8.3. Cropping Plan of the Irrigation Projects Relating with the Lebir Dam Project

8.3.1. Irrigation Project Area

The discharge in the lower reaches of the Kelantan River will undoubtedly increase as a result of discharge into the upper reaches of the river from the Lebir dam, chiefly by operation of the power station.

It is anticipated that the seven irrigation projects proposed in the Kelantan River Basin Study (KRBS) carried out by ENEX and the two on-going KADA and Kemasin-Semerak projects will benefit from the aforesaid increased discharge.

Irrigable Area of Nine Projects:

The following table gives the names of the nine projects and their respective irrigable areas:

Name of Project	Irrigable Area (ha)	Status
1. KADA	31,800	On-going Project
2. Kemasin & Semerak	15,000	On-going Project
3. North Lemal Phase I	3,644	Included in KRBS, ENEX
4. Ulu Lemal	3,806	ditto
5. Sg. Bagan	1,620	ditto
6. Tasek Garu	18,650	ditto
7. Sg. Sat	1,822	ditto
8. Panyit	1,234	ditto
9. Kusial	1,250	Not included in KRBS
Total	78,826	

The locations of these nine projects are shown on Fig.8-1.

Note: 1. The KADA area is mentioned based on the KADA II Improvement Project Report. The area of 31,800 ha consists of 20,090 ha in Kemubu & Salor and 11,710 ha in Lemal, Alor Pasir & Pasir Mas. The area reduced by six percent for on-farm works is estimated at 29,893 ha.

2. Kemasin-Semerak 15,000 ha is based on the data of DID, Kelantan. Area proposed in the Kemasin-Semerak Agricultural Development Project Report is 14,815 ha.

3. The areas from No.2 to No.9 are referred to from the data of DID, Kelantan.

Development Plan of Nine Projects:

A long term development plan has been prepared by KRBS, ENEX for the nine projects which will benefit from increased discharge in the Kelantan River through water released from the Lebir Dam. (See Table 8-2) Table 8-2 includes an implementation schedule for the Lebir Dam Project prepared for this study. KADA II is provided based on the Improvement Project Report. The schedule for Kemasin-Semerak provided in the ENEX plan has been revised. It is assumed that the North Lemal Phase II, Ulu Lemal and Sg. Bagan will be delayed in realization by about ten years in comparison with the ENEX schedule.

Hence, the project benefits will be generated in the staged development as shown in Table 8-2.

Acreage of Existing and Proposed Paddy Fields in the Project Area:

The acreage of paddy field in the six irrigation projects excluding KADA, Kemasin-Semerak and North Lemal Phase II was roughly measured on the Map of Kelantan with a scale of 190,080 : 1 (air-photo in 1973 and reprint in 1982). The area (excluding that of Tasek Garu) roughly corresponds to the information provided by DID Kelantan. There is, however, a big difference in the acreage between Tasek Garu : 18,650 ha according to DID Kelantan and 6,400 ha measured on the map.

It is assumed that the Tasek Garu project area with 18,650 ha includes a new reclamation area of about 12,250 ha. The area cultivated with paddy, which is reported to be by district-by-district in the statistics, SEPU, is classified into two areas: the inside irrigation scheme and the outside irrigation scheme (DALAM/LUAR PERAIRAN). The study is made on the basis of consideration that the former is irrigated paddy and the latter is rainfed paddy.

The paddy field acreages by each project are as follows.

- Paddy Field Acreage -

(unit: ha)

Project	Without Project Irrigated Paddy	Rainfed Paddy	Total	With Project Irrigated Paddy
1. KADA	31,800	-	31,800	31,800
2. Kemasin-Semarak	15,000	-	15,000	15,000
3. North Lemal Phase II	1,094	2,550	3,644	3,644
4. Ulu Lemal	1,110	2,696	3,806	3,806
5. Sg. Bagan	1,130	490	1,620	1,620
6. Tasek Garu	1,730	4,670	6,400	18,650
7. Sg. Sat.	1,270	552	1,822	1,822
8. Panyit	860	374	1,234	1,234
9. Kusial	310	940	1,250	1,250
Total	54,304	12,272	66,576	78,826

8.3.2. Prospect on Demand and Supply of Rice in Kelantan State

During six (6) years from 1980 to 1985 the production of rice in Kelantan State was in range from about 10 percent to 12 percent against the national total as shown in the following table. Based on this share, the study on the prospect of the paddy production in Kelantan was executed.

- Share of Rice Production in Kelantan
State to National (Malaysia) Production -

Item	Unit	1980	1981	1982	1983	1984	1985
Malaysia (A)	1,000 t	1,318	1,303	1,213	1,117	1,010	1,269
Kelantan (B)	1,000 t	132.1	130.3	95.6	131.4	78.5	116.1
B/A	%	10.0	10.0	7.9	11.8	7.8	9.1

The production area of paddy in the Kelantan River basin is divided into the granary areas and the area outside the granary area. The former are KADA and Kemasin-Semerak. Demand and supply of rice in KADA and Kemasin-Semerak can be estimated by using the respective plans made in the Feasibility Studies. The production of paddy to be prospected on and after 2006 is assumed to be the same as that in 2005.

The rice or paddy requirements in Kelantan State are estimated by using the forecasted population and apparent rice consumption per head. In comparison of the rice requirement in the State with the paddy production in the granary area, the latter is expected to exceed the former as shown in the following table.

However, when the paddy production in Malaysia meets with a lean year in the future, import of rice should be increased. Maintaining paddy production in Kelantan State and increasing paddy export to other States are necessary, in order to secure the national self-sufficiency level of rice supply targeted by NAP. It is assumed that the surplus level of paddy production in Kelantan State required by other States' consumers is dependent on the share of paddy production in Kelantan State against the National production.

The following cases are understandable with respective interpretation.

In case of the share of 10 percent, the rice requirement in Kelantan State will be satisfied by production in KADA and Kemasin-Semerak Project Area and others. Hence, supplemental products will not be necessary.

In case of the share of 11 percent, supplemental products will not be necessary in 2000 and 2005.

In case of the share of 12 percent and over, supplemental paddy will be necessary. The supplemental paddy should be supplied from production by the seven (7) new irrigation projects excluding KADA and Kemasin-Semerak Projects.

- Prediction of Rice Requirement in Kelantan State -

Item	Unit	1985	1990	1995	2000	2005	2010
1. Population 1/	Million	1.026	1.174	1.334	1.509	1.692	1.877
2. Apparent Rice Consumption per Head 2/	kg	108	100	100	100	95	95
3. Total Apparent Rice Consumption	1,000 t	110.8	117.4	133.4	150.9	160.7	178.3
4. Consumption in Terms of Paddy 3/	1,000 t	170.4	180.6	205.2	232.2	247.2	274.3

Note: 1/ Annual growth rate of population is assumed as follows. 1986-'90 2.7%, 1991-'95' 2.6%, 1996-2000 2.5%, 2001-2005 2.3%, 2006-2010 2.1%.

2/ Apparent rice consumption per head after 1995 is assumed to be 5% larger than that in the projection in Malaysia.

3/ Milling rate is 65%.

- Prediction of Paddy Production in Kelantan State -

Item	Unit	1985	1990	1995	2000	2005	2010
1. Paddy Production, Malaysia	1,000 t	1,952	2,200	2,354	2,637	2,785	3,091
2. Share of Kelantan	1,000 t						
10%	"	178.4	220	235	264	279	309
11%	"	178.4	242	259	290	306	340
12%	"	178.4	264	282	316	334	371
13%	"	178.4	286	306	343	362	402
14%	"	178.4	308	330	369	390	433
3. Paddy Production in the Granary Area (KADA + Kemasin-Semerak)	"	159	198	232	271	287	287
4. Paddy Production in the Non-granary Area	"	19.4	21	22	23	25	26
5. Paddy Production in Kelantan State (3 + 4)	"	178.4	219	254	294	312	313
6. Supplemental Paddy (2 - 5) Share of Kelantan							
10%	"	0	1	0	0	0	0
11%	"	0	23	5	0	0	26
12%	"	0	45	28	22	22	58
13%	"	0	67	52	49	50	89
14%	"	0	89	76	75	78	120

8.3.3. Analysis of Crop Damage

(1) Area Cropped and Harvested with Paddy

(a) Rainy Season Paddy in Kelantan State

Trend of Cultivated Area:

The trend of paddy cultivating acreage for 15 years from 1970/71 to 1984/85 is shown in Table 8-2 in Appendix. The maximum cultivation acreage in the main season was 70,389 ha in 1972/73 and 70,286 ha in 1974/75. After 1974/75, the acreage decreased to 43,602 ha in 1981/82 and 33,189 ha in 1984/85. The decrease of cultivated area in 1981/1982 and 1983/84 was notably large. Tumpat, Pasir Mas and Kota Bharu recorded the smallest cultivated area.

The cultivated area in the KADA Project and the other areas from 1974/75 to 1984/85 are shown in Tables 8-2 and 8-3 in Appendix. Since the last five years, the area cultivated in the main season outside KADA has decreased gradually. (See Tables 8-4 and 8-5 in Appendix)

Trend of Harvested Area:

The cultivated area mentioned above has been decreased due to the flood, insects and drought.

The trend of the area cultivated with paddy over the past 15 years is shown in Table 8-2 in Appendix and indicates that the average rate of damage per year was 5.5 percent of the total cultivated area.

The statistics reveal that large areas were damaged by floods from 1970/71 to 1974/75, but only small areas of damage after 1975/76 (except 1983/84). The change in cropped area shows the same trend as the above.

Since 1975/76, the cropped area has decreased continuously although the damage due to floods remained small.

The main reasons are considered as follows:

i. Local farmer's enthusiasm for paddy cropping has declined due to their anxiety about flooding.

ii. Flood damage to nursery beds has resulted in spoiling seedlings for transplanting.

iii. There was a shortage of supplemental seedlings for replacing damaged seedlings.

iv. Drought in April is prone to cause delay to transplanting of the second crops and also to the harvesting. Hence, the proper opportunity for nursery and transplanting for the first crop is lost. The period for such works, therefore, unavoidably falls on the flood season.

According to Table 8-2 in Appendix, the main season paddy tends to be damaged by drought. The area damaged in the main season is greater than that damaged in the off-season as shown in Table 8-7 in Appendix. The main reason for this seems that each growing period (panicle, booting and flowering) encounters shortage of irrigation water from March to April. As paddy plants need much irrigation water in the above growing stages, the water shortage damages the main season paddy cropping. The drought damage incurred in 1981/82 was the largest in acreage. The damaged area occupied 16.8 percent of the cropped area. Damage in the districts of Pasir Puteh, Pasir Mas, Tumpat and Bachok reached 38.1 percent, 27.8 percent, 23.3 percent and 10.7 percent, respectively. (See Table 8-6 in Appendix).

(a) Off Dry Season Paddy in Kelantan State

Trend of Cropped Area:

The trend of area cropped and harvested with paddy for 14 years from 1972 to 1985, is shown on Table 8-7 in

Appendix. An annual fluctuation is found more stable than that of main season paddy. Around 86-96 percent of the area cropped with off season paddy in Kelantan State is occupied by KADA Project Area. Hence, the trend in Kelantan State can be considered to show a similar trend to KADA Project Area if irrigation facilities are provided. The cropped area decreased from 1980 to 1983, and the decrease by each district is shown in the following table. The maximum rate is found in Tumpat.

- Trend of Area Cropped with Off Season Paddy by District -

District	Average 1980-1983(A) (ha)	Average 1973-1979(B) (ha)	(A) / (B) (%)
Tumpat	1,329	2,665	49.9
Pasir Mas	4,049	5,050	80.2
Kota Bharu	9,981	11,839	84.3
Bachok	3,176	3,532	89.9
Pasir Puteh	1,169	1,313	89.0
Others	693	853	81.2
Kelantan	20,397	25,252	80.8

Trend of Harvested Acreage:

About 2.8 percent of the cropped area was damaged in the last 14 years. The most serious damage was caused by insects, accounting for about 60 percent of the total damaged acreage.

Drought damage amounted to only about five percent. Flood damage amounted to 25 percent. Insect damage occurred every year. The districts suffering most heavily from floods are Bachok, Pasir Mas and Kota Bharu.

(c) Trend of Cropped Area in KADA Project Area

The gross area of KADA Project Area is 60,438 ha. The land use is classified as follows:

Paddy field 33,651 ha (55.7 percent), rubber 7,714 ha (12.8 percent), coconuts 310 ha (0.5 percent), mix-plantation 11,500 ha (19.0 percent), others including forest and swamp 6,673 ha (11.0 percent).

The trend of cropped area over the last ten years in KADA Project Area is as follows.

i. The cropped area decreased to 54,119 ha in 1974, 44,341 ha in 1978 and 26,227 ha in 1983. (Refer to Table 8-9 in Appendix).

ii. The annual cropping intensity also decreased by as much as 161 percent in 1974, 132 percent in 1978 and 78 percent in 1983. (Refer to Tables 8-9 and 8-10 in Appendix).

iii. The trend of reduction of cropped area by sub-areas is shown in Fig.8-2. The trend indices are calculated by setting the value for 1974/75 at 100. The reduction in area in Kemubu/Salor is gradual, while that in Lemal/Alor Pasir is steep and irregular. Acreage in Pasir Mas has been decreasing since 1979/80.

iv. The annual cropping intensity for the last ten years is shown in the following table. Years with an intensity of over 150 percent appear five times each in Kemubu/Salor and Pasir Mas and nil in Lemal/Alor Pasir.

Annual Cropping Intensity	Kemubu/Salar	Lemal/Alor Pasir	Pasir Mas
above 150 %	5	0	5
above 140 %	6	1	5
above 130 %	7	1	6

Annual Trend of Cropped Area of Paddy (1974-1975 =100%)

(2) Analysis of Paddy Yielding Factors

According to the KADA statistics, paddy yield by sub-project area is shown as follows.

- Paddy Yield -

(tons/ha)

	Main Season				Off Season		
Crop Year	Kemubu Salor	Lemal Alor Pasir	Pasir Mas	Crop Year	Kemubu Salor	Lemal Alor Pasir	Pasir Mas
1974/75	1.96	1.46	1.95	1975	2.41	2.37	2.91
1975/76	2.21	1.68	2.49	1976	2.74	2.94	2.68
1976/77	2.04	1.62	2.36	1977	2.94	3.62	3.26
1977/78	2.59	1.72	2.32	1978	3.08	2.76	3.18
1978/79	3.35	2.92	3.26	1979	3.50	2.76	3.57
1979/80	3.97	2.72	3.15	1980	4.15	2.78	2.97
1980/81	2.92	1.63	2.76	1981	2.69	2.27	1.96
1981/82	2.44	2.50	3.44	1982	3.62	3.53	3.08
1982/83	3.32	2.66	2.65	1983	4.04	3.31	3.41
1983/84	2.11	2.02	3.42	1984	3.30	1.86	1.99

Source: Statistical Digest (Kumusan Beraugkan), KADA

The cropping schedule for paddy in the main and off seasons from 1980 to 1987 was obtained from the KADA office. Fig.8-4 in Appendix shows these schedules.

The paddy growing stages such as rooting, panicle and booting stage, require much irrigation water. The growing stages for four years from 1980 to 1983, were confirmed based on Fig.8-4 in Appendix.

The correlation between rainfall/pumping discharge and paddy yield was studied for each growing stage. Table 8-11 in Appendix shows this original data on the Kemubu-Salor sub-project area. The correlation coefficient between paddy yield and rainfall/pumping discharge was studied on a monthly basis by using the original data as follows:

The correlation between the paddy yield and the monthly rainfall in the panicle stage shows a relatively high coefficient.

- Correlation of Yield -

Relation	Correlation Coefficient	Sample
Monthly rainfall in rooting stage - yield	- 0.56	7
Monthly rainfall in panicle stage - yield	0.78	7
Monthly rainfall in booting stage - yield	0.88	7
Monthly pumping discharge in rooting stage - yield	0.78	7
Monthly pumping discharge in panicle stage - yield	0.67	7
Monthly pumping discharge in booting stage - yield	0.41	7

Note: The sample is the total of the values in both main and off seasons.

The relationship between paddy yield for 1975 to 1984 (10 years) and the three factors of: rainfall at the Project Area, pumping hours, and water level of Kelantan River at the Pumping Station was studied. In particular, the relation between Kemubu-Salor and Pasir Mas sub-project area was qualitatively studied by applying these factors for the years in question.

The following table shows reasonable correlation.

- Relation in Panicle Stage and Booting Stage -

Pump Station	Year	Month	Rainfall (mm)	Pumping Hour (hours)	Minimum Water Level (m)	Paddy Yield (ton/ha)
Kemubu Salor	1981	Jul.	128.0	2,535	5.06	2.69
	1979	Jul.	233.6	2,248	5.15	3.50
	1981	Aug.	35.3	2,171	5.00	2.69
	1979	Aug.	134.7	1,921	5.09	3.50
Pasir Mas	1981	Jul.	128.0	1,444	1.50	1.96
	1980	Jul.	223.0	1,444	2.22	2.97
	1981	Aug.	35.3	1,433	1.86	1.96
	1980	Aug.	268.5	1,154	2.65	2.97

8.3.4. Cropping Plan

(1) Cropping Pattern

Paddy production in Kelantan State will be mainly dependent on the future output from the granary area of Kemubu and Kemasin-Semerak. On the other hand, the higher the share of paddy production in Kelantan State rises, the more new supply of paddy from the non-granary area will be necessary.

The paddy field of seven (7) irrigation projects is related to the Lebir Dam Project and amounts to 19,776 ha. This does not include Kemubu and Kemasin-Semerak. These paddy fields consist of 38 percent irrigated and 62 percent rainfed fields.

The discharge at the lower reaches of the Kelantan River will undoubtedly increase as a result of discharge into the upper reaches of the river from the Lebir Dam. Conversion of the rainfed paddy of 12,272 ha into the irrigated paddy will be planned in order to use effectively the increased discharge available. This plan will be important to promote the revitalization of agriculture in the coastal region. According to the Fifth Malaysia Plan, the existing paddy land outside the granary areas will be gradually phased out and replaced by other more remunerative crops. Diversification and intensification in cropping pattern are one of the most important strategies with the future agricultural development in the coastal region.

The cropping plan of the seven (7) projects is studied by the following two systems.

1st - Mono cropping system of paddy for both seasons; main and off.

2nd - The upland crops and partial paddy are cropped in the off or dry season in an irrigated rotational cropping pattern. Paddy in the crops in the main or rainy season.

The second cropping pattern is similar to that preferred in the ENEX (KRBS) Main Report Vol.4 Agriculture. "Indicator crops" are preferred in the 30 years long term plan of ENEX Report with crops of paddy, tobacco, groundnut, maize, soybean, sorghum, vegetable, pasture, sugarcane, cashew nut, rambutan, durian, duku, rubber, coconut and oil palm. In this study, the six "Indicator crops" selected for the seven (7) projects are paddy, tobacco, groundnut, maize, sorghum and vegetable. The profit from soybean is negative or low according to the production cost data of KADA. The ENEX Report says that the upland crops preferred in the

Tasek Garu project are tobacco, groundnut, maize, soybean, sorghum, vegetable, pasture and sugarcane. Sugarcane occupies about 40 percent of the total acreage of these crops. Paddy is not selected. Such singularity in the plan for Tasek Garu has been noted with respect to the present study but, since information regarding sugarcane was unavailable during the field study, paddy was selected as one of the "Indicator crops". Sorghum was selected in the Tasek Garu project only. The ENEX Report does not include the Kusial project. The planned cropping pattern in this project area is the mono-cropping system of paddy, chosen for its convenience.

(2) Mono-Cropping System of Paddy

The cropping ratio of paddy for the new irrigation projects was planned as follows.

Cropping ratio without the Project

The trend of the area cropped with paddy for 15 years from 1970/71 to 1984/85 by each district shows a decrease in the last five years (1980/81 through 1984/85). The annual average cropping ratio for these five years is taken as the cropping ratio without the Project. (Refer to Tables 8-3 and 8-8 in Appendix).

- Cropping Ratio of Paddy without the Project -

unit: %)

Season	North Lemal	Ulu Lemal	Sungai Bagan	Tasek Garu	Sungai Sat	Panyit	Kusial
Main	0.29	0.40	0.76	0.40	0.76	0.76	0.40
Off	0.12	0.17	0.07	0.17	0.07	0.07	0.06
District Concerned	Tumpat	Pasir Mas Tanah Merah	Machang	Pasir Mas Tanah Merah	Machang	Machang	Tanah Merah

Cropping ratio with the Project

The trend of the area cropped with paddy for ten years from 1970/71 through 1979/80 by each district, is steady as shown in the following table. The ratio of off-season paddy is low except for Kota Bharu.

In this study, the ratio of Kota Bharu is assumed as the cropping ratio with the Project. The target of the cropping ratio is fixed at 90 percent for main season, 80 percent for off-season and 170 percent per year.

- Cropping Ratio from 1970/71 to 1979/80 -

(unit: %)

Season	Tumpat	Pasir Mas	Kota Bharu	Bachok	Pasir Puteh	Machang	Tanah Merah
Main	80	89	92	78	96	91	70
Off	26	26	75	31	11	2	11

The cropping ratio by year with the Project is planned as follows.

(unit: %)

Season	1st Year	2nd Year	3rd Year	4th Year	5th Year
Main	80	80	90	90	90
Off	60	70	70	80	80
Total	140	150	160	170	170

In using the cropping ratio mentioned above, the area cropped with paddy for the new irrigation projects is proposed as the following table.

- Acreage of Paddy without the Project -

(unit: ha)

Project	Main Season Irrigated Paddy	Main Season Rainfed Paddy	Main Season Total	Off Season Irrigated Paddy	Off Season Rainfed Paddy	Off Season Total
North Lemal Phase I	317	740	1,057	131	306	437
Ulu Lemal	444	1,078	1,522	194	460	654
Sungai Bagan	859	372	1,231	79	34	103
Tasek Garu	692	1,868	2,560	228	572	800
Sungai Sat	965	420	1,385	89	39	128
Panyit	654	284	938	60	26	86
Kusial	124	376	500	19	56	75

Paddy cultivated area assuming implementation of the project is planned as follows.

- Acreage of Paddy with the Project -

(unit: ha)

Project	Season	1st Year	2nd Year	3rd Year	4th Year	5th Year
North Lemal Phase I	Main	2,915	2,915	3,280	3,280	3,280
	Off	2,816	2,550	2,550	2,915	2,915
Ulu Lemal	Main	3,045	3,045	3,425	3,425	3,425
	Off	2,284	2,664	2,664	3,045	3,045
Sg. Bagan	Main	1,296	1,296	1,458	1,458	1,458
	Off	972	1,134	1,134	1,296	1,296
Tasek Garu	Main	14,920	14,920	16,785	16,785	16,785
	Off	11,190	13,055	13,055	14,920	14,920
Sg. Sat	Main	1,458	1,458	1,640	1,640	1,640
	Off	1,093	1,275	1,110	1,458	1,458
Panyit	Main	987	987	1,110	1,110	1,110
	Off	740	964	964	987	987
Kusial	Main	1,000	1,000	1,125	1,125	1,125
	Off	750	875	875	1,000	1,000

(3) Irrigated Rotational Cropping System

Acreage cultivated with upland crops in the new six (6) projects excluding the Kusial project is forecast as follows.

Maize:

Maize is one of the imported foods. According to the Food Balance Sheets, 1978-81 Average, 1984, FAO, 95.7 percent of the domestic maize supply is imported. The acreage planted with maize, therefore, should be increased in future.

The acreage in Kelantan State shows a tendency to decrease, though the area in 1977 was 1,367 ha.

- Acreage Cropped with Maize in Kelantan -

Year	1981	1982	1983	1984	1985
Area (ha)	1,089	987	533	676	591

The annual supply of maize per capita in Malaysia is 2.3 kg according to the aforesaid Food Balance Sheets. Estimation of maize supply available in Kelantan State is studied at 2,360 tonnes in 1985 and 4,317 tonnes in 2,010 using the above value of 2.3 kg. The acreage necessary to meet the requirements is estimated at about 760 ha for 1985 and about 1,400 ha for 2010 on the basis of an average yield of 3.097 tonnes per ha. According to the ENEX Report, the acreage of maize planned for 30 years is projected at 4,360 ha.

- Planning Acreage of Maize in Kelantan -

(unit: ha)

Year	Without Project	With Project
1985	591	591
2010	1,400	4,360

Groundnut:

The area planted with groundnuts in Kelantan has been decreased very much recently. The maximum acreage, however, was 3,500 ha in 1979.

- Acreage Cropped with Groundnut in Kelantan -

(unit: ha)

Year	1981	1982	1983	1984	1985	Average
Area	2,715	3,273	1,313	718	601	1,387

The area for groundnut, which was projected in the ENEX Report is 27,089 ha in the coastal region and 7,590 ha in the six (6) project area associated with the Lebir Dam Project. According to the cropping pattern in the Kemasin-Semerak Report, the area for groundnut is planned by about 4,100 ha.

In this study, the acreage forecast for 2010 in Kelantan would be about 30 percent of that of ENEX.

- Planning Acreage of Groundnut in Kelantan -

(unit ha)

Year	Without Project	With Project
1985	601	601
2010	1,390	8,100

Tobacco:

The trend with crop acreage of tobacco from 1976 to 1985 in the Kelantan State is studied using the following linear regression equations.

Virginia tobacco $Y = 6,872 + 198X$
Local tobacco $Y = 168 + 14.6X$

In using these equations, the area planted with tobacco in 2010 can be estimated at 11,600 ha of Virginia tobacco and 530 ha of local tobacco, and the total will be 12,130 ha. Of this, the area planned for the new six (6) projects related to the Lebir Dam Project amounts to 3,415 ha. In the ENEX Report, the prospected area planted with tobacco was 13,863 ha. On this, the forecast area related to the Lebir Dam Project was 4,840 ha. The planned acreage of tobacco is projected as follows in consideration of forecast mentioned above.

- Planning Acreage of Tobacco in Kelantan -

Year	Without Project	With Project
1985	6,985	6,985
2010	12,130	13,555

Sorghum:

The acreage planted with sorghum in the Compartment No.4 of the Tasek Garu project is planned as 4,570 ha according to the ENEX Report. In this study, the Tasek Garu project was only considered to plant 4,570 ha of sorghum after 2006.

Vegetable:

The trend of crop acreage of vegetable from 1976 to 1985 in Kelantan State is shown by the linear regression equation, Y = 1,331 + 47.2X. Using this equation, acreage of vegetable without the Project would be planned as follows.

- Planning Acreage of Vegetable without the Project in Kelantan -

Year	1985	1990	1995	2000	2005	2010
Crop Area	1,545	1,567	1,803	2,015	2,130	2,130

The annual supply of fresh vegetables per capita in Malaysia is 23 kg according to the Food Balance Sheets. The supply of fresh vegetables in Kelantan State is estimated at about 43,700 tonnes in 2010, on the basis of 23 kg per capita.

The acreage necessary to meet the requirements projected is estimated at about 3,970 ha in the year of 2010, on the basis of an average yield of 11 tonnes. In this study, the planning acreage of vegetable in the year of 2010 was projected as 80 percent of the necessary acreage.

- Acreage Planned for Vegetable with Project in Kelantan -

(unit: ha)

Year	Without Project	With Project
1985	1,545	1,545
1986	2,130	3,179

The acreage planted with upland crops mentioned above are classified into the coastal area and the hinterland area.

- Diversification Crop in 2010 -

- Without Project -

(unit: ha)

Item	Maize	Ground-nut	Tobacco	Sorghum	Vegetable	Total
A. Coastal Region	1,200	1,190	17,945	-	1,944	16,279
A-1. Projects Associated with Lebir Dam	600	680	3,415	-	584	5,339
A-2. KADA, Kemasin-Semerak & Others	540	510	8,530	-	1,360	10,940
B. Hinterland Region	200	200	185	-	186	771
C. Kelantan State	1,400	1,390	18,130	-	2,130	17,050

Note: Areas in the Projects associated with the Lebir Dam are considered as areas planned generally in the districts where the new six (6) projects are located.

- With Project -

(unit: ha)

Item	Maize	Ground-nut	Tobacco	Sorghum	Vegetable	Total
A. Coastal Region	4,111	7,900	13,370	4,570	2,993	32,944
A-1. Projects Associated with Lebir Dam	1,511	3,290	4,840	4,570	1,633	15,884
A-2. KADA, Kemasin-Semerak & Others	2,600	4,610	8,530	-	1,360	17,100
B. Hingerland Region	250	200	185	-	186	821
C. Kelantan State	4,361	8,100	13,555	4,570	3,179	33,765

Note: Areas planted in A-1 are projected as those planted in the six (6) project areas.

The present acreage of upland crops planted on paddy field is not available from the general statistics. The present cropping calendar for short term crops like chili, tobacco, groundnuts, vegetables, cucumbers and maize is shown in Fig. 8-1-1 in Appendix extracted from the KADA Report.

In consideration of the facts mentioned above, the acreage of diversified crops planted in the paddies of the new six (6) projects was assumed as 2,670 ha or 50 percent of 5,339 ha projected in the year of 2010 in the districts where the new six (6) projects are located. This acreage is assumed as that without the Project in 2010.

The area planted with each crop by seven (7) projects are shown in the following table.

- Planning Acreage with the Project
in the year of 2012 -
(R : Rainy season, D : Dry season)

(unit: ha)

Project	Season	Upland Crop	Paddy	Total
North Lemal	R	-	3,280	3,280
	D	2,574	341	2,915
Ulu Lemal	R	-	3,425	3,425
	D	2,704	341	3,045
Sg. Bagan	R	-	1,458	1,458
	D	1,162	134	1,296
Tasek Garu	R	-	16,785	16,785
	D	7,816	7,104	14,920
Sg. Sat	R	-	1,640	1,640
	D	951	507	1,458
Panyit	R	-	1,110	1,110
	D	637	350	987
Kusial	R	-	1,125	1,125
	D	-	1,000	1,000
Total	R	-	28,823	28,823
	D	15,844	9,777	25,621

8.4. Water Requirement

8.4.1. Peak Water Requirement of Irrigation Projects

According to the KRBS (ENEX 1977), the irrigation water requirements of the fields along the Kelantan River are forecast to be 112.3 CMS (131,955 ha) by the year of 2000 as shown in Table 8-1. The JICA Study Team collected up-dated data from the local authorities, DID Kelantan and KADA. Based on such data, future peak demand can be projected to be 127.6 CMS (78,826 ha), of which 101.1 CMS (54,250 ha) are required for the Kelantan main stream, and 26.5 CMS (24,576 ha) for tributary streams. The total requirements are calculated by aggregating the requirements for each of the nine projects. However, it should be noted that peak demand for water does not occur simultaneously in the KADA and Kemasin-Semerak project areas. The peak demand for the two areas occurs in April and May respectively.

The peak water requirement occurs in April and the requirement in the future is forecast to be 111.6 CMS (127.6 CMS - 16.0 CMS).

8.4.2. Water Requirement along the Kelantan River by Uses

DID Kelantan has summarized the water requirements by user along the Kelantan River as follows.

1. Irrigation water supply	90 CMS
2. Domestic and industrial water supply	20 CMS (Kemasin-Semerak Study estimated at 5 (CMS)
3. Residual flow for saline abatement	80 CMS
Total	190 CMS

The respective water requirements of 71.6 CMS for KADA and 16.0 CMS for Kemasin-Semerak will not exceed the allowable limit of 90 CMS for irrigation water supplied from the Kelantan River discharge.

However, as water requirements for the seven new projects gradually increase in future, the gross requirements for irrigation water will exceed the above required discharge limit.

The supply of water for the seven new projects will be ensured by the increasing discharge of the Kelantan River following completion of the Lebir Dam. However, the discharge of 85 CMS to 100 CMS in the total requirements for domestic and industrial water supply and the residual flow for saline abatement must be maintained. This will limit the discharge available to maintain the irrigation water supply to the new projects.

Hence, a water balance to ensure stable supply of irrigation water for the seven new projects should be carried out within the scope of the total basin-wide water balance study on the assumption that both Dabong and Lebir Dams proposed in KRBS are constructed.

For the purposes of this study, assumptions regarding seven new projects will be made based on the anticipated increasing discharge in the Kelantan River after completion of the Lebir Dam.

Thus, the water balance analysis made in this report is not comprehensive but partial only.

8.4.3. Water Operation Study

(1) The development plan of the nine irrigation projects mentioned above was assumed tentatively based on the ENEX KRBS as shown in Table 8-2. The water requirements for these projects are expected to increase in stages. Hence, the following four cases have been studied.

Irrigable Area (ha) -

Project	Case 1	Case 2	Case 3	Case 4
KADA	31,800	31,800	31,800	31,800
Kemasin-Semerak	15,000	15,000	15,000	15,000
North Lemal Phase I	-	3,644	3,644	3,644
Ulu Lemal	-	3,806	3,806	3,806
Sg. Bagan	-	1,620	1,620	1,620
Tasek Garu	-	-	18,650	6,400
Sg. Sat	-	-	1,822	1,822
Panyit	-	-	1,234	1,234
Kusial	-	-	1,250	-
Total	46,800	55,870	78,826	65,326

Note: Tasek Garu, 6,400 ha in Case 4 is explained in Section 8.3.1. Each case is assumed to be planned with paddy.

(2) An increase of discharge in the Kelantan River by water release from the Lebir Dam is classified into the following three cases.

A : without the Lebir Dam
B : release of 70 CMS
C : release of 80 CMS

(3) The following water operation for ten cases was studied.

	Case 1	Case 2	Case 3	Case 4
A	0	0	0	0
B	-	0	0	0
C	-	0	0	0

(4) A basic condition of the study

(a) Basic data (MCM)

: - Discharge for ten-days at the Guillemard Bridge of the Kelantan River from 1967 to 1984.

: - Unit water requirement (Lit/s/ha) over ten-day periods from 1967 to 1984.

(b) Total irrigation water duty (MCM)

Unit water requirement x irrigable area of the four cases.

(c) Discharge of river water (MCM)

Discharge of river water by ten-days at the Guillmard Bridge for each of the ten cases mentioned above.

(d) Required discharge of the Kelantan River excluding the irrigation water requirement.

Discharge for domestic/industrial water supply
... 20 CMS or 5 CMS
Residual flow for saline abatement
... 80 CMS
Total ... 85 CMS or 100 CMS

(e) Remaining discharge of the Kelantan River (MCM)

Discharge of river water over ten-day periods (1967 to 1984)

- irrigation water duty over ten-days (1967 to 1984) = remaining discharge in the Kelantan River

The pumping sites for nine irrigation projects are proposed in different locations. However, for the sake of convenience this difference should be disregarded.

(f) Calculation of the unit water requirement over ten-day periods as mentioned above is made based on the cropping calendar used in the KADA Improvement Project Study. The difference in peak period of irrigation water requirement between the KADA Kemubu Irrigation Scheme and Kemasin-Semerak Project is not taken into account for the purpose of calculating unit water requirements. In March and April, especially, the remaining discharge at the Kelantan River, which is, calculated in the above (e) is lower than the correct figures.

To maintain a high level with accuracy, the remaining discharge by ten-day period which will be less than the reserved discharge of Kelantan River (85 CMS and 100 CMS), should be revised in consideration to take account of the difference in irrigation water requirement between April and May at Kemubu Irrigation Scheme and Kemasin-Semerak Project.

(g) The water operation study conducted in the KADA II Improvement Project has determined the discharge at Kemubu pumping station for the first ten-day period in September, 1975 to the third ten-day period in August, 1976. The discharge at the Guillemard Bridge is different from that at the Kemubu pumping station.

This difference can be explained by the local inflow between the Guillemard Bridge and the Kemubu pumping station site. The remaining discharge less than the reserved discharge (85 CMS and 100 CMS) should be revised to take account this local inflow.

(h) According to the KADA II Report, the average water balance between pumping and natural flow for Kemubu and Lemal Alor Pasir Irrigation Schemes is estimated at about 90 percent and 68 percent respectively by pumping. The other necessary water is supplied by return flow and some other rivers. In this water operation study, the water balance for the project area will be disregarded.

(5) Analysis

Case 1 (46,800 ha, without the Lebir Dam):

The water shortage, which is the shortfall of discharge below the reserved discharge of 85 CMS, occurred twice in 17 years (in 1969 and 1983). The limiting factor of 100 CMS for the reserved discharge makes the water balance study very difficult.

Shortage of irrigation water occurred as follows:

March, 3rd ten-days: three times in 16 years (1969, 1781, 1982)

April, 1st ten-days: twice in 17 years (1968, 1983)

April, 2nd ten-days: three times in 17 years (1968, 1969, 1983)

April, 3rd ten-days: twice in 16 years (1969, 1983)

Case 2 (55,870 ha):

Without the Lebir Dam:-

Irrigation water shortage occurs from the second 10-days of March to the first 10-day period of May. The maximum frequency in occurrence is three times each in 16 years and 17 years.

In the case of the 85 CMS reserved discharge:-

70 CMS and 80 CMS released from the Lebir Dam will not result in any shortage in irrigation water.

In the case of the 100 CMS reserved discharge:-

70 CMS and 80 CMS released from the Lebir Dam will cause water shortage to occur once in 16 years, on the basis of past records.

<u>Case 3 (78,826 ha)</u>:

Without the Lebir Dam:-

Irrigation water shortage will occur more frequently than in Case 2. The maximum frequency will be four times in 16 years or 17 years respectively during the second 10-day period of March through the second 10-day period of May.

In case of the 85 CMS reserved discharge:-

70 CMS and 80 CMS released from the Lebir Dam will cause water shortage to occur once in 16 or 17 years.

In case of the 100 CMS reserved discharge:-

Under water release of 70 CMS, irrigation water shortage will occur from the first ten-days of March to the first ten-days of May. Occurrence frequency will be only once in 16 or 17 years. Past exceptions were two occasions in 1968 and 1969 when shortage occurred on the second 10-days of April in 17 years.

Release of 80 CMS will result in a lesser frequency of occurrence than in the case of 70 CMS.

Case 4 (65,326 ha):

Without the Lebir Dam:-

Occurrence of irrigation water shortage will be more frequent than Case 2. The maximum frequency in occurrence will be three times for 16 or 17 years, from the first ten-days of March to the first ten-days of May.

70 CMS and 80 CMS release from the Lebir Dam will cause water shortage once in 17 years, on the basis of past records.

The probability analysis in the KADA II Improvement Project Report was adopted to obtain ten-year return period rainfall data. This data is taken into consideration in deciding the design unit water requirements for the main and secondary canals and pumps. When this basis for planning is applied to the above case study, it is anticipated that the release of water exceeding 70 CMS from the Lebir Dam will stabilize the supply of irrigation water to the nine irrigation projects each, in Case 2, Case 3 and Case 4, except for the 100 CMS reserved discharge in Case 3.

The following table shows the frequency of occurrence of ten-day periods of irrigation water shortage when the remaining discharge is below the reserved discharge of 85 CMS or 100 CMS.

- Occurrence of Shortage of Irrigation Water Supply -

Case	Discharge released	Discharge reserved	April 1st-10 days	April 2nd-10 days	April 3rd-10 days	May 1st-10 days
2	70 CMS	85 CMS	-	-	-	-
	70	100	-	-	1/16	-
	80	85	-	-	-	-
	80	100	-	-	1/16	-
3	70	85	-	1/17	1/16	1/17
	70	100	2/17	3/17	1/16	1/17
	80	85	-	1/17	1/16	-
	80	100	1/17	1/17	1/16	-
4	70	85	-	1/17	1/16	-
	70	100	-	1/17	1/16	-
	80	85	-	1/17	1/16	-
	80	100	-	1/17	1/16	-

Note: Detail is shown by Appendix Table 8-1.

8.5. Agricultural Benefit

8.5.1. Benefit in the KADA II Project Area

(1) The west bank area of the KADA II Project Area, or the Lemal & Alor Pasir and Pasir Mas sub-project areas, includes the area covered by the Lemal Irrigation Component under the loan from the World Bank and the area under the implementation of KADA II. The original implementation schedules of both the above projects showed the construction of the former from 1976 through 1984 and that of the latter from 1982 through 1987. The present situation is that the former is already completed, while the works of the latter has not been commenced yet. However, it is included in the Fifth Malaysia Plan (1986-1990).

According to information obtained from KADA, some of the paddy fields at the terminal of the irrigation canal system are located at high elevation. When the pumping discharge is small and the actual water level in the irrigation canal is lower than the designed water level, the irrigation water can not reach the fields at the high elevation in the furthest canals downstream of the system. It is reported that severe water shortage has taken place in an area of about 1,000 ha located at the southern part of Jong Bakar in Lemal area, and 540 ha, (about 30 percent) of Pasir Mas sub-project area. These areas are among areas chronically damaged in drought due to their comparatively high elevations.

Even if the Lemal Irrigation Component and the pumping facilities in KADA II were constructed in the near future, shortage of irrigation water at the most distant area will continue when the available discharge from the Kelantan River is small in quantity.

Hence, the areas of 1,540 ha (1,000 ha + 540 ha) mentioned above can be assumed as those which will benefit little by the discharge from the Lebir Dam.

According to the results of water operation study, irrigation water shortage in Case 1 will occur from the second ten-days of March to the third ten-days of April. Such water shortage is an obstacle to expansion of cropping area of off-season paddy. This problem should be solved by water to be supplied from the Lebir Dam. Therefore, an increase in the area of 1,540 ha for the off-season paddy can be assumed as one of the benefits generated by the Project.

(2) The areas damaged by drought in Kelantan State for the ten years from 1975 to 1984 are estimated at 1,172 ha on an annual basis for the main season, and 35 ha for the off season. About 60 percent of the damaged areas is assumed to be in the Kemubu sub-project area, east bank of KADA II.

- Drought - Damaged Area in Kelantan State -

Item	Main Season Paddy	Off Season Paddy
Statistical Year	1974/75 to 83/84	1975 - 1984
Annual Average Damaged Area	1,172 ha (Kemubu 693 ha)	35 ha (Kemubu 29 ha)
Maximum Damaged Area	7,319 ha (1981/82)	129 ha (1982)

Note: Statistics of Paddy

The main season paddy in 1981/82 suffered extreme damage as recorded in the above table. The period from the third ten-days of February to the third ten-days of March in 1982 fully covered the panicle, booting and flowering stage. Both rainfall and discharge at Kelantan River were insufficient for paddy planting.

After completion of the KADA II Improvement Project, the period of panicle and booting stages will be shifted to December or January under the cropping calendar planned in the hydrological analysis for irrigation of the KADA II Project. Hence, it is reasonable to assume that drought damage to the main season paddy mentioned above will be mitigated after completion of the KADA II Project.

On the other hand, Case 1 of the water operation study shows that shortage of irrigation water occurs during the second 10-days of March and the third 10-days of April. Such shortage will be an obstacle to expansion of the cultivated area for off season paddy in the Kemubu sub-project area. It is difficult to forecast these damaged areas.

Consequently, an annual increase of the areas protected from drought, amounting to 722 ha (693 ha of the main season paddy and 29 ha of the off season paddy), can be assumed as one of the benefits from the Project.

8.5.2. Benefit in the Kemasin-Semerak Project Area

The irrigation water for the Kemasin-Semerak Project area will be supplemented through the Kemubu pumping station at the Kelantan River. The main canal of the Kemubu system will be widened to convey this water more than seven kilometers. The water will then be carried to the Ketereh River (the upper reaches of the Kemasin River), and flow into the Melor (or Gunong) which will transport the water to the lower Kemasin and towards the Semerak River.

Discharge by the Kelantan pumping station for the Kemasin-Semerak Project is compared with that for the Kemubu as shown in the following table. The peak water demand for each project does not overlap.

- Kelantan Pumping Discharge-

(unit: CMS)

Project	Jan.	Feb.	Mar.	Apr.	May	Jun.	Jul.	Aug.	Sep.	Oct.	Nov.	Dec.
1. Kemasin 1/	-	-	0.63	0.47	4.71	3.67	2.36	-	-	-	-	-
Semerak	-	-	1.35	1.50	9.03	9.03	4.44	-	-	-	-	-
Total	-	-	1.98	1.97	13.74	12.70	6.8	-	-	-	-	-
Include loss	-	-	2.28	2.27	15.80	14.61	7.82	-	-	-	-	-
2. Kemubu 2/	-	-	39.9	39.9	26.7	26.7	26.7	-	39.9	39.9	20.7	-

Note: Maximum discharge 15.80 CMS = 16.0 CMS

Source: 1/. Hydrological analysis, Appendix Report, Kemasin - Semerak Agricultural Development Project

2/ KADA office, March, 1987.

According to Case 1 in the water operation study made in Section 8.4.3., every ten-day period can secure sufficient water over 80 - 100 CMS in May when the Kemasin-Semerak Project Area is wholly in its peak water demand. Therefore, the benefit is not considered to be generated by discharge released from the Lebir Dam.

8.5.3. Benefit in the Seven New Irrigation Projects

The water operation study referred to in Section 8.4.3. is a partial water balance study in the Kelantan River Basin. Hence, a stable supply of irrigation water for the seven new irrigation projects will not always be available. It will depend upon discharge from the Lebir Dam.

In this study, however, it has been assumed that a stable supply of irrigation water for the seven projects can be realized by discharge secured from the Lebir Dam, with the prerequisite of securing the necessary discharge of 85 CMS to 100 CMS. As a result, the benefit of the seven projects can be estimated on the basis of each irrigable area in Case 2, Case 3 and Case 4.

The main factors giving benefit are an increase in irrigable areas by conversion of rainfed paddy fields through the supply of additional water. Positive expansion of irrigable area is limited to Tasek Garu Area.

An increase in the paddy yield by additional irrigation water supply is not considered, in order to keep the benefit moderate. The paddy yield per hectare is estimated in consideration of the trend of natural increase.

8.5.4. Target Yield by Crops

Paddy:

The estimated annual average growth rate of the paddy yield per hectare in Kelantan State is found to differ, corresponding to the use of two kinds of basic data, which are mutually inconsistent. In order to make the target yield moderate, the annual average growth rate is taken to be 1.2 percent for the main season and 0.7 percent for the off season. (refer to Table 8-12 and 13 in Appendix).

- Annual Average Growth Rate of Paddy Yield in Kelantan -

Data Source	Main Season	Off Season
1. Average yield by district, SEPU 1976 to 1985	1.2 %	1.7 %
2. Agricultural Basic Statistics, Department of Agriculture (Various Issues), 1970 to 1980	1.9 %	0.7 %

The present yields per hectare for irrigated paddy cropping and rainfed paddy cropping were estimated based on the statistics by each district available from SEPU. The future annual yields are estimated by use of the annual growth rate mentioned above (refer to Table 8-14 in Appendix). The estimated yields are 3.76 tons per hectare for the main season paddy and 3.96 tons for the off season. These are the same figures as that of the KADA II Improvement Project. The statistical yield of SEPU is reported by hectare harvested. The harvested area ratio to cultivated area is estimated at 96 percent for the main season and 98 percent for the off season on the basis of the annual rates for 11 years from 1970 to 1980 by Agricultural Basic Statistics. The project benefits are estimated based on the harvested area by using the above ratio.

Upland Crop:

The target yields for diversified crops are planned based on the production cost reports by crops offered from KADA and ENEX Report, Vol. 4: Agriculture.

- Target Yield by Upland Crop -

(unit: kg/ha)

Crop	Without Project	With Project
Maize	2,500	3,125
Groundnut	2,500	3,750
Tobacco (Crude leaf)	6,100	7,930
Sorghum	3,750	4,500
Cabbage	12,500	15,000
Kailan	9,500	11,000
Long Bean	10,000	12,000
Petola	12,000	15,000
Mustard	13,000	16,000
Broccoli	6,250	7,500
Okra	12,500	15,000

8.5.5. Commodities Prices

Evaluation of the Project's benefit is made applying the market price, economic price and accounting price. The project evaluation using the market price is said to be the financial analysis or cash flow analysis and that using the economic price or accounting price is said to be the economic analysis.

The market prices and the economic or accounting prices of paddy represented by main crops for this study are estimated as follows.

Market or Financial Prices of Paddy:

(a) Paddy Statistics (Peranakaan Padi), Malaysia, 1984

Monthly average price of paddy in Kelantan State

Long grain	0.50M$/kg
Medium grain	0.46M$/kg

(b) Data, SEPU, Kelantan

Purchased price of paddy, LPN, Kelantan
46.30 - 49.60 M$/100 kg, Jul. 16, 1980
Plut 3.31 M$ (Jan. 10 to Oct. 16, 1980)
(Subsidy) 16.54 M$ (Oct. 17, 1980)

(c) Economic Report, 1986/87 (Malaysia, MOF)

Subsidy to paddy price 49.61 M$/100 kg

(d) In case of an economic evaluation with the market price, 500 M$/ton will be used.

Economic Price of Paddy:

The estimated economic price of paddy (based on the paddy prices forecasted by the World Bank), are as follows (details are shown in Table 8-17 in Appendix).

- Price Structure for Rice -

Description	1986		2000	
	Financial	Economic	Financial	Economic
US$/mt				
1. Export price of Thai, 5% broken rice, FOB, Bangkok	177	177	216	216
2. CIF of rice, Klang port	189	189	225	225
M$/mt (US$1 = M$2.50)				
3. CIF of rice, Klang port	473	473	563	563
4. Wholesale price of rice, Kota Bharu	643	587	733	677
5. Ex-mill price of rice, KADA area	638	583	728	673
6. Paddy equivalent price	415	379	473	437
7. Farm gate paddy price	366	335	424	393

For any economic evaluation study with the economic or accounting price, 424 M$ or 393 M$/ton in the year 2000 shall be used.

Market or Financial Prices of Upland Crops:

Market prices of upland or diversification crops are planned based on the production costs report by crop, offered from KADA, and data collected from SEPU, Kelantan.

- Market or Financial Prices of Upland Crop - M$/kg

Maize	0.64	Groundnut	1.0	Tobacco	0.67
Sorghum	0.60	Cabbage	0.50	Kailan	0.50
Long Bean	0.60	Petola	0.40	Mustard	0.60
Broccoli	1.00	Okra	0.60		

Economic Price of Upland Crops:

The boundary price obtained by using a general conversion factor of 0.8 is applied for the economic price of upland crops.

- Economic Price of Upland Crop - M$/kg

Maize	0.51	Groundnut	1.0	Tobacco	0.67
Sorghum	0.48	Cabbage	0.50	Kailan	0.50
Long Bean	0.48	Petola	0.40	Mustard	0.60
Broccoli	0.80	Okra	0.60		

Shadow Rate of Unskilled Labour:

The shadow rate of the agricultural wage was studied in the KADA II Improvement Project Report. The shadow rate estimated by using the opportunity cost curve of agricultural wage was about $5.5 to 6.1 although the average market rate was $10. Hence, the shadow rate of unskilled labour for this study is set at 60 percent of the average market rate.

8.5.6. Crop Production Cost

The production costs by crop per hectare are estimated as follows based on the production costs report offered from the Agricultural Department, KADA.

- Production Cost - M$/ha

Crop		Market/Financial Price	Economic Price
Paddy			
Transplant	(W/o Pro.)	1,297	934
Direct. seed	(W/Pro.)	1,136	856
Maize		1,337	815
Groundnut		1,826	1,331
Tobacco		4,047	2,670
Sorghum		1,212	951
Cabbage		4,692	3,638
Kailan		4,526	3,085
Long bean		5,264	3,858
Petola		4,281	3,216
Mustard		4,535	3,093
Broccoli		4,562	3,559
Okra		5,043	3,731

8.5.7. Incremental Quantity of Products

The agricultural benefit is evaluated in five cases. The project areas defined by respective four (4) cases correspond to each case from 1 to 4 given in the water operation study in Section 8.4.3. The cropping system in these four (4) cases is practised by mono cropping. The cropping in Case 5 is to be by irrigated rotational cropping. The crops planted are paddy and upland crops, or diversified crops. The cropping system of Case 5 is planned to convert the mono-cropping with the dry season paddy planned in Case 3 into the mixed cropping of both the upland crops and paddy.

The area planted with crop are shown by each case in the following table.

- The Cropped Area with the Project -

(unit: ha)

Case	Crop	1998/99 1999	'99/00 2000	'05/06 2006	'06/07 2007	'08/09 2009	'11/12 2012	'17/18 2018
1	Paddy	2,268	2,268	2,268	2,268	2,268	2,268	2,268
2	Paddy	5,738	13,442	14,933	14,933	14,933	14,933	14,933
3	Paddy	7,710	15,872	28,877	45,662	55,420	56,712	56,712
4	Paddy	7,710	15,872	21,527	27,288	32,847	33,764	33,764
5	Paddy	2,268	9,524	14,621	31,406	39,579	40,868	40,868
5	Upland Crop	7,784	7,803	14,887	14,893	15,841	15,844	15,844
	Total	10,052	17,327	29,508	46,299	55,420	56,612	56,612

Note: Key Year
1999: Start of the dry season paddy in North Lemal, Ulu Lemal, Sg. Bagan.
2000: Start of the rainy season paddy in North Lemal, Ulu Lemal, Sg. Bagan.
2006: Start of the dry season paddy in Tasek Garu.
2007: Start of the rainy season paddy in Tasek Garu.
2009: Start of Sg. Sat, Panyit and Kusial.
2018: Attainment to full benefit.

- The Cropped Area without the Project -

(unit: ha)

1	Paddy	-	-	-	-	-	-	-
2	Paddy	1,092	3,670	3,670	3,670	3,670	3,670	3,670
3	Paddy	1,194	5,004	5,804	8,364	11,476	11,476	11,476
4	Paddy	1,194	5,004	5,804	8,364	10,901	10,901	10,901
5	Paddy	1,194	5,004	5,804	8,364	11,476	11,476	11,476
5	Upland Crop	2,284	2,320	2,530	2,564	2,634	2,670	2,670
	Total	3,478	7,324	8,334	10,928	14,110	14,146	14,146

The production of paddy and upland crops are shown by cases in the following table.

- Production Quantities with the Project -

(unit 1,000 ton)

Case	Crop	1998/99 1999	'99/00 2000	'05/06 2006	'06/07 2007	'08/09 2009	'11/12 2012	'17/18 2018
1	Paddy	7.1	7.2	7.7	7.7	7.8	7.9	8.1
2	Paddy	20.1	45.0	52.8	53.0	53.4	53.7	54.7
3	Paddy	25.7	53.2	99.5	158.9	196.0	202.1	205.4
4	Paddy	25.7	53.2	75.3	95.9	117.2	125.1	126.9
5	Paddy	7.1	31.4	50.9	110.0	140.5	146.2	147.6
"	Maize	2.6	2.6	4.7	4.7	4.7	4.7	4.7
"	Groundnut	7.3	8.4	12.3	12.3	12.3	12.3	12.3
"	Tobacco	21.3	22.9	32.7	38.4	38.4	38.4	38.4
"	(crude)							
"	Sorghum	-	-	20.6	20.6	20.6	20.6	20.6
"	Vegetable	10.4	11.0	18.4	18.4	21.4	21.4	21.4

- Production Quantities without the Project -

(unit: 1,000 ton)

Case	Crop	1999	2000	2006	2007	2009	2012	2018
1	Paddy	-	-	-	-	-	-	-
2	Paddy	3.4	10.1	10.8	10.8	11.0	11.1	11.1
3	Paddy	3.8	14.2	17.6	25.1	35.9	36.1	36.2
4	Paddy	3.8	14.2	17.6	25.1	34.1	34.3	34.4
5	Paddy	3.8	14.2	17.6	25.1	34.1	34.3	34.4
"	Maize	0.6	0.7	0.8	0.8	0.8	0.8	0.8
"	Groundnut	0.8	0.8	0.8	0.8	0.8	0.8	0.8
"	Tobacco	8.5	8.7	9.7	9.9	10.2	10.4	10.4
"	(crude)							
"	Vegetable	3.2	3.2	3.2	3.2	3.2	3.2	3.2

- Incremental Production Quantities -

(unit: 1,000 ton)

Case	Crop	1999	2000	2006	2007	2009	2012	2018
1	Paddy	7.1	7.2	7.7	7.7	7.8	7.9	8.1
2	Paddy	16.6	34.9	42.0	42.2	42.4	42.6	43.6
3	Paddy	21.9	39.0	81.9	133.8	160.1	166.0	169.2
4	Paddy	21.9	39.0	57.7	70.8	83.1	90.8	92.5
5	Paddy	3.3	17.2	33.3	84.9	106.4	111.9	113.2
"	Maize	2.0	1.9	3.9	3.9	3.9	3.9	3.9
"	Groundnut	6.5	7.6	11.5	11.5	11.5	11.5	11.5
" "	Tobacco (crude)	13.2	14.2	23.0	28.5	28.2	28.0	28.0
"	Sorghum	-	-	20.6	20.6	20.6	20.6	20.6
"	Vegetable	7.2	7.8	15.2	15.2	18.2	18.2	18.2

Expected incremental production of paddy in 2012 is compared with the forecasted total paddy production of Kelantan State in 2010.

- Prospect of Total Paddy Production in Kelantan and Incremental Paddy Production in the Projects -

Share of paddy production of Kelantan to those of Malaysia	Supplemental paddy production in Kelantan (2010 years)	Incremental paddy production in 2012 in the irrigation projects associated with Lebir Dam
10%	0 '000 ton	-
11%	26 "	Case 1 7.9 '000 ton
12%	58 "	Case 2 42.6 "
13%	89 "	Case 4 90.8 "
14%	120 "	Case 5 111.9 "
15% above		Case 3 166.0 "

The larger the expectation is given to paddy production increase in Kelantan State for meeting the national rice requirements, the more meaningful will become the irrigation components relating to the Lebir Dam Project.

8.5.8. Gross Production Value, Production Cost and Net Production Value

Gross production value, production cost and net production value are estimated using both the market/financial price and economic price.

The cropping system schemed in Case 5 is a diversified cropping selection of paddy and the remunerative crops. Hence, maximum incremental net production value and quick benefits, can be expected from the project as Case 5. However, it is noted that an appropriate distribution of labour, promotion of supporting services and improvement of marketing outlets are necessary in order to gain the benefits mentioned above.

(1) Market Price

(With the Project)

Gross Production Value (unit: M$ million)

Case	1999	2000	2006	2007	2009	2012	2018
1	3.55	3.60	3.85	3.87	3.91	3.97	4.06
2	10.03	22.52	26.42	26.52	26.68	26.86	27.36
3	12.83	26.62	49.77	79.47	98.02	101.05	102.72
4	12.83	26.62	37.67	47.95	58.59	62.57	63.45
5	33.29	47.58	86.01	115.57	136.41	139.28	139.99

Production Cost (unit: M$ million)

Case	1999	2000	2006	2007	2009	2012	2018
1	2.58	2.58	2.58	2.58	2.58	2.58	2.58
2	6.52	15.27	16.87	16.87	16.87	16.87	16.87
3	8.76	18.03	32.80	56.44	62.96	63.94	63.94
4	8.76	18.03	24.45	31.00	37.31	38.36	38.36
5	26.37	34.68	53.56	72.65	85.93	87.40	87.40

Net Production Value (unit: M$ million)

Case	1999	2000	2006	2007	2009	2012	2018
1	0.97	1.02	1.27	1.29	1.33	1.39	1.48
2	3.51	7.25	9.55	9.65	9.81	9.99	10.49
3	4.07	4.59	16.97	23.03	35.06	37.11	38.78
4	4.07	4.59	13.22	16.95	21.28	24.21	25.09
5	6.92	12.90	32.45	42.92	50.48	52.88	52.59

(Without the Project)

Gross Production Value (unit: M$ million)

Case	1999	2000	2006	2007	2009	2012	2018
1	-	-	-	-	-	-	-
2	1.72	5.06	5.38	5.41	5.49	5.53	5.54
3	1.90	7.12	8.82	12.56	17.93	18.06	18.09
4	1.90	7.12	8.82	12.56	17.03	17.15	17.18
5	10.49	15.85	18.30	22.17	27.79	28.05	28.08

Production Cost (unit: M$ million)

Case	1999	2000	2006	2007	2009	2012	2018
1	-	-	-	-	-	-	-
2	1.42	4.00	4.00	4.00	4.00	4.00	4.00
3	1.55	6.49	7.53	10.85	14.88	14.88	14.88
4	1.55	6.49	7.53	10.85	14.14	14.14	14.14
5	9.53	14.60	16.38	19.82	24.10	24.23	24.23

Net Production Value (unit: M$ million)

Case	1999	2000	2006	2007	2009	2012	2018
1	-	-	-	-	-	-	-
2	0.30	1.06	1.38	1.41	1.49	1.53	1.54
3	0.35	0.63	1.29	1.71	3.05	3.18	3.21
4	0.35	0.63	1.29	1.71	2.89	3.01	3.04
5	0.96	1.25	1.92	2.35	3.69	3.82	3.85

(Incremental NPV) (unit: M$ million)

Case	1999	2000	2006	2007	2009	2012	2018
1	0.97	1.02	1.27	1.29	1.33	1.39	1.48
2	3.21	6.19	8.17	8.24	8.32	8.46	8.95
3	3.72	3.96	15.68	21.32	32.01	33.93	35.57
4	3.72	3.96	11.93	15.24	18.39	21.20	22.05
5	5.96	11.65	30.53	40.57	46.79	48.06	48.74

(2) Economic Price

(With the Project)

Gross Production Value (unit: M$ million)

Case	1999	2000	2006	2007	2009	2012	2018
1	2.81	2.83	3.03	3.04	3.07	3.12	3.19
2	7.88	17.70	20.77	20.85	20.97	21.11	21.50
3	10.08	20.93	39.12	62.46	77.04	79.42	80.73
4	10.08	20.93	29.61	37.69	46.05	49.18	49.87
5	26.95	38.97	70.25	95.32	112.57	114.99	115.59

Production Cost (unit: M$ million)

Case	1999	2000	2006	2007	2009	2012	2018
1	1.94	1.94	1.94	1.94	1.94	1.94	1.94
2	4.91	11.51	12.78	12.78	12.78	12.78	12.78
3	6.60	13.59	24.71	39.09	47.44	48.55	48.55
4	6.60	13.59	18.43	23.46	28.12	28.90	28.90
5	18.27	24.52	38.41	52.78	62.49	63.60	63.60

Net Production Value (unit: M$ million)

Case	1999	2000	2006	2007	2009	2012	2018
1	0.87	0.89	1.09	1.10	1.13	1.18	1.25
2	2.97	6.19	7.99	8.07	8.19	8.33	8.72
3	3.48	7.34	14.41	23.07	29.60	30.87	32.18
4	3.48	7.34	11.18	14.33	17.93	20.27	20.97
5	8.68	14.45	31.84	42.54	50.08	51.39	51.99

(Without the Project)

Gross Production Value (unit: M$ million)

Case	1999	2000	2006	2007	2009	2012	2018
1	-	-	-	-	-	-	-
2	1.35	3.98	4.19	4.23	4.28	4.34	4.36
3	1.49	5.60	6.94	9.87	14.09	14.19	14.22
4	1.49	5.60	6.94	9.87	13.38	13.48	13.50
5	8.52	13.05	15.10	18.37	23.14	23.34	23.37

Production Cost (unit: M$ million)

Case	1999	2000	2006	2007	2009	2012	2018
1	-	-	-	-	-	-	-
2	1.02	1.02	1.02	1.02	1.02	1.02	1.02
3	1.12	4.67	5.42	7.81	10.72	10.72	10.72
4	1.12	4.67	5.42	7.81	10.18	10.18	10.18
5	8.52	13.05	15.10	18.37	16.91	16.99	16.99

Net Production Value (unit: M$ million)

Case	1999	2000	2006	2007	2009	2012	2018
1	-	-	-	-	-	-	-
2	0.33	2.96	3.17	3.21	3.26	3.32	3.34
3	0.37	0.93	1.52	2.06	3.37	3.47	3.50
4	0.37	0.93	1.52	4.06	3.20	3.30	3.32
5	2.03	2.92	3.74	4.54	6.23	6.31	6.38

(Incremental NPV) (unit: M$ million)

Case	1999	2000	2006	2007	2009	2012	2018
1	0.87	0.89	1.09	1.10	1.13	1.18	1.25
2	2.64	3.23	4.82	4.86	4.93	5.01	5.38
3	3.11	6.41	12.89	21.01	26.23	27.40	28.68
4	3.11	6.41	9.56	12.27	14.73	16.97	17.65
5	6.65	11.53	28.10	38.00	43.85	45.04	45.61

8.6. Project Cost

The layout of the irrigation projects proposed in the coastal region is shown in Fig.8-1 obtained from DID Kelantan. Among these Kemubu-Salor, Lemal-Alor & Pasir Mas and Kemasin-Semerak are the ongoing projects. The other seven projects are the large scale water supply projects for irrigation development worked out by DID, Kelantan. These seven projects originated from the Kelantan River Basin Study (KRBS) conducted by ENEX in 1977, but are not included in the Fifth Malaysia Plan.

It can be considered, however that DID, in the near future, will independently make promotion of some works in these new project areas, shown to be economic by the total water balance study of the Kelantan River. Hence, DID has not participated in the cost allocation of the Lebir Dam Project.

The discharge to the upstream of the Kelantan River from the Lebir Dam will generate agricultural benefit from the projects shown in Table 8-2. In the economic evaluation of the Lebir Dam Project, the construction costs of the seven new irrigation projects and their annual operation and maintenance costs should be counted as cost-flow corresponding to benefit-flow.

The following data were used in order to make a rough cost estimate of the seven new projects.

Data-1: KRBS, Main Report, Volume 2: Drainage and Irrigation, 1977.

Data-2: North Lemal Irrigation Project Phase 1-Stage 1 Pre-Investment Report, 1978, DID, Kelantan

Data-3: Final Report, KADA II Improvement Project, Kelantan, MOA, Malaysia, 1982.

According to KRBS, for preliminary design and costing purposes, each project was examined in terms of three components - a bulk water supply system, a reticulation system, and an on-farm distribution system.

. The bulk water supply system consists of the facilities to supply irrigation water from the source of water to the boundary of the irrigation projects including major booster pumps within the project area.

. The reticulation system provides the distribution facilities of water throughout the project area. Roads, bridges and the operation & maintenance facilities are included.

. The on-farm distribution system comprises the terminal facilities required to supply the water to the cultivated area.

At first, KRBS proposed three alternative schemes on the bulk water supply system.

. A series of major pumping stations on the Kelantan River

. A barrage across the Kelantan River

. Storage reservoirs on the Sg. Nal, Sg. Sokor and Sg. Taku, tributaries of the Kelantan River.

As a result of study, KRBS concluded that both the Kelantan River barrage and the system of pumping stations are potentially feasible, as water supply schemes for large scale irrigation development.

In consideration of a result of the study conducted by KRBS, the estimation of the cost, with water supply system, is made based on the facility of pumping stations. The cost of the pumping stations in the seven (7) irrigation projects is estimated by use of the cost equation studied in KRBS.

Secondly, KRBS needed the formulation of some conceptual design criteria to form the base for estimating the cost of the reticulation component of each irrigation project. The basic reticulation costs for a typical paddy irrigation scheme were studied. The costs are based largely on the present equivalent value costs for the Kemubu and Lemal Irrigation Schemes. The capital cost of this basic reticulation scheme was expressed as a function of the gross area and peak irrigation demand. Hence, the reticulation cost in this study is estimated using the same cost per hectare for the proposed gross area, as in the project which was studied in KRBS.

KRBS studied the average cost per ha of the land irrigated in the four kinds of on-farm irrigation systems, flood irrigation furrow, sprinkler and trickle. The on-farm costs in this study are estimated using the cost per ha in the water distribution system with flood type.

KRBS estimated the annual cost of the irrigation project. This cost covers the operation and maintenance cost and energy cost. The cost equation and unit cost used in estimation of operation and maintenance cost for pumping stations and on-farm system are available in KRBS, Main Report Vol.2: Appendix, while the base of estimation of operation and maintenance cost for the reticulation system is not available.

Though the cost equation used in estimation of energy costs needs the mean annual irrigation demand quantity (m^3/sec), those data are not available in KRBS Report. Hence, the operation and maintenance costs estimated for the east bank sub-project area of KADA II is used in each irrigation project of this study. (refer to Tables 33 to 43 in Appendix)

The project cost used in the economic evaluation is shown in the following table.

(Market/Financial Price Base)

- Capital Cost -

(unit: M$1,000)

Project	1977 Price	1986 Price
North Lemal Phase 1	13,096	19,251
Ulu Lemal	12,080	17,758
Sg. Bagan	5,074	7,459
Tasek Garu	65,014	95,570
Sg. Sat	5,856	8,608
Panyit	3,973	5,840
Kusial	4,025	5,917

Note: Consumer price index is estimated at 100 in 1977 and 147 in 1986. Capital costs at 1986 prices are estimated (using 147 percent) at 1.47 x 1977 prices.

- Operation and Maintenance Cost -

Project	1981 Price	1986 Price
North Lemal Phase 1	419	486
Ulu Lemal	438	508
Sg. Bagan	186	216
Tasek Garu	2,145	2,488
Sg. Sat	210	244
Panyit	142	165
Kusial	144	167

Note: Consumer price index is estimated at 100 in 1981 and 116 in 1986. O & M costs at 1986 prices are estimated (using 116 percent) at 1.16 x 1981 prices.

(Economic Price Base)

- Capital Cost at 1986 Prices -

(unit: M$1,000)

Project	Economic Cost		Total
	Foreign Cost	Local Cost	
North Lemal Phase 1	7,700	8,894	16,594
Ulu Lemal	7,103	8,204	15,307
Sg. Bagan	2,984	3,446	6,430
Tasek Garu	38,228	44,153	82,381
Sg. Sat	3,443	3,977	7,420
Panyit	2,336	2,698	5,034
Kusial	2,367	2,734	5,101

Note: 1. Ratio between foreign currency and local currency is assumed at 40%: 60% based on KADA II Main Report, Table 4-24, page 4-123.

2. Conversion factors used for project and annual costs are 0.77 and 0.80 respectively.

- Operation and Maintenance Cost at 1986 Prices -

(unit: M$1,000)

Project	Economic O & M Cost
North Lemal Phase 1	389
Ulu Lemal	406
Sg. Bagan	173
Tasek Garu	1,990
Sg. Sat	195
Panyit	132
Kusial	134

Note: Economic costs are estimated using 0.8 for the general conversion factor.

8.7. Internal Rate of Return

Case 5 shows the highest internal rate of return as indicated in the following table. It is considered that the irrigation project of Case 5 is in the appropriate level from the national point of view. *(Refer to Tables 44 and 45 in Appendix)*

- Internal Rate of Return -

Case	FIRR	EIRR
2	11.0%	11.6%
3	12.5	12.5
4	12.8	12.7
5	18.3	19.9

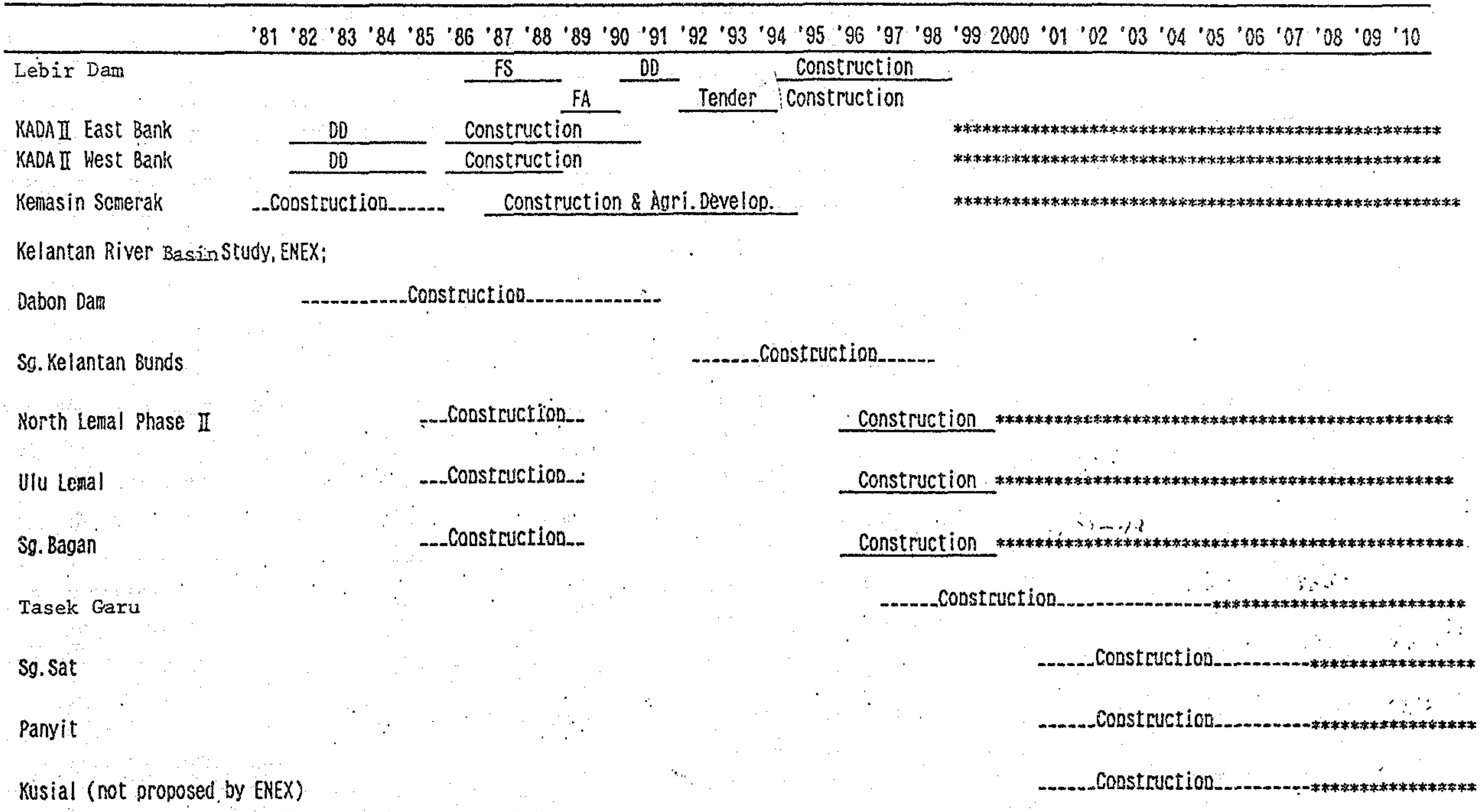
Table 8-1 Comparison of Up to Date Irrigation Projects with the Kelantan River Basin Study of ENEX

Water demand in 2000 years based on KRBS				Water demand in future based on DID & KADA				
	Name of Project	Irrigable Area (ha)	Peak Water Demand (cu.m)		Name of Projects	Irrigable Area (ha)	Peak Water Demand (cu.m)	Source
1.	Kemubu & Salor Ext.	32,623	28.6	1.	Kemubu & Salor	20,090 5/	41.9	KADA
2.	Lemal, Alor Pasir & Pasir Mas	15,463	15.4	2.	Lemal, Alor Pasir, Pasir Mas	11,710 5/	29.7	KADA
3.	Kemasin & Semerak	35,401	29.4	3.	Kemasin & Semerak	15,000 6/	16.0	DID
4.	North Lemal	9,265	10.9	4.	North Lemal (I)	3,644	6.5	DID
5.	Ulu Lemal	7,371	4.6	5.	North Lemal (II)	5,621	10.0 1/	DID
6.	Upper Ulu Lemal	758	0.4	6.	Ulu Lemal (including Upper Ulu Lemal)	3,806	7.0 4/	DID
7.	Sg. Bagan	4,281	2.5					
8.	Sg. Tasek Garu	18,650	15.0	7.	Sg. Bagan	1,620	2.8 2/	DID
9.	Sg. Sat	6,652	3.5	8.	Tasek Garu	18,650	16.0 3/	DID
10.	Pertok & Putat Ext.	1,491	1.1	9.	Sg. Sat	1,822	3.2 2/	DID
				10.	Panyit (included Pertok & Putat)	1,234	2.5 2/	DID
				11.	Rantau Panjang	2,024	3.5 1/	DID
				12.	Kusial	1,250	2.0 3/	DID

Kelantan River Barrage Scheme : Planned			Not Proposed			
Water requirement from Kelantan River	(ha) 131,955	(cu.m) 112.3		(ha) 51,250	(cu.m) 101.1	
Tributary of Kelantan River			Sg. Nal (Storage) Sg. Sokor (Storage)	4,676 19,900	8.5 18.0	) 26.5)127.6

Note: 1/ Sg. Golok 2/ Sg. Nal (Storage) 3/ Sg. Sokor (Storage) 4/ At present, water source is planned to be converted into Nal Dam. 5/ KADA II Improvement Project. The land on right of way in on-farmworks is not reduced. 6/ Kemasin-Semerak Agricultural Development Project Report.

Table 8-2 DEVELOPMENT PLAN OF IRRIGATION PROJECT



Note : FS:Feasibility Study FA:Financial Arrangement DD:Detail Design

------------:Plan by ENEX ————————:Assumtive plan revised by JICA Study

************:Period of benefit formation due to water release from Lebil Dam

WATER SOURCE FOR IRRIGATION DEVELOPMENT

Fig. 8-1 PUNCA AIR UNTUK PEMBANGUNAN PENGAIRAN

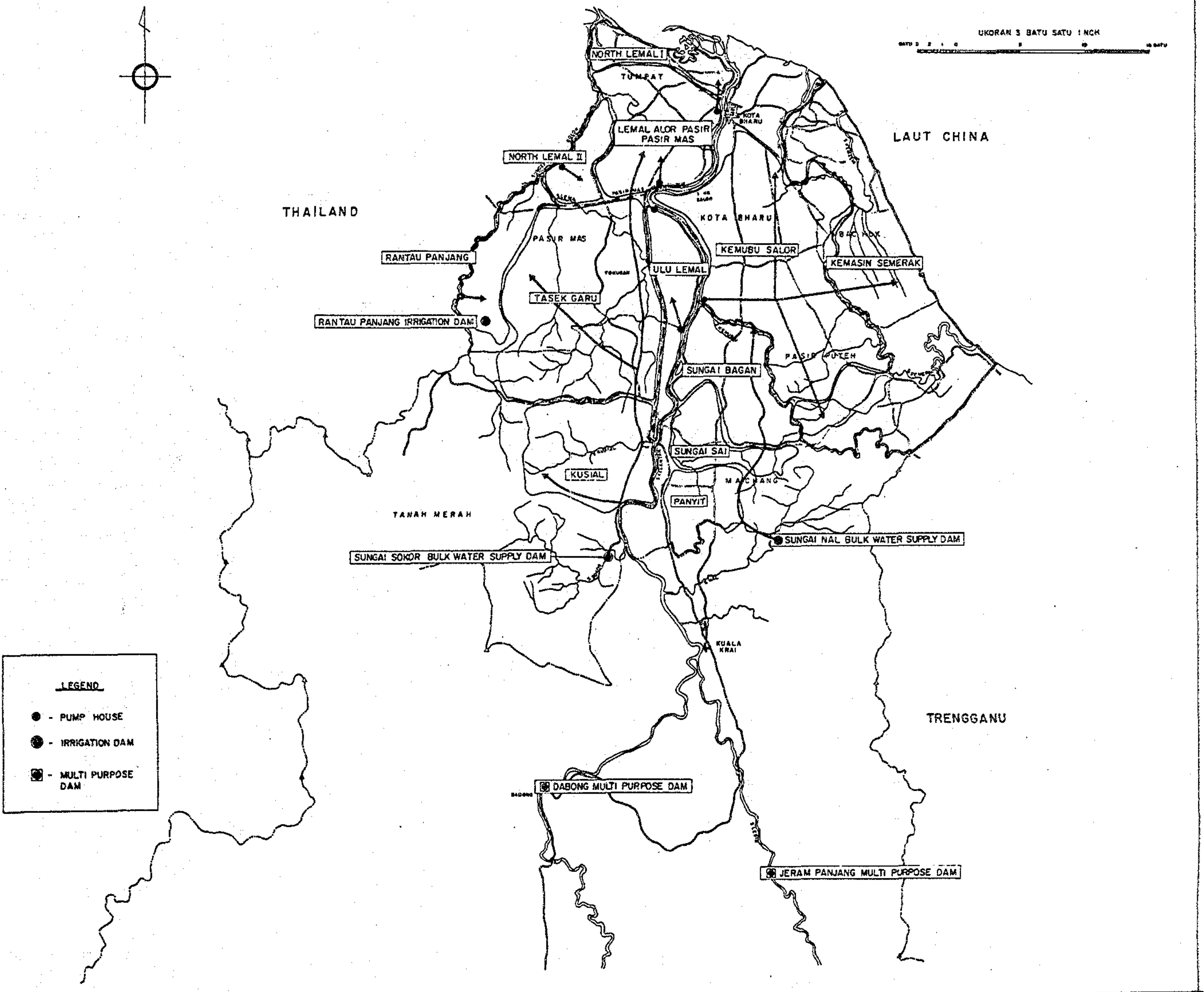
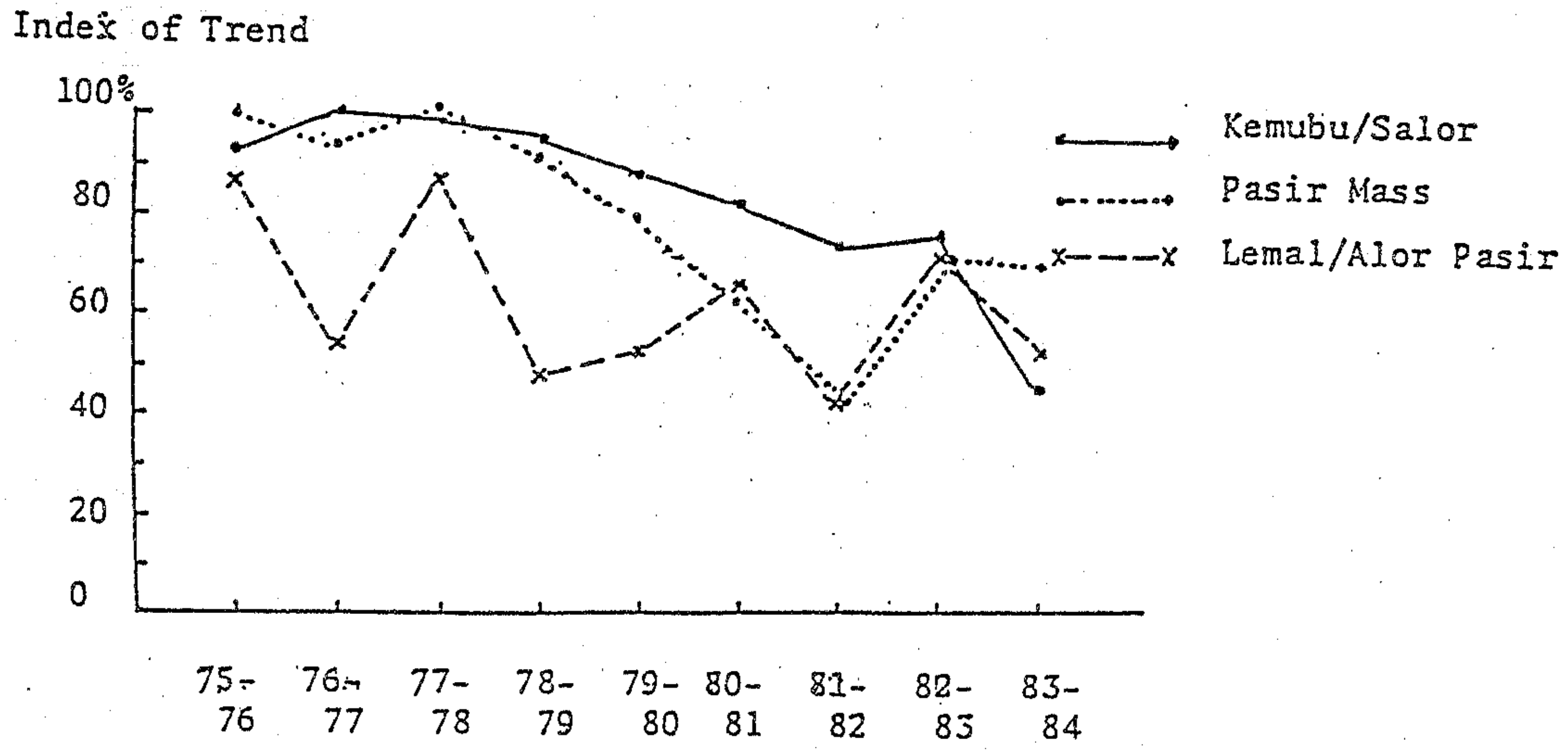


Fig. 8-2 Annual Trend of Cropped Area of Paddy

(1974/1975 = 100%)



9. Feasibility Grade Design of Structures

9. Feasibility Grade Design of Structures

9.1. Layout Study of the Project

(1) Topographical and Geological Features of the Main Dam Site and Arrangement of Structures

The main dam site is located at a point about 3 km upstream of the Tualang Bridge where the Lebir River meanders in an S-shape.

Starting from this point, rapid currents continue along Jeram Kushon, Jeram Chendawru, Jeram Panjang, and Jeram Tembeling with the elevation of the river bed lowered by 4 m over a river course of 2,500 m.

The river bed width at the main dam site is 150 m, covered by exposed bedrock. Normally, the river runs in a 60 m wide channel closer to the right bank side, and the lowest elevation of the riverbed is 24 m above sea level.

The mountain slopes on both banks at the main dam site are as steep as 30° to 40° up to EL. 45 m, and then turn to gentler slopes of about 15° on average, at higher elevations.

On the left bank of the river, a ridge of the mountain which forms a watershed between the Lebir River and the Sam River, spreads towards the east and the west, and it presents a flat terrace with an elevation of some 110 m near the Lebir main stream. The terrace, extending 500 m to the east, slopes down to the Lebir river on the left river bank.

The right bank of the river is formed by a branch of the small mountain forming a watershed between the Lebir river and the Rek river. The mountain side of an independent ridge spreading about 1,500 m long toward the south-southwest from a point 1,000 m north of the upper stream portion of the S-shape river section forms the right bank slope at the dam site. The elevation of the ridge is only about 110 m near the dam site, and it is a thin ridge with a thickness of 500 m at the river bed elevation. Fresh rock is exposed for the most part in the riverbed. However, both slopes are covered with heavily weathered laterite whose thickness is 5 m to 7 m on the terrace at EL. 45 m. Weakly weathered laterite at higher elevations is 8 m to 16 m thick.

The topographical and geological features are suitable for the construction not only of a fill-type dam, but of a concrete gravity type dam. If the fill-type dam is selected, the ridge on the right bank will provide a suitable site for a spillway. The full water level of the reservoir should be limited to EL. 90 m as some of the fresh bedrock exposures on the peak of right bank ridge are at as low as EL 84 m.

The powerhouse and switchyard can be arranged at flat land on the left bank without any layout problems, and the foundation bedrock under the powerhouse site may not pose any problem. An access road, if branching off from the Kuala Krai-Gua Musang Highway, would be only 3 km long and convenient for transportation to the site.

The tailrace outlet level will be EL 28 m, given that the plant releases water at the rate of 640 m^3/sec.

The topography justifies the location of the river diversion tunnels on the right bank. However, the geological features may not necessarily be the best, if large section diversion tunnels are planned in consideration of large flood discharges. Large flood discharges and a high water level of floods may reduce the possibility of adopting a two-staged open channel diversion for this project.

A most suitable quarry site is located on the right bank of the river, some 1.5 km north and downstream of the proposed main dam site. Borrow sites for core materials are located east-northeastward from the dam site. A granite borrow site is some 4.0 km and a conglomerate borrow site some 1.5 km away from the damsite in this direction.

(2) Access Roads and In-situ Construction Roads

It is recommended that an access road to the left bank of the main dam should branch off from the Kuala Kerai-Gua Musang Highway at a point several hundred meters distant toward Gua Musang from the Tualang Bridge. This road can be used as an access to the sites for the waterway and powerhouse. It is also necessary to provide roads to the dam site from the quarry and borrow pit sites located on the right bank. Access roads to the quarry site and the borrow pit site should branch off from the existing logging road. As the existing logging road passes through the Saddle dam I site and may hinder the development of the quarry site, road relocation will be required to bypass these two places.

The crest of the coffer dam will be used as a connecting road between the right and the left banks at the main dam site, after the river diversion is completed. Since the major construction activities will be done on the right bank side, base camps and temporary construction facilities are planned to be set up on the right bank.

For the whole project layout, refer to Fig. 9-0-0.

9.2. Design Concepts of Structures

9.2.1. Main Dam

(1) Type of Dam

The type of a dam must be determined, taking into consideration the foundation bedrock condition, availability of embankment materials and measures against possible floods which may occur during the construction and after its completion. So far as the project site is concerned, it is possible to construct either a fill type dam or a concrete gravity dam. A decisive factor for the selection of the dam type will, therefore, depend on the economic comparison between both types. A preliminary comparison of total construction cost of dams, spillways and river diversion works for both types is as shown in the table below. It shows the fill-type dam to be the more economical alternative, and to be recommendable for this project.

Comparison of Construction Cost
of 73 m High Dam by Type

Item	Fill-type (M$)	Concrete gravity type (M$)
Main Dam	46,810,000	104,404,000
Spillway	33,494,000	10,499,000
Temporary River Diversion	32,814,000	18,548,000
Total	113,118,000	133,451,000
	(0.85)	(1.00)

Assumptions:

Concrete gravity type:

Unit price of concrete: 112 M$/m^3 (rate for RCC)

Dam concrete volume : 850,000 m^3

Rockfill type:

Unit price of rockfill materials: 16.3 M$/m^3

Dam embankment volume : 2,900,000 m^3

(2) Typical Cross Section of the Main Dam

(a) Dam crest height

The maximum rise of the reservoir water level due to floods is discussed in Section 7.2.1. The resultant maximum reservoir level against the design flood and the projected flood is as given below:

	Return Period	Maximum Inflow (m^3/s)	Maximum Reservoir Level (m)
Design Flood	10,000	10,580	88.1
Projected Flood	50	5,250	84.9

The height of waves generated by winds can be determined by applying the following Sverdrup-Munk-Bretschneider (SMB) equation:

$$Hw = 0.00086V^{1.1}F^{0.45}$$

Where,

V = 20.8 m/sec (average wind velocity of 10 minutes)

F = 4,500 m (fetch with HWL at 80 m and on the ES direction)

Therefore,

$Hw = 1.07$ m

The height of waves hitting against the upstream face of the dam is 0.86 m as discussed in Section 7.2.1.

Hence, it is considered adequate to determine the height of waves due to winds to be 1.1 m.

The height of waves generated by earthquake can be determined by applying the following equation:

$$He = \frac{1}{2} x \frac{K}{\pi}(gHo)^{1/2}$$

Where,

K = 0.10 (seismic coefficient of the dam at normal water level)

≈ 1 sec (seismic period)

Ho = 61 m (80-19) (depth of reservoir water at normal water level)

g = 9.8 m/s^2 (acceleration of gravity)

Therefore,

$He = 0.389$ m ≑ 0.4 m

Now that,

The design flood water level with the wave effect due to winds is 89.2 m (88.1 + 1.1),

The surcharged water level is 86.2 m (84.9 + 1.1 + 0.4/2), and the normal water level with the combined wave effect due to winds and earthquake is EL.81.5 m (80.0 + 1.1 + 0.4).

It is therefore decided to determine the dam crest height at EL.92.0 m, taking a freeboard of 2.8 m above the design flood water level (with the wave effect due to winds).

(b) Dam Crest Width

The dam crest width is determined to be 10 m, taking into consideration convenience during construction and after completion of the works. A width of 10 m is also consistent with normal practice in building dams of a similar sort.

(c) Impervious Core

The dam type will be a rockfill dam with a central impervious core, taking advantage of local geological conditions in the vicinity of the proposed main dam site. The area contains abundant rock materials and material suitable for use in the central core. The width of the core at the bottom will be 26.3 m, or 36 % of the dam height. Two layers comprising coarse and fine filter will be provided both upstream and downstream of the core zone. The width of each of these layers will be 4 m, the smallest width possible to facilitate the embankment work.

(d) Upstream and Downstream Face Slopes of the Main Dam

The gradient of the upstream and downstream face slopes of the main dam can be decided based on the result of calculation of safety factors against sliding. Assuming the horizontal seismic coefficient (K) of the dam to be 0.1, and the internal friction angle of rockfill materials to be 41°, it is possible to obtain the slope gradient of 1:1.75 as a critical limit. In fact, there exist some rockfill dams built of late in Peninsular Malaysia, whose slope gradient is very close to the above limit.

In Japan, it is expressly provided for in the dam design standards that the safety factor of a fill-type dam against sliding should be $1.1 + \alpha$, and the value of "α" should usually read as 0.1. For this Project, however, it is considered adequate to use the value of 1.1 as the safety factor against sliding, disregarding the value "α", in view of the fact that there is very little seismic activity in Peninsular Malaysia.

As a result of the above consideration, it is decided to adopt the following values as the gradient of the dam slope:

Upstream face slope : 1 : 1.85
Downstream face slope : 1 : 1.75

Horizontal berms will be provided in both upstream and downstream faces of the dam, in conjunction with the construction of cofferdams. The upstream face berm will be 12.5 m wide and provided at EL 59.0 m, while the downstream face berm will be 10.0 m wide and at EL 40.0 m.

Rockfill materials with internal friction angle of 43° will be used as the outershell for the surface layer (10 m in horizontal width) of the upstream face slope. The outershell at higher elevations than EL 60 m will be constructed of riprap. The riprap will be of a size no smaller than 50 cm, in consideration of the possible wave rise of 1.0 m due to winds. The size is determined by using applicable equations (such as those used by U.S. Corps of Engineers).

The cofferdams provided both upstream and downstream of the main dam will form part of the main dam after completion. They will be constructed as independent structures prior to construction of the main dam.

The upstream cofferdam is 39 m high with a crest elevation of EL 59 m. The slope of the downstream face cofferdam will be 1:1.3, the steepest for a rockfill dam. The upstream face slope will become part the main dam and the gradient will therefore be a 1:1.85. An impervious core and filter zones will be provided at the upstream face, and these will be protected by an outer shell of riprap. The slope of the riprap embankment will be 1:1.90.

The downstream cofferdam will be 20 m high with its crest at EL 40 m. The upstream face slope of the dam will be 1:1.3, while that of the downstream face slope will be 1:1.75, the same as that of the main dam. An impervious core and filter zones will be provided at the downstream face, and these will be protected by riprap. The outermost riprap embankment will have a slope of 1:1.87.

(3) Materials for Embankment

Tuffaceous rockfill materials are planned to be taken from the quarry site. The result of boring investigation and laboratory tests of sample rock materials as given in Chapter 4 can be summarized as follows:

Sample rock materials:

- Green, fine grain tuff
- Purple, coarse grain tuff
- Purple, fine grain tuff
- Purple tuffaceous breccia and conglomerate

Strength by the uniaxial compression tests:

Low : 154-211 kg/cm^2 (five samples)
Medium : 399-586 kg/cm^2 (four samples)
High : 687-1,195 kg/cm^2 (seven samples)

Specific gravity: 2.76

Absorbability : 0.38 %

Stability by the sodium sulphate tests: 1.3 - 4.6 % (three samples)

RQD : 35 - 66 (four bore holes)

As shown above, the test result indicates the rock materials taken from the quarry site are hard enough and quite adequate to be used not only as rockfill materials but as concrete aggregates. However, it may be fairly stated that the rock masses at the quarry site are extensively cracked, and accordingly, pieces of quarried rocks are only of 20 to 30 cm in size on average. Since no natural filter materials are available in the vicinity of the main dam site, quarried rocks will be crushed to produce filter materials.

With regard to the possible adverse effect on concrete structures of sulphide pyrites contained in tuffaceous rocks that was raised by a GSD representative at the meeting on the interim report of February 1988, the geological expert of the study team is of the following opinion:

"The general geology in the vicinity of the proposed dam site consists of tuffs, tuffaceous breccias, lavas or the like, and it is observed on rare occasions that sulphide pyrites layers intermingle with these rock groups through cracks or small voids, but it is estimated from visual checks that the ratio of their content would not exceed 0.1%.

In this connection, reference can be made to the study on the effect of sulphide pyrite on concrete structures made by Tohoku Electric Power Co., Inc., a Japanese utility, for their Miyashita dam. According to this study, the admixture of sulphide pyrite in the rockfill materials with the content ratio of less than 0.2 % (2 kg per ton) would pose no problems in using the materials as concrete aggregate.

Since the content of sulphide pyrites in rock materials in the project area is believed extremely low with its ratio being less than a half of that for the Tohoku Electric's study case, it is considered that no particular sulphide-derived problems should arise out of the use of rock materials in the area."

It is proposed to use some 400,000 m^3 of rock available from excavation of the proposed intake site as rockfill material for the main dam. They are not only suitable as rockfill materials, but their use would also be advantageous from the economical point of view in view of the very short haulage distance. A larger volume of rock than required will therefore be excavated and stockpiled for use as rockfill materials.

Two borrow sites are available for core materials. The characteristics of materials at each site are as given below:

Weathered granite borrow site:

Distance from the main dam site: About 4 km
Grain specific gravity : 2.67
Liquid limit : 34 - 61% (6 samples)
Plastic limit : 29 - 37% (6 samples)
Plasticity index : 5 - 26% (6 samples)
Natural water content: 15 - 30% (6 samples)
Optimum water content: 18 - 23% (6 samples)
Dry density : 1.55 - 1.74 ton/m^3 (6 samples)
Parameters for shearing strength: (=0.12 kg/cm^2, ϕ=34.8°)
Permeability coefficient : K=1.60 x 10^{-6} cm/sec.

Weathered conglomerate borrow site:

Distance from the main dam site: About 1.5 km
Grain specific gravity : 2.72
Liquid limit : 28 - 38% (3 samples)
Plastic limit : 11 - 23% (3 samples)
Plasticity index : 7 - 14% (3 samples)
Natural water content : 13 - 17% (3 samples)
Optimum water content : 15 - 17% (3 samples)
Dry density : 1.79 - 1.84 ton/m^3 (3 samples)
Parameters for shearing strength: C=0.27 kg/cm^2, ϕ=32.8°
Permeability coefficient : K=0.36x10^{-6} cm/sec.

As is seen above, the materials taken from the two borrow sites are both suitable for use as core materials. The conglomerate borrow site, is closer to the dam site and should therefore be selected. Suitable material excavated at the proposed intake site may also be available for use as core materials, but suitability remains to be checked.

(4) Foundation Bedrock

The baserock masses underlying the proposed main dam consist mainly of green and purple tuffs, and partly of tuffaceous sandstones, shales and tuffaceous conglomerates.

The baserock masses underlying the impervious core and filter zones of the dam should be excavated until harder rock layers appear. It should be a basic practice to excavate rock masses until CH class hard bedrocks are exposed. Excavation of heavily-weathered rock masses at higher elevations of the proposed abutments on both banks should be limited to the level where CM class hard bedrock is exposed. For the foundation of rockfill zones, D class hard bedrock which may underlie top soils and overburdens may be satisfactory. The depth of excavation should be 2 to 3 m on average. Based on this criterion, the lowest elevation of the core zone foundation would be EL 19 m. As to the hardness classification of bedrock, refer to the corresponding table in Section 4.3.

The result of the geological investigation for this Study indicates that no large, geologically weak zones exist, in the project area, such as faults or fractured belts, and the bedrock masses are considered adequate to support a 70 m high rockfill type dam.

(5) Treatment of Dam Foundation

The surface of the foundation bedrock on which the soil core materials are to be placed must be free from any materials which are prone to erosion. It is therefore essential to remove any such materials, or where necessary to cover them with concrete (dental work). In a case where there exists numbers of voids such as cracks in the baserock, it is necessary to apply slush grouting treatment.

The grouting treatment to the foundation bedrock involves consolidation grouting and curtain grouting. The former will be applied to the depth of 10 m to the entire surface of bedrock on which core and filter materials are to be placed. The latter will be applied in a single line on the center of the core zone to the depth of 70 m at the deepest section and 30 m at the shallowest.

According to the result of the boring investigation (at boreholes D-1, D-2 and D-3), the permeability of bedrock underlying the core zone is generally 5 or less in Lugeon test value, except for the case of the unusually high values of 22-25 in the upper portions of D-2. Hence, it is considered unnecessary to apply any specifically designed grouting treatments.

However, it is considered necessary to provide a curtain wall on a narrow ridge, extending from the dam abutment on the right bank to the spillway overflow, as some of its bedrock is as low as EL 84 m. This is lower than the design flood water level of EL 88.1 m. The wall should be provided for the whole length from the abutment to the spillway.

(6) Stability Analysis of the Main Dam

The stability of the main dam embankment is checked by slip-circle analysis. The earthquake effects are considered on the basis of the horizontal seismic coefficient method. The physical properties of the embankment materials are as given below:

Zone No.	Materials	Dry Unit Weight (ton/m^3)	Saturation Unit Weight (ton/m^3)	Internal Friction Angle (degree)	Cohesion (kg/cm^2)
1	Outershell	1.85	2.10	43.0	0
2	Rockfill	1.85	2.10	41.0	0
3	Filter	1.85	2.10	35.0	0
4	Core	1.60	1.80	30.0	0

Shown below are the least safety factors of the slopes against sliding under various loading conditions:

Case	Water Level (m)	Static Loading Condition		Earthquake Effect		Horizontal Seismic Coefficient
		Upstream Face	Downstream Face	Upstream Face	Downstream Face	
1. Reservoir Empty	-	1.68	1.52	1.50	1.36	0.05
2. Design Flood Water Level	88.1	1.59	1.52	-	-	0
3. Surcharged Water Level	85.1	1.61	1.52	1.33	1.36	0.05
4. Normal Water Level	80.0	1.56	1.52	1.12	1.22	0.10
5. Medium Water Level	70.0	1.56	1.52	1.18	1.22	0.10
6. Low Water Level	60.0	1.68	1.52	1.12	1.22	0.10

The pore water pressure of the core zone is dealt with as follows. For the empty reservoir condition, it is considered equivalent to 50 % of the earth loading. For the normal water level, it is considered equivalent to 100%, in consideration of an abrupt drawdown from the design flood water level. For the medium and low water levels, it is considered zero, assuming that the water level would decrease gradually.

The safety factors of the slopes against sliding under various loading conditions calculated by the Slip circle method are shown in Fig.9-2-1.

The same method is applied to the stability analysis of the coffer dams. No earthquake effects are considered in the analysis, because they will be in existence as independent structures only for a relatively short period of time during construction. The least safety factors of the slopes of these dams against sliding are as given below:

	Upstream face	Downstream face
Upstream cofferdam	1.22	1.13
Downstream cofferdam	1.23	1.09

(7) Quantitative Estimates of Construction Work

Quantitive estimates of the main earthworks for the main dam are as given below:

Item	Upstream Cofferdam	Downstream Cofferdam	Sub-total	Main Dam	Total
Earthwork:	m^3	m^3	m^3	m^3	m^3
Earth excavation	89,100	32,000	121,100	249,300	
Rock excavation	38,200	13,700	51,900	106,900	
Excavation-total	127,300	45,700	173,000	356,200	529,200
Rockfill embankment	380,200	108,300	488,500	1,488,100	1,976,600
Filter embankment	49,900	18,300	68,200	321,200	389,400
Outer-shell embankment	37,000	13,400	50,400	161,100	211,500
Riprap embankment	37,000	13,400	50,400	-	50,400
Total	504,100	153,400	657,500	1,970,400	2,627,900
Core embankment	36,700	13,700	50,400	391,600	442,000
Embankment-total	540,800	167,100	707,900	2,362,000	3,069,900
Grand total	668,100	212,800	880,900	2,718,200	3,599,100
Grouting:	m	m	m	m	m
Consolidation	0	0	0	3,070	3,070
Curtain	1,700	0	1,700	9,980	11,680
Total	1,700	0	1,700	13,050	14,750

Miscellaneous work involves the following:

- Construction of a permanent road
- Construction of a dam control center
- Provision of cut-off walls on the ridge at the right bank

The above estimates are made based on the dimensions shown in Figs. 9-0-1, 9-0-2 and 9-0-3.

9.2.2. Saddle Dam I

(1) Type of the Dam

The proposed site for the Saddle Dam I is located at a saddle where there are only few streams of water. The provision of either a temporary river diversion channel or a spillway is therefore unnecessary. A fill-type dam is most suitable from the point of view of economy. The choice of this type is also supported by considerations of site geology, which consists of deep layers of weathered bedrocks.

(2) Typical Cross-Section of the Dam

The typical-cross section of the Saddle Dam I is determined in similar manner to that described in the preceding Sub-section. The dam is of the following dimensions and is similar in configuration to the main dam:

Crest height	:	EL 92.0 m
Crest width	:	10.0 m
Upstream face slope	:	1 : 1.85
Downstream face slope	:	1 : 1.75
Upstream face berm elevation	:	EL 59.0 m
Upstream face berm width	:	10.0 m
Core zone base width	:	24.5 m
Filter zone base width	:	

8.0 m each both upstream and downstream of the core zone. The width of two layers is made up of coarse and fine filter.

(3) Materials for Embankment

Similarly to the case of the main dam, rockfill materials are planned to be taken from the tuffaceous rock quarry site, and core materials from the conglomerate borrow site.

No plans are envisaged to use excavated materials from the sites of other structures as rockfill and core materials.

(4) Foundation Bedrock

The bedrock masses underlying the proposed Saddle Dam I consist mainly of tuffaceous sandstones and conglomerates, interbedded with some distribution of tuffs. The result of boring investigation (at four boreholes, S-1, S-2, S-3 and S-4) reveals that the layers of weathered bedrock are 10 m deep at the river bed, and 23 to 27 m deep at the upper portions of the abutments on both banks. Excavation of the foundation bedrock should, therefore, be deep enough to encounter harder rock masses. The bedrock underlying the *impervious core and filter zones should be excavated until* CL or CM class hard rock is exposed. Heavily weathered layers at the upper portions of both banks should be removed to a depth of some 30 m, until the required rock is exposed. The lowest elevation of the core zone foundation would be EL 25 m after excavation to the depth of some 15 m. D class bedrock may be adequate foundation for rockfill zones and this may only require excavation to a depth of 8-10 m.

(5) Treatment of Dam Foundation

Similar treatment to the case of the main dam should be applied. Curtain grouting will be applied in a single line to the depth of 65 m at the deepest section and 30 m at the shallowest. The permeability of bedrock is a little higher than that of the main dam. Lugeon test results indicate permeability to be 30-40.

(6) Stability Analysis of the Dam

Stability analysis is made by the method similar to the case of the main dam. The physical properties of the embankment materials are same as those given in Sub-section 9.2.1(6). Shown below are the least safety factors of the slopes against sliding under various loading conditions:

Case	Water Level (m)	Static Loading Condition		Earthquake Effect		Horizontal Seismic Coefficient
		Upstream Face	Downstream Face	Upstream Face	Downstream Face	
1. Empty Dam Reservoir	-	1.70	1.51	1.51	1.35	0.05
2. Design Flood Water Level	88.1	1.63	1.51	-	-	0
3. Surcharged Water Level	85.1	1.60	1.51	1.53	1.35	0.05
4. Normal Water Level	80.0	1.56	1.51	1.12	1.21	0.10
5. Medium Water Level	70.0	1.56	1.51	1.13	1.21	0.10
6. Low Water Level	60.0	1.70	1.51	1.13	1.21	0.10

Fig. 9.2.2 shows the safety factor of the slopes against sliding under various loading conditions calculated by slip circle analysis.

(7) Quantitative Estimates of Construction Work

The quantitative estimates made of the main earthwork for Saddle Dam I are as given below:

Earthwork:

Earth excavation	868,100 m^3 (Used for the Saddle Dam II embankment)
Rock excavation	0
Total (Excavation)	868,100
Rockfill embankment	914,400
Filter embankment	220,100
Outer-shell embankment	136,400
Total (Embankment-rock materials only)	1,270,900
Core embankment	260,900
Total (Embankment-all materials)	1,531,800
Grand total (Excavation and embankment)	2,399,900

Grouting:

Consolidation	2,200 m
Curtain	6,620
Total	8,820

Miscellaneous work involves the following:

- Protection of excavated slope
- Construction of a permanent road
- Installation of dam monitoring instruments

The above estimates are made based on the dimensions shown in Figs. 9-0-4 and 9-0-5.

9.2.3. Saddle Dam II

(1) Type of the Dam

The Saddle Dam II is planned to be located at a basin-like depression formed by three small ridges. The dam will be built by filling the basin with rock and earth materials from the Saddle Dam I excavation. In other words, the dam site is a spoil bank of surplus excavated material, which in itself creats an earthfill dam.

(2) Typical Cross-Section of the Dam

The crest height of the dam will be EL 92.0 m, the same as that of the main dam. The reservoir face slope will be 1:3.5. A berm with a width of 15 m will be provided on this face slope at EL 67.0 m. The surface of the slope will be protected by riprap. The slope at elevations higher than the reservoir water level will be 1:3.0. The area around the outlet of the base drains will be protected by rockfill structures.

(3) Materials for Embankment

As mentioned in (1) above, the surplus excavated weathered bedrock from the Saddle Dam I site will be used as the material for embankment of the Saddle Dam II. The properties of the materials are believed to be similar to those taken for testing from the proposed conglomerate borrow site, and are considered adequate for the use as earthfill materials. Riprap materials for the reservoir face slope protection, and filter materials for the base drain systems, will be supplied from the quarry site.

(4) Foundation Bedrock

The bedrock underlying Saddle Dam II consist of tuffs and tuffaceous sandstones, which are intruded with meta-dacites. The result of boring investigation (at bore hole S-5) indicates that the weathered rock masses lie to a depth of 27 m in the vicinity of the abutment on the left bank. For the foundation of an earthfill dam, however, it is not necessary to completely remove such weathered rock masses. Excavation to a depth of some 3 m is considered sufficient.

(5) Treatment of Dam Foundation

The configuration of Saddle dam II is, unlike the normal dimensions of a dam. It width of 280 m is large compared with the water head of 48 m. Therefore, no grouting treatment will be given, and instead, base drain systems will be provided. For the drain systems, drain trenches will be provided, and perforated Hume pipings will be laid in the trenches. The systems will be back-filled with filter materials.

(6) Stability Analysis of the Dam

Stability analysis is made by the method similar to the case of the main dam. The physical properties of embankment materials are as given below:

Zone No.	Materials	Dry Unit Weight (ton/m^3)	Saturation Unit Weight (ton/m^3)	Internal Friction Angle (degree)	Cohesion (kg/cm^2)
1	Riprap	1.85	2.10	43.0	0
2	Earthfill	1.85	2.10	30.0	0
3	Foundation (D class)	2.20	2.20	20.0	5.0
4	Foundation (C class)	2.30	2.30	25.0	15.0

Shown below are the least safety factors of the slopes against sliding under various loading conditions:

Case	Water Level (m)	Static Loading Condition		Earthquake Effect		Horizontal Seismic Coefficient
		Upstream Face	Downstream Face	Upstream Face	Downstream Face	
1. Reservoir Empty	-	2.06	1.64	1.72	1.23	0.05
2. Design Flood Water Level	88.1	1.97		-		0
3. Surcharged Water Level	85.1	1.85		1.47		0.05
4. Normal Water Level	80.0	1.80		1.20		0.10
5. Medium Water Level	70.0	1.77		1.22		0.10
6. Low Water Level	60.0	1.79		1.21		0.10

(7) Quantitative Estimates of Construction Work

The quantitative estimates made of the main earthwork for the Saddle Dam II are as given below:

Earth excavation	195,100 m^3
Earth embankment	653,100
Riprap embankment	63,300
Filter embankment	15,800
Protection rockfill embankment	9,500
Rockfill embankment - Total	88,600
Total (Excavation + embankment)	936,800

Miscellaneous work involves the following:

- Construction of a permanent road
- Installation of dam monitoring instruments

The above estimates are made based on the dimensions shown in Figs. 9-0-4 and 9-0-6.

9.2.4. Spillway

(1) Type of Spillway

The type of a spillway basically relates to considerations ot the planned system of flood control and inflow design flood. Also, taken into consideration in this Study are the layout of other project structures and the site topography, and an overflow chute spillway type was selected. As for provision of gates, the spillway was designed as ungated free overflow system which is simple in both operation and maintenance. However, for comparison, other types were also studied. These include a gated chute spillway and a tunnel spillway utilizing a part of diversion tunnels after the dam completion. The result of this comparative study is given later in this section.

(2) Spillway Design Flood and Spillway Capacity

Probable floods at the proposed damsite were examined in Section 5.4 of Chapter 5 "Flood Analysis". These floods are listed below:

Probable Floods at the Lebir Damsite

Probable Return Period (years)	Flood Flow (m^3/sec)
1.15	620
2	1,700
5	2,850
10	3,600
20	4,320
50	5,260 (Diversion design flood)
100	5,950
200	6,660
1,000	8,280
10,000	10,600 (Spillway design flood)

Dam design standard in Japan stipulates that inflow design flood for rockfill dams shall be a flood of 200-year return plus 20% thereof.

If this standard is applied for the Lebir dam, the inflow design flood becomes 8,000 m^3/sec. This volume of design flood almost corresponds to the probable flood of 1,000 years return period. For this Study, however, the probable flood of 10,000 years, which is equivalent to the PMF, is proposed to be applied.

The flood hydrograph of each probable flood listed above is given in Fig.9-2-4.

When the inflow design flood is set, the spillway capacity can be determined in the relation between the overflow length and overflow water depth through flood routing calculation. By calculation, the overflow length was determined as 150 m, and the crest of the overflow section was set at the same level as generation high water level.

The relationship between spillway discharges and reservoir level in this case is shown in Fig.9-2-6.

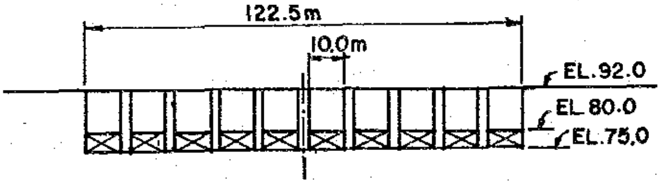
The relationship between reservoir level and storage volume is given in Fig.9-2-5 and Table 9-2-1.

The maximum spillway discharge and the highest reservoir level for the respective probable floods are tabulated below. The discharge/stage-duration curves for the respective cases are as depicted in Fig.9-2-7.

Probability	Flood Peak	Max. Spillway Discharge	Highest Reservoir Level
years	m^3/s	m^3/s	m
10,000	10,600	6,400	88.1
1,000	8,260	4,880	86.8
200	6,650	3,840	85.8
100	5,940	3,390	85.3
50	5,250	2,950	84.9
20	4,310	2,370	84.2
5	2,840	1,490	83.1
2	1,700	860	82.1

(3) Gated Spillway Alternative

In the Interim Report issued in February 1988, a plan of a gated spillway was studied (Refer to Sub-section 7.2.2. of Section 7). Its basic dimensions are as illustrated below:



For large floods, a gated spillway performs almost same hydraulic function as an ungated spillway does. It has an advantage, in that it can control spilling discharges to some extent by the gate operation when a flood is of small to medium magnitude. For realization of such control, however, it is essential that hydro-meteorological observation, water stage measurement, and a flood forecasting system should be established, and strict dam operation control should be conducted by setting up the gate operation regulation, and through vocational training of personnel who operate the gates.

The extent of construction works may differ depending on the provision of gates. The following is a comparison in the quantities of major work items.

Works	Ungated	Gated
Excavation	1,318,000 m^3	1,301,700 m^3
Concrete	121,600 m^3	124,800 m^3
Gate Metal	0	300 tons

The gated spillway can be given a shorter overflow length. In this case, it is 122.5 m, which is 27.5 m less than the ungated spillway. However, the overflow depth is 5m, which requires a lower elevation for installation of the structure. This will not always result in reduction of construction quantity. It is considered from the above that an ungated spillway has more economic advantage.

(4) Tunnel Spillway Alternative

Two diversion tunnels each of 12 m inner diameter are planned to be excavated through the right abutment for the construction of the main dam.

It is proposed that one of these tunnels be used as a bottom outlet after the completion of the dam, and the other is to be plugged. Described hereinafter is the utilization of these tunnels as spillways. The basic dimensions of the tunnel inlet considered are as shown below;

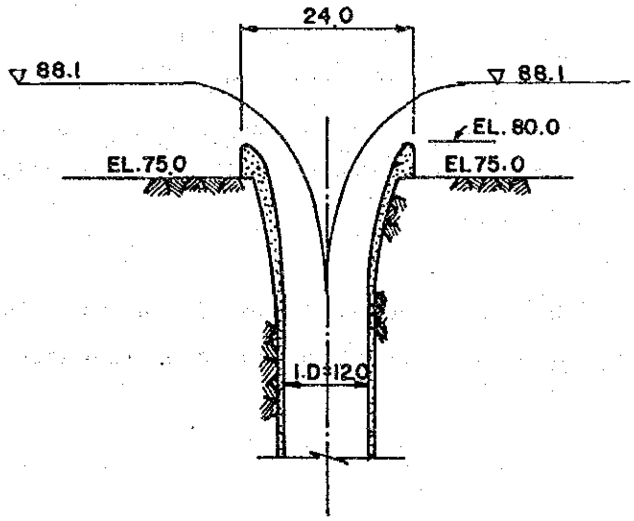


Fig. 9-2-8 shows the relationship between reservoir water level and spillway discharge in the case of both lines of diversion tunnels being utilized as spillways. As seen from the curve in this Figure, the spillway discharge will be 4,800 m^3/s at reservoir elevation EL.88 m, which is deficient by 1,600 m^3/s compared with the required discharge capacity of 6,400 m^3/s, and therefore, another spillway dealing with excess discharge beyond the diversion tunnel capacity is still required.

This additional spillway is planned to be of the free overflow chute type. The sizing of spillways was examined and compared as below:

Reservoir level	(1) Chute spillway	(2) Tunnel Spillway (2 lines)	(1)-(2)	(1) - $\frac{(2)}{2}$(1 line)
m	m^3/s	m^3/s	m^3/s	m^3/s
80	0	0	0	0
81	280	250	30	155
82	760	900	-140	310
83	1,420	1,670	-250	580
84	2,200	2,500	-300	950
85	3,060	3,400	-340	1,360
86	4,000	4,160	-160	1,920
87	5,100	4,600	500	2,800
88.1	6,400	4,800	1,600	4,000

Where only one diversion tunnel is used as a spillway, the 4,000 m^3/s (max) of remaining flow must be released by other spillway. This amount of water is about 60% of the chute type spillway capacity discussed earlier and proposed for the Project.

In a plan which includes use of the two diversion tunnels for spillways, 1,600 m^3/s has to be released elsewhere. If this amount of water is discharged at an overflow depth of 5 m, the net overflow length will be 77 m and the crest of overflow will be EL.83.1 m, which requires a 5 m high gate.

The costs of above-mentioned alternative plans were estimated and compared as follows:

Work	Chute Type	One Tunnel Type	Two Tunnel Type
	10^6M$	10^6M$	10^6M$
Chute type construction	33.49	20.09	13.40
Tunnel type construction	0	1.92	3.84
Bottom outlet*/	0	0	0.60
Gates	0	0	2.00
Total	33.49	22.01	19.84

*/ A bottom outlet becomes necessary if both diversion tunnels have been used for spillways. Therefore, the additional cost for construction of the bottom outlet is estimated.

It is known from the above cost comparison that some ten million Malaysian Dollars can be saved by utilizing the diversion tunnels. In this Study, however, this type of solution is kept pending, for there is a feeling among some engineers the tunnel spillways involve restriction in hydraulic function.

(5) Stilling Basin

Basic parameters necessary for hydraulic design of a stilling basin are given below. (The river stage - discharge curve is shown in Fig.9-2-9).

No.	Reservoir Water Level	Spillway Discharge	Probability	Stilling Basin Floor Level	Total Hydraulic Head	Velocity at Downstream End
	EL. m	m^3/s		EL. m	m	m/s
1	88.1	6,400	10,000	23.0	65.1	31.0
2	85.3	3,400	100	23.0	62.3	29.5

No.	Spillway Discharge	Spillway Width	Discharge per unit width (q)	Water Depth at Downstream End (hi)	Froude's Number (Fr1)
	m^3/s	m	$m^3/s/m$	m	
1	6,400	95	67	2.2	6.7
2	3,400	95	36	1.2	8.7

(a) Natural jump in case of a horizontal channel (Handbook of Hydraulic Formulas of JSCE 1971 Version, Page 296)

$$h2'/h1 = \frac{1}{2}(\sqrt{1 + 8Fr1^2} - 1)$$

No.	Spillway Discharge	Water Depth after Jump (h2)	$\frac{1}{2}(\sqrt{1+8Fr1^2}-1)$	h2'	Head Loss	Apron Length
	m^3/s	m		m	m	m
1	6,400	47.5 - 23 = 24.5	9.0	19.8	31.3	106
2	3,400	38.0 - 23 = 15.0	11.8	14.2	32.2	78

Apron will be required to have a length of more than 100 m if it is intended to generate a natural jump in the basin.

(b) Jump in use of end sill (the above Handbook, Page 301)

No.	Spillway Discharge	Weir Height W	$H = (\underline{q})^{2/3}$	h2 = H + W	Apron Length (5 h2)
	m^3/s	m	m	m	m
1	6,400	8.67	10.9	19.6	98
2	3,400	6.89	7.2	14.1	71

Note: c = 1.85

In this case, the apron length needed will be about 100 m if the jump is to be caused by a wedge-shaped weir.

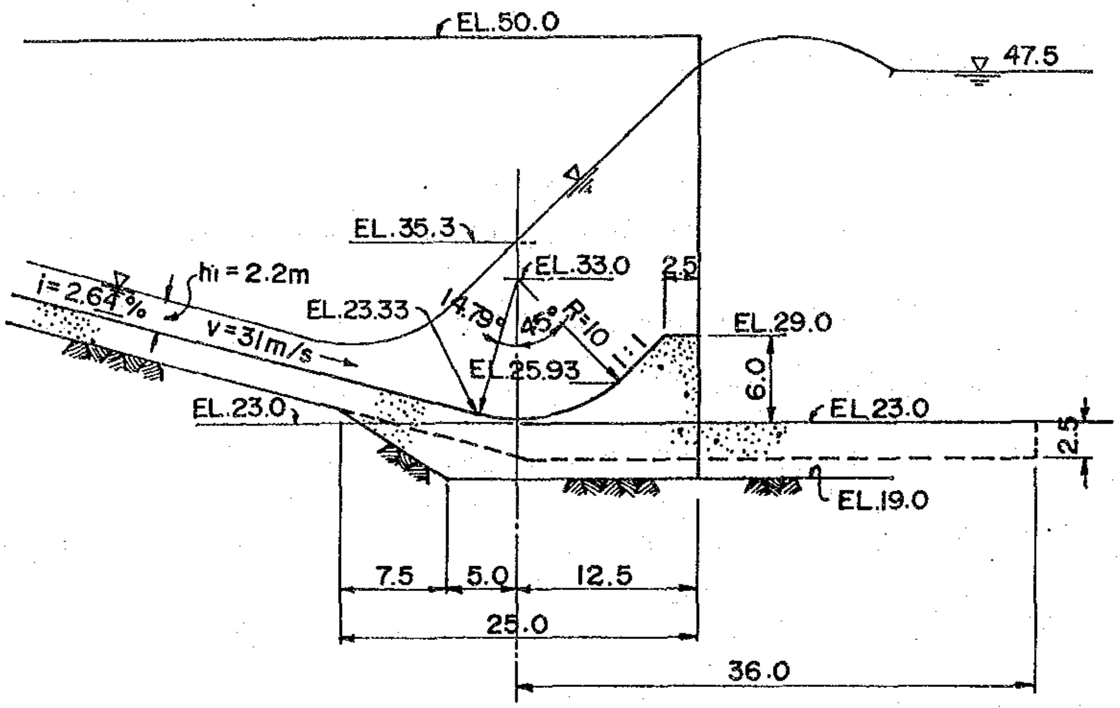
(c) Descending Slope Apron Type Stilling Basin (The above Handbook, Page 300)

No.	Spillway Discharge	Water Depth Before Jump (h1)	L/n1	Apron Length (L)
	m^3/s	m		m
1	6,400	2.2	16.2	35.6
2	3,400	1.2	10.4	12.5

(d) Bucket type stilling basin

No.	Spillway Discharge	Total Head	Downstream water Depth after Jump (h2)	h2/Z	hb	hb/Z	R=Z/b
	m^3/s	m	m		0.5h2		m
1	6,400	65.1	24.5	0.376	12.3	0.19	10.8
2	3,400	62.3	15.0	0.242	7.5	0.12	10.4

No.	Discharge per Unit Width (q)	$q/\sqrt{g}\,Z^{3/2}$
	$m^3/s/m$	
1	67	0.041
2	36	0.023



For this Project, a bucket type stilling basin, which requires the least extensive construction, is proposed.

(6) Foundation Bedrock

It was known from the drilling data at D-6 that a bedrock layer of CM class exists at EL.84.5 m in the right bank mountain ridge where the overflow crest of spillway will be located. Since spillway structures are constructed in locations below this elevation, bedrock at the sites should be sufficiently strong.

Two other drill holes (D-5, D-7) bank ridge also confirm that good bedrock is available within a depth of 10 m.

The spillway excavation slopes will be 41 m high at the left bank side of the overflow section, and 37 m high at the left bank of the bucket stilling basin. After the stability analysis of these excavated slopes, the following safety factors were obtained.

Location	Normal	Earthquake
Left bank of overflow section	1.38	1.21
Left bank of bucket stilling basin	1.54	1.42

N.B. Horizontal seismisity k = 0.10

Mechanical properties of bedrocks

C class rock C = 25 t/m^2, ϕ = 30°

D class rock C = 5 t/m^2, ϕ = 20°

As mentioned in the main dam foundation treatment, the elevation of bedrock in the right bank comes, in some locations, lower than the reservoir design flood water level of EL.88.1 m. It is supposed that this trend extends continuously for about 300 m eastwards across the spillway site. Additional investigation will, therefore, be necessary, and cut walls may be installed where required. Final decision on this aspect is reserved due to lack of detailed data.

(7) Quantity of Spillway Construction Work

Estimated quantities of the major items of spillway construction work are as follows:

Earthwork

Excavation, common	1,259,600 m^3	(incl. excavation in the downstream portion of bucket)
Excavation, rock	59,200 m^3	
Backfill, random	255,500 m^3	
Concrete	121,600 m^3	(incl. the downstream protection concrete)

Miscellaneous work includes the following items:

- Curtain grouting at the overflow section
- Various drain work
- Permanent road
- Cut-off walls on the right bank ridge

The estimate of these quantities were based on Figs. 9-0-1 and 9-9-7.

9.2.5. Diversion Channel

(1) Method of River Diversion

For fill type dams, river diversion through a tunnel or tunnels in the abutments is the only practical option. In this Project, diversion tunnels are proposed to be driven in the right bank of the dam site for reasons of topography.

(2) Diversion Design Flood

In determining the capacity of river diversion provisions, the magnitude of diversion flood to be accommodated often raises much discussion. In Japanese practice, a 20-year return period flood is adopted for most cases of fill type dams. On the other hand, in the dam construction projects in Peninsular Malaysia, a 50-year flood has been often chosen for the diversion flow.

In Lebir Dam Project, it is proposed to adopt a 50-year flood after the comparative study of tunnel and upper cofferdam sizes required for three different probable floods, i.e. of 20-year, 50-year and 100-year recurrence intervals.

(3) Capacity of Diversion Channels

With the magnitudes of the diversion flood, the capacity of the diversion channels is determined from the relationship between the tunnel diameter and the height of the upper cofferdam. This related tunnel capacity for the operating head created by a cofferdam, and floods of three recurrence intervals mentioned above is shown in Fig.9-2-12. From the result given in this figure, two lines of tunnel each with a diameter of 12 meters and an upstream cofferdam having a crest elevation of EL.59.0 m can provide the protection against a 50-year return flood. Therefore, this scheme is proposed for the Project. Principal features of the upstream and downstream cofferdams are as follows:

	Upstream Cofferdam	Downstream Cofferdam
Crest elevation (EL)	59.0 m	40.0 m
Dam height	39.0 m	12.0 m
Slope - Upstream face	1 : 1.90	1 : 1.3
- Downstream face	1 : 1.3	1 : 1.87
Rock embankment	504,100 m^3	153,400 m^3
Core embankment	36,700 m^3	13,700 m^3
Total volume of embankment	540,800 m^3	167,100 m^3

The relationship between upstream water level and diversion discharge with two tunnels each having a 12 meter dia. is shown in Fig.9-2-13.

In this diversion scheme, the inlet invert level of the tunnels is set at EL.29 m and of their outlets, EL.26 m. The slope of channel is 0.5128% in No.1 tunnel (585 m in length) and 0.5208% in No.2 tunnel (576 m in length).

The discharges through the tunnels in conditions of open channel flow are as follows:

Upstream water level EL. (m)	Diversion Discharge (m^3/s)
31	160
32	350
33	580
34	860
35	1,160
36	1,480
37	1,720
38	1,950
39	2,200
40	2,580

Where probable floods of 100-year, 50-year and 20-year recurrences occur in the river with these diversion works, upstream water level, tunnel discharge and downstream water level in the respective cases will be as follows:

Probable Flood	Upstream Water Level	Tunnel Discharge	Downstream Water Level
	EL.(m)	m^3/s	EL.(m)
100	60.9	3,450	39.2
50	58.3	3,250	38.8
20	54.5	3,020	38.0

In the above calculation, the storage effect of the reservoir (storage volume: 408 x 10^6m^3 at EL.58 m) created by the upstream cofferdam was taken into account. Hydrographs of flood routing through the diversion channels in each of above probable floods is given in Figs. 9-2-14(1)-(3).

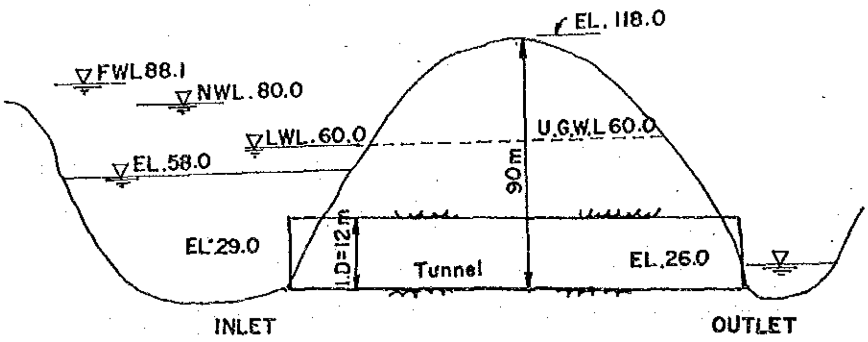
(4) Geological Conditions Along Diversion Tunnels

Since the proposed location of diversion tunnels is near to the spillway site, the geological conditions along the tunnel route are considered good, as inferred from the drilling data carried out at the spillway site. However, the rocks spreading over the upstream side of the route are mostly of CM class.

(5) Tunnel Lining Concrete

As the tunnel diameter will be as large as 12 meters, its lining concrete is designed to be 1.0 meter in thickness. The ratio of the thickness to the tunnel diameter is 8.3 %. Reinforcing bars will be arranged in the lining concrete. The necessary quantity of such reinforcing bars is obtained after checking concrete stress by use of calculation formula for thick-lined circular conduits the restraint of surrounding rock being considered.

The basic parameters for design consideration are as follows:



(a) Design Conditions

- Internal Pressure

88.1 - 26.0 = 62.1 m (Temporary)
58.0 - 26.0 = 22.0 m
80.0 - 60.0 = 20.0 m

Designed such that tensile stress generated in the concrete will be about 10 kg/cm^2 when internal pressure is 2.2 kg/cm^2.

- External Pressure

Water pressure equivalent
to the ground covering 90 m x 1.0 t/m^2=90 t/m^2
(Applying to the whole section)

- Temporary ExternalPressure

Grouting pressure 5 kg/cm^2
(50% up of allowable stress)
(Applied over an arc of 90° at the crown of the tunnel, symetrically on the vertical centerline)

(b) Allowable Stress

Concrete comp. $\sigma ca = 90\ kg/cm^2$ (ck=240 kg/cm^2)

ten. $\sigma ct = 10\ kg/cm^2$

Reinforcing bar $\sigma ca = 1,800\ kg/cm^2$ (SD30)

(c) Elasticity of Surrounding Rocks

Following three elasticities are considered:

Er = 10,000 kg/cm^2, 30,000 kg/cm^2, 50,000 kg/cm^2

(d) Necessary Reinforcement

At any elasticity of surrounding rock, no reinforcement is necessary when the load is due to external pressure or temporary pressure.

Where the load is due to internal pressure and the elasticity of surrounding rocks is 10,000 kg/cm^2, hoop reinforcing bars of D22 ctc 200 mm (area of a bar 3.871 cm^2 x 5 pcs = 19.355 cm^2) will be required per meter of inner face of lining concrete in the tunnel axis direction. No reinforcement is necessary if the elasticity of surrounding rocks is more than 30,000 kg/cm^2.

For the Project, the above reinforcing bars will be provided for the entire length of tunnel as a measure of safety. The weight of reinforcing bars per m^3 of concrete is calculated as follows:

Hoop bars	40.84 m x 5 pcs x 3.04 kg/m = 621 kg (D22)
Distribution bars	1.0 m x 8 pcs x 1.56 kg/m = 13 kg (D16)

Concrete volume per meter of tunnel axis = 41 m^3

Reinforcing bars per m^3 of concrete = 15.5 kg

(6) Quantity of Diversion Tunnel Work

Estimated quantities of major work items in the construction of the diversion tunnels are as follows:

Earth Work	Inlet (m^3)	Outlet (m^3)	Total (m^3)
Excavation, common	45,000	49,700	94,700
Excavation, rock	21,100	25,400	46,500
Total	66,100	75,100	141,200

Tunnelling	Tunnel No.1	Tunnel No.2	Total
Length	585 m	576 m	1,161 m
Excavation	95,300 m^3	95,800 m^3	189,100 m^3

Concreting	Open	Tunnel Lining	Plugging	Total
Concrete	13,200 m^3	57,800 m^3	8,500 m^3	79,500 m^3

Other miscellaneous works include the work items listed below:

- Consolidation/contact grouting
- Primary river coffering work
- Coffer dikes at the inlets and outlets of tunnels

The estimates of these quantities were based on Figs.9-0-1 and 9-0-8.

9.2.6. Intake

(1) Type of Intake

For the intake, a type of vertical shaft side intake with gates is a compact structure. As a result of the exploratory drilling (D-1) at the left upper abutment of the main dam site, a good and hard rock was confirmed and the excavation of a vertical shaft is judged not to be a problem.

Therefore, this type of intake is adopted.

(2) Intake Water

Power intake proposed in this Project is 320 m^3/sec. per gate.

(3) Intake Sill Level

The bottom level of intake is set at EL.48 m, i.e. 1 m above the reservoir siltation level.

(4) Inlet

The difference between the bottom level of intake EL.48 m, and the lowest water level (LWL) EL.60 m is 12 m. As stated later in this section, the penstock tunnel has an internal diameter of 8.4 m. The difference between the top level of tunnel at its inlet and the LWL therefore becomes 3.6 m. The velocity head being $V^2/2g$ = 1.54 m, as the flow velocity inside the tunnel is 5.5 m/sec.

In this Project, the screen is given an inclination of 1:0.3, a width of 13.3 m (at the top) to 15.0 m (at the bottom), and is 11.6 m in height. The total opening area of the mouth is 164.1 m^2. However, its net area is reduced to 94 m^2 due to three 2 m wide piers being provided at the inlet. The water velocity here is as relatively high, at 3.4 m/sec. In the detailed design stage, the shape should be determined based on the results of hydraulic test. The screen will have a trash rack along its 1:0.3 slope face.

(5) Gate Shaft

Intake gates will be installed in the vertical shaft of 12 m inner diameter, excavated below the platform at EL.92 m.

(6) Foundation Bedrock

It is expected from the result of the drilling investigation carried out at the left upper abutment of the dam site, that good rocks of CH class will be encountered below a depth of 10 m (EL.80 m) from the ground surface at the intake site.

(7) Stability Analysis of Excavation Slope

Excavation for the intake structure is planned to be carried out in a section 47 m high from its bottom level EL.47 m up to EL.92 m, with a steep slope of 1:0.3. Therefore, the slope stability against sliding is of particular importance, and a stability analysis of the cut slope was conducted.

In the analysis, physical properties of bedrock materials were assumed as follows:

C-class bedrock $C = 25\ t/m^2$, $\phi = 30°$
D-class bedrock $C = 5\ t/m^2$, $\phi = 20°$

The factor of safety obtained is as described below:

	WL 47.0 m	WL 85.1 m
Normal condition	1.55	1.90
At earthquake (k = 0.10)	1.42	1.62

Relevant sliding stability analysis diagrams are given in Figs. 9-2-15 and 9-2-16.

(8) Ground Cut at Intake

An excavation plan of more than one million m^3 is being considered. Of this large quantity, about half is the minimum requirement of excavation for conveyance of reservoir water into the intake. In anticipation of good rock available around this site, the surplus excavated materials are intended for use as rockfill for main dam embankment.

(9) Quantities of Intake Civil Work

Quantities of major work items are estimated as follows:

Earth work

Excavation, common	502,900 m^3
" , rock	539,200 m^3
Total	1,042,100 m^3

Tunnelling

Intake tunnel	2,600 m^3
Gate shaft	14,700 m^3
Total	17,300 m^3

Concrete

Open	3,600 m^3
Tunnel	670 m^3
Gate shaft	8,000 m^3
Total	12,270 m^3

Miscellaneous works include the following items:

- Clearing and grading around EL.92 m ground
- Tunnel grouting
- Mesh provision
- Boat house

The estimates of these quantities are based on Figs.9-0-1 and 9-0-9.

9.2.7. Waterway Pressure Tunnel

(1) Shape of Profile

The length of waterway from the intake to the powerhouse will be about 250 m, which is shortened by about 100 m from the length estimated from the site topography, due to the large volume of excavation made in front of the intake site. The profile of this waterway is to connect EL.48 m at the intake bottom level and EL. 21.1 m which is the center of turbine height in the powerhouse.
In the Interim Report of February 1988, a combination of

pressure tunnel and surface laid penstock was proposed. However, a large scale open excavation near the powerhouse would lead to a complex construction operation and could trigger a big problem in the construction schedule control. To avoid occurrence of such a situation, the entire route of waterway is planned in pressure tunnel. The waterway consists of the three following tunnel sections:

	Upstream Section*	Inclined Section	Downstream Section	Total
No.1	123.43 m	50.34 m	23.0 m	196.77 m
No.2	148.43 m	50.34 m	10.0 m	208.77 m
Total	271.86 m	100.68 m	33.0 m	405.54 m

* Not including the 49.0 m length of intake tunnel.

The downstream section of tunnel was made the shortest in order to avoid complication with the powerhouse erection works.

The upstream section will have a downward gradient of about 4.5 %. This is for the purpose of preventing negative pressure arising from water hammer. Due to the short tunnel length of approximately 250 m, a surge tank is not provided.

(2) Horizontal Shape

In principle, the lines of pressure tunnel are separated at an interval of 3 times the excavation diameter of the tunnel. Accordingly, the net interval was taken as more than 30 m.

The tunnel route to the relation of intake location with the direction of the powerhouse structure.

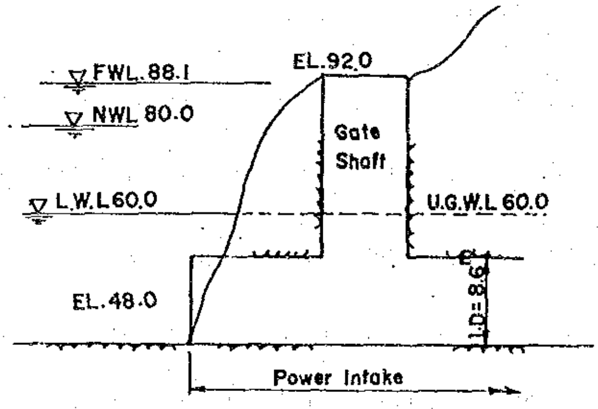
(3) Shape of Tunnel Section

The inner diameter of the tunnel was determined as 8.6 m after the economic comparative study discussed in Section 6.3.2. The Interim Report of February 1988 proposed steel pipes embedded over the entire length of the tunnel. It is, however, expected that the surrounding rocks are good and the internal pressure acting on the tunnel is relatively small. The upstream section therefore is only of reinforced concrete, while the inclined to downstream tunnel sections were provided with steel pipes. By this change the necessary weight of steel pipes is reduced to 920 tons, one-third of the previous plan.

Shape of Reinforced Concrete Section

The thickness of the tunnel lining concrete is determined to be 60 cm, which corresponds to 7.0 % of the tunnel diameter.

Basic parameters to be considered in design are listed below. Stress analysis is made as the same as that for the river diversion channel.



(a) Design Conditions

- Internal Pressure 88.1 m - 48.0 m = 40.1 m
  80.0 m - 60.0 m = 20.0 m
  60.0 m - 48.0 m = 12.0 m

Designed such that tensile stress generated in concrete will be about 10 kg/cm^2 when internal pressure is 2.0 kg/cm^2.

- External Pressure

  Water pressure equivalent
  to the ground covering 44 m x 1.0 k/m^2 = 44 t/m^2
  (Applying to the whole section)

- Temporary External Pressure

  Grouting pressure 3 kg/cm^2
  (50% up of allowable stress)
  (Applied over an arc of 90° at the crown of the tunnel, symetically on the vertical centerline).

(b) Allowable Stress

Concrete comp. ca = 90 kg/cm^2 (ck = 240 kg/cm^2)
ten. ta = 10 kg/cm^2
Reinforcing bar sa = 1,800 kg/cm^2 (SD30)

(c) Elasticity of Surrounding Rock

Following three elasticities are considered:

Er = 10,000 kg/cm^2, 30,000 kg/cm^2, 50,000 kg/cm^2

(d) Necessary Reinforcement

Where the load is due to external pressure or temporary pressure, no reinforcement is necessary even at any of the above elasticities.

When the load is due to internal pressure, and the surrounding rock is at Er = 10,000 kg/cm^2, hoop reinforcing bars of D22 ctc 200 mm (Area of a bar 3.871 cm^2 x 5 pcs = 19.355 (m^2) will be required per meter of inner face of lining concrete in the tunnel axis direction. If the elasticity of surrounding rocks is more than 30,000 kg/cm^2, no reinforcement is needed.

For the Project, the above reinforcement will be provided for the entire length of the upstream tunnel as a measure of safety. The weight of reinforcing bars per m^3 of concrete is calculated as follows:

Hoop bars 30.8m x 5 x 3.04 kg/m = 468 kg (D22)
Distribution bars 10 m x 8 x 1.56 kg/m = 13 kg (D16)
Total 481 kg

Concrete volume per meter of tunnel axis = 17.3 m^3
Reinforcing bars per m^3 of concrete = 28 kg

Shape of Steel Pipe Section

The steel pipe inner diameter is determined as 8.6 m. The thickness of the steel pipes was determined as that required to withstand internal pressure due to hydrostatic pressure and water hammer, or by the minimum thickness required to maintain the diameter of pipes, whichever the largest. The following formulae were used:

(a) For internal pressure,

$$t_s = \frac{ro \cdot Pi}{Fa} \text{ (cm)}$$

where, γo = Inner diameter of a pipe (cm)
Pi = Internal pressure (30% increase of hydrostatic pressure to include water hammer) (kg/cm^2)
Fa = Allowable stress in the pipe 1,200 kg/cm^2 (SM41)

(b) Min. thickness $t_s = 6$ mm

(c) Min. thickness for handling

$$t_s = \frac{D + 800}{400} \text{ (mm)}$$

(D = Inner dia. of pipe)

The gap between the steel pipe and the surrounding rock should be 70 mm which is the minimum requirement for pipe installation.

To withstand the external pressure, stiffeners are to be provided. For this purpose, 15% of the required steel weight based on the internal pressure is added for their provision.

(4) Quantity of Tunnel Work

Quantities of the major work item are estimated as follows:

	Tunnel No.1	Tunnel No.2	Total
Excavation	15,100 m^3	16,000 m^3	31,100 m^3
Concrete	3,900 m^3	4,200 m^3	8,100 m^3

Miscellaneous works include the following items:

- Tunnel consolidation grouting
- Tunnel measurement

The estimates of these quantities are based on Figs.9-0-1 and 9-0-9.

9.2.8. Powerhouse

(1) Type and Size of Powerhouse

A surface constructed powerhouse with ground elevation at EL.45 m and height of the center of hydraulic turbine at EL.21.1 m is planned.

The approximate dimension of the powerhouse structure is 29 m wide, 73 m long and 59 m high as shown in Figs. 9-0-11, 9-0-12, 9-0-13, 9-0-14, 9-0-15 and 9-0-16. The height is determined by the factors of ground level, height of turbine center, and hoisting span of cranes for installation or repair/maintenance of generators, while the width and length are determined by the sizes of the main equipment and space requirement for overhaul.

The ground elevation of EL.45 m is determined to protect the facilities from the 1,000 year flood which will cause a rise in river water level to EL 43.5 m. Thus a ground elevation of EL 45 m provides a 1.5 m margin against flooding.

(2) Powerhouse Structure

The powerhouse is laid out to enclose two units of turbine-generators, their auxiliary equipment, control devices, and switching equipment, including the areas necessary for assembling, disassembling and overhauling works. The powerhouse will also be equipped with overhead cranes of necessary capacity for erection and maintenance purposes.

The main power equipment will be placed on a barrel type multi floor foundation. The floor level of the assembly bay is set at EL.36.0 m. There will be a fairly large height difference between the ground surface EL.45.0 m and the assembly bay. A platform will therefore be provided at the same level as the ground surface in order to ensure the necessary space for transportation and assembly of generating equipment.

Four floors will be provided below the assembly bay. The floor immediately beneath the assembly bay will house main circuit equipment for the generators, and the second to fourth floors will be arranged for auxiary equipment of the turbine-generators. The drainage pit for the powerhouse will be provided below the bottom floor.

A building, annexed to the main powerhouse building, will be constructed to enclose a control room, relay room, offices, air conditioning equipment, station service equipment and store rooms, etc.

(3) Location of Main Transformers

A transformer yard will be provided adjacent to the main powerhouse building at the tailrace side, where two units of main transformers and 275 kV steel structures for connection to the switchyard will be installed.

(4) Powerhouse Foundation

The drilling investigation carried out at the powerhouse site revealed that there is a good bedrock layer of CH class at about 10 m deep below the ground surface (EL.30 m). No problem is anticipated. The deepest part of foundation excavation will reach EL. 1.6 m.

(5) Stability Analysis of Excavation Slope

Generally, deep excavation is required into the mountain at which a hydro-power station is located. From past experience, such excavation often creates large cut slopes resulting in troubles due to instability of the cut slope. Similarly, in this Project the slope is expected to rise to a maximum 62 m during construction.

Stability analysis against sliding was carried out and slope stability has been checked.

Physical properties of bedrock material are assumed as follows:

C-class bedrock	$C = 25.0\ t/m^2$, $\phi = 30°$
D-class bedrock	$C = 5.0\ t/m^2$, $\phi = 20°$

In this stability analysis, the seismic load is not considered since the cut slope is a temporary one only, during construction and not a permanent condition. The safety factors are calculated and obtained as below:

Condition	Safety Factor (Normal)
No ground water	1.25
Ground water	1.02
Ground water with PS anchor (300 ton/m)	1.27

Under the ground water condition, the factor of safety is marginal to 1.0. Therefore, reinforcement by PS anchors is planned.

In the above calculation, PS anchors of 300 ton per one meter of the lateral width of the slope have been adopted, but this is only preliminary, and is therefore subject to final design of this reinforcement after detailed investigation of the physical properties of bedrock materials.

The relative stability analysis diagrams are given in Figs. 9-2-17 and 9-2-18.

(6) Quantity of Powerhouse Building Work

Quantities of major work items are estimated as given below. The boundary with the tailrace work has been taken to the beginning point of tailrace outlet stab.

<u>Earth Work</u>

Excavation, common	73,700 m^3
Excavation, rock	161,300 m^3
Backfill	19,600 m^3
Total	254,600 m^3

<u>Concrete</u>

Draft tube	25,400 m^3
below EL 26 m	32,800 m^3
EL 26-45 m	15,800 m^3
Total	74,000 m^3

Architectural works include the following:

- Structural concrete above EL.45 m - 5,600 m^3

- Building facilities including air-conditioning, lighting, water supply and elevator, etc.

- Roofing

- External/Internal finishing

Miscellaneous works include the following:

- Annex building
- Gardening around the powerhouse
- Permanent roads

The estimates of these quantities are based on Figs. 9-0-1, 9-0-9, 9-0-11, 9-0-12, 9-0-13, 9-0-14, 9-0-15 and 9-0-16.

9.2.9. Tailrace

(1) Shape of Tailrace Section

The standard cross section of the tailrace was determined to have an invert floor width of 20 m and side walls sloped at 1:1, following the economic comparative study presented in Section 6.3.4. The tailrace channel will be lined with 50 cm thick reinforced concrete. The top elevation of concrete tailrace walls is set at EL.30 m and has a longitudinal gradient of 1/3000. When the maximum power discharge of 640 m^3/sec is made, the flow depth and velocity in the channel will be 7.0 m and 3.4 m/sec respectively. Water from the power station is first discharged into a 40 m long discharge basin which is connected to the tailrace channel. Total length of the tailrace up to the river is 499 m, and its floor elevation is EL.21.0 m.

(2) Layout of Tailrace

It is a basic concept that a tailrace should connect a powerhouse and the river in the shortest distance. For this Project, however, a consideration was given to avoid the influence of spillway discharges and therefore, the tailrace channel was curved toward the downstream direction in order to facilitate smooth return of turbine dischange to the downstream river course.

(3) Geological Condition Along Tailrace Channel

Bedrock along the route of the tailrace channel is mainly composed of green conglommerate, which is covered by a terrace deposit layer about 4 m thick. As the invert floor level of the tailrace is EL.21 m, relatively deep excavation (15 - 25 m) has to be carried out and it is expected that foundation rock for the tailrace will be bedrock of CM-CH class.

(4) Quantity of Tailrace Work

Major work items in the tailrace construction were estimated in the following quantities.

<u>Earthwork</u>

Excavation, common	277,500 m^3
Excavation, rock	241,700 m^3
Total	519,200 m^3

Miscellaneous works include the following:

- Fencing
- Excavation slope protection

The estimates of these quantities are based on Fig.9-0-10.

9.2.10. Switchyard

(1) Location and Size

A switchyard was located at the left bank of the tailrace, and is about 100 m distant from the powerhouse at EL 53.0 m.

In this switchyard, 275 kV switching equipment, including four circuits of outgoing facilities, and double bus-bar connections will be erected. Total area of the switchyard is approximately 11,000 m^2 or 124 m by 89 m wide. The switchyard layout outline is shown in Fig.10-2.

(2) Land Preparation

A ground cut ranging from EL 50 - 70 m will be executed for preparation of the switchyard site. During the excavation, the cut slope may reach as high as 29 m, but no particular problem is expected. Excavation slope stability analysis is shown in Figs. 9-2-19 and 9-2-20.

(3) Quantity of Switchyard Construction Work

Major work items and their estimated quantities are as follows:

Earthwork

Excavation, common	63,100 m^3
Excavation, rock	11,800 m^3
Total	74,900 m^3

Concrete

Miscellaneous works include following items.
(Buildings are not necessary)

- Fencing
- Perimeter roads

The estimates of these quantities are based on Figs.9-0-1 and 9-0-14.

9.2.11. Bottom Outlet

(1) Location and Type

The river Diversion Tunnel No.1 will be provided with a gate chamber and used as a bottom outlet. An inlet facility with the top elevation of EL.50.0 m will be constructed at the portal of the diversion tunnel.

Two gate facilities will be provided: One is a jet flow gate for making normal in-service discharges and the other is a ring-follower gate that will be installed at the upstream side and function as the guard gate.

A tunnel access will be constructed to approach the gate chamber from the downstream dam abutment at approx. EL.50 m.

(2) Outline of Bottom Outlet Facility

Jet Flow Gate

Design Head	54.6 m
Inlet Dia.	2.0 m
Max. Discharge	84 m^3/sec (WL 88.1 m)
	77 m^3/sec (WL 80.0 m)
	46 M^3/sec (WL 50.0 m)

Ring Follower Gate

Design Head	54.6 m
Inlet Dia.	2.0 m

Guide Pipe

Design Head	54.6 m
Design Ext. Load	3.0 kg/cm^2
Inner Dia.	2.0 - 2.5 m

(3) Quantity of Bottom Outlet Work

Major work items and their estimated quantities are as follows:

Tunnel excavation	3,600 m^3
Tunnel concrete	1,500 m^3
Gate steel	360 ton

Miscellaneous works include following items:

- Lighting facility
- Drainage work
- Inlet gate

The estimates of these quantities are based on Figs.9-0-1 and 9-0-8.

9.2.12. Intake Gate

(1) Design Features

Type	:	Roller gates
Number	:	2 units
Width	:	8.6 m
Height	:	8.6 m
F.W.L.	:	EL.88.1 m
H.W.L.	:	EL.80.0 m
Sill Level	:	EL.48.0 m
Hoist Plaform Level	:	EL.92.0 m
Max. Hoisting Height	:	44 m
Hoisting Speed	:	0.3 m/min.
Emergency Shutdown speed	:	3.0 m/min.
Bypass valve	:	Inlet Dia. 30 cm
Weight (2)		
Gross Weight	:	400 ton
Gate body	:	83 ton/gate (Service Gate) 43 ton/gate (Maintenance Use)

9.2.13. Tailrace Gate

(1) Design Features

Type	:	Roller gates
Number	:	6 units
Width	:	6.4 m
Height	:	5.7 m
Sill Level	:	EL.4.3 m
Water Level of Stilling Basin	:	EL.28.0 m

Hoist Foundation Level	: EL.45.0 m
Max. Hoisting Height	: 25 m
Hoisting Speed	: 0.3 m/min.
Weight (2)	
Gross Weight	: 300 ton
Gate body	: 23 ton/gate

9.3. Quarry Site Plan

The proposed quarry site where geology is mainly tuffacious conglomerates was described in detail in Section 4.11 based on the investigation results. Listed below are the estimated quantities of construction materials to be obtained from this quarry site.

	Construction Quantity $10^6 m^3$	Run-of-mine Collection $10^6 m^3$
Rockfill material for dam	3.0	2.8
Filter material for dams	0.63	0.8
Structural concrete aggregate	0.33	0.5
Roadway base material	0.83	0.9
Total	4.79	5.0

The quarrying area will cover 16 ha (400 m x 400 m) and the removal of weathered rock in the overburden is estimated to amount 1,250,000 m^3.

Quarry operation at the site is planned on the basis of the bench cut method, with 15 m high benches. The plan and longitudinal drawings of the quarry site are given in Figs. 9-3-1 and 9-3-2 respectively.

9.4. Borrow Site Plan

Geology of the borrow site is as described in Section 9.11. Core material to be used for the main dam, upstream and downstream cofferdams, and saddle dams is estimated to be 700,000 m^3 in total. The necessary borrow site development will extend over an area of 14 ha (375 m x 375 m) assuming that above amount of core material is collected within an average depth of 5 m.

The surface soil grubbing, estimated at about 2 m, will produce about 281,000 m^3 of spoil material.

9.5. Spoil Areas

Soil and rock debris produced in the excavation operations at various structure sites are estimated as follows:

Type	Quantity 10^3m^3
Soil	2,837
Rock	1,056
Total	3,893

Out of these materials, about 1,330 x 10^6m^3 will be utilized for rock embankment of the main dam, earth material for Saddle Dam II, back-fill material at the spillway and powerhouse sites, and the remainder will be disposed of into spoil banks.

Total volume of spoil materials is estimated as follows:

	Construction Quantity 10^3m^3	Disposal Quantity 10^3m^3
Soil	2,840	4,828
Rock	780	1,170
Unsuitable rock quarries	1,250	2,125
Overburden at the borrow pit	280	476
Total	5,150	8,599

If the above disposal material amounting to 8,600 x $10^3 m^3$ is deposited to an average height of 8 m, the total disposal area required will be 108 ha.

In order to reduce such spoil disposal, every measure for use of excavated materials should be considered.

Sizable potential disposal areas are at the east side of quarry site, around Saddle Dam II, the upstream bank at the main dam site, and the immediate downstream area of the main dam site.

9.6. Construction use and Permanent Roads

The necessary permanent roads are listed below:

(a)	Tualang - Power Station (Left bank of the Lebir River)	8 m width
(b)	Power Station - Intake - Main Dam - Spillway (Left bank)	5 m "
(c)	Power Station - Main Dam Downstream Face - access tunnel to Bottom Outlet	4 m "
(d)	Saddle Dam I (Left bank) - Spillway (Right bank)	5 m "

The following construction roads are required.

(a)	Existing logging road - Quarry site	10 m width
(b)	Existing logging road - Saddle Dam II	15 m "
(c)	Existing logging road - Diversion Tunnel Portal	8 m "
(d)	Quarry site - Main Dam Site	15 m "
(e)	Borrow site - Main Dam Site	15 m "