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FEASIBILITY STUDY
ON
THE ASUE RIVER BASIN
AGRICULTURAL DEVELOPMENT PROJECT

VOLUME 3
APPENDICES VII-XV

AUGUST 1985

JAPAN INTERNATIONAL COOPERATION AGENCY

FEASIBILITY STUDY

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THE ASUE RIVER BASIN

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LIST OF REPORTS

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APPENDIX VII

IRRIGATION AND DRAINAGE

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IRRIGATION AND DRAINAGE

1. PRESENT STATUS

1.1 GENERAL

The proposed service area consists of two distinct alluvial plains along two separate rivers. One area extends along the alluvial plain of the north-south flowing Asue River and is referred to as the Asue Area in this report. The other service area is located east of the Asue Area and extends along the Hasohoy and Tabagay rivers which flow south to north. This area is referred to as the Eastern Area.

The Asue Area is bounded on the south by Ajuy Bay, on the east by the hill area which is the boundary of the Eastern Area and on the north and west by the mountain range which divides the catchment areas of the Panay and Asue rivers. The eastern area is bounded on the north by Bagacay Bay, while on the other sides it is surrounded by mountains.

The average slope of the existing paddy fields is approximately 1/300, and the elevation of the service area varies from 2m to 40m. The lower portion of the service area adjoins fish ponds in the coastal area. The average size of farm plot varies from 0.3-0.5ha. The western portion of the Asue area is undulating and sugarcane is planted in the elevated area.

There are four main rivers in the Project area; namely, the Asue River, Gubaton River, Pasaka Creek and Tabagay River. These rivers are not only the main sources of irrigation water supply for existing paddy but also the main drainage channels in the Project area. Along these rivers, concrete weirs and pumps are installed by farmers in order to tap river flow for irrigation. However, the present irrigation area is quite limited and the majority of existing paddy is rainfed.

Small canal networks are provided in the paddy area, although their canal layout is not systematically arranged. Moreover, the location of these small canals is sometimes partially rearranged by the farmers. Most of these canal networks are connected with existing intake structures, such as weirs and pumps. However, farmers have set-up some parts of these canals in the middle of a paddy area in an attempt to obtain the return flow from paddy located in the upstream.

As for the drainage system, very few drainage canals can presently be seen in paddy area. The canal networks mentioned above function as drainage channels. The main drainage system at present is plot to plot as is the irrigation system, and surplus water is drained into the nearest creek or river. The Serruco Communal Irrigation System is the only organized irrigation system in the Project area. In general, farmers are always threatened with shortage of irrigation water due to lack of systematic irrigation and drainage facilities.

1.2 Irrigation Conditions

A portion of existing paddy in the Project area is irrigated by several irrigation methods. The existing paddy field can be categorized into the following five types:

- Serruco Communal Irrigation System
- Water impounding component under the KABSAKA Project
- Pump irrigation
- Private diversion dam irrigation
- Rainfed

In general, paddy fields in the upstream area are irrigated by private diversion weirs, while those in the lower area are normally irrigated by pumps.

(1) Communal Irrigation System

Serruco Communal Irrigation System (CIS), the only existing communal irrigation system in the Project area, was constructed in December 1980, and presently serves about 300ha to 400ha of paddy field area through the Serruco diversion dam. Although the original service area was planned at 700ha, only 300ha have adequate irrigation facilities, such as lateral canals, turnouts, etc. For the remaining area, main concrete structures, such as turnouts and siphons have been constructed, but construction of the main canal and lateral canal has been deferred due to incompletion of land compensation for the canal routes.

The number of irrigation associations established and operated by farm households is 106 at present. The organization consists of a president, vice-president, secretary and treasurer appointed by a 6-member Board of Directors for a period of 2 years.

Irrigation association members residing near canal diversion facilities control the gates taking into consideration crop conditions in paddy fields. The same management method applies to dam facilities. Fees to cover construction costs are collected by association members and are equivalent to ₱150/ha/crop while an operation fee of ₱50/ha/crop is collected seperately by the same. Water management costs include the following:

- Dam Keeper ₱100/month
- Gate Keeper ₱250/month

An outline of the Serruco CIS is presented in TABLE VII-1.

(2) Water Impounding Component of KABSAKA Project

The water impounding component of the KABSAKA Project has been implemented under the supervision of the Bureau of Soils, Ministry of Agriculture and Food.

In 1982, six out of the 40 water impounding component projects were constructed in the eastern part of Panay Island. Another 10 impounding dams were constructed in 1983. Out of the total 16 constructed water impounding projects, 5 dams are located within or near the Project area, while 3 more water impounding projects are under planning in the Project area.

The size and dimension of each water impounding project on Panay Island is listed in TABLE VII-2, and the water impounding projects related to this Project are listed in TABLE VII-3. Besides these KABSAKA Water Impounding Projects, there are private water impoundings within or near the Project area. Their sizes are listed in TABLE VII-4 and the locations are shown in FIG. VII-1.

(3) Private Irrigation Facilities

There may be a few hundred privately owned small pumps in the service area. Pump irrigated areas can be categorized into 2 types; fixed and portable. The fixed pump type is installed in a pump house and normally has a concrete lined canal. The portable type of pump is carried by farmers to available water sources such as rivers or creeks. These pumps are operated intermittently with 2 or 3 day intervals and the irrigation area of one pump is approximately 2ha.

There are several privately owned fixed type concrete weirs along the Asue River and related creeks. According to field investigation, 12 concrete weirs are located in the service area; 7 on the Asue River, 2 on Sara Creek, 2 on the Pasaka River and one on the Lanjagan River. Most of the weirs are stoplog type about 2-3m in height and 4 weirs out of 12 have been washed out by floods. The size and type of each weir are listed in TABLE VII-5 and the location is shown in FIG. VII-1.

Fixed pumps and private intake weirs were installed by large land owners and, although some of these facilities were constructed in 1938, the majority were established in the latter half of the 1950s. The scale of weir and pump capacity are dependent on the amount of land area belonging to the owner and consequently directly irrigated area ranges from about 50 to 150ha.

As a result of land reform, privately owned land as well as cultivated area per farmer is being reduced. Private irrigation systems are maintained, however, and priority is given to the owner in the water distribution system.

(4) Acreage of Present Irrigation Area

In the present irrigated area mentioned above, there are only two kinds of irrigation systems with reliable and durable facilities namely the Serruco CIS and KABSAKA Water Impounding Projects. These irrigation systems were constructed recently and are not yet fully operative. Construction of both systems is continuing and completion of the same is envisioned in the near future. Irrigation by private weirs, pumps and water impoundings, on the other hand, is unreliable and maintenance of facilities is largely dependent on the farmers and their willingness to irrigate.

Irrigated area and facilities were investigated in field surveys during the rainy season from June to August in 1984. Present irrigated area under the Serruco CIS and KABSAKA WIP were confirmed by the related authorities. However, the acreage of private facilities was diffcult to verify due to the plot to plot irrigation system. Irrigated areas according to type of irrigation system are presented in FIG. VII-1. Total irrigated area in the wet season as of 1984 is approximately 1,700ha although some of them are located outside of the Project area as shown in the following table.

PRESENT IRRIGATED AREA

Type of Irrigation	Acreage (ha)	Remarks
Serruco CIS	400	Project Area
KABSAKA Water Impounding Projects	232	
Castol WIP	32	Project Area
Moto WIP	50	Project Area
Bondolan WIP	50	Project Area
Aglosong WIP	50	Outside of Project Area
Belen WIP	50	Outside of Project Area
Private Water Impounding Projects	28	
Sanson	7	Outside of Project Area
Pacig	12	Partially in the Project Area
Salcedo	9	Partially in the Project Area
Private Weir	460	Project Area
Private Pump	580	Project Area
Total	1,700	

1.3 Drainage Conditions

1.3.1 General

In general, no severe drainage problems were observed in the service area although some problems occur in the Eastern Area, along the Hasohoy River, where the soil is heavy clay. According to local inhabitants of the Eastern Area, water stagnates for 1-3 days with a maximum depth of about 0.6m during heavy rain. There are 2 possible reasons for the drainage problem; heavy clay soil which has a very low percolation rate, and no drainage canals in the flat areas. In some areas near Barangay Pasi, the clay is extracted for ceramics manufacturing. Normally, clay soil is about 4m in depth where soil excavation for ceramic material is carried out. The soil is black.

In the upstream area of the Asue River basin, paddy fields are sometimes affected by flood. These floods are mainly caused by backwater from privately owned concrete weirs because the farmers are not able to remove stop-logs from the same during floods. The flood problem could be easily solved by removing the existing weirs.

Existing rivers and creeks are used for drainage purposes as well as for irrigation. At present, there are a series of weirs along the Asue River and related creeks. This means that the water has been repeatedly used for irrigation. The limited river water is efficiently utilized, though backwater from existing weirs causes flooding upstream of the weir during heavy rains.

1.3.2 Flood Mark Investigation

Typhoon "Undang" hit Panay Island on November 5 1984 during the course of the second field investigation. As mentioned in APPENDIX II METEOROLOGY AND HYDROLOGY, existing rain gauges could not measure total rainfall due to the strong wind. However, it is assumed that total rainfall caused by the typhoon was at least 200 to 300mm and the duration of rainfall was about 12 hours. In spite of the short duration of rainfall, almost all of the service area was abnormally inundated. The flood probability return period was estimated at about 25 to 50 years or more. In order to determine flooding conditions, the following investigations were conducted through interviews of the local people.

On a square kilometer mesh, the service area was divided into 98 meshes. The flood events were investigated in at least 3 points per mesh by interviewing the local people and about 300 points were thereby investigated for the following items.

- Location of flood at interview site
- Time flood commenced
- Time of maximum flooding
- Time flood ended
- Flooding conditions in the paddy field stated as follows:

  o Normal Water Depth
  o Farm dike could be seen
  o Spilled over the farm dike
  o Farm dike, paddy field were all submerged

- Flow direction
- Maximum depth of water
- Maximum height of water level in the river or creek

The location of the interview sites are shown in FIG. VII-2 with the flood flow direction. Maximum flood conditions in the paddy field categorized into 4 grades are shown in FIG. VII-3 and maximum flood water level near the river is shown in FIG. VII-4. Duration and area of water stagnation are illustrated in FIG. VII-5.

According to flood conditions in the paddy (see FIG. VII-3) the area near Ajuy, along the Lanjagan and Serruco rivers was not submerged, but most areas were totally submerged during maximum flood conditions. Peak flood occured at about 2:00 p.m. in the upstream area of the Asue River and at 3:00 to 5:00 p.m. in the downstream area. In addition, inundated area 24 hours after the typhoon was investigated to determine the area where water stagnated for more than 24-hours. FIG. VII-6 shows the above-mentioned area and the acreage was estimated at about 1,500ha.

According to the results of flood investigation, the following conclusions can be made.

a) Approximately 5,000ha of paddy was submerged during peak flood conditions (see FIG. VII-3).

b) The maximum flooding depth was observed at 1.8m along the Tabagay River (see FIG. VII-3).

c) The severest inundation in the Project area occurred along the Tabagay River in Barangay Capinang and near

San Dionisio Town where about 130ha was submerged for more than 3 days. Areas along the Tabagay River, upstream of Padios Creek in Barangay Lanciola and the middle part of the Asue River in Barangy Padios and Salcedo were submerged for more than 2 days. About 2000ha in the Eastern Area and along the Asue River was submerged for more than 24 hours. However, water in most paddy stagnated about 20 hours or less (FIG. VII-5).

d) Field investigation results concerning area submerged more than 24 hours, (FIG VII-6) correlated very closely to the area indicated by the interview survey (FIG. VII-5) showing that the results of interviews are reliable.

e) Consequently, it was determined that the Project area does not have a severe drainage problem, as flood water in the service area did not stagnate long enough to damage paddy. The duration of water stagnation in most of the service area was less than 24 hours in spite of the very rare flood frequency of the typhoon. The area submerged more than 2 days, resulting in heavy damage, amounted to only about 400ha. In such areas, drainage improvement will be necessary.

2. PROPOSED IRRIGATION PLAN

2.1 Formulation of Irrigation Plan

2.1.1 Basic Irrigation Plan Concept

A 24-hour supply gravity irrigation system is proposed for the irrigation development plan of paddy in the Project area. The following concepts for specific conditions are also considered in the irrigation plan.

a) In order to reduce the required reservoir capacity in the Catipayan River, river flow and return flow in the irrigation area should be utilized as much as possible by constructing new diversion dams and utilizing the existing Serruco diversion dam.

b) The proposed diversion dams or main canals should be interlinked with the Asue main canal to supplement irrigation water deficit from the Catipayan reservoir. Moreover the proposed main canals should also be interlinked with each other to supply excess water from one canal to another when available.

c) Since potential irrigable area is rather limited due to topographical conditions the main canal should be arranged to cover as much irrigable area as possible.

d) Since proposed diversion dam sites are located in the flat area, a fixed type diversion dam is not preferable. Accordingly, an automatic collapsable type, such as a rubber dam, has been proposed for the diversion dam to avoid a backwater effect in the upstream area during flood.

e) The slope of the irrigation service area is approximately 0.3% (1/300). Considering this slope, the alignment of on-farm facilities should be carefully studied.

2.1.2 Basic Policy for the Irrigation Plan

In view of the above conditions, the following countermeasures were considered in irrigation planning.

(1) Diversion Dams

The following three diversion dams are proposed to utilize river flow in the irrigation area.

a) The Asue diversion dam is located in the upstream of the Asue River to intake diversion water from the Catipayan reservoir and also water from the Asue catchment area.

b) The Gubaton diversion dam is located in the upstream Gubaton (or Lanjagan) River and will supplement irrigation water.

c) The Bakabak diversion dam is located in the downstream Asue River to intake water not only from its own catchment area but also the return flow from the upper irrigated area.

In addition to the three newly proposed diversion dams mentioned above, the existing Serruco diversion dam has also been used by interlinking the Serruco main canal and the proposed Asue main canal. All three diversion dams are proposed as automatic collapsible types, such as rubber dams, to avoid backwater effect in the upper area during flooding.

(2) Interlinkage of the Main Canal

The Asue main canal leading from the Asue diversion dam will be extended to the Serruco and Gubaton irrigation area so as to supplement the irrigation water to these areas when available river flow is insufficient. The Asue main canal will be interlinked to the Serruco main canal and the Gubaton main canal.

Diversion canals or supplemental canals are proposed for the Serruco right main canal and the Gubaton main canal to utilize local river flow. The diversion canals will be interlinked with the Asue main canal. Therefore, when the irrigation water in the Serruco and Gubaton area is insufficient, supplemental irrigation water can be supplied from the Catipayan reservoir through the Asue main canal. Moreover, when excess water is expected from the Serruco and Gubaton rivers, water will be supplied to the portion of the area under the Asue main canal.

By interlinking these main canals, mutual support systems among water supply networks will be established among the available water resources.

(3) Enriched Benefit Area

As for the existing Serruco Communal Irrigation System, the elevation of the service area is about EL.32 to EL.20m. The elevation of the Asue diversion dam in the Asue River will be about EL.35m, and consequently the Serruco CIS, at present irrigating about 400ha, cannot be supplied with irrigation water from the Catipayan River. However, the 300ha lower portion of the Serruco CIS with incomplete irrigation facilities can be covered by the Asue main canal. Although the upper 400ha of the Serruco CIS cannot be irrigated by the Catipayan River, the same can be included as an indirect benefit area of the Project by utilizing Serruco River flow more intensively than in the original plan. Also, the same on-farm development and extension services as direct irrigation area can be performed in the upper 400ha. The area is therefore included as enriched benefit area.

The same conditions have been considered for the KABSAKA Water Impounding Projects. The related projects are Castor WIP, Moto WIP and Bondolan WIP. In accordance with the layout of the main canals, present irrigation area under these water impoundings was divided into direct irrigation area and enriched benefit area.

(4) On-farm Development

The slope of the service area is rather steep at approximately 0.3% (1/300). Taking into consideration the steep slope, the supplementary farm ditch was aligned along the contour line to reduce the number of drop structures. Furthermore, taking advantage of the steep slope, the supplementary farm ditch is designed to function as both an irrigation and a drainage channel.

2.1.3 Irrigation Plan Alternatives

(1) Asue Diversion Dam

In order to intake diverted water from the Catipayan reservoir, the Asue diversion dam is proposed in Barangay Aguirre in the upstream Asue River. The Asue diversion dam can also intake water from the Asue catchment area. Although generally a diversion dam at a far upstream location can cover a much wider irrigation area because of the higher elevation, the Asue diversion dam will be situated further downstream to increase the catchment area despite the decrease in elevation.

FEATURES OF ALTERNATIVE SITES FOR ASUE DIVERSION DAM

Location / Items	A - 1	A - 2	A - 3
Catchment Area	2.0km^2	13.7km^2	16.8km^2
Water Level at Intake	EL. 36	EL. 33	EL. 29

Site A-1 is too high for the main canal, and the left main canal route should be aligned near the foot of the mountain in the first 5km. Also both sides of the main canals in the first 5km command only about 100ha of irrigation area, and consequently construction cost will be rather high compared with location A-2 and A-3. Furthermore, the expected catchment area at site A-1 is so small that a larger Catipayan reservoir capacity is required than for site A-2 and A-3.

However, due to the high elevation of site A-1, the main canal route to the Asue Area (A-1-1) as shown in FIG. VII-7 can be shortened by introducing a tunnel about 600m in length, while the open canal route (A-1-2 as shown in FIG. VII-7) is about 8km. With a discharge capacity of $5m^3/s$, the construction cost of the tunnel is 41,500₱/m and that of the open unlined canal is about 1,600₱/m. The cost of the tunnel is about 26 times greater than that of the open canal, or equivalent to 15.6km of the open canal. The tunnel route plan therefore is too expensive.

The elevation of site A-2 is sufficient to command the eastern hilly area near Barangay Castol. However, existing paddy field from the outlet of the proposed diversion tunnel to site A-2 (about 100ha) can not be irrigated by site A-2. Therefore an additional canal the same size as the main farm ditch, should be provided to supply irrigation water directly from the Catipayan reservoir. Site A-2 has a $13.7km^2$ catchment area which is expected to reduce the reservoir capacity at the Catipayan dam site.

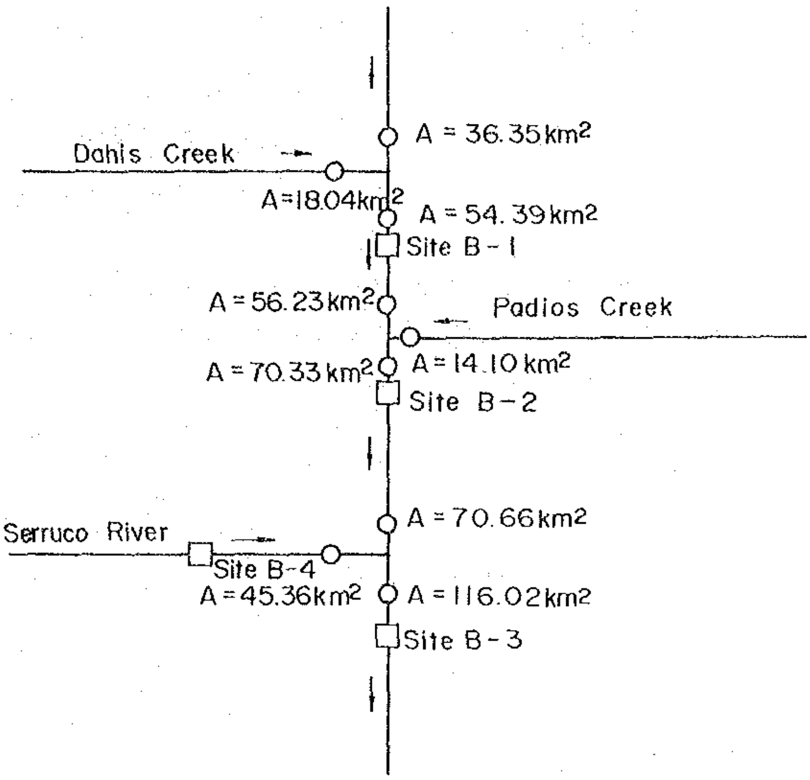
As for site A-3, the route of the main canal is quite flat but the elevation is too low to cover the eastern hilly area. The command area for irrigation is thus excessively reduced. Accordingly, site A-2 was selected for the Asue diversion dam site.

(2) Gubaton Diversion Dam

In order to cover a wider area, the Gubaton diversion dam site was selected to cover the upstream portion of the existing paddy area. Also, the diversion site was located at elevation 17.5m to supply water for the Asue Area.

(3) Bakabak Diversion Dam

In order to utilize the return flow from the upper irrigation area, a diversion dam in the downstream Asue River is proposed. Three alternatives sites have been studied as shown in FIG. VII-7, and the schematic diagram is on the following page.



Note : A is the Catchment Area

The diversion dam can utilize not only return flow from the upstream area but also flow from its own catchment area. Site B-1 is located in the Asue River just downstream of the Dahis Creek confluence with an intake water level of EL.10.3m. River flow from the Serruco River and Padios Creek can not be utilized for the irrigation area; the topographically irrigable area is estimated at 1500ha. Site B-2 is located just downstream of the Padios Creek confluence and intake of Serruco river flow is not possible at the site.

Site B-3 has the largest catchment area among the 3 sites. However, the irrigable area is only 1000ha because of low elevation. The comparison of these three alternative sites is shown in the table below together with a combination site of B-1 and B-4.

FEATURES OF ALTERNATIVE SITES FOR THE BAKABAK DIVERSION DAM

Alternatives / Items	Site B-1	Site B-2	Site B-3	Site B-1 & B-4
Location (Downstream Confluence)	Dahis Creek	Padios Creek	Serruco River	2-Diversion Dams at B-1 & B-4
Catchment Area (km^2)	54.39	70.33	116.02	94.39
Intake Water Level	EL.10.3	EL.7.6	EL.7.5	EL.10.3
Topographically Irrigable Area (ha)	1,470	1,040	1,000	1,470
Ratio of Catchment Area to Command Area	3.6	6.7	11.6	6.4
Irrigable Area by the Catchment Area 1/	540ha	700ha	1,160ha	940ha
Water Deficit Area 2/	960ha	350ha	0	560ha
Main Canal Length	12.0km	7.5km	5.8km	12.5km
Right Main Canal	6.5km	5.0km	4.4km	6.5km
Left Main Canal	6.0km	2.5km	1.4km	6.0km

1/ The irrigatable area is estimated at 10% of the catchment area, on the basis of water resources operation study results.

2/ Water deficit area should be replenished from the Catipayan reservoir.

The catchment areas for sites B-1, B-2 and B-3 are $54.39km^2$, $70.33km^2$ and $116.02km^2$, respectively. The ratios of the catchment areas at site B-1 and B-2 to site B-3 are 0.469 and 0.606, respectively. The main reason for these small ratios is exclusion of the Serruco watershed at sites B-1 and B-2. Exclusion of river flow and return flow mainly of the Serruco River at sites B-1 and B-2 thus presents a disadvantage. According to the water balance study, a catchment area of about 10 times the irrigable area is required in cases without a reservoir. The command area cannot be irrigated by river flow alone at sites B-1 and B-2 return flow and supplementary supply from the Catipayan reservoir is required. Site B-3 on the other hand has a catchment area more than 10 times the irrigation area, and the said site thus has sufficient discharge to irrigate the area without supplement from the reservoir.

Another alternative plan for utilization of the Serruco river flow is considered for site B-1. In this plan, the right main canal crosses the Serruco River at site B-4 as shown in FIG. VII-7. A combination of B-1 and B-4 allows a catchment area of 94.39km^2 with 54.39km at site B-1 and 40km^2 at site B-4. However, one more diversion dam at site B-4 is required. In addition to this, the left main canal from site B-1 should be aligned at the foot of the mountain area for about 4km. The route is too narrow to locate the main canal between Padios Creek and the mountain . Thus river flow from Padios creek can not be utilized. Consequently the combinaiton of B-1 and B-4 would be more costly than site B-3 because of the additional diversion dam required and the difficulties involved in the left main canal route. Therefore, site B-3 is, proposed as the Bakabak diversion dam site.

2.2 Design Water Requirement

2.2.1 Measurement of Percolation Rate

Percolation rate in paddy, which is a basic factor for determining irrigation water requirement, was measured in the Project area from October to November 1984. The measuring sites were selected on the basis of 2km mesh with 400ha per mesh, covering the service area. Accordingly, measurement of percolation rate was conducted at 20 sites with Rapid Percolation Measuring Equipment. The Team also used N-type equipment. Data measured with the latter, however, seems unreliable due to the influence of frequent rainfall.

The observed maximum, minimum, and average percolation rates are 1.9, 0.1, and 0.88, respectively. The obtained values were found to be almost the same with values obtained through surveys conducted by NIA from April to May 1982 (dry season). The results of the measurements conducted by the Team and NIA are shown in FIG. VII-8 and tabulated in TABLE VII-6.

Each measuring point was categorized into three types by soil type in the irrigation area and an arithmetic mean value was obtained in accordance with the soil type as shown in the table below.

PERCOLATION RATE

Soil Type	Area (ha)	Average Percolation Rate (mm/day)
Clay	1,884	0.48
Sandy Clay Loam	5,180	0.81
Sandy Loam	1,873	1.47
Total/Average	8,937	0.90 1/

1/ weighted average by area of soil type

Average value of percolation rate considering soil type is estimated at about 1.0mm per day. However, the percolation rate in the Project area is expected to increase due to drainage improvement works at the on-farm level after project implementation. Therefore, the percolation rate for the entire Project area as a basic factor to be used for determination of irrigation water demand was determined at 1.5mm/day.

2.2.2 Design Water Requirement Analysis

(1) Consumptive Use

1) Consumptive use

Observed daily pan evaporation was available at Barangay Aguirre from 1979 to 1984. The consumptive use for paddy was taken as 80% of the pan evaporation.

In order to find the maximum consumptive use for various durations, the analysis of maximum evaporations for 1 to 60-day durations was made with the observed data. The annual maximum for each year for various durations is shown in TABLE VII-7 and the extremes are plotted in FIG. VII-9. Due to small variations in evaporation, the extremes for each duration were adopted to compute the consumptive use of paddy. As seen from TABLE VII-7 the maximum consumptive use is 8.4mm/day for a 1-day duration decreasing to 5.8mm/day for a 60-day duration.

The supply capacities of the quarternary and tertiary canals are affected by the maximum daily values due to the smaller command areas. On the other hand, the supply capacities of secondary and main canals are affected by maximum average consumptive use during the presaturation period due to the larger command areas of the canal. Accordingly, the capacities for the quarternary and tertiary canals should be decided on the basis of maximum daily consumptive use, the secondary capacity should be decided on the basis of the maximum 10-day average value and the main canal capacities should be based on the maximum average consumptive use during the 40-day presaturation period as shown in FIG. VII-9.

2) Presaturation requirements

Presaturation requirements consist of the following items.

a) Land Soaking Capacity

	Wet Season	Dry Season
Root Depth	250mm	250mm
Dryness	70%	60%
Porosity	50%	50%
Land Soaking	87.5mm	75.0mm

b) Standing Water

With the present plot-to-plot irrigation system, deep standing water requires a prolonged presaturation period. Standing water should therefore be reduced as much as possible. The depth of standing water was decided at 20mm.

c) Consumptive Use

As mentioned above, the maximum average consumptive use for a duration corresponding to the presaturation period was adopted.

d) Percolation Rate

Percolation rates were adopted on the basis of field measurement data considering soil classification in the paddy area. A weighted average of the percolation rates was used for the irrigation schemes covering either of these areas and 1.5mm/day was decided as the percolation rate in the Project area.

e) Areal Factor

During the presaturation period, the irrigated acreage increases at a uniform rate. Taking into consideration the incremental acreage, the peak water requirement occurs on the last day of the presaturation period.

Daily water requirement during presaturation can be computed by the following procedure considering the incremental acreage, as shown in TABLE VII-8.

The computation was made according to the following parameters:

Presaturation Period	40 days	
Presaturation Requirement	Wet Season	Dry Season
Land Soaking	87.5mm	75.0mm
Standing Water	20.0mm	20.0mm
Total	107.5mm	95.0mm

Maintenance Water

Consumptive Use	6.1mm/day
Percolation Rate	1.5mm/day
Total	7.6mm/day

3) Effective rainfall

As for effective rainfall during crop growth, daily rainfall of 5.0mm or less is considered ineffective as it will be intercepted by paddy leaves. For daily values exceeding 5.0mm, 100% is considered effective. Normally the daily consumptive use and percolation rate are about 8.0 to 10.0mm/day. When effective rainfall exceeds the daily water requirement, the surplus is stored in the paddy field and used subsequently. Effective rainfall however, is limited to 60mm/day due to over spill.

4) Growing stage requirements

Water requirement during crop growth has been computed as the sum of the daily consumptive use and the percolation rate.

(2) Irrigation Efficiencies

1) General

Irrigation efficiency is the ratio between the quantity of irrigation water effectively used by the crops and the total quantity supplied. It is therefore an important aspect to be considered when planning, designing and operating an irrigation system. Irrigation efficiencies of the Project are generally comprised of three main components: conveyance, operation and farm efficiencies. The sources of conveyance, operation and field losses in the Study area are briefly described below.

a) Conveyance Loss

Conveyance losses consist of seepage through the canal bed and evaporation.

b) Operation Loss

Operation losses are caused by insufficient control of water distribution from intakes, especially discharge from double orifice gates, which are easily affected by water level fluctuation upstream and downstream of the gates. The long time-lag for water to flow from an intake to the field makes it difficult to efficiently utilize rain falling directly onto the paddy.

Operation losses can be reduced by careful operation of gates and by introducing a comprehensive field-monitoring system.

c) Farm Loss

The S.F.D. of irrigation water supplied to a paddy field takes into account the consumptive use of paddy and the percolation rate. The excess irrigation water which runs off a paddy field into a drainage channel and is not re-used, is farm loss.

Farm loss may also be due to additional water required to maintain higher standing water in an undulating rotational unit.

2) Estimation of conveyance losses

Conveyance losses from the canals were estimated as follows:

a) Seepage Loss

In order to estimate the canal seepage loss, the Moritz formula developed by USBR was adopted as follows:

$$S = 0.0116072 \times C \times \sqrt{\frac{Q}{V}}$$

Where
S = Seepage losses (m^3/km)
Q = Discharge in m^3/s
V = Velocity in m/sec.
C = Constant value depending on soil type,
(For clay and clayey loam soil, C = 0.41)
(For sandy loam or sandy clay loam soil, C = 0.60)

As $\frac{Q}{V} = A$ and $C = 0.60$

the above equation becomes

$$S = 0.00696 \times \sqrt{A}$$

The area A can be estimated as follows:

$$A = (B - 1.5\ H) \times H$$

b) Estimated conveyance loss

In order to estimate the conveyance loss by the Moritz formula in each type of canal from the main, secondary and tertiary canals, the longest route of the canal network system was taken into consideration and the Asue main canal, Lateral A-L2 canal and tertiary canal were selected. As a temporary value, 2.0ℓ/s/ha of water duty was adopted for the estimated discharge capacity of the canal.

Name of Canal	Main Canal Asue Main Canal	Secondary Canal A-12	Tertiary Canal -
Length (km)	13.09	7.14	1.2
Service Area (ha)	2,250	580	52
Estimated Discharge (m^3/s) (at 2.0 ℓ/s/ha) (2)x2.0/1000	5.100 1/	1.160	0.104
Average Discharge (m^3/s)	3.130	0.632	0.052
Average Velocity (m/s)	0.8	0.7	0.4
Average Area (m^2) (4)/(5)	3.912	0.903	0.130
Seepage Loss (m^3/s) $(7)=(1) \times 0.00696\ \sqrt{(6)})$	0.180	0.0472	0.0030
Loss percentage (%) (8) = (7)/(3) x 100	3.5	4.1	2.9

1/ Discharge includes $0.6m^3/sec$ of replenishment water to the Gubaton area.

As a result, a conveyance loss for the main, secondary and tertiary canals was determined at 3.0%, 4.0% and 3.0%, respectively with an overall conveyance efficiency of 90%.

3) Efficiencies in each type of canal

As for operation loss, overall operation efficiency can be assumed at 90% and this value is also applied to the main, secondary and tertiary canals considering the number of gate structures along the canal as shown in the table below. Field efficiency was adopted at 70% during wet season considering the effect of rainfall and 75% during dry season.

Accordingly, each irrigation efficiency factor in accordance with the type of canal was decided as shown in the below. The period from December to May is designated as the dry season and the rest as the wet season.

OVERALL IRRIGATION EFFICIENCIES

	Conveyance	Operation	Field	Total	Overall
Main	0.97	0.98	-	0.95	0.57 (0.61)
Secondary	0.96	0.97	-	0.93	0.60 (0.64)
Tertiary	0.97	0.95	-	0.92	0.64 (0.69)
SFD & On-farm	-	-	0.70 (0.75)	0.70 (0.75)	0.70 (0.75)
Total	0.90	0.90	0.70 (0.75)	0.57 (0.61)	

Note: The figures in brackets refer to the dry season.

(3) Design Water Requirements

Net water requirements for the on-farm level together with those for the quarternary and tertiary canals are based on peak daily consumptive use. On the other hand secondary and main canals are based on peak 10-day and 40-day average consumptive use, respectively.

Considering the estimated irrigation efficiencies for each irrigation system, the design unit water requirements were decided as shown in TABLE VII-9, and the results are as shown below.

Canal	Unit Design Water Requirement
Main	2.051ℓ/s/ha
Secondary	2.122ℓ/s/ha
Tertiary & SFD	2.242ℓ/s/ha

2.3 Potential Irrigable Area

2.3.1 Irrigation Area

In accordance with the basic concepts and the selected proposed diversion dam sites, the layout of the main canals from the diversion dams were studied to maximize the command area. At the same time interlinking of main canals was considered to enable exchange of surplus river flow and supplementary flow from the Catipayan reservoir.

The irrigation service area can be divided into two types; direct irrigation area and indirect benefit area or the enriched benefit area of the Serruco CIS and KABSAKA WIP. The direct irrigation area can be divided into three sections in accordance with the command areas of the proposed diversion dams, the Asue, Gubaton and Bakabak sections. The Asue section, however, is further divided into the Asue Area which is covered by the right main canal and extends along the Asue River, and the Eastern Area which is covered by the left main canal and mainly extends along the Hasohoy and Tabagay Rivers.

(1) Enriched Benefit Area

1) Serruco CIS

Due to the high elevation required for the Asue main canal, water from the Catipayan River cannot be used to supplement irrigation water supply to the existing Serruco CIS. As a result, the upper 360ha area of the Serruco CIS is planned as an enriched benefit area. A maintenance road along both existing main canals is proposed along with on-farm development.

The existing right main canal has sufficient capacity to convey irrigation water up to 1.024m^3/sec. Therefore, extension of the main canal to interlink the same with the Asue main canal is proposed to convey surplus water to the Asue Area.

2) KABSAKA WIP

In the Eastern Area, three existing KABSAK WIP are partially covered by the Eastern main and lateral canals. The original irrigation areas of Castor, Moto and Bondolan WIP are 32ha, 50ha and 50ha, respectively. With the proposed canal route, 5ha in the Castor WIP, 10ha in the Moto WIP and 25ha in the Bondolan WIP can not be covered by the Eastern main canal. This 40ha area is considered as enriched benefit area.

(2) Direct Irrigation Area

1) Bakabak Area

As explained in section 2.1.3, the proposed Bakabak diversion dam site has sufficient catchment area to supply necessary irrigation water for 1,000ha of the command area even with annual triple cropping. Therefore, no supplementation from the Catipayan dam is required although the return flow from the upper irrigated area is partially used for irrigation.

2) Gubaton Area

Based on the proposed Gubaton diversion dam site and the layout of the main canal route, the Gubaton irrigation area was designated at 520ha. During a normal flood however, surplus water in the Gubaton River can be brought to the lower portion of the Asue Area. Linkup of the diversion canal from the Gubaton main canal to the lateral canal in the Asue Area is proposed. (This diversion canal was designated as diversion canal (2)).

On the other hand, river flow is insufficient for irrigation of the entire irrigation area during droughts. Therefore, supplemental irrigation water supply from the Catipayan reservoir is proposed. The Asue main canal, would be extended to the Gubaton main canal for this purpose and the connecting canal which will link the Serruco right main canal is termed diversion canal (1).

3) Asue Area

The layout of the Asue main canal was formulated to maximize the command area and an irrigation area of 2,250ha commanded by the Asue main canal is proposed. As mentioned above, interlinking canals from the Serruco right main canal and Gubaton main canal are proposed to convey surplus water to the southern portion of the Asue Area. From the topographical point of view, 491ha in the latter can be replenished from the Serruco and Gubaton rivers when surplus water is available.

4) <u>Eastern Area</u>

The southern part of the Eastern Area, (i.e., the southern part of the San Dionisio - Santol Road) near Barangay Agrosong, has elevations of about 20.0 to 40.0m and an area of about 500ha. The area slopes from south to north with the lowest portion in the center where a river drains the area. Lift irrigation facilities will therefore be required to irrigate the same.

Alignment of canal slope with topography to permit irrigation from the Eastern Main Canal will be difficult. Accordingly, installation of a pump will be required in the southern part of the area near Barangay Agnaga. This method will require about 3km of pipe line for lifting with the highest elevation of the pipe line at 60m. Considering the power requirement for the pump and construction cost of the pipe line, irrigation of the entire 500ha area would not be economical. Consequently, the southern part of the Eastern Area has been omitted from the service area.

In the northern part from San Dionisio to Santol, the elevation ranges from about 16-18m. Topographical conditions in the area are quite complex. The service area slopes from the higher western hilly area to the lower Barangay Santol with EL.±14m and from there ascends eastwards toward San Dionisio at elevation ±18.0m. The area is referred to as the San Dionisio irrigation area, and can be irrigated by a long elevated flume, by use of a pump or by construction of a long siphon structure.

During Typhoon Undang, most of the San Dionisio area was flooded. Water spilled over the road from Santol to San Dionisio and flowed toward the Hasohoy River, as the volume exceeded the capacity of the existing creek, the outlet of which is located in the southern part of San Dionisio Town. Generally, for a smaller flood, the existing creek has sufficient capacity to convey flood waters into the San Dionisio estuary. However, when the flood is too high, flood waters flow into the Hasohoy River, increasing flood damage in the Hasohoy area.

In order to protect the Hasohoy area from flood damage, a new drainage canal is planned along the Santol - San Dionisio road to connect with the existing creek, the capacity of which should be enlarged by excavation. If the purpose of San Dionisio area development is only drainage improvement works, land acquisition for the drainage canal will be rather difficult and paddy production will not greatly increase. Therefore, irrigation water supply for the area is included in the improvement works. A part of lateral canal E-L2 in the San Dionisio area can be changed into a siphon about 1km long thereby including 250ha of paddy in the service area.

5) Area Along the Trans-diversion Canal

Construction of a trans-diversion canal is planned from the proposed Catipayan dam to the Asue River. There are about 100ha of existing paddy field along the proposed canal route which can be included in the service area.

In order to utilize the rivers of the Asue River catchment area as much as possible, the proposed diversion dam is located about 4km downstream from the tunnel outlet. At the proposed Asue diversion dam site, a catchment area of $13.6km^2$ is expected. Due to the location of the Asue diversion dam, irrigation of the existing 90ha irrigation area between the tunnel outlet and the diversion dam by means of the diversion dam is not possible. In order to supply irrigation water to the area, construction of a small lateral irrigation canal is proposed from the outlet of the tunnel to the diversion dam.

2.3.2 Proposed Irrigation Area and Networks

In consideration of main canal layouts and present land use conditions, the gross irrigation area was determined from the topographical map. Right of way for irrigation and drainage canals as well as proposed facilities, were also estimated on the basis of canal layout. The relation of gross area and net irrigation area is shown in TABLE VII-10.

Along with study of the three proposed diversion dams, main and lateral canal alignment was studied to maximize the irrigation area. The layout of irrigation canals is shown in FIG. VII-10, while a schematic diagram of irrigation blocks is presented in FIG. VII-11.

The proposed irrigation service area by diversion dam system and main canal are summarized in the following table.

PROPOSED IRRIGATION SERVICE AREA

Diversion Dam	River	Main Irrigation Canal	Net Irrigable Area
Asue D.D. Asue	Asue	Asue M.C.	2,250ha
		Eastern M.C.	2,400
		Sub-total	4,650
Bakabak D.D.	Asue	Bakabak Rigth M.C.	610
		Bakabak Left M.C.	390
		Sub-total	1,000
Gubaton D.D.	Gubaton	Gubaton M.C.	520
Trans-diversion Canal	Catipayan	Trans-diversion Canal	190
Sub-Total			6,360
Enriched Benefit Area			
Serruco D.D. (existing)	Serruco	Serruco Rigth M.C.	175
		Serruco Left M.C.	185
		Sub-total	360
KABSAKA Water Impounding area (Existing)			
		Castor WIP	5
		Moto WIP	10
		Bondolam WIP	25
		Sub-total	40
Sub-Total			400
Total			6,760

2.4 Proposed Irrigation System

2.4.1 Peak Replenishment Water

(1) Trans-diversion Canal

The necessary trans-diversion capacity from the Catipayan River was estimated at 6.0m^3/sec at the dam site on the basis of water balance study. Diverted water will be used to irrigate 100ha of area before the tunnel inlet. Moreover, water will be diverted to irrigate 90ha of upper Asue Area after the tunnel outlet just downstream of the canal route hydropower station. The 5.8m^3/sec of diverted water from the Catipayan reservoir will be released to a small tributary of the Asue River at Barangay Malapaya for intake at the Asue diversion dam. Capacity of the small tributary of the Asue River, however, is insufficient to convey the diverted water.

Field investigation indicates that an extension of 650m in river length will sufficiently increase the discharge capacity of the tributary to allow a discharge of 5.8m^3/sec.

(2) Linkage Canal

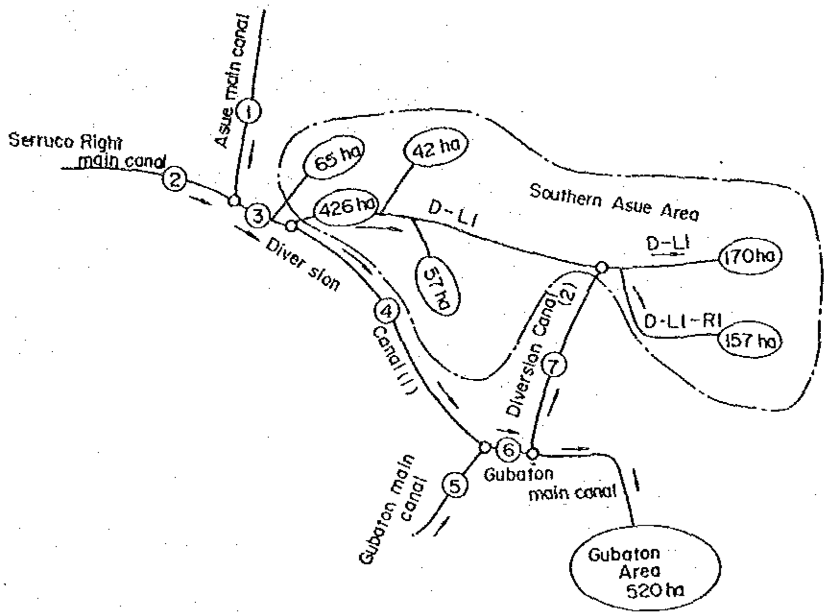
Interlinked main canal networks among the Asue, Serruco and Gubaton main canals make possible mutual replenishment of irrigation water. There are three routes for supplementary supply of irrigation water as explained below. The routes are shown in FIG. VII-12.

a) Catipayan Reservoir to Gubaton Area
In the Gubaton area, Gubaton river flow will be utilized for irrigation in preference to water from the Catipayan reservoir. However, when surplus water in the Serruco River is available, it will be utilized to replenish the Gubaton area after intake of Gubaton river flow.

b) Serruco River to Southern Asue Area
As shown in FIG. VII-12 when surplus Serruco river flow occurs after supplying water to the Serruco enriched area, the surplus water is conveyed to the Southern Asue area through the Serruco right main canal and diversion canal (1).

c) Gubaton River to Southern Asue Area
Surplus water in the Gubaton area can be diverted to a part of southern Asue area through the Gubaton main canal and diversion canal (2).

Based on the above mentioned routes and reservoir water operation studies for a 20-year period, maximum replenishment water capacities were obtained. The canal routes are schematically illustrated below.



a) Serruco to Southern Asue area

The maximum replenishment capacity occured in June in the second 10-day period of 1969 at 0.380 cu.m/sec, and the routes are from (2)-(3) to D-L1.

b) Gubaton to Southern Asue area

Maximum capacity occured in June during the first 10-days of 1967 at 0.428m^2/sec and the routes are from (5)-(6)-(7)-D-L1

c) Asue to Gubaton Area

The maximum capacity occured in June during the first 10-day period of 1973 at 0.503m^3/sec along the routes of (1)-(3)-(4)-(6) to the Gubaton area. However, when no surplus water is available from either the Serruco and Gubaton rivers, irrigation water is supplied from the Catipayan Reservoir through the Asue main canal and the routes are from (1)-(3) to D-L1. The design capacity of these canal networks was decided as presented in the table on the following page.

Name of Canal	Max. replenishment Capacity (m^3/sec)	Irrigation Area (ha)	Irrigation[1/] Capacity (m^3/sec)	Total Capacity
1. End of Asue main canal	0.503	491	1.007	1.510
2. End of Serruco right main canal	0.380	0	0	0.380
3. Top of diversion canal (1)	0.503	491	1.007	1.510
4. Diversion canal (1)	0.503	0	0	0.503
5. Gubaton main canal	0.428	520	1.067	1.495
6. Gubaton main canal	0.428	520	1.067	1.495
7. Diversion canal (2)	0.428	0	0	0.428

1/ The design unit water requirement for the main canal is 2.051ℓ/sec/ha.

2.4.2 Design Capacity of Main Canals

The design capacity of each type of canal can be decided from the irrigation command acreage, adopting the unit design water requirement in accordance with the type of irrigation canal. As explained above however, some of the main canal routes will also be used to supply water to the irrigation water deficit area. The obtained maximum replenishment capacity occurs during the land soaking period. Therefore, the design capacities for such canals should be decided on the basis of irrigation water duty plus maximum replenishment water capacity.

The design capacity of the main canals was decided as shown in the following table.

Name of Main Canal	Max. Replenishment Capacity (m^3/sec)	Irrigation Area (ha)	Irrigation[1/] Capacity (m^3/sec)	Total Capacity
Asue main canal	0.503	2,250	4.615	5.118
Eastern main canal	0	2,400	4.922	4.922
Bakabak right M.C.	0	610	1.251	1.251
Bakabak left M.C.	0	390	0.800	0.800
Gubaton M.C.	0.428	520	1.067	1.495
Serruco left M.C.	0	185	0.379	0.379
Serruco right M.C.	0.380	175	0.359	0.739

1/ The design unit water requirement for the main canal is 2.051 ℓ/sec/ha.

As for the Serruco main canals, the existing main canals should be utilized to avoid double investment. The existing capacity of the right main canals is $1.024 m^3/sec$ which is sufficient for the design capacity of $0.739 m^3/sec$.

2.4.3 Design Capacity of On-farm Facilities

At the on-farm level, main farm ditches (M.F.D) and supplementary farm ditches (S.F.D.) are proposed to facilitate the introduction of modern irrigation water management. In addition, the S.F.D. has two functions, irrigation and drainage, to take advantage of the steep slope of the paddy field.

Water management at the on-farm level includes not only simultaneous irrigation, especially during the land soaking period due to the high water requirement, but also rotational irrigation during the growing stage. Therefore, the design capacity of the S.F.D. should satisfy the maximum capacity of the above three functions i.e. simultaneous and rotational irrigation and drainage.

DESIGN CAPACITY OF S.F.D.

Method / Item	Simultaneous Irrigation during Land Soaking Period	Rotational Irrigation during Growing Period	Drainage Capacity in 10-year Return Period
Command area (ha)	10ha	10ha	10ha
Unit requirement	2.242ℓ/s/ha	1.790ℓ/s/hax5day[1/]	5.0ℓ/s/ha[2/]
Design discharge	22.42ℓ/s	89.5ℓ/s	50.0ℓ/s

Note: 1/ Daily maximum consumptive use is 8.4 mm/day, the percolation rate is 1.5mm/day and the on-farm efficiency is 0.64, so that (8.4 + 1.5)/0.64/8.64 = 1.790ℓ/s/ha

2/ The unit drainage discharge from the paddy field was obtained at 5.0ℓ/s/ha as explained in Chapter 3.

Accordingly, the design capacity of the S.F.D. has been decided at 89.5ℓ/s.

As for the M.F.D. the command area is about 50ha and the design unit water requirement is 2.242 ℓ/s/ha. The M.F.D. is not used for drainage purposes due to the alignment of the canal, and the design capacity for the same was determined at 2.242ℓ/s/ha x 50ha = 112.1ℓ/s.

2.4.4 Proposed O & M Roads

All weather gravel roads are to be provided along the embankment of main canals, laterals and sub-laterals at an intensity of about 20m of O & M and linkage roads for each hectare of service area based on the following criteria.

a) When a small canal runs parallel to an existing road within a center to center distance of 40m, the road embankment may be omitted.

b) Generally, the road embankment shall be placed at the service area except in existing canals where a road has been set up along the canal embankment, in which case, the location of the existing road will be maintained.

c) When both sides of the canal are serviced, the road shall be placed at the wider embankment of the existing canal.

d) If possible, the road along the same side of the canal should be maintained from the head gate to the end to prevent vehicles/equipment from switching from left to right embankment or vice versa.

e) Generally, the canal operating roads should terminate at the furthest turnout of a canal. However, when a canal terminates near an existing road or another canal, a connecting road may be provided.

f) Bridges are to be provided along the O & M roads parallel with siphons where concrete pipe or box culverts are not economical. Bridges most commonly used are reinforced concrete deck girder (RCDG) type with 3.50m roadway width and 0.46m sidewalks on both sides.

3. PROPOSED DRAINAGE PLAN

3.1 Concept of Drainage Plan

Typhoon Undang provided actual evidence of drainage problems in the field, as explained in section 1.3 The events of the typhoon clearly indicated several urgently required drainage improvements. Moreover, a sophisticated runoff analysis was introduced to anticipate drainage discharge analysis for conditions after project completion. The following basic concepts are proposed for the drainage plan.

(1) Design Return Period

A 10-year return period was adopted for designing the drainage facilities, taking into acount the events of typhoon Undang. The return probability of typhoon Undang is assumed at about 20-years or more, and the water stagnation period in the field in most cases was 20-30 hours. This indicated that the present river courses normally have the capacity to discharge a flood with about a 10-year return period except for small portions of the creek and rivers.

(2) Existing Weirs

Existing private concrete weirs normally have no control structures to mitigate flood effects in the upstream area and accordingly these weirs sometimes exaggerate upstream flood damage. The same should therefore, be removed. The slope of the present river course, however, is approximately 1/300 to 1/500 which is almost the critical slope of the river. As a result, removal of the existing weirs will make the river flow steeper than at the present. Main river training or straightening of the main river course is thus in advisable without construction of drop structures, except in the rather flat portion of rivers such as the Hasohoy River or creeks.

(3) S.F.D as Drainage Facilities

As explained in the concepts of the irrigation plan, the S.F.D is planned for drainage purposes as well. When existing creeks or rivers are not available near the S.F.D networks, however, new project drains or tertiary drains are proposed to discharge excess water smoothly.

(4) Main Drainage Improvement Works

As for the drainage improvement plan, careful and intensive investigations were made to determine drainage problems in flood conditions during Typhoon Undang. Based on the results of the same the following three improvement works are proposed for implemention in such areas.

a) Re-excavation of existing creeks and rivers

The Hosohoy River and Padios Creek do not have sufficient capacity to drain flood water. As drainage improvement at the on-farm level may increase flood water in the rivers, re-excavation works are proposed.

b) Re-construction or rehabilitation of existing culverts

As seen during the flood, the capacity of several cross drain structures upstream under the road is insufficient to discharge river flow. Rehabilitation of such structures is recommended.

c) New drainage canal construction

Near the San Dionisio area, flood water from the southern portion of the Eastern Area spilled over the existing road and flowed into the Hasohoy area. In order to intercept flood water, a new drainage canal is proposed along the road. Also where the existing creek or rivers are not sufficient for drainage, new drainage canals are proposed.

3.2 Drainage System Networks

(1) Schematic Drainage Networks

The catchment areas of the rivers in the service area were divided into sub-catchment areas according to topography and existing tributary and drainage systems. These sub-catchment areas were further sub-divided into two types of land use; paddy fields which have storage capacity, and slope area such as uplands, residential areas, sugarcane plantations, etc.

The boundaries of sub-catchment and paddy field areas are shown in FIG. VII-13 and schematic drawings of the drainage system for each river are shown in FIG. VII-14

(2) Characteristics of Sub-catchment Areas

The shapes of the sub-catchments are modified rectangles in accordance with the land use. The lengths and width of the same are presented in TABLE VII-11. The adopted numbering system, paddy and non-paddy acreages, and sub-catchment slopes are also indicated in the table and these correspond to the numbering system in FIG. VII-13 and FIG. VII-14.

(3) Characteristics of Drains and Rivers

Cross-sections of existing rivers, tributaries and drains were surveyed at approximately 500m intervals. In accordance with the obtained cross-sections, discharge-flow area relationships were derived based on Manning's equation, a roughness coefficient of 0.03 and slopes of rivers. Flow area and discharge equations were fitted by the least squares method.

3.3 Runoff Analyses for Drainage System

3.3.1 Rainfall Analysis

(1) Design Rainfall

As shown in the Hydrology Section, the incremental rainfall from one day to two days for a 10-year return period is 21.6mm/day which is about 10% of daily rainfall. Moreover, as confirmed during typhoon Undang, water stagnation in most cases lasts less than 24 hours. Consequently, one day maximum rainfall (24 hours maximum) has been considered for drainage analysis rather than adoption of successive rainfall.

Several observed hourly rainfall patterns were examined for design rainfall. The flood concentration time in the Asue River is approximately 5 to 6 hours. Accordingly, the selection of the patterns involved not only peak hourly rainfall but also 6 hour duration rainfall.

The pattern of design rainfall was estimated on the basis of the ratio between the observed and 10-year return period of rainfall corresponding to the maximum durations of one hour, 6 hours and 24 hours. The design rainfall patterns for 10-year and 25-year return periods are shown in FIG. VII-15.

(2) Effective Rainfall

The runoff characteristics in the Asue River basin are rather different from those in the Catipayan River catchment area as the geological and topographical conditions are quite different. The geological conditions in the Asue River basin consist of deeply weathered diolite features similar to weathered granite.

Therefore, the effectiveness of rainfall is normally quite low and loss is rather high. From the observed river discharges in the Catipayan, Serruco and Asue rivers, the runoff ratios are 74%, 54% and 43%, respectively on an annual average basis. The runoff ratio for floods in the Asue River is also expected to be quite low. Unfortunately, however, no observed flood records could be obtained in the Asue River.

In paddy fields, the runoff mechanization can be presumed from the hydraulic point of view as explained hereinafter and the cumulative rainfall to cumulative loss of rainfall for paddy fields, was estimated as shown in FIG. VII-16. The same relationship was estimated for hilly and mountain area in the surrounding service area. The overall loss in the Asue River can be estimated from the long term discharge records. The runoff ratio in the hilly area can be estimated by subtracting paddy loss from the overall loss. The runoff mechanism in the Asue River can also be represented in the runoff model by the tank model method as shown in Appendix I.

The estimated curve for cumulative rainfall to cumulative loss for the hilly and mountainous area is shown in FIG. VII-16.

3.3.2 Runoff Analysis for Hilly Area

(1) Method of Runoff Analysis

Most catchment runoff models for estimation of floods are based on empirical approaches and rely heavily on observed data to determine model parameters. Consequently, when no observed data is available or when the features of the catchment area are expected to change in future, empirical methods can be very difficult to adopt.

At present, the Asue Area has very few terminal drainage facilities in paddy fields. Under Project implementation however, terminal drainage facilities will be provided to improve drainage conditions. The runoff model to be adopted for the Asue Area should therefore be able to simulate such changes in the catchment area.

In order to meet the above requirement, the Kinematic Wave method, or Characteristic method, an approach which utilizes hydraulic equations to solve unsteady flow in open channels, can be adopted to simulate overland and channel flows. However, runoff characteristics from paddy are somewhat different from overland flow because of the storage capacities in the paddy field. These storage characteristics must also be integrated into the model. Consequently, a runoff model incorporating these two features was established for runoff simulation analysis. The basic concepts are presented below.

(2) Equation of Motion and Continuity

The equation of motion and continuity for unsteady flow at constant lateral inflow is as follows:

$$\frac{\partial u}{\partial t} + \alpha u \frac{\partial u}{\partial x} - (\alpha - 1) \frac{u \partial A}{A \partial t} + g \cos\theta \frac{\partial h}{\partial x} = g \sin\theta - \frac{\tau o}{\rho R} - \frac{\alpha u g}{A} \quad \ldots (1)$$

$$\frac{\partial A}{\partial t} + \frac{\partial u}{\partial x} = q \quad \ldots\ldots\ldots\ldots\ldots\ldots (2)$$

where, q: lateral inflow in unit length of the canal

u: mean velocity

A: water area

h: water depth

R: hydraulic radius

Q: discharge

α: coefficient

θ: canal slope angle

ρ: density of water

τ_o: shear stress on the canal bottom

g: gravitational acceleration

x: distance

t: time

Determination of the exact solution of the above equations is extremely elaborate and complex. However, approximate solutions are obtained by assuming the lateral inflow to be steady and uniform as follows:

$\alpha = 1$ and the shear stress to be given by the equation

$$\frac{\tau_o}{R\rho} = \frac{n^2 g\, u^2}{R^{3/4}}$$

Where n is the coefficient of roughness. The approximate characteristic solution to the equations is as follows:

$$\frac{dx}{dt} = \left(1 - \frac{2}{3\beta}\right) u + \frac{\left(1 - \frac{2}{3}\beta\right) u g R^{4/3}}{2n^2 g A u + g R^{4/3}} \quad \ldots\ldots \; (3)$$

Where $\frac{dA}{dt} = q$ . . . . (4)

or $\frac{dQ}{dx} = q$ . . . . (5)

subject to $= (R/A)/(dR/dA)$

The relation of A and u can be given as follows:

$$u = \sqrt{\left\{\left(1 + \frac{2}{3\beta}\right) \frac{gR^{4/3}}{2gn^2A}\right\}^2 + \frac{R^{4/3}}{n^2} \sin\theta - \frac{2\left(1-\frac{2}{3}\beta\right)u\left(qR^{4/3}/2gn^2A\right)^2}{u + qR^{4/3}/2gn^2A}}$$

$$- \left(1 + \frac{2}{3\beta}\right) \frac{qR^{4/3}}{2gn^2A} \quad \ldots\ldots\ldots \; (6)$$

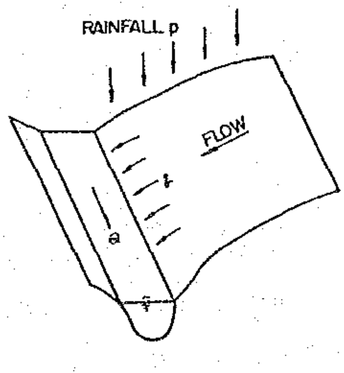
When $q = 0$

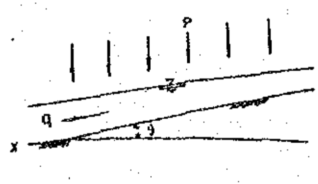
$$\frac{dx}{dt} = \left(1 + \frac{2}{3\beta}\right) u \quad \ldots\ldots\ldots \; (7)$$

$A = \text{const}$ or $Q = \text{const}$ . . . . (8)

$$Q = Au = \frac{A}{n} R^{2/3} (\sin\theta)^{\frac{1}{2}} \quad \ldots\ldots\ldots \; (9)$$

(3) Overland Flow





The continuity and resistance equations of channel flow can be expressed as:

$$\frac{\partial A}{\partial t} + \frac{\partial Q}{\partial x} = q \qquad \text{. . . . . (1)}$$

$$Q = \frac{A}{n} R^{2/3} i^{\frac{1}{2}} \qquad \text{. . . . . (2)}$$

where i is the energy gradient

For a steady state uniform flow condition, the discharge can also be represented by

$$A = KQ^P \qquad \text{. . . . . (2)}$$

where K and P are constants.

From the above three equations, the following equations are derived:

$$\log t = \log K + P \log Q - \log q \qquad \text{. . . . . (4)}$$

$$\log t = \log K + \log x + (P - 1) \log Q \qquad \text{. . . . . (5)}$$

and if the lateral inflow is neglected (i.e. q = o),

$$x = \frac{1}{KP} Q^{1-P} t \qquad \text{. . . . . (6)}$$

When applied to overland flow, Q and q must be replaced by q and r respectively where r is the rainfall rate and the coefficient of surface runoff.

Equations (4) and (5) can then be rewritten as:

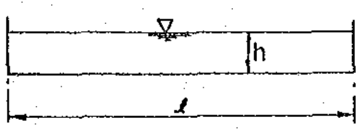
Equi-Rainfall Line

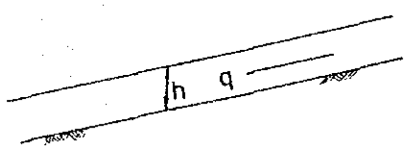
$$\log t = \log K - \log r + P \log q \qquad \text{. . . . . (7)}$$

Equi-Distance Line

$$\log t = \log K + \log x + (P-1) \log q \qquad \text{. . . . . (8)}$$

For a unit width of a wide rectangular channel





$R = h$

$A = 1 \times h = h$

$q = \frac{A}{n} R^{2/3} i^{\frac{1}{2}} = \frac{A^{5/3} i^{\frac{1}{2}}}{n}$

$$A = \left(\frac{n}{i^{\frac{1}{2}}}\right)^{0.6} q^{0.6} \qquad \ldots\ldots (9)$$

Replacing Q in Equation (3) by q and comparing,

$K = \left(\frac{n}{i^{\frac{1}{2}}}\right)^{0.6}$ and

$P = 0.6$

Substituting the values of K and P in equation (6) and replacing Q by q, and thus t is expressed as follows:

$$t = 0.6 q^{-0.4} \left(\frac{n}{i^{\frac{1}{2}}}\right)^{0.6} x \qquad \ldots\ldots (10)$$

3.3.3 Runoff from Paddy Field after the Project

With the calibrated runoff model, improvement of drainage channels in the paddy field is incorporated into the model. The proposed improvement of drainage channel systems at the on-farm level is as shown in FIG. VII-17.

The basic concept of the improvement method is maximum utilization of the storage capability of the paddy as a high discharge rate from paddy will have serious effects on downstream areas. Paddy crops, on the other hand, are not damaged by water stagnation in the field for periods of 1 to 1-1/2 days.

Paddy runoff has the following characteristics:

a) During the early stage of rainfall, all of the rainfall in the paddy fields will be stored up to the crest of the notch of the "farm spillway" which becomes the point of initial loss. The height of the notch is about 50mm to 100mm and is equal to standing water depth in the field.

b) Runoff will occur when water depth in the field becomes higher than the crest of the notch. However, the height of the notch varies from plot to plot.

c) When rain falls continuously, the water level will increase and runoff will occur from every notch in the plot. At this time, all fields are inundated and further rainfall will discharge across the notch.

d) All the over spill from the notch is drained into quarternary drains. The capacity of these quaternary drains is determined by their slope and size and if their capacity is too great, a high rate of discharge to downstream sections will occur. The size of the channel should therefore be designed to allow stagnation of water in the paddy field. In other words, discharge should be initially controlled by the size of the quaternary drainage channel.

e) If the tertiary canal crosses a farm road, culverts will be required under the same. The size of culverts can also be designed to control discharge.

The characteristics of runoff from paddy fields are illustrated in FIG. VII-18.

3.3.4 Design Discharge for Drainage Facilities

In accordance with the schematic diagram of the river basin in the service area and the runoff model, runoff analysis for 10-year and 25-year return periods was carried out.

A specific runoff hydrograph for the 10-year return period of paddy is shown in FIG. VII-19. The peak discharge occured at 24 hours from the beginning of rainfall at 1.8mm/hour which corresponds to 5.0ℓ/sec/ha in the paddy field. Design unit drainage discharge from pure paddy was determined at 5.0ℓ/sec/ha.

The design hydrograph of a 10-year return period at several points in the Asue River basin are shown in FIG. VII-20 and those for other rivers are shown in FIG. VII-21. In the Asue River basin, peak discharge occurred at 13 hours from the start of rainfall and 5 hours after peak hourly rainfall.

Based on the results of runoff analysis, the peak discharge is varied due to the shape of the sub-catchment area, land use and slope. Specific discharge from the hilly area therefore can not be determined simply. The relationship between catchment area and peak discharge along the Asue main river were plotted as shown in FIG. VII-22. The specific runoff varies from 22ℓ/s/ha to 14ℓ/s/ha in accordance with the size of catchment area.

As for the design discharge for 10-year and 25-year return periods, the peak discharge at various points along the rivers in the service area are shown in accordance with the schematic diagram in FIG. VII-23 and FIG. VII-24, respectively. The 25-year return period can be adopted for designing culverts under the main roads such as national and provincial roads.

3.4 Proposed Drainage Improvement Works

Based on the results of the drainage discharge analysis, the majority of the river course was found to have a capacity of about a 10-year return period discharge. River course training works will in general be unnecessary; however, removal of existing private weirs will be required. In addition, a supplementary project drainage canal will be necessary to facilitate drainage in the flat portion of the Eastern Area and some sections of the Asue Area where no existing creeks are available.

(1) Drainage Improvement Works

In accordance with the drainage discharge analysis along the rivers and creeks in the service area and the results of the flood mark investigation on typhoon Undang, the following three types of drainage improvement works are proposed.

- New drainage canal construction
- Improvement of existing drainage structures
- Excavation of existing creeks and rivers

The most severe flood area, determined from area inundated more than 24-hours, is in the Hasohoy area. According to the topographic map scale 1 to 10,000, the area along the Hasohoy River is very flat with a small river slope. Excavation to enlarge the flow section and to obtain a steeper slope of river flow is therefore recommended. A similar situation occurs in Padios Creek and excavation of the same is also recommended.

In the San Dionisio area and southern part of the Eastern Area, flood water flowed over the road between Santol and San Dionisio and into the Hasohoy area. As these flood waters exaggerated drainage problems in the Hasohoy area, construction of new drainage channels along the road is required. The location of the drainage improvement works are presented in FIG. VII-10.

(2) Design Flood Discharge for Diversion Dam

As for the design flood discharge for diversion dams, the 50-year return period flood by the hyetogaph mentioned in APPENDIX I, was adopted. A rubber type dam was selected for the diversion dam and accordingly the cross section of the dam site during the flood was assumed as the collapsed condition of the rubber dam. High water level during the flood was thus calculated.

The same condition was adopted to estimate the design flood at the proposed diversion dam sites. The design flood at each diversion dam site is presented in the table below.

Diversion Dam	River	Catchment Area (km^2)	Design Flood (m^3/sec)
Asue	Asue	13.70	94.74
Bakabak	Asue	116.02	611.92
Gubaton	Gubaton	18.80	154.36

4. ON-FARM DEVELOPMENT PLAN

4.1 Objectives and Measures of On-farm Development Plan

4.1.1 Objectives

On-farm development can be categorized into the following main objectives.

(1) Irrigation

To introduce modern irrigation techniques and efficient water management for rotational irrigation in order to attain timely water supply and control water supply in accordance with crop demand and growth.

(2) Drainage

To drain excess water or excess rainfall within a short period from the farm field and to reduce the ground water table in order to increase aeration of the crop root zone, to facilitate the introduction of farm machinery and to increase the efficiency of farm machinery in the field.

(3) Farm Roads

To increase accessibility to farm plots and attain high efficiency of modern farming practices and efficient transport of farm input materials and products.

In order to attain successful on-farm development, provision of the following facilities is planned at the on-farm level. On-farm facilities are defined below.

1) Plot

A plot is the minimum unit of farm field encircled by ridges. Farm work from seeding to harvesting should be carried out on the basis of this plot as a unit. The existing plots generally cover approximately 0.2 to 0.3ha each.

2) Rotational Unit (RU)

The rotational unit is the minimum area for the operation of irrigation and drainage facilities at the on-farm level. It covers an area of about 10ha and is made up of several plots. Rotational units are served by supplementary farm ditches (S.F.D).

3) Rotational Area (RA)

Each rotational area consists of about five rotational units, covering a total area of about 50ha and served by one turn-out through the main farm ditch (M.F.D).

4) Supplementary Farm Ditch (S.F.D)

S.F.D are connected with the M.F.D at the division boxes and supply water to the respective rotational units.

5) Rotational Irrigation

Rotational irrigation is carried out through the S.F.D. by supplying irrigation water on a plot to plot basis.

4.1.2 Countermeasures

(1) Irrigation Facilities

As for irrigation facilities, new main farm ditches and supplementary farm ditches are aligned as shown in FIG. VII-24.

In the layout of supplementary farm ditches (S.F.D), existing small canals in paddy fields can be fully utilized. Furthermore, existing small ditches can also be connected by the systematically arranged S.F.D. Accordingly, irrigation water can be conveyed more effectively and the time required for water to reach each plot can be shortened.

(2) Drainage Facilities

The S.F.D alignment is proposed along the contour line in 200m intervals. The slope of existing paddy is approximately 1/300, therefore the difference in elevation between a certain pair of two S.F.Ds is an average of about 200m/300 ≑ 0.7m. The lower S.F.D is able to intercept surplus water from the upper rotational unit. Accordingly, the S.F.D can be utilized not only for irrigation water supply but also for drainage of the surplus water in the field.

(3) Farm Roads

The operation and maintenance roads along the main, lateral and sub-lateral canals can also be used as farm roads. The layout of the O&M road, however, was basically made in accordance with the layout of the irrigation canals.

In order to establish a better transportation network, linkage roads are proposed where necessary. By a combination of O&M roads and linkage roads, not only can efficient farming practices and transportation of farm products be achieved, but also these new road networks can contribute to improved transportation and communication for public utilities as well.

4.2 Sample Areas for On-Farm Development

4.2.1 Layout of On-farm Facilities

Layout of on-farm facilities was planned on the basis of the two selected sample areas in the Project area. One sample area, Area "A", is located in the flat area and served by lateral irrigation canal E-L1 near Barangay Devera and the other sample area, Area "B", is located in the steep area served by lateral irrigation canal A-L3 near Barangay Alibyg.

The basic conception for layout of on-farm facilities is described hereunder.

(1) Farm Ditches

The average distance between supplementary farm ditches in a rotation area is about 200m. Main and supplementary farm ditches should be located along tenant or lot boundaries and such locations should be selected so that the service area can be irrigated with a minimum water surface in the farm ditch of 20cm above the highest natural ground level in the area. Existing small ditches in the paddy field should be utilized as much as possible.

There should be one supplementary farm ditch for each rotation unit. However, a common supplementary farm ditch for two adjacent rotation units may be constructed depending upon the topography. To have a permanent visual boundary between rotation units, the supplementary farm ditches should be located along sides of rotation units except when due to topography a farm ditch must be located inside the rotation unit.

(2) Farm Drains

Supplementary farm ditches should be used for farm drains as well. Farm drains should be located where paddy fields are partially depressed or where surplus water should be drained into the nearest creek or river.

(3) Farm Roads

Farm roads are provided along the main farm ditch where needed, but basically O&M roads along irrigation canals can be used as farm roads.

(4) Turnout

The ideal location of a turnout is at the inlet of a check structure although in certain instances when there is sufficient head and irrigable area at the downstream side of the structure, it will be more economical to locate the turnout at the outlet of a road crossing provided with a check.

When turnout is necessary further upstream of a check structure, its operating head is to be based on a water surface elevation equal to the checking height of the downstream sector of the water flow.

4.2.2 Proposed On-farm Development

(1) Comparison of Terminal Water Supply System

Generally, there are no constraints to alignment of on-farm facilities in the flat area. However, in steeply sloped area such as the Project area, which has an average land slope of about 0.3%, on-farm facilities should be carefully aligned as construction cost will vary in accordance with the number of drop structures in the canal.

Two types of alignment were considered for the terminal water supply system as follows:

Type I: Supplementary farm ditches aligned along the contour line so that drops for the supplementary ditches are not necessary

Type II: Supplementary ditches provided across the contour line in which many drops for the supplementary ditches are required.

From an economic point of view, Type I is more advantageous than Type II. In addition, the supplementary farm ditches for Type I, of which the average distance between the ditches is about 200m, can be used simultaneously as farm drains facilitating drainage of excess water from the field during floods.

Type I is therefore recommended as the water distribution system for the steep area due to topographical conditions and the same should be adopted wherever possible in the Project area.

4.3 On-farm Facilities

On-farm facilities and the functions of the same are outlined as follows:

a) Turnout - Provided with a steel gate to control and regulate the flow of water to farm ditches

b) Measuring Device - Constant head orifice

c) Main Farm Ditch - Conveys water from the turnout to the supplementary farm ditches

d) Supplementary Farm Ditch - Conveys water from the main farm ditch to the paddy in a rotation unit

e) Division Box - Checks and diverts water from the main farm ditch to the supplementary farm ditch

f) End Check - Prevents leakage of irrigation water from the supplementary farm ditch to the drainage ditch when it is needed in paddy fields and releases irrigation water from the same when not needed

g) Farm Ditch Crossing - Provides access to farm equipment from farm road to the paddy fields in a rotation area; this is installed only on main farm ditches. On supplementary farm ditches, equipment may cut across.

h) Farm Ditch Check and Drop - In steep slopes, limits the flow of water in the farm ditch within the non-scouring velocity

i) Farm Drain - Drains the paddy fields when necessary and drains surface runoff during floods

The quantity of the proposed on-farm facilities is discussed in the following section. These quantities were estimated based on the sample layout of on-farm facilities.

5. PROPOSED FACILITIES

5.1 Diversion Dam

5.1.1 General

Three diversion dams were proposed on the Asue River and the Gubaton River. According to the investigation for drainage, in the upstream along the Asue River, paddy was affected by flood. These floods were mainly caused by backwater from existing weirs. An automatic collapsible type dam is recommended for its economic feasibility and to prevent backwater.

The main features of the three diversion dams are tabulated in the following table.

Diversion Dam	River	B(m)	H(m)	Irrigable Area (ha)
Asue	Asue	12.0	2.4	4,650
Bakabak	Asue	27.6 x 2	3.0	1,000
Gubaton	Gubaton	20.0	5.0	520

5.1.2 Asue Diversion Dam

(1) River Condition

The location of the proposed Asue Diversion Dam is about 700m downstream from the existing bridge crossing the provincial road. Present river width is about 15m with a 1/550 slope and a meandering river course. River embankments do not exist on either side.

(2) Intake Water Level

The required water level at the beginning of both main canals, the Asue Main Canal and Eastern Main Canal, is determined at EL 33.0m, and intake water level at EL 33.3m with an estimated intake loss of 0.30m.

(3) <u>High Water Level</u>

Design flood discharge of the Asue River at the diversion dam analysed by the Kinematic Wave Method are as follows:

25-year return period	$71.69m^3/s$
50-year return period	$94.74m^3/s$

Design flood discharge for the existing Serruco Diversion Dam under CIS is $99.04m^3/s$. This value is nearly the same as the 25-year return period discharge of $101.71m^3/s$ analysed by the Kinematic Wave Method. Accordingly, 25-year return period could be adopted for the Asue diversion dam. The design flood discharge for the Asue diversion dam, however, has been conservatively determined on the basis of a 50-year return period discharge similar to other national projects in the Philippines.

High Water Level of the dam was determined at EL 33.8m which is on the water depth timed 0.2 of dam height from the upper part of the dam.

(4) <u>Preliminary Design</u>

Major dimensions of structural features are as follows:

- Diversion Dam

Bottom Length	12.0m
Dam Height	2.4m
Sill Elevation	EL 30.9m for upstream
	El 29.9m for downstream

- Intake Structure

Intake Water Level	EL 33.3m
Design Intake Capacity	$5.118m^3/s$ for Asue Main Canal $4.992m^3/s$ for Eastern Main Canal
Gate Type	Sluice gate
Gate Dimension	2.0x1.4x2 nos for Asue Main Canal
	2.0x1.4x2 nos for Eastern Main Canal

5.1.3 Bakabak Diversion Dam

Considering the performance of the rubber dam, the Bakabak Diversion Dam should be divided into a double span due to the large river width.

(1) River Condition

The Bakabak Diversion Dam was proposed on the Asue River 250m downstream from the confluence with the Serruco River.

Present river width is about 60m, slope is 1/700 and the flood basin is clearly identified.

(2) Intake Water Level

The required water levels of the Bakabak main canals, (left, right) are determined at EL 7.5m, and intake water level at EL 7.8m adding an intake loss of 0.30m.

(3) High Water Level

Design flood discharge of the Asue River at the dam, which was analysed by the Kinematic Wave method in the same way as for the Asue Diversion Dam are as follows:

25-year return period	471.76m^3/s
50-year return period	611.92m^3/s

Adopting the 50-year return period discharge as the design flood discharge, the high water level of the dam was determined at EL 8.4m.

(4) Preliminary Design

Major dimensions of the structures are presented as follows:

- Diversion Dam

Bottom Length	27.6m x 2m
Dam Height	3.0m
Sill Elevation	EL 4.8m for upstream
	El 3.8m for downstream

- Intake Structure

Intake Water Level	EL 7.8m
Design Intake Capacity	$1.251m^3/s$ for Bakabak Right Main Canal
	$0.800m^3/s$ for Bakabak Left Main Canal
Gate Type	Sluice gate
Gate Dimension	1.5x1.0 for Bakabak Right Main Canal
	1.0x1.0 for Bakabak Left Main Canal

5.1.4 Gubaton Diversion Dam

(1) River Condition

The Gubaton Diversion Dam site is proposed on the Gubaton River 2km upstream where it crosses the national road which runs from south to north.

Present river width is about 20m and slope is 1/750. The river regime is nearly ravine and the riverbed is deeply excavated. There is a provincial road along the river on the left side, which will be utilized as the access road for construction, and as a maintenance road after completion of the dam.

(2) Intake Water Level

The required water level of the Gubaton Main Canal is determined at EL 17.5m, and intake water level at EL 17.8m adding an intake loss of 0.30m.

(3) High Water Level

Design flood discharge of the Gubaton River at the proposed dam site, analysed by the Kinematic Wave method are as follows:

25-year return period	$115.04m^3/s$
50-year return period	$154.36m^3/s$

Adopting the 50-year return period discharge as the design flood discharge, the high water level of the dam was determined at EL 18.4m.

(4) Preliminary Design

- Diversion Dam

Bottom Length	20.0m
Dam Height	5.0m
Sill Elevation	EL 12.8m for upstream
	El 10.8m for downstream

- Intake Structure

Intake Water Level	EL 17.8m
Design Discharge	1.495m^3/s for Gubaton Main Canal
Gate Type	Sluice gate
Gate Dimension	1.5x1.2 for Gubaton Main Canal

5.2 Irrigation System

5.2.1 Irrigation Canal and Appurtenant Structures

(1) Canal Alignment

After thorough investigation of the topographical conditions in the service area, layout of the canal network was plotted on a 1:4,000 topographical map. Particular attention was given to maximize acreage of irrigable land and maintain existing irrigation systems wherever possible. Proposed canal lengths are shown in TABLE VII-12.

(2) Design Criteria

1) Mean velocity formula

Manning's Formula was used to determine the canal cross section, as presented below.

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

$$Q = A \cdot V$$

$$= 1/n \cdot A \cdot R^{2/3} \cdot I^{1/2}$$

where,

V : Mean velocity (m/s)

n : Roughness coefficient

R : Hydraulic radius (m)

I : Surface slope

Q : Discharge (m^3/s)

A : Cross sectional area (m^2)

Where it is assumed that a steady and uniform flow is a given factor, the water surface slope is equal to the canal bed slope.

(3) Roughness Coefficient

Roughness coefficient (n) of the Manning Formula is mostly affected by configuration of the canal and its material. The general "n" value adopted for planning is given in the table presented below.

ROUGHNESS COEFFICIENT

Facility	"n" value
Main Lateral Earth Canal	0.025
Concrete Flume	0.015
Concrete Pipe and Culvert	0.015
Farm Ditch	0.030

(4) Allowable Velocity

Water velocity should be within the minimum velocity of anti-hydrophyte to growth and maximum velocity which will not cause erosion nor an unstable flow. Factors to identify minimum velocity are indefinite, but generally they are assumed to be in the range of 0.25-0.45m/s for unlined canals. Maximum velocity can be determined by Kennedy's Formula provided below which is designed to prevent erosion:

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

where, Va: allowable maximum velocity (m/s)

D : water depth (m)

C : coefficient by different nature of soil; in this plan 0.546 of the general value for sandy loam is adopted.

(5) Canal Cross Section

Cross section of the canal is planned as a trapezoid type, and the design criteria are as given below.

1) Base-depth ratio

When taking the smallest B/d ratio into consideration for the earth canal, seepage losses can be controlled with reduced wetted perimeter. However, this approach is not convenient for operation and maintenance. Moreover, a large variation flux effects the variation of water level resultant in diversion. On the other hand, the large B/d ratio requires an extensive land area. It is thus rarely economical. In the case of a gradually decreasing cross sectional area with reduced flow in the lower reaches of a long canal system, rapid change of water depth is undesirable. Therefore, the B/d ratio should be small in accordance with the reduced canal section.

Considering the above, the B/d ratio was classified by size of canal and in line with the design criteria of NIA.

Base-depth Ratio by Different Discharge

Discharge (m^3/s)	B/d ratio
less than - 4.0	2.0
4.0 - 9.0	2.5

2) Inside slope

To maintain a stable inside canal face, the inside slope is determined by the nature of the soil. The standard slope for different soils differs slightly with the cutting and mounting approach applied. However, as a standard value for sandy clay, the inside slope was decided at 1:1.5.

3) Outside slope

The outside slope was considered in light of dike saturation and stability of the embankment. After the study, the outside slope of the canal was determined at 1:1.5. The following points were given due consideration in planning embankments above 5m:

a) compacted material will be carefully selected;
b) toe drains, etc. for slope drainage will be provided;
c) anti-seepage will be ensured by compaction, etc.; and,
d) outside slope of the embankment will be more gradual than the standard value 1:1.5.

4) Bank top width

In principal, one side of the canal bank is planned as a service road, and the other side as an inspection path. The scale of these roads depends on the design discharge of the canal as is presented below.

Item	Design Discharge (m^3/s)	Service Road (m)	Inspection Path (m)
Main and Lateral Canal	3.0 - 5.0	4.0	2.0
	1.0 - 3.0	4.0	1.5
	0.3 - 1.0	4.0	1.0
Lateral Canal	- 0.3	3.0	1.0

5) Freeboard

Water level in a canal always has the potential to rise above the designed water level. Some of the major reasons are:

a) unexpected roughness coefficient caused by improper construction;

b) structural defects;

c) waving by wind; and

d) increase in rainfall.

Accordingly, freeboard has been designed as follows with due consideration of the above points:

$$Fb = 0.4\ d \qquad (d < 2.0)$$

Fb : Freeboard (m)(should not be below 0.3m)

d : water depth for design discharge (m)

The criteria appears appropriate and in accordance with NIA criteria, such as Design Guides and Criteria for Irrigation Canals, O & M Roads Design Channels & Appurtenant Structures.

(6) Bed Gradient

Bed gradient should be determined on the condition of allowable water velocity as mentioned above. FIG. VII-25 shows the relationship of bed gradient and velocity based on roughness coefficient and canal cross sectional configuration. The figures indicate that the allowable velocity and the economical cross section has a slope range of 1/3,500 above $4.0m^3/s$ and 1/3,000 below $4.0m^3/s$.

There are cases of determining the bed gradient by the condition of canal alignment. The main canal longitudinal profile is shown in VOLUME 4 DRAWINGS.

(7) Others

In principal, all canals will be earth-lined except some part of the Asue Main Canal. A concrete flume canal is proposed for a 300m length north of Sara where the slope is too steep for an earth-lined canal.

5.2.2 Canal Related Structures

(1) Diversion and Measurement Structures

Div^rsion and water measurement structures are designed as double orifice (constant head orifice). Diversion of water above 1.0m^3/s from the main to lateral canal however, will be by a head gate and parshall frume. Diversion from lateral to sub-lateral canals is designed to maintain a fixed rate using a fixed proportional divisor.

(2) Check Gate

Behind the diversion facilities, construction of a check gate is planned to maintain a fixed water level for cases in which the discharge amount is below the designed value. However, depending on the interval of diversion facilities, it may sometimes be unnecessary to use the check gate.

(3) Drop

A drop structure will be constructed to maintain the design bed slope. The vertical drop is a simple structure well suited for small discharge, but not for a large discharge and drop. Therefore, for main canals, the chute type structure will be adopted.

(4) Wasteway and Spillway

In principal, wherever a change occurs in the main canal cross section a wasteway is planned. The wasteway incorporates a spillway to automatically drain overflow water.

(5) Crossing Facilities

An inverted siphon is planned for drainage canals in consideration of the relationship between water level in the canal and probable water level in crossing the drainage canal, and in comparison of the flow amount in the irrigation canal and estimated flood in the drainage canal.

Although irrigation siphons are economical when estimated, flood drainage is noticeably larger than the flow amount in the irrigation canal. When a small drainage canal crosses an irrigation canal, a drainage culvert or drainage culvert pipe is more advantageous.

(6) Road Crossing Facilities

A bridge is planned where the irrigation canal crosses the existing road. As shown in DRAWINGS the general structural design of the bridges to be constructed is concrete slab type.

When the flow amount is small, a road cross pipe will be constructed in light of the economics of this approach.

(7) Combined Structures

According to the basic Irrigation System, main canals will be interlinked with each other in order to exchange irrigation water supply among the same.

A combined structure with sluice gates is proposed at the confluence of main canals.

(8) Over Chute

When drainage flow is relatively small, an overchute to convey drainage water over the irrigation canal is planned rather than a drainage culvert in view of economy.

Numbers of structures discussed above are based on the results of field investigations and are tabulated in TABLE VII-13, VII-14 and VII-15.

5.2.3 Structures for Eastern Area Irrigation

As mentioned in Irrigation Area, the Eastern Area is included within the Project area. Regarding structures to convey water from the Eastern Main Canal to the higher portion at EL.18.0m in the southern part of the San Dionisio-Santol Road, there are three alternative ways; namely, siphon, pump and elevated flume.

Comparison of cost for the alternatives was carried out and is tabulated in TABLE VII-16 illustrated in FIG. VII-26. Based on the results of the same, the siphon is proposed for irrigation of the Eastern Area.

5.2.4 Structures for Irrigation along the Trans-diversion Canal

Paddy fields along the proposed Trans-diversion Canal (100ha) are included within the Project area. Installation of one outlet gate was proposed to supply water for the area upstream from the tunnel inlet EL 83.

Paddy fields between the tunnel outlet and the Asue Diversion Dam (90ha) can not be irrigated due to the location. An outlet gate is also proposed for the area downstream of the canal route power station at EL 42.5.

Regarding irrigation canals for both areas, main farm ditches to be completed as on-farm facilities will be utilized for the same. This is the same method used as that where irrigation water is directly supplied to the main canal without lateral or sub-lateral canals.

5.2.5 Asue River Course Improvement

As a result of reservoir operation study, the maximum water to be conveyed through the trans-diversion tunnel after irrigation water supply to the 190ha area, is $5.8m^3/s$ from the Catipayan River to the Asue River. At the south of the canal route power station, water is released into the upper stream of the Asue River to intake water at the Asue Diversion Dam.

For the existing river course from the release point to the diversion dam, the discharge capacities of the cross-sections were checked to determine whether the same have sufficient capacity for not only drainage discharge but also $5.8m^3/s$ discharge into the Asue River. It was determined that 650m of the river course requires improvement to allow a discharge of $5.8m^3/s$. Concrete lining is proposed for this portion of the river because of the steep riverbed.

5.3 Drainage System

5.3.1 New Drainage Canal

(1) Canal Alignment

After the completion of the Project, irrigation water supply will be sufficient throughout the Project area, even in those portions presently unirrigated. Smooth drainage of surplus irrigation water as well as flood water is also required. New drainage canals are planned for construction in the unirrigated portions, and in areas where drainage canal systems are insufficient.

Locations of the newly proposed drainage canals are shown in FIG. VII-10, and canal lengths are tabulated below.

LENGTH OF NEW DRAINAGE CANALS

Canal	Length (m)
Dr - 1	500
Dr - 2	2,000
Dr - 3	500
Dr - 4	1,700
Dr - 5	2,400
Dr - 6	1,400
Dr - 7	1,500
Dr - 8	1,400
Dr - 9	2,000
Dr - 10	1,300
Dr - 11	1,500
Dr - 12	1,600
Dr - 13 - 1	1,500
Dr - 13 - 2	600
Dr - 14	1,600
Total	21,500

5.3.2 Design Criteria

(1) Mean Velocity Formula

Mean velocity was determined by adoption of the Manning Formula with a roughness coefficient "n" of 0.03.

(2) Allowable Velocity

Allowable velocity was determined according to Kennedy's Formula which considers scouring as shown on the following page.

$$Va = 0.546 \cdot D^{0.64}$$

Va : allowable velocity (m/s)

D : water depth (m)

For the drainage canal design, the maximum Va was fixed at 1.0m/s.

(3) Canal Cross Section

Criteria for the drainage canal cross section are given below:

1) Bed-depth ratio

Although a large bed-depth ratio helps stabilize the discharge flow, it is not usually economically viable due to increased construction work and land requirements. Accordingly the trapezoid cross section, which has maximum hydraulic radius per unit cross-sectional area, was adopted as the most effective drainage system and the following formula was used.

$$B = 2H\ (\ 1 + m^2 - m)$$

B : bed width (m)

H : water depth (m)

m : side slope

When m = 1.0, B is 0.828H.

2) Drainage canal side slope

The side slope of the drainage canal should be stable to prevent landsliding and scouring. Generally, the side slope is determined by the soil conditions. As soils in the Project area are mainly composed of clay and sandy-loam, the side slope has been designated as 1:1.

3) Freeboard

Design criteria for freeboard corresponds to the following formula used for designing irrigation canals:

$$Fb = 0.25\ H + 0.3 \qquad (H \geqq 2.0m)$$
$$= 0.4\ H \qquad (H < 2.0m)$$

Fb : freeboard (m)

H : water depth (m)

(4) Design Drainage Discharge

The capacity of each drainage canal is calculated with a drainage area multiplied by the following unit area drainage, which is a 10-year return period discharge analysed by the Kinematic water method.

Unit: ℓ/sec/ha

Description	Unit Discharge
Paddy runoff	5.0
Hilly area runoff	22-14.0

5.3.3 Improvement of Existing Drainage Structures

Six drainage culvert road crossings are proposed to resolve the drainage problem. Locations of the structures are shown in FIG. VII-10.

The capacity of each structure was decided using a 25-year return period analysed by the Kinematic Wave method.

5.3.4 Excavation of Existing Creeks and Rivers

During the field survey chronic flooding caused by shortage in drainage capacity was identified in some creeks and rivers. Rehabilitation of the same is proposed to meet the design criteria discussed in DESIGN CRITERIA, with emphasis on the following points:

a) expansion of narrow sections; and,

b) improvement of extreme meandering.

The creeks and rivers to be excavated are tabulated below.

Creeks, River	Length (m)
Hasohoy River	2,500
Padios Creek	1,500
San Dionisio Creek	2,000
Total	6,000

The design drainage discharge to calculate the capacity of each creek or river is the 10-year return period discharge mentioned in New Drainage Canal.

5.3.5 Others

The paddy fields along the upstream Asue River are sometimes inundated due to privately owned concrete weirs. Removal of the weirs is proposed to prevent backwater.

5.4 On-Farm Facilities

5.4.1 Typical Plan for On-farm Facilities

Based upon the plan mentioned in ON-FARM DEVELOPMENT PLAN, irrigation water will be supplied to each rotation block composed of 5 rotation units of 10ha. The typical layout of on-farm facilities is illustrated as shown in FIG. VII-27.

5.4.2 On-farm Facilities

Based on the typical on-farm layout discussed above, the following facilities are proposed to introduce modern water management and farming practices to the farmers concerned.

1) Turnout

For diversion to main farm ditches from lateral canals or directly from the main canal, a double orifice turnout is proposed. Service area per turnout is generally 50ha, while design capacity of the main farm ditch derived form the turn-out was calculated at 2.242ℓ/s/ha as discussed in DESIGN WATER REQUIREMENTS.

Proposed orifice size is 0.60 XO. 40m with ϕ18" RC PIPE.

2) Main farm ditches (MFD)

Main farm ditches (MFD) are generally designed parallel to lateral canals. Assuming adoption of simultaneous irrigation, the longitudinal canal section is designed to maintain a minimum water level of more than 10cm above land surface during the crop maintenance stage.

The top bank elevation will be determined according to the maximum water level during land preparation with an additional 20cm of freeboard. This will allow distribution of water of supplementary farm ditches. The side slope and bank top width is designed at 1:1.0 and 0.4m, respectively while planned minimum and maximum allowable velocity will be 0.2m/s and 0.65m/s. The canal bed slope was decided in consideration of the minimum and maximum allowable velocity, and design cross section determined by the Manning method with a roughness coefficient of 0.03.

3) Supplementary farm ditches (SFD)

Supplementary farm ditches deliver water to farm plots from the MFD. Moreover, SFD will function as farm drains owing to the steep topographic condition of the Project Area. As much as possible, farm ditches are designed to follow existing property lines and tenant boundaries.

Design water level for the SFD is the same as that for the MFD. Top bank elevation was determined with 15cm freeboard according to the maximum expected water level not only obtained by the design water requirement with assumed adoption of the rotational system but also estimated for drainage in flood periods. The side slope and bank top width are 1:1.0 and 0.35m, respectively. Minimum and maximum allowable velocity is the same as that for the M.F.D..

4) Farm drains

As mentioned above, the SFD will be utilized as a dual purpose farm drain. In addition, new farm drains are proposed in the portion which can not be drained smoothly by the proposed SFD alone.

The sizes of these farm drains are the same as the SFD.

5) Division boxes

Division boxes are planned at the origin of each SFD. The division box consists of 2 reinforced concrete leaf wall type checks to be installed in both sides of the SFD and MFD.

6) End checks

End checks will be installed at the tail end of each SFD to prevent runoff water from entering the drainage canal. Design structure is reinforced concrete leaf wall.

5.4.3 On-farm Facilities in the Sample Area

Two sample areas were selected in the Project Area to formulate proposed on-farm facilities, and to estimate cost for on-farm development. One site was selected as a representative flat area, Sample A; the other site was a hilly area, sample B. Present and proposed land use in the sample aeas are presented in the table below.

(Unit: ha)

Item	Sample A		Sample B	
	Present	Proposed	Present	Proposed
Paddy Field	143.1	144.0	85.4	82.0
Sugarcane	3.1	0.5	15.2	14.6
Coconut	3.7	3.0	7.7	7.2
Bamboo	3.9	1.9	-	-
Grass	-	-	6.8	6.4
Residential Area	2.5	2.5	2.3	2.3
Road (Existing)	0.5	0.5	2.0	2.0
Canal(- do -)	1.5	1.5	0.6	0.6
Main, Lateral Canal	-	2.2	-	2.5
Farm Ditch	-	1.5	-	2.2
Farm Drain	-	0.2	-	-
Road, ICC etc.	-	0.5	-	0.2
River, Creek	1.2	1.2	2.6	2.6
Total	159.5	159.5	122.6	122.6

Land parcelling and designing for on-farm facilities were conducted on the 1:4,000 topographical map illustated as shown in FIG. VII-28. Rotational area, required on-farm facilities and proposed length of on-farm facilities for the sample areas are summarized in TABLE VII-17, VII-18 and VII-19, respectively.

MAIN FEATURES OF SERRUCO COMMUNAL IRRIGATION SYSTEM

1. Completion Date	Nov. 31, 1980
2. Turn-over Date	May 7, 1981
3. No. of Members	65 farmers
4. Area	700ha
5. Source of Water Supply	Serruco River
6. Diversion Works	
a) Serruco Diversion Dam	
- Type	Fixed weir and stop-log gate (B=1.5 m x 2)
- Height	3.0 m
- Length	30.0 m
b) Design discharge capacity	99.04 cu. m/s
7. Main canal	2 nos. - 8,475 m
8. Lateral canal	4 nos. - 7,738 m
9. Canal Structures	
a) Head gates	3 nos.
b) Drops	1 nos.
c) Road/Thresher Crossing	12 nos.
d) Siphon	6 nos.
e) Inlet and Outlet	4 nos.
f) Check	Combined with other structures
g) End Check	6 nos.
h) Drainage Siphon	4 nos.

TABLE VII-2

KABSAKA WATER IMPOUNDING COMPONENT PROJECTS

Project/Location	Length of Dam (m)	Top Width (m)	Height (m)	Service Area (m)	Water-shed Area (ha)	Pond Area (ha)	Storage Capacity (cu.m)	No. of Benefi-ciaries	Average Farm Size (ha)	B/C Ratio	I R R (%)
6 Finished WIP's in 1982											
1. Aglosong WIP, Concepcion, Iloilo	82	6.0	10.0	50	67	6.5	425,125	30	1.6	1.96	22.0%
2. Ajuy WIP, San Dionisio, Iloilo	146	5.0	10.0	47.20	48	1.46	56,500	25	1.72	1.68	15.5%
3. Castor WIP, Sara, Iloilo	122	5.0	10.0	32	30	1.10	38,550	16	2.0	1.36	11.0%
4. Moto WIP, San Dionisio, Iloilo	148	5.0	10.0	50	25	1.20	40,500	25	2.0	1.30	11.5%
5. PILI-I WIP, Ajuy, Iloilo	154	5.0	10.0	47	58	1.82	71,416	25	2.0	1.58	13.6%
6. Porolan, WIP, Dingle, Iloilo	126	5.0	10.0	50	43.3	1.48	60,500	47	1.06	1.41	12.0%
10 Finished WIP's in 1983											
1. Hda. Conchite WIP San Dionisio, Iloilo	156	5.0	10.0	50	48.0	2.25	112,600	27	1.85	1.81	16.2%
2. Iprog WIP, San Enrique	118	5.0	10.0	50	73.56	0.95	33,370	15	1.70	1.32	10.2%
3. Madarag WIP, San Enrique	55	5.0	10.0	40	47	1.0	40,000	25	1.60	1.26	10.5%
4. Buwang WIP, Lambunao	70	5.0	10.0	30	25	1.5	45,000	16	1.70	1.32	11.0%
5. Zaragpza WIP, Balasan	182	6.0	8.0	50	204	9.5	525,000	15	2.14	1.89	18.5%
6. Bondolan WIP San Dionisio, Iloilo	110	5.0	10.0	50	25	2.7	100,000	20	2.5	1.45	13.0%
7. Sta. Ana WIP, Estancia	98	5.0	10.0	24	18	2.0	70,000	15	1.5	1.36	11.5%
8. Barasan WIP, Leon	65	4.5	7.0	20	50	1.0	21,200	15	1.3	1.29	10.5%
9. Capangyan WIP, Lambunao	110	5.0	10.0	35	70	2.0	74,000	18	1.8	1.24	10.5%
10. Belen (Calamigan) WIP, Concepcion											
5 Deferred Indefinitely											
1. Bacabac WIP, Sara	154	5.0	10.0	50	70	2.50	74,500	16	3.12	1.45	13.0%
2. Pamoringao WIP, Cabatuan	150	5.0	10.0	38	34.5	1.10	37,125	30	1.25	1.37	11.0%
3. Santiago WIP, Cabatuan Biejo, Iloilo	170	5.0	10.0	30	70	1.75	135,000	14	1.80	1.36	12.0%
4. Doming, SAra	70	5.0	10.0	30	170	1.10	47,500	18	1.70	1.32	10.2%
5. Apelo Small Diversion Dam, Sara											

Advise of the World Bank representative in a meeting with the Ministry of Agriculture key personnel

KABSAKA WIP IN THE PROJECT AREA

NAME OF PROJECT	COORDINATES LONGITUDINAL	COORDINATES LATITUDINAL	DAM STORAGE CAPACITY (C.M.)	DAM LENGTH (M.)	DAM TOP WIDTH (M.)	DAM HEIGHT (M.)	WATER-SHED AREA (ha)	DAM POND AREA (ha.)	NO. OF BENEFI-CIARIES	AVE. FARM SIZE (ha.)	SERVICE AREA (ha.)	SOURCE OF WATER	BRGY./MUNICI-PALITY COVERED
1. AGLOSONG	123°-04'-10"	11°-14'-10"	525,125	82.00	6.00	10.00	67.00	6.50	30.00	1.60	50.00	Agsalong Springs	Brgy. Agsalong Concepcion
2. CASTOR	123°-02'-02"	11°-18'-52"	38,550	122.00	5.00	10.00	30.00	1.10	16.00	2.00	32.00	Tabagay Streams	Brgy. Castor, Sn. Dionisio
3. MOTO	123°-03'-54"	11°-16'-56"	40,500	148.00	5.00	10.00	25.00	1.20	25.00	2.00	50.00	Hasohoy River Tribs.	Brgy. Moto, Sn. Dionisio
4. BONDOLAN	123°-04'-15"	11°-17'-08"	100,000	110.00	5.00	10.00	25.00	2.70	20.00	2.50	50.00	Hasohoy River Tribs.	Brgy. Bondolan Sn. Dionisio
5. BELEN (Cala-migan)	123°-04'-26"	11°-13'-33'	61,143	77.00	5.00	10.00	75.00	3.53	31.00	1.70	50.00	Belen Springs	Brgy. Belen, Concepcion
6. DOMINGO	122°-59'-01"	11°-14'-00"	47,500	70.00	5.00	10.00	170.00	1.10	18.00	1.70	30.00		Brgy. Domingo Sara
7. APELO	122°-54'-00"	11°-14'-50"	no data	no data	no data	no data	no data	no data	no data	no data		Marimbon Crk. Tibs.	Brgy. Apelo, Sara
8. BACABAC			74,500	154.00	5.00	10.00	70.00	2.50	16.00	3.12	50.00		Brgy. Bacabac, Sara
TOTAL											342.00		

Note: * Marked projects are deferred indefinitely.

PRIVATE IMPOUNDINGS IN THE PROJECT AREA

NAME OF IMPOUNDING	YEAR CONSTRUCTED	DAM STORAGE CAPACITY (CM.)	DAM HEIGHT (M.)	DAM TOP LENGTH (M.)	SERVICE AREA (HA.)	R E M A R K S
1. SANSON	1962	no data	5.0	150.0	7.0	without spillway
2. PACIG	1960	no data	3.0	40.0	12.0	spillway for intake
3. SALCEDO	1970	no data	3.0	100.0	9.0	with spillway(B=3.5)
TOTAL					28.0	

TABLE VII-5

EXISTING WEIRS IN THE PROJECT AREA

1/ NO.	YEAR CONSTRUCTED	LOCATION	TYPE OF STRUCTURE	CONDITION	REMARKS
1	1950	Asue River	Stop-log type B=1.5	operative	private
2	-	"	1 barrel gate B=1.2	"	"
3	1952	"	3 barrel gates B=1.1, 1.3, 1.1	washed out in 1982	"
4	-	"	fixed weir	washed out in March 1984,under repair	"
5	1938	"	2 barrel gates B=1.0, 1.0	operative	"
6	-	"	Stop-log type B=1.8	"	"
7	-	"	Stop-log type B=1.5	"	"
8	1938	"	Stop-log type B=1.5	"	"
9	-	"	Stop-log type B=1.5	"	"
10	1962	Pasaka Stream	Stop-log type B=2.0 x 4	"	"
11	1978	Gubaton River	USBR criteria type	washed out in 1981	constructed by FSDC
12	-	Pasaka Stream	fixed weir	washed out	private

1/ refer to the location map of existing structures

RESULTS OF PERCOLATION TEST

Test Site No.1/	Percolation Rate	Measured	Textural Class2/
1	0.2	JICA	C
2	0.5	-do-	SCL
3	1.7	-do-	SL
4	0.1	-do-	C
5	1.0	-do-	C
6	0.1	-do-	SCL
7	1.3	-do-	SCL
8	1.0	-do-	SCL
9	0.3	-do-	SCL
10	1.2	-do-	SCL
11	0.1	-do-	SCL
12	1.5	-do-	SCL
13	1.8	-do-	SCL
14	1.8	-do-	SCL
15	1.6	-do-	SL
16	1.9	-do-	SCL
17	0.2	-do-	SCL
18	0.7	-do-	SCL
19	0.1	-do-	SCL
20	0.5	-do-	SCL
21	0.5	NIA	SCL
22	0.6	-do-	SCL
23	0.9	-do-	SCL
24	0.8	-do-	SCL
25	1.10	-do-	SL
26	0.5	-do-	SCL
27	0.6	-do-	C
28	0.7	-do-	SCL
29	0.8	-do-	SCL

1/ Refer to FIG. VII-8
2/ C: Clay, SCL: Sandy Clay Loam, SL: Sandy Loam

ANNUAL MAXIMUM EVAPORATION FOR VARIOUS DURATIONS

Unit: mm

Duration (Days)	Year								
	1979	1980	1981	1982	1983	1984	Max.	mm/day	ET(x0.8)
1	10.1	7.6	9.6	9.8	10.5	7.3	10.5	10.5	8.4
2	19.5	15.2	18.8	15.2	19.1	13.3	19.5	9.8	7.8
3	28.7	22.8	24.4	21.6	27.3	13.9	28.7	9.6	7.7
4	37.8	30.4	29.2	27.3	33.1	14.3	37.8	9.4	7.5
5	46.8	38.0	36.3	33.5	33.6	15.7	46.8	9.4	7.5
6	55.7	45.6	43.5	39.2	38.8	16.7	55.7	9.3	7.4
7	64.5	53.2	50.1	43.7	40.8	17.3	64.5	9.2	7.4
8	73.7	60.8	56.5	47.5	48.5	17.7	73.7	9.2	7.4
9	82.8	68.4	62.2	51.5	49.5	19.1	82.8	9.2	7.4
10	91.3	76.0	68.2	56.7	50.3	19.5	91.3	9.2	7.3
15	129.7	110.8	93.9	85.1	66.6	25.0	129.7	8.6	6.7
20	173.5	148.8	115.7	111.2	87.9	33.7	173.5	8.7	7.0
30	243.6	196.8	174.6	164.1	126.6	41.6	243.6	8.1	6.5
40	307.8	237.0	227.4	219.6	159.2	50.8	307.8	7.7	6.2
50	378.1	293.4	271.3	263.8	192.9	66.8	378.1	7.6	6.1
60	440.6	354.1	310.2	317.7	207.5	77.5	440.6	7.3	5.8

TABLE VII-8

CALCULATION OF LAND SOAKING REQUIREMENTS

Days (n)	(1) Land Soaking Requirements	(2) Areal rate per day (1/40)	(3) Land Soaking Requirement with Rotational Irrigation (mm/day) (1) x (2)	(4) Maintenance Water Duty	(5) Areal Factor $\frac{n-1}{40}$	(6) Maintenance Water with Rotational Irrigation (mm/day) (4) x (5)	(7) Total Water Requirement per day (mm/day) (3) + (6)
Land Soaking							
1	107.5	0.025	2.69	-	-	-	2.69
2	107.5	0.025	2.69	7.6	0.025	0.19	2.88
3	107.5	0.025	2.69	7.6	0.050	0.38	2.88
4	107.5	0.025	2.69	7.6	0.075	0.57	3.26
5	107.5	0.025	2.69	7.6	0.100	0.76	3.45
6	107.5	0.025	2.69	7.6	0.125	0.95	3.64
7	107.5	0.025	2.69	7.6	0.150	0.14	3.83
.	.	.	.	.	.	.	.
.	.	.	.	.	.	.	.
.	.	.	.	.	.	.	.
.	107.5	0.025	2.69	7.6	.	.	.
.	.	.	.	.	.	.	.
.	.	.	.	.	.	.	.
.	.	.	.	.	.	.	.
35	107.5	0.025	2.69	7.6	0.850	6.46	9.15
36	107.5	0.025	2.69	7.6	0.875	6.65	9.34
37	107.5	0.025	2.69	7.6	0.900	6.84	9.53
38	107.5	0.025	2.69	7.6	0.925	7.03	9.72
39	107.5	0.025	2.69	7.6	0.950	7.22	9.91
40	107.5	0.025	2.69	7.6	0.975	7.41	10.1
Growing Stage							
41	-	-	-	7.6	1.00	7.6	7.6
42	-	-	-	7.6	1.00	7.6	7.6
43	-	-	-	7.6	1.00	7.6	7.6
44	-	-	-	7.6	1.00	7.6	7.6

UNIT DESIGN WATER REQUIREMENTS

Items	Tertiary & S.F.D.		Secondary Canal		Main Canal	
	1st Crop	2nd Crop	1st Crop	2nd Crop	1st Crop	2nd Crop
Land Soaking Capacity						
Land Soaking (mm)	87.5	75.0	87.5	75.0	87.5	75.0
Standing Water (mm)	20.0	20.0	20.0	20.0	20.0	20.0
(1) Subtotal (mm)	107.5	95.0	107.5	95.0	107.5	95.0
(2) Daily Peak (mm/day)	2.7	2.4	2.7	2.4	2.7	2.4
(2) = (1)/40 day						
Maintenance Water						
Consumptive Use (mm/day)	8.4	8.4	7.0	7.0	6.1	6.1
Percolation Rate (mm/day)	1.5	1.5	1.5	1.5	1.5	1.5
(3) Subtotal	9.9	9.9	8.5	8.5	7.6	7.6
(4) Daily Peak (mm/day)	9.7	9.7	8.3	8.3	7.4	7.4
(4) = (3) x 39/40						
(5) Total Water Requirements						
(Net Water Requirements)	12.4	12.1	11.0	10.7	10.1	9.8
(5) = (2) + (4) (mm/day)						
(6) = (5)/8.64 (ℓ/s/ha)	1.435	1.400	1.273	1.238	1.169	1.134
(7) Irrigation Efficiency (%)	64	64	60	60	57	57
(8) Unit Design Water Requirement	<u>2.242</u>	2.188	<u>2.122</u>	2.063	<u>2.051</u>	1.989
(8) = (6)/(7) (ℓ/s/ha)						

Note: The figures underlined are used for the unit design water requirement.
For the safety of the canal capacity, no effective rainfall is considered.

POTENTIAL IRRIGABLE AREA

Land Use	Gross Area: Total Area	Gross Area: Within the Project Boundary 4/	Potential Service Area: Irrigation Area	Potential Service Area: Enriched Area	Potential Service Area: Sub-total	Net Area: Irrigation Area: Net Irrigable	Net Area: Irrigation Area: Right of Way	Net Area: Enriched Area: Net Irrigable	Net Area: Enriched Area: Right of Way	Net Area: Total: Net Irrigable	Net Area: Total: Right of Way
A. Present Paddy Field	6,635 1/	6415	5900	420	6320	5600	300	400	20	6000	320
1. Irrigated Area	2,110	1590	1170	420	1590	1110	60	400	20	1510	80
1) Serruco CIS	700 1/	400	22	378	400	20	2	360	18	380	20
2) KABSAKA WIP(8 dams)	342 2/	132	90	42	132	85	5	40	2	125	7
3) Private Water Imp.(3 dams)	28 3/	18	18	-	18	15	3	-	-	15	3
4) Private Weir(12 weirs)	460	460	460	-	460	440	20	-	-	440	20
5) Private Pump	580	580	580	-	580	550	30	-	-	550	30
2. Non-Irrigated	4825 1/	4825	4730	-	4730	4490	240	-	-	4490	240
1) Asue & Eastern Area	4720 1/	4720	4625	-	4625	4390	235	-	-	4390	235
2) Trans-diversion Area	105	105	105	-	105	100	5	-	-	100	5
B. Convertible Area	1255	1255	800	-	800	760	40	-	-	760	40
1. Sugarcane	755	755	380	-	380	360	20	-	-	360	20
1) Asue & Eastern Area	690	690	380	-	380	360	20	-	-	360	20
2) Trans-diversion Area	65	65	-	-	-	-	-	-	-	-	-
2. Coconuts	240	240	200	-	200	190	10	-	-	190	10
3. Grassland	260	260	220	-	220	210	10	-	-	210	10
TOTAL	7,890	7670	6700	420	7120	6360	340	400	20	6760	360

Note: 1/ Serruco CIS presently irrigates only 400ha but the original plan is 700ha. The 300ha is at present rainfed area and double counted in the non-irrigated area, as the total present paddy is adjusted in the total area.

Note: 2/ 5 completed water impounding projects and 3 planned projects, as shown in TABLE VII-3

Note: 3/ See TABLE VII-4

Note: 4/ Present land use, see TABLE I-10

Note: 5/ The upper portion of Serruco CIS and KABSAKA cannot be included as irrigable area due to the high elevation but can be included in the benefit area.

CHARACTERISTICS OF THE ASUE RIVER SUB-BASIN

Sub-Basin No.	Total Area (ha)	Cumulative Total Area (ha)	Hilly Area (ha)	Cumulative Hilly Area (ha)	Paddy Area (ha)	Cumulative Paddy Area (ha)	Length of Catchment (m)	Slope (I:S)
110	1370		1172		198		6000	
(L)	780	780	680	680	100	100		30
(R)	590	1370	492	1172	98	198		35
1	1370		1172		198			
120	1275		595		680		7360	
(L)	885	2255	520	1692	365	563		30
(R)	390	2645	75	1767	315	878		35
2	2645		1767		878			
130	378		242		136		2260	
(L)	52	52	198	198	128	128		60
(R)	326	378	44	242	8	136		70
140	468		377		91		4820	
(L)	207	585	199	420	29	165		40
(R)	261	846	619	619	62	227		35
3	846		619		227			
150	84		9		75			
(L)	50	896	1	627	42	269	2580	140
(R)	34	930	8	628	33	302		300
4	930		628		302			
5	3575		2395		1180			
160	60		4		56			
(L)	13	3588	0	2395	9	1189	470	—
(R)	47	3635	4	2399	47	1236		300
6	3635		2399		1236			
170	1231		854		377		6690	
(L)	961	961	749	749	212	212		30
(R)	270	1231	105	854	165	377		50
180	275		153		122		2400	
(L)	138	1369	89	89	49	426		40
(R)	137	1506	64	153	73	499		60
7	1506		153		499			
190	298		177		121		3260	
(L)	116	1622	26	1033	90	589		160
(R)	182	1804	151	1184	31	620		40

CHARACTERISTICS OF THE SUB-BASIN OF THE ASUE RIVER

Sub-Basin No.	Total Area (ha)	Cumulative Total Area (ha)	Hilly Area (ha)	Cumulative Hilly Area (ha)	Paddy Area (ha)	Cumulative Paddy Area (ha)	Length of Catchment (m)	Slope (I:S)
8	1804		1184		620			
9	5439		3583		1856			
200	184		10		174		2020	
(L)	57	5496	0	3583	57	1913		
(R)	127	5623	10	3593	117	2030		220
10	5623		3593		2030			
210	195		68		127		1960	
(L)	107	107	49	49	58	58		40
(R)	88	195	19	68	69	69		140
220	325		225		100		2600	
(L)	157	352	132	200	25	152		55
(R)	168	520	93	293	75	227		80
11	520		293		227			
230	410		40		370		1220	
(L)	254	774	34	327	220	447		115
(R)	156	930	6	333	150	597		300
12	930		333		597			
240	480		176		304		3350	
(L)	286	1216	166	499	120	717		35
(R)	194	1410	10	509	184	901		300
13	1410		509		901			
14	7033		4102		2931			
250	33		14		19		580	
(L)	10	7043	0	4102	10	2941		
(R)	23	7066	14	4116	9	2950		35
15	7066		4116		2950			
260	2290		2255		35		5840	
(L)	1350	1350	1333	1333	17	17		25
(R)	940	2290	922	2255	18	35		25
16	2290		2255		35			
270	975		377		598		5735	
(L)	587	2877	254	2509	333	368		25
(R)	388	3265	123	2632	265	633		25

CHARACTERISTICS OF THE SUB-BASIN OF THE ASUE RIVER

Sub-Basin No.	Total Area (ha)	Cumulative Total Area (ha)	Hilly Area (ha)	Cumulative Hilly Area (ha)	Paddy Area (ha)	Cumulative Paddy Area (ha)	Length of Catchment (m)	Slope (I:S)
280	791		549		242		5840	
(L)	562	3827	430	3062	132	765		35
(R)	229	4056	119	3181	110	875		40
17	4056		3181		875			
290	480		161		319		5330	
(L)	299	4355	155	3336	144	1019		60
(R)	181	4536	6	3342	175	1194		300
18	4536		3342		1194			
19	11602		7458		4144			
300	339		34		305		42	
(L)	109	11711	15	7473	94	4238		120
(R)	230	11941	19	7492	211	4449		180
20	11941		7492		4449			
310	1054		527		527			
(L)	869	12810	487	7979	382	4831	2810	35
(R)	185	12995	40	8019	145	4976		75
21	12995		8019		4976			
320	77		21		56		870	
(L)	25	13020	0	8019	25	5001		—
(R)	52	13072	21	8040	31	5032		60
22	13072		8040		5032			
330	660		393		267			
(L)	360	13432	232	8272	128	5160		30
(R)	300	13732	161	8433	139	5299		35
23	13732		8433		5299			

Characteristics of Sub-basin in Pasaka Creek

Sub-Basin No.	Total Area (ha)	Cumulative Total Area (ha)	Hilly Area (ha)	Cumulative Hilly Area (ha)	Paddy Area (ha)	Cumulative Paddy Area (ha)	Length of Catchment (m)	Slope (I:S)
340	1005		94		911		7220	
(L)	365	365	36	36	329	329		170
(R)	640	1005	58	94	582	911		135
24	1005		94		911			

CHARACTERISTICS OF THE SUB-BASIN OF THE GUBATON RIVER

Sub-Basin No.	Total Area (ha)	Cumulative Total Area (ha)	Hilly Area (ha)	Cumulative Hilly Area (ha)	Paddy Area (ha)	Cumulative Paddy Area (ha)	Length of Catchment (m)	Slope (I:S)
350	1880		1775		105		11060	
(L)	995	995	920	920	75	75		35
(R)	885	1880	855	1775	30	105		40
25	1880		1775		105			
360	650		134		516		4640	
(L)	260	2140	24	1799	236	341		30
(R)	390	2530	110	1909	280	621		35
26	2530		1909		621			
Characteristics of Sub-basin in Tabagay River								
370	1174		907		267		6110	
(L)	538	538	440	440	98	98		25
(R)	636	1174	467	907	169	267		30
380	1290		505		785		2060	
(L)	915	2089	282	1189	633	900		65
(R)	375	2464	223	1412	152	1052		30
27	2464		1412		1502			
390	580		274		306		2210	
(L)	345	2890	102	1514	243	1295		50
(R)	235	3044	172	1686	63	1358		25
28	3044		1686		1358			
Characteristics of Sub-basin in the San Dionisio Creek								
400	253		93		160		2560	
(L)	125	125	45	45	80	80		35
(R)	253	253	48	93	80	160		55
410	1245		604		641		5200	
(L)	832	1085	473	566	359	519		35

CHARACTERISTICS OF THE SUB-BASIN OF SAN DIONISIO CREEK

Sub-Basin No.	Total Area (ha)	Cumulative Total Area (ha)	Hilly Area (ha)	Cumulative Hilly Area (ha)	Paddy Area (ha)	Cumulative Paddy Area (ha)	Length of Catchment (m)	Slope (I:S)
(R)	413	1498	131	697	282	801		55
31	1498		697		801			

PROPOSED LENGTH OF IRRIGATION CANAL

Main Canal		Lateral Canal	
Canal	Length	Canal	Length
	(m)		(m)
A - 1	4,640	A-L1	3,910
A - 2	3,740	A-L2	7,140
A - 3	4,710	A-L3	2,420
Subtotal	13,090	A-L4	1,170
E - 1	2,030	A-L5	1,380
E - 2	4,130	A-L6	3,070
E - 3	1,120	Subtotal	19,090
Subtotal	7,280	E-L1	7,160
BR - 1	1,850	E-L1-1L	1,170
BR - 2	2,510	E-L2	8,000
BL	1,420	E-L2-1L	870
Subtotal	5,780	E-L2-2L	6,000
SR	(2,780)	E-L2-1R	870
SL	(2,100)	E-L3	2,690
Subtotal	4,880	E-L3-1L	2,690
G	2,600	E-L4	2,530
D1	(1,940)	Subtotal	31,980
D2	1,120	BR-L1	2,570
Total	29,870	BR-L2	750
	(36,690)	BR-L3	380
		BL-L1	1,030
		BL-L2	2,700
		Subtotal	7,430
		SR-L1	(1,350)
		SR-L2	(820)
		SL-L1	(1,890)
		Subtotal	4,060
		G-L1	860
		G-L2	1,050
		G-L3	2,600
		Subtotal	4,510
		D-L1	(5,230)
		D-L1-1R	1,860
		Subtotal	7,090
		Total	64,870
			(74,160)

Note: A (Asue Main Canal), E (Eastern Main Canal), BR (Bakabak Right Main Canal), BL (Bakabak Left Main Canal), SR (Serruco Right Main Canal), SL (Serruo Left Main Canal), G (Gubaton Main Canal), D1 (Diversion Canal: 1), D2 (Diversion Canal: 2), (   ) = Existing Canal

NUMBERS OF CANAL RELATED STRUCTURES

Structures	Main Canal	Lateral	Total
1. Head Gate and Parshall Flume	6	-	6
2. Double Orifice	31	129	160
3. Check	25	115	140
4. Siphon	8	3	11
5. Drainage Culvert	8	8	16
6. Drainage Culvert Pipe	8	21	29
7. Bridge	18	3	21
8. Road Crossing	4	57	61
9. Wasteway	8	-	8
10. Chute	6	-	6
11. Vertical Drop	-	117	117
12. Fixed Proportional Divisor	1 1/	6	7
13. Combined Structure	3	-	3
14. Overchute	44	21	65
Total	170	480	650

Note: 1/ Proportional divisor with gate

NUMBERS OF STRUCTURES: MAIN CANAL

Name	Head Gate	Double Orifice	Check	Siphon	Drainage Culvert	Drainage C.P.	Bridge	Road Crossing	Waste-way	Chute	Fixed P.Div.	Comb. Stru.	Over-chute
A-1	1	3	3	-	1	2	-	1	1	-	-	-	9
A-2	1	1	2	1	1	2	4	1	1	-	-	-	7
A-3	-	6	6	2	1	-	5	-	-	3	-	-	7
subtotal	2	10	11	3	3	4	9	2	2	3	0	0	23
E-1	1	1	1	-	1	1	1	-	1	-	-	-	4
E-2	1	1	2	-	3	2	1	-	1	1	-	-	8
E-3	1	2	2	1	-	1	1	-	1	-	-	-	2
subtotal	3	4	5	1	4	4	3	0	3	1	0	0	14
BR-1	-	2	1	1	-	-	1	-	1	-	-	-	-
BR-2	-	3	3	-	-	-	1	-	1	1	-	-	-
BL	-	4	2	-	1	-	1	-	1	1	-	-	-
subtotal	0	9	6	1	1	0	3	0	3	2	0	0	0
SR	-	1	1	-	-	-	-	-	-	-	-	-	2
SL	-	2	1	1	-	-	-	-	-	-	-	-	-
subtotal	0	3	2	1	0	0	0	0	0	0	0	0	2
G	0	4	0	1	0	0	2	1	0	0	1	1	5
D1	1	1	1	0	0	0	1	0	0	0	0	1	0
D2	0	0	0	1	0	0	0	1	0	0	0	1	0
Total	6	31	25	8	8	8	18	4	8	6	1	3	44

NUMBERS OF STRUCTURES: LATERAL CANAL

Name	Double Orifice	Check	Siphon	Drainage Culvert	Drainage C.P.	Bridge	Road Cross.	Verti. Drop	Fixed P.Div.	Over-chute
A-L1	8	6	-	-	1	-	4	6	-	-
A-L2	13	12	2	2	-	-	10	11	-	-
A-L3	6	6	-	-	-	-	5	2	-	-
A-L4	2	2	-	-	-	-	1	2	-	2
A-L5	2	2	-	-	2	-	-	3	-	-
A-L6	3	3	1	-	-	1	2	7	1	4
Subtotal	34	31	3	2	3	1	22	31	1	8
E-L1	13	13	-	1	-	-	4	10	1	-
E-11-1L	2	2	-	-	-	-	-	2	-	-
E-L2	10	8	-	-	5	-	3	14	3	-
E-L2-1L	1	1	-	-	-	-	2	2	-	-
E-L2-2L	7	7	-	-	3	-	5	12	-	6
E-L2-1R	1	1	-	-	-	-	2	2	-	-
E-L3-	8	7	-	-	1	-	1	2	1	-
E-L3-1L	4	4	-	1	-	1	1	5	-	-
E-L4	3	3	-	1	2	1	3	5	-	2
Subtotal	49	46	0	3	11	2	21	54	5	8
BR-L1	5	4	-	-	-	-	3	4	-	-
BR-L2	3	2	-	-	-	-	1	1	-	-
BR-L3	3	2	-	-	-	-	1	-	-	-
BL-L1	3	3	-	-	-	-	1	1	-	-
BL-L2	4	4	-	-	6	-	1	5	-	3
Subtotal	18	15	0	0	6	0	7	11	0	3
SR-L1	4	2	-	-	-	-	-	-	-	-
SR-L2	1	1	-	-	-	-	-	-	-	-
SL-L1	4	3	-	1	-	-	-	-	-	-
Subtotal	9	6	0	1	0	0	0	0	0	0
G-L1	3	2	-	-	-	-	1	1	-	-
G-L2	2	2	-	-	-	-	2	2	-	-
G-L3	4	4	-	2	-	-	3	4	-	2
Subtotal	9	8	0	2	0	0	6	7	0	2
D-L1	6	6	-	-	-	-	-	11	-	-
D-L1-1R	4	3	-	-	1	-	1	3	-	-
Subtotal	10	9	0	0	1	0	1	14	0	0
Total	129	115	3	8	21	3	57	117	6	21

COST COMPARISON OF STRUCTURES FOR EASTERN IRRIGATION AREA

Structure	Required Facilities and Length	Cost ('000P)	Remarks
Siphon	42" RC Pipe Siphon 1,000m	2,092	
Pump	Earth Canal 1,000m Pump Station; 350m/m Centrifugal Pump X 2	3,247 1/	1/: excluding pump operation cost
Elevated Flume	B H 1.0 X 1.0 Elevated Concrete Flume 1,200m 2/	3,600	2/: extended in order to provide adequate clearance above existing road

ROTATION AREA OF SAMPLE AREAS

Unit :ha

Rotation Block No.	1	2	3	4	5	Total
Sample A, No. 1	9.8	7.7	10.6	11.9	12.0	52.0
No. 2	11.5	13.1	8.7	7.9	12.8	54.0
No. 3	5.4	8.1	11.4	8.6	4.5	38.0
Subtotal						144.0
Sample B, No. 1	7.1	5.2	10.5	3.4	3.8	30.0
No. 2	6.6	9.3	10.7	13.8	11.6	52.0
Subtotal						82.0

ON-FARM FACILITIES OF SAMPLE AREAS

Rotation Block No. (m)	Length of Farm Ditch MFD (m)	SFD (m)	Total (m)	Number of Structures Farm Drains	Turn-outs	Road crossing	Division Boxes	End Checks
Sample A, No. 1	540 (-)	2,260 (750)	3,010 (750)	-	1	-	5	5
No. 2	1,190 (520)	2,510 (1,020)	3,700 (1,540)	880 (300)	1	3	5	5
No. 3	830 (700)	1,810 (590)	2,640 (1,290)	1,350 (1,150)	1	1	5	5
Subtotal	2,560 (1,220)	6,790 (2,360)	9,350 (3,580)	2,230 (1,450)	3	4	15	15
Sample B, No. 1	2,300 (900)	4,250 (770)	6,550 (1,650)	-	1	3	5	5
No. 2	850 (600)	2,010 (430)	2,860	-	1	-	5	5
Subtotal	3,150 (1,500)	6,260 (1,200)	9,410 (2,700)	-	2	4	10	10

Note: Figure in parenthesis shows the length of existing canal which will be utilized even after completion of the project.

TABLE VII-19

PROPOSED LENGTH OF ON-FARM FACILITIES IN SAMPLE AREAS

Rotation Block No.	MFD (m)	SFD 1 (m)	SFD 2 (m)	SFD 3 (m)	SFD 4 (m)	SFD 5 (m)	Sub-total (m)	Total (m)	Farm Drain (m)
Sample A, No. 1	540 (-)	500 (-)	600 (270)	450 (150)	420 (-)	500 (330)	2,260 (750)	3,010 (750)	-
No. 2	1,190 (520)	590 (260)	790 (30)	400 (-)	310 (310)	420 (420)	2,510 (1,020)	3,700 (1,540)	880 (300)
No. 3	830 (700)	250 (-)	250 (150)	330 (130)	520 (-)	460 (310)	1,810 (590)	2,640 (1,290)	1,350 (1,150)
Subtotal	2,560 (1,220)						6,790 (2,360)	9,350 (3,580)	2,230 (1,450)
Sample B, No. 1	2,300 (900)	860 (-)	370 (-)	770 (770)	1,200 (-)	1,050 (-)	4,250 (770)	6,550 (1,650)	-
No. 2	850 (600)	280 (-)	420 (100)	480 (-)	350 (-)	480 (330)	2,010 (430)	2,860 (1,030)	-
Subtotal	3,150 (1,500)						6,260 (1,200)	9,410 (2,700)	=

Note: Figure in parenthesis shows the length of existing canal which will be utilized even after completion of the project.

PRESENT IRRIGATION FACILITIES AND AREAS

Present Irrigation Area

Area No	Type of Irrigation	Acreage (ha)	Area No	Type of Irrigation	Acreage (ha)
	Communal irrigation System	400		Private Weir	460
①	Serruco CIS	400	⑩	Private Weir	80
	KABSAKA Water Impounding Proj	232	⑪	-do-	30
②	Castol WIP	32	⑫	-do-	350
③	Moto WIP	50		Private Pump	580
④	Bondolan WIP	50	⑬	Private Pump	25
⑤	Aglosong WIP	50	⑭	-do-	40
⑥	Belen WIP	50	⑮	-do-	15
	Private Water Impounding	28	⑯	-do-	500
⑦	Sanson	7			
⑧	Pacig	12			
⑨	Salcedo	9		Total of Present Irrigation Area	1700

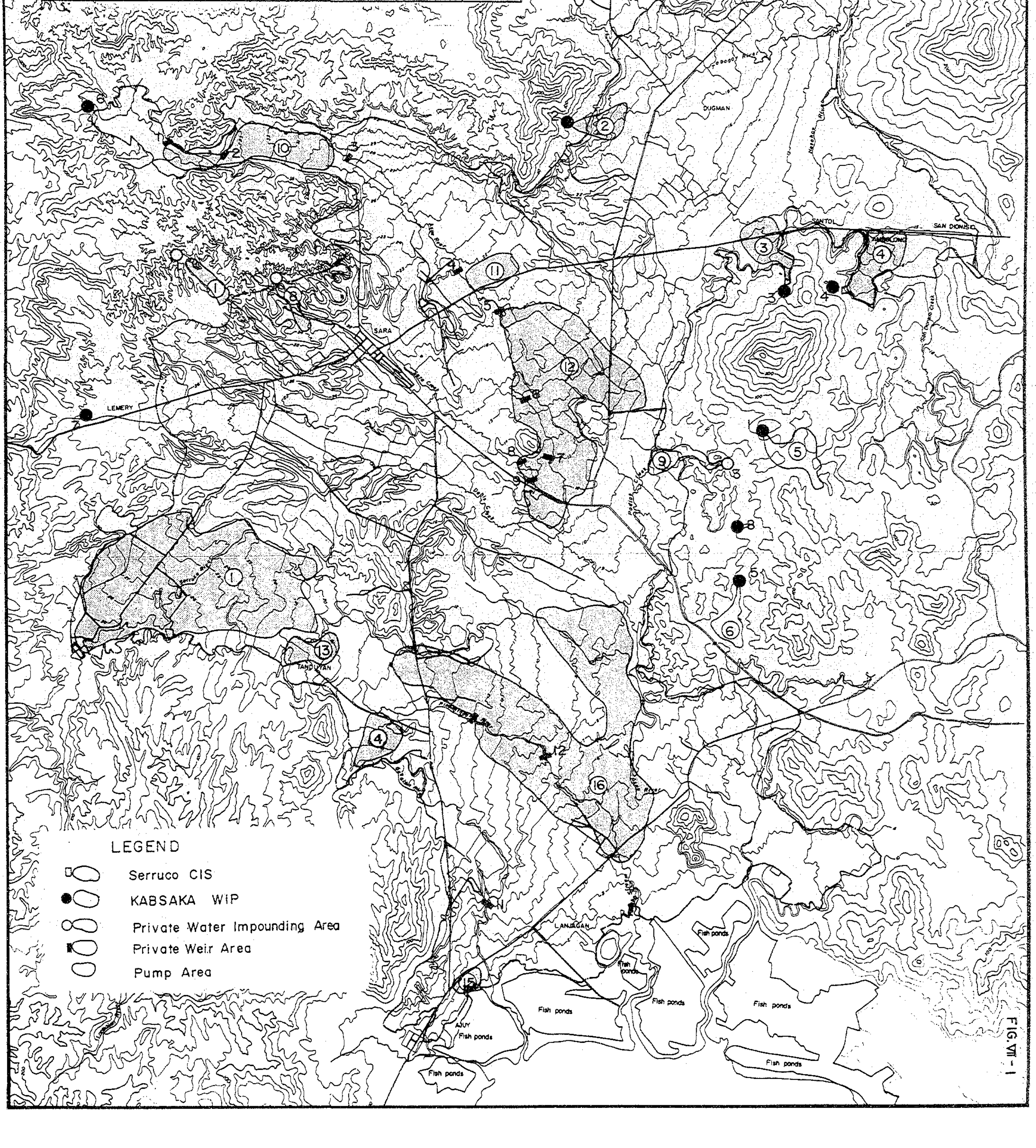
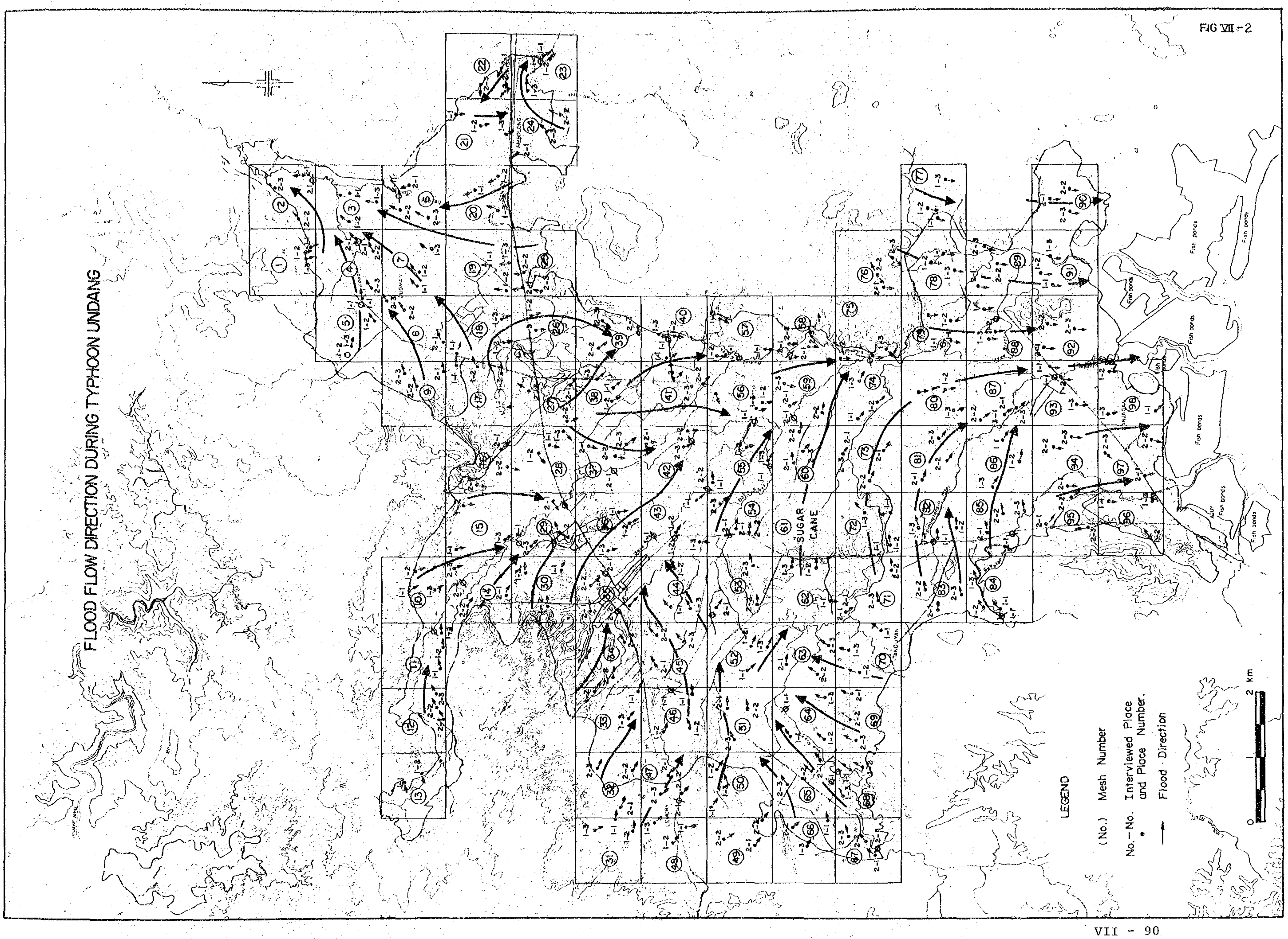


FIG. VII - 1

FIG VII-2


FLOOD FLOW DIRECTION DURING TYPHOON UNDANG
LEGEND
(No.) Mesh Number
No.-No. Interviewed Place and Place Number.
Flood · Direction
SUGAR CANE
Fish ponds
0 1 2 km