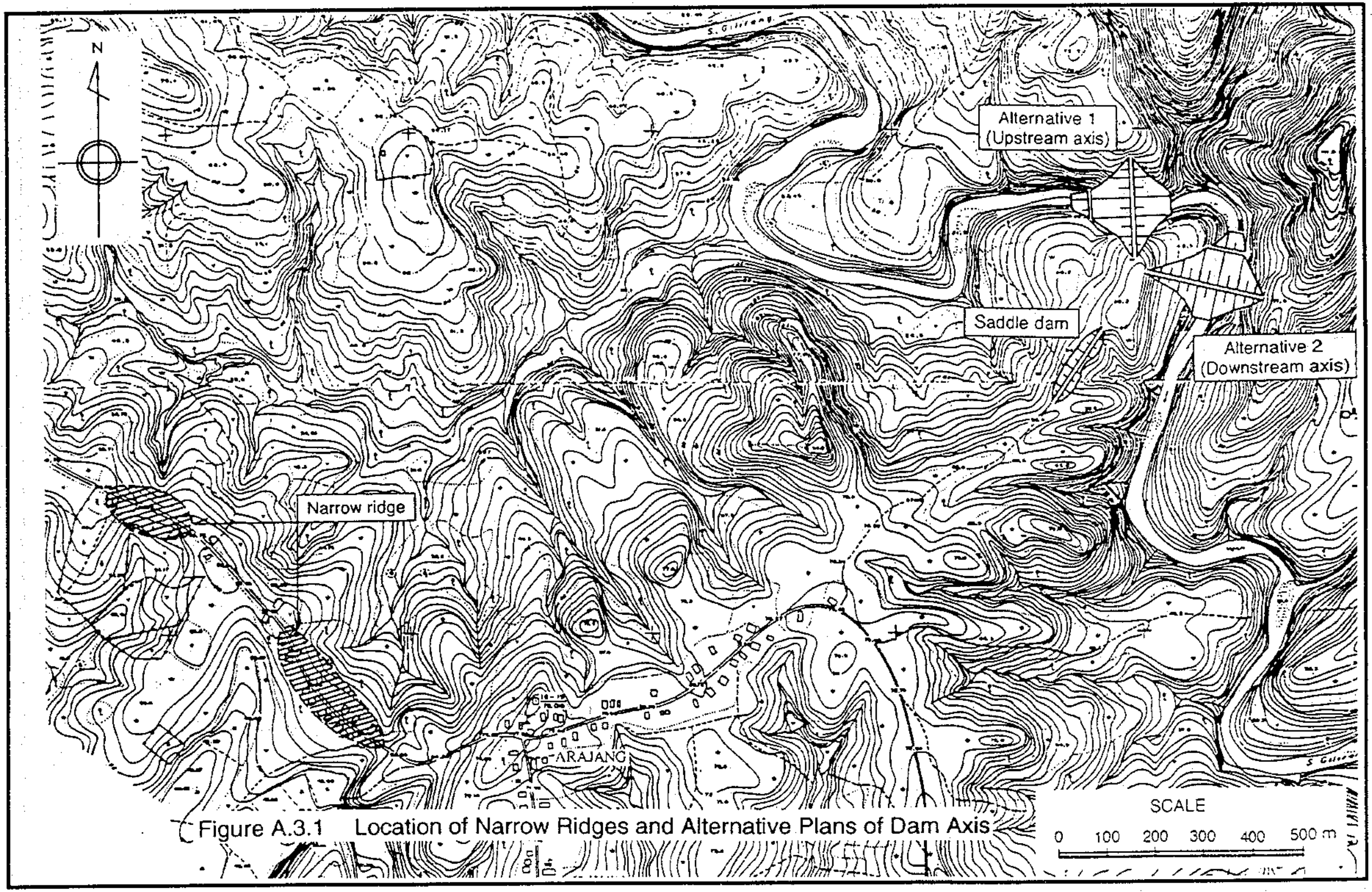


Figure A.3.1 Location of Narrow Ridges and Alternative Plans of Dam Axis

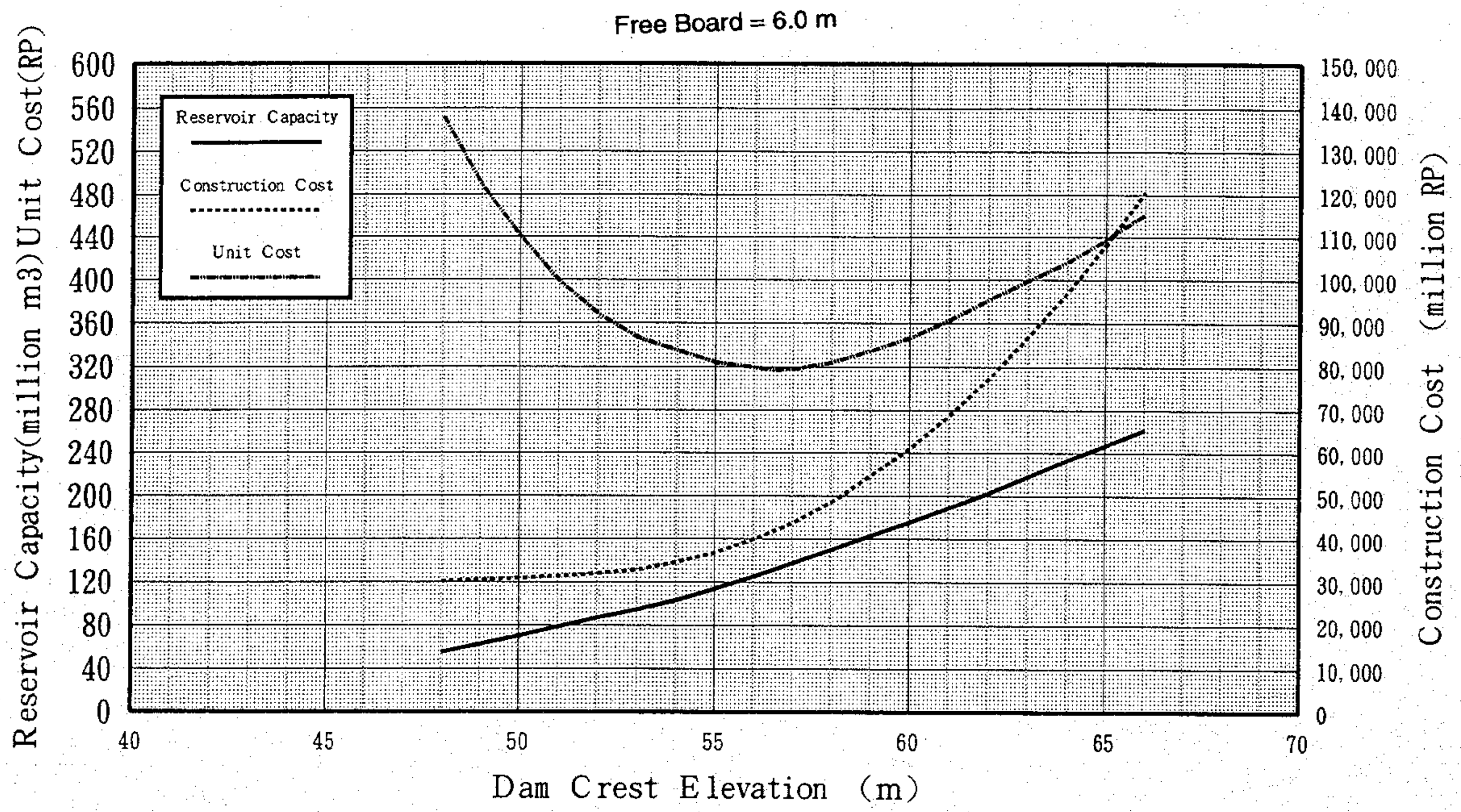


Figure A.3.2 Relation between Crest Elevation and Construction Cost

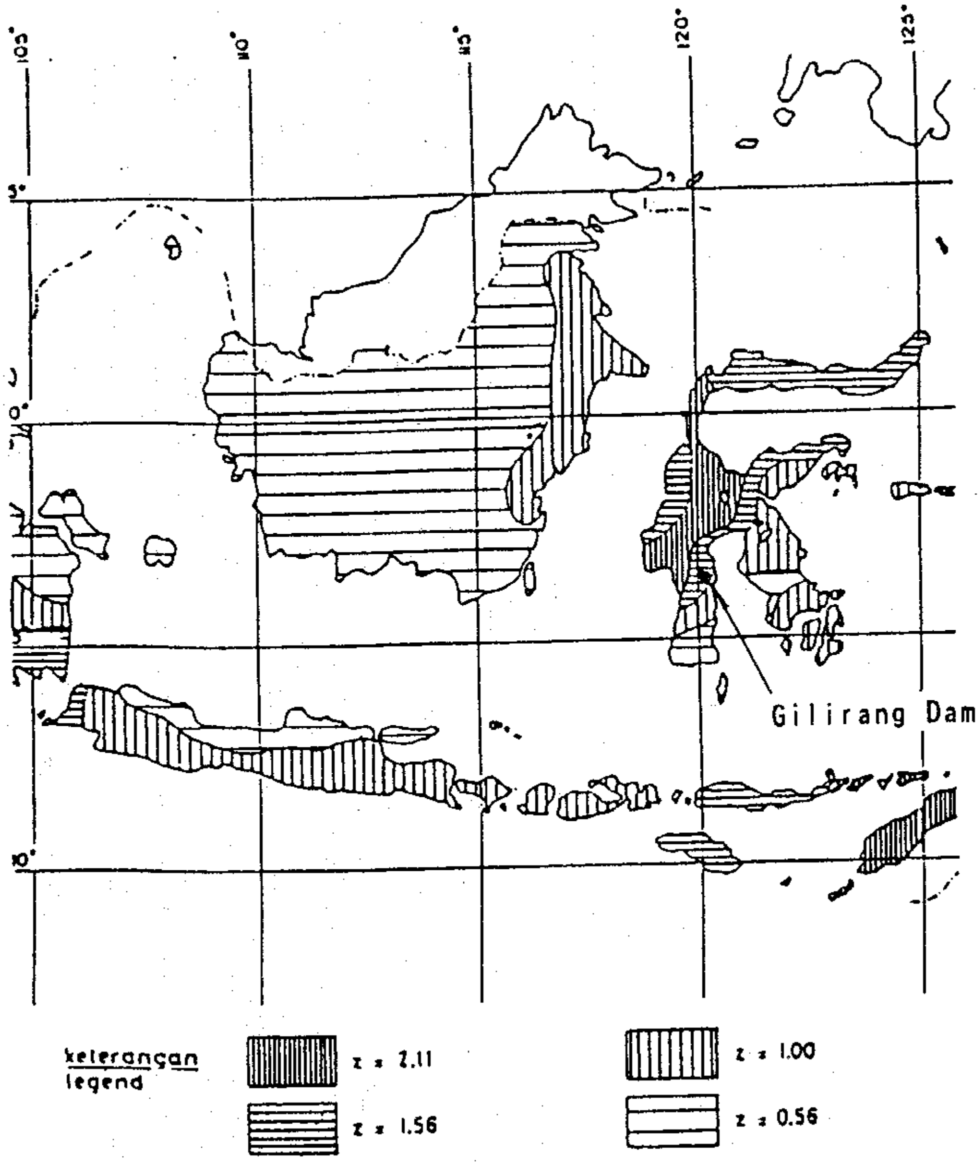


Figure A.3.3 Earthquake Zone in Indonesia (Central)

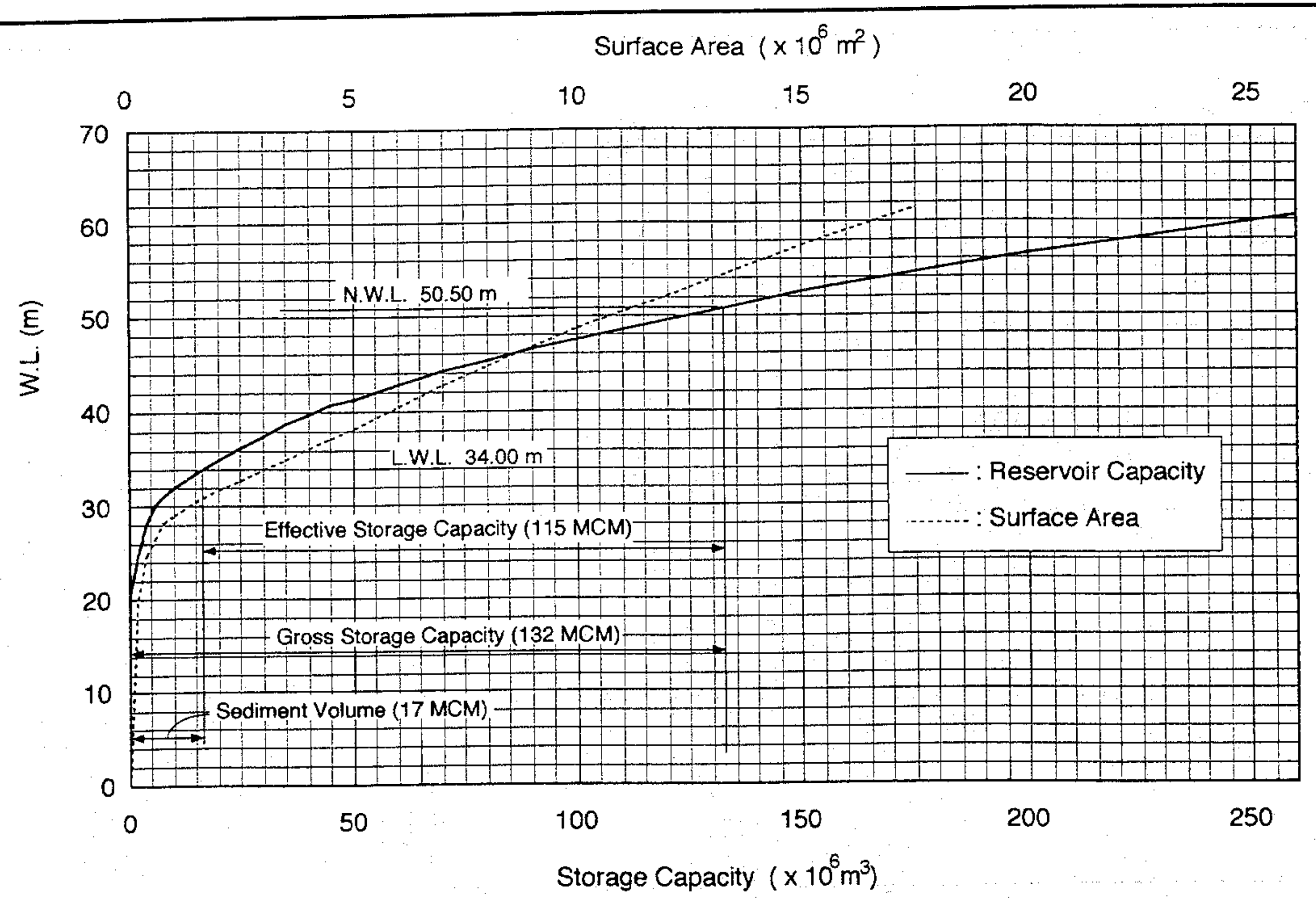


Figure A.3.4 Relations between Water Level and Capacity or Surface Area

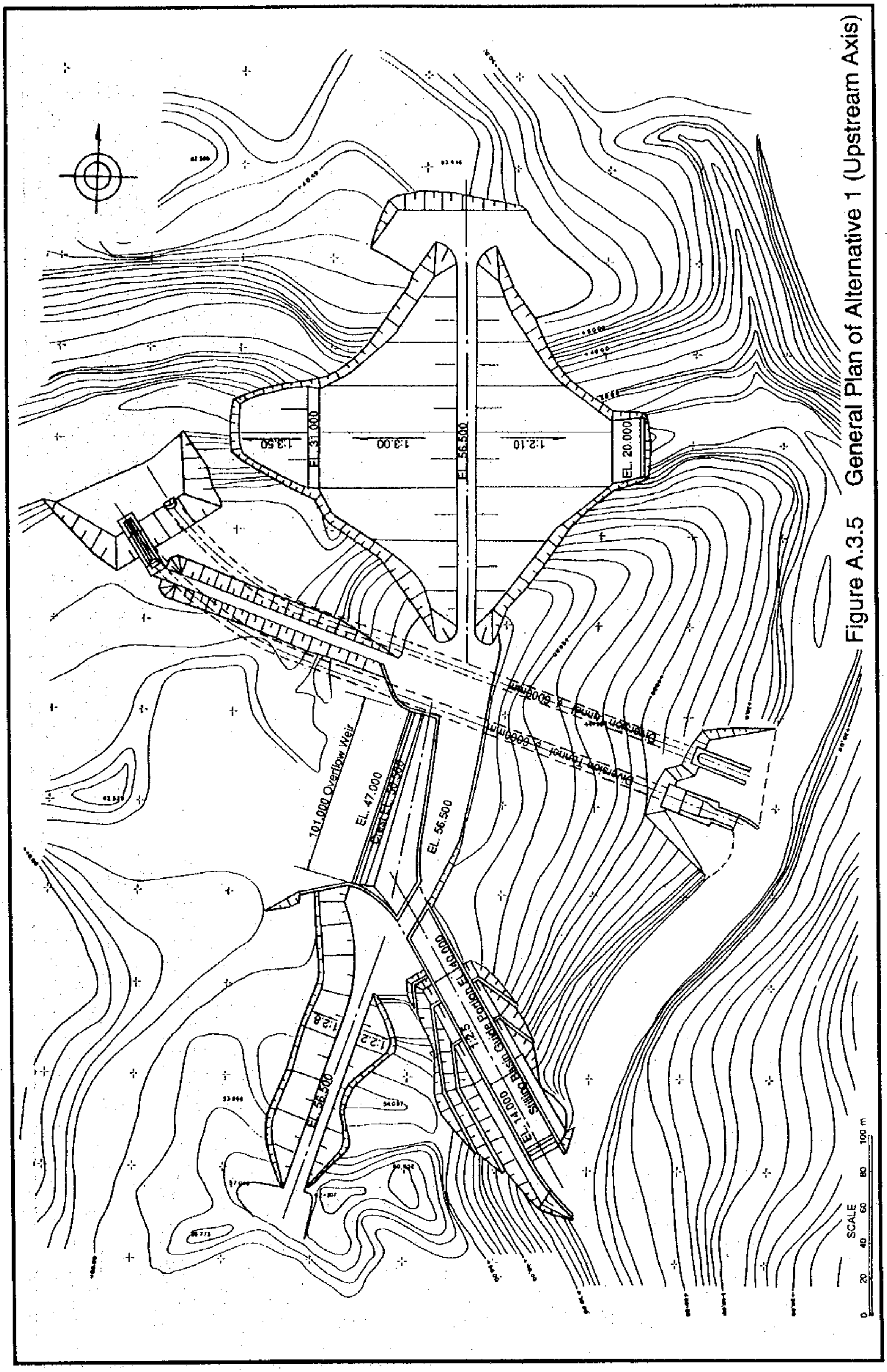


Figure A.3.5 General Plan of Alternative 1 (Upstream Axis)

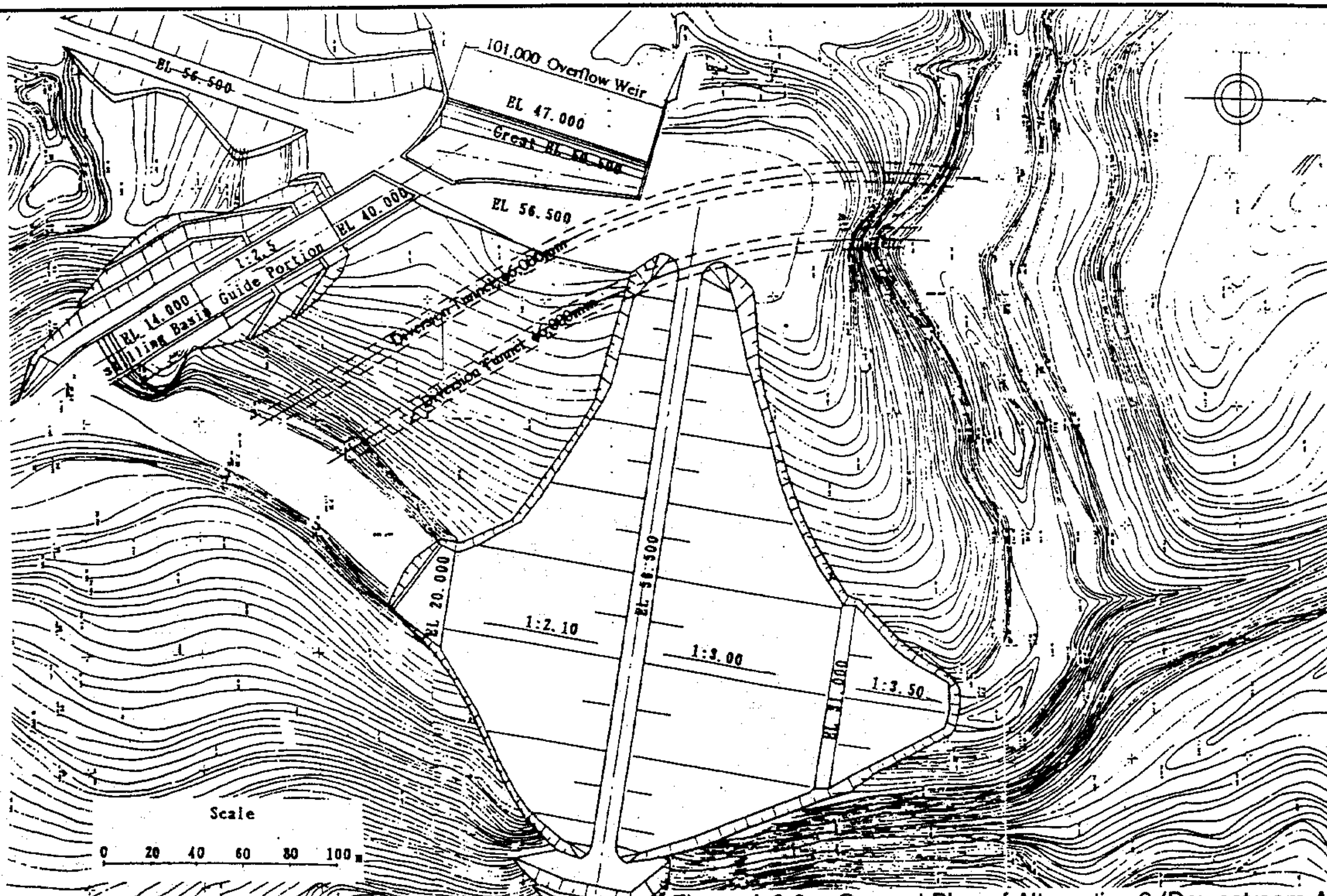


Figure A.3.6 General Plan of Alternative 2 (Downstream Axis)

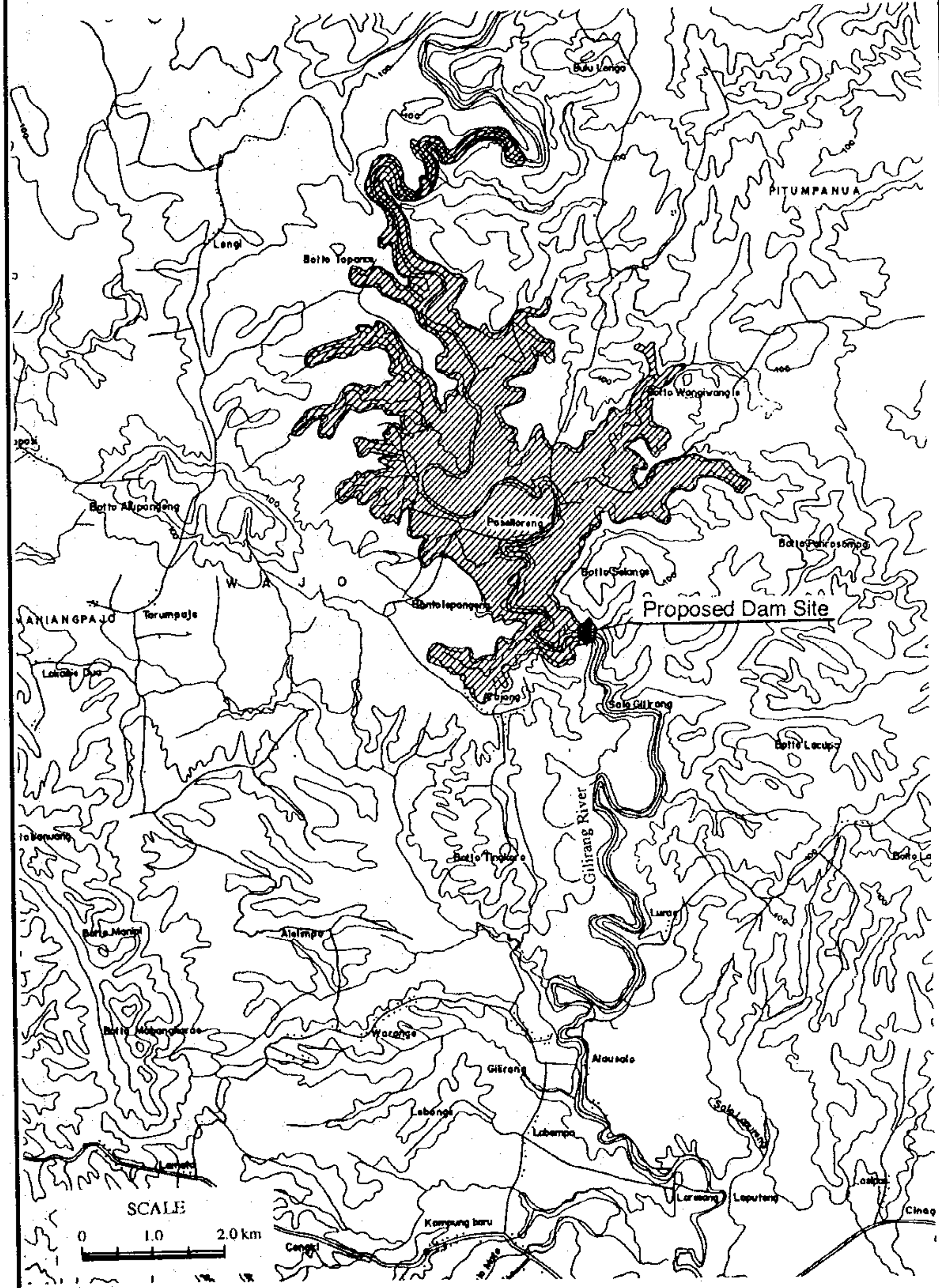
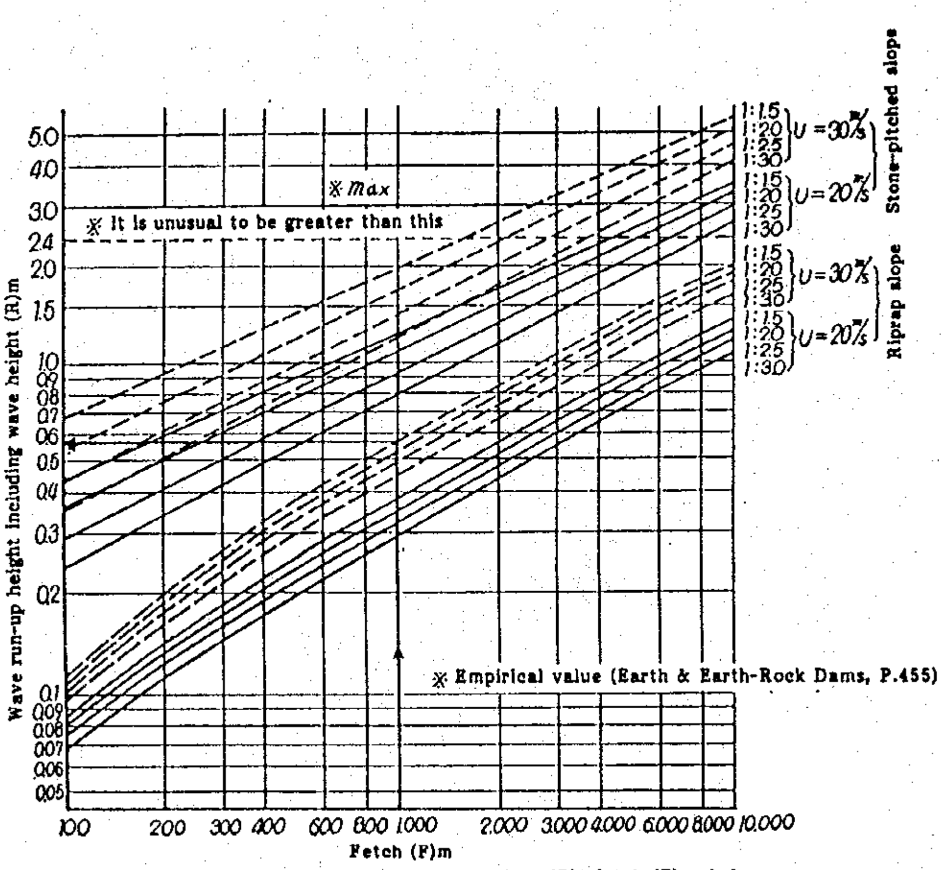
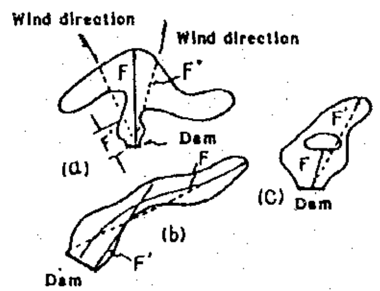


Figure A.3.7 Reservoir Area of Pasellorong Dam

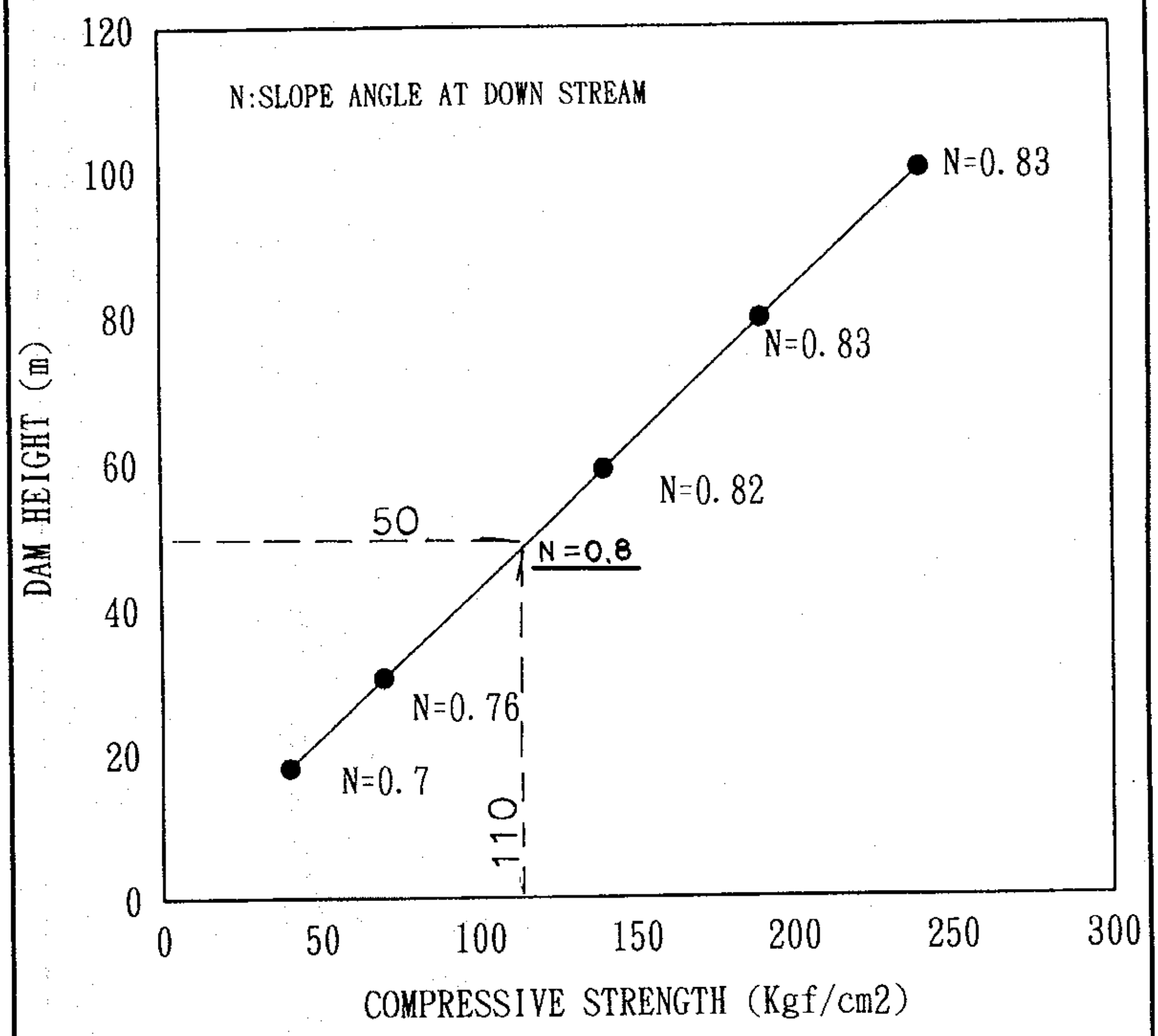


In order to determine wave run-up height (R), fetch (F), wind velocity (V), wave height (hw) and wave length (L) are required to be found by the S.M.B method, and slope and slope roughness by the Saville method



Source: Engineering Manual for Irrigation & Drainage of The Japanese Institute of Irrigation & Drainage

Figure A.3.8 Wave Run-up by the S.M.B Method and Saville Method



Source:Engineering Manual for Irrigation & Drainage of The Japanese Institute of Irrigation & Drainage

Figure A.3.9 Required Strength of Foundation and Dam Height

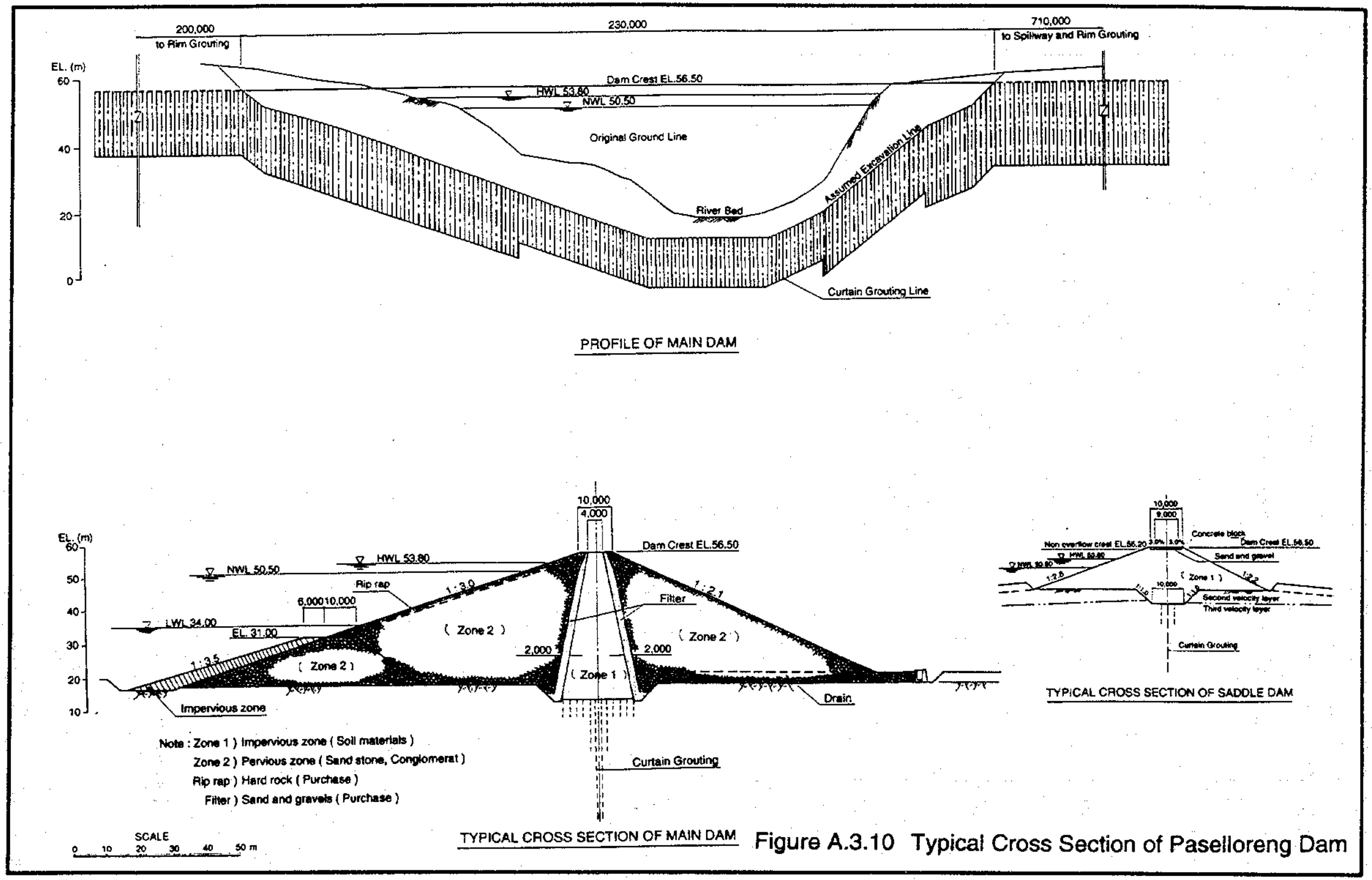


Figure A.3.10 Typical Cross Section of Paselloreng Dam

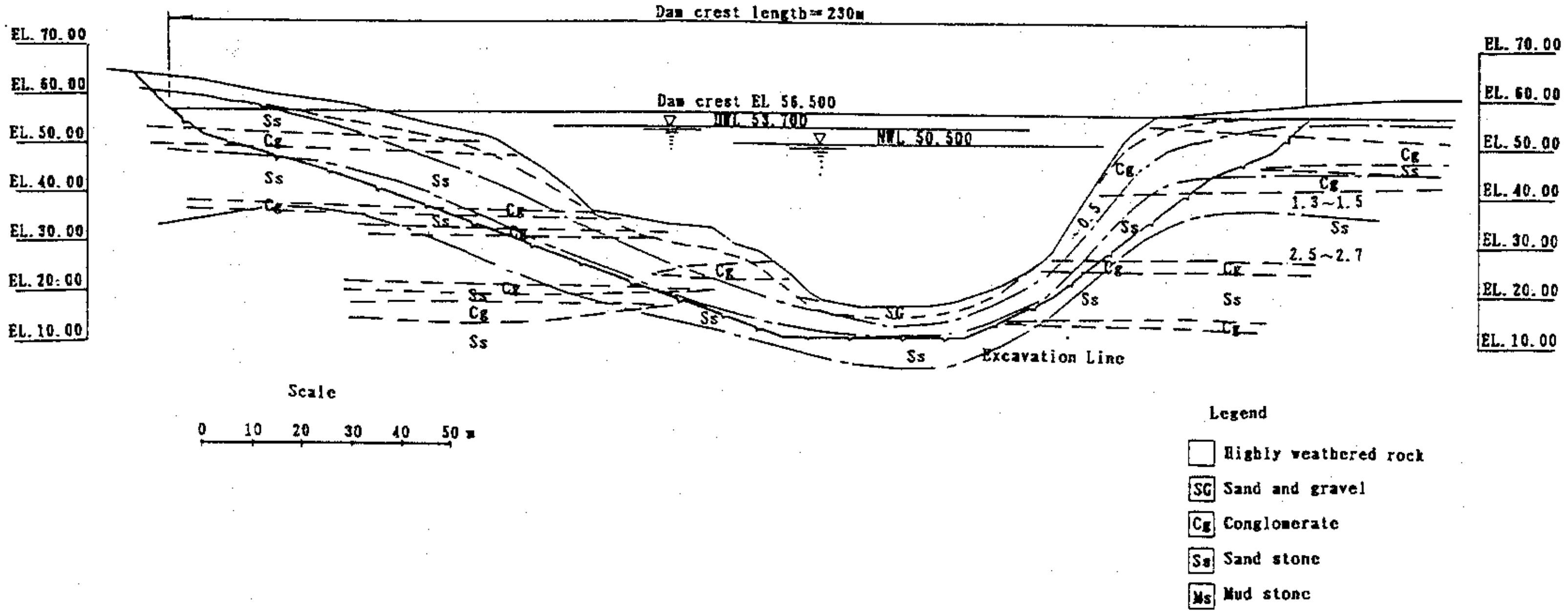
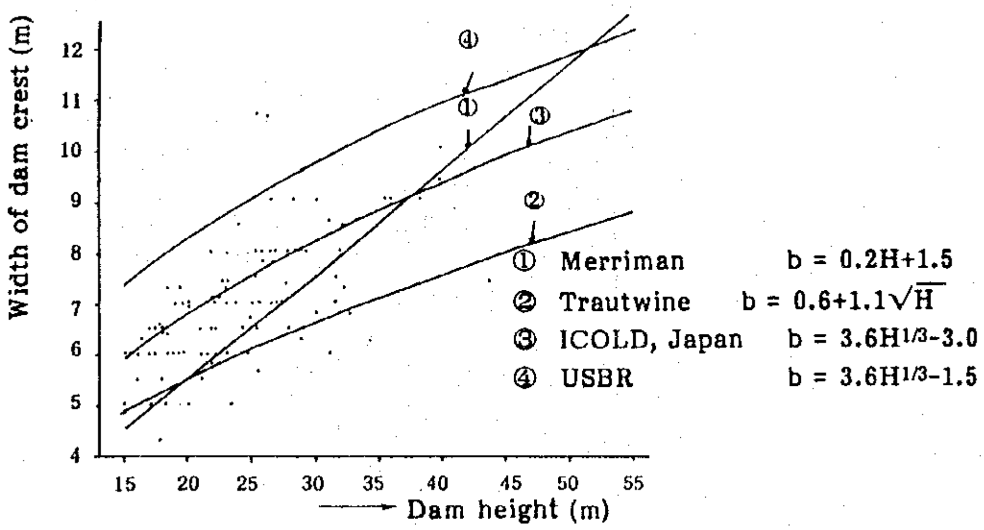


Figure A.3.11 Cross Section of Dam Axis

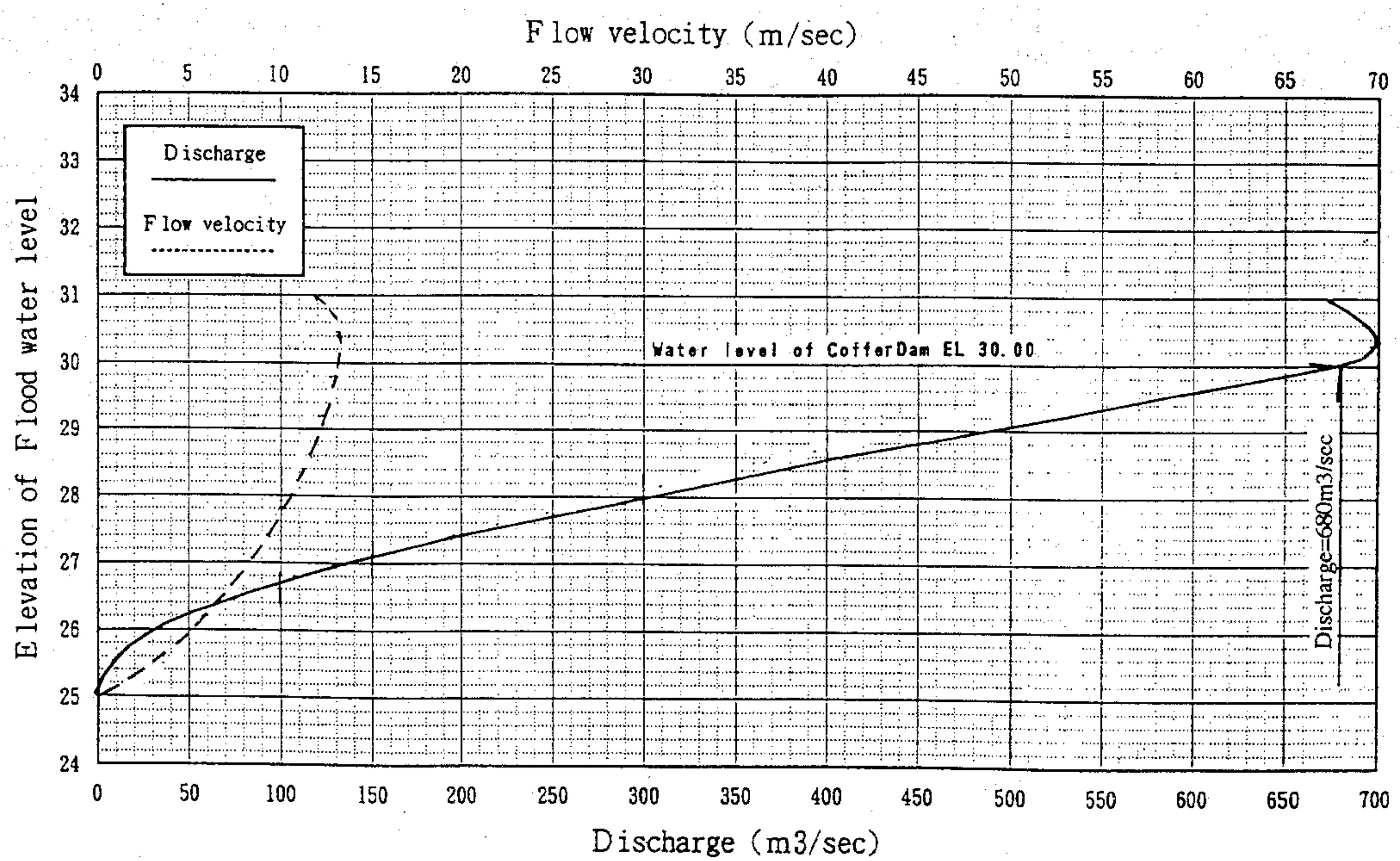


Crest width is often estimated by the following equations:

U.S.B.R.	$b = 3.6\ H^{1/3} - 1.5$
Merriman	$b = 0.2\ H + 1.5$
Trautwine	$b = 0.6 + 1.1\ \sqrt{H}$
Where,	b = Crest width (m)
	H = Dam height (m)

Source:Engineering Manual for Irrigation & Drainage of The Japanese Institute of Irrigation & Drainage

Figure A.3.12 Width of Dam Crest and Dam Height



φ6,000mmx2lines

Figure A.3.13 Discharge and Flow Velocity of Diversion Tunnel

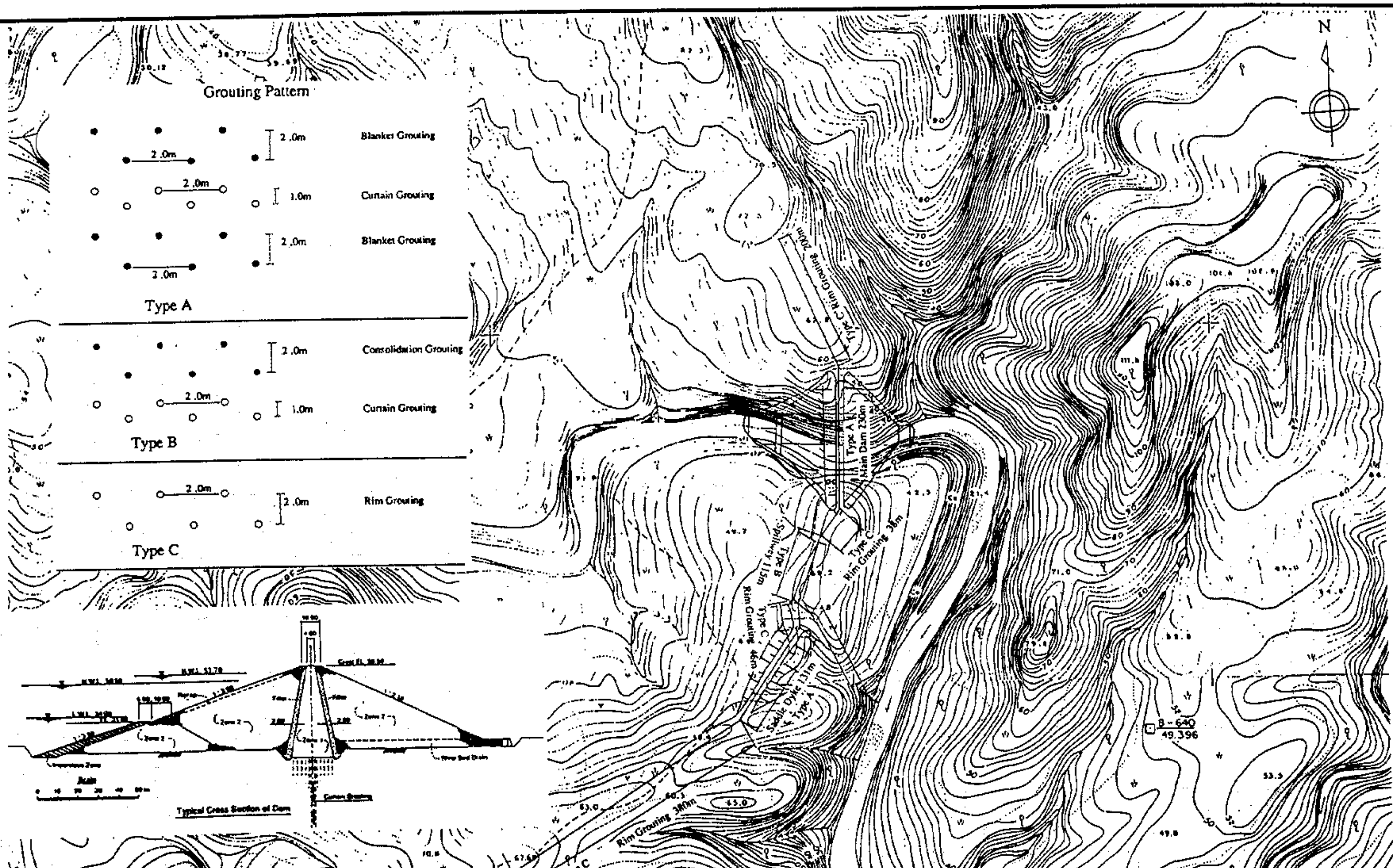
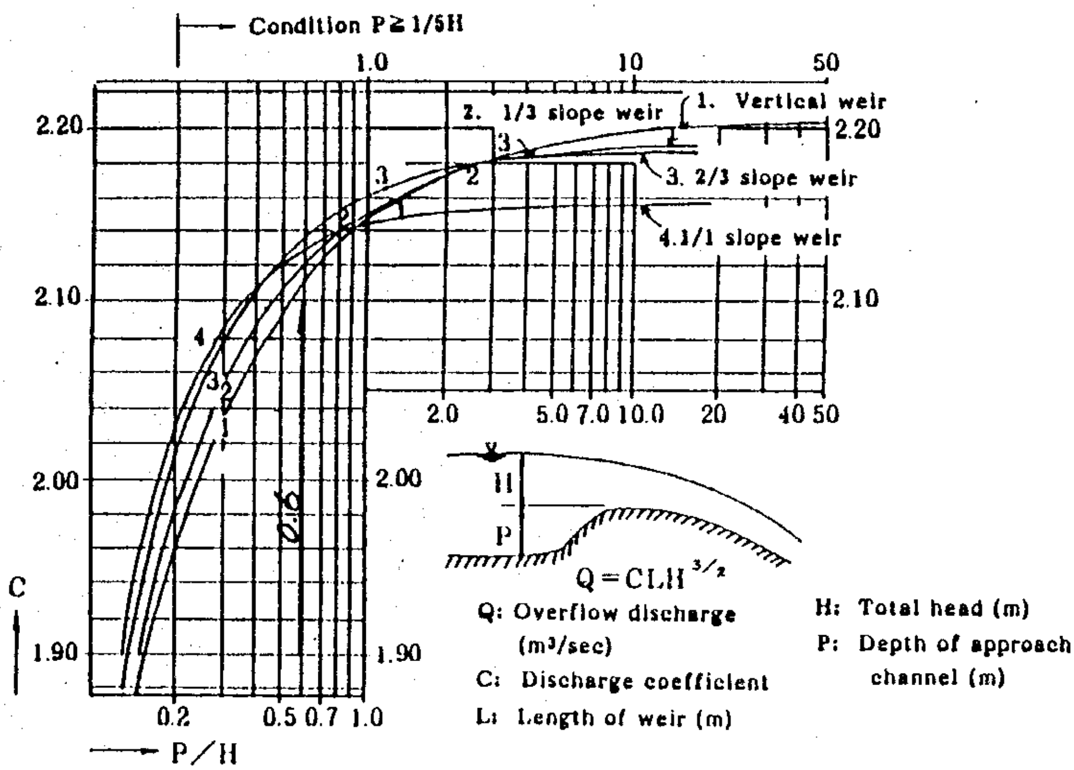


Figure A.3.14 Foundation Treatment Plan of Paselloreng Dam



Source: Engineering Manual for Irrigation & Drainage of The Japanese Institute of Irrigation & Drainage

Figure A.3.15 Discharge Coeficient of Weir

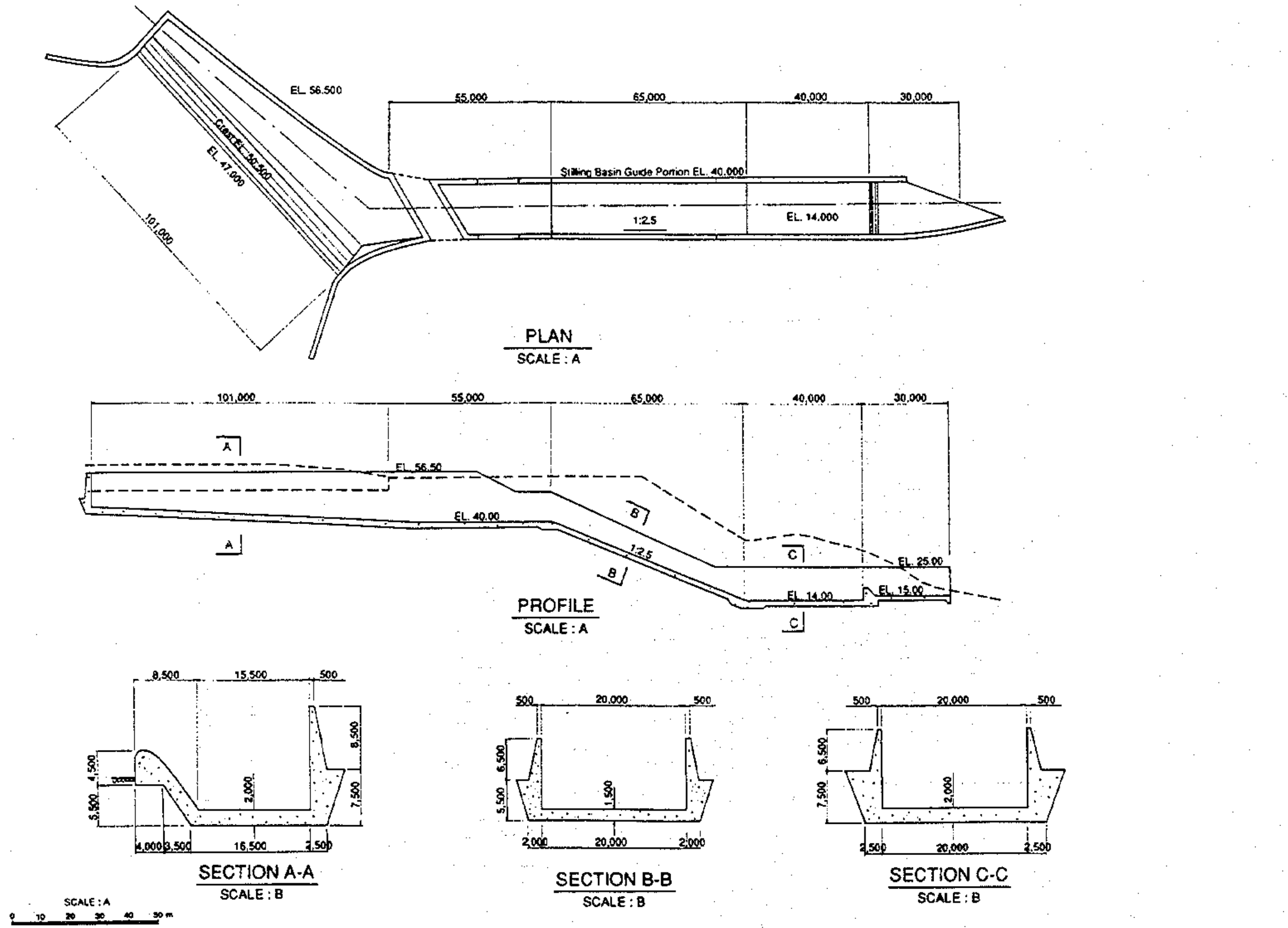


Figure A.3.16 Spilway of Paselloreng Dam

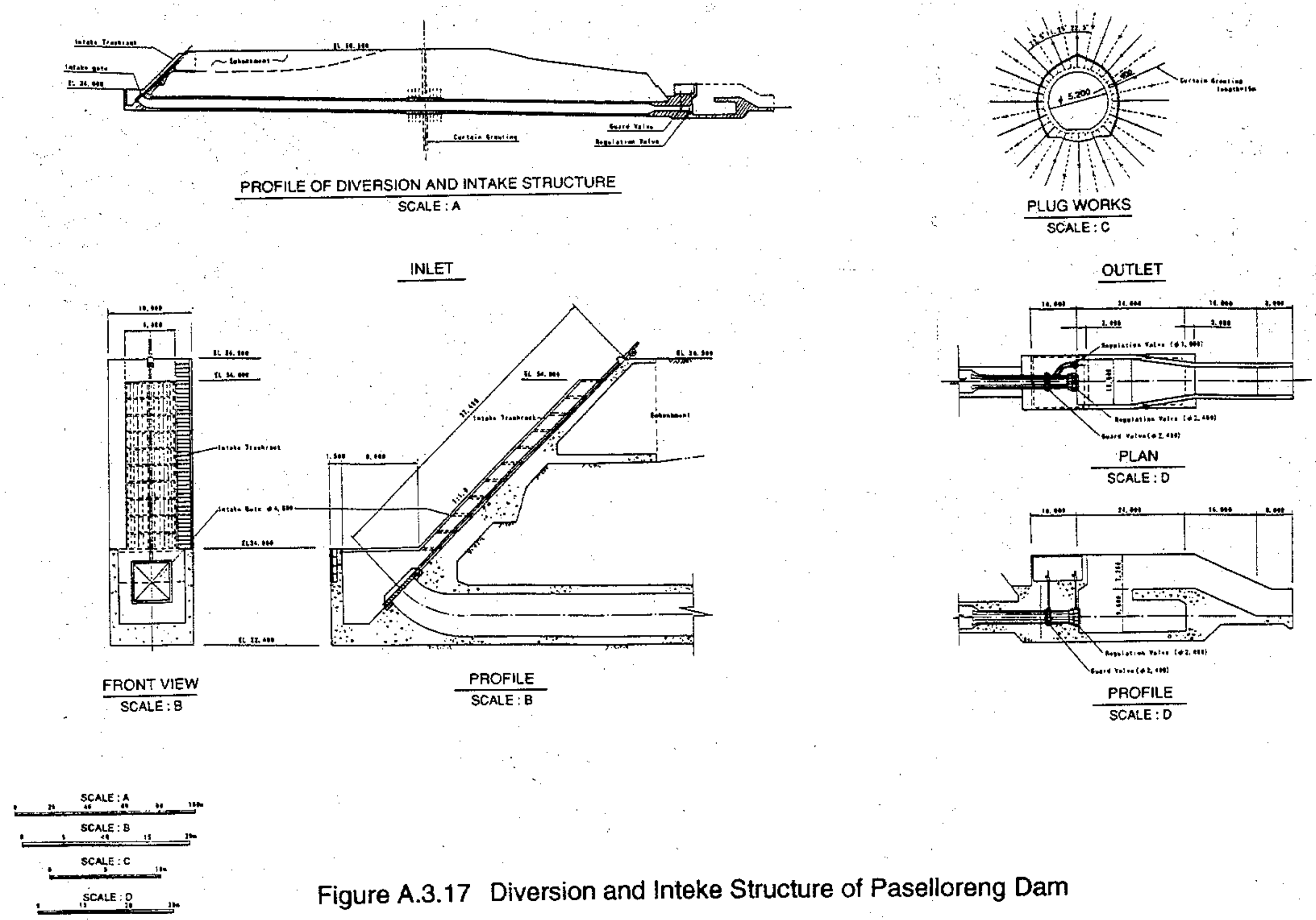


Figure A.3.17 Diversion and Inteke Structure of Paselloreng Dam

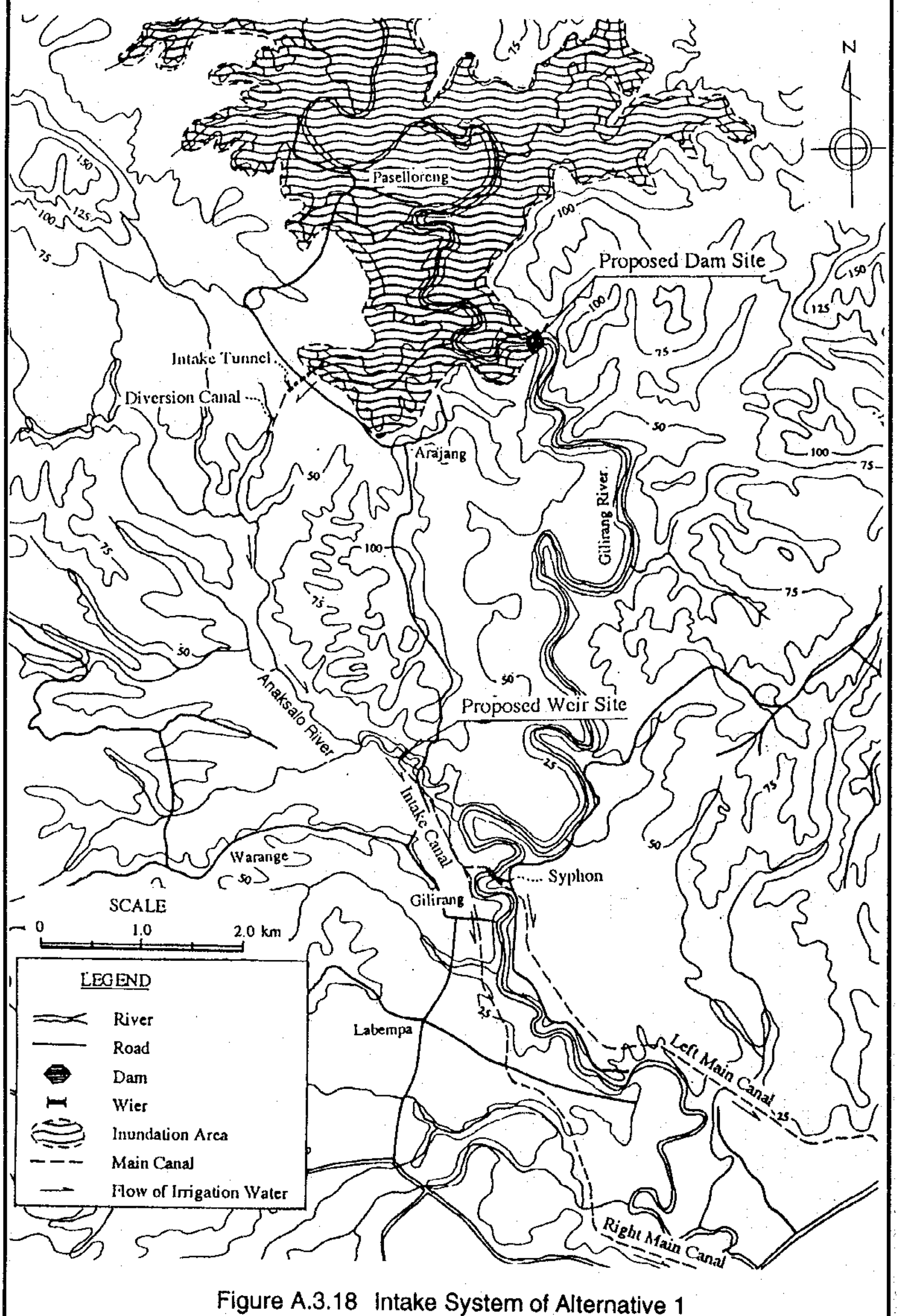


Figure A.3.18 Intake System of Alternative 1

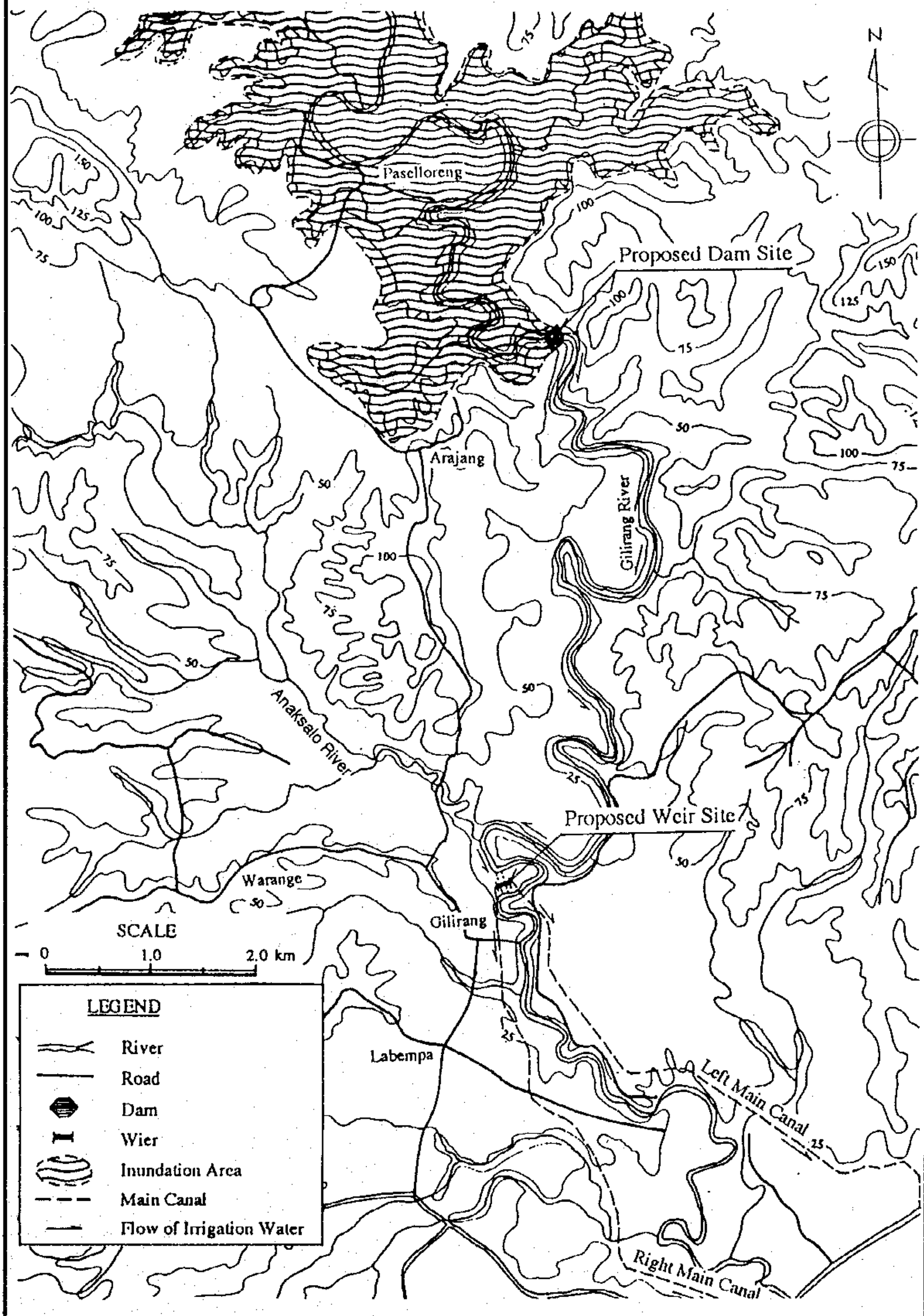


Figure A.3.19 Intake System of Alternative 2

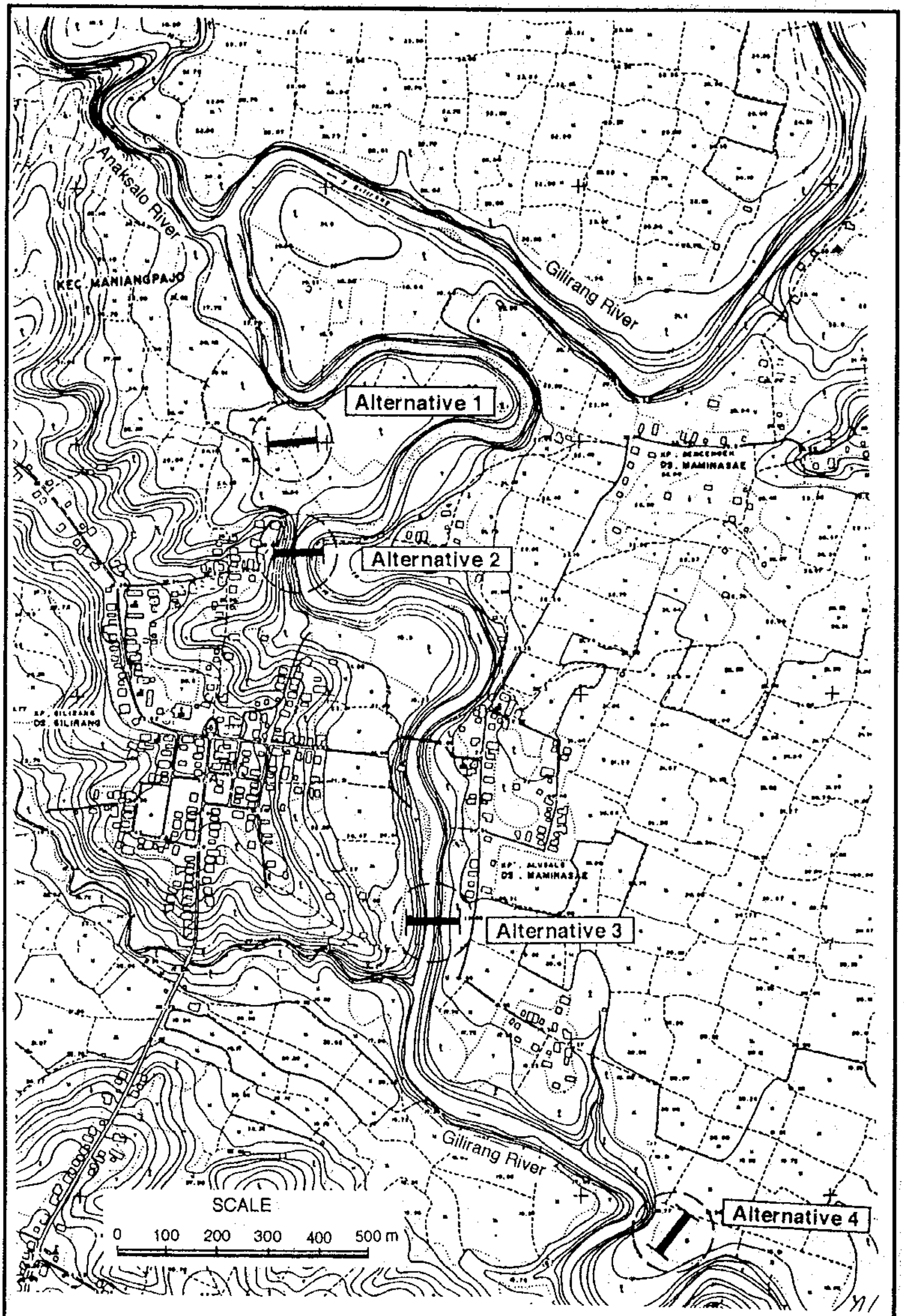


Figure A.3.20 Possible Site of Intake Weir

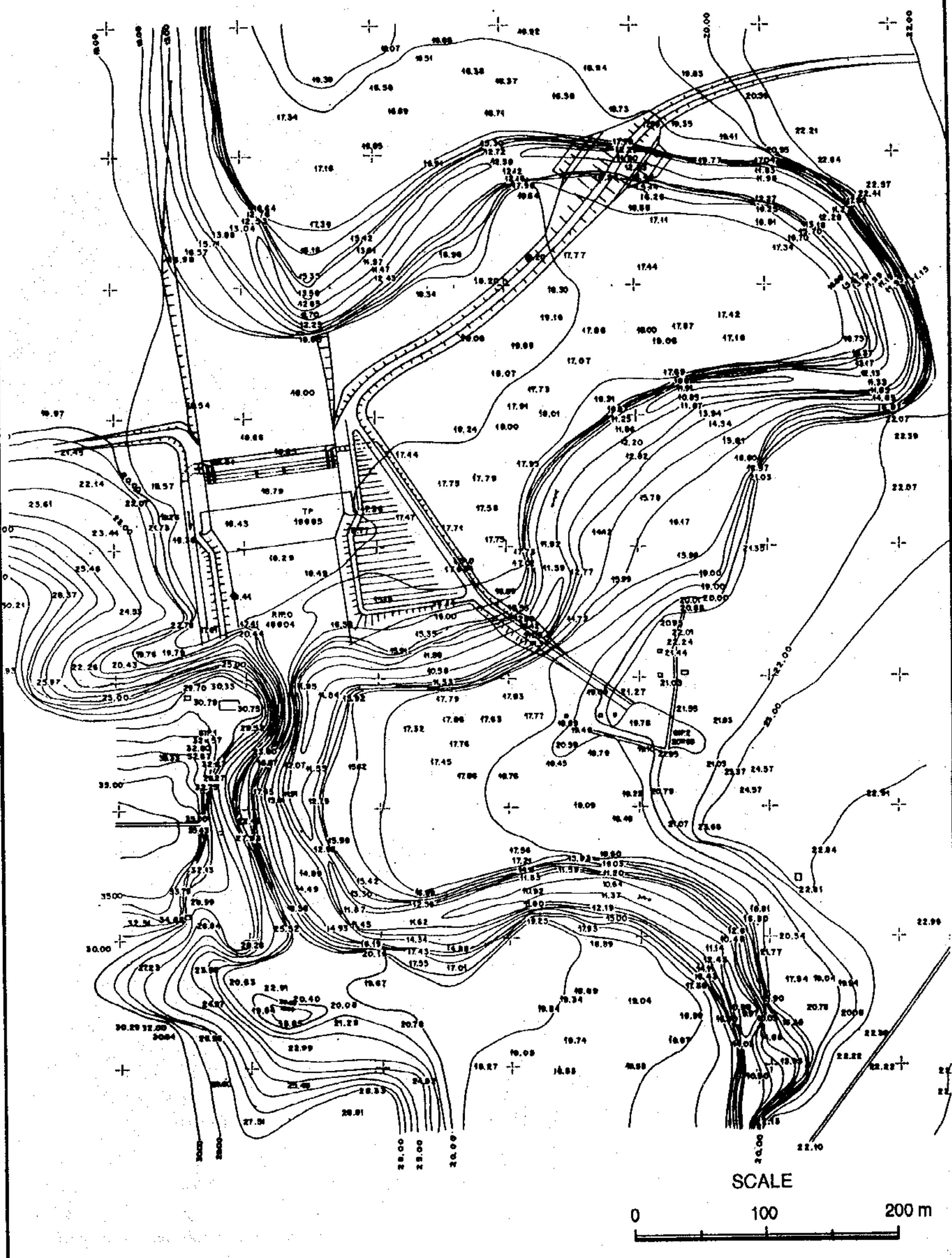


Figure A.3.21 General Plan of Alternative 1 Weir Site

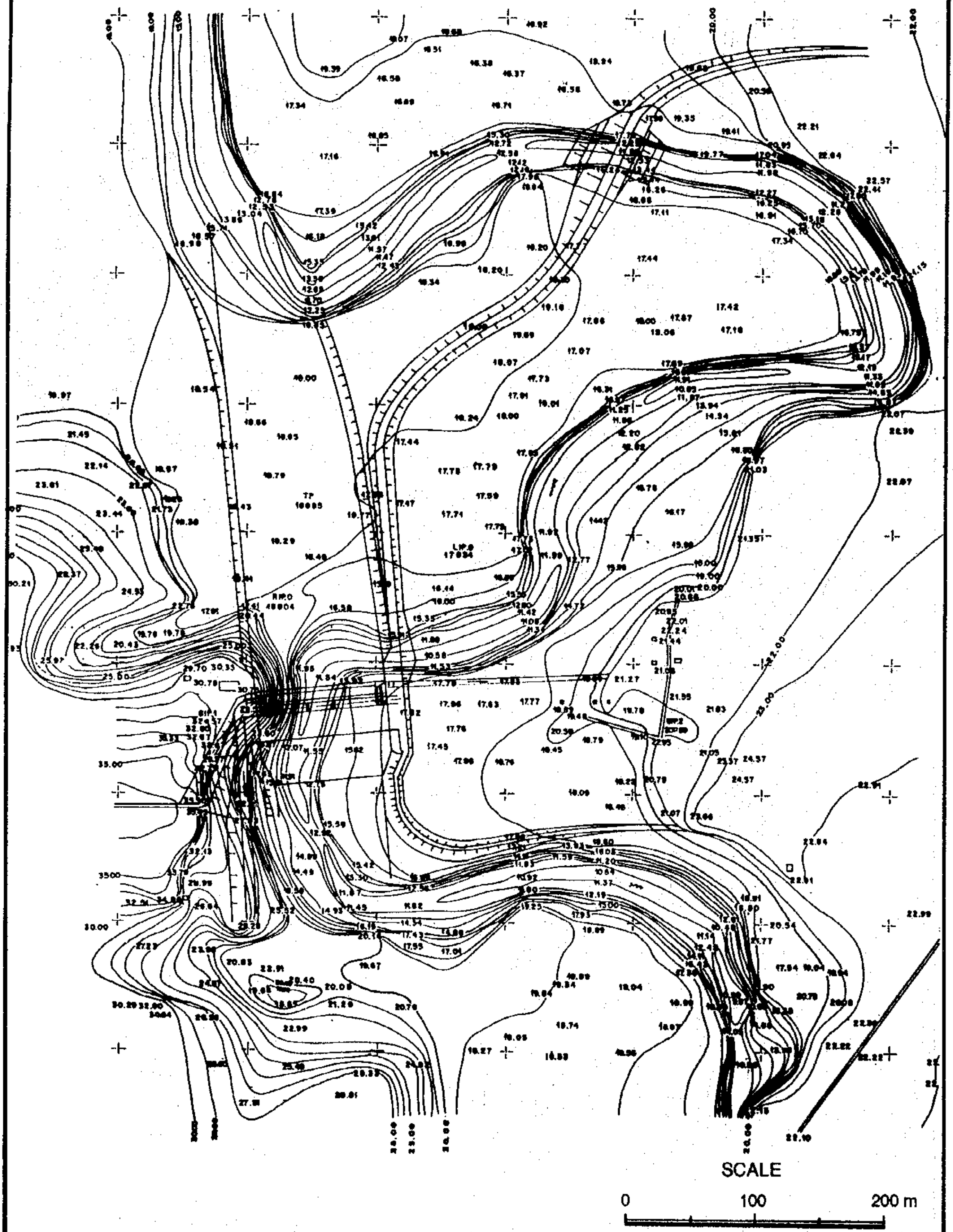


Figure A.3.22 General Plan of Alternative 2 Weir Site

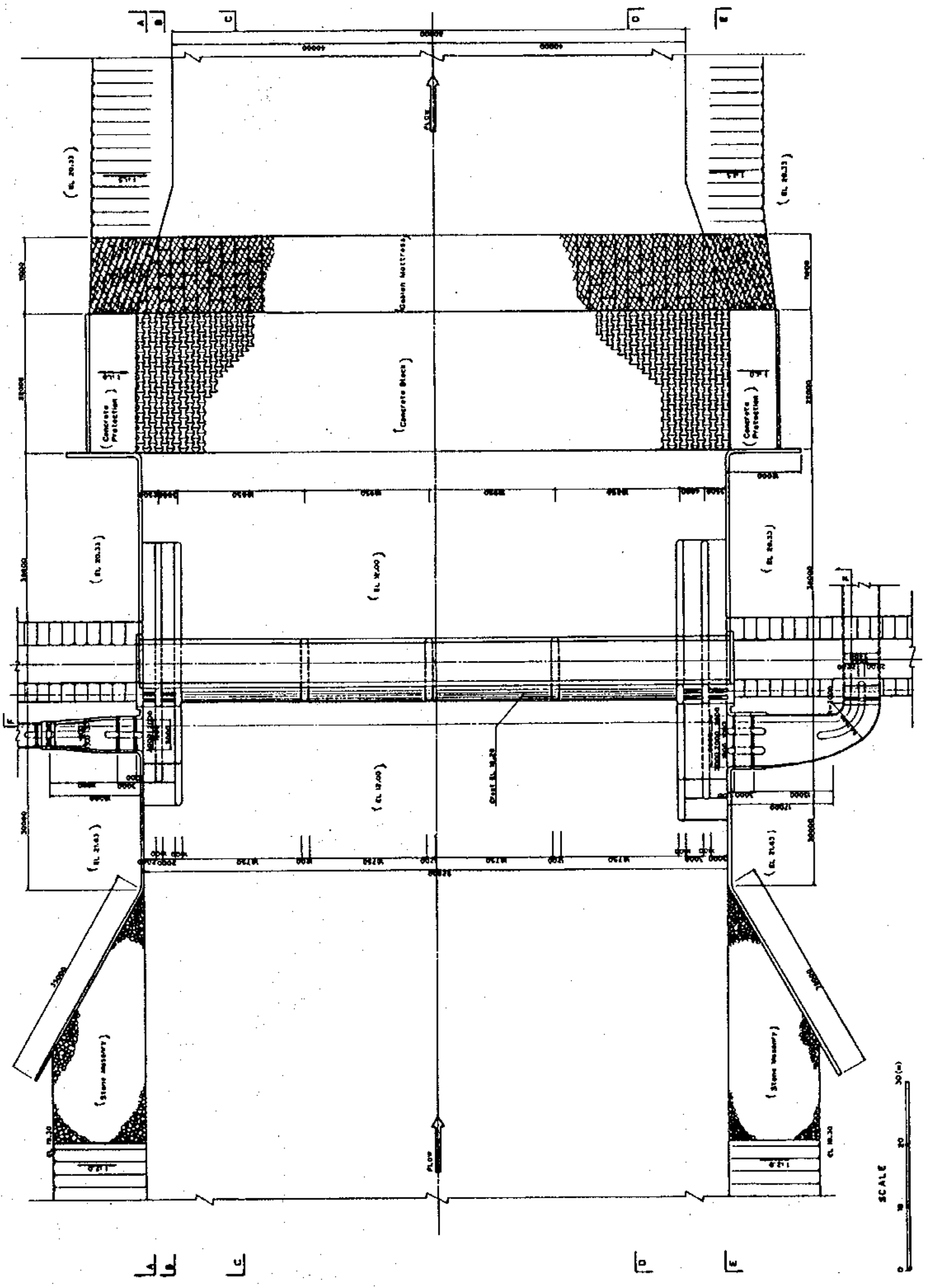


Figure A.3.23 Plan of Gilirang Weir

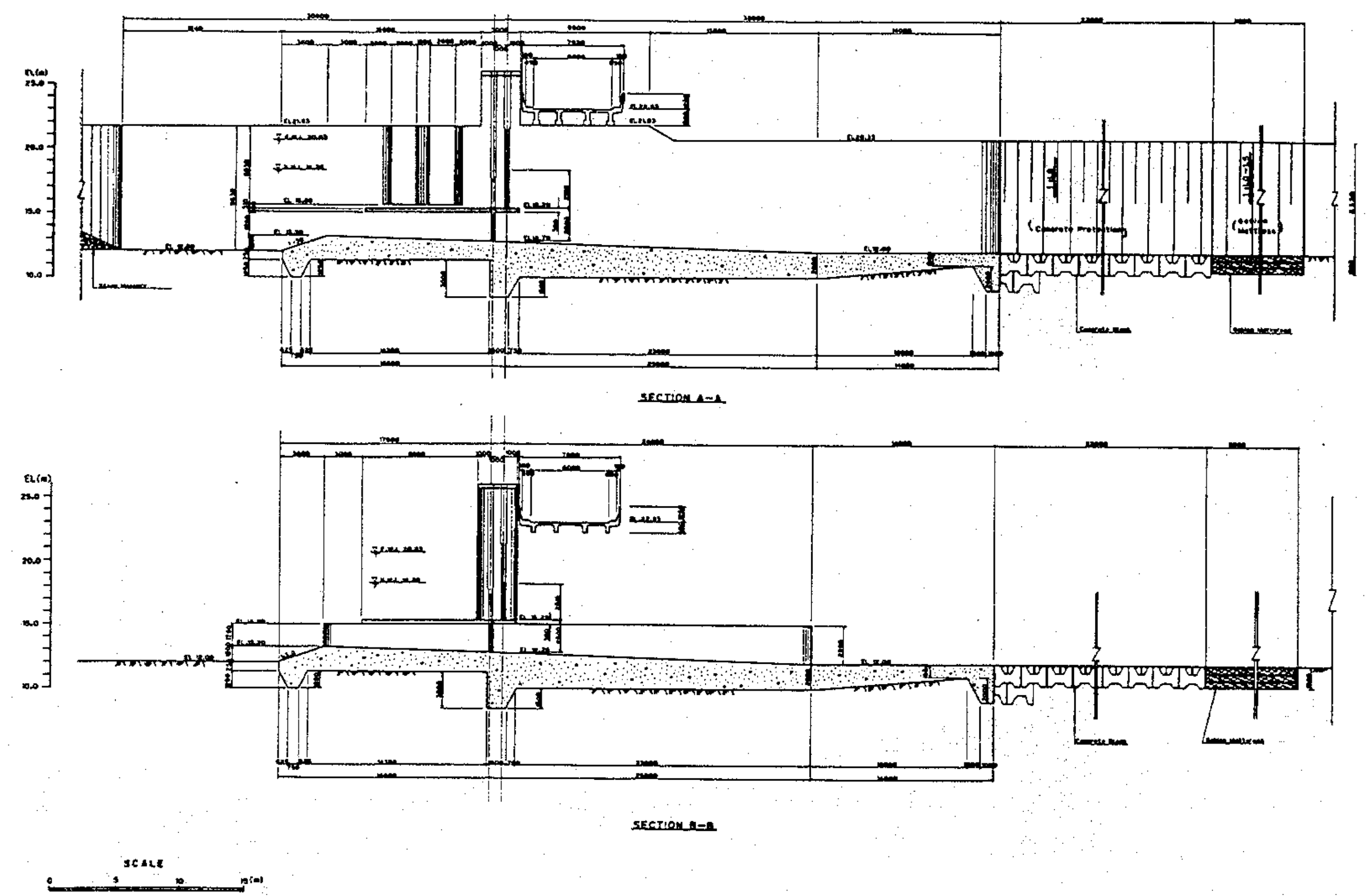


Figure A.3.24 Typical Section of Gilirang Weir (1/3)

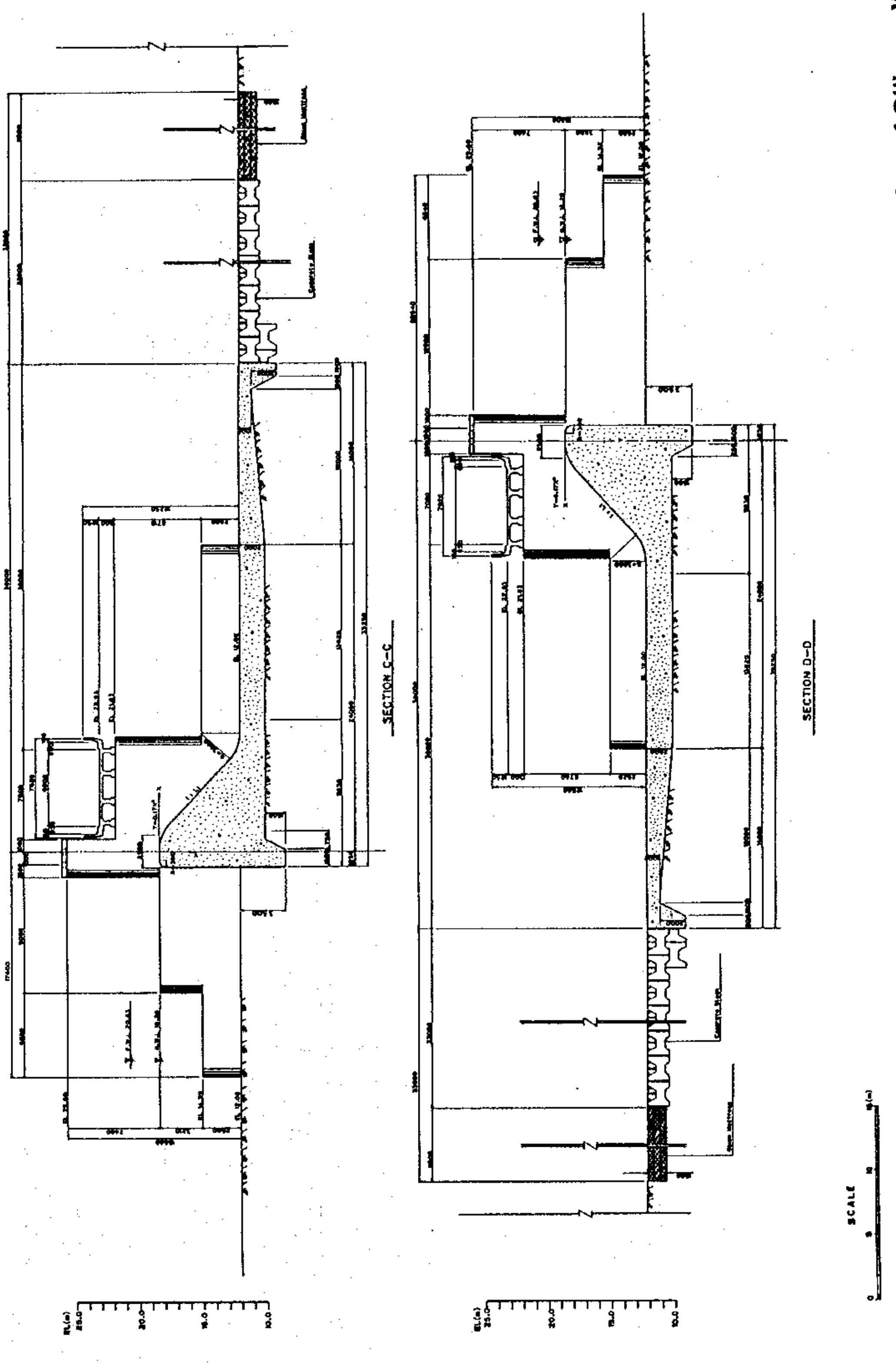


Figure A.3.25 Typical Section of Gilirang Weir (2/3)

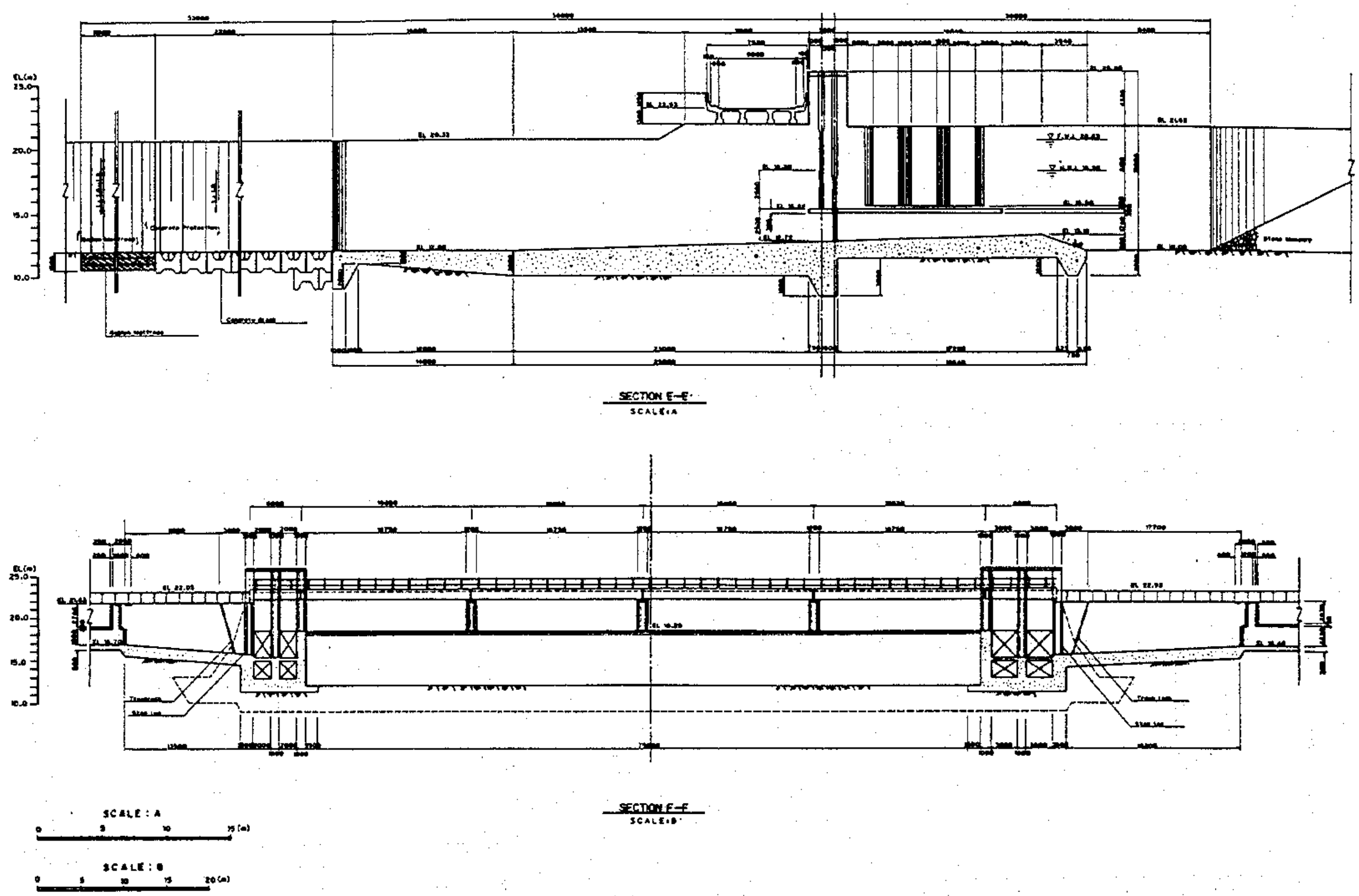


Figure A.3.26 Typical Section of Gilirang Weir (3/3)

ANNEX 4

IRRIGATION AND DRAINAGE

ANNEX 4 IRRIGATION AND DRAINAGE

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ANNEX 4 IRRIGATION AND DRAINAGE

1. EXISTING IRRIGATION AND DRAINAGE SYSTEM

1.1 Existing Pumping Irrigation System

There is no gravity irrigation system in the study area, even "non-technical and/or semi-technical grade of irrigation system ". And only small pumping irrigation system is found being operated by farmers themselves. Water source of the system depends mostly on the Gilirang river and pumping heads from the river are estimated at 5 to 15 m depending on topographic conditions. The pump bores vary 3 to 8 inches with 12 to 24 HP engine, covering about 40 ha of irrigation area per unit on an average.

All the pumps are operated only for the dry season paddy cultivation. Pumping operation periods and hours vary from 60 to 80 days per crop season and 12 to 24 hours per day respectively, depending on climate conditions and farmers' experiences. Canal networks are very primitive and only on-farm ditches are allocated to distribute irrigation water to plot, and they are maintained by users. The pumps are generally owned by private farmers; some are hired from private companies on contract base and the others are supported by the government.

There are 20 units of pumping irrigation facilities in the study area, irrigating 640 ha of dry season paddy field in total.

The pumping irrigation in the upper Gilirang river basin such as Paselloreng and Arajang has been extended to 200 ha or more and operated for 8 to 12 years so far; that in the downstream such as Toduma and Benceng-2 has been started in the recent year and still at trial stage. The high extension of pumping irrigation in the upper Gilirang river basin may show the critical cropping conditions under the limited available farm land thereabouts.

The owner farmers organize their users' member, whose average number is 42 farmers per unit. Water charge is collected to be 20% of harvesting amount or equivalent to Rp. 135,000/ha. Operation cost (fuel and lubricant cost) varies according to capacity of pump, pump head, rainfall distribution and operation technique, etc. Preliminary estimate of the operation cost by the Study Team shows Rp. 47,000/season/ha based on the interview with the farmers. In addition to the operation cost, operator charge and depreciation costs should be further considered.

Some pumping facilities were leased under name of village chief or farmers' leader with a price of Rp. 1,500,000/unit/crop season for the year of 1993. Inventory of pump irrigation facilities in the study area and their distributions are summarized in Table A.4.1 and Figure A.4.1.

1.2 Drainage Condition

There are scattered some poor drainage tracts in the study area , since no drainage system have been developed until now. The poor drainage tracts are generally located around the confluence of the Gilirang river and its tributaries. Because of gentle gradient of the Gilirang river, the drainage conditions along the riparian tracts of its tributaries are much effected with the water level of the Gilirang river; the conveyance capacity of the tributaries themselves are extremely insufficient, and it may cause flooding in the surrounding tracts.

In addition to excess runoff from upper basins, low-lying coastal belts near the sea, such as Akkajeng and Doping, are much influenced by tide when peak runoff will meet high tide. According to farmers interviews, frequency of flood which will cause one-day inundation will be once a year , and 2-3 days continuous inundation , once every 2-3 years. The damages to crops by inundation are depending on kinds of crops, their growing stage, depth and duration of inundation. However, in order to minimize the depth and duration of inundation, improvement of drainage system is highly essential in this coastal belts. There are six (6) typical poor drainage areas with a total area of 810 ha in the study area; details and their locations are summarized in Table A.4.2 and Figure A.4.1.

2. IRRIGATION WATER REQUIREMENT

2.1 General

Irrigation water requirement is one of the most important constituents to determine the final project scale through the analysis of available water sources for irrigation. In addition, the irrigation water requirement is basic component for the determination of the design discharges for all irrigation facilities. In this Annex, estimate of the irrigation water requirement is carried out.

The estimate of the irrigation water requirements for paddy and for palawija are made on half-monthly basis as follows:

(a) Irrigation water requirement for paddy

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

(b) Irrigation water requirement for palawija

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

where,
- WRD : Irrigation water requirement (mm)
- CU : Consumptive use of water (mm)
- PL : Percolation loss (mm)
- LP : Land preparation requirement (mm)
- ER : Effective rainfall (mm)
- E : Combined irrigation efficiency

2.2 Consumptive Use of Water

2.2.1 General

There is no actual measurement data of consumptive use in this area. The consumptive use of water by crops is estimated based on empirical prediction method, using the climatic data and crop coefficient at rating crop growth stages, as expressed below :

$$CU = Kc \times ETP$$

where,
- CU : Consumptive use of water (mm/day)
- Kc : Crop coefficient at rating crop growth stages
- ETP : Potential evapotranspiration estimated by the empirical prediction method with climatic data (mm/day)

Though there exist various prediction methods for the estimate of ETP, the modified Penman method is employed for the estimate of ETP in accordance with the recommendation in the DGWRD Standard.

2.2.2 Crop coefficients (Kc)

(1) Kc for Paddy

The following coefficients for paddy which are determined according to the FAO Irrigation and Drainage Paper No. 24 are adopted. These are the same coefficients with the Bila Irrigation

Project.

Growth Stage		Wet Season Paddy	Dry Season Paddy
1st month	1st half month	1.1	1.1
	2nd half month	1.1	1.1
2nd month	1st half month	1.1	1.1
	2nd half month	1.08	1.09
3rd month	1st half month	1.03	1.05
	2nd half month	0.95	0.95

(2) Kc for Palawija

The coefficients for palawija for the Project are estimated as the combined coefficients of maize, green beans and soybean as follows :

Growth Stage		Palawija
1st month	1st half month	0.5
	2nd half month	0.75
2nd month	1st half month	1.01
	2nd half month	1.05
3rd month	1st half month	0.96
	2nd half month	0.6

2.2.3 Potential evapotranspiration (ETP)

The modified Penman method is applied for the estimation of continuous 18 year's potential evapotranspiration from 1976 through 1993 as shown in Table A.4.3. The calculation is made using 18 years period of meteorological data obtained at the Sengkang meteorological station. The data used for the calculation of the evapotranspiration are summarized in Annex 2.

2.3 Percolation Rate

The field measurement was carried out in the stage of Master Plan Study and the results show the average rate is 1 mm/day in both dry and wet seasons. Furthermore, there are some results of 1 to 2 mm/day in accordance with percolation tests in the Bila Irrigation Project and Langkeme Irrigation Project. Considering above conditions, 2 mm/day of percolation rate is adopted in this study.

2.4 Land Preparation Requirement

Land preparation requirement includes the requirement for water layer replacement, nursery water requirement and puddling water requirement. As for the requirement for the water layer replacement, 50 mm each are applied at 1 and 2 month after transplanting. For the calculation of irrigation requirement during the land preparation, the recommended method in the DGWRD Standard which was developed by Van de Goor and Zijlstra is applied. The total requirement of 250 mm (including 50 mm for water layer replacement) is taken and the land preparation period is considered as 30 days. The calculation is made for continuous 15 years from 1979 through 1993. The results are shown in Table A.4.4. The adopted formula is as follows :

$$IR = Me^k/(e^k - 1)$$

where, IR = irrigation requirement at field level in mm/day

M = water requirement to compensate for evaporation and percolation in the field which is already saturated,

$M = Eo + P$ in mm/day

where, Eo = open water evaporation taken at 1.1 ETP during land preparation in mm/day

P = percolation in mm/day

K = MT/S

where, T = land preparation period in days

S = pre-saturation requirement added with 50 mm water layer in mm

2.5 Effective Rainfall

Rainfall data observed at Sakkoli rainfall station is used for the estimate of effective rainfall. Sakkoli rainfall station locates at the center of the project area and the data in this station seems to be most reliable comparing with the other stations. Lack of data are compensated with the data observed at Peneki rainfall station.

2.5.1 Effective Rainfall for Paddy

Effective rainfall during a growing period of paddy is estimated by the daily depth balance method based on the resent 15 years rainfall records from 1979 through 1993. The following assumptions are made prior to the calculation :

- rainfall less than 5 mm/day is ineffective due to evaporation from soil surface,
- rainfall more than 40 mm/day is also ineffective considering the height of ridge in the project area, and
- 80 % of the rainfall which is greater than 5 mm/day and less than 40 mm/day is effective.

The effective rainfall is estimated on half monthly basis and its calculation results are shown in Table A.4.5.

2.5.2 Effective Rainfall for Palawija

Effective rainfall for palawija is estimated based on the evapotranspiration/precipitation method prepared by USDA.

$$ER = 0.2 \times R^{0.95} \times Cu^{0.31}$$

where, ER : Monthly effective rainfall (mm)

R : Monthly rainfall (mm)

Cu : Crop water requirement (mm)

In the above calculation, the effective rainfall should not exceed crop water requirement. The monthly effective rainfalls are converted into half monthly effective rainfalls on pro rata basis. The results are tabulated in Table A.4.6.

2.6 Irrigation Efficiency

Irrigation efficiencies of paddy field irrigation and upland field irrigation are determined, taking into account the following conditions :

(i) Upland field irrigation is conducted by surface irrigation methods.
(ii) Lining canals up to rotation blocks are adopted.

The overall irrigation efficiencies for respective paddy and upland irrigation are estimated as follows :

Irrigation efficiency	Paddy field	Upland field
Conveyance efficiency	85 %	85 %
Application efficiency	75 %	60 %
Overall efficiency	64 %	51 %

2.7 Irrigation Water Requirements

2.7.1 Field Water Requirement

Field water requirement is defined as water required for crop growth at the field. The field water requirement is estimated on half monthly basis for proposed cropping pattern in the period of 15 years from 1979 to 1993. The sample calculations of the field water requirement are shown in Table A.4.7 and A.4.8. The field water requirements for paddy (dry season paddy and wet season paddy) and for palawija are shown in Table A.4.9 and A.4.10 respectively. The total field water requirements are shown in Table A.4.11.

2.7.2 Unit Diversion Water Requirement

The irrigation water requirement per unit irrigation area at the intake-site is called unit diversion water requirement. The diversion water requirement includes water required for crop growth and for compensation of losses of water during conveyance and operation. The unit diversion water requirement is usually indicated as the required water discharge for the unit irrigation area, l/sec/ha.

The calculation is made for continuous 15 years period from 1979 through 1993 and the unit diversion water requirements for paddy and palawija are shown in Table A.4.12 and A.4.13 respectively. These table indicate that the maximum unit diversion water requirement is 1.74 l/sec/ha for dry season paddy, 1.69 l/sec/ha for wet season paddy and 1.40 l/sec/ha for palawija.

3. DRAINAGE WATER REQUIREMENT

3.1 General

In the irrigation project, there are two kinds of drainage water requirements, in general. One is that for the inside of the project area and the other one is that for the outside of the project areas. The first one is the drainage water requirement to remove the excess water in the project area, and the second one is that to transport the runoff from the outside of the project area. The drainage water requirement for both areas are estimated in accordance with DGWRD Standard.

3.2 Inside of the Project Area

The drainage water requirement is calculated by using the following formula:

$$Qd = 1.62\, A^{0.92}\, Dm$$
$$Dm = D(3)/(3 \times 8.64)$$
$$D(3) = R5 + 3(IR - ET - P) - \Delta S$$

where,
- Qd : Design drainage requirement (l/sec/ha)
- Dm : Drainage modulus (l/sec/ha)
- A : Drainage area (ha)
- D(3): Surface drainage runoff in 3-day (mm)
- R5 : 3-day consecutive rainfall with A return period of 5-year (mm)
- IR : Irrigation supply (mm/day)
- ET : Evapotranspiration (mm/day)
- P : Percolation (mm/day)
- ΔS : Additional storage (mm)

For the computation of the drainage modulus, the components can be taken as follows in accordance with the DGWRD Standard.

- Irrigation supply (IR) equals zero as irrigation is stopped in heavy rainfall days.

- Additional storage (ΔS) at the end of consecutive days is 50 mm at maximum.
- Percolation (P) is assumed 0 mm/day.
- Average evaporation for rainy season from March to August is assumed to be 4.5 mm/day.
- Probable 3-day rainfall with a return period of 5 year is estimated by using data obtained at Sakkoli station which locates almost center of the project area. The probable 3-day rainfall is estimated at 299 mm by Gumbel method.

Moreover, drainage water requirements in paddy field are determined to involve areal size of paddy field. So that, the drainage water requirements in the project area are summarized as follows:

Irrigation Area (ha)	Drainage Water Requirement (l/sec/ha)
$A \leq 400$	9.1
$A > 400$	$1.62 \times A^{0.92} \times 9.1$

3.3 Outside of the Project Area

For the estimate of the peak discharge for drainage areas in the outside of the project area, Der Weduwen formula is adopted by using probable daily rainfall with a return period of 5-year, in accordance with the DGWRD Standard. The probable rainfall for outside of the project area is estimated at 165 mm/day from the data observed at Sakkoli station. The peak discharges in various drainage area and land gradient are shown in Figure A.4.2.

Design drainage requirement for the out side is estimated by the following formula as described in the DGWRD Standard. The probable daily rainfall with a return period 5-year at Sakkoli station is also adopted.

$$Qd = 0.116\ \alpha\ R_5\ A^{0.92}$$

where, Qd : Design discharge (l/sec)

α : Runoff coefficient (0.75)

R_5 : Probable daily rainfall (165 mm/day)

A : Drainage area (ha)

4. DELINEATION OF THE PROJECT AREA

4.1 Assessment of Irrigable Area

4.1.1 General

The topographical conditions of main canal route proposed by the Master Plan was reviewed through the 1/5,000 topographical map (due to lack of mapping area of the east part of the Project area, partly reviewed through the 1/25,000 topographical map) and the field reconnaissance was made by the Study Team. The results showed that the proposed main canal route should be examined very carefully to minimize the construction cost and to maximize the irrigation area due to complicated topographical condition. Especially, the route of head race (from the proposed dam site to the starting point of main canals) with a length of 6 km should be carefully examined from both technical and economical points of view. Therefore, the possible intake system by intake weir downstream of the Gilirang river was considered as alternatives in addition to the direct diversion from the proposed reservoir.

4.1.2 Maximum Potential Irrigable Area

The proposed route of main canals proposed in the Master Plan was examined based on the 1/5,000 topographical map (partly with 1/25,000 map) to maximize and identified irrigable area. The scattering upstream areas are also examined and proposed to be included in the basic development concept, because of giving priority to the upstream areas and existing pump irrigation areas. The whole potential irrigable area of the existing paddy field with direct diversion from the reservoir by gravity was estimated at 9,500 ha in gross and 8,600 ha in net (assumed at 90 % of gross area). The distribution of the above areas by elevation is summarized below;

Irrigable Existing Paddy Field from Topographical Condition
(with direct diversion from the reservoir by gravity)

	(Gross)	(Net)
EL 30 m - 20 m	900 ha	800 ha
EL 20 m - 10 m	4,400 ha	4,000 ha
EL 10 m >	4,200 ha	3,800 ha
Total	9,500 ha	8,600 ha

4.1.3 Intake Weir

Alternative intake system other than direct diversion from the reservoir is considered, which gets water by an intake weir downstream the Gilirang river after discharging from the reservoir. There are a lot of candidate sites for the intake weir though, the design intake water level is limited at less than EL 18.0 m in order to avoid the effect of back water to the proposed dam upstream the Gilirang river, which requires the seepage observation after the completion of dam.

Due to the limitation of the design intake water level (EL 18.0 m), the reason of which was mentioned above, the irrigable area with an intake weir by gravity is estimated at 5,880 ha.

4.1.4 Small Scale Pumping Irrigation

As described in the section of "Exploitable water resources by dam construction" of the main report, water resource availability will be possible to irrigate more than 5,880 ha, which is the irrigable area by intake weir mentioned above. In addition, the followings are taken into account;

- Small pumping irrigation from the Gilirang river is already familiar with the farmers in the project area.
- The results of the house hold survey show the positive intention of pumping irrigation, if water is available.
- Small pumping irrigation to some outside areas of the main canals is easy and economical.

Considering the above conditions, the possibility of small scale pumping irrigation was examined and identified 1,120 ha of the area summarized below;

Proposed Small Scale Pumping Irrigation Area	
Pumping from Main Canals	680 ha
Pumping from Upstream the Gilirang River	440 ha
Total	1,120 ha

4.1.5 Assessment of Irrigable Area

Based on the conditions of topography, water availability and the economical possibility of pumping irrigation, the irrigable area is classified into three (3) cases, namely, (I) 8,600 ha, (II) 5,880 ha, (III) 7,000 ha with respective irrigation systems, summarized below;

Irrigation System		Irrigable Area (ha)
(I)	Divert from reservoir	8,600
(II)	Intake from weir	5,880
(III)	Intake from weir and Pump	7,000
(Weir : 5,880 ha + Pump : 1,120 ha)		

4.2 Preliminary Layout and Cost Estimate

4.2.1 Preliminary Layout

Considering the conditions above, three(3)alternatives are taken into account for comparative study, summarized below;

Irrigation System		Irrigation Area (ha)
Alternative-I	Divert from reservoir	8,600
Alternative-II	Intake weir	5,880
Alternative-III	Intake weir in combination with Pumping	7,000
(Weir : 5,880 ha + Pump : 1,120 ha)		

Where, Alternative-I is an idea that all the maximum irrigable area from the topographical condition estimated at 8,600 ha is irrigated by the water from the reservoir through a headrace and two(2) main canals.

Alternative-II is an idea that the irrigation area will concentrate in downstream from proposed intake weir, covering 5,880 ha.

Alternative-III is basically the same as Alternative-2, covering 5,880 ha and additionally some grouped areas upstream along the Gilirang river and out side of the proposed main canals be taken up to irrigate with pump covering about 1,120 ha, with a total irrigation area of 7,000 ha. Among the pumping irrigation areas above, the area upstream along the Gilirang river is 440 ha and the areas outside of the proposed main canals is 680 ha, respectively. General layout of three (3) alternatives is shown in Figure A.4.3 - A.4.5.

4.2.2 Preliminary Costs of Alternatives

In order to compare the costs of alternatives, the quantities of major work items such as excavation, embankment, concrete and gate were assumed and the direct cost of main facilities were preliminary estimated based on the unit prices of Bila and Langkeme Irrigation Projects.

The cost comparison of major facilities including dam and on-farm development, was made based on the following design conditions and features;

Design conditions and Dimensions of Major Facilities			
Alternative-I:			
(1)	Rockfill Dam :	Maximum storage capacity	: 132 MCM
		Crest elevation	: EL 56.5 m
		Dam height	: 44.5 m
(2)	Headrace :	Length	: 6.0 km

		Design canal discharge	: 15.0 m3/sec
		Type of canal	: Unlined canal
(3)	Main canal :	Total length of canal	: 74.0 km
		Design discharge (left)	: 4.4 m3/sec
		Design discharge (right)	: 10.6 m3/sec
		Type of canal	: Unlined canal
Alternative-II:			
(1)	Rockfill Dam :	Maximum storage capacity	: 125 MCM
		Crest elevation	: EL 56.0 m
		Dam height	: 44.0 m
(2)	Weir :	Type of weir	: Fixed type
		Material of weir	: Concrete
		Intake water level	: EL 18.0 m
		Width of scoring sluice	: 2.0 m x 2 nos.
			3.0 m x 2 nos.
(3)	Main canal :	Total length of canal	: 47.5 km
		Design discharge (left)	: 3.5 m3/sec
		Design discharge (right)	: 6.7 m3/sec
		Type of canal	: Unlined canal
Alternative-III:			
(1)	Rockfill Dam :	Maximum storage capacity	: 132 MCM
		Crest elevation	: EL 56.5 m
		Dam height	: 44.5 m
(2)	Weir :	Type of weir	: Fixed type
		Material of weir	: Concrete
		Intake water level	: EL 18.0 m
		Width of scoring sluice	: 2.0 m x 2 nos.
			3.0 m x 2 nos.
(3)	Main canal :	Total length of canal	: 47.5 km
		Design discharge (left)	: 3.7 m3/sec
		Design discharge (right)	: 7.6 m3/sec
		Type of canal	: Unlined canal
(4)	Pumping unit :	- 1.4 m3/min., 4 inch	: 6 nos.
		- 3.5 m3/min., 6 inch	: 22 nos.
		- 6.0 m3/min., 8 inch	: 13 nos.

The estimation of cost comparison shows that the cost of Alternative-I, including all administrative project costs is 252 billion Rp., Alternative-II is 157 billion RP. and Alternative-III is 161 billion RP. respectively, as summarized below and details in Table A.4.14.

Preliminary Cost of Alternatives	
Alternative-I	252 billion Rp.
Alternative-II	157 billion Rp.
Alternative-III	161 billion Rp.

It is clear that the high cost of Alternative-I comes from big volumes of earth work and high unit prices for headrace and main canal construction due to topographical and geological conditions.

4.3 Delineation of the Project Area

The proposed route of main canals of Alternative-III was further examined based on the 1/5,000 topographical map (partly with 1/2,5000 map) to maximize and identified irrigable area. After reconnaissance survey of proposed main canal's routes, the longitudinal and cross sectional survey for main canals was carried out to design and estimate earthwork quantities of the canal construction.

The scattering upstream areas and outside the proposed main canals for small scale pumping irrigation are also examined and proposed to be included in the basic development concept. The whole potential irrigable area of the existing paddy field was estimated at 7,780 ha in gross and 7,000 ha in net (assumed at 90 % of gross area). Among the above areas, 5880 ha will be irrigated by gravity irrigation system with an intake weir at the down stream the proposed dam site and the remaining 1,120 ha will be irrigated by small scale pumping system, out of which the areas of 440 ha are located upstream along the Gilirang river and the remaining areas of 680 ha are located along the proposed main canals, respectively. The proposed irrigation area (the Project Area) is summarized below and shown in Figure A.4.5.

Proposed Irrigation Area (the Project Area)	
Gravity Irrigation Area by Intake Weir	5,880 ha
Pumping Irrigation Area	1,120 ha
(from Upstream Gilirang river : 440 ha +	
from Main Canals : 680 ha)	
Total	7,000 ha

5. PRELIMINARY DESIGN OF IRRIGATION FACILITIES

5.1 Layout of Irrigation and Drainage Canal Networks

5.1.1 Irrigation Canal System Development

Irrigation canal system in the project area consists of main canals, secondary canals, tertiary canals and quaternary canals. For the alignment of main canals, followings were taken into account.

- To minimize the canal construction costs, since the construction costs of canals will occupy one of the largest parts of the whole project cost.

- To maximize the potential irrigable area by maintaining as much higher elevation of canals as possible, keeping the necessary longitudinal slope.

- To minimize the work quantity of canal construction by such a way of short-cut based on the topographical condition, though the main canal generally runs along contour lines.

- To examine carefully the sections where the canals run at the foot of hills, in order to avoid the damage by heavy rain.

- To examine the length of main canals in the form of straight lines, as much as possible, and deliver the water to the command areas with secondary canals.

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

The Left Main Canal will be constructed to serve an area of 2,105 ha located in the left bank of the Gilirang river. Out of the area of 2,105 ha, the area of 2,030 ha will be irrigated by gravity and the remaining area of 75 ha be irrigated by pump. The Right Main Canal will be constructed to serve an area of 4,445 ha located in the right bank of the Gilirang river. Out of the area of 4,455 ha, the area of 3,850 ha will be irrigated by gravity and the remaining area of 605 ha be irrigated by pump. Those pump irrigation areas above will be directly served with tertiary supply canals from the main canals. As for the upstream scattered pumping irrigation areas of 440 ha will be directly served with secondary and/or tertiary canals from the Gilirang river.

Total length of the main canals and secondary canals is estimated at 21.0 km for Left Main Canal , 26.5 km for Right Main Canal and 37.2 km for the secondary canals respectively, summarized below;

Canals	Length (km)	Irrigation Area (ha)		
		Gravity	Pump	Total
Downstream				
Left Main Canal	21.0	2,030	75	2,105
Right Main Canal	26.5	3,850	605	4,455
(Total Main Canal)	47.5 km	5,880	680	6,560
Secondary Canals	35.5			
Upstream				
Secondary	1.7		440	
(Total Secondary Canals)	37.2 km	5,880 ha	1,120 ha	7,000 ha

5.1.2 Drainage Canal System Development

There are scattered some poor drainage tract in the study area, since no drainage system have been developed until now. The poor drainage tracts are generally located around the confluence of the Gilirang river and its tributaries. Because of gentle gradient of the Gilirang river, the drainage conditions along the riparian tracts of its tributaries are much effected with the water level of the Gilirang river; the conveyance capacity of the tributaries themselves are extremely insufficient, and it may cause flooding in the surrounding tracts.

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

The drainage canal layout was worked out based on the 1/5,000 topographical map and the field investigation considering the following matter.

- Drainage water requirements, drainage method, required canal elevations at key points and general layout of drainage system are first confirmed.
- Drainage canal routes are laid out along the low land and as straight as possible.
- The alignment is worked out so as not to pass through village areas and not to give damages to public facilities.
- Raised portions of drains are minimized in order to keep canal water level below ground surface as much as possible.

The drainage canal system consists of major, tertiary and quaternary drains. The tertiary and quaternary drainage canals are developed in on-farm level development. The function of the major drain is to transport water from tertiary drains and flood water from surrounding mountainous area to the disposal points. The layout of the irrigation system and topography are the main factors in determining the location of all the drainage canals. The route of major drains are generally selected in the natural stream lines and low depressions.

5.2 Irrigation Canal and its Related Structures

5.2.1 Design Criteria

(1) Function and requirement of canal

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

(a) Main canal
In the project area, two main canals are provided; Left Main Canal and Right Main Canal. The main function of the main canal is to deliver irrigation water from the Gilirang weir to the project area in the shortest or economical way.

(b) Secondary canal
This is a canal branching off from the main canal to distribute water up to the secondary block. The size of secondary block varies from 130 to 1,250 ha, which is divided into 2 to 15 tertiary units. The canal is unlined and trapezoidal.

(c) Tertiary system
The tertiary units includes one tertiary canal and 10 - 15 quaternary canals. The maximum size of tertiary unit is about 150 ha. Whereas, a quaternary can covers 0 - 15 ha. Some tertiary units of which size exceeds 150 due to topography and/or administrative matters would be divided into sub tertiary units.

(2) Design discharge

The unit design discharge for the main and secondary canals is 1.74 l/sec/ha as calculated in Section 2.7.2 in this Annex, which is the water requirement with a peak of the past 15 years.

(3) Design criteria

The design of the proposed canals was made based on the DGWRD standard in principle, summarized below;

Summary of Design Criteria for Earth Canal

- Velocity	(m/sec)
Earth canal	0.3 - 0.7
- Roughness (Value for manning's formula) :	
(Discharge : m3/sec)	
< 1.0	0.0285
1.0 - 5.0	0.025
5.0 - 10.0	0.0235
(Concrete flume)	(0.015)
- Freeboard	
(Discharge : m3/sec)	(m)
0.5 - 1.5	0.5
1.5 - 5.0	0.6
5.0 - 10.0	0.75
- Canal base width/water depth (B/h) ratio	
(Discharge : m3/sec)	
0.5 - 1.5	1.2 - 1.8
1.5 - 3.0	1.8 - 2.3
3.0 - 5.0	2.3 - 2.9
5.0 - 7.5	2.9 - 3.5
7.5 - 10.0	3.5 - 3.9
- Side slope	
(Discharge : m3/sec)	

	<	1.5	1.0
1.5	-	10.0	1.5

5.2.2 Proposed Main and Secondary Irrigation Canals

Irrigation canal system in the project area consists of main canals, secondary canals, tertiary canals and quaternary canals. The proposed layout of the irrigation system is shown in Figure A.4.6. and the irrigation diagram in Figure A.4.7. The design of canal and structures are shown in the attached Drawings.

(a) Main canals

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

The Left Main Canal will be constructed to serve an area of 2,105 ha located in the left bank of the Gilirang river. This canal will run approximately eastwards from the intake along the skirts of the northern hill slopes. The total length will be 21.0 km. The alignment of the canal route is made to maximize the potential irrigable area by maintaining as much higher elevation of canals as possible, keeping the necessary longitudinal slope. This canal is designed for the discharge of 3.66 m3/sec at its head.

The Right Main Canal will be constructed to serve an area of 4,455 ha located in the right bank of the Gilirang river. This canal will run approximately south-eastwards from the intake along the skirts of the southern hill slopes with some short cuts of hills about less than 10 m in height. The total length of the canal is estimated at 26.5 km and the design discharge at its head is 7.75 m3/sec.

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

(b) Secondary canals

These canal will branch off from the above mentioned main canals to distribute water to the secondary irrigation units of which size will vary from 1,250 ha to 130 ha depending on topography. Fourteen (14) secondary canals with a total length of 37.2 km will be constructed in the Project area. These canals are designed principally as unlined earth canal with trapezoidal cross section of side slope of 1:1.5 to 1.0.

The typical cross sections of those canals adopted in this design are shown in Figure A.4.8.and the longitudinal profiles of main canal are in Figure A.4.9.

The number and total length of the main, secondary canals and the number of their related structures are as shown below;

Description	Left Bank	Right Bank	Total
Main Canal			
- Canal length (km)	21.0	26.5	47.5
- Related structure			
Turnout w/check (for gravity)	16	23	39
Turnout w/check (for pump)	5	23	28
Aqueduct	0	1	1
Road crossing (culvert)	7	8	15
Spillway/wasteway	8	10	18
Cross drain (box culvert)	8	7	15

Cross drain (pipe culvert)	57	39	96
Measuring device	1	1	2
Secondary canal			
- Nos. of secondary canal	5	9	14
- Length of secondary canal	8.1	29.1	37.2
- Related structures			
Turnout w/check (for gravity)	14	38	52
Road crossing (culvert)	2	7	9
Spillway/wasteway	1	5	6
Cross drain (box culvert)	0	1	1
Cross drain (pipe culvert)	13	44	57
Drop	4	2	6
Syphon	0	1	1

5.3 Pumping unit

5.3.1 Required Pumping Unit

According to the field survey about existing small scale pumping irrigation in and around the Project area, almost along the Gilirang river, the farmers have much intention for introducing dry season paddy cultivation with pump irrigation. The house hold survey results also show the farmers' much intention for pump irrigation in the dry season. The constraints for development of pumping irrigation are lack of available water and technical and management experience of the farmers.

Considering the conditions above, the following points are taken into account to estimate the required pump units;

- Introduction of small scale pump, which is more familiar with the farmers in the Project area
- Introduction of the same type and capacity of pump, which is easier for maintenance and getting spare parts, etc.
- Introduction of the movable type of pump, which will be housed during no irrigation period

Based on the above conditions and irrigation area of each tertiary block, required discharge of irrigation water and required total pump head, the proposed pump capacity was classified into three (3), namely Type (I) with a pumping capacity of 0.9 m3/min. and 10 PS of engine, Type (II) with a capacity of 3.5 m3/min. and 18 PS of engine and Type (III) with a capacity of 6.0 m3/min. and 27 PS of engine. There are 32 tertiary blocks for pump irrigation, having command areas of 3 ha to 134 ha, respectively. In addition, a secondary block with a command area of 307 ha, further divided into 6 tertiary blocks, located in Alusalo village is also proposed for pump irrigation. Required total pump head of each block also varies form about 3 m to 9.5 m. Accordingly, required number of pump units was estimated at 6 units of Type (I), 22 units of Type (II) and 13 units of Type (III), summarized below;

Required No. of pump units	
Nos. of pump irrigation block	32 tertiary blocks 1 secondary block
Type (I) : 1.4 m3/min., 4 inch, 10 PS	6 units
Type (II) : 3.5 m3/min., 6 inch, 18 PS	22 units
Type (III) : 6.0 m3/min., 8 inch, 27 PS	13 units
Total	41 units

5.3.2 Operation Cost

Comparative study for pump operation was made between the use of the micro hydropower at the Pasellorеng dam and diesel engine pump. Based on the results of the study above, suggesting that the use of diesel engine pump is more economical, the Project applies the pump units with diesel engine for pumping irrigation area. The detailed discussion is described in Capter 6 in this Annex.

Operation and maintenance costs of pumping irrigation are estimated based on the proposed cropping pattern (paddy+palawija+paddy). The annual operation hour is estimated at 3,112 hours and annual fuel consumption per pumping unit is estimated at 3,641 lit./unit for Type (I) pump with 10 PS engine, 6,566 lit./unit for Type (II) pump with 18 PS engine and 9,834 lit./unit for Type (III) pump with 27 PS engine, respectively. This shows that the annual operation costs per pumping unit will be about 1,700,000 Rp./unit for Type (I), 3,100,000 Rp./unit for Type (II), and 4,600,000 Rp./unit for Type (III) including lubricant cost (20 % of fuel cost), respectively and total annual pumping operation cost of the project will be about 137 million Rp. Since proposed pump irrigation area is 1,120 ha in total, annual pump operation cost per ha will be about 123,000 Rp./ha, summarized below;

Pumping Operation Cost

Pump Type	Type(I)	Type(II)	Type(III)
PS(Horse Power)	10	18	27
Unit fuel consumption (lit./PS/hr)	0.117	0.117	0.117
Required units of pump facilities(nos.)	6	22	13
Annual operation hour (hour)	3,112	3,112	3,112
Annual fuel consumption/unit (lit.)	3,641	6,566	9,834
Total annual fuel consumption (lit)	21,844	144,172	127,789
Diesel oil price (Rp./lit.)	389.6	389.6	389.6
Lubricant (20% of fuel)	77.9	77.9	77.9
Fuel & lubricant cost per unit (mill. Rp.)	1.7	3.06	4.6
Total fuel & lubricant cost (mill. Rp)	10.2	67.3	59.8
Total pump irrigation area		1,120 ha	
Total pump operation cost (whole the project)		Rp.137.3 mill.	
Pump operation cost (per ha)		Rp.123,000 /ha	

5.4 Drainage Canal and its Related Structures

5.4.1 Function and Requirement of Drainage Canal

This drainage canal system consists of major, tertiary and quaternary drains. The tertiary and quaternary drainage canals are described in Section 5.5 in this Annex. Herein mentioned is the design of major drainage canals. The function of the major drain is the major drains transport water from tertiary drains and flood water from surrounding mountainous areas to the disposal points. The layout of the irrigation system and topography are the main factor in determining the location of all the drainage canals. The location of major drains is dominated by natural streams and rivers exist in the Project area. These natural streams and rivers are used as much as possible as the major drains.

5.4.2 Design of Drainage Canal System

(1) Design discharge

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

(a) Drainage requirement for removal of excess rainfall in the paddy fields (= Qp),
(b) Drainage requirement for passing the high flow coming from the out side project area (= Qo),

The design discharge (= Qd), is determined by the following basis;

Qd = Qp + Qo

Qp and Qo are detailed in Chapter 3 in this Annex.

(2) Canal section

The drainage canal sections are designed for the following criteria:

Type of canal	:	Trapezoidal earthen canal
Permissible velocity	:	0.3 - 0.6 m3/sec
Roughness coefficient	:	0.03
		(for the use of manning's formula)
Side slope of canal	:	1:1.0

(3) Related structures

The structures related to the drainage system are bridges, drops and drainage junctions. The bridge is provided at the road crossing. The drops are of cascade type with trapezoidal section. The gabion mattresses are used for the drainage drops. The drainage junctions are provided at the connecting points of major drains to protect drains from bed erosion.

5.4.3 Proposed Drainage Canal System

The drainage canal system is networked so as to evacuate the excess water in the fields and to transport the stream flows occurred in the outside project area to the Gilirang river, the Muala river. The drainage system will consist of major drains, tertiary drains and quaternary drains which are constructed with in the tertiary blocks, and transport collected water inclusive of stream flows to the above rivers. The routes of major drains are generally selected in the natural stream lines and low depressions.

There are 17 major drains (natural streams, branches of the Gilirang river) with a total length of 151.9 km in the Project area. Based on the above considerations, it is proposed to be excavated some parts of existing major drains with a total length of 57.2 km, summarized below. The typical cross sections of drainage canal adopted in the design are shown in Figure A.4.8.and the drainage diagram showing the drainage area and the design discharge is as shown in Figure A.4.10.

Major Drains in the Project Area	
Nos. of Major Drains	17 drains
Total length within the Project Area	151.9 km
Proposed Length for Excavation	57.2 km

5.5 On-farm Development

5.5.1 General

On-farm development program aims at efficient water management by establishing well organized tertiary system and through refined rotational irrigation program. The design and operational programming for the tertiary system of the Project are based on the DGWRD standard and guideline manual. The tertiary development program is prepared for every tertiary block. This tertiary block is further divided into several subordinate blocks like sub-tertiary blocks and quaternary blocks. The definition and the recommended size of each irrigation block are briefed as follows:

(a) Tertiary block

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

(b) Sub-tertiary block

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

(c) Quaternary block

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

5.5.2 Tertiary Irrigation System

(1) Canal system

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

(a) Tertiary canal

The tertiary canal delivers irrigation water from secondary irrigation canal or sometimes directly from main canal to sub-tertiary canals and/or quaternary canals. The irrigation water should not be given directly to fields from the tertiary canal. For the alignment of these canals in the area with steep topography, the canal should be perpendicular to the contour line.

(b) Sub-tertiary canal

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

(c) Quaternary canal

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

(2) Related structures

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

(a) Tertiary division box

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

(b) Quaternary division box

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

(c) Measuring device

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

(d) Drop structure

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

(e) Culvert

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

(f) Cross drain

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

5.5.3 Tertiary Drainage System

(1) Drainage canal system

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

(a) Quaternary drain

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

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

(2) Related structure

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

(a) Drainage drop structure

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

(b) Drainage culvert

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

The typical layout of tertiary system is shown in Figure A.4.11. The layouts of tertiary systems of the representative are shown in Figure A.4.12 - A.4.14. There are 139 tertiary/sub-tertiary blocks in the Project area with a total irrigation area of 7,000 ha and a total tertiary canal of 236 km. The average length of tertiary canal per block is estimated at 1.7 km/block. The area of each block varies from 3 ha to 197 ha due to topographical conditions, some of vast blocks will be divided into several sub-tertiary blocks for better water management.

5.6 Farm Road Network and Inspection Road

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

5.6.1 Farm Road

The farm roads of 27.5 km long in total are improved and well networked with the provincial roads. The roads is design as to have a width of 6.0 m with gravel pavement. Catch drains are provided on both sides along the road to carry water of rainfall on the road surface and the surrounding into drainage canals. Cross drains with concrete pipe culvert are also constructed under the road at approximately 500 m intervals to pass drainage water in the catch drain. The proposed farm roads to be improved are four (4) sections summarized below;

Proposed Farm road to be improved

Section (Passage Village)	Length (km)
1. Gilirang - Arajang - Dam site	7.5
2. Benceng-2 - Bacubaccue - Padewakeng	6.2
3. Pontoe - Sarammae - Allapporeng	8.3
4. Sarammae - Bacubacue	5.5
Total	27.5

5.6.2 Inspection Road

(1) Main inspection road

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

Considering the future increase of vehicles for the inspection and operation and heavy construction equipment to be required for the canal maintenance and repair, the main inspection road is designed so as to have a width of 6.0 m and to be gravel surfaced with a pavement width of 3.0 m. The road is also used for the movement of agricultural products and equipment and for the day to day services between villages and from them to the trunk road.

(2) Secondary inspection road

The secondary inspection road is mainly provided alongside the secondary canals. All these roads have a width of 6 m and are gravel surfaced with a pavement width of 3.0 m. These roads link the paddy fields to the main road and are used for the purpose of farm operation.

(3) Tertiary inspection road

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

The following table shows the respective inspection road length;

Road	Length (km)
Main inspection road	47.5
Secondary inspection road	37.2
Tertiary inspection road	236.0

Typical cross sections of proposed farm road and inspection road are shown in Figure A.4.8.

6. OTHER INFRASTRUCTURES

6.1 Hydropower Development

6.1.1 Introduction

The hydropower development would be made by using the waterhead of released irrigation water of the Paselloreng dam. As described in the minutes of meeting for inception report, the basic concept of the hydropower generation is to be utilized only for the irrigation facilities such as gate operations of the proposed intake weir and dam and small scale pump irrigation facilities. In this section, to examine the possibility of development of mini-hydropower, the potential of annual power generation, required power generation for the above irrigation facilities will be investigated and economic evaluation will be made comparing with using the electricity from PLN (Persahaan Listrik Negara) together with generators.

6.1.2. Hydropower Development Plan

(1) Maximum discharge

The maximum discharge for hydropower generation is determined based on the released water governed by the reservoir operation. The maximum discharges for hydropower generation was determined at 4.0 m3/sec by using the released flow duration curve as shown in Figure A.4.15, which was made on the Paselloreng dam operation in the water balance study.

(2) Head loss

The head losses include loss due to trashrack, loss due to inlet, loss due to friction, etc. By using the released irrigation water of 6.0 m3/sec, out of which the discharge of 4.0 m3/sec be used for hydropower generation, the head losses were calculated at 0.60 m.

(3) Headwater level

The hydropower generation plant would be operated within the effective head of the water turbine between 120 % and 70 %. The high headwater level for operation of water turbine is EL 50.5 m, while the tail water level is EL 34.0 m.

The effective head of water turbine was calculated as follows:

(a) Effective head (120 %)

Total head = EL 50.50 m- EL 34.00 m = EL 16.50 m
Effective head = 16.50 m - 0.60 (head loss) = 15.90 m

(b) Effective head (100 %)

Effective head = 15.90 m / 1.2 = 13.25 m
Headwater elevation = EL 34.00 m +13.25 m + 0.60 m (head loss) / 1.2
= EL 47.75 m

(c) Effective head (70 %)

Effective head = 13.25 m x 0.7 = 9.28 m
Headwater elevation = EL 34.00 m + 9.28 m + 0.6 m (head loss) x 0.7 / 1.2
= EL 43.63 m

Consequently, the plant would be operated within the headwater elevation between EL 50.50 m and EL 43.63 m.

(4) Generated output

The generated out put is calculated by using the following equation:

$$P = 9.8 \times Q \times H \times Et \times Eg$$

where;

P = Generated output (kW)
Q = Discharge (m^3/sec)
H = Effective head (m)
Et = Efficiency of water turbine
Eg = Efficiency of generator

The generated output was calculated as follows;

	Reservoir Water Level		
	EL 50.50 m	EL 47.80 m	EL 43.60 m
Q	4.0	3.6	3.1
H	15.9	13.3	9.3
Et	0.86	0.83	0.77
Eg	0.85	0.82	0.77
P	455.6	319.4	167.5

(5) Annual power generation

On the basis of the results of the water balance calculation shown in Figure A.4.16 and Table A.4.15, the maximum annual power generation was calculated below;

Maximum annual power generation (1987) : 3,144,000 kWH / year
Minimum annual power generation (1982) : 646,000 kWH / year

(6) Selection of water turbine and generator

A tubular turbine of S-type was selected by using the turbine selection diagram, taking the effective head and maximum discharge into consideration An induction generator was also selected because of its simple electrical equipment. The selected water turbine and generator are shown below.

Item	Capacity
Tubular turbine (S-type)	350 kW
Induction generator	350 kVA

(7) Preliminary Cost estimate

Preliminary cost of the hydropower generating facilities was estimated as shown below.

(Unit = million Rp.)

Item	Q'ty	Amount
1. Generating Equipment	1 lot	1,920
Tubular turbine (350 kW)		
Induction generator (350 kVA)		
Control equipment,		
Indoor and outdoor switch gear, etc.		
2. Civil Works	1 lot	900
Earth works and concrete works		
Control house		
3. Transmission Line (from dam to weir)	1 lot	177
20 kV, 10 km		
Toal Cost		2,997

6.1.3 Required Power Generation

(1) Weir and dam

The use of generated power will be limited only for irrigation facilities such as gate operation of proposed dam, weir and pumping facilities. For the intake weir, equipped with 12 gates in total, namely 4 nos. of intake gates, 4 nos. of flood sluice way gates and 4 nos. of sand sluice way gates.

Generally, manual operation will be proposed for the gate operation of weir, taking account of economic points of operation and maintenance. However, considering the number and weight of gates, electric operation is recommended to make more successful and easier operation at flood. The electric power for the gate operation is estimated at 30 kW in total based on the following assumption;

Weight of gate = 5.0 ton/gates
Required electric power = 2.0 kW/gate
Total required electric power for gate operation = 2.0 kW x 12 gates = 24 kW
Miscellaneous = 10 KW
Total required electric power for the weir site = 24 + 10 = 34 kW (say, = 30 kW)

As for the dam site, a total power requirement of 60 kW was assumed for the operation of all gates, valves, lighting and other utilities of Paselloreng dam.

(2) Pumping facilities

Small scale pumping irrigation is included in this project, introducing 41 units of pump in total summarized below;

Required No. of Pump Units		
Nos. of pump irrigation block	32 tertiary blocks 1 secondary block	
Type (I) :	1.4 m3/min., 4 inch, 10 PS	6 units

Type (II) :	3.5 m3/min., 6 inch, 18 PS	22 units
Type (III) :	6.0 m3/min., 8 inch, 27 PS	13 units
	Total	41 units

In the Project, the proposed power for pumping is 10 PS to 27 PS diesel engine depending on the areas of the tertiary irrigation block, total pumping head and discharge requirement, etc. Total required power for pumping is estimated at about 1,100 kW based on the following assumption;

Total Required Power (Capacity) for Pumping

Type(I) pump (4 inch, 1.4 m3/min.): 10 PS engine (13.6 kW) x 6 units = 82 kW
Type(II) pump (6 inch, 3.5 m3/min.): 18 PS engine (24.5 kW) x 22 units = 539 kW
Type(III) pump (8 inch, 6.0 m3/min.): 27 PS engine (36.8 kW) x 13 units = 478 kW
Total required power = 1,099 (say = 1,100 kW)

Accordingly the required power for the operation of weir, dam and pumping irrigation will be 1,190 KW in total. Since the available capacity of generator will be 350 kVA, the use of hydropower generation will be limited only for 10 units of pump, 4 units of type(II) and 6 units of type(III) near weir site in addition to the dam and weir operation, summarized below;

Proposed Use of Hydropower Generation	
Available capacity of generator	350 kW
Required power	
Dam operation	60 kW
Weir operation	30 kW
Pump operation : Type(II) (6 inch) x 4 units	100 kW
Type(III) (8 inch) x 6 units	220 kW
Total required power in gross	410 kW
Total required power in net (*)	approx. 350 kW

(*) : It is assumed that all the gates operation and pumping irrigation will not be made at the same time.

6.1.4 Technical Discussion

(a) The potential annual power generation is about 646,000 to 3,144,000 kW.

(b) Total required power generation for all the gate operation of intake weir, dam and small pump irrigation will be 1,190 kW however, the capacities of the water turbine and generator by hydropower development will be 350 kW and 350 kVA, respectively. Accordingly, the generated power by hydropower at Paselloreng dam will cover only 1/3 of all the power requirement, including all the gate operation of intake weir, dam and 10 units of pump operation.

(c) It is planned that the hydropower plant would be connected with the operation house of the proposed weir site by transmission line of 10 km.

(d) Since the hydropower development plan is formulated by using the released irrigation water from the Paselloreng dam, the electricity can not be supplied in the irrigation off-season for about 57 days a year in average, shown in Table A.4.15.

(e) To stand by for the gate operation, an engine generator will be installed for both weir and dam sites..

(7) Since PLN (Perusahaan Listrik Negara) transmission line of 20 kV is available at both dam and weir site, the Project will introduce electricity from the existing transmission line for all the gates operation of dam and weir with additional distribution lines, 0.5 km

for weir site and 1.0 km for dam site.

6.1.5 Economic Consideration

As described above, the construction cost of the hydropower generating facilities with a capacity of 350 kW is estimated at Rp.2,997 x 10^6, thus, the construction costs per kW at Rp.8.56 x 10^6, respectively. For comparative examination, operating cost by the electricity from PLN and operation cost of 10 units pump by generator is estimated as follows.

Objective Operations for Comparison (Assumption)

Operation	Capacity	Annual operation hour
Dam operation	60 kW	8 x 365 = 2,920
Weir operation	30 kW	8 x 365 = 2,920
Pump operation :		
Type(II) (6 inch) x 4 units	100 kW (136 PS)	3,112
Type(III) (8 inch) x 6 units	220 kW (300 PS)	3,112

(1) Alternative-1 (Hydropower at Paselloreng dam site)

The construction cost of the hydropower development is changed into the annual equivalent cost by using the following formula :

$$Ca = Cs \times i \times (1+i)^n / \{ (1+i)^n - 1 \}$$

where,
- Ca = Annual equivalent cost
- Cs = Construction cost
- i = Discount rate (10%)
- n = Useful life year of facilities (20 year)

The annual equivalent cost is calculated as follows :

$$\text{Rp. } 2{,}997 \times 10^6 \times 0.10 \times (1+0.1)^{20} / \{ (1+0.1)^{20} - 1 \} = \text{Rp. } 352 \times 10^6$$

(2) Alternative-2

The initial cost for Alternative-2 is only for the distribution lines including some accessories, estimated at Rp. 39.1 x 10^6, summarized below;

Initial Cost of Alternative-2

Distribution line installation at Dam site (L=1.0 km) :	Rp. 23,800,000
Distribution line installation at Weir site (L=0.5 km) :	Rp.15,300,000
Total	Rp.39,100,000

And annual equivalent cost of the installation is calculated as follows:

$$\text{Rp. } 39{,}100 \times 10^3 \times 0.10 \times (1+0.1)^{20} / \{ (1+0.1)^{20} - 1 \} = \text{Rp.}4.6 \times 10^6$$

In addition, the annual consumption cost of fuel for the pump is calculated as follows;

- Fuel consumption of generator per hour = 0.117 lit. / PS. / hr
- 10 units of pump is equivalent to 436 PS.
- Rp.389.6 x 0.117 x 436 x 3,112 = Rp. 61.8 x 10^6

Further, the electric charge for both weir and dam site is calculated as follows;

Electric charge for PLN	
Dam site (60 kW)	
Monthly basic charge : Rp.5,300 x 60 (kW) x 12 (month) =	Rp.3,816,000
Consumption charge : Rp.250 x 8 (hour) x 365 (day) =	Rp. 730,000
Weir site (30 kW)	
Monthly basic charge : Rp.5,300 x 30 (kW) x 12 (month) =	Rp.1,908,000
Consumption charge : Rp.250 x 8 (hour) x 365 (day) =	Rp. 730,000
Total Annual Electric Charge	Rp.7,184,000

Total annual cost of Alternative-2 is summarized below;

Annual Equivalent Cost of Alternative-2	
Installation cost of distribution line	: RP.4,600,000
Fuel consumption	: Rp.61,800,000
Electric charge	: Rp. 7,184,000
Total	Rp.73.6 x 10^6

The cost comparison above shows that the annual equivalent costs of the hydropower is about 4.7 times as high as that of the utilization of electricity from PLN for dam and weir operation and diesel engine for pump irrigation, summarized below.

Annual Equivalent Cost
Hydropower = Rp.352 x 10^6
Generator = Rp.73.6 x 10^6

6.2 Domestic Water Supply Development

6.2.1 Introduction

As described in the minutes of meeting for inception report, the examination about the possibility of potable water supply to Sengkang was done by the Study team. The examination was made according to such items as water requirement for domestic purpose in Sengkang, water availability from the Paselloreng dam, topographical conditions of the pipe line route location and preliminary design of required facilities, shown below.

6.2.2 Water Requirement and Availability

The result of hydrological and water balance study of the Paselloreng dam shows that there will be possible to release necessary amount of water for domestic purpose in Sengkang based on the following assumptions;

Population of Sengkang :	23,000 (population of Tempe in 1992)
Predicted Population in 2000 :	25,000 (with a 0.09 % of annual increase)
Water requirement /person/day :	60 lit./person/day (minimum requirement)
Total water requirement :	1,500 m^3/day

6.2.3 Topographical Conditions and Water Intake Site

Based on the available topographical map with a scale of 1/50,000, the route of pipe line is examined, proposing that the water will be taken at the proposed intake weir site for irrigation with an elevation of EL 18.0 m. Consequently, the water will convey by pipe line along the provincial road via Anabanua to Sengkang with a total distance of 35 km. As shown in Figure A.4.17, the pipe line must go through the high land with an elevation of EL 110 m on the way to Sengkang and this will be one of the critical points of this plan, which may require additional facilities such as booster pumps and other equipment, described below.

6.2.4 Required Facilities and Pleliminary Cost

(1) Total diversion requirement

Total diversion requirement is assumed at 1,650 m^3/day, considering 10% of the conveyance loss in addition to the total water requirement.

(2) Required pump capacity

The maximum water consumption per hour is applied for the design maximum diversion requirement per day for domestic water supply plan, as below.

Design maximum diversion requirement per day
= (Total diversion requirement) x 1.3 = 2,145 m^3/day
Required pump capacity = 2,145 / (24 x60) = 1.5 m^3/min.

(3) Required number of pump unit

Two units of pump are required for this plan, one is for regular use and the other is for stand-by use, considering the changes of conveyance water volume and dispersion of risk, etc.

(4) Conveyance system

The following conveyance system is proposed based on the topographical conditions described above.

- A pump station will be installed at the intake site
- A distribution reservoir will be constructed at the highest land (EL 110 m), about 9.5 km from the intake site
- A water tower and elevated tank will be constructed in Sengkang, about 25.5 km from the highest land
- Water will be pumped up from the intake site to the distribution reservoir
- Water will be conveyed by gravity from the distribution reservoir to the elevated tank in Sengkang
- A 536 m^3 of 6 hours' water requirement will be applied as effective capacity for both the distribution reservoir and elevated tank.(2,145 m^3/day x 6/24 = 536 m^3)

(5) Required diameter of steel pipe

(5)-1 From intake site to the distribution reservoir

250 mm of steel pipe is recommended comparing the diameter of pipe and pump specification, based on the following conditions.

- Total length of conveyance pipe line = 9.5 km
- Total pumping head = 90 m
- Diversion requirement = 1.5 m^3/min.
- Required diameter of steel pipe = 250 mm
- Specification of pump : Pumping capacity = 1.5 m^3/min., Pumping head = 111m, Motor = 1,450 rpm, 45 kW

(5)-2 From the distribution reservoir to Sengkang

250 mm of steel pipe is also recommended considering the head loss of conveyance pipe

as below.
- Total length of conveyance pope line = 25.5 km
- Total head = 92 m
- Diversion requirement = 1.5 m^3/min.

(6) Summary of required main facilities and preliminary cost

Required main facilities from the proposed intake site to Sengkang and preliminary cost are summarized below;

Item	Description	Quantity	Cost (miil. Rp.)	
- Pump Station				
Pump facilities				
	Capacity : 1.5 m^3/min.	2 units	504	(*)
	Pumping head : 111 m			
	Motor : 1,450 rpm, 45 kW			
Pump house		1 house	100	
- Distribution reservoir				
	Capacity : 536 m^3	1 unit	26	
- Water tower and elevated tank				
	Capacity : 536 m^3	1 unit	26	
- Steel pipe	Diameter = 250 mm	35 km	26,746	(**)
Total Cost			27,402	

Note : (*) : including installation of equipment and spare parts, etc.
(**) : including installation of pipe, civil work

Table A.4.1 Inventory of Pumping Irrigation Facilities

Kecamatan (District)	Desa (Village)	Dusun (Sub-village)	Existing Pump System: Capacity (Dia.=inch)	Nos.ofUnit	Irrigation Area (ha)	Water Source	Ownership ,1)	Nos. of Users /Members	Location (See Figure)	Remark (Experience)	Operation Cost: Rp./season	Rp./season/ha
Maniangpajo												
1.	Arajang	Lawareng	6	1	60	Gilirang river	(P)	70	A	12 years	3,240,000	54,000
			6	1	25	Gilirang river	(P)	30	B	5 years	1,620,000	64,800
			6	1	10	Gilirang river	(P)	10	C	3 years	648,000	64,800
			6	1	10	Gilirang river	(P)	N.A., 5)	D, 6)			
2.	Gilirang	Ale Limpo	3	1	5	Gilirang river	(P)	10	E, 6)	2 years		
3.	Paselloreng	Lurae	8	1	50	Gilirang river	(P)	50	F			
			6	1	50	Gilirang river	(P)	50	G			
			6	1	50	Gilirang river	(P)	45	H	8 years	2,280,000	45,600
			4	1	15	Gilirang river	(P)	15	I	8 years	480,000	32,000
	(Sub-total - I)			9	275							
Sajoanging												
1.	Akkajeng	Mualla	8	2	70	Mualla river	(G), 3)	60	J, 6)			
2.	Doping	Doping baru	4	1	25	Ground water	(P)	20	K			
3.	Lawesso	Doping lama	3	1	20	Spring/pond	(P)	15	L			
4.	Barangmamase	Bottotella	4	1	15	Kulampu river	(P)	15	M			
5.	Sakkoli	Cinaga	3	1	5	Ground water	(G), 4)	N.A.,5)	N			
6.	Akkotengeng	Toduma	8	4	200	Gilirang river	(C), 2)	150	O	2 years	5,832,000	29,160
	(Sub-total - II)			10	335							
Majauleng												
1.	Botto Benteng	Benceng-2	6	1	30	Girilang river	(G), 3)	30	P	4 years	1,080,000	36,000
	(Sub-total - III)			1	30							
	(Total)			20	640						(Average)	46,623

Source: Kantor Dinas Pertanian and interviewed with the farmers by the Study team

Note:
1) : (P) Private, (C) Company, (G) Government
2) : BWR (Binawan Wajo Raya)
3) : Supported by DINAS
4) : P2AT (PU Pengairan Propinsi Sul-Sel, Kab. Wajo)
5) : Not Available
6) : Changing location by season

Table A.4.2 Poor Drainage Area

	Desa	Sub-village	Area (ha)	Drainage Condition	Remark	Location (See Figure)
1	Sakkoli/ Laerung	Taruntpakkai	80	1 day inundation once a year Flooding of Girilang river	E.L. approx. 17 .0 m	(1)
2	Salobulo	Salobulo	150	Joinnig point of Gilirang river and its branch (Marepi river), 2-3 days continuous inundation in every 2-3 years, 0.5 m of depth	Drainage improvement will be efective E.L. approx. 6.0 m	(2)
3	Akkajeng	Babana	300	2 - 3 days continuous inundation once a year, with tidal influence	E.L. approx. 2.0 - 3.0 m	(3)
4	Padaelo	Allapporange	200	Poor drainage condition of the small stream (branch of Girilang river) flowing through near Polewali village 1-2 days inundation once a year	Drainage improvement will be efective E.L. approx. 8.0 m	(4)
5	Akkajeng	Mualla	50	Poor drainage condition of Muala river(small stream) Inundated with heavy rain, much influenced by tide	E.L. approx. 7.0 m	(5)
6	Doping	Dopinglama	30	Poor drainage condition of Doping river(small stream) Inundated with heavy rain, much influenced by tide	E.L. approx. 6.0 m	(6)
		(Taotal area)	810			

Source : Interviewed with Extension Workers of Agricultural Office and the farmers

Table A.4.3 **Potential Evapotranspiration at Sengkang**

(Unit : mm/day)

	Jan.	Feb.	Mar.	Apr.	May	Jun.	Jul.	Aug.	Sept.	Oct.	Nov.	Dec.
1976	5.18	5.97	5.52	5.11	4.50	3.95	4.29	5.51	6.14	5.69	5.22	4.95
1977	5.05	5.47	5.60	4.96	4.61	3.75	4.60	5.29	5.68	6.49	5.36	4.31
1978	4.31	4.81	4.29	4.24	4.19	3.67	3.89	4.28	4.46	5.15	4.65	4.08
1979	4.21	4.21	4.12	4.06	4.21	3.46	4.12	5.36	5.27	5.83	5.00	4.10
1980	4.07	4.06	4.49	3.88	3.95	3.67	4.05	4.34	5.69	5.04	4.43	3.89
1981	5.13	4.33	4.47	4.21	3.87	3.96	3.45	4.78	4.06	5.31	5.11	4.02
1982	4.35	4.13	3.90	3.86	4.07	3.41	4.60	5.10	5.16	5.84	5.33	4.52
1983	4.16	4.67	4.44	4.31	3.69	3.57	3.59	4.68	5.60	5.13	4.61	3.98
1984	4.67	4.54	4.39	3.76	3.52	3.30	3.76	4.44	4.52	5.47	4.55	4.06
1985	4.78	4.48	4.73	4.29	3.36	3.41	3.45	4.38	5.19	4.74	4.43	4.49
1986	4.24	4.60	4.49	4.24	4.22	3.63	3.61	4.76	5.35	4.49	4.43	4.32
1987	4.39	4.94	4.38	4.62	4.06	4.29	4.55	5.40	6.46	6.49	4.94	4.00
1988	4.32	4.33	4.66	4.39	4.03	3.31	3.65	4.19	4.98	4.86	4.61	4.27
1989	4.53	5.04	4.52	4.53	4.01	3.89	3.40	4.00	5.24	5.04	4.88	4.41
1990	4.47	4.47	5.06	4.41	3.95	3.88	4.09	4.50	5.46	5.44	5.32	4.01
1991	4.50	4.52	4.80	4.46	4.06	4.17	3.99	4.53	5.49	5.98	4.73	4.49
1992	4.41	4.71	4.52	4.62	4.04	3.66	3.74	4.69	4.92	5.23	4.92	4.51
1993	4.66	4.95	4.36	3.92	3.79	3.72	3.65	4.84	5.69	6.55	4.79	4.50
Ave.	4.52	4.68	4.60	4.33	4.01	3.71	3.92	4.73	5.30	5.49	4.85	4.27

Table A.4.4 Land Preparation Requirement

(Unit : mm/day)

	Jan.	Feb.	Mar.	Apr.	May	Jun.	Jul.	Aug.	Sept.	Oct.	Nov.	Dec.
1979	12.1	-	12.0	12.0	12.1	-	-	-	-	-	12.6	12.0
1980	12.0	-	12.3	11.9	11.9	-	-	-	-	-	12.2	11.9
1981	12.7	-	12.3	12.1	11.8	-	-	-	-	-	12.7	12.0
1982	12.2	-	11.9	11.8	12.0	-	-	-	-	-	12.9	12.3
1983	12.0	-	12.2	12.2	11.7	-	-	-	-	-	12.4	11.9
1984	12.4	-	12.2	11.8	11.6	-	-	-	-	-	12.3	12.0
1985	12.5	-	12.4	12.1	11.5	-	-	-	-	-	12.2	12.3
1986	12.1	-	12.3	12.1	12.1	-	-	-	-	-	12.2	12.2
1987	12.2	-	12.2	12.4	12.0	-	-	-	-	-	12.6	11.9
1988	12.2	-	12.4	12.2	12.0	-	-	-	-	-	12.4	12.1
1989	12.3	-	12.3	12.3	11.9	-	-	-	-	-	12.6	12.2
1990	12.3	-	12.7	12.2	11.9	-	-	-	-	-	12.9	11.9
1991	12.3	-	12.5	12.3	12.0	-	-	-	-	-	12.4	12.3
1992	12.2	-	12.3	12.4	12.0	-	-	-	-	-	12.6	12.3
1993	12.4	-	12.2	11.9	11.8	-	-	-	-	-	12.5	12.3
Ave.	12.3	-	12.3	12.1	11.9	-	-	-	-	-	12.5	12.1

Table A.4.5 **Half-monthly Average of Effective Rainfall for Paddy**

(Unit : mm/day)

	Jan.		Feb.		Mar.		Apr.		May		Jun.		Jul.		Aug.		Sept.		Oct.		Nov.		Dec.	
	I	II	I	II	I	II	I	II	I	II	I	II	I	II	I	II	I	II	I	II	I	II	I	II
1979	1.1	2.4	3.1	0.6	0.0	5.6	11.2	5.4	7.1	5.7	7.3	9.6	4.1	2.3	0.0	-	-	-	-	-	1.2	0.0	0.4	5.6
1980	0.0	0.0	0.5	3.9	4.7	2.4	8.5	5.5	12.3	9.4	4.2	8.6	3.3	2.2	5.1	-	-	-	-	-	1.2	1.9	3.9	2.2
1981	0.0	0.0	0.0	0.6	1.1	10.0	4.7	8.9	12.2	7.3	4.4	5.1	9.5	11.0	2.9	-	-	-	-	-	5.0	1.3	0.5	0.0
1982	3.1	0.0	0.0	0.0	7.1	3.5	5.1	7.7	5.6	6.8	10.8	2.1	0.0	0.0	0.8	-	-	-	-	-	0.0	0.0	1.4	2.6
1983	4.8	5.4	1.1	0.7	2.8	2.6	4.4	0.9	7.7	8.1	12.6	14.8	0.9	10.0	1.4	-	-	-	-	-	1.3	0.0	0.9	0.0
1984	2.9	1.3	6.7	0.0	5.1	2.2	5.7	5.9	17.3	11.1	10.5	3.8	8.2	10.2	3.6	-	-	-	-	-	3.6	4.7	1.0	0.7
1985	0.0	2.5	4.7	2.5	5.2	3.0	2.9	1.2	6.8	17.6	6.0	0.0	5.6	6.4	2.0	-	-	-	-	-	6.6	2.6	2.1	3.8
1986	5.3	2.7	0.3	0.0	5.8	3.3	1.8	7.5	6.8	5.2	9.7	7.2	6.9	0.8	0.0	-	-	-	-	-	5.2	5.8	6.4	2.9
1987	1.0	3.9	0.0	0.0	6.1	8.3	10.2	7.7	5.3	7.4	5.0	10.7	2.1	5.0	0.0	-	-	-	-	-	7.2	3.5	3.4	2.8
1988	0.0	2.0	2.3	2.7	3.0	4.7	0.9	13.8	12.7	2.4	5.3	7.2	8.7	5.7	7.5	-	-	-	-	-	5.9	3.8	2.5	0.8
1989	1.8	3.4	1.2	0.0	0.0	0.5	11.8	0.0	7.9	1.3	3.5	6.8	10.5	9.6	1.4	-	-	-	-	-	1.0	1.4	1.3	1.0
1990	0.0	0.4	3.9	3.4	2.3	3.4	4.9	8.3	15.7	9.0	3.5	3.2	4.0	5.6	1.8	-	-	-	-	-	0.0	0.0	0.0	0.0
1991	2.5	1.9	0.0	0.7	2.9	2.0	3.3	7.7	6.2	10.5	4.6	2.1	2.4	0.0	1.8	-	-	-	-	-	0.0	3.2	2.6	3.3
1992	0.7	3.3	4.3	7.6	2.9	6.5	0.5	9.1	9.2	3.4	15.6	5.7	7.4	4.5	2.4	-	-	-	-	-	1.7	2.1	1.3	2.4
1993	2.1	0.7	0.0	0.0	2.6	0.3	7.6	7.8	4.1	3.0	12.2	6.8	8.1	2.0	0.0	-	-	-	-	-	0.6	1.4	2.0	0.6
Average	1.7	1.9	1.8	1.6	3.7	3.7	5.2	6.6	9.3	7.3	7.7	6.0	5.5	5.2	2.2	-	-	-	-	-	2.8	2.3	2.1	1.6

Table A.4.6 Half-monthly Average of Effective Rainfall for Palawija

(Unit : mm/day)

	Jan.		Feb.		Mar.		Apr.		May		Jun.		Jul.		Aug.		Sept.		Oct.		Nov.		Dec.	
	I	II	I	II	I	II	I	II	I	II	I	II	I	II	I	II	I	II	I	II	I	II	I	II
1979	-	-	-	-	-	-	-	-	-	-	-	-	-	-	0.0	0.0	1.2	1.6	0.7	0.5	0.3	0.1	-	-
1980	-	-	-	-	-	-	-	-	-	-	-	-	-	-	0.6	2.4	0.0	0.0	0.7	0.5	0.8	0.2	-	-
1981	-	-	-	-	-	-	-	-	-	-	-	-	-	-	0.2	0.6	0.7	0.9	2.8	2.0	1.6	0.4	-	-
1982	-	-	-	-	-	-	-	-	-	-	-	-	-	-	0.0	0.2	0.0	0.0	0.0	0.0	0.0	0.0	-	-
1983	-	-	-	-	-	-	-	-	-	-	-	-	-	-	0.3	1.1	0.5	0.7	1.7	1.2	0.4	0.1	-	-
1984	-	-	-	-	-	-	-	-	-	-	-	-	-	-	0.2	0.7	2.6	3.6	0.1	0.1	2.4	0.6	-	-
1985	-	-	-	-	-	-	-	-	-	-	-	-	-	-	0.4	1.6	0.1	0.2	3.3	2.3	1.9	0.5	-	-
1986	-	-	-	-	-	-	-	-	-	-	-	-	-	-	0.0	0.0	0.6	0.8	3.9	2.8	2.5	0.6	-	-
1987	-	-	-	-	-	-	-	-	-	-	-	-	-	-	0.0	0.0	0.9	1.2	0.4	0.3	2.6	0.6	-	-
1988	-	-	-	-	-	-	-	-	-	-	-	-	-	-	0.8	2.9	2.8	3.9	3.1	2.3	2.4	0.6	-	-
1989	-	-	-	-	-	-	-	-	-	-	-	-	-	-	0.5	2.0	2.2	3.0	2.1	1.5	0.6	0.1	-	-
1990	-	-	-	-	-	-	-	-	-	-	-	-	-	-	0.1	0.4	0.5	0.7	2.4	1.7	0.1	0.0	-	-
1991	-	-	-	-	-	-	-	-	-	-	-	-	-	-	0.1	0.4	0.0	0.0	0.5	0.4	0.8	0.2	-	-
1992	-	-	-	-	-	-	-	-	-	-	-	-	-	-	0.2	0.6	0.6	0.9	2.3	1.6	0.8	0.2	-	-
1993	-	-	-	-	-	-	-	-	-	-	-	-	-	-	0.1	0.2	0.0	0.1	0.8	0.6	0.5	0.1	-	-
Average	-	-	-	-	-	-	-	-	-	-	-	-	-	-	0.2	0.9	0.8	1.2	1.6	1.2	1.2	0.3	-	-

Table A.4.7 Sample Calculation of Net Field Water Requirement for Paddy

Items	Unit	Jan. I	Jan. II	Feb. I	Feb. II	Mar. I	Mar. II	Apr. I	Apr. II	May I	May II	Jun. I	Jun. II
		Dry Season Paddy										Wet Season Paddy	
A. Land Preparation Requirement													
1. Land Preparation Intensity		1/6					1/6	1/3 1/6	1/6 1/3 1/6	1/6 1/3	1/6		
Total		1/6					1/6	1/2	2/3	1/2	1/6		
2. Land Preparation Requirement	(mm/day/area)	12.08	12.08	12.08	12.08	12.02	12.02	11.98	11.98	12.08	12.08	11.57	11.57
	(mm/day)	2.01					2.00	5.99	7.99	6.04	2.01		
3. Water Layer Replacement Intencity		1/3	1/3	2/3	1/3	1/3					1/3	1/3	2/3
4. Water Layer Replacement Requirement	(mm/day/area)	3.33	3.33	3.33	3.33	3.33					3.33	3.33	3.33
	(mm/day)	1.11	1.11	2.22	1.11	1.11					1.11	1.11	2.22
5. Total Requirement for Land Preparation	(mm/day)	3.12	1.11	2.22	1.11	1.11	2.00	5.99	7.99	6.04	3.12	1.11	2.22
B. Crop Water Requirement													
1. Crop Intensity		1/3 1/3 1/6	1/3 1/3 1/3	1/3 1/3 1/3	1/6 1/3 1/3	1/6 1/3	1/6		1/6	1/3 1/6	1/3 1/3 1/6	1/3 1/3 1/3	1/3 1/3 1/3
Total		5/6	1	1	5/6	1/2	1/6		1/6	1/2	5/6	1	1
2. Crop Coefficient		1.10 1.10 1.10	1.09 1.10 1.10	1.05 1.09 1.10	1.00 1.05 1.09	1.00 1.05	1.00		1.10	1.10 1.10	1.10 1.10 1.10	1.08 1.10 1.10	1.03 1.08 1.10
Weighted average		1.10	1.10	1.08	1.06	1.03	1.00		1.10	1.10	1.10	1.09	1.07
3. Potential ET	(mm/day/area)	4.21	4.21	4.21	4.21	4.12	4.12	4.06	4.06	4.21	4.21	3.46	3.46
4. Consumptive Use	(mm/day/area)	4.63	4.62	4.55	4.45	4.26	4.12		4.47	4.63	4.63	3.78	3.70
5. Percolation	(mm/day/area)	2.00	2.00	2.00	2.00	2.00	2.00		2.00	2.00	2.00	2.00	2.00
6. Crop Water Requirement	(mm/day/area)	6.63	6.62	6.55	6.45	6.26	6.12		6.47	6.63	6.63	5.78	5.70
7. Crop Water Requirement	(mm/day)	5.53	6.62	6.55	5.37	3.13	1.02		1.06	3.32	5.53	5.78	5.70
C. Effective Rainfall	(mm/day/area)	1.12	2.35	3.09	0.62	0.00	5.55	11.20	5.39	7.09	5.65	7.25	9.60
D. Net Field Water Requirement	(mm/day)	7.53	5.38	5.67	5.87	4.24	0.00	0.00	3.68	2.26	3.00	0.00	0.00
	(l/sec/ha)	0.87	0.62	0.66	0.68	0.49	0.00	0.00	0.43	0.26	0.35	0.00	0.00

Items	Unit	Jul. I	Jul. II	Aug. I	Aug. II	Sept. I	Sept. II	Oct. I	Oct. II	Nov. I	Nov. II	Dec. I	Dec. II
A. Land Preparation Requirement													
1. Land Preparation Intensity										1/6	1/3 1/6	1/6 1/3 1/6	1/6 1/3
Total										1/6	1/2	2/3	1/2
2. Land Preparation Requirement	(mm/day/area)	12.02	12.02	12.90	12.90	12.83	12.83	13.24	13.24	12.64	12.64	12.01	12.01
	(mm/day)									2.11	6.32	8.01	6.01
3. Water Layer Replacement Intencity		1/3	1/3										
4. Water Layer Replacement Requirement	(mm/day/area)	3.33	3.33										
	(mm/day)	1.11	1.11										
5. Total Requirement for Land Preparation	(mm/day)	1.11	1.11							2.11	6.32	8.01	6.01
B. Crop Water Requirement													
1. Crop Intensity		1/6 1/3 1/3	1/6 1/3	1/6								1/6	1/3 1/6
Total		5/6	1/2	1/6								1/6	1/2
2. Crop Coefficient		0.95 1.03 1.08	0.95 1.03	0.95								1.10	1.10 1.10
Weighted average		1.03	1.00	0.95								1.10	1.10
3. Potential ET	(mm/day/area)	4.12	4.12	5.36	5.36	5.27	5.27	5.83	5.83	5.00	5.00	4.10	4.10
4. Consumptive Use	(mm/day/area)	4.26	4.13	5.09								4.51	4.51
5. Percolation	(mm/day/area)	2.00	2.00	2.00								2.00	2.00
6. Crop Water Requirement	(mm/day/area)	6.26	6.13	7.09								6.51	6.51
7. Crop Water Requirement	(mm/day)	5.22	3.07	1.18								1.09	3.26
C. Effective Rainfall	(mm/day/area)	4.05	2.25	0.00	0.00	0.00	0.00	0.00	0.00	1.17	0.00	0.43	5.55
D. Net Field Water Requirement	(mm/day)	2.27	1.93	1.18	0.00	0.00	0.00	0.00	0.00	0.93	6.32	8.67	3.71
	(l/sec/ha)	0.26	0.22	0.14	0.00	0.00	0.00	0.00	0.00	0.11	0.73	1.00	0.43

Table A.4.8 **Sample Calculation of Net Field Water Requirement for Palawija**

Items		Jan. I	Jan. II	Feb. I	Feb. II	Mar. I	Mar. II	Apr. I	Apr. II	May I	May II	Jun. I	Jun. II	Jul. I	Jul. II	Aug. I	Aug. II	Sept. I	Sept. II	Oct. I	Oct. II	Nov. I	Nov. II	Dec. I	Dec. II
																				Palawija					
A. Crop Water Requirement																									
1. Crop Intensity																1/6	1/3	1/3	1/3	1/3	1/6				
																	1/6	1/3	1/3	1/3	1/3	1/6			
																		1/6	1/3	1/3	1/3	1/3	1/6		
Total																1/6	1/2	5/6	1	1	5/6	1/2	1/6		
2. Crop Coefficient																0.50	0.75	1.01	1.05	0.96	0.60				
																	0.50	0.75	1.01	1.05	0.96	0.60			
																		0.50	0.75	1.01	1.05	0.96	0.60		
Weighted average																0.50	0.67	0.80	0.94	1.01	0.92	0.84	0.60		
3. Potential ET	(mm/day/area)	4.21	4.21	4.21	4.21	4.12	4.12	4.06	4.06	4.21	4.21	3.46	3.46	4.12	4.12	5.36	5.36	5.27	5.27	5.83	5.83	5.00	5.00	4.10	4.10
4. Consumptive Use	(mm/day/area)															2.68	3.57	4.24	4.94	5.87	5.39	4.20	3.00		
5. Crop Water Requirement	(mm/day)															0.45	1.79	3.53	4.94	5.87	4.49	2.10	0.50		
B. Effective Rainfall (50 %)	(mm/day/area)	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	1.18	1.65	0.71	0.51	0.33	0.08	0.00	0.00
C. Net Field Water Requirement	(mm/day)															0.45	1.79	2.35	3.29	5.15	3.98	1.77	0.42		
	(l/sec/ha)															0.05	0.21	0.27	0.38	0.60	0.46	0.20	0.05		

Table A.4.9 Field Water Requirement for Paddy

(Unit : l/sec/ha)

	Jan.		Feb.		Mar.		Apr.		May		Jun.		Jul.		Aug.		Sept.		Oct.		Nov.		Dec.	
	I	II	I	II	I	II	I	II	I	II	I	II	I	II	I	II	I	II	I	II	I	II	I	II
1979	0.87	0.62	0.66	0.68	0.49	0.00	0.00	0.43	0.26	0.35	0.00	0.00	0.26	0.22	0.14	0.00	0.00	0.00	0.00	0.00	0.11	0.73	1.00	0.43
1980	0.98	0.88	0.94	0.28	0.00	0.09	0.00	0.40	0.00	0.00	0.34	0.00	0.34	0.23	0.00	0.00	0.00	0.00	0.00	0.00	0.10	0.48	0.58	0.80
1981	1.11	1.01	1.03	0.69	0.38	0.00	0.16	0.03	0.00	0.12	0.35	0.39	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.59	0.99	1.07
1982	0.65	0.91	1.00	0.74	0.00	0.00	0.10	0.14	0.42	0.20	0.00	0.66	0.78	0.51	0.04	0.00	0.00	0.00	0.00	0.00	0.25	0.75	0.92	0.81
1983	0.44	0.27	0.94	0.72	0.18	0.06	0.20	0.97	0.14	0.01	0.00	0.00	0.57	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.08	0.72	0.94	1.06
1984	0.72	0.81	0.28	0.78	0.00	0.10	0.03	0.35	0.00	0.00	0.00	0.46	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.17	0.94	0.99
1985	1.07	0.68	0.51	0.49	0.00	0.02	0.36	0.93	0.21	0.00	0.10	0.91	0.02	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.41	0.84	0.67
1986	0.39	0.59	1.03	0.79	0.00	0.00	0.49	0.19	0.29	0.41	0.00	0.10	0.00	0.37	0.13	0.00	0.00	0.00	0.00	0.00	0.00	0.03	0.33	0.76
1987	0.91	0.47	1.11	0.82	0.00	0.00	0.00	0.20	0.45	0.13	0.32	0.00	0.53	0.00	0.14	0.00	0.00	0.00	0.00	0.00	0.00	0.32	0.65	0.74
1988	1.02	0.68	0.76	0.45	0.17	0.00	0.60	0.00	0.00	0.70	0.17	0.07	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.27	0.77	1.00
1989	0.84	0.55	0.98	0.83	0.51	0.31	0.00	1.08	0.15	0.83	0.44	0.18	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.13	0.57	0.93	0.99
1990	1.03	0.89	0.60	0.38	0.28	0.00	0.14	0.12	0.00	0.00	0.44	0.60	0.27	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.25	0.75	1.04	1.06
1991	0.75	0.71	1.05	0.70	0.19	0.14	0.33	0.19	0.35	0.00	0.36	0.76	0.44	0.48	0.00	0.00	0.00	0.00	0.00	0.00	0.24	0.35	0.78	0.74
1992	0.94	0.54	0.58	0.00	0.18	0.00	0.66	0.04	0.01	0.59	0.00	0.28	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.05	0.48	0.93	0.84
1993	0.81	0.87	1.11	0.83	0.21	0.32	0.00	0.13	0.57	0.61	0.00	0.16	0.00	0.22	0.13	0.00	0.00	0.00	0.00	0.00	0.17	0.56	0.85	1.04
Average	0.83	0.70	0.85	0.61	0.15	0.07	0.22	0.34	0.18	0.26	0.18	0.33	0.21	0.13	0.03	0.00	0.00	0.00	0.00	0.00	0.09	0.46	0.82	0.90

Table A.4.10 Field Water Requirement for Palawija

(Unit : l/sec/ha)

	Jan.		Feb.		Mar.		Apr.		May		Jun.		Jul.		Aug.		Sept.		Oct.		Nov.		Dec.	
	I	II	I	II	I	II	I	II	I	II	I	II	I	II	I	II	I	II	I	II	I	II	I	II
1979	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.05	0.21	0.27	0.38	0.60	0.46	0.20	0.05	0.00	0.00
1980	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.44	0.61	0.50	0.39	0.12	0.03	0.00	0.00
1981	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.03	0.11	0.24	0.33	0.29	0.24	0.07	0.02	0.00	0.00
1982	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.04	0.18	0.40	0.56	0.68	0.52	0.25	0.06	0.00	0.00
1983	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.01	0.06	0.38	0.53	0.40	0.32	0.18	0.04	0.00	0.00
1984	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.02	0.09	0.05	0.07	0.63	0.48	0.00	0.00	0.00	0.00
1985	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.39	0.54	0.18	0.15	0.00	0.00	0.00	0.00
1986	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.05	0.18	0.35	0.49	0.08	0.08	0.00	0.00	0.00	0.00
1987	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.05	0.20	0.40	0.56	0.71	0.55	0.00	0.00	0.00	0.00
1988	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.06	0.08	0.20	0.17	0.00	0.00	0.00	0.00
1989	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.16	0.22	0.34	0.27	0.17	0.04	0.00	0.00
1990	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.03	0.13	0.37	0.52	0.36	0.29	0.24	0.06	0.00	0.00
1991	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.03	0.13	0.43	0.60	0.63	0.49	0.13	0.03	0.00	0.00
1992	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.03	0.11	0.31	0.43	0.35	0.28	0.14	0.03	0.00	0.00
1993	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.04	0.16	0.44	0.61	0.67	0.52	0.17	0.04	0.00	0.00
Average	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.03	0.10	0.31	0.43	0.44	0.35	0.11	0.03	0.00	0.00

Table A.4.11 Field Water Requirement (Total)

(Unit : l/sec/ha)

	Jan.		Feb.		Mar.		Apr.		May		Jun.		Jul.		Aug.		Sept.		Oct.		Nov.		Dec.	
	I	II	I	II	I	II	I	II	I	II	I	II	I	II	I	II	I	II	I	II	I	II	I	II
1979	0.87	0.62	0.66	0.68	0.49	0.00	0.00	0.43	0.26	0.35	0.00	0.00	0.26	0.22	0.19	0.21	0.27	0.38	0.60	0.46	0.31	0.78	1.00	0.43
1980	0.98	0.88	0.94	0.28	0.00	0.09	0.00	0.40	0.00	0.00	0.34	0.00	0.34	0.23	0.00	0.00	0.44	0.61	0.50	0.39	0.22	0.51	0.58	0.80
1981	1.11	1.01	1.03	0.69	0.38	0.00	0.16	0.03	0.00	0.12	0.35	0.39	0.00	0.00	0.03	0.11	0.24	0.33	0.29	0.24	0.07	0.60	0.99	1.07
1982	0.65	0.91	1.00	0.74	0.00	0.00	0.10	0.14	0.42	0.20	0.00	0.66	0.78	0.51	0.08	0.18	0.40	0.56	0.68	0.52	0.50	0.81	0.92	0.81
1983	0.44	0.27	0.94	0.72	0.18	0.06	0.20	0.97	0.14	0.01	0.00	0.00	0.57	0.00	0.01	0.06	0.38	0.53	0.40	0.32	0.27	0.76	0.94	1.06
1984	0.72	0.81	0.28	0.78	0.00	0.10	0.03	0.35	0.00	0.00	0.00	0.46	0.00	0.00	0.02	0.09	0.05	0.07	0.63	0.48	0.00	0.17	0.94	0.99
1985	1.07	0.68	0.51	0.49	0.00	0.02	0.36	0.93	0.21	0.00	0.10	0.91	0.02	0.00	0.00	0.00	0.39	0.54	0.18	0.15	0.00	0.41	0.84	0.67
1986	0.39	0.59	1.03	0.79	0.00	0.00	0.49	0.19	0.29	0.41	0.00	0.10	0.00	0.37	0.17	0.18	0.35	0.49	0.08	0.08	0.00	0.03	0.33	0.76
1987	0.91	0.47	1.11	0.82	0.00	0.00	0.00	0.20	0.45	0.13	0.32	0.00	0.53	0.00	0.19	0.20	0.40	0.56	0.71	0.55	0.00	0.32	0.65	0.74
1988	1.02	0.68	0.76	0.45	0.17	0.00	0.60	0.00	0.00	0.70	0.17	0.07	0.00	0.00	0.00	0.00	0.06	0.08	0.20	0.17	0.00	0.27	0.77	1.00
1989	0.84	0.55	0.98	0.83	0.51	0.31	0.00	1.08	0.15	0.83	0.44	0.18	0.00	0.00	0.00	0.00	0.16	0.22	0.34	0.27	0.29	0.61	0.93	0.99
1990	1.03	0.89	0.60	0.38	0.28	0.00	0.14	0.12	0.00	0.00	0.44	0.60	0.27	0.00	0.03	0.13	0.37	0.52	0.36	0.29	0.49	0.80	1.04	1.06
1991	0.75	0.71	1.05	0.70	0.19	0.14	0.33	0.19	0.35	0.00	0.36	0.76	0.44	0.48	0.03	0.13	0.43	0.60	0.63	0.49	0.37	0.38	0.78	0.74
1992	0.94	0.54	0.58	0.00	0.18	0.00	0.66	0.04	0.01	0.59	0.00	0.28	0.00	0.00	0.03	0.11	0.31	0.43	0.35	0.28	0.20	0.52	0.93	0.84
1993	0.81	0.87	1.11	0.83	0.21	0.32	0.00	0.13	0.57	0.61	0.00	0.16	0.00	0.22	0.17	0.16	0.44	0.61	0.67	0.52	0.34	0.60	0.85	1.04
Average	0.83	0.70	0.85	0.61	0.15	0.07	0.22	0.34	0.18	0.26	0.18	0.33	0.21	0.13	0.05	0.10	0.31	0.44	0.43	0.34	0.20	0.49	0.82	0.90

Table A.4.12 Unit Diversion Water Requirement for Paddy

(Unit : l/sec/ha)

	Jan.		Feb.		Mar.		Apr.		May		Jun.		Jul.		Aug.		Sept.		Oct.		Nov.		Dec.	
	I	II	I	II	I	II	I	II	I	II	I	II	I	II	I	II	I	II	I	II	I	II	I	II
1979	1.36	0.97	1.03	1.06	0.77	0.00	0.00	0.67	0.41	0.54	0.00	0.00	0.41	0.35	0.21	0.00	0.00	0.00	0.00	0.00	0.17	1.14	1.57	0.67
1980	1.54	1.37	1.47	0.44	0.00	0.14	0.00	0.62	0.00	0.00	0.54	0.00	0.54	0.36	0.00	0.00	0.00	0.00	0.00	0.00	0.16	0.76	0.91	1.25
1981	1.74	1.58	1.61	1.08	0.60	0.00	0.25	0.05	0.00	0.19	0.55	0.61	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.92	1.54	1.67
1982	1.02	1.43	1.57	1.16	0.00	0.00	0.15	0.21	0.66	0.31	0.00	1.04	1.22	0.80	0.06	0.00	0.00	0.00	0.00	0.00	0.39	1.17	1.43	1.27
1983	0.69	0.42	1.47	1.12	0.29	0.10	0.31	1.52	0.22	0.01	0.00	0.00	0.90	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.13	1.12	1.47	1.65
1984	1.12	1.26	0.43	1.22	0.00	0.16	0.04	0.55	0.00	0.00	0.00	0.72	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.26	1.47	1.55
1985	1.67	1.07	0.79	0.77	0.00	0.03	0.56	1.45	0.33	0.00	0.16	1.42	0.03	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.64	1.31	1.05
1986	0.61	0.92	1.60	1.23	0.00	0.00	0.77	0.30	0.46	0.64	0.00	0.16	0.00	0.57	0.20	0.00	0.00	0.00	0.00	0.00	0.00	0.05	0.52	1.19
1987	1.41	0.74	1.73	1.29	0.00	0.00	0.00	0.31	0.71	0.21	0.50	0.00	0.83	0.00	0.21	0.00	0.00	0.00	0.00	0.00	0.00	0.50	1.02	1.16
1988	1.59	1.06	1.19	0.70	0.27	0.00	0.94	0.00	0.00	1.10	0.26	0.10	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.43	1.21	1.56
1989	1.31	0.86	1.53	1.30	0.80	0.48	0.00	1.69	0.23	1.30	0.69	0.28	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.20	0.89	1.45	1.55
1990	1.61	1.39	0.93	0.59	0.44	0.00	0.23	0.18	0.00	0.00	0.69	0.94	0.42	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.39	1.17	1.63	1.66
1991	1.18	1.11	1.65	1.10	0.30	0.22	0.51	0.30	0.54	0.00	0.56	1.18	0.69	0.74	0.00	0.00	0.00	0.00	0.00	0.00	0.37	0.54	1.22	1.15
1992	1.48	0.85	0.90	0.00	0.27	0.00	1.02	0.07	0.01	0.93	0.00	0.44	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.08	0.75	1.45	1.31
1993	1.26	1.36	1.73	1.29	0.33	0.51	0.00	0.21	0.89	0.95	0.00	0.25	0.00	0.35	0.20	0.00	0.00	0.00	0.00	0.00	0.27	0.87	1.34	1.63
Average	1.30	1.10	1.33	0.95	0.24	0.12	0.34	0.53	0.29	0.40	0.28	0.51	0.33	0.20	0.05	0.00	0.00	0.00	0.00	0.00	0.14	0.72	1.28	1.40

Table A.4.13 Unit Diversion Water Requirement for Palawija

(Unit : l/sec/ha)

	Jan.		Feb.		Mar.		Apr.		May		Jun.		Jul.		Aug.		Sept.		Oct.		Nov.		Dec.	
	I	II	I	II	I	II	I	II	I	II	I	II	I	II	I	II	I	II	I	II	I	II	I	II
1979	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.10	0.41	0.53	0.75	1.17	0.90	0.40	0.10	0.00	0.00
1980	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.86	1.20	0.99	0.76	0.24	0.06	0.00	0.00
1981	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.05	0.22	0.47	0.65	0.57	0.47	0.14	0.03	0.00	0.00
1982	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.09	0.34	0.78	1.10	1.33	1.02	0.50	0.12	0.00	0.00
1983	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.02	0.11	0.74	1.03	0.79	0.62	0.36	0.09	0.00	0.00
1984	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.04	0.17	0.10	0.14	1.23	0.94	0.00	0.00	0.00	0.00
1985	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.76	1.06	0.34	0.30	0.00	0.00	0.00	0.00
1986	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.09	0.36	0.68	0.95	0.15	0.16	0.00	0.00	0.00	0.00
1987	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.10	0.40	0.78	1.09	1.40	1.08	0.00	0.00	0.00	0.00
1988	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.12	0.17	0.40	0.34	0.00	0.00	0.00	0.00
1989	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.31	0.43	0.67	0.54	0.33	0.08	0.00	0.00
1990	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.06	0.25	0.72	1.01	0.71	0.57	0.48	0.11	0.00	0.00
1991	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.06	0.25	0.83	1.17	1.24	0.96	0.26	0.06	0.00	0.00
1992	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.05	0.22	0.60	0.84	0.68	0.55	0.28	0.07	0.00	0.00
1993	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.08	0.32	0.86	1.20	1.31	1.01	0.34	0.08	0.00	0.00
Average	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.05	0.20	0.61	0.85	0.87	0.68	0.22	0.05	0.00	0.00

Table A.4.14 Feature of Alternatives and Cost Comparison

	Alternative - I	Alternative - II	Alternative - III
Total Irrigation Area	8,600 ha	5,880 ha	7,000 ha
Intake System	Divert from reservoir	Intake from weir for 5,880 ha	Intake from weir for 5,880 ha and pumping for 1,120 ha (440 ha from upstream Gilirang river and 880 ha from main canals)
Main facilities	1. Dam 2. Headrace, connected with main canal 3. Main canal	1. Dam 2. Weir 3. Main canal	1. Dam 2. Weir 3. Main canal 4. Pumping facilities
Dimension of main facilities			
1. Dam			
Type of Dam	Rockfill	Rockfill	Rockfill
Maximum storage capacity	132 MCM	125 MCM	132 MCM
Dam height	44.5 m	40.0 m	44.5 m
Crest length	230 m	220 m	230 m
Crest elevetion	EL 56.5m	EL 56.0 m	EL 56.5 m
Catchment area	169 km2	169 km2	169 km2
2. Weir			
Type of Weir	-	Fixed type	Fixed type
Material of weir	-	Concrete	Concrete
Intake water level	-	EL 18.00 m	EL 18.00 m
Width of scoring sluice	-	2.0 m x 2 Nos. 3.0 m x 2 Nos.	2.0 m x 2 Nos. 3.0 m x 2 Nos.
3. Headrace			
Design canal discharge	15.0 m3/sec	-	-
Length	6 km	-	-
Type of canal	Unlined canal	-	-
4. Main canal			
Total length of canal	74 km	47.5 km	47.5 km
Design discharge of left main	4.4 m3/sec	3.5 m3/sec	3.7 m3/sec
Design discharge of right main	10.6 m3/sec	6.7 m3/sec	7.6 m3/sec
Type of canal	Unlined canal	Unlined Canal	Unlined Canal
5. Pumping facilities			
Nos of pump unit			
(I) 1.4 m3/min., D=4 inch, 10HP	-	-	6
(II) 3.5 m3/min., D=6 inch, 18 HP	-	-	22
(III) 6.0 m3/min., D=8 inch, 27 HP	-	-	13
Preliminary cost estimate (Rp. Million)			
Item			
1 Dam	35,901	34,331	35,901
2 Weir	0	8,984	8,984
3 Headrace	41,854	0	0
4 Irrigation & drainage system	84,758	36,077	37,480
5 Pumping facilities	0	0	234
6 Others	89,306	77,919	78,088
Grand total	251,819	157,311	160,687

Table A.4 .15 Potential Annual Power Generation (1979 - 1993)

		1979		1980		1981		1982		1983		1984		1985		1986	
		H (m)	Q (m3/sec)	H (m)	Q (m3/sec)	H (m)	Q (m3/sec)	H (m)	Q (m3/sec)	H (m)	Q (m3/sec)	H (m)	Q (m3/sec)	H (m)	Q (m3/sec)	H (m)	Q (m3/sec)
Jan.	I	45.62	10.58	45.44	11.75	44.20	13.91	47.45	8.13	34.38	5.55	44.53	9.14	46.94	13.08	46.95	4.31
	II	43.76	7.82	43.43	10.79	41.16	12.69	46.15	11.42	34.38	2.57	42.59	9.77	44.73	8.54	46.54	7.52
Feb.	I	42.03	5.92	41.02	11.45	37.79	12.90	43.84	12.53	34.56	11.78	40.49	0.56	42.94	4.24	45.10	12.72
	II	42.16	8.55	38.45	3.05	34.38	8.82	41.14	9.37	34.38	4.60	42.33	9.26	43.32	6.13	42.57	9.92
Mar.	I	40.49	6.38	38.22	0.13	34.38	5.08	39.20	0.21	36.31	2.55	40.84	0.11	42.29	0.25	40.64	0.11
	II	38.89	0.03	38.38	1.22	34.38	0.00	39.27	0.04	35.69	1.13	40.95	1.31	42.29	0.44	40.77	0.00
Apr.	I	39.15	0.00	38.26	0.00	35.02	0.11	39.52	1.44	35.44	2.53	40.82	0.62	42.29	4.79	41.30	4.25
	II	41.23	2.63	40.38	3.60	45.14	0.02	39.26	0.50	35.00	6.27	40.70	1.76	41.18	11.60	41.61	0.13
May	I	42.41	0.18	40.57	0.00	46.44	0.00	40.18	4.39	37.75	0.10	42.05	0.00	38.37	0.25	43.17	0.20
	II	45.07	2.01	42.82	0.00	48.15	0.08	39.84	0.14	39.84	0.00	45.48	0.00	40.16	0.00	45.98	0.45
Jun.	I	46.00	0.00	45.77	0.35	49.85	3.76	42.47	0.00	45.95	0.00	49.58	0.00	43.25	0.07	48.08	0.00
	II	49.28	0.00	47.56	0.00	49.67	1.19	45.14	3.22	47.98	0.00	50.84	4.71	46.20	9.01	50.84	0.07
Jul.	I	50.84	0.18	50.21	3.92	50.61	0.00	47.00	9.69	50.84	5.86	50.68	0.00	45.75	0.01	50.84	0.00
	II	50.84	1.66	49.94	3.15	50.84	0.00	45.36	6.56	50.64	0.00	50.84	0.00	47.70	0.00	50.84	0.25
Aug.	I	50.84	1.38	49.52	0.00	50.84	0.01	43.99	1.14	50.84	0.00	50.84	0.01	48.45	0.00	50.84	0.10
	II	50.84	0.78	50.15	0.00	50.84	0.79	43.65	1.22	50.84	0.01	50.84	0.02	49.44	0.00	50.84	0.05
Sept.	I	50.83	0.07	50.38	2.24	50.72	0.06	43.27	2.25	50.84	1.95	50.84	0.01	50.55	1.69	50.84	1.55
	II	50.84	1.89	50.10	3.11	50.84	1.57	42.66	2.95	50.61	2.65	50.84	0.22	50.43	2.61	50.72	1.73
Oct.	I	50.65	3.00	49.67	2.64	50.70	0.61	41.88	3.47	50.30	2.13	50.84	2.28	50.14	0.88	50.69	0.02
	II	50.30	2.46	49.27	2.11	50.84	0.06	40.96	2.78	50.02	0.88	50.78	2.29	50.07	0.50	50.81	0.02
Nov.	I	49.94	2.47	48.90	2.20	50.84	0.02	40.12	4.63	50.13	1.79	50.55	0.39	50.13	0.00	50.84	0.00
	II	49.65	9.44	48.53	6.44	50.84	5.91	38.87	9.56	50.00	9.29	50.46	2.28	50.21	5.29	50.84	0.02
Dec.	I	48.41	12.42	47.53	7.50	50.66	12.22	36.46	10.64	48.84	11.88	50.22	11.68	49.61	10.36	50.84	4.23
	II	46.53	5.59	46.25	10.11	49.37	13.18	34.38	10.34	47.05	13.22	48.87	12.37	48.29	8.40	50.45	9.48

Annual Power Generation ('000kWH)

1979	1980	1981	1982	1983	1984	1985	1986
2,330	2,008	2,647	646	2,045	2,368	2,008	2,598

No Operation period (off-irrigation season) (days)

1979	1980	1981	1982	1983	1984	1985	1986
45	90	60	15	75	75	60	45

		1987		1988		1989		1990		1991		1992		1993	
		H (m)	Q (m3/sec)	H (m)	Q (m3/sec)	H (m)	Q (m3/sec)	H (m)	Q (m3/sec)	H (m)	Q (m3/sec)	H (m)	Q (m3/sec)	H (m)	Q (m3/sec)
Jan.	I	49.35	11.26	46.92	12.51	48.33	9.17	46.39	12.88	45.01	6.21	44.21	11.79	46.69	10.24
	II	47.82	5.61	44.77	7.30	47.49	5.99	43.99	11.17	45.46	9.07	41.72	6.95	44.81	10.94
Feb.	I	47.07	13.69	43.93	9.47	46.91	11.72	41.39	7.02	43.50	13.22	40.00	5.81	42.40	13.90
	II	44.67	10.35	42.02	5.03	45.06	10.17	40.13	0.26	40.53	8.94	39.62	0.00	39.06	10.49
Mar.	I	42.84	0.00	41.46	1.94	43.45	6.32	45.41	3.53	38.62	2.66	40.28	2.24	36.63	3.03
	II	43.09	0.00	41.32	0.00	42.25	3.84	44.83	0.06	37.98	2.05	39.92	0.00	35.74	4.30
Apr.	I	43.51	0.00	41.91	6.24	41.47	0.00	44.94	1.25	37.46	4.16	40.26	5.41	34.50	0.39
	II	47.30	0.14	41.37	0.00	41.88	12.83	45.11	1.02	36.52	0.13	40.86	0.03	34.44	0.61
May	I	49.91	0.31	44.84	0.00	39.28	1.29	45.27	0.00	42.16	0.99	42.67	0.00	35.41	0.39
	II	50.84	0.09	49.04	6.01	39.56	9.63	46.96	0.00	43.98	0.00	44.84	6.64	41.26	2.06
Jun.	I	50.84	0.22	49.24	0.12	37.58	5.09	47.82	3.14	47.38	0.25	43.96	0.00	44.31	0.00
	II	50.84	0.00	50.84	0.04	36.76	0.12	48.35	5.63	50.28	8.60	46.40	2.32	46.86	0.11
Jul.	I	50.84	5.75	50.84	0.00	41.99	0.00	48.29	3.28	49.58	0.30	46.65	0.00	49.29	0.00
	II	50.53	0.00	50.84	0.00	48.93	0.00	47.90	0.00	50.84	4.93	48.30	0.00	50.84	2.25
Aug.	I	50.66	2.13	50.84	0.00	50.84	0.00	49.39	0.01	50.63	0.40	50.07	0.01	50.79	2.09
	II	50.43	1.15	50.84	0.00	50.84	0.00	49.76	0.66	50.57	0.40	50.84	0.72	50.55	0.98
Sept.	I	50.27	2.13	50.84	0.02	50.84	0.04	49.71	1.58	50.61	2.18	50.76	0.34	50.41	2.36
	II	49.98	1.52	50.84	0.02	50.84	0.05	49.59	1.56	50.35	2.99	50.84	0.11	50.08	3.16
Oct.	I	50.11	3.43	50.84	0.05	50.84	0.09	49.67	1.18	49.96	3.23	50.84	0.77	49.62	3.35
	II	49.67	2.55	50.84	0.04	50.84	0.07	49.70	0.07	49.50	2.44	50.84	1.46	49.12	2.48
Nov.	I	49.33	0.00	50.84	0.08	50.84	2.10	50.53	4.03	49.13	3.94	50.67	1.68	48.73	3.19
	II	49.68	3.82	50.84	3.44	50.73	7.46	50.18	9.29	48.52	4.84	50.45	6.01	48.25	7.35
Dec.	I	49.31	8.01	50.55	9.59	49.95	11.64	49.19	12.99	47.73	9.89	49.89	11.70	47.09	10.57
	II	48.33	9.14	49.54	11.45	48.47	12.45	47.35	13.06	46.07	9.30	48.37	10.40	45.32	12.85

Annual Power Generation ('000kWH)

1987	1988	1989	1990	1991	1992	1993
3,144	2,756	2,111	2,583	2,281	2,117	2,068

No Operation period (off-irrigation season) (days)

1987	1988	1989	1990	1991	1992	1993
75	90	60	45	15	75	30

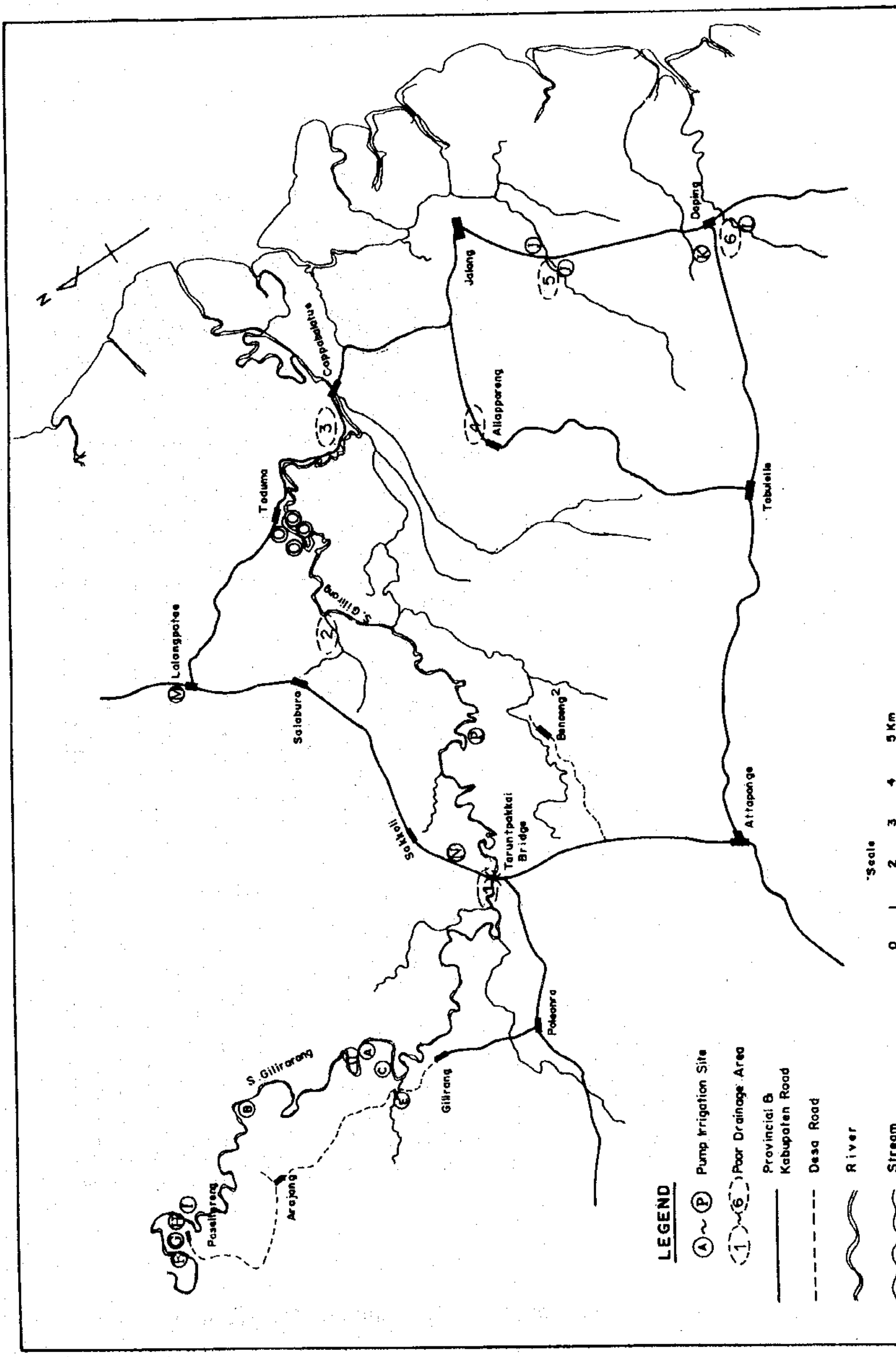

Figure A.4.1 Existing Pumping Irrigation Site and Poor Drainage Area