5.5 Probable Discharge of Main Rivers

Run-off analysis in the K. Progo and the K. Opak basins is carried out to estimate probable peak discharge at several representative sites along the rivers using the Storage Function Method.

(1) Run-Off Model

The run-off model of the Storage Function Method should express actual flood characteristics correctly under given rainfall conditions. The reliability of the run-off model is checked in terms of a comparison of the actual hydrographs at the gauging stations with the computed hydrographs using the run-off model tests. The K. Progo and K. Opak basins are divided into 6 and 3 sub-basins respectively based on the constitution of the river systems, the topographic features of the river basins and location of gauging stations.

Furthermore, the sub-basins and the one-channel models are arranged in the run-off model of K. Progo and K. Opak respectively. They function as an existing river channel which causes reduction and delay of arrival time of peak discharge.

The run-off models of the two river systems are shown in Fig. 14.

(2) Storage Function

Storage function for sub-basin and channel models are given as follows:

(Sub-Basin)

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

$$(S_2 - S_1)/\Delta T = \bar{Y} - (q_2 + q_1)/2 \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (5.4)$$

$$Q' \ (t = t - te) = q \cdot A/3.6 + Q_B$$

(Channel)

$$S_r = KQ^P - T \cdot Q$$

$$(S_2 - S_1)/\Delta T = \bar{Y} \ (I_1 + I_2)/2 - (Q_1 + Q_2)/2 \quad \ldots\ldots\ldots \quad (5.5)$$

$$Q \ (t = t - te) = Q$$

where:

S_b : Storage quantity of rainfall over sub-basin in mm

K & p : Constants of storage function

q : Outflow from sub-basin in mm/hr

$\bar{Y}$: Rainfall intensity in T over sub-basin in mm

ΔT : Routing period (= 3,600 sec.)

A : Catchment area of sub-basin in sq.km

QB : Base flow of sub-basin in cu.m/sec.

T_e : Basin lag of sub-basin in hr

Q' : Discharge from sub-basin in cu.m/sec.

S_r : Storaged quantity of flood water in channel model in cu.m/ΔT

I : Inflow to channel model from upstream basin cu.m/sec.

Q" : Outflow from channel model in cu.m/sec.

and sub-script numbers "1" and "2":

"1" denotes time at "t"
"2" denotes time at "t + ΔT"

(Base Flow)

The discharge record at Duwet, Santolo and Kranggan sites shows a specific discharge of 0.1 cu.m/sec./sq.km on the average just before the flow condition of a flood starts.

Accordingly, the specific discharge of 0.1 cu.m./sec./sq.km was adopted to estimate the base flow- "OB" for both sub-basins of K. Progo and K. Opak. "QB" is given a value by multiplying the specific discharge of the area of each sub-basin.

(Rainfall Loss)

The original storage function includes the term of rainfall losses; however, the rainfall losses are not included in the utilized storage function because the rainfall losses caused by infiltration are not significant when the ground is saturated and the continuous rainfall in rainy season would certainly cause high ground saturation. Accordingly, the rainfall loss was taken as zero or excluded from the computation in this case.

(Constants K and P of Storage Function)

The constants included in the storage function may be estimated with the following equations. In the equations, a rectangular plate and a rectangular aqueduct (both of which are fixed in slope) are assumed to be a sub-basin and a channel model respectively. The flow over the plate and in the aqueduct is assumed to obey the Manning's mean velocity formula.

$$K = 4.6\ (N/\sqrt{Is})^{0.6} + 0.446^{0.4}(n/\sqrt{Is}) \quad \text{..... Sub-Basin (5.6)}$$

$$P = 0.6$$

$$K = L\ 6^{0.4}\ (n/\sqrt{Ir})^{0.6}/3.6 \quad \text{.............. Channel ..(5.7)}$$

$$P = 0.6$$

where:

N	: Manning's "n" value on slope of sub-basin
n	: Manning's "n" value in channel model
Is	: Mean slope of sub-basin after replaced with plate
Ir	: Mean slope of channel model after replaced with aqueduct
B	: Mean width of sub-basin in Km after replaced with plate
b	: Mean width of channel model in m after replaced with aqueduct
L	: Stretch of channel model in Km after replaced with aqueduct

The values "Is", "Ir", "B", "b" and "L" are determined by the topographic features of the sub-basins. The "n" value may be assumed as n = 0.045 considering the irregular figure of river channel flowing through the mountain valley. Accordingly, the "N" value is the control factor of the run-off model. The optimum "N" value which can compute hydrographs similar to actual flood hydrographs will be selected using the run-off model test.

(3) Run-Off Model Test

The hydrological data (hyetographs to be given in run-off model and hydrographs to check the reliability of the run-off model) are as follows:

(Hyetograph)

The hydrograph of the two storm rainfalls, November 23 and 26, 1976 floods, were adopted as the hyetographs to test the suitability of the run-off model. The hourly rainfall records of the floods were used as hourly rainfall patterns over the sub-basins for the run-off model located adjacent to the rainfall station. The hourly rainfall figure of the sub-basins was adjusted to the mean of daily rainfall during the floods which was observed in the sub-basins by multiplying an appropriate rate to make the total hourly rainfall equal to the mean of the daily rainfall.

(Discharge Hydrograph)

The actual discharge hydrographs of the two floods was prepared by applying the stage records of Kranggan and Duwet sites along the K. Progo to their rating curves. The peak discharge was estimated at 705 m^3/s and 695 m^3/s for the November 23 and 26,1976 floods at the Duwet site respectively.

However, there is no available discharge hydrographs in the Opak river due to the absence of any stage gauging station along the river.

(Simulation)

The run-off simulation was carried out for K. Progo run-off model. The computation was performed for two floods under the several sub-basin constants of "K" which were given by different "N" values in Eq. (5.6). An "N" value which produced computed hydrographs similar to the actual discharge hydrographs was adopted as the optimum. Then the optimum "N" value was used in K. Opak run-off model to determine its sub-basin constant.

The constants from the run-off model thus obtained are shown in Table 10. The computed hydrographs using the constants together with the actual discharge hydrographs are shown in Fig. 15 and Fig. 16.

(4) Probable Discharge Estimation

The probable discharge at significant sites along K. Progo and K. Opak was estimated based on the relation between daily rainfall over the basin and the peak discharge computed through the run-off models.

The probable discharge for specified return period may be estimated as the computed peak discharge applying the following hyetographs to the run-off model where the total hourly rainfall of a day is equivalent to the daily rainfall corresponding to the return period. The hyetographs were produced by multiplying the constants to the two hyetographs utilized in the run-off simulation. The estimated probable discharges based on the hyetographs are shown in Table 11.

The probable discharge estimated by run-off simulation was checked by the Rational Formula at the river mouth of K. Progo and K. Opak. The estimation of the peak discharge was computed for 100- and 50-year of return periods respectively by using the same procedure described in section 5.4. The results are given in Table 11 (3), and they coincide with the estimated probable discharge through the run-off simulation.

The coincident of probable discharge calculated in two different ways verifies the reliability of the probable discharges from the run-off simulation. Therefore, the probable discharges by run-off simulation were adopted as the probable discharges in this study. The discharge distributions corresponding to several return periods along K. Progo and K. Opak are shown in Fig. 17.

Table 7 Flow Regime at Duwet Site K. Progo

Unit: cu.m/s

Year	Discharge in order from the largest				
	1st	95th	185th	235th	355th
1969	408	122	48	18	5
1970	265	97	62	22	10
1971	268	112	70	35	11
1972	294	104	43	11	3
1973	264	124	88	55	28
1974	256	117	73	45	23
1975	352	159	114	56	18
1976	228	88	43	13	3
Mean	292	115	68	32	13

Table 8 Frequency Analysis of Discharge at Weirs

Return Period	Probable Discharge	
	Blawang Weir (Opak River)	Krasak Weir (Krasak River)
2-year	330 cu.m/s	170 cu.m/s
10	640	340
20	780	420
50	960	520
100	1,100	580

Table 9 Estimated Peak Discharge of 100-year Flood of Tributaries (1)

RIVER	(1) K. PABELAN												
Site	Upper Part	Con-fluent 1	C.D 1	C.D 2	Con-fluent 2	Con-fluent 3	N.R.	No.2	Con-fluent 4	N.R 2	No.3	Con-fluent 5	Whole Area
Catchment Area (Km^2)	35.86	42.09	51.44	62.32	65.07	73.18	81.98	83.44	90.71	91.81	92.37	100.51	103.23
Probable Daily Rainfall (mm)	184.3	176.7	165.3	152.1	148.7	138.9	132.8	132.7	132.3	132.2	132.2	131.8	131.6
Length of Channel (Km)	25.50	25.50	30.88	30.88	30.88	30.88	40.01	43.39	43.39	44.79	45.89	45.85	45.89
Mean Slope of Channel	0.0714	0.0714	0.0375	0.0375	0.0375	0.0375	0.0240	0.0107	0.0107	0.0107	0.0107	0.0107	0.0107
Mean Velocity (m/s)	9	9	7	7	7	7	5	4	4	4	4	4	4
Basin Lag (hr)	1.00	1.00	1.25	1.25	1.25	1.25	1.75	2.00	2.00	2.00	2.25	2.25	2.25
Rainfall Intensity During Basin Lag (mm/hr)	85.0	81.5	68.9	63.4	62.0	57.9	46.3	42.8	42.7	42.7	39.7	39.6	39.6
Peak Discharge (m^3/s)	635	715	740	825	840	880	880*	880*	880*	880*	880*	880*	880*
Specific Discharge ($m^3/s/Km^2$)	18	17	14	13	13	12	11	11	10	10	10	9	9

Notes: C.D. : Checkdam
N.R. . National Road
No.– : Division Point Number
* Estimated design discharge

Table 9 Estimated Peak Discharge of 100-year Flood of Tributaries (2)

RIVER	(2) K. BLONCKENG							
Site	Upper Part	Con-fluent 1	N.R.	No.1	Con-fluent 1	No.2	No.3	Whole Area
Catchment Area (Km^2)	13.40	17.93	21.38	22.16	36.41	41.35	69.15	71.20
Probable Daily Rainfall (mm)	211.7	206.2	202.0	201.0	183.7	177.6	143.8	141.3
Length of Channel (Km)	12.50	12.50	14.65	16.23	16.23	22.43	22.43	22.43
Mean Slope of Channel	0.0429	0.0429	0.025	0.025	0.0143	0.0143	0.0143	0.0143
Mean Velocity (m/s)	7	7	6	6	6	4	4	4
Basin Lag (hr)	0.75	0.75	0.75	1.00	1.00	1.50	1.50	1.50
Rainfall Intensity During Basin Lag (m^3/s)	109.2	106.3	104.2	92.7	84.7	67.5	54.7	53.7
Peak Discharge (m^3/s)	305	400	465	465*	645	645*	790	795
Specific Discharge ($m^3/s/Km^2$)	23	22	22	21	18	16	11	11

Notes: C.D. : Checkdam
N.R. : National Road
No.– : Division Point Number
* Estimated design discharge

Table 9 Estimated Peak Discharge of 100-year Flood of Tributaries (3)

RIVER	(3) K. PUTIH								(4) K. BATANG					(5) K. KRASAK				
Site	C.D 1	C.D 2	No.1	N.R.	No.2	Con-fluent 2	N.R.	Whole Area	N.R.	No.1	Con-fluent	No.2	Whole Area	Upper Reach	Check Dam	N.R.	K. Bebeng	Whole Area
Catchment Area (Km^2)	7.70	8.58	12.46	14.21	15.65	24.69	25.77	26.60	8.07	9.06	16.86	19.65	22.82	6.90	23.0	24.9	9.43	31.7
Probable Daily Rainfall (mm)	218.6	217.5	212.8	210.7	208.9	198.0	196.6	195.6	218.2	217.0	207.5	204.1	200.2	219.6	200.0	197.9	216.5	189.4
Length of Channel (Km)	6.33	6.33	15.98	15.98	17.78	17.78	19.86	21.84	11.05	13.55	13.55	16.30	18.80	4.25	10.25	12.53	13.45	22.03
Mean Slope of Channel	0.025	0.0625	0.0625	0.0333	0.01	0.01	0.01	0.01	0.0455	0.025	0.025	0.025	0.025	0.10	0.0375	0.0375	0.173	0.022
Mean Velocity (m/s)	9	9	6	6	4	4	4	4	8	6	6	6	6	11	7	7	15	5
Basin Lag (hr)	0.5	0.5	0.75	0.75	1.0	1.0	1.25	1.5	0.75	0.75	0.75	1.0	1.0	0.5	0.5	0.75	0.5	1.25
Rainfall Intensity During Basin Lag (mm/hr)	128.0	127.4	109.8	108.7	96.3	91.3	81.9	74.4	112.5	111.9	107.0	94.1	92.3	128.6	117.1	102.0	126.8	78.9
Peak Discharge (m^3/s)	205	230	285	320	315	470	470*	470*	190	210	375	385	440	185	560	530	250	530*
Specific Discharge ($m^3/s/Km^2$)	27	26	23	23	20	19	18	18	23	23	22	20	19	27	24	21	26	17

Notes: C.D. : Checkdam
N.R. : National Road
No.– : Division Point Number
* Estimated design discharge

Table 9 Estimated Peak Discharge of 100-year Flood of Tributaries (4)

RIVER	(6) K. BOYONG				(7) K. KUNING						
Site	C.D.	N.R.	K. Boyong	K. Code	C.D.	No. 1	Confluent 1	No. 2	Confluent 2	No. 3	Whole Area
Catchment Area (Km)	6.15	12.33	14.14	76.0	7.20	15.74	22.93	26.84	34.23	36.49	47.68
Probable Daily Rainfall (mm)	220.5	213.0	210.8	135.4	219.2	208.8	200.1	195.3	186.3	183.6	169.9
Length of Channel (Km)	5.75	12.45	18.20	37.0	7.05	21.63	21.63	26.81	26.81	27.76	34.34
Mean Slope of Channel	0.125	0.0529	0.0308	0.0143	0.125	0.0455	0.0455	0.01	0.01	0.01	0.01
Mean Velocity (m/s)	12	8	7	4	12	8	8	4	4	4	4
Basin Lag (hr)	0.50	0.50	0.75	2.0	0.50	1.00	1.00	1.50	1.50	1.50	2.00
Rainfall Intensity During Basin Lag (mm/hr)	129.1	124.7	108.7	43.8	128.3	96.3	92.2	74.3	70.8	69.8	55.0
Peak Discharge (m^3/s)	165	320	320	695	195	315	440	415	505	530	545
Specific Discharge ($m^3/s/Km^2$)	27	26	23	9	27	20	19	16	15	15	12

Notes: C.D. : Checkdam
N.R. : National Road
No.– : Division Point Number
* Estimated design discharge

Table 9 Estimated Peak Discharge of 100-year Flood of Tributaries (5)

RIVER	(8) K. GENDOL						(9) K. WORO				
Site	C.D. 1	C.D. 2	C.D. 3	C.D. 4	Whole Area	C.D. 5	Planned Check Dam	Sand Pocket	C.D.	K. Simpin	Whole Area
Catchment Area (Km	10.37	12.86	13.06	13.25	14.59	55.5	10.29	12.82	17.06	17.43	42.14
Probable Daily Rainfall (mm)	215.4	212.3	212.1	211.9	210.2	160.4	215.5	212.4	207.2	206.8	176.7
Length of Channel (Km)	8.35	12.18	13.28	14.38	19.38	19.38	11.5	16.0	22.8	27.2	27.2
Mean Slope of Channel	0.786	0.050	0.050	0.050	0.03	0.03	0.092	0.0385	0.0278	0.0125	0.0215
Mean Velocity (m/s)	10	8	8	8	6	6	11	7	6	4	4
Basin Lag (hr)	0.50	0.50	0.75	0.75	1.00	1.00	0.50	0.75	1.00	1.50	1.50
Rainfall Intensity During Basin Lag (mm/hr)	126.1	124.3	109.4	109.3	96.9	73.9	126.1	109.6	95.5	78.6	67.2
Peak Discharge (m^3/s)	275	335	300	305	295	855	270	295	340	285	590
Specific Discharge ($m^3/s/Km^2$)	26	26	23	23	20	15	26	23	20	16	14

Notes: C.D. : Checkdam
N.R. : National Road
No.– : Division Point Number
* Estimated design discharge

Table 10 Constants of Storage Function

(1) K. Progo Basin

	Catchment Area (Km^2)	K	P	Te (hr)	$Q_B(m^3/s)$
Sub-basin - 1	424.0	10.0	0.6	0.0	4.2
- 2	570.0	15.0	0.6	0.0	5.7
- 3	441.0	30.0	0.6	0.0	4.4
- 4	328.0	20.0	0.6	0.0	3.3
- 5	199.0	19.0	0.6	0.0	2.0
- 6	334.9	29.0	0.6	0.0	3.3
Channel - 1	-	20.0	0.6	0.0	-
- 2	-	15.0	0.6	0.0	--
- 3	-	15.0	0.6	0.0	-
- 4	-	40.0	0.6	0.0	-

(2) K. Opak Basin

	Catchment Area (Km^2)	K	P	Te (hr)	$Q_B(m^3/s)$
Sub-basin - 1	453.2	37.0	0.6	0.0	4.5
- 2	740.5	24.0	0.6	0.0	7.4
- 3	62.2	24.0	0.6	0.0	0.6
- 4	830.0	35.0	0.6	0.0	8.3
Channel - 1	-	25.0	0.6	0.0	-

Table 11 Estimated Probable Peak Discharge in Main Rivers

(1) K. Progo Basin

Site	Catchment Area	10-year	30-year	50-year	100-year
Kranggan	424 Km^2	1,200(2.1)	1,600(3.8)	1,700(4.0)	2,000(4.7)
Borobudur	994	2,100(2.1)	2,800(2.8)	3,100(3.1)	3,600(3.6)
Mendut	441	500(1.1)	600(1.4)	800(1.8)	900(2.0)
Duwet	1,763	3,100(1.8)	3,900(2.2)	4,600(2.6)	5,300(3.0)
Sentolo	1,962	3,500(1.8)	4,300(2.2)	5,100(2.6)	5,800(3.0)
River Mouth	2,297	3,700(1.6)	4,500(2.0)	5,700(2.5)	6,500(2.8)

(2) K. Opak Basin

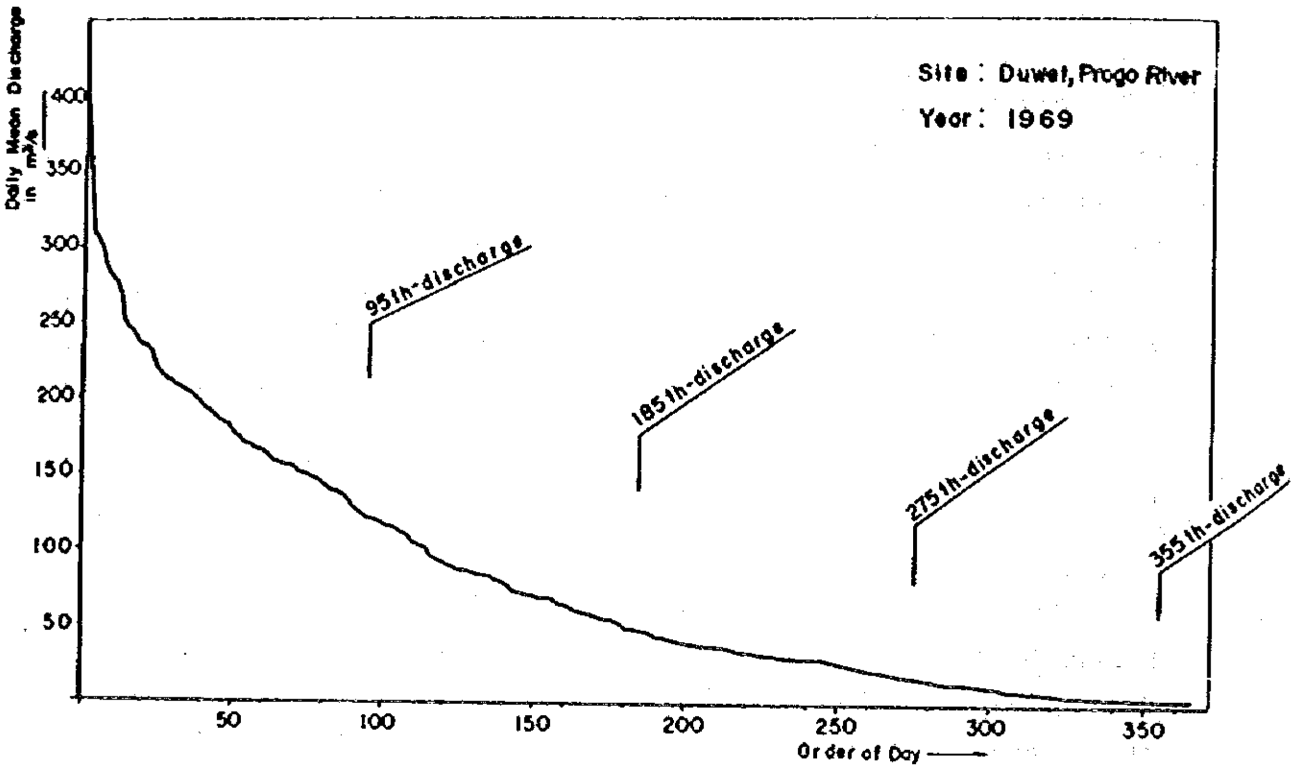
Site	Catchment Area	10-year	30-year	50-year	100-year
Karang Semut	453	1,600(3.5)	1,800(4.0)	2,000(4.4)	2,200(4.9)
Dogongan	740	1,650(2.2)	2,050(2.8)	2,300(3.1)	2,600(3.5)
River Mouth	1,250	2,900(2.3)	3,500(2.8)	3,900(3.1)	4,400(3.5)

Note: Values in ( ) denote the specific peak discharge in $m^3/s/Km^2$

(3) Checks Based on Rational Method (100-year Flood)

Site	Catchment Area	f	R100	$T\ell$	$\bar{Y}$	Qp
K. Progo Mouth	2,297 Km^2	0.75	113 mm	7.8 hr	13.4 mm/hr	6,412 m^2/s
K. Opak Mouth	1,256	0.75	114	6.0	16.8	4,396

Fig. 10 FLOW REGIME CURVES at DUWET SITE PROGO (I)



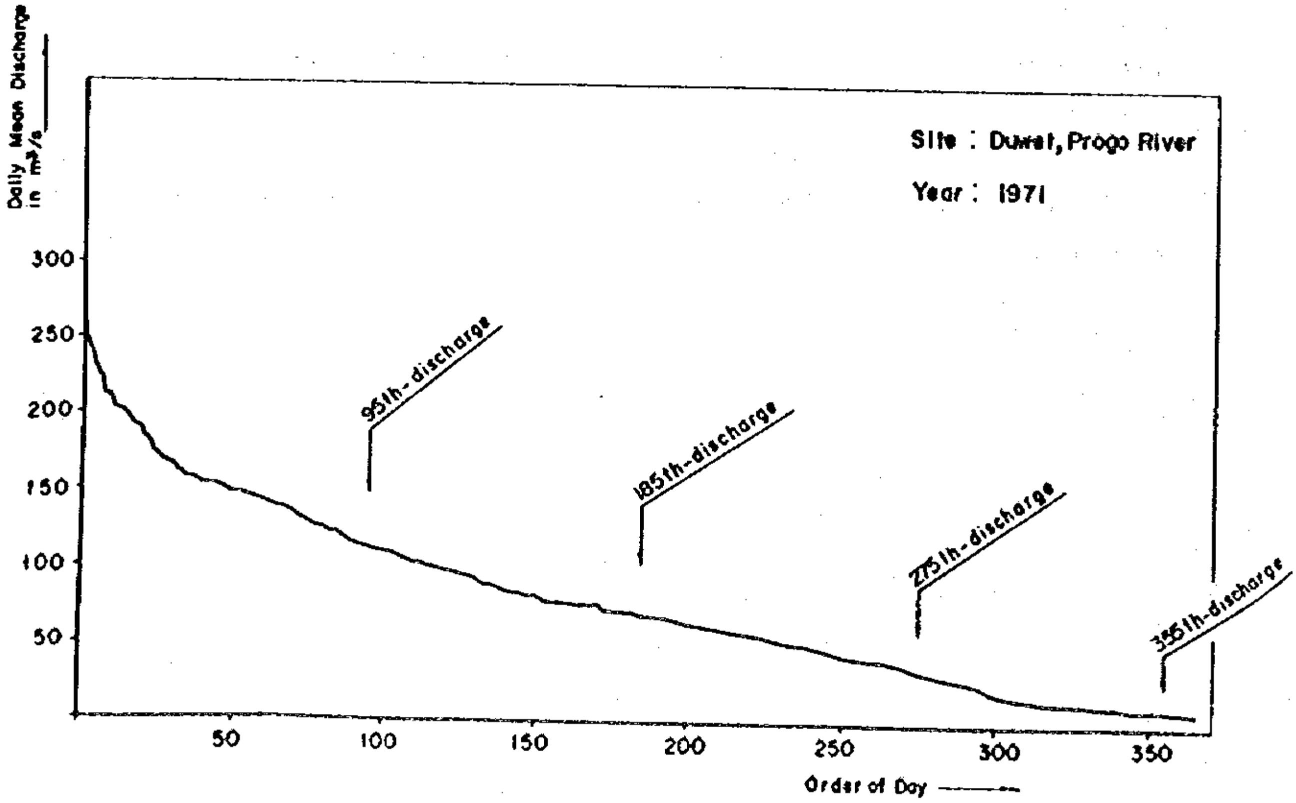
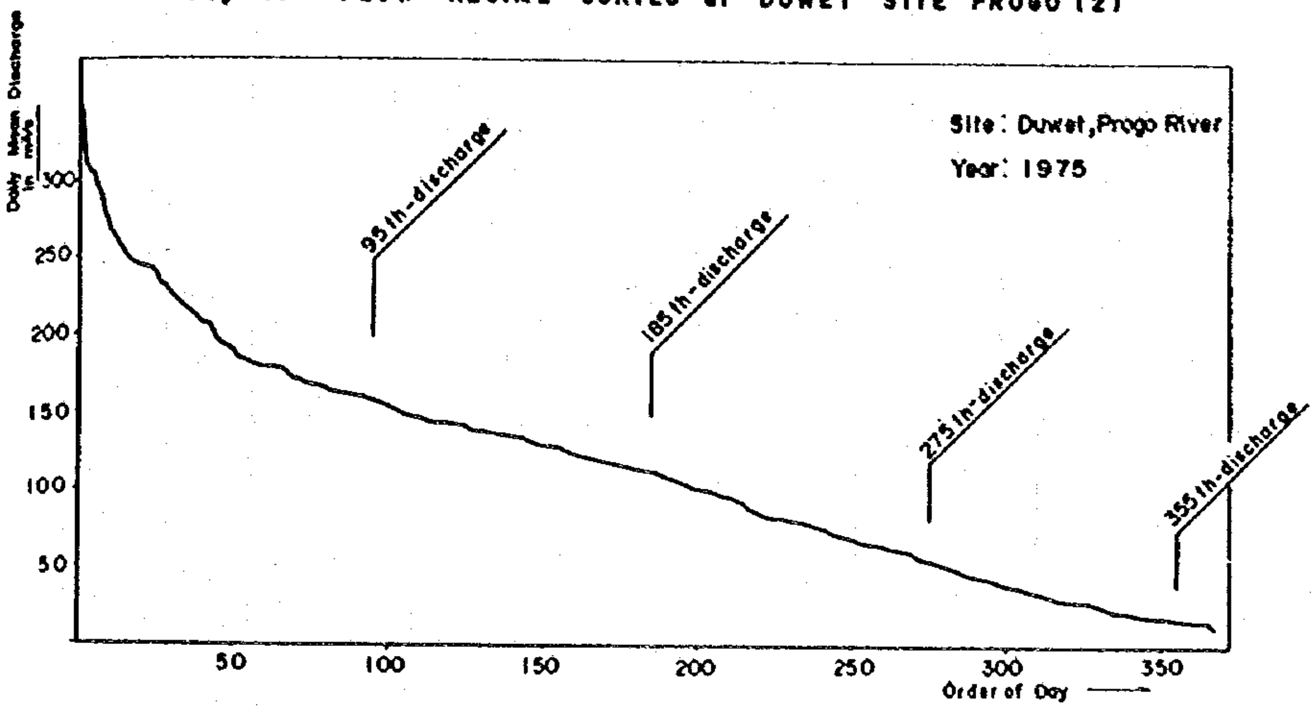


Fig. 10 FLOW REGIME CURVES at DUWET SITE PROGO (2)



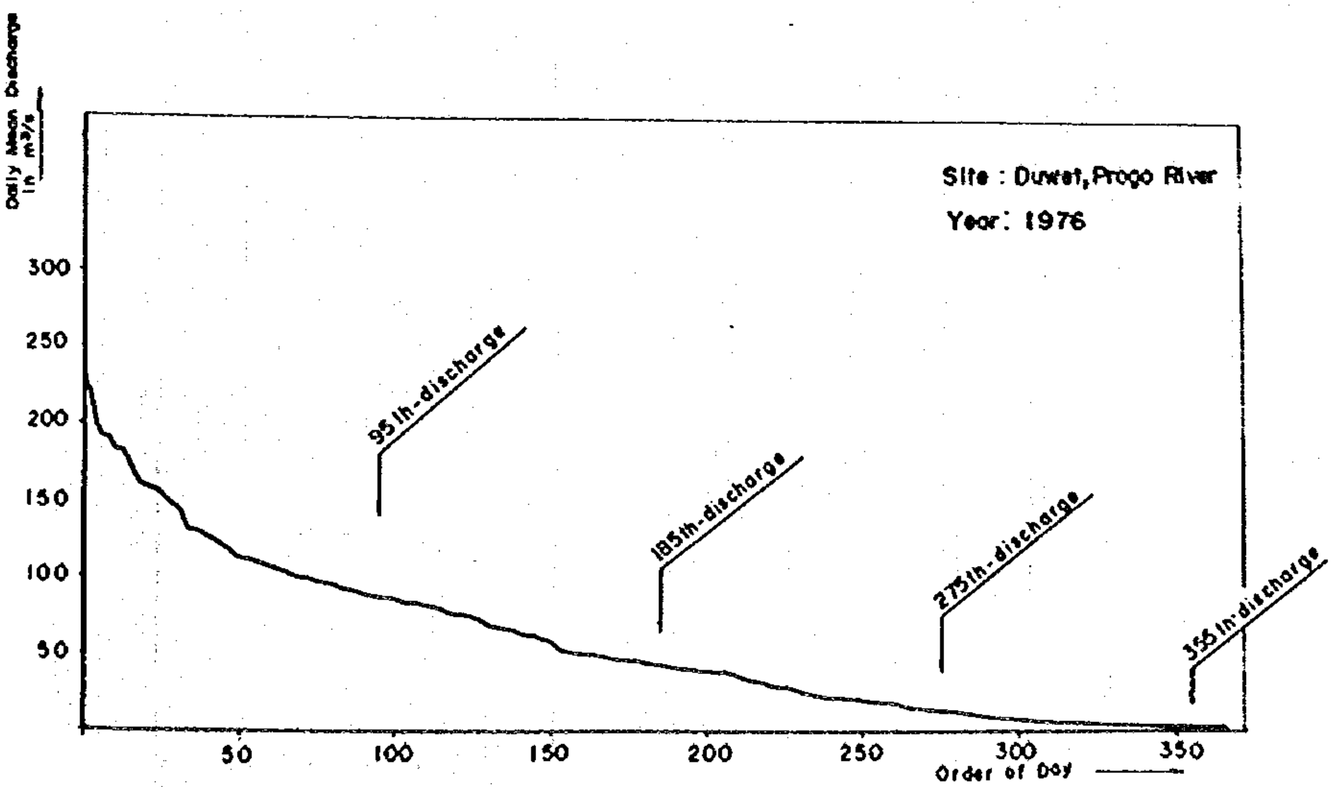


Fig. 11 FREQUENCY ANAYSIS OF DISCHARGE AT WEIRS

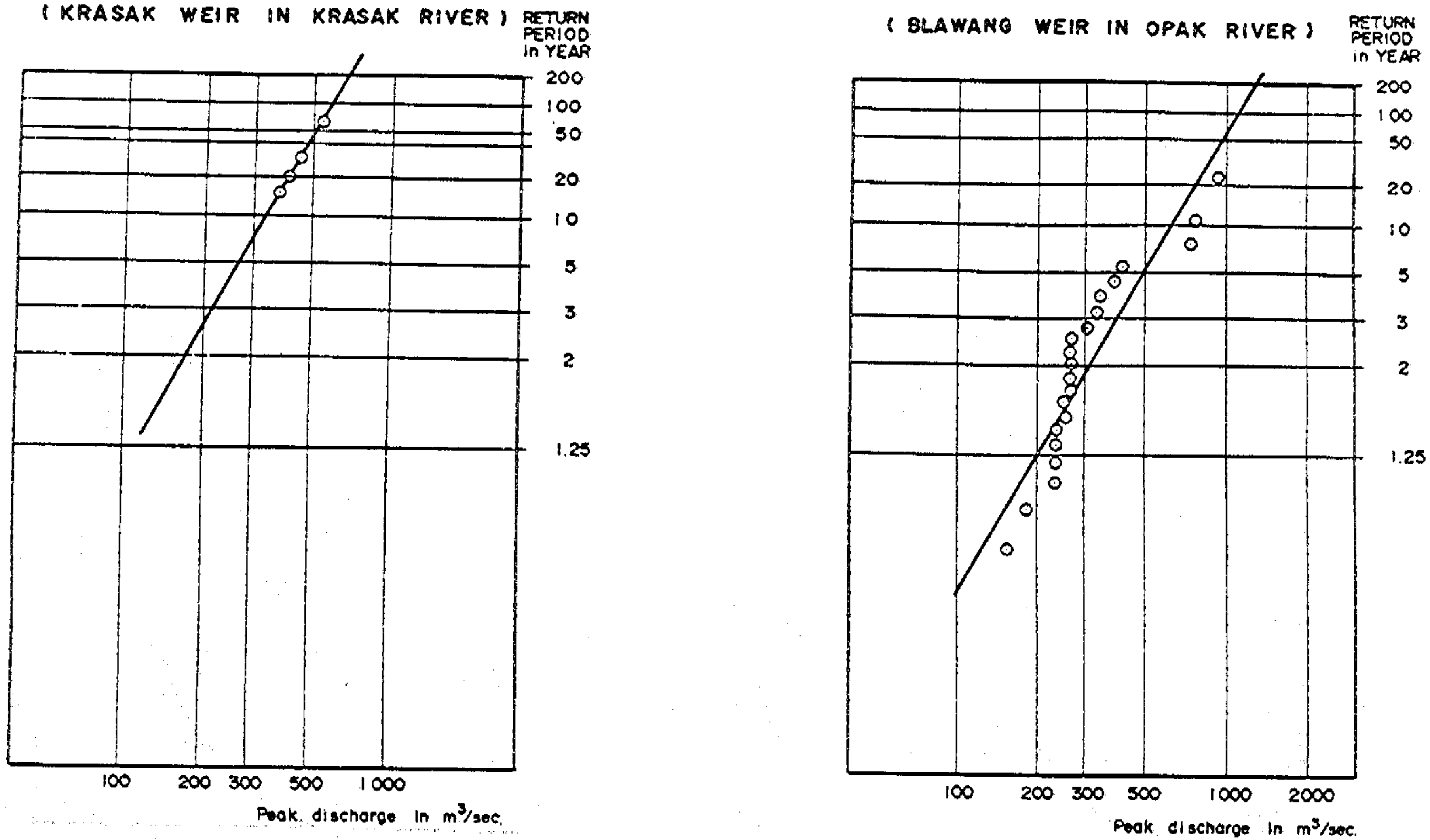


Fig. 12 ESTIMATED BASIN LAG VERSUS CATCHMENT AREA

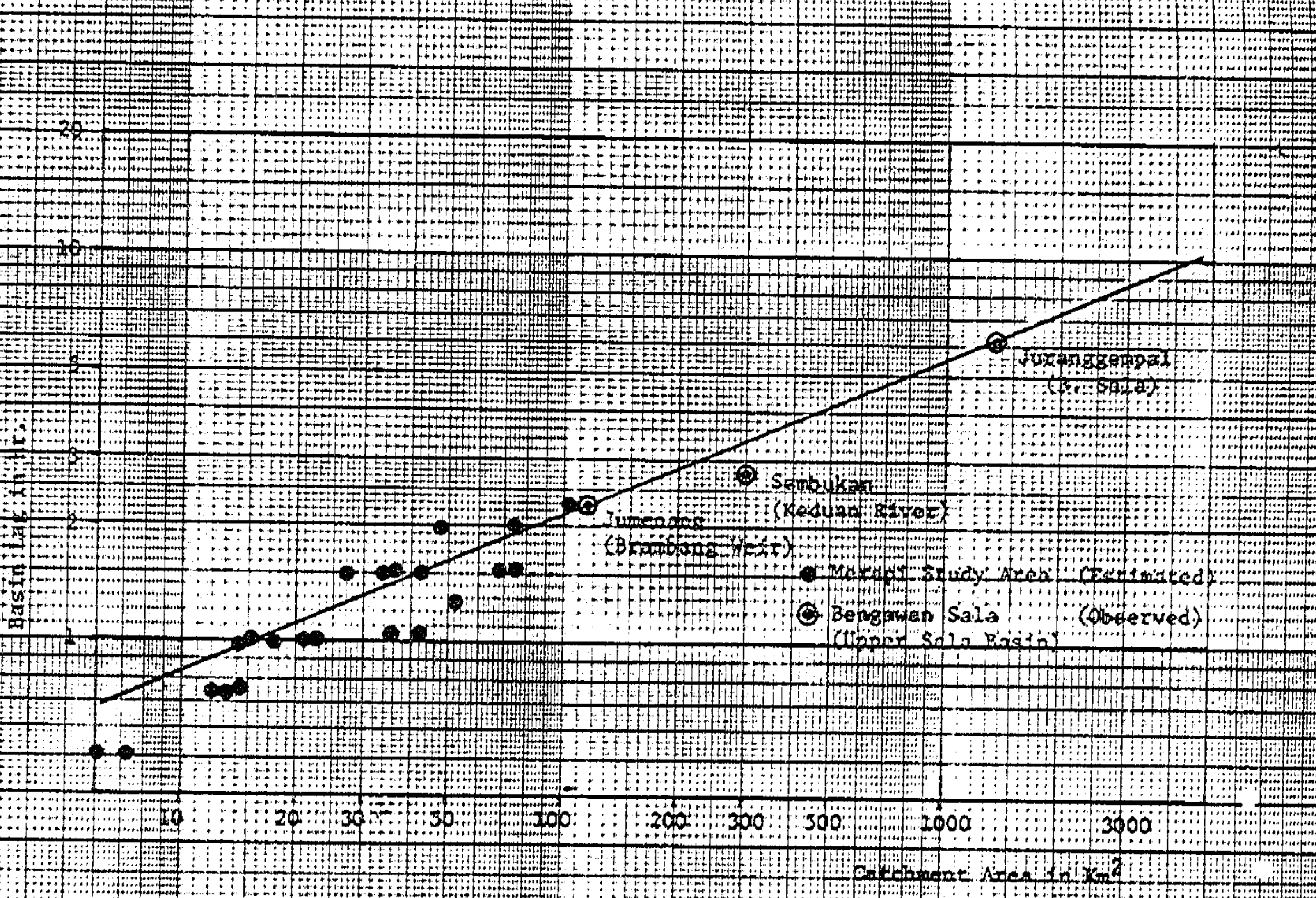


Fig. 13 SPECIFIC DISCHARGE VERSUS CATCHMENT AREA

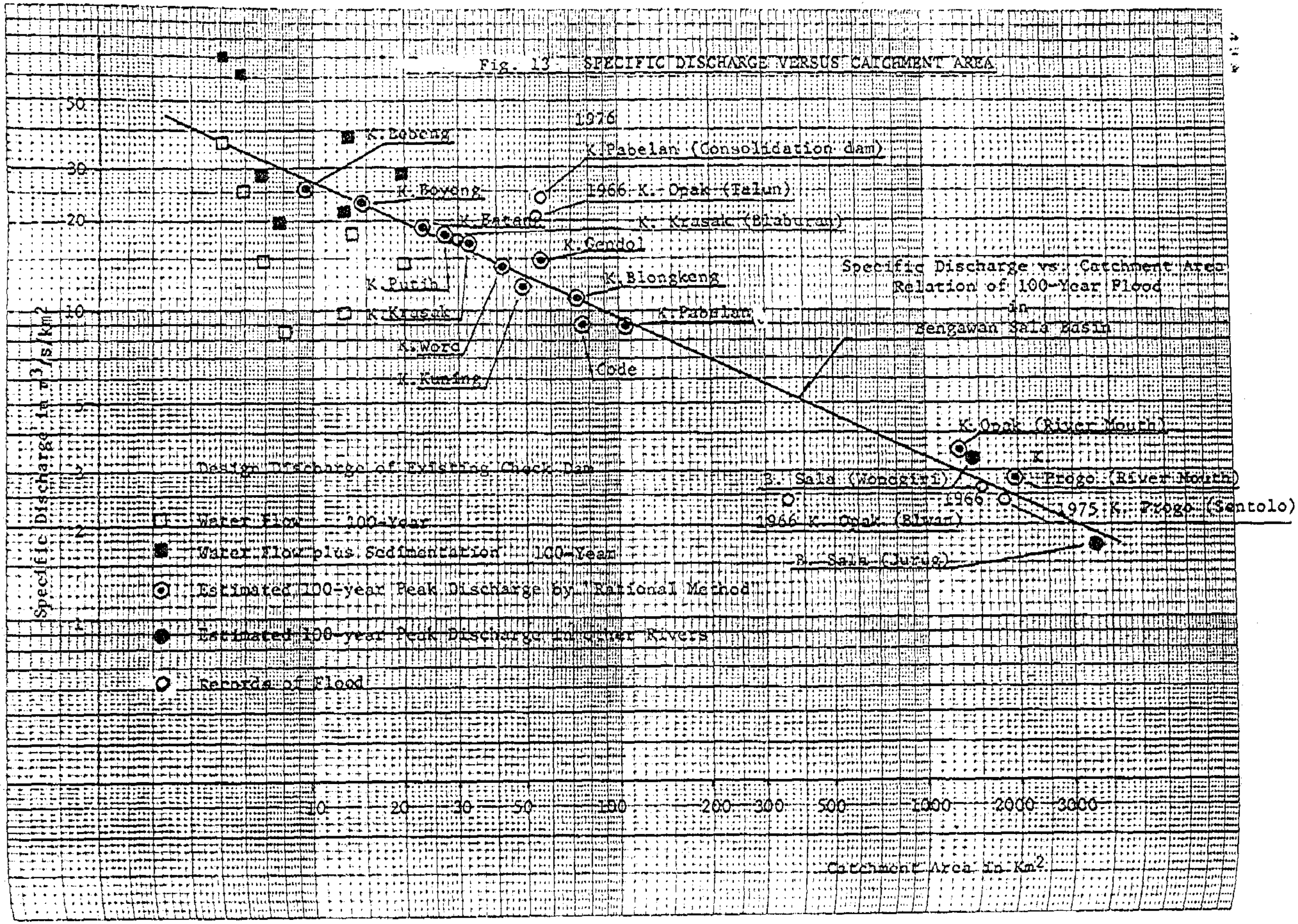


Fig. 14 Run-Off Model of K. Progo and K. Opak River Systems

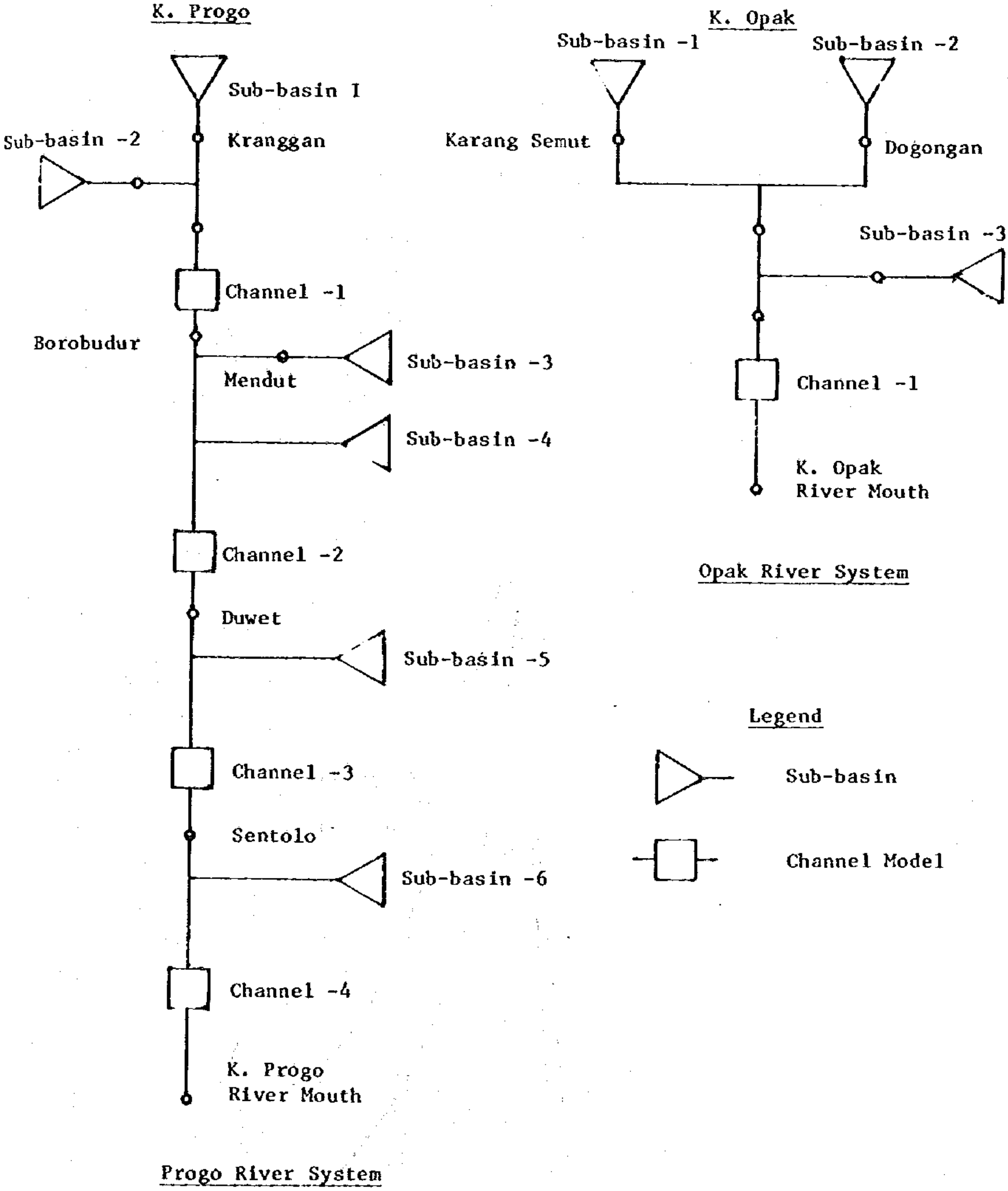


Fig. 15 COMPARISON OF DISCHARGE HYDROGRAPH OF NOV. 22, 1976 FLOOD

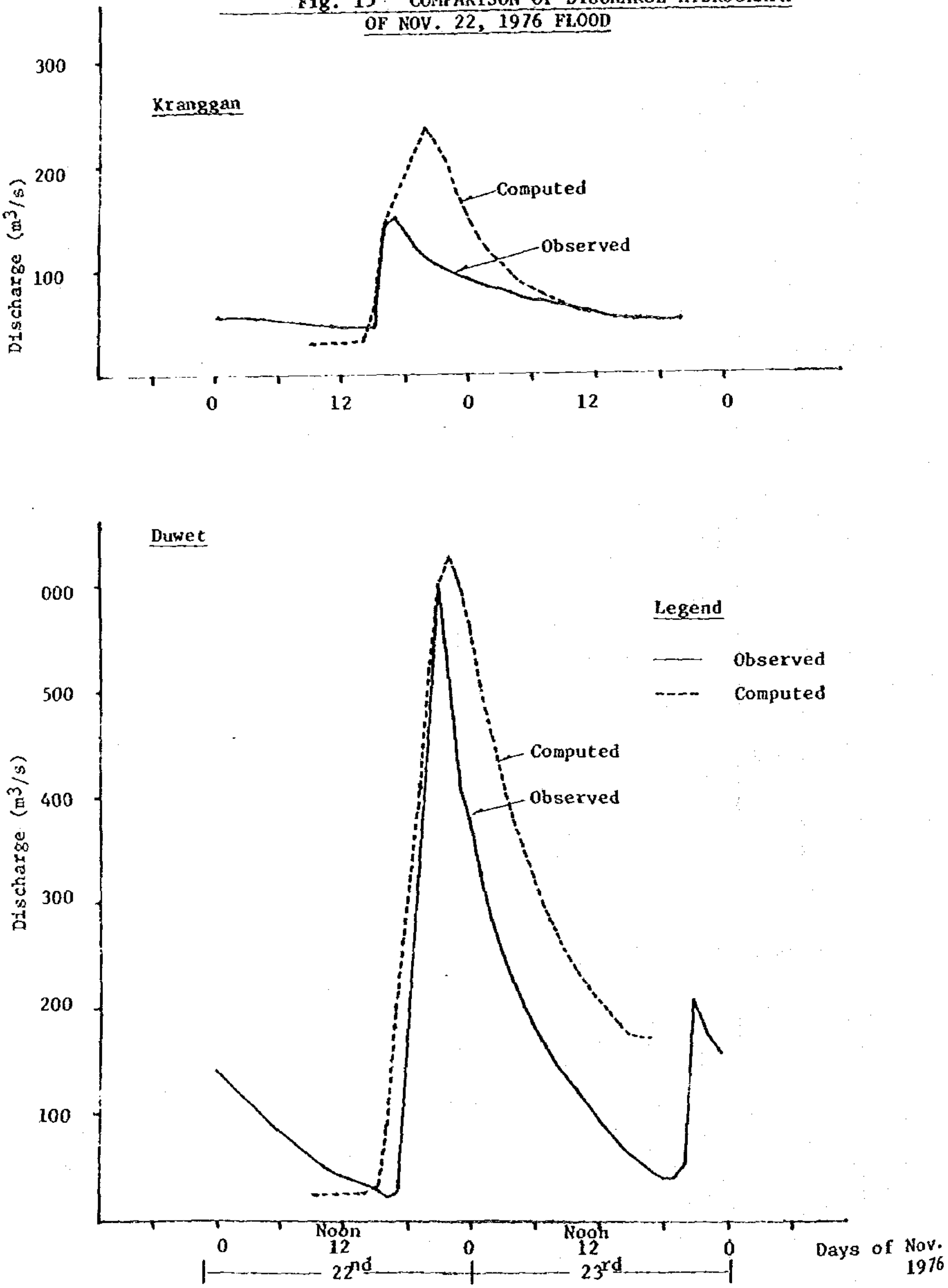


Fig. 16 COMPARISON OF DISCHARGE HYDROGRAPH OF NOV. 25, 1976 FLOOD

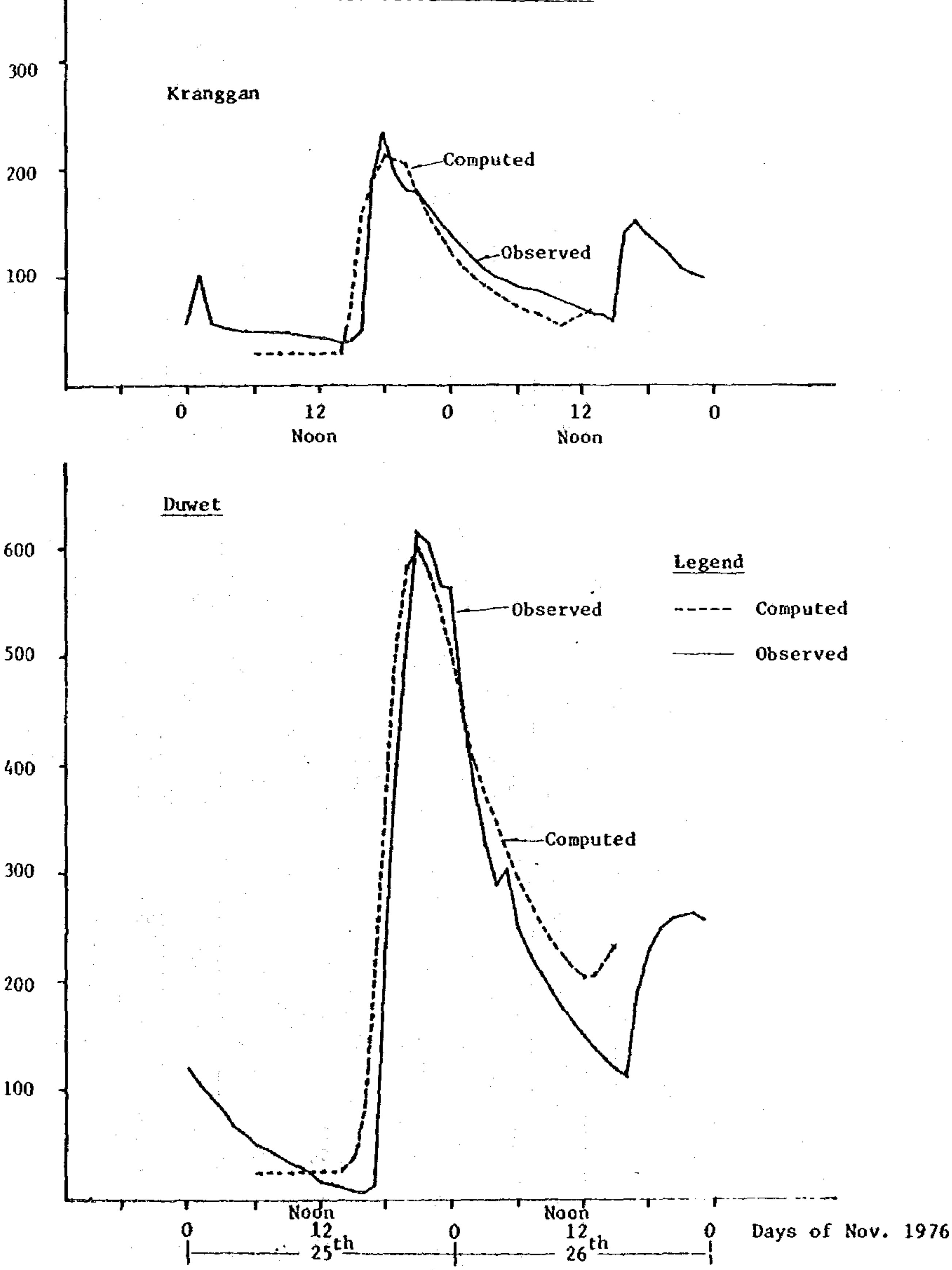
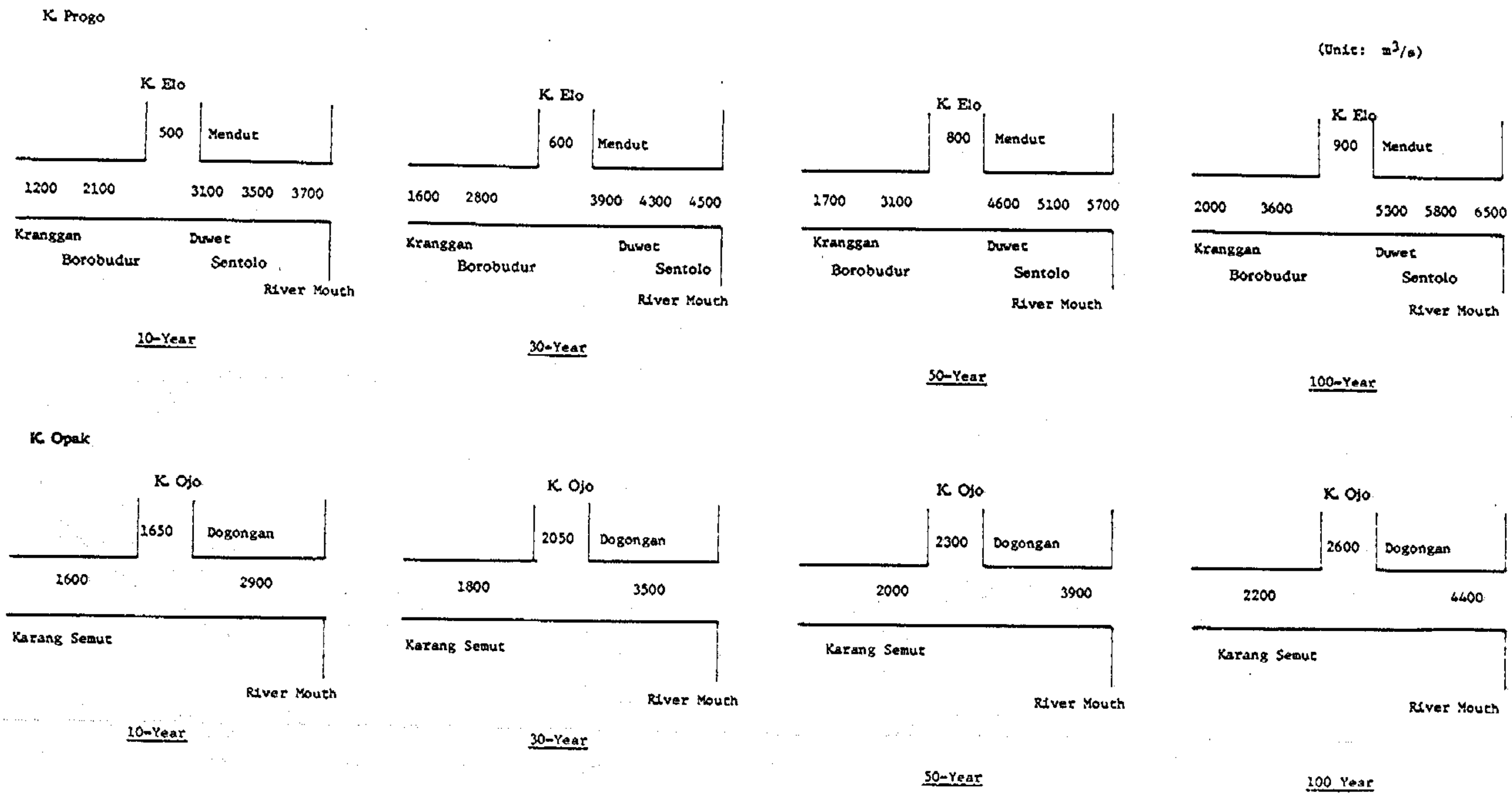


Fig. 17 PROBABLE PEAK DISCHARGE DISTRIBUTION



6. SEDIMENTATION

The quantity of sediment conveyed by rivers from the G. Merapi (area) and the effect of the sediment on the downstream areas of the main rivers was studied.

6.1 Available Data

The suspended loads have been measured by D.P.M.A. at several sites along K. Progo and K. Opak. The results are shown in Fig. 18 in which the relative quantity of suspended load per day vs. daily mean discharge is given.

The relation is given in a following equation.

$$Vs = 8.98\ Q^{1.35} \quad \text{................} \quad (6.1)$$

where:

Vs : Volume of suspended load in cu.m/day

Q : Daily mean discharge in cu.m/sec.

The suspended load in the tributaries in the project area was measured only two times in K. Krasak by K. Progo basin study team. The results are also shown in Fig. 18 to indicate the large difference in the quantity of sediment between the main rivers and K. Krasak. According to the figure, the suspended load in K. Krasak is approximately five to ten times greater than that in the main rivers.

Bed materials were sampled in K. Progo, K. Krasak, K. Opak and K. Woro to measure their size and specific gravity. The results given in Table 12 show that mean diameters of bed material in the downstream area of the tributaries and the main rivers was less than 1.0 mm and the coarser diameter in the upstream area of the tributaries ranged from 2.0mm to 25.0mm. The specified gravity of the bed materials shows small deviation from a value centering around 2.80 g/cu.cm.

Table 12 Mean Diameter of Bed Material

1. K. Progo

No.	Sampling Site	Mean diameter	Specific Gravity
1	Upstream of K. Pabelan junction	30.5 mm	2.94 g/cu.cm
2	Downstream of K. Pabelan junction	12.5	2.89
3	Downstream of K. Krasak junction	0.9	2.66
4	Sentolo	0.9 *	
5	Kamijoro	0.5 *	2.80 (Mean)
6	Srandakan	0.4 *	

* from Reference Book RB-2

2. K. Opak

No.	Sampling Site	Mean diameter	Specific Gravity
1	Prambanan	29.8 mm	2.85 g/cu.cm
2	Karangsemut	3.9	2.79
3	Kretek	4.9	-
4	Upstream of K. Code junction	8.8	2.75

3. K. Dengkeng (K. Woro)

No.	Sampling Site	Mean diameter	Specific Gravity
1	Sand Pocket	26.0 mm	-
2	Check Dam	10.0	-
3	National Highway	1.9	-

4. Tributary

No.	Tributary	Sampling Site	Mean diameter	Specific Gravity
1	K. Pabelan	Junction	25.3 mm	2.88 g/cu.cm
2	K. Krasak	Kemiri	5.0	2.83
3	"	Jarakah	9.8	2.78
4	"	Jombong	11.6	2.90
5	"	Sudimoro	18.6	2.81
6	"	Salam Bridge	5.4	2.80
7	"	Krasak Weir	4.8	2.80
8	K. Boyong	Gadja Mada Univ.	25.3	2.97

6.2 Sedimentation Formula

The above-mentioned suspended load formula cannot be used to estimate suspended loads in both main rivers and tributaries in the study area due to the observed large different suspended loads shown in Fig. Furthermore, there is no observed data to construct the relation of bedload quantity vs. discharge.

Therefore, the Brown's formula shown in the following equation was adopted to estimate the sedimentation. The formula gives volume of sedimentation consisting of bedload and suspended load under specific hydraulic river conditions.

$$\frac{qB}{U_*d} = 10\left[\frac{U_*^2}{(\sigma/\rho - 1)gd}\right]^2 \quad \ldots\ldots\ldots\ldots\ldots \quad (6.2)$$

$$U_* = \sqrt{gHI}$$

where:

- qB : Volume of sediment in unit width of river in cu.m/sec.m.
- U* : Friction velocity in m/s
- d : Diameter of bed material in m
- σ : Density of bed material
- ρ : Density of water
- g : Gravitational acceralation in 9.8 m/s^2
- I : Energy slope of flow (= slope of riverbed)

The hydraulic parameters required in the above equation are given based on Manning's mean velocity equation shown below.

$$Q = \frac{1}{n} \cdot H^{2/3} \cdot I^{1/2} \cdot H \cdot B \quad \ldots\ldots\ldots\ldots \quad (6.3)$$

where:

- Q : Discharge in m^3/s
- n : Manning's n
- H : Depth of flow in m
- I : Slope of riverbed
- B : Width of flow in m

The combining of Manning's formula and Brown's formula result in the following equation used to calculate sedimentation.

$$qB = 10\,\frac{(ngQ/HB)^3\ I^{4.5}}{(\ \delta/\rho - 1)^2\ d} \quad \ldots\ldots\ldots\ldots \quad (6.4)$$

6.3 Sediment Yield

(1) Sediment rating curve

Based on the equation mentioned above, the sediment rating curves at 16 sites in the main river and the tributaries shown in Fig. 19 were constructed under the hydraulic conditions given in Table 13 The constructed sediment rating curves are given in Table 14

(2) Annual sediment yield

Annual sediment yield at the respective sites may be estimated by accumulating data throughout a year of the quantity of sedimentation corresponding to respective daily discharge at the sites. The daily discharge at sites was based on discharge obtained at Duwet site in the Progo River by multiplying the ratio of the extent of watershed area at the site vs. those at Duwet site to the Duwet discharge. The annual sediment yields were estimated using above-mentioned procedure for 4 years, 1969, 1971, 1975 and 1976 to determine mean. The results of the estimation are given in Table 15.

Table 13 River Condition to Calculate Sediment Load

No.	Site	Catchment Area (sq.km)	River Bed Slope	Mean Diameter of Bed Material
1. K. Progo				
1	Upstream of K. Pabelan junction	1,435	1/260	30.5 (mm)
2	Downstream of K. Krasak junction	1,795	1/190	0.9
3	Srandakan	2,297	1/610	0.4
2. K. Opak				
1	K. Gendol	15	1/33	6.5
2	K. Opak - U22 site	56	1/36	29.8
3	- 25 km site	330	1/450	8.8
4	- 22 km site	377	1/450	8.8
5	- 13 km site	453	1/450	4.9
6	- 12 km site	1,194	1/450	4.9
7	- 5 km site	1,256	1/450	4.9
3. K. Dengkeng (K. Woro)				
1	Check dam	17	1/100	10.0
2	Woro - Pt. 47 site	17	1/600	1.9
3	- Pt. 49 site	42	1/600	1.9
4	- Pt. 67 site	58	1/940	1.9
5	- Pt. 69 site	76	1/940	1.9
4. Tributary				
1	K. Pabelan	103	1/30	25.3
2	K. Blongkeng	71	1/70	25.3
3	K. Lamat	14	1/43	25.3
4	K. Putih	27	1/10	4.8
5	K. Batang	23	1/40	25.3
6	K. Krasak	32	1/46	4.8
7	K. Kuning	48	1/260	8.8
8	K. Code	76	1/70	8.8
9	K. Boyong	12	1/80	25.3

Table 14 Sediment Rating Curve

1. K. Progo

(1) Upstream of K. Pabelan Junction

Q	U*	Qs
0	0.000	0.0
250	0.204	0.6
700	0.278	2.9
1,300	0.362	6.8
2,100	0.373	13.5

(2) Downstream of K. Pabelan Junction

Q	U*	Qs
0	0.000	0.0
290	0.231	3.9
900	0.313	19.1
1,800	0.378	50.5
2,950	0.431	100.2

(3) Srandakan

Q	U*	Qs
0	0.000	0.00
40	0.075	0.03
500	0.120	0.6
1,750	0.174	3.9
3,550	0.214	11.1

2. K. Opak

(1) Opak - U22 Site

Q	U*	Qs
0	0.000	0.0
45	0.446	0.4
100	0.547	1.2
275	0.686	4.1
540	0.794	9.5

(2) K. Opak - 25km Site

Q	U*	Qs
0	0.000	0.000
10	0.098	0.001
85	0.156	0.014
235	0.199	0.055
410	0.203	0.088

(3) K. Opak - 22km Site

Q	U*	Qs
0	0.000	0.000
13	0.088	0.001
115	0.162	0.023
305	0.209	0.090
570	0.244	0.208

(4) K. Opak - 13km Site

Q	U*	Qs
0	0.000	0.000
33	0.118	0.006
160	0.175	0.054
405	0.207	0.164
850	0.243	0.422

Note: Q : Discharge (m/s)
U* : Friction Velocity (m/s)
Qs : Sediment load (m/s)

Table 14 Sediment Rating Curve (Cont'd)

2. K. Opak (cont'd)

(5) K. Opak - 12km Site

Q	U*	Qs
0	0.000	0.0000
10	0.013	0.0005
65	0.088	0.0039
330	0.139	0.0400
750	0.174	0.1313

(6) K. Opak - 5km Site

Q	U*	Qs
0	0.000	0.0000
10	0.061	0.0005
80	0.085	0.0040
415	0.139	0.0480
945	0.174	0.1538

3. K. Dengkeng (K.Woro)

(1) K. Woro - Pt 47 Site

Q	U*	Qs
0	0.000	0.0000
10	0.115	0.0007
40	0.164	0.0072
90	0.206	0.0194
160	0.241	0.0331

(2) K. Woro - Pt 49 Site

Q	U*	Qs
0	0.000	0.0000
8	0.798	0.0018
30	0.112	0.0110
65	0.140	0.0344
115	0.162	0.0740

(3) K. Woro - Pt 67 Site

Q	U*	Qs
0	0.000	0.0000
14	0.093	0.0024
47	0.127	0.0128
103	0.157	0.0381
180	0.186	0.0891

(4) K. Woro - Pt 69 Site

Q	U*	Qs
0	0.000	0.0000
45	0.118	0.0120
110	0.149	0.0408
205	0.179	0.1025
260	0.192	0.1471

4. Tributary

(1) K. Pabelan

Q	U*	Qs
0	0.000	0.0000
305	0.361	0.7199
1,900	0.608	10.1981
4,600	0.790	38.1802

(2) K. Blongkeng

Q	U*	Qs
0	0.000	0.0000
950	0.791	8.8294
1,315	0.856	13.5308
1,735	0.921	19.9304

Note: Q : Discharge (m/s)
U* : Friction Velocity (m/s)
Qs : Sediment load (m/s)

Table 14 Sediment Rating Curve (cont'd)

4. Tributary (cont'd)

(3) K. Putih

Q	U*	Qs
0	0.000	0.0000
30	0.270	0.2310
130	0.402	1.7680
290	0.498	5.3380
490	0.575	11.2870

(4) K. Batang

Q	U*	Qs
0	0.000	0.0000
50	0.439	0.4180
180	0.635	2.7954
210	0.479	1.3929
710	0.661	7.7860

(5) K. Krasak

Q	U*	Qs
0	0.000	0.0000
30	0.298	0.6560
100	0.421	3.6750
200	0.517	10. 2080
325	0.597	21.0370
635	0.731	57.7440

(6) K. Code

Q	U*	Qs
0	0.000	0.0000
40	0.349	0.3340
70	0.407	0.7699
175	0.502	2.4675
315	0.583	5.5254

(7) K. Kuning

Q	U*	Qs
0	0.000	0.0000
10	0.167	0.0068
25	0.200	0.0185
45	0.226	0.0371
70	0.248	0.0641
110	0.276	0.1133

Note: Q : Discharge (m/s)
U* : Friction Velocity (m/s)
Qs : Sediment load (m/s)

Table 15 Estimated Sediment Yield

(Unit: x 10^3cu.m)

No.	River / Item		Site	Catchment Area (km^2)	ANNUAL SEDIMENTATION 1969	1971	1975	1976	Mean
(1)	K. Progo, Upstream		K. Pabelan	1,453.0	581.1	788.7	1,162.3	577.0	777.3
			K. Pabelan	103.2	253.1	343.6	506.3	251.3	338.5
			K. Blongkeng	71.2	228.0	309.4	456.0	226.4	304.9
			K. Lamat	14.2	130.0	176.0	259.5	128.8	173.6
			K. Putih	26.6	197.5	268.0	395.0	196.1	264.1
			K. Batang	22.8	209.4	284.1	418.7	207.9	280.0
			K. Krasak	31.7	672.5	912.6	1,345.0	667.7	899.4
(2)	Sub-Total				1,560.5	2,117.7	1,345.0	1,549.4	2,087.2
(3)	K. Progo, Duwet			1,794.7	25,008.8	33,940.5	5,001.6	24,830.2	33,449.3
(4)	K. Progo, Surandadan			2,297	2,248.2	3,301.4	5,197.2	2,227.1	3,243.5
	Difference	(1) + (2) – (4)			–106.6	–395.0	–913.9	–100.7	–379.0
(5)	Check dam		Pt. 47	17.10	17.9	24.1	35.8	17.9	23.9
(6)	K. Woro		Pt. 47	17.43	4.3	5.8	8.6	4.3	5.8
(7)			Pt. 49	42.14	48.3	65.5	96.5	47.9	64.6
(8)			Pt. 67	57.74	15.7	21.3	31.4	15.6	21.0
(9)			Pt. 69	76.14	14.1	19.1	28.2	14.8	18.9
	Difference	(7) – (6)			32.6	44.2	65.1	32.3	43.6
(10)	K. Gendol			14.6	156.5	211.1	313.0	156.5	209.3
(11)	K. Opak, U22			55.51	501.0	679.9	1,002.0	497.4	670.1
(12)	K. Opak,	25 km site		329.5	33.4	45.3	66.8	33.2	44.7
(13)		K. Kuning		47.7	28.4	38.5	56.8	28.2	38.0
(14)	K. Boyong			12.3	67.2	91.1	134.3	66.7	89.8
(15)	K. Opak,	22 km site		377.2	31.4	42.6	62.8	31.2	42.0
(16)		K. Code		76.0	714.4	969.5	1,428.8	709.3	955.5
(17)	K. Opak,	13 km site		453.2	82.3	118.1	182.6	81.6	116.2
(18)	K. Opak,	12 km site		1,193.7	70.5	97.7	146.6	69.9	96.2
(19)	K. Opak,	5 km site		1,255.9	65.8	90.2	134.1	65.3	88.9
	Difference	(11) – (12)		–	467.6	634.6	935.2	464.2	625.9
	Difference	(12) + (13) – (15)		–	30.4	61.2	60.8	30.2	40.7
	Difference	(15) + (16) – (17)		–	663.5	894.0	1,309.0	658.9	881.4
	Difference	(18) – (19)		–	4.7	7.5	12.5	4.6	7.3

Note: Difference: Positive (–) denotes erosion of sedimention
Negative (+) denotes deposition of sedimentation

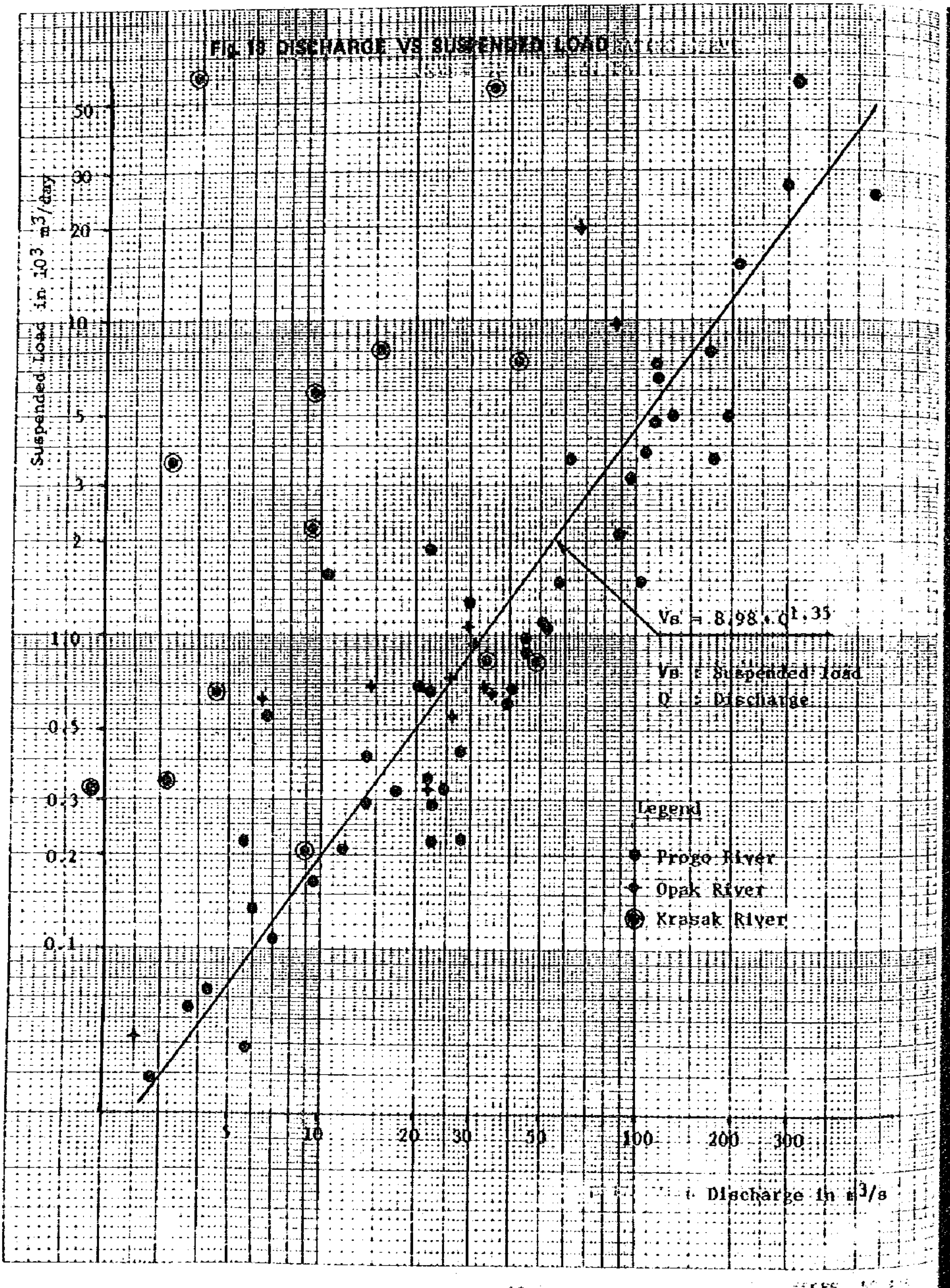
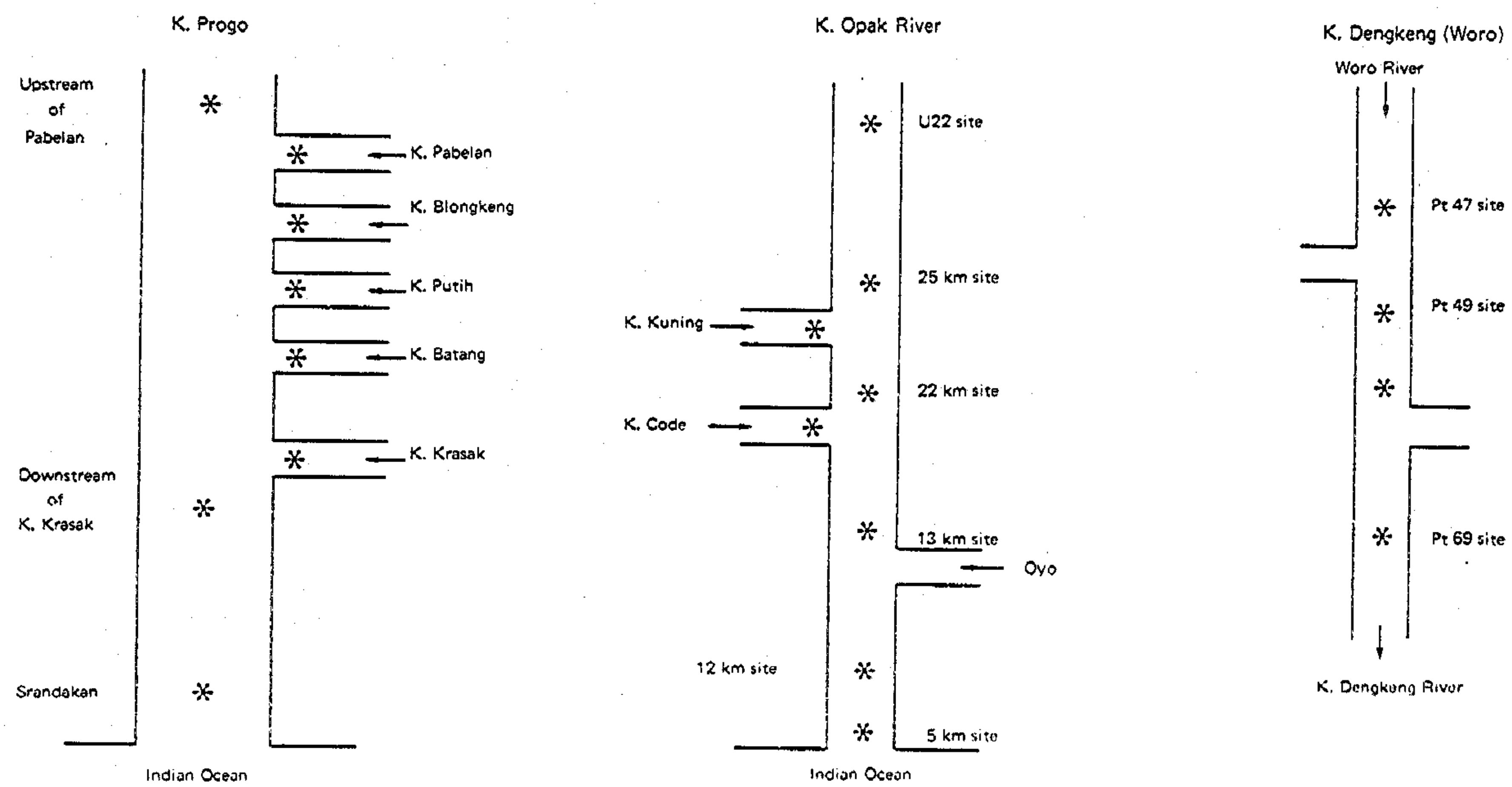


FIG. 18 DISCHARGE VS SUSPENDED LOAD
Suspended Load in 10^3 m^3/day
50
30
20
10
5
3
2
1.0
0.5
0.3
0.2
0.1
5
10
20
30
50
100
200
300
Discharge in m^3/s
$Vs = 8.98 \cdot Q^{1.35}$
Vs : Suspended load
Q : Discharge
Legend
Progo River
Opak River
Krasak River

Fig. 19 Location of Sites to Estimate Sediment Load



SUPPORTING REPORT D

RIVER SURVEY

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1. PROBLEM SPOTS ALONG K. PROGO AND K. OPAK

The scope of this river survey covers the basins of the K. Progo and K. Opak from upstream down to the estuaries which contains areas severely affected by volcanic debris from the G. Merapi. (Refer to Fig. 1)

The main object of the river survey is to study characteristics of inundated areas and problem spots along the river courses.

1.1 General River Characteristics

1.1.1 K. Progo

The K. Progo has a catchment basin of 2,300 sq. km. and a total length of about 140 km including about 40 km between the estuary and the confluence of the K. Krasak.

G. Sundoro and G. Sumbing form the western boundary and G. Merbabu and G. Merapi form the eastern boundary of the catchment basin of the K. Progo, a main river. The river flows southward, after joining a tributary group consisting of K. Pabelan, K. Blongkeng, K. Batang and K. Krasak, all on the western slopes of G. Merapi; it then empties into the Indian Ocean. According to the survey report in 1977, the stream gradient is about 1:600 over the area between the estuary and the point about 20 km upstream. The river width is about 400 m between the estuary and the point about 20 km upstream, and about 200 m between the 20 km point and the confluence of the K. Krasak.

1.1.2 K. Opak

The K. Opak has a catchment basin of 1,250 sq.km and a total length of about 70 km. The K. Opak joins tributaries such as K. Boyong, K. Kuning and K. Gendol on the southern slope of G. Merapi, maintains a southward flow, and, at point about 13 km upstream from the estuary, meets to the K. Ojo (the biggest tributary with a catchment basin of about 700 sq.km) from the east and then empties into the Indian Ocean. The stream gradient is 1:780, 1:450 and 1:260, over the sections between the estuary and points 12 km, 30 km and 37 km upstream respectively. The river width varies around a roughly estimated average of 120 m.

1.1.3 River Inundation

The magnitude of a river inundation has varied with the rate of flow capacity of the channel, and the scale of flood discharge. The rate of channel flow capacity is observed on the decline due to a steady rise in the riverbed caused by an ever-increasing amount silt deposits originating from G. Merapi, an active volcano. Flood damage in the lowland on both banks of the K. Progo between the estuary and the point about 10 km upstream was reported in 1969, 1974 and again in 1975. As for the K. Opak, floods in 1965 and 1966 were reported as causing heavy damage over an area of 2,000 ha. in the catchment basin. (Refer to Fig. 2, 3, 4, 5, 6 & 7)

(1) K. Progo

Floods in recent years were reported in 1969, 1974 and 1975. The heaviest of the three was the flood in March 1969, and details of the damage compiled from field survey reports and the existing data are roughly as shown below.

Inundation Zone	: As shown in Fig. 2
Inundation Area	: 880 ha.
Right Bank	: 330 ha.
Left Bank	: 500 ha.
Houses Collapsed	: 26
Submerged Houses	: 392
Inundation Depth	: 1.0 - 1.5 m (see Fig. 3)

An approximate of estimate of damage caused by inundation is shown in Table 1.

(2) K. Opak

Floods in recent years were reported in 1965 and 1966. The flood in 1966 was by far the worst, and the details of damage are as shown below.

Inundation Zone	: As shown in Fig. 4
Inundation Area	: 1,800 ha.
Right Bank	: 500 ha.
Left Bank	: 1,300 ha.
Houses Collapsed	: 330
Submerged Houses	: 5,700
Deaths	: 8
Inundation Depth	: 2.0 - 2.5 m (see Fig. 5)

1.1.4 Flow Capacity of River Channel

(1) K. Progo

The area of investigation in the river channel was limited to the section between the estuary and the point about 20 km upstream as follows:

Flood Plain	: From the estuary to the point 13 km upstream
Mountains	: Upstream from the 13 km point

For the area between the estuary and the point about 20 km upstream, non-uniform flow computations were performed in 1977 on different arbitrary discharges using the coefficient of roughness of 0.035 and cross levelling at 1-km intervals (refer to Fig. 8). The data itself comes from the section entitled "Hydraulic and Hydrology". The flow capacity of the channel is summarized both before and after the construction (1976-78) of a 4.0 km embankment on the left bank downstream of the Srandakan Bridge (located at point about 8 km upstream from the estuary).

K. Progo Flow Capacity

	Embankment Construction	
	Before	After
	(cu.m/sec approx.)	
Left Bank	1,000	5,000
Right Bank	2,000	-

A longitudinal distribution of the flow capacity of the channel is shown in Fig. 9 and Table 2. (Refer also to Tables 3 & 4 and Fig. 10)

(2) K. Opak

The area of investigation in the river channel was limited to the section between the estuary and the point about 37 km upstream as follows:

Flood Plain	: From the estuary to the point about 20 km upstream
Mountains	: Upstream from the 20 km point

For the above-mentioned 37 km section, non-uniform flow computations were performed on different arbitrary discharges using the coefficient of roughness of 0.035 and the available data on cross levelling (Refer to Fig. 12).

The flow capacity of the channel is summarized below both before and after the construction of embankments after the flood of 1966. The embankments have been constructed almost completely on the right bank, and partially on the left bank along the river channel between the estuary and the point about 17 km upstream from the estuary.

K. Opak Flow Capacity

	Embankment Construction	
	Before	After
	(cu.m/sec approx.)	
Left Bank	500	1,000
Right Bank	300	2,500 - 3,000

A longitudinal distribution of the flow capacity of the channel is shown in Fig. 12 and Table 5. (Refer also to Tables 6 & 7 and Fig. 11)

1.1.5 Present Condition of Riverbed Evolution

The existing data of the riverbed evolution is based on the outcome of the cross levelling at the sites of the bridge and the intake, and also on verbal information obtained from local inhabitants.

(1) K. Progo

Site	Location	Past Record	Bed Evolution, Sectional Area sq m.
Srandakan Bridge	From the estuary to the point abt. 8 km upstream	Comparative studies of the cross levelling data in 1966 and 1978	+700
Kamijoro Intake	From the estuary to the point abt. 17.5 km upstream	Comparative studies of the cross levelling data in 1924 and 1978	-212 (1924-33) +109 (1933-34) +4 (1934-35) +19 (1935-36) -19 (1936-37) +14 (1937-38) -79 (1938-39) +43 (1939-41) -84 (1941-61) +400 (1961-70) -200 (1970-77)
Bantar Bridge	from the estuary to the point abt. 28 km upstream	Comparative studies of the cross levelling data in 1971 and 1978	+200

Note: + shows rising tendency - "deposit"
- shows declining tendency - "scouring"

The evolution of the riverbed of the K. Progo in the year 1969 between the estuary and the confluence of the K. Krasak is estimated below. However, it may be necessary to conduct further survey and study as the data so far obtained is far from complete.

The bed deposit (cross sectional area of riverbed) of about 400 sq.m was observed between 1961 and 1970 (including effects of eruption of the G. Merapi in 1969) at the Kamijoro Intake, a representative point along the river channel where annual changes of the riverbed evolution were recorded.

On the other hand, the bed deposit of 200 sq.m approx. was observed at the Bantar Bridge located relatively close to the Kamijoro Intake during the seven years (1971 - 1978).

There are two different types of change in riverbed evolution --- one is normal years and the other is years when G. Merapi was active.

Based on the foregoing, the bed deposit at the Kamijoro Intake in 1969 was estimated at approx. 200 sq.m which is about 1/2 of the 400 sq.m deposit.

The length of the channel of the K. Progo between the estuary and the confluence of the K. Krasak is about 40 km. Out of the above-mentioned 40 km, the length of the channel where the riverbed evolution is prominent may be estimated at 35 km, except the section upstream where tractive force is higher on the bed, and the bed deposit (cross sectional area of the riverbed) may also be estimated at 200 sq.m. From the foregoing, the riverbed deposited volume was estimated at about 7,000,000 cu.m.

(2) K. Opak

There are a number of weirs blocked with debris preventing discharge in the upper stream of the river and exposed rocks are found over the riverbed of the middle and downstream channel of the K. Opak. As a result, the tendency toward scouring is observed. According to information based on the above-mentioned interviews with local inhabitants regarding riverbed evolution, the estimated rates of debris deposition are shown below.

Location	K. Opak Riverbed Depositing (est.) cm per year
Keringan-Tulung	25
Jiwan	8
Panjangrejo	4 - 8

1.1.6 Estimation of Flood Discharge

(1) K. Progo

Discharge of the flood in March 1969, the biggest in recent years, may reasonably estimated at 5,000 cu.m/sec. based on the flow capacity of the existing river channel, the rate of evolution of the riverbed to-date, and also the depth of inundation at the time of the above-mentioned flood.

(2) K. Opak

Discharge of the flood in March 1966 may reasonably be estimated at 1,500 cu.m/sec. based on flow capacity of the existing river channel and the depth of inundation at the time of the above-mentioned flood. The riverbed evolution to-date is not considered on the presumption that the change, if any, was negligible.

1.1.7 Facts Regarding Actual River Inundation

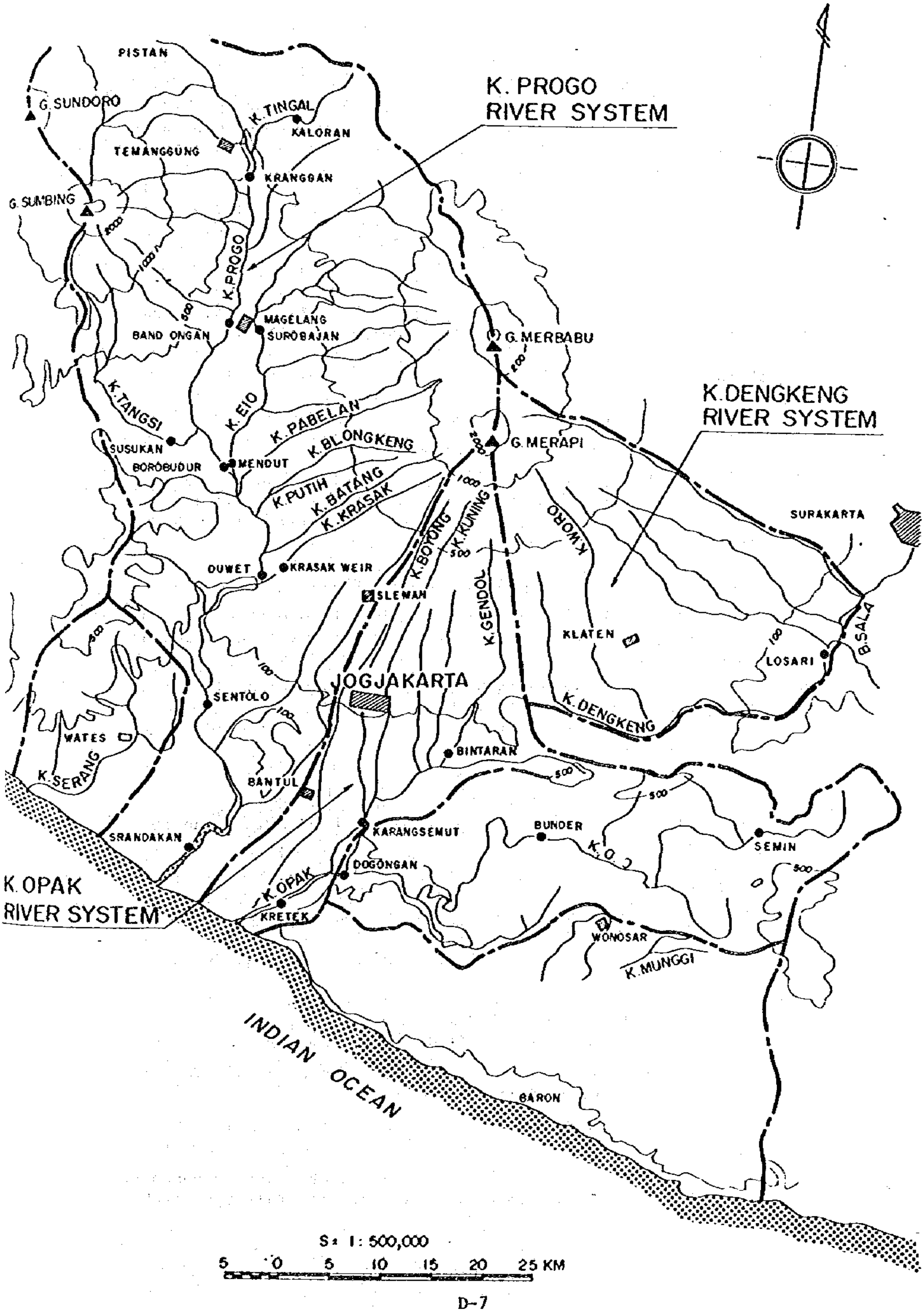
(1) K. Progo

Since the flow capacity of the existing river channel with the completed embankment is estimated at about 5,000 cu.m/sec., the channel capacity may be large enough to safely take care of floods equivalent to the one in 1969.

(2) K. Opak

The flow capacity of the existing river channel with the completed embankments is estimated between 2,500 and 3,000 cu.m/sec. on the right bank (heavily populated area). This capacity will diminish the chance of flood inundation based on the discharge experienced in the flood in 1966 of 1,500 cu.m/sec. and provided there is no appreciable deviation in the rate of the riverbed evolution.

Fig.1 Map Of Main River Systems



1.2 Types of Problem Spots

A careful survey of spots along the rivers revealed various problems listed below in detail such as damage to irrigation intakes caused by silt deposit, embedding, estuary block-up, water inundation due to landslides, riverbank lateral erosion, etc.

1.2.1 Damage to Irrigation Intakes

(1) K. Progo

Location of intakes along the K. Progo is shown in Fig. 13. The actual condition of damage of the intakes is as follows:

Jati Intake:

- Year of Construction: 1973
- Irrigation Area: 400 ha.
- Suffered heavy flow-in and deposit of silt. Maintenance by annual excavation requiring 500 workers over 5 days.

Sapon Intake:

- Found at present in good working order
- The existing intake was built in 1969; before then, it was located at Bakalan about 1.5 km downstream from the present location.
- The Bakalan Intake suffered heavy lateral erosion from the flood in 1969 so that a width of about 40 m was destroyed.

Kamijoro Intake:

- Year of Construction: 1924
- Several years after its completion, it was buried beneath the silt

Makam Bulan Intake:

- Year of Construction: 1927
- Ceased useful functioning because of a heavy silt deposit

Mangir Intake:

- Year of Construction: 1965
- Intake from the main river by an open cut channel. To-date, it does properly fulfill its function

(2) K. Opak

As shown in the location map (Fig. 14) the intakes are composed of six weirs of the full transverse-type and six lateral-type intakes.

All intakes, including Blawong and Canden, which were completed in 1977, are located at points more than 10 km upstream from the estuary. Though the intakes are filled with silt, they still functions properly.

The lateral-type intakes are closed during rainy seasons to avoid intrusion of silt and their functions are suspended accordingly during the season. The bottom of the intakes are located about 0.5 m above the riverbed.

Water-intake during dry seasons are by means of temporary full transverse-type weirs (construction cost: Approx. Rp.1,000,000.-) Since the stream flow is rather meager and the bottom of the intakes are located above the riverbed. Bamboo, banana leaves and sand are used for the construction of the temporary weirs which are usually flushed away by floods in the following rainy season.

1.2.2 Maintenance Problems of Intakes

It is necessary to maintain the bottom of the intake as close as possible to the surface of the riverbed. This makes maintenance extremely difficult where the riverbed depositing is heavy.

1.2.3 Landside Water Inundation

(1) K. Progo

Besides inundation caused by overflowing, inundation caused by landsides is observed particularly in the neighborhood of the estuary. Floods were reported along the right bank of the K. Galur which flows into the main river in the neighborhood of its estuary.

Since the estuary of the main river is almost always blocked up during dry seasons, its discharge of the main river, flows back into the channel of the K. Galur, at a point close to the estuary. This backwater is the cause of poor drainage and inundation over the area along the tributary. At present, the backwater from the main river is held by a wooden balance gate using natural water level control. Although gate is used for control of inland and the main river water, its operation has deteriorated and is hampered by the outgrowth of aquatic plants in well nourished water of the protected lowland along the K. Galur.

This situation is also generally true of the right bank area where inundation is caused by inland water along the K. Trihudati, a tributary joining to the main in the neighborhood of the estuary. In this case, too, a wooden balance gate is used for the control of the water level of inner and outer sides of the gate.

More efforts towards improvement of the administration and maintenance of the gates is highly advisable.

(2) K. Opak

As yet, no significant damage caused by inland water inundation was reported; however, as the estuary block-up is more prominent in dry seasons, an ample cause exists for the occurrence of a problem in the future.

1.2.4 Estuary Block-Up

Due to tidal flows caused by high westerly winds from the sea and meager stream flow during dry seasons, the estuaries of both the K. Progo and K. Opak are usually blocked during dry seasons by a tremendous volume of silt and debris transported by floods during preceeding wet season. Excavation by manual labor is now used to prevent an inundation problem caused by the inland water.

1.2.5 Lateral Erosion of River Banks

Heavy damage on the river banks caused by lateral erosion in several places was reported. In one case mentioned previously, the wash-out of intakes and loss of farmland resulted from about 40 m of lateral erosion, at the point about 10 km upstream from the estuary of the K. Progo at the time of the flood in March 1969.

1.2.6 Summary of Problem Causes

In conclusion, the following is a summary of causes of the primary problems along the river channels.

a) Large volume of sediment discharge from upstream

b) Riverbed evolution and unstable channel gradient

c) Fine granulation of riverbed materials

d) Unsteady meandering characteristics

e) Considerable differences in discharge between wet and dry seasons

f) The resultant decrease of river flow maintenance due to the location of irrigation intakes upstream.

g) Lack of proper administration and maintenance of river structures

h) Effects of strong tidal flow

i) Steep declining slope of coastal seabed

Table 1 Inundation Damage

	K. PROGO						K. OPAK			
	1969		1974		1975		1965		1966	
	L	R	L	R	L	R	L	R	L	R
Inundated Villages (Desa)	19	10	19	10	19	10	23	48	25	62
Inundated Area (ha.)	500	330	140	130	460	190	235	520	520	1,160
Farmland (ha.)	40	225	-	-	-	-	191	300	450	650
Yards (ha.)	33	100	-	-	-	-	138	300	220	420
Pop. Injured	61KK 793	- 28	- -	- -	- -	- -	- 2,193	- 3,000	- 6,900	411KK 9,068
Pop. Dead or Disappeared	0	0	-	-	-	-	-	1	7	1
Broken Houses	-	26	-	-	-	-	65	100	178	153
Inundated Houses	-	392	-	-	-	-	-	-	630	3,230
Broken Bridges	-	4	-	-	-	-	-	7	-	10
Broken Irrigation Facilities	-	-	-	-	-	-	-	17	-	17
Broken Dikes (m.)	-	-	-	-	-	-	-	-	-	-

Note: L - Left Bank
R - Right BAnk
KK -

Table 2 Flow Capacity of K. Progo

Section	Ground Height				Flow Capacity			
	Left Side		Right Side		Left Side		Right Side	
	Levee	River Bank	Levee	River Bank	Levee	River Bank	Levee	River Bank
	DL. m.	DL. m.	DL. m.	DL. m.	cum/s	cum/s	cum/s	cum/s
3	3.2	2.5	6.0	4.0	-	550	-	1,650
4	-	4.8	-	5.5	-	950	-	1,900
5	8.5	6.5	-	8.0	5,200	900	-	3,650
6	10.0	8.0	-	9.5	5,500	750	-	3,300
7	13.0	11.0	-	10.5	**	4,250	-	3,100
8	-	14.8	-	15.8	-	7,500	-	9,800
9	-	17.5	-	19.0	-	7,100	-	**
10	19.8	15.5	-	19.5	6,500	1,000	-	5,800
11	-	21.5	-	17.6	-	4,600	-	1,300
12	-	24.0	-	23.0	-	3,250	-	5,900
13	-	23.0	-	22.0	-	5,250	-	3,850
14	-	27.6	-	26.5	-	**	-	**
15	-	30.5	-	26.5	-	**	-	8,600
16	-	35.0	-	31.0	-	**	-	**
17	33.7	32.5	-	*	**	**	-	**
18	-	40.5	-	38.0	-	**	-	**
19	-	36.5	-	*	-	**	-	**
20	-	39.0	-	39.0	-	**	-	**
21	-	45.0	-	41.0	-	**	-	**
22	-	43.0	-	43.0	-	8,700	-	8,700
23	-	44.0	-	44.0	-	**	-	**

Note: * Sufficient Height
** Sufficient Flow Capacity

Table 3 Riverbed DL and Cross Levelling Intervals
K. Progo:

Cross Section No.	Unit Distance	Accumulated Distance	Riverbed, Height	
			Lowest	Mean
	m.	m.	DL. m.	DL. m.
1	0	0	-9.712	-7.149
2	398	398	-0.348	0.251
3	1,005	1,403	1.066	2.065
4	1,012	2,415	2.896	3.857
5	1,018	3,433	4.826	5.560
6	1,022	4,435	6.530	7.376
7	1,016	5,451	2.552	3.886
8	979	6,430	10.214	11.121
9	997	7,427	9.906	11.637
10	994	8,421	12.642	12.969
11	981	9,402	13.242	14.122
12	1,011	10,413	14.998	17.124
13	990	11,403	18.048	18.564
14	986	12,389	18.748	19.957
15	1,000	13,389	20.708	21.867
16	953	14,342	22.798	23.694
17	1,000	15,342	21.720	24.155
18	1,011	16,353	23.230	25.238
19	989	17,342	27.528	28.565
20	1,001	18,343	29.836	30.793
21	1,000	19,343	30.084	32.045
22	994	20,337	32.528	33.633
23	98	20,435	32.508	33.441

Table 4 Channel Width of K. Progo

Section No.	Channel Width, B	Section No.	Channel Width, B	Section No.	Channel Width, B
	m.		m.		m.
1.0	200	11.0	240	21.0	330
2.0	490	12.0	540	22.0	210
3.0	600	13.0	600	23.0	210
4.0	360	14.0	730	-	-
5.0	670	15.0	420	-	-
6.0	720	16.0	550	-	-
7.0	240	17.0	330	-	-
8.0	520	18.0	360	-	-
9.0	300	19.0	420	-	-
10.0	150	20.0	330	-	-

Table 5 Flow Capacity of K. Opak

Section	Ground Height				Flow Capacity			
	Left Side		Right Side		Left Side		Right Side	
	Levee	River Bank	Levee	River Bank	Levee	River Bank	Levee	River Bank
	DL. m	DL. m	DL. m	DL. m	cum/s	cum/s	cum/s	cum/s
3	5.4	3.5	5.8	3.5	1,700	250	2,180	250
4	8.5	5.5	7.5	5.0	2,700	330	1,430	180
5	9.5	7.0	8.5	6.5	1,280	400	700	300
6	-	14.3	11.3	8.5	-	4,400	2,200	300
7	-	10.5	12.3	10.5	-	650	1,950	650
8	-	11.5	14.0	11.0	-	500	2,330	300
9	-	19.3	15.7	12.5	-	7,000	2,250	200
10	-	15.0	18.2	14.0	-	450	2,850	200
11	18.0	16.5	19.5	15.5	1,400	650	2,450	300
12	-	23.0	20.7	17.5	-	3,930	2,150	530
13	-	19.0	21.7	19.0	-	450	2,000	450
14	22.5	21.0	23.2	21.5	850	280	1,300	430
15	25.4	23.0	25.1	23.5	1,030	250	850	350
16	27.0	26.5	28.0	26.0	850	700	1,180	550
17	29.2	28.5	28.7	26.0	1,380	1,050	1,130	300
18	-	31.0	-	31.0	-	880	**	880
19	-	34.0	-	32.5	-	880	**	400
20	-	37.5	-	38.0	-	2,000	**	2,300
21	-	39.0	39.5	38.5	-	1,500	1,800	1,250
22	-	52.0	-	42.5	-	8,000	**	2,000
23	-	*	-	44.5	-	**	**	1,600
24	47.3	45.0	-	49.0	1,900	600	**	3,250
25	-	48.5	-	49.0	-	1,250	**	1,650
26	-	51.5	-	51.5	-	1,450	**	1,450
27	-	54.5	-	56.0	-	1,900	**	3,000
28	-	-	-	73.0	-	-	-	8,000
29	-	61.0	-	60.0	-	2,250	-	1,750
30	-	64.0	-	64.0	-	2,200	-	2,200
31	-	71.0	-	67.0	-	4,400	-	2,150
32	-	73.0	-	73.0	-	3,900	-	3,900
33	-	74.0	-	74.0	-	2,000	-	2,000
34	-	80.0	-	79.5	-	5,300	-	4,250
35	-	-	-	85.0	-	-	-	-
36	-	92.0	-	87.8	-	-	-	-
37	-	92.8	-	92.8	-	-	-	-

Note: * Sufficient Height
** Sufficient Flow Capacity

Table 6 Riverbed and Cross Levelling Intervals
K. Opak:

Section No.	Distance		Riverbed Height	
	Unit	Accumulated	Lowest	Mean
	m.	m.	DL. m.	DL. m.
1	0	0	0.146	0.633
2	947	947	1.138	1.866
3	1,009	1,956	2.216	2.884
4	1,034	2,990	4.092	4.854
5	1,005	3.995	4.946	5.604
6	1,025	5,020	6.472	7.355
7	1,028	6,048	7.336	8.201
8	1,053	7,101	9.264	10.094
9	1,018	8,119	10.396	11.350
10	1,033	9,152	11.590	12.441
11	1,025	10,177	12.072	13.723
12	1,030	11,207	14.996	15.579
13	1,005	12,212	15.852	16.587
14	1,002	13,214	18.424	19.362
15	1,053	14,267	19.976	20.852
16	1,047	15,314	23.404	24.088
17	977	16,291	24.460	25.921
18	1,020	17,311	27.292	27.817
19	1,019	18,330	28.680	29.895
20	1,043	19,373	31.732	32.579
21	1,003	20,376	33.028	34.570
22	1,004	21,380	36.166	36.998
23	1,000	22,380	38.440	39.257
24	1,003	23,383	41.142	42.411
25	1,005	24,388	43.404	45.091
26	1,012	25,400	45.522	47.033
27	1,045	26,445	48.162	49.744
28	1,017	27,462	49.606	51.441
29	1,024	28,486	52.356	53.678
30	1,037	29,523	53.934	55.113
31	1,024	30,547	56.976	58.112
32	1,048	31,547	64.348	66.217
33	1,007	32,602	66.380	67.596
34	1,002	33,604	69.306	70.560
35	987	34,591	73.618	74.876
36	1,054	35,645	79.318	80.372
37	786	36,431	80.998	82.704

Table 7 Channel Width of K. Opak

Section No.	Channel Width, B m.	Section No.	Channel Width, B m.	Section No.	Channel Width, B m.
1.0	540	14.0	70	27.0	40
2.0	580	15.0	60	28.0	60
3.0	300	16.0	220	29.0	30
4.0	220	17.0	180	30.0	30
5.0	190	18.0	60	31.0	35
6.0	200	19.0	90	32.0	30
7.0	180	20.0	90	33.0	35
8.0	170	21.0	80	34.0	50
9.0	90	22.0	60	35.0	90
10.0	100	23.0	40	36.0	90
11.0	130	24.0	60	37.0	120
12.0	140	25.0	40	-	-
13.0	70	26.0	60	-	-

Fig. 2 Inundation Area of the 1969, 1974 and 1975 Floods Along K. Progo

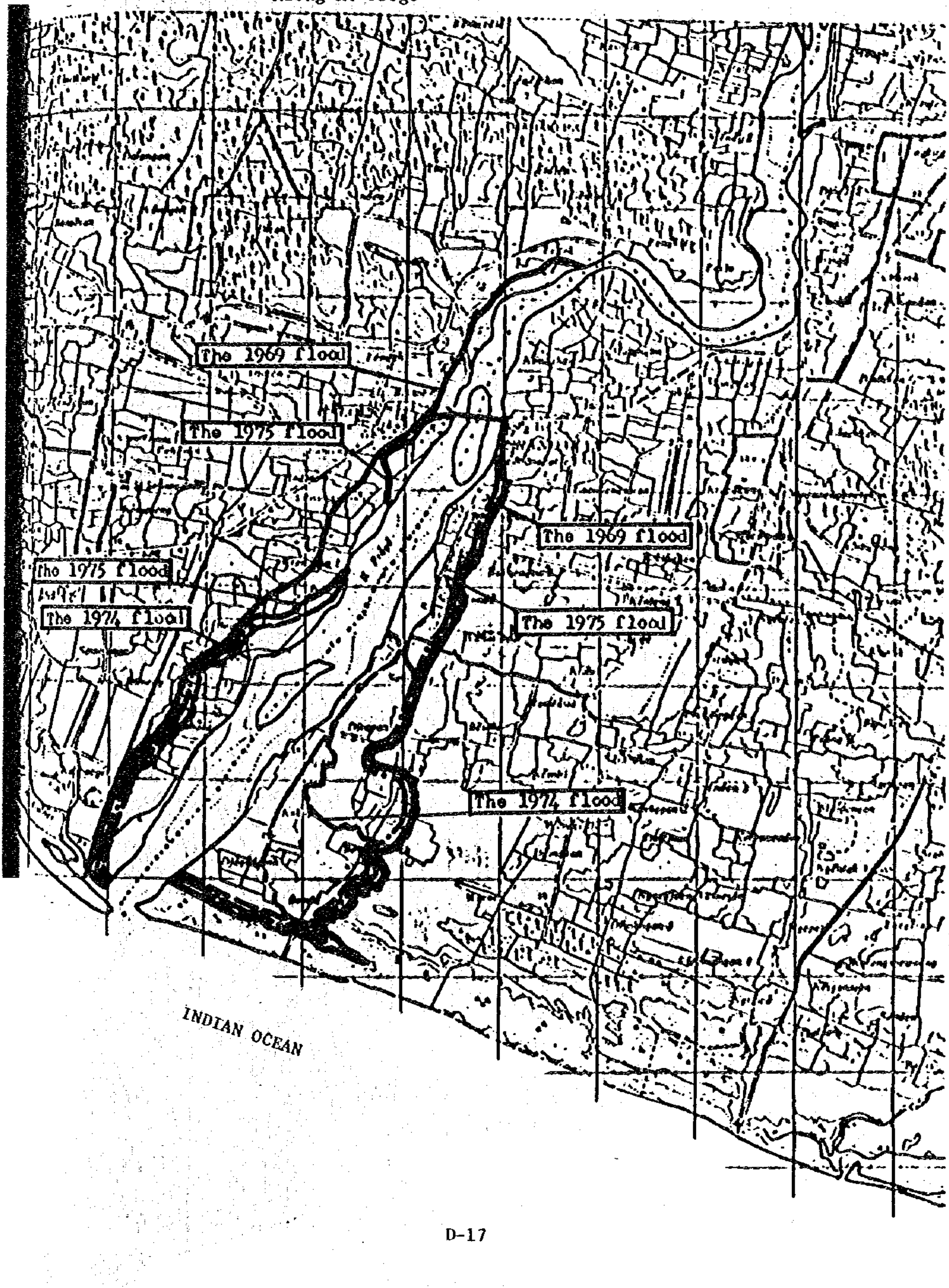


Fig. 3 Water Depth in K. Progo Inundated Area

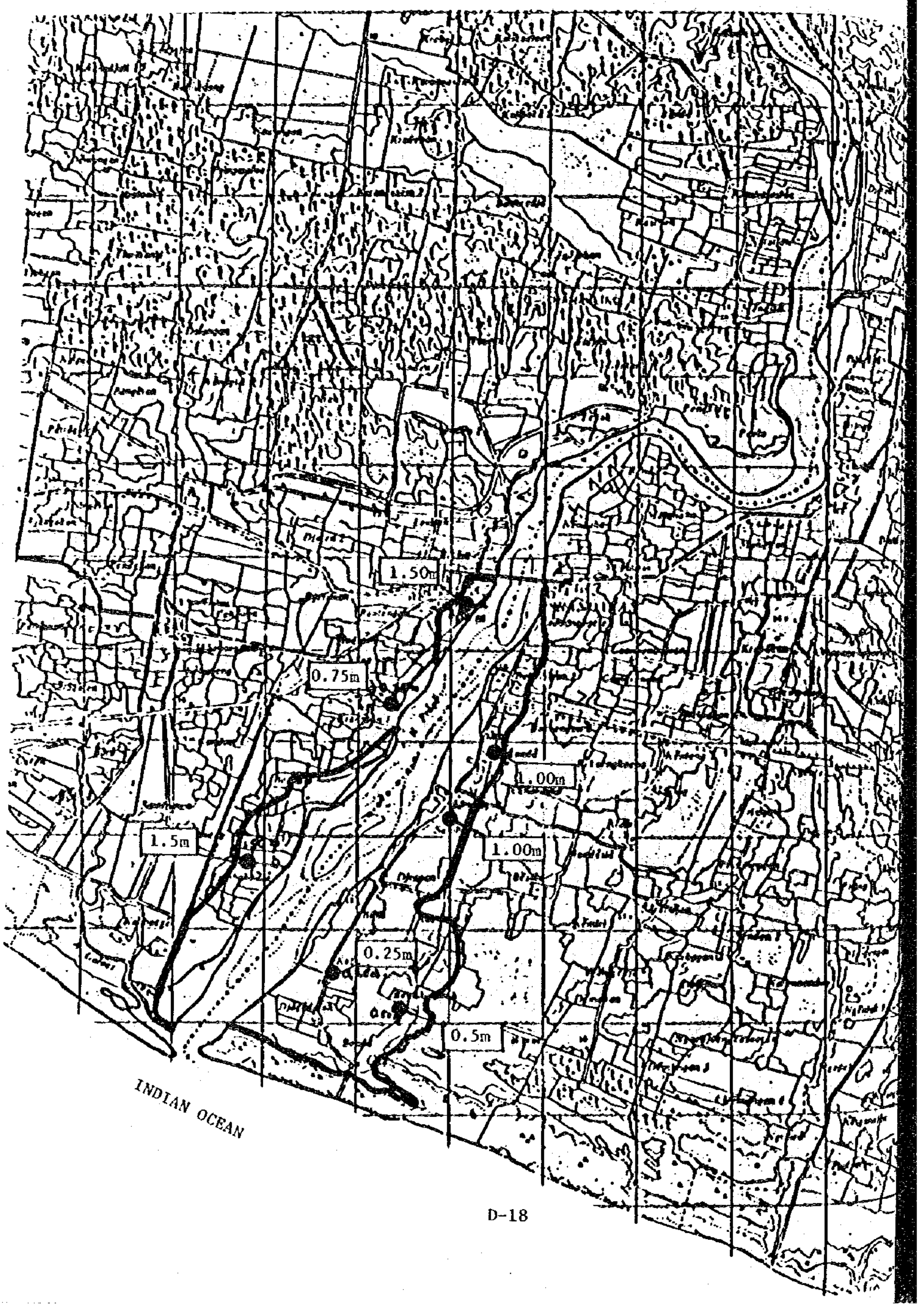


Fig. 4 Inundation Area of the 1966 and 1966 Floods Along K. Opak

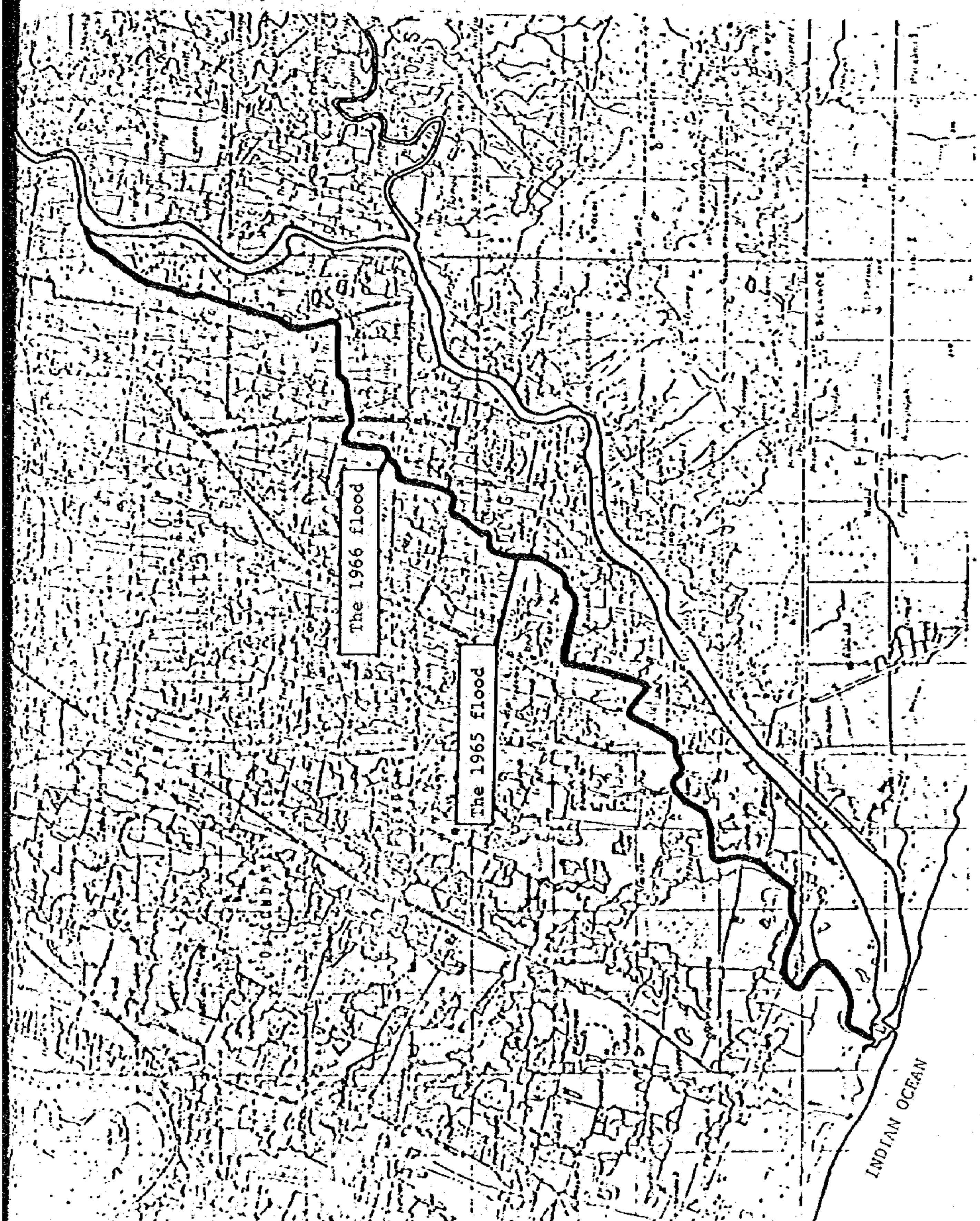


Fig. 5 Water Depth in Inundated Area

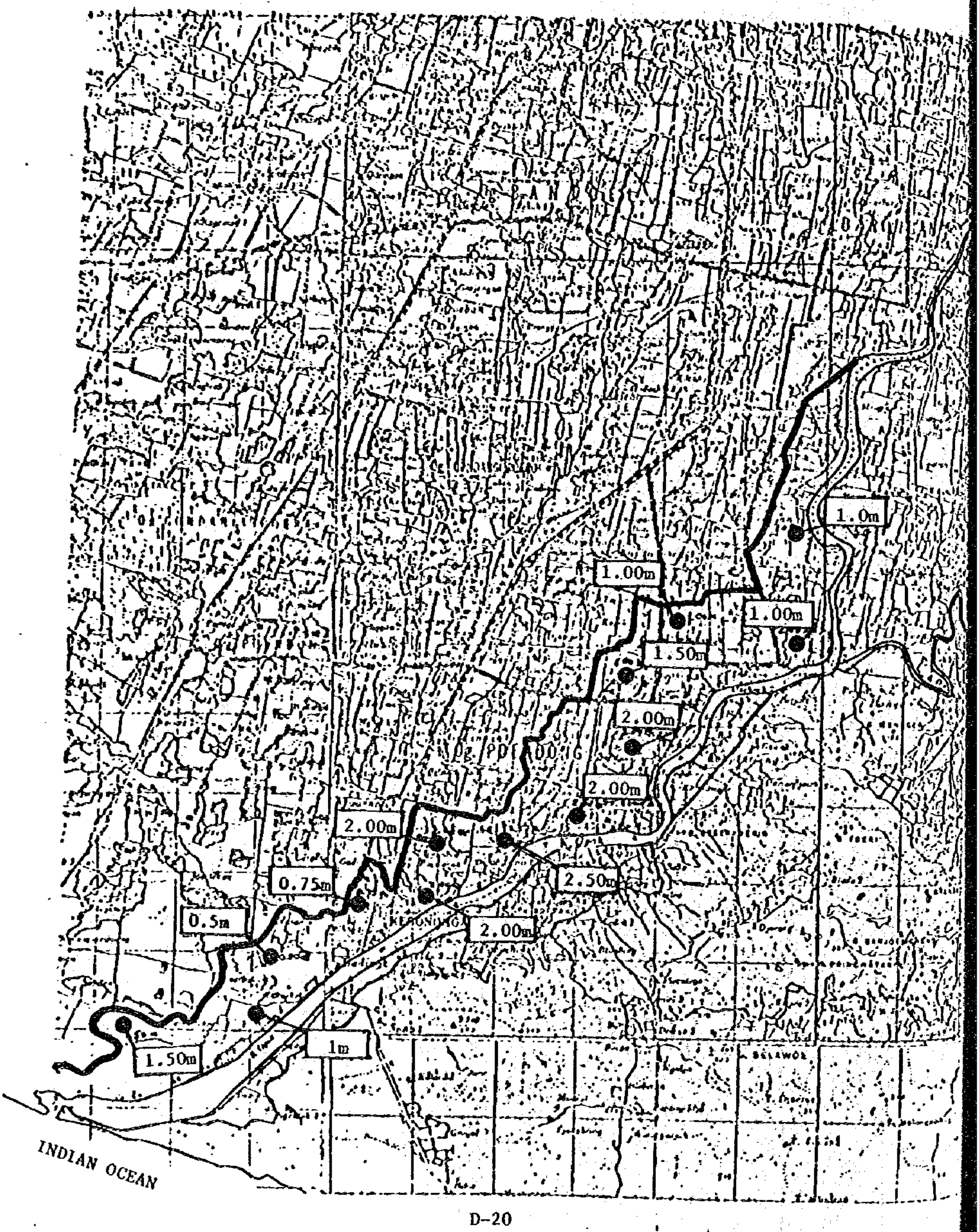
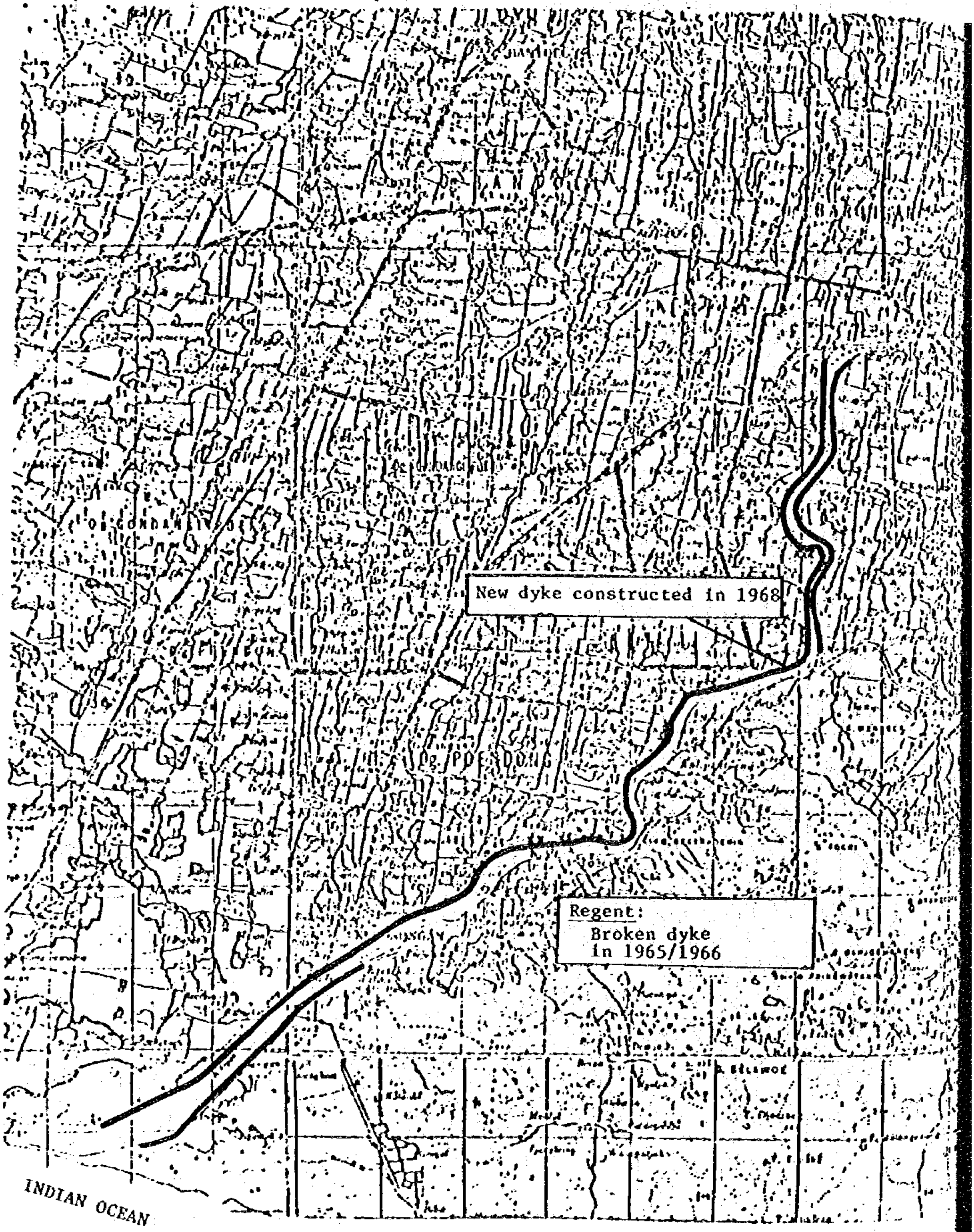


Fig. 6 Location of Existing Dyke Along K. Progo

Fig. 7 Location of Existing Dyke and Broken Dyke Along K. Opak



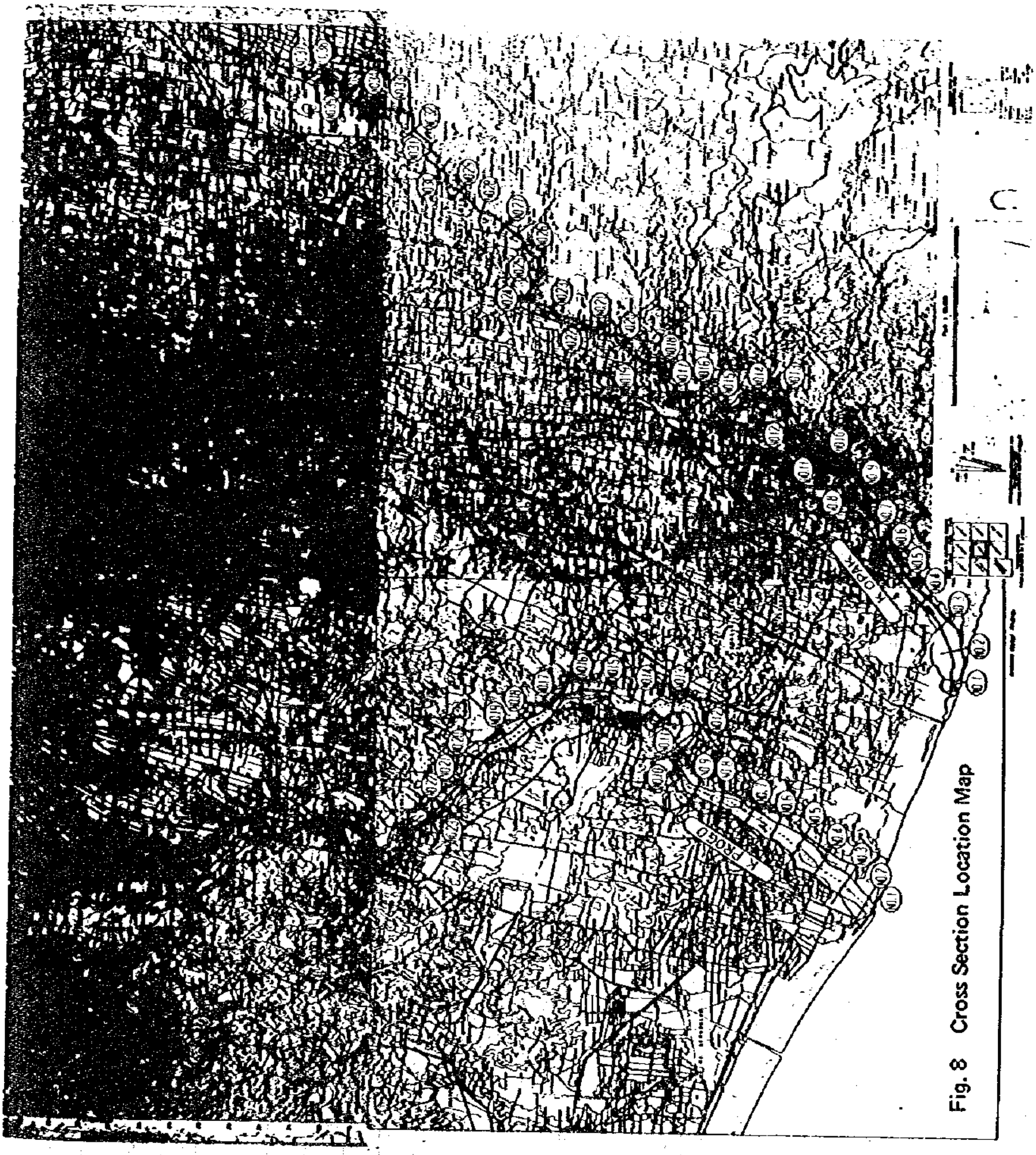


Fig. 8 Cross Section Location Map

Fig. 9 Longitudinal Section and Flow Capacity of K. Progo

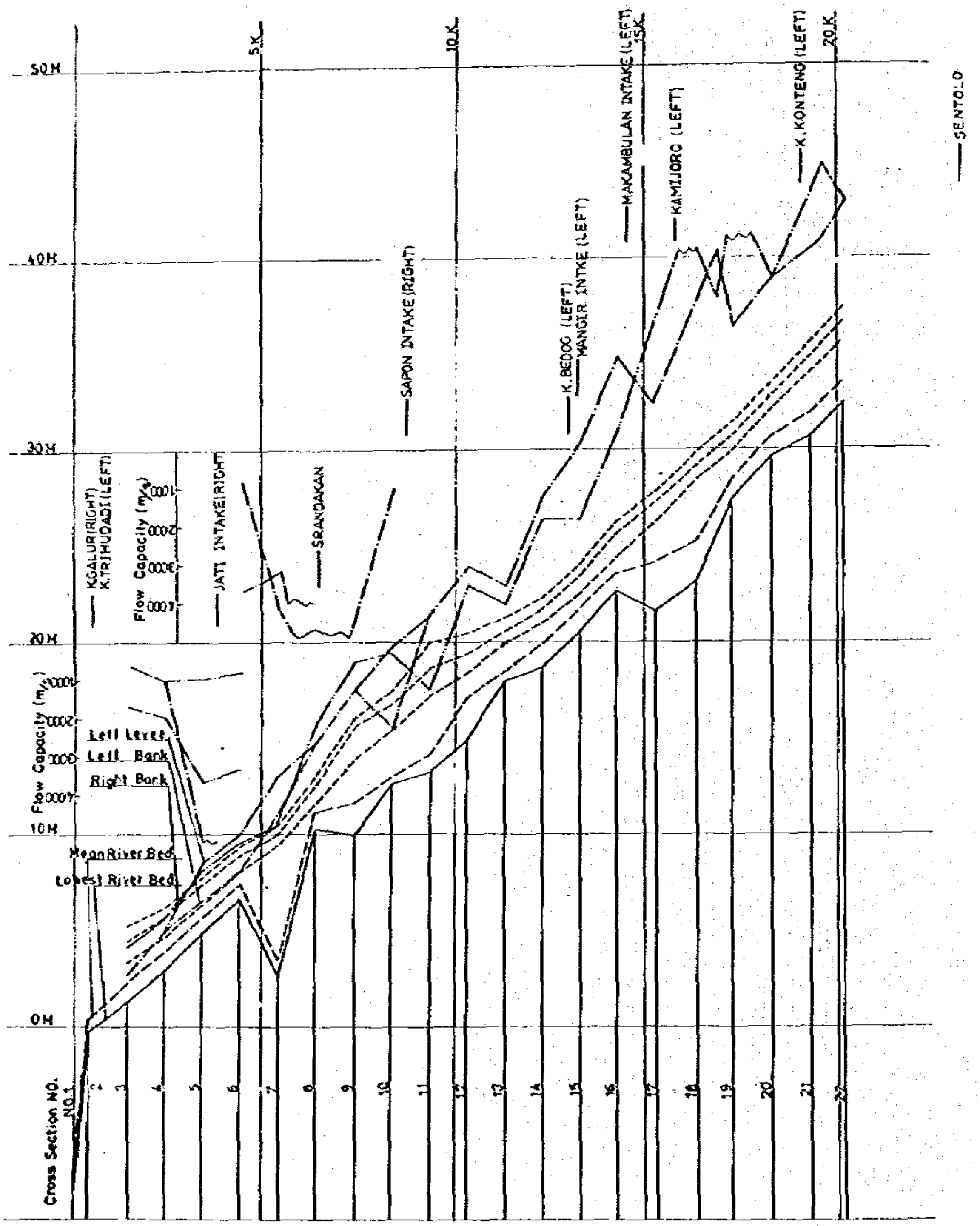
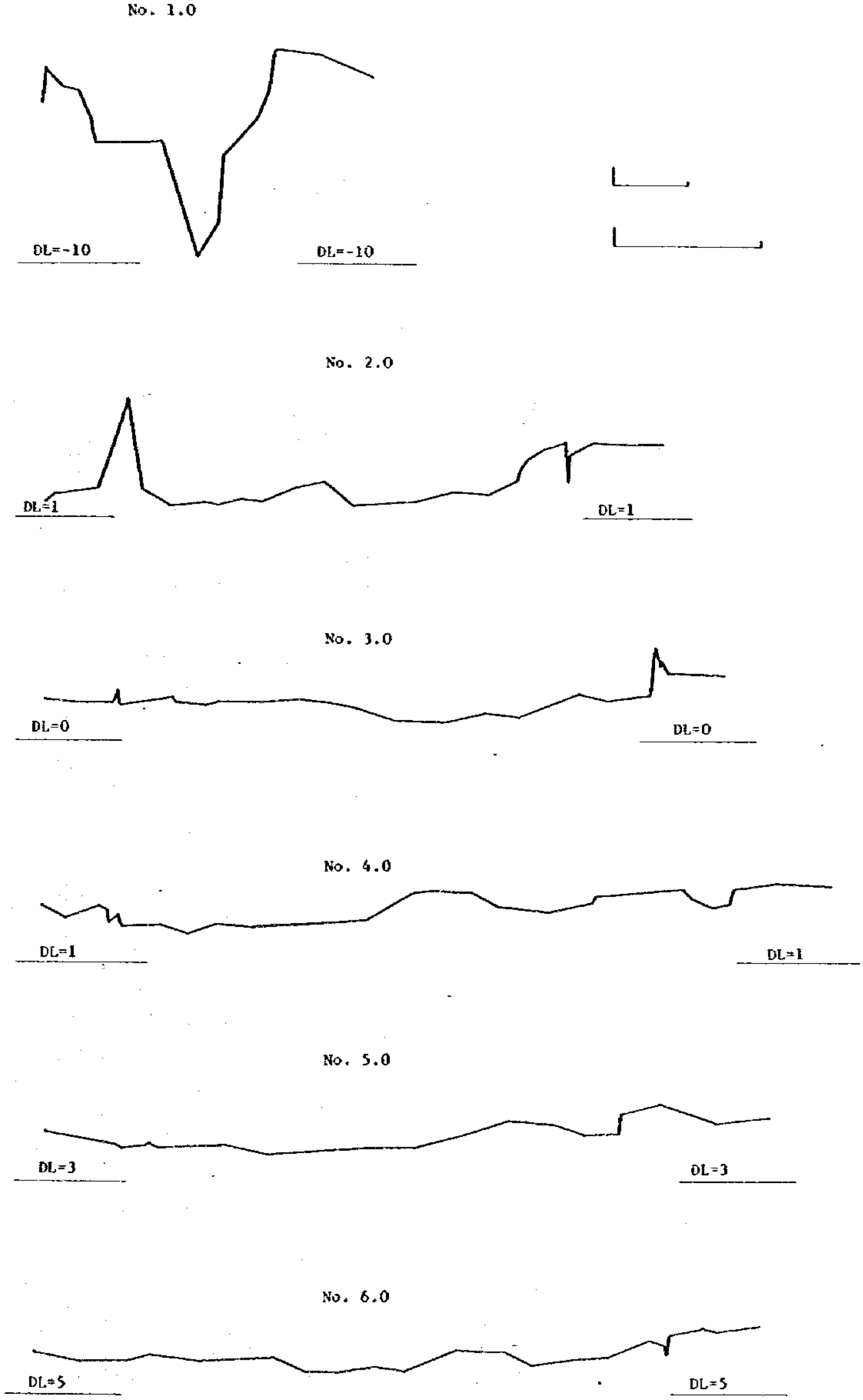


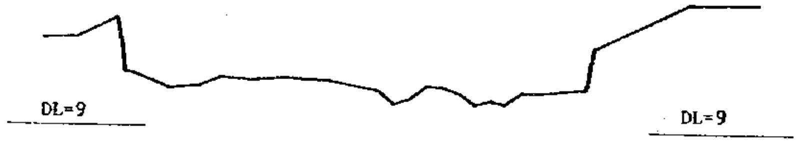
Fig. 10 Cross Section of K. Progo



Cross Section of K. Progo

No. 7.0

No. 8.0



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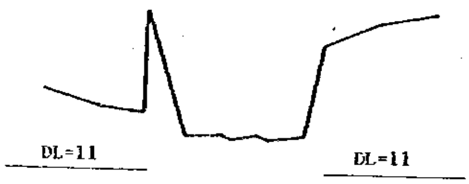

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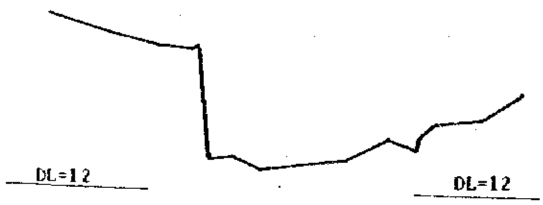

Cross Section of K. Progo

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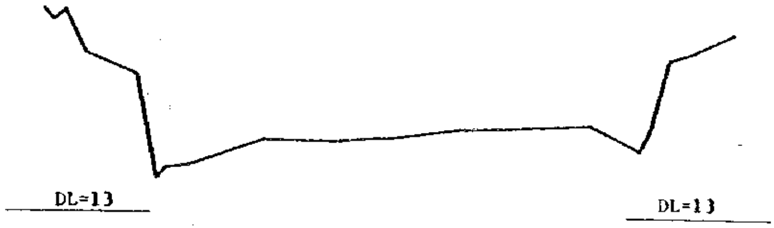

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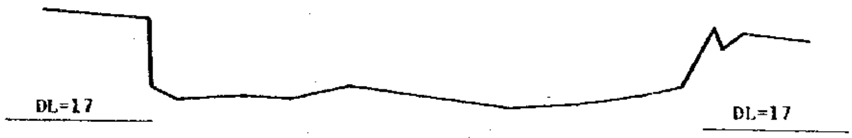

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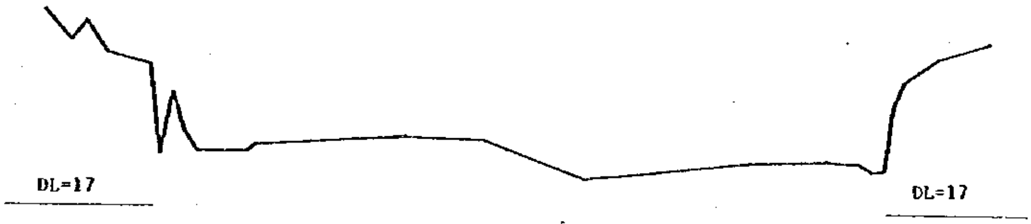

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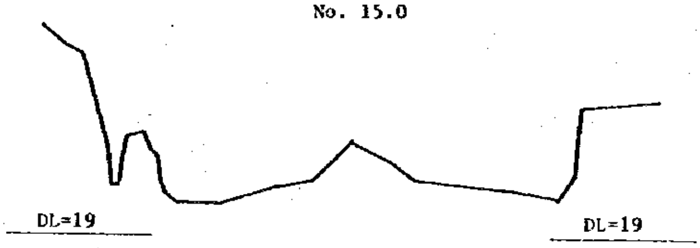

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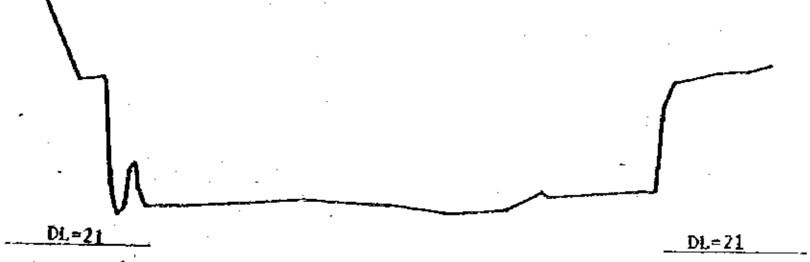
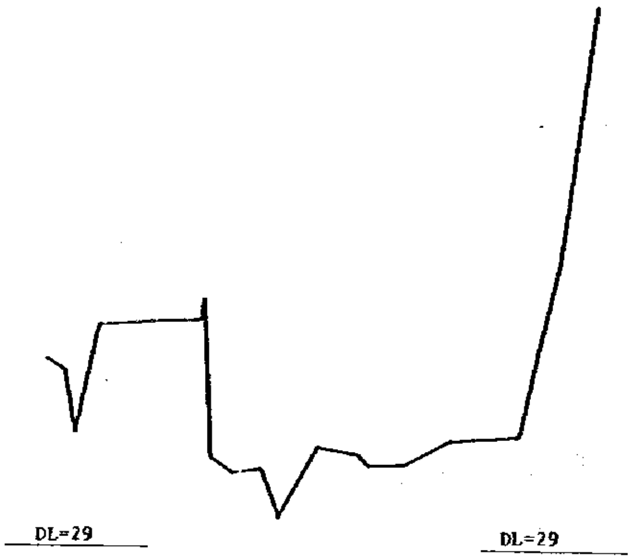


Cross Section of K. Progo

No. 17.0



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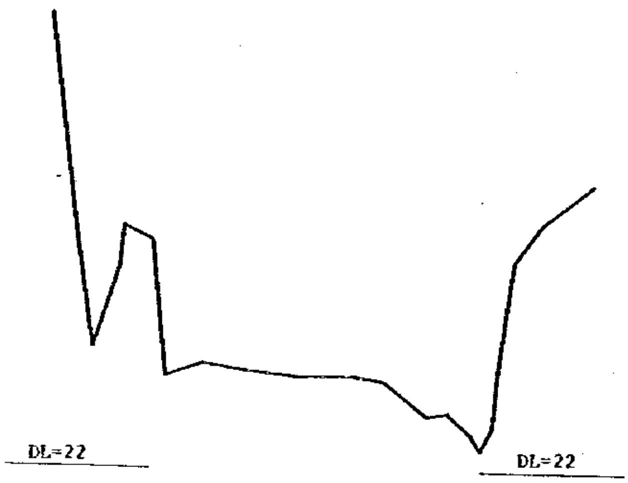

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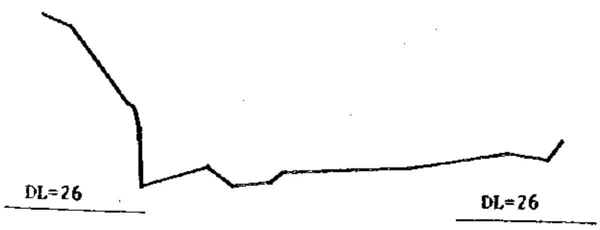
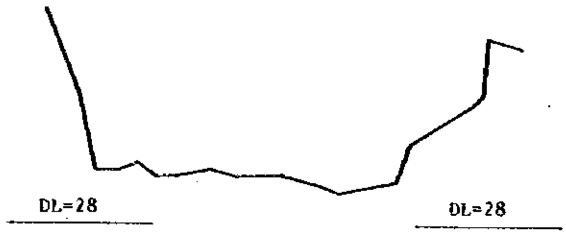


Cross Section of K. Progo

No. 20.0



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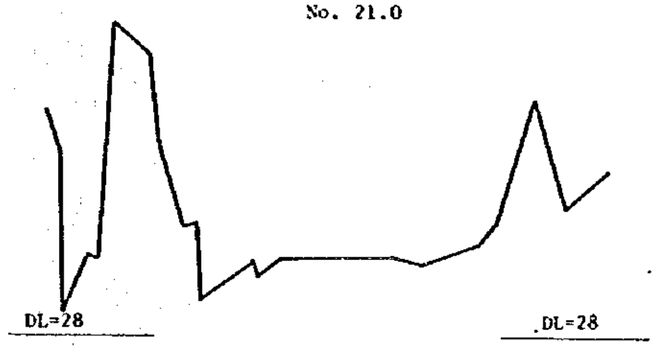

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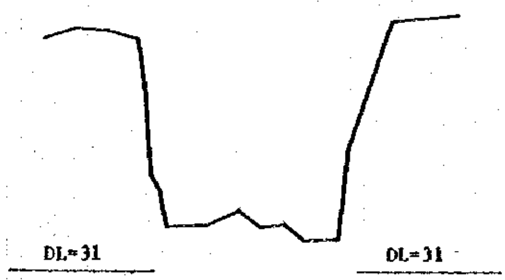


Fig. 11 Longitudinal Section and Flow Capacity of K. Opak

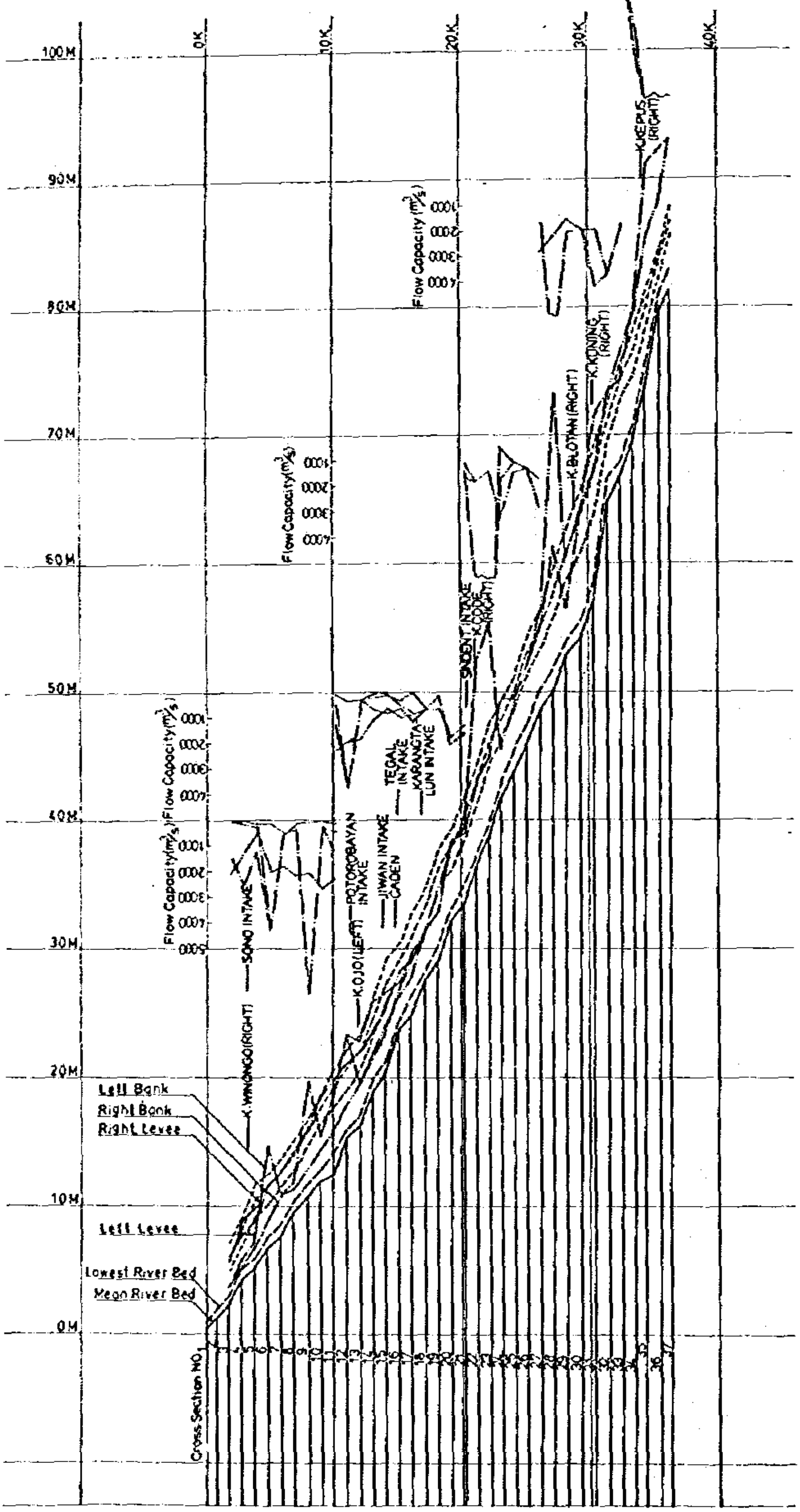
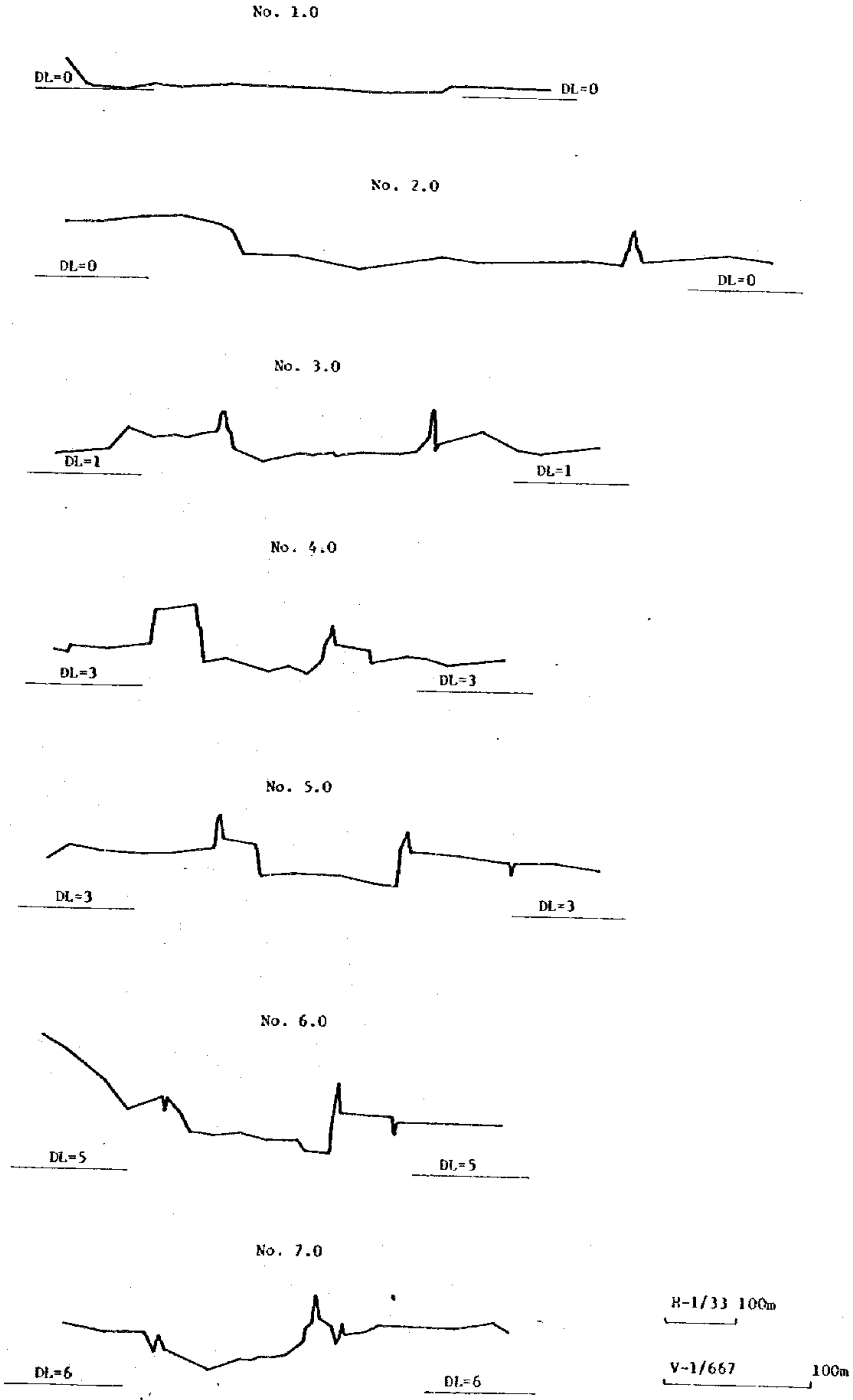


Fig. 12 Cross Section of K. Opak



Cross Section of K. Opak

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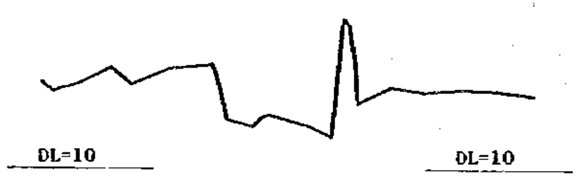

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Cross Section K. Opak

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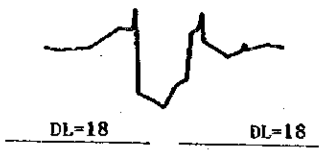

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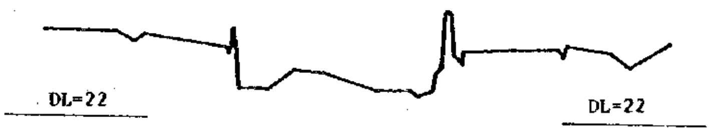

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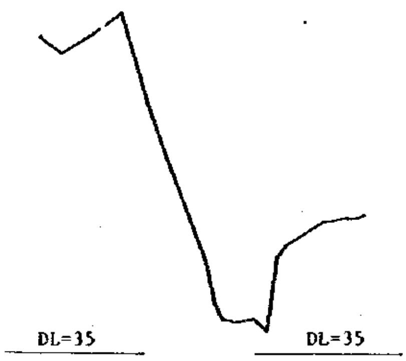

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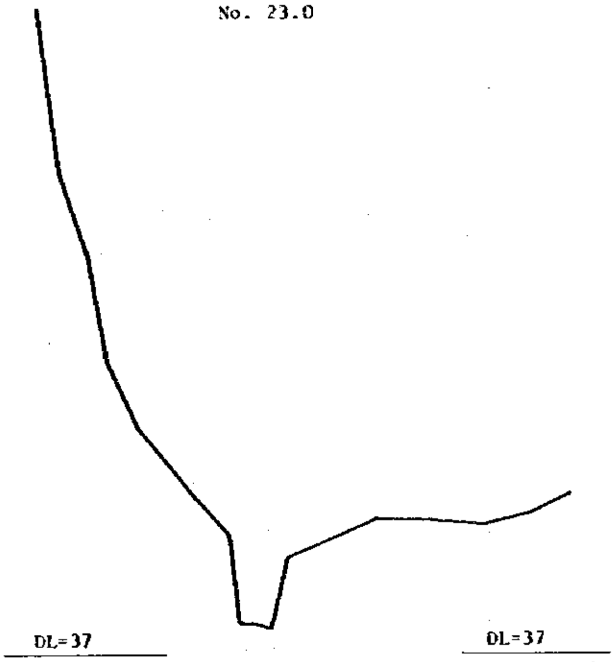

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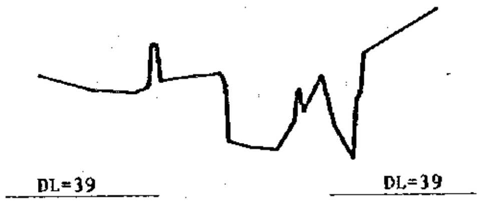

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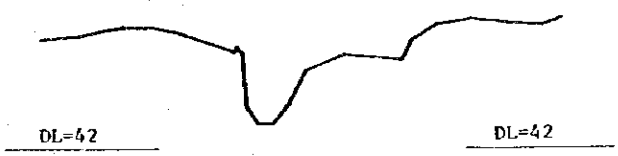

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Cross Section of K. Opak

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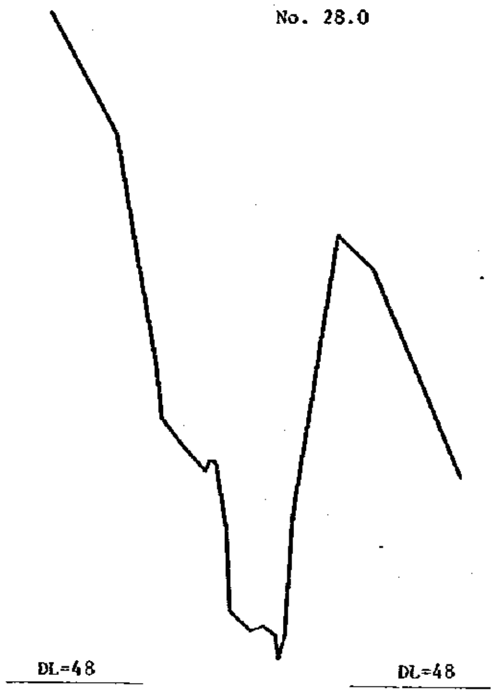

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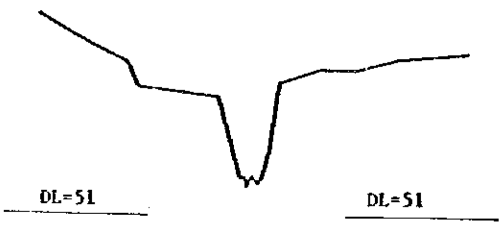

CROSS SECTION OF K. OPAK

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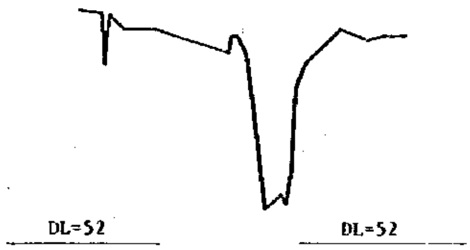

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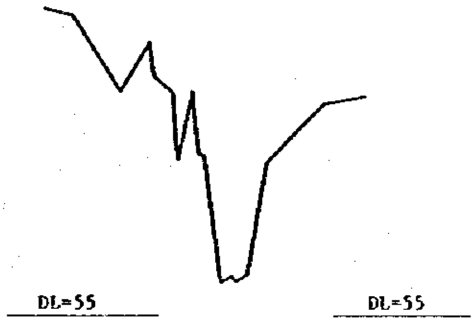

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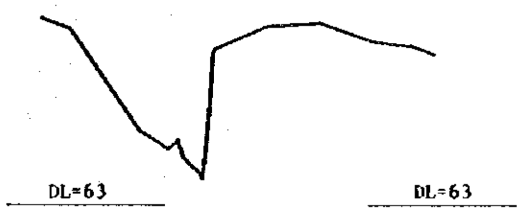

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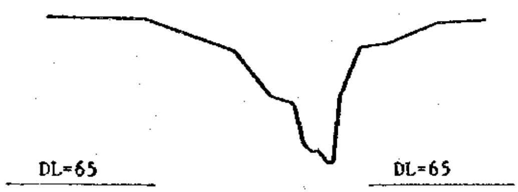

Cross Section K. Opak

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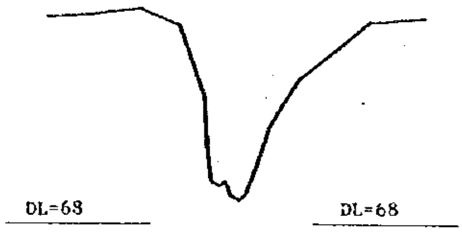

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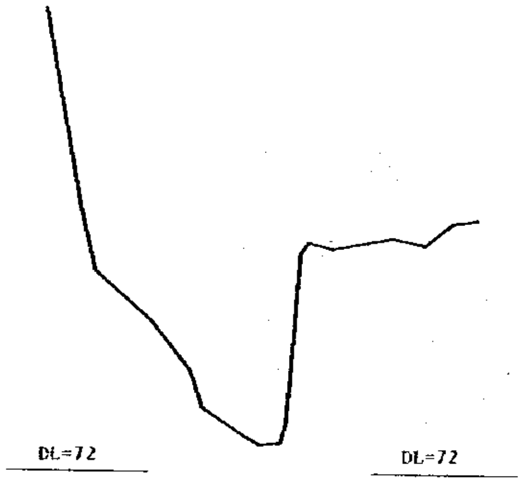

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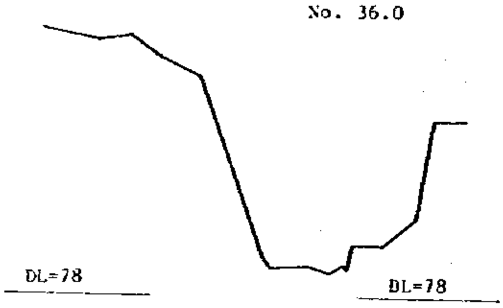

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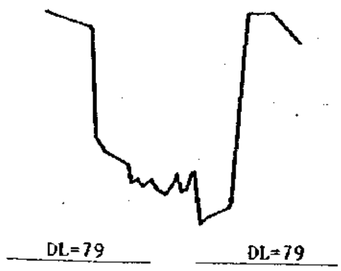


Fig. 13 Location of Irrigation Intakes Along K. Progo

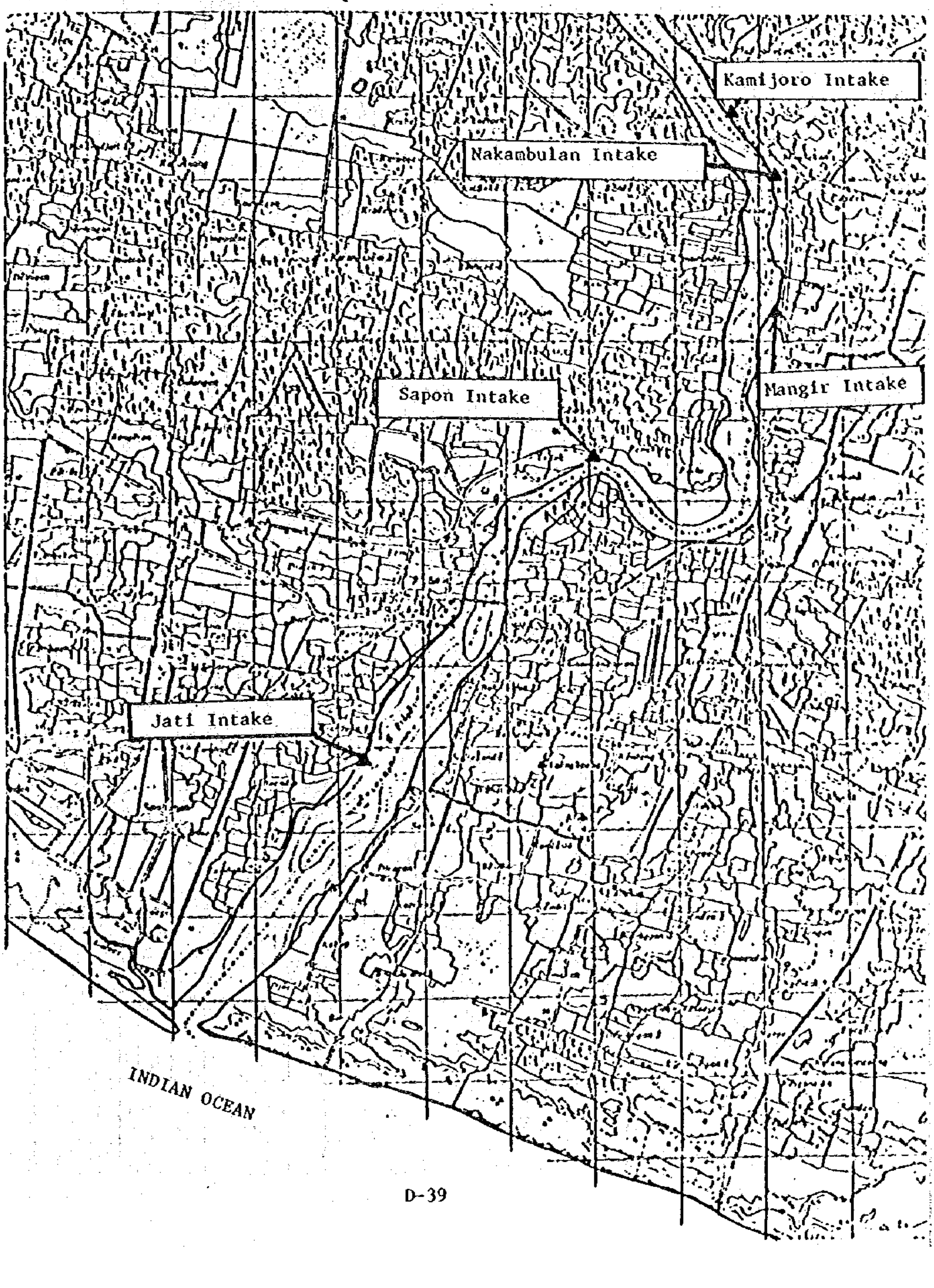
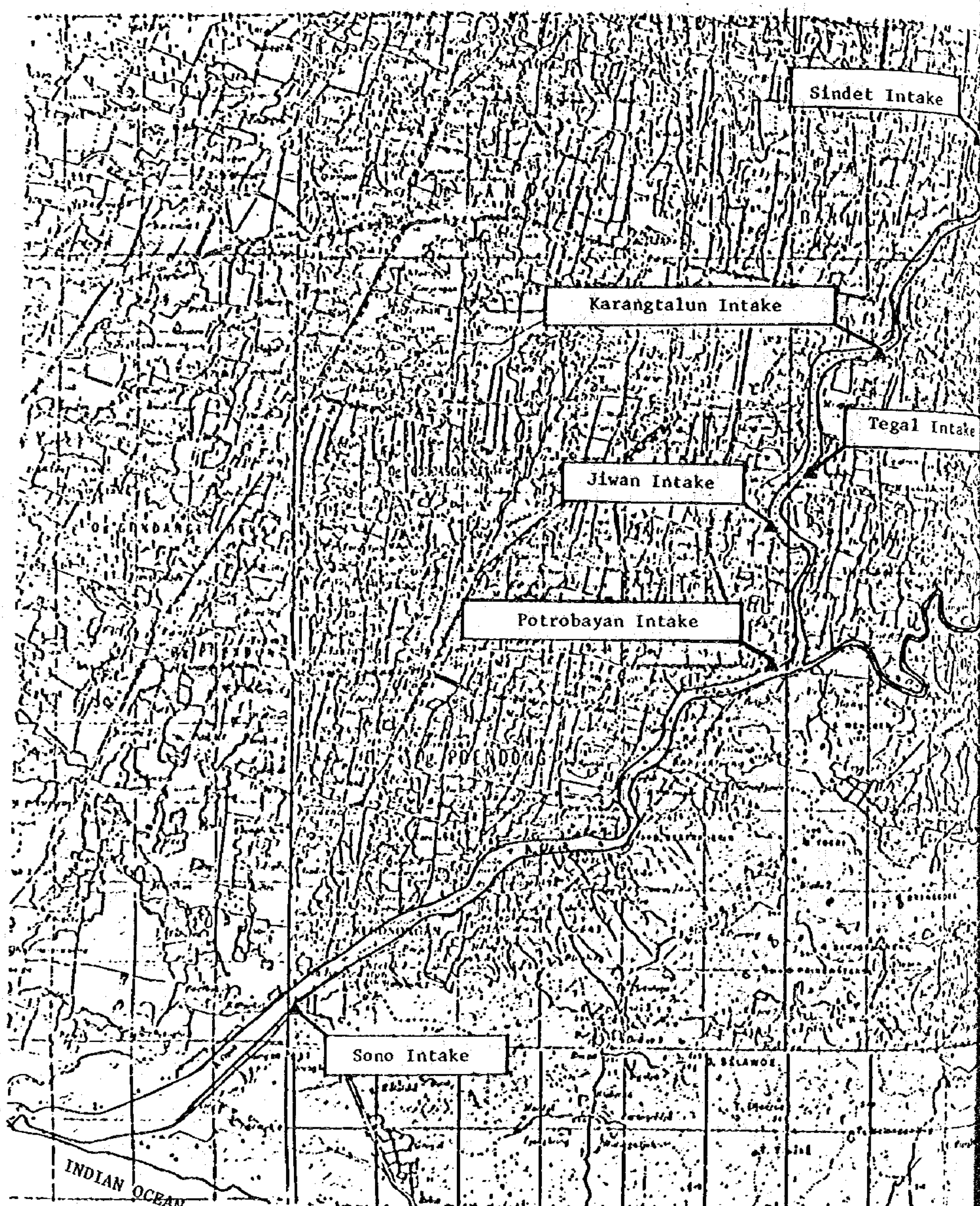


Fig. 14 Location of Irrigation Intakes Along K. Opak



2. COUNTERMEASURES TO RIVER PROBLEM SPOTS

Since flood inundation and blockage due to damage of irrigation intakes are considered as major damage caused by the rivers, the following basic policies are proposed to prevent such damage from occuring in the future.

2.1 Basic Countermeasures

2.1.1 Control of Sediment Load

The K. Progo suffers from sediment loads transported down by repeated floods and a considerable volume of silt deposit can be observed over the channel of the river. The eruption of G. Merapi in 1969 and floods during the succeeding wet season brought a silt deposit of about 7,000,000 cu.m to the river channel between the confluence of the K. Krasak and the estuary.

The basic concept of flood control is to secure the steadiness of the riverbed is practically possible.

Accordingly, it is essential to check and control the sediment discharge below the rated flow capacity of the river channel by effective Sabo control projects to be constructed in the upstream area.
the above mentioned section into consideration, it is recommended that
On the other hand, it is also essential to increase the flow capacity of the river channel to the possible maximum. The flow capacity of the river channel reported in supporting report "C", estimated that the sediment load of K. Progo is 3,240,000 cu.m/year at the site downstream of the Srandakan Bridge, and 33,400,000 cu.m/year at the Duet Gauging Station in the upstream area. The sediment load capacity of the latter is about five times higher than that of the former. It is reported that the stream gradient of the section of the river between Duet and the estuary is an almost steady 1:600. To regulate the flood discharge below the proposed HWL and taking an anticipative increase of flow capacity of the above mentioned section into consideration, it is recommended that the river width in the lower reach, presently 500 - 700 m, should be narrowed to a width reasonably corresponding to the existing river width of the upstream area and that the riverbed should also be excavated as required.

2.1.2 Stabilization of Meander

A steep stream gradient and alluvial fans are observed along the river channels of both the K. Progo and K. Opak. In the lower reach of the K. Progo where the river has a great width, it is observed that sand dunes are distributed extensively in the riverbed.

Under these circumstances, the major stream line at the time of flood is extremely unstable; hence, the impact zone is also unstable. This situation enhances the risk of lateral erosion of the river bank, and as a result, loss of valuable land along the full length of the river channel. Countermeasures should include a narrowing down of the existing river width in the way mentioned above, a change of the scale meander with alternating sand bars to eliminate the unstable meandering as much as possible, and protection of banks along the impact zones.

2.1.3 Construction of Training Dikes at Estuaries

The estuaries of the K. Progo and K. Opak are almost completely blocked during dry seasons without development of a delta to the sea at the river mouth from east to west.

This is caused by the tremendous volume of sediment transported to and deposited over the estuary during rainy seasons, and further by the low stream flow during dry seasons, the change of wind from east to west, and the strong tidal flow of the sea in front of the estuaries. On the island of Java, the above-mentioned estuary block-ups are generally observed in rivers flowing into the Indian Ocean; however, since the Java sea is shallow enough to allow the growth of deltas, practically no estuary block-up is observed there.

Construction of training dikes is recommended as an effective measure of prevention of estuary block-ups and elimination of damage caused by inland water inundation along the rivers.

2.1.4 Proposed River Bed Improvements

The chief aim of the proposed river improvement is to increase the tractive force by limiting the width of flow by the construction of new dikes in the downstream course where the width is extensive. In addition, the aim is to reduce damage caused by inundation by means of new dikes. Since the inundation damage is rather small at present, it is recommended that the schedule to construct the proposed dikes should start from the lowest dike required and follow a step-by-step increase in height to the final dike.

Based on the observation data previously outlined, a channel improvement plan for the K. Progo was proposed as described below. The plan is a rough one as it is prepared from extremely limited data available for longitudinal and cross levelling, and plane map surveying. One of the most urgent and important matters yet to be performed is to make river channel survey data as complete as is practically possible. The proposed river improvements are limited to the section between the estuary to the point about 20 km upstreams.

<u>K. Progo Channel Improvement Plan</u>

a) Design the downstream channel cross sections big enough to take care of sediment transport from the upstream area taking stabilization of the riverbed into consideration.

b) Improve the present longitudinal riverbed gradient of 1:600 to a steeper 1:550. (Refer to Fig. 15)

c) Make standard designs of the cross sections of the channel taking the desired gradient into consideration is shown in Fig. 16. As shown, the low water channel along the section between the Srandakan Bridge and the estuary, is currently about 500 m in width; as shown, the new low water channel of about 200 m in width and 2 to 3 m in depth will be excavated in the existing low water channel in all sections. The result is a composit channel with a major bed

d) The remaining portion of the existing channel, 500 m in width, will be regarded as a reclaimed land for utilization in agriculture, etc.; however, for the sake of safety, the utilization may have to be limited only during dry seasons.

e) For the proposed flood stage, it is recommended that a stage be high enough to allow the discharge of 5,000 cu.m/sec. (the biggest flood in the past) through the present cross section of the channel.

f) Since it is estimated that the volume of excavation of the riverbed from the estuary to the point about 20 km upstream will be about 15,000,000 cu.m, it is advisable to utilize excavated soil for construction of embankments and reclamation of hinterland.

g) With the progress of the proposed improvement plan, some modifications and repairs of the existing irrigation intakes and bridges may become advisable.

2.2 Countermeasures against Problems

2.2.1 Countermeasures against Blockage of Irrigation Intakes

For a steady intake for the whole year round operation, the following measures should be taken.

a) Construction of full lateral weirs instead of lateral intakes

b) To cope with unexpected riverbed evolution, and also convert the intakes to overflow and underflow types for the whole year round operation, addition of a gate in cases of intakes without a gate, and conversion to double gate for intakes with gate.

c) Thorough administration and maintenance programs.

2.2.2 Countermeasures against General Problems

Execution of the basic measures mentioned in sections 2.1.1 ∿ 2.1.4 will serve as countermeasures against inundation caused by inland water, blocking-up of the estuary, and lateral erosion of the river banks; through administration and maintenance will serve the purpose as well.

Fig. 15 Longitudinal Section of K. Progo River Improvement

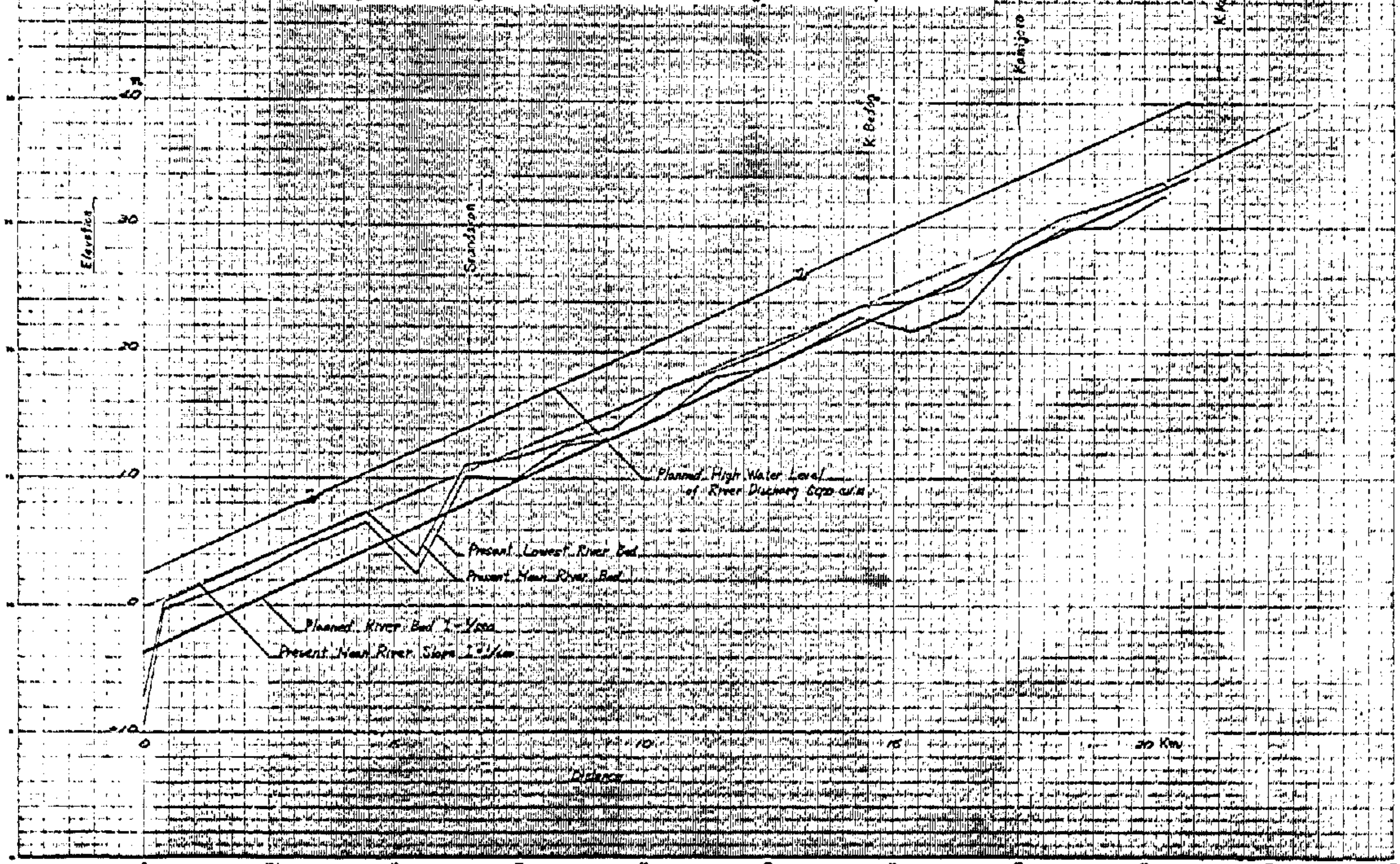
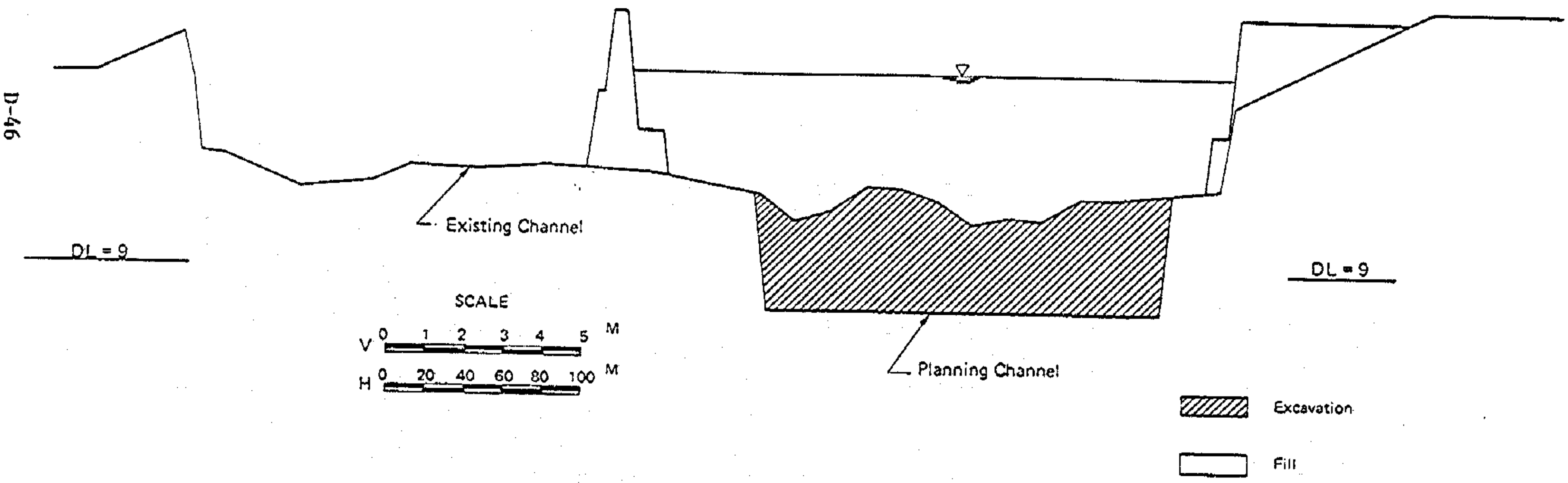


Fig. 16 Cross Section of K. Progo River Improvement at Surandakan Bridge



SUPPORTING REPORT E

RECORD OF DISASTER AND HAZARD AREAS

CONTENTS

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1. INTRODUCTION

This chapter deals with volcanic damage trends in the area covered by the plan, the extent of potential hazard areas, the warning and evacuation system, the disaster prevention facilities and other countermeasures, and discussion of the main problems for the formation of criteria for the disaster prevention master plan.

2. DAMAGE: CAUSES AND TRENDS

2.1 Records of Damage

There are many topographical signs on the footfills of G. Merapi of active volcanic activity, but few records were kept of damage caused by such activity until the last major eruption in 1969. Table 1 lists earlier eruptions and the amount of damage that was supposed to have been caused by them.

Although it is not known exactly how much damage was caused by such eruptions, it is clear that they dealt tremendous blows to local society. Some of the more recent main eruptions and the estimated number of deaths that occurred these were as follows:

Main Eruption and Deaths Caused

Date of Eruption (year, mo.)	Deaths	Location
1972. 4	3,000	Borobudur
1822.12	100	"
1832.12	32	"
1849. 9	800	"
1872. 4	200	"
1930.12	1,369	-

2.2 Causes of Damage

The following is a classification of the causes of damage after volcanic eruptions of G. Merapi:

(1) On the Slopes and Foothills of the Mountain (Tributary Area)

(a) Damage directly due to the eruptions (nuée ardente).

(b) Damage due to secondary displacement of volcanic debris (lahar/banjir).

(2) River Areas (K. Progo and K. Opak)

Damage resulting from harmful sediment deposits in the riverbeds.

2.3 Damage Trends

(1) Nuée Ardente

Nuée ardente, which results directly from eruptions, has high temperature, and high liquidity and is therefore extremely dangerous. Probably most of the reported deaths following eruptions in the past were due to it. (see Fig. 2)

(Scope of Occurrence)

According to records kept during the period 1930-1969, the area of danger from nuée ardente (Type I area) extended about 10 km from the crater (to an elevation of about 650 m) on the western slopes of the mountain, covering an area of approx. 136 km^2, or 16% of the area covered by the plan.

There are also signs that at times in the past when there was greater volcanic activity than now, nuée ardente flowed 11 km down from the crater to an elevation of about 550 m in the K. Woro area on the southeast slopes of the mountain (Type II and III areas).

(Forms of Damage)

Besides damage due to extremely hot volcanic debris and gases, volcanic debris also fills the valleys of tributaries upstream, changing the conditions of their basins. In river systems in which the basins have rapidly increased, the state of flow changes, and in some cases there is abnormal increase in sediment (produced and discharged) causing sediment damage along the rivers downstream.

In recent cases of nuée ardente the upstream sections of K. Batane and K. Blongkeng shifted into the K. Bebeng (K. Krasak) and K. Putih systems, respectively, with a sharp increase in both produced sediment and discharged sediment in both tributaries, causing enormous damage in downstream areas.

(2) Lahar

Lahar is volcanic debris deposited unstably on the upper slopes of the mountain which has begun to flow as the result of rains. In form, it resembles mud flows in (Dosekiryu in Japanese)

As it flows down, it erodes the river banks and beds, carrying a large amount of sediment with it and causing tremendous damage.

(Scope of Occurrence)

Lahar occurs on the upper slopes of the mountain between elevations of 1,000 m and 2,000 m. The damage from it occurs from the upper slopes to the hamlet and agricultural areas on the middle slopes of the mountain.

In the tributary areas on the slopes of the mountain, the hazard zone with respect to lahar and banjir comprises an area of approximately 286 km^2, or 34% of the area covered by the plan.

(Damage Trends)

Lahar occurs mainly in the rainy season, especially right after an eruption, since it results from unstable deposits of volcanic debris and rainfall. Furthermore, depending on the direction of the crater, it tends to be concentrated in a particular area and a particular river system. Since the crater of G. Merapi now faces in the direction of K. Krasak, K. Putih, and other rivers in the Type-I areas on the southwest slopes of the mountain, that is where the lahar damage has been concentrated recently.

(3) Banjir

Banjir generally means flooding, but in the tributary areas of G. Merapi, the sediment content of the flooding is very high except in a few areas such as downstream sections of K. Woro. Most of the damage tend to be caused by a form between lahar and banjir. In the area along K. Krasak, downstream from the national road where there was damage in 1976, there are a lot of round boulders, and sediment deposited to a thickness of 60-80 cm. In tributary areas, however, it is hard to make a clear distincition between banjir and lahar.

Most of the banjir damage in the areas along the main rivers such as K. Progo, K. Opak, and K. Dengkeng, (as opposed to the tributaries), is due to inundation; however, since the basins are comparatively small, the proportions of the inundation (area, duration, and depth) are small, and there is less damage in monetary terms than in the case of lahar.

(4) Sediment Deposits

Excessive sediment production and discharge make the river courses unstable by, for one thing, causing the riverbeds to rise. What results is damage to river structures, irrigation facilities, bridges, and other facilities and a heightening of the danger of a disaster occurring.

(Scope of Occurrence)

At present the rivers most seriously affected by excessive sediment discharge are K. Krasak and K. Progo.

As a result of the 1969 eruption, K. Krasak underwent a change in its basin. Subsequently there was an abnormal increase in sedment discharge which resulted in sediment damage in the downstream areas and along K. Progo. The riverbed of K. Krasak rose abruptly, by 7.0 m in the vicinity of the national road and by almost 20 m further downstream.

Because of the increase in sediment discharge from K. Krasak and other tributaries on the southwest slopes of the mountain, the riverbed of K. Progo, also rose abnormally downstream at a point 16 km from the mouth of the river, and since 1961, the intake of the Kamidjoro irrigation area (2,300 ha), which was constructed in 1924, has not been able to function at all due to the large rise in the riverbed. A new intake has been established on the tributary.