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REPUBLIC OF INDONESIA

MINISTRY OF PUBLIC WORKS
DIRECTORATE GENERAL OF WATER RESOURCES DEVELOPMENT

FEASIBILITY STUDY ON THE SANREGO IRRIGATION PROJECT

ANNEX VOLUME II

VI. IRRIGATION AND DRAINAGE PLAN
VII. PRELIMINARY DESIGN OF PROJECT FACILITIES
VIII. CONSTRUCTION PLAN AND COST ESTIMATE
IX. PROJECT EVALUATION
X. WATERSHED MANAGEMENT

MARCH 1983

JAPAN INTERNATIONAL COOPERATION AGENCY

TOKYO, JAPAN

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ANNEX - VI

IRRIGATION AND DRAINAGE PLAN

ANNEX - VI IRRIGATION AND DRAINAGE PLAN

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ANNEX - VI IRRIGATION AND DRAINAGE PLAN

1. GENERAL

The Government of Indonesia has been making an endeavour to realize the irrigation development of the Sanrego area in 1972, and the Sanrego Irrigation Project has been formulated so as to supply irrigation water to the area of 8,071 ha by utilizing the available water from the Sanrego river.

The irrigation development plan in this Feasibility Study is formulated based on the assessment of the water resources, estimate of irrigation water requirement and water balance study. The assessment of water resources is made through the hydrological analysis for not only the Sanrego river but also small tributaries to make the maximum effective use of water resources for the Project. The irrigation water requirement is calculated for the several cropping patterns to determine the optimum cropping pattern. Based on the available irrigation water and irrigation water requirement, the water balance study is made to estimate the guarantee irrigation area by the Sanrego river and tributaries.

The proposed irrigation system is determined on the basis of the formulated development plan, taking into consideration the best use of the irrigation canal system designed by DOI through review works for the design results. The drainage and farm road systems are carefully studied in relation with the irrigation system since they are important components of the Project works equally to the irrigation system.

2. IRRIGATION WATER REQUIREMENT

2.1 General

Irrigation water requirement is estimated for obtaining the basic information on evaluation of the availability of water resources and determination of the project scale. For evaluation of the water resources in and around study area, the long term water balance between the water requirement and river flow is employed in this study. Since the meteorological data as well as discharge data of the Sanrego river is available for 9 years from 1974 to 1982, the water requirements are estimated for a series of such period on 10-days basis.

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

- Irrigation water requirement of paddy, WRD:

$$WRD = (CU + PL + NW + PW - ER)/E$$

- Irrigation water requirement of polowijo crops, WRP:

$$WRP = (CU - ER)/E$$

where, CU: consumptive use of water

PL: percolation loss

NW: nursery water

PW: puddling water

ER: effective rainfall

E : combined irrigation efficiency

2.2 Consumptive Use of Water

The consumptive use of water is estimated based on the empirical prediction method using the climate data and crop coefficient rating to crop growth stages. There exist various prediction methods developed for estimate the consumptive use of water so far. Among them, the modified Penman method is best selected in consideration of the availability of climate data for applying the method and the accuracy of the results obtained from this method. It is generally accepted as the most reliable prediction method among others.

The consumptive use of water is calculated by following formula for each 10-days.

$$CU = Kc \times ETp$$

where, CU : consumptive use of water (mm/day)

Kc : crop coefficient rating crop stage of the cropping pattern

ETp: Potential evapotranspiration estimated by the modified Penman method (mm/day)

2.2.1 Crop coefficient

The crop coefficient for paddy is determined on the basis of the coefficient used in the on-going and planned irrigation projects around the Project area. (Ref. Guideline of Exploitation and Maintenance of Sadang Sub-Project and Feasibility Study on the Bila Irrigation Project). In order to check the crop coefficient, field measurements on water consumption of paddy are carried out at three (3) stations in Maradda existing irrigation scheme for two (2) months. The data obtained from field measurement are summarized in Table VI.2.1.

The crop coefficient for polowijo crops is determined by referring the FAO Irrigation and Drainage Paper No. 24 "Crop Water Requirement" and "Penelitian Water Requirements Untuk Palawija" published by Proyek Irigasi I.D.A.(PROSIDA).

The crop coefficients for paddy and polowijo crops are shown in Fig. VI.2.1 and VI.2.2, respectively. Based on the above figures, the variances of the crop coefficient rating the growing period are estimated as shown below:

(Paddy)

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

(Polowijo crops)

First month			Second month			Third month			Last month		
E	M	L	E	M	L	E	M	L	E	M	L
0.36	0.38	0.47	0.65	0.86	0.96	0.98	0.96	0.89	0.72	0.50	-

2.2.2 Potential evapotranspiration

The kind and type of climate data necessary for applying the modified Penman method are temperature, relative humidity, sunshine duration and wind velocity on daily basis. The meteorological stations which provide such data are Camming and Ujung Lamuru. The Camming station located at north edge of the study area and Ujung Lamuru is situated at about 25 km north from Camming. In consideration of the locations of both stations, the data obtained from Camming station is employed to estimate potential evapotranspiration.

The calculation is made with the reference to the FAO paper "Crop Water Requirement". The form of equation is expressed as follows:

$$ETp = C \cdot \{W \times Rn + (1-W) \times f(u) \times (ea - ed)\}$$

where,

ETp: Potential evapotranspiration (mm/day)

W: temperature x related weighting factor

Rn: net radiation in equivalent evaporation (mm/day)

f(u): wind x related function

(ea-ed): difference between the saturation vapour pressure at mean air temperature and the mean actual vapour pressure of the air (mbar)

C: adjustment factor to compensate for the effect of day and night weather conditions

The sample calculation of potential evapotranspiration is shown in Table VI.2.2. The calculation result for nine (9) years is shown in Table VI.2.3 and the average value of potential evapotranspiration is summarized below:

(Unit: mm/day)

J	F	M	A	M	J	J	A	S	O	N	D
4.5	4.7	4.6	4.4	3.7	3.5	4.0	4.9	5.8	5.8	5.2	4.3

In order to evaluate the propriety of the above calculation result, cross checking is made using the meteorological data at Ujung Lamuru. Furthermore, the evaporation data by U.S. Class A pan at Ujung Lamuru is employed to evaluate the estimated values.

The meteorological data at Ujung Lamuru for the period from 1978 to 1981 is summarized in Table VI.2.4 and evapotranspiration estimated using such data and pan-evaporation data are shown in Table VI.2.5. The sample calculation of evapotranspiration at Ujung Lamuru is shown in Table VI.2.6.

According to the result of cross checking mentioned above, the estimated evapotranspiration at Camming is proved to be appropriate. The comparison of estimated values of evapotranspiration and U.S. Class A pan data is shown in Fig. VI.2.3.

2.3 Percolation Rate

Percolation rates differ depending on the soil characteristics, topography, groundwater level, etc. Measurement data on the percolation rate are not available in the study area. In order to estimate the percolation rate, field measurements are conducted in the paddy fields. The locations of measurement sites are shown in Fig. VI.2.4. The result of measurement are summarized below:

(Unit: mm/day)

Station	Measurement Period	Average Value
Biru	August 19 - 31	1.0
Sanrego	August 19 - 31	1.3
Maradda	August 20 - 31	0.7
Oddi	August 13 - 18	1.2
Paccing	July 28 - August 1	0.8
Toritori	July 28 - August 4	1.3
Average		1.1

As regards the groundwater affect to the percolation rate, the well observation for three (3) months are carried out in the representative points of the study area as shown in Fig. VI.2.4. Observation period ranges from end of wet season to the dry season. The result shows that the groundwater surface fluctuates in the small ranges even in the transitional period from wet season to dry season. According to such results, the affect of groundwater to the percolation rate is judged to be small. The result of well observation is shown in Fig. VI.2.5.

In addition to the field measurements mentioned above, the references of the percolation rate used in the similar projects around the study area are made.

In the Sadang Sub-Project located in the adjacent area of the northern part of the Central South Sulawesi, the following measurement data for the short period of 4 to 6 days are obtained:

Measurement Place	Soil Type	Percolation Rate
Kanyuara	Silty clay	0.87 mm/day
Padaelo	Silty clay	0.72 mm/day

(Ref. Water Requirement for Paddy and Other Crops, PROSIDA, May, 1976)

In the Feasibility Study on the Bila Irrigation Project carried out by JICA in 1981 - 1982, the percolation rate was determined to be 2 mm/day based on the field measurements.

Taking into consideration with the result of the field measurements and data obtained from other similar projects around the study area, the percolation rate of 2 mm/day is adopted both for wet season paddy and dry season paddy.

2.4 Nursery Water Requirement

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:

- Area required for nursery bed: 1/20 of main field
- Nursery period : 25 days
- Water required for 25 days :

	Wet Season Paddy	Dry Season Paddy
Preparation of nursey bed	100 mm	100 mm
Evapotranspiration	103 mm (4.1 mm x 25 days)	120 mm (4.8 mm x 25 days)
Percolation (2 mm/day x 25 days)	50 mm	50 mm
Total	253 mm (260 mm)	270 mm

2.5 Puddling Water Requirement

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

The assumptions used in the estimation of water needed for saturation of soil profile are elaborated on the basis of the results of soil survey and laboratory test. The soil moisture content before water supply is resulted by daily soil moisture balance calculation during the period from the end of previous cropping season to the beginning of the puddling works for eight years. The result of soil moisture balance calculation reveals that the average value of soil moisture just before puddling works in each year is about 28% with the narrow ranges on the dry season cropping. Therefore, the soil moisture of 20% before water supply is used in the following estimation for wet and dry season. The details are shown in Table VI.2.7.

The puddling water 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 soil after puddling: 5%

(c) Soil moisture before water supply : 20%

(2) Water loss occured 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 (4.0 mm/day x 10 days): 40 mm
- dry season (4.7 mm/day x 10 days): 47 mm

(3) Average standing water depth after puddling 50 mm

Total	Wet season	185 mm (190 mm)
	Dry season	192 mm (200 mm)

2.6 Effective Rainfall

2.6.1 Design rainfall

There exist three rainfall gauging stations in the study area, Palattae, Maradda and Camming. The daily rainfall data of said stations are available for nine (9) years from 1974 to 1982. The annual and seasonal rainfalls are shown in the Table VI.2.8 and average value are summarized below:

(Unit: mm)

	Palattae	Maradda	Camming
Annual	2,069	1,804	1,965
Period of Wet season paddy (Apr. - Aug.)	1,263	1,069	1,030
Period of Dry season paddy (Nov. - Mar.)	489	457	610

To calculate the effective rainfall, the basin rainfall to the whole study area is estimated on the basis with the daily rainfall data obtained from the said three (3) stations. The concepts of the basin rainfall used in the study are:

(1) Rainfall pattern (rainy days) is defined as same as the Palattae rainfall gauging station. The seasonal rainy days at three (3) stations are shown in Table VI.2.9.

(2) Daily rainfall value is defined as the weighting average of rainfall values at three (3) stations. The weighting average of rainfall value is calculated with the ratio of command area of each station to the expected project area (8,000 ha). The said ratio used in the estimation of basin rainfall are as follows:

Palattae: 48.8%

Maradda : 45.4%

Camming : 5.8%

The basin rainfall used in the estimation of the effective rainfall, so called design rainfall, is shown in Table VI.2.8 and the average as follows:

for Wet Season Cropping (April - August) :	1,026 mm
for Dry Season Cropping (November - March):	398 mm

2.6.2 Effective rainfall

(1) Effective rainfall for paddy

The daily balance method is adopted for the estimation of the effective rainfall for paddy. The calculation is carried out for the period from 1974 to 1982 under the following conditions:

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

(b) Average water holding capacity of the field is assumed to be 50 mm. If the water depth is less than 10 mm, the irrigation water is supplied to fill up the holding capacity of the field,

(c) Intercepted loss of every rainfall is assumed as 2 mm.

Based on the result of the daily water balance calculation, the effective rainfalls for wet season cropping and dry season cropping are averaged as 470 mm and 250 mm, respectively. They are equivalent about 46% and 63% of total design rainfall. The details are shown in Table VI.2.8 and relation between design rainfall and effective rainfall on 10 days basis is shown in Fig. VI.2.6.

(2) Effective rainfall for polowijo crops

Effective rainfall for polowijo crops during the growing period is estimated by using the USDA - SCS method which is developed in the U.S. Department of Agriculture Soil Conservation Service. The seasonal effective rainfall and the ratio of the effective rainfall to the total design rainfall are shown in Table VI.2.10 and summarized in Table VI.2.11.

2.7 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) Combined irrigation efficiency (E): the ratio between water made directly available to the crop and that released at the intake structure, $E = Ec \times Ea$

In consideration of earth canals on major parts and canal control facilities to be provided, the conveyance loss of 20% is adopted. The application loss inclusive field losses for paddy and for polowijo crops are difined as 20% and 30%, respectively.

The combined irrigation efficiencies for paddy and polowijo crops, therefore, as follows:

	Paddy	Polowijo Crops
Conveyance efficiency :	80%	80%
Application efficiency :	80%	70%
Combined irrigation efficiency:	64%	56%

2.8 Diversion Water Requirement

For the calculation of the diversion water requirement, the following four (4) alternative cropping patterns are taken into account. The details of alternative cropping patterns are discribed in ANNEX - V.

Cropping Pattern	Wet Season	Dry Season
A	Paddy	Paddy
B	Paddy	Paddy Polowijo crops (Po-100B)
C	Paddy	Paddy Polowijo crops (Po-100C) Polowijo crops (Po-90C)
D	Paddy	Polowijo crops (Po-100B) Polowijo crops (Po-100C) Polowijo crops (Po-90C)

Based on the above, the diversion water requirements are estimated on the wet season paddy, dry season paddy and three (3) cases of dry season polowijo crops.

2.8.1 Farm water requirement

As the previous step for estimation of the diversion water requirements, the seasonal farm water requirements are estimated on 10-days basis for the period from 1974 to 1982. Table VI.2.12 and VI.2.13 show the sample calculations of the farm water requirements for paddy and polowijo crops, respectively. The nursery and puddling water requirements are estimated as shown in Table VI.2.14.

2.8.2 Unit diversion water requirement

The unit diversion water requirements are calculated on the basis of the farm water requirement, effective rainfall and irrigation efficiency. Table VI.2.15 shows the sample calculations of unit diversion water requirement for paddy and polowijo crops.

The calculation results of the unit diversion water requirements for paddy and polowijo crops for nine (9) years from 1974 to 1982 are summarized in Table VI.2.16. The maximum value of unit diversion water requirement for each crop is shown below:

W.S.P	D.S.P	Po-100B	Po-90C	Po-100C
1.43	1.75	0.93	1.24	1.06

Remarks; W.S.P: Wet season paddy

D.S.P: Dry season paddy

Po-100B: Polowijo of 100 days growing period for pattern B & D

Po-90C: Polowijo of 90 days growing period for pattern C & D

Po-100C: Polowijo of 100 days growing period for pattern C & D

3. WATER BALANCE STUDY

3.1 General

The irrigation water requirements vary throughout the year and from year to year as estimated in the previous section 2. The discharge records of the Sanrego river, which are made available for nine (9) years since 1974, indicate the large fluctuation as mentioned in ANNEX - I. There is difference between the time distribution of the available irrigation water from the river flow 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 1974 to 1982 to assess the available irrigation water from the rivers and to estimate the irrigable area guaranteed by the river flow.

3.2 Irrigable Area with the Sanrego River Flow

The balance calculation is made by means of dividing the river discharge by the diversion water requirement on the daily basis for the period of nine (9) years from 1974 to 1982. The irrigation water requirements used in the balance calculation are estimated on the four alternative cropping patterns mentioned in the previous section.

In making determination of guarantee irrigation area, it is assumed that the lack of water supply for a short period would not affect the successful crop yield in consideration of the effect of the standing water in case of paddy and crop characteristics in case of polowijo crops. Therefore, the acceptable period for the lack of water supply is assumed to be 5 days.

Based on the above, 10-days mean values of irrigable area in the critical low flow period are taken as the guarantee irrigation area by the Sanrego river flow. The irrigable area of wet season paddy and dry season crops in the critical low flow period are shown in Table VI.3.1 and VI.3.2, respectively and summarized in Table VI.3.3.

By using the above results, the irrigable area with the irrigation dependability level of 80% is estimated for each alternative cropping pattern as shown in Fig. VI.3.1, Fig. VI.3.2 and Table VI.3.3.

The results of water balance study and dependability analysis are summarized below:

(Unit: ha)

Cropping	Average Irrigable Area	Irrigable Area with 80% Dependability
Wet season paddy	7,400	6,300
Dry season paddy		
(Pattern A)		
Paddy	5,000	3,700
(Pattern B)		
Paddy	3,400	2,800
Polowijo (100)/1	3,400	2,800
(Pattern C)		
Paddy	3,100	2,200
Polowijo (90)/2	3,500	2,300
Polowijo (100)	7,100	3,800
(Pattern D)		
Polowijo (90)	5,500	2,800
Polowijo (100)	4,500	3,300
Polowijo (100)	7,700	(4,000)/3

Notes; /1: 100 days growing period

/2: 90 days growing period

/3: Without dependability analysis

3.3 Irrigable Area with the Sanrego River and Three Tributaries

The water balance study in the previous paragraph indicates that the Sanrego river flow would be short to supply water for the potential area of 8,000 ha for the irrigation development studied in ANNEX - IV. Besides, there exist some tributaries of the Sanrego and the Walanae rivers, which tributaries would be expected as the supplemental water resources for the project.

Taking into consideration with the above situations, three tributaries, namely, Biru, Parota and Macinaga, are employed to be supplemental water resources to the Sanrego river in order to maximize the effective use of water and land resources in the area. As described in ANNEX - I, the available water in the said three tributaries is about 24% of that in the Sanrego river during the wet and transitional season. The water balance calculation is made with the expected water discharge of the tributaries in addition to that of the Sanrego river and the irrigation water requirements on each alternative cropping pattern.

The guarantee irrigation areas by the river flows of the Sanrego river and three (3) tributaries for four (4) alternative cropping patterns are shown in Table VI.3.4 and VI.3.5, and summarized in Table VI.3.6.

Based on the above results the irrigable area with the irrigation dependability level of 80% is estimated for each cropping pattern as shown in Fig. VI.3.1, Fig. VI.3.3 and Table VI.3.6. The result is summarized below:

(Unit: ha)

Cropping	Average Irrigable Area	Irrigable Area with 80% Dependability
Wet season paddy	9,200	8,000
Dry season paddy		
(Pattern A)		
Paddy	5,700	4,000
(Pattern B)		
Paddy	4,100	3,000
Polowijo (100)/1	4,000	3,000
(Pattern C)		
Paddy	3,300	2,300
Polowijo (90)/2	3,800	2,400
Polowijo (100)	- /4	(4,000)/3
(Pattern D)		
Polowijo (90)	6,000	3,000
Polowijo (100)	5,000	3,400
Polowijo (100)	-	(4,000)

Remarks; /1: 100 days growing period

/2: 90 days growing period

/3: Without dependability analysis

/4: Value of more than 10,000

4. IRRIGATION DEVELOPMENT PLAN

4.1 Irrigable Area

The calculation of irrigation water requirement and water balance study are made for four (4) alternative cropping patterns as described in the previous section. Based on the water balance study result, the proposed cropping pattern is determined in ANNEX - III, taking into account the profitability, irrigation water requirement and labour requirement. Based on the proposed cropping pattern, the irrigable area with the irrigation dependability level of 80% is estimated on two (2) cases of project frame which depends on the water sources for the project. The irrigable areas on such cases are shown as follows:

(Unit: ha)

Alternative	Irrigable Area with 80% Dependability	
	Wet season paddy	Dry season paddy
Case 1		
Water source: Sanrego river only	6,300	3,700
Case 2		
Water source: Sanrego river + three tributaries	8,000	4,000

From the viewpoints of the present land use and land classification, the potential area for irrigation development is estimated to be about 8,000 ha in net as mentioned in ANNEX - IV. Out of 8,000 ha, 6,800 ha of land is used for paddy field and the remaining 1,200 ha of land is occupied by upland, orchard and grass land which exist sporadically in the project area. The irrigable area of 6,300 ha on case 1 is selected from the existing paddy fields. On case 2, the area of 1,200 ha is reclaimed and applied as the irrigable area from upland, orchard and grass land in addition to the existing paddy field.

4.2 Determination of Development Plan

Based on the land and water resources in the study area, two (2) alternative plans are taken into account for irrigation development. In order to select the optimal plan for irrigation development of the area, the comparatively study is made for those alternatives from technical and economical viewpoints. The general features of two alternative plans are shown below:

Alternative - 1:

This plan aims to serve the irrigation area of 6,300 ha depending upon only the Sanrego river flow. The irrigation facilities in this plan are required the modification of the DOI design as shown follows:

(1) Sanrego intake weir - cascade type intake discharge, 10.1 m^3/sec

(2) Main canal - canal length, 11.6 km max. design discharge, 10.1 m^3/sec

(3) Secondary canal - canal length, 86.3 km max. design discharge, 5.0 m^3/sec

(4) Related structures - 268 nos.

(5) Tertiary system - 6,300 ha

Alternative - 2:

This plan aims to serve the irrigation area of 8,000 ha by utilization the available water from the Sanrego river and three tributaries. The irrigation water is supplied to the area with the irrigation system based on the built-up DOI design and new supplemental facilities to be incorporated into the above system. The irrigation facilities required in this plan as follows:

(1) Sanrego intake weir - cascade type intake discharge, 12.9 m^3/sec

(2) Parota intake weir - tirol type intake discharge, 1.4 m^3/sec

(3) Biru intake weir - overflow type intake discharge, 0.9 m^3/sec

(4) Macinaga intake weir - tirol type intake discharge, 0.4 m^3/sec

(5) Main canal - canal length, 11.6 km max. design discharge, 12.9 m^3/sec

(6) Secondary canal - canal length, 97.5 km max. design discharge, 6.2 m^3/sec

(7) Connecting canal - canal length, 4.9 km max. design discharge, 1.4 m^3/sec

(8) Related structures - 287 nos.

(9) Tertiary system - 8,000 ha.

The preliminary cost estimate is made for the above two plans and the result is shown below:

(Unit: $x10^6$Rp)

Description	Construction Cost	
	Alternative - 1	Alternative - 2
1. Preparatory works	721	844
2. Sanrego intake weir	4,010	4,010
3. Main canal and related structures	1,536	1,700
4. Secondary canals	4,978	6,252
5. Three small-scaled intake weir	-	63
6. Farm road system	797	797
7. Tertiary system	3,107	3,945
8. Reclamation work	-	384
Total	15,149	17,995

To select the optimum plan for the Project, the preliminary economic comparison is carried out based on the economic cost and benefit for the above Alternatives. The calculation conditions and results are shown as follows:

Description	Alternative - 1	Alternative - 2
Development area	6,300 ha	8,000 ha
Conditions of comparison		
- Project life	50 years	50 years
- Construction period	8 years	8 years
- Build-up period to full development stage	5 years	5 years for existing paddy field 8 years for reclamation area
Economic cost and benefit		
- Total economic cost (10^6Rp)	19,632	22,668
- Benefit (10^6Rp)	5,858	7,155
Internal rate of return (IRR) (%)	14.6	15.1

The economic evaluation indicates that Alternative 2 shows higher IRR of 15.1% than that of 14.6% of Alternative 1. Besides, Alternative 2 will ensure the irrigation for possible maximum land of 8,000 ha making the maximum effective use of the water sources for the Project. This means that this plan will largely contribute to the regional socioeconomic development in the Project area. Consequently, the Sanrego Irrigation Project would be implemented successfully with the irrigation canal system designed by DOI with incorporation of new supplemental facilities for irrigation of 8,000 ha by utilizing the available water from the Sanrego river and three tributaries, i.e., Parota, Biru and Macinaga.

5. IRRIGATION CANAL SYSTEM

5.1 Design Discharge of Canals

In the previous section 2.8, the unit diversion water requirement is estimated for each year during the study period of 1974 to 1982. Depending on the irrigation area of 8,000 ha in wet season and of 4,000 ha in dry season, the peak water requirement to be diverted from the Sanrego river and three (3) tributaries in wet season is far larger than that in dry season. Therefore, the maximum value of the unit water requirements in wet season during nine (9) years is adopted to be the design unit diversion water requirement. The maximum value worked out through the study is of 1.43 lit/sec/ha. The design discharge of main and secondary canal is calculated by the following simple formula:

$$Qd = 1.43 \times A \times \frac{1}{1,000}$$

where, Qd: design discharge (m^3/sec)

A: commanding area (ha)

Meanwhile, the design discharges used in the existing design conducted by DOI was estimated by using the following formula:

$$Qd = a \times C \times A \times \frac{1}{1,000}$$

where, Qd: design discharge (m^3/sec)

a: design unit diversion water requirement (lit/sec/ha)

C: tegal coefficient

A: commanding area (ha)

The design unit diversion water requirement (a) used in the above formula was estimated at 2.0 lit/sec/ha, which was required for dry season paddy. The calculation of the said value was made by using the meteorological data on monthly basis.

The maximum design discharges at the beginning points of main canal and each secondary canal are estimated by using both formulae as shown in Table VI.5.1.

The preliminary cost comparison on the earth works of canals between the canals designed in this study and those designed by DOI is made and the result is shown in Table VI.5.2. It reveals that the construction cost on earth works of canals designed by DOI is a little higher than that of designed in this study. But, the difference of the costs estimated above is judged not to affect seriously to the total construction cost of project facilities.

The detailed design of main and secondary irrigation canal system was already completed by DOI. Preparation of re-design will spend the time and money, and will delay the implementation of the Project.

Taking into consideration the above situation, the design discharge estimated by DOI on the main and secondary irrigation canals are applied to the study for the Project.

In the design of tertiary and quarternary canals, the design discharge of the canal is determined by using the following formula, considering the rotational irrigation on the quarternary block basis.

$$Qd = 1.43 \times C \times A \times \frac{1}{1{,}000}$$

where, Qd: design discharge (m^3/sec)

C: tegal coefficient

A: commanding area (ha)

The irrigation diagram of the Project is shown in ANNEX - XII, Drawings.

5.2 Irrigation Canal System

5.2.1 Function and requirement of canal

Irrigation canal system of the Project consists of a main canal, secondary canals, connecting canals and tertiary system. The layout planning of these canals is carried out after understanding their respective functions and requirements mentioned below:

(1) Main canal

The main function of the canal is to deliver irrigation water from the Sanrego intake weir and the supplemental water from the Biru intake weir to the Project area in the most economical way. The length of the main canal is about 11.6 km and the maximum design discharge is about 12.9 m^3/sec.

(2) Secondary canal

The secondary canal is branched off from the Sanrego main canal or another secondary canal to distribute water to the secondary block area. The size of secondary block varies from about 3,700 ha to 100 ha. Due to the topography, the Parota, Palakka and Aming secondary canals command the comparatively large areas ranging from about 1,400 ha to 3,700 ha which total is about 70% of the Project area. The irrigable areas on the secondary block basis are shown in Table VI.5.3.

(3) Connecting canal

Three (3) connecting canals are proposed to be constructed to convey supplemental water from three (3) small intake weirs on the tributaries. Parota connecting canal of 1.0 km long conveys water of about 1.40 m^3/sec to Parota secondary canal. Biru connecting canal of 1.4 km long conveys water of about 0.93 m^3/sec to the Sanrego main canal and Macinaga connecting canal of 2.5 km along conveys water of about 0.4 m^3/sec to Aming secondary canal.

(4) Tertiary system

A tertiary block includes one (1) tertiary canal and 10 - 20 quaternary canals. The maximum size of tertiary block is proposed to be 77 ha in consideration with the proper irrigation rotation of the quaternary block level. The numbers of tertiary blocks amount 200 in total. The irrigable areas on the tertiary block are shown in Table VI.5.4.

5.2.2 Irrigation layout

Layout planning of irrigation canal system is carried out by checking the existing canal alignment designed by DOI. The following items are taken into consideration in the checking:

(a) Canal alignment should be planned so as not to give damage to public facilities,

(b) Embankment portions should be minimized as much as possible,

(c) Canal water level should be kept as high as possible for easy operation of canal system, and

(d) Canal layout should be convenient for the grouping of future water users association.

In order to check the build-up layout by DOI on the above items, the field investigations on topography, soil mechanics, construction materials, agriculture, and agroeconomics are carried out. Based on the results of the above checking, the canal layout designed by DOI is judged to be used as the proposed irrigation canal system.

The proposed irrigation canal layout is shown in Fig. VI.5.1 and the irrigation diagram is shown in ANNEX - XI, Drawings.

6. PROPOSED DRAINAGE SYSTEM

6.1 Drainage Water Requirement

6.1.1 General

It is essential in the drainage planning to maintain the paddy field in a adequately drained condition for the purpose of successful irrigation farming. In general, the critical 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 is surrounded by the low mountainous ranges and the flood runoff from these mountainous areas concentrates 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 irrigated paddy field. The drainage water requirement, therefore, is estimated for such two sources of water to be drained. The calculation of drainage water requirement is shown hereunder.

6.1.2 Drainage water requirement

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

(a) removal of excess rainfall in the paddy field, and

(b) transporting the runoff coming from the outside of Project area.

(1) Drainage requirement for removal of excess rainfall in the paddy field

In estimating the drainage water requirement for removal of excess rainfall in the paddy field, the following criteria are applied:

Four (4) days consecutive rainfall with five (5) year return period should be removed within four (4) days.

The rainfall data at Palattae are used in the calculation. Four (4) days consecutive rainfall with five (5) year return period is shown in Fig. VI.6.1. The design value is estimated by the following formula:

$$Qp = F \times \frac{C \times R4 \times A}{T} \times 10$$

where, Qp: drainage water requirement (m^3/sec)

F: peak factor, 1.25

C: runoff coefficient, 0.8

$R4$: 4-days consecutive rainfall, 280 mm

A: drainage area (ha)

T: drainage period (sec), $4 \times 24 \times 3{,}600$

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

(2) Drainage requirement for transporting the runoff from the outside of Project area

To estimate the surface runoff coming from the land covered with forest, bush, etc., the McMatch'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 Palattae and Maradda. The rainfall with return period of 5 years is shown in Fig. VI.6.2. The formula applied is as follows:

$$Qo = 9.15 \times 10^{-3} \times C \times i \times S^{1/5} \times A^{4/5}$$

where, Qo: peak flow (m^3/sec)

C: coefficient representing the watershed characteristics, 0.63

i: rainfall intensity for the time of flood concentration (mm/hr)

S: average ground slope

A: drainage area (ha)

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

$$i = \frac{R24}{24} \times \left(\frac{24}{T}\right)^{2/3} (\text{mm/hr})$$

where, $R24$: daily rainfall with return period of 5 years, 150 mm

T: time of flood concentration

The average ground slope is expressed as follows:

$$S = h/l$$

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. VI.6.3.

(3) Design drainage water requirement

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

$$Qd = f \times (Qp + Qo)$$

Where, Qd: design drainage canal discharge (m^3/sec)

f: allowance factor, 1.15

Qp: drainage water requirement for the paddy fields (m^3/sec)

Qo: drainage water requirement for transporting the flood runoff from the outside of the Project area (m^3/sec)

6.2 Drainage Canal System

6.2.1 Function and requirement of drainage canal

The drainage canal system consist of major, tertiary and quaternary drains.

The function of the major drains is to transport water from tertiary drains and flood water from surrounding mountainous areas to the disposal points. The tertiary drains are provided to lead the excess water to be collected by the quaternary drains in the Project area to the major drains. The quaternary drains collect excess water from the paddy field and the surface runoff from the non-irrigation area, farm road, village area, etc.

The layout of the irrigation system and topography are the main factors in determining the alignment 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.

6.2.2 Layout planning of major drains

In the layout planning of major drains, the following basic concepts are examined throughout the field investigations: (1) where should the major drainage canal to disposal points be located? (2) How will the excess water in the area by collected and transported to the disposal points ?. On the view point of the above concepts, the following items relating to drainage planning are investigated over the Project area.

(a) field damage due to ill-drainage and/or flood,

(b) present drainage mechanism in the Project area and in its vicinal areas,

(c) Analysis of intensity and duration of rainfall in the Project area,

(d) present land use in the Project area and in the drainage area of small streams and tributaries of the Walanae and the Sanrego rivers,

(e) soil characteristics in and around the Project area, and

(f) present farming practices, socio and agro-economic conditions in and around the Project area.

In addition to the investigation and analysis on the above items, the sample checking of the conveyance capacity of the natural stream is carried out based on the topographic survey of the river cross section and its profile as shown in Fig. VI.6.4. The location and result of the sample checking are shown as follows:

Description	Sample - 1	Sample - 2
1. Name of river	The Biru river	The Paccing river
2. Location	Upstream of confluence with D-11-2-1/1	Upstream of confluence with D-8-2/1
3. River bed gradient	1/169	1/287
4. River section below terrace in average (m^2)	82	48
5. Conveyance capacity below terrace (m^3/sec)	273	124
6. Design discharge (m^3/sec)	44	33

Note: /1: No. of the natural river as a major drain, ref. Fig. VI.6.5 and Drainage Diagram in ANNEX - XI, Drawings.

The results of the field investigations and analyses on the basic concepts of layout planning reveal that the natural streams and rivers interlaced in the project area have full functions as the major drains for the project. Therefore, no major drains are proposed to be newly provided for the project.

The natural streams and rivers used as the major drains are about 180 km long in total and density of the major drains in the Project area is estimated at 22 m/ha. The alignment of the streams as major drains is shown in Fig. VI.6.5 and Drainage diagram is shown in ANNEX - XI, Drawings.

7. ROAD NETWORK

7.1 General

The Project area is put under the poor road conditions during the wet season. For the proper operation and maintenance of the project facilities and for agricultural and agro-economic activities after project implementation, the well arranged road network is of vital importance.

The proposed road network consists of the existing all-weather roads, roads planned in the Integrated Rural Development Project of the Sanrego Area, canal inspection roads and new roads proposed to be constructed in this study. In the proposed road network, the all roads mentioned above are well linked each other. The proposed road network is shown in Fig. VI.7.1 and general features of each kind of road are shown in Table VI.7.1.

7.2 Existing Road

There exist two (2) kinds of all-weather roads in the Project area.

One is the provincial road leading to Ujung Lamuru in north and to Sinjai in southeast. This provincial road is paved with gravel and with sufficient width. The Provincial Government promotes the improvement work of this provincial road up to the tar-macadam road as well as the improvement of the bridges on the road. The improvement work of this road in the Project area will be completed in 1982 fisical year on the schedule. This road will play a major role as the trunk road of the Project area and its surroundings from the regional economic point of view.

The other is the construction road for the Sanrego intake weir, which is laid in the southern edge of the Project area, connecting the above provincial road with the intake site. This road is paved with gravel. In the road network for the Project, this road will be put a role of the inspection road of the Sanrego intake weir and Biru intake weir.

7.3 Roads Planned in the Integrated Rural Development Project

Construction of new roads and up-grade works of the rural feeder roads are planned in the Integrated Rural Development Project conducted by the Government of Indonesia and Canadian International Development Agency (CIDA). Kind of roads planned in the said project are mainly laid in the northern east of the Project area. Those roads are proposed to be gravel pavement roads, and the implementation of them will be completed up to 1987. The general features of these roads planned in the Project area are shown in Table VI.7.1.

7.4 Canal Inspection Road

For the purposes of inspection, operation and maintenance of the main and secondary irrigation canals, the inspection roads will be constructed along the said canals.

Considering the future increase of vehicles for the inspection and operation and heavy construction equipment to be required for the canal maintenance and repair, the inspection roads are designed so as to have a width of 6.0 m and to be gravel pavement.

These roads are also used for the movement of agricultural products and equipments as well as for the daily services between villages and from those roads to the trunk road. The general features of the inspection roads are shown in Table VI.7.1.

7.5 Proposed Road

7.5.1 Farm road

The farm road proposed in the Project consists of two (2) routes. One is branched from the access road for the Sanrego intake weir at near Palattae and leads to Sanrego village via Toritori. This road is proposed to be connected with the road planned in the Integrated Rural Development Project at Sanrego village and also connected with inspection roads of six (6) secondary irrigation canals. The other is branched from the above farm road at Toritori and leads to the provincials road. This road would make daily services and/or agronomic activities between the area of right and left bank of the Walanae river smooth. These proposed farm roads are constructed by up-grading of the existing feender roads.

The farm roads proposed in the project would play a role as the trunk roads as well as the existing provincial road. The general features of the roads are shown in Table VI.7.1 and typical cross section of the road is shown in Fig. VI.7.2.

7.5.2 Related structures

In relation with the proposed roads mentioned above, six (6) drain culverts and three (3) bridges are provided on the roads to cross the Walanae river, its tributaries and the Sanrego river. The drain culvert consists of single or double box barrel, transitions at up and downstream of the structure and wing walls. The structures are mainly constructed with wet stone masonry.

The bridge with maximum span length of 8 m and concrete T-beam type is applied. The width of bridge is 7 m and the length is 30 m at the Biru river and 40 m at the Walanae and the Sanrego rivers.

In addition to the above, the road crossing culverts are provided at every 400 m of road length to release the excess water conveyed by the side ditches to the disposal point. The road crossing culvert consists of a concrete pipe conduit and two (2) collecting boxes to be made with wet stone masonry.

Table VI.2.1 Evapotranspiration

Month	Date	Growing Stage (%)	Evapotranspiration (mm)			
			A	B	C	Average
(1982)						
Aug.	11-15	33	4.3	6.3	5.0	5.2
	16-20	38	5.3	6.0	-	5.7
	21-25	42	6.8	7.3	6.4	6.8
	26-31	46	-	6.5	6.0	6.3
Sep.	1-5	50	7.0	6.0	7.0	6.7
	6-10	54	7.5	7.6	6.0	7.0
	11-15	58	6.5	6.5	7.6	6.9
	16-20	60	6.8	6.0	5.0	5.9
	21-25	66	8.0	7.5	9.0	8.2
	26-30	71	8.0	8.0	9.4	8.5

Remark: A, B, C is names of the observation stations.

Table VI.2.2 Sample Calculation of Potential Evapotranspiration at Camming (1980)

No.	Data (Monthly Mean)	Jan.	Feb.	Mar.	Apr.	May	Jun.	Jul.	Aug.	Sep.	Oct.	Nov.	Dec.
1.	Temperature (°C), T	26.6	26.0	26.3	26.3	25.7	25.3	25.2	25.4	25.6	27.6	28.3	26.0
2.	Relative humidity (%), RH	83	86	82	89	91	90	82	84	76	75	76	86
3.	Sunshine duration n/N	0.25	0.31	0.40	0.38	0.34	0.42	0.55	0.59	0.77	0.57	0.53	0.33
4.	Wind velocity (m/s), U	4.68	5.18	3.26	3.46	2.77	4.09	3.88	4.33	5.38	5.51	4.49	4.71
Calculation													
5.	Saturation vapour pressure (mbr), ea	34.9	33.6	34.2	34.2	33.0	32.3	32.1	32.5	32.8	37.0	38.5	33.6
6.	Actual vapour pressure (mbr), ed	29.0	28.9	28.0	30.4	30.0	29.1	26.3	27.3	24.9	27.8	29.3	28.9
7.	Difference in vapour pressure (mbr), ea-ed	5.9	4.7	6.2	3.8	3.0	3.2	5.8	5.2	7.9	9.3	9.2	4.7
8.	Wind function, f(u)	0.52	0.53	0.45	0.46	0.42	0.49	0.48	0.50	0.56	0.57	0.51	0.52
9.	Weighting factor for U & RH, 1-W	0.25	0.25	0.25	0.24	0.25	0.26	0.26	0.26	0.25	0.24	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	5.8	6.4	7.0	6.6	5.8	6.1	7.0	7.8	9.6	8.3	8.0	6.4
12.	Net short wave radiation(mm/day),Rns	4.4	4.8	5.3	5.0	4.6	4.6	5.3	5.9	7.2	6.2	6.0	4.8
13.	Effect on long wave radiation												
	(a) f(t) for T	16.0	15.9	16.0	16.0	15.8	15.7	15.7	15.8	15.8	16.2	16.4	15.9
	(b) f(ed) for ed	0.10	0.10	0.11	0.10	0.10	0.10	0.11	0.11	0.12	0.11	0.10	0.10
	(c) f(n/N) for n/N	0.33	0.38	0.46	0.44	0.41	0.48	0.60	0.63	0.79	0.61	0.58	0.40
14.	Net long wave radiation (mm/day),Rnl (13.a)x(13.b)x(13.c)	0.5	0.6	0.8	0.7	0.6	0.8	1.0	1.1	1.5	1.1	1.0	0.6
15.	Net radiation (mm/day), Rn	3.9	4.2	4.5	4.3	3.8	3.8	4.3	4.8	5.7	5.1	5.0	4.2
16.	Weighting factor for Rn, W	0.75	0.75	0.75	0.76	0.75	0.74	0.74	0.74	0.75	0.76	0.77	0.76
17.	Adjustment factor for Rn, C	1.04	1.06	1.08	1.07	1.05	1.05	1.08	1.10	1.16	1.12	1.10	1.07
18.	Potential evapotranspiration (mm/day), PET												
	(a) Radiation term (15)x(16)	2.9	3.2	3.4	3.3	2.9	2.8	3.2	3.6	4.3	3.9	3.9	3.2
	(b) Aerodynamic term (7)x(8)x(9)	0.8	0.6	0.7	0.4	0.3	0.4	0.7	0.7	1.1	1.3	1.1	0.6
	(c) PET (17)x[(18.a)+(18.b)]	3.8	4.0	4.4	4.0	3.4	3.4	4.2	4.7	6.3	5.8	5.5	4.1

Table VI.2.3 Potential Evapotranspiration at Camming

(Unit: mm/day)

Year	Jan.	Feb.	Mar.	Apr.	May	Jun.	Jul.	Aug.	Sep.	Oct.	Nov.	Dec.	Annual (mm)
1974						3.5	3.4	5.1	5.5	5.4	4.8	4.4	-
1975	4.0	5.1	4.5	3.4	3.0	3.4	4.4	4.8	5.1	4.8	4.7	4.6	1,242
1976	5.6	5.8	6.0	5.4	4.3	3.8	4.4	6.1	7.1	6.4	5.7	4.8	1,994
1977	5.1	5.7	4.9	5.3	4.5	3.5	4.2	4.7	6.3	6.8	7.0	4.2	1,889
1978	4.6	4.8	4.6	4.7	4.3	3.7	4.0	4.7	5.4	5.1	4.1	4.2	1,648
1979	4.1	4.2	4.4	4.3	3.6	3.2	4.6	5.1	5.7	6.1	5.3	4.0	1,662
1980	3.8	4.0	4.4	4.0	3.4	3.4	4.2	4.7	6.3	5.8	5.5	4.1	1,634
1981	4.2	4.0	4.6	4.4	3.3	4.1	3.4	4.5	5.3	6.0	4.4	3.9	1,585
1982	4.4	4.3	3.7	3.6	3.4	3.3	3.5	4.5					-
Average	4.5	4.7	4.6	4.4	3.7	3.5	4.0	4.9	5.8	5.8	5.2	4.3	1,665

Table VI.2.4 Meteorological Condition at Ujung Lamuru

Item		Jan.	Feb.	Mar.	Apr.	May	Jun.	Jul.	Aug.	Sep.	Oct.	Nov.	Dec.	Average
Temperature	(°C)	27.5	27.4	27.6	27.5	27.4	26.7	25.8	26.0	26.8	27.7	28.2	27.5	27.2
Relative Humidity	(%)	75	85	87	88	89	87	85	84	81	80	82	87	84
Wind Velocity	(km/hr)	2.7	2.1	1.7	1.5	1.8	1.9	2.3	2.7	3.1	3.1	2.6	2.1	2.3
Sunshine Duration	(hr)	4.3	4.4	4.9	5.2	4.9	5.8	6.5	7.0	7.9	8.4	6.4	4.6	5.9

Table VI.2.5 Pan-Evaporation and Estimated Potential Evapotranspiration at Ujung Lamuru

(Unit: mm/day)

Year	Jan.	Feb.	Mar.	Apr.	May	Jun.	Jul.	Aug.	Sep.	Oct.	Nov.	Dec.	Annual (mm)
(A-Pan Evaporation, Ep)													
1978	5.7	5.6	4.7	5.1	4.3	4.6	4.7	4.5	5.8	6.1	5.2	4.9	1,860
1979	4.6	4.8	4.8	4.7	5.0	3.5	4.4	5.5	6.2	6.9	6.1	5.1	1,875
1980	5.5	5.1	4.9	4.1	4.3	4.2	4.2	4.7	6.4	6.6	6.2	4.3	1,844
1981	4.9	5.2	5.3	4.8	4.4	4.3	4.3	5.0	5.6	6.1	4.9	4.5	1,803
Average	5.2	5.2	4.9	4.7	4.5	4.2	4.4	4.9	6.0	6.4	5.6	4.7	1,846
(Potential Evapotranspiration, ETp)													
1978	4.1	4.1	4.1	4.3	3.9	3.6	3.9	3.8	4.6	5.3	4.4	4.2	1,530
1979	4.2	4.1	3.9	3.5	3.2	3.3	3.6	4.8	5.2	5.6	4.9	4.1	1,533
1980	3.7	3.9	4.2	4.0	3.8	3.9	4.1	4.5	5.8	5.9	5.4	3.8	1,616
1981	4.5	4.7	4.9	4.7	3.9	4.4	3.9	5.0	5.3	6.0	4.7	4.2	1,709
Average	4.1	4.2	4.3	4.1	3.7	3.8	3.9	4.5	5.2	5.7	4.9	4.1	1,597
ETp/Ep(%)	79	81	88	87	82	90	89	92	87	89	88	87	Average (87)

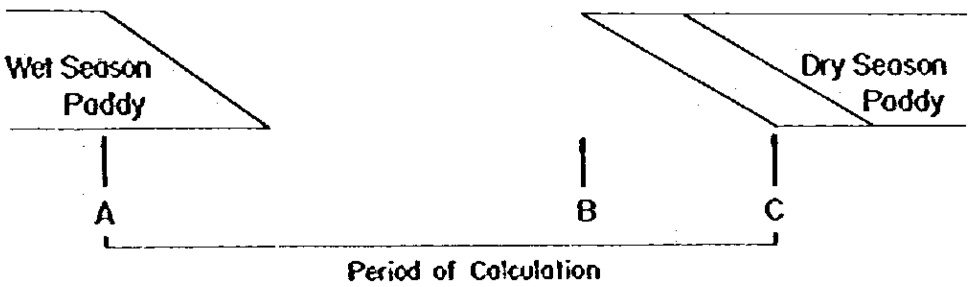
Table VI.2.6 Sample Calculation of Potential Evapotranspiration at Ujung Lamuru (1980)

No.	Data (Monthly Mean)	Jan.	Feb.	Mar.	Apr.	May	Jun.	Jul.	Aug.	Sep.	Oct.	Nov.	Dec.
1.	Temperature (°C), T	27.9	27.3	27.3	27.8	26.0	26.6	26.3	26.1	26.6	28.4	28.7	27.3
2.	Relative humidity (%), RH	86	89	82	86	86	83	85	85	78	78	79	85
3.	Sunshine duration n/N	0.27	0.34	0.41	0.40	0.43	0.49	0.55	0.58	0.74	0.64	0.57	0.33
4.	Wind velocity (m/s), U	1.25	0.92	0.58	0.38	1.83	2.29	2.43	3.04	3.71	3.58	2.66	2.33
	Calculation												
5.	Saturation vapour pressure (mbr), ea	37.6	36.3	36.3	37.4	35.5	34.9	34.2	33.8	34.9	38.7	39.4	36.3
6.	Actual vapour pressure (mbr), ed	32.3	32.3	29.8	32.1	30.5	28.9	29.1	28.7	27.2	30.2	31.1	33.5
7.	Difference in vapour pressure (mbr), ea-ed	5.3	4.0	6.5	5.2	5.0	5.9	5.1	5.1	7.7	8.5	8.3	2.8
8.	Wind function, f(u)	0.35	0.32	0.30	0.29	0.38	0.41	0.4[illegible]	0.46	0.51	0.50	0.44	0.42
9.	Weighting factor for U & RH, 1-W	0.23	0.24	0.24	0.23	0.24	0.24	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	15.2
11.	Solar radiation (mm/day), Rs	6.0	6.6	7.1	6.7	6.4	6.5	7.0	7.7	9.4	8.9	8.3	6.4
12.	Net short wave radiation (mm/day), Rns	4.5	5.0	5.3	5.0	4.8	4.9	5.3	5.8	7.0	6.7	6.2	4.8
13.	Effect on long wave radiation												
	(a) f(t) for T	15.9	15.9	15.9	15.9	15.9	15.9	15.9	15.9	15.9	16.4	16.4	15.9
	(b) f(ed) for ed	0.09	0.09	0.10	0.09	0.10	0.10	0.10	0.10	0.11	0.10	0.09	0.09
	(c) f(n/N) for n/N	0.34	0.41	0.47	0.46	0.49	0.54	0.60	0.62	0.77	0.68	0.61	0.40
14.	Net long wave radiation (mm/day), Rnl (13.a)x(13.b)x(13.c)	0.5	0.6	0.8	0.7	0.8	0.9	1.0	1.0	1.3	1.1	0.9	0.6
15.	Net radiation (mm/day), Rn	4.0	4.4	4.6	4.4	4.0	4.0	4.3	4.8	5.7	5.6	5.3	4.2
16.	Weighting factor for Rn, W	0.77	0.76	0.76	0.77	0.76	0.76	0.75	0.75	0.76	0.77	0.77	0.76
17.	Adjustment factor for Rn, C	1.06	1.07	1.07	1.07	1.07	1.07	1.07	1.08	1.10	1.10	1.09	1.07
18.	Potential evapotranspiration (mm/day), PET												
	(a) Radiation term (15)x(16)	3.1	3.4	3.5	3.4	3.1	3.1	3.3	3.6	4.3	4.3	4.1	3.2
	(b) Aerodynamic term (7)x(8)x(9)	0.4	0.3	0.5	0.4	0.5	0.6	0.5	0.6	0.9	1.0	0.8	0.3
	(c) PET (17)x[(18.a)+(18.b)]	3.7	3.9	4.2	4.0	3.8	3.9	4.1	4.5	5.8	5.9	5.4	3.8

Table VI.2.7 Results of Daily Soil Moisture Balance Calculation (Dry Season Paddy)

Year	Soil Moisture During Paddling Work (Oct. 31 to Jan. 10)	
	Minimum Value (%)	Average Value (%)
1974	23.0	28.5
1975	20.6	29.7
1976	17.6	29.7
1977	12.3	34.0
1978	18.6	35.1
1979	12.3	16.8
1980	12.3	20.0
1981	24.6	34.4
Average	17.7	28.5

Remark: Period of Calculation



where, A: water in paddy field be drained out and moisture content be assumed as the minimum of 10%.

B to C: paddling works be commenced. Soil moisture content during B to C is shown above

A to C: period of daily soil moisture balance calculation

Table VI.2.8 Seasonal Rainfall and Effective Rainfall for Paddy

(Unit : mm)

Year	Annual Rainfall /1			Wet Season (Apr. - Aug.)						Dry Season (Nov. - Mar.)					
	Rp	Rm	Rc	Rp	Rm	Rc	DR	ER	R	Rp	Rm	Rc	DR	ER	R
1974-75	-	1,742	-	-	899	-	899	386	43	577	302	613	369	176	48
1975-76	2,585	2,471	2,889	1,618	1,592	1,520	1,463	621	42	311	336	513	229	141	62
1976-77	1,676	1,844	1,944	1,212	1,205	1,259	1,077	493	46	331	512	651	240	161	67
1977-78	1,914	1,575	1,259	1,163	1,026	724	979	390	40	717	517	392	548	358	65
1978-79	2,224	1,431	1,346	1,273	861	548	965	564	58	528	468	731	534	315	59
1979-80	1,753	1,405	1,684	1,135	748	799	870	465	53	429	485	659	373	237	64
1980-81	2,062	1,600	1,856	1,522	1,092	1,023	1,163	426	37	474	482	662	416	271	65
1981-82	2,272	2,367	2,774	939	1,345	1,498	881	363	41	546	557	657	475	339	71
1982	-	-	-	1,241	855	869	940	529	56	-	-	-	-	-	-
Average	2,069	1,804	1,965	1,263	1,069	1,030	1,026	470	46	489	457	610	398	250	63

Note ; Rp : Rainfall at Palattae
Rm : Rainfall at Maradda
Rc : Rainfall at Camming

/1 : Amount in the calender year

DR : Design rainfall
ER : Effective rainfall
R : ER/DR (%)

Table VI.2.9 Seasonal Rainy Days at Three Rainfall Stations

Year	Period of Wet Season Paddy (Apr.-Aug.)			Period of Dry Season Paddy (Nov.-Mar.)		
	Palattae	Maradda	Camming	Palatta	Maradda	Camming
1974-75	-	52	-	64(132)	30(105)	58(150)
1975-76	85	86	108	45(51)	28(90)	65(81)
1976-77	58	37	69	19(112)	38(167)	60(108)
1977-78	55	55	58	76(151)	45(280)	53(98)
1978-79	78	65	77	66(141)	51(145)	75(170)
1979-80	61	53	73	48(107)	49(132)	78(179)
1980-81	65	63	86	40(175)	52(132)	66(172)
1981-82	47	55	68	61(116)	51(109)	78(110)
1982	65	57	63	-	-	-
Average	64	58	75	52(123)	43(145)	67(134)

Note ; volume in parenthesis is the maximum 10-days rainfall in mm

Table VI.2.10 Summary of Seasonal Design Rainfall and Effective Rainfall for Polowijo

Year	Po-100(Pattern B&D)			Po-90(Pattern C&D)			Po-100(Pattern C&D		
	DR	ER	ER/DR	DR	ER	ER/DR	DR	ER	ER/DR
1974-75	403	217	54	(585)	247	42	574	215	37
1975-76	353	172	49	607	236	39	512	230	45
1976-77	339	175	52	183	105	57	427	213	50
1977-78	607	242	40	401	142	35	712	261	37
1978-79	690	280	41	637	258	41	700	275	39
1979-80	406	213	52	278	152	55	732	262	36
1980-81	599	196	33	253	115	45	700	236	34
1981-82	596	228	38	714	311	44	788	260	33
Average	499	215	45	457	196	45	643	244	39

Note; DR : Seasonal design rainfall (mm)

ER : Seasonal effective rainfall (mm)

ER/DR : (%)

PO-100 : Polowijo crop of 100 days growing period

PO-90 : Polowijo crop of 90 days growing period

Table VI.2.11 Effective Rainfall for Polowijo Crop

Month	10-days	1974-75			1975-76			1976-77			1977-78			1978-79			1979-80			1980-81			1981-82		
		(1)	(2)	(3)	(1)	(2)	(3)	(1)	(2)	(3)	(1)	(2)	(3)	(1)	(2)	(3)	(1)	(2)	(3)	(1)	(2)	(3)	(1)	(2)	(3)
Aug.	E		10			6			0			6			4			14			36			3	
	M		13			17			4			10			38			2			2			18	
	L		9			17			0			11			3			10			3			16	
Sep.	E		22			36			0			0			13			51			0			32	
	M		32			12			0			0			24			0			0			6	
	L		18			16			0			0			2			3			0			15	
Oct.	E		10			36			0			0			15			0			0			85	
	M		62			10			0			0			0			0			0			1	
	L	11	13		15	30		20	34		0	0		16	50		2	2		14	16		15	17	
Nov.	E	0	0		5	5		0	0		0	0		3	4		8	9		0	0		21	35	
	M	16	18		8	9		0	0		3	4		0	0		0	0		0	0		10	17	
	L	11	12	14	8	9	13	19	23	16	41	49	20	35	40	12	14	17	15	0	0	0	13	22	13
Dec.	E	15	15	14	6	6	6	14	14	13	38	38	25	4	4	3	7	7	7	23	23	15	20	20	13
	M	3	3	3	5	5	5	0	0	0	12	12	8	34	34	22	30	30	27	23	23	15	7	7	5
	L	10	10	9	22	22	20	30	30	27	12	12	8	27	27	18	7	7	7	12	12	8	17	17	12
Jan.	E	5		4	0		0	17		17	0		0	20		19	4		4	24		22	55		48
	M	4		3	14		13	10		10	28		27	12		12	18		17	10		9	19		16
	L	59		53	17		17	10		10	2		2	14		13	13		12	17		16	2		1
Feb.	E	30		30	4		4	7		7	15		15	24		25	13		13	10		10	15		15
	M	5		5	9		10	7		7	25		25	12		13	19		19	5		5	17		17
	L	13		13	2		2	0		0	3		3	17		17	34		34	6		6	12		12
Mar.	E	22		23	23		24	9		11	24		27	17		19	16		17	11		15	8		16
	M	11		11	3		4	8		9	11		13	11		11	12		12	14		19	0		1
	L	2		2	31		33	24		27	28		31	35		38	16		17	27		35	20		41
Apr.	E			16			37			39			34			12			15			44			24
	M			5			8			20			10			1			30			10			14
	L			10			34			0			13			40			16			7			12
Total	(a)	217	(247)	215	172	236	230	175	105	213	242	142	261	280	258	275	213	152	262	196	115	236	228	311	260
Design Rainfall in Growing Period	(b)	403	(585)	574	353	607	512	339	183	427	607	401	712	690	637	700	406	278	732	599	253	700	596	714	788
(a)/(b)	(%)	54	42	37	49	39	45	52	57	50	40	35	37	41	41	39	52	55	36	33	45	34	38	44	33

Note : (1) : Polowijo of 100 days growing period (Pattern B & D)
(2) : Polowijo of 90 days growing period (Pattern C & D)
(3) : Polowijo of 100 days growing period (Pattern C & D)

Table VI.2.12 Sample Calculation of Farm Water Requirement for Paddy (1980-1981)

No.	Items / Month	MAR.			APR.			MAY			JUN.			JUL.			AUG.			SEP.			OCT.			NOV.			DEC.			JAN.			FEB.		
	10 days	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											Wet Season Paddy																				Wet Season Paddy				
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								1/12	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
	6.	1/6	1/12								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
	Total	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									1/12	1/4	5/12	7/12	9/12	11/12	1	11/12	9/12	7/12	5/12
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.	1.28								1.00	1.02	1.07	1.18	1.30	1.38	1.36	1.28														1.00	1.02	1.07	1.18	1.30	1.38	1.38
	6.	1.38	1.28								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
	Weight Av.	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									1.00	1.01	1.04	1.08	1.13	1.17	1.22	1.26	1.31	1.34	1.36
4.	P.E.T	46	46			40	40	34	34	37	34	34	34	42	42	46	47	47									55	55	41	41	45	42	42	46	40	40	32
5.	Consumptive Use 3x4	62	59			40	40	35	37	42	40	41	43	55	56	63	63	60									55	56	43	44	51	49	51	58	52	54	44
6.	Percolation loss	20	20			20	20	20	20	22	20	20	20	20	20	22	20	20									20	20	20	20	22	20	20	22	20	20	16
7.	Farm Water Requirement 5+6	82	79			60	60	55	57	64	60	61	63	75	76	85	83	80									75	76	63	64	73	69	71	80	72	74	60

Table VI.2.13 Sample Calculation of Farm Water Requiremant for Polowijo (1980-1981)

(Po-100B, 1980-81)

No	Items	Oct.			Nov.			Dec.			Jan.			Feb.			Mar.		
		E	M	L	E	M	L	E	M	L	E	M	L	E	M	L	E	M	L
1.	Cropping Pattern			Po - 100 B															
2.	Cropping intensity			1/12	1/4	5/12	7/12	9/12	11/12	1	1	1	1	11/12	9/12	7/12	5/12	1/4	1/12
3.	Crop Coefficient																		
	1			0.36	0.38	0.47	0.65	0.86	0.96	0.98	0.98	0.90	0.72	0.50					
	2				0.36	0.38	0.47	0.65	0.86	0.96	0.98	0.98	0.90	0.72	0.50				
	3					0.36	0.38	0.47	0.65	0.86	0.96	0.98	0.98	0.90	0.72	0.50			
	4						0.36	0.38	0.47	0.65	0.86	0.96	0.98	0.98	0.90	0.72	0.50		
	5							0.36	0.38	0.47	0.65	0.86	0.96	0.98	0.98	0.90	0.72	0.50	
	6								0.36	0.38	0.47	0.65	0.86	0.96	0.98	0.98	0.90	0.72	0.50
	Weight. Average			0.36	0.37	0.41	0.48	0.56	0.64	0.72	0.81	0.88	0.90	0.87	0.84	0.81	0.74	0.65	0.50
4.	P.E.T			64	55	55	55	41	41	45	42	42	46	40	40	32	46	46	51
5.	E.W.R.			23	20	22	26	23	26	32	34	37	41	33	34	26	34	30	25

(Po-90C, 1980)

No	Items	Jul.			Aug.			Sep.			Oct.			Nov.			Dec.		
		E	M	L	E	M	L	E	M	L	E	M	L	E	M	L	E	M	L
1.	Cropping Pattern				Po - 90 C														
2.	Cropping intensity				1/12	1/4	5/12	7/12	9/12	11/12	1	1	1	11/12	9/12	7/12	5/12	1/4	1/12
3.	Crop Coefficient																		
	1				0.36	0.39	0.50	0.75	0.92	0.98	0.98	0.91	0.75	0.50					
	2					0.36	0.39	0.50	0.75	0.92	0.98	0.98	0.91	0.75	0.50				
	3						0.36	0.39	0.50	0.75	0.92	0.98	0.98	0.91	0.75	0.50			
	4							0.36	0.39	0.50	0.75	0.92	0.98	0.98	0.91	0.75	0.50		
	5								0.36	0.39	0.50	0.75	0.92	0.98	0.98	0.91	0.75	0.50	
	6									0.36	0.39	0.50	0.75	0.92	0.98	0.98	0.91	0.75	0.50
	Weight. Average				0.36	0.38	0.43	0.52	0.61	0.68	0.75	0.84	0.88	0.87	0.86	0.83	0.76	0.67	0.50
4.	P.E.T				47	47	52	63	63	63	58	58	64	55	55	55	41	41	45
5.	F.W.R.				17	18	22	33	38	43	44	49	56	48	47	46	31	27	23

(Po-100C, 1980-81)

No	Items	Nov.			Dec.			Jan.			Feb.			Mar.			Apr.		
		E	M	L	E	M	L	E	M	L	E	M	L	E	M	L	E	M	L
1.	Cropping Pattern			Po - 100 C															
2.	Cropping intensity			1/12	1/4	5/12	7/12	9/12	11/12	1	1	1	1	11/12	9/12	7/12	5/12	1/4	1/12
3.	Crop Coefficient			0.36	0.37	0.41	0.48	0.56	0.64	0.72	0.81	0.88	0.90	0.87	0.84	0.81	0.74	0.65	0.50
4.	P.E.T			55	41	41	45	42	42	46	40	40	32	46	46	51	44	44	44
5.	F.W.R.			20	15	17	22	23	27	33	32	35	29	40	39	41	32	29	22

Note : Po - 100 B : Polowijo of 100 days growing period, for pattern B and D

Po - 100 C : Polowijo of 100 days growing period, for pattern C and D

Po - 90 C : Polowijo of 90 days growing period, for pattern C and D

P.E.T. : Potential Evapotranspiration (mm)

F.W.R. : Farm water Requirement (= Consumptive use of water) (mm)

Table VI.2.14 Nursery and Puddling Water Requirement

No.	Items \ Month	MAR.			APR.			MAY			JUN.			OCT.			NOV.			DEC.			JAN.		
	10-days	E	M	L	E	M	L	E	M	L	E	M	L	E	M	L	E	M	L	E	M	L	E	M	L
I.	NURSERY WATER								Wet Season Paddy												Dry Season Paddy				
1	Nursery Water Requirement (mm)		104	104	104	104	104	104	104	104	104				108	108	108	108	108	108	108	108	108		
2	Cropping Intensity		1/960	1/120	1/64	1/48	1/48	1/48	1/960	1/64	1/240				1/960	1/120	1/64	1/48	1/48	1/48	19/960	1/64	1/240		
3	Farm Water Requirement (mm) ① x ②		1	1	2	2	2	2	2	2	1				1	1	2	2	2	2	2	2	1		
II.	PUDDLING WATER								Wet Season Paddy												Dry Season Paddy				
1	Puddling Water Requirement (mm)				190	190	190	190	190	190	190						200	200	200	200	200	200	200		
2	Working Intensity				1/12	1/6	1/6	1/6	1/6	1/6	1/12						1/12	1/6	1/6	1/6	1/6	1/6	1/12		
3	Farm Water Requirement (mm)				16	32	32	32	32	32	16						17	33	33	33	33	33	17		

Table VI.2.15 Sample Calculation of Unit Diversion Water Requirement (1980 - 1981)

Year	Month	10-Days	Wet Season Paddy & Dry Season Paddy (Pattern A&B)									
			ER	NR	PW	PI	PR	FR	FI	FN	FWD	DWR
				(1)			(2)			(3)	(1)+(2)+(3)	
1980	Mar.	M		1							1	0.02
		L		1							1	0.02
	Apr.	E	6	2	190	1/12	15				17	0.31
		M	23	2	190	1/6	28	60	1/12	3	33	0.60
		L	40	2	190	1/6	25	60	1/4	5	32	0.58
	May	E	64	2	190	1/6	21	55	5/12	-	23	0.42
		M	25	2	190	1/6	28	57	7/12	19	49	0.89
		L	66	2	190	1/6	21	64	9/12	-	23	0.37
	Jun.	E	28	1	190	1/12	14	60	11/12	29	44	0.79
		M	42					61	1	19	19	0.37
		L	54					63	11/12	8	8	0.15
	Jul.	E	4					75	9/12	53	53	0.96
		M	27					76	7/12	29	29	0.52
		L	4					85	5/12	34	34	0.55
	Aug.	E	43					83	1/4	10	10	0.18
		M	0					80	1/12	7	7	0.12
		L										
	Sep.	E										
		M										
		L										
	Oct.	E										
		M		1							1	0.02
		L		1							1	0.02
	Nov.	E	0	2	200	1/12	17				19	0.34
		M	0	2	200	1/6	33	75	1/12	6	42	0.75
		L	0	2	200	1/6	33	76	1/4	19	54	0.98
	Dec.	E	36	2	200	1/6	27	63	5/12	21	41	0.73
		M	51	2	200	1/6	25	64	7/12	8	34	0.62
		L	22	2	200	1/6	30	73	9/12	38	70	1.15
1981	Jan.	E	36	1	200	1/12	14	69	11/12	30	45	0.81
		M	11					71	1	60	60	1.09
		L	23					80	11/12	52	52	0.86
	Feb.	E	14					72	9/22	44	44	0.79
		M	3					74	7/12	41	41	0.75
		L	7					60	5/12	22	22	0.40
	Mar.	E	22					82	1/4	15	15	0.27
		M	46					79	1/12	3	3	0.05
		L										
	Apr.	E										
		M										
		L										

Year	Month	10-Days	Polowijo-100 (Pattern H&D)					Polowijo-90 (Pattern C&D)					Polowijo-100 (Pattern C&D)				
			ER	FR	FI	FWD	DWR	ER	FR	FI	FWD	DWR	ER	FR		FWD	DWR
1980	Mar.	M															
		L															
	Apr.	E															
		M															
		L															
	May	E															
		M															
		L															
	Jun.	E															
		M															
		L															
	Jul.	E															
		M															
		L															
	Aug.	E						36	17	1/12	-	-					
		M						2	18	1/4	4	0.08					
		L						3	22	5/12	8	0.15					
	Sep.	E						0	33	7/12	19	0.40					
		M						0	38	9/12	29	0.59					
		L						0	43	11/12	39	0.81					
	Oct.	E						0	44	1	44	0.91					
		M						0	49	1	49	1.01					
		L	14	23	1/12	1	0.01	16	56	1	40	0.75					
	Nov.	E	0	20	1/4	5	0.10	0	48	11/12	44	0.91					
		M	0	22	5/12	9	0.19	0	47	9/12	35	0.73					
		L	0	26	7/12	15	0.31	0	46	7/12	27	0.55	0	20	1/12	2	0.03
	Dec.	E	23	23	9/12	-	-	23	31	5/12	3	0.07	15	15	1/4	-	-
		M	23	26	11/12	3	0.06	23	27	1/4	1	0.02	15	17	5/22	1	0.02
		L	12	32	1	20	0.38	12	23	1/12	1	0.02	8	22	7/12	8	0.15
1981	Jan.	E	24	34	1	10	0.21						22	23	9/12	1	0.02
		M	10	37	1	27	0.56						9	27	11/22	17	0.34
		L	17	41	1	24	0.45						16	33	1	17	0.32
	Feb.	E	10	35	11/12	23	0.47						10	32	1	22	0.45
		M	5	34	9/12	22	0.45						5	35	1	30	0.62
		L	6	26	7/12	12	0.39						6	29	1	23	0.59
	Mar.	E	11	34	5/12	10	0.20						15	40	11/12	23	0.47
		M	14	30	1/4	4	0.08						19	39	9/12	15	0.31
		L	27	25	1/12	-	-						35	41	7/12	4	0.07
	Apr.	E											44	32	5/12	-	-
		M											10	29	1/4	5	0.10
		L											7	22	1/12	1	0.03

Remarks:
ER; Effective rainfall (mm)
NR; Nursery water requirement (mm)
PW; Puddling water (mm)
PI; Puddling work intensity
PR; Puddling water requirement (mm) = (PW - ER)·PI
FR; Farm water requirement (mm)
(Evapotranspiration x Crop coefficient + Percolation loss)

FI; Cropping Intensity
FN; Net farm water requirement (mm) = (FR - ER)·FI
FWD; Farm water demand (mm) for Paddy = (NR + PR + FN)
for Polowijo = (FR - ER)·FI
DWR; Unit diversion water requirement (lit/sec/ha) = FWD/(8.64 · Days · C)
C; Irrigation efficiency for Paddy; 0.64
for Polowijo; 0.56

Table VI.2.16 Summary of seasonal Maximum Unit Diversion Water Requirement

(Unit : lit/sec/ha)

Year	W.S.P.	D.S.P.	Po-100B	Po-90.C	Po-100C
1974-75	1.32	1.41	0.64	0.80	0.83
1975-76	1.06	1.75	0.93	0.68	1.03
1976-77	1.21	1.40	0.80	1.12	1.06
1977-78	1.30	1.55	0.81	1.24	0.80
1978-79	1.19	1.05	0.50	0.89	0.50
1979-80	0.85	1.35	0.56	1.07	0.40
1980-81	0.96	1.15	0.56	1.01	0.62
1981-82	1.43	1.24	0.77	1.01	0.64
1982	1.07	-	-	-	-
Average	1.15	1.36	0.70	0.98	0.74
Maximum	1.43	1.75	0.93	1.24	1.06

Note ; W.S.P : Wet season paddy

D.S.P : Dry season paddy

Po100B : Polowijo of 100 days growing period (for Pattern B & D)

Po90C : Polowijo of 90 days growing period (for Pattern C & D)

Po100C : Polowijo of 100 days growing period (for Pattern C & D)

Table VI.3.1 Irrigable Area of Wet Season Paddy in Critical Low Flow Period (with Sanrego River Flow)

Year	Month	Date	Discharge of Sanrego River	Diversion Water Requirement	Irrigable Area	Year	Month	Date	Discharge of Sanrego River	Diversion Water Requirement	Irrigable Area	Year	Month	Date	Discharge of Sanrego River	Diversion Water Requirement	Irrigable Area
			(m3/sec)	(l/sec/ha)	(ha)				(m3/sec)	(l/sec/ha)	(ha)				(m3/sec)	(l/sec/ha)	(ha)
1974	Jun.	1	7.8	1.32	5,900	1977	May	11	6.5	1.30	5,000	1980	Jul.	1	10.4	0.96	10,800
		2	8.0	"	6,100			12	6.5	"	5,000			2	10.0	"	10,400
		3	8.4	"	6,400			13	6.6	"	5,100			3	9.6	"	10,000
		4	8.4	"	6,400			14	6.6	"	5,100			4	8.8	"	9,200
		5	8.6	"	6,500			15	6.6	"	5,100			5	8.8	"	9,200
		6	8.6	"	6,500			16	6.6	"	5,100			6	8.4	"	8,800
		7	8.6	"	6,500			17	6.5	"	5,000			7	8.4	"	8,800
		8	8.8	"	6,700			18	7.3	"	5,600			8	8.0	"	8,300
		9	8.8	"	6,700			19	9.0	"	6,900			9	8.0	"	8,300
		10	8.8	"	6,700			20	8.0	"	6,200			10	8.0	"	8,300
Average			8.5	1.32	6,400	Average			7.0	1.30	5,400	Average			8.8	0.96	9,200
1975	Jul.	1	10.9	1.06	10,300	1978	May	22	9.2	1.19	7,700	1981	Jun.	1	10.9	1.43	7,600
		2	10.9	"	10,300			23	8.4	"	7,100			2	10.9	"	7,600
		3	11.4	"	10,800			24	7.6	"	6,400			3	10.4	"	7,300
		4	10.0	"	9,400			25	7.3	"	6,100			4	10.0	"	7,000
		5	8.2	"	7,700			26	7.0	"	5,900			5	9.6	"	6,700
		6	8.8	"	8,300			27	6.8	"	5,700			6	9.6	"	6,700
		7	9.0	"	8,500			28	6.2	"	5,200			7	9.2	"	6,400
		8	9.0	"	8,500			29	7.0	"	5,900			8	9.2	"	6,400
		9	9.2	"	8,700			30	8.4	"	7,100			9	8.8	"	6,200
		10	9.0	"	8,500			31	7.3	"	6,100			10	8.4	"	5,900
Average			9.6	1.06	9,100	Average			7.5	1.19	6,300	Average			9.7	1.43	6,800
1976	May	12	9.2	1.21	7,600	1979	May	11	8.8	0.85	10,400	1982	May	11	9.2	1.07	8,600
		13	8.6	"	7,100			12	9.2	"	10,800			12	9.2	"	8,600
		14	8.4	"	6,900			13	10.9	"	12,800			13	7.6	"	7,100
		15	8.2	"	6,800			14	9.6	"	11,300			14	7.6	"	7,100
		16	8.4	"	6,900			15	8.0	"	9,400			15	7.3	"	6,800
		17	8.2	"	6,800			16	7.0	"	8,200			16	6.8	"	6,400
		18	8.0	"	6,600			17	6.8	"	8,000			17	6.8	"	6,400
		19	7.8	"	6,400			18	6.5	"	7,600			18	6.2	"	5,800
		20	7.7	"	6,400			19	6.2	"	7,300			19	6.0	"	5,600
		21	7.7	0.85	9,100			20	6.8	"	8,000			20	6.0	"	5,600
Average			8.2	1.17	7,100	Average			8.0	0.85	9,400	Average			7.3	1.07	6,800

Table VI.3.2 Irrigable Area of Dry Season Crop in Critical Low Flow Period (with Sanrego River Flow)

Year	Pattern A				Pattern C							
	D.S.P.				Po-90 and D.S.P.				D.S.P. and Po-100			
	(1)	(2)	(3)	(4)	(1)	(2)	(3)	(4)	(1)	(2)	(3) △1	(4) △1
1974 - 75	Dec.20-29	6.5	1.40	4,700	Dec.11-20	6.5	0.80	8,100	Feb.11-20	6.0	0.86	7,000
1975 - 76	Jan. 1-10	7.6	1.75	4,500	Dec. 1-10	8.1	0.69	12,000	Feb. 1-10	7.0	1.08	6,500
1976 - 77	Dec.11-20	6.0	1.40	4,300	Nov.11-20	3.3	0.76	4,400	Feb. 1-10	8.2	(0.81)	(7,000)
1977 - 78	Jan.22-31	6.5	1.30	5,000	Oct.21-30	4.0	0.63	6,300	Jan.22-31	6.5	(0.64)	(3,600)
1978 - 79	Nov.11-20	5.3	0.73	7,300	Nov.11-20	5.3	0.64	8,300	Jan.22-31	6.4	(0.36)	(7,700)
1979 - 80	Nov.18-27	2.4	0.78	3,000	Nov.11-20	2.6	0.73	3,600	Feb. 1-10	7.4	(0.39)	(15,300)
1980 - 81	Nov.17-26	3.2	0.88	3,700	Nov.11-20	3.1	0.74	4,200	Feb.21-28	7.4	(0.59)	(11,100)
1981 - 82	Dec.17-26	8.5	1.12	7,400	Oct.11-20	4.8	0.51	9,300	Jan.22-31	9.2	(0.64)	(5,300)
Average	-	-	-	5,000	-	-	-	-	-	-	-	-

Year	Pattern B				Pattern D							
	D.S.P. and Po-100				Po-90 and Po-100(1)				Po-100(1) and Po-100(2)			
	(1)	(2)	(3)	(4)	(1)	(2)	(3) △2	(4) △2	(1)	(2)	(3) △3	(4) △3
1974 - 75	Dec.20-29	6.5	0.94	7,000	Nov. 1-10	7.2	0.45	15,900	Feb.11-20	6.0	0.71	8,500
1975 - 76	Jan. 1-10	7.8	1.34	5,800	Dec. 1-10	8.1	0.31	26,000	Feb. 1-10	7.0	0.88	8,000
1976 - 77	Dec.11-20	6.0	1.00	6,000	Oct.11-20	3.4	(1.12)	(3,100)	Feb. 1-10	8.2	(0.81)	(5,500)
1977 - 78	Jan.22-31	6.5	1.06	6,100	Oct.21-30	4.0	0.64	6,300	Jan.22-31	6.5	(0.64)	(6,100)
1978 - 79	Jan.22-31	6.4	0.68	9,500	Oct.11-20	5.6	(0.89)	(6,300)	Jan.22-31	6.4	0.42	15,200
1979 - 80	Nov.18-27	2.4	0.46	5,100	Oct.21-30	2.9	0.55	5,200	Feb. 1-10	7.4	(0.39)	(16,100)
1980 - 81	Nov.17-26	3.2	0.57	5,700	Oct.11-12	2.9	(1.01)	(2,900)	Feb.21-28	7.4	(0.59)	(10,400)
1981 - 82	Jan.22-31	9.2	1.01	9,100	Oct.11-20	4.5	(1.01)	(4,700)	Jan.22-31	9.2	(0.64)	(7,600)
Average	-	-	-	6,800	-	-	-	-	-	-	-	-

Note : D.S.P.: Dry season paddy Po-90 : Polowijo crop of 90 days growing period Po-100 : Polowijo crop of 100 days growing period

(1) : Critical low flow period

(2) : Average value of Sanrego River flow discharge in the critical low flow period (m3/sec)

(3) : Combined diversion water requirement (l/sec/ha)

(4) : Irrigable area (ha)

△1 : Value in ( ) is for Po-100

△2 : Value in ( ) is for Po-90

△3 : Value in ( ) is for Po-100(2)

x : Value of more than 20,000

Table VI.3.3 Summary of Irrigable Area with the Sanrego River Flow

(Unit : ha)

Year	Wet Season	Dry Season Crop									
		Pattern A	Pattern B		Pattern C			Pattern D			
	W.S.P.	D.S.P.	D S.P.	Po-100	D.S.P.	Po-90	Po-100	Po-90	Po-100(1)	Po-100(2)
1974-75	6,400	4,700	3,500	3,500	3,500	4,000	3,500	8,000	4,300	4,200
1975-76	9,100	4,500	2,900	2,900	3,300	6,000	3,200	13,000	4,000	4,000
1976-77	7,100	4,300	3,000	3,000	2,200	2,200	7,000	3,100	4,700	5,500
1977-78	5,400	5,000	3,100	3,000	3,200	3,100	3,600	3,100	3,200	6,100
1978-79	6,300	7,300	4,800	4,700	4,200	4,100	7,700	6,300	7,600	7,600
1979-80	9,400	3,000	2,600	2,500	1,800	1,800	15,300	2,600	2,600	16,100
1980-81	9,200	3,700	2,900	2,800	2,100	2,100	11,100	2,900	4,000	10,400
1981-82	6,800	7,400	4,600	4,500	4,700	4,600	5,300	4,700	5,700	7,600
1982	6,800	-	-	-	-	-	-	-	-	-
Average	7,400	5,000	3,400	3,400	3,100	3,500	7,100	5,500	4,500	7,700
80% Dependability	6,300	3,700	2,800	2,800	2,200	2,300	3,800	2,800	3,300	(4,000)

Note : W.S.P. ; Wet Season Paddy
D.S.P. ; Dry Season Paddy
Po-90 ; Polowijo of 90 days growing period
Po-100 ; Polowijo of 100 days growing period
(4,000) ; Without dependability analysis

Table VI.3.4 Irrigable Area of Wet Season Paddy in Critical low Flow Period (with Sanrego River and Tributaries Flow)

Year	Critical Low Flow Period	Average Flow Discharge			Diversion Water Requirement	Irrigable Area
		Sanrego River	Tributaries	Total		
		(m3/sec)	(m3/sec)	(m3/sec)	(l/sec/ha)	(ha)
1974	Jun. 1-10	8.5	2.0	10.5	1.32	7,900
1975	Jul. 1-10	9.6	2.3	11.9	1.06	11,300
1976	May 12-21	8.2	2.0	10.2	1.17	8,800
1977	May 11-20	7.0	1.7	8.7	1.30	6,700
1978	May 22-31	7.5	1.8	9.3	1.19	7,800
1979	May 11-20	8.0	1.9	9.9	0.85	11,700
1980	Jul. 1-10	8.8	2.1	10.9	0.96	11,400
1981	Jun. 1-10	9.7	2.3	12.0	1.43	8,400
1982	May 11-20	7.3	1.8	9.1	1.07	8,400
Average						9,200

Table VI.3.5 Irrigable Area of Dry Season Crop in Critical Low Flow Period
(with Sanrego River and Tributaries Flow)

Year	Pattern A				Pattern C							
	D.S.P.				Po-90 and D.S.P.				D.S.P. and Po-100			
	(1)	(2)	(3)	(4)	(1)	(2)	(3)	(4)	(1)	(2)	(3) ⊿1	(4) ⊿1
1974 - 75	Dec.20-29	6.1	1.40	5,800	Dec.11-20	8.1	0.80	10,100	Feb.11-20	7.4	0.86	8,600
1975 - 76	Jan. 1-10	9.7	1.75	5,600	Nov.21-30	8.9	0.62	14,400	Feb. 1-10	8.7	1.08	8,700
1976 - 77	Nov.11-20	3.3	0.76	4,400	Nov.11-20	3.3	0.72	4,400	Feb. 1-10	10.2	(0.81)	(9,400)
1977 - 78	Jan.22-31	8.1	1.30	6,200	Oct.21-30	4.0	0.63	6,300	Jan.22-31	8.1	(0.64)	(6,200)
1978 - 79	Nov.11-20	5.3	0.73	7,300	Nov.11-20	5.3	0.64	8,300	Jan.22-31	7.9	(0.36)	(11,800)
1979 - 80	Nov.18-27	2.4	0.78	3,000	Nov.11-20	2.6	0.73	3,600	Feb. 1-10	9.2	(0.39)	(x)
1980 - 81	Nov.17-26	3.2	0.88	3,700	Nov.11-20	3.1	0.74	4,200	Feb.21-28	9.2	(0.59)	(14,500)
1981 - 82	Dec.17-26	10.4	1.12	9,200	Oct.11-20	4.8	0.51	9,300	Jan.22-31	11.4	(0.64)	(12,200)
Average	-	-	-	5,700								

Year	Pattern B				Pattern D							
	D.S.P. and Po-100				Po-90 and Po-100(1)				Po-100(1) and Po-100(2)			
	(1)	(2)	(3)	(4)	(1)	(2)	(3) ⊿2	(4) ⊿2	(1)	(2)	(3) ⊿3	(4) ⊿3
1974 - 75	Dec.20-29	8.1	0.94	8,700	Nov. 1-10	7.2	0.45	15,900	Feb.11-20	7.4	0.71	10,400
1975 - 76	Jan. 1-10	9.7	1.34	7,200	Dec. 1-10	10.0	0.31	x	Feb. 1-10	8.7	0.88	10,000
1976 - 77	Nov.11-20	3.3	0.48	7,000	Oct.11-20	3.4	(1.12)	(3,100)	Feb. 1-10	10.2	(0.81)	(9,500)
1977 - 78	Jan.22-31	8.1	1.06	7,600	Oct.21-30	4.0	0.64	6,300	Jan.22-31	8.1	(0.64)	(8,700)
1978 - 79	Jan.22-31	7.9	0.66	11,800	Oct.11-20	5.6	(0.89)	(6,300)	Jan.22-31	7.9	0.42	18,800
1979 - 80	Nov.18-27	2.4	0.46	5,100	Oct.21-30	2.9	0.55	5,200	Feb. 1-10	9.2	(0.39)	(x)
1980 - 81	Nov.17-26	3.2	0.57	5,700	Oct.11-12	2.9	(1.01)	(2,900)	Feb.21-28	9.2	(0.59)	(14,900)
1981 - 82	Jan.22-31	11.4	1.01	11,300	Oct.11-20	4.5	(1.01)	(4,700)	Jan.22-31	11.4	(0.64)	(15,000)
Average	-	-	-	8,100	-	-	-	-	-	-	-	-

Note : D.S.P. : Dry season paddy
Po-90 : Polowijo crop of 90 days growing period
Po-100 : Polowijo crop of 100 days growing period

(1) : Critical low flow period
(2) : Average value of Sanrego river and Tributaries flow discharge in the critical low flow period (m3/sec)
(3) : Combined diversion water requirement (l/sec/ha)
(4) : Irrigable area (ha)

⊿1 : Irrigable area (ha)
⊿2 : Value in ( ) is for Po-100
⊿3 : Value in ( ) is for Po-100(2)
x : Value of more than 20,000

Table VI.3.6 Summary of Irrigable Area with Sanrego and Tributaries Flows

(Unit : ha)

Year	Wet Season	Dry Season Crop								
		Pattern A	Pattern B		Pattern C			Pattern D		
	W.S.P.	D.S.P.	D.S.P.	Po-100	D.S.P.	Po-90	Po-100	Po-90	Po-100(1)	Po-100(2)
1974-75	7,900	5,800	4,400	4,300	4,300	5,000	4,300	8,000	5,200	5,200
1975-76	11,300	5,600	3,600	3,600	4,100	7,200	4,000	16,100	5,000	5,000
1976-77	8,800	4,400	3,500	3,500	2,200	2,200	9,400	3,100	4,700	9,500
1977-78	6,700	6,200	3,800	3,800	3,200	3,100	6,200	3,100	3,200	8,700
1978-79	7,800	7,300	5,900	5,900	4,200	4,100	11,800	6,300	9,400	9,400
1979-80	11,700	3,000	2,600	2,500	1,800	1,800	(20,000)	2,600	2,600	(20,000)
1980-81	11,400	3,700	2,900	2,800	2,100	2,100	14,500	2,900	4,000	14,900
1981-82	8,400	9,200	5,700	5,600	4,700	4,600	12,200	4,700	5,700	15,000
1982	8,400	-	-	-	-	-	-	-	-	-
Average	9,200	5,700	4,100	4,000	3,300	3,800	-	6,000	5,000	-
80% Dependability	8,000	4,000	3,000	3,000	2,300	2,400	(4,000)	3,000	3,400	(4,000)

Note : W.S.P. ; Wet Season Paddy
D.S.P. ; Dry Season Paddy
Po-90 ; Polowijo of 90 days growing period
Po-100 ; Polowijo of 100 days growing period
(4,000) ; Without dependability analysis

Table VI.5.1 Maximum Design Discharge of Main and Secondary Canals

Name of Canal	Design Discharge (m^3/sec) F/S (1)	DOI Design (2)	(1)/(2)
Main Canal	11.44	12.91	0.89
Secondary Canal			
Palakka	2.88	3.51	0.82
Parota	2.06	2.58	0.80
Labosi	0.44	0.57	0.70
Topale	0.23	0.33	0.70
Apale	0.70	0.95	0.74
Pao	0.42	0.55	0.76
Maradda	0.27	0.43	0.63
Batu	0.43	0.63	0.68
Jaramele	0.61	0.36	1.69
Cendranae	0.37	0.39	0.95
Barang	0.20	0.25	0.80
Aming	5.27	6.20	0.85
Lume	0.14	0.26	0.54
Masago	0.47	0.55	0.85
Paccing	0.77	0.86	0.90
Hulo	0.17	0.27	0.63
Sama Enre	1.01	1.11	0.91
Poparapa	0.21	0.25	0.84

Table VI.5.2 Comparison of the Construction Cost of the Earth Work of Main and Secondary Canals

Working Item	In case of Design by D.O.I.			In case of Design by F/S Team		
	Unit	Quantities	Approx.Cost (10^6 x Rp.)	Unit	Quantities	Approx.Cost (10^6 x Rp.)
1. Stripping	10^3 x m3	421	547.3	10^3 x m3	410	533.0
2. Excavation						
- Common soil	10^3 x m3	506	328.9	10^3 x m3	430	279.5
- Weathered Rock	10^3 x m3	126	541.8	10^3 x m3	110	473.0
- Rock	10^3 x m3	132	950.4	10^3 x m3	120	864.0
3. Embankment	10^3 x m3	1,155	1,617.0	10^3 x m3	1,085	1,519.0
T o t a l			3,985.4 (3,990.0)			3,668.5 (3,670.0)

Table VI.5.3 Irrigable Area on the Secondary Block

Name of Canal	No. of Turnout	Irrigable Area (ha)
Main Canal	At Intake	8,000
	B S 1	8,000
	B S 2	5,940
	B S 3	5,786
	B S 4	5,424
	B S 5	5,350
	B S 6	5,088
	B S 7	5,067
	B S 8	4,662
	B S 9	4,200
	B S 10	3,872
Aming Sec.	B. Point	3,685
Palaka Sec.	B. Point	2,015
Pao Sec.	B. Point	294
Maradda Sec.	B. Point	188
Batu Sec.	B. Point	304
Jaramele Sec.	B. Point	425
Cendranae Sec.	B. Point	257
Barang Sec.	B. Point	143
Lume Sec.	B. Point	100
Masago Sec.	B. Point	331
Paccing Sec.	B. Point	539
Poparapa Sec.	B. Point	144
Parota Sec.	B. Point	1,442
Labosi Sec.	B. Point	306
Tapale Sec.	B. Point	162
Apale Sec.	B. Point	491
Hulo Sec.	B. Point	118
Sama Enre Sec.	B. Point	705

Table VI.5.4 Irrigable Area on the Tertiary Block (1/2)

Name of Canal	Name of Turnout	Name of Tertiary Block	Irrigable Area (ha)
Main Canal	BS1	S1kr	45
	BS2	S2krkr	31
		S2krtgkr	61
		S2krtgkn	30
		S2krkn	32
	BS3	S3krkr	43
		S3krkn	25
	BS4	S4krkr	14
		S4krkn	60
	BS5	S5krkr	42
		S5krkn	32
	BS6	S6kr	21
	BS7	S7krkr	49
		S7krkn	52
	BS8	S8krkr	15
		S8krkn	22
	BS9	S9krkr	13
		S9krkn	58
	BS10	S10krkr	13
		S10krkn	31
Aming Sec.	BAm1	Am1kr	28
	BAm2	Am2krkr	51
		Am2krkn	40
	BAm3	Am3kr	52
	BAm4	Am4kr	69
	BAm5	Am5kr	47
	BAm6	Am6kr	26
	BAm7	Am7krkr	40
		Am7krkn	43
	BAm8	Am8krkr	28
		Am8krtg	47
		Am8krkn	36
	BAm9	Am9kr	17
	BAm10	Am10kr	19
		Am10kn	31
	BAm11	Am11kr	56
	BAm12	Am12krkr	56
		Am12krtg	37
		Am12krkn	54
	BAm13	Am13krkr	63
		Am13krkn	16
		Am13kn	11
	BAm14	Am14kr	36
		Am14knkr	77
		Am14knkn	25
	BAm15	Am15kr	19
	BAm16	Am16kr	56
	BAm17	Am17krkr	31
		Am17krkn	45
		Am17krtgkr	49
		Am17krtgkn	60
	BAm18	Am18kr	25
Aming Sec.	BAm19	Am19krkr	29
		Am19krkn	54
	BAm20	Am20kr	25
	BAm21	Am21kr	70
	BAm22	Am22kr	18
	BAm23	Am23krkr	20
		Am23krkn	16
	BAm24	Am24kr	38
	BAm25	Am25kr	26
	BAm26	Am26kr	61
	BAm27	Am27kr	64
	BAm28	Am28kr	51
	BAm29	Am29krkr	34
		Am29krkn	34
Samo Enre Sec.	BSE1	SE1krkr	60
		SE1krkn	36
	BSE2	SE2krkr	42
		SE2krkn	34
	BSE3	SE3kr	63
		SE3knkr	57
		SE3knkn	38
	BSE4	SE4kr	42
		SE4krtg	56
	BSE5	SE5tgkr	33
		SE5tgtg	62
		SE5tgkn	21
		SE5kn	22
Palaka Sec.	BPa1	Pa1kn	26
	BPa2	Pa2kn	40
	BPa3	Pa3kr	51
		Pa4kn	54
	BPa4	Pa4kr	39
		Pa4kn	58
	BPa5	Pa5krkr	49
		Pa5krtg	32
		Pa5krkn	28
		PA5kn	28
	BPa6	Pa6kr	42
		Pa6kn	44
	BPa7	Pa7kr	24
		Pa7kn	58
Pao Sec.	BP1	P1kr	64
		P1kn	14
	BP2	P2kr	35
		P2tg	63
		P2kn	35
	BP3	P3kr	29
		P3kn	53
Maradda Sec.	BMa1	Ma1krtg	23
		Ma1krkn	55
		Ma1kn	28
	BMa2	Ma2kr	20
		Ma2tg	40
		Ma2kn	22
Batu Sec.	BBT1	BT1kr	38
		BT1knkr	40
		BT1knkn	50
	BBT2	BT2kr	20
		BT2kn	26
	BBT3	BT3kr	13
		BT3tg	68
		BT5kn	40
Jaramele Sec.	BJr1	Jr1kr	51
		Jr1kn	36
	BJr2	Jr2kr	56
		Jr2tg	41
		Jr2kn	68
	BJr3	Jr3kr	31
	BJr4	Jr4kn	38
	BJr5	Jr5kr	53
		Jr5kn	51
Cendranae Sec.	BCa1	Ca1kr	55
		Ca1kn	12
	BCa2	Ca2knkr	24
		Ca2kntg	44
		Ca2knkn	20
	BCa3	Ca3kr	49
		Ca3tg	53
Masego Sec.	BM1	M1krkr	35
		M1krkn	42
		M1kn	48
	BM2	M2kr	57
		M2tg	42
		M2knkr	59
		M2knkn	48
Perota Sec.	BPr1	Pr1kn	16
	BPr2	Pr2kn	40
	BPr3	Pr3kn	38
	BPr4	Pr4kn	53
	BPr5	Pr5knkr	32
		Pr5knkn	20
	BPr6	Pr6knkr	46
		Pr6knkn	31
	BPr7	Pr7kn	56
	BPr8	Pr8krkr	35
		Pr8krkn	22
		Pr8knkr	28
		Pr8knkn	66

Table VI.5.4 Irrigable Area on the Tertiary Block (2/2)

Name of Canal	Name of Turnout	Name of Tertiary Block	Irrigable Area (ha)
Apale Sec.	BAp1	Ap1kr	36
	BAp2	Ap2kr	63
		Ap2kn	53
	BAp3	Ap3kr	61
		Ap3kn	32
	BAp4	Ap4kr	23
		Ap4kn	46
	BAp5	Ap5krkr	48
		Ap5krkn	16
		Ap5tg	64
		Ap5kn	49
Lume Sec.	BL1	L1kr	25
	BL2	L2kr	53
		L2kn	22
Paccing Sec.	BPc1	Pc1kr	50
		Pc1kn	33
	BPc2	Pc2kr	49
		Pc2kn	33
	BPc3	Pc3krkr	29
		Pc3krkn	40
		Pc3kn	16
Paccing Sec.	BPc4	Pc4knkr	41
		Pc4knkn	32
	BPc5	Pc5kr	41
		Pc5tg	65
		Pc5kn	32
Bulo Sec.	BB1	B1kr	42
		B1krtg	57
		B1kn	19
Poparapa Sec.	BPp1	Pp1kr	27
		Pp1knkr	62
		Pp1knkn	55
Labosi Sec1	BLb1	Lb1kr	66
		Lb1kn	23
	BLb2	Lb2knkr	44
		Lb2knkn	77
	BLb3	Lb3kr	48
		Lb3kn	48
Tapale Sec.	BTp1	Tp1kr	44
		Tp1kn	31
	BTp2	Tp2kr	65
		Tp2kn	42
Barang Sec.	BBa1	Ba1kr	20
		Ba1kn	26
	BBa2	Ba2kr	53
		Ba2kn	44
Baru A Sec.	BBA1	BA1kr	57
		BA1kn	56
Baru B Sec.	BBB1	BB1kr	55
		BB1kn	52

Table VI.7.1 General Features of Farm Road

Kind of Road	Total Length	Width of road		Pavement	
		Paved portion	Total	Paved material	Thickness
	(km)	(m)	(m)		(mm)
1. Existing all-weather road					
(a) Provincial road	20.3	-	-	Tar-macadam	-
(b) Construction road of intake weir	10.8	-	-	Gravel	-
2. Canal Inspection road					
(a) Main Canal	11.6	3.0	6.0	Gravel	200 (350)
(b) Secondary canal	85.0	3.0	6.0	"	200 (350)
3. Road Planned in Integrated Rural Development Project					
(a) Type 1	6.3	6.0	9.0	Gravel	275 (425)
(b) Type 2	5.8	4.5	7.5	"	"
(c) Type 3	12.1	3.5	6.5	"	"
4. Farm Road planned in F/S study	13.2	3.0	6.0	Gravel	200 (350)

Note : Value within brackets includes a compacted subgrade of 150 mm

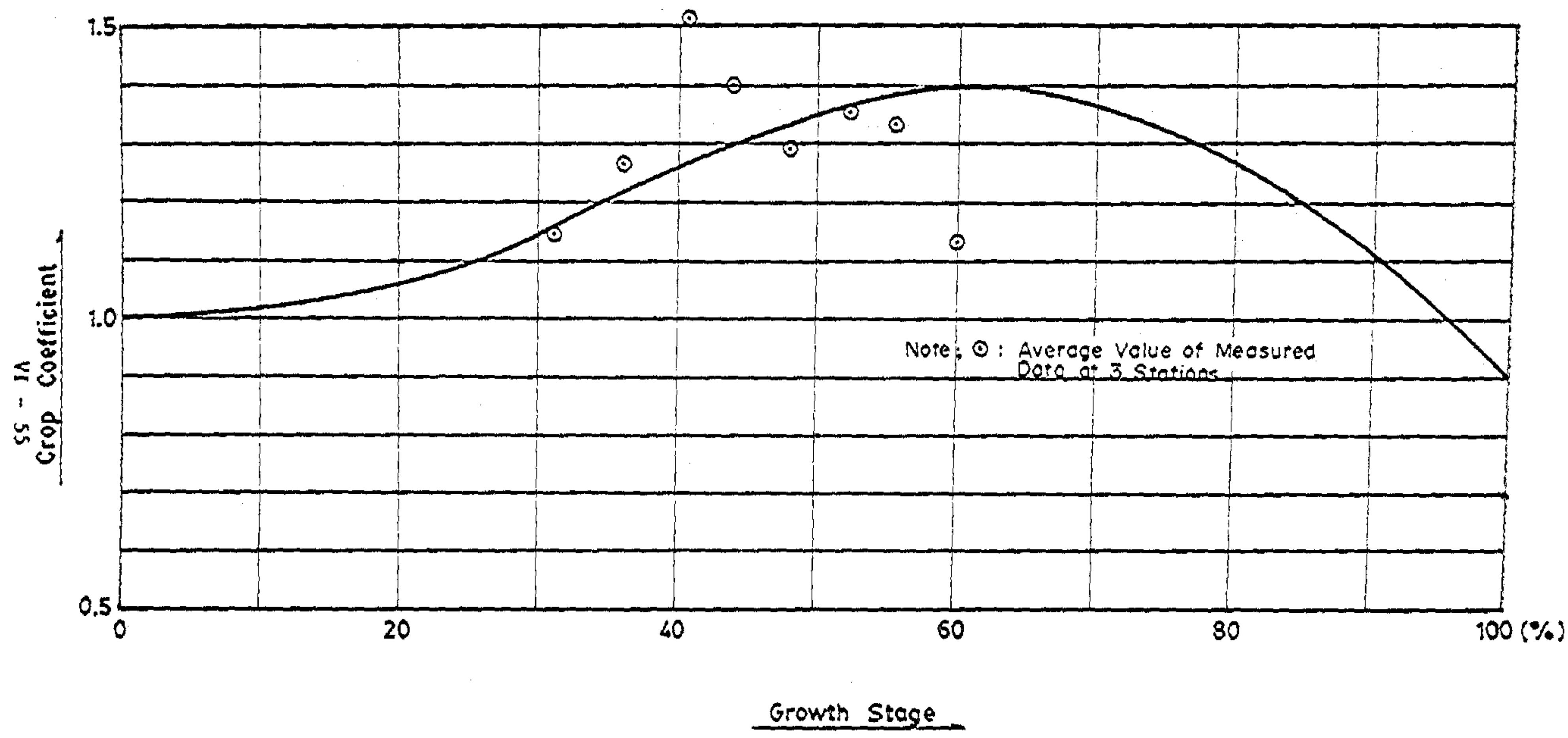


Fig. VI.2.1 CROP COEFFICIENT CURVE FOR PADDY

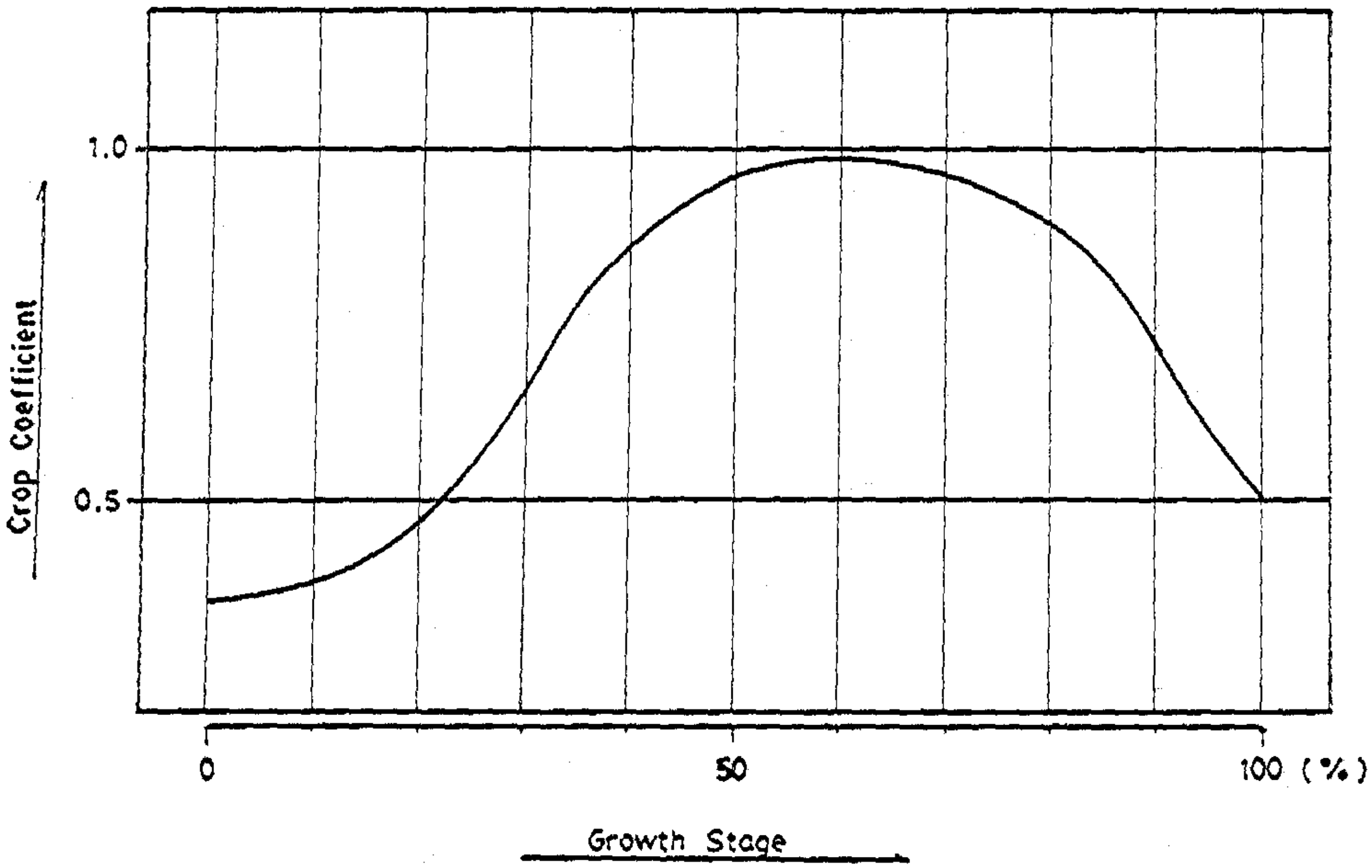


Fig. VI.2.2 CROP COEFFICIENT CURVE FOR POLWIJO CROP

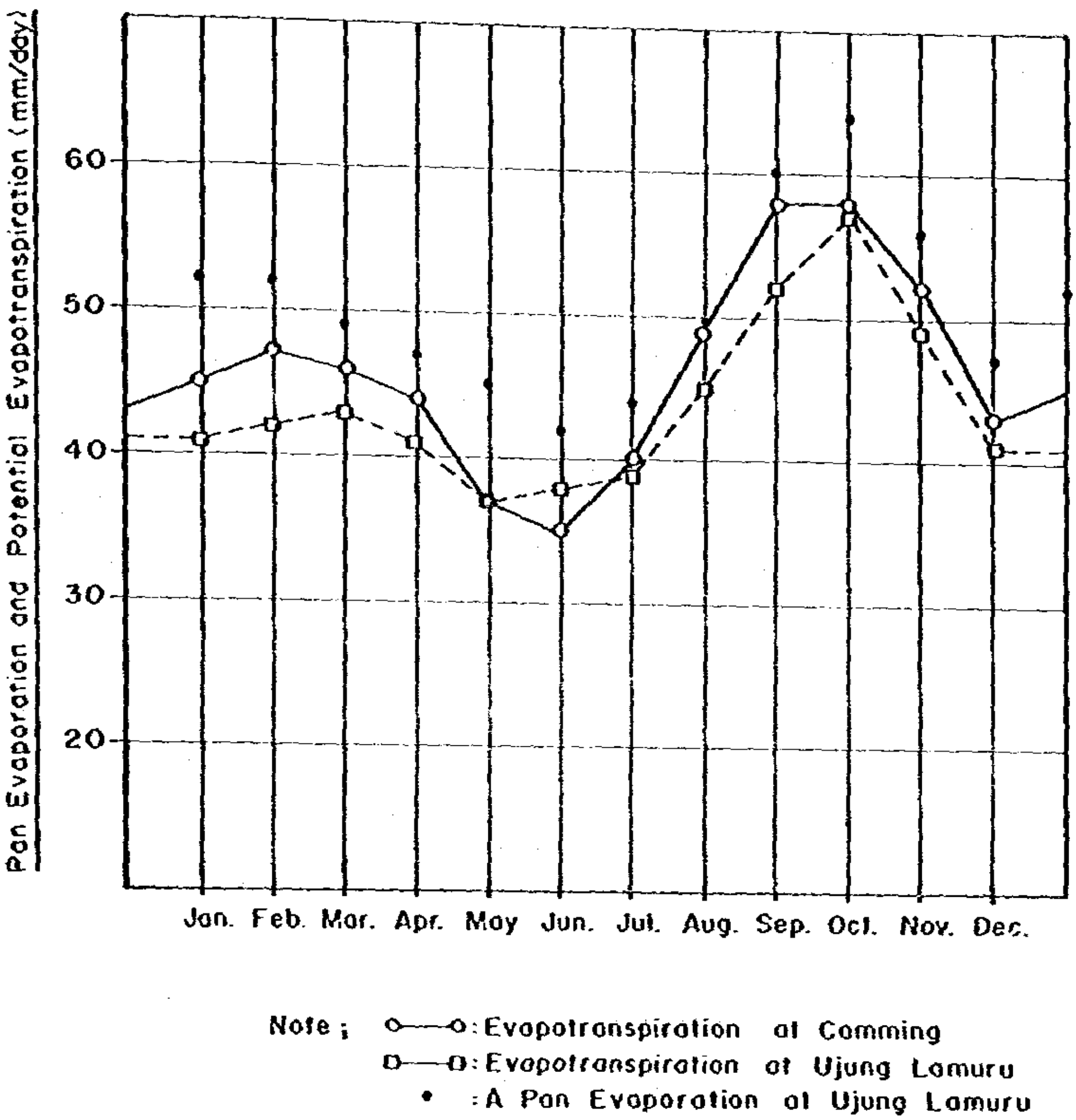


Fig. VI.2.3 COMPARISON OF ESTIMATED EVAPOTRANSPIRATION AT CAMMING AND UJUNG LAMUR

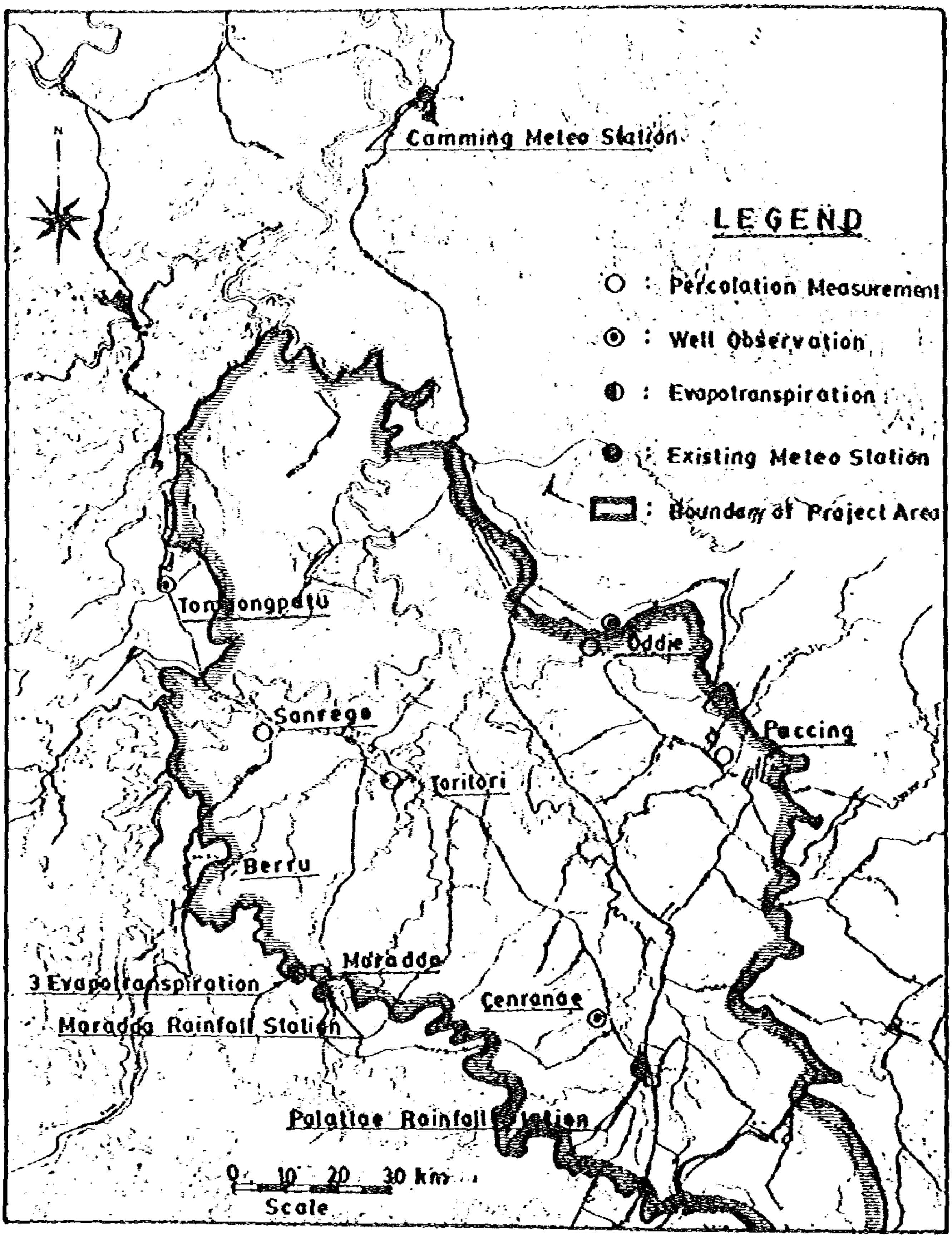


Fig. VI.2.4 LOCATIONS OF FIELD MEASUREMENT SITES

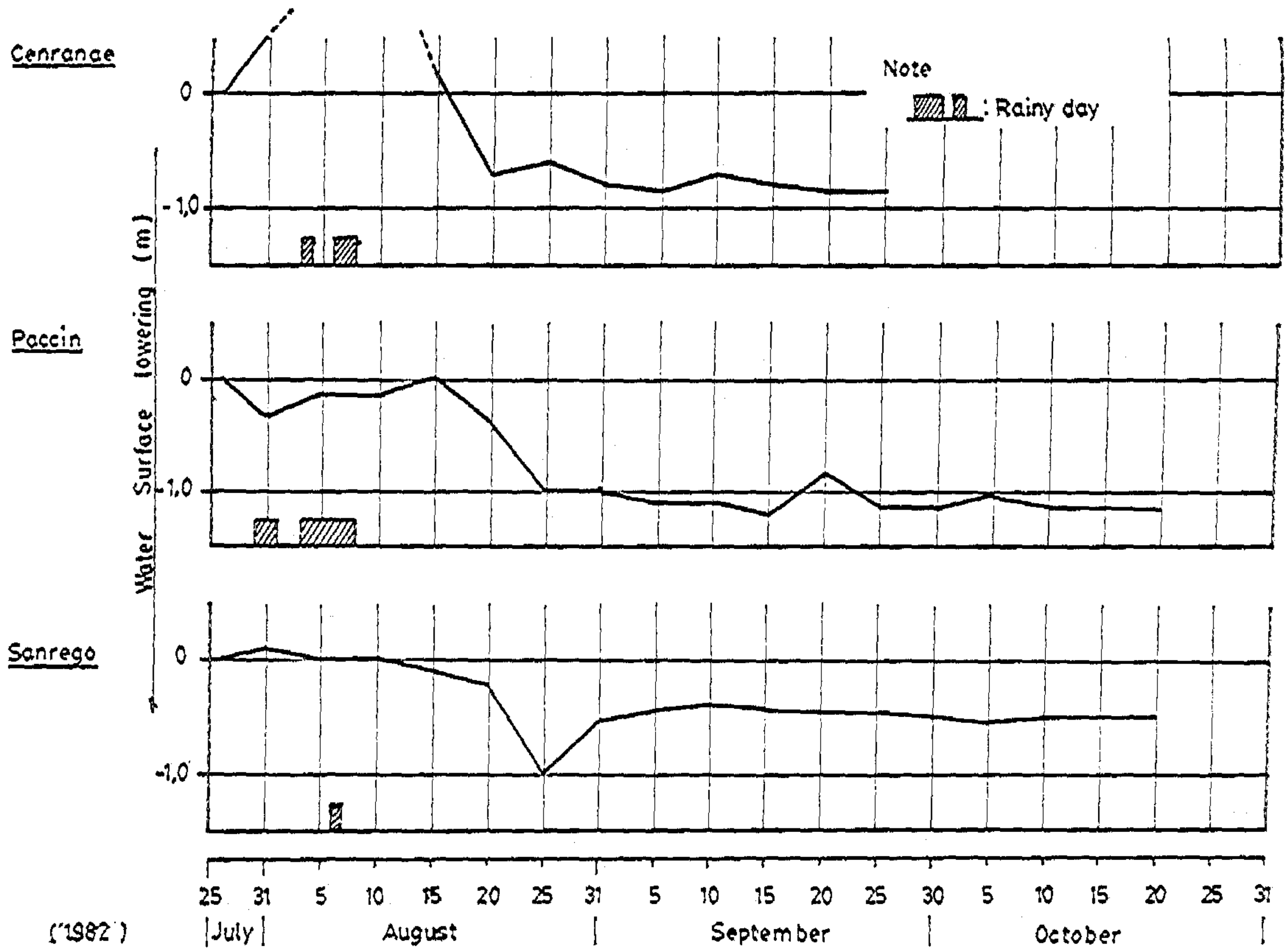


Fig. VI.2.5 WELL OBSERVATION RESULT

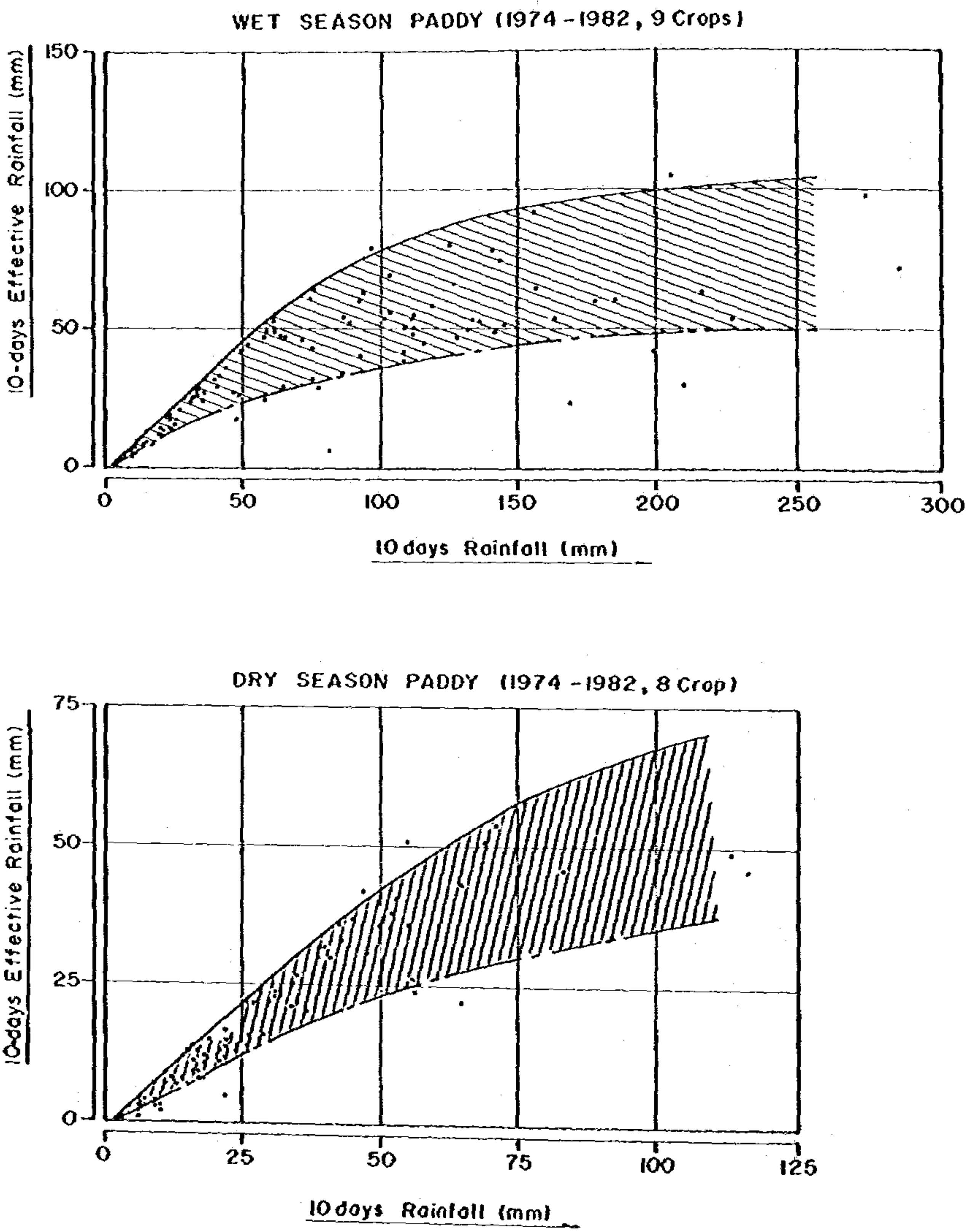


Fig. VI.2.6 RELATION BETWEEN DESIGN RAINFALL AND EFFECTIVE RAINFALL FOR PADDY (10 DAYS BASIS)

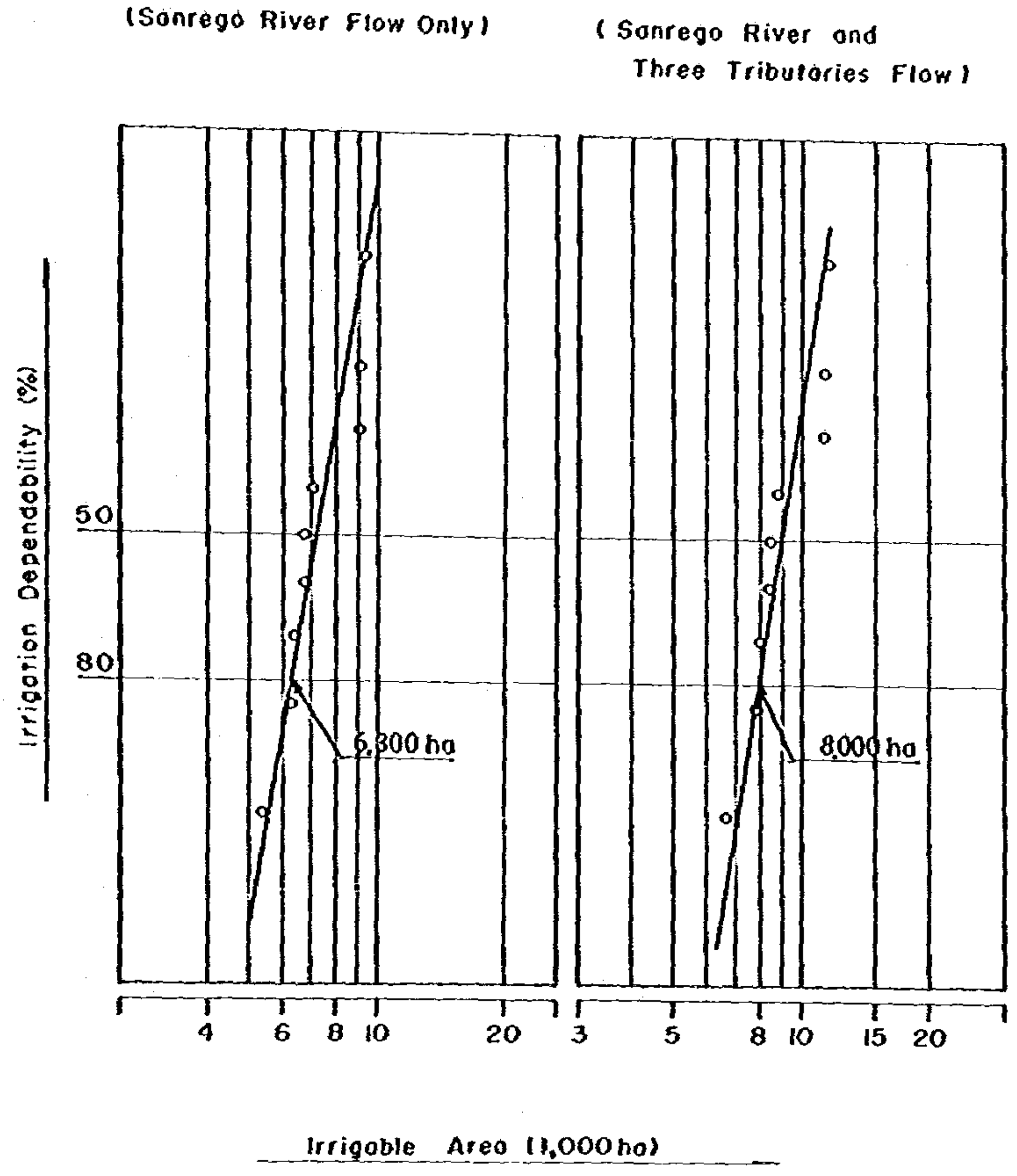


Fig. VI.3.1 IRRIGABLE AREA OF WET SEASON PADDY

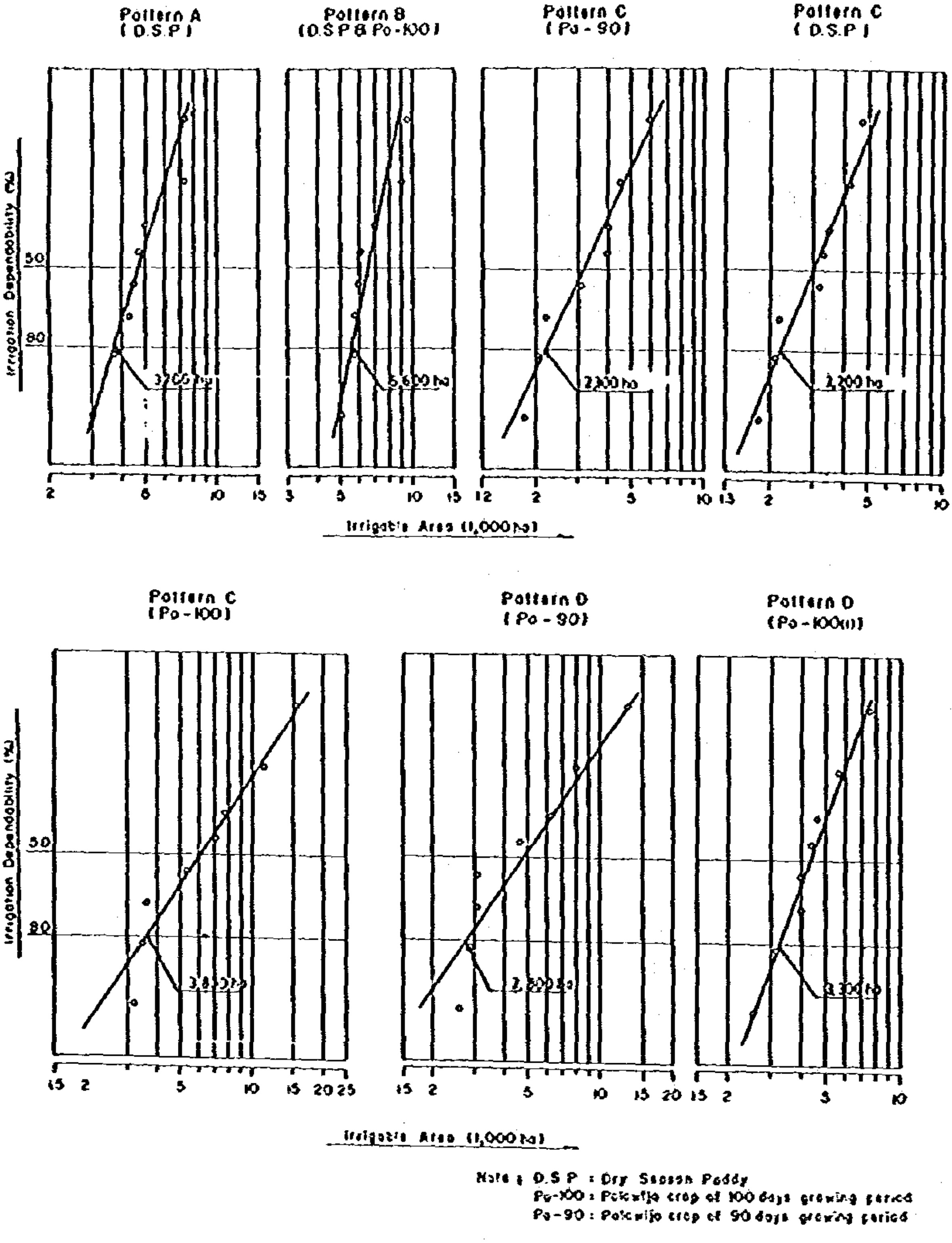


Fig. VI.3.2 IRRIGABLE AREA OF DRY SEASON CROP (WITH SANREGO RIVER FLOW)

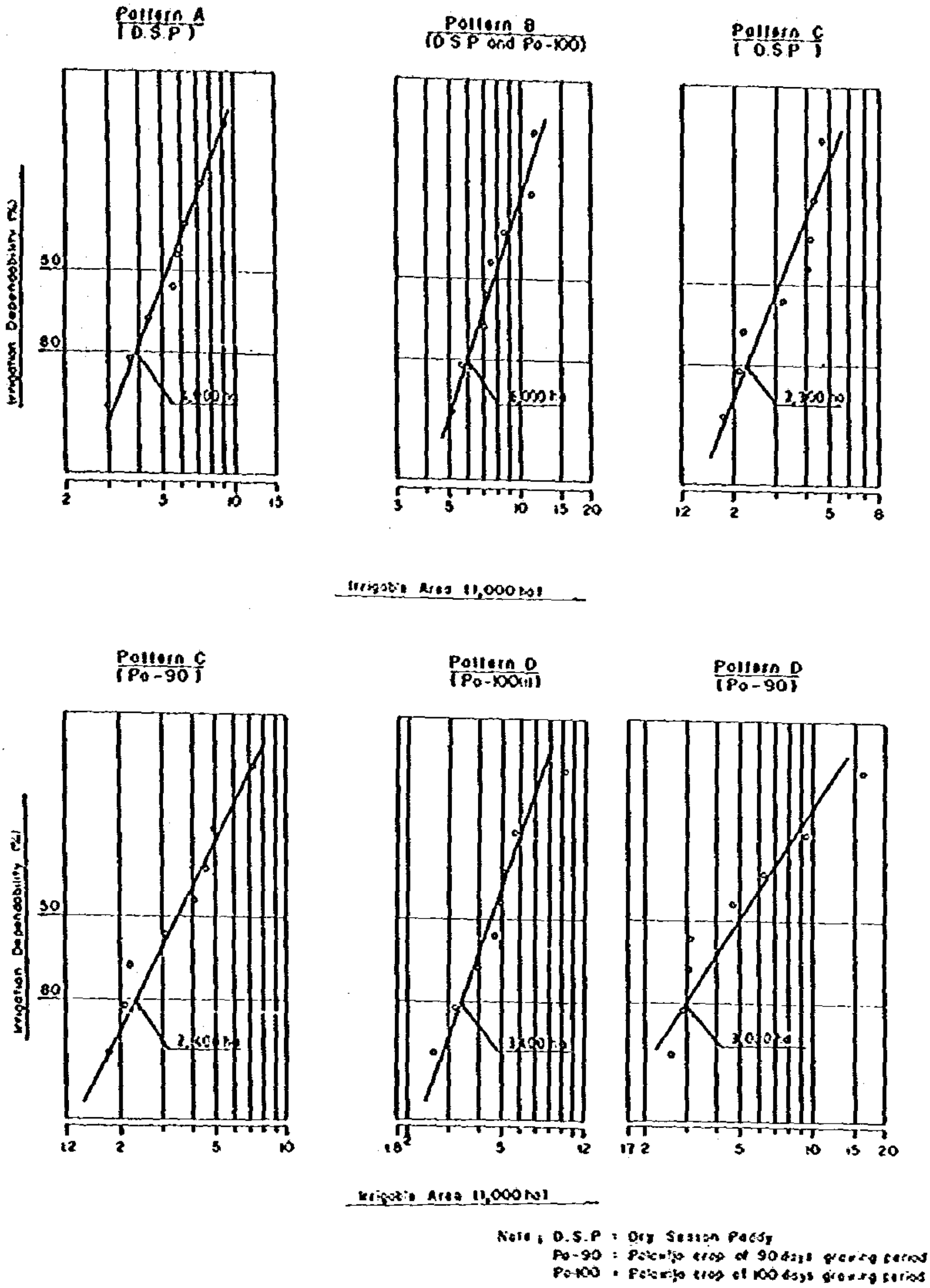


Fig. VI.3.3 IRRIGABLE AREA OF DRY SEASON CROP (WITH SANREGO RIVER AND TRIBUTARIES FLOW)

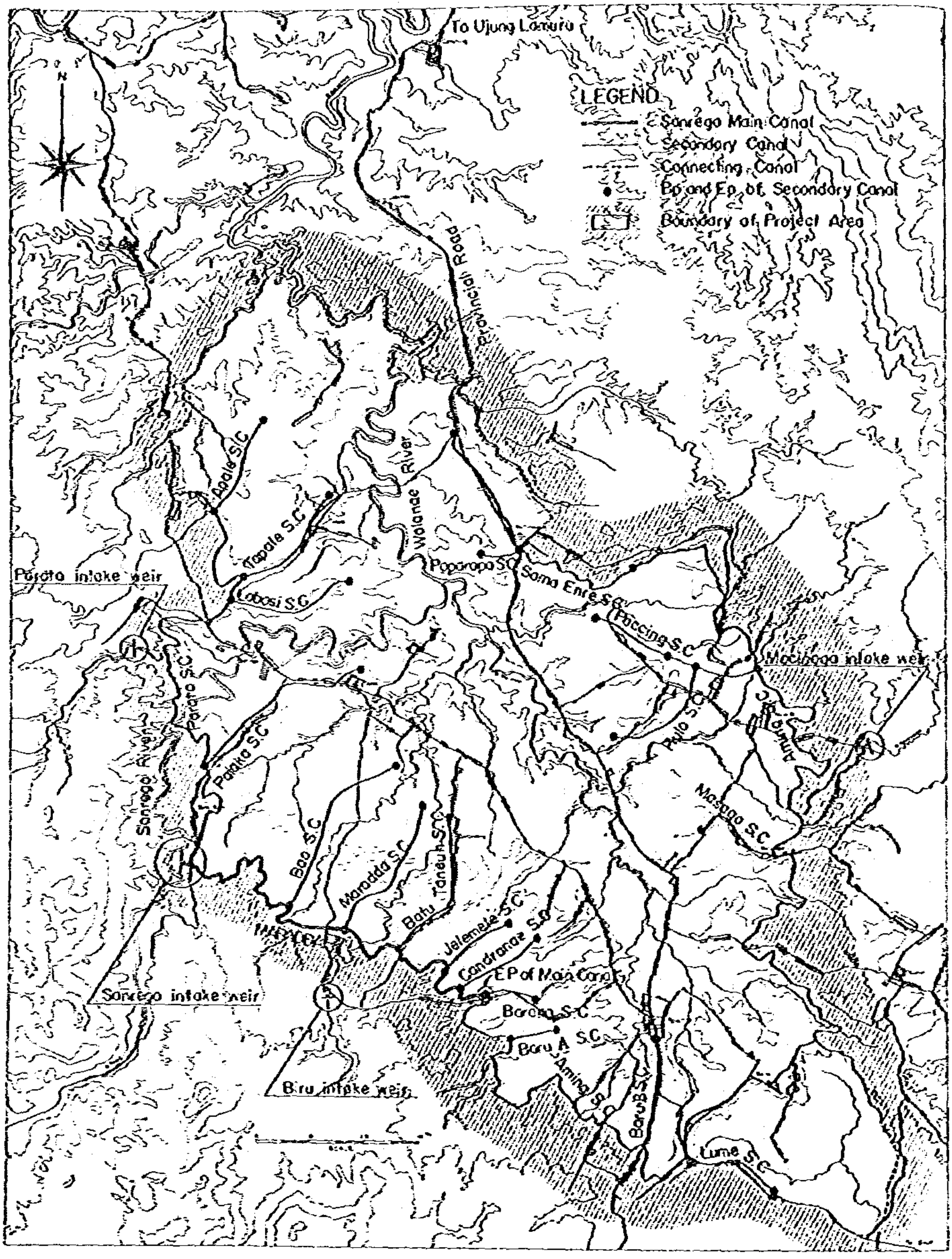
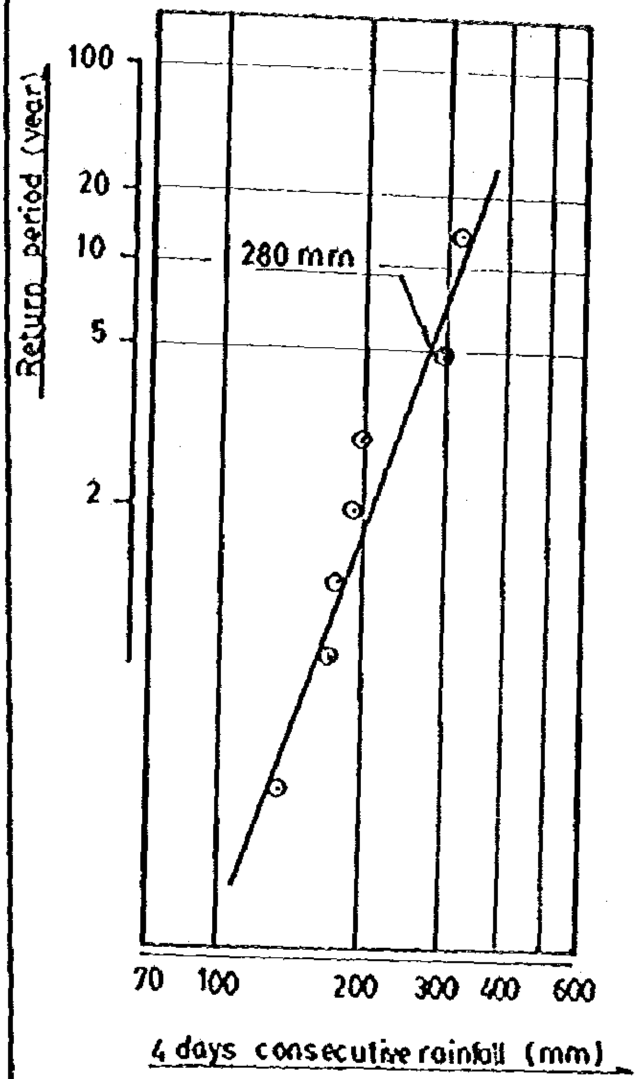


Fig. VI.5.1 IRRIGATION CANAL SYSTEM



Data Available

Year	4 days consecutive rainfall (mm)
1975	180
1976	199
1977	192
1978	172
1979	135
1980	322
1981	298
Average	214
5 Year returnperiod	280

Fig. VI.6.1 4 DAYS CONSECUTIVE RAINFALL AT PALATTAE

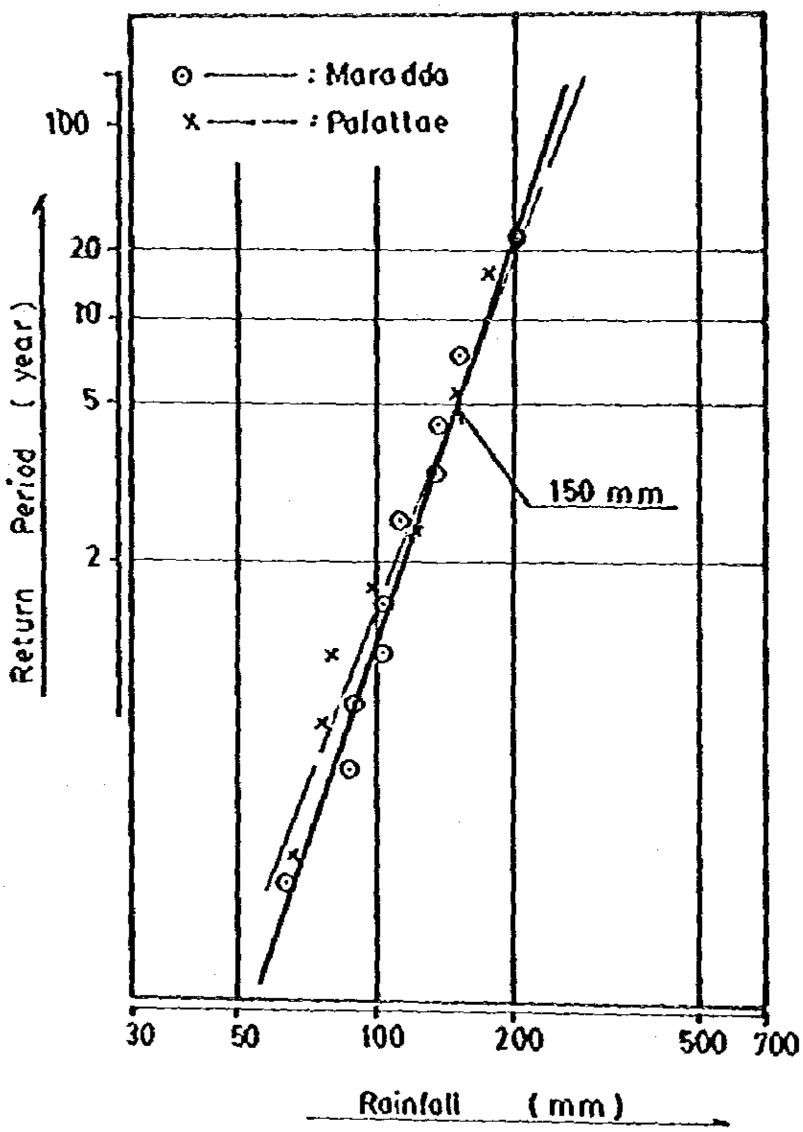


Fig. VI.6.2 PROBABLE DAILY RAINFALL AT PALATTAE AND MARRADA

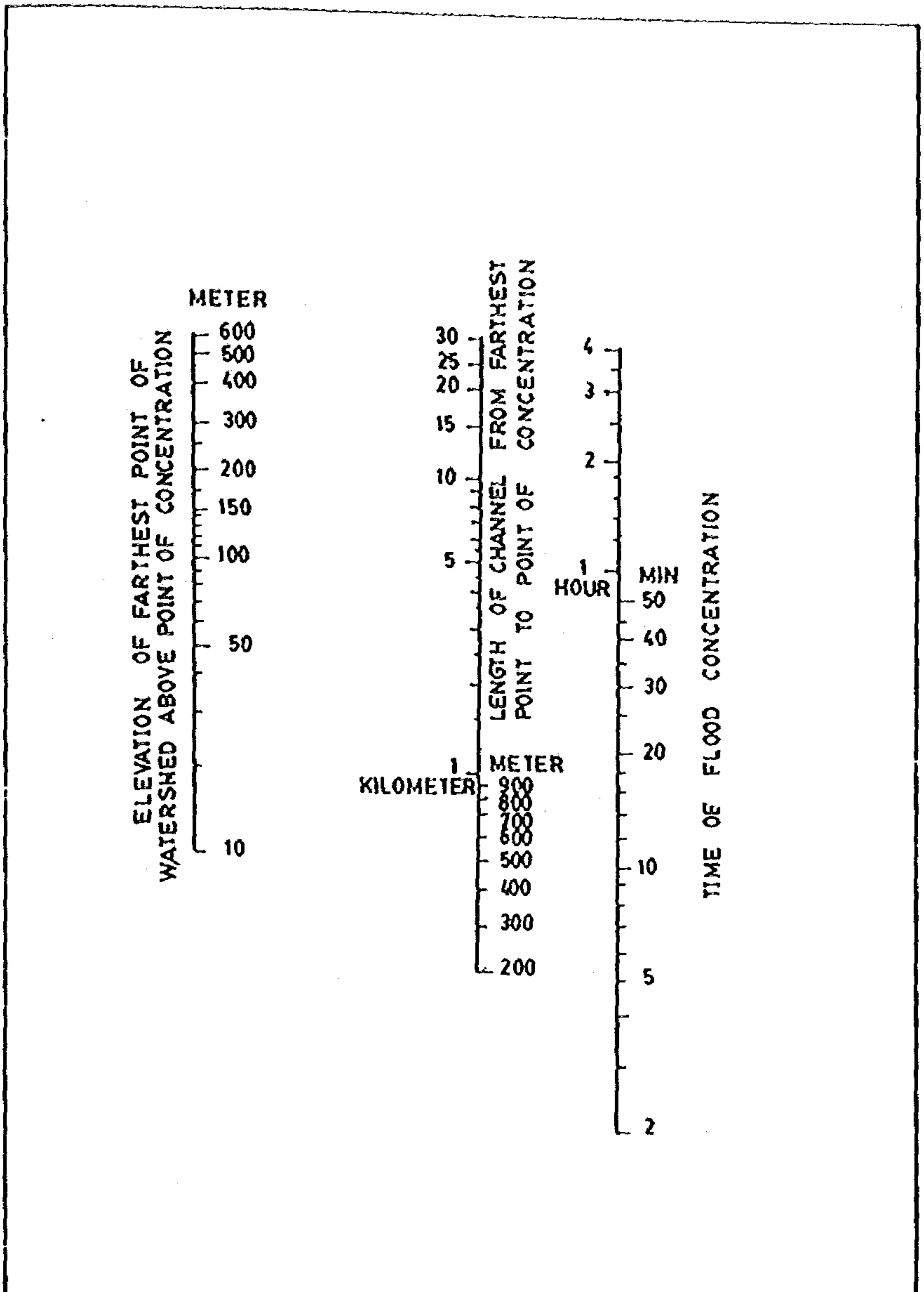


Fig. VI.6.3 NOMOGRAPH FOR ESTIMATING TIME OF FLOOD CONCENTRATION

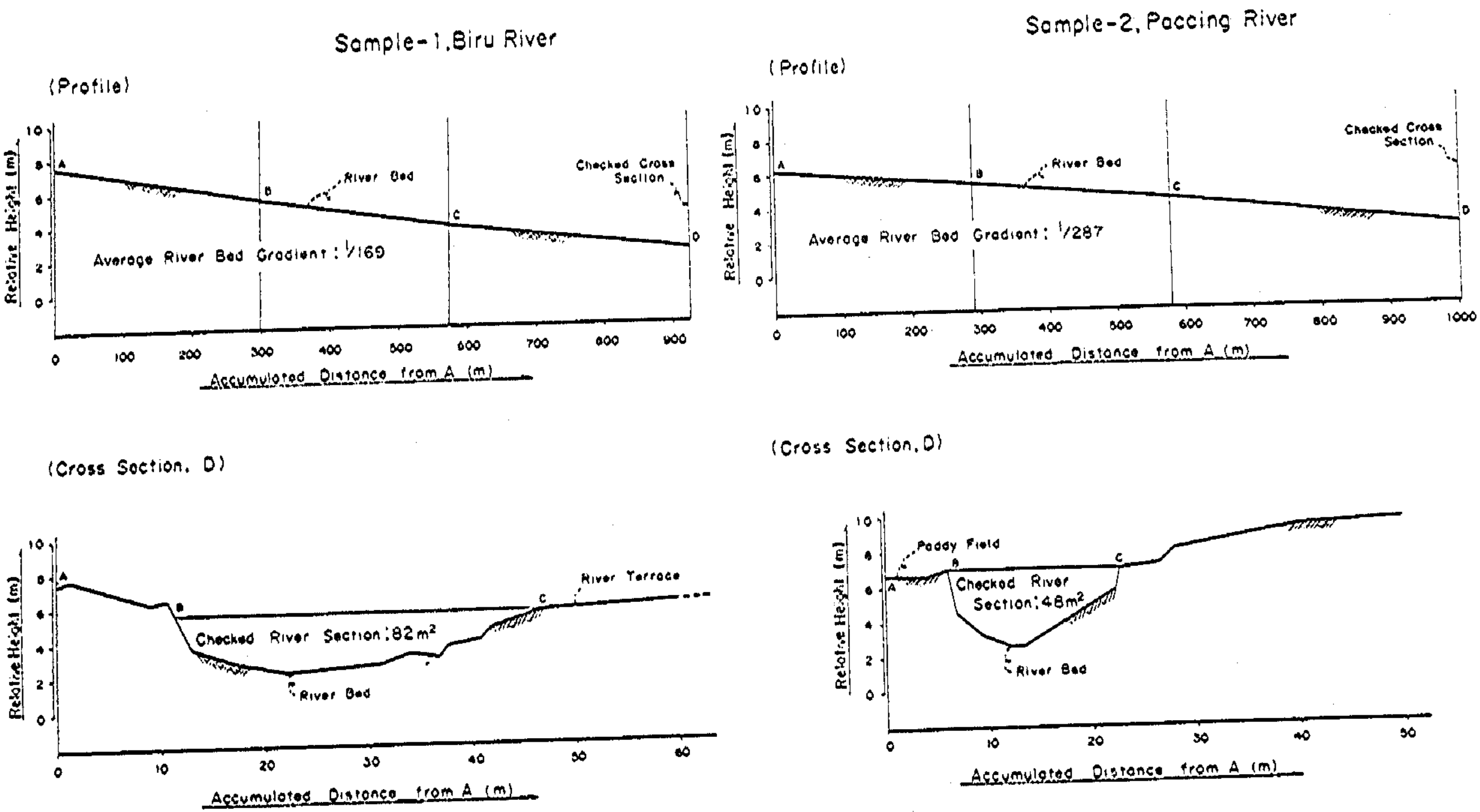


Fig. VI.6.4 RIVER CROSS SECTIONS AND PROFILES

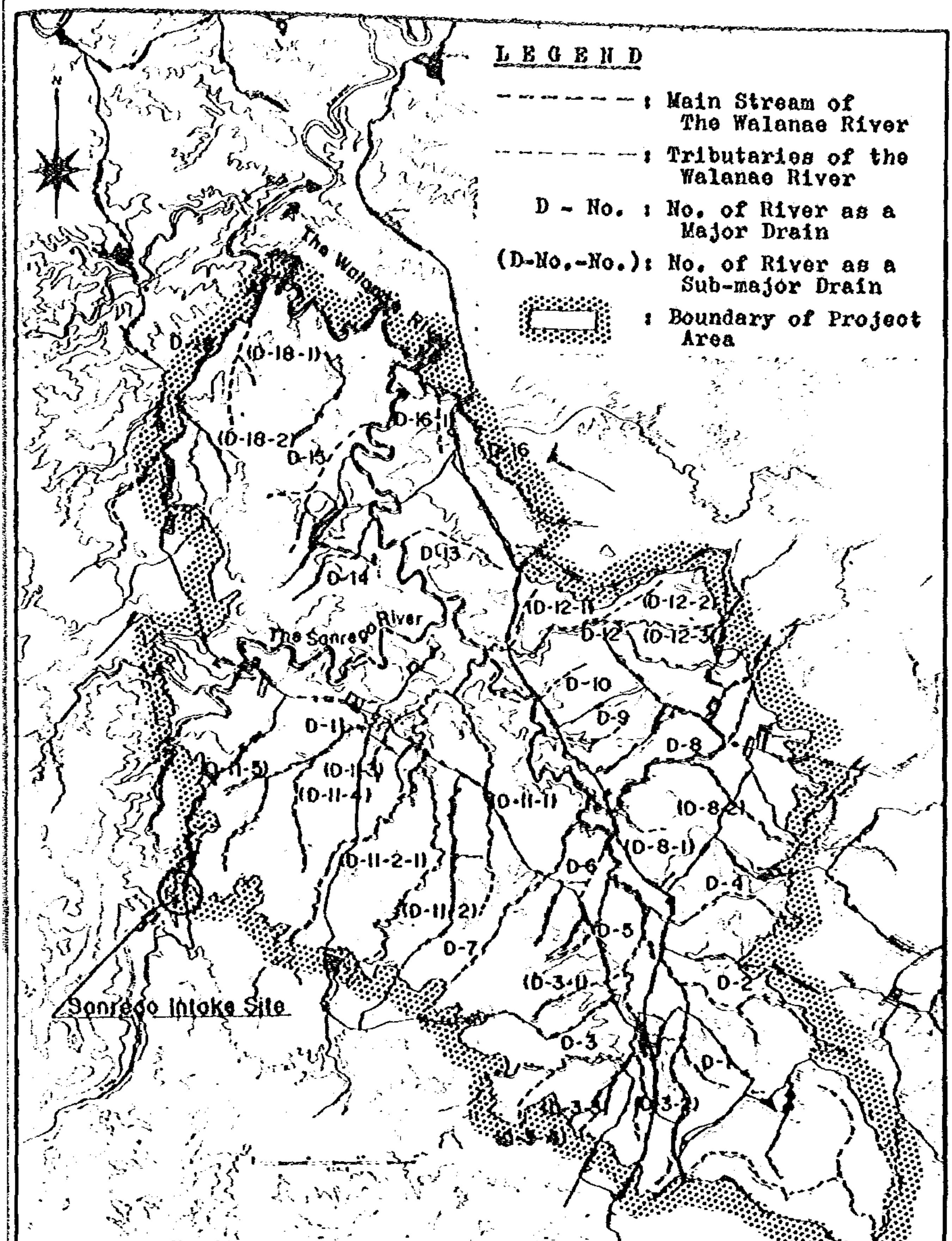


Fig. VI.6.5 NATURAL STREAMS AS MAJOR DRAINS

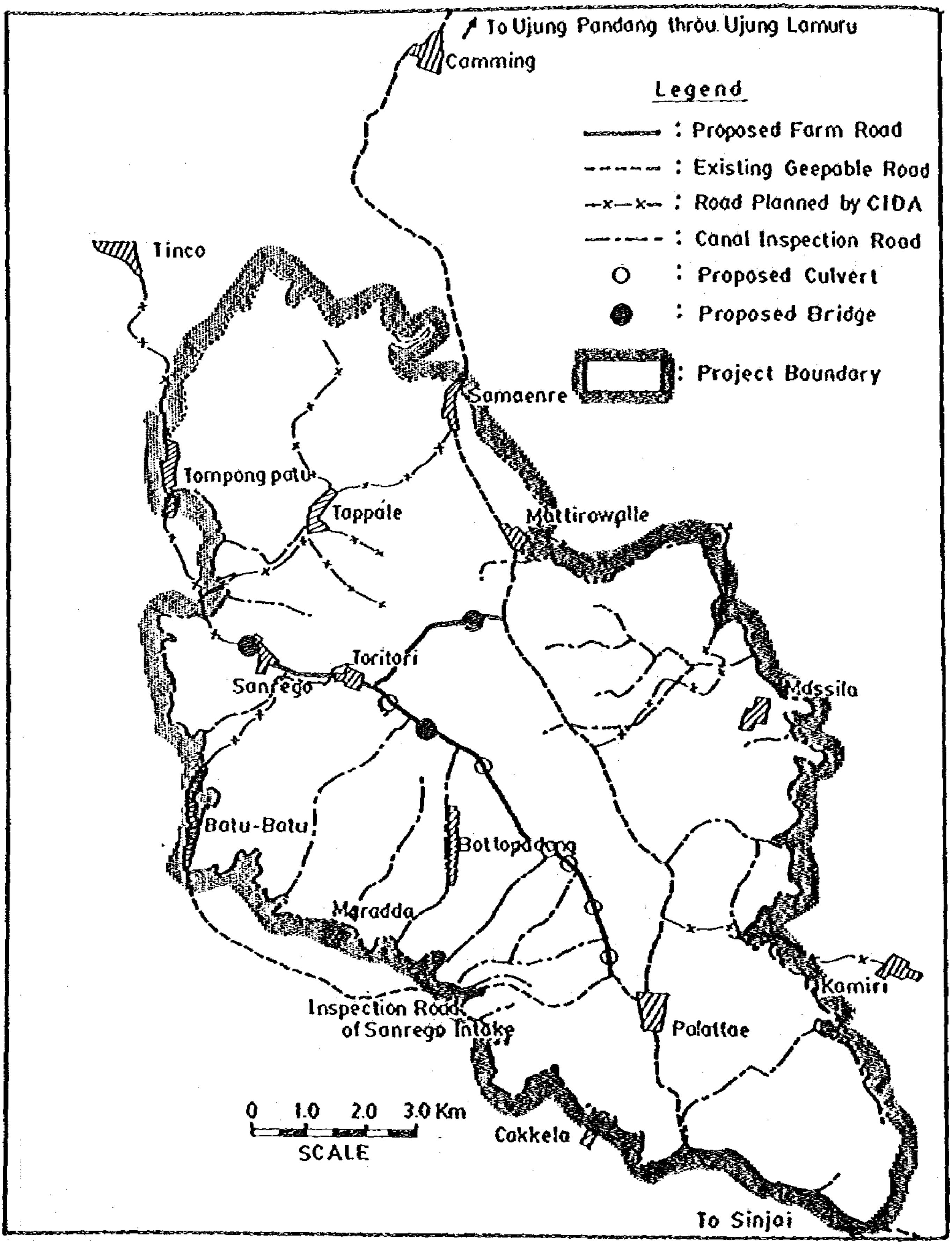


Fig. VI.7.1 FARM ROAD NETWORK

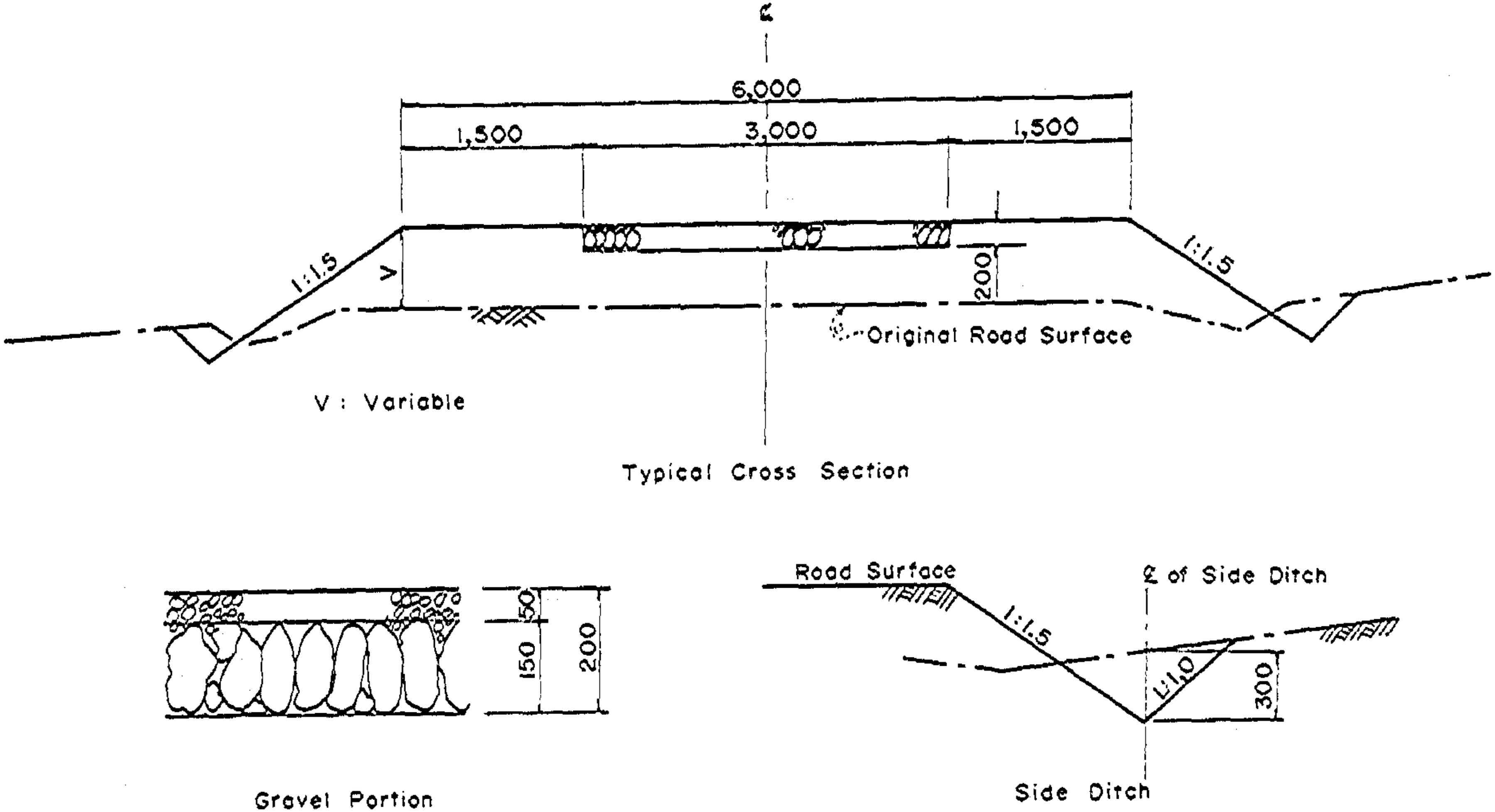


Fig. VI.7.2 TYPICAL CROSS SECTION OF PROPOSED FARM ROAD

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ANNEX - VII

PRELIMINARY DESIGN OF PROJECT FACILITIES

ANNEX - VII PRELIMINARY DESIGN OF PROJECT FACILITIES

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ANNEX - VII PRELIMINARY DESIGN OF PROJECT FACILITIES

1. GENERAL

The major feature of the Sanrego Irrigation Project is to supply irrigation water to the area of 8,000 ha from the Sanrego river and supplementaly from three tributaries, namely, Parota, Biru and Macinaga. The facilities required for the Project include Sanrego intake weir, supplemental small-scaled intake weirs on the three tributaries, irrgation canals and related structures and farm roads.

The Directorate of Irrigation (DOI), Directorate General of Water Resources Development, has been making an endeavour to realize the irrigation development of the Sanrego area since 1972. The detail design of the Sanrego intake weir and main and secondary irrigation system was almost completed in July, 1982 by DOI based on the field investigations on topography, geology and soil mechanics. The preliminary design of the project facilities in this study is, therefore, made mainly based on the review works on the DOI design so as to make the best use of the design results conducted by DOI since the repreparation of the different plan will delay the implementation of the Project and will spend much time and money. The design of new supplemental facilities is carried out so as to incorporate into the irrigation system designed by DOI.

2. SANREGO INTAKE WEIR

2.1 General

The Sanrego river is a major water source for the Project. The Sanrego intake weir is proposed to be constructed on the Sanrego river for introducing required quantity of irrigation water from the river to the Project area.

The detailed design of the intake weir was already completed together with the hydraulic model test by the Directorate of Irrigation (DOI) and the excavation works of coupure channel were commenced since 1980 by the local contractor. The tendering for the construction works of the intake weir including the closing dike was finished based on the local tender by the end of 1982, and the actual construction works are expected to be started at the beginning of 1983.

2.2 Main Features of Intake Weir Designed by DOI

(1) Location: About 14.5 km upstream from the confluence of the Walanae and the Sanrego rivers

(2) Catchment area: A = 157.8 km^2
(based on the map with a scale of 1:250,000)

(3) Design flood discharge: Qd = 814 m^3/sec
Calculation method - Merchior Method
Probably rainfall - R_{100} = 306 mm/day

(4) Design flood water level: EL. 174.847 m

(5) Scale of structure:

5-1 Fixed weir

- Type: Cascade type (double energy dissipator type)

- Upper crest elevation	:	EL. 170.747 m
- Upper crest length	:	40.0 m
- Elevation of upper stilling basin	:	EL. 160.447 m
- Length of upper stilling basin	:	17.5 m
- Lower crest elevation	:	EL. 164.447 m
- Lower crest length	:	47.0 m
- Elevation of lower stilling basin	:	EL. 151.947 m
- Length of lower stilling basin	:	16.8 m

5-2 Movable weir (scouring sluice)

- Bottom elevation	:	EL. 166.947 m
- Width	:	2.0 m x 2 nos.
- Gate	:	2.0 m x 3.8 m x 2 nos.

5-3 Coupure channel

- Width	:	47.0 m
- Length of upstream	:	180.0 m
- Bed elevation at upstream	:	EL. 166.947 m
- Length of downstream	:	250.0 m
- Bed elevation at downstream	:	EL. 162.447 m - EL. 154.947 m

5-4 Intake structure

- Intake water level	:	EL. 170.647 m
- Bed elevation	:	EL. 169.047 m
- Width	:	2.0 m x 3 nos.
- Gate	:	2.0 m x 1.7 m x 3 nos.

5-5 Closing dike

- Crest elevation	:	EL. 177.047 m
- Crest length	:	230.0 m
- Crest width	:	8.0 m
- Dam height	:	26.1 m

5-6 Temporary diversion channel

- Location : Center of coupure channel
- Diversion discharge: 297.0 m^3/sec (2 years return period)
- Bottom elevation : EL. 162.447
- Width : 2.0 m x 2 nos.

Fig. VII.2.1 shows the typical section of the Sanrego intake weir designed by DOI.

2.3 Review Works

2.3.1 Intake weir site

For the selection of the intake weir site, the following items should be considered.

(a) Relation between the height of weir crest and the length of main canal:

In order to attain the designed intake weater level, if the intake weir site is selected upstream, the height of weir crest is less but the length of main canal is more. Whereas, if the intake weir site is selected downstream, the crest height is more but the length of main canal is less.

(b) Topography and geology of the site:

(i) Narrow portion of river course will first be selected.

(ii) The site with stable rock foundation is preferable.

(iii) The river course at the site should be stable.

(c) Affection of backwater to upstream reaches:

Due to construction of weir, backwater will occur at the upstream of the weir and sometimes give damage to farmlands, houses, bridges and other structures along the river. This should carefully be examined.

(d) Affection of structure to sediment transport in the river:

Due to construction of weir, sediment transport in the river will be examined, which will cause the riverbed errosion in the downstream of the weir and may give damage to the existing structures in the downstream reaches of the river. Careful survey and study are required for this matter.

(e) Construction:

The site should ensure easy and cheap construction work.

(f) Operation and maintenance:

The site should provide good access for operation and maintenance.

The Sanrego intake weir site proposed by DOI is located at about 14.5 km upstream from the confluence of the Walanae and Sanrego rivers. This site is suitable for construction of intake weir in consideration of the following conditions:

(a) The site is situated near the proposed irrigation area and the intake water level is enough for supplying irrigation water to the area.

(b) The foundation of the site is composed of hard and stable rock.

(c) The Sanrego river flows in the narrow valley upstream of the proposed intake site. There exist few facilities or fields in the upstream reaches of the site. Therefore, no affection will not be occured by backwater due to construction of weir.

(d) The coupure method is adopted for the construction of the weir. This method is very familiar in Indonesia and the site is suitable for the application of this method.

2.3.2 Hydrological and hydraulic conditions

(1) Design intake water level

The design intake water level is decided based on the water level to be required for the irrigation of total project area. Since the highest elevation of the proposed irrigation area is around +169.0 m based on the topographic map with a scale of 1/5,000, the intake water level of +170.647 m designed by DOI is enough in consideration of hydraulic losses at canals and structures.

(2) Design flood discharge

The design flood discharge is estimated on the basis of the 100-year return period. In this review, based on the available data such as discharge records in the Sanrego river and rainfall data obtained from the adjacent area of the Sanrego river watershed, the design flood discharge is calculated by use of three empirical and statistical methods, i.e., Merchior method, rational formula and exponential distribution method. The details of calculation methods are mentioned in ANNEX-I and the maximum value of 820 m^3/sec among the above three methods is employed as the design flood discharge. This value is almost equal to the estimated one of 814 m^3/sec in the DOI design report.

(3) Design flood water level

Overflow depth at the weir is given by the following formula:

$$H = \left(\frac{Q}{B \times C}\right)^{2/3}$$

where,

H: overflow water head (m)

Q: flood discharge (m^3/s)

B: crest length (m)

C: coefficient of overflow (c = 2.1)

$$H = \left(\frac{820}{43.2 \times 2.1}\right)^{2/3} = 4.34 \text{ m}$$

Crest elevation is +170.747 m, then the design flood water level is as follows:

EL. 170.747 + 4.34 = EL. 175.087 m

2.3.3 Hydraulic study

The design of intake weir by DOI was made with the confirmations by the hydraulic model test which was carried out by use of the flood discharge of 700 m^3/sec, although the design flood discharge was estimated at 814 m^3/sec in the design report. In this review, the hydraulic study is made based on the design flood discharge of 820 m^3/sec as described in the previous paragraph.

(1) Stilling basin

The length of the stilling basin was determined by DOI design as shown below based on the hydraulic model test:

for upper stilling basin: L = 17.50 m

for lower stilling basin: L = 16.80 m

The hydraulic check for energy dissipating function of the stilling basin is made for the following flood discharges:

Q_{100} = 820 m^3/sec

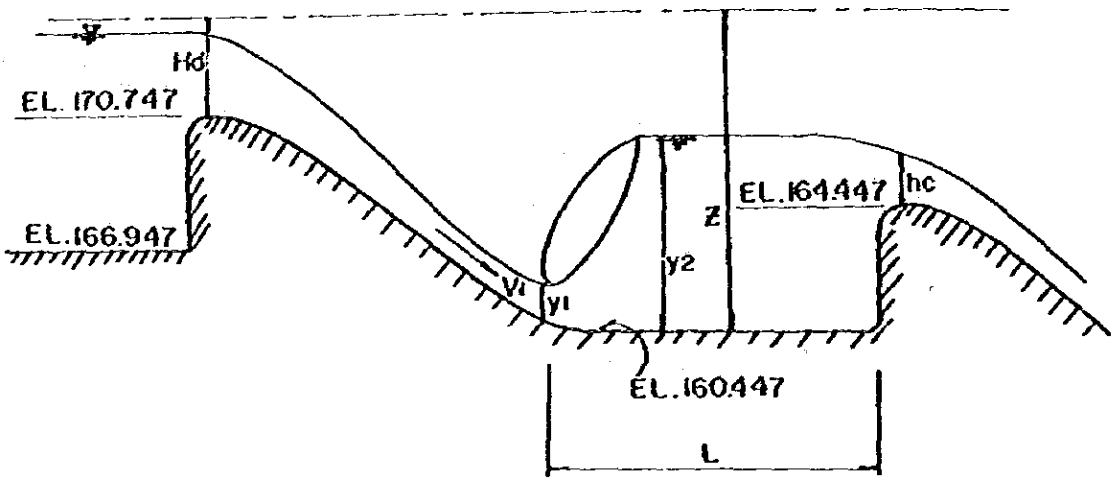
Q_{20} = 600 m^3/sec

Q_2 = 300 m^3/sec

Q_1 = 100 m^3/sec

(a) <u>Upper stilling basin</u>

The length of upper stilling basin is estimated by the following formulae.



$$Hd = \left(\frac{Q}{C \cdot B_1}\right)^{2/3}$$

$$V_1 = \sqrt{2g\ (Z - 0.5 \cdot Hd)}$$

$$y_1 = \frac{Q}{B_2 \cdot V_1}$$

$$Fr = \frac{V_1}{\sqrt{g \cdot y_1}}$$

$$y_2 = \frac{y_1}{2}\ (\sqrt{1 + 8Fr^2} - 1)$$

$$L = K \cdot y_2$$

where,

C: overflow coefficient, 2.1

B_1: crest length, 43.2 m

g: gravity acceleration, 9.8 m/sec^2

Z: EL.170.747 + Hd - EL.160.447 = Hd + 10.3

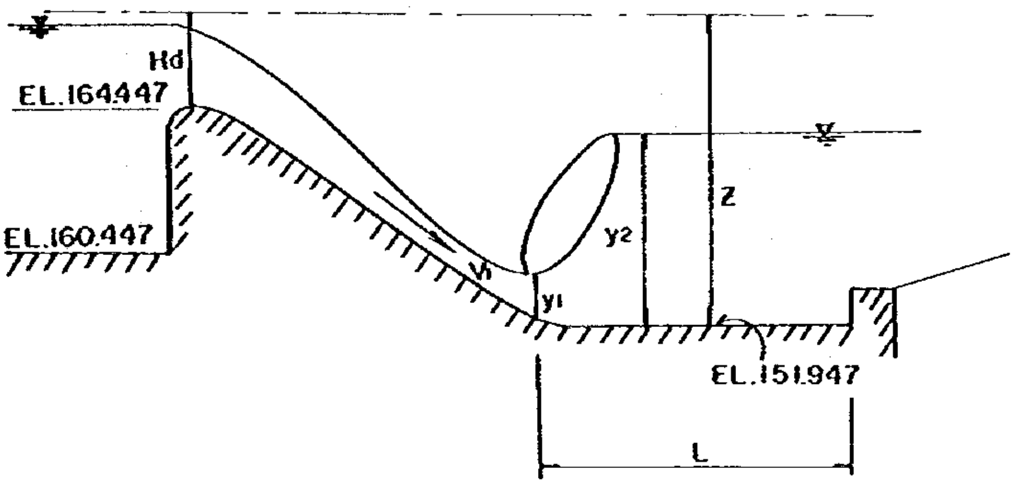
K: ratio of conjugate depth and length of stilling basin, from the hydraulic model test

$K = L/y_2 = 17.5/6.38 = 2.74$

Item	Flood Discharge (m^3/sec)			
	820	600	300	100
Hd (m)	4.34	3.52	2.22	1.07
Z (m)	14.64	13.82	12.52	11.37
V_1 (m/sec)	15.63	15.37	14.95	14.57
y_1 (m)	1.12	0.83	0.43	0.15
Fr	4.72	5.39	7.28	12.02
y_2 (m)	6.94	5.93	4.22	2.48
K	2.74	2.74	2.74	2.74
L (m)	19.10	16.30	11.60	6.80

From the above result, the length of upper stilling basin is required to be taken at 19.1 m for the design flood discharge of 820 m^3/sec. It is 1.6 m longer than the length designed by DOI.

(b) Lower stilling basin



$$Hd = \left(\frac{Q}{C \cdot B_1}\right)^{2/3}$$

$$V_1 = \sqrt{2g\ (Z - 0.5\ Hd)}$$

$$y_1 = \frac{Q}{B_2 \cdot V_1}$$

$$Fr = \frac{V_1}{\sqrt{g \cdot y_1}}$$

$$y_2 = \frac{y_1}{2}\left(\sqrt{1 + 8Fr^2} - 1\right)$$

$$L = K \cdot y_2$$

where,

- C: overflow coefficient, 2.1
- B_1: crest length, 47.0 m
- g: gravity acceleration, 9.8 m/sec^2
- Hd: EL.164.447 + Hd - EL.151.947 = Hd + 12.5
- B_2: width of stilling basin, 47.0 m
- K: ratio of conjugate depth and length of stilling basin, this ratio is obtained based on Froud number (Fr)

Item	Flood Discharge (m^3/sec)			
	820	600	300	100
Hd (m)	4.10	3.33	2.10	1.01
Z (m)	16.60	15.83	14.60	13.51
V_1 (m/sec)	16.89	16.66	16.30	15.97
y_1 (m)	1.03	0.77	0.39	0.13
Fr	5.32	6.06	8.34	14.15
y_2 (m)	7.25	6.23	4.41	2.54
K	2.45	2.60	2.70	2.80
L (m)	17.80	16.20	11.90	7.10

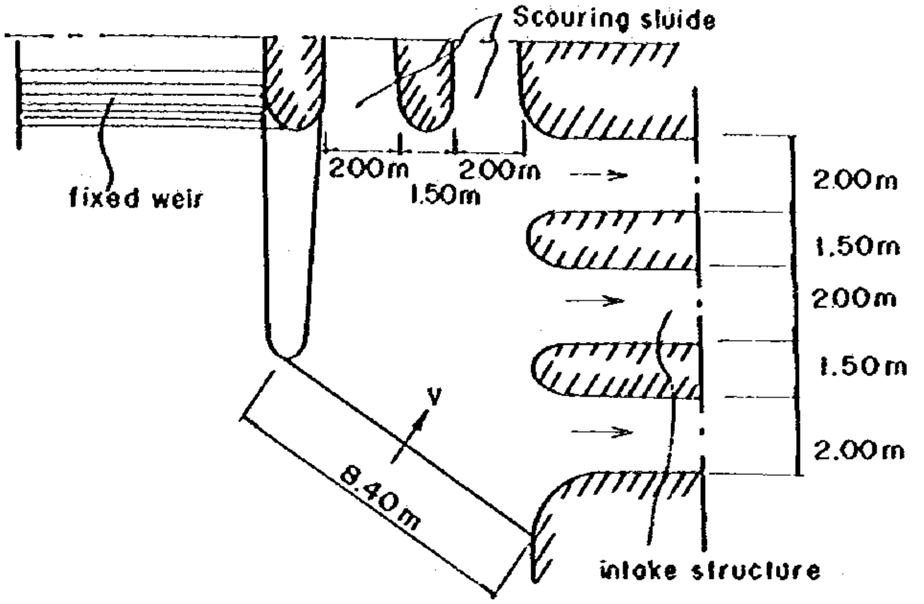
The length of lower stilling basin with the design flood discharge of 820 m^3/sec is estimated at 17.8 m as shown above. Therefore, it is required to be 1.0 m longer than the length designed by DOI.

(2) Scouring sluice

The function of the designed scouring sluice by DOI was checked by the hydraulic model test and the test result proved that the designed scouring sluice can fulfill its function at the normal water level of +170.747 m.

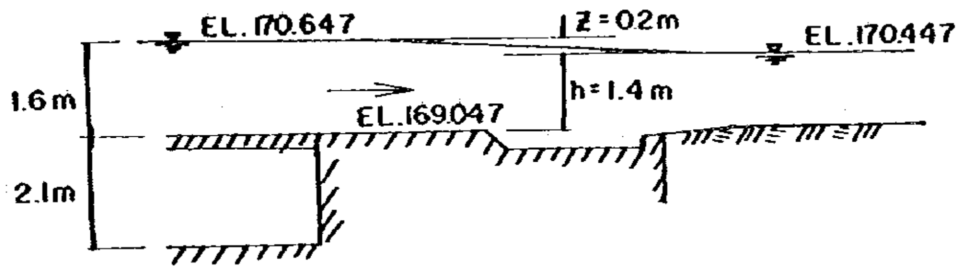
(3) Intake structure

In general, the inflow velocity at the intake structure is taken to be 0.6 - 1.0 m/sec to prevent sediment loads from entering into the irrigation canal. In the design report by DOI, the inflow velocity was estimated at 0.96 m/sec as shown below:



$$V = \frac{12.914}{8.4 \times 1.6} = 0.96 \text{ m/sec}$$

The width of the intake structure was decided as follows:



$$b = \frac{Q}{ch\sqrt{2gZ}} = \frac{12.914}{0.80 \times 1.4 \times \sqrt{2 \times 9.8 \times 0.2}} = 5.82 \text{ m} < 2.0 \times 3 = 6.0 \text{ m}$$

From the above, it is judged that the design of intake structure conducted by DOI is reasonable for the design discharge of 12.914 m^3/sec.

(4) Backwater due to construction of weir

In order to check the affection of backwater to upstream reaches of the Sanrego river due to construction of weir, calculation of backwater is made based on the design flood discharge with 100-year return period of 820 m^3/sec and the extraordinary flood discharge with 1,000-year return period of 1,136 m^3/sec. The calculation of backwater is made by using the Escoffier's graphical method and the calculation result is shown below (see Fig. VII.2.2):

(Unit: m)

Distance from Intake Weir	Water Surface Elevation	
	In Case of Q_{100} = 820 m^3/sec	In Case of Q_{1000} = 1,136 m^3/sec
300	EL. 175.1	EL. 175.4
600	175.1	175.4
900	175.3	175.8
1,200	175.5	176.0
1,500	176.5	177.7
1,800	179.1	180.7
2,100	180.7	181.5
2,400	182.8	184.4

In upstream reaches of the Sanrego river, a few houses exist at about 2.0 km upstream of the intake weir and these houses are situated on the high lands with the elevation of 200 m or more. Backwater due to construction of weir will, therefore, not give damage to upstream reaches of the Sanrego river.

2.3.4 Structural study

The foundation of the intake weir is composed of the alternation of sandstone and siltstone. The bearing capacity of the foundation is very high and so there is no problem for construction of intake weir. According to the stability analysis, the intake weir designed by DOI is safe for overturing and sliding.

2.3.5 Study of closing dike

(1) Crest elevation

The closing dike is a high dam, having the height of about 26.0 m. Therefore, the design flood discharge with 1,000-year return period is used for determination of crest elevation. It is estimated at 1,136 m^3/sec.

The crest elevation of the closing dike is determined as follows:

Crest elevation = Hd + hw + hi

where,

- Hd: flood water level
  EL.175.437 m at Q_{1000} = 1,136 m^3/sec
- hw: height of wave due to wind
- hi: height of allowance according to dam type, in case of fill dam - 1.0 m

"hw" is calculated by using the following SMB method:

$$hw: \ 0.00086\ V^{1.1}F^{0.45}$$

where, V: wind velocity, 30 m/sec

F: fetch, 500 m

$$\therefore hw = 0.00086 \times 30^{1.1} \times 500^{0.45} = 0.59 \text{ m}$$

Therefore, the crest elevation is estimated at

EL.175.437 + 0.59 + 1.00 = EL.177.027 m

The designed crest elevation by DOI is EL.177.047 m, which shows the sufficient value for the flood discharge of Q_{1000} = 1,136 m^3/sec.

(2) Cross section

The stability analysis of closing dike is made based on the soil mechanical investigation for embankment materials as described in ANNEX-III. From the result of stability analysis, the downstream section of closing dike designed by DOI is required to be modified as shown in Fig. VII.2.3.

(3) Leakage and piping

Although the foundation of closing dike is composed of hard and stable rocks, alternating sandstone and siltstone, many cracks are found along the laminations of these rocks. From the result of geological investigation, the permeability of foundation is rather high, showing the order of 10^{-3} cm/sec.

In order to check the leakage from the foundation of closing dike, the following calculation is made:

$$Q = q \cdot B$$

$$q = k \cdot K' \ (h_1 - h_2)/2K$$

$$k^k = \tanh(\pi C/2H)$$

where,

Q: leakage from foundation (m^3/sec)

q: unit leakage per meter (m^3/sec/m)

B: width of foundation, 300 m

k: coefficient of permeability, 5 x 10^{-5}m/sec

h_1: water depth at upstream, EL.170.747 - EL.152.947 = 17.8 m

h_2: water depth at downstream, 0 m

K',K: value obtained from k^k

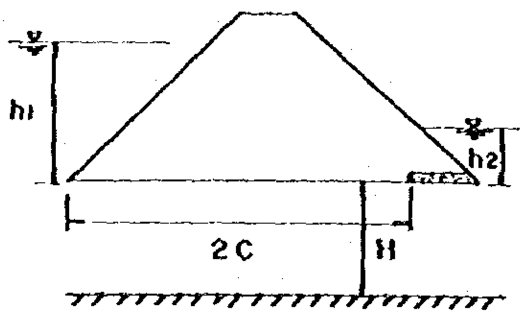
C: half of bottom width of closing dike, 85.0/2 = 42.5 m

H: depth of permeable foundation, 8.0 m

k^k = tanh (3.14 x 42.5/2 x 8)
= 1.0

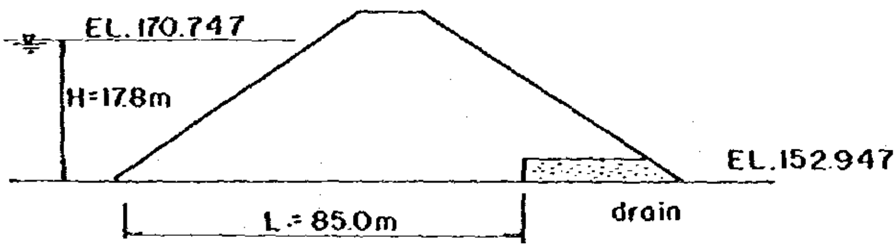
$q = 5 \times 10^{-5} \times 38 \times 17.8/2 \times 38$
$= 4.5 \times 10^{-4}$ m^3/sec/m

$Q = 4.6 \times 10^{-4} \times 300$
$= 0.14$ m^3/sec



The leakage estimated above is negligible small for discharge of the Sanrego river.

For piping at the bottom of closing dike contacting with the foundation, the following check is made:



Weighted-creep ratio is calculated as follows:

C = L/H = 85.0/17.8 = 4.78 > 2.0

The embankment materials of closing dike consist of medium and hard clays. In general, such materials are considered to be safe for piping in case that weighted-creep ratio is more than 2.0. Therefore, the above calculation result shows that there is no problem for piping in the design of closing dike. However, it is recommended that the special attention should be paid to compact carefully for embanking at the construction stage.

(4) Left side wall contacting with closing dike

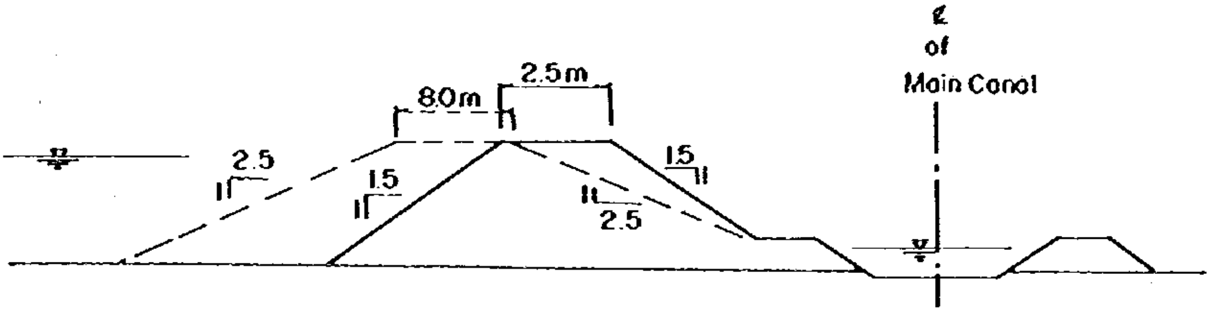
In the DOI design, the left side wall of intake weir was designed by changing the cross-section of closing dike at the contact place.

The height of left side wall is proposed to be more higher so as not to change the corss section of closing dike as shown in Figs. VII.2.4 and VII.2.5. In addition, the length of left side wall is also proposed to be extended toward upstream as shown in Figs. VII.2.4 and VII.2.5.

Concerning the cross section of left bank wall, the back slope of the said wall is recommended to be more than 1:0.3 to prevent piping at between wall and embankment of closing dike, and the fillet designed by DOI is proposed to be taken off to avoid unequal settlement of closing dike as much as possible, as shown in Fig. VII.2.6.

(5) Embankment at right bank of intake weir

In the DOI design, the cross-section of embankment at right bank of intake weir was designed with both upstream and downstream slopes of 1:1.5. Taking into account the water level at upstream, these slopes are proposed to be determined at 1:2.5 to keep stability of embankment as shown below.



——— : DOI design
– – – – : Proposed plan

2.4 Recommendation

Based on the review works for the intake weir designed by DOI, recommendations are summarized as follows:

(1) The length of stilling basin is recommended to be taken as shown below.

- for upper stilling basin: L = 19.1 m (1.6 longer than the design length)
- for lower stilling basin: L = 17.8 m (1.0 longer than the design length)

(2) The downstream section of closing dike is recommended to be modified as shown in Fig. VII.2.3, based on the stability analysis.

(3) The height and length of left side wall of intake weir are recommended to be more higher and longer as shown in Figs. VII.2.4 and VII.2.5, so as not to change the cross-section of closing dike.

(4) The back slope of left bank wall contacting the closing dike is recommended to be more than 1:0.3 to prevent piping at between wall and embankment of closing dike, and the fillet provided at the bottom of wall is recommended to be taken off to avoid unequal settlement of embankment, as shown in Fig. VII.2.6.

(5) Upstream and downstream slopes of embankment at the right bank of intake weir are recommended to be modified from 1:1.5 to 1:2.5 for keeping the stability of embankment.

3. SUPPLEMENTAL THREE INTAKE WEIRS ON THE TRIBUTARIES

3.1 General

There are many small tributaries in the Project area and among of them, three (3) tributaries, namely, Parota, Biru and Macinaga, are comparatively large. For the maximum effective use of water resources, these tributaries are selected as the supplemental water sources for the Project. On the Biru river, there exists an intake weir which was constructed in 1934 to supply irrigation water to the Maradda area of 250 ha. This existing weir can be used with some rehabilitation works as one of the Project facilities. On another two tributaries, Parota and Macinaga, no facilities can be found and so, new small-scaled intake weirs are proposed to be constructed to get irrigation water. The design of these intake weirs is made so as to incorporate into the irrigation canal system designed by DOI.

3.2 Parota Intake Weir

3.2.1 Intake weir site

The Parota river is one of the tributaries of the Sanrego river and its watershed is located on the northwest of the Sanrego river watershed. The catchment area is estimated at 32.0 km^2 at the junction with the Sanrego river. The Parota intake weir site is selected at about 2.0 km upstream from the junction with the Sanrego river in taking consideration of diverting water level into the Parota secondary canal in the irrigation canal system designed by DOI.

The river channel at the proposed intake weir site is about 15 m wide and the gradient of the riverbed is estimated at 1 to 400. The geological condition at the site is favourable for construction of the intake weir, composing of hard and stable rocks.

3.2.2 Type of weir

The discharge in the Parota river sharply fluctuates in the wide ranges and flood discharge at the proposed intake weir site is comparatively large. Taking into account the above hydrological condition, simplicity of structure and easiness of operation and maintenance, the tirol type weir is proposed to be constructed on the hard bed rock.

3.2.3 Preliminary design

The discharge to be taken by the tirol type weir is estimated by the Mostkow's formula which is described as follows:

$$Q_I = L.U.B.M\sqrt{2gHo}$$

where,

Q_I: intake discharge (m^3/sec)

L: length of intake screen (m)

U: coefficient of intake flow

B: width of intake weir (m)

M: $\Sigma S/B$

S: opening of two screen bars (m)

g: acceleration of gravity (m/sec^2)

Ho: $\left(\frac{Q_I}{C \cdot B}\right)^{2/3}$

C: overflow coefficient

In the above formula,

Q_I = 880 ha x 1.6 ℓ/sec/ha = 1.40 m^3/sec

$U = 0.6$

$B = 10.0$ m

$M = 0.25$

$$Ho = \left(\frac{1.40}{1.7 \times 10.0}\right)^{2/3} = 0.19 \text{ m}$$

Therefore,

$$L = \frac{Q_I}{U.B.M\sqrt{2gHo}}$$

$$= 0.48 \text{ m}$$

The typical section of the weir is decided as shown on Fig. VII.3.1 in view of stability and installation of intake screen.

No apron is proposed to be equipped at the downstream of the weir because the hard rock is exposed at the foundation of the weir.

3.3 Biru Intake Weir

3.3.1 Intake weir site

Biru river is one of the tributaries of the Walanae river and flows from south to north in the Project area. On the Biru river, there exists an intake weir constructed as the main facility of the Maradda semi-technical irrigation system. The existing intake weir is located at about 11.5 km upstream from the junction with the Walanae river. The catchment area at the site is measured to be 20.3 km^2.

The river channel at the existing intake weir site is about 30 m wide and the gradient of the river bed is about 1 to 300. The hard and stable rocks are found at the foundation of the weir site.

3.3.2 Type of weir

The existing intake weir consists of overflow type weir with a scouring sluice. Main features of the existing weir are as follows:

(1) Fixed weir

- crest elevation : EL.170.30 m
- crest length : 27.5 m
- height of weir : max. 3.0 m

(2) Scouring sluice

- width : 1.0 m x 1 no.
- bottom elevation: EL.168.30 m
- gate : stop log

(3) intake

- width : 1.0 m x 1 no.
- bottom elevation: EL.168.80 m
- gate : stop log

3.3.3 Preliminary design

The crest elevation of the existing intake weir is enough for diverting water into the Sanrego main canal in the irrigation canal system designed by DOI. The design intake discharge is estimated at 580 ha x 1.6 ℓ/sec/ha = 0.93 m^3/sec and for this design discharge, the existing intake structure has sufficient capacity. Therefore, the existing intake weir is proposed to be used as the one of the project facilities with minor rehabilitation works. The typical section of the existing intake weir is shown in Fig. VII.3.2.

3.4 Macinaga Intake Weir

3.4.1 Intake weir site

The Macinaga river is upper reaches of the Baruttung river which is one of the tributaries of the Walanae river. The Macinaga intake weir is proposed to be newly constructed at about 7.5 km upstream from the junction with the Walanae river so as to incorporate into the Aming secondary canal in the irrigation canal system designed by DOI.

At the proposed intake weir site, the catchment area is measured at 8.7 km^2. The width of river channel is about 7.0 m and the gradient of the river bed is about 1 to 300 at the site. The geological condition at the site is favourable for construction of the intake weir since the exposed rocks are hard and stable.

3.4.2 Type of weir

In taking consideration of design intake discharge, hydraulic and topographic conditions of the river, the tirol type weir is proposed to be applied to the Macinaga intake weir. The full span of 8 m wide would be constructed on the hard bed rock.

3.4.3 Preliminary design

The length of intake screen of the tirol type weir is determined as follows:

$$L = \frac{Q_I}{U.B.M\,\sqrt{2gHo}}$$

where,

L: length of intake screen (m)

Q_I: design intake discharge
250 ha x 1.6 ℓ/sec/ha = 0.40 m^3/sec

U: coefficient of intake flow, 0.6

B: width of intake weir, 5.0 m

C: $\Sigma S/B = 0.25$

g: acceleration of gravity, 9.8 m/sec^2

Ho: $$\left(\frac{Q_I}{C \cdot B}\right)^{2/3} = \left(\frac{0.40}{1.7 \times 5.0}\right)^{2/3} = 0.13 \text{ m}$$

Therefore,

$$L = \frac{0.40}{0.6 \times 5.0 \times 0.25 \times \sqrt{19.6 \times 0.13}} = 0.34 \text{ m}$$

The typical section of the weir is decided in view of stability and installation of intake screen, and is illustrated on Fig. VII.3.3.

4. IRRIGATION CANAL SYSTEM

4.1 General

The Directorate of Irrigation (DOI) planned the irrigation canal system for the irrigation area of 8,071 ha by utilizing the available water from the Sanrego river and almost completed the design works of main and secondary irrigation canals and related structures. In this study, some supplemental water sources are required to supply irrigation water to the irrigation area proposed by DOI based on the water balance study described in ANNEX-VI. Three tributaries, Parota, Biru and Macinaga, are selected as the supplemental water sources for the Project and new facilities, such as small-scaled intake weirs and connecting canals, are proposed to be constructed to incorporate into the irrigation canal system designed by DOI. The study of irrigation canal system is, therefore, to review the DOI design and to make preliminary design of connecting canals.

4.2 Main Features of Irrigation Canals and Related Structures Designed by DOI

4.2.1 Irrigation canals

Irrigation canal system designed by DOI consists of one main canal, 18 secondary canals and tertiary systems. The lengths and design discharges of main and secondary canals are shown below:

Name of Canal	Canal Length (m)	Covering Area (ha)	Maximum Design Discharge (m^3/sec)
1. Sanrego Main Canal	11,582	8,071	12.914
2. Secondary Canal			
(1) Palaka S.C	5,949	2,193	3.509
(2) Parota S.C	8,720	1,613	2.581
(3) Labosi S.C	2,227	320	0.566
(4) Tapale S.C	1,216	171	0.328
(5) Apale S.C	4,165	576	0.945
(6) Pao S.C	2,345	310	0.550
(7) Maradda S.C	1,855	235	0.432
(8) Batu Taneuh S.C	3,783	365	0.631
(9) Jaramele S.C	1,334	191	0.359
(10) Candranae S.C	1,628	212	0.394
(11) Barang S.C	1,476	121	0.252
(12) Aming S.C	32,864	3,872	6.195
(13) Lume S.C	1,999	125	0.258
(14) Masago S.C	1,844	311	0.552
(15) Paccing S.C	5,168	519	0.858
(16) Hulo S.C	2,268	133	0.270
(17) Same Enre S.C	6,277	693	1.110
(18) Poparapa S.C	1,224	117	1.246
Total	97,924	-	-

All canals were designed to be unlined earth canals with trapezoidal sections and canal dimensions are shown in Table VII.4.1

4.2.2 Related structures

A number of canal structures of various type are generally required in conjunction with the irrigation canal. In the DOI design, the following related structures were proposed in irrigation canal system:

(1)	Turnout	88 nos.
(2)	Drop	64 nos.
(3)	Chute	7 nos.
(4)	Aqueduct	3 nos.
(5)	Bridge	25 nos.
(6)	Cross drain culvert	79 nos.

Table VII.4.2 shows the list of canal structures related to main and secondary canals designed by DOI and Table VII.4.3 shows detailed features of these structures.

4.3 Review Works

4.3.1 Irrigation canals

(1) 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 in the canal. In the DOI design, the maximum and minimum velocities were determined to be 0.70 m/sec and 0.30 m/sec, respectively. Considering the characteristics of soil materials and the conditions of aquatic vegetation, these designed values are judged to be reasonable.

(2) Roughness coefficient

The following roughness coefficients of the canals were used for determination of canal hydraulic properties in the DOI design:

Design Discharge: Q (m^3/s)	Strickler's K	Manning's n
$Q \leq 4.5$	45	0.022
$4.5 < Q \leq 10.0$	47.5	0.021
$Q > 10.0$	50.0	0.020

These are appropriate values for the design of earth canals.

(3) Freeboard

The freeboard height is normally subject to canal size and location, velocity, water surface fluctuations and availability of materials for embankment. In the DOI design, the minimum freeboard for the respective canal discharge were determined as follows:

Design Discharge: Q (m^3/sec)	Minimum Freeboard (m)
$Q \leq 1.5$	0.5
$1.5 < Q \leq 3.0$	0.6
$3.0 < Q \leq 6.0$	0.7
$6.0 < Q \leq 8.5$	0.8
$Q > 8.5$	1.0

The above minimum freeboard is judged to be reasonable because it is almost same as the design values used in another similar projects in Indonesia.

(4) Bottom width - water depth ratio (B/h)

B/h ratio in the DOI design was determined based on the 1980 standard of IRIGASI as shown below:

Design Discharge: Q (m^3/sec)	B/h
$Q \leq 0.3$	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

(5) Inside slope

The inside slope of the canal depends upon the characteristics of materials to be used for canal construction and also on height of canal embankment. The inside slope under the normal condition based on the 1980 standard of IRIGASI is given as follows:

Design Discharge: Q (m^3/sec)	Inside Slope
$Q \leq 1.5$	1 : 1.0
$1.5 < Q \leq 15.0$	1 : 1.5

The DOI design was made based on the above standard.

(6) Bank width

In the DOI design, banks of main and secondary canals were designed so as to be used as inspection roads. The bank width was determined to be 6.0 m with the gravel metalled width of 3.0 m and shoulders at both sides, each having a width of 1.5 m. This inspection road will be very useful for maintenance and operation of canals and also for agricultural activities in the area.

(7) Lining of canal

In the DOI design, all canals were proposed to be unlined. Based on the soil mechanical and geological investigations along the proposed canal routes in this study, for the following portions, the canal sections are recommended to be lined to prevent seepage and erosion:

Name of Canal	Location	Length to be lined (km)
Main canal	B.S.1 - B.S.2c	0.9
Palaka secondary canal	B.S.1 - B.Pa.2	1.7
Aming secondary canal	B.Am.12 - B.Am.14	1.3
Aming secondary canal	B.Am.28 - B.SE.1	3.5
Total		7.4

At the portions to be lined in main canal and Palaka secondary canal, the geological condition is constituted by alternation of silt-stone and sandstone. These rocks are comparatively hard but many cracks are found. Therefore, to prevent seepage, both slopes of the canal are required to be lined with mortar.

At the portions to be lined in Aming secondary canal, the geological condition is mainly constituted by siltstone. This siltstone is highly weathered and so, it is likely to be considerably weak and much permeable. The canal sections of these portions are, therefore, proposed to be lined with wet stone masonry to prevent seepage and erosion.

4.3.2 Related structures

Various structures related to the main and secondary irrigation canals were proposed to be provided for crossing of road and river, regulating and control of discharge and distribution of irrigation water in the DOI design as mentioned in the previous section 4.2.2. These structures except the cross drain culvert were designed with sufficient capacities hydraulically and structurally based on the design standard used in other similar projects in Indonesia.

As regards the cross drain culvert, the water flowing capacity of conduit was determined on the basis of the design flood discharge with 20-year return period, and the design discharge was estimated by using the Weduwen method. The drainage area at each structure site was measured on the map with a scale of 1:250,000.

In order to review the capacity of drainage culvert designed by DOI, the design discharge of that is checked. The drainage area at the structure site is measured on the topographic map with a scale of 1:25,000 prepared by JICA, 1978. It is revealed in the review works that the drainage area obtained from the map prepared by JICA is comparatively larger than that used in the DOI design, and that the area is highly occupied by the paddy field.

According to the land use condition in the drainage area, the capacity of a drain culvert designed by DOI, which has a drainage area of less than 1 km^2, is distinctly enough to drain the flood water from the drainage area. Therefore, the capacity of the drain culvert with a drainage area of more than 1 km^2 is checked in this review work.

The design discharge of the culvert with 20-year return period is estimated by using following formula:

$$Qd = f(Qp + Qo)$$

$$Qp = F(\frac{C1 . R_4 . A}{T} \times 10)$$

$$Qo = 9.15 \times 10^{-3} \cdot C_2 \cdot i \cdot S^{1/5} \cdot A^{4/5} \text{ (McMatch Formula)}$$

where,

Qd: design discharge (m^3/sec)

Qp: drainage water from paddy field (m^3/sec)

Qo: drainage water from non-paddy field (m^3/sec)

f: allowance factor; 1.15

F: peak factor; 1.25

C_1: runoff coefficient; 0.8

R_4: 4-day consecutive rainfall with 20-year return period (mm); 380

A: drainage area (ha)

T: drainage period of paddy field (sec); 345,600

C_2: coefficient representing the watershed characteristics; 0.63

i: rainfall intensity for the time of concentration (mm/hr) by using rational formula

S: average ground slope

According to the result of the above estimation, the capacity of three (3) drain culverts, out of 79 culverts designed by DOI are proposed to be enlarged in order to drain out the design flood discharge safely.

The proposed design discharges and sizes of these three conduit sections are shown as below:

Name of Drain Culvert	DOI Design		Proposed Design	
	Design Discharge (m^3/sec)	Size of Section (m)	Design Discharge (m^3/sec)	Size of Section (m)
BS 8a	4.81	2.0 x 1.9	7.40	3.0 x 4.0
BAm3c	27.42	6.0 x 5.6	40.40	3.0x2.5x 3 barrels
BAm26a	19.21	2.5x2.4x 2 barrels	39.30	3.0x2.5x 3 barrels

4.3.3 Recommendation

Through the review works, it is judged that the design of main and secondary irrigation canals and related structures conducted by DOI was made with the adequacy and soundness on the basis of the design standard used in other similar irrigation projects in Indonesia. Therefore, the recommendation is given only for the followings:

(1) To prevent seepage and erosion, the canal sections are recommended to be lined with mortar and wet stone masonry for the following portions:

- for Main canal : 0.9 km (B.S.1 - B.S.2c)
- for Palaka secondary canal: 1.7 km (B.S.1 - B.Pa.2)
- for Aming secondary canal : 4.8 km (B.Am.12 - B.Am.14 & A.Am.28 - B.SE.1)

(2) Out of 79 cross drain culverts designed by DOI, three (3) culverts are recommended to be enlarged their capacities so as to safely drain the design flood discharge with the return period of 20-years as shown in section 4.3.2.

4.4 Preliminary Design of Connecting Canals from Supplemental Three Intake Weirs on the Tributaries

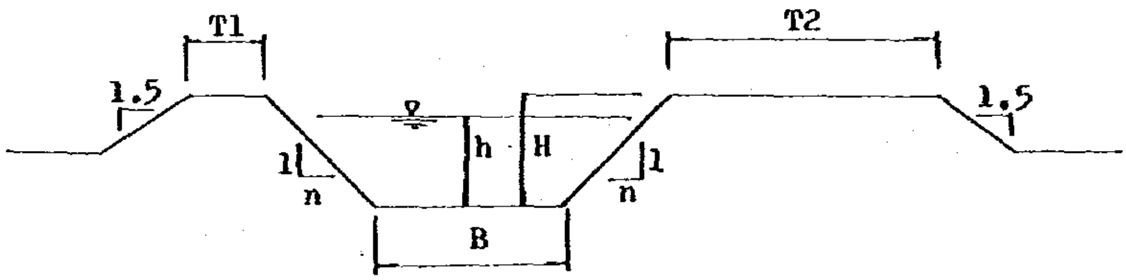
4.4.1 General

The small-scaled intake weirs are proposed to be constructed on three tributaries, Parota, Biru and Macinaga, to supply irrigation water supplementarly for the Project area of 8,000 ha as mentioned in the previous section 3. The connecting canals are also proposed to be constructed to convey intake water from weirs to the main and secondary irrigation canals designed by DOI. The design of these connecting canals is carefully made so as to incorporate into the irrigation canal system proposed by DOI.

4.4.2 Canal section

All connecting canals are proposed to be unlined earth canals with trapezoidal sections. The canal section is designed based on the design standard used in the DOI design as described in the previous section 4.3.1.

The canal sections of three connecting canals are determined as follows:



Name of Canal	Canal Length (km)	Design Discharge (m^3/sec)	Canal Dimension B (m)	h (m)	H (m)	n	T_1 (m)	T_2 (m)
Parota	1.00	1.40	1.80	0.90	1.40	1.0	1.50	4.00
Biru	1.40	0.93	1.50	0.75	1.25	1.0	1.50	4.00
Macinaga	2.50	0.40	0.95	0.63	1.13	1.0	1.50	4.00

The canal bank (T_2) is proposed to be used as inspection road with gravel metalled width of 3.0 m. At where the existing roads run in parallel with the proposed canal route, the inspection roads are not planned on the bank and the width of bank will be the same of that berm (T_1).

4.4.3 Related structures

Relating to connecting canals, the following structures are proposed to be constructed:

(Unit: nos.)

Name of Canal	Name of Structure			
	Spillway	Bridge	Cross Drain Culvert	Junction
Parota	1	-	1	1
Biru	-	-	-	1
Macinaga	1	1	2	1

Junction structures are provided to protect the main and secondary canals at the connecting points with connecting canals.

Table VII.4.1 List of Main and Secondary Irrigation Canals Designed by DOI (1/3)

Name of Canal	Design Discharge (m^3/sec)	Canal Length (m)	Canal Dimension/1 B (m)	h (m)	H (m)	n	T (m)	L_1 (m)	L_2 (m)
I. Main Canal									
Section 1	12.914	50	9.40	1.570	2.570	1.5	17.11		
" 2	9.296	2,096	7.15	1.430	2.430	1.5	14.44	3.0	6.0
" 3	9.163	2,237	7.10	1.420	2.420	1.5	14.36	3.0	6.0
" 4	8.558	850	6.40	1.488	2.422	1.5	13.67	3.0	6.0
" 5	8.539	795	6.40	1.422	2.422	1.5	13.67	3.0	6.0
" 6	8.035	1,251	6.20	1.378	2.178	1.5	12.73	3.0	6.0
" 7	7.995	424	6.20	1.378	2.178	1.5	12.73	3.0	6.0
" 8	7.315	1.334	5.50	1.375	2.175	1.5	12.02	3.0	6.0
" 9	6.918	1.077	5.40	1.350	2.150	1.5	11.85	3.0	6.0
" 10	6.465	1.438	5.20	1.300	2.100	1.5	11.50	3.0	6.0
Total		11,582							
II. Secondary Canal									
2-1 Palaka S.C.									
Section 1	3.569	914	3.40	1.133	1.833	1.5	8.90	2.0	6.0
" 2	3.485	779	3.49	1.133	1.833	1.5	8.90	1.5	6.0
" 3	3.342	1,124	3.35	1.117	1.817	1.5	8.80	1.5	6.0
" 4	0.734	783	1.30	0.682	1.282	1.0	3.96	1.5	6.0
" 5	0.522	757	1.20	0.585	1.185	1.0	3.57	1.5	6.0
" 6	0.331	1,592	0.95	0.622	1.222	1.0	3.39	1.5	6.0
Sub-total		5,949							
2-2 Parota S.C.									
Section 1	2.581	882	2.70	1.080	1.680	1.5	7.74	1.5	6.0
" 2	2.510	744	2.65	1.060	1.660	1.5	7.63	1.5	6.0
" 3	2.410	1,492	2.60	1.040	1.640	1.5	7.52	1.5	6.0
" 4	2.381	1,595	2.60	1.040	1.640	1.5	7.52	1.5	6.0
" 5	2.302	717	2.55	1.020	1.620	1.5	7.41	2.0	6.0
" 6	1.630	762	2.15	0.860	1.460	1.5	6.53	2.0	6.0
" 7	1.293	1,378	1.65	0.925	1.425	1.0	4.70	2.0	6.0
" 8	1.228	750	1.80	0.900	1.400	1.0	4.60	1.5	6.0
Sub-total		8,720							
2-3 Labosi S.C.									
Section 1	0.566	1,044	1.25	0.625	1.325	1.0	3.50	1.5	6.0
" 2	0.409	1,183	0.95	0.633	1.133	1.0	3.22	1.5	6.0
Sub-total		2,227							
2-4 Tapale S.C.									
Section 1	0.328	1,216	0.90	0.600	1.100	1.0	3.10	1.5	6.0
2-5 Apale S.C.									
Section 1	0.945	950	1.50	0.775	1.275	1.0	4.05	1.5	6.0
" 2	0.795	1,125	1.40	0.700	1.200	1.0	3.80	1.5	6.0
" 3	0.672	600	1.35	0.675	1.175	1.0	3.70	1.5	6.0
" 4	0.530	690	1.20	0.600	1.100	1.0	3.40	1.5	6.0
" 5	0.409	800	0.95	0.633	1.133	1.0	3.22	1.5	6.0
Sub-total		4,165							
2-6 Pao S.C.									
Section 1	0.550	1,438	1.25	0.625	1.125	1.0	3.50	1.0	6.0
" 2	0.332	907	0.90	0.600	1.100	1.0	3.10	1.0	6.0
Sub-total		2,345							
2-7 Maradda S.C.									
Section 1	0.432	848	0.95	0.637	1.137	1.0	3.22	1.5	6.0
" 2	0.199	1,007	0.55	0.541	1.041	1.0	2.63	1.5	6.0
Sub-total		1,855							

Table VII.4.1 List of Main and Secondary Irrigation Canals Designed by DOI (2/3)

Name of Canal	Design Discharge (m^3/sec)	Canal Length (m)	Canal Dimension/1 B (m)	h (m)	H (m)	n	T (m)	L_1 (m)	L_2 (m)
2-8 Batu Tanueh S.C.									
Section 1	0.631	1,157	1.30	0.650	1.150	1.0	3.60	1.5	1.5
" 2	0.440	2,160	0.95	0.633	1.133	1.0	3.22	1.5	1.5
" 3	0.379	466	0.95	0.633	1.133	1.0	3.22	1.5	1.5
Sub-total		3,783							
2-9 Jaramele S.C.									
Section 1	0.359	1,334	0.90	0.600	1.100	1.0	3.10	1.5	6.0
2-10 Cendranae S.C.									
Section 1	0.394	698	0.95	0.633	1.133	1.0	3.22	1.5	6.0
" 2	0.287	930	0.65	0.650	1.150	1.0	2.95	1.5	6.0
Sub-total		1,628							
2-11 Barang S.C.									
Section 1	0.252	1,476	0.60	0.609	1.109	1.0	2.82	1.5	6.0
2-12 Aming S.C.									
Section 1	6.195	880	5.10	1.275	1.975	1.5	11.03	2.5	6.0
" 2	6.150	776	5.05	1.263	1.963	1.5	10.94	2.5	6.0
" 3	5.861	856	4.70	1.343	2.043	1.5	10.83	2.5	6.0
" 4	5.769	1,373	4.50	1.286	1.986	1.5	10.46	2.5	6.0
" 5	5.714	1,181	4.60	1.314	2.014	1.5	10.64	2.5	6.0
" 6	5.627	550	4.60	1.314	2.014	1.5	10.64	2.5	6.0
" 7	5.510	697	4.40	1.257	1.957	1.5	10.27	2.5	6.0
" 8	5.426	1,002	4.50	1.286	1.986	1.5	10.46	2.5	6.0
" 9	5.053	697	4.40	1.257	1.957	1.5	10.27	2.5	6.0
" 10	5.070	1,206	4.35	1.243	1.943	1.5	10.18	2.5	6.0
" 11	4.805	955	4.25	1.214	1.914	1.5	9.99	2.5	2.5
" 12	4.573	1,372	4.15	1.186	1.886	1.5	9.81	2.5	2.5
" 13	4.066	682	3.70	1.233	1.933	1.5	9.50	2.5	6.0
" 14	3.949	627	3.60	1.200	1.900	1.5	9.30	2.5	6.0
" 15	3.723	1,041	3.50	1.167	1.867	1.5	9.10	2.5	6.0
" 16	3.704	1,233	3.50	1.167	1.867	1.5	9.10	2.5	6.0
" 17	3.611	1,726	3.45	1.150	1.850	1.5	9.00	2.5	6.0
" 18	3.336	2,152	3.35	1.117	1.817	1.5	8.80	2.5	6.0
" 19	3.260	1,460	3.30	1.100	1.800	1.5	8.70	2.5	6.0
" 20	2.653	1,152	2.75	1.100	1.800	1.5	8.15	2.5	6.0
" 21	2.638	1,287	2.70	1.080	1.780	1.5	8.04	2.5	6.0
" 22	2.526	718	2.65	1.060	1.760	1.5	7.93	2.5	6.0
" 23	2.504	1,427	2.65	1.060	1.760	1.5	7.93	2.5	6.0
" 24	2.400	1,285	2.60	1.040	1.740	1.5	7.82	2.5	6.0
" 25	2.346	1,021	2.55	1.020	1.720	1.5	7.71	2.5	6.0
" 26	2.285	837	2.55	1.020	1.720	1.5	7.71	2.5	6.0
" 27	1.381	1,062	1.90	0.950	1.550	1.0	5.00	2.5	6.0
" 28	1.286	1,099	1.85	0.925	1.525	1.0	4.90	2.5	1.5
" 29	1.182	2,170	1.80	0.900	1.500	1.0	4.80	1.5	6.0
Sub-total		32,864							
2-13 Luse S.C.									
Section 1	0.258	856	0.65	0.650	1.150	1.0	2.95	1.5	1.5
" 2	0.212	1,143	0.55	0.550	1.050	1.0	2.65	1.5	1.5
Sub-total		1,999							
2-14 Masago S.C.									
Section 1	0.552	1,094	1.25	0.625	1.125	1.0	3.50	1.5	2.5
" 2	0.391	750	0.95	0.633	1.133	1.0	3.22	1.5	6.0
Sub-total		1,844							

Table VII.4.1 List of Main and Secondary Irrigation Canals Designed by DOI (3/3)

Name of Canal	Design Discharge (m^3/sec)	Canal Length (m)	Canal Dimension/1 B (m)	h (m)	H (m)	n	T (m)	L_1 (m)	L_2 (m)
2-15 Paccing S.C.									
Section 1	0.858	955	1.45	0.725	1.225	1.0	3.90	1.5	2.5
" 2	0.788	770	1.40	0.700	1.200	1.0	3.80	2.5	6.0
" 3	0.524	1,280	1.20	0.600	1.100	1.0	3.40	2.5	6.0
" 4	0.352	945	0.90	0.600	1.100	1.0	3.10	2.5	6.0
" 5	0.165	1,218	0.50	0.500	1.000	1.0	2.50	2.5	6.0
Sub-total		5,168							
2-16 Hulo S.C.									
Section 1	0.270	2,268	0.65	0.650	1.150	1.0	2.95	2.5	6.0
2-17 Sama Enre S.C.									
Section 1	1.110	1,336	1.70	0.850	1.350	1.0	4.40	2.5	6.0
" 2	1.005	1,878	1.60	0.800	1.300	1.0	4.20	2.5	6.0
" 3	0.621	993	1.30	0.650	1.150	1.0	3.60	1.5	6.0
" 4	0.381	1,613	0.95	0.633	1.133	1.0	3.22	1.5	6.0
" 5	0.320	397	0.85	0.567	1.067	1.0	2.98	1.5	6.0
Sub-total		6,217							
2-18 Poparapa S.C.									
Section 1	0.245	1,224	0.60	0.600	1.100	1.0	2.80	1.5	6.0
Total		39,342							

Remarks: /1:

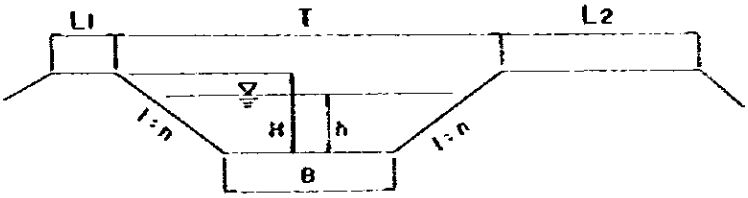


Table VII.4.2 List of Canal Related Structures Designed by D.O.I

(Unit: nos.)

Name of Canal		Turn Out	Drop	Chute	Aque-duct	Bridge	Cross Drain Culvert
I. Main Canal		10	-	-	1	4	21
II. Secondary Canal							
2-1	Palaka S.C.	6	3	2	-	-	7
2-2	Perota S.C	8	-	1	2	-	8
2-3	Labosi S.C.	2	4	-	-	-	4
2-4	Tapale S.C.	1	3	-	-	-	-
2-5	Apale S.C.	5	-	-	-	-	-
2-6	Pao S.C.	2	9	-	-	-	-
2-7	Maradda S.C.	2	8	-	-	-	1
2-8	Batu Tanueh S.C.	3	10	-	-	2	-
2-9	Jaramele S.C.	1	5	-	-	-	-
2-10	Cendranae S.C.	2	3	-	-	-	-
2-11	Barang S.C.	1	1	1	-	-	1
2-12	Aming S.C.	29	11	-	-	12	30
2-13	Lume S.C.	2	3	3	-	-	1
2-14	Masago S.C.	2	1	-	-	1	1
2-15	Paccing S.C.	5	-	-	-	1	3
2-16	Hulo S.C.	1	-	-	-	1	1
2-17	Sama Enre S.C.	5	3	-	-	3	1
2-18	Poparapa S.C.	1	-	-	-	1	-
	Total	88	64	7	3	25	79

Table VII.4.3 Detailed Features of Canal Related Structures (1/6)

Name of Canal	Kind of Structure	Design Discharge (m^3/sec)	Size of Structure
I. Main Canal	1. Turn out		
	(1) B.S.1	3.618	Rooijn B = 0.6m x 1
			Sluice B = 3.0m x 1, B = 2.75m x 3
	(2) B.S.2	0.133	Rooijn B = 0.6m x 1
	(3) B.S.3	0.605	" B = 1.0m x 2, 0.5m, 0.3m
	(4) B.S.4	0.019	" B = 0.3m x 1
	(5) B.S.5	0.504	" B = 1.3m x 1, 0.5m, 0.3m
	(6) B.S.6	0.040	" B = 0.3m x 1
	(7) B.S.7	0.680	" B = 1.0m x 2, 0.4m, 0.3m
	(8) B.S.8	0.397	" B = 1.3m x 1, 0.4m, 0.3m
	(9) B.S.9	0.452	" B = 1.3m x 1, 0.5m, 0.3m
	(10) B.S.10	0.271	" B = 0.4m, 0.3m, Sluice 2.6m x 2
	2. Aqueduct		
	(1) B.S.6a	8.035	L = 31.0m
	3. Bridge		
	(1) B.S.2a	9.295	B = 3.0m, L = 2.60m x 3
	(2) B.S.5b	8.539	B = 3.0m, L = 2.4m x 3
	(3) B.S.7	7.315	B = 3.0m, L = 3.82m x 2
	(4) B.S.10	6.195	B = 3.0m, L = 2.6m x 2
	4. Cross drain culvert		
	(1) B.S.2b	6.378	b = 3.0m x h = 1.9m
	(2) B.S.2c	3.573	b = 2.0m x h = 1.6m
	(3) B.S.2d	1.329	b = 1.0m x h = 1.3m
	(4) B.S.2e	1.196	b = 1.0m x h = 1.3m
	(5) B.S.3a	1.353	b = 1.0m x h = 1.3m
	(6) B.S.3b	15.606	b = 3.0m x h = 1.83m x 2
	(7) B.S.3c	2.373	b = 1.5m x h = 1.4m
	(8) B.S.4a	5.020	b = 3.0m x h = 1.6m
	(9) B.S.4b	27.370	b = 5.0m, R = 2.5m, h = 5.25m
	(10) B.S.5a	8.910	b = 2.4m x 2, h = 1.28m
	(11) B.S.6b	3.600	b = 1.5m x h = 1.90m
	(12) B.S.8a	4.810	b = 2.0m x h = 1.90m
	(13) B.S.8b	1.320	b = 1.0m x h = 1.30m
	(14) B.S.8c	1.080	b = 1.0m x h = 1.10m
	(15) B.S.9a	0.340	ϕ = 0.8m
	(16) B.S.9b	0.210	ϕ = 0.8m
	(17) B.S.9c	0.370	ϕ = 0.8m
	(18) B.S.9d	26.580	b = 5.0m R = 2.5m, h = 5.16m
	(19) B.S.9e	1.457	b = 1.0m x h = 1.3m
	(20) B.S.10a	1.962	b = 1.0m x h = 1.6m
	(21) B.S.10b	3.310	b = 2.0m x h = 1.1m
II. Secondary Canal			
2-1 Palaka S.C.	1. Turn out		
	(1) B.Pa.1	0.075	Rooijn B = 0.3m x 1
	(2) B.Pa.2	0.143	" B = 0.8m x 1
	(3) B.Pa.3	2.581	" B = 1.3m x 2, 0.4m, 0.3m
	(4) B.Pa.4	0.331	" B = 0.6m x 1, 0.5m
	(5) B.Pa.5	0.249	" B = 0.4m x 2
	(6) B.Pa.6	0.391	" B = 0.8m x 1, 0.5m, 0.4m
	2. Drop		
	(1) B.Pa.1a	3.509	Z = 1.5m
	(2) B.Pa.1b	3.509	Z = 1.5m
	(3) B.Pa.4b	0.734	Z = 2.5m
	3. Chute		
	(1) B.Pa.4a	0.734	H = 3.973m, L = 61.7m
	(2) B.Pa.6a	0.931	H = 8.075m, L = 234m

Table VII.4.3 Detailed Features of Canal Related Structures (2/6)

Name of Canal	Kind of Structure	Design Discharge (m^3/sec)	Size of Structure
	4. Cross drain culvert		
	(1) B.Pa.1c	0.450	ϕ = 0.8m
	(2) B.Pa.2a	4.880	b = 1.6m x h = 2.0m
	(3) B.Pa.2b	0.940	ϕ = 0.8m
	(4) B.Pa.3a	3.030	b = 1.2m x h = 1.80m
	(5) B.Pa.3b	3.630	b = 1.5m x h = 1.90m
	(6) B.Pa.3c	2.900	b = 1.2m x h = 1.90m
	(7) B.Pa.5a	0.890	ϕ = 0.8m
2-2 Parota	1. Turn out		
	(1) B.Pr.1a	0.071	Romijn B = 0.3m x 2
	(2) B.Pr.3a	0.100	" B = 0.5m x 1
	(3) B.Pr.3b	0.029	" B = 0.5m x 1
	(4) B.Pr.4a	0.077	" B = 0.5m x 1
	(5) B.Pr.5a	2.302	" B = 1.0m x 2, 0.4m, 0.3m
	(6) B.Pr.6a	1.009	" B = 1.0m x 1, 0.4m, 0.3m
	(7) B.Pr.7a	0.065	" B = 0.4m x 1
	(8) B.Pr.8a	0.125	" B = 0.8m x 1, 0.4m x 2
	2. Chute		
	(1) B.Pr.2a	2.510	H = 9.781m, L = 80.0m
	3. Aqueduct		
	(1) B.Pr.2b	2.510	L = 90m, H = 12.0m
	(2) B.Pr.3c	2.410	L = 60m, H = 7.0m
	4. Cross drain culvert		
	(1) B.Pr.1a	1.820	b = 1.0m x h = 1.5m
	(2) B.Pr.3a	16.860	b = 4.0m x h = 3.0m
	(3) B.Pr.3b	3.850	b = 1.5m x h = 1.9m
	(4) B.Pr.4a	21.110	b = 2.0m x 3, h = 2.0m
	(5) B.Pr.5a	7.690	b = 1.5m x 2, h = 2.0m
	(6) B.Pr.6a	6.720	b = 2.0m h = 2.4m
	(7) B.Pr.7a	10.110	b = 1.7m x 2, h = 2.0m
	(8) B.Pr.8a	13.020	b = 2.0m x 2, h = 2.8m
2-3 Labesi S.C.	1. Turn out		
	(1) B.Lb.1	0.157	Romijn B = 0.8m, 0.3m
	(2) B.Lb.1	0.191	" B = 0.8m x 2, 0.3m
	2. Drop		
	(1) B.Lb.1b	0.566	Z = 2.225m
	(2) B.Lb.1c	0.566	Z = 2.225m
	(3) B.Lb.1d	0.566	Z = 2.225m
	(4) B.Lb.2	0.218	Z = 1.700m
	3. Cross drain culvert		
	(1) B.Lb.1a	0.766	ϕ = 0.8m
	(2) B.Lb.2a	1.503	b = 1.0m x h = 1.3m
	(3) B.Lb.2b	1.782	b = 1.0m x h = 1.3m
	(4) B.Lb.2c	1.579	b = 1.0m x h = 1.2m
2-4 Tapale S.C.	1. Turn out		
	(1) B.TP.1	0.095	Romijn B = 0.4m, 0.3m
	2. Drop		
	(1) B.TP.1a	0.328	Z = 2.4m
	(2) B.TP.1b	0.328	Z = 2.4m
	(3) B.TP.1c	0.328	Z = 2.4m
2-5 Pao S.C.	1. Turn out		
	(1) B.P.1	0.351	Romijn B = 0.8m, 0.4m
	(2) B.P.2	0.352	" B = 0.8m, 0.5m, 0.3m

Table VII.4.3 Detailed Features of Canal Related Structures (3/6)

Name of Canal	Kind of Structure	Design Discharge (m^3/sec)	Size of Structure
	2. Drop		
	(1) B.P.1a	0.574	Z = 2.034m
	(2) B.P.1b	0.574	Z = 2.034m
	(3) B.P.1c	0.574	Z = 2.034m
	(4) B.P.1d	0.574	Z = 2.034m
	(5) B.P.1e	0.574	Z = 2.034m
	(6) B.P.1	0.342	Z = 1.750m
	(7) B.P.2a	0.342	Z = 1.761m
	(8) B.P.2b	0.342	Z = 1.761m
	(9) B.P.2	0.162	Z = 1.710m
2-7 Marada S.C.	1. Turn out		
	(1) B.Ma.1	0.372	Roaljo B = 0.8m, 0.4m
	(2) B.Ma.1	0.207	" B = 0.5m, 0.3m
	2. Drop		
	(1) B.Ma.1a	0.455	Z = 1.700m
	(2) B.Ma.1b	0.455	Z = 1.700m
	(3) B.Ma.1c	0.455	Z = 1.700m
	(4) B.Ma.1	0.207	Z = 1.444m
	(5) B.Ma.2a	0.207	Z = 1.444m
	(6) B.Ma.2b	0.207	Z = 1.444m
	(7) B.Ma.2d	0.207	Z = 1.444m
	(8) B.Ma.2	0.157	Z = 1.400m
	3. Cross drain culvert		
	(1) B.Ma.2c	5.885	b = 3.0m x H = 1.5m
2-8 Batu Tasaeh S.C.	1. Turn out		
	(1) B.BT.1	0.336	Roeljo B = 0.6m, 0.4m
	(2) B.BT.2	0.157	" B = 0.3m x 2
	(3) B.BT.3	0.376	" B = 0.8m, 0.6m, 0.3m
	2. Drop		
	(1) B.BT.1a	0.653	Z = 2.00m
	(2) B.BT.1b	0.648	Z = 2.00m
	(3) B.BT.1c	0.648	Z = 2.00m
	(4) B.BT.1	0.443	Z = 1.95m
	(5) B.BT.2a	0.443	Z = 1.90m
	(6) B.BT.2b	0.443	Z = 1.90m
	(7) B.BT.2c	0.443	Z = 1.90m
	(8) B.BT.2d	0.443	Z = 1.90m
	(9) B.BT.2e	0.443	Z = 1.90m
	(10) B.BT.2	0.376	Z = 1.45m
	3. Bridge		
	(1) B.BT.1	0.443	B = 8.1m, L = 1.6m
	(2) B.BT.2f	0.443	B = 3.0m, L = 1.6m
2-9 Jarapele S.C.	1. Turn out		
	(1) B.Jr.1	0.380	Roaljo B = 0.8m, 0.4m, 0.3m
	2. Drop		
	(1) B.Jr.1a	0.380	Z = 2.258m
	(2) B.Jr.1b	0.380	Z = 2.258m
	(3) B.Jr.1c	0.380	Z = 2.258m
	(4) B.Jr.1d	0.380	Z = 2.258m
	(5) B.Jr.1e	0.380	Z = 2.258m
2-10 Cendranas S.C.	1. Turn out		
	(1) B.Ca.1	0.224	Roaljo B = 0.5m, 0.3m
	(2) B.Ca.2	0.316	" B = 0.5m, 0.4m, 0.3m

Table VII.4.3 Detailed Features of Canal Related Structures (4/6)

Name of Canal	Kind of Structure	Design Discharge (m^3/sec)	Size of Structure
	2. Drop		
	(1) B.Ca.2a	0.316	Z = 3.00m
	(2) B.Ca.2b	0.316	Z = 3.00m
	(3) B.Ca.2	0.149	Z = 2.25m
2-11 Barang S.C.	1. Turn out		
	(1) B.Ba.1	0.280	Romijn B = 0.8m, 0.3m
	2. Drop		
	(1) B.Ba.1b	0.257	Z = 1.974m
	3. Chute		
	(1) B.Ba.1c	0.257	H = 5.953m, L = 107m
	4. Cross drain culvert		
	(1) B.Ba.1a	0.432	ϕ = 0.8m
2-12 Asing S.C.	1. Turn out		
	(1) B.As.1	0.097	Romijn B = 0.3m
	(2) B.As.2	0.472	" B = 0.8m, 0.5m, 0.4m
	(3) B.As.3	0.132	" B = 0.4m
	(4) B.As.4	0.136	" B = 0.4m
	(5) B.As.5	0.149	" B = 0.5m
	(6) B.As.6	0.180	" B = 0.6m
	(7) B.As.7	0.141	" B = 0.5m
	(8) B.As.8	0.515	" B = 0.8m, 0.5m, 0.4m
	(9) B.As.9	0.050	" B = 0.4m
	(10) B.As.10	0.393	" B = 1.0m, 0.5m
	(11) B.As.11	0.360	" B = 0.6m, 0.4m
	(12) B.As.12	0.725	" B = 0.8m x 3, 0.5m
	(13) B.As.13	0.218	" B = 0.6m, 0.4m
	(14) B.As.14	0.365	" B = 0.8m, 0.4m x 2
	(15) B.As.15	0.053	" B = 0.4m
	(16) B.As.16	0.153	" B = 0.5m
	(17) B.As.17	0.503	" B = 0.6m, 0.5m, 0.4m x 2
	(18) B.As.18	0.112	" B = 0.4m
	(19) B.As.19	0.256	" B = 1.3m, 0.4m, 0.3m
	(20) B.As.20	0.097	" B = 0.4m
	(21) B.As.21	0.176	" B = 0.6m
	(22) B.As.22	0.060	" B = 0.4m
	(23) B.As.23	0.212	" B = 0.4m x 2
	(24) B.As.24	0.110	" B = 0.4m
	(25) B.As.25	0.119	" B = 0.4m
	(26) B.As.26	0.592	" B = 1.3m x 2, 0.6m
	(27) B.As.27	0.152	" B = 0.5m
	(28) B.As.28	0.163	" B = 0.6m
	(29) B.As.29	0.134	" B = 0.4m
	2. Drop		
	(1) B.As.8	0.245	Z = 2.2m
	(2) B.As.9a	5.083	Z = 1.0m
	(3) B.As.9	5.070	Z = 1.0m
	(4) B.As.10a	5.070	Z = 2.0m
	(5) B.As.10b	5.070	Z = 1.4m
	(6) B.As.10	0.253	Z = 2.5m
	(7) B.As.11b	4.805	Z = 1.5m
	(8) B.As.11c	4.805	Z = 2.0m
	(9) B.As.13b	4.065	Z = 0.73m
	(10) B.As.20a	2.683	Z = 0.7m
	(11) B.As.25a	2.345	Z = 1.1m
	3. Bridge		
	(1) B.As.7	5.510	B = 4.0m, L = 2.75m x 2
	(2) B.As.8	5.426	B = 4.0m, L = 2.85m x 2
	(3) B.As.9	5.083	B = 4.0m, L = 2.15m x 2

Table VII.4.3 Detailed Features of Canal Related Structures (5/6)

Name of Canal	Kind of Structure	Design Discharge (m^3/sec)	Size of Structure
	(4) B.Am.10	5.070	B = 4.0m, L = 3.10m x 2
	(5) B.Am.11a	4.895	B = 4.0m, L = 2.95m x 2
	(6) B.Am.12a	4.573	B = 4.0m, L = 2.85m x 2
	(7) B.Am.13a	4.066	B = 4.0m, L = 2.70m x 2
	(8) B.Am.17	3.391	B = 4.0m, L = 2.20m x 2
	(9) B.Am.26	2.285	B = 4.0m, L = 2.70m
	(10) B.Am.27a	1.381	B = 4.0m, L = 2.80m
	(11) B.Am.28a	1.286	B = 4.0m, L = 2.70m
	(12) B.Am.28c	1.286	B = 4.0m, L = 2.70m
	4. Cross drain culvert		
	(1) B.Am.1a	0.783	ϕ = 0.8m
	(2) B.Am.1b	23.712	B = 4.0m x H = 2.96m
	(3) B.Am.1c	1.373	B = 0.8m x H = 1.40m
	(4) B.Am.2a	4.910	B = 1.4m x H = 1.80m
	(5) B.Am.3a	0.370	ϕ = 0.8m
	(6) B.Am.3b	0.908	ϕ = 0.8m
	(7) B.Am.3c	27.424	B = 6.0m x H = 5.6m
	(8) B.Am.4a	0.759	ϕ = 0.8m
	(9) B.Am.4b	6.931	B = 3.0m x H = 3.0m
	(10) B.Am.4c	7.663	B = 3.0m x H = 4.0m
	(11) B.Am.5a	0.970	B = 0.8m x H = 1.1m
	(12) B.Am.5b	0.856	B = 0.8m x H = 1.1m
	(13) B.Am.6a	9.395	B = 2.0m x 2, H = 1.7m
	(14) B.Am.6b	13.452	B = 3.0m x 2, H = 1.6m
	(15) B.Am.7a	1.542	B = 1.0m x H = 1.3m
	(16) B.Am.8a	4.604	B = 2.0m x H = 1.6m
	(17) B.Am.8b	5.020	B = 2.0m x H = 1.8m
	(18) B.Am.9b	3.960	B = 2.0m x H = 1.5m
	(19) B.Am.13c	1.640	B = 1.0m x H = 1.4m
	(20) B.Am.15a	11.610	B = 2.5m x 2, H = 1.7m
	(21) B.Am.16a	16.715	B = 3.0m x 2, H = 1.9m
	(22) B.Am.18a	35.766	B = 3.0m x 3, H = 2.5m
	(23) B.Am.19a	8.495	B = 3.0m x H = 1.9m
	(24) B.Am.20b	60.700	B = 6.0m x 2, H = 5.8m
	(25) B.Am.22a	1.361	B = 1.5m x H = 0.95m
	(26) B.Am.24a	4.200	B = 2.0m x H = 1.50m
	(27) B.Am.25b	9.937	B = 2.5m x H = 2.50m
	(28) B.Am.25c	9.937	B = 2.5m x H = 2.50m
	(29) B.Am.26a	19.212	B = 2.5m x 2, H = 2.40m
	(30) B.Am.28b	17.200	B = 2.25m x 2, H = 2.30m
2-13 Luze S.C.	1. Turn out		
	(1) B.L.1	0.102	Romijn b = 0.5m
	(2) B.L.2	0.172	" b = 0.8m, 0.5m
	2. Drop		
	(1) B.L.1b	0.258	Z = 2.50m
	(2) B.L.1d	0.258	Z = 2.50m
	(3) B.L.2b	0.212	Z = 2.50m
	3. Chute		
	(1) B.L.1a	0.258	H = 6.974m, L = 137m
	(2) B.L.1c	0.258	H = 6.022m, L = 112m
	(3) B.L.2a	0.212	H = 22.488m, L = 638m
	4. Cross drain culvert		
	(1) B.L.2c	4.750	B = 1.90m x H = 1.50m
2-14 Masago S.C.	1. Turn out		
	(1) B.M.1	0.216	Romijn b = 0.5m, 0.4m
	(2) B.M.2	0.306	" b = 0.6m x 2, 0.4m
	2. Drop		
	(1) B.M.2c	0.391	Z = 1.50m

Table VII.4.3 Detailed Features of Canal Related Structures (6/6)

Name of Canal	Kind of Structure	Design Discharge (m^3/sec)	Size of Structure
	3. Bridge		
	(1) B.M.2a	0.391	B = 3.0m, L = 1.6m
	4. Cross drain culvert		
	(1) B.M.2b	8.770	B = 2.50m x H = 2.30m
2-15 Paccing S.C.	1. Turn out		
	(1) B.Pc.1	0.154	Romijn b = 0.4m x 2
	(2) B.Pc.2	0.169	" b = 0.8m, 0.4m x 2
	(3) B.Pc.3	0.193	" b = 0.5m, 0.4m x 2
	(4) B.Pc.4	0.259	" b = 0.4m x 3
	(5) B.Pc.5	0.317	" b = 0.4m x 3
	2. Bridge		
	(1) B.Pc.1a	0.858	B = 4.0m, L = 1.45m
	3. Cross drain culvert		
	(1) B.Pc.3a	2.810	B = 1.5m, H = 1.4m
	(2) B.Pc.5a	0.165	B = 1.0m, H = 0.9m
	(3) B.Pc.5b	2.210	B = 1.5m, H = 1.7m
2-16 Hulo S.C.	1. Turn out		
	(1) B.H.1	0.211	Romijn B = 0.5m, 0.4m x 2
	2. Bridge		
	(1) B.H.1	0.270	B = 4.0m, L = 1.3m
	3. Cross drain culvert		
	(1) B.H.1a	3.570	B = 2.0m x H = 1.40m
2-17 Sama Fare S.C.	1. Turn out		
	(1) B.SE.1	0.185	Romijn b = 0.6m
	(2) B.SE.2	0.407	" b = 0.8m, 0.5m, 0.4m x 2
	(3) B.SE.3	0.371	" b = 0.6m x 2
	(4) B.SE.4	0.121	" b = 0.5m
	(5) B.SE.5	0.447	" b = 0.6m, 0.5m, 0.4m
	2. Drop		
	(1) B.SE.4a	0.381	Z = 2.0m
	(2) B.SE.5a	0.320	Z = 2.0m
	(3) B.SE.5b	0.320	Z = 1.5m
	3. Bridge		
	(1) B.SE.3a	0.621	B = 4.0m, L = 1.95m
	(2) B.SE.4b	0.381	B = 4.0m, L = 1.60m
	(3) B.SE.5c	0.320	B = 4.0m, L = 1.40m
	4. Cross drain culvert		
	(1) B.SE.2a	4.020	B = 2.0m x L = 1.50m
2-18 Poparapa S.C.	1. Turn out		
	(1) B.Pp.1	0.314	Romijn b = 0.5m x 2
	2. Bridge		
	(1) B.Pp.1a	0.245	B = 6.0m, L = 1.2m