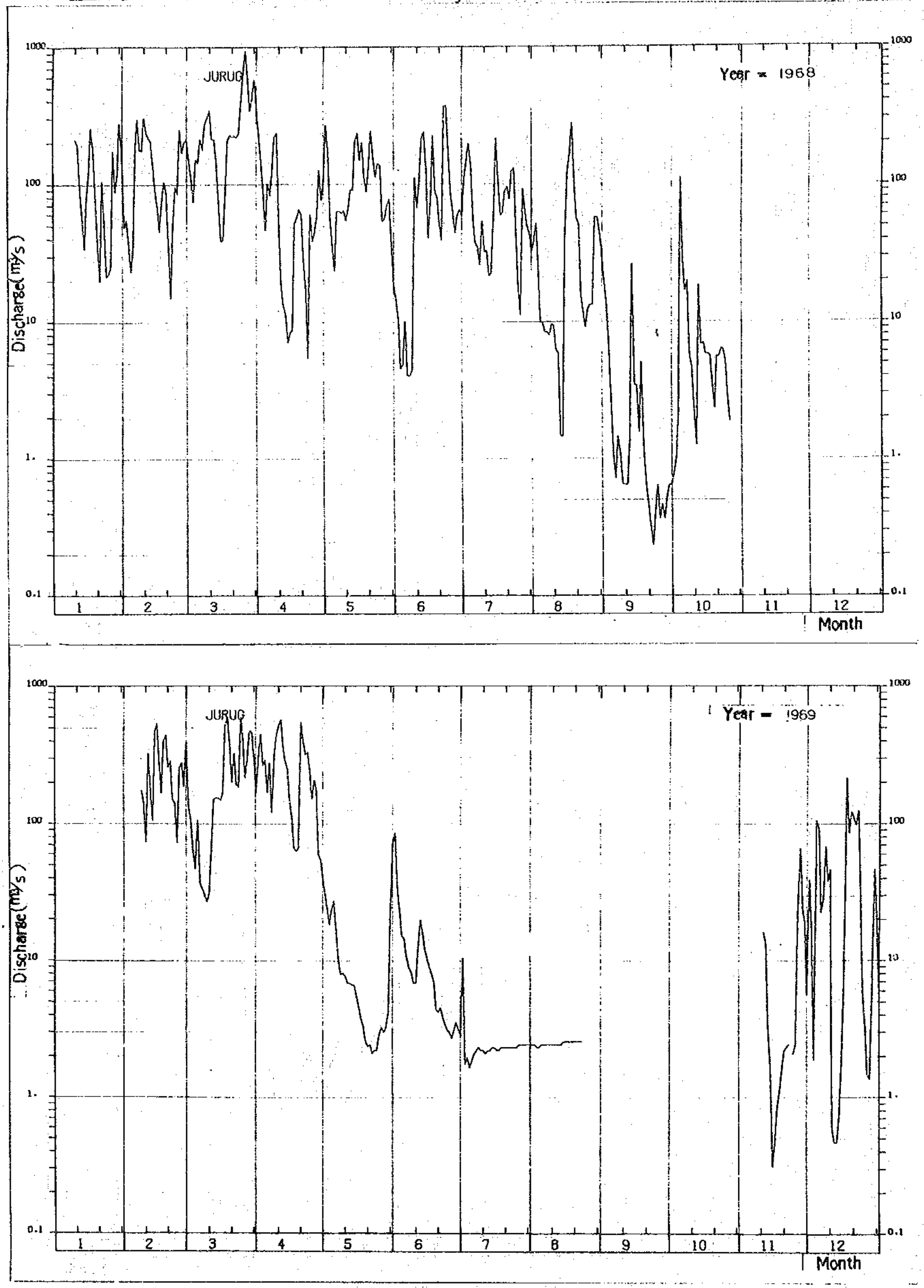
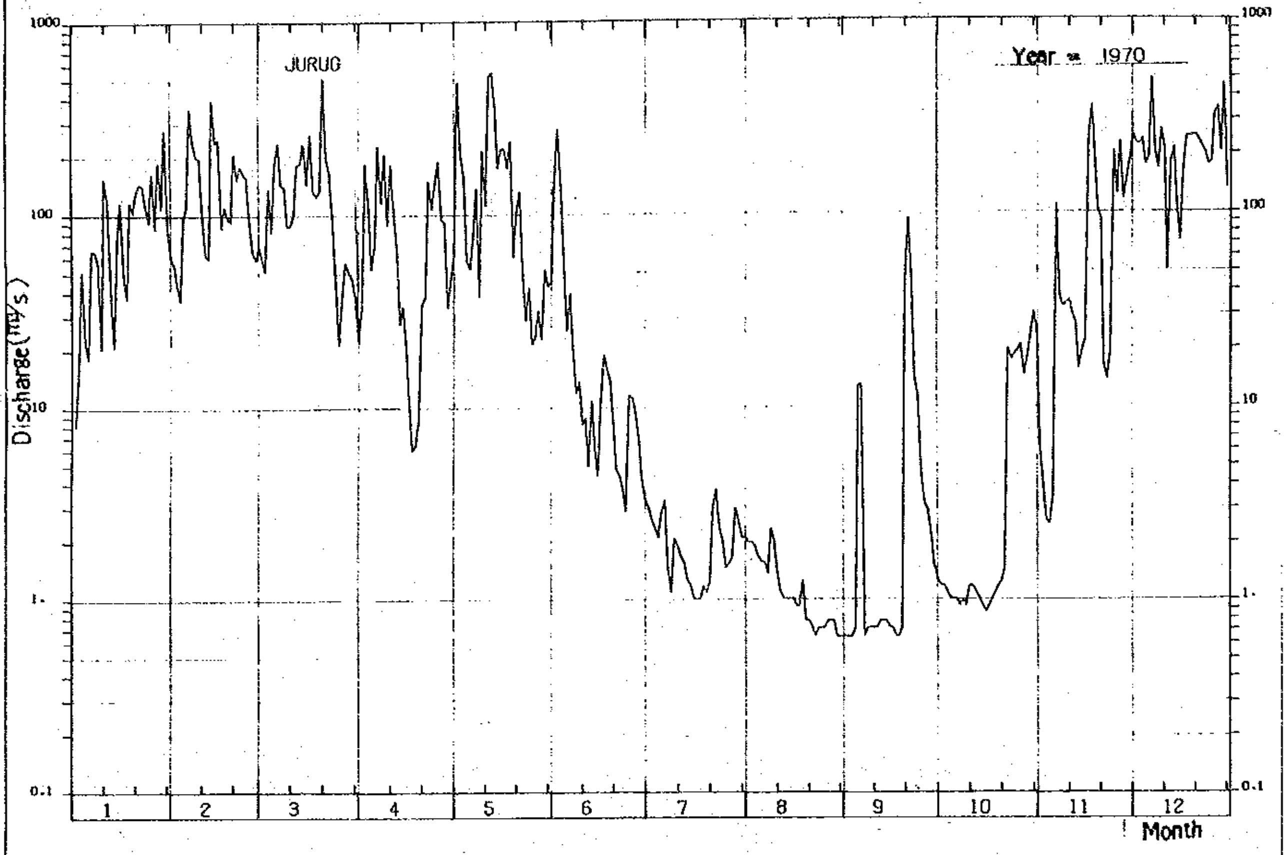


Fig. 2.3.5 Daily mean discharge



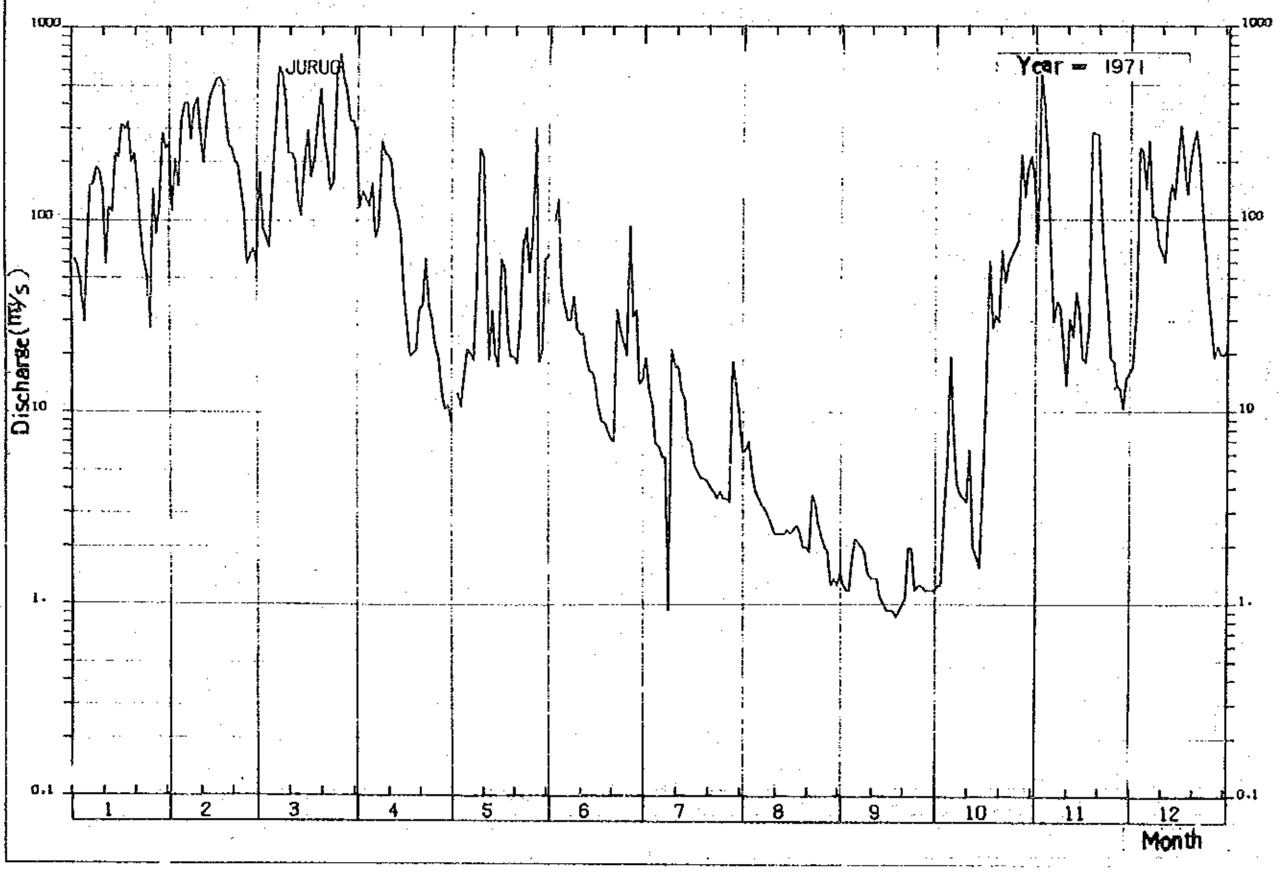
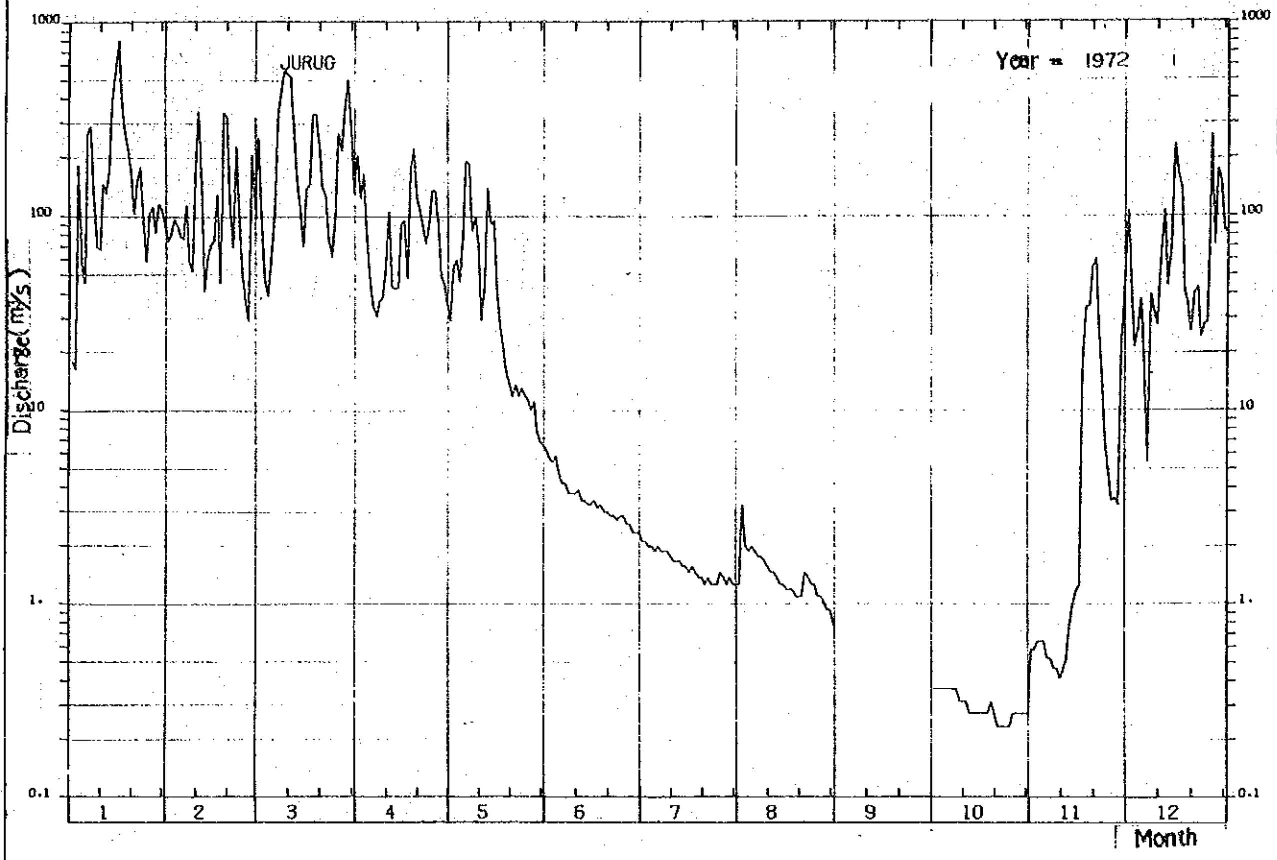


Fig. 2.3.5 Daily mean discharge



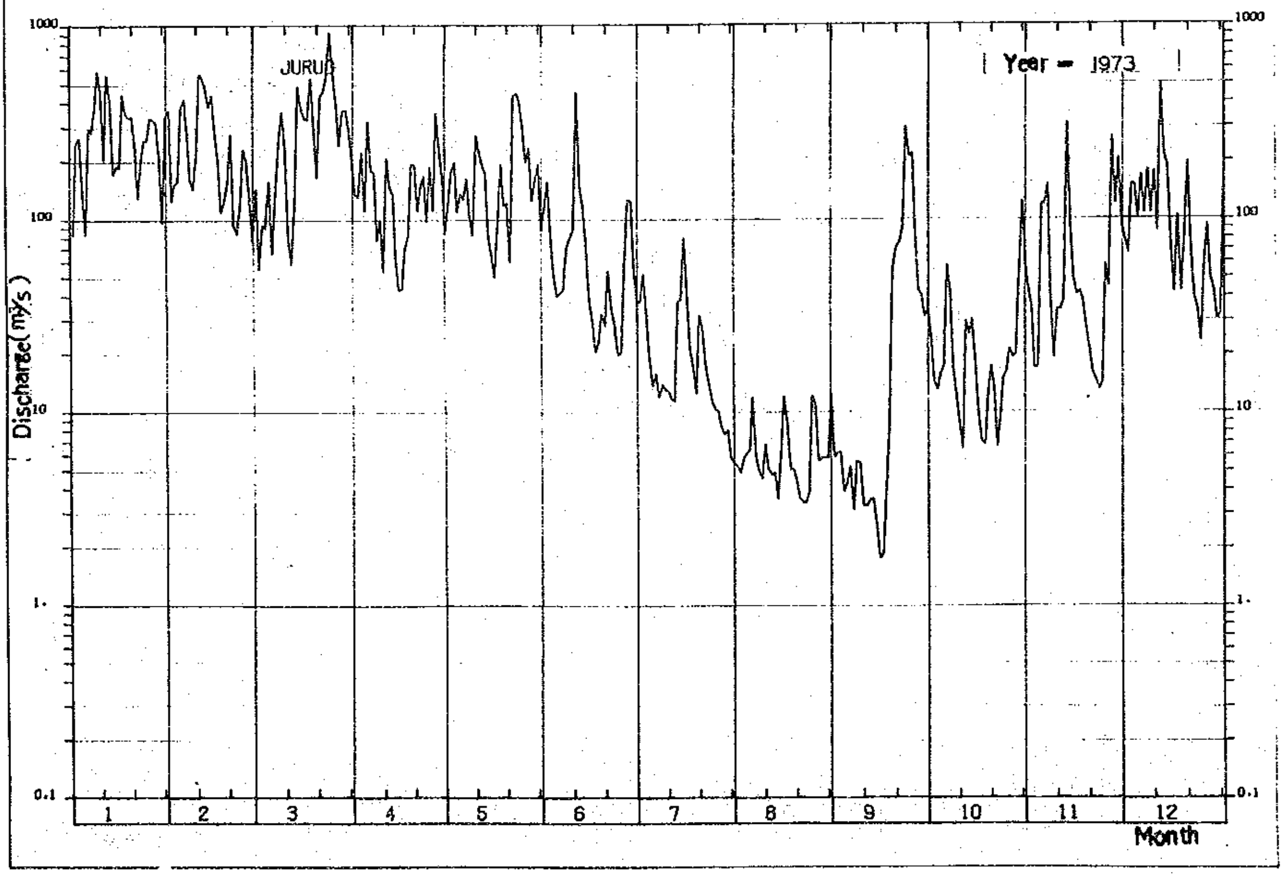
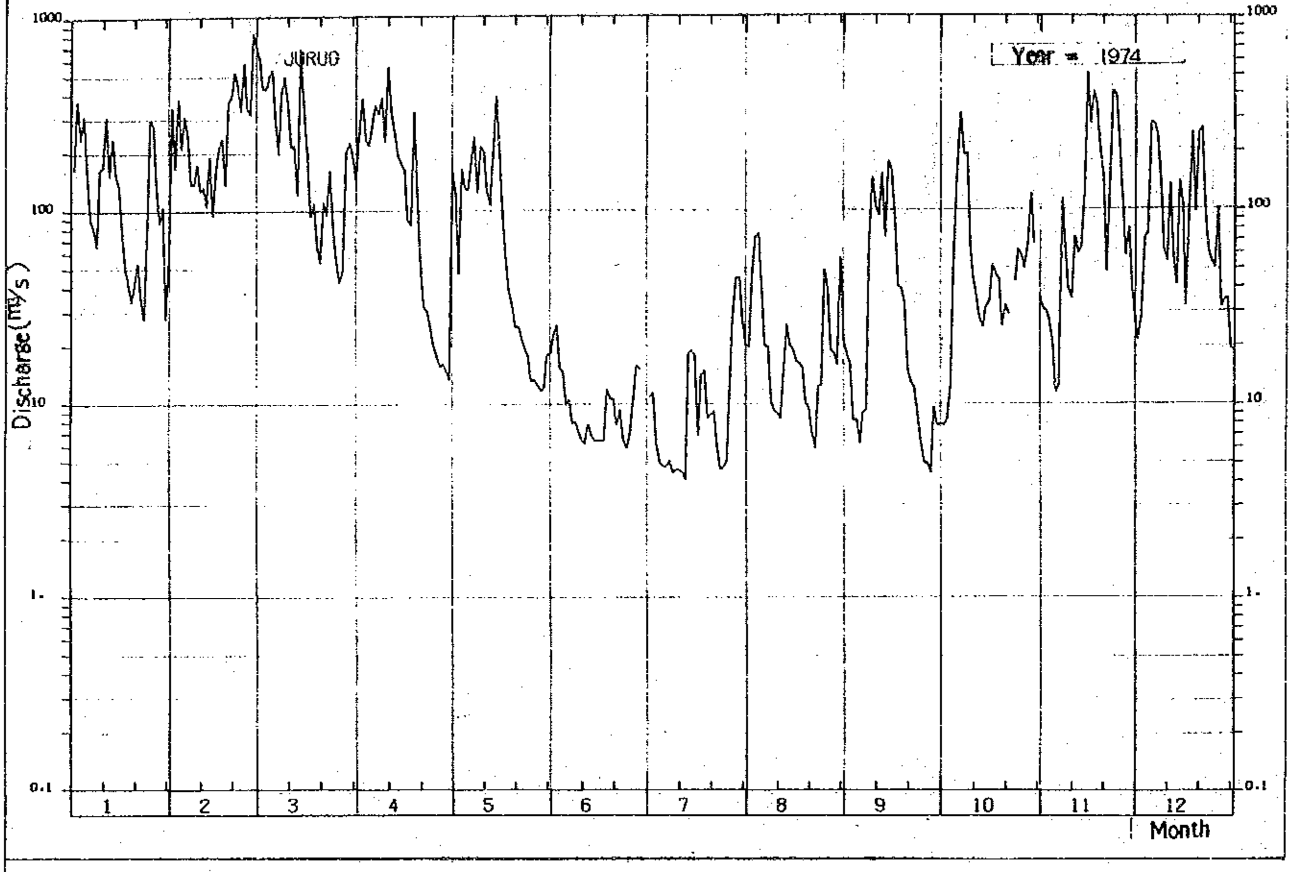


Fig. 3.3.5 Daily mean discharge



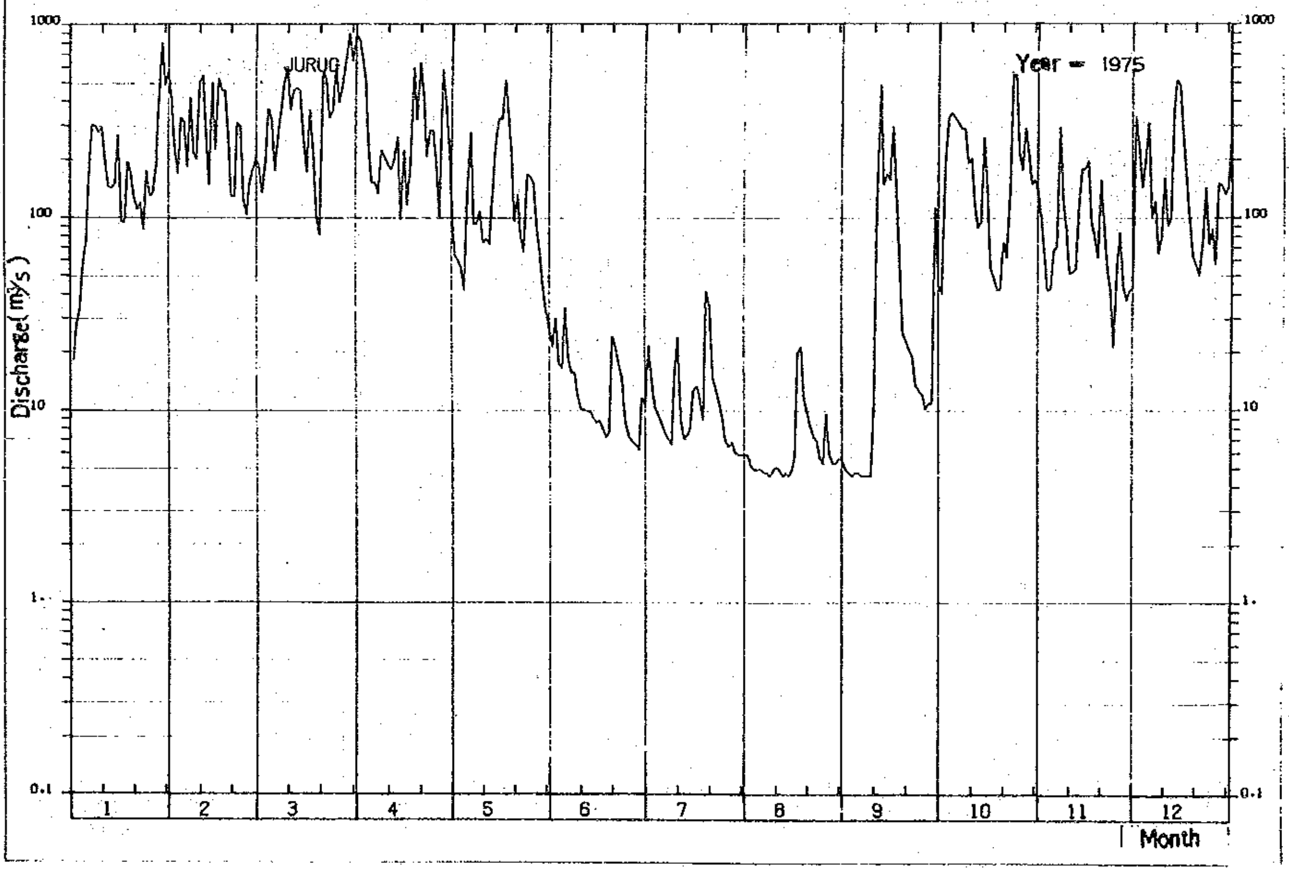
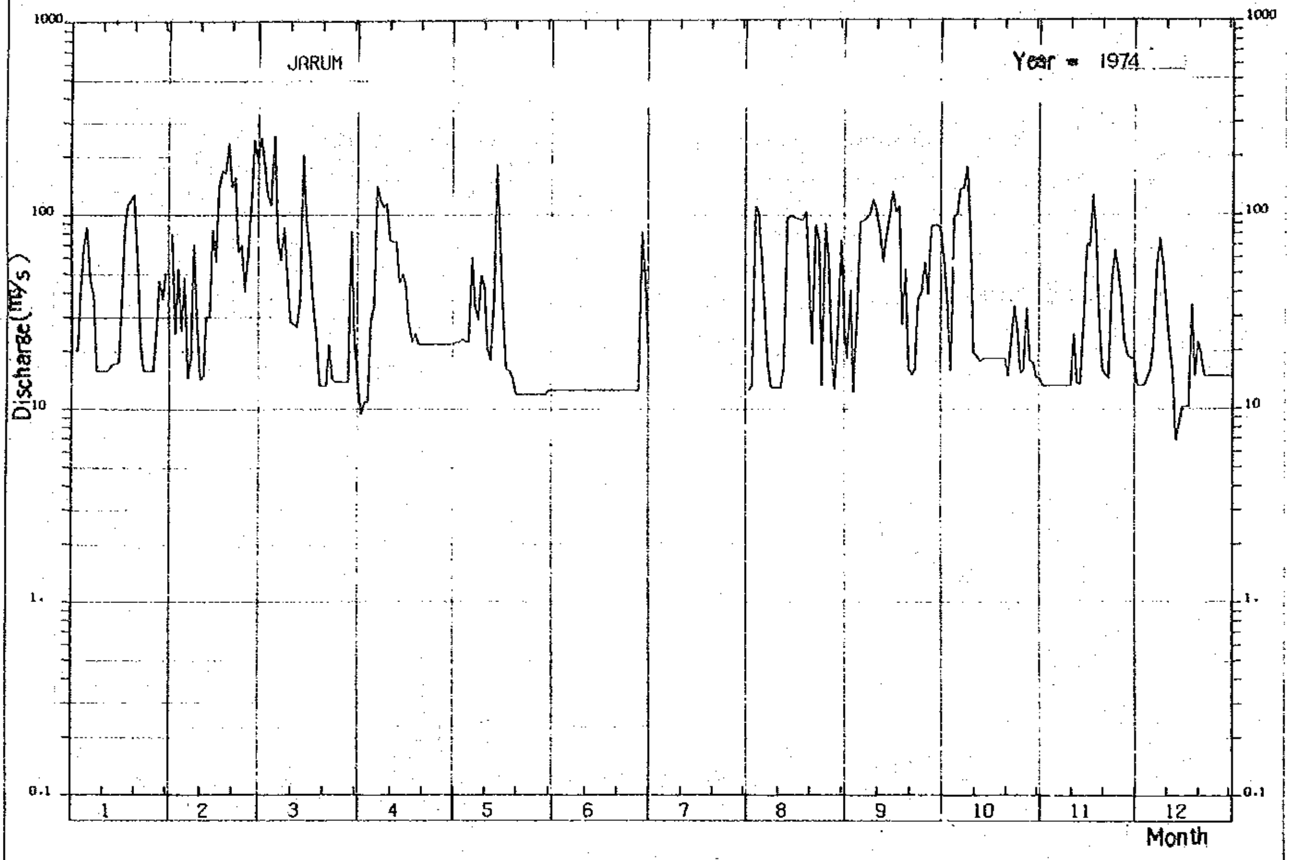


Fig. 2.3.6 Daily mean discharge



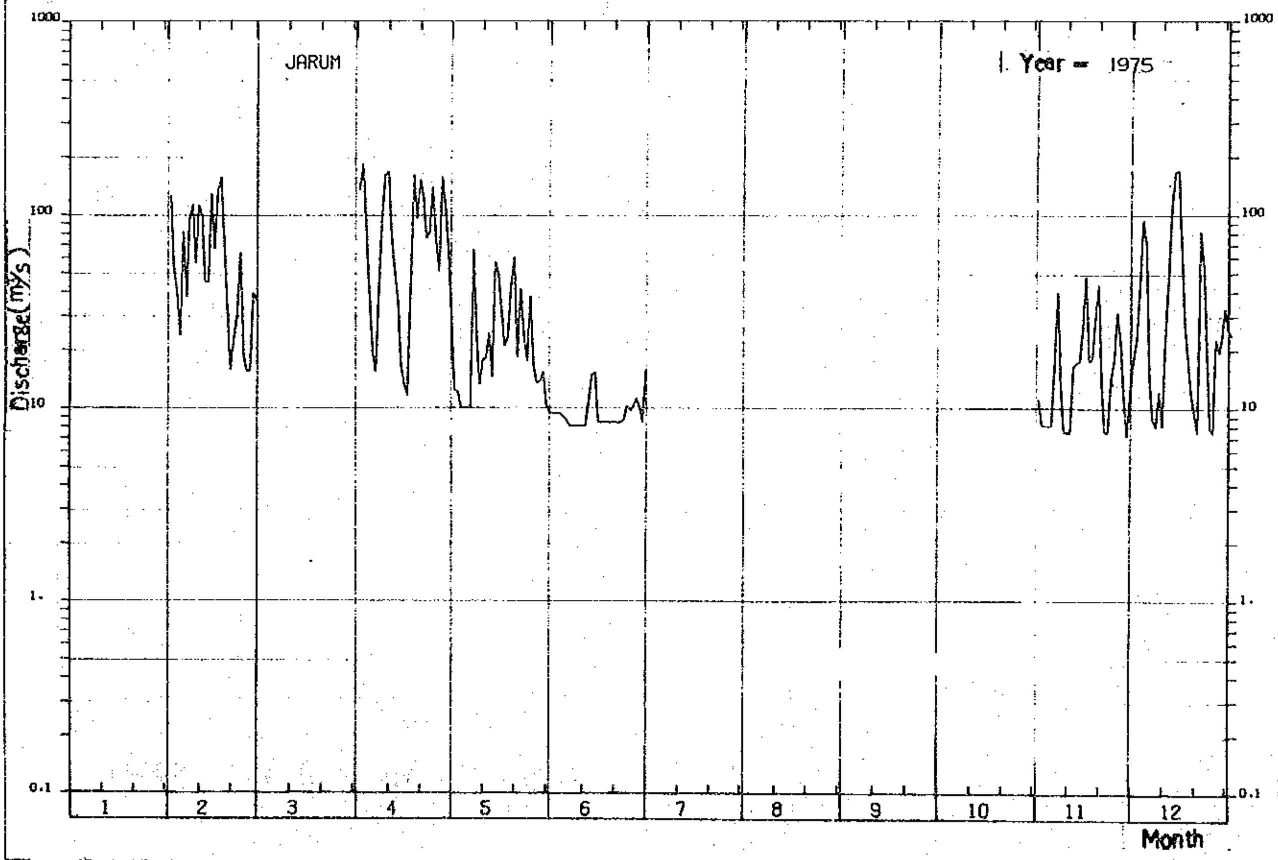


Fig. 2.3.7 Companison of New and Old Discharge Data

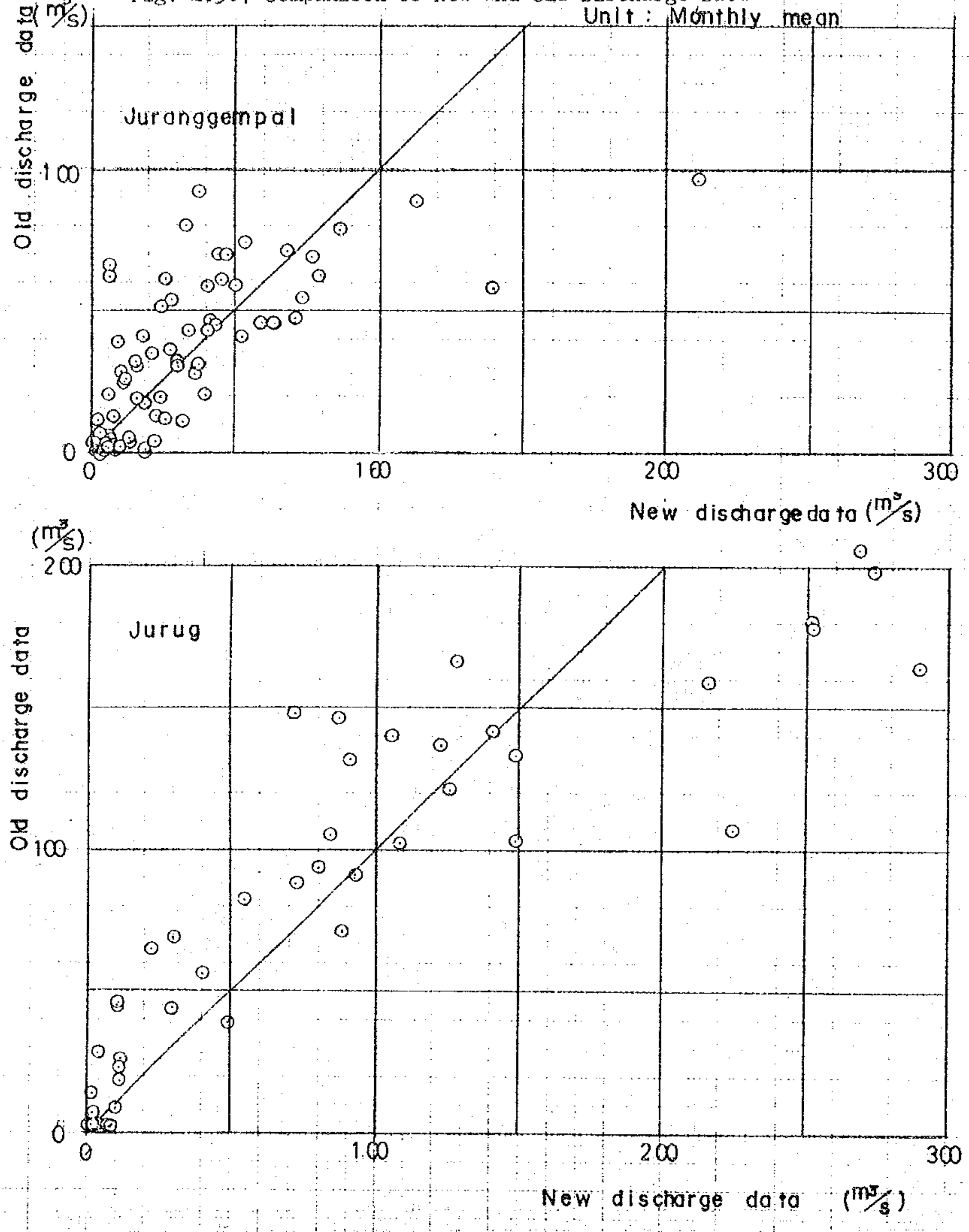
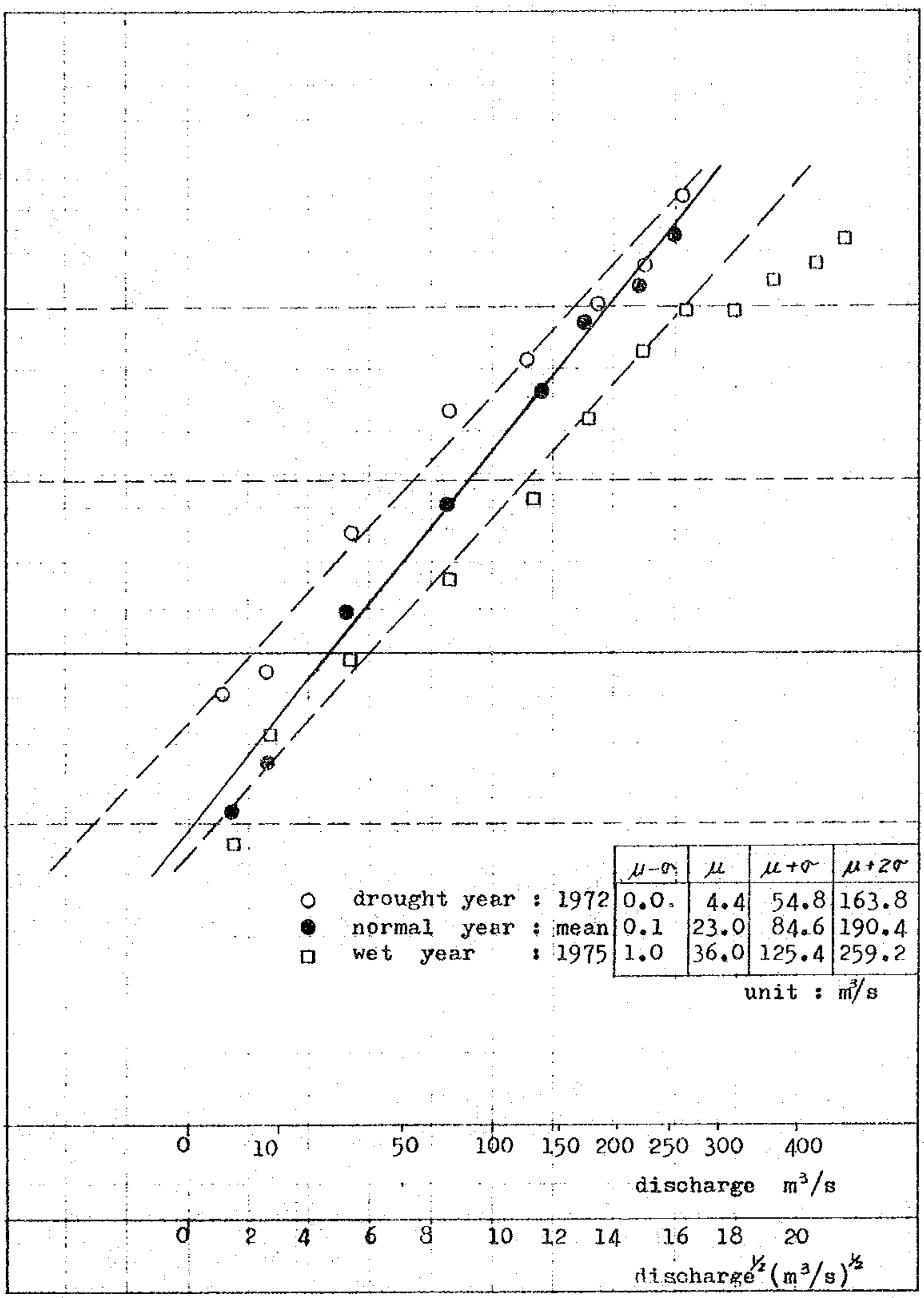


Fig.2.38 Probability distribution of daily mean discharge at Juraggempal



	$\mu-\sigma$	μ	$\mu+\sigma$	$\mu+2\sigma$
○ drought year : 1972	0.0	4.4	54.8	163.8
● normal year : mean	0.1	23.0	84.6	190.4
□ wet year : 1975	1.0	36.0	125.4	259.2

unit : m^3/s

Fig. 239 Probability distribution of daily mean discharge at Jurug

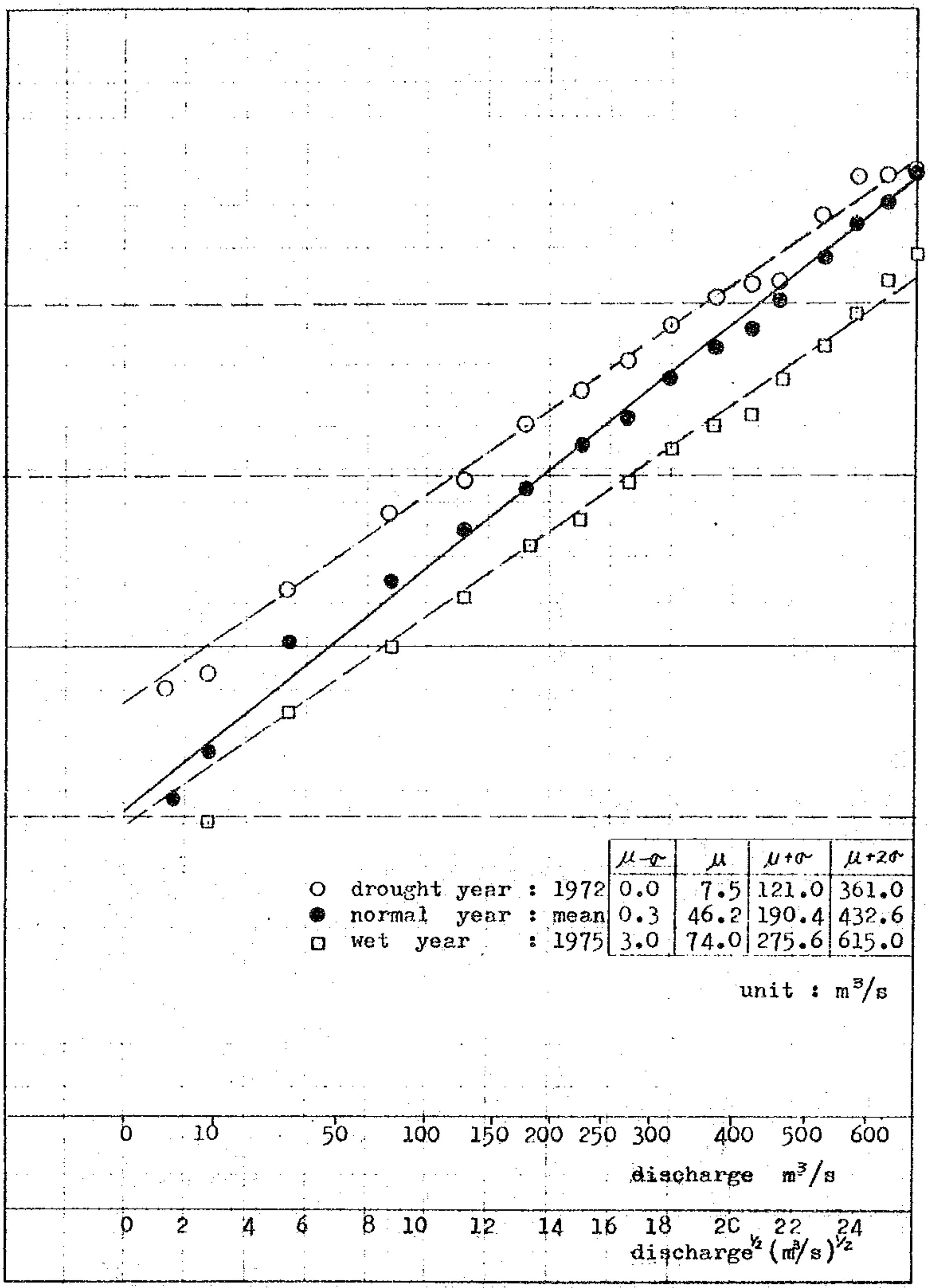
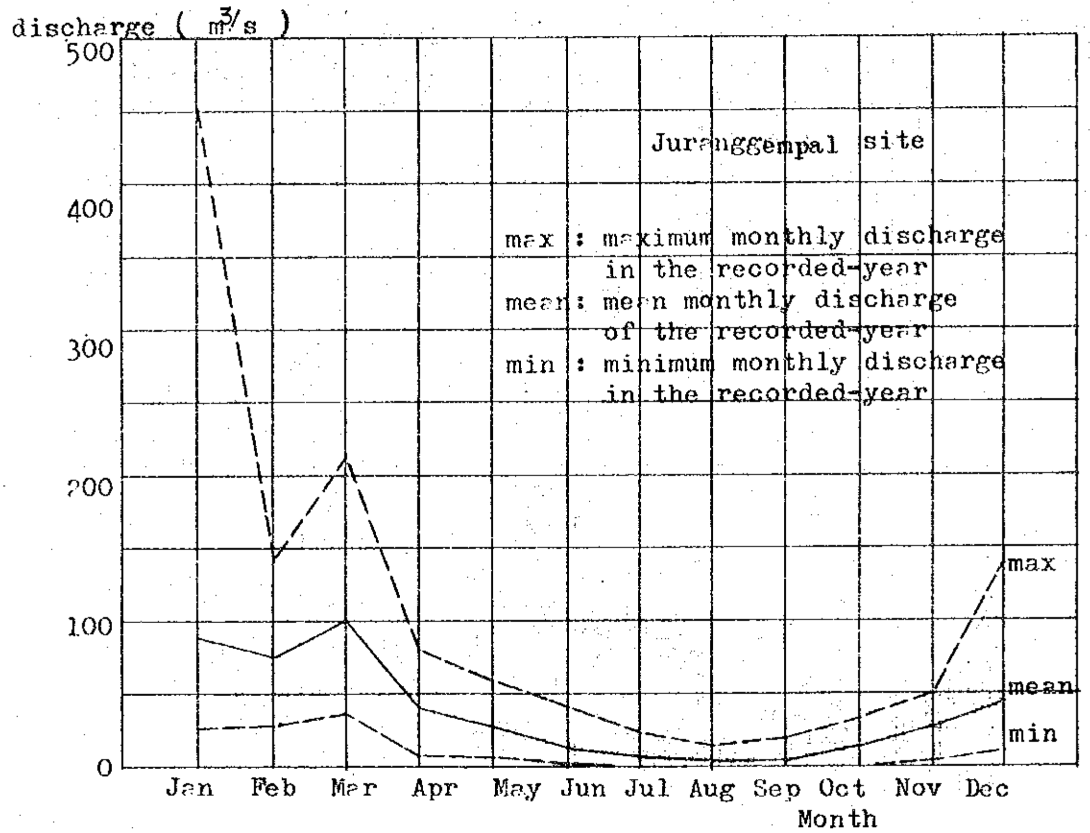
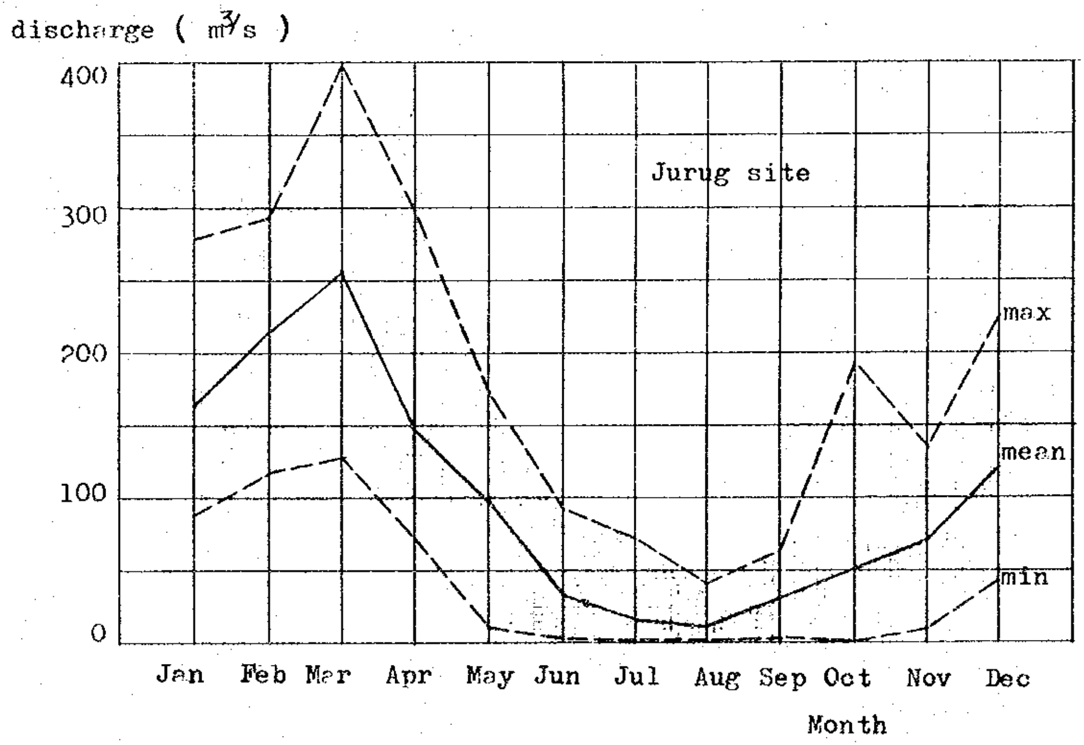


Fig.23.10 Monthly mean discharge at Juranggempal and at Jurug





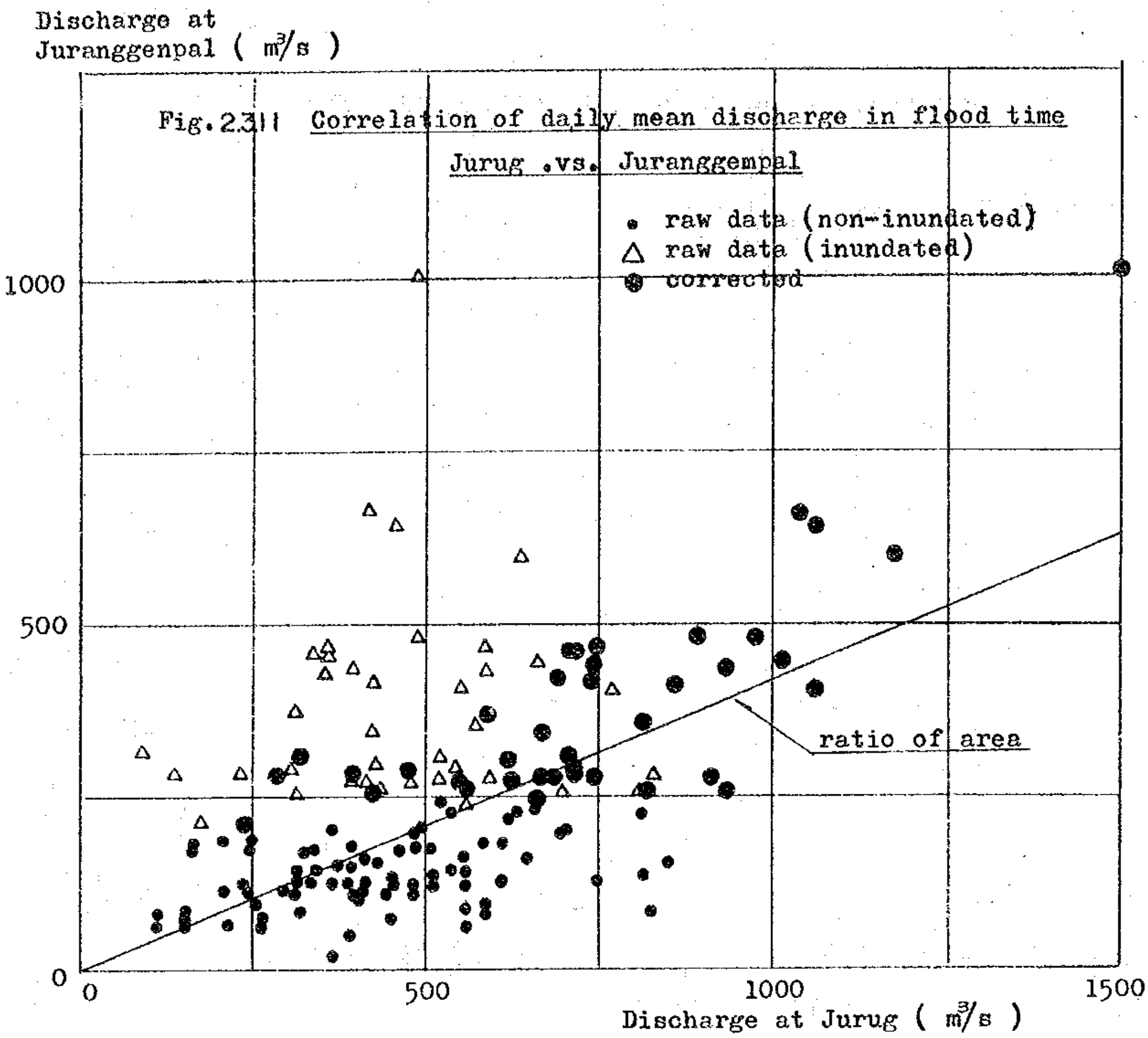

Discharge at
Juranggenpal (m³/s)
Fig.2.3.11 Correlation of daily mean discharge in flood time
Jurug .vs. Juranggempal
raw data (non-inundated)
raw data (inundated)
corrected
ratio of area
1000
500
0
0
500
1000
1500
Discharge at Jurug (m³/s)

Fig. 2.3.12 Annual Rainfall .vs. Annual Rainfall-Loss

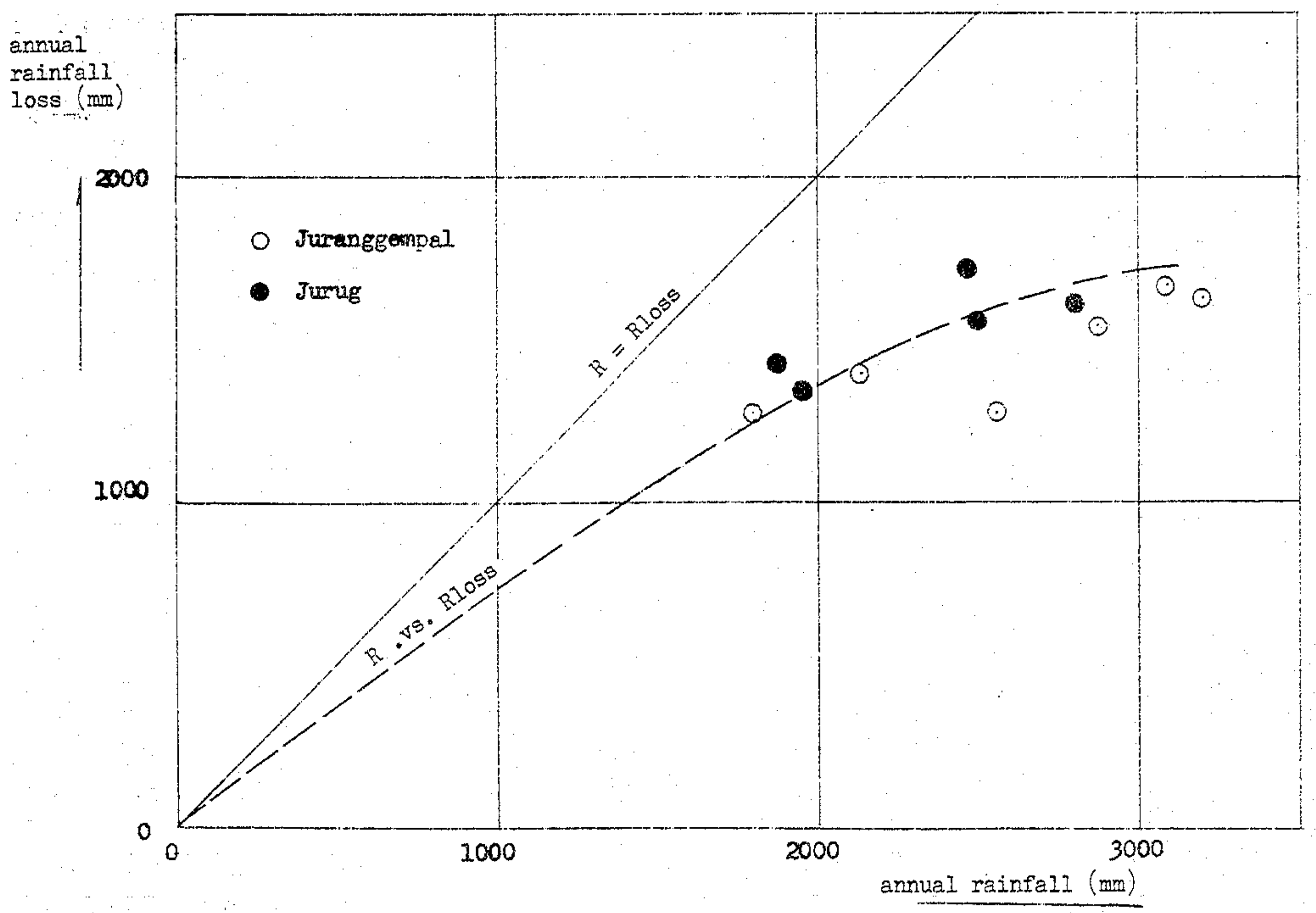


Table 2.3.1 Monthly Means Discharge

(Unit: m^3/s)

(a) Juranggempal site

Year	Jan	Feb	Mar.	Apr.	May	Jun.	July	Aug.	Sep.	Oct.	Nov.	Dec.	Meam
1965	-	-	73.2	20.9	13.1	9.8	6.3	4.9	2.8	-	-	30.2	(20.1)
1966	52.8	75.9	211.3	40.6	15.1	12.0	2.1	0.9	1.0	22.8	15.9	24.6	39.6
1967	453.8	67.5	63.7	69.5	23.6	14.9	8.1	2.6	1.8	1.2	19.0	27.7	62.8
1968	43.7	28.2	37.1	6.0	6.1	18.6	9.0	11.1	7.7	32.6	37.3	139.5	31.4
1969	26.1	86.0	44.5	33.2	6.1	2.4	0.4	0.1	0.2	1.9	6.1	10.0	18.1
1970	30.2	79.4	54.2	41.4	59.3	24.3	3.1	1.5	7.9	19.2	39.8	70.6	35.9
1971	50.1	102.3	113.7	34.6	27.7	16.1	5.8	3.9	2.3	18.9	40.4	45.3	38.4
1972	73.2	33.9	87.8	28.2	24.1	2.0	0.9	0.6	1.7	0.2	3.6	11.5	22.3
1973	55.6	53.9	106.9	55.6	47.6	40.3	22.4	11.2	4.7	14.3	32.2	-	(44.5)
1974	49.7	142.3	124.5	45.0	42.0	5.0	6.6	11.4	11.9	16.7	49.9	39.0	45.3
1975	62.3	93.3	206.5	80.2	43.6	6.8	5.6	4.3	-	-	-		(62.8)
Meam	89.8	76.3	102.1	41.4	28.03	13.8	6.4	.4.8	4.2	14.2	27.1	44.3	62.8

(b) Jurug site

Year	Jan.	Feb.	Mar.	Apr.	May	Jun.	Jul.	Aug.	Sep.	Oct.	Nov.	Dec.	Meam
1968	108.8	126.7	268.0	72.7	106.4	92.8	72.2	40.7	3.2	10.9	-	-	(90.3)
1969	-	252.0	216.6	251.2	10.9	11.6	2.5	2.5	-	-	9.9	42.7	(88.9)
1970	88.5	141.1	128.0	84.2	149.0	29.6	1.9	1.1	8.7	7.3	87.7	225.3	79.4
1971	149.3	274.3	290.0	80.5	55.4	30.4	8.3	2.7	1.3	49.0	91.0	123.0	96.3
1972	178.9	116.8	214.2	83.4	49.1	3.4	1.6	1.4	-	0.3	11.5	72.1	(66.6)
1973	279.7	244.3	293.9	143.9	174.3	76.2	20.7	6.3	60.6	23.8	70.6	110.7	125.4
1974	135.4	294.4	245.6	174.9	97.1	10.3	12.3	23.4	42.8	66.9	135.9	103.6	111.9
1975	204.0	284.4	397.1	296.6	139.6	13.4	11.8	6.8	63.8	194.5	86.7	168.2	155.6
Meam	163.5	216.8	256.6	148.4	97.7	33.5	16.4	10.6	30.1	50.4	70.5	120.8	101.8

(c) Jarum site

Year	Jan.	Feb.	Mar.	Apr.	May	Jun.	Jul.	Aug.	Sep.	Oct.	Nov.	Dec.	Meam
1974	4.1	84.6	64.0	42.0	27.9	16.3	-	52.5	67.2	40.8	30.1	20.3	(90.9)
1975	-	63.1	-	83.9	24.3	9.5	-	-	-	-	17.1	40.8	(39.8)
Meam	4.1	73.9	64.0	63.0	26.1	12.9	-	52.5	67.2	40.8	24.1	30.6	65.4

the grain size is fine again and yet more or less coarser than that of the Jurug-Colo reach.

In the site investigations, it has been found that sand collection is going on by manual excavation in the vicinities of the Jurug Railway Bridge, Bacem Bridge and Nguter Bridge over the main river; it was also noted that along the tributar is, the K. Brambang in particular, quite a volume of sand is being dug out manually. The sands from the Nguter Bridge and K. Brambang are of less silt content and is said to be selling at a high price as construction material.

2.4.2 Suspended load

The suspended load was checked at 8 points once or thrice respectively. The examination sites are as shown in Fig. 2.4.1 with the results of the examination in Table 2.4.1.

The relation between discharge and suspended load discharge is as shown in Fig. 2.4.1, but when judged as a whole, the suspended load discharge is in proportion to the aquare of the discharge. This relation is obtainable from the following formula.

$$Qs = 0.0198\ Q^{2.0}$$

Q : discharge (m^3/S)

Qs: suspended discharge (kg/s).

When the annual total of the suspended load discharge calculated by this relation is multiplied by the daily discharge distribution in Fig. 2.3.9, it gives 2.61×10^6 m^3 for the drought year, 5.48×10^6 m^3 for the normal, and 7.60×10^6 m^3 for the wet year. When each of these is converted into the annual land loss height, it corresponds to 810, 1,700 and 2,360 m^3/year/km^2 respectively.

It is supposed that the influence of the suspended load on the river-bed formation is very small. Namely, the relation between the discharge and suspended load density equals to the primary proportion as indicated in Fig. 2.4.4. Furthermore, the discharge and area are again in a relation equal to the primary proportion. Consequently, the suspended load from the tributary into the main river is carried away to the lower stream, and never scours the river bed or is deposited on it. For this reason, it may well be considered that the suspended discharge is of the same character as the wash load.

2.4.3 Bed-load

From the results of the field survey and suspended load observation, it can be judged that the bed load plays an important role in the formation of river bed of the Sala river.

The measuring of the bed load was not feasible because of the faulty conditions of the required instruments. After conducting the trial measuring several times, the sand, which was confirmed to have formed the bed-load, was obtained.

Therefore, the bed-load discharge was studied by using the value calculated from the bed-load discharge formula. For bed-load discharge calculations, there are a number of different formulas such as Du Boys formula, Einstein formula, Sato-Kikkawa-Asida formula, etc. The Sato-Kikkawa-Asida's handy formula was used this time.

$$Q_B = q_B \cdot B$$

$$q_B = Wg^{0.5} R^{1.5} I^{1.5}$$

where

Q_B: Bed-load discharge (kg/s)

q_B: Unit bed-load discharge (kg/m.s)

B: River width (m)

W: Unit weight of water (kg/m^3)

g: 9.8 (m/s^2)

R: Hydraulic mean depth (m)

I: Energy slope or river bed slope.

It can be obtained from the Sato-Kikkawa-Asida formula of $q_B = \wp \cdot (\sigma/\sigma-\rho) - To\text{-}U_* f(Tc/Td)$, $To = WRI/U_* = \sqrt{gRI}$ and with the natural river as $\wp(\sigma/\sigma-\rho) \fallingdotseq 1$ $f(Tc/Td)^* \fallingdotseq 1$.

As to the bed-load discharge (Q_B), a comparatively large discharge has been observed lately. The calculations were made for the Nguter bri. site and Bacem bri. site of the main Sala river, the results of which are shown in Fig. 2.4.5. From the results we find that Q_B at the two sites is approximately the same, and the bed-load discharge has a linear relation with discharge. According to the calculations, the an- annual load discharge obtained is 21 m^3/year/ km^2 which, as compared with the suspended load discharge (1,700 m^3/year, km^2), stands for a conspicuously small value.

Quoted above is the result of the calculations, but according to the past examples there are many occasions in which the difference between the calculated value and the actual value is as large as several-ten times. As to the bed-load, therefore, the measuring will have to be progressed continuously hereafter.

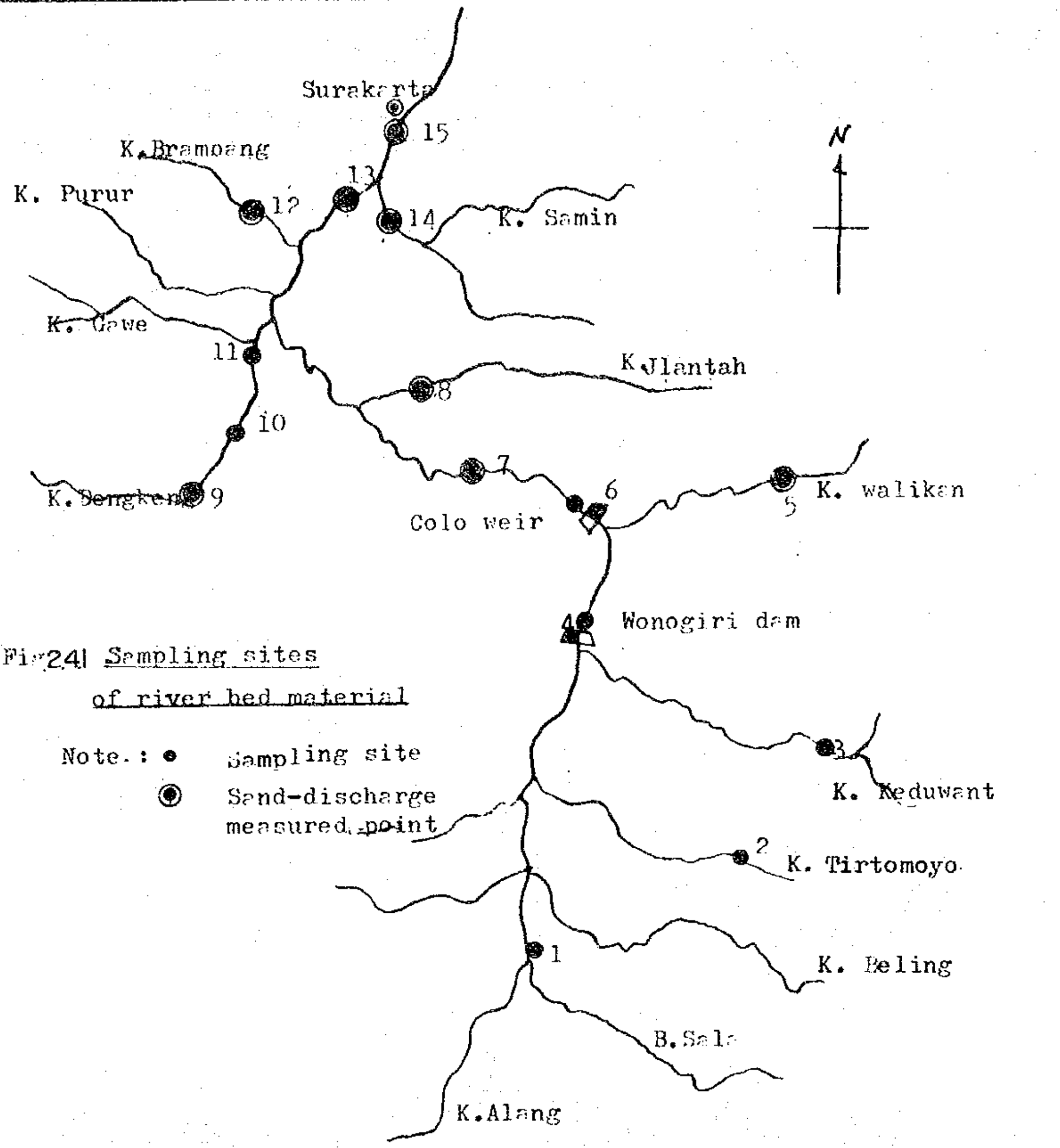


Fig.241 Sampling sites of river bed material

Note. : ● Sampling site
◉ Sand-discharge measured point

No.	Site	River	No.	Site	River
1	Clesung Bri.	B.Sala	9	Djasan	K.Dengkeng
2	Kulurejo	K.Tirtomoyo	10	Madjasto	K.Dengkeng
3	Sembuwan	K.Keduwant	11	Pandjang	K.Dengkeng
4	Dam site	B.Sala	12	Tembalan	K.Brambang
5	Kedoengsari	K.Walikan	13	Bacem Bri.	B.Sala
6	Weir site	B.Sala	14	Guntur	K.Samin
7	Nguter Bri.	B.Sala	15	Mojo	B.Sala
8	Keroeh	K.Jlantah			

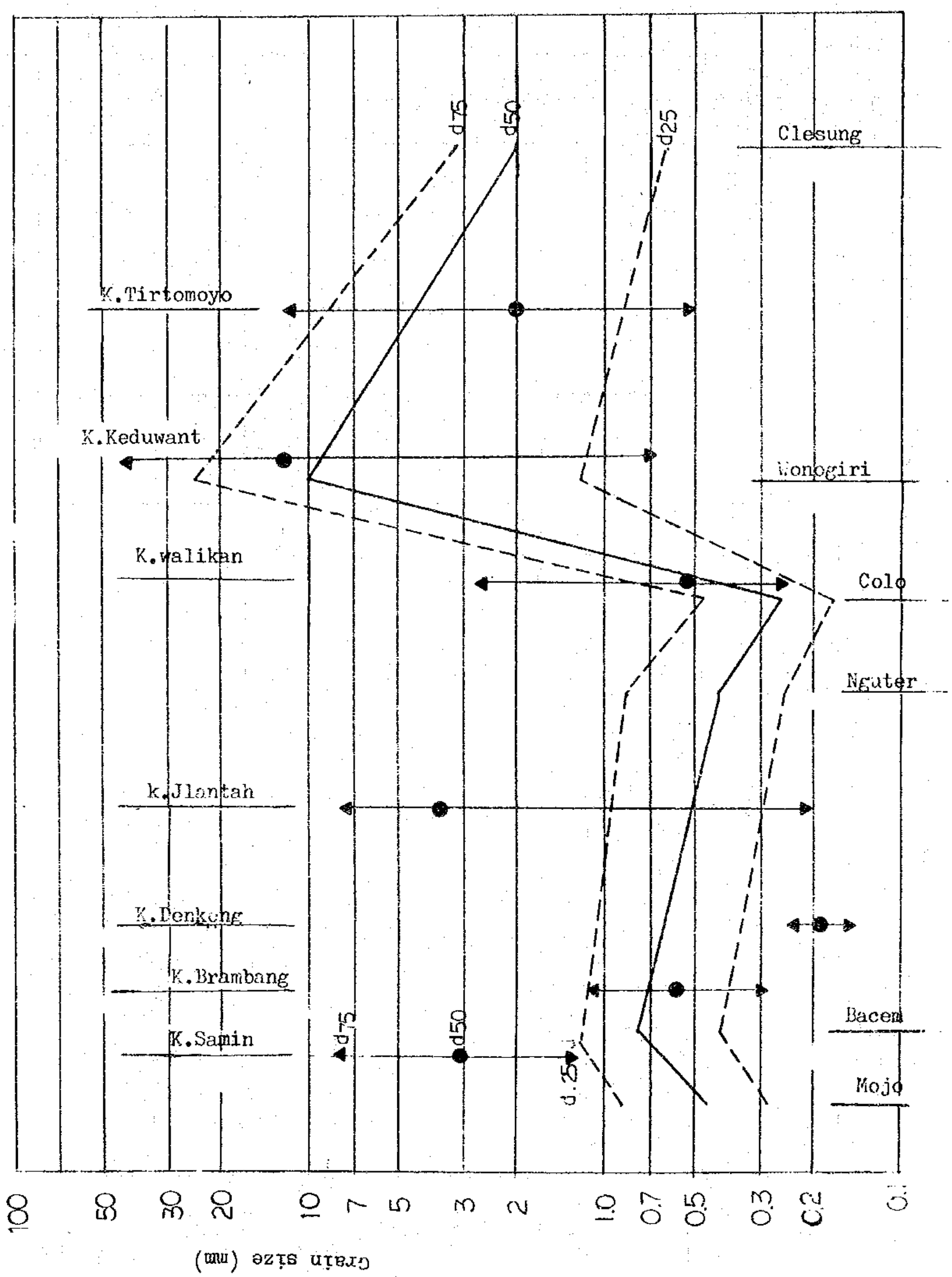


Fig.24.2 Profile of grain size in upper Jurug reach

Fig. 2.4.3 Suspended load discharge.

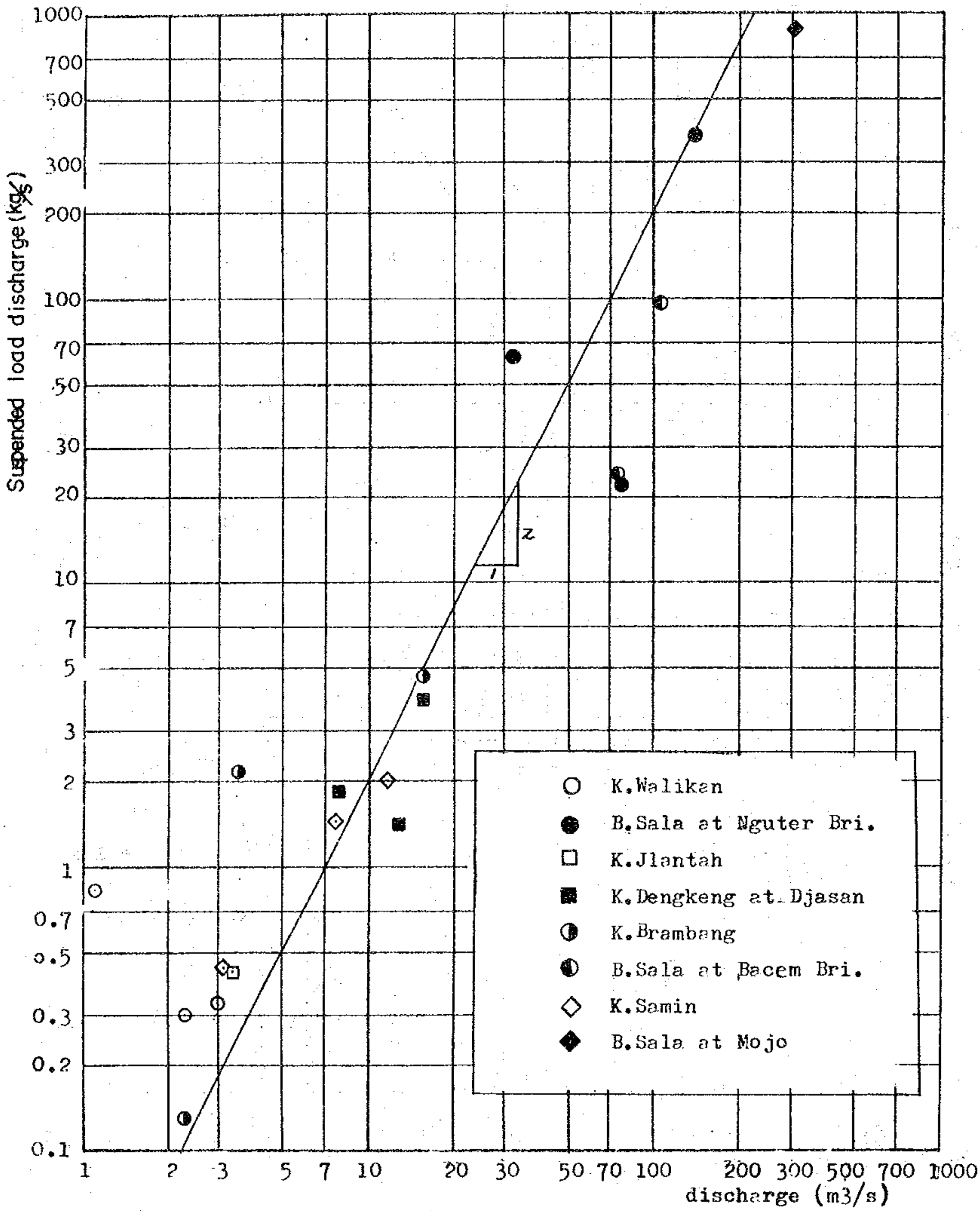


Fig 2.4.4 Suspended load density

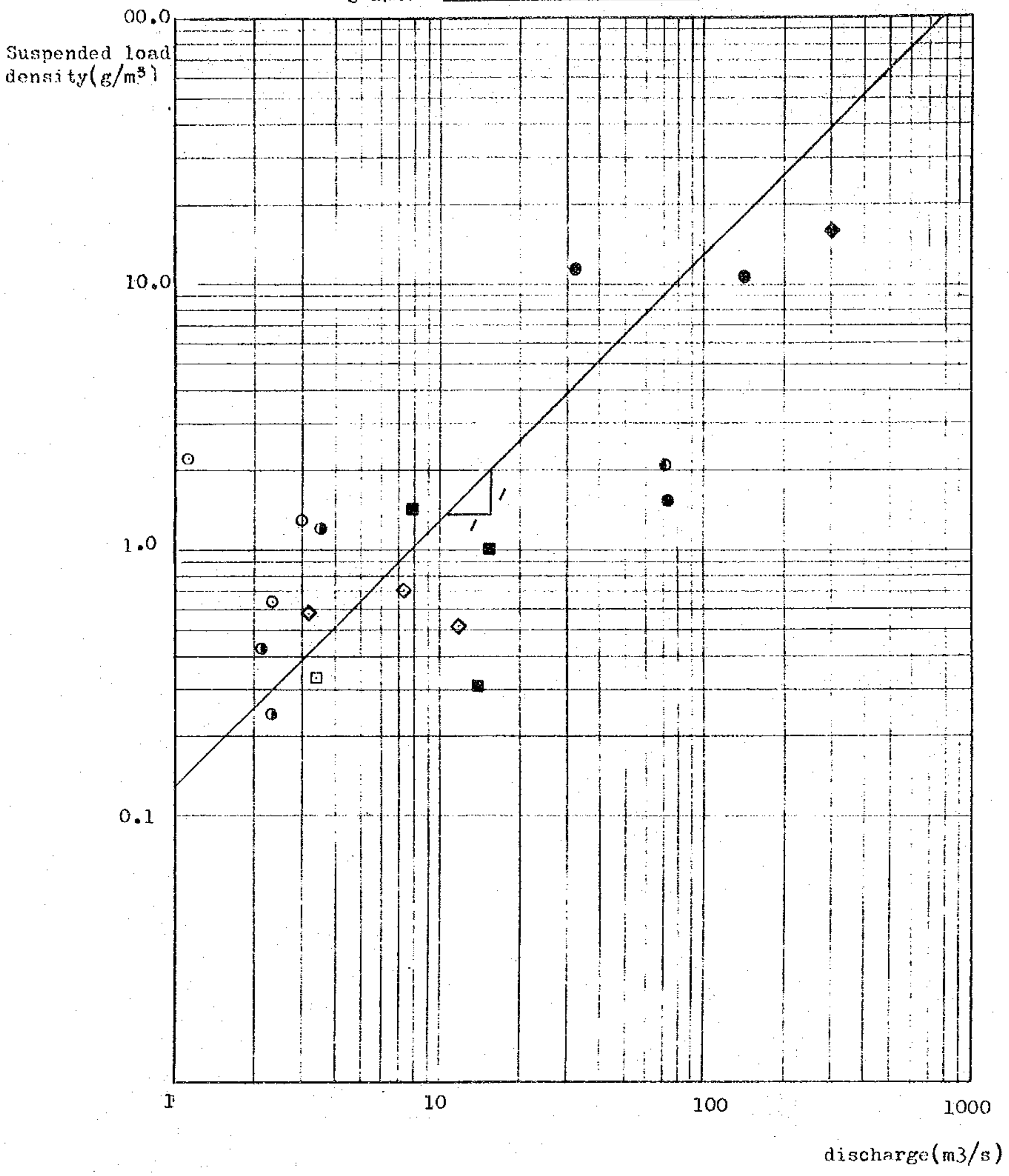


Fig. 2.45 Bed load discharge. (calculated)

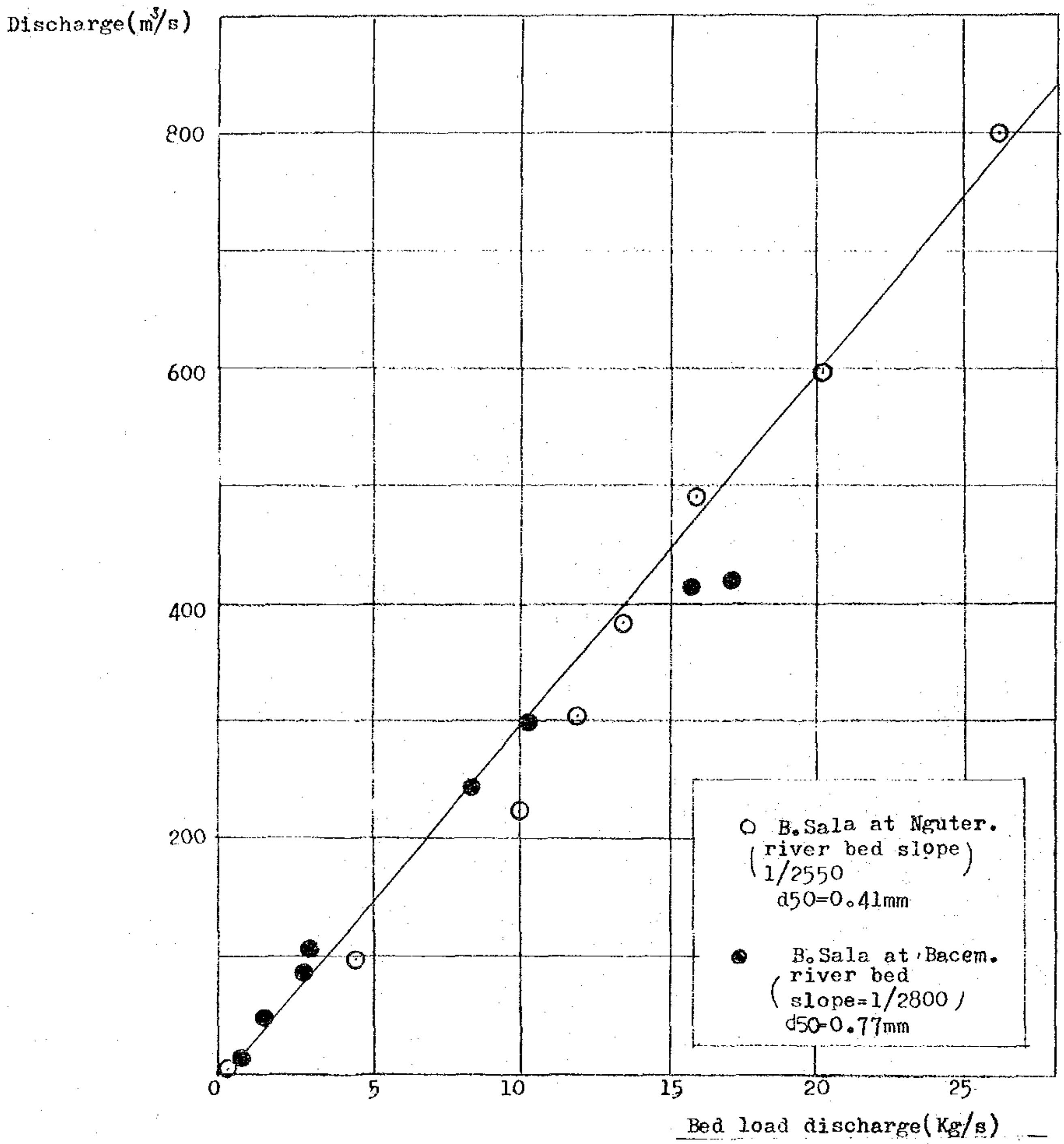


Table 2.4.1 Results of Suspended-load Observation

No.	Site	Grain Size (mm)			Specific gravity of sand (g/cm^3)	Observed results of Suspended load								
						1			2			3		
		25	50	75		Q (m^3/S)	V (m/S)	Qs (kg/S)	Q (m^3/S)	V (m/S)	Qs (kg/S)	Q (m^3/S)	V (m/S)	Qs (kg/S)
1	Clesurg Bri.	0.61	2.00	3.20	-	-	-	-	-	-	-	-	-	-
2	Kulurejo	0.52	2.00	12.00	-	-	-	-		-	-	-		-
3	Sembuwan	0.70	12.00	40.00	-	-	-	-	-	-	-	-	-	-
4	Dam site	1.20	10.00	24.00	-	-	-	-	-	-	-	-	-	-
5	Kedoengsaru	0,25	0,42	2.60	(3.49)	1.10	0.16	0.82	3.00	0.33	0.93	2.30	0.29	0.30
6	Weir site	0.17	0.26	0.47	2.88	-	-	-	-	-	-	-	-	-
7	Nguter Bri.	0.25	0.41	0.83	2.65	140.00	0.87	374.90	72.60	0.86	22.13	31.70	0.76	63.00
8	Kepoeh	0.22	4.00	7.30	(3.57)	3.40	0.43	0.43	-	-	-	-	-	-
9	Djasan	0.18	0.22	0.29	3.02	15.30	0.50	3.88	7.80	0.41	1.81	13.70	0.53	1.40
10	Madijasto	0.23	0.30	0.44	2.69	-	-	-	-	-	-	-	-	-
11	Pandiang	0.14	0.19	0.23	3.02	-	-	-	-	-	-	-	-	-
12	Tembalan	0.30	0.57	1.10	2.77	2.10	0.62	0.22	2.30	0.67	0.13	3.50	0.80	2.20
13	Bacem Bri.	0.41	0.77	1.20	2.87	70.60	1.13	23.38	103.40	0.74	97.14	-	-	-
14	Guntur	1.30	3.10	7.90	2.91	3.20	0.55	0.43	7.40	0.60	1.42	11.70	0.49	2.00
15	Mojo	0.28	0.46	0.88	2.88	302.60	1.12	889.10	-	-	-	-	-	-

Note : Q = discharge (m^3/S)

V = mean velocity (m/S)

Qs = suspended load (kg/S)

2.5 FLOOD

2.5.1 Noted floods

In the upper Sala basin, big floods occur late in the rainy season of March, but in other seasons, too, middle class floods take place quite often.

Particularly, large floods took place during the latter periods in 1966, 1968 and 1975 about which the records are available. By the scales of precipitation and discharge, the 1966 flood was the largest experienced when the peak discharge at the Jurug site recorded 2,160 m^3/s and the inundated area in the Jurug-Wonogiri area marked about 200 km^2.

The flood in 1968 marked the peak discharge of 1,540 m^3/s at the Jurug site which suggests that the inundation in the Madiun river basin was of a larger scale than in the upper Sala basin.

The 1975-flood caused a big inundation to the K. Dengkeng basin of the Jurug-Wonggiri basin, but the peak discharge at the Jurug site was 1,020 m^3/s which was rather small as compared to the aforementioned 2 cases of the 1966 and 1968 floods.

Explanations on the floods will be given mainly on the inundations of 1966 and 1968.

1) Isohyetal map
The isohyetal map of the 1966 and 1968 floods in the Upper Jurug basin is shown in Figs. 2.5.1 and 2.5.2.

2) Discharge hydrograph
The discharge hydiograph of the 1966, 1968 and 1975 floods at Jurug and Juranggempal, along with the hyetograph for the Upper Jurug basin, is shown in Fig. 2.5.3.

3) Rainfall
The rainfall in the noted floods, and in other maximum annual floods for reference, are shown in Table 2.2.1.

4) Inundation area
The inundated areas of the noted floods in the Jurug-Wonogiri area are indicated in Fig. 2.5.4.

Flood	Peak Discharge at Jurug (m^3/s)	Inundated area (km^2)
1958	-	93.1
1966	2,160	198.0
1968	1,540	-
1975	1,020	86.3

2.5.2 Flood routing

The flood routing has been conducted for the 1966 and 1968 floods according to the storage-function method.

In the storage-function method, the Upper Jurug basin is represented by the 2 basins of Upper Wonogiri and Jurug-Wonogiri and by the Jurug-Wonogiri channel.
The calculation formula of the storage-function method was as follows:

Basin:

$$Q = \frac{1}{3.6} q_{(T-T_L)} \cdot A$$

$$S = K \cdot q^P$$

$$\frac{ds}{dt} = r - q$$

Q: discharge (m^3/s)

r: Input rainfall

q: Output rainfall

T_L: Time lag

A: Area

K. & P: Constants

Channel:

$$S = KQ_L^P - T_L Q_L$$

$$\frac{ds}{dt} = Q_u - Q_L$$

Q_L: Output discharge

T_L: Time lag.

As the result of the analysis, the constants (K and P) contained in the calculation formula were given as indicated in Fig. 2.5.5. The result of the calculations for the 1966 flood was very accurate as shown in the table below.

Item	Peak Discharge (m^3/s)		
	Juranggem pal (1,350 km^2)	Jurug (3,220 km^2)	Ngawi (6,072 km^2)
Observed	3,800-4,000	2,300	1,850
Calculated	3,950	2,160	1,890

2.6 BASIC DATA FOR RIVER IMPROVEMENT PLANNING

In this chapter, explanations will be given on the basic data for river improvement planning: the probable discharge, the discharge distribution in the maximum experienced flood, and the flood control effect of the Wonogiri dam.

2.6.1 Probable discharge in the existing condition

Probable discharges under the existing conditions at Juranggempal and Jurug were obtained from the observed annual maximum discharge data in Table 2.6.1.

These data were plotted by the Thomas method into the log-normal distribution paper of Fig. 2.6.1 and of Fig. 2.6.2.

The result of the probable discharge calculations is shown in Table 2.6.3 . As is known from the table that of the maximum experienced floods the 1966-flood at Jurug corresponds to the 40-year flood and the 1968-flood to the 10-year flood.

2.6.2 Probable discharge in several construction cases

1) Construction cases

In the construction plan of the Wonogiri dam, the coffer dam of the Wonogiri Dam is to be completed in 1978, and the Wonogiri Dam as a whole in 1980. As to the river improvement works, the first stage of the plan is to be started in 1978 and completed in 1983, i.e., 3 years after the dam construction.

To cope with this program, the probable discharge in Jurug at each construction stage is examined:

Case 0 ... Existing condition.

Case 1 ... Main coffer damming + existing river.

Case 2 ... Wonogiri dam + existing river. (400 m^3/s constant outflow).

Fig.2.5.1 Isohyetal Map in 1966 Flood

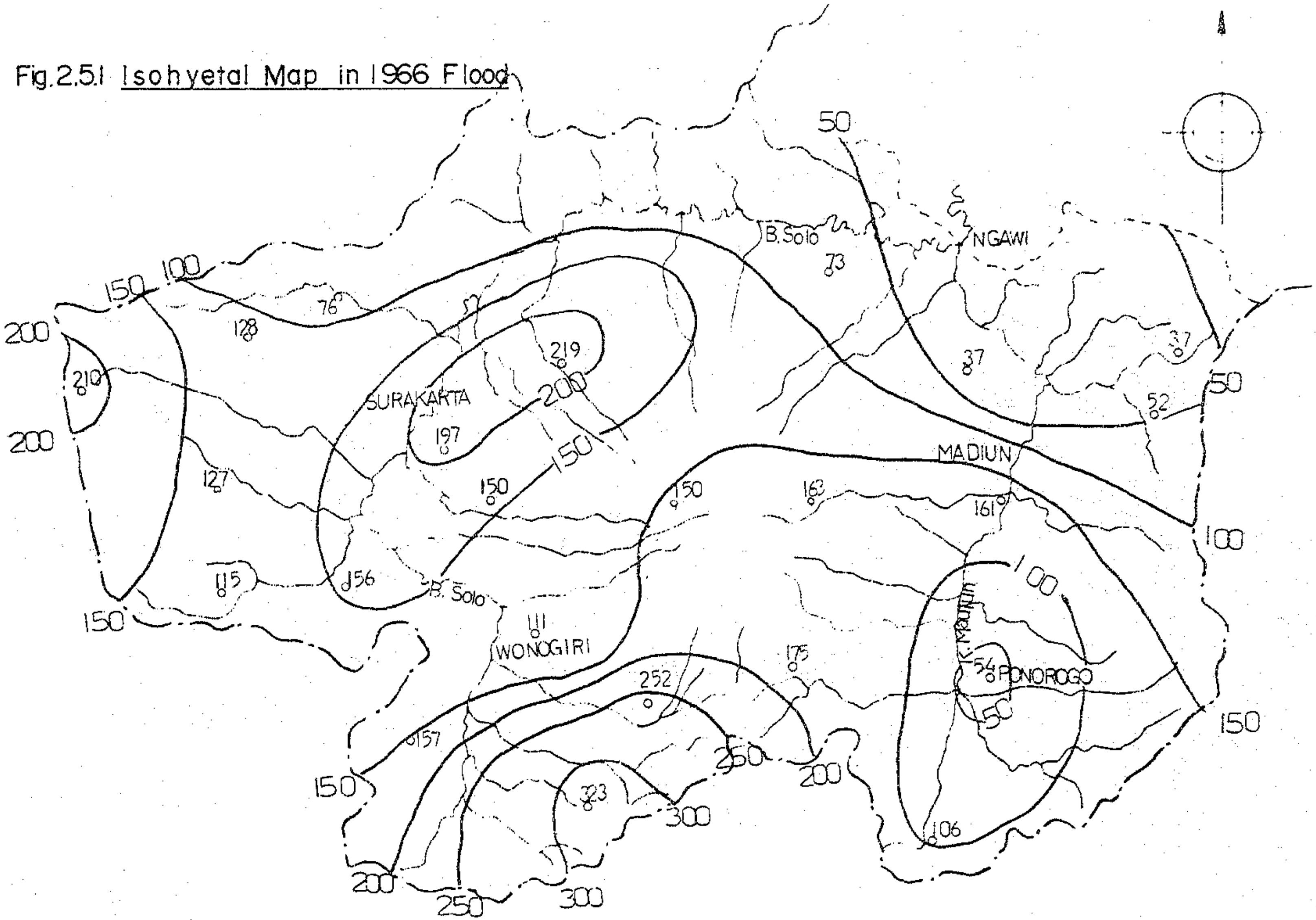


Fig. 2.52 Isohyetal Map in 1968 Flood

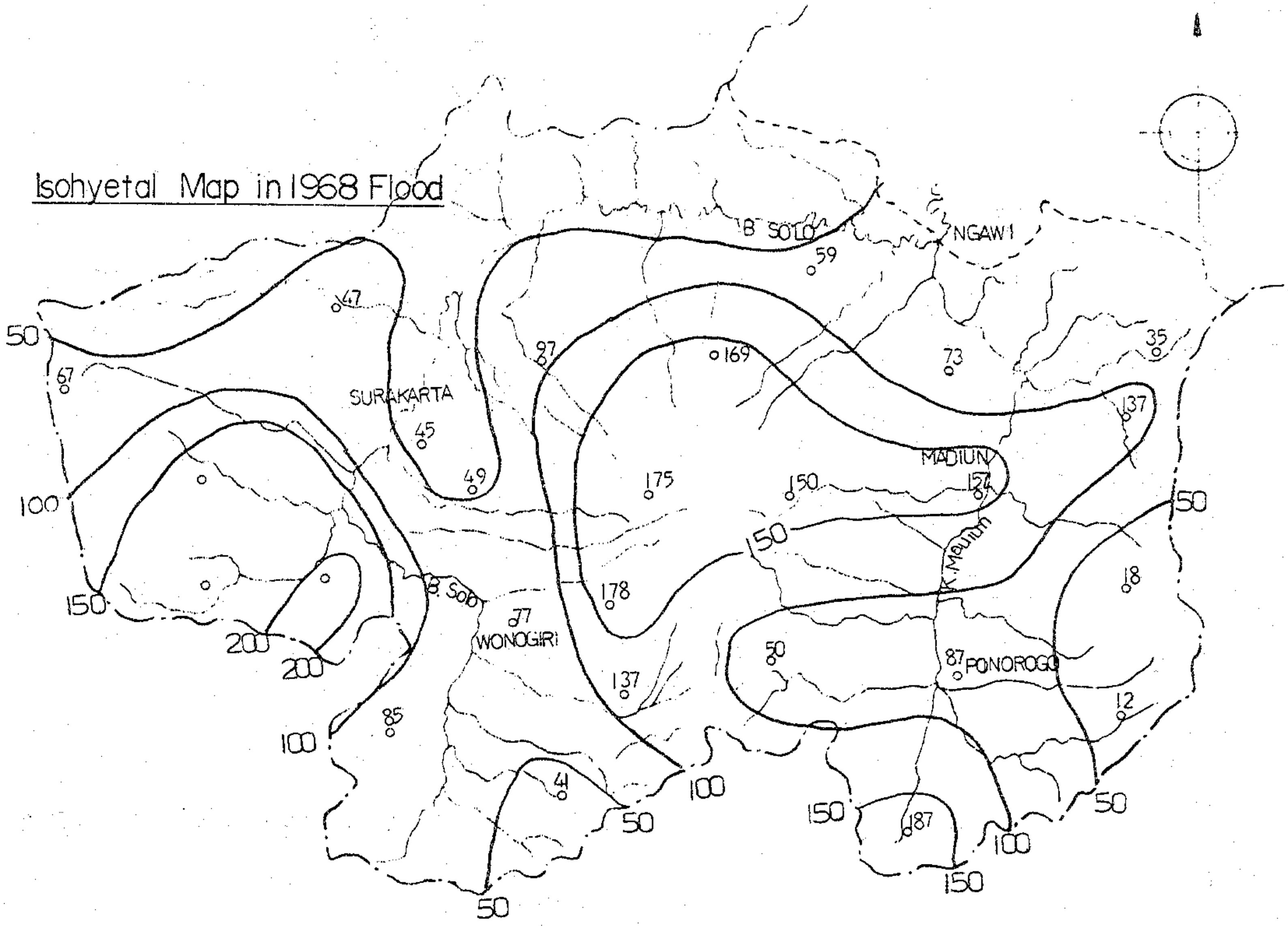


Fig. 2.5.3 Discharge Hydrograph under Existing Condition

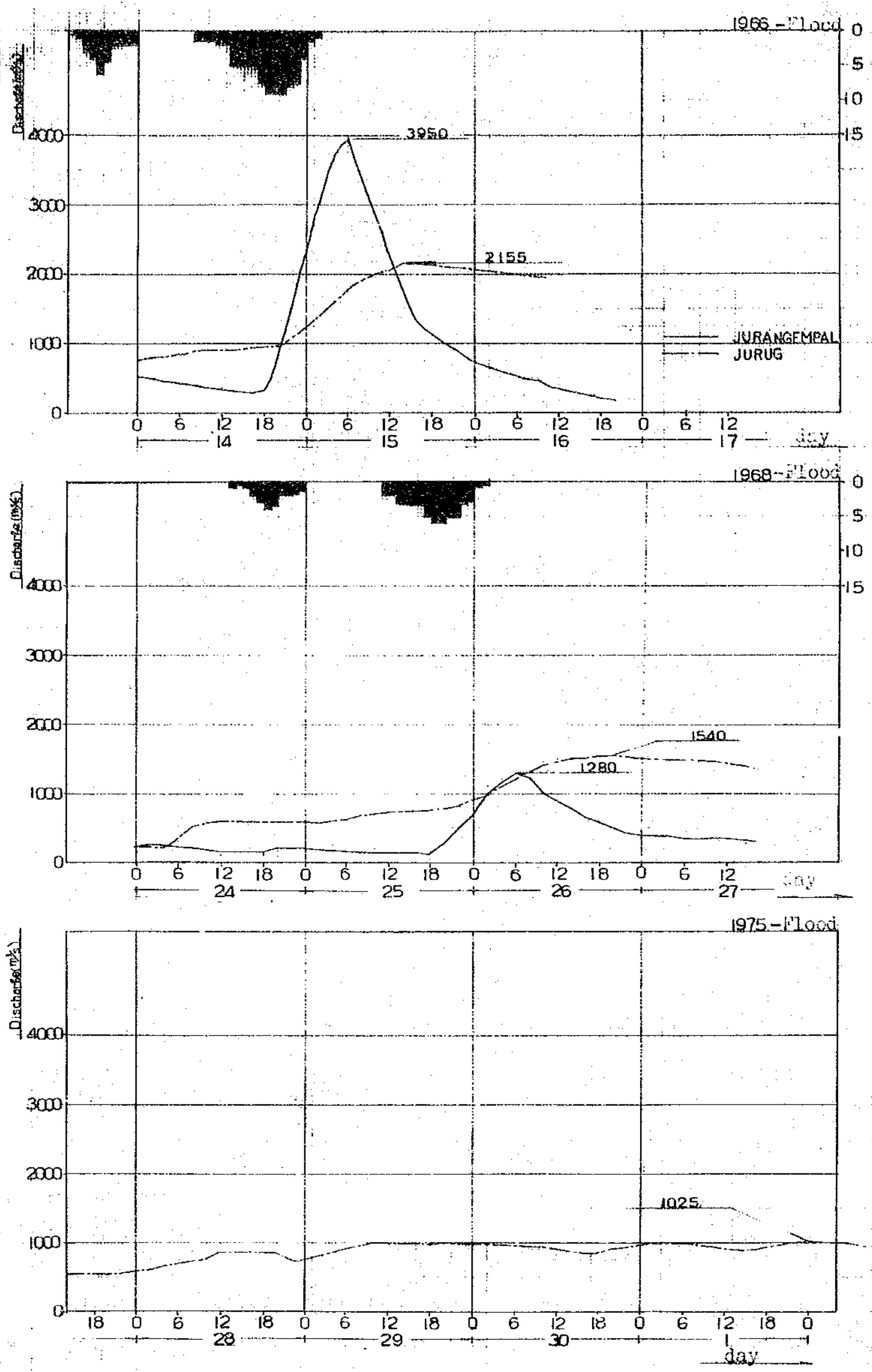
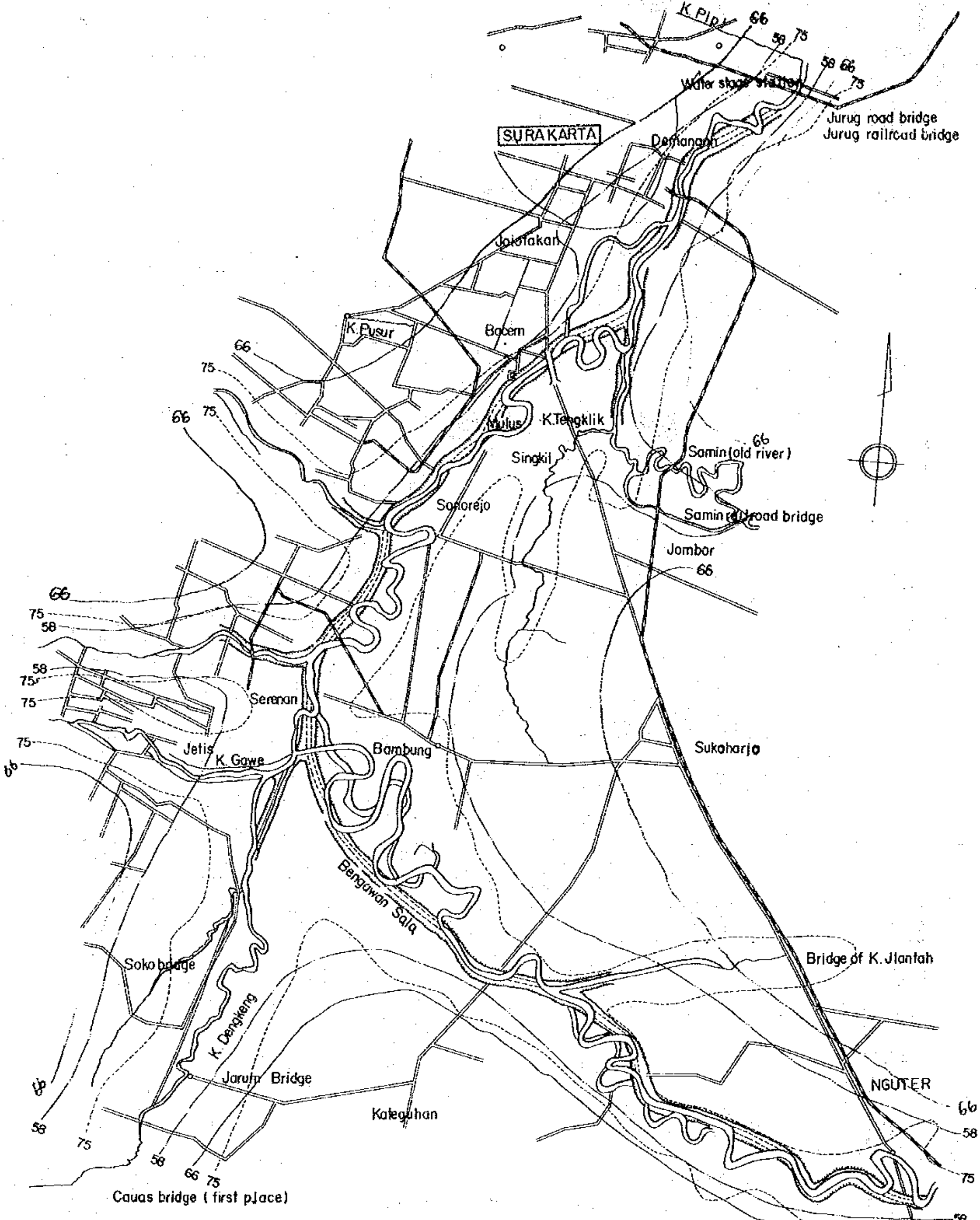


Fig. 2.5.4 Inundation Area in Large Floods



Remarks

---- Inundation area in 1958 flood

—— 〃 in 1966 flood

-------- 〃 in 1975 flood

Fig 2.5.5 Flood Routing Model

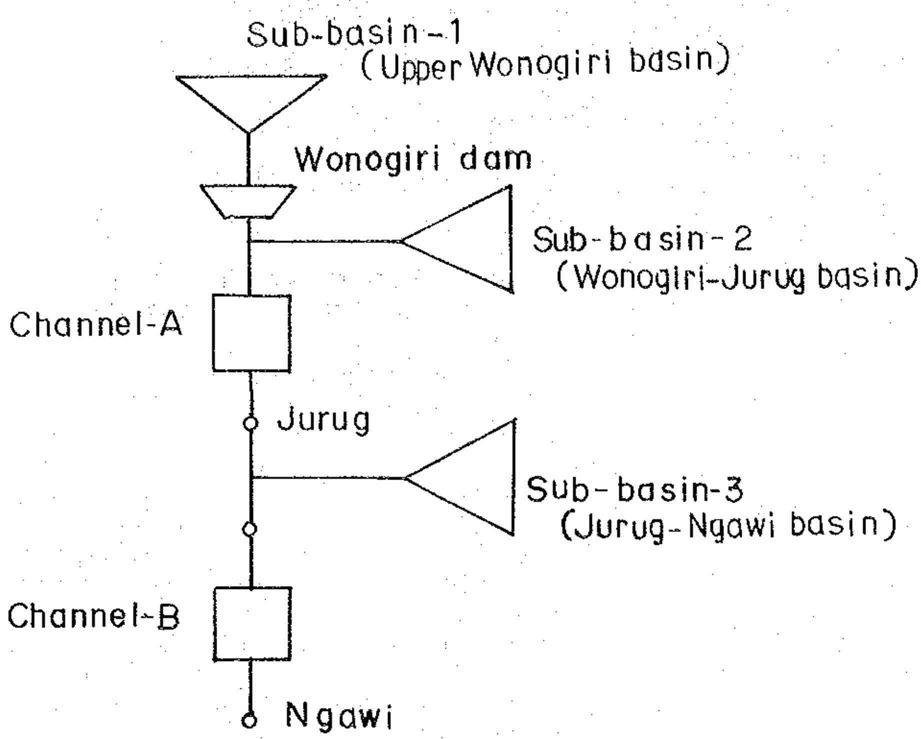


Table I-21 Constants of Storage Function.

(a) Constants for watershed.

Watershed	Area	K	P	TL	f	Qb	Rsa
Sub-basin1	1350 km^2	27.0	0.6	6hr.	1.0	86 m^3/s	0.0 mm
2	1870	49.3	0.6	7	1.0	120	0.0
3	2852	73.7	0.6	9	1.0	188	0.0

(b) Constants for channel in Existing condition

Channel	Length	K	P	TL	Range of Q
Channel-A	64 km	0.075	1.772	2 hr.	Q ≤ 540 m^3/s
		82.0	0.65	2	> 540
-B	143 km	1.6	1.462	3	Q ≤ 500 m^3/s
		340.0	0.60	3	> 500

(c) After completion of river improvement facilities

Channel	Length	K	P	TL	Range of Q
Channel-A	64 km	82	0.66	2	Q ≥ 0 m^3/s
-B	143	340	0.60	3	≥ 0

Case 3 ... Wonogiri dam + improved river. (400 m^3/s constant outflow).

Case 4 ... Improved river without flood control by Wonogiri dam.

2) Estimate of Peak Discharge at Jurug Site

The peak discharge for each stage was determined as follows:

a) Under the existing conditions, the 1966- and 1968-floods at the Jurug site correspond to the 40-year and 10-year flood, respectively.

b) The hydrograph, in which the inflow discharge hydrograph of each flood was multiplied by 1.5 and 0.5, was given to the flood routing model to calculate the discharge. In this case, as compared to the discharge probability under the existing condition, the 1966-flood type corresponded to the 500-year flood (1.5 times as large) and 5-year flood (0.5 times as large), and the 1968-flood type to the 50-year (1.5 times as large) and 2-year flood (0.5 times as large).

c) By using the storage-function model constants to correspond to each construction case , the flood routing was performed to work out the discharge at the Jurug site. The result of the calculations thereof is given in Table 2.6.2.

d) The peak discharge obtained in (c) above corresponded to the return period determined for each case by (b) above. From the 6 plotting positions, i.e., three 1966-flood type and three 1968-flood types, the probability line and probable discharge for each construction case can be surmised on the log-normal probability paper. The probable discharge at each construction case surmized as shown in Fig. 2.6.3 and Table 2.6.3.

2.6.3 Discharge distribution in the 1966-flood

On the assumption that the Wonogiri dam functions to control the flood with 400 m^3/s of the constant outflow and there is no inundation in the Wonogiri-Jurug reach, i.e., Case 3, the discharge hydrograph of the 1966, 1968 and 1975 floods at the Jurug site is given in Fig. 2.6.4.

Even after functioning of the flood control, the peak discharge at the Jurug site in the 1966 flood could have been approximately 2,000 m^3/s, the largest as compared with the other two floods.

For this reason, the surmising in advance of the confluent discharge from the tributary of the Jurug-Wonogiri reach as of the 1966-flood is deemed to offer the useful data for determinings later on the design discharge distribution in the river-improvement planing. The surmise has been carried out as follows:

1) The outflow from the Wonogiri dam is 400 m^3/s with the peak discharge of 2,000 m^3/s at the Jrug site, and the maximum confluent discharge from the Jurug-Wonogiri basin (1,870 km^2) is 1,600 m^3/s.

2) In addition to the smallness of the discharge capacity of the tributary, it has a vast inundation area in its upstream, and therefore, the discharged flow is pooled. On that account, the tributary discharge hydrograph becomes gentle which eventually makes the time lag between the main river peak discharge and tributary peak discharge negligible. Furthermore, it could surmised that the peak discharge falls under the control of the minimum discharge capacity of the tributary. However, as the K. Walikan (198 km^2) which joins the main river immediately on the upstream of the Colo weir has no inundation area and, its discharge flows directly into the main river. Besides, discharge from the K. Wingko is ponded in the vast inundation area around Sala city and does not directly increase the discharge of the main stream.

3) The minimum discharge capacity of the tributary was judged from its cross section immediate upsteam of the main river inundation area by the uniform flow formula or the rating curve in relation to the bank-full stage. And for the K. Walikan the peak discharge was calculated from the flood-mark of the 1966-flood and the uniform-flow formula.
The result thereof is given in Table 2.6.4 and Fig. 2.6.5.

4) Excluding the discharge from the K. Wingko, the total confluent discharge of the tributary is estimated at 1,490 m^3/s which leaves a difference of 110 m^3/s from the estimated confluence discharge of 1,600 m^3/s. Accordingly, this difference of 110 m^3/s was distributed to the tributaries, other than the K. Walikan and K. Wingko, for adjustment of the total confluence discharge thereof to 1,600 m^3/s. The result is shown in Table 2.6.4.
The discharge distribution thus estimated for the 1966-flood is given in Fig. 2.6.6.

2.6.4 Discharge distributions of the 15-year, 2-year and 1.2-year floods

The 1966-flood corresponds to the 40-year flood, and the surmised discharge distributions, corresponding to the 15-year, 2-year and 1.2-year floods, are shown in Figs. 2.6.7, 2.6.8 and 2.6.9.

The peak discharges corresponding to these probable floods at the Jurug site are 1,600, 900 and 600 m^3/s, respectively.

2.6.5 Estimated discharge of the tributary after improvement

The tributary discharge under the existing condition is dealt with in Item 2.6.3. This corresponds to the 1-year return period according to the Jurum station records.

Upon completion of the improvement works of the tributary, the area-discharge line under the present condition, as shown in Fig. 2.6.5, is estimated to move in parallel with and pass the plotting position of the K.Walikan which is under the non-inundation condition even at present.

The estimated discharge is given in Table 2.6.4. And to judge from the area-specific discharge relation now being examined in Indonesia, it is likely that this discharge approximately corresponds to the 100-year flood.

Fig. 2.6.1 Probable Peak Discharge at Juranggempal
(Existing Condition)

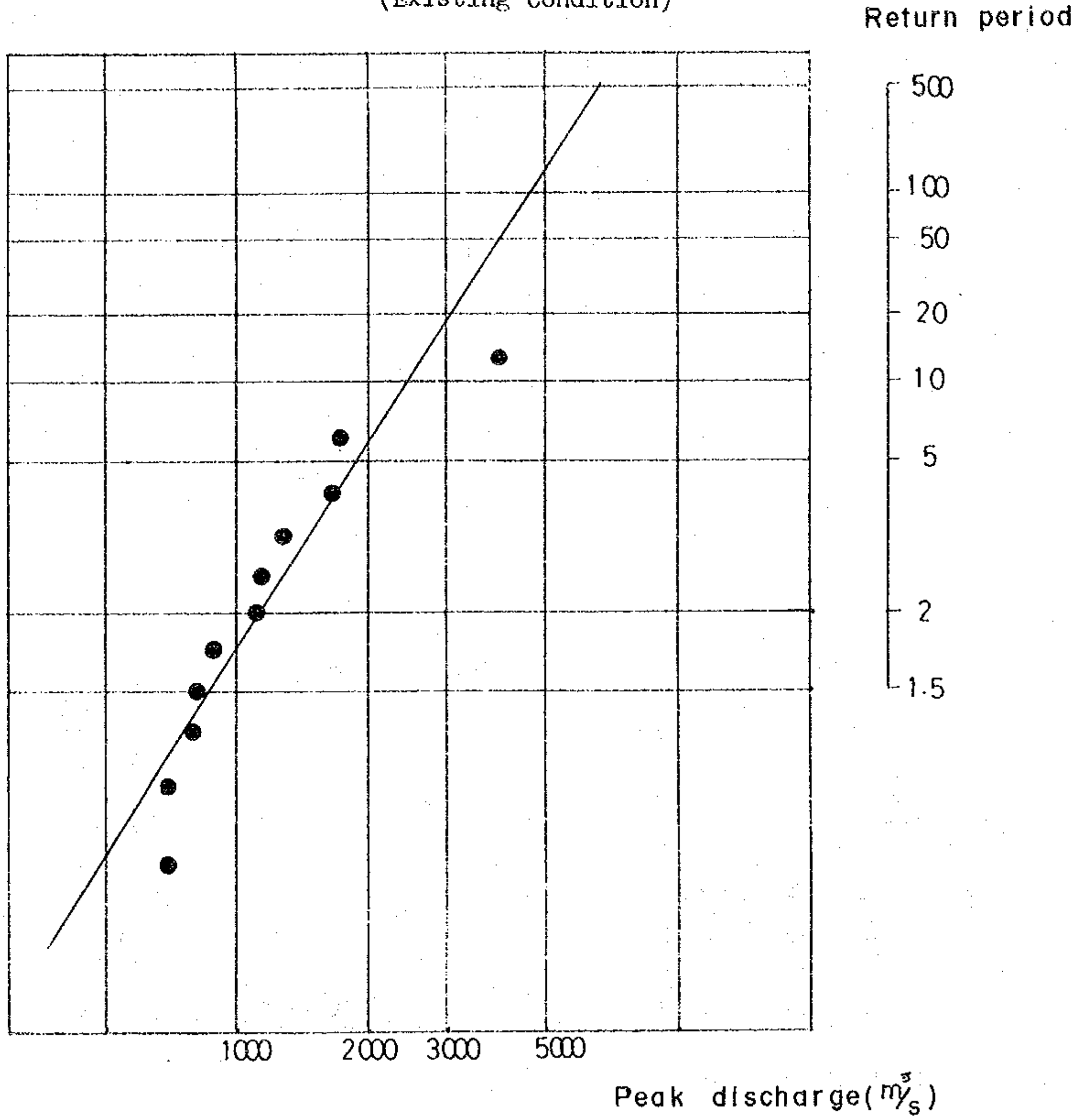
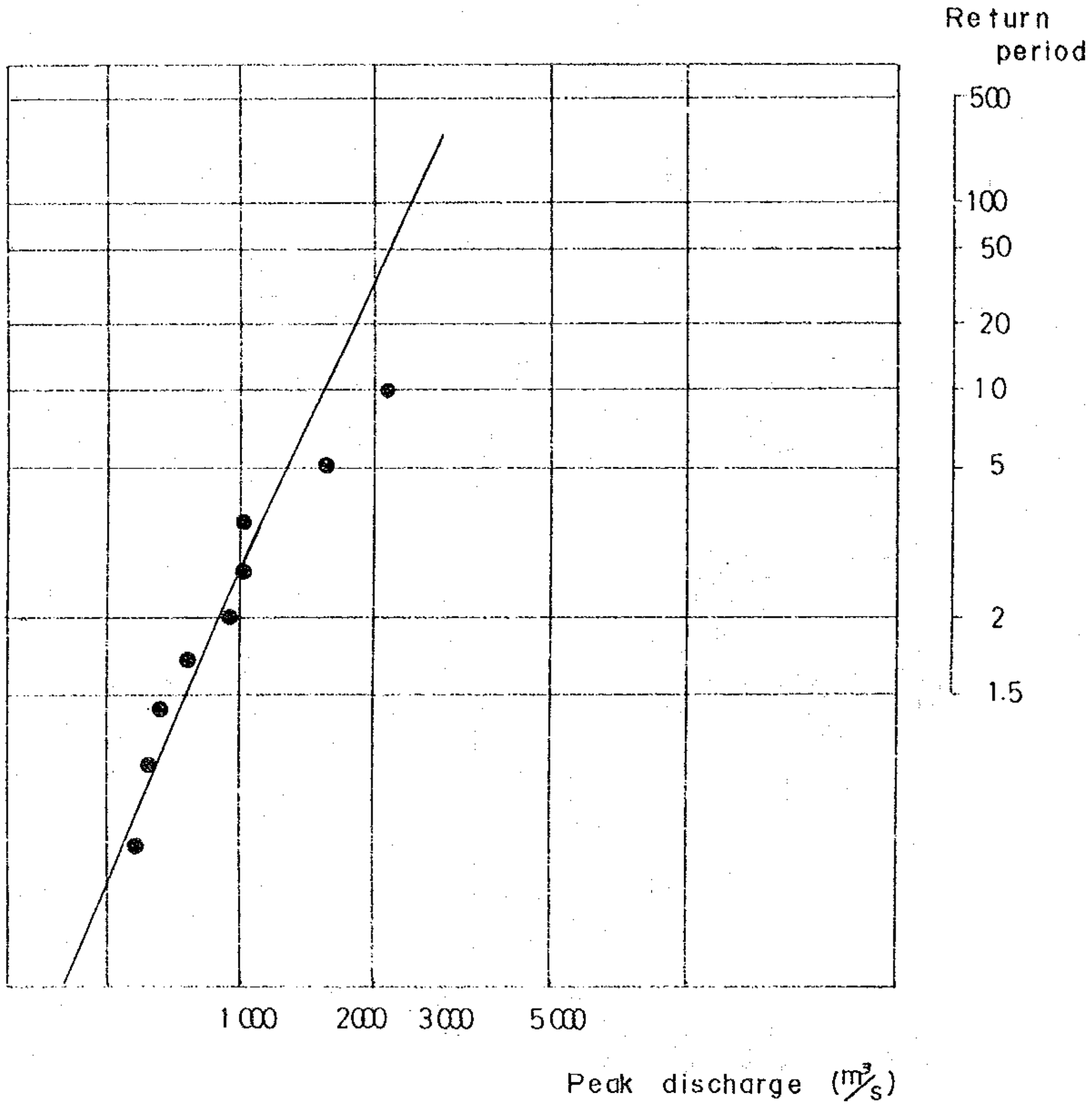
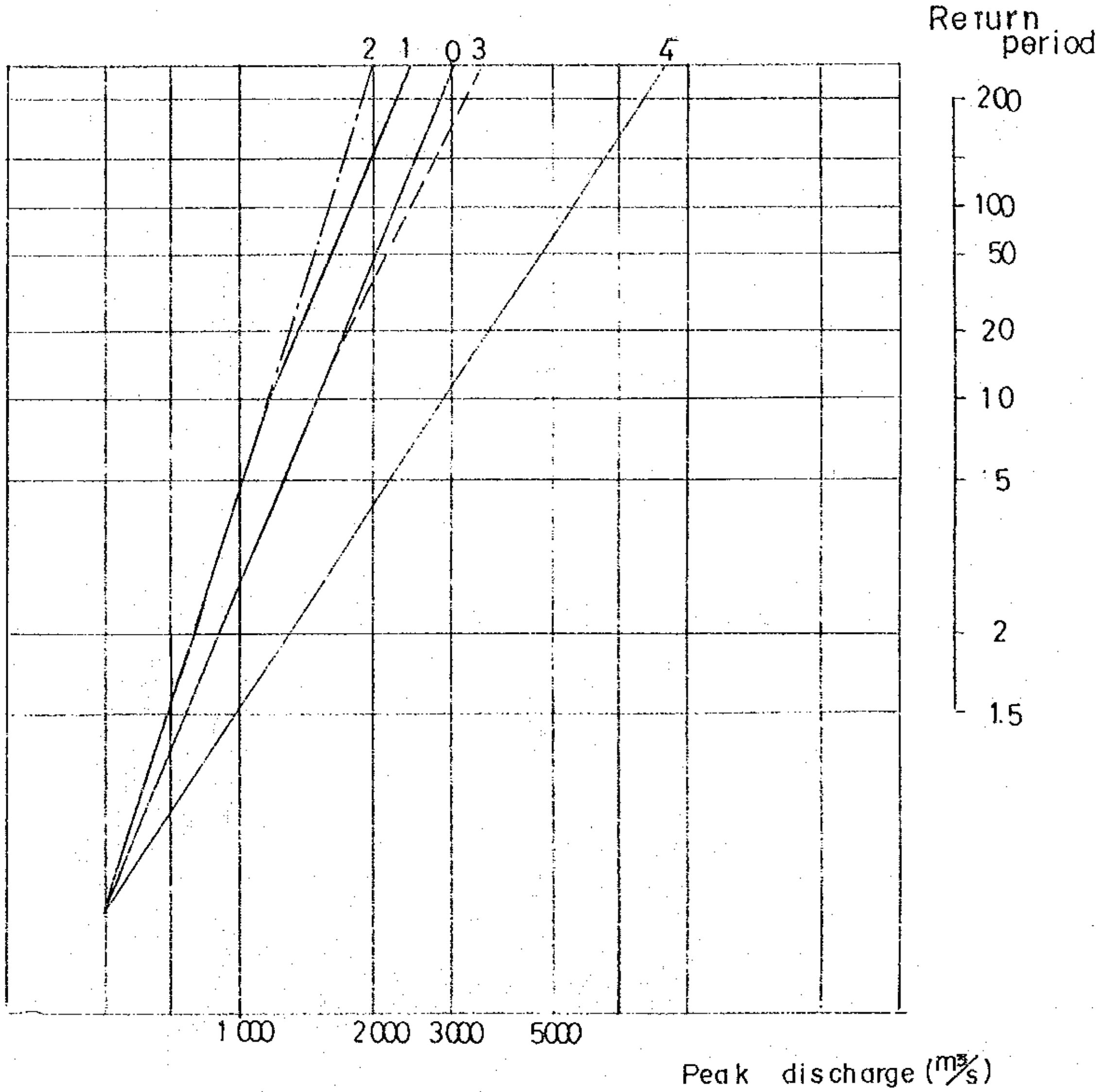


Fig. 2.6.2 Probable Peak Discharge at Jurug
(Existing Condition)



Fij. 2.6.3 Peak-Discharge Probability in Various Construction stage at Jurug
(Construction case)



case 0 -- ⓪ Existing condition.

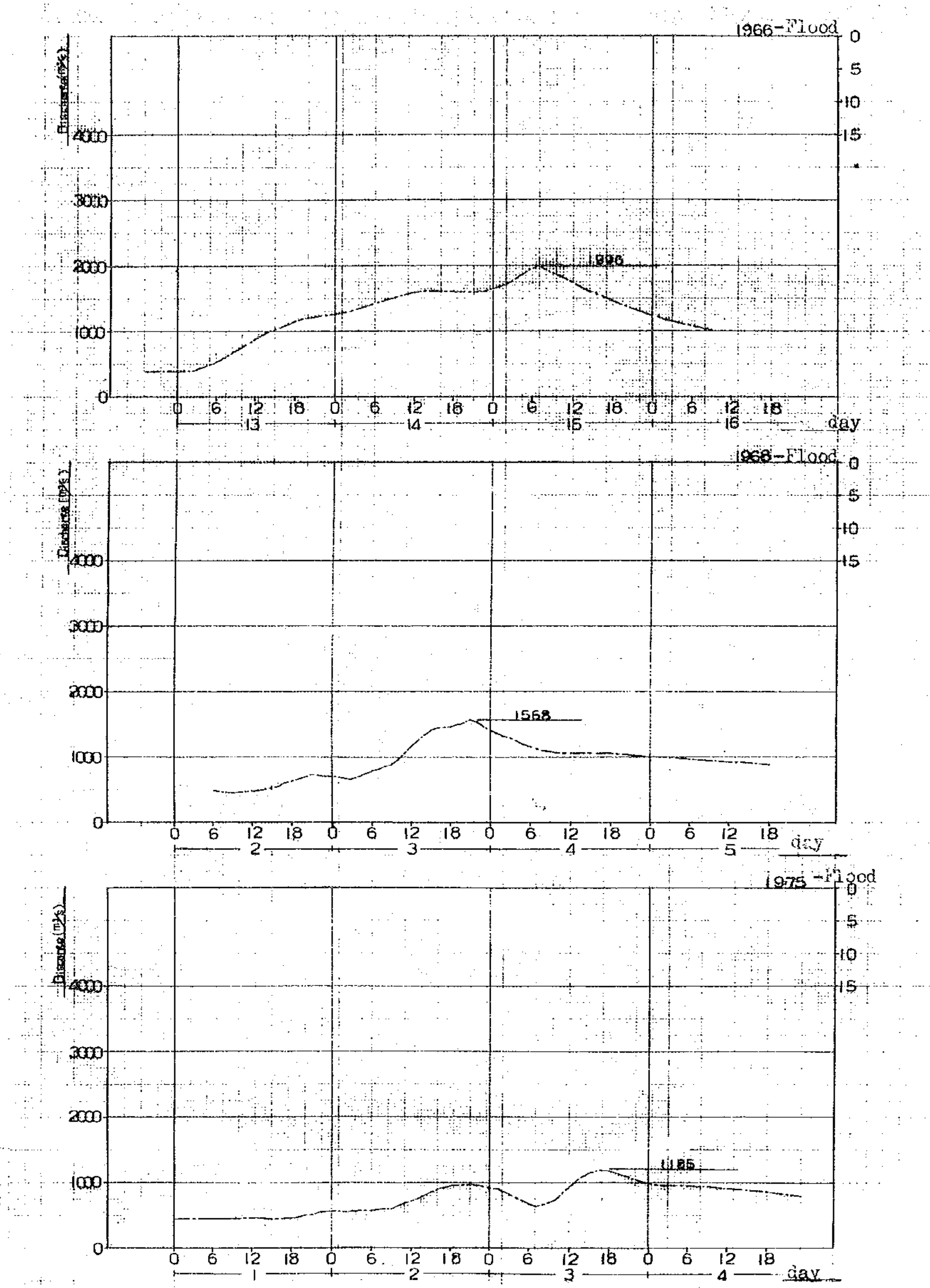
case 1 ---- ① Coffer dam construction

case 2 -- ② Wonogiri. dam . construction

case 3 -- ③ Improved river without retarding basin.

case 4 -- ④ Non Wonogiri dam control.

Fig 2.6.4 Discharge hydrograph at Jurug (case 3)



Fjg.2.6.5 Estimated Discharge of Tributary

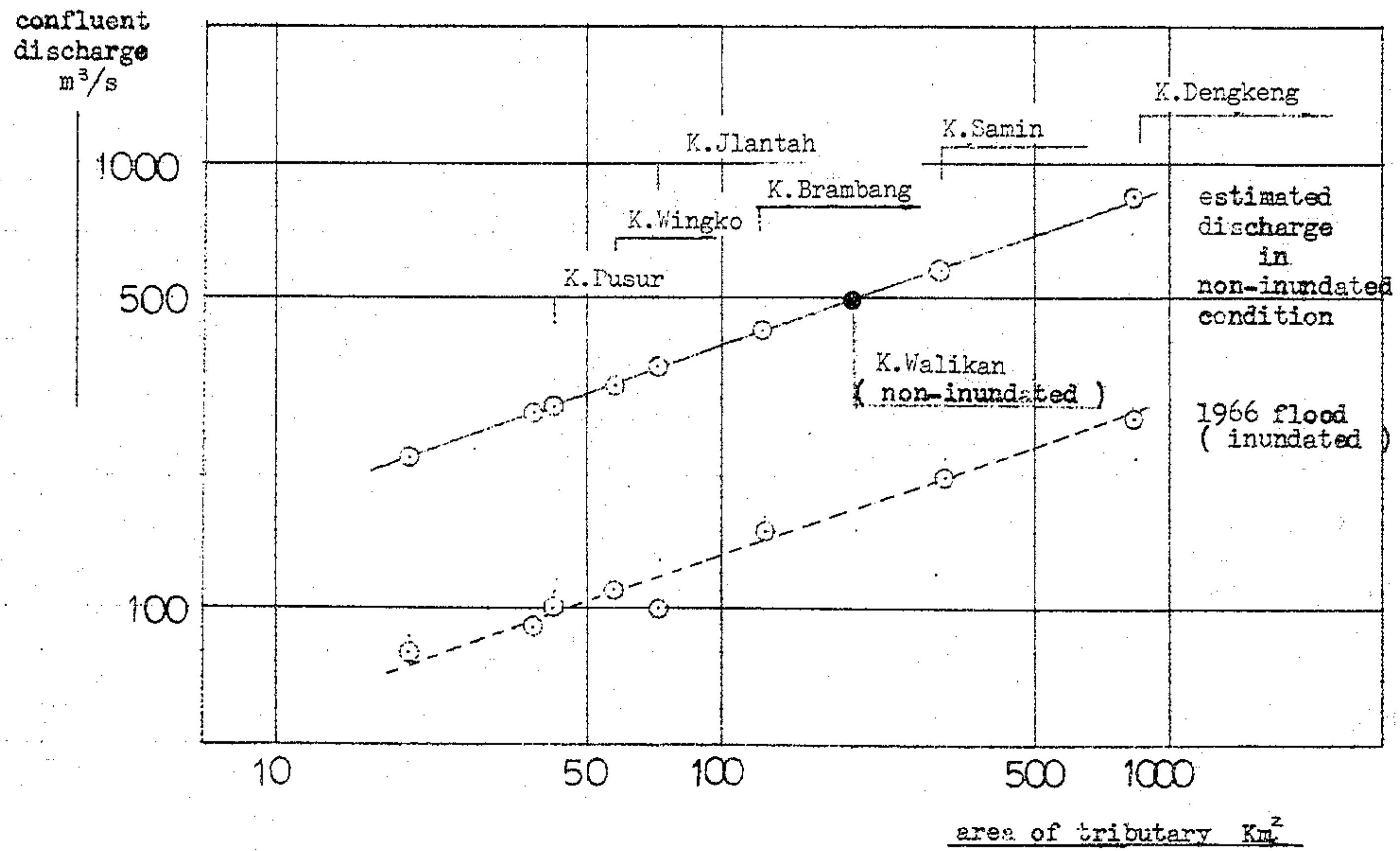


Fig.2.6.6 Specific discharge vs. area in Indonesia

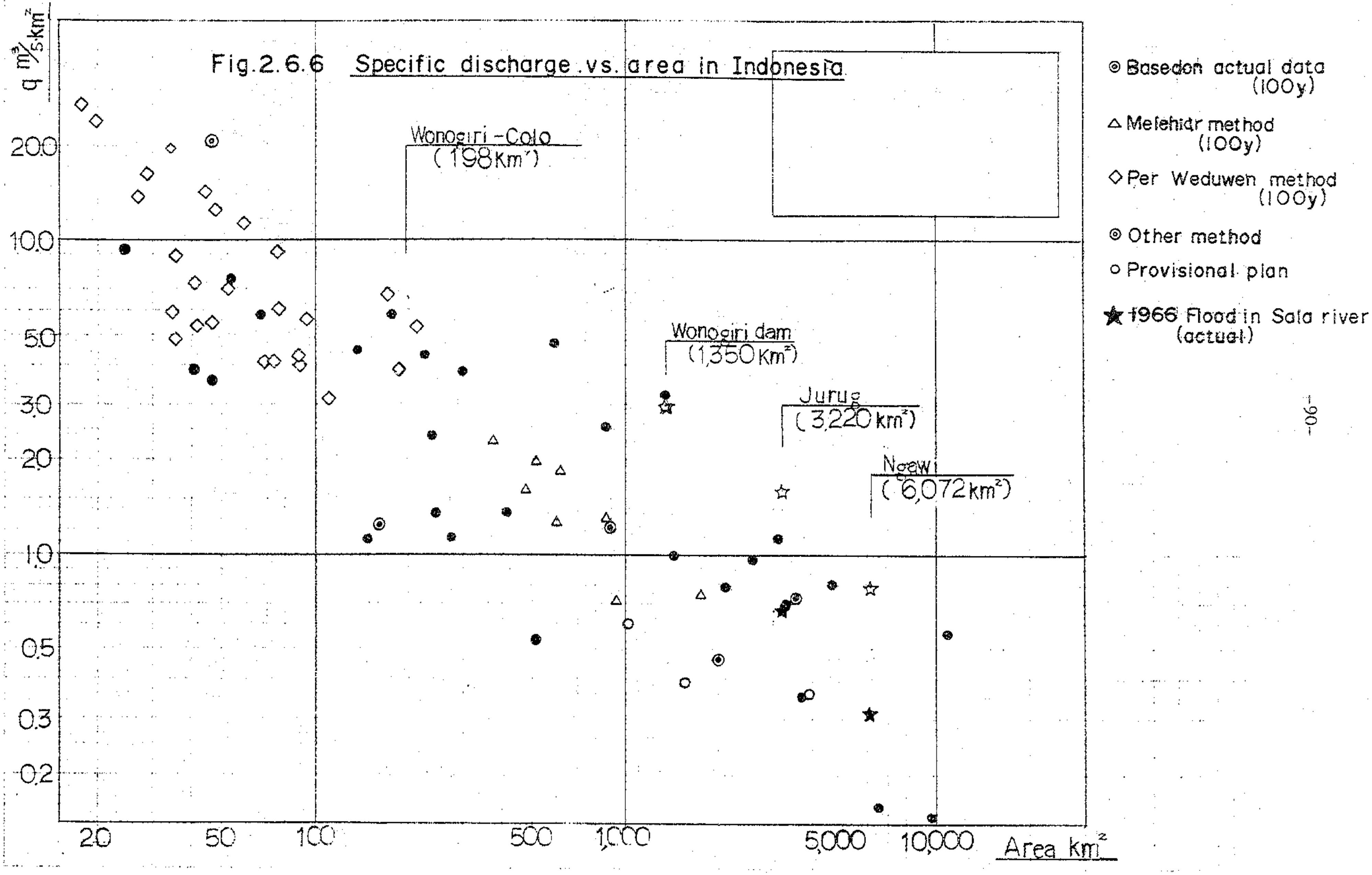


Fig.2.6.7 **Estimated maximum discharge in the 1966 flood.**

(after the dam-controled and the river improvement worked out)

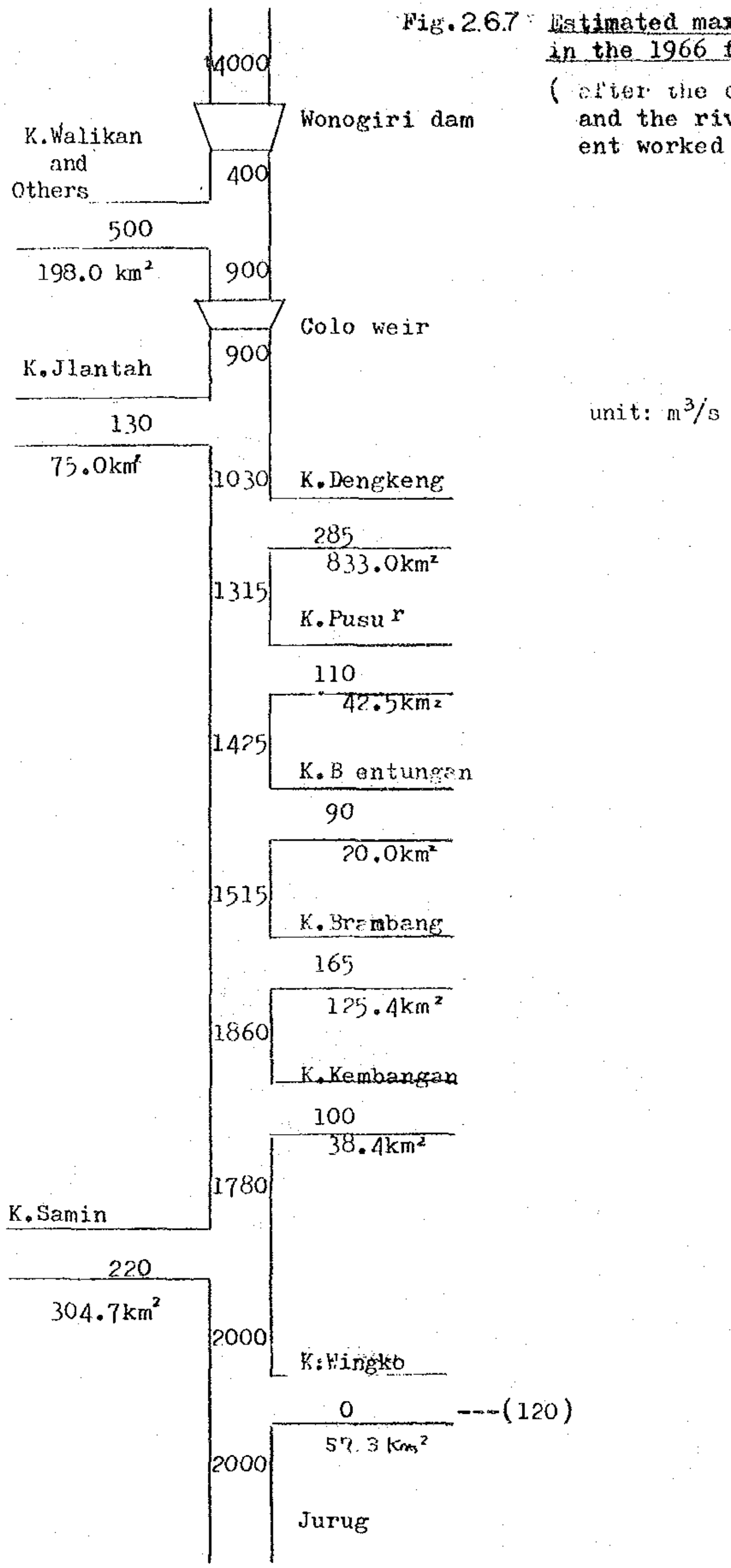


Fig. 2.6.8 Discharge distribution in case of 1,600 m³/s at Jurug.

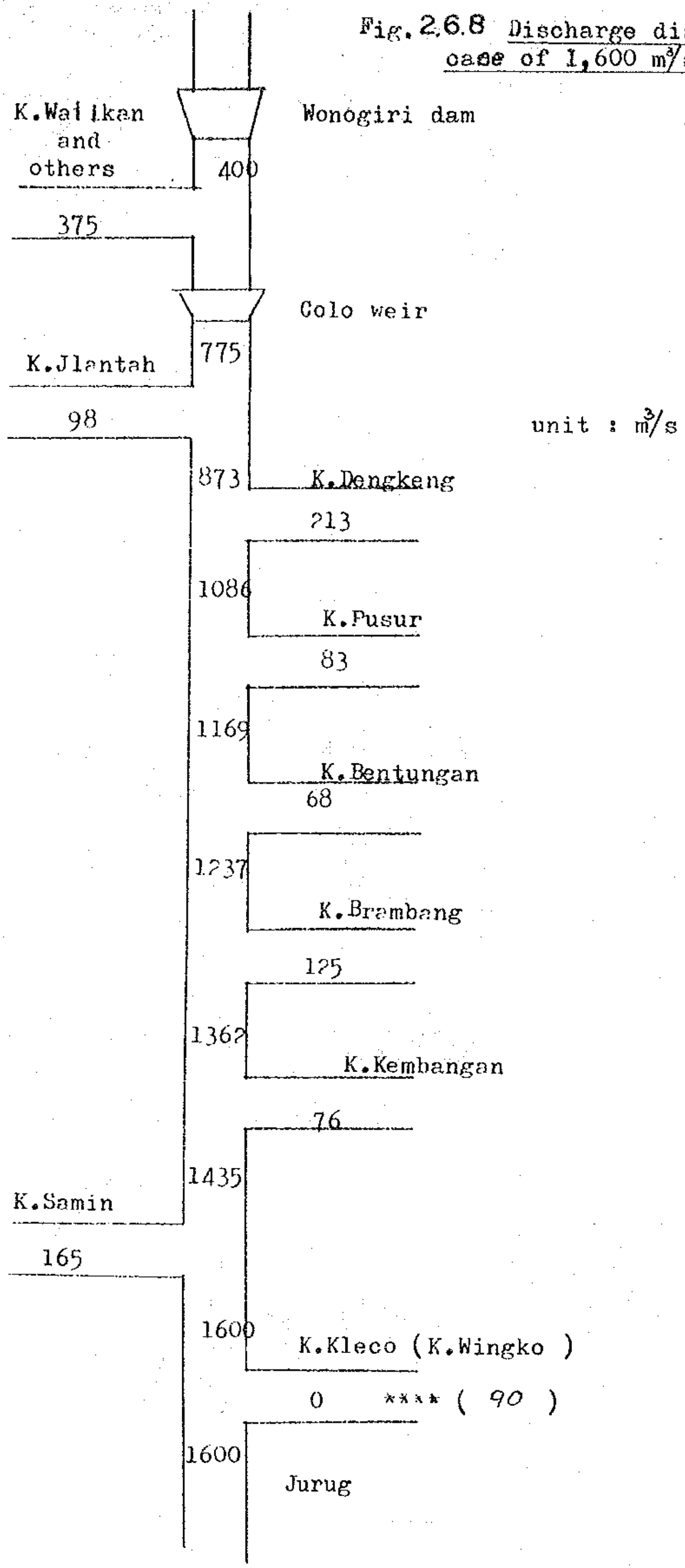


Fig. 2.6.9 Discharge distribution for 2-year flood at Jurug site.

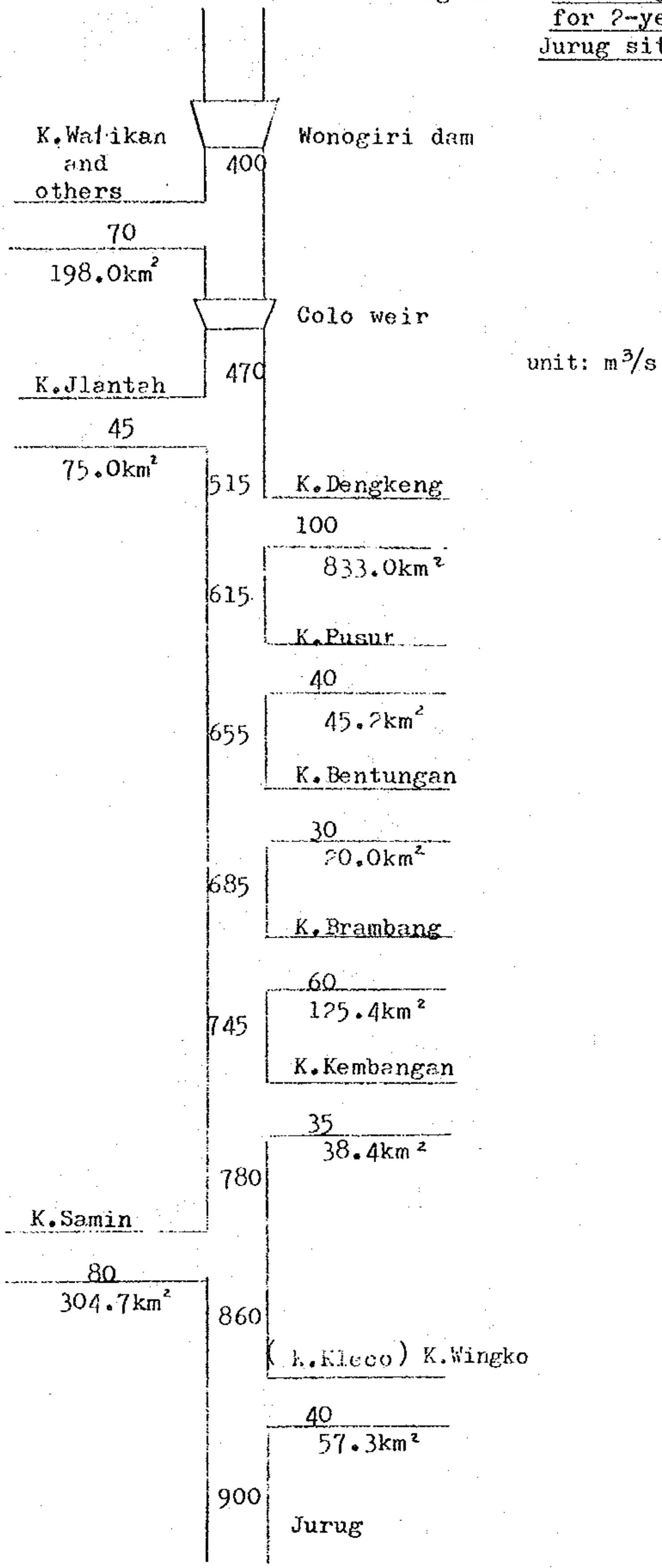


Fig. 2.6.10 Low water discharge distribution in case of 600 m^3/s at Jurug

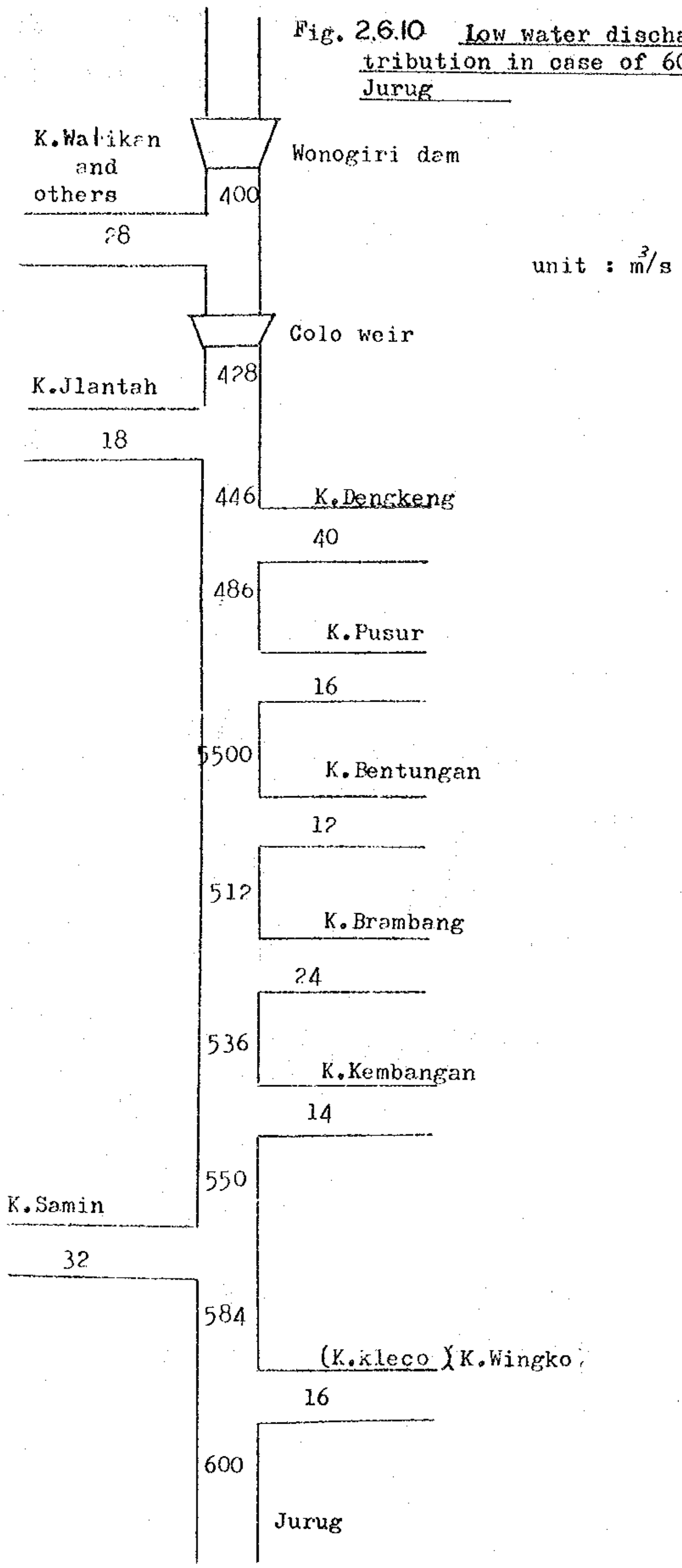


Table 2.6.1 Annual Maximum Discharge

(a) Juranggempal Site

No.	Year	Daily Mean (m^3/S)	Peak (m^3/S)	Estimated Peak (m^3/S)
1	1965	406	-	700*
2	1966	3,876	3,950	3,950
3	1969	1,019	-	1,650*
4	1968	796	1,280	1,280
5	1969	419	-	700*
6	1970	528	-	900*
7	1971	468	-	800*
8	1972	482	820	820
9	1973	651	1,140	1,140
10	1974	669	-	1,120*
11	1975	1,098	-	1,710*

(b) Jurug Site

No.	Year	Daily Mean (m^3/S)	Peak (m^3/S)	Estimated Peak (m^3/S)
1	1966	-	2,160	2,160
2	1968	938	1,540	1,540
3	1969	597	-	620*
4	1970	544	-	580*
5	1971	720	-	760*
6	1972	803	660	660*
7	1973	929	1,020	1,020
8	1974	822	950	950
9	1975	899	1,020	1,020

Note: * Values are estimated by the relationship between daily mean and peak discharge.

Table 2.6.2 Maximum Discharge at Jurug Site in the Several Construction Cases

(a) 1966 - Flood Case

Multiplied Ratio	Existing Condition (m^3/S)	Estimated Return Period (1/T)	Stage 1 (m^3/S)	Stage 2 (m^3/S)	Stage 3 (m^3/S)	Stage 4 (m^3/S)
1.50	2,830	1/500	2,390	2,013	3,305	7,628
1.00	2,156	1/40	1,650	1,316	1,996	5,240
0.50	1,281	1/5	886	1,874	1,165	2,363

(b) 1968 - Flood Case

1.50	2,048	1/50	2,110	1,734	2,158	4,061
1.00	1,540	1/10	1,310	1,297	1,568	2,440
0.50	894	1/2	865	854	976	1,290

Note: Case 1 = Main Coffer Dam + Existing river

Case 2 = Wonogiri Dam + Existing river

Case 3 = Wonogiri Dam + Improved river

Case 4 = Non-Wonogiri Dam Control + Improved river

Table 2.6.3 Probable Peak Discharge

(a) Juranggempal Site

N-Year	Peak Discharge (m^3/S)	Note
100	4,300	
50	4,000	
20	3,050	
10	2,450	
5	1,870	
2	1,030	

(b) Jurug Site

N-Year	Peak Discharge (m^3/S)				
	Existing Condition	Case 1	Case 2	Case 3	Case 4
200	2,500	2,400	2,000	2,750	6,600
100	2,300	2,200	1,800	2,400	5,500
50	2,030			2,150	4,700
40	2,000	1,800	1,550	2,050	4,600
20	1,750	1,360	1,350	1,750	3,600
10	1,500	1,170	1,170	1,500	2,900
5	1,250	1,000	1,000	1,250	2,200
2	900	800	800	900	1,300

Note: Case 1 = Main Coffer Dam + Existing river
Case 2 = Wonogiri Dam + Existing river
Case 3 = Wonogiri Dam + Improved river (without retarding basin)
Case 4 = Non-Wonogiri Dam + Improved river (without retarding basin)

Table 2.6.4 Discharge in Tributary

(a) Minimum Discharge Capacity in Tributary

Tributary	Area (km^2)	Estimated Minimum Discharge Capacity (m^3/S)
K. Walikan	198	500
K. Jlantah	75	100
K. Dengkeng	833	270
K. Pusur	43	100
K. Bentungan	20	80
K. Brambang	125	150
K. Kembangan	38	90
K. Samin	305	200
(K. Wingko)	(57)	(120)
Total	1,637	1,490

(b) Proposed Discharge and Estimated Discharge in Future

Tributary	Area (km^2)	Proposed Discharge of Tributary (m^3/S)	Estimated Discharge in Future (m^3/S)
K. Walikan	198	500	500
K. Jlantah	75	130	350
K. Dengkeng	833	285	830
K. Pusur	43	110	290
K. Bentungan	20	90	220
K. Brambang	125	165	410
K. Kembangan	38	100	280
K. Samin	305	220	580
K. Wingko	(57)	120	320
Total	1,637	1,600	3,655

Note: The cross sections of the tributaries are designed according to the proposed discharge considering the estimated discharge in future.

3. RIVER IMPROVEMENT WORK

3.1 EXISTING RIVER CONDITION

3.1.1 Profile, width, and discharge capacity of the river

The profile of the existing main river is 1/2,000 in the downstream of the objective reach and 1/1,450 in the upstream reach as shown in Fig. 3-1-2.

The river width and the discharge capacity are shown in Fig. 3-1-3; it shows that its mean width is 100 m and a mean discharge capacity is 500 m^3/s in the proposed reach.

The existing river meanders severely especially at the confluences with K.Jlantah, K.Dengkeng and K.Samin. Due to such meandering, the river length along the existing river in the proposed reach is about 55 km, while the existing axis line of meander has a length of only 33 km.

3.1.2 River water use

The use of the river and its water has been confined to the conveniences of the people living along the river, mainly in:

reach	mean width (m)	mean slope	mean discharge capacity (m^3/s)
Lower Jurug	-	1/2,800	-
Jurug - K.Samin	100	1/3,300	700
K.Samin - K.Dengkeng	90	1/2,600	500
K.Dengkeng - K.Jlantah	80	1/2,600	300
K.Jlantah - Nguter	110	1/2,600	450
Upper Nguter	-	1/1,200	-

Note: Discharge capacity is calculated for the bank-full stage

1) Irrigation: Some water is used for pumping irrigation; 21 small pump stations, with the capacities ranging from 0.2 m^3/s to 0.5 m^3/s, are installed along the river.

2) Fishing: Some people catch fish in the river. But due to the fluctuation of its discharge and the water with much suspended-load, the kinds and the quantities of the living fish are very limited.

3) Sand collection: Though the river-water contains much suspended-load, the river bed is formed by sand. Therefore, good qualified sand is being collected at some places along the river, particularly at around Nguter bridge, Bacem bridge, Jurug railway bridge and K.Brambang, for sale as construction.

3.1.3 Existing riparian structures

Existing riparian structures along the reach for the proposed river improvement work are shown in Table 3.1.1 and their locations are shown in DRG. No.RW 57 and in Fig. 3.1.4.

All of the road bridges and railway bridges in the reach are old. The discharge capacity at each bridge site is shown in Table below; it is rather small especially at the upstream reach bridge.

Location	A(m^2)	I	n	B(m)	V(m/s)	Q(m^3/s)
Jurug Road Bridge	1,230	1/3,300	0.035	175.0	1.82	2,200
Jurug Rail Road Bridge	1,350	"	"	180.0	1.90	2,500
Mojo Rail Road Bridge	1,220	"	"	150.0	1.79	2,180
Bacem Road Bridge	580	1/2,600	"	120.0	1.60	900
Nguter Road Bridge	500	"	"	110.0	1.54	750
Nguter Rail Road Bridge	530	1/1,200	"	110.0	2.33	1,235

Note:
A: area of cross-section
I: slope of river bed
n: Manning's n
B: Bridge length
V: velocity of flow
Q: discharge

There are 21 small pump stations for pumping up irrigation water from the Sala river. The pump capacity is ranging from 0.2 m^3/s to 0.5 m^3/s.

Among other riparian structures, there are ferries, groines and improved levees along the reach but their scale is small.

Along the tributaries of K.Dengkeng and K.Jlantah, improved continuous levees, sluiceways, intake weirs, and boat stations do exist.

3.1.4 Sediment

The sediment load is estimated at 1,700 $m^3/year/km^2$ in terms of the suspended load and at 21 $m^3/year/km^2$ in terms of the bed load.

The suspended load is functioning as wash load and it has very little influence on the river bed formation.

Therefore, attention should be paid to the bed load rather than to the suspended load.

3.1.5 Flood damage

The annual mean flood damages in the Surakarta area which forms a part of the project area, and in the Sragen area located on its downstream are as given below (see Item 3.3. and 3.4):

(Flood damage)

Surakarta area .... 6.56 x US$$10^6$/year

Sragen area .... 3.56 x US$$10^6$/year

Fig - 3.1.1 General Map of the project reach

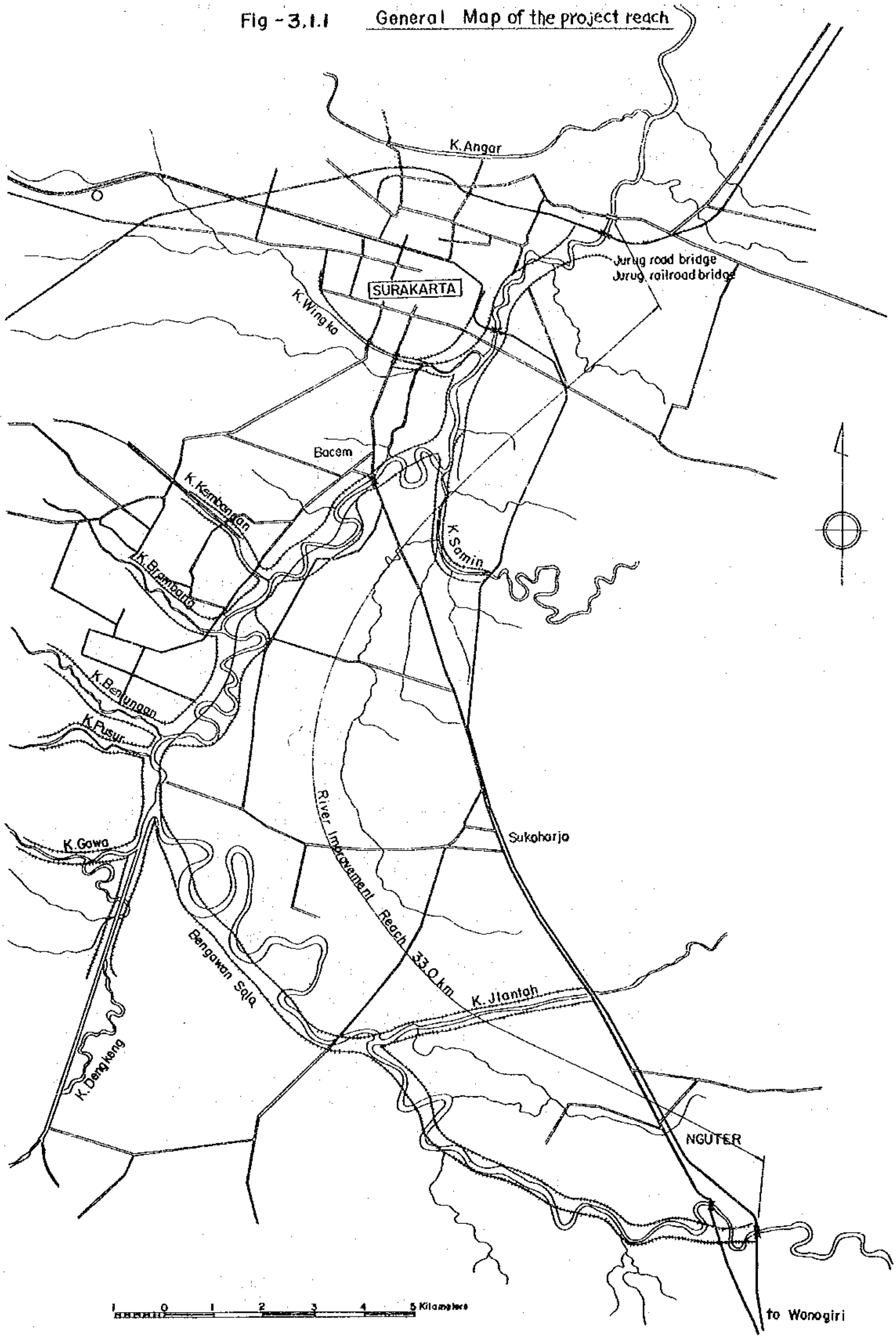
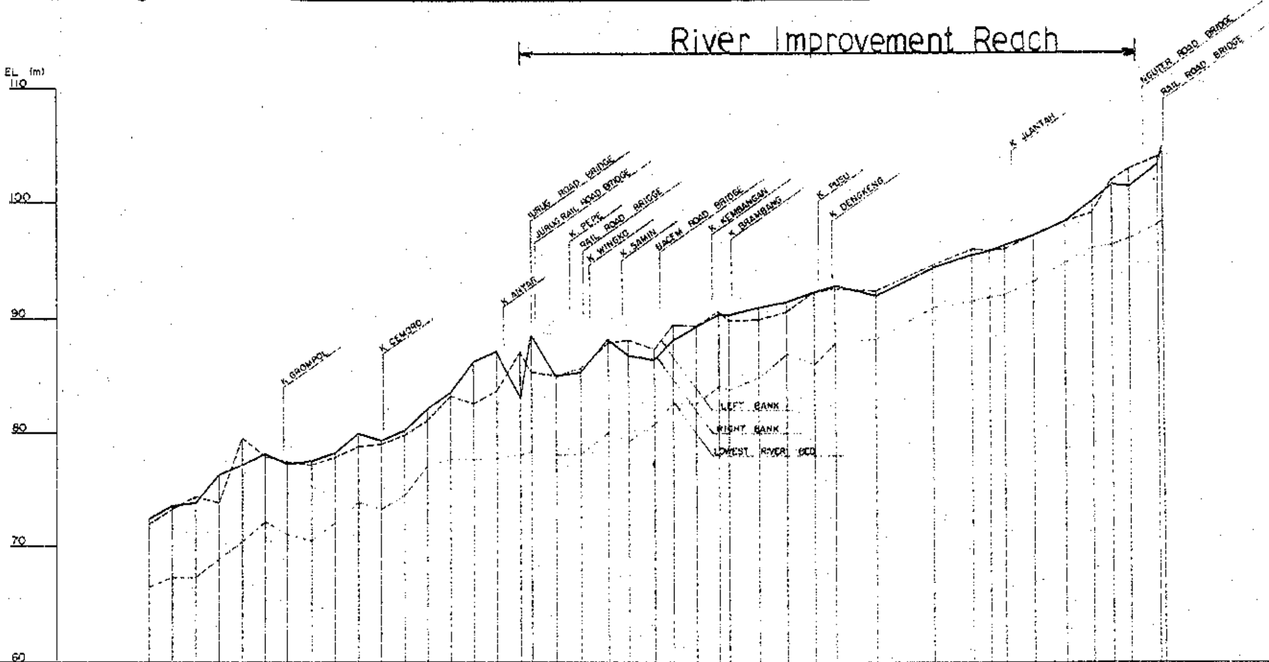


Fig-3.1.2 PROFILE of EXSISTING RIVER COURSE



Longitudinal slope: 1/3,300 ; 1/2,600

Cross section number: Original (No.)	Cross section number: Modified (No.)	Unit distance (m)	Accumulated distance (m)	Present elevation (EL m): Lowest river bed	Present elevation (EL m): Left bank	Present elevation (EL m): Right bank
422			-33.038	66.51	71.97	72.41
424		2,000	-31.038	67.33	73.26	73.56
426		2,037	-29.001	67.35	74.23	73.81
428		1,993	-27.008	68.95	73.82	76.34
430		2,022	-24.986	70.41	79.64	77.12
432		2,001	-22.985	72.06	77.90	78.10
434		1,914	-21.071	70.35	77.41	77.31
436		2,038	-19.033	70.45	77.15	77.52
438		2,024	-17.009	71.99	77.83	78.15
440		1,962	-15.047	73.76	78.74	79.92
442		2,021	-13.026	73.19	77.95	79.28
444		2,004	-11.022	74.41	79.70	80.12
446		1,979	-9.043	77.05	81.12	82.00
448		2,032	-7.011	77.55	83.11	83.48
450		2,008	-5.003	77.55	82.51	85.22
452		1,981	-3.022	77.62	83.61	87.12
454		2,022	-1.000	77.95	86.95	83.07
136	1	1,000	0	78.33	85.36	88.62
109	2	2,100	2.100	78.00	85.09	84.59
88	3	2,100	4.200	78.12	85.56	85.29
60	4	2,400	6.600	79.99	87.91	88.08
35	5	1,800	8.400	79.26	88.10	86.82
14	6	2,200	10.600	80.69	87.50	86.40
259	7	1,600	12.200	82.61	89.42	88.13
251	8	2,000	14.200	82.50	89.27	89.26
234	9	2,000	16.200	84.12	90.61	90.42
227	10	800	17.000	83.76	89.66	90.43
203	11	2,600	19.600	84.77	89.55	90.97
179	12	2,400	22.000	86.65	90.54	91.41
150	13	2,400	24.400	85.95	92.23	92.27
129	14	1,800	26.200	87.65	92.61	92.83
97	15	3,500	29.700	88.21	92.33	92.01
172	16	4,900	34.600	90.88	94.55	94.37
140	17	3,400	38.000	91.51	95.81	95.33
126	18	1,500	39.500	91.80	95.72	95.70
114	19	1,300	40.800	92.01	95.95	96.18
90	20	2,500	43.300	93.16	97.08	97.13
64	21	2,700	46.000	94.73	98.25	98.19
225	22	2,400	48.400	95.96	99.14	100.06
196	23	1,700	50.100	96.26	101.56	101.46
157	24	1,500	51.600	96.79	102.84	101.55
96	25	2,500	54.100	98.05	103.93	103.13
88	26	400	54.500	98.37	104.31	103.48

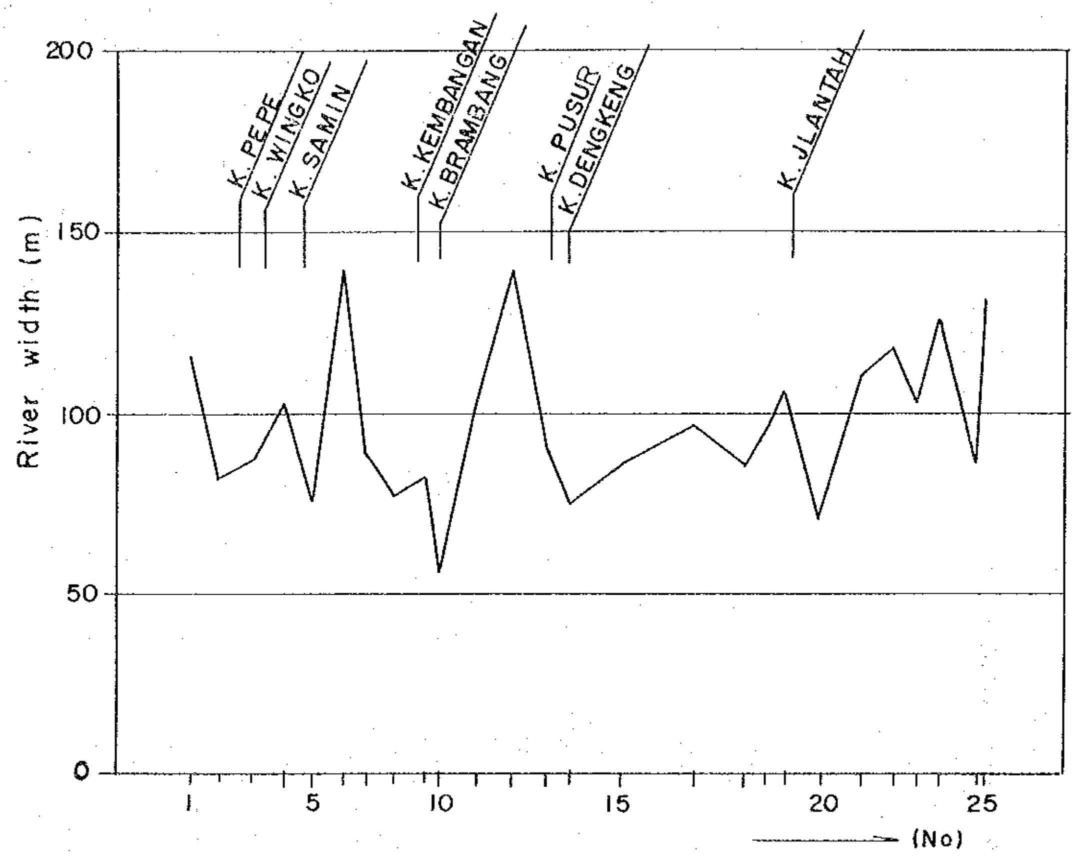
SCALE
H 0 2 4 6 8 10 (×10³) m
V 0 2 4 6 8 10 m

WONOGIRI IRRIGATION AND UPPER SALA RIVER IMPROVEMENT INDONESIA

PROFILE of PRESENT RIVER COURSE

Date: June 30, 1976 | DWG. No. WR. 5

Fig.3.1.3 Exsisting condition of River



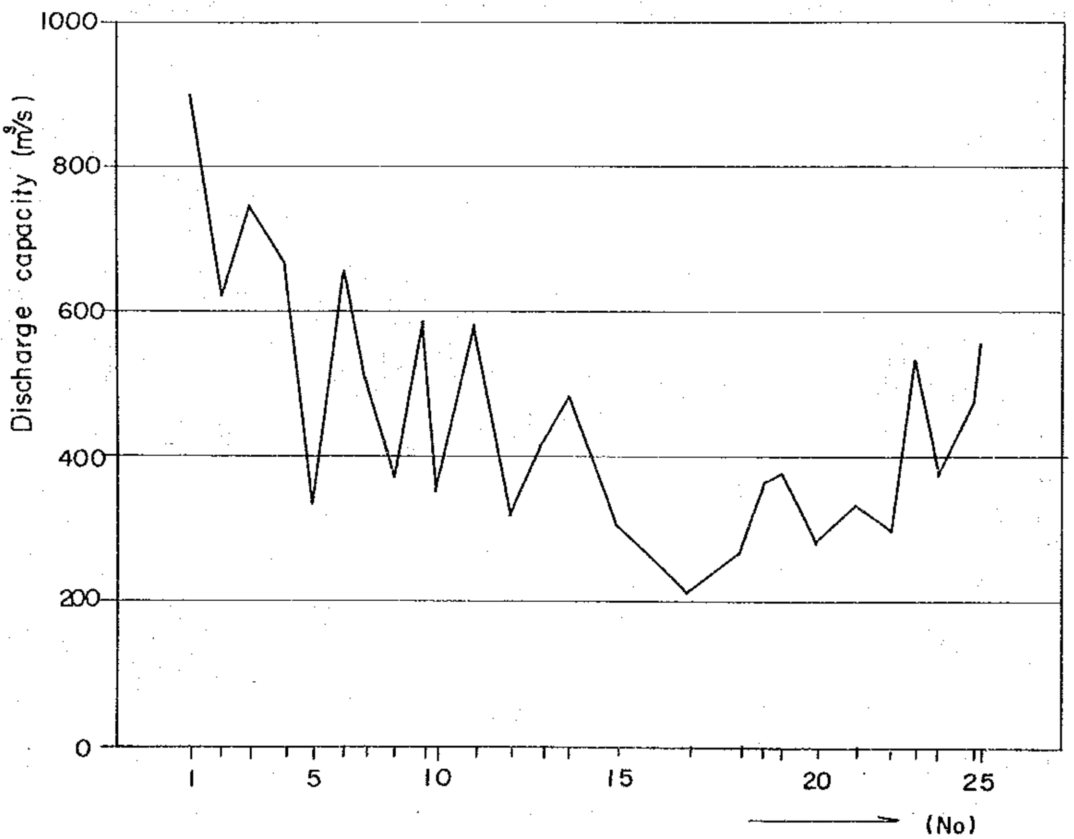


Fig-3.14(a) LOCATION of EXISTING RIPARIAN STRUCTURE

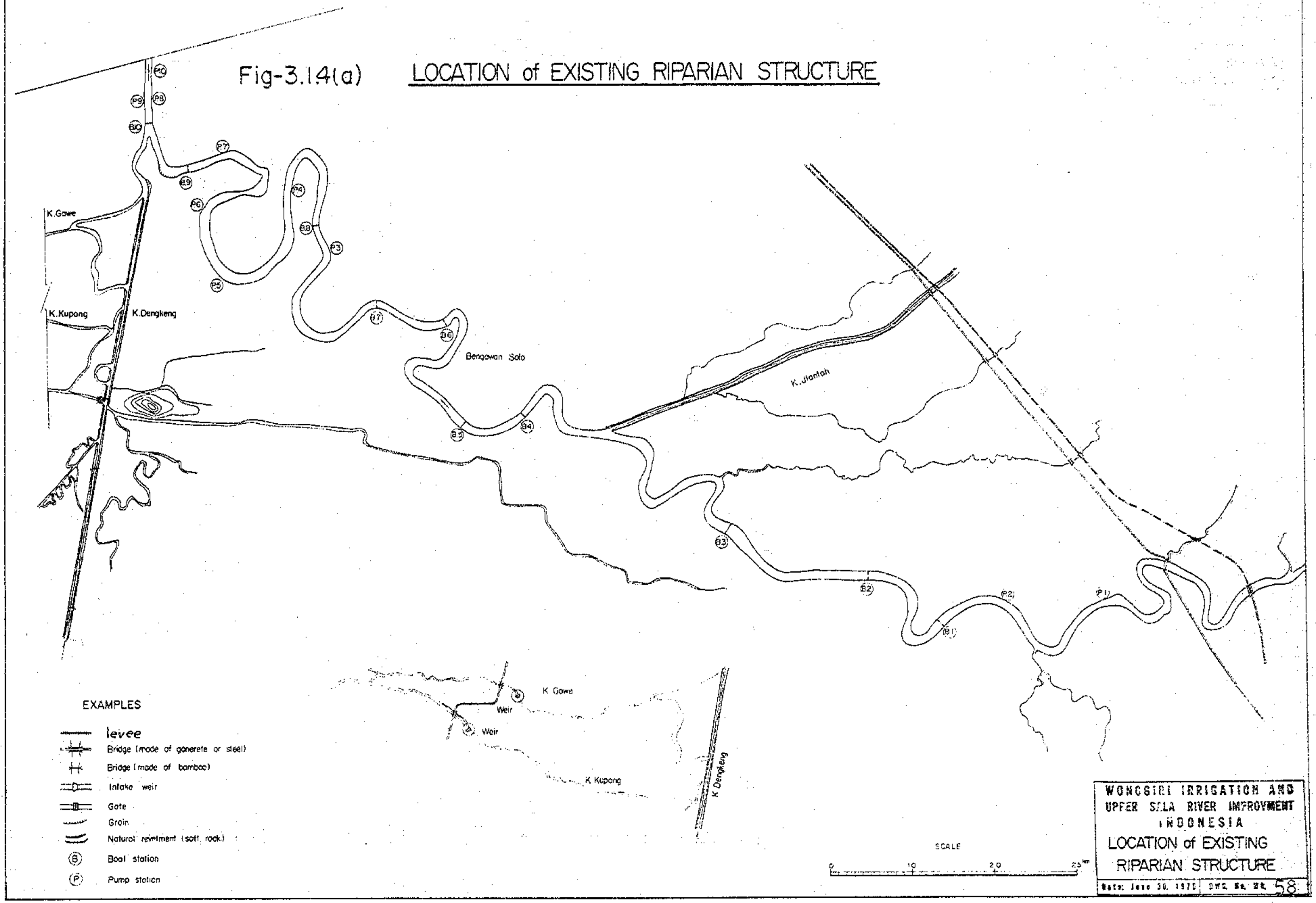


Fig - 3.1.4 (b) LOCATION of EXISTING RIPARIAN STRUCTURE

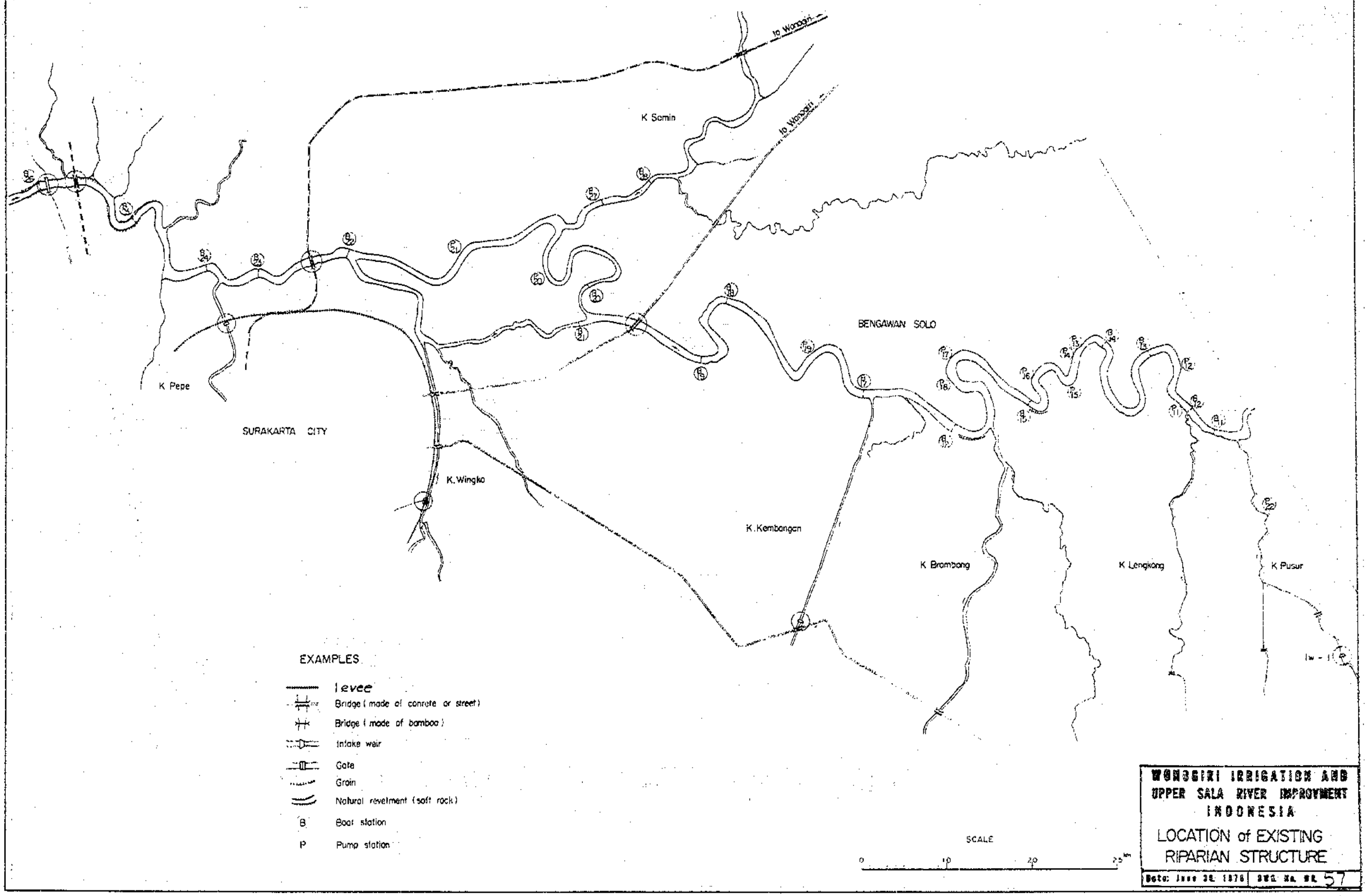


Table 3.1.1 (a) Existing Riparian Structure

name of river	kind of stucture	place	note	
Sala	bridge-1		Nguter railroad bridge	l=120.0m
	bridge-2		Nguter road bridge	l=110.0m
	bridge-3		Bacem road bridge	l=120.0m
	bridge-4		Mojo rail road bridge	l=150.0m
	bridge-5		Jurug rail road bridge	l=180.0m
	bridge-6		Jurug road bridge	l=170.0m
	groin	2	Made of coconut tree	
	levee	1	L = 7,000m (Surakarta circle levee)	
	pump station	21	q - $0.2m^3/s$ - $0.5m^3/s$ admin: strated by each village	
	boat station	26	Made of bamboo or cutting of river bank	
	natural revetment		Outcrop of soft rock near Surakarta city	
K.Jlantah	levee		Continuous levee	l = 4.0km h = 1m - 2m
	bridge	2	Made of bamboo	
	levee		Continuous levee	l = 8.0km h = 2m - 3m
	gate	1	(h) 2.50m x (b) 2.25m x 4	
	bridge	2	Made of bamboo	
	groin	1	Made of coconut tree	
K. Kupang	intake weir	1	b = 40m	

Table 3.1.1 (b) Existing Riparian Structures

K. Gawe	intake weir	1	$b = 20^m$
K. Pusur	intake weir	1	$b = 10^m$
K.Kenbangan	intake weir	1	$b = 8^m$
K.Samin	boat station	2	made of bamboo on the cutting of river bank
K.Wingko	gate	1	(h) 4.0^m x (b) 2.0^m x 2
K.Pepe	gate	1	(h) 3.0^m x (b) 2.0^m x 10
	pump station	1	for drainage, under construction

3.2 RIVER IMPROVEMENT WORK PLANNING

The river improvement project proposed for the 33-km Nguter-Jurug reach is planned for the 1966-flood, the worst-ever flood in the Upper Jurug area, with an estimated peak discharge of 2,000 m^3/s or the 40-year flood discharge under the Wonogiri dam flood control.

First of all, 2 cases as detailed below were investigated.

1) River channel having a discharge capacity of 2,000 m^3/s at Surakarta (hereinafter called 2,000 m^3/s-case)

2) River channel having a discharge capacityof 1,600m^3/s at Surakarta (hereinafter called 1,600 m^3/s-case). The low water discharge was determined upon consideration of the progressive contruction.

As the result of such studies, the 2,000 m^3/s-case has been adopted for the proposed river improvement plan, since the project area should be protected from the ever-worst flood and the construction cost does not differ much.

In the proposed plan, 2 retarding basins are suggested so that the improved channel will not exert bad influences on the down-stream reach, also,3 emergency inundation areas will be provided against the floods over 2,000 m^3/s.

3.2.1 General Conceptions of Planning

The river improvement work has been planned in full consideration of the technical, social and economic aspects as mentioned below:

1) Technical aspect

a. As the existing river is meandering disorderly and heavily in many parts of its stream, it is impossible to work out the river improvement plan while keeping the river flowing down along the existing channel. Short-cutting should be planned at the parts where the river meanders too heavily.

b. On the other hand, it is difficult for a straight river channel to maintain and preserve its banks from erosion. The reason is that it is quite difficult to find out flow attacking points and construct protective revetment in straight channels. The proposed channel, therefore, should be aligned so that the river water would flow with a gentle mandering.

c. Taking the slope of compensation into consideration, the longitudinal profile of the proposed river channel has been planned as low as possible to keep the water-level low. As a result of this, the landside water will be easily drained out to the main-stream thus reducing the actual damage due to landside water as well as the potential damage due to the flowing water confined in the levees.

d. As the seasonel fluctuation of the river flow are extremely big, a composite cross section has been proposed to stabilize. The low-water channel in medium and small flood taking frequency of occurrence of flood into consideration.

e. At the time of flood, a large scale bank erosion may take place, scale of which reaches, sometimes, to several tens meters. Therefore, the river width should be made as large as possible within the economically justifiable limits.

f. The proposed river channel should be designed so that its semiment load can be maintained dynamically.

g. In the river improvement planning, careful attention should be paid to avoid the negative influences, due especially to medium and small floods, on the Sragen area in the downstream of Surakarta.

h. After construction of the proposed river channel, a system for its efficient maintenance and smooth administration will be required. From this view-point, an appropriate road system will be required for the maintenance and administration of the river channel, and such road system will also be useful for flood prevention activities in case of emergency.

2) Socio-economic aspect

a. There are many towns and villages along the course of the existing river although they are subject to flood damages caused by the flooding of the Sala river every year. Naturally the river improvement work will protect them from such disasters, but for such purpose the people living in the district will have to be moved. Therefore, the resettlement of the population must be minimized in the alignment planning of the river.

b. The livelihood of the people living along the river depends much on the river, for example, they use the water for bathing, washing, fishing, and for agricultural purposes. The river, after completion of the river improvement work, may fail to provide them with the same advantages as before; it is, however, necessary to avoid giving and direct disadvantages to them.

c. The river improvement plan studied this time includes a considerable short-cut. In this case, even in areas where the total move of the people is not required, the villages, administrative districts, and farm lands may be separated by the new channel. Therefore, special care must be taken to avoid social disorder due to the river improvement.

3.2.2 Design Flood

The design flood for the river improvement plan has been fixed at 2,000 m^3/s-case based on the 1966-flood, and that of the alternative plan has been scaled down to 1,600 m^3/s-case.

The discharge distribution of the 1966 flood is shown in Fig. 2.6.7. For the design discharge distribution used in the river improvement plan of the 2,000 m^3/s-case, the modified figures shown in Fig. 3-2-1 have been used. The modified discharge distribution is larger in value than that of the actual flood of 1966.

This is because the possibility of larger discharge than the design scale from the mountain river like K. Walikan has been taken into consideration. For the purpose of river bed stabilization, a flow discharge of high frequency is used, generally probable frequency of once or twice a year, in the proposed design low-water discharge.

The discharge of the low water channel for the 2,000 m^3/s-case has been decided at 900 m^3/s which corresponds to 2-year flood at Jurug site. The discharge distribution for low water channel is shown in Fig. 3-2-1.

900 m^3/s discharge may seem rather large for the design discharge of 2,000 m^3/s, but in considration of the fact that the major bed is planned to be used as the farmland and that the average ponding frequency of the farmland and the major bed is made once in two years, the low water channel of 900 m^3/s discharge is rather small.

The discharge of low water channel for the 1,600 m^3/s-case has been at employed 600 m^3/s. 600 m^3/s corresponds to 1.2-year flood at Jurug site. The discharge distribution in that case is shown in Fig. 3-2-2. Design discharge of the tributaries is also shown in Fig. 3-2-2. The design discharge of the tributaries is based on specific discharge of rivers in Indonesia as practically no hydraulic and hydrological data are available.

3.2.3 Design High Water Level

While it is advisable that the design high water level of the river channel is planned as low as possible to reduce potential damages, the river improvement work would require much excavation which is rather costly, if the design high water level should be set at relatively low elevation. Although the design high water level is generally set at the highest flood level recorded in the past, it would be too high (4.5m above the landside height, on an average) since it hampers the drainage of landside water and endangers the safety of the levees.

Therefore, the design high water level proposed in the plan may better be set at lower elevation, from the economic view-point also.

For the safety of the levees, the design high-water level has been set at less than 2 meters above the landside height at all sections along the river channel except in the reaches coming under the influence of the backwater which is caused by the downstream narrow channel sections and the existing bridges, such as the Jurug Road and Railway Bridges and the Mojo Bridge which are being left untouched owing to the tremendous re-construction costs of the existing bridges.

3.2.4 Alignment of River

The existing river meanders heavily at several locations. To secure the stability of the proposed river channel, it is strongly recommended that the low water channel meanders as gentle as practically possible in the proposed design river channel alignment.

For the subject channel section, three alternative plans were formulated, considering channel alignment, amount of excavation, convenience of construction and removal of houses, the alignment of the river as shown in Fig. 3-2-6 was selected as most suitable for the proposed river channel, based on technical, economical and social viewpoints.

The pitch of the existing meandering ranges from 500 to 1,000 meters. The most suitable river channel alignment usually can be obtained empirically.

Anderson gave a formula which has a relationship between the pitch of meandering and Froude number as follows.

$$\frac{L}{\sqrt{A}} = 72Fr^{\frac{1}{2}}$$

where:

L: meandering pitch of river (m)

A: Cross sectional area of river (m^2)

Fr: Froude number

The suitable pitch of meandering for the subject river will be 500 to 1,000 meters according to the formula mentioned above.

Although it was originally intended to work out the river improvement plan in accordance with the existing river elements, it was found out that short-cutting intended to moderate the excessive meandering of the river was indispendable at many locations. The total short-cutting length is 13km out of the entire river course of 33km. As this area is densely populated, short-cutting at the heavily meandering sections would cause the removal of the inhabitants or splitting of the existing villages.

For reference, given below are the comparative test results at the confluence of the K. Jlantah, K. Brambang and K. Samin where meandering is most prominent.

Three alternative alignment plans are considered in each reach as shown in Figs. 3-2-3, 3-2-4 and 3-2-5.

Naturally, the earth works occupy the main part of construction works and much part of the construction cost of this improvement work, taking alignment of river channel, convenience of construction and removal of houses into consideration.

Therefore, a plan which needs the minimum amount of excavation in the three plans is adopted in the proposed plan in each reach considering also the suitable alignment of the river course. Consequently, the Plan-A is adopted for the three reaches.