CHAPTER IV. THE PROJECT

Alternative Study on Optimal Scale of the Project

A. Basic Conception of Alternative Studies

Considering the topographical conditions of the surface water resources basin, four potential dam sites in the Pamacsalan, Wahig, Bagunan and Binabae rivers have been selected. In addition, a diversion weir could be proposed at just downstream of the confluence of the Pamacsalan and Wahig rivers, and named the Malinao diversion dam. The advantages of the diversion dam construction are i) discharge from the remaining catchment area located between the dam site and the weir site can be utilized for irrigation water, ii) the return flow from upper areas, located just downstream of the Pamacsalan and Wahig dams which is presently irrigated area as communal area, can also be utilized again for a lower irrigation area, and iii) no leading canal from dam to irrigation area is necessary.

As for the irrigation area, it can be categolized into three areas, that is i) Pamacsalan upper area located between proposed Pamacsalan dam and Malinao diversion dam, which is presently irrigated as the communal area of which the potential irrigable area is 138 hectares in net irrigation area, ii) Wahig upper area located along the both river sides of the Wahig river between the proposed Wahig dam and Malinao diversion dam which is also irrigated by several brush dams as communal area and the potential irrigable area is 482 hectares in net irrigation area, and iii) the lower irrigation area located downstream of Malinao diversion dam, of which potential irrigable area is 4,800 hectares in net irrigation area.

Comparing with the river bed elevation at the Wahig dam site (EL. 230) and the Pamacsalan dam site (EL. 190), there is about 40 meters difference, so that it might be possible to have a trans-basin of the Wahig river water to the Pamacsalan reservoir through a trans-basin tunnel about 5 kilometers in length.

As for the Binabae dam site, the available storage capacity is only 3 million cubic meters and the catchment area is only 13 square kilometers. In addition, the dam site is rather wide and the water cost becomes very high compared with other dams. Therefore, the Binabac dam is neglected from the alternative studies.

As for the Malinao diversion dam, it would be possible to have a stroage function in about 3 million cubic meters by heightening the weir by gates structure, though some of existing communal area would be submerged. The advantages of the storage function at Malinao diversion dam are as follows; i) the excess water of power released from the upper reservoir can be regulated for irrigation water use especially during non irrigation season, and a peak power generating would become possible, ii) the river water from the remaining catchment area can also be regulated by the Malinao site, and iii) compared with the size of catchment area at the Malinao site (138.8 sq.km) and the storage capacity, the stored water can be repeatedly used for irrigation. Considering the above advantages, it would be quite attractive to have a storage function at the Malinao site in spite of submerging the existing communal area in several hectares.

It was found that the discharge in the Bagun-an river is quite small compared with its catchment area. Therefore, the Bagunan dam has been considered as an after-bay function only to regulate the excess water of power from irrigation water demand.

The alternative studies have been categolized into the following three cases in view of configuration of water resources development.

i) Single reservoir plan: In order to check the potentiality of each proposed reservoir, single reservoir case has been studied.

ii) Multi-reservoir plan: Considering the size of the maximum potentiality of the irrigation area, it would be rather difficult to supply sufficient irrigation water by only one reservoir. Combinations of the proposed dams have been studied.

iii) Trans-basin plan: Based upon the trans-basin from Wahig dam to Pamacsalan dam, combination with the Malinao or Bagunan dam have been studied.

In addition, from the purposes of the water demand side, above mentioned alternatives have been grouped into i) irrigation only and ii) irrigation and power.

Consequently, 14 cases have been selected with the combination of proposed reservoirs and the purposes, and the configuration of each cases have been illustrated in Figure 4B-1. In order to find the optimal scale of development, the same level of studies have been performed for the 14 cases on the basis of reservoir water operation studies and economic evaluation by cost-benefit analysis.

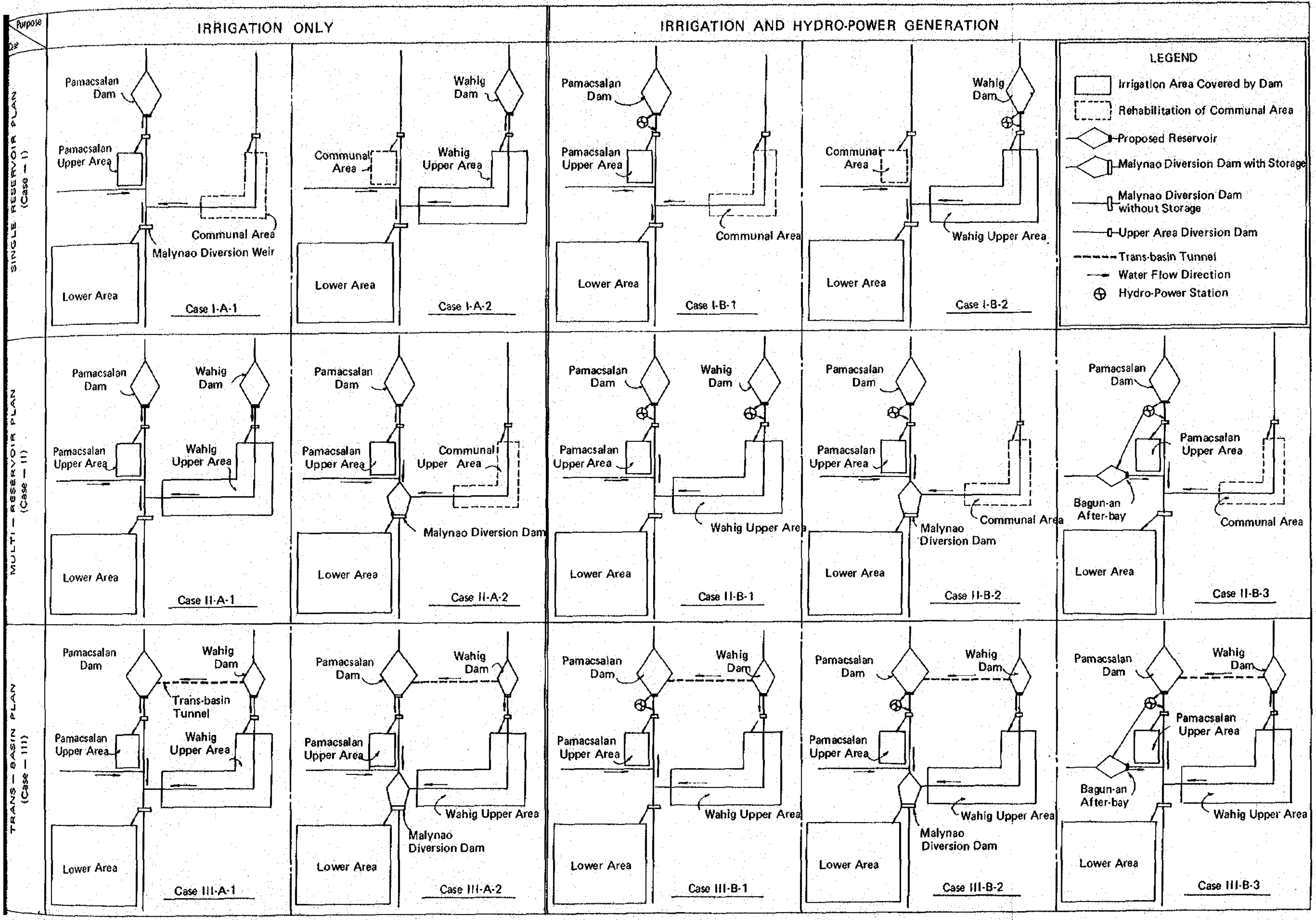
B. Initial Studies for the Alternative Studies

1. Optimal Cropping Calendar for Irrigation Demand

In the tropical and semi-tropical countries, there would be less problem to adopt a double-cropping of paddy because the temperature is almost evenly distributed through a year with the sufficient high degree of caltivating paddy rice. Therefore, the cropping calendar can be started from every period of a year. However in view of water availability there should be an optimal cropping calendar to maximize the natural rainfall use, in other words, to minimize an artificial water supply from reservoir or river flow through an irrigation canal. Normally, land soaking period brings the maximum irrigation water demand during the irrigation season, of which peak demand is closely related to a required land soaking period.

The land soaking period of 30-day, 45-day, 50-day and 60-day as well as the shifting of cropping calendar every 10-day interval have been studied to find the minimum water supply capacity through an irrigation canal. The 30-day land soaking requires the lowest water supply volume in annual total demand but the peak demand becomes the highest

FIGURE 4B-1 ALTERNATIVE CASE STUDIES ON OPTIMAL SCALE OF DEVELOPMENT



among them. On the other hand the 60-day one requires the highest water supply volume in a year but the peak demand occures during growing period. The 50-day one shows that the peak demand during land soaking period is almost the same as the one during growing stage.

As for the starting of a cropping calendar, the present one is normally started from 1st June. The earlier start of the calendar than the present one requires the more water in the first crop and the less water in the second crop. On the contrary, the later start of the calendar requires the less water in the first crop and the more water in the second crop. Therefore, it will be required to consider the double crop as one cropping calendar on annual basis. Nine types of cropping calendar starting from first 10-day of May, shifting every 10-day interval upto the last 10-day of June have been studied to estimate the crop water requirement, effective rainfall, diversion water requirement and the peak demand for the first and second crops. As the results, the starting from the last 10-day of May becomes the lowest diversion water requirements as well as the peak demand becomes the lowest. Starting from the first June, the condition also becomes the same as the one mentioned above. However, considering the frequency of typhoon in November, the cropping calendar starting from the last 10-day of May has been recommended to be adopted for the project.

In order to decide the land soaking period, the available labour force and possibility of farm mechanization have also been considered from the agricultural side. The reservoir water operation study has been made for the land soaking period of 30-day and 50-day in order to compare the view point of available water resources. The required storage capacity in 1972-1973, which is the drought year of 10-year return period, becomes almost the same for the both types, but in the normal year the 50-day one requires less storage capacity than the one for the 30-day. Accordingly 50-day land soaking period has been selected to be adopted for the irrigation of the Project Area.

2. Trans-basin Discharge from Wahig Reservoir

It is necessary to estimate a dependable discharge from the Wahig reservoir through the trans-basin tunnel beforehand. The trans-basin discharge, however, is varied in accordance with the water surface elevation of Wahig reservoir. Considering the estimated water surface elevation at Wahig and full water surface elevation of Pamacsalan, the out-let of the trans-basin tunnel will not be affected by the water surface elevation of the Pamacsalan reservoir. There would be more than five meters difference between them.

Adapting the Manning formula for the trans-basin tunnel, the discharge Q can be estimated from the following equation.

$$Q = a \times V = a \times \sqrt{\frac{2gH}{1 + \zeta e + \Sigma\zeta b + f\cdot\frac{\ell}{D}}}$$

where a; Cross section area of the tunnel (m^2)

V; Velocity in the tunnel (m/sec)

H; Total head

ζe; Entrance loss

ζb; Bend loss

ℓ; Total length of tunnel (m)

D; Diameter of tunnel (m)

f; Coefficient of friction loss

$$f = \frac{12.7gn^2}{D^{1/3}}$$

n; Coefficient of roughness $n = 0.015$

g; Acceleration of gravity (m/sec^2)

Considering the minimum diameter of tunnel in the practical construction, the diameter has been decided at two meters, and the total length of the tunnel is estimated at 5,200 meters from the topographical map of 1:50,000.

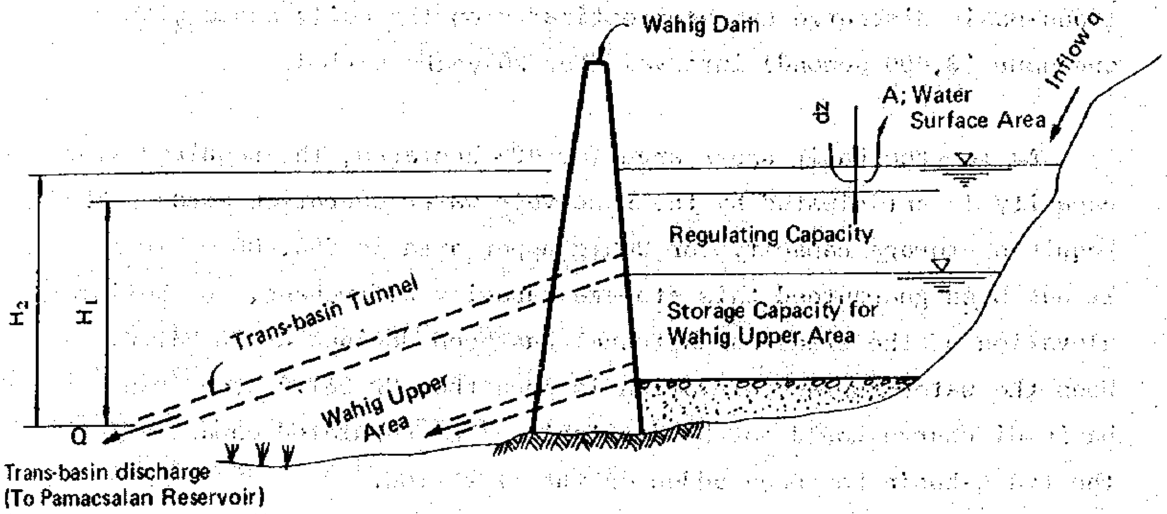
Therefore, the above equation becomes

$$Q = Ca\sqrt{2gH}$$

where $$C = \frac{1}{\sqrt{1 + \zeta e + \Sigma\zeta b + f\frac{\ell}{D}}}$$

Then $$C = \frac{1}{\sqrt{1 + 0.5 + 44.763}} = 0.12987$$

$$Q = 1.8065\sqrt{H}$$



The water surface elevation is varied dz within the unit time dt on the basis of trans-basin discharge from tunnel Q(cu.m/sec), the inflow q(cu.m/sec) and the reservoir surface area of A (sq.m).

Therefore, $A \cdot dz = (q - Q)dt$

where $Q = 1.8065\sqrt{H}$

If the dt is small enough compared with the changing of discharge Q, and head of dz, and the initial value of H_1 is known, the above equation is

$$dz = H_1 - H_2 = \frac{q - 1.8065\sqrt{H_1}}{A_1} \times dt$$

So, the next step of H_2 can be obtained from the above equation, and the discharge from the trans-basin tunnel for the next step is

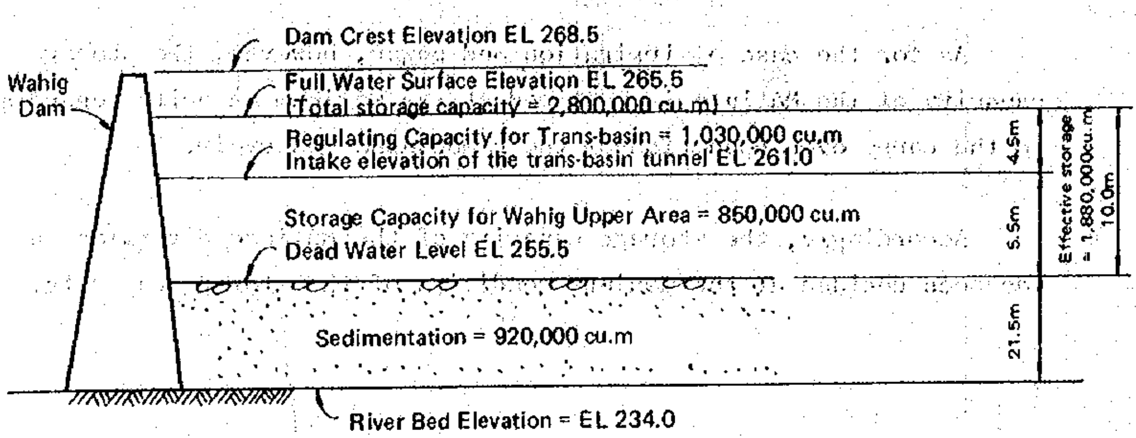
$$Q_2 = 1.8065\sqrt{H_2}$$

Considering the size of reservoir capacity and inflow amount, the trans-basin discharge has been estimated on the daily basis with the one hour (3,600 second) interval for 20-years period.

As for the Wahig upper area for 482 hectares, the required storage capacity is anticipated by the reservoir water operation study. The required storage capacity for Wahig upper area is 850,000 cubic meters. It has been guaranteed this storage capacity beforehand, so the intake elevation of the trans-basin tunnel has been decided at EL 261.0. When the water surface elevation is less than EL 261.0, the trans-basin discharge could not be available. The regulated capacity for the trans-basin has been added on the elevation.

Several trial-and-error method have been performed to decide the most effective storage capacity. The function of the Wahig reservoir is the same function of flood control reservoir. If the storage capacity is too small, there would be so much overflow from the reservoir, on the contrary, if the storage capacity is too big, the reservoir might not be full up. Considering these condition, the reservoir capacity has been decided to have a regulating capacity for a flood of once a two year. The regulating capacity has been decided at 1.03 million cubic meters.

Accordingly, the allotment of the storage capacity at the Wahig reservoir becomes as following figures.



3. Storage Capacity of the Malinao Diversion Dam

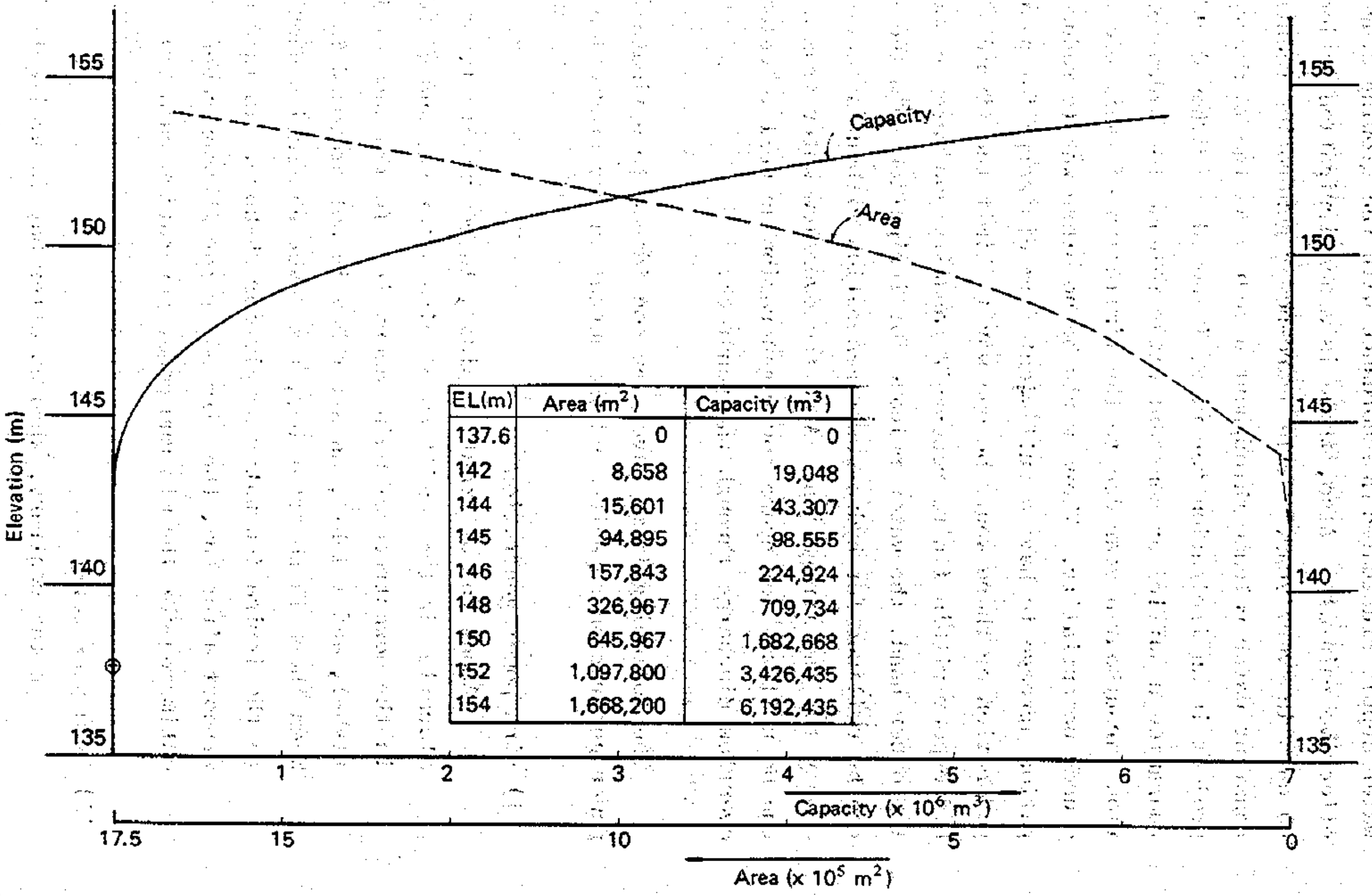
Increasing of the storage capacity of the Malinao diversion dam will affect to the submerged area of the existing communal area. Decreasing of the storage capacity can not have a enough function of after bay. As far as a function of after bay is concerned to regulated the excess water of hydro-power generation, especially during non-irrigation period, the bigger capacity of the storage is the more preferable. According to the topographical condition and submerged area of the existing communal area, the maximum full water surface is EL 152.0 and the available total storage is 3,426,000 cubic meters. Within the available storage capacity, the optimum storage capacity has been selected in accordance with the possibility of the expantion of irrigation acreage and firm peak power capacity.

As for the irrigation only, the combination of the Malinao dam and Pamacsalan dam can satisfy the enough irrigation water supply for the maximum irrigation acreage of the lower area with the capacity of 2,500,000 cubic meters at Malinao diversion dam of which full water surface elevation is EL 151.0. The maximum condition of the storage capacity at the Malinao diversion dam can be reduce the required storage capacity of the Pamacsalan reservoir.

As for the case of irrigation and power, however, the storage capacity of the Malinao is required at the maximum capacity regarding to the carry over condition of the Pamacsalan reservoir.

Accordingly, the storage capacity of the Malinao diversion dam has been decided at the maximum condition of the storage capacity.

FIGURE 4B-2 AREA - CAPACITY CURVES OF MALINAO DIVERSION DAM SITE



EL(m)	Area (m²)	Capacity (m³)
137.6	0	0
142	8,658	19,048
144	15,601	43,307
145	94,895	98.555
146	157,843	224,924
148	326,967	709,734
150	645,967	1,682,668
152	1,097,800	3,426,435
154	1,668,200	6,192,435

4. Features of Hydro-Power Generation

It would be rather difficult to combine the hydro-power generation and irrigation in the water deficit area because the irrigation water supply fluctuates in accordance with the growing stage of rice and effective rainfall. Also, there is a non-irrigation period for about 40 days in every year. While hydro-power generation requires almost constant water release throughout a year. As the main purpose of the project is an irrigation for rice, it would be rather difficult to guarantee a sufficient constant water release for hydro-power generation throughout a year due to the limitation of available water resource potentiality. There would be no other way to reduce the irrigation acreage to have a sufficient firm discharge for hydro-power generation or there would be one way to limit the firm discharge for hydro-poer within finite reservoir capacity and water resource. On the other hand, the water release for irrigation from the reservoir can be utilized for hydro-power generation without affection of water supply for the irrigation. Also the overflow water from a spillway can be utilized for hydro-power generation as well to release the overflow discharge through the penstock.

The following items have been considered to estimate the hydro-power generation capacity and the annual energy production value.

i) Firm power generation

Among the alternative plans, there are two types of reservoir alocation in the condition of with after bay and without after bay in the downstream of main reservoir. In the case of I-B-1, I-B-2, II-B-1, and III-B-1, an after bay is not proposed in the plan. When there is no after bay, the power generation should follow to the irrigation water supply for 24-hour. In this case of without after bay, the firm power for 24-hour generation has been estimated for the minimum power generation capacity.

ii) Firm peak power

On the contrary, when an after bay is available in the downstream of main dam, the water release for power can be regulated by the after bay capacity to supply for irrigation water for 24-hour. In the case of II-B-2, II-B-3, III-B-2 and III-B-3, the Malinao diversion dam or Bagunan dam can have a function of after bay to regulate the discharge of hydro-power. Considering a demand pattern in a day, the demand for power will increase in the night for lightening. Therefore, in this case of with after bay, 6-hour firm peak power generation has been estimated for the minimum power generation capacity.

iii) Dependable power generation capacity

In order to estimate the fixed value of KW benefit, the installed capacity would be rather high considering the KW value as a worth of propaty. As the firm power or the firm peak power can be guaranteed to generate throughout the year, the KW value of the firm power or firm peak power will be too low to estimate the KW value. Considering these condition, a dependable power generation capacity has been adopted to estimate the KW value. The dependable power capacity has been estimated by the average generation capacity during the computed 20-year period on 10-day basis.

iv) Annual energy production

The annual energy production of KWH has been estimated in accordance with the generated time in a day for the firm peak power and firm power.

v) The friction loss of the penstock and other losses have been anticipated to estimate the effective head of power generation.

vi) Considering the difference of the maximum power discharge and firm discharge, two units of hydraulic turbines have been adopted to be installed at the power station.

vii) The optimization of the installed capacity has been made on each case on the basis of dependable capacity and annual energy production value. The dependable capacity and annual energy production value have been computed on several different maximum power conditions for 21-year on 10-day basis. The most optimal maximum power capacity has been selected among the varios maximum power generation capacity on the basis economic analysis of the cost and benefit. The cost of power has been included the allcated cost of dam.

5. Operation Rules

The following criteria were adopted in the reservoir water operation studies on the basis of 10-day interval.

(a) The period of the study is the last 10-day of October 1956 to the middle 10-day of October 1976 for 21 years starting with the full water condition of reservoir in the last 10-day of October 1956. So, a crop year of the last 10-day October 1956 to the middle 10 - day October 1957 is indicated as 1956 - 1957 and so other year as well.

(b) The 100-year sedimentation volume has been adopted for the dead water.

(c) The reservoir loss of 8 percent has been adopted for the water release from the stored water in the reservoir considering the size of water surface area, evaporation value, and seepage loss from the reservoir area.

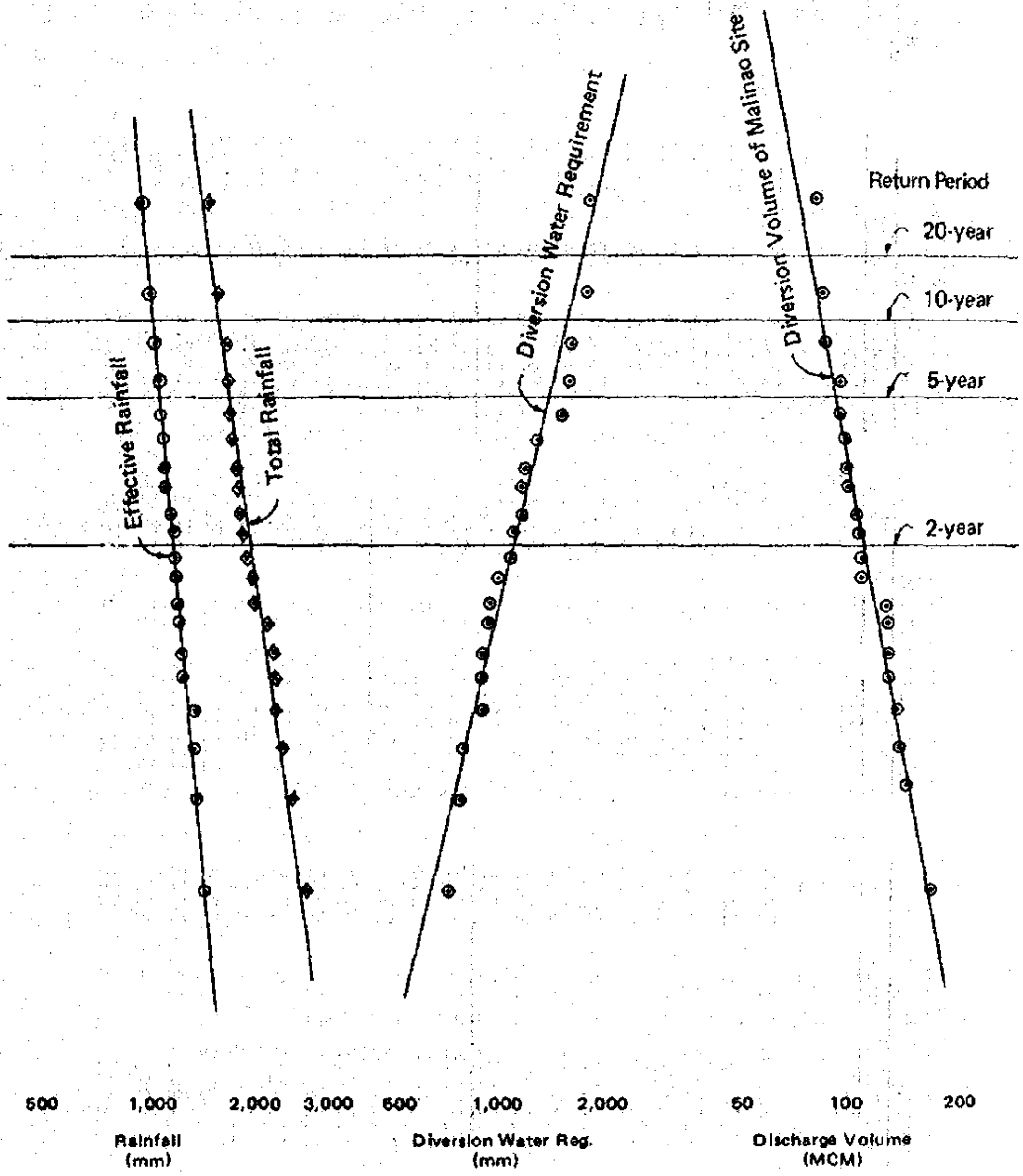
(d) As for the return flow from the upper areas, 30 percent of intake water has been considered as return flow that can be utilized again for the lower area.

(e) A 50-day land soaking period has been adopted for the estimation of water requirements.

Table 4B-1 Probability Analysis of Drought

	Total Rainfall During Irrigationn			Effective Rainfall for Irrigation			Division Water Requirement			Annual Discharge Volume at Malinao Site		
	Rainfall (mm)	Order	%	Rainfall (mm)	Order	%	Rainfall (mm)	Order	%	Discharge (MCM)	Order	%
1956-57	-	-	-	-	-	-	-	-	-	132.77	17	82.5
1957-58	1,875.4	11	52.5	1,072.5	6	27.5	1,137.3	8	37.5	76.45	1	2.5
1958-59	1,642.2	5	22.5	1,087.8	8	37.5	1,061.0	11	52.5	90.79	5	22.5
1959-60	1,789.9	9	42.5	1,175.7	11	52.5	903.3	14	67.5	93.36	6	27.5
1960-61	1,804.0	10	47.5	1,198.3	14	67.5	874.6	17	82.5	100.63	9	42.5
1961-62	2,248.9	15	72.5	1,348.2	18	87.5	877.0	16	77.5	126.84	16	77.5
1962-63	2,530.5	19	92.5	1,447.4	20	97.5	705.6	20	97.5	135.50	18	87.5
1963-64	1,953.6	13	62.5	1,076.6	7	32.5	1,453.5	5	22.5	108.47	11	52.5
1964-65	2,807.5	20	97.5	1,343.6	17	82.5	1,129.3	9	42.5	169.78	20	97.5
1955-66	1,507.7	2	7.5	998.1	3	12.5	1,703.9	2	7.5	90.58	4	17.5
1966-67	2,381.2	18	87.5	1,231.2	16	77.5	1,061.6	10	47.5	112.26	12	57.5
1967-68	1,430.9	1	2.5	1,027.2	4	17.5	1,562.1	3	12.5	95.51	8	37.5
1968-69	1,690.6	6	27.5	945.4	2	7.5	1,766.9	1	2.5	81.44	3	12.5
1969-70	1,713.8	7	32.5	1,182.1	13	62.5	1,250.8	6	27.5	94.46	7	32.5
1970-71	2,252.7	16	77.5	1,389.4	19	92.5	761.9	19	92.5	140.07	19	92.5
1971-72	2,141.5	14	67.5	1,226.9	15	72.5	768.5	18	87.5	125.98	14	67.5
1972-73	1,625.3	4	17.5	912.4	1	2.5	1,533.9	4	17.5	80.76	2	7.5
1973-74	2,282.2	17	82.5	1,141.2	9	42.5	917.6	13	62.5	126.78	15	72.5
1974-75	1,934.2	12	57.5	1,172.9	10	47.5	880.5	15	72.5	125.08	13	62.5
1975-76	1,768.0	8	37.5	1,179.0	12	57.5	973.4	12	57.5	106.88	10	47.5
1976-77	1,606.9	3	12.5	1,039.7	5	22.5	1,157.3	7	32.5	-	-	-

FIGURE 4B-3 PROBABILITY ANALYSIS OF DROUGHT



(f) The upper areas have been given the first priority to be supplied the irrigation water, though the intake water for the commual rehabilitation area is limited within the available discharge at the communal diversion weir site;

(g) As for the trans-basin plan, the trans-basin discharge has been computed beforehand and added to the Pamacsalan discharge from its own catchment area. The total discharge of the both has been considered as the inflow discharge for the Pamacsalan reservoir.

(h) Overflow discharge from dam has been utilized for hydro-power generation within the limitation of the maximum discharge for hydro-power.

(i) The required storage capacity has been decided on the basis of 10-year return period of drought condition. Therefore, the selected reservoir capacity has not any water shortage in the condition of 10-year return period.

(j) In order to maximize the attainable irrigation acreage and/or hydro-power generation, the carry over condition of the reservoir has been allowed upto 3 years.

6. Drought Year

In order to find severe drought years in the analyzed 21 years from 1956 to 1976, probability analysis of total rainfall during irrigation season, effective rainfall for irrigation and discharge volume at Malinao diversion dam site have been performed as shown in Table 4B - 1 and illustrated in Figure 4B - 3.

According to the analysis, the 10-year return period of drough year varies in the above mentioned factors. The results of the drought analysis on each return period and its corresponding year are shown in the following table.

	Probable Drought Year			
Return Period	20-year	10-year	5-year	2-year
Total Rainfall (mm)	1,450	1,510	1,660	1,905
Corresponding year	1967-67	1965-66	1958-59	1957-58
Effective Rainfall (mm)	940	990	1,020	1,140
Corresponding year	1968-69	1965-66	1967-68	1973-74
Diversion Water Requirement (mm)	1,670	1,500	1,340	1,080
Corresponding year	1967-68	1972-73	1969-70	1966-67
Discharge Volume at Malinao Site (MCM)	74.0	80.0	88.0	105.0
Corresponding year	1957-58	1972-73	1965-66	1975-76

From the table, the year of 1967-68 to 1968-69 can be said the continuous drought year. The same characteristic has been appeared in the long term stochastic trend analysis. Also in the diversion water requirement, the 5-year corresponding year is 1969-70, so these four years from 1967 to 1970 can be said four year successive drought year. The carry over condition of the reservoir will be affected in these years.

As for the 10-year return period, 1965-66 or 1972-73 is correspond to that of drought year. The required storage capacity will be decided in the condition of 1965-66 or 1972-73.

7. Irrigation Acreage for Wahig Communal Area

As for the upper area located between the proposed Pamacsalan or Wahig dams and Malinao diversion dam, the existing communal irrigation area is 130 hectares and 244 hectares for Pamacsalan upper area and Wahig upper area respectively. The Pamacsalan upper area consists of 130 hectares communal and 9 hectares rainfed area. The total available land is 139 hectares. The Wahig upper area consists of 244 hectares communal and 20 hectares of existing pump irrigation area. The total available land is 264 hectares but the existing grassland of 224 hectares can be converted into arable land if irrigation water can be supplied.

As for the Pamacsalan upper area, the total available land can be irrigated by diversion dam only. Considering the reduction of acreage by irrigation canal, drainage canal and farm road, the available net irrigation acreage has been estimated at 136 hectares. However, out of 136 hectares, 16 hectares will be submarged by the Malinao diversion dam with storage function. Therefore, the available land of Pamacsalan upper area is decided at 120 hectares when Malinao diversion dam with storage function is constructed.

In the Case I-A-1, I-B-1, II-A-2, II-B-2 and II-B-3, the Wahig dam is not proposed, so the Wahig upper area can not be served by reservoir water. Accordingly, these cases of Wahig upper area is supplied the irrigation water by diversion dam only. The possible irrigable area by diversion dam only has been estimated by the probability analysis as shown in Table 4B-2, and Figure 4B-4.

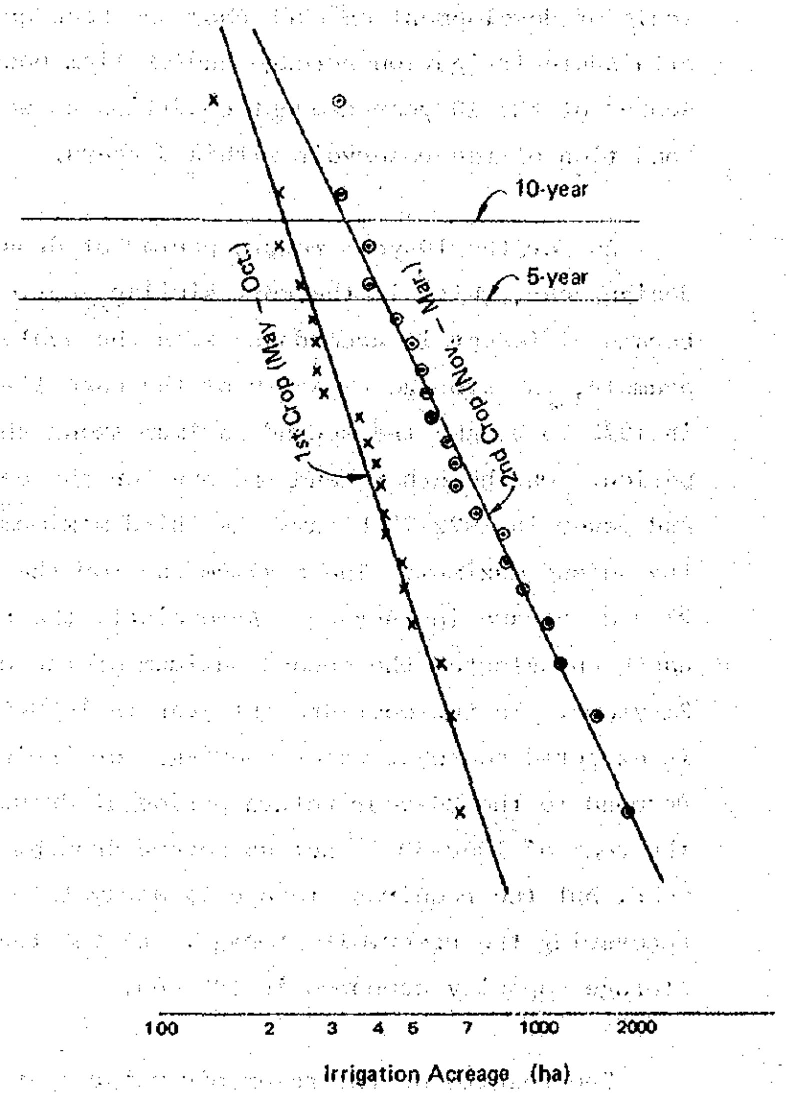
As the irrigation area is served by diversion dam only, 5-year return period of drought condition has been adopted to decide the possible irrigation acreage. As the results of probability analysis, the first crop is 250 hectares and the second crop is 400 hectares in the condition of 5-year return period. Considering the reduction acreage by construction of irrigation and drainage canal, the available land has been estimated at 256 hectares. Therefore, the first crop has been reviced to 256 hectares. The maximum available land in the Wahig upper area, when the Wahig dam is proposed, has been estimated at 482 hectares. However, out of 482 hectares, 3 hectares will be submarged by the Malinao diverison dam with the storage function. So, 472 hectares has been decided in case of Malinao diversion dam with storage has been proposed.

Table 4B-2 Possible Irrigation Acreage on Wahig Upper Area by Diversion Dam Only

	Second Crop (Nov. - Mar.)				First Crop (May - Oct.)			
Year	Critical Period	Possible Irrigation Acreage (ha)	Order	Probability (%)	Critical Period	Possible Irrigation Acreage (ha)	Order	Probability (%)
1956-57	11/1 1/	308.1	2	7.5	5/3 1/	395.3	12	57.5
57-58	11/3	475.9	6	27.5	9/1	461.6	16	77.5
58-59	12/3	432.6	5	22.5	6/2	345.8	9	42.5
59-60	11/2	628.0	11	52.5	8/3	414.4	14	67.5
60-61	3/1	887.1	15	72.5	6/3	208.6	3	12.5
61-62	1/3	859.8	14	67.5	5/3	587.2	18	87.5
62-63	2/1	979.9	16	77.5	6/2	283.0	8	37.5
63-64	12/1	507.8	7	32.5	8/2	385.6	11	52.5
64-65	10/3	1,158.9	17	82.5	8/1	407.8	13	62.5
65-66	2/1	363.2	4	17.5	6/3	255.6	5	22.5
66-67	11/3	720.7	13	62.5	6/2	271.3	7	32.5
67-68	12/1	544.2	9	42.5	9/2	238.7	4	17.5
68-69	2/3	301.1	1	2.5	8/1	207.1	2	7.5
69-70	1/2	599.8	10	47.5	5/3	260.2	6	27.5
70-71	12/3	1,568.7	19	92.5	5/3	659.4	20	97.5
71-72	12/1	1,993.3	20	97.5	8/2	481.6	17	82.5
72-73	2/1	362.1	3	12.5	5/3	134.7	1	2.5
73-74	10/3	524.9	8	37.5	7/1	626.7	19	92.5
74-75	11/3	1,267.9	18	87.5	5/3	456.1	15	72.5
75-76	11/3	634.7	12	57.5	6/1	367.8	10	47.5

Note: 1/ critical period of 11/1 means that November the 1st 10-day.

FIGURE 4B-4 PROBABILITY ANALYSIS ON IRRIGABLE AREA FOR WAHIG COMMUNAL AREA (Diversion dam only)



C. Results of the Alternative Studies

The each case of the alternatives has been studied by varying the irrigation acreage and/or firm power discharge to find the maximum attainable scale of development within the scope of the operation rules. According to the study of the reservoir water operation, water costs for several different irrigation acreages and their required reservoir capacity have been compared. The obtained water costs indicate that the bigger scale of development become the lower water cost in the reservoir. Therefore, the selection of the optimal scale of development on each case has been considered the maximum attainable irrigation acreage and/or firm power discharge within the scopes of the 10-year drought condition as well as the carry over condition of the reservoir within 3 years.

As for the 10-year return period of drought condition, the design year, which is the most similar year of the 10-year drought, became different in accordance with the scale of development. For example, the storage capacity of the case II-A-2 for irrigation only in 1972-73 became the second maximum among the 21-years of the study period. On the other hand, the one for the case II-B-2 for irrigation and power in 1972-73 became the third maximum and in 1965-66 became the second maximum. The maximum one for the above both cases during 21-year became in 1968-69. Accordingly the required storage capacity has been selected the second maximum of the capacity in the analyzed 21 years. In the most drought year in 1968-69, the reservoir capacity is expected somewhat water shortage for irrigation, which year correspond to the 20-year return period of drought condition. However, the year of 1968-69 is not so severe drought condition on the single year, but the required storage is accumulated since 1967 without recovering the reservoir storage. As the results the highest required storage capacity appeared in 1968-69.

The results of the reservoir water operation studies in the selected scale of development in the 14 cases have been summarized in the Table 4B-3.

Table 4B-3 Results of Alternative Case St

Case			Case I (Single Reservoir Plan)				Case II (Multi-			
Purpose			Irrigation Only		Irrigation and Power		Irrigation Only			
Case Number			I-A-1	I-A-2	I-B-1	I-B-2	II-A-1		II-A-2	
Reservoir Name			Pamacsalan	Wahig	Pamacsalan	Wahig	Pamacsalan	Wahig	Pamacsalan	Malinao
Configuration of the Alternatives	Irrigation Acreage	Communal Area (ha)	400	136	400	136	–	–	400	–
		Upper Area (ha)	136	482	136	482	136	482	120	–
		Lower Area (ha)	4,040	3,460	3,800	3,300	3,462	1,338	4,800	–
		Sub-total	4,576	4,078	4,336	3,918	3,598	1,820	5,320	–
		Total Irrigation Area (ha)	4,576	4,078	4,336	3,918	5,418		5,320	
	Reservoir	Effective Storage (x1,000 m^3)	27,030	22,230	27,030	22,230	24,280	13,400	28,380	3,300
		Total Storage (x 1,000 m^3)	28,650	23,150	28,650	23,150	25,400	14,320	29,500	3,426
		Full Water Surface Elevation (m)	246.5	292.5	246.5	292.5	243.5	284.1	246.9	152.0
		Dam Crest Elevation (m)	249.5	295.5	249.5	295.5	246.5	287.1	249.9	154.5
		Height of Dam (m)	56.5	61.5	56.5	61.5	53.5	53.1	56.9	16.9
	Hydro-power Generation	Max. Power Discharge (m^3/sec)			2.0	1.5				
		Firm Power Discharge (m^3/sec)1/			0.2	0.2				
		Firm Peak Power Discharge (m^3/sec)2/								
		Installed Capacity (KW)			1,100	1,950				
		Firm Power Capacity (KW)1/			100	95				
		Firm Peak Power Capacity (KW)2/								
		Dependable Capacity (KW)3/			505.1	383.1				
		Annual Energy Production (MWH)			4,434.8	3,368.1				
		Plant Factor (%)			46.5%	41.0%				
Economic Evaluation	Over-all	Total Construction Cost (10^3\$)4/	21,655	22,329	22,973	23,735	31,722		24,829	
		Overall Internal Rate of Return	16.1%	14.0%	14.9%	12.8%	13.9%		16.9%	
		Allocated Cost for Power (10^3\$)5/	—	—	500	570				
	Irrigation	Construction Cost for Irrigation (10^3\$)	21,655	22,329	20,713	21,435	31,722		24,830	
		Construction Cost per Hectare (\$/ha)	4,732	5,475	4,777	5,471	5,855		4,667	
	Power	Construction Cost for Power (10^3\$)	—	—	2,260	2,300				
		Construction Cost per KW (\$/KW)6/	—	—	4,474	6,004				
		Construction Cost per KWH (\$/KWH)	—	—	0.510	0.683				
		B/C of Power7/	—	—	0.462	0.256				

Note: 1/ When an after bay is not available in the downstream of main dam, peak power generation will not be possible, therefore, firm power generation for 24 hours has been adopted considering the irrigation water supply.

2/ When an after by is available in the downstream of main dam, peak power generation is possible, therefore, firm peak power generation for 6 hours has been adopted.

3/ The average of generated capacity on 10-day basis for analyzed 20 years has been considered as dependable capacity.

4/ The construction cost is not considered the price esculation but included a contingency.

5/ The allocated cost has been preliminaryly estimated on the basis of separable cost of main dam only. However, in case of an after bay is available, the storage capacity for daily peak power generation has been allocated in the percentage of the total effective storage capacity of after bay.

6/ Construction cost per kilowatt has been estimated on the basis of dependable capacity.

7/ The discount rate of 8.3% has been adopted to estimate the B/C of hydro-power.

-3 Results of Alternative Case Studies on Optimal Scale of Development

	Case II (Multi-Reservoir Plan)									Case III (				
	Irrigation Only			Irrigation and Power						Irrigation Only				
		II-A-2		II-B-1		II-B-2		II-B-3		III-A-1		III-A-2		
	Wahig	Pamacsalan	Malinao	Pamacsalan	Wahig	Pamacsalan	Malinao	Pamacsalan	Bagun-an	Pamacsalan	Wahig	Pamacsalan	Wahig	Malinao
	—	400	—	—	—	400	—	400	—	—	—	—	—	—
	482	120	—	136	482	120	—	136	—	136	482	120	479	—
	1,338	4,800	—	3,462	1,338	4,800	—	4,300	—	4,800	—	4,800	—	—
	1,820	5,320	—	3,598	1,820	5,320	—	4,836	—	4,936	482	4,920	479	—
8		5,320		5,418		5,320		4,836		5,418			5,399	
	13,400	28,380	3,300	24,800	13,880	30,180	3,365	28,880	3,000	35,080	1,880	26,480	1,880	3,365
	14,320	29,500	3,426	25,920	14,800	31,300	3,426	30,000	3,500	36,200	2,800	27,600	2,800	3,426
	284.1	246.9	152.0	244.0	284.7	248.5	152.0	247.5	177.0	252.3	265.5	245.8	265.5	152.0
	287.1	249.9	154.5	247.0	287.7	251.5	154.5	250.5	180.0	255.3	268.5	248.8	268.5	154.5
	53.1	56.9	16.9	54.0	53.7	58.5	16.9	57.5	12.0	62.3	34.5	55.8	34.5	16.9
				1.0	1.0	3.0		2.5						
				0.10	0.1	0.4		(0.3)						
						1.6		1.2						
				550	450	1,700		1,600						
				55	45									
						850		750						
				326	274	1,225		1,020						
				2,856.5	2,402.4	5,175.0		4,765.4						
				59.3%	60.9%	36.2%		34.0%						
2		24,829		35,062		27,592		27,855		33,585			33,989	
3.9%		16.9%		13.7%		15.9%		14.2%		13.4%			13.6%	
				150		500		500		—			—	
2		24,830		31,722		24,792		25,225		33,585			33,989	
5		4,667		5,855		4,660		5,216		6,199			6,295	
				3,340		2,800		2,630		—			—	
				10,245		2,286		2,348		—			—	
					0.635		0.541		0.521	—			—	
					0.302		0.640		0.576	—			—	

sible, therefore,

re, firm peak

le capacity.

owever, in case
n the percentage

Case III (Trans-basin Plan)

Irrigation Only					Irrigation and Power							
III-A-1		III-A-2			III-B-1		III-B-2			III-B-3		
Pamacsalan	Wahig	Pamacsalan	Wahig	Malinao	Pamacsalan	Wahig	Pamacsalan	Wahig	Malinao	Pamacsalan	Wahig	Bagun-an
—	—	—	—	—	—	—	—	—	—	—	—	—
136	482	120	479	—	136	482	120	479	—	136	482	—
4,800	—	4,800	—	—	4,800	—	4,800		—	4,800	—	—
4,936	482	4,920	479	—	4,936	482	4,920	479	—	4,936	482	—
5,418		5,399			5,418		5,399			5,418		—
35,080	1,880	26,480	1,880	3,365	47,280	1,880	35,480	1,880	3,365	31,880	1,880	3,000
36,200	2,800	27,600	2,800	3,426	48,400	2,800	35,600	2,800	3,426	33,000	2,800	3,500
252.3	265.5	245.8	265.5	152.0	260.3	265.5	251.8	265.5	152.0	250.0	265.5	177.0
255.3	268.5	248.8	268.5	154.5	263.3	268.5	254.8	268.5	154.5	253.0	268.5	180.0
62.3	34.5	55.8	34.5	16.9	70.3	34.5	61.8	34.5	16.9	60.0	34.5	12.0
					3.0		4.0			4.0		
					0.3		(1.4)			(0.8)		
							4.0(8hour)			2.4(8hour)		
					2,000		2,400			2,400		
							-					
					200		2,400			1,200		
					1,012.5		2,400.0			1,652.0		
					8,884.5		9,468.3			9,460.8		
					53.1%		45.9%			45.0%		
33,585		33,989			38,105		39,879			39,639		
13.4%		13.6%			13.1%		13.2%			13.0%		
—		—			2,110		2,810			2,810		
33,585		33,989			33,585		33,889			33,649		
6,199		6,295			6,199		6,277			6,211		
—		—			4,520		5,990			5,990		
—		—			4,464		2,496			3,626		
—		—			0.509		0.633			0,633		
—		—			0.463		0.523			0.324		

As seen from the Table 4B-3, the followings can be said;

(a) The Pamacsalan reservoir has the highest potentiality among the proposed reservoirs to develop the irrigation acreage as well as the possible storage capacity.

(b) The multi-reservoir case of II-A-2 has the highest internal rate of return (hereinafter IRR) of 16.9 percent.

(c) The Case I-A-1 can emulate to the Case II-A-2, but the Case I-A-2 can not satisfy the total available irrigation acreage in the lower area. Moreover in the cases of with power, the Case I-B-1, which is correspond to the Case I-A-1 with power, has low applicability of power due to without after bay.

(d) As for the irrigation and power cases, the Case II-B-2 has the highest IRR of 15.9 percent and also the potential acreage of the lower area can be supplied sufficient irrigation water at the same time. Also the construction cost per hectare becames the lowest among the alternatives. The B/C ratio of power shows the highest compared with other cases, though the ratio indicates that the power scheme can not be justified from the economical point of view.

(e) Compared with the Case II-A-2 and Case II-A-1, the required storage capacity of the Pamacsalan dam is almost same in the both cases, but the Case II-A-2 can expand the irrigation acreage because of the storage function of Malinao diversion dam. Therefore, it can be said that the difference acreage of 744 hectares between the cases can be expanded by the Malinao diversion dam.

(f) As for the trans-basin cases, the total available land can be supplied enough irrigation water. In order to find the maximum potentiality of the water resource development, the firm power discharge has been expanded as much as possible. As the result, the allocated cost for power in the dam construction cost has also increased. The obtained B/C ratio of power could not be increased in spite of the increasing of power capacity and annual energy production.

As for the hydro-power generation scheme, the increasing of the firm power discharge of 0.1 cu.m/sec decreases about 100 hectares of irrigation acreage as comparing with the Case I-A-1, I-A-2 and I-B-1, II-B-2. The benefit from the irrigation of the 100 hectares corresponds to about 5 times of the benefit of hydro-power generated by 0.1 cu.m/sec. It can be understood that the IRR of irrigation only case becomes higher than the irrigation and power case. The hydro-power generation decreases the IRR. However, the necessity of hydro-power generation can not be neglected by IRR only, because the hydr-power generation can be greatly contribute to the rural electrification and social benefit. These intangible benefit has not been included in the economic analysis. Moreover, the water use in this project, even the irrigation only, will affect to the other project located in the downstream, such as the downstream of the Wahig river hydro-power generation scheme. Therefore, it will be inevitable to include the hydro-power scheme in this project and also it will be better to include the scheme from the view point of the compound utilization of limited water resource.

Consequently, the Case II-B-2 has been selected as the optimal scale of development to be proposed for the implementation of the project.

Proposed Scheme of Development

The Case II-B-2 has been selected to be implemented for the project as the most optimal scale of development. The Pamacsalan reservoir and the Malinao diversion dam with storage function can guarantee the sufficient irrigation water supply for 5,320 hectares as well as the hydro-power generation of 1,700 KW in the installed capacity.

The configuration of the project and the allocation of storage water are shown as below.

Pamacsalan Dam:

Catchment Area:	28.0 sq.km
Total effective storage capacity:	30,180,000 cu.m
out of which for irrigation:	28,380,000 cu.m
for power:	1,800,000 cu.m
Sedimentation capacity (100 year):	1,120,000 cu.m
Total storage capacity:	31,300,000 cu.m
Full water surface area:	1,260,000 sq.m
Full water surface elevation:	EL. 248.5 m
Dead water elevation:	EL. 207.5 m
Dam crest elevation:	EL. 251.5 m
Height of dam from the foundation:	67.5 m

Malinao Diversion Dam:

Catchment Area:	138.8 sq.km
Total effective storage capacity:	3,365,000 cu.m
out of which for irrigation:	3,300,000 cu.m
for power for daily peak regulation:	65,000 cu.m
Sediment capacity1/:	61,000 cu.m
Total storage capacity:	3,426,000 cu.m
Full water surface area:	1,098,000 sq.m
Full water surface elevation:	EL. 152.0 m
Dead water elevation2/:	EL. 143.0 m

Dam crest elevation: EL. 154.5 m

Height of Dam from the foundation: 24.5 m

Note: 1/ As the diversion dam is gated type for the whole width of river cource, the sedimentation can be flushed out to the downstream. The sediment capacity has been estimated so that the dead water capacity is below the bottom of intake.

2/ Dead water elevation is set at the bottom of intake elevation.

Irrigation Area:

Lower Area: 4,800 hectares

Pamacsalan upper area: 120 hectares

Wahig communal rehabilitation area
for 1st crop (May to October): 256 hectares
for 2nd crop (November to March): 400 hectares

Total Irrigation Acreage: 5,320 hectares

Hydro-power Generation:

Installed capacity: 1,700 KW

Installed unit: 850KW x 2units

Firm peak power capacity: 850 KW

Dependable capacity: 1,225 KW

Annual energy production: 5,175 MWH

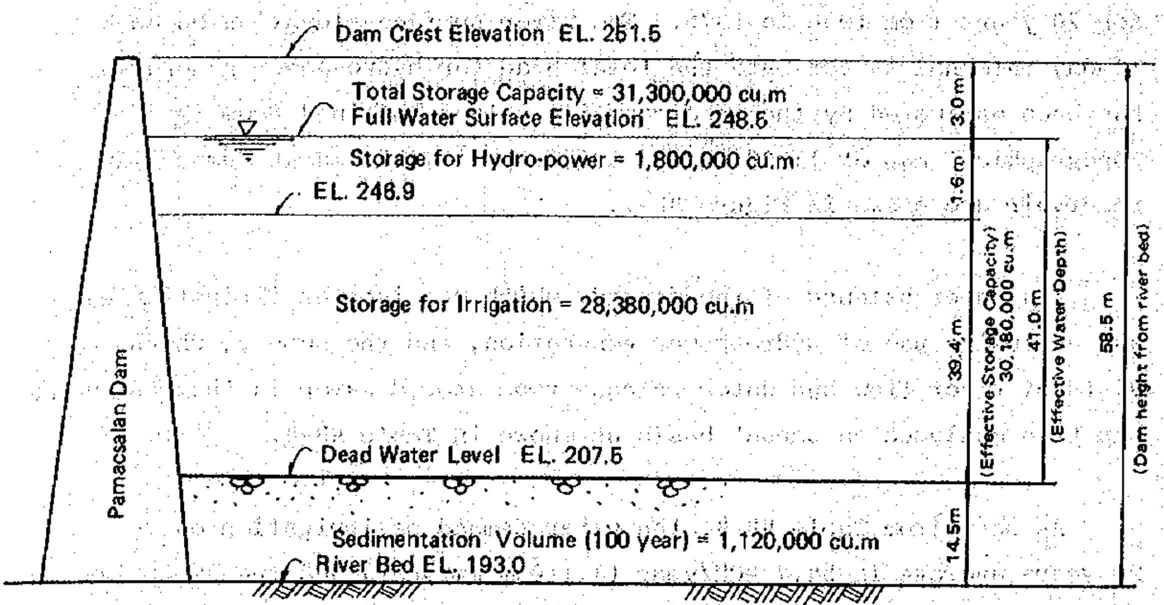
Plant factor: 36.2 %

Transmission line to carmen: 69 KV

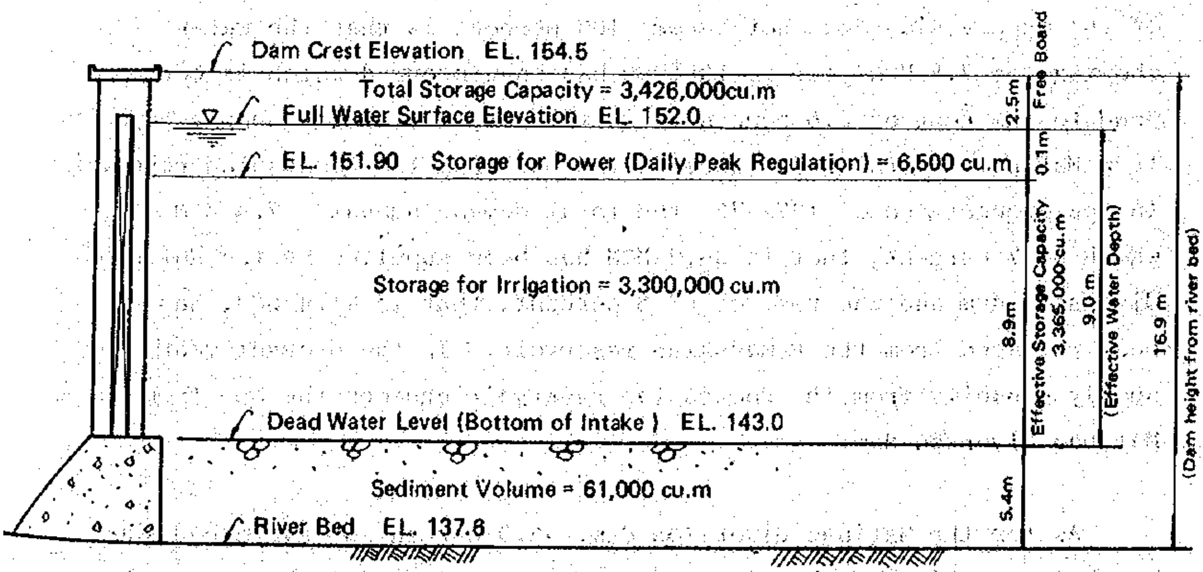
The allocation of storage capacity are illustrated in Figure 4B-5.

FIGURE 4B-5 ALLOCATION OF STORAGE CAPACITY

Damacsalan Reservoir



Malinao Diversion Dam



Reservoir Plan

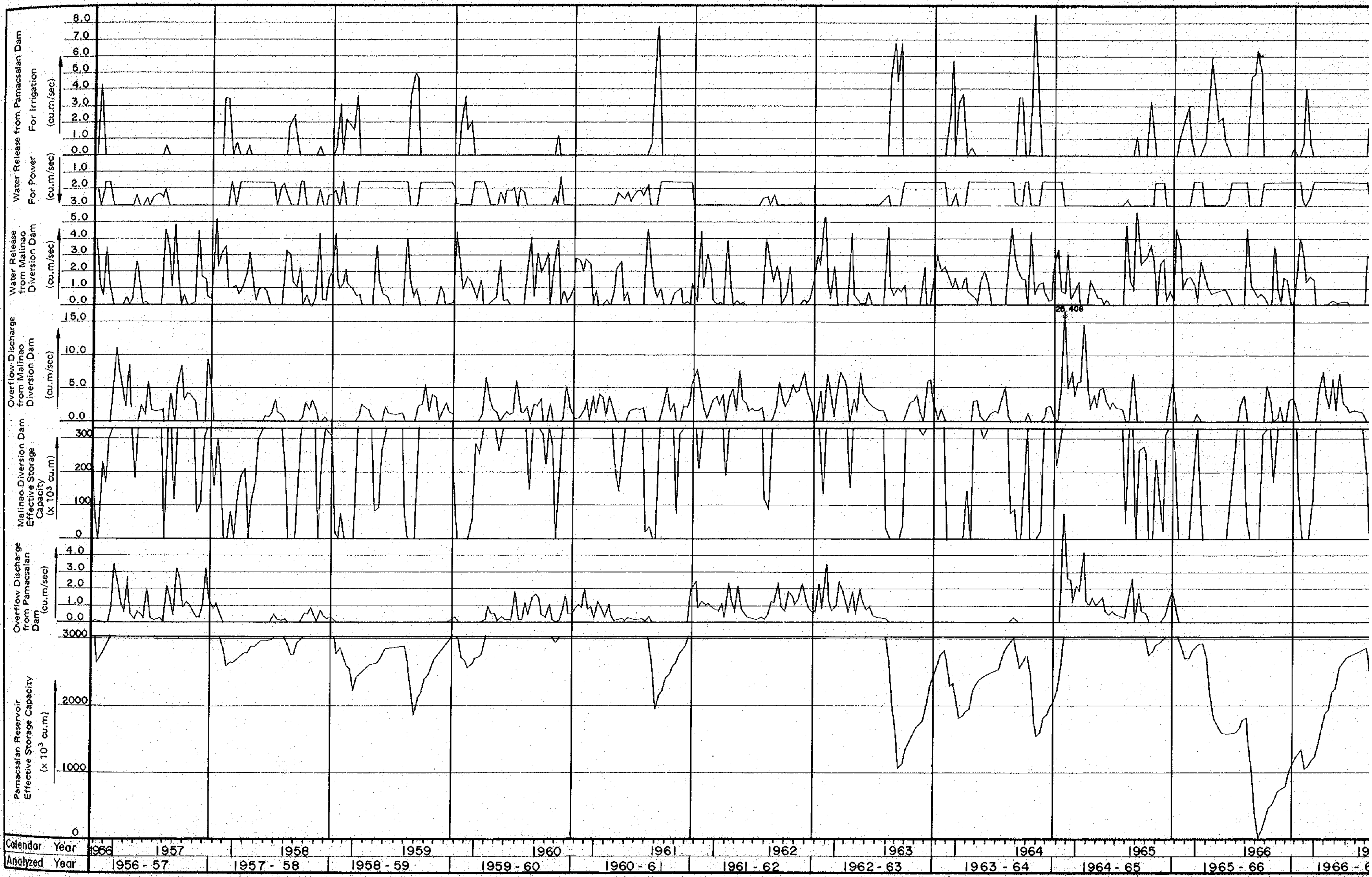
The results of reservoir water operation studies on the proposed scheme of development are illustrated in Figure 4B-6 on 10-day internal for 20 years from 1956 to 1975. The water surface elevation on each 10-day internal to estimate the total head for hydro-power generation has been estimated by the area-capacity curves obtained from the topographical map of 1:4000. The area-capacity curves at Pamacsalan reservoir are shown in Figure 4B-7.

The water balance of the demand, which consists of irrigation water and exclusive use of hydro-power generation, and the supply, which consists of river flow and water release from stored water in the reservoir, has been analyzed on annual basis as shown in Table 4B-4.

As seen from Table 4B-4, the water demand of irrigation on the 20 years average is 54.7 MCM/year (1,110.9 mm/year) and the exclusive use of water for hydro-power generation is 9.7 MCM/year. The average total demand, therefore, is 64.3 MCM/year. On the other hand, 59.3 percent of total demand, that is 38.1 MCM/year, has been supplied by the Malinao diversion dam, and 40.1 percent of that, that is 25.8 MCM/year, has been supplied by the Pamacsalan reservoir. The reasons of the supply side dose not become 100 percent is that the water shortage of 7.8 MCM/year in 1968-69 has been occured which is correspond to the rest of 0.6 percent. On the normal year the supply capacity from Malinao diversion dam exceeds the one from the Pamacsalan reservoir. In the drought year of 1972-73, the total demand amounts 77.6 MCM of which 46.7 percent, that is 36.2 MCM has been supplied by the Malinao diversion dam and the rest of 53.3 percent, that is 41.4 MCM, has been released from the Pamacsalan reservoir. In the drought year, the supply capacity from the Pamacsalan reservoir exceeds the one from the Malinao diverson dam.

As for the Malinao diversion dam, 43.9 percent of average demand has been supplied by the river flow without storaged in the dam and 15.9 percent of the demand has been release from the stored water.

FIGURE 4B-6 RESULT OF RESERVOIR WATER OPERATION STUDY (Pamacsalan Reservoir and Malinao Diversion Dam,



and Malinao Diversion Dam, Irrigation and Hydro-power)

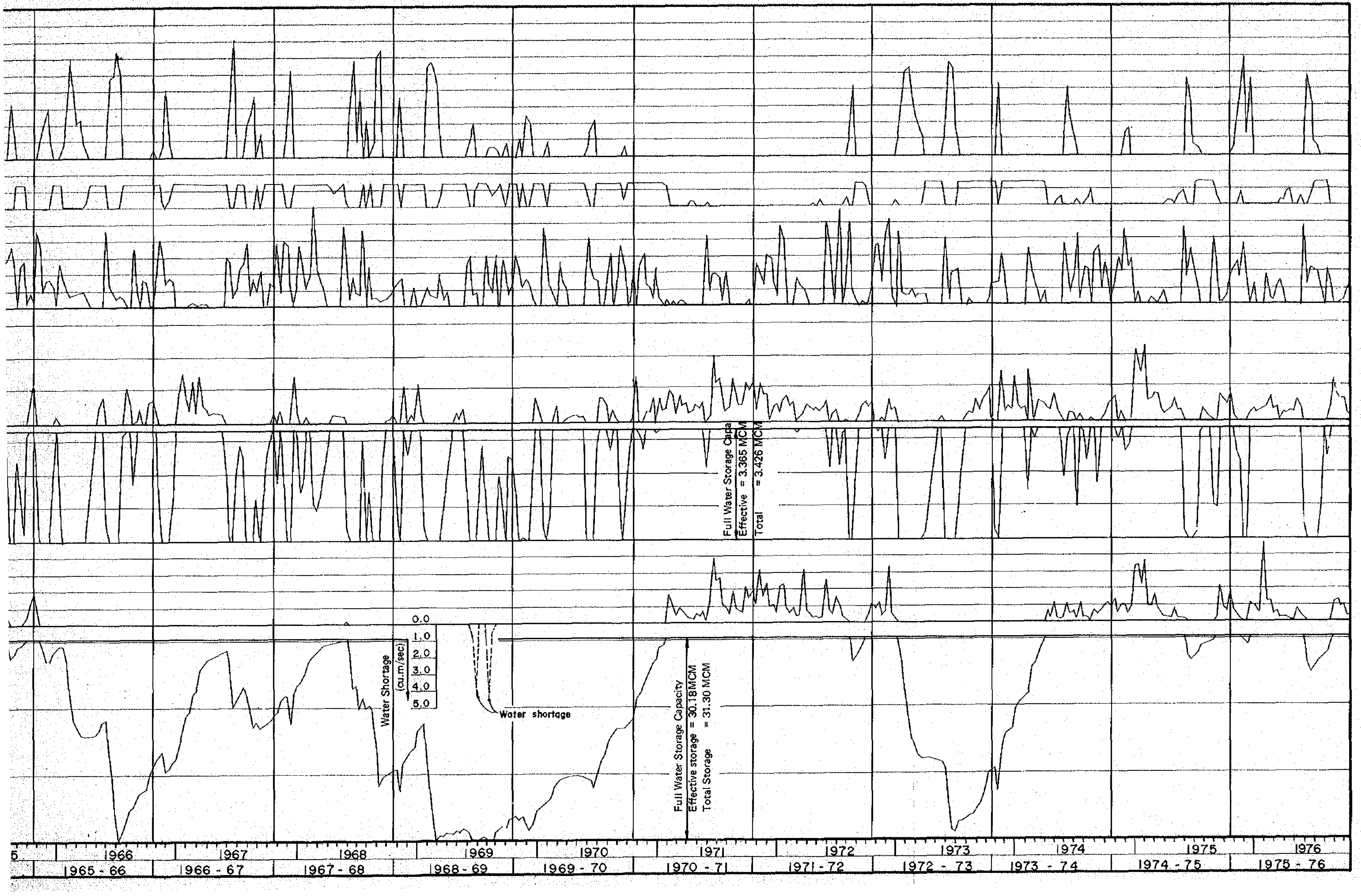
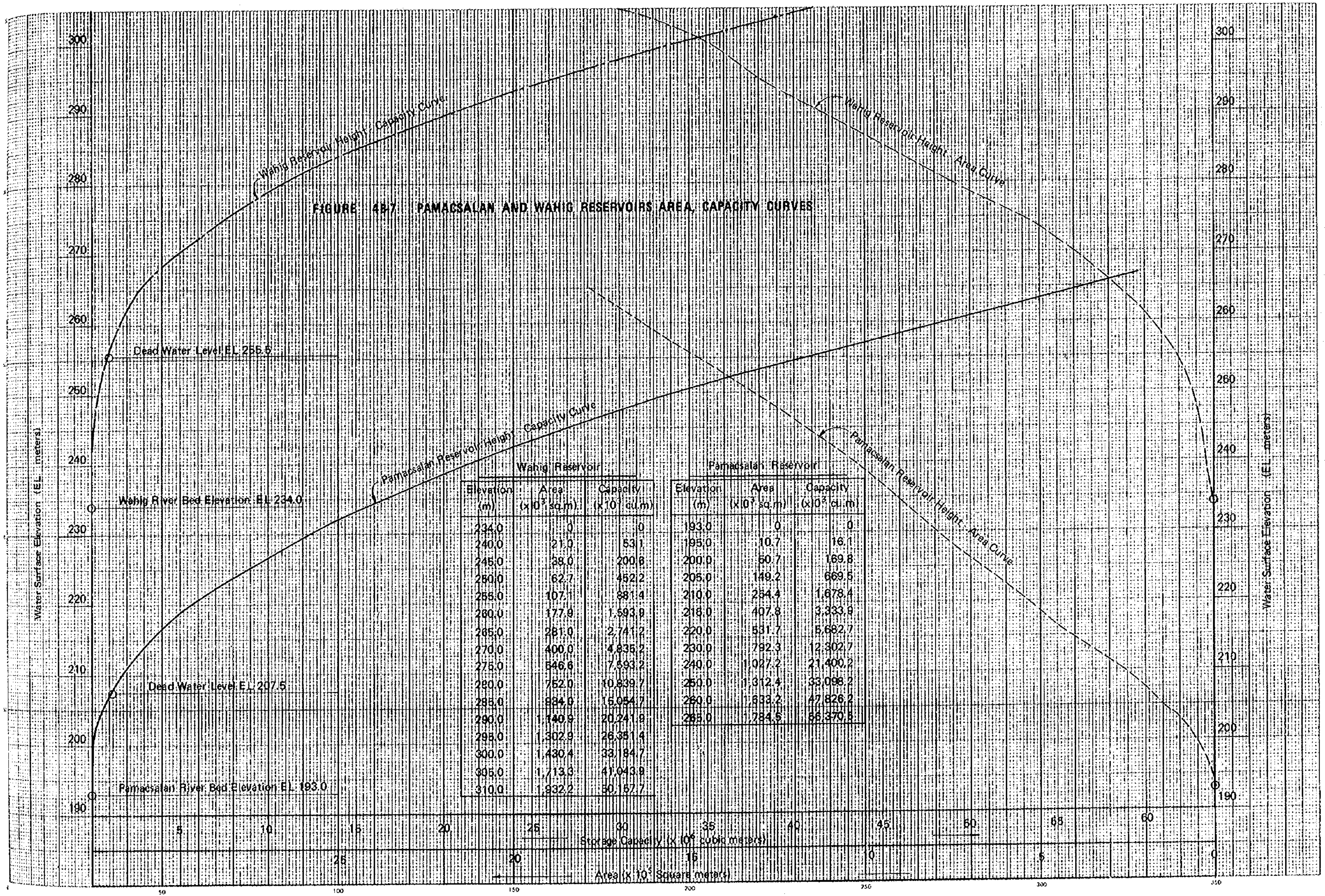


FIGURE 4B-7 PAMACSALAN AND WAHIG RESERVOIRS AREA, CAPACITY CURVES



Wahig Reservoir

Elevation (m)	Area (x10³ sq.m)	Capacity (X10³ cu.m)
234.0	0	0
240.0	21.0	53.1
245.0	38.0	200.6
250.0	62.7	452.2
255.0	107.1	881.4
260.0	177.9	1,593.9
265.0	281.0	2,741.2
270.0	400.0	4,835.2
275.0	546.6	7,593.2
280.0	752.0	10,839.7
285.0	884.0	15,054.7
290.0	1,140.9	20,241.9
295.0	1,302.9	26,351.4
300.0	1,430.4	33,184.7
305.0	1,713.3	41,043.9
310.0	1,932.2	50,157.7

Pamacsalan Reservoir

Elevation (m)	Area (x10³ sq.m)	Capacity (x10³ cu.m)
193.0	0	0
195.0	10.7	16.1
200.0	50.7	169.8
205.0	149.2	669.5
210.0	254.4	1,678.4
215.0	407.8	3,333.9
220.0	531.7	5,682.7
230.0	792.3	12,302.7
240.0	1,027.2	21,400.2
250.0	1,312.4	33,098.2
260.0	1,633.2	47,826.2
265.0	1,784.5	56,370.5

Table 4B-4 Water Balance of Demand and Supply

	Demand				Water Supply								
					Malinao Diversion Dam				Pamacsalan Reservoir				
Year	Upper Area	Lower Area	Hydro-Power Exclusive	Total Demand Volume	Inflow Through Reservoir	Water Re-lease from Storage	Sub-total	Repeaded Use (Times/year)	Inflow Through Reservoir	Water Re-lease from Storage	Sub-total	Repeaded Use (Times/year)	Total Inflow Discharge
1956-57	998.9	39,557.2	11,132.6	51,678.7	22,878.0	12,692.0	35,570.0	3,846	12,445.8	3,662.9	16,108.7	0.012	48,528.2
1957-58	1,395.4	55,815.8	9,135,3	66,346.5	30,562.6	14,262.2	44,824.8	4.322	14,264.9	7,256.8	21,521.7	0.240	27,942.5
1958-59	1,200.3	48,012.9	9,073.5	58,286.7	16,623.6	9,622.9	26,246.5	2,916	12,776.0	19,264.2	32,040.2	0.638	33,184.4
1959-60	1,187.7	47,509.8	10,139.8	58,837.3	28,007.1	11,081.5	39,088.6	3,358	14,116.0	5,632.7	19,748.7	0.187	34,124.5
1960-61	1,105.0	44,200.4	10,713.9	56,019.3	24,857.7	8,375.5	33,233.2	2,538	12,188.3	10,597.7	22,786.1	0.351	36,778.7
1961-62	762.8	30,511.7	11,851.6	43,126.1	25,194.3	5,317.5	30,511.8	1.611	12,614.3	0.0	12,614.3	0.0	46,360.4
1962-63	1,260.0	50,398.4	10,371.0	62,029.4	23,413.9	7,536.1	30,950.0	2,284	11,874.3	19,205.1	31,079.4	0.636	49,525.3
1963-64	1,974.9	78,994.3	7,739.2	88,708.4	35,666.1	10,834.9	46,501.0	3.283	14,421.4	27,786.0	42,207.4	0.921	39,644.6
1964-65	1,214.5	48,579.4	10,646.6	60,440.5	31,637.9	12,488.8	44,126.7	3,784	13,817.4	2,496.4	16,313.8	0.083	62,053.3
1965-66	1,991.2	79,649.1	6,555.9	88,196.2	27,673.6	11,573.6	39,247.2	3,507	14,088.5	34,860.5	48,949.0	1.155	33,105.8
1966-67	1,423.1	56,926.0	8,981.9	67,331.0	24,808.7	10,675.2	35,483.9	3.235	13,687.7	18,159.4	31,847.1	0.602	41,030.4
1967-68	2,009.9	80,396.4	8,149.8	90,556.1	31,957.3	17,571.1	49,528.4	5.325	12,891.2	28,136.5	41,027.7	0.932	34,908.1
1968-69	1,825.0	72,998.3	7,532.6	82,355.9	24,043.0	15,925.0	39,968.0	4,826	12,178.2	22,360.5	34,538.7	0.741	29,766.4
1969-70	1,307.4	52,297.8	8,832.5	62,437.7	28,920.4	13,616.7	42,537.1	4,126	15,429.7	4,470.9	19,900.6	0.148	34,526.2
1970-71	602.8	24,113.0	12,011.5	36,727,3	20,809.2	3,303.8	24,113.0	1.001	12,614.3	0.0	12,614.3	0.0	51,195.4
1971-72	1,328.5	53,139.2	10,862.9	65,330.6	42,993.7	5,843.7	48,837.4	1.771	13,041.2	3,452.0	16,493.2	0.114	46,046.8
1972-73	1,702.5	68,099.2	7,775.3	77,577.0	28,340.3	7,888.8	36,229.1	2,391	12,012.1	29,335.8	41,347.9	0.972	29,519.3
1973-74	1,178.7	47,149.4	11,076.2	59,404.3	31,405.0	11,925.5	43,330.5	3,614	12,813.4	3,260.4	16,073.8	0.108	46,337.9
1974-75	1,088.3	43,530.2	10,485.2	55,103.7	29,901.5	7,083.4	36,984.9	2.146	14,773.3	3,345.5	18,118.8	0.111	45,716.7
1975-76	1,114.5	44,579.8	9,975.1	55,669.4	27,800.1	7,072.2	34,872.3	2,143	14,777.4	6,019.7	20,797.1	0.199	39,063.1
Average	1,333.1	53,322.9	9,652.1	64,308.1	28,214.7	10,234.5	38,109.2	3.101	13,563.7	12,465.2	25,806.4	0.413	40,467.9
(%)	2.07	82.91	15.02	100.00	43.87	15.91	59.26		21.09	19.38	40.13		
								Ratio to the River Discharge			63.77%		100.0%

4 Water Balance of Demand and Supply

(Unit: 1,000 cu.m)

Water Supply												
Malinao Diversion Dam				Pamacsalan Reservoir					Malinao Diversion Site		Pamacsalan Reservoir	
Inflow Through Reservoir	Water Re-lease from Storage	Sub-total	Repeaded Use (Times/year)	Inflow Through Reservoir	Water Re-lease from Storage	Sub-total	Repeaded Use (Times/year)	Total Inflow Discharge	Total Water Supply Volume	Total Inflow Volume at Malinao	Water Shortage Volume	Required Storage Capacity
22,878.0	12,692.0	35,570.0	3,846	12,445.8	3,662.9	16,108.7	0.012	48,528.2	51,678.7	132,772.8	0	3,650.0
30,562.6	14,262.2	44,824.8	4.322	14,264.9	7,256.8	21,521.7	0.240	27,942.5	66,346.5	76,450.8	0	4,578.3
16,623.6	9,622.9	26,246.5	2,916	12,776.0	19,264.2	32,040.2	0.638	33,184.4	52,286.7	90,792.4	0	11,876.1
28,007.1	11,081.5	39,088.6	3,358	14,116.0	5,632.7	19,748.7	0.187	34,124.5	58,837.3	93,364.4	0	5,053.7
24,857.7	8,375.5	33,233.2	2,538	12,188.3	10,597.7	22,786.1	0.351	36,778.7	56,019.3	100,626.3	0	10,597.8
25,194.3	5,317.5	30,511.8	1,611	12,614.3	0.0	12,614.3	0.0	46,360.4	43,126.1	126,841.8	0	0
23,413.9	7,536.1	30,950.0	2,284	11,874.3	19,205.1	31,079.4	0.636	49,525.3	62,029.4	135,500.4	0	19,205.1
35,666.1	10,834.9	46,501.0	3.283	14,421.4	27,786.0	42,207.4	0.921	39,644.6	88,708.4	108,467.2	0	15,012.5
31,637.9	12,488.8	44,126.7	3,784	13,817.4	2,496.4	16,313.8	0.083	62,053.3	60,440.5	169,776.8	0	9,639.2
27,673.6	11,573.6	39,247.2	3,507	14,088.5	34,860.5	48,949.0	1.155	33,105.8	88,196.2	90,577.4	0	30,126.5
24,808.7	10,675.2	35,483.9	3.235	13,687.7	18,159.4	31,847.1	0.602	41,030.4	67,331.0	112,258.4	0	19,798.0
31,957.3	17,571.1	49,528.4	5.325	12,891.2	28,136.5	41,027.7	0.932	34,908.1	90,556.1	95,508.2	0	21,884.9
24,043.0	15,925.0	39,968.0	4,826	12,178.2	22,360.5	34,538.7	0.741	29,766.4	74,506.7	81,440.9	7,849.2	30,180.0
28,920.4	13,616.7	42,537.1	4,126	15,429.7	4,470.9	19,900.6	0.148	34,526.2	62,437.7	94,463.4	0	28,744.7
20,809.2	3,303.8	24,113.0	1.001	12,614.3	0.0	12,614.3	0.0	51,195.4	36,727.3	140,070.1	0	9,257.6
42,993.7	5,843.7	48,837.4	1.771	13,041.2	3,452.0	16,493.2	0.114	46,046.8	65,330.6	125,983.4	0	3,452.0
28,340.3	7,888.8	36,229.1	2,391	12,012.1	29,335.8	41,347.9	0.972	29,519.3	77,577.0	80,764.9	0	29,335.9
31,405.0	11,925.5	43,330.5	3.614	12,813.4	3,260.4	16,073.8	0.108	46,337.9	59,404.3	126,780.1	0	23,329.8
29,901.5	7,083.4	36,984.9	2.146	14,773.3	3,345.5	18,118.8	0.111	45,716.7	55,103.7	125,080.3	0	3,345.5
27,800.1	7,072.2	34,872.3	2,143	14,777.4	6,019.7	20,797.1	0.199	39,063.1	55,669.4	106,876.4	0	4,989.6
28,214.7	10,234.5	38,109.2	3.101	13,563.7	12,465.2	25,806.4	0.413	40,467.9	63,915.6	110,719.8	392.5	
43.87	15.91	59.26		21.09	19.38	40.13			99.39		0.61	
			Ratio to the River Discharge			63.77%		100.0%	57.73%	100%		

The average of stored water release amounts 10.2 MCM/year so that the stored water becomes 3.1 times of the its storage capacity of 3.3 MCM. Such a repeated use of the storage capacity makes possible to expand the irrigation acreage. Especially, in 1967-68 the stored water release amounts 17.6 M.M that correspond to 5.3 times of the storage capacity.

As for the Pamacsalan reservoir, the storage capacity is too big compared with its catchment area. As the results, the repeated use of the stored water becomes only 0.4 time/year in average. However, in the drought year in 1965-66, the Pamacsalan reservoir has been released 1.2 time of the storage capacity.

Compared with the original river flow volume of 40.5 M.M (1,445 mm/year) and the utilized river flow of 25.8 MCM in the 20 years average, 63.8 percent of the original river flow has been utilized for irrigation and power at the Pamacsalan dam site. While, the total water utilization at Malinao diversion dam site at 63.9 MCM corresponds to utilized 57.7 percent of the available river stream flow of 110.7 MCM. The ratio of the effective storage capacity of Pamacsalan reservoir, 30.18 MCM (1,078 mm/year), to the river flow becomes 74.6 percent.

The reasons of such high ratio of river flow utilization will be as flowings, i) the Malinao storage capacity has been repeatedly used, ii) the storage capacity of the Pamacsalan reservoir is too big, compared with the catchment area and iii) the carry over storage for about 2 year and 8 month has been allowed.

On the contrary, the utilization ratio of 57.7 percent at the Malinao diversion dam means that the rest of 42.3 percent of total discharge could not be used for the project purposes, that is 46.8 MCM in average of discharge has been over-flowed from the diversion dam. Even the droughtest year in 1968-69, 17.8 MCM and 26.6 MCM in 1972-73 have been over-flowed to the downstream without utilized in the project. There will be still potentiality to develop the other project.

Table 4B-5 Percolation Rate in the Project Area

(unit: cm)

	Location	1st Observation				2nd Observation				Average Percolation Rate
		Start	5 minu.	10 minu.	Rate	Start	5 minu.	10 minu.	Rate	
1.	Barrio Pilar	6.210	6.220		0.01	5.020	5,020		0.00	0.005
2.	Barrio Estaca	5.880	5.920		0.04	3.900		3.95	0.05	0.045
3.	Barrio Katipunan	3.700		3.750	0.05	5.500		5.500	0.00	0.025
4.	Barrio Babag	4.100		4.100	0.00					0.000
5.	Barrio San Miguel	5.400		5.450	0.050					0.050
6.	Barrio Dagohoy	4.350		4.390	0.040					0.040
	Average									0.028

Note: Location of measuring site is given in Figure 4B-8

FIGURE 4B-8 LOCATION OF MEASURING SITE OF PERCOLATION RATE

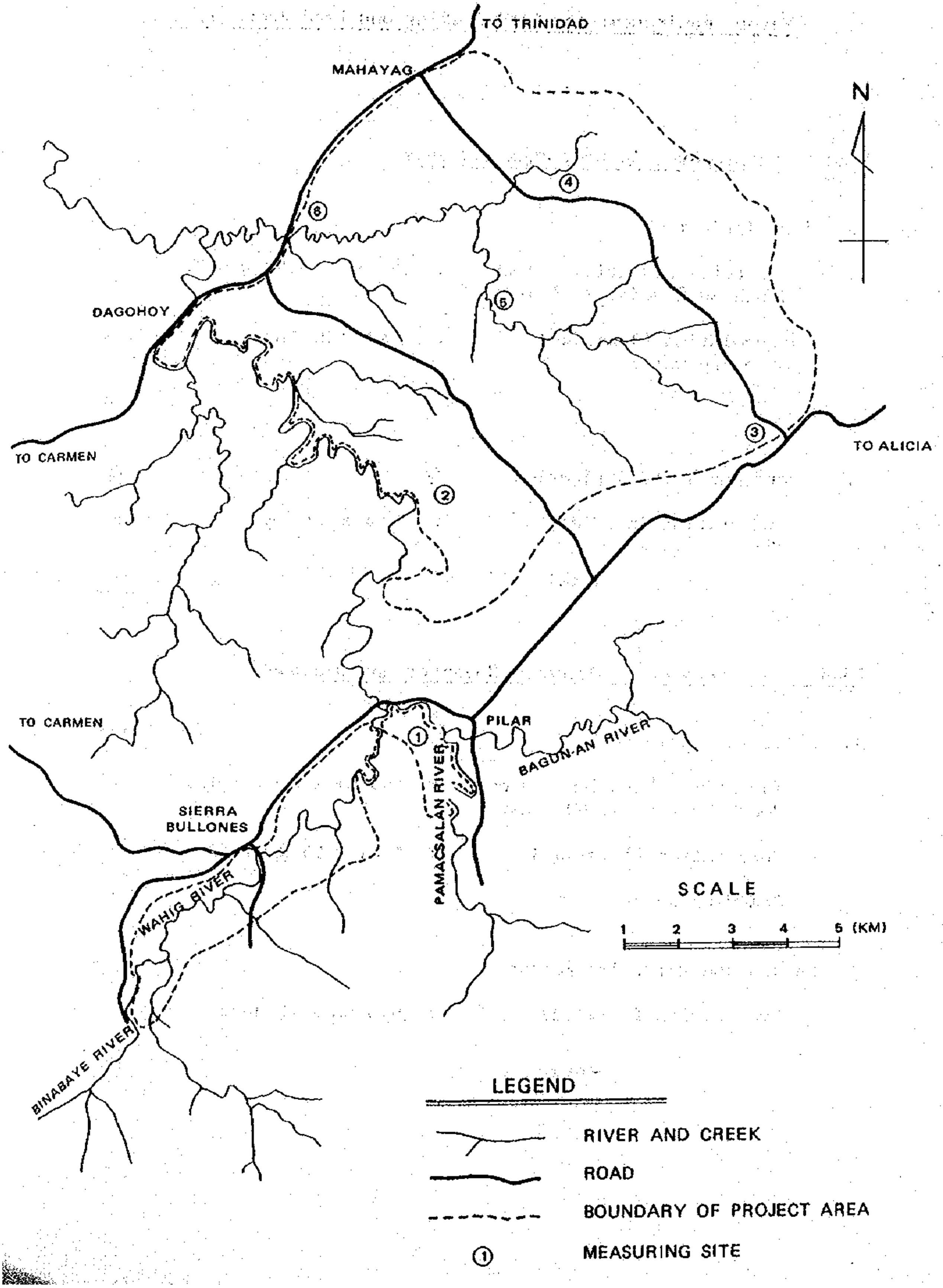


Table 4B-6

Water Requirement for Land Soaking and Land Preparation

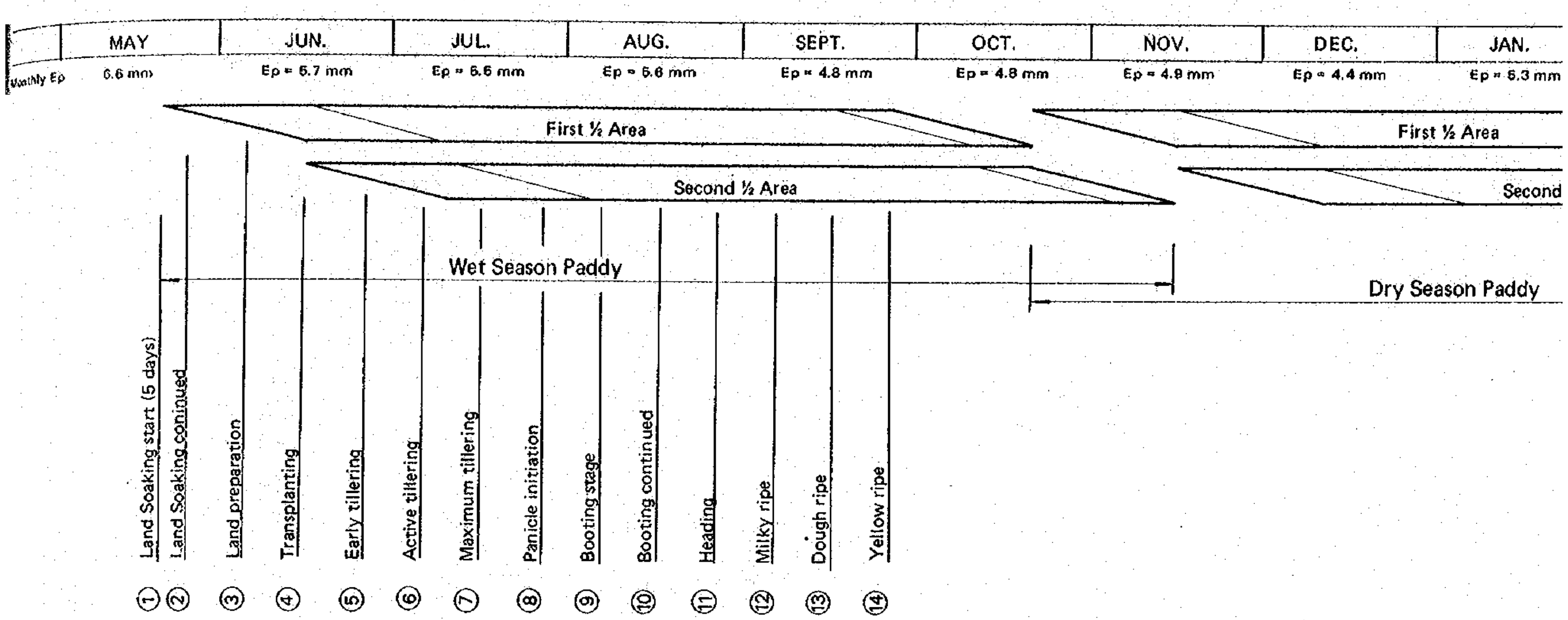
Land to be prepared in May, June and July

1. First Irrigation				150 mm
Top soil saturation 150 mm depth 60 % void, 70 % dry.	:	150 mm x 0.60 x 0.7	=	65
Percolation (1 mm/day)	:	1 mm x 25 days	=	25
Standing water				60
2. Second and Third Irrigation				60
Evaporation in 12 days	:	5 mm x 12 days	=	60
Total				210

Land to be prepared in October, November and December

1. First Irrigation				120
Top soil saturation 150 mm depth 60 % void, 40 % day	:	150 mm x 0.60 x 0.4	=	35
Percolation (1 mm/day)	:	1 mm x 25 days	=	25
Standing water				60
2. Second and third irrigation				50
Evaporation in 12 days	:	4.5 mm x 12 days		50
Total				170

FIGURE 4B-9 CALCULATION OF WEIGHTED CROP WATER REQUIREMENTS

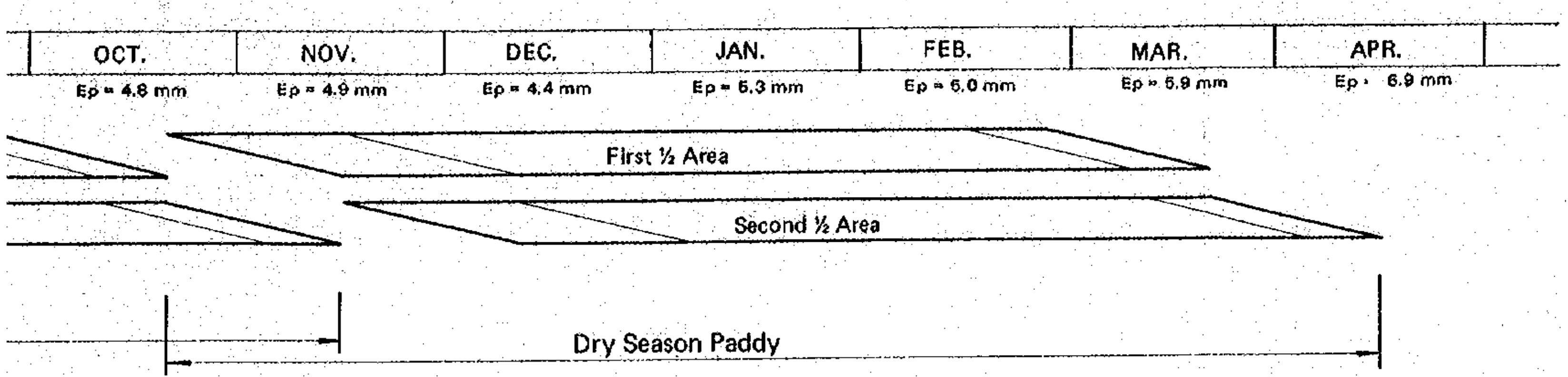


	Growing Stage with 10-day Interval	Water Req. (mm)	Equation for Weighted Irrigation Requirement	Wet Season Paddy (May - Nov.) Weighted 10-day Water Requirement (mm)		Average Daily (mm)
1.	Land soaking start	P_2	$1/2 \times 1/2 \times P_3$	1/2 x 1/2 x 150	= 37.5	3.8
2.	Land soaking continued	P_1	$1/2 \times (1/2 \times P_2 + 2/10 \times P_1)$	1/2 x (1/2 x 150 + 2/10 x 70)	= 44.5	4.5
3.	Land preparation	P	$1/2 \times (8/10 \times P_1)$	1/2 x (8/10 x 70)	= 28.0	2.8
4.	Transplanting	I_1	$1/2 \times (I_1 + P_2)$	1/2 x (67 + 150)	= 108.5	10.9
5.	Early tillering	I_2	$1/2 \times (I_2 + 7/10 \times P_1)$	1/2 x (66 + 7/10 x 75)	= 59.3	5.9
6.	Active tillering	I_3	$1/2 \times (I_3 + 5/10 \times P_1)$	1/2 x (65 + 5/10 x 75)	= 51.3	5.1
7.	Maximum tillering	I_4	$1/2 \times (I_4 + I_4)$	1/2 x (65 + 65)	= 65.0	6.5
8.	Panicle initiation	I_5	$1/2 \times (I_5 + I_5)$	1/2 x (65 + 65)	= 65.0	6.5
9.	Booting stage	I_6	$1/2 \times (I_6 + I_6)$	1/2 x (66 + 66)	= 66.0	6.6
10.	Booting continued	I_7	$1/2 \times (I_7 + I_7)$	1/2 x (66 + 66)	= 66.0	6.6
11.	Heading	I_8	$1/2 \times (I_8 + I_8)$	1/2 x (62 + 62)	= 62.0	6.2
12.	Milky ripe	I_9	$1/2 \times (I_9 + I_9)$	1/2 x (58 + 58)	= 58.0	5.8
13.	Dough ripe	I_{10}	$1/2(I_{10})$	1/2 x 58	= 29.0	2.9
14.	Yellow ripe	I_{11}	$1/2(I_{10})$	1/2 x 58	= 29.0	2.9

Note: 1 $$\frac{10.9 \times 10^{-3} \times 1.0^{ha} \times 10^{4} \times 0.6 \times 10^{3}}{86,400 \times 0.70 \times 0.85 \times 0.9} = 1.414\ \ell/sec/ha$$

0.70 : application efficiency
0.85 : conveyance efficiency
0.90 : operation efficiency

ION OF WEIGHTED CROP WATER REQUIREMENTS



Wet Season Paddy (May - Nov.)			Dry Season Paddy (Oct. - Apr.)			Water Requirement per hectare (ℓ/sec/ha)		
Weighted 10-day Water Requirement (mm)		Average Daily (mm)	Weighted 10-day Water Requirement (mm)		Average Daily	Cropping Area (%)	Wet Season Paddy	Dry Season Paddy
1/2 x 1/2 x 150	= 37.5	3.8	1/2 x 1/2 x 120	= 48.0	4.8	5	0.041	0.052
1/2 x (1/2 x 150 + 2/10 x 70)	= 44.5	4.5	1/2 x (1/2 x 120 + 2/10 x 50)	= 35.0	3.5	20	0.195	0.1513
1/2 x (8/10 x 70)	= 28.0	2.8	1/2 x (8/10 x 50)	= 20.0	2.0	40	0.242	0.1729
1/2 x (67 + 150)	= 108.5	10.9	1/2 x (59 x 120)	= 89.5	9.0	60	1.414[1/](max.)	1.167
1/2 x (66 + 7/10 x 75)	= 59.3	5.9	1/2 x (57 + 7/10 x 50)	= 46.0	4.6	80	1.124	0.7952
1/2 x (65 + 5/10 x 75)	= 51.3	5.1	1/2 x (54 + 54)	= 54.0	5.4	100	1.102	1.167
1/2 x (65 + 65)	= 65.0	6.5	1/2 x (54 + 54)	= 54.0	5.4	100	1.405	1.167
1/2 x (65 + 65)	= 65.0	6.5	1/2 x (59 + 59)	= 59.0	5.9	100	1.405	1.275
1/2 x (66 + 66)	= 66.0	6.6	1/2 x (63 + 63)	= 63.0	6.3	100	1.426	1.361
1/2 x (66 + 66)	= 66.0	6.6	1/2 x (63 + 63)	= 63.0	6.3	100	1.426	1.361
1/2 x (62 + 62)	= 62.0	6.2	1/2 x (62 + 62)	= 62.0	6.2	100	1.390	1.340
1/2 x (58 + 58)	= 58.0	5.8	1/2 x (60 + 60)	= 60.0	6.0	100	1.253	1.297
1/2 x 58	= 29.0	2.9	1/2 x 60	= 30.0	3.0	100	0.627	0.648
1/2 x 58	= 29.0	2.9	1/2 x 60	= 30.0	3.0	90	0.627	0.648

- = 1.414 ℓ/sec/ha

Table 4B-7 **Water Losses**

Water losses for paddy fields consist of the following two losses;

i) Farm application losses

ii) Conveyance losses

The former is considered to be on-farm losses due to mostly farmer's capacity of farm management and topographical conditions, then in the Project the 30 per cent of the average crop water requirement for the wet season paddy and 27 per cent for dry season paddy respectively is taken as on-farm losses, based on the following assumption.

Wet Season Paddy;

Present irrigated and rainfed fields	:	0.15
Reclaimed paddy fields	:	0.35

Dry Season Paddy;

Present irrigated and rainfed fields	:	0.15
Reclaimed paddy field	:	0.30

An average farm application losses are calculated as follows;

Wet season:

$$(1{,}450^{ha} \times 0.15 + 3{,}870 \times 0.35)/5{,}320 = 0.30$$

Dry season:

$$(1{,}450^{ha} \times 0.15 + 3{,}870 \times 0.30)/5{,}320 = 0.27$$

On the other hand, the latter which is water losses during the conveyance stage, furthermore, can be classified into two factors, namely; physical and non-physical factors, and physical factors are composed of seepage, leakage and evaporation losses while non-physical factors are rather related to operational factors such as over-application of irrigation water in the fields, inscheduled drainage and illegal diversion.

However, in the Project Area, no data on the water conveyance losses exist entirely, so that the conveyance losses for main and lateral canals are decided as shown below, Although the Morits empirical formula[1] could be considered as the procedure for estimation of seepage losses of canal.

Description	Conveyance Losses (%)
Phisical losses	15
Non-phisical losses	10

As a result, total water losses for paddy field irrigation are decided at 46.5 [2] percent for the wet season paddy and 41.1[3] percent for the dry season paddy.

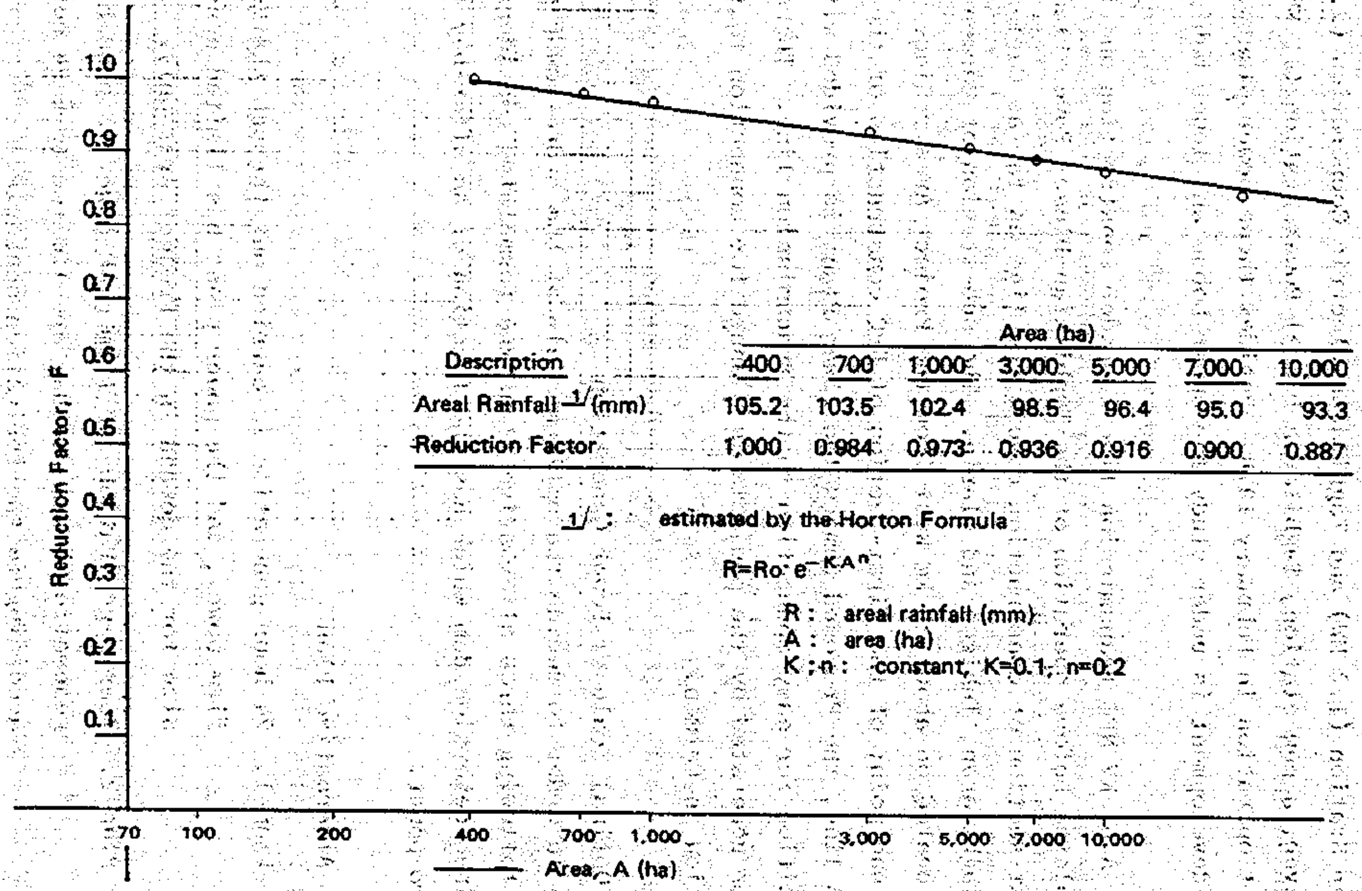
---

[1] : Morits Formula: Water losses (cu.m/sec/km) = $0.04C \cdot (Q/V)^{1/2}$ where, Q = design discharge (cu.m/sec), V = velocity of flow (cu.m/sec) and C = 0.16

[2] : 100 -[(100 - 30%) x (100 - 15%) x (100 - 10%)]

[3] : 100 -[(100 - 30%) x (100 - 15%) x (100 - 10%)]

FIGURE 4B-10 AREA-REDUCTION FACTOR FOR DRAINAGE MODURUS



Description	400	700	1,000	3,000	5,000	7,000	10,000
	Area (ha)						
Areal Rainfall 1/ (mm)	105.2	103.5	102.4	98.5	96.4	95.0	93.3
Reduction Factor	1,000	0.984	0.973	0.936	0.916	0.900	0.887

1/ : estimated by the Horton Formula

$R = Ro \cdot e^{-KA^n}$

R : areal rainfall (mm)
A : area (ha)
K ; n : constant, K=0.1, n=0.2

Electric Hydro-power Development

1. General

Existing power plants in the Province of Bohol are Loboc Hydro Power Station (1.2 MW) and Tagbilaran Electric Cooperation (2.9 MW). However, a new station, Tagbilaran Diesel Power Station (11 MW) is to start operation in February 1978. Besides these power plants, a plan for development is under study for upper Loboc and lower Wahig rivers.

A 69 KV NPC transmission line is under construction and is to be completed before operation of the Tagbilaran Diesel Power Plant. The first section of construction covers the line as far as Carmen by way of Tagbilaran, Tubigon and Clarin. The sub-station of Carmen (5 MW) is located on about 20 km west from the Project Area. Since Carmen is the end point of the first-section of NPC construction of the transmission line, the Proposed Pamacsalan Power Plant is provided with favorable condition for easy connection to the transmission line. Therefore, the power generated by the projects is planned for connection to the sub-station in Carmen.

2. Optimal Scale of Power Generation

In order to decide the optimum scale of the hydro-power plant, the following three alternative plans are formulated considering the Malinao deversion dam which has the function of an after-bay for hydro-power.

(a) Alternative Plan

Plan-1. 24-hour generation is carried out through the year for firm power, and the power plant has limited generation size for the demand in the Project Area only.

Plan-2. Yearly generation for firm peak power is possible. But generation hours per day are limited to 6-24 hours according to the irrigation schedule.

Plan-3. Only peak power is taken into consideration. Peak generation is carried on as much as the performance allows.

These plans have the following merits and demerits as shown in Table 4B-10.

(b) Design Discharge for Determination of Turbine

From the result of reservoir operation study, dependable discharge for hydro-power varies from 5.5 cu.m/sec to 0.4 cu.m/sec as shown in the duration curve (see Figure 4B-11), and then the output of each plan is computed as follows;

Power Output at Each Flow

Item	Discharge (cu.m/sec)					
	1.0	2.0	2.5	3.0	3.5	4.0
Max output (KW)[1/]	573	1,124	1,385	1,633	1,865	2,078
Max energy production (MWH)	5,019	9,846	12,133	14,305	16,337	18,203
Annual production (MWH)	3,500	4,689	4,989	5,175	5,263	5,286
Plan factor (%)	69.7	47.7	41.1	36.2	32.2	29.0

Note: [1/] KW = 9.8 x He x Q x 0.8

He : Effective head (see Figure 4B-13)
Q : Discharge (cu.m/sec)

From the above table, the relation of output and discharge of each plan is decided as follows:

Plan-1. Max discharge 1.0 cu.m/sec
0.5 cu.m/s per unit suitable for minimum discharge of 0.4 cu.m/s. The generated capacity is 300 KW. Required number is 2 sets, and the plant factor is 69.7%.

Plan-2. Max discharge 3.0 cu.m/sec
In order to guarantee high efficient operation even at

the 6 hours peak discharge of 1.6 cu.m/s instead of the minimum discharge of 0.4 cu.m/s. The discharge is set at 1.5 cu.m/s per unit. The generated capacity is 850 KW, the required number is 2 sets, and the plant factor is 36.2%.

Plan-3. Max discharge 3.0 cu.m/sec
The generated capacity is 1,700 KW. The required number is 1 set and the plan factor is 32.6%.

Dischargeable days in a year are calculated by using the duration curve of plan-2 as follows;

Dischargeable Days per Year

Hour	Discharge (%)																	
	100			80			60			50			40			30		
	(1)	(2)	(3)	(1)	(2)	(3)	(1)	(2)	(3)	(1)	(2)	(3)	(1)	(2)	(3)	(1)	(2)	(3)
24	40	70	-	50	70	-	70	80	30	80	100	40	90	100	40	130	170	50
18	60	70	10	60	80	30	100	110	40	100	140	140	130	170	50	210	190	70
12	80	100	40	80	130	40	130	160	50	160	190	60	220	190	80	290	190	80
8	110	160	50	130	170	60	220	190	80	270	190	80	340	360	360	360	360	360
6	160	180	60	180	190	80	290	190	80	350	360	360	360	360	360	360	360	360
	Two operation Limit									One operation Limit								

Note: (1):
(2): Irrigation base year (1972 - 1973)
(3): Irrigation drought year (1968 - 1969)

According to the above table, 290 days a year is possible average with 60% of max. discharge of 3.0 cu.m/s for 6 hours, and 350 days operation with a single generator is possible at 100% dischange. The after-bay is used to control discharge, and the schedule should be decided according to weather conditions and dam water level each time.

Table 4B-10. Merits and Demerits of Alternative Plans

Plan	Merits	Demerits
Plan-1.	1. Firm generation is possible year round, and a stable power supply is maintained.	1. In case 24 hour generation for stable power is carried all through the year, the size of power plant becomes smaller. 2. More over flow. 3. In order to enable generation during off-irrigation period, water storage capacity of the dam should be larger.
Plan-2.	1. Generation is possible year round. 2. Afterbay enables peak generation even during off-irrigation period, and the water may be used effectively for irrigation. 3. A larger generation capacity is allowed.	1. Water storage capacity of the dam should be large enough to enable generation during off-irrigation period. Also, the capacity of the afterbay should be larger. 2. In order to maintain a highly efficient operation during off-irrigation periods, two machines are needed and the construction costs become higher.
Plan-3.	1. One turbine with a capacity to cover the maximum discharge is enough, and construction costs are lower. 2. Generation cost is lower.	1. Turbine effeciency becomes lower during off-irrigation periods, and this may harm the machine. 2. Since there is only a single turbine, generation would stop for a while if the machine broke down. 3. In order to continue generation during off-irrigation period, the water storage capacity of both dam and diversion weir should be larger.

FIGURE 4B-11. DAM WATER RELEASE

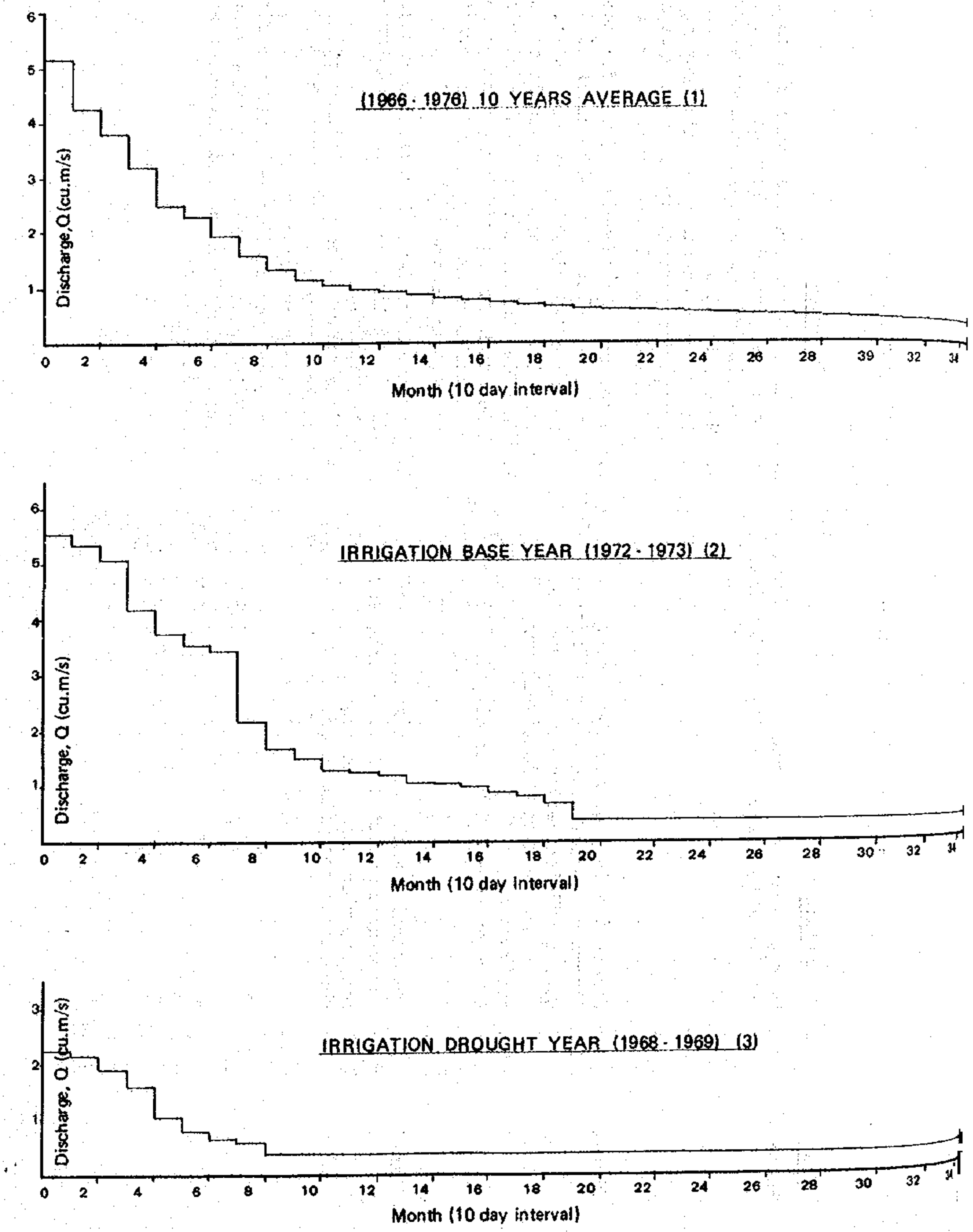
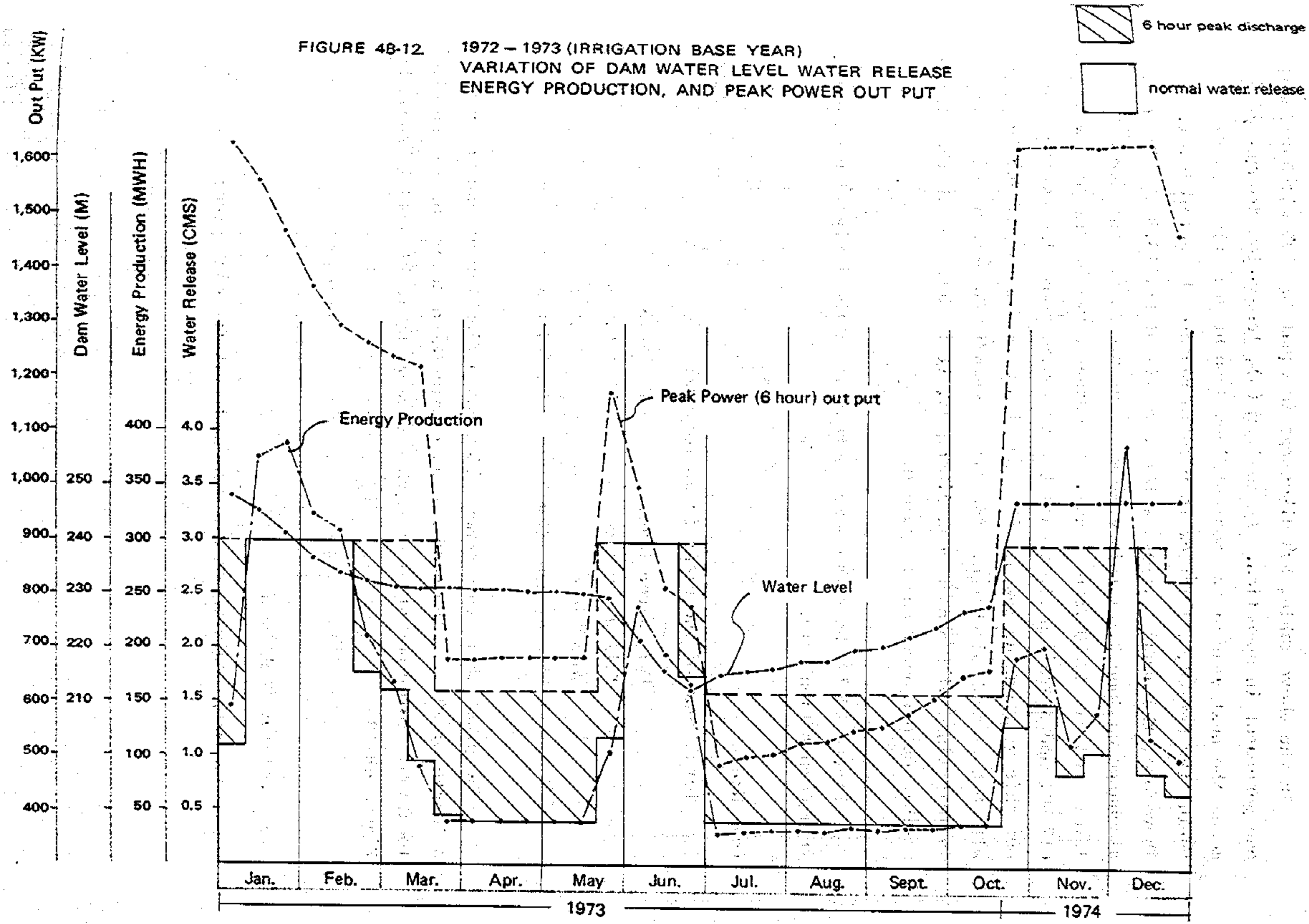


FIGURE 4B-12 1972 – 1973 (IRRIGATION BASE YEAR)
VARIATION OF DAM WATER LEVEL WATER RELEASE
ENERGY PRODUCTION, AND PEAK POWER OUT PUT



In the above table, 50% discharge is equal to 100% discharge with one generator in operation. As to Item (3), Irrigation drought year means the next year of the most serious drought year (1967-1968). During the period, all inflow is used to restore the water level. Therefore, the annual average water level is at EL.218.8 m. The max. discharge on June 3, of this year was 2.6 cu.m/s, but the dam water level then was EL.207.5 m. Thus, the output power became as low as 617 Kw, and operation efficiency became lower, too. The average output this year was as low as 287.6 Kw with farm discharge, and 641.3 Kw with 6 hour peak discharge. Therefore, even if 80 percent discharge is maintained with 8 hour peak discharge, it is doubtful whether the prescribed output power can be gained.

Figure 4B-12 shows the power potential of the irrigation base year, from computation data.

(c) Head

Total head of the plant is as follows:

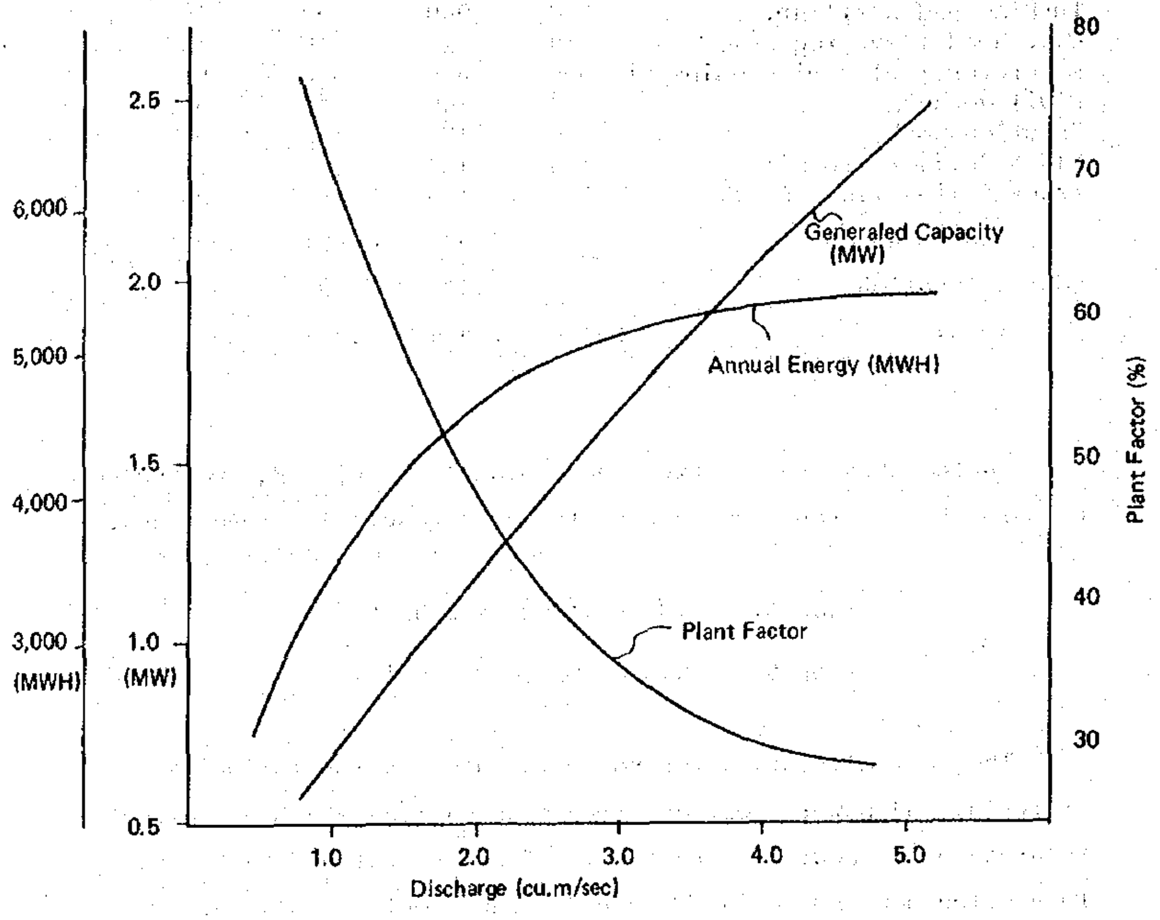
Full water level	EL 248.5 m
Tail water level	EL 175.0 m
Total head	73.5 m
Pile of sand level	EL 207.0 m

The penstock in this computation has a diameter of one meter and a length of 250 m. The head may be recalculated according to the plant location. Power potential on each discharge by total head is shown in Figure 4B-13.

(d) Cost for Each Plan

Following table shows the required cost for each plan:

FIGURE 4B-13. POWER POTENTIAL



Note: Computation Factor as Follows

Head Loss = $0.2791 \times (\frac{Q}{A})^2$ A : $0.5^2 \times \pi$

Total Head = EL 248.5M – EL 175 M = 73.5 M

Effective Head = Total Head – Head Loss

MW = E'HEAD x 9.8 x Discharge x 0.8 x 1,000

Annual Energy from Computation Data

Description	Plan-1	Plan-2	Plan-3	Plan-A	Plan-B
	cost (US$1,000)				
Turbine and auxiliary	848	960	900	950	1,150
Over head traveling crane	14	16	16	16	16
Generator & electric equipment	306	346	306	340	421
Civil works	87	109	109	109	130
Erection works	140	172	150	160	190
CIF & delivery to site	124	140	100	130	155
Transmission line, 69 kV	180	180	180	180	180
Sub-Total	1,699	1,923	1,761	1,885	2,242
Cost allocation	431	531	531	531	531
Total	2,130	2,454	2,292	2,416	2,773

In the above table, two plans, plan-A and Plan-B, are included to compare the reference cost under the same generation conditions.

Plan-A: 1.25 cu.m/sec x 2 units

Plan-B: 1.75 cu.m/sec x 2 units

The cost allocation for power is based on the following criteria: a) power plant and transmission line which are directly related to the power generation is of power cost and b) the costs of Pamacsalan dam and Malinao diversion dam should be allocated.

Allocation cost is estimated at US$531 x 10^3, consisting of US$400 x 10^3 for water level raising to enable generation during off-irrigation period, US$100 x 10^3 for water level raising of Malinao diversion dam and US$31 x 10^3 for engineering services. No allocation of diversion dam cost is needed in Plan-1 because of firm power.

(e) Benefits of Each Plan

According to the premise of benefit estimation, the benefit of each plan is as follows:

Description	Benefit (US$'000) Plan-1	Plan-2	Plan-3	Plan-A	Plan-B
Installed capacity[1] (KW)	300 x 2	850 x 2	1,700	700 x 2	950 x 2
Dependable capacity (KW)	487	1,225	1,225	1,103	1,328
Energy available[2] (MWH)	3,360	4,968	4,968	4,789	5,052
KW value $85.4/KW	41.59	104.62	104.62	94.20	113.4
KWH value $0.0258/KWH	86.69	128.17	128.17	123.56	130.34
Annual benefit	128.28	232.77	232.77	217.76	243.74

Note: [1] the lowest average of six hours peak capacity for 20 years is applied. In Plan-1, it is equivalent to 80 percent of the installed capacity.

[2] 96 percent of annual energy production

(f) Economic Evaluation

Economic evaluation for each plan was made by applying B/C ratio method and in this evaluation, the following are assumed;

1. Replacement period

   Transmission line and electric equipment: 25 years

   Turbine and generator : 30 years

2. Interest: 8.5 %

3. Operation and Maintenance cost include expenses of spare parts for repairs or miscellaneous, personnel cost for operation and allocation cost of O & M cost for Pamacsalan dam and Malinao diversion dam.

   Expenses of spare part:

   2 % of power plant and transmission line costs

   Personnel cost:

(1) Mechanical engineer	@₱9,410 x 1 person	= ₱ 9,410
(2) Electrical engineer	@₱6,320 x 1 "	= 6,320
(3) Mechanical operator	@₱5,910 x 6 "	= 35,460
(4) Electrical operator	@₱5,910 x 3 "	= 17,730
Total		₱68,920
		≒ (US$9,200)

Allocated O & M cost:

Allocated dam O & M cost

$$₱214 \times 10^3 \times 0.077^{1/} = 16.48 \times 10^3 = \$2.20 \times 10^3$$

Allocated diversion weir O & M cost

$$₱80.5 \times 10^3 \times 0.038^{2/} = 3.1 \times 10^3 = \$0.41 \times 10^3$$

Note: 1/ $$\frac{\text{Dam allocated cost}}{\text{Dam construction cost}} = \frac{₱3,000 \times 10^3}{₱39,070 \times 10^3} = 0.077$$

2/ $$\frac{\text{Diversion dam allocated cost}}{\text{Diversion dam construction cost}} = \frac{₱750 \times 10^3}{₱19,530 \times 10^3} = 0.038$$

Operation and Maintenance cost is estimated as follows;

Description	O & M Cost (US$'000)				
	Plan-1	Plan-2	Plan-3	Plan-4	Plan-5
Repair and miscellaneous	33.98	33.46	35.22	37.70	44.84
Annual salary	9.2	9.2	9.2	9.2	9.2
Dam O & M	2.2	2.2	2.2	2.2	2.2
Diversion weir O & M	-	0.41	0.41	0.41	0.41
Total	45.38	50.27	47.03	49.51	56.65

Based upon the above assumption and estimation, economic evaluation for each plan was made and results are given in Table 4B-11.

According to Table 4B-11, B/C ratio, in general, is not favorable. Among these plans, however, Plan-3 with the B/C ratio of 0.71 is better than any other plan. The energy prime cost in this plan is the lowest among the plans and it could be considered to be competitive enough against that of Tagbilaran Diesel Power Plant. But, in this plan, thereis only one generator within installed capacity of 1,700 KW, so, effective operation can not be expected.

In Plan-1, the B/C ratio is 0.43 and it is almost impossible to compete with other plans. This shows a general trend of hydro-power plant construction, and a small-capacity generation does not make a proper profit.

Annual equivalent cost is figured out from total investment cost at present worth. The prime energy cost in Plan-2 comes to US$0.039/KWH (₱0.293/KWH). The prime energy cost of power plant and transmission line of the Tagbilaran Diesel Power Plant in 1974 has been estimated at US$ 0.031 (₱0.233). This cost is equivalent to US$0.039 in 1977 and is almost the same as the one in case of Plan-2. Therefore, it can be said that Plan-2 would be competitive with Plan-3.

Through these studies on the most suitable scale of hydro-power, Plan-2 is finally recommended as the project plan.

Table 4B-11. Economic Evaluation of Each Plan

	Description		Plan-1	Plan-2	Plan-3	Plan-A	Plan-B
1.	Max discharge	(cu.m/s/unit)	0.5	1.5	3.0	1.25	1.75
2.	Installed capacity	(KW)	300 x 2	850 x 2	1,700 x 1	700 x 2	950 x 2
3.	Dependable capacity	(KW)	487	1,225	1,225	1,103	1,328
4.	Annual generated energy	(MWH)	3,500	5,175	5,175	4,989	5,263
5.	Annual energy available	(MWH)	3,360	4,968	4,968	4,789	5,052
6.	Annual Cost						
	Construction cost	(US$'000)	2,130	2,454	2,292	2,416	2,773
	O & M cost	(US$'000)	45.38	50.27	47.03	49.51	56.65
	Replacement	(US$'000)	1,410	1,636	1,462	1,565	1,861
7.	Annual benefit	(US$'000)	128.28	233.77	233.77	217.76	243.74
8.	Present worth factor	(%)	8.5	8.5	8.5	8.5	8.5
9.	Present worth of cost	(US$'000)					
	Investment cost		1,574.83	1,836.01	1,739.03	1,813.25	2,026.93
	O & M cost		299.55	324.06	303.19	319.18	365.20
	Replacement cost		72.65	82.18	75.27	80.58	95.83
	Total Cost (C)		1,940.03	2,242.25	2,117.49	2,213.01	2,487.96
10.	Present worth of benefit	(US$'000)					
	Benefits (B)		826.98	1,507.04	1,507.04	1,403.84	1,571.32
11.	B/C		0.426	0.672	0.712	0.634	0.631
12.	Annual equivalent cost						
	Investment		136.14	158.72	150.34	165.75	175.23
	O & M		25.29	28.01	26.21	27.59	31.57
	Replacement		6.28	7.10	6.51	6.97	8.28
	Total	(US$'000)	167.71	193.83	183.06	200.31	215.08
	C/KWH	(US$)	0.0499	0.0390	0.0368	0.0418	0.0426

Note: Table 4B-12 shows the present worth of cost of plan-2

Table 4B-12. Present Worth of Cost for Plan-2 (i = 8.5%)

	Cost (US$'000)				Present Worth of Cost (US$'000)				
Year	Investment	O & M	Replacement	Power Benefits	Investment	O & M	Replacement	Total	Power Benefits
1	25	-	-	-	29.04	-	-	23.04	-
2	6	-	-	-	5.1	-	-	5.1	-
3	132	-	-	-	103.34	-	-	103.34	-
4	90	-	-	-	64.94	-	-	64.94	-
5	69	-	-	-	45.89	-	-	45.89	-
6	1,653	-	-	-	1,141.22	-	-	1,141.22	-
7	704	-	-	-	452.48	-	-	452.48	-
8-31	-	50.27	-	233.77	-	286.93	-	286.93	1,334.31
32	-	50.27	480	233.77	-	3.69	35.28	26.42	17.18
33-41	-	50.27	-	233.77	-	22.60	-	22.60	105.10
42	-	50.27	1,443	233.77	-	1.63	46.90	31.24	7.60
43-50	-	50.27	-	233,77	-	9.21	-	9.21	42.85
Total					1,836.01	324.06	82.18	2,242.25	1,507.04

Total annual investment cost = US$2,242.25 x 10^3

Total annual power benefit = US$1,507.04 x 10^3

B/C = 0.672

Paddy Production with Project

1. Estimation of Target Yield

(a) Estimation of Potential Yield

In Bohol, any experimental data on paddy yield are not available for the estimation of potential yield because only one newly opened national experiment station is existing in the Province. The following is the experimental data on paddy yield at the three national experiment stations inclusive of Visayas Rice Experiment Station.

Experimental Yield

Season	Nitrogen Application (kg/ha)				
	0	30	90	120	150
Wet	3.3	3.9	4.5	4.2	-
Dry	3.5	-	4.7	5.3	5.4

Note: (1) Unit of yield: ton/ha
(2) Experimental yield: Average yield for the latest 6 HYVs at three national experiment stations BPI, 1975

The quadratic equations of regression between yield (y) and nitrogen application (x) are shown as follows:

Wet season crop: $y = 3.28 + 0.025x - 0.00014x^2$

Dry season crop: $y = 3.50 + 0.029x - 0.00011x^2$

The curves of the equations are shown in Figure 4C-1

(b) Attained Paddy Yield in the Project and Its Vicinity

The data of attained paddy yield are collected from some outstanding farmers, one compact farm, DAR demonstration farms, and others in the Project Area and its vicinity. (See Table 3D-7, Appendix 3D-1) From the collected data, following yield are taken as the representative yields which are attained at the fields where irrigation and drainage conditions are existing on similar level to the proposed facilities for the Project;

	Yield (ton/ha)	Nitrogen application (kg/ha)
Wet season crop	4.4	60
Dry season crop	4.7	80

This yield data are substituted for above mentioned regression equations of experimental yields. As a result, following equations are obtained.

Wet setason crop: $y = 3.40 + 0.025x - 0.00014x^2$

Dry season crop: $y = 3.08 + 0.029x - 0.00011x^2$

These equations are assumed as the yield Curves which expresse the yield of "after Project". The figure of the curves are shown in Figure 4C-1.

(c) Yield at Optimum Amount of Nitrogen Application

Optimum amount of nitrogen application and their yields are computed as follows:

Wet season crop:

$$\text{Opt. nitrogen (kg/ha)} = \frac{0.025Py - Pn}{2(0.00014xPy)} = 80\ \text{kg}$$

Yield at the nitrogen application = 4.5 ton/ha

Dry season crop:

$$\text{Opt. nitrogen (kg/ha)} = \frac{0.029\ Py - Pn}{2(0.00011xPy)} = 120\ \text{kg}$$

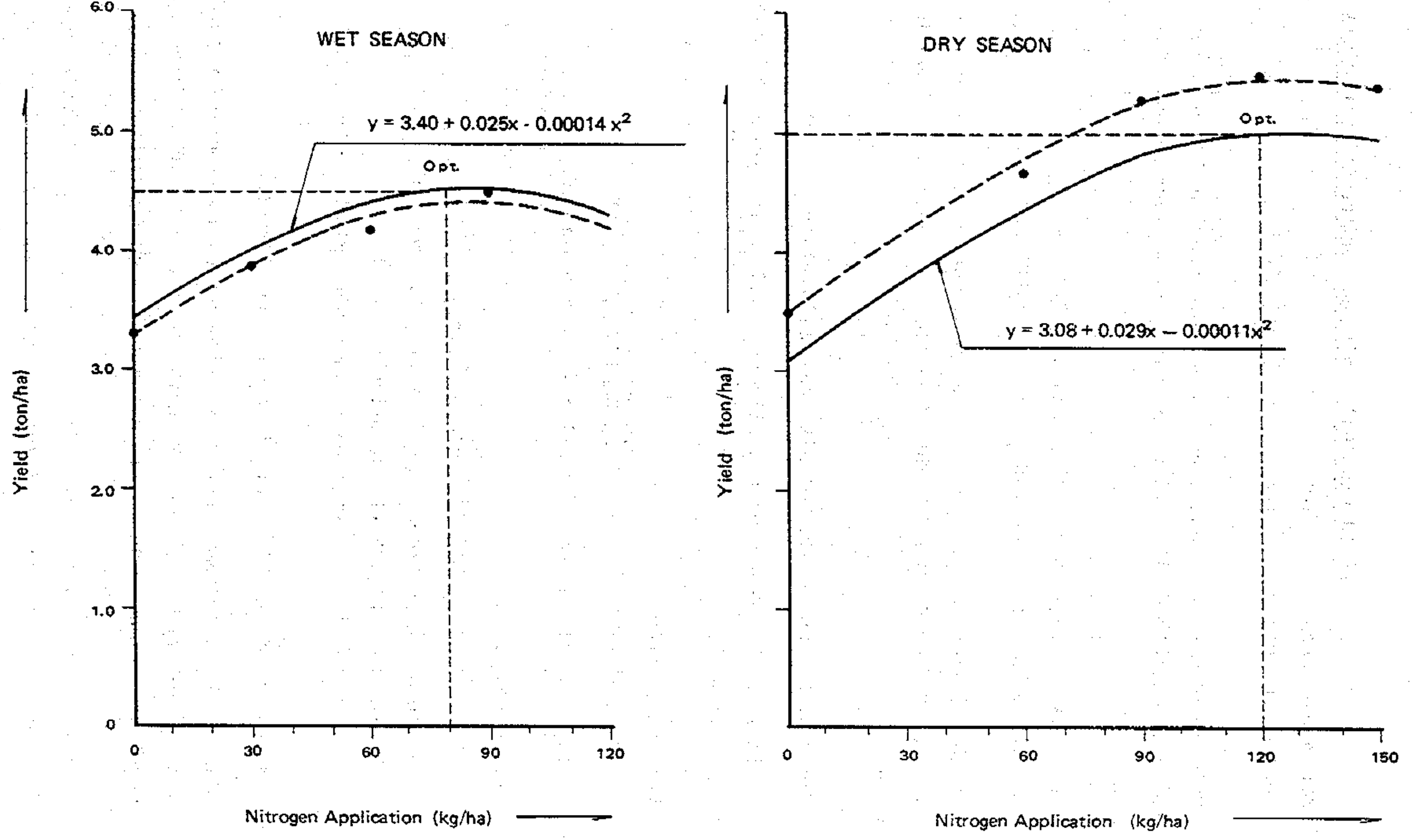
Yield at the nitrogen application = 5.0 ton/ha

Where Py = ₱1,375 (Paddy price per ton)
Pn = ₱ 3.6 (Nitrogen price per kg)

(d) Estimation of the Yield at Different Land Classes

The yields at optimum amount of nigrogen application are regarded as the yield of land class 1R. Then the potential yields and the yields at the different land classes are estimated as follows;

FIGURE 4C-1 EXPERIMENTAL AND PREDICTED YIELDS FOR PROJECT AREA



Remarks: ------ Experimental yield: Effect of levels of nitrogen on yield of rice varieties (IR26, 28, 30, 32, 34, 36) at three national experiment stations (Maligaya Rice Research and Training Center, Bicol Rice and Corn Experiment Station and Visayas Rice Experiment Station), 1975

Yield at Different Land Classes

Land Class	Range of Productivity (%)	Average Productivity Rating (%)	Estimated Yields (ton/ha)	
			Wet Season	Dry Season
Potential Yield	100	100	4.74	5.26
1 R	90 - 100	95	4.5	5.0
2 R	80 - 90	85	4.0	4.4
3 R	70 - 80	75	3.5	4.0

The yield for Wahig-Headwork irrigation area is estimated at 95% of the above estimated yields. It is the reason why the yields are reduced that the irrigation water in Wahig-Headwork area is designed to be supplied at the level of the probability of 1/5, comparing to the Pamacasalan-Dam irrigation area where the probability is 1/10.

2. Total Amount of Paddy Production

The paddy production by land class and by irrigation area is shown in Table 4C-1.

The total amount is estimated at about 42,200 tons per year. And average yield in the Project is 3.8 ton/ha and 4.2 ton/ha for wet and dry seasons, respectively.

Table 4C-1 Paddy Production "with Project" in Future by Land Class

Area	Season	2 R			3 R			Total	
		Area (ha)	Yield (ton/ha)	Production (ton)	Area (ha)	Yield (ton/ha)	Production (ton)	Area (ha)	Production (ton)
1. Upper Area									
(a) Wahig	Wet	235	3.8	893.0	21	3.3	24.3	256	917.3
	Dry	257	4.2	1,079.4	143	3.8	543.4	400	1,622.8
(b) Pamacsalan	Wet	91	4.0	364.0	29	3.5	101.5	120	465.5
	Dry	91	4.4	400.4	29	4.0	116.0	120	516.4
Sub-total	Wet	326	-	1,257.0	50	-	125.8	376	1,382.8
	Dry	348	-	1,479.8	172		530.2	520	2,139.2
2. Lower Area	Wet	2,970	4.0	11,880.0	1,830	3.5	6,405.0	4,800	18,285.0
	Dry	2,970	4.4	13,068.0	1,830	4.0	7,320.0	4,800	20,388.0
3. Total	Wet	3,296	-	13,137.0	1,880	-	6,530.8	5,176	19,667.8
	Dry	3,318	-	14,690.6	2,002	-	7,850.2	5,320	22,527.2

Note: (1) Average yield in the Project area
Wet season: 3.8 ton/ha
Dry season: 4.2 ton/ha
(2) Total amount of wet and dry season: 42,195.0 ton

Table 4C-2 Recommended Farm Practices and Input Materials for Paddy Cultivation

Operation	Recommended Practices	Input Materials per Hectare
1. Preparation of Seedbed	(a) Form and size of seedbed; There are two recommendable methods in raising seedlings, which are wet and dapog seedbed. In case of wet seedbed, prepare puddled plots having one to 1-1/2 m width and any convenient length. The total plot area is 400 m^2 per hectare of transplanting area.	
	(b) To mix in the seedbed plots the recommended amount of fertilizer.	Fertilizer for seedbeds; 10kg of 14-14-14
2. Sowing	Seed rate and seed preparation; - Seed rate; 110g per m^2 of seedbed - Selection; Remove unfilled grains by soaking seeds in water - Soaking; In fungicided clean-water or running water for 24 hours. Be sure to change the water every 8 hours in case of clean water.	Seeds: 45 kg
3. Care of Seedlings	(a) 3 days after sowing, start irrigation in filmy shallow water level. And increase gradually the water level up to 2-3 cm as the seedlings grow tall.	
	(b) 10 days after sowing, apply insecticides to protect insects/pests such as green leafhoppers, brown planthoppers, whorl maggots, stemborers, etc.	Insecticides for seedlings; BPMC+Chlorphyrifos (e.g. 153cc of Brodan EC, 21.0%+ 10.5% in 9 gallon of water) or any other kinds of insecticides.
	(c) The age of seedlings for transplanting; 16 days after sowing for the early maturing varieties (115 days or less) and 21 days for the medium maturing varieties (120 - 140 days)	

Operation	Recommended Practices	Input Materials per Hectare
4. Land Preparation	(a) Irrigate the soils if it is too dry to plow.	
	(b) Repair all dikes to destroy rat dwellings and also to minimize seepage.	
	(c) For the fields which are very weedy or having plenty of plant stubbles, start plowing 3 weeks before transplanting to allow these materials to decompose. After plowing, flood the fields to prevent the loss of nitrogen and to prevent further growth of weeds.	
	(d) Harrowing shall be performed as follows; 1st step: Within 3 to 5 days after plowing, harrow the fields longitudinally and cross-wide with enough flooded water to break up and to puddle partially clods while burying the weeds. Continue to keep the water about one cm deep.	
	2nd step: 5 to 7 days after the 1st harrowing, harrow the fields again to puddle fully and make the field relatively level. Be sure to keep the field flooded.	
	3rd step: One day before transplanting, drain most of the water from the field and apply basal fertilizer. Immediately after the application, harrow the field to mix the fertilizer with soil. Then level the field very well.	Basal fertilizer: - Wet season: 200kg of 14-14-14 (N: 28, P_2O_5: 28kg, K_2O: 28kg) - Dry season: 200kg of 14-14-14 (N: 28kg, P_2O_5: 28kg, K_2O: 28kg)

Operation	Recommended Practices	Input Materials per Hectare
5. Transplanting	(a) Soak roots of pulled seedlings in fungicided water 24 hours before transplanting to protect the seedings from insects/pests as mentioned in 3, (b).	Insecticides for treatment of seedlings: Carbofuran (e.g. 740cc of Furadan 2F in 150 liters of water) or any kinds of chemicals.
	(b) Transplant 2 - 3 seedlings per hill at a depth of 2-3 cm with a spacing of 20cm x 20cm (in case of wet-seedbed seedlings).	
	(c) 3 days after transplanting, start irrigating the field to have 2 cm of standing water depth and gradually increase the standing water depth up to 5cm as the crops grow taller. Be sure to maintain continuously shallow irrigation up to milk-ripe stage. Especially during the period from panicle initiation to one week after flowering, sustain water depth of 5cm.	
6. Weed Control	(a) 3-6 days after transplanting, apply the pre-emergence herbicides. For the maximum results, maintain water in the field with a depth of 3-5cm at least 3 weeks after the application.	Pre-emergence herbicides; Benthiocarb (e.g. 25kg of SaturnS, 5%, G) or any other kinds of herbicides.
	(b) Within 20 to 25 days after transplanting, control remnant weeds by hand weeding and rotary weeding or application of any of the post-emergence herbicides.	
7. Spraying Insecticides	(a) At 15 and 35 days after transplanting, apply insecticides to protect the plant from the insects/pests including whorl maggots, green leafhopper, brown leafhoppers and stemborers, etc.	Insecticides; - 15 and 35 days after transplanting; Carbophenothion (e.g. 0.5 and 0.75qts of Lethox EC, 48%, in 100 and 150 gallons of water respectively) or any other kinds of insecticides.

Operation	Recommended Practices	Input Materials per Hectare
	(b) At early booting stage (65 days after sowing for 115 days or less maturing varieties and 75 days for 120 - 140 maturing varieties), apply insecticides to protect the plant from the insects/pests such as brown planthoppers, stemborers and rice bug, etc.	- 65 or 75 days after transplanting: monocrotophos (e.g. 3.5kg of Azodrin EC, 20.2% in 300 gallons of water) or any other kinds of insecticides.
8. Topdressing	(a) 1st topdressing: 35 days (for the early maturing varieties) or 50 days (for the medium maturing varieties) after sowing.	1st topdressing: - Dry season: 50kg of urea (N:23kg)
	(b) 2nd topdressing: 50 days (for the early maturing varieties) or 66 days (for the medium maturing varieties after sowing).	2nd topdressing: - Wet season: 100kg of ammonium sulfate (N:21kg) - Dry season: 100kg of ammonium sulfate (N:21kg)
9. Drainage	At the hard dough stage (10 days before harvesting), stop irrigation and drain the water in the field completely.	
10. Harvesting & Threshing	(a) When at least 80% of the grains in panicle have turned yellow, start to harvest the rice (do not delay harvesting beyond 30 days after flowering to minimize much harvesting losses)	
	(b) Thresh the harvests immediately, at least within 3 days after harvesting especially during the rainy season to prevent deterioration of grain quality and viability .	
11. Cleaning & Drying	Clean and dry the paddy promptly to assure quality grains. Reduce the moisture content to 14% by sunshine or dryer.	

Note: (1) For rat control, Chronic rodenticides (e.g. 0.25kg of Ratoxin, 0.5%) may be necessary for the sustained baiting after transplanting to harvesting.

(2) The total fertilizer requirement per hectare is estimated from "the masagana-99 fertilizer recommendation in some part of the Project area by Bureau of Soil" as shown below.

	N	P_2O_5	K_2O	Remarks
Wet season	72 kg	28 kg	28 kg	The fertilizer requirement shall be provided for the whole Project Area before the Project implementaton.
Dry season	49 kg	28 kg	28 kg	

Forecasting of Population

Table 4C-3 Household Population of Bario Concerned

		1975		1970
		Household Population	Number of Household	Household Population
I.	S. Bullones	7,269	1,245	6,918
	1. Poblacion	1,685	274	2,388
	2. Anibogan	525	102	465
	3. Santa Cruz	576	108	523
	4. Vill Garcia	725	129	622
	5. Sarvador 1/	1,026	175	-
	6. San Jose	425	72	369
	7. Bogsoc	1,224	209	1,523
	8. Canlangit	1,083	176	1,028
II.	Pilar	4,600	741	3,598
	9. Delpilar	338	57	233
	10. Lumbay	935	148	628
	11. Caguwasan	364	62	225
	12. Estaca	913	148	722
	13. Buena Serte	823	132	676
	14. San Isidro	825	123	796
	15. La Suerte 2/	402	71	318
III.	Dagohoy	4,828	833	3,935
	16. Pabocion	1,367	232	1,098
	17. San Miguel	940	170	819
	18. Malitbog	735	114	570
	19. Calwasan	788	143	672
	20. Mahayag	381	62	346
	21. Babag	617	112	430
IV.	San Miguel	3,273	553	(964)
	22. Mahayang	913	155	964
	23. San Pascual 3/	2,360	398	n.a.
V.	Alicia	1,314	228	944
	24. Katipunum	1,314	228	944
	Total	21,284	3,600	(16,359)

(Annual Growth Rate ; 2.9%)

Note: 1. New Bario established in 1975
2. New Bario established in 1975
3. Included in Ubay

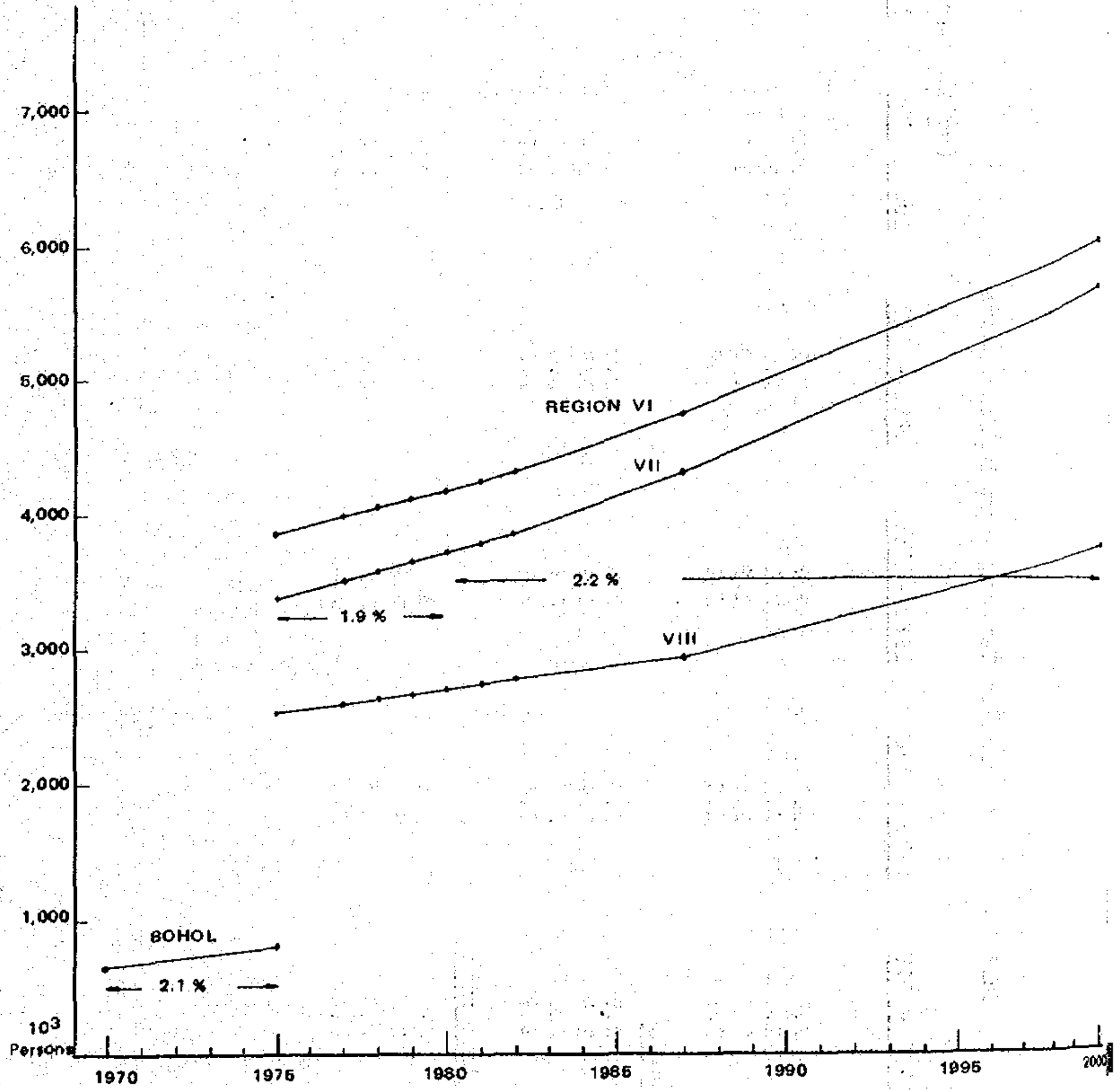
Source: Census of Population

Table 4C-4 Population Target, Annual Growth Rate

(unit: percent)

	1970 - '75	1975 - '80	1980 - '85	1985 - '90	1990 - '95	1995 - 2000
Bohol						
Case 1.	2.1	2.1	2.2	2.2	2.2	2.2
Case 2.	2.1	2.1	2.1	2.1	2.1	2.1
Case 3.	2.1	2.0	1.9	1.9	1.9	1.9
Case 4.	2.1	1.9	1.9	1.9	1.9	1.9
Project Area, 24 Bario						
Case 1.	2.9	2.9	2.9	2.9	2.9	2.9
Case 2.	2.9	2.9	2.8	2.6	2.4	2.2
Case 3.	2.9	2.7	2.6	2.4	2.2	2.2
Case 4.	2.9	2.6	2.4	2.2	2.2	2.2
Five Municipality Concerned						
Case 1.	3.0	3.0	3.0	3.0	3.0	3.0
Case 2.	3.0	3.0	2.8	2.6	2.4	2.2
Case 3.	3.0	2.8	2.6	2.4	2.2	2.2
Case 4.	3.0	2.6	2.4	2.2	2.2	2.2

FIGURE 4C-2 REGIONAL POPULATION TARGET 1975 - 2000



Note: Region VI : West Visayas
Region VII : Central Visayas
Region VIII : East Visayas

Source: Long-Term and Five Year (1978 - 82) Development Plans Draft Summary 1977, NEDA

Table 4C-5 Population Target

(unit: persons)

	1970	1975	1980	1985	1990	1995	2000
Bohol							
Case 1.	683,297	759,370	842,520	939,330	1,047,260	1,167,590	1,301,740
Case 2.	683,297	759,370	842,520	934,730	1,037,140	1,150,700	1,276,700
Case 3.	683,297	759,370	838,340	921,000	1,011,810	1,111,570	1,221,170
Case 4.	683,297	759,370	834,240	916,500	1,006,870	1,106,150	1,215,220
Five Municipality Concerned							
Case 1.	54,754	63,525	73,620	84,520	97,040	111,400	127,800
Case 2.	54,754	63,525	73,620	84,520	96,090	108,190	120,620
Case 3.	54,754	63,525	72,930	82,910	93,350	104,080	116,040
Case 4.	54,754	63,525	72,220	81,310	90,650	101,070	112,680
Project Area							
Case 1.	18,400	21,284	24,500	28,300	32,600	37,600	43,400
Case 2.	18,400	21,284	24,500	28,100	31,900	35,900	40,000
Case 3.	18,400	21,284	24,300	27,600	31,000	34,600	38,600
Case 4.	18,400	21,284	24,200	27,200	30,300	33,800	37,700

Table 4C-6 Population 10 Years Old and Over Classified by Major Gainful Occupation, Bohol: 1970 and 1975

Major Gainful Occupation Group	1975 [1]		1970 [2]	
	Number	Percent	Number	Percent
Total	232,824	100.0	255,021	100.0
Farmers, fishermen, hunters, loggers and related workers	150,940	64.8	147,684	57.9
Craftsmen, production process workers and related laborers	32,966	14.2	46,342	18.2
Sales workers	14,446	6.2	15,976	6.3
Services, sports and related workers	10,861	4.7	14,901	5.8
Professional, technical and related workers	9,095	3.9	14,614	5.7
Workers in transport and communications	4,486	1.9	4,957	1.9
Stevedores, related freight handlers and laborers n.e.c.	4,178	1.8	3,706	1.5
Clerical workers	2,298	1.0	3,205	1.3
Administrative, executive and managerial workers	1,121	0.5	1,910	0.8
All others	2,433	1.0	1,726	0.6

Source: Population Census, 1975

Table 4C-7 Ratio of Unemployment

Province & Municipality	Economical Active (person)	Employed (person)	Unemployed (person)	Ratio of Unemployment (%)
Inside of the Project Area				
Bohol	245,544	232,351	13,193	5.4
Sierra-Bullones	4,181	3,811	370	8.8
Pilar	3,747	3,692	55	1.5
Dagohoy	2,369	2,343	26	1.1
San Miguel	4,920	4,735	185	3.8
Alicia	3,362	3,162	200	5.9
Total	18,579	17,743	836	4.5
Outside of the Project Area				
(Adjacent)				
Ubae	9,373	8,711	662	7.1
Candijay	4,252	3,712	540	12.7
Guindulman	5,590	5,092	498	8.9
Duero	3,118	2,862	256	8.2
Jagna	6,342	5,473	869	13.7
Carmen	6,664	6,548	116	1.7
Mabini	5,176	4,874	302	5.8
Batuan	2,025	1,695	330	16.3
Total	42,540	38,967	3,573	8.4
(Other)				
Tagbilaran	13,706	12,736	970	7.1
Trinidad	4,811	4,614	197	4.1
Talibon	13,059	12,486	573	4.4
Tubigon	8,412	7,927	485	5.8
Bilar	4,051	3,793	258	6.4
Loboc	4,261	3,989	272	6.4
Dimiao	2,862	2,638	224	7.8
Maribojoc	4,837	4,701	136	2.8

Source: Population Census, 1970

Table 4C-8 Employed Situation 1970

Province & Municipality	All persons 10 years and over (Total)	Type of Activity					
		Economical Active		Not Economical Active			
		Employed	Unemployed	House keeper	Student	Others	Not Stated
Bohol	490,137	232,351	13,193	132,430	23,995	87,216	952
Male	234,734	154,294	7,180	10,632	12,243	50,092	293
F.M.	255,403	78,057	6,013	121,798	11,752	37,124	659
S. Bullones	8,857	3,811	370	2,044	114	2,510	8
Male	4,290	2,597	235	120	9	1,329	-
F.M.	4,567	1,214	135	1,924	105	1,181	8
Pilar	7,470	3,692	55	1,829	51	1,334	-
Male	3,548	2,586	27	102	-	670	-
F.M.	3,922	1,106	28	1,727	51	664	-
Dagohoy	4,685	2,343	26	1,696	282	338	-
Male	2,323	1,723	26	159	184	231	-
F.M.	2,362	620	-	1,537	98	107	-
S. Miguel	6,691	4,735	185	1,127	134	510	-
Male	3,302	2,860	26	87	22	307	-
F.M.	3,389	1,875	159	1,040	112	203	-
Alicia	8,877	3,162	200	4,166	212	1,110	21
Male	4,384	2,651	162	696	142	733	-
F.M.	4,493	511	38	3,470	70	377	21
S. Munici	36,580	17,743	836	10,862	793	5,802	29
Male	17,847	12,417	476	1,164	357	3,270	-
F.M.	18,733	5,326	360	9,662	436	2,522	29

Farm Mechanization

1. Improvement of Farm Operation System

Improved farm operaton system for paddy cultivation, as shown in Figure 4C - 3, (3), will be introduced in the Project area after the Project. The system requires to introduce such farm machineries as following for the mechanization of the major operations.

Machinery to be Introduced

Operation	Machinery
Plowing	Hand tractor (7 - 8 HP) with plow
Threshing	Powered thresher (7 - 8 HP, throw-in type)
Drying	Dryer (Flat bed type, 2.0 tons bin)

For such operations as puddling and transportation, carabao will be utilized. Threshing by pedal thresher and drying of paddy by sunshine will remain in some part of area as a supplement of threshing by powered thresher and drying by dryer respectively.

Capacity and efficiency of the machineries are shown in Table 4C - 9. And all of the machinereies are available in the Philippines.

2. Plan of Farm Machinery Utilization

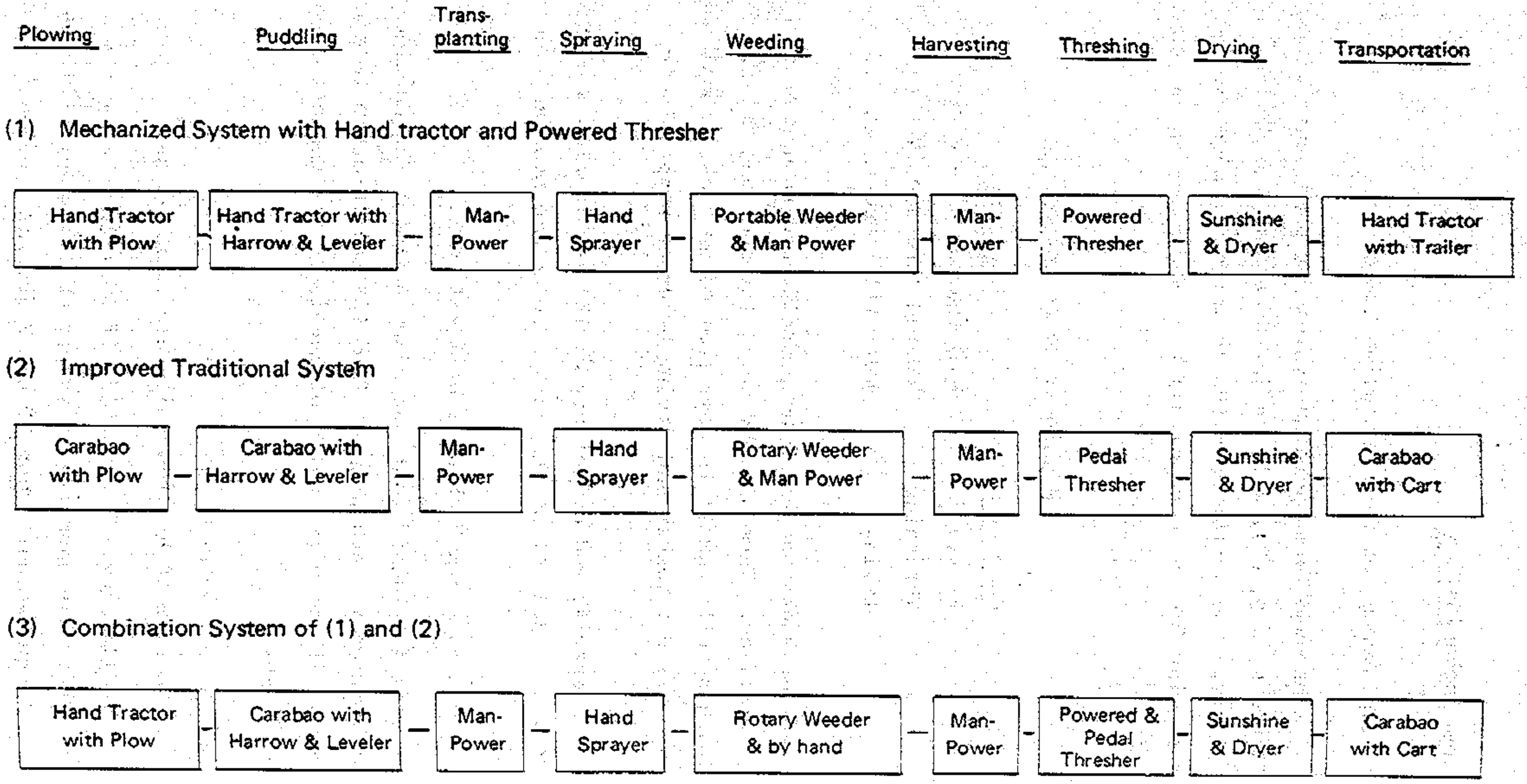
It is planned that necessary units of the machineries will be introduced in each compact farm (50 ha) in order to minimize machinery cost. The required units of machinery per compact farm are computed as follows.

(a) Hand tractor

To plow 50 ha, two units are required as;

Efficiency: 2 days/ha/unit (See Table 4C - 9)
Possible operation days per crop season: 50 days
Required units per 50 ha: 2 days/ha x 50 ha ÷ 50 days = 2 unit

FIGURE 4C-3 FARM OPERATION SYSTEM OF PADDY CULTIVATION



	Plowing	Puddling	Trans-planting	Spraying	Weeding	Harvesting	Threshing	Drying	Transportation
(1) Mechanized System with Hand tractor and Powered Thresher	Hand Tractor with Plow	Hand Tractor with Harrow & Leveler	Man-Power	Hand Sprayer	Portable Weeder & Man Power	Man-Power	Powered Thresher	Sunshine & Dryer	Hand Tractor with Trailer
(2) Improved Traditional System	Carabao with Plow	Carabao with Harrow & Leveler	Man-Power	Hand Sprayer	Rotary Weeder & Man Power	Man-Power	Pedal Thresher	Sunshine & Dryer	Carabao with Cart
(3) Combination System of (1) and (2)	Hand Tractor with Plow	Carabao with Harrow & Leveler	Man-Power	Hand Sprayer	Rotary Weeder & by hand	Man-Power	Powered & Pedal Thresher	Sunshine & Dryer	Carabao with Cart

(b) Thresher

One unit of powered thresher will deal with the amount of paddy in half area of 50 ha and four units of pedal thresher will be used in another half area as:

Powered Thresher

Efficiency: 1.2days/ha/unit (See Table 4C-9)

Possible operation days per crop season: 50days x 0.7[1/] = 35 days

Required units per 25 ha: 1.2days/ha x 25ha ÷ 35days ≑ on unit

Pedal Thresher

Efficiency: 4.0days/ha/unit (See Table 4C-9)

Possible operation day per crop season: 35 days

Required units per 25 ha: 4.0days/ha x 25ha÷35days÷0.8[2/] ≑ 4 units

Note: 1/ 0.7: The estimated ratio of possible operation days to full operation days

2/ 0.8: The estimated reduction rate of operation efficiency

(c) Dryer

To dry up 2 tons of threshed paddy in moisture content from 26% to 14%, it will take 8 houres per unit. One unit of drier will covers the area of 31 ha as follows;

Capacity: 2.0tons x two rotations per day per unit

Possible operation days: 50days x 0.66 ≑ 33 days

Area coverage: 4tons x 33days ÷ 4.2ton per ha
(Ave. yield of dry season crop) ≑ 31ha

The amount of paddy in the remaining area will be dried up in the traditional method by sunshine. But practically speaking, whole amount of the threshed paddy will be dealt with in mixture method, namely by drier and by sunshine.

(d) Working carabao

Harrowing will be performed by using carabaos in whole area. The required heads of work carabao per 50 ha. is computed as follows:

Efficiency: 1st harrowing = 4 animal days/ha
2nd harrowing = 2 animal days/ha
3rd harrowing = 3 animal days/ha
Total: 9 animal days/ha

Possible operation day: 69 days
Required heads of work carabao: 9 animal days/hax50ha÷69days≒7heads

Carabao will be utilized for other operations such as plowing in the small areas along dikes or in the corner of plots and transportation of input materials. More heads of carabao will be needed when carabao will be utilized for these operations during land-preparation period.

3. Labor Requirement and Machinery Cost

The labor requirement for the above-mentioned farm operation systems is estimated at 101 man-days per hectare as seen in Table 4C-10.

The machinery cost of the system is calculated at ₱ 194 per hectare. (See Table 4C-11)

Table 4C-9 Efficiency of Each Operation in Paddy Cultivation

Operation	Machinery or Animal	(1) Ope. width (m)	(2) Ope. Speed (km/hr)	(3) Theoretic Ope. Capacity (ha/hr)	(4) Field Efficiency (%)	(5)=(3)x(4) Field Ope. Capacity (ha/hr)	(6) Ope. Efficiency (%)	(7)=(5)x(6) Actual Ope. Capacity (ha/hr)	(8) Hours per ha (hr/ha)	(9) Ope. times (time)	(10)=(9)x(9) Hours per ha (hr/ha)	(11) Ope. hours per day (hr/day)	(12)=(9)÷(1 days per ha (day/ha)
Plowing	Hand tractor (7-8HP) with plow	0.24	4.3	0.103	84	0.087	70	0.061	16.4	1	16.4	8	2.0
1st Harrowing	Carabao with Harrow	1.0	2.5	0.250	80	0.200	65	0.130	7.7	2	15.4	8	2.0
2nd Harrowing	Carabao with Harrow	1.0	2.5	0.250	80	0.200	65	0.130	7.7	2	15.4	8	2.0
Final Harrowing & Leveling	Carabao with Harrow & leveler	0.7	2.5	0.175	80	0.140	65	0.091	11.0	2	22.0	8	2.8
Threshing	Powered thresher (7-8HP, Throw-in type)	-	-	0.222 (1.0ton)	80	0.178 (0.8ton)	75	0.134 (0.6ton)	7.5	1	7.5	6	1.2
	Pedal Thresher	-	-	0.070 (0.32ton)	80	0.056 (0.25ton)	75	0.042 (0.19ton)	24.0	1	24.0	6	4.0
Drying	Driyer (Flat bed, 2.0tons bin)	-	-	0.111 (0.50ton)	80	0.089 (0.40ton)	80	0.071 (0.32ton)	14.0	1	14.0	16	0.9
Transportation	Carabao with Cart	-	2.5	0.2ton	80	0.16ton	0.11ton	-	-	-	-	-	-

Note: The efficiency of dryer is in case of drying paddy in moisture content from 18% to 14%.

Table 4C-10 Labor Requirement of Paddy Cultivation, with Project

(Unit: day/ha)

Operation	Man-day	Machinery or Animal-day	Remarks
1. Seed-bedding			
a. Land Preparation/sowing	1.5	0.5	same as item 2
b. Care of seedlings	1.5		
Sub-total	3.0	0.5	
2. Land Preparation			
a. Plowing	2.0	2.0	by hand tractor
b. 1st Harrowing	4.0	4.0	by animal power
c. 2nd Harrowing	2.0	2.0	
d. Final Harrowing & Leveling	3.0	3.0	
e. Repair of dikes	3.0		
Sub-total	14.0	11.0	
3. Transplanting			
a. Pulling & Delivery of Seedlings	7.5	0.5	carrying by animal power
b. Transplanting	20.0		
Sub-total	27.5	0.5	straight-row planting
4. Fertilizer Application			
a. Basal application	1.0	0.2	carrying by animal power
b. Top dressing	1.0	0.2	
Sub-total	2.0	0.4	
5. Spraying			
a. Insecticides	3.0		
b. Herbicides	1.0		
Sub-total	4.0		
6. Weeding			
a. By rotary weeder	7.0		by rotary weeder
b. By hand	6.0		
Sub-total	13.0		
7. Irrigation/Drainage	5.0		
8. Harvesting			
a. Cutting/Bundling	15.0		(0.6) (2.0)
b. Hauling/Piling	2.0		
c. Threshing1/	8.5	2.6	
Sub-otal	25.5	2.6	by powered or pedal thresher
9. Post Harvesting			
a. Drying2/	3.5	2.5	by drier or sunshine
b. Sacking	2.0		
c. Piling/Delivery	1.5	0.3	carrying by animal power
Sub-total	7.0	2.8	
Total	101.0	17.8	(Machinery day: 7.2 days Animal day: 10.6 days)

Note: 1/ Each half area will be threshed by powered and pedal thresher
2/ Each half area will be dried by drier & sunshine

Table 4C-11 Farm Machinery Cost

1. Fixed Cost

Machinery	(1) Purchasing Price (₱)	(2) Durable Period (Year)	(3) Depreciation Cost[1] (₱/year)	(4) Repair Cost (₱/year)	(5) Other Fixed Cost[2] (₱/year)	(6) Total Cost (₱/year)	(7) Coverage per unit (ha)	(8) Total cost per hectare (₱)	(9) Area Coverage (%)	(10) Fixed Cost per hectare (₱)
Hand tractor	8,900	5	1,602	712 (8%)	89	2,403	25.0x2	48	100	48
Thresher	17,200[3]	8	1,935	516 (3%)	172	2,623	12.5x2	105	50	53
Pedal thresher	500	6	75	-	5	80	3.6x2	11	50	6
Dryer	7,500	8	844	375 (5%)	75	1,294	25.0x2	26	50	13
Total										120

Note: [1] Computed as (1) x 0.9 ÷ (2)

[2] Computed as (1) x 0.01

[3] Price without engine because the engine of hand tractor can be used for thresher.

2. Variable Cost

Operation	Machinery	(1) Ope. hours per ha (hr/ha)	(2) Fuel Consumption (ℓ/hr)	(3) Fuel (ℓ)	(4) Unit Cost (₱/ℓ)	(5) Cost of Fuel (₱)	(6)=(5)x1.3 Cost inclusive of oil (₱)	(7) Area Coverage (%)	(8) Variable Cost per hectare (₱)
Plowing for seedbed	Hand treactor with plow	0.1	G. 4.0	0.4	1.71	6.8	8.8	100	8.8
Plowing	Hand tractor with plow	2.0	G. 4.0	8.0	1.71	13.7	17.8	100	17.8
Threshing	Powered thresher	1.2	G. 4.0	4.8	1.71	8.2	10.7	50	5.4
	Pedal thresher	4.0		-				50	
Drying	Drier	14.1	G+O 0.75 K 1.5	G+O 10.6 K 21.2	2.20 1.30	23.3 27.7	30.3 36.0	50	33.2
	Total:								74.0

Note: G: Gasoline, O: Oil, K: Kerosin.

3. Machinery Cost per Hectare

Fixed cost + Variable cost = ₱120 + ₱74 = ₱194

FIGURE 4C-4 LABOR USE FOR ALL CROPS
(WITH PROJECT, FUTURE)

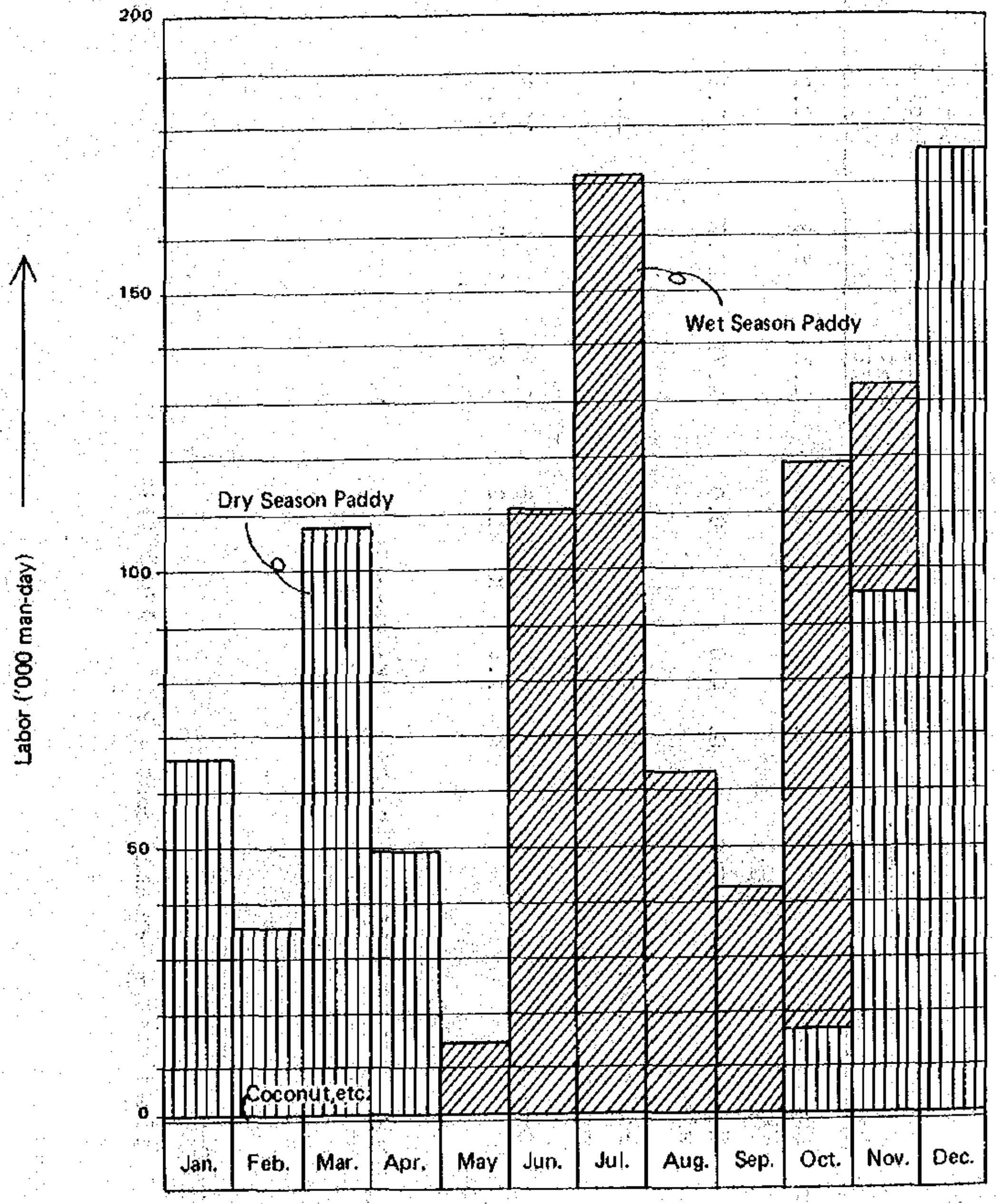


Table 4C-12 Farm Labor Balance with Project

(Unit: man-day & 1,000 man-day)

Items	Area (ha)	Jan.			Feb.			Mar.			Apr.			May			Jun.			Jul.			Aug.			Sep.			Oct.			Nov.			Dec.			Total
1. Labor Requirement per hectare (man-day)																																						
(a) Paddy																																						
- Irrigated, Wet	1															2.6	2.7	6.2	8.6	9.6	10.9	12.4	5.9	3.7	2.6	1.3	0.6	6.1	6.8	6.7	6.7	6.3	1.3					101.0
- Irrigated, Dry	-do-	5.9	3.7	2.6	1.3	0.6	4.7	6.8	6.7	6.7	6.5	2.5																			2.6	2.7	6.2	8.6	9.6	10.9	12.4	101.0
(b) Coconut, etc.	-do-	1.1	1.1	1.1	1.1	1.1	1.1	1.1	1.1	1.1	1.1	1.1	1.1	1.1	1.1	1.1	1.1	1.1	1.1	1.1	1.1	1.1	1.1	1.1	1.1	1.1	1.1	1.1	1.1	1.1	1.1	1.1	1.1	1.1	1.1	1.1	1.1	39.6
2. Total Labor Requirement (1,000 man-day)																																						
(a) Paddy																																						
- Irrigated, Wet	5,176															13.5	14.0	32.1	44.5	50.0	56.4	64.2	30.5	19.2	13.5	6.7	3.1	31.6	35.2	34.7	34.7	32.6	6.7					523.2
- Irrigated, Dry	5,320	31.4	19.7	13.8	6.9	3.2	25.0	36.2	35.6	35.6	34.6	13.3																			13.8	14.4	33.0	45.8	51.1	58.0	66.0	537.4
(b) Coconut, etc.	194	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.2	7.2
Total	10,690	31.6	19.9	14.0	7.1	3.4	25.2	36.4	35.8	35.8	34.8	13.5	0.2	0.2	0.2	13.7	14.2	32.3	44.7	50.2	56.6	64.4	30.7	19.4	13.7	6.9	3.3	31.8	35.4	34.9	48.7	47.2	39.9	46.0	51.3	58.2	66.2	1,067.8
3. Available Farm Labor Force		45.3	45.3	50.7	45.3	45.3	39.9	45.3	45.3	50.7	45.3	45.3	45.3	45.3	45.3	50.7	45.3	45.3	45.3	45.3	45.3	50.7	45.3	45.3	50.7	45.3	45.3	45.3	45.3	45.3	50.7	45.3	45.3	45.3	45.3	45.3	50.7	1,663.2
4. Balance (3 - 2)		13.7	25.4	36.7	38.2	41.9	14.7	8.9	9.5	14.9	10.5	31.8	45.1	45.1	45.1	37.0	31.1	13.0	0.6	-4.9	-11.3	-13.7	14.6	25.9	37.0	38.4	42.0	13.5	9.9	10.4	2.0	-1.9	5.4	-0.7	-6.0	-12.9	-15.5	595.4

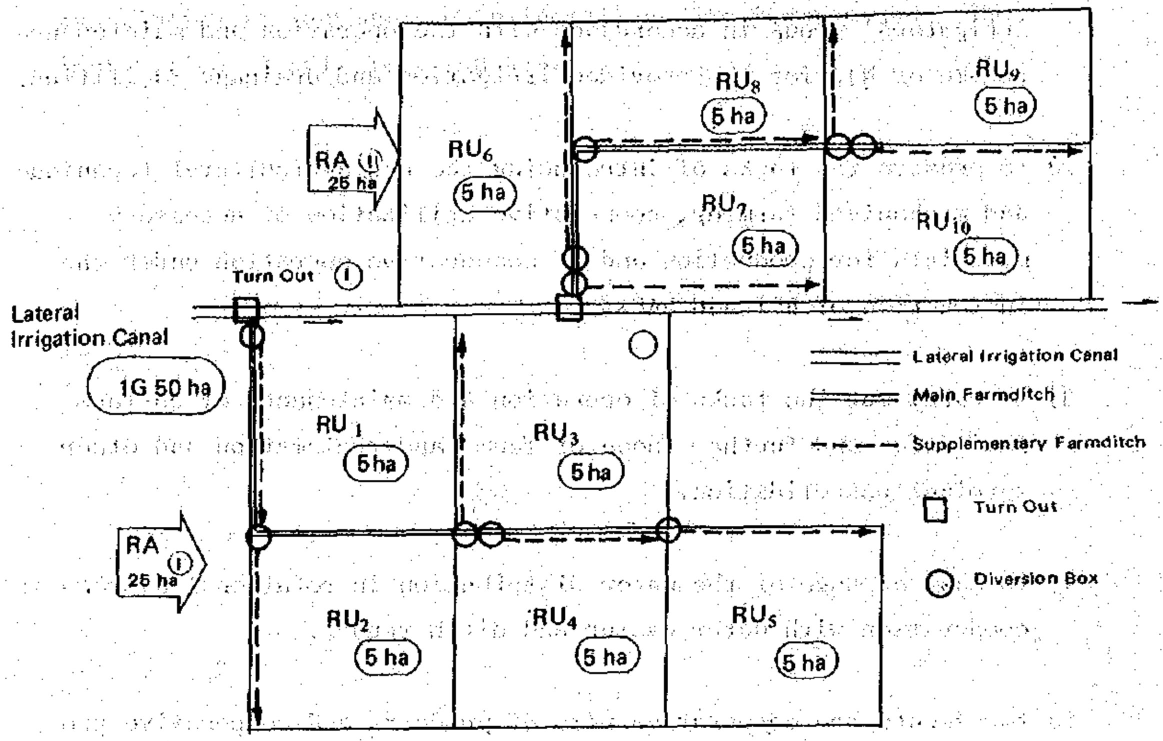
Remarks (1) Available labor forces per month are based on that as shown below
2,289 (total farm households) x 2.5 (average available labor force per farm household) x 26day x 0.95
(rate of farm labor) = 141,346 man-day (per 30 day-month)

Table 4C-13 Total Amount of Input Materials for Paddy Production with Project

Input Materials	Per Hectare			Total Amount			
	Unit	Wet	Dry	Unit	Wet (5,176ha)	Dry (5,320ha)	Total
1. Seeds	kg	45	45	ton	233	239	472
2. Fertilizer							
Urea (45 - 0 - 0)	kg	-	50	ton	-	266	266
Ammosul (21 - 0 - 0)	kg	100	100	ton	518	532	1,050
Compound (14-14-14)	kg	210	210	ton	1,087	1,117	2,204
3. Insecticides							
Liquid	quart	2.2	2.2	1,000 quart	11	12	23
Wetable Powder	kg	3.5	3.5	ton	18	19	37
4. Herbicides							
Granular	kg	25	25	ton	129	133	362

Irrigation System of On-farm Level

Diagram of irrigation system of on-farm level is illustrated as follows:



RU (Rotational Unit);

One unit of terminal irrigation area (about five hectares) served by one supplementary farm ditch. RU involves two to three farmers.

RA (Rotational Area);

Area of about 25 ha surved by one turn-out (about five rotational units). RA involves 20 to 30 farmers.

IG (Irrigators' Group);

Farmers group established within the area of about 50 ha (about two rotational areas). IG involves 25 to 30 farmers.

IA (Irrigators' Association);

Farmers group established within the area of 200 to 300 ha as same as Samahang Nayon (Barrio Association) in the Barrio level. IA involves 80 to 120 farmers.

Items to be carried out by Irrigators' Association

1) to execute the irrigated agricultural farming smoothly under the well-controled water management through the control of related irrigators' group in accordance with the operation and maintenance scheme of NIA for the provided irrigation and drainage facilities.

2) to promote the tasks of introducing the new agricultural technique and mechanized farming, cooperative utilization of necessary materials for production and of cooperative operation under the instruction of BPI and BAEx.

3) to carry out the tasks of operation and maintenance of on-farm facilities and further those of farm land reclamation and other terminal consolidation.

4) to make arrange of the water distribution in rotational systems in cooperation with water master and ditch tender.

5) to execute the cooperative sale of products and cooperative procurement of input materials under a guidance of Samahang Nayon and Kilusang Bayan.

6) to undertake the task of getting necessary fund on loan under joint garantee of the member of Irrigators' Association and Irrigators' Group to make payment more smooth.

7) to collect the water charge, repayment for project cost and administration cost for operation of Irrigators' Association.

8) to assist Samahang Nayon and Kilusang Bayan in encouragement of saving and rural development.

9) to be responsible for increase of carabao, major animal power to farm practices, so that self-sufficiency is possible within own

irrigators' Associations, importing it from other association or areas out of the Project Area.

10) to conduct the necessary training of farmes within Irrigators' Association to increase their incentives for farming under the instruction of BPI, BAEx and NIA.

Table 4C-14 Proposed Irrigators' Group in Each Barrio

			Farmers			
Municipality	Barrio	Area (ha)	Existing Farmers	Immigrated Farmers	Irrigators'Group Number to be located	Irrigators' Association Number to be located
Upper Area						
Sierra Bullones	Poblacion	130	110		9, 10, 11	
	Anibogan	22	31		8	
	Santa Cluz	36	43		8	
	Villagarucia	112	106		6, 7	2
	Salvador	52	52		5	
	San Jose	47	36	5	2	
	Bogsoc	25	41	16	1	
	Canlangit	96	87		3, 4	
	Sub-total	520	506	21	12	2
Pilar	Delpilar	18	11	2	11	
	Lumboy	108	89		12, 13	
	Sub-total	126	100	2	3	
Lower Area						
Pilar	Cagawasan	110	37	16	14, 15, 16	6
	Estaca	377	148	35	17, 18, 19, 20 21, 22, 23	
	Buenasuerte	535	132		24, 25, 26, 27, 28, 29, 30, 31, 32, 33, 34, 35	
	San Isidro	318	123	74	50, 51, 52, 53, 54, 55	
	Lasuerte	134	71		56, 57, 58	
	Sub-total	1,600	611	127	34	6
San Miguel	Mahayag	277	93	15	61, 62, 63, 64	
	San Pascuel	110	31		59, 60	
	Katipunan	704	204	9	36, 37, 38, 39, 40, 41, 42 43, 44, 45, 46, 47, 48, 49	4
	Sub-total	1,091	328	24	21	4
Dagohoy	Poblacion	97	23	24	101, 102	
	San Miguel	379	170	9	66, 67, 68, 69, 70, 71, 72	
	Malitbog	633	114	29	81, 82, 83, 84, 85, 86, 87, 88, 89, 90, 91, 92, 93	8
	Caluwasan	406	143	2	73, 74, 75, 76, 77, 78, 79, 80	
	Mahayag	253	62		103, 104, 105, 106, 107	
	Babag	341	112	8	94, 95, 96, 97, 98, 99, 100	
	Sub-total	2,109	624	72	42	8
	Total	5,320	2,069	220	107	20

Table 4C-15 Required Seeds of Rice

Description	W/O Project Cropping Area (ha)	W/O Project Seed per Hectare (ton/ha)	W/O Project Volume (A) (ton)	W/Project Cropping Area (ha)	W/Project Seed per Hectare (ton/ha)	W/Project Volume (B) (ton)	(B) - (A)
Wet Season							
Irrigated paddy	468	0.060	28	5,176	0.045	233	
Rainfed paddy	1,060	0.058	62				
Upland rice	287	0.0525	15				
Sub-total	1,815		105	5,176		233(117)1/	128
Dry Season							
Irrigated paddy	468	0.055	26	5,320	0.045	239	
Rainfed paddy	1,060	0.050	53				
Sub-total	1,528		79	5,320		239(120)	160
Total	3,343		184	10,496		472	288

Note: 1/: renewed every two crops.

Table 4C-16 Required Volume of Fertilizers

Description	Cropping Area (ha)	Fertilizer per Hectare (ton/ha)	Volume (ton)	Amount (₱ '000/ton)	Amount (₱ '000)
W/O Project					
Wet Season					
Irrigated paddy	468	0.198	93	1.5	139.5
Rainfed paddy	1,060	0.107	113	1.5	169.5
Upland rice	287	0.054	16	1.5	24.0
Sub-total	1,815		222		333.0
Dry Season					
Irrigated paddy	468	0.215	100	1.5	150.0
Rainfed paddy	1,060	0.110	117	1.5	175.5
Sub-total	1,528		217	1.5	325.5
Total (A)	3,343		439		658.5
W/Project					
Wet Season	5,176	0.310	1,605	0.412	2,133
Dry Season	5,320	0.310	1,649	0.412	2,192
Total (B)	10,496		3,254		4,325
(B) - (A)	7,153		2,815		3,666.5

Table 4C-17 Required Volume of Chemicals

Description	Cropping Area (ha)	Chemicals per Hectare (ton/ha)	Volume (ton)	Amount (₱ '000/ton)	Amount (₱ '000)
W/O Project					
Wet Season					
Irrigated paddy	468	0.011	5.0	0.125	58.5
Rainfed paddy	1,060	0.0004	0.4	0.004	4.2
Upland rice	287	-	-		-
Sub-total	1,815		5.4		62.7
Dry Season					
Irrigated paddy	468	0.0014	0.7	0.016	7.5
Rainfed paddy	1,060	0.00038	0.4	0.004	4.2
Sub-total	1,528		1.1		11.7
Total (A)	3,343		6.5		74.4
W/Project					
Wet Season	5,176	0.02975	154.0	0.333	1,773
Dry Season	5,320	0.02975	158.0	0.333	1,722
Total (B)	10,496		312.0		3,495
(B) - (A)	7,153		305.5		3,420.6

Table 4C-18 Required Number of Farm Machineries

Equipment	One Irrigators' Group	Whole Project Area	Amount (₱ '000/unit)	 (₱ '000)
Hand tractor (7 - 8 HP)	2	214	11.7	2,504
Power Thresher (7 HP)	1	107	15.2	1,626
Pedal Thresher	4	428	0.5	214
Dryer (NGA Type)	1	107	7.5	802
Total				5,146

Study on Required Rice Mill Capacity

1. Existing Rice Mill Capacity

a) Daily milling capacity (12 hours operation): 88.9ton (1,775 cav.)

b) Total milling capacity: (half year) 88.9ton x 130days = 11,557 ton

2. Required Rice Mill Capacity

a) Rice production: 5,320ha x 4.2ton/ha = 22,344ton (dry season paddy)

b) Home consumption rice in the area:
25,300 x 183kg/year x 1/2 = 2,315ton

c) Marketing volume of rice: 22,344ton - 2,315ton = 20,029ton

d) Marketing volume of paddy (20%):
20,029ton x 0.2 = 4,006ton

e) Milling volume by Ubay Grain Center (10%):
(20,029 - 4,006ton) x 0.1 = 1,602ton

f) Milling volume by newly installed rice mill in the area
20,029 - 4,006 - 1,602 = 14,421ton

g) Total milling volume by newly installed rice mill (required rice mill capacity)
14,421 + 2,315 - 11,557ton = 5,179ton

h) Required unit of rice mill
5,179ton/10ton 1/ x 130days ≑ 4 units

Note: 1/ milling capacity 10ton (39HP, 200 cav/12hr)

Table 4C-19 Disbursement Schedule of Supporting Services

(Unit: ₱ '000)

Description	Year												Total	Remarks
	1979	1980	1981	1982	1983	1984	1985	1986	1987	1988	1989	1990		
1. Schedule of on-farm development (ha)				1,365	1,392	1,598	965						5,320	
2. Cropping Schedule (ha)														
Wet Season					1,365	2,757	2,757	3,955	5,176	5,176	5,176	5,176		
Dry Season					1,365	2,757	4,355	5,320	5,320	5,320	5,320	5,320		
3. Required cost for establishment of Irrigators' Association (₱ '000)														
Preparation of cadastral map		145	148	169	103								565	₱106.2/ha
Establishment of Irrigators' Group		14	14	16	10								54	₱500/Irrigators' Group
Construction of Working station				150	300	150	150						750	3 Kilusang Bayan x ₱250,000
Operation cost of development center	15	15	15	15	15	15	15	15	15	15			150	
Total	15	174	177	350	428	165	165	165	15	15			1,519	
4. Required cost for Supporting Services(₱ '000)														
Establishment of Samahang Nayon			3	2	2								7	
Establishment of Kilusang Bayan			100	100	100								300	
Facilities of Kilusang Bayan				1,511	1,541	1,770	1,068						5,890	
Sub-total			103	1,613	1,643	1,770	1,068						6,197	₱1,165/ha ≒ US$155/ha
Supply of seed by BPI														
Wet season					21	41	41	59	78	78	78	78	474	Cropping area x 45kg/ha x 1/4 x ₱60/45kg
Dry season					21	41	65	80	80	80	80	80	527	
Supply of fertilizer														
Wet season					393	851	1,021	1,481	1,883	2,033	2,133	2,133	11,928	₱412/ha
Dry season					475	1,027	1,708	2,342	2,548	2,644	2,644	2,644	16,032	₱497/ha
Supply of Pesticide														
Wet season					455	918	918	1,317	1,724	1,724	1,724	1,724	10,504	₱333/ha-year
Dry season					455	918	1,450	1,772	1,772	1,772	1,772	1,772	11,683	
Delivery of farm mashinery				1,321	1,347	1,547	932						5,147	
Purchase of paddy by Kilusang Bayan														
Wet season					1,690	3,380	3,380	4,810	6,300	6,300	6,300	6,300	38,460	
Dry season					1,690	3,380	5,330	6,500	6,500	6,500	6,500	6,500	42,900	
Sub-total				1,321	6,547	12,103	14,845	18,361	20,885	21,131	21,231	21,231	137,655	
Community development				480	489	562	339						1,870	
Total			103	3,414	8,679	14,435	16,252	18,361	20,885	21,131	21,231	21,231	145,722	

Proposed Dam and Reservoir

A. Investigation Performed

1. Surface Geological Investigations
2. Bore-Hole Drillings
3. Test Pits
4. Soil Tests
5. Rock Tests

B. Technical Supporting

1. Geology of Damsite and Reservoir
2. Dam Type
3. Freeboard and Dam Crest Elevation
4. Stability Analysis
5. Spillway
6. Diversion Facilities
7. Countermeasure for Leakage from Dam Abutment
8. Seismicity

A. Investigations Performed

The following investigations at the considered storage sites of Pamacsalan and Wahig have been implemented up to date.

Pamacsalan storage site; —

- Surface geology for the damsite and reservoir area — Geological map.
- Bore-hole drillings for the damsite — Bore-hole logs.
- Test pits excavation for the borrow areas — Test pit profiles.
- Soil test for the embankment material — Table of soil tests.
- Rock test for the embankment material — Table of rock tests.

Wahig storage site; —

- Surface geology for the damsite and reservoir area — Geological map.
- Bore-hole drillings for the damsite — Bore-hole logs.
- Test pits excavation for the borrow areas — Test pit profiles.
- Soil test for the embankment material — Table of soil tests.
- Rock test for the embankment material — Table of rock tests.

Moreover, the basic concrete aggregate tests were carried out during the feasibility study.

Brief descriptions of the investigation executed are as follows:

1. Surface Geological Investigations

Geological maps of the Pamacsalan and Wahig damsite with location of the Karstism marks and explorations which have been prepared based upon the field reconnaissance survey with a 1:4,000 topographical maps as well as the results of bore-hole drillings, are shown in Figure 4D-1 and 4D-2 respectively.

Surface geological investigations covering the reservoir areas at both dam sites were carried out by NIA Geologists with topographical maps (1:4,000 and 5 meters contours).

Main findings and some considerations obtained through the field reconnaissance survey and geological maps are described in the Technical Supporting in this Appendix on Dam and Reservoir.

FIGURE 4D-1

GEOLOGICAL MAP OF PAMACSALAN RESERVOIR AREA

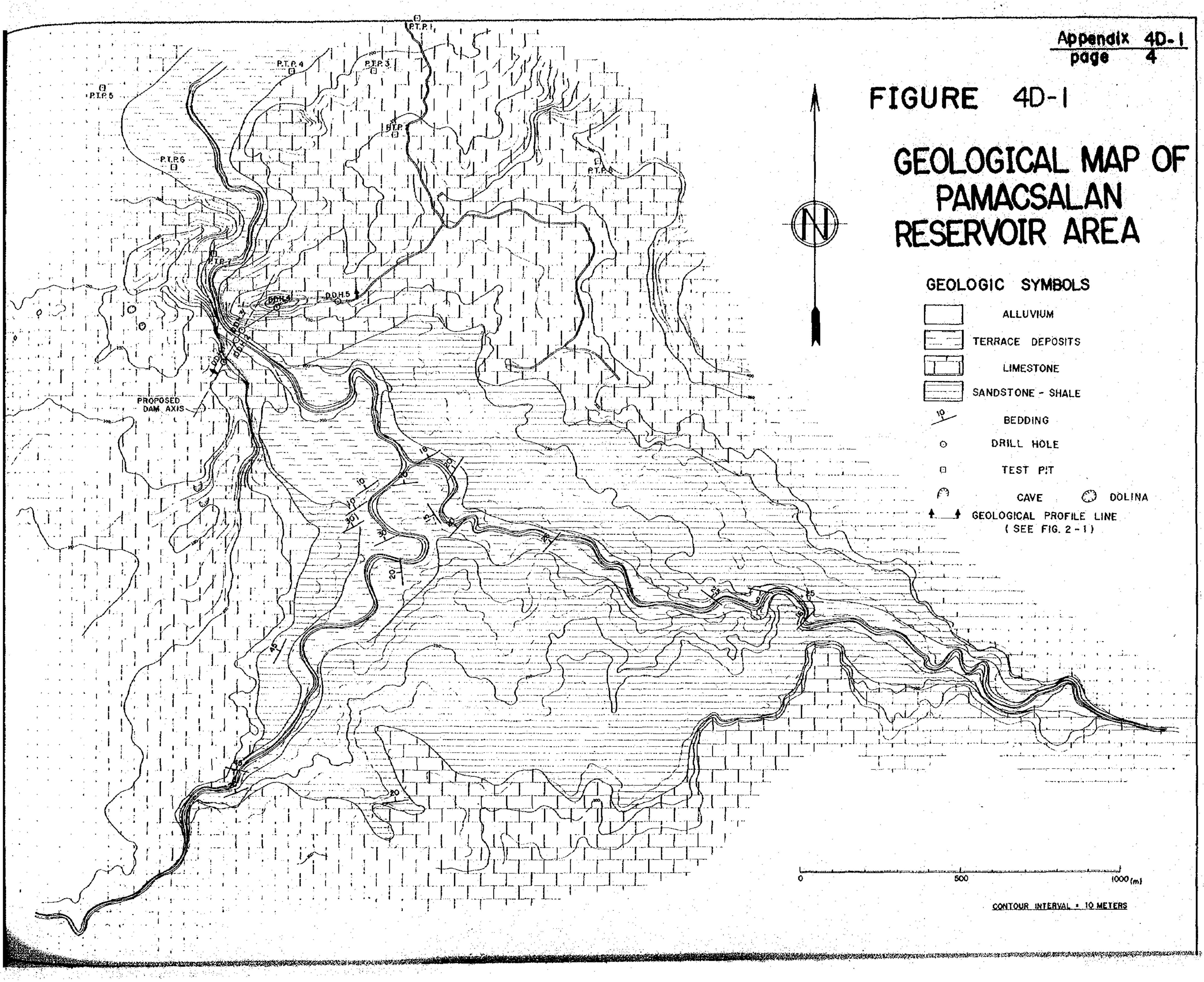


FIGURE 4D-2

GEOLOGICAL MAP WAHIG RESERVOIR

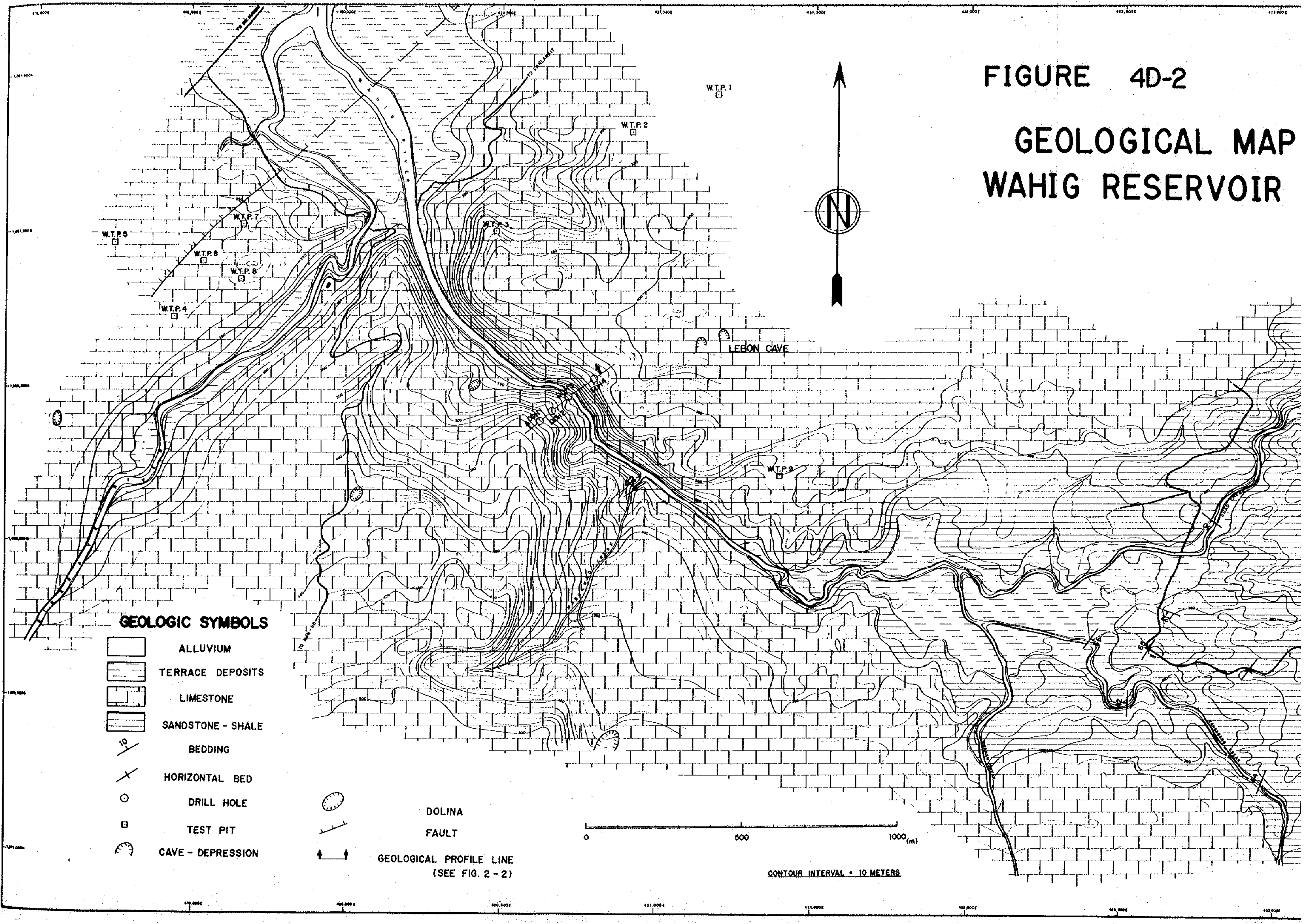
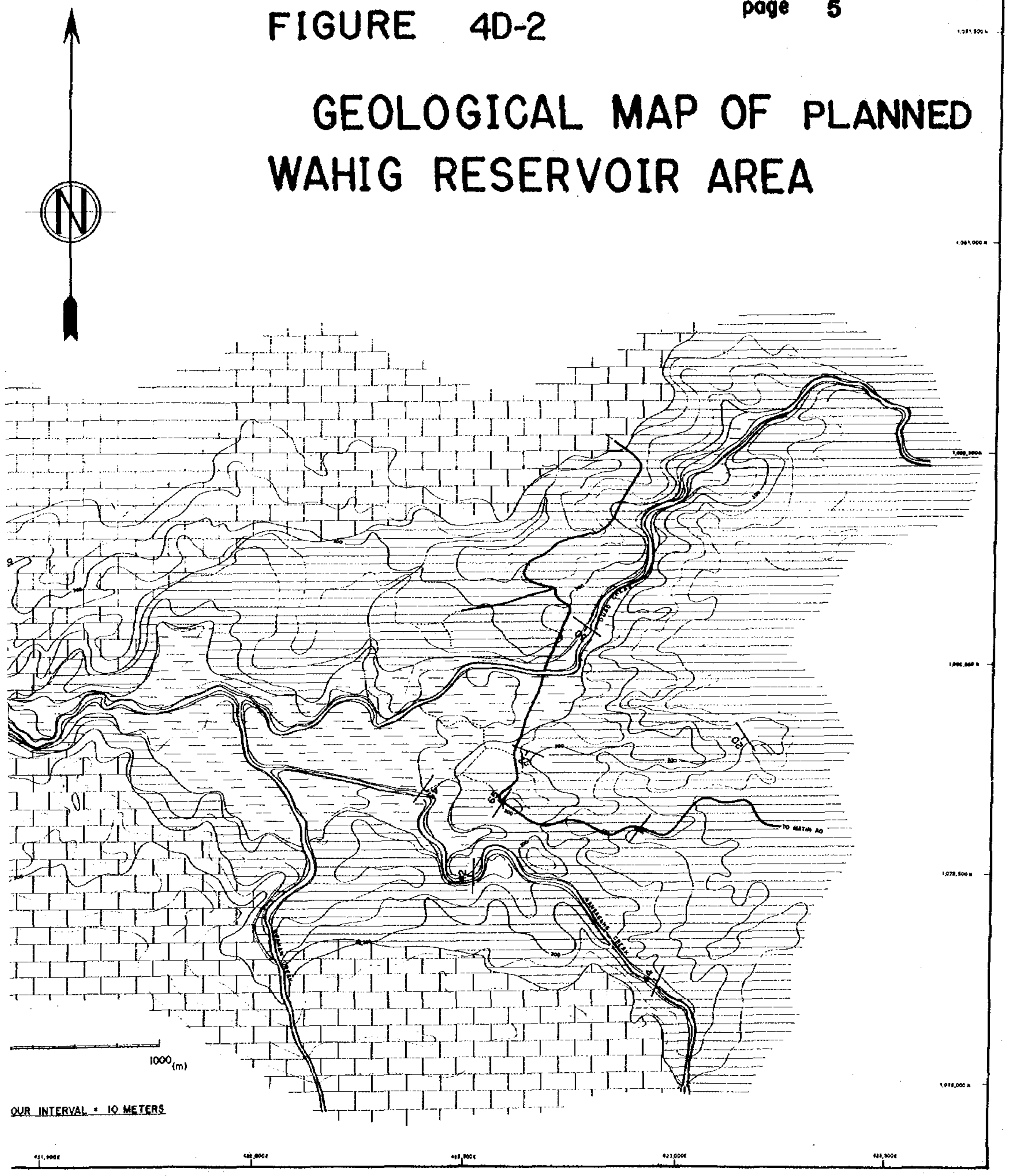


FIGURE 4D-2

GEOLOGICAL MAP OF PLANNED WAHIG RESERVOIR AREA



2. Bore-Hole Drillings

(a) Pamacsalan Damsite

5 bore-holes in total have been drilled during the feasibility study by NIA, as shown in the following table.

Location	Hole Number	Drilled Length (m)	Remarks
Left Abutment	P. DDH 1	60.0	Vertical
River Bed	P. DDH 2	60.0	Inclined
River Bed	P. DDH 3	60.0	Inclined
Right Abutment	P. DDH 4	40.0	Vertical
Right Saddle	P. DDH 5	70.0	Vertical

Location of the bore-holes and each log are shown on Figures 4D-3 and 4D-4 (Geological Profile), respectively.

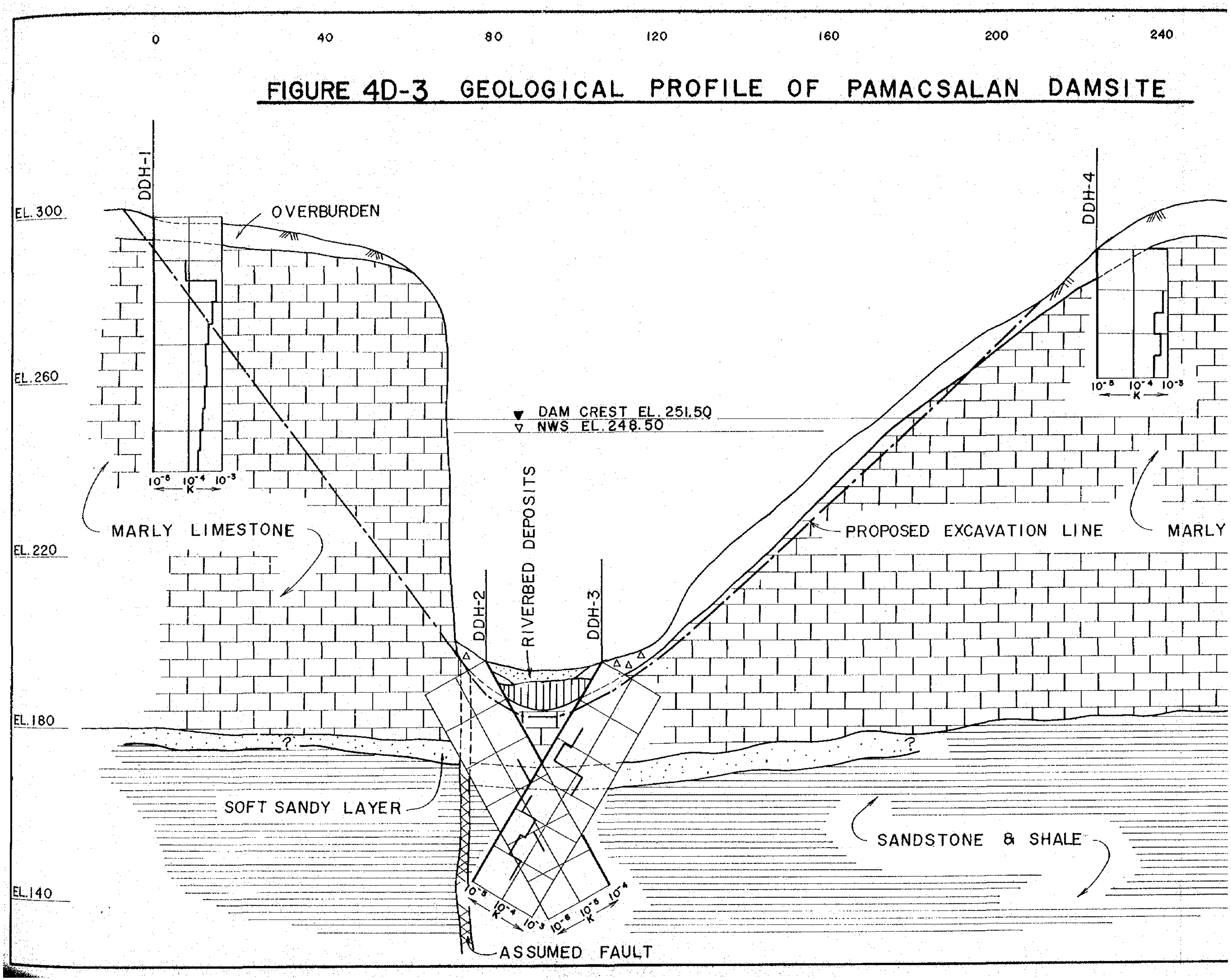
(b) Wahig Damsite

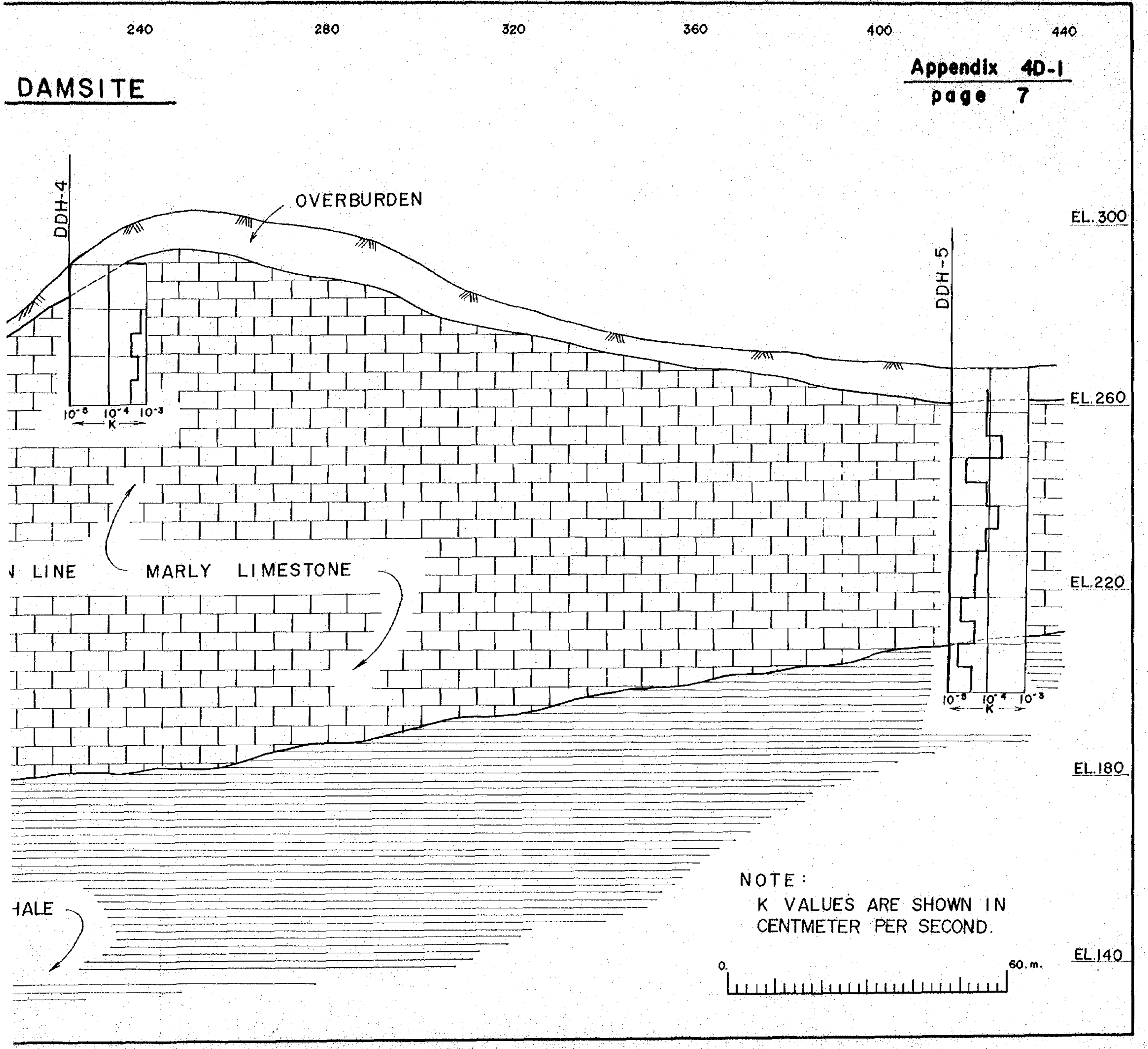
4 bore-holes in total as shown in the following table were drilled to data by NIA.

Location	Hole Number	Drilled Length (m)	Remarks
Left Abutment	W. DDH 1	40.0	Vertical
Left Abutment	W. DDH 2	35.0	Vertical
River Bed	W. DDH 3	40.0	Vertical
Right Abutment	W. DDH 4	35.0	Vertical

Location of all bore-holes and each log are indicated in the Figures 4D-5 and 4D-6 (Geological Profile), respectively.

FIGURE 4D-3 GEOLOGICAL PROFILE OF PAMACSALAN DAMSITE

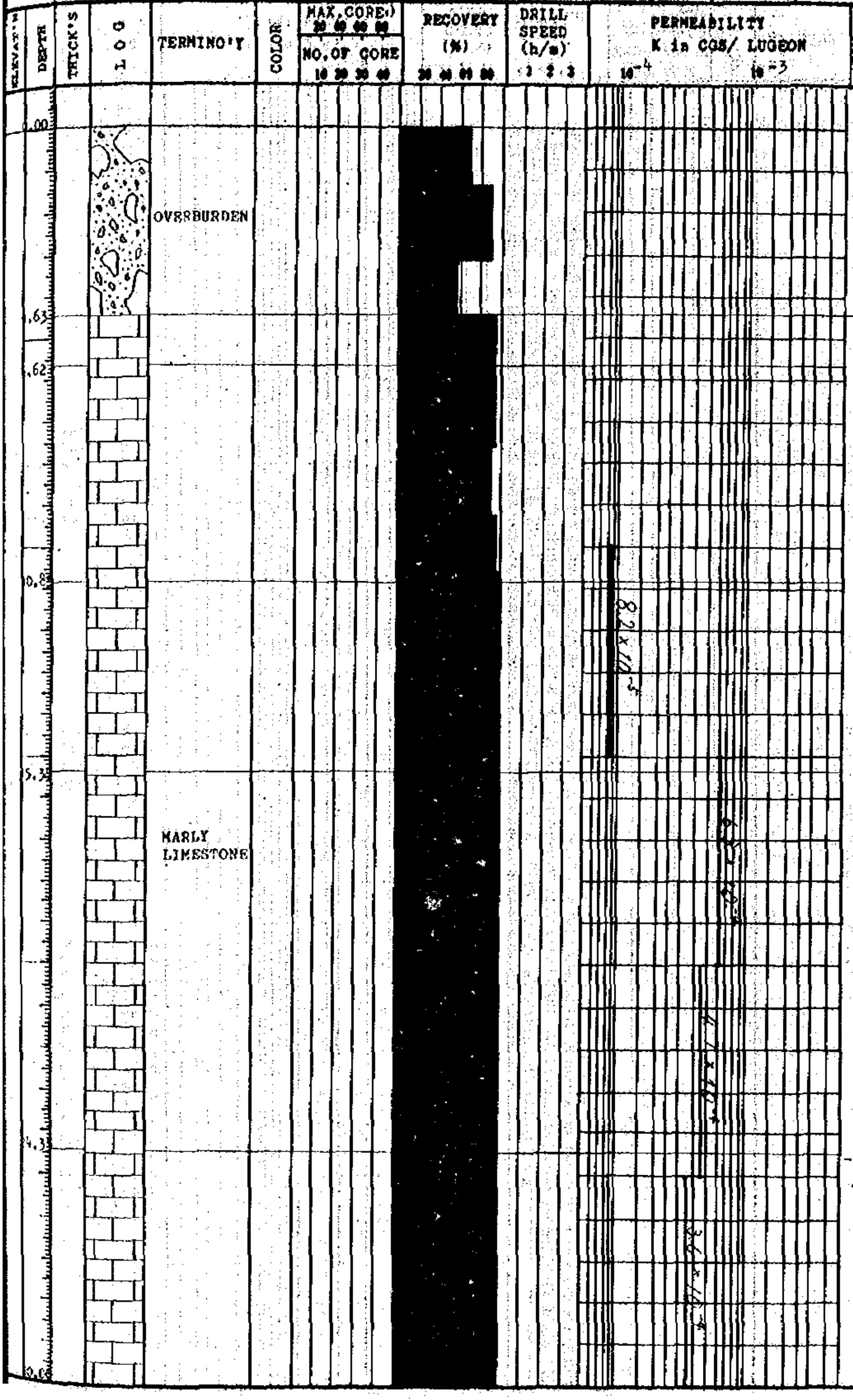




NOTE :
K VALUES ARE SHOWN IN
CENTMETER PER SECOND.

B O R E H O L E L O G

JECT	BOHOL I.A.D. PROJECT						SITE	PAMACSALAN DAMSITE		
H-1	ELEVATION		ANGLE	VERTICAL	MACHINE	ACKER	BEGUN	10/16/77	SITE ENGINEER	
	DEPTH	60.00 M	BIT		PUMP		COMPLETED	10/26/77	FOREMAN	
	DIAMETER	BQ	GW LEVEL		ENGINE		DAYS REQUIRED		DRILLER	



DESCRIPTIONS

0.00-4.63 OVERBURDEN
Highly fractured formation of limestone. Fractures are open and rusty. Extremely to moderately weathered. Buff in color, moderately soft to moderately hard. Core sizes varie from pebble to cobbles

4.63-5.62
MARLY LIMESTONE
Fresh, hard, light gray in color. Fracturing is open and rusty. Shale content approx. 5-10%.

5.62-10.85 -DO-
Maximum length of core is 50cms and the minimum is 1.5cms. At 6.18 meters depths, approx. 1ft length brecciated core recovery. Shale composition approx. 5-10%.

10.85-15.35-DO-
Fresh, light gray in color, hard and competent. Approx. 5% shale and 5-10% SST composition. But as if goes deeper, the SST composition is less and shale is becoming higher in percentage. At 15.35 meters depth, patchable clayey materials possible fracture fillings.

15.35-24.35-DO-
Buff in color, SW to MW AT 19.00-24.75 meters depth, badly fractured and show open cavities. Maximum length of core 3.5cms and the minimum 3cms, forming an average of 15-20cms, minite voids are distributed this section.

-Do-but fresh to SW. At 26.25-27.35 meters depth, badly fractured possible passage of water.

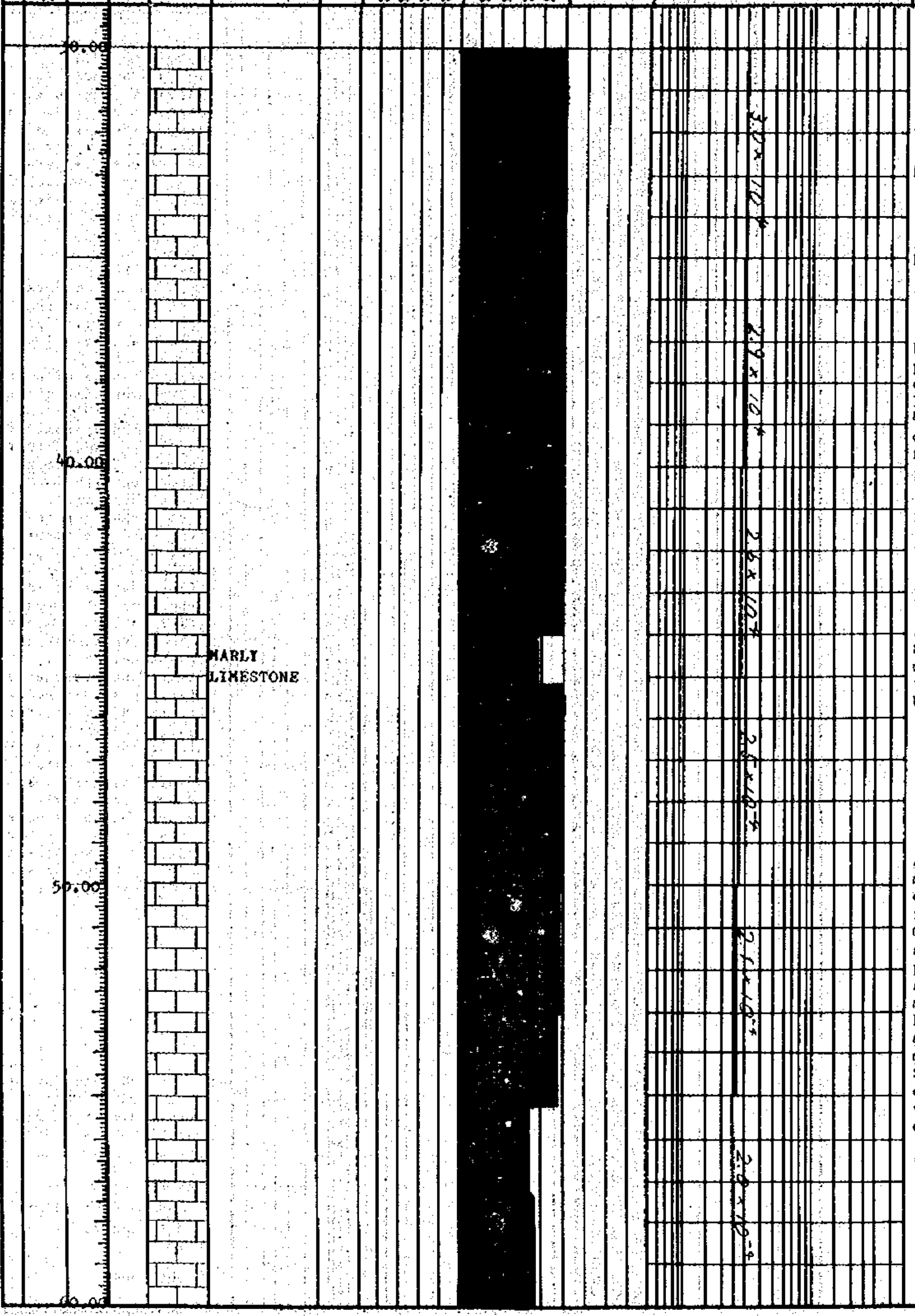
Some rock type but as if goes deeper, color tend to become reddish brown.

(continues)

BOREHOLE LOG

PROJECT	BOHOL I.A.D. PROJECT							**SITE**	PAMACSALAN DAMSITE		
HOLE NO. DDH-1	**ELEVATION**		**ANGLE**	VERTICAL	**MACHINE**	ACKER		**BEGUN**	10/16/77	**SITE ENGINEER**	
	DEPTH	60.00 M	**BIT**		**PUMP**			**COMPLETED**	10/26/77	**FOREMAN**	
	DIAMETER	BQ	**GW LEVEL**		**ENGINE**			**DAYS REQUIRED**		**DRILLER**	

DATE	**ELEVAT'N**	**DEPTH**	**THICK'S**	**LOG**	**TERMINO'Y**	**COLOR**	**MAX. CORE(cm)** 20 40 60 80 **NO. OF CORE** 10 20 30 40	**RECOVERY (%)** 20 40 60 80	**DRILL SPEED (h/m)** 1 2 3	**PERMEABILITY K in CGS/ LUGEON** 10^{-4} 10^{-3}	**DESCRIPTIONS**



(continued)

Do.

Do.

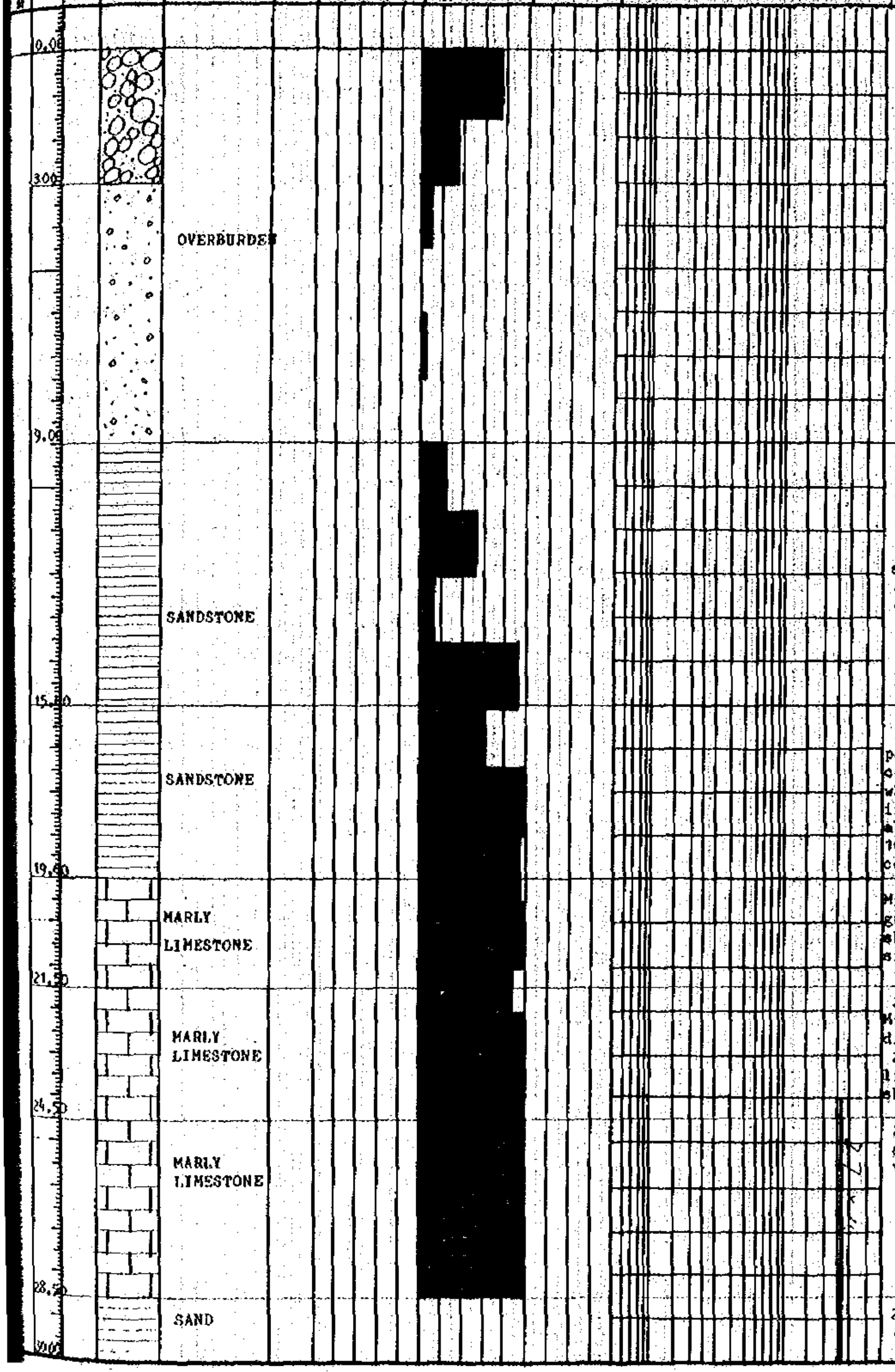
Do but fresh and presence of minite cavities/voids espe ly at section 42.00 43.95 meters. Shale composition varies from 5-10%.

SW-MW but at 48.00 meters depth, appro 15cms length EW pos ble passage of wate

At section 49.85-5 meters, shows a goug charactristic, possi fault? or sheared Gray in color, whe shows plasticity presence of brecci At 53.57 and 55.40 meters depth, fract EW and SW respecti and at section 57. meters down to bot SW but plenty of cavities/voids. Si varies from 0.3 to cm in diameter.

B O R E H O L E   L O G

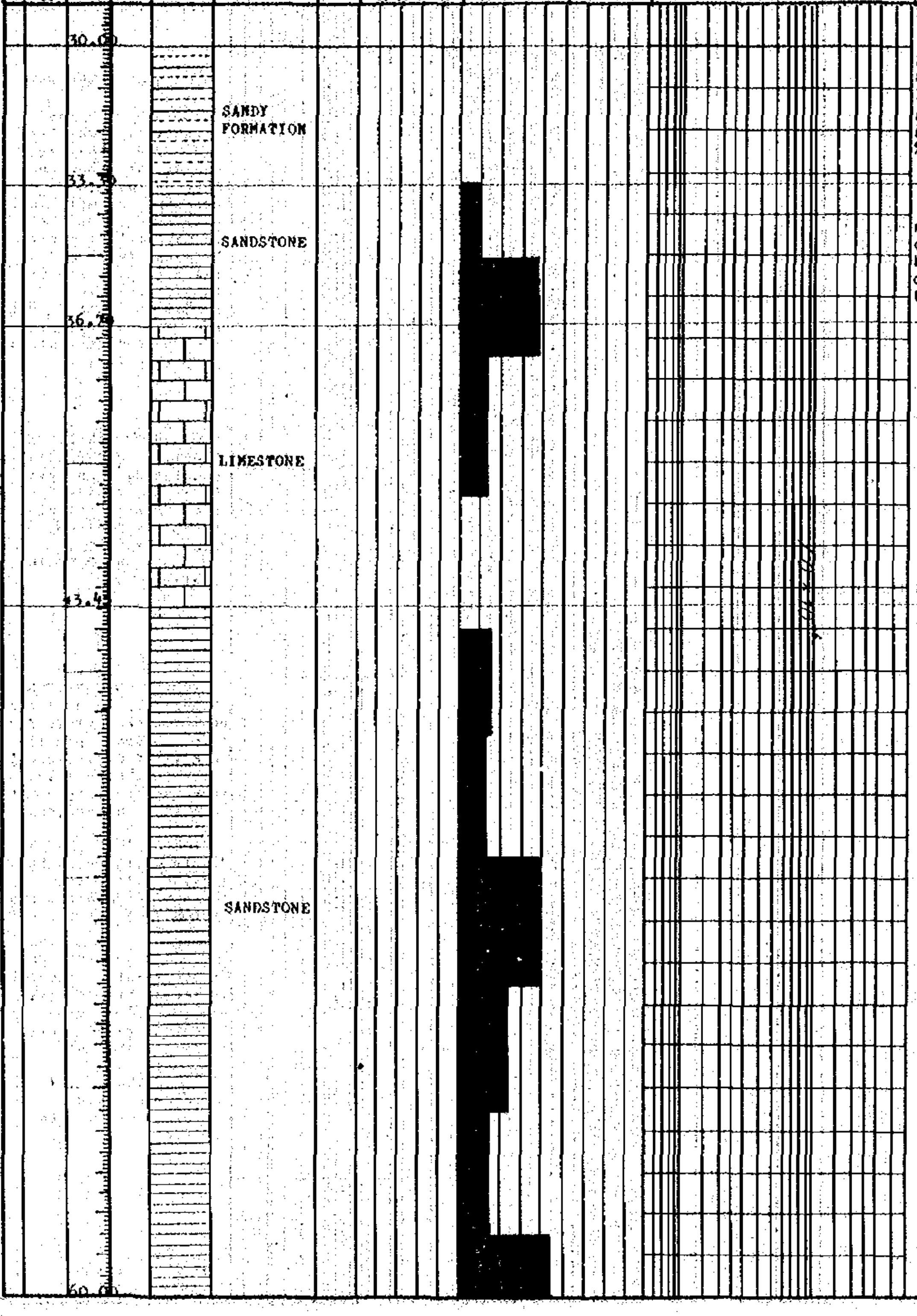
OJECT	BOHOL I.A.D. PROJECT						SITE	PAMACSALAN DAMSITE		
BOH-2	ELEVATION		ANGLE	30 Degree	MACHINE	ACKER	BEGUN	9/16/77	SITE ENGINEER	
	DEPTH	60.00 M	BIT		PUMP		COMPLETED	17/10/77	FOREMAN	
	DIAMETER	NQ	GW LEVEL		ENGINE		DAYS REQUIRED		DRILLER	



ELEVAT'N	DEPTH	THICK'S	LOG	TERMINO'Y	COLOR	MAX.CORE() / NO.OF CORE	RECOVERY (%)	DRILL SPEED (h/m)	PERMEABILITY K in CGS/ LUGEON	DESCRIPTIONS
	0.00									
				OVERBURDEN						0.00-9.00 OVERBURDEN 0.00-3.00 Composed chiefly of moderately weathered cobble to boulder of limestone, buff in color, hard & presence of small cavities/voids, iron stain is noticable especially along fractured plannes, presence of fossil inclusion. 3.00-9.00 Sandy formation recovered as sledge, fine to medium grained grey in color.
	3.00									
	9.00			SANDSTONE						9.00-15.00 SANDSTONE Poorly to moderately compacted, light grey in color, finegrained with fossil inclusion, friable & does not show plasticity, approx.5 to 10 % $CaCO_3$ content.
	15.00			SANDSTONE						15.00-19.00 SANDSTONE Moderately to well compacted, dark grey to dark color, moderate plastic when wet, friable, fossil inclusion, hader than above section, appro.5 to 10 % lime content, max. core length 20 cm.
	19.00			MARLY LIMESTONE						19.00-21.50 LIMESTONE Minute cavities, light grey, hard, plastic, shale & sandstone composition 25 %, fossiliferous
	21.50			MARLY LIMESTONE						21.50-24.50 LIMESTONE Minute veinlet of calcite different directions. 24.00-24.50 m shows a limestone phenocryst, shale/sst are matrix .
	24.50			MARLY LIMESTONE						24.50-28.50 - do - absence of minute veinlets of calcite.
	28.50			SAND						28.50-33.30 SANDY FORMATION (Continues)
	30.00									

BOREHOLE LOG

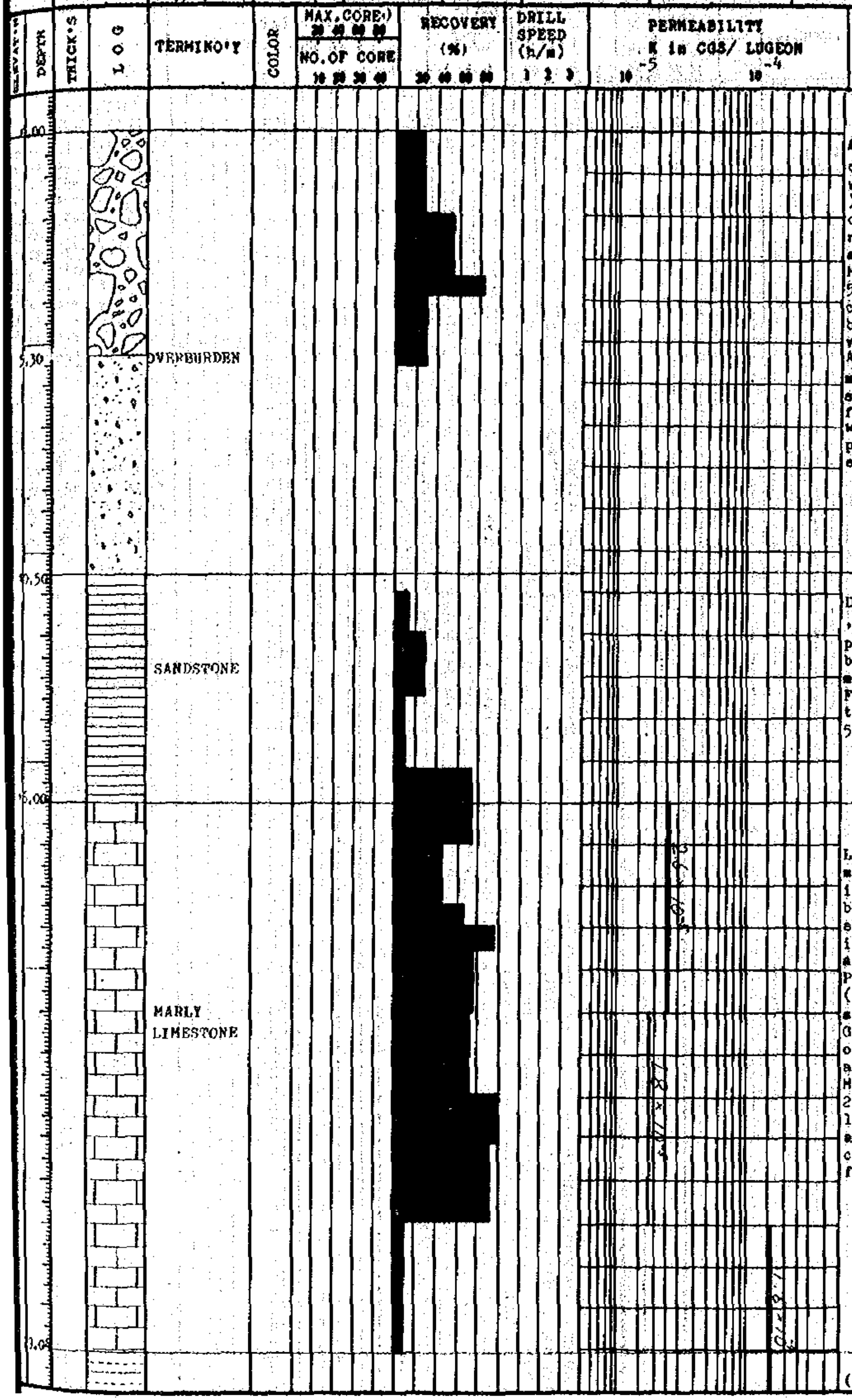
PROJECT	BOHOL I.A.D. PROJECT					SITE	PAMACSALAN DAMSITE		
HOLE NO.	DDH-2	ELEVATION		ANGLE		MACHINE		BEGUN		SITE ENGINEER	
		DEPTH		BIT		PUMP		COMPLETED		FOREMAN	
		DIAMETER		GW LEVEL		ENGINE		DAYS REQUIRED		DRILLER	



DATE	ELEVAT'N	DEPTH	THICK'S	LOG	TERMINO'Y	COLOR	MAX.CORE) 20 40 60 80 NO.OF CORE 10 20 30 40	RECOVERY (%) 20 40 60 80	DRILL SPEED (h/m) 1 2 3	PERMEABILITY K in COS/ LUGEON 10^{-6} 10^{-5}	DESCRIPTIONS
		30.00									(Continued)
		33.30			SANDY FORMATION						28.50-33.30 SANDY FORMATION Recovered as sledg grey, fine to mediu grained, more or les % $CaCO_3$ composition
		36.70			SANDSTONE						33.30-36.70 SANDS Well compacted, fin medium grained, dark color, deeper portio becomes harder, mor compacted, friable, p tic, fossiliferous.
		43.45			LIMESTONE						36.70-43.45 LIMES Light grey color, f to medium grained, and compacted, soli crystal growth alo open cavities, gene apperence is simil to sandstone, max. length of core is cm.
		60.00			SANDSTONE						43.45-60.00 SANDS Same rock facies section 33.3-36.7 but harder, appro. to 20 % $CaCO_3$ con deeper portion be harder and more co ted,

BOREHOLE LOG

JECT	BOHOL I.A.D. PROJECT						SITE	PAMACSALAN DAMSITE		
	ELEVATION		ANGLE	30 Degree	MACHINE		BEGUN	9/12/77	SITE ENGINEER	
N-3	DEPTH	60.00 M	BIT		PUMP		COMPLETED	10/15/77	FOREMAN	
	DIAMETER	NQ,BQ	GW LEVEL		ENGINE		DAYS REQUIRED		DRILLER	



DESCRIPTIONS

0.00-10.50
OVERBURDEN

At 0.00-5.30m, composed chiefly of moderately weathered (cobble-boulder) limestone. Buff in color, iron staining is noteciable especially along fracture plaines. Maximum length of core 50 cms. The minimum is cobble sizes. Hard rocks but presence of minute voids.

At 5.30-10.50m, sandy formation recovered as slude, light gray in color, fine to medium grained with some inclusion of pebble sizes of limestone.

10.50-16.00 SANDSTONE

Dark gray to dark in color, poorly indurated/compacted. Friable when dry, but when wet shows a moderate plasticity. Fossils is present. Fine to medium grained approx. 5-10% $CaCO_3$ component.

16.00-29.04 LIMESTONE

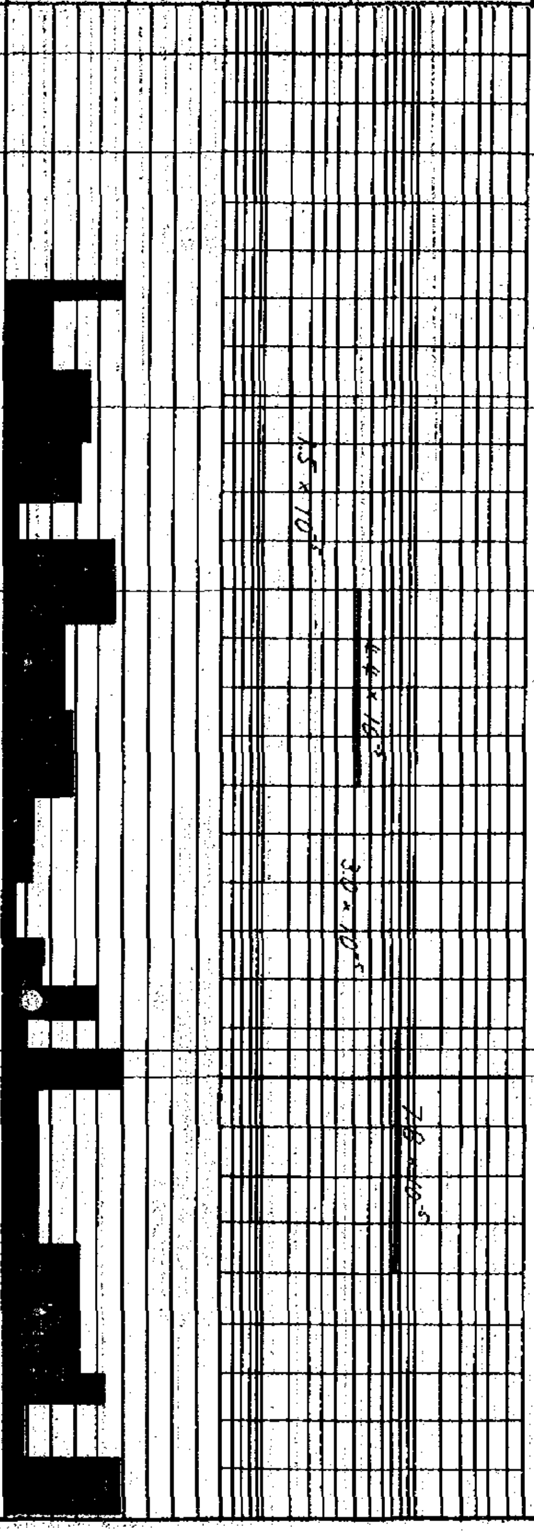
Light gray in color, moderately soft but as if goes deeper tend to become harder when wet shows a moderate plasticity. Approx. 20-25% shale and 5-10% SST contents. Patiches of white color (calcite) especially at section 22.00-22.50M. Going deeper percentage of shale is decreasing and SST is becoming more. Maximum length of core 24cms. and the minimum length of core 24 cms. and the minimum is cobb cobble sizes few fossils present.

29.04-32.04
(continues)

BOREHOLE LOG

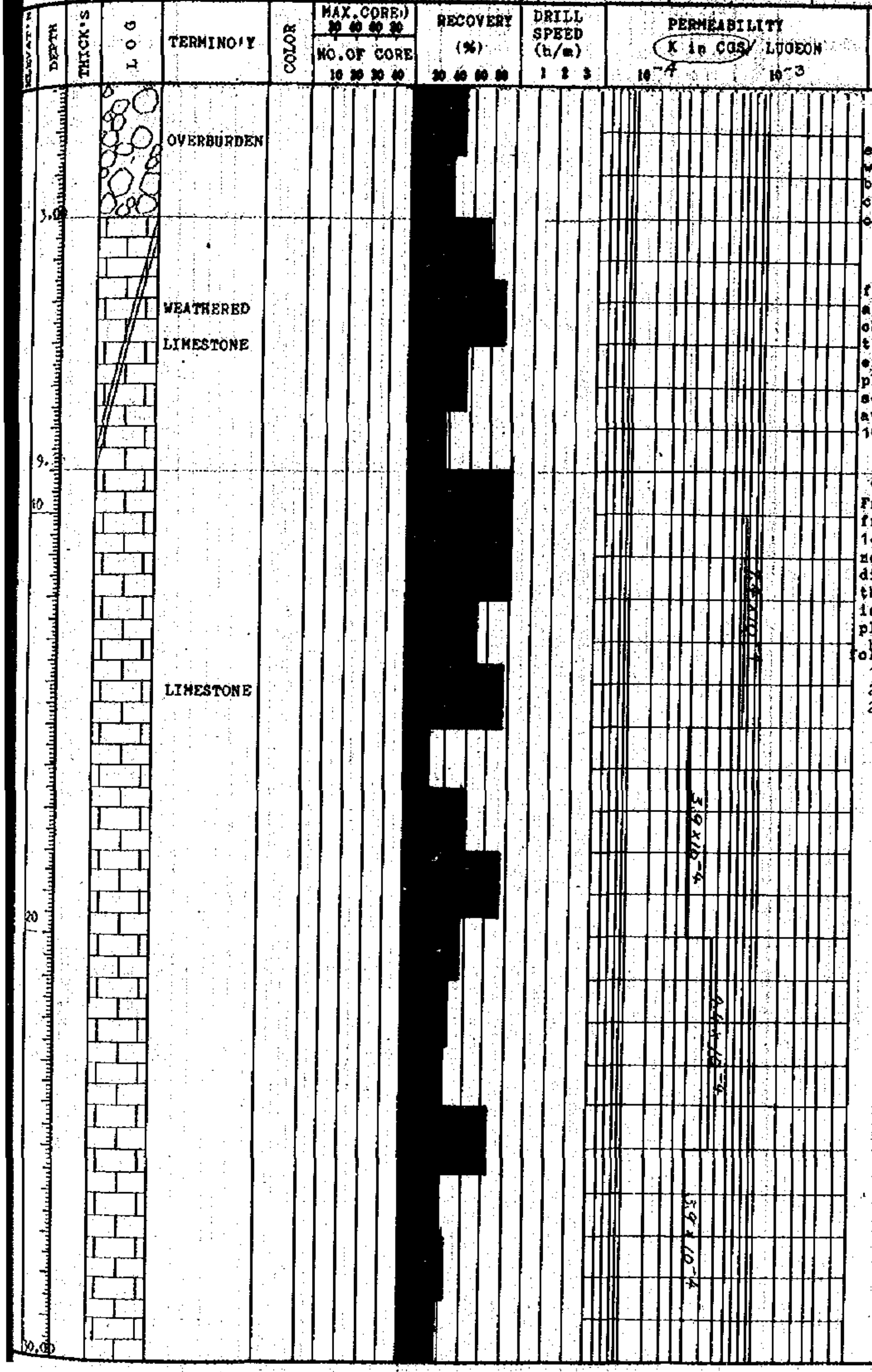
PROJECT		BOHOL I.A.D. PROJECT						SITE	PAMACSALAN DAMESITE	
HOLE NO.	DDH-3	ELEVATION		ANGLE	30 Degree	MACHINE	TONK	BEGUN	9/12/77	SITE ENGINEER
		DEPTH	60.00 M	BIT		PUMP		COMPLETED	9/15/77	FOREMAN
		DIAMETER	NQ,BQ	GW LEVEL		ENGINE		DAYS REQUIRED		DRILLER

DATE	ELEVAT'N	DEPTH	THICK'S	LOG	TERMINO'Y	COLOR	MAX.CORE 20 40 60 80 / NO.OF CORE 10 20 30 40	RECOVERY (%) 20 40 60 80	DRILL SPEED (h/m) 1 2 3	PERMEABILITY K in CGS/ LUGEON 10^{-5} 10^{-4}	DESCRIPTIONS
											(continued)
		30.0			SANDY FORMATION						SANDY FORMATION Recovered as sludg light gray in colo fine to medium gra Approx. more or le % CaCO3 compositio
		32.0			SANDSTONE						32.04-37.34 SANDS Moderately compac fine to medium gra dark gray in color when wet shows a plasticity, soft a when dry friable, fossils content.
		37.3			LIMESTONE					1.5 × 10^{-5}	37.34-41.00 LIMES Appearance is simi from a sandstone,l gray in color,fine medium grained,har competent. Maximum length of core 30c
		41.0			SANDSTONE					4.4 × 10^{-5} 3.0 × 10^{-5}	41.00-50.40 SANDS Approx. 5-10% CaCO content,and as if deeper becoming mo compacted. From 36.34m. dept Reduced hole from NQ to BQ because conebarrel can'nt penetrate down du to the loose form encountered.
		50.40			LIMESTONE						50.40-51.00 LIMES
		51.00			SANDSTONE					7.8 × 10^{-5}	51.00-60.00 SAND Same rock type as section 32.04-37.3 41.00-50.65m.,but Approx. 15-20% Ca content, and as if deeper tends to b compacted and har



B O R E H O L E   L O G

JECT	BOKOL IAD PROJECT						SITE	PANAOSALAN DAMSITE		
	ELEVATION		ANGLE	vertical	MACHINE	TONE	BEGUN	7/10/77	SITE ENGINEER	
22.4	DEPTH	30.00 M	BIT		PUMP		COMPLETED	29/10/77	FOREMAN	
	DIAMETER	NW/BW	GW LEVEL		ENGINE		DAYS REQUIRED	23	DRILLER	



DESCRIPTIONS

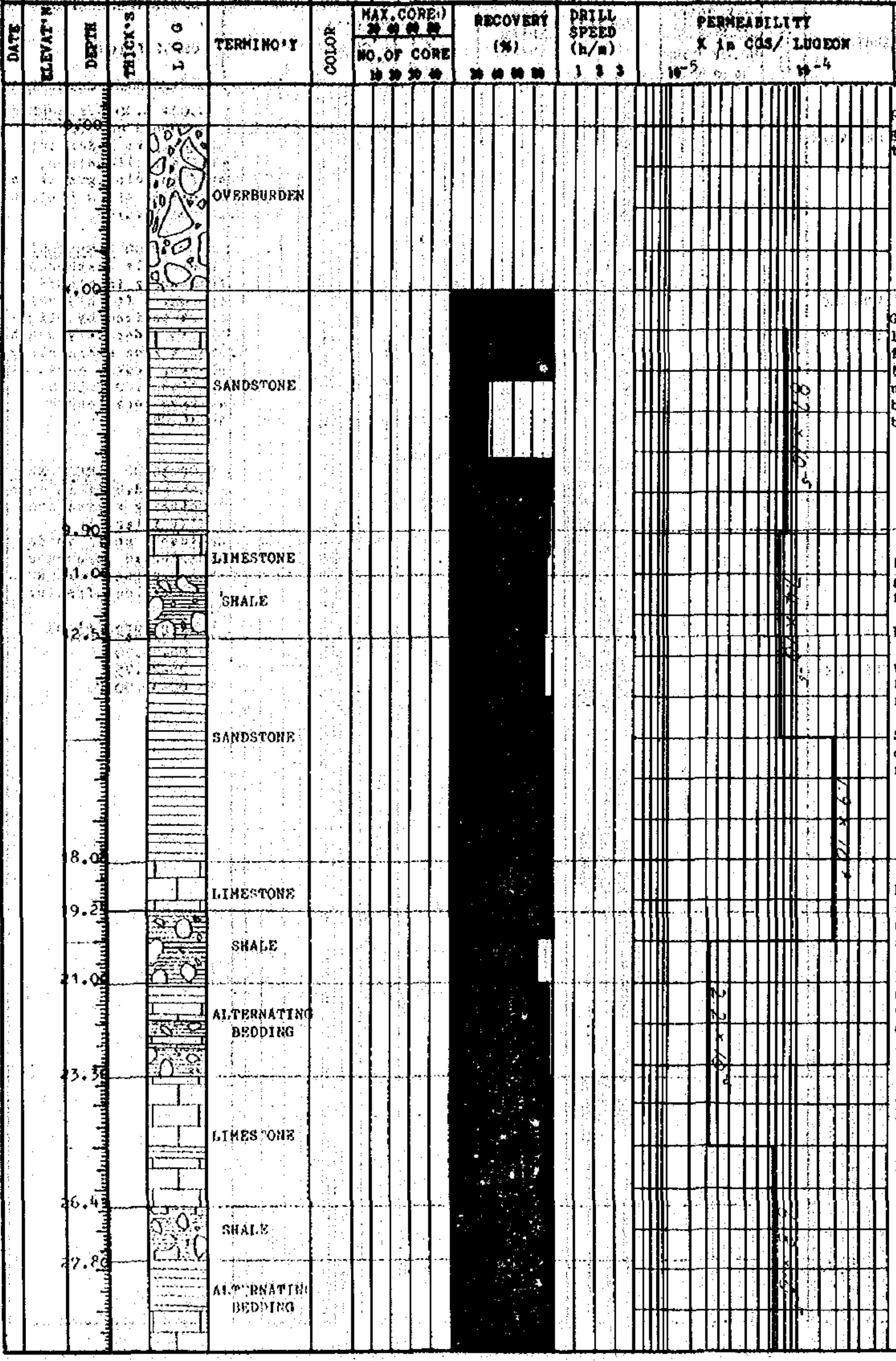
0.00- 3.00 <u>OVERBURDEN</u>
Composed chiefly of slightly to moderately weathered limestone, buff in color, gravel to cobble in sizes & plenty of small voids.

3.00- 9.00 <u>LIMESTONE</u>
Moderately weathered, fracturing is closed approx. 1 ft interval, characterized by slightly to moderately weathered & iron stain along planes, broken cores at section 7.00-7.50 M, average core length 14-16 CM.

9.00-30.00 <u>LIMESTONE</u>
Fresh, hard, buff in color, fracturing varies from 1-2 ft interval, noticable small voids distributed throughout this section, weathering is more along fracture planes,
broken cores along following section:
15.00-16.50
24.25-24.75
28.00-30.00 M.

B O R E H O L E   L O G

PROJECT	BOHOL Y.A.D. PROJECT					**SITE**	PAMACSALAN DAMSITE	
HOLE NO. DDH-5	**ELEVATION**		**ANGLE** VERTICAL	**MACHINE**		**BEGUN** 9/5/77	**SITE ENGINEER**	
	DEPTH 70.00 M		**BIT**	**PUMP**		**COMPLETED** 9/12/77	**FOREMAN**	
	DIAMETER NW		**GW LEVEL**	**ENGINE**		**DAYS REQUIRED**	**DRILLER**	



DESCRIPTIONS

0.00-4.00 OVERBU
Recovered as sludge,
granish in color,co
texture.

Rock bit drilling
0.00-4.00m good dri
condition up to 7.0

4.00-9.90 SANDSTON
Grayish in color,sl
ly fractured,mediu
fossiliferous with
thin beds of limest
noted at 5.15-5.6m,
breaks at moderate
blows.

9.90-11.00 LIMESTO
Dirty white to ligh
grayish in color,
fossiliferous.
11.00-12.55 SHALE
Light greyisch in c
,fairly hard,calcar
with interclasts fr
fragments of limest
fossiliferous.
12.55-18.08 SANDST
Slightly fractured
core generally hard
fossiliferous.

18.08-19.24 LIMEST
Light brownish in c
,hard with argilla
materials along it
section/fracture
19.24-21.00 SHALE
Light grayish in c
medium hard,calcar
with interclasts fr
ments of limestone,
fossiliferous.
21.00-23.30
ALTERNATIG BEDDI
Alternation of sa
limestone and shal
fossiliferous.
23.30-26.41 LIMES
Light grayish in c
fossiliferous with
argillaceous mater
along its section
minor thin bed of
stone noted at25.
41m. Breaks at
hammer blows.
27.80-34.20 SHALE
(continues)

B O R E H O L E L O G

OJECT	BOHOL I.A.D. POJECT					SITE	PAHACSALAN DAMSITE		
DH-5	ELEVATION		ANGLE	VERTICAL	MACHINE	ACKER	BEGUN	9/5/77	SITE ENGINEER
	DEPTH	20.00 M	BIT		PUMP		COMPLETED	9/12/77	FOREMAN
	DIAMETER	NW	GW LEVEL		ENGINE		DAYS REQUIRED		DRILLER

ELEVAT'N	DEPTH	THICK'S	LOG	TERMINO'Y	COLOR	MAX.CORE 20 40 60 80 / NO.OF CORE 10 20 30 40	RECOVERY (%) 20 40 60 80	DRILL SPEED (h/m) 1 2 3	PERMEABILITY K in CGS/ LUGEON 10^{-5} 10^{-4}
	0.0			ALTERNATIC BEDDING					1.6 X 10^{-4}
	4.20			SHALE					1.0 X 10^{-5} 4.7 x 10^{-5}
	5.9			LIMESTONE					4.0 x 10^{-5}
	8.0			SHALE					
	1.8			LIMESTONE					1.8 x 10^{-5}
	3.8			SHALE					3.8 x 10^{-5}
	6.1			SANDSTONE					
	7.36			LIMESTONE					
	.80			SHALE					

DESCRIPTIONS

(continued)

ALTERNATING BEDDING
Alternation of sandstone, limestone and shale, fossiliferous.

90% water return at 19.90-23.30m

34.20-45.95 SHALE
Light grayish in color, medium hard, calcareous, with interclasts fragments of limestone, fossiliferous. Minor thin beds of limestone noted at 31.15-31.86; 35.35-35.41; 36.53-37.01; 41.13-41.33; 43.75-44.05; 44.75-45.10m and thik bed of sandstone noted at 37.01-37.65m, fossilferous.

80% water return. Grayins in color at 31.20-34.20m.

At 34.20-70.00m, 90% water return, grayish in color.

45.95-48.00 LIMESTONE
Ligth grayish in color, hard with argillaceous materials noted along its section, fossiliferous

48.00-51.25 SHALE
Light grayish in color, hard calcareous, with interclasts fragments of limestone, fossilliferous. Minor thin bed of calcareous shale at 48.99-49.29 meters.

51.25-53.89 LIMESTONE
Light gray in color, hard with argillaceous materials noted along its section, fossiliferous. Minor thin bed of calcareous shale at 51.89-52.20m

53.89-56.15 SHALE
Grayish in color, hard, calcareous with interclasts fragments of limestone, fossiliferous.

56.15-57.36 SANDSTONE
Grayish in color, hard, fossiliferous.

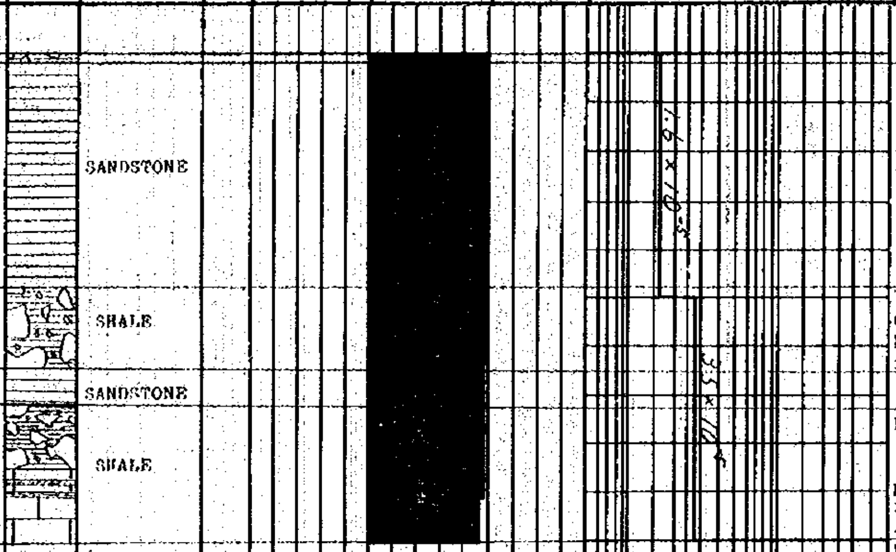
57.36-58.80 LIMESTONE
Grayish in color, hard.

58.80-60.20 SHALE

(continues)

BOREHOLE LOG

PROJECT	BOHOL I.A.D. PROJECT						**SITE**	PAMACSALAN DAMSITE		
HOLE NO. DDH-5	**ELEVATION**		**ANGLE**	VERTICAL	**MACHINE**	ACKER	**BEGUN**	9/5/77	**SITE ENGINEER**	
	DEPTH	70.00 M	**BIT**		**PUMP**		**COMPLETED**	9/12/77	**FOREMAN**	
	DIAMETER	NW	**GW LEVEL**		**ENGINE**		**DAYS REQUIRED**		**DRILLER**	



DATE	ELEVAT'N	DEPTH	THICK'S	LOG	TERMINO'Y	COLOR	MAX. CORED / NO. OF CORE	RECOVERY (%)	DRILL SPEED (h/m)	PERMEABILITY K in CGS/ LUGEON	DESCRIPTIONS
											(continued)
		60.0			SANDSTONE					1.6×10^{-3}	60.20-64.74 SANDS Dark grayish in col fairly hard to fri fossiliferous. Break at light ham blows.
		64.7			SHALE					3.5×10^{-4}	64.74-66.40 SHALE Grayish in color, hard to friable,fos iliferous with inter fragments of limes
		66.4			SANDSTONE						66.40-67.15 SANDS Dark grayish in col friable.
		67.1			SHALE						67.15-70.00 SHALE Moderately fractur fairly hard to fri fossiliferous,calc with interclasts fr ments of limeston. Minor thin bed of stone noted at 68. 68.50;68.55-68.65 68.75-70.00m.
		70.0									

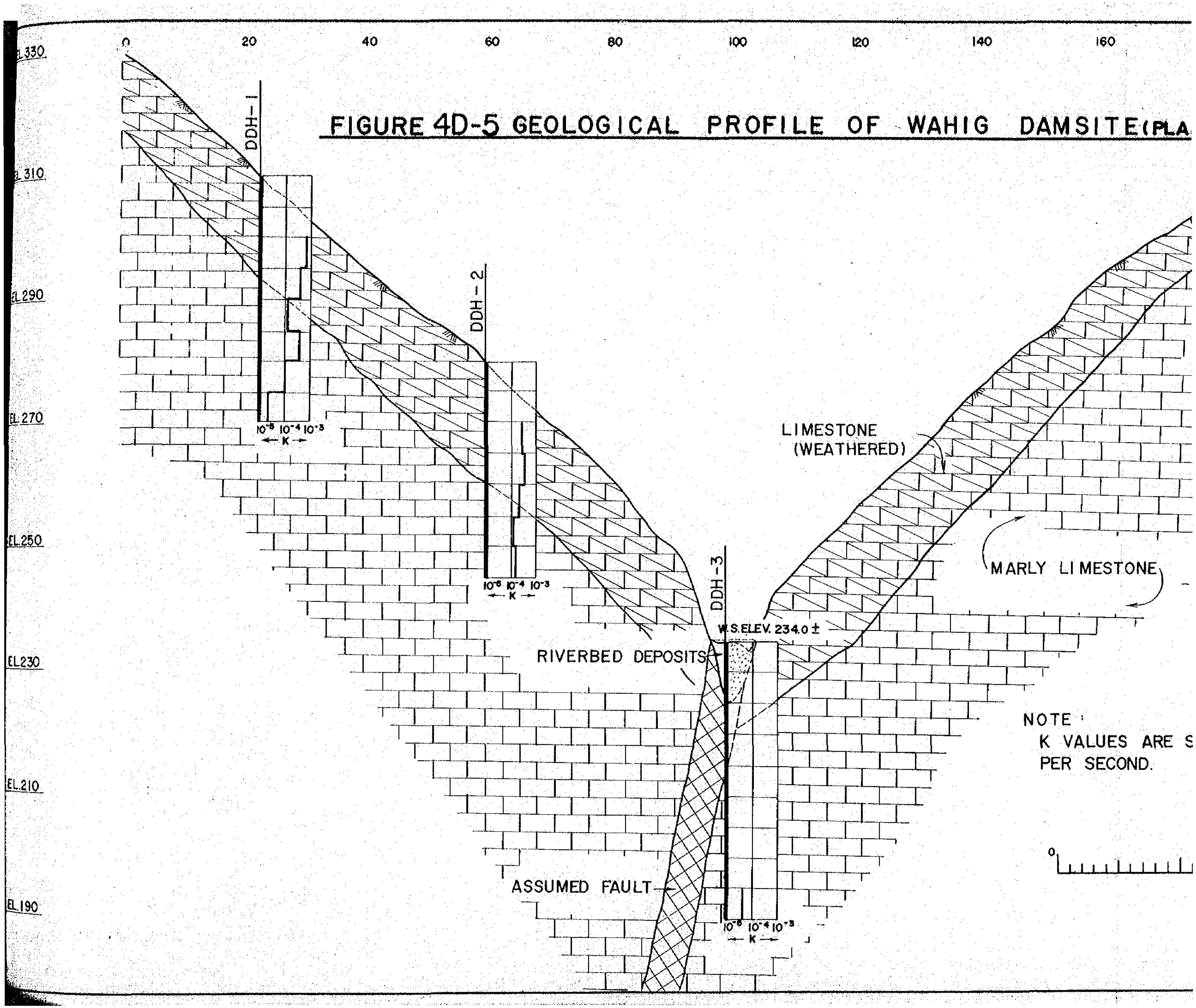
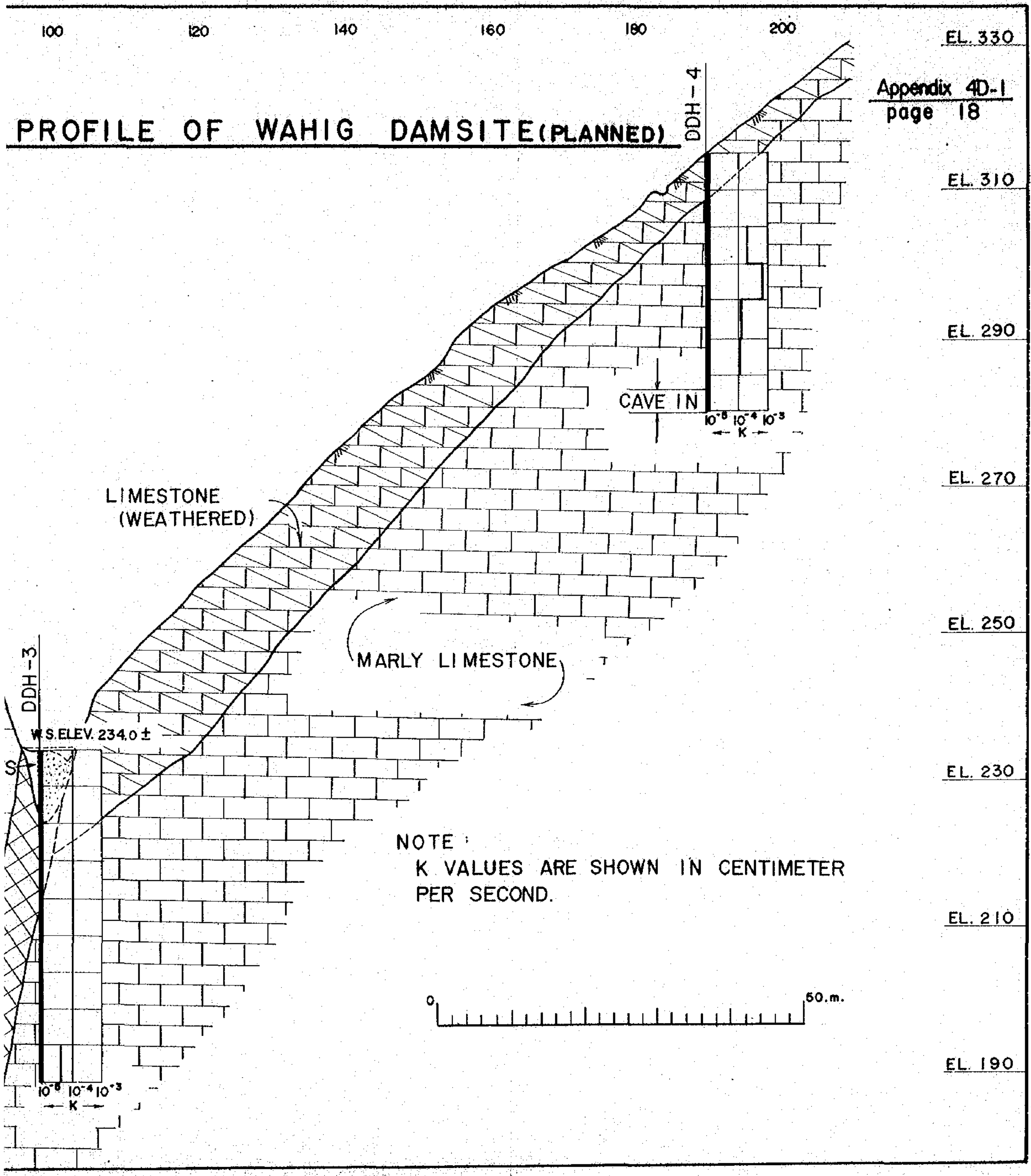


FIGURE 4D-5 GEOLOGICAL PROFILE OF WAHIG DAMSITE(PLA

NOTE :
K VALUES ARE S
PER SECOND.

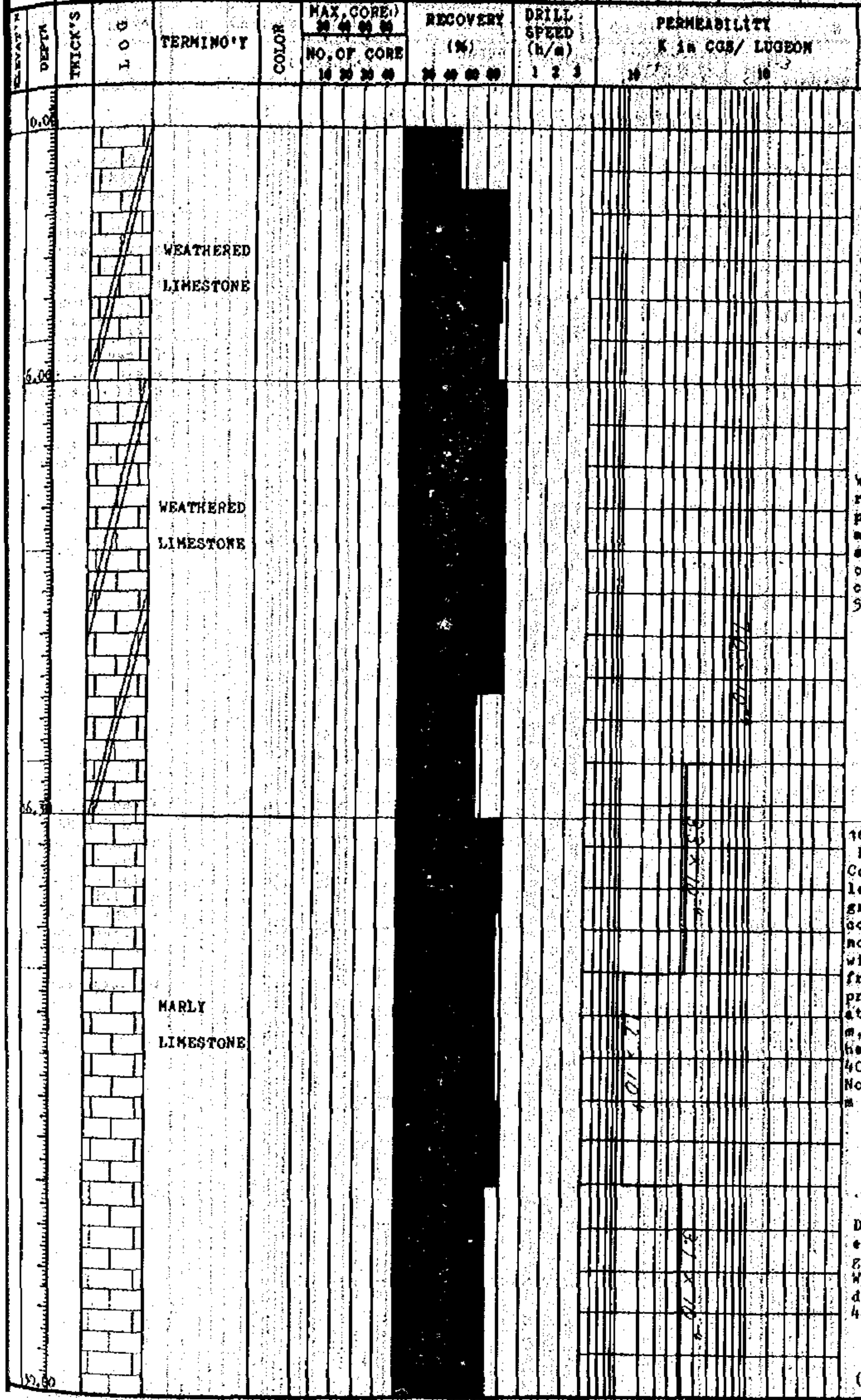
PROFILE OF WAHIG DAMSITE(PLANNED)



NOTE :
K VALUES ARE SHOWN IN CENTIMETER PER SECOND.

BOREHOLE LOG

JECT	BOHOL I.A.D. PROJECT					SITE	/ WAHIG PROPOSED DAMSITE		
	ELEVATION		ANGLE	VERTICAL	MACHINE	ACE 2(ACKER	BEGUN 11/29/77	SITE ENGINEER	
H-5	DEPTH	40.00 m	BIT		PUMP		COMPLETED 12/?/77	FOREMAN	
	DIAMETER	NQ	GW LEVEL	17.20 m	ENGINE		DAYS REQUIRED	DRILLER	



DESCRIPTIONS

0.00-6.00 LIMESTONE
Weathered, cores recovered are mostly broken but firm & hard, noted abundant presence of iron stains, also foreign materials(shells, cementing materials), friable with finger press, max.& min.core lengths are 28 and 6 cm respectively.

6.00-16.30 LIMESTONE
Weathered, cores recovered are softer than previous run, few are also firm & hard, friable still abundant presence of iron stain, max.& min. core lengths are 34 & 5 cm respectively.
return water is 90%.

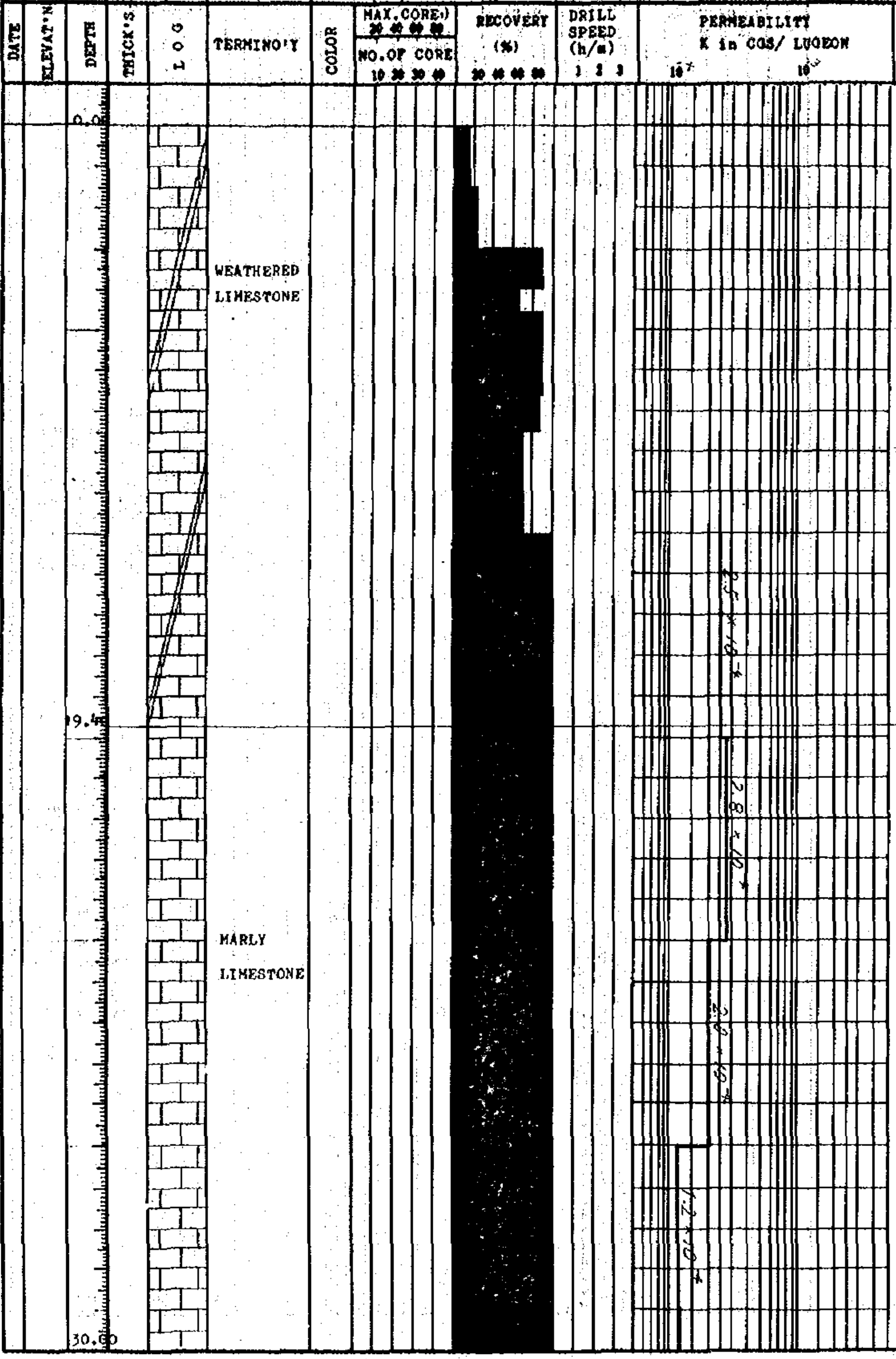
16.30-40.00
MARLY LIMESTONE
Cores recovered mostly less than 10 cm long, grey in color, most cores are hard & firm, noted few broken cores with sharp edges, friable with finger press, soft cores noted at sections 35.20-37.20 m, easily disturbed by hammer blows.
40 % water return.
No water level is 19.00 m depth.

Drilling condition for entire hole was fair and good.
Water losses noted at depth 29.20 m down to 40.00 m.

(the log continues)

BOREHOLE LOG

PROJECT	BOHOL I.A.D. PROJECT					SITE	WAHIG PROPOSED DAMSITE		
HOLE NO. DDH-2	ELEVATION		ANGLE	VERTICAL	MACHINE	TAS	BEGUN	12/5/77	SITE ENGINEER
	DEPTH	35.00 m	BIT		PUMP		COMPLETED	12/8/77	FOREMAN
	DIAMETER	NQ	GW LEVEL	19.00 M	ENGINE		DAYS REQUIRED	4	DRILLER



DESCRIPTIONS

0.00-19.40
LIMESTONE(weath
cores recovered are
mostly 10 cm length
noted abundant pre
of iron stains espe
lly along broken su
face,
section 0.00 m dow
13.00 m is a soft (
mation, friable wit
finger press,
max.& min. lengths
solid core are 35 c
and 3 cm respectiv

80 to 90 % water re
No water level.

19.40-35.00
MARLY LIMESTONE
cores are hard and
firm, grey in color
noted some sections
are friable due to
slight weathering,
max.and min.core le
are 42 cm and 5 cm.

No water return.

(Continues)

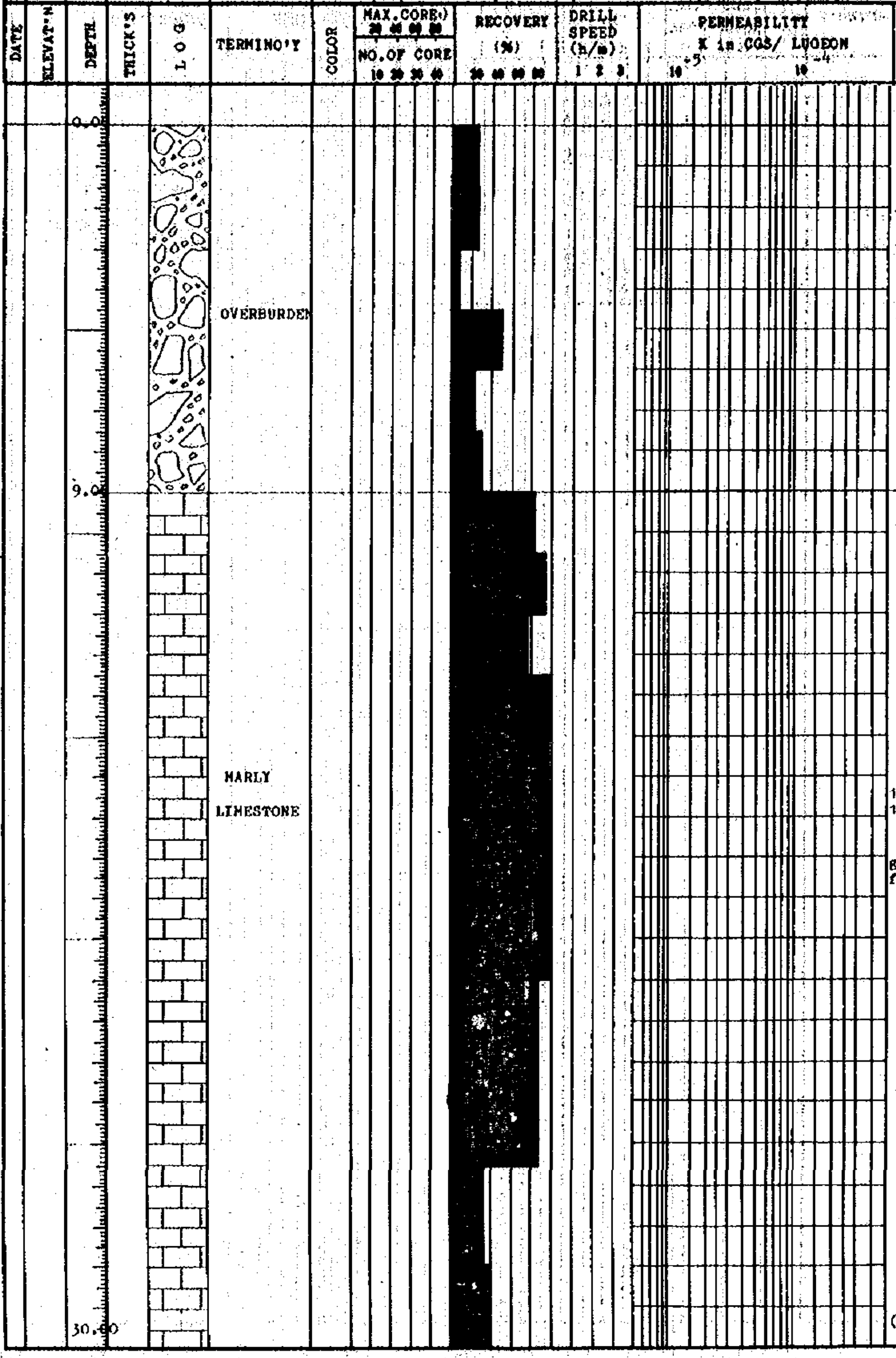
BOREHOLE LOG

JECT	BOHOL I.A.D. PROJECT					SITE	WANIG PROPOSED DAMSITE		
	ELEVATION		ANGLE		MACHINE		BEGUN		SITE ENGINEER	
8-2	DEPTH		BIT		PUMP		COMPLETED		FOREMAN	
	DIAMETER		GW LEVEL		ENGINE		DAYS REQUIRED		DRILLER	

ELEVAT'N	DEPTH	THICK'S	LOG	TERMINO'Y	COLOR	MAX.CORE(cm) 20 40 60 80 NO.OF CORE 10 20 30 40	RECOVERY (%) 20 40 60 80	DRILL SPEED (h/m) 1 2 3	PERMEABILITY K in CGS/ LUGEON 10^{7} 10^{3}	DESCRIPTIONS
	30.00									(Continued)
				MARLY LIMESTONE					1.14×10^{-4}	Drilling condition for the entire-hole was slightly hard to fair. Loss water noted at depth 14.00 down to 35.00 m.
	35.00									

BOREHOLE LOG

PROJECT	BOHOL I.A.D. PROJECT					SITE	WAHIG PROPOSED DAMSITE		
HOLE NO. DDH-3	ELEVATION		ANGLE	VERTICAL	MACHINE	ACE-1 ACKER	BEGUN	11/12/77	SITE ENGINEER
	DEPTH	45.00 m	BIT		PUMP		COMPLETED	11/18/77	FOREMAN
	DIAMETER	NQ	GW LEVEL		ENGINE		DAYS REQUIRED	7	DRILLER



DESCRIPTIONS

0.00- 9.00 OVERB
River-bed deposits,
hard,massive,yello
color limestone ra
from gravel to bo
sizes;fresh.

No water return.

9.00-45.00
MARLY LIMESTONE
Hard indurated, s
what clastic,fossi
ferous,light grey,
greyish to yellowi
color,highly frac
weathering varies
slightly to moder
weathered,mostly
recovered are bro
brecciated.

Minor broken cores
taken from section
9.13 - 9.33 m
9.48 - 10.50 m
12.40 - 12.70 m
14.85 - 15.00 m.

Good water return
grey to grey in col
from 9.00 - 45.00 m

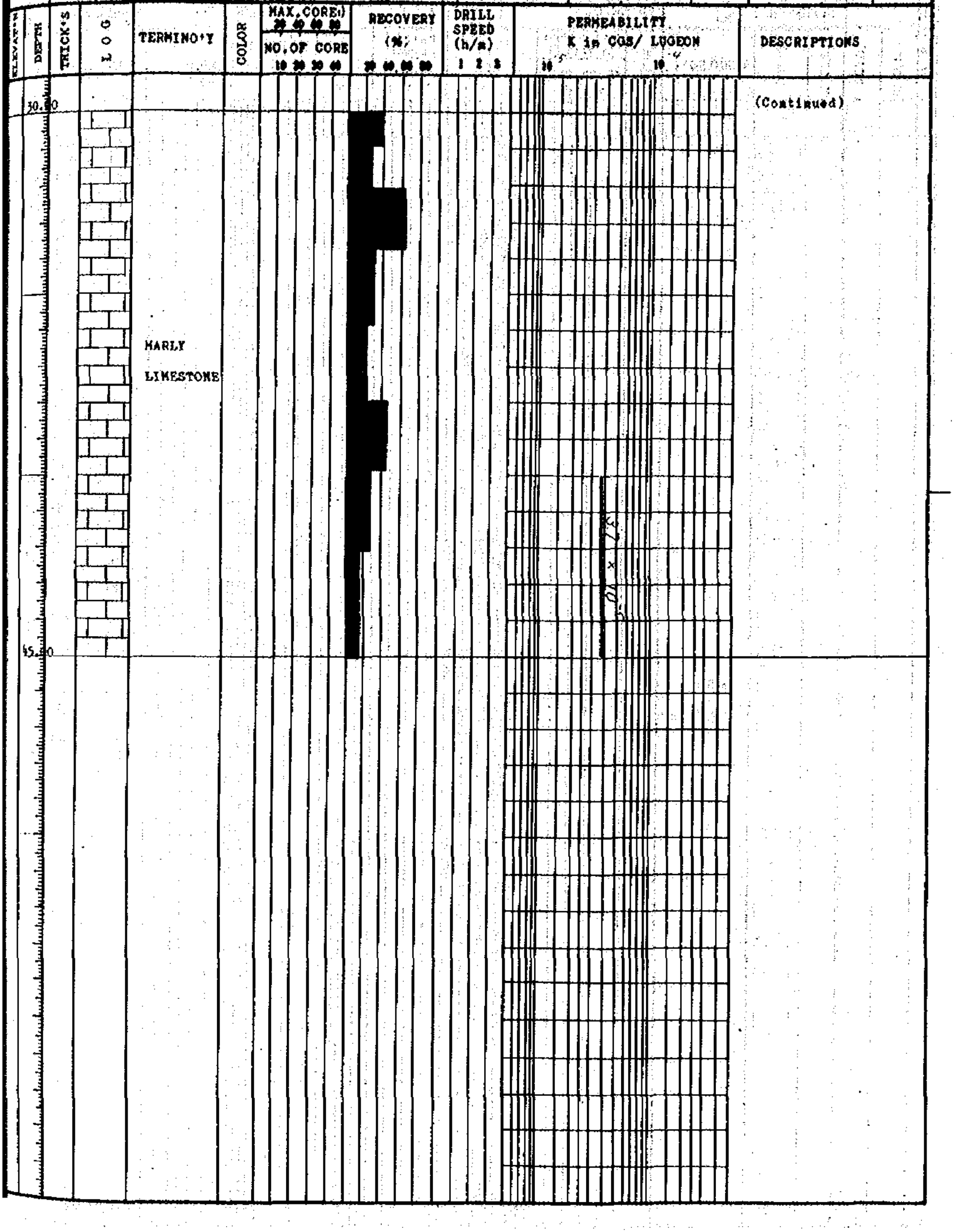
Brecciated zones;
15.37 - 15.49 m
18.10 - 18.30 m
20.20 - 21.30 m
23.40 - 28.50 m (f
30.30 - 32.10 m
33.60 - 42.20 m
42.20 - 45.00 m (f

Water-pressure te
conducted section
from 10.00 m down
40.00 m were disc
nued due to exces
leakage.

(Continued)

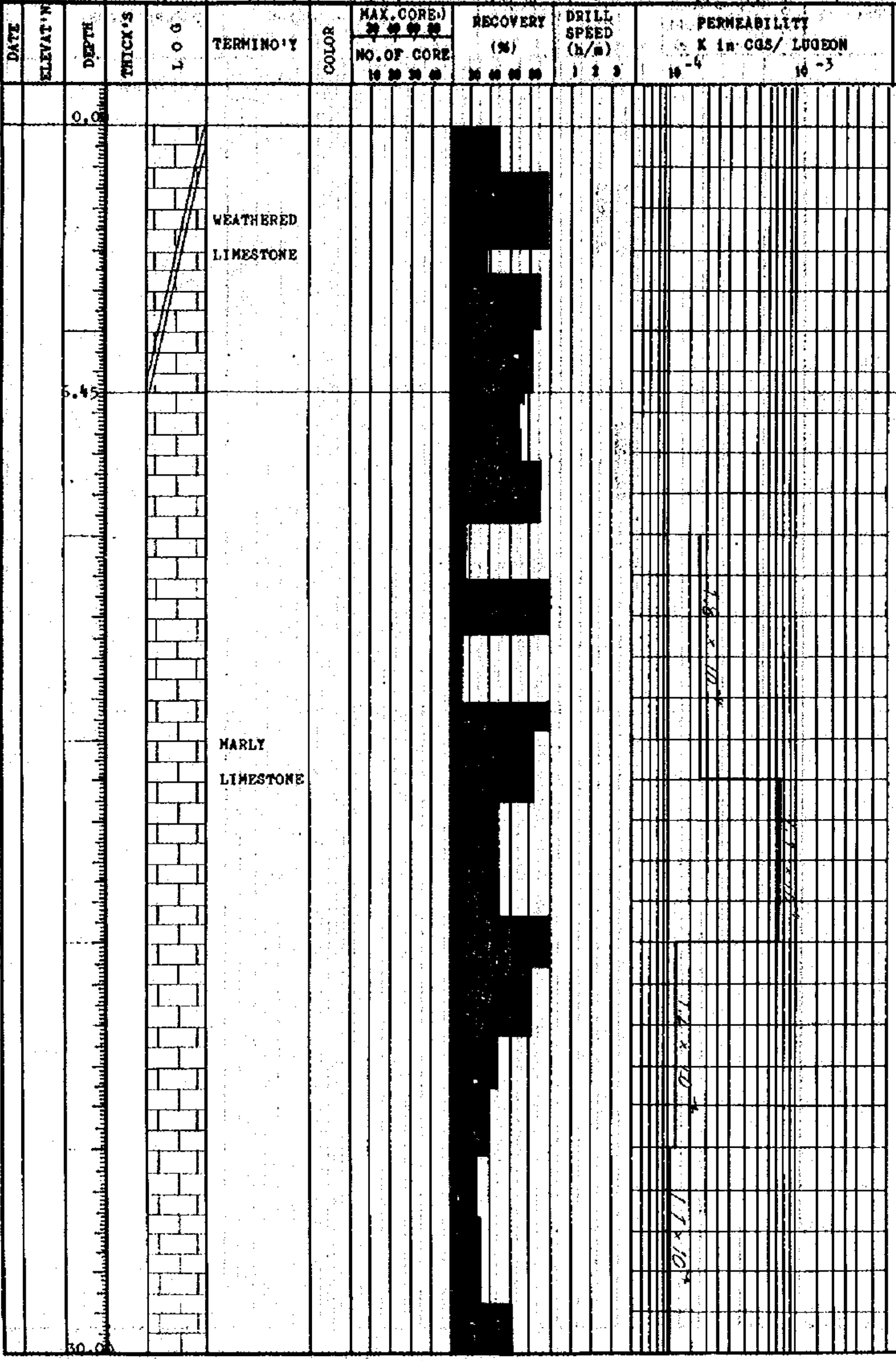
BOREHOLE LOG

JECT	BOHOL I.A.D. PROJECT					SITE	WAHIG PROPOSED DAMSITE	
R-3	ELEVATION		ANGLE		MACHINE		BEGUN		SITE ENGINEER	
	DEPTH		BIT		PUMP		COMPLETED		FOREMAN	
	DIAMETER		GW LEVEL		ENGINE		DAYS REQUIRED		DRILLER	



BOREHOLE LOG

PROJECT	BOHOL I.A.D. PROJECT						SITE	WAHIG PROPOSED DAMSITE	
HOLE NO. DDH-4	ELEVATION		ANGLE	VERTICAL	MACHINE	AOE 1,ACKER	BEGUN	12/11/77	SITE ENGINEER
	DEPTH	35.00 M	BIT		PUMP		COMPLETED	12/14/77	FOREMAN
	DIAMETER	NQ	GW LEVEL		ENGINE		DAYS REQUIRED	4	DRILLER



DESCRIPTIONS

0.00- 6.45
LIMESTONE
Weathered,cores rec
red mostly broken
sharp to rounded i
shape,friable with
finger press,brown
color probably due
oxidation/weatheri
max.length of core
14 cm and min.lengt
3 cm.
50 % water return.

6.45-35.00
MARLY LIMESTONE
Solid cores recover
mostly at section
to16.50 m,firm and
the rest of core is
broken with sharp
subrounded in shap
also froable with
quite great resist
to hammer blow, m
min. core lengths
30 cm & 5 cm respe
ely.

Upto 9.37 m depth,
water returns are
to 100 %.

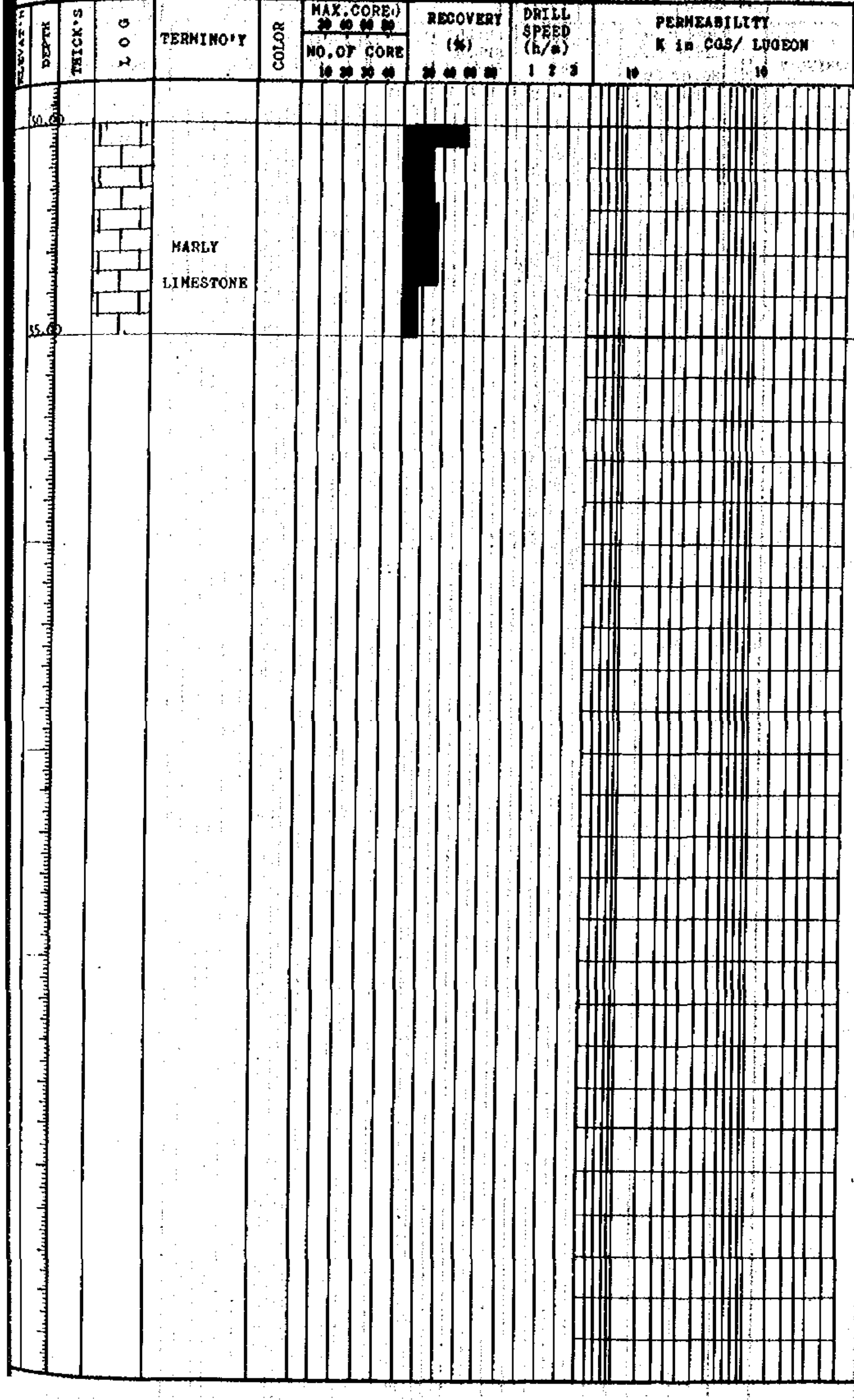
Water loss noted a
depth 17.10 m.

Deeper sections th
20.00 m, no water
return.

(Continues)

BOREHOLE LOG

JECT	BOHOL I.A.D. PROJECT					SITE	WAHIG PROPOSED DAMSITE	
NH-4	ELEVATION		ANGLE		MACHINE		BEGUN	SITE ENGINEER
	DEPTH		BIT		PUMP		COMPLETED	FOREMAN
	DIAMETER		GW LEVEL		ENGINE		DAYS REQUIRED	DRILLER



DESCRIPTIONS

(Continued)

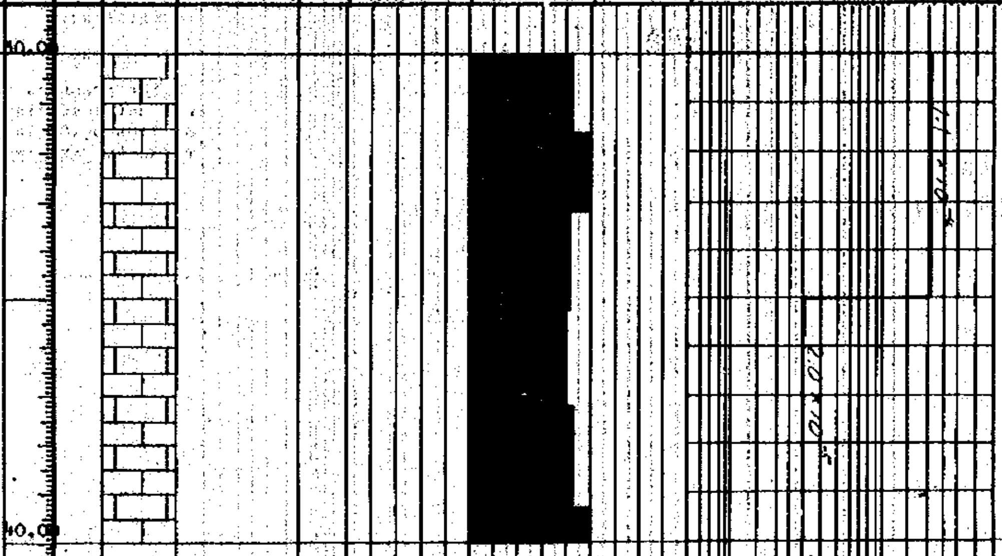
30.00-35.00

Water-pressure test was not conducted at section 30.00 to 35.00 m due to cave-in.

BOREHOLE LOG

PROJECT	BOHOL I.A.D. PROJECT					SITE	WAHIGPROPOSED DAMSITE		
HOLE NO. DDH-1	ELEVATION		ANGLE		MACHINE		BEGUN		SITE ENGINEER
	DEPTH		BIT		PUMP		COMPLETED		FOREMAN
	DIAMETER		GW LEVEL		ENGINE		DAYS REQUIRED		DRILLER

DATE	ELEVAT'N	DEPTH	THICK'S	LOG	TERMINO'Y	COLOR	MAX.CORE) 20 40 60 80 / NO.OF CORE 10 20 30 40	RECOVERY (%) 20 40 60 80	DRILL SPEED (h/m) 1 2 3	PERMEABILITY K in CGS/ LUGEON 10 10	DESCRIPTIONS
											(continued)



3. Test Pits

Excavation of test pits were carried out around both dam sites to grasp the obtainable quantity of the impervious material, and to sample the soil test materials by NIA.

(a) Pamacsalan Dam

8 test pits in total were excavated around the Pamacsalan dam site during the initial feasibility study. Location and log of these test pits are shown in Figures 4D-1 and 4D-7, respectively.

(b) Wahig Dam

Excavation of 8 test pits were carried out around the dam site during the initial feasibility study. Location and log of these test pits are indicated in Figures 4D-2 and 4D-8, respectively.

FIGURE 4D-7 TEST PITS LOG AT PAMACSALAN DAM SITE

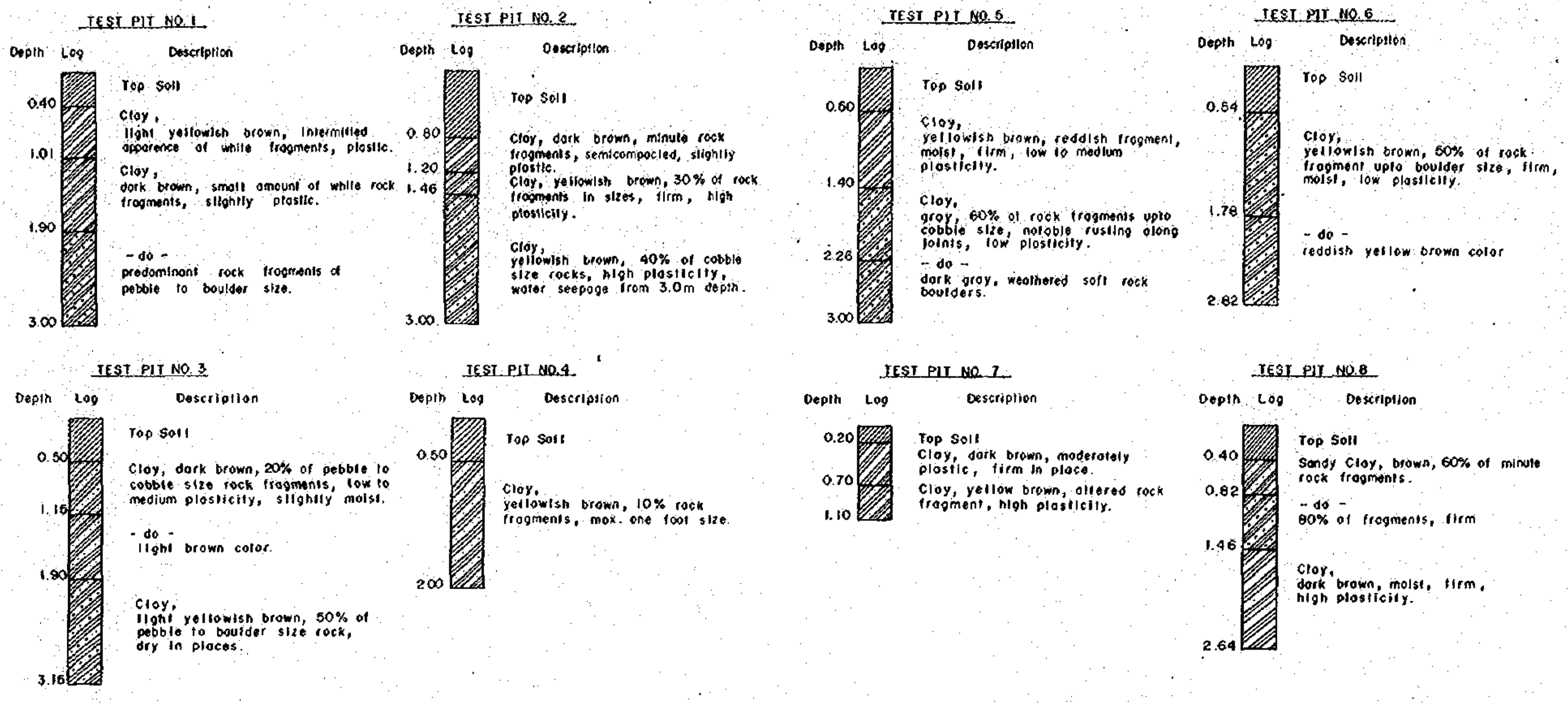
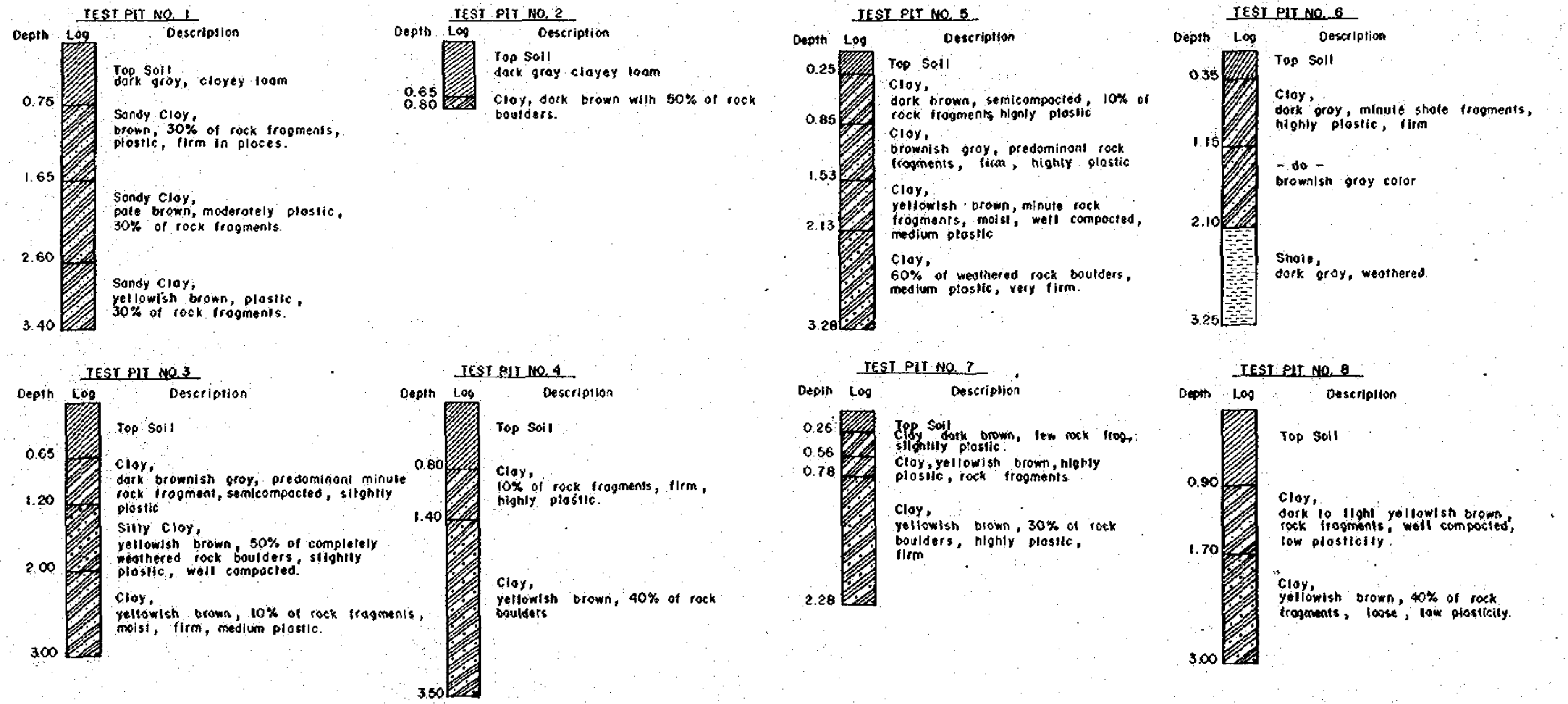


FIGURE 4D-8 TEST PITS LOG AT PLANED WAHIG DAM SITE



4. Soil Tests

The materials taken from the test pits were sent to NIA. (Materials Testing Laboratory of Upper Pampanga River Project, Cabanatuan City) and the physical tests were carried out. On the other hand, almost of the dynamic tests were performed in Japan by the sampled materials from the test pits.

The results of physical and dynamic tests including auxiliary test are shwon in Table 4D-1.

Table 4D-1. Table of Soil Tests

Sample	Depth (GL.-m)	GS[1]	F.M (%)[2]	Gradation Analysis Gravel (%)	Sand (%)	Silt (%)	Clay (%)	Atterberg Limit L.L[3] (%)	P.L[4] (%)	P.I[5] (%)	USCS[6]	Compaction Test E.C[7] (%)	Wopf[8] (%)	γd max.[9] (t/m³)	Initial Condition of Specimen E.C (%)	D-value[10] (%)	W[11] (%)	γd[12] (t/m³)	e[13]	Sr[14] (%)	
Pamacsalan																					
P.TP-1	0.40-1.01	2.67	38	40.68	19.89	31.03	8.40	55.50	26.26	29.24		100	23.80	1.492							
	1.01-1.90		30																		
	1.90-3.00		36																		
P.TP-2	0.80-1.20	2.64	28	15.82	31.05	39.03	14.10	55.35	26.67	28.68											
	1.20-1.46		25																		
P.TP-3	0.50-1.15	2.67	53	12.16	37.20	35.54	15.10	51.30	23.89	27.41		100	25.40	1.457							
	1.15-1.90		36																		
	1.90-3.15		12																		
P.TP-4	0.50-2.00		12																		
P.TP-5	0.50-1.40		27																		
	1.40-2.26		31																		
	2.26-3.00		23																		
P.TP-6	0.54-1.78	2.70	22	8.85	49.86	26.29	15.00	48.10	30.44	17.66											
	1.78-2.82		38																		
P.TP-7	0.20-0.70		53																		
	0.70-1.10		22																		
Wahig																					
W.TP-1	0.75-1.65	2.63	22	39.65	44.63	10.92	4.80	48.10	20.00	28.10		100	15.35	1.732							
	1.65-2.60		14																		
	2.60-3.40		7																		
W.TP-3	0.65-1.20	2.66	19	11.81	51.46	25.73	11.00	37.25	21.25	16.00											
	1.20-2.00		17																		
	2.00-3.20		20																		
W.TP-4	0.80-1.40	2.71	27	2.61	58.44	28.85	10.10	34.25	22.19	12.06		100	17.72	1.699							
	1.40-3.30		30																		
W.TP-5	0.25-0.85	2.69	69	4.26	40.53	42.21	13.00	72.40	42.30	30.10		100	36.30	1.204							
	1.53-2.13		35																		
	2.13-3.28		49																		
W.TP-6	0.35-1.15		64																		
	1.15-2.10		39																		
	2.10-3.35		30																		
W.TP-7	0.26-0.56		35																		
	0.56-0.78		29																		
	0.78-2.28		26																		
W.TP-8	0.90-1.70		17																		
	1.70-3.00		17																		
W.TP-9	0.50-2.20		10																		
Pamacsalan																					
P.TP-3	S-1[23]	2.79	34.5	24.0	16.3	31.2	28.5	72.6	27.2	45.4	CH										
	S-2[23]	2.78	33.8	25.1	16.1	31.3	27.5	72.6	28.4	44.2	CH										
	4.76 mm under mixed	2.78	34.0	-	20.9	40.1	39.0	75.4	27.4	48.0	CH	150	26.0	1.500	100	100	28.4	1.458	0.907	87.1	3
												100	28.4	1.458	150	95	31.4	1.425	0.951	91.8	6
Wahig																					
W.TP-5	S-1[23]	2.73	29.6	24.5	8.8	37.0	29.7	56.9	26.3	30.6	CH										
	S-2[23]	2.71	29.3	17.7	8.9	43.1	30.3	56.2	26.6	29.6	CH										
	4.76 mm under mixed	2.71	29.0	-	9.3	54.2	36.5	56.7	24.5	32.2	CH	150	20.5	1.625	100	100	22.2	1.588	0.707	85.1	1
												100	22.2	1.588	150	95	26.0	1.544	0.755	93.3	1
W.TP-6	S-1[23]	2.75	41.9	62.8	5.3	17.4	14.5	86.9	40.8	46.1	QC	150	31.7	1.367							
	S-2[23]	2.75	36.0	0	24.8	41.2	34.0	82.8	38.2	44.6	CH	100	33.6	1.335							

Notes:

1/	specific gravity
3/	liquid limit
5/	plasticity index
7/	degree of compaction
9/	maximum dry density
10/	D-value = $\frac{\gamma d}{\gamma d \text{ max.}} \times 1$
12/	dry density
14/	degree of saturation
16/	compression index
18/	$U = \frac{e_0 - e}{1+e_0} \times 100$
19/	cohesion at unconsoli
20/	angle of internal fri
21/	cohesion at consolida
22/	angle of internal fri
23/	natural gradation con
24/	after compaction cond.

Table 4D-1. Table of Soil Tests

	Atterberg Limit				Compaction Test			Initial Condition of Specimen							Consolidation Test			Triaxial Test U-U Test		Direct Shear Test C-U Test		
lay (%)	L.L3/ (%)	P.L4/ (%)	P.I5/ (%)	USCS 6/	E.C7/ (%)	Wopf8/ (%)	γd max.9/ (t/m^3)	E.C (%)	D-value10/ (%)	W11/ (%)	γd12/ (t/m^3)	e13/	Sr14/ (%)	K15/ (cm/sec)	Cc16/	Cv17/ (cm^2/sec)	U18/ (%)	Cu-u19/ (t/m^2)	φu-u20/ (°)	Cc-u21/ (t/m^2)	φc-u22/ (°)	Remarks
.40	55.50	26.26	29.24		100	23.80	1.492															All soil tests were carried c by NIA.
.10	55.35	26.67	28.68																			
.10	51.30	23.89	27.41		100	25.40	1.457															
.00	48.10	30.44	17.66																			
.80	48.10	20.00	28.10		100	15.35	1.732															
.00	37.25	21.25	16.00																			
.10	34.25	22.19	12.06		100	17.72	1.699															
.00	72.40	42.30	30.10		100	36.30	1.204															
.5	72.6	27.2	45.4	CH																		All soil test were carried in Japan
.5	72.6	28.4	44.2	CH																		
.0	75.4	27.4	48.0	CH	150 100	26.0 28.4	1.500 1.458	100 150	100 95	28.4 31.4	1.458 1.425	0.907 0.951	87.1 91.8	3.5×10^{-8} 6.9×10^{-9}	0.278	$5.8\times10^{-3}\sim$ 1.23×10^{-4}	12.55	10.8 8.30	10°00' 5°40'	10.8 8.2	14°20' 8°20'	
.7	56.9	26.3	30.6	CH																		
.3	56.2	26.6	29.6	CH																		
.5	56.7	24.5	32.2	CH	150 100	20.5 22.2	1.625 1.588	100 150	100 95	22.2 26.0	1.588 1.544	0.707 0.755	85.1 93.3	1.2×10^{-7} 1.1×10^{-8}	0.252	$7.74\times10^{-3}\sim$ 4.46×10^{-4}	12.70	10.8 8.0	16°30' 4°10'	8.8 6.3	21°40' 14°30'	
.5	86.9	40.8	46.1	QC	150	31.7	1.367															
.0	82.8	38.2	44.6	CH	100	33.6	1.335															

Notes:

1/ specific gravity
2/ field moisture
3/ liquid limit
4/ plastic limit
5/ plasticity index
6/ unified soil classification system
7/ degree of compaction
8/ optimum moisture content
9/ maximum dry density
10/ D-value = $\frac{\gamma d}{\gamma d\ max.}$ x 100
11/ moisture content
12/ dry density
13/ void ratio
14/ degree of saturation
15/ permeability coefficient
16/ compression index
17/ coefficient of consolidation
18/ U = $\frac{e_0-e}{1+e_0}$ x 100
19/ cohesion at unconsolidated undrain condition test
20/ angle of internal friction at unconsolidated undrain condition test
21/ cohesion at consolidated undrain condition test
22/ angle of internal friction at consolidated undrain condition test
23/ natural gradation condition
24/ after compaction condition

5. Rock Tests

Rock tests were carried out for the boring core on both dam sites to grasp the lithic character of bed rock and as embankment material. The tests were performed in Japan and the results are shown in the following table:

Sample	Depth (Gl.-m)	Appurtenant Specific Gravity	Absorption (%)	Compressive Strength (kg/cm²)
P. DDH-2	24.0 ∿ 25.0	2.219	9.56	60.57
P. DDH-5	17.0 ∿ 19.0	2.224	9.29	64.07
P. DDH-5	31.2 ∿ 34.2	2.451	6.88	117.40
W. DDH-3	13.5 ∿ 14.0	2.302	5.60	122.00

The basic concrete aggregate tests for sampled materials of the Manaba and Guindulman river were carried out by D.P.H. soils and materials quality control service, Manila and the results are shown in the following table.

Sample	Specific Gravity	Washing Loss (%)	Absorption (%)	Fineness Modulus
Manaba River	2.63	2.0	3.82	6.15
Guindulman River	2.66	2.0	3.71	2.33
Guindulman River	2.68	2.0	3.43	5.07

B. Technical Supporting

1. Geology of Damsite and Reservoir

(a) General Geology

The most of Bohol Island consists of a marine clastics of the upper Miocene to Pliocene. The most of Project area is composed of a formation of dark grey shale and sandstone. And the south of the Project area, the catchment areas of planned reservoirs, consists of aforesaid formation and a thick limestone formation overlain on it.

The formations show a structure in the northeast-southwest direction, main trend of the Province, and a syncline runs central part of the Project area and an anticline forms the southern-most border of watershed of reservoirs.

In the southwest and east parts of Bohol, some igneous intrusives and volcanics crop out in some extent.

The quarternary deposits in and around the Project area are, in general, in a small scale.

(b) Engineering Geology

(1) General

The Pamacsalan and reservoir are planned for main water resource of the Project. The dam is located at a tributary of the upper reach of Wahig River.

The reservoir area and dam site are situated at the northeast wing of aforesaid anticline structure. The strate composed of dam site inclines 10 to 30 degrees toward northeast, as shown Figures 4D-1.

The reservoir area composed of a formation of sandstone and shale. The topography of reservoir area is, in general, flat and gentle. Residual overburden developes rather thickly. Landslides in small scale spread out in the reservoir area.

Dangers of leakage through reservoir bottom are deemed to be very less.

A thick limestone formation lies unconformably on the said sandstone and shale formation rimming the reservoir area. The limestone is merly, partly it is limy marl, massive and somewhat clastic. It intercalates thin layers of shale and sandstone.

The formation decreases its elevation up to the riverbed, and forms narrow gorge where the damsite is planned.

So far as on-foot geological survey has concerned Karstism in the lower portion of site is not remarkable, while on the elevation of around 300 meters or more, sinkholes and caves in scales are seen elsewhere.

On the vicinity of the Planned Wahig Dam site a cave in Class A, socalled Levon Cave develops on an elevation of around 400 meters on the right bank side. Dolines, Karst sink holes, moderately large size are lined up in the immediate downstream of left bank of the site.

Whereas on the very neighborhood of left abutment of the Pamacsalan site, a large scale depression is spread out. A narrow waterfall drains water out of basin in the moment. Numbers of dolines and caves in sizes can be seen inside of basin.

As the said facts, the Karstism on the Planned Wahig site is deemed to be seen rather in progress than that on the Pamacsalan.

(2) Pamacsalan Damsite

Topography

The topography of Pamacsalan Reservoir area shows an ideal feature for water storage. A tributary joins rectangularily to the Pamacsalan at the middle of the area, and widens the reservoir area toward upstream side.

The Pamacsalan flows down toward the north-northwest direction within the reservoir area and shifts itself to north at the proposed damsite. The river slope is one to 37 on an average.

The higher portions which rim the reservoir area consists of limestone formation. Reflecting the geological structure, the rimming limestone goes down toward downstream of the river. The limestone base is deemed to cross with the riverbed about 200 meters upstream of proposed dam axis.

A very thin ridge made of limestone extends from east to west up to the right abutment of proposed dam. The elevation of ridge top varies from 265 to 300 meters. The minimum thickness of the ridge at the elevation of 260 meters is only 60 meters.

Whereas in the left bank side, a narrow ridge in the same feature of right bank side developes in the north-south direction. The ridge top varies elevation from 260 to 360 meters. The reservoir side slope of the ridge forms a vertical cliff around 100 meters high.

The topography of damsite shows insymmetrical features. As described above, the left abutment is about vertical. Whereas the slope on the right abutment is one to one. The cord-height rate of the damsite at the 270 meter elevation is one to 1.67.

Geological Condition

Five boreholes have been drilled along dam axis and ridges. The profile section availed from dorilling results is illustrated in Figure 4D-3.

As seen in these illustrations, the actual foundation of proposed Pamacsalan Dam is the marly limestone.

The permeability of limestone is in rather higher order ranging from 1.0×10^{-5} to 6.0×10^{-4} cu.m/sec. or 10 to 620 ft/year. The permeability logs of the boreholes is shown on Figure 4D-3.

According to examination of core samples taken from the boreholes, open cavities formed by karstism are hard to recognize.

So far as the drilled portions are concerned, it can probably be said the limestone in this damsite is in a comparatively better category for dam engineering, though the topographical short passes on the both bank sides are deemed to involve potential danger of leadage.

Further investigations might be required to clarify the potentials along the ridges on the both banks.

An alternation of sandstone and shale lies underneath the limestone in a shallower depth. Two inclined boreholes on the riverbed, DDA 2 and 3 encounter the layer around 28 meter vertical depth through six-meter thick soft sandy layer. On the riverbed, about 10-meter thick deposits develop. Therefore actual thickness of limestone beneath riverbed is only 15 meters. The top elevation of sandstone and shale formation in this portion is 165 meters.

Whereas by the borehole, DDH 5 drilled on the saddle portion of ridge in the right bank side, the top elevation of sandstone and shale formation is confirmed to be at 205 meters.

To assure the bearing capacity of bedrock, unconfined compression tests has been carried out on the core samples taken from boreholes.

The result is shown below;

Sample No.	1	2	3
Borehole No.	DDH-2	DDH-5	DDH-5
Depth (m)	24-25	17-19	31-34
Specific Gravity	2.219	2.234	2.448
Water Absorption (%)	9.56	9.39	6.88
Compression Strength (kg/sqcm)	60.6	64.1	117.4

Note; compression strengths are under saturated condition

The bearing capacity of bedrock in the lower portion on the right bank saddle is rather good. That on the river bed seems, however, to be not so high, probably due to weathering effect.

(3) Planned Wahig Damsite

Topography

The basic topographic feature of the Planned Wahig reservoir area is as same as that of the Pamacsalan.

The Wahig river joining two major tributaries at the mid basin, flows down west ward and shifts itself toward northwest direction forming 1.3 kilometer-long gorge, socalled the Wahig Gorge, where the damsite is planned.

An average river slope in the area is more gentle than that of the Pamacsalan being one to 50. Nevertheless, since the river bed situates higher elevation than the Pamacsalan, storage effectiveness of planned reservoir is lower than the Pamacsalan.

The planned damsite has been selected at the middle of the Wahig Gorge which consists of limestone formation. The planned damsite shows a quite symmetric topography sloping one to one. The cord-height rate becomes, therefore, one to 2.0.

Geological Condition

Four bore-holes has been drilled along the planned dam axis. The geological profile is shown in Figure 4D-5.

As seen in the goelogical profile, the site consists of only marly limestone. The weathered zone covers the both abutments for five to 15 meters in vertical thickness. Beneath the weathered zone, fairly fresh but moderately fractured bedrock lies. The overall bedrock condition of this site is deemed to be worse than that of the Pamacsalan comparing with the core recoveries.

The result from the bore-hole DDH-3 which has been drilled on the river bed is the worst one among all. The recovery of core material is only about 40% and the obtained materials are highly fractured marly limestone.

The water-pressure tests have been applied on the hole every five meter depth for seven times. Due to the high water-intake of bedrock, only one result of permeability could be got out of seven tests. It shows 2.5 x 10^{-5} cm/sec. or 15 ft/year at the section of depth between 40 to 45 meters. Judging from the adopted pump capacity, permeabilities of remained sections, depths between 10 to 40 meters, seem to be exceed 8.0 x 10^{-4} cm/sec. or 820 ft/yr.

The result of rock tests carried out on the core sample from the hole are as follows;

depth	13.5 - 14.0 (m)
specific gravity	2.302
water absorption	5.60 (%)
unconfined compression strength	115.0 - 135.7 (kg/sqcm)

This condition of bed rock does not represent overall condition of the site. The results from water tests on the both abutments indicate that the permeabilities range 2 to 8 x 10^{-4} cm/sec in the weathered zone and 2 x 10^{-5} to 3 x 10^{-4} cm/sec in the lower zone.

The bore-hole DDH-3 meets probably with a fractured zone of fault at least the section shallower than the elevation of 220 meters.

The bore-gole DDG-4 meets cave-in at the depth from 30 to 35 meters, an elevation around 280 meters. The location of the hole is not so far from the Lebon cave. However, it is unclear this cave belongs to the Lebon cave system.

There is no insurance that this kind of karst caves exists no longer in and around the site. Therefore, much more detailed geological investigations and careful considerations should be paid in case of constructing of high dam.

As a result, a careful consideration should be given to construct a high dam onto limestone formation in general, even if mark of karstism is invisible. That is from the reason why to grasp grade and extent of karstism are always difficult even though to apply the latest techniques of prospecting in various manner, and evaluation of possiblity for countermeasure against seepage problems, foundation treatment, becomes impossible.

The geological conditions of the proposed Pamacsalan and planned Wahig damsites have been investigated through on-foot survey, nine holes of boring and in-situ tests in the holes. As described in the foregoing chapters, the results indicate only general situation on components and structures of geology, permeability of foundation, grade of weathering, general grade and extent of karstism etc.

It is, therefore, necessary to pay very much boldness to introduce a conclusion that whether dam under sufficient effectiveness and safety might be constructed on the proposed sites or not, and that, supposing it is possible, which method and how much cost would be required for foundation. However, on the condition that there comes additional detailed investigations, the Pamacsalan Dam may be considered possible to be construted, since the thickness of limestone formation is limited above an elevation of 180 meters.

On the other hand, as to the Wahig, since the karstism in moderate scale and extent are supposed to exist over the Wahig Gorge, it is desired to postpone to make conclusion until much more information on geology has been collected, however it may be unfeasible to construct a high dam in this site judging from the existing data.

2. Dam Type

At the proposed Pamacsalan dam site, two different types of the dam such as concrete gravity and fill were considered from the view points of topographical, geological and hydrological conditions. According to the result of reservoir operation studies, the height of Pamacsalan dam was assumed at about 68 meters from the deepest core zone base at the river bed.

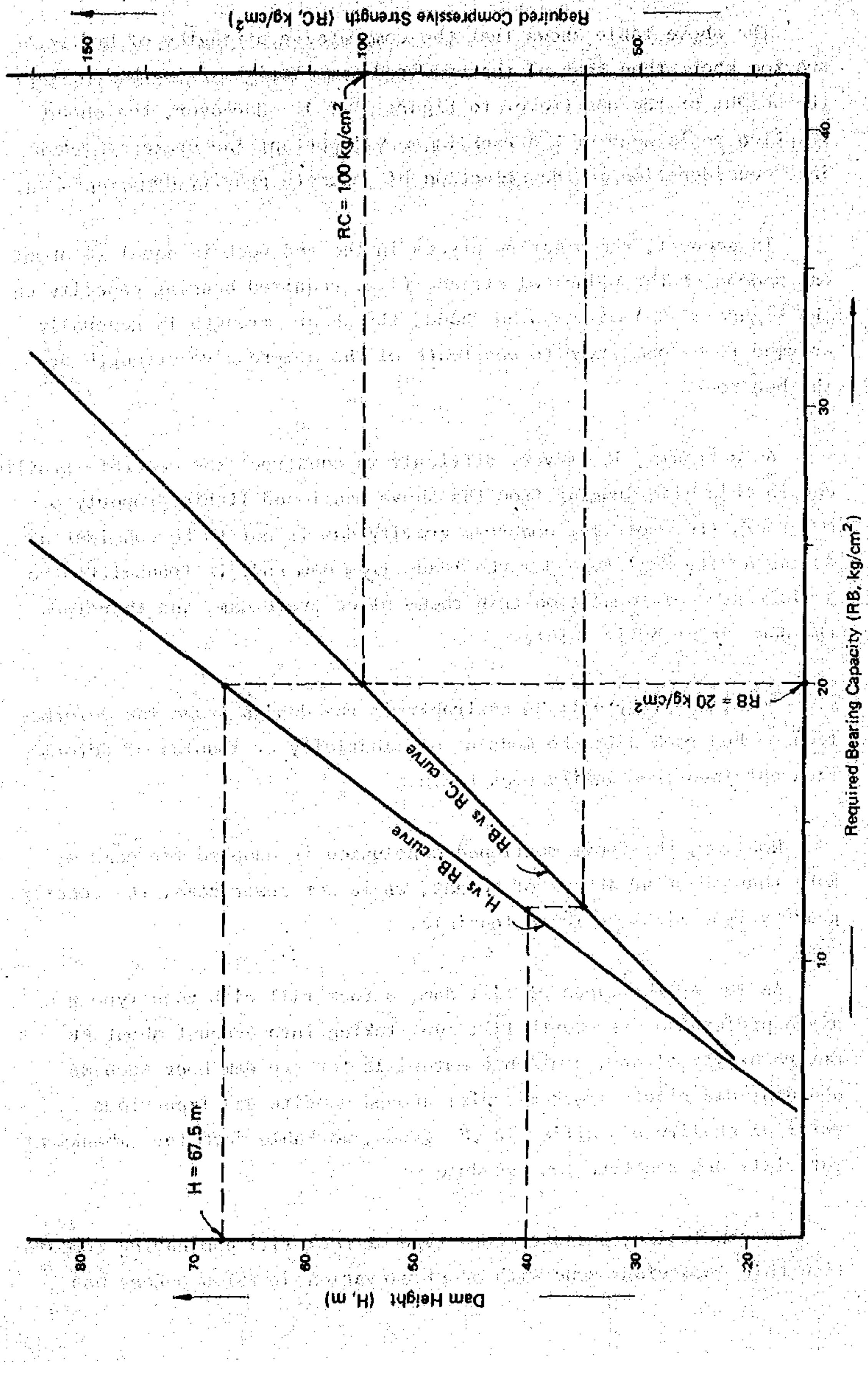
The concrete gravity dam was considered as the most suitable to the shape of gorge and surface geological conditions around damsite. For the concrete gravity dam, the bed rock shall be safe aginst normal and tangential component forces transmitted by the dam body.

Figure 4D-9 which is prepared from the results of stress analysis gives the relationship between the dam height and the required bearing capacity in the bed rock, and also shows the relationship between the required bearing capacity and the required comperssive strength of the bed rock. Required compressive strength was determined by multiphying the safety factor of five to the required bearing capacity taking into account the ununiform character of bed rock, effect of fault and joint, and fluctuation of compressive strength.

The results of compressive strength test with cylindrical test pieces which was made from sound boring cores executed in the damsite, is presented in the following table.

Location	Hole Number	Depth (m)	Compressive Strength (kg/cm²)
River Bed	P. DDH-2	24.0 ∿ 25.0	51.8 ∿ 75.6 (average 60.6)
Right Abutment	P. DDH-5	17.0 ∿ 19.0	49.1 ∿ 83.7 (average 64.1)
Right Abutment	P. DDH-5	31.2 ∿ 34.2	89.3 ∿ 152.1 (average 117.4)

FIGURE 4D-8. RELATION CURVE BETWEEN DAM HEIGHT AND REQUIRED BEARING CAPACITY, REQUIRED COMPRESSIVE STRENGTH ON CONCRETE GRAVITY DAM



The above table shows that the compressive strengths of bed rock are too short than that of the required compressive strength due to the height of the dam (refer to Figure 4D-9). Moreover, the shear friction resistance of bed rock is very important factor to be taken into consideration in the selection of concrete gravity dam.

In general, the shearing stress in the bed rock is equal to about one-second of the principal stress, (i.e. required bearing capacity on the Figure 4D-9) on the other hand, the shear strength is generally assumed to be one-sixth to one-tenth of the compressive strength of the bed rock.

As a result, it is very difficult to construct the concrete gravity dam in this site judging from the above mentioned lithic property of bed rock, therefore the concrete gravity dam is not to be considered. As far as the fill type dam the loads from dam body is transmitted to a wider area of foundation than those of concrete dam, and therefore the dam can be built safely.

Detailed study will be desirable in the design stage for deformation of bed rock with the modulus of elasticity or modulus of deformation obtained from insitu rock tests.

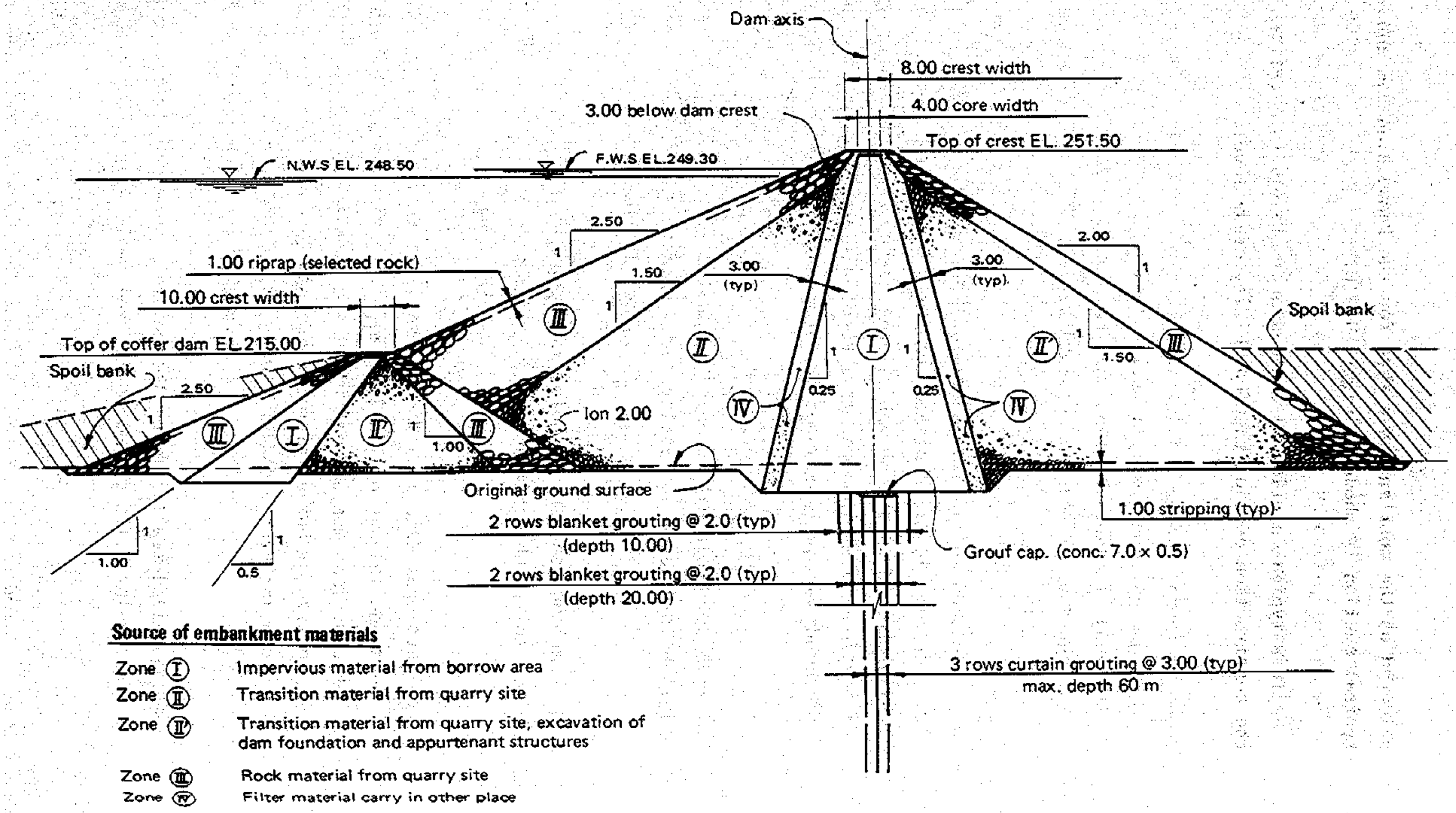
However, the above mentioned conclusion is adopted for dams of more than about 40 meters of height, while for lower dams, the concrete gravity type might be found feasible.

As far as the types of fill dam, a rock fill with zone type was given preference over earth fill type taking into account about 68 meters height of dam, available materials for the dam body such as abundant distributed rock material around damsite and impervious material quality classified in CH. group, workable days for embankment materials and construction schedule.

In conclusion, a center core type of rock fill dam having comparation thin impervious zone with crest elevation at 251.5 meters was selected as the most suitable type for the Pamacsalan dam and the typical section is shown in Figure 4D-10.

This typical section is decided sufficiently in detail to estimate the construction cost on the feasibility study level.

FIGURE 4D-10. ASSUMED TYPICAL SECTION OF CENTER CORE TYPE FILL DAM
(Not to Scale)



Source of embankment materials

Zone Ⅰ	Impervious material from borrow area
Zone Ⅱ	Transition material from quarry site
Zone Ⅱ'	Transition material from quarry site, excavation of dam foundation and appurtenant structures
Zone Ⅲ	Rock material from quarry site
Zone Ⅳ	Filter material carry in other place

3. Freeboard and Dam Crest Elevation

Freeboard is the difference between the crest elevation and the full water surface level in a reservoir and is shown in the following equation in consideration of various factors according to the Design Criteria For Dams which was published by Japanese National Committee on Large Dams.

$$Hf \geqq \Delta H + (R \text{ or } he/2) + ht + hs$$

where,

Hf: freeboard of dam

ΔH: rise of water surface level due to release of unexpected flood discharge (See "Spillway, Ultimate Flow-out Capacity")

R : height of wave due to wind

he: height of wave due to earthquake

ht: rise of water level due to unexpected accident in operating spillway gates, standard value ht is 0.5 m adopted

hs: addition of allowance according to type and importance of dam; standard value hs is 1.0 m adopted for fill type and zero for concrete type dam.

(a) Height of Wave due to Wind

Height of wave due to the wind is considered to be caused by deep-water wave, and then, the height of significant wave is adopted based on S.M.B. (Sherdrup-Munk-Breschneider) method which is derived from factors such as fetch and wind speed. On the other hand, since uprushing height varies considerably with embankment slope and roughness of slope, height of significant wave should be adjusted adequately with Saville method to obtain height of wave due to the wind with consideration of uprushing height as well.

The calculation results with various slope, fetch and wind speed are shown in the Figures 4D-11 and 4D-12.

In order to obtain the height of wave due to wind in the Pamacsalan dam site, the wind speed of 30 meters per second in 10 minutes on an

FIGURE 4D-11. WAVE HEIGHT OBTAINED BY S.M.B METHOD

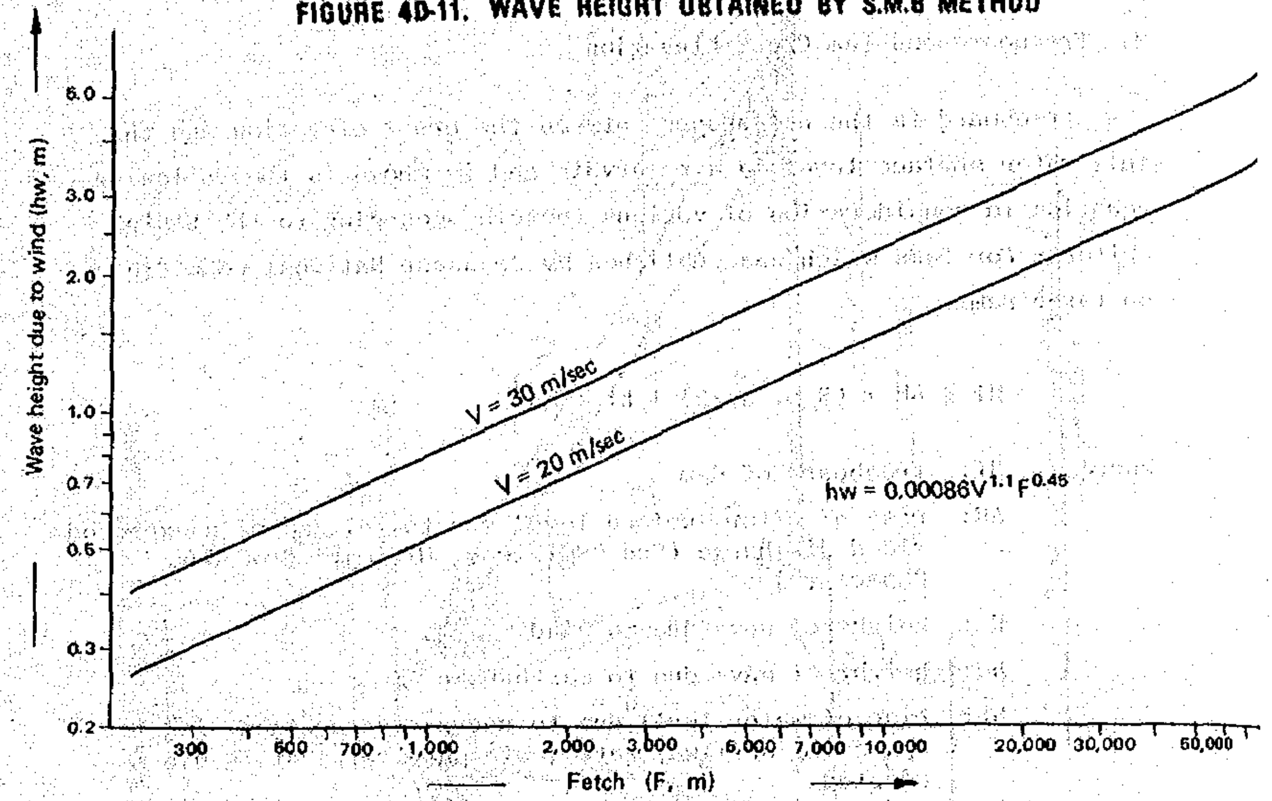
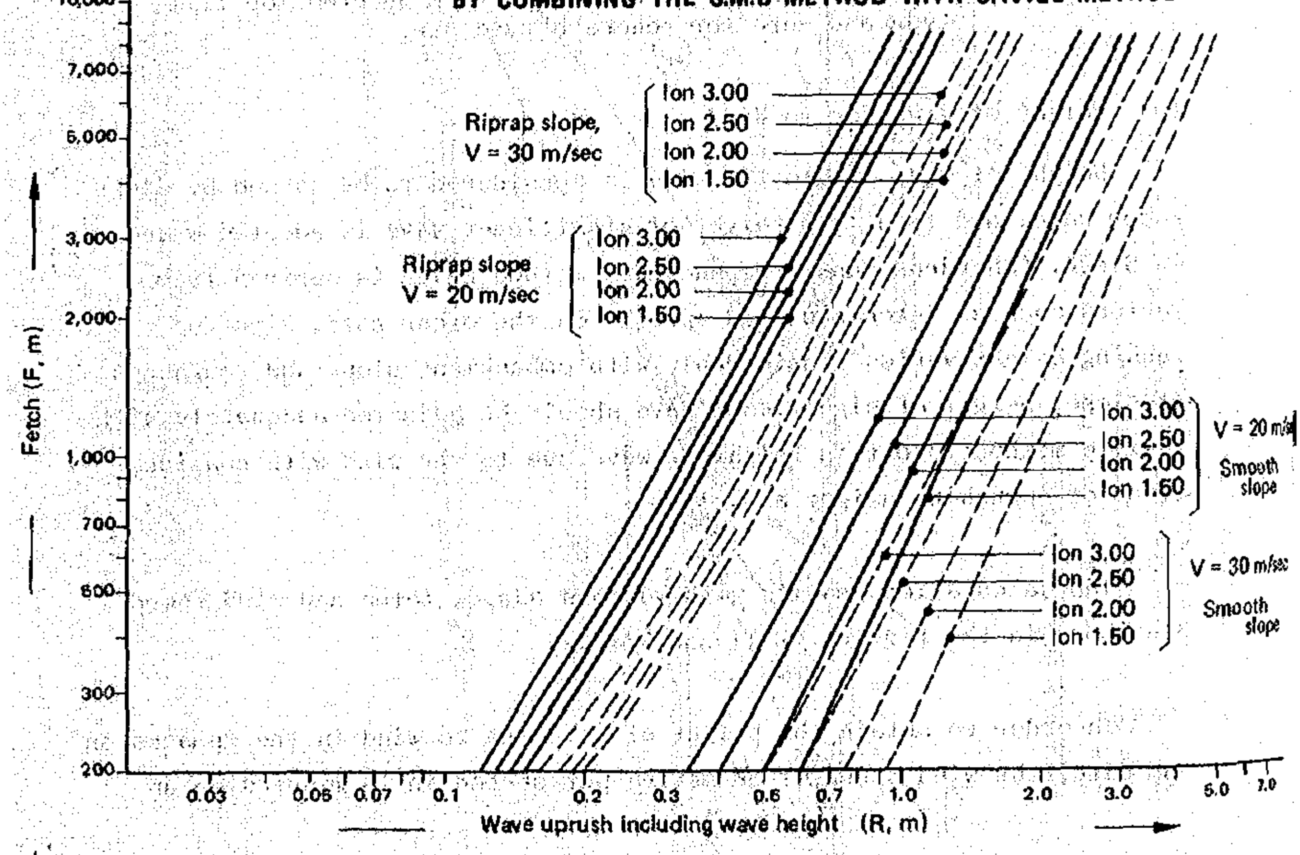


FIGURE 4D-12. WAVE UPRUSH (INCLUDING WAVE HEIGHT) OBTAINED BY COMBINING THE S.M.B METHOD WITH SAVILL METHOD



average is to be adopted taking into account the observed data of maximum instantancous wind speed in Tagbilaran city (observed maximum value is 36 m/sec in 1968) and the following conditions.

- The maximum instantaenous wind speed does not last for the blow time (usually 10 minutes) which is required for bringing about the wave due to wind.
- In many case, the wind direction does not accord with the maximum fetch direction.
- The wind speed in the Pamacsalan damsite decrease by the topographical and vegetative conditions.

(b) Height of Wave due to Earthquake

The height of wave due to earthquake can be obtained by Sato's formula, as follows;

$$he = \frac{K\tau}{2\pi} \cdot \sqrt{g.Ho}$$

where, he: height of wave at upstream face of the dam due to earthquake.

K : horizontal seismic coefficient. (K is 0.1 adopted)

τ : period of seismic wave in second. (usually, τ is 1.0 second adopted)

g : gravitational acceleration ($g = 9.8$ m/sec^2)

Ho: depth of reservoir water

(c) Freeboard

Estimated freeboard of the Pamacsalan dam is shown in the following table. The upstream surface of Pamacsalan dam is formed with rockzone by the materials obtained from quarry site, therefore, the riprap slope was adopted as the height of wave due to wind.

ΔH[1/] (m)	F (m)	R (m)	Ho (m)	he (m)	ht (m)	hs (m)	Hf (m)
0.80	1,850	0.70	55.5	0.37	0.50	1.00	3.00

Note: [1/] see spillway ultimate flow-out capacity

(d) Dam Crest Elevation

The Pamacsalan dam is constructed not only for irrigation but also for hydro-electric power, however the storage capacity for hydro-electric power purpose is about few percent of total storage capacity, therefore almost storage water is used for both irrigation and hydro-electric power purposes.

According to the results of reservoir operation studies, the storage capacity of the Pamacsalan dam with about 8 percent of reservoir losses in consideration of the geological condition and its corresponding water surface elevation are tabulated as follows;

Water level	Storage capacity ($\times 10^3 m^3$)	Water surface elevation (EL.m)	Area of water surface ($\times 10^3 m^2$)
Full water	31,300	248.5	1,269.6
Dead water	1,120	207.5	201.8

From the above table, the Pamacsalan dam crest elevation without extra bank can be obtained by adding the freeboard to full water surface;

Dam crest elevation EL.248.50 + 3.00 = EL.251.50 m

4. Stability Analysis

Stability of the dam body means that a soil mass with skeleton stress and pore pressure can keep its equilibrium state in resisting to the external forces. Taking into account the above mentioned condition, the stability analysis is made by effective stress method where the pore pressure was considered.

There are two situations for pore pressure; the one is due to unsteady flow in the course of embankment and immediately after completion of embankment, and the other is due to steady flow at full or rapid drawdown condition of the reservoir.

(a) Design Values

Design values of density and shearing strength to be used for stability analysis of the dam body vary with the moisture contents and degree of compaction by the roller. The design values of impervious material for the Pamacsalan dam should be decided according to the results of soil test, however the values of transition and rock materials, the estimations are made on the data which have been obtained through various past soil test in similar nature.

(1) Impervious Material

Taking into account the property of soil mechanics, field moisture content and the dam scale, the dry density of impervious material should be controlled at more than 95 percent of the maximum dry density of compaction test, and the design values are determined from the results of soil test as follows:

Density			Shearing Strength				Permea-bility co-efficient
			U-U test [4]		C-U test [5]		
γd [1]	γt [2]	γsat [3]	φ [6]	C [7]	φ	C	
(t/m³)	(t/m³)	(t/m³)	(°)	(t/m²)	(°)	(t/m²)	(cm/sec)
1.43	1.87	1.91	5°40'	6.6	6°40'	6.6	6.9×10^{-9}

[1] dry density

[2] wet density

[3] saturated density

[4] values use for the conditions during construction or immediately after completion of embankment

[5] values use for the conditions at full or rapid drawdown of the reservoir

[6] angle of internal friction

[7] cohesion

(2) Transition Material

The dam foundation, spillway and quarry site, various materials should be mixed with in ranging fine materials to blasted coarse rock materials, however, almost of them may be consisted of blasted rock materials. The soil test for this material has not been carried out, therefore the design values of transition material were assumed with intermediate values as those of impervious material and rock material, and the results are shown in the following table.

Density			Shearing strength	
γd 1/ (t/m^3)	γt 2/ (t/m^3)	γsat 3/ (t/m^3)	ϕ 4/ (°)	c 5/ (t/m^2)
1.50	1.70	1.88	35°00'	0.0

1/ dry density, 2/ wet density, 3/saturated density
4/ angle of internal friction, 5/cohesion

(3) Rock Material

Since effective test has not been conducted for determination of design values of the rock material (limestone) except the auxiliary test, the estimation is made based on the data obtained from the past soil tests. Figure 4D-13 shows relationship between coefficient uniformity and embanked void-ratio of the blasted rock materials and the angle of internal friction.

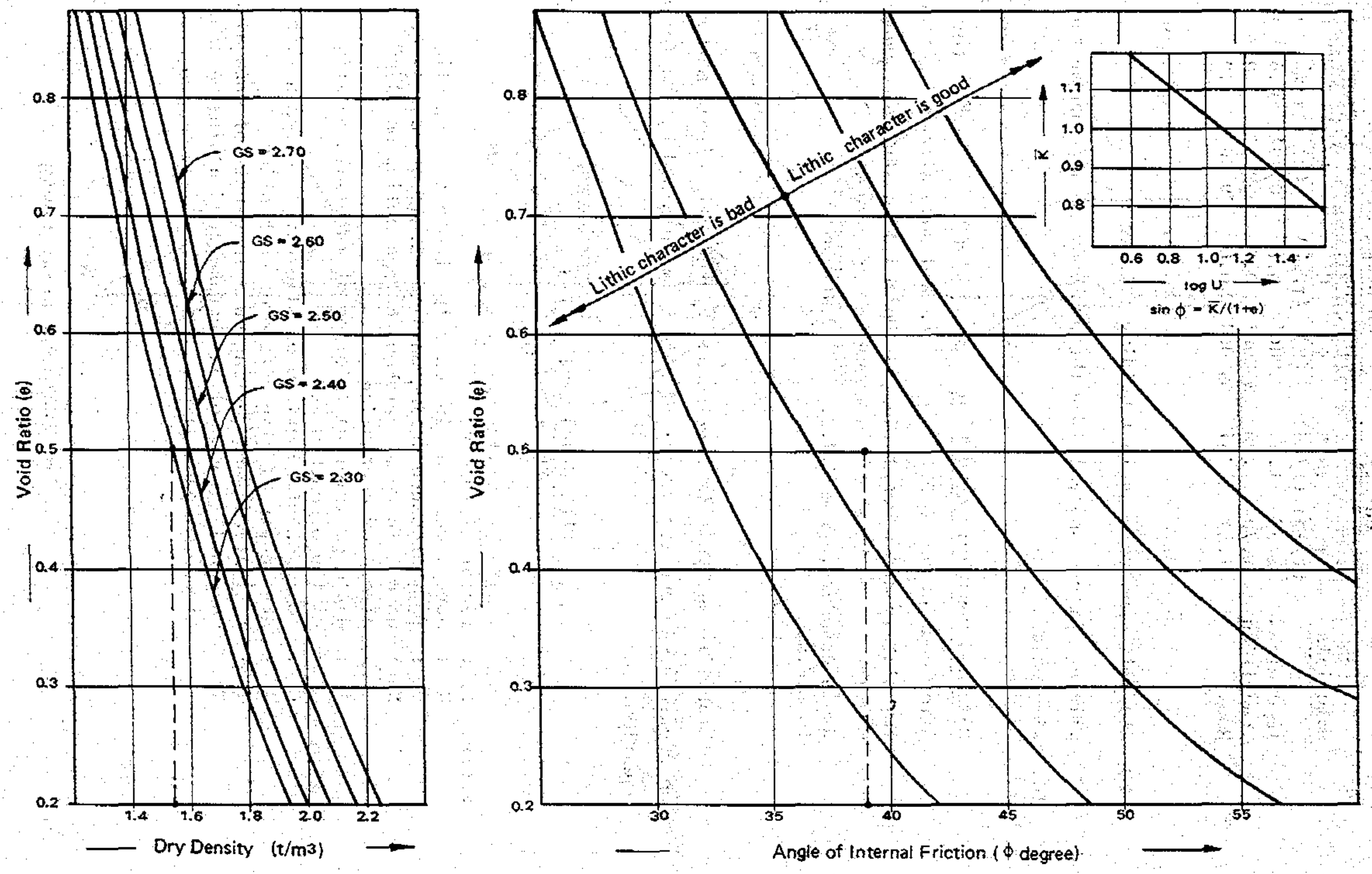
The design values of rock material are estimated as follows based on the results of auxiliary test and the above mentioned figure.

Specific gravity (G.s)	Void 1/ ratio (e)	Moisture 2/ content (%)	Density			Shearing strength	
			γd 3/ (t/m^3)	γt 4/ (t/m^3)	γsat 5/ (t/m^3)	ϕ 6/ (°)	c 7/ (t/m^2)
2.30	0.50	5.0	1.53	1.61	1.87	39°00'	0.0

1/ 2/ assumed values, 3/ dry density, $\gamma d = Gs/(1+e)$
4/ wet density, $\gamma t = \gamma d(1+M.C/100)$
5/ saturated density, $\gamma sat = (GS+e)/(1+e)$

Design values of the above mentioned embankment materials for the Pamacsalan dam are summarized in the following table.

FIGURE 4D-13. RELATION CURVE BETWEEN DRY DENSITY AND ANGLE OF INTERNAL FRICTION FOR ROCK MATERIAL



Materials	Density		Shearing strength				Permeability coefficient
			U-U test		C-U Test		
	γt	γsat	φ	C	φ	C	
	(t/m³)	(t/m³)	(°)	(t/m²)	(°)	(t/m²)	(cm/sec)
Impervious	1.87	1.91	5°40'	6.6	6°40'	6.6	6.9×10^{-9}
Transition	1.70	1.88	35°00'	0.0	35°00'	0.0	-
Rock	1.61	1.87	39°00'	0.0	39°00'	0.0	-

(b) Stability Analysis against Sliding Failure

Sliding failure analysis for the dam body is made under the following cases:

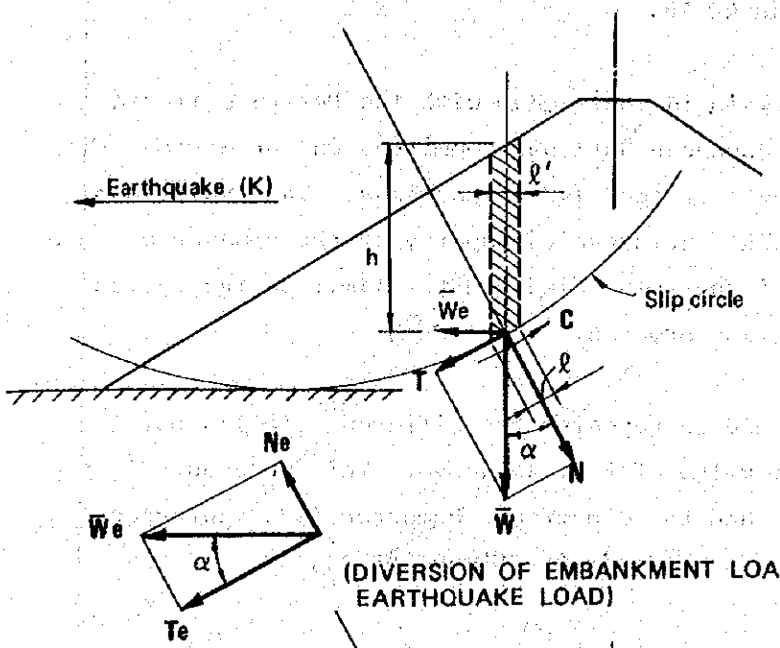
Case and reservoir condition	Slope	Pore pressure
After completion, N.W.S.	Up and down stream	Steady flow
Immediately after completion, empty	Up and down stream	Unsteady flow
Rapid drawdown, from N.W.S. to D.W.S.	Upstream	Steady flow

Stability analysis is carried out by the slip circle method shown on Figure 4D-14 and the factor of safety is obtained by the following equation.

$$F.S = \frac{\Sigma[(N-U-Ne).\tan\phi + c.\ell]}{\Sigma(T + Te)}$$

where: F.S: factor of safety
N : normal force acting on slip circle of each slice
U : Pore pressure acting on slip circle of each slice
Ne : normal force of earthquake load acting on slip circle of each slice
ϕ : angle of internal friction of materials on slip cirlce of each slice
C : cohesion of materials on slip circle of each slice
ℓ : arc length of slip circle of each slice
T : tangential force acting on slip circle of each slice
Te : tangential force of earthquake load acting on slip circle of each slice

FIGURE 4D-14. STABILITY ANALYSIS WITH SLIP CIRCLE METHOD



$\bar{W} = h \times \ell' \times \gamma$

ℓ': width of slice

γ : unit weight

$N = \bar{W} \times \cos\alpha$

$T = \bar{W} \times \sin\alpha$

$\ell = \ell'/\cos\alpha$

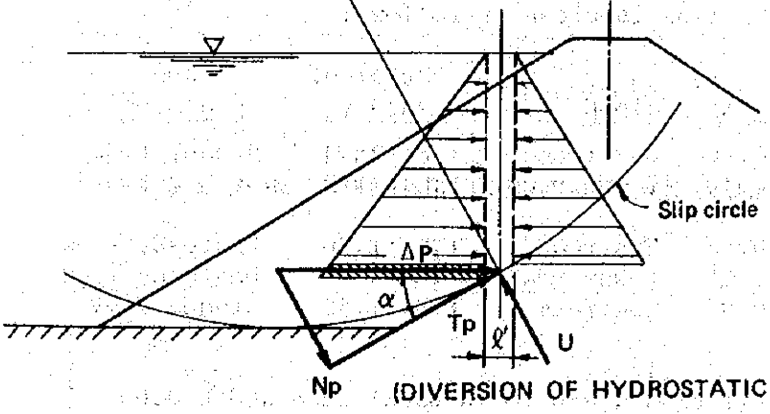
$\bar{W}e = \bar{W} \times K$

K: seismic coefficient

$Ne = \bar{W}e \times \sin\alpha$

$Te = \bar{W}e \times \cos\alpha$

(DIVERSION OF EMBANKMENT LOAD AND EARTHQUAKE LOAD)



$Np = \Delta P \times \sin\alpha$

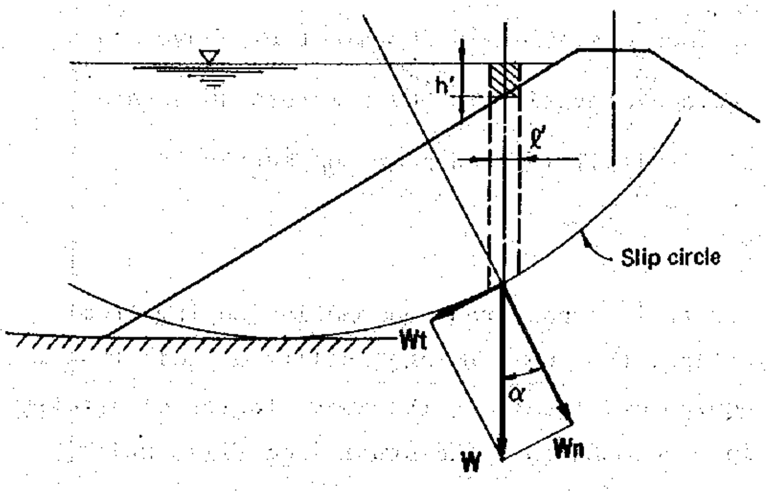
$Tp = \Delta P \times \cos\alpha$

ΔP: difference of hydrostatic pressure between both side of slice

ℓ': width of slice

U : pore pressure

(DIVERSION OF HYDROSTATIC PRESSURE)



$W = h' \times \ell' \times \gamma'$

ℓ': width of slice

γ': unit weight of storage water

$Wn = W \times \cos\alpha$

$Wt = W \times \sin\alpha$

(DIVERSION OF SURCHARGE WATER)

Pore pressure to load has been calculated by means of Hilf's method from consolidation test. And the result of Pamacsalan dam is shown in Figure 4D-15 and 4D-16.

The factor of safety is in conformity with the Design Criteria For Dams established by Japanese National Committee On Large Dams, that is to say, it must not be less than 1.2 in any case. This value comes from the consideration that the dynamic property of the embankment materials in earthquake is not clarified and the limit design system is introduced to calculation process.

The calculation should be repeated for different circles until obtained in the smallest value of F.S. as above. This procedure is the work of trial and error, and the electronic computer is advantageously used with the flow chart as shown in the Figure 4D-17.

The results of calculation is shown as follows:

Reservoir condition	K[1]	Slope	Factor of safety	Pore pressure
After completion with Full water surface (N.W.S)[2].	0.10	Upstream	F.S.=1.21[4]	Steady flow
	0.10	Downstream	F.S.=1.28	Steady flow
Immediately after Completion with empty	0.05	Upstream	F.S.=1.76	Unsteady flow
	0.05	Downstream	F.S.=1.44	Unsteady flow
Rapid drawdown from N.W.S to D.W.S [3]	0.05	Upstream	F.S.=1.55	Steady flow

[1] seismic coefficient, K=0.10 in usual case and k=0.05 in special case.

[2] full water surface elevation is 248.5 meters above sea level.

[3] dead water surface elevation is 207.5 meters above sea level.

[4] most critical circle is located near the surface of upstream slope.

The above factor of safety is considered reasonable judging from the design of feasibility stage, the quantity and quality of soil tests, and the capacity of test equipment. Besides, the above factor of safety will be increased taking into account that the stabilized fills which

FIGURE 4D-15. RELATION CURVE BETWEEN LOAD AND POREPRESSURE

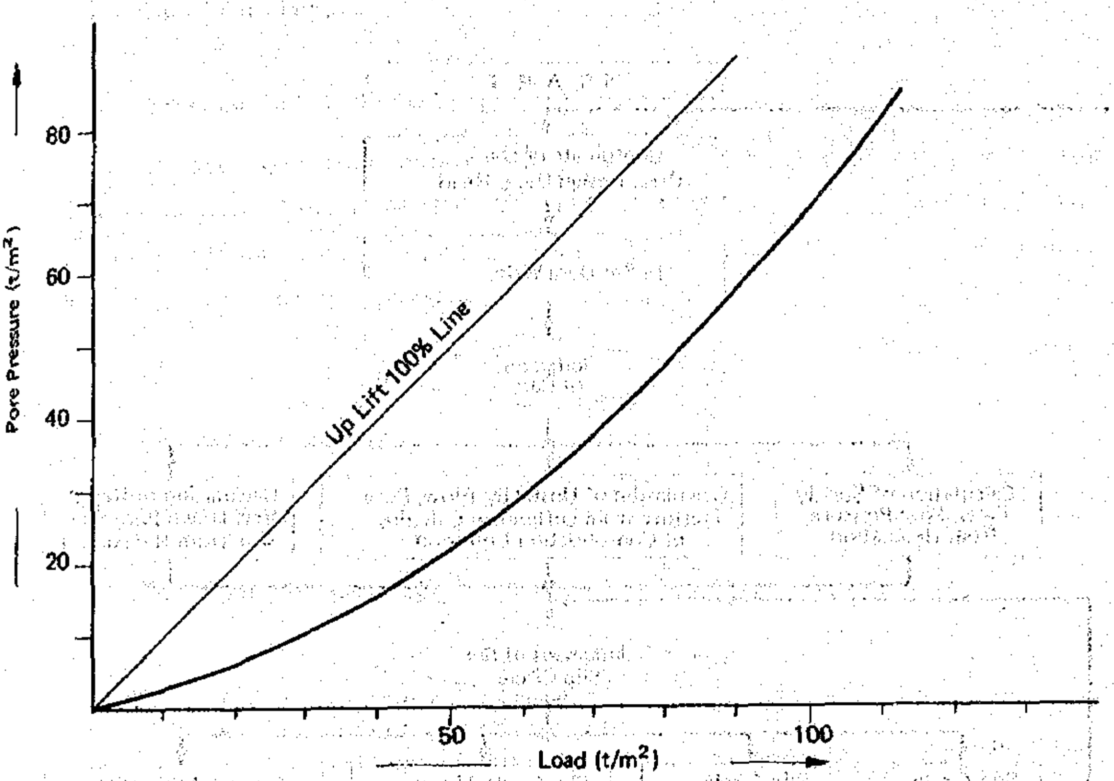


FIGURE 4D-16. RELATION CURVE BETWEEN LOAD AND SETTLEMENT

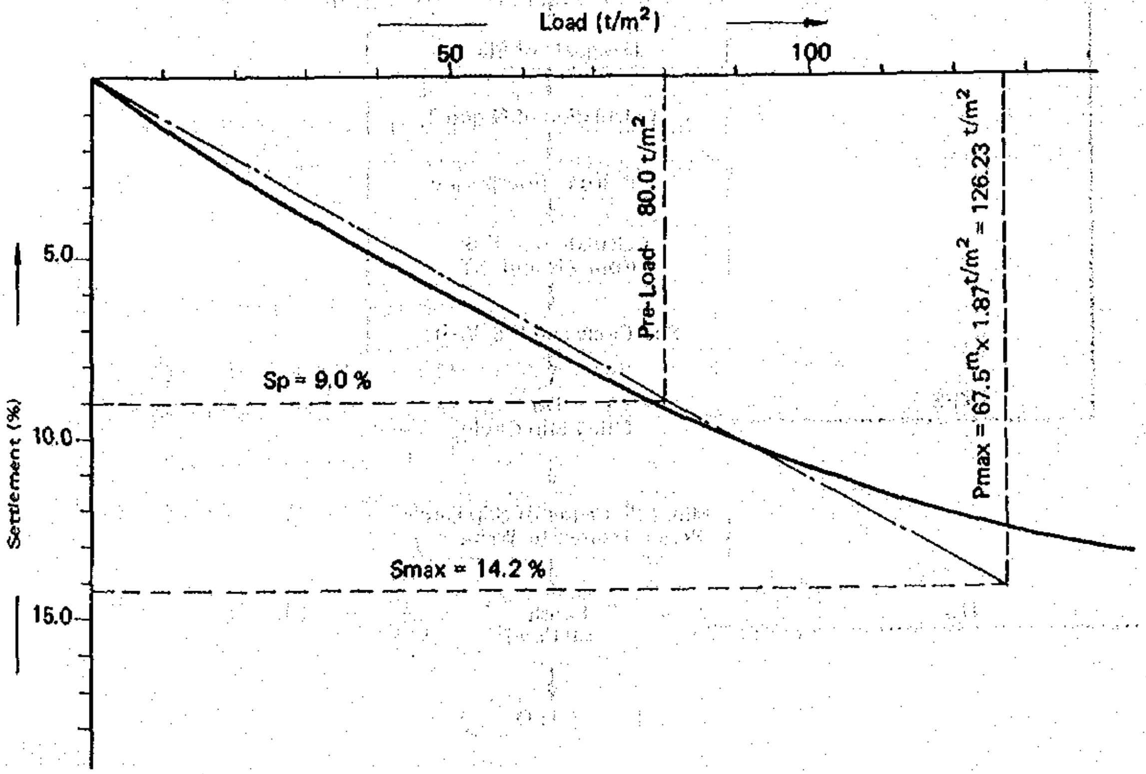
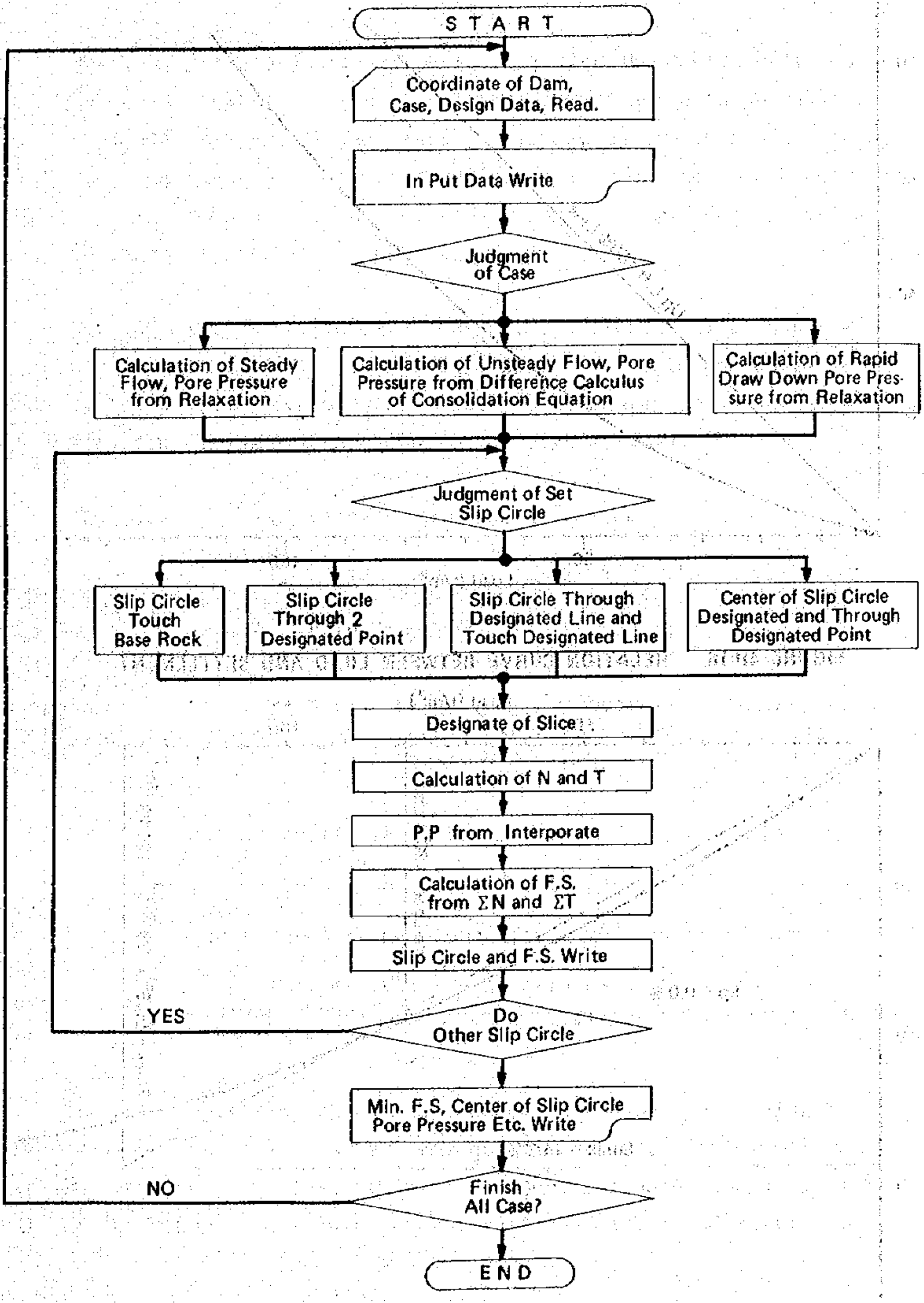


FIGURE 4D-17. FLOW CHART OF STABILITY ANALYSIS BY COMPUTER



are executed by spoil bank at the upstream and downstream sides of dam body is effective.

(c) Surface Slope Stability

It may sometimes happen that in case of dam body is constructed with cohesion less materials, critical slip circle approaches to the surface of dam body. In this case, the factor of safety can be obtained from the following formulas:

For upstream slope
$$F.S = \frac{(1-K.\frac{\gamma sat}{\gamma sub}.\tan\alpha)}{K.\frac{\gamma sat}{\gamma sub} + \tan\alpha} . \tan\phi$$

For downstream slope
$$F.S = \frac{1 - K.\tan\alpha}{K + \tan\alpha} . \tan\phi$$

where,
F.S : factor of safety
K : seismic coefficient
γsat: saturated density of outer shell material (rock zone)
γsub: submerged density of outer shell material
($\gamma sub = \gamma sat - 1$)
α : tangential value of slope
ϕ : angle of internal friction of outer shell material

Factor of safty for the surface sliding is 1.21 in upstream with slope of 1 vertical to 2.50 horizontal and 1.28 in downstream with slope of 1 vertical to 1.20 horizontal.

(d) Settlement of Dam Body

The settlement of dam body which may be caused by the self-weight and the storage water pressure should be considered in designing fill dam. The settlement after completion of dam is given by the following equation taking into account the result of consolidation test.

$$S = (1 - \frac{Sp}{Smax.}). \frac{\gamma t}{2E} . Z^2 , \qquad E = \frac{Pmax}{Smax}$$

where, S : settlement after completion of dam
Sp : settlement by pre-load in percent
Smax: settlement by maximum load in percent
γt : unit weight of dam body. (wet density)
E : coefficient of settlement from consolidation test
Z : depth below dam crest
Pmax: maximum load

In case of taking contact pressure by compaction roller, as preload the settlement by pre-load Sp. is 9.0 percent from the Figure 4D-16. The settlement after completion of the dam is equal to 1.75 meters obtained from the above equation and this value is equivalent to approximately 2.6 percent for the dam height. This is a little large for high dam caused by much content of fine materials at the soil test specimen, however considering the content of about 25 percent of coarse materials (more than 4.75 mm in diameter) in the borrow area, the value may actually be decreased and may not much affect to the point of shearing cracks due to differential settlement of dam body. Soil test with large-scale equipment and detailed study should be considered in the design stage for this problem. The profile of dam crest including extra bank is decided at zero meter on both abutment and 1.50 meters on river bed portion in trapezoidal shape.

5. Spillway

(a) Design Flood Discharge

The maximum peak discharge for spillway is assumed from an envelope curve of observed data in Philippines as shown on Figure 4D-18.

$$Q = 235A. \frac{1}{\sqrt{A} + 22} \quad \text{(extreme)}$$

where:
Q: maximum peak discharge (m^3/sec)
A: area of catchment (km^2)

The design flood discharge of spillway is in conformity with the Design Criteria For Dams established by Japanese National Committee On Large Dams, that is to say, it must be determined from the statistically estimated for a return period of one hundred years and for the fill dam, it must be based on a discharge of twenty percent more than that for concrete dam.

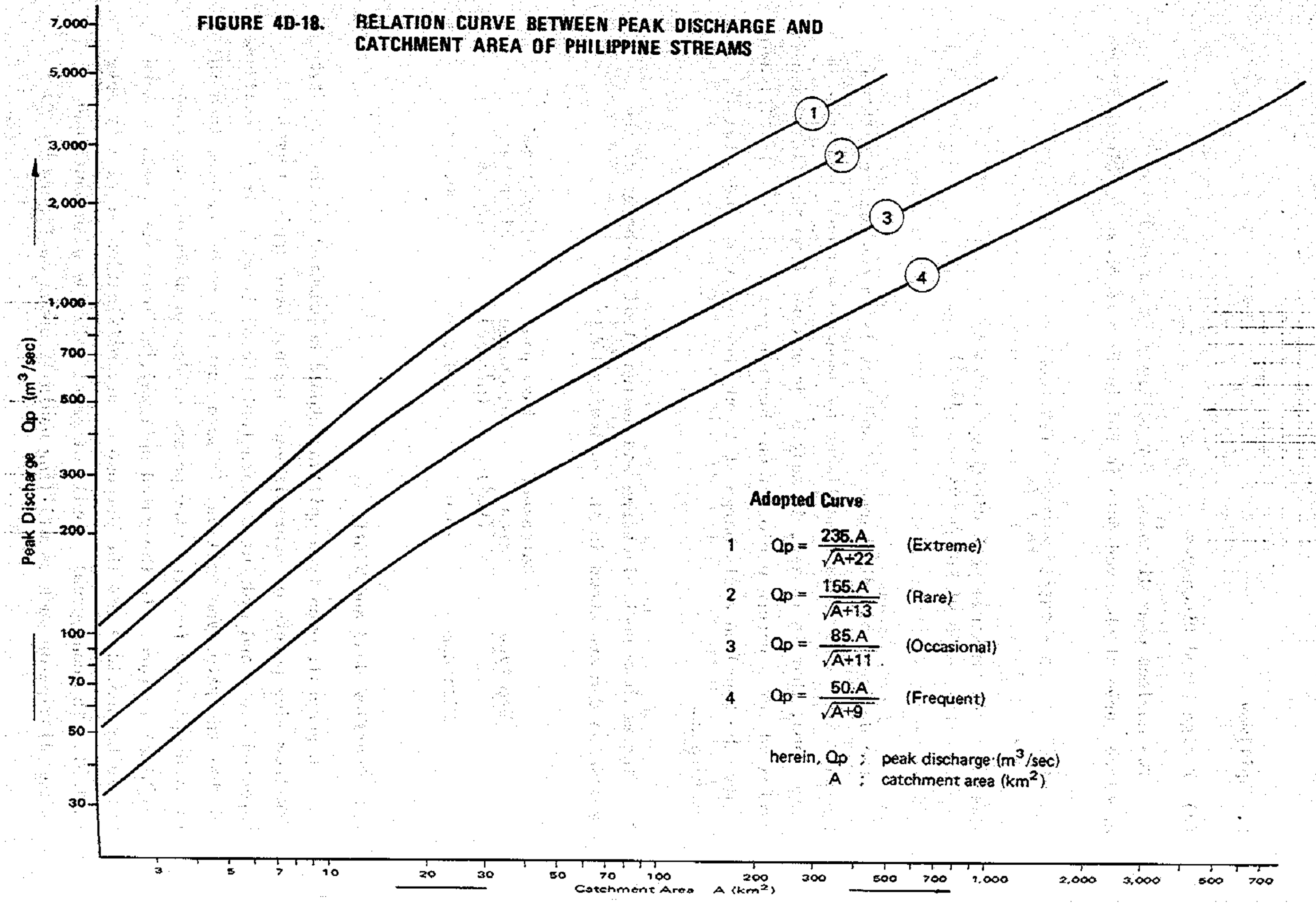
The maximum peak discharge and design flood discharge of the Pamacsalan dam are shown in the following table.

Dam type	Catchment area (km^2)	Max. peak discharge (m^3/sec)	Design flood discharge (m^3/sec)
Fill dam	28.0	930.0	600.0

(b) Overflow Head and Width

In general, open type spillway should be adopted to the fill type dam from view points of nonresistance against overtopping from unexpected flood and hydraulic characteristic of itself. It is considered that the gate type spillway is more suitable to be adopted than the ungated spillway from the view points of design discharge and topographical feature. In the gated spillway, at least two gates or more should be provided for the purpose of diversification of risk by gate control. The conceptual profile of gated spillway with chute and energy dissipator such as stilling basin is shown on Figure 4D-19.

FIGURE 4D-18. RELATION CURVE BETWEEN PEAK DISCHARGE AND CATCHMENT AREA OF PHILIPPINE STREAMS



Adopted Curve

1 $Qp = \frac{235.A}{\sqrt{A+22}}$ (Extreme)

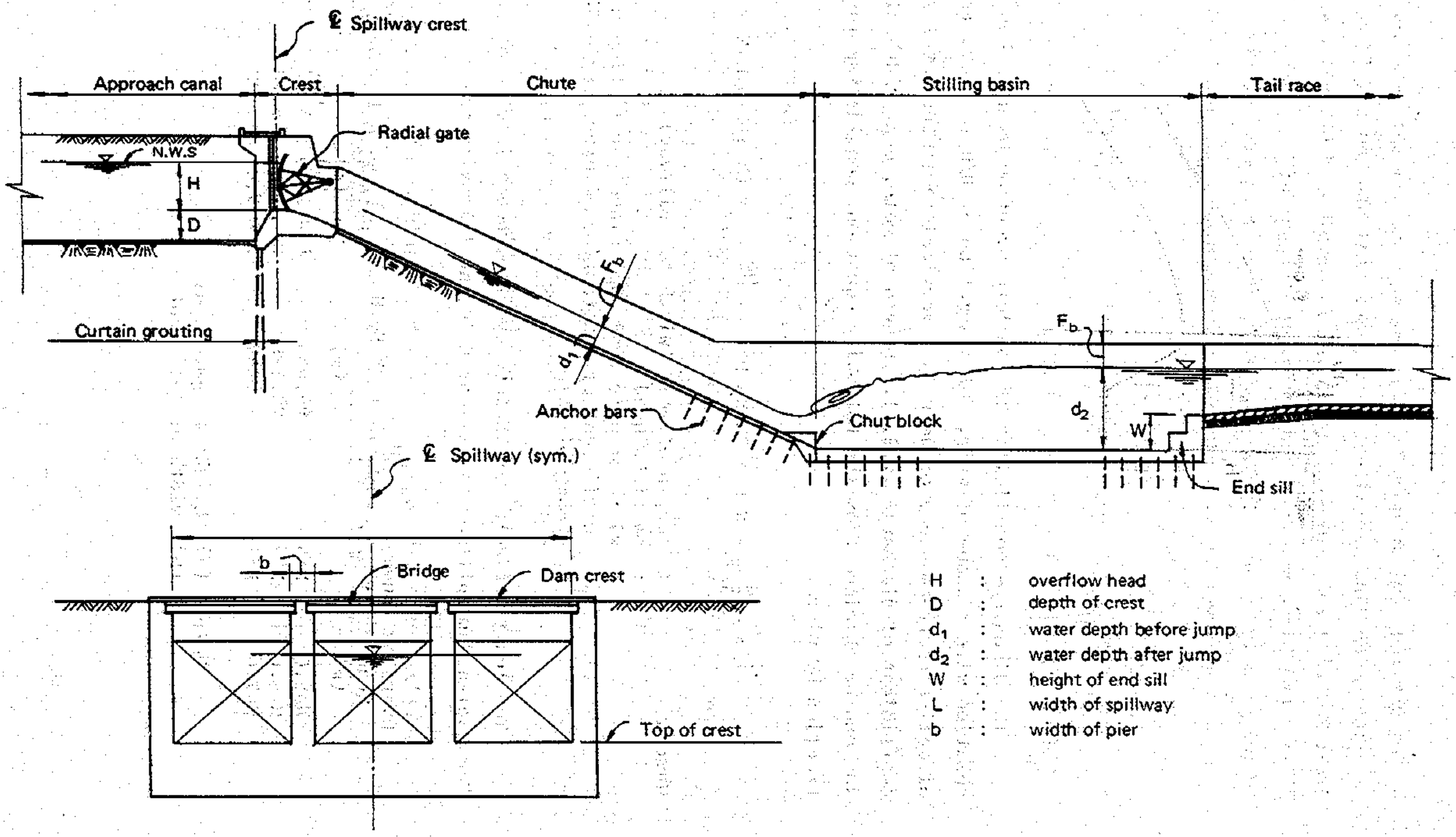
2 $Qp = \frac{155.A}{\sqrt{A+13}}$ (Rare)

3 $Qp = \frac{85.A}{\sqrt{A+11}}$ (Occasional)

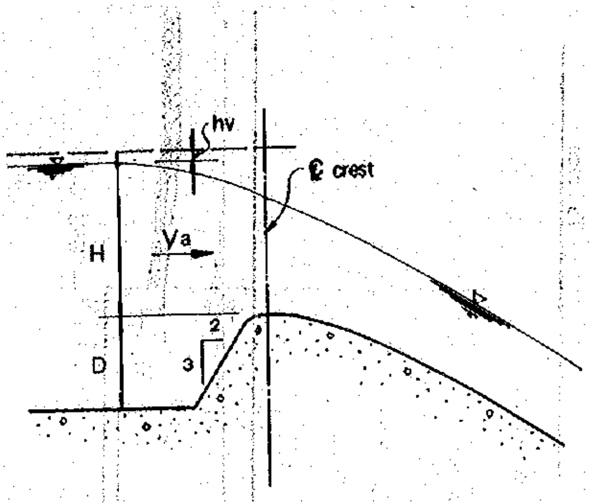
4 $Qp = \frac{50.A}{\sqrt{A+9}}$ (Frequent)

herein, Qp ; peak discharge (m³/sec)
A ; catchment area (km²)

FIGURE 4D-19. CONCEPTUAL PROFILE OF GATED SPILLWAY FOR FILL TYPE DAM
(Not to Scale)



H : overflow head
D : depth of crest
d_1 : water depth before jump
d_2 : water depth after jump
W : height of end sill
L : width of spillway
b : width of pier



The discharge coefficient of complete overflow on weir has a close relation with a shape of weir. On assumption of D/H is 0.40, the most effective upstream slope of weir is 3 vertical to 2 horizontal and coefficient of discharge is 2.11. Length of weir, e.g. width of spillway can be obtained from the following equation, considering the contraction by piers.

$$L = \frac{Qd}{CH^{3/2}} + N.b + 2N.Kp.H = \frac{Qd}{2.11.H^{3/2}} + (b + 2Kp.H)N$$

where,
L : length of weir (width of spillway)
H : overflow head
b : width of peir (b=2.20m, place for lifting unit)
Kp: coefficient of contraction on pier. (Kp=0.035)
N : number of pier

The length of weir for various overflow heads in case of the Pamacsalan dam spillway is estimated in the following table.

Overflow head (m)	Number of pier	$\frac{Qd}{CH^{3/2}} + 2N.Kp.H$ (m)	Gate size (m) (a) (b)	Length of weir (m)
H=5.0	N=4	26.835	5 - 5.40 x 5.37	35.65
H=6.0	N=2	20.188	3 - 6.40 x 6.73	24.59
H=7.0	N=1	15.844	2 - 7.40 x 7.92	18.04

Note: (a) height of gate leaf
(b) width of gate leaf

Generally, the radial type gate can be adopted for spillway gate acting on high head. In this case, it is desirable that gate leaf height and its width could nearly be equal from the view point of gate control mechanism. Taking into account the above mentioned and connect to existing river, the overflow head and the length of weir (width of spillway) for the Pamacsalan dam is decided as follows;

Overflow head (m)	Length of weir (m)	Gate size (m)
H = 6.0	L = 24.59	3 - 6.40(h) x 6.73(b)

(c) Hydraulic Dimensions

The water depth on chute section is roughly obtained in applying the following formula.

$$He(1-\alpha) = d + [q^2/2gd^2]$$

where, He: difference of elevation between reservoir full water surface and calculating point of water depth.

α : coefficient of friction loss (usually, α is 0.10 - 0.15 on fill type dam)

d : water depth of calculating point

q : discharge per unit width of spillway

g : gravitational acceleration ($g = 9.8\ m/sec^2$)

The results of calculation of water depth on the chute section for the Pamacsalan dam is shown in the following table.

Elevation (EL.m)	Water depth d (m)	Velocity[1/] V(m/sec)	Froude number[2/] Fr
238.50	2.14	14.19	3.10
218.50	1.10	22.18	6.76
198.50	0.84	29.05	10.12
180.00	0.72	33.89	12.76
170.00	0.67	36.41	14.21

[1/] $V=q/d$, [2/] $Fr=V/\sqrt{g.d}$
α is adopted 0.125 on fill dam

The running water through the chute posseses considerably high energy to bring about erosion and scouring. Actually, in the downstream area of the Pamacsalan damsite, there exist small villages and cultivated lands where the river walls and beds have no sufficient resistance to erosion and scouring. Judging from velocity and Froude number, intensive hydraulic jump shall be occurred in the energy dissipator, therefore the USDIBR (United States Department of Interior, Bureau of Reclamation) type II in providing chute blocks and end sill in the stilling basin should be adopted.

The conjugate depth in the stilling basin d_2 can be obtained by the following formula;

$$d_2 = \frac{1}{2} . d_1(\sqrt{1 + 8Fr^2} - 1)$$

where, d_2: water depth after hydraulic jump
d_1: water depth before hydraulic jump
Fr: Froude number $(Fr = V_1/\sqrt{gd_1})$

The required length of stilling basin Ls can be obtained from relation curve between Ls/d_2 and Fr. (Ls/d_2 is 4.3 on filldam)

Standard height of end sill can be calculated by the following formula.

$$\frac{W}{d_1} = \frac{(1+2Fr^2).\sqrt{1+8Fr^2}-1-5Fr^2}{1+4Fr^2-\sqrt{1+8Fr^2}} - \frac{3}{2} Fr^{2/3}$$

where,
W_1: standard height of end sill
d_1: water depth before hydraulic jump
Fr: Frould number ($Fr=V_1/\sqrt{gd_1}$)

The results of calculation for d_2, Ls and W are shown in following table.

Floor elevation (EL.m)	d1 (m)	Fr	d2 (m)	Ls (m)	W (m)
170.00	0.67	14.21	13.13	56.5	7.4

The freeboard of chute and stilling basin can be obtained from the following formulas and the results of calculation are tabulated as follows;

Chute $Fb = 0.1Vd^{1/2}$, $H = (d+Fb)/\cos\theta$
Stilling basin $Fb = 0.1(V_1+d_2)$, $H = (d_2+Fb)$

where,
Fb: freeboard of chute and stilling basin
H : vertical height of wall
V,d: velocity and water depth of calculating point
V_1 : velocity before hydrualic jump
d_2: water depth after hydraulic jump
θ : slope angle of chute floor

Calculating point	Chute (m)		Stilling basin (m)	
(EL.m)	Fb	H	Fb	H
238.50	2.08	4.5	-	-
218.50	2.33	3.8	-	-
198.50	2.66	3.9	-	-
180.00	2.88	4.0	-	-
170.00	-	-	4.95	18.1

(d) Ultimate Flow-out Capacity

It is assumed that an unexpected flood discharge would flow into the reservoir at quantity over the design flood discharge of spillway, the flowout capacity can roughly be estimated from the following formula taking into account the reservoir routing.

$$\Delta H = \frac{2}{3}.\alpha. \frac{H}{1 + \frac{(1+\beta)Af.H}{Qd.T}}$$

where,
ΔH: rise of water surface level due to unexpected discharge
α : rate of addition for unexpected flood discharge
H : design overflow head
β : increase ratio of reservoir surface area at unexpected flood.
Af: reservoir water surface area at time of design flood discharge
Qd: design flood discharge
T : duration time of discharge over the design flood discharge. (usually, $T<3$ hours, T is 2 hours=7200 sec)

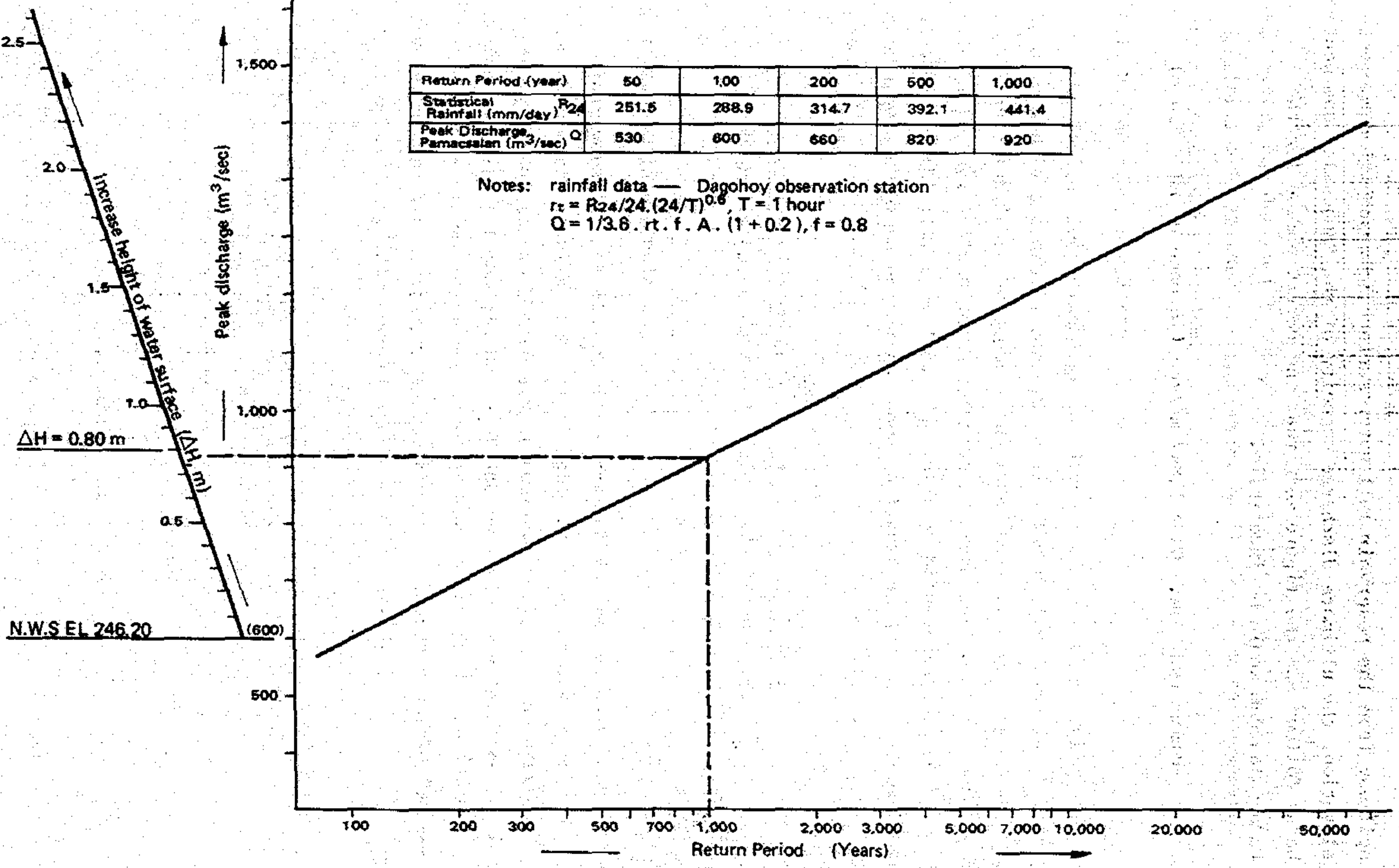
The discharge capacity of spillway for the Pamacsalan dam estimated in the following table and the relationship between the increases of water surface, discharge and return period for the Pamacsalan dam is shown on Figure 4D-20 and summarized below.

α	0.1	0.3	0.5	0.7	1.0	1.5	2.0
$(1+\alpha)Qd$ (m^3/sec)	660	780	900	1,020	1,200	1,500	1,800
ΔH (m)	0.14	0.43	0.72	1.01	1.44	2.17	2.89

$Qd = 600\ m^3/sec$, $H = 6.00\ m$, $Af = 1,269.6 \times 10^3 m^2$

It is learned from the above mentioned table that the proposed reservoir of Pamacsalan have a considerably large reservoir routing, and also from Figure 4D-20 that there would be out of question in point of the stability of the Pamacsalan dam in increase of the water surface at 1.30 meters, even if the unexpected flood with 5,000 years discharge flows into the reservoir. In defining that the ultimate discharge capacity of spillway is the discharge which flows out through the

FIGURE 4D-20. RELATION CURVE BETWEEN RETURN PERIOD AND FLOW-OUT DISCHARGE OF SPILLWAY ON PAMACSALAN DAM



Return Period (year)	50	1,00	200	500	1,000
Statistical Rainfall (mm/day) R_{24}	251.5	288.9	314.7	392.1	441.4
Peak Discharge, Pamacsalan (m^3/sec) Q	530	600	660	820	920

Notes: rainfall data —— Dagohoy observation station
$r_t = R_{24}/24.(24/T)^{0.6}$, T = 1 hour
Q = 1/3.6 . rt . f . A . (1 + 0.2), f = 0.8

spillway from the reservoir full water surface to reach the dam crest elevation, the ultimate discharge capacity of the Pamacsalan dam would be as follows and the figure mean that the return period for the Pamacsalan dam would be indefinite.

Pamacsalan Dam; Qu = 1,847 m^3/sec (in case of ΔH = 3.0 m)

6. Diversion Facilities

The tunnel type diversion facilities are provided for the Pamacsalan dam due to the topographical condition and the diversion facilities will be used for the outlet facilities from the reservoir after completion of the dam.

(a) Design Flood Discharge

The design flood discharge for diversion facilities varies case by case and is decided in conformity with the Design Criteria For Dams which was established by Japanese National Committee On Large Dams, that is to say, a flood discharge with a return period of 10 years is adopted for the fill type dam. The relation between various peak discharge and return period is shown on Figure 4D-21 and is summarized below.

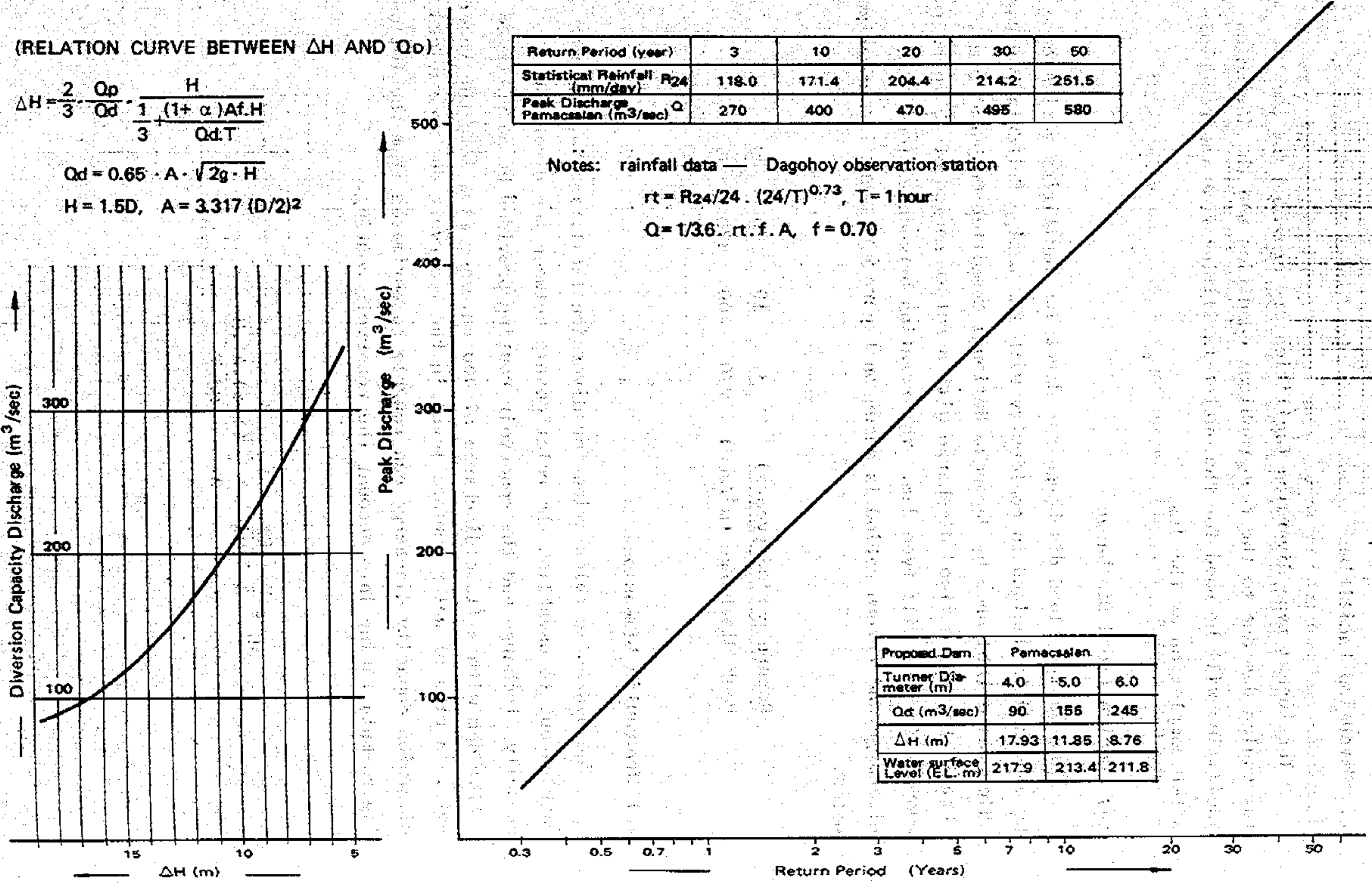
Return period (years)	3	10	20	30
Peak discharge (m^3/sec)	270	400	470	495

(b) Hydraulic Dimensions

The hydraulic calculation for diversion facilities have been carried out to establish the relationship between the diversion capacity discharge and the reservoir routing provided by an upstream coffer dam with a flood discharge of 400 cu.m/sec flow into the reservoir. A rise of water surface level in the reservoir due to various diversion capacity discharge can roughly be estimated from the following formula taking into account the reservoir routing.

$$\Delta H = \frac{2}{3} \cdot \frac{Qp}{Qd} \cdot \frac{H}{\frac{1}{3} + \frac{(1+\alpha) \cdot Af \cdot H}{Qd \cdot T}}$$

FIGURE 4D-21. RELATION CURVE BETWEEN RETURN PERIOD AND PEAK DISCHARGE, RISE WATER SURFACE LEVEL



(RELATION CURVE BETWEEN ΔH AND QD)

$$\Delta H = \frac{2}{3} \cdot \frac{Qp}{Qd} \cdot \frac{H}{\frac{1}{3} + \frac{(1+\alpha)Af.H}{Qd.T}}$$

$$Qd = 0.65 \cdot A \cdot \sqrt{2g \cdot H}$$

$$H = 1.5D, \quad A = 3.317 (D/2)^2$$

Return Period (year)	3	10	20	30	50
Statistical Rainfall (mm/day) R_{24}	118.0	171.4	204.4	214.2	251.5
Peak Discharge Pamacsalan (m3/sec) Q	270	400	470	495	580

Notes: rainfall data — Dagohoy observation station

$rt = R_{24}/24 . (24/T)^{0.73}$, T = 1 hour

$Q = 1/3.6 . rt . f . A$, f = 0.70

Proposed Dam	Pamacsalan		
Tunner Diameter (m)	4.0	5.0	6.0
Qd (m3/sec)	90	155	245
ΔH (m)	17.93	11.85	8.76
Water surface Level (EL. m)	217.9	213.4	211.8

where, ΔH: rise of water surface level due to diversion capacity discharge
Qp: peak discharge (Qp is adoped 400 m^3/sec)
Qd: diversion capacity discharge $Qd = 0.65.A.\sqrt{2g.H}$
H : head at diversion capacity discharge $H = 1.5D$
α : increase ratio of reservoir surface area at rise of water surface level
Af: reservoir surface area at time of head on diversion capacity discharge
T : duration time of discharge over the diversion capacity discharge (T is 2 hours = 7,200 sec)
A : sectional area of diversion tunne. $A = 3.317(\frac{D}{2})^2$
D : inside diameter of diversion tunnel.

The results of calculation for various capacity discharge are shown in the following table and the relationship between the increase of water surface level and diversion capacity discharge are shown on Figure 4D-21.

Capacity discharge (m^3/sec)	D (m)	ΔH (m)	Water level (EL.m)	Storage capacity ($\times 10^3 m^3$)
90	4.0	17.93	317.9	4,706.9
155	5.0	11.85	213.4	2,814.8
245	6.0	8.76	211.8	2,285.1

In selecting the diversion capacity discharge to be required for determining coffer dam crest elevation, the following are taken into account:

- The construction schedule of dam is assumed at a short period of 4 or 5 years.
- With rise of main dam embankment over the coffer dam, the discharge capacity of the diversion facilities are increased.
- The large plants and tree trunks will flow into the diversion facilities.
- Since the Pamacsalan diversion facilities is utilized as outlet

facilities after the completion of the dam, it is desirable for these facilities to have maximum velocity at 8 m/sec or less against erosion during the diversion period.

- In general, construction of the high coffer dam has a risk to flood in itself and, therefore the coffer dam height should be decided at approximately 1/3 of the main dam height.

Taking into account the above mentioned matters, the diversion capacity discharge and the inside diameter of the horse-shoe shaped concrete lined tunnel for the Pamacsalan dam is decided as follows;

Diversion capacity discharge (m^3/sec)	Inside diameter (m)	Water level (EL.m)
155	5.0	213.4

(c) Crest Elevation of Coffer Dam

The freeboard of coffer dam can be obtained as same estimation method of the freeboard on main dam. The result of calculation and adopted crest elevation of coffer dam is shown in the following table.

Fetch (m)	R1/ (m)	Freeboard2/ (m)	Water level (m)	Crest elevation (EL. m)
1,200	0.56	1.56	213.4	214.96 ≒ 215.0

1/ wind speed is 30 m/sec and U/S slope of fill type coffer dam is 1 on 2.5 with dumped riprap, therefore the height of wave due to wind R is obtained from the Figure 4D-12

2/ freeboard can be obtained as follows; $Fb = R + hs$

7. Countermeasure for Leakage from Dam Abutment

Both abutment of the Pamacsalan dam composed of marly limestone formation of the Upper Miocene to Pliocene where the Karstism marks such as cave, sinkhole and depression were found. As for the Pamacsalan damsite, it was not yet found any of existing cave or sinkhole on the right abutment, however, on the left abutment, the so called Pamacsalan cave forms with two parallel horizontal holes which was connected by the roof falling and a large scale depression was seen.

As the above mentioned geological condition, the storage water may probably be leaked out through the limestone formation of dam abutment without any countermeasure for the leakage in reservoir filling.

In general, there are two methods of the countermeasure for leakage such as grouting and blanket with impervious material methods. A grouting method is adopted from the viewpoints of the topographical, geological and economical conditions for the Pamacsalan dam. Moreover, it is almost difficult to find out the leakage path through limestone formation before reservoir filling, so that additional countermeasure may usually be provided during or after reservoir filling due to the obtained data from observation system such as an extensive net of piezometer holes. In this case, a grouting method with working platform or grouting adit is most economical and suitable one for the countermeasure of leakage problem.

It is very difficult to define the scope of countermeasure for leakage through the dam abutment with numerical value, however, it may be assumed by referring to the creep-ratio which is used for the foundation treatment of usual dams as shown in the following table.

Abutment	Extend grouting length (m)	Grouting adit length (m)	Piezometer hole (unit)
Left	300	2 rows - 430	2
Right	400	2 rows - 428	3

Fortunately, almost part of the reservoir area will be stable with comparative impervious alternation of shale and sandstone formation and this formation is continued to the downstream of damsite where the diversion dam is located. Therefore, if the leaked water through the limestone formation will have no detrimental effects on the stability of the dam and vicinity of abutment and appurtenant structures, the considerable water losses from reservoir will eventually be covered by the diversion dam and should be available again for irrigation use.

Based on the above mentioned fact, it seems that the diversion dam should be planned located near the downstream of damsite in order to diminish the loss of leakage water from reservoir.

8. Seismicity

Since Bohol Island is located in a part of the Circum Pacific Earthquake Zone, it is naturally required to pay a careful consideration on an effect of earthquake to the dam and other structures.

The earthquake around Bohol Island may almostly be occurred by the tectonic movement along Visayas and Mindanao Block and the epicenters should be located on the probable major structure lines along the above mentioned blocks. According to the data collected by the PAGASA (Philippine Atmospheric, Geophysical and Astronomical Services Administration) during the period of 1949 ~ 1970, the scale of earthquakes around Bohol Island was mostly less than magnitude 6 and the type of earthquakes was shallow or intermediate. Figure 4D-22 shows the significant epicenters of earthquakes and seismic intensity recorded around Bohol Island from 1949 to 1970. The strongest earthquake around Bohol Island recorded recently occurred on 16 November 1965 with a magnitude 5.3 based on the U.S.C.G.S scale, however there were no significant epicenter in Bohol Island.

From the observed data of earthquake around Bohol Island and the seismic coefficient used in the design of other large dams in Philippines, it seems to be reasonable that the number of 0.10 will be adopted as a coefficient of horizontal seismic force in designing the dam and other structures.

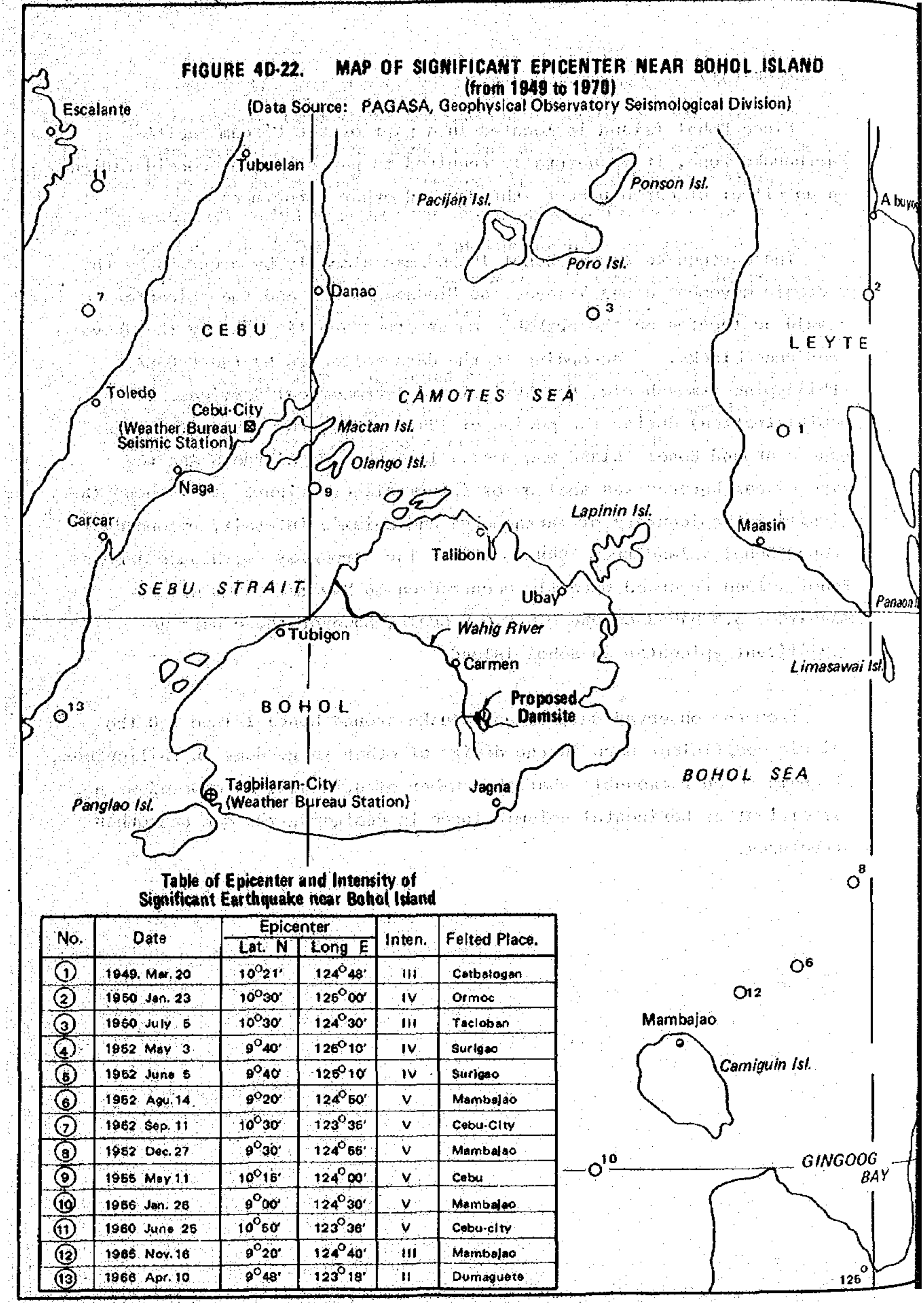


FIGURE 4D-22. MAP OF SIGNIFICANT EPICENTER NEAR BOHOL ISLAND (from 1949 to 1970)

(Data Source: PAGASA, Geophysical Observatory Seismological Division)

Table of Epicenter and Intensity of Significant Earthquake near Bohol Island

No.	Date	Epicenter		Inten.	Felted Place.
		Lat. N	Long E		
①	1949. Mar. 20	10°21'	124°48'	III	Catbalogan
②	1950 Jan. 23	10°30'	125°00'	IV	Ormoc
③	1950 July 5	10°30'	124°30'	III	Tacloban
④	1962 May 3	9°40'	126°10'	IV	Surigao
⑤	1962 June 5	9°40'	125°10'	IV	Surigao
⑥	1962 Agu. 14	9°20'	124°50'	V	Mambajao
⑦	1962 Sep. 11	10°30'	123°35'	V	Cebu-City
⑧	1962 Dec. 27	9°30'	124°55'	V	Mambajao
⑨	1965 May 11	10°15'	124°00'	V	Cebu
⑩	1966 Jan. 26	9°00'	124°30'	V	Mambajao
⑪	1960 June 25	10°50'	123°36'	V	Cebu-city
⑫	1965 Nov. 16	9°20'	124°40'	III	Mambajao
⑬	1966 Apr. 10	9°48'	123°18'	II	Dumaguete

Proposed Irrigation Canal

A. Irrigation Canal

1. Design Criteria for Irrigation Canal

(a) Canal Intensity

The canal (main, lateral and sub-lateral canals) length per hectare in the Project Area should be more than 20 meters, and also more than one (1) kilometer of canal should be needed for every 50 hectares.

(b) Water Requirement

Depending on the irrigation water requirement, the maximum water requirement is estimated at 1.414 ℓ/sec/ha including canal seepage and conveyance losses.

(c) Canal Capacities

Canal capacities will be determined to satisfy the water demand at various points in the system. The total water requirement for every canal, which is equal to the sum of maximum water requirement as mentioned above, is reflected in the schematic diagram as shown in Figure 4D-23.

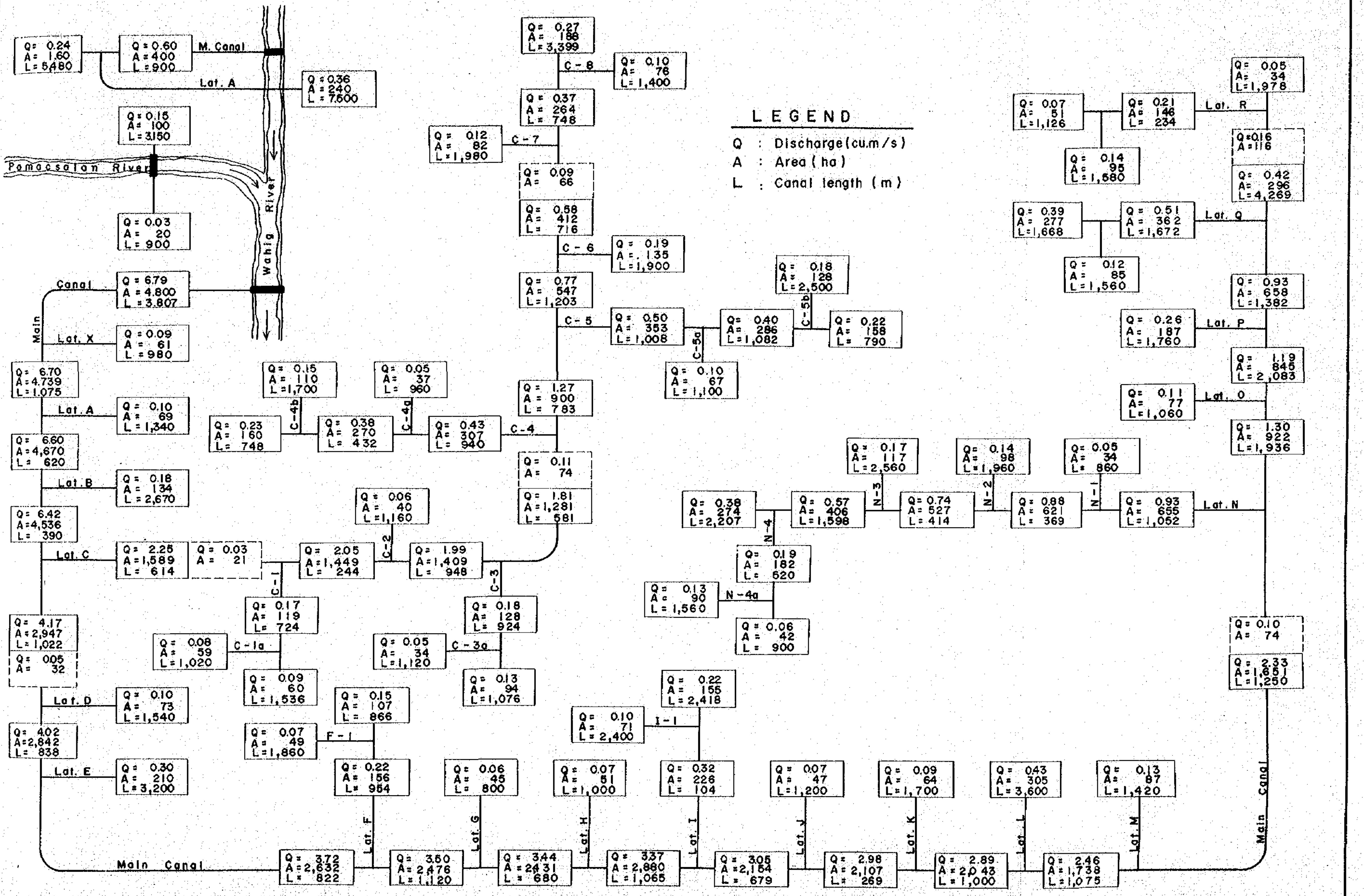
(d) Flow formula

Manning's open channel formula will be used to determine the canal elements. It is expressed in the metric system by the following;

$$V = \frac{1}{n} R^{2/3} \cdot S^{1/2}$$

where: V = velocity in meters per second
R = hydraulic radius in meter
S = slope of the canal
n = coefficient of roughness
(0.025 for earth canals with ordinary soil material)

FIGURE 4D-23 SCHEMATIC DIAGRAM OF PROPOSED IRRIGATION SYSTEM AND CANAL CAPACITY



The coefficient of roughness "n" and slope "S" are usually fixed values, thus leaving the hydraulic radius "R" as the only variable. The hydraulic radius is depending on the cross sectional area of the water and the wetted perimeter. The formula for steady and uniform flow is Q = AV, thus, following equation is produced;

$$Q = \frac{1}{n} A \cdot R^{2/3} \cdot S^{1/2}$$

where: Q = discharge in cubic meter per second
A = cross sectional area of the water in square meter

(e) Coefficient of Roughness

Manning's coefficient of roughness "n" depends largely on canal condition; however, the value of 0.025 is usually applied for ordinary earth main, lateral and sub-lateral canals. For various kinds of canal materials, the values of "n" applied are;

Coefficient of roughness "n"

Canal material	"n" values
Main and lateral earth canal	0.025
Concrete lined canal	0.015
Concrete pipe	0.014
Steel or metal pipe	0.012
Farm ditch	0.040

(f) Allowable Velocity

For the unlined canals, the maximum allowable velocity of flow should be determined as to prevent scouring, and the minimum allowable velocity should be decided so as to prevent deposition of silt or growth of aquatic plants and moss. If the velocity is not within this requirements, the slope should be revised accordingly. It may be steeper to increase the velocity if said velocity is silting or it may be made flatter to reduce the velocity if scouring. Velocity in unlined canals ordinarily vary from a minimum of 0.30 meter per second to a maximum of 1.0 meter per second.

In case where the velocity won't be as appricable as changing the slope, Kennedy's formula for critical velocity is used as given by the expression;

$$Vs = C \cdot D^{0.64}$$

where: Vs = velocity for non-silt and non-scoure or critical velocity in meter per second
D = depth of water in meter
C = coefficient for various types of soil

Values of coefficient C are as follows:

for fine, light, sandy soil	0.46
for coarse, light, sandy soil	0.51
for sandy, loamy silt	0.56
for coarse silt or hard soil debris	0.60

A suggested modification of Kennedy's formula for clear water is;

$$Vs = C \cdot D^{0.5}$$

(g) Canal Slope

The water surface grade line is tentatively drawn on the profile of the natural ground with the most appropriate slope; the flattest is 0.0002 for main canal with discharge of 10 cubic meter per second or more, and the steepest slope is 0.003 for canals with discharge of 0.50 cubic meter per second or less. Usually a slope of 0.002 is the steepest adopted for canals with discharges between 0.50 to 1.00 cubic meter per second. These limiting slope are applicable to canals on soil of the average loam.

(h) Canal Section

The section of the canal as previously determined should be such that it can take in the maximum capacity as discussed in sub-paragraph A-1-(c) and satisfy the relationships between bottom width, water depth, side slopes, freeboard and width of bank top or berm.

Bottom width and water depth

The depth of water should not exceed 2.0 meters except in uncommonly large canals. A bottom width-depth ratio of 2.5 has been adopted as standard for canals located on cut and fill on a relatively level ground. From experience, this proportion is the most economical under such condition.

Inside slope

The stability of construction materials for canalization is the determining factor in deciding the side slope of canal. Usually, the side slope adopted for unlined earth canal is 1:1.5 (vertical versus horizontal), which is approximately the angle of repose of ordinary earth.

Outside slope

When water runs against the fill embankments, the line of saturation tends to bend downward from the water surface through the embankment material. The rate of bend is a bariable slope of the saturation lines, depends mainly on the character and relative placement of the different types of embankment materials. The empirical slope of the saturation line is 1:4.0, commonly under ordinary conditions.

In view of above, the outside slope of 1.5:1 (horizontal to vertical) is derived for ordinary earth and height of embankment lower than 4 meters.

For the embankment higher than 4 meters, the following countermeasures should be considered for proper maintenance;

a) to use good embankment materials
b) to change the side slope to a gentler slope
c) to adopt toe-drain or flat-drain
d) to construct enough transvers structure
e) to use slope protection either by sodding or rip-rapping

Bank top width

The width of the bank top or barm is also based on the depth of water, to be more specific, to the nearest ten centimeters of that value. However, if one side of the bank top would be utilized as a roadway, a width of 6.00 meters for the main canal and 3.50 meters for the lateral or sub-lateral canal would be adopted.

Free board

Free board of canal will normally be governed by condition of the canal size and location, storm water, inflow, watersurface fluctuations caused by checks, wind action, soil characteristics, percolation gradients, operating road requirements, and availability of excavated material.

U.S. Bureau of Reclamation recommends that preliminary estimated of the free board required under ordinary conditions may be calculated by the expression;

$$Fb = \sqrt{cd}$$

where: Fb = free board in meter
c = coefficient
d = depth of water in meter

The coefficient "C" varies from 1.5 for a canal capacity of 0.5 cubic meter per second to 2.5 for a canal capacity of 30 cubic meter per second or more. Generally, it will be over estimated if the above formula will be used for deep canal. According to the Handbook of Applied Hydraulics, freeboard in the unlined canal, i.e., height of bank above water surface, varies from 30 centimeters for a small canal with a shallow depth to 120 centimeters for a big capacity canal of 30 cubic meter per second or more. Consequently, the formula for function of water depth can be explained as;

$$Fb = 0.30 + \frac{d}{4}$$

It is considered that the above formula is most applicable for earth canal with a depth of 1.0 to 3.0 meters. However, for deepness less than 2.0 meters, the usual practice is to make the height of the dike 1.4 times the depth of water or $Fb = 0.4d$, but a minimum of 30 centimeters.

2. Proposed Canal Length

The following table shows the proposed length of canals.

Table 4D-2 : List of Canal Length and Service Area

Canal	Canal length (m)	Service area (ha)	Capacity (m^3/s)
Wahig (Upper area)			
Main canal	6,380	400	0.60
Lateral A	7,500	240	0.36
Sub-total	13,880		
Pamacsalan (Upper area)			
Main canal (L)	3,150	100	0.15
Main canal (R)	900	20	0.03
Sub-total	4,050		
Malinao (Lower area)			
Main canal	27,360	4,800	6.79
Sub-total	27,360		
Lateral X	980	61	0.09
" A	1,340	69	0.10
" B	2,670	134	0.18
" C	9,240	1,589	2.25
" D	1,540	73	0.10
" E	3,200	210	0.30
" F	1,820	156	0.22
" G	800	45	0.06
" H	1,000	51	0.07
" I	2,520	226	0.32
" J	1,200	47	0.07
" K	1,700	64	0.09
" L	3,600	305	0.43
" M	1,420	87	0.13
" N	5,640	655	0.93
" O	1,060	77	0.11
" P	1,760	187	0.26
" Q	3,340	362	0.51
" R	1,360	146	0.21
Sub-total	46,190		
Sub-lateral C-1	2,260	119	0.17
" C-1a	1,020	59	0.08
" O-2	1,160	40	0.06
" C-3	2,000	128	0.18
" C-3a	1,120	34	0.05

Canal		Canal length (m)	Service area (ha)	Capacity (m^3/s)
Sub-lateral	C-4	2,120	307	0.43
"	C-4a	960	37	0.05
"	C-4b	1,700	110	0.15
"	C-5	2,880	353	0.50
"	C-5a	1,100	67	0.10
"	C-5b	2,500	128	0.18
"	C-6	1,900	135	0.19
"	C-7	1,980	82	0.12
"	C-8	1,400	76	0.10
"	F-1	1,860	49	0.07
"	I-1	2,400	71	0.10
"	N-1	860	34	0.05
"	N-2	1,960	98	0.14
"	N-3	2,560	117	0.17
"	N-4	1,420	132	0.19
"	N-4a	1,560	90	0.13
"	Q-1	1,560	85	0.12
"	R-1	1,580	95	0.14
	Sub-total	39,860		
	Total	131,338		

B. Related Structures

1. Conveyance Structures

(a) Canal Lining

The reaches of canal where excess seapages or serious slope sliding occur shall be lined with concrete or rip-rap. Drawing No.010 shows the typical drawing of lining canal sections.

(b) Road Crossings

Where hydraulic head is available and the discharge is less than 3.0 cubic meter per second a road crossing of reinforced concrete pipe with concrete transition will be used instead of a bridge. The pipe shall be set on a minimum slope of 0.005, and provided with a minimum of 0.90 meter of earth cover except for farm roads which will have a minimum cover of 0.60 meter. Drawing No.013 shows a typical road crossing with check and Drawing No.014 shows a typical pipe crossing.

(c) Inverted Siphons

Precast reinforced concrete pipes will be used for siphons conveying water less than 3 cubic meter per second, but concrete box section for siphons of above 3 cubic meter per second. The velocity or siphon should not exceed 1.5 meter per second. Pipe slopes should not be steeper than 2 to 1 and flatter than a slope of 0.005. A typical box siphon or a pipe siphon are shown in No.011 and No.012.

(d) Bench Flumes

Bench flumes will be made of reinforced concrete in rectangular shape with inlet and outlet transitions. The reach of a flume subject to heavy silting from adjacent steep slopes of sidehills shall be covered with reinforced concrete slabs.

(e) Drops

The grade control structures are required to convey water over relatively steep slopes to lower the grades and water surface in the canals and laterals. The types of drops will be determined according to slope and drop of the canal. In practice, drops are convined with check gate. (See Drawing No.015 or No.016)

2. Regulating Structures

(a) Checks

Checks shall be built where needed to regulate the canal water surface upstream of the structure and to control the downstream flow. Checks would be combined, where possible, with the inlets to other structures such as siphon or drops. An interval of checks will be determined based on the canal slopes and canal properties.

Use of stoplogs shall be limited accordingly to velocity, discharge and depth of water passing through a check structure.

Overflow walls should be provided on both sides of the gates or stoplogs, and the top of these walls will be set at or slightly above the control water surface. (See Drawing No.013 or No.015)

(b) Headgates

Turnout should be placed, avoiding places of high embankment or deep excavation in canal reaches, according to the following conditions; 1) nearest to the command area as much as practicable, 2) where flow is steady, and 3) operation and maintenance of the structure is convenient. The maximum velocity in the pipe should be about 1.0 meter per second more or less. In order to check the amount of flow, a parshall flume will be installed on the downstream of the headgate. Drawing No.017 and No.018 are typical head gates and Drawing No.019 shows a parshall flume.

3. Protective Structures

(a) Wasteways

Wasteways if needed shall be placed at the immediate downstream of a reach of a canal where 1) the storm water or irrigation water entering into a canal is more than 120 percent of the design in flow of the canal, 2) the point where a natural drainage-channel of adequate capacity exists, 3) the upstream of a long reach of high embankment, 4) the inlet of a long and large siphon and terminals of main canals and laterals.

(b) Drainage Culverts

Drainage culverts would be required to drain storm run-off under canals. Design capacities for the culvert shall be computed by estimating run-off from the drainage area. Drawing No.020 and No.021 are typical drainage culverts.

(c) Drain Inlets

Drain inlets will be used to carry relatively small amount of storm runoff or drainage water into the canal when an economical means of crossing the canal is not available. But the maximum drainage inlet capacity at any point should not exceed 10 percent of the capacity of the canal unless overflow wasteway facility is provided for a reach of canal within which the inlet is located. Drawing No.022 shows a typical drain inlet structure.

4. Proposed Structures

The numbers of proposed related structures are estimated on the topographic map with scale of 1:4,000 and the profile prepared from topo-map. The following table is the list of structures.

Table 4D-3. List of Related Structures

Name of structures	Number
1. Siphon	7
2. Siphon and thresher crossing	2
3. Siphon and drop	1
4. Drop	145
5. Drop with check	1
6. Drop and thresher crossing	65
7. Drop with check and headgate	5
8. Road crossing	19
9. Road crossing with drop	5
10. Road crossing with check	1
11. Thresher crossing	28
12. Thresher crossing with check	23
13. Thresher crossing with check and headgate	1
14. Thresher crossing with check, drop and headgate	8
15. Check and headgate	5
16. Headgate	1
17. Headgate and thresher crossing	1
18. End check	47
19. Parshall flume	41
20. Drainage culvert	51
Total:	457

Proposed Drainage Canal

A. Design Criteria of Drainage Canal

1. Design Discharge of Canal

The design discharge of drainage canal is estimated by multiplying the drainage area by the design discharge as shown below;

Discharge Criteria for Drainage

Area (ha)	Drainage Modulus (lit/sec/ha)
0 - 400	6.60
400 - 1,000	6.40
1,000 - 3,000	6.14
3,000 - 5,000	6.01
5,000 - 10,000	5.87

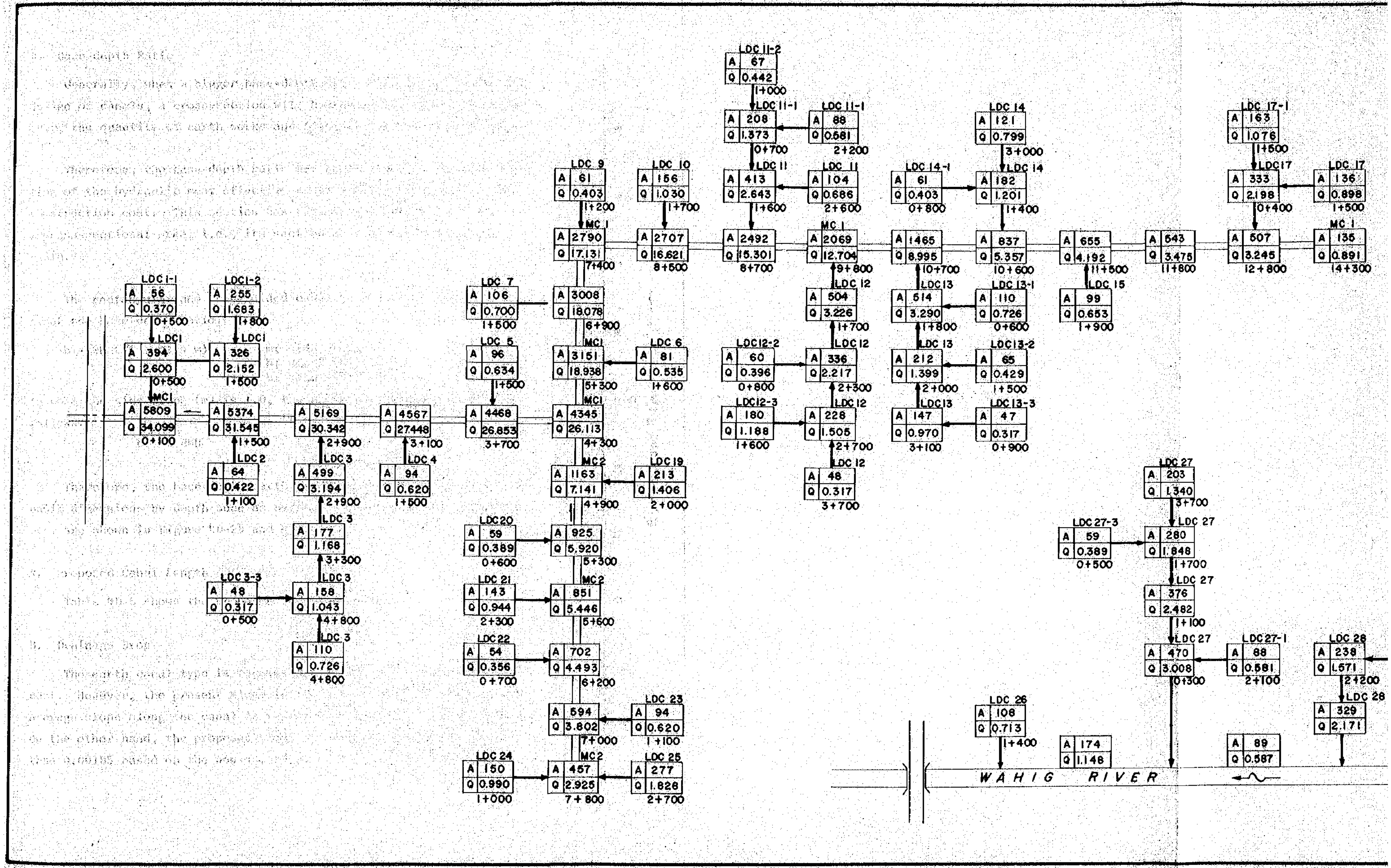
Figure 4D-24 indicates the schematic diagram of drainage system and its canal capacity.

2. Determination of Side Slope

The side slope of earth canals should be stabilized without a slip, scour or erosion against the water in the canal and natural conditions. Generally, the side slope is determined based on the soil in the project area. The soil is ranged clay or clay-loam in the project area after the field survey. Therefore, the side slope is decided at 1:1. This side slope is the same as NIA's criteria.

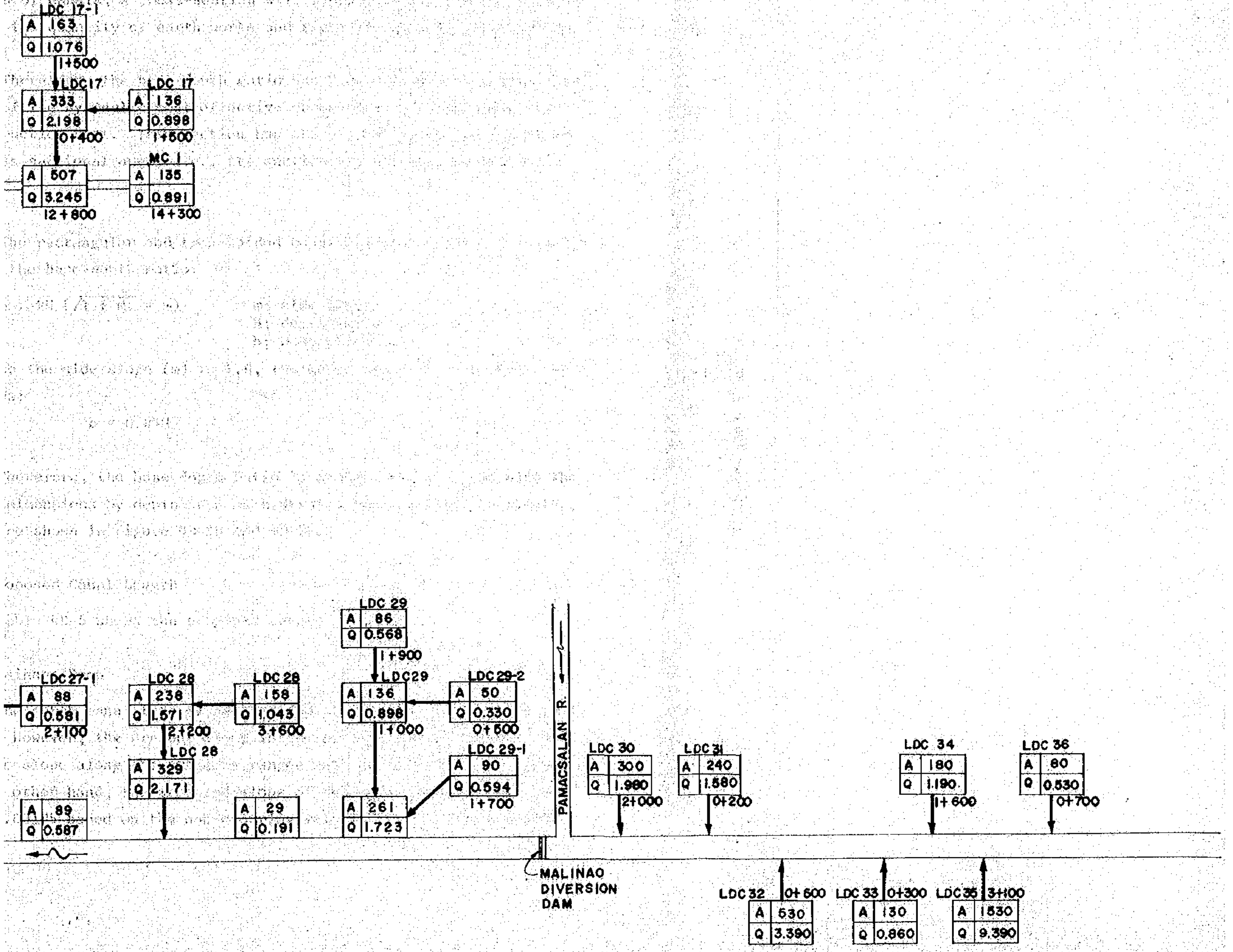
Typical Side Slope by Soil

Cut in firm soil:	1:1
Cut or fill in loam:	1:1.5
Cut or fill in sandy loam:	1:2.5



LDC 11-2
A 67
Q 0.442
1+000
LDC 11-1
A 208
Q 1.373
LDC 11-1
A 88
Q 0.581
2+200
0+700
LDC 11
A 413
Q 2.643
LDC 11
A 104
Q 0.686
2+600
1+600
LDC 9
A 61
Q 0.403
1+200
LDC 10
A 156
Q 1.030
1+700
MC 1
A 2790
Q 17.131
7+400
A 2707
Q 16.621
8+500
A 2492
Q 15.301
8+700
MC 1
A 2069
Q 12.704
9+800
LDC 14
A 121
Q 0.799
3+000
LDC 14-1
A 61
Q 0.403
0+800
LDC 14
A 182
Q 1.201
1+400
A 1465
Q 8.995
10+700
A 837
Q 5.357
10+600
A 655
Q 4.192
11+500
A 543
Q 3.475
11+800
LDC 17-1
A 163
Q 1.076
1+500
LDC17
A 333
Q 2.198
0+400
LDC 17
A 136
Q 0.898
1+500
A 507
Q 3.245
12+800
MC 1
A 135
Q 0.891
14+300
LDC 15
A 99
Q 0.653
1+900
LDC 12
A 504
Q 3.226
1+700
LDC 13
A 514
Q 3.290
1+800
LDC 13-1
A 110
Q 0.726
0+600
LDC 7
A 106
Q 0.700
1+500
A 3006
Q 18.078
6+900
LDC 1-1
A 56
Q 0.370
0+500
LDC 1-2
A 255
Q 1.683
1+800
LDC 1
A 394
Q 2.600
0+500
LDC 1
A 326
Q 2.152
1+500
LDC 5
A 96
Q 0.634
1+500
MC 1
A 3151
Q 18.938
5+300
LDC 6
A 81
Q 0.535
1+600
LDC 12-2
A 60
Q 0.396
0+800
LDC 12
A 336
Q 2.217
2+300
LDC 13
A 212
Q 1.399
2+000
LDC 13-2
A 65
Q 0.429
1+500
MC 1
A 5809
Q 34.099
0+100
A 5374
Q 31.545
1+500
A 5169
Q 30.342
2+900
A 4567
Q 27.448
3+100
A 4468
Q 26.853
3+700
MC 1
A 4345
Q 26.113
4+300
LDC 12-3
A 180
Q 1.188
1+600
LDC 12
A 228
Q 1.505
2+700
LDC 13
A 147
Q 0.970
3+100
LDC 13-3
A 47
Q 0.317
0+900
LDC 2
A 64
Q 0.422
1+100
LDC 3
A 499
Q 3.194
2+900
LDC 4
A 94
Q 0.620
1+500
MC 2
A 1163
Q 7.141
4+900
LDC 19
A 213
Q 1.406
2+000
LDC 12
A 48
Q 0.317
3+700
LDC 27
A 203
Q 1.340
3+700
LDC 3
A 177
Q 1.168
3+300
LDC 20
A 59
Q 0.389
0+600
A 925
Q 5.920
5+300
LDC 27-3
A 59
Q 0.389
0+500
LDC 27
A 280
Q 1.848
1+700
LDC 3-3
A 48
Q 0.317
0+500
LDC 3
A 158
Q 1.043
4+800
LDC 21
A 143
Q 0.944
2+300
MC 2
A 851
Q 5.446
5+600
LDC 27
A 376
Q 2.482
1+100
LDC 3
A 110
Q 0.726
4+800
LDC 22
A 54
Q 0.356
0+700
A 702
Q 4.493
6+200
LDC 27
A 470
Q 3.008
0+300
LDC 27-1
A 88
Q 0.581
2+100
LDC 28
A 238
Q 1.571
2+200
LDC 28
A 329
Q 2.171
LDC 26
A 108
Q 0.713
1+400
A 594
Q 3.802
7+000
LDC 23
A 94
Q 0.620
1+100
A 174
Q 1.148
A 89
Q 0.587
LDC 24
A 150
Q 0.990
1+000
MC 2
A 457
Q 2.925
7+800
LDC 25
A 277
Q 1.828
2+700
WAHIG RIVER

FIGURE4D-24 SCHEMATIC DIAGRAM OF PROPOSED DRAINAGE SYSTEM AND CANAL CAPACITY.



LEGEND

A = DRAINAGE AREA (HAS.)
Q = DISCHARGE (CUM/S)
MC = MAIN DRAINAGE CANAL
LDC = LATERAL DRAINAGE CANAL
0+000 = STATION NO.

3. Base-depth Ratio

Generally, when a bigger base-depth ratio will be chosen for the design of canals, a cross-section will hydraulically stabilize. However, the quantity of earth works and right-of-way will become large.

Therefore, the base-depth ratio has been determined in consideration of the hydraulic most effective cross-section to minimize the construction cost. This section has the minimum wetted perimeter to a cross-sectional area, i.e., its section has the maximum hydraulic radiu.

The rectangular and trapezoided canal have the following equation about the base-depth ratio.

$b = 2H(\sqrt{1 + m^2} - m)$

m; side slope
H; depth (m)
b; base width (m)

In case the side slope (m) is 1.0, the above equation is modified as follows:

$b = 0.84H$

Therefore, the base-depth ratio is decided at 0.8 m and also the basic dimensions by depth such as hydraulic radius, wetted perimeter, etc. are shown in Figure 4D-25 and 4D-26.

4. Proposed Canal Length

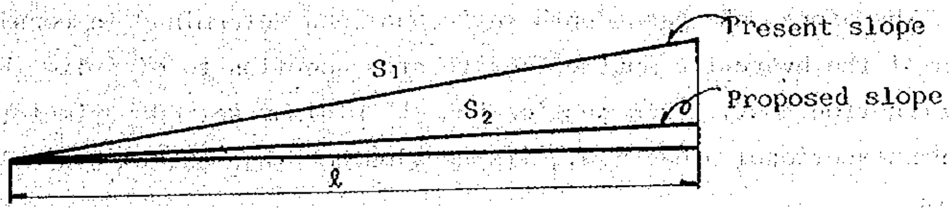
Table 4D-5 shows the proposed length of canals.

B. Drainage Drop

The earth canal type is recommended to minimize the construction cost. However, the present slope in the project area is steep. The average slope along the canal is ranged with about 0.01 on an average. On the other hand, the proposed slope of drainage canal will be less than 0.00185 based on the non-scouring velocity in the flood period.

1. Number of drops

The number of drops has been estimated in consideration of the present and proposed slope based on the topo-map (1/4,000). The standard drop head is 1.0 meter.



$No. = l \times (S_1 - S_2)/1.0$ No.; Number of drop

The number of drops is shown in the Table 4D-7 and 4D-8.

2. Design of Drainage Drops

(a) Type of Drops

There are many types of drainage drop. These drops should be designed in consideration of the non-scouring and non-erosion of approach canals because of the earth type of drainage canals.

The type of drops will be decided by the topographical condition, economic construction cost, drop head and the drainage discharge in the canals, etc. Most of the discharge are ranged with less than two cu.m/sec, and the standard drop head is one meter in the project. In this case, the drop head and discharge are not so large, therefore, the stilling pool type is recommendable. However, in case that the drop head is more than one meter and the discharge is less than 0.5 cu.m/sec, the drop with the impact box is adopted. And the inclined drop will be adopted in case both head and discharge are higher.

(b) Length of Approach Canal L (m)

$L = 1.2 + 3\sqrt{Q}/2$ Q; Discharge (cu.m/sec)

(c) Thickness of Base of Approach Canal t (m)

$t = 0.2 + 0.1\sqrt{h}$ h; Depth of uniform flow (m)

(d) Free-board of Approach Canal h_1 (m)

$h_1 = 1/3\ h > 0.30$ m

(e) Width of Notch b (m)

$h_c = 2/3\ (h + 1.1\ \nu^2/2g)$

$q = 2.98\ hc^{3/2}$

$b = Q/q$

hc; Critical depth (m)
ν; Velocity of uniform flow (m/sec)
h; Depth of uniform flow (m)
q; Unit discharge at notch (cu.m/m)

(f) Length of Stilling Pool Ls (m)

$Ls = 3(E.F)^{1/2}$

F; Drop head (m), $E = h + \frac{\alpha\nu^2}{2g}$

(g) Depth of Stilling Pool D (m)

$D = 1/2 \cdot (E.F)^{1/2}$

(h) Thickness of Base of Stilling Pool Ts (m)

$Ts = 0.1 + 0.1\ \sqrt{q.F}$

(i) Free-board of Stilling Pool F_b (m)

$Fb = 0.10 + 0.3\ \sqrt{Q}$

Table 4D-4 Average Discharge of Drop

	Number of drop by discharge								
	More than 30[1]	30 to 20	20 to 10	10 to 8	8 to 6	6 to 4	4 to 2	Less than 2	Total
Main drainage canal	2	2	5	-	2	4	-	11	26
Lateral canal	-	-	-	-	-	-	15	294	309
Total	2	2	5	-	2	4	15	305	335

Weight average discharge Qa = Σ(Discharge x No. of Drop)/No. of drop

For main canals:

More than 10 cu.m/s Qa = 185/9 = 20.56 cu.m/sec Type I

Less than 10 cu.m/s Qa = 45/17 = 2.65 cu.m/sec Type II

For lateral canals:

Less than 4 cu.m/s Qa = 339/309 = 1.10 cu.m/sec Type III

Note: 1/ Discharge in the canal (cu.m/sec)

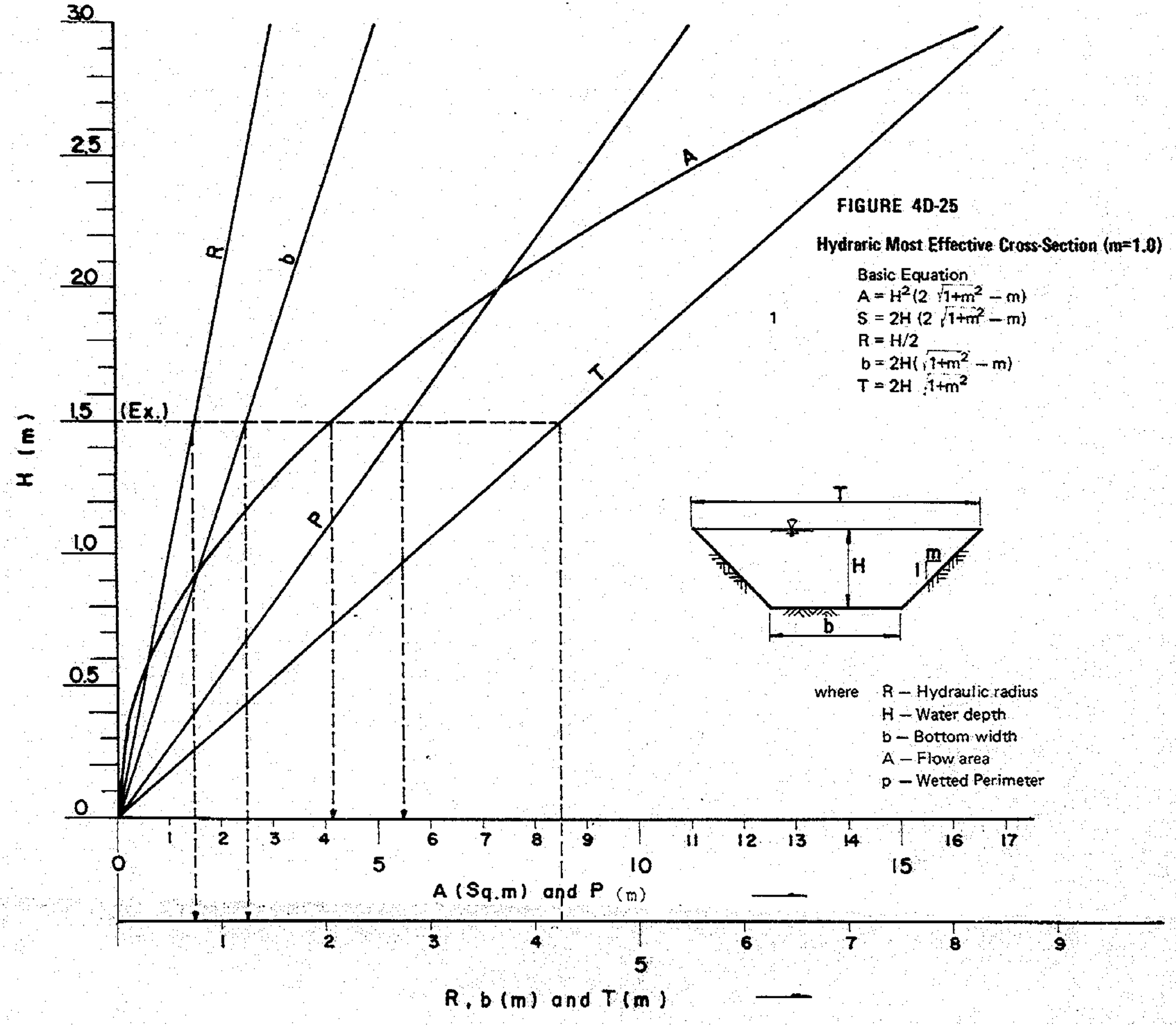


FIGURE 4D-25

Hydraric Most Effective Cross-Section (m=1.0)

Basic Equation

$A = H^2(2\sqrt{1+m^2} - m)$

$S = 2H(2\sqrt{1+m^2} - m)$

$R = H/2$

$b = 2H(\sqrt{1+m^2} - m)$

$T = 2H\sqrt{1+m^2}$

where
R — Hydraulic radius
H — Water depth
b — Bottom width
A — Flow area
p — Wetted Perimeter

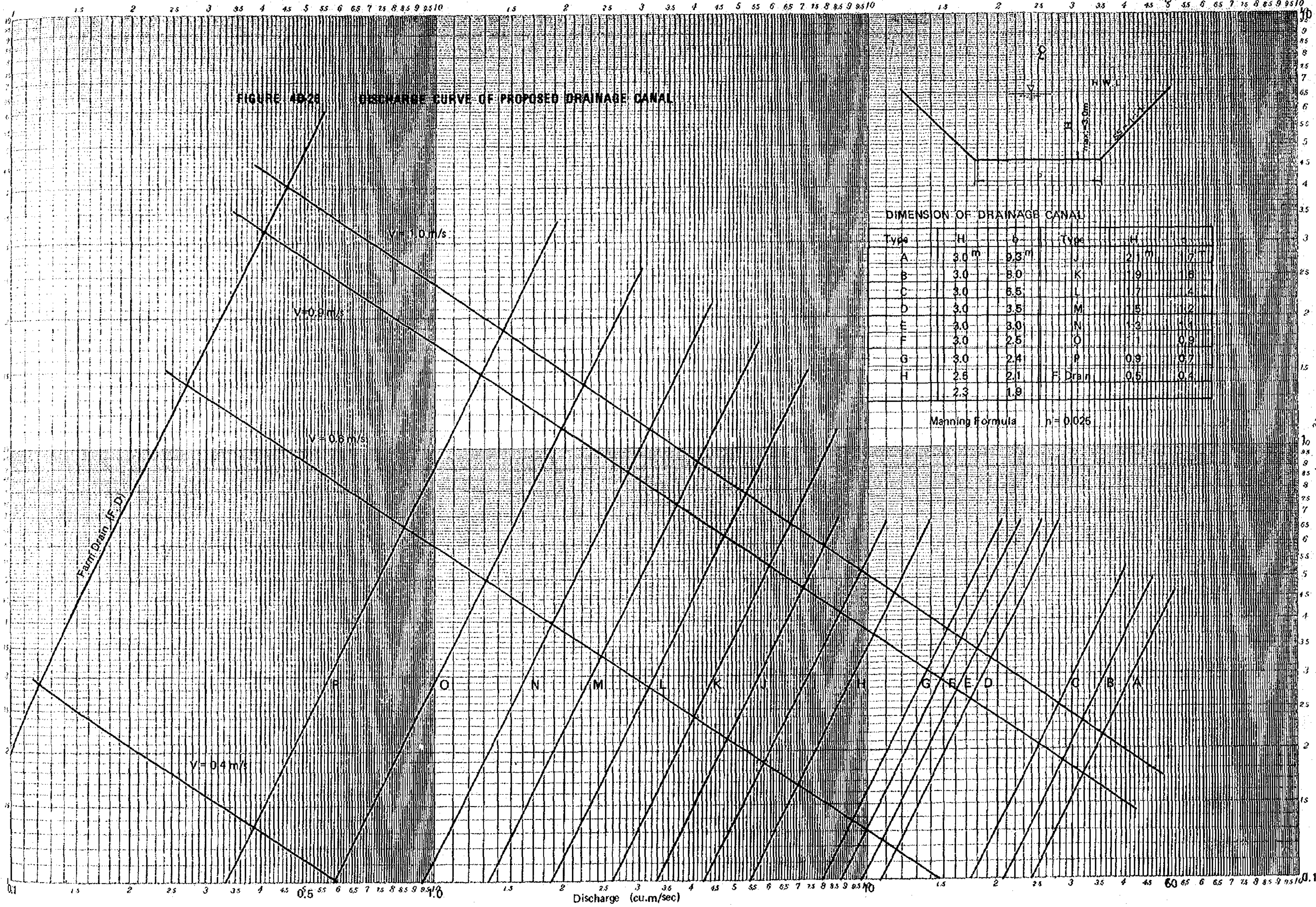


FIGURE 4D-28 DISCHARGE CURVE OF PROPOSED DRAINAGE CANAL

DIMENSION OF DRAINAGE CANAL

Type	H	b	Type	H	b
A	3.0 m	9.3 m	J	2.1 m	1.7
B	3.0	8.0	K	1.9	1.6
C	3.0	6.5	L	1.7	1.4
D	3.0	3.5	M	1.5	1.2
E	3.0	3.0	N	1.3	1.1
F	3.0	2.5	O	1.1	0.9
G	3.0	2.4	P	0.9	0.7
H	2.5	2.1	F. Drain	0.5	0.4
	2.3	1.9			

Manning Formula n = 0.025

Table 4D-5 Total Length of Drainage Canal by Type

Type	With and without Improvement			New Construction			
	M.D.C.1/ (m)	L.D.C.2/ (m)	Total (m)	M.D.C.1/ (m)	L.D.C.2/ (m)	Total (m)	Total (m)
A	100	-	100	-	-	-	100
B	2,800	-	2,800	-	-	-	2,800
C	1,400	-	1,400	-	-	-	1,400
D	2,600	-	2,600	-	-	-	2,600
E	500	-	500	-	-	-	500
F	1,100	-	1,100	-	-	-	1,100
G	200	-	200	-	-	-	200
H	1,100	-	1,100	-	-	-	1,100
I	900	3,100	4,000	-	-	-	4,000
J	600	-	600	-	-	-	600
K	800	-	800	-	-	-	800
L	1,300	-	1,300	-	-	-	1,300
M	1,100	500	1,600	-	1,000	1,000	2,600
N	-	2,700	2,700	-	-	-	2,700
O	-	8,400	8,400	-	1,800	1,800	10,200
P	1,500	29,100	30,600	-	22,900	22,900	53,500
W/o Improvement	1,800	10,500	12,300	-	-	-	12,300
Grand Total	17,800	54,300	72,100	-	25,700	25,700	97,800

Note: 1/ Main drainage canal (M.D.C.)

2/ Lateral drainage canal (L.D.C.)

The length of drainage canals is estimated based on the topographical map with a scale of 1:4,000 surveyed by NIA.

Table 4D-6 List of Main Drainage Canal

		Present Conditions						Proposed Length					
								Without	With Improvement			New Construction	
Canal Name	Section	Length (m)	Discharge Capacities (cu.m/sec)	Slope ($x10^{-3}$)	Velocities (m/sec)	Drainage Area (ha)	Discharge (cu.m/sec)	Improvement (m)	Slope ($x10^{-3}$)	Type	Length (m)	Type	Length (m)
MDC I	1	100				5,809	34.10	-	0.22	A	100		-
	2	1,400				5,374	31.55	-	0.23	B	1,400		-
	3	1,400	4.26	2.3	0.93	5,169	30.34	-	0.23	B	1,400		-
	4	200				4,567	27.45	-	0.26	C	200		-
	5	600				4,468	26.85	-	0.26	C	600		-
	6	600	3.62	0.33	0.43	4,345	26.11	-	0.26	C	600		-
	7	1,000				3,151	18.94	-	0.32	D	1,000		-
	8	1,600	25.46	2.75	1.61	3,008	18.01	-	0.32	D	1,600		-
	9	500				2,790	17.13	-	0.34	E	500		-
	10	1,100	9.38	0.36	0.60	2,707	16.62	-	0.36	F	1,100		-
	11	200				2,492	15.30	-	0.38	G	200		-
	12	1,100	11.41	1.36	1.05	2,069	12.70	-	0.46	H	1,100		-
	13	900	7.63	0.33	0.55	1,465	9.00	-	0.51	I	900		-
	14	100	49.11	14.00	3.61	837	5.36	-	0.66	K	100		-
	15	700	16.83	2.9	1.47	655	4.19	-	0.78	L	700		-
	16	300				543	3.48	-	0.92	M	300		-
	17	1,000	6.95	1.5	0.94	507	3.25	1,000					-
	18	1,500	1.05	8.6	1.12	135	0.89	-	1.38	P	1,500		-
Sub-total		14,300						1,000			13,300		-
MDC II	1	600				1,163	7.14	-	0.59	J	600		-
	2	400				925	5.92	-	0.66	K	400		-
	3	300				851	5.45	-	0.66	K	300		-
	4	600				702	4.49	-	0.78	L	600		-
	5	800				594	3.80	-	0.92	M	800		-
	6	800	3.18	2.25	0.98	457	2.93	800					
Sub-total		3,500						800			2,700		-
Total		17,800						1,800			16,000		-

		Present Conditions						Proposed Length					
Canal Name	Section	Length (m)	Discharge Capacities (cu.m/sec)	Slope (x10-3)	Velocities (m/sec)	Drainage Area (ha)	Discharge (cu.m/sec)	Without Improvement (m)	With Improvement Slope (x10-3)	With Improvement Type	With Improvement Length (m)	New Construction Type	New Construction Length (m)
LDC 1	1	500	14.39	4.4	1.76	394	2.60	-	1.10	N	500		-
1	2	1,000	10.22	2.1	1.12	326	2.15	-	1.38	O	1,000		-
1-1		500				56	0.37	-	1.85	P	500		-
1-2		1,800				255	1.68	-	1.38	O	1,800		-
2		1,100				64	0.42	-			-	P	1,100
3	1	1,900	5.00	1.47	0.85	499	3.19	1,900			-		-
3	2	1,000				499	3.19	-			-	M	1,000
3	3	400				177	1.17	-			-	P	400
3	4	1,500				158	1.04	-			-	P	1,500
3-3		500				48	0.32	-			-	P	500
4	1	800				94	0.62	-	1.85	P	800		-
4	2	700				94	0.62	-			-	P	700
5	1	1,500	23.21	3.40	1.78	96	0.63	-	1.85	P	1,500		-
6	1	1,600				81	0.54	-			-	P	1,600
7	1	1,500	3.51	5.73	1.36	106	0.70	-	1.85	P	1,500		-
9	1	1,200				61	0.40	-	1.85	P	1,200		-
10	1	1,600	3.25	8.06	1.41	156	1.03	-	1.85	P	1,600		-
11	1	1,400	2.34	6.86	1.20	413	2.64	-	1.10	N	1,400		-
11	2	1,200	5.65	6.08	1.56	104	0.69	-	1.85	P	1,200		-
11-1	1	700				208	1.37	-	1.85	P	700		-
	2	1,000				88	0.58	-	1.85	P	1,000		-
	3	500				88	0.58	-			-	P	500
11-2		1,000				62	0.44	-			-	P	1,000
12	1	1,700	4.65	1.76	0.88	504	3.23	1,700					-
12	2	600				336	2.22	-	1.38	O	600		-
12	3	400				228	1.51	-	1.38	O	400		-
12	4	200				48	0.32	-			-	P	200
12	5	800				48	0.32	-			-	P	800
12-2		800				60	0.40	-	1.85	P	800		-
12-3		1,600				180	1.19	-			-	P	1,600
13	1	1,800	6.66	1.61	0.92	514	3.29	1,800			-		-

Canal Name	Section	Length	Present Conditions			Drainage Area	Discharge	Proposed Length					
			Discharge Capacities	Slope	Velocities			Without Improvement	With Improvement			New Construction	
									Slope	Type	Length	Type	Length
		(m)	(cu.m/sec)	(x10-3)	(m/sec)	(ha)	(cu.m/sec)	(m)	(x10-3)		(m)		(m)
LDC 13	2	200				212	1.40	-	1.85	P	200		-
13	3	1,100				147	0.97	-	1.85	P	1,100		-
13-1		600				110	0.73	-	1.85	P	600		-
13-2		1,500				65	0.43	-	1.85	P	1,500		-
13-3	1	300				47	0.32	-	1.85	P	300		-
	2	600				47	0.32	-			-	P	600
14	1	1,400				182	1.20	-	1.85	P	1,400		-
14	2	900				121	0.80	-	1.85	P	900		-
14	3	700				121	0.80	-			-	P	700
14-1	1	300				61	0.40	-	1.85	P	300		-
	2	500				61	0.40	-			-	P	500
15	1	1,000	0.64	6.10	0.922	99	0.65	-	1.85	P	1,000		-
	2	900				99	0.65	-			-	P	900
17	1	400	6.89	1.75	1.02	333	2.20	-	1.38	O	400		-
	2	1,100				136	0.90	-	1.85	P	1,100		-
17-1		1,500				163	1.08	-	1.85	P	1,500		-
19	1	2,000				213	1.41	-			-	P	2,000
20	1	600				59	0.39	-	1.85	P	600		-
21	1	600				143	0.94	-	1.85	P	600		-
	2	1,700				143	0.94	-			-	P	1,700
22	1	100				54	0.36	-	1.85	P	100		-
	2	600				54	0.36	-			-	P	600
23		1,100				94	0.62	-			-	P	1,100
24	1	100				150	0.99	-	1.85	P	100		-
	2	900				150	0.99	-			-	P	900
25	1	1,700	3.33	2.82	0.99	277	1.83	1,700			-		-
	2	1,000				277	1.83	-			-	O	1,000
26		1,400	19.78	6.57	2.20	108	0.71	-	1.85	P	1,400		-
27	1	300	8.19	0.67	0.71	470	3.01	300			-		-
	2	800	1.01	1.38	0.54	376	2.48	-	1.10	N	800		-

Canal Name	Section	Length (m)	Present Conditions: Discharge Capacities (cu.m/sec)	Present Conditions: Slope (x10-3)	Present Conditions: Velocities (m/sec)	Drainage Area (ha)	Discharge (cu.m/sec)	Proposed Length: Without Improvement	Proposed Length: With Improvement: Slope (x10-3)	With Improvement: Type	With Improvement: Length (m)	New Construction: Type	New Construction: Length (m)
LDC 27	3	600				280	1.85	-	1.38	O	600		-
	4	1,200	20.39	5.25	2.01	203	1.34	-	1.85	P	1,200		-
	5	800				203	1.34	-			-	P	800
27-1		2,100	3.47	0.43	0.50	88	0.58	2,100			-		-
27-3		500				59	0.39	-	1.85	P	500		-
28	1	1,400	19.79	4.36	1.84	329	2.17	-	1.38	O	1,400		-
	2	800				238	1.57	-			-	O	800
	3	1,400				158	1.04	-			-	P	1,400
29	1	1,000	1.40	1.10	0.58	136	0.90	1,000			-		-
	2	400				86	0.57	-	1.85	P	400		-
	3	500				86	0.57	-			-	P	500
29-1	1	400				90	0.59	-	1.85	P	400		
	2	1,300				90	0.59	-				P	1,300
29-2		500				50	0.33	-	1.85	P	500		-
Sub-total		71,600						10,500			35,400		25,700
LDC 30		2,000				300	1.98	-	1.38	O	2,000		-
Sub-total		2,000						-			2,000		-
LDC 31		200				240	1.58	-	1.38	O	200		-
32		500				530	3.39	-	0.92	M	500		-
33		300				130	0.86	-	1.85	P	300		-
34		1,600				180	1.19	-	1.85	P	1,600		-
35		3,100				1,530	9.39	-	0.51	I	3,100		-
36		700				80	0.53	-	1.85	P	700		-
Sub-total		6,400									6,400		-
Total		80,000						10,500			43,800		25,700
Grand Total		97,800						12,300			59,800		25,700

Table 4D-7 Number of Drops (Main Drainage Canals)

Name of Canal	Section	Elevation		H	L	Slope (x10^{-3})		No. of Drop
		B.P.	E.P.			Present	Proposed	
		(m)	(m)	(m)	(m)			
MDC 1	1	101.4	101.6	0.2	100	2.00	0.22	-
	2	101.6	103.5	1.9	1,400	1.36	0.23	2
	3	103.5	105.5	2.3	1,400	1.64	0.23	2
	4	105.5	105.6	0.1	200	0.50	0.26	-
	5	105.6	105.7	0.1	600	0.17	0.26	-
	6	105.7	105.9	0.2	600	0.33	0.26	-
	7	105.9	106.0	0.1	1,000	0.10	0.32	-
	8	106.0	110.4	4.4	1,600	2.75	0.32	4
	9	110.4	110.5	0.1	500	0.20	0.34	-
	10	110.5	110.9	0.4	1,100	0.36	0.36	-
	11	110.9	111.0	0.1	200	0.50	0.38	-
	12	111.0	112.5	1.5	1,100	1.36	0.46	1
	13	112.5	112.8	0.3	900	0.33	0.51	-
	14	112.8	114.2	1.4	100	14.00	0.66	1
	15	114.2	116.2	2.0	700	2.86	0.78	1
	16	116.2	116.3	0.1	300	0.33	0.92	-
	17	116.3	117.8	1.5	1,000	1.50	0.92	-
	18	117.8	130.7	12.9	1,500	8.60	1.38	11
Sub-total								22
MDC 2	1	105.9	108.7	2.8	600	4.70	0.59	2
	2	108.7	108.8	0.1	400	0.25	0.66	-
	3	108.8	110.1	1.3	300	4.33	0.66	1
	4	110.1	111.5	1.4	600	2.33	0.78	1
	5	111.5	112.7	1.2	800	1.50	0.92	-
	6	112.7	114.5	1.8	800	2.25	-	-
Sub-total								4
TOTAL								26

Table 4D-8 Number of Drops (Lateral Drainage Canals)

Number of Canal	Section	Elevation		H	L	Slope		No. of Drop
		B.P. (m)	E.P. (m)	(m)	(m)	Present (x 10⁻³)	Proposed (x 10⁻³)	
LDC 1	1	101.6	103.8	2.2	500	4.40	1.10	2
LDC 1	2	103.8	105.9	2.1	1,000	2.10	1.38	1
LDC 1-1		103.8	107.6	3.8	500	7.60	1.85	3
LDC 1-2		105.9	120.2	14.3	1,800	7.94	1.38	12
LDC 2		103.5	109.5	6.0	1,100	5.45	1.85	4
LDC 3	1	105.8	108.6	2.8	1,900	1.47	-	-
LDC 3	2	108.6	109.4	0.8	1,000	0.80	0.92	-
LDC 3	3	109.4	113.5	4.1	400	10.25	1.85	3
LDC 3	4	113.5	115.0	1.5	1,500	1.00	1.85	-
LDC 3-3		113.5	115.0	1.5	500	3.00	1.85	1
LDC 4	1	105.9	109.2	3.3	800	4.13	1.85	2
LDC 4	2	109.2	117.7	8.5	700	12.14	1.85	7
LDC 5		105.7	110.8	5.1	1,500	3.40	1.85	2
LDC 6		105.9	119.8	13.9	1,600	8.69	1.85	11
LDC 7		110.3	118.9	8.6	1,500	5.73	1.85	6
LDC 9		110.5	116.4	5.9	1,200	4.92	1.85	4
LDC 10		110.9	123.8	12.9	1,600	8.06	1.85	10
LDC 11	1	110.9	120.5	9.6	1,400	6.86	1.10	8
LDC 11	2	120.5	127.8	7.3	1,200	6.08	1.85	5
LDC 11-1	1	120.5	123.1	2.6	700	3.71	1.85	1
LDC 11-1	2	123.1	129.3	6.2	1,000	6.20	1.85	4
LDC 11-1	3	129.3	134.9	5.6	500	11.20	1.85	5
LDC 11-2		123.1	135.0	11.9	1,000	11.90	1.85	10
LDC 12	1	112.5	115.5	3.0	1,700	1.76	-	-
LDC 12	2	115.5	115.9	0.4	600	0.67	1.38	-
LDC 12	3	115.9	120.8	4.9	400	12.25	1.38	4
LDC 12	4, 5	120.8	124.9	4.1	1,000	4.10	1.85	2

(Continued)

LDC 12-2		115.9	124.0	8.1	800	10.13	1.85	7
LDC 12-3		120.8	123.8	3.0	1,600	1.88	1.85	-
LDC 13	1	112.8	115.7	2.9	1,800	1.61	-	-
LDC 13	2	115.7	116.7	1.0	200	5.00	1.85	1
	3	116.4	125.6	9.2	1,100	8.36	1.85	7
LDC 13-1		115.7	118.9	3.2	600	5.33	1.85	2
LDC 13-2		116.4	129.4	13.0	1,500	8.67	1.85	10
LDC 13-3	1	124.3	128.7	4.4	300	14.67	1.85	4
LDC 13-3	2	128.7	135.0	6.3	600	10.50	1.85	5
LDC 14	1	114.2	123.8	9.6	1,400	6.86	1.85	7
LDC 14	2	123.8	127.5	3.7	900	4.11	1.85	2
LDC 14	3	127.5	129.6	2.1	700	3.00	1.85	8
LDC 14-1	1	123.8	123.9	0.1	300	0.33	1.85	-
LDC 14-1	2	123.9	126.0	2.1	500	4.20	1.85	1
LDC 15	1	116.2	122.3	6.1	1,000	6.10	1.85	4
LDC 15	2	122.3	127.0	4.7	900	5.22	1.85	3
LDC 17	1	117.8	118.5	0.7	400	1.75	-	-
LDC 17	2	118.5	134.0	15.5	1,100	14.09	1.85	13
LDC 17-1		118.5	135.9	17.4	1,500	11.60	1.85	15
LDC 19		108.7	114.1	5.4	2,000	2.70	1.85	2
LDC 20		108.8	113.0	4.2	600	7.00	1.85	3
LDC 21	1	110.7	114.3	3.6	600	6.00	1.85	2
LDC 21	2	110.1	110.7	0.6	1,700	0.35	1.85	-
LDC 22	1, 2	111.5	112.0	0.5	700	0.71	1.85	-
LDC 23		112.7	123.0	10.3	1,100	9.36	1.85	8
LDC 24	1, 2	114.5	119.5	5.0	1,000	5.00	1.85	3
LDC 25	1	114.5	119.3	4.8	1,700	2.82	1.85	-
LDC 25	2	119.3	124.7	5.4	1,000	5.40	1.38	4
LDC 26		104.6	113.8	9.2	1,400	6.57	1.85	7
LDC 27	1	109.6	109.8	0.2	300	0.67	-	-
LDC 27	2	109.8	110.9	1.1	800	1.38	1.10	-
LDC 27-1		108.9	109.8	0.9	2,100	0.43	1.85	-

(Continued)

LDC 27-3		113.5	118.0	4.5	500	9.00	1.85	4
LDC 28	1	120.4	126.5	6.1	1,400	4.36	1.36	4
LDC 28	2	126.5	130.8	4.3	800	5.38	1.38	3
LDC 28	3	130.8	139.0	8.2	1,400	5.86	1.85	-
LDC 29	1	124.8	125.9	1.1	1,000	1.10	-	-
LDC 29	2, 3	125.8	136.0	10.2	900	11.33	1.85	9
LDC 29-1	1	124.8	125.8	1.0	400	2.50	1.85	-
LDC 29-1	2	125.8	131.5	5.7	1,300	4.38	1.85	3
LDC 29-2		125.9	129.4	3.5	500	7.00	1.85	3
LDC 30		145.5	156.4	10.9	2,000	5.45	1.38	8
LDC 31		158.8	158.9	0.1	200	0.50	1.38	-
LDC 32		162.0	164.5	2.5	500	5.00	0.92	2
LDC 33		171.8	172.4	0.6	300	2.00	1.85	-
LDC 34		174.6	188.6	14.0	1,600	8.75	1.85	11
LDC 35		175.6	190.5	14.9	3,100	4.81	0.51	13
LDC 36		187.3	195.0	7.7	700	11.00	1.85	6
LDC 27	3	110.9	113.5	2.6	600	4.33	1.38	2
LDC 27	4	113.5	119.8	6.3	1,200	5.25	1.85	4
LDC 27	5	119.8	127.0	7.2	800	9.00	1.85	6
Total								309
Grand Total								335