No. 68

REPUBLIC OF INDONESIA

MINISTRY OF PUBLIC WORKS
DIRECTORATE GENERAL OF WATER RESOURCES DEVELOPMENT

FEASIBILITY STUDY
ON
THE BILA IRRIGATION PROJECT

ANNEX VOLUME–II

VI. IRRIGATION AND DRAINAGE PLAN
VII. PROJECT IMPLEMENTATION SCHEDULE
VIII. COST ESTIMATE
IX. FLOOD CONTROL PLAN
X. WATERSHED MANAGEMENT
XI. PROJECT EVALUATION

JUNE 1982

JAPAN INTERNATIONAL COOPERATION AGENCY
TOKYO, JAPAN

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国際協力事業団
受入月日 '84. 9.04 | 108
登録No. 409665 | 83.3
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ANNEX - VI

IRRIGATION AND DRAINAGE PLAN

ANNEX-VI IRRIGATION AND DRAINAGE PLAN

CONTENTS

ANNEX-VI IRRIGATION AND DRAINAGE PIAN

1. EXISTING IRRIGATION AND DRAINAGE SYTEM

1.1 Classification of Irrigation System

In Indoneisa, irrigation system is generally classified into three grades in accordance with the following basis:

Grade of system	Structures: Intake facilities	Structures: Division facilities	Structures: Other facilities	Canal density	Water manage-ment
Non-technical	Temporary facilities with stone or gabion	Not provided or temporary facilities with stone, wood or soil	Poorly provided	Low	Not control-lable Not measurable
Semi-technical	Permanent facilities with wet masonry without measuring divice	Permanent facilities with wet masonry without measuring divice	Relatively well provided	Medium	Control-lable but not measurable
Technical	Permanent facilities of wet masonry with measuring device	Permanent facilities of wet masonry with measuring device	Well provided	High	Control-lable and measurable

1.2 Existing Irrigation System

Six (6) small scale irrigation systems have been developed in the Project area consisting of one (1) semi-technical irrigation system and five (5) non-technical irrigatin systems, covering about 500 ha and 700 ha respectively. Technical irrigatin system has not been developed in the area. Existing irrigation systems are classified into DPU irrigation scheme and Desa (village) irrigation scheme according to operation and maintenance body. The location of the existing irrigation systems are as shown in Fig. 1.1.

Water sources of the existing systems depend on small streams originating in eastern mountainous ranges. Since the catchment areas of them are small in scale, most of the streams dry up in the dry season. Irrigation for the areas served by the existing systems, therefore, is limited only in the wet season. The irrigation canals of the existing systems are lacking entirely or provided poorly with distribution system and canal structures, such as division structures, culverts, measuring devices, etc. The existing canals function less effectively and are not free from damage. Most of canals are unlined and heavily silted. They are superannuated.

Semi-technical irrigation system developed in the Project area is only Salodua scheme serving an area 520 ha, of which operation and maintenance are carried out by DPU. The irrigatin water for the scheme is diverted from the Manumanu river originating in eastern hill slopes. The irrigation system consists of a permanent intake weir constructed with wet stone masonry at a catchment area of 8 km^2, and a main canal of 5,190 m with four diversion structures. Tertiary canals are provided for the limited extent, for 60 m in length. Therefore most area is served through plots to plots. The irrigation canal system is presently used for drainage in the wet season due to lack of technical drainage system.

Five Desa irrigation schemes, which are all non-technical irrigation systems, are provided with intake facilities including permanent intake weir constructed with wet stone masonry. Their conditions, however, are poor and operation is not effective. The canal system are primitive and the distribution system is not provided. Irrigation is also limited in the wet season. The construction and operation and maintenance have been carried out by farmers themselves.

The irrigation areas, irrigation facilities of the existing systems are as shown in Table 1.1.

2. WATER REQUIREMENT

2.1 Irrigation Water Requirements

2.1.1 General

Irrigation water requirement is estimated for obtaining the basic information on (1) evaluation of the availability of water resources and (2) determination of irrigation system capacities. For evaluation of the Bila river flow, the long term water balance between the water requirement and the river flow is employed in this study. Since the discharge data are available for 8 years from 1973 to 1981, the water requirements are estimated for a series of such period on 10-day basis. For determination of the system capacities, the maximum requirement with dependability level of four out of five years is adopted.

The estimate of the irrigation water requirement is made as follows:

Irrigation water requirement of paddy, WRP:

$$WRP = (CU + PR + NW + PW - ER)/E$$

where: CU; Consumptive use of water
PR; Percolation rate
NW; Nursery water
PW; Puddling water
ER; Effective rainfall
E ; Total irrigation efficiency

2.1.2 Consumptive use of water

The consumptive use of water is estimated based on the empirical prediction method using the climatic data and crop coefficients relating to crop growth stages. Among the various prediction methods developed so far, the modified Penman method is best selected in consideration of the availability of climatic data for applying the method and the accuracy of the results obtained by it. It is generally accepted as the most reliable prediction method among others.

The consumptive use of water (CU) is calculated by the following formula for each 10 days.

$$CU = Kc \times PET$$

where: CU; Consumptive use of water (mm/day)
Kc; Crop coefficient relating crop growth stage of the proposed cropping pattern
PET; Potential evapotranspiration estimated by the modified Penman method (mm/day)

(1) Crop coefficient, Kc

Three sets of consumptive use measurement tank were installed in the southern, central and northern parts of the project area, to obtain the data on water consumption in the field. The locatin and equipment of measurement are shown in Fig. 2.1, and Fig. 2.2.

Since crop coefficients are one of the factors to predict consumptive use of water, it should well coincide with the concept of the potential evapotranspiration estimated by the applied method. Further the field measurements are not necessarily reliable to estimate the long term tendency. Then, the crop coefficients are determined by cross checking the field measurement on consumptive use of water by coefficients used in the on-going irrigation schemes around the Project area. They are as shown in Fig. 2.3.

Based on the above figure, the variances of the crop coefficient relating to the growing stages are estimated as shown below:

First month			Second month			Last month		
F	M	L	F	M	L	F	M	L
1.00	1.02	1.07	1.18	1.30	1.38	1.38	1.28	-

(2) Potential evapotranspiration

The kind and type of climatic data necessary for applying the modified Penman method are (1) temperature, (2) relative humidity, (3) sunshine duration or solar radiation and (4) wind velocity on daily basis. The meteorological stations to provide such data are Sengkang and Kanyuara. To minimize the effect of their location characteristics, the average values of calculated potential evapotranspiration from the above two stations are used in this estimate. The climatic data used in the estimate are shown in Table 2.1.

The calculation is made with the reference to the Crop Water Requirements, Irrigation and Drainage Paper Riviced Edition 1977 by FAO. The form of equation is expressed as follows:

$$PET = c \times \left[W \times Rn + (1-W) \times f(u) \times (ea-ed) \right]$$

where: PET; potential evapotranspiration (mm/day)
W ; temperature-related weighting factor
Rn ; net radiation in equivalent evaporation (mm/day)
f(u); wind-related function
(ea-ed); difference between the saturation vapour pressure at mean air temperature and the mean actual vapour pressure of the air (mm bar)
c ; adjustment factor to compensate for the effect of day and night weather conditions

The calculation for two stations is as shown in Table 2.2 and 2.3. The results are summarized in Table 2.4 and are shown in Fig. 2.4 together with the comparison of evaporation from the standard class A pan.

(Unit: mm/day)

Month	Sengkang	Kanyuara	Average
Latitude	S 4°07'	S 3°56'	-
Altitude	14.0 m	11.7 m	-
Jan.	4.7	4.7	4.7
Feb.	4.9	4.6	4.8
Mar.	5.0	5.3	5.2
Apr.	4.5	5.0	4.8
May	4.4	5.1	4.8
Jun.	3.7	4.5	4.1

(To be continued)

(Unit: mm/day)

Month	Sengkang	Kanyuara	Average
Jul.	4.4	4.4	4.4
Aug.	5.4	4.6	5.0
Sep.	5.7	5.2	5.5
Oct.	6.0	5.9	6.0
Nov.	5.3	5.6	5.4
Dec.	4.5	4.9	4.7
Annual total	1,789	1,820	1,807

(3) Consumptive use of water

The consumptive use of water is calculated by production of crop coefficient, Kc, and potential evapotranspiration, PET as calculated above on 10 day basis.

The average values of consumptive uses of water (CU) for the proposed cropping pattern are given in Table 2.5 and summarized as follows:

Crop	Apr.		May			Jun.			Jul.			Aug.	
	2nd	3rd	1st	2nd	3rd	1st	2nd	3rd	1st	2nd	3rd	1st	2nd
Wet season paddy	48	48	50	52	60	48	50	52	58	59	65	68	64

Crop	Oct.	Nov.			Dec.			Jan.			Feb.		
	3rd	1st	2nd	3rd	1st	2nd	3rd	1st	2nd	3rd	1st	2nd	3rd
Dry season paddy	66	55	56	58	53	55	63	59	62	70	65	65	49

2.1.3 Percolation rate

Percolation rates differ depending on the soil characteristics, topography, groundwater levels, etc. In the representative fields of the northern, central and southern parts in the Project area, field measurements are conducted during wet season cropping of paddy. The location and equipment of measurement are as shown in Fig. 2.1, and Fig. 2.2.

(Unit: mm/day)

Station	Measurement period	No. of available date	Average value
1. DS. Bila (Northern part)	Aug. 13 - 21	7	1.4
2. DS. Lowa (Southern part)	Aug. 15 - Sep. 22	23	1.4
3. DS. Wele (Middle part)	Sep. 1 - 29	12	2.0

The results show fairly low values ranging from 1 mm/day to 2 mm/day. Further, making reference to the value presently applied in the on-going irrigation schemes around the project area, the percolation rate of 2.0 mm/day is adopted in the assessment of the irrigation water requirement.

2.1.4 Nursery water

The nursery water requirement is estimated for the amount of water necessary for (1) preparation of nursery bed, (2) evapotranspiration from nursery fields and (3) percolation loss.

The nursery water requirement is estimated under the following conditions:

(1) Area required for nursery bed : 1/20 of main fields

(2) Nursery period : 20 days

(3) Water required for 20 days :

Preparation of nursery bed	:	100 mm
Evapotranspiration (5 mm/day x 20)	:	100 mm
Percolation (2 mm/day x 20 days)	:	40 mm
Total		240 mm

2.1.5 Puddling water

The puddling water requirement is estimated for the amount of (1) water needed for saturation of soil profile, (2) loss water occurred during water supply consisting of percolation and evaporation losses and (3) standing water after puddling.

The puddling water requirement is much influenced with irrigation practice and the field layout. The on-going irrigation schemes around the Project area apply such water supply method as to gradually give water to a field for a long period more than 10 days. With reference to this method, the puddling water requirement is estimated on the basis of 10-day water supply.

The on-farm irrigation system will be provided so as to cover five or more numbers of fields lots by a outlet on a field ditch, and each field will be served with water coming from the upstream field. In this system, the standing water is inevitably needed in high depth of water.

In consideration of the above, the average depth of standing water of 70 mm is adopted.

Thus, the puddling water requirement is estimated as follows:

(1) Water needed for saturation of soil profile 75 mm

(a) Depth of soil and porosity

surface soil: 20 cm, 50%
sub-soil : 10 cm, 50%

(b) Vapor phase in soils after puddling: 5%

(c) Soil moisture before water supply : 20%

(2) Water loss occurred during water supply

(a) Percolation loss (2 mm/day x 10 days) 20 mm

(b) Evaporation loss, assuming the same as evapotranspiration under the initial paddy field condition

wet season (4.8 mm/day x 10 days): 48 mm
dry season (5.4 mm/day x 10 days): 54 mm

(3) Average standing water depth after puddling 70 mm

Total	Dry season:	219 mm
	Wet season:	213 mm
		(220 mm)

2.1.6 Effective rainfall

The rate of effective rainfall to total rainfall is much influenced with water consumption rate, rainfall intensity, water holding capacity relating to the field sizes and shapes, and water supply method. To estimate the effective rainfall, the daily water balance in the field is carried out for the period from 1973 to 1981 under the following conditions:

(1) Water consumption rate is the sum of percolation rate and consumptive use of water,

(2) Continuous water supply method is adopted for field water supply management,

(3) Average water holding capacity of the field is assumed to be 40 mm from the maximum holding capacity less minimum standing water,

(4) Intercepted loss rainfall of 2 mm is considered for every rainfall.

The daily rainfall data at Tanru Tedong located on the central part of the Project area is used for the estimate of the effective rainfall. The effective rainfall estimated is shown in Table 2.6 after summarized on 10-day basis.

2.1.7 Irrigation efficiency

The irrigation efficiencies are determined by the following categories:

(1) Conveyance efficiency, Ec: the ratio between water received at the field inlet and that released at the project intake structure.

(2) Application efficiency, Ea: the ratio between water made directly available to the crop and that received at field inlet.'

(3) Total irrigation efficiency, Et: the ratio between water made directly available to the crop and that released at the intake structure, Et = Ec x Ea

In consideration of earth canals on major parts and canal control facilities to be provided, the conveyance loss of 20% and application loss inclusive of field losses of also 20% are applied with reference to the design values prepared by the Directorate of Irrigation, DGWRD. The total irrigation efficiency, therefore, is as follows:

(1) Conveyance efficiency : 80%
(2) Application efficiency : 80%
(3) Total irrigation efficiency: 64%

2.1.8 Diversion water requirement

(1) Unit diversion water requirement

The seasonal unit diversion water requirements are estimated for the period from 1973 to 1981 on 10-day basis by use of values obtained as above for the proposed cropping pattern, taking into account the cropping intensity in each growing period. The results are as shown in Table 2.7.

(2) Design water requirement

The irrigation system capacities are determined on the basis of 80% dependability level for irrigation for 10-day requirement. The unit design water requirements with occurrence of once out of five years are estimated, based on the annual maximum unit water requirement shown in Table 2.8.

(Unit: ℓ /sec/ha)

Season	Unit design water requirement
Dry season	1.65
Wet season	1.35

2.2 Drainage Water Requirements

2.2.1 General consideration

The drainage condition of the Project area is classified into well drained, imperfectly drained and poorly drained types. The area of 5,300 ha or 47% of the possible land falls in imperfectly drained and poorly drained conditions during the rainy season. It is essential to maintain the paddy fields in a adequately drained condition for the purpose of successful irrigatin farming.

In general, the criteria for calculating unit drainage water requirement defines the rainfall intensity with certain probability and a drain period necessary for removal of excess water to an allowable extent. In on-going irrigation schemes in Indonesia, the drainage requirements have been estimated by applying different ways considering the natural and physical conditions prevailing over the Project area.

The Project area extends over the foot of the low mountainous ranges and continues to the low-lying area around Lake Buaya. The flood runoffs from the above mountainous area concentrate in the Project area. Therefore the drainage system should have the capacity enough to transport the flood without hampering the Project area, in addition to the capacity to remove the excess water of rainfall in the irrigation area.

The drainage water requirement, therefore, is estimated for such two sources of water to be drained. The drainage for the runoff from the outside Project area is determined from the peak discharge of flood flow. The drainage for paddy fields is estimated for draining rather long period rainfall gradually. The calculatio of them is shown hereunder.

2.2.2 Drainage water requirement

The drainage water requirement for the drainage system of the Project is estimated for:

(1) removal of excess rainfall in the paddy fields

(2) transporting the runoff coming from the outside Project area

(a) Drainage requirement for removal of excess rainfall in the paddy fields

In estimating the drainage water requirement for removal of excess rainfall in the paddy fields, the following criteria are applied: four (4) day consecutive rainfall with five (5) year return period is removed within four (4) days. The rainfall data at Tanru Tedong is used for estimate of the design rainfall. The design value is estimated by the following formula:

$$Qp = F \times \left(\frac{C \times R_4 \times A}{T} \times 10\right)$$

where: Qp; Drainage water requirement (m^3/sec)
F ; Peak factor, 1.25
C ; Runoff coefficient, 0.8
R_4; 4-day consecutive rainfall, 280 mm
A ; Drainage area (ha)
T ; Drainage period, 4 x 24 x 3,600 sec

The above calculation gives the unit drainage water requirement of 8.1ℓ/sec/ha.

(b) <u>Drainage requirement for transporting the runoff from the outside Project area</u>

To estimate the surface runoff coming from the land covered with forest, bush, etc., the McMath's formula suggested in the DRAINAGE MANUAL, USBR, is applied. The design rainfall is estimated for daily rainfall with return period of 5 years using the rainfall data at Tanru Tedong. The formula applied is as follows:

$$Qo = 7.2 \times 10^{-3} \times C \times i \times S^{1/5} \times A^{1/5}$$

where: Qo; Peak flow (m^3/sec)
C ; Coefficient representing the watershed characteristics, 0.6
i ; Rainfall intensity for the time of concentration (mm/hr)
S ; Average ground slope
A; Drainage area (ha)

The rainfall intensity for the time of concentration is calculated by use of rational formula.

$$i = \frac{R_{24}}{24} \times \left(\frac{24}{T}\right)^{2/3} \quad \text{(mm/hr)}$$

where: R_{24}; Daily rainfall with return period of 5 years (Tanru Tedong rainfall data), 150 mm/24 hr
T ; Time of concentration

The average ground slope is expressed as follows:

$$S = h/\ell$$

where: h; Fall of channel between the farthest contributing point and the point of the concentration (m)
ℓ; length of channel (m)

The nomograph is prepared for estimate of the time of concentration of flood for different h and ℓ as shown in Fig. 2.6.

(c) Design drainage water requirement

The design discharge of the drainage canal is estimated by use of the following formula:

$$Qd = f \times (Q_p + Q_o)$$

where: Qd; Design drainage canal discharge (m^3/sec)
f ; Allowance factor, 1.15
Q_p; Drainage water requirement for the paddy fields
Q_o; Drainage water requirement for transporting the flood runoff

3. WATER BALANCE STUDY

3.1 General

The irrigation water requirements vary from year to year, and throughout the year. The discharge records of the Bila river, which are made available for eight (8) years since April, 1973 near the Project intake site with a catchment area of 379 km^2, indicate the large fluctuation. There is difference between the time distribution of the available irrigation water and that of the irrigation water requirement. In this context, the seasonal water balance study between the supply and the requirement is conducted for the period of 1973 to 1981 to assess the available irrigation water of the Bila river and to estimate the irrigable area guaranteed by the Bila river flow.

3.2 Irrigation Area with the Bila River Natural Flow

The balance calculation is made by means of dividing the river discharge by the diversion water requirement on the daily basis. In making determination of guarantee irrigation area on the short period basis in the water balance, there is a problem on the sensibility of the crop against water shortage and the water supply management. In this study it is assumed that the lack of water supply for a short period will not result in severe damage for crop in consideration of the effect of the standing water, and that the period of 5 days is acceptable for the lack of water supply. Therefore, 10 days mean values of irrigable areas in the critical low flow period are taken as the guarantee irrigation area by the natural river flow. The results are as shown in Fig. 3.1, and the irrigable areas in the critical low flow periods in the wet and dry seasons are shown in Table 3.1. The summary is as shown below:

(Unit: ha)

Year	Wet season paddy First lowest	Wet season paddy Second lowest	Dry season paddy First lowest	Dry season paddy Second lowest
1973	17,580	N.T.	7,530	13,260
1974	6,350	6,930	5,510	6,280
1975	3,480	17,830	4,670	6,390
1976	3,790	4,650	2,850	4,280
1977	7,610	8,390	2,030	2,950
1978	13,700	N.T.	5,500	5,540
1979	6,990	7,560	2,090	3,790
1980	8,050	10,460	3,160	3,370

Based on the above result, the irrigable area with the irrigation dependability level of 80% is estimated for the dry season cropping and wet season cropping.

(Unit: ha)

Cropping	Irrigation area
Dry season	2,600
Wet season	4,600

4. REVIEW OF EXISTING PLANS

4.1 General

Different approaches to the development of the Bila irrigation project have been made, i.e., Master Plan and DOI Plan. In order to formulate the most optimum development plan of the Project, both plans are first reviewed and updated based on the results of investigation obtained so far. The principal development concepts of the existing two plans are briefed as follows:

In the Master Plan, the Bila Irrigation Project was ranked first in the development priority. In the study, the main role of the water resources development projects in the Central South Sulawesi area was analyzed from the viewpoints of national policy, national economy and regional socio-economy. Through those analyses, the basic concept for agricultural development in the Bila area was established, stressed on the following:

(1) increase of paddy production taking into consideration attainment of self-sufficiency of foodstuff at national level, and the increase of farmers' income,

(2) equalizing of regional development gap in view of the equal distribution of social justice.

In due consideration of the above, the Bila Irrigation Project was formulated, aiming at the efficient and economical utilization of the resources with principal development concept that the Project area should cover the large extent of irrigable land 10,500 ha to be served with the Bila river natural flow through the Bila intake having an intake water level 35 m.

On the other hand, the Government has commenced the study and investigation since 1975 in response to the strong request on the early implementation of the Bila irrigation project by the local inhabitants. The DOI plan aims to serve an area of 9,288 ha with the Bila river flow through the Bila intake having an intake water level 30 m.

The general features and the general layouts of both plans are as shown in Table 4.1 and Fig. 4.1, respectively.

4.2 General Features of the Existing Plans

4.2.1 Master plan

The Master Plan identified the natural resources endowed in the Bila area and proposed the development strategy in the Bila area.

The irrigation development plan aims to serve the land lying on the left bank and a bit of sandwitched area by the Bila and Boya rivers. With the natural flow of the Bila river, an area of 10,500 ha is irrigated. The intake structure is constructed with a multiple stage diversion method at the site about 3 km upstream from the intake site proposed by DOI, having an intake water level of 35 m. The irrigation area was selected mainly on the alluvial plain starting from 35.0 m in elevation at the head to 9.0 m in elevation around Lake Buaya.

The project area extends for 9,300 ha over the left bank and for 1,200 ha over the right bank. Irrigation water of 13.8 m^3/sec at maximum is taken at the Bila intake structure, consisting of 12.2 m^3/sec for the left bank area and 1.6 m^3/sec for the right bank area. The general features of the Project proposed in the Master Plan are as shown in Table 4.1.

The irrigation area was determined by means of water balance calculation on the monthly basis for the period of 5 years from 1973 to 1977, which results in the irrigatin area of 10,500 ha and 6,800 ha (average) in rainy and dry seasons respectively as shown in Table 4.2.

4.2.2 DOI plan

The Government has conducted the investigation for making the detailed design for the project covering topography, geology, soil mechanics and hydraulic model test of intake weir. The detailed design of intake and main irrigation system has been completed. Those activities made so far by the Government are indicated in Table 4.3. Reports and drawings prepared are listed in Table 4.4.

With the natural flow of the Bila river through the Bila intake to be constructed approximately 8 km upstream of the junction of the Boya river in a bee line, an area of 9,288 ha is irrigated. The intake structure is constructed with so-called coupure method on meanders of the Bila river, having an intake water level of 30 m. The irrigation area lies from 30 m in elevation at the head to 7 m in elevation around Lake Buaya.

The Project area extends for 7,608 ha over the left bank and for 1,680 ha over the right bank of the Bila river. The maximum irrigation water at the head of main canals is 12.11 m^3/sec for the left bank area and 2.53 m^3/sec for the right bank area.

The principal irrigation development plan was identified in 1975 based on the field reconnaissance, subsequently the topographic maps on a scale of 1:5,000 were prepared covering an area 13,000 ha. Based on those maps the development plan was studied.

The irrigation water requirement was estimated based on the empirical prediction method by means of production of potential evapotranspiration by crop coefficient. To estimate potential evapotranspiration, the modified Penman method was adopted. The basic values used for estimate of the irrigation water requirement are summarized in Table 4.5. The design unit diversion water requirement is determined to be 1.6 ℓ/sec/ha.

The irrigation area was determined by use of the estimated monthly discharge. Through the water balance study, the project area was determined to be 6,886 ha. It was extended to 9,288 ha in the final design.

4.3 Review of Existing Plans

4.3.1 Irrigation area

The irrigation plan conceived by both plans is the complete irrigation in the wet season and supplemental irrigation in the dry season by use of the Bila river natural flow through Bila intake structure. Those plans were formulated under the conditions of limited hydrological informations.

The water balance study based on the accumulated data on the Bila river discharge indicates that the Bila river flows are far short to irrigate either of the Project areas of the DOI Plan and the Master Plan, resulting in 4,600 ha and 2,600 ha in the rainy and dry seasons with the dependability level of 80% as shown in CHAPTER 3. Therefore the stable and adequate irrigation would not be ensured for either envisaged project irrigation areas of 10,500 ha and 9,288 ha for the Master Plan and the DOI Plan respectively.

4.3.2 Project area

The project areas of 10,500 ha and 9,288 ha were selected for the land lying in the elevations ranging from 35 m to 9 m and 30 m to 7 m in the Master Plan and the DOI Plan. The hydrological investigations and study on the low-lying area around Lake Buaya, however, disclosed the severe inundation and ill-drainage conditions of the land up to elevations of 10 m in the wet season as shown in Table 4.6. For the sake of efficient and safety irrigation farming and efficient allocation of water in the Bila area, exclusion of such inundation area from the Project irrigation area is proposed, resulting in the area of 10,000 ha of the Master Plan and of 8,500 ha of the DOI Plan.

4.3.3 Project facilities

The irrigation systems for both plans were designed with different water requirements. The Master Plan is based on the unit diversion water requirement of 1.31 ℓ/sec/ha. The DOI plan is based on that of 1.60 ℓ/sec/ha. The present study on the irrigation water requirement made in Chapter 2, however, shows the unit diversion water requirement of 1.65 ℓ/sec/ha at the intake site. The irrigation system capacity of the Master Plan would be insufficient to meet the irrigation water requirement even for the revised area of 10,000 ha. The main irrigation system of the DOI Plan would have some allowance in the flow capacity for the revised area of 8,500 ha.

The intake structures are contemplated at the different sites. Those sites have the following characteristics. The intake site proposed in the DOI plan is adequate to supply the water with the required water level of 30 m in short distance to the Project area. The intake site proposed in the Master Plan is the most desirable site to divert water for the large area with the water level of 35 m in view of topography, geology and hydrology.

The design of the DOI plan was made for the main irrigation facilities such as intake structure, main and secondary canals and related canal structures. The drainage and farm road systems, however, are additionally required.

4.4 Preliminary Design of Intake and Canals Proposed in the Master Plan

4.4.1 General

There exists the difference in the results of the facility designs. The design in the Master Plan is prepared on the preliminary basis. On the other hand, the DOI plan is presented in the detailed design. In order to make comparison of both plans, the preliminary design of the intake and irrigation canal is carried out.

4.4.2 Bila intake

The main function of the intake weir is to raise the water level up to 35 m in elevation necessary for diverting water to main canals which supply water for an area of 10,000 ha.

The intake weir site is selected about 3 km upstream from the Bila intake site proposed by the DOI, in consideration of topography, geology construction access and efficient operation and maintenance in accordance with the proposed plan in the Master Plan.

At the site, topographic survey and geological and soil mechanical investigation as presented in ANNEX-III and IV were carried out.

The above mentioned investigation disclosed the following design considerations:

(1) The fixed type concrete weir can be placed on the baserock in the present river channel.

(2) The "highly weathered baserock" is not useful for the foundation of the weir because of poor shearing strength and high permeability.

(3) The embankment is efficient for the right bank which is composed of mainly terrace deposit. However, sufficient cutoff to a firm baserock is necessary against leakage through the terrace deposits and highly weathered loose baserock.

(4) The foundation treatment work, such as curtain grouting etc. is required because the baserock is not so hard and well cemented.

Hydrological and hydraulic conditions for the design of the intake weir are taken as follows:

(1) Design intake discharge: The design intake discharge is taken as the peak diversion requirement of 16.5 m^3/sec for the project area of 10,000 ha.

(2) Design intake water level: Based on the topographic relationship between the intake site and irrigation area, the intake water level is determined to be EL. 35.20 m so as to supply the enough water to the area of 10,000 ha.

(3) Design flood discharge: The flood with 100-year return period, 1,200 m^3/sec, is taken as the design flood discharge.

(4) Design flood water level: Based on the estimate of the overflow depth at the intake weir, the design flood water level is estimated to be EL. 38.0 m.

Based on the above conditions, the intake weir is designed as follows:

(1) The intake weir is of overflow concrete weir combined with earthfill embankment. The excavation line is determined to be EL. 18 m taking into account the stable bedrock for the weir foundation. Then the height of weir is determined to be 17.2 m at the maximum point.

(2) The crest elevation of an overflow weir section is given to be same as the intake water level of EL. 35.20 m. The crest elevation of a nonoverflow weir section is determined to be EL. 41.0 m. The crest elevation of earth embankment section is determined to be EL. 40.0 m.

(3) Shape of weir is of hydraulically favourable type for the overflow of flood water. Hydraulic jump basin type is applied to a stilling basin in consideration of Froude numbers and downstream water depth. Earth embankment is of homogeneous earthfill type with cutoff trenches in view of foundation conditions.

(4) Fundation treatment for concrete weir and embankment is needed;

- for the earth embankment, sufficient trench cutoff to baserock is necessary against leakage through the terrace deposits and highly weathered base rock,

- curtain grouting is applied for the foundation treatment at the filet of the concrete dam and at the trench of each embankment.

The preliminary design of the Bila intake weir is as shown in Fig. 4.2 to Fig. 4.8.

4.4.3 Irrigation canal

The irrigation canal system is worked out to command the possible largest irrigable area of 10,000 ha starting from the above mentioned Bila intake with the intake water level of 35.0 m.

The layout planning of irrigation canal system is based on the maps on a scale of 1:5,000 prepared by DOI and on a scale of 1:25,000 prepared by JICA in 1978. Especially for the secondary canal layout, the detailed map of 1:5,000, and the proposed layout by DOI are primarily referred.

Based on the selected main canal route, the following field investigation was carried out.

(1) The field reconnaissance is made along the alignments to know the micro-topography, hydrological conditions and soil conditions, and to collect farmers' opinion toward the development plan.

(2) The following topographic survey was carried. The results are as shown in Fig. 4.9.

(a) check levelling for existing benchmarks,

(b) route survey including cross section and profile survey of the main canals.

(3) Soil mechanical and geological investigation including,

(a) penetration tests and swedish sounding test at main structure sites,

(b) soil mechanical survey along main canals by pit observation and soil mechanical test in laboratory by the Government.

(4) The construction material survey is made for their availabilities and prices.

(5) For the layout planning, agricultural, economical and sociological data are also collected.

Based on the field investigation of the canal alignment, the route of the main irrigation canal is determined as shown in Fig. 4.9. The canal profile of the main canal is established as shown in the longitudinal profile Fig. 4.10. The design of canal is based on the design criteria presented in CHAPTER 7.

A number of canal structures of various types are required in conjunction with the irrigation canals. The configuration of these structures are made properly considering their functions, canal layout, operational schedule and social conditions in the Project area. Related canal structures consists of check structures, turnouts, siphons, culverts, spillways, cross drainage culverts, drops.

4.5 Economic Evaluation of Existing Plans

4.5.1 General

In order to assess the present economic situation of both plans, the economic evaluatin is made on the following conditions:

(1) The construction cost of the Master Plan is estimated based on the preliminary designs made in Section 4.4. The construction cost of the DOI is estimated based on the results of the present design inclusive of additionally required facilities such as drainage and road systems.

(2) The irrigation benefits for both plan is estimated based on the result of water balance study which indicates the irrigation areas of 4,600 ha and 2,600 ha in the wet and dry seasons cropping respectively.

The general features of the facilities required and the work volumes for both plans are as shown in Table 4.6 and Table 4.7 respectively. Based on the above, the construction costs are estimated as shown in Table 4.8 by use of cost data presented in ANNEX-VIII. The anticipated benefits are estimated as shown in Table 4.9, making reference to ANNEX-V.

4.5.2 Economic evaluation

The economic feasibility of both plans is examined by calculating the internal rate of return (IRR) on the following assumptions:

	Description	Master plan	DOI plan
1	Project area	10,000 ha	8,500 ha
2	Irrigation area		
	(1) Wet season	4,600 ha	2,600 ha
	(2) Dry season	2,600 ha	2,600 ha
3	Project life	50 years from 1983	50 years from 1983
4	Total economic cost ($\times 10^6$Rp.)	27,531	21,864
5	Construction period (Years from 1983 including design and tender)	7	7
6	Annual net incremental benefits ($\times 10^6$Rp.)	3,759	3,759
7	Build-up period (Years from 1983)	7 (12)	7 (13)
	Internal Rate of Return (IRR)	8.9 %	10.3 %

The internal rate of return is calculated at 8.9% for Master Plan and 10.3% for DOI Plan, indicating the rather low economic viability of both plans. The main reason for such low economic feasibility is the low rate of irrigation areas to the total area under the project; in other words, the project scale proposed in both plans is too large compared to the available water of the Bila river.

5. STUDY ON RESERVOIR PLANS

5.1 General

5.1.1 Necessity of reservoir plan

The water balance study indicates that the Bila river natural flows will fall short to irrigate either of the project irrigation areas envisaged by the Master Plan and DOI Plan. The economic evaluation of both plans made in Chapter 4 shows the low economic viability of 8.9% and 10.3% in terms of IRR for the Master Plan and DOI Plan respectively. If the existing plans are implemented as originally formulated, they would not be economically justified. In these situations, a reservoir plan is necessary and effective for both plans in consideration of the following:

(1) the possible irrigable land of 10,000 ha in the Bila area is far larger than the guarantee irrigation area (4,600 ha) with the Bila river natural flow as mentioned in Chapter 4,

(2) the water resources in the Bila river basin are abundant if regulated, and

(3) the irrigation system with the natural flow intake system, which falls in the small scale, is not suitable from the viewpoint of socio-economic conditions.

5.1.2 Possible reservoir plans

The possible alternative sites for constructing storage reservoirs are revealed in the Bila river and the kalola river in view of hydrology, topography and geology at the hydraulic structure site.

An alternative Bila dam site is selected in the middle reaches, near the Bila intake site proposed in the Master Plan having a catchment area of 376 km^2 as shown in Fig. 5.1. The Kalola dam site is situated near the debouchment with a catchment area of 122 km^2.

In consideration of the reservoir sites and potential water resources to be utilized from the relevant rivers, the irrigation system incorporating a reservoir can be taken as follows:

(1) Bila dam irrigation system; This system is a dam irrigation system to use the regulated flow of the Bila river. Irrigation water is diverted from a reservoir, therefore the Bila intake weir will not be necessary.

(2) Bila intake and Kalola dam combinated system; This system is aimed at stabilization and supplement of the Bila river diversion water with storage of the Kalola river flow. Two hydraulic structures of the headworks are needed, i.e., an intake weir in the Bila river and a dam in the Kalola river.

5.2 Alternative Reservoir Plans

5.2.1 Bila reservoir plan

(1) Plan formulation of Bila dam

The Bila river is the main irrigation water source for the Bila area, having the largest catchment area among the rivers to be utilized in the area and providing a suitable hydraulic structure site at the head of the Project area. In order to assess the development potential of the Bila reservoir plan, the proper size of a reservoir capacity is first studied based on the observed discharge data for eight years.

Four (4) alternative scales of reservoir are selected for the following different irrigation dependability:

(a) Case 1: Reservoir having a capacity to ensure the irrigation fully in the rainy season for the past eight years,

(b) Case 2: Reservoir having an average capacity of the past eight years occurred in the dry season cropping,

(c) Case 3: Reservoir having a capacity corresponding to the second largest requirement in the past eight years,

(d) Case 4: Reservoir having the largest capacity required in the past eight years.

(2) Storage capacity

The storage capacity of the reservoir is estimated on five days basis for the period of eight years since 1973. The storage includes the irrigation requirements, the river maintenance flow, the domestic water and evaporatin losses.

(a) Irrigation requirements
The seasonal diversion water requirements as estimated in Table 2.8 are used.

(b) River maintenance flow
The water to be released from the reservoir is estimated to be 0.4 m^3/sec as shown in Table 5.1. This is the same as the amount of water to flow down from the intake weir in case of an intake weir irrigation system, as shown in Fig. 5.2.

(c) Domestic water
This is estimated for the population of 83,900 on the basis of 0.1 m^3/day/person.

(d) Evaporation loss
This is estimated based on pan evaporation observed at Sengkang station after converting to the equivalent evaporation rate from the free water surface.

(e) Sedimentation
The annual sediment load transported from the upper reaches of the dam is determined to be 450 m^3/km^2/year, based on the results of sediment analysis. The sedimentation storage in the reservoir is calculated for 100 years.

Based on the above, the water deficit of the Bila river flow for the case of irrigation of possible irrigable area 10,000 ha is estimated as shown in Table 5.2, and the required storage capacities for the above alternative cases are determined as shown in Table 5.3.

(3) General features of alternative Bila dams

The dam site is selected at the narrow section in-between the proposed and alternative Bila intake sites. The storage-water surface curve (Q-A curve) at the site is prepared by use of the maps on a scale of 1:25,000 as shown in Fig. 5.3.

The design criteria applied in this study are:

(a) The Dam type is of rockfill type with central core judging from available materials around the site and topographic features.

(b) The design flood discharge is estimated at 1,500 m^3/sec, which is calculated for the peak flood discharge of 1,250 m^3/sec with 200-year probability of occurrence plus 20% allowance. This is equivalent to that with 1,000-year probability of occurrence.

(c) The spillway is of non-gate control type.

The typical corss section and profile of the dam are as shown in Fig. 5.4. The general features of the alternative dams are as shown below.

Description		Alternative Case			
		Case-1	Case-2	Case-3	Case-4
1. Irrigation area (ave.)					
Wet season	(ha)	10,000	10,000	10,000	10,000
Dry season	(ha)	4,800	6,400	7,200	10,000
2. Storage					
Effective	($10^6 m^3$)	12.1	28.8	37.8	68.2
Dead	($10^6 m^3$)	17.0	17.0	17.0	17.0
Total	($10^6 m^3$)	29.0	45.8	54.8	85.2
3. Water surface level					
L.W.L.	(EL. m)	41.5	41.5	41.5	41.5
N.H.W.L.	(EL. m)	43.5	45.5	47.0	51.0
F.W.L.	(EL. m)	46.5	48.5	50.0	54.0

(to be continued)

Description	Alternative Case Case-1	Case-2	Case-3	Case-4
4. Dam features				
Crest length (m)	1,200	1,400	1,500	1,900
Crest width (m)	8.0	8.0	8.0	8.0
Crest EL. (El. m)	49.5	51.5	53.0	57.0
5. Spillway				
Design flood (m^3/sec)	1,500	1,500	1,500	1,500
	non-gate control type			

(4) Construction cost

The construction costs of the alternative dams are estimated based on the preliminary design. The estimated costs are as shown in Table 5.4.

5.2.2 Kalola reservoir plan

(1) Storage capacity

The main purpose of the Kalola reservoir is to supplement the water deficit of the Bila river diversion water. The required storage, therefore, is estimated so as to make up the deficit of intake water based on the result of the water balance calculation between the Bila river flow and the diversion requirement. Since the observed data on the Kalola river discharge is scare, the estimate of the discharge was made by analysing the runoff characteristics of the Bila river, the Gilirang river and the Kalola river, as detailed in ANNEX-I.

The storage of the Kalola reservoir is estimated on the following conditions:

(a) River maintenance flow
The water to be released to the lower reaches is determined to be 0.1 m^3/sec which corrresponds to about 0.1 m^3/sec/100 km^2, similar to the released water from the Bila intake weir in volume.

(b) Domestic use, evaporation loss and dead storage
The principal values for estimate of those items are the same as mentioned in Section 5.2.1.

The storage requirement of the Kalola reservoir for the possible irrigable area of 10,000 ha is estimated for the period of eight years since 1973, as shown in Table 5.5.

(2) General features of Kalola dam

The dam site is selected at the debouchment about 1 km northeast of village Watang Kalola, having a catchment area of 122 km^2. The storage-reservoir area curve (Q-A curve) at the site is prepared based on the maps on a scale of 1:25,000 as shown in Fig. 5.5.

The dam features are based on the following:

(a) The dam type is of zoned rock fill type with central core according to the results of topographic, geological and soil mechanical investigation.

(b) The design flood discharge is estimated at 800 m^3/sec based on the peak flood discharge of 645 m^3/sec with 200-year probability of occurrence plus 20% allowance. This is corresponding to the flood with 1,000-year probability of occurrence.

(c) The spillway is of non-gate control type.

The general features of the Kalola dam are as shown below:

Description		General features
1. Irrigation area		
Wet season	(ha)	9,800
Dry season	(ha)	9,800
2. Storage		
Effective	($10^6 m^3$)	37
Dead	($10^6 m^3$)	6
Total	($10^6 m^3$)	43
3. Water surface level		
L.W.L.	(EL. m)	30.0
N.H.W.L.	(EL. m)	36.0
F.W.L.	(EL. m)	39.5
4. Dam features		
Crest length	(m)	230
Crest width	(m)	8
Crest EL	(EL. m)	42.5
5. Spillway		
Design discharge	(m^3/sec)	800
Type		non-gate control type

(3) Constructin cost

The construction cost of the kalola dam is estimated based on the result of the construction cost estimate mentioned in ANNEX-VIII, as shown in Table 5.6.

5.2.3 Selection of reservoir plan

In order to determined the most desirable reservoir plan between the above mentioned alternatives, the economic comparison was carried out for the construction cost of Bila dam and the combined construction cost of Bila intake, Kalola dam and relevant facilities. The construction cost of Bila intake is referred to that presented in ANNEX-VIII. The selection of the reservoir plan is made in terms of unit cost per annual cropping area. The summary is as follows:

	Construction cost	Irrigation area Wet season	Dry season	Annual	Construction cost per unit cropping area
	(10^6 Rp)	(ha)	(ha)	(ha)	(Rp/ha)
Bila reservoir plan					
Case-1	21,224	10,000	4,800	14,800	1,434,000
Case-2	22,330	10,000	6,400	16,400	1,362,000
Case-3	23,886	10,000	7,200	17,200	1,389,000
Case-4	29,156	10,000	10,000	20,000	1,458,000
Kalola reservoir plan	10,323	10,000	10,000	20,000	516,000

As shown in the above table, the Kalola reservoir plan, which consists of the Bila intake and kalola dam for the Project headworks, is that most economical results, compared with the different sizes of Bila reservoir. Therefore the Kalola reservoir plan is selected for the proposed Project.

6. PROJECT FORMULATION

6.1 Potential Area for Irrigation Development

6.1.1 General

The field investigation and studies cover an area of about 20,000 ha in gross relevant to the Project study particularly making study on land suitability for irrigation development and the present drainage condition for proper planning of drainage improvement.

In selecting the most suitable irrigation development area, the following principal factors are taken into consideration: (1) land use, (2) soil condition, (3) irrigability, (4) drainability, (5) regional socio-economic conditions and (6) optimum project scale in terms of economic viability.

The present conditions on the major factors are revealed as follows:

6.1.2 Factors affecting selection of irrigation area

(1) Land use

The land use pattern in the survey area is closely correlated to topographic condition, soil condition and availability of water sources. In particular, development of the paddy fields, mainly extending over the alluvial plains, is well correlated to above conditions. The paddy fields have been presently developed to the possible maximum extent, even over the undulating lands as far as the above conditions are allow paddy cultivation. There exist about 13,700 ha of paddy fields in and around the study area. The details are as shown in ANNEX-V.

(2) Soil condition

The existing paddy fields are developed on the two soil units, Eutric Fluvisols and Eutric Gleysols. These soils extend over the flat alluvial plain and have suitable characteristics for paddy cultivation like flat topography, deep surface soils, heavy soil texture, easy water availability, etc. Other soil units observed in the study area are Ferric Acrisols, Plinthic Acrisols and Eutric Regosols which extend on the hilly regions and along the river stretches, and are not suitable for rice cultivation due to shallow soil depth, stoniness and undulating topography. This means that existing paddy fields have been developed to the possible maximum extent and there is no unused arable land to be newly reclaimed under the project. The details are as shown in ANNEX-II.

(3) Irrigability

The irrigability is assessed through engineering studies on intake structures and canal systems. The possible maximum height of intake water level has been determined at 35 m in elevation in the Master Plan, in view of topographic conditions of intake site, and the available water. The possible alignment of irrigation canal is then laid out from 35 m intake water level down to the existing paddy fields, with a view to inclusion of more existing paddy fields.

(4) Drainability

The drainability of lands is evaluated in relation to river flows and lake water level fluctuations. The Lake Buaya collects the run-offs and/or drain water from the Bila and Kalola rivers and other small streams. The land locating around Lake Buaya is presently put under poor drainage condition in the rainy season due to the stagnation of the raised lake water. The field investigation on the inundation condition for the area around Lake Buaya revealed that lowlying area with ground levels less than 10 m was inundated for a long period.

The existing paddy fields lying above 10 m in elevation are classified into 2 groups, i.e., (1) well drained and (2) moderately well drained. This condition could be economically improved by provision of drainage system.

6.1.3 Potential area for development

The study area is classified by superimposing the above study results. The potential area for irrigation development is selected to be 11,200 ha as shown in the following table:

Present land use	Main factor of land evaluation	Land capability class	Extent of area (ha)
I. Potential area for development			
1. Existing paddy field	well drained	I	5,900
2. "	moderately well drained, affected with surface water	IIa	3,000
3. "	moderately well drained, affected with ground water	IIg	1,500
4. "	moderately well drained, affected with both surface and ground water	IIga	800
Sub-total			11,200
II. Exclusion from potential development area			
1. Existing paddy field	high permeable soil above possible canal line	IIIℓe	1,600
2. "	poorly drained due to Lake Buaya water	IIIa	900

(To be continued)

Present land use	Main factor of land evaluation	Land capability class	Extent of area (ha)
3. Upland, orchard and village area	high permeable soil	IIIℓ	2,500
4. Forest	shallow soil	IIItdf	900
5. Grass land	heavily shallow soil	IVd	2,900
Sub-total			8,800
Total			20,000

6.2 Alternative Development Plans

6.2.1 Alternative Plans

The land capability study identifies the possible irrigable land endowed in the area of about 10,000 ha in net. The water balance study indicates that the irrigation water for the above possible irrigation area will be sufficiently enough with incorporating the Kalola river flow by constructing the Kalola reservoir. The review and updating of the existing plans show that the Master Plan will command the possible largest extent of the irrigable land but the construction of canal and intake will fall expensive, on the other hand the DOI Plan will serve irrigation of somewhat smaller area but the selected canal route and intake site will economize on their construction.

The development plans on the basis of the existing plans will result in the different sizes of development of water and related land resources. In order to select the optimal development plan of the Project, elaborate technical and economical comparison is made for the following alternative cases:

(1) Case-1 (Proposed Plan) : This aims to serve the large irrigation area of 9,800 ha with the irrigation system based on the built-up DOI design of the intake and canals including additional canal starting from the Kalola dam for commanding the southern elevated area.

(2) Case-2 (Alternative-I) : This aims to serve the possible largest area of 10,000 ha with the irrigation system proposed in the Master Plan incorporating the Kalola dam.

(3) Case-3 (Alternative-II): This aims to serve the irrigation area of 8,500 ha with the irrigation system agreed with the built-up DOI design of the intake and canals incorporating the Kalola dam.

The general layouts of the above plans are as shown in Fig. 6.1 and Fig. 6.2, and the irrigation diagrams of those plans are as shown in Fig. 6.3 and Fig. 6.4. The general feature of the those plans are as shown in Table 6.1, and summarized as follows:

Description		Proposed Plan	Alternatives I	Alternatives II
1. Project area	(ha)	9,800	10,000	8,500
2. Irrigation area				
(1) Wet season	(ha)	9,800	10,000	8,500
(2) Dry season	(ha)	9,800	10,000	8,500
3. Project facilities				
(1) Bila intake				
Basic plan		designed by DOI	proposed in Master Plan	designed by DOI
Intake WEL		30.0	35.0	30.0
(2) Kalola dam				
Total storage	(10^6m^3)	43.0	44.0	35.0
Effective storage	(10^6m^3)	37.0	38.0	29.0
Crest elevation	(m)	42.5	42.5	41.0
Crest length	(m)	230.0	230.0	210.0
(3) Irrigation canal				
Main canal	(km)	46.1	45.7	27.5
Secondary canal	(km)	98.3	74.2	103.3
Tertiary system	(ha)	9,800	10,000	8,500
(4) Drainage canal	(km)	86.5	92.5	81.0
(5) Road	(km)	28.0	29.0	28.0

6.2.2 Basic conditions for evaluation

The evaluation of alternative cases is made by means of economic evaluation in terms of the internal rate of return (IRR).

The project benefit is assessed in the primary increased crop production benefit per annum. This amounts to 942,500 Rp/ha for respective plans, as detailed in Table 6.2. Whereas there exist the paddy fields about 400 ha in the Kalola reservoir area. After completion of the Project, those paddy fields will get into non-productive land under water. The loss of farm land is evaluated as negative benefit by the Project, reducing the aforementioned incremental crop production value. This corresponds to 15,000 Rp/ha per annum.

The construction cost for alternative cases are estimated at 1981 price level based on the preliminary design and quantities taking for the required facilities as shown in Table 6.3. The construction costs include the transfer payment such as custom duty, income tax, cooperation tax, etc., which corresponds to 10% of the direct construction cost. In the estimate of the economic cost, such amount is deducted from the construction cost. The cost required for land acquisition and price contingency are also not incorporated in the estimate of economic cost. The construction costs and the economic costs estimated for alternative cases are as shown in Table 6.4.

6.2.3 Selection of proposed development plan

To evaluate the economic feasibility of alternative plans, the internal rates of return are calculated under the following conditions:

Description	Proposed Plan	Alternatives I	Alternatives II
1. Project area			
(1) Development area (ha)	9,800	10,000	8,500
(2) Irrigation area			
Wet season (ha)	9,800	10,000	8,500
Dry season (ha)	9,800	10,000	8,500
2. Conditions of comparison			
(1) Project life (year from 1983)	50	50	50
(2) Construction period (years from 1983 incl. design and tender)	7	7	7
(3) Build-up period to full development stage (year from 1983)	5 (11)	5 (11)	5 (11)
3. Economic cost and benefit			
(1) Total economic cost (10^6Rp)	35,178	37,836	31,829
(2) Annual net incremental benefit (10^6Rp)	9,552	9,756	8,278
Internal rate of return (IRR) (%)	15.3	15.0	15.0

As shown in the above result, the proposed Plan indicates the highest IRR of 15.3%, followed by Alternatives-I and II. The irrigation development area of the proposed plan reaches to approximately possible maximum irrigable land in the Bila area. That means the large amount of irrigation benefit and the large number of beneficiary can be ensured. This plan will contribute to the regional socio-economic development in the Project area. Consequently, the Bila Irrigation Project would be formulated with Case-1.

7. PRELIMINARY DESIGN OF FACILITIES

7.1 General

The facilities required for the Project include Bila intake structure, Kalola dam, irrigation canal and their related structures, drainage facilities and farm roads. The project facilities should be provided in the most effective and economical manner so that each function can be combined with the fully compatible with farming operation introduced in the Project area. In consideration of the above, the preliminary design of the facilities is carried out as mentioned below.

7.2 Bila Intake

7.2.1 General

The detailed design of the Bila intake weir has been completed by DOI, based on the results of the hydraulic model test. The details of the results of the hydraulic model test and the design are described in the reports titled:

(1) LAPORAN PENYELIDIKAN HIDROLIS DENGAN MODEL, TERHADAP RENCANA BENDUNGAN BILA, JUL. 1, 1977, DIRECTORAT PENYE LIDIKAN MASALAH AIR

(2) FINAL DESIGN BENDUNG BILA, JUL. 1, 1980, PT WECON LTD., DIRECTORAT IRIGASI

With scrutinizing the above mentioned reports and drawings and the technical review of the weir design to be mentioned in the succeeding section, the Bila intake weir design by the DOI is proposed for the intake of the Project as summarized below.

7.2.2 General feature of the Bila intake

(1) Function

The main function of the Bila intake is to divert the required quantity of irrigation water from the Bila river to the Project area of 9,800 ha. In order to fulfil this purpose, the structure will consist of various components such as intake weir, operation bridge, coupure channel and closure embankment. For well functioning as the intake structure, each component will be combined and fully compatible with each other. The details are shown in the attached DRAWINGS.

(2) Features of intake structure

(a) Hydrological and hydraulic conditions

(i) Design flood discharge
The design flood discharge with 100-year return period is taken to be 1,200 m^3/sec as estimated in ANNEX-I.

(ii) Design flood water level
The design flood water level in the upstram section of the weir is calculated to be EL. 34.40 m which will not give effect of inundation in the upstream area.

(iii) Design diversion discharge
The design diversion discharge is determined to be 12.71 m^3/sec consisting of 10.73 m^3/sec for the left main canal and 1.98 m^3/sec for the right main canal.

(iv) Design intake water level
To serve the project area smoothly, the intake water level is set at 30.3 m, which provides the water level of EL. 30.0 m at the canal head.

(b) Geological and foundation condition at weir site

The bedrock exists at the depth of 9 m below the surface. The becrocks are Tertiary Pliocene sedimentary rocks consisting of alternating bedrocks of conglomerate, sandstone and siltstone, overlain by Quaternary Deposits. The permeability of this layer is in the order of 10^{-51}cm/sec.

(c) Design structure features

(i) The intake weir is of cascade type to be constructed with wet stone masonry. The crest elevation of weir is set at EL. 30.3 m. At the middle part of the weir, the scoring sluices provided with stoplogs are constructed.

(ii) The scoring sluices are provided at both ends of the weir. The type of sluice is of under sluice. At the left side, two numbers of sluiceway are constructed with the width of 2.0 m each, and at the right side, one number of sluiceway is provided with the same width. The gate is of wooden gate manual operated type.

(iii) The intakes are provided at both banks. Bottom height of intake is set at EL. 28.70 m for both sides. Three sets of intake gate and one set of intake gate are installed for the left and right side intakes respectively. The design velocity at the intake gate is 1.2 m/sec and 1.5 m/sec for the left and right side intakes respectively.

(iv) The operation bridge is constructed about 25 m upstream of the weir, having a total width of 5.1 m and the total length of 70.0 m supported with four piers.

(v) The closure embankment is of homegeneous eaarth embankment with a crest width of 5.0 m and crest elevation of 36.15 m. The maximum height from the riverbed is 12.65 m. The crest of the closure embankment is connected to the left embankment of intake weir and the bridge.

(vi) The coupure channel is excavated with the base width of 70.0 m and the base gradient of 1:651 corresponding to the riverbase gradient around the weir.

The general features of the Bila intake are summarized below:

(1) Diversion weir

(a)	Tye of weir	Cascade type
(b)	Material of weir	Wet stone masonry
(c)	Crest elevation	EL. 30.3 m
(d)	Water level at canal head	EL. 30.0 m
(e)	Max. diversion discharge	12.71 m^3/sec
(f)	Crest length of overflow weir	47.5 m
(g)	Width of scoring sluce including piers	
	Left side	7.0 m
	Right side	3.5 m
	Centre	12.0 m
(h)	Width of intake	
	Left side (gate size)	8.5 m (2.0 m x 3 Nos.)
	Right side (gate size)	1.3 m (1.3 m x 1 No.)
(i)	Height of weir	
	Upstream weir (from stilling basin)	8.65 m
	Downstream weir (from stilling basin)	9.85 m
(j)	Bridge	
	Total width	5.1 m
	Total length	70.0 m

(2) Closure embankment

(a)	Type of embankment	Homogeneous
(b)	Crest elevation	EL. 36.15 m
(c)	Crest width	5.0 m

(d) Max. height (from riverbed) 12.65 m

(e) Crest length 60 m

7.2.3 Technical review of the design

(1) Design flood discharge

The design flood discharge of the weir was estimated at 1,136 m^3/sec in the existing design. Based on the observed data of the flood of the Bila river, the design flood discharge is estimated at 1,200 m^3/sec with 100-year probability of occurrence as mentioned in ANNEX-I. This is about 6% higher than the originally estimated velue.

(2) Water levels

In order to check the hydraulic properties of the flow over the weir, the water levels in the upstream and downstream sections of the weir are calculated based on the investigation data.

The water level gauging station of the Bila river has been established approximately 1 km downstream from the proposed weir site. At the station the discharge-water level relation has been obtained. In addition, to confirm the discharge-water level relation at the flood condition, the water profile of the Bila river flood is estimated by use of the results of the topographic survey of the Bila river course, as shown in Fig. 7.1.

Based on the above, the water levels at the downstream of the weir for the different discharge are estimated as follows:

Discharge (m3/sec)	1,200	750	400	200
Estimated downstream WBL (m)	28.40	27.50	26.00	25.10

The above calculation shows that the water level in the downstream section at the peak flood seems to be higher than the design water level EL. 25.6 m.

Based on the estimated water level in the downstream section, the water profiles over the weir are also estimated as shown below:

Dischaege	Water level in the middle stilling basin			Water level in the upstream section			
	C	Ht	WEL	C	Le	Ht	WEL
1,200 m^3/sec	2.03	4.14	29.53	2.14	61.5	4.10	34.40
750	2.00	3.07	28.60	2.06	61.8	3.15	33.45
400	1.99	2.02	27.62	1.99	62.0	2.15	32.45
200	1.89	1.31	26.95	1.90	62.2	1.40	31.70

Remarks: C ; Overflow coefficient
Ht ; Total head of flow over the weir
WEL; Waterr level
Le ; Effective length of weir portion

According to the above, the water levels in the middle stilling basin and in the upstream sections would be 0.25 m higher than the desing water levels respectively.

(3) Weir stability against turning, sliding, subsidence and seepage under weir

Under the varied hydraulic conditions as estimated above, the weir stability is examined. The calculation shows that the weir is stable against turning, sliding and subsidence. The seepage length of 90 m of the weir is also long enough for reducing the energy underneath the structure.

(4) Riprap in the downstream section

The intake weir is not provided with the riprap of the downstream section. The flexible riprap of 40 m in length will be needed in the downstream section.

7.2.4 Discussion

The type of intake weir designed by DOI has been determined to be of cascade type through the hydraulic model test for the reason this type would effectively and smoothly dissipate the excess energy of the released water over the weir. The magnitude of the excess energy depends on the difference in water levels between the downstream and upstream sections. The Study Team estimated the higher water level of 28.4 m than the designed water level of 25.6 m in the downstream section at the flood stage, based on the results of the topographic survey of the Bila river channel and the hydraulic and hydrological studies thereon.

The investigation and study do not prove the downstream water level used in the design, even with scrutinizing available documents and data on the weir design and the study results. However, in due consideration of the fact that the existing design of the weir has been confirmed through the hydraulic model test, the design of the weir prepared by DOI was selected as the proposed plan in this stage of the feasibility study.

In case of expectation of higher water level than the adopted value, however, it is likely that (1) the cascade type of weir would be of expensive type under the small difference in the water levels between the upstream and downstream sections, (2) the sidewalls in the downstream basin are required to be heightened and (3) the bottom level of the downstream stilling basin can be raised. It is recommended in the design stage, therefore, that the downstream water level of the weir should be proved.

7.3 Kalola Dam

7.3.1 General

The main function of Kalola dam is to supplement the water deficit of the Bila river and to stabilize the irrigation of the Project. The details are shown in the attached DRAWINGS.

The proposed dam site is selected at a narrow neck of the Kalola river about 10 km upstream of the confluence of the Bila river, in consideration of the following:

(1) Topography and geology of the site

(a) Narrow portion of the river course,

(b) The site having stable baserock,

(c) The site to permit the low dam height and the enough storage capacity and to have large catchment area.

(2) Construction: The site to economize on construction

(3) Operation and maintenance: The site to provide good access for operation and maintenance.

7.3.2 Geological features of the Kalola dam site

According to the results of geological investigation the following conditions were disclosed:

(1) Baserock of the dam site is composed of moderately coarse sandstone and partly conglomerate.

(2) The left bank is formed by a thin ridge and steep slope, composed of mainly coarse sandstone covered with surface soil.

(3) The right bank rises on a cliff of a terrace for about 5 meteres height from the general river water level. The terrace with the width of about 70 m is composed of mainly silty materials and partly sand and gravel deposits which are very loose and soft.

(4) The geological conditions of right bank may be nearly similar to the left bank.

Based on the above finding, the following conditions should be taken into consideration for the dam design:

(1) The fresh and firm baserock is suitable for the foundation of a low height concrete gravity dam and fill type dam.

(2) The highly weathered zone of the baserock and terrace deposits is not adequate for the foundation of any dam type because of the poor shearing strength and the high permeability.

(3) The foundation treatment work, such as curtain grouting, etc. is required since the baserock, even the fresh rock, seems not so hard and less cemented.

(4) The dam height should be limited less than 35 - 40 m by the topographic feature, especially by the height and width of the ridge of left bank abutment.

7.3.3 Selection of dam type

The selection of the type of dam calls for through consideration of the characteristics of each type, as related to the physical features of the proposed dam site and the adaptation to the purposes the dam is supposed to serve, as well as economy, safety, and other pertinent limitation. Among those, the greatest factor determining the final choice of type of dam will be the cost of construction. In this context, the following factors were considered in the choice of type of dam.

(1) topographical conditions

(2) geology and foundation conditions

(3) material available

(4) location of related structures

Based on the results of topographic survey and geological investigation, the choice of dam type is made:

(a) Topography: Fill type dam is proposed due to the slightly wide section for application of concrete dam.

(b) Geology: Fill type dam is also proposed because of poor shearing strength and high permeability of the baserock.

(c) Fill type dam will economize on the construction by using materials excavated from related structures such as spillway, intake, etc.

Consequently, the fill type dam is selected at the Kalola site.

7.3.4 Preliminary design of Kalola dam

(1) Design criteria

The design criteria applied to the preliminary design of dam and related structures are as follows:

(a) "The Design Criteria for Dam" (issued by J.N.C.L.D, 1978)

(b) "The Design Criteria for Dam" (issued by M.A.F.F, 1981)

(c) "Manual for River and Sabo Works" (issued by M.C, 1977)

(2) Design flood

The peak flood discharge and flood hydrograph are analyzed as presented in ANNEX-I. The spillway and river diversion are designed for the following conditions:

Structure	Peak flood discharge	
Spillway	800 m^3/sec	1,000-year flood (equivalent 200-year flood plus 20% allowance)
River diversion	485	20-year flood

The flood hydrograph at the Kalola dam site is as shown in Fig. 7.2.

The probable maximum flood (PMF) is estimated based on the Probable Maximum Rainfall by means of statistical estimates with reference to Manual for Estimation of Probable Maximum Rainfall, WMO 1973. The calculation shows the PMF will be 1,300 m^3/sec as shown in Fig. 7.3.

(3) Storage capacity

The storage will consist of the following:

(a) irrigation requirement

(b) river maintenance flow

(c) domestic use of water

(d) evaporation loss

(e) sedimentation

The storage requirement of the irrigation and the river maintenance is first calculated by means of water balance between the irrigation water requirement, and the Bila river and the Kalola river discharges as shown in Table 5.5.

The design storage required for the above is estimated at 30 x 10^6 m^3 with irrigation dependability level of four out of five years, as shown in Fig. 7.4.

The other storage is estimated on the following basis:

Domestic use of water	:	This is estimated for populatin of 83,900 on the basis of 0.1 m^3/person/day (0.1 m^3/person/day x 83,900 x 150 days)
Exportation loss	:	This is based on pan evaportion measured at Sengkang after converting it to the equivalent evaporation rate from free water surface. (5.1 mm x 8.1 km^2 x 150 days)
Sedimentation	:	The annual sediment volume in the reservoir is determined at 450 m^3/km^2/year from the investigation result, inclusive of allowance. The design year for sediment volume in the reservoir is taken to be 100 year. (450 m^3/km^2/years x 122 km^2 x 100 years)

The storage of the Kalola dam is determind as follows:

(Unit: 10^6 m^3)

Effective storage	37
Irrigation and river maintenance	30
Domestic use of water	1
Evaporation loss	6
Dead storage	6
Sediment volume	6
Total storage	43

(4) Normal high water level and low water level

Normal high water level and low water level are determined to be EL. 36.0 m and EL. 30.0 m respectively as shown in Fig. 5.5.

(5) Crest elevation

The crest elevation of a main dam is determined to be the maximum design water surface plus a free board. The dam crest elevation is given as the highest elevation among the following calculations:

(a) $Hc \geq Hn + hw + he + hi$ or
$Hn + 3.0$ m

(b) $Hc \geqq Hd + hw + hi$ or
$Hd + 2.0$ m

where: Hn; normal high water level (36.0 m)
Hd; flood water level (39.5 m)
hw; height of wave due to wind
he; height of wave due to earthquake
hi; addition of allowance according to type of dam
(fill dam hi = 1.0 m)

In the above, "hw" and "he" is obtained using the following formula:

$$hw = 0.00086\ V^{1.1}F^{0.45} \text{ (method of S.M.B)}$$

$$he = 1/2^{k} \cdot \text{ V g.Ho (S. Sato's formula)}$$

where: V ; wind velocity (m/s) (10 minutes average)
F ; fetch (m)
k ; coefficient of seismicity
; period of seismic wave per second
Ho; depth of reservoir at N.W.L.

As a resut of calculation of the above items, the dam crest elevation at the impervious zone is determined to be EL. 42.5 m.

(6) Type of main dam

In selecting the optimal type of a main dam from the possible types of fill dam, the following consideration and study are made:

(a) Judging from topography and geological condition at the dam site, each type; zoned type, homogeneous type, can be applied.

(b) In order to utilize excavated materials from related structures and to economize on construction, however, the zone type is proposed.

(c) In addition, to ensure the stabilization of dam and efficient quality control of embankment, zoned rockfill dam having central earth core is finally selected.

As mentioned previously, the base rock below the foundation will need the foundation treatment. The foundation treatment to be provided is trench cutoff 8 m depth and grouting. Three rows curtain grouting with depth of 25 m at maximum and five rows blanket grouting with depth of 5 m will be provided.

(7) Spillway

The spillway is proposed on the right abutment of the main dam. The type will be of non-gated side overflow weir spillway provided with shuteway and stilling basis. The spillway is designed so as to release the design flood inflow of 800 m^3/sec at the peak.

The crest length of spillway relates to the height of dam. The optimum size of spillway, therefore, is studied from comparison of combined costs of the dam and spillway with different crest lengths. General feature and mark quantities of studied spillways are as shown in Table 7.1. The result is as shown in Fig. 7.5.

According to the result, the crest length of spillway will be 57 m and the crest elevation will be EL. 36 m, corresponding to the normal high water level. The flood water level will be EL. 39.5 m.

The probable maximum flood of 1,300 m^3/sec will be discharged safety with the overflow depth of 2.04 m above the crest, which is within the height of a freeboard. The reservoir water levels for the period of the design flood and the probable maximum flood are estimated by means of flood routing. The results are as shown in Fig. 7.2.

(8) <u>River diversion</u>

A double line of diversion tunnel is proposed at the left bank to regulate the flood inflow of a 20-year flood of 485 m^3/sec.

The diversion tunnel consists of 115 m and 90 m long circular tunnels with 6 m in diameter. After completion of dam construction, one of them will be used for the irrigation outlet.

(9) <u>Intake</u>

Intake tower is provided at the entrance of the diversion tunnel, capable of releasing the irrigation water of 12.1 m^3/sec. The intake tower will be provided with two sets of sluice gates. The general features of the Kalola dam are as shown below:

(a) <u>Kalola dam and reservoir</u>

(i) General

- Catchment area	122 km^2
- Reservoir surface area at F.W.L.	12 km^2
- Storage capacity	
Total storage capacity	43×10^6 m^3
Effective storage capacity	37×10^6 m^3
Dead water volume	6×10^6 m^3
- Water level	
Flood water level	EL. 39.5 m
Normal high water level	EL. 36.0 m
Low water level	EL. 30.0 m

(ii) Dam

- Type	Rockfill dam having central impervious earth core
- Crest elevation	EL. 42.5 m
- Dam height	30.5 m
- Crest length	230 m

(iii) Spillway

- Type	Non-gated side channel overflow weir
- Design discharge	800 m^3/sec
- Crest elevation	EL. 36.0 m
- Crest length	57.0 m

(iv) Diversion tunnel

- Type	Pressured tunnel
- Design diversion discharge	485 m^3/sec
- Diameter	6.0 m

(v) Intake

- Design discharge	12.01 m^3/sec
- Intake gate	Sluice gate (1.8 m wide x 1.8 m high x 2 Nos.)

7.4 Irrigation Canal System

7.4.1 Design criteria

(1) Function and requirement of canal

Irrigation canal system of the Project consists of main canals, secondary canals and tertiary system. The canal system design is carried out to meet the following function and requirements:

(a) Main canal

In the project area, two main canals are provided; Left Main Canal and Right Main Canal. The main function of the main canal is to deliver irrigation water from two water sources, the Bila and Kalola rivers, to the Project area in the shortest or in the economical way. The canals are basically unlined and trapezoidal.

(b) Secondary canal

This is a canal branching off from the main canal and/or parent secondary canal to distribute water up to the secondary block. The size of secondary block varies from 300 to 4,000 ha which is divided into approximately 5 to 30 tertiary units. The canal is unlined and trapezoidal.

(c) Tertiary system

The tertiary unit includes one tertiary canal and 10 - 15 quaternary canals. The maximum size of tertiary unit is about 150 ha. Whereas, a quaternary canal covers 10 - 15 ha.

Some tertiary units of which size exceeds 150 ha due to topography and/or administrative matters would be divided into subtertiary units.

(2) Design discharge

The unit design discharge for the main and secondary canals is 1.65ℓ/sec/ha as calculated in Section 2.1, which is the water requirement with irrigation dependability level of four out of five years in the dry season cropping.

The design discharge for the tertiary canal is calculated by using the following formula:

$$Q = 5.32 \times a \times A^{2/3}$$

where: Q; Design discharge (ℓ/sec)
a; Unit irrigation water requirement, 1.65ℓ/sec/ha
A; Commanding area (ha)

(3) Velocity

The maximum permissible velocity in unlined canals is determined so as not to give the erosion. The minimum permissible velocity is determined so as not to induce the growth of aquatic plant and moss. Considering the characteristics of soil materials and the conditions of aquatic vegetation, the maximum and minimum permissible velocities are determined as follows:

(Unit: m/sec)

Kind of canal lining	Minimum	Maximum
Earth canal	0.3	0.7
Lined canal	0.3	2.0

(4) Roughness coefficient

The roughness coefficient of canals for determination of their hydraulic properties are as follows:

	N value for manning formula
Earth canal	
$Q \leq 10.0$ m^3/sec	0.020
5.0 m^3/sec $\leq Q \leq$ 10.0 m^3/sec	0.021
$Q <$ 5.0 m^3/sec	0.0222
Sub-secondary canal (Saluran Muka)	0.0235
Tertiary canal	0.025
Lined canal	0.0166
Concrete pipe	0.013
Concrete flume	0.015

(5) Freeboard

The freeboard height is normally subject to canal size and location, velocity, water surface fluctuations caused by check gates and wind action and availability of materials for embankment. The minimum freeboard for the respective canal discharge is determined as follows:

Discharge (m^3/sec)	Freeboard (m)
$Q \leq 0.3$	0.30
$0.3 < Q \leq 0.5$	0.40
$0.5 < Q \leq 1.0$	0.50
$1.0 < Q \leq 7.5$	0.60
$7.5 < Q \leq 15.0$	0.75
$15.0 < Q \leq 25.0$	0.90

(6) Canal base width/water depth (B/h) ratio

The following ratios of B/h are adopted for the respective canal design discharges:

Discharge (m^3/sec)	B/h ratio
$Q \leq 0.3$	1.0
$0.3 < Q \leq 0.5$	1.5
$0.5 < Q \leq 1.5$	2.0
$1.5 < Q \leq 3.0$	2.5
$3.0 < Q \leq 4.5$	3.0
$4.5 < Q \leq 6.0$	3.5
$6.0 < Q \leq 7.5$	4.0
$7.5 < Q \leq 9.0$	4.5
$9.0 < Q \leq 11.0$	5.0
$11.0 < Q \leq 15.0$	6.0
$15.0 < Q \leq 25.0$	8.0

(7) Side slope

The side slopes of canal sections are determined taking into consideration the results of soil mechanical investigation as follows:

Discharge (m^3/sec)	Unlined canal	lined canal
$Q \leq 3.0$	1.0	0.5
$3.0 < Q \leq 15.0$	1.5	1.0
$15.0 < Q$	2.0	1.5

7.4.2 Proposed main and secondary irrigation canals

The proposed irrigation canal system is determined based on the results of the review of the existing canal design prepared by the DOI as mentioned in the succeeding section.

Irrigation canal system in the Project area consists of main canals, connecting canal, secondary, subsecondary canals, tertiary canals and quaternary canals. The proposed layout of the irrigation system is shown in Fig. 6.1. The designs of canals and structures are shown in the attached DRAWINGS.

(a) Main canals

In the Project area, two main canals; Left Main Canal and Right Main Canal, are required to deliver water from the Bila intake to the Project area.

The Left Main Canal will be constructed to serve an area of 6,500 ha located in the left bank of the Bila river. This canal will run approximately southwards from the intake along the skirts of the eastern hill slopes, further, after crossing the kaloia river the canal will run southwords. The total length will be 30.5 km. The alignment of the canal route is made based on the required water level of 30.0 m in elevation. This canal is designed for the discharge of 10.73 m^3/sec at its head.

The Right Main Canal will be constructed to serve an area of 1,200 ha located in the sandwitched small strip between the Bila and the Boya rivers. This canal will run for about 10.5 km from the intake along the skirts of the western hills. The design discharge at its head is about 1.98 m^3/sec.

All the main canal are designed principally as unlined earth canal with trapezoidal cross section of side slope of 1:1.5 to 1.0.

(b) Connecting canal

From the Kalola reservoir, a connecting canal will be constructed to deliver reservoir water to the Left Main Canal approximately. The canal will join the left main canal after the main canal posses. This canal will run towards the South, then stretch to the west along the Kalola rivercourse. The total length will be about 5.1 km. This canal is designed as unlined earth canal with trapezoidal cross section. The design discharge at its head is about 11.72 m^3/sec.

(c) Secondary and sub-secondary canals

These canals will branch off from the above mentioned main canal or secondary canal itself to distribute water to the secondary irrigation units of which size will vary from 1,900 ha to 80 ha depending on topography. About 10 secondary canals and 18 sub-secondary canals with the total length of 98.3 km will be constructed in the Project area. These canals are designed principally as unlined earth canal with trapezoidal cross section of side slope of 1:1.5 to 1.0.

The typical cross sections of those canals adopted in this design are shown in Fig. 7.6.

The number and the total length of the main, connecting, secondary, sub-secondary canals and the number of their related structures are as shown below:

Description		Left bank area Main	Connecting	Right bank area	Total
Main and connecting canals					
- Canal length	(km)	30.5	5.1	10.5	46.1
- Related structure					
Turnout w/check	(nos.)	28	3	11	42
Culvert	(nos.)	2	-	-	2
Spillway	(nos.)	3	-	2	5
Drop	(nos.)	1	3	2	6
Cross drain	(nos.)	45	-	5	50
Syphon		1	-	-	1
Measuring device	(nos.)	1	1	1	3
Secondary and sub-secondary canals					
- Canal length	(km)	93.2		5.1	98.3
- Related structure					
Turnout w/check	(nos.)	67		1	68
Culveret	(nos.)	6		-	6
Spillway	(nos.)	9		1	10
Cross drain	(nos.)	29		-	29
Drop	(nos.)	6		2	8
Syphon	(nos.)	3		-	3

7.4.3 Technical review of canal design prepared by DOI

(1) Irrigation area

The boundary of the irrigation area in the low-lying area around Lake Buaya is established at the ground level ten (10) m, differing from the existing design being set at the ground level seven (7) m. This varies the irrigation area commanded at the respective canal sections. In addition, the soil survey was conducted over the Project area and it revealed the land capability for paddy cropping. Based on those findings the command areas at the canal sections are re-estimated as shown in Fig. 7.7.

(2) Canal layout

The longitudinal profile drawings for the main and secondary canals have been prepared based on the route survey results. Most of those alignments are used for the proposed canal system. The improvements on the above canal system are made to meet the requirement of the proposed canal system; (1) the irrigation area directly commanded by the Bila intake should be limited to 2,600 ha according to the result of water balance, (2) the canals contemplated in the low-lying area lower than the ground level 10 m are neglected and (3) in order to serve the southern slopes from the Kalola dam, which are located inlands from the DOI canal route, the additional canal along the foot of such slopes is required.

(3) Canal capacities

Based on the unit irrigation water requirement 1.65 ℓ/sec/ha and the re-estimated commanding areas at each canal section, the canal capacities of the original design were examined whether to meet the varied conditions. The comparison of the canal capacity between the original and revised ones is as shown in Fig. 7.8, showing together with the original estimate. Hydraulic properties, such as canal dimentions, velocity, gradient, etc. were compared between the revised and original ones as shown in Table 7.2. According to the above table, the upstream reaches of the left main canal, about 10 km in length, have an allowance in the flow capacity. Whereas, the remainings fall in the lack of the capacity. Therefore, the canal sections should be modified to suit the varied conditions.

7.5 Drainage Canal System

7.5.1 Function and requirement of drainage canal

This drainage canal system consists of major, tertiary and quaternary drains. The tertiary and quaternary drainage canals are described in Section 7.7. Herein mentioned is the design of major drainage canals. The function of the major drain is that major drains transport water from tertiary drains and flood water from surrounding mountainous areas to the disposal points. The layout of the irrigation system and topography are the main factors in determining the location of all the drainage canals. The location of major drain is dominated by natural streams and rivers existed in the project area. These natural streams and rivers are used as much as possible as the major drains.

7.5.2 Layout planning of drainage canal

The layout planning of drainage canals is carried out through the following procedure:

(1) Establishment of basic concept

First of all, the following basic concepts for planning the drainage system are examined:

(a) Where should the major drainage canal to disposal points be located?

(b) How will the excess water in the area be collected and transported to the disposal points?

(2) Field survey on drainage planning

The following items relating to drainage planning were investigated over the Project area:

(a) Field damage due to ill-drainage is surveyed for its extent and magnitude.

(b) Present drainage mechanism is observed in the project area and in its vicinal areas.

(c) Reconnaissances along the Kalola and other rivers are made to examine the highest flood water level in the past and their present flow capacities.

(d) Analysis is made for examination of the intensity and duration of rainfall in the area and for the estimate of drainage requirements.

(e) Present land use in the area is surveyed for analysis on drainage requirements.

(f) Soil characteristics in the area is surveyed.

(g) The field surveys on the present farming practices, socio and agroeconomic conditions are carried out in the project area and in its vicinal areas.

(3) Drainage canal layout

The drainage canal layout is worked out primarily based on the topographic maps and layout of canal system prepared by DOI, supplemented by the topographic maps of 1:25,000. In determination of the drainage canal layout, the following matters are taken into consideration:

(a) Drainage water requirements, drainage method, required canal elevations at key points and general layout of drainage system are first confirmed.

(b) Drainage canal routes are laid out along the low land and as straight as possible.

(c) The alignment is worked out so as not to pass through village areas and not to give damages to public facilities.

(d) Raised portions of drains are minimized in order to keep canal water level below ground surface as much as possible.

(e) The canal alignment thus obtained is confirmed weather the alignment will satisfy the operational and social requirements or not.

7.5.3 Design of drainage canal system

(1) Design discharge

The design discharge of the drainage canal at respective cross sections consists of:

(a) drainage requirement for removal of excess rainfall in the paddy fields, Qp.

(b) drainage requirement for passing the high flow coming from the outside project area, Qo.

The design discharge, Qd, is determined by the following basis including the allowance of capacities:

$$Qd = 1.15 \times (Qp + Qu)$$

The drainage diagram showing the drainage area and the design discharge is as shown in Fig. 7.9.

(2) Canal section

The drainage canal sections are designed for the following criteria:

Type of canal	:	Trapezoidal earth canal
Permissible velocity		
Maximum velocity	:	0.6 m/sec
Minimum velocity	:	0.3 m/sec
Roughness coefficient for the use of manning's formula	:	0.03
Side slope of canal	:	1:1.0

(3) Related structures

The structures related to the drainage system are bridges, drops and drainage junctions.

The bridge is provided at the road crossing. The maximum span length is 8 m and concrete T-bean type is applied.

The drops are of cascade type with trapezoidal section. The gabion mattresses are used for the drainage drops.

The drainage junctions are provided at the connecting points of major drains to protect drains from bed erosion.

7.5.4 Proposed drainage canal system

The drainage canal system is networked so as to evacuate the excess water in the fields and to transport the stream flows occurred in the outside project area to the Bila river, the Kalola river and Lake Buaya. The drainage system will consist of major drains, tertiary drains and quaternary drains. The major drains are designed to collect water from tertiary drains and quaternary drains which are constructed within the tertiary blocks, and to transport collected water inclusive of stream flows to the above rivers and lake. The routes of major drains are generally selected in the natural stream lines and low depressions. For the project, about 49 major drains with total length of 86.5 km will be excavated. The drainage layout of the Project is shown in the attached DRAWINGS. The typical cross sections of drainage canal adopted in this design are shown in Fig. 7.6.

The total required canal length and the number of the related structures are as shown below:

Description			Left bank area	Right bank area
Major drains				
(1)	Canal length	(km)	80.6	5.9
(2)	Related structure			
	(a) Drop	(nos.)	121	8
	(b) Junction	(nos.)	16	1
	(c) Bridge	(nos.)	5	-

7.6 Farm Road Networks

For the proper construction, operation and maintenance of the Project facilities, well arranged road network is of vital importance. The project area is presently put under the poor road conditions during the wet season. For the construction purposed the selected village roads transversing the area are improved, which will be transferred to the village link roads after the Project implementation. Main, secondary and tertiary irrigation canals are provided with canal inspection roads. Those roads are also used for farm roads.

(1) Construction road

The construction roads of 28 km long in total area constructed and well networked with the national road. The road is design as to have a width of 6.0 m with asphalt pavement. Catch drains are provided on both sides along the road to carry water of rainfall on the road surface and the surrounding into drainage canals. Catch basins are also constructed under the road at approximately 500 m intervals to pass drainage water in the catch drain.

(2) Main inspection road

The main inspectin roads are required for inspection, operation and maintenance of the main canals.

Considering the future increas of vehicles for the inspection and operation and heavy construction equipment to be required for the canal maintenance and repair, the main inspection road is designed so as to have a width of 6.0 m and to be asphalt-surfaced. The road is also used for the movement of agricultural products and equipment and for the day-to-day services between villages and from them to the trunk road.

(3) Secondary inspection road

The secondary inspection road is mainly provided alongside the secondary canals. All these roads have a width of 6 m and are paved with asphalt. These roads link the paddy fields to the main road and are used for the purpose of farm operation.

(4) Tertiary inspection road

For the same purpose as that of the secondary farm roads, the tertiary farm roads are constructed along one side of all the tertiary canals. These roads have a width of 3 meters and is of earth without any metalling.

The following table shows the respective road length:

(Unit: km)

Roads	Length
Construction road	28.0
Main inspection road	46.1
Secondary inspection road	98.3
Tertiary inspection road	294.0

7.7 Tertiary Development

7.7.1 General

Tertiary development program aims at efficient water management by establishing well organized tertiary system and through refined rotational irrigation program. For this subject, the Directorate of Irrigation has prepared the report titled as "Guideline Manual for Planning of Tertiary Network". The design and operational programming for the tertiary system of the Project are based on this guideline manual.

The tertiary development progarm is prepared for every tertiary block. This tertiary block is further divided into several subordinate blocks like sub-tertiary blocks and quaternary blocks. The definition and the recommended size of each irrigation block are briefed as follows:

(1) Tertiary block

The tertiary block is covered by one tertiary canal. The distribution of irrigation water in the tertiary block is managed by farmers themselves. In some cases, however, it is difficult for farmers to manage the distribution of water to vast land and large number of farmers equally. Considering the appropriate organization of water users' group in future, the maximum size of tertiary block is proposed to be 150 ha.

(2) Sub-tertiary block

The tertiary block is somewhere divided into several sub-tertiary blocks depending on the topographic and/or administrative boundaries of villages to simplify the irrigatin system and the organization of water users' association.

(3) Quaternary block

In order to distribute irrigation water equally and efficiently to all parts of the fields through more intensive water control, it is advisable to sub-divide the tertiary block into several sub-tertiary blocks and the quaternary blocks. The quaternary block is served by respective quaternary canals. The recommended size of one quaternary block is 10 to 15 ha. The rotational irrigation is practiced on the quaternary basis.

The typical alyout of tertiary system is shown in Fig. 7.10. The layouts of tertiary systems of the represntative are shown in Fig. 7.11 and Fig. 7.12.

7.7.2 Tertiary irrigation system

(1) Canal system

The tertiary system consists of tertiary canal, sub-tertiary canals and quaternary canals which respectively cover the tertiary block, sub-tertiary blocks and quaternary blocks as mentioned above. The following respective function and design principle are taken into considertion.

(a) Tertiary canal

The tertiary canal delivers irrigation water from secondary irrigation canal or sometimes directly from main canal to the subtertiary canals and/or quaternary canals. The irrigation water should not be given directly to fields from the tertiary canal.

For the alignment of these canals in the area with steep topography, the canal should be perpendicular to the contour line (perpendicular type).

(b) Sub-tertiary canal

The sub-tertiary canal leads irrigation water from the tertiary canal to the quaternary canals. In this case also, irrigation water should not be given directly to fields from this canal. In principle, the alignment of this canal is made in the same manner as that of the tertiary canal.

(c) Quaternary canal

The quaternary canal is terminal system. Irrigation water to be carried by this canal is distributed to fields directly. The end of quaternary canal is connected to nearby drainage canal so as to drain off excess water in the canal. Especially in steep slopes, the canal should be aligned in parallel to the contour line (contour type). The average interval of quaternary canal is limited to 200 m at maximum. All the quaternary canals except the canal to be constructed in the highest position in the respective area are so designed as to have dual function; irrigation and drainage functions, where possible.

(2) Related structure

In order to attain its primary objective, the canal system thus aligned requires the following structures:

(a) Tertiary division box

Many division boxes are constructed on the tertiary canals and all of them are equipped with stoplogs to regulate irrigation water in accordance with the rotational irrigation program.

(b) Quaternary division box

All the division boxes to be constructed on the quaternary canal are equipped with stoplogs.

(c) Measuring device

The measuring device as Cipoletti weir is installed at the head of tertiary block.

(d) Drop structure

A drop structure is provided whre the ground surface slope is steeper than the required canal gradient. In principle, the drop structure is not provided on the canal system as an independent structure but as combined structure with division box.

(e) Culvert

A culvert is constructed at the crossing of canal with road. This structure is of combined type with the division box as far as possible.

(f) Crossdrain

A crossdain is provided where the irrigation canal has to cross over the drainage canal.

7.7.3 Tertiary drainage system

(1) Drainage canal system

In the tertiary block, the quaternary drains and tertiary drains are required to evacuate excess water from the block. In the layout planning of these drainage canals, the following respective function and design principle are taken into consideration:

(a) Quaternary drain

Quaternary drains are provided to collect excess water in the quaternary block and drain off the water to the tertiary drain. Generally, the quaternary canal has dual functions, then, the quaternary drain is not provided independently.

(b) Tertiary drain

Tertiary drains are provided to lead the excess water to be collected by the quaternary drains in the tertiary block to the major drain or directly to the river.

(2) Related structure

In order to facilitate the proper function to the drainage system mentioned above, the following structures are required on the canals:

(a) Drainage drop structure

This structure is placed where the natural ground slope is steeper than the designed gradient of drain bed.

(b) Drainage culvert

A drainage culvert is provided where the drainage canal will cross under the road. For crossing, the precast concrete pipe is installed.

The total length of tertiary and quaternary irrigation canals, tertiary drains, and tertiary inspection roads are as shown below:

7.8 Further Investigation

Project facilities mentioned above are designed on the basis of results of the preliminary investigations conducted by the survey team and D.O.I's design. In order to perform detailed design of the project facilities and realize the project by means of construction of the facilities, further investigations on topography, geology, and soil mechanics at the proposed sites of major facilities and detailed survey on construction materials should be carried out. In addition to the further investigtions, the aerial photo maps which would be great use for the detailed design and construction of the facilities in the tertiary systems should be prepared.

Specifications and work quantities of the further investigations and aerial photo mapping are shown as follows.

(1) Topographic survey

Facilities	Quantity	Description
1. Bila Intake	50 ha	preparation of contoured maps, scale in 1/500 with 0.25 m contour interval
2. Kaiola Dam	70 ha	preparation of contoured maps, scale in 1/500 with 0.25 m contour interval
3. Irrigation canal		
Main Canal	41.0 km	check survey includ. centering and levelling on the route
Connecting Canal	5.1 km	Centering and levelling on the route, and cross levelling every 50 m along the route with 100 m width
Secondary Canal	98.3 km	check survey includ. centering and levelling on the route
Tertiary System	9,800 ha	centering and levelling on the route of tertiary canals and drains, cross levelling every 100 m with 50 m width
4. Major Drainage Canal	86.5 km	centering and levelling on the route, cross leveling every 100 m with 100 m width
5. Construction Road	28 km	centering and leveling on the route, cross leveling each 100 m with 50 m width

(2) Geological investigation

(a) Bila intake site; - drilling investigation at the intake weir and closure dam sites, 5 holes of each 20 m deep (depth of 100 m in total)

(b) Kalola dam site; - drilling investigation on the axis of main dam, 4 holes of each 25 m deep, (depth of 100 m in total)

- pit investigation on the axis of main dam, 9 test pits of each 3 m deep (depth of 27 m in total)

(3) Soil mechanical investigation

(a) Sounding test of foundation at the canal structure sites
- main canal and connecting canal route; 40 sites
- secondary canal route ; 80 sites

(b) Test pit observation on the main canal and connecting canal route with soil sampling and laboratory tests ; 20 sites

(4) construction material survey

The requireds volumes and proposed quarry sites of the construction materials for the Kalola dam are shown in the following table.

Materials	Characteristics t	Characteristics ø	Soil Classification	Required Volume	Proposed Quarry Site
	(cm/s)	(°)		(m^3)	
Core	1^{-8} - 1^{-6}	20	CH	42,000	- right bank on the lower reaches of the Kalola river
Filter	-	33	GW	20,000	- river base deposit of the Kalola river
Randam Rock	-	35	-	67,000	- excavated materials at the dam site
Rock	-	35	-	134,000	- near the dam site

t: Coefficient of permeability
o: Angle of internal friction

In order to confirm the results of preliminary investigations on the above construction materials, drilling investigations, geophysical exploration and pit observations should be carried out at the proposed quarry sites and borrow area. Specifications and work quantities of geological investigations are roughly described as below.

(a) Rock and randam rock materials;
- 5 cross section survey,
  drilling investigation, 5 holes of each 20 m deep (depth of 100 m in total)
- geophysical exploration (seismic survey)

(b) Core and filter materials
- pit observation, 10 pits of each 5 m deep (depth of 50 m in total)

(5) Aerial photo mapping

Preparation of aerial photo maps on the area of 20,000 ha, scale in 1 to 5,000 with 0.5 m contour interval.

Table 1.1 Existing Irrigation System

No.	Name of System	Location Kabupaten	Irrigation Area (ha)	Irrigation Facilities Intake	Main Canal (m)	Secondary Canal (m)	Tertiary Canal (m)	Drainage Canal (m)	Classification of Irrigation System
1.	Salodua	Wajo	500	Masonry weir	5,000	-	60	-	DPU irrigation, semi-technical
2.	Callacu I	Wajo	125	Masonry intake	2,500	-	-	-	Desa irrigation, semi-technical
3.	Callacu II	Wajo	100	Masonry intake	3,000	-	-	-	Desa irrigation, semi-technical
	Sub-total		725		10,500	-	60	-	
4.	Kalosi	Sidrap	250	Masonry weir	3,000	-	-	-	Desa irrigation, semi-technical
5.	Jampai	Sidrap	40	Masonry weir	950	-	-	-	Desa irrigation, semi-technical
6.	Kannung	Sidrap	200	Masonry weir	2,300	-	-	-	Desa irrigation, semi-technical
	Sub-total		490		6,250	-	-	-	
	Total		1,215		16,750	-	60	-	1-DPU irrigation, semi-technical 5-Desa irrigation, semi-technical

Table 2.1 Climatic Data of Sengkang and Kanyuara (1/6)
(Mean Monthly Air Temperature)

Sengkang

(Unit: °C)

Year	Jan.	Feb.	Mar.	Apr.	May	Jun.	Jul.	Aug.	Sep.	Oct.	Nov.	Dec.	Ave.
1975	-	-	-	-	26.9	26.3	26.1	25.9	27.1	27.4	28.0	27.8	-
1976	27.9	28.4	28.3	27.8	27.6	25.1	24.8	25.3	26.0	29.2	28.4	28.5	27.3
1977	28.7	29.0	28.0	28.7	29.0	27.0	27.0	27.6	26.9	28.0	28.0	27.0	27.9
1978	27.5	28.0	27.5	27.3	27.6	27.8	26.1	26.3	26.6	28.7	27.1	27.2	27.3
1979	27.4	27.2	27.6	27.2	26.6	26.2	25.7	26.6	27.0	27.7	28.2	27.4	27.1
1980	28.0	27.4	27.3	27.2	26.9	26.4	25.8	26.2	27.1	28.2	27.9	27.4	27.2
1981	27.3	27.5	27.3	27.2	27.2	27.0	-	-	-	-	-	-	-
Aver-age	27.8	27.9	27.7	27.6	27.4	26.5	25.9	26.3	26.8	28.2	27.9	27.5	27.3

Kanyuara

Year	Jan.	Feb.	Mar.	Apr.	May	Jun.	Jul.	Aug.	Sep.	Oct.	Nov.	Dec.	Ave.
1975	26.7	26.7	27.9	27.8	27.0	26.3	26.0	26.4	26.9	27.9	27.8	26.9	27.0
1976	26.7	27.1	27.5	27.4	27.0	26.1	25.9	26.4	27.0	27.5	27.6	26.9	26.9
1977	27.4	27.1	27.4	27.4	27.4	26.7	26.8	27.1	27.5	27.1	28.3	26.9	27.3
1978	26.9	27.1	26.6	27.6	28.7	25.7	27.0	25.8	26.8	27.2	26.6	27.9	27.0
1979	26.1	26.2	26.5	28.6	26.8	26.4	26.4	26.9	27.3	27.8	27.8	27.4	27.0
1980	27.2	27.2	27.7	27.7	26.5	27.3	26.8	26.3	28.5	28.3	27.3	28.0	27.4
1981	26.6	26.2	27.1	27.4	26.9	26.7	-	-	-	-	-	-	26.8
Aver-age	26.8	26.8	27.2	27.7	27.2	26.5	26.5	26.4	27.3	27.6	27.6	27.3	27.1

Table 2.1 Climatic Data of Sengkang and Kanyuara (2/6)
(Mean Monthly Relative Humidity)

Sengkang (Unit: %)

Year	Jan.	Feb.	Mar.	Apr.	May	Jun.	Jul.	Aug.	Sep.	Oct.	Nov.	Dec.	Ave.
1975	-	-	-	-	78.3	78.7	75.7	75.2	74.8	75.5	72.2	71.9	-
1976	70.7	66.1	68.0	69.8	73.2	75.5	75.0	68.4	60.7	63.4	68.7	65.2	68.7
1977	66.7	65.0	62.0	68.0	69.0	80.0	72.8	71.3	80.7	74.0	81.0	88.0	73.2
1978	83.7	80.4	86.6	84.9	87.2	87.4	86.8	84.2	85.4	86.4	84.0	90.0	85.6
1979	86.4	90.8	86.8	92.9	85.5	87.3	84.7	76.8	80.2	65.9	77.0	89.1	83.6
1980	86.4	89.6	85.1	89.0	87.5	81.9	85.6	84.0	76.2	84.7	90.8	89.3	85.8
1981	72.1	85.7	88.4	86.8	88.3	-	-	-	-	-	-	-	-
Aver-age	77.7	79.6	79.5	81.9	81.3	81.8	80.1	76.7	76.3	75.0	79.0	82.3	79.2

Kanyuara

Year	Jan.	Feb.	Mar.	Apr.	May	Jun.	Jul.	Aug.	Sep.	Oct.	Nov.	Dec.	Ave.
1975	96	98	96	80	82	79	95	95	93	89	86	88	90
1976	90	89	90	91	91	93	95	94	96	97	96	96	93
1977	95	97	97	92	91	94	97	97	97	96	84	94	94
1978	96	99	99	98	99	89	87	84	85	82	84	89	91
1979	88	84	83	83	85	93	89	82	88	82	81	83	85
1980	86	83	82	85	86	86	86	86	77	79	81	88	84
1981	87	88	86	88	90	88	-	-	-	-	-	-	-
Aver-age	91	91	90	88	89	89	92	90	89	88	85	90	90

Table 2.1 Climatic Data of Sengkang and Kanyuara (3/6)
(Mean Monthly Sunshine Duration)

Sengkang

(Unit: hrs, hrs/day)

Year	Jan.	Feb.	Mar.	Apr.	May	Jun.	Jul.	Aug.	Sep.	Oct.	Nov.	Dec.	Annual
1975	-	-	-	-	-	139	179	209	221	215	218	155	-
	-	-	-	-	-	4.6	5.8	6.8	7.4	6.9	7.3	5.0	-
1976	194	204	188	210	188	162	200	262	260	213	210	169	2,460
	6.3	7.3	6.1	7.0	6.1	5.4	6.5	8.4	8.7	6.9	7.0	5.5	6.7
1977	147	117	192	185	201	145	206	229	279	307	231	174	2,413
	4.7	4.2	6.2	6.2	6.5	4.8	6.6	7.4	9.3	9.9	7.7	5.6	6.6
1978	155	145	162	169	211	178	198	194	183	254	194	161	2,204
	5.0	5.2	5.2	5.6	6.8	5.9	6.4	6.3	6.1	8.2	6.5	5.2	6.0
1979	155	149	145	183	230	144	216	272	231	244	188	154	2,311
	5.0	5.4	4.7	6.1	7.4	4.8	7.0	8.8	7.7	7.9	6.3	5.0	6.3
1980	124	132	169	148	177	162	226	206	266	215	177	138	2,140
	4.0	4.6	5.5	5.0	5.7	5.4	7.0	6.7	8.9	7.0	5.9	4.4	5.8
1981	159	141	195	179	183	204	-	-	-	-	-	-	-
	5.0	5.0	6.3	5.9	5.9	6.8	-	-	-	-	-	-	-
Average	156	148	175	179	198	162	204	228	240	241	203	158	2,292
	5.0	5.3	5.7	6.0	6.4	5.4	6.6	7.4	8.0	7.8	6.8	5.1	6.3

Table 2.1 Climatic Data of Sengkang and Kanyuara (4/6)
(Mean Monthly Sunshine Distillation)

Kanyuara

(Unit: ml)

Year	Jan.	Feb.	Mar.	Apr.	May	Jun.	Jul.	Aug.	Sep.	Oct.	Nov.	Dec.	Annual
1975	16.5	15.9	18.6	15.8	15.3	14.0	14.1	14.8	15.8	19.9	18.3	16.7	195.7
1976	15.3	16.6	16.4	18.2	17.2	16.4	19.2	17.3	17.2	19.0	18.2	15.4	206.4
1977	14.8	15.3	18.4	17.7	17.5	16.6	16.8	17.5	18.9	18.4	18.8	20.2	210.9
1978	16.6	15.7	16.2	13.3	17.5	16.9	16.5	15.9	17.5	19.5	17.4	15.4	198.4
1979	18.4	17.7	15.8	17.8	17.5	13.9	14.5	15.0	15.9	18.2	17.5	17.4	199.6
1980	19.1	18.9	20.4	16.7	16.6	15.8	15.6	15.1	18.9	17.4	18.3	14.8	207.6
1981	16.8	16.1	19.4	19.2	17.9	17.6	-	-	-	-	-	-	-
Average	16.8	16.6	17.9	17.0	17.1	15.9	16.1	15.9	17.4	18.7	18.1	16.7	204.2

Table 2.1 Climatic Data of Sengkang and Kanyuara (5/6)
(Mean Monthly Wind Velocity)

Sengkang (Unit: m/sec)

Year	Jan.	Feb.	Mar.	Apr.	May	Jun.	Jul.	Aug.	Sep.	Oct.	Nov.	Dec.	Annual
1975	-	-	-	-	-	-	-	1.0	0.8	0.6	0.7	1.4	-
1976	1.3	1.5	1.5	1.1	1.3	1.2	1.5	1.8	1.6	1.3	0.9	1.5	1.4
1977	1.5	2.0	1.3	1.1	1.0	1.4	1.8	2.0	2.0	2.0	1.4	1.2	1.5
1978	1.2	1.2	1.0	1.1	1.1	1.0	1.1	1.4	1.3	1.3	1.0	1.3	1.1
1979	1.2	1.2	1.1	1.0	0.96	1.3	1.7	1.9	1.8	1.3	1.2	1.3	1.3
1980	1.5	1.3	1.3	0.9	1.7	1.3	1.4	1.5	1.7	1.5	1.4	1.3	1.4
1981	1.8	1.3	1.1	0.95	1.1	0.9	-	-	-	-	-	-	-
Average	1.4	1.4	1.2	1.0	1.2	1.2	1.5	1.6	1.5	1.3	1.1	1.3	1.3

Kanyuara

Year	Jan.	Feb.	Mar.	Apr.	May	Jun.	Jul.	Aug.	Sep.	Oct.	Nov.	Dec.	Annual
1975	2.5	2.2	2.9	2.5	2.5	2.4	2.3	2.0	2.4	1.9	2.6	2.9	2.4
1976	2.7	2.4	2.4	2.3	2.6	2.2	2.2	2.8	3.2	2.6	1.8	2.4	2.5
1977	2.1	3.3	2.3	2.2	1.8	2.0	2.5	2.9	3.2	3.0	1.4	1.6	2.4
1978	1.7	1.8	1.6	2.1	1.7	2.8	1.8	2.3	2.1	2.5	1.7	1.9	2.0
1979	1.9	1.6	2.1	1.9	1.4	1.8	2.0	2.7	2.4	1.8	1.8	1.8	1.9
1980	1.6	1.4	2.0	1.9	2.6	2.1	1.7	2.2	2.3	2.3	1.5	2.1	2.0
1981	2.3	1.4	1.6	1.8	2.7	1.5	-	-	-	-	-	-	-
Average	2.1	2.0	2.1	2.1	2.2	2.1	2.1	2.5	2.6	2.3	1.8	2.1	2.2

Table 2.1 Climatic Data of Sengkang and Kanyuara (6/6)
(Monthly Evaporation)

Sengkang (Unit: mm/month)

Year	Jan.	Feb.	Mar.	Apr.	May	Jun.	Jul.	Aug.	Sep.	Oct.	Nov.	Dec.	Annual
1975	-	-	-	-	-	116	128	159	174	135	187	159	-
1976	185	184	185	158	145	113	143	180	206	197	158	168	2,022
1977	168	177	188	131	159	110	134	175	240	303	227	181	2,193
1978	176	176	175	169	140	135	138	160	154	164	148	145	1,880
1979	163	176	153	136	131	119	163	209	186	210	185	122	1,953
1980	189	162	176	142	134	109	145	164	225	222	172	288	2,128
1981	188	134	214	134	126	-	-	-	-	-	-	-	-
Average	178	168	182	145	139	117	141	174	197	205	180	177	2,003

Kanyuara

Year	Jan.	Feb.	Mar.	Apr.	May	Jun.	Jul.	Aug.	Sep.	Oct.	Nov.	Dec.	Annual
1975	175	192	219	167	196	202	192	247	135	141	190	172	2,228
1976	151	203	156	160	183	137	148	213	172	227	182	187	2,119
1977	164	130	149	148	161	139	144	142	180	204	177	141	1,879
1978	150	171	159	174	128	92	108	150	162	196	173	157	1,820
1979	176	155	202	184	170	121	158	224	205	275	229	212	2,311
1980	220	220	255	174	167	147	174	193	281	268	219	156	2,474
1981	181	153	188	176	170	146	-	-	-	-	-	-	-
Average	174	175	190	169	167	141	154	195	206	219	195	171	2,156

Table 2.2 Potential Evapotranspiration at Senkang (1980)

Data (Monthly mean)	Jan.	Feb.	Mar.	Apr.	May	Jun.	Jul.	Aug.	Sep.	Oct.	Nov.	Dec.
1. Temperature (°C); T	28.0	27.4	27.3	27.2	26.9	26.4	25.8	26.2	27.1	28.2	27.9	27.4
2. Relative humidity (%); RH	86.4	89.6	85.1	89.0	87.5	81.9	85.6	84.0	76.2	84.7	90.8	89.3
3. Sunshine duration; n/N	0.33	0.37	0.45	0.41	0.48	0.46	0.62	0.55	0.74	0.57	0.48	0.36
4. Wind velocity (km/day); U	129.6	112.3	112.3	77.8	146.9	112.3	121.0	129.6	146.9	129.6	121.0	112.3
Calculation												
5. Saturation vapour pressure (m & r); ea	37.8	36.5	36.3	36.1	35.5	34.4	33.3	34.0	35.9	38.3	37.6	36.5
6. Actual vapour pressure (m & r); ed	32.7	32.7	30.9	32.1	31.1	28.2	28.5	28.6	27.4	32.4	34.1	32.6
7. Difference in vapour pressure (m & r); (ea-ed)	5.1	3.8	5.4	4.0	4.4	6.2	4.8	5.4	8.5	5.9	3.5	3.9
8. Wind function; f(u)	0.6	0.6	0.6	0.5	0.7	0.6	0.6	0.6	0.7	0.6	0.6	0.6
9. Weighting factor for U & RH; 1-W	0.23	0.24	0.24	0.24	0.24	0.25	0.25	0.25	0.24	0.23	0.23	0.24
10. Extra tervestial radiation (mm/day); Ra	15.5	15.8	15.6	14.9	13.8	13.2	13.4	14.3	15.1	15.6	15.5	15.4
11. Solar radiation (mm/day); Rs	6.4	6.9	7.4	6.8	6.8	6.3	7.5	7.5	7.9	8.3	7.6	6.6
12. Net short wave radiation (mm/day); Rns	4.8	5.2	5.6	5.1	5.1	4.7	5.6	5.6	5.9	6.2	5.7	5.0
13. Effect on long wave radiation												
(a) f(t) for T	16.3	16.2	16.2	16.1	16.1	16.0	15.9	15.9	16.1	16.3	16.3	16.2
(b) f(ed) for ed	0.09	0.09	0.10	0.09	0.09	0.11	0.11	0.10	0.11	0.09	0.08	0.09
(c) f(n/N) for n/N	0.40	0.43	0.51	0.47	0.53	0.51	0.51	0.60	0.77	0.61	0.53	0.42
14. Net long wave radiation (mm/day); Rnl (13.a) x (13.b) x (13.c)	0.6	0.6	0.8	0.7	0.8	0.9	0.9	1.0	1.4	0.9	0.7	0.6
15. Net radiation (mm/day); Rn	4.2	4.6	4.8	4.4	4.3	3.8	4.7	4.6	4.5	5.3	5.0	4.4
16. Weighting factor for Rn; W	0.77	0.76	0.76	0.76	0.76	0.75	0.75	0.75	0.76	0.77	0.77	0.76
17. Adjustment factor; C	1.00	1.02	1.04	1.01	1.01	0.99	1.04	1.04	1.06	1.08	1.05	1.00
18. Potential evapotranspiration (mm/day); PET												
(a) Radiation term (15) x (16)	3.2	3.5	3.6	3.3	3.3	2.9	3.5	3.5	3.4	4.1	3.9	3.3
(b) Aerodynamic term (7) x (8) x (9)	0.7	0.5	0.8	0.5	0.7	0.9	0.7	0.8	1.4	0.8	0.5	0.6
(c) PET; (17) x [(18.a) + (18.b)]	3.9	4.1	4.6	3.8	4.0	3.8	4.4	5.3	5.1	5.3	4.6	3.9

Table 2.3 Potential Evapotranspiration at Kanyuara (1980)

Data (Monthly mean)	Jan.	Feb.	Mar.	Apr.	May	Jun.	Jul.	Aug.	Sep.	Oct.	Nov.	Dec.
1. Temperature (°C); T	27.2	27.2	27.7	27.7	26.5	27.3	26.8	26.3	28.5	28.3	27.3	28.0
2. Relative humidity (%); RH	86	83	82	85	86	86	86	86	77	79	81	88
3. Radiation (ml/day); Rc	19.1	18.9	20.4	16.7	16.6	15.8	15.6	15.1	18.9	17.4	18.3	14.8
4. Wind velocity (km/day); U	141.8	120.9	170.8	160.6	223.6	178.5	143.1	188.4	197.2	194.7	125.8	184.8
Calculation												
5. Saturation vapour pressure (mbar); ea	36.1	36.1	37.2	37.2	34.7	36.3	35.3	34.2	39.0	38.5	36.3	37.8
6. Actual vapour pressure (mbar); ed	31.1	30.0	30.5	31.6	29.8	31.2	30.3	29.4	30.0	30.4	29.3	33.3
7. Difference in vapour pressure (mbar); (ea-ed)	5.0	6.1	6.7	5.6	4.9	5.1	5.0	4.8	9.0	8.1	7.0	4.5
8. Wind function; f(u)	0.65	0.60	0.73	0.70	0.87	0.75	0.66	0.78	0.80	0.80	0.61	0.77
9. Weighting facotr for U & RH; 1-W	0.24	0.24	0.23	0.23	0.24	0.24	0.24	0.25	0.23	0.23	0.24	0.23
10. Net radiation, (mm/day); Rn /1	6.7	6.6	7.3	5.6	5.6	5.2	5.2	4.9	6.6	6.0	6.4	4.8
11. Weighting factor for Rn; W	0.76	0.76	0.77	0.77	0.76	0.76	0.76	0.75	0.77	0.77	0.76	0.77
12. Adjustment factor; C	1.01	1.00	1.03	0.97	0.97	0.96	0.96	0.95	1.00	0.98	1.00	0.94
13. Potential evapotranspiration (mm/day); PET												
(a) Radiaiton term; (10) x (11)	5.1	5.0	5.6	4.3	4.3	4.0	3.9	3.7	5.1	4.6	4.8	3.7
(b) Aerodynamic term; (7) x (8) x (9)	0.79	0.88	1.12	0.90	1.01	0.92	0.78	0.93	1.65	1.49	1.01	0.80
(c) PET; (12) x [(13.a) + (13.b)]	5.9	5.9	6.9	5.0	5.2	4.7	4.5	4.4	6.8	6.0	5.8	4.2

/1 : Refer to Fig. 2.5

Table 2.4 Summary of Potential Evapotranspiration

Sengkang

(Unit: mm/day)

Year	Jan.	Feb.	Mar.	Apr.	May	Jun.	Jul.	Aug.	Sep.	Oct.	Nov.	Dec.
1975	-	-	-	-	-	-	-	4.7	5.5	5.1	5.4	4.9
1976	5.6	6.4	5.8	5.3	4.8	3.7	4.3	6.1	6.6	6.4	5.7	5.6
1977	5.2	5.5	5.9	5.3	4.9	3.7	4.9	5.8	6.5	7.2	5.7	4.2
1978	4.3	4.6	4.4	4.2	4.3	3.7	4.0	4.5	4.5	5.7	4.9	4.1
1979	4.2	4.1	4.2	4.1	4.4	3.3	4.3	6.1	5.8	6.3	5.2	4.0
1980	3.9	4.1	4.6	3.8	4.0	3.8	4.4	5.3	5.1	5.3	4.6	3.9
1981	5.2	4.4	4.8	4.4	3.9	4.1	-	-	-	-	-	-
Average	4.7	4.9	5.0	4.5	4.4	3.7	4.4	5.4	5.7	6.0	5.3	4.5

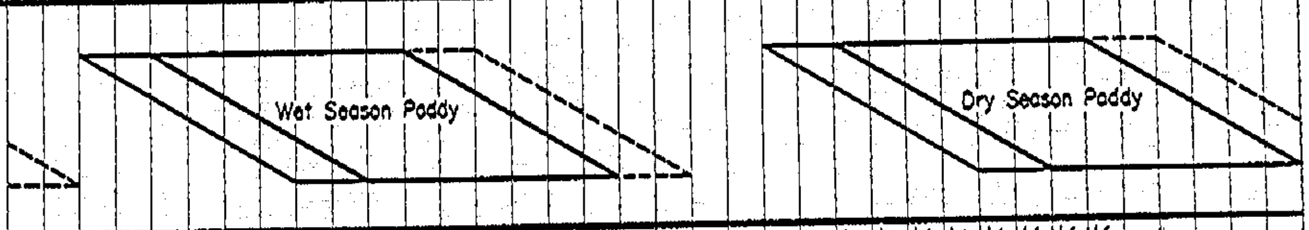
Kanyuara

Year	Jan.	Feb.	Mar.	Apr.	May	Jun.	Jul.	Aug.	Sep.	Oct.	Nov.	Dec.
1975	4.3	4.0	5.3	5.2	4.9	4.5	3.5	3.7	4.3	6.2	6.0	5.1
1976	4.3	4.9	4.8	5.4	5.1	4.4	5.5	4.8	4.8	5.4	5.1	4.0
1977	3.7	3.9	5.1	5.0	5.0	4.5	4.4	4.7	5.3	5.0	5.8	6.0
1978	4.4	3.9	4.0	3.1	4.6	5.0	4.8	4.9	5.4	6.6	5.3	4.4
1979	5.5	5.4	4.9	5.7	5.2	3.5	3.9	4.8	4.7	5.9	5.6	5.4
1980	6.0	5.9	6.9	5.0	5.2	4.7	4.5	4.4	6.8	6.0	5.8	4.2
1981	5.0	4.4	6.0	5.9	5.4	5.1	-	-	-	-	-	-
Average	4.7	4.6	5.3	5.0	5.1	4.5	4.4	4.6	5.2	5.9	5.6	4.9

Average on two stations (1973 - 1981)

4.7	4.8	5.2	4.8	4.8	4.1	4.4	5.0	5.5	6.0	5.4	4.7

Table 2.5 Farm Water Requirement



	Mar.			Apr.			May			Jun			Jul.			Aug.			Sep.			Oct.			Nov.			Dec.			Jan.			Feb.		
	E	M	L	E	M	L	E	M	L	E	M	L	E	M	L	E	M	L	E	M	L	E	M	L	E	M	L	E	M	L	E	M	L	E	M	L
① CROPPING PATTERN																																				
② CROPPING INTENSITY 1					1/12	1/6	1/6	1/6	1/6	1/6	1/6	1/12												1/12	1/6	1/6	1/6	1/6	1/6	1/6	1/12					
2						1/12	1/6	1/6	1/6	1/6	1/6	1/6	1/12												1/12	1/6	1/6	1/6	1/6	1/6	1/6	1/12				
3							1/12	1/6	1/6	1/6	1/6	1/6	1/6	1/12												1/12	1/6	1/6	1/6	1/6	1/6	1/6	1/12			
4								1/12	1/6	1/6	1/6	1/6	1/6	1/6	1/12												1/12	1/6	1/6	1/6	1/6	1/6	1/6	1/12		
5									1/12	1/6	1/6	1/6	1/6	1/6	1/6	1/12												1/12	1/6	1/6	1/6	1/6	1/6	1/6	1/12	
6										1/12	1/6	1/6	1/6	1/6	1/6	1/6	1/12												1/12	1/6	1/6	1/6	1/6	1/6	1/6	1/12
Total					1/12	1/4	5/12	7/12	9/12	11/12	1	11/12	9/12	7/12	5/12	1/4	1/12							1/12	1/4	5/12	7/12	9/12	11/12	1	11/12	9/12	7/12	5/12	1/4	1/12
③ CROPPING COEFFICIENT 1					1.00	1.02	1.07	1.18	1.30	1.38	1.38	1.28												1.00	1.02	1.07	1.18	1.30	1.38	1.38	1.28					
2						1.00	1.02	1.07	1.18	1.30	1.38	1.38	1.28												1.00	1.02	1.07	1.18	1.30	1.38	1.38	1.28				
3							1.00	1.02	1.07	1.18	1.30	1.38	1.38	1.28												1.00	1.02	1.07	1.18	1.30	1.38	1.38	1.28			
4								1.00	1.02	1.07	1.18	1.30	1.38	1.38	1.28												1.00	1.02	1.07	1.18	1.30	1.38	1.38	1.28		
5									1.00	1.02	1.07	1.18	1.30	1.38	1.38	1.28												1.00	1.02	1.07	1.18	1.30	1.38	1.38	1.28	
6										1.00	1.02	1.07	1.18	1.30	1.38	1.38	1.28												1.00	1.02	1.07	1.18	1.30	1.38	1.38	1.28
Weight. Av.					1.00	1.01	1.04	1.08	1.13	1.17	1.22	1.26	1.31	1.34	1.36	1.35	1.28							1.00	1.01	1.04	1.08	1.13	1.17	1.22	1.26	1.31	1.34	1.36	1.35	1.28
④ POTENTIAL EVAPOTRANSPILATION					48	48	48	48	53	41	41	41	44	44	48	50	50							66	54	54	54	47	47	52	47	47	52	48	48	38
⑤ CONSUMPTIVE USE ③ x ④					48	48	50	52	60	48	50	52	58	59	65	68	64							66	55	56	58	53	55	63	59	62	70	65	65	49
⑥ PERCOLATION LOSS					20	20	20	20	22	20	20	20	20	20	22	20	20							22	20	20	20	20	20	22	20	20	22	20	20	16
⑦ ⑤ + ⑥					68	68	70	72	82	68	70	72	78	79	87	88	84							88	75	76	78	73	75	85	79	82	92	85	85	65
⑧ WATER REQUIREMENT ⑦ x ②					6	17	29	42	62	62	70	66	59	46	36	22	7							7	19	32	46	55	69	85	72	62	54	35	21	5
⑨ WATER REQUIREMENT FOR NURSERY			1	2	2	2	2	2	2	1												1	2	2	2	2	2	2	1							
⑩ WATER REQUIREMENT FOR PUDDLING				18	37	37	37	37	37	18													18	37	37	37	37	37	18							
⑪ FARM WATER REQUIREMENT ⑧+⑨+⑩			1	20	45	56	68	81	101	81	70	66	59	46	36	22	7					1	20	46	58	71	85	94	88	85	72	62	54	35	21	5

Note : Unit for ④ to ⑪ : mm

⑨, ⑩ : Details in Fig.

Table 2.6 Effective Rainfall
(Tanru Tedong)

(Unit: mm/10days)

Month		1973	1974	1975	1976	1977	1978	1979	1980	1981
Jan.	F	5	7	0	0	32	0	10	16	0
	M	46	0	0	3	1	40	12	63	0
	L	28	23	1	27	28	4	0	0	0
Feb.	F	23	0	52	2	14	7	22	1	2
	M	56	21	2	0	12	0	21	5	0
	L	0	16	0	10	3	40	1	61	3
Mar.	F	-	-	-	-	-	-	-	-	-
	M	-	-	-	-	-	-	-	-	-
	L	11	40	5	0	1	22	70	68	77
Apr.	F	94	105	37	11	78	11	63	77	29
	M	72	56	65	69	57	0	0	36	14
	L	14	0	17	70	35	96	82	39	22
May	F	15	54	89	52	65	63	62	54	68
	M	41	61	77	14	23	54	1	17	63
	L	76	75	63	37	69	41	32	92	9
Jun.	F	95	0	95	96	13	82	65	36	80
	M	66	64	45	60	54	66	96	66	83
	L	72	61	11	13	0	63	15	67	14
Jul.	F	67	17	26	10	0	19	1	3	111
	M	2	57	56	21	0	60	65	0	71
	L	11	40	87	6	0	3	6	3	57
Aug.	F	112	2	22	0	40	41	0	47	-
	M	64	0	56	0	42	43	0	6	-
	L	-	-	-	-	-	-	-	-	-
Sep.	F	-	-	-	-	-	-	-	-	-
	M	-	-	-	-	-	-	-	-	-
	L	-	-	-	-	-	-	-	-	-
Oct.	F	0	16	78	63	13	43	0	0	-
	M	15	81	42	0	0	0	0	0	-
	L	97	43	77	40	0	22	0	14	-
Nov.	F	23	44	16	28	0	40	41	43	-
	M	0	11	6	58	16	40	26	29	-
	L	12	0	6	48	101	0	0	64	-
Dec.	F	2	7	8	6	13	10	3	0	-
	M	40	1	6	0	14	9	4	0	-
	L	21	27	6	21	23	5	0	50	-

Table 2.7 Unit Diversion Water Requirement (1/4)

Year		NR	PR	CU	P	ER	C1	C2	C3	ND	DR
1973 Mar.	E										
	M										
	L	120				11	1/240			1	0.01
Apr.	E	120	220			94	1/80	1/12		11	0.20
	M	120	220	48	20	72	1/60	1/6	1/12	25	0.45
	L	120	220	48	20	14	1/60	1/6	1/4	50	0.90
May	E	120	220	50	20	15	1/60	1/6	5/12	59	1.06
	M	120	220	52	20	41	1/60	1/6	7/12	49	0.89
	L	120	220	60	22	76	1/80	1/6	9/12	29	0.47
Jun.	E	120	220	48	20	95	1/240	1/12	11/12	11	0.20
	M			50	20	66			1	4	0.07
	L			52	20	72			11/12	-	-
Jul.	E			58	20	67			9/12	8	0.14
	M			59	20	2			7/12	45	0.81
	L			65	22	11			5/12	32	0.53
Aug.	E			68	20	112			3/12	-	-
	M			64	20	64			1/12	2	0.04
	L										
Sep.	E										
	M										
	L										
Oct.	E	120				0	1/240			1	0.01
	M	120	220			15	1/80	1/12		35	0.63
	L	120	220	66	22	97	1/60	1/6	1/12	21	0.33
Nov.	E	120	220	55	20	23	1/60	1/6	1/4	47	0.85
	M	120	220	56	20	0	1/60	1/6	5/12	70	1.27
	L	120	220	58	20	12	1/60	1/6	7/12	75	1.36
Dec.	E	120	220	53	20	2	1/80	1/6	9/12	91	1.65
	M	120	220	55	20	40	1/240	1/12	11/12	49	0.89
	L			63	22	21			1	64	1.05
1974 Jan.	E			59	20	7			11/12	66	1.19
	M			62	20	0			9/12	62	1.12
	L			70	22	23			7/12	40	0.65
Feb.	E			65	20	0			5/12	35	0.63
	M			65	20	21			3/12	16	0.29
	L			49	16	16			1/12	4	0.09

Year		NR	PR	CU	P	ER	C1	C2	C3	ND	DR
1974 Mar.	E										
	M										
	L	120				40	1/240			1	0.01
Apr.	E	120	220			105	1/80	1/12		10	0.18
	M	120	220	48	20	56	1/60	1/6	1/12	29	0.52
	L	120	220	48	20	0	1/60	1/6	1/4	56	1.01
May	E	120	220	50	20	54	1/60	1/6	5/12	35	0.63
	M	120	220	52	20	61	1/60	1/6	7/12	34	0.61
	L	120	220	60	22	75	1/80	1/6	9/12	30	0.49
Jun.	E	120	220	48	20	0	1/240	1/12	11/12	81	1.46
	M			50	20	64			1	6	0.11
	L			52	20	61			11/12	10	0.18
Jul.	E			58	20	17			9/12	46	0.83
	M			59	20	57			7/12	13	0.24
	L			65	22	40			5/12	20	0.33
Aug.	E			68	20	2			3/12	22	0.40
	M			64	20	0			1/12	7	0.13
	L										
Sep.	E										
	M										
	L										
Oct.	E	120				16	1/240			1	0.01
	M	120	220			81	1/80	1/12		12	0.22
	L	120	220	66	22	43	1/60	1/6	1/12	35	0.57
Nov.	E	120	220	55	20	44	1/60	1/6	1/4	38	0.69
	M	120	220	56	20	11	1/60	1/6	5/12	64	1.16
	L	120	220	58	20	0	1/60	1/6	7/12	84	1.52
Dec.	E	120	220	53	20	7	1/80	1/6	9/12	86	1.56
	M	120	220	55	20	1	1/240	1/12	11/12	80	1.59
	L			63	22	27			1	58	0.95
1975 Jan.	E			59	20	0			11/12	72	1.30
	M			62	20	0			9/12	62	1.12
	L			70	22	1			7/12	53	0.87
Feb.	E			65	20	52			5/12	14	0.25
	M			65	20	2			3/12	21	0.38
	L			49	16	0			1/12	5	0.11

Remark: NR; Nursery Water Requirement (mm)
PR; Puddling Water Requirement (mm)
CU; Consumptive Use of Water (Evapotranspiration x Crop Coefficient) (mm)
P ; Percolation (mm)
ER; Effective Rainfall (mm)
ND; Net Water Demand
DR; Diversion Water Requirement (lit/sec/ha)
C1; Ratio of Nursery Area to the Total Area
C2; Ratio of Puddling Area to the Total Area
C3; Cropping Intensity of the Project Area

Table 2.7 Unit Diversion Water Requirement (2/4)

Year		NR	PR	CU	P	ER	C1	C2	C3	ND	DR
1975 Mar.	E										
	M										
	L	120				5	1/240			1	0.01
Apr.	E	120	220			37	1/80	1/12		17	0.31
	M	120	220	48	20	65	1/60	1/6	1/12	27	0.49
	L	120	220	48	20	17	1/60	1/6	1/4	48	0.87
May	E	120	220	50	20	89	1/60	1/6	5/12	22	0.40
	M	120	220	52	20	77	1/60	1/6	7/12	25	0.45
	L	120	220	60	22	63	1/80	1/6	9/12	41	0.67
Jun.	E	120	220	48	20	95	1/240	1/12	11/12	11	0.20
	M			50	20	45			1	25	0.45
	L			52	20	11			11/12	56	1.01
Jul.	E			58	20	26			9/12	39	0.71
	M			59	20	56			7/12	13	0.24
	L			65	22	87			5/12	-	-
Aug.	E			68	20	22			3/12	17	0.31
	M			64	20	56			1/12	2	0.04
	L										
Sep.	E										
	M										
	L										
Oct.	E	120				78	1/240			1	0.01
	M	120	220			42	1/80	1/12		16	0.29
	L	120	220	66	22	77	1/60	1/6	1/12	25	0.41
Nov.	E	120	220	55	20	16	1/60	1/6	1/4	50	0.90
	M	120	220	56	20	6	1/60	1/6	5/12	67	1.21
	L	120	220	58	20	6	1/60	1/6	7/12	80	1.45
Dec.	E	120	220	53	20	8	1/80	1/6	9/12	85	1.54
	M	120	220	55	20	6	1/240	1/12	11/12	83	1.50
	L			63	22	6			1	79	1.30
1976 Jan.	E			59	20	0			11/12	54	0.98
	M			62	20	3			9/12	59	1.07
	L			70	22	27			7/12	38	0.62
Feb.	E			65	20	2			5/12	35	0.63
	M			65	20	0			3/12	21	0.38
	L			49	16	10			1/12	5	0.08

Year		NR	PR	CU	P	ER	C1	C2	C3	ND	DR
1976 Mar.	E										
	M										
	L	120				0	1/240			1	0.01
Apr.	E	120	220			11	1/80	1/12		19	0.34
	M	120	220	48	20	69	1/60	1/6	1/12	26	0.47
	L	120	220	48	20	70	1/60	1/6	1/4	26	0.47
May	E	120	220	50	20	52	1/60	1/6	5/12	37	0.67
	M	120	220	52	20	14	1/60	1/6	7/12	70	1.27
	L	120	220	60	22	37	1/80	1/6	9/12	66	1.08
Jun.	E	120	220	48	20	96	1/240	1/12	11/12	10	0.18
	M			50	20	60			1	10	0.18
	L			52	20	13			11/12	54	0.97
Jul.	E			58	20	101			9/12	-	-
	M			59	20	21			7/12	34	0.62
	L			65	22	6			5/12	34	0.56
Aug.	E			68	20	0			3/12	22	0.46
	M			64	20	0			1/12	7	0.13
	L										
Sep.	E										
	M										
	L										
Oct.	E	120				63	1/240			1	0.01
	M	120	220			0	1/80	1/12		20	0.36
	L	120	220	66	22	40	1/60	1/6	1/12	35	0.57
Nov.	E	120	220	55	20	28	1/60	1/6	1/4	45	0.81
	M	120	220	56	20	58	1/60	1/6	5/12	36	0.65
	L	120	220	58	20	48	1/60	1/6	7/12	47	0.85
Dec.	E	120	220	53	20	6	1/80	1/6	9/12	87	1.57
	M	120	220	55	20	0	1/240	1/12	11/12	89	1.61
	L			63	22	21			1	64	1.05
1977 Jan.	E			59	20	32			11/12	43	0.78
	M			62	20	1			9/12	61	1.10
	L			70	22	28			7/12	37	0.61
Feb.	E			65	20	14			5/12	30	0.54
	M			65	20	12			3/12	18	0.33
	L			49	16	3			1/12	5	0.10

Table 2.7 Unit Diversion Water Requirement (3/4)

Year		NR	PR	CU	P	ER	C1	C2	C3	ND	DR
1977 Mar.	E										
	M										
	L	120				1	1/240			1	0.01
Apr.	E	120	220			78	1/80	1/12		12	0.22
	M	120	220	48	20	57	1/60	1/6	1/12	42	0.76
	L	120	220	48	20	35	1/60	1/6	1/4	41	0.67
May	E	120	220	50	20	65	1/60	1/6	5/12	29	0.52
	M	120	220	52	20	23	1/60	1/6	7/12	63	1.14
	L	120	220	60	22	69	1/80	1/6	9/12	36	0.58
Jun.	E	120	220	48	20	13	1/240	1/12	11/12	68	1.23
	M			50	20	54			1	16	0.29
	L			52	20	0			11/12	66	1.19
Jul.	E			58	20	0			9/12	59	1.07
	M			59	20	0			7/12	46	0.83
	L			65	22	0			5/12	36	0.59
Aug.	E			68	20	40			3/12	12	0.22
	M			64	20	42			1/12	4	0.07
	L										
Sep.	E										
	M										
	L										
Oct.	E	120				13	1/240			1	0.01
	M	120	220			0	1/80	1/12		20	0.36
	L	120	220	66	22	0	1/60	1/6	1/12	46	0.76
Nov.	E	120	220	55	20	0	1/60	1/6	1/4	57	1.03
	M	120	220	56	20	16	1/60	1/6	5/12	61	1.10
	L	120	220	58	20	101	1/60	1/6	7/12	20	0.36
Dec.	E	120	220	53	20	13	1/80	1/6	9/12	81	1.47
	M	120	220	55	20	14	1/240	1/12	11/12	75	1.36
	L			63	22	23			1	62	1.02
1978 Jan.	E			59	20	0			11/12	72	1.30
	M			62	20	40			9/12	32	0.58
	L			70	22	4			7/12	50	0.82
Feb.	E			65	20	7			5/12	33	0.60
	M			65	20	0			3/12	21	0.38
	L			49	16	40			1/12	2	0.05

Year		NR	PR	CU	P	ER	C1	C2	C3	ND	DR
1978 Mar.	E										
	M										
	L	120				22	1/240			1	0.01
Apr.	E	120	220			11	1/80	1/12		19	0.34
	M	120	220	48	20	0	1/60	1/6	1/12	44	0.79
	L	120	220	48	20	96	1/60	1/6	1/4	21	0.38
May	E	120	220	50	20	63	1/60	1/6	5/12	30	0.54
	M	120	220	52	20	54	1/60	1/6	7/12	39	0.70
	L	120	220	60	22	41	1/80	1/6	9/12	62	1.01
Jun.	E	120	220	48	20	82	1/240	1/12	11/12	12	0.22
	M			50	20	66			1	4	0.07
	L			52	20	63			11/12	8	0.14
Jul.	E			58	20	19			9/12	44	0.79
	M			59	20	60			7/12	11	0.20
	L			65	22	3			5/12	35	0.57
Aug.	E			68	20	41			3/12	12	0.22
	M			64	20	43			1/12	3	0.05
	L										
Sep.	E										
	M										
	L										
Oct.	E	120				43	1/240			1	0.01
	M	120	220			0	1/80	1/12		20	0.36
	L	120	220	66	22	22	1/60	1/6	1/12	40	0.65
Nov.	E	120	220	55	20	40	1/60	1/6	1/4	40	0.73
	M	120	220	56	20	40	1/60	1/6	5/12	46	0.83
	L	120	220	58	20	0	1/60	1/6	7/12	84	1.37
Dec.	E	120	220	53	20	10	1/80	1/6	9/12	84	1.51
	M	120	220	55	20	9	1/240	1/12	11/12	80	1.45
	L			63	22	5			1	80	1.31
1979 Jan.	E			59	20	10			11/12	63	1.13
	M			62	20	12			9/12	53	0.95
	L			70	22	0			7/12	54	0.88
Feb.	E			65	20	22			5/12	26	0.47
	M			65	20	21			3/12	16	0.29
	L			49	16	1			1/12	5	0.08

Table 2.7 Unit Diversion Water Requirement (4/4)

Year		NR	PR	CU	P	ER	C1	C2	C3	ND	DR
1979 Mar.	E										
	M										
	L	120				70	1/240			1	0.01
Apr.	E	120	220			63	1/80	1/12		14	0.25
	M	120	220	48	20	0	1/60	1/6	1/12	44	0.80
	L	120	220	48	20	82	1/60	1/6	1/4	20	0.36
May	E	120	220	50	20	62	1/60	1/6	5/12	31	0.55
	M	120	220	52	20	1	1/60	1/6	7/12	80	1.44
	L	120	220	60	22	32	1/80	1/6	9/12	70	1.15
Jun.	E	120	220	48	20	65	1/240	1/12	11/12	16	0.29
	M			50	20	96			1	-	-
	L			52	20	15			11/12	52	0.94
Jul.	E			58	20	1			9/12	58	1.04
	M			59	20	65			7/12	8	0.15
	L			65	22	6			5/12	34	0.55
Aug.	E			68	20	0			1/4	22	0.40
	M			94	20	0			1/12	7	0.13
	L										
Sep.	E										
	M										
	L										
Oct.	E	120				0	1/240			1	0.01
	M	120	220			0	1/80	1/12		20	0.36
	L	120	220	66	22	0	1/60	1/6	1/12	46	0.76
Nov.	E	120	220	55	20	41	1/60	1/6	1/4	40	0.72
	M	120	220	56	20	26	1/60	1/6	5/12	55	0.99
	L	120	220	58	20	0	1/60	1/6	7/12	84	1.52
Dec.	E	120	220	53	20	3	1/80	1/6	9/12	90	1.77
	M	120	220	55	20	4	1/240	1/12	11/12	85	1.54
	L			63	22	0			1	85	1.40
1980 Jan.	E			59	20	16			11/12	58	1.04
	M			62	20	63			9/12	14	0.26
	L			70	22	0			7/12	53	0.88
Feb.	E			65	20	1			5/12	35	0.63
	M			65	20	5			1/4	21	0.36
	L			49	16	61			1/12	1	0.01

Year		NR	PR	CU	P	ER	C1	C2	C3	ND	DR
1980 Mar.	E										
	M										
	L	120				68	1/240			1	0.01
Apr.	E	120	220			77	1/80	1/12		12	0.23
	M	120	220	48	20	36	1/60	1/6	1/12	35	0.63
	L	120	220	48	20	39	1/60	1/6	1/4	39	0.70
May	E	120	220	50	20	54	1/60	1/6	5/12	35	0.64
	M	120	220	52	20	17	1/60	1/6	7/12	68	1.22
	L	120	220	60	22	92	1/80	1/6	9/12	14	0.23
Jun.	E	120	220	48	20	36	1/240	1/12	11/12	45	0.81
	M			50	20	66			1	4	0.07
	L			52	20	67			11/12	5	0.08
Jul.	E			58	20	3			9/12	56	1.02
	M			59	20	0			7/12	46	0.83
	L			65	22	3			5/12	35	0.58
Aug.	E			68	20	47			1/4	9	0.16
	M			64	20	6			1/12	7	0.12
	L										
Sep.	E										
	M										
	L										
Oct.	E	120				0	1/240			1	0.01
	M	120	220			0	1/80	1/12		20	0.36
	L	120	220	66	22	14	1/60	1/6	1/12	42	0.69
Nov.	E	120	220	55	20	43	1/60	1/6	1/4	39	0.70
	M	120	220	56	20	29	1/60	1/6	5/12	53	0.96
	L	120	220	58	20	64	1/60	1/6	7/12	35	0.63
Dec.	E	120	220	53	20	0	1/80	1/6	9/12	93	1.68
	M	120	220	55	20	0	1/240	1/12	11/12	89	1.61
	L			63	22	50			1	35	0.58
1981 Jan.	E			59	20	0			11/12	72	1.31
	M			62	20	0			9/12	62	1.11
	L			70	22	0			7/12	54	0.88
Feb.	E			65	20	2			5/12	35	0.63
	M			65	20	0			1/4	21	0.38
	L			49	16	3			1/12	5	0.12