REPUBLIC OF INDONESIA

MINISTRY OF PUBLIC WORKS
DIRECTORATE GENERAL OF
WATER RESOURCES DEVELOPMENT

FEASIBILITY STUDY
ON
THE LANGKEMME IRRIGATION PROJECT

ANNEX-II

HYDROLOGY
IRRIGATION
CONSTRUCTION PLAN

MARCH 1981

JAPAN INTERNATIONAL COOPERATION AGENCY
TOKYO JAPAN

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ANNEX-II HYDROLOGY, IRRIGATION, AND CONSTRUCTION PLAN

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Glossary of Terms and Abbreviation

1. Local Administrative Organization

Kabupaten (Kab.)	:	District
Kecamatan (Kec.)	:	Sub-district
Desa	:	Village
Bupati	:	Chief of Kabupaten
Camat	:	Chief of Kecamatan
Kepala Desa	:	Chief of Desa

2. Organization for Irrigation and Agricultural Development

DPU	:	Ministry of Public Works
DGWRD	:	Directorate General of Water Resources Development
P3SA	:	Sub-directorate of Planning and Programming
PLN	:	Public Corporation of Electricity
BRI	:	Indonesia People's Bank
BIMAS/INMAS	:	Mass Guidance for Self-sufficiency in Food
DOLOG	:	Provincial Rice Purchasing Agency
BUUD/KUD	:	Village Unit Executive Body/Agricultural Cooperative Organization
P3A	:	Water User's Association
BPP	:	Rural Extension Center

3. Other Local Terms

Polowijo	:	Second Crops, Planted after Harveste of Wet Season Paddy
Pelita I	:	First Five-Year Development Plan
Pelita II	:	Second Five-Year Development Plan
Pelita III	:	Third Five-Year Development Plan
PPL	:	Field Extension Worker
PPM	:	Extension Supervisor
PPS	:	Subject Matter Specialist

4. Area and Volume

m^2	:	square meter
ha	:	hectare
km^2	:	square kilometer
l	:	liter
m^3	:	cubic meter
t	:	ton

5. Derived Measures based on the Same Symbols

m^3/sec	:	cubic meter per second
t/ha	:	ton per hectare
m^3/km^2	:	cubic meter per square kilometer
mm/day	:	millimeter per day
l/sec/ha	:	liter per second per hectare
l/day	:	liter per day
m^3/km^2/year	:	cubic meter per square kilometer per year
meq/100g	:	milli-equivalent per 100 gram of soil
km/sec	:	kilometer per second
kg/cm^2	:	kilogram per square centimeter
cm/sec	:	centimeter per second
t/m^3	:	ton per cubic meter
t/m^2	:	ton per square meter

6. Electric Measures

kV	:	kilovolt
kW	:	kilowatt
kWh	:	kilowatt-hour
MW	:	megawatt
kVA	:	kilovolt ampere
Hz	:	Hertz

7. Currency

US$	:	United States Dollars
Rp	:	Rupiah

US$1 = Rp 625

8. Others

%	:	percent
No.	:	number
Nos.	:	numbers
vs.	:	versus
MSL	:	Mean Sea Level

CHAPTER III HYDROLOGY

CHAPTER III HYDROLOGY

3.1 DATA AVAILABLE

Ten pluviometric stations including automatic rain gauges are located in and around the central Walanae river basin as shown in Fig. 3.1.1. The stations are operated by P3SA /1, PMG /2, DIPERITA /3 and PMA /4 as mentioned in Table 3.1.1. The Watan Soppeng pluviometric station, which has the longest operational duration among them, has been operated since 1906. The station has provided long range monthly rainfall records and daily rainfall since 1963. Nine remaining stations have also commenced the records of daily rainfall in recent five years (see Table 3.1.2). As regards hourly rainfall, no data is available at all in and around the project area.

As illustrated in Fig. 3.1.1, networks of pluviometric stations are extremely uneven and all of the stations are located at relatively lowlying basins. No rainfall data is available in the mountaneous watersheds relevant to the water resources for the project. Additional rainfall gauges should be urgently installed in the mountaneous watersheds for the future study and the operation/maintenance of the project.

While, seven water level gauging stations are located on the Walanae river system in and around the project area as shown in Fig. 3.1.1. These stations have been operated by P3SA, PMA and DPMA /5. Among them, two gauging stations have been installed on the Langkemme and the Sero river relevant to the project. The Langkemme Water Level Gauging Station equipping an automatic recorder has been operated since May, 1974 by the PMA, and the Sero Water Level Gauging Station equipping a water gauging staff, since July 1975 by the P3SA. (see Table 3.1.2)

3.2 WATER RESOURCES

3.2.1 Water Sources

The Langkemme Irrigation Project would depend its water resources on the Langkemme river, the Sero river system, and the seven small tributaries of the Walanae river. The catchment areas of respective rivers are estimated at discharge measurement sites as tabulated below:

Langkemme River	104 km^2
Sero River	459 km^2
Tributaries (in Total)	104 km^2

/1 Parancangan dam Pengembangan Sumber Air
/2 Pusat Meteorology
/3 Dinas Partanian
/4 Penyelidikan Masalar Air
/5 Directorate Penyelidikan Masalah Air

3.2.2 Characteristics of Rainfall in the Langkemme and the Sero River Valley

As illustrated in Fig. 3.2.1, the Langkemme and the Sero river valley including Soppeng plain are completely covered with isohyeto-line of greater than 1,500 mm of annual rainfall. Annual basin rainfall of a little greater than 2,000 mm has occurred in the vicinity of the southern and western boundary of the watersheds of the Langkemme and the Sero river. Annual mean rainfall in the project area is estimated at approximately 1,550 mm averaging the rainfall records of the Watan Soppeng and Takkalala stations. While, the annual average basin rainfall graced in the mountaneous watersheds of the Langkemme and the Sero river is assessed to be about 2,300 mm at least, taking into account of the annual evapotranspiration of 900 mm and the average annual runoff of 1,200 mm from the both rivers.

In addition to the uneven rainfall distributions caused by topography and geography, the rainfall in the relevant watersheds is characterized by extremely high rainfall intensity and narrow rainfall spread. The average basin rainfall seems to be, therefore, considerably small as compared with the point rainfall recorded in the stations located at lowlying plain.

Seasonal distribution of rainfall in and around the project area is also extremely uneven. About 85% of the annual rainfall of 1,550 mm concentrates within nine months of the transitional and the rainy season. The rainfall during dry season, from August to October, is only about 220 mm on an average, accounting for 15% equivalence of the annual rainfall.

3.2.3 River Flow

The water levels in the Langkemme and the Sero river have been recorded since 1974 at the respective gauging stations. The mean daily discharges are converted from the water level records by using the respective rating curves which were worked out through discharge measurements by the Government and are compiled in the Data Book. Based on these discharge, mean 10-day discharges are averaged and also compiled in the Data Book.

The flow in the Sero river sharply fluctuates season to season as well as the flow in the Langkemme river. In particular, the Sero river is extremely depleted during dry season. Despite of its threefold catchment as compared with that of the Langkemme river, the droughty discharge in the Sero river is smaller than that of the Langkemme river.

The minimum mean 10-day discharges of 0.79 m^3/sec and 0.57 m^3/sec are recorded at the Langkemme station in 1976 and at the Sero station in 1977, respectively. Meanwhile, the maximum mean 10-day discharge of 26.04 m^3/sec are also recorded at the Langkemme station in 1977 and that of 139.68 m^3/sec at the Sero station in the same year.

The records collected from the both water level gauging stations are partially absent owing mainly to troubles of gauges. To inter-supplement the lack of records, correlation study on specific runoff is made between both stations, as shown in Fig. 3.2.2. The correlative equations are developed as,

$Y = 2.5\,X$, in case of $Y < 7.5$
$Y = 1.25X + 3.75$, in case of $Y > 7.5$
where,
- Y : Specific discharge of the Langkemme river by 10-day basis ($m^3/sec/100\ km^2$)
- X : Specific discharge of the Sero river by 10-day basis ($m^3/sec/100\ km^2$)

3.2.4 Dependable Water Resources

The dependable water resources for the Langkemme Irrigation Project are contributed by the Langkemme river, the Sero river and the seven tributeries of the Walanae river as mentioned hereinbefore. Fig. 3.2.3(1) and (2) show the river system in and around the project area.

(1) Langkemme River

The Cennae irrigation scheme commanding about 200 ha of paddy field and accompanying a micro hydropower station has been operated by the DPU in the vicinity of the Langkemme Water Level Gauging Station. The intake structure of the scheme is located at about 2 km upstream from the gauging station.

While, the intake structure for the Langkemme Irrigation Project would be proposed at about half km further upstream from the existing Cennae intake, as schematically illustrated in Fig. 3.2.4. The dependable water resources at the proposed intake site would be clarified on the basis of the discharge recorded at the gauging station and the discharge consumptively used in the Cennae Irrigation scheme.

To estimate the dependable water resources at the proposed intake site, water balance survey in the Cennae scheme has been made since August by measuring the diverted flow into the Cennae intake, the return flow through the Cennae head reach and the outflow from the Cennae tail reach. It is clarified through the measuring survey that the intaked discharge in the Cennae scheme is closely related to the discharge in the main reach of the langkemme river during drought season, because no regulator is equipped in the Cennae intake.

Through the discharge measurements during drought season, a constant ratio of 0.22 is estimated between the intaked discharge in the Cennae scheme and the discharge being concurrently recorded at the gauging station. While, minimum 10-day mean discharge in the Langkemme station was recorded to be 0.79 m^3/sec in 1976.

The intaked discharge in the Cennae scheme during drought season is estimated at 0.17 m^3/sec by multiplying the droughtest discharge of 0.79 m^3/sec by the constant ratio of 0.22. The discharge of 0.17 m^3/sec is likely to be equivalent to the average irrigation requirement for the Cennae scheme. The available discharge at the proposed intake site would be estimated by summing the recorded discharge at the station and the intaked discharge in the Cennae scheme. To conservatively estimate the available water resources, the discharge of 0.17 m^3/sec would be constantly added to the discharge recorded at the station. The discharge at the proposed intake site is summarized in Table 3.2.1(1).

(2) Sero River System

Three intake structures for the Sero diversion canal system would be proposed on the three tributaries of the Sero river system, viz., the Jupang, the Unyi and the Pisig river. No data is available at all on the runoff of the respective tributaries. The dependable discharge at the confluence of the Pising and the Jupang river would be converted from the discharge recorded at the Sero Water Level Gauging Station on the assumption that the runoff intensity in the Sero river system is varied in proportion to the catchment area. The conversion ratio would be,

$$\frac{\text{The catchment area at the confluence of The Jupang and Pising rivers}}{\text{The catchment area of the Sero gauging station}} = \frac{335\ km^2}{459\ km^2} = 0.73$$

The dependable discharge at the confluence of the Jupang and Pising rivers is worked out by multiplying the discharge recorded at the station by conversion ratio of 0.73 and summarized in Table 3.2.1(2).

(3) Tributaries of the Walanae River

Seven tributaries of the Walanae river flow across the project area from the southwest to the northeast, and irrigate about 6,500 ha of paddy field under the Desa irrigation schemes at present. The water resources endowed in the seven tributaries are essential for the irrigation plan in this project, but no data is available on the runoff of these tributaries at present. As shown in Fig. 3.2.3(2), total catchment area of the seven tributaries extends to 104 km^2, being just equal to the catchment area of the Langkemme river.

The catchment area of tributaries extends adjacent northward to that of the Langkemme river. The geological, topographic and forestational conditions in the watershed of the tributaries are almost similar to that of the Langkemme river. In this view, the discharge of the tributaries might be assessed on the basis of the discharge recorded in the Langkemme gauging station.

Discharge measurements have been undertaken in the seven tributaries in the course of the study, as compiled in the Data Book. The ratio of specific discharge between both rivers are seasonally varied. To estimate the discharge of the tributaries, conversion coefficients are obtained by monthly based on the data of discharge measured in the tributaries and the records in the Langkemme station.

According to the discharges measured in August, the runoff during rainy season in the tributaries is likely to be as stable as the runoff in the Langkemme river. The conversion coefficient obtained in August would be applicable for whole rainy season. The following three conversion coefficients (c) are worked out according to the seasonal fluctuation of specific runoffs.

i) November thru August

Qt (Tributaries) = 1.273 $m^3/s/100\ km^2$
Ql (Langkemme) = 1.221 $m^3/s/100\ km^2$
$C = \frac{Qt}{Ql} = \underline{1.0426}$

ii) September

$Qt = 0.471\ (m^3/sec/100\ km^2)$
$Ql = 1.221\ (m^3/sec/100\ km^2)$
$C = \underline{0.3857}$

iii) October

$Qt = 1.221\ (m^3/sec/100\ km^2)$
$Ql = 0.514\ (m^3/sec/100\ km^2)$.
$C = \underline{0.4210}$

The dependable discharges of the tributaries at the junctions of the main canal are worked out by 10-day basis as shown in Table 3.2.1(3), multiplying the recorded discharge at the Langkemme gauging station by the conversion coefficients mentioned above. Based on Table-3.2.1(1)(2)(3), annual runoffs of three rivers for recent five years are illustrated as shown in Fig. 3.2.5.

3.2.5 Frequency Study on Drought Discharge

The annual minimum 10-day discharge of the Langkemme and Sero rivers are picked out from the discharge data compiled in the Data Book and tabulated below;

Year	Minimum 10-day Discharge at Intake Site	
	Langkemme River	Sero River
1975	0.90	1.90
1976	0.79	0.90
1977	1.05	0.57
1978	1.41	2.29
1979	1.09	1.17

The five samples are insufficient for theoretical frequency study. The recurrence of the minimum discharge is graphically analyzed by the Thomas plots as shown in Fig.-3.2.6. The probable minimum 10-day discharge corresponding to respective recurrences and the recurrence of the minimum 10-day discharge in recent five years are obtained from the Thomas plots and tabulated below respectively;

Return Period (year)	Probable Minimum 10-day Discharge	
	Langkemme River (m3/sec)	Sero River (m3/sec)
20	0.65	0.34
10	0.72	0.45
5	0.81	0.63
3	0.92	0.87
2	1.03	1.23

Year	Recurrence of Minimum 10-day Discharge	
	Langkemme River (Year)	Sero River (Year)
1975	3.1	1.4
1976	6.1	2.9
1977	1.9	6.1
1978	1.2	1.3
1979	1.7	2.1

3.2.6 Technical Remarks

Specific droughty discharge of ten water sources for the Luwu and the Sadang project are collected and summarized by monthly basis in Table 3.2.2, together with the droughty discharge analyzed in the Langkemme and the Sero rivers.

As clarified in Table 3.2.2, both the Sero and Langkemme rivers seem to be extremely exhausted during dry season as well as the Pemali river under the Sadang project. Watershed management and development of storage reservoir would be essential for the both rivers in future to conserve the endowed water resources and to regulate the natural runoff.

3.3 FLOOD

3.3.1 Characteristics of River Basin

The Langkemme Irrigation Project would comprise mainly the Langkemme irrigation canal system and the Sero diversion canal system. The both systems construct various structures in and across the tributaries of the Walanae river. Eleven tributaries are relevant to the systems. The characteristic of each river basin is as summarized in Table 3.3.1.

3.3.2 Concentration Time

Concentration time comprises runoff time and inflow time. The averaged riverbed gradient of each tributary concerned is far steeper than 1/100 as shown in Table 3.3.1. On reference to the wellknown Kraven's Table, the average runoff velocity of flood is assumed to be

3.5 m/s for each river reach of the tributary. Runoff time of flood is obtained by dividing length of river reach by runoff velocity.

Inflow time or a time required for rainfall water to flow into river reach is also assumed at a half hour in due consideration of slope of basin, distance to inflow, roughness of slope and anticipated rainfall intensity. The concentration time of each tributary, consisting of runoff time plus inflow time, is calculated and summarized in Table 3.3.2.

3.3.3 Runoff Coefficient

Runoff coefficient is variable according to magnitude of flooding and closely related to rainfall intensity and concentration time. From empirical viewpoints, however, runoff coefficient is assumed at 60% on an average for flood analysis in this study.

3.3.4 Probable Maximum Daily Rainfall

Annual maximum rainfall data picked out from the Watan Soppeng and Takalala pluviometric stations are Thomas-plotted as shown in Fig. 3.3.1. The probable maximum daily rainfalls are read on the Thomas plottings and summarized as follows,

Return Period (year)	Probable Daily Rainfall	
	Takalala St.	Watan Soppeng St.
100	230 (mm)	260 (mm)
50	192	232
30	176	213
20	162	195
10	141	168
5	118	138
2	87	95

3.3.5 Average Basin Rainfall

To work out average basin rainfall, Tiessen coefficient is not available in this study due to lack of meteorological networks. The average basin rainfall in the relevant catchment must be estimated from the point rainfall recorded at the Watan Soppeng and Takalala stations.

Fig. 3.3.2 indicates conversion ratios from point rainfall into average basin rainfall. Two curves in the figure are worked out in the reports on the irrigation project in Indonesia and the remaining is a curve recommended by USBR based on the various storm analysis.

Generally, storm rainfalls in the South Sulawesi seem to be less intensive as compared with those in the East Jawa. The curve recommended by USBR would be applied for estimate of basin rainfall; the conversion ratio of 0.82, 0.74 is used for the estimate of the basin rainfall of the Langkemme and the Jupang river, respectively. For catchment smaller than 50 km^2, point rainfall would be taken for basin rainfall.

3.3.6 Rainfall Intensity

No data is available on rainfall intensity of short duration. Rainfall intensity within concentration time of flood is estimated by an empirical formula prepared by Dr. Mononobe as follows,

$$Ir = \frac{R24}{24} \left(\frac{24}{Tc}\right)^{2/3}$$

where,

Ir: Maximum average rainfall intensity within concentration time (mm/hr)
R24: Daily rainfall (mm)
Tc: Concentration time (hr)

The probable rainfalls estimated by the records at the Watan Soppeng station are conservatively applicable for the calculation of the rainfall intensity. The calculated rainfall intensity in each river basin is summarized in Table 3.3.3. The rainfall intensities in the Langkemme and Jupang river basins are estimated bsed on the probable rainfall reduced by the conversion ratio of 0.82, 0.74, respectively.

3.3.7 Probable Flood

Rational formula as presented below is applied for estimate of probable peak flood discharge:

$$Q = 0.2778\ C \cdot I \cdot A$$

where,

Q : Peak flood (m^3/sec)
C : Coefficient of runoff
I : Rainfall intensity within time of concentration (mm/ha)
A : Catchment area (km^2)

The calculated peak floods in the respective tributaries are presented in Table 3.3.4.

3.3.8 Comparison with Creager's Curve

The calculated specific peak floods are compared with the Creager's Curve of C = 100 and 30 as shown in Fig. 3.3.3. The probable specific peak floods estimated with a return period of 30 years are closely plotted on the Creager's Curve developed by C = 30.

3.3.9 Flood Water Level

High water level during a period of peak flood discharge are estimated by Manning's Formula, based on the probable peak discharge previously estimated and the longitudinal and cross sections of river channel prepared by topographic survey. The estimated water levels are summarized in Table 3.3.5, corresponding to the probable flood discharges of various return periods.

Table 3.1.1 (1) Rainfall Gauging Stations Available in the Walanae River Basin

(1) No.	Station Name	(2) Belonging		Setting Year	River System
33	Matango	N	P3SA	May 1975	Walanae
34	Ujung Lamuru	N	PMG	1914	"
		A	P3SA	Mar. 1974	"
35	Bengo	N	PMG		"
		N	PHA		"
		N	DIPERTA	1971	"
36	Paciro	N	P3SA	May 1975	"
37	Turucinnae	N	P3SA	May 1975	"
38	Takalala	N	PMG	1928	"
		N	DIPERTA		"
39	Malanroe	N	DIPERTA	Sep. 1972	"
40	Watan Soppeng	N	PMG	1906	"
		N	DIPERTA		"
41	Cabenge	N	PMG	Jun. 1971	"
		A	P3SA	1974	"
46	Sero	N	P3SA	1975	"

Notes : (1) ; Rainfall station Number in Fig. 3.1.1
(2) ; Classification of automatic or normal gauge

P3SA ; Proyek Perancangan dan Pengembangan Sumber Sumber Air

PHA ; Penyelidikan Masalah Air

PMG ; Pusat Meteorologi dan Geofisika

DIPERTA ; Dinas Pertanian

Table 3.1.1 (2) Water Level Gauging Stations Available in the Walanae River Basin

(1) No.	Station Name	(2) Belonging		Setting Year	River System	Remarks
12	Ujung Lamuru	A	P3SA	Apr. 1974	Walanae	
13	Langkemme	A	PMA	Jul. 1974	"	
14	Sero	N	P3SA	Jul. 1975	"	
15	Kalempang	A	P3SA	May 1977	"	
16	Pacongkang (Mong)	N.A	P3SA	Sep. 1975	"	
17	Lakibong	N	P3SA	Aug. 1970	"	
18	Cabenge	A	DPMA	Oct. 1974	"	

Notes : (1) ; Water level station Number in Fig. 3.1.1

(2) ; Classification of automatic or normal gauge

(3) ; Pacongkang is automatic gauging station after may 1978, and the name of station was changed to Mong

P3SA ; Proyek Perancangan dan Pengembangan Sumber Sumber Air

PMA ; Penyelidikan Masalah Air

DPMA ; Direktorat Penyelidikan Masalah Air

PMG ; Pusat Meteorologi dan Geofisika

DIPERTA ; Dinas Pertanian

Table 3.1.2 (1) Available Rainfall Records in the Walanae River Basin

STATION \ YEAR (19..)	17	18	19	20	21	22	23	24	25	26	27	28	29	30	31	32	33	34	35	36	37	38	39	40	41	42	43	44	45	46	47	48
33 MATANGO																																
34 UJUNG LAMURU	O	O	O	O	O	O	O	O	O	O	O	O	O	O	O	O	O	O	O	O	O	O	O	O	O							
35 BENGO									O	O	O	O																				
36 PACIRO																																
37 TURUCINNAE																																
38 TAKALALA																																
39 MALLANROE																																
40 WATAN SOPPENG	O	O	O	O	O	O	O	O	O	O	O	O	O	O	O	O	O	O	O	O	O	O	O	O	O							
41 CABENGE																																
46 SERO																																

STATION \ YEAR (19..)	49	50	51	52	53	54	55	56	57	58	59	60	61	62	63	64	65	66	67	68	69	70	71	72	73	74	75	76	77	78	79	80
33 MATANGO																											⊙	⊙	⊙	⊙	⊙	⊙
34 UJUNG LAMURU																										⊙	⊙	⊙	⊙	⊙	⊙	⊙
35 BENGO																							⊙	⊙	⊙	⊙	⊙	⊙	⊙	⊙	⊙	⊙
36 PACIRO																											⊙	⊙	⊙	⊙	⊙	⊙
37 TURUCINNAE																											⊙	⊙	⊙	⊙	⊙	⊙
38 TAKALALA			O	O	O	O	O		O	O	O	O	O	O	O	O	O	O	O	O	O	O	⊙	⊙	⊙	⊙	⊙	⊙	⊙	⊙	⊙	⊙
39 MALLANROE																												⊙	⊙	⊙	⊙	⊙
40 WATAN SOPPENG				O	O	O	O		O	O	O	O	O	O	⊙	⊙	⊙	⊙	⊙	⊙	⊙	⊙	⊙	⊙	⊙	⊙	⊙	⊙	⊙	⊙	⊙	⊙
41 CABENGE																								O	O	⊙	⊙	⊙	⊙	⊙	⊙	⊙
46 SERO																													⊙	⊙	⊙	⊙

REMARKS :

O Monthly Rainfall Data Available

• Daily Rainfall Data Available

Table 3.1.2 (2) Available Water Level Records in the Walanae River System

☆	YEAR	1973												1974												1975												1976											
	STATION NAME	1	2	3	4	5	6	7	8	9	10	11	12	1	2	3	4	5	6	7	8	9	10	11	12	1	2	3	4	5	6	7	8	9	10	11	12	1	2	3	4	5	6	7	8	9	10	11	12
12	UJUNG LAMURU	/	/	/	/	/	/	/	/	/	/	/	/	/	/	/	△	O	O	O	O	O	O	O	O	△	/	△	O	O	O	O	△	O	O	O	/	/	/	O	O	△	O	O	O	O	O	O	O
13	LANGKEMME	/	/	/	/	/	/	/	/	/	/	/	/	/	/	/	/	△	O	O	O	O	O	△	O	O	O	O	△	/	△	△	O	O	O	O	O	O	O	O	O	O	O	O	O	O	O	O	O
14	SERO	/	/	/	/	/	/	/	/	/	/	/	/	/	/	/	/	/	/	/	/	/	/	/	/	/	/	/	/	/	/	△	O	O	O	O	O	O	O	O	O	O	O	O	O	O	O	O	O
15	KALEMPANG	/	/	/	/	/	/	/	/	/	/	/	/	/	/	/	/	/	/	/	/	/	/	/	/	/	/	/	/	/	/	/	/	/	/	/	/	/	/	/	/	/	/	/	/	/	/	/	/
16	PACONGKANG	/	/	/	/	/	/	/	/	/	/	/	/	/	/	/	/	/	/	/	/	/	/	/	/	/	/	/	/	/	/	/	/	△	O	O	△	O	O	O	O	O	O	△	O	O	O	O	O
17	LAKIBONG	/	/	/	/	/	/	/	/	/	/	/	/	/	/	/	/	/	/	/	/	/	/	/	/	/	/	/	/	/	/	/	/	△	△	/	/	/	△	O	O	O	O	O	O	O	O	O	O
18	CABENGE	/	/	/	/	/	/	/	/	/	/	/	/	/	/	/	/	/	/	/	/	/	/	△	O	O	O	O	O	O	O	O	O	O	O	△	/	/	/	/	/	/	/	/	/	/	/	△	O

☆	YEAR	1977												1978												1979												1980											
	STATION NAME	1	2	3	4	5	6	7	8	9	10	11	12	1	2	3	4	5	6	7	8	9	10	11	12	1	2	3	4	5	6	7	8	9	10	11	12	1	2	3	4	5	6	7	8	9	10	11	12
12	UJUNG LAMURU	O	O	O	O	O	△	O	O	O	O	/	O	O	/	O	△	O	O	O	O	O	O	O	O	O	O	O	O	O	O	O	O	O	O	O	O	*											
13	LANGKEMME	O	O	O	O	O	O	△	O	O	O	O	O	△	O	O	O	△	O	O	O	O	O	△	/	O	O	O	O	O	O	O	O	O	/	/	/	*											
14	SERO	O	O	O	O	O	O	O	O	O	O	O	O	O	O	O	O	O	O	O	O	O	O	O	O	O	O	O	O	O	O	O	O	O	O	O	O	*											
15	KALEMPANG	/	/	/	/	/	/	/	/	/	/	/	/	△	△	△		△	O	O	O	O	O	O	O	O	O	O	O	O	O	O	O	O	O	O	O	*											
16	PACONGKANG	O	O	O	O	O	O	O	O	△	△	O	△	△	O	O	O	O	O	O	O	O	O	O	O	O	O	O	O	O	O	O	O	O	O	O	O	*											
17	LAKIBONG	△	O	O	O	O	△	O	O	O	O	O	O	O	O	O	O	O	O	O	O	O	O						out of order																				
18	CABENGE	O	O	O	O	O	O	O	O	O	O	△	/	O	O	O	O	O	O	O	O	O	O	O	O	O	O	O	O	O	O	O	O	O	O	O	O	*											

REMARKS:

- ☆ Water Level Gauging Station Number
- / No Data (No Information or Gauge Trouble)
- △ Data Partialy Available
- O Data Available
- * Seems To Be Available But Not Formally Arranged Yet

Table 3.2.1 (1) Mean 10 - Day Discharge (at Langkemme Intake Site)

Month \ Year / Period	1975 Early	1975 Middle	1975 Late	1976 Early	1976 Middle	1976 Late	1977 Early	1977 Middle	1977 Late	1978 Early	1978 Middle	1978 Late	1979 Early	1979 Middle	1979 Late	Max.	Min.	Average
Jan.	3.98	6.88	1.72	1.42	12.90	1.18	12.24	9.42	19.25	3.21	4.47	5.41	13.18	10.69	3.96	19.25	1.18	7.33
Feb.	3.08	5.58	3.47	2.02	1.10	1.61	4.69	26.21	17.59	2.02	4.00	3.31	2.25	3.81	8.76	26.21	1.10	5.92
Mar.	4.54	3.24	3.17	1.07	1.92	2.23	9.18	4.62	6.40	3.67	4.34	4.28	6.59	2.31	3.59	9.18	1.07	4.08
Apr.	3.75	4.19	[3.91]	2.41	2.36	1.20	9.77	9.13	3.14	3.50	2.35	2.59	4.81	2.57	2.98	9.77	1.20	3.91
May	[3.70]	[3.70]	[3.70]	3.79	1.12	1.20	2.55	6.78	4.46	4.01	5.42	2.89	7.48	2.16	2.44	7.48	1.12	3.70
Jun.	[4.04]	[4.04]	1.07	2.78	2.45	2.17	3.79	13.38	2.57	3.83	3.35	3.04	4.52	6.12	3.45	13.38	1.07	4.04
Jul.	1.16	1.11	3.81	4.88	2.45	1.54	5.22	3.24	1.93	4.28	3.78	3.01	2.34	2.66	1.87	5.22	1.11	2.88
Aug.	1.31	4.13	1.37	1.33	1.74	1.39	2.01	2.24	1.53	2.39	1.58	1.72	2.00	1.40	1.26	4.13	1.26	1.83
Sep.	1.73	1.54	1.46	0.97	0.96	1.33	1.39	1.22	1.82	3.78	1.72	3.55	1.27	1.27	2.09	3.78	0.96	1.74
Oct.	1.47	1.13	1.20	2.18	2.38	1.75	2.58	1.49	1.29	5.01	2.15	1.87	(2.20)	(1.08)	(1.14)	5.01	1.13	2.04
Nov.	1.34	1.44	1.29	1.36	2.48	1.63	1.25	1.31	2.21	2.63	4.28	(9.46)	(1.31)	(1.28)	(2.74)	4.28	1.25	1.93
Dec.	4.11	8.94	1.80	4.23	5.81	2.16	2.96	6.43	6.11	(7.98)	(8.76)	(12.39)	(1.52)	(8.59)	(8.37)	8.94	1.80	4.73
Max.		8.94			12.9			26.21			5.42			13.18		26.21		
Min.		1.07			0.96			1.22			1.58			1.26		0.96		
Average		2.83			2.38			5.87			3.36			3.59				

( ) : Presumed from R.Sero

[ ] : Substituted from average

Discharge in m^3/s

Average : Excluding ( ) and [ ]

Table 3.2.1 (2) Mean 10 - Day Discharge (at the Confluence of Jupang and Pising Rivers)

Year / Period / Month	1975 Early	1975 Middle	1975 Late	1976 Early	1976 Middle	1976 Late	1977 Early	1977 Middle	1977 Late	1978 Early	1978 Middle	1978 Late	1979 Early	1979 Middle	1979 Late	Max.	Min.	Average
Jan.	(3.58)	(6.32)	(1.46)	9.96	79.73	9.29	24.26	23.14	52.48	14.54	7.79	14.10	38.27	28.22	10.40	79.73	1.46	21.57
Feb.	(2.74)	(5.10)	(3.10)	49.92	7.22	11.07	15.52	102.06	63.02	9.02	8.48	7.10	5.20	9.70	19.55	102.26	2.74	21.25
Mar.	(4.11)	(2.90)	(2.82)	6.24	29.88	11.35	34.02	8.75	9.16	5.33	4.84	4.98	17.26	9.71	14.11	34.02	2.82	11.03
Apr.	(3.37)	(3.78)	[7.74]	7.89	3.29	5.18	7.39	20.03	6.77	5.17	4.75	6.78	14.12	5.92	5.53	20.03	3.29	7.18
May	[8.90]	[8.90]	[8.90]	53.53	3.40	1.43	1.45	0.83	0.58	6.29	8.45	2.65	17.75	6.60	3.80	53.53	0.58	8.90
Jun.	[19.29]	[19.29]	0.85	20.53	4.18	4.36	10.60	56.71	6.31	10.70	8.26	4.47	12.98	16.86	7.33	56.71	0.85	13.51
Jul.	0.94	1.84	22.23	18.66	7.25	1.34	1.48	0.88	0.86	8.82	8.03	6.92	2.67	8.68	2.98	22.23	0.86	6.24
Aug.	2.92	14.81	2.40	2.05	1.10	1.07	0.75	0.74	0.57	10.29	3.88	2.20	1.90	1.44	1.32	14.81	0.57	3.16
Sep.	5.70	5.35	1.39	1.05	0.66	0.66	0.50	0.46	0.42	2.79	1.59	1.84	3.55	1.50	1.27	5.70	0.42	1.92
Oct.	2.57	2.70	4.47	0.66	0.66	1.66	0.42	0.42	0.42	5.62	1.67	1.77	1.90	0.86	0.91	5.62	0.42	1.78
Nov.	16.13	6.23	13.12	1.10	2.76	2.89	0.42	0.42	0.71	3.79	7.73	11.10	1.07	1.05	2.42	16.13	0.42	4.73
Dec.	11.58	18.38	11.40	4.09	27.55	9.06	7.90	12.50	17.77	8.06	9.66	17.14	1.27	8.51	8.10	27.55	1.27	11.53
Max.		22.2			79.73			102.06			17.14			38.27		102.06	-	-
Min.		0.85			0.66			0.42			1.59			0.91		-	0.42	-
Average		7.15			11.19			13.63			6.85			8.19			-	9.40

( ) : Presumed from R.Langkemme

[ ] : Substituted from average value

Discharge in m^3/s

Average : Excluding ( ) and [ ]

Table 3.2.1 (3) Mean 10 - Day Discharge of Tributaries

Year / Period / Month	1975			1976			1977			1978			1979			Max.	Min.	Average
	Early	Middle	Late	Early	Middle	Late	Early	Middle	Late	Early	Middle	Late	Early	Middle	Late			
Jan.	4.15	7.17	1.79	1.48	13.45	1.23	12.76	9.82	20.07	3.35	4.66	5.64	13.74	11.15	4.13	20.07	1.23	
Feb.	3.21	5.82	3.62	2.11	1.15	1.68	4.89	27.33	18.34	2.11	4.17	3.45	2.35	3.32	9.13	27.33	1.15	
Mar.	4.73	3.38	3.31	1.12	2.00	2.32	9.57	4.82	6.67	3.83	4.52	4.46	6.87	2.41	3.74	9.57	1.12	
Apr.	3.91	4.37	[4.08]	2.51	2.46	1.25	10.19	9.52	3.27	3.65	2.45	2.70	5.01	2.68	3.11	10.19	1.25	
May	[3.86]	[3.86]	[3.86]	3.95	1.17	1.25	2.66	7.07	4.65	4.18	5.65	3.01	7.80	2.25	2.54	7.80	1.17	
Jun.	[4.21]	[4.21]	1.12	2.88	2.55	2.26	3.95	13.95	2.68	3.99	3.49	3.17	4.71	6.38	3.60	13.95	1.12	
Jul.	1.21	1.16	3.97	5.09	2.55	1.61	5.44	3.38	2.01	4.46	3.94	3.14	2.44	2.77	1.95	5.44	1.16	
Aug.	1.37	4.31	1.43	1.39	1.81	1.45	2.10	2.34	1.60	2.49	1.65	1.79	2.09	1.46	1.31	4.31	1.31	
Sep.	0.67	0.59	0.56	0.37	0.37	0.51	0.54	0.47	0.70	1.46	0.66	1.37	0.49	0.49	0.81	1.46	0.37	
Oct.	0.62	0.48	0.51	0.92	1.00	0.74	1.09	0.63	0.54	2.11	0.91	0.79	(0.93)	(0.45)	(0.48)	2.11	0.48	
Nov.	1.40	1.50	1.34	1.42	2.59	1.70	1.30	1.37	2.30	2.74	4.46	(9.86)	(1.37)	(1.33)	(2.86)	4.46	1.30	
Dec.	4.29	9.32	1.88	4.41	6.06	2.25	3.09	6.70	6.37	(8.32)	(9.13)	(12.92)	(1.58)	(8.96)	(8.73)	9.32	1.88	
Max.		9.32			13.45			27.33			5.65			13.74				
Min.		0.48			0.37			0.47			0.66			(0.45) 0.49				
Average		2.77			2.31			5.95			3.14			3.88				

( ) : Presumed from R. Sero

[ ] : Quotation from average

Discharge in m^3/s

Average : Excluding ( ) and [ ]

Table 3.2.2 List of Design Lowest Monthly Discharge in South Sulawesi

Name of Project	Name of River	Catchment Area A (km^2)	Discharge Q (m^3/s)	Specific Discharge q ($m^3/s/100$ km)
Langkemme	Langkemme R.	104	1.48	1.42
"	Sero R	335	0.57	0.17
Luwu	Rongkong R.	1,030	49.0	4.76
"	Baebunta R.	40	1.6	4.00
"	Radda R.	40	1.7	4.25
"	Masamba R.	105	4.4	4.19
"	Balease R.	855	43.0	5.03
Sadang	Penali R.	790	4.5	0.57
"	Gung R.	120	2.4	2.00
"	Tjomal R.	4	0.26	6.50
"	Mamasa R.	1,215	22.5	1.85
"	Sadang R.	5,875	155.0	2.64

Table 3.3.1 Characteristics of River Basin at Intake Sites and Cossing Points of Major Canal

No.	River Name	Catchment Area A (km^2)	River Length l (m)	Head h (m)	Slope i (l/h)
1.	R.Belo	39	15,000	786	1/19
2.	R.Maccope	12	8,150	747	1/11
3.	R.Panincong	14	12,250	875	1/14
4.	R.Congkai	13	7,250	647	1/11
5.	R. Labempa	18	10,750	820	1/13
6.	R.Baruttungnge	11	5,750	501	1/11
7.	R.Maddenra	14	11,250	800	1/14
8.	R.Langkemme	104	20,000	1,370	1/15
9.	R.Pising	37	9,500	816	1/12
10.	R.Unyi	32	8,500	1,002	1/8
11.	R.Jupang	237	31,000	1,386	1/22

Table 3.3.2 Concentration Time

No.	River Name	Slope i (h/l)	Velocity of Arrival V (m/s)	River Length l (m)	Concentration Time: Stream flow T2 (hr)	Concentration Time: Inflow T1 (hr)	Concentration Time: Tc=T1+T2 (hr)
1.	R. Belo	1/19	3.5	15,000	1.2	0.5	1.7
2.	R. Maccope	1/11	3.5	8,150	0.6	0.5	1.1
3.	R. Panincong	1/14	3.5	12,250	1.0	0.5	1.5
4.	R. Congkai	1/11	3.5	7,250	0.6	0.5	1.1
5.	R. Labempa	1/13	3.5	10,750	0.9	0.5	1.4
6.	R. Baruttungnge	1/11	3.5	5,750	0.5	0.5	1.0
7.	R. Maddenra	1/14	3.5	11,250	0.9	0.5	1.4
8.	R. Langkemme	1/15	3.5	20,000	1.6	0.5	2.1
9.	R. Pising	1/12	3.5	9,500	0.8	0.5	1.3
10.	R. Unyi	1/8	3.5	8,500	0.7	0.5	1.2
11.	R. Jupang	1/22	3.5	31,000	2.5	0.5	3.0

Table 3.3.3 Average Rainfall Intensity within Concentration Time of Flood

No.	River Name	Concentration Time Tc (hr)	Return Period in Years			
			r1/10	r1/30	r1/50	r1/100
1.	R. Belo	1.7	40.9	51.8	56.5	63.3
2.	R. Maccope	1.1	54.7	69.3	75.5	84.6
3.	R. Panincong	1.5	44.4	56.4	61.4	68.7
4.	R. Congkai	1.1	54.7	69.3	75.5	84.6
5.	R. Labempa	1.4	46.5	59.0	64.3	72.0
6.	R. Baruttungnge	1.0	58.2	73.8	80.4	90.1
7.	R. Maddenra	1.4	46.5	59.0	64.3	72.0
8.	R. Langkemme	2.1	35.5	45.0	49.1	55.0
9.	R. Pising	1.3	48.9	62.0	67.5	75.7
10.	R. Unyi	1.2	51.6	65.4	71.2	79.8
11.	R. Jupang	3.0	28.0	35.5	38.7	43.3
	Probable Daily Rainfall (Watan Soppeng)		R24= 168mm	R24= 213mm	R24= 232mm	R24= 260mm

Note : Mononobe's formula $Ir = \frac{R24}{24} \left(\frac{24}{T}\right)^{2/3}$

Table 3.3.4 Probable Peak Flood Discharge

Unit : m^3/s

No.	Name of River	Rainfall Area A (km^2)	Return Period in Year			
			10	30	50	100
1.	R. Belo	39	245.4 (6.3)	336.7 (8.6)	367.3 (9.4)	411.5 (10.6)
2.	R. Maccopc	12	109.4 (9.1)	138.6 (11.6)	151.0 (12.6)	169.2 (14.1)
3.	R. Panincong	14	103.6 (7.4)	131.6 (9.4)	143.3 (10.2)	160.3 (11.5)
4.	R. Congkai	13	118.5 (9.1)	150.2 (11.6)	163.6 (12.6)	183.3 (14.1)
5.	R. Labempa	18	139.5 (7.8)	177.0 (9.8)	192.9 (10.7)	216.0 (12.0)
6.	R. Baruttungnge	11	106.7 (9.7)	135.3 (12.3)	147.4 (13.4)	165.2 (15.0)
7.	R. Madenra	14	108.5 (7.8)	137.7 (9.8)	150.0 (10.7)	168.0 (12.0)
8.	R. Langkemme	85	502.9 (4.8)	637.5 (6.1)	695.6 (6.7)	779.2 (7.5)
9.	R. Pising	37	301.6 (8.2)	382.3 (10.3)	416.3 (11.3)	466.8 (12.6)
10.	R. Unyi	32	275.2 (8.6)	348.8 (10.9)	379.7 (11.9)	425.6 (13.3)
11.	R. Jupang	175	816.7 (3.4)	1,035.4 (4.4)	1,128.8 (4.8)	1,262.9 (5.3)

Note : ( ) Specific Discharge - q $m^3/s/km^2$

Table 3.3.5 Flood Water Level

Name of River	Return Period Years	Discharge Q m^3/s	Water Level H (m)
Langkemme R.	1/100	779.2	172.1
	1/50	695.6	171.9
	1/30	637.5	171.8
	1/10	502.9	171.4
Jupang R.	1/100	1,262.9	181.2
	1/50	1,128.8	180.9
	1/30	1,035.4	180.7
	1/10	816.7	180.2
Pising R.	1/100	466.8	173.7
	1/50	416.3	173.5
	1/30	382.3	173.4
	1/10	301.6	173.1

* Roughness coefficient n = 0.040

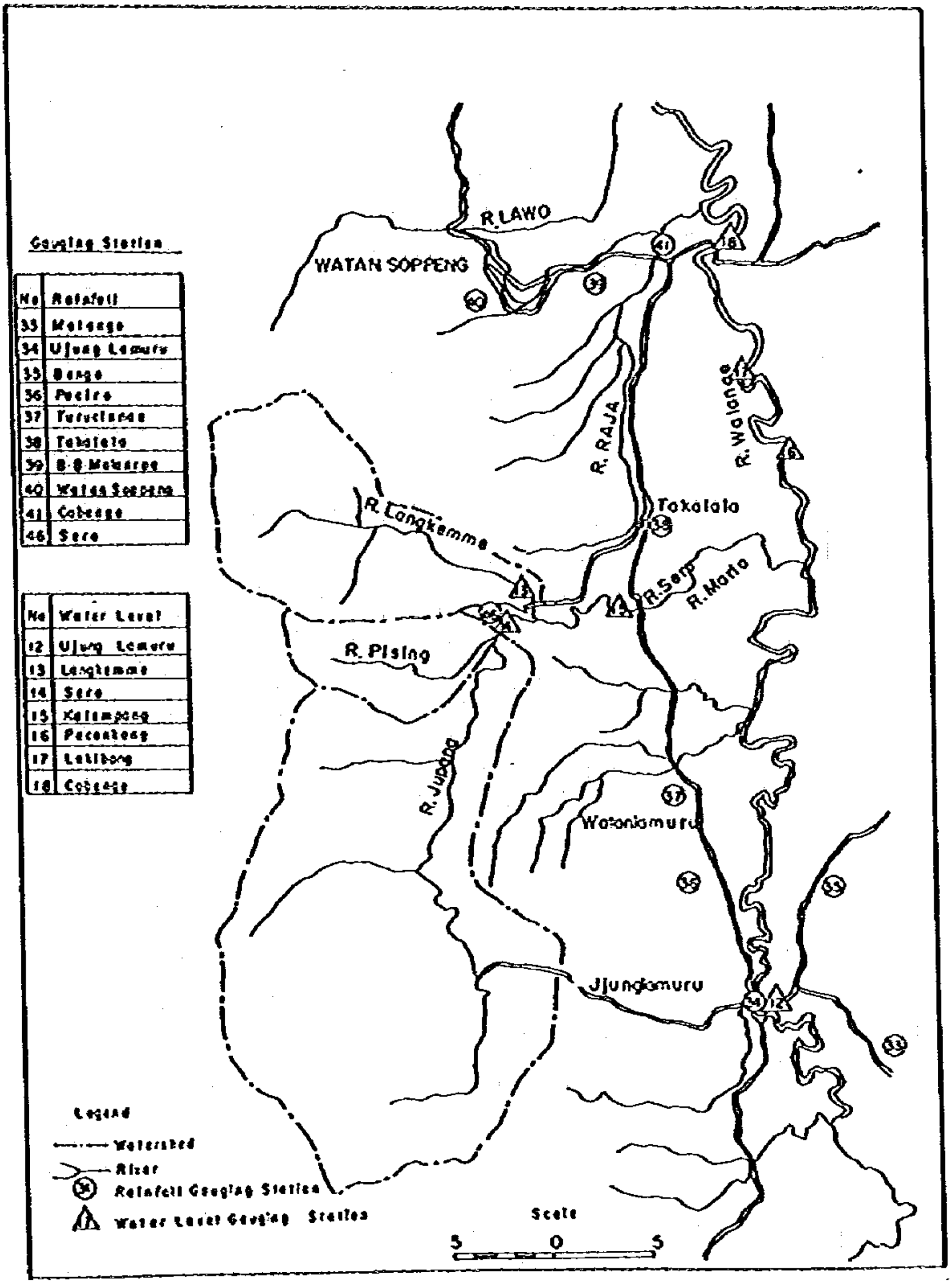


Fig. 3.1.1 NETWORKS OF METEO-HYDROLOGICAL STATIONS

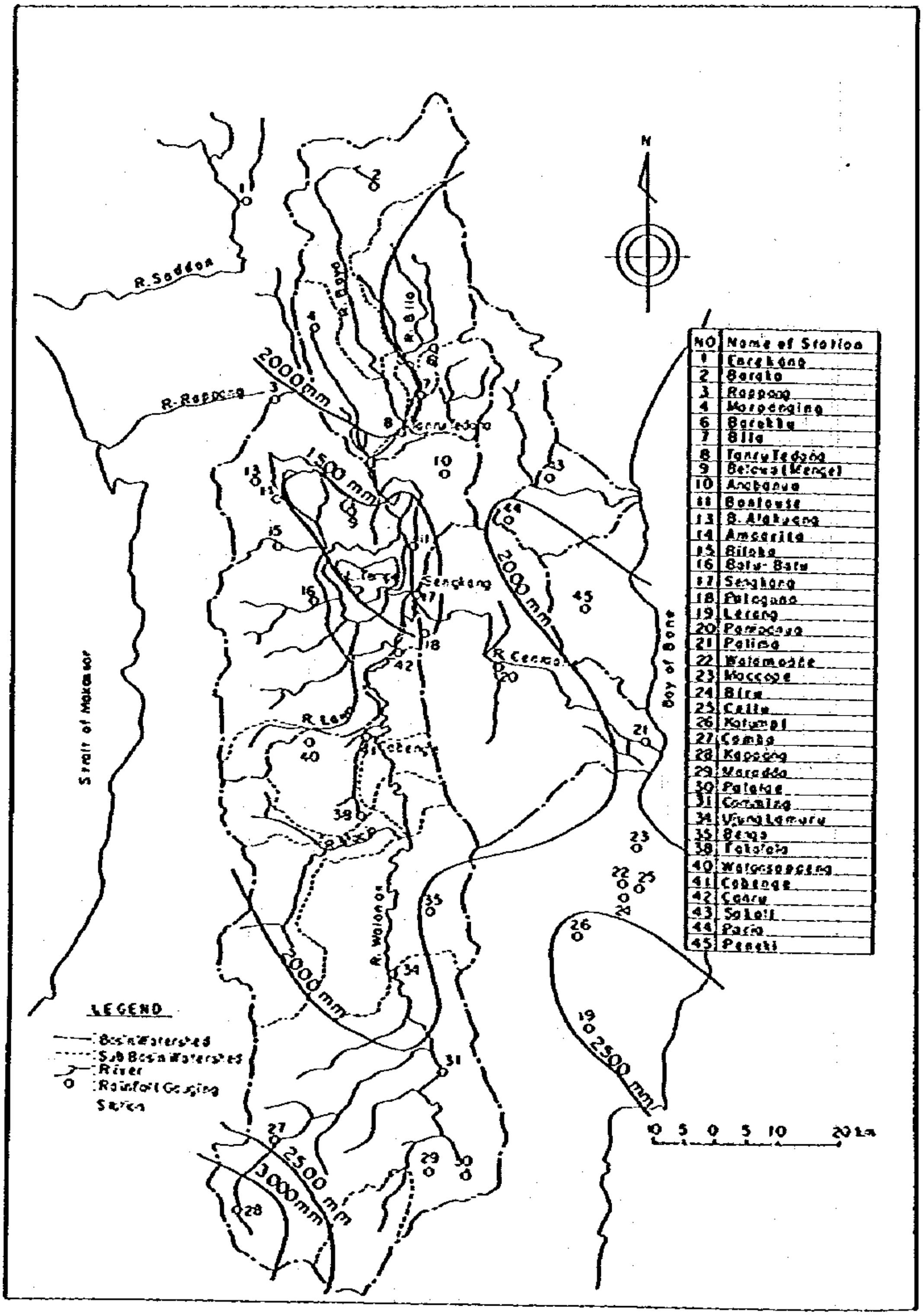


Fig. 3.2.1 ISOHYETAL MAP IN SOUTH SULAWESI

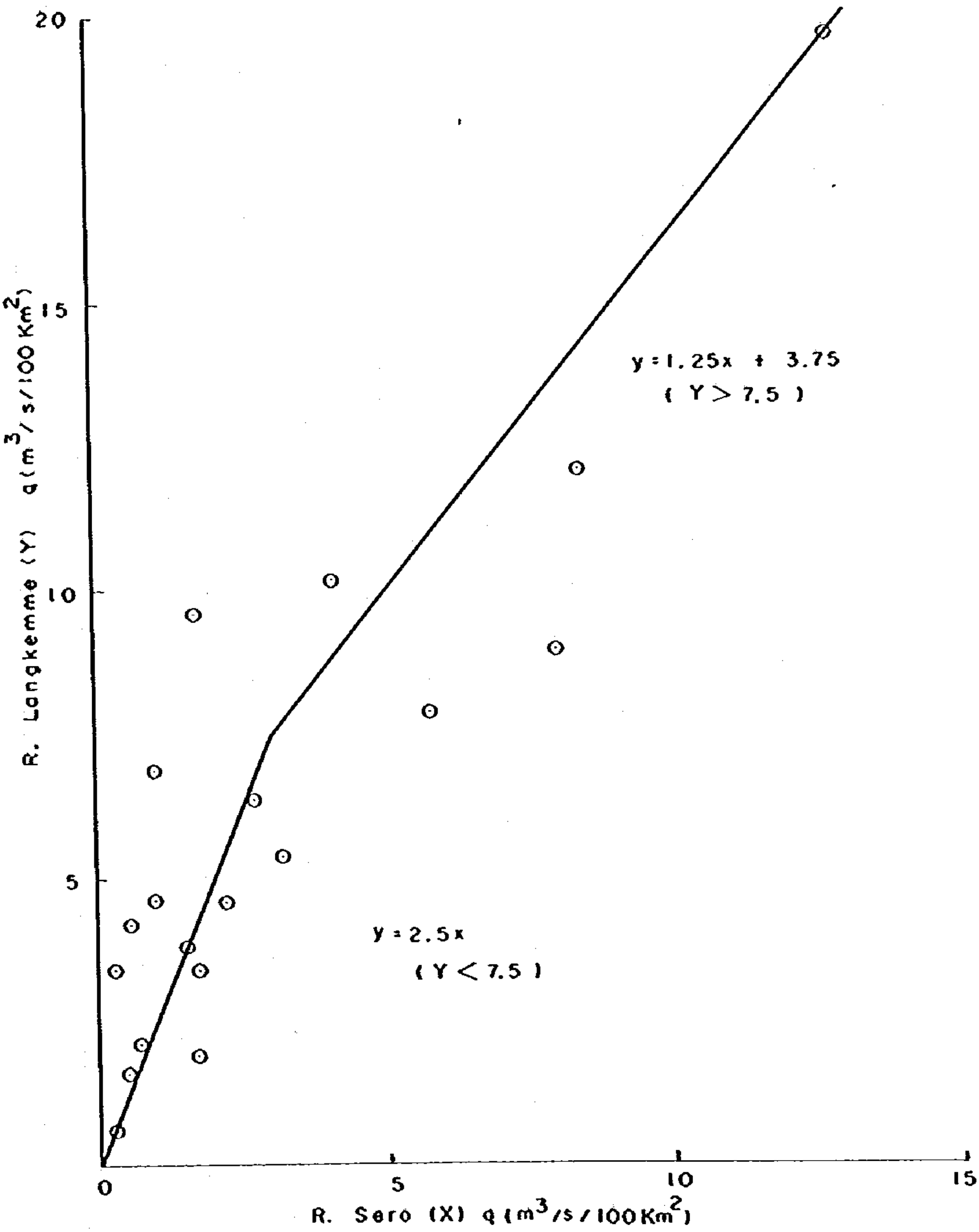


Fig. 3.2.2 CORRELATION BETWEEN SPECIFIC DISCHARGES OF R.LANGKEMME AND R.SERO

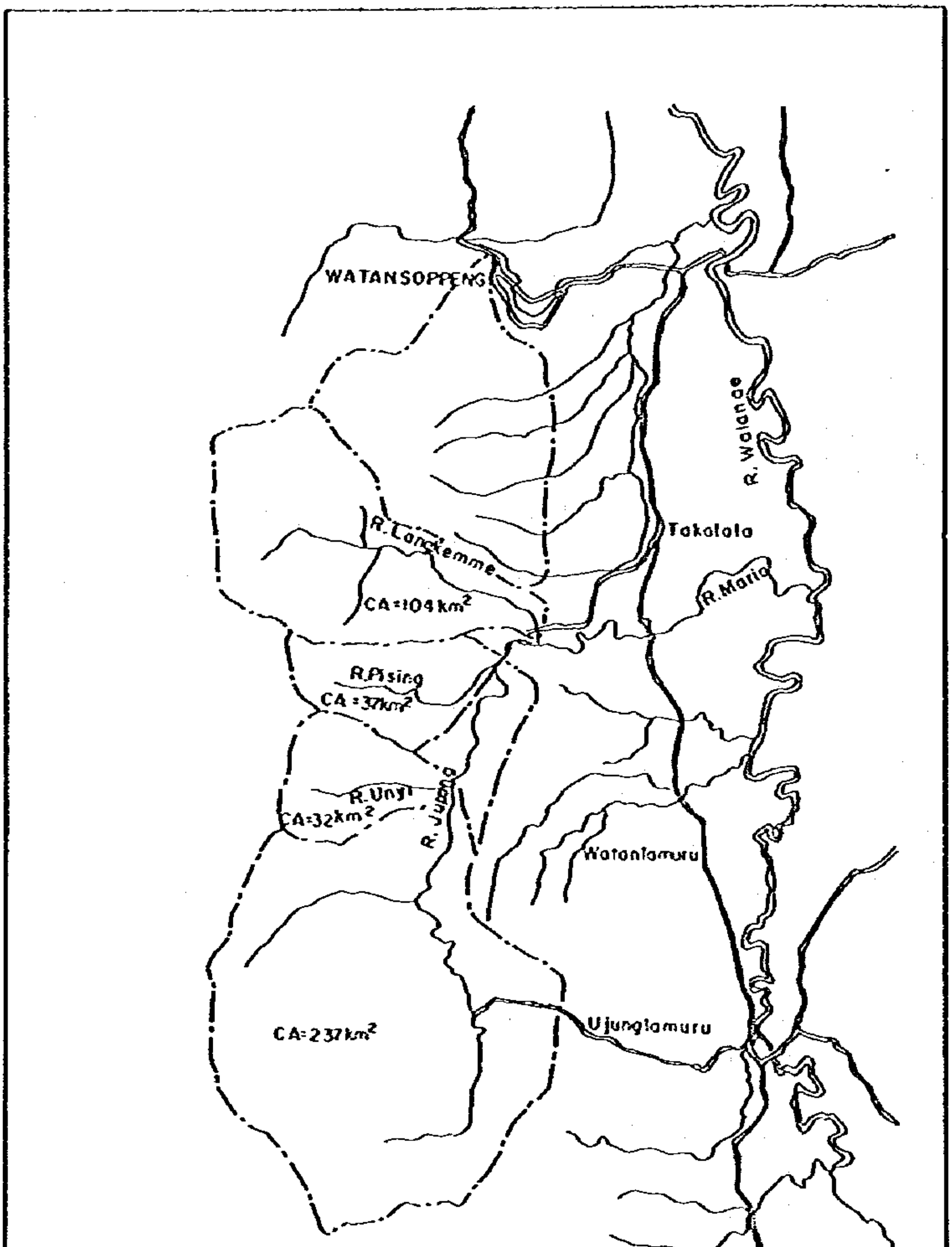


Fig. 3.2.3(1) BASIN MAP OF MAIN RIVERS

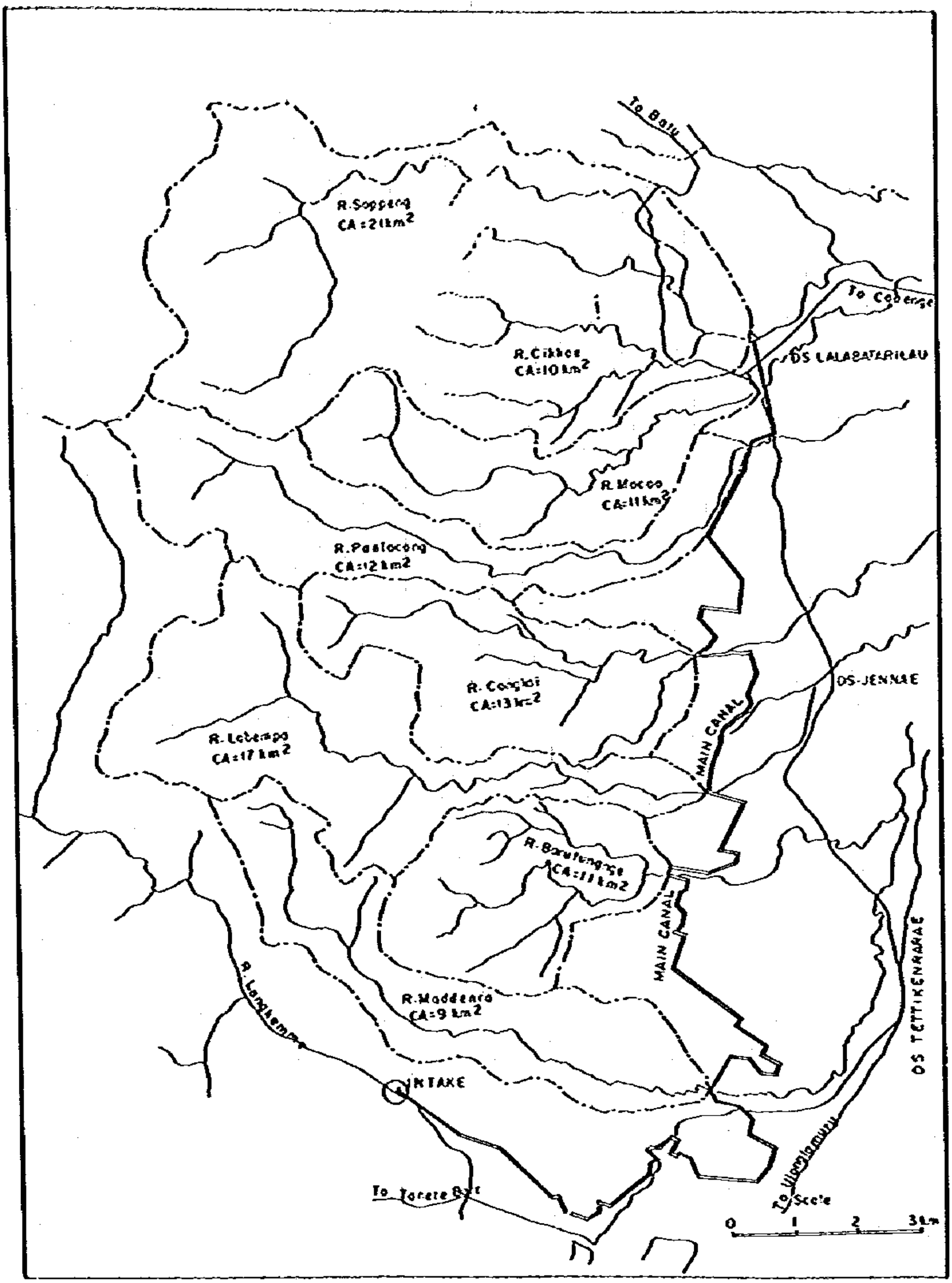


Fig. 3.2.3(2) BASIN MAP OF TRIBUTARIES

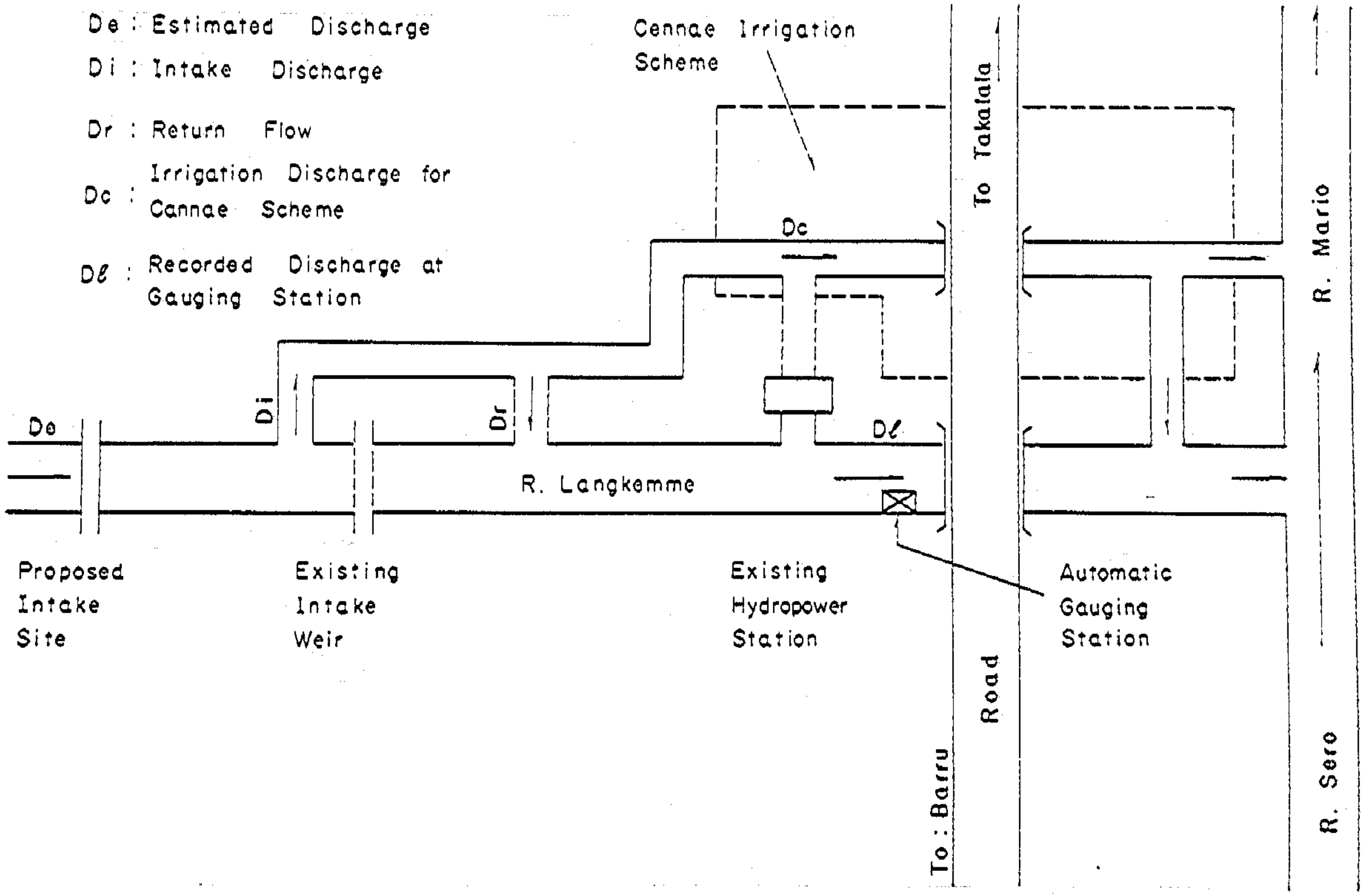


Fig. 3.2.4 SCHEMATIC PLAN IN AND AROUND THE CENNAE PROJECT

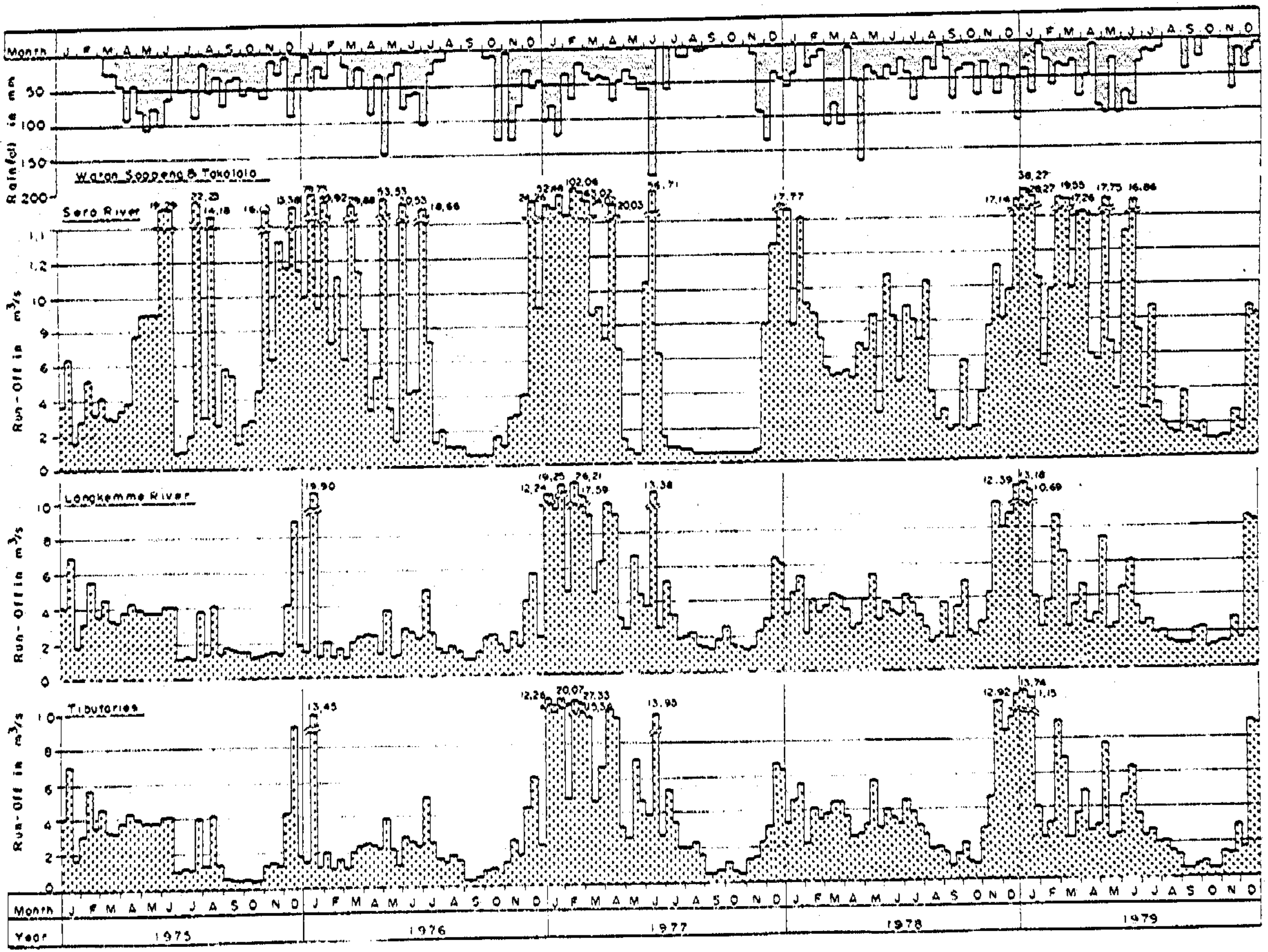


Fig.3.2.5 ANNUAL RUN-OFF BY 10-DAY BASIS

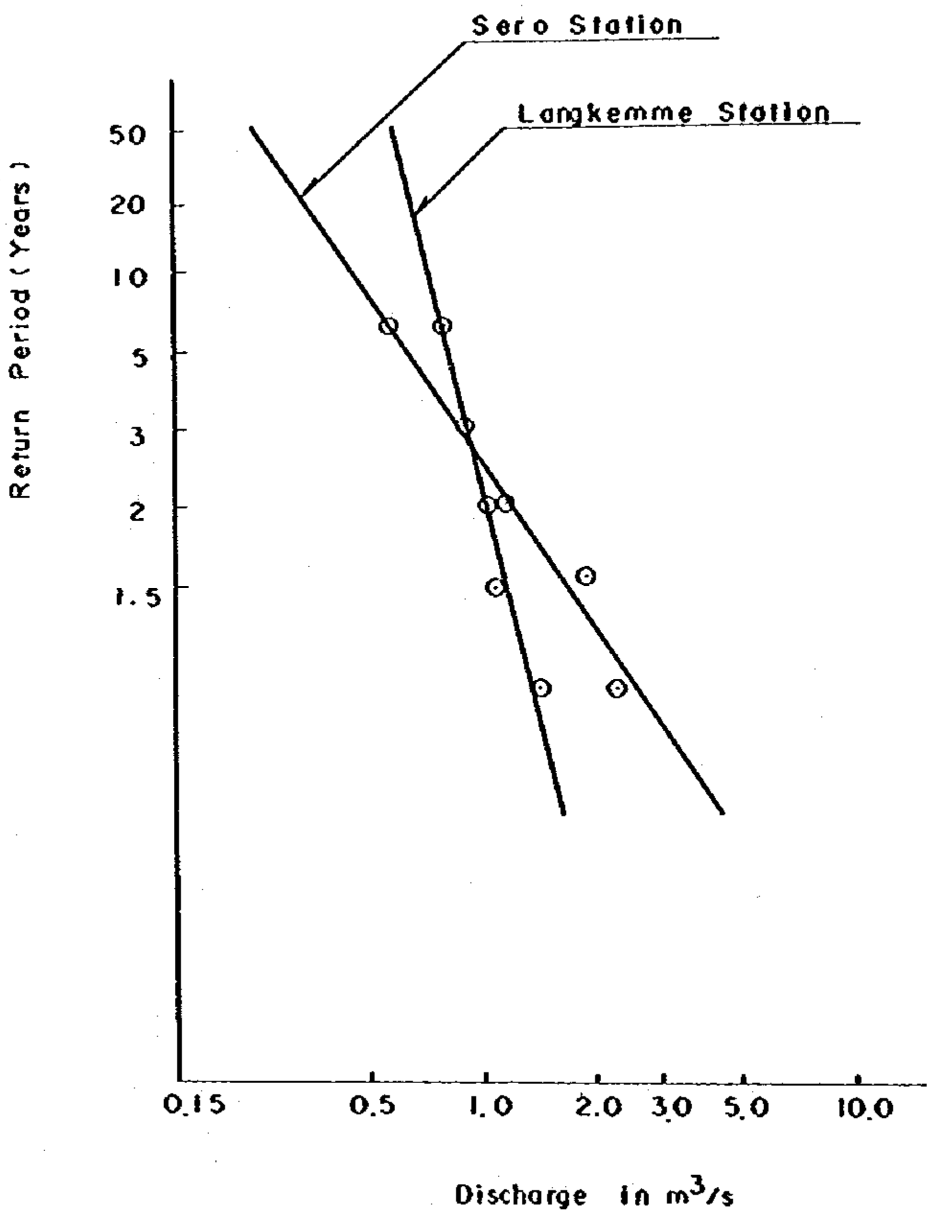


Fig. 3.2.6 PROBABLE 10-DAY DROUTHY DISCHARGES AT LANGKEMME STATION AND SERO STATION

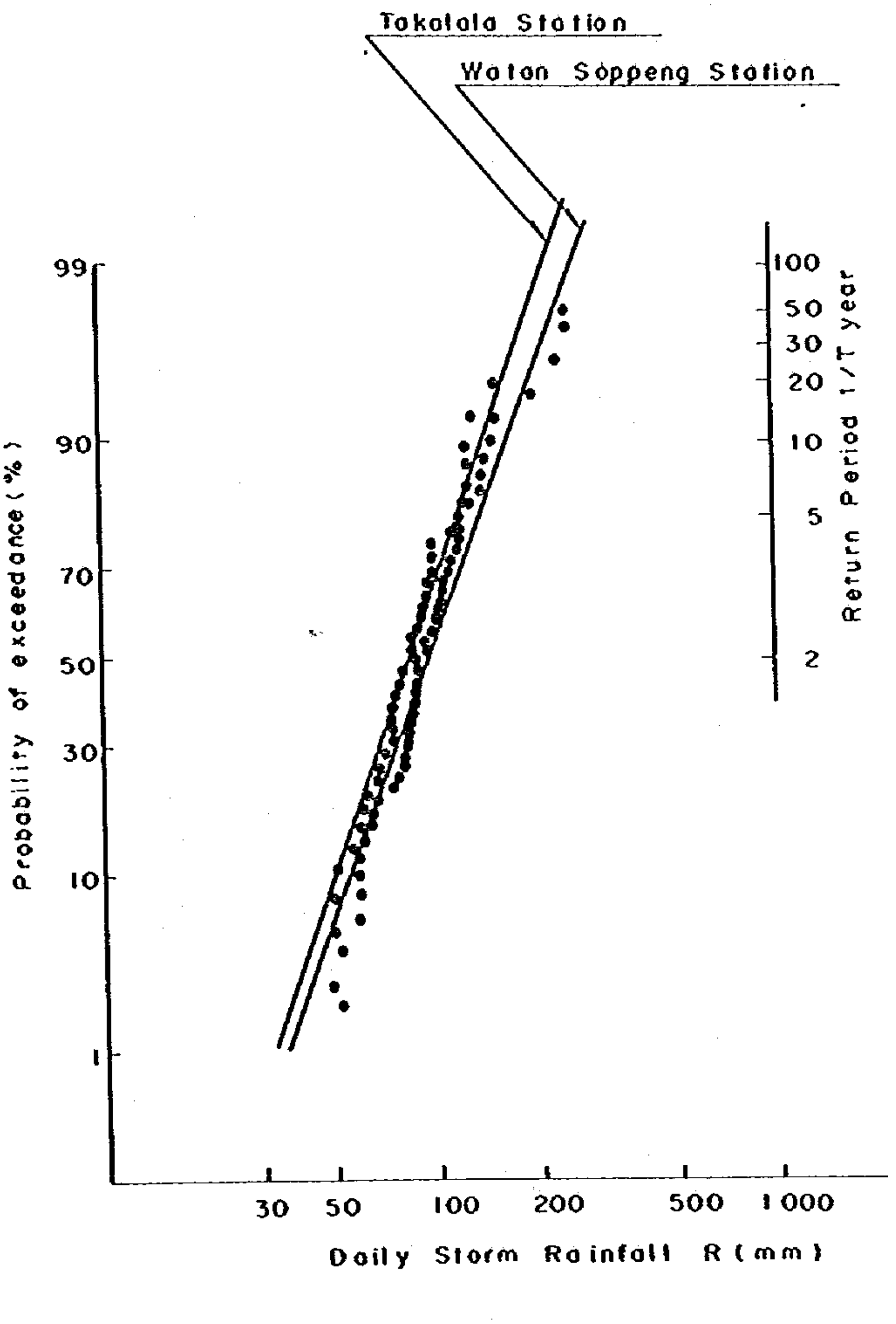


Fig. 3.3.1 PROBABLE DAILY STORM RAINFALLS AT WATAN SOPPENG STATION AND TAKALALA STATION

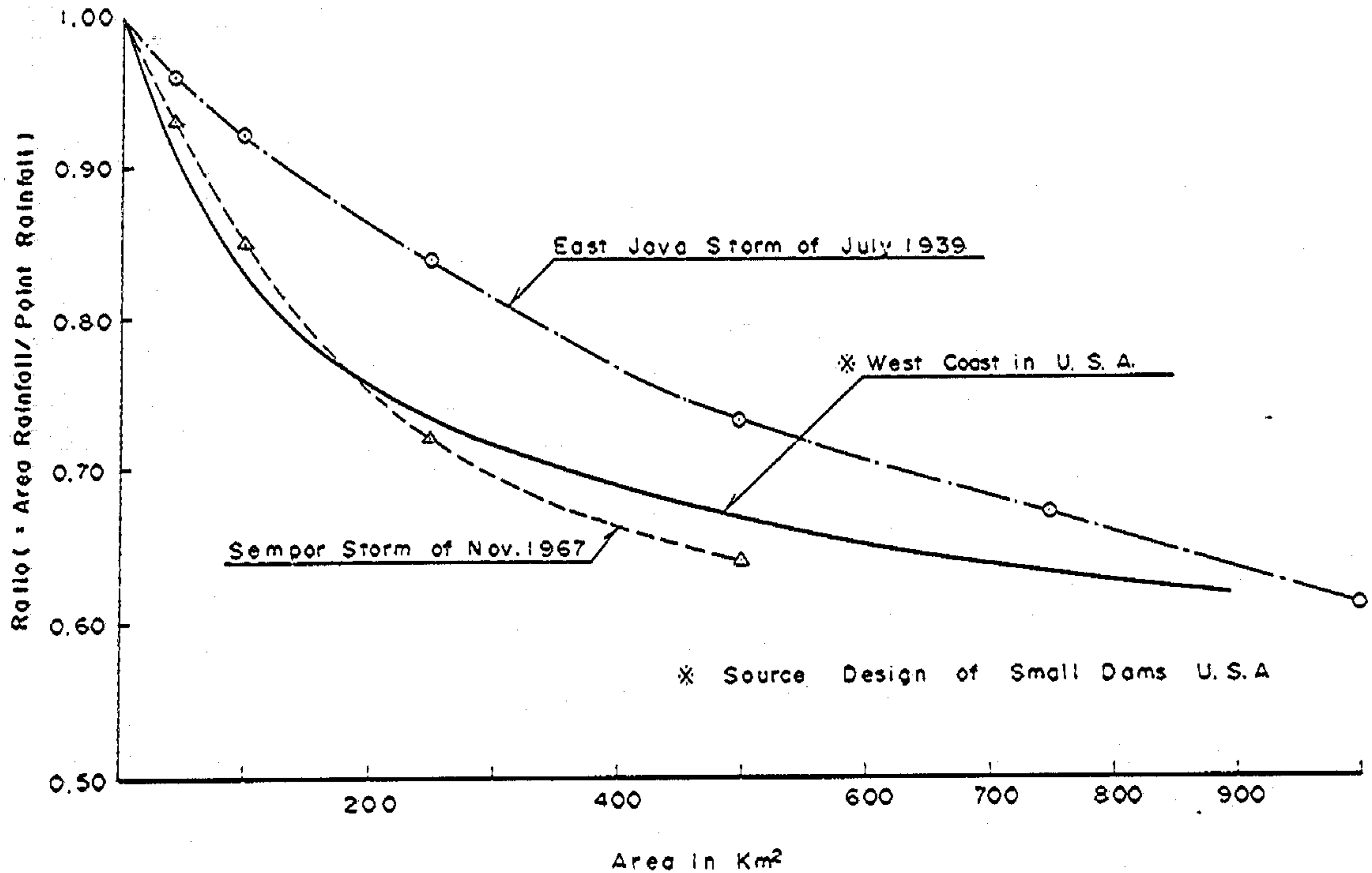


Fig. 3.3.2 CONVERSION RATIO FROM POINT RAINFALL INTO BASIN RAINFALL

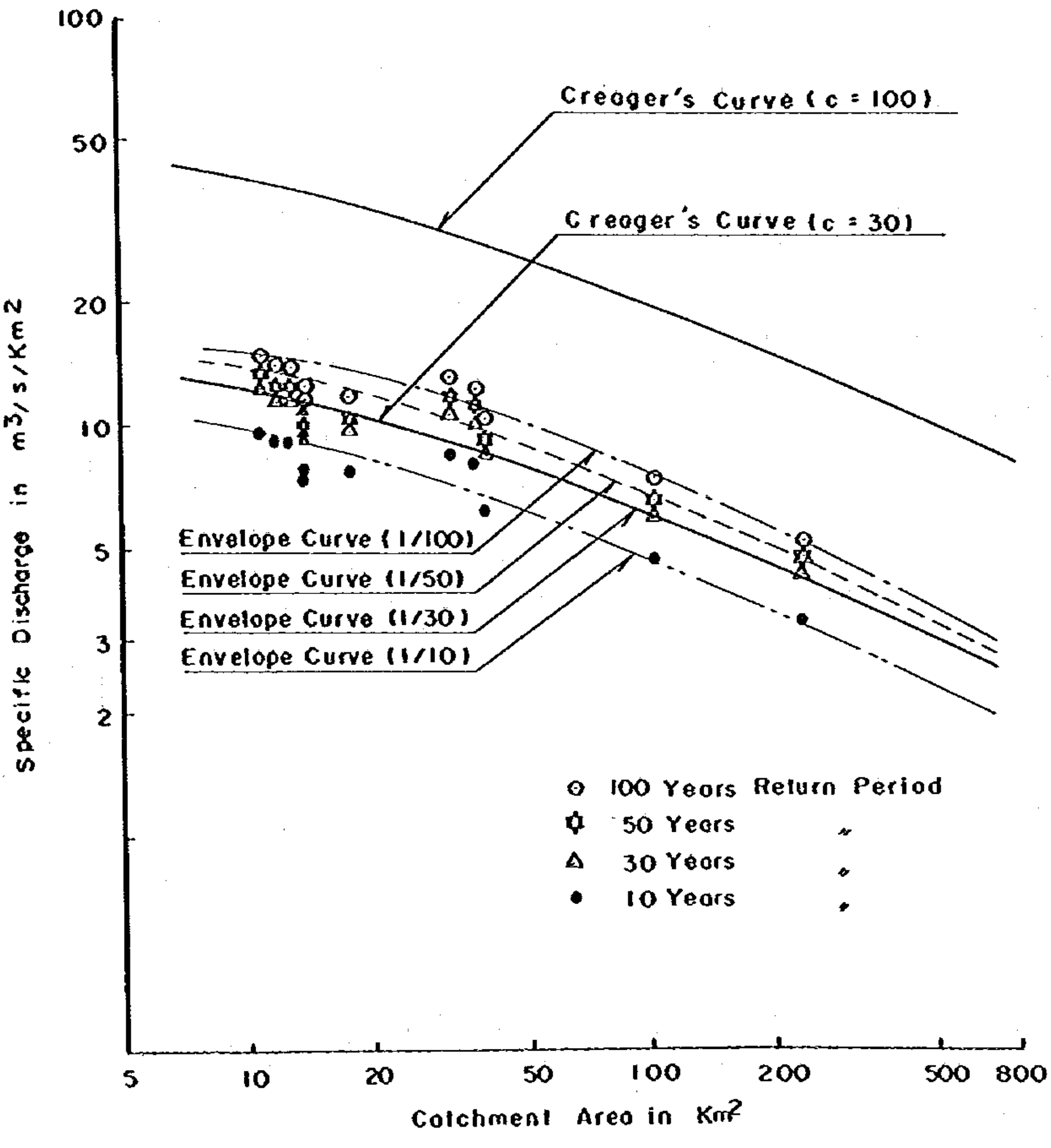


Fig. 3.3.3 SPECIFIC DISCHARGE WITH CREAGER'S CURVE

CHAPTER IV IRRIGATION

CHAPTER IV IRRIGATION

4.1 IRRIGATION PLAN

4.1.1 Project Area and Existing Irrigation Schemes

(1) Project Area

The project area is located at about 130 km northeast along the provincial road remote from Ujung Pandang, the capital of South Sulawesi Province. It extends due southward of Watan Soppeng, the capital of Kabupaten Soppeng, slenderly from north to south astride the provincial road, and is approximately bounded by the provincial road from Takalala to Sengkang in the east, the Lawo river in the north and the Mario river in the south. The western boundary is skirted along the foot of hilly range extending westward. The total gross and net areas are estimated at 8,100 ha and 6,400 ha, respectively.

(2) Existing Irrigation Schemes

(A) Classification of Irrigation System

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

Grade of System	Structures: Intake Facilities	Structures: Division Facilities	Structures: Other Facilities	Canal Density	Water Management
Non-technical	Temporary facilities with stone or gabion	Not provided or Temporary facilities with stone, wood or soil	Poorly provided	Low	Not controllable Not measurable
Semi-technical	Permanent facilities with wet masonry without measuring device	Permanent facilities with wet masonry without measuring device	Relatively well provided	Medium	Controllable but not measurable
Technical	Permanent facilities of wet masonry with measuring device	Permanent facilities of wet masonry with measuring device	Well provided	High	Controllable and measurable

(B) Existing Irrigation Schemes

Fourty eight (48) small scaled irrigation schemes have been developed in the project area of 6,400 ha comprising 44 Desa irrigation schemes and four DPU semi-technical irrigation schemes. The Desa and DPU schemes cover about 4,300 ha, 2,100 ha respectively as shown in Table 4.1.1. The Desa irrigation schemes are further devided into two categories in terms of the grade of irrigation facilities, namely, non-technical and semi-technical levels. Thirty four (34) schemes among them still remain non-technical level covering 2,900 ha and ten remaining schemes have been already up-graded to the semi-technical level, covering about 1,400 ha, as summarized below.

Scheme	Grade	Nos. of Scheme	Net Area (ha)
Desa Irrigation	Non-technical	34	2,900
	Semi-technical	10	1,400
DPU Irrigation	Semi-technical	4	2,100

Note: Details are shown in Table 4.1.1.

i) Existing Intake Facilities

In order to divert irrigation water into respective irrigation schemes, 66 intake facilities have been constructed on the seven tributaries of the Walanae river. Eleven intakes among them are perennial ones constructed with wet masonry. The remainings are ephemeral ones constructed with stones or humble gabions. The existing intake facilities are classified into three grades of A, B and C in view of the construction material and the present condition as tabulated below.

Class	Condition of Intake	Nos.
A	Wet masonry (New)	10
B	Wet masonry (Old)	1
C	Stone, or Gabion	55

Note: Details are shown in Table 4.1.1.

ii) Existing Irrigation Canals

Most of canals aligned in the existing irrigation schemes are unlined and heavily silted. The capacity of canals is generally unchanged throughout the entire reach. As the result, the canals usually fulfill dual functions of irrigation and drainage. The density of irrigation canal networks widely ranges from about 10 m/ha to 100 m/ha. In undulating area, the density is relatively high since plot-to-plot irrigation practice is difficult. The low canal density appears in relatively gently slanting area. The details of canal density are shown in Table 4.1.2.

iii) Existing Canal Related Structures

In order to convey the irrigation water diverted from the tributaries by intake facilities, some of canal and related structures in the existing irrigation schemes have been constructed with wet masonry and/or concrete.

Bridges and culverts have been provided as required. Spillways have been usually constructed at 100 m to 200 m down stream from intake facilities in order to spill out the excess water diverted into head reach. At almost diversion points, temporary weirs with stone, wood, or soil are used, which are to be made by every season.

The numbers of existing canal related structures are shown below.

Permanent Structure	Nos.
Bridge	31
Culvert	39
Diversion Structure	14
Drop	6
Spillway	3
Check	2
Aqueduct	1
Parshall Flume	1
Total	97

(C) Desa Irrigation Schemes

Fourty four (44) Desa irrigation schemes have been developed so far in the project area, covering about 4,300 ha of paddy field. The size of these Desa irrigation schemes ranges from 10 ha to 500 ha.

In these schemes, not only the construction but also the operation and maintenance of the facilities are carried out by farmers themselves. Usually, one or two water masters (Ulu Ulu) nominated by farmers are undertaking the inspection of facilities and the control of irrigation water. While, the maintenance of irrigation facilities is directly executed by farmers.

(D) DPU Semi-technical Irrigation Schemes

Four DPU semi-technical irrigation schemes have been developed in the project area, covering about 2,100 ha of paddy field. In these DPU semi-technical irrigation schemes, some reaches of main canal are lined with wet masonry and/or concrete. Most of the canals networked in the scheme still remain unlined but relatively better maintained. The density of canal system ranges from about 20 m/ha to 30 m/ha. Natural rivulets adjacent to paddy fields function main drainage canal in these schemes. No technical drainage system has been developed in these schemes. The operation and maintenance of these four schemes are carried out by DPU.

4.1.2 Irrigation Water Requirement

(1) General

The calculation on irrigation water requirement is made by 10-day basis from 1975 to 1979 as shown in Fig. 4.1.1 on the basis of meteorological records and the proposed cropping patterns.

(2) Meteorological Data

The meteorological data measured at the Sengkang meteorological station, about 30 km northward the project area, are available for the estimate of potential evapotranspiration. The data at the station from middle of 1975 to middle of 1980 are compiled in the Data Book. To apply for the calculation of evapotranspiration, these data are averaged as tabulated below:

(A) Mean Monthly Air Temperature (t)

(^{o}C)

Jan.	Feb.	Mar.	Apr.	May	Jun.	Jul.	Aug.	Sep.	Oct.	Nov.	Dec.
27.9	28.0	27.7	27.6	27.4	26.5	26.1	26.3	26.7	28.2	27.9	27.6

(B) Mean Monthly Relative Humidity (Hm)

(%)

Jan.	Feb.	Mar.	Apr.	May	Jun.	Jul.	Aug.	Sep.	Oct.	Nov.	Dec.
78.8	78.4	77.7	80.9	80.1	82.3	79.0	75.2	76.4	73.0	76.6	80.0

(C) Mean Monthly Sunshine Duration (n)

(hr/month (hr/day))

Jan.	Feb.	Mar.	Apr.	May	Jun.	Jul.	Aug.	Sep.	Oct.	Nov.	Dec.
155 (5.0)	149 (5.3)	171 (5.6)	179 (6.0)	202 (6.5)	155 (5.2)	204 (6.6)	233 (7.5)	235 (7.8)	247 (8.0)	208 (7.0)	163 (5.3)

(D) Mean Monthly Wind Velocity (U_2)

(miles/day)

Jan.	Feb.	Mar.	Apr.	May	Jun.	Jul.	Aug.	Sep.	Oct.	Nov.	Dec.
1.7	1.4	1.2	1.0	1.2	1.2	1.5	1.6	1.5	1.3	1.0	2.7

(E) Mean Monthly Pan Evaporation (Measured by class A pan)

(mm/month (mm/day))

Jan.	Feb.	Mar.	Apr.	May	Jun.	Jul.	Aug.	Sep.	Oct.	Nov.	Dec.
176	175	184	147	142	117	141	176	192	202	181	155
(5.7)	(6.2)	(5.9)	(4.9)	(4.6)	(3.9)	(4.5)	(5.7)	(6.4)	(6.5)	(6.0)	(5.0)

(3) Potential Evapotranspiration (ETo)

Three empirical formula are applied for the estimate of potential evapotranspiration. The estimated values are cross-checked by the records of class A pan evaporation at Sengkang Station.

(A) Calculation Methods

The calculation is made by use of following three empirical formulas.

i) Penman Method

The Penman method has made the most complete theoretical approach showing the consumptive use in inseparably connected with the solar energy. The formula representing the potential evapotranspiration is as follows.

$$ETo = \frac{\Delta \cdot H + 0.27 \cdot Ea}{\Delta + 0.27}$$

$$H = (1 - r) \cdot Rs$$

$$- \sigma \cdot Ta^4 \cdot (0.56 - 0.092 \sqrt{ed}) \cdot (0.10 + 0.90 \cdot \frac{n}{N})$$

$$Ea = 0.35 \cdot (ea - ed) \cdot (1 + 0.0098 \cdot U_2)$$

Where, ETo : Potential evapotranspiration (mm/day)

H : Daily heat budget (mm H_2O/day)

Rs : Solar radiation (mm H_2O/day)

$Rs = Ra \cdot (a + b \cdot \frac{n}{N})$

Ra : Extra terrestrial radiation (mm H_2O/day) (See Table 4.1.3)

a, b : Experimentally determined constants
$a = 0.21$, $b = 0.52$ (See Table 4.1.4)

n : Sunshine hours (hr/day)

N : Maximum possible sunshine hours (hr/day)
(See Table 4.1.5)

r : Reflection coefficient, $r = 0.25$
(See Table 4.1.6)

$\sigma \cdot Ta^4$: Radiation from field surface (mm H_2O/day)
(Stefan - Boltzmann Low)
(See Table 4.1.7)

σ: Stefan constant
$= 8.26 \times 10^{-11}$ cal/(cm^2.min)/$^oK^4$
$= 8.26 \times 10^{-11}$ mm H_2O/hr/$^oK^4$
$= 24 \times 8.26 \times 10^{-11}$ mm H_2O/day/$^oK^4$

Ta : Temperature (^oK : ^{o}Kelvin) or
(^{o}Abs : Absolute Temperature)

Ea : Evaporation (mm H_2O/day)

ea : Saturation vapor pressure (mm Hg)
(See Fig. 4.1.2)

ed : Actual vapor pressure (mm Hg)
$ed = ea \times Hm$

Hm : Relative humidity (%)
(See Data Book)

U_2 : Wind velocity at 2 m above the field (miles/day)

Δ : Slope of saturation vapor pressure,
ea (mm Hg) curve of temperature, T (^oF)
$= d\ ea/dT$
(See Fig. 4.1.2 and 4.1.3)

The calculation process is shown in Table 4.1.8.

ii) Christiensen-Hargreaves Method

The Christiensen-Hargreaves method is a modification of the Hargreaves formula in terms of wind, sunshine, and elevation factors. The method is explained as follows.

$$ETo = 17.4 \cdot D \cdot Tc\ (Fh \cdot Fw \cdot Fe)$$

$$Fh = 0.59 - 0.55 \cdot Hn^2$$

$$Fw = 0.75 + 0.0255 \cdot Wkd$$

$Fs = 0.478 + 0.58 \cdot S$

$Fe = 0.950 + 0.0001 \cdot E$

Where, ETo : Potential evapotranspiration (mm/day)

D : Day-time coefficient (See Table 4.1.9)

Tc : Temperature (°C)

Hn : Noon humidity (%)
$Hn = 0.4\ Hm + 0.6\ Hm^2$

Hm : Relative humidity (%)
(See Data Book)

Wkd : Wind velocity at 2 m above the field (km/day)

S : Sunshine hour ratio
$S = n/N$

n : Sunshine hours (hr/day)
(See Data Book)

N : Maximum possible sunchine hours (hr/day)
(See Table 4.1.5)

E : Elevation above the sea level (m)

iii) Radiation Method

The Radiation method is essentially an adaptation of the Makkink formula (1957). This method should be more reliable than the Blaney-Criddle approach. The Radiation method may be more applicable to equatorial zones, small islands, and high altitude places.

$ETo = C\ (W \cdot Rs)$

Where, ETo : Potential evapotranspiration (mm/day)
(See Fig. 4.1.4)

Rs : Solar radiation (mm H_2O/day)
$Rs = Ra\ (a + b \cdot \frac{n}{N})$

Ra : Extra terrestrial radiation
(See Table 4.1.3)

a, b: Experimentally determined constants
$a = 0.21$, $b = 0.52$ (See Table 4.1.4)

n : Sunshine hours (hr/day)
(See Data Book)

N : Maximum possible sunshine hours (hr/day) (See Table 4.1.5)

W : Weighting factor which depends on temperature and altitude (See Table 4.1.11)

C : Adjustment factor which depends on mean humidity, Hm (%) and day-time wind velocity, U_2 daytime (m/sec)

Hm : Relative humidity (%) (See Data Book)

U_2 daytime: Daytime wind velocity at 2 m above the field (m/sec) U_2 daytime = K . U_2

U_2 : Wind velocity at 2 m above the field (m/sec)

K : Correction factor to obtain U_2 daytime (See Table 4.1.12)

The calculation of evapotranspiration based on each formula is given in Table 4.1.8, 4.1.10 and 4.1.13, respectively.

(B) Results of Calculation

Potential evapotranspiration is calculated by the above three empirical formulas as tabulated below and illustrated in Fig. 4.1.6 comparing with the recorded Pan-evaporation in Fig. 4.1.5. The averaged value would be applied to estimate of water requirement. The maximum and minimum values of averaged potential evapotranspiration are 5.0 mm/day in October and 3.1 mm/day in June, respectively.

mm/day

Method	Jan.	Feb.	Mar.	Apr.	May	Jun.	Jul.	Aug.	Sep.	Oct.	Nov.	Dec.
Penman	3.9	4.0	4.0	3.8	3.7	3.2	3.6	4.1	4.5	4.7	4.3	4.2
C-Hargreaves	3.9	3.9	3.9	3.5	3.8	3.1	3.8	4.8	4.6	5.3	4.3	3.9
Radiation	4.0	3.8	3.9	3.9	3.7	3.0	3.7	4.2	4.7	5.1	4.4	4.1
Average	3.9	3.9	3.9	3.7	3.7	3.1	3.7	4.4	4.6	5.0	4.3	4.1

(C) Cross-Check by Pan Evaporation Method

Evaporation pans provide a measurement of the integrated effect of radiation, wind, temperature, and humidity on evaporation from a specific open water surface. To relate pan evaporation (Epan) to potential evapotranspiration (ETo) empirically derived, coefficients (Kp) are given as follows, taking into account climate and environmental conditions around the evaporation pan.

$ETo = Kp \cdot Epan$

Where, ETo : Potential evapotranspiration (mm/day)

Kp : Pan coefficient
(See Table 4.1.14)

Epan : Pan evaporation (mm/day)
(See Data Book)

The result of calculation as follows:

mm/day

Jan.	Feb.	Mar.	Apr.	May	Jun.	Jul.	Aug.	Sep.	Oct.	Nov.	Dec.
4.3	4.7	4.4	3.7	3.5	2.9	3.4	4.3	4.8	4.9	4.5	3.8

Note: Kp = 0.75 is applied, taking into account climate and environmental conditions of the location of pan at the Sengkang Meteorological station. (See Table 4.1.14)

Comparing the respective values derived from the three empirical formulas with the recorded Pan evaporation, the averaged potential evapotranspiration (ETo) derived from Penman, Christiensen-Hargreaves and Radiation is applicable to the Project.

(4) Crop Coefficient (Kc)

The crop coefficient (Kc) curves for paddy and polowijo crop are shown in Fig. 4.1.7, which was prepared in the master plan stage refferring to data measured in the objective area and other several projects in Indonesia. The calculations deciding the crop coefficient are shown in Table 4.1.20 and 4.1.21, respectively. The results are summarized as follows:

Crop	Apr.			May			Jun.			Jul.		
	1st	2nd	3rd	1st	2nd	3rd	1st	2nd	3rd	1st	2nd	3rd
1st Paddy	0.80	0.83	0.88	0.96	1.09	1.22	1.30	1.30	1.21	1.06	0.95	0.70

Crop	Nov.		Dec.			Jan.			Feb.			Mar.	
	2nd	3rd	1st	2nd	3rd	1st	2nd	3rd	1st	2nd	3rd	1st	2nd
2nd Paddy	0.80	0.83	0.91	0.99	1.08	1.21	1.29	1.27	1.18	1.12	1.02	0.84	0.70

Crop	Jul.		Aug.			Sep.			Oct.			
	2nd	3rd	1st	2nd	3rd	1st	2nd	3rd	1st	2nd	3rd	1st
Polowijo	0.31	0.38	0.44	0.54	0.65	0.73	0.74	0.71	0.64	0.56	0.50	0.40

(5) Consumptive Use (Cu)

The consumptive use (Cu) is calculated by the following formula.

Cu = Kc ETo

Where, Cu : Consumptive use (mm/10-day basis)

Kc : Crop coefficient

ETo : Potential evapotranspiration (mm/10-day basis)

The consumptive uses (cu) for paddy and polowijo are as given in Table 4.1.20 and 4.1.21, respectively and summarized as follows;

mm/10-day basis

Crop	Apr.			May			Jun.			Jul.		
	1st	2nd	3rd	1st	2nd	3rd	1st	2nd	3rd	1st	2nd	3rd
1st Paddy	30	31	33	35	40	50	40	40	38	39	35	29

Crop	Nov.		Dec.			Jan.			Feb.			Mar.	
	2nd	3rd	1st	2nd	3rd	1st	2nd	3rd	1st	2nd	3rd	1st	2nd
2nd Paddy		34	36	37	40	49	47	50	55	46	44	32	33

Crop	Jul.		Aug.			Sep.			Oct.				
	2nd	3rd	1st	2nd	3rd	1st	2nd	3rd	1st	2nd	3rd	1st	
Polowijo		11	15	19	24	31	33	34	33	32	28	28	17

(6) Percolation (Pc)

Field measurement on percolation loss (Pc) are made by twin sylinders in the irrigated paddy fields in the course of this study and the collected data are compiled in the Data Book. The data range from 1.0 mm/day to 2.0 mm/day excepting extraordinary values. Making reference to the field data, the percolation loss (Pc) of 2.0 mm/day is incorporated in the calculation of the irrigation water requirement for the project.

(7) Effective Rainfall (Re)

Paddy

Effective rainfall (Re) during a growing period of paddy is estimated by the daily water depth balance method based on the recent five years rainfall records of both stations in Watan Soppeng and Takalala. The following assumptions are made prior to the calculation;

- rainfall less than 5 mm/day is ineffective,
- excess beyond 50 mm/day is also ineffective, and
- 80 % of the rainfall which is greater than 5 mm/day and less than 50 mm/day is effective.

The effective rainfall is estimated by every 10-day period through five years from 1975 to 1979, and its calculation results are shown in Table 4.1.15.

Polowijo

Effective rainfall (Re) during a growing period of polowijo is estimated by the USDA-SCS/1 method. The calculation results are shown in Table 4.1.16.

Based on the above calculation, annual effective rainfall is estimated at 815 mm on an average, 715 mm for paddy and 100 mm for polowijo. It is equivalent to about 45 % of total annual rainfall. (See Table 4.1.17)

(8) Farm Water Requirement (Fw)

Paddy

The farm water requirement (Fw) for paddy is expressed by the following formula.

$$Fw = Wp + (Wn + Wd)$$

Where, Fw : Farm water requirement for paddy field (mm/10-day basis)

Wp : Water requirement for main paddy field after transplanting (mm/10-day basis)

Wn : Nursery water requirement (mm/10-day basis)

Wd : Puddling water requirement (mm/10-day basis)

The water requirement for main paddy field (Wp) is expressed by the following formula.

$$Wp = (Cu + Pc - Re) \times Ic$$

Where, Wp : Water requirement for main paddy field (mm/10-day basis)

Cu : Consumptive use for paddy (mm/10-day basis)

Pc : Percolation loss in paddy field (mm/10-day basis)

/1 : U.S. Department of Agricultures Soil Conservation Service.

Re : Effective rainfall for paddy (mm/10-day basis)

Ic : Crop intensity

The nursery water requirement (Wn) is given by the following formula. The calculation is made by 10-day basis through the nursery period of 20 days.

$$Wn = Sn + Pwn$$

$$Sn = (1.5 \cdot n \cdot d - Re) \times \frac{Au}{20}$$

$$Pwn = (Pwo - Re) \times \frac{Au}{20}$$

Where, Wn : Nursery water requirement (mm/10-day basis)

d : Potential evapotranspiration (ETo) + percolation loss (Pc) (mm/day)

Re : Effective rainfall (mm/10-day basis)

$\frac{Au}{20}$: Ratio of transplanting area to total paddy field area

Pw : Puddling water (mm)
(puddling water of 120 mm is applied on reference to following paragraph)

The puddling water requirement (Wd) is presented by the following formula.

$$Wd = (Pw - Re) \times Ap$$

Where, Wd : Puddling water requirement (mm/10-day basis)

Pw : Puddling water (mm) (mentioned below)

Re : Effective rainfall (mm/10-day basis)

Ap : Ratio of puddling area in 10-day period to total puddling area

The puddling water quantity (Pw) is theoretically assessed by soil to be saturated and porosity, and presented by the following formula.

$$Pw = Ds + Ws$$

Where, Pw : Puddling water quantity (mm)

Ds : Water depth above soil surface after puddling (mm)

Ws : Difference of soil moisture before and after puddling (mm)

Following assumptions are applied.

- Water depth above soil surface after puddling (Ds) is 45 mm.

- Polosity (Po) is 50 % in both surface soil (d_1) with depth of 200 mm and sub-soil (d_2) with depth of 100 mm.

- Soil moisture (Ms) before water supply is 20 % in volume.

- Vapour phase (Vp) in soils after puddling is 5 %.

Then, the puddling water quantity (Pw) is calculated as follows.

$$
\begin{aligned}
Pw &= Ds + Ws \\
&= Ds + (d_1 + d_2) \times (Po - Ms - Vp) \\
&= 120 \text{ (mm)}
\end{aligned}
$$

The nursery and puddling requirements are estimated in Table 4.1.18 and 4.1.19 respectively and furthermore, the farm water requirement sums up in Table 4.1.20, based on the relevant formula and procedures.

Polowijo

The farm water requirement (Fw) for polowijo is presented by the following formula.

$$Fw = (Cu - Re) \times Ic$$

Where, Fw : Farm water requirement for polowijo (mm/10-day basis)

Re : Effective rainfall (mm/10-day basis)

Ic : Crop intensity

The calculations are shown in Table 4.1.21.

(9) Irrigation Efficiency (E)

The irrigation efficiency (E) is usually defined as follows;

$$E = \frac{Ea}{100} \times \frac{Eco}{100} \times 100 \text{ (\%)}$$

Where, Ea : Water application efficiency (%)

Eco: Water conveyance and operation efficiency (%)

i) Water Application Efficiency (Ea)

The water application efficiencies for paddy and polowijo are assumed as 80 % and 75 % respectively in due consideration of the proposed irrigation methods which will be discussed in the latter section.

ii) Water Conveyance and Operation Efficiency (Eco)

The water conveyance and operation efficiency (Eco) depend upon condition of the irrigation system and the operation skill of the irrigation facilities. The common value of the efficiency is usable for irrigations of both paddy and polowijo. Taking into account the size of the project and the technical level of water management in the project area, the water conveyance and operation efficiency (Eco) for the Project is assumed as 80 %.

iii) Irrigation Efficiency (E)

The irrigation efficiencies (E) for both paddy and polowijo are calculated based on the above mentioned assumptions. The results are summarized as follows.

Crop	Irrigation Efficiency (E)
Paddy	64 %
Polowijo	60 %

(10) Diversion Water Requirement (Dw)

The diversion water requirement (Dw) is calculated by the following formula.

$$Dw = Fw \times \frac{100}{E}$$

Where, Dw : Diversion water requirement (mm/10-day basis)

Fw : Farm water requirement (mm/10-day basis)

E : Irrigation efficiency (%)

The calculations are shown in Table 4.1.20 and 21.

(11) Unit Diversion Water Requirement (Qe)

The calculation and its result are shown in Table 4.1.20 and 21, and summarized in Table 4.1.22, taking into account total water requirement during overlaped growing period of two kind of crops in the proposed cropping pattern.

The peak unit diversion water requirement (Qemax) for paddy is 1.26 l/s/ha on late January, 1978, which occurs during the growing stage of the 2nd paddy in 1977. As for polowijo, the peak unit diversion requirement (Qemax) of 0.656 l/s/ha occurs on Middle September, 1976, 1977 and 1979.

4.1.3 Dependable Irrigation Water

The dependable irrigation water is assumed to be 80 % of the run-off in the respective water sources in due consideration of the river cource conservation and the domestic water for inhabitants along the downstream of each river.

(A) Tributaries

The seven tributaries of the Walanae river, such as Madenra, Baruttungnge, Labempa, Congko, Panincong, Maccope, and Belo, would be firstly developed for the project. The total dependable water discharge on these tributaries is shown in the Data Book. The minimum discharge of 0.30 m^3/sec occurred on early and middle September, 1976.

(B) Langkemme River

The dependable water on the Langkemme river is to be used to supplement the water deficit caused by water supply from the tributaries. The dependable water discharge in the Langkemme river is shown in the Data Book. The minimum discharge of 0.42 m^3/sec occurred on middle September, 1976.

(C) Sero River

The dependable water on the Sero river is to be diverted to supplement the water deficit which would be caused even if the irrigation water were supplied by both the tributaries and the Langkemme river. The dependable water discharge on the Sero river is shown in the Data Book. The minimum discharge of 0.42 m^3/sec occurred in the period from late September to middle November, 1977.

4.1.4 Balance Calculation between Diversion Requirement and Dependable Water

To clarify the shortage of irrigation water, a balance calculation is annually made by 10-day basis between the water resources dependable on the respective rivers and the seasonal diversion requirement from 1975 to 1979.

The balance calculation is made according to the Work Devisions which are formulated in the construction plan (Chapter-V). In the Work Division-I, 2,900 ha of non-technical area would be up-graded to semi-technical level. The balance is made between the diversion requirement for the 2,900 ha of non-technical area and the dependable water resources in the tributaries, and the deficits of irrigation water for the 2,900 ha after the completion of the Work Division-I are clarified through the balance calculation. Intake and irrigation efficiencies are assumed as 80 % and 56 %, respectively.

In the Work Division-II, 4,500 ha of the semi-technical area including the up-graded area in the Work Division-I would be up-graded to technical area. The balance is made between the water requirement for the 4,500 ha of the semi-technical area and the dependable water resources in the tributaries and the Langkemme river. As clarified in the balance calculation. The both dependable water resources are almost balanced with the water requirement of 4,500 ha. The intake and irrigation efficiencies are modified as 80 % and 64 %, respectively, in due consideration of the up-graded condition in the area.

In the Work Division-III, 1,900 ha of DPU semi-technical would be up-graded to technical level. The balance is made between the water requirement for the 6,400 ha of the technical area including 4,500 ha under work division-II and the dependable water resources in the tributaries, the Langkemme river and the Sero river. The intake and irrigation efficiencies are assumed as 80 % and 64 %, respectively.

After completion of the Work Division-III, the maximum irrigation waters deficit of 4,060 l/sec occurs on late June, 1975. The water balance can be brokendown as tabulated below. The maximum continuous deficit period of 40 days appears from late August to late September, 1976.

		(late June, 1975)
Diversion Water Requirements		6,670 (l/sec)
Dependable flow	in the Tributaries	900 (l/sec)
"	in the Langkemme river	860 (l/sec)
"	in the Sero river	850 (l/sec)
Sub-total		2,610 (l/sec)
Irrigation Water Deficit		4,060 (l/sec)

The details of calculations are shown in Table 4.1.23 and Fig. 4.1.8.

4.1.5 Design Discharges of Sero Diversion Canal and Langkemme Main Canal

(A) Sero Diversion Canal

According to the balance calculation, the Sero diversion system would be operated for only 420 days during past five years. The maximum intake discharge of 5.5 m^3/sec has occurred on early January in 1976. In this view, it seems to be uneconomical for the design of Sero diversion canal to propose the maximum discharge of low operation frequency. The discharge of 2.5 m^3/sec, equivalent to 94 % frequency, would be applied for the design of the Sero diversion canal.

(B) Langkemme Main Canal

The operation frequency of the Langkemme intake is 78 times of 10-day period for the five years. Among them, the maximum intake discharge is 3.6 m^3/s on late January in 1978. The design discharge of the Langkemme main canal is determined at 2.5 m^3/sec, equivalent to 96 % operation frequency, taking into account the economical use of the intake capacity. Thus, the design discharge of the Langkemme main canal would be 5.0 m^3/sec, adding 2.5 m^3/s conveyed by the Sero diversion canal.

4.1.6 Irrigation Method

(1) Irrigation Method for Paddy

(A) Recommendable Irrigation Method

Two alternative irrigation methods of paddy field are conceivable for the Project. One is the continuous irrigation method and the other is the intermittent irrigation method. Considering the situation of, dependable water discharge and irrigation water deficit discussed in the foregoing section, the continuous irrigation method is not recommendable even though the water management is easy.

As regards the intermittent irrigation method, the low permeability of the field is required to storage the irrigation water during an interval. Then, the storage ability of the field is enough for the intermittent irrigation method in the project area. Finally, the intermittent irrigation method is recommended for the Project considering such conditions as mentioned above.

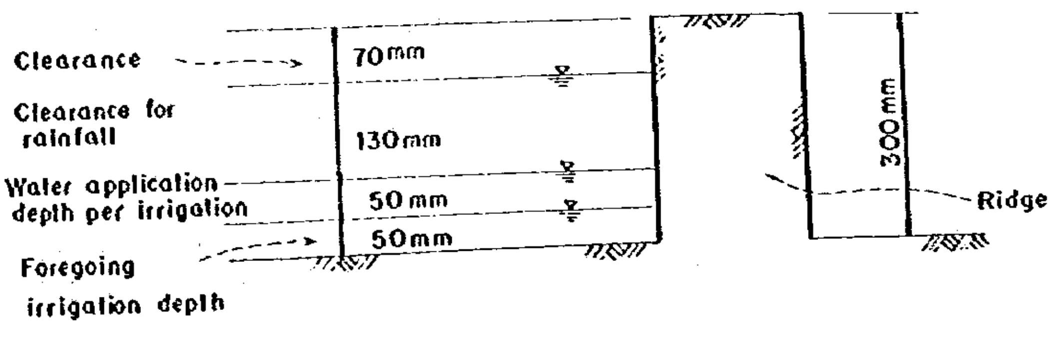
(B) Irrigation Interval

A rotation area consists of several rotation units. A tertiary block or a sub-tertiary block in the Project is generally less than 100 ha, so it is to be a rotation area.

While, the size of rotation units depends upon natural boundaries of drainage ditches, canals, creeks and roads, and then it is generally to be around 10 ha.

(C) Irrigation Interval

The ridge of paddy field would be proposed to be 30 cm high. To use an occasional rainfall for irrigation effectively, excessively deep irrigation is unfavourable. Daily rainfall of 130 mm of five years return period is assumed using the average of probable daily rainfalls at both Takalala and Watan Soppeng stations. As illustrated below regarding share of water depth in paddy field, the water depth to be once irrigated should be confined within 50 mm for effective use of occasional rainfall. The peak farm water requirement (Fw) is estimated at 77 mm/10-day as shown in Table 4.1.21. It is equivalent to 7 mm/day of daily requirement. Thus, seven days irrigation interval is recommendable for the Project.



(2) Irrigation Method for Polowijo

(A) Recommendable Irrigation Method

The polowijo cropping in the Project would be introduced during only three months of dry season. Considering the land preparation for upland crops in which the paddy field would be reformed, it is uneconomical to apply such typical field irrigation method as furrow irrigation. So, it is recommended that small in-field ditches would be dug by every season in order to take the irrigation water quickly into the field and supply it equally to the field. The ditches would be made in checkerboard pattern with an interval of 10 m or so. The field surface would be flooded by the water overflowed from the ditches.

(B) Irrigation Interval

i) Available Moisture (AM)

Available moisture (AM) on the field irrigation has been defined as moisture difference in effective root zone between the field capacity (pF 1.5 - 2.0) and the first wilting point (pF 3.5 - 3.8). While, it is defined as moisture difference in effective soil layer between the water holding capacity after 24 hours of soil saturation and the moisture equivalent (PF 3.0) to depletion of moisture content optimum growth. No data on the effective soil layer of the project area is available. Hence, these definitions would be applied to the calculation.

ii) Effective Root Zone Depth (D)

The effective root zone of polowijo crops is determined considering that the paddy field after harvest is used for polowijo cropping. The effective root zone depth (D) would be limited to 30 cm of plowing depth.

iii) Soil Moisture Extraction Pattern (SMEP)

The soil moisture extraction patterns of polowijo crops are determined in accordance with the "Basic Moisture Extraction Pattern" /1 established by Sockley (U.S.A), as shown in Fig. 4.1.9.

iv) Total Readily Available Moisture (TRAM)

The calculation of total readily available moisture (TRAM) is carried out in the following procedures. The avalalble moisture (AMi) in a soil layer is calculated by the use of following formula.

$$AMi = \frac{1}{100} \cdot (Fc - Wpf) \cdot h$$

/1 Figure of the "Basic Moisture Extraction Pattern" is given in SPRINKLER IRRIGATION (1953). Published by Sprinkler Irrigation Association.

Where, AMi : Available moisture in a soil layer (mm)

Fc : Soil moisture at field capacity (volume %)

Wpf : Soil moisture at first wilting point (volume %)

h : Thickness of a soil layer (mm)

The soil moistures at both the field capacity and the first wilting point are determined considering soil type of the Project area in accordance with empirical data reported in "The Study on Field Irrigation, No.IX" (Feb., 1967, Tokai-Kinki Agricultural Experiment Station, Japan), as summarized below. (See Fig. 4.1.10)

Soil Type : Clay	Soil Moisture (volume %)
Field Capacity, Fc (pF 1.5)	55 %
First Wilting Point, Wfp (pF 3.5)	39 %

The thickness (h) of a soil layer in effective root zone is assumed at 75 mm. (See Fig. 4.1.9) Then the available moisture in a soil layer (AMi) is estimated at 12 mm. The total soil moisture extraction in effective root zone is calculated by the use of following formula and summarized below:

$$WDi = AMi \times \frac{100}{RMEi}$$

Where, WDi : Soil moisture extraction based on each soil layer's characteristics (mm)

RMEi: Moisture extraction ratio in each soil layer (%)

Soil Layer	Total Soil Moisture Extraction in Effective Root Zone (mm)
1st Layer	30
2nd "	40
3rd "	60
4th "	120

The total readily available moisture (TRAM) is defined as the minimum value of total soil moisture extraction in effective root zone.

v) Irrigation Interval

The maximum irrigation interval is given by the following formula.

$$\text{Max. Irrigation Interval (day)} = \frac{\text{(TRAM, mm)}}{\text{(Peak Consumptive Use, mm/day)}}$$

The TRAM of 30 mm is estimated as shown in the above table and the peak consumptive use is estimated at 3.4 mm/day as shown in Table 4.1.21. Then, the maximum irrigation interval is calculated at eight days.

Finally, to meet the irrigation interval for paddy mentioned hereinbefore, the design irrigation interval for polowijo would be also proposed at seven days. The water application depth per irrigation (Dn) is estimated at 24 mm by multiplying the irrigation interval by the peak consumptive use.

4.2 PROJECT FACILITIES

4.2.1 General

The general features of the Langkemme Irrigation Project is to irrigate the area of about 6,400 ha in maximum by developing water in three water resources, the Langkemme, the Jupang, and the tributaries of the Walanae river.

The facilities required for the project mainly comprise head works, canals, and their related structures. The basis for planning and design of the Project facilities is the most effective use of water resources. The followings are brief description of the comparative study, design criteria, and design of the project facilities.

4.2.2 Intake Weirs

Four intake weirs are proposed in the Langkemme, the Jupang, the Unyi, and the Pising river to divert the required irrigation water for the project area. In addition, 66 existing intakes which have been installed in the project area would be integrated into 21 weirs for the effective use of water resources in the tributaries.

Three types of intake weirs are recommendable i.e. fixed weir with a scouring sluice, tirol type weir, and gabion weir. Selection of the weir type to be applied to each intake system depends on flood discharge, hydraulic and topographic condition of the river, diversion discharge, and availability of construction materials.

The followings are brief description of the study on each intake weir.

(1) Langkemme Intake Weir

(A) Selection of Site

As a result of the detailed field reconnaissance survey, four attractive sites for construction of the intake weir are selected at the up and down streams of the existing hydropower station.

The most eleveted paddy fields in the project area extend at an altitude of EL 160 m. In order to distribute irrigation water to such area, the intake water level should be a few meters higher than EL 160 m considering the friction loss in the canals and head losses caused in the related structures. Meanwhile, the river bed elevation at the existing hydropower station is surveyed as EL 152 m. In case of the site located in the down stream of the existing hydropower station, the intake weir height would be more than 10 m above the river bed. In consideration of the hydropower station, the back water damed up by the weir will exert decesive influence on the operation of the hydropower station. It is, therefore, concluded that the intake site located in the downstream of the existing hydropower station is technically infeasible.

The comparative study for the selection of the intake site is made on the remaining three alternative sites located in the upstream of the hydropower station. Each alternative site of the intake weir and irrigation system are shown in Fig. 4.2.1 and briefly described as follows:

(Alternative - 1)

Alternative-1 is the site proposed in the master plan in 1979. It is located at about 5.5 km upstream of the confluence with the Sero river. The site of Alt.-1 requires an intake weir of about one m high and 35 m long. The irrigation water is taken at elevation of 204 m. In the system, the head reach of about 200 m long is provided to divert the water into the main canal. The topographic conditions at the sites of the intake structure and the head reach are undulating steep. The main canal is also aligned in the hilly area at the begining half reach of it's route. The construction cost of the main canal would be considerably high.

(Alternative - 2)

Alternative-2 is the site being located at about 1.5 km downstream of the Alt.-1 site. An intake weir of about 2 m high and 50 m long is required in the Alternative-2. The irrigation water is taken at elevation of 180 m and conveyed by the head reach of about one km along the left bank of the Langkemme river.

(Alternative - 3)

Alternative-3 is the site newly proposed by the survey team based on the detailed field reconnaissance survey. It is located at about one km downstream of the Alt.-2 site. An intake weir of about 2.5 m high and 40 m long is required at the site. The irrigation water is taken at the elevation of 170 m and conveyed along the left bank of the Langkemme river for about a half km.

The main canal alignments are made on the topographic maps in scale of 1 to 25,000. The main canal in each alternative would take the same route at the lower reach from the proposed hydropower station so that the upper reaches of main canal from the proposed hydropower station would be considered in the cost comparison. The main features of irrigation system proposed in each alternative are summarized below.

Facilities	Alt.-1	Alt.-2	Alt.-3
1. Intake Weir			
- Weir Height (m)	3.0	4.0	4.5
- Crest Length (m)	35.0	50.0	40.0
- Crest EL. (m)	203.0	180.0	170.0
2. Main Canal Length (km) (Upper reach from proposed hydropower station)	12.3	15.3	14.6
3. Related Structures			
- Culvert (Nos.)	15	18	18
- Syphon (Nos.)	3	2	1
- Bridge (Nos.)	5	6	7

The design conditions of the main canal used in the alternative study are as follows.

1.	Design Discharge	5 m^3/sec.
2.	Hydraulic Gradient	1 : 5,000
3.	Roughness Coefficient for Manning's formula	0.03
4.	Hydraulic Depth	1.89 m
5.	Mean Velocity	0.497 m/s
6.	Side Slop of Canal Section	1 : 1.5
7.	Canal Base Width	2.5 m
8.	Free board	0.4 m
9.	Width of Beam	2.0 m

The cost comparison is made on head works, main canal, and its related structures. To estimate quantity of earth work, the profile of each alignment of main canal is prepared based on the topographic maps in scale of 1 to 25,000. The quantities of earth excavation and embabkment are estimated on the assumed canal section and profile of each alternative route. In the above estimation, rock excavation is differentiated in due consideration of geological condition of each alternative route. As regards unit cost, data worked out in the master plan is available for the study.

The construction costs of respective alternatives aforementioned are summarized as follows and further brokendown in Table 4.2.1.

(Unit: US$)

Work Item	Alternative-1	Alternative-2	Alternative-3
I. Earth Works	1,693,000	1,228,000	1,176,000
II. Related Structure	920,000	1,014,000	933,000
III. Intake Weir	1,240,000	1,430,000	1,260,000
IV. Construction Road	73,000	-	-
Total	3,926,000	3,672,000	3,369,000

As clarified above, the alternative-3 is the most eligible in view of the construction cost. In addition, the alternative sites are closely related with the length of the Sero diversion canal mentioned hereinafter. The alternative-1 requires the longest diversion canal since the head and tail reaches of the Sero diversion canal are extended depending on the location of the Langkemme intake site and its intake water level. The anticipated length of the Sero diversion canal in each alternative is estimated as follows.

	(Unit: Km)
Alternative 1	19.0
Alternative 2	16.5
Alternative 3	15.0

Provided the additional cost for extended length of the Sero diversion canal is taken into account, a decisive difference of construction cost arises among three alternatives.

As regards the construction of the intake weir, construction road is not required for the Alternative-2 and -3, excepting short access road, since village roads passable by heavy construction machinery extend up to the vicinity of Alternative-2 site and most of the road will be asphalt-paved before long. The village road must be extended one km at least for the construction of weir in the Alternative-1.

Depending on the result of the study, the Alternative-3 is superior to the other alternatives in view of the construction cost. And, the Alternative-3 is provided with better conditions on the construction works and the operation/maintenance of the intake system. From the overall viewpoints, the Alternative-3 would be proposed as the optimum intake site of the Langkemme Intake weir.

(B) Design Condition

i) Topography of the Intake Site

The proposed site of the Langkemme intake weir is located at the head of the alluvial fan. There exist the narrow strip of the alluvial deposit of about 500 m wide in the both banks of the river and the skirt of the hilly region with steep inclination of about 1 to 2. At the proposed intake site the river is about 50 m wide and about 10 m deep with the bottom elevation of 168.40 m above mean sea level. The river bed gradient is measured as 1 to 100 on an average at the proposed intake site.

ii) Geology

As described in the ANNEX III, the geological condition of the proposed intake site is broadly constituted by the following two layers; sediment deposit and base rock. The thickness of deposits is estimated at about 2 m on an average and the base rock is relatively hard and cemented excepting the weathered surface of about 0.5 m thick.

iii) Hydrology

As no rating curve was available at the proposed intake site, the curve at the Langkemme Bridge at 1.5 Km downstream of the proposed intake site was elaborated by applying Mannings formula based on the existing data. According to the study in CHAPTER III the minimum and maximum mean 10-day discharge in recent five years are estimated as 0.79 m^3/sec and 26.04 m^3/sec, respectively.

As a result of the study mentioned hereinbefore, the optimum diversion requirement at the intake site is calculated at 2.50 m^3/sec. However, the intake discharge of 5.0 m^3/s is adopted for the intake structure on the basis of the effective water use of the Langkemme river.

The probable flood discharge at the site is estimated in the CHAPTER III. The probable flood of 100 years return period is adopted for the design flood discharge of the weir. The flood discharge and the corresponding flood water level at the proposed intake site are quoted from the Table 3.3.5 and summarized as follows.

Return Period in Years	Flood Discharge (m3/sec.)	Flood Water EL. (m3/sec.)
1/100	779.2	172.1
1/50	695.6	171.9
1/30	637.5	171.8
1/10	502.9	171.4

(C) Design of Weir

i) Main Features

As described in the "Design Condition", the longitudinal gradient of the Langkemme river at the intake site is very steep as 1 : 100 on an average. Besides, the design flood discharge is large and the water velocity exceeds 5 m/sec. In such conditions, the overflow type weir of 37.5 m long with scouring sluice of 4.0 m wide is proposed for the Langkemme intake. The full width of weir would be constructed on the base rock to secure the stability of weir.

A syphon would be installed inside the weir to link the Sero diversion canal to the Langkemme main canal. The weir is constructed with wet stone masonry which is much economical and convenient for procuring the materials. Meanwhile, the conduit of the syphon should be constructed with reinforced concrete for resisting against shearing forces.

ii) Hydraulic Calculation

Overflow discharge of water under complete overflowing condition can be calculated by;

$$Q = C\ B\ H^{3/2}$$

Where, Q : Discharge (m^3/sec)
B : Width of Weir (m)
H : Upstream Water Depth above Crest (m)
C : Coefficient of Discharge (= 1.7)

iii) Typical Section

Typical section of the weir is decided considering such forces as external water pressures, uplift pressures, silt pressures, earth quake, and own weight of the weir. The stability of the weir would be examined in view of overturning, sliding, and overstressing with sufficient safety factor.

As the bed rock at the weir site has enough bearing capacity for the weir, the stability analyses are carried out on (a) overturning and (b) sliding. The analyses are made on following two cases;

Case I Extraordinary condition:
- Water level: intake water level
- External force: seismic condition (seismic coefficient, K = 0.15)

Case II Ordinary condition:
- Water level: design flood water level
- External force: non-seismic condition

Details of the stability analyses are shown in Table 4.2.2 - 4.2.4 and Fig. 4.2.2 - 4.2.3.

The upstream corner of crest is rounded with a circle of 0.5 m radius and the downstream slope is transfigured with parabola curve. At the toe of the weir, upturned backet type basin of 4.0 m radius is installed as the stilling basin. The details of the typical section of the weir is shown in the ANNEX IV.

iv) Intake Structure

The intake structures are designed based on the intake water discharge of 5.0 m^3/s. The water level at the head of the Langkemme main canal would be EL. 170.0 m and the elevation of the intake sill is designed as to be 169.2 m. In order to prevent the sand intrusion from the river, a permissible intake velocity is limited to be around 1.0 m/sec. The intake width of 6.0 m is, therefore, required for the Langkemme intake structure.

(2) Jupang Intake Weir

(A) Selection of Site

The site of the Jupang intake weir is closely related with the proposed site of the Langkemme intake weir as mentioned hereinbefore. The water level required for the tail of the Sero diversion canal is assumed at EL. 170.6 m, accounting the hydraulic headless due to crossing the Langkemme river. The diversion canal is aligned on the map of 1/25,000 on the basis of the topographic condition along the Sero river and the water level at the tail of the canal. As a result of the canal alignment, the intake water level at the Jupang intake weir is estimated at EL. 176.6 m. To meet the requirement of the intake water level, the intake site is selected at about 7.5 km upstream from the confluence of the Pising river. A plan table survey is made at the site and the topographic map of 1/500 is prepared for the design of the intake weir.

(B) Design Condition

i) Topography of the Intake Site

The river of the site forms deep gorge and is about 40 m wide. The altitude of riverbed is clarified at about 176.20 m above mean sea level through topographic survey. The longitudinal gradient of the river bed is measured as 1 to 100 on an average at the proposed intake site.

ii) Geology

According to the study in the ANNEX III, the geological condition of the proposed itake site is constituted by the layer of tuff breccia and the seidmentated deposit layer including considerably sizable rocks. The former is much favourable for the foundation of weir. The thickness of the later is estimated at two meters, more or less.

iii. Hydrology

As no discharge data for the Jupang river is available, the runoff of the Jupan river is estimated based on the data recorded at the Sero gauging staff. Depending on the study in CHAPTER III, the maximum and minimum 10-day mean discharge in the recent five years at just downstream of the confluence with the Pising river are estimated as to be 102.06 m^3/sec and 0.42 m^3/sec, respectively.

The study on "Irrigation plan" clarifies the diversion requirement of 1.92 m^3/sec at the Jupang intake. However, an intake capacity of 2.5 m^3/sec would be given for the Jupang intake to effectively use the available water resources in the Jupang river.

The probable flood discharge at the proposed intake site is estimated in the CHAPTER III. The probable flood of 100 years return period is adopted for the design of the weir. The flood discharge and corresponding flood water level at the proposed intake site are quoted from Table 3.3.5 and summarized as follows.

Return Period in Years	Flood Discharge (m3/sec.)	Flood Water EL. (m3/sec.)
1/100	1,262.9	181.2
1/50	1,128.8	180.9
1/30	1,035.4	180.7
1/10	816.7	180.2

(C) Design of Weir

i) Type of Weir

The discharge in the Jupang river sharply fluctuates in the wide ranges as described in the CHAPTER III. In addition, the probable flood discharge at the proposed intake site is remarkably large, resulting in excessively high water level. Therefore, the tirol type weir would be suitable for the Jupang intake weir.

The full span of weir of 35 m wide would be constructed on the bed rock after excavating sediment materials to secure the stability of the weir.

ii) Hydraulic Calculation

The overflow discharge of the weir can be calculated by the same formula applied for the design of the Langkemme intake weir. Meanwhile, the discharge to be taken by the tirol type weir is estimated by the Mostkow's formula which is descrived as follows.

$$QI = Qo = Ll \cdot U \cdot B \cdot M \sqrt{2gHo}$$

Where, QI : Intake discharge (m^3/sec.)

Qo : Discharge at the upstream of the intake screen (m)

Ll : Length of intake screen in case of taking 100% of Qo (m)

U : Discharge Coefficient, 0.6 (in case of the screen slope = 30^o)

B : Width of intake weir (m)

M : $\sum S/B$

g : Acceleration of gravity (9.8 m/sec^2)

S : An openning of the two screen bars (m)

iii) Typical Section

The typical section of the weir is decided in view of stability and installation of intake screen. The details of typical section and installation of the intake screen are shown in the ANNEX IV. The stability analyses are made on overturning and sliding. The details of the stability analyses are shown in Table 4.2.5 to 4.2.7 and Fig. 4.2.4 and 4.2.5. The upstream corner of crest is rounded with a circle of 0.5 m radius and the downstream slope is transformed with parabola curve. The details of the designed section is shown in the ANNEX IV.

iv) Stilling Basin

In order to dissipate hydraulic energy and to protect the river bed from erosion, an apron is equipped at the downstream of the weir. The length of the apron is calculated by the Echevery's formula taking into consideration of the design flood. The formula is described as follows.

$$L = 3 \sqrt{H \cdot F}$$

Where, L : Length of a apron (m)

H : Water depth above the crest in the upstream of a weir

F : Difference of water level between up and downstream of weir

The apron of 15.6 m long is estimated based on the above formula. It would be constructed with masonry and gabion works.

v) Desilting Facilities

In order to prevent the sand intrusion to the main canal, desilting facilities such as desilting basin and desilting chamber are provided. The former is provided in the diversion canal, and the later is installed at the outlet of the collecting channel of the weir. The invert of the collecting channel is given a slant of 1/20. Therefore, jet flow occurs in the channel and flushes the sediment materials out into the desilting chamber.

(3) Unyi and Pising Intake Weir

(A) Selection of Sites

The sites for intake weirs in the Unyi and the Pising rivers are determined in taking consideration of diverting water level into the Sero diversion canal. The Water surface elevation at the confluence of the Sero diversion canal with the Unyi and the Pising offtake channels are 175.47 m and 174.11 m respectively. On the basis of these water surface elevations, offtake channels are aligned along the respective river banks and then, the both intake sites are selected so as to meet the respective required water level.

The proposed site of the Unyi intake is located at about 0.5 km upstream from the confluence of the Jupang and the Unyi river, while the proposed site of the Pising intake, about three km upstream from the confluence of the Jupang and the Pasing rivers.

The both intake sites form narrow and deep valleys. The river channels at both sites are about 30 m wide. The gradients of the river bed in the Unyi and the Pising river are estimated at 1 to 25 and 1 to 40, respectively. The geological condition at both intake sites are favourable for construction of the intake structures.

(B) Design Condition

At the confluence with the Jupang river, the Unyi and the Pising river have catchment areas of 32 Km^2 and 37 Km^2, respectively. The probable flood discharge and water level at both intake sites are remarkably large and high. The probable flood discharge and water level worked out in the study are summarized as follows.

River	Return Period in Year	Flood Discharge (m^3/sec)	Flood Water EL. (m)
Pising	1/100	466.8	173.7
	1/50	416.3	173.5
	1/30	382.3	173.4
	1/10	301.6	173.1
Unyi	1/100	425.6	-
	1/50	379.7	-
	1/30	348.8	-
	1/10	275.2	-

As results of the studies on "HYDROLOGY" and "IRRIGATION PLAN" the diversion requirement at proposed intake sites are estimated at 0.28 m^3/sec on the Unyi and 0.30 m^3/s on the Pising river. Meanwhile, the design intake discharge of 0,5 m^3/sec would be proposed for the design of both intake structures and offtake channels.

(C) Design of Weirs

The diversion requirement at both intake sites are rather small. A gabion weir is recommendable for the both intake weirs, the design conditions described above and construction materials available insitu being considered. Design of gabion weirs is made based on the Indonesian design criteria for gabion structures and refering to the designs on the existing gabion weirs constructed in the South Sulawesi. Five types of gabion are prepared to meet the size for each portion of the weirs.

The water surface elevation at the head of the Unyi and the Pising offtake channel would be 176.3 m and 174.7 m, respectively. The permissible intake velocity is limited to be around 0.8 m/sec in order to prevent the sand intrusion to the offtake channel. The width of intake and water depth at the intake are designed to be 2.0 m and 0.3 m, respectively. Consequently, the design water velocity at intake would be 0.83 m/sec.

Intake structure should be installed as close as possible to the gut of river and the gut must be stably maintained. In order to keep the gut of river toward the intake, the weir would be constructed with an angle of about 64 degrees to the center line of the river. The notch of 2 m wide and 0.3 m deep would be provided with the weir for the stabilization of the gut of river. The details of the Unyi and the Pising weir are shown in the ANNEX IV.

(4) <u>Existing Intake Weir</u>

(A) Present Conditions

There exist 66 intake weirs on the tributaries of the Walanae river. The existing irrigation schemes in the project area are mainly contributed on the seven tributaries with the existing intake weirs. An existing intake weir commands the irrigable land of 70 ha on an average. Therefore, the diverted discharge of water at an intake can be estimated as to be about 0.1 m^3/sec.

The existing intake weirs are divided into 3 grades depending upon the necessities of their rehabilitation and integration, which are (a) the weirs newly constructed with stone masonry work, (b) the timeworn weirs with stone masonry work and (c) Gabion and piled stone weirs. The numbers of intake weirs in each grade is as follows:

Grade	Nos. of weir
(a)	10
(b)	1
(c)	55
Total	66

(B) Integration of the Existing Weirs

The sixty six (66) existing intakes in the tributaries would be integrated into 21 intakes. In addition, an intake weir is proposed to be newly constructed in the Laja river which will command the irrigable land of 60 ha with the diverted water of 0.1 m^3/sec.

Generally, the longitudinal gradient of river bottom in the tributaries is rather steep than 1 to 100 so that the river runoff would sharply fluctuate. Taking into account of the high magnitude of probable flood and the required water to be diverted, the tirol type weir would be recommended. As regards the intake weir to be constructed in the Belo river, the ordinary type of intake weir with a scouring sluice would be proposed since considerable amount of bed-loads rush and deposit on the river bed.

All the intake facilities would be constructed with wet stone masonry, since materials for stone masonry work are easy obtainable at the respective constructing sites. The ANNEX IV shows the details of dual intakes of tirol type weir. The required numbers of respective intake facilities to be integrated and newly constructed in the tributaries are shown as follows.

Type of Intake Facilities	Nos.
I. Ordinary type	3
II. Tirol type	
1. Mono intake	13
2. Dual intakes	6
Total	22

4.2.3 Design of Canals

(1) Canal System

The canal system of the project consists of the Langkemme main canal, the Sero diversion canal, link canals, offtake channels, and tertiary canals. The Langkemme main canal is the trunk canal which conveys water from the Langkemme intake site to entrances of the secondary blocks. The route of main canal is selected at the skirt of the western mountain ranges. The design discharge of irrigation water conveyed through the main canal would range from 5.0 m^3/sec to 0.804 m^3/s. The total length of the Langkemme main canal is 29.5 Km including a tunnel of 720 m.

The Sero diversion canal is constructed in order to convey the irrigation water exploited in the Sero river system to the Langkemme main canal. The diversion canal runs in the skirt of the mountain ranges in the left bank of the Sero river. The design discharge of the irrigation water to be conveyed through the Sero diversion canal would be 2.5 m^3/sec.

The link canal is defined as the canal which conveys the water diverted at integrated weir to integrated tertiary blocks. Some link canals would be joined with the Langkemme main canal in order to effectively use the surplus water from tributaries and/or to supplement irrigation water from the main canal. The design water discharge of the link canal would range from 0.9 m^3/sec. to 0.1 m^3/sec. The supplemental water for some secondary blocks is released into the tributaries of the Walanae river and conveyed through the natural river course to the sites of integrated weirs which are constructed in the downstreams of the tributaries.

The offtake channels are aligned in order to convey the water from the Unyi and the Pising intake site to the Sero diversion canal.

The tertiary canal is defined as the canal which conveys water diverted from the link canal to distribute the irrigation water to the quarternary blocks.

The total length of each canal to be provided in the project is shown as follows.

Canals	Total length (Km)
1. Langkemme Main Canal	29.5
2. Sero Diversion Canal	14.9
3. Link Canal	39.5
4. Offtake Channel	0.6
5. Tertiary Canal	51.0
Total	135.5 (Km)

The canal system of the project and the general layout of the canals are shown in Fig. 4.2.6 and the ANNEX IV. The profiles of the Langkemme main canal and the Sero diversion canal are also shown in the ANNEX IV.

(2) Comparative Study on the Main Canal Route

The topographic condition in the reach of the main canal between BL1-4 Culvert and BL-2 Turnout is sharply undulating and steep. The average slope in this reach is estimated as 1 to 2.5. It is considered that construction cost for open channel in such topography become excessively high. In order to select the optimum route of the main canal in the reach between BL1-4 culvert and BL-2 turnout, two alternative routes are prepared as shown in Fig. 4.2.7 and the cost comparison is made on the alternatives.

To estimate the quantity of earth works and numbers of required structures, profile of each alternative route is prepared based on the result of the route survey and the contoured map in scale of 1/5,000. The profile of each alternative and the typical section of canals and tunnel are shown in Fig. 4.2.8 and Fig. 4.2.9. The main features of each alternative routes are summarized as follows:

Main features	Alternative I	Alternative II
1. Canal length (km)	1.8	5.8
2. Length of Tunnel (km)	0.72	-
3. Nos. of Canal Structures		
- Cross drain	-	5
- Culvert	-	1
- Junction	1	1

The quantities of earth excavation and embankment are estimated on the assumed section and the said profile. Excavation is classified into rock, common, and tunnel based on the result of the topographic survey and the soil mechanic investigation along the alternative routes.

The construction cost of each alternative is estimated as brokendown in Table 4.2.8 and summarized as follows:

		(10^3 x Rp.)
Work item	Alternative I	Alternative II
1. Earth Work	137,370	979,300
2. Related Structures	1,000	19,500
3. Tunnel	590,400	-
Total	728,770	998,800

The Alternative-I is more economical than the Alternative-II as shown above. Furthermore, geological investigation clarifies that the route of Alternative-I is considerably favourable for tunnel excavation. The Alternative-I is technically and economically more eligible than the Alternative-II and would be proposed as the optimum route.

(3) Alternative Study on Secondary System

As regards the link canal system to be constructed in the project, the study is made on the following three alternatives in view of construction cost, water management, and effective use of water resources. Each alternative plan is briefly discribed as follows:

(Alternative-1) Existing intake structure would be stabilized with masonry works. The irrigation water is directly released into natural tributaries from the Langkemme main canal. The irrigation water conveyed through the natural river course is diverted into tertiary blocks by the rehabilitated intake structure of respective tertiary block. No link canal is aligned in principle.

(Alternative-2) The irrigation water is distributed to tertiary blocks through the link canal which would be constructed to join the Langkemme main canal with the respective tertiary block. Instead, most of the existing intakes would not be rehabilitated.

(Alternative-3) This system is a combined system of alternative-1 and - 2. The irrigation water is distributed to the tertiary blocks through the link canals and the tributaries. In case of the later, the existing intake structure would be stabilized for diverting the water conveyed through the tributaries. Prior to making layout of the Alternative-3, length of link canal which is equivalent to construction cost of one intake weir is worked out to obtain the minimum construction cost as summarized in Table 4.2.9.

The main features and direct construction costs are roughly estimated on the respective alternative as shown in Table 4.2.10 and are summarized as follows:

(Unit: 10^3Rp)

	Direct Construction Cost
Alternative 1	491,565
Alternative 2	1,487,845
Alternative 3	989,465

In view of construction cost, the Alternative-1 is more economical than the other, but is not necessarily appropriate for better management and effective use of water resources. From the overall viewpoints, the alternative-3 would be proposed as the optimum secondary system of the Langkemme Irrigation Project.

(4) Design of Canals

All canals proposed for the project are earth canals. In order to prevent the erosion of the side slope, the allowable maximum velocity in the canals is designed to be 0.7 m/sec. Based on the results of the study in ANNEX III, CHAPTER VII, the side slope of embanked is designed to be 1 to 1.5. As regards the excavated canal, the side slope is designed to be 1 to 1.5 and 1 to 1.0 in the alluvial and deluvial deposits. Meanwhile, in the hilly region where the weathered rock and/or base rock is outcropped, the side slopes of 1 to 0.5 and 1 to 0.3 are adopted. Depending on the side slope, the canals proposed for the project are divided into four types. Type I to III canals are mainly adopted to the Langkemme main canal and the Sero diversion canal and type IV is adopted to link canals and tertiary canals. The hydraulic features and typical cross section of each type of canal are shown in Table 4.2.11 to 4.2.14 and the ANNEX IV.

The hydraulic calculation in the design of canals is made by using the manning's formula as follows.

$$Q = A \cdot V$$

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

Where, Q : Discharge (m^3/sec)
A : Flow area (m^2)
V : Velocity (m/sec)
R : Hydraulic radius (m)
I : Hydraulic gradient
n : Manning's coefficient of roughness

The freeboard and the width of berm are designed based on the criteria of L.P.M.A (Institute of Research for Hydraulics and Hydrology, Bandung) which is summarized in Table 4.2.15. The inspection road of 3.5 m wide is installed along the main canal and the link canal in case that the ground slope is less than 1 to 7.

In case that a canal is constructed in the teracce deposit along the river and/or limestone bed rock, lining work is made by wet masonry of 0.3 m thick to prevent seepage from the canal banks and bottom and to protect the side slopes against erosion. Total length of canal to be lined is about 4.0 km in the Langkemme main canal and the Sero diversion canal. The turfing is proposed for the Langkemme main canal and the Sero diversion canal to protect the erosion of the canal slope.

4.2.4 Canal Related Structures

Various related structures would be provided for crossing of hill, road, and tributaries, regulating and control of discharge, and distribution of irrigation water. The structures proposed for the main canal are broadly categorized as listed below:

(A) Crossing facilities

i. Culvert
ii. Tunnel
iii. Syphon & Aqueduct
iv. Cross drain

(B) Regulating and control facilities

i. Check
ii. Spillway
iii. Drop and Chute
iv. Junction

(C) Distribution facilities

i. Turnout
ii. Release structure

The required numbers of the canal related structures are summarized in Table 4.2.16 and the general characteristics and design criteria of those structures are mentioned hereinafter.

(1) <u>Turnout</u>

Turnout structure is constructed to distribute the required water from a parent canal to a branching canal. Turnout structure is designed to be combined with a release structure and/or a waste way. In the proposed canal system, four types of turnout are proposed depending on their combination.

A slide gate is provided at the outlet of the structure and a notch with a staff gauge is installed as a measuring divice. In case that the no release structure and/or wasteway is combined, a reinforced concrete pipe is laid under canal embankment to distribute irrigation water. The details are shown in the ANNEX IV.

(2) <u>Check Structure</u>

Check structure is provided at the just downstream of a turnout and/or a spillway to raise the water surface to the required level during the period of small discharge. The check structure would regalate the water supply and aid to drain out the excess water flow out through a spillway. The check structures are divided into three types depending on the discharge in the canal.

The transition of five m long is provided in the up and down streams of the structure respectively to minimize the head loss. Two slide gates are installed to regulate water surface in the upstream of canal. The details are shown in the ANNEX IV.

(3) <u>Culverts</u>

Culverts are provided for the road crossing. The culverts in the proposed canal system are wet-masoned and divided into two type, one has the double barrels of rectangular section and another has single barrel. The former is further divided into four types depending on the discharge in the canal.

Design water depth in the conduit is taken to be about 80 % of conduit height. And the crown of conduit is designed to be equal to road surface.

Transition of about five m long is provided in the up and down streams of the culvert respectively to minimize the head loss. The conduit is of reinforced concrete to bear the passing load on it. The details of the culverts are shown in the ANNEX IV.

(4) Tunnel

As described hereinbefore a tunnel of 720 m long is proposed to minimize the construction cost of the main canal.

The free-flow tunnel of 2R-horse-shoe type (R=1.25 m) is introduced. The design discharge is 4.75 m^3/s and design velocity in the conduit is defined to be less than 1.1 m/sec. The water depth in the conduit is taken to be 80 % of 2R and the earth cover over the conduit is more than 10 m.

The crown of tunnel is lined with concrete and the walls and invert, with wet stone masonry. At the both portals of tunnel, the conduit of 10 m long is lined with concrete only. The drain with cobble is employed under invert to control the ground water pressure on the tunnel lining. Transition of about 10 m long is provided at the both portals of tunnel respectively to minimize the head loss. A cross section is shown in the ANNEX IV.

(5) Syphon and Aqueduct

Inverted syphons and aqueducts are provided in the proposed canal system for river crossing. The syphon is superior to the aqueduct in view of the structural stability for being free from seismic load and passage of the river flood, but requires high head loss and complicated maintenance.

The Langkemme main canal would run across the some tributeries. Syphons are provided in the Langkemme main canal system so as to be free from unforeseen flood. While, in the Sero diversion canal system, aqueducts are proposed in principle to minimize the head loss.

Six syphons provided in the Langkemme main canal are divided into two types depending on the design discharge; one has a reinforced concrete barrel with a circular section and another has a barrel with a rectangular section. The former is adopted in case the design discharge is less than 2.0 m^3/s. The transition is provided in the up and down streams of the syphon to minimize the head loss. The design velocity in the conduit is defined to be about 1.5 times of the velocity in the upstream canal. The earth cover over the barrel is greater than one meter. Details are shown in the ANNEX IV.

Three aqueducts are proposed in the Sero diversion canal and one aqueduct, in the Langkemme main canal. An aqueduct consists of reinforced concrete duct with a rectangular section, abutments of wet stone masonry and transitions installed in the up and down streams of the structure. Two types of duct are adopted depending on the design discharge, one has a section of 3.0 m x 1.5 m and another has a section of 2.0 m x 1.5 m. Required numbers of piers depends on the cross section of river. The maximum span of duct is limited to be 8 m. The abutments are constructed on the base rock and the river section under the aqueduct is protected by the gabion mats. The design velocity is defined to be around 1.0 m/sec. The water depth in the duct is adopted 80 % of the height of the duct. The details are shown in the ANNEX IV.

(6) Cross Drain

A number of cross drains are provided to drain excess runoff from hill sides extending along canal route. Cross drains are divided into two types depending on the design discharge, one has a wet stone masonry barrel with a rectangular section and another has a reinforced concrete barrel with a circular section. The former is adopted in case the design discharge is more than 2.5 m^3/s and the later, less than 2.5 m^3/sec. Slab of the barrel is of reinforced concrete. The inlet and outlet of the structure are protected by wet stone masonry. The bottom and side slopes of canal above a cross drain structure are protected by wet stone masonry. The details are shown in the ANNEX IV.

(7) Spillway and Wasteway

Side spillways are provided in the proposed canal system to automatically spillout excess water from the canal when the water surface in the canal exceeds designed high water level. The spillway would prevent canal and related structures from damages to be caused by over-topping of canal water. Three types of spillway are adopted depending on the design discharge of water to be spilled out. The structure is constructed with wet stone masonry and the guide channel is provided to convey the spilled water to natural river. Details are shown in the ANNEX IV.

Wasteways are provided to empty the canal for it's maintenance. The wasteway is generally constructed in combination with a turnout structure. A slide gate is installed at the outlet of the structure and a guide channel is provided to convey the water to the natural river. Details are shown in the ANNEX IV.

(8) Drop and Chute

Drop structures are divided into seven types depending on the design discharge. A drop structure consists of inlet transition, steep portion with a rectangular section, stilling pool and outlet transition. The inlet transition is hardly protected by wet stone masonry to prevent canal from scouring. The steep portion is inclined of 1 : 1.0. The length of the stilling pool is designed based on the standard of L.P.M.A. Details are shown in the ANNEX IV.

A chute is constructed at the proposed site for a micro hydropower station which would be constructed in the middle reach of the Langkemme main canal after completion of the project. The chute is provided in connection with the structures of the micro hydropower station as shown in the ANNEX IV. After completion of the micro hydropower station, the chute can be used as a bypass of the penstock of the hydropower station.

The chute consists of the inlet structure, chute channel and stilling pool. A check weir is installed at the inlet of the chute channel to prevent the racing upstream and scouring of the canal, and the inlet section of about 0.7 m x 1.5 m is designed to discharge the design water of 2.9 m^3/sec into the chute channel with normal depth of the upstream canal. The chute channel is 1.5 m wide and incline with a slope of 1 : 1.20. The stilling pool is 1.0 m deep and about 7.5 m long. In the stilling pool, chute blocks of 0.2 m wide each and floor blocks of 0.3 m wide each are installed.

(9) Others

Junction structures are provided to protect the Main Canal from scouring at the connecting point with the link canal which conveys water from integrated weir to the main canal.

Release structure is constructed in combination with a turnout to distribute the required water to the integrated weir through the natural river. A slide gate is installed at the inlet of the structure and a notch with a staff gauge is installed as a measuring device. The guide channel is provided to release water into the natural river.

4.3 CONSTRUCTION COST ESTIMATE

4.3.1 General

The following assumptions are made for the cost estimate of the project;

(1) The conversion rate between Rupiah and U.S. dollar is assumed at US$ 1.00 = Rp.625 as of November, 1980.

(2) All of the construction works would be executed by full contract basis; major works, such as the Langkemme main canal system and the Sero diversion canal system would be constructed by foreign contractors and the remaining minor works, by local contractors. The machinery and equipments required for construction works would be provided by the contractors themselves. Therefore, depreciation cost of machinery and equipments would be taken into account of the construction cost, instead of the procurement cost of machinery and equipments.

(3) Tax exemptions are made for the construction materials to be imported.

(4) Unit price is divided into foreign and local currency portions. Local currency portion is estimated on the basis of the current price in South Sulawesi in late 1980 and of the data obtained from the ongoing and completed irrigation project around the project area. Foreign currency portion is estimated based on the CIF prices at Ujung Pandang. The currency is classified into local and foreign portions according to the following criteria;

Local Currency Portion

i. Labour wage
ii. Wooden materials such as boad, prank, pile.,
iii. Fuel, oil etc.,
iv. Inland transportation charge
v. Transfer payment for local portions, such as general expences, profit.,
vi. Cement

Foreign Currency Portion

i. Reinforcement bar
ii. Metal works such as steel gates, structual steel
iii. Transfer payment for foreign currency.
iv. Expense and fees of engineering services by foreign consultants.
v. Depreciation cost of machinery and equipment.

(5) For the construction of the quarternary network, only the costs of material are included in the construction cost. The construction works of the quarternary network are carried out by water users' association concerned under the guidance of the local Government and the project O/M office.

(6) The associated costs to be financed by the government such as for extension services, for improvement of social infrastructure, are not taken into account for the cost estimate.

4.3.2 Cost Estimate

(1) Unit Cost

Prior to unit cost analysis, construction plan and method are set up based on the selection of the proper numbers and size of constsruction equipment. Unit cost analyses are detaily made taking into account of such depreciation cost and operation charge of the required equipments. The analyses are made at 1980 price level and summarized in Data Book.

(2) Project Cost

Project cost is estimated on the basis of quantity-taking of the project works obtained from the drawings and the respective unit costs. The total project cost is estimated at about US$34.6 million, comprising US$14.5 million of foreign currency portion and US$20.1 million equivalence of local currency portion, at financial basis including construction cost, engineering and administration cost, and price contingency. The cost is summarized in Table 4.3.1 and the further breakdowns are given in Table 4.3.2.

4.3.3 Operation and Maintenance Cost

Operation and maintenance cost of the project mainly comprises the expences for the project O/M offices including personnel cost as well as maintenance cost of the project facilities. These costs are estimated at about US$0.472 million equivalence on reference to the proposed organizations for the operation and maintenance of the project. The breakdown of the cost is given in Table 4.3.3.

4.3.4. Replacement Cost

Steel gates provided for intake weirs and canal related structures, other metal works and wires for gabion works would have to be periodically replaced throughout a period of the project life. The replacement costs of respective materials are estimated as shown in Table 4.3.4, according to the economic life of respective equipments to be replaced.

4.3.5 Disbursement of Construction Cost

The annual disbursement schedule of the construction cost is worked out as shown in Table 4.3.5, according to the construction time schedule which is compiled in the CHAPTER V. The disbursement can be summarized as follows:

Year	Local Currency (10^3 US$)	Foreign Currency (10^3 USB)	Total (10^3 US$)
1982	238.7	967.6	1,206.3
1983	3,542.9	2,645.8	6,188.7
1984	5,335.8	3,647.8	8,983.6
1985	4,136.1	4,073.2	8,209.3
1986	6,481.8	3,093.8	9,575.6
1987	323.1	87.2	410.3
Total	20,058.4	14,515.4	34,573.8

Table 4.1.1 Existing Irrigation Scheme

Inventory of Sheme	Name of Scheme	Grade of System	Net Irrigable Area (ha)	Canal Density (m/ha)	Inventory of Intake Structure	Grade of Intake Structure
2.	Cennae	Semi-Technical	210	20	2	C
3.	(Madenra-I)	Non-Technical	9	50	3	C
4.	(Madenra kanan)	Non-Technical	23	50	3,4	C
5.	Madenra	Non-Technical	26	30	4	C
6.	(Tokebbeng kiri)	Non-Technical	14	40	5,6	C
7.	Tokebbeng	Non-Technical	106	20	5,8	C
8.	(Congko I)	Non-Technical	61	10	7,9	C
9.	(Congko II)	Non-Technical	37	20	10	C
10.	(Pakkali kanan)	Non-Technical	50	30	11,12,13,15	A(11), C(12,13,15)
11.	Pakkali	Semi-Technical	65	100	11,14,16,17	A(11), C(14,16,17)
12.	(Labessi kanan)	Non-Technical	45	20	18	C
13.	Labessi-I	Semi-Technical	186	50	19,20,23	A(20,23), C(19)
14.	Latasi	Semi-Technical	43	50	20,21,22	A(22), C(20,21)
15.	Kadeppe	Non-Technical	54	50	33	C
17.	Timusu	Non-Technical	131	60	34,35	C
18.	Tenga Padange-I	Non-Technical	70	50	36,37	C
19.	Tenga Padange-II	Non-Technical	204	30	38	C
20.	Kalempang	Non-Technical	113	40	39	C
21.	Attebunge	Non-Technical	156	40	40	C
22.	Labessi-II	Semi-Technical	118	20	24,26	A(24,Aqueduct), C(26)
23.	(Kalempang-I)	Non-Technical	18	30	41	C
24.	(Jawi-Jawie)	Non-Technical	86	40	42,43,44	C
25.	(Pattojo)	Non-Technical	29	90	45	A
26.	Toasiabeng	Non-Technical	97	30	50	C
27.	Latana	Non-Technical	236	30	48,49,51,52	C
28.	Lamogo	Semi-Technical	478	30	54	A
29.	Ompo Pattojo	Semi-Technical	175	10	46,47	C
30.	Kompe Cadung	Semi-Technical	90	30	53	C
31.	Centanae	Semi-Technical	41	20	28,29	C
32.	(Lamanggarae)	Non-Technical	26	20	30	C
33.	Togigi	Non-Technical	60	50	55	C
34.	(Kubba kanan)	Non-Technical	93	30	56	C
35.	Kubba	Non-Technical	46	20	57	C
36.	Talagae	Non-Technical	60	80	58	C
37.	Maccope-I	Non-Technical	377	20	60,64	C
38.	Maccope-II	Non-Technical	24	40	60	C
39.	Pasaammeng	Non-Technical	64	40	61,65	C
40.	Talagae	Non-Technical	196	20	58	C
41.	Mattimpajoe	Non-Technical	180	10	32	C
42.	Akampeng-I	Semi-Technical	80	70	67	C
43.	Belo	Semi-Technical	82	50	59	A
44.	(Soppeng)	Non-Technical	48	60	63	C
45.	(Pasaammeng kiri)	Non-Technical	72	30	62	C
50.	Akampeng-II	Semi-Technical	868	30	68	A
51.	Lalange	Semi-Technical	700	20	25,27	A(27), B(25)
52.	Cangadi	Non-Technical	60	10	-	-
53.	Lagarigi	Semi-Technical	332	20	31	A
55.	Malanroe	Non-Technical	46	60	66	C

Summary :

Scheme	Grade of System	Nos. of Scheme	Net Irrigable Area
1. Desa irrigation	Non-Technical	34	2,900
	Semi-Technical	10	1,400
(Sub-total)		44	4,300
2. DPU irrigation	Semi-Tec-nical	4	2,100
(Total)		48	6,400

Table 4.1.2 Canal Density under the Existing Irrigation Scheme

Inventory of Scheme	Name of Scheme	Irrigation Area (ha)	Tertiary Canal Length (m)	Quarternary Canal Length (m)	Total Canal Length (m)	Canal Density (m/ha)
2.	Cennae	210	1,140	3,060	4,200	20
3.	(Madenra I)	9	420	0	420	50
4.	(Madenra kanan)	23	1,240	0	1,240	50
5.	Madenra	26	770	0	770	30
6.	(Tokebbeng kiri)	14	580	0	580	40
7.	Tokebbeng	106	2,510	0	2,510	20
8.	(Congko I)	61	340	520	860	10
9.	(Congko II)	37	820	0	820	20
10.	(Pakkali kanan)	50	240	1,200	1,440	30
11.	Pakkali	65	4,550	2,070	6,620	100
12.	(Labessi kanan)	45	670	170	840	20
13.	Labessi-I	186	3,380	5,670	9,050	50
14.	Latasi	43	1,740	360	2,100	50
15.	Kadeppe	54	880	1,700	2,580	50
17.	Timusu	131	5,600	1,960	7,560	60
18.	Tenga Padange-I	70	3,320	0	3,320	50
19.	Tenga Padange-II	204	2,720	3,350	6,070	30
20.	Kalempang	113	3,700	620	4,320	40
21.	Attebunge	156	3,010	2,700	5,710	40
22.	Labessi-II	118	2,400	120	2,520	20
23.	(Kalempang I)	18	600	0	600	30
24.	(Jawi-Jawie)	86	2,100	1,530	3,630	40
25.	(Pattojo)	29	2,170	280	2,450	90
26.	Tossiabeng	97	1,970	750	2,720	30
27.	Latana	236	6,570	920	7,490	30
28.	Lamogo	478	1,080	13,400	14,480	30
29.	Ompo Pattojo	175	1,120	180	1,300	10
30.	Rompe Gading	90	2,510	140	2,650	30
31.	Cenranae	41	630	0	630	20
32.	(Lamanggarae)	26	420	110	530	20
33.	Togigi	60	2,950	0	2,950	50
34.	(Kubba kanan)	93	870	1,980	2,850	30
35.	Kubba	46	340	530	870	20
36.	Talagae	60	3,650	880	4,530	80
37.	Maccope-I	377	6,580	1,540	8,120	20
38.	Maccope-II	24	1,050	0	1,050	40
39.	Passammeng	64	2,820	0	2,820	40
40.	Talagae	196	3,470	190	3,660	20
41.	Mattimpajoe	180	1,870	540	2,410	10
42.	Akampeng-I	80	1,500	4,080	5,580	70
43.	Belo	82	2,640	1,630	4,270	50
44.	(Soppeng)	48	2,660	230	2,890	60
45.	(Passammeng kiri)	72	1,830	320	2,150	30
50.	Akampeng-II	868	6,330	8,500	14,830	30
51.	Lalange	700	2,720	3,210	5,930	20
52.	Cangadi	60	190	200	390	10
53.	Lagarigi	332	4,330	1,960	6,290	20
55.	Malanroe	46	1,210	1,490	2,700	60

Table 4.1.3 Extra Terrestrial Radiation (Ra)

(mm/H20/day)

Northern Hemisphere

Latitude	Jan.	Feb.	Mar.	Apr.	May	Jun.	Jul.	Aug.	Sep.	Oct.	Nov.	Dec.
10°	13.2	14.2	15.3	15.7	15.5	15.3	15.3	15.5	15.3	14.7	13.6	12.9
8	13.6	14.5	15.3	15.6	15.3	15.0	15.1	15.4	15.3	14.8	13.9	13.3
6	13.9	14.8	15.4	15.4	15.1	14.7	14.9	15.2	15.3	15.0	14.2	13.7
4	14.3	15.0	15.5	15.5	14.9	14.4	14.6	15.1	15.3	15.1	14.5	14.1
2	14.7	15.3	15.6	15.3	14.6	14.2	14.3	14.9	15.3	15.3	14.8	14.4
0	15.0	15.5	15.7	15.3	14.4	13.9	14.1	14.8	15.3	15.4	15.1	14.8

(mm/H20/day)

Southern Hemisphere

Latitude	Jan.	Feb.	Mar.	Apr.	May	Jun.	Jul.	Aug.	Sep.	Oct.	Nov.	Dec.
10°	16.4	16.3	15.5	14.2	12.8	12.0	12.4	13.5	14.8	15.9	16.2	16.2
8	16.1	16.1	15.5	14.4	13.1	12.4	12.7	13.7	14.9	15.8	16.0	16.0
6	15.8	16.0	15.6	14.7	13.4	12.8	13.1	14.0	15.0	15.7	15.8	15.7
4	15.5	15.8	15.6	14.9	13.8	13.2	13.4	14.3	15.1	15.6	15.5	15.4
2	15.3	15.7	15.7	15.1	14.1	13.5	13.7	14.5	15.2	15.5	15.3	15.1
0	15.0	15.5	15.7	15.3	14.4	13.9	14.1	14.8	15.3	15.4	15.1	14.8

Table 4.1.4 Experimentally Determined Constants for the Radiation Equation

$$Rs = (a + b.n/N).Ra$$

Source		Location or Range of Locations	Constants a	Constants b	Latitude (°)
As listed by Linacre (1967)					
Fitzpatrick	(1965)	Kimberley, S.Africa	0.33	0.43	16 S
Cockett et al.	(1964)	Central Africa	0.32	0.47	15 S
Page	(1961)	Dakar, Senegal	0.10	0.70	15 N
Yadov	(1965)	Madras, India	0.31	0.49	13 N
Davies	(1965)	Kano, Nigeria	0.26	0.54	12 N
Smith	(1960)	Trinidad	0.27	0.49	11 N
Stanhill	(1963)	Benin City/1, Nigeria	0.26	0.38	7 N
		Mean	0.26	0.50	13°
Davies	(1965)	Accra, Ghana	0.30	0.37	6 N
Black et al.	(1954)	Batavia (Djakarta)	0.29	0.59/2	6 S
Page	(1961)	Kinshasa, Zaire	0.21	0.52	4 S
Page	(1961)	Singapore	0.21	0.48	1 N
Glover et al.	(1958b)	Kabete, Kenya	0.24	0.59	1 S
Page	(1961)	Kisangani, Zaire	0.28	0.40	1 S
Rijks et al.	(1964)	Kampala, Uganda	0.24	0.46	0
		Mean	0.25	0.49/3	3°

/1 : Davies (1965) gave 0.28 and 0.33 for a and b respectively

/2 : Table by Linacre (1967) indicated 0.29 for Batavia, a likely error since Chidley and Pike (1970) give 0.59 for Djakarta, the same location

/3 : Based on revised figure for Batavia

Table 4.1.5 Maximum Possible Sunshine Hours (N) for Different Months and Latitudes

Northern Lats	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sep.	Oct.	Nov.	Dec.
Southern Lats	**Jul.**	**Aug.**	**Sep.**	**Oct.**	**Nov.**	**Dec.**	**Jan.**	**Feb.**	**Mar.**	**Apr.**	**May**	**Jun.**
50°	8.5	10.1	11.8	13.8	15.4	16.3	15.9	14.5	12.7	10.8	9.1	8.1
48	8.8	10.2	11.8	13.6	15.2	16.0	15.6	14.3	12.6	10.9	9.3	8.3
46	9.1	10.4	11.9	13.5	14.9	15.7	15.4	14.2	12.6	10.9	9.5	8.7
44	9.3	10.5	11.9	13.4	14.7	15.4	15.2	14.0	12.6	11.0	9.7	8.9
42	9.4	10.6	11.9	13.4	14.6	15.2	14.9	13.9	12.6	11.1	9.8	9.1
40	9.6	10.7	11.9	13.3	14.4	15.0	14.7	13.7	12.5	11.2	10.0	9.3
35	10.1	11.0	11.9	13.1	14.0	14.5	14.3	13.5	12.4	11.3	10.3	9.8
30	10.4	11.1	12.0	12.9	13.6	14.0	13.9	13.2	12.4	11.5	10.6	10.2
25	10.7	11.3	12.0	12.7	13.3	13.7	13.5	13.0	12.3	11.6	10.9	10.6
20	11.0	11.5	12.0	12.6	13.1	13.3	13.2	12.8	12.3	11.7	11.2	10.9
15	11.3	11.6	12.0	12.5	12.8	13.0	12.9	12.6	12.2	11.8	11.4	11.2
10	11.6	11.8	12.0	12.3	12.6	12.7	12.6	12.4	12.1	11.8	11.6	11.5
5	11.8	11.9	12.0	12.2	12.3	12.4	12.3	12.3	12.1	12.0	11.9	11.8
0	12.0	12.0	12.0	12.0	12.0	12.0	12.0	12.0	12.0	12.0	12.0	12.0

Table 4.1.6 Reflection Coefficient (r)

	Water Surface	Surface covered with Crops
r	0.05 - 0.07	0.15 - 0.25

Table 4.1.7 Values of $\sigma\ Ta^4$ for Various Temperatures for the Penman Method

Temperature, Ta	$\sigma\ Ta^4$	Temperature, Ta^4	$\sigma\ Ta^4$
K (°Abs)	mm H_2O/day	°F	mm H_2O/day
270	10.73	35	11.48
275	11.51	40	11.96
280	12.40	45	12.45
285	13.20	50	12.94
290	14.26	55	13.45
295	15.30	60	13.96
300	16.34	65	14.52
305	17.46	70	15.10
310	18.60	75	15.65
315	19.85	80	16.25
320	21.15	85	16.85
325	22.50	90	17.46
		95	18.10
		100	18.80

Table 4.1.8 Calculation Sheet of Potential Evapotranspiration (ETo) Penman Method

Penman Method

ETo = (△. H + 0.27 . Ea) / (△ + 0.27)

	Jan.	Feb.	Mar.	Apr.	May	Jun.	Jul.	Aug.	Sep.	Oct.	Nov.	Dec.
1. Mean monthly temperature : t (°C)	27.9	28.0	27.7	27.6	27.4	26.5	26.1	26.3	26.7	28.2	27.9	27.6
(°F)	(82.2)	(82.4)	(81.9)	(81.7)	(81.3)	(79.7)	(79.0)	(79.3)	(80.1)	(82.8)	(82.2)	(81.7)
2. Mean monthly realative humidity : Hm	0.79	0.78	0.78	0.81	0.80	0.82	0.79	0.75	0.76	0.73	0.77	0.81
3. Sunshine rate : $\frac{n}{N}$ (= S)	0.41	0.43	0.46	0.50	0.55	0.44	0.56	0.63	0.65	0.66	0.57	0.43
4. Wind velocity : U_2 (Mile/day)	91.23	75.13	64.40	53.66	64.40	64.40	80.50	85.86	80.50	69.76	53.66	144.89
5. Mean monthly extra terrestrial radiation : Ra (mmH_2O/day)	15.5	15.8	15.6	14.9	13.8	13.2	13.4	14.3	15.1	15.6	15.5	15.4
6. Reflection coefficient : r = 0.25												
7. (1 - r) = 0.75												
8. A = (a + b·$\frac{n}{N}$) , (= 0.21 + 0.52 x 3)	0.42	0.43	0.45	0.47	0.50	0.44	0.50	0.54	0.55	0.55	0.51	0.43
9. B = Ra·(1-r)·(a+b·$\frac{n}{N}$) , (= 5 x 7 x 8)	4.92	5.14	5.26	5.25	5.13	4.34	5.04	5.77	6.21	6.47	5.89	5.01
10. Vapor pressure												
(a) Saturated , ea (See Figure with t°C)	28	28	28	27	27	27	26	26	27	28	28	28
(b) Actual , ed = Hm x ea (= 2 x 10a)	22	22	22	22	22	22	21	20	21	20	21	23
(c) $\sqrt{ed}$	4.70	4.69	4.67	4.67	4.65	4.71	4.53	4.42	4.54	4.52	4.63	4.76
11. $\sigma \cdot Ta^4$ (see Table with °F = 1.8·t° + 32)	16.5	16.5	16.4	16.4	16.4	16.1	16.1	16.1	16.3	16.5	16.5	16.4
12. (0.56 - 0.092·$\sqrt{ed}$), (= 0.56 - 0.092 x 10c)												
13. (0.10 + 0.90 · $\frac{n}{N}$) , (= 0.10 + 0.90 x 3)												
14. I = $\sigma \cdot Ta^4$· (0.56 - 0.092 · $\sqrt{ed}$)·(0.10+0.90·$\frac{n}{N}$) . (= 11 x 12 x 13)	0.99	1.04	1.10	1.17	1.29	1.01	1.39	1.65	1.59	1.65	1.36	0.98
15. H = Ra·(1-r)·(0.21 + 0.52·$\frac{n}{N}$) - ·Ta^4 (0.56 - 0.092·$\sqrt{ed}$)·(0.010 + 0.90·$\frac{n}{N}$), (= 9 - 14)	3.93	4.10	4.15	4.08	3.84	3.34	3.65	4.12	4.62	4.82	4.53	4.03
16. 0.35·(ea - ed), (= 0.35 x (10 - 10b))												
17. (1 + 0.0098·U_2), (= 1 + 0.0098 x 4)												
18. Ea = 0.35·(ea - ed)·(1 + 0.0098·U_2), (= 16 x 17)	3.94	3.68	3.56	2.75	3.07	2.73	3.42	4.16	3.99	4.45	3.50	4.55
19. △ (see Figure with t°C)	0.96	0.96	0.96	0.96	0.88	0.88	0.84	0.84	0.88	0.96	0.96	0.96
20. △ ·H, (= 19 x 15)	3.77	3.94	3.98	3.92	3.38	2.94	3.07	3.46	4.07	4.63	4.35	3.87
21. 0.27·Ea, (= 0.27 x 18)	1.06	0.99	0.96	0.74	0.83	0.74	0.92	1.12	1.08	1.20	0.95	1.23
22. ·H + 0.27·Ea, (= 20 + 21)	4.83	4.93	4.94	4.66	4.21	3.68	3.99	4.58	5.15	5.83	5.30	5.10
23. + 0.27, (= 19 + 0.27)	1.23	1.23	1.23	1.23	1.15	1.15	1.11	1.11	1.15	1.23	1.23	1.23
24. (△·H + 0.27·Ea)/(△ + 0.27), (= 22 / 23) : Eto	3.93	4.01	4.02	3.79	3.66	3.20	3.59	4.13	4.48	4.74	4.31	4.15

Table 4.1.9 <u>Monthly Day-time Coefficient, D</u>

(only for use with Hargreaves equations)

Latitude degree	Jan.	Feb.	Mar.	Apr.	May	Jun.	Jul.	Aug.	Sep.	Oct.	Nov.	Dec.
North												
10	0.97	0.89	1.01	1.01	1.06	1.03	1.06	1.05	0.99	0.99	0.95	0.97
8	0.98	0.89	1.01	1.01	1.05	1.02	1.05	1.04	0.99	0.99	0.95	0.97
6	0.98	0.90	1.01	1.01	1.05	1.02	1.05	1.04	0.99	1.01	0.95	0.98
4	0.98	0.91	1.02	1.00	1.04	1.01	1.04	1.04	0.99	1.01	0.95	0.98
2	1.01	0.91	1.02	0.99	1.02	0.99	1.02	1.02	0.98	1.02	0.98	1.01
0	1.02	0.92	1.02	1.00	1.02	0.99	1.02	1.02	0.98	1.02	0.99	1.02
South												
2	1.02	0.93	1.02	0.98	1.01	0.98	1.01	1.01	0.98	1.02	0.99	1.03
4	1.04	0.93	1.02	0.98	1.01	0.97	0.98	1.01	0.98	1.03	1.00	1.04
6	1.05	0.94	1.02	0.97	1.00	0.96	0.98	1.00	0.98	1.03	1.01	1.05
8	1.05	0.94	1.02	0.97	0.99	0.95	0.98	1.00	0.98	1.02	1.02	1.06
10	1.06	0.94	1.02	0.97	0.98	0.94	0.97	0.99	0.98	1.04	1.02	1.07

Table 4.1.10 Calculation Sheet of Potential Evapotranspiration (ETo) Christiensen - Hargreaves Method

Christiensen - Hargreaves Method

ETo = 17.4 · D · Tc · (Fh · Fw · Fs · Fc)

	Jan.	Feb.	Mar.	Apr.	May	Jun.	Jul.	Aug.	Sep.	Oct.	Nov.	Dec.
1. Monthly day-time coefficient ; D	1.04	0.93	1.02	0.98	1.01	0.97	0.98	1.01	0.90	1.03	1.00	1.04
2. Mean monthly temperature ; Tc (°C)	27.9	28.0	27.7	27.6	27.4	26.5	26.1	26.3	26.7	28.2	27.9	27.6
3. Mean daily relative humidity ; Hm	0.79	0.78	0.78	0.81	0.80	0.82	0.79	0.75	0.76	0.73	0.77	0.81
4. Mean wind velocity ; Wkd (Km/day)	146.88	120.96	103.68	86.4	103.68	103.68	129.6	138.24	129.6	112.32	86.4	233.28
5. Mean monthly sunshine hour ; S	0.41	0.43	0.46	0.50	0.53	0.44	0.56	0.63	0.65	0.66	0.57	- 0.43
6. Elevation above the sea level ; E (= 50m)												
7. Mean noon humidity ; Hn (%) (= 0.4 Hm+0.6Hm^2)	0.69	0.68	0.673	0.72	0.71	0.74	0.69	0.64	0.66	0.61	0.66	0.71
8. Fh (= 0.59 - 0.55 Hn^2)	0.33	0.33	0.34	0.31	0.32	0.29	0.33	0.36	0.35	0.38	0.35	0.31
9. Fw (= 0.75 + 0.0255 $\sqrt{Wkd}$)	1.06	1.03	1.01	0.99	1.01	1.01	1.04	1.05	1.04	1.02	0.99	1.14
10. Fs (= 0.478 + 0.58 S)	0.72	0.73	0.74	0.77	0.80	0.73	0.80	0.84	0.86	0.86	0.81	0.73
11. Fc (= 0.950 + 0.0001 E)	0.96	0.96	0.96	0.96	0.96	0.96	0.96	0.96	0.96	0.96	0.96	0.96
12. Fh·Fw·Fs·Fc (= 8 × 9 × 10 × 11)	0.24	0.24	0.24	0.22	0.24	0.21	0.26	0.31	0.30	0.32	0.27	0.24
13. 17.4 · D · Tc · (Fh·Fw·Fs·Fc), (= 17.4 × 1 × 2 × 12) ; ETo	3.88	3.87	3.88	3.50	3.78	3.08	3.75	4.75	4.56	5.25	4.29	3.94

Table 4.1.11 Weighting Factor (W) at Different Temperatures and Altitude

Temperature °C	10	12	14	16	18	20	22	24	26	28	30	32	34	36	38	40
W at altitude (m)																
0	.55	.58	.61	.64	.66	.68	.71	.73	.75	.77	.78	.80	.82	.83	.84	.85
500	.57	.60	.62	.65	.67	.70	.72	.74	.76	.78	.79	.81	.82	.84	.85	.86
1000	.58	.61	.64	.66	.69	.71	.73	.75	.77	.79	.80	.82	.83	.85	.86	.87
2000	.61	.64	.66	.69	.71	.73	.75	.77	.79	.81	.82	.84	.85	.86	.87	.88
3000	.64	.66	.69	.71	.73	.75	.77	.79	.81	.82	.84	.85	.86	.88	.88	.89
4000	.66	.69	.71	.73	.76	.78	.79	.81	.83	.84	.85	.86	.88	.89	.90	.90

Table 4.1.12 Correction Factor (K) of Wind Velocity

U day/ U night Ratio	1.0	1.5	2.0	2.5	3.0	3.5	4.0
Correction Factor (K)	1.0	1.2	1.3	1.43	1.5	1.56	1.6

Table 4.1.13 Calculation Sheet of Potential Evapotranspiration (ETo) Radiation Method

Radiation Method ETo = C · (W · Rs)	Jan.	Feb.	Mar.	Apr.	May	Jun.	Jul.	Aug.	Sep.	Oct.	Nov.	Dec.
1. Mean monthly extra terrestrial radiation ; Ra (mH_2O/day)	15.5	15.8	15.6	14.9	13.8	13.2	13.4	14.3	15.1	15.6	15.5	15.4
2. Constant ; a = 0.21, Constant ; b = 0.52												
3. Sunshine rate $\frac{n}{N}$ = S	0.41	0.43	0.46	0.50	0.55	0.44	0.56	0.63	0.65	0.66	0.57	0.43
4. Rs = (a + b × $\frac{n}{N}$) · Ra (= (a + b × 3) · 1)	6.56	6.85	7.01	7.00	6.84	5.79	6.72	7.69	8.27	8.63	7.85	6.68
5. Mean monthly temperature : t (°C)	27.9	28.0	27.7	27.6	27.4	26.5	26.1	26.3	26.7	28.2	27.9	27.6
6. Weighting factor effect of radiation ; W (See Table with 5)	0.77	0.77	0.77	0.77	0.76	0.76	0.75	0.75	0.76	0.77	0.77	0.77
7. Mean monthly relative humidity ; Rm (%)	78.8	78.4	77.7	80.9	80.1	82.3	79.0	75.2	76.4	73.0	86.4	80.8
8. Mean monthly wind velocity ; U_2 (m/s)	1.7	1.4	1.2	1.0	1.2	1.2	1.5	1.6	1.5	1.3	1.0	2.7
9. U daytime / U nighttime (= 2.5)												
10. K (= 1.43)												
11. U_2 daytime = K · U_2. (= 10 × 8)	2.43	2.00	1.72	1.43	1.72	1.72	2.145	2.29	2.15	1.86	1.43	3.86
12. W · Rs , (= 6 × 4)	5.05	5.28	5.40	5.39	5.20	4.40	5.04	5.77	6.29	6.65	6.04	5.14
13. ETo , (See Figure with 7, 11 and 12)	4.0	3.8	3.9	3.9	3.7	3.0	3.7	4.2	4.7	5.1	4.4	4.1
14. C = ETo / (W·Rs) , (= 13 / 12)	7.92	7.58	7.41	7.42	7.69	9.09	7.94	6.94	6.36	6.02	6.62	7.78

Table 4.1.14 Pan Coefficient (Kp) for Class A Pan for Different Groundcover and Levels of Mean Relative Humidity and 24 hour Wind

Class A pan	Case A : Pan placed in short green cropped are				Case B[Δ] : Pan placed in dry fallow area			
RH mean %		low 40	medium 40-70	high 70		low 40	medium 40-70	high 70
Wind km/day	Windward side distance of green crop m				Windward side distance of dry fallow m			
Light 175	1	.55	.65	.75	1	.7	.8	.85
	10	.65	.75	.85	10	.6	.7	.8
	100	.7	.8	.85	100	.55	.65	.75
	1000	.75	.85	.85	1000	.5	.6	.7
Moderate 175-425	1	.5	.6	.65	1	.65	.75	.8
	10	.6	.7	.75	10	.55	.65	.7
	100	.65	.75	.8	100	.5	.6	.65
	1000	.7	.8	.8	1000	.45	.55	.6
Strong 425-700	1	.45	.5	.6	1	.6	.65	.7
	10	.55	.6	.65	10	.5	.55	.65
	100	.6	.65	.7	100	.45	.5	.6
	1000	.65	.7	.75	1000	.4	.45	.55
Very Strong 700	1	.4	.45	.5	1	.5	.6	.65
	10	.45	.55	.6	10	.45	.5	.55
	100	.5	.6	.65	100	.4	.45	.5
	1000	.55	.6	.65	1000	.35	.4	.45

Δ : For extensive areas of bare-fallow soils and no agricultural development, reduce K pan by 20% under hot, windy conditions; by 5-10% for moderate wind, temperature and humidity conditions.

Table 4.1.15 Effective Rainfall for Paddy

		1975			1976			1977			1978			1979		
		Sop	Tak	Mean	Sop	Tak	Mean	Sop	Tak	Mean	Sop	Tak	Mean	Sop	Tak	Mean
Jan.	1	-	-	-	-	-	-	56	100	78	53	36	45	6	56	31
	2	36	19	28	23	45	34	50	71	61	24	32	28	7	47	27
	3	16	9	13	12	16	14	90	101	96	-	-	-	-	6	3
Feb.	1	7	31	19	38	19	29	13	38	26	26	18	22	-	44	22
	2	7	15	11	-	-	-	58	54	56	8	17	13	36	66	51
	3	10	9	10	7	-	4	18	9	14	-	6	3	6	51	29
Mar.	1	36	40	38	10	20	15	19	36	28	67	114	91	15	48	32
	2	37	6	22	36	41	39	30	39	35	40	85	63	14	35	25
	3	22	26	24	37	-	19	49	17	33	50	102	76	69	63	66
Apr.	1	35	40	38	28	48	38	49	14	32	-	-	-	38	43	41
	2	89	61	75	69	66	68	48	11	30	59	22	41	-	6	3
	3	31	38	35	22	30	26	57	18	38	46	61	54	53	90	72
May	1	47	84	66	98	124	111	14	30	22	10	36	23	81	84	83
	2	74	97	86	36	7	22	37	24	31	34	30	32	18	19	19
	3	48	62	55	26	-	13	42	54	48	22	55	39	64	99	82
Jun.	1	75	85	80	52	71	62	30	65	48	8	37	23	76	42	59
	2	68	30	49	25	74	50	118	171	145	16	54	35	71	84	78
	3	-	-	-	40	52	46	-	-	-	-	29	15	25	27	26
Jul.	1	23	60	42	78	89	84	37	55	46	40	31	36	-	20	10
	2	62	10	36	29	18	24	-	-	-	54	78	66	-	25	13
	3	98	48	73	22	-	11	22	-	11	46	35	41	4	14	9
Oct.	3	20	58	39	78	121	100	-	-	-	98	-	49	5	-	3
Nov.	1	39	60	50	4	-	2	-	-	-	25	19	22	-	-	-
	2	12	5	9	116	67	92	14	7	11	50	34	42	37	76	57
	3	11	32	22	33	42	38	46	98	72	18	103	61	13	10	12
Dec.	1	-	7	4	16	22	19	88	118	103	26	28	27	32	-	32
	2	85	52	69	38	49	44	14	41	28	39	37	38	16	-	16
	3	15	27	21	18	52	35	28	46	37	70	111	91	9	-	9

Note : Sop and Tak are abbreviations of Watan Soppeng and Takalala respectively.

Table 4.1.16 Effective Rainfall for Polowijo

		1975			1976			1977			1978			1979		
		Sop	Tak	Mean	Sop	Tak	Mean	Sop	Tak	Mean	Sop	Tak	Mean	Sop	Tak	Mean
	1															
Jul.	2	11	7	9	11	11	11	-	-	-	11	11	11	-	11	6
	3	15	15	15	5	-	3	15	-	8	15	15	15	3	11	7
	1	4	13	9	9	9	9	7	4	6	18	10	14	-	-	-
Aug.	2	24	13	19	-	-	-	-	-	-	24	24	24	-	-	-
	3	14	28	21	-	-	-	4	4	4	-	6	3	-	-	-
	1	33	33	33	-	-	-	4	-	2	21	11	16	25	33	29
Sep.	2	21	26	24	-	-	-	-	-	-	34	34	34	-	-	-
	3	24	18	21	-	-	-	-	-	-	29	21	25	13	27	20
	1	32	7	20	3	6	5	-	-	-	32	10	21	-	4	2
Oct.	2	19	28	24	-	11	6	-	-	-	25	14	20	-	-	-
	3	17	28	23	28	28	28	-	-	-	28	18	23	4	-	2
	1	17	17	17	12	-	6	-	-	-	17	15	16	-	-	-
Nov.	2															
	3															

Note : Sop and Tak are abbreviations of Watan Soppeng and Takalala respectively.

Table 4.1.17 Estimated Effective Rainfall

	Paddy				Paddy Total		Polowijo	
	(Jan.1st - Jul.2nd)		(Nov.1st - Dec.3rd)				(Jul.3rd - Oct.3rd)	
	R	Re	R	Re	R	Re	R	Re
1975	798	430	235	115	1,033	545	517	217
1976	926	484	346	158	1,272	642	181	38
1977	1,166	600	329	175	1,495	775	40	17
1978	1,048	434	366	195	1,414	629	410	177
1979	1,032	536	170	85	1,202	621	96	54
Total					6,414	3,212	1,244	503

Mean Annual Effective Rainfall for Paddy $= \frac{3,212\ (mm/4.5\ year)}{4.5\ (year)} = 714\ (mm/year)$

Mean Annual Effective Rainfall for Polowijo $= \frac{503\ (mm/5\ year)}{5\ (year)} = 101\ (mm/year)$

Effective Rainfall Ratio for Paddy $= \frac{3,212}{6,414} = 50\%$

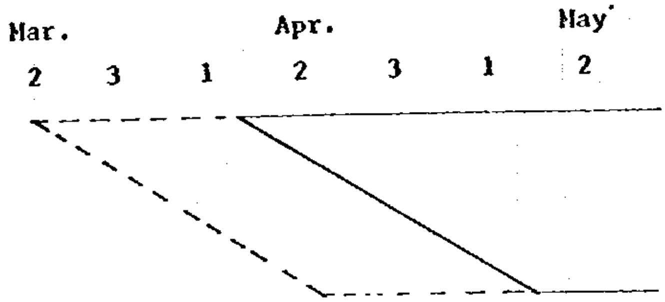
Effective Rainfall Ratio for Polowijo $= \frac{503}{1,244} = 40\%$

Note : R means Rainfall (See Annex - V (Data Book))

Re means Effective Rainfall (See Table 4.1.15 and Table 4.1.16)

Table 4.1.18 (1) Nursery and Puddling Water Requirement

1st Paddy (1975)



(1) Nursery Water Requirement

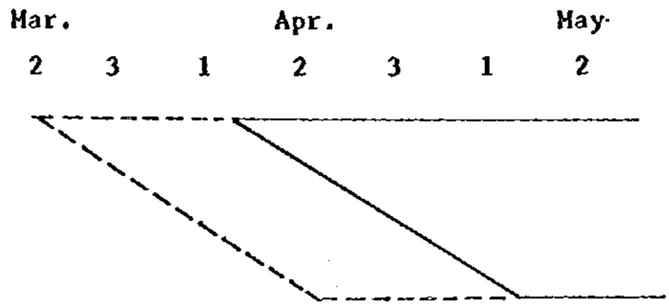
Au	$\frac{1}{24}$	$\frac{1}{3}$	$\frac{5}{8}$	$\frac{5}{8}$	$\frac{1}{3}$	$\frac{1}{24}$
Au x $\frac{1}{20}$	$\frac{1}{480}$	$\frac{1}{60}$	$\frac{1}{32}$	$\frac{1}{32}$	$\frac{1}{60}$	$\frac{1}{480}$
Re (mm/n days)	22	24	38	75	35	66
Eto (mm/ days)	3.9	3.9	3.7	3.7	3.7	3.7
d (mm/n days)	5.9	5.9	5.7	5.7	5.7	5.7
n (mm/n days)	10	11	10	10	10	10
Sn (mm/n days)	0	1	1	0	1	0
Pwn (mm/n days)	0	2	3	1	1	0
Wn (mm/n days)	0	3	4	2	2	0

(2) Puddling Water Requirement

Ap			$\frac{1}{6}$	$\frac{1}{3}$	$\frac{1}{3}$	$\frac{1}{6}$
Wd (mm/n days)			14	15	28	9

Table 4.1.18 (2) Nursery and Puddling Water Requirement

1st Paddy (1976)



(1) Nursery Water Requirement

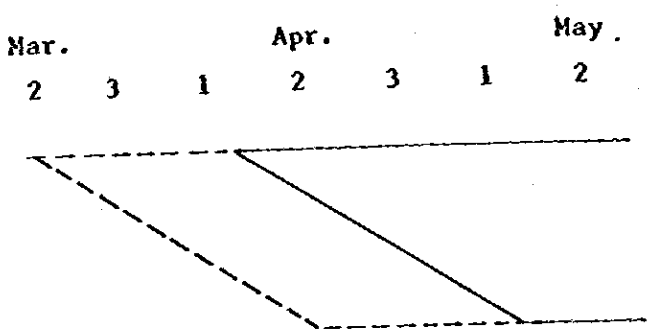
Au	$\frac{1}{24}$	$\frac{1}{3}$	$\frac{5}{8}$	$\frac{5}{8}$	$\frac{1}{3}$	$\frac{1}{24}$
Au $\times \frac{1}{20}$	$\frac{1}{480}$	$\frac{1}{60}$	$\frac{1}{32}$	$\frac{1}{32}$	$\frac{1}{60}$	$\frac{1}{480}$
Re (mm/n days)	39	19	38	68	26	111
ETo (mm/ days)	3.9	3.9	3.7	3.7	3.7	3.7
d (mm/n days)	5.9	5.9	5.7	5.7	5.7	5.7
n (mm/n days)	10	11	10	10	10	10
Sn (mm/n days)	0	1	1	1	1	0
Pwn (mm/n days)	0	2	3	2	2	0
Wn (mm/n days)	0	3	4	2	3	0

(2) Puddling Water Requirement

Ap			$\frac{1}{6}$	$\frac{1}{3}$	$\frac{1}{3}$	$\frac{1}{6}$
Wd (mm/n days)			14	15	28	9

Table 4.1.18 (3) Nursery and Puddling Water Requirement

1st Paddy (1977)



(1) Nursery Water Requirement

Au	$\frac{1}{24}$	$\frac{1}{3}$	$\frac{5}{8}$	$\frac{5}{8}$	$\frac{1}{3}$	$\frac{1}{24}$
Au x $\frac{1}{20}$	$\frac{1}{480}$	$\frac{1}{60}$	$\frac{1}{32}$	$\frac{1}{32}$	$\frac{1}{60}$	$\frac{1}{480}$
Re (mm/n days)	35	33	32	30	38	22
ETo (mm/days)	3.9	3.9	3.7	3.7	3.7	3.7
d (mm/n days)	5.9	5.9	5.7	5.7	5.7	5.7
n (mm/n days)	10	11	10	10	10	10
Sn (mm/n days)	0	1	2	2	1	0
Pwn (mm/n days)	0	1	3	3	1	0
Wn (mm/n days)	0	3	3	4	2	0

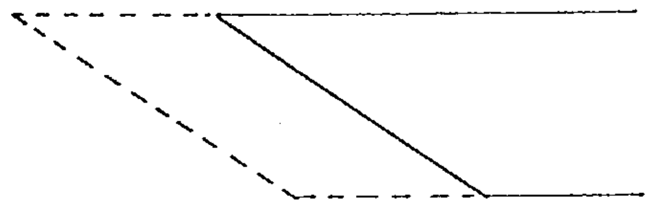
(2) Puddling Water Requirement

Ap			$\frac{1}{6}$	$\frac{1}{3}$	$\frac{1}{3}$	$\frac{1}{6}$
Wd (mm/n days)			15	30	27	16

Table 4.1.18 (4) Nursery and Puddling Water Requirement

1st Paddy (1978)

Mar.			Apr.			May
2	3	1	2	3	1	2



(1) Nursery Water Requirement

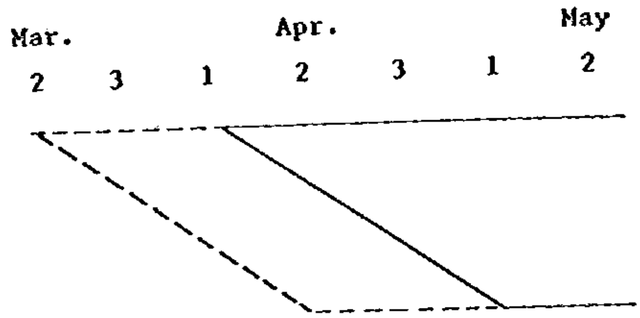
Au	$\frac{1}{24}$	$\frac{1}{3}$	$\frac{5}{8}$	$\frac{5}{8}$	$\frac{1}{3}$	$\frac{1}{24}$
Au x $\frac{1}{20}$	$\frac{1}{480}$	$\frac{1}{60}$	$\frac{1}{32}$	$\frac{1}{32}$	$\frac{1}{60}$	$\frac{1}{480}$
Re (mm/n days)	63	76	-	41	54	23
ETo (mm/days)	3.9	3.9	3.7	3.7	3.7	3.7
d (mm/n days)	5.9	5.9	5.7	5.7	5.7	5.7
n (mm/n days)	10	11	10	10	10	10
Sn (mm/n days)	0	0	3	1	1	0
Pwn (mm/n days)	0	1	4	2	1	0
Wn (mm/n days)	0	1	5	4	2	0

(2) Puddling Water Requirement

Ap			$\frac{1}{6}$	$\frac{1}{3}$	$\frac{1}{3}$	$\frac{1}{6}$
Wd (mm/n days)			20	26	22	16

Table 4.1.18 (5) Nursery and Puddling Water Requirement

1st Paddy (1979)



(1) Nursery Water Requirement

Au	$\frac{1}{24}$	$\frac{1}{3}$	$\frac{5}{8}$	$\frac{5}{8}$	$\frac{1}{3}$	$\frac{1}{24}$
Au x $\frac{1}{20}$	$\frac{1}{480}$	$\frac{1}{60}$	$\frac{1}{32}$	$\frac{1}{32}$	$\frac{1}{60}$	$\frac{1}{480}$
Re (mm/n days)	25	66	41	3	72	83
ETo (mm/days)	3.9	3.9	3.7	3.7	3.7	3.7
d (mm/n days)	5.9	5.9	5.7	5.7	5.7	5.7
n (mm/n days)	10	11	10	10	10	10
Sn (mm/n days)	0	1	1	3	0	0
Pwn (mm/n days)	0	1	2	4	1	0
Wn (mm/n days)	0	1	4	5	1	0

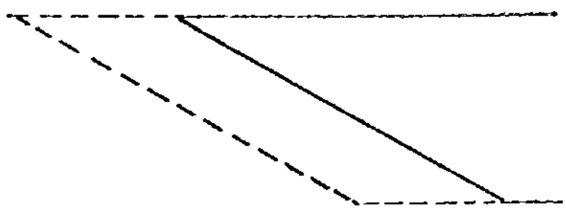
(2) Puddling Water Requirement

Ap			$\frac{1}{6}$	$\frac{1}{3}$	$\frac{1}{3}$	$\frac{1}{6}$
Wd (mm/n days)			13	39	16	6

Table 4.1.19 (1) Nursery and Puddling Water Requirement

2nd Paddy (1975 - 1976)

	Oct.			Nov.			Dec.
1	2	3	1	2	3	1	2



(1) Nursery Water Requirement

Au	$\frac{1}{8}$	$\frac{3}{8}$	$\frac{1}{2}$	$\frac{1}{2}$	$\frac{3}{8}$	$\frac{1}{8}$
Au x $\frac{1}{20}$	$\frac{1}{160}$	$\frac{3}{160}$	$\frac{1}{40}$	$\frac{1}{40}$	$\frac{3}{160}$	$\frac{1}{160}$
Re (mm/n day)	33	50	9	22	4	69
ETo (mm/day)	5.0	4.3	4.3	4.3	4.1	4.1
d (mm/n day)	7.0	6.3	6.3	6.3	6.1	6.1
n (day)	11	10	10	10	10	10
Sn (mm/n day)	0.5	0.8	2.1	1.8	1.6	0.1
Pwn (mm/n day)	0.5	1.3	2.8	2.4	2.2	0.3
Wn (mm/n day)	1	1	5	4	4	0

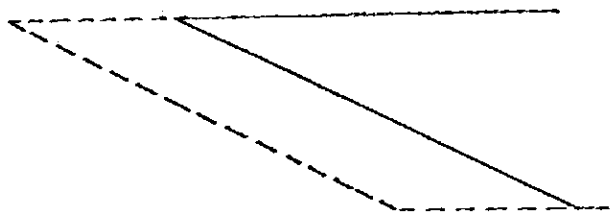
(2) Puddling Water Requirement

Ap	$\frac{1}{4}$	$\frac{1}{4}$	$\frac{1}{4}$	$\frac{1}{4}$
Wd (mm/n day)	28	25	29	13

Table 4.1.19 (2)   Nursery and Puddling Water Requirement

2nd Paddy (1976 - 1977)

	Oct.	Nov.			Dec.	
	3	1	2	3	1	2



(1) Nursery Water Requirement

Au	$\frac{1}{8}$	$\frac{3}{8}$	$\frac{1}{2}$	$\frac{1}{2}$	$\frac{3}{8}$	$\frac{1}{8}$
Au x $\frac{1}{20}$	$\frac{1}{160}$	$\frac{3}{160}$	$\frac{1}{40}$	$\frac{1}{40}$	$\frac{3}{160}$	$\frac{1}{160}$
Re (mm/n day)	96	2	92	38	19	44
ETo (mm/day)	5.0	4.3	4.3	4.3	4.1	4.1
d (mm/n day)	7.0	6.3	6.3	6.3	6.1	6.1
n (day)	11	10	10	10	10	10
Sn (mm/n day)	0.16	1.74	0.06	1.42	1.36	0.30
Pwn (mm/n day)	0.16	2.22	0.70	2.06	1.90	0.48
Wn (mm/n day)	0	4	1	3	3	1

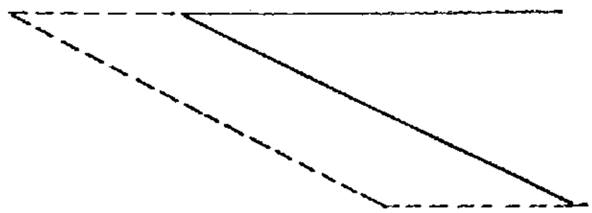
(2) Puddling Water Requirement

Ap			$\frac{1}{4}$	$\frac{1}{4}$	$\frac{1}{4}$	$\frac{1}{4}$
Wd (mm/n day)			7	21	25	19

Table 4.1.19 (3) Nursery and Puddling Water Requirement

2nd Paddy (1977 - 1978)

Oct.		Nov.			Dec.	
2	3	1	2	3	1	2



(1) Nursery Water Requirement

Au	$\frac{1}{8}$	$\frac{3}{8}$	$\frac{1}{2}$	$\frac{1}{2}$	$\frac{3}{8}$	$\frac{1}{8}$
Au x $\frac{1}{20}$	$\frac{1}{160}$	$\frac{3}{160}$	$\frac{1}{40}$	$\frac{1}{40}$	$\frac{3}{160}$	$\frac{1}{160}$
Re (mm/n day)	0	0	11	72	103	28
ETo (mm/n day)	5	4.3	4.3	4.3	4.1	4.1
d (mm/n day)	7.0	6.3	6.3	6.3	6.1	6.1
n (day)	11	10	10	10	10	10
Sn (mm/n day)	0.72	1.78	2.08	0.56	0	0.40
Pwn (mm/n day)	0.76	2.26	2.72	1.20	0.32	0.58
Wn (mm/n day)	1	4	5	2	0	1

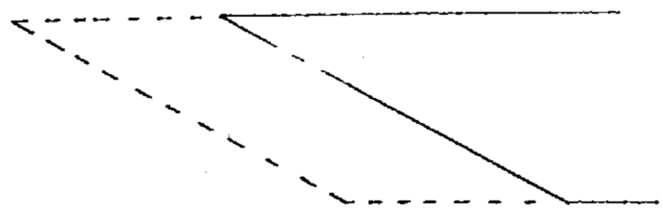
(2) Puddling Water Requirement

Ap			$\frac{1}{4}$	$\frac{1}{4}$	$\frac{1}{4}$	$\frac{1}{4}$
Wd (mm/n day)			27	12	4	23

Table 4.1.19 (4) Nursery and Puddling Water Requirement

2nd Paddy (1978 - 1979)

	Oct.			Nov.			Dec.
1	2	3	1	2	3	1	2



(1) Nursery Water Requirement

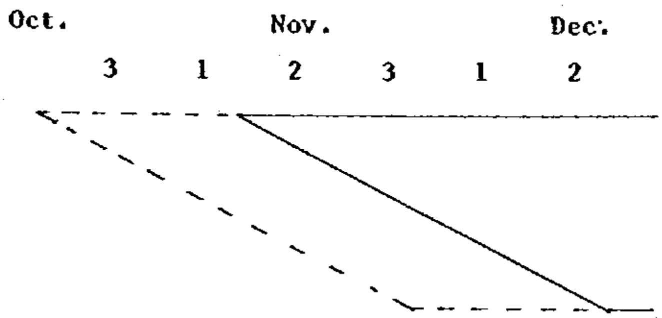
Au	$\frac{1}{8}$	$\frac{3}{8}$	$\frac{1}{2}$	$\frac{1}{2}$	$\frac{3}{8}$	$\frac{1}{8}$
Au x $\frac{1}{20}$	$\frac{1}{160}$	$\frac{3}{160}$	$\frac{1}{40}$	$\frac{1}{40}$	$\frac{3}{160}$	$\frac{1}{160}$
Re (mm/n day)	33	50	9	22	4	69
ETo (mm/day)	5.0	4.3	4.3	4.3	4.1	4.1
d (mm/n day)	7.0	6.3	6.3	6.3	6.1	6.1
n (day)	11	10	10	10	10	10
Sn (mm/n day)	0	1	1	1	1	0
Pwn (mm/n day)	0	2	2	1	2	0
Wn (mm/n day)	0	3	3	2	3	0

(2) Puddling Water Requirement

Ap			$\frac{1}{4}$	$\frac{1}{4}$	$\frac{1}{4}$	$\frac{1}{4}$
Wd (mm/n day)			20	15	23	21

Table 4.1.19 (5) Nursery and Puddling Water Requirement

2nd Paddy (1979 - 1980)



(1) Nursery Water Requirement

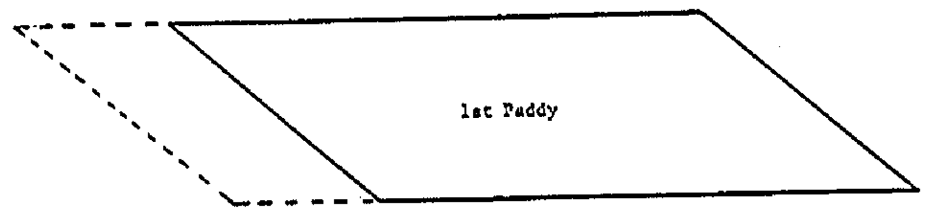
Au	$\frac{1}{8}$	$\frac{3}{8}$	$\frac{1}{3}$	$\frac{1}{2}$	$\frac{3}{8}$	$\frac{1}{8}$
Au x $\frac{1}{20}$	$\frac{1}{160}$	$\frac{3}{160}$	$\frac{1}{40}$	$\frac{1}{40}$	$\frac{3}{160}$	$\frac{1}{160}$
Re (mm/n day)	0	0	57	12	32	16
ETo (mm/n day)	5.0	4.3	4.3	4.3	4.1	4.1
d (mm/n day)	7.0	6.3	6.3	6.3	6.1	6.1
n (day)	11	10	10	10	10	10
Sn (mm/n day)	1	2	11	2	1	0
Pwn (mm/n day)	1	2	2	3	2	1
Wn (mm/n day)	2	4	3	5	3	1

(2) Puddling Water Requirement

Ap			$\frac{1}{4}$	$\frac{1}{4}$	$\frac{1}{4}$	$\frac{1}{4}$
Wd (mm/n day)			15	27	22	26

Table 4.1.20 (1) Calculation Sheet of Water Requirement for Paddy

Cropping Pattern

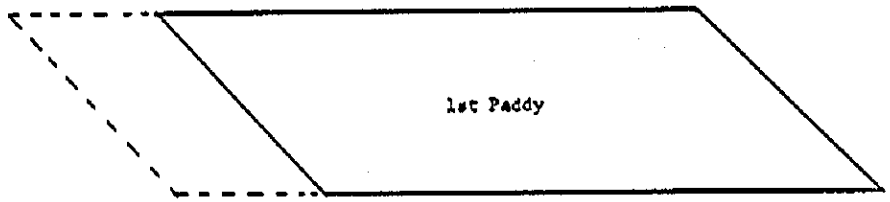


Calculation

1 9 7 5	Mar. 1st	Mar. 2nd	Mar. 3rd	Apr. 1st	Apr. 2nd	Apr. 3rd	May 1st	May 2nd	May 3rd	Jun. 1st	Jun. 2nd	Jun. 3rd	Jul. 1st	Jul. 2nd	Jul. 3rd
A. Cropping intensity (Kc)															
Early cropping				1/24	7/24	1/3	1/3	1/3	1/3	1/3	1/3	1/3	1/6		
Middle "					1/24	7/24	1/3	1/3	1/3	1/3	1/3	1/3	1/3	1/6	
Late "						1/24	7/24	1/3	1/3	1/3	1/3	1/3	1/3	1/3	1/6
Total				1/24	1/3	2/3	23/24	1	1	1	1	1	5/6	1/2	1/6
B. Crop coefficient (Kc)															
Early cropping				0.80	0.83	0.94	1.09	1.23	1.34	1.33	1.23	1.08	0.70		
Middle "					0.80	0.83	0.94	1.09	1.23	1.34	1.33	1.23	1.08	0.70	
Late "						0.80	0.83	0.94	1.09	1.23	1.34	1.33	1.23	1.08	0.70
Weighting average				0.80	0.83	0.88	0.96	1.09	1.22	1.30	1.30	1.21	1.06	0.95	0.70
C. Potential evapotranspiration (ETo)				37	37	37	37	37	41	31	31	31	37	37	41
D. Consumptive use of water (Cu)				30	31	33	35	40	50	40	40	38	39	35	29
E. Percolation loss (Pc)				20	20	20	20	20	22	20	20	20	20	20	22
F. Effective rainfall (Re)				38	75	35	66	86	55	80	49	-	42	36	73
G. Water req. paddy field (Wp)				0	0	12	0	0	17	0	11	58	14	10	0
H. Water req. for nursery (Wn)		0	3	4	2	2	0								
I. Water req. for puddling (Wd)		0	2	14	15	28	9								
J. Farm water req. (Fw) (mm/N days)		0	3	18	17	42	9	0	17	0	11	58	14	10	0
K. Diversion water req. (Lw) (mm/N days)		0	5	28	27	66	14	0	27	0	18	90	23	15	0
L. Unit Diversion req. (Qe) (l/s/ha)		0	0.049	0.326	0.312	0.759	0.163	0	0.280	0	0.204	1.042	0.262	0.174	0

Table 4.1.20 (2) Calculation Sheet of Water Requirement for Paddy

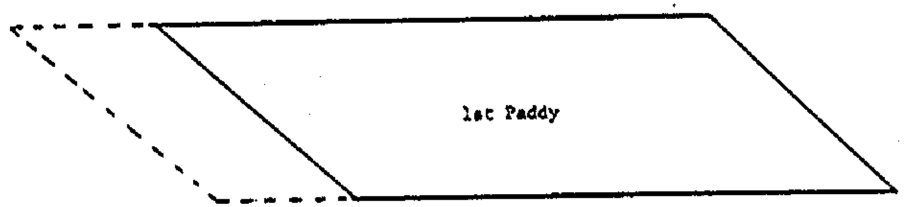
Cropping Pattern



	1976														
	Mar.			Apr.			May			Jun.			Jul.		
	1st	2nd	3rd	1st	2nd	3rd	1st	2nd	3rd	1st	2nd	3rd	1st	2nd	3rd
Calculation															
A. Cropping intensity (Ic)															
Early cropping				1/24	7/24	1/3	1/3	1/3	1/3	1/3	1/3	1/3	1/6		
Middle "					1/24	7/24	1/3	1/3	1/3	1/3	1/3	1/3	1/3	1/6	
Late "						1/24	7/24	1/3	1/3	1/3	1/3	1/3	1/3	1/3	1/6
Total				1/24	1/3	2/3	23/24	1	1	1	1	1	5/6	1/2	1/6
B. Crop coefficient (Kc)															
Early cropping				0.80	0.83	0.94	1.09	1.23	1.34	1.33	1.23	1.08	0.70		
Middle "					0.80	0.83	0.94	1.09	1.23	1.34	1.33	1.23	1.08	0.70	
Late "						0.80	0.83	0.94	1.09	1.23	1.34	1.33	1.23	1.08	0.70
Weighting average				0.80	0.83	0.88	0.96	1.09	1.22	1.30	1.30	1.21	1.06	0.95	0.70
C. Potential evapotranspiration (ETo)				37	37	37	37	37	41	31	31	31	37	37	41
D. Consumptive use of water (Cu)				30	31	33	35	40	50	40	40	38	39	35	29
E. Percolation loss (Pc)				20	20	20	20	20	22	20	20	20	20	20	22
F. Effective rainfall (Re)				38	68	26	111	22	13	62	50	46	84	24	11
G. Water req. for paddy field (Wp)				0	0	18	0	38	59	0	10	12	0	16	7
H. Water req. for nursery (Wn)		0	3	4	2	3	0								
I. Water req. for puddling (Wd)				14	17	31	1								
J. Farm water req. (Fw) (mm/N days)		0	3	18	19	32	1	38	59	0	10	12	0	16	7
K. Diversion water req. (Dw) (mm/N days)		0	5	28	30	81	2	60	92	0	16	18	0	24	10
L. Unit Diversion req. (Qe) (l/s/ha)		0	0.052	0.324	0.347	0.938	0.018	0.691	0.970	0	0.186	0.210	0	0.283	0.109

Table 4.1.20 (3) Calculation Sheet of Water Requirement for Paddy

Cropping Pattern

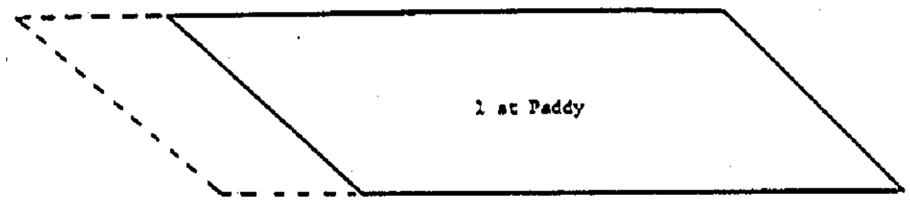


Calculation

	1977 Mar. 1st	Mar. 2nd	Mar. 3rd	Apr. 1st	Apr. 2nd	Apr. 3rd	May 1st	May 2nd	May 3rd	Jun. 1st	Jun. 2nd	Jun. 3rd	Jul. 1st	Jul. 2nd	Jul. 3rd
A. Cropping intensity (Ic)															
Early cropping				$\frac{1}{24}$	$\frac{7}{24}$	$\frac{1}{3}$	$\frac{1}{3}$	$\frac{1}{3}$	$\frac{1}{3}$	$\frac{1}{3}$	$\frac{1}{3}$	$\frac{1}{3}$	$\frac{1}{6}$		
Middle "					$\frac{1}{24}$	$\frac{7}{24}$	$\frac{1}{3}$	$\frac{1}{3}$	$\frac{1}{3}$	$\frac{1}{3}$	$\frac{1}{3}$	$\frac{1}{3}$	$\frac{1}{3}$	$\frac{1}{6}$	
Late "						$\frac{1}{24}$	$\frac{7}{24}$	$\frac{1}{3}$	$\frac{1}{3}$	$\frac{1}{3}$	$\frac{1}{3}$	$\frac{1}{3}$	$\frac{1}{3}$	$\frac{1}{3}$	$\frac{1}{6}$
Total				$\frac{1}{24}$	$\frac{1}{3}$	$\frac{2}{3}$	$\frac{23}{24}$	1	1	1	1	1	$\frac{5}{6}$	$\frac{1}{2}$	$\frac{1}{6}$
B. Crop coefficient (Kc)															
Early cropping				0.80	0.83	0.94	1.09	1.23	1.34	1.33	1.23	1.08	0.70		
Middle "					0.80	0.83	0.94	1.09	1.23	1.34	1.33	1.23	1.08	0.70	
Late "						0.80	0.83	0.94	1.09	1.23	1.34	1.33	1.23	1.08	0.70
Weighting average				0.80	0.83	0.88	0.96	1.09	1.22	1.30	1.30	1.21	1.06	0.95	0.70
C. Potential evapotranspiration (ETo)				37	37	37	37	37	41	31	31	31	37	37	41
D. Consumptive use of water (Cu)				30	31	33	35	40	50	40	40	38	39	35	29
E. Percolation loss (Pc)				20	20	20	20	20	22	20	20	20	20	20	22
F. Effective rainfall (Re)				32	30	38	22	31	48	48	145	-	46	-	11
G. Water req. for paddy field (Wp)				1	7	10	32	29	24	12	0	58	11	28	7
H. Water req. for nursery (Wn)		0	3	3	4	2	0								
I. Water req. for pudding (Wd)				15	30	27	16								
J. Farm water req. (Fw) (mm/N days)		0	3	19	41	39	48	29	24	12	0	58	11	28	7
K. Diversion water req. (Dw) (mm/N days)		0	5	30	64	61	75	46	38	19		90	17	43	10
L. Unit diversion req. (Qe) (l/s/ha)		0	0.052	0.347	0.741	0.706	0.869	0.528	0.395	0.222	0	1.042	0.201	0.500	0.109

Table 4.1.20 (4) Calculation Sheet of Water Requirement for Paddy

	1978 Mar. 1st	Mar. 2nd	Mar. 3rd	Apr. 1st	Apr. 2nd	Apr. 3rd	May 1st	May 2nd	May 3rd	Jun. 1st	Jun. 2nd	Jun. 3rd	Jul. 1st	Jul. 2nd	Jul. 3rd
Cropping Pattern															



	Mar. 1st	Mar. 2nd	Mar. 3rd	Apr. 1st	Apr. 2nd	Apr. 3rd	May 1st	May 2nd	May 3rd	Jun. 1st	Jun. 2nd	Jun. 3rd	Jul. 1st	Jul. 2nd	Jul. 3rd
Calculation															
A. Cropping intensity (Ic)															
Early cropping				$\frac{1}{24}$	$\frac{7}{24}$	$\frac{1}{3}$	$\frac{1}{3}$	$\frac{1}{3}$	$\frac{1}{3}$	$\frac{1}{3}$	$\frac{1}{3}$	$\frac{1}{3}$	$\frac{1}{6}$		
Middle "					$\frac{1}{24}$	$\frac{7}{24}$	$\frac{1}{3}$	$\frac{1}{3}$	$\frac{1}{3}$	$\frac{1}{3}$	$\frac{1}{3}$	$\frac{1}{3}$	$\frac{1}{3}$	$\frac{1}{6}$	
Late "						$\frac{1}{24}$	$\frac{7}{24}$	$\frac{1}{3}$	$\frac{1}{3}$	$\frac{1}{3}$	$\frac{1}{3}$	$\frac{1}{3}$	$\frac{1}{3}$	$\frac{1}{3}$	$\frac{1}{6}$
Total				$\frac{1}{24}$	$\frac{1}{3}$	$\frac{2}{3}$	$\frac{23}{24}$	1	1	1	1	1	$\frac{5}{6}$	$\frac{1}{2}$	$\frac{1}{6}$
B. Crop coefficient (Kc)															
Early cropping				0.80	0.83	0.94	1.09	1.23	1.34	1.33	1.23	1.08	0.70		
Middle "					0.80	0.83	0.94	1.09	1.23	1.34	1.33	1.23	1.08	0.70	
Late "						0.80	0.83	0.94	1.09	1.23	1.34	1.33	1.23	1.08	0.70
Weighting average				0.80	0.83	0.88	0.96	1.09	1.22	1.30	1.30	1.21	1.06	0.95	0.70
C. Potential evapotranspiration (ETo)				37	37	37	37	37	41	31	31	31	37	37	41
D. Consumptive use of water (Cu)				30	31	33	35	40	50	40	40	38	39	35	29
E. Percolation loss (Pc)				20	20	20	20	20	22	20	20	20	20	20	22
F. Effective rainfall (Re)				-	41	34	23	32	39	23	35	15	36	66	41
G. Water req. for paddy field (Wp)				2	3	0	31	28	33	37	25	43	19	0	2
H. Water req. for nursery (Wn)		0	1	5	4	2	0								
I. Water req. for puddling (Wd)				20	26	22	16								
J. Farm water req. (Fw) (mm/N days)		0	1	27	33	24	47	28	33	37	25	43	19	0	2
K. Diversion water req. (Dw) (mm/N days)		0	2	42	52	37	74	44	52	58	40	67	30	0	3
L. Unit diversion req. (Qe) (l/s/ha)		0	0.016	0.486	0.602	0.428	0.852	0.510	0.543	0.675	0.458	0.771	0.352	0	0.027

Table 4.1.20 (5) Calculation Sheet of Water Requirement for Paddy

	1979 Mar. 1st	Mar. 2nd	Mar. 3rd	Apr. 1st	Apr. 2nd	Apr. 3rd	May 1st	May 2nd	May 3rd	Jun. 1st	Jun. 2nd	Jun. 3rd	Jul. 1st	Jul. 2nd	Jul. 3rd
Cropping Pattern															

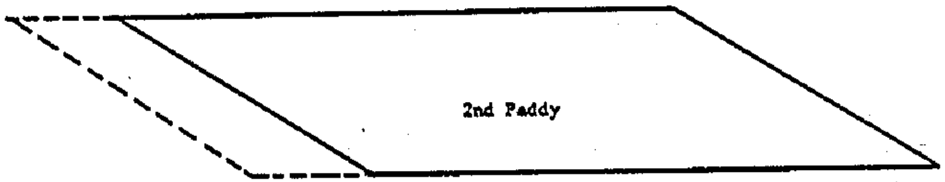
Calculation

	Mar. 1st	Mar. 2nd	Mar. 3rd	Apr. 1st	Apr. 2nd	Apr. 3rd	May 1st	May 2nd	May 3rd	Jun. 1st	Jun. 2nd	Jun. 3rd	Jul. 1st	Jul. 2nd	Jul. 3rd
A. Cropping intensity (Ic)															
Early cropping				$\frac{1}{24}$	$\frac{7}{24}$	$\frac{1}{3}$	$\frac{1}{3}$	$\frac{1}{3}$	$\frac{1}{3}$	$\frac{1}{3}$	$\frac{1}{3}$	$\frac{1}{3}$	$\frac{1}{6}$		
Middle "					$\frac{1}{24}$	$\frac{7}{24}$	$\frac{1}{3}$	$\frac{1}{3}$	$\frac{1}{3}$	$\frac{1}{3}$	$\frac{1}{3}$	$\frac{1}{3}$	$\frac{1}{3}$	$\frac{1}{6}$	
Late "						$\frac{1}{24}$	$\frac{7}{24}$	$\frac{1}{3}$	$\frac{1}{3}$	$\frac{1}{3}$	$\frac{1}{3}$	$\frac{1}{3}$	$\frac{1}{3}$	$\frac{1}{3}$	$\frac{1}{6}$
Total				$\frac{1}{24}$	$\frac{1}{3}$	$\frac{2}{3}$	$\frac{23}{24}$	1	1	1	1	1	$\frac{5}{6}$	$\frac{1}{2}$	$\frac{1}{6}$
B. Crop coefficient (Kc)															
Early cropping				0.80	0.83	0.94	1.09	1.23	1.34	1.33	1.23	1.08	0.70		
Middle "					0.80	0.83	0.94	1.09	1.23	1.34	1.33	1.23	1.08	1.70	
Late "						0.80	0.83	0.94	1.09	1.23	1.34	1.33	1.23	1.08	0.70
Weighting average				0.80	0.83	0.88	0.96	1.09	1.22	1.30	1.30	1.21	1.06	0.95	0.70
C. Potential evapotranspiration (ETo)				37	37	37	37	37	41	31	31	31	37	37	41
D. Consumptive use of water (Cu)				30	31	33	35	40	50	40	40	38	39	35	29
E. Percolation loss (Pc)				20	20	20	20	20	22	20	20	20	20	20	22
F. Effective rainfall (Re)				41	3	72	83	19	82	59	78	26	10	13	9
G. Water req. for paddy field (Wp)				0	16	0	0	41	0	1	0	32	41	21	7
H. Water req. for nursery (Wn)		0	1	4	5	1	0								
I. Water req. for puddling (Wd)				13	39	16	6								
J. Farm water req. (Fw) (mm/N days)		0	1	17	60	17	6	41	0	1	0	32	41	21	7
K. Diversion water req. (Dw) (mm/N days)		0	2	27	94	27	9	64		2		49	64	33	11
L. Unit diversion req. (Qe) (l/s/ha)		0	0.016	0.312	1.088	0.307	0.109	0.745	0	0.024	0	0.572	0.744	0.382	0.114

Table 4.1.20 (6) Calculation Sheet of Water Requirement for Paddy

	1975									1976					
	Nov.			Dec.			Jan.			Feb.			Mar.		
	1st	2nd	3rd	1st	2nd	3rd	1st	2nd	3rd	1st	2nd	3rd	1st	2nd	3rd

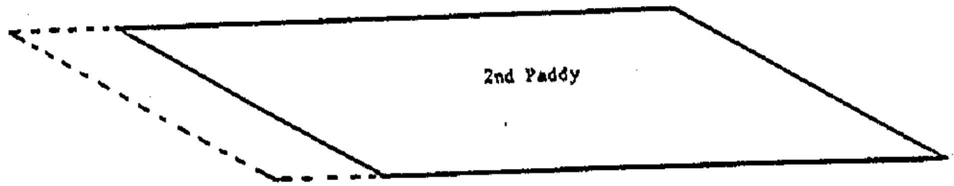
Calculation



Calculation	Nov. 1st	Nov. 2nd	Nov. 3rd	Dec. 1st	Dec. 2nd	Dec. 3rd	Jan. 1st	Jan. 2nd	Jan. 3rd	Feb. 1st	Feb. 2nd	Feb. 3rd	Mar. 1st	Mar. 2nd	Mar. 3rd
A. Cropping intensity															
Early Cropping			$\frac{1}{8}$	$\frac{23}{72}$	$\frac{1}{3}$	$\frac{1}{3}$	$\frac{1}{3}$	$\frac{1}{3}$	$\frac{1}{3}$	$\frac{1}{3}$	$\frac{29}{96}$	$\frac{25}{288}$			
Middle "				$\frac{1}{18}$	$\frac{5}{18}$	$\frac{1}{3}$	$\frac{1}{3}$	$\frac{1}{3}$	$\frac{1}{3}$	$\frac{1}{3}$	$\frac{1}{3}$	$\frac{95}{288}$	$\frac{1}{6}$	$\frac{1}{288}$	
Late "					$\frac{1}{72}$	$\frac{5}{24}$	$\frac{1}{3}$	$\frac{1}{3}$	$\frac{1}{3}$	$\frac{1}{3}$	$\frac{1}{3}$	$\frac{1}{3}$	$\frac{1}{3}$	$\frac{71}{288}$	$\frac{1}{32}$
Total			$\frac{1}{8}$	$\frac{3}{8}$	$\frac{5}{8}$	$\frac{7}{8}$	1	1	1	1	$\frac{31}{32}$	$\frac{3}{4}$	$\frac{1}{2}$	$\frac{1}{4}$	$\frac{1}{32}$
B. Crop coefficient (Kc)															
Early Cropping			0.80	0.84	0.98	1.14	1.28	1.36	1.32	1.16	0.95	0.70			
Middle "				0.80	0.83	0.94	1.09	1.23	1.34	1.33	1.23	1.08	0.70	0.70	
Late "					0.80	0.82	0.88	1.04	1.20	1.32	1.35	1.26	1.18	0.84	0.70
Weighting average			0.80	0.83	0.91	0.99	1.08	1.21	1.29	1.27	1.18	1.12	1.02	0.84	0.70
C. Potential evapotranspiration (ETo)			43	43	41	41	45	39	39	43	39	39	35	39	39
D. Consumptive use of water (Cu)			34	36	37	40	49	47	50	55	46	44	36	33	27
E. Percolation loss (Pc)			20	20	20	20	22	20	20	20	20	20	18	20	20
F. Effective rainfall (Re)			9	22	4	69	21	-	34	14	29	-	4	15	39
G. Water req. for paddy field (Wp)			6	13	33	0	50	67	36	63	36	48	25	9	0
H. Water req. for nursery (Wn)	1	1	5	4	4	0									
I. Water req. for puddling (Wd)			28	25	29	13									
J. Farm water req. (Fw) (mm/N days)	1	1	39	42	66	13	50	67	36	63	36	48	25	9	0
K. Diversion water req. (Dw) (mm/N days)	2	2	61	66	103	20	78	105	57	98	56	74	39	15	0
L. Unit diversion req. (Qe) (l/s/ha)	0.016	0.018	0.706	0.764	1.192	0.235	0.818	1.215	0.654	1.029	0.651	0.862	0.499	0.170	0

Table 4.1.20 (7) Calculation Sheet of Water Requirement for Paddy

	1976						1977								
	Nov.			Dec.			Jan.			Feb.			Mar.		
	1st	2nd	3rd	1st	2nd	3rd	1st	2nd	3rd	1st	2nd	3rd	1st	2nd	3rd
Cropping Pattern															



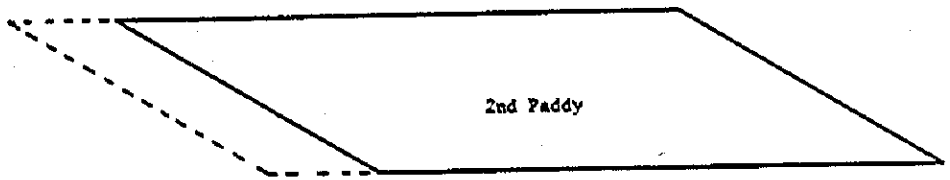
Calculation

	Nov. 1st	Nov. 2nd	Nov. 3rd	Dec. 1st	Dec. 2nd	Dec. 3rd	Jan. 1st	Jan. 2nd	Jan. 3rd	Feb. 1st	Feb. 2nd	Feb. 3rd	Mar. 1st	Mar. 2nd	Mar. 3rd
A. Cropping Intensity (Ic)															
Early cropping			$\frac{1}{8}$	$\frac{23}{72}$	$\frac{1}{3}$	$\frac{1}{3}$	$\frac{1}{3}$	$\frac{1}{3}$	$\frac{1}{3}$	$\frac{1}{3}$	$\frac{29}{96}$	$\frac{25}{286}$			
Middle "				$\frac{1}{18}$	$\frac{5}{18}$	$\frac{1}{3}$	$\frac{1}{3}$	$\frac{1}{3}$	$\frac{1}{3}$	$\frac{1}{3}$	$\frac{1}{3}$	$\frac{95}{288}$	$\frac{1}{6}$	$\frac{1}{288}$	
Late "					$\frac{1}{72}$	$\frac{5}{24}$	$\frac{1}{3}$	$\frac{1}{3}$	$\frac{1}{3}$	$\frac{1}{3}$	$\frac{1}{3}$	$\frac{1}{3}$	$\frac{1}{3}$	$\frac{71}{288}$	$\frac{1}{32}$
Total			$\frac{1}{8}$	$\frac{3}{8}$	$\frac{5}{8}$	$\frac{7}{8}$	1	1	1	1	$\frac{1}{32}$	$\frac{3}{4}$	$\frac{1}{2}$	$\frac{1}{4}$	$\frac{1}{32}$
B. Crop coefficient (Kc)															
Early Cropping			0.80	0.84	0.98	1.14	1.28	1.36	1.32	1.16	0.95	0.70			
Middle "				0.80	0.83	0.94	1.09	1.23	1.34	1.33	1.23	1.08	0.70	0.70	
Late "					0.80	0.82	0.88	1.04	1.20	1.32	1.35	1.26	1.18	0.84	0.70
Weighting average			0.80	0.83	0.91	0.99	1.08	1.21	1.29	1.27	1.18	1.12	1.02	0.84	0.70
C. Potential evapotranspiration (ETo)			43	43	41	41	45	39	39	43	39	39	31	39	39
D. Consumptive use of water (Cu)			34	36	37	40	49	47	50	55	46	44	32	33	27
E. Percolation loss (Pc)			20	20	20	20	22	20	20	22	20	20	16	20	20
F. Effective rainfall (Re)			92	38	19	44	35	78	61	96	26	56	14	28	35
G. Water req. for paddy field (Wp)			0	7	24	14	36	0	9	0	39	6	17	6	0
H. Water req. for nursery (Wn)	0	4	1	3	3	1									
I. Water req. for puddling (Wd)			7	21	25	19									
J. Farm water req. (Fw) (mm/N days)	0	4	8	31	52	34	36	0	9	0	39	6	17	6	0
K. Diversion water req. (Dw) (mm/N days)	0	6	12	48	81	53	56		14		61	9	26	10	0
L. Unit diversion req. (Qe) (l/s/ha)	0	0.069	0.139	0.556	0.937	0.613	0.588	0	0.166	0	0.704	0.102	0.380	0.112	0

Table 4.1.20 (8) Calculation Sheet of Water Requirement for Paddy

	1977 Nov. 1st	Nov. 2nd	Nov. 3rd	Dec. 1st	Dec. 2nd	Dec. 3rd	1978 Jan. 1st	Jan. 2nd	Jan. 3rd	Feb. 1st	Feb. 2nd	Feb. 3rd	Mar. 1st	Mar. 2nd	Mar. 3rd

Cropping Pattern



Calculation

	Nov. 1st	Nov. 2nd	Nov. 3rd	Dec. 1st	Dec. 2nd	Dec. 3rd	Jan. 1st	Jan. 2nd	Jan. 3rd	Feb. 1st	Feb. 2nd	Feb. 3rd	Mar. 1st	Mar. 2nd	Mar. 3rd
A. Cropping intensity (Ic)															
Early cropping			$\frac{1}{8}$	$\frac{23}{72}$	$\frac{1}{3}$	$\frac{1}{3}$	$\frac{1}{3}$	$\frac{1}{3}$	$\frac{1}{3}$	$\frac{1}{3}$	$\frac{29}{96}$	$\frac{25}{288}$			
Middle "				$\frac{1}{18}$	$\frac{5}{18}$	$\frac{1}{3}$	$\frac{1}{3}$	$\frac{1}{3}$	$\frac{1}{3}$	$\frac{1}{3}$	$\frac{1}{3}$	$\frac{95}{288}$	$\frac{1}{6}$	$\frac{1}{288}$	
Late "					$\frac{1}{72}$	$\frac{5}{24}$	$\frac{1}{3}$	$\frac{1}{3}$	$\frac{1}{3}$	$\frac{1}{3}$	$\frac{1}{3}$	$\frac{1}{3}$	$\frac{1}{3}$	$\frac{71}{288}$	$\frac{1}{32}$
Total			$\frac{1}{8}$	$\frac{3}{8}$	$\frac{5}{8}$	$\frac{7}{8}$	1	1	1	1	$\frac{31}{32}$	$\frac{3}{4}$	$\frac{1}{2}$	$\frac{1}{4}$	$\frac{1}{32}$
B. Crop coefficient (Kc)															
Early cropping			0.80	0.84	0.98	1.14	1.28	1.36	1.32	1.16	0.95	0.70			
Middle "				0.80	0.83	0.94	1.09	1.23	1.34	1.33	1.23	1.08	0.70	0.70	
Late "					0.80	0.82	0.88	1.04	1.20	1.32	1.35	1.26	1.18	0.84	0.70
Weighting average			0.80	0.83	0.91	0.99	1.08	1.21	1.29	1.27	1.18	1.12	1.02	0.84	0.70
C. Potential evapotranspiration (ETo)			43	43	41	41	45	39	39	43	39	39	31	39	39
D. Consumptive use of water (Cu)			34	36	37	40	49	47	50	55	46	44	32	33	27
E. Percolation loss (Pc)			20	20	20	20	22	20	20	22	20	20	16	20	20
F. Effective rainfall (Re)			11	72	103	28	37	45	28	-	22	13	3	91	63
G. Water req. for paddy field (Wp)			5	0	0	28	34	22	42	77	43	38	22	0	0
H. Water req. for nursery (Wn)	1	4	5	2	0	1									
I. Water req. for puddling (Wd)			27	12	4	23									
J. Farm water req. (Fw) (mm/N days)	1	4	37	14	4	52		22	42	77	43	38	22	0	0
K. Diversion water req. (Dw) (mm/N days)	2	6	58	22	6	81	53	35	66	120	67	59	35	0	0
L. Unit diversion req. (Qe) (l/s/ha)	0.018	0.072	0.669	0.253	0.072	0.940	0.555	0.401	0.763	1.260	0.774	0.685	0.504	0	0