REPUBLIC OF INDONESIA
MINISTRY OF PUBLIC WORKS AND ELECTRIC POWER

FEASIBILITY REPORT
ON
ULAR RIVER FLOOD CONTROL
AND IMPROVEMENT OF IRRIGATION PROJECT

VOLUME II
STUDY REPORT

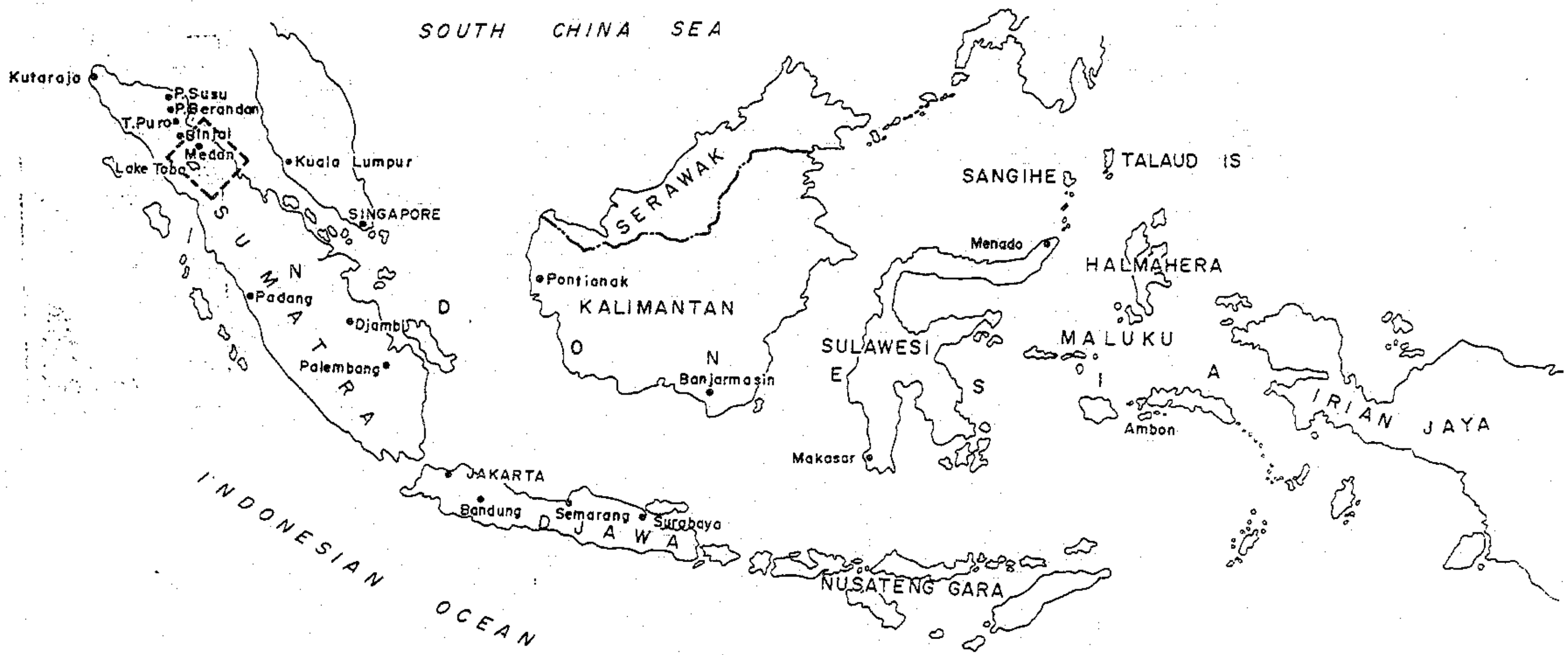
JULY 1978

JAPAN INTERNATIONAL COOPERATION AGENCY

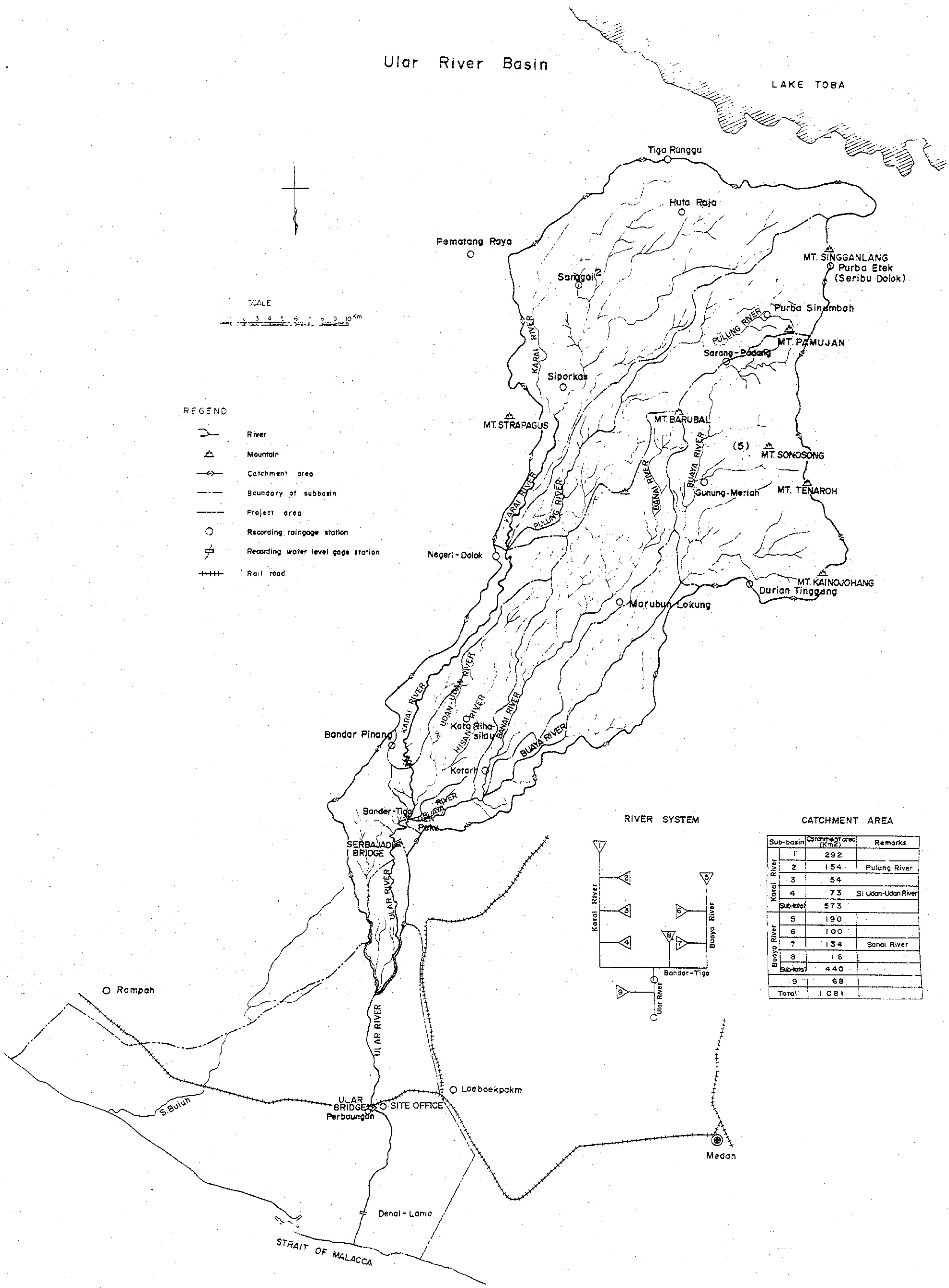
The Feasibility Report on Ular River Flood Control and Improvement of Irrigation Project is composed of three volumes.

Volume	I :	Main Report
Volume	II :	Study Report
Volume	III :	Supporting Report

Location Map



Ular River Basin

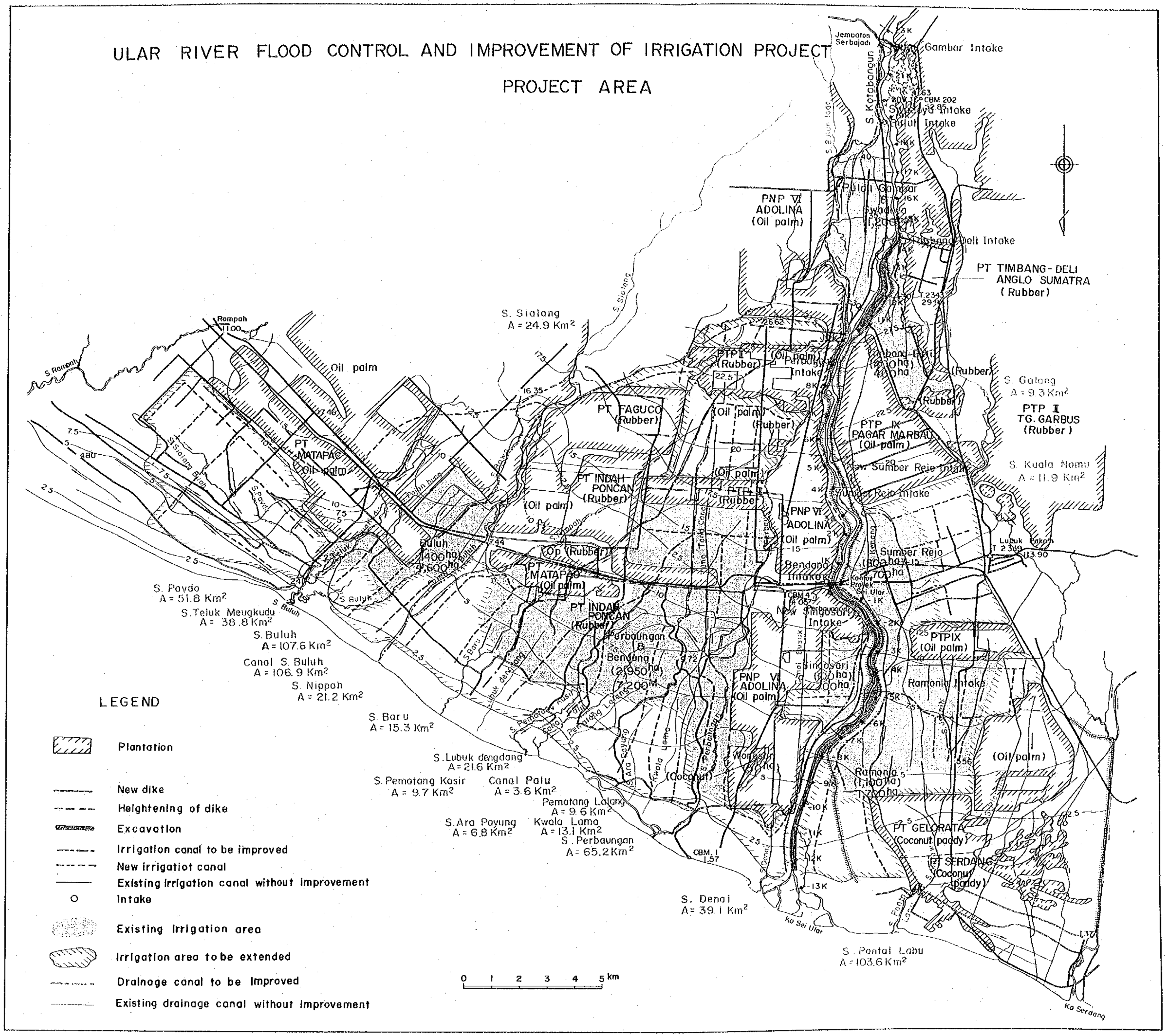


CATCHMENT AREA

	Sub-basin	Catchment area (Km2)	Remarks
Karai River	1	292	
	2	154	Pulung River
	3	54	
	4	73	Si Udan-Udan River
	Sub-total	573	
Buaya River	5	190	
	6	100	
	7	134	Banai River
	8	16	
	Sub-total	440	
	9	68	
	Total	1081	

ULAR RIVER FLOOD CONTROL AND IMPROVEMENT OF IRRIGATION PROJECT

PROJECT AREA



C O N T E N S

LIST OF TABLES

LIST OF FIGURES

DEFINITIONS

Abbreviations

Inception Report	Inception Report on Feasibility Study of Ular River Flood Control and Improvement of Irrigation Project.
Overall Plan	Overall Ular River Improvement Plan mentioned in Study Report on Overall Ular River Improvement Project.
Overall Plan Study	Study of Overall Ular River Improvement Project (Including Flood Control, Reclamation of Downstream Plain and Possible Irrigation Project) conducted by JICA in 1977 and 1978.
Study Team	Feasibility Study Team of JICA for Ular River Flood Control and Improvement of Irrigation Project.
Urgent Project	Ular River Urgent Flood Control Project.

Administrative Districts

Propinsi	Province.
Kabupaten	District.
Kecamatan	Subdistrict.
Desa or Kampung	Village.
Kota Madya	Municipality.
Bupati	Head of Kabupaten.
Camat	Head of Kecamatan.
Walikota	Head of Municipality, Mayor.

Acronyms

ADC	Agricultural Development Center.
BIMAS	Bimbingan Massal (Mass Guidance Program).
BMTD	Badan Musyawarah Tani Desa (Village Agricultural Cooperation Association).

BRI	Bank Rakyat Indonesia (Indonesian People's Bank).
BUUD	Badan Usaha Unit Desa (Village Unit Executive Body).
CIF	Cost, insurance and freight.
CRIA	Central Research Institute of Agriculture.
DGWRD	Directorate General of Water Resources Development, Ministry of Public Works and Electric Power.
DOLOG	Depot Logistik (Food Agency).
DPMA	Direktorat Penyelidikan Masalah Air (Directorate of Research of Water Problem).
DPU or PU	Dinas Pekerjaan Umum, Propinsi Sumatera Utara (Public Works Service, North Sumatra Province).
GDP	Gross Domestic Product.
IBRD	International Bank for Reconstruction and Development.
INMAS	Intensifikasi Massal (Agricultural Intensification Program).
IRR	Internal Rate of Return.
JICA	Japan International Cooperation Agency.
KUD	Koperasi Unit Desa (Village Unit Agricultural Cooperative).
NPV	Net Present Value.
PELITA	Pembangunan Lima Tahun (Five-Year Development Plan).
PNP	Perusahaan Negara Perkebunan (State Estate Enterprise).
PPL	Penyuluh Pertanian Lapangan (Extension Worker).
PPM	Penyuluh Pertanina Madya (Extension Supervisor).
PPS	Penyuluh Pertanian Spesialis (Subject-matter Specialist).

PROSIDA	Proyek Irigasi IDA (IDA Irrigation Project).
PTP	Perusahaan Terbatas Perkebunan (Private Estate Enterprise).
P3A	Petani Pengarap Pemakai Air (Water User's Association).
REC	Rural Extension Center.
RISPA	Research Institute of the Sumatran Planters Association.
S.	Sei or Sungai (river).
VHF	Very high frequency.
WUD	Wilayah Unit Desa (Village Unit).

Terminology

Balai Benih	Seed station.
Kebun Benih Sentral	Seed Center.
Polowijo	Second crop.

Datum of Elevation

LWS	Low Water Springs at Belawan Harbor.
UP	Reference datum of leveling for the Ular river project. No particular gage is installed for the UP, but the Bench Mark No.T-2339 installed in Lubuk Pakam is fixed to be 13.900 m UP in the reading.

Unit of Measurements

km	kilometer.
m	meter.
cm	centimeter.
mm	millimeter.
t	metric ton.

kg	kilogram.
gr	gram.
yr	year.
hr	hour.
s	second.
ha	hectare.
l	litter.
me	milligram equivalent.

Currency Equivalents

US$ 1 = Rp 415.

US$ 1 = ¥ 241.

CHAPTER I

FLOOD CONTROL

1.1. Topography of the Ular River Basin.

The Ular river basin is mainly composed of those of the Karai and the Buaya which originate in the northwestern somma of Lake Toba and its adjoining plateau about 1,200 m high in elevation. The Karai river flows to the northeast gathering the water from such tributaries as the Pulang and the Udan-Udan and joins with Buaya river which gathers the water from the tributary Banai. The Ular river, after joining the Karai and Buaya rivers, flows through the center of the agricultural area approximately 400 km^2 of the alluvial plain of the Ular river, and it debouches into the Strait of Malacca at a poin about 30 km east from Medan. The basin stretches over both Districts of Simalungun and Deli/Serdan.

The general aspect of the basin is shown in Fig.1-1-1 together with the river system and the catchment area of the sub-basins. The Ular river has a catchment area of 1,081 km^2 in total and a length of about 115 km from the river mouth to the headwaters. Profiles of the Ular river and its tributaries were drawn based on the topographic maps on a scale of 1/50,000 and are shown in Fig.1-1-2. As is seen from Fig.1-1-1 and Fig. 1-1-2, the Karai river basin is different from that of the Buaya in the shape and condition of the headwater region. The Karai river basin is long and narrow, and the headwater forms a plateau which has mean gradient of approximately 1/50, while the Buaya river basin is relatively short and wide, and the headwater forms steep-sloped mountains.

The Ular river runs through an area covered with andesite form in the Barisan volcano range, but the basin of the river is widely covered with acidic tuff, under which dacite and dacite tuff are found exposed on the banks. Near Mabar Bridge on the upstream course of the Buaya river, andesite mass is found (refer to the geological map in Study Report Overall Ular River Improvement Project, JICA, 1978). The downstream area of the Ular river is covered with fluvial and alluvial deposits mainly consisting of clay, gravel and sand. Near the coast line facing the Strait of Malacca, marine deposits are found.

1.2. Surveying and Soil Survey.

Topographic surveying, mapping and soil survey were carried out in the project area for obtaining data the study and planning. Their results are included in Vol. III, Supporting Report.

1.3. Hydrology.

1.3.1. Rainfall.

In and around the Ular river basin, seventeen recording rain-gages and fourteen ordinary rain-gages have been installed by the Ular River Project Office for observation of rainfall. The names of their stations and the years of starting of observation are as follows.

(1) Recording rain-gage stations.

	Station	Starting.
1)	Tiga - Runggu	Oct, 1972
2)	Sarang - Padang	Oct, 1972
3)	Gunung Meriah	Sep, 1972
4)	Negeri Dolok	Sep, 1972
5)	Kotari	Oct, 1972
6)	Perbaungan	Aug, 1972
7)	Paku	May, 1977
8)	Bandar Pinang	May, 1977
9)	Kota Rihasilau	May, 1977
10)	Marubun - Lokung	May, 1977
11)	Durian Tinggung	May, 1977
12)	Purba Sinumbah	May, 1977
13)	Siporkas	May, 1977
14)	Sanggai - Sanggai	May, 1977
15)	Pematang - Raya	May, 1977
16)	Huta - Raja	May, 1977
17)	Purba Etek (Seribu Dolok)	May, 1977

(2) Ordinary rain-gage stations.

1)	Paku	August, 1975
2)	Silinda	"
3)	Rumah - Deleng	"
4)	Bandar - Negeri	"
5)	Tiga - Juhar	"
6)	Marubun - Lokung	"

7)	Negeri - Kasihan	August, 1975
8)	Bah - Bah	"
9)	Sarang - Ganjang	"
10)	Siporkas	"
11)	Sanggai - Sanggai	"
12)	Pematang - Raya	"
13)	Huta - Raja	"
14)	Seribu - Dolok	"

At Silinda, the Directorate of Research of Water Problems (DPMA: Direktorat Penyelidikan Masalah Air) has a climatologic station equiped with recording rain-gage, thermometer, hydrometer, anemometer, evaporimeter and actinometer. Data are available at this station from February 1975. The DPMA also has a staff gage at Serbajadi Bridge, where data are available from August 1971.

In addition, the PNP (Government-owned Estate Enterprise; Perusahaan Negara Perkebunan) has a number of ordinary rain-gage stations in the downstream area of the Ular river. Data are available at these stations for more than 20 years, and monthly data are available at some stations for longer period. The locations of the above-mentioned rain-gage stations are shown in Fig.1-3-1.

The annual isohyetal map, Fig.1-3-2, of the northern part of Sumatra Island drawn by the Meteorological and Geophysical Institute, Department of Communication, based on the record from 1911 to 1940 shows that mean annual rainfall is 1,500 mm to 2,500 mm in the downstream area of the Ular river and increases toward upstream to reach the maximum 2,500 mm to 3,000 mm in the hilly area and decreases toward further upstream to 2,000 mm or 2,500 mm in the upmost plateau. This variation implies a close relation between rainfall and topography.

The records at the recording rain-gage stations of the Ular River Project Office and the ordinary rain-gage stations of the PNP indicate that this basin has the following hydrologic features (Table 1-3-1 and Figs.1-3-3 and 1-3-4).

a. Mean annual rainfall data obtained by the DPU and the PNP both show the similar tendency mentioned above that the annual rainfall has a close relation to topography as shown in the isohyetal map.

b. The hilly area seems to have two rainy seasons in a year; the first rainy season has the maximum rainfall in April or May and the second one in October or November.

c. The downstream plain seems to have one rainy season in a year;

the maximum rainfall appears in October or November.

d. In the upstream plateau, rainy and dry seasons are not distinguishable.

Correlations of daily rainfall between Bandar Pinang and Bandar Kuala, Siporkas and Sanggai-Sanggai, Purba Etek and Purba Sinumbah, and Durian Tinggung and Negri Dolok are shown in Figs. 1-3-5 to 1-3-9. There seems to be no correlation except Bandar Kuala and Bandar Pinang.

1.3.2. Discharge.

For observing water level of the Ular river, the Ular River Project Office has installed six recording water-level gages with ordinary staff gages. Their names and the starting years of observation are as follows.

Name	Starting
Bandar Tiga	August 1972
Perbaungan	Ditto
Denai-Lama	Ditto
Paku Bridge	May 1977
Bandar Pinang	Ditto
Serbajadi Bridge	Ditto

There are three gaging stations which provide discharge rating curves. One is located at Plau-Tagor (Serbajadi Bridge) under the management of the DPMA, another at Ular Bridge and the other at Bandar Tiga both under the management of the Ular River Project Office.

The Bandar Tiga station is located just downstream of the confluence of the Karai and the Buaya, the Pulau-Tagor station is located about 4 km downstream of the confluence and the Ular Bridge station about 22.6 km downstream of the Pulau-Tagor station. About 12.6 km downstream of the Pulau-Tagor station, the Pulau Gambar canal (S. Kotabangun) joins to the Ular river.

The discharge rating curve at the Ular Bridge station was drawn in 1972 before the commencement of construction works of the railway and the highway bridges. The present condition of the river channel around the bridges seems to be different from that in 1972 owing to the construction works. Therefore, the discharge rating curve at the Ular Bridge station may have to be redrawn after the completion of the whole construction works being done around the two bridges and scheduled excavation of low water channel and protection works.

According to the discharge duration curve at Pulau-Tagor (Fig.1-3-11) drawn based on the record of water level, it is found that the Ular river has abundant discharge. Even in 1972 when the discharge was a minimum in recent years, the minimum daily discharge exceeded 26 m^3/s and the mean daily discharge was larger than 50 m^3/s. Even in the dry season, average monthly rainfall is approximately more than 180 mm in the hilly area and approximately 100 mm in the plain area and the plateau.

On the occasion of the study of the Overall Ular River Improvement Project, study was made on the return period of peak discharges by the Thomas' Plot method by use of annual maximum discharges at Serbajadi Bridge. This was mainly by reason that no rainfall records for a period long enough for the analysis were available in the upper and middle regions of the basin and the runoff mechanism of the Ular river basin had not been made clear yet. This condition is also the same in the present study, or the same procedure was adopted to estimate the return period. However, the study was made adding the data obtained in 1977 and by use of the Gumbel and the Ven Te Chow methods beside the Thomas' plot method.

(1) Recalculation of return period by use of the data up to 1976.

According to the study of Overall Plan, the peak discharges at Serbajadi Bridge were estimated as follows.

1972	340 m^3/s
1973	610 "
1974	372 "
1975	255 "
1976	392 "

By use of these data, the peak discharge of return period of 2 years at the point of Serbajadi Bridge was calculated as follows.

380 m^3/s

Further, according to the Overall Plan Study Report, the biggest flood discharge at Serbajadi Bridge in the last 23 years was estimated at 865 m^3/s (1954), the second one at 610 m^3/s (1973) and the third one at 540 m^3/s (1969).

By use of the above-mentioned data, return period of the discharge 800 m^3/s at Serbajadi Bridge was calculated by the three methods.

(a) Thomas' Plot method.

33 years for 800 m^3/s.

(b) Gumbel method.

The equation for estimation of 2-year discharge is

$$X = 325.6 + 148.6\ Y$$

By this equation, 2-year discharge was estimated at 380 m^3/s. The equation for estimation of return period of peak discharge is

$$X = 258.3 + 170.7\ Y.$$

By this equation, the return period of 800 m^3/s is calculated at 24 years.

(c) Ven Te Chow method.

The equation for estimation of 2-year discharge is

$$X = 103.4\ K + 439.04.$$

By this equation, 2-year discharge was estimated at 422 m^3/s. The equation for estimation of return period of peak discharge is

$$X = 183.5\ K + 406.5.$$

By this equation, the return period of 800 m^3/s is calculated at 28 years.

(2) Calculation by use of the data including the data in 1977.

The annual maximum discharges at Serbajadi Bridge are as follows.

1972	340 m^3/s
1973	610 "
1974	372 "
1975	255 "
1976	392 "
1977	453 "

It is not exactly confirmed that the above-mentioned value 453 m^3/s in 1977 is the annual maximum, but this is probably the maximum or very close to it. This value was estimated based on the reading on the staff gage at Serbajadi Bridge on November 15 and 22, 1977. By use of these data and by the Thomas' Plot, the return period of 800 m^3/s was estimated at 32 years (refer to Fig.1-3-13).

By use of the Gumbel method, the equation for estimation of 2-year discharge is as follows.

$$X = 342.3 + 130.9\ Y$$

By this equation, 2-year discharge was estimated at 390 m^3/s. The equation for estimation of return period is as follows.

$$X = 267.0 + 164.9\ Y$$

By this equation, the return period of 800 m^3/s was calculated at 26 years.

By use of the Ven Te Chow method, the equation for estimation of 2-year discharge is as follows. By this equation,

$$X = 164.4\ K + 417.7$$

2-year discharge was calculated at 391 m^3/s. The equation for estimation of return period is as follows. By this equation,

$$X = 195.1\ K + 381.0$$

the return period of 800 m^3/s was calculated at 29 years.

The above calculations are summarized below.

Return Period of 800 m^3/s, in Years

	Thomas' Plot	Gumbel method	Ven Te Chow method	Average
Cal. in 1976	33	24	28	28.3
Cal. in 1977	32	26	29	29.0

It is seen from this table that the discharge 800 m^3/s has a return period of nearly 30 years. But it must be noted that the orthodox procedure for estimating return period could not be taken because of shortage of hydrological data. Studies should be continued on return period with the accumulation of data on hydrology. According to a paper/1 presented in 1976 on flood estimation from short records in Australia, the 10-year flood estimated from 30 or 50 year record was more than 50 % higher than the estimate from 10-year sequential samples which were selected from the said 30 or 50 year record as a sequential 10 year of low values. This fact tells us that data on hydrology must be accumulated before the accurate prediction of return period can be made. Therefore, it cannot be said that the calculation in the present study is more accurate than that in the study of the Overall Plan. In conclusion, it was decided to adopt the return periods calculated on the occasion of the Overall Plan Study.

/1 Walter C. Boughton: Flood Estimation from Short Recores, Journal of the Hydraulics Division, ASCE, March 1976.

The adopted return periods of discharges at Serbajadi Bridge are summarized as follows.

Discharge (m^3/s)	600	800	1,000	1,200
Return period (yr)	8	33	133	500

1.3.3. Tide Level.

(1) Observation of Tide Level.

Tide level was observed on 23, 24 and 25 of December in 1977 at S. Pantai Labu and at S. Buluh. The river mouth of S. Pantai Labu is located about 5 km northwest of the river mouth of the Ular, and the river mouth of the S. Buluh is located about 20 km southeast of the river mouth of the Ular. On the S. Pantai Labu, observation was made at the river mouth and at the river bank just before the tax office which is located about 2 km from the river mouth. On the S. Buluh, observation was made at the river bank just before the tax office of Sialang Buah which is located about 60 m upstream from the river mouth.

Observed data are listed in Table 1-3-2(1) and (2). In these tables, Pantai Labu[1] shows the data observed at the tax office and Pantai Labu[2] gives the data observed at the river mouth. The observation at the river mouth was conducted only in the day time owing to the security of safety for the observers. Fig.1-3-14 shows tide curves at the tax office and the river mouth of the S. Pantai Labu during the periods where the observations at both places were conducted at the same time. By use of these tide curves, correlation between both tide levels was studied and is shown in Fig.1-3-15.

(2) Tide Level at the River Mouth of the Ular.

Tide table is issued every year by Dinas Hidrografi TNI Angkatan Laut (Department of Hydrography, Navy) concerning tide levels at 60 places of the Indonesian archipelago. For the comparison of tide levels between the observed and the predicted at Belawan Deli, seven days' data around the vernal equinox day in 1977 have been taken out. The data are shown in Table 1-3-3 and are plotted in Fig.1-3-16. Correlation between the observed water level and the predicted one at the high water springs and the low water springs at Belawan Deli is shown in Fig.1-3-17.

It is said that the reading of the tide gage at Belawan Harbor (this is called Belawan Deli in the tide table) is larger by 0.4 m than that of tide level predicted by the tide table, that is, the reading of the Belawan Harbor gage minus 0.4 m makes the reading of the tide table. The said Table 1-3-3, Fig.1-3-16 and Fig.1-3-17 were made by this correction. It is seen from Fig.1-3-16 that the correlation is quite good with scattering of points within 10 to 20 cm. Considering that prediction is given at the accuracy of decimeter, it can be said that the predictions are sufficiently correct and also the correction of 0.4 m is correct.

Study was made of the correlation of tidal ranges between Belawan Deli and S. Asahan and between Belawan Deli and Teluk Aru during the period from April 13, 1976 to April 19, 1976 as shown in Table 1-3-3 by making use of data predicted in the tide table. The river mouth of the S. Asahan is located approximately 127 km southeast from the river mouth of the Ular, and Teluk Aru is located approximately 97 km northwest from the river mouth of the Ular. The results are shown in Figs.1-3-18 and 1-3-19, indicating that the tidal range at Sungai Asahan is about 60 % larger than that of Belawan Deli and the tidal range at Teluk Aru is nearly equal to that of Belawan Deli. On the other hand, the river mouth of the Ular is located about 30 km southeast from Belawan Deli. It may be assumed, therefore, that the tidal range at the river mouth of the Ular is nearly equal to or larger to some extent than that of Belawan Deli.

By use of the correlation between tide levels at the tax office and at the river mouth of the S. Pantai Labu (Fig.1-3-15) and the data on water levels observed at the tax office on the S. Pantai Labu during the period from December 23, 1977 to December 25, 1977 (Table 1-3-2(1) and (2)), tide levels at the river mouth of the S. Pantai Labu were estimated concerning the same period. These are shown in Table 1-3-4. On the other hand, the tide levels observed at Belawan Harbor during the same period as mentioned above have been corrected by 0.4 m and listed in Table 1-3-5.

Fig.1-3-20 shows the comparison of three tide curves of the corrected tide curve at Belawan Deli, the tide curve at the river mouth of the S. Pantai Labu and the tide curve at Sialang Buah (Table 1-3-2(1) and (2)) putting them in such a way that the mean water levels of these three curves during this period coincide with each other. It is easily seen that the high water levels at the river mouth of the S. Pantai Labu and at Sialang Buah nearly coincide with each other, while the low water levels do not coincide and are affected by water depths. Since the tidal range at the river mouth of the Ular must be equal to or a little larger than that of Belawan Deli as mentioned previously, the low water levels of the sea off S. Pantai Labu and Sialang Buah should be far lower than those shown in Fig.1-3-20. In view of this fact and judging from the shapes of the tide curves around the low water levels, there seems to be a possibility that the mean water level of the tide at S. Pantai Labu or Sialang Buah is higher than Fig.1-3-20 by about 30 cm. On this assumption, the relation between UP in meters and LWS in meters will be expressed as follows.

$$0 \text{ m UP} \fallingdotseq 2.1 \text{ m LWS}$$

The river mouth of the Ular is located between S. Pantai Labu and Sialang Buah, or located about 5 km southeast from S. Pantai Labu and about 20 km northwest from Sialang Buah. Therefore, the high water level at the river mouth of the Ular must be nearly equal to those at S. Pantai Labu and Sialang Buah. From Table 1-3-2, the normal high tide level at the river mouth of the Ular may be fixed at about 0.2 m UP. This tide level is converted to 2.3 m LWS by the above-mentioned relation between UP in meters and LWS in meters.

At Belawan Harbor located about 30 km northwest from the river mouth of the Ular, it is recorded that there occured a very high tide level in the past to have reached 3.0 m LWS or 0.9 m UP. Since the harbor is not so far from the river mouth, it can be considered that this height of tide level occured in the past also at the river mouth of the Ular.

1.3.4. Network of Hydrological Observation Stations.

The existing hydrological observation stations in and around the basin of the Ular river are listed below, where Project in parentheses denotes that the station belongs to the Ular River Project Office, PNP denotes that the station is owned by the PNP and DPMA means that the station is owned by the DPMA.

a. Recording rain-gage stations.

Tiga Runggu (Project)
Purba Etek (Seribu Dolok)(Project)
Huta Raja (Project)
Sangai-Sangai (Project)
Gunung Meriah (Project)
Durian Tinggung (Project)
Negeri Dolok (Project)
Marbun Lokung (Project)
Silinda (DPMA)
Purba Sinumbah (Project)
Sarang Padang (Project)
Siporkas (Project)
Pematang Raya (Project)
Kota Rihasilau (Project)
Kotari (Project)
Bandar Pinang (Project)
Paku (Project)
Perbaungan (Project)

b. Ordinary rain-gage stations.

Tiga Juhar (PNP)
Bandar Negeri (PNP)
Serbajadi (PNP)
Pulau Tagor (PNP)
Sungai Putih (PNP)
Sungai Karang (PNP)
Timbang Deli (PNP)
T. Garbus (PNP)
Tiga Juhar (Project)
Negeri Kasihan (Project)
Rumah Deleng (Project)

c. Recording water-level gage stations.

Bandar Pinang (Project)
Paku (Project)

Bandar Tiga (Project)	Serbajadi (Project)
Perbaungan (Project)	Denai Lama (Project)

The locations of these rain-gage stations and water-level gage stations are shown in Fig.1-3-1. Among the rain-gage stations, the following six stations were set in 1972.

Tiga Runggu :	in plateau area
Sarang Padang :	in plateau area
Gunung Meriah :	in hilly area
Negeri Dolok :	in hilly area
Kotari :	in plain area
Perbaungan :	in plain area

Data at these stations are available for more than four years. Correlations of rainfall among these stations were studied on the occasion of the Overall Plan Study, but no correlation was found. In 1977, some other stations were set in addition to the above. However, no significant correlation was found even when the new data were added in the present study.

The final purpose of rain-gage stations is to catch areal and time distribution of rainfall in order to grasp the run-off mechanism of the basin. From this viewpoint, the density of distribution of the existing rain-gage stations seems to be still thin. A rain-gage station only catches a point rainfall. Therefore, the representativeness of rain-gage stations must be reviewed.

In Japan, a standard on necessary density of distribution of rain-gage stations has been established and gives a value of 50 km^2 for one rain-gage in the case of a catchment area as large as that of the Ular. From this viewpoint, it is recommendable to set two more recording rain-gage stations; one is at Tiga Juhar in line with the proposal of the Overall Plan Study made by JICA in 1978 and the other one is at Serbajadi where an ordinary rain-gage is installed by the PNP.

After these two recording rain-gage stations have additionally been set, observation of rainfall and discharge should be continued for some period by use of all the rain-gages and the water gages until it is made clear whether the present density of distribution of hydrological gages is sufficient or not. In making the analyses, special attention must be paid to the tropical characteristics of behavior of rainfall and the relation between the topography and rainfalls.

In 1978, the JICA Study Team for the Overall Plan proposed to set additionally 8 recording water-level-gage stations, among which the following 5 stations are not installed yet.

Balapulung on the Buaya river (near the bridge).

Sipinggan on the Buaya river (no bridge).

Mabar on the Buaya river (near the bridge).

Negeri Dolok on the Karai river (near the bridge).

Esperance on the Ular river (no bridge).

Hence it is recommendable to set these stations as soon as possible. If these stations have been added, the analysis of runoff will fairly be improved and at the same time, it will serve to supply informations of floods. During the analyses, study must be made of the density of distribution of gaging stations which will serve for supplying data on water levels or discharges to be used for the future flood warning and forecasting.

The present flood information system is as follows.

a. If water level at the Bandar Tiga station has become higher than 2.0 m on the graduations of the staff gage, information is soon sent to the Ular River Project Site Office at Perbaungan by VHF radio telephone.

b. For the above purpose, three persons are standing by at the Bandar Tiga station for 24 hours.

c. The above information is soon sent to the following agencies under circumstances.

The military.

The police.

The district government.

The PNP.

d. On receiving information, necessary activities are commenced for flood fighting.

e. For the above purpose, five persons are standing by for 24 hours at the Ular River Project Site Office at Perbaungan.

f. The above information is soon sent to the Ular River Project Office in Medan.

In order to improve the present system of communication between the Ular River Project Office and the observation stations, it is recommendable to set the following 6 VHF radio stations.

VHF radio stations at Serbajadi Bridge, Bandar Tiga and Perbaungan Site Office.

Three mobile stations.

At present, only simple information can be made, as mentioned above, on occured flood because of insufficient distribution of rain-gage stations and gaging stations. This project expects to strengthen the networks of the stations. This will serve for the establishment of flood-forecasting and warning system in the future. However, in order to establish the system, it is necessary to reinforce the study on analysis of the relations between rainfalls and runoffs as well as the relations between water levels or discharges studying the suitability of distribution of the stations on the one hand and strengthening the communication system for transmitting the informations on hydrologic data on the other hand.

1.4. Flood Control Plan.

1.4.1. Present Condition of the River.

(1) Ular River.

The total catchment area of the Ular river basin is 1,081 km^2 and the total length is 115 km. Slope of the river bed is 1/1,200 to 1/1,800 in the lower reaches between the river mouth and Ular Bridge, and 1/800 to 1/700 between Ular Bridge and Serbajadi Bridge. Average grain size of bed materials is approximately 1.15 mm at Ular Bridge. Sediment discharge is estimated at approximately 800,000 m^3/yr. Fig.1-4-1 shows the stretch of improvement extending from -12.25 km to 22.65 km.

(a) From river mouth to -7.5 km.

No large-scale levees are constructed on both sides but small dikes have been built along the low-water channel to protect paddy fields and residences from floods. The dikes are not high enough for protecting large floods. This area thus forms a flood plain and moreover suffers from habitual inundations mainly due to insufficient capacity of the low-water channel.

The main river is divided into two courses at a point near -12.5 km. At present, the right course is the main channel and the left one is a distributary because the channel has become shallow due to sedimentation. This condition of the river indicates us that it is too early to fix the river channels downstream from the bifurcation.

(b) From -7.5 km to -2.5 km.

On the left side, a levee has already been constructed. On the right side between -6.0 km to -2.5 km, a levee has also been constructed. However, since the river width near -4.0 km is only 150 m, widening must be made near this constriction.

(c) From -2.5 km to 0.0 km.

Levees on both sides between -2.5 km and 0.0 km have already been constructed. Height of the landside ground is almost the same as the river bed.

(d) From 0.0 km to 10.0 km.

Levees on both sides were constructed with the design discharge of 600 m^3/s during the period of Urgent Flood Control Project from 1972 to 1976. This project resulted in remarkable decrease in damages in the middle reaches. For maintenance of levees, weedings are carried out once a month by man power with tools. The constrictions formed by the old bridges for highway and railway are to be mitigated by the rebuilding, but still remain some constrictions especially due to the railway bridge. However, the authorities have a plan to expand the total span further.

(e) From 10.0 km to 15.0 km.

On the right side between 10.0 km and 15.0 km, there exist small dikes of 1 m to 1.5 m in height along the low-water channel. Pulau Gambar area which is located inside of the small dikes suffers damages from habitual inundation owing to insufficiency of carrying capacity of the river channel. On the left side of the river, a levee is constructed and connected with a hill at 15.0 km.

(f) From 15.0 km to 22.65 km.

The left side of the river is formed by hills. The right side consists of hilly land and some levees. The levee, however, is not sufficient for confining the design discharge.

(2) Pulau Gambar Canal.

Pulau Gambar Canal, S. Kotabangun, which pours into the Ular river at a point near 10.0 km on the right side, is a small tributary that has a drainage area of about 38 km^2 and a length of 15 km. It has two tributaries named S. Kotabangun and S. Buluh laga. S. Kotabangun is an irrigation canal diverted from the Ular river at a point near 22 km, while S. Buluh originates in low hills about 75 m high above the sea level. S. Kotabangun and S. Buluh join to the old Ular river which is now used as an irrigation canal named Buluh Canal. The old Ular river (Buluh Canal) is divided into two canals at Sennah divergence; one is Buluh irrigation canal and the other is Pulau Gambar Canal (S. Kotabangun).

The Sennah divergence has gates for irrigation-water supply and drainage (spillway). When irrigation water is enough, drainage water collected by S. Kotabangun and S. Buluh laga is discharged to Pulau Gambar Canal (S. Kotabangun) through the Sennah drainage gates (spillway).

At present, the lower basin of Pulau Gambar Canal (S. Kotabangun) plays a role of natural retarding basin since the right-side levee of the main river is not closed yet.

The carrying capacity of the lower reaches of Pulau Gambar Canal is estimated at about 6 m^3/s, which is extremely small compared with the design discharge 25 m^3/s obtained by the study of drainage requirement.

1.4.2. Carrying Capacity of the Existing River Channel.

For reviewing the carrying capacity of the existing river channel of the Ular, non-uniform flow calculation was applied to the stretch from -12.25 km to 22.65 km. The conditions for the non-uniform flow calculation are as follows.

(1) Cross-sections of river channel.

Cross-sectional levelings were conducted by the JICA Surveying Team at intervals of 250 m on the two stretches between -13.5 km and 0.0 km and between 10.0 km and 23.0 km during the period of August to October in 1977. The stretch between 0.0 km and 10.0 km was surveyed in 1976 also by the JICA Surveying Team. Cross-sectional surveying on this stretch was conducted at intervals of 1 km. The cross sections drawn by the JICA Surveying Team were used for the calculations.

(2) Coefficient of roughness.

Coefficients of roughness for the calculations were assumed as follows considering the present condition of the river channel.

For the low-water channel,

$n = 0.033$ in case of 200 m^3/s and 400 m^3/s, and

$n = 0.030$ in case of 600 m^3/s and 800 m^3/s.

For the high-water channel,

$n = 0.070$ downstream from 15 km, and

$n = 0.060$ upstream from 15 km.

(3) Water stage at the lower end of the river.

In calculating water level, the lower end was set at -12.25 km, and a stage-discharge curve at this point was drawn by uniform-flow calculation. Water levels corresponding to several discharges were read on this curve and adopted as the stating water levels for calculation of water levels on the upstream reaches.

(4) Calculated water level.

Calculations of water level were conducted by use of the above-mentioned coefficient of roughnesses with regard to four cases of 200 m^3/s, 400 m^3/s, 600 m^3/s and 800 m^3/s. The results are shown in Fig.1-4-2. This figure indicates the following.

a. In the reaches downstream from a point near -6.25 km, the carrying capacity is only 200 m^3/s or less.

b. In the reaches between -6.0 km and -4.0 km, the carrying capacity is about 400 m^3/s. Around the point -4.0 km, the river width is extremely small. This constriction raises the water level between -4.0 km and -2.0 km and reduces the carrying capacity to less than 400 m^3/s. The reaches between -2.0 km and 0.0 km have a carrying capacity of more than 400 m^3/s but less than 600 m^3/s.

c. The reaches between 11.0 km and 15.0 km which include the Pulau Gambar area have a carrying capacity less than 400 m^3/s because no levee exists on the right-side bank.

d. In the reaches upstream from 15.0 km, the carrying capacity is almost more than 400 m^3/s except a few places, but less than 600 m^3/s.

1.4.3. Study of Retarding Basin in Pulau Gambar Area.

(1) General.

An area located upstream of the confluence (near the distance-mark 10 km) of Pulau Gambar Canal (S. Kotabangun) and the main stream of the Ular river was selected as a candidate site for retarding basin, and the function as a retarding basin was studied in this Section. The area named as Pulau Gambar including the candidate area is administratively managed by Galang Subdistrict. The Pulau Gambar area plays an important role in rice production, but this area suffers from flood about once or twice a year.

Plan of retarding basin is outlined as below.

a. A levee shall be constructed on the right side of the Ular river along the Pulau Bambar area in accordance with the Overall Ular River Improvement Plan.

b. Deversoir shall be provided on a part of the said levee. A part of discharge including the peak shall be cut by the deversoir.

c. Flood discharge of the lower reaches downstream from the deversor will be reduced. The present inundation shall be confined within the retarding basin. Furthermore, the frequency of inflow into the area will be reduced by provision of new levee compared with the present state.

The retarding basin shall be composed of the following elements.

a. One deversoir for the purpose of inflow of a part of flood discharge of the main stream.

b. Enclosing levee which shall be constructed on the circumference of the retarding basin.

c. One sluice for drainage of water stored in the retarding basin. The sluice shall be opened after the flood is over.

The required volume of the retarding basin must be increased and the crown of the enclosing levee must also be made high as the discharge to be cut becomes large.

In the present study, under the condition of the hydrograph with 800 m^3/s in peak (Fig.1-4-3), the following three cases were studied as the alternatives of retarding basins, and construction costs were estimated.

a. 50 m^3/s is cut from the peak discharge of 800 m^3/s and the flood discharge will be reduced to 750 m^3/s on the lower reaches from the deversoir.

b. 100 m^3/s is cut from the peak discharge of 800 m^3/s and the flood discharge will be reduced to 700 m^3/s on the lower reaches from the deversoir.

The costs for improvement of the reaches downstream from the deversoir must be decreased as the design discharge is decreased to 750 m^3/s, 700 m^3/s and 600 m^3/s. These were estimated in comparison with the cost for 800 m^3/s. Next, comparison was made of the increase in cost due to the provision of retarding basin and the decrease in cost due to cutting of the peak discharge by the retarding basin.

(2) Hydraulic elements of deversoir.

Water level in a retarding basin was calculated by the following equations.

$$H_u = f(Q)$$

$$H_r = f(V)$$

$$V_t = \frac{q_{t-1} + q_t}{2} \Delta t$$

where H_u : water level of the river, in m

H_r : water level of the retarding basin, in m

Q : discharge of the river, in m^3/s

V : volume of the retarding basin, in m^3

V_t : stored volume until time t

q : overflow discharge, in m^3/s

Δt : time interval, in sec.

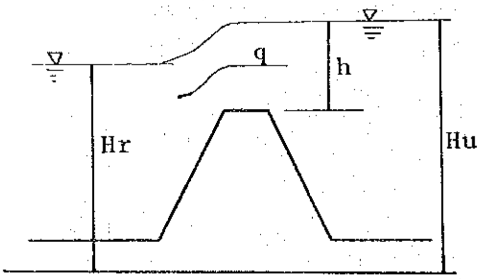
For calculation of q, the Ito-Homma formula was applied.

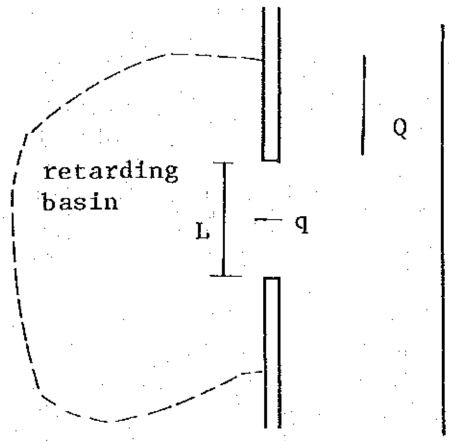
$$q = \frac{2}{3} \mu\sqrt{2g}\cdot L\ h^{3/2}$$

where μ : coefficient of overflow

h : water depth above the crest of deversoir, in m

L : length of deversoir, in m.





Calculations were conducted on the conditions mentioned below

a. The hydrograph shown in Fig.1-4-3 (refer to Fig.3-4-16 in Overall Plan Study Report, 1977) can be used as the hydrograph of the main river at the site of proposed deversoir.

b. Overflow into the retarding basin begins at 400 m^3/s of the main river (about 2-year return period).

c. Storage capacity of the retarding basin is given as shown in Fig.1-4-4.

d. Water level of the main river channel at the deversoir is calculated by applying the design cross section and the discharge volume in the river.

Storage volume necessary for retarding in each case of the three alternatives is shown in Fig.1-4-3. Based on these calculations, the elements of the deversoir were set as shown in the following table.

Case No.	Discharge on lower reaches	Max. overflow discharge	Location of deversoir	Length of deversoir	Storage capacity	Area of retarding basin	Max. W.L. basin	H.W.L. river
1	750 m^3/s	50 m^3/s	10.75 k +185 m	90 m	$1.0 \times 10^6 m^3$	1.4 km^2	30.5 m UP	31.0 m UP
2	700	100	11.25 k + 65 m	175	1.9×10^6	1.9	31.0	31.5
3	600	200	11.75 k +305 m	350	3.9×10^6	2.3	32.0	32.5

(3) Basic design of retarding basin.

The plans of the three alternatives are shown in Fig.1-4-5. The crown height of enclosing levee was set 80 cm higher than the design water level of the retarding basin as shown in the following table.

Case No.	H.W.L. of retarding basin	Crown height of enclosing levee
1	30.5 m UP	31.3 m UP
2	31.0 "	31.8 "
3	32.0 "	32.8 "

Longitudinal profile of the levee on the right bank of the Ular river is shown in Fig.1-4-6 and longitudinal profile of the enclosing levee on the right side of Pulau Gambar Canal is shown in Fig.1-4-7.

Crown width was set at 4 m and gradient of slope of 1 : 2 was adopted. Typical cross sections are shown in Fig.1-4-8. Basic design of the deversoir of case 2 is shown in Fig.1-4-9.

The dimensions of drainage sluices were estimated in three cases of alternatives on the assumption that the stored water is drained in 35 hours in every case. The calculated dimensions are as follows.

Case No.	Area required (sq.m)	Dimension H × W × number (in m)
1	4	2 × 1 × 1
2	8	2 × 2 × 2
3	16	2 × 2 × 4

(4) Cost estimate of three alternatives.

Increases in costs due to construction of retarding basin in comparison with the construction of dike under the plan of Overall Ular River Improvement Project are as follows.

Case 1

Embankment	33,900	m^3	900 Rp/m^3	30,510,000 Rp
Main river	2,300			
Pulau Gambar canal	26,300			
Upper side	5,300			
Deversoir	90	m	800,000 Rp/m	72,000,000
Drain sluice	1	gate		26,900,000
Land	1.4	km^2	30 Rp/m^2	42,000,000
Miscellaneous			(30%)	51,590,000
Total				223,000,000

Case 2

Embankment	64,600	m^3	900 Rp/m^3	58,140,000 Rp
Main river	7,900			
Pulau Gambar canal	50,500			
Upper side	6,200			
Deversoir	175	m	800,000 Rp/m	140,000,000
Drain sluice	2	gate		53,800,000
Land	1.9	km^2	30 Rp/m^2	57,000,000
Miscellaneous			(30%)	93,060,000
Total				402,000,000

Case 3

Embankment	144,200	m^3	900 Rp/m^3	129,780,000 Rp
Main river	31,300			
Pulau Gambar canal	106,800			
Upper side	6,100			
Deversoir	350	m	800,000 Rp/m	280,000,000
Drain sluice	4	gate		107,600,000
Land	2.3	km^2	30 Rp/m^2	69,000,000
Miscellaneous			(30%)	176,620,000
Total				763,000,000

Decreases in costs due to cutting of peak discharge by retarding basin were estimated in comparison with the plan of Overall Ular River Improvement Project and the improvement works for 600 m^3/s, 700 m^3/s and 750 m^3/s were compared with those for 800 m^3/s as shown in the following table. As seen in this table, the height of embankment of the former is lower than the latter, the necessary land for the former is smaller than the latter and consequently the construction costs of the former becomes smaller than the latter.

Height of Embankment (from design river bed)

Section	800 m^3/s	750 m^3/s	700 m^3/s	600 m^3/s
-12.25k to -10.75k	3.67 m	3.59 m	3.50 m	3.11 m
-10.75 to -5.75	3.60	3.52	3.43	3.04
-5.75 to 2.00	3.58	3.50	3.42	3.04
2.00 to 10.00+343^m	3.52	3.45	3.37	3.00

Decrease in Embankment Volume

Section	length of embank.	750 m^3/s	700 m^3/s	600 m^3/s
-12.25k to -10.5k	1,820 m	3,200 m^3	6,600 m^3	19,800 m^3
-10.5 to - 7.5	3,035	7,100	14,900	46,000
-7.5 to - 2.5	4,860	13,900	28,000	87,700
-2.5 to 0.0	2,720	9,900	19,500	63,200
0.0 to 10.0+343^m	10,163	27,100	57,300	189,800
Total	22,598	61,200	126,300	406,500

Decrease in Land Area

Section	length of embank.	750 m^3/s	700 m^3/s	600 m^3/s
-12.25k to -10.5k	1820 m	1,200 m^2	2,500 m^2	8,200 m^2
-10.5 k to - 7.5k	3035	1,900	4,100	13,600
-7.5 k to - 2.5k	2720	1,700	3,500	11,800
0.0 k to 10.0+343^m	Already acquired			
Total	7,575 m	4,800 m^2	10,100 m^2	33,600 m^2

Based on the above-mentioned decreases, decrease in construction costs were estimated as shown in the following table.

Case 1 (750 m^3/sec)

Embankment	61,200 m^3	900 Rp/m^3	55,080,000 Rp
Land	4,800 m^2	30 Rp/m^2	144,000
Miscellaneous		(30%)	16,567,000
Total			71,791,000

Case 2 (700 m^3/sec)

Embankment	126,300 m^3	900 Rp/m^3	113,670,000 Rp
Land	10,100 m^2	30 Rp/m^2	303,000
Miscellaneous		(30%)	34,192,000
Total			148,165,000

Case 3 (600 m^3/sec)

Embankment	406,500 m^3	900 Rp/m^3	365,850,000 Rp
Land	33,600 m^2	30 Rp/m^2	1,008,000
Miscellaneous			110,057,000
Total			476,915,000

(5) Comparison of construction costs of three alternatives.

Elements of the alternatives of retarding basin are summarized as follows.

Case NO.	Cut of dis-charge	Deversoir			Retarding basin			Length enclosing Levee	Drain sluice
		Place	Length	EL. of crown	Area	Storage capacity	Max. W.L.		
									H×W×No.
1	50m^3/s	10.75k +185m	90m	30.5m	1.4km^2	$1.0\times10^6 m^3$	30.5m	3860m	2×2×1
2	100	11.25k + 65m	175	31.0	1.9	1.9×10^6	31.0	5120	2×2×2
3	200	11.75k +305m	350	32.0	2.3	3.9×10^6	32.0	5970	2×2×4

Increases in costs by construction of retarding basin compared with the costs of the river improvement works in the Overall Ular River Improvement Project are as follows.

Increase in Construction costs unit:Rp.1,000.

Case NO.	Embank-ment	Deversoir	Drain sluice	Land	Miscellaneous	Total
1	30,510	72,000	26,900	42,000	51,590	223,000,000
2	58,140	140,000	53,800	57,000	93,060	402,000,000
3	129,780	280,000	107,600	69,000	176,620	763,000,000

Decrease in Construction costs unit:Rp.1,000.

Case NO.	Embank-ment	Deversoir	Drain sluice	Land	Miscellaneous	Total
1	55,080	---	---	144	16,567	71,791
2	113,670	---	---	303	34,192	148,165
3	365,850	---	---	1,008	110,057	476,915

Cost compared with the river improvement with 800 m /sec.

Case 1	223,000,000 - 71,791,000 = + 151,209,000 Rp
Case 2	402,000,000 - 148,165,000 = + 253,835,000 Rp
Case 3	763,000,000 - 476,915,000 = + 286,085,000 Rp

Channel improvement plan without artificial retarding basin, as was proposed in the Study Report on Overall Ular River Improvement Project, JICA, 1978, is recommendable from the following point of view.

a. The candidate area for construction of retarding basin has well been developed to be high-productive rice field. The loss to be caused by the construction of retarding basin should be avoided.

b. The construction costs with retarding basin are larger than those without it.

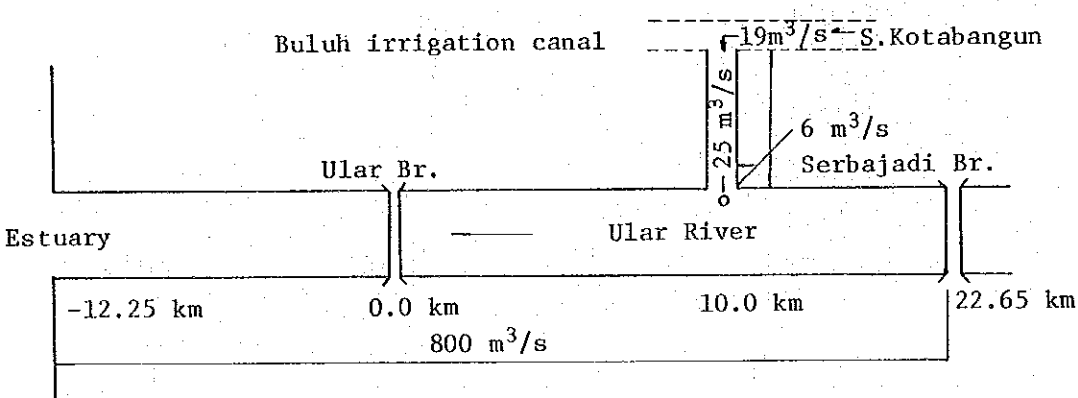
1.4.4. Design Discharge.

A discharge of 800 m^3/s was adopted as the design discharge at a point of Serbajadi Bridge in accordance with the decision of the Overall Plan Study that was made by the JICA in 1978. Its return period was estimated at approximately 30 years also in the present study.

As concluded in the previous paragraph 1.4.3, it was decided to improve the river channel of the Ular without artificial retardation of flood in the Pulau Gambar area. Therefore, it was decided to apply the design discharge of 800 m^3/s to the whole stretch of the Ular from Serbajadi Bridge to a point of -12.25 km near the river mouth.

Concerning the tributary, Pulau Gambar Canal or S. Kotabangun, the peak discharge at the Sennah divergence or the peak discharge before diversion to the Ular river was estimated at 19 m^3/s as mentioned in the paragraph (3), 2.3.4. Adding 6 m^3/s of a runoff from the remaining area to the above discharge 19 m^3/s, the discharge 25 m^3/s was adopted as the design discharge of the S. Kotabangun at its confluence with the Ular river.

Thus the discharge allocation adopted in the present study for the improvement of the rivers is as follows.



1.4.5. Improvement plan of the Ular river.

(1) General.

In this study, planning for improvement was made on the stretch of river channel extending from -12.5 km near the river mouth (0 km is located at Ular Bridge) to +22.65 km upstream of the Ular river. The stretch from -12.25 km to the real river mouth was put out of planning because, in our judgement as well, this stretch is located in a too swampy area to fix a low-water channel.

As mentioned in the previous paragraph, the capacity of the existing river channel is not enough for carrying the design discharge of 800 m^3/s. Therefore, to meet this discharge, improvement of the river channel was planned in line with the decision of the Overall Plan Study. Concerning the Pulau Gambar area, an artificial retarding basin was studied with the view of reducing the magnitude of design discharge. But this plan was not adopted from the viewpoint of economy.

The principles taken in planning the improvement are as follows.

a. The existing alignment of river channel shall be paid high regard in planning new alignment.

b. The width between new levees shall be set at 250 m as standard.

c. In case the width between the existing levees is enough, the levees shall be used by heightening or widening at need.

d. Necessary carrying capacity shall be secured by excavation and/or dredging works together with by building new levees or strengthening the existing levees.

e. The total spans of the railway and highway bridges are assumed to be left as they are after the completion of the present re-building works.

f. Concerning the treatment of the confluence of the Ular river and Pulau Gambar Canal (S. Kotabangun), the enclosing-levee system shall be adopted in line with the conclusion of the study mentioned in 1.4.6.

g. In designing cross sections of the river, attention shall be paid to the water level of intake.

In planning the improvement of the river channel, the following data were used.

a. The topographical maps on a scale of 1/10,000 for planning the alignment of the river channel. These were provided by the JICA.

b. For planning cross sections of the river channel, (1) 106 cross sections which were surveyed at intervals of 250 m and drawn in 1977 by the JICA Surveying Team over a stretch of about 35 km upstream from the river mouth except a stretch of about 10 km upstream from the highway bridge, and (2) the cross sections which were surveyed at intervals of 1 km and drawned in 1976 by the JICA Surveying Team over a stretch of about 10 km upstream from the highway on the occasion of the Overall Plan Study.

c. The results of soil survey that was conducted by the Soil Survey Group of the Study Team, as compiled in Vol. III of this Report.

(2) Hydraulic conditions for design.

a. Design discharge.

A discharge of 800 m^3/s was adopted as the design discharge for the main river channel of the Ular as mentioned previously. The return period of this discharge is about 30 years.

b. Water level at the river mouth.

At the lower end of the stretch for improvement, -12.25 km, a stage-discharge curve was drawn by uniform-flow calculation. On this curve, a water level which corresponds to the design discharge 800 m^3/s was read at 3.46 m UP. On the other hand, calculation was made of a water level which will appear at the point -12.25 km on condition of 800 m^3/s and a river width of 200 m by use of the non-uniform flow method starting from the sand bar located at the real river mouth which is about 1.5 km downstream from the lower end of the river. Comparison of the two water levels showed that the former is a little higher than the latter. From the viewpoint of safety, the former value 3.46 m UP was adopted as the design water level at the lower end of the river.

c. Manning's coefficient of roughness.

In consideration of the present condition of river channel including both the major and minor beds and the level of practical maintenance in future, the following Manning's coefficients of roughness were adopted for design, where n_1 is coefficient of roughness for low-water channel and n_2 is coefficient of roughness for high-water channel.

n_1 = 0.028 and n_2 = 0.060 for -12.25 km to 0.00 km.

n_1 = 0.030 and n_2 = 0.060 for 0.00 km to 22.65 km.

(3) Proposed improvement of levees.

Improvement of levees was planned as follows.

a. The levees located between 0.0 km and 10.0 km were already rebuilt on the occasion of the Urgent Flood Control Project. Therefore, the alignment of these levees shall be left as they are, but only heightening of the levees was planned so as to meet the design discharge 800 m^3/s. In this stretch, the standard river width was set at approximately 250 m.

b. On the stretches located downstream from 0.0 km and upstream from 10.0 km, new levees or heightening of levees were planned respectively at the river width of 260 m and 250 m considering to utilize the existing levees or natural banks as much as possible.

c. The existing levee located on the left side of the stretch upstream from 10.0 km was planned to be heightened and strengthened and connected with a hill located near 15.0 km.

d. On the right side between 10.0 km and 22.65 km, a new levee was planned on the stretch from 10.0 km to 19.0 km, and heightening was planned of the existing levee located from 19.0 km to 22.65 km.

e. On the left side of the stretch from 0.0 km to the river mouth, heightening was planned of the existing levee located from 0.0 km to -7.5 km, and a new levee was planned on a stretch from -7.5 km to -12.25 km.

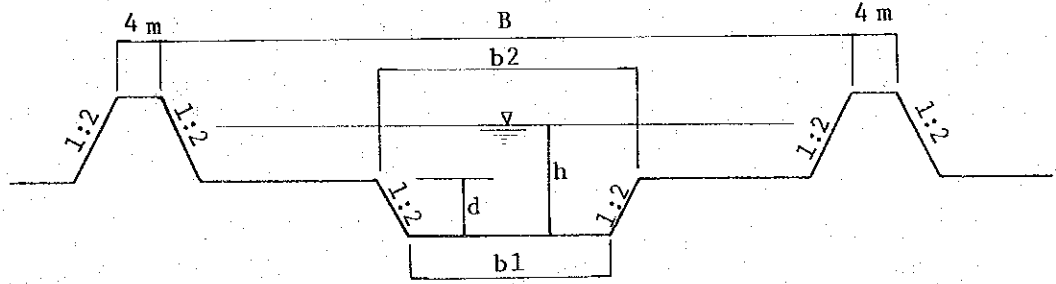
f. On the right side of the stretch from 0.0 km to the lower end of the river, heightening was planned of the levee located between 0.0 km and -2.5 km, and a new levee was planned from -2.5 km to -12.25 km.

(4) Longitudinal profile.

Design elevations of low-water and high-water channels design freeboard were fixed following the principle on the occasion of the Overall Plan Study. It was confirmed by non-uniform flow calculation that the carrying capacity of every section of the design channel is sufficient. Thus the design longitudinal profile of the Ular river is shown in Fig.1-4-10.

(5) Proposed cross sections.

The cross section shown below was adopted as standard in accordance with the Overall Plan.



	d (m)	h (m)	b_1 (m)	b_2 (m)	B (m)
-12.25 to -10.75 km	1.2	3.07	85.5	90.3	260
-10.75 to - 5.75 +159 m	1.2	3.00	76.0	80.8	260
-5.75 +159 m to 2.00 +204 m	1.2	2.95	69.2	74.0	250
2.00 +204 m to 10.00 +343 m	1.2	2.95	65.2	70.0	250
10.00 +343 m to 19.00	1.2	2.89 to 2.79	70.0	74.8	250
19.00 to 22.65		Existing			

Typical river cross-sections proposed based on the above standard are shown in Figs.1-4-11 and 1-4-12. Stability of levees was confirmed with regard to the three sections shown in Fig.1-4-13. Levees located from -12.25 km to 0.0 km shall be coated with cohesive soil 0.5 m in thickness and levees from 0.0 km to 22.65 km with cohesive soil 0.3 m in thickness.

A calculation of sediment discharge was made for conjecturing the tendency of change in river bed after the completion of river channel improvement. In general, a river bed is mainly formed by bed load and change in river bed is also mainly affected by bed load. As it seems to be the same in the case of the Ular river, attention was paid to only bed load in this case.

The sediment discharge was calculated by use of the Sato-Kikkawa-Ashida formula on the whole stretch of improvement between -12.25 km at its lower end and 22.75 km a little upstream of Serbajadi Bridge. The grain size of sediment was assumed at

$$d_m = 1.15 \text{ mm}$$

over the whole stretch of river channel in the same way as in the Overall Plan Study.

The Sato-Kikkawa-Ashida formula is expressed as follows.

$$q_B = \Psi \cdot u_*^3 \cdot F(\tau/\tau_c)/(\sigma/\rho - 1) \cdot g$$

where

q_B : bed load per unit width per unit time ($m^3/s/m$; net volume of sediment particles without void),

u_* : friction velocity (m/s),

τ : tractive force of flow (gr/cm^2),

τ_c : critical tractive force (gr/cm^2),

$F(\tau/\tau_c)$: a given function of τ/τ_c (non-dimension),

ρ : density of water ($gr \cdot s^2/cm^4$),

σ : density of sediment particle ($gr \cdot s^2/cm^4$)

g : acceleration of gravity (m/s^2), and

Ψ : coefficient (non-dimension; $\Psi = 0.62$ for $n > 0.025$).

The function F is given as follows.

τ_c/τ	0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9	1.0	1.1	1.2
F	1	0.997	0.978	0.927	0.835	0.697	0.527	0.355	0.212	0.111	0.048	0.018	0.006

With a view of comparing the bed loads in the upper and lower parts of the whole stretch, a reach between 20.75 km and 22.75 km was chosen as the upper part and another reach between -6.0 km and -4.0 km as the lower part. The latter part was chosen by reason that the stretch which has levees on both sides ends at -6.0 km. For both the parts, bed-load discharge was calculated on several water discharges at five sections at intervals of 250 m in the respective upper and lower reaches of 2 km. The bed-load discharge averaged over the five sections at a discharge was regarded as the representative bed-load discharge in the reach corresponding to the water discharge.

Correlations were obtained between water discharge and bed-load discharge respectively in both the upper and lower reaches in respect of the two cases of existing and design river channels. These are shown in Fig.1-4-14.

Next, a calculation was made of discharge frequency distribution during one year based on the average discharge duration curve drawn by use of the discharge data obtained at Serbajadi Bridge in the period of 3 years from 1972 to 1974. By use of the discharge frequency distribution and the above-mentioned correlation curves regarding water discharge and sediment discharge, the sediment discharge distribution during one year with regard to different magnitudes of water discharge was obtained in respect of the two cases of existing and design river channels. These are shown in Figs.1-4-15 and 1-4-16. In the figures, black circles indicate the upstream part of the river channel (from 20.75 km to 22.75 km) and white circles the downstream part (from -4.0 km to -6.0 km). The sediment discharge in this calculation is expressed by m^3/s, where the volume of sediment is expressed by net volume of sediment particles without void.

It is seen from the figures that, in this river, a water discharge 50 m^3/s gives the maximum sediment discharge throughout the year with regard to both the cases of existing and design river channels. This discharge therefore implies that the sediment transported by this discharge will contribute to the formation of river bed. Based on this conception, a profile of sediment discharge caused by the water discharge 50 m^3/s was drawn and shown in Fig.1-4-17, where the full line shows the existing river channel and the broken line shows the design river channel. It is easily seen from this figure that the logitudinal distribution of sediment discharge in the design river channel will remarkably be improved compared with the existing river channel. However, it is also seen from the figure that the sediment discharge in the upper reaches is somewhat larger than that in the lower reaches. Therefore, there will be a tendency that a sediment deposit may occur in the lower reaches. For maintaining the lower channel, some dredging work will be needed.

(6) Revetment.

Revetment works were planned at some places where levees are critical due to erosion at river banks. The standard design for them is shown in Fig.1-4-18. The locations of planned revetment works are shown in Fig.1-4-1. The total length is 1,800 m.

1.4.6. Treatment of Confluence of Pulau Gambar Canal and the Ular River.

(1) General.

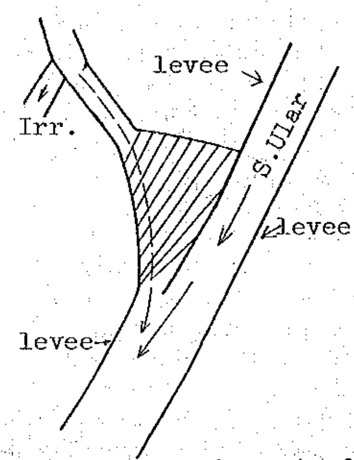
The present condition of Pulau Gambar Canal was already mentioned in (2), 1.4.1. The drainage area is shown in Fig.1-4-19. As mentioned before, the carrying capacity of the existing Pulau Gambar Canal is estimated at 6 m^3/s on its lower reaches. This is extremely small compared with the design discharge 25 m^3/s. On the other hand, as concluded in the previous paragraph, no artificial retarding basin will be set in this area. As a result, Pulau Gambar Canal must be improved in order to protect this area from the flooding of the Ular as well as the canal's own floods. Since the embankment works of the Ular river is being planned, the improvement of the confluence of Pulau Gambar Canal to the Ular as well as the improvement of the canal itself must be planned at this time.

(2) Alternative plans.

Three alternative plans were considered for improvement of the confluence of Pular Gambar Canal and the Ular river.

a. Alternative 1. Open levee system.

When a flood flow of the Ular river is increased, a part of the water is flooded into the adjacent land through the open levee and stored during the high water stage. In this case, velocity of inflow is extremely small and the land will not severely be damaged on the surface. However, the area will become unproductive year by year due to sedimentation by flooding.



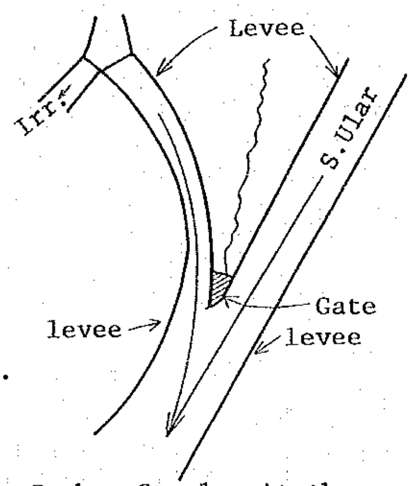
For the lower reaches of the Ular river downstream from the confluence, it is expected that safety will be increased due to some decrease in peak discharge by retarding effect.

The carrying capacity of the existing low-water channel of the Ular river is estimated at about 180 m^3/s; hence the lowland adjoining to open levee will be submerged by flood exceeding 180 m^3/s which would occur two or ten times a year.

In this plan, improvement work will have only to be carried out on the upper reaches of Pulau Gambar Canal upstream from the inundated lower area. Consequently, discharge of Pulau Gambar Canal (S. Kotabangun) will be poured into the submerged area.

b. Alternative Plan 2. Enclosing levee system.

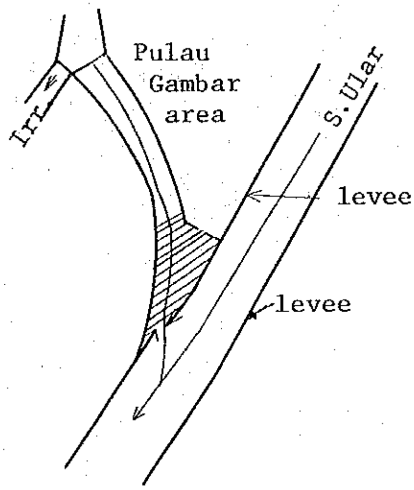
In this plan, the right-side levee of the Ular river and the left-side levee of Pulau Gambar Canal (S. Kotabangun) shall be connected and the height of levee crown of Pulau Gambar Canal shall be built the same as that of the Ular over a distance of back-water by the Ular. Hence the flood water of the Ular over a distance of back-water by the Ular. Hence the flood water of the Ular will not flow into the inner land which is located between the Ular and Pulau Gambar Canal. Discharge from the upper reaches of Pulau Gambar Canal is drained directly to the Ular river; however, discharge from the remaining catchment area, named Pulau Gambar area, must be drained by sluice or pump.



c. Alternative Plan 3. Gate system.

In this plan, the right side levee of the Ular shall be continuously constructed crossing Pulau Gambar Canal. At the crossing site, a gate shall be provided for drainage of water of Pulau Gambar Canal and excess water from Pulau Gambar area as well as for stopping the flood water of the Ular.

When water level of the Ular river is rising, the gate will be closed. During a period that water level of the Ular is higher than that of inner land, the gate will be closed. Accordingly, drained water from Pulau Gambar Canal and Pulau Gambar area ought to be stored in the inner neighboring area or in Pulau Gambar area during the said period. If the left-side levee of Pulau Gambar Canal is connected with the levee of the Ular, another sluice must be provided for drainage of Pulau Gambar area.



(3) Facilities of alternative plans.

Studies were made on facilities of the three alternative plans over a range from the confluence to the end of backwater by the Ular. The facilities adopted in the three plans are as follows.

(i) Alternative Plan 1. Open levee system.

a. Length of the opening is about 160 m.

b. Protection work is needed at the end of the right-side levee of the Ular river.

c. Land to be used for retardation must be acquired because the area will become unproductive due to yearly sedimentation.

d. Maintenance must be made of the area to be used for retardation.

(ii) Alternative Plan 2. Enclosing levee system.

a. On Pulau Gambar Canal, a levee is needed extending over the distance of back-water to be affected by the Ular river.

b. A levee is needed to enclose the inner land.

c. A sluice must be provided for drainage of Pulau Gambar area.

(iii) Alternative Plan 3. Gate system.

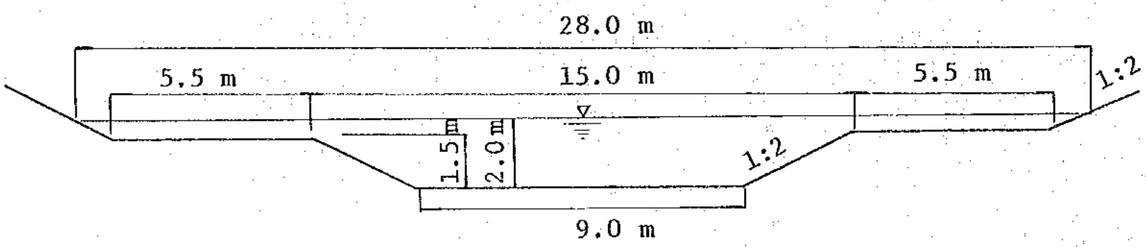
a. A gate must be provided on the right-side levee of the Ular river.

b. A levee is needed to enclose the inner land.

Location of the alternative plans are shown in Fig.1-4-20.

Longitudinal profiles of the Alternative Plans 2 and 3 were planned considering the existing canal bed and the design river bed of the Ular. These are shown in Fig.1-4-21. In the Alternative Plan 1, since the lower reaches of Pulau Gambar Canal is included in the retardation area, the improvement of the canal shall be made on the upper reaches upstream from the distance-mark 10.75 km, and the longitudinal profile of the reaches to be improved is the same as those of the Alternative Plans 2 and 3.

Cross section of Pulau Gambar Canal (S. Kotabangun) was designed to meet the design discharge 20 m^3/s by use of the Manning formula with $n = 0.03$ and $I = 1/1,000$. Standard cross section in design is shown below, and some cross section are shown in Fig.1-4-22.



According to the study of drainage improvement, peak discharge from 10.6 km^2 of the catchment area of Pulau Gambar was estimated at 6 m^3/s. This water was planned to be drained from a sluice, the elements of which are shown below.

Elements of Sluice Gate of Alternative Plan 2

Elevation of bed	: 27.01 m UP.
Gates	: H = 1.2 m, W = 1.4 m, 2 gates.
Structure	: Reinforced concrete.
Gate type	: Type will be studied in the detail design.

In Alternative Plan 3, the gate was designed as below so as to meet the discharge 70 m^3/s,

Elements of Gate of Alternative Plan 3

Elevation of bed	: 27.01 m UP.
Gates	: W = 12.8 m, 2 gates.
Structure	: Reinforced concrete.

(4) Water level in the inner land.

After the completion of improvement of the main channel of the Ular, discharges exceeding 180 m^3/s of the Ular will enter into the inner land if no enclosing levee is constructed. The following table shows the recent peak-discharges at Bandar Tiga exceeding 180 m^3/s. These values were estimated by use of the discharge rating curve and the records of water level at Bandar Tiga.

Peak-discharge at Bandar-Tiga (Q $\geqq$ 180 m /sec)

Year	Date	Peak Discharge m^3/sec	Year	Date	Peak Discharge m^3/sec	Year	Date	Peak Discharge m^3/sec
1973	Apr.15	230	1974	Jan.19	190	1976	Feb. 5	393
	Apr.20	205		Sep. 7	325		June10	205
	June 5	425		Sep.29	376		Nov. 7	335
	July23	203		Oct. 3	365		Nov.21	406
	Oct.24						Dec.20	266
	Nov.10		1975	Apr.15	260		Dec.30	190
	Nov.21			Dec.30	185			
	Dec. 9							
	Dec.16							
	Dec.25							

(a) Alternative Plan 1 : Open levee system.

The above table shows that flood water would enter into the inner land 2 to 10 times a year in this plan. When the discharge of of the Ular is 800 m^3/s, submerged area will reach to 0.7 km^2 and the water level will be 29.7 m UP.

(b) Alternative Plan 2 : Enclosing levee system.

Hydraulic elements were estimated as below.

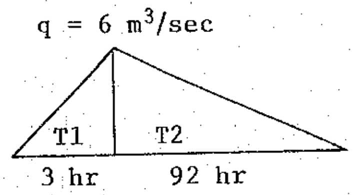
Peak water level in inner land	: 29.10 m UP.
Volume of water to be stored	: 230,000 m^3.
Area to be submerged	: 0.49 km^2.

These elements were estimated by the following procedures.

a. Storage capacity curve shown in Fig.1-4-23 was used.

b. Water level at the confluence with the Ular river, as shown in Fig.1-4-24, was estimated by use of the hydrograph shown in Fig.1-4-25 /1 and the design cross section at the confluence.

c. The hydrograph with the peak discharge of 6 m^3/s /2 was assumed as below.



(c) Alternative Plan 3 : Gate system.

Hydraulic elements were estimated as below.

Peak water level in inner land	: 29.60 m UP.
Volume of water to be stored	: 500,000 m^3.
Area to be submerged	: 0.67 km^2.

These elements were estimated by the following procedures.

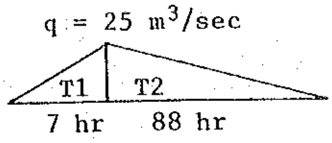
a. Storage capacity curve shown in Fig.1-4-23 was used.

b. Water level at the confluence with the Ular river was estimated as shown in Fig.1-4-24.

/1 Overall Study Report, Fig.3-4-16.

/2 See Design discharge for drainage of this report.

c. The discharge hydrograph for the catchment area of Pulau Gambar Canal (S. Kotabangun) was assumed as below.



(5) Costs of alternative plans.

The estimated economic costs of the three alternative plans are as follows. In this estimation, loss or minus benefit which may accrue from implementation of the plan was aocounted as a cost for the convenience of comparison of the three plans.

(a) Alternative Plan 1 : Open levee system.

Protection work of levee	200 m	20,000,000 Rp
Excavation of canal	1,300 m^3, @ 250 Rp/m^3	325,000
Land acquisition	70 ha, @ 30 Rp/m^2	21,000,000
Miscellaneous work	20 %	8,675,000
Subtotal		50,000,000
Maintenance cost (economic cost for 50 years)		36,000,000

on condition of
3% of construction cost,
@ 3 Rp/m^2 for retardation area, and
discount rate of 10 % (9.915 for 50 years)

Loss that the present productive area will become unproductive due to inundation by flood water.	93,000,000

on condition that
submerged area is 70 ha,
normal unit yield is 3,300 kg/ha,
unit price of paddy is 65 Rp/kg,
unit production cost is 81,000 Rp/ha and
discount rate is 10 % (9.915 for 50 years).

Total 50,000,000 + 36,000,000 + 93,000,000 = 179,000,000 Rp

(b) Alternative Plan 2 : Enclosing levee system.

Embankment	38,500 m^3, @ 900 Rp/m^3	34,650,000 Rp
Left 30,800 m^3		
Right 7,700		
Excavation	1,300 m^3, @ 250 Rp/m^3	325,000
Enclosing levee, 160 m	3,000 m^3, @ 900 Rp/m^3	2,700,000

Drainage sluice	1.2 m × 1.4 m, 2 gates	36,300,000 Rp
Miscellaneous work	20 %	14,825,000
Subtotal		88,800,000
Maintenance cost (economic cost for 50 years)		26,413,000
Decrease in production in the submerged area		2,000,000

on condition that
submerged area is 49 ha,
rate of damage to paddy is 30 % of normal yield,
frequency of submergence is 1/10 and
discount rate is 10 % (9.915 for 50 years)

Total   88,800,000 + 26,413,000 + 2,000,000 = 117,213,000 Rp

(c) Alternative Plan 3 : Gate system.

Embankment	18,000 m^3, @ 900 Rp/m^3	16,200,000 Rp
Left 18,000 m^3		
Right 0		
Excavation	1,300 m^3, @ 250 Rp/m^3	325,000
Enclosing levee, 160 m	3,000 m^3, @ 900 Rp/m^3	2,700,000
Gate	12.4 m × 3.49 m, 2 gates	500,000,000
Miscellaneous	20 %	99,775,000
Subtotal		619,000,000
Maintenance cost (619,000,000 × 0.05 × 9.915)		305,000,000
Decrease in production in the submerged area		2,700,000

on condition that
submerged area is 67 ha,
rate of damage to paddy is 30 % of normal yield,
frequency of submergence is 1/10 and
discount rate is 10 % (9.915 for 50 years)

Total   619,000,000 + 305,000,000 + 2,700,000 = 926,700,000 Rp

(6) Comparison of three alternative plans.

The three alternative plans are summarized as follows.

Description	Alternative 1.	Alternative 2.	Alternative 3.
Economic cost.	179,000,000.Rp	117,213,000.Rp	926,700,000.Rp
Water level of submergence.	29.7 m UP	29.1 m UP	29.6 m UP
Volume to be stored.	550,000 m^3	230,000 m^3	500,000 m^3
Area to be submerged after works.	70 ha.	49 ha.	67 ha.