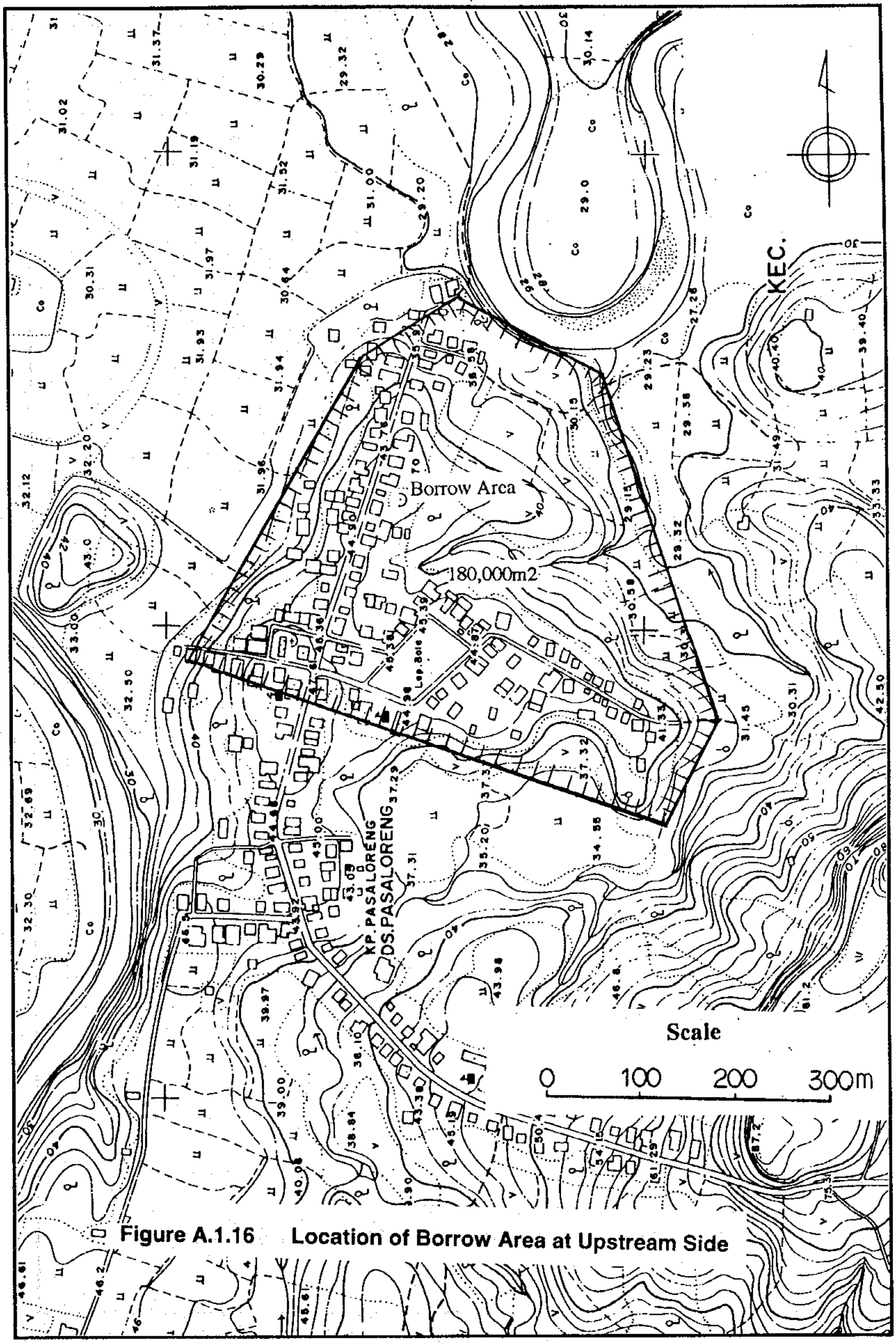
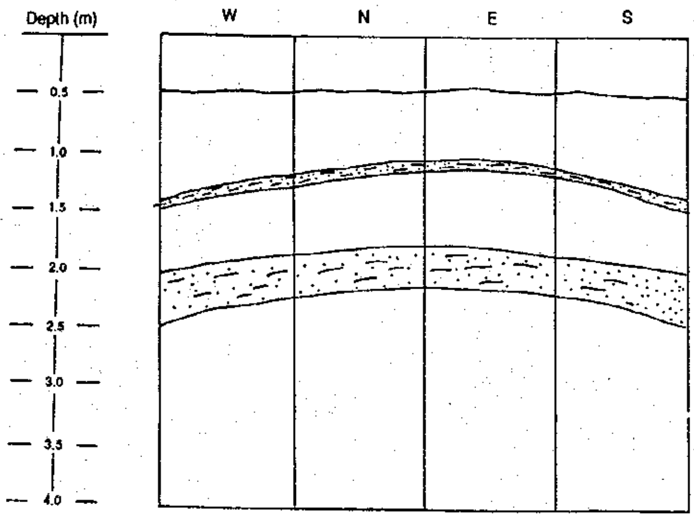
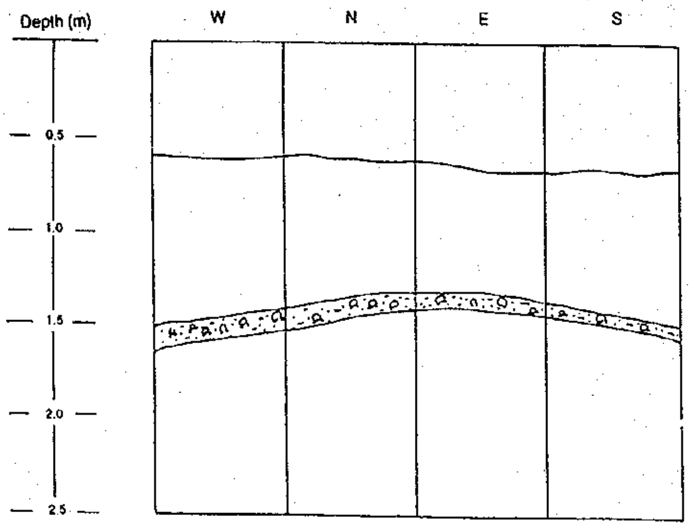


Figure A.1.16 Location of Borrow Area at Upstream Side



Sampling	Color	Description
	Dark grey	Humic soil
	Yellow grey	Sandysilt qu = 20-50kg/cm2
	Yellow grey	Sandyclay qu ≤ 50kg/cm2
	Dark grey	Sandyclay
	Grey	Sandyclay

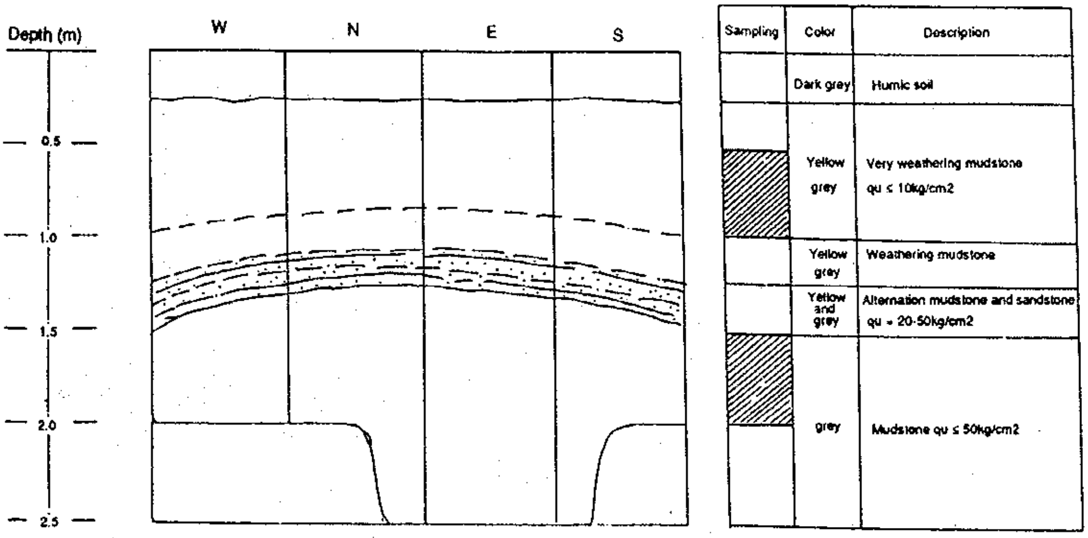
Location : Left bank site
Bore No. : TP-3
Start : March/29/94
Completed : April/2/94



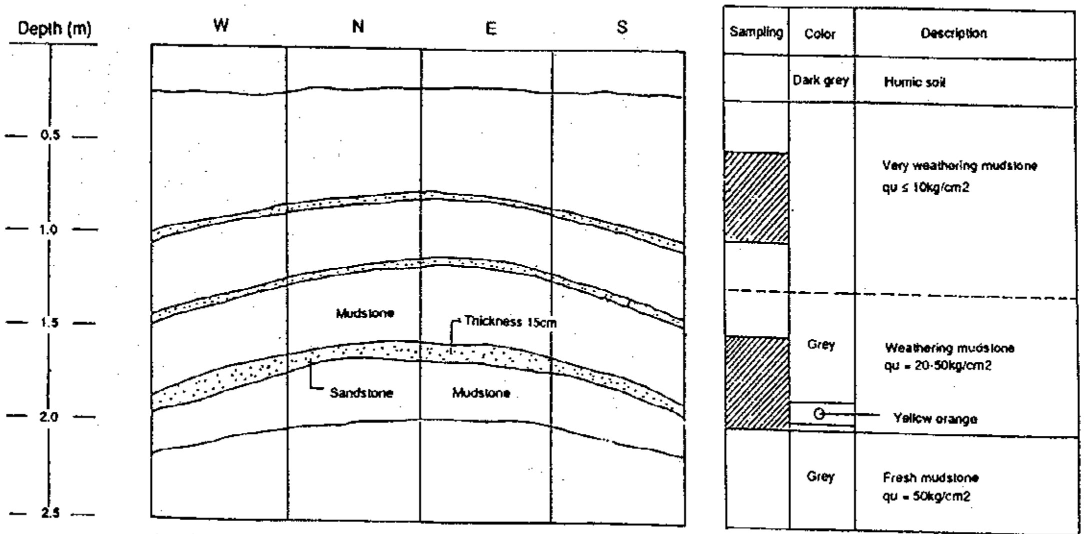
Sampling	Color	Description
	Dark grey	Humic soil
	Dark orange and dark grey	Weathering conglomerate qu = 20-50kg/cm2

Location : Left bank site
Bore No. : TP-4
Start : March/29/94
Completed : April/2/94

Figure A.1.17 Shaft Geological Profile at Borrow Area (1/2)



Location : Right bank site
Bore No. : TP-1
Start : March/26/94
Completed : March/28/94



Location : Right bank site
Bore No. : TP-2
Start : March/26/94
Completed : March/28/94

Figure A.1.17 Shaft Geological Profile at Borrow Area (2/2)

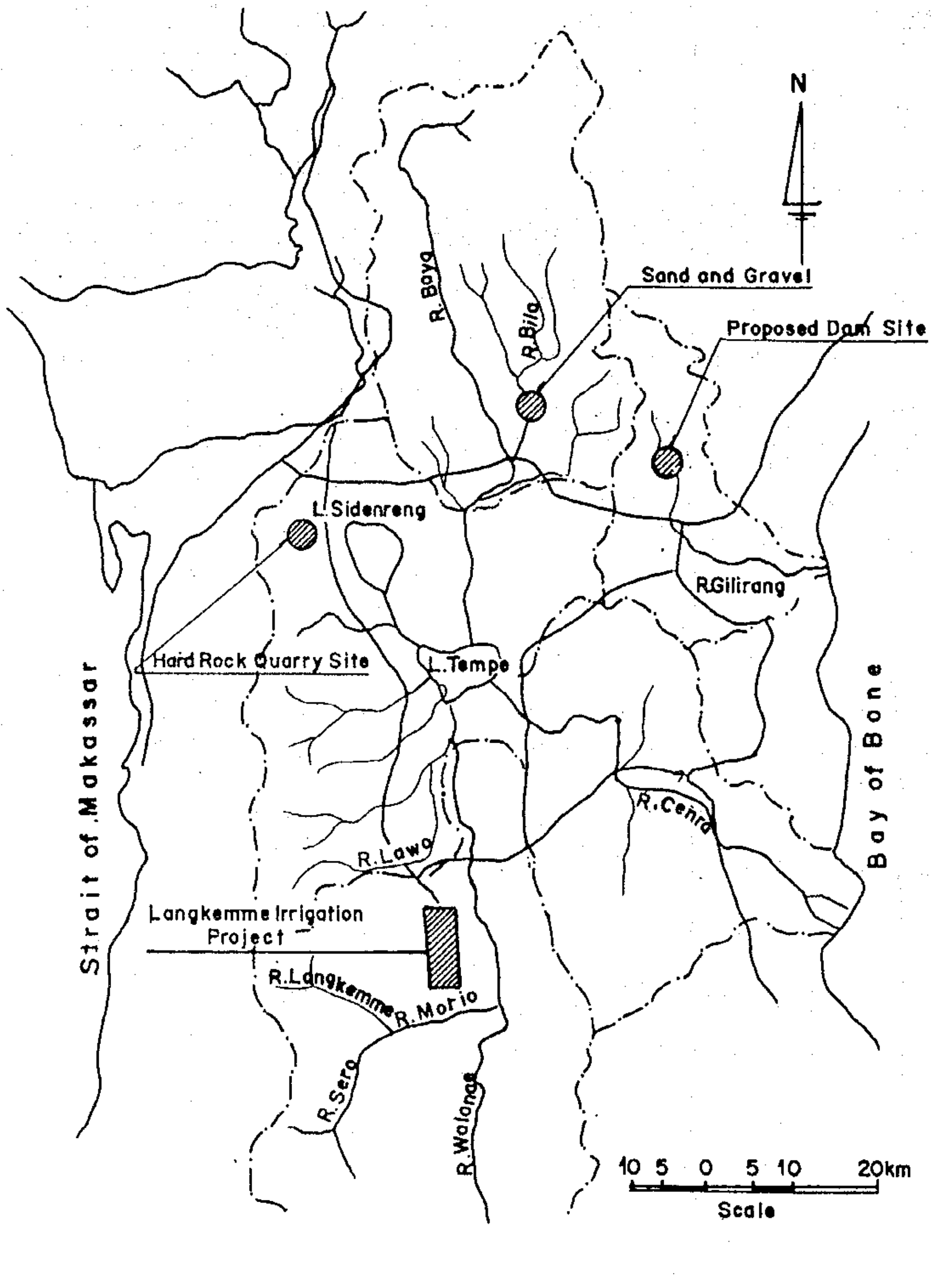


Figure A.1.18 Location of Quarry Site of Purchase Material

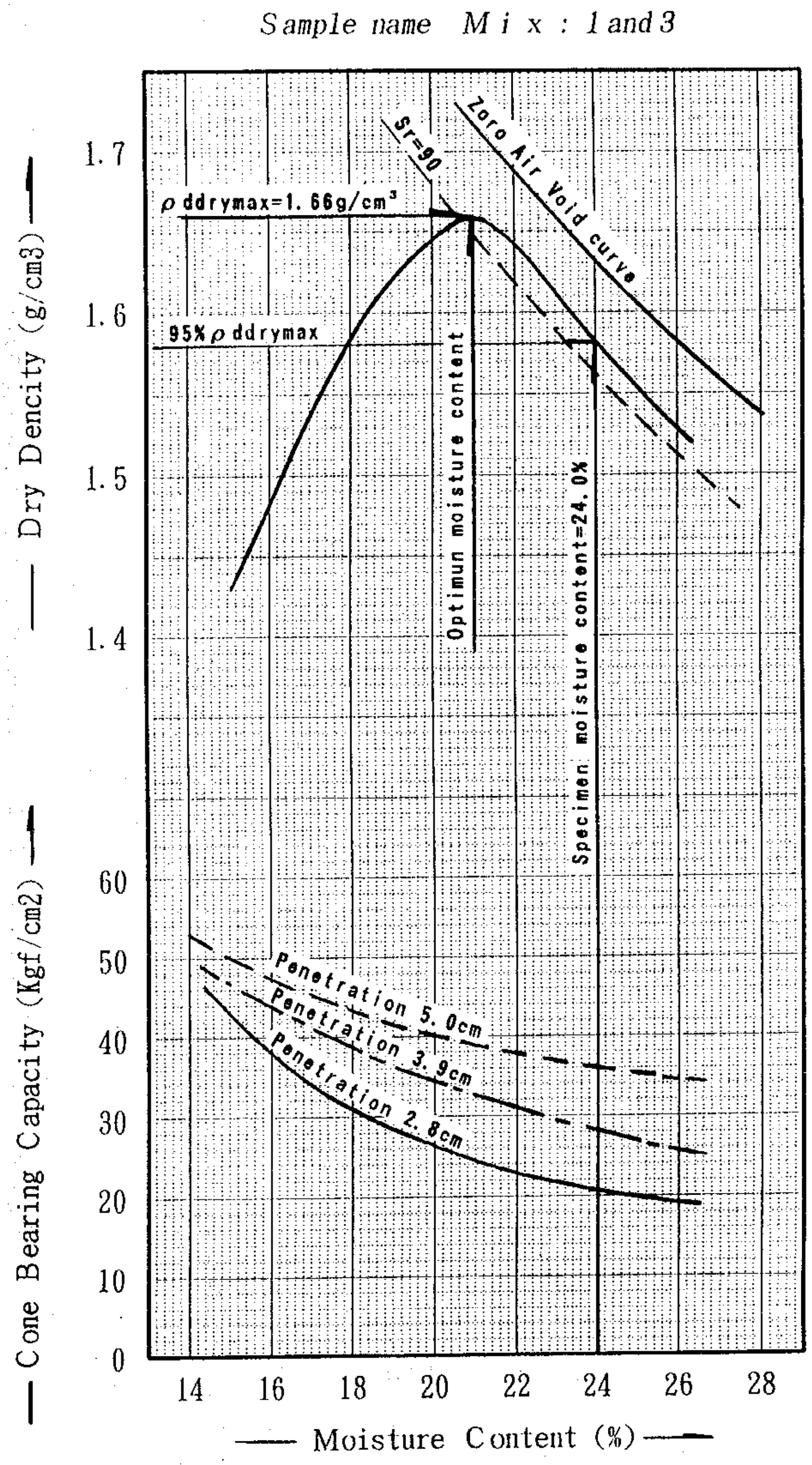


Figure A.1.19 Result of Compaction Test (Highly Weathered Mudstone)

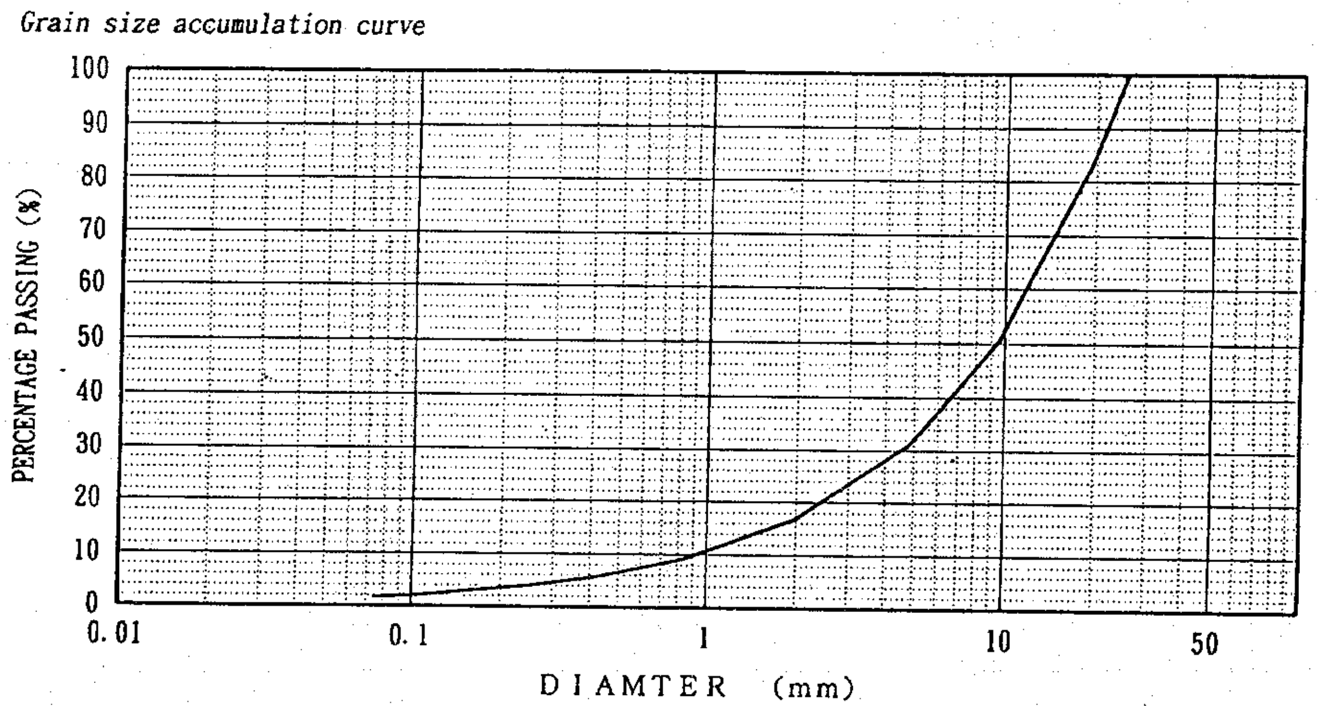
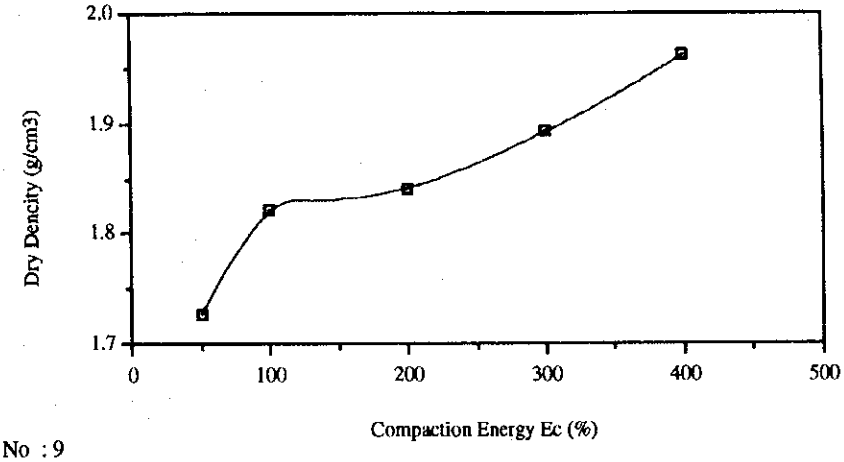
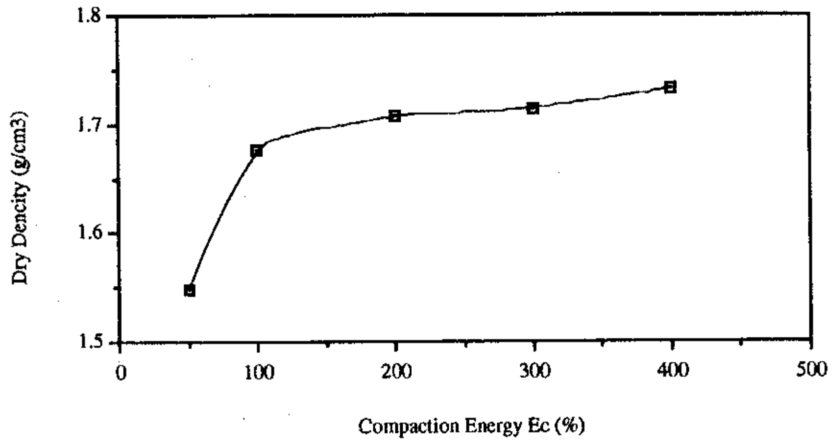


Figure A.1.20 Grain Size Curve of Rock Materials for Mechanical Test

Sample No : 8
Location : Outcrop
Soil : Conglomerate



Sample No : 9
Location : Outcrop
Soil : Sand Stone



Sample No : Mix 8 & 9
Location : Outcrop
Soil : Conglomerate + Sand Stone

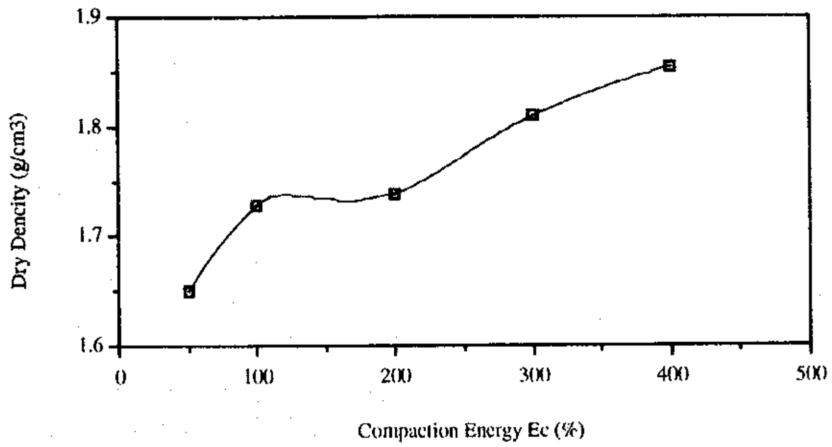


Figure A.1.21 Result of Compaction Test

ANNEX 2

METEOROLOGY, HYDROLOGY AND WATER BALANCE STUDY

ANNEX 2 METEOROLOGY, HYDROLOGY AND WATER BALANCE STUDY

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ANNEX 2 METEOROLOGY, HYDROLOGY AND WATER BALANCE STUDY

1. INTRODUCTION

This Annex describes the results of meteoro-hydrological study and water balance study which were carried out in the Feasibility Study on the Gilirang Irrigation Project. In the hydrological study, investigation and studies on long-term runoff are carried out to clarify the potentiality of surface water resources for the Project. The water balance study, which is the basic component for the delineation of the project scale, is executed in accordance with the results of the hydrological study. The main objective of the study in this Annex covers the following items:

(1) to collect and analyze the meteoro-hydrological data in and around the study area,

(2) to check the availability of the meteoro-hydrological data for the study,

(3) to prepare monthly meteorological data for continuous 15 years period from 1979 through 1993 in the study area,

(4) to prepare half-monthly rainfall data for continuous 15 years period from 1979 through 1993 in the study area,

(5) to prepare half-monthly runoff discharge data of the Gilirang river for continuous 15 years period from 1979 through 1993 at the proposed dam site and weir site,

(6) to estimate the probable floods of the Gilirang river at the proposed dam site and weir site,

(7) to clarify the possible cropping intensity according to the results of the water balance study.

2. METEOROLOGY

2.1 Climate

The study area is located in south Sulawesi, at just 4° of south latitude and about 120° of east longitude. It belongs to the equator climatic zone. Its climate is characterized by tropical monsoons, having distinct dry and wet seasons. The east monsoon predominant from may to June and which brings the rainiest month in May.

There are three (3) meteorological stations and fifteen (15) rainfall stations in and around the study area. The location of the stations are mentioned in Figure A.2.1.

2.2 Rainfall

Though the rainfall data are available at 15 gauging stations mentioned above, their observation periods are very different and there exist not many stations which have enough length of data for the hydrological analysis. The observation period of daily rainfall at these stations are illustrated in Figure A.2.2 and the annual rainfall pattern of each station is mentioned in Figure A.2.3.

Sakkoli rainfall station is located at the center of the study area and the data in this station seems to be most reliable comparing with the data in the other stations, though there are some interruptions of observation period. Lack of data can be compensated with the data observed at Peneki rainfall station. Half-monthly average rainfalls at the Sakkoli rainfall station are shown in Table A.2.1. There is a marked difference in the yearly rainfalls, though there is a clear trend in annual rainfall pattern. The average yearly rainfall is about 2,200 mm in the study area. The yearly rainfalls widely vary from year to year ranging between approximately 1,500 mm to 2,900 mm.

2.3 Other Meteorological Conditions

Among three meteorological stations, Bontouse station has only a few available data comparing with Sengkang station, which has certain length of reliable data. Sakkoli station has just started its observation from last April, and has no available data, at all. Therefore, the meteorological data observed at Sengkang station are employed for meteorological analyses in this study.

2.3.1 Temperature

The mean monthly temperature varies from 26 °C to 28 °C with a little seasonal variation. From October to November is the hottest season, having a mean maximum temperature over 32 °C. The coolest season is from June to August which has a mean minimum temperature less than 23 °C. The average daily fluctuation is only about 4 °C through a year. The mean monthly temperature observed at Sengkang meteorological station is shown in Table A.2.2 and Figure A.2.4.

2.3.2 Relative Humidity

The mean monthly relative humidity is from about 72 % to 84 %. The highest relative humidity occurs in May and June, and the lowest occurs in October. The mean monthly relative humidity observed at Sengkang meteorological station is shown in Table A.2.3 and Figure A.2.5.

2.3.3 Sunshine Duration

Annual average sunshine percentage is 53% at Sengkang. The mean monthly sunshine duration varies widely from 5.2 hr/day at minimum in December to 8.1 hr/day at maximum in September. The mean monthly sunshine duration observed at Sengkang meteorological station is shown in Table A.2.4 and Figure A.2.6.

2.3.4 Wind Velocity

The monthly average of wind velocity is generally low in a range of 1.0 m/sec to 1.5 m/sec. The mean monthly wind velocity observed at Sengkang meteorological station is shown in Table A.2.5 and Figure A.2.7.

2.3.5 Evaporation

The annual evaporation from the Class A-pan varies from 1,840 mm (5.0 mm/day) in 1978 to 2,290 mm (6.3 mm/day) in 1987, and the average of annual evaporation is 2,080 mm (5.7 mm/day). The mean monthly evaporation is in the range of 4.6 mm/day in June to 7.1 mm/day in October. The mean monthly evaporation observed at Sengkang meteorological station is shown in Table A.2.6 and Figure A.2.8.

3. LONG-TERM RUNOFF OF GILIRANG RIVER

3.1 River Basin

The Gilirang river is the only surface water source in the study area. The river originates from the mountainous zone between Bila and Awo river basins. The river meanders from the north to the south near the junction of provincial road and turns its trend to the east, and then flows directly into the Bay of Bone traversing through the alluvial plain in the lower Gilirang river basin.

Its total reach is about 100 km. The catchment area of the river are 518 km^2, 230 km^2 and 169 km^2 at the river mouth, Gilirang gauging station, and at the proposed dam site respectively. The proposed dam site is located at about 11 km upstream of the Gilirang gauging station. In the upstream of the station, the river basin is classified into three categories by its coverage. About 70 % of the basin is mountainous area with forest, 20 % of that is hilly area with no trees and the remaining is grass land. The average gradient in the first 10 km from river origin is very steep and it gradually becomes gentle toward the river mouth. The average gradient between proposed dam site and the river mouth is very gentle and it is about 1/2,500 (See Figure A.2.9).

3.2 Available Data

There are three water-level gauging stations in the Gilirang river as shown Figure A.2.1. However, Arajang station just started its observation in February 1994 and has no available data yet. Among two remaining stations, the observation at Tarumpakkae station started by P3SA in 1975 and terminated in 1978 in accordance with the installation of the Gilirang station in 1978, and there exist only short period data. The Gilirang gauging station has continuous data from 1979 up to date by P3SA and BPPDSA, though there are some interruptions. The observation at Gilirang station is carried out by an automatic level recorder, though that at Tarumpakkae was carried out by manual reading of three times a day.

Considering above conditions, the observed data at the Gilirang gauging station is the most reliable data for hydrological analysis. The Gilirang station also has certain amount of discharge data with water level data which were measured directly using current meter. These data are very useful to know the relation between the water level and the discharge in the river by formulation of so called rating curve. There are sixty-seven (67) available data of the direct measurements, and the estimated rating curve using these data is shown in Figure A.2.10.

The nearest river basin to the Gilirang river basin, which has certain period of reliable daily flow records, is the Bila river basin. There are two gauging stations of Bila (Upstream) and Bila (Downstream). The Bila (Upstream) station has continuous data from 1978 to date and the Bila (Downstream) has continuous data from 1973 to 1988 with some interruptions at both stations.

3.3 Half-monthly Discharge

The daily discharge at the Gilirang gauging station is estimated using the rating curve which is estimated using the direct measurement data. The half monthly discharge for 15 years (from 1979 through 1993) is calculated using these data. The interrupted periods of the data are filled by estimated discharge using the relationships of discharge between the Gilirang gauging station and Bila (Upstream) or Bila (Downstream) gauging stations. Regression formulas among these stations are shown in Table A.2.7. The half monthly discharge at Gilirang station is shown in Table A.2.8.

The estimated half-monthly discharge at the Gilirang gauging station is converted into the discharge at proposed dam site and the discharge from residual basin between proposed dam site and proposed weir site using the ratio of catchment areas. Estimated half-monthly discharge at the proposed dam site and inflow discharge into the Gilirang river from the residual basin are shown in Table A.2.9 and A.2.10 respectively. The discharge varies with seasons and also drastically fluctuates year by year. Average half-monthly discharge at the proposed dam site for 15 years (1979 - 1993) is mentioned in Figure A.2.11. Average annual runoff-coefficient is about 55 % assuming annual rainfall of the Gilirang river basin is same as that at Sakkoli rainfall station.

4. FLOOD ANALYSIS

4.1 Procedure of Analysis

The purpose of the flood analysis is to determine the design flood discharge for the design of various structures such as spillway, diversion tunnel, etc. As there is not enough record of floods which can be used for the estimate of flood discharge for longer return periods, synthetic unit hydrograph method is employed to estimate the probable flood discharge and pattern of the flood in various return periods in this study.

4.2 Design Rainfall

4.2.1 Probable Daily Rainfall

The rainfall data at Sakkoli rainfall station is used for the estimate of probable daily rainfalls in the Gilirang river basin. The probable daily rainfalls are estimated in various methods for different return periods in Table A.2.11, and values estimated by Gumbel method are applied for the flood analysis as follows :

	Return Period (Year)								
	5	10	20	50	100	200	500	1,000	PMP
Probable Daily Rainfall (mm/day)	165	193	221	256	282	309	343	370	504

4.2.2 Design Storm Pattern

The Thiessen method is not applicable to this study to estimate average basin rainfall due to the lack of rain gauge network. Therefore, the average basin rainfall in the catchment area is estimated from the point rainfall at Sakkoli station. The applied conversion ratio from the point rainfall into the average basin rainfall in the catchment area is estimated from the curves which were recommended by US National Weather Service as shown in Figure A.2.12. The conversion ratio of 0.92 is used for the estimate of the basin rainfall in the catchment area.

Storms in South Sulawesi, as in most of Indonesia, tend to be relatively short duration thunderstorms. Since the data on rainfall pattern in South Sulawesi for the storms are not available, the rainfall intensity-duration curve at Jakarta is used with duration time of 6 hours for this study as follows:

$$Rt = \frac{Rd \times 8.65}{6\,(t + 2.65)}$$

where, Rt : Rainfall intensity of the design storm (mm/hr)
Rd: Total rainfall depth of the design storm (mm)
t : Duration time (6 hours)

The estimated storm pattern and intensity is shown in Figure A.2.13.

4.3 Unit Hydrograph

4.3.1 Concentration Time

Concentration time is defined as flood travel time and which can be obtained from time lag between the beginning of rise of direct flow and its peak. Some recorded hydrographs of direct flow are available at Gilirang gauging station, and the concentration times obtained from these hydrographs are tabulated below :

Date of Occurrence of Peak	Peak Discharge (m3/sec)	Lag Time (hr)
Jul. 21, 1992	116	10
Jul. 23, 1992	166	9
Jun. 18, 1993	126	5
Jun. 26, 1993	207	9
Jun. 2, 1994	265	11
Jun. 10, 1994	220	11
Average	-	9

Based on the above observation, 9 hours is adopted as the concentration time at proposed weir site and 8 hours is adopted as that at proposed dam site assuming the flood velocity between the proposed dam site and weir site is about 11 km/hr.

4.3.2 Rainfall-runoff Model

As there is not enough hourly rainfall data corresponding with flood records, construction of the unit hydrograph from actual rainfall-runoff relationship is impossible. Therefore the synthetic unit hydrograph method developed by Nakayasu is applied.

The Nakayasu's unit hydrograph is described as follows:

(a) Flood peak

$$Qp = \frac{AR}{3.6} \cdot \frac{1}{(0.3\ Ti + T3)}$$

(b) Rising curve ($0 \leq t \leq Ti$)

$$Q/Qp = (T/T1)^{2.4}$$

(c) Recession curve

i) $0.3 \leq Q/Qp \leq 1$

$$Q/Qp = 0.3^{(T-T1)/T3}$$

ii) $0.09 \leq Q/Qp \leq 0.3$

$$Q/Qp = 0.3^{(T-T1+0.5\ T3)/1.5\ T3}$$

iii) $Q/Qp < 0.09$

$$Q/Qp = 0.3^{(T-T1+1.5\ T3)/2.0\ T3}$$

where,

Q	:	Discharge of unit hydrograph at time T (m^3/sec)
Qp	:	Peak discharge (m^3/sec)
A	:	Catchment area (km^2)
Ro	:	Unit rainfall in unit time (mm)
T1	:	Rising time from beginning to peak (hour)
T2	:	Recession time from peak to 0.3 Qp (hour)

T1 and T3 are determined using the following empirical equations developed by Nakayasu.

$$T1 = TG + 0.8\ Tr$$
$$T3 = 0.47 \cdot (A \cdot L)^{0.25}$$
$$TG = 0.4 + 0.058 \cdot L \quad (\text{if } L > 15 \text{ km})$$
$$TG = 0.21 \cdot L^{0.7} \quad (\text{if } L \leq 15 \text{ km})$$

where,

TG	:	Basin lag time (hour)
Tr	:	Duration of unit rainfall (hour)
L	:	Length of river course (km)

Figure A.2.14 and A.2.15 show the unit hydrograph of the Gilirang river at the proposed dam site and at the proposed weir site for a storm with duration of 1 hour and intensity of 1 mm/hour respectively.

4.4 Probable Flood for Reservoir and Weir Plan

4.4.1 Probable Flood for Paselloreng Dam

Probable floods with various return periods and probable maximum flood for the Paselloreng dam are estimated using unit hydrograph method, under the assumption that the runoff coefficient during intense rainfall is 0.70 and base flow is 4 m^3/sec. The result is shown in Figure A.2.16 and peak flood discharges are summarized as follows:

Probable Peak Flood at Paselloreng Dam

(Unit : m^3/sec)

	Return Period (Year)						
	10	25	50	100	200	1,000	PMF
Probable Peak Flood	677	802	896	987	1,081	1,293	1,760

4.4.2 Probable Flood for Gilirang Weir

Probable floods with various return periods and probable maximum flood for the Gilirang weir before the dam construction are estimated using unit hydrograph method as shown in Figure A.2.17, under the assumption that the runoff coefficient during intense rainfall is 0.70 and base flow is 5 m^3/sec.

After the dam construction, the probable floods which pass through the reservoir will be regulated because of storage effect of the reservoir. For example, the peak discharge of the probable maximum flood which flow into the reservoir becomes less than half when the flood spillout from the spillway as shown in Figure A.2.18. Therefore, the probable floods at the weir site after dam construction are different with those before the dam construction. Figure A.2.19 shows the probable flood discharge with 1,000-year return period at weir site before and after the dam construction. The peak flood discharge is reduced to less than half after the dam construction. The peak flood discharge at the weir site before and after the dam construction is summarized bellow:

Probable Peak Flood at Gilirang Weir

(Unit : m³/sec)

Probable Peak Flood	Return Period (Year)						
	10	25	50	100	200	1,000	PMF
Before Dam Construction	836	991	1,107	1,219	1,335	1,598	2,175
After Dam Construction	362	444	508	570	637	792	1,152

In Figure A.2.20, specific discharge of design flood at the Pasellorеng dam and the Gilirang weir for various return period are plotted with Creager 100 and 30 flood lines, as well as design floods of dam or weirs in Bila and Langkeme Irrigation Project in South Sulawesi.

5. SEDIMENT ANALYSIS

Sediment load is one of the major factors to define the reservoir dimension, such as dead storage capacity. For the estimate of sediment transport at proposed dam site of Paselloreng Dam, sediment load is divided into two (2) components of bed load and suspended load including wash load.

The bed load is assumed to be 20% of suspended load, which is a commonly applied value in Indonesia unless relevant data are available. The suspended load is usually estimated using the actual relation between suspended load discharge and water discharge at relevant river. There are five (5) actual measurements of suspended load at Gilirang gauging station as shown in Table A.2.12. The results of the measurement are plotted on Figure A.2.21 and an equation which indicates the relation between the water discharge and the suspended load discharge is obtained as follows;

$$Qs = 5.012\ Q^{0.906}$$

where, Qs : Suspended sediment discharge (ton/day), and
Q : Water discharge (m³/sec)

In order to calculate the average total sediment inflow, the year of 1983 is selected as the representative average year from 1980 to 1993. The suspended load and bed load are estimated using above relations and applying daily discharge data in 1983.

Bed load	54.9 m³/km²/year
Suspended load	11.0 m³/km²/year
Total	65.9 m³/km²/year

However, estimated value above is rather small comparing with the estimate of 1,000 m³/km²/year for Kalola dam of Bila Irrigation Project which is the nearest project to the Gilirang Irrigation Project. The following matters should be considered to get the final estimate of the sediment load at proposed dam site.

1) The base rock of the Gilirang river is Tertiary Pliocene sedimentary rock which is same as that of Kalola river.

2) Data number is not sufficient and no sample data with large discharge is included, though the suspended load with large discharge easily affect the amount of total sediment load.

3) According to the geological investigation by the geologist, there is not much difference of geological condition in the upstream area of both river.

Considering the matters above, the total sediment load at proposed dam site is estimated as 1,000 $m^3/km^2/year$ adding some safety allowance. Therefore, dead storage capacity and dead water level of the Paselloreng dam is assumed to be 17,000,000 m^3 and EL. 34.0 m, respectively.

6. WATER QUALITY

6.1 Water Quality of Gilirang River

Water samplings were carried out at the proposed dam site and at the Gilirang gauging station on 7th April, 1994 for the clarification of the water quality of Gilirang river water. The samples were sent to the Maros Soil Research Station for analyses. The results of the water quality analyses for both sites are shown in Table A.2.13 and which show that the water in the Gilirang river seems to be harmless for irrigation purpose.

6.2 Variation of Salinity in Gilirang River

The salinity of the water in the Gilirang river was examined on October 19, 1994 as shown in Figure A.2.22. The salinity in the lower part is very high and the river water between river mouth and about 17 km upstream of the river mouth is useless for the irrigation purpose due to its high salinity. This is because that the river water is affected by sea water especially in the dry season when the river discharge is very small (0.3 m^3/sec at Gilirang station), as the gradient of the Gilirang river is very gentle .

7. WATER BALANCE STUDY FOR ALTERNATIVES

7.1 Basic Concept for Water Balance Study

The purpose of the water balance study is to clarify the relationship among the proposed dam scale, irrigable area and cropping intensity for each alternative development plan. The proposed alternative plans are as follows:

(1) Alternative I

- The maximum irrigable area (existing paddy field) of 8,600 ha would be irrigated by the storage water in the reservoir with the maximum storage capacity of 132 MCM.
- Direct diversion from the reservoir would be applied for whole irrigation area of 8,600 ha by facilitating a headrace to connect with irrigation canal system.

(2) Alternative II

- The existing paddy field of 5,880 ha would be irrigated by the storage water in the reservoir with the maximum storage capacity of 132 MCM.
- The intake weir at the downstream of the reservoir would be installed to divert the water to the irrigation area of 5,880 ha. The whole irrigation area would be irrigated by gravity irrigation system.

(3) Alternative III

- The existing paddy field of 7,000 ha would be irrigated by the storage water in the reservoir with maximum storage capacity of 132 MCM.
- Small scale portable pumping units would be provided for scattering 440 ha of the upstream area , and the intake weir at the downstream of the reservoir would be installed to divert the water to the remaining 6,560 ha. 680 ha out of 6,560 ha would be also

irrigated using small scale portable pumping units and the remaining 5,880 ha would be irrigated by gravity irrigation system.

The water balance calculation is carried out with following basic condition.

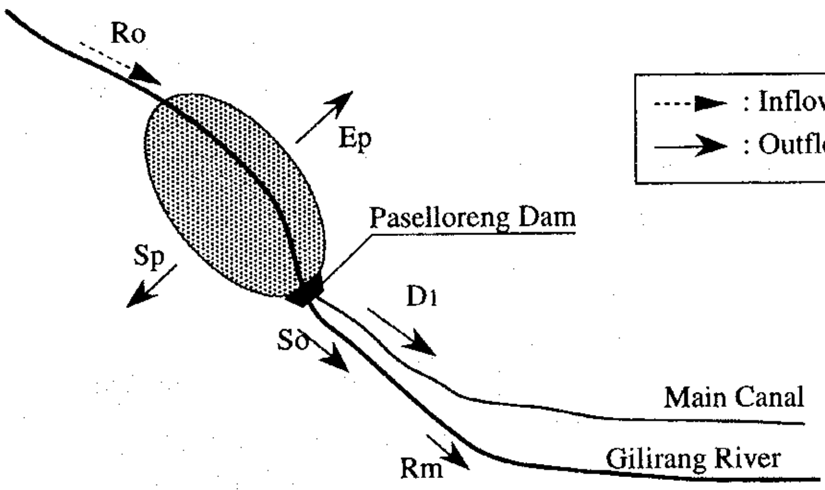
- Cropping pattern of "paddy-palawija-paddy" is examined first as the most preferable cropping pattern (refer to Annex 6).
- If the amount of irrigation water is not enough, the cropping intensity of palawija is reduced and that of dry season paddy is reduced next.
- If there is any excess of irrigation water, the storage capacity of reservoir is reduced.

Calculation of water balance for each alternative is carried out for the period of fifteen (15) years from 1979 to 1993 on the basis of runoff estimated in the hydrological study and estimated irrigation water requirement for each alternative (refer to Annex 4). The calculations are made on half-monthly basis.

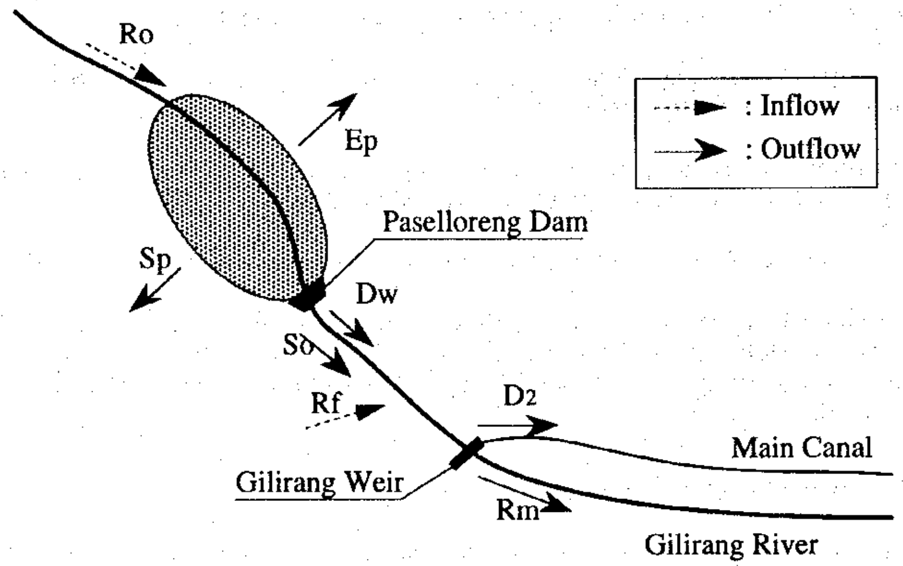
7.2 Basic Components for Water Balance Calculation

The water balance calculation is executed in accordance with some basic components. The components of inflow and outflow in the water balance calculation for each alternative are illustrated below and details of the components are described here in after.

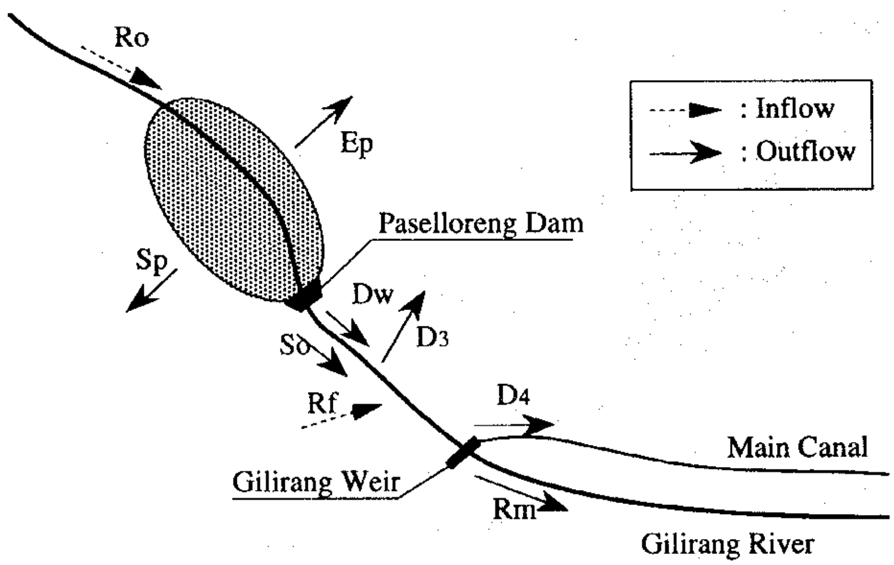
<u>**Alternative I**</u>



Alternative II



Alternative III



7.2.1 Inflow Component

(1) Runoff at proposed dam site (Ro)

Half-monthly average discharge at the proposed dam site from Gilirang river basin for 15 years from 1979 through 1993 is used for the calculation.

(2) Inflow from residual basin between proposed dam site and intake weir site (Rf)

Half-monthly average inflows from residual basin between the proposed dam site and weir site for 15 years from 1979 through 1993 are used for the calculation.

7.2.2 Outflow Component

(1) Evaporation (Ep)

Evaporation from the reservoir surface is estimated by converting observed evaporation of A-pan at Sengkang meteorological station from 1979 through 1993 using 0.7 of converting rate.

(2) Seepage loss from reservoir (Sp)

Seepage loss from the reservoir is assumed to be about 0.03 %/day of gross storage volume.

(3) Irrigation water demand for 8,600 ha (D_1)

Diversion water requirements at proposed dam site for 8,600 ha of gravity irrigation schemes.

(4) Irrigation water demand for 5,880 ha (D_2)

Diversion water requirements at the proposed weir site for 5,880 ha of gravity irrigation schemes.

(5) Irrigation water demand for 440 ha (D_3)

Irrigation water requirements for 440 ha of pump irrigation schemes which locate along the Gilirang river between proposed dam site and weir site.

(6) Irrigation water demand for 6,560 ha (D_4)

Diversion water requirements at the proposed weir site for 5,880 ha of gravity irrigation schemes and 680 ha of pump irrigation schemes.

(7) River maintenance flow (Rm)

River maintenance flow in the downstream reach of the proposed intake weir site is designed to be 0.89 m^3/sec in minimum, which is present average discharge in driest half month (the first half of December) from 1979 through 1993.

(8) Diversion water requirement from dam (Dw)

Diversion water requirements at the proposed dam site from 1979 through 1993 are calculated using the above components and adopted for the further calculation. Dw is same as D1 in case of Alternative 1.

(9) Spillout discharge from dam (So)

Spillout discharge is considered if there is any excess storage which exceed the maximum storage capacity of dam.

7.2.3 Efficiency

(1) Intake efficiency at weir

Intake efficiency at weir is assumed as 90 % including conveyance efficiency between proposed dam site and weir site. Intake loss (10 %) is considered as a part of the river maintenance flow (Rm).

(2) Irrigation efficiency

Total irrigation efficiencies of 64 % for paddy and 51 % for palawija are adopted for both pump and gravity irrigation schemes in the calculation.

7.2.4 Characteristics of Reservoir

Characteristic curves between water level and reservoir capacity or surface area which are mentioned in Figure A.2.23 are used for the calculation. Dead storage capacity is designed to be 17,000,000 m^3.

7.3 Procedure of Water Balance Calculation

The water balance study is made in the following manner.

$$Se = Sb + I - Or - Os$$

where, Se : Reservoir storage volume at the end of the period
Sb : Reservoir storage volume at the beginning of the period
I : Inflow to the reservoir during the period (Ro)
Os : Spillout discharge (So) during the period in case the reservoir storage volume at the end of the period exceeds maximum storage capacity, if any
Or : Outflow from the reservoir during the period including diversion water requirements (Dw), seepage loss (Sp) and evaporation loss (Ep) (where, $Dw = D_1 + (D_2/0.9) + Rm - Rf$)

If the storage volume at the end of the period is less than the dead water storage , the outflow in the period is assumed to be zero for the convenience of calculation. Therefore minimum storage volume is dead storage capacity (17,000,000 m^3), and the period of the dead storage capacity is considered as drought period in the calculation.

The calculation is carried out for 15 years from 1979 through 1993 on half monthly basis. The maximum storage capacity of which frequency of drought time is two (2) times is defined as Gross storage capacity corresponding to each cropping pattern or intensity.

7.4 Results of Water Balance Calculation

The fluctuations of storage volume in the reservoir, inflow and out flow volumes at proposed dam site and spillout volume from the reservoir for each alternative are illustrated in Figure A.2.24, A.2.25 and A.2.26. The possible frequencies of full storage in the reservoir for 15-year period are 9/15 (Alternative I), 12/15 (Alternative II) and 11/15 (Alternative III) .

The results of the water balance calculation are summarized below:

	Irrigable Area and Cropping Intensity			Max Storage
	Wet Season Paddy	Palawija	Dry Season Paddy	Capacity of Dam
Alternative I	8,600 ha	0 ha	7,400 ha	132 MCM
	(100 %)	(0%)	(86 %)	
Alternative II	5,880 ha	5,880 ha	5,880 ha	125 MCM
	(100 %)	(100 %)	(100 %)	
Alternative III	7,000 ha	2,000 ha	7,000 ha	132 MCM
	(100 %)	(29 %)	(100 %)	

8. OVERALL WATER BALANCE IN GILIRANG RIVER BASIN

8.1 Definition

Overall water balance study in the Gilirang river basin is carried out to presume the discharge condition in the Gilirang river due to project implementation. This study is indispensable not only for the estimate of available water in the Gilirang river in future but also for the environmental assessment study in and around the project area. The water balance calculation under condition of with and without project is executed for Alternative 3. Discharge condition of the Gilirang river is presumed at proposed weir site and river mouth.

(1) Calculation period and interval

Overall water balance calculation is carried out for 15 years period from 1979 to 1993 on the basis of the result of water balance calculation for Alternative 3. The calculation is made on half-monthly basis.

(2) Runoff caused by rainfall

Rainfall in the residual river basin in the lower reach of the proposed weir site is represented with the rainfall observed at Sakkoli station which locates almost center of the area. Runoff into the Gilirang river from residual river basin is estimated as follows:

Effective runoff = 0.55 x (Rainfall - Effective Rainfall) x Residual river basin

Time lag between rainfall and runoff into the river is assumed to be half month.

(3) Return flow from seepage of reservoir

50 % of the seepage from reservoir is assumed to be return flow into the Gilirang river. Time lag between seepage and return flow into the river is assumed to be 1 month.

(4) Return flow from irrigation water

50 % of irrigation loss (conveyance loss and application loss) and 50 % of seepage loss from paddy field are assumed to be return flow into the Gilirang river. Time lag between seepage and return flow into the river is assumed to be 1 month.

8.2 Result of Overall Water Balance Study

Through the water balance calculation, variations of discharge in the Gilirang river are presumed. The variation of discharge at proposed weir site under condition with and without project is shown in Figure A.2.27 and that at river mouth of the Gilirang river is shown in Figure A.2.28. Figure A.2.27 shows that the discharge at lower reach of the proposed weir site is decreased in most seasons after implementation of the project, except for the driest seasons. The discharge in the dry season becomes rather constant after the project implementation, though there are much fluctuations under without project condition. Figure A.2.28 shows the tendency that the discharge in the dry season is increased in spite of decrease in the rainy season after the project implementation.

From the above considerations, it is obvious that the river discharge in the Gilirang river will become more constant and steady than the present discharge condition especially in the dry season, and the frequency of large discharges will be decreased after the project implementation. This tendency will be appropriate for the fish culture in the lower reach of the project area, especially for the fish farmers who are going to introduce the shrimp cultivation.

Table A.2.1 Half-monthly Rainfall at Sakkoli Rainfall Station

(Unit : mm/Half month)

	Jan.		Feb.		Mar.		Apr.		May		Jun.		Jul.		Aug.		Sept.		Oct.		Nov.		Dec.		Total
	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	21	47	67	10	0	146	343	101	182	114	156	225	76	50	0	0	140	0	6	50	27	0	8	146	1,915
1980	14	7	11	72	117	57	253	112	315	297	89	183	66	45	171	88	3	0	36	23	26	44	121	48	2,198
1981	0	0	2	12	28	217	93	207	327	303	92	165	181	443	58	5	0	83	96	150	109	27	19	4	2,621
1982	81	2	6	10	175	74	95	253	166	141	239	109	8	0	17	0	0	0	0	0	0	3	27	59	1,465
1983	191	179	27	19	55	71	82	32	231	225	293	327	26	242	35	74	55	0	65	79	30	0	28	0	2,366
1984	61	29	178	4	124	52	109	127	406	375	234	104	183	249	73	1	277	61	1	4	70	151	20	16	2,909
1985	0	49	115	50	121	87	65	27	136	537	115	0	173	135	38	129	0	15	166	130	133	48	40	116	2,425
1986	135	58	6	2	108	80	34	177	208	115	198	195	232	22	0	0	15	51	132	229	116	124	149	66	2,452
1987	19	84	6	0	222	236	281	204	158	154	96	650	90	189	1	2	0	96	0	26	136	98	69	60	2,877
1988	0	47	46	51	77	95	19	363	272	50	102	146	188	153	215	111	231	125	246	37	132	92	50	23	2,871
1989	39	67	25	0	0	18	227	5	188	25	75	199	203	366	32	192	40	224	47	136	24	28	24	22	2,206
1990	3	11	73	63	43	67	93	188	354	244	96	71	78	115	41	0	16	37	2	200	3	6	0	0	1,804
1991	52	38	6	19	59	59	62	158	139	363	128	40	53	0	41	0	0	0	41	0	0	74	78	65	1,475
1992	17	32	83	146	59	244	14	269	186	77	377	124	138	113	45	15	76	0	118	77	31	40	25	48	2,354
1993	47	14	0	1	53	6	154	149	76	59	453	128	157	47	0	20	0	4	43	18	13	32	37	22	1,533
Average	45	44	43	31	83	101	128	158	223	205	183	178	123	145	51	42	57	46	67	77	57	51	46	46	2,231

Table A.2.2(1/2) Mean Monthly Air Temperature at Sengkang (1/2)

(Unit:°C)

Year		Jan.	Feb.	Mar.	Apr.	May	Jun.	Jul.	Aug.	Sept.	Oct.	Nov.	Dec.
1976	Max.	31.9	32.3	32.3	32.1	30.7	29.0	27.8	29.7	32.0	32.5	32.7	31.4
	Min.	-	24.8	24.7	23.3	22.8	22.4	21.9	21.3	20.5	22.9	23.2	17.1
	Mean	-	28.6	28.5	27.7	26.8	25.7	24.9	25.5	26.3	27.7	28.0	24.3
1977	Max.	31.5	31.9	32.5	31.5	31.2	29.1	28.8	29.6	31.7	33.4	34.7	33.1
	Min.	23.6	23.6	23.5	23.4	23.5	23.2	21.9	21.7	21.6	22.4	23.0	23.9
	Mean	27.6	27.8	28.0	27.5	27.4	26.2	25.4	25.7	26.7	27.9	28.9	28.5
1978	Max.	32.7	37.2	32.8	32.3	31.8	31.0	31.7	30.7	31.4	32.0	32.6	31.8
	Min.	23.5	23.2	24.4	23.5	24.4	23.3	22.9	22.7	22.8	22.7	23.1	23.9
	Mean	28.1	30.2	28.6	27.9	28.1	27.2	27.3	26.7	27.1	27.4	27.9	27.9
1979	Max.	32.1	32.3	32.5	31.9	32.2	29.9	29.4	31.2	31.5	33.3	33.7	33.0
	Min.	23.7	23.5	23.5	23.6	24.0	23.5	21.9	21.8	22.5	21.9	23.5	23.4
	Mean	27.9	27.9	28.0	27.8	28.1	26.7	25.7	26.5	27.0	27.6	28.6	28.2
1980	Max.	33.1	32.6	32.1	32.7	30.5	29.6	30.1	30.2	32.3	34.1	34.5	31.9
	Min.	24.1	23.8	23.9	23.0	24.0	23.6	22.5	22.4	21.3	23.1	23.7	23.6
	Mean	28.6	28.2	28.0	27.9	27.3	26.6	26.3	26.3	26.8	28.6	29.1	27.8
1981	Max.	32.0	32.7	32.0	31.7	30.7	30.8	30.0	30.9	32.0	32.5	32.1	31.0
	Min.	24.2	23.3	22.9	23.8	24.3	23.2	22.7	21.7	22.9	22.7	23.8	23.9
	Mean	28.1	28.0	27.5	27.8	27.5	27.0	26.4	26.3	27.5	27.6	28.0	27.5
1982	Max.	31.6	31.7	31.6	30.4	30.0	28.9	29.6	30.3	31.6	33.1	34.0	33.7
	Min.	23.8	23.1	23.9	24.8	25.6	25.0	23.1	23.8	24.9	25.1	26.9	25.8
	Mean	27.7	27.4	27.8	27.6	27.8	27.0	26.4	27.1	28.3	29.1	30.5	29.8
1983	Max.	31.2	31.7	32.3	32.0	30.1	29.4	28.5	29.4	31.5	32.5	31.5	30.0
	Min.	25.3	25.5	26.4	26.0	25.8	25.3	24.5	24.2	25.3	25.7	25.6	25.0
	Mean	28.3	28.6	29.4	29.0	28.0	27.4	26.5	26.8	28.4	29.1	28.6	27.5
1984	Max.	31.1	31.3	30.9	29.3	30.1	28.4	27.8	29.5	30.3	32.0	32.1	31.0
	Min.	25.2	26.2	25.1	24.0	24.0	23.6	23.0	22.1	23.0	22.3	23.1	22.9
	Mean	28.2	28.8	28.0	26.7	27.1	26.0	25.4	25.8	26.7	27.2	27.6	27.0
1985	Max.	31.2	30.9	30.9	30.4	29.7	29.0	27.9	28.6	30.4	31.1	31.6	31.6
	Min.	23.4	24.1	23.5	22.6	22.1	21.1	21.0	21.5	21.9	22.3	22.7	22.8
	Mean	27.3	27.5	27.2	26.5	25.9	25.1	24.5	25.1	26.2	26.7	27.2	27.2

Table A.2.2(2/2) Mean Monthly Air Temperature at Sengkang (2/2)

(Unit:°C)

Year		Jan.	Feb.	Mar.	Apr.	May	Jun.	Jul.	Aug.	Sept.	Oct.	Nov.	Dec.
1986	Max.	29.9	30.4	30.6	30.5	30.2	29.7	29.1	30.1	31.4	31.1	30.6	31.4
	Min.	22.6	20.7	20.1	20.0	20.6	21.1	22.0	21.7	21.9	22.9	22.9	22.9
	Mean	26.3	25.6	25.4	25.3	25.4	25.4	25.6	25.9	26.7	27.0	26.8	27.2
1987	Max.	31.0	31.4	30.5	30.8	29.6	29.7	30.0	30.7	32.1	32.6	32.6	30.1
	Min.	23.1	23.7	22.6	22.7	23.1	23.1	23.0	23.4	24.0	24.7	24.1	23.8
	Mean	27.1	27.6	26.6	26.8	26.4	26.4	26.5	27.1	28.1	28.7	28.4	27.0
1988	Max.	31.3	29.8	31.2	30.6	30.8	28.5	28.6	28.7	28.9	31.2	30.9	29.1
	Min.	24.1	24.1	24.3	24.2	24.1	23.3	23.0	23.5	24.3	24.2	24.6	24.4
	Mean	27.7	27.0	27.8	27.4	27.5	25.9	25.8	26.1	26.6	27.7	27.8	26.8
1989	Max.	29.7	29.9	30.5	30.6	30.2	30.0	28.4	28.9	30.2	30.9	31.2	30.3
	Min.	24.2	24.5	24.0	24.2	23.9	23.5	23.5	23.4	24.1	24.3	24.5	24.5
	Mean	27.0	27.2	27.3	27.4	27.1	26.8	26.0	26.2	27.2	27.6	27.9	27.4
1990	Max.	30.5	30.5	31.9	31.3	30.5	30.0	29.6	30.0	31.4	32.8	32.7	31.3
	Min.	24.5	23.7	24.3	24.1	24.0	23.9	23.2	23.2	24.7	25.1	24.3	24.1
	Mean	27.5	27.1	28.1	27.7	27.3	27.0	26.4	26.6	28.0	29.0	28.5	27.7
1991	Max.	31.6	31.3	31.8	31.1	29.6	29.8	29.6	30.3	31.8	33.0	33.1	32.1
	Min.	23.9	23.9	23.7	23.7	24.1	23.9	23.3	23.9	23.6	24.7	24.5	24.4
	Mean	27.8	27.6	27.8	27.4	26.9	26.9	26.5	27.1	27.7	28.9	28.8	28.3
1992	Max.	31.2	32.0	32.3	31.9	31.0	30.2	29.8	30.8	31.6	31.9	32.6	31.9
	Min.	24.6	24.2	24.8	25.0	24.3	24.0	23.7	22.5	24.0	24.3	24.1	24.5
	Mean	27.9	28.1	28.6	28.5	27.7	27.1	26.8	26.7	27.8	28.1	28.4	28.2
1993	Max.	31.9	32.5	29.6	30.6	30.4	29.9	28.9	30.6	32.0	33.4	31.4	31.4
	Min.	24.5	24.1	23.3	23.4	24.3	24.1	23.2	22.9	24.2	24.3	24.4	24.2
	Mean	28.2	28.3	26.5	27.0	27.4	27.0	26.1	26.8	28.1	28.9	27.9	27.8
	Max.	31.4	31.8	31.6	31.2	30.5	29.6	29.2	30.0	31.3	32.4	32.5	31.5
Average	Min.	24.0	23.9	23.8	23.6	23.8	23.4	22.8	22.6	23.2	23.6	24.0	23.6
	Mean	27.7	27.8	27.7	27.4	27.2	26.5	26.0	26.3	27.3	28.0	28.2	27.5

Table A.2.3 Mean Monthly Relative Humidity at Sengkang

(Unit: %)

Year	Jan.	Feb.	Mar.	Apr.	May	Jun.	Jul.	Aug.	Sept.	Oct.	Nov.	Dec.
1976	71	66	68	70	73	76	75	68	61	63	69	65
1977	67	65	62	68	69	80	73	71	81	74	81	88
1978	84	80	87	85	87	87	87	84	85	86	84	90
1979	86	91	87	93	86	87	85	77	80	66	77	89
1980	86	90	85	89	88	85	86	84	76	85	91	89
1981	72	86	88	87	88	83	88	81	94	86	73	89
1982	85	88	92	93	92	93	79	76	80	72	73	81
1983	84	82	80	84	88	89	88	85	78	85	85	84
1984	79	83	86	87	85	84	84	77	80	72	77	79
1985	79	82	80	81	84	85	85	79	75	77	81	79
1986	82	78	81	78	82	83	83	80	74	81	82	81
1987	79	77	81	82	83	81	77	71	67	67	77	82
1988	81	81	81	83	84	87	84	84	80	80	80	81
1989	81	77	80	81	85	83	87	82	78	80	79	80
1990	79	82	76	80	83	84	80	79	74	78	81	79
1991	78	80	80	81	82	80	79	76	71	72	72	79
1992	79	79	79	79	84	83	82	79	79	77	77	78
1993	78	73	81	84	88	84	84	79	79	72	77	79
Average	79	80	81	83	84	84	83	78	77	76	79	82

Table A.2.4 **Mean Monthly Sunshine Duration at Sengkang**

(Unit: hrs/day)

Year	Jan.	Feb.	Mar.	Apr.	May	Jun.	Jul.	Aug.	Sept.	Oct.	Nov.	Dec.
1976	6.3	7.3	6.1	7.0	6.1	5.4	6.5	8.4	8.7	6.9	7.0	5.5
1977	4.7	4.2	6.2	6.2	6.5	4.8	6.6	7.4	9.3	9.9	7.7	5.6
1978	5.0	5.2	5.2	5.6	6.8	5.9	6.4	6.3	6.1	8.2	6.5	5.2
1979	5.0	5.4	4.7	6.1	7.4	4.8	7.0	8.8	7.7	7.9	6.3	5.0
1980	4.0	4.6	5.5	5.0	5.7	5.4	7.0	6.6	8.9	7.0	5.9	4.4
1981	5.0	5.0	6.3	5.9	5.9	6.8	5.0	8.2	6.3	8.7	6.2	4.7
1982	5.1	4.8	4.6	5.1	7.0	5.6	7.8	7.6	6.9	6.8	5.4	5.2
1983	4.3	5.4	4.6	5.6	5.0	4.5	4.0	7.8	8.1	7.2	5.9	4.4
1984	5.4	4.8	5.4	5.1	4.4	4.2	6.0	7.6	6.2	8.2	6.6	4.0
1985	6.3	5.9	6.5	6.2	4.1	5.5	5.2	6.9	8.6	7.8	6.0	5.6
1986	5.2	6.3	6.7	6.1	7.5	6.1	5.8	8.5	8.3	6.0	6.4	6.1
1987	5.4	6.5	5.7	7.5	6.5	7.8	8.0	7.9	9.7	9.5	6.7	4.5
1988	5.6	5.3	6.3	6.6	6.5	5.0	5.6	6.3	7.7	7.7	6.2	5.3
1989	5.9	6.7	5.7	6.8	6.8	6.9	5.0	5.3	8.0	7.5	7.2	5.8
1990	5.4	6.0	6.9	6.9	6.1	6.6	6.6	7.3	8.7	8.3	8.6	5.2
1991	5.1	5.5	6.7	6.8	6.0	7.4	6.3	6.4	8.2	9.1	5.6	6.0
1992	5.5	6.2	5.6	6.8	6.7	5.9	5.8	8.4	7.8	8.0	7.3	5.5
1993	5.9	5.6	5.8	5.2	6.0	6.0	5.8	8.6	9.7	8.6	6.5	5.4
Average	5.3	5.6	5.8	6.1	6.2	5.8	6.1	7.5	8.1	8.0	6.6	5.2

Table A.2.5 **Mean Monthly Wind Velocity at Sengkang**

(Unit: m/sec)

Year	Jan.	Feb.	Mar.	Apr.	May	Jun.	Jul.	Aug.	Sept.	Oct.	Nov.	Dec.
1976	1.3	1.5	1.5	1.1	1.3	1.3	1.5	1.8	1.6	1.3	0.9	1.5
1977	1.5	2.0	1.3	1.1	1.0	1.4	1.8	2.0	2.0	2.0	1.4	1.2
1978	1.2	1.2	1.0	1.1	1.1	1.0	1.1	1.4	1.3	1.3	1.0	1.3
1979	1.2	1.2	1.1	1.0	1.0	1.3	1.7	1.9	1.8	1.3	1.2	1.3
1980	1.5	1.3	1.3	0.9	1.7	1.3	1.4	1.5	1.7	1.5	1.4	1.3
1981	1.8	1.3	1.0	1.0	1.1	0.9	1.1	1.3	0.2	1.3	1.3	1.6
1982	1.5	1.3	1.2	1.7	1.3	1.2	1.6	1.7	1.7	1.9	1.6	1.0
1983	1.3	1.3	1.0	0.9	1.1	2.2	2.6	-	1.7	1.4	1.3	1.0
1984	1.3	1.5	1.2	0.5	0.9	1.0	1.2	1.2	1.0	1.0	0.3	1.1
1985	1.2	0.7	0.9	0.8	0.8	0.8	0.9	1.2	0.9	0.1	0.7	1.1
1986	1.1	0.9	0.7	0.8	1.0	0.8	0.7	1.1	1.2	0.8	0.6	0.5
1987	0.9	1.0	0.8	0.8	1.0	1.2	1.2	1.7	1.7	1.5	0.9	0.9
1988	0.6	0.8	0.8	0.7	0.7	0.8	1.0	1.3	1.2	0.3	0.8	1.0
1989	1.1	1.2	0.9	0.8	0.6	0.7	0.9	1.2	1.3	0.9	0.7	0.8
1990	1.0	0.7	0.9	0.3	0.7	0.9	1.1	0.9	0.9	0.8	0.8	0.1
1991	1.1	0.9	0.8	0.6	1.1	0.9	0.9	1.1	1.1	1.1	0.7	0.6
1992	0.7	0.7	0.6	0.6	0.5	0.6	0.7	0.7	0.6	0.7	0.5	0.9
1993	0.9	1.1	0.7	0.6	0.7	0.8	0.8	0.9	1.1	2.3	0.8	1.1
Average	1.4	1.3	1.2	1.0	1.1	1.2	1.5	1.6	1.4	1.3	1.1	1.2

Table A.2.6 **Mean Monthly Evaporation at Sengkang**
(Class A Pan)

(Unit: mm/day)

Year	Jan.	Feb.	Mar.	Apr.	May	Jun.	Jul.	Aug.	Sept.	Oct.	Nov.	Dec.
1976	6.0	6.3	6.0	5.3	4.7	3.8	4.6	5.8	6.9	6.4	5.3	5.4
1977	5.4	6.3	6.1	4.4	5.2	3.7	4.3	5.6	8.0	9.8	7.6	5.8
1978	5.7	6.3	5.6	5.6	4.5	4.4	4.0	5.2	5.2	5.1	4.5	4.6
1979	5.3	6.3	5.4	4.5	4.2	4.0	5.3	6.7	6.2	6.8	6.2	5.4
1980	6.1	5.6	5.7	4.7	4.3	3.7	4.7	5.3	7.5	7.2	5.7	5.5
1981	6.1	4.8	6.9	4.5	3.9	4.5	3.5	6.8	7.2	7.4	6.4	5.5
1982	6.3	5.6	5.5	4.1	5.0	4.3	6.0	7.5	8.3	8.9	7.6	5.9
1983	5.4	4.8	5.0	4.8	4.3	4.6	4.3	6.9	7.9	6.7	6.2	5.3
1984	5.4	5.8	7.9	7.7	3.2	4.2	3.9	4.8	5.3	6.5	5.7	4.7
1985	5.8	5.0	6.0	5.7	4.4	4.6	3.6	5.1	6.9	5.3	5.0	6.1
1986	5.6	6.1	6.1	5.2	5.9	4.6	4.7	6.1	7.3	5.5	5.6	5.3
1987	5.8	6.4	5.2	5.2	5.2	5.3	5.7	7.3	8.4	8.8	7.1	5.0
1988	6.0	5.0	5.5	5.3	5.2	4.1	4.9	4.8	5.5	6.0	5.2	5.1
1989	4.8	6.3	6.1	6.4	5.0	4.6	3.7	4.3	6.1	5.9	5.8	5.2
1990	5.8	5.0	6.6	5.6	4.6	4.5	4.7	5.6	7.4	7.3	7.3	5.8
1991	6.4	5.4	6.1	5.8	5.1	6.0	6.1	6.6	7.7	7.9	6.5	5.6
1992	5.5	5.8	5.5	5.7	5.2	4.3	4.7	5.0	6.3	6.7	5.9	6.1
1993	5.8	7.3	5.8	5.9	4.8	6.9	6.1	6.7	8.2	9.1	6.1	5.4
Average	5.7	5.8	5.9	5.4	4.7	4.6	4.7	5.9	7.0	7.1	6.1	5.4

Table A.2.7 Regression Formula of Monthly Discharge

Month	Regression Formula Bila (Upstream) v.s. Gilirang	 Bila (Downstream) v.s. Gilirang
January	Y = 0.166 X - 0.790	Y = 0.203 X - 0.105
February	Y = 0.638 X - 5.547	Y = 0.360 X - 0.444
March	Y = 0.031 X + 1.017	Y = 0.042 X + 1.080
April	Y = 0.079 X + 8.920	Y = -0.643 X + 27.916
May	Y = 0.293 X + 6.711	Y = 0.244 X + 11.868
June	Y = 0.727 X - 4.879	Y = 0.589 X + 2.467
July	Y = 0.805 X - 2.395	Y = 1.034 X - 7.569
August	Y = 1.681 X - 14.433	Y = 0.858 X - 0.952
September	Y = 0.527 X - 2.993	Y = 0.367 X - 0.049
October	Y = 0.198 X + 0.336	Y = 1.208 X - 2.190
November	Y = 0.171 X + 0.232	Y = 0.468 X - 0.400
December	Y = -0.008 X + 1.017	Y = -0.020 X + 0.939

Table A.2.8

Half-monthly Average Discharge at Gilirang Gauging Station (S. Gilirang)

Unit : m3/sec

	1979	1980	1981	1982	1983	1984	1985	1986	1987	1988	1989	1990	1991	1992	1993	Average
Jan.	1.77	2.53	0.13	1.11	1.06	0.14	1.38	3.40	0.74	1.06	5.48	0.46	12.81	0.53	0.19	2.19
	0.83	1.21	0.17	0.46	4.54	1.96	0.92	0.58	2.32	5.29	4.34	0.26	0.19	0.54	0.39	1.60
Feb.	9.59	1.65	0.26	0.51	0.51	12.55	9.02	0.79	0.75	1.10	2.26	2.70	0.12	6.37	0.02	3.21
	0.69	3.21	0.22	0.47	17.37	2.78	1.32	0.57	0.52	3.49	1.65	40.83	0.31	4.88	0.03	5.22
Mar.	0.27	1.49	0.23	1.22	0.71	1.60	1.06	1.57	2.34	2.46	1.60	1.48	0.71	1.53	0.06	1.22
	1.87	1.54	3.16	1.86	0.68	1.81	1.32	3.59	3.36	4.02	1.43	1.78	0.69	2.42	0.52	2.00
Apr.	12.43	12.22	59.84	0.96	1.57	0.94	0.38	8.29	29.92	5.84	3.22	3.85	1.29	11.46	0.52	10.18
	11.46	6.52	11.02	6.30	22.63	11.29	0.51	10.77	28.58	23.64	2.98	3.48	32.13	11.82	5.75	12.59
May	19.15	14.40	16.10	4.61	11.90	24.14	10.67	21.37	25.55	37.82	3.79	14.39	14.04	15.67	32.39	17.73
	10.21	20.32	18.60	15.50	38.43	36.64	18.38	18.83	39.94	11.33	3.54	8.20	25.76	3.98	22.00	19.44
Jun.	31.51	16.30	3.97	19.05	18.49	39.87	22.39	35.33	25.21	41.64	3.06	10.37	33.05	19.06	20.77	22.67
	58.73	29.97	15.38	20.08	46.24	5.13	9.49	10.62	7.74	10.98	29.87	8.03	4.07	6.05	25.07	19.16
Jul.	19.37	2.83	44.85	1.00	6.05	58.27	17.33	23.62	4.37	37.36	55.55	1.75	32.96	15.97	31.78	23.54
	5.89	0.61	78.11	0.54	35.20	72.53	7.74	20.20	3.17	25.97	93.63	15.56	5.07	19.92	3.73	25.86
Aug.	3.87	9.14	3.42	0.20	4.73	27.29	11.83	12.33	1.05	54.65	21.67	5.72	1.00	12.92	0.57	11.36
	2.43	4.23	0.84	0.25	10.00	5.35	14.82	5.91	1.00	49.48	70.46	1.55	2.57	1.11	0.95	11.40
Sept.	22.16	0.70	13.27	0.04	0.79	3.55	1.93	1.83	0.51	55.04	29.47	2.06	0.73	5.73	0.25	9.20
	1.09	0.27	1.54	0.02	0.63	2.30	0.97	3.41	5.38	48.42	21.48	4.52	0.44	11.59	0.06	6.81
Oct.	0.42	0.28	4.46	0.03	0.57	3.67	1.54	3.24	0.73	7.26	9.81	3.42	0.21	4.79	0.34	2.72
	0.25	0.42	8.91	0.02	3.87	1.22	2.61	3.73	1.36	4.75	16.99	11.95	0.77	1.04	1.10	3.93
Nov.	0.97	0.25	12.20	0.02	2.10	0.52	2.24	4.83	5.51	1.69	2.53	2.13	0.20	0.35	0.64	2.41
	0.51	0.22	6.71	0.95	0.37	1.09	0.72	11.20	2.23	1.46	0.41	1.92	0.08	1.86	0.32	2.00
Dec.	0.83	0.26	0.90	3.61	0.10	0.76	1.01	1.12	1.52	1.01	0.25	0.46	0.23	0.18	1.06	0.89
	0.46	0.19	0.83	0.11	0.33	0.58	1.01	0.90	1.28	4.41	0.26	1.02	0.51	0.90	1.11	0.93

Table A.2.9 Half-monthly Average Discharge at Proposed Dam Site

Unit : m3/sec

	1979	1980	1981	1982	1983	1984	1985	1986	1987	1988	1989	1990	1991	1992	1993	Average
Jan.	1.30	1.86	0.10	0.82	0.78	0.11	1.01	2.49	0.54	0.78	4.02	0.34	9.42	0.39	0.14	1.61
	0.61	0.89	0.13	0.34	3.33	1.44	0.67	0.43	1.70	3.89	3.19	0.19	0.14	0.39	0.29	1.18
Feb.	7.04	1.21	0.19	0.38	0.37	9.22	6.63	0.58	0.55	0.81	1.66	1.99	0.09	4.68	0.01	2.36
	0.51	2.36	0.16	0.35	12.76	2.04	0.97	0.42	0.38	2.56	1.21	30.00	0.23	3.58	0.02	3.84
Mar.	0.20	1.10	0.17	0.89	0.52	1.18	0.78	1.16	1.72	1.81	1.17	1.09	0.52	1.13	0.05	0.90
	1.37	1.13	2.32	1.36	0.50	1.33	0.97	2.64	2.47	2.95	1.05	1.31	0.51	1.78	0.38	1.47
Apr.	9.13	8.98	43.97	0.71	1.16	0.69	0.28	6.09	21.99	4.29	2.37	2.83	0.95	8.42	0.38	7.48
	8.42	4.79	8.10	4.63	16.63	8.30	0.37	7.91	21.00	17.37	2.19	2.56	23.61	8.69	4.22	9.25
May	14.07	10.58	11.83	3.39	8.74	17.74	7.84	15.70	18.78	27.79	2.79	10.57	10.32	11.51	23.80	13.03
	7.50	14.93	13.67	11.39	28.24	26.92	13.50	13.83	29.35	8.33	2.60	6.02	18.93	2.93	16.17	14.29
Jun.	23.16	11.97	2.92	14.00	13.59	29.29	16.45	25.96	18.52	30.60	2.25	7.62	24.28	14.01	15.26	16.66
	43.16	22.02	11.30	14.75	33.98	3.77	6.98	7.80	5.69	8.07	21.95	5.90	2.99	4.44	18.42	14.08
Jul.	14.23	2.08	32.96	0.73	4.45	42.82	12.73	17.36	3.21	27.45	40.81	1.29	24.22	11.74	23.35	17.30
	4.33	0.45	57.39	0.39	25.86	53.29	5.68	14.84	2.33	19.08	68.80	11.43	3.72	14.64	2.74	19.00
Aug.	2.85	6.71	2.51	0.14	3.48	20.05	8.69	9.06	0.77	40.16	15.92	4.21	0.73	9.50	0.42	8.35
	1.78	3.11	0.61	0.18	7.35	3.93	10.89	4.34	0.73	36.35	51.77	1.14	1.89	0.81	0.70	8.37
Sept.	16.28	0.51	9.75	0.03	0.58	2.61	1.42	1.34	0.37	40.44	21.65	1.51	0.54	4.21	0.18	6.76
	0.80	0.20	1.13	0.02	0.46	1.69	0.71	2.51	3.95	35.58	15.78	3.32	0.32	8.51	0.04	5.00
Oct.	0.31	0.21	3.28	0.02	0.42	2.69	1.13	2.38	0.54	5.34	7.21	2.51	0.15	3.52	0.25	2.00
	0.19	0.31	6.55	0.02	2.84	0.90	1.92	2.74	1.00	3.49	12.49	8.78	0.57	0.77	0.81	2.89
Nov.	0.71	0.18	8.96	0.02	1.55	0.38	1.64	3.55	4.05	1.24	1.86	1.57	0.14	0.26	0.47	1.77
	0.38	0.16	4.93	0.70	0.27	0.80	0.53	8.23	1.64	1.07	0.30	1.41	0.06	1.36	0.23	1.47
Dec.	0.61	0.19	0.66	2.66	0.08	0.56	0.74	0.82	1.12	0.74	0.19	0.34	0.17	0.14	0.78	0.65
	0.34	0.14	0.61	0.08	0.24	0.43	0.74	0.66	0.94	3.24	0.19	0.75	0.38	0.66	0.82	0.68

Table A.2.10 **Half-monthly Average Inflow from Residual Basin between Dam and Weir**

Unit : m3/sec

	1979	1980	1981	1982	1983	1984	1985	1986	1987	1988	1989	1990	1991	1992	1993	Average
Jan.	0.47	0.67	0.03	0.29	0.28	0.04	0.37	0.90	0.20	0.28	1.45	0.12	3.40	0.14	0.05	0.58
	0.22	0.32	0.05	0.12	1.20	0.52	0.24	0.15	0.61	1.40	1.15	0.07	0.05	0.14	0.10	0.42
Feb.	2.54	0.44	0.07	0.14	0.14	3.33	2.39	0.21	0.20	0.29	0.60	0.72	0.03	1.69	0.01	0.85
	0.18	0.85	0.06	0.12	4.61	0.74	0.35	0.15	0.14	0.92	0.44	10.83	0.08	1.29	0.01	1.38
Mar.	0.07	0.40	0.06	0.32	0.19	0.42	0.28	0.42	0.62	0.65	0.42	0.39	0.19	0.41	0.02	0.32
	0.50	0.41	0.84	0.49	0.18	0.48	0.35	0.95	0.89	1.07	0.38	0.47	0.18	0.64	0.14	0.53
Apr.	3.30	3.24	15.87	0.25	0.42	0.25	0.10	2.20	7.94	1.55	0.86	1.02	0.34	3.04	0.14	2.70
	3.04	1.73	2.92	1.67	6.00	3.00	0.14	2.86	7.58	6.27	0.79	0.92	8.52	3.14	1.52	3.34
May	5.08	3.82	4.27	1.22	3.16	6.40	2.83	5.67	6.78	10.03	1.01	3.82	3.72	4.15	8.59	4.70
	2.71	5.39	4.93	4.11	10.19	9.72	4.87	4.99	10.59	3.01	0.94	2.17	6.83	1.06	5.84	5.16
Jun.	8.36	4.32	1.05	5.05	4.90	10.57	5.94	9.37	6.69	11.04	0.81	2.75	8.77	5.06	5.51	6.01
	15.58	7.95	4.08	5.33	12.26	1.36	2.52	2.82	2.05	2.91	7.92	2.13	1.08	1.60	6.65	5.08
Jul.	5.14	0.75	11.90	0.27	1.61	15.46	4.60	6.27	1.16	9.91	14.73	0.46	8.74	4.24	8.43	6.24
	1.56	0.16	20.71	0.14	9.34	19.24	2.05	5.36	0.84	6.89	24.83	4.13	1.34	5.28	0.99	6.86
Aug.	1.03	2.42	0.91	0.05	1.26	7.24	3.14	3.27	0.28	14.50	5.75	1.52	0.27	3.43	0.15	3.01
	0.64	1.12	0.22	0.07	2.65	1.42	3.93	1.57	0.27	13.12	18.69	0.41	0.68	0.29	0.25	3.02
Sept.	5.88	0.19	3.52	0.01	0.21	0.94	0.51	0.48	0.13	14.60	7.82	0.55	0.19	1.52	0.07	2.44
	0.29	0.07	0.41	0.01	0.17	0.61	0.26	0.91	1.43	12.84	5.70	1.20	0.12	3.07	0.02	1.81
Oct.	0.11	0.07	1.18	0.01	0.15	0.97	0.41	0.86	0.19	1.93	2.60	0.91	0.06	1.27	0.09	0.72
	0.07	0.11	2.36	0.01	1.03	0.32	0.69	0.99	0.36	1.26	4.51	3.17	0.21	0.28	0.29	1.04
Nov.	0.26	0.07	3.24	0.01	0.56	0.14	0.59	1.28	1.46	0.45	0.67	0.57	0.05	0.09	0.17	0.64
	0.14	0.06	1.78	0.25	0.10	0.29	0.19	2.97	0.59	0.39	0.11	0.51	0.02	0.49	0.08	0.53
Dec.	0.22	0.07	0.24	0.96	0.03	0.20	0.27	0.30	0.40	0.27	0.07	0.12	0.06	0.05	0.28	0.24
	0.12	0.05	0.22	0.03	0.09	0.15	0.27	0.24	0.34	1.17	0.07	0.27	0.14	0.24	0.29	0.25

Table A.2.11 Probable Daily Rainfall at Sakkoli

(Unit : mm/day)

Return Periods (Year)	Iwai Method	Hazen Method	Peason III Method	Gumbel Method	Hershfield Method
1.01	80	99	88	50	-
1.5	108	107	106	105	-
2	120	114	116	122	-
5	150	141	147	165	-
10	173	175	174	193	-
20	195	209	203	221	-
30	208	244	222	236	-
40	218	268	237	247	-
50	226	288	249	256	-
80	242	328	275	274	-
100	250	347	289	282	-
200	275	456	334	309	-
500	310	605	405	343	-
1000	339	713	469	370	-
P.M.P	-	-	-	-	504

Table A.2.12 **Measurement of Suspended Sediment at Gilirang Gauging Station**

No.	Date	Sample Volume (ml)	Weight of Suspended Sediment (mg)	Consentration of Suspended Sediment (mg/l)	Water Discharge (m3/sec)	Suspended Sediment (ton/day)
1	Apr. 26, 1993	250	2.5	10.0	5.5	4.75
		250	3.3	13.2	5.5	6.27
		250	2.9	11.6	5.5	5.51
						(5.51)
2	May 28, 1993	250	10.6	42.4	7.3	26.74
		250	12.6	50.4	7.3	31.79
		250	15.9	63.6	7.3	40.11
						(32.88)
3	Jul. 21, 1993	250	14.9	59.6	6.5	33.47
		250	11.9	47.6	6.5	26.73
		250	12.4	49.6	6.5	27.86
						(29.35)
4	Dec. 16, 1993	250	16.6	66.4	0.3	1.72
		250	14.5	58.0	0.3	1.50
		250	21.7	86.8	0.3	2.25
						(1.82)
5	May 28, 1994	250	11.6	46.4	3.0	12.03
		250	6.6	26.4	3.0	6.84
		250	15.9	63.6	3.0	16.49
						(11.78)

Table A.2.13 **Result of Water Quality Analysis**

Parameter	Unit	Proposed Dam Site				Gilirang Gauging Station			
		No.1	No.2	No.3	Average	No.4	No.5	No.6	Average
Chemical & Physical Properties									
E.C.	dS/m	0.14	0.14	0.14	0.14	0.27	0.24	0.24	0.25
p.H.	-	8.0	8.1	8.0	8.0	7.9	8.2	8.2	8.1
BOD	ppm	0.10	0.12	0.08	0.10	0.08	0.00	0.02	0.03
COD	ppm	32.11	29.32	30.37	30.60	33.50	32.11	33.50	33.04
DO	ppm	6.2	6.3	6.2	6.2	6.3	5.9	6.0	6.1
SS	mg/l	0.18	0.17	0.08	0.14	0.31	0.12	0.18	0.20
Cation Ion									
Na	me./l	0.001	0.001	0.001	0.001	0.001	0.001	0.001	0.001
Ca	me./l	2.24	0.02	1.98	1.41	3.32	3.31	0.32	2.32
Mg	me./l	0.39	0.31	0.30	0.33	0.55	0.55	0.54	0.55
Anion Ion									
F	me./l	0.001	0.001	0.001	0.001	0.001	0.001	0.001	0.001
Cl	me./l	20.0	16.0	20.0	18.7	10.0	10.0	12.0	10.7
NH4	me./l	0	0	0	0	0	0	0	0
NO3	me./l	0.00	0.01	0.00	0.00	0.01	0.02	0.01	0.01
NO2	me./l	0.00	0.01	0.01	0.01	0.01	0.02	0.03	0.02
TN	me./l	0.05	0.05	0.05	0.05	0.01	0.02	0.01	0.01
TP	me./l	0.007	0.005	-	0.006	0.008	0.008	-	0.008
PO4	me./l	0.002	0.003	-	0.003	0.007	0.005	-	0.006

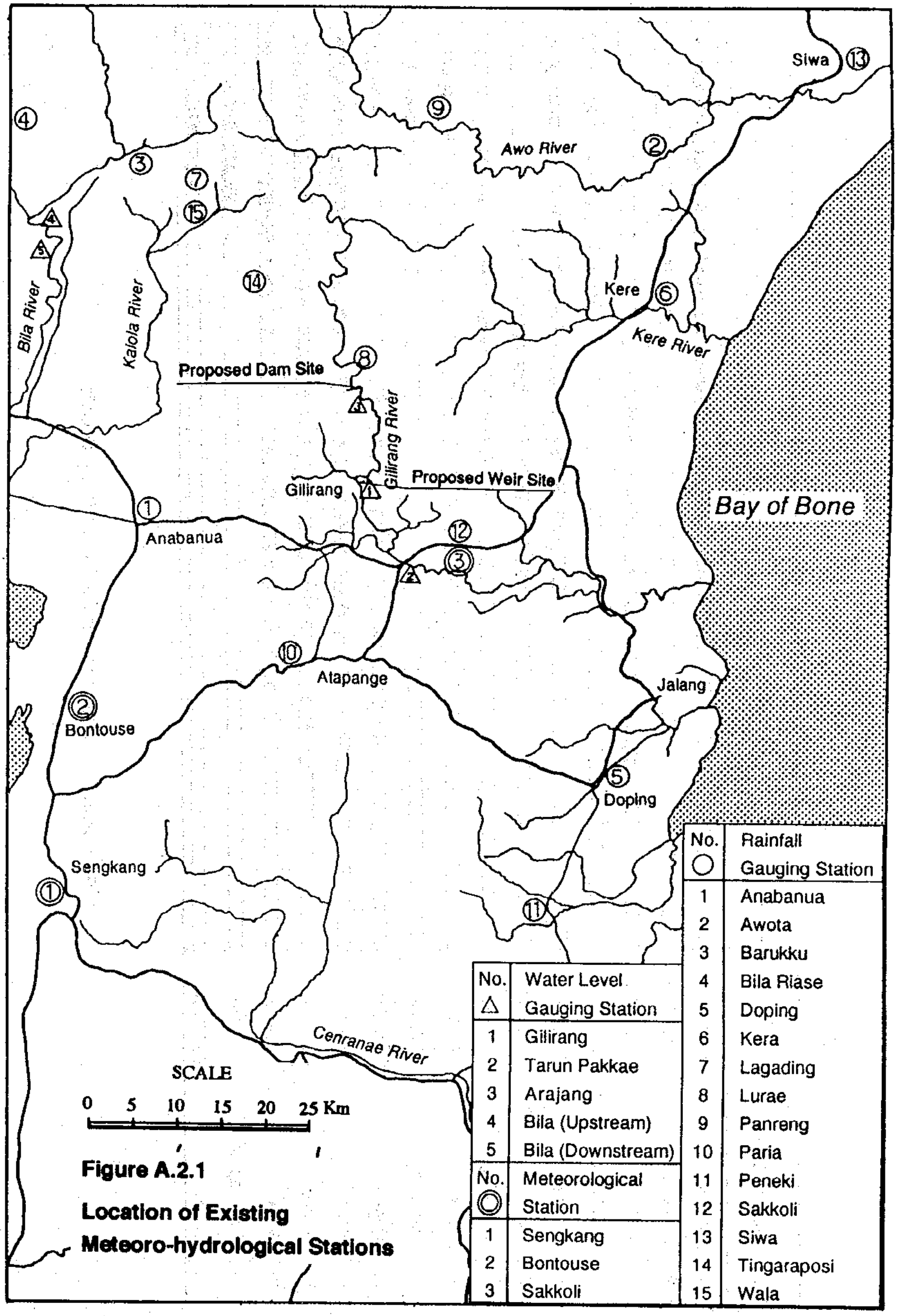


Figure A.2.1

Location of Existing Meteoro-hydrological Stations

No. △	Water Level Gauging Station
1	Gilirang
2	Tarun Pakkae
3	Arajang
4	Bila (Upstream)
5	Bila (Downstream)

No. ◎	Meteorological Station
1	Sengkang
2	Bontouse
3	Sakkoli

No. ○	Rainfall Gauging Station
1	Anabanua
2	Awota
3	Barukku
4	Bila Riase
5	Doping
6	Kera
7	Lagading
8	Lurae
9	Panreng
10	Paria
11	Peneki
12	Sakkoli
13	Siwa
14	Tingaraposi
15	Wala

No.	Rainfall Station	Owner
1	Anabanua	Pertanian
2	Awota	P3SA/BPPDSA
3	Barukku	P3SA/BPPDSA
4	Bila Riase	BPPDSA
5	Doping	Pertanian
6	Kera	Pertanian
7	Lagading	P3SA/BPPDSA
8	Lurae	BPPDSA
9	Panreng	BPPDSA
10	Paria	Pertanian
11	Peneki	Pertanian
12	Sakkoli	Pertanian
13	Siwa	Pertanian
14	Tingaraposi	BPPDSA
15	Wala	BPPDSA

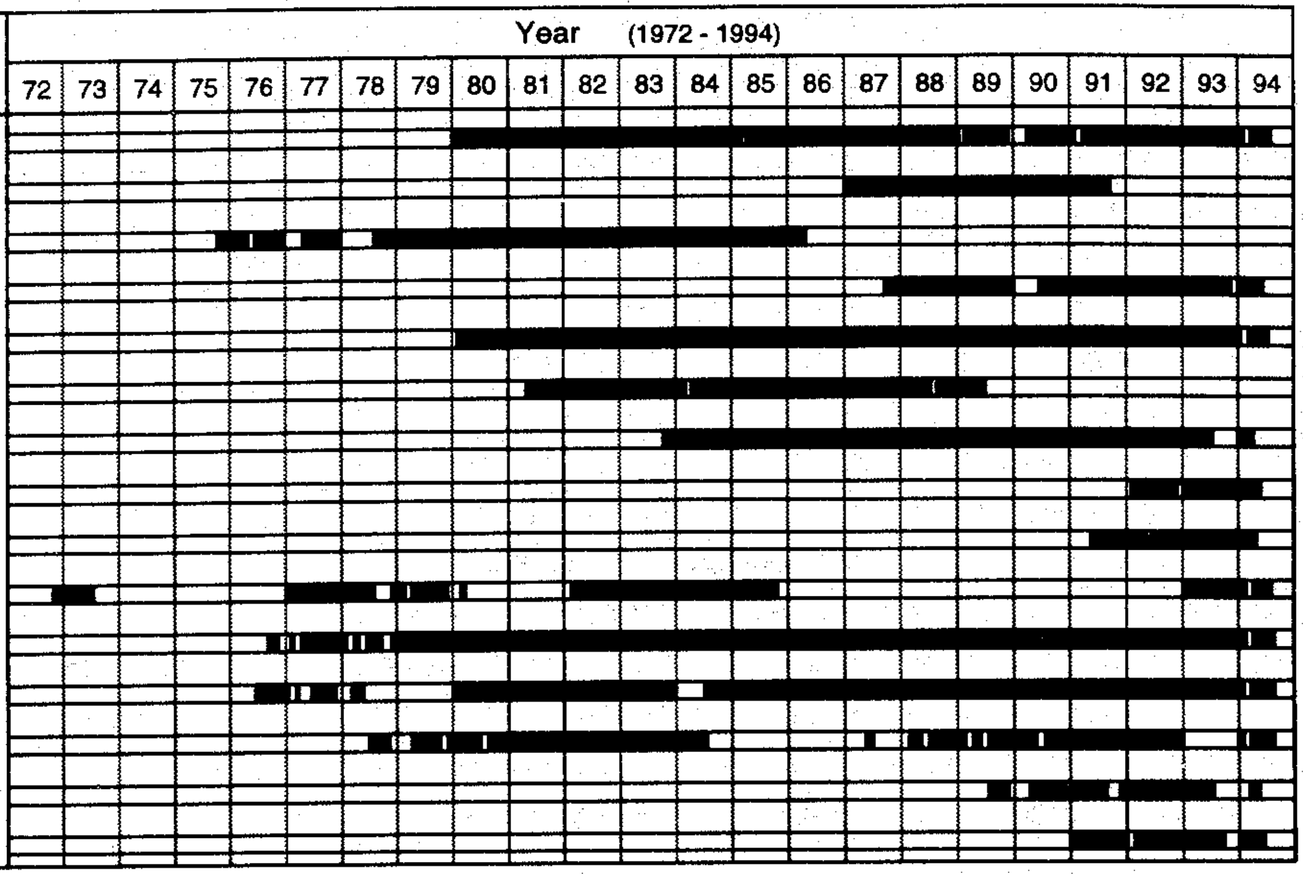


Figure A.2.2 Observation Period of Daily Rainfall

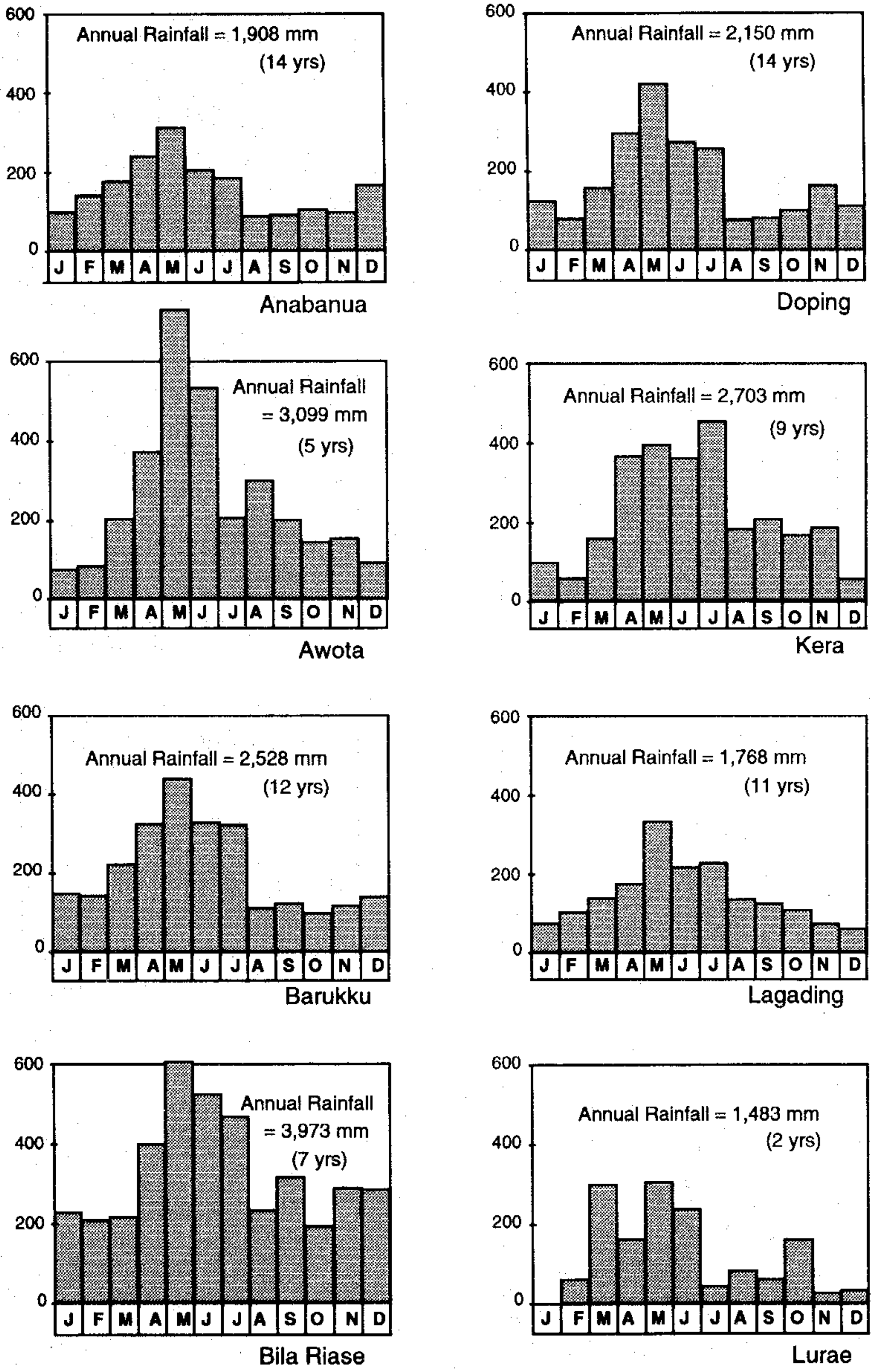


Figure A.2.3 (1/2) Annual Rainfall Pattern in and around the Project Area (mm/month)

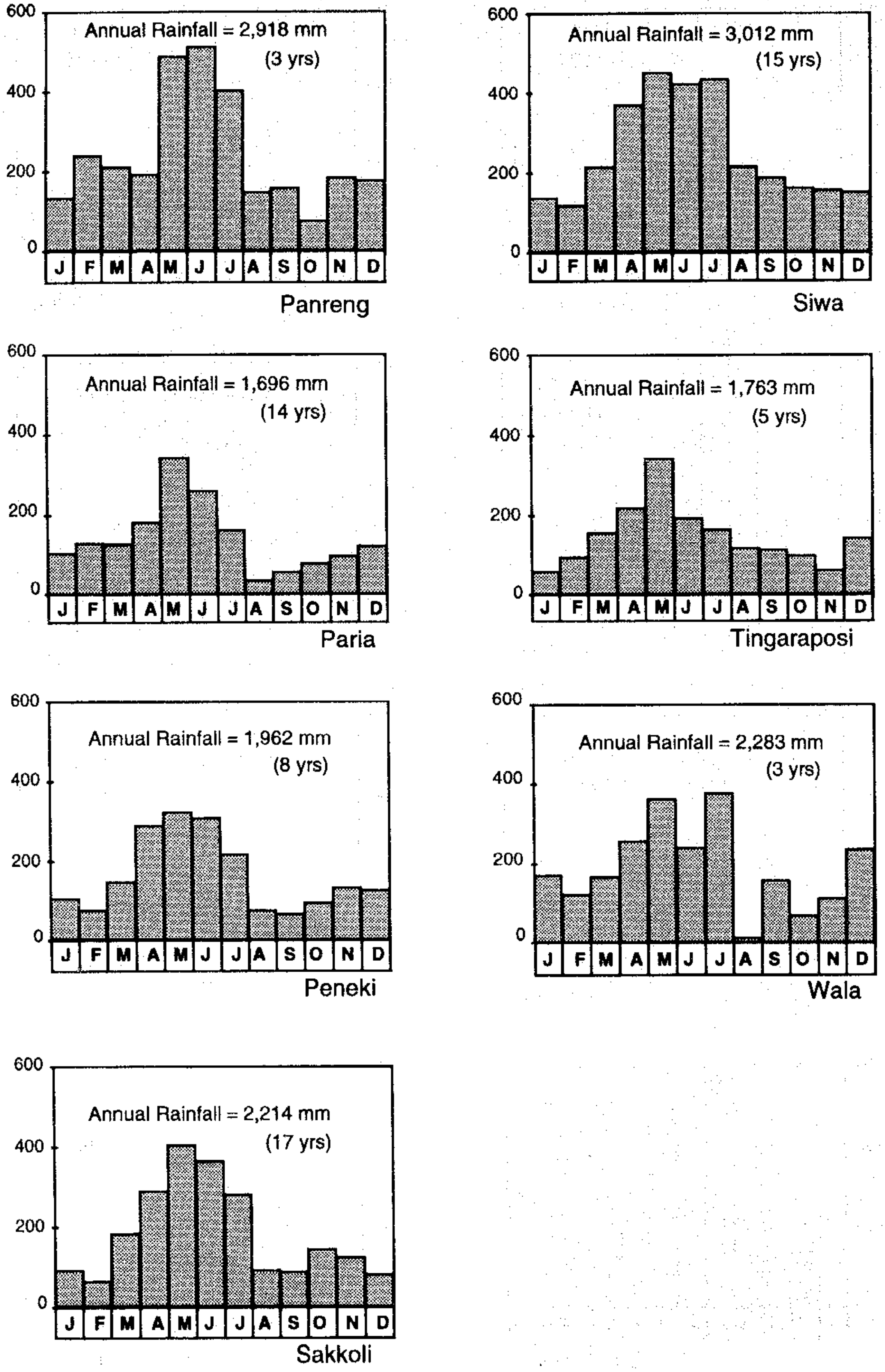


Figure A.2.3 (2/2) Annual Rainfall Pattern in and around the Project Area
(mm/month)

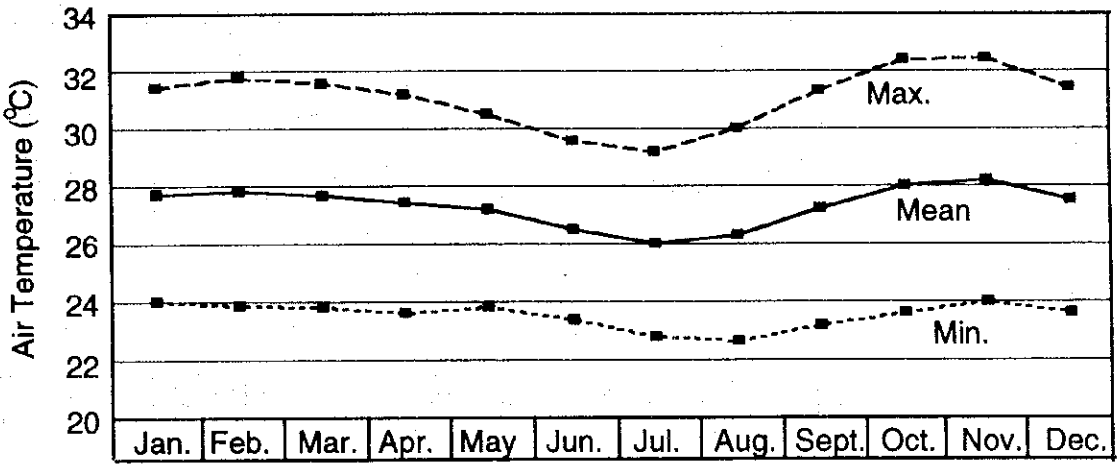


Figure A.2.4 Mean Monthly Air Temperature at Sengkang

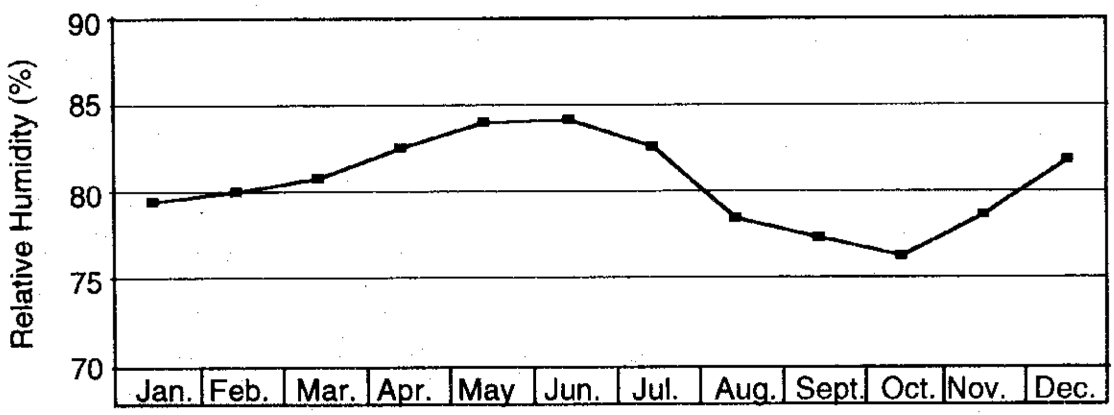


Figure A.2.5 Mean Monthly Relative Humidity at Sengkang

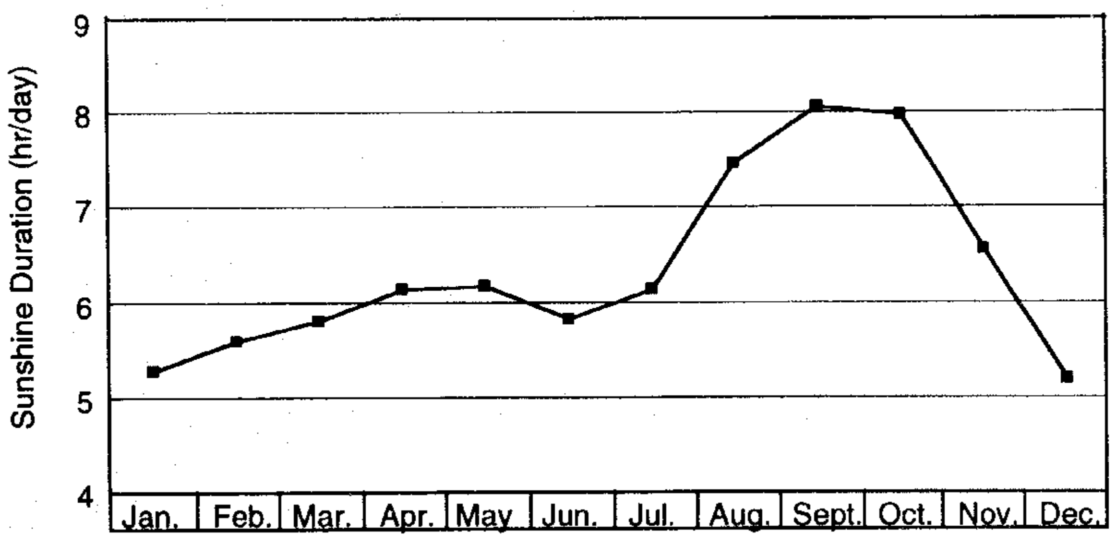


Figure A.2.6 Mean Monthly Sunshine Duration at Sengkang

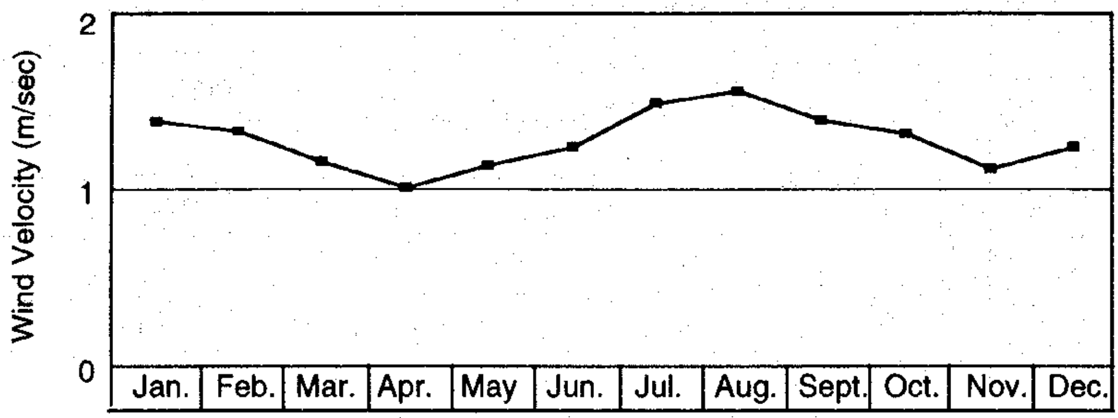


Figure A.2.7 Mean Monthly Wind Velocity at Sengkang

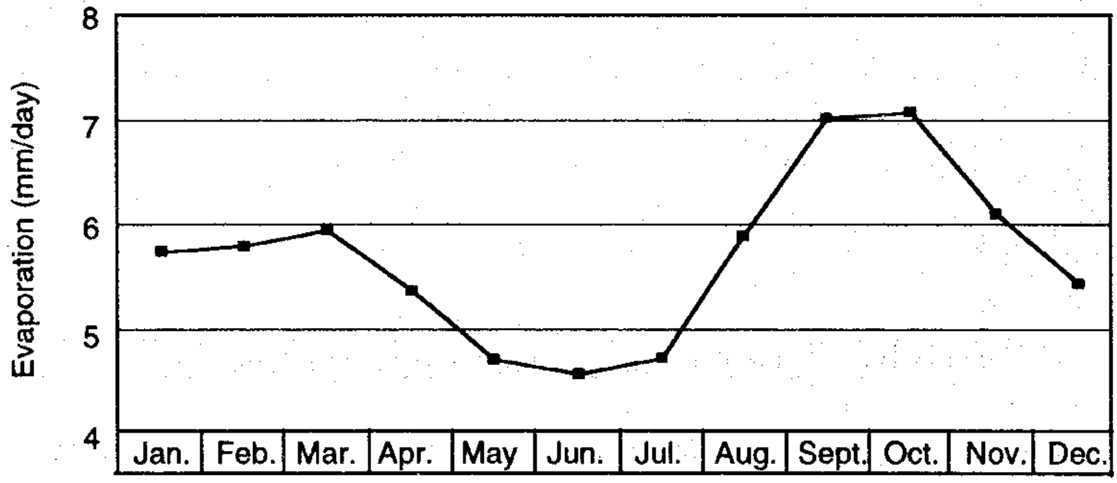


Figure A.2.8 Mean Monthly Evaporation at Sengkang

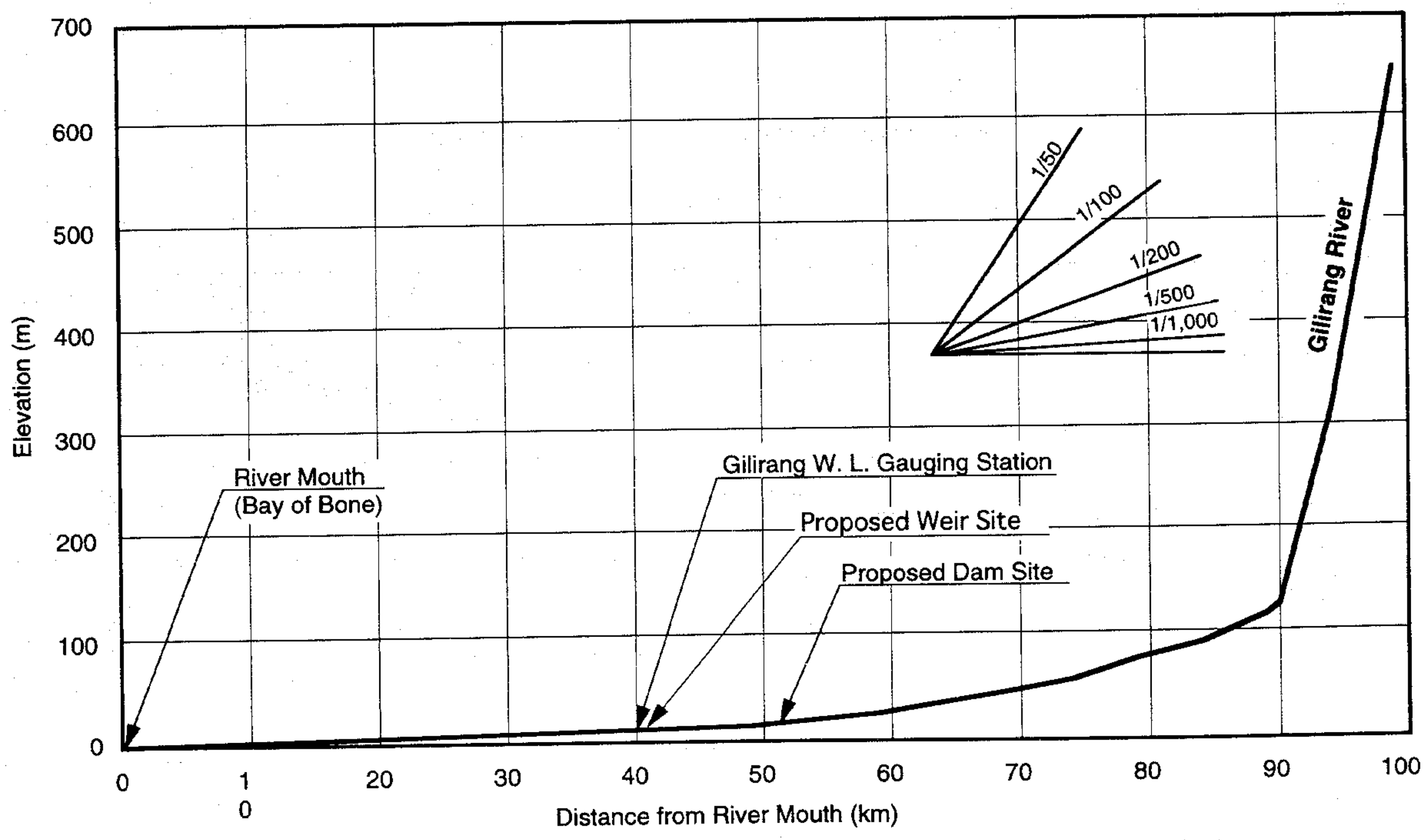


Figure A.2.9 Profile of Gilirang River

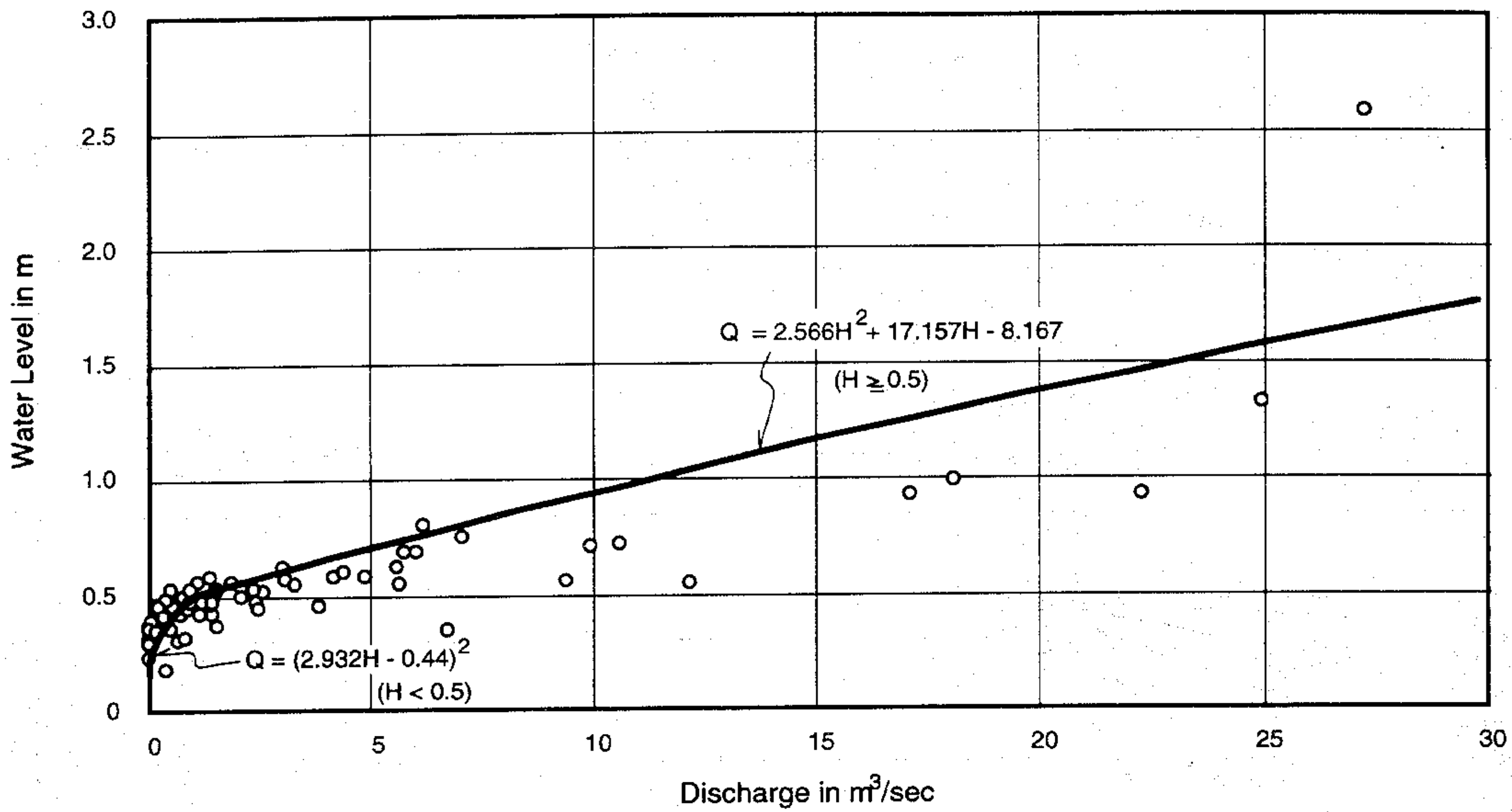


Figure A.2.10 Rating Curve at Gilirang Station

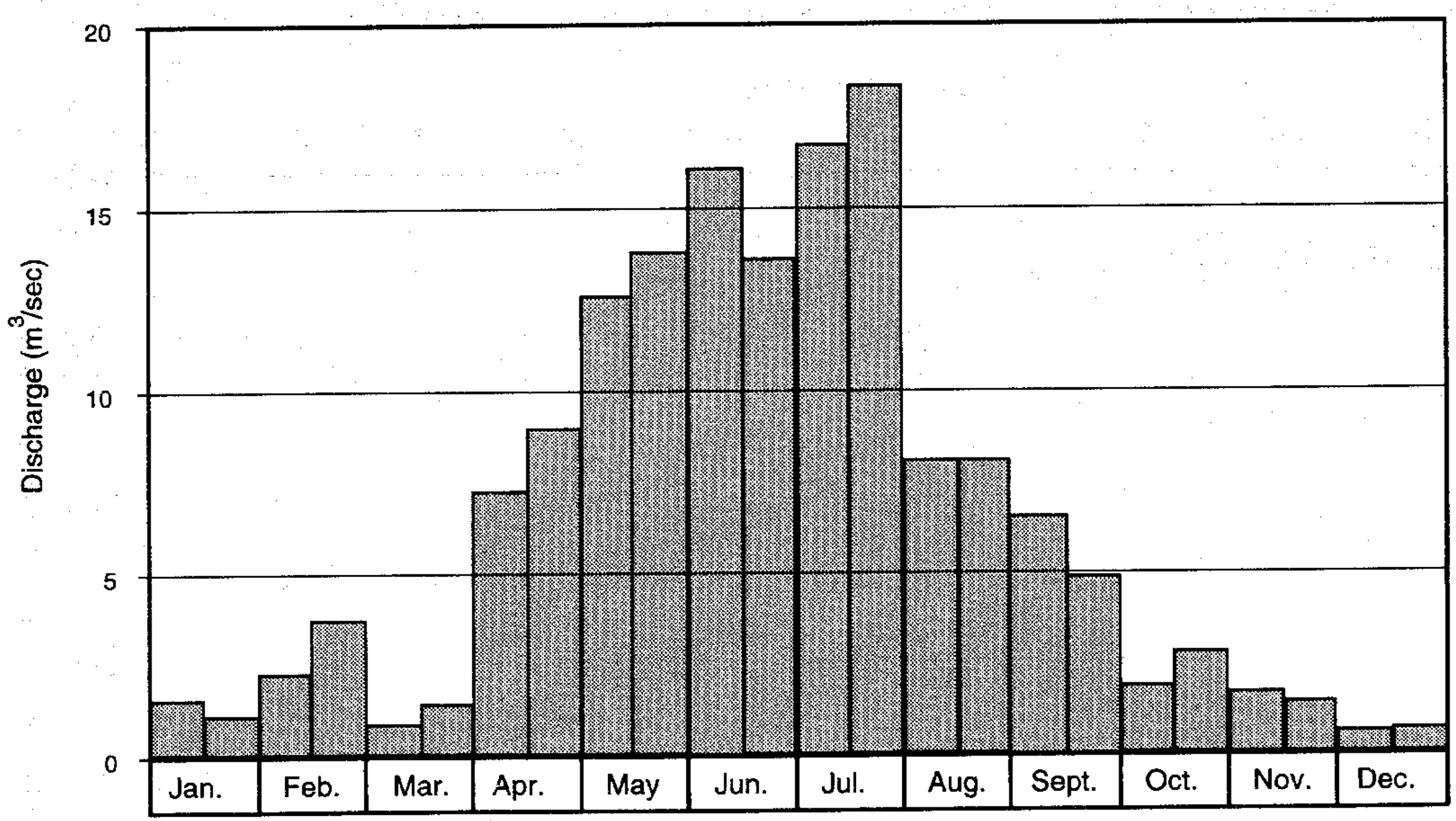


Figure A.2.11 Half-monthly Average Discharge at Proposed Dam Site

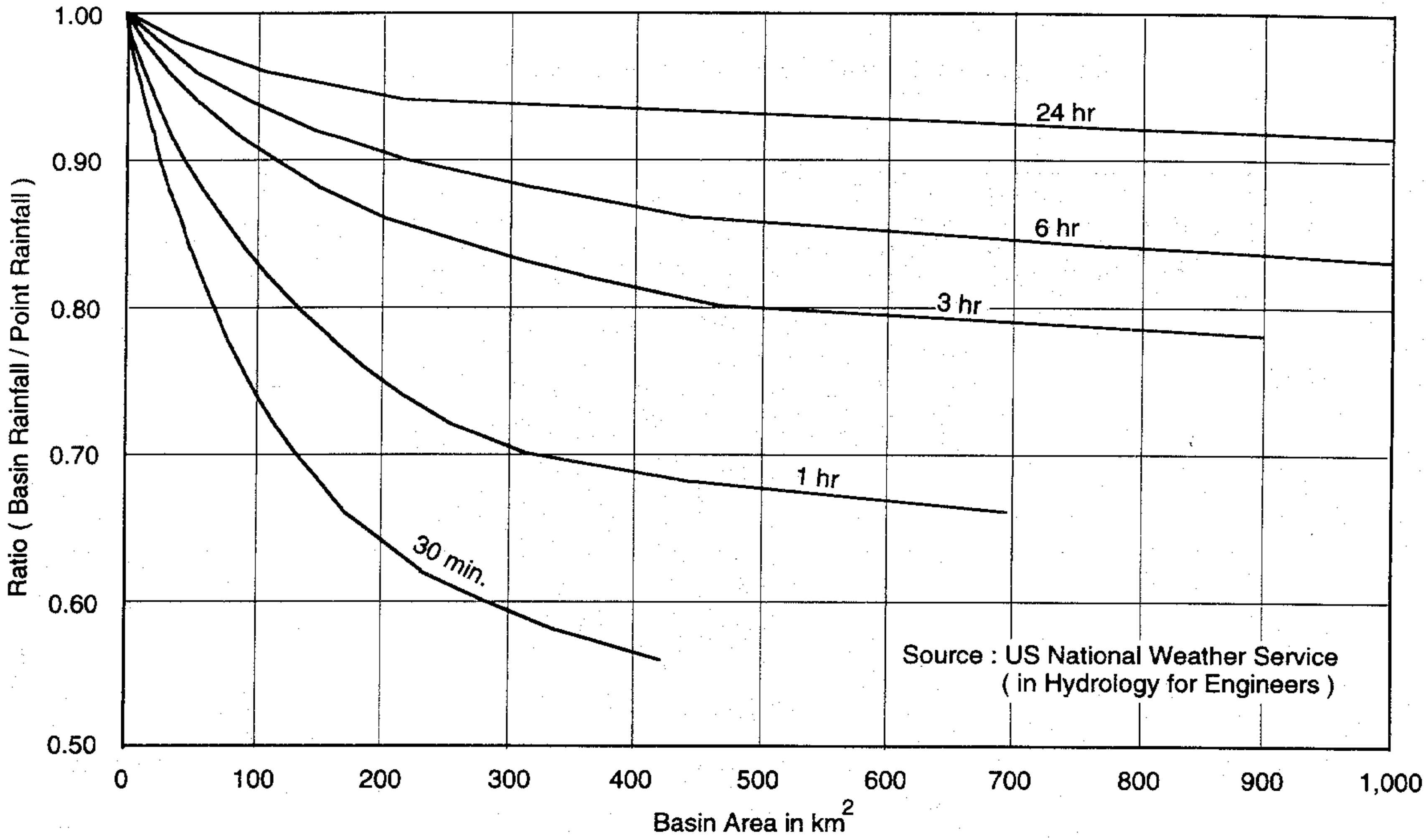


Figure A.2.12 Conversion Ratio from Point Rainfall into Basin Rainfall

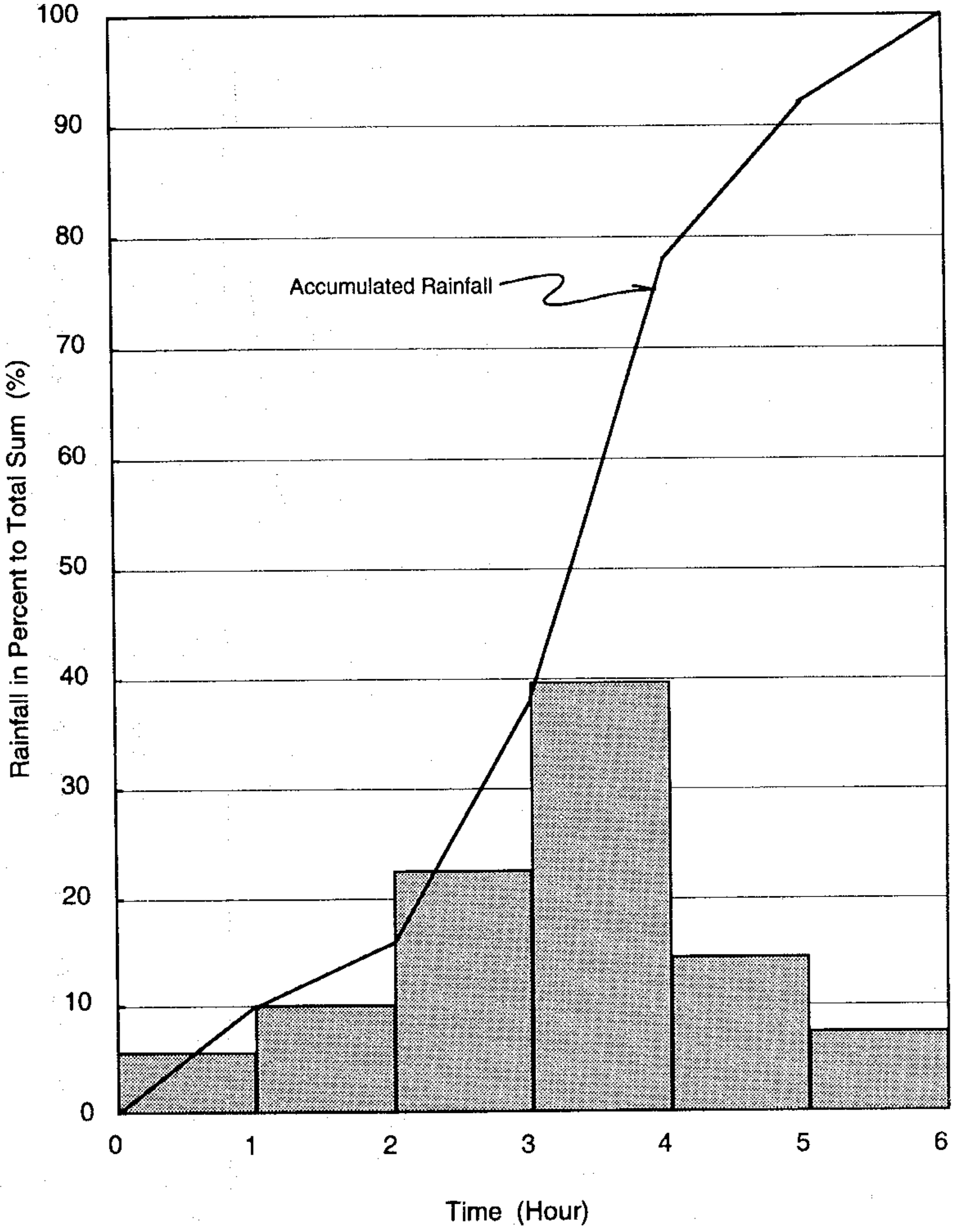


Figure A.2.13 Hourly Rainfall Distribution of Design Storm

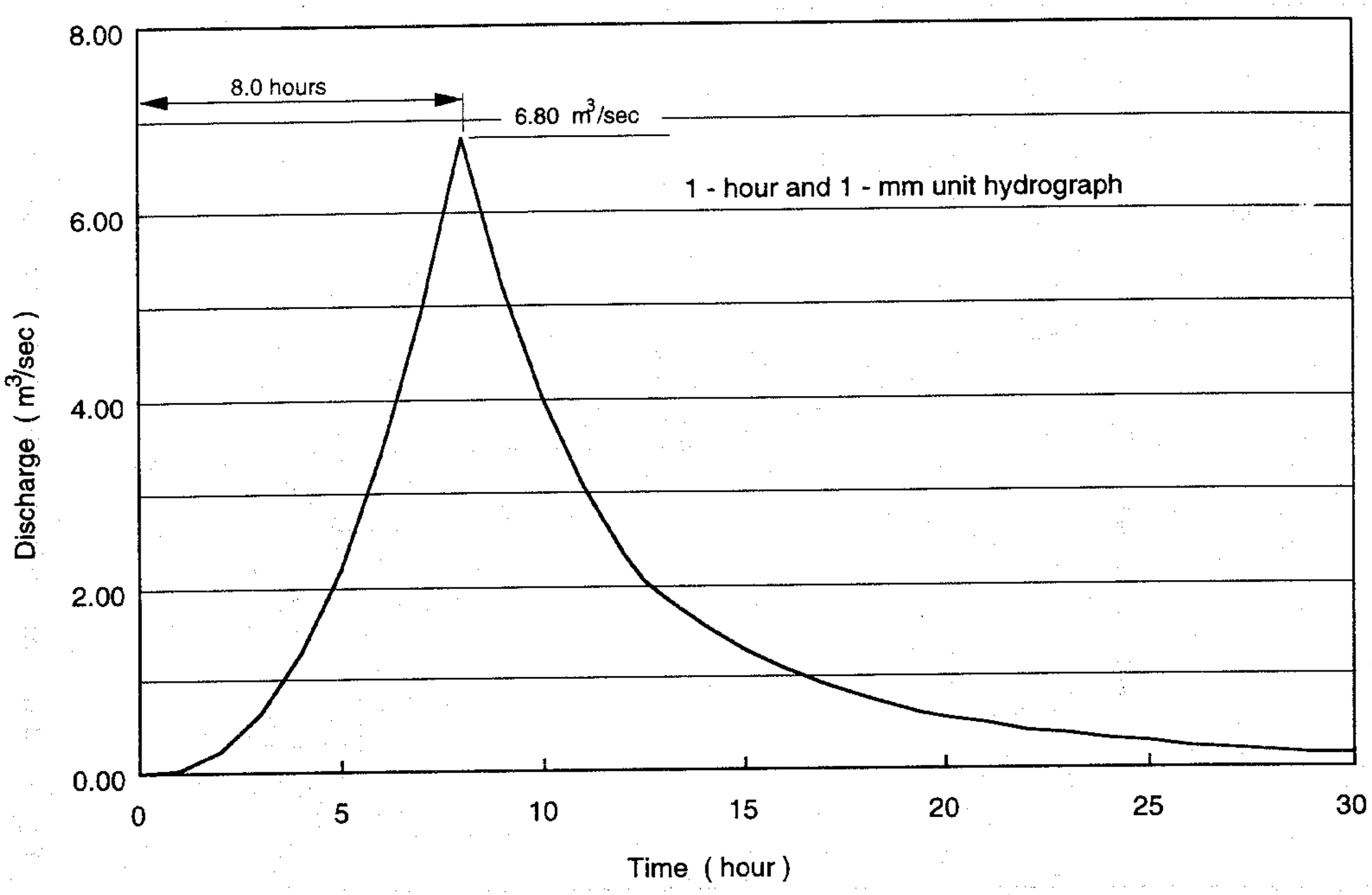


Figure A.2.14 Unit Hydrograph of Gilirang River at Proposed Dam Site

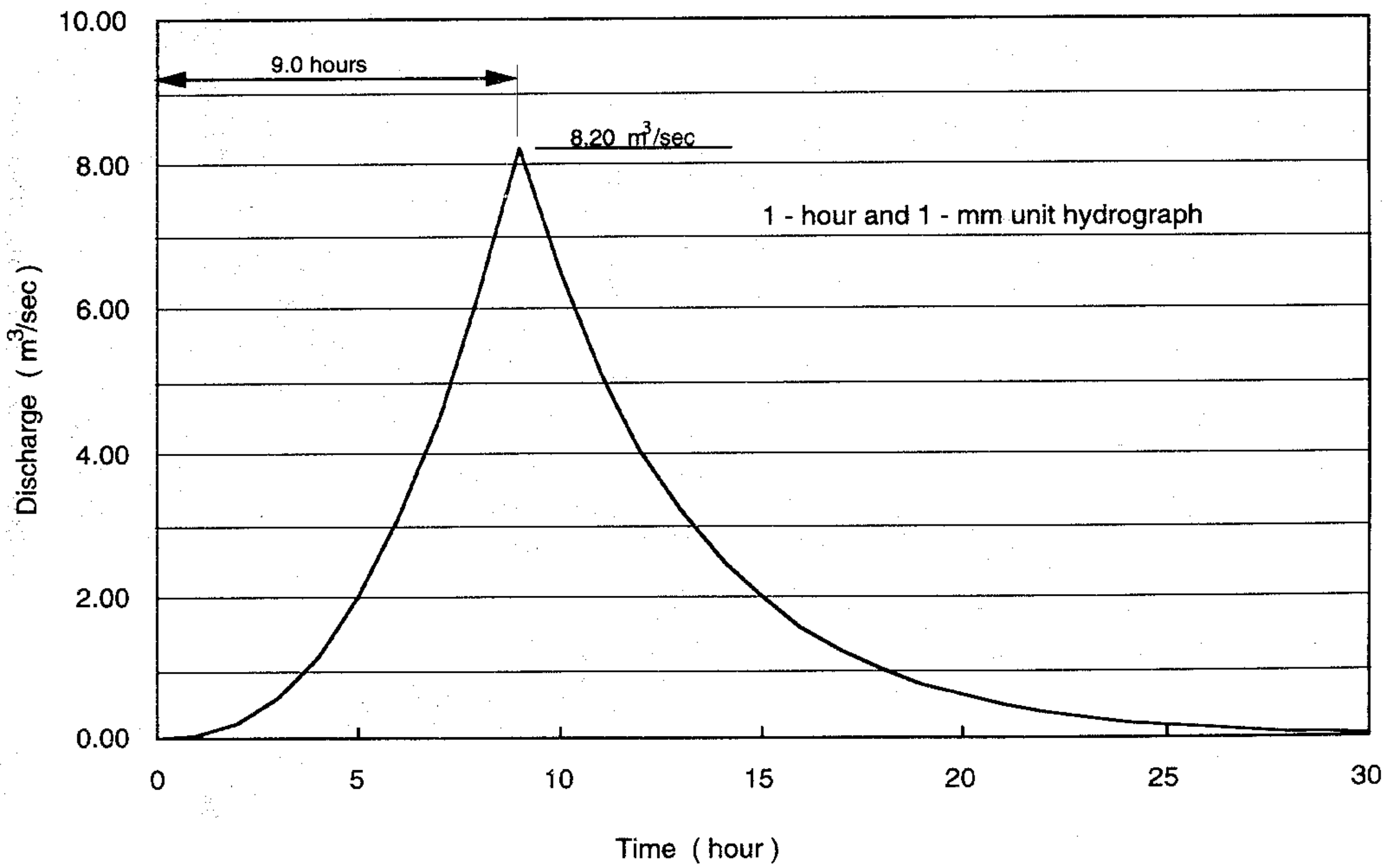


Figure A.2.15 Unit Hydrograph of Gilirang River at Proposed Weir Site

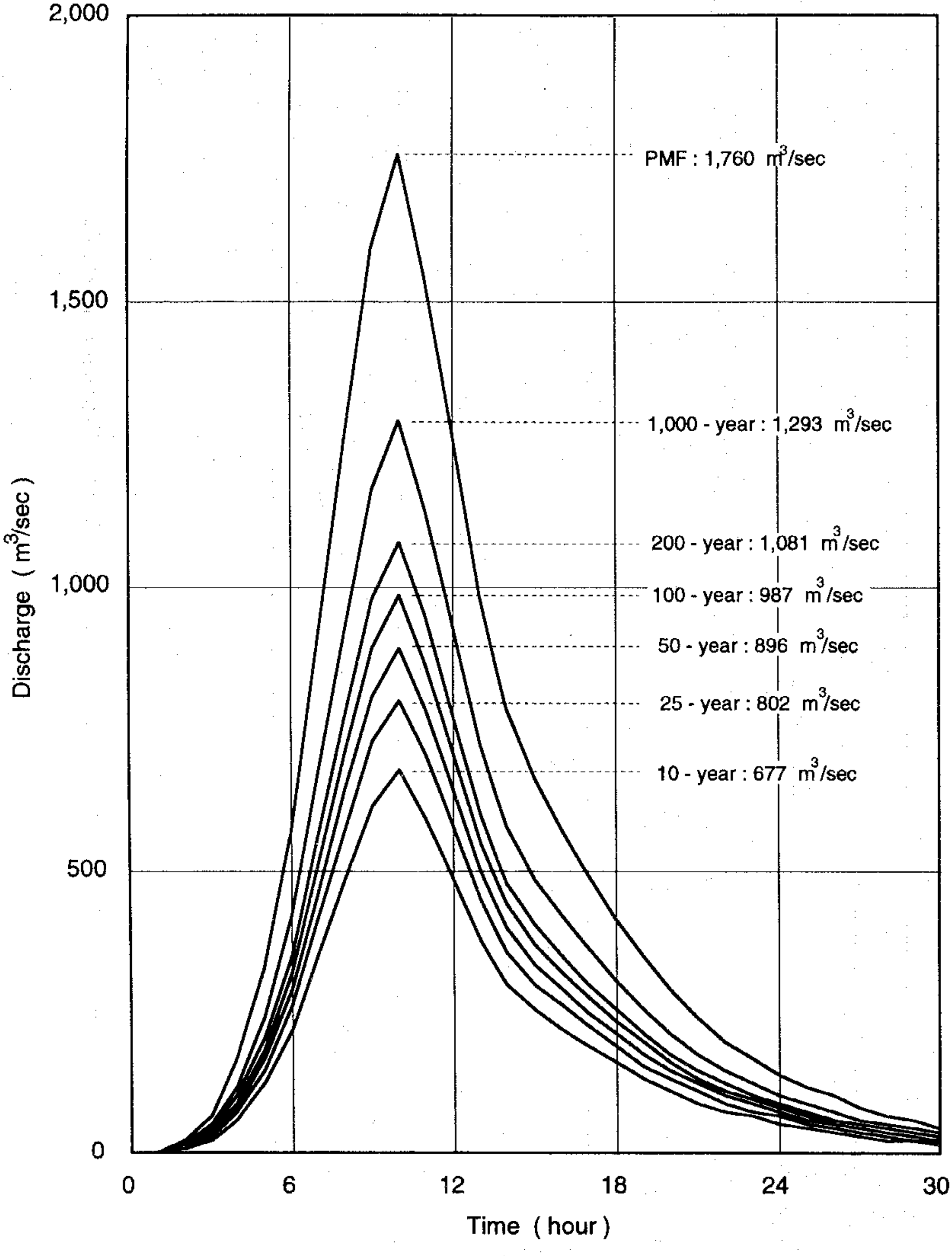


Figure A.2.16 Probable Floods at Proposed Dam Site

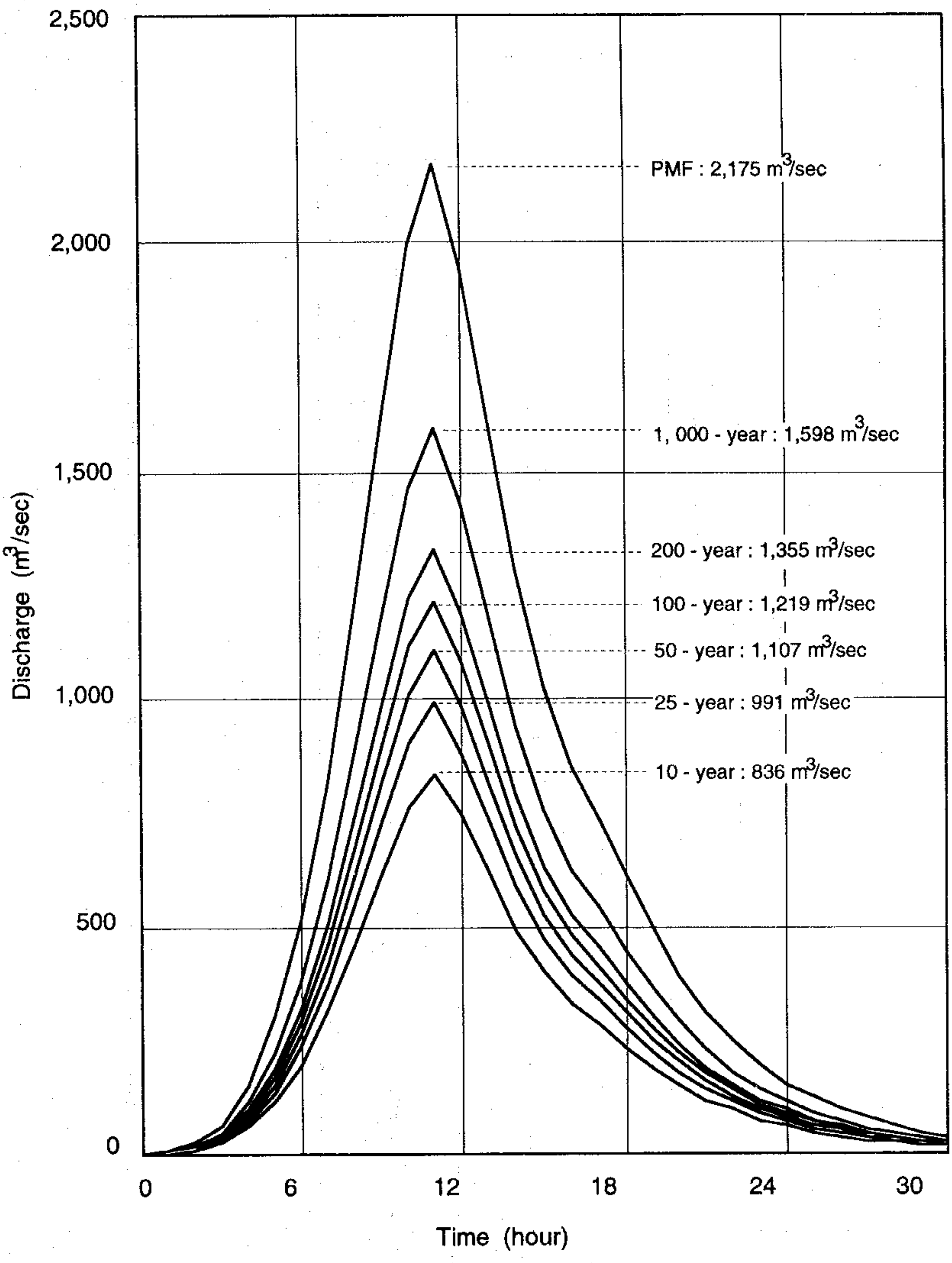


Figure A.2.17 Probable Floods at Proposed Weir Site (Before dam construction)

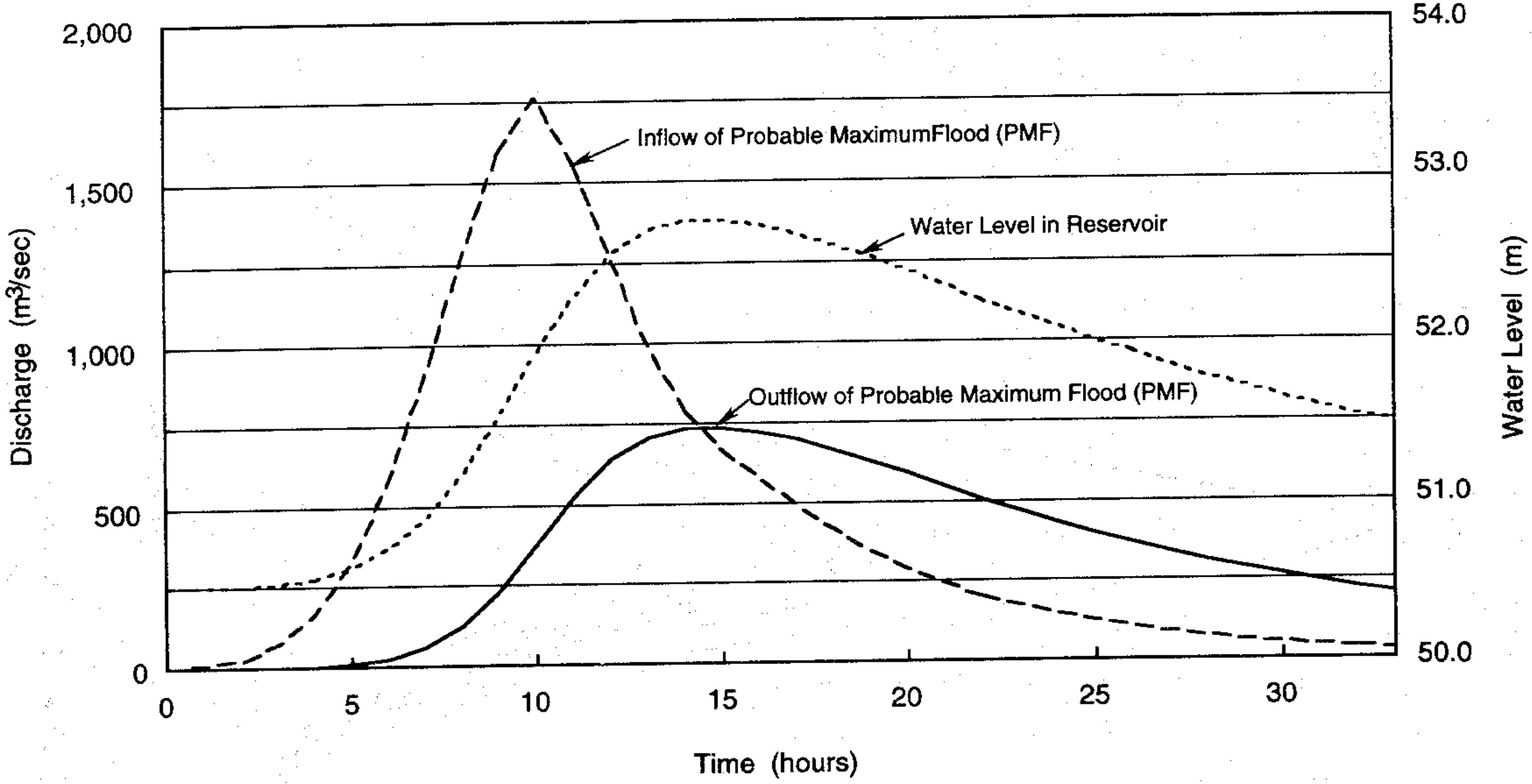


Figure A.2.18 Storage Effect of Reservoir against Probable Maximum Flood

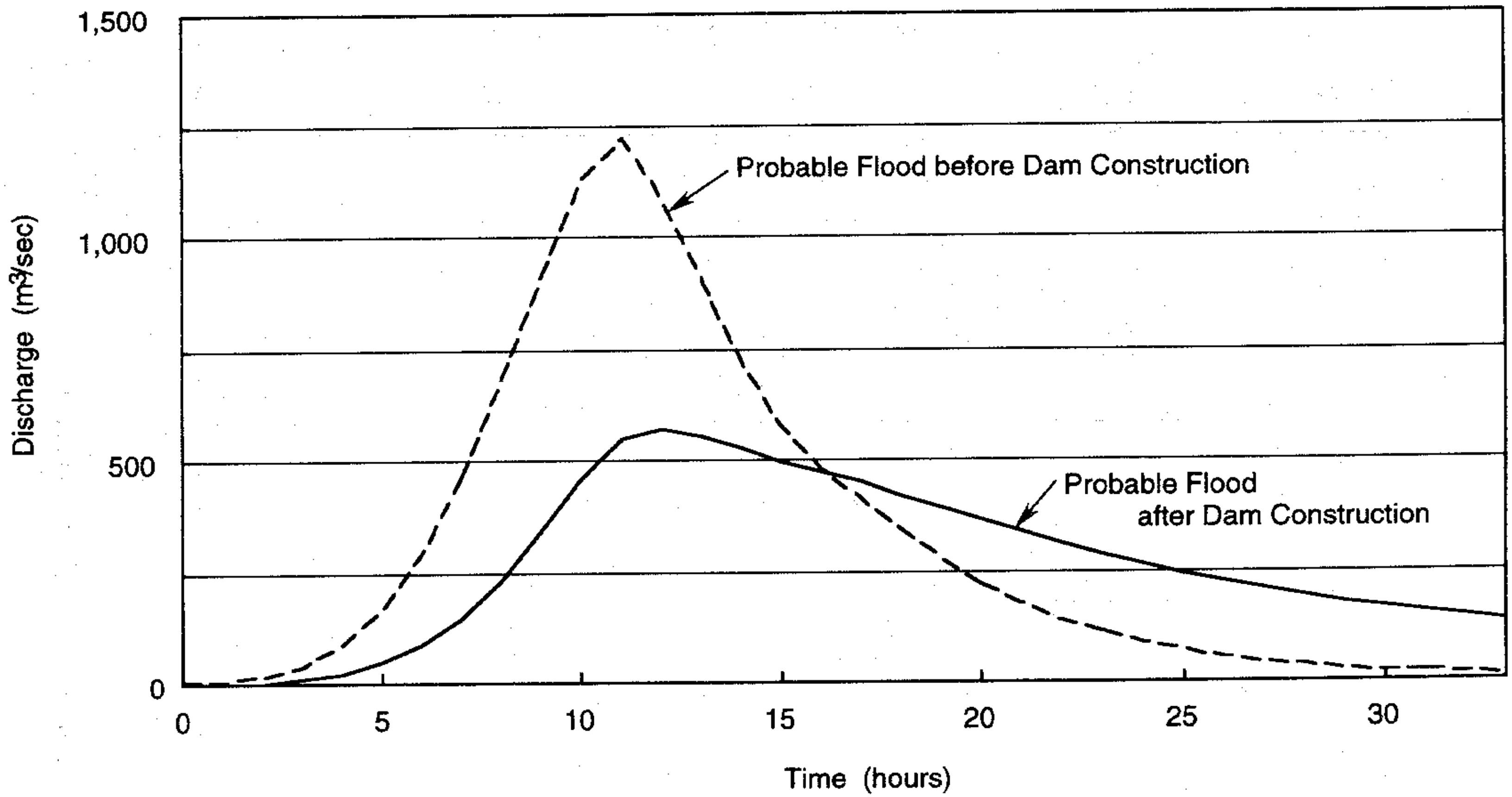


Figure A.2.19 Probable Flood Discharge at Proposed Weir Site (100 Year Return Period)

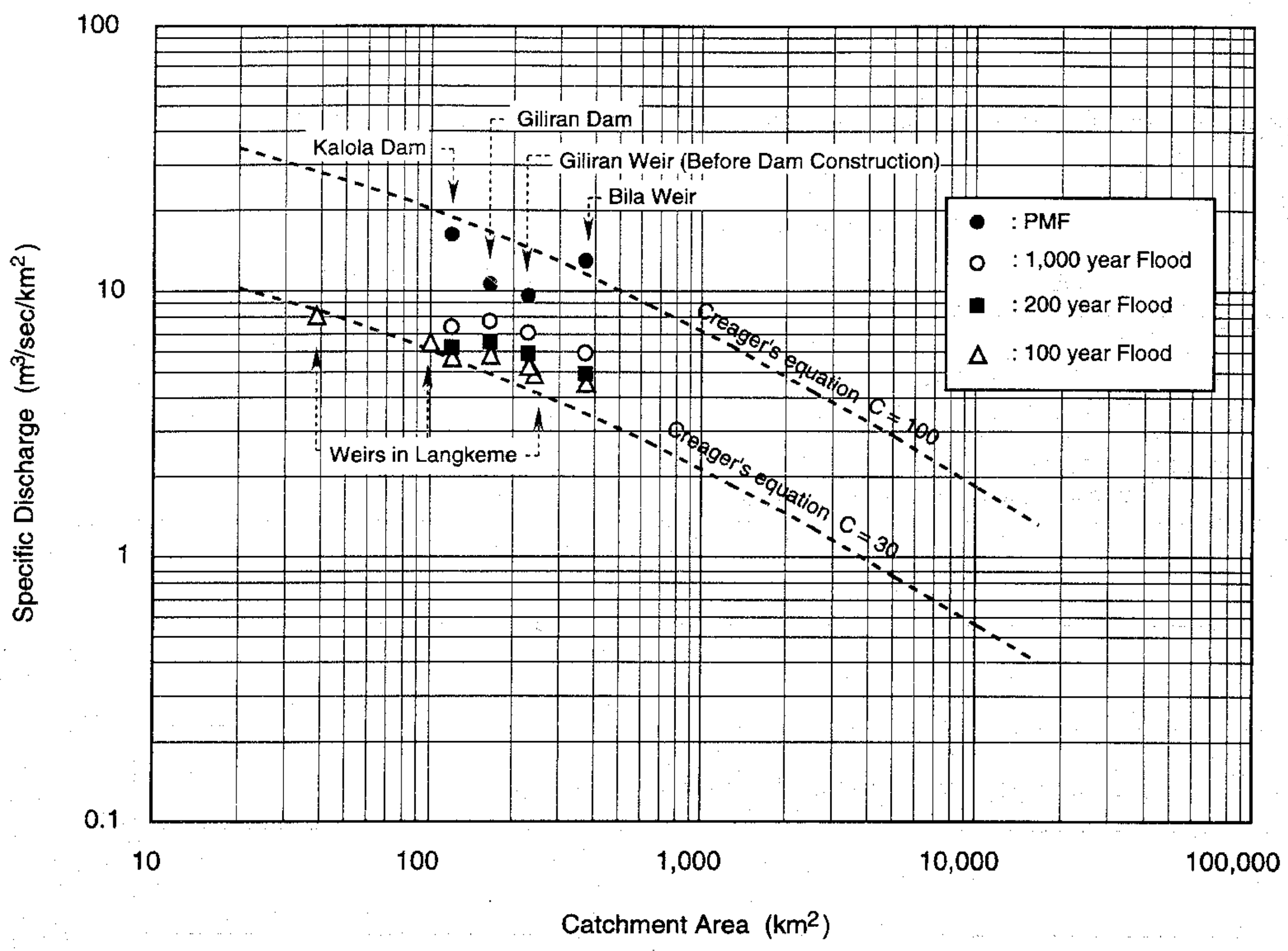


Figure A.2.20 Specific Discharge of Design Flood in South Sulawesi

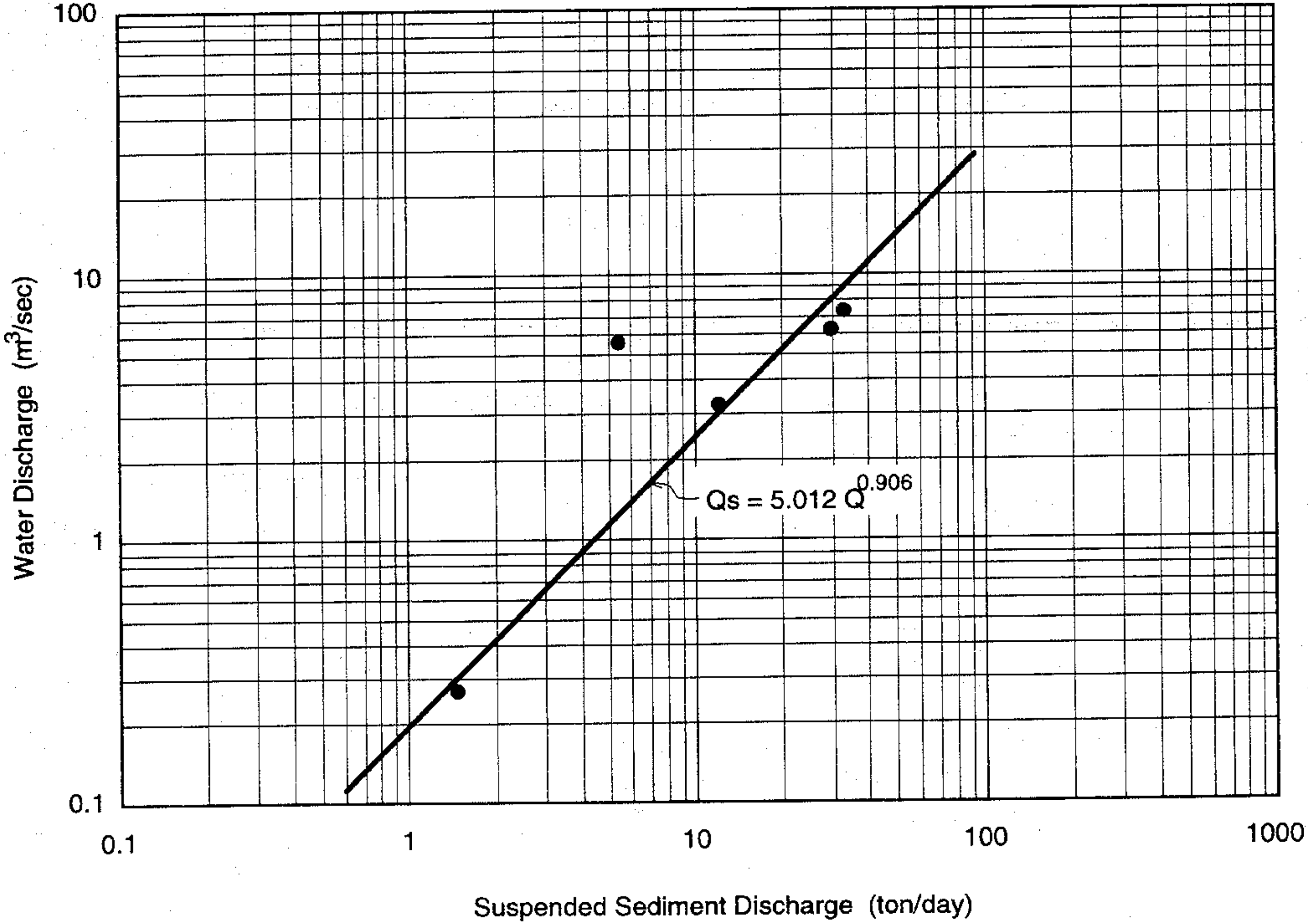


Figure A.2.21 Sediment Rating Curve for Gilirang River at Gilirang Gauging Station

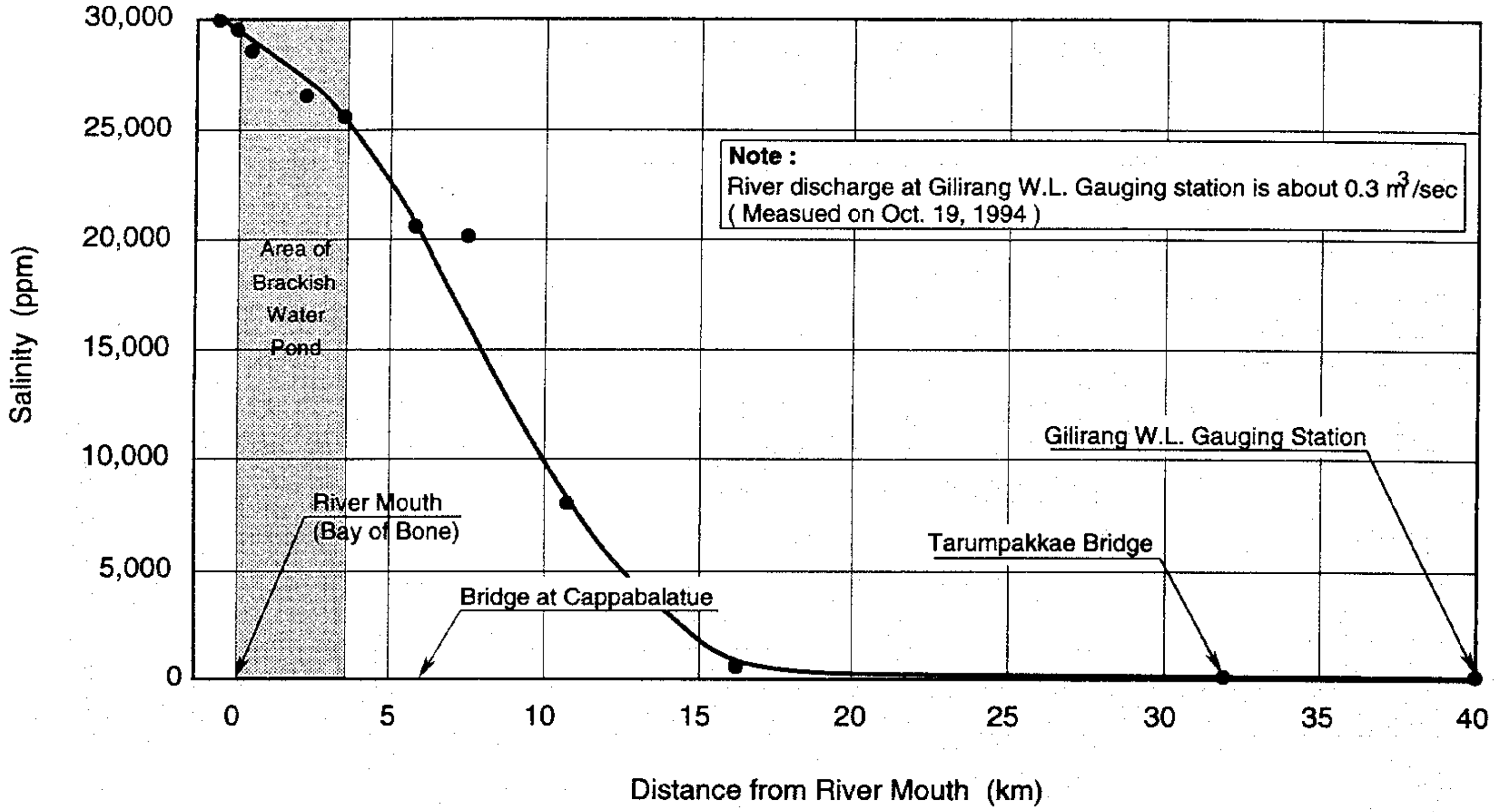


Figure A.2.22 Variation of Salinity in the Gilirang River

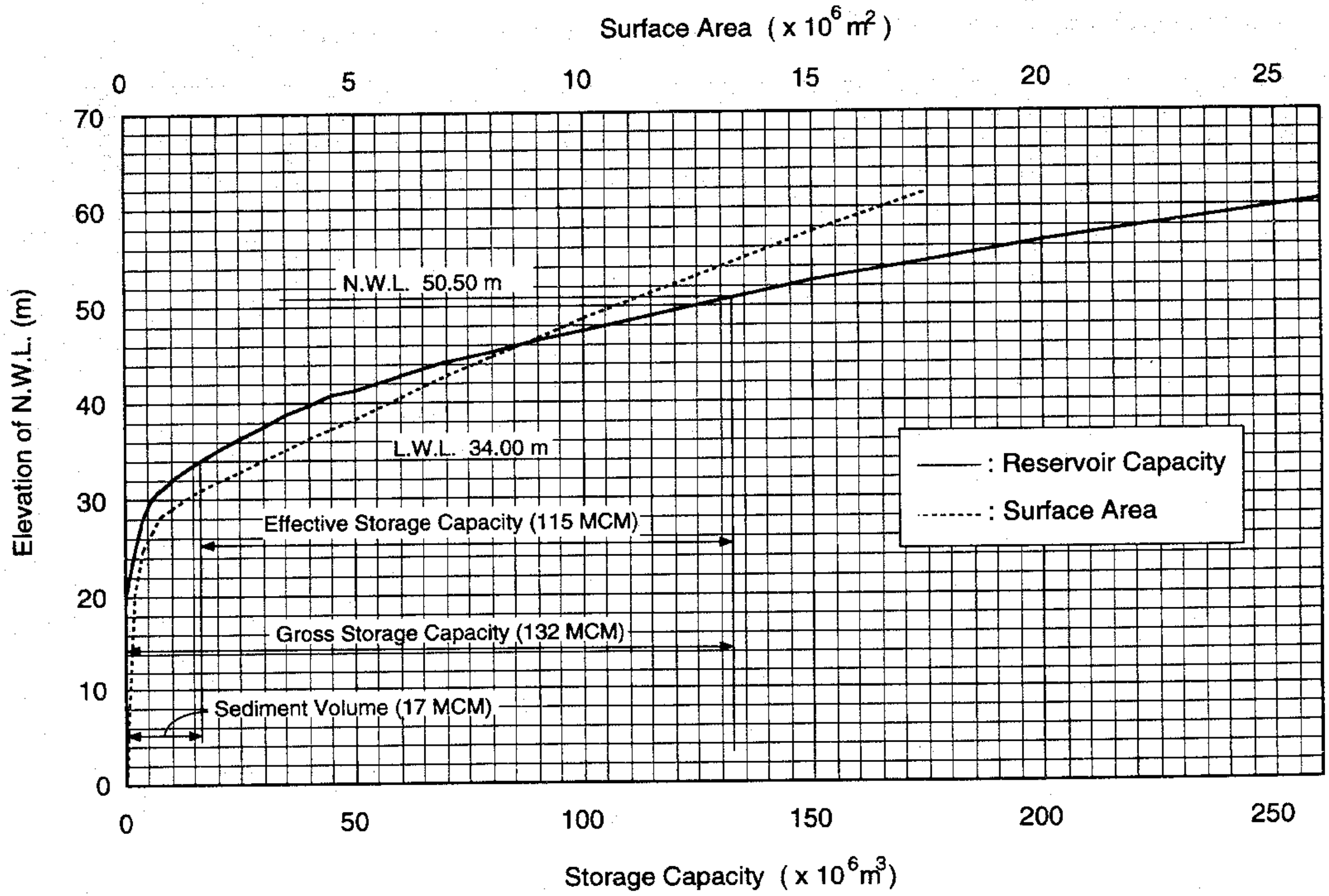


Figure A.2.23 Relations between Normal Water Level and Storage Capacity or Surface Area

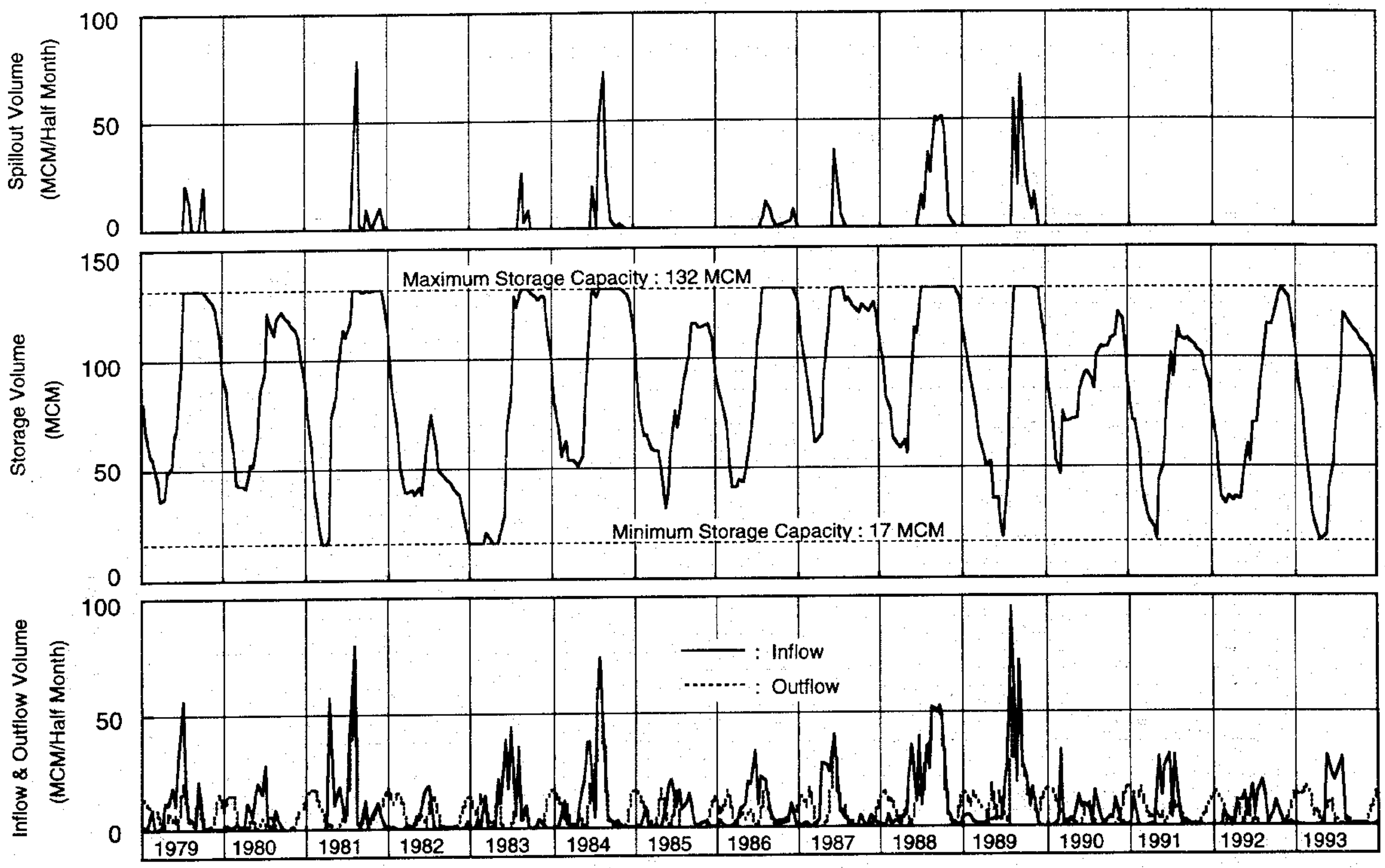


Figure A.2.24 Result of Water Balance Study in the Paselloreng Dam (Alternative I)

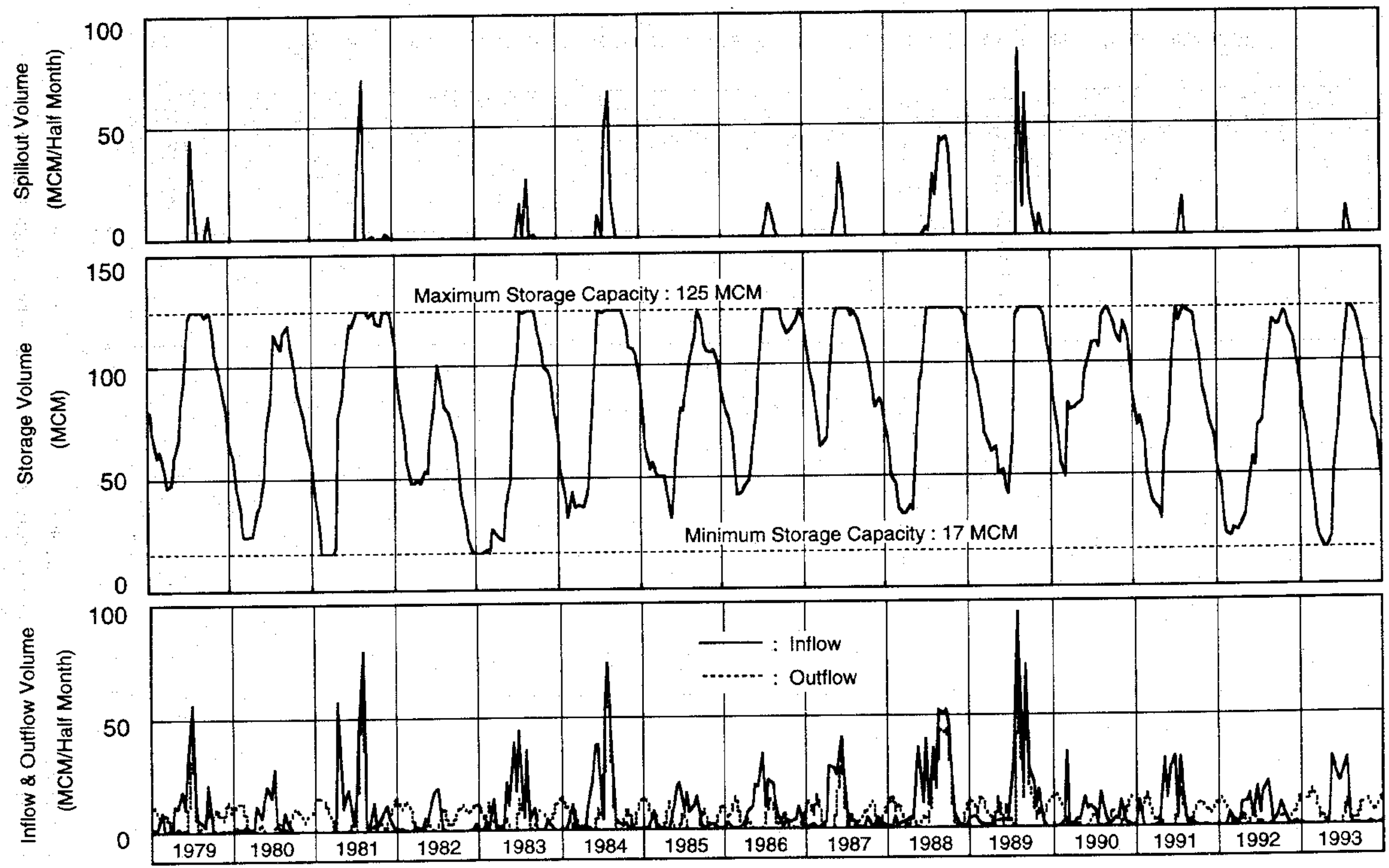


Figure A.2.25 Result of Water Balance Study in the Paselloreng Dam (Alternative II)

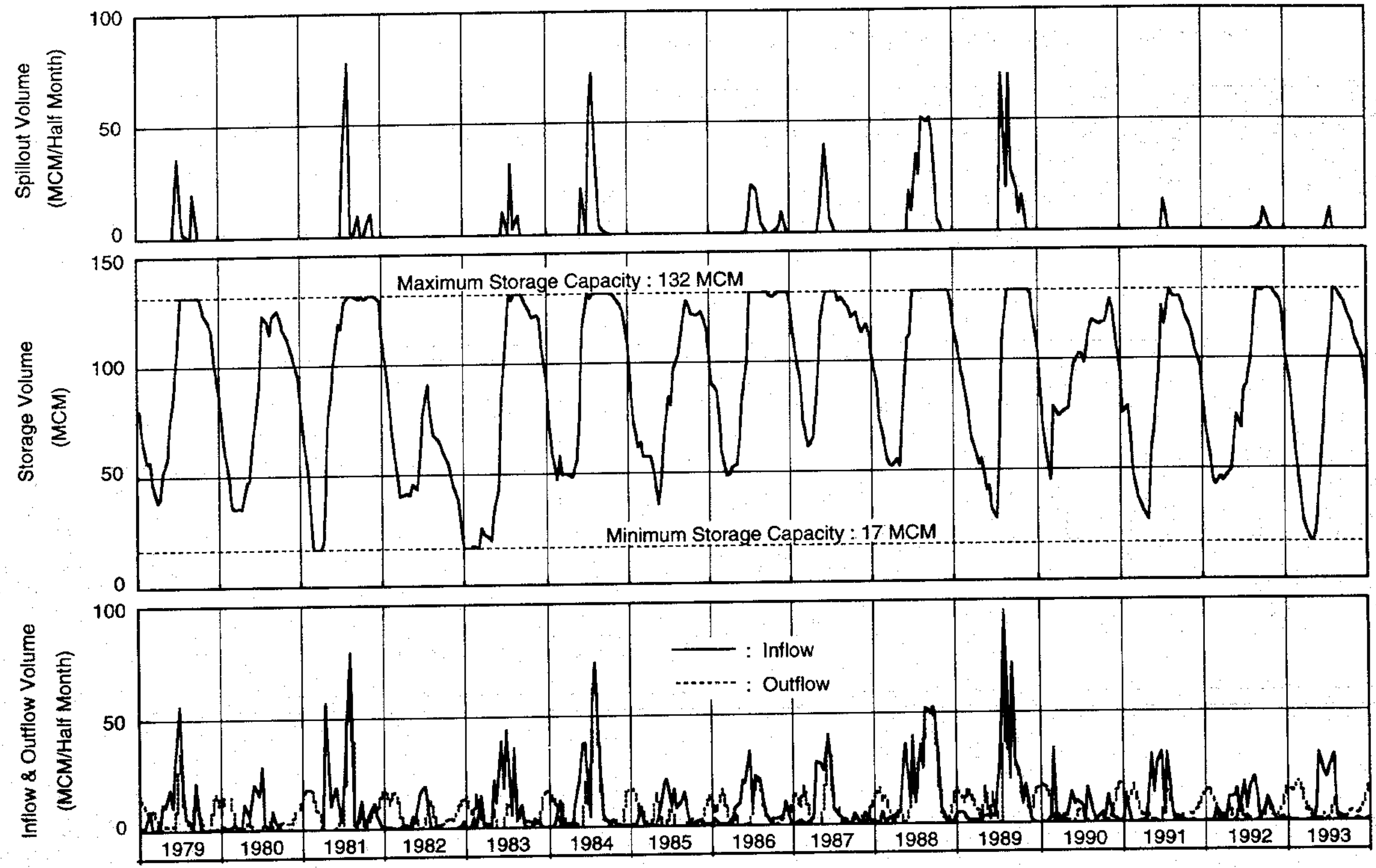


Figure A.2.26 Result of Water Balance Study in the Paselloreng Dam (Alternative III)

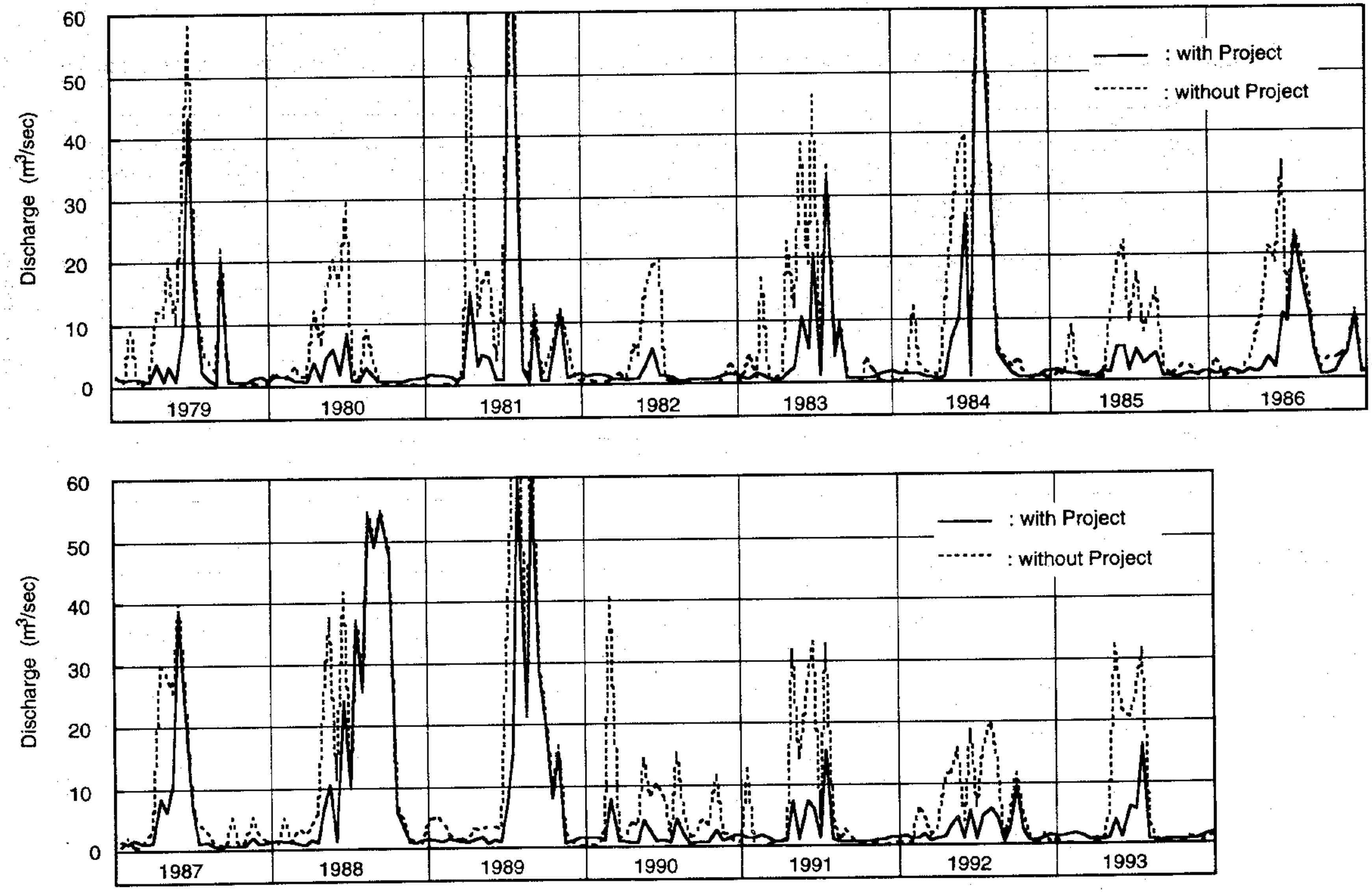


Figure A.2.27 Variation of Discharge at Proposed Weir Site with and without Project Condition (Alternative III)

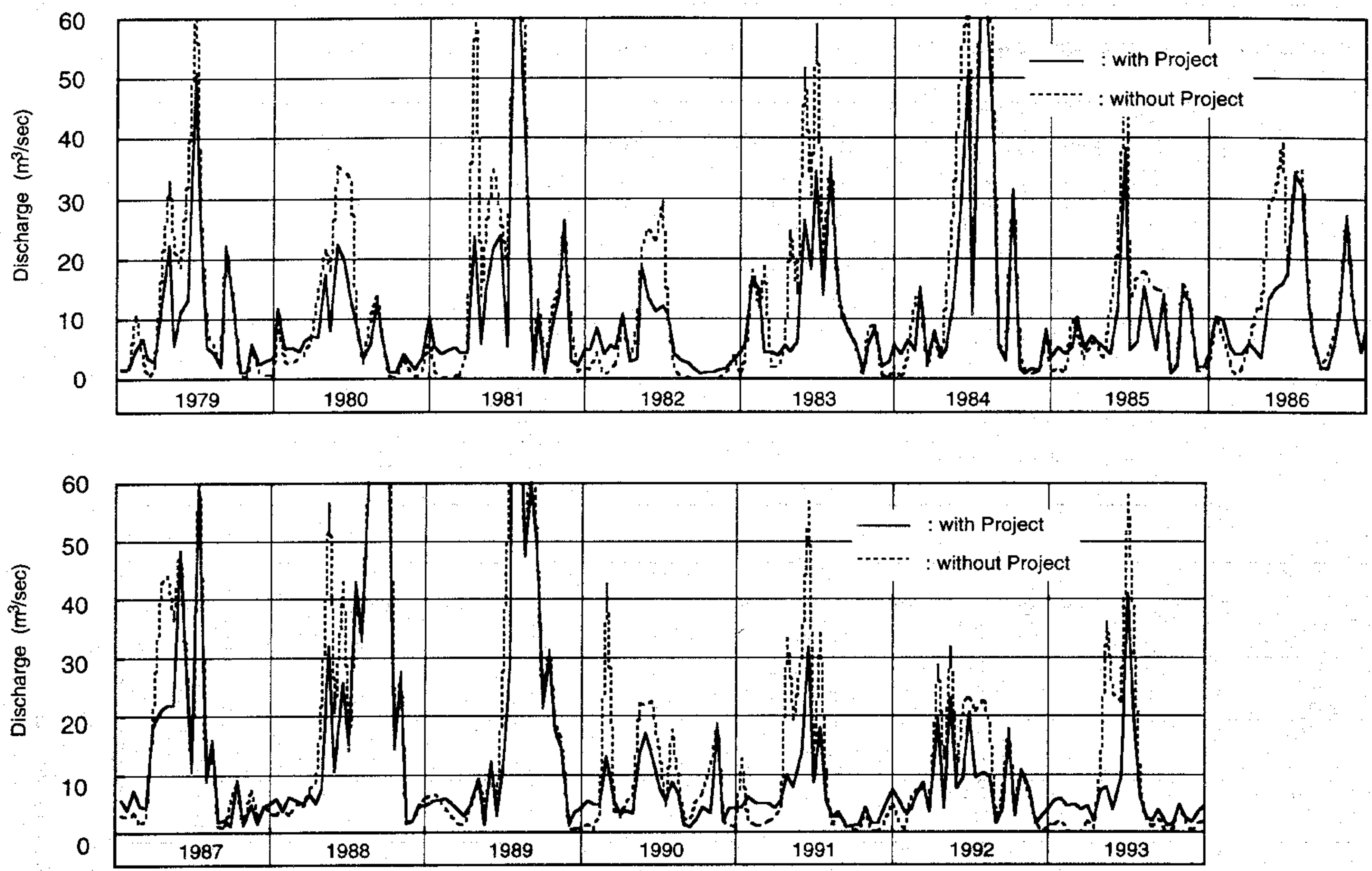


Figure A.2.28 Variation of Discharge at River Mouth of Gilirang River with and without Project Condition (Alternative III)

ANNEX 3

DAM AND WEIR

ANNEX 3 DAM AND WEIR

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ANNEX 3 DAM AND WEIR

1. DESIGN OF DAM

1.1 Basic Consideration for the Design

Design of the dam is made based on the results of investigations, studies and analysis on topography, geology and soil mechanics in and around the proposed dam site and hydrology of the Gilirang river basin, which are described in respective Annex. Basic considerations to the design are mentioned hereinafter.

1.1.1 Design Standards or Criteria on Dam

Design standards or criteria on dam is not enacted in Indonesia except for seismic coefficient and design flood discharge of closure dike and diversion channel. The seismic coefficient is enacted in "Structural Parameters" of DGWRD Design Standards and the design flood discharge is enacted in "Headworks" of DGWRD Design Standards. Therefore, design of dam is based on Japanese Criteria except for the seismic coefficient and the design flood discharge.

1.1.2 Topography and Geology

(1) General

The proposed dam site is located about 10 km upstream from Gilirang village, and forming narrow gorge. The ratio between dam crest length and dam height roughly ranges from 6 to 7. River bed slope is rather gentle as 1/1,600. Selection of dam type would not be affected by the topographic condition thereabouts.

Foundation rock in and around the proposed dam site is configured by conglomerate and sand stone originating Tertiary Pliocene sedimentary rock. Compressive strength of basement rock is inferred 100 kgf/cm^2 to 300 kgf/cm, and river bed deposits and overburdens thereabouts are rather thin.

According to drilling log, lugion values of basement rock would be rather small or less than about 5 Lu at layer deeper than 15 m and ranges from 10 to 30 Lu at surface layer shallower than 15 m. Judging from the lugion value at the dam site, the foundation can be easily improved by proper treatment. Meanwhile, bearing capacity and shearing strength at the foundation would be rather enough for construction of both concrete gravity and fill type dams of about 40 m high. The ground water level of left ridge is higher than normal water level at down stream dam axis. At both ridge of upstream dam axis and right ridge of down stream dam axis (refer to alternative study of dam axis hereinafter), the ground water level is rather deeper than normal water level, and so proper treatment shall be performed.

(2) Topographic limitation for design of dam

Topographic condition at proposed dam site was carefully examined based on 1/500 topographic map which was prepared in the phase (I) field survey and further detailed topographic survey in the phase (II) field survey. As the result of the examination, following limitations for design of dam dimension were detected.

1) The lowest elevation of narrow ridge on the right bank beside proposed dam site is EL. 55.0 m. Accordingly, a saddle dam is indispensable if the dam crest elevation is more than EL. 55.0 m. Furthermore, the lowest elevation of another narrow ridges at Desa Arajang is EL. 56.7 m. Therefore, in case that the dam crest elevation become higher than EL. 56.5 m, the saddle dam section and numbers suddenly increase (Refer to Figure A.3.1).

2) If the elevation of non-overflow crest is higher than that of the narrow ridge, the possibility of erosion of embankment increases due to wave action in case of flood, and so the safety of the dam would deteriorate at maintenance.

1.1.3 Optimum Scale of Dam

Considering the topographic limitation of dam site, economical investigation for dam scale are carried out as shown in Figure A.3.2, with assumption that the dam type is rockfill type. The incline of the curve of construction cost becomes more steep as the crest elevation increases and an inflection point is between EL. 56.0 m to 57.0 m. Furthermore, unit cost (Total cost/Gross storage capacity) is lowest at about EL. 56.5 m. In due consideration of the above investigation, it is recommended that the maximum crest elevation would be EL. 56.5 m.

1.1.4 Construction Materials

Laboratory rock test clarifies that the basement rock would be available for rock materials for rock fill type dam. But, this rock is unsuitable for concrete aggregate due to lack of hardness which is medium to soft.

Weathered mudstone crops out sporadically around the dam site. The laboratory test also clarifies that this materials would be available for impervious materials for fill type dam. According to the results of field investigation, this mudstone is rather widely distributed, and seems to be enough for required volume of the fill type dam.

Following the results of the laboratory tests and field investigation, availability of construction materials for respective type of the dam are as follows:

Dam Type	Material	Availability
Fill Type Dam	Rock materials	Available at site and enough in quantity and quality
	Impervious materials	- ditto -
Concrete Gravity Dam	Concrete aggregate	Unavailable at site

1.1.5 Seismic Coefficient

Seismic coefficient is calculated by the following formulas authorised by DPMA:

$$ad = N\,(ac \cdot z)^{m}$$

$$k = ad/g$$

where,
- ad : shock acceleration in cm/s^2
- n, m : coefficient for soil type (n = 2.76, m = 0.71)
- ac : basic shock acceleration in cm/s^2 (ac = 160 cm/s^2)
- k : earthquake coefficient
- g : acceleration of gravity in cm/s^2 (g = 980)
- z : factor depending on geographic position (z = 1.56) (Refer to Figure A.3.3)

$$ad = 2.76 \times (160 \times 1.56)^{0.71} = 139 \ m/s^2$$

$$k = 139/980 = 0.14$$

Furthermore, making reference to k = 0.15 for design of the Kalola dam, the seismic coefficient of 0.15 is applied for design of the proposed dam.

1.2 Selection of Dam Axis and Dam Type

Comparative study is made on both dam axis and dam type in order to establish the optimum dam plan.

1.2.1 Alternative Plan for Dam Axis and Dam Type

Two alternatives for dam axis are picked out and compared from technical and economical viewpoints. Alternative 1 (or upstream axis) was initially picked out in the Master Plan Study and geological investigation was made by DPMA. Meanwhile, Alternative 2 (or downstream axis) is newly picked out by the Study Team. Locations of the alternatives are shown in Figure A.3.1. Furthermore, two types of dam consisting of fill dam and concrete gravity dam are combined with the above two alternatives. Comparative study is carried out on condition that the maximum storage capacity is fixed. As the maximum crest elevation is EL. 56.5 m, the normal water level for this comparative study is fixed at EL. 50.5 m considering the freeboard of fill dam is 6.0 m including the overflow depth at spillway and wave height. The maximum storage capacity in this condition is fixed at 132 MCM for both alternatives based on H-V curve of the reservoir (See Figure A.3.4).

The general plans of fill type dam for Alternative 1 (Upstream axis) and Alternative 2 (Downstream axis) are shown in Figure A.3.5 and A.3.6 respectively. Dimensions of the dam and reservoir for alternative study are summarized below:

(1) Maximum storage capacity	:	132 MCM
(2) Effective storage capacity	:	115 MCM
(3) High water level	:	EL. 53.8 m
(4) Normal water level	:	EL. 50.5 m
(5) Low water level	:	EL. 34.0 m
(6) Design flood discharge		
- Dam, Spillway	:	1,300 m^3/sec
- Energy dissipater	:	1,000 m^3/sec
- Diversion tunnel	:	680 m^3/sec
(7) Catchment area	:	169 km^2
(8) Sediment volume	:	17 MCM

Following conditions are given to the respective type of dam in consideration of the site conditions.

(1) Fill type dam

1) Typical cross section

Dam type would be determined to be central impervious rockfill dam, as rock material is obtained in large quantities at the proposed dam site.

As a result of laboratory test, internal friction angle of rock materials are 43 degree to 45 degree. Design shear strength would be determined at 40 degree, in consideration of fluctuate of quality in construction work.

Slope of embankment would be determined at 1 to 3.0 for upstream 1 to 2.1 for downstream.

2) Excavation line

As a result of seismic prospecting, the excavation line would be determined as follows ;

a) Impervious portion : layer of third velocity (= 1.3 to 1.5 km/sec : CM class)

b) Pervious portion : layer of second velocity (= 0.8 to 0.9 km/sec : CL class)

3) Foundation treatment

Foundation treatment measures would be grouting method. Depth of injection hole would be calculated using the following formula proposed by Simonds.

$$d = (h/3) + c$$

where h : reservoir storage depth
c : constant, generally 5 to 25 m (mean 15 m)
$d = 42/3 + 15 = 30$ m (river bed portion)

This range of treatment is satisfied at more than 5 Lu. Injection holes of curtain grouting are arranged in two lines with 2.0 m intervals. Injection holes of blanket grouting are arranged in 4 line with intervals of 3.0 m.

Treatment range of right ridge is as far as spot of drilling hole GB-1. Treatment range of left ridge is as follows ;

a) Downstream dam axis :
Treatment range would be determined as far as the point of intersection of normal water level and ground water level.
b) Upstream dam axis :
Treatment range would be determined to satisfy creep ratio (=18) for silt or fine sand proposed by Bligh.(as far as 230 m from abutment)
c) Injection holes are arranged in one line with interval of 1.5 m as lugion values of left ridge are from 3 Lu to 6 Lu.
d) Depth of injection hole is as far as layer of fourth velocity (= EL. 25.0 m).

4) Diversion tunnel

Diversion tunnel for design discharge is as follows :

a) Upstream dam axis :

diameter : 6,000 mm x length : 350 m x 1 line
diameter : 6,000 mm x length : 370 m x 1 line

b) Downstream dam axis :

diameter : 6,000 mm x length : 250 m x 1 line
diameter : 6,000 mm x length : 300 m x 1 line

5) Spillway

Design discharge is 1,300m^3/sec, and so overflow depth and length of weir would be determined as follows :

Overflow depth = 3.3 m
Length of weir = 101 m

6) Crest elevation of fill type dam

Crest elevation would be determined at EL. 56.50 m in consideration of overflow depth and wave height and dam crest protection.

(2) Concrete gravity dam

1) Typical cross section

Slope angle would be determined at 1 to 0 for upstream and 1 to 0.8 for downstream. Slope angle of fillet would be determined at 1 to 1.0 for upstream.

2) Excavation line

Excavation line would be determined at layer of fourth velocity (2.5 to 2.7 km/sec)

3) Foundation treatment

Foundation treatment is determined as same as method of fill type dam.

4) Crest elevation

Crest elevation would be determined at EL. 55.0 m taking into account wave height.

5) Diversion tunnel

Design flood would be determined at half of fill type dam, because the concrete gravity type can bear to overflow during construction.

a) Upstream dam axis

diameter : 6,000 mm x length : 350 m x 1 line

b) Downstream dam axis

diameter : 6,000 mm x length : 250 m x 1 line

1.2.2 Result of Comparison

The fill type dam and the concrete gravity dam are compared at both alternative sites based on the estimated costs for construction as shown below:

Alternative 1 (Upstream dam axis)

1) Fill type dam

Crest elevation : EL. 56.50 m, NWL : EL. 50.5 m

Works	Quantity	Amount (1,000 Rp.)
Embankment	384,000 m^3	6,528,000
Foundation treatment	14,400 m	1,310,400
Diversion tunnel	26,000 m^3	7,566,000
Spillway	30,000 m^3	9,690,000
Plug works	2 set	764,000
Intake structure	1 set	7,000,000
Total cost		32,858,400

2) Concrete gravity dam

Crest elevation : EL. 55.1 m, NWL : EL. 50.5 m

Works	Quantity	Amount (1,000 Rp.)
Concrete works	38,000 m^3	41,400,000
Foundation treatment	14,400 m	1,310,400
Diversion tunnel	12,700 m^3	3,695,700
Plug works	1 set	382,000
Intake structure	1 set	4,700,000
Total cost		51,488,100

Alternative 2 (Downstream dam axis)

1) Fill type dam

Crest elevation : EL. 56.50 m, NWL : EL. 50.5 m

Works	Quantity	Amount (1,000 Rp.)
Embankment	538,000 m^3	9,146,000
Foundation treatment	9,900 m	900,900
Diversion tunnel	20,000 m^3	5,820,000
Spillway	30,000 m^3	9,690,000
Plug works	2 set	764,000
Intake structure	1 set	7,000,000
Total cost		33,320,900

2) Concrete gravity dam

Crest elevation : EL. 55.1 m, NWL : EL. 50.5 m

Works	Quantity	Amount (1,000 Rp.)
Concrete works	212,000 m^3	63,600,000
Foundation treatment	9,900 m	900,900
Diversion tunnel	9,000 m^3	2,619,000
Plug works	1 set	382,000
Intake structure	1 set	4,700,000
Total cost		72,201,900

The tables show that the fill type dam is in lower cost at both alternative sites, and upstream dam axis is in lower cost in both dam types.

Construction of concrete gravity dam would be technically feasible taking foundation condition at the proposed dam site into account. However, fill type dam is more safe and recommendable, because the basement rock at dam site originates from the Tertiary formation. Thus, the fill type dam at upstream site is proposed to be selected.

1.3 Design of Dam

1.3.1 Dimensions of Dam

(1) Storage capacity (Refer to Figure A.3.4)

1) Maximum storage capacity

As a result of water balance study, the maximum storage capacity is fixed at 132 MCM.

2) Dead storage capacity

Dead storage capacity would be determined to be 17 MCM based on the estimate of sediment volume for 100 years.

3) Effective storage capacity

Effective storage capacity is 115 MCM as the difference of the maximum storage capacity and the dead storage capacity.

(2) Water level

1) Normal water level

Based on H-V curve shown in Figure A.3.4, normal water level which produces maximum storage capacity is EL. 50.5 m. Submerged area with this normal water level is 11.0 km^2 as shown in Figure A.3.7.

2) Low water level

Low water level is fixed at EL. 34.0 m from the dead storage capacity.

3) High water level

Overflow depth of 3.3 m is assumed in consideration of magnitude of flooding. Thus, high water level is set out EL. 53.80 m by adding the overflow depth to the normal water level.

(3) Elevation of non-overflow crest

Elevation of non-overflow section is determined considering normal water level, high water level, height of wave by wind, and wave movement by earthquake.

1) By applying the S.M.B method, wave run-up height (R), fetch (F), wind velocity (V), wave height (Hw) and wave length (L) are determined. (Refer to Figure A.3.8) Slope roughness is determined by applying Saville method:

Fetch	: 6 km
Wind velocity	: 30 m/sec
Slope of dam	: 1 to 3.0 vertical to horizontal
Slope protection	: Rip rap

Thus, the wave height by wind (Hw) is estimated at 1.4m.

2) In order to estimate wave height from reservoir surface caused by earthquake, Sato formula authorised in Japan is applied as follows :

$He = 1/2 \times k \cdot t/\pi \cdot \sqrt{gHo}$

where,
He : Wave height from reservoir surface by earthquake (m)
K : Design seismic coefficient, K=0.15
t : Seismic period, t=1.0 second
Ho : Depth of reservoir at normal water level (m), Ho = 30.0 m,
g : acceleration of gravity, $g = 9.8m/sec^2$

Thus, $He = 1/2 \times (0.15 \times 1.0) / \pi \times \sqrt{9.8 \times 30.0} = 0.41$ m

3) No surcharge water level would be considered, because the proposed dam does not function regulation, but water use. The elevation of non-overflow section would be determined to be EL. 56.20 m by adding the wave height by wind, wave height caused by earthquake, and some allowance to the high water level (HWL). (Refer to Table A.3.1)

(4) Elevation of dam crest

Furthermore, the dam crest would be paved against erosion with a thickness of 0.3 m ; the elevation of dam crest, thus, is determined to be EL. 56.5 m by adding the thickness of pavement to the elevation of non-overflow section.

(5) Height of dam

Dam foundation would be necessarily excavated up to EL. 12.0 m in order to obtain layer of the third velocity rock (CM class rock) for the impervious zone at the basement ; the height of dam from the excavation line of the foundation would be 44.5m. (Refer to Figure A.3.9)

1.3.2 Layout of Typical Cross Section

(1) Type of rock fill dam

Fill dam is classified into two typical type in cross section, i.e. homogeneous type and zone type. Generally, to get the same value of safety factor on stability analysis, cross section of zone type is smaller than that of homogeneous type, since residual pore water pressure in the zone type is usually lower than that in the homogeneous type in case of rapid draw down.

As previously mentioned, though proper and sufficient rock materials are obtainable in the vicinity of the proposed dam site; proper impervious materials are rather limited and insufficient for the requirement for homogenous type. In order to effectively use the available proper materials in the vicinity of the dam construction site, the zone typed rockfill dam is economically and technically the optimal, and would be selected for the dam plan in this project.

(2) Zoning

Impervious zone of rockfill dam is divided into three types ;

a) Central impervious zone type,
b) Inclined impervious zone type, and
c) Surface impervious zone type.

Among them, the inclined and the surface impervious zone types have a more gentle upstream slope than the central impervious zone type, because the critical sliding line may intersect the impervious zone in case of the inclined and surface impervious zone types. Thus, the central impervious zone type can give the smallest embankment volume among three types mentioned above.

In case that workable days for embankment would be limited due to weather condition, the inclined and surface impervious zone types are sometimes prior to the central impervious zone type, because the embankment work of the downstream rock zone would be made in advance of the embankment work of upstream impervious zone. The workable days at the proposed dam site, however, is relatively sufficient, making reference to the construction date obtained at the Kalola dam and the Bila intake weir. In view of the workable days, the inclined and surface impervious zone types are not superior to the central impervious zone type.

Foundation treatment for the inclined and surface impervious zone types is usually wider than that for the central impervious zone type. In this view, the central impervious zone type is superior to the other two types. Following the discussion above, the central impervious zone type would be selected for the proposed dam. The following are the embankment materials for respective zones ;

a)	Impervious zone :	Soil materials (weathered mud stone)
b)	Pervious zone :	Conglomerate and sand stone
c)	Filter zone :	Materials to be purchased and transported from Bila river

(3) Typical cross section

Figures A.3.10 and A.3.11 illustrate the typical cross section of the proposed dam and the cross section of dam axis. Dimensions of typical cross section are summarized below:

1) Crest width of dam :	10.00 m for workability (Refer to Figure A.3.12)
2) Thickness of impervious zone :	Greater than 50 % of water depth at excavation line, and 4.0 m at crest ; slope of 1 to 0.2
3) Thickness of filter zone :	2.0 m for workability
4) Slope protection for upstream:	Riprapping by hard and durable rock which is transported from outside the dam site.
5) Slope of embankment :	1 to 3.0 for upstream and 1 to 2.1 for downstream.

1.3.3 Stability Analysis of Dam

(1) Stability analysis by surface sliding method

1) Surface sliding for downstream

$$Fs = ((1 - m \cdot K)/(m + K))\tan Ø'$$

2) Surface sliding for upstream

$$Fs = \{((1 - m \cdot K(r_{sat}/r')) \cdot \tan Ø'\}/(m + (r_{sat}/r') \cdot K)$$

where,
Fs : Safety factor
Ø' : Internal friction angle
a : Angle of slope
m : slope = tan a
K : Seismic intensity
r_{sat}: Saturated unit weight of rock materials or sand and grovels
r' : Submerged unit weight of rock materials or sand and grovels
r^w : Unit weight of water.

(2) Design values of rock materials

Design unit weight is determined at compaction energy = 100 %

a) Dry unit weight : $1.70\ tf/m^3$
b) Specific gravity : 2.7
c) Void ratio : $e = Gg \cdot rw/rd - 1 = 0.59$
d) Saturated unit weight : $r_{sat} = (Gg + e)/(1 + e) \cdot rw = 2.07\ tf/m^3$
e) Submerged unit weight : $r_{sub} = r_{sut} - rw = 1.07\ tf/m^3$
f) Internal friction angle : Ø = 40°-00'

According to laboratory test, internal friction angle of rock materials are 43 degree to 45 degree. However design shear strength would be determined at 40 degree, take into fluctuate of quality at conservation work.

(3) Safety factor

The allowable safety factor should be greater than 1.20 in order to get a stability of embankment slope. As given below, both the proposed downstream and upstream slopes are fully stable.

For the proposed downstream slope of 1 to 2.1 : Safety factor of 1.244 under seismic intensity of 0.15

For the proposed upstream slope of 1 to 3.0 : Safety factor of 1.216 under seismic intensity of 0.15

1) In case of downstream : seismic intensity = 0.15

slope angle	Safety factor
1/1.80	1.090
1/1.90	1.143
1/2.00	1.194
1/2.10	1.244
1/2.20	1.293
1/2.30	1.341
1/2.40	1.388

2) In case of upstream : seismic intensity = 0.15

slope angle	Safety factor
1/2.70	1.134
1/2.80	1.162
1/2.90	1.189
1/3.00	1.216
1/3.10	1.241
1/3.20	1.260
1/3.30	1.290

1.4 Diversion Tunnel and Coffer Dam

1.4.1 Design of Diversion Tunnel

In due consideration of magnitude of flood discharge and scale of the proposed dam, river diversion would be made by diversion tunnel. Two barrelled tunnel with a diameter of 6.0 m would be proposed so as to smoothly divert such a intensive flood discharge. (Refer to Figure A.3.13)

1.4.2 Design Discharge for Coffer Dam

Diversion discharge is enacted as a flood discharge with a return period of 25 years unless risk considerations make another appropriate return period by Design Criteria of Headworks in Indonesia. However, design discharge would be determined a return period of 10 years taking into account that the discharge of 25 years recurrence is too large from economical viewpoint, and design discharge of Kalola Dam was determined 10 years recurrence, too. In this consideration, coffer dam would be designed on the basis of the flood discharge of 10 years recurrence which is estimated to be 680 m^3/sec by the Hydrological study in Annex 2.

Outlet invert of diversion tunnel would be set out to be El. 19.00 m which is 1.0 m higher than the ordinary water level of the river. Water level at the inlet side of the diversion tunnel, therefore, would be estimated to be El. 30.00 m on the basis of the flood discharge of 10 years recurrence and length and diameter of the diversion tunnel. Thus, crest elevation of the coffer dam would be set out to be El. 31.00 m adding 1.00 m of allowance.

For the sake of reduction of embankment volume for main dam and shortening of diversion tunnel, coffer dam would be involved into the main dam. Materials for coffer dam, therefore, would be as similar rock zone as the main dam. The coffer dam would be designed with a surface impervious zone type.

1.5 Saddle Dam and Cut-off Trench

1.5.1 General

Saddle dam or cut-off trench would be constructed at the narrow ridge on the right bank side, because the elevation of the right ridge is less than the crest elevation of main dam (EL. 56.5 m) in some places and surface layer of the narrow ridge is consist by high permeability materials.

Range of the saddle dam would be determined as for as 200 m from the spillway. Cut-off trench would be set at narrow ridge in Desa Arajang. Range of the cut-off trench would be determined at elevation of the ridge less than EL. 60 m, because thickness of permeability layer is evaluated about 5 m from drilling log. Length of cut-off trench is estimated about 650 m.

1.5.2 Typical Cross Section

(1) Saddle dam

1) Type of saddle dam
Type of saddle dam would be determined homogeneous dam, as the cross section can be rather small.

2) Excavation line
Depth of excavation line would be determined at 5 m, taking into account the thickness of the permeability layer.

3) Slope of embankment
Slope of embankment would be determined at 1 to 2.8 for upstream and 1 to 2.2 for downstream.

Typical cross section of the saddle dam is shown in Figure A.3.10.

(2) Cut-off trench

Depth of excavation line would be determined at 5 m taking into account the thickness of permeable layer. Width of cut-off trench would be determined at 5 m for workability.

1.6 Foundation Treatment

(1) Excavation line

As a result of seismic prospecting, the excavation line would be determined as follows :

a) Impervious portion : layer of third velocity (= 1.3 to 1.5 km/sec : CM class)

b) Pervious portion : layer of second velocity (= 0.8 to 0.9 km/sec : CL class)

(2) Grouting

Foundation treatment measures would be grouting method. Depth of injection hole would be calculated using the following formula proposed by Simonds.

$$d = (h/3) + c$$

where h : reservoir storage depth
c : constant, generally 5 to 25 m (mean 15 m)
d = 42/3 + 15 = 30 m (river bed portion)

This range of treatment is satisfied at more than 5 Lu. Injection holes of curtain grouting are arranged in two lines with 2.0 m intervals. Injection holes of blanket grouting are arranged in 4 line with intervals of 3.0 m.

Treatment of both narrow ridges is as follows:

a) Treatment range would be determined to satisfy creep ratio (= 18) for silt or fine sand proposed by Bligh, as shown in Figure A.3.10 & A.3.14.

b) Injection holes are arranged in one line with interval of 1.5 m as lugion values of left ridge are from 3 Lu to 6 Lu. Depth of injection hole is as far as layer of fourth velocity (= EL. 25.0 m).

1.7 Design of Spillway

1.7.1 Location of Spillway

Two alternative sites for spillway would be considered from topographic and hydraulic conditions, i.e. left bank side and right bank side close to the proposed dam axis. In case that spillway would be located at left bank side, stilling basin must be located across confluence of a small tributary from left river bank, resulting in technical troublesome and increase of additional cost. In the meantime, in case of the right bank location, all the spillway facilities can be smoothly laid out owing to topographic condition, and can be safely operated and better maintained owing to easy access from the main road. Thus, the right bank site would be selected for the location of spillway.

1.7.2 Layout of Spillway

(1) Design discharge

Design flood discharge of closure dikes is enacted as a flood discharge with a return period of 1,000 years by Design Criteria of Headworks in Indonesia.
Therefore, recurrence flood of 1,000 years would be applied for design of inlet of the spillway. For maintaining the safety of dam, the recurrence flood would be estimated without any consideration of retarding effect in the reservoir. Furthermore, the spillway capacity would be

checked by the probable maximum flood (PMF) on assumption that the PMF would be regulated in the proposed reservoir.

Recurrence flood of 100-year would be applied for design of energy dissipater of the spillway.

(2) Layout of spillway

1) Inlet :	Straight overflow type without crest gate
2) Guide Channel :	Side spillway type from topographic
3) Energy Dissipater :	Forced hydraulic jump type
4) Length of overflow weir :	The following formula is applied for calculation of length of weir. (Refer to Figure A.3.15)

$$Q = CLH^{3/2}$$

where
- Q : Spilling out discharge(m^3/sec), 1300m^3/sec
- C : Coefficient, 2.15
- L : Effective length of weir
- H : Overflow depth include. approach velocity head

The length of weir is determined at 101 m from topographic condition. Thus, overflow depth would be determined at 3.3 m based on the formula.

Overflow depth (m)	Effective length of the weir (m)
3.0	134
3.1	111
3.2	106
3.3	101
3.4	96

Structure of the spillway is shown in Figure A.3.16.

1.8 Design of Intake Structure

1.8.1 Selection of Location and Type

One of two temporary diversion tunnels would be converted into the intake structure because of economical viewpoint. Therefore, location of the intake structure would be determined at left side bank side, and type of the intake structure would be determined as an intake tunnel.

1.8.2 Capacity of Intake Structure

Capacity of the intake structure is added the function of outlet discharge for reservoir draw down. The intake capacity is determined depend on the discharge for reservoir draw down due to which is larger than that required for intake.

If the time required for draw down is assumed to be about 7 days, the mean discharge is as 115 MCM/(7 x 86,400 sec) = 190 m^3/sec. Mean velocity of discharge is determined 9 m/sec, and so diameter is decided as follows:

Area of cross section = 190 m^3/sec/9m/sec = 21 m^2 -----> Diameter = 5.2m

However, it is not practical to install the outlet valve of such large scale from an economic point of view. Therefore, the diameter of outlet valve would be fixed at 2,400 mm with condition that the time for draw down is allowed up to 30 days. The intake tunnel would be reinforced by concrete lining of 0.4 m thickness.

1.8.3 Layout of Intake Structure

(1) Intake section type

An inclined roller gate is set out at intake section from technical and topographic view points. Top portion of inclined gate would be set on embankment as the ground elevation of the point is lower than high water level.

(2) Energy dissipater

The energy dissipater is set out outside the intake tunnel. The energy dissipater of submerged type is adopted. Profile of the intake structure is shown in Figure A.3.17.

The general features of the Passelloreng Dam are summarized below:

Dam and reservoir

(a) General

- Catchment area	: 169 km^2
- Reservoir surface area at N.W.L.	: 11.0 km^2
- Storage capacity	
Maximum storage capacity	: 132 MCM
Effective storage capacity	: 115 MCM
Dead storage capacity	: 17 MCM
- Water level	
High water level	: EL. 53.8 m
Normal water level	: EL. 50.5 m
Low water level	: EL. 34.0 m

(b) Dam

- Type	: Rockfill dam having central impervious earth core
- Crest elevation	: EL. 56.5 m
- Dam height	: 44.5 m
- Crest length	: 230.0 m

(c) Spillway

- Type	: Non-gated side channel overflow weir
- Design discharge	: 1,300 m^3/sec
- Crest elevation	: EL. 50.5 m
- Crest length	: 101.0 m

(d) Diversion tunnel

- Type	: Pressured tunnel
- Design diversion discharge	: 680 m^3/sec
- Diameter	: 6.0 m

(e) Intake

- Design discharge : 13.5 m^3/sec

2. DESIGN OF WEIR

2.1 Basic Consideration for the Design

Design of the weir is carried out in accordance with the results of investigations, study and analysis on various items, which are described in respective Annex.

Design standard and criteria on headworks are enacted on December 1986 by DGWRD. In this Annex, the plan for the weir is made mainly based on the DGRWD Standard and the preliminary design for the weir is carried out mainly based on the Japanese criteria.

2.2 Selection of Intake System

In the study area, two alternative plans with intake weir facility can be considered as the possible intake systems which divert the irrigation water into main irrigation canals of the Project. The first alternative is to divert the water in the reservoir into the upstream of the tributary of the Gilirang river (Anaksali river) through intake tunnel and diversion canal, and the diverted water is taken into the main canal at the intake weir in the lower reach of the tributary (Refer to Figure A.3.18). The second alternative is to release the water in the reservoir again into the Gilirang river, and the released water is diverted into the main canals at the proposed weir in the lower reach of the Gilirang river (Refer to Figure A.3.19).

The main irrigation facilities for these two alternatives except for dam and canal facilities are as follows:

Alternative 1

1)	Intake tunnel	:	: Diameter : 2.8 m x Length : 500 m
2)	Diversion valve	:	: 1 No.
3)	Diversion canal	: Reservoir side	: 200 m
		Weir side	: 1,500 m
4)	Intake weir	: Crest width	: 65 m
		: Scouring sluice gate	: 2.0 m x 2.0 m x 4 Nos.
		: Intake gate	: 1.3 m x 2.0 m x 3 Nos.
			2.2 m x 2.0 m x 2 Nos.
5)	Intake canal	:	1,300 m
6)	Siphon	:	: 200 m

Alternative 2

1)	Intake weir	: Crest width	: 79 m
		: Scouring sluice gate	: 2.0 m x 2.0 m x 2 Nos.
			3.0 m x 2.0 m x 2 Nos.
		: Intake gate	: 1.3 m x 2.0 m x 2 Nos.
			: 2.2 m x 2.0 m x 2 Nos.

Considering above facilities, it is obvious that the cost for Alternative 1 will be very high comparing with that of Alternative 2, though the total area of gravity irrigation system increases about 200 ha. Therefore, the intake system, which to release the storage water again into the Gilirang river and to divert the water into the main canals at the weir in the lower reach is adopted in this Project.

which to take in the released water from the reservoir at the weir in the Gilirang river and divert into the irrigation canals, is adopted in this Project.

2.3 Selection of Weir Site

Possible site for the intake weir was examined through 1/5,000 topographic map from technical, economical and environmental points of view. In order to avoid the serious effect of back water to the dam, the design intake water level is fixed at EL. 18.0 m. Furthermore, for the effective use of runoff from residual catchment area between the dam and the weir, it is recommended that the weir should be constructed in the lower reach of confluence where the Anaksalo river joins the Gilirang river.

2.3.1 Possible Site for Intake Weir

In accordance with the consideration above, several weir sites came out. Figure A.3.20 shows four alternatives as the possible weir sites. The features for each alternative are summarized below :

(1) Alternative 1

The land space for the Coupure method, which is most common construction method for the weir in Indonesia, is large enough and the length of required embankment is shortest in the four alternatives.

(2) Alternative 2

This is only the alternative where the bed rock is appeared on the ground surface. So the weir can be constructed on the bed rock and the stability of the weir will be highest.

(3) Alternative 3

This alternative locates on straight portion of the Gilirang river, so the morphological condition is in good shape. However, long embankments are required to prevent the flood in the upper reach and this affects the houses near proposed weir site. The removal of more than ten houses (about one third of houses in Kampung Alusalo) is indispensable for the execution of this alternative and there is anxiety for environmental effect of construction to the residents.

(4) Alternative 4

This alternative has biggest amount of runoff water from the residual catchment area in the four alternatives. However, land space for construction is rather small for Coupure method. In addition, this alternative needs very long embankment to prevent the inundation of paddy fields due to flood.

2.3.2 Selection of Weir Site

Considering the features of alternatives, Alternative 1 and 2 are preferable as the possible weir site from technical, economical and environmental points of view. Therefore, alternative study for these two alternatives is executed to select the final weir site in this paragraph. General plans for both alternatives are shown in Figure A.3.21 and A.3.22 respectively.

The following conditions are given to the respective alternative in consideration of the site condition.

1) Maximum design flood for the diversion structure is the flood discharge with a return period of 100 years. Probable peak flood discharge of 570 m^3/sec which is recurrent flood of 100 years after the dam construction is adopted for the design according to the

result of hydrological study in Annex 2.

2) The height of closure dike needs the flood height with a return period of 1,000 years. Probable peak flood discharge of 1,152 m^3/sec which is recurrent flood of 1,000 years is adopted in accordance with the result of hydrological study.

3) The intake discharge for the left main canal is 3.66 m^3/sec and that for the right main canal is 7.75 m^3/sec.

4) Elevation of base rock surface is EL. 9.0 m in Alternative 1 and EL. 12.0 m in Alternative 2 in accordance with the result of the geological investigation. Therefore, the weir of Alternative 1 is designed as floating type concrete weir and that of Alternative 2 is designed as fixed type concrete weir.

5) Construction method of Coupure method is adopted for Alternative 1 and two stage wise diversion method is employed for Alternative 2.

2.3.3 Results of Comparison

The estimated costs for the main works of the intake weir construction for both alternatives are summarized below:

(Unit : 1,000 Rp)

Works	Alternative 1	Alternative 2
1. Intake Weir	5,147,400	4,856,800
2. Foundation Treatment	1,056,000	0
2. Diversion Channel	816,600	1,020,800
3. Closure Embankment	535,300	704,400
4. Operation Bridge	91,100	91,100
5. Hydromechanical Works	1,866,000	1,866,000
6. Electrical System	15,300	15,300
Sub-total	9,527,700	8,554,400
7. Temporary Works *	352,000	434,000
Total	9,879,700	8,988,400

*: 2 % (Alternative 1) and 7 % (Alternative 2) of sub-total are assumed to be the costs for temporary works respectively, due to the difference of construction methods.

The table shows that the Alternative 2 is in lower cost. This is because of high cost for foundation treatment of Alternative 1 due to unstable foundation condition. The weir of Alternative 2 can be constructed on the base rock and no remarkable foundation treatment is required. Furthermore, the construction of Alternative 2 can be executed in dry condition, as same as Coupure method, with the construction of diversion channel.

In accordance with the above consideration, the Alternative 2 would be favourable as the weir site.

2.4 Design of Weir

2.4.1 Function

The main function of the proposed weir is to divert the required quantity of irrigation water, most part of which is once stored in the reservoir and again released into the Gilirang river.(Refer Water Balance Study for Alternatives in Annex 2) In order to fulfil this function, the weir consists of several components such as intake weir, operation bridge, closure embankment, etc.

2.4.2 Design Condition

(1) Topography of the weir site

The proposed site of the Gilirang intake weir is located at about 11 km downstream of proposed dam site. The river meanders through alluvial deposit with a shape of deep gorge which is about 30 m wide. The longitudinal gradient of the river bed is very gentle as 1 to about 1,500.

(2) Geology

The base rock of the proposed weir site is mudstone of Tertiary Pliocene sedimentary rock. In this site, the outcrop of the baserock is observed in the river course with EL. 12.0 m. This base rock has enough capacity as the foundation for fixed type concrete gravity weir.

(3) Hydrology

After the dam construction, the probable floods which pass though the reservoir are regulated because of storage effect of the reservoir. Therefore, the probable peak flood discharges after the dam construction are adopted for the design of intake weir facilities. Probable peak flood discharge with a return period of 100 years (570 m^3/sec) is used for the design of the diversion structure, and that of 1,000 years (1,152 m^3/sec) is used for the design of closure dike.

Design diversion discharge is determined to be 11.41 m3/sec consisting of 3.66 m^3/sec for the left main canal and 7,75 m^3/sec for the right main canal. The left main canal covers 2,105 ha and the right main canal covers 4,455 ha of irrigation area.

2.4.3 Design of Weir

As described in the "Design Condition", the longitudinal gradient of the Gilirang river is very gentle. In order to avoid the serious effect of back water to the dam, the design intake water level is limited at EL. 18.0 m. Therefore, the design intake water levels for both main canals (left and right main canals) are fixed at EL. 18.0 m.

(1) Design structure features

1) Intake weir

The intake weir is combined type of fixed weir and gated weir. The fixed weir is constructed with concrete. The crest elevation of the weir is fixed at EL. 18.2 m for both fixed and gated weirs.

2) Scouring sluice

The scouring sluices are equipped at both sides of the weir. The type of sluice is under sluice. Two numbers of sluiceway are constructed at both sides with the width of 2.0 m for left side and 3.0 m for right side. The gates are steel gates with electric motor operation.

3) Intakes

The intakes are provided at both banks. Bottom height of intake is set at EL. 16.70 m for the left side intake and EL. 16.46 m for the right side intake. Two sets of intake gate are installed for both side intakes respectively. The design velocities at the intake gate are 0.78 m/sec for left intake and 0.93 m/sec for right intake.

4) Operation bridge

The operation bridge is constructed on the intake weir with 6.0 m width and total length of 93.6 m. The bridge is connected to the maintenance road for both main canals.

5) Closure embankment

The closure embankment is of homogeneous earth embankment with a crest width of 5.0 m and crest elevation of EL. 21.63 m. The crest of the closure embankment is connected to the left embankment of intake weir and the bridge.

The structures of the Gilirang intake weir are shown in Figure A.3.23 to A.3.26 and summarized below:

Diversion weir

(a) Type of weir	: Fixed type
(b) Material of weir	: Concrete
(c) Crest elevation	: EL. 18.20 m
(d) Intake water level	: EL. 18.00 m
(e) Design flood discharge	: 570 m^3/sec
(f) Diversion discharge	
Left main canal	: 3.66 m^3/sec
Right main canal	: 7.75 m^3/sec
(g) Crest length of fixed weir including piers	: 78.6 m
(h) Width of scoring sluice	
Left side	: 2.0 m x 2 Nos.
Right side	: 3.0 m x 2 Nos.
(i) Width of intake	
Left side	: 1.3 m x 2 Nos.
Right side	: 2.2 m x 2 Nos.
(j) Height of weir (from stilling basin)	: 6.20 m
(k) Operation bridge	
Total width	: 6.0 m
Total length	: 93.6 m

Closure embankment

(a) Type of embankment	: Homogeneous
(b) Crest elevation	: EL. 21.63 m
(c) Crest width	: 5.0 m
(d) Max. height (from riverbed)	: 9.63 m
(e) Crest length	: 740 m

Table A.3.1 Elevation of Non-overflow Section (Unit : m)

	Hd>2.5m	Hd≤2.5m
Dam with gated spillway	Hf+hw+he+1.5 (when hw+he<1.5,Hf+3) Hs+hw+he/2+1.5 (when hw+he/2<1.5,Hs+3) Hh+hw+1.5 (when hw<0.5,Hh+2)	Hf+hw+he+1.5 (when hw+he<1.5,Hf+3) Hs+hw+he/2+1.5 (when hw+he/2<1.5,Hs+3) Hh+hw+1.5 (when hw<0.5,Hh+2)
Dam with non-gate spillway	Hf+hw+he+1 (when hw+he<2,Hf+3) Hs+hw+he/2+1 (when hw+he/2.Hs+3) Hh+hw+1 (when hw<1,Hh+2)	Hf+hw+he+1 (when hw+he<1,Hf+2) Hs+hw+he/2+1 (when hw+he/1.Hs+3) Hh+hw+1 (when hw<1,Hh+2)

(Note)

Hf : Normal water level(m)
Hs : Surcharge water level(m)
Hh : Design flood level(m)
hw : Wind induced wave height from reservoir surface (m)
he : Earthquake induced wave height from reservoir surface by earthquake (m)
Hd : overflow depth when design flood discharge overflows spillway(m)