CHAPTER 7

AGRICULTURAL PRODUCTION IMPROVEMENT

7. Agricultural Production Improvement

7.1. The Proposed Cropping Pattern

The full scale introduction of high yielding varieties is one of the most important items for setting up the future cropping pattern. The proposed cropping pattern is decided as shown in Figure 7-1-1, aiming at preventing as much as possible damages to be caused by typhoons as well as downpour and taking into account the existing cropping pattern in the project area.

7.2. Yield of Paddy in the Future

The yield of paddy in the Philippines generally evidences a steady tendency of increase. In 1976, the yield was 1.721 ton/ha; while, in 1980, it increased to 2.124 ton/ha. The average annual rate of increase during the period of 5 years from 1975 to 1980 was 4.5%.

In the "Five Year Development Plan of the Philippines" (1978-1982) the yield is planned to be increased from 1.865 ton/ha in 1978 to 2.635 ton/ha in 1987, with an annual average rate of increase of 3.9% during the period.

If the aforesaid growth rate is kept unchanged over the coming decade, the production of paddy will be 1.47 times through 1.55 times as large as that one prevailing presently. However, it is perfectly feasible to expect an increase of 1.45 times through 1.50 times in the production of paddy after completion of the Mabini Agricultural Development project owing to the following reasons. The irrigation project will make possible the supply of a sufficient amount of irrigation water, which makes the capital investment for the agricultural production effective, and increases of agricultural production and income of farmers by strengthening the agricultural extension services.

Data regarding the future yield of paddy per unit area are presented in the Table 7-1. In this project, the proposed yield per unit area is 4.58 ton/ha for the rainy season crop and 4.79 ton/ha for the dry season crop. The proposed yield can be achieved when introducing high yielding varieties in the whole project area and carrying out the paddy cultivation in accordance with the Masagana 99 Project.

7.3. Agricultural Technique

7.3.1. Damage Caused by Blight and Insects

1) Major items of damages caused by diseases and insects

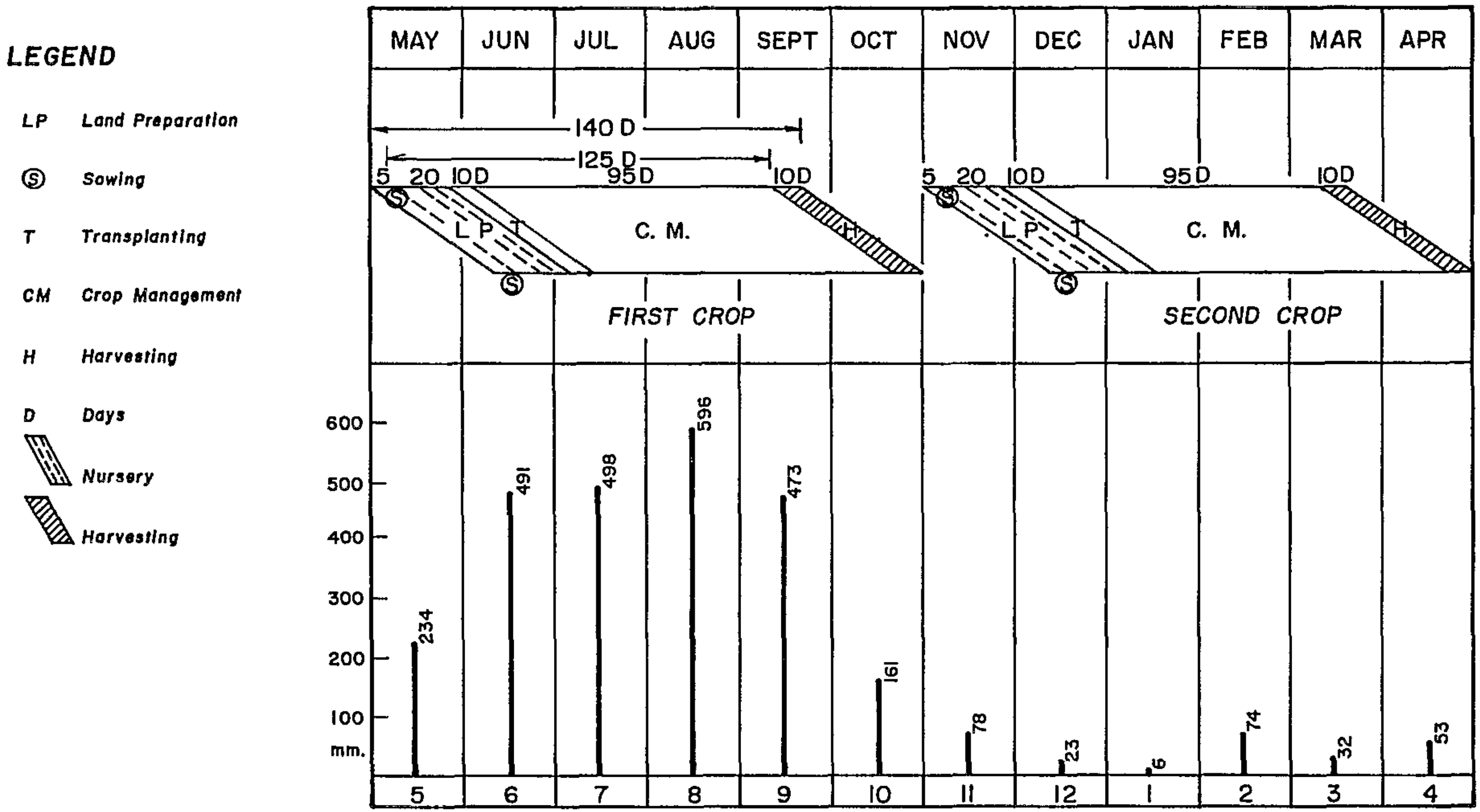
According to the results of observation of paddy field and of interview with farmers living in the project area, the major items of damages caused by disease and insects can be summarized as follows. The said damages seem to be more remarkable in the rainy season than in the dry season. However, during the dry season, some damages will be caused by rats.

Major harmful insects

- Green leaf hopper
- Rice case worm
- Rice stem boder
- Rice paddy bug

Fig.7.1.1 **POTENTIAL CROPPING PATTERN**

Maturity - 125 day



NORMAL RAINFALL, MABINI (20 years of record), prepared by WEATHER BUREAU

The Steps of 11 instructed in the Masagana 99 Project recommends the eradication of the green leaf hopper, because it is considered to carry the tungro virus. The selection of the variety with resistance against the green leaf hopper and against the virus carried by the said insect is the most important theme regarding the extension of the high yielding variety in the project area.

The fact that most of the farmers mentioned the name of the insects at the occasion of the interview, evidences their high concerns to damages caused by the disease and the insects.

2) Prevention of epidemics in plants

In course of the study, 10 farmers were interviewed in the project area. Only one farmer used no insecticide at all and all other farmers interviewed informed that they were using 1 through 5 kinds of insecticides. The financing granted to farmers through the Masagana 99 Project seems to be promoting the use of insecticides. (Part of the loan is given to the farmers in form of the coupons for procurement of insecticides and fertilizers).

The most important measures regarding prevention of epidemics is to take timely correct countermeasures. Therefore, it is indispensable to make simple observations regarding the emerge of the insects and break out of damages. The observations should be simple enough in order to make possible its execution by any farmer.

7.3.2. Farmers

The intention of the farmers to increase agricultural production is generally high, but the present productivity shows considerable variations in each individual case, depending on the cultivating technique. The farmers of Alaminos and Sual who have been achieveing high productivity of paddy are well aware of the 16 technical steps of the M-99 Project, and have a high technical level even in the steps of the paddy nursery and paddy transplantation steps.

7.3.3. Extension Services

Serveral technical experts are working in the local office of the Ministry of Agriculture served for the four towns of the project area for the extension of agricultural techniques. However, they are too busy with jobs regarding the M-99 and other activities to extend technics into their whole territory.

As for their technical level, there are 1 or 2 highly qualified experts in prevention of damages caused by disease and insects.

The insecticide dealers offer the technical services to the farmers, but there seems to be no other remarkable activities to extend services to the farmers. With regard to the supply of guaranteed seeds by the BPI, there is no production of the seeds within the project area. Accordingly, it is indispensable to go to other areas in order to obtain the seeds.

7.3.4. Agricultural Experiment Stations

The extension services and the improvement of the agricultural technics of the project area can be made under the functions of the Central Luzon Station, but there is no agricultural experimental station in the Pangasinan Province and the project area can be considered as a "desert" in terms of the agricultural technique extension.

7.3.5. Agricultural Technique

Elementary education is provided at every villages or hamlets located within the project area. However, there is no institution at all for the training of the future experts, leaders and successors of the farming activities with high technics. Therefore, it is suggested that the establishment of an institution aimed at providing training course of the agricultural technics in the West Pangasinan Zone, will result a success, because the local people are showing interesting in education.

7.4. Investements for Production of Paddy

In the project, investments for the production of paddy are estimated as summarized in Table 7.4.1, taking into consideration the situation prevailing presently and the future plan proposed by the Agro-Economical Survey Report of the NIA (1979). The investment amounts presented in the aforesaid table are determined by the results of local hearings and the collected data and information. With regard to the investments for the agricultural production, it is necessary to point out that the Masagana 99 Project itself presents financial problems. For example, members of M-99 can obtain low interest loans of 300 Pesons/per hectare for the application of the nitrogen fertilizers, but that amount makes possible the application of only 50 Kg/ha, and that can not be considered as sufficient.

Accordingly, establishment of an agricultural financing system is desirable in addition to the M-99.

Table 7.4.1 Input for Rice Crop Production

Input		Unit	Wet Season		Dry Season
			Rainfed	Irrigated	Irrigated
Present:					
Cultivation	- Mechanical	(% Area)	5	55	40
	- Animal	(% Area)	95	45	60
Seed	- Transplanted	(kg)	50	50	50
Fertilizer	- N	(Nutrient kg)	22	51	52
	- P	(Nutrient kg)	15	28	28
	- K	(Nutrient kg)	9	27	27
Pesticide	- Liquid	(qt)	.65	1.45	1.30
	- Granule	(kg)	2.00	24	20
Herbicide	- Liquid	(qt)	.58	.20	.20
	- Granule	(kg)	8.18	12	14

Harvesting	- Mechanical	(% Area)	100	100	100
	- Manual	(% Area)	-	-	-
<u>Future With Project:</u>					
Cultivation	- Mechanical	(% Area)	-	70	80
	- Animal	(% Area)	-	30	20
Seed	- Transplanted	(kg)	-	65	70
Fertilizer	- N	(Nutrient kg)	-	70	80
	- P	(Nutrient kg)	-	30	30
	- K	(Nutrient kg)	-	0	0
Pesticide	- Liquid	(qt)	-	2.00	1.95
	- Granule	(kg)	-	33.00	30.00
Herbicide	- Liquid	(qt)	-	2.00	2.00
	- Granule	(kg)	-	16.5	21.00
	- Mechanical	(% Area)	-	85	90
	- Manual	(% Area)	-	15	10

7.5 Strengthening of Supporting Facilities and Services

7.5.1 Required Labor Force

The monthly requirements of labor forces occurring presently and expected to occur after the completion of the project are summarized in the following table.

Monthly Labor Requirement
(1,000 man.day/month)

Month	Present Requirement	After Completion of Project
Jan.	7.7	138.0
Feb.	9.1	92.0
Mar.	3.5	207.0
Apr.	-	218.5
May	74.5	69.0
June	322.0	414.0
July	129.5	126.5
Aug.	41.3	69.0
Sept.	60.4	218.5
Oct.	51.5	253.0
Nov.	158.2	69.0
Dec.	20.3	425.5
Total	878.0	2,300.0

Presently the peak requirement of labor force occurs in June, but it may be shifted to December due to the new cropping pattern after completion of the project.

Labor requirement in December is anticipated to be 400,000 man-day/month. It is foreseen that some 20,000 farm labores will be required by assuming 20 working days per month. There is 11,500ha of the project area, while the number of farmings families living in the area is approximately 7,700 (1.5 ha of farm holding per farmers family). 15,000 workers will be available from the farming families living in the project area, by assuming that 2 workers can be avaiable from a family.

It is still necessary to recruit 5,000 workers from sources other than the farming families living in the project area. According to data of 1970, there are 3,700 unemployed persons in the 4 towns and villages located in the project area. It may therefore be possible to absorb them in the agricultural sector.

Furthermore, there are also landless farmers living in the region and therefore it could be assumed that the labor requirements can be perfectly met within neibouring areas of the project.

7.5.2 Source of Seeds

In the Pangasinan Province, there are presently 18 seed centers which produce and distribute high yielding varieties of paddy. These seed centers have a total land area of 225ha. However, there is no seed center in the project area, and in addition, there is no seed grower of certified seed too. In view of the aforesaid circumstances, it seems to be indispensable to create a new seed center in the project area. The seed supply system of the Bureau of Plant Industry (BPI) is as follows.

Breeder's seed	Produced mainly at Experiment Stations
Registered seed	Produced by seed growers
Certified seed	Produced by seed growers

Seeds called "good seed" are also used as substitutes of the breeder's seeds, certified seeds and registered seeds mentioned above.

Assuming that 50% of the seed requirements of the benefited area of 11,500ha is provided by each farmer, the quantity of certified seeds required for 5,000ha of paddy field is calculated as follows.

5,000ha x 0.05 ton (1 cav. per ha) = 250 ton

125ha of paddy field is required for production of seed paddy, by assuming an yield of 2 ton/ha, because rice seed requires a severe selection. The aforesaid area is equivalent to approximately 1/2 of the seed centers existing presently in the Pangasinan.

Instead of creating many small sized seed centers, it is recommendable to create four seed centers with an area of approximately 30ha in the vicinity of Alaminos, Bani, Mabini and Sual. It is necessary to construct an experiment station with an area of approximately 1.5ha in Alaminos, in order to provide the Breeder's seed.

7.5.3 Fertilizers and Chemicals

The quantities of fertilizers and chemicals required per unit area are presented in Table 7.4.1. The quantities of fertilizers and chemicals required in the whole project area are calculated as follows.

Requirement of Fertilizers

(Unit: ton)

Fertilizer	Present Situation			After Completion of Project		
	Wet Season	Dry Season	Total	Wet Season	Dry Season	Total
Nitrogen (N)	302.3	36.4	338.7	805	920	1,725
Phosphorus (P)	194.6	19.6	214.2	345	345	690
Potassium (K)	134.1	18.9	153.0	-	-	-
Total	631.0	74.9	705.9	1,150	1,265	2,415

Requirement of Chemicals

Chemicals	Present Situation			After Completion of Project		
	Wet Season	Dry Season	Total	Wet Season	Dry Season	Total
Insecticide						
- Liquid(q.t)	8,835	910	9,745	23,000	22,425	45,425
- Powder(ton)	60.4	14	74.4	379.5	345	724.5
Herbicide						
- Liquid(q.t)	6,024	140	6,164	-	-	-
- Powder(ton)	100.6	9.8	110.4	189.8	241.5	431.3

The required amount of fertilizers, which is presently 630 tons during the rainy season, is anticipated to be 1.265 tons, i.e., approximately 2 times as large as the present requirement. The additional fertilizer of 630 tons will be required after completion of the project. Approximately $200m^2$ of warehouse is required in order to ensure the temporary storage of the aforesaid additional quantity of fertilizers, and furthermore, both retail sellers of fertilizers and farmers are required to make efforts to ensure an appropriate storage of fertilizers.

7.5.4 Threshing and Polishing Facilities

The implementation of the present project is expected to bring the production of unhulled rice described below. In the future, the padd production of 55,000 tons will be harvested during the dry season and expected to surpass the present paddy productions of 24,000 tons in the rainy season. As can be seen from the figures above, an additional production of 30,000 tons is expected and threshing and polishing facilities will be required as follows.

(1) Polishing Facilities

The existing polishing machines of paddy have very small processing capacities, with an average of the order of 4 tons/day. The installation of an additional quantity of 60 units of polishing machines is required, assuming that the additional quantity of paddy would be processed by machines of the same capacity as the existing facilities.

30,000 tons ÷ 6 months ÷ 20 days/month ÷ 4 tons/day
= 60 units

The additional paddy production will be distributed through the National Food Authority (NFA). Therefore, the polishing facilities should be strengthened under the responsibility of the NFA.

(2) Threshing Facilities

The existing threshing machines have very small processing capacities, with an average of the order of 3 ton/day. Therefore, a total of 340 units of thresher is required, by assuming that the threshing of the additional paddy production of 30,000 tons is carried out within the harvesting period lasting 1.5 month, as shown below.

30,000 ton ÷ 1.5 month ÷ 20 days ÷ 3 ton/day
= 340 units

The quantity of machines required to cope with the total production of 55,000 tons of paddy will be 600 units. This figure corresponds to approximately 1 unit of threshing machine per 12 farmers, because there are approximately 7,700 farmers living in the project area.

7.5.5 Rice Warehouse

The increase of paddy production per cropping season is 30,000 ton, and this quantity will be distributed through the National Food Agency (NFA). When paddy should be packed in bags and piled up in the warehouse, the storage capacity per square meter of warehouse is of the order of 3 tons. Accordingly, the National Food Agency should construct 10,000m^2 of warehouse in order to store the additional quantity of paddy harvested in the project area. The construction of warehouses at 10 places is required, by assuming that each one has an area of approximately 1,000m^2.

7.5.6 Credit

Financing is presently provided to the farmers by the Masagana 99, through the Rural Bank and the Philippines National Bank. The upper limit of the aforesaid financing is 1,600 Pesos per ha. The total capital required to cover the whole project area is calculated as follows, by assuming that all irrigated paddy field are benefited by the credit system.

Capital for Financing by the Masagana 99 System

(Unit: 1,000 ₱)

Cropping Season	Present Situation	After Completion of Project
Wet Season	2,720	18,400
Dry Season	1,120	18,400

In the Masagana 99 System, 500 Pesos among the total financing amount is composed of fertilizers and chemicals. However, approximately 1,100 Pesos of fertilizers and chemicals are required according to data of the project. The balance of the financing provided by Masagana 99 is 600 Pesos per ha, and a new financing system is required in order to cover the balance. The total amount of capital required in order to make possible the aforesaid additional financing is 6,900 thousands Pesos per one cropping season.

7.5.7 Extension

According to the studies carried out in Alaminos, Bani, Mabini and Sual, there are 5 to 10 technicians in charge of the extension services, and the extension system seems to be good, prima facie.

However, the measures described below are required in order to increase the paddy productivity in the project area.

(1) Personnel

From the quantitative point of view the personnel existing presently seems to be sufficient. However, it is very important to upgrade the technical level of the experts in charge of extension services.

(2) Technique

Technical extension centers should be created in the project area and the extension service officers should acquire the technique required to teach every details of paddy cultivation to the farmers, by stepping in the paddy field with them. These extension service officers should join an adequate training in the technical centers described later in this report.

(3) Facilities and Equipment

With exception of Mabini, the extension offices are located in the town offices, but they are small in size and the equipments in such extention offices presently provided are very poor. Therefore, a considerable reinforcement is required.

Motorcycles and scooters should be urgently provided, because means of transportation and communication are very essential for extension services.

(4) Contents of the Work

More importance should be emphasized on the actual field technology in cultivating and harvesting paddy.

(5) Extension Organization

It is estimated that there are approximately 7,000 farmers within the project area. It is recommendable to carry out the extension of the cultivation techniques by assigning one contact farmer for each 10 farmers and one extension service officer for every 70 contact farmers.

7.5.8 Research

Sophisticated research activities feem not to be appropriate in the project area. Fied techniques directly necessary for the extension services and education of extension technicians are indispensable. The major items of the research activities should be in compliance with the technical needs listed below.

(1) Masagana 99

These are standard techniques for the whole national territory of the Philippines. Therefore, they should be modified in such a way to match the conditions prevailing in the project area and appropriate technology for the project area should be adapted.

The 16 technical steps of Masagana 99 are composed of a praiseworthy technical system and therefore efforts are required in order to adapt them to the project area after necessary modification.

(2) Technical Center

It is recommendable to construct a technical center provided with the functions required to carry out the experiments listed below, in addition to the seed production and education of the extension service officers. The center should have a total area of 22ha of land provided with complete irrigation and drainage facilities, subdivided as follows.

- 15.0 ha for experiments
- 2.0 ha for training
- 5.0 ha for construction of the required facilities

(3) Technical Demands after Supply of Irrigation Water

An increasing demand for agricultural techniques is expected after completion of the project, a preparatory period of 10 years may be too short to meet the said demands. The major items of the said demands are those ones affixing the "o" mark in the list below.

O 1) Agricultrual meteorological observation

2) Crop production

O (1) Adaptation and screening of high yield variety
O (2) Optimum plantation time and harvesting time
(3) Optimum type of cultivation, especially weeds control
O (4) High temperature damage of rice plant in dry season
O (5) Rice cultivation method on swampy paddy

3) Soil and fertilizer

O (1) Soil management of swampy paddy
(2) Soil management method of every soil type
O (3) Optimum fertilizer application for every soil type
(4) Application method of dry season rice crop
(5) Soil erosion of mountain sides

4) Plant protection

O (1) Pathogenesis of insects and diseases
(2) Preparation of simple occurrence forcast
O (3) Preparation of plant protection standard

5) Mechanization

(1) Land preparation technics by low power tracter
(2) Improvement of customary plow and harrow
O (3) Rice planting by small machine
(4) Cutting by machine on wet condition
O (5) Threshing machine and its use technics

6) Post harvesting

O (1) Method of decreasing harvesting loss
O (2) Drying method, facilities and system about after harvesting

7) Irrigation and drainage

O (1) Irrigation management cooperation system
O (2) Irrigation method in dry season
O (3) Drainage method and its effect on swampy paddy

Table 7.5.1 DEPLOYMENT OF FARM MANAGEMENT TECHNOLOGIST

(a)	FARM MANAGEMENT TECHNOLOGIST	BARANGAY COVERAGES
	MRS. LIGAYA G. ARIOLA	1. BALANGOBONG 2. SAN VICENTS 3. TANGCARANG
	MRS. GLORIA T. CABATIC	1. POCALPOCAL 2. BUED 3. SABANGAN 4. TELBANG 5. VICTORIA
	MR. POBERIO DELA CRUZ	1. AMANGBANGAN 2. DULACAC 3. INERANGAN 4. STA. MARIA 5. TAWINTAWIN
	MISS CORAZON PADUYOS	1. CABATUAN 2. BOLANEY 3. AMANDIEGO 4. BALAYANG
	MISS CRISPINA ONATE	1. TANAYTAY 2. MACATIW 3. MAGSAYSAY 4. LUCAP 5. POBLACION
	MRS. FEB. RABAGO	1. POGO 2. POLO 3. SAN ROQUE 4. SAN JOSE
	MISS MERCURIA RABANAL	1. ALOS 2. BISOCOL 3. QUIBUAR 4. LINANSANGAN
	MR. EDILBERTO R. RAPUES	1. PANGAPISAN 2. CAYUCAY 3. MONA 4. BALEYADAAN
	MRS. TERESITA B. MALAPOTE (MIS DISTRICT OFFICER)	1. TOCOC-PALAMIS
(b)	HOME MANAGEMENT TECHNOLOGIST	
	MRS. FELICIDAD BACAY	1. WHOLE MUNICIPALITY
(c)	LIVESTOCK INSPECTOR	
	MR. ROMED PADAONG	1. WHOLE MUNICIPALITY

Table 7.5.2 Republic of the Philippines
MINISTRY OF AGRICULTURE
Region No. 1
Dagupan City

Number and Specialty of Each Technicians by Municipalities

Municipalities	No. of Technician	Position	Specialty
Alaminos	1	Mun. Agric'l. Officer	Municipal Supervisor
	7	FMT - I	All MA Programs
	1	FMT - II	- do -
	1	PPCT	Crop Protection
Bani	1	Mun. Agric'l. Officer	Mun. Supervisor
	2	FMT - I	All MA Programs
	1	L.I.	Livestock & Poultry
	1	HMT	Home Management
	1	RYDO	Rural Youth Development
Mabini	1	Mun. Agric'l. Officer	Mun. Supervisor
	1	FMT - I	All MA Programs
	1	HMT	Home Mgt.
	1	L.I.	Livestock & Poultry
Sual	1	Mun. Agric'l. Officer	Mun. Supervisor
	1	FMT - II	All MA Programs
	2	FMT - I	- do -
	1	PPCW	Crop Protection

NOTE: All MA Programs on Crop Production, Cooperatives, Soils and Extension works.

FMT : Farm Management technologist
PPCT: Plant Pest Control Technologist
L.T.: Livestock Inspector
HMT : Home Management Technisian
RYDO: Rural Youth Development Officer
PPCW: Plant Pest Control Worker

CHAPTER 8

WATER REQUIREMENT

8. Water Requirement

8.1. Proposed Cropping Pattern

The proposed cropping pattern for the project area is described in 7.1 of this report, taking into consideration the prevailing cropping pattern at present. Accordingly, the irrigation water requirement is calculated in this section, based upon the cropping pattern.

8.2. Water Requirement

8.2.1. Reference Crop Evapotranspiration, ETo

The methods listed below are proposed and recommended by the international institutions, for the purpose of calculating the evapotranspiration of crops.

(1) Blaney-Criddle Method

(2) Radiation Method

(3) Penman Method

(4) Pan Evaporation Method, etc.

After examining the available data, it was decided to calculate the value of evapotranspiration by the Penman method, by utilizing the observation data collected at the Dagupan City. The Dagupan Observation Station is located in the vicinity of the project area and supplies sufficient data of necessary facoters for the calculation of the Panman method. (Refer to Supporting Report 8.1.1.)

8.2.2. Crop Water Requirement, ETcrop

The crop water requirement (ETcrop) is calculated by multiplying the reference crop evapotranspiration (ETo) with the crop efficiency (Kc).

$$ETcrop = Kc \cdot ETo$$

where Kc; Crop Efficiency

As for the crop efficiency Kc of paddy, the values listed below are adopted (among the values of crop efficiency adopted by the FAO), by taking into consideration factors like local peculiarities, wind velocity and other relevant conditions.

Period of growth	Rainy season	Dry season
1st month	1.1	1.1
2nd month	1.1	1.1
Intermediate period	1.05	1.25
Last 4 weeks	0.95	1.0

8.2.3. Net Farm Requirement

The net farm requirement of paddy is calculated by adding the water requirements corresponding to the following items

- Water requirement for the land preparation and the nursery
- Supplying water for the cultivation
- Percolation

to the crop water requirement subtracting the effective rainfall.

(1) Water Requirement for Land Preparation & Nursery

The following quantities of water are required for the purpose of land preparation and nursery.

- Rainy season (ETo + P + 80)mm
- Dry season (ETo + P + 70)mm

(2) Supplying Water for Cultivation

The total depth of water required for the cultivation during the growth period of paddy is as follows.

Upper limit	Lower limit	Rainy season	Dry season
150	0mm	May 1st - May 25th	Nov. 1st - Nov. 25th
50	20mm	May 26th - July 20th	Nov. 26th - Jan. 20th
150	25mm	July 21st - Oct. 7th	Jan. 21st - Apr. 9th
	0mm	Oct. 8th - Oct. 31st	Apr. 10th - Apr. 30th

(3) Deep Percolation

Results of the measurement of deep percolation carried out in this area indicate that it has values ranging from 0.69mm/day to 1.10mm/day, with an average of 0.88mm/day. According to the results, a design value of the order of 1mm/day seems to be appropriate in this case. However, the value of 2mm/day, mentioned also in the design standard of the NIA, is adopted because the deep percolation is expected to be increased after improvement of the paddy field, the consolidation and the drainage system.

(4) Effective Rainfall

The effective rainfall in case of the cultivation of paddy is calculated by means of the "Paddy Operation Study", where the water balance on the paddy field level is calculated. In the "Paddy Operation Study" method, the actual rainfall is taken as the inflow and the crop water requirement is taken as run-off and the balance between them is stored in the paddy, within the limit values of the "Supplying Water for Cultivation" mentioned in the item (2) above. In case of the surplus water, it is discharged in the drainage canals, while in case of the shortage, the required quantity of water is replenished. The value of effective rainfall can be obtained by subtracting the quantity of replenished water calculated as described above from the crop water requirement.

8.2.4. Diversion Water Requirement

The diversion water requirement is calculated from the net farm requirement, taking into consideration the overall efficiency.

The following values of efficiency are taken for the calculation of the overall efficiency.

	Dry season	Rainy season
Farm waste	0.8	0.7
Conveyance efficiency	0.8	0.8
Operation effeciency	0.9	0.9

Accordingly, the overall efficiency is the following values.

Overall efficiency	0.58	0.50

As for the irrigation efficiency, the values of the overall efficiency adopted in the projects of the NIA do not exceed 50%. Observation of irrigation efficiency has been carried out in some existing irrigation projects, and the obtained data are almost same as the aforesaid value.

However, a high overall irrigation efficiency should be adopted, because irrigation water is stored in the dam and distributed over an area exceeding 10,000ha in this project. In order to achieve the high overall efficiency mentioned above, the water distribution system and the maintenance & operation system should be upgraded. In other words, the efficiency can be improved by reinforcing the training program of the employees of the NIA and farmers in charge of the maintenance and operation of the irrigation system and by equipping more complete diversion facilities (e.g. installation of turn-out gates).

The gross quantity of irrigation water per unit area is calculated as follows, as a result of the considerations above.

Values of the diversion water requirement per unit area are calculated as follows.

	1967	1968	1969	1970
Jan.	-	484	477	414
Feb.	-	472	411	445
Mar.	-	249	239	226
Apr.	-	13	2	0
May	82	41	82	-
June	107	145	51	-
July	108	114	52	-
Aug.	0	0	0	-
Sep.	0	0	0	-
Oct.	0	0	0	-
Nov.	16	118	115	-
Dec.	449	460	416	-

Unit: mm

8.2.5. Maximum Water Requirement

The unit water requirement in the basic drought years are

January 1968 R = 15.61mm/day

February 1968 R = 16.28mm/day

Therefore, the maximum value of the unit water requirement occurs in February, 1968. Consequently, the maximum unit water requirement at the intake becomes

1.884 Liter/sec/ha.

Since the service area for irrigation of the project is estimated at 11,500ha (Refer to Chapter 9. Water Balance Analysis of the Reservoir), the maximum water amount is calculated as below,

$$Q_{max} = 1{,}884 \times 11{,}500 = 21{,}666 \text{ Liter/sec}$$
$$= 21.666 \text{ m}^3\text{/sec.}$$

CHAPTER 9

WATER BALANCE ANALYSIS OF THE RESERVOIR

9. Water Balance Analysis of the Reservoir

9.1. General

The Mabini reservoir is planned to ensure the quantity of water required for the proposed irrigable area. A series of water balance analyses regarding the quantity of water required for the project area and the quantity of water dischargeable from the reservoir with the assumed scale is carried out below. The quantity of required water for the project area is estimated in accordance with the proposed cropping pattern.

9.2. Water Balance of the Reservoir

The study of the water balance between the available inflow and the required quantity of water was carried out throughout the period of 3 years, by assuming that at the beginning of the study (May of 1967), the stored water level is the low water surface (LWS).

At first, the normal water surface (NWS) is set at EL63.0m, in order to determine the maximum irrigable area and the lowest intake level (Refer to 6.2 Maximum Water Surface). The study of the water balance was carried out in several cases of irrigable areas. Next step is carried out the comparative study of the results, and the results obtained in "10.3. Intake Level and Service Area" of the main report.

The river maintenance flow of the Balincaguin River at the downstream of the proposed damsite is set at 2.3 m^3/s for 225 square kilometers of catchment area by using $q = 1.0 m^3/s/100Km^2$ and is kept constant flow throughout the year. (Refer to 4.6. of main report)

9.3. Determination of Irrigable Area and Intake Water Level

The comparison study resulted the planned irrigable area and intake level as 11,500ha and EL38.0m, respectively. The required capacity of the Mabini reservoir is estimated at 240 million cubic meters.

Accordingly, the major items of the Mabini reservoir are as follows.

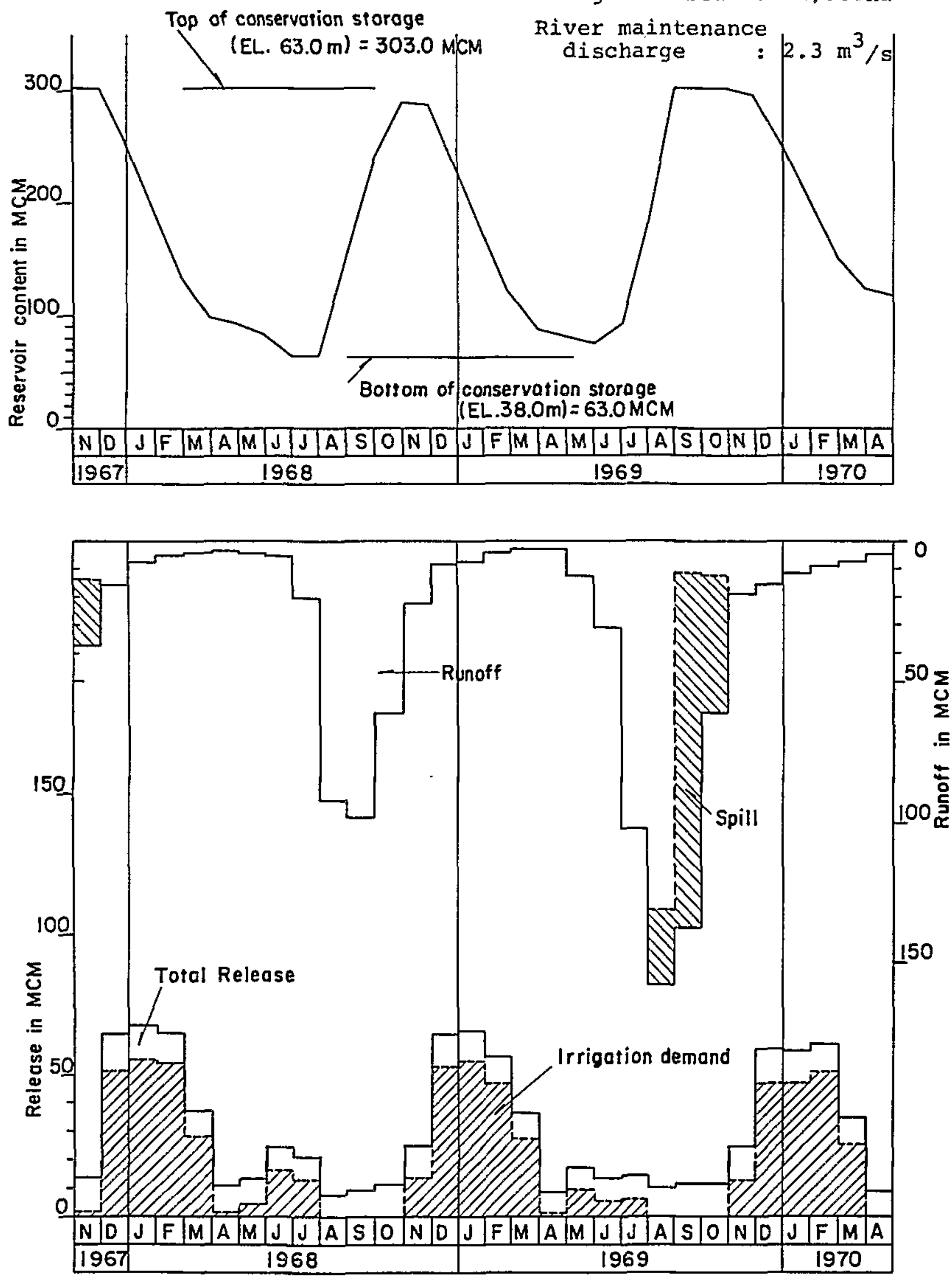
Normal water surface (NWS)	EL63.0m
Low water surface (LWS)	EL39.0
Effective storage capacity	240 million cubic meters
Dead water	63 million cubic meters
Total storage capacity	303 million cubic meters

The operation of the Mabini reservoir is presented in Figure 9.2.1. According to the diagram of operation, the stored water level does not recover to the normal water surface (NWS), EL63.0m immediately after the rainy season in 1968. However, as shown in the diagram, no water shortage is made during the following dry season as well as its succeeding period. Accordingly, it is considered that the use of water during 1968 does not have any deffect on the following cropping.

Fig.9.2.1 Reservoir Operation

Irrigable area : 11,500ha

River maintenance discharge : 2.3 m^3/s



9.4. Additional Study

Request regarding the studies on the water balance in operation of the reservoir for a long term basis and on the maximum possible irrigation area was presented at the occasion of the meeting with the NIA authorities, held in 24th of February of 1982. The results of the aforesaid studies are as follows.

The period of 11 years ranging from May, 1959 to April, 1979 is selected for the said study, in view of the availability of the hydrological and meteorological data.

Results of the additional water balance analysis are presented in Figure 9.4.1, and seasonal runoff and rainfall are presented in Table 9.4.3.

Fig. 9.4.1 Reservoir Operation

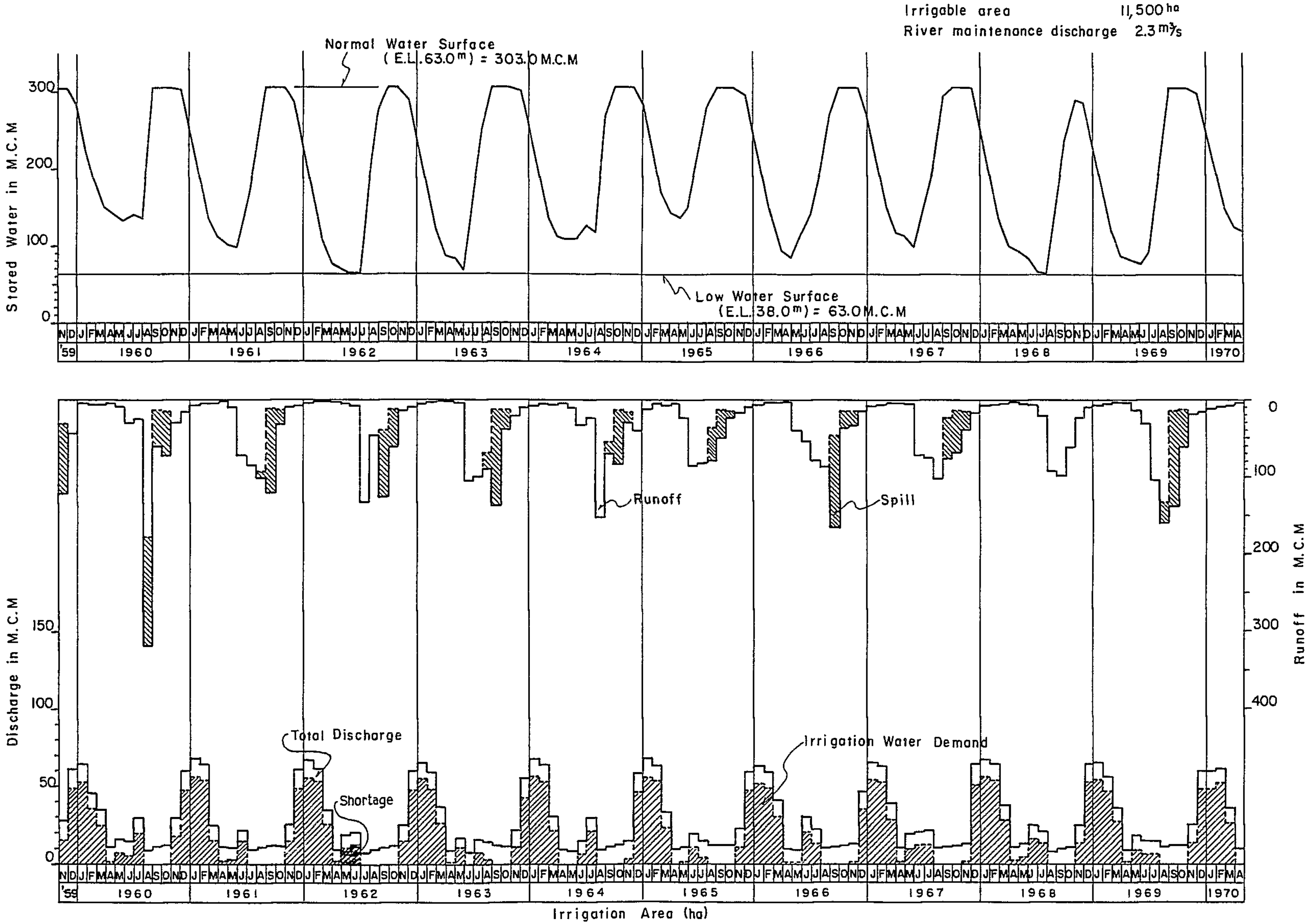


Table 9.4.1 Shortage Amount

	11,500 ha Wet season	11,500 ha Dry season	12,000 ha Wet season	12,000 ha Dry season	12,500 ha Wet season	12,500 ha Dry season	13,000 ha Wet season	13,000 ha Dry season	13,500 ha Wet season	13,500 ha Dry season	14,000 ha Wet season	14,000 ha Dry season	15,000 ha Wet season	15,000 ha Dry season	16,000 ha Wet season	16,000 ha Dry season	17,000 ha Wet season	17,000 ha Dry season	18,000 ha Wet season	18,000 ha Dry season
1959/60	-		-		-		-		-		-		-		-		-		-	7.14 (Mar. Apr.)
1960/61	-		-		-		-		-			1.13 (Apr.)		16.96 (Mar. Apr.)		32.93 (Mar. Apr.)	11.19 (May Jul.)	49.24 (Feb. Mar. Apr.)	19.83 (May Jul.)	65.55 (Feb. Mar. Apr.)
1961/62	-			2.67 (Apr.)		10.95 (Mar. Apr.)		19.34 (Mar. Apr.)		27.72 (Mar. Apr.)	3.30 (May)	36.80 (Mar. Apr.)	3.54 (May)	52.92 (Feb. Mar. Apr.)	3.77 (May)	69.97 (Feb. Mar. Apr.)	4.00 (May)	87.02 (Feb. Mar. Apr.)	4.23 (May)	104.08 (Feb. Mar. Apr.)
1962/63	21.30 (May Jun.)		27.64 (May Jun.)		28.59 (May Jun.)		29.54 (May Jun.)	4.56 (Apr.)	30.49 (May Apr.)	12.72 (Mar. Apr.)	31.44 (May Jun.)	20.89 (Mar. Apr.)	33.34 (May Jun.)	37.23 (Mar. Apr.)	35.24 (May Jun.)	53.64 (Feb. Mar. Apr.)	37.14 (May Jun.)	70.26 (Feb. Mar. Apr.)	39.03 (May Jun.)	86.88 (Feb. Mar. Apr.)
1963/64	-		3.69 (May)		11.94 (May)		15.73 (May)		16.20 (May)		16.67 (May)		17.62 (May)	11.24 (Mar. Apr.)	18.56 (May)	26.81 (Mar. Apr.)	19.50 (May)	42.41 (Feb. Mar. Apr.)	20.44 (May)	58.26 (Feb. Mar. Apr.)
1964/65	-		-		-		-		-		-		-		-			7.93 (Mar. Apr.)	1.81 (Jul.)	22.94 (Mar. Apr.)
1965/66	-		-		-			2.27 (Apr.)		10.23 (Mar. Apr.)		18.33 (Mar. Apr.)		34.53 (Mar. Apr.)		50.70 (Mar. Apr.)		67.02 (Feb. Mar. Apr.)		83.5 (Feb. Mar. Apr.)
1966/67	-		-		-		-		-		-			0.23 (Apr.)		14.58 (Mar. Apr.)		29.07 (Mar. Apr.)		43.55 (Mar. Apr.)
1967/68	-		-		-		-			1.42 (Apr.)	1.39 (May)	9.34 (Mar. Apr.)	15.75 (May)	25.43 (Mar. Apr.)	16.57 (May)	41.52 (Mar. Apr.)	17.39 (May)	57.67 (Feb. Mar. Apr.)	18.21 (May)	74.05 (Feb. Mar. Apr.)
1968/69	-		7.70 (Jun. Jul.)		16.73 (Jun. Jul.)		25.79 (May Jun. Jul.)	7.36 (Mar. Apr.)	33.54 (May Jun. Jul.)	15.58 (Mar. Apr.)	35.04 (May Jun. Jul.)	23.79 (Mar. Apr.)	38.04 (May Jun. Jul.)	40.20 (Mar. Apr.)	41.04 (May Jun. Jul.)	56.77 (Feb. Mar. Apr.)	44.03 (May Jun. Jul.)	73.49 (Feb. Mar. Apr.)	47.02 (May Jun. Jul.)	90.20 (Feb. Mar. Apr.)
1969/70	-		-		5.09 (May)		6.19 (May)		6.59 (May)		7.00 (May)		7.82 (May)		8.63 (May)	12.55 (Mar. Apr.)	9.45 (May)	28.13 (Mar. Apr.)	10.26 (May)	43.67 (Mar. Apr.)
	9/10	11/11	7/10	10/11	6/10	10/11	6/11	7/11	6/10	6/11	4/10	5/11	4/10	3/11	4/10	2/11	3/10	1/11	2/10	0/11

Table 9.4.2 List of Shortage Amount

Unit ; Million Cubic Meters(%)

Year	Season	Irrigable Area									
		(1.000)	(1.043)	(1.087)	(1.130)	(1.174)	(1.217)	(1.304)	(1.391)	(1.478)	(1.565)
		11,500ha	12,000ha	12,500ha	13,000ha	13,500ha	14,000ha	15,000ha	16,000ha	17,000ha	18,000ha
1959/60	Wet										
	Dry										7.14(2.0)
1960/61	Wet									11.19(3.3)	19.83(5.5)
	Dry						1.13(0.4)	16.96(5.7)	32.93(10.3)	49.24(14.5)	65.55(18.2)
1961/62	Wet						3.30(1.2)	3.54(1.2)	3.77(1.2)	4.00 (1.2)	4.23 (1.2)
	Dry		2.67 (1.1)	10.95 (4.4)	19.34 (7.4)	27.72(10.3)	36.80(13.1)	52.92(17.6)	69.97(21.9)	87.02(25.6)	104.08(28.9)
1962/63	Wet	21.3 (9.3)	27.64(11.5)	28.59(11.4)	29.54(11.4)	30.49(11.3)	31.44(11.2)	33.34(11.1)	35.24(11.0)	37.14(10.9)	39.03(10.8)
	Dry				4.56(1.8)	12.72(4.7)	20.89(7.5)	37.23(12.4)	53.64(16.8)	70.26(20.7)	86.88(24.1)
1963/64	Wet		3.69(1.5)	11.94(4.8)	15.73(6.1)	16.20(6.0)	16.67(6.0)	17.62(5.9)	18.56(5.8)	19.50(5.7)	20.44(5.7)
	Dry							11.24(3.7)	26.81(8.4)	42.41(12.5)	58.26(16.2)
1964/65	Wet										1.81(0.5)
	Dry									7.93(2.3)	22.94(6.4)
1965/66	Wet										
	Dry				2.27(0.9)	10.23(3.8)	18.33(6.5)	34.53(11.5)	50.70(15.8)	67.02(19.7)	83.50(23.2)
1966/67	Wet										
	Dry							0.23(0.1)	14.58(4.6)	29.07(8.6)	43.55(12.1)
1967/68	Wet						1.39(0.5)	15.75(5.3)	16.57(5.2)	17.39(5.1)	18.21(5.1)
	Dry					1.42(0.5)	9.34(3.3)	25.43(8.5)	41.52(13.0)	57.67(17.0)	74.05(20.6)
1968/69	Wet		7.70(3.2)	16.73(6.7)	25.79(9.9)	33.54(12.4)	35.04(12.5)	38.04(12.7)	41.04(12.8)	44.03(13.0)	47.02(13.1)
	Dry				7.36(2.8)	15.58(5.8)	23.79(8.5)	40.20(13.4)	56.77(17.7)	73.49(21.6)	90.20(25.1)
1969/70	Wet			5.09(2.0)	6.19(2.4)	6.59(2.4)	7.00(2.5)	7.82(2.6)	8.63(2.7)	9.45(2.8)	10.26(2.9)
	Dry								12.55(3.9)	28.13(8.3)	43.67(12.1)
Total of Shortage %		(9.3)	(17.3)	(29.3)	(42.7)	(57.2)	(73.2)	(111.7)	(151.1)	(192.8)	(233.7)
Total Water Requirement(Ax2000) mm		230 (100%)	240 (100%)	250 (100%)	260 (100%)	270 (100%)	280 (100%)	300 (100%)	320 (100%)	340 (100%)	360 (100%)

Table 9.4.3 Runoff and Rainfall

Water year	Wet season runoff (MCM)	Wet season Rainfall (mm)	Dry season Runoff (MCM)	Dry season Rainfall (mm)	Water year Runoff (MCM)	Water year Rainfall (mm)
1959/60	703.2	1,782.9	192.0	225.9	895.2	2,008.8
1960/61	536.9	3,124.3	60.7	142.4	597.6	3,266.7
1961/62	-	3,229.4	29.4	53.2	-	3,282.6
1962/63	457.9	2,618.6	40.5	96.4	498.4	2,715.0
1963/64	508.9	3,609.6	57.6	278.4	566.5	3,888.0
1964/65	395.1	2,900.4	96.1	-	491.2	-
1965/66	363.8	-	41.9	238.7	405.7	-
1966/67	490.9	3,018.5	74.4	388.1	565.3	3,406.6
1967/68	422.9	3,531.4	79.1	210.2	502.0	3,741.6
1968/69	303.8	2,595.9	-	127.9	-	2,723.8
1969/70	-	3,508.4	69.5	207.2	-	3,715.6
1970/71	338.1	3,245.8	120.2	269.0	458.3	3,514.8
1971/72	395.0	2,243.0	-	169.2	-	2,412.2
1972/73	-	4,493.2	-	128.9	-	4,622.1
1973/74	-	2,298.3	70.9	144.0	-	2,442.3
1974/75	809.2	3,621.4	-	-	-	-
Mean	487.4	2,935.6	76.0	204.9	583.3	3,220.2

CHAPTER 10

SERVICE AREA

10. Service Area

10.1. General

The service area is a flat and low paddy cultivation area extending over the basins of the Alaminos River and the Balincaguin River. From the administrative point of view, it extends over four municipalities (refer to "town", hereafter), namely, Alaminos, Bani, Mabini and Sual. The total arable area of the 4 towns is reported to be of the order of 19,000ha, most of which is located in existing paddy area around the town of Alaminos.

According to a land classification, a survey has already been carried out by the NIA in the area of 17,250ha which is composed of 12,146ha of arable land and of the remaining non-arable land, i.e., the dwelling land, rivers, etc. In addition to the above mentioned area, study of the project area is carried out by including the low and flat paddy area located in left bank of the Bani River and the mountanious areas Study of additional project area resulted in 19,200ha totally and the 13,820ha of arable land.

10.2. Topography of the Project Area

Generally speaking, the topography of the project area presents a gentle slope (of the order of 1/300 through 1/400) from the south to the north, and therefore the topographical conditions prevailing therein are favourable for formulating an irrigation project.

The southern part of the project area is a diluvium plateau, most of which is paddy field cultivated up to the altitudes of approximately EL.40m. The northern part of the project area, particularly the vicinities of the estuary of the Inerangan River, consists of lowland area. Paddy fields extends throughout the whole area.

There are no significant rivers in the project area except the Alaminos River and the Inerangan River. Particularly the Alaminos River which joins many tributaries should be taken into consideration at the occasion of drawing out layout of the irrigation canal systems.

10.3. Intake Water Level and Service Area

The relation between the area and altitude of the arable land within the project area was studied from the topographical map of 1:4,000 in scale which is still incomplete. Therefore, the topographical map of 1:50,000 in scale was used while the 1:4,000 topographical map is used complementarily. The relation between the area and altitude of arable land is presented in Figure 10.1.1 of the supporting report.

The intake water level can be determined from the altitude of the main irrigation canal, by taking into account the slope of the irrigation canal and the head losses of the intake facility to enable to supply required irrigation water to the planned irrigable area. The relation between the irrigable area and the corresponding intake water level is shown in Figure 10.2.1 of the supporting report.

The planned irrigable area of the Project was decided to be 11,500ha after water balance study utilizing effective storage capacity of the reservoir between EL.63.00m of normal water surface and EL.38.00m of low water surface (lowest intake water level). The above mentioned irrigable area can be identified within the arable land of 13,820ha.

CHAPTER 11

DAM

11. Dam

11.1. Determination of Dam Height

11.1.1. Crest Width

The crest width of the Mabini Dam is designed as 10m, taking into account the safety against waves, percolation and earthquakes as well as the consideration for the transportation connecting both sides of the river in the future and for the construction methods.

11.1.2. Freeboard

The free-board of the Mabini Dam is determined by formula prescribed in the dam design criteria (Ministry of Agriculture, Forestry and Fisheries of Japan). There are 2 formulae as presented below, and the height of free-board should be adopted whichever is higher

FORMULA (1) = Hf + hw + he + 1.5 (if hw + he < 1.5, Hf + 3)

FORMULA (2) = Hh + hw + 1.5 (if hw < 0.5, Hh + 2)

where	Hf:	Normal water surface	63.00m
	Hh:	High water surface	65.00m
	hw:	Height of wave due to wind	1.0 m
	he:	Height of wave due to earthquake	0.75m

FORMULA (1) = 66.35m

FORMULA (2) = 67.60m

In view of the results of the above calculations, the elevation of core crest is determined as 68.00m, by adding 0.40m for allowance. The elevation of dam crest is determined as 68.50m, by adding 0.5m of core protection material. (Refer to 11.3 of Supporting Report)

11.2. Foundation Treatment

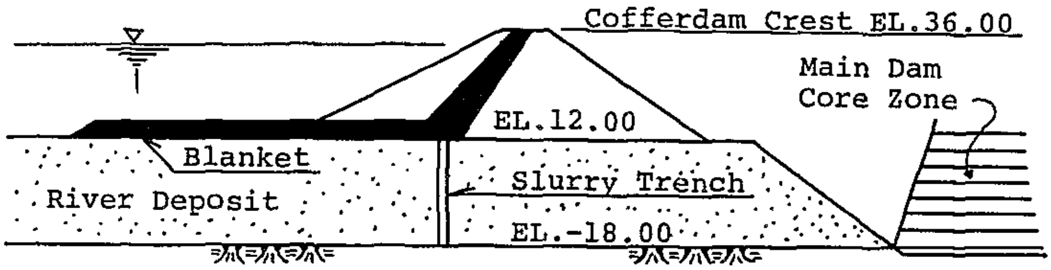
(1) Main Dam

The river deposit of an approximately 30m thick has been sedimented at the proposed dam-site. The results of geological survey show that the layer is composed of sand and gravel with seepage coefficient of the order of 1×10^{-1} through 10^{-2} cm/sec. The foundation rock consists of the basalt, has a weathered layer of 1 through 2m in thickness. Accordingly, the foundation treatment should be studied regarding two kinds of materials, the river deposit and the foundation rock. Particular importance is in the measures to deal with the sand and grand layer of the riverbed. The cut-off trench method, the continuous cut-off wall method and the blanket method can be considered. As a result of a comprehensive study, it was decided that the cut-off trench method is the most advantageous one in view of the safety and the economy. (Refer to 11.5 Dam Type.)

(2) Cofferdam

The amount of seepage water comes out from the river-bed sand and gravel layer under the cofferdam is estimated as the following table.

Case	Countermeasures	Seepage Coefficient of Riverbed Material	
		1×10^{-1} cm/sec	Remarks
		m^3/min	
1	No Countermeasures	323.438	
2	Extension of Blanket Upstream of Cofferdam	22.990	
3	Slurry Trench	0.847	



The amount of the above mentioned seepage through foundation of the cofferdam was calculated for the case of the probable flood of 1/20. As of now, the geological study shows that the seepage coefficient of the existing riverbed materials is the order of 10^{-1} to 10^{-2} cm/sec. In the rainy season, the blanket method is not sufficient to secure the safety during the construction of cut-off trench. Therefore, it is recommended that the slurry trench methods should be applied to implement safer excavation of cut-off trench for the Mabini Dam. (Refer to 11.1 of Supporting Report)

(3) Grouting for Main Dam

The curtain grout and the blanket grout should be carried out for the foundation treatment. The curtain grout will be arranged in 3 rows, with space of 2m. The depth of the grouting is calculated by means of the Simonds's formula.

$$d = H/3 + C$$

where

d:	Grouting depth	
H:	Water storage depth	63 - (-20) = 83m
C:	Constant	5 through 85m

The depth of the curtain grout is decided to be 15m through 35m, as a result of the calculation described above.

Four rows of blanket grout of 5m through 10m are planned at the upstream side of the curtain grout, throughout the section of 450m ranging from No. 3 through No. 12 in cross section of river at dam-site, to improve the cut-off effect and reinforce the bearing power of bedrock.

11.3. Seismic Coefficient

The seismic observation stations near the dam-site are located in Iba and in Dagupan. The record of previous earthquakes near the dam-site is shown in Table 11.3.1. The seismic coefficient for the stability analysis of a fill-type dam will be determined, taking into account the relation between seismic acceleration and the foundation conditions. The relationship of these are shown in Table 11.3.2. Since the proposed dam will be constructed on the foundation of the sand and gravel layer, the seismic coefficient should be 0.20 based on the recorded maximum seismic acceleration.

Table 11.3.1 Earthquake Recorders

DATE	RANK	EPICENTER LATITUDE	EPICENTER LONGITUDE	MAGNITUDE (M)	DISTANCE TO DAMSITE (D) (km)	ACCELERATION (gal)
1927.4.13	2	16.0	120.5	6.75	57.40	137.94
1927.4.13	8	16.0	120.5	6.75	57.40	90.34
1927.4.19	1	16.0	120.0	6.75	5.60	312.05
1928.8. 5	4	16.0	119.5	6.25	48.56	107.82
1932.8.24	9	16.5	120.5	6.25	77.54	60.20
1934.2.14	10	17.5	119.0	7.60	192.50	48.35
1963.3.17	11	15.6	120.2	5.50	54.35	42.01
1963.3.17	7	16.25	120.0	5.50	24.84	98.48
1963.3.17	5	16.25	120.0	5.60	24.84	107.30
1974.2. 9	6	16.2	120.1	5.50	24.09	100.62
1974.3.15	3	16.05	119.92	5.00	4.59	127.43

Data from PAGASA "UNDP Seismological Programe for Southeast ASIA" 1927 - 1979

$$* \quad \log_{10} \frac{galmax}{640} = \frac{D + 40}{100} (-7.604 + 1.7244M - 0.1036M^2)$$

Table 11.3.2 Seismic Coefficient

Acceleration (gal)	FOUNDATION ROCK	FOUNDATION SOIL
400	0.20	0.25
400 - 200	0.15	0.20
200 - 100	0.12	0.15
100	0.10	0.12

11.4. Embankment Materials

(1) Outline of the embankment materials

The embankment materials will be obtained in accordance with the plan described below, taking the topographical and geological conditions of the damsite into account.

Core material - The reddish brown soil, which originated from the weathered basalt, and the dark brown soil, which consists of the alluvium, can be used as the core materials. The borrow area is shown in Figure 11.3.1. The quantity of the core material existing therein is more than 720,000 m^3, which is sufficient in volume.

Transition material - The weathered basalt and the limestone can be utilized as transition materials. It is expected that the transition material will be composed mostly of the excavated material from the spillway, which is estimated at approximately 3,000,000 m^3 of material.

Filter material - Large amount of river deposit found at the damsite can be utilized as filter material, after screening and washing of river deposit.

Rockfill material - The fresh basalt will be used as the rockfill material. The quarry site for the rockfill is shown in Figure 11.3.1. It is estimated at about 5,000,000 m^3 of the rock. However, further survey is required prior to the implementation of the project because drilling survey and other detailed studies have not been carried out.

It is recommended that the borrow area should be limited within reservoir area from emvironmental stand of view.

(2) Results of the Survey of Embankment Materials

Five test pits were digged, to make sure the available materials for embankment. A total of 11 samples were collected from the test pits and a soil test was carried out. Mechanical tests of rock materials were carried out by the sample taken from the boring cores of dam-site. Test results of embankment material are shown in Figure 11.3.2..

a) Core material

The reddish brown soil, the dark brown soil and the light brown soil are examined as core materials.

Reddish brown soil - This material consists of the residual soil and the weathered basalt. The samples are obtained from ATP.1. This soil is also found very often at the right bank of the downstream. This material is classified as MH in the unified soil classification.

Dark brown soil - This is the alluvium composing the paddy field located at the upstream side of the dam axis. Samples of this material are obtained from ATP.2. This layer partially contains cobbles, but this material is classified as CL and CH.

Light brown soil - This is the residual soil found in the limestone zones. Samples of this materials are collected from ATP.5 of the left bank of the downstream.

It is classified as SM.

All materials are of less seepage coefficients, of the order of $\alpha \times 10^{-6}$ through 10^{-8} cm/sec, presenting, therefore, sufficient impermeability. The natural moisture content of almost all materials exceeds by 10% over the optimum moisture content. It is considered that this is caused by the sampling in the rainy season, but the extent of the influence is unknown. The core materials were examined by triaxial compression test of specimens with a density equivalent to 95% of the maximum dry density and having wet side moisture content. The results of the soil tests are presented in Table 11.3.1.

b) Transition material

Samples of the weathered basalt was obtained from the test pit APT.4. This material is classified as SM and SW-SM according to the laboratory tests. The seepage coefficient is of the order of 10^{-5} cm/sec, because its particles are fragile. The specimen with a density equivalent to 95% of the maximum dry density were examined by triaxial compression test. The results of soil tests are presented in Table 11.3.1.

c) Rockfill materials

Rockfill materials were examined by compressive strength test, the specific gravity test and the water absorption test, by using samples taken from boring cores. The test results are presented in Table 11.3.2. The results of additional tests carried out in Japan are presented in Table 11.3.3. The results of such tests show low strength of both the limestone and the basalt. The rockfill material for the riprap should be selected from better quality rock material excavated from the spillway and quarry site.

d) River deposit

The river deposit can be used as the filter. The test samples are obtained from the test pit ATP.3. It was not possible to estimate the density of the samples in the field due to the insufficient test equipments. In the laboratory, a triaxial compression tests is performed on the material passing No. 4 sieve. The density of the specimens is made as low as possible. (Dry density = 1,497 g/cm^3), being obtained a cohesion $C = 0.65\ kg/cm^2$ and an angle of internal friction $\phi = 26°20'$.

Fig.11.4.1 Location Map of Test Pits

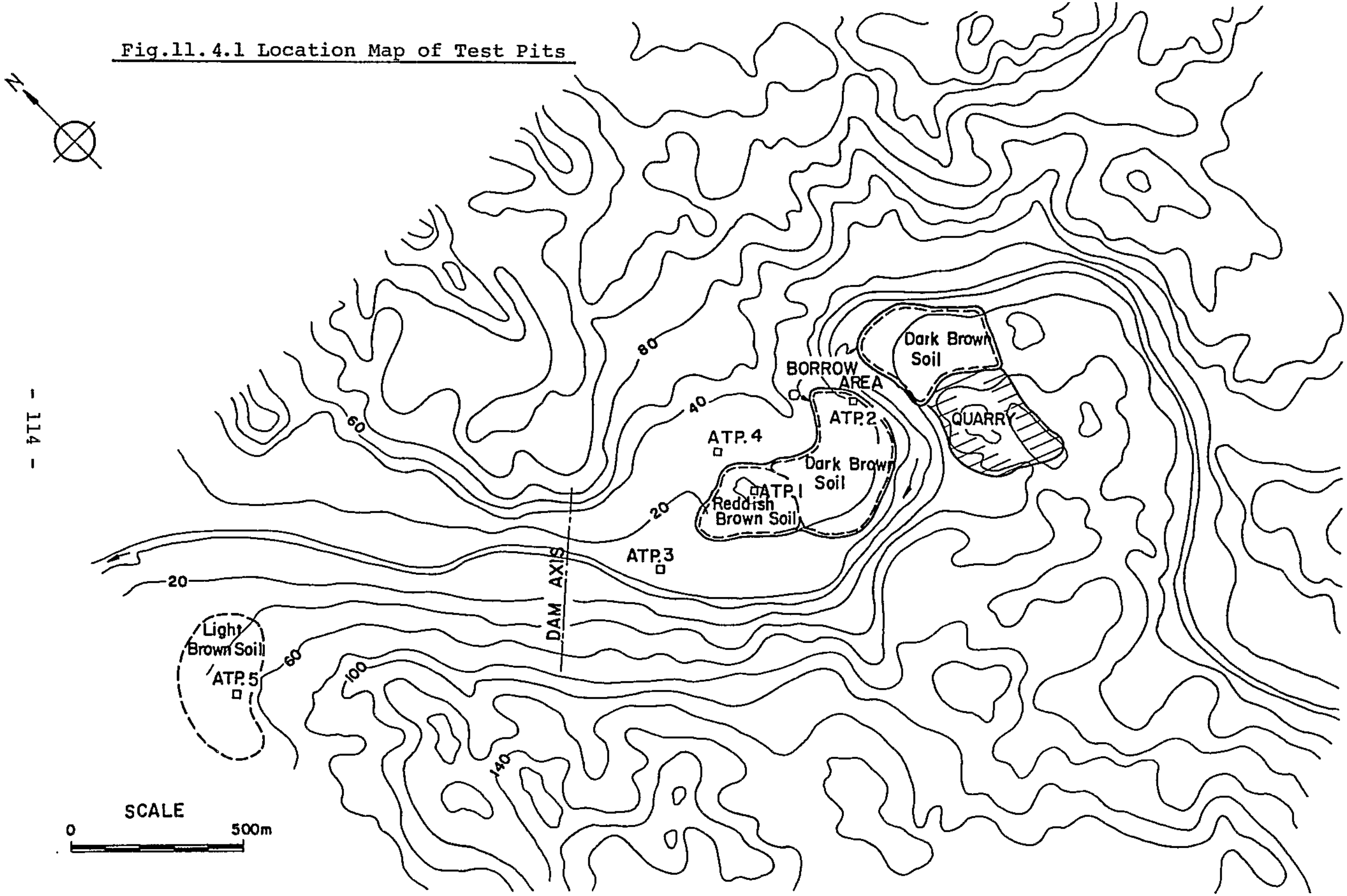


Table 11. 4. 1 Results of Soil Test

ITEM		Test Pit No.	Soil								Weathered Basalt		River Deposit
		Test Pit No.	ATP. 1			ATP. 2			ATP.5		ATP.4		ATP. 3
		Sample No.	1	2	3	1	2	3	1	2	1	2	1
Natural Moisture Content		(%)	50.96	64.05	56.93	24.53	44.00	51.35	40.63	44.62	44.63	25.38	
Specific Gravity		Gs	2.258	2.610	2.624	2.845	2.878	2.663	2.607	2.417	2.515	2.518	2.833
Gradation	Gravel	(%)	4.10	2.00	0.40	0.40	24.50	1.90	1.50	8.10	1.50	42.50	74.60
	Sand	(%)	9.65	20.45	31.01	45.21	19.30	22.68	48.58	44.40	55.00	67.27	24.36
	Silt	(%)	75.45	69.25	61.39	49.09	48.80	66.62	44.12	45.50	36.50	6.93	1.04
	Clay	(%)	10.80	8.30	7.20	5.30	7.40	8.80	5.80	2.00	7.00	3.30	0.00
	D_{60}	(mm)	0.036	0.0405	0.058	0.087	0.094	0.0463	0.11	0.11	0.123	2.65	21.0
	D_{10}	(mm)	0.034	0.0118	0.0143	0.020	0.0099	0.0057	0.0275	0.042	0.012	0.070	0.4
	Cu		10.59	3.43	4.06	4.35	9.49	8.12	4.00	2.62	10.25	37.86	52.50
Atterberg Limit	Liquid Limit	(%)	94.66	74.70	65.00	33.90	60.75	68.35	74.55	71.85	51.97	43.80	-
	Plastic Limit	(%)	45.69	44.49	43.85	20.06	26.60	29.43	40.49	37.48	35.99	30.21	-
	Plasticity Index		48.97	30.21	21.15	13.84	34.15	39.92	34.06	34.37	15.98	13.59	-
Compaction	Max. Dry Density	(g/cm^3)	1.186	1.099	1.217	1.758	1.428	1.241	1.188	1.251	1.281	1.406	1.966
	Optimum M. C.	(%)	42.5	49.90	41.20	18.50	27.25	36.30	43.30	32.60	33.30	29.70	13.90
Permeability		k (cm/sec)	OPT. 1.2×10^{-7}	OPT. 1.8×10^{-6}			OPT. 4.0×10^{-8}			OPT. 1.4×10^{-7}		OPT. 2.3×10^{-5}	
				WET 2.4×10^{-7}			WET 1.3×10^{-7}			WET 9.3×10^{-8}		DRY 4.6×10^{-6}	
Consolidation Coefficient of Consolidation		Cv											
Triaxial Compression UU Test	Cohesion	C (kg/cm^2)	0.84	0.04	0.89	0.43	0.63	0.97	0.56	0.20	0.86	0.00	0.65
	Angle of Internal Friction		10°20'	5°40'	18°40'	9°30'	1°45'	11°15'	5°50'	3°30'	17°50'	34°15'	26°30'
Triaxial Compression CU Test	Cohesion	C (kg/cm^2)	1.25	0.70	0.79	0.63	0.58	0.80	0.78	0.82	0.57	0.42	
	Angle of Internal Friction		7°00'	10°00'	17°20'	13°05'	12°20'	13°30'	1°25'	11°30'	22°40'	31°00'	

Table 11. 4. 2 TEST RESULTS OF SAMPLES TAKEN FROM BORING CORE

Description of Rock	Hole NO.	Depth in m.	Compressive Strength in kg/cm^2	Specific Gravity	Absorption in Percent
Limestone	DH.10	6.30-6.45	81.04	2.248	1.05 %
		11.10-11.30	39.2		
Basalt	DH.2	7.90-8.10	181.7	2.25	9.30 %
		8.32-8.52	100.6	2.218	11.64 %
		19.17-19.45	172.5		
			202.6		
		28.15-28.45	85.0		
			54.9		
	DH.8	62.20-61.30	108.5	2.25	9.30 %
	DH.9	41.05-41.20	52.3		
	DH.10	44.60-44.75	99.3		

Table 11.4.3 ADDITIONAL TEST RESULTS OF SAMPLES TAKEN FROM BORING CORE

Hole No.		DH.1	DH.8	DH.9
Depth in m		20.3	63.0	25.9
Description of Rock		Volcanic Breccia	Basalt	Basalt
Visual Inspection		Grayly green in color Gravel rich no crack	Grayly green rock with feldspar crystal Rich of calcite vein	Grayly green rock with feldspar crystal Rich of calcite vein
Moisture Content in percent		8.8	13.8	12.7
Wet Density in 8/cu.cm		1.941	1.994	2.191
Compressive Strength	Sample Diameter in cm	4.718	4.695	4.722
	Sample Height in cm	8.442	9.637	9.527
	Area in sq.cm	17.48	17.31	17.51
	Max. Load in kgf	1,380	2,200	1,010
	Compressive Strength in kg/sq.cm	78.9	127.1	57.7

(3) Design parameters

The design data are selected from the results of the tests of the embankment materials. The stability analysis is carried out by the effective stress analysis, but it is not possible to obtain the effective stress, because the triaxial compression test equipment furnished by the NIA is not equipped with pore pressure measurement device. Accordingly, the design strength is determined based upon the total-stress parameter. The selected design parameters are presented in Table 11.4.4.

Table 11.4.4 DESIGN PARAMETER FOR EMBANKMENT

Item		Density		Strength	
		Wet Density in ton/cu.m	Saturated Density in ton/cu.m	Cohesion in ton/sq.m	Angle of Internal Friction
Core	Immediate after completion	1,797	1,822	7.1	10°37'
	Without pore pressure	1.797	1,822	7.9	12°12'
Filter		2,000	2,150	0	38°00'
Transition		1,824	1,848	0	38°00'
Rockfill		1,643	1,902	0	42°00'
Random Zone		1,850	2,000	0	36°00'
River Deposit			2,100	0	38°00'

11.5 Dam Type

The concrete type is not suitable for the Mabini Dam, in view of the topographical and geological conditions. The earth-fill dam is also excluded because of the availability of the earth material for embankment. There are two types of rock-fill dam, namely, the central core type and the inclined core type. In the case of the Mabini Dam, results of the stability calculations suggest that the inclined core type is not appropriate, due to the small shearing strength of the core material. (Refer to Supporting Report 11.3.1.)

As a result of the above considerations, the central core type rock-fill dam is selected for the Mabini Dam. Another problem of the Mabini dam is the foundation treatment of the riverbed which affects the cross section of the dam. Therefore, a comprehensive study including the foundation treatment of the riverbed is carried out herein. The combinations of the various treatment and dam type for alternatives are as follows.

	Treatment of River Deposit	Dam Type
Type-A	Cut-off Trench	Central Core, Rockfill
Type-B	Cut-off Wall	Central Core, Rockfill
Type-C	Soletanche Grouting	Central Core, Rockfill

The standard cross sections of the 3 types of combination are presented in Figure 11.5.1. After a comparative study of the aforesaid 3 alternatives, it is decided to adopt the central core type, TYPE-A, which carries out the excavation down to the foundation bedrock, in view of the following reasons. (Refer to Figure 11.5.3.)

1) <u>The Construction Cost is Economical</u>

The alternative of TYPE-A requires a construction cost approximately 150 million Pesos (US$150 million) cheaper than the alternatives of TYPE-B and TYPE-C. (Refer to Supporting Report 11.3.2.)

2) <u>This Type is Safer in Terms of Permeability</u>

The continuous cut-off wall has possibility to present water leakage from concrete joints and water leakage from the boundary of the continuous cut-off wall and the core at the ocassion of earthquakes, etc.

3) <u>This is the Type Offering the Safety for Subsidence of Riverbed Material</u>

Subsidence of river deposit is anticipated after completion of dam embankment. In the case of the continuous cut-off wall, there is a risk of destruction of the continuous wall, because a negative friction force is caused by the differential subsidence between cut-off wall and the riverbed sand and gravel layer, which are loaded by the dam body weight (approximately 120 t/m^2). However, the differences of subsidence amounts are minor, because the dam of the TYPE-A alternative is made from the same material.

4) <u>This is Safer Type in Case of Earthquakes</u>

In the case of continuous wall, the characteristics frequency of the wall is defferent from the characteristic frequency of the foundation bedrock at the occasion of earthquakes. Therefore, cracks will be developed at the boundary between the continuous wall and the bedrock, being prone to cause water leakage and piping.

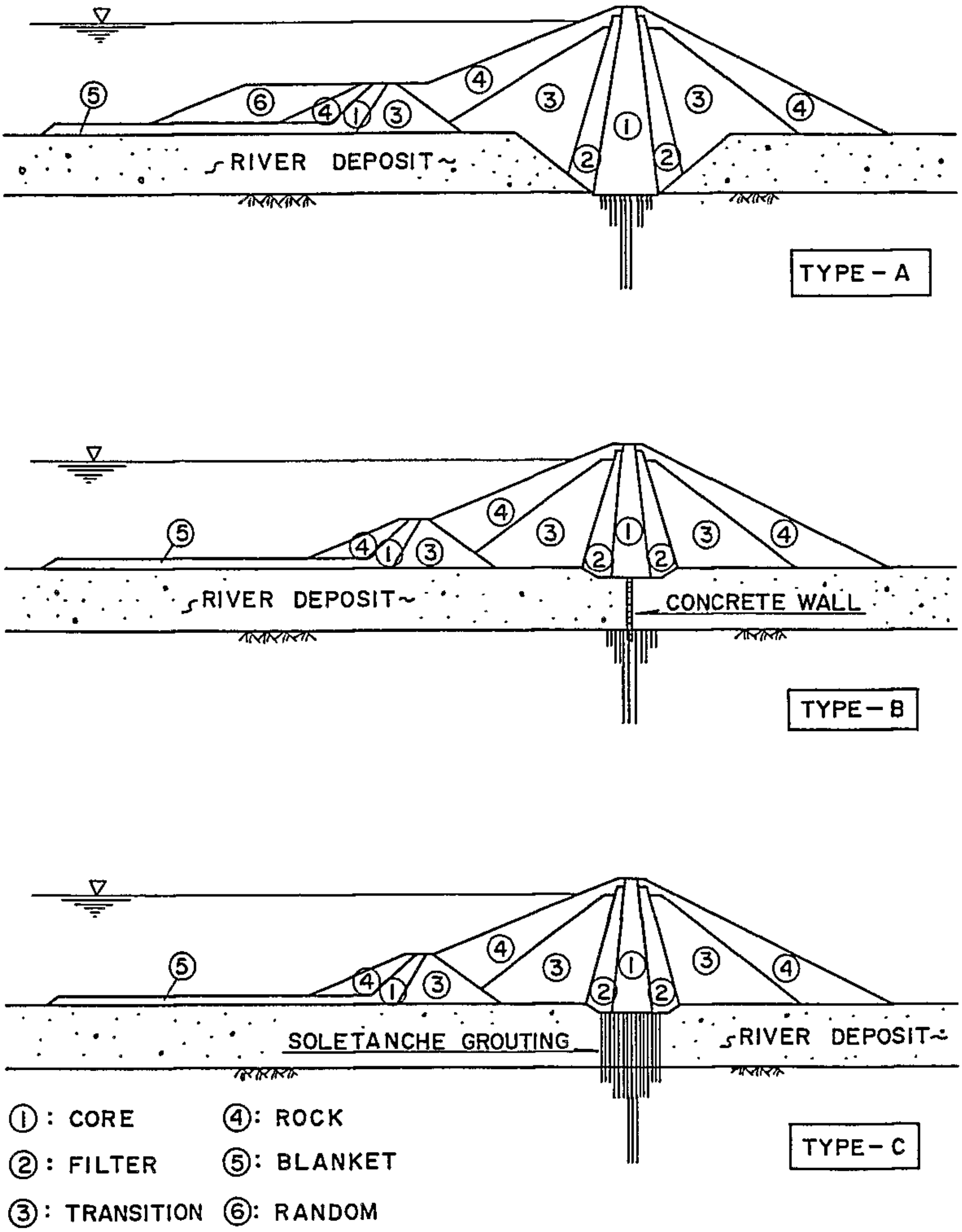
5) <u>There are many Cases of this type in Philippines</u>

The central core type rock-fill dams are prevailing in the Philippines. In other words, the Philippines Engineering Force has much experience in the construction of such type of dam.

6) <u>This Alternative Makes Possible to Check by Direct Observation of the Conditions of the Foundation Rock</u>

In this method, it is possible to check the conditions of the foundation rock by direct observation because the excavation of the cut-off trench reaches the foundation bedrock. It is also possible to undertake grouting activities with high reliability.

Fig.11.5.1 Cross Section of Dam Types



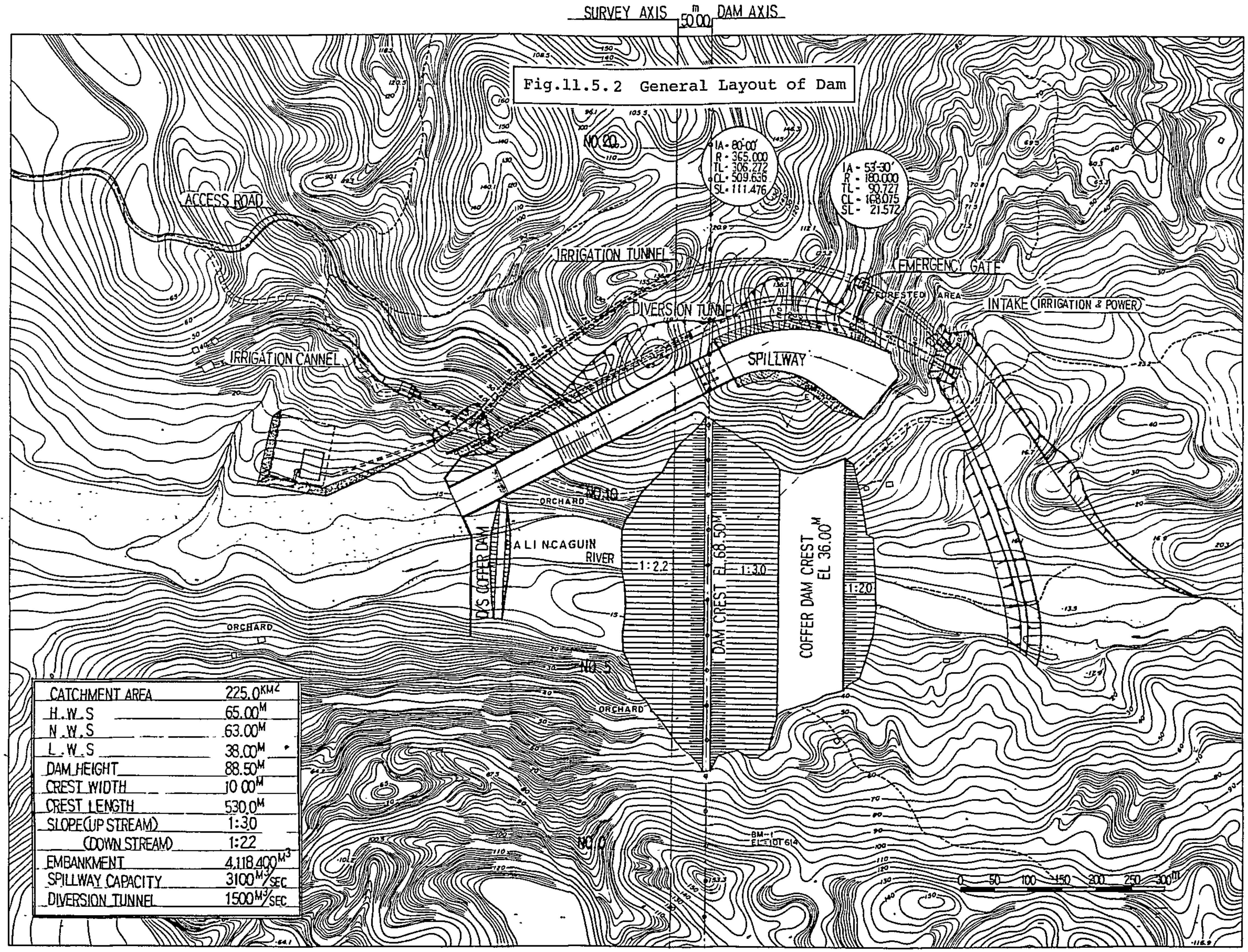
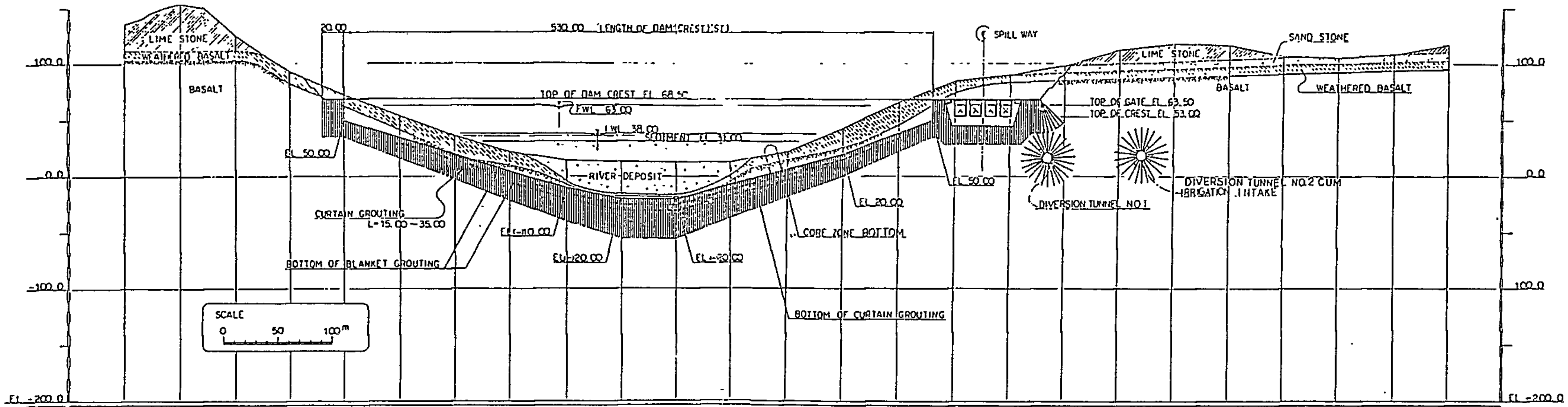


Fig.11.5.2 General Layout of Dam

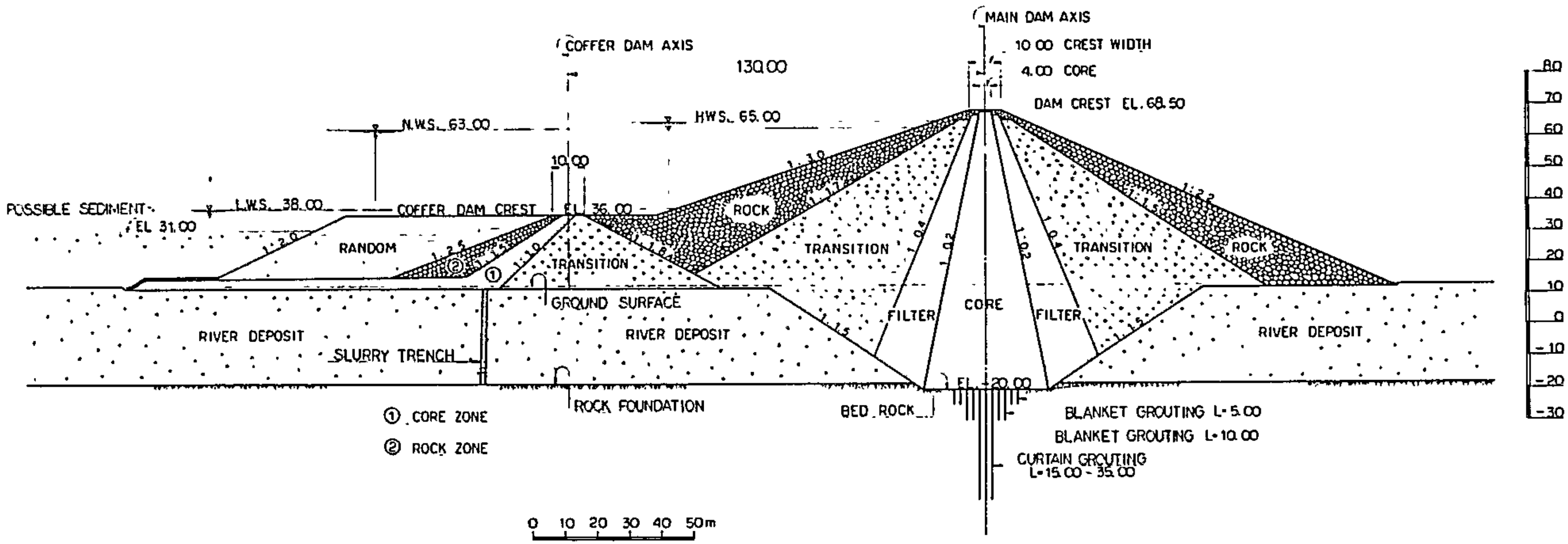


STA	DIST	GL	BL
NO.-2	-100.0	135.0	
NO.-1	-50.0	153.3	
NO.0	0.0	125.0	
NO.1	50.0	93.0	
NO.2	100.0	72.5	50.00
NO.3	150.0	54.0	35.00
NO.4	200.0	36.0	20.00
NO.5	250.0	22.0	5.00
NO.6	300.0	14.0	-10.00
NO.7	350.0	12.0	-20.00
NO.8	400.0	13.0	-20.00
NO.9	450.0	13.0	-6.67
NO.10	500.0	22.0	6.67
NO.11	550.0	43.0	20.00
NO.12	600.0	66.0	38.75
NO.13	650.0	85.0	
NO.14	700.0	90.0	
NO.15	750.0	98.0	
NO.16	800.0	112.0	
NO.17	850.0	118.0	
NO.18	900.0	117.0	
NO.19	950.0	107.0	
NO.20	1000.0	105.0	
NO.21	1050.0	108.0	
NO.22	1100.0	117.5	

NOTE. STA : STATION NUMBER; BL : CORE BOTTOM ELEVATION
DIST : CUMULATIVE DISTANCE; GL : GROUND ELEVATION

Fig.II.5.2
PROFILE ALONG CENTER LINE OF DAM CREST

FIG.11.5.4 TYPICAL CROSS SECTION OF DAM



11.6. Design of Dam Embankment

11.6.1. Stability Analysis of Dam Embankment

The stability analysis of the Mabini Dam is carried out by "The Slip Circle Method" in the 6 cases of which conditions are listed in the table below. (Refer to Fig. 11.6.1).

Condition	Water Level of the Reservoir		Horizontal Seismic Coefficient	Upstream	Downstream
After Completion	Normal Water Surface	63.00^{m}	0	Case 1	Case 2
After Completion	Normal Water Surface	63.00	0.2	Case 3	Case 4
After Completion	Intermedaite Water Level	38.00	0.2	Case 5	-
Immediately after Completion	Empty	-	0.1	Case 6	

The minimum safety factor is upstream SF = 1.23 and downstream SF = 1.21 at the occasion of normal water surface. The results of the safety analysis in the various cases are as follows.

	Water Level	Seismic Coefficient	Upstream or Downstream	Minimum Safty Factor
Case 1	N.W.S	0	Up	2.77
Case 2	N.W.S	0	Down	1.93
Case 3	N.W.S	0.2	Up	1.23
Case 4	N.W.S	0.2	Down	1.21
Case 5	L.W.S	0.2	Up	1.41
Case 6	Empty	0.1	Up	-

The upstream slope of the Mabini Dam is determined as 1:3.0 and the downstream slope is determined as 1:2.2, as a result of the stability analysis.

11.6.2. Study of the Seepage of Dam Embankment

The diagram of the seepage flow drawn out when the normal water surface (EL.63m) of the Mabini Dam is shown in Figure 11.6.2. The downstream water level is assumed to be EL12.00m. The water leakage from the dam body is calculated as follows.

Water leakage per day

$$Qday = 2.3664\ \ell/sec \times 86{,}400\ sec = 204.5\ m^3/day$$

Seepage loss from reservoir is estimated at 0.05% of total reservoir capacity. (Refer to 9.2.(4) of Supporting report)

$$303{,}000{,}000 \times 0.0005 = 151{,}500m^3 > 240.5m^3/day$$

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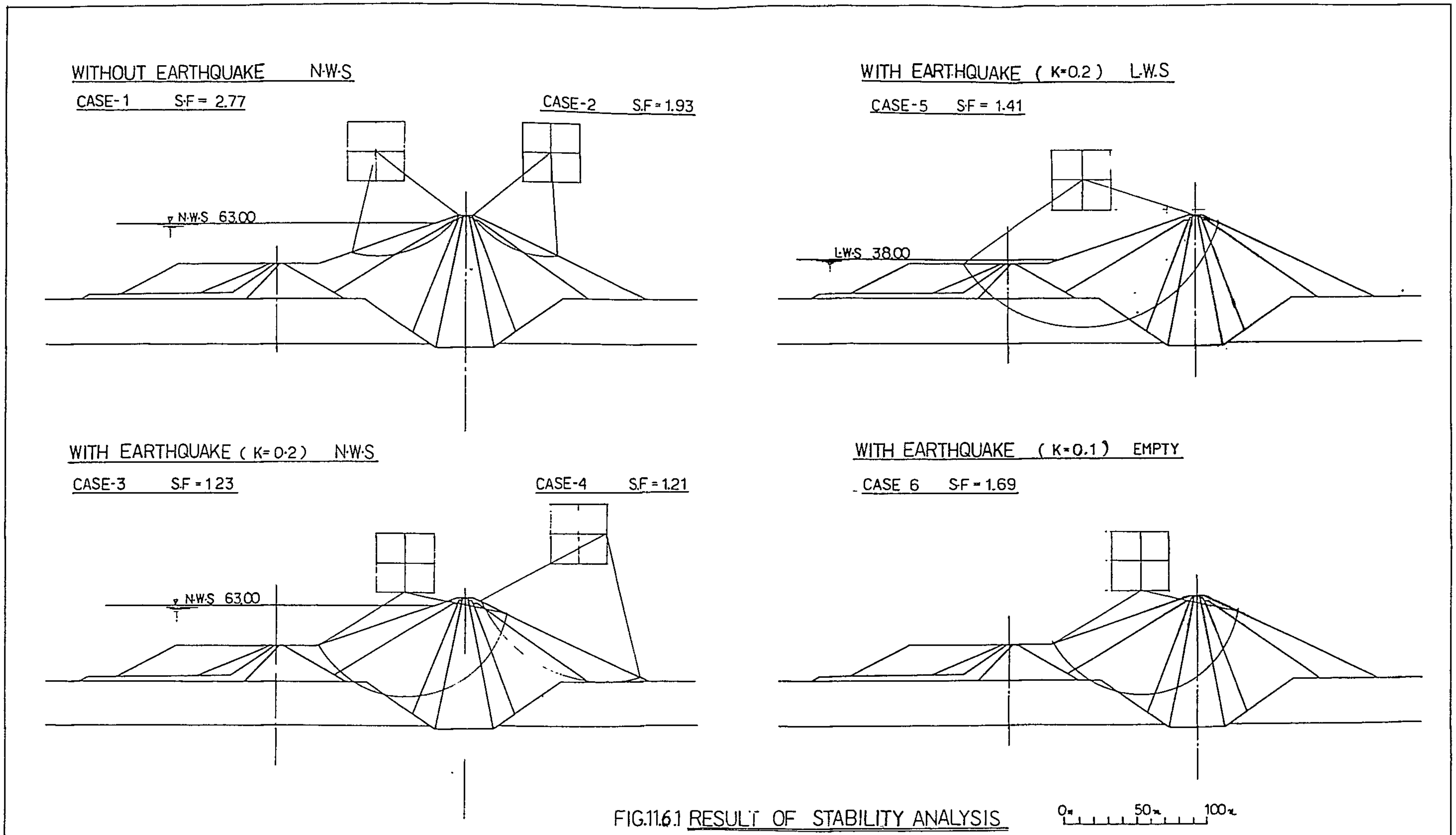


FIG.11.6.1 RESULT OF STABILITY ANALYSIS

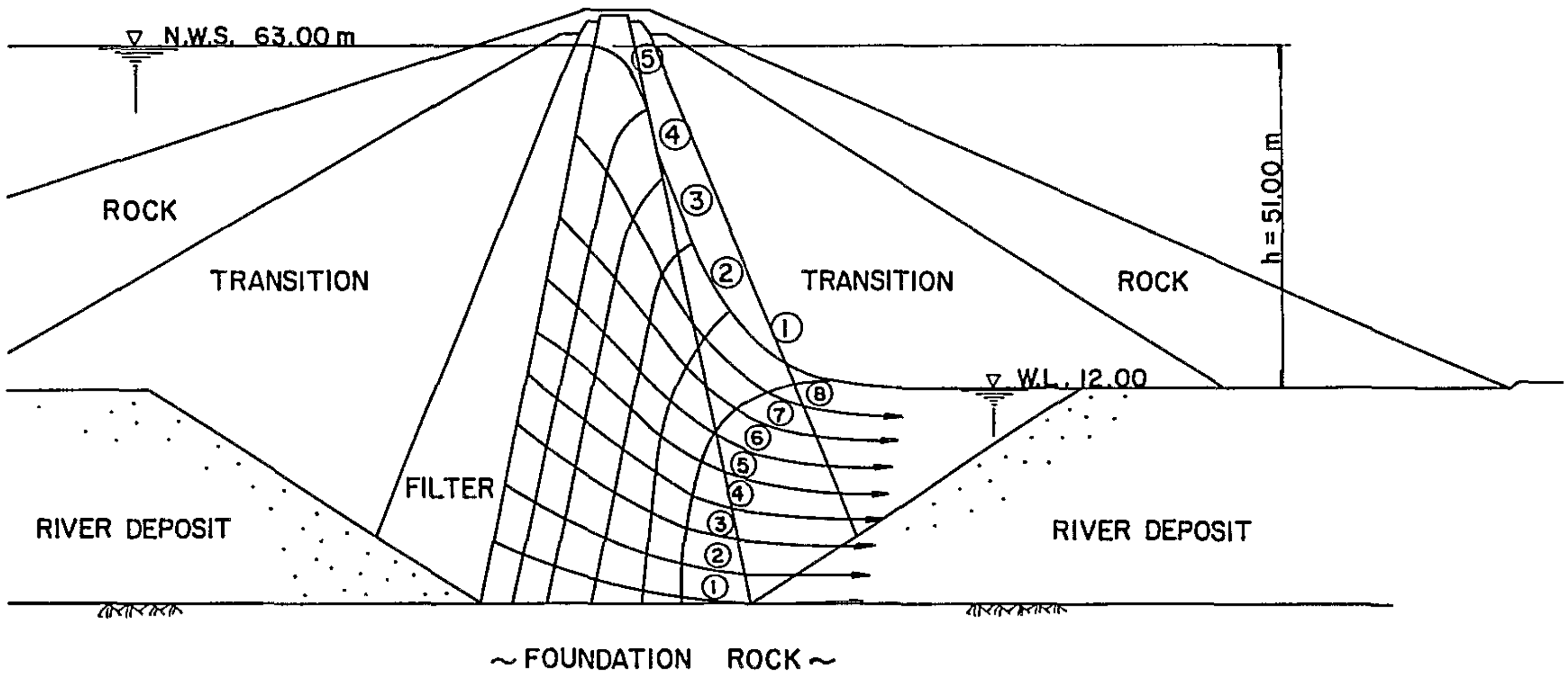


Fig.11.6.2 Flow Net of Seepage

11.7. Land Acquisition

It is expected that the land for the dam-site and the reservoir area should be acquired prior to the commencement of dam construction work and that the trees should be cut before storing water.

The number of families who would require resettlement is 85.

Currently, there is no road in the upstream of the proposed dam-site. However, it is necessary to construct a new road around the reservoir of 2.00m in width for pedestrians as a service for residents in the upstream area.

Land Compensations

Inundated Area	1,300ha
Families Requiring Relocation	85 families +1 Elementary School (3 Classrooms)
Total Length of Road Relocated	32.5Km

CHAPTER 12

SPILLWAY STRUCTURE

12. SPILLWAY STRUCTURE

12.1. Location and Route

It is decided to locate the spillway structure at the right bank side, after having taken consideration as listed below.

(1) The spillway with straight configuration, which is more advantageous from the hydraulic point of view, can be constructed at the right bank side.

(2) The confluence to the original river is more smooth at the right bank side.

(3) The geological survey disclosed a low velocity zone at the left bank side. The construction of any important structure like spillway at left bank is not recommendable.

12.2. Flood Tracing

The flood control effect of the reservoir results from the balance between the inflow and the outflow. The relation between the stored water, the inflow and the outflow is given by the following expression, by taking a time interval Δt.

$$S = Qi \cdot \Delta t - Qo \cdot \Delta t$$

where:

S : The quantity of water stored during the interval of time Δt.

Qi: The average inflow during the interval of time Δt.

Qo: The average outflow during the interval of time Δt.

The hydrograph of the flood obtained from the Probable Maximum Precipitation is used for the inflow (Refer to 4.4.) The variation in the discharge curve of the flood spillway are influenced not only by the size and configuration of the spillway, but also by the method of adjustment of the discharge. The Flood Tracing calculation is carried out for the gated spillway and for the side-channel spillway.

(1) In case of the gated spillway

a. The design flood is assumed to start when the reservoir water is at the normal water surface.

b. The gate is assumed to be operated in such a way to keep always the normal water surface.

(2) In case of side-channel spillway

a. The length of the crest of overflow is limited to be 180 m, due to the topographical conditions.

b. The side-channel spillway will be operated without man power.

The results of the flood discharge calculation regarding the gated spillway and the side-channel spillway, which was carried out by the above conditions are referred to Figure 12-1-1 and Figure 12-1-2 of Supporting Report.
The results of calculation are presented in the table below.

	Gated Spillway	Side-Channel Spillway
Maximum Rate of Outflow	3,100 m^3/sec	3,630 m^3/sec
H.W.S	EL. 65.00 m	EL. 65.00 m
N.W.S	EL. 63.00 m	EL. 60.30 m
Effective Storage Capacity	240,000,000 m^3	201,000,000 m^3
Capacity at N.W.S	303,000,000 m^3	264,000,000 m^3
Capacity at L.W.S	63,000,000 m^3	63,000,000 m^3

12.3. Selection of the Type of the Spillway

In the gated spillway, it is possible to use a design flood equivalent to 78% (3,100 m^3/s) of the inflow due to the effect of surcharge, but in the side-channel spillway (ungated type spillway), it is required to use 91% (3,630 m^3/s) of the inflow. In other words, the peak flood can be reduced at 3,100 m^3/s from 4,000 m^3/s of PMP because of the surcharge of 2 m by using gated spillway.

The highest possible water level of the proposed reservoir will be less than EL. 65.00 m in view of the topographical and geological conditions. After reducing 2 m of surcharge, normal water surface will be resulted at NWS EL.63.00 m and the effective storage capacity of 240,000,000 m^3 in case of a gated type spillway. However, in case of ungated type spillway, the crest elevation becomes EL.60.30 m and the effective storage capacity 201,000,000 m^3 even in case of adopting the maximum crest length (L = 180 m) allowed by the topographical conditions. Therefore, the gated type spillway is advantageous from the hydrological point of view, compared with the ungated type spillway. Also the comparison of the construction costs evidences the advantage of the gated type spillway. (Refer to 12-2 of supporting report)

From the future operation and maintenance point of view, the gated spillway is disadvantageous compared with the side-channel spillway (ungated type spillway). However, spillways of dams existing in Philippines are mostly of gated type.

Therefore it is decided to adopt the gated spillway. The design flood discharge is estimated at 3,100 m^3/sec, the design level of the high water surface (HWS) at EL.65.00 m and the normal water surface (NWS) at EL.63.00 m. For this purpose, a gate type which can be possible to overflow will be adopted in this spillway in case of emergency.

12.4. Structural design

12.4.1. Alignment

The chute of the spillway and the energy dissipator are designed with straight line in formation. The centerline of the spillway crosses the dam with an angle of about 63°.

12.4.2. Determination of the scale of the gate

The study for the determination of the scale of the gates is carried out to discharge a designed flood of Q = 3,100 m^3/s, a high water surface of EL.65.00 m and cases of 4 gates, 3 gates and 2 gates, respectively (Refer to 12.3.2 of Supporting Report). All cases present no problem from the hydrological point of view, but the number of gates is determined as 4 from the points of view of easier operation of the gates and economical advantages.

Accordingly, 4 gates with 9.50 m width x 10.5 m height will be installed in the spillway. The total width of the gate structure will be about 50.0 m including three piers of 3 m wide.

12.4.3. Chute portion and stilling basin

The width of the chute is about 50 m, same as gate structure. Chute portion is devided into two, namely steep slope and gentle slope in conforming to the topographical conditions, and the gradients in each case are 1:2.5 and 1:20, respectively.

To disperse the energy of flood, the auxiliary dam type is adopted in this spillway, in view of its advantages regarding hydraulic safety, adequacy to the topographical and geological conditions of the related area, etc..

12.4.4. Warning system for flood discharge

In case of flood, the gates of spillway structure must be open step by step to discharge the flood from reservoir. However, there is a possibility to give calamities to the villigers and fishermen working in the river downstream. Therefore, facilities of warning system should be necessary to inform the villigers living downstream of increasing the river water amount before opening the gates of spillway.

The Major Items of the Spillway

Location	:	Right abutment of the dam
Type	:	Chute type with gates
Designed Discharge	:	3,100 m^3/sec.
Water Head	:	12.0 m (designed discharge)
Gate Size	:	H = 10.5 m, B = 9.5 m
Gate Number	:	Four units
Width of Chute Way	:	47.0 m
Slope of Chute Way	:	1:20 upstream and 1:2.5 downstream portion

Elevation

High Water Surface	:	65.0 m
Normal Water Surface	:	63.0 m
Top of the Weir Crest	:	53.0 m
Bottom of Aproach Channel	:	48.0 m
Bottom of Stilling Basin	:	7.4 m
Crest of Auxiliary Dam	:	14.1 m
River Bed	:	12.0 m

Velocity of Flow through Spillway Structure

Aproach Channel	:	3.88 m/sec
Crest	:	7.37 m/sec
Max. Velocity of Chute Portion	:	28.34 m/sec

13. Diversion Works

13.1. Selection of Diversion System

There are two alternatives in temporary diversion of river when construction of dams, namely, a diversion tunnel system and the open channel system passing through a dam body. The open channel system is adopted often in cases of a concrete dam and a combined dam constructed in wide riverbed, where about 1/2 of the width of riverbed is sufficient to handle the design flood discharge during construction period. The tunnel system, which can be possible to construct the dam with more safety and reliability, is adopted in this project, in view of the following reasons.

- The rockfill type dam is selected for this project
- The amount of the design flood discharge amount is too large compared with the river width
- The foundation treatment in the riverbed requires a long period of time, etc.

13.2. Diversion Flood Discharge

The value of the design discharge to be adopted in the diversion work should be determined by taking into consideration the type of dam, the characteristics of the flood, the flood frequency during the construction period and the anticipated damages in case of overflow. Generally speaking, the 10-20 year probable discharge is adopted in case of fill-dams. The adoption of the 20-year probability seems to be appropriate in the case of the Mabini Dam, because the design standard of the Philippines is 1/20-1/50, in addition to the adoption of the open-cut method for the foundation treatment. Accordingly, the diversion flood discharge is estimated to be 1,500 m^3/sec (Refer to 4.4 Hydrology of the Main Report).

13.3. Selection of Route

It is recommendable to locate a diversion tunnel at the right bank side of the river, because the tunnel length results approximately 150 m shorter than that of the left bank alternative. The right bank side is also advantageous to be able to install the intake facility inside of the diversion tunnel, because the intake facility should be at right bank side where project area to be irrigated spreads.

13.4. Tunnel and Cofferdam

13.4.1. Cross section of the diversion tunnel and elevation of the coefferdam

The elevation of the cofferdam and the diameter of the diversion tunnel are mutually related. Accordingly, the combination of the size of the diversion tunnel and the elevation of the cofferdam is determined by carrying out the economic comparison. As can be seen in Table 13.1.1 and Table 13.1.2. of supporting report, the tunnel diameter of 8.5 m and the cofferdam elevation at EL.36.00 m is the most advantageous combination.

13.4.2 Number of tunnels

In case of using only one tunnel, an extremely large one, having a 2R type standard horseshoe type cross section with

12.2 m diameter and excavation cross section of 130 m^2 is required in order to discharge the flood flow of Qmax = 1500 m^3/s. (Refer to Table 13.2.1 of supporting report).
It is therefore decided to provide two diversion tunnels by taking into consideration the amount of the diversion discharge, the easy construction of the tunnel plug work, etc. One of the diversion tunnels will be used intake facility later on.

13.4.3 Combination of tunnel diameters

Economical comparison is carried out for 5 different combinations. As a result, the combination of 2 tunnels with standard horseshoe type cross section with 2R = 8.5 m is adopted (Table 13.2.2 of supporting report).

13.5. Major Dimension of the Diversion Work

Major dimension of the diversion work is summarized as followings:

Diversion Tunnels

Location	:	Right abutment of the dam
Cross Section	:	Standard horseshoe type
Tunnel Number	:	2-units
Tunnel Diameter	:	D = 8.5 m
Slope	:	1/350, 1/125
Tunnel Lenght	:	750 m, 790 m
Velocity	:	12.5 m/s (discharge: 1500 m^3/s)

Elevation of Tunnel Entrance : EL 13.20 m, EL 17.32 m

Elevation of Cofferdam Crest: EL 36.00 m

CHAPTER 14

INTAKE FACILITY

[illegible]

[illegible]

14. Intake Facility

14.1. Location

A maximum quantity of 25.31 m^3/s (21.666 m^3/s for irrigation + 3.644 m^3/s for river maintenance flow in maximum) of water is taken from the stored water between a normal water surface of HWS = 63.00 m and a low water surface of LWS = 38.00. (Refer to 9.3) The intake facility should be located at the right bank side, because all service area are located at the right bank side.

Two standard horseshoe type diversion tunnels with 8.5 m diameter are constructed at the right bank side in this project. One of the said diversion tunnels will be used as intake facility.

14.2. Intake Type and Structure

14.2.1. Selection of the type

There are various possible types of intake facilities, namely, tower type, inclined tower type, the shaft type, etc. (Refer to Figure 14.2.1). The shaft type is adopted as a result of the comparison study of the possible alternatives (Table 14.1.1 of supporting report), in view of the following study.

Tower type

The maintenance and inspection of the whole intake facility is easy in this type of structure, because an emergency gate and control gates are provided at the entrance of the intake tower.

However, this type of structure is appropriate for dams with shallow reservoir depth, or steep sloped abutment requiring a short connecting bridge and offering satisfactory conditions of bedrock. In view of the foregoings, this alternative seems to be not economical, because about 150 m long bridge with 5 through 6 spans will be required in this case.

Inclined tower type

This type of structure is applicable when the slope is steeper than 1:1.2. (This type of intake structure should have such slope as the gate comes down by the gravity). This might be one of the cheapest alternatives if the topography and geology is permitted.. However, this alternative is not recommendable due to the gentle slope (1:3.5) of the abutment where structure is located.

Shaft type

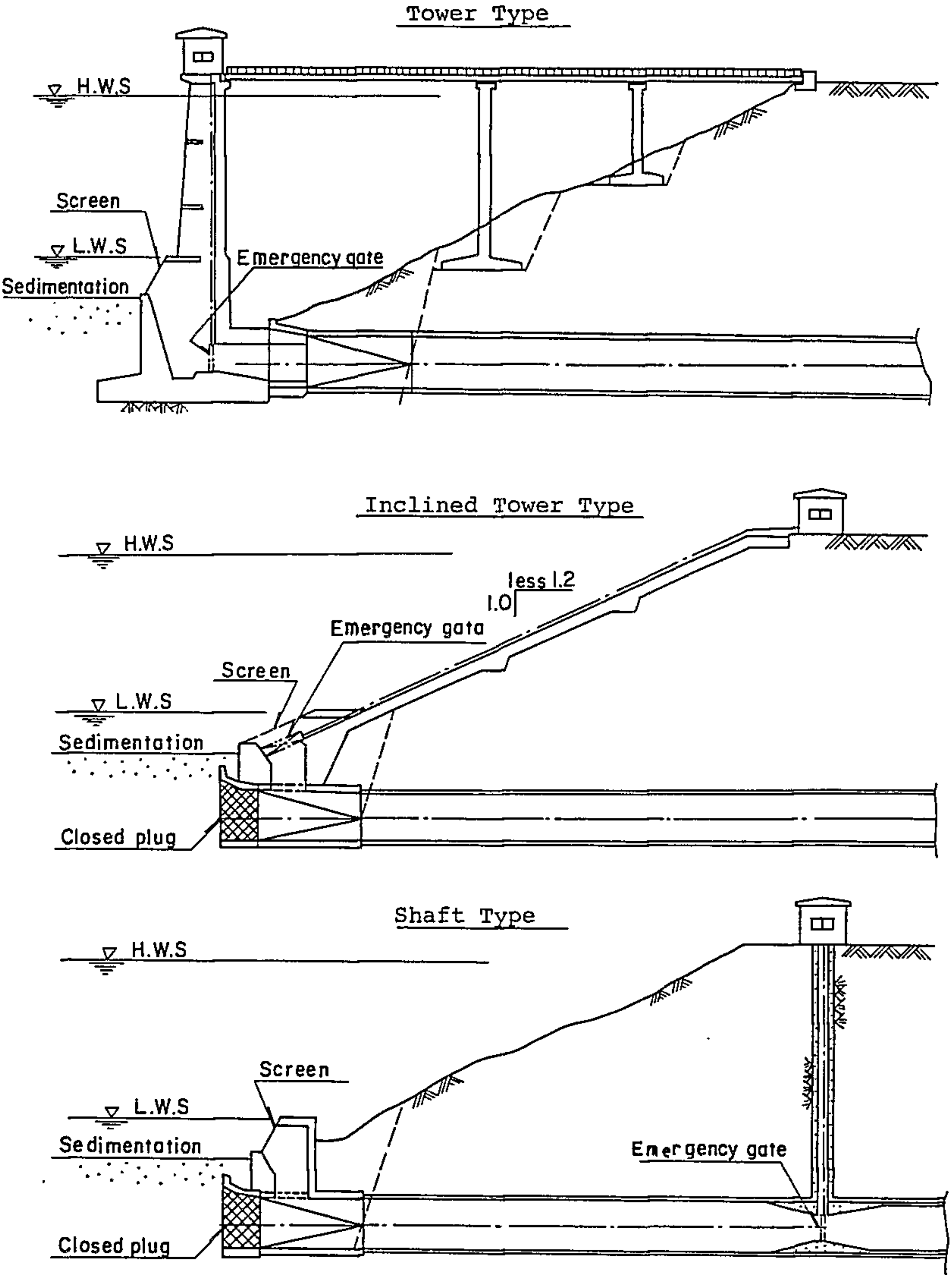
This type can be adopted, if the intake of the slope tower is not preferable. This is the standard type of pressure tunnel according to the "Design of Small Dams", published by the Bureau of Reclamation of the United States. However this type presents disadvantage, as it is almost impossible to inspect the tunnel portion upstream side of the emergency gate after completion. Therefore, it is required to locate the shaft as close as possible to the reservoir.

The economical comparison of these alternatives indicates that the construction cost of the tower type structure is 1175 x 10^3 US$, while the cost of the shaft type structure is 328 x 10^3 US$, i.e., the latter one costs 1/3.5 times lower than the former one. (Refer to Tables 14.1.1 and 14.1.2 of supporting report).

14.2.2. Structure

The intake structure will be a 10 m wide x 10 m high concrete structure with a steel screen and installed at the entrance of one diversion tunnel No.1. An emergency gate will be provided at upstream position of the diversion tunnel. The intake facility will use the existing diversion tunnel with the concrete plug at the middle reaches of the tunnel (length of plug portion is approximately 40 m), while at the downstream side of the plug, steel pipes will be installed.

Fig.14.2.1 Type of Intake Works



CHAPTER 15

IRRIGATION PROJECT

15. Irrigation Component

15.1. Alternatives of the Driving Channel

The driving channel of an open canal is constructed along contour lines. A straight line position between the point A and B of the length of 2.4 Km was studied for an alternative. (Section A-B, Refer to Figure 15.1.1) However, it was decided to adopt the open canal for the driving channel, because of the following reasons.

(a) The ground of the tunnel route and its vicinities is composed of the limestone layer, which presents risks to encounter spring water and caves. This is not appropriate for the construction of tunnels.

(b) The open channel is more advantageous from the economic point of view. (Refer to Supporting Report, Table 15.1.1).

15.2. Driving Channel

15.2.1. Tunnel of the driving channel

The alitude of the ground located behind the intake is of approximately EL.100 m, while the elevation of the intake is EL.38.00 m. Therefore, the upstream portion of the driving channel should be a tunnel. As mentioned before, two diversion tunnels will be excavated at the right side abutment. Accordingly, the most advantageous alternative is to use the aforesaid diversion tunnel as driving channel. Results of the study indicate that the use of the diversion tunnel for the irrigation and the hydroelectric power generation do not present problems. Therefore, it was decided to utilize one of the two diversion tunnels for driving channel.

15.2.2. Open section of the driving channel

(1) Selection of route

The open driving channel will be located along the topographical contour lines.

(2) Type of channel

The driving channel is required to pass a large discharge of the order of $Q = 21.666\ m^3/sec$, and there is a possibility of the leakage loss from the channel if it is unlined, because its foundation contains porous limestone. Therefore, the driving channel should be lined. There are two possible alternatives regarding the type of a lining channel, i.e., flume type and grouted masonry type. The grouted masonry type channel is adopted in this channel, in view of its economic advantages.

(3) Standard cross section of the channel

The driving channel will be a three face lined type one, and the road for the maintenance and operation purposes (B = 6.0 m, T-14t) will be constructed along the channel. The standard cross section stipulated in the design standards of the NIA will be adopted for the driving channel. The hydraulic specifications and standard cross section of the driving channel are as follows.

Discharge	:	$Q = 21.666\ m^3/sec$
Type of channel	:	Grouted masonry type
Applicable formula	:	Manning's formula
Coefficient of roughness	:	$n = 0.025$

The standard cross section of the driving channel is shown in Figure 15.2.1 of the Supporting Report.

15.3. Main Canals and Lateral Canals

15.3.1. Main canal Mo (Mabini Diversion Works - Diversion Works devided into East and West main canal Wo/Eo)

$Q = 20.536\ m^3/sec \qquad L = 3.0\ km$

The main canal will be an open canal type one. Some excavation will be necessary because the ground elevation is 10 m to 15 m higher than that of the bottom of the canal. (Refer to Figure 15.3.1. of the Supporting Report).

15.3.2. East and west main canal and lateral canal

(1) Routes of the canal

The East and west main canal will extend towards the east and west direction at the diversion point Wo/Eo. As can be seen from the discharge distribution diagram, it has a discharge of the order of 10 m^3/sec at its starting point, while at the downstream end of its discharge is approximately 1.0 m^3/s. The route of the main canal is located along the contour lines on the mountain, and the selection of route is made in such a way to attain a satisfactory balance between the excavations and the embankments. The lateral canals are arranged as either perpendicular or parallel to the main canal. The amount of the diversion discharge is of the order of 0.5 m^3/sec through 1.0 m^3/sec.

(2) Type of canal

In principle, both main canal and lateral canal will be earth lined type ones. The slope of the topography of this area is gentle (1/2000 through 1/4000) and the construction of the lining in the canal results only slight reduction of the cross section and is not economical.
The ground of the paddy area where these main and lateral canals are located is mostly composed of clayey soil or silty clay, and practically impermeable. The earth lines system will be adopted in both the main canal and the lateral canal, in view of the merit of using excavated earth materials to the embankment of the canal. (Refer to Figure 15.3.1 of the Supporting Report).

(3) Cross section of the canal

The standard design criteria of the NIA will be adopted as cross section of the canal. The standard design criteria of the NIA are presented in the next page. The distribution of slopes of the main canal and the cross section of the canal in accordance with the standard design criteria are presented in Table 15.3.1 and Table 15.3.2 of the Supporting Report.

(a) Side slope 1:1.5

(b) Allowable velocities 0.5 m/s through 1.0 m/s for big canal

Minimum velocity: 0.3 m/sec for small canal.

(c) Hydraulic formula for open canal flow

Manning's Formula $V = \frac{1}{n} \cdot R^{2/3} \cdot I^{1/2}$

$n = 0.025$ for earth-lined canals

$n = 0.030$ for farm ditches

(d) The base width (b) to depth (d) relationship

$b = 2d$ ..... $Q < 4.00\ m^3/sec$

$b = 2.5d$ ... $4.0 \leqq Q < 9.0\ m^3/sec$

$b = 3.0d$ ... $Q \geqq 9.0\ m^3/sec$

(e) Elevation of the water surface during crop cultivation should be set at least 30 centimeters above the ground surface from the point where the canal can already be irrigated.

(f) Free board

(i) $Fd = 0.4d > 0.3$ m in case of $d < 1.99$ m

(ii) $Fd = 0.25d + 0.30 < 2.0$ m $d > 2.0$ m

(g) Top berm width

For operation and maintenance

B = 6.0 m for main canal

B = 4.0 m for laterals and sub-laterals

15.3.3. Existing irrigation facilities

The maximum potential irrigable area covered by the existing irrigation facilities is approximately 3,000ha, according to data provided by the NIA. However, only 30-50 percentage of the area is irrigated. As a matter of fact, there are existing irrigation facilities in the area but the capacity is insufficient and the shortage of water occurs during the dry season. The irrigable area is located at elevations lower than new irrigation system of the project.

Therefore, the area irrigated by the existing facilities should be included in the service area of this project and is necessary to rehabilitated in some portion.

The regional water utilization associations has been organized around the existing irrigation facilities, and composed a local comunity from the historic and social points of view. Therefore, in case of setting up layout of irrigation systems, it should be considered in such a way to put such water utilization organizations into practical use, and the existing canals should be connected with newly constructed irrigation systems.

15.4. Related Canal Structures

15.4.1. Diversion works/check gates

In principle, the diversion works from the main canals will be used together with check gates. Each diversion work from the secondary canals will be planned in such a way to cover an paddy area of approximately 50ha.

15.4.2. Siphon

Siphons will be used at places where the canal crosses rivers, while aqueducts will be used at places where the canal crosses deep valleys. Siphons will be constructed at approximately 10 places in the main canal and at approximately 3 places in the lateral canals. The largest ones are the siphon crossing the Alaminos River in the main canal and the siphon crossing the Maseden River in the lateral canal.

15.4.3. Culverts

Culverts will be provided as drainage culverts, at places where the canals cross the drainage ditches. Box culverts made of concrete are recommendable, but pipe culverts can also be used, depending upon the size of the drainage ditches. Culverts will be installed at approximately 10 places in the main canal and at approximately 7 places in the lateral canals.

15.4.4. Bridges

At places where the canals cross provincial roads, it is necessary to construct bridges.

Bridges will be constructed at 10 places in the main canal and 18 places in the lateral canals.

Besides those ones mentioned above, the construction of foot bridges (B = 2.0 m) is required at intervals of 2 km in the lateral canals.

15.4.5. Spillway

Spillways will be provided at the upstream side of siphons and in principle at places where the capacity and cross section of the canal is changed by 5% through 10%. A total of 22 spillways will be required at the main canal.

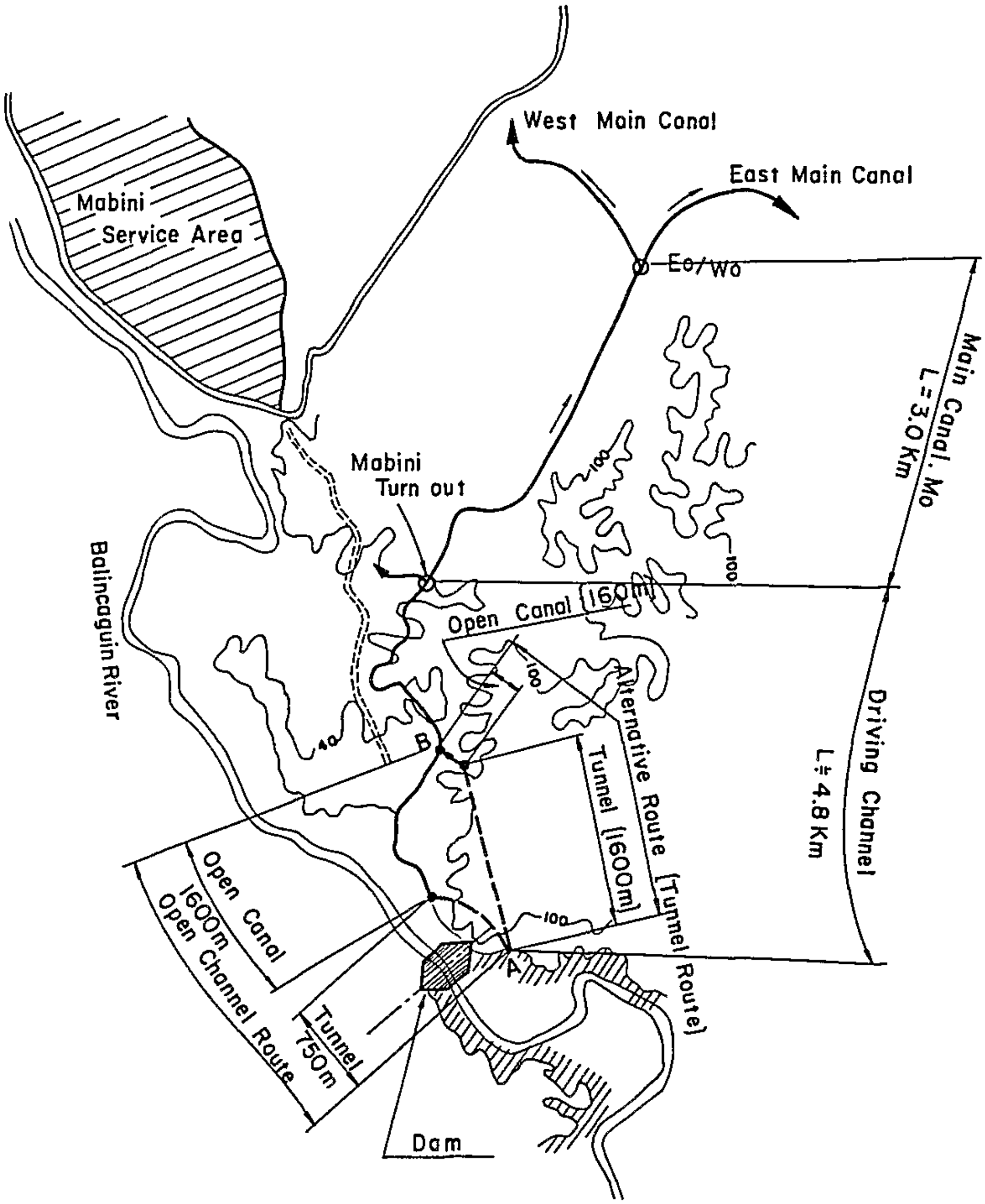
The outline of the structures mentioned above are presented in the Design Criteria of the NIA and therefore they will be used in the project. The list of the main structures and their locations are presented in Figure 15.4.1 and Table 15.4.1 of the Supporting Report.

15.5. Roads for Operation and Maintenance

There are two provincial roads located crosswise at the center of the 11,500ha service area. They have a very convenient configuration for the sake of planning the future road network. The network of roads required for maintenance and operation of the irrigation system will be connected to the said provincial roads.

The extensions of the main canal and lateral canals are 50 km and 135 km, respectively. These canals require the respective maintenance and operation roads. Data regarding the width of the said roads are contained in the Design Criteria of the NIA and they are perfectly able to be used in the present project. Excerpts of the Criteria are presented in the paragraph 15.5 of the Supporting Reports.

Fig.15.1.1 Location of Driving Channel



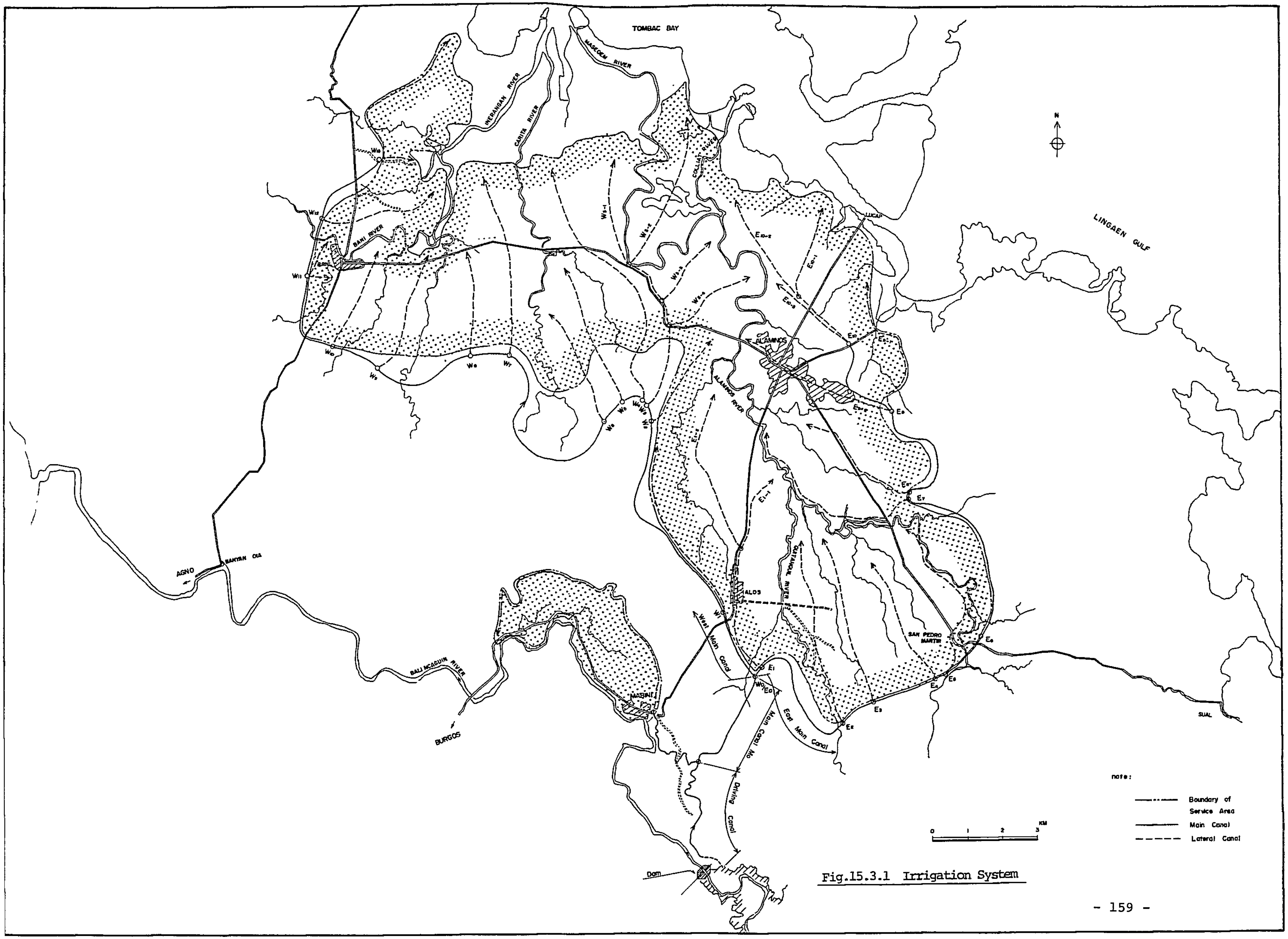
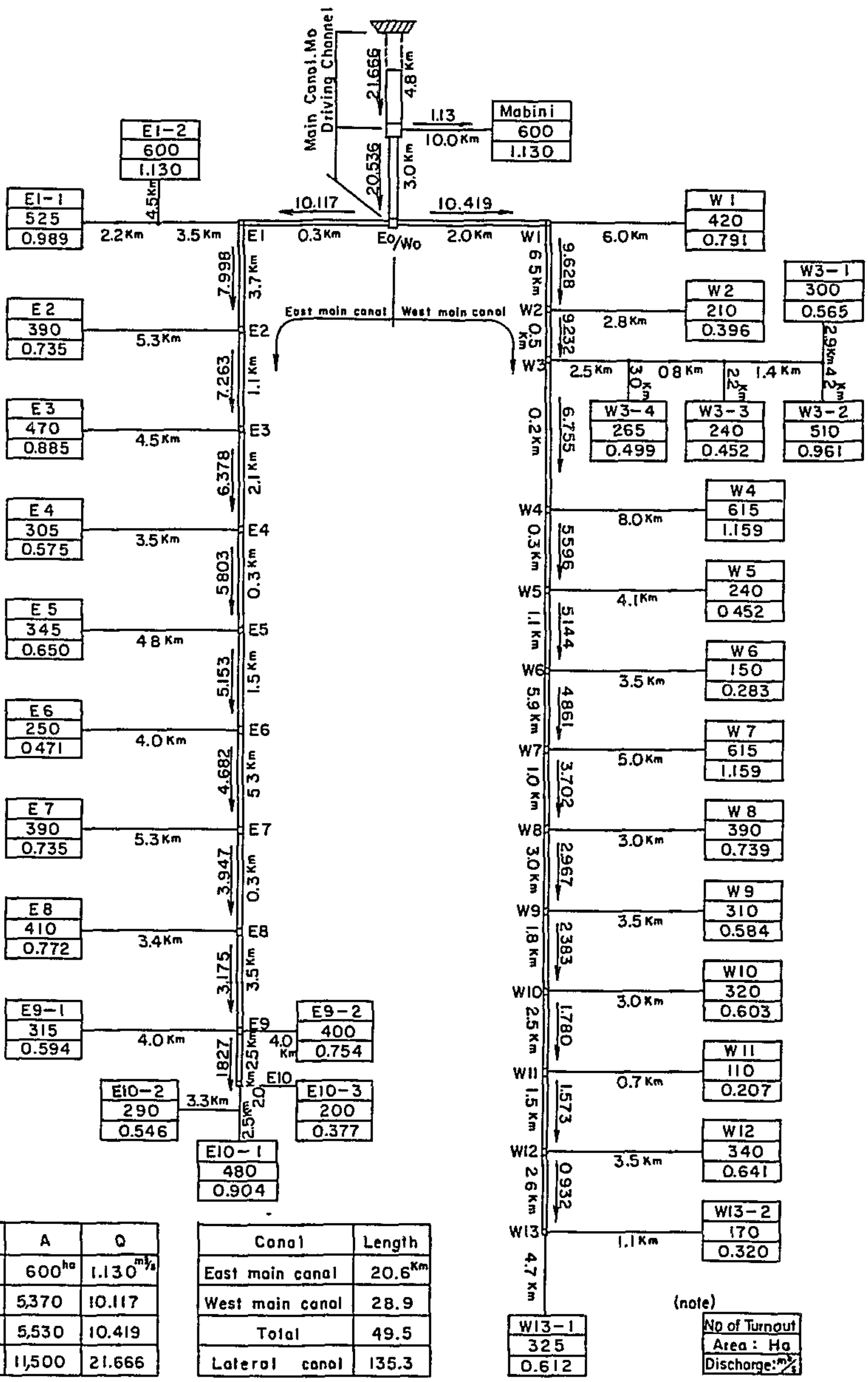


Fig.15.3.1 Irrigation System

Fig.15.3.2 Distribution Diagram from Main Canal



Area	A	Q
Mabini	600 ha	1.130 m³/s
Σ Ei	5,370	10.117
Σ Wi	5,530	10.419
Total	11,500	21.666

Canal	Length
East main canal	20.6 Km
West main canal	28.9
Total	49.5
Lateral canal	135.3

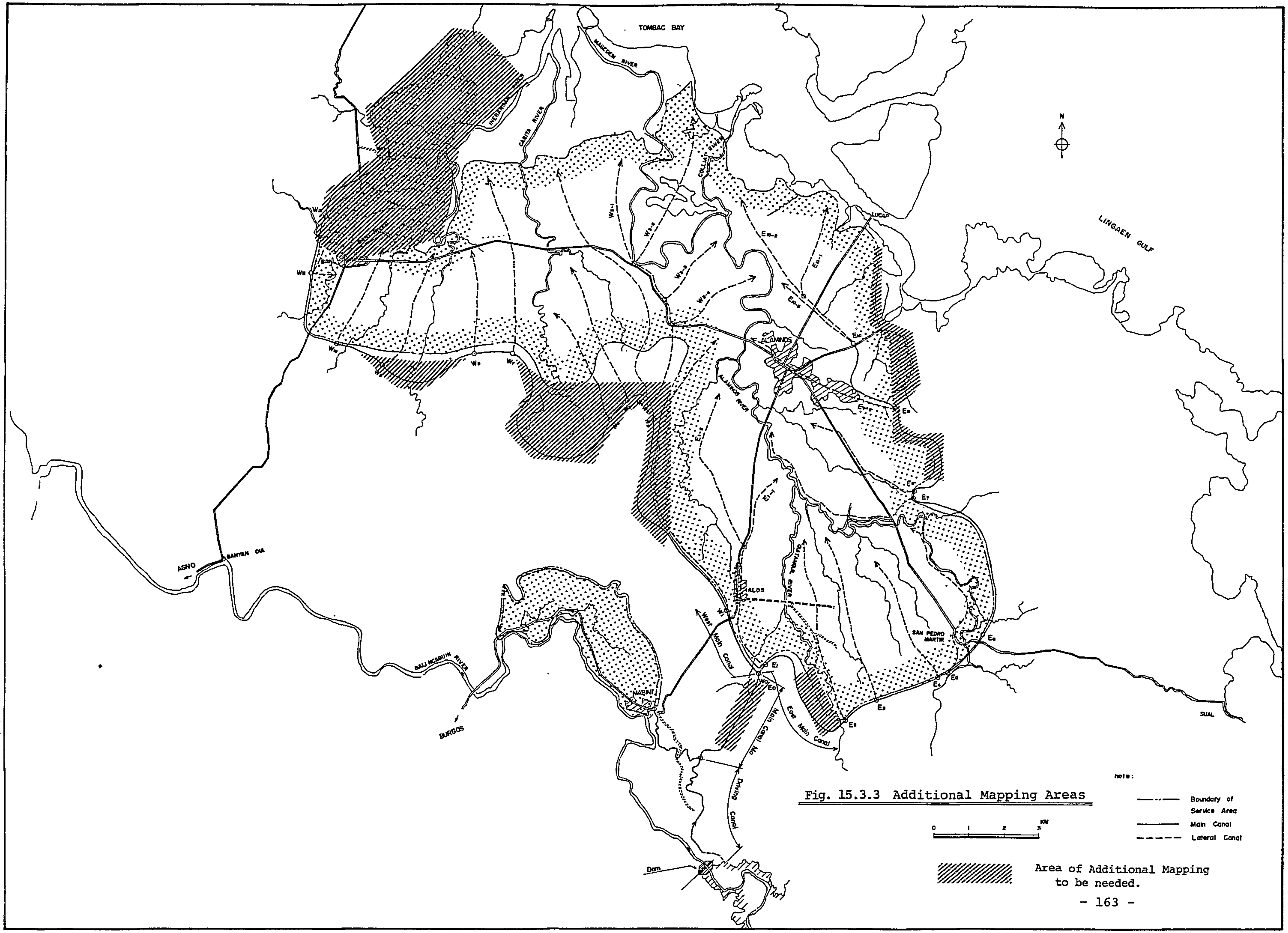


Fig. 15.3.3 Additional Mapping Areas

CHAPTER 13

DIVERSION WORKS

15.6. Drainage

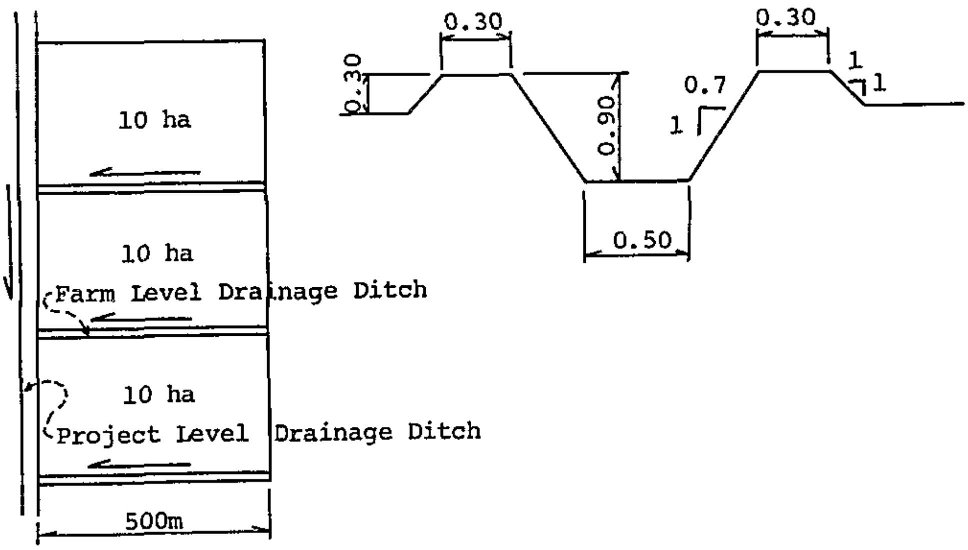
The drainage component of the Mabini Agricultural Development Project may be devided into two; namely project level drainage and farm level drainage. It may be preferable to implement project level drainage after the irrigation system of the project is completed, as farmers of the area are likely to be eager to obtain water for irrigation as soon as possible and are not keen to eliminate drainage problems without securing irrigation water. Farm level drainage is therefore included in this stage of the project.

In formulating the farm level drainage, unit water amount of drainage in the area was taken from the Design Criteria of the NIA as follows;

$$q = 6.0 \text{ Liter/sec/ha}$$

Density of drainage ditches in farm level will be about 50 m/ha. The construction cost of drainage system was estimated.

Layout and cross section of the drainage ditch are showing as follow.



CHAPTER 16

CONSTRUCTION MANNER OF THE PROJECT

16. Construction Manner of the Project

(1) Outline of Construction Manner

The construction of the Mabini Dam is characterized by the excavation of the cut-off trench in the river deposit with a depth of approximately 30m. Prior to starting the excavation of the trench, it is required to complete the construction of the diversion tunnels and the construction of the cofferdam. The excavation of the both abutments and spillway, which has no influences from the river flow should be started before completion of the cofferdam. The grouting of both abutments should be carried out in parallel with the excavation of the riverbed trench. The embankment with a total volume of 3,854,800 cubic meters should be started after the riverbed grouting. Excavated materials, except topsoil and organic materials, should be used for the embankment materials, either directly or after stockpiling. The filter material should be obtained by screening and washing the river deposit. Concrete should be mixed at a mixing-plant installed at the damsite, by using river deposit as aggregate. The CRITICAL PATH of the construction of the project is presented in Figure 16.1.

(2) Diversion Tunnel

Two diversion tunnels will be constructed at the right bank of the damsite. The half-face excavation method will be adopted in the construction of these two tunnels, because they have a larger diameter (8.5m). The excavation will be started from the upstream and downstream end of the tunnels, working in two shifts. Excavation will be carried out by using tractor shovels and dump trucks. The plan for the construction of these two tunnels should be drawn out in such a way to make possible the diversion of the sliding form of the arch and the sliding form and center of the side walls. These sliding form of 10.5m and the concrete pump should be used for arch lining, while the sliding forms of 7.5m and two sets of centers of 3m should be used for the side wall lining.

(3) Excavation of the Riverbed Trench

The most critical problem at the excavating of the river deposit with thickness exceeding 30m is how to dewater the seepage water. It is recommendable to construct cutoff walls of slurry trenches at the upstream and downstream side in order to reduce the infiltration of water. The submersible pump should be adopted in order to drain infiltrated water springing out at the excavation site and continued until the back fill of embankment will be reach to the original river bed. The excavation at the two abutments of the dam-site should be carried out by bulldozers and ripper-dozers, and the river deposit should be removed away by motor-scrapers with bull-dozers working as pushes.

(4) Grouting

Grouting should be started after finishing the excavation of the abutments and the riverbed, by using sets composed of two drilling machines, one grouting pump and one grouting mixer, 5 sets will be used for the grouting work. The consumption of the cement grouting is estimated at 3 bags of cement per meter.

(5) Embankment

Core

Borrow area is selected in the reservoir areas. Excavation and hauling by motor scrapers with pushers, spreading by bulldozers and compaction by sheepsfoot rollers will be used.

Filter

Sand and gravel of the river deposit will be used as filter material, after washing and screening. The loading and hauling will be carried out with a combination of tractor-shovels and dump-trucks, while spreading and compaction will be carried out by bulldozers and vibrating rollers.

Transition

For transition zone, excavated materials from other structures (mainly spillway) are expected to be transported either directly or via storage stockpiling. The combination of tractor-shovels and dump-trucks will be used for loading and hauling, while bulldozer and vibrating rollers will be used for spreading and compaction.

Rockfill

It is expected that large quantities of quarry-run materials will be used as rockfill. The quarry site will be excavated by the bench-cut method and the type of explosive used in this case will be dynamite. The same types of construction machinery used for the construction of the transition will be used.

(6) Spillway

The construction of the spillway should be set forth practically in parallel with the embankment, because the large quantities of excavated material will be utilized for the dam embankment. The excavation of rocks of which seismic wave velocity not exceeding 2 Km/sec is planned by using ripper-dozers, because it is considered that such kinds of rock is slightly soft and rippable. However, the fresh rocks will be excavated by the bench-cut method using dynamite. The finishing surface of excavation will be protected by pre-spliting. Concrete will be transported from the mixing plant by concrete-mixer trucks and will be placed by concrete pumps.

(7) Irrigation facilities

The construction of irrigation canals will be set forth in parallel with the construction of the dam, and they will be concluded within 5 years. Backhoes, dump-trucks, bulldozers and sheepsfoot rollers will be the major machinery to be used for the construction of the earth lining canals. It is expected that the lateral canals will be constructed mainly by embankment. Accordingly, earth required for the construction of the lateral canals will be excavated and transported from the borrow areas selected in advance.

The schedule of the construction work described above is presented in Figure 16.2.

Fig.16.1 Critical Path

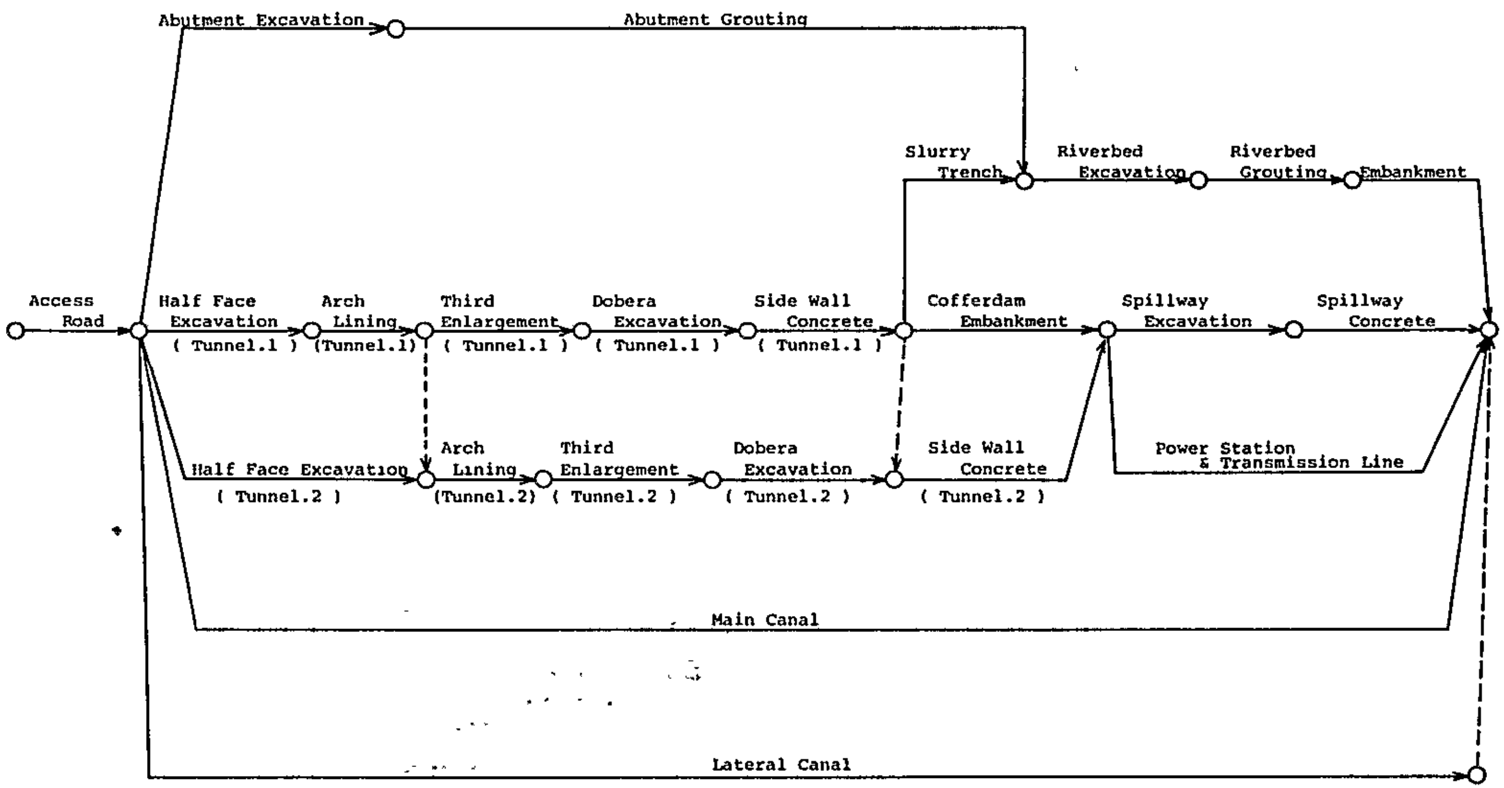
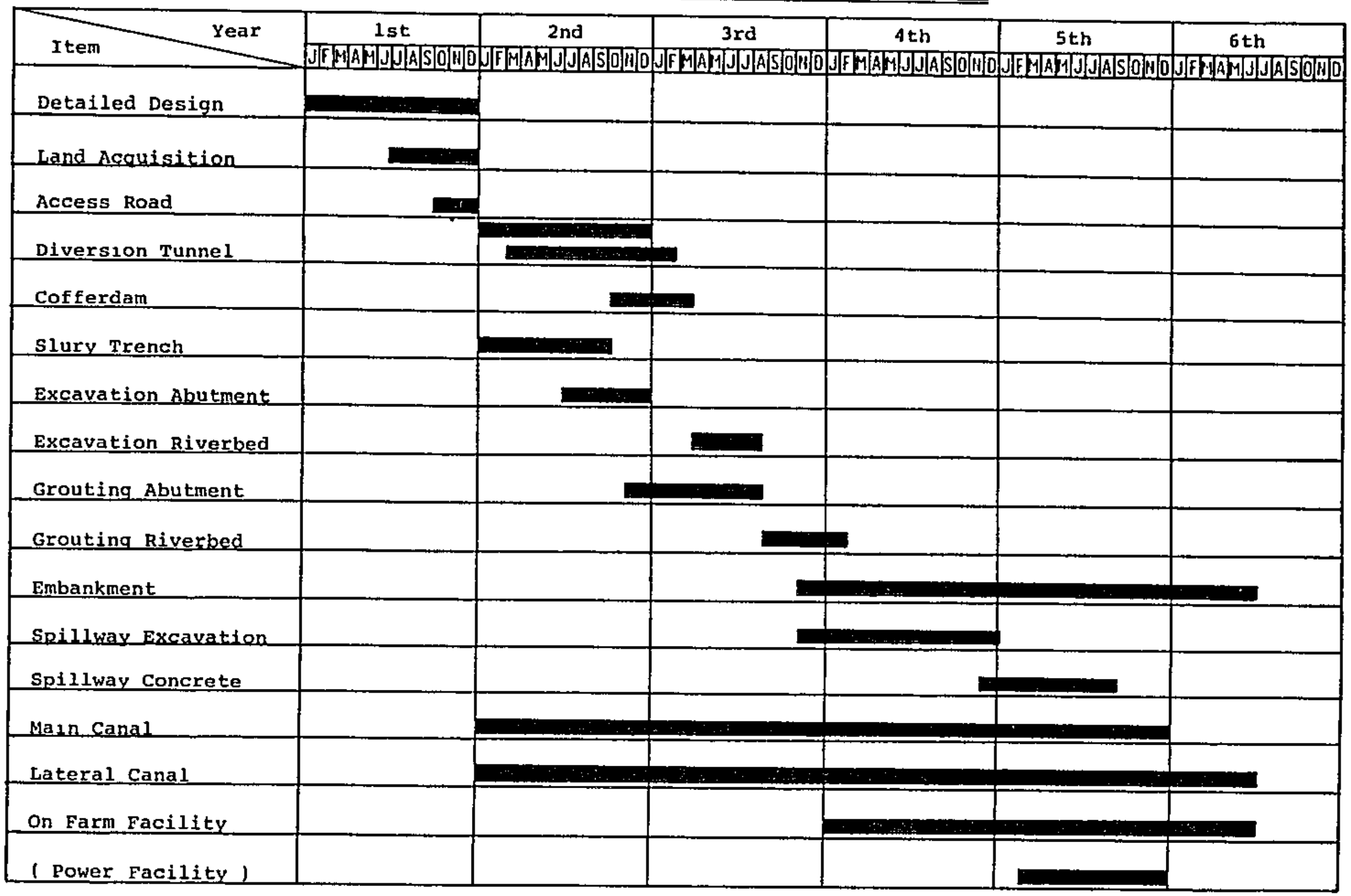


Fig. 16.2 Construction Schedule



CHAPTER 17

CONSTRUCTION COST

17. Construction Cost

(1) Construction Cost

The total construction cost of 1,017.0 million pesos (127.1 million US$) consists of 571.5 million pesos (71.4 million US$) of foreign currency and 445.6 million pesos (55.7 million US$) of local currency. The breakdown of the construction cost is presented in Table 17.1

The construction cost is divided into 697.7 million pesos (87.2 million US$) of the cost of dam construction cost and 319.3 million pesos (39.9 million US$) of the cost of the irrigation facilities. The breakdown of the these construction costs are presented in the Tables 17.2 and 17.3.

(2) Unit Prices

For the cost estimation, the unit prices in 1981 are adpoted. The cost of machinery is calculated by converting the rental rates - October 11, 1979 "Revised Rental Rates for Use by All NIA Construction Equipment and Motor Vehicles" - into unit prices of 1981. The rental rates are applicable for only machinery possessed by the Philippine Government. Regarding the other types of machinery, the depreciation costs and the repair costs are estimated.

(3) Contingencies

Physical contingencies and price escalation due to the inflation should be included in the project cost. An amount equivalent to 10% of the construction cost will be physical contingencies. As for the price escalation, the price escalation ratios presented in Table 17.4 are adopted.

(4) Annual Construction Costs

The annual distribution of the construction cost, calculated based upon the project construction schedule, are presented in Table 17.5. The first year of implementation of the project is assumed to be 1983 and the price escalation is estimated.

(5) Classification of Local Currency and Foreign Currency

The following percentage of foreign currency of the various required materials determined by NIA "Basic Rate for Cost Estimate" are adopted.

1)	Cement	75%
2)	Steel bars & hardware	80%
3)	Fuel & oil	50%
4)	Equipment rental	75%
5)	Sheet pile	100%

Table 17.1 CONSTRUCTION COST

Item	COST (x 10^3) ₱		
	F.C	L.C	TOTAL
1. Main Works			
Dam	250,508	139,514	390,022
Irrigation	90,023	101,780	191,803
Sub-Total	340,531	241,294	581,825
2. Access Road	5,200	4,550	9,750
3. Land Acquisition			
Dam	0	34,000	34,000
Irrigation	0	5,962	5,962
Sub-Total	0	39,962	39,962
4. O/M Cost	0	4,000	4,000
5. Engineering Service	32,000	0	32,000
6. Physical Contingency	37,773	28,981	66,754
7. Price Escalation	155,946	126,797	282,743
TOTAL	571,451	445,583	1,017,034
Doller Equivalent to 1,000US$	71,431	55,698	127,129

Table 17.2 DAM CONSTRUCTION COST

Item	COST (x 10^3) ₱		
	F.C	L.C	TOTAL
1. Main Works			
Diversion	39,147	19,592	58,739
Foundation	33,527	13,654	47,181
Embankment	46,373	23,801	70,174
Spillway	98,786	64,270	163,056
Preparatory Work	32,675	18,197	50,872
Sub-Total	250,508	139,514	390,022
2. Access Road	5,200	4,550	9,750
3. Land Acquisition	0	34,000	34,000
4. Engineering Service	16,000	0	16,000
5. Physical Contingency	27,171	17,806	44,977
6. Price Escalation	121,573	81,409	202,982
TOTAL	420,452	277,279	697,731

Table 17.3 IRRIGATION FACILITIES CONSTRUCTION COST

Item	COST (x 10^3) ₱		
	F.C	L.C	TOTAL
1. Main Works			
Intake	12,961	4,439	17,400
Driving Canal	18,223	13,607	31,830
Main Canal Mo	962	658	1,620
East Main Canal	2,918	1,748	4,666
East Lateral Canal	7,448	3,964	11,412
West Main Canal	4,612	2,826	7,438
West Lateral Canal	8,692	4,632	13,324
Structure	19,015	22,130	41,145
On Farm Facility	3,450	34,500	37,950
Preparatory Work	11,742	13,276	25,018
Sub-Total	90,023	101,780	191,803
2. Land Acquisition	0	5,962	5,962
3. Engineering Service	16,000	0	16,000
4. Operation and Maintenance	0	4,000	4,000
5. Physical Contingency	10,602	11,174	21,776
6. Price Escalation	34,373	45,388	79,761
TOTAL	150,998	168,304	319,302

Table 17.4 PRICE ESCALATION RATIO IN PERCENT

		1981	1982	1983	1984	1985	1986	1987	1988
Foreign		9.0	8.5	8.0	7.5	7.0	6.0	6.0	6.0
Local	Civil	8.0	8.0	8.0	8.0	8.0	8.0	8.0	8.0
	O/M	8.0	8.0	8.0	8.0	8.0	8.0	8.0	8.0

Source ; IBRD-Appraisal Report,1981

Table 17.5 ANNUAL DISTRIBUTION OF CONSTRUCTION COST

Unit: ₱ 1,000

Works		Total	1st Year	2nd Year	3rd Year	4th Year	5th Year	6th Year
1. Civil Works		591,575	47,694	98,301	144,400	144,747	127,385	29,045
	F/C	345,731	27,408	65,025	84,177	83,247	70,747	15,127
	L/C	245,844	20,289	33,276	60,223	61,500	56,638	13,918
2. Land Acquisition		39,962	39,962	-	-	-	-	-
	F/C	-	-	-	-	-	-	-
	L/C	39,962	39,962	-	-	-	-	-
3. Engineering Service		36,000	16,000	3,200	3,200	3,200	3,200	7,200
and O/M	F/C	32,000	16,000	3,200	3,200	3,200	3,200	3,200
	L/C	4,000	-	-	-	-	-	4,000
4. Physical Contingency		66,754	10,366	10,150	14,760	14,795	13,059	3,624
	F/C	37,774	4,341	6,822	8,738	8,645	7,395	1,833
	L/C	28,980	6,025	3,328	6,022	6,150	5,664	1,791
Sub-Total		734,291	114,025	111,651	162,360	162,742	143,644	39,869
	F/C	415,505	47,749	75,047	96,115	95,092	81,342	20,160
	L/C	318,786	66,276	36,604	66,245	67,650	62,302	19,709
5. Price Escalation		282,743	19,231	28,995	57,316	72,519	78,410	26,272
	F/C	155,946	8,203	19,489	33,435	40,769	41,847	12,203
	L/C	126,797	11,028	9,506	23,881	31,750	36,563	14,069
TOTAL		1,017,034	133,256	140,646	219,676	235,261	222,054	66,141
	F/C	571,451	55,952	94,536	129,550	135,861	123,189	32,363
	L/C	445,583	77,304	46,110	90,126	99,400	98,865	33,778

CHAPTER 18

ECONOMIC EVALUATION

18. Economical Evaluation

18.1. Benefits of Irrigation

The four towns which is located in the project area have an administrative area of approximately 70,000ha, with approximately 26,000ha of agricultural land. The paddy harvested area in the rainy season of 1981 was 17,441ha, 15% of that corresponding to 2,559ha were of irrigated paddies. During the dry season of 1981 the paddy harvested area was only 40% of the irrigated area of the rainy season, i.e., 1,036ha.

Cultivation of paddy in two seasons by irrigation facilities will be possible throughout the whole service area (11,500ha) after completion of the project, being therefore expected a considerable increase in the production of paddy in the project area.

Utilization of land (ha)

	Present Situation	After completion of the project
Rainy season		
- Irrigated	1,700	11,500
- Rainfed	9,800	
Dry season		
- Irrigated	700	11,500
TOTAL	12,200	23,000

The paddy yield per unit area in the project area during the rainy season and during the dry season are 3.16 ton/ha and 3.19 ton/ha, respectively. The yield per unit area prevailing presently are low, because the supply of a sufficient irrigation water is not possible through the existing irrigation facilities.

However, the stable supply of a sufficient irrigation water will be possible after completion of the project. The yields in the rainy season and the dry season are expected to be increased by approximately 1.45 times and 1.5 times, respectively, as a result of the strengthening of favourable factors like the introduction of high yielding varieties, effective application of fertilizer and agricultural chemicals, etc., in addition to the supply of irrigation water.

<u>Yield (ton/ha)</u>

	Present Situation	After completion of the project
Rainy season		
- Irrigated	3.16	4.48
- Rainfed	1.93	(1.93)
Dry season		
- Irrigated	3.19	4.79

The annual production of paddy is expected to become 81,236 tons, as a result of the expansion of the cultivated area and as well as of the yield per unit area.

Production of paddy (ton)

	Present Situation	After completion of the project
Rainy season		
- Irrigated	5,372	52,670
- Rainfed	18,914	-
Dry season		
- Irrigated	2,233	55,085
TOTAL	26,519	107,755

The benefit attributable to the increase of production, calculated based upon the export price of rice, will be 112,497,000 Pesos per year.

18.2. EIRR

(1) Costs

The cost of project consists of the construction costs and the maintenance and operation costs. The taxes and interests are excluded, and the labour costs are calculated from the economic cost taking into consideration the shadow wage rates.

The conversion factor (0.827) prevailing in the Philippines is applied to the construction costs in general, while the standard conversion factor (0.820) is applied to the maintenance and operation cost, for the conversion of the financial costs into economic costs.

Cost of the Project
(1,000 Pesos, Economic Cost, 1981 Constant Price)

1st year	104,948
2nd year	92,945
3rd year	134,881
4th year	135,197
5th year	119,402
6th year	33,550
TOTAL	620,923

The maintenance and operation cost required annually is 3,772,000 Pesos.

(2) Benefits

The amount of annual benefit caused by the irrigation facilities and others is 115,331,000 Pesos. This benefit will be raised after the 7th year from starting of the project construction, when the dam will be full to its high water level. The construction of the dam and other related facilities will be completed in the 6th year.

(3) Life of the Project Facilities

It is assumed that the project will have a life duration of 50 years after starting full scale operation.

(4) IRR

The IRR (Internal Rate of Return) of the project calculated at 12.8%.

18.3. Indirect Benefits

18.3.1. Increase of Income of Farmers

The average farm land holding of farmers living in the project area is about 1.5ha. The future farm budget of the farmers living in this area, calculated based upon the farming area mentioned above is presented in Table 18.4.1.

If this project is not implemented, the annual income of an average farmer living in this area is expected to be 9,191 Pesos even in case of cropping two seasons of paddy. However, after the completion of the project the income is expected to increase to 16,667 Pesos, approximately 1.8 times of the income without project. However, the figures refer to land owner farmers, and in case of tenant farmers, it is required to discount the sharecropping charge from the said income. Presently, most of the tenant farmers pay tenant in fixed amount basis, and the tenant charge is of the order of 6 cav/ha/crop (1 cav = 50Kg). In case of share-croppers, 25% of the harvested quantity is shared by the landowner. The incomes corresponding to the farming modes are presented in the table below. From the considerations above, it is evident that the setting of an appropriate tenant system and fee is indispensable in order to give more impact for the paddy production to the tenant farmers.

Farm incom (Peso)

	Without Project			With Project
	Rainfed Paddyfield	Irrigated Paddyfield		Irrigated Paddyfield
		One Cropping	Two Cropping	Two Cropping
Land owner farmers	3,183	4,503	8,191	16,667
Tenant farmers (fixed amount)	2,303	3,623	7,432	14,908
Tenant farmers (share croppers)	1,768	2,186	4,536	9,798

18.3.2. Increase of Employment Opportunity

A labour force of 2,300,000 man-day will be required for agriculture sector after completion of the project, with an increase of 1,411,000 man-day compared with the situation of the without project.

The rate of unemployment in the four municipalities was 13.5% in 1975, which means 3,686 jobless workers. It will be possible to absorb such idle manpower in the agricultural sector and the dry season cropping will be possible in full scale after the completion of the project, contributing therefore to create employment opportunity all the year round.

18.3.3. Utilization of surplus water

The dam has a surplus of storage capacity in normal years, being therefore possible to supply the irrigation water to the additional areas by using the surplus water. (Refer to 9.4)

Table 18.1.1 NET VALUE OF PRODUCTION

	Area (ha.)	Yield (ton/ha)	Farmgate Price (₱/ton)	Gross Value of Production (₱/ha.)	Cost of Production (₱/ha.)	Cost of Farm Labor (₱/ha.)	Net Value of Production (₱/ha.)	Net Ret. for Proj. Area (1,000₱)
Wet Season								
Irrigated Rice								
W/O	1,700	3.16	1,995	6,304	1,740	524	4,040	6,868
With	11,500	4.58	1,995	9,137	2,180	655	6,302	72,473
Rainfed Rice								
W/O	9,800	1.93	1,995	3,850	1,010	459	2,381	23,334
With	-	-	-	-	-	-	-	-
Dry Season								
Irrigated								
W/O	700	3.19	1,995	6,364	1,685	524	4,155	2,909
With	11,500	4.79	1,995	9,556	2,295	655	6,606	75,969
TOTAL								
W/O	12,200	-	-	-	-	-	-	33,111
With	23,000	-	-	-	-	-	-	148,442

148,442 - 33,111 = 115,331

Table 18.2.1

IRR = 12.8 %

YEAR	COST	BENEFIT	(B-C)	PRESENT VALUE (B-C)
1	104948	0	-104948	-93039
2	92945	0	-92945	-73048
3	134881	0	-134881	-93977
4	135197	0	-135197	-83509
5	119402	0	-119402	-65383
6	33550	0	-33550	-16287
7	3280	115331	112051	48223
8	3280	115331	112051	42751
9	3280	115331	112051	37899
10	3280	115331	112051	33599
11	3280	115331	112051	29786
12	3280	115331	112051	26406
13	3280	115331	112051	23410
14	3280	115331	112051	20753
15	3280	115331	112051	18398
16	3280	115331	112051	16311
17	3280	115331	112051	14460
18	3280	115331	112051	12819
19	3280	115331	112051	11364
20	3280	115331	112051	10075
21	3280	115331	112051	8931
22	3280	115331	112051	7918
23	3280	115331	112051	7020
24	3280	115331	112051	6223
25	3280	115331	112051	5517
26	3280	115331	112051	4891
27	3280	115331	112051	4336
28	3280	115331	112051	3844
29	3280	115331	112051	3408
30	3280	115331	112051	3021
31	3280	115331	112051	2678
32	3280	115331	112051	2374
33	3280	115331	112051	2105
34	3280	115331	112051	1866
35	3280	115331	112051	1654
36	3280	115331	112051	1467
37	3280	115331	112051	1300
38	3280	115331	112051	1153
39	3280	115331	112051	1022
40	3280	115331	112051	906
41	3280	115331	112051	803
42	3280	115331	112051	712
43	3280	115331	112051	631
44	3280	115331	112051	560
45	3280	115331	112051	496
46	3280	115331	112051	440
47	3280	115331	112051	390
48	3280	115331	112051	346
49	3280	115331	112051	306
50	3280	115331	112051	272
51	3280	115331	112051	241
52	3280	115331	112051	213
53	3280	115331	112051	189
54	3280	115331	112051	168
55	3280	115331	112051	149
56	3280	115331	112051	132
TOTAL				-1310

Table 18.3.1 FARM BADGET

		Without Project			With Project
			Irrigated		Irrigated
	Unit	Rainfed	1st Crop Only	1st & 2nd Crop	1st & 2nd Crop
Total Cropped Area	(ha)	1.5	1.5	3.0	3.0
Cropping Intensity	(%)	100	100	200	200
Total Crop Production	(ton)	2.895	4.74	9.525	14.055
Gross Value of Production	(₱)	5,660	9,267	18,621	27,478
Production Cost(Excluding Labor)	(₱)	1,658	2,948	5,798	7,738
Cost of Farm Labor	(₱)	819	936	1,872	2,340
Net Value of Production (Before Water Charges)	(₱)	3,183	5,383	10,951	17,400
Water Charges*	(₱)	-	880	1,760	733
Net Value of Production (After Water Charges)	(₱)	3,183	4,503	9,191	16,667
Farm Labor Requirement	(man-days)	105	120	240	300

* Without Project: 6 cav/ha/crop (1 cav = 50 kg)
With Project: Rainy Season 2 cav/ha, Dry Season 3 cav/ha

18.3.4. Flood Control Benefits

(1) General

In addition, damages of flood used to occur in the area along the Balincaguin River from time to time. After the construction of the dam, it will be possible to control the flood flow of the river, and eliminate the flood damages.

(2) Study of the Previous Flood Tracing

1) Inflow Hydrograph

Measurement records of inflow hydrograph are not available. The available data and information presently are as follows.

- Annual maximum flood discharge rate.
- Data regarding the time sequence of the aforesaid maximum flood.
- Cronological table of the daily rate of discharge.
- Data of the adimensional graph method for preparation of unit hydrograph in accordance with the NIA, etc.

The inflow hydrograph required for the study is set up by the available data with comprehensive consideration.

2) Method of Operation of the 4 Gates of the Sill-way

The gates should be operated in such a way to keep the high water level of 63.0m at the beginning and decline stage of the flood, while when the water level exceeds the high level of 63.0m they should be fully opened.

Accordingly, the study of flood tracing is carried out by using the 4 gates planned in the dam as variables.

3) Analysis and Results of the Flood Tracing

As shown in Table 18.3.2, the 5 cases of flood with the highest levels of water are selected for the study. The number of spillway gates to be opened in order to discharge water and the summary of the results of flood tracing studies corresponding to them are presented in Table 18.3.2.

The graphical results of the flood tracking study regarding the floods occurred in August of 1968 and September of 1966 are presented in Figure 18.3.1 and Figure 18.3.2.

Table 18.3.2 Maximum Discharge Records and Results of Flood Control by the Dam

Year	Date	Maximum Inflow (m^3/s)	Maximum Outflow (m^3/s)			
			4 Gates	3 Gates	2 Gates	1 Gate
1968	Aug. 30, 12:00 Noon	1,626.0	1,626.0 (63.00)	1,626.0 (63.00)	1,319.0 (63.64)	822.0 (65.32)
1966	Sept. 9, 4:00 P.m.	1,498.0	1,498.0 (63.0)	1,498.0 (63.0)	1,244.0 (63.23)	722.0 (64.30)
1962	July 20, 5:00 p.m.	1,302.0	1,302.0 (63.0)	1,302.0 (63.0)	1,212.0 (63.05)	684.0 (63.90)
1960	Aug. 13, 7:00 p.m.	988.0	988.0 (63.0)	988.0 (63.0)	988.0 (63.0)	645.0 (63.48)
1964	Aug. 7, 8:00 p.m.	762.0	762.0 (63.0)	762.0 (63.0)	762.0 (63.0)	625.0 (63.26)

Notes: ( ) ; Water level (E.L.m)

Width of Spillway ; 9.5m x 4 Gates

Creast Elevation ; E.L. 53.0m

Normal Water Level; E.L. 63.0m

High Water Level ; E.L. 65.0m

Fig. 18-3- 1 (1)Flood routing (observed flood: Aug. 30, 1968)

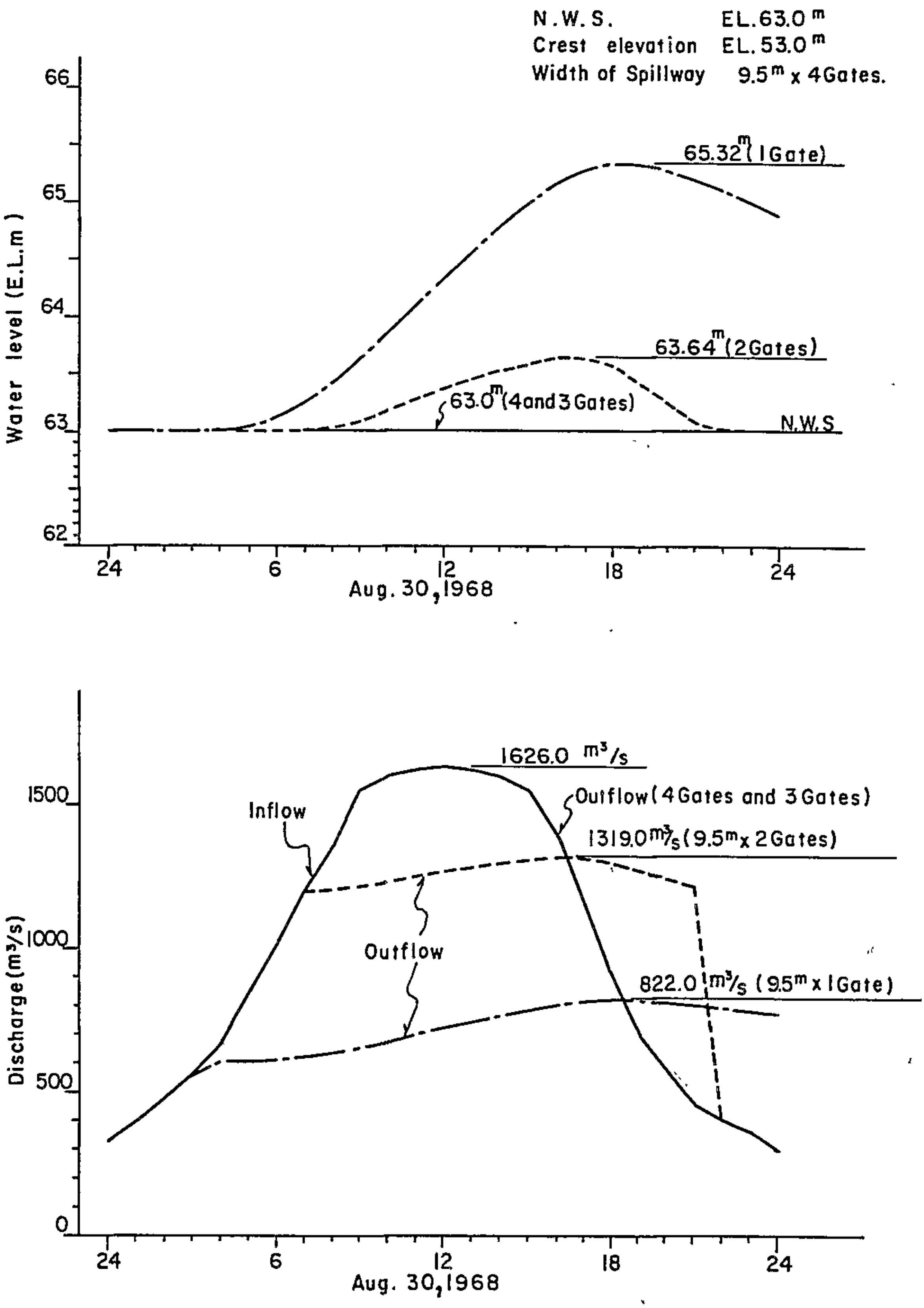
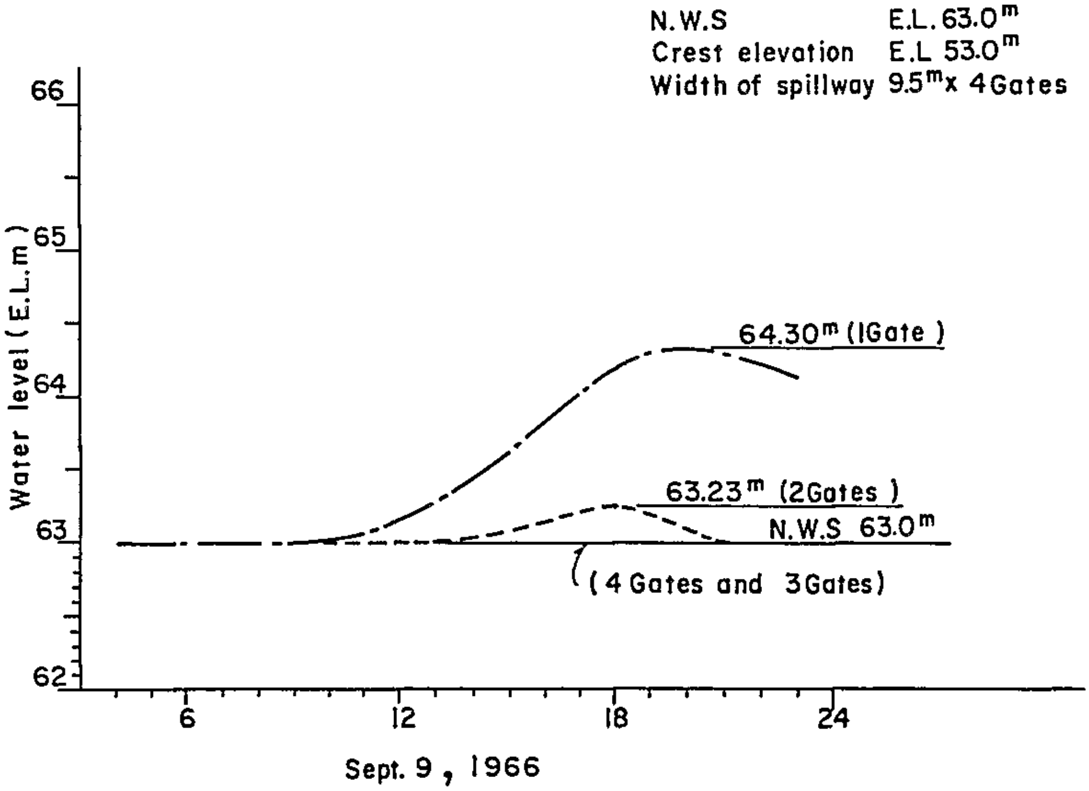
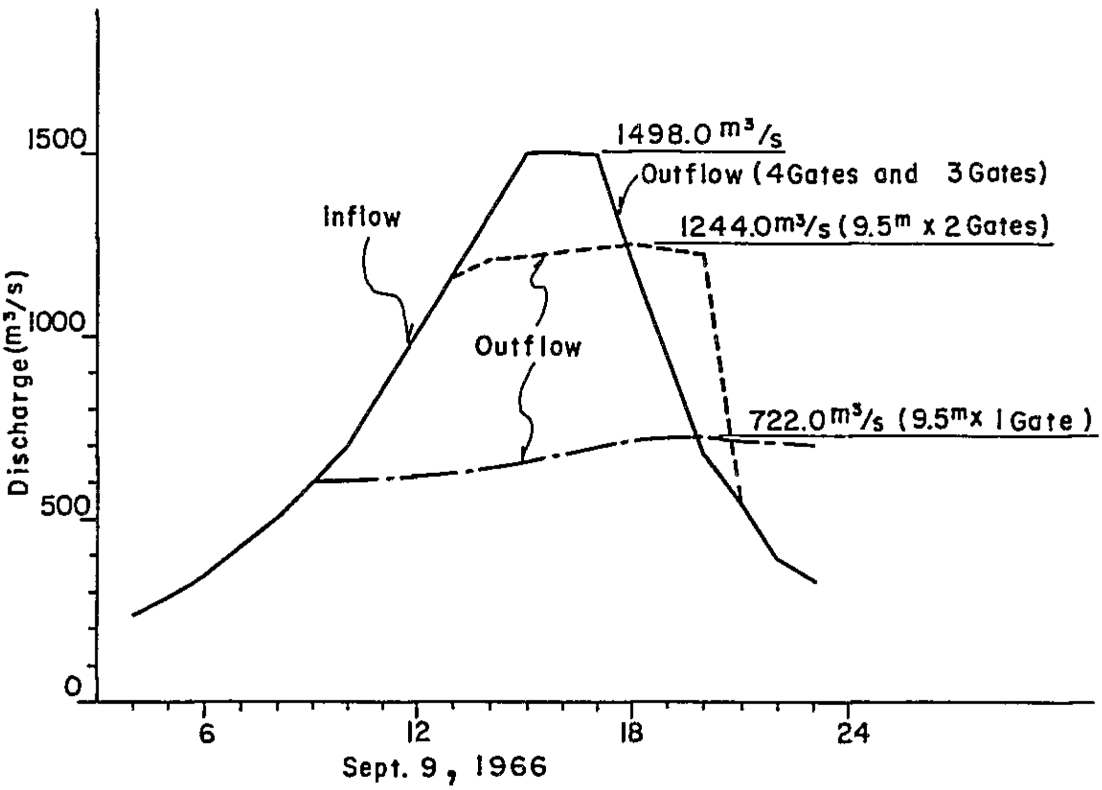


Fig. 18-3-1 (2) Flood routing (observed flood: Sept. 9, 1966)





CHAPTER 19

ORGANIZATION AND OPERATION

19. Organization and Operation

19.1 Implementation of the Civil Work

Construction of civil works

The construction of civil works of the project will be carried out under the responsibility of the NIA and the power generating facilities under the responsibility of the NPC. At the occasion of the detail design, the preparation of specifications and the purchase of construction machinery, equipment and materials, the NIA and the NPC will employ the foreign consultants for the technical cooperation. The supervision of the construction work will be carried out by the foreign consultant. The organization for the implementation of the construction work is shown in Figure 19.1

Compensation and indemnification

Resettlement of the people living in reservoir area and the vicinity of the dam-site to the service area should be performed with particular consideration.

19.2. Operation and Maintenance

The organization proposed for the operation and maintenance of the various facilities after completion is presented in Figure 19.2. The annual operation and maintenance cost estimated at 4 million Pesos and the breakdown of the cost is presented in Table 19.1.

Fig. 19.1 Proposed Organization for Construction

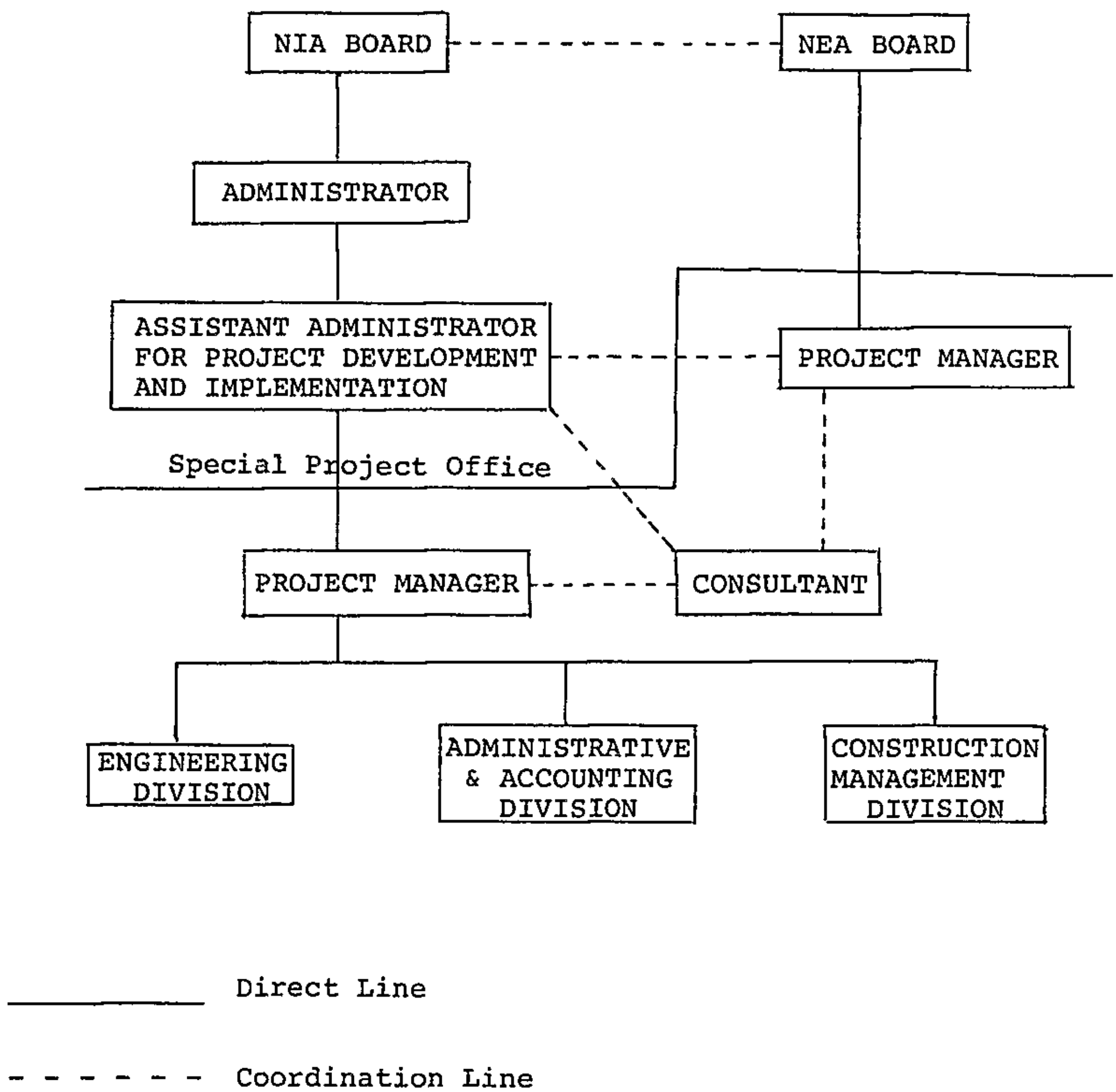
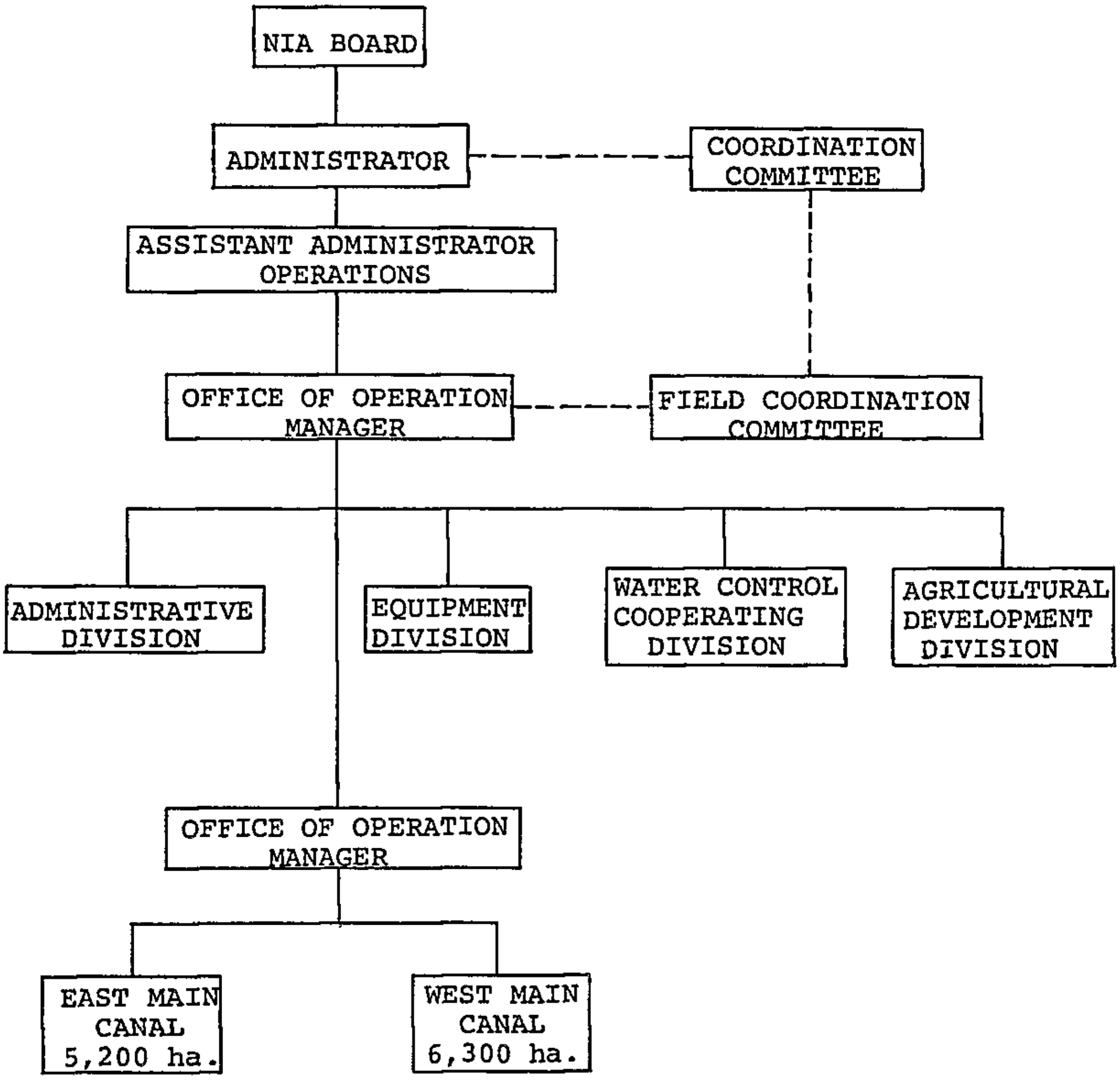


Table 19.1 Cost for Operation and Maintenance

	No. of Personnel	Total Salary per Annum
I. Salaries and Wedges		
A. Office		
1. Office of Operation Manager	4	90,000
2. Administrative Division	9	120,000
3. Equipment Division	10	120,000
4. Water Control Coordinating Division	10	170,000
5. Agricultural Development Division	9	170,000
6. Office of Superintendent	117	1,238,000
TOTAL A		₽1,908,000
B. Cost of Living Allowance		₽ 572,000
C. Incentive Allowance		₽ 159,000
D. Personal Insurance		₽ 181,000
TOTAL = A + B + C + D		= ₽2,820,000
II. Maintenance of Facilities		
Canal		₽ 100,000
Roadway Maintenance		₽ 425,000
Others		₽ 105,000
TOTAL		₽ 630,000
III. Materials and Supplies		₽ 170,000
IV. Administrative and General Expenditures		₽ 380,000
GRAND TOTAL		₽4,000,000

Fig. 19.2 Proposed Organization for Operation and Maintenance of Irrigation Project



ANNEX POWER GENERATION

A. Power Generation

A-1 Present Situation and Future Plan of the Philippines' Electrification

Due to the oil-crisis which shook the world economy in 1970's, the socioeconomic development program of the Philippines had to be dramatically revised.

The government of the Phillipines has been carring out the Five Year Development Plan for the period from 1978 to 1982, and one of its most important issues is the development of the national energy. Nevertheless, it is foreseen that the energy development should be accelerated after 1983, with faster pace and in much larger scale.

The total installed capacity of power plants in the Philippines in 1980 is 3,766 MW and the total annual generated energy is 15,086 (GWh). Even if all power plants now under construction are completed, the overall installed capacity of the power plants in the Philippines' would be only 5,737.5 MW, while the whole demand of power in 1991 would reach some 10,000 MW when we assume the fugure rate of increase as 7%, taking into consideration of both the future rate of population increase (3%) and the tendency of the improvement of living standard of the people. The government has to make its utmost efforts to solve the future energy problems from all aspects, by installing new power plants of more than 4,000 MW in addition.

The electricity in the Philippines has been supplied by the National Power Corporation (NPC), the Meralco System, the National Electrification Administration (NEA), and the Local Electric Cooperative Inc. which has been holding power plants built by private enterprises. The next tables, (Table A-1 and A-2 of Supporting Report) show the summarized installed capacity of the existing power plants (in 1980) devided in three systems in the Philippines and also the installed capacities of plants under construction. As seen in the Table A-2, the Philippines' power plants are of various types which include hydro, thermal, nuclear, geothermal and others. For the non-oil producing country like the Philippines, however, the development of the hydro-potential which still amply remains in various parts of this country should be, first of all, taken up and be implemented as fast as possible; it would be needless to emphasize that the development which utilizes the nation's own natural resources, i.e. water, would not cause air-pollution problems, while helping people living in the remote areas of the country. Once it is developed, it would not only supply cheap energy but also be operated quite easily.

The local electrification in the Philippines has been carried out along the line with the plan and policy of the National Electrification Administration (NEA). According to the plan, it is expected that 20, 50, and 100% of the electric energy required by the Local Electric Cooperative Inc. could be supplied by 1982, 1987, and 2000, respectively. On the other hand, the development of the small scale hydro-power plants has been initiated in 1930's, and, at present, many power plants are being operated such as the ones in Benquet Province in Luzon and others. Today, the total installed capacity of such small hydro power plants in Luzon, Mindanao, and Visaya exceeds 6,700 kW, 3,000 kW, and 1,000 kW, respectively (See Table A-3 of supporting report). The development of such mini-hydro plants should certainly be expedited faster and in the larger scale all over the country in the future in view of the increasing cost of the imported oil.

The Philippines is blessed with its high humidity and temperature, and its average annual precipitation exceeds 2,000 mm (3,000 mm even in some areas). It is also blessed with its topography, and we can find many places suitable for creating dams and reservoirs in various parts of the Philippines. It is therefore to be always taken into account to check a possibility to develop hydro-potential as much as possible whenever the construction of a dam and reservoir is considered even if the dam project was mainly planned to increase agricultural production.

It was therefore concluded that the physical, socio-economic feasibility should be studied for the direction to provide hydro-power plants in the Mabini Agricultural Development Project.

A-2 Power Generation

(1) Power Generation

The power generation is planned based upon the following conditions: the hydroelectric power plants are to be provided as an accessory facility of the irrigation project, and the design of civil engineering works of the irrigation project is not modified by the construction of the power plants. Therefore, it is not required to allocate the construction costs of dam and related facilities to the power component.

a) Power generation with irrigation water

It is possible to generate electricity by irrigation water during the irrigation season during the dry season, at the downstream end of the tunnel, by utilizing the water head between the water level of reservoir and the stilling basin elevation.

b) Generation of power with surplus water during rainy season

It is possible to generate electricity with water discharged from the dam to the river just at the downstream of the damsite during the rainy season, by utilizing head between the water level or reservoir and of the downstream riverbed.

(2) Installed Capacity

The following considerations are applied to the power generation with regard to the irrigation as well as the river maintenance.

- Maximum consumption of irrigation water (approximately) — 21.7 m^3/sec
- Average discharge required for river maintenance purpose at the downstream side — 2.3 m^3/sec

Total — 24.0 m^3/sec

a)	Normal water surface	EL 63.00 m
	Low water surface	EL 38.00 m
	Turbine center water surface	EL 37.00 m
	Rated water surface	EL 55.00 m
	Rated water head (Hr)	18.00 m
	Length of pressure tunnel (L)	1,000.00 m

Water head loss (Hl) = 1/1,200 x L + Hr x 0.02 = 1.2 m

Effective water head He = 18.00 m - 1.20 m = 16.8 m

P = Et x Eq x 9.8 x Q x He ≒ 3,000 kW

P :	Power generating capacity	
Et:	Turbine efficiency	80%
Eq:	Generator efficiency (3-phase synchronous)	95%
Q :	Discharge 24.0 m^3/sec ... Dry Season	

(Hereinafter this power station is called POWER STATION A)

b)	Normal water surface	EL 63.00 m
	Low water surface	EL 38.00 m
	Water level of turbine surface	EL 15.00 m
	Riverbed elevation	EL 12.50 m
	Rated water surface	EL 55.00 m
	Rated water head (Hr)	40.00 m
	Length of pressure tunnel (L)	1,200.00 m

Water head loss Hl = 1/1,200 x L + Hr x 0.02 = 1.8 m

Effective water head He = 40.00 m - 1.80 m = 38.2 m

Q: Discharge 24.0 m^3m/sec ... Wet Season

P = Et x Eq x 9.8 x Q x He ≒ 7,000 kW

(Hereinafter this power station is called POWER STATION B)

The Power Station A is operated during the dry season by using irrigation water, while the Power Station B is operated during the rain season by using water for river maintenance and the surplus water.

(3) Generated Energy

As described in the Supporting Report, the total quantity of energy to be generated by the Power Stations A and B is approximately 25 x 10^6 kWH. This generated energy can be consumed with ease in the 69 kV transmission line system of the NPC located in the project area, because the 69 kV transmission system has an extremely large capacity compared with the power generating capacities of the power stations A and B.

A-3 Revenue and Expenses

(1) Construction Cost

Table A-1 Construction cost

Unit: ₱1,000.

Item	Nos.	Price	Total Price
A Power Station 3,000 kW			
Including accessories, outdoor switching station	1 lot	20,000.	
Installation works, foundation and power house	1 lot	5,000.	25,000.
B Power Station 7,000 kW			
Including accessories, outdoor switching station	1 lot	40,000.	
Installation works, foundation, power house & residence	1 lot	15,000.	55,000.
Gates for A & B Power Station and Bypass			
Gates and accessories	1 lot	3,000.	
Installation works and civil works	1 lot	1,500.	4,500.
Penstock 100 ton	1 lot	2,500.	
Laying work & civil works	1 lot	3,500.	6,000.
Surge tank 100 ton and accessories	1 lot	2,500.	
Installation works & civil works	1 lot	2,500.	5,000.
69 kV transmission line 12 km			
Conductor & accessories	1 lot	1,400.	
Wooden pole & installation works	1 lot	1,600.	3,000.
69 kV Switching Station			
Including accessories	1 lot	1,000.	
Installation works and connection work to NPC line	1 lot	500.	1,500.

Table A-1 Construction cost (Cont'd)

Unit: ₱1,000.

Item	Nos.	Price	Total Price
69 kV distribution line & communication devices			
(Intake - Dam - A PS. - B PS.)			
Including accessories, line, remote control system devices and water level devices and communication devices	1 lot	6,000.	
Installation works	1 lot	1,000.	7,000.
Contigency		13,000.	13,000.
Total			120,000.

The main civil works for the construction of the irrigation facilities are not influenced by the construction of the power station A and B.

(2) Power Rate

1) The Annual income

According to the study at the Panganisan I Electric Corp. Inc. Bani Pangasinan, the unit price of electric power in the project area is as follows.

Table A-2 Unit cost of electricity

Type of Consumer	Content	Rate₱
Residantial & Public Building	Minimum Bill 1 - 12 kWh Excess / kWh	9.45 0.79
Commercial	Minimum Bill 1 - 12 kWh Excess / kWh	10.20 0.85
Industrial	Demand Charge / kW Energy Charge / kWh	15.00 0.78
Irrigation	Demand Charge / kW Energy Charge / kWh	15.00 0.71
Street Lights	175 W bulb / month Rate / watt	(0.626) 39.50 0.226
	Mean rate / kWh	0.751

According to the report of the NPC, the unit price per kWh in the Luzon System is 0.48 Pesos.

The annual income is calculated as follows, by assuming this unit price.

0.48 x 25,120,000 kWh = 12,057,600 (Pesos)

2) Calculation for Investment cost/kW and Generating power cost/kWh:

Assuming the total Investment costs for A and B Power Station including Transmission line and Switching station are ₱120,000,000, the Investment cost/kW is as follows.

₱120,000 ÷ (3,000 + 7,000(kW)) = ₱12,000 = $1,568/kW

Assuming the factors undermentioned as follows,

(a)	Rate of interest per year	4.5%
(b)	Rate of depreciation per year (Life: 50 years)	2.0%
(c)	Rate of fixed property tax per year	1.4%
(d)	Rate of operation & miantenance per year	0.5%
(e)	Rate of incidental expenses per year	0.5%
	Total	8.9%

Assuming the total kWh per year is 25,120,000 kWh, the generating power cost per kWh is as follows:

$$\text{kWh cost} = \frac{₱120{,}000{,}000}{25{,}120{,}000} \times 8.9\% = ₱0.425$$

According to the World Bank Report for the developing countries in 1980, the Investment cost/kW, Fuel cost/kWh and Generating power cost/kWh are as follows:

Table A-3

Kind		Investment cost/kW * unit Dollar	Fuel cost/kWh unit Cent	Generating power cost/kWh unit Cent
Hydro	Large scale, High head	1,100	non	2.4
	Small scale, Low head	3,500	non	12.7
Diesel	Large scale, Heavy oil	1,000	4.2	6.7
	Small scale, Light oil	800	10.9	13.2
Thermal	Large scale, Natural gas	800	0.4	2.4
	Large scale, Coal	1,000	2.7	5.2
	Large scale, Imported oil	800	5.5	7.5
	Small scale, Heavy oil	1,400	7.3	11.4
	Small scale, Wood	1,500	3.0	10.0
Geothermal	Dry steam	1,400	non	3.0
	Wet steam or Hot water	2,800	non	6.0
Nuclear	Large scale, Composite units	1,600	1.0	5.1
	Small scale, Single unit	2,200	1.0	7.4
Solarheat		20,000 - 30,000	non	100 - 300
Wind force		5,000 - 15,000	non	30 - 100

* Including the cost of Transmission and Distribution line in the Investment cost/kW. Excepting the cost of Battery in the Investment cost/kW of Solar heat and Wind force.

3) Benefits expected from the generation of electricity

Power will be generated in the present project by utilizing the head of water used for irrigation purpose. The annual generation of energy will be 25,000,000 kWh. This electricity will be supplied for the general use through the NEA.

The benefit expected from the generation of electricity is expressed in terms of cost required for the development of alternative power sources. In the present case, a small scale thermal generation would be considered as an alternative source of power. In case of a small scale thermal generation, power rate at generation end is 11.4 Cent/kWh or 0.91Pesos/kWh (refer to Table A-3). Accordingly, the benefit expected from the generation of electricity will be as follows;

25,120,000kWh x 0,91Pesos/kWh = 22,859,200Pesos

A-4 Conclusion

The Mini hydro power development supplemented to the Project will be able to generate power which will be usefull for the promotion of the social economic, and industrial growth and the commercial activities, and will achieve the electrification all over this region.

The hydro-electric power may be resulted in the reduction of dependence on the imported foffil fuel in its own way.

At any rate, the mini hydro power development shall be considered to be provided in the Mabini Agricultural Development.

Including power generation benefit, EIRR is 13.3%

Fig. A-1 Peak Demand & Capability in Luzon Grid

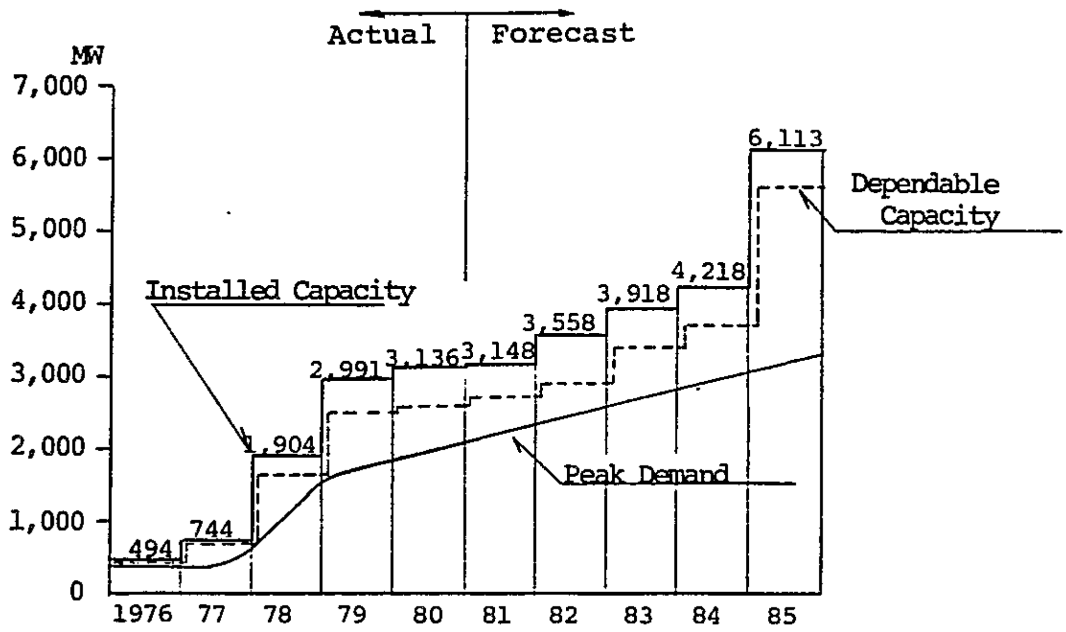
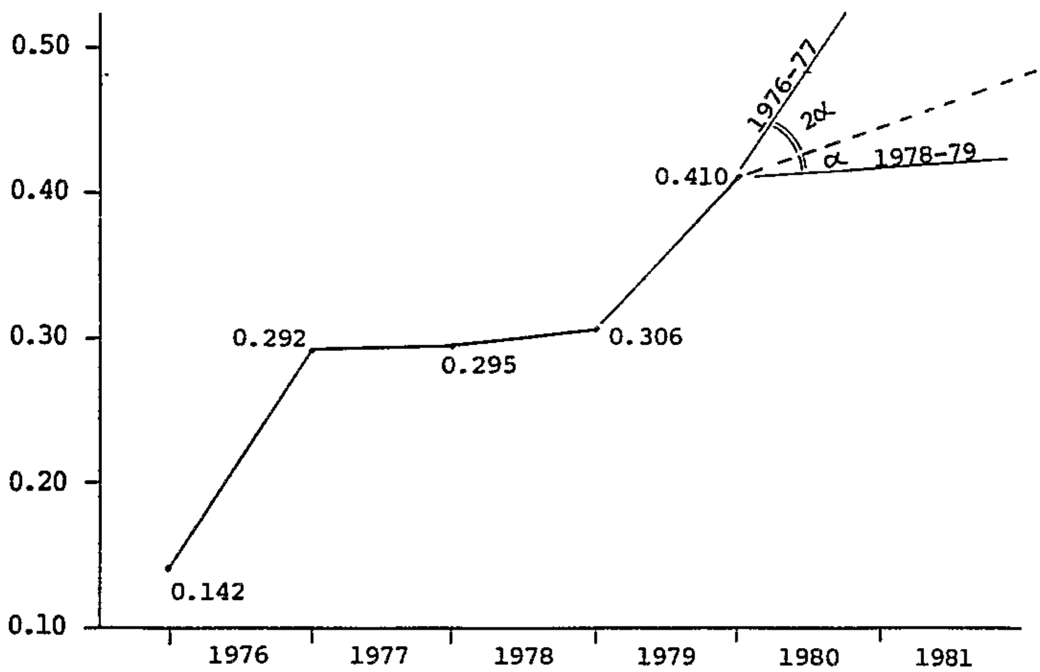


Fig. A-2 Average Rate per KWH in Luzon Grid



(N.P.C. 1980 Annual Report)

Table A-1 CONSTRUCTION COST

Item	COST (x 10^3) ₱		
	F.C	L.C	TOTAL
1. Main Works			
Dam	250,508	139,514	390,022
Irrigation	90,023	101,780	191,803
Power	76,400	30,600	107,000
Sub-Total	416,931	271,894	688,825
2. Access Road	5,200	4,550	9,750
3. Land Acquisition			
Dam	0	34,000	34,000
Irrigation	0	5,962	5,962
Sub-Total	0	39,962	39,962
4. O/M Cost	0	4,000	4,000
5. Engineering Service	32,000	0	32,000
6. Physical Contingency	45,414	32,040	77,454
7. Price Escalation	199,181	146,551	345,732
TOTAL	698,726	498,997	1,197,723
Doller Equivalent US 1,000$	87,341	62,374	149,715

Table A-2 POWER FACILITIES CONSTRUCTION COST

Item	COST (x 10^3) ₱		
	F.C	L.C	TOTAL
1. Main Works			
Power Station A 2,500Kw	20,000	5,000	25,000
Power Station B 5,000Kw	40,000	15,000	55,000
Gates for A & B Power Station	3,000	1,500	4,500
Penstock	2,500	3,500	6,000
Surge Tank	2,500	2,500	5,000
Transmission Line L=12Km	1,400	1,600	3,000
Switching Station 69KV	1,000	500	1,500
69KV Distribution Line & Communication Device	6,000	1,000	7,000
Sub-Total	76,400	30,600	107,000
2. Pysical Contingency	7,640	3,060	10,700
3. Price Escalation	43,235	19,754	62,989
TOTAL	127,275	53,414	180,689

Table A-3 ANNUAL DISTRIBUTION OF CONSTRUCTION COST

Unit: ₱ 1,000

Works		Total	1st Year	2nd Year	3rd Year	4th Year	5th Year	6th Year
1. Civil Works		698,575	47,694	98,301	144,400	144,747	234,385	29,045
	F/C	422,131	27,408	65,025	84,177	83,247	147,147	15,127
	L/C	276,444	20,289	33,276	60,223	61,500	87,238	13,918
2. Land Acquisition		39,962	39,962	-	-	-	-	-
	F/C	-	-	-	-	-	-	-
	L/C	39,962	39,962	-	-	-	-	-
3. Engineering Service and O/M		36,000	16,000	3,200	3,200	3,200	3,200	7,200
	F/C	32,000	16,000	3,200	3,200	3,200	3,200	3,200
	L/C	4,000	-	-	-	-	-	4,000
4. Physical Contingency		77,454	10,366	10,150	14,760	14,795	23,759	3,624
	F/C	45,414	4,341	6,822	8,738	8,645	15,035	1,833
	L/C	32,040	6,025	3,328	6,022	6,150	8,724	1,791
Sub-Total		851,991	114,025	111,651	162,360	162,742	261,344	39,869
	F/C	499,545	47,749	75,047	96,115	95,092	165,382	20,160
	L/C	352,446	66,276	36,604	66,245	67,650	95,962	19,709
5. Price Escalation		345,732	19,231	28,995	57,316	72,519	141,164	26,272
	F/C	199,181	8,203	19,489	33,435	40,769	85,082	12,203
	L/C	146,551	11,028	9,506	23,881	31,750	56,317	14,069
TOTAL		1,197,723	133,256	140,646	219,676	235,261	402,743	66,141
	F/C	698,726	55,952	94,536	129,550	135,861	250,464	32,363
	L/C	498,997	77,304	46,110	90,126	99,400	152,279	33,778

Table A-4

IRR = 13.3 %

YEAR	COST	BENEFIT	(B-C)	PRESENT VALUE (B-C)
1	104948	0	-104948	-92628
2	92945	0	-92945	-72405
3	134881	0	-134881	-92739
4	135197	0	-135197	-82044
5	221288	0	-221288	-118524
6	33550	0	-33550	-15860
7	4264	138190	133926	55880
8	4264	138190	133926	49320
9	4264	138190	133926	43531
10	4264	138190	133926	38421
11	4264	138190	133926	33911
12	4264	138190	133926	29930
13	4264	138190	133926	26417
14	4264	138190	133926	23316
15	4264	138190	133926	20579
16	4264	138190	133926	18163
17	4264	138190	133926	16031
18	4264	138190	133926	14149
19	4264	138190	133926	12488
20	4264	138190	133926	11022
21	4264	138190	133926	9728
22	4264	138190	133926	8586
23	4264	138190	133926	7578
24	4264	138190	133926	6689
25	4264	138190	133926	5904
26	4264	138190	133926	5211
27	4264	138190	133926	4599
28	4264	138190	133926	4059
29	4264	138190	133926	3583
30	4264	138190	133926	3162
31	4264	138190	133926	2791
32	4264	138190	133926	2463
33	4264	138190	133926	2174
34	4264	138190	133926	1919
35	4264	138190	133926	1694
36	4264	138190	133926	1495
37	4264	138190	133926	1319
38	4264	138190	133926	1164
39	4264	138190	133926	1028
40	4264	138190	133926	907
41	4264	138190	133926	801
42	4264	138190	133926	707
43	4264	138190	133926	624
44	4264	138190	133926	550
45	4264	138190	133926	486
46	4264	138190	133926	429
47	4264	138190	133926	378
48	4264	138190	133926	334
49	4264	138190	133926	295
50	4264	138190	133926	260
51	4264	138190	133926	230
52	4264	138190	133926	203
53	4264	138190	133926	179
54	4264	138190	133926	158
55	4264	138190	133926	139
56	4264	138190	133926	123
TOTAL				904

APPENDIX I

MEMBERS OF SUPERVISORY GROUP ON FEASIBILITY STUDY ON MABINI AGRICULTURAL DEVELOPMENT PROJECT IN THE REPUBLIC OF THE PHILIPPINES

Leader	Norio UCHIYAMA	Chief of Land Improvement Construction Department, Agricultural Structure Improvement Bureau, Ministry of Agriculture, Forestry and Fisheries (MAFF)
Irrigation	Hiromichi HENMI	Deputy Chief of Disaster Prevention Division Construction Department Agricultural Structure Improvement Bureau, Ministry of Agriculture, Forestry and Fisheries (MAFF)
Dam and Structure	Kenji HORII	Deputy Chief of Development Division Construction Department Agricultural Structure Improvement Bureau, (MAFF)
Economy	Kimihiko KITAKURA	Senior Project Economist Agricultural Investigation Division Hokkaido Development Bureau
Agronomy	Saburo NEGAYAMA	Deputy Chief of Resources Division Planning Department Agricultural Structure Improvement Bureau, MAFF

CONCERNED STAFF AND COUNTERPART PERSONNEL ON NATIONAL IRRIGATION ADMINISTRATION

Mr. Cesar L. Tech	Assistant Administration for Project Development and Implementation
Mr. Jose B. del Rosario, Jr.	Director Project Development Department
Mr. Avelino S. Rivera	Chief Water Resources Division
Mr. Rogelio P. de la Rosa	Chief Project Investigation Division
Mr. Edgardo B. Bernal	Sr. Investigation Engineer
Mr. Isidro Digal	Chief, Plan Formulation Division
Mr. Clemente T. Alanano	Chief Dams and Reservoirs Section
Mr. Manuel U. Estefanio	Dam Engineer
Mr. Edilberto B. Punzal	Chief Irrigation Works Section
Mr. Reynaldo R. Santos	Irrigation Engineer
Mr. Patricio C. Marquez, Jr.	Supervising Hydrologist
Mr. Romeo F. Potenciano	Chief Surface Water Section
Mr. Lolito E. Miguel, Sr.	Chief Geologist
Mr. Danilo A. Fajardo	Sr. Geologist
Mr. Erwin P. Ancheta	Geologist
Mr. Orlando D. Pascual	Chief Hydrogeology Section
Mr. Orlando C. Villalon	Sr. Hydrogeologist
Mr. Pio S. Gregorio	Hydrogeologist
Mr. Edgardo D. Rosario	Geologist

Mr. Epifanio Gacusan	Chief Land Resources Utilization and Economics Division
Mr. Dominador D. Pascua	Chief Land Use Section
Mr. Francisco T. Orense	Agronomist III
Mr. Juanito P. Pacleb	Sr. Soil Technologist
Mr. Faustino M. Galit	Chief Survey and Mapping Section
Mr. Thomas A. Filart	Chief Drafting Section
Mr. Emerson M. Coloma	Drainage Investigation Section
Mr. Romeo S. Roque	Director Region I Office
Mr. Daniel D. Asprec	Sr. Irrigation Engineer Region I

APPENDIX 3

FEASIBILITY STUDY TEAM
ON THE MABINI AGRICULTURAL DEVELOPMENT PROJECT
IN THE REPUBLIC OF THE PHILIPPINES

	Assignment	Name
1.	Leader	Yoshimi UCHIYAMA
2.	Irrigation Engineer	Megumi MORI
3.	Irrigation Structural Engineer	Sumitada OKAMOTO
4.	Irrigation Structural Engineer and Tunnel Engineer	Shiro KIKUCHI
5.	Agronomist	Takanosuke MARUSUGI
6.	Dam Design Engineer	Shinichiro MATSUMOTO
7.	Dam Structural Engineer	Masatoshi HIGASHIDE
8.	Hydrologist	Nobuyuki OKABE
9.	Geologist	Akinori TAKAKU
10.	Soil Mechanical Engineer and Construction Planner	Kazuo MIBAYASHI
11.	Hydro-Power Engineer	Taroh ITOH
12.	Economist	Takashi INOUE
13.	Geographical Engineer	Hiroshi SATO
14.	Geophysical Exploration Engineer	Ryoji IMAI
15.	Geophysical Exploration Engineer	Masaharu KAWASAKI
16.	Coordinator	Junji OHAMA