Table VI-1 Production of Main Crops in Whole Kalimantan and South Kalimantan

Crop	Area	Year 1972 (ton)	1973 (ton)	1974 (ton)	1975 (ton)	1976 (ton)	1976 (ton)
Rice	Kalimantan	649,119	719,440	783,525	766,048	837,797	100
	South Kalimantan	273,589	321,852	329,519	318,740	330,280	39.4
Dry paddy	Kalimantan	948,713	1,051,489	1,145,152	1,119,608	1,224,472	100
	South Kalimantan	399,860	470,400	481,604	465,851	482,717	39.4
Dry stalk paddy	Kalimantan	1,248,306	1,383,538	1,506,779	1,473,169	1,611,148	100
	South Kalimantan	526,132	618,947	633,690	612,962	635,154	39.4
Maize	Kalimantan	10,373	10,308	11,765	11,711	11,478	100
	South Kalimantan	2,901	1,915	2,147	1,205	1,310	11.4
Cassava	Kalimantan	267,713	283,246	326,446	278,180	273,159	100
	South Kalimantan	17,633	21,584	19,870	23,616	18,775	68.7
Sweet potato	Kalimantan	28,160	28,876	32,698	32,928	34,307	100
	South Kalimantan	9,469	7,250	6,925	9,001	9,396	27.4
Soybean	Kalimantan	1,945	2,000	2,452	2,115	2,482	100
	South Kalimantan	425	306	373	259	513	20.7
Peanut	Kalimantan	1,594	1,847	1,279	2,457	4,447	100
	South Kalimantan	1,194	1,338	753	1,835	3,920	88.1

Data Source: Statistical Year Book of Indonesia, 1976

Note: Applied conversion rate for paddy rice is as follows:

Dry stalk paddy	Dry paddy	Rice
100	76	52

Table VI-2 Gross Value of Agricultural Products in South Kalimantan and Kabupaten Banjar

	South Kalimantan 1974 (Rp.10^6)	South Kalimantan 1975 (Rp.10^6)	South Kalimantan 1976 (Rp.10^6)	South Kalimantan 1976 (%)	Kabupaten Banjar/1 1976 (Rp.10^6)	Kabupaten Banjar/1 1976 (%)
1. Food crops						
Paddy	43,457	38,682	52,506	65.8	9,410	60.2
Maize	107	118	153	0.2	11	0.1
Cassava	431	355	740	1.1	62	0.4
Sweet potato	186	204	344	0.2	16	0.1
Peanuts	208	462	1,030	1.3	224	1.4
Soybean	57	64	78	0.1	2	0.0
Mung bean	50	41	52	0.1	2	0.0
Vegetables	78	585	1,047	1.3	320	2.0
Fruits	772	1,751	7,288	9.1	1,278	8.2
Sub-total	45,346	42,262	63,238	79.2	11,325	72.4
2. Industrial crops	8,207	4,678	5,942	7.5	1,541	9.8
3. Livestock	5,332	7,138	10,646	13.3	2,783	17.8
Total	58,885	54,078	79,826	100.0	15,649	100.0

/1 : Figures are estimated from the Agricultural Year Book, Kabupaten Banjar.

Data source : Statistical Year Book in South Kalimantan Province, 1974 - 1976.

Table VI-3 Number and Area of Farms by Size of Holding in the Representative Kecamatan in the Project Area

Size of holding (ha)	No. of farm	Percentage (%)	Area (ha)	Percentage (%)
0.1	82	3.3	5.1	0.3
0.1 - 0.5	742	29.5	226.0	9.3
0.51 - 1.00	891	35.4	647.7	26.5
1.01 - 1.50	369	14.7	457.3	18.7
1.51 - 2.00	210	8.4	372.6	15.3
2.01 - 3.00	139	5.6	341.9	14.0
3.01 - 4.00	46	1.8	159.8	6.5
4.01 - 5.00	24	0.9	106.2	4.3
More than 5.01	15	0.4	125.5	5.1
Total	2,518	100.0	2,442.1	100.0
Average			0.97	

Note : This table is prepared on the basis of data obtained from three Kecamatan, Banjarbaru (sub-area A), Martapura (sub-area B) and Kertak Hanyar (sub-areas C and D) in the project area.

Data source : Kecamatan Banjabaru, Martapura and Kertak Hanyar offices, 1978.

Table VI-4 Land Tenure by Number of Farms in the Representative Kecamatan in the Project Area

Kecamatan	Land owner (%)	Land owner cum tenant (%)	Tenant (%)
Karang Intan (sub-area A)	80	10	10
Martapura (sub-area B)	85	5	10
Kertak Hanyar (sub-area C)	65	5	30
Gambut (sub-area D)	60	5	35
Average	72.5	6.3	21.2

Data source : This table is prepared based on the data collected from the selected farmers and Desa.

Table VI-5 Present Land Use in the Survey Area

Description	Area (ha)	Proportion (%)
1. Agricultural Land		
Paddy	40,500	43.7
Plantation /1	4,800	5.2
2. Non-agricultural Land		
Alang-alang area /2	9,800	10.5
Shrub	4,980	5.3
Bush	6,400	6.9
Swamp forest	18,200	19.6
Swamp grass area	700	0.8
Others /3	7,400	8.0
3. Total	92,780	100.0

/1: This includes rubber and coconut plantations.

/2: The alang-alang area includes the small-scale clove plantations being under development and upland crop field for casava, maize, etc. mainly for home consumption.

/3: Others include non-agricultural land such as village compounds, roads, canals rivers, dikes, etc.

Table VI-6 Present Land Use in the Survey Area and the Project Area

Land category	Survey area	Project area					
		Sub-area A	Sub-area B	Sub-area C	Sub-area D	Sub-area E	Total (A-E)
	(ha)	(ha)	(ha)	(ha)	(ha)	(ha)	(ha)
Paddy field	40,500	1,200	4,100	3,900	12,200	8,560	29,960
Plantation	4,800 /1	760	200	100	-	-	1,060
Alang-alang area	9,800 /2	-	-	-	-	-	-
Shrub	4,980	90	4,000	-	-	-	4,090
Bush	6,400	-	-	-	-	-	-
Swamp forest	18,200	-	-	-	-	-	-
Swamp grass	700	-	-	-	-	-	-
Others	7,400 /3	150	400	400	1,350	950	3,250
Total	92,780	2,200	8,700	4,400	13,550	9,510	38,360

/1 : This includes rubber and coconut plantations covering 2,500 ha and 2,300 ha, respectively.

/2 : The alang-alang area includes the small scale clove plantations being under development and upland crop field for cassava, maize, etc.

/3 : Others are primarily defined as infrastructural land including village yard, roads, channels, rivers, etc.

Note : Sub-area F is excluded from the project area because of unsuitable land for economical irrigation development.

Fig. VI-1 Present Land Use Map

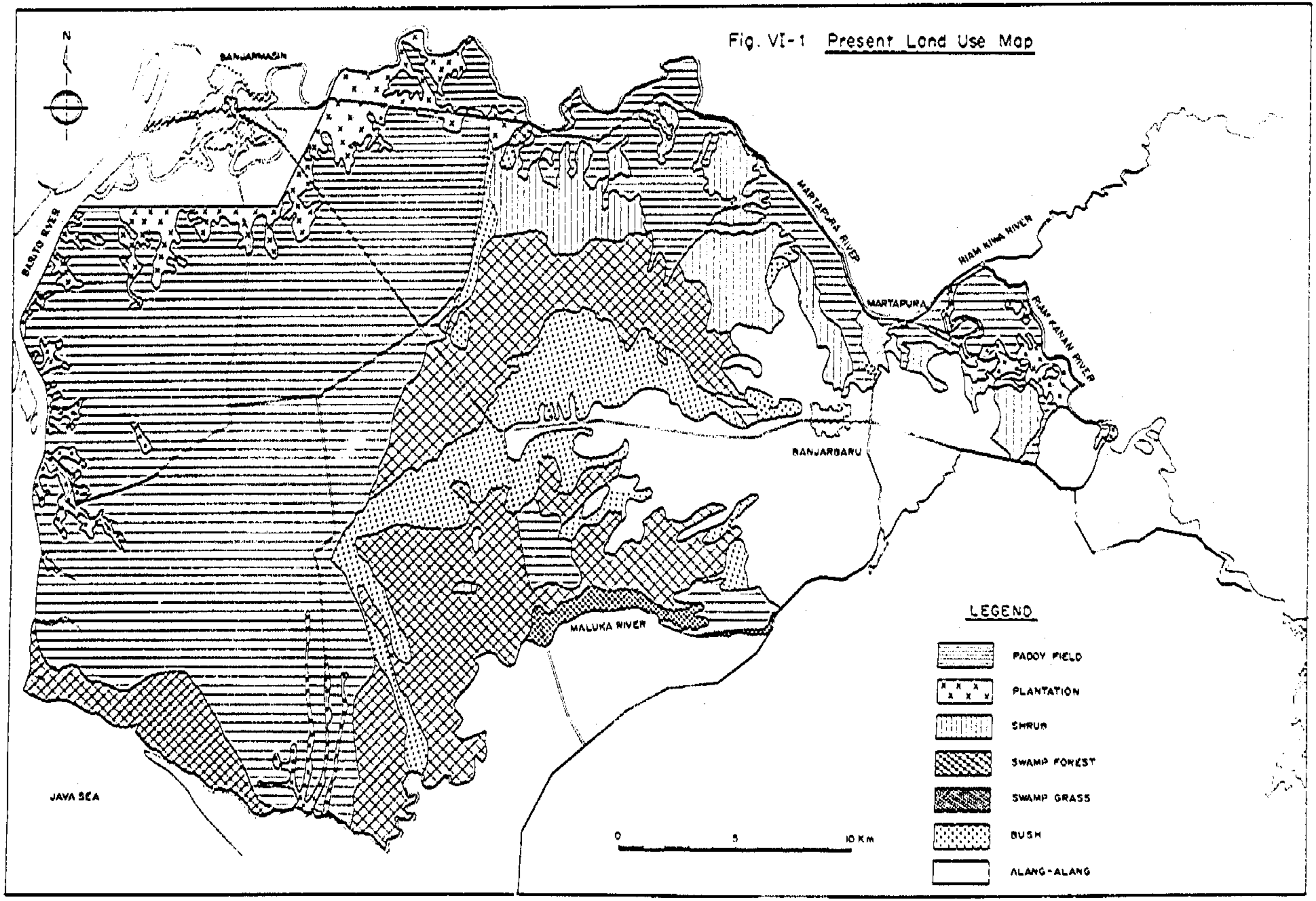
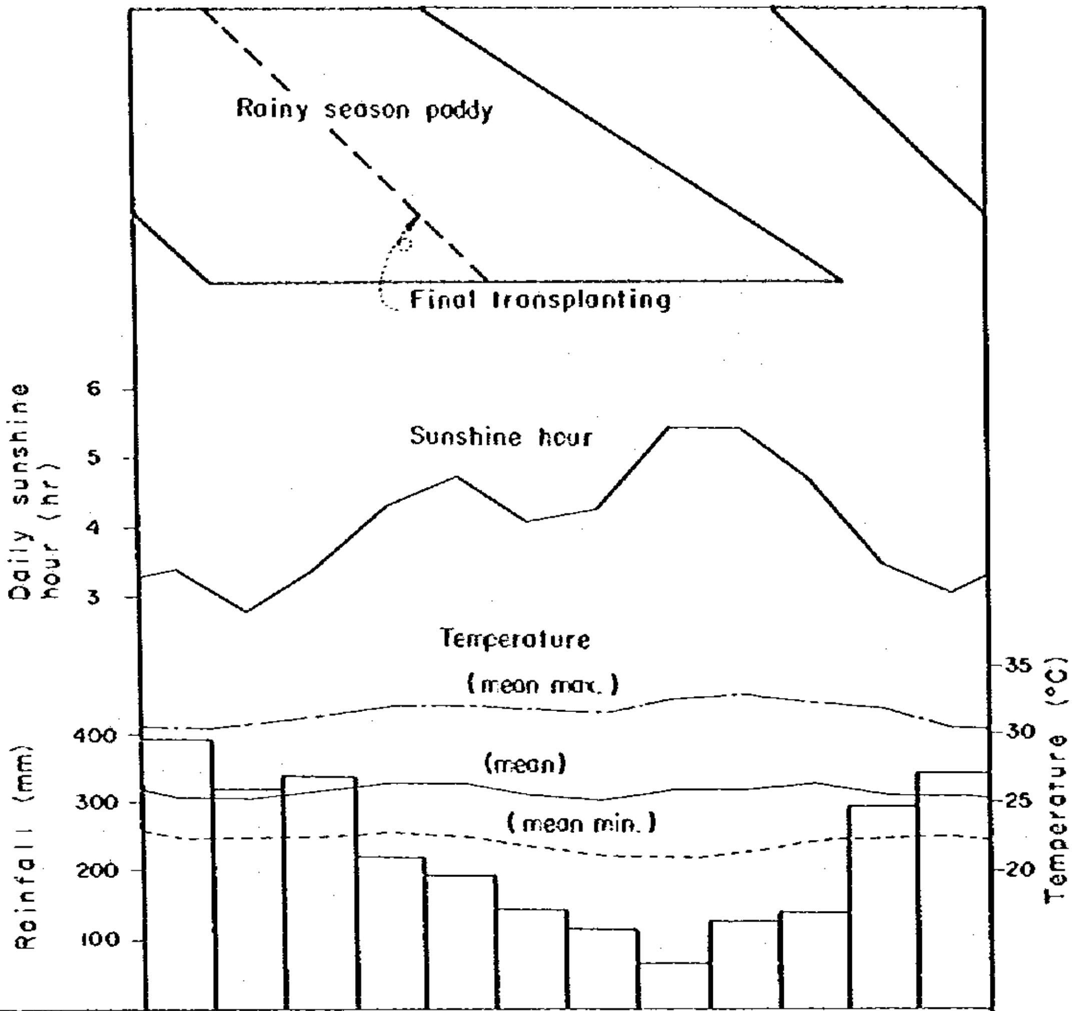


Fig. VI-2 Present Cropping Pattern



Month	J	F	M	A	M	J	J	A	S	O	N	D
Rainfall (mm)	396	318	343	220	193	142	118	66	125	137	290	343
Mean temp (°C)	25.7	25.6	25.9	26.3	26.4	25.7	25.3	25.9	26.0	26.5	25.9	25.8
Mean max. temp. (°C)	30.7	30.7	31.5	32.2	32.3	31.9	31.7	32.6	33.0	32.6	31.9	30.6
Mean min. temp. (°C)	22.7	22.6	22.7	22.9	22.7	21.9	21.2	21.2	21.4	22.3	22.7	22.8
Relative humidity (%)	87	89	88	87	85	85	83	79	82	82	85	88
No. of rain days	19	16	16	12	11	9	9	8	8	9	15	18
Sunshine hour (hr/day)	3.4	2.8	3.4	4.4	4.8	4.1	4.3	5.5	5.5	4.7	3.5	3.2

Note : 1) In the sub-areas A and B, the dry season paddy is grown in small area.

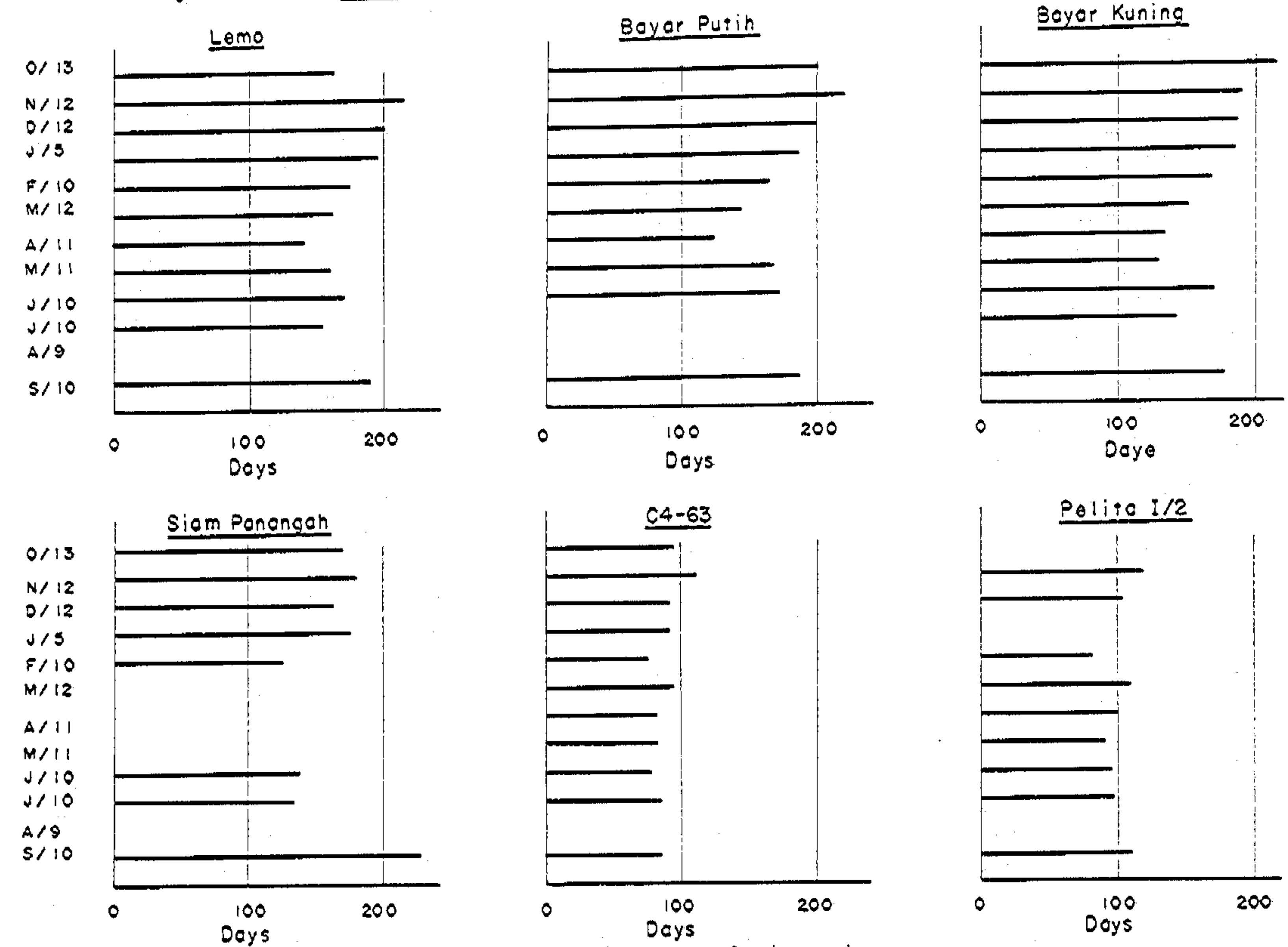
2) Rainfall and temperature shown in this table are the average values for 17 years at Banjarbaru station.

Table VI-7 List of Main Varieties grown in the Project Area

Variety of paddy	Growing duration (days)	Plant height (cm)
Lemo	255	155
Bayar putih	287	195
Bayar kuning	265	175
Bayar raden rata	255	150
Bayar raden jawa	265	160
Siam ganal	270	165
Siam panangah	270	160
Siam serai	270	150
Kencana	235	155
Randah padang	230	150
Pandak	240	150
Karang dukuh	245	150
Simon	235	165
Pelita I/1	135 - 145	130
C_4 - 63	125-130	125

Data source : Agricultural Extension Services, Kabupaten Banjar

Fig. VI-3 Growing Period from Seeding to Flowering by Date of Seeding



Data source : Central Research Institute for Agriculture, Banjarmasin.

Table VI-8 Farm Input Requirements per Hectare for Present Paddy Cultivation

Item	Requirements
1. Labor	
Preparation of seedlings	8 man-day
Field preparation	35 man-day
Transplanting	35 man-day
Weeding	20 man-day
Harvesting	35 man-day
Threshing, drying & transportation	15 man-day
Total	148 man-day
2. Seeds, Fertilizers & Chemicals	
Seeds	10 kg
Fertilizers	-
Insecticides	-
Fungicide	-
Rodenticide	-
3. Miscellaneous	
Bags, mats, tools, etc.	about 10% of total production cost.

Data source : Monografi Daerah 1976.
Agricultural Extension Services, South Kalimantan.

Monografi Daerah 1973.
Agricultural Extension Services, Kabupaten Banjar.

Table VI-9 Planted Area, Damaged Area, Harvested Area and Production of Lowland Paddy in Kabupaten Banjar

Description		Year					
		1973	1974	1975	1976	1977	Average
Planted area	(ha)	49,000	50,100	50,300	51,700	50,200	50,300
Damaged area	(ha)	3,400	800	300	3,800	700	1,800
Harvested area	(ha)	45,600	49,300	50,000	47,900	49,500	48,500
Production	(ton)	91,300	126,500	121,500	108,300	108,800	112,300
Unit Yield	(ton/ha)	2.0	2.6	-	-	-	-
Dry stalk paddy		2.0	2.6	2.4	2.3	2.2	2.3
Dry grain paddy		1.5	2.0	1.8	1.75	1.7	1.75

Data source : Monografi Daerah 1976. Department of Agricultural Extension Services, South Kalimantan 1977. Annual Report 1973 and 1977. Agricultural Extension Services, Kabupaten Banjar.

Table VI-10 Losses of Rice During Operations through Harvest to the Market in Indonesia

	Loss	Percentage (%)
1.	Field loss in harvesting by "Ani-ani" system	8.0
2.	Field loss in transportation	2.0
3.	Loss in drying	2.0
4.	Loss from drying yard to storage	1.5
5.	Loss in storage by farmer	4.0
6.	Loss in storage	1.0
7.	Loss in rice mill	4.5
8.	Handling loss in bag by hook, etc.	1.0
9.	Loss in transportation to the market	1.0
	Total	25.0

Data source : Data from Badan Ursah Logistic, 1977.

Table VI-12 Present Annual Production of Paddy in the Project Area

Sub-area	Cropped area (ha)	Unit yield (ton/ha)	Production (ton)
A	1,200	1.75	2,100
B	4,100	1.75	7,175
C	3,900	1.75	6,825
D	12,200	1.75	21,350
Sub-total	21,400		37,450
E	8,560	1.75	14,980
F	0	-	0
Total	29,960		52,430

Table VI-11 Result of Yield Check Survey in the Project Area

	Location	Variety	Number of hill (per m2)	Average number of panicle per hill	Average number of grain per panicle	Percentage/1 of ripened grain (%)	Weight of 1000 grain (gram)	Yield (t/ha)	Date of seeding	Date of harvesting
1.	Kec. Karang Intan Jingah Habang	Pandak	13.0	10.0	143.0	50.9	20.5	1.9	Dec.	23rd Aug.
2.	Kec. Kertak Hanyar Manarap	Siam halus kuning	10.0	13.8	165.3	82.6	16.5	3.1	Oct.	25th Aug.
3.	Kec. Kertak Hanyar Tatah Pemangkih laut	Tilang	9.6	12.0	98.4	86.6	21.8	2.1	Dec.	25th Aug.
4.	Kec. Kertak Hanyar Tatah Pemangkih Darat	Bayar pahit	9.5	16.3	88.8	89.6	21.4	2.6	Jan.	25th Aug.
5.	Kec. Aluh-aluh Bunipah	Bayar kuning	10.1	9.9	124.9	81.1	25.4	2.6	Dec.	25th Aug.
6.	Kec. Aluh-aluh	Tilang	10.3	12.6	88.6	68.6	21.5	1.7	Dec.	28th Aug.
	Average		10.4	12.4	118.2	76.6	21.2	2.33	-	-

/1 : Selection of ripened grain was made using water with the specific gravity of 1.0. and the grains which sunk in the water were defined as ripened grain.

Fig. VI-4

Marketing Flow of Farm Inputs

Location	Organization	Price: Seed (Rp/Kg) /1	Fertilizers (Rp/Kg): Urea	Fertilizers (Rp/Kg): T.S.P	Fertilizers (Rp/Kg): Kcl	Chemicals (Rp./lit): Diazinon	Chemicals (Rp./lit): Sumithion	Chemicals (Rp./lit): Sevin
Jakarta	Center of P.T. Tawison		70	70	70	900	900	900
Province (Banjarmasin)	Branch of P.T. Tawison		70	70	70	900	900	900
Kabupaten/ Kecamatan	BUUD/KUD	150	70	70	70	900	900	900
Desa	BIMAS farmers, INMAS farmers, other farmers	150	70	70	70	900	900	900

/1 : Farmers can get seeds from Balai Benih Seed center in Kecamatan Sungai Tabuk, Kabupaten Banjar.

Data Source : P.T. Tawison in Banjarmasin.

Table VI-13 Financial Prices of Farm Inputs

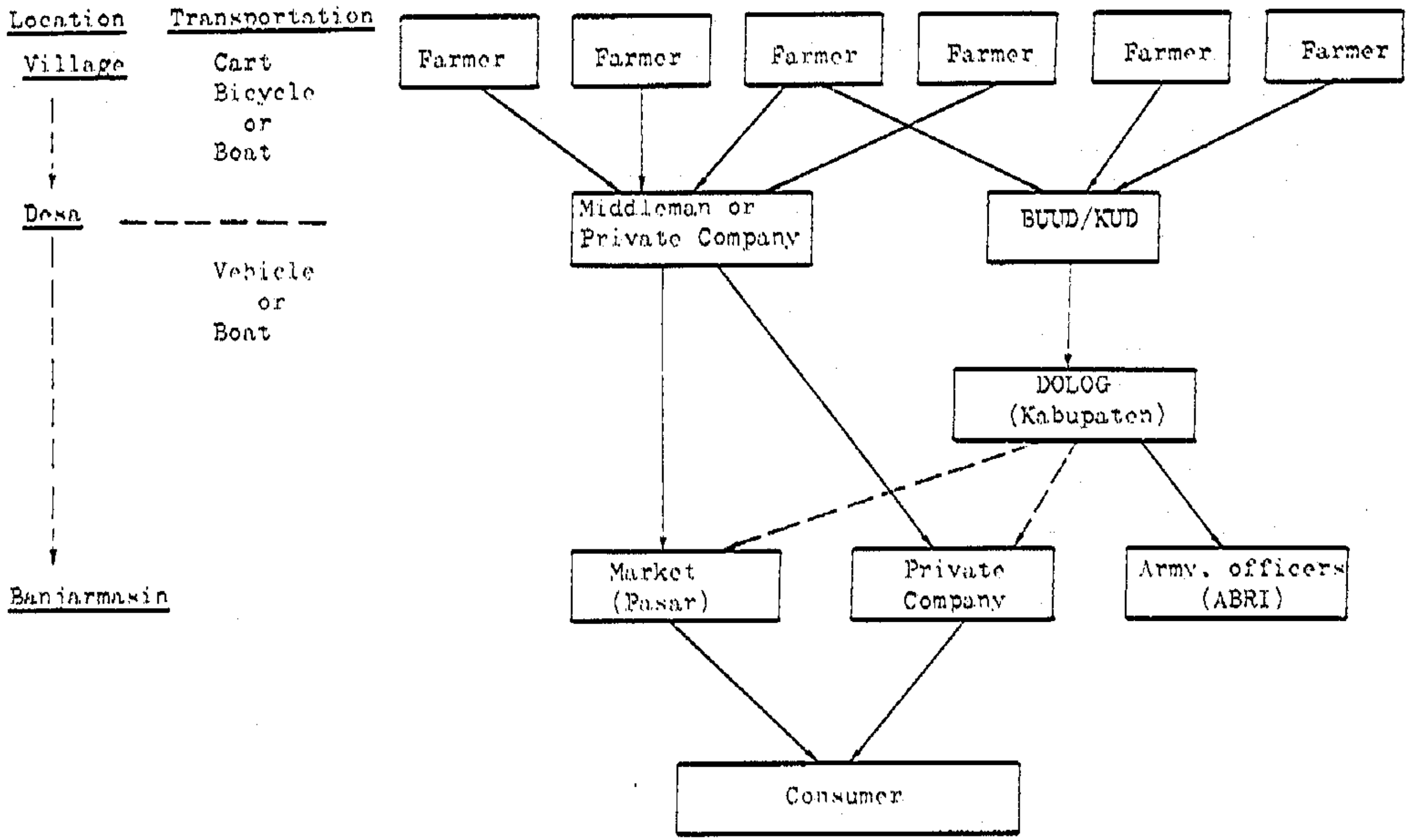
Item	Price
Seed (dry paddy)	150 (Rp/kg)
Fertilizers	
Urea	70 (Rp/kg)
T.S.P	70 (Rp/kg)
Kcl	125 (Rp/kg)
Agro-chemicals	
Insecticide	900 (Rp/lit)
Fungicide	900 (Rp/lit)
Rodenticide	2,300 (Rp/kg)
Labour	
Preparation of Seedling	500 (Rp/man-day)
Field Preparation	350 (Rp/man-day)
Transplanting/1	500 (Rp/man-day)
Weeding	300 (Rp/man-day)
Fertilizing & Spraying	200 (Rp/man-day)
Water management	200 (Rp/man-day)
Harvesting	500 (Rp/man-day)
Processing, Drying & Transportation	300 (Rp/man-day)

Date Source: 1) Agricultural Extension Services in Kabupaten Banjar, 1978

2) Farm Economy Survey, 1978

/1 : Labour charge for transplanting includes meal.

Fig. VI-5 Market Flow of Rice



Depends on price situation.

Table VI-14 Financial Prices of Farm Products

(Average of Semi-Annual)

Crop	Market	1975 Jan.-June (Rp/kg)	1975 July-Dec. (Rp/kg)	1976 Jan.-June (Rp/kg)	1976 July-Dec. (Rp/kg)	1977 Jan.-June (Rp/kg)	1977 July-Dec. (Rp/kg)	Average of 1975-1977 (Rp/kg)
Rice	Banjarmasin	99.9	100.2	156.4	125.0	157.0	131.5	
	Banjarbaru	99.1	101.4	152.5	125.3	128.8	124.8	
	Average	99.5	100.8	154.5	125.2	142.9	128.2	125
Dry paddy	Banjarmasin	60.0	59.2	108.7	79.8	*	*	
	Banjarbaru	59.7	57.8	98.9	82.3	98.3	98.3	
	Average	59.9	58.5	103.8	81.1	98.3	98.3	83
Maize	Banjarmasin	78.9	82.2	123.9	107.3	120.2	92.5	
	Banjarbaru	80.5	82.5	128.9	123.3	101.3	75.4	
	Average	79.7	82.4	126.4	115.3	110.8	84.0	100
Soybean	Banjarmasin	246.0	273.5	237.5	180.5	270.5	192.5	
	Banjarbaru	173.3	238.2	252.7	247.5	291.7	329.2	
	Average	209.7	255.9	245.1	214.0	281.1	260.9	245
Peanuts	Banjarmasin	271.5	269.7	323.0	287.7	315.0	355.3	
	Banjarbaru	252.2	275.5	348.3	335.8	325.0	350.0	
	Average	261.9	272.6	335.7	311.8	320.0	352.7	309
Cassave	Banjarmasin	23.0	22.5	36.0	39.2	49.7	41.7	
	Banjarbaru	16.5	28.3	41.2	41.7	31.7	40.0	
	Average	19.8	25.4	38.6	40.5	40.7	40.9	34
Green bean	Banjarmasin	229.8	232.3	268.8	275.5	286.8	241.3	
	Banjarbaru	221.3	233.2	298.3	293.3	257.5	273.3	
	Average	225.6	232.8	283.6	284.4	272.2	257.3	259

* : No data are available.

Data source: Agricultural Extension Services in Kabupaten Banjar

Table VI-15 Capacity and Number of Rice Mills in Kecamatan concerned with the Project

Kecamatan	Item	Capacity and Number (capacity: ton/hr) 1.0	1.0-2.0	2.0-3.0	3.0-4.0	more than 4.0	Total
Karang Intan	Number	-	2	-	-	-	2
	Capacity	-	2.8	-	-	-	2.8
Astambul	Number	8	9	2	-	-	19
	Capacity	4.5	9.1	4.2	-	-	17.8
Martapura	Number	2	12	2	2	1	19
	Capacity	1.4	14.9	4.6	8	8	36.9
Banjarbaru	Number	-	1	-	-	-	1
	Capacity	-	1.0	-	-	-	1.0
Gambut	Number	-	15	2	-	-	17
	Capacity	-	23.7	5.0	-	-	28.7
Sungai Tabuk	Number	-	4	2	1	-	7
	Capacity	-	6.3	4.5	3.5	-	14.3
Kertak Hanyar	Number	2	14	4	1	-	21
	Capacity	1.4	28.1	9.6	3.5	-	42.6
Aluh Aluh	Number	2	1	-	-	-	3
	Capacity	0.7	1.1	-	-	-	1.8
Kurau	Number	-	-	-	-	10	10
	Capacity	-	-	-	-	4	4
Total	Number	14	58	12	4	11	99
	Capacity	8.0	87.0	27.9	15.0	12	149.9

Note: 1) The capacity shown in this table is the input capacity.

2) Number of rice mills in this table includes the mills owned by private companies. Kecamatan concerned are as follows:

Sub-area A: Kec. Karang Intan, Astambul and Martapura
Sub-area B: Kec. Martapura, Banjarbaru, Gambut and Sungai Tabuk
Sub-area C: Kec. Gambut, Sungai Tabuk and Kertak Hanyar
Sub-area D: Kec. Gambut, Kertak Hanyar and Aluh Aluh
Sub-area E: Kec. Aluh Aluh, Gambut, Banjarbaru and Kurau

Data source: Monografi Daerah, Agricultural Extension Service, Kabupaten Banjar, 1976.
Agricultural Extension Service in Kabupaten Tanah Laut.

Table VI-16 Present Annual Budget of Typical Owner Farmer

Farm size : 1 ha

Family size : 6 persons

Description	Amount (Rp)
1. **Gross Income**	
Farm income	
Paddy	145,250
Upland crops	2,000
Livestock income (poultry)	16,500
Miscellaneous	64,220
Total	227,970
2. **Out-go**	
Farming expenses	
Seeds	1,500
Miscellaneous	150
Livestock expenses	1,650
IPEDA tax, etc.	2,800
Family living expenses	220,320
Total	226,420
3. **Balance or Capacity to Pay**	Rp. 1,550 or US$ 2.5

Table VI-17 Present Staffing and Equipment for Rural Extension Centers (R.E.C) in Kecamatan related to the Project

R.E.C	Kecamatan	Staffing			Equipment			
		P.P.S	P.P.M	P.P.L	Jeep	motor cycle	Ha per P.P.L	No. of farmer per P.P.L
Astambul	Karang Intan			2			7,030	2,130
	Astambul			5			1,040	630
	Martapura			1			13,950	9,390
	Sub-total		2	8		6	7,340	4,050
Gambut	Banjarbaru			2			2,250	7,330
	Gambut			6			990	530
	Sungai Tabuk			3			2,750	1,570
	Kertak Hanyar			3			2,270	1,170
	Kota Madya/1 Banjarmasin			1			-	-
	Sub-total		2	15		5	2,060	2,650
Aluh Aluh	Aluh Aluh		2	2		3	2,390	860
Kurau	Kurau		2	9		2	1,440	450
	Total		8	34		16	3,790	2,670

/1 : Kota Madya Banjarmasin is outside the project area.

Note: Kecamatan related to the Project are as follows:

Sub-area A : Kec. Karang Intan, Astambul and Martapura

Sub-area B : Kec. Martapura, Banjarbaru, Gambut and Sungai Tabuk

Sub-area C : Kec. Gambut, Sungai Tabuk and Kertak Hanyar

Sub-area D : Kec. Gambut, Kertak Hanyar and Aluh Aluh

Sub-area E : Kec. Aluh Aluh, Gambut, Banjarbaru, and Kurau

Data source: 1) Department of Agriculture in South Kalimantan Province

2) Agricultural Extension Service Section in Kabupaten Banjar

3) Agricultural Extension Service Section in Tanah Laut.

Fig. VI-6 Organization for Aluh Aluh Rural Extension Center

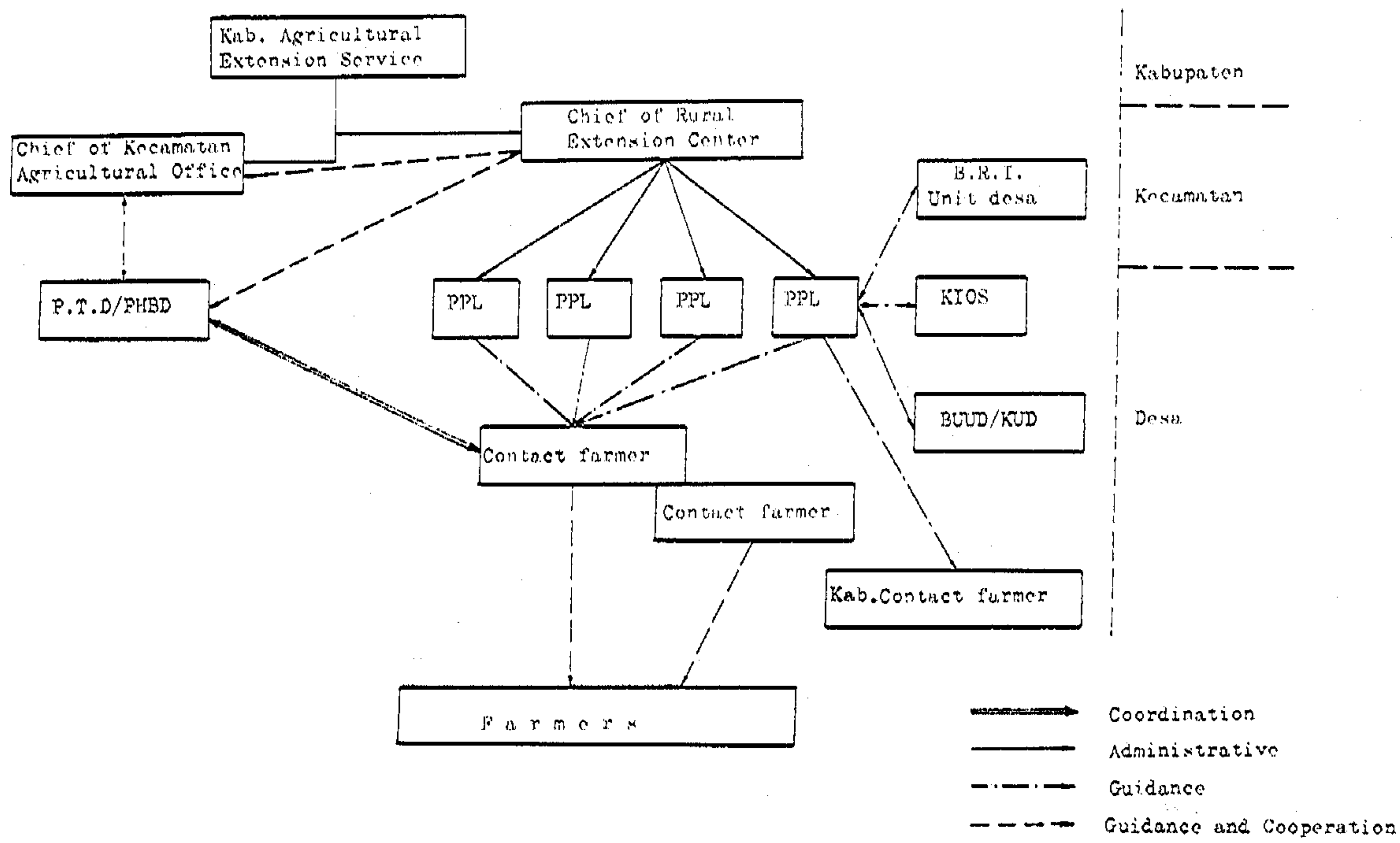


Fig. VI-7 Organization for Agricultural Extension Service in South Kalimantan Province

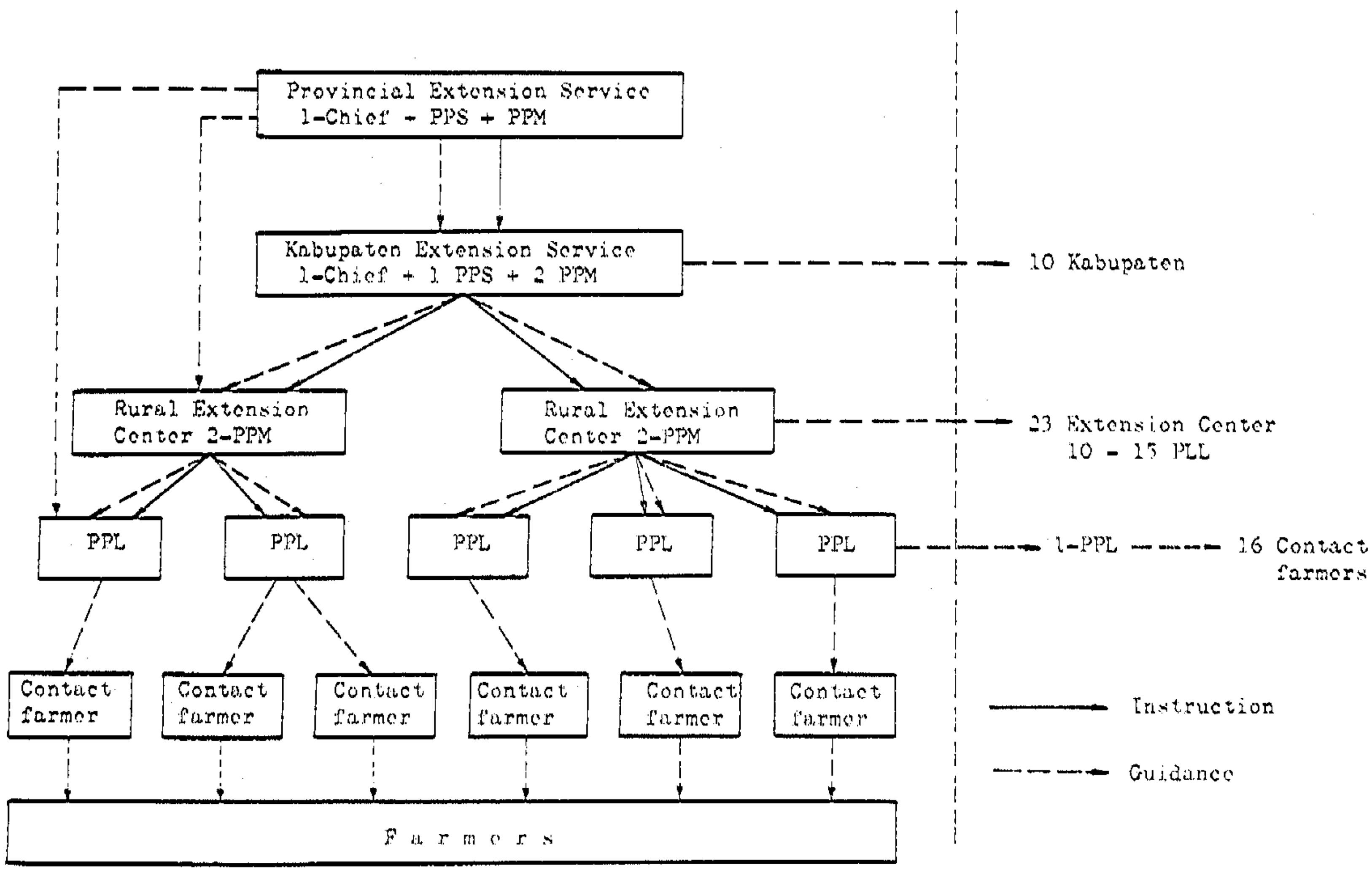
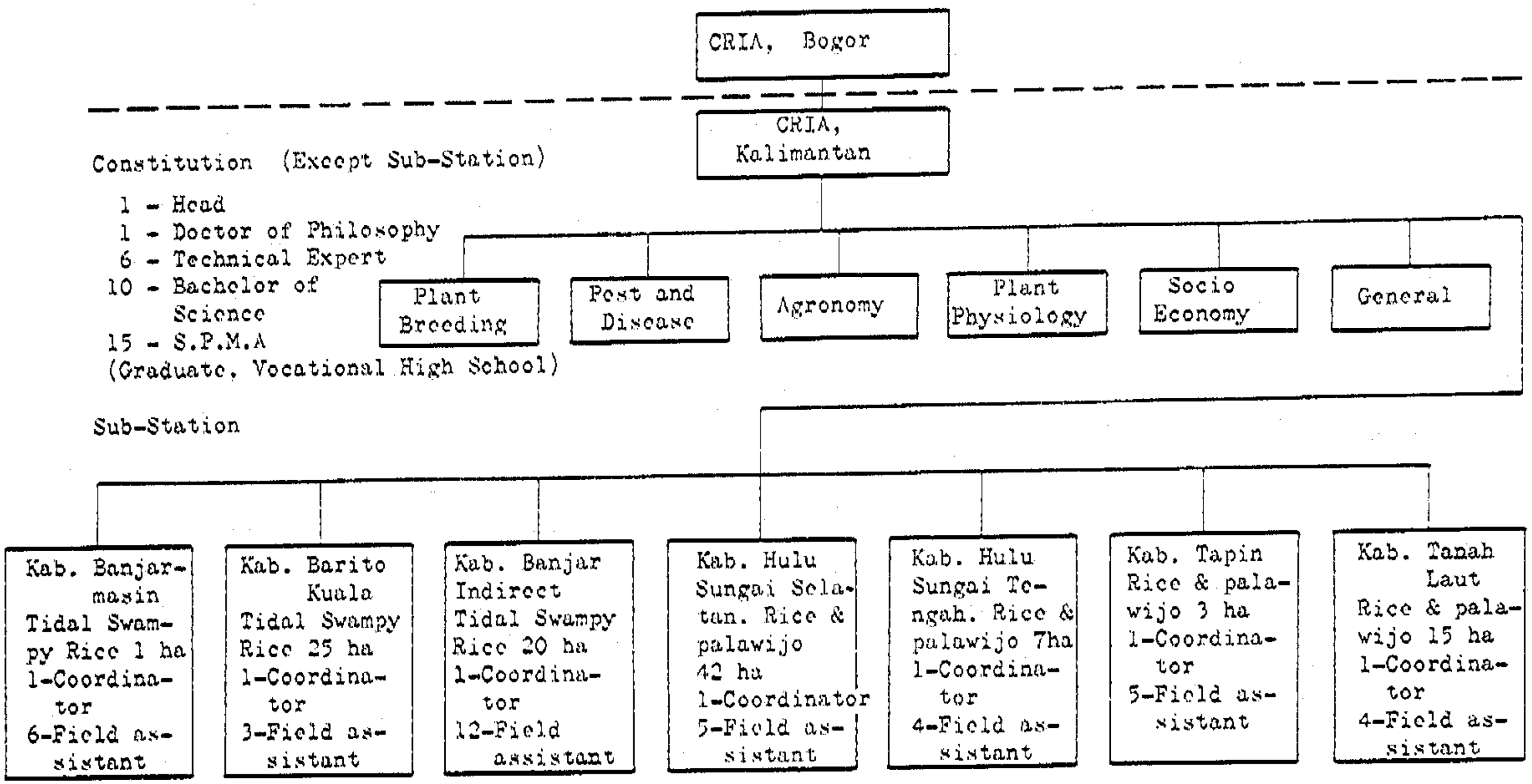


Fig. VI-8 Organization of Central Research Institute for Agriculture, Kalimantan



Note: Objective crops: Rice, Maize, Sorgham, pulses (Ground nuts, Soybean, Mang bean, Cowpea)
Tuber crops (Cassava, Sweet potato, etc.)

Fig. VI-9 Organization of Seed Multiplication and Distribution

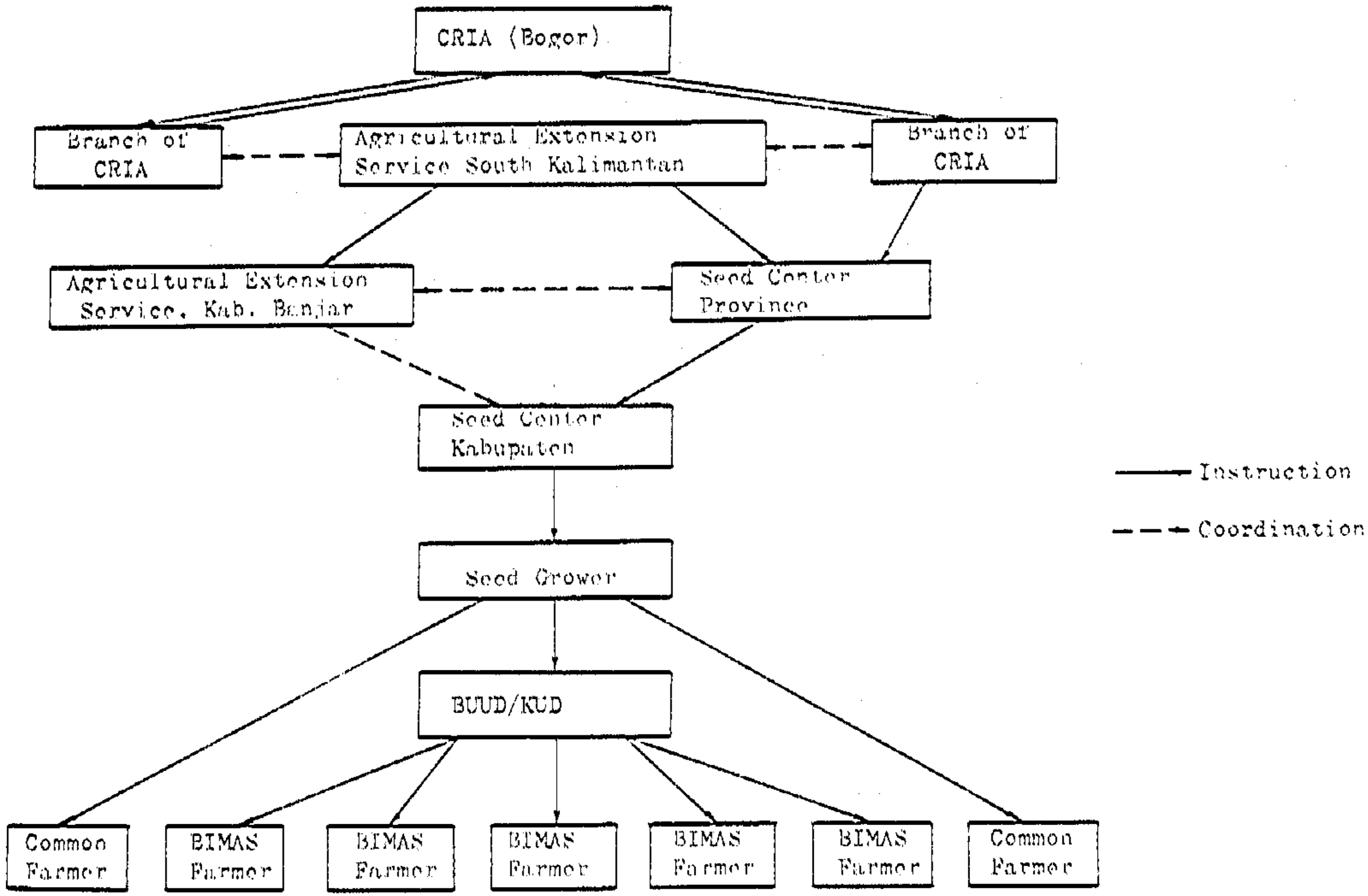


Table VI-18 Area under BIMAS and INMAS Programs in the Project Area

Cropping season	BIMAS Target (ha)	BIMAS Actual progress (ha)	INMAS Target (ha)	INMAS Actual progress (ha)	Total Target (ha)	Total Actual progress (ha)
1974/75	1,250	377	1,250	250	2,500	627
1975	*	*	250	120	250	120
1975/76	1,250	159	1,250	500	2,500	659
1976	200	275	125	1,000	325	1,275
1976/77	1,675	1,327	2,300	2,625	3,975	3,902
1977	250	213	500	1,400	750	1,613
1977/78	1,320	842	5,300	865	6,620	1,707
1978**	250	126	875	35	1,125	161
Average/1	1,374	676	2,525	1,060	3,899	1,724
Average/2	233	205	438	640	613	792

* : No data are available.

** : The progress as of August in 1978.

/1 : Average of rainy season paddy area under the Programs.

/2 : Average of dry season paddy area under the Programs.

Note: The figures shown in this table are estimated based on the data from the agricultural office in Kabupaten Banjar.

Table VI-19 Credit and Repayment of BIMAS Program in the Project Area

Cropping season	Credit (Rp. 1,000)	Repayment (Rp. 1,000)	Repayment percentage (%)	Outstanding (Rp. 1,000)
1974/75	6,508	626	9.6	5,883
1975	*	*	*	*
1975/76	4,064	308	7.6	3,756
1976	3,458	1,098	31.6	2,366
1976/77	22,885	2,266	9.9	20,620
1977	2,839	137	4.8	2,702
1977/78	18,446	28	0.2	18,418
1978	2,538	0	0	2,538

* : No data are available.

Note: The figures shown in this table are estimated based on the data from Agricultural Extension Services, South Kalimantan Province.

Table VI-20 Number of Cooperatives in the Project Area

Sub-area	Desa	BUUD/KUD	KIOS	B.R.I/1 (Unit desa)	R.M.U/2
A	7	1	0	0	0
B	9	0	0	0	0
C	4	1	1	1	0
D	13	5	5	5	1
E	9	1	0	0	0
Total	42	8	6	6	1

/1 : Bank Rakyat Indonesia

/2 : Rice mill unit under BUUD/KUD

Data source: Agricultural office in Kabupaten Banjar.

Table VI-21 Present and Proposed Land Use in Each Sub-area

Land Category	Present Land Use (ha)	Proposed Land Use: Irrigated paddy field, Wet season (ha)	Proposed Land Use: Irrigated paddy field, Dry season (ha)	Proposed Land Use: Land to be occupied by facilities and others (ha)
Sub-area A				
Paddy field	1,200	810	945 (340)	50
Plantation	760	650	485	110
Shrub	90	70	50	20
Other lands	150	-	-	150
Sub-total	2,200	1,530	1,480	330
Sub-area B				
Paddy field	4,100	3,660	2,910 (170)	270
Plantation	200	170	130	30
Shrub	4,000	3,400	2,540	600
Other lands	400	-	-	400
Sub-total	8,700	7,230	5,580	1,300
Sub-area C				
Paddy field	3,900	3,650	2,730	250
Plantation	100	90	70	10
Shrub	-	-	-	-
Other lands	400	-	-	400
Sub-total	4,400	3,740	2,800	660
Sub-area D				
Paddy field	12,200	11,520	8,620	680
Other lands	1,350	-	-	1,350
Sub-total	13,550	11,520	8,620	2,030
Sub-area E				
Paddy field	8,560	8,080	6,050	480
Other lands	950	-	-	950
Sub-total	9,510	8,080	6,050	1,430
Total (A - E)	38,360	32,100	24,530 /1	5,750

/1: This includes 510 ha of land to be used for single cropping in the dry season only.

Table VI-22 Economic Comparison between Rubber Plantation and New Paddy Field

Description		Project year 1st	2nd	3rd	4th	5th	6th	7th	8th	9th	10th	11th	----	50th	Present worth/1 (Rp.1,000)
1. Rubber plantation without renewal of productivity															
Yield	(ton/ha)	0.8	0.8	0.8	0.8	0.8	0.7	-	-	-	-	-	----	0.7	
Gross production value	(Rp.x1.000)	320	320	320	320	320	280	-	-	-	-	-	----	280	
Production cost	(Rp.x1.000)	100	100	100	100	100	90	-	-	-	-	-	----	90	783
Net production value	(Rp.x1.000)	220	220	220	220	220	190	-	-	-	-	-	----	190	1.293
2. Rubber plantation with renewal of productivity															
Yield	(ton/ha)	-	-	-	-	-	1.2	1.2	1.2	1.2	1.2	1.6	----	1.6	
Gross production value	(Rp.x1.000)	-	-	-	-	-	480	480	480	480	480	640	----	640	
Cost for re-newing	(Rp.x1.000)	410													
O & M cost	(Rp.x1.000)	-	297	198	122	122									
Production cost	(Rp.x1.000)	-	-	-	-	-	110	110	110	110	110	120	----	120	
Total cost	(Rp.x1.000)	410	297	198	122	122	110	110	110	110	110	120	----	120	1.541
Net production value	(Rp.x1.000)	-	-	-	-	-	370	370	370	370	370	520	----	520	2.137
3. Paddy field shifted from the existing rubber plantations															
Yield	(ton/ha)	-	5.4	6.1	6.5	6.9	7.8	8.0	8.5	----	----	----	----	8.5	
Gross production value	(Rp.x1.000)	-	972	1,098	1,170	1,242	1,404	1,440	1,530	----	----	----	----	1,530	
Cost for reclamation	(Rp.x1.000)	820													732
Production cost	(Rp.x1.000)	-	179	202	216	229	259	265	282	----	----	----	----	282	1.728
Net production value	(Rp.x1.000)	-	793	896	954	1,013	1,145	1,175	1,248	----	----	----	----	1,248	8.231

/1 : This is estimated using the present worth factor i = 0.12

Note: Prices of crude-rubber and paddy used in this table are Rp. 400 and Rp. 180 per kg. respectively.

Fig. VI - 10 Proposed Cropping Patterns

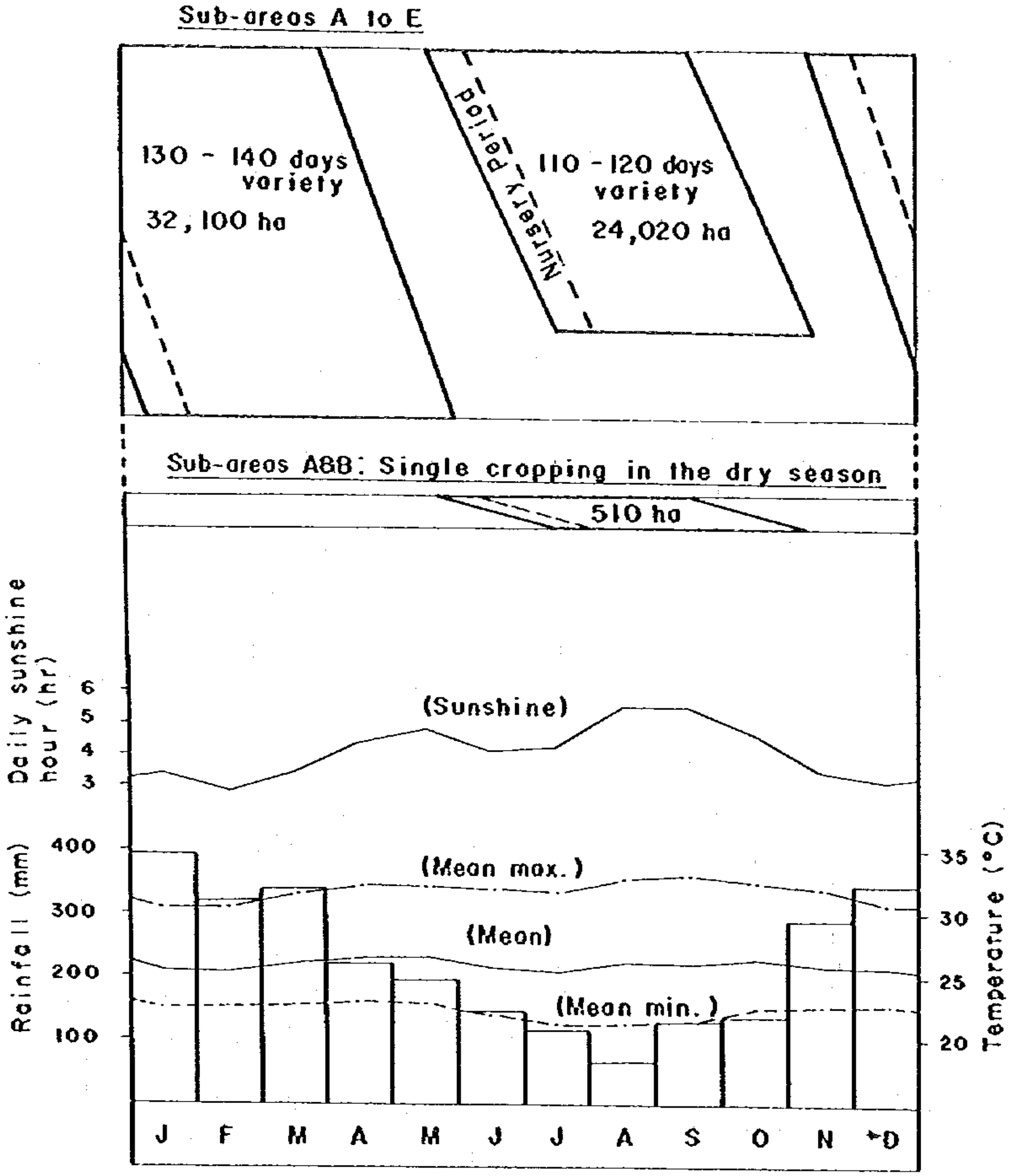


Table VI-23 Proposed Cropping Area in Each Sub-area

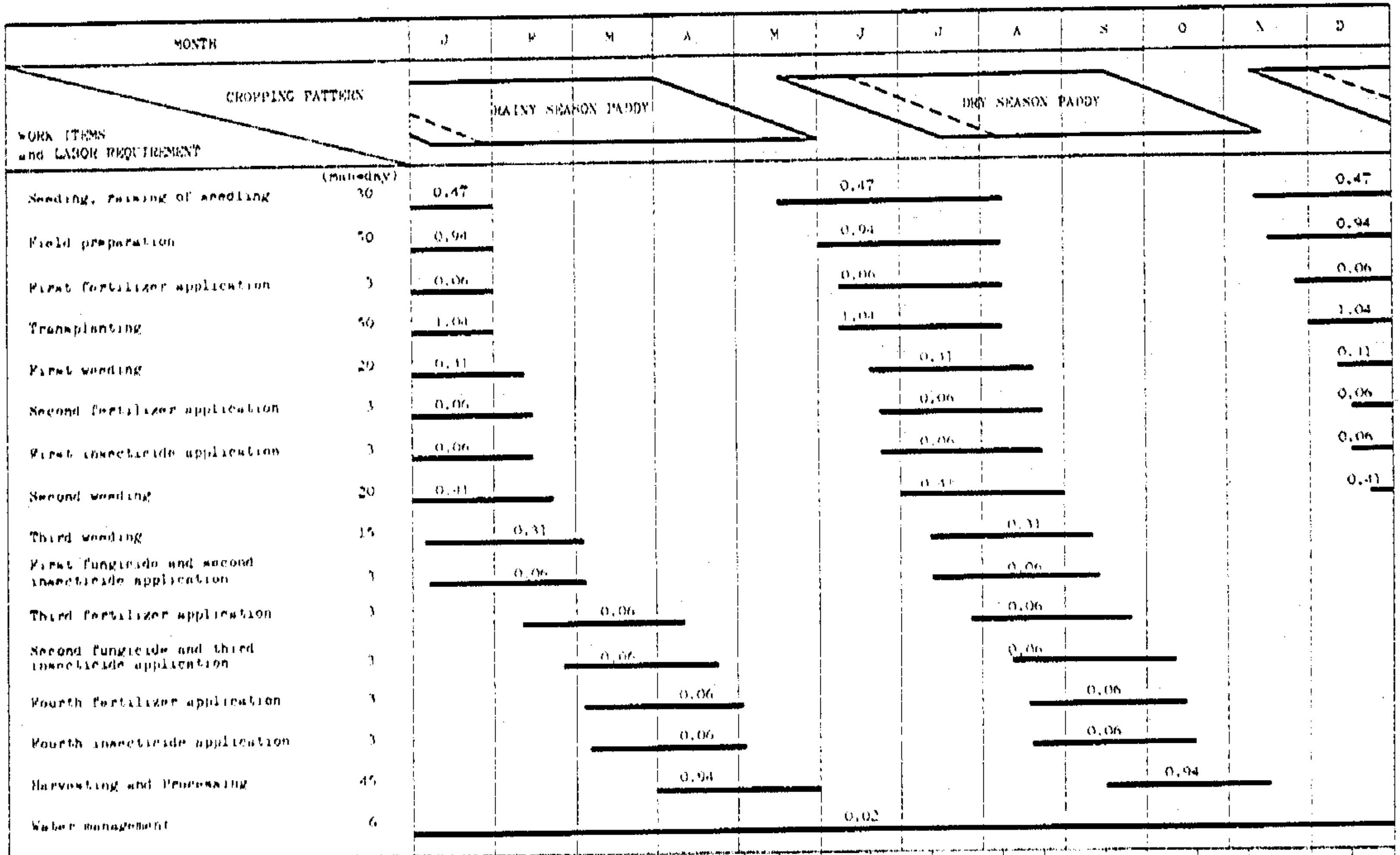
Sub-area	Cropping pattern	Dry season paddy (ha)	Rainy season paddy (ha)	Total (ha)
A	Double cropping	1,140	1,140	2,280
	Single cropping	340	390	730
Sub-total		1,480	1,530	3,010
B	Double cropping	5,410	5,410	10,820
	Single cropping	170	1,820	1,990
Sub-total		5,580	7,230	12,810
C	Double cropping	2,800	2,800	5,600
	Single cropping	-	940	940
Sub-total		2,800	3,740	6,540
D	Double cropping	8,620	8,620	17,240
	Single cropping	-	2,900	2,900
Sub-total		8,620	11,520	20,140
E	Double cropping	6,050	6,050	12,100
	Single cropping	-	2,030	2,030
Sub-total		6,050	8,080	14,130
Total (A - E)		24,530	32,100	56,630

Table VI-24 Monthly Labour Requirements

Month	J	F	M	A	M	J	J	A	S	O	N	D	Total
Total Labour Requirement	90.5	23	10.5	27	30	60.5	90.5	44	20.5	26.5	24.5	72.5	520
Available family labour	72	72	72	72	72	72	72	72	72	72	72	72	864
Required seasonal labour to be hired /1	35	-	-	-	-	12	30	12	-	-	-	19	108

/1 : These seasonal labours to be hired would be required to supplement a shortage of family labour in a short time, and these do not mean the monthly balance between the total labour requirement and the available family labour, refer to Fig. VI-11.

Fig. VI-11 Seasonal Labor Requirement for Paddy Rice Cultivation per Ha under the Proposed Farming Practices



Total daily labor requirement [1]	man-day	3.81	1.39	0.9	0.26	1.2	1.14	0.9	1.43	2.53	3.47	3.9	1.57	1.06	1.2	1.14	0.96	2.37	2.94	3.45
Available family labor a day [2]	man-day	2.4	2.4	2.4	2.4	2.4	2.4	2.4	2.4	2.4	2.4	2.4	2.4	2.4	2.4	2.4	2.4	2.4	2.4	2.4
Balance of labor requirement	man-day	1.41	-	-	-	-	-	-	-	0.13	1.07	1.5	-	-	-	-	-	-	0.5	1.05
Required seasonal labor to be hired	man-day	35	-	-	-	-	-	-	-	2	20	12	-	-	-	-	-	-	6	13

[1]: This is estimated based on the number of workable days, 24 days per month.

[2]: The workability of family labour is estimated at 80% with some allowance for sickness and other works.

Table VI-25 Potential Labour Force

A. Number of registered labour force

Speciality	Registration Office: Banjarbaru/1	Banjarmasin	South Kalimantan and Central Kalimantan
Mechanics	94	126	352
Economics	3	19	52
Administration	162	252	620
Marketing	18	44	95
Social services	28	44	653
Agriculture	17	233	298
Transportation and Communication	579	837	915
Total	901	1,555	2,985
B. Number of latent labour force/2			
	9,000	7,500	29,000
C. Number of employee requested			
	7	50	130
D. Potential labour force			
(A+B-C)	9,894	9,005	31,855

Data source: Kantor Wilayah Directorat Jenderal Pembinaan dan Penggunaan Tenaga Kerja, South Kalimantan and Central Kalimantan Banjarbaru.

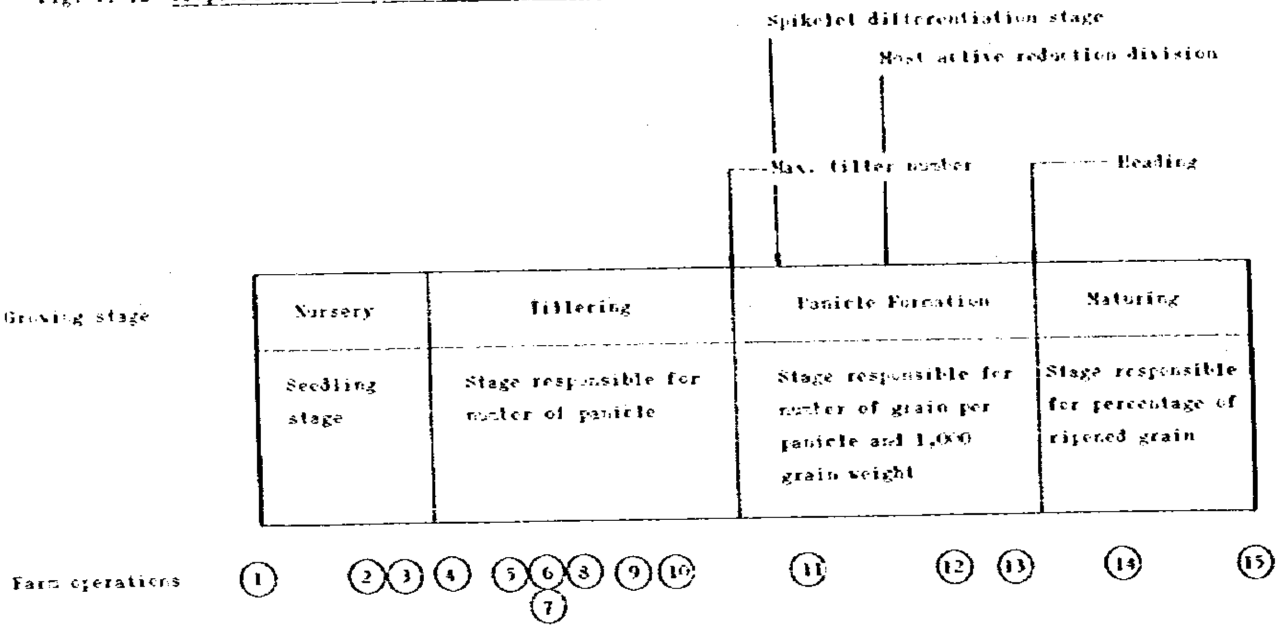
/1 : This office covers three Kabupaten of Banjar, Tapin and Hulu Sungai Selatan.

/2 : These numbers are estimated based on the information obtained from the above registration offices.

Table VI-26 Merit and Demerit of Transplanting and Direct Seeding Methods

Work	Transplanting method		Direct seeding method	
	Merit	Demerit	Merit	Demerit
1. Seed requirement	Small. 25 kg/ha approx.			Rather high. 50 to 75 kg/ha
2. Maintenance of nursery & seedlings	Small nursery bed with small labour requirement and farm inputs		No nursery bed is required.	Large labour requirement and farm inputs for maintenance of seedlings
3. Soil preparation	Very smooth land levelling is not required for transplanting.			Very smooth land levelling of paddy field, which calls for high labour requirement, is required for even germination of seeds.
4. Seeding	Seeding is easier in small nursery bed under dry condition.			Careful seeding is required particularly for heavy clayey soils which will disturb even germination, because seeds sown are coated with clay.
5. Transplanting		Large labour requirement is needed for transplanting.	Small labour requirement is needed, approximately 20 % of total labour required for transplanting method.	
6. Weeding	Weeding is easily carried out by hand or using rotary weeder.			Weed control using chemicals is required, which will be costly and will result in environment pollution.
7. Lodging	Paddy rice transplanted has tolerance to lodging.			Rather low tolerance to lodging

Fig. VI-12 Proposed Schedule of Farm Works and Farm Inputs Required for Each Growing Stage of Paddy



Farm operation	Farm input/ha		Days after Transplanting: Dry season paddy	Days after Transplanting: Rainy season paddy
1. Seeding, raising of seedling (0.03 ha)	Seed	25 kg	- 20	- 30
	Urea	10 kg		
2. Field preparation (Final)	Chemicals for seed treatment	100 g	- 5	- 5
3. First fertilizer application	Urea	60 kg		
	T.S.P	100 kg	- 5	- 5
	KCL	60 kg		
4. Transplanting(30cmx15cm in planting space)			0	0
5. First weeding			+ 10	+ 10
6. Second fertilizer application	Urea	60 kg	+ 15	+ 15
7. First insecticide application	Insecticide	1 liter	+ 15	+ 15
8. Second weeding			+ 20	+ 20
9. Third weeding			+ 30	+ 30
10. First fungicide and second insecticide application	Fungicide	1 liter	+ 30	+ 30
	Insecticide	1 liter		
11. Third fertilizer application	Urea	60 kg	+ 50	+ 70
12. Second fungicide and third insecticide application	Fungicide	1 liter	+ 65	+ 85
	Insecticide	1 liter		
13. Fourth fertilizer application	Urea	60 kg	+ 70	+ 90
14. Fourth insecticide application	Insecticide	1 liter	+ 75	+ 95
15. Harvest			+100	+120

Table VI-27 Requirement of Labour, Farm Inputs and Equipment for One Crop Paddy Cultivation per Ha with the Project

Item		Requirements
1.	**Labour**	
	Raising of seedling	30 man-day
	Field preparation	50 "
	Transplanting	50 "
	Weeding	55 "
	Fertilizing & spraying	24 "
	Water management	6 "
	Harvesting	30 "
	Threshing, drying & transportation	15 "
	Total	260 man-day
2.	**Seed, Fertilizers & Chemicals**	
	Seed	25 kg
	Fertilizers, Urea	250 kg
	T.S.P /1	100 kg
	KCL /2	60 kg
	Insecticide	4 lit
	Fungicide	2 lit
	Chemicals for seed treatment	100 gr
	Rodenticide	200 gr
3.	**Equipment**	
	Rotary weeder	2 sets
	Treadle thresher	1 set
	Winnover	1 set
	Knap-sac type mist-duster	1 set

/1 : Triple-super phosphate

/2 : Potassium chloride

Table VI-28 Crop Experiments on Alluvial Soils/1

Fertilizer Treatment			Average Paddy Yield (ton/ha)	Incremental Paddy Yield (ton/ha)
N (kg/ha)	P_2O_5 (kg/ha)	K_2O (kg/ha)		
(1) Test Farm				
0	0	0	2.97	-
60	0	0	3.45	0.48
120	0	0	5.10	2.13
90	23	0	5.25	2.28
120	23	0	6.45	3.48
120	23	30	6.20	3.23
(2) Individual Farm under BIMAS Program/2				
9.2	18.4	0	4.69	1.72

/1: This is the results of fertilizer dosage test on the high-yielding varieties of paddy carried out in the Brantas river basin, East Java Province, 1975/'76.

/2: These figures are estimated based on the average dosage in the Brantas river basin under the BIMAS Program. Paddy yield is obtained from the irrigated paddy field in the basin.

Data source:

Annual report on crop experiment, 1975/'76; the Central Research Institute for Agriculture, East Java, Malang

Table VI-29 Record of the Unit Yield of High-Yielding Varieties Cultivated in and around the Project Area

Locations	Year	Variety	Cultivated Area (ha)	Yield (ton/ha)
In the Project Area/1				
Kecamantan Gambut:				
Pemagantan	1977	IR-36	3	4.0
Guntung papuyu	1977	IR-36	2	4.2
Malintang	1977	IR-32	3	4.0
Kec. Kertak hanyar:				
Simpang empat	1977	IR-32	8	3.8
Kecamantan Karang Intan:				
Penyambaran	1977	IR-32	4	3.9
Tingah Habang	1977	IR-32	4	3.9
Mali-mali	1977	IR-36	7	3.9
Kecamatan Astambul:				
Pematang Danau	1977	IR-26	8	3.9
Kecamatan Sungai Tabuk:				
Sungai Lulut	1977	IR-26	1	4.1
Average yield				3.9
Area Under the BIMAS Program/2				
Intangan	1978	IR-32	0.1	4.2
Kahakan	1978	IR-28	0.1	5.1
Average yield (dry paddy)				4.7

/1: Data obtained from the demonstration farm in the farmers' field, directed by the Agricultural Extension Services, Kabupaten Banjar. No chemical fertilizers were applied, but four times of insecticide were applied and cultivated under the rain-fed condition.

/2: Data obtained from the Rural Extension Worker who is working in the demonstration farms carried out under BIMAS Program in the Irrigation Project of Intangan and Kahakan, Kabupaten Hulu Sungai Tengah.

Table VI-30 Projected Increase in Yield of Paddy under the Project

Condition/1	Weight (%)			Initial Year/2	1st Year	2nd Year	3rd Year	4th Year	5th Year	6th Year	7th Year
1. Level-up of farmers' cultivation technics	65	Projected progress of development	(%)	30	45	55	60	65	75	85	100
		Effect on crop production	(%)	19.5	29.3	35.8	39.0	42.3	48.8	55.3	65.0
2. Level-up of operation technics of on-farm facilities	30	Projected progress of development	(%)	0	30	45	55	70	100		
		Effect on crop production	(%)	0	9.0	13.5	16.5	21.0	30.0		
3. Stabilization of soil and land conditions	5	Projected progress of development	(%)	0	30	60	100				
		Effect on crop production	(%)	0	1.5	3.0	5.0				
Total		Integrated effect on crop production increase		19.5	39.8	52.3	60.5	68.3	83.8	90.3	100.0

/1: This shows the conditions to be improved under the Project which would affect the increase of crop production.

/2: Cultivation of paddy under the Project is commenced at the end of initial year.

Fig. VI-13 Prospective Development of Essential Agricultural Background and Foreseeable Build-up Period of the Unit Yield

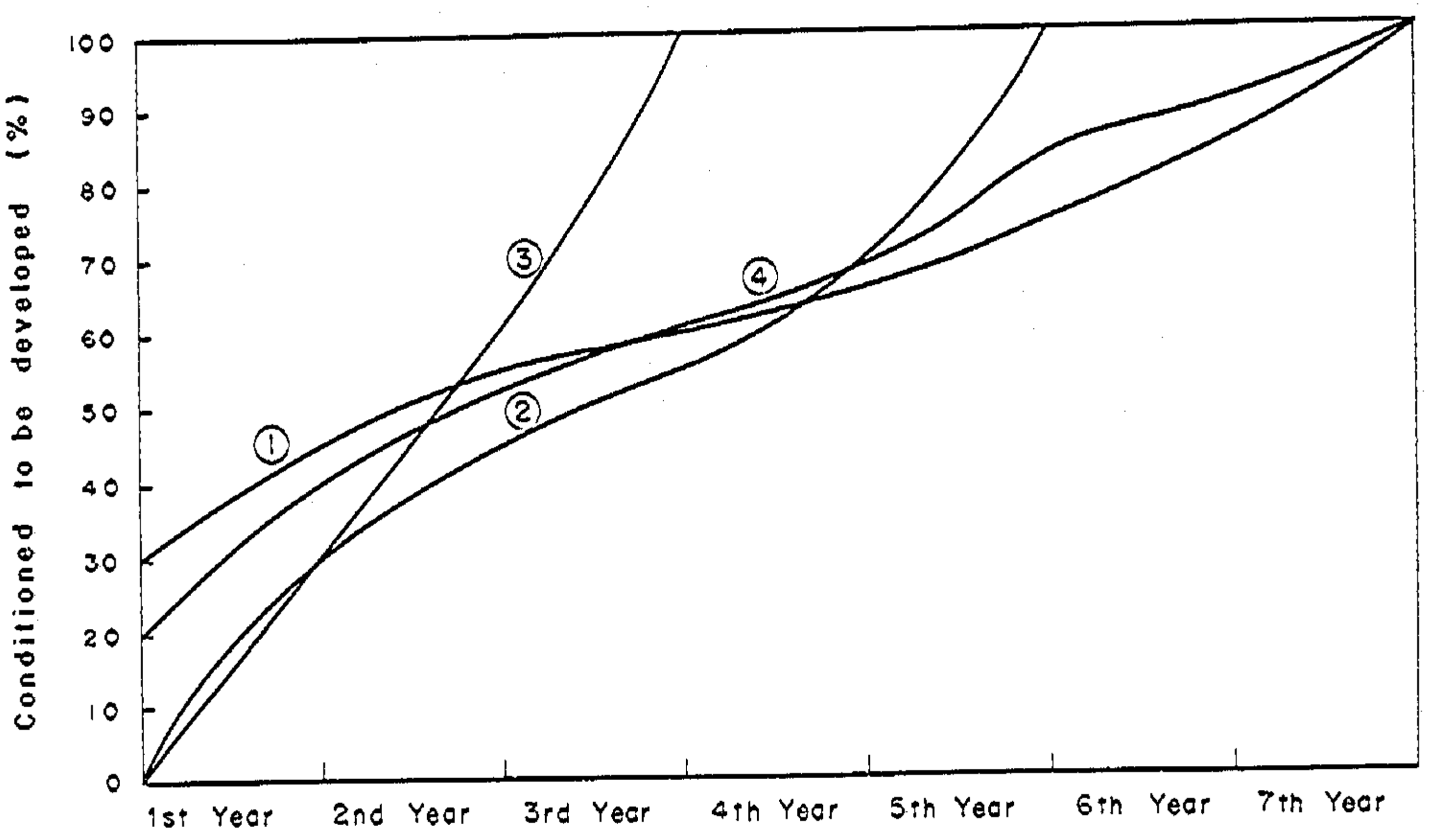


Table VI-31 Increase in the Unit Yield of Paddy

	Build-up Period (year)						
	1st	2nd	3rd	4th	5th	6th	7th
A. Projected progress of increase to target yield (%) /1	39.8	52.3	60.5	68.3	83.8	90.3	100.0
B. Target yield with the Project							
Dry season paddy (ton/ha)	4.5	4.5	4.5	4.5	4.5	4.5	4.5
Rainy season paddy (ton/ha)	4.0	4.0	4.0	4.0	4.0	4.0	4.0
C. Anticipated unit yield without the Project (ton/ha) /2	1.75	1.75	1.80	1.80	1.90	1.90	2.0
D. Anticipated yield (B - C) x A + C							
Dry season paddy (ton/ha)	2.8	3.2	3.4	3.6	4.1	4.2	4.5
Rainy season paddy (ton/ha)	2.6	2.9	3.1	3.3	3.7	3.8	4.0

/1: See Table VI-28.

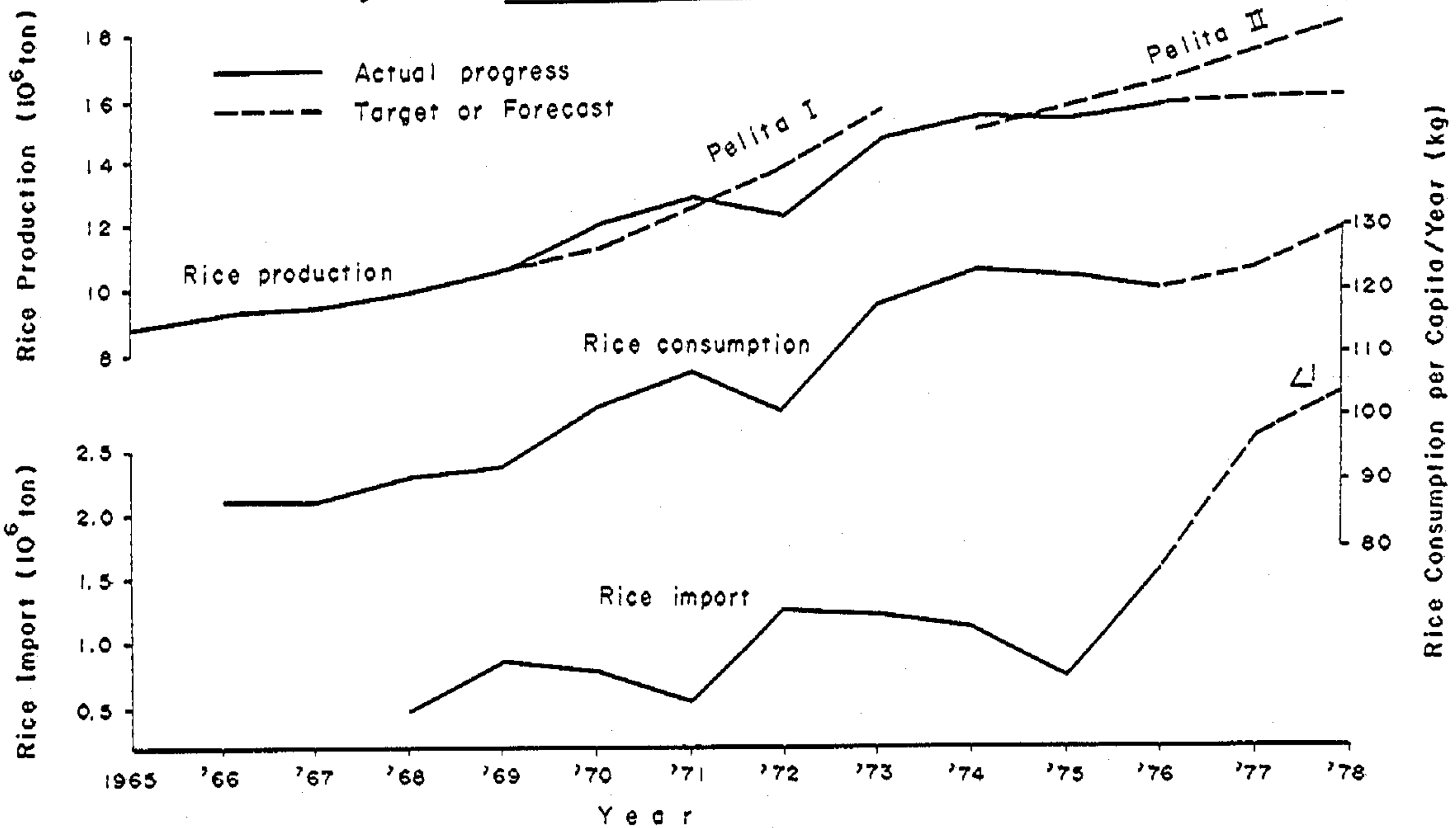
/2: The anticipated unit yield of paddy rice without the Project is expected to increase by efforts which will be made under the Pelita III and BIMAS/INMAS Programs. Considering the prevailing constraints in the project area, the expected yield increase would be very small under the condition without the Project.

Table VI-32 Anticipated Paddy Rice Production

Year		Future With-Project Condition: Cropped Area (ha) Sub-area A	Sub-area B	Sub-area C	Sub-area D	Sub-area E	Total	Production (ton) Sub-area A	Sub-area B	Sub-area C	Sub-area D	Sub-area E	Total	Future Without-Project Condition: Cropped Area (ha)	Production (ton)	Production Increment (ton)
1984	W	1,530	2,700	-	-	-		4,000	7,000	-	-	-				
	D	1,870	2,700				8,800	5,200	7,600				23,800	3,450	6,000	17,800
1985	W	1,530	7,230	1,580	-	-		4,400	19,600	4,100	-	-				
	D	1,870	7,400	1,580			21,190	6,000	21,800	4,400			60,300	6,860	12,000	48,300
1986	W	1,530	7,230	3,740	4,670	-		4,700	21,500	10,200	12,100	-				
	D	1,870	7,400	3,740	4,670		34,850	6,400	24,200	11,100	13,100		103,300	14,080	25,300	78,000
1987	W	1,530	7,230	3,740	11,520	2,420		5,000	22,900	11,200	31,300	6,300				
	D	1,730	6,740	3,400	10,460	2,200	50,970	6,200	23,400	11,200	31,000	6,200	154,700	23,970	43,100	111,600
1988	W	1,530	7,230	3,740	11,520	8,080		5,700	24,900	11,900	34,400	21,700				
	D	1,480	5,580	2,800	8,620	6,050	56,630	6,100	21,100	9,700	28,300	17,700	181,500	29,960	56,900	124,600
1989	W	1,530	7,230	3,740	11,520	8,080		5,800	27,100	12,900	36,600	23,900				
	D	1,480	5,580	2,800	8,620	6,050	56,630	6,200	23,100	10,600	30,000	19,800	196,000	29,960	56,900	139,100
1990	W	1,530	7,230	3,740	11,520	8,080		6,100	28,000	14,000	39,900	25,500				
	D	1,480	5,580	2,800	8,620	6,050	56,630	6,700	24,100	11,600	32,800	20,900	209,600	29,960	59,900	149,700
1991	W	1,530	7,230	3,740	11,520	8,080		6,100	28,900	14,500	43,000	27,700				
	D	1,480	5,580	2,800	8,620	6,050	56,630	6,700	25,100	12,100	35,700	22,700	222,500	29,960	59,900	162,600
1992	W	1,530	7,230	3,740	11,520	8,080		6,100	28,900	14,900	44,700	30,100				
	D	1,480	5,580	2,800	8,620	6,050	56,630	6,700	25,100	12,600	37,200	25,000	231,300	29,960	59,900	171,400
1993	W	1,530	7,230	3,740	11,520	8,080		6,100	28,900	14,900	46,100	31,200				
	D	1,480	5,580	2,800	8,620	6,050	56,630	6,700	25,100	12,600	38,800	25,900	236,300	29,960	59,900	176,400
1994	W	1,530	7,230	3,740	11,520	8,080		6,100	28,900	14,900	46,100	32,300				
	D	1,480	5,580	2,800	8,620	6,050	56,630	6,700	25,100	12,600	38,800	27,200	238,700	29,960	59,900	178,800

Note: W: Wet Season
D: Dry Season

Fig. VI-14 Rice Demand and Supply Condition in Indonesia



∠1: World grain situation outlook for 1978/79, FAO, Aug. in 1978.

Data Source: 1) Statistic Indonesia 1972/73, 1975/76.
2) Indicator Economy, 1973.
3) Data from BULOG, 1977.
4) Data from Embassy of Japan (J.K.T), 1977.
5) Data from Asian Economy Institute, Japan 1972.
6) Asian Economy Institute, Japan (Yearly Economy Report, Indonesia), 1978.

Table VI-33 Rice Production and Consumption in Kalimantan

Year	East Kalimantan (1,000 ton)	Central Kalimantan (1,000 ton)	West Kalimantan (1,000 ton)	South Kalimantan (1,000 ton)	Total (1,000 ton)
1971	- 55	- 3	- 40	+ 48	- 50
1972	- 58	- 9	- 41	+ 77	- 31
1973	- 81	- 2	- 41	+109	- 15
1974	- 56		- 23	+ 51	- 28
1975	- 53	- 6	- 28	+ 36	- 51
1976	- 67	- 8	- 36	+ 64	- 47

Note: Minus figures in this table mean shortage of rice, and plus figures show surplus of rice.

Data source: BAPPEDA in South Kalimantan

Table VI-34 Rice Production and Consumption in South Kalimantan

Year	Production (1,000 ton)	Consumption (1,000 ton)	Surplus (1,000 ton)
1971	253	205	48
1972	286	209	77
1973	323	213	109
1974	326	275	51
1975	318	282	36
1976	330	266	64
1977	348	290	58

Data source : BAPPEDA in South Kalimantan

Fig. VI-15 Paddy Production and Demand Prospects in Kalimantan

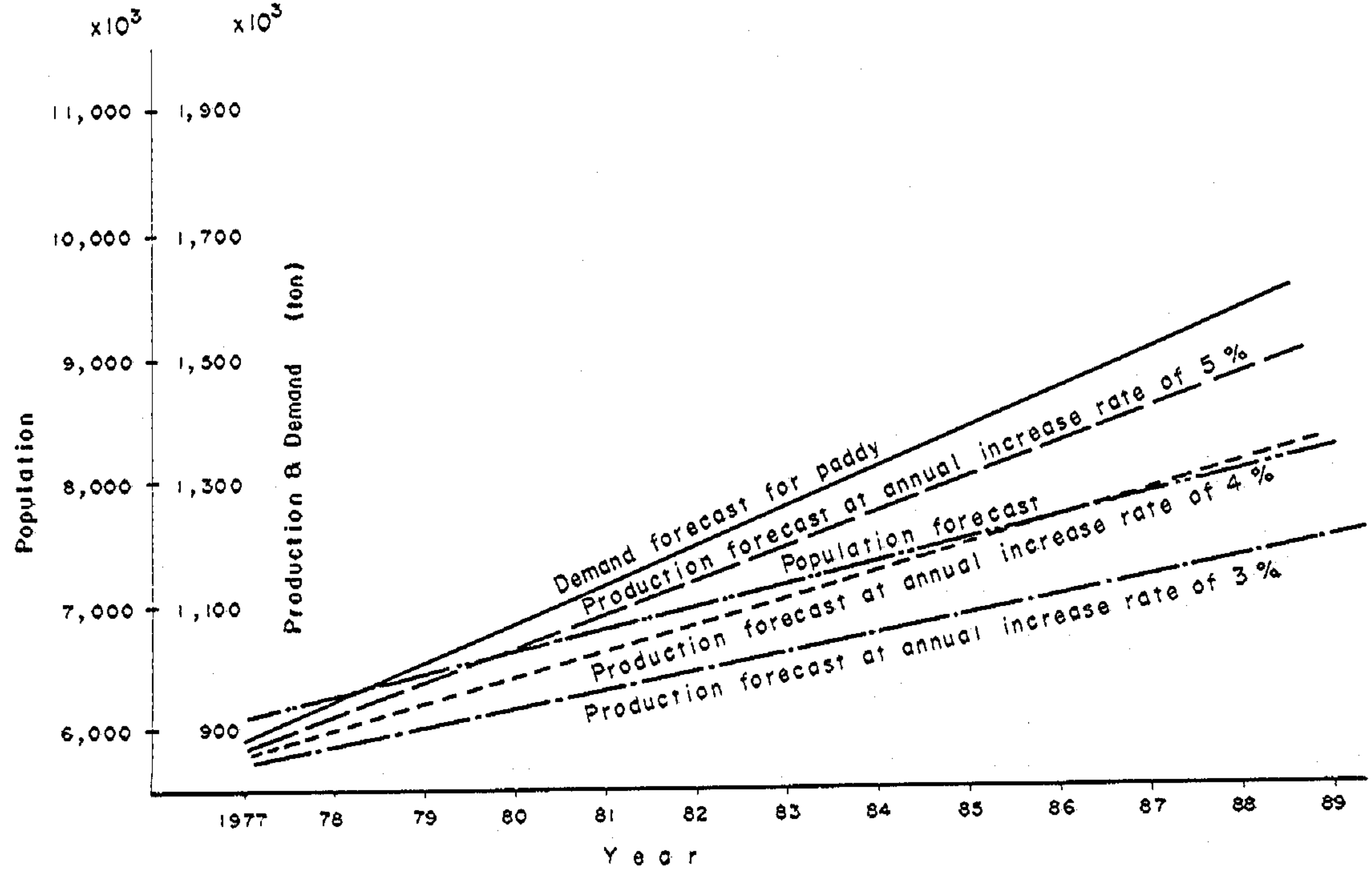
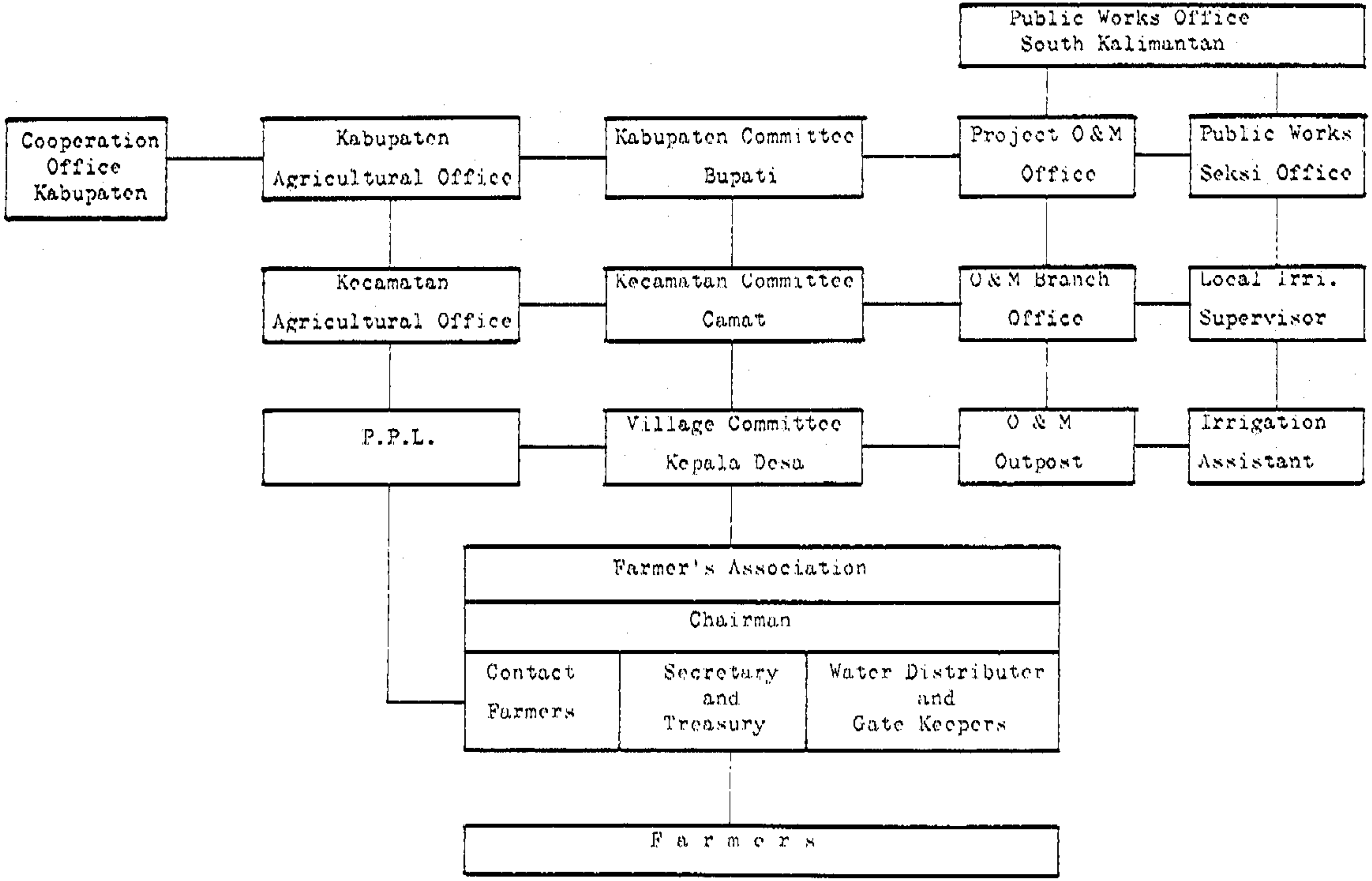


Fig. VI-16 Organization Chart for Farmer's Association



ANNEX VII

PLANS FOR IRRIGATION, DRAINAGE AND RURAL WATER SUPPLY

Table of Contents

List of Tables

List of Figures

ANNEX VII PLANS FOR IRRIGATION, DRAINAGE AND RURAL WATER SUPPLY

VII.1 PLANS FOR IRRIGATION

General

There is no competition between the irrigation and drainage systems in an irrigation project. Irrigation canals and distributaries occupy the higher elevations throughout the areas, the drainage system the lower areas. The irrigation water is distributed over the land whilst the drainage water is removed from the land.

Considering the topographical features, the project area can be divided into six, sub-areas A, B, C, D, E and F, in view of the alignment of irrigation and drainage systems. Out of the above six sub-areas, the sub-area F is excluded from the Project on the basis of the following discussion.

In order to supply the irrigation water for the sub-area F, the water would have to be lifted from the main irrigation canal in a few steps instead of one due to complicated topography for lifting water. This would incur high construction costs and difficult operations. In addition, in view of the topography in the most areas, it is rather difficult to arrange a suitable field plot for proper operation of irrigation. Besides, the soil having coarse texture with exposed gravels throughout the profile limits the field management of irrigation and farm operations. It is considered that the most land in this area are economically unsuitable for irrigation development.

In determining the location of the main irrigation canal, there exist two major constraints in this project. One is the existence of the Riam Kanan dam, and the other is the soil conditions along the canal. Both restrict the location and elevation of diversion weir and the main canal. The diversion weir would have to be designed so as not to disturb dam operations due to backwater caused by damming up the river water level of the Riam Kanan, and the main canal would have to be aligned avoiding the swamp areas in which the soil conditions for the construction of the canal are obviously bad.

These two restrictions confine the water level at the main canal head between El. 9.00 m and 11.00 m. Within this range, the economic comparative study is made in combination of the diversion weir and the main canal. The study result indicates that the optimum water level would be El. 9.80 m at the canal head. The location of the canal route is then inevitably settled. The irrigability of the area for the Project is examined within the areas which are effectively irrigable with this main canal.

Sub-area A

Out of the total gross area of 3,600 ha, the suitable irrigable land with an area of 3,000 ha is selected in view of soil, drainability, present land use and vegetation. The irrigability is examined for the suitable land. It is found that the easternmost land with an area of 500 ha and a part of land along the main canal with an area of 300 ha could not be irrigated properly by gravity.

The reason is that both of the lands are higher than the design water level in the main canal. The irrigable area would then be 2,200 ha in gross.

Sub-area B

Out of the total gross area of 18,380 ha, the suitable irrigable land with an area of 9,180 ha is selected in view of soil, drainability, present land use and vegetation. The study on the irrigability for the suitable land indicates that there are two areas where irrigation could not be practiced properly. One is located in the south of the area adjacent to the main canal, and the other is situated in the northernmost of the area in between the river Martapura and the road. Irrigation of the latter land is difficult, because there is a fairly large depression on the route for proposed secondary canal which leads to the land to be irrigated, and very large embankment is expected. The area of the former is 270 ha and the latter 210 ha, totaling 480 ha. The irrigable area would then be 8,700 ha in gross.

Sub-area C

The total gross area of the sub-area C is 4,400 ha. The investigation on soil, drainability, present land use and vegetation reveals that the whole area of 4,400 ha are suitable for irrigation farming. The area is characterized by very flat topography in view of irrigation. The provisional alignment of the irrigation canal system indicates that the entire area can be irrigated by gravity without any difficulty. Therefore, the irrigable area would be 4,400 ha in gross.

Sub-area D

Out of the total gross area of 18,050 ha, the suitable irrigable land with an area of 13,550 ha is selected in view of soil, drainability, present land use and vegetation. The suitable land thus selected is characterized by very gentle slope from east to west. There exist very few local depressions. The provisional alignment of the irrigation canal system for the said suitable land indicates that the whole area can be irrigated by gravity without any difficulty. Therefore, the irrigable area would be 13,550 ha in gross.

Sub-area E

The total gross survey area of the sub-area E is 26,100 ha. The irrigable land to be developed in view of soil class, drainability, present land use and vegetation is provisionally estimated at 9,510 ha, which is bounded on the north by the sub-area D, on the east by the land unsuitable for development in terms of soil and drainability, on the south by the river Maluka, and on the east by the sand dune.

The possibility of a further irrigation development in the sub-area E using the Maluka river is studied in two ways. One is to use the water from the river Maluka by gravity. Since the elevation of the area to be developed ranges between El. 0.3 m and El. 1.8 m, the size and the location of the headworks on the river Maluka would have to be determined to feed water with water level higher than El. 2.0 m. In this context, it is noted that the longitudinal slope of the river is very gentle, and to satisfy the above condition the suitable site for headworks is found at about 25 km upstream from the area to be developed. The discharge in the dry season at this point is tentatively estimated at 2 to 3 m^3/sec, despite the fact that the gross water requirements to irrigate this possible area in the dry season are as 11.88 m^3/sec. Moreover, it is also noted that the topographical and soil conditions along the proposed route for the irrigation canal are obviously bad, because marshy depression extends most of the proposed route. An enormous amount of embankment is anticipated.

The other plan is to pump-up water in the lower reaches of the river, since the discharge increases as the lower the river reaches are. There are two suitable pumping sites just downstream of the confluence of the river Panggaaungan, a tributary of the river Maluka. The river discharge at these points is provisionally estimated at 4.8 m^3/sec. The biggest trouble of the second plan is the intrusion of the sealine water into the Maluka. This would require the construction of the tide gate structure on the river Maluka, and the operating hour of the pumps would be restricted.

VII.2 IRRIGATION WATER REQUIREMENTS

VII.2.1 General

Field measurement of crop consumptive use water by paddy rice was carried out from the middle of August at the experimental farm in the sub-station of Central Research Institute for Agriculture located in the sub-area C. The study on water requirements is made based on this measurement.

VII.2.2 Evaporation

After processing the meteorological records available at Banjarbaru for the period of 1965 to 1977, a set of potential evapotranspiration is prepared using the Penman method. A comparison between the calculated potential evapotranspiration and the evaporation from class-A pan located at the same station is made for the cross checking. The result indicates that the former is less than the latter by about 10 %.

The reason is thought to be the fact that the sunshine hour recorded is somewhat shorter than expected. Therefore, it seems practical to use the value of evaporation from class-A pan in connection with the crop coefficient. As the pan evaporation is measured for only three years, it is impossible to assess the probable value statistically. In view of giving some allowance, the highest mean values of the three years, among which there are small differences, are adopted. The evaporation records and calculated potential evapotranspiration values using the Penman method are shown below.

Pan Evaporation (EP) and
Potential Evapotranspiration (ETP)
(mm/day)

	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sept.	Oct.	Nov.	Dec.
EP	3.7	3.5	4.2	4.1	3.9	3.8	4.5	5.1	5.5	5.4	4.1	3.5
ETP	3.5	3.3	3.6	3.8	3.6	3.3	3.4	4.4	4.4	4.5	3.7	3.3

Crop Coefficient

Six sets of measurement tank were installed to obtain the various records. Since the time period for experimentation was limited to obtain all the data from the transplanting to the yellow ripening stages, two growing stages of paddy were subjected to the experiments. One is Pelita I/1 at the stage of 14th day after seeding, and the other is IR-5 at the stage of 64th day. Pan-evaporation and rainfall were observed simultaneously to corelate with the evapotranspiration.

It is noted, however, that the measurements are not necessarily sufficient in view of reliability to estimate the crop coefficients because of only one experiment for the dry season paddy. In addition to these, those derived from the experimental results at the trial farms conducted by FAO staff near Palembang city are also taken into consideration. The comparative study reveals that the crop coefficients in this project area are higher than those at the said trial farm at the early and the late growing stages.

The crop coefficient curve at the growing stages between transplanting and maturing of paddy is prepared as shown in Fig. VII-1, based on the experimental results obtained in the project area and in Palembang. This crop coefficient curve is practically acceptable to estimate the consumptive use of water by paddy rice to be introduced into the Project. The period of the growing stage expressed in percentage varies with the variety of paddy. Based on this curve, the variance of the coefficients to the growing stages between transplanting and maturing is estimated as shown below.

Crop Coefficient of Paddy

Percentage of growing stage (%)	10	20	30	40	50	60	70	80	90	100
Crop coefficient	0.86	0.97	1.10	1.22	1.32	1.37	1.30	1.17	0.98	0.67

VII.2.3 Consumptive Use by Paddy Rice

The consumptive use of water by crops may be expressed as the products of class-A pan evaporation and crop coefficient which take specific value with the growing stages as stated above. Thus, evapotranspiration or consumptive use of paddy rice is worked out by multiplying evapotranspiration by crop coefficient as shown below.

10-Day Consumptive Use by Rainy Season Paddy in mm/day

Dec.			Jan.			Feb.			Mar.			Apr.			May		
E	M	L	E	M	L	E	M	L	E	M	L	E	M	L	E	M	L
2.9	3.2	3.3	3.7	3.9	4.1	4.4	4.6	4.6	5.1	4.7	4.5	4.1	3.8	3.4	2.6	-	-

10-Day Consumptive Use by Dry Season Paddy in mm/day

June			July			Aug.			Sept.			Oct.		
E	M	L	E	M	L	E	M	L	E	M	L	E	M	L
-	3.3	3.5	4.5	4.8	5.0	5.9	6.2	6.2	6.3	6.2	5.9	5.2	4.5	3.6

VII.2.4 Puddling Water Requirements

The quantity of water required for puddling works is theoretically assessed by soil depth to be saturated and porosity. However, both vary relatively from place to place. Averagely, the puddling water requirement is estimated by using the following formula and assumptions :

$$PW = DS + WS$$

where,

PW : Puddling water requirement in mm

DS : Water depth above soil surface after puddling in mm

WS : Difference in soil moisture before and after puddling water needed for saturation of soil in mm

(1) Water depth on soil surface after puddling is 45 mm.

(2) Porosity is assumed to be 50 % on both surface soil with depth of 20 cm and sub-soil with depth of 10 cm.

(3) Vapour phase in soils after the puddling is assumed to be 5 %.

(4) Soil moisture before irrigation is 20 % in volume.

Then, the puddling water requirement is calculated as follows :

$$PW = 45 + 300 \times (0.5 - 0.05) - 20 \times 300 = 120 \text{ mm}$$

VII.2.5 Percolation Rate

Three sets of cylinder were installed in the said substation of the institute for the measurements of percolation only.

The results of the measurement show very small values even in the driest season. Therefore, the percolation rate is assumed to be 1 mm/day in the dry season and 0.5 mm/day in the wet season.

Horizontal percolation through the ponds in terraced land supplies water to the lower adjacent fields. Since this water is not lost, such percolation need not be considered in the overall irrigation requirements.

VII.2.6 Washing Water

The percolation rate suitable for paddy cultivation is commonly reported to be the range between 2 to 3 mm/day and 80 mm/day. According to the experiments in the project area, however, percolation rate is about 1 mm/day even in the driest season. Some experiments are being carried out for the land reclamation project in Japan in order to obtain data on additional water requirement for dilution of salinity in the soils. According to the interim report on this experiment, about 300 mm in depth of water in total would be required for one cropping of paddy rice to expect high yields.

In this context, the supply of 2 mm/day of water to paddy land would be proposed so as to provide the land conditions favourable for paddy cultivation through increased horizontal seepage and percolation actions, as well as dilution of acid water. The supply of 2 mm/day of water means that about 500 mm in depth of water will be used for the above purposes for two croppings of paddy rice in this project area.

VII.2.7 Effective Rainfall

Effective rainfall in the project area is estimated by applying the daily water depth balance method, using the following assumptions :

(1) Rainfall less than 5 mm/day is ineffective.

(2) The excess beyond 50 mm/day is ineffective.

(3) 90 % to the total of each 10-day rainfall through the above procedure is effective.

Daily rainfall data for the period of 18 years at Banjarbaru station are used. The results of the calculation with the recurrence interval of once five drought years are shown below :

10-Day Effective Rainfall for Rainy Season Paddy in mm/day

Dec.			Jan.			Feb.			Mar.			Apr.			May		
E	M	L	E	M	L	E	M	L	E	M	L	E	M	L	E	M	L
5.9	6.9	8.0	8.9	7.6	7.5	6.9	6.6	5.4	6.0	7.5	7.4	4.9	3.8	4.6	4.2	-	-

10-Day Effective Rainfall for Dry Season Paddy in mm/day

June			July			Aug.			Sept.			Oct.		
-	2.7	3.1	1.5	2.8	2.7	1.1	1.2	2.0	2.6	2.7	2.9	3.2	1.9	3.4

VII.2.8 Farm Requirements

The farm requirements or net irrigation water requirements are obtained by using the following formula :

$$FR = CU + PL + WW + PW - ER$$

where, FR : Farm requirements

CU : Consumptive use of water

PL : Percolation loss

WW : Washing water

PW : Puddling water

ER : Effective rainfall

The results of calculation of the farm requirements are shown below.

10-Day Farm Requirements for Rainy Season Paddy in mm/day

Nov.			Dec.			Jan.			Feb.			Mar.		
E	M	L	E	M	L	E	M	L	E	M	L	E	M	L
-	-	2.0	2.0	2.0	2.0	2.0	2.0	-	-	0.5	1.7	1.6	-	-

Apr.			May		
E	M	L	E	M	L
1.3	1.3	0.4	0.2	-	-

10-Day Farm Requirements for Dry Season Paddy in mm/day

June			July			Aug.			Sept.			Oct.		
E	M	L	E	M	L	E	M	L	E	M	L	E	M	L
2.0	2.6	3.1	5.0	5.3	6.4	7.8	8.0	7.2	6.7	5.4	4.0	2.5	1.9	0.5

VII.2.9 Gross Irrigation Requirements

Certain losses are unavoidable for conveying water and supplying it to the farm. The gross irrigation water requirements are obtained by dividing the farm irrigation requirements by the conveyance and operation efficiency. The most of irrigation canals over the project area are planned as unlined, and, thus, they incur rather high conveyance losses which are estimated at 20 % and operation losses at 15 %, resulting in

68 % of total efficiency. This value is generally adopted for the other irrigation projects in Indonesia.

The gross irrigation requirements are estimated as shown below.

10-Day Gross Irrigation Requirement for Rainy Season Paddy in mm/day

Nov.			Dec.			Jan.			Feb.			Mar.		
E	M	L	E	M	L	E	M	L	E	M	L	E	M	L
-	-	2.9	2.9	2.9	2.9	2.9	2.9	-	-	0.7	2.5	2.4	-	-

Apr.			May		
E	M	L	E	M	L
1.9	1.9	0.6	0.3	-	-

10-Day Gross Irrigation Requirement for Dry Season Paddy in mm/day

June			July			Aug.			Sept.			Oct.		
E	M	L	E	M	L	E	M	L	E	M	L	E	M	L
2.9	3.8	4.6	7.4	7.8	9.4	11.5	11.8	10.6	9.9	7.9	5.9	3.7	2.8	0.7

The maximum 10-day water requirement would occur in mid-August and would be 11.8 mm/day which are translated as 1.37 lit/sec/ha. Table VII-1 summarizes the discussions so far made.

VII.3 RURAL WATER SUPPLY

The total population in the sub-areas A, B, C, D and E is about 340,000, excluding Banjarmasin city where municipal water supply facility is available. Provided that per-capita consumption of water is 100 lit/day, the total consumption would be 0.39 m^3/sec, which corresponds to 1.1 % to the maximum total water requirements for the above areas. In the design of the capacity of the main canal, 0.40 m^3/sec of water for rural supply is taken into consideration, giving allowance.

VII.4 PRESENT CONDITIONS OF FLOODS AND DRAINS

It is noted that each of the sub-areas is separated in terms of drainage conditions either by the lay of the land or roads. There are no incoming and outgoing of water among the sub-areas. Therefore, the water balance studies for each sub-area could be made independently.

The survey results indicate that the lands could be divided into four grades in view of the present flood conditions as stated below.

Grade 1 : Areas which are free from the seasonal floodings, including the areas poorly drained due to micro-relief condition.

Grade 2 : Areas which suffer from the seasonal floodings from the beginning of December to the end of March. The maximum water standing is assumed to be 30 cm.

Grade 3 : Areas which suffer from the seasonal floodings from early November to the end of May. The maximum water standing is assumed to be 50 cm.

Grade 4 : Areas which are submerged throughout the year. The maximum water standing is assumed to be more than 100 cm (mid-February) and the minimum, 30 cm (beginning of September) on an average.

The map showing the present flooding status is presented in Fig. VII-2, which indicates that there exists a considerable extent of areas suffering from inundation annually.

In order to estimate the present outflow condition in the sub-area B, of which retarding effect seems very large, a water balance among rainfall, evapotranspiration, percolation and the amount of submergence is made using the present available data. Evapotranspiration from the land surface is assumed to be as high as 80 % of class-A pan evaporation, since the area is more or less submerged or very wet condition throughout the year. Percolation is regarded to be practically zero.

<u>Sub-area A</u>

The sub-area A is divided into two, north (sub-area A_1) and south (sub-area A_2) in view of drainage, since it is separated by a road running from east to west and no incoming and outgoing of water between the two sub-areas are observed.

Such of the catchment areas is classified as follows :

Sub-area	Total catchment area (ha)	Grade 1 (ha)	Grade 2 (ha)	Grade 3 (ha)	Grade 4 (ha)
Sub-area A_1	1,550	80	420	580	470
Sub-area A_2	6,550	-	-	1,030	5,520
Total :	8,100	80	420	1,610	5,990

The sub-area A_1 is characterized by a polder surrounded almost the whole area by a dike. There is no inflow from the outside areas except for the Riam Kanan river, from which inflow is checked by the gates at the outlet. Since the area is fairly low and flat, there is a remarkable retarding effect within the area. The water balance study reveals that the pattern of discharge would be flatter than that of rainfall and, hence, there would be a lag of time of peak discharge from that of rainfall. The volume of water to be stored in this sub-area is estimated to be 4.9 million m^3.

The features of the sub-area A_2 could be characterized by steep slope and relatively large catchment basin behind it. Since there is not so large area to be inundated, the runoff pattern of the area would be consistent with rainfall pattern. Rain water is collected to a few numbers of small rivers, which join a fairly large channel connecting the river Riam Kanan with the river Martapura.

Sub-area B

It is noted that the vast extent of the area is occupied by peat which is submerged throughout the year. As the adjacent land, which is also low and flat, is subject to attack of floods, a very large area suffers from seasonal submergence. These areas other than the peat area are being used as paddy field depending on the degree of submergence.

The area is classified based on the depth and duration of inundation as follows :

Total catchment area (ha)	Grade 1 (ha)	Grade 2 (ha)	Grade 3 (ha)	Grade 4 (ha)
22,010	3,680	4,230	7,020	7,080

The above table indicates that about 7,000 ha of land are submerged from early November, and additional 4,200 ha from the beginning of December. The total area to be submerged are, therefore, 18,300 ha including the area perenially submerged. The volume of storage water would be as big as 82 million m^3 in the middle of February.

Fig. VII-3 shows the rough relationship among the storage volume (submerged depth is converted to the depth to the total catchment area), rainfall and outflow in mm to the total catchment area. There is a remarkable time lag between the rainfall pattern and outflow pattern. The peak discharge would take place in the middle of February, whereas the biggest rainfall in January. The highest average discharge is estimated at 25 m^3/sec through the existing drains.

Sub-area C

Since the sub-area C is enclosed by roads and the river Martapura, there is no water coming from and going to the adjacent area. Despite the fact that the area is low and flat, drainage is made naturally fairly well through the existing three small rivers. The area submerged before December and after April is small. Most of the area are used as paddy field.

The area is classified in view of drainage condition as follows :

Total catchment area (ha)	Grade 1 (ha)	Grade 2 (ha)	Grade 3 (ha)	Grade 4 (ha)
7,790	70	7,270	450	-

Sub-area D

The area is characterized by the tidal effect from the river Barito. Since the western one-quarter of the area is flat and lower than the high tide except for only the river side belt, the area is more or less submerged throughout the year. Three-fourths of the eastern area, which are a little bit elevated and free from the large tidal effect, are well developed as paddy field, because the area is flooded only from December to March with shallow water depth.

The area is classified based on the depth and duration of inundation as follows :

Total catchment area (ha)	Grade 1 (ha)	Grade 2 (ha)	Grade 3 (ha)	Grade 4 (ha)
25,300	1,980	15,520	2,430	5,370

Sub-area E

The total catchment area is 83,050 ha. The gross area which could be taken up for the survey area is 26,100 ha, as the possible area to be developed. The area is bounded on the north by the sub-area D, on the west by the river Barito, on the south by the river Maluka and on the east by the sub-area F.

The western area is characterized by the tidal effect from the river Barito, whereas the eastern area by the flood from the river Maluka. Since the western one-fifth of the area is flat and lower than the high tide except for only the river side belt, the area is more or less submerged throughout the year. Two-fifths of of the central area, which is slightly higher and free from the large tidal and flood effects, are well developed as paddy field. It is noted that the vast extent of the eastern area (two-fifths) is occupied by swamp which is submerged throughout the year. As the adjacent land which is also low and flat is subject to floods, the most of the area suffers from seasonal submergence.

The area is classified based on the depth and duration of inundation as follows :

Total survey area (ha)	Grade 1 (ha)	Grade 2 (ha)	Grade 3 (ha)	Grade 4 (ha)
26,100	850	7,950	7,260	10,0240

It is necessary to note from the above table that the drainage plays a very important role in developing this area.

VII.4.1 Present Drainage System

Before making the future drainage plan in the project area, the watershed map is prepared based on the 1/5,000 maps with countour interval of 1 m as shown in Fig. VII-4. The map indicates that the sub-areas B, D and E have fairly large catchment basin, whereas the sub-areas A and C, small.

Based on the above map, the present drainage system is schematically presented as shown in Fig. VII-5. The figures in the chart show the catchment area of each of the existing rivers and drains. The possibility of using these rivers and drains in the future plan will be discussed in the subsequent Annex IX in relation to the water level of the rivers to be drained.

VII.5 PLANS FOR DRAINAGE

VII.5.1 Selection of Drainage Method

The method of drainage is classified into three types: (1) gravity drainage, (2) pump drainage, (3) a combination of gravity and pump. When the water level in the area is higher than the water level outside for most of the time, the method (1) is adopted with sluice gate, flap gate or miter gate in the outfalls of drains which is closed when the outside stage is higher than inside.

In very low-lying areas, where water cannot be drained by gravity, the method (2) is adopted and is also used for lowering the groundwater table in the field. When the reclaimed land from the low-lying area is lower than the sea level or flood stage in the river during a certain period of time and the land has not enough storage capacity to keep the inside water level lower than the level required by only gravity drainage, the method (3) is adopted to achieve better drainage.

It is very common in planning the reclamation of a low-lying area to make investigation of the methods of drainage and ratio of gravity to pump drainage with relation to the benefits produced. Taking into account the higher operating costs of pump drainage than gravity, the selection of the project area in view of the drainage condition will be made in the subsequent section provided that the areas for irrigation farming are to be secured by gravity only.

VII.5.2 Criteria for Drainage Planning

Most of the project area is low-lying and flat, and is affected by both flood and tide. The water stage in the rivers Barito, Martapura and Maluka periodically becomes higher than the ground elevation of the project area. Under these conditions, submerged in the paddy fields during floods would be allowed to some extent in order to minimize the construction cost for drainage facilities. The allowable depth and duration of submergence in the paddy fields would have to be determined taking into account the degree of damage to paddy rice by submergence. Table VII-2 shows the relation between the yield reduction rate of paddy rice and depth and duration of submergence at different growing stages of paddy rice, which is prepared based on data obtained through the field investigation on crop damages by floods in Philippines.

From this table, the following considerations could be made :

(1) The submergence at the growing stage of young panicle formation gives the serious damage to the yield of paddy rice, on the contrary, damage due to submergence at the stage of maturing is insignificant.

(2) The duration of submergence within 1 to 2 days is not significant, but damage of paddy rice remarkably increases due to submergence beyond 2 days.

(3) When a part of leaves still remains above water surface, the damage to paddy rice is decreased as compared with that when leaves are completely submerged.

While, the midest rainy season in the project area occurs in the period between December and March. As seen the proposed cropping pattern shown in Figs. VI-10 and 12, the growing stage of paddy rice between middle stage of tillering and beginning stage of panicle formation would corresponds to the midest rainy season. The height of paddy rice at this stage would range from 25 cm to 60 cm.

Taking into account the above considerations, the following design criteria would be applied for making the future drainage plan in the Project.

(1) The allowable depth of submergence in the paddy fields should be 30 cm, and duration of submergence should not exceed 5 days.

(2) The submergence more than 30 cm in depth should not last more than 24 hours.

VII.6 DRAINAGE REQUIREMENTS

It is essential to maintain soil moisture in an adequate condition where the irrigation farming is practiced. If, on the other hand, the areas where drainage could not be made within a feasible range, these areas would have to be abandoned. This sub-section deals with the drainage requirements for the areas where the drainage is practiced economically.

The drainage requirements in the project area are estimated on the basis of the following assumptions and procedures :

(1) Rainfall data covering 17 years from 1960 to 1976 at Banjarbaru, 18 years from 1960 to 1977 at Syamsudin Noor and 15 years from 1962 to 1977, except for 1968, at Banjarmasin are used. The data of each station are applied to the following sub-areas :

Sub-area	Data
A	at Banjarbaru
B	at Syamsudin Noor
C	at Banjarmasin
D	at Banjarmasin
E	at Syamsudin Noor

(2) Design rainfall is estimated at 240.7 mm of 72-hour rainfall at Banjarbaru, 259.2 mm at Syamsudin Noor and 217.6 mm at Banjarmasin during the wet season paddy from December to March taking 10-year return period. Three days (72 hours) maximum in each year at the above three stations are tabulated in Table VII-3.

The probable rainfall is thus estimated as shown below.

Return period	Rainfall in mm		
	Banjarbaru	Syamsudin Noor	Banjarmasin
5 years	209.4	217.9	187.0
10 years	240.7	259.2	217.6

(3) Based on the average rainfall pattern, the distribution percentage of the design daily rainfall is estimated as follows (see Table VII-3) :

Date	Percentage		
	Banjarbaru	Syamsudin Noor	Banjarmasin
1st day	46	34	41
2nd day	29	29	30
3rd day	25	37	29

(4) Relationship between rainfall and runoff distribution is assumed as follows :

Relationship between cumulative rainfall and total runoff in lowland

Cumulative rainfall (mm)	10	10-30	30-50	50-100	100-300
Runoff coefficient (f)	0	0.1	0.3	0.5	0.8

Relationship between rainfall and runoff distribution

Rainfall (mm)	1st day (%)	2nd day (%)	3rd day (%)	4th day (%)
Less than 30	100			
30-50	70	30		
50-100	60	30	10	
More than 100	50	30	15	5

(5) Based on the above assumptions, the drainage requirements are estimated as shown below :

Design Drainage Requirement in Sub-area A

Design rainfall (mm)	Cumulative rainfall (mm)	f	Runoff (mm) 1st day	2nd day	3rd day	4th day	5th day
110.7	110.7	0.8	44.3	26.6	13.3	4.4	-
69.8	180.5	0.8	-	33.5	16.8	5.6	-
60.2	240.7	0.8	-	-	28.9	14.4	4.8
Total :			44.3	60.1	59.0	24.4	4.8
	lit/sec/ha		5.2	7.0	6.9	2.9	0.6

Design Drainage Requirement in Sub-area B and E

Design rainfall (mm)	Cumulative rainfall (mm)	f	Runoff (mm) 1st day	2nd day	3rd day	4th day	5th day
88.1	88.1	0.5	26.4	13.2	4.5	-	-
75.2	163.3	0.8	-	36.1	18.0	6.0	-
95.9	259.2	0.8	-	-	46.0	23.0	7.7
Total :			26.4	49.3	68.5	29.0	7.7
	lit/sec/ha		3.1	5.7	8.0	3.4	0.9

Design Drainage Requirement in Sub-area C and D

Design rainfall (mm)	Cumulative rainfall (mm)	f	Runoff (mm) 1st day	2nd day	3rd day	4th day	5th day
89.2	89.2	0.5	26.8	13.4	4.5	-	-
65.3	154.5	0.8	-	31.3	15.7	5.2	-
63.1	217.6	0.8	-	-	30.3	15.1	5.0
Total :			26.8	44.7	50.5	20.3	5.0
	lit/sec/ha		3.1	5.2	5.2	2.4	0.6

The drainage requirements, accordingly, are estimated at 7.0 lit/sec/ha in the sub-area A, 8.0 lit/sec/ha in the sub-areas B and E and 5.9 lit/sec/ha in the sub-areas C and D, which are used for the design of the drainage system in the project area.

VII.7 OUTSIDE WATER LEVEL (WATER LEVEL ALONG THE RIVERS)

VII.7.1 Available Data

There are three gauging stations along the Martapura and one in the Barito river estuary. Fig. VII-6 shows the location of them, i.e. Banjarmasin (station X), Sungai Tabuk (station Y), Martapura (station Z) and Barito (station U). Out of the above stations, stations X and Z made observation for a fairly long period (about two years), whereas stations Y and U were newly established. In analyzing the water level fluctuations, the data available at stations X and Z are mostly used. Since the data available at stations Y and U are so limited, they are used only for correlation with those at stations X and Z.

VII.7.2 Effects of Runoff on Water Level Fluctuation

It is stated in the previous section that the design discharge due to flood is estimated on the basis of the probable heavy rainfall which occurs once ten years. If the same probability is employed for the estimation of the water level on the rivers, the overall probability would become too small. Practically, therefore, the normal highest water level which takes place during the heavy rainy season is adopted as the design high water level.

First of all, the tidal effect on the upmost stream in the project area, at station Z, is examined. The regular daily fluctuation of the water level at station Z indicates that the water level outside, the river Martapura, is under the effect of tide.

Fig. VII-7 shown maximum, minimum and average water levels in each month using the data available from September, 1976 to July, 1978 at station Z. Since the data available are only two years, the highest value that took place in March, 1977 and the lowest value that was recorded in September, 1977 are employed. The fluctuation of the water level due to runoff is estimated at 2.6 m, because the average elevation of the former is El. 2.3 m and the latter, El.-0.3 m.

The same analysis is made for the data available at station X as shown in Fig. VII-8, which reveals that the daily maximum water level varies from El. 0.02 m (September, 1977) to El. 0.58 m (January, 1978). Therefore, the fluctuation of the water level due to runoff is estimated at 0.56 m.

As the data at station Y are too short to make the same procedure with the above, the proportional allotment is made based on the distance. Since the distance along the river course from station X to station Y and from station Y to station Z is 25.25 km and 24.55 km, respectively, the following calculation would be valid.

$$\frac{2.60\ \text{m} - 0.56\ \text{m}}{25.25 + 24.55\ \text{km}} \times 25.25\ \text{km} = 1.03\ \text{m}$$

There are no available data to estimate the seasonal fluctuation in the Barito at station U so far. The data available at station X are used, taking into account the smaller seasonal variation due to runoff, which assumed to be 0.44 m (the average difference of water level between the rainy season and the dry season at station X).

VII.7.3 Tidal Effects on Water Level Fluctuation

Table VII-4 shows the daily variation of the water level at stations X and Z in January, 1978. The table indicates that average fluctuations due to tidal effect are 1.41 m at station X and 0.15 m at station Z.

As the data on the daily fluctuations at station Y are not available in the flood season, the simultaneous observation among station X, Y and Z is made in September, 1978 as shown in Fig. VII-9.

The figure indicates that the daily fluctuation at station X was 1.64 m, whereas 1.36 m at station Y. Provided that this ratio is applicable in the flood season, the daily fluctuation at station Y due to tidal effect would be :

$$\frac{1.36\ \text{m}}{1.64\ \text{m}} \times 1.41 = 1.17\ \text{m}$$

VII.7.4 Standardization of Tidal Range

It is necessary to standardize the tidal cycle to determine the drainage canal capacity. For this, data recorded at station X in January, 1978 are used. Fig. VII-10 shows a part of actual tidal record observed at Banjarmasin port. As seen in this Figure, though the two cycles of peak tide and low tide are observed within 24 hours, the difference in elevation between the peak tide and low tide in one cycle is very small as compared with that in another cycle. For the study on drainage improvement, it would be practical that only one tide cycle (one peak and one bottom) will occur within 24 hours.

Fig. VII-11 shows the monthly mean tidal cycle in the said month. The upper diagram illustrates the relationship between the time and the water level. The lower diagram illustrates the duration of time between the two same water levels from the ebb tide to the flood tide. Superimposed in the diagram at station X is the same diagram at station Y. The latter is modified from the former assuming that the ratio of amplitude is 1.41 m to 1.17 m.

The standardization of tidal range is made as shown in Fig. VII-12 and VII-13 for stations X and Y, respectively, using the information obtained from Fig. VII-11.

Since there are no data available at station U in the flood season, a comparative study between station X and U is made using the data so far available. The result indicates that there is clear trend in tidal ranges between the two stations in terms of the amount of runoff from the upper streams. In the driest season, when the runoff is very little, the tidal range between the two stations is more or less the same, whilst as the amount of runoff increases the tidal range at station U becomes bigger than that at station X. In the heaviest rainy season, it is expected that the tidal range at station U would be bigger than that at station X by 1.29 times. Through the same procedure as stated in Fig. VII-11, the standardization of tidal range is made as shown in Fig. VII-14.

VII.7.5 Longitudinal Profile of Water Level along the River Martapura

The longitudinal profile of the water level along the Martapura, taking into account the effects of flood and tide, is worked out as shown in Fig. VII-15 as the summary of discussions in this section. Based on the figure, the classification of the area in terms of drainability will be made in the following section.

VII.7.6 Water Level along the River Maluka

The river Maluka is characterized by three distinct features. Out of the total length of 70 km, the upper stretch (until the crossing point with the road connecting Batibati with Banjarbaru) with a length of 22 km is characterized by little flow and the river is not flooded even in the heaviest rainy season owing to its relatively large capacity, the middle stretch (until the confluence with the river Panggaaungan) with a length of 34.4 km is characterized by very gentle slope with small capacity and the surrounding areas are more or less submerged throughout the year, and the lower stretch with a length of 13.6 km is characterized by large daily water level fluctuations due to the tidal effect.

The drainage of the area to be developed in the sub-area E is practiced in relation to the water level at the site of proposed drainage canal outlet in the said lower stretch (See Drawing No. 1). The river water fluctuation at the outlet, which is planned to be located at about 2 km upstream from the estuary, is regarded to be approximately the same with the sea tide fluctuation in the dry season judging from the observation data at Kurau which is shown in Fig. VII-11. As for the water level fluctuation in the rainy season there is no data at the outlet site. The high water level is assumed to be El. 0.5 m through interview with the local people.

VII.8 DRAINABILITY

The previous sections show that the drainability of the project area would be affected by both flood and tide. Based on the criteria for drainage planning discussed in Section VII-5, the study on drainability in each sub-area is made taking into account the relation among the fluctuation of outside and inside water levels, and drainage discharge. As a result, the improved drainability of land is classified into the following four categories :

Category 1 : Good. The ground level is higher than the water level outside throughout the year. No measures are needed in the outfalls of the drains.

Category 2 : Favourable. The submergence more than 30 cm does not last more than 24 hours. Miter or flap gates are required in the outfalls of drains.

Category 3 : Poor. In the rainy season, the submergence more than 30 cm lasts more than 24 hours. But in the dry season, the submergence does not occur.

Category 4 : Bad. No gravity drainage can be made throughout the year.

In the previous sections, which deal with the present flood conditions, the discussion is made based on the total catchment area. However, in this section, the classification of the land will be made based on the areas which are effectively irrigable through the main irrigation canal, as the alignment of it has already been settled by the economical comparative studies.

Sub-area A

The sub-area A would be divided into two areas, sub-area A_1 and sub-area A_2, since the area is separated in terms of drainage by a road running from east to west as stated in the previous section. The drainability of part of the sub-area A_1 is poor due to the depression in the central area. Whilst, the drainability of the most of the sub-area A_2 is very good.

Each of the sub-areas is classified as follows :

	Total area (ha)	Category 1 (ha)	Category 2 (ha)	Category 3 (ha)	Category 4 (ha)
Sub-area A_1	1,550	900	250	400	-
Sub-area A_2	2,110	2,060	50	-	-
Total area	3,660	2,960	300	400	-

The above table shows that drainability of about 400 ha which are located in the central part of the sub-area A is poor, and no perfect drainage could be expected during the heavy rainy season by gravity from January to March. The submerging depth ranges from 30 cm to 60 cm depending on the ground elevation. This indicates that the rainy season paddy (improved variety) could not be introduced. However, it is recommended that this area would be included in the project area, because it is located in the center of the sub-area A_1. The alignment of drains would be made to save this area in the dry season.

Sub-area B

The sub-area B would be divided into four areas in view of the location and the alignment of drains to be planned. The sub-area B_1 is the small area located between the Martapura and the road which connects Banjarmasin and Martapura cities running along the river Martapura. The sub-area B_2 is located along the Martapura road. The area is composed of present paddy fields and shrub lands.

The sub-area B_3 which is located in the central part consists of swamp land of peat with trees and grasses. The sub-area B_4 is high land, which is located in the south and consists of plantation of rubber and wild land of alang-alang.

Each of the areas is classified in view of drainability as follows :

	Total area (ha)	Category 1 (ha)	Category 2 (ha)	Category 3 (ha)	Category 4 (ha)
Sub-area B_1	810	200	-	610	-
Sub-area B_2	8,370	3,380	4,800	190	-
Sub-area B_3	5,050	-	-	-	5,050
Sub-area B_4	4,150	4,150	-	-	-
Total area	18,380	7,730	4,800	800	5,050

The above table indicates that the sub-area B_3 with 5,050 ha could not be drained by gravity. Therefore, the area is to be excluded from the Project in view of poor drainability. Drainability of 800 ha in the sub-area B_1 and B_2 is poor, and no perfect drainage could be expected by gravity during the heavy rainy season from January to March. The submerging depth varies between 30 cm to 60 cm depending on the ground elevation. This indicates that the rainy season paddy (improved variety) could not be introduced. But, it is proposed to include in the project area, because these lands could be used for cultivation of paddy rice during the dry season. Since the remaining areas of 12,530 ha could be favourably drained, these areas could be selected as the irrigable area, provided that the other conditions such as soils, land use, topography, etc. permit.

Sub-area C

The sub-area C would be divided into two areas in terms of the present land use. The majority of the area belongs to the sub-area C_1 which is well developed as paddy field. The sub-area C_2 which is located near Banjarmasin consists of coconut plantation.

Each of the areas is classified in view of drainability as follows :

	Total area (ha)	Category 1 (ha)	Category 2 (ha)	Category 3 (ha)	Category 4 (ha)
Sub-area C_1	4,400	1,690	2,710	-	-
Sub-area C_2	920	-	920	-	-
Total area	5,320	1,690	3,630	-	-

The above table indicates that the drainability of the whole area of the sub-area C is favourable. Double cropping of paddy rice could be expected throughout the area, since the submergence more than 30 cm in depth does not last more than 24 hours even in the heaviest rainy season with sluice, flap or miter gates at the outfalls of the drains.

Sub-area D

The sub-area D could be divided into four areas based on the drainage system. The sub-area D_1 is located on the north-east of the area and is encircled by roads. The direction of drain is toward the river Martapura. The sub-area D_2 occupies the north-west of the area and is bounded by a road on the east and the south. The direction of flow is toward the river Barito. The sub-area D_3 is situated in the south of the area and is separated from the sub-areas D_1 and D_2 by a road located at high land. The sub-area D_4 is located on the east of the area and is separated from the sub-areas D_1 and D_3.

Each of the areas is classified in view of drainability as follows :

	Total area (ha)	Category 1 (ha)	Category 2 (ha)	Category 3 (ha)	Category 4 (ha)
Sub-area D_1	4,990	1,860	2,640	490	-
Sub-area D_2	7,440	-	3,430	4,010	-
Sub-area D_3	3,850	340	3,510	-	-
Sub-area D_4	1,770	1,770	-	-	-
Total area	18,050	3,970	9,580	4,500	-

The above table indicates that the drainability of the area of 4,500 ha situated along the Barito is very poor. Of course it is not impossible to make drainage even in the rainy season, since the tidal fluctuation is about 1.8 m. For this, however, a fairly large scale polder would be necessary to protect the tidal effect.

As discussed in the previous section, since the amount of water available from the river Riam Kanan is limited, it is suggested to exclude this area from the Project in view of drainability also.

Sub-area E

The sub-area E could be divided into two areas based on the drainage system. The sub-area E_1 occupies the northern half of the area. The direction of drain is toward the river Barito.

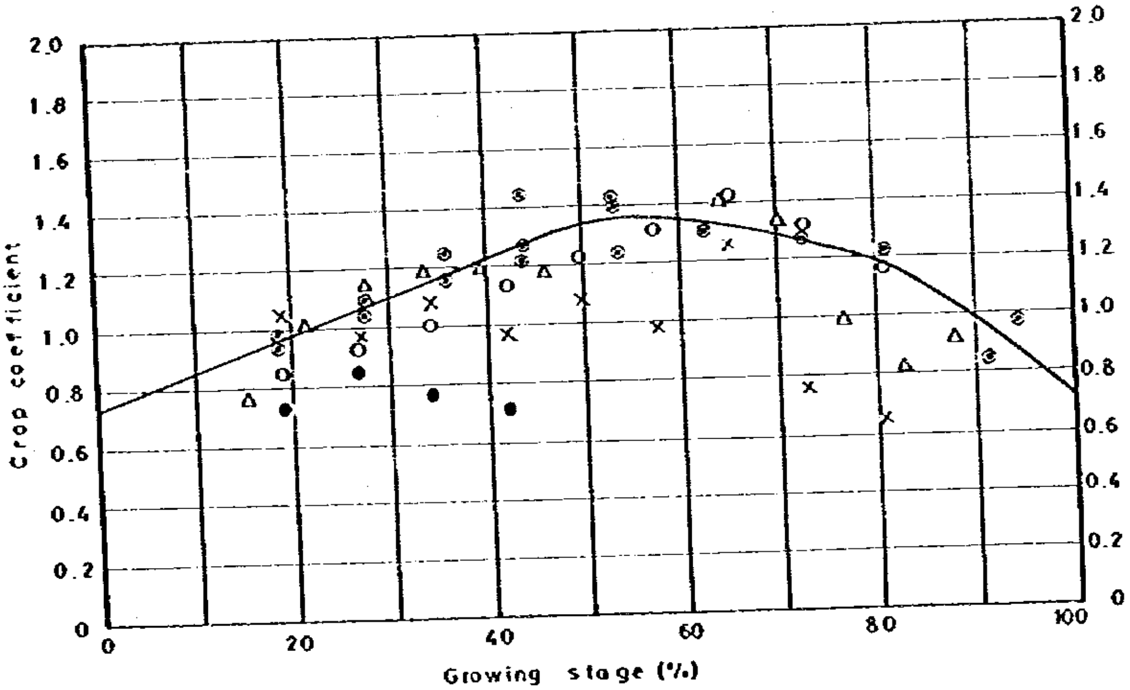
The sub-area E_2 is located on the southern half of the area. The direction of flow is toward the river Maluka.

Each of the areas is classified in view of drainability as follows :

	Total area (ha)	Category 1 (ha)	Category 2 (ha)	Category 3 (ha)	Category 4 (ha)
Sub-area E_1	6,130	1,200	2,880	2,050	-
Sub-area E_2	8,620	1,790	4,330	2,500	-
Total area	14,750	2,990	7,210	4,550	-

The drainability of the area is similar to the sub-area D. The above table indicates that the land with an area of 4,550 ha which is located along the rivers Barito and Maluka could not be drained properly. It is also not impossible to make drain excess water from these lands as stated above. The final decision whether or not to include these areas in the Project is subject to other restrictions such as soils, amount of water available, etc.

Fig VII-1 Crop Coefficient Curve for Paddy



⊚ Observed in the Project Area (dry season)

× Observed at Cintamanis (wet season)

• Observed at Cintamanis (dry season)

○ Observed at Belitang (wet season)

Δ Observed at Belitang (dry season)

Table VII-1 Irrigation Water Requirement for Newly Introduced Varieties

Description	Nov.		Dec.			Jan.			Feb.			Mar.			Apr.			May		
Schematic cropping pattern			Nursery stage						135 to 140 - day variety IR-26, IR-34 etc.						Ripening stage					
A. Consumptive water requirements																				
(1) Crop coefficient, kc			0.84	0.95	1.06	1.17	1.23	1.34	1.37	1.27	1.14	0.96	0.67							
				0.84	0.95	1.06	1.17	1.23	1.34	1.37	1.27	1.14	0.96	0.67						
					0.84	0.95	1.06	1.17	1.23	1.34	1.37	1.27	1.14	0.96	0.67					
						0.84	0.95	1.06	1.17	1.23	1.34	1.37	1.27	1.14	0.96	0.67				
							0.84	0.95	1.06	1.17	1.23	1.34	1.37	1.27	1.14	0.96	0.67			
								0.84	0.95	1.06	1.17	1.23	1.34	1.37	1.27	1.14	0.96	0.67		
(2) Average, kc			0.84	0.90	0.95	1.01	1.05	1.10	1.19	1.24	1.25	1.22	1.13	1.08	1.01	0.92	0.82	0.67		
(3) Pan evaporation (Class-A pan), mm/day			3.5	3.5	3.5	3.7	3.7	3.7	3.5	3.5	3.5	4.2	4.2	4.2	4.1	4.1	4.1	3.9		
(4) Consumptive water requirement, mm/day			2.9	3.2	3.3	3.7	3.9	4.1	4.4	4.6	4.6	5.1	4.7	4.5	4.1	3.8	3.4	2.6		
B. Percolation, mm/day			0.5	0.5	0.5	0.5	0.5	0.5	0.5	0.5	0.5	0.5	0.5	0.5	0.5	0.5	0.5	0.5		
C. Washing water, mm/day			2.0	2.0	2.0	2.0	2.0	2.0	2.0	2.0	2.0	2.0	2.0	2.0	2.0	2.0	2.0	2.0		
D. Effective rainfall, mm/day			5.9	6.9	8.0	8.9	7.6	7.5	6.9	6.6	5.4	6.0	7.5	7.4	4.9	3.8	4.6	4.2		
E. (A)+(B)+(C)-(D), mm/day			-	-	-	-	-	-	-	0.5	1.7	1.6	-	-	1.9	2.5	1.3	0.9		
F. Crop intensity to total area			1/6	2/6	3/6	4/6	5/6	1	1	1	1	1	1	5/6	4/6	3/6	2/6	1/6		
G. Puddling water, 120 mm		2.0	2.0	2.0	2.0	2.0	2.0	-	-	-	-	-	-	-	-	-	-	-		
H. Net irrigation requirement (E)x(F)+(G), mm/day		2.0	2.0	2.0	2.0	2.0	2.0	-	-	0.5	1.7	1.6	-	-	1.3	1.3	0.4	0.2		
I. Irrigation water requirement(H) / E /1 mm/day		2.9	2.9	2.9	2.9	2.9	2.9	-	-	0.7	2.5	2.4	-	-	1.9	1.9	0.6	0.3		
J. Diversion requirement /2 m3/sec		8.1	8.1	8.1	8.1	8.1	8.1	-	-	1.9	7.0	6.7	-	-	5.3	5.3	1.7	0.8		

Description	Jun.			Jul.			Aug.			Sept.			Oct.			Nov.		
Schematic cropping pattern		Nursery stage					120 - day variety, IR-30, IR-28, etc.						Ripening stage					
A. Consumptive water requirements																		
(1) Crop coefficient, kc		0.86	0.99	1.14	1.27	1.34	1.37	1.21	1.01	0.67								
			0.86	0.99	1.14	1.27	1.34	1.37	1.21	1.01	0.67							
				0.86	0.99	1.14	1.27	1.34	1.37	1.21	1.01	0.67						
					0.86	0.99	1.14	1.27	1.34	1.37	1.21	1.01	0.67					
						0.86	0.99	1.14	1.27	1.34	1.37	1.21	1.01	0.67				
							0.86	0.99	1.14	1.27	1.34	1.37	1.21	1.01	0.67			
(2) Average, kc		0.86	0.93	1.00	1.07	1.12	1.16	1.22	1.22	1.15	1.12	1.07	0.96	0.84	0.67			
(3) Pan evaporation (Class-A pan), mm/day		3.8	3.6	4.5	4.5	4.5	5.1	5.1	5.1	5.5	5.5	5.5	5.4	5.4	5.4			
(4) Consumptive water requirement, mm/day		3.3	3.5	4.5	4.8	5.0	5.9	6.2	6.2	6.3	6.2	5.9	5.2	4.5	3.6			
B. Percolation, mm/day		1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0			
C. Washing water, mm/day		2.0	2.0	2.0	2.0	2.0	2.0	2.0	2.0	2.0	2.0	2.0	2.0	2.0	2.0			
D. Effective rainfall, mm/day		2.7	3.1	1.5	2.8	2.7	1.1	1.2	2.0	2.6	2.7	2.9	3.2	1.9	3.4			
E. (A)+(B)+(C)-(D), mm/day		3.6	3.4	6.0	5.0	5.3	7.8	8.0	7.2	6.7	6.5	6.0	5.0	5.6	3.2			
F. Crop intensity to total area		1/6	2/6	3/6	4/6	5/6	1	1	1	1	5/6	4/6	3/6	2/6	1/6			
G. Puddling water, 120 mm	2.0	2.0	2.0	2.0	2.0	2.0	-	-	-	-	-	-	-	-	-			
H. Net irrigation requirement (E)x(F)+(G), mm/day	2.0	2.6	3.1	5.0	5.3	6.4	7.8	8.0	7.2	6.7	5.4	4.0	2.5	1.9	0.5			
I. Irrigation water requirement(H) / E /1 mm/day	2.9	3.8	4.6	7.4	7.8	9.4	11.5	11.8	10.6	9.9	7.9	5.9	3.7	2.8	0.7			
J. Diversion requirement /2 m3/sec	8.2	10.8	13.1	21.0	22.1	26.7	32.6	33.5	30.1	28.1	22.4	16.8	10.5	7.9	2.0			

/1 : Irrigation efficiency, 68%

/2 : Irrigation area

Rainy season paddy : 24,020 ha

Dry season paddy : 24,[illegible]0 ha

Fig. VII-2 Present Drainage Condition

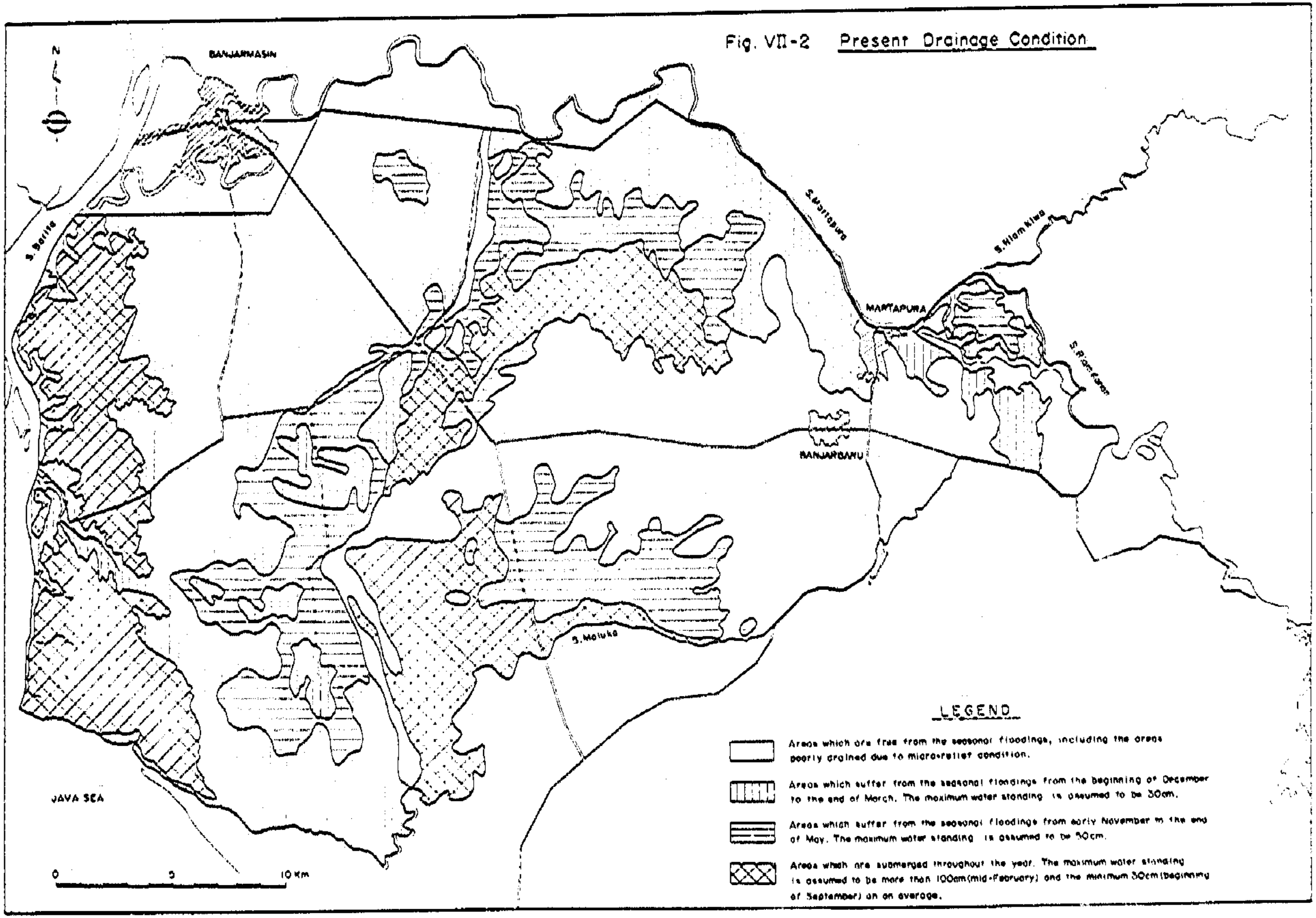


Fig. VII-3 Relationship among Rainfall minus Evapotranspiration, Submergence and Runoff in Sub-area B

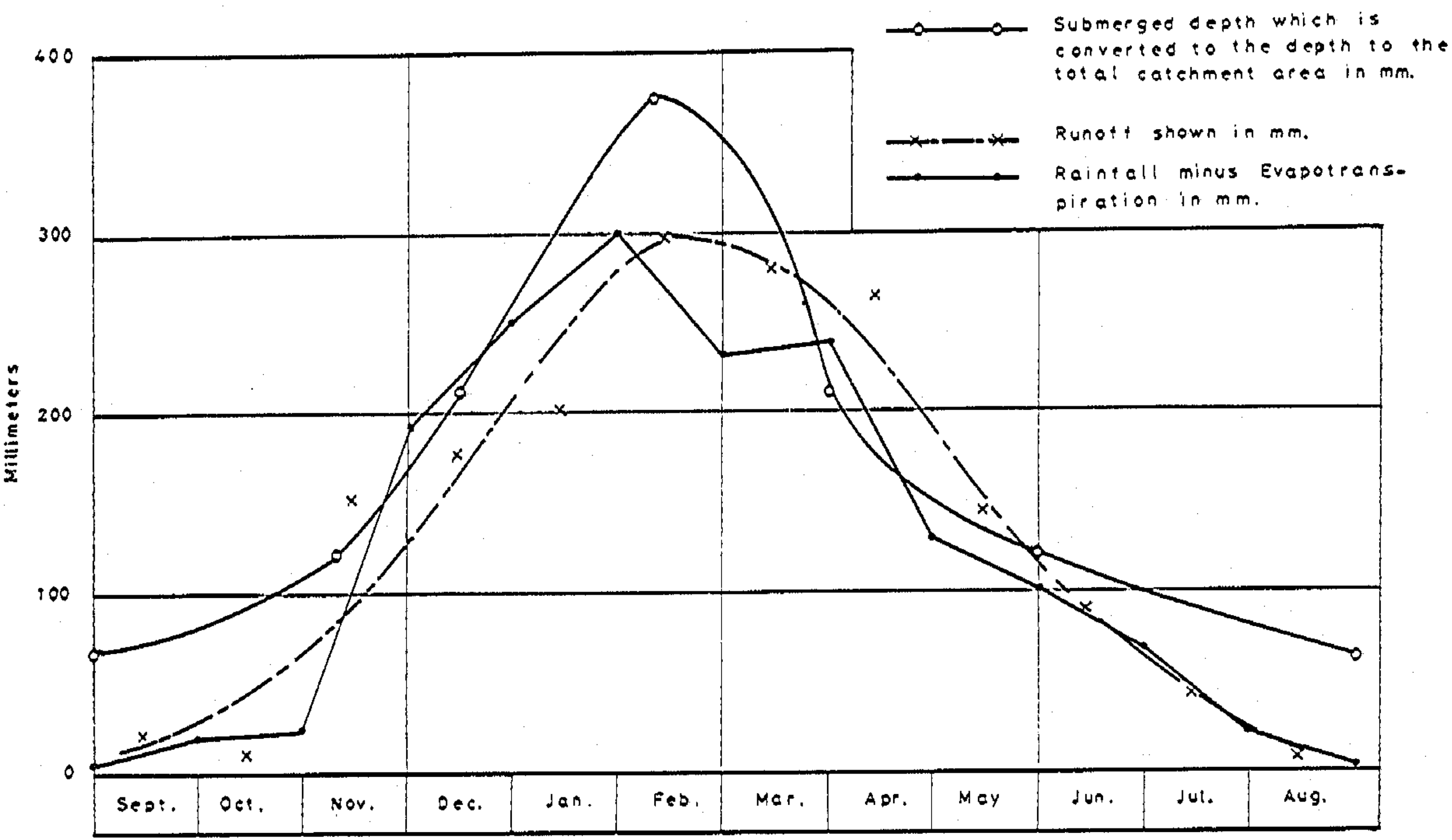


Fig. VII-4 Watershed Map (ha)

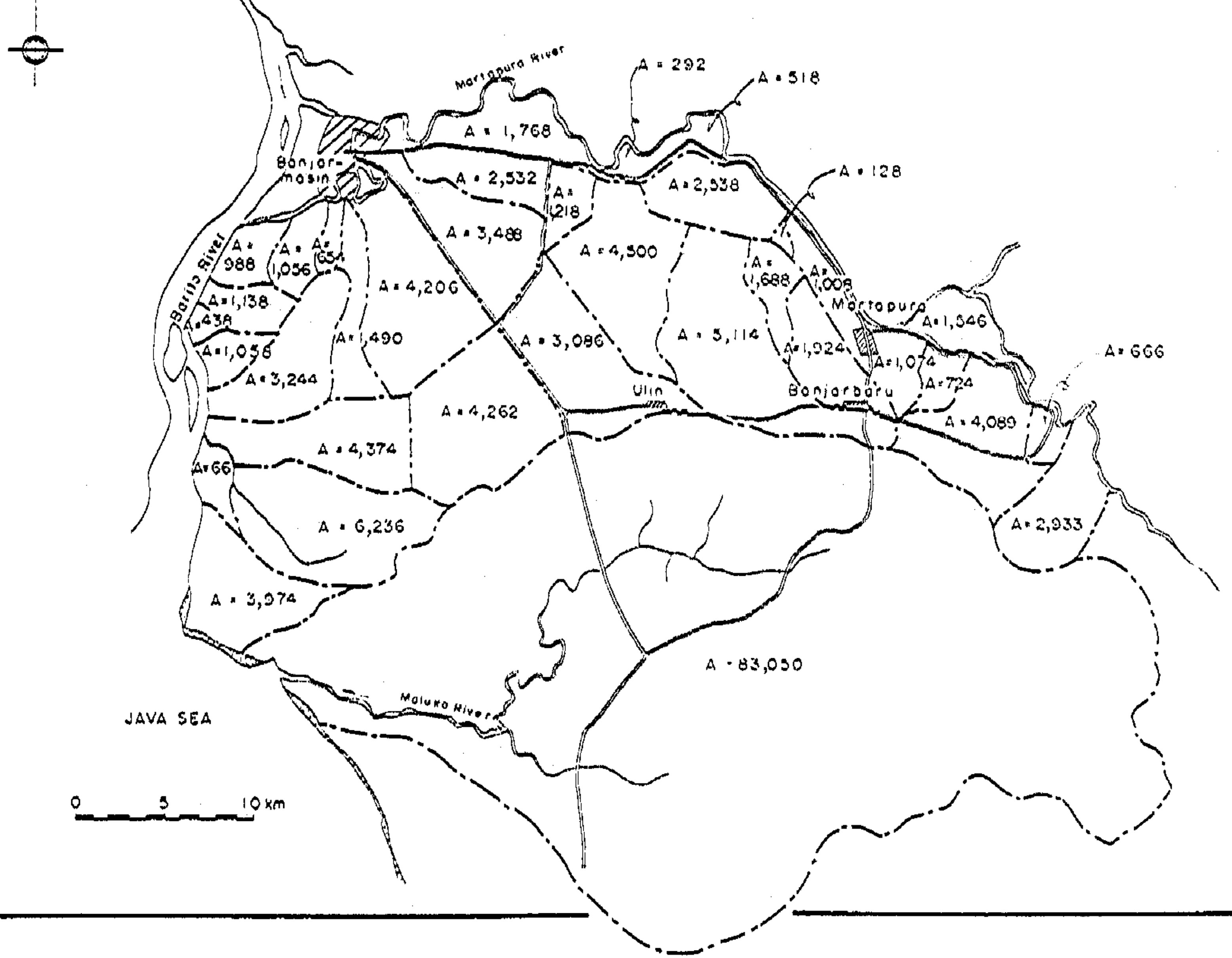


Fig. VII-5 Present Drainage System

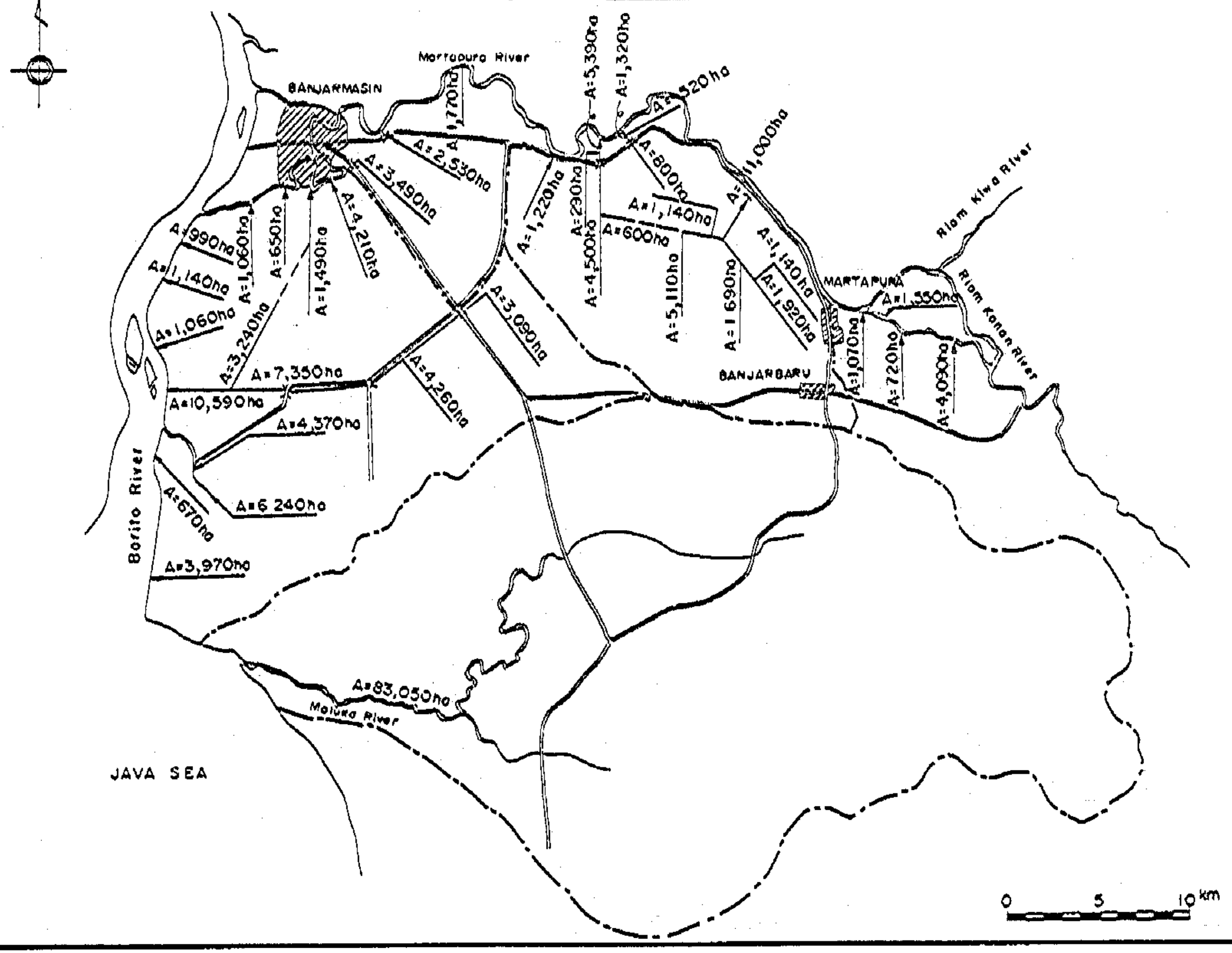


Table VII-2 Yield Reduction Rate (%)

Growing stage	Depth of submergence (cm)	Yield reduction of different duration of submergence			
		1-2 days	3-4 days	5-7 days	7 days
		Clean water			
20 days after transplanting	25-30	10	20	30	35
Young panicle formation partly inundated	45-60	10	30	65	95-100
Young panicle formation completely inundated	50-65	25	45	80	90-100
Head stage	75-90	15	25	30	70
Ripening stage	100	0	15	20	20
		Muddy water			
Yound panicle formation partly /1 inundated	45-60	20	50	85	90-100
Young panicle formation completely inundated	50-65	70	80	85	90-100
Heading stage	75-90	30	80	90	90-100
Ripening stage	100	5	10	30	30

/1 : "Partly" means that leaves (9-15 cm in length) remain above water surface.

Table VII-3 Three Days Maximum Rainfall (mm)

Year	Banjargaru				Syamsudin Noor				Banjarmasin			
	1st Day	2nd Day	3rd Day	Total	1st Day	2nd Day	3rd Day	Total	1st Day	2nd Day	3rd Day	Total
1960	36	78	0	114	46	60	53.5	159.5	-	-	-	-
1961	74	77	24	175	56	80	14.1	150.1	-	-	-	-
1962	137	22	35	194	44	18	51	113	39	117	25	181
1963	2	163	10	175	72	12	158	242	127	135	8	270
1964	2	130	40	172	75	46.2	12	133.2	106	17	23	146
1965	44	0	88	132	63	28	45	136	78	26	26	130
1966	53	0	104	157	30	34.2	85	149.2	100	5	15	120
1967	118	0	31	149	26	14	297.5	337.5	68	1	63	132
1968	112	27	144	283	19	137	7	163	-	-	-	-
1969	110	0	68	178	75	16	36	127	101	0	37	138
1970	77	30	73	180	25	78	87	190	35	48	25	108
1971	148	26	20	194	128	95	32	255	-	-	-	-
1972	87	0	5	92	56	10	43	109	52	14	30	96
1973	35	92	19	146	64	5.2	72	141.2	2	3	175	180
1974	38	58	12	108	85.3	15.9	17	118.2	17	7	100	124
1975	142	71	12	225	19	68	44	131	34	85	19	138
1976	95	44	25	164	50	53	6	109	51	90	10	151
1977	-	-	-	-	52	69.4	32.1	153.5	23	60	30	113
	(46%)	(29%)	(25%)	(100%)	(34%)	(29%)	(37%)	(100%)	(41%)	(30%)	(29%)	(100%)

Fig. VII-6 General Map Showing the Location of Gauging Stations

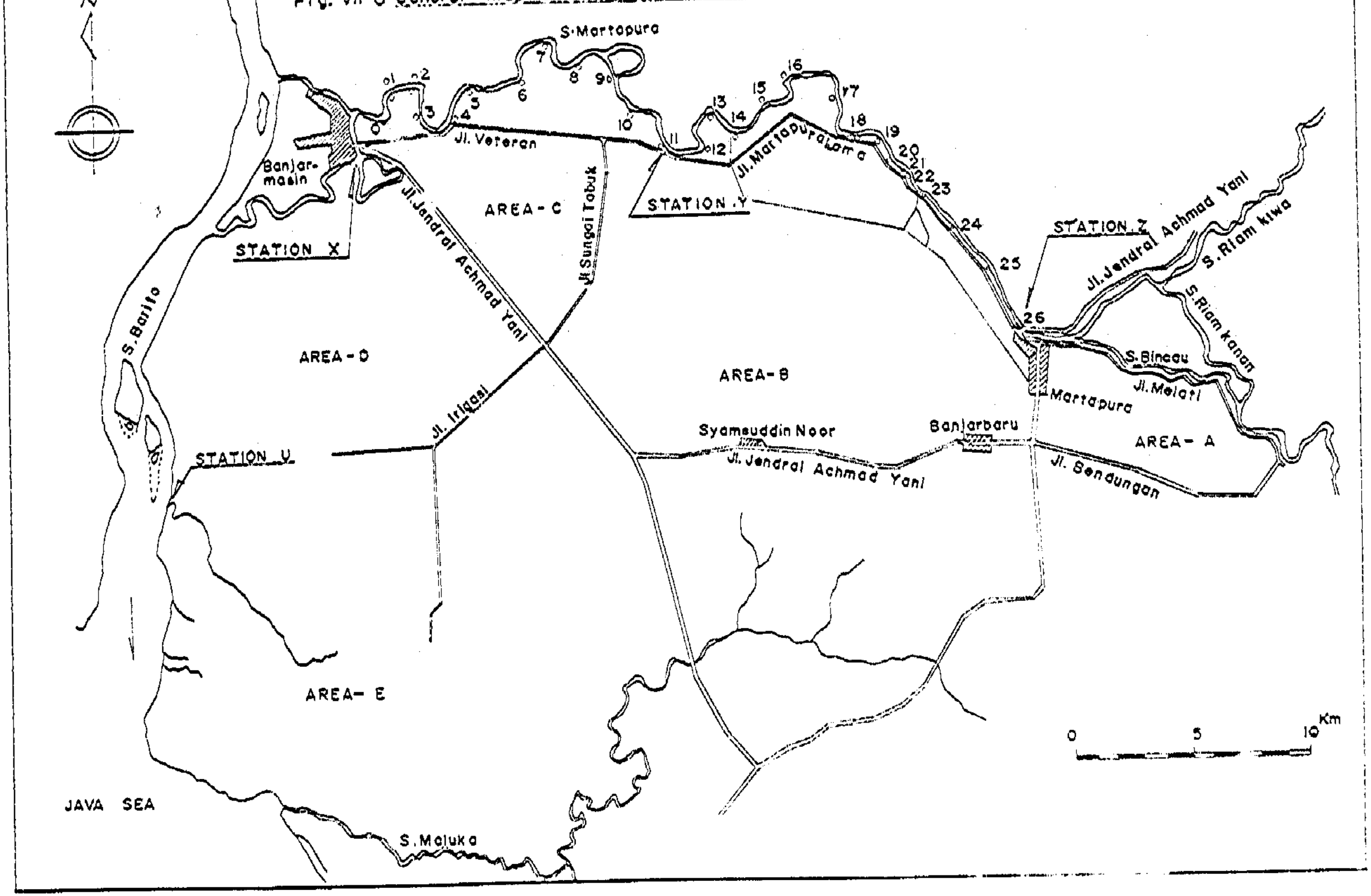


Fig. VII-7 Water Level in the Martapura River, Station Z

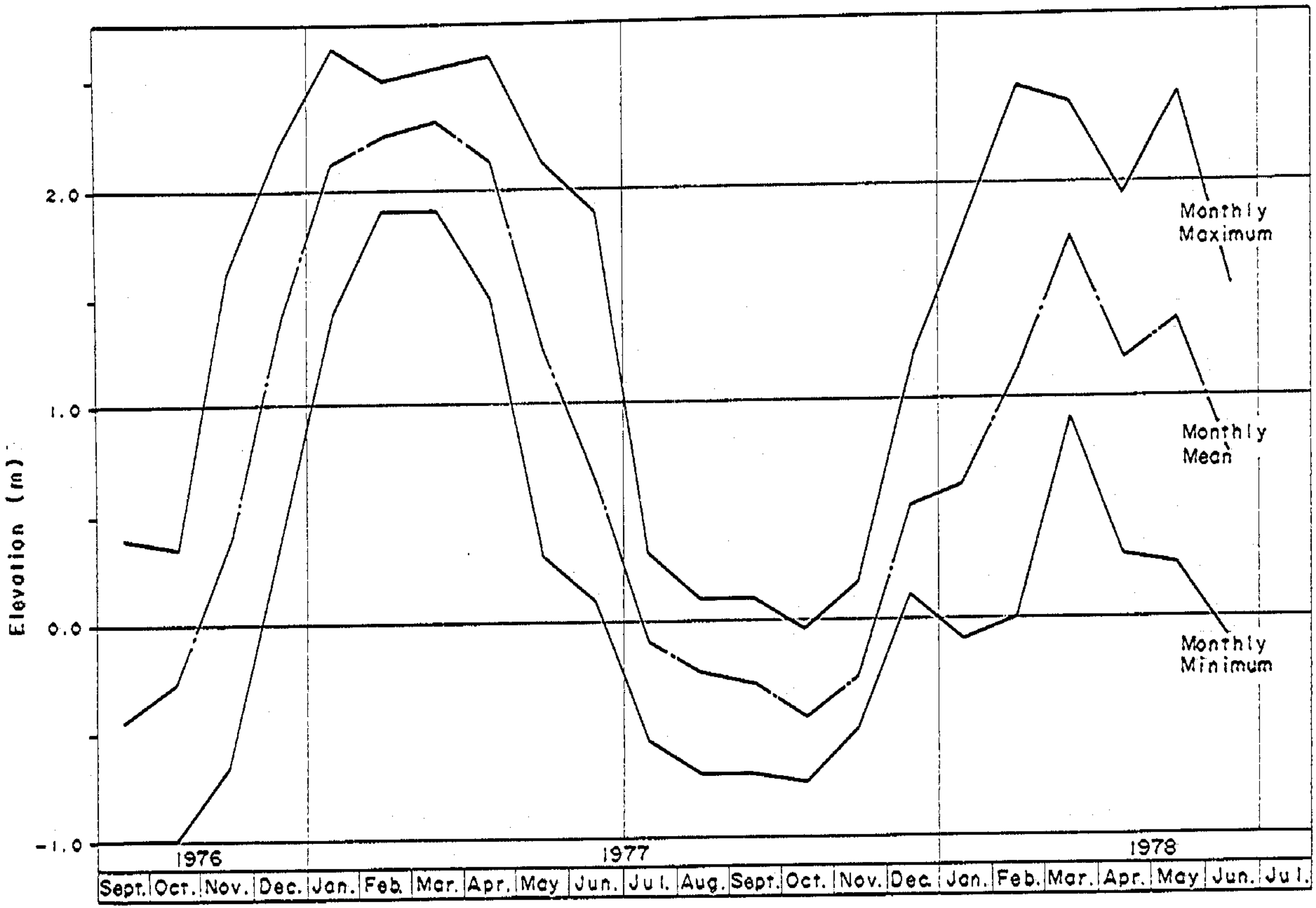


Fig. VII-8 Water Level in the Martapura River, Station X

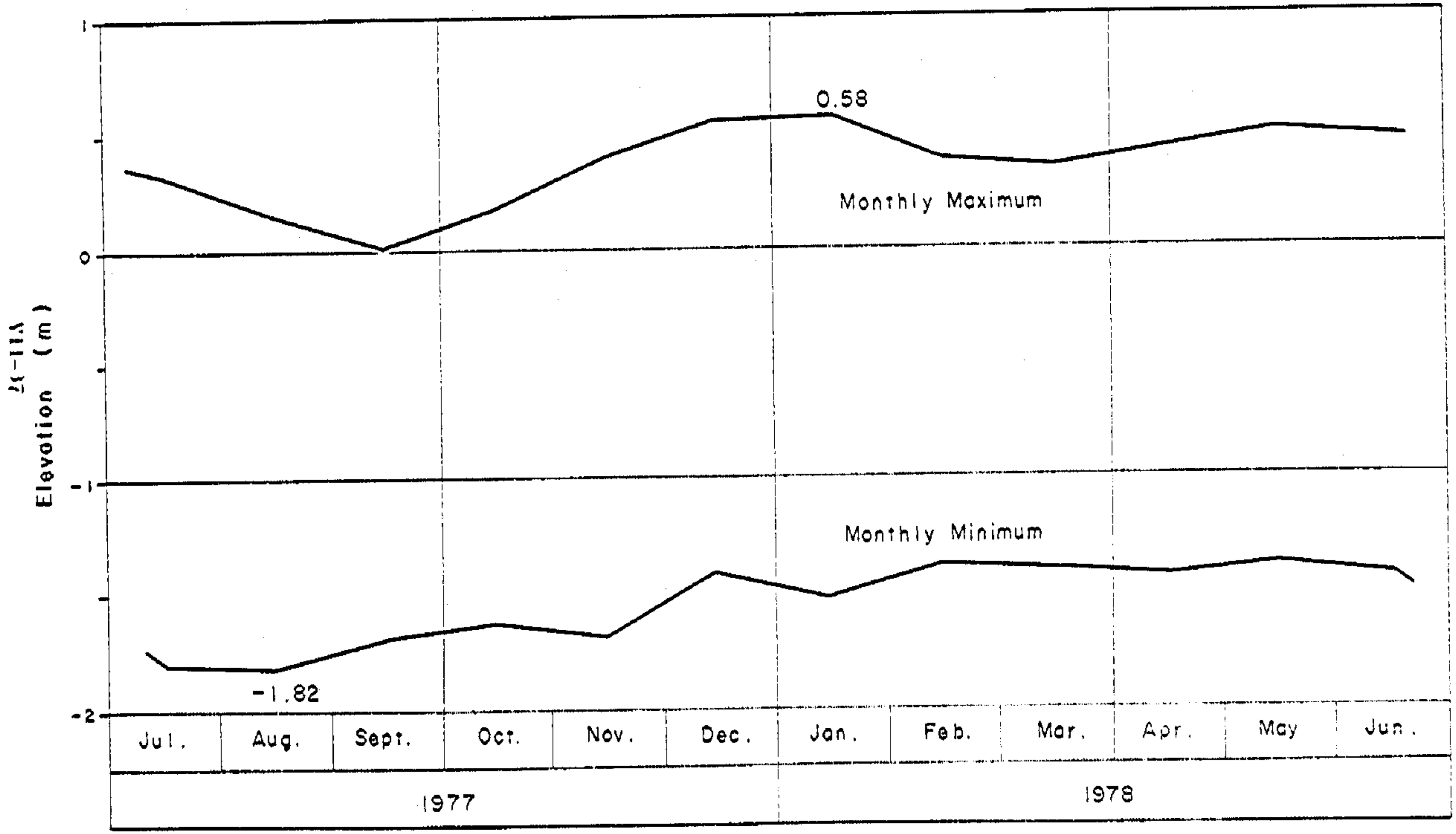


Table VII-4 Daily Variation of Water Level at Stations X and Z in January, 1978

Day	Martapura (Station Z)	Banjarmasin (Station X)
1	0.05	1.34
2	0.05	1.19
3	0.05	1.25
4	0.05	1.23
5	0.10	1.47
6	0.40	1.53
7	0.10	1.26
8	0.25	1.74
9	0.20	1.91
10	0.10	1.76
11	0.20	1.84
12	0.10	1.67
13	0.20	1.64
14	0.15	1.45
15	0.20	1.29
16	0.15	1.21
17	0.10	1.27
18	0.20	1.37
19	0.30	1.32
20	0.10	1.43
21	0.10	1.40
22	0.35	1.50
23	0.10	1.60
24	0.05	1.50
25	0.30	1.46
26	0.10	1.51
27	0.05	1.48
28	0.10	1.34
29	0.10	1.24
30	0.10	0.89
31	0.15	0.95
Total :	4.550	43.840
Average :	0.147	1.414

Fig. VII-9 Relationship of Water Level Fluctuations

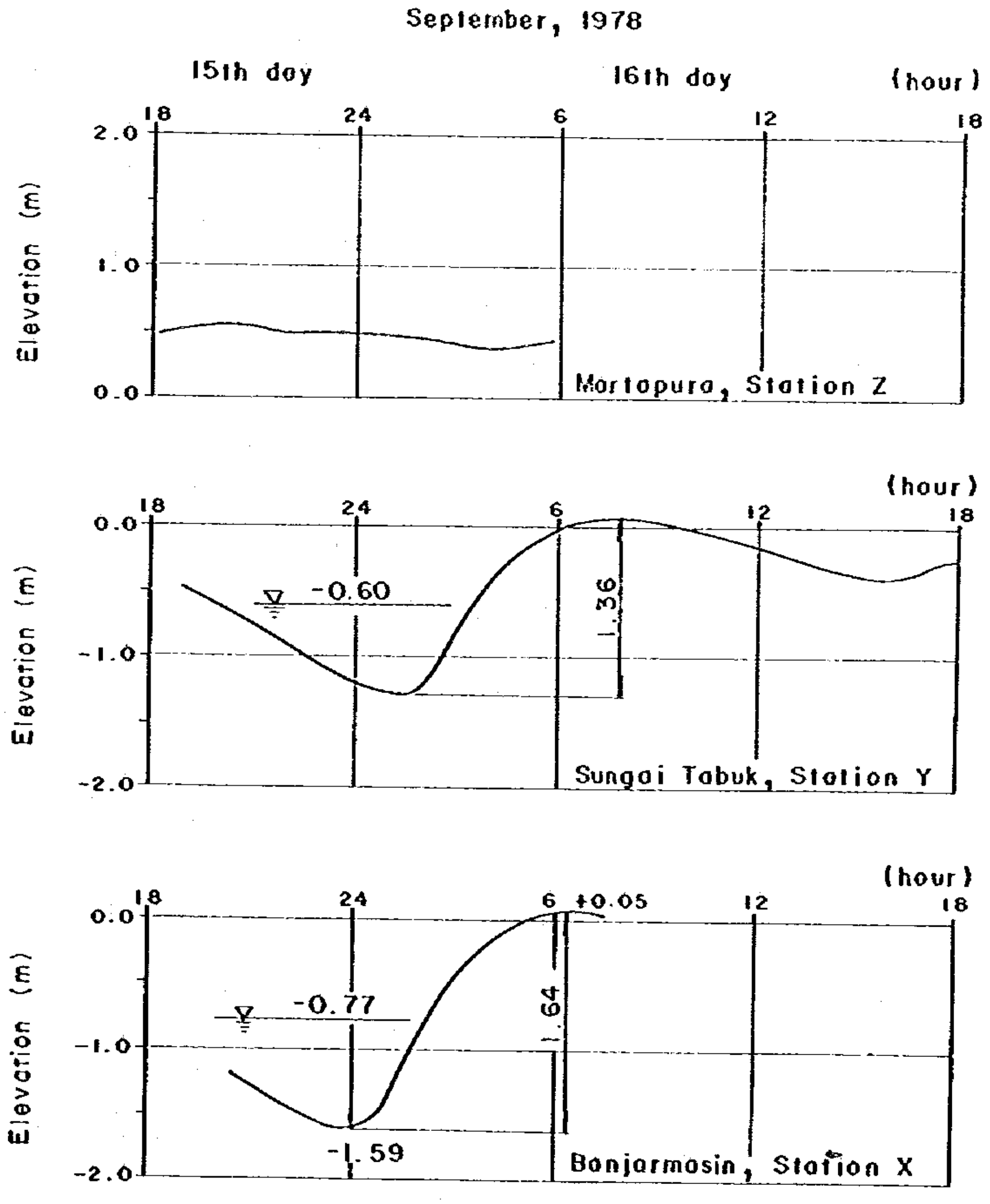


Fig. VII-10 Tidal Record at Banjarmasin Port

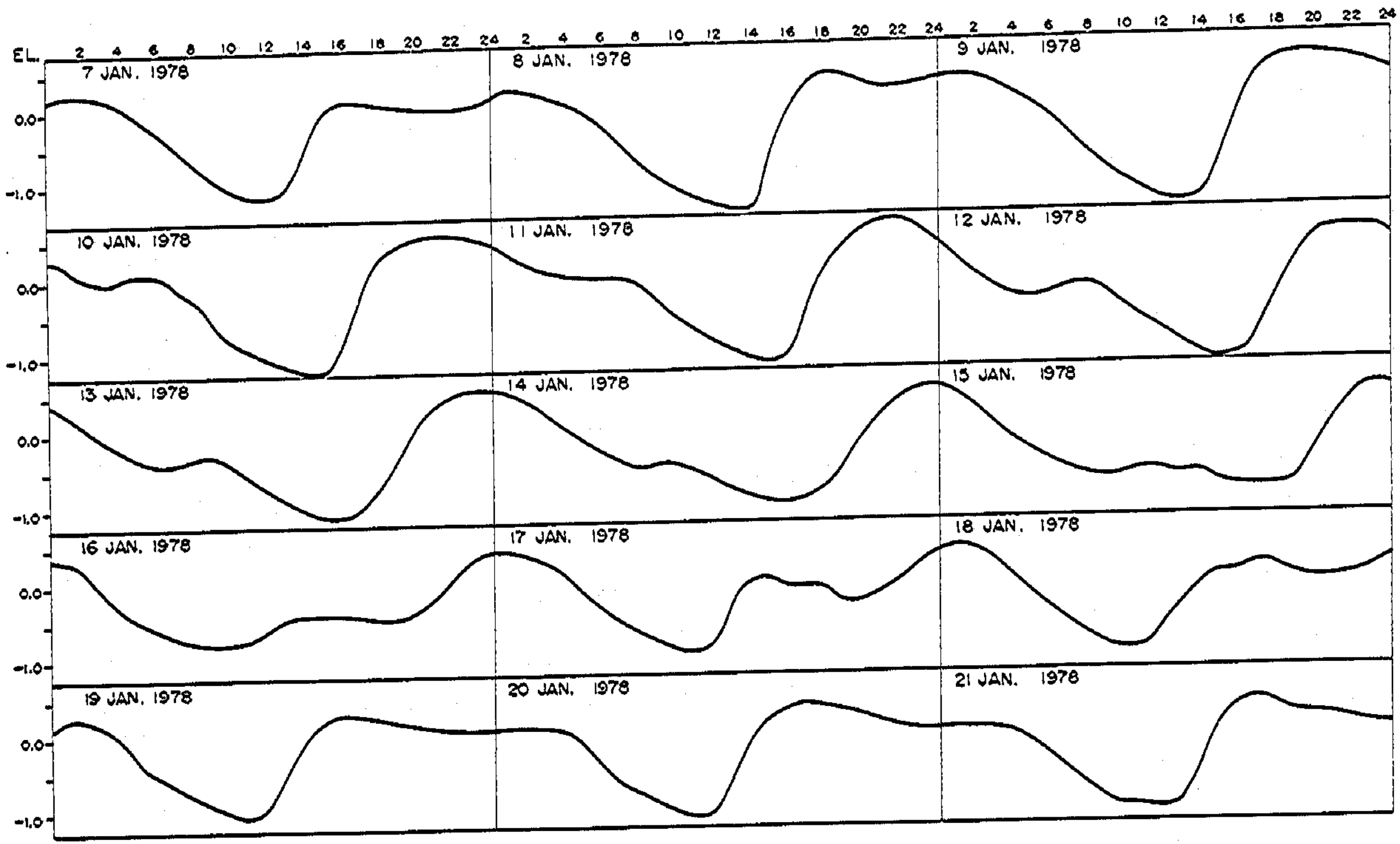
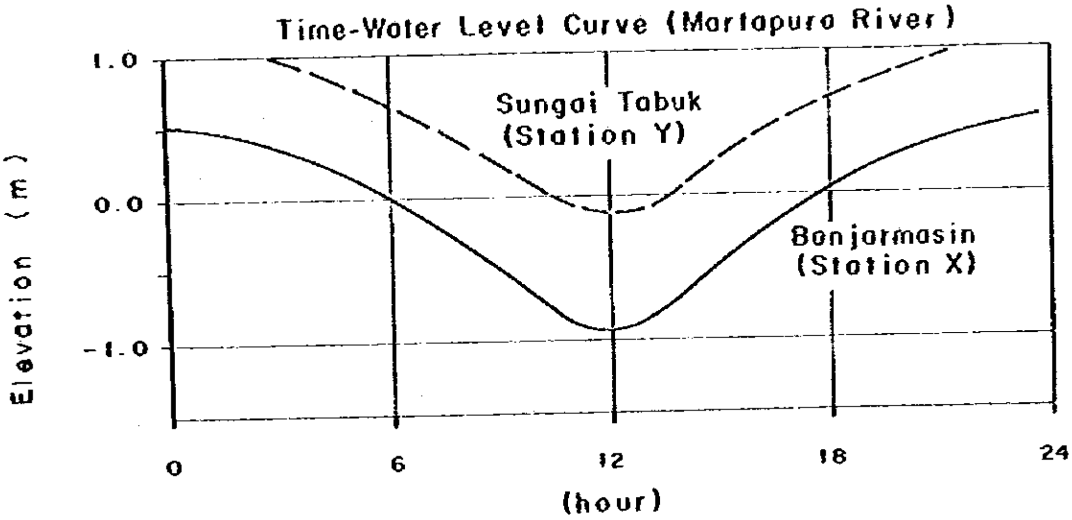


Fig. VII-11 Mean Daily Tidal Cycle (January, 1978)



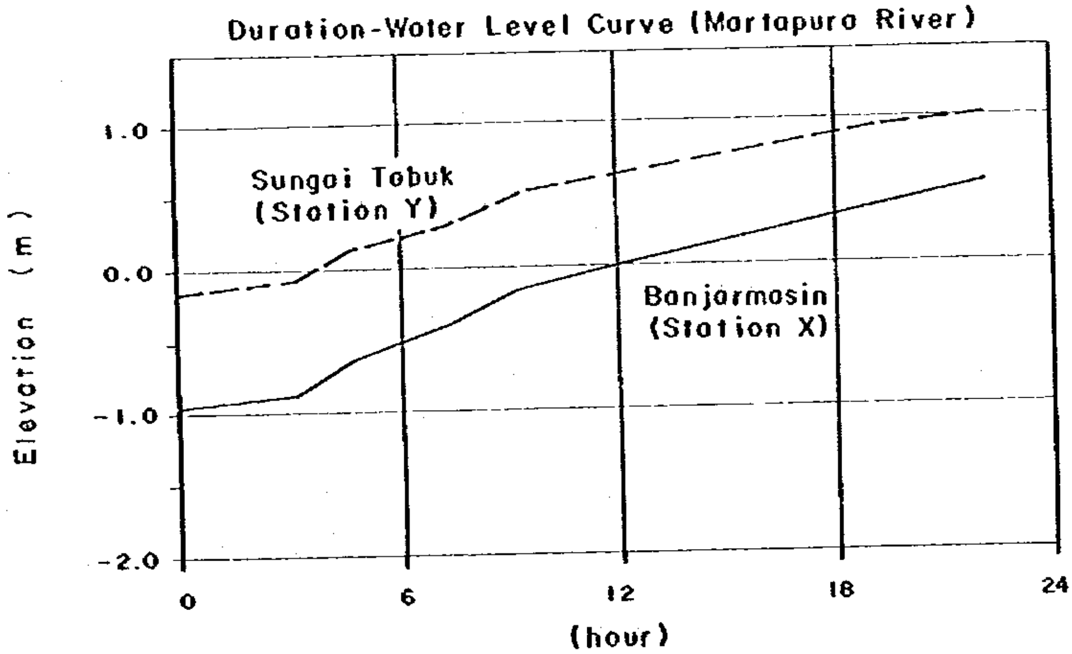


Fig. VII-12 **Standardized Tidal Range at Banjarmasin, Station X**

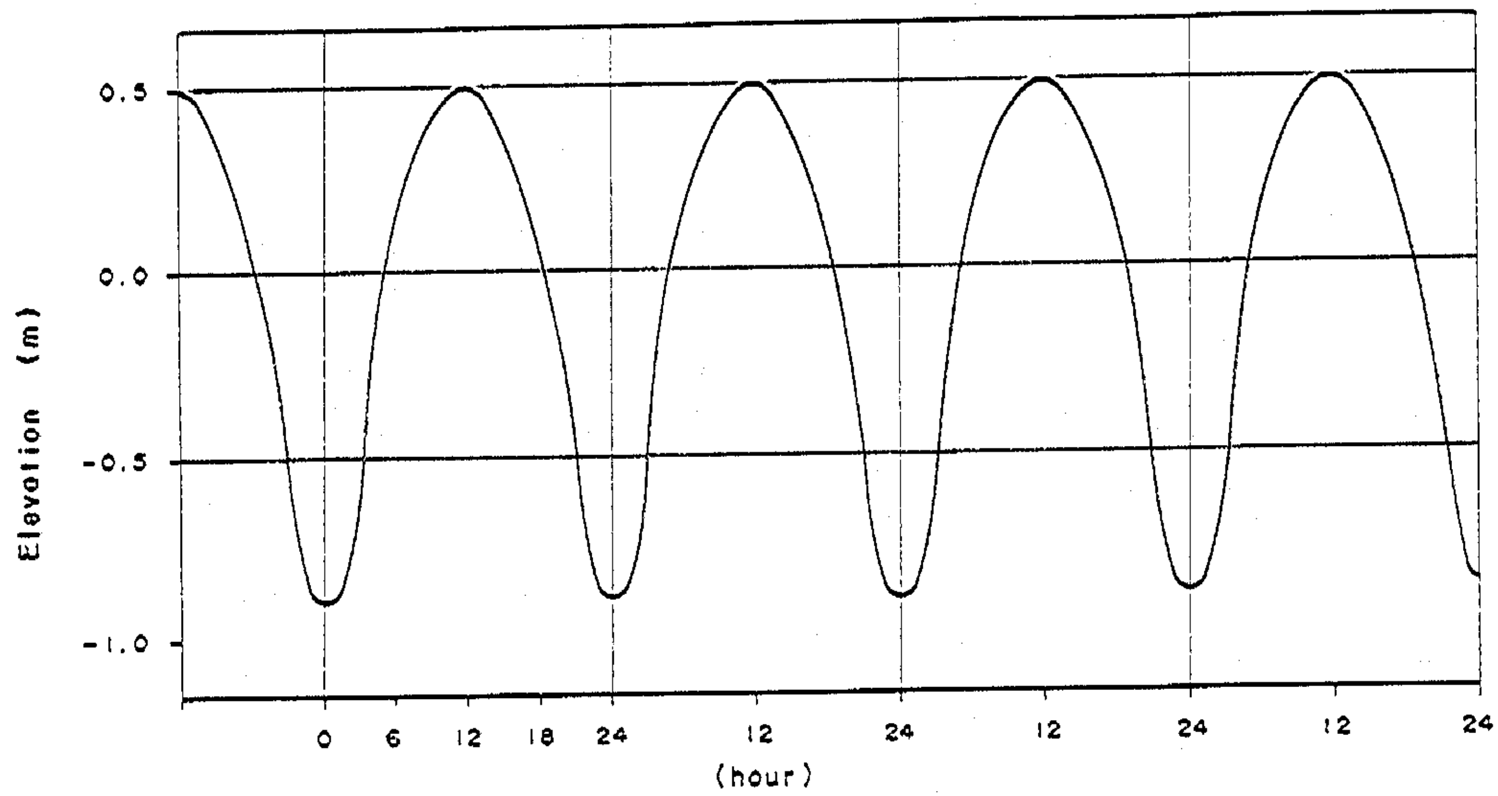


Fig. VII-13 Standardized Tidal Range at Sungai Tabuk, Station Y

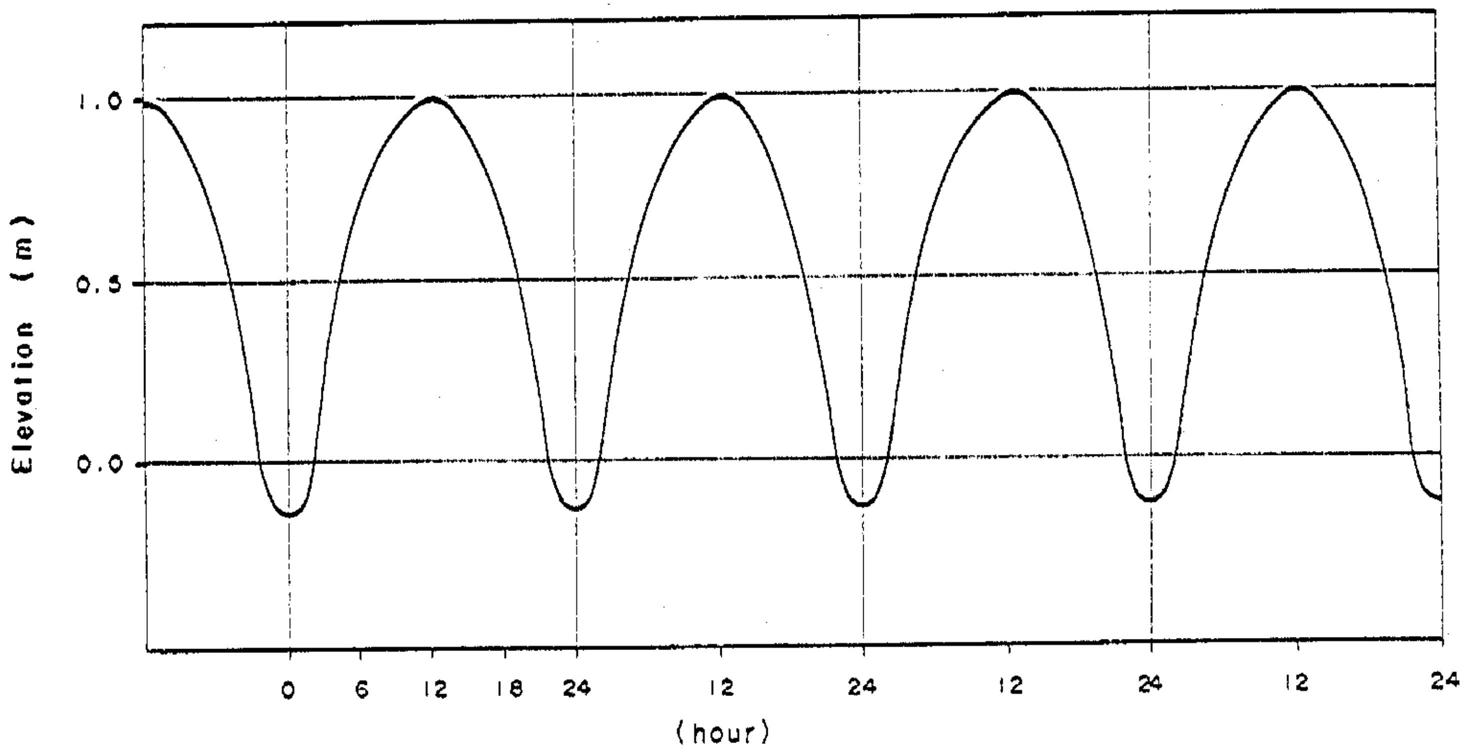


Fig. VII-14 Standardized Tidal Range at Barito, Station U

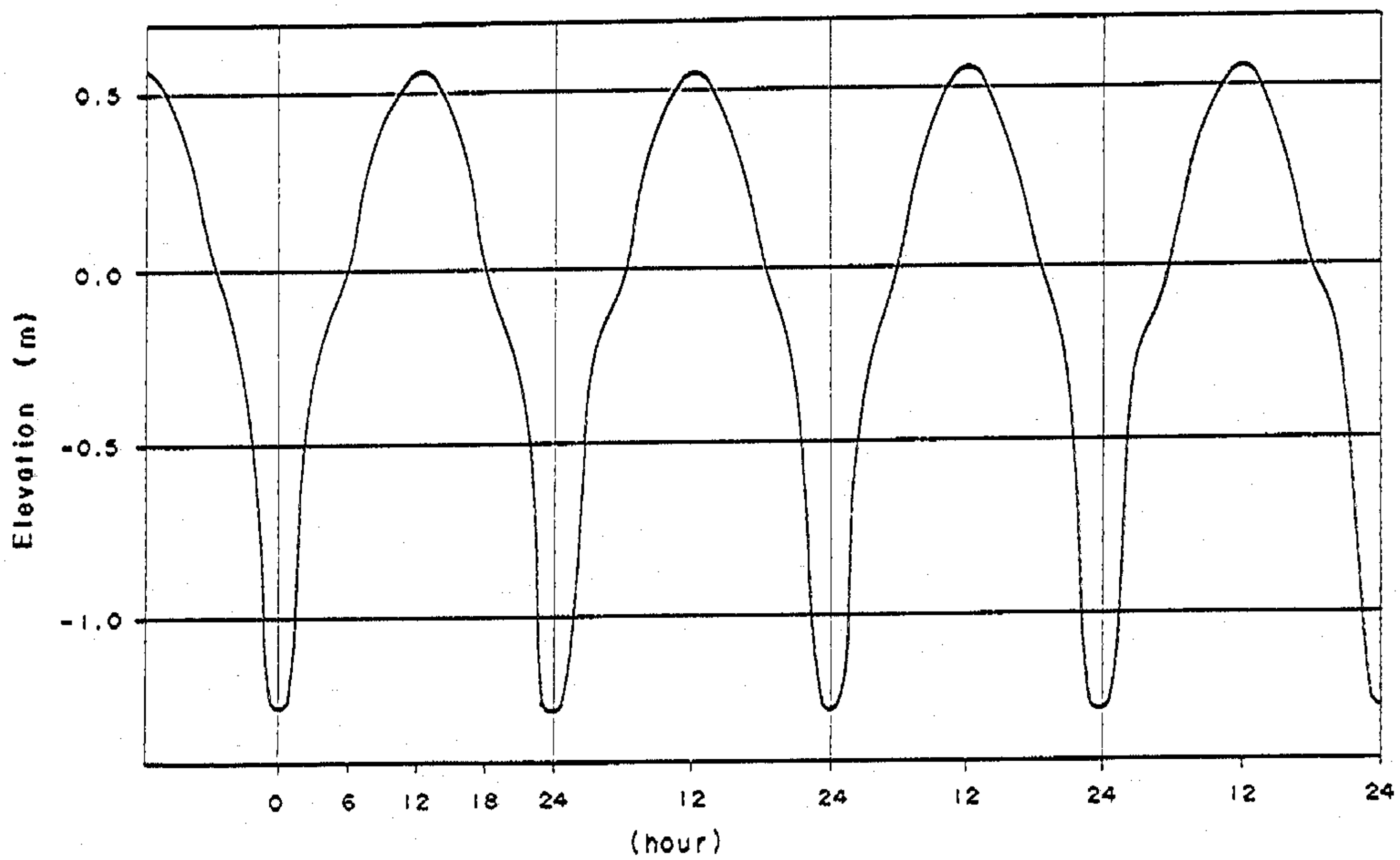
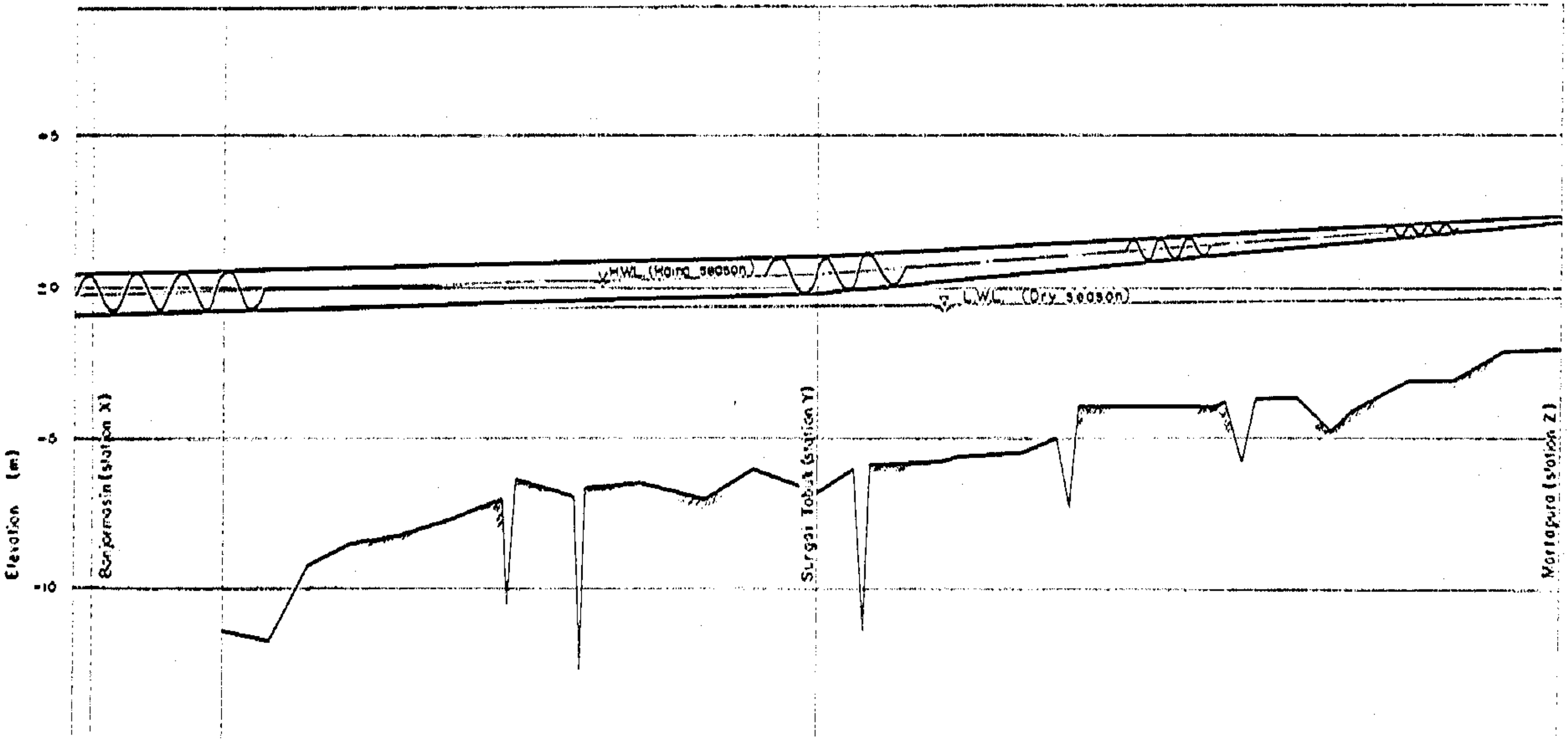


Fig. VII-15 Longitudinal Profile of Water Level along the Martapura River



Station No.	Passed Dist. (km)	Average HWL (El. m)	Amplitude HWL (El. m)
Port (WLR)	0.00	-0.17	1.41
No 0	4.60	-0.07	1.37
No 1	6.00	-0.03	1.35
No 2	7.75	0.01	1.34
No 3	9.25	0.05	1.32
No 4	11.05	0.03	1.30
No 5	12.65	0.13	1.29
No 6	14.65	0.18	1.27
No 7	17.15	0.24	1.25
No 8	19.10	0.28	1.23
No 9	21.35	0.34	1.21
No 10	23.00	0.38	1.19
No 11	25.25	0.43	1.17
No 12	26.85	0.55	1.10
No 13	29.30	0.74	1.00
No 14	29.55	0.79	0.97
No 15	31.56	0.94	0.83
No 16	33.55	1.06	0.83
No 17	36.10	1.26	0.72
No 18	38.10	1.41	0.64
No 19	39.16	1.49	0.59
No 20	41.00	1.63	0.52
No 21	42.10	1.71	0.47
No 22	42.77	1.77	0.44
No 23	44.65	1.91	0.36
No 24	46.13	2.02	0.30
No 25	47.83	2.15	0.23
No 26	49.80	2.30	0.15

ANNEX VIII

SELECTION OF AREAS FOR DEVELOPMENT

Table of Contents

List of Table

ANNEX VIII SELECTION OF AREAS FOR DEVELOPMENT

VIII.1 GENERAL

A variety of complex and interrelated land data were collected and analyzed in selecting the area for irrigation development. Systematic appraisal for the soils and topography in terms of irrigation and drainage conditions is briefed in this section, which aims at the delineation of the design area for the comprehensive development plan of the Riam Kanan Irrigation Project.

VIII.2 CONSIDERATIONS AFFECTING SELECTION

The land classification survey reveals the grade of irrigation suitability. Typical characteristics of the natural soil bodies involved are: soil texture quality,.effective soil depth, soil acidity, topography and drainage.

Micro and macro topography are evaluated with respect to degree and direction, land capability and land development requirements. Irrigability in relation to location and topography is the main point in this context.

The drainability of the area as a whole is considered in relation to the drainage characteristics of topography, and river and sea water fluctuations. For this, data were collected and analyzed regarding (1) drainage pattern of the area and subdivision of the area into sub-areas of different hydrological characteristics, (2) position and fluctuation of inundation water level, (3) ground contours and main direction of water flow, (4) identification of barriers and runoff characteristics and (5) natural drainage of the area and eventual inflow from the surrounding areas.

Last but not least, the present land use and vegetation of the area are taken into consideration, since these conditions have large effects on the relative difficulty in making reclamation for irrigation farming.

The delineation of the Project is made based on the above four conditions as well as the predictions of future alternations such as sea water intrusion due to completion of the project works. The areas which satisfy these factors are described hereinafter.

VIII.2.1 <u>Delineation of the Project</u>

As stated in Annex VII, the economical comparative study which is made in combination of the diversion weir and the main canal indicates that the optimum water level is EL. 9.80 m at the canal head. Since the location of the canal route is inevitably settled by the study stated above, the selection of the area for irrigation development is made within the areas which are effectively irrigable with this main canal.

Since the amount of water available from the river Riam Kanan is limited, part of the project area would not be irrigated in the dry season. For further increase of irrigation area during the dry season, the selection of the area is made using the water from the river Maluka which is another surface water source in the project area.

Sub-area A

The total gross area of this sub-area is 3,660 ha.

According to the soil survey results, the area is graded into three classes, the suitable land (Class II) with an area of 2,600 ha, the marginally suitable land (Class IV) with an area of 400 ha and the economically unsuitable land for irrigation farming (Class V) with an area of 660 ha.

The present land use survey explains that Class II land consists of 940 ha of paddy field, 900 ha of small productive rubber plantations, 200 ha of shrub land, 560 ha of alang-alang grass land and others for the canals, roads, etc. Class IV land consists of 360 ha of paddy field and 40 ha of other lands, and Class V is wasted gravel soil land covered with alang-alang grass.

The study on the drainability indicates that the area is classified into three, the perfect drainable land (Category 1) with an area of 2,960 ha, the favourable drainable land (Category 2) with an area of 300 ha and poorly drainable land in the rainy season (Category 3) with an area of 400 ha.

The investigation on the irrigability reveals that the water could be distributed to the land with an area of 2,200 ha by gravity through the proposed main canal.

The systematic appraisal conducted as an integrated study with the above four disciplines results in the selection of land with an area of 1,800 ha suitable for paddy cultivation. In this context, it would be suggested to shift the present small productive rubber plantations into paddy field. Since the land in Class IV with an area of 400 ha which is inundated in the rainy season is located in the central part of this sub-area, it would have to be included in the Project to make comprehensive drainage system in the area. The area would be used for single cropping of paddy. The land in Class V, 660 ha, and technically non-irrigable land, 800 ha, would be excluded from the Project.

The total irrigable land in this sub-area would, therefore, be 2,200 ha in gross.

Sub-area B

The total gross area of this sub-area is 18,380 ha.

According to the land classification study results, the area is graded into four classes, the suitable land (Class II) with an area of 7,800 ha, the moderately suitable land (Class III) with an area of 1,380 ha, the marginally suitable land (Class IV) with an area of 2,900 ha and the economically unsuitable land (Class V) with an area of 6,300 ha.

The present land use survey indicates that Class II land consists of 4,530 ha of paddy field, 200 ha of small productive rubber plantations, 2,850 ha of shrub land and 220 ha of other lands. Class III land includes 50 ha of small productive rubber plantations, 1,150 ha of shrub land and 180 ha of other lands. Classes IV and V consist of paddy field, bush, alang-alang grass land, swamp forest and other lands.

The study on the drainability indicates that the area is classified into four, the perfect drainable land (Category 1) with an area of 7,730 ha, the favourably drainable land (Category 2) with an area of 4,800 ha, the poorly drainable land (Category 3) with an area of 800 ha and the non-irrigable land (Category 4) with an area of 5,050 ha.

The investigation on the irrigability reveals that a part of the area which is locally elevated could not be irrigated by gravity. The extent is estimated to be 480 ha. Other than this elevated land, it is expected that the irrigation water would be distributed to the entire area without any difficulty.

An integration is made among the above four conditions in selecting the area for the Project. The land which is classified into Class II with an area of 7,800 ha is taken up as the project area, because it falls into either Category 1 or 2 in terms of the drainability. In this context, it would also be suggested to shift the present small productive rubber plantations into paddy field. 900 ha of shrub land in Class III would be included in the project area, because the land falls mostly into Category 2 (about 200 ha in Category 3) in view of drainage conditions. The lands classified into Classes IV and V would be excluded from the Project due to poor soil conditions, low irrigability and drainability.

Then, total gross irrigable land in the sub-area B would be 8,700 ha including 200 ha of land for single cropping of paddy.

Sub-area C

The total gross area of the sub-area C is 5,320 ha. Since the coconut plantations occupy 920 ha of land, the area envisaged for the Project will be 4,400 ha in gross.

According to the land classification study, the area is graded into two classes, the suitable land (Class II) with an area of 4,000 ha and the moderately suitable land (Class III) with an area of 400 ha.

The present land use survey shows that classes II and III lands consist of paddy field with an area of 3,900 ha, small productive rubber plantations with an area of 100 ha and other lands with an area of 400 ha.

The study on the drainability indicates that the area is classified into two, the perfect drainable land (Category 1) with an area of 1,690 ha and the favourably drainable land (Category 2) with an area of 2,710 ha (excluding the area for coconut plantations).

From the proposed location of the main canal, the entire area could be irrigated by gravity.

It is concluded, therefore, that the whole area excluding the coconut plantations will be taken up for the Project, including the present small productive rubber plantations which would be shifted into paddy field under the Project.

The total irrigable land in this sub-area would be 4,400 ha in gross.

Sub-area D

The total gross area of the sub-area D is 18,050 ha.

The land classification study shows that the area is graded into three classes, the suitable land (Class II) with an area of 11,450 ha, the moderately suitable land (Class III) with an area of 2,100 ha and the marginally suitable land (Class IV) with an area of 4,500 ha.

The present land use survey indicates that Classes II and III consist of paddy field and other lands. Whilst, Class IV land which is located in the western swampy area, consists of paddy field with coconut trees and other lands.

From the viewpoint of the drainability, the area is classified into three, the perfect drainable land (Category 1) with an area of 3,970 ha, the favourable drainable land (Category 2) with an area of 9,580 ha and poorly drainable land (Category 3) with an area of 4,500 ha.

The entire area, except for the elevated river shore belt along the river Barito, could be irrigated by gravity with the proposed main canal.

With these studies, the lands, which are classified into Classes II and III with an area of 13,550 ha, are included in Category 1 or 2 in terms of drainability. Therefore, these land would be taken up for the Project with irrigation and drainage improvement. While, the land classified into Class IV submerges during the rainy season without any drastic measures such as the construction of large-scale polder with check devices. Moreover, the land in Class IV is shallowly bottomed by the mud-clay (potential acid soil). The fact would result in very acidic soils, once the soils are dried by the drainage works. Therefore, it would be suggested that Class IV land should be left as it is, excluding from the Project.

The total gross irrigable land in this sub-area would be 13,550 ha.

Sub-area E

The total gross area which could be taken up for the sub-area E is 14,750 ha.

According to the land classification study, the area is graded into four classes, the suitable land (Class II) with an area of 6,650 ha, the moderately suitable land (Class III) with an area of 3,550 ha and the marginally suitable land (Class IV) with an area of 4,550 ha. The economically unsuitable land (Class V) is not included in the above gross area.

The studies show that the drainability is poor as a whole, except partially for the areas which are used as paddy field at present. The poor drainability is thought to be the fact that the slope of the river Maluka is very gentle and that tidal fluctuation exerts an effect on the water level up to the fairly upper reaches of the river. The area taken up for the study is classified into three, the perfect drainable land (Category 1) covering 2,990 ha of land, the favourably drainable land (Category 2) with an area of 7,210 ha and the poorly drainable land in the rainy season (Category 3) with an area of 4,550 ha.

In the southern part of the sub-area E, the land with an area of 690 ha is being developed under the existing reclamation project. This area is excluded from the Riam Kanan irrigation project.

The total gross irrigable land in the sub-area E would be 9,510 ha.

As for the future stage development plan, a comparative study is made between the gravity irrigation and the pumping irrigation for the dry season crops with the water from the Maluka as stated in Annex VII. The study result indicates that the pumping irrigation scheme would be preferable in view of construction cost and the extent of the area to be developed. The amount of water available at the proposed pumping site is estimated at 4.8 m^3/sec

Table VIII-1 Selection of the Project Area

	Sub-area A (ha)	Sub-area B (ha)	Sub-area C (ha)	Sub-area D (ha)	Sub-area E (ha)	Total (ha)
A. Area selected for final delineation of Project Area from land suitability (gross area)						
Class II	2,600	7,800	4,000	11,450	6,650	32,500
Class III	-	1,380	400	2,100	3,550	7,430
Class IV	400	-	-	-	-	400
Sub-total	3,000	9,180	4,400	13,550	10,200	40,330
B. Area selected for final delineation of Project Area from drainability (gross area)						
Category 1	2,300	3,580	1,690	3,970	2,990	14,530
Category 2	300	4,800	2,710	9,580	7,210	24,600
Category 3	400	800	-	-	-	1,200
Sub-total	3,000	9,180	4,400	13,550	10,200	40,330
C. Final Project Area in gross /1	2,200	8,700	4,400	13,550	9,510	38,360
D. Net irrigable area	1,870	7,400	3,740	11,520	8,080	32,610
D-1 Net irrigation area in wet season	1,530	7,230	3,740	11,520	8,080	32,100
D-2 Net irrigation area in dry season	1,480 (340)	5,585 (170)	2,800	8,620	6,045	24,530 (510)
E. Maximum available water for irrigation in the dry season (m^3/sec)	2.02	7.65	3.84	11.81	8.28	33.6

/1 : This is determined, excluding the land which is not irrigated economically from the potential area delineated through the studies on soils, land suitability and drainability.

Figures in parentheses show the area to be used for single cropping of dry season paddy.

ANNEX Ⅸ

PROPOSED PROJECT WORKS

Table of Contents

Page

List of Tables

List of Figures

Page

ANNEX IX PROPOSED PROJECT WORKS

IX.1 ALTERNATIVE PLANS FOR DIVERSION WEIR

IX.1.1 Selection of Site

A diversion weir was proposed to be constructed at Sungai Asam, about 12 km downstream from the Riam Kanan dam, in the Preliminary Survey Report. It was planned to adopt the Copure method for the construction of the diversion weir.

Detailed field reconnaissance of the proposed weir site was carried out for this study based on the detailed topographic maps on a scale of 1 to 5,000. As a result, another suitable site from the topographic point of view was found near the village Mandikapau, about 1 km upstream from the Sungai Asam site.

Further field reconnaissance was made in both upper stream area of Mandikapau and lower stream area of Sungai Asam for possible alternative sites which might be found. In the lower stream area than Sungai Asam, the elevation of both banks of the river is too low to construct the diversion weir suitable for this irrigation project. Under such topographic condition, a large scale diversion weir with a long crest would be required, which would call for high construction cost as compared with those at Sungai Asam and Mandikapau.

If the diversion weir is constructed in the upper stream area of Mandikapau, the main irrigation canal would cross the manifold hilly area. Due to the increase of canal length and earthwork volume, the construction cost of the main irrigation canal would be high.

For the above two main reasons, the suitable weir sites are not considered in both upper and lower stream areas, except for the two sites already selected at Sungai Asam and Mandikapau. The sites are shown in Fig. IX-6.

Therefore, either Sungai Asam site (site-A) or Mandikapau site (site-B) is finally proposed for the Project. The comparative study on these two sites is made to determine the final weir site and the most suitable type of weir from both technical and economic points of view as presented below.

IX.1.2 Topographic and Geological Investigations on Weir Sites

The plan table surveying and check lewelling were carried out for the two sites. As for the site-A, the topographic map on a scale of 1 to 2,000 was prepared by the Indonesian Government based on B.P. benchmark system. For the site-B, the topographic map on a scale of 1 to 1,000 was prepared by the Team using T.P. system.

The geological investigation for the site-A was made by the Government by means of core boring and test pits in 1972, whereas for the site-B during this survey period by means of core boring. As described in Annex IV, both sites have favourable geological conditions for the foundation of the proposed concrete weir.

However, it is noted from the above investigations that special measures would be required for the treatment of the right banks at both sites-A and B. Since the right bank at site-A is very flat topographically, the crest length would be very long, say 800 m. Again, since the soil on fresh rock layer on the right bank at the site-B is fairly deep geologically, a special attention would have to be paid for foundation treatment. In this context, it is proposed for the right banks to adopt earthfill type for the site-A, and floating type for the site-B.

IX.1.3 Hydrological Analysis of the Weir Sites

The discharge of the river Riam Kanan is completely controlled by the Riam Kanan dam. Hydrological conditions at the proposed weir sites depend entirely on the operation of the dam, power generation and flood control.

Small reservoir to be created by constructing the diversion weir would have a function as an after-bay for regulating the daily variation of discharges released through power plants in order to obtain the stable irrigation water and to release certain amount of water for the maintenance of the river Martapura. Hydrological values of the proposed diversion weir would have to be determined so that power generation would not be affected by the backwater caused by the construction of the weir.

The rating curve at the proposed weir sites is estimated using the Manning's formula and is cross-checked by hydraulic calculation of non-uniform flow between Mali Mali, about 9 km downstream from the weir sites, and the Riam Kanan dam site, based on the following investigation and available data:

(1) River cross sectional survey at the proposed weir sites.

(2) The past flood traces obtained through interview with the local people, especially for the floods in 1937 and 1961 estimated to be more than 1,000 m^3/sec and 800 m^3/sec, respectively, at Mali Mali.

(3) Available rating curve at three gauging stations on the river Riam Kanan, at Mali Mali, Awangbangkal and Riam Kanan dam site.

(4) The reports on the Riam Kanan dam project and operation manual of the dam.

The estimated rating curve at the proposed weir sites is shown in Fig. IX-1.

According to the design report/1 on the Riam Kanan dam, the peak flood discharge is 1,950 m^3/sec which is defined as 1.44 times the 100 year-flood peak. The regulated peak discharge from the dam is 500 m^3/sec in case of this flood. The 100 year-flood peak at the Riam Kanan dam site is estimated at 1,400 m^3/sec, and the regulated peak discharge in this case is 420 m^3/sec as shown in Fig. IX-2.

The design flood from the additional catchment area in between the dam and weir site is estimated in the same manner as in the above. Since the additional catchment area (There is only little difference between the sites-A and B in terms of the catchment area.) is 80 km^2. the design flood is estimated at 150 m^3/sec, which is equivalent to 1.44 times the 100 year-flood peak, and at 110 m^3/sec which is equivalent to the 100 year-flood peak.

The design flood for the diversion weir would be given as the sum of regulated flood discharge from the Riam Kanan Reservoir and flood discharge from the additional catchment area.

As mentioned before, the proposed diversion weir at the site-A would be concrete weir with earth dike of about 800 m in length as shown in Drawing No. 4. On the other hand, the diversion weir at the site-B would be concrete weir without earth dike as shown in Drawing No. 5. In view of type of diversion weirs at both sites, the maximum design flood discharge to be applied to each site would be as follows:

	site-A (m^3/sec)	site-B (m^3/sec)
Regulated discharge from dam	500	420
Flood from the additional catchment area	150	110
Total	650	530

/1: Report on the design of emergency spillway for the Riam Kanan Project Jan. 1971.

The ordinary and ordinary maximum discharges at the proposed weir sites are considered to be the discharge from the power station plus discharge from the additional catchment area. According to the design report on the Riam Kanan dam, maximum discharge from the power station is 87 m^3/sec and average daily discharge is 44 m^3/sec at the final stage. The discharge from the additional catchment area is expected to be 4 m^3/sec. Therefore, the ordinary and the ordinary maximum discharges at the weir sites would be 48 m^3/sec and 91 m^3/sec, respectively.

It is noted that there is an important restriction in determining the crest elevation of the weir. In other words, it is absolutely necessary not to heighten the present tail race water level due to the backwater effect by constructing the weir. Otherwise the original operation program of the power plant would be altered. In this context, the examinations on the backwater effect would have to be made in two cases. One is at the flood time and the other is at the ordinary maximum discharge time. The results of the calculation for the above two cases are shown in Figs. IX-3 and IX-4.

Fig. IX-3 shows the effect of the damming-up caused by the weir on the water level between the weir site and the dam site in case of the design flood time. When the flood water level would be raised up to 13.0 m in elevation from the original water level, there would be a small backwater effect at the dam site, design water level at which is 19.607 m in elevation. The fact suggests that the design of the weir would have to be made so that the water level would not exceed 13 m in elevation at the weir site.

The effect of weir on the water level at the dam site in case of the ordinary maximum water level is shown in Fig. IX-4, which indicates that if the water level at the weir site is more than 12.0 m in elevation, the backwater would exert an effect on the tail race water level, which was designed to be 14.807 m in elevation.

Consequently, the crest elevation of the diversion weir would have to be determined so that the water level in the upstream of the weir would not exceed 13 m in elevation at the maximum design flood time and 12 m in elevation at the time of ordinary maximum discharge.

IX.1.4 Regulating Capacity

As described in Annex III, the expected average discharge from the Riam Kanan dam is estimated at 42 m^3/sec with 15.6 MW of average output. The maximum discharge from the dam is estimated at 65.6 m^3/sec with 24 MW of peak power generation. The duration of this peak power generation would be 5 hours, making reference to the present actual operation pattern in Surabaya. With these figures, the regulating capacity required for the diversion weir is calculated as follows:

$$V = (65.6\ m^3/sec - 42\ m^3/sec) \times 5\ hours \times 3,600\ sec = 425,000\ m^3$$

Based on the topographic maps on a scale of 1 to 5,000, storage volume curves at both site-A and site-B are prepared as shown in Fig. IX-5.

The fluctuation range of the water level in the after-bay necessary to regulate 425,000 m^3 of water and the water levels at the head of the main irrigation canal are tabulated below and are shown in Fig. IX-5 in three cases of the crest elevation of 11, 10 and 9 m at both site-A and site-B.

Crest elevation (m)	Fluctuation range of water level: Site-A (m)	Fluctuation range of water level: Site-B (m)	Water level at the head of canal in elevation: Site A (m)	Water level at the head of canal in elevation: Site-B (m)
11	0.13	0.15	10.87	10.85
10	0.17	0.20	9.83	9.80
9	0.24	0.27	8.76	8.73

According to the report on the Riam Kanan Dam Project/1, annual sedimentation in the Riam Kanan reservoir was estimated at 86 $m^3/year/km^2$, based on the results of observation at Awangbangkal. With this figure, the 100-year life sedimentation in the after-bay from the remaining catchment area, about 80 km^2, is estimated as follows:

$$86\ m^3/year/km^2 \times 80\ km^2 \times 100\ year = 700,000\ m^3 < 2,700,000\ m^3*$$

* Gross storage capacity of after-bay in case of El. 9 m of weir crest at the site-B.

Accordingly, total sedimentation would not exceed the capacity of the after-bay.

IX.1.5 Determination of Diversion Weir Site and Crest Elevation

As mentioned before, the crest elevation of the diversion weir is to be determined so that the water level in the upstream of the weir does not exceed 12 m in elevation at the time of ordinary maximum discharge. Considering the overflow water depth on the crest of the weir, the maximum crest elevation of the weir is determined to be 11 m. The minimum crest elevation of the weir required for the supply of irrigation water keeping suitable head is to be 9 m. Even with this variation of the crest elevations between 11 m and 9 m, there would be no variation of irrigated land in terms of extent of area to be served by the weir. The

/1: Preliminary Report on the Riam Kanan Project, July in 1962.

higher crest elevation would call for the higher costs for the construction of the weir and compensation for houses and lands to be submerged by the construction of the weir. Since the proposed main irrigation canal would be aligned avoiding the swamp area in principle, the canal route would be selected so as to pass through the elevated land as far as possible. This means that the lower crest elevation would require the higher construction cost of the canal due to the increase of excavation work.

In order to determine the most suitable weir site and its crest elevation, the comparative study is made, estimating the cost of the following items:

- Costs for concrete works and earth works for the construction of the weir,
- Costs for earth works for the main irrigation canal, and
- Compensation costs for houses and lands to be submerged.

The comparison of these costs is made for the following six cases based on the preliminary design shown in Fig. IX-6.

Crest elevation (m)	Site-A	Site-B
9	Case A-1	Case B-1
10	Case A-2	Case B-2
11	Case A-3	Case B-3

The main features of the weir such as dimensions, hydrological values, etc. in the above six cases are shown in Table IX-1. The work quantities, costs of weir, main canal and compensation are shown in Tables IX-2, IX-3 and IX-4, respectively. The unit costs used for the similar irrigation projects completed and/or under construction in both South Kalimantan and Java, and the survey results of the compensation are used for this comparative study.

The total costs of each case are summarized below.

Item	Site-A			Site-B		
	Case A-1	Case A-2	Case A-3	Case B-1	Case B-2	Case B-3
Weir	3,363	3,735	4,790	1,932	2,034	2,142
Main irrigation canal	6,128	6,079	5,913	6,897	6,316	6,490
Compensation	331	508	697	154	380	631
Total	9,822	10,322	11,400	8,983	8,730	9,263

Note: All costs in the above table are shown in 1,000 US$.

Depreciation costs of the construction equipment are included.

From this cost comparison, the total costs in each case at the site-B are all lower than those at the site-A, and case B-2 shows the lowest total cost. At the site-A, the total cost in the case A-1 is the lowest. It is concluded that the case B-2 is taken up as the most suitable type weir for the following studies.

The preliminary design of the weir in the cases A-1 and B-2 are shown in Drawings No.4 and No.5, respectively.

IX.2 PRELIMINARY DESIGN OF DIVERSION WEIR

IX.2.1 Type of Weir

The present cross section of the river at the proposed weir site consists of two portions; one is flow channel (about 40 m in width) through which normal discharges flow down, and the other is river land (about 40 m in width) which becomes flow channel when small and middle scales of floods less than 150 m^3/sec occur.

There is a depression beyond the right bank levee of the river. The width of the depression is about 160 m. In order to dam-up the river water level up to 10 m in elevation to divert the irrigation water, the construction of an embankment is required in this depression. It is noted, however, that the water level of the downstream would rise up to 12.2 m in elevation at the design flood time, which are higher than the existing river levee. In order to construct the diversion weir with a sufficiently high stability, it is preferable to construct the whole of the weir with concrete without any earthfill portion.

As described in Annex IV, the geological conditions of the weir site are favourable in view of bearing capacity for the concrete weir. However, the Quaternary sediments underlain on the fresh rock are fairly thick on the right major bed of the river, whereas they are thin on the left bank.

In this context, the weir is proposed to consist of two types. One is of floating type under which the fresh rock lies deeply, and the other is of fixed type under which the fresh rock exists with shallow layer.

The principal features of the proposed diversion weir would be as follows:

Type of weir	Fixed weir (on the river bed)
	Floating weir (on the major bed at the right bank)
River bed elevation	El. 0.00 m
Intake water level	El. 9.80 m

Max. intake discharge	34 m^3/sec
Regulating capacity	425,000 m^3
Crest elevation	El. 10.0 m
Design flood discharge	530 m^3/sec
Flood water level (downstream)	El. 12.2 m
Flood water level (upstream)	El. 12.25 m
Total length of weir	228 m
Length of fixed weir	48 m
Length of floating weir	180 m
Minimum release of discharge to the downstream	8 m^3/sec
Gates for releasing water to the downstream	2.0 m x 4.0 m x 2 nos.
Height of weir (from apron to crest of weir)	9.0 m

IX.2.2 Hydraulic Conditions and Backwater Effect

The hydraulic conditions would be altered by the construction of the diversion weir. The water level in the upstream and the downstream of the weir at the time of the flood discharge, the ordinary maximum discharge and the ordinary discharge, would be as shown below.

	Discharge (m^3/sec)	The water level in downstream (m)	The water level in upstream (m)
At the time of design flood discharge	530.0	12.20	12.28
At the time of ordinary maximum discharge	91.0	5.90	10.85
At the time of ordinary discharge	48.0	4.70	10.57

As described in the previous section IX.1.3, the operation of power station would not be affected by backwater under above both hydraulic conditions, design flood discharge and ordinary maximum discharge. In addition to the above cases, hydraulic analysis of backwater would have to be made in the various hydraulic conditions. The results of the analysis in the following eight cases are shown in Fig. IX.7.

Case	Condition	Discharge At weir site (m^3/sec)	At power station (m^3/sec)
Case 1	Extreme high flood	650	500
Case 2	Design flood	530	420
Case 3	Ordinary flood	357	300
Case 4	- do -	228	200
Case 5	Ordinary discharge	91	87
Case 6	- do -	46	42
Case 7	- do -	39	35
Case 8	- do -	24	20

As proved in Fig. IX-7, the water level at the dam site in any cases of discharge would not be higher than the design water level in the tail race of the dam.

IX.2.3 Foundation and Stability Analysis

The foundation of the fixed type of weir is the fresh hard rock which has enough bearing capacity and solidity for the concrete weir. No foundation treatment on this part would be required. On the other hand, the foundation of the floating type of weir is the Quaternary sediments and weathered rock with permeability coefficient ranging from 10^{-3} to 10^{-4} cm/sec. The foundation treatment with cut-off of concrete and long apron would be required to prevent piping through the foundation.

The weir is designed as an overflow type. The stability analyses are made for the overturning and sliding against the external forces for the above two cases as shown in Table IX-5 and Fig. IX-8. The slope of the upstream and downstream sides are thus determined to be 1:0.1 and 1:0.8, respectively, and the width of crest, 3.0 m.

IX.2.4 Apron

In case that the weir is fixed directly on the fresh rock, the construction of apron would not be necessarily required. Since the weir is proposed to be combined with the floating type, it seems essential to provide with the apron for the downstream protection against overflowing water, rolling stones and seepage. In this context, it is noted that the design of the apron is made on the assumption that the whole weir body is of floating type. The length and the thickness of the apron and the length of rock riprap are calculated to be 25.0 m, 1.0 m and 25.0 m, respectively, using the Bligh formula as shown in Drawing No. 5.

IX.2.5 Intake Structure

The design of the intake structure is made based on the discharge which would be 34 m^3/sec, the water level at the head of the main irrigation canal which would be 9.80 m in elevation and the fluctuation range of the water level in the after-bay which would be 0.2 m, taking into account the velocity of flow in the main canal and at the inlet of the intake structure, together with topographical and geological conditions. As stated in the preceeding section, since the sedimentation is not expected to exceed the capacity of the after-bay, no sand settling basin is considered to be installed on the intake structure. The width of the intake is determined to be 26.0 m, and the elevation of the intake sill, El. 7.4 m. The number of the gate for control of intake discharge would be nine with the dimension of 2.0 m (width) x 3.0 m (height).

IX.2.6 Coffering Works

In order to construct the diversion weir, coffering works would be necessary for dewatering. The time period for the construction would be limited only in the dry season, from May to October. Upon completion of the Riam Kanan dam, the maximum discharge from the power station is estimated at 65.6 m^3/sec with 24 MW of peak power generation in the future, and the maximum discharge for the past five dry seasons from the Riam Kanan dam was about 70 m^3/sec. Therefore, the design discharge for coffering is taken at 70 m^3/sec which seem to be able to flow river runoff without any obstacle for the works during the construction period from May to October.

The construction period is anticipated to be two years, and hence, coffering is required for two dry seasons. In the first year, the floating type weir on the right bank would be constructed in the dry work. A part of the crest of this floating type weir would have to be lowered in order to divert the design runoff for the construction of the remaining part of the weir. In the second year, another coffer dam would be constructed by utilizing the center guide wall of the weir. The fixed type weir on the river and the intake structure would be constructed during this period. The plan of coffering is given in Fig. IX-9.

IX.3 IRRIGATION AND DRAINAGE SYSTEMS

IX.3.1 Definition of Terms for Canals and Drains

Irrigation Canals

- Main canal is defined as the trunk canal which conveys water from the diversion weir to the entrance of the subareas.

- Secondary canal is defined as the canal which is diverted from the main canal to distribute water to each sub-area.

- Sub-secondary canal is defined as the canal which is diverted from the secondary canal to distribute water to each sub-divided sub-area.

- Tertiary canal is defined as the canal which is diverted from either the secondary canal or sub-secondary canal to distribute water to each farm block with an interval of 600 m, and the length ranging from 800 m to 1,600 m.

- Field ditch (quaternary canal) is defined as the ditch which is diverted from the tertiary canal to each field patch with an interval of 400 m and a length of 600 m.

Drains

- Field drain (quaternary drain) is defined as the drain which corresponds with the field ditch. The interval and the length are 400 m and 600 m, respectively.

- Tertiary drain is defined as the drain which collects water from the field drain and corresponds with the tertiary canal with an interval of 600 m. The length of the tertiary drain ranges from 800 m to 1,600 m.

- Sub-secondary drain is defined as the drain which collects water from the tertiary drains and corresponds with the sub-secondary canal.

- Secondary drain is defined as the drain which collects water from the sub-secondary drains and tertiary drains.

- Main drain is defined as the drain which collects water from the secondary drains.

IX.3.2 Main Irrigation Canal

As stated in Annex VIII, the total net area to be irrigated in the dry season would be 24,530 ha. Since the peak unit water requirement is estimated at 1.37 lit/sec/ha as shown in Annex VII, the gross irrigation requirement would be 33.6 m^3/sec. By adding the amount of water, 0.4 m^3/sec for rural water supply, the total amount of water to be conveyed through the main canal would be 34.0 m^3/sec.

In the preliminary design of the main canal, 85% of the above gross irrigation requirement plus rural water supply is adopted for easier control of the water level in the canal and for economical construction. The amount of water which is more than 85% of the peak would be flowed by the use of the freeboard of the canal section. This would last for one month. The main canal diagram is given in Fig. IX-10.

The type of the main canal is open channel. The canal is designed in order that there would be little seepage losses and the inside slopes of the canal would not be eroded.

The soil mechanical investigation carried at C.1 (see Fig. IV-4 in Annex IV) indicates that the top humic soils are underlain by white sand. Since the ground water level at this site is as high as 0.5 m even in the dry season, there would be erosion due to spring water. On the other hand, if the ground water level is lowered, high seepage losses would be expected. Moreover, as the allowable velocity for sand layer is very low (less than 0.45 m/sec), scouring would occur due to flow. The same trend in terms of soil conditions and ground water level is observed at point C.3 (see Fig. IV-4 in Annex IV) and on the route along the highway. It is therefore proposed to make lining with concrete for this part, which is the downstream reaches of the canal from Turnout No. 5 (see Fig. IX-10).

The soil mechanical investigation at point C.4 (see Fig. IV-4 in Annex IV) reveals that the ground water level around this part is low, and hence, no erosion due to the spring water would be expected. However, since there exists gravel layer in this part, the permeability of which is fairly high, big losses of water due to seepage would take place. In this view, it is proposed to make lining with earth for this part (between Turnouts No. 2 and No. 3 in Fig. IX-10).

Since the remaining part of the canal is favourable in view of topographical and soil conditions, unlined canal is proposed to save the construction costs.

Then, the length of the concrete lining canal would be 24,860 m, the earth lining canal,7,100 m and the unlined canal, 16,400 m, totalling 48,360 m.

As stated in the previous section in this Annex, there are two restrictions in determining the longitudinal slope of the canal. One is the existence of the Riam Kanan dam, and the other is the poor soil conditions along the proposed main canal. Taking into account these, the intake water level of the main canal is decided to be 9.80 m in elevation. The slope of the canal would be settled at 1/7,000. The velocity of the earth canal would range between 0.5 m/sec and 0.6 m/sec, whereas the concrete lining canal between 0.8 m/sec and 1.1 m/sec.

Drawing No. 6 shows the longitudinal section of the main canal, which is proposed to be located at the lowest possible limits to minimize the excavation works within the limits of favourable conditions or avoiding the swamp areas. Due to the low elevation of water level at the canal head and the undulating topography along the canal route, the volume of soil excavation would be inevitably large.

The dimension of the canal depends on the amount of discharge, longitudinal slope and the canal material. Drawing No.6 shows the dimensions of each canal section in relation to discharge and velocity.

Two highway bridges, five local road bridges, three checks with gates, ten turnouts, twenty-six cross drains, one inverted syphon and six washing basins would be the principal structures on the main canal. The location of the structures is shown in Drawing No.1.

IX.3.3 Secondary Alignments

The secondary and the sub-secondary canals in the whole project area are designed as unlined with trapezoidal cross section. The preliminary design of the irrigation canals is made based on the basic design criteria described hereunder.

Simultaneous water distribution method is adopted since it has much advantage that investment in the irrigation system would be lower because of fewer water control structures and measuring devices as well as managing staff in the system.

The design discharge is calculated by multiplying the unit peak water requirement by the area to be served. The water level in the canals is designed to be 30 cm to 50 cm above the paddy field.

The maximum permissible velocity in unlined canal is decided in order to avoid the scouring of canal surface. The minimum permissible velocity is defined as the lowest velocity that does not cause silt depositing. The velocity range is thus determined to be between 0.2 m/sec and 0.6 m/sec. Considering the allowable water velocity and natural gradient of the surface, longitudinal slopes of the canals are designed to be within the range of 1/10,000 to 1/2,000.

The bank stability is analyzed based on the soil mechanical tests to determine the stable side slope, which would be 1:1.5 for both the secondary and the sub-secondary canals.

The drains in the whole project area are designed as the unlined canal with trapezoidal section. The preliminary design of the drains is made based in the design criteria described below.

Considering the natural gradient of the ground surface and the role of the drains, longitudinal slopes of the main, the secondary and the sub-secondary drains are designed to be in the range of 1/10,000 to 1/3,000.

The slope stability is analyzed based on the soil mechanical tests to determine the stable side slope. The side slopes are proposed to be 1:2 for the main drain and 1:1.5 for the secondary and the sub-secondary drains.

In the design of the capacity of drains, where there is a tidal effect, at the first step, the tentative decision for the capacity of the drains is made, then after calculation of water stage inside the area, the final capacity of the drains is chosen to meet the requirements (submergence is less than 30 cm). The storage-volume curve and water stage outside the area are used in this context. The stage-volume curve for storage calculation is obtained from topographic map. The water stage curve in the rivers is determined in Annex VII. The water level inside the area is calculated by routing a design hydrograph. Through the above procedure, the duration of gate closure, and hence, the capacity of the drains are determined.

The alignments of the canals and drains are referred to in Figs. IX-10 and IX-12, respectively. The length of and the number of the related structures on the secondary and the sub-secondary canals and drains are listed in Table IX-6. The design of the typical turnout structures and diversion boxes are shown in Drawing No.8, check gates for the irrigation canals and drains in Drawing No.9, bridges and culverts in Drawing No.11, and syphon and washing basins in Drawing No.10. The location of these structures is given in Drawing No.1.

Alignments in Sub-area A

As stated in the previous section, the sub-area A is sub-divided into two, the north (sub-area A_1) and the south (sub-area A_2) in view of drainage, because it is separated by a road running from east to west and there are no coming and outgoing of water between the two areas.

Sub-area A_1 is characterized by a polder surrounding the whole area, a depression in the center and a natural channel connecting the river Riam Kanan with the river Martapura a little to the west of the area. Since the water standing in the center would have to be drained, a secondary drain would be aligned to pass through the center of the area connecting with the Martapura. The existing channel would be used as another secondary drain by improving its course and capacity.

In order to irrigate both banks of the former secondary drain, this would be placed between the two secondary canals. The area separated by the latter drain would be irrigated by extending one of the secondary canals to west.

Sub-area A_2 is characterized by its relatively steep slope and undurating topography. There are five natural depressions running from south to north. Five sub-secondary drains would be arranged in these depressions by connecting with the Riam Kanan.

In order to irrigate the separated areas by the above drains, four sub-secondary canals would be needed. Each of the sub-secondary canals would be located between each of the sub-secondary drains.

The alignment of the secondary and the sub-secondary canals, and that of the secondary and the sub-secondary drains are schematically shown in Figs. IX-11 and IX-13, respectively.

In the design of the canals the peak unit water requirement is adopted in connection with the area to be served.

In the design of the drains for this area, the peak unit drainage requirement, 7.0 lit/sec/ha, is used by multiplying the area to be drained. Since the daily fluctuation of the outside water level due to tidal effect is very small in this area, no checks would be required at the outfalls of the drains. The capacity of the drains are designed so as to lower the water level in the drain 10 cm below the surface of the paddy fields.

Alignment in Sub-area B

The sub-area B is characterized by the vast extent of the unsuitable land located on the southern half of the area. The land consists of wasted swamp bush submerged throughout the year and wasted hilly area with alang-alang. As the land on the northern half which is used as paddy field at present is also low and flat, a very large area suffers from seasonal flooding.

In order to develop the suitable area on the northern half, it seems essential to protect the area from floods which come from the southern half. In this context, it is proposed to provide this area with a main drain which would separate the two contrasted areas running from east to west and connecting with the river Martapura. The function of the main drain would be then to undertake all of the floods from the southern half.

The suitable land in the northern half is characterized by a shallow depression extending from east to west. It is expected that the rain water within this area would be collected to this depression. To drain this water, another main drain is proposed to be located at the center of the area which would run from east to west and be drained to the river Martapura.

The said alignment of the main drains would separate the suitable land into two, one of which is located between the two main drains and the other, between the main drain and the river Martapura. In this context, two secondary canals would be necessary. One would be proposed to be located along the former main drain to irrigate the southern half and the other along the river Martapura to irrigate the northern half. The third secondary canal would be required to irrigate the southwestern part of the area.

The micro relief in the irrigable area would necessitate to arrange sub-secondary drains. Eight numbers of sub-secondary drain which runs from south to north and six numbers of the sub-secondary drain which runs from north to south is proposed to be connected with the latter main drain. In order to irrigate the sub-divided areas by these drains, 13 numbers of sub-secondary canal would be needed.

The alignment of the secondary and the sub-secondary canals, and that of the main, the secondary and the sub-secondary drains are schematically shown in Figs. IX-11 and IX-13.

In the design of the canals the peak unit water requirement is adopted in connection with the area to be served.

In the design of the drains for this area, the peak unit drainage requirement, 8.0 lit/sec/ha is used. In determining the capacity of the main drains, the considerations are taken in two ways. Since the land which is located north of the former main drain would be excluded from the Project due to various bad conditions for irrigated land, the design capacity of the drain is determined taking into account the retarding effect of the land. Whilst as the latter main drain is situated in the center of the land to be developed for irrigation farming, no retarding effect can be taken into consideration.

For instance, Fig. IX-14 shows the relationship between the discharge to be drained and the volume of water to be stored for the former main drain. As the catchment area of the drain is 8,600 ha, the peak discharge would be as high as 70 m^3/sec, when the design value of 8.0 lit/sec/ha would be applied. According to Fig. IX-14, if the design discharge of the drain would be fixed at 25 m^3/sec, the amount of water to be stored would be 6,000,000 m^3, which is translated as 2.0 m in water surface elevation or 1.0 m in water depth. In this context, it is proposed that a levee with a height of 1.5 m would be constructed inside area of the drain, capacity of which would be 25 m^3/sec.

As the outside water level fluctuates daily as discussed in Annex VII, the necessity and the location, if necessary, for the installation of flap gates are examined in relation to the inside water level. Since the function of the former main drain is to prevent water from flowing into the area to be developed and the submergence of the upper area is allowed, no check structures would be required.

On the other hand, the check measures for the latter main drain would be indispensable, because the outside water level becomes higher than the part of the land elevation. A comparative study on the location of the flap gates is made whether to be located at the outfall of the main drain, at the outfalls of the secondary drains or at the outfalls of the tertiary drains. The study result indicates that the best location of the flap gates is at the outfall of each secondary drain. The reason is attributed to the facts that if the gates would be located at the outlet of the main drain, a vast extent of land would be submerged during the high outside water level because of large drainage basin behind it, and that if the gates would be located at the outfalls of the tertiary drains, the number, and hence, the costs of check structures would be very large. The number of the check structures on the secondary or sub-secondary drains would be 15 in total.

The relationship among the drainage discharge through check structure, depth and duration of submergence in the paddy fields, and the fluctuation of outside water level, would have to be examined in each sub-divided area. Fig. IX-16 shows the result of examination in the model area covered by the sub-secondary drain SD-O-7. The figure indicates that maximum depth of submergence at the lowest paddy fields doesn't exceed 30 cm in depth.

Alignment in Sub-area C

Since sub-area C is enclosed by roads, there is no water coming from the adjacent areas. (In the description of the present condition of this area, the area facing to the river Martapura is included in the topic, but at the stage of delineation this area is excluded.) Despite the fact that the area is low and flat, drainage is made naturally fairly well through the existing two small rivers connecting the river Martapura.

There would be no competition to use these rivers as secondary drains by improving their capacity because these are located at the lowest parts of the area. Since these drains separate the area into three, three secondary canals would be necessary to irrigate each sub-divided area.

The alignment of the secondary and the sub-secondary canals, and that of the secondary and sub-secondary drains are schematically shown in Figs. IX-11 and IX-13, respectively.

In the design of the canals the peak unit water requirement is adopted in connection with the area to be served.

In the design of the drains for this area, the peak unit drainage requirement, 5.9 lit/sec/ha is used by multiplying the area to be drained. As the daily fluctuation of the outside water level due to tidal effect is very large in this area, it seems indispensable to provide the drains with check devices. A comparative study on the location of the flap gates is made whether to be located at the outfalls of the secondary drains or at the outfalls of the tertiary drains. According to the study result, the location is proposed at the outfalls of the secondary drains, because if the gates would be located at each outfall of tertiary drains, the number, and hence, the costs of check structures would be very large. The number of check structures on the secondary or sub-secondary drains would be three in total.

Fig. IX-17 shows the result of examination on the relationship among the drainage discharge through check structure, depth and duration of submergence in the paddy fields, and fluctuation of outside water level in the model area covered by the secondary drain SD-7. The figure indicates that maximum depth of submergence at the lowest paddy fields doesn't exceed 30 cm in depth.

Alignments in Sub-area D

Sub-area D could be divided into three areas in view of the future drainage system. Sub-area D_1 is located on the northeast of the area and is encircled by roads. The direction of drain is toward the river Martapura. Sub-area D_2 occupies the northwest of the area and is bounded by roads on the east and the south. The direction of flow is toward the river Barito. Sub-area D_3 is situated in the south of the area and is separated from the sub-areas D_1 and D_2 by a road located at higher land.

As there are two small rivers draining into the river Martapura in sub-area D_1, these would be used as secondary drains by improving their courses. Since these two secondary drains and another sub-secondary drain divide the area into three parts, three secondary canals would be necessary to be constructed. The micro relief in this sub-area would necessitate to arrange sub-secondary drains. The number would be six excluding the one stated above. To correspond with these subdivided areas, the same number of the sub-secondary canal would be needed.

For the development of sub-area D_2, two numbers of artificially made secondary drain would be required toward the river Barito together with a few numbers of sub-secondary drains, because there are no notable existing drains at present. Irrigation water necessary for thus subdivided four areas would be fed by the same number of either secondary or sub-secondary canals.

In sub-area D_3, one main drain would be proposed to be located along the road which separates the area from the sub-areas D_1 and D_2. This would collect all of the waters from the upper reaches. For the remaining areas, two secondary drains would be needed. The areas thus subdivided into three would be favourably irrigated by two secondary canals. According to the micro relief, four sub-secondary drains and five sub-secondary canals would be aligned.

The alignment of the secondary and the sub-secondary canals, and that of the main, the secondary and the sub-secondary drains are schematically shown in Figs. IX-11 and IX-13.

In the design of the canals the peak unit water requirement is adopted in connection with the area to be served.

In the design of the drains for this area, the peak unit drainage requirement, 5.9 lit/sec/ha is used. In determining the capacity of drains, the considerations are taken in two ways. Since the land which is located west of the sub-area D would be excluded from the Project due to various bad conditions for irrigation farming, the design capacity of the main drain situated in the sub-area D_3 is determined taking into account the retarding effect of land. Whilst as the secondary and the sub-secondary drains are situated within the land to be developed, no retarding effect could be taken into consideration.

The same procedure is applied with the sub-area B in this context. Fig. IX-15 shows the relationship between the discharge to be drained and the volume of water to be stored for the main drain. As the catchment of the drain at its head is 4,700 ha, the peak discharge would be as large as 28 m^3/sec, when the design value of 5.9 lit/sec/ha would be applied. According to Fig. IX-15, if the design discharge of the drain at the head would be fixed at 8 m^3/sec, the amount of water to be stored would be 2,200,000 m^3, which is translated as 1.8 m in water surface elevation or 0.8 in water depth. Since the roads which separate the Project area from the outside area is higher than 1.0 m, it is proposed to leave the area with the submerged condition.

As the outside water level fluctuates daily as discussed in Annex VII, the necessity and the location, if necessary, for installation of flap gates are examined in relation to the inside water level. Since the function of the main drain is to drain the water from the adjacent outside area through the sub-area D, no check structures would be required in principle. However, actually, two flap gate structures would be installed at the crossing point with the newly proposed road, because the water inside is checked by the road.

On the other hand, full-scale check measures would be required in the low lying land where the outside water level would become higher. A comparative study on the location of the flap gates is made whether to be located on the secondary drains, the sub-secondary drains or the tertiary drains.

In the sub-area D_1, there are two secondary drains. The design discharge of one of them is small, but the other is fairly large. For the former drainage system, the gates are proposed to be located at the outfalls of the secondary drain, because the submergence due to the closure of the gates is expected to be small and the construction cost would be the lowest. For the latter drainage system, it is suggested to install the check structures at the outfalls of the sub-secondary drains and at those of the tertiary drains, water of which would be drained directly to the secondary drain, because the only one check structure at the outfall of the secondary drain would cause very large inundation in the downstream reaches during the high tide. The number of the check structure would be one on the secondary drain, two on the sub-secondary drains and six on the tertiary drains.

Since the four secondary drains in the sub-areas D_2 and D_3 are small in their drainage areas, the design capacity would not be so large. In this context, it is proposed to install the check structures at the outlet of each secondary drain, except for the one which is located in relatively high land. The number of the check structure would be three on the secondary drains, one on the sub-secondary drain and four on the tertiary drains.

Fig. IX-18 shows the result of examination on the relationship among the drainage discharge through check structure, depth and duration of submergence in the paddy fields, and fluctuation of outside water level in the model area covered by the secondary drain SD-13. The figure indicates that maximum depth of submergence at the lowest paddy fields doesn't exceed 30 cm in depth.

Alignment in Sub-area E

The area to be developed in the sub-area E is characterized by the fairly large drainage basin behind it and the adjoining river, the Maluka, the capacity of which is too small to accommodate big flood discharge. In order to develop the proposed area, it would be necessary to protect the area against floods from the upper stretches and from the river. For these, two major drains are proposed to be constructed. One would be located on the eastern boundary of the area running from north to south joining the river Maluka. By this, the flood from the swamp area situated on the east would be intercepted. The other would be constructed along the river Maluka, function of which would be to intercept flood from the river and to collect water within the area. Both of the major drains would therefore be constructed with dikes to prevent from water intrusion.

The area thus protected against the outside water could be divided into two in view of the future drainage system. Sub-area E_1 is located along the river Maluka. The direction of flow is toward the river Maluka. Sub-area E_2 is located between the sub-area D and the sub-area E_1. The direction of drain is toward the river Barito.

In the sub-area E_1, there are several existing channels which are draining into the river Maluka. Three of them could be selected as sub-secondary drains in relation to the alignment of the tertiary canals and drains. These would be improved and connected with the major drain running along the river Maluka.

For the development of the sub-area E_2, two existing small rivers would be utilized by improving their courses and capacities. These would be connected with one of the tributaries of the river Barito. In addition to these, two sub-secondary drains would be aligned.

In order to irrigate the sub-area E, one of the secondary canals in the sub-area D would be extended to the south. Since the sub-area E is sub-divided into two, sub-areas E_1 and E_2, the secondary canal would be diverted into two directions.

The one which is for the use of the sub-area E_1 would be diverted to four sub-secondary canals as the sub-secondary drains would separate the area into four. Whilst the other one which is for the use of the sub-area E_2, would be diverted to three sub-secondary canals as the sub-secondary drains would separate the area into three.

The alignment of the secondary and the sub-secondary canals, and that of the secondary and sub-secondary drains are schematically shown in Fig. IX-11 and IX-13, respectively.

In the design of the canals, the peak water requirement is adopted in connection with the area to be served.

In the design of the drains for this area, the peak unit water requirement, 8.0 lit/sec/ha is used by multiplying the area to be drained. As the daily fluctuation of the outside water level due to tidal effect is very large, it seems indispensable to provide the drains with check devices. A comparative study on the location of the flap gates is made whether to be located at the outfalls of the secondary drains or at the outfalls of the tertiary drains. According to the study result, the location is proposed at the outfalls of the secondary drains, because if the gate would be located at each outfall of tertiary drains, the number, and hence, the costs of check structures would be very large. The number of the check structures on the secondary drains would be three in total.

IX.4 TERTIARY DEVELOPMENT

IX.4.1 General

This subsection deals with the basic consideration of the alignment of the tertiary canals and drains and downwards. Fig. IX-19 is referred to in this context.

The preliminary survey on the present farmers' status indicates that the average land holding is about 1 ha per household with the range between 0.5 ha and 2.0 ha (more than 80% of total farmers), and the area of one field patch is smaller than 1 ha. The basic area of 1 ha, which satisfies these two conditions, is adopted to make the alignment of the tertiary development.

IX.4.2 Shape and Size of Field Block

The shape and the size of the field block are planned to minimize the construction costs within the permissible limits of various conditions. Therefore, (1) the length of the longer side of the field block is planned to become equal to the permissible length of the field ditch, that is, 600 m, (2) the length of the shorter side of the field block, is determined to be 200 m taking into account the distance from the field ditches and minimum allowable drainage condition of each field patch, and (3) the size of the field block would become, in principle, 200 m x 600 m (12 ha).

Provided that the average land holding would be 1 ha, it is expected that 12 households would be accommodated in one farm block.

IX.4.3 Irrigation Canals

Since the topography of the project area is almost flat or very gentle in slope, it is suggested, in principle, to separate the canals for irrigation and drainage completely in order to make the perfect control of the water in the fields.

The alignment of the canal networks would be necessarily settled according to the shape and size of the field block. The distance between the turnouts on the sub-secondary canals would be 600 m and the distance between the adjacent two tertiary canals, which are diverted from the said turnouts and aligned in parallel each other, would be 600 m. The length of each field ditch would be 600 m with an interval of 400 m (see Fig. IX-19).

The type of tertiary canal downwards is planned to be of earth in order to minimize the construction costs. The design of the tertiary canal and the field ditch is made on the assumption that the rotational irrigation would be practiced within the tertiary unit. The capacity of them is designed to be the same throughout the whole line. The design discharge of the former would be 150 lit/sec taking into account the peak discharge which could be conveyed rotationally, whereas the latter 80 lit/sec, provided that the puddling work in the farm ditch unit (24 ha) would last for five days. The typical dimensions for tertiary canal and field ditch are shown in Table IX-7.

IX.4.4 Drainage Canals

The arrangement of drains would be decided according to the shape and size of the field block, and the alignment of the canal networks. The length of the field drain would become 600 m and the interval between the two, 400 m. Therefore, the field drains join the tertiary drain, which is aligned to make the right angle with them, in every 600 m.

The drainability of the project area is fairly poor on the whole. The depth of the groundwater table from the surface is assumed to be less than 1 m even in the non-irrigated period. During the irrigation period, it is expected that the groundwater would rise and be connected with the submerged water in saturated condition.

For this kind of fields, it is preferable for channels to have the capacity to drain the groundwater. However, as stated previously, in view of the high water surface elevation of the outside and construction costs, it is proposed to design the channels to have the capacity to drain the surface water only. The typical dimensions for tertiary drain and field drain are shown in Table IX-7.

IX.4.5 Farm Roads

The government intends to introduce farming machinery into the project area in future. It is, therefore, proposed to construct the farm roads in a minimum density or at least along the tertiary canals. The total width and height are to be 3.5 m and 0.5 m, respectively. Details will be described in the following section.

IX.4.6 Development in Each Sub-Area

Table IX-8 summarizes the length and density of tertiary canals, tertiary drains, field ditches, field drains and farm roads.

Sub-area A

The typical alignment of the sub-area A is seen in (1) of Fig. IX-20. The area is characterized by a small extent of land and fairly undulating topography on the south. These conditions necessarily restrict the alignment of tertiary and quaternary. The density of the tertiary canals, tertiary drains and the farm roads would then be small as seen in Table IX-8, whereas the density of the field ditches and field drains would be fairly high.

Sub-area B

The typical alignment of the sub-area B is seen in (2) of Fig. IX-20. Since the area is characterized by a large extent of land, very flat topography and few restrictions for the alignment of structures, it is proposed to make tertiary development regular as much as possible. The density of both tertiary canals and tertiary drains would be as high as 9.5 m/ha, and farm roads, 11.1 m/ha. The density of the field ditches and field drains is also high.

Sub-area C

The typical alignment of the sub-area C is seen in (3) of Fig. IX-20. The area is characterized by very flat topography and well developed existing channels. There are coconut trees and houses along these channels. Since these are proposed to be used as either tertiary drains or secondary drains, the alignment of tertiary canals and tertiary drains would be inevitably settled. The density of tertiary canals would be fairly high, due to complicated alignment, and that of tertiary drains, low. The quaternary system (field ditches and field drains) is restricted by the existence of coconut trees and houses. Sometimes it would be necessary to place these between two field ditches.

In this context, it is noted that the density of field ditch would be as high as 36.5 m/ha, whereas that of field drain is not so high. Since the length of farm roads corresponds with that of tertiary canal, the density would be necessarily high.

Sub-area D

The typical alignment of the sub-area D is seen in (4) of Fig. IX-20. The characteristics of the area is more or less the same with those of the sub-area C except for the density of the existing channels which is rather low. The fact would suggest that the alignment could be made more regular in shape because of less restrictions. Because of smaller number of existing channels which could be applied for the tertiary development, the density of new tertiary canals and tertiary drains would be higher than that in the sub-area C. On the other hand, as the number of coconut trees and houses are less, the alignment of quarternary system (field ditches and field drains) could be made easier. The density of both field ditches and field drains would not be so high.

Sub-area E

The typical alignment of the sub-area E is seen in (5) of Fig. IX-20. The characteristics of the area are similar with those of the sub-area D. The alignment and density of the tertiary drains would be more or less the same with those of the sub-area D. The same could be said true with the alignment and density of the field ditches and field drains.

IX.5 ROAD NETWORK

In the project area, there is one trunk road paved with asphalt which connects Banjarmasin with the Riam Kanan dam. Other than this, there are a few earth roads. One is located along the river Martapura, which links the sub-areas A, B and C with Banjarmasin. These two roads are linked with another road which separates the sub-area C from the sub-area B. Two local roads connect the sub-areas D and E with Banjarmasin. The proposed new road routes are shown in Fig. IX-21.

For the Project, three kinds of road would be proposed. The typical dimensions for the three are also shown in Fig. IX-21.

One is the main road which would be located along the main canal, along the main drains and on the connecting route which links the highway and the main drain in the sub-area B, and along the western border of the sub-areas C and D. This kind of road would be constructed in such a manner that after the stripping with a depth of 0.2 m, laterite pavement would be made with a thickness of 0.6m, the base course would be prepared with sand having a thickness of 0.1 m, and the gravel metalling would follow with a thickness of 0.2 m. The total road width would be 5.0 m and the effective width would be 3.5 m with the shoulder on both sides each having a length of 0.75 m.

Another is the secondary road which would be located along the secondary and the sub-secondary canals, and the connecting route between the secondary canals (or sub-secondary canals). This kind of road would be constructed in such a way that after the stripping with a depth of 0.2 m, laterite pavement would be made with a thickness of 0.6 m, the base course would be prepared with sand having a thickness of 0.1 m, and the gravel metalling would follow with a thickness of 0.2 m. The total road width would be 4.0 m and the effective width would be 3.5 m with the shoulder on both sides each having a length of 0.5 m.

The third is the farm road which would be located along the tertiary canals. This kind of road is constructed in such a manner that after the stripping with a depth of 0.2 m, laterite pavement would be made with a thickness of 0.5 m. No gravel and sand metaling is proposed to be made for this kind of road.

IX.6 LAND RECLAMATION

IX.6.1 Land Reclamation Plan

The possibility for the land reclamation of the shrub land into paddy fields is discussed in Annex V, and that of the rubber plantations is Annex VI. The total areas which could be reclaimed for paddy fields are 5,150 ha in gross. The distribution of the lands is tabulated as follows:

Present land use	Total area (ha)	Sub-area A (ha)	Sub-area B (ha)	Sub-area C (ha)
Rubber plantation	1,060	760	200	100
Shrub land	4,090	90	4,000	-
Total	5,150	850	4,200	100

The main works for the reclamation of the small-productive rubber plantations will consist of woods cutting, stumping, land clearing and firing. The density of rubber trees is about 500 nos./ha with the average diameter of 25 cm. As the land is very flat, no large scale cutting and banking of earth would be required.

The reclamation of the shrub lands would be easier than that of rubber plantations. The main works for the reclamation would be also woods cutting, stumping and firing. The density of trees is about 2,000 nos/ha with the average diameter of 5 cm. As the land is very flat, no cutting and banking would be required.

Taking into account the very flat topography for both rubber plantation and the shrub land, the size of the farm block would be designed as large as possible with the permissible range of various conditions. In view of drainage, the shorter side would be 200 m. In view of the maximum reach of a field ditch, the longer side would be 600 m.

The distance between the two adjacent farm roads would be 600 m or along the tertiary canals. No farm roads along the quaternary canal (farm ditch) would be considered in order to save construction costs and to increase cropping rate of reclaimed land.

Therefore, for the tertiary development of the newly reclaimed lands, the typical alignment of which is shown in Fig. IX-19 would be applicable.

An improvement of soil would not be necessary because the rubber plantation is favourable in view of soil texture quality, effective soil depth and soil acidity. The soil in the shrub area which is proposed to be developed is also favourable provided that drainage would be practiced properly because the soil-drying effect would proceed the decomposition of humic soil.

IX.6.2 Procedure of Construction

Woods cutting for rubber plantation would be made by manpower. Since the diameter of woods is fairly big, a combination of chain-saws and axes would be applied. Cutting would be made at about 20 to 30 cm above the ground.

For the reclamation of the shrub land, bush clearing would be made either by manpower or machines. Since the precondition is firing, cutting would be made as low as possible by means of scythe, bush cutter, chainsaw, etc.

The above works would be followed by firing. The small trees and the bushes which are cut would be well dried during the dry season. The area for one firing would range between 3 ha and 5 ha.

Stumping or uprooting would be made after firing. There would be several methods for stumping such as by manpower, by animal power, by machine, by explosive, etc. It seems practical to use either bulldozer or rakedozer taking into account the present vegetation of rubber trees and shrub.

Exclusion of wood roots would be practiced after the above works. It is usually very common to use rakedozers. The distance between the lines of excluded wood roots would be expected to be 100 m taking into account the proposed layout of farm block.

Finally plowing would be practiced. The objective is to make the cultivable land. Although there are several ways, it seems most preferable to use breakers or bottom plows.

IX.7 PLAN OF PUMP IRRIGATION

As stated in the previous sections, there are several restriction in developing the river Maluka, which is characterized by its little water resources and its very gentle slope. The restrictions are: (1) there are no favourable locations for dam construction for regulating the river discharge because of very flat topography of both of the river banks throughout the whole stretches, (2) a large scale gravity irrigation would not be practiced because of very gentle slope and low elevation of the river, and (3) the sea water intrudes fairly upper reaches of the river because it directly drains to the Java sea.

The land which could be taken up for irrigation development in view of soil, vegetation, topography and drainability is located on the downstream stretch of the river Maluka excluding the unsuitable land which faces to the river Barito. The study on the gravity irrigation for this area indicates that the location of the headworks would be very far from the area to be irrigated. The catchment area at the proposed headwork (near to the crossing point with road connecting Banjarbaru with Bati Bati) would be as small as 385 km^2 and the discharge would be 2.4 m^3/sec with the probable recurrence interval of once five years. The area which could be irrigated by gravity would be very small, and the construction costs for canal would be very high. Therefore the pump irrigation at the lower reach (downstream of the confluence with the river Panggaaungan) is recommended because of much larger catchment basin and discharges.

In the selection of the possible pump sites, the following dominant factors are taken into consideration at the stage of reconnaissance. They are: (1) the canal alignment from the pump site would be favourable for irrigable area, (2) the site would have an enough space for the construction of pump station, (3) the site would provide a short distance between pump station and a head reach as much as possible, (4) the site would have a suitable foundation for construction of pump station, and (5) the topographical and soil conditions for tide gate construction which would be installed downstream of the pump station would be preferable.

Our field reconnaissance made in due considerations of the above shows that there are two possible sites. One is located at immediately downstream of the confluence, and the other about 800 m further downstream of the said site. Provisionally the former site would be recommendable because the canal alignment from the pump site and the tide gate installation would be more preferable despite the fact that the other conditions are more or less the same.

There are big differences among the seasonal water requirements. To solve this unfavourable factor for the selection of size and number of pump units, it is conceivable mathematically to select an assortment of different capacities of pumps. However, such selection would cause difficult problems on operation and repair. In selection, therefore, several units with the same capacity is recommendable. Taking into account the water requirements for the rainy season paddy which would be averagely one-quarter of the peak water requirements for the dry season paddy, four units of pump are proposed to be installed.

The types of pump applicable for a large discharge pump station are divided into three kinds, namely, axial flow type, mixed flow type and volute type having different operating characteristics, respectively. In this context, mixed flow type is proposed to be adopted. This pump would be suitable for medium delivery head pump station, in general, the applicable delivery head ranges from 3 m to 20 m.

The selection of either horizontal shaft or vertical shaft is made considering the topography of the pump station and conveniency for maintenance. As a result, the pumps could be designed for the vertical shaft type.

Since electric power is not supplied around the proposed pump station at present, the preliminary design is proposed to be made by adopting diesel engines as motive power until the future electric power supply system would be planned.

Thus, the salient features of the pump station would be as follows:

Total head:	4 m
Type of pump:	Vertical shaft mixed flow type
Number of pump:	4 nos.
Discharge:	80 m^3/min each
Shaft power:	70 kW
Rotational speed of pump:	300 rpm
Diameter of pipe:	800 mm
Motive power:	Diesel engine
Type of pump house:	Double floor type

The preliminary design of the pump station is made as shown in Drawing No. 12.

The installation of tide gates on the river Maluka at the downstream of the pump station seems essential as the saline water intrusion would occur during the low discharge, and flood water would have to be discharged without causing any backwater effect in the upper reaches. In addition, the consideration is also taken to the navigation of the river.

For the above, the features of the tide gates would be as follows:

Type of gate:	Roller gate
Size and number of gates:	8 m (H) x 8 m (W) x 3 nos.
Size and number of gates for navigation	6 m (H) x 4 m (W) x 2 sets for up and downstream

The preliminary design of the tide gates is made as shown in Drawing No. 12.

IX.8 PRELIMINARY STUDY ON BY-PASS STRUCTURE ON RIAM KANAN DAM

IX.8.1 General

As described in Annex III, the amount of irrigation water from the Riam Kanan dam depends entirely on the discharge released from the power plants in the dry season. No outlet facilities for the specific irrigation purpose are provided in the dam. This means that would the output of power be stopped due to the occurrence of troubles with the power plants, transmission line, and other related structures, especially in the dry season, no irrigation water would be supplied to the project and a considerable damage to crops would occur.

In order to expect the possible highest return from the project throughout the year, it would be better to construct the specific intake or by-pass structure in due consideration of the unforeseeable troubles of the power generation. This section deals with the study on the technical possibilities of the construction of such intake facility, though which is still very preliminary, in this context.

IX.8.2 Alternative Plans on By-pass

Though several alternative plans on the construction of by-pass are considered, following two plans would be proposed in view of technical and economic possibilities. The first alternative is to draw the water directly from the penstock as shown in Fig. IX-22 (Case-I). The second alternative is to release the water directly from the reservoir to the Aranio river through the channel of emergency spillway, located in the left bank of the reservoir, as shown in Fig. IX-23 (Case-II).

In the design of the by-pass, 42 m^3/sec of discharge at the low water level, 52.707 m in elevation, in the reservoir would be adopted on the assumption that all of the three generating plants would be out of operation. The detailed explanation on the above two cases are as follows:

Case-I

As seen in Fig. IX-22, the steel conduit with a diameter of 1,700 mm is connected to the penstock between the surge tank and spherical branch. The diverted water is released into the tail race of dam from the Hollow jet valve which is provided at the outlet of the conduit.

The construction itself would be simple, and time required for the construction would be less than one month. However, partial interruption of electric supply would be required during the construction in Banjarmasin and other districts.

Case-II

As seen in Fig. IX-23, the intake structure for by-pass is constructed at just near-by the dike for the emergency spillway. The diverted water is released into the Aranio river, which joins the Riam Kanan river at the immediate downstream of the dam, through the tunnel and outlet open channel. The outlet open channel is constructed by deepening and widening the existing water way for the emergency spillway.

The power operation would not be disturbed during the construction of by-pass. However, the complicated constructions would be required as compared with that of Case-I, and the volume of soil and rock excavation would be considerably large.

IX.8.3 Cost Estimate and Recommendation

The preliminary cost estimate for both cases are made, and total cost are as follows:

Case	Cost (US$1,000)
I	1,400
II	3,700

The breakdown of cost estimates are shown in Table IX-9.

In due consideration of the technical and economic advantage, it is recommended to take up the Case-I as the most suitable plans for the construction of by-pass.

Fig. IX-1 Rating Curve at Weir Site

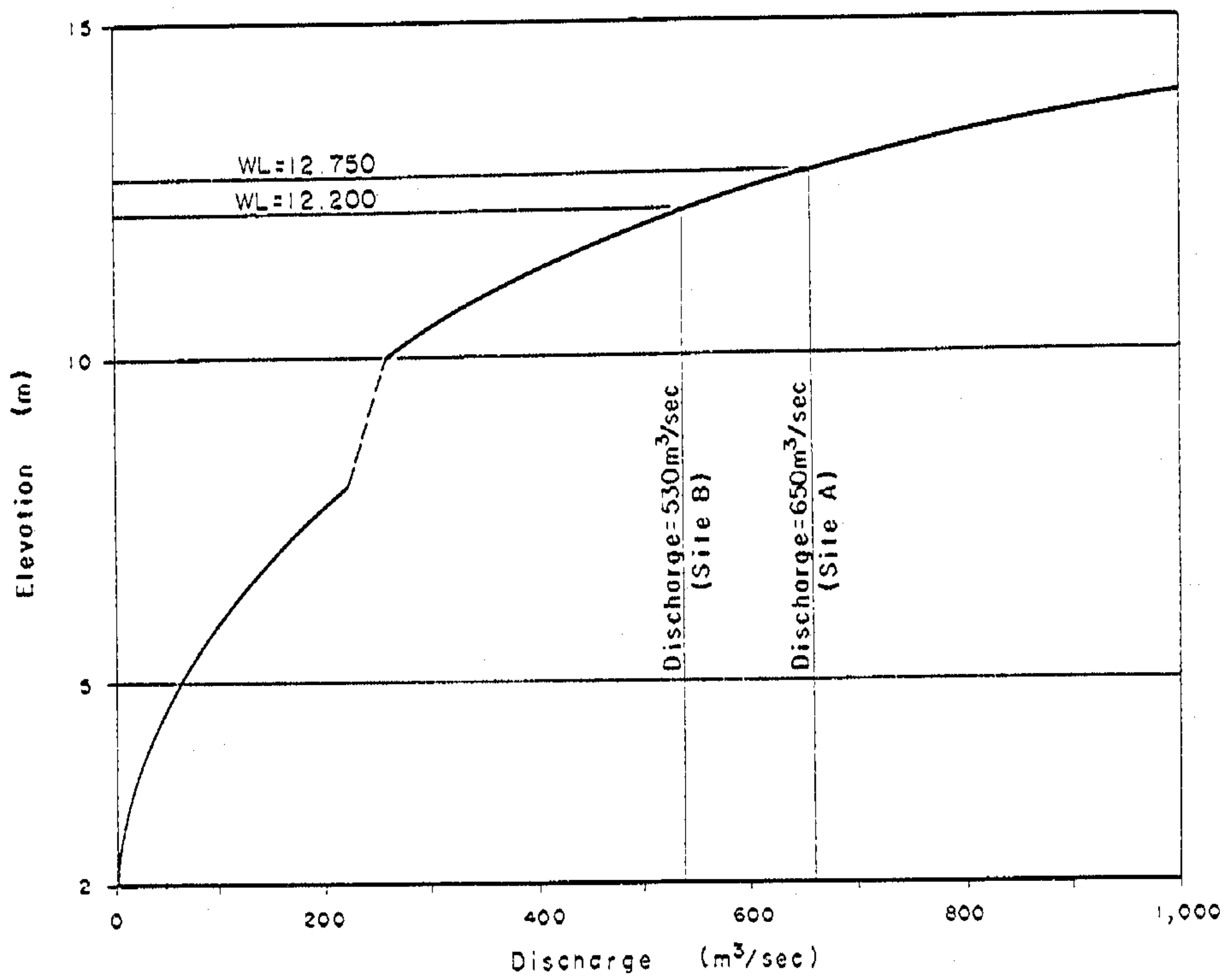


Fig. IX-2 Regulated Flow and Reservoir Water Level

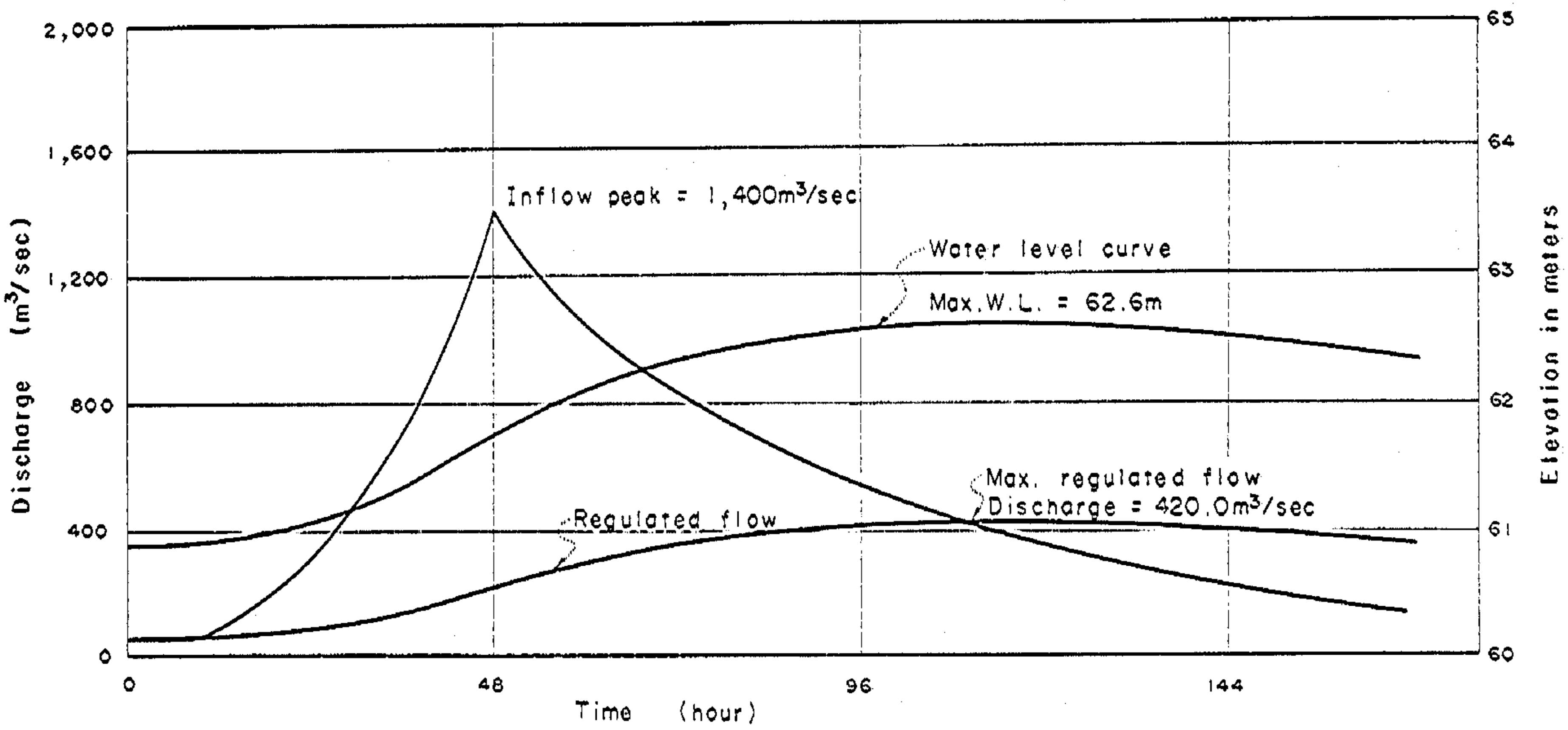


Fig. IX-3 Backwater Curve (Discharge = 500 m^3/sec ⚠1)

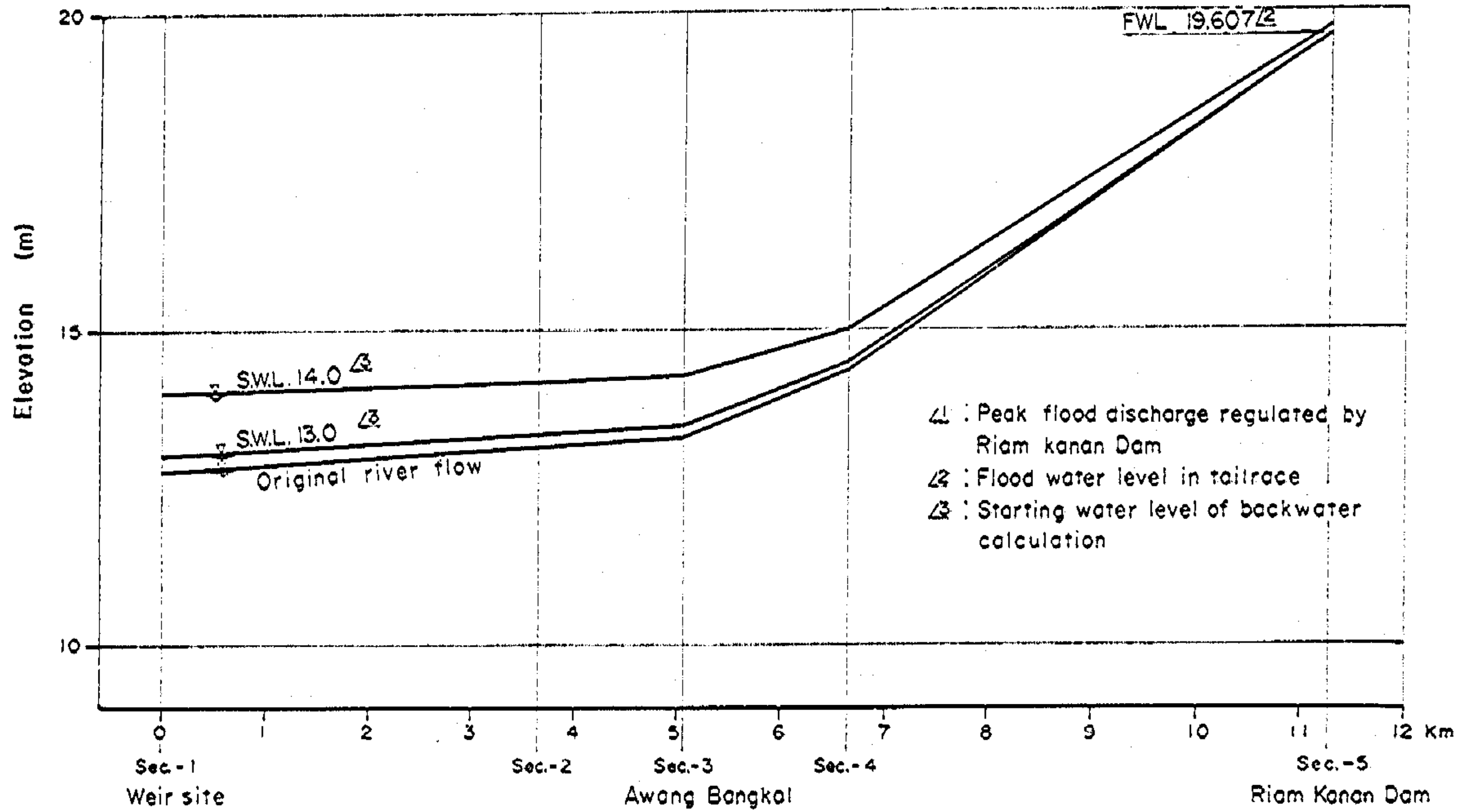


Fig. IX-4 Backwater Curve (Discharge = 87 m³/sec △1)

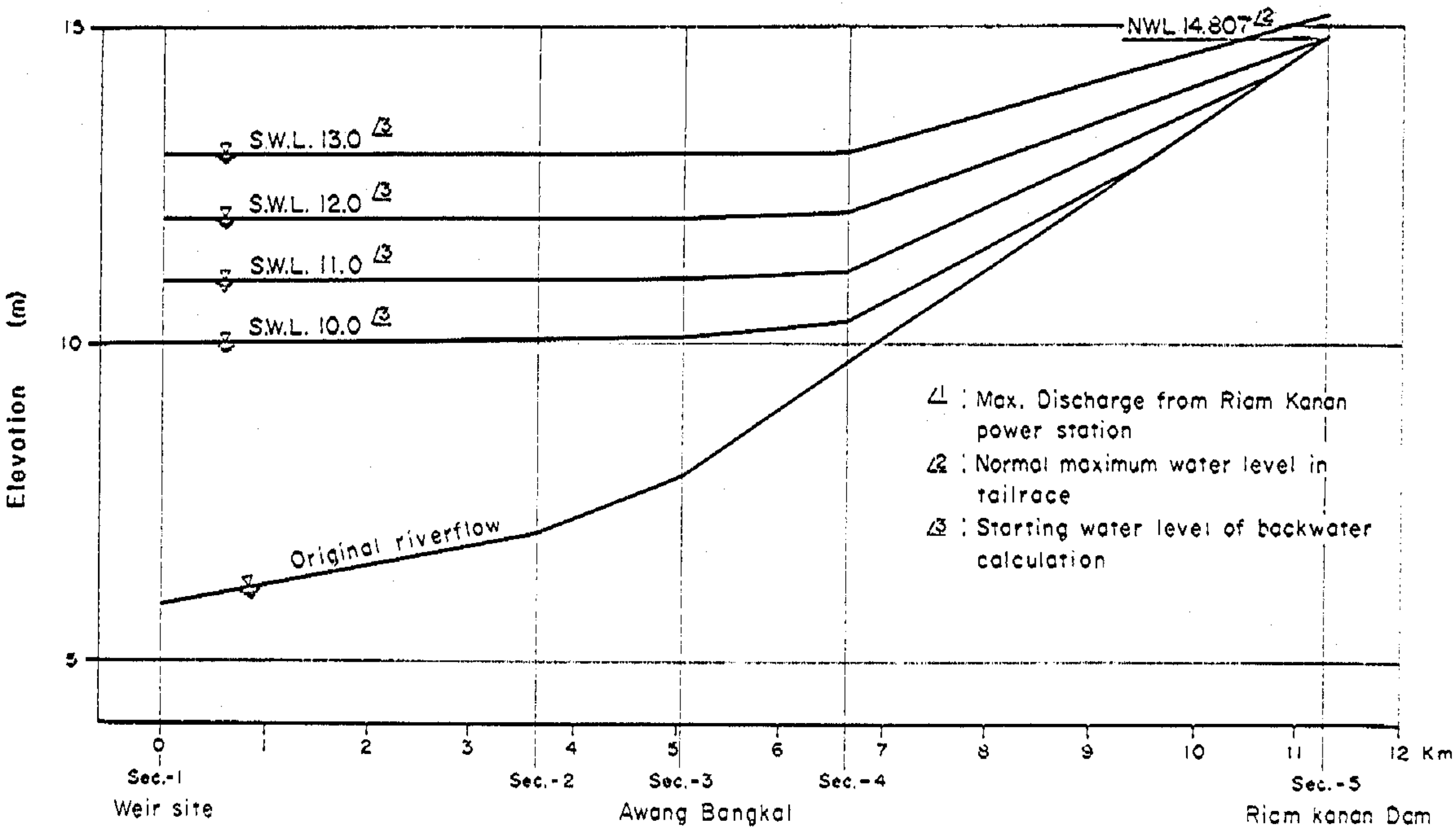


Fig. IX - 5 Storage Volume Curve

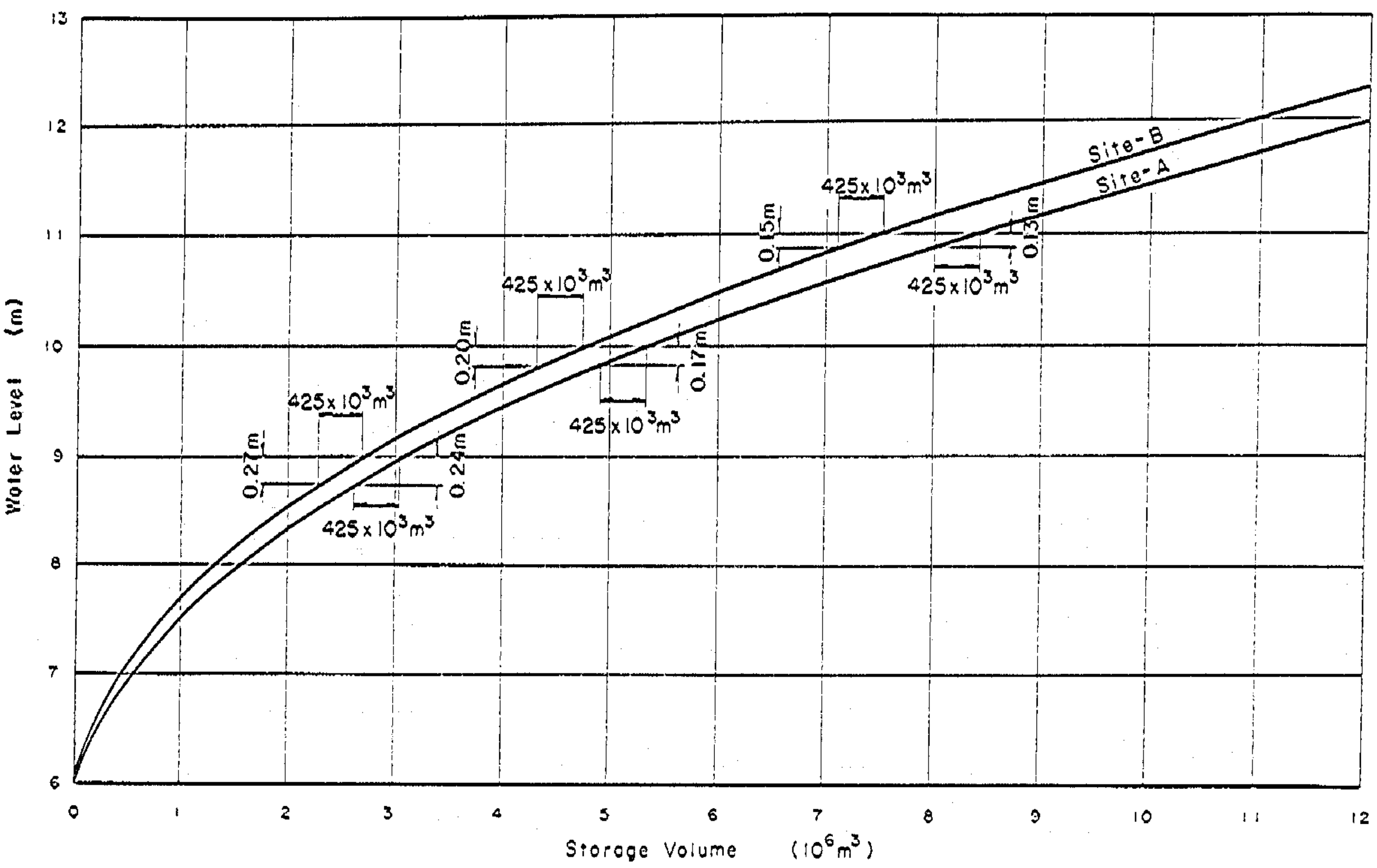


Fig. IX-6 Alternative Headworks, site-A and site-B