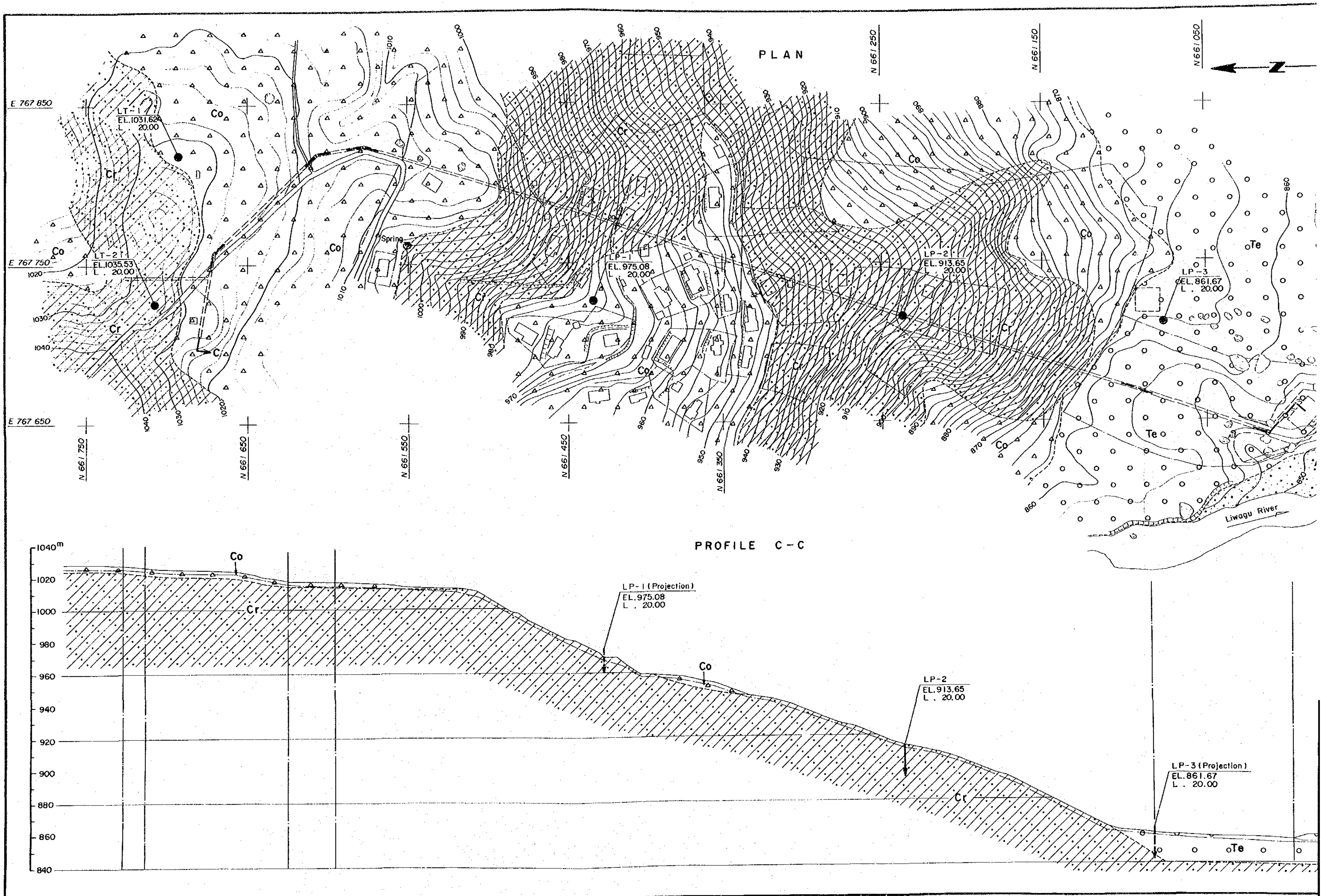
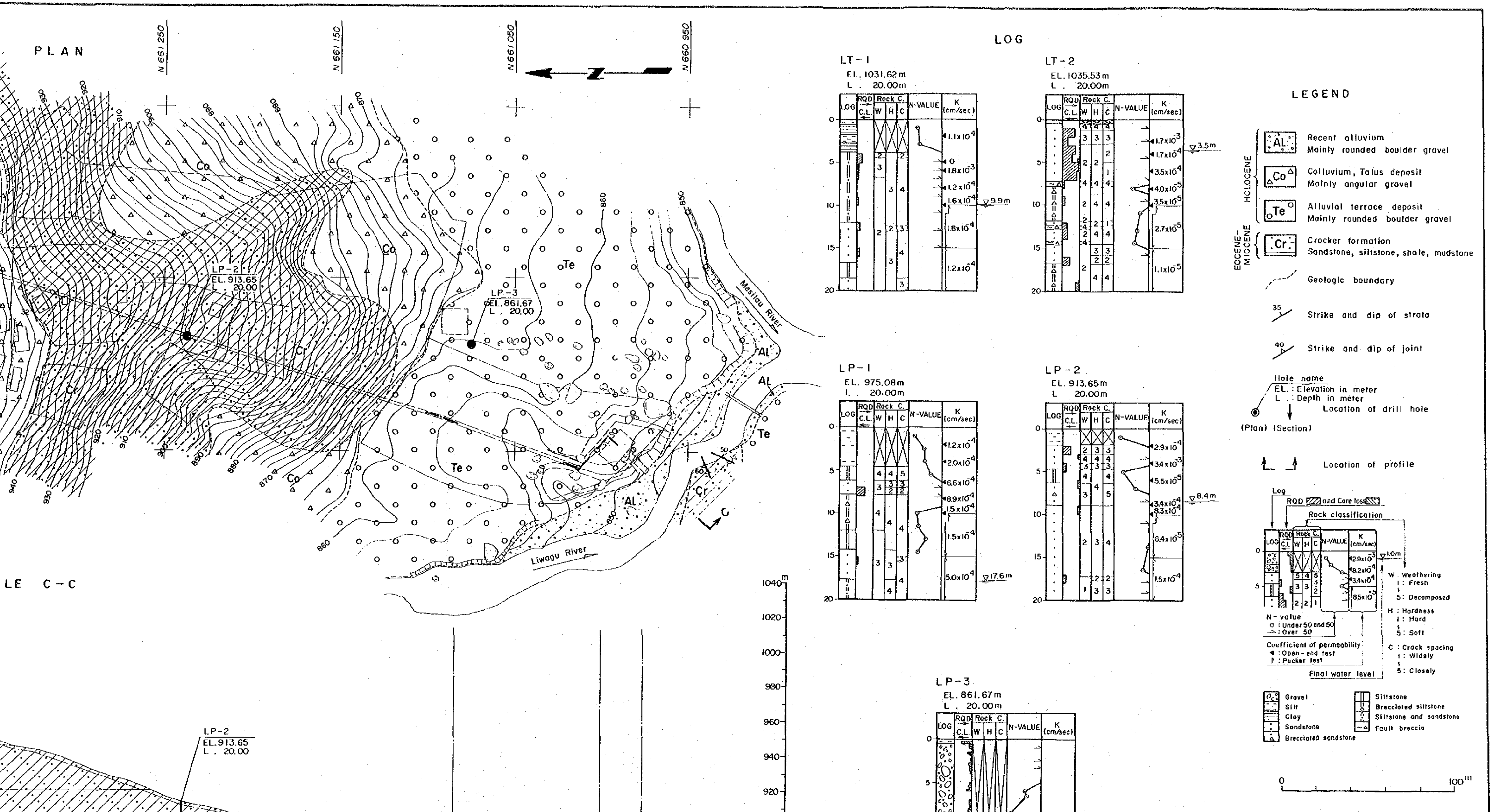


PLAN
PROFILE C-C
N 661 750
N 661 650
N 661 550
N 661 450
N 661 350
N 661 250
N 661 150
N 661 050
E 767 850
E 767 750
E 767 650
LT-1
EL.1031.62
L. 20.00
LT-2
EL.1035.53
L. 20.00
LP-1
EL.975.08
L. 20.00
LP-2
EL.913.65
L. 20.00
LP-3
EL.861.67
L. 20.00
Spring
Liwagu River
Co
Cr
Te
LP-1 (Projection)
LP-3 (Projection)
1040m
1020
1000
980
960
940
920
900
880
860
840



SMALL SCALE HYDROELECTRIC POWER DEVELOPMENT PROJECT AT UPPER LIWAGU RIVER BASIN

NARADAW PROJECT
GEOLOGICAL PLAN AND PROFILE OF PENSTOCK AND POWERHOUSE

FIGURE 7-5

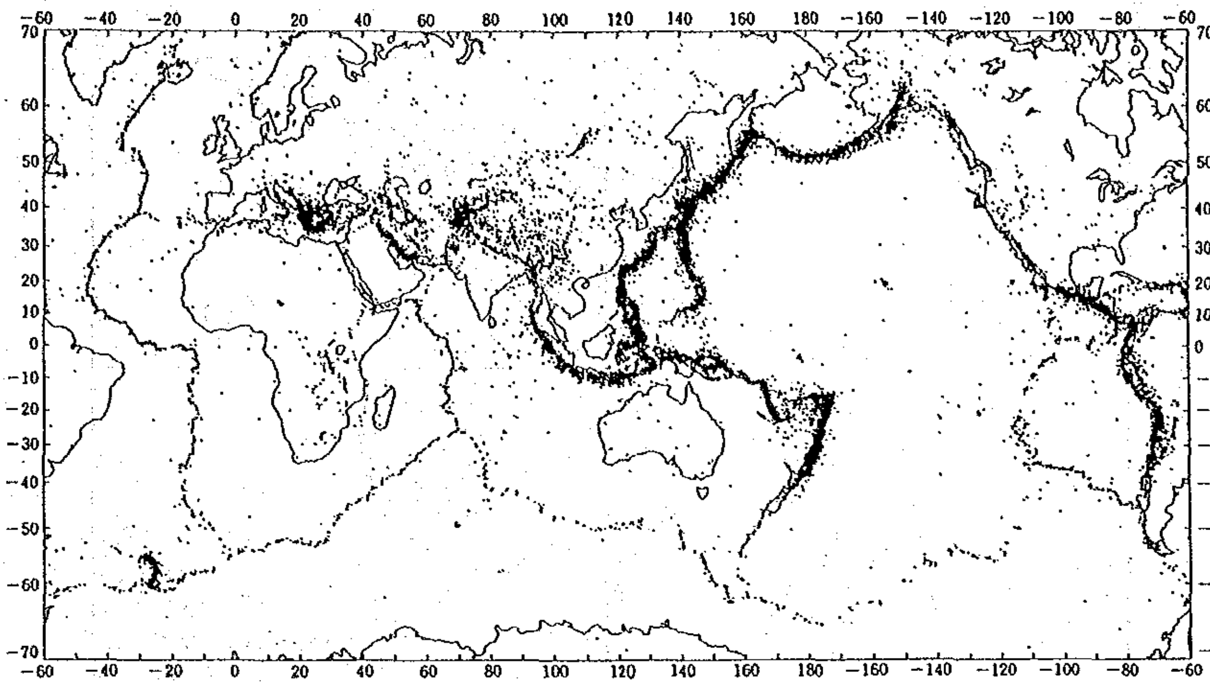


Figure 7-6 Distribution of Earthquake Epicenters of Magnitude:$M \geq 4$ and Focal Depth :$H \leq 100$km during the Period from 1970 to 1985 in the World (National Astronomical Observatory of Japan,1990)

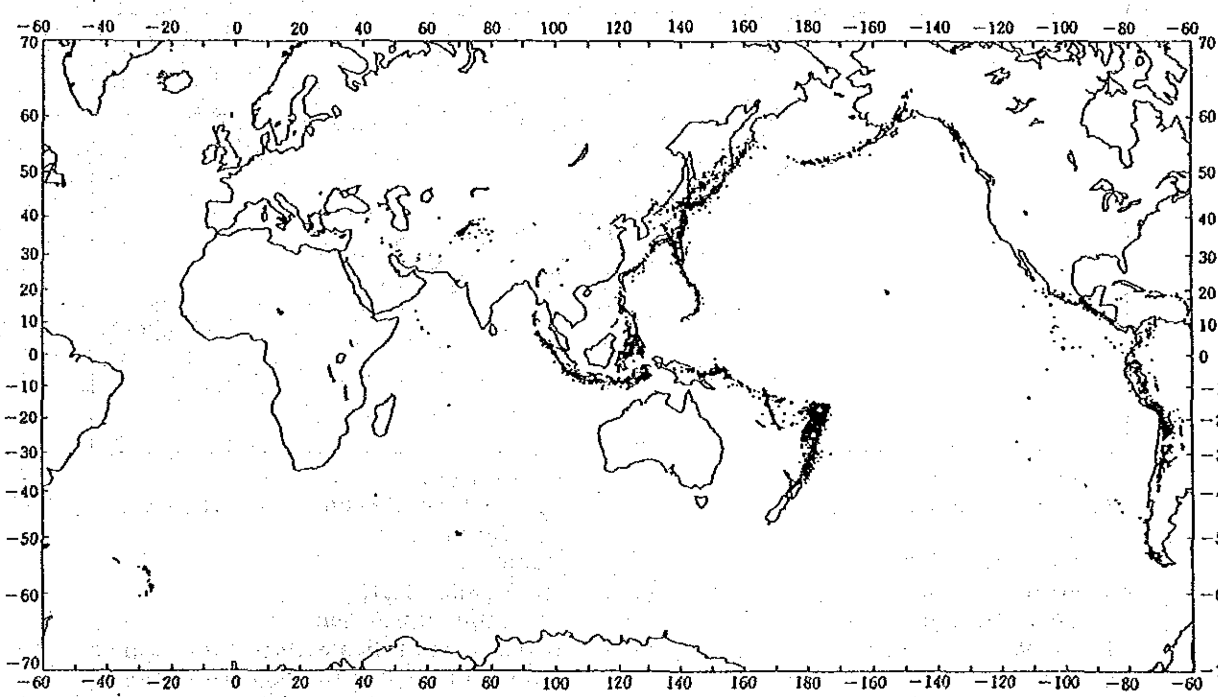
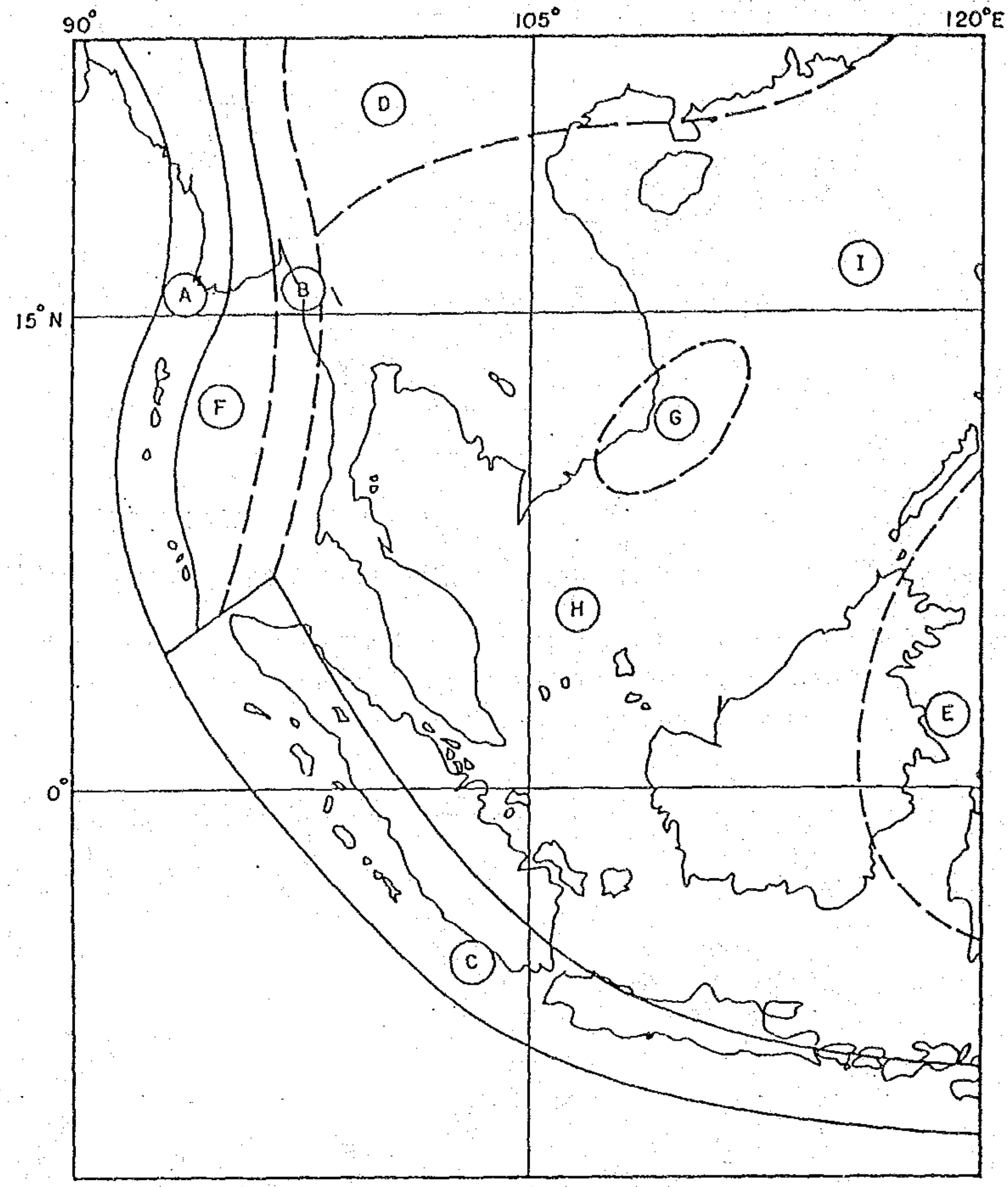


Figure 7-7 Distribution of Earthquake Epicenters of Magnitude:$M \geq 4$ and Focal Depth :$H \geq 100$km during the Period from 1970 to 1985 in the World (National Astronomical Observatory of Japan,1990)



Strongly Active :
A. Outer Burmese Arc.
B. Inner Burmese Arc.
C. Indonesian Arc.
Moderately Active :
D. Shan Plateau
E. Sabah and East Kalimantan
Active :
F. Irrawaddy / Andaman Trough

Weakly Active :
G. Junction of South China Sea and Sunda Shelf
Stable :
H. Sunda Shelf
I. South China Sea
Broken Lines Indicate Uncertain Position and not Doubt that the Feature Exists

Figure 7-8 Seismotectonic Setting Map of Malaysia(Southeast Asia Association of Seismology and Earthquake Engineering,1985)

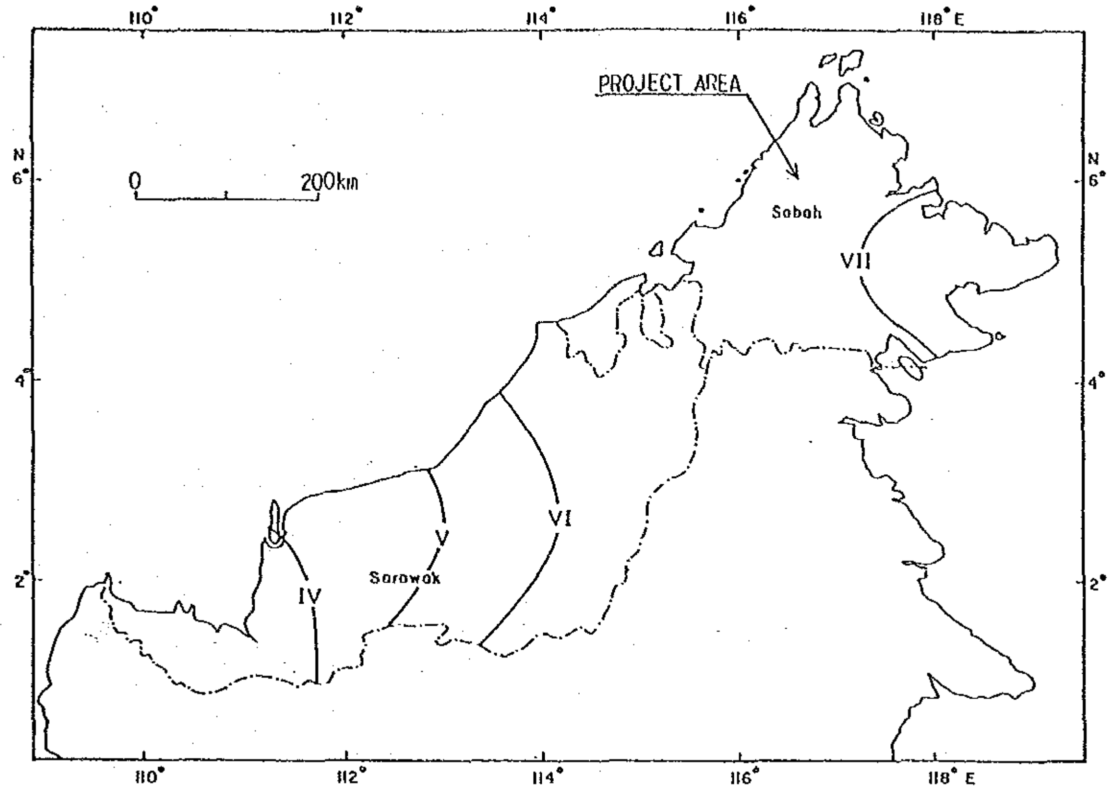


Figure 7-9 Maximum Observed Intensity (MM Scale) of Sabah and Sarawaku (1875-1983)
(Southeast Asia Association of Seismology and Earthquake Engineering,1985)

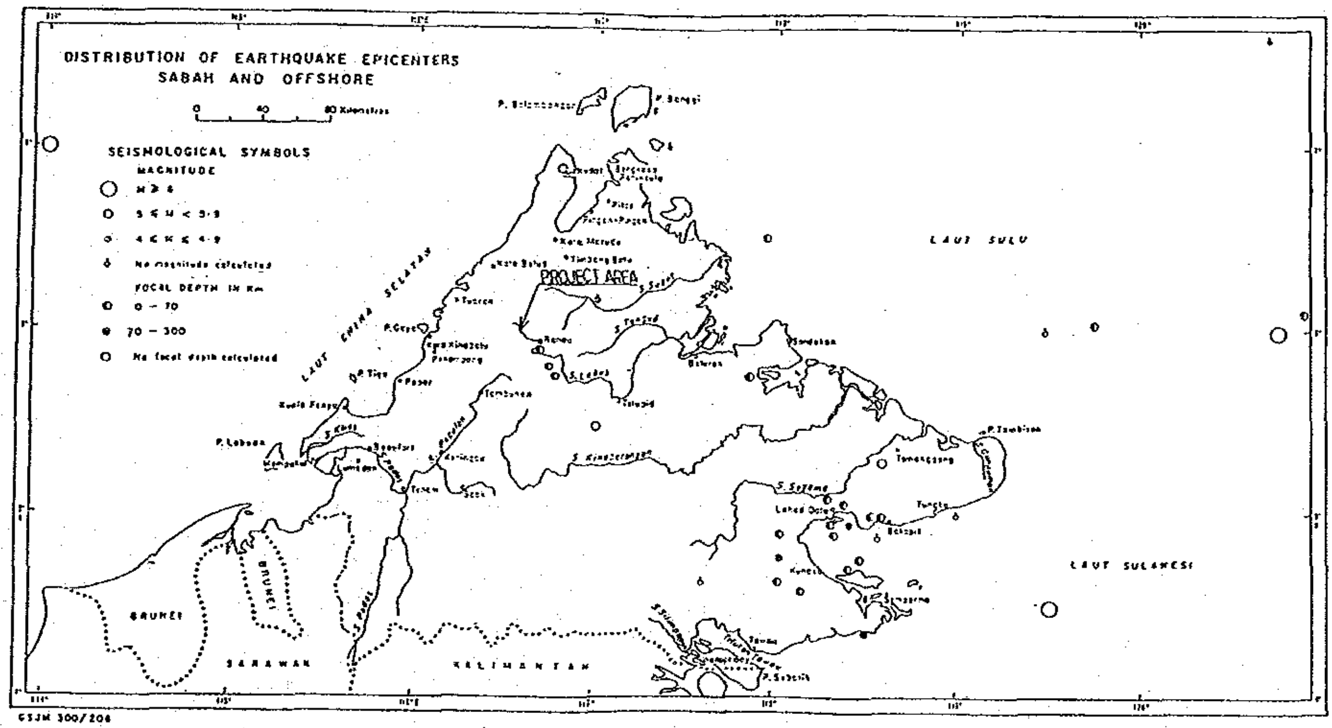


Figure 7-10 Distribution of Earthquake Epicenters,Sabah and Offshore (Lim,P.S.,1991)

Chapter 8 METEOROLOGY AND HYDROLOGY

Chapter 8

METEOROLOGY AND HYDROLOGY

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8. METEOROLOGY AND HYDROLOGY

The meteorological and hydrological features of Liwagu River basin have been analyzed based on the precipitation data of Kundassang Rainfall Gauging Station, the river discharge data at Bedukan Gauging Station, and the suspended sediment data of Porog Gauging Station on Labuk River. (Refer to **Appendix 3**)

The river discharge, flood discharge and sediment load at the intake dam sites of Mesilau and Liwagu, and the powerhouse site of Naradaw, as calculated from the acquired observation data are as presented in the table below.

Item		Mesilau Intake Dam Site	Liwagu Intake Dam Site	Naradaw Powerhouse Site
Catchment Area	(km^3)	28	31	34
Mean Discharge	(m^3/s)	1.06	1.18	--
95% Firm Discharge	(m^3/s)	0.21	0.24	--
50 Year Return Period Flood	(m^3s)	180	200	220
Sediment Load	(m^3/year)	470	520	--

The installation of a gauging station and the river discharge measurement proposed on Scope of Work were not carried out for the reasons described in **Appendix 9**.

8.1 Meteorological Profile in the State of Sabah

8.1.1 General Conditions in the State of Sabah

The state of Sabah of East Malaysia belongs to the Asian monsoon climate zone. This climate of Sabah state is divided into the southwest monsoon season during May through August, and the northeast monsoon season during November through March. (Refer to **Fig. 8-1**)

On the west coast (Kota Kinabalu) and the south coast (Tawau), the southwest monsoon season is a rainy season while on the east coast (Sandakan) the northeast monsoon season is a rainy season.

Annual precipitation varies widely with locality ranging from 2,000 mm up to 4,000 mm. The rainfall concentrates in an area around the Crocker mountains in the northeastern region.

Temperature averages 27°C annually and remains almost constant and does not have any seasonal fluctuation in the coastal region, but it varies widely with locality and weather in the state of Sabah as a whole.

8.1.2 Precipitation and Temperature in the State of Sabah

As is mentioned in **8.1.1**, it is a rainy season during May through August on the west and the south coast, and during November through February on the east coast in general. However, the difference between them has become less distinctive recently, which means that it is a rainy season between May and February, and a dry season between February and April in the entire area of the state of Sabah, since it is under influence of both southwest and northeast monsoons.

In the rainy season a transitory thundershower usually occurs in the afternoon or evening everyday, while it is not so rare that there is no rainfall for more than a few weeks in the dry season.

Especially there are reports on severe drought which were experienced in 1972 & 1973, in 1983 and in 1986 & 1987, and brought about bad effects on agricultural products and water supply, and some mountain fires.

Fig. 8-2 to **3** show the precipitation and temperatures on the west coast (Kota Kinabalu), the east coast (Sandakan), the south coast (Tawau), and the southern slope of Mount Kinabalu (Ranau) between 1981 and 1989.

Daily mean temperature in the coastal region is constantly 27°C all the year round in every area. Maximum temperature and minimum temperature also show a similar tendency as daily mean temperature, which are 34°C and 20°C respectively.

Humidity is as high as 80% to 85% showing a feature of a typical tropical climate.

8.2 Meteorology and Hydrology in the Upper Liwagu River Basin

8.2.1 Outline of the Liwagu River Basin

Originating from the southeastern and southern slope of Mt. Kinabalu at an elevation of 4,101 m, which is located in the county boarder of Ranau and Tuaran counties, the Liwagu River pours into the Sulu Sea.

The river joins with the Kegibangan River in the vicinity of the Tampias river gauging station to form the Labuk River and the distance between the water source and its river-mouth is approximately 200 km. The upper Liwagu River basin designates, in this text, the upstream area of Ranau.

The upper Liwagu River basin was once covered with a primeval forest which has been replaced with a secondary forest recently, as a part of it was developed for farming and a large-scale lumbering was conducted.

Especially, the opening of Tamparuli-Ranau Highway on September in 1982 promoted the development of the river basin, and a primeval forest is rarely seen in between Kundasang and Ranau, while the farmlands for growing highlands vegetables, houses scattered up to the summit, and a secondary forest are widely seen. Development of golf courses is also active in the river basin.

As shown in **Fig. 8-4**, development area is located almost in the center of the upper Liwagu River basin which is 4 km south east to Kudassang.

Basic items of the Liwagu River and the Mesilau River in the project site are as follows.

	Liwagu Intake Dam Site	Mesilau Intake Dam Site
Catchment Area	31 km^2	28 km^2
Mean River Gradient	1/12	1/8
Elevation of River Bed	EL. 1,045 m	EL. 1,035 m

8.2.2 Meteorological and Hydrological Survey at the River Basin and Its Vicinity

Fig. 8-5 shows the locations where meteorological and hydrological survey has been conducted in the Liwagu River basin and its vicinity, while **Table 8-1** to **4** show the items and periods of observation at rainfall gauging stations and river gauging stations.

Among them all, the most important items, when analyzing meteorological and hydrological characteristics of the upper Liwagu River basin, are precipitation data at the Kundasang rainfall gauging station, discharge data at the Bedukan river gauging station which is 10 km downstream of Ranau, and suspended sediment data at the Porog river gauging station of the Labuk River.

8.2.3 Precipitation in the River Basin

Table 8-5 and **Fig. 8-6** show daily precipitation data at the Kundasang rainfall gauging station which is located in the central part of the Liwagu River basin.

According to the above data, annual mean precipitation for 10 years from 1970 through 1979 is approximately 2,300 mm. Difference of annual precipitation is minimal as the maximum annual precipitation is 2,760 mm (1975) and the minimum annual precipitation is 1,912 mm (1976).

Rainfall characteristics show an influence of the northeast monsoon like Sandakan on the east coast, with the maximum daily mean precipitation of 279 mm (October) and the minimum of 120 mm (February).

Precipitation in the dry season fluctuates extensively with the maximum monthly precipitation of 542 mm (February 1977) and the minimum monthly precipitation of 21 mm (February 1973). The data show that dry days continued for a longer period of time than any other month during January through March in 1973 when a low monthly precipitation of 20 mm to 30 mm was recorded.

8.3 Discharge at the Project Site

8.3.1 Discharge Calculation Method

In the vicinity of Naradaw project site, Kinabalu National Park gauging station and Bedukan gauging station have been established to measure discharges.

Kinabalu National Park gauging station has been installed in the Kinabalu National Park at the foot of the Mount Kinabalu and placed under the jurisdiction of the park's administration office. This gauging station has a small catchment area of 11 km^2, which has resulted in widely-varying calculation of discharges obtained from the water level observed. And as a result of poor access to the gauging station, a large amount of data is missing.

Bedukan gauging station is located 17 km downstream of Naradaw project site and has a large catchment area of 200 km^2. Having an easy access, the amount of its missing data is less than those of Kinabalu National Park gauging station.

However, Bedukan gauging station has been closed since October 1981.

Judging from the observation conditions at these gauging stations, the discharge at the project site has been calculated based on the discharges for ten years from 1970 through 1979, and in proportion to the ratio of catchment area of the project site and Bedukan gauging station.

8.3.2 Discharge Data at Bedukan Gauging Station

Missing data of Bedukan gauging station during a ten-year period from 1970 through 1979 is concentrated to the period between December 1974 and January 1976 as shown in **Table 8-6**.

The total number of days which do not have data amounts to 491 days in the ten-year period.

An analysis was conducted to supplement the missing data for the relevant period by the following method.

- As for relatively a short period of time excluding from December 1974 through January 1976, the missing data is to be supplemented by statistical methods such as periodic analysis.

- As for a long period of data missing from December 1974 through January 1976, the missing data is to be supplemented from correlations between discharging data obtained at Tampias and Tomboloi gauging station, and that of Bedukan gauging station.

8.3.3 Result of Discharge Data Supplementation

Results of correlation of data obtained at Bedukan gauging station, Tampias gauging station, and Tomboloi gauging station are shown below.

	Bedukan G/S - Tampias G/S	Bedukan G/S - Tomboloi G/S
Period of Data	1977 - 1980	1970 - 1977
R	0.689	0.776
A	0.06	0.05
B	2.53	0.70

where,

$Y = AX + B$

R : Coefficient of Correlation

Y : Data obtained at Bedukan Gauging Station

X : Data obtained at Tampias Gauging Station or Tomboloi Gauging Station

Discharge data of Bedukan gauging station after data supplementation is shown in **Fig. 8-7** and **Table 8-7**.

Daily mean discharge for the ten-year period is 3.80 $m^2/s/100\ km^2$, maximum daily mean discharge is 165.20 $m^3/s/100\ km^2$ (February 7, 1971), minimum daily mean discharge is 0.02 $m^3/s/100\ km^2$ (April 19, 1975).

Maximum monthly mean discharge and minimum monthly mean discharge for the ten-year period both appear in February. Especially, discharge from January through March 1973 is extremely small, and this is, as was mentioned in **8.2.3**, closely related with precipitation in the relevant river basin.

8.3.4 Calculated Discharge

Based on the discharge data for the ten-year period between 1970 and 1979 obtained in **8.3.3**, discharges at Liwagu intake dam site and Mesilau intake dam site for Naradaw project have been calculated by the following method.

- Prepare Duration Curve based on parallel average of the ten-year data obtained at Bedukan Gauging Station (Refer to **Fig. 8-8**).

- Prepare Discharge Duration of Liwagu intake dam site and Mesilau intake dam site based on the ratio of catchment area of each intake dam site versus Bedukan gauging station.

Upon calculating the discharge in the project site, catchment areas of Bedukan gauging station, Liwagu intake dam site, and Mesilau intake dam site, were calculated by planimeter.

	Catchment Area (km²)	
	Hydrological Year Book by DID	Planimetry 1:25000 maps (This Report)
Bedukan Gauging Station	440	200
Liwagu Intake Dam Site	--	31
Mesilau Intake Dam Site	--	28

The Discharge Duration at Liwagu intake dam site and Mesilau intake dam site which were calculated based on the Discharge Duration at Bedukan gauging station is shown in the table below.

	Daily Mean Discharge ($m^3/s/100\ km^2$)	Daily Mean Discharge (m^3/s)	
Duration (%)	Bedukan Gauging Station Catchment Area 200 km^2 (A)	Liwagu Intake Dam Site Catchment Area 31 km^2 (A) x 0.31	Mesilau Intake Dam Site Catchment Area 28 km^2 (A) x 0.28
10	7.16	2.22	2.00
20	5.25	1.63	1.47
30	4.06	1.26	1.14
40	3.33	1.03	0.93
50	2.72	0.84	0.76
60	2.20	0.68	0.62
70	1.77	0.55	0.50
80	1.38	0.43	0.39
90	0.97	0.30	0.27
95	0.76	0.24	0.21

8.4 Flood Discharge in the Project Site

8.4.1 Method of Flood Discharge Calculation

The following method was applied in calculating the flood discharge.

- Calculate flood discharge in return period of 5, 10, 20, 50 and 100 years, by Gumbel distribution and Log-Normal distribution using the maximum discharges at Bedukan gauging station for each year during the period between 1970 and 1979.

- Based on the calculation of flood discharge by the above method, compute the flood discharge at Liwagu intake dam site, Mesilau intake dam site, and the power station, using the relational curves in terms of the catchment area versus specific flood discharge. (Refer to **Fig. 8-9**)

Maximum discharges of Bedukan gauging station for 1970 through 1979 which were used for analysis are shown below.

Date of Occurrence	Maximum Discharge (m^3/s)
December 3, 1970	83
November 18, 1971	235
January 18, 1972	123
September 15, 1973	153
February 13, 1974	543
February 24, 1975	85
May 23, 1976	159
February 21, 1977	299
January 13, 1978	120
October 18, 1979	79

8.4.2 Calculated Flood Discharge

The result of flood discharge calculation is shown in **Table 8-8** and **Fig. 8-10**.

8.5 Sediment Load in the Project Site

8.5.1 General Outline of the Resources of Sediment Load in the Upper Reaches of the Liwagu River

Most of the areas in the upper reaches of the Liwagu River are mountains and hills of an elevation of 500 m to 4,100 m. The Liwagu River and the Mesilau River are weaving through this area from west-northwest to east-southeast. Since the upper reaches of the Liwagu River have been developed, small-scale villages are scattering here and there and secondary forests are widely seen while there is few primeval forest left. The gradients of Liwagu River and Mesilau River in the project sites are 1/19 and 1/15 respectively.

The grade of slopes is 20 to 45 degrees in general, and old landslide geography and new landslips are noticed on some parts of the slopes. It is thought that a large amount of sediment load has been supplied from the old landslide geography and new landslips because of the facts that there are rocks of a few meters in diameter scattering in the Liwagu intake dam site and Mesilau intake dam site, and that the water of Mesilau River was found to contain a lot of suspended sediment on the occasion of the second site survey conducted in October 1991. This is also inferred from the sediment load at the existing intake dam site of Carabau hydroelectric power station.

8.5.2 Estimation of Sediment Inflow

Naradaw project is a power generation scheme of a run-off-river plant which does not have a large-scale reservoir and takes water from a small-scale intake dam. In this case, the storage capacity in the upstream of the intake dam is very small in comparison with the river discharge, and storage of river flow does not cause suspended load accumulate to form a large amount of sediment load. However, bed load components made of larger debris are intercepted by the intake dam and accumulated in the upstream area.

Therefore, when designing the intake dam in this project, inflow of bed load components of larger debris has to be taken into consideration.

(1) Estimation of Suspended Sediment

It is only on the occasion of flood when suspended sediment is observed in the Liwagu River and Mesilau River. However, as there is no observation record of suspended sediment on the occasion of flood, it is impossible to estimate the quantity of the inflow of sand and gravel in a direct manner. Therefore, in this report, estimation is done based on the data of Porog gauging station of the Liwagu River. The Porog gauging station's data of suspended sediment is as follows.

Name of River	Name of Gauging Station	Catchment Area km^2	Suspended Sediment	
			10^3 ton/year	ton/year/km^2
Labuk	Porog	3,240	374	115

The Source: National Water Resources Study, Malaysia (Sectoral Report Vol. 2 Meteorology and Hydrology 1982) JICA

When suspended sediment of the upper reaches of the Liwagu River is to be 115 ton/year/km^2, sediment inflow at Mesilau intake dam site (28 km^2) and Liwagu intake dam site (31 km^2) is 3,220 ton/Year and 3,565 ton/year respectively.

Supposing that average sediment density of the mixture of clay, silt and sand is 1.0 ton/m^3 and the entire amount is to accumulate, 3,220 m^3/year and 3,565 m^3/year is to accumulate in the upstream area of each dam site.

However, as was mentioned earlier, it is deemed that most of the suspended sediment is flushed down with flood and will not accumulate in the upstream area of the intake dam in large quantity.

(2) Estimation of Bed Load

Although it is difficult, as a matter of reality, to observe bed load in natural rivers to make an accurate quantitative estimation, there is a strong possibility that the quantity of the inflow of gravel and rocks at the intake dam is large enough not to be disregarded in comparison with its storage capacity.

It is said that bed load does not exceed 25% of the quantity of suspended sediment. In consideration of the fact that the tractive force is strong in the upper reaches of the Liwagu River we are to take an assumption that bed load is as much as 25% of the suspended sediment. In this case, annual inflow of bed load in Mesilau intake dam and Liwagu intake dam will be 805 ton/year and 891 ton/year respectively. Supposing that average sediment density is 1.7 ton/m^3 though it is actually dependent on the distribution of gravel grading, total volume will be 470 m^3/year and 520 m^3/year respectively when all of the inflow is to accumulate.

If bed load which moves on the river bed is intercepted by the intake dam, a considerable amount is to accumulate in the upstream area of the dam.

As the storage capacity of Mesilau intake dam and Liwagu intake dam is 900 m^3 and 400 m^3 respectively, the intake dam is to be filled with sand within a year or two if there is an inflow of sediment that exceeds 400 m^3/year in quantity.

Estimation shown here is merely reference data based on the data obtained at Porog gauging station of the Labuk River, and actual figures may be less. However, as the relevant river basin is developed further, it is very possible that the source of sediment supply increases in the future and the inflow of sediment into the intake dam is also to increase.
Therefore, when designing the intake dam, enough consideration has to be given to the removal of sand and gravel that flow in.

Table 8-1 Outline of Rainfall Gauging Stations

Ref. No.	Station Name	Altitude m	Latitude ° ′ N	Longitude ° ′ E	Gauged Period
6065001	Kambarangan	2,146	06 02	116 33	1957 - 1983
6065002	Kinabalu National Park	1,623	06 02	116 32	1971 - 1990
5966002	Kundasang	1,372	05 58	116 40	1961 - 1986
5966001	Ranau Agriculture	548	05 56	116 39	1954 - 1986
5966001	Ranau JPT	549	05 58	116 42	1980 - 1990
5968001	Tampias	220	05 42	116 51	1978 - 1989

Table 8-2 Outline of River Gauging Stations

Ref. No.	Station Name	Altitude m	Latitude ° ′ ″ N	Longitude ° ′ ″ E	Catchment Area km²
6065401	Kinabalu National Park	1,460	60 00 05	116 09 10	11
5966401	Bedukan	390	05 55 00	116 09 10	200
5768401	Tampias	220	05 43 05	116 51 35	2,010
5770401	Tomboloi	98	05 44 25	117 03 00	2,460
5872401	Porog	17	05 51 15	117 13 40	3,240

Table 8-3 Gauged Period of Rainfall Gauging Stations

Ref. No.	Station Name	Gauged Period	'70	'71	'72	'73	'74	'75	'76	'77	'78	'79	'80	'81	'82	'83	'84	'85	'86	'87	'88	'89	'90	Remarks
6065001	Kambarangan	'57-'83																						
6065002	Kinabalu N.P.	'71-'90																						
5966002	Kundasang	'61-'86																						
5966001	Ranau JPT	'80-'90																						
5966001	Ranau Agricaluture	'54-'90																						
5968001	Tamplas	'78-'87																						

Table 8-4 Gauged Period of River Gauging Stations

Ref. No.	Station Name	Catchment Area km²	Gauged Period	Year '70	'71	'72	'73	'74	'75	'76	'77	'78	'79	'80	'81	'82	'83	'84	'85	'86	'87	'88	'89	'90	Remarks
6065401	Kinabalu N.P.	11																							Water Level
5966401	Bedukan	200	'70-'80																						
5768401	Tamplas	2,010	'77-'87																						
〃	〃	2,010																							Water Level
5770401	Tombolol	2,460	'64-'77																						
5872401	Porog	3,240	'63-'89																						
〃	〃	3,240																							Water Level

Table 8-5 Monthly Rainfall at Kundasang Rainfall Gauging Station (1970-1979)

(Unit:mm)

YEAR	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP	OCT	NOV	DEC	TOTAL
1970	200.10	35.20	183.30	168.20	244.90	275.20	120.50	206.40	187.80	253.00	204.20	205.80	2284.60
1971	173.40	455.00	86.70	42.52	69.40	170.10	71.90	329.40	193.00	273.90	311.60	166.60	2343.52
1972	213.30	149.80	254.90	46.30	279.60	69.60	26.90	212.40	329.60	208.40	247.20	181.30	2219.30
1973	32.70	20.80	26.90	217.90	177.70	111.80	233.30	119.50	463.60	256.90	254.30	159.60	2075.00
1974	178.80	421.00	177.30	140.60	256.90	252.30	134.90	204.50	177.60	236.10	118.30	277.35	2575.65
1975	284.40	190.30	89.10	79.80	331.20	85.00	201.10	260.20	231.70	264.90	389.20	353.20	2760.10
1976	281.90	96.60	73.50	71.30	228.10	68.20	226.20	185.10	98.40	237.40	208.70	136.60	1912.00
1977	272.00	541.50	47.10	103.30	139.90	251.20	164.00	184.80	134.20	314.50	249.00	218.90	2620.40
1978	131.80	107.90	107.40	146.80	160.00	294.70	149.40	110.00	179.10	280.30	253.70	191.00	2112.10
1979	28.50	25.60	149.50	67.00	180.30	223.10	238.10	140.60	260.00	467.10	193.30	189.50	2162.60
TOTAL	1796.90	2043.70	1195.70	1083.72	2068.00	1801.20	1566.30	1952.90	2255.00	2792.50	2429.50	2079.85	23065.27
MEAN	179.69	204.37	119.57	108.37	206.80	180.12	156.63	195.29	225.50	279.25	242.95	207.98	2306.53

Table 8-6 Number of Missing Data of Daily Mean Discharge at Bedukan Gauging Station (1970-1979)

Gauging Station	Gauged Period	Month	Year									
			1970	1971	1972	1973	1974	1975	1976	1977	1978	1979
Bedukan	1970−1980	Jan.	1∼29	1∼5				1∼29	1∼31			
		Feb.							1∼6, 12			
		Mar.					11∼18, 26∼31	17∼31				
		Apr.	7, 8					1∼30			6	
		May				1	26∼31	1∼31	18			
		Jun					1∼30	1∼30				
		Jul.					1∼9	1∼31				
		Aug.					22∼31	1, 15∼21			1∼10	
		Sep.					1∼10	22∼30				
		Oct.					11∼31	1∼31				
		Nov.		2∼4	7∼14		1∼6	1∼30				
		Dec.	22∼31				17∼31	1∼31				
Number of Missing Data		Total	41	8	8	1	113	265	39	0	16	0

Total 491 days

Table 8-7 Monthly Discharge at Bedukan Gauging Station (1970-1979)

(Unit: $10^6 m^3/100km^2$)

Year	Jan.	Feb.	Mar.	Apr.	May	Jun.	Jul	Aug.	Sep.	Oct.	Nov.	Dec.	Total
1970	13.77	4.41	3.29	5.30	7.09	11.49	8.34	9.43	8.96	14.84	12.76	13.31	112.98
1971	12.31	68.65	17.13	4.61	4.05	5.52	3.73	13.71	11.18	15.95	21.66	9.47	187.98
1972	18.06	11.61	10.04	8.11	7.80	4.22	1.90	4.28	9.05	11.56	8.57	8.57	103.78
1973	2.98	1.12	2.18	4.23	6.38	5.62	6.92	6.70	15.41	8.81	13.28	9.38	83.01
1974	17.85	38.56	10.03	5.63	11.69	7.93	7.26	7.95	3.39	5.73	3.77	6.10	125.88
1975	15.29	9.37	8.37	1.33	19.67	7.86	8.42	12.35	15.18	12.56	11.96	16.60	138.97
1976	17.65	6.53	4.43	3.26	7.16	3.08	4.85	5.48	4.00	9.40	13.05	8.32	87.20
1977	16.05	35.31	8.02	4.22	6.60	11.83	9.28	8.35	4.86	14.62	15.28	10.54	144.95
1978	11.28	5.36	4.66	6.50	7.77	13.57	10.23	6.54	6.46	11.91	14.17	13.06	111.51
1979	5.73	2.63	6.30	3.29	5.54	11.20	11.68	6.12	12.56	19.78	16.43	15.10	116.37
Total	130.97	183.53	74.46	46.48	83.73	82.33	72.60	80.90	91.05	125.16	130.95	110.45	1212.61
Mean	13.10	18.35	7.45	4.65	8.37	8.23	7.26	8.09	9.11	12.52	13.09	11.04	121.26

Table 8-8 Flood Discharge at Mesilau & Liwagu Intake Dam Sites and Naradaw P/S Site

Return Period (Year)	Flood Discharge (m^3/s)							
	Bedukan Gauging Station (C.A. 200 km^2)		Mesilau Intake Dam Site (C.A. 28 km^2)		Liwagu Intake Dam Site (C.A. 31 km^2)		Naradaw Intake Dam Site (C.A. 34 km^2)	
	Gumbel	Log-Normal	Gumbel	Log-Normal	Gumbel	Log-Normal	Gumbel	Log-Normal
5	340	260	90	70	100	80	110	90
10	440	340	120	90	130	100	140	110
20	540	430	150	120	160	130	180	140
50	670	560	180	150	200	160	220	180
100	770	660	210	180	230	190	250	210

Figures of the Gumbel distribution are adopted.

Figure 8－1 Wind movement over South-East Asia

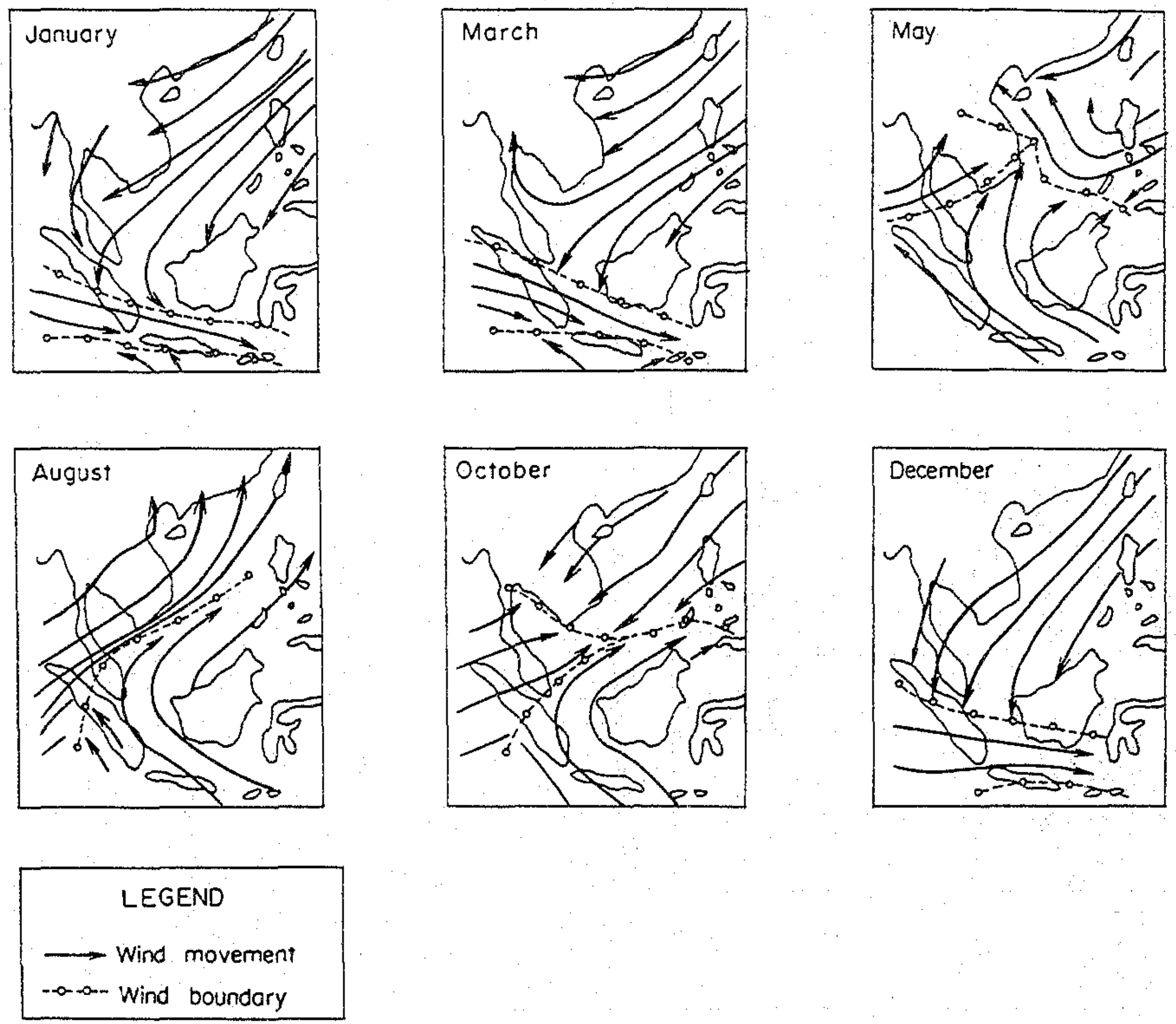


Figure 8-2 Monthly Mean Rainfall (1981 – 1989)

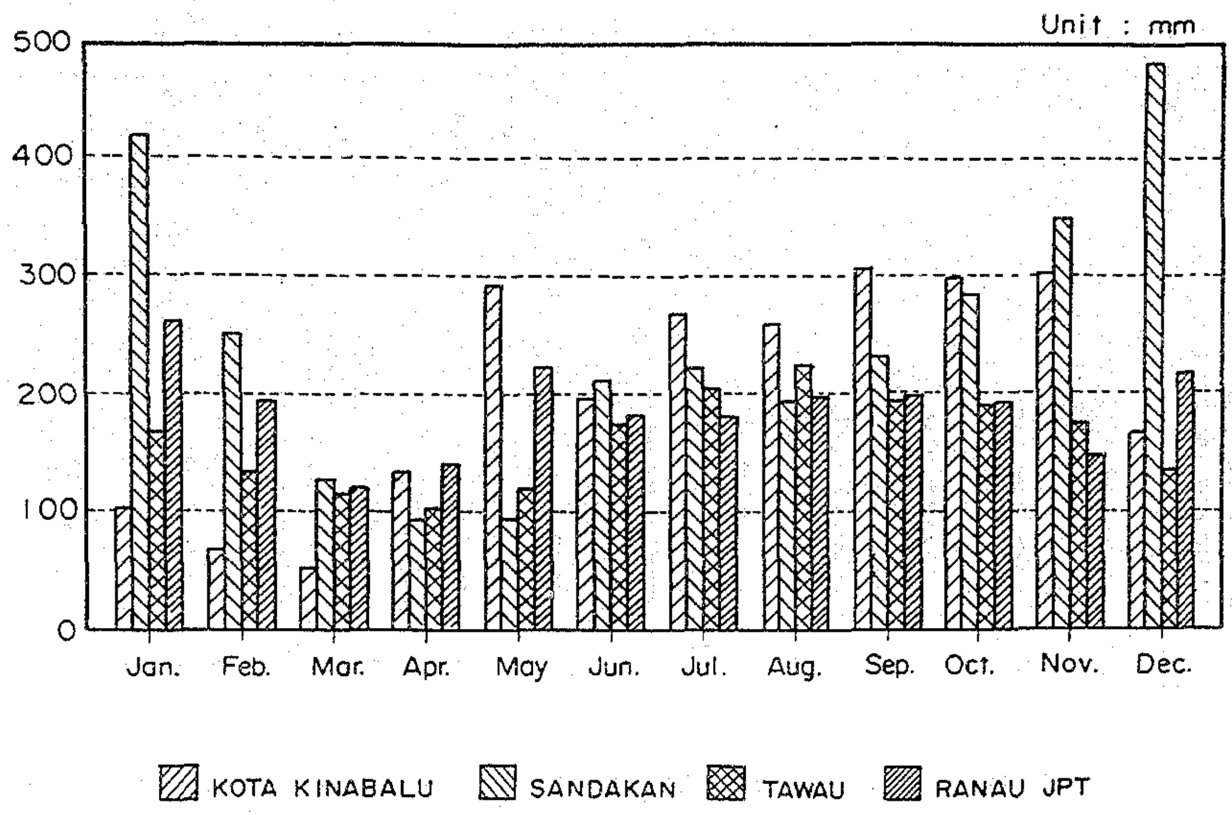


Figure 8-3 Monthly Mean Temperature (1981-1989)

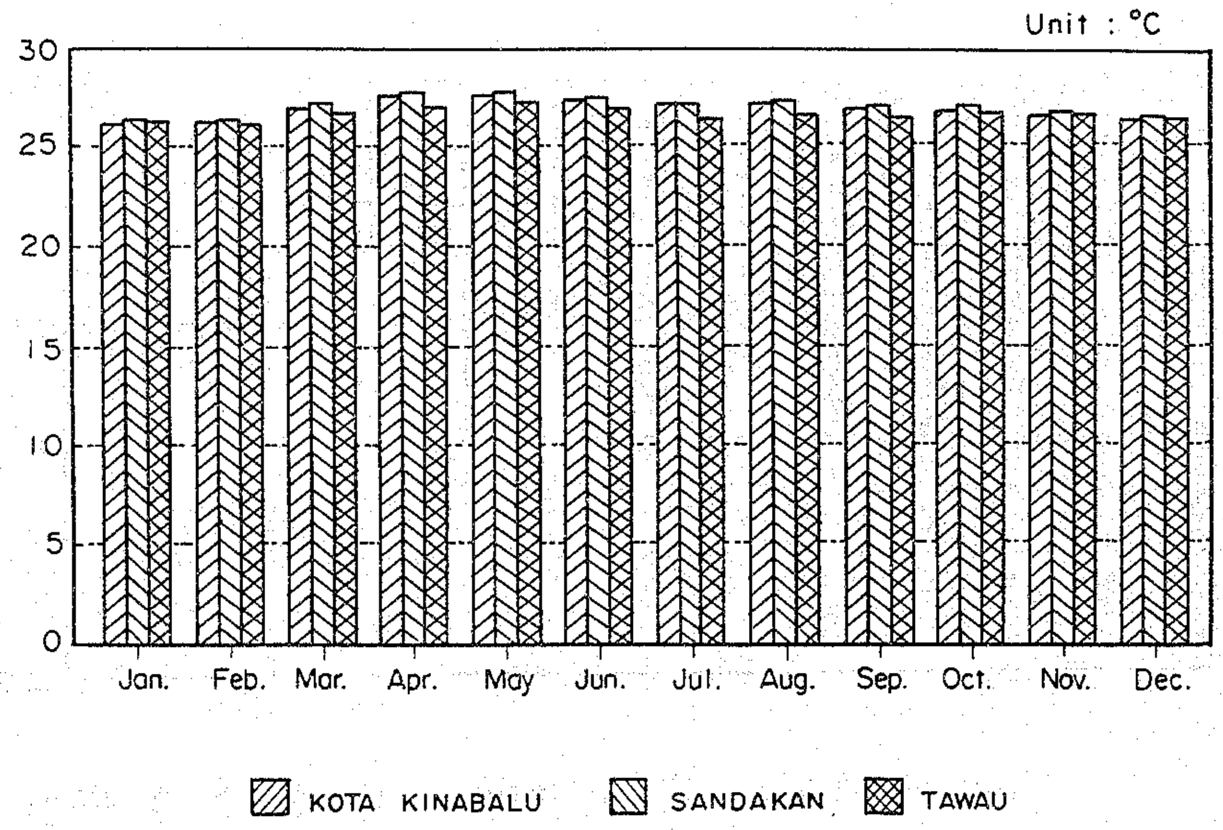


Figure 8-4 Location of Naradaw Scheme

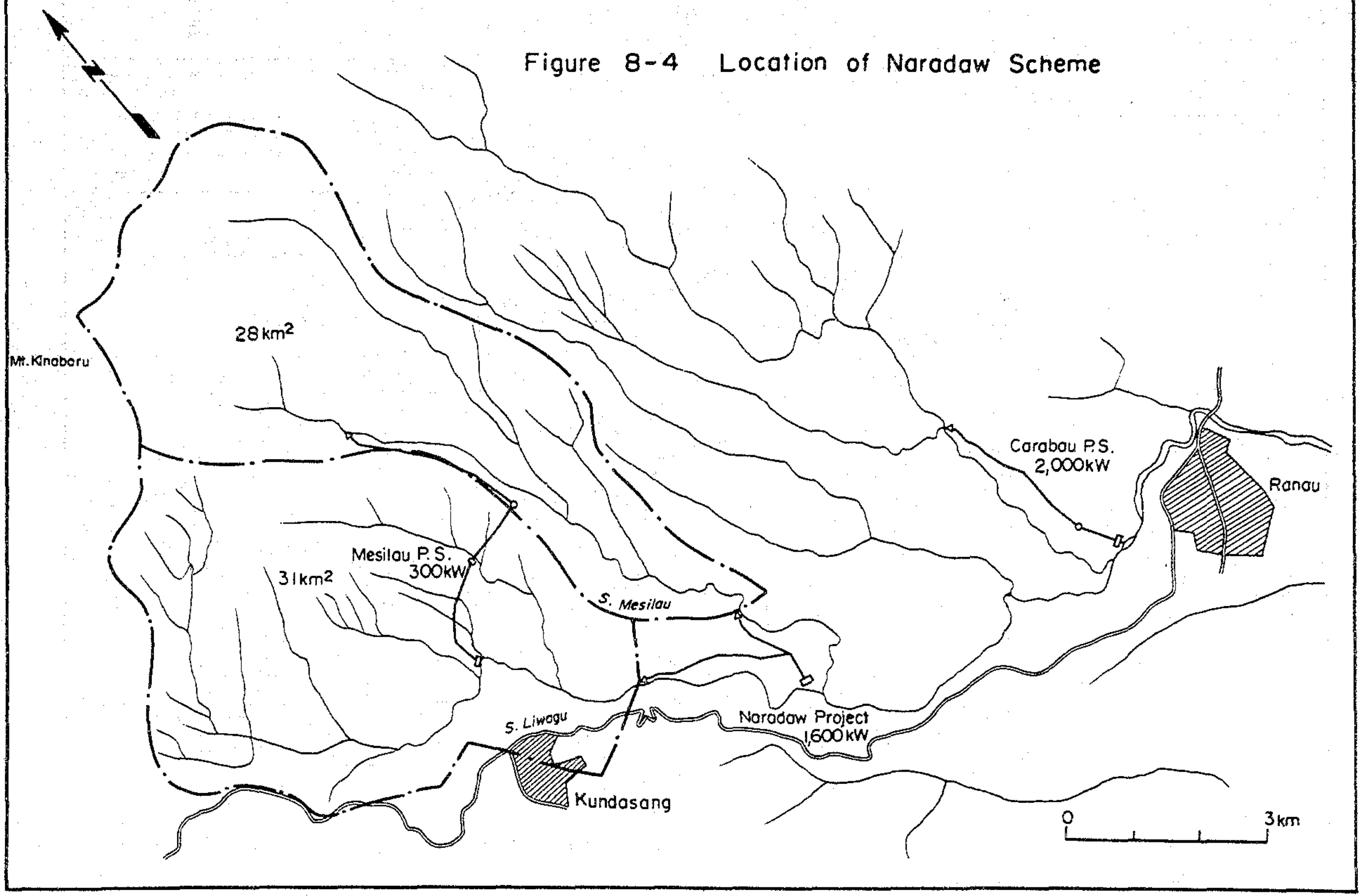


Figure 8-5 Location of Observatory

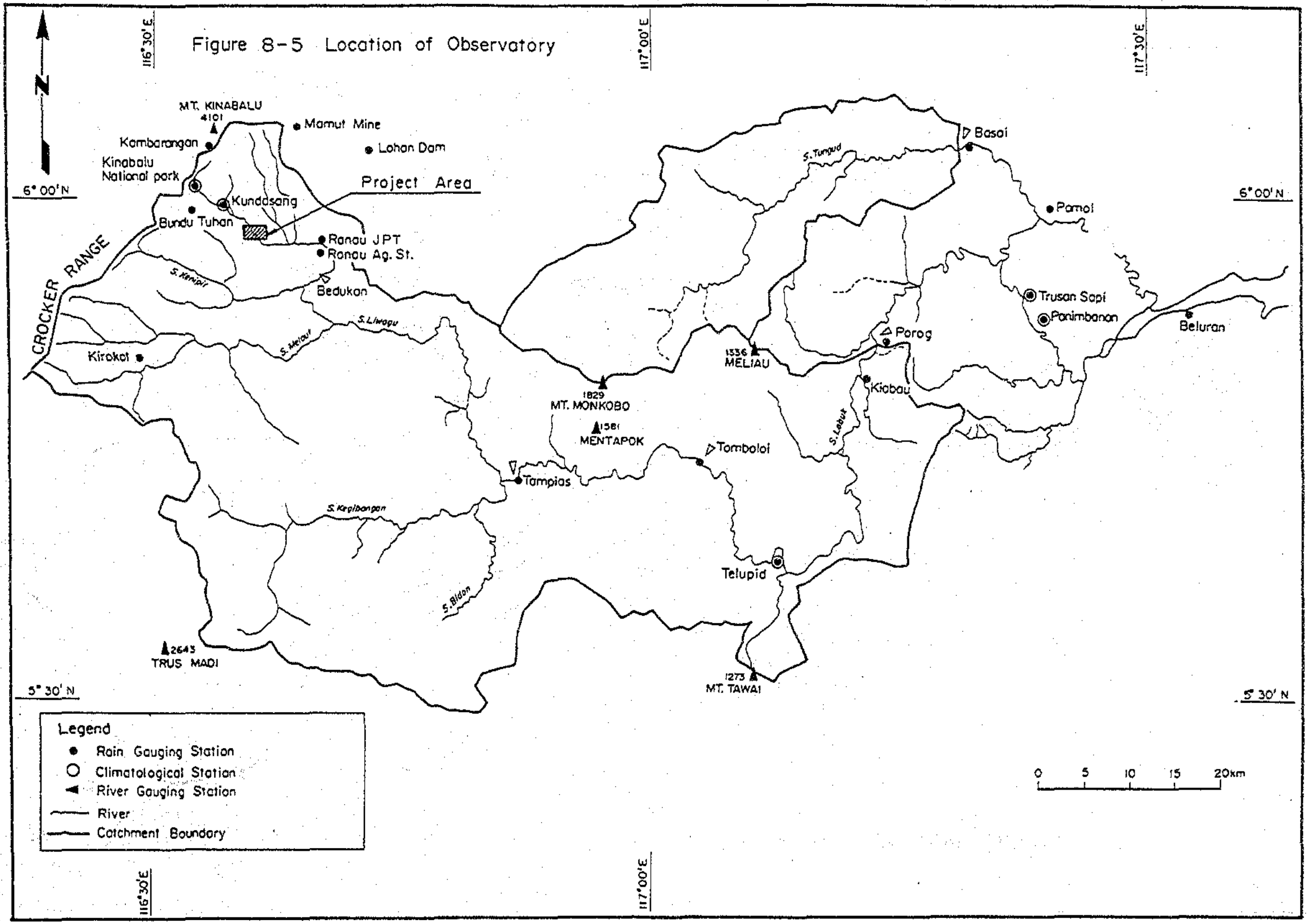


Figure 8-6 Monthly Rainfall at Kundasang Rainfall Gauging Station

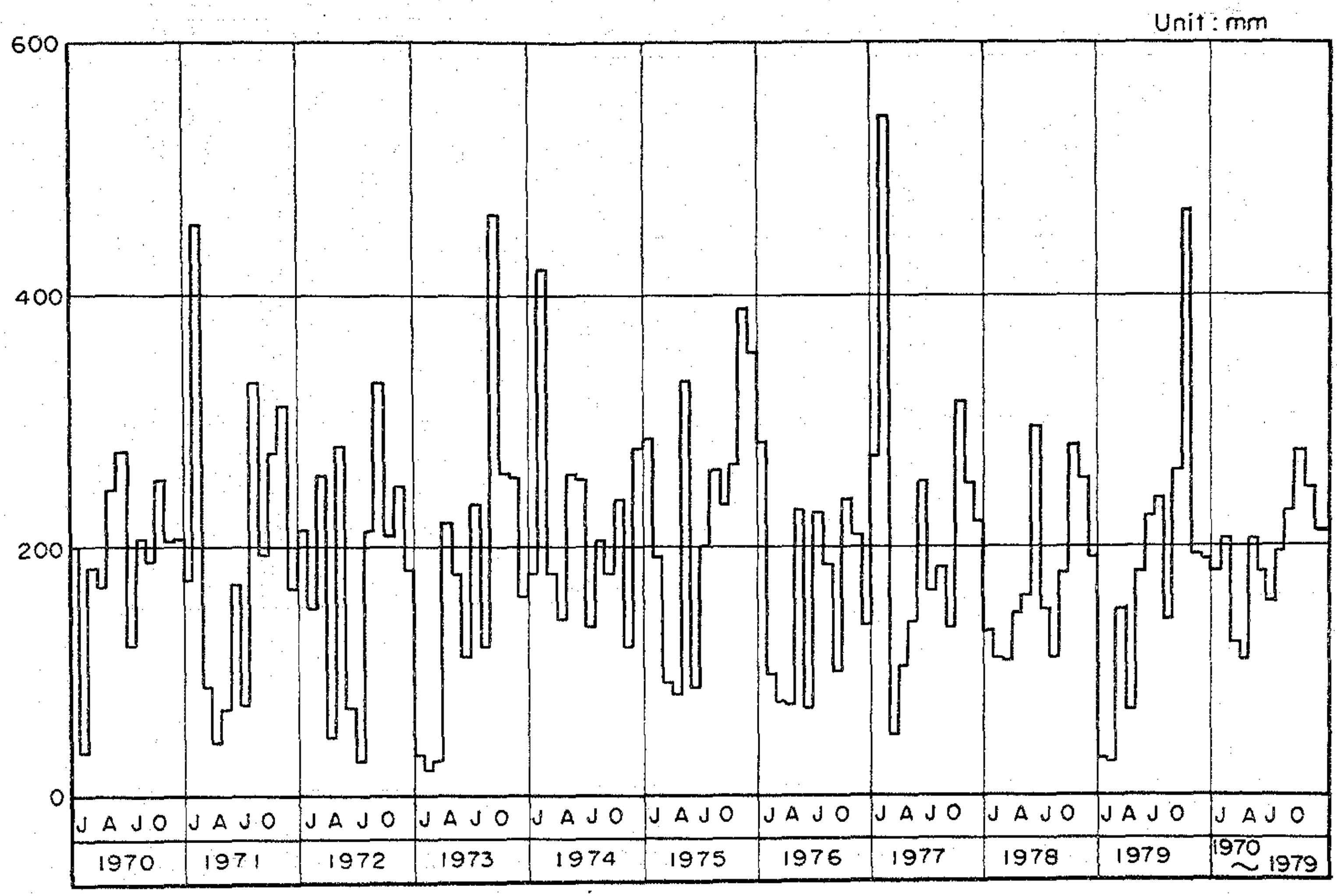


Figure 8-7 Monthly Discharge at Bedukan Gauging Station

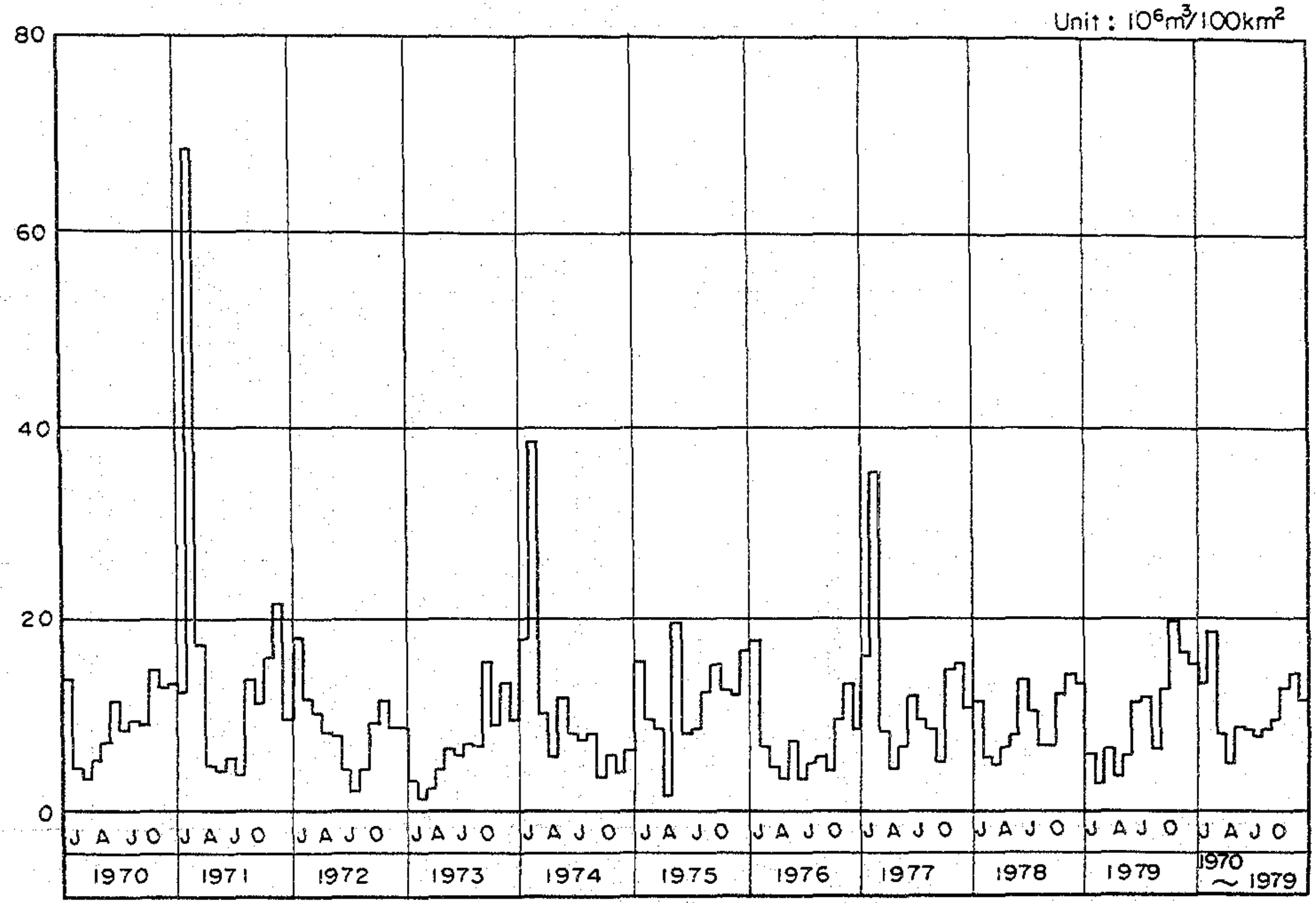


Figure 8-8 Discharge Duration Curve at Bedukan Gauging Station

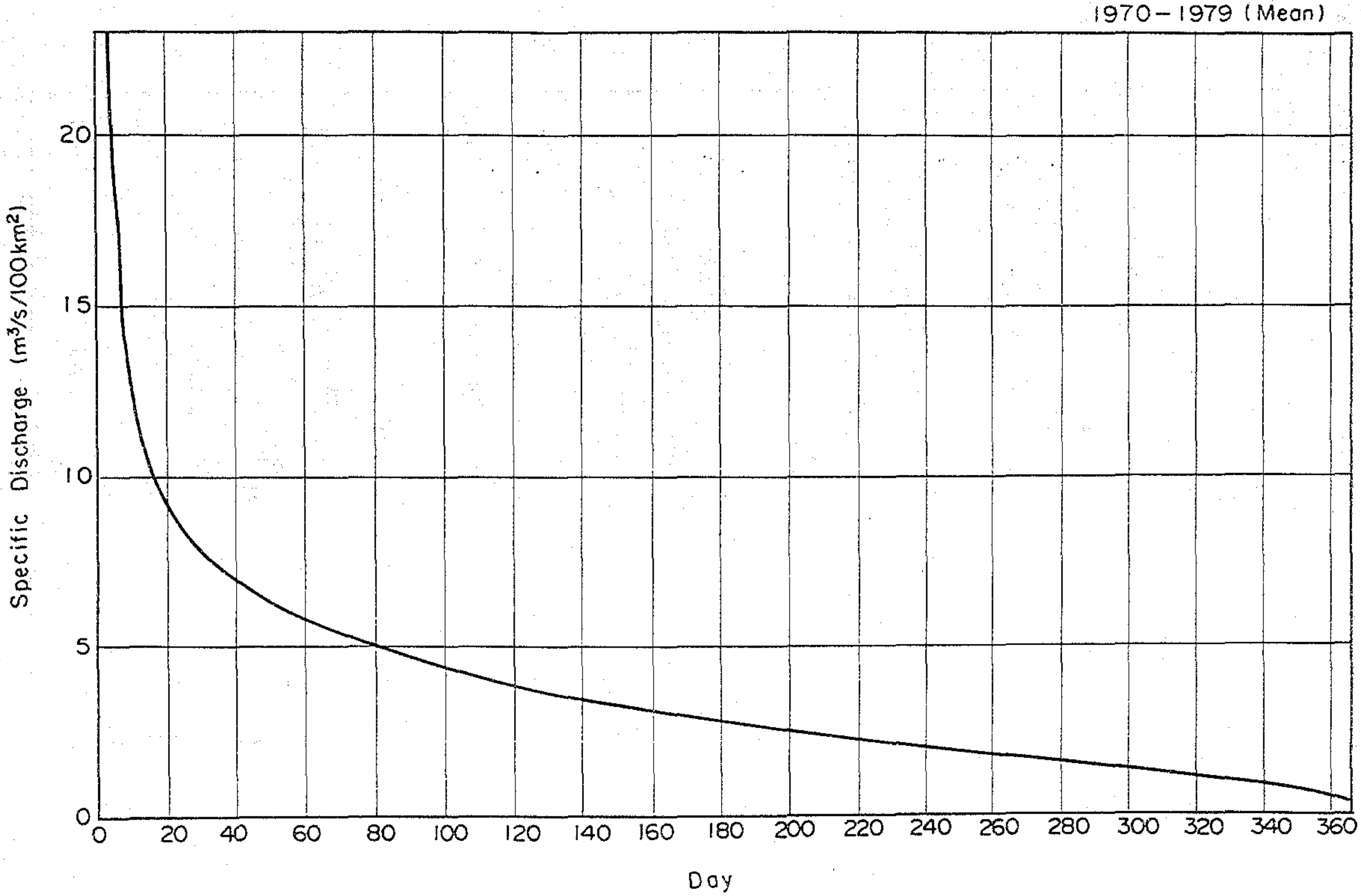


Figure 8-9 Relation between Specific Discharge and Catchment Area

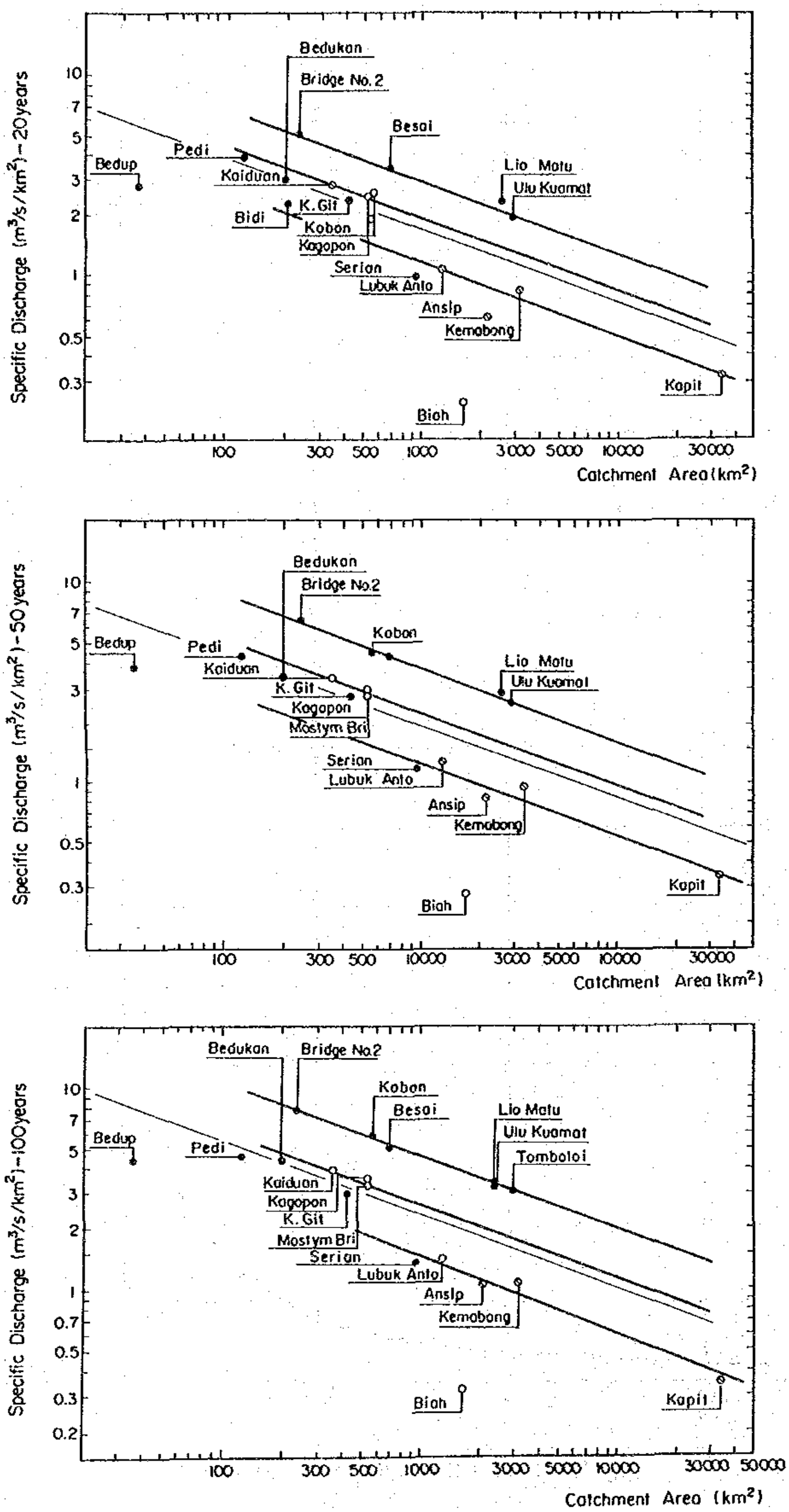
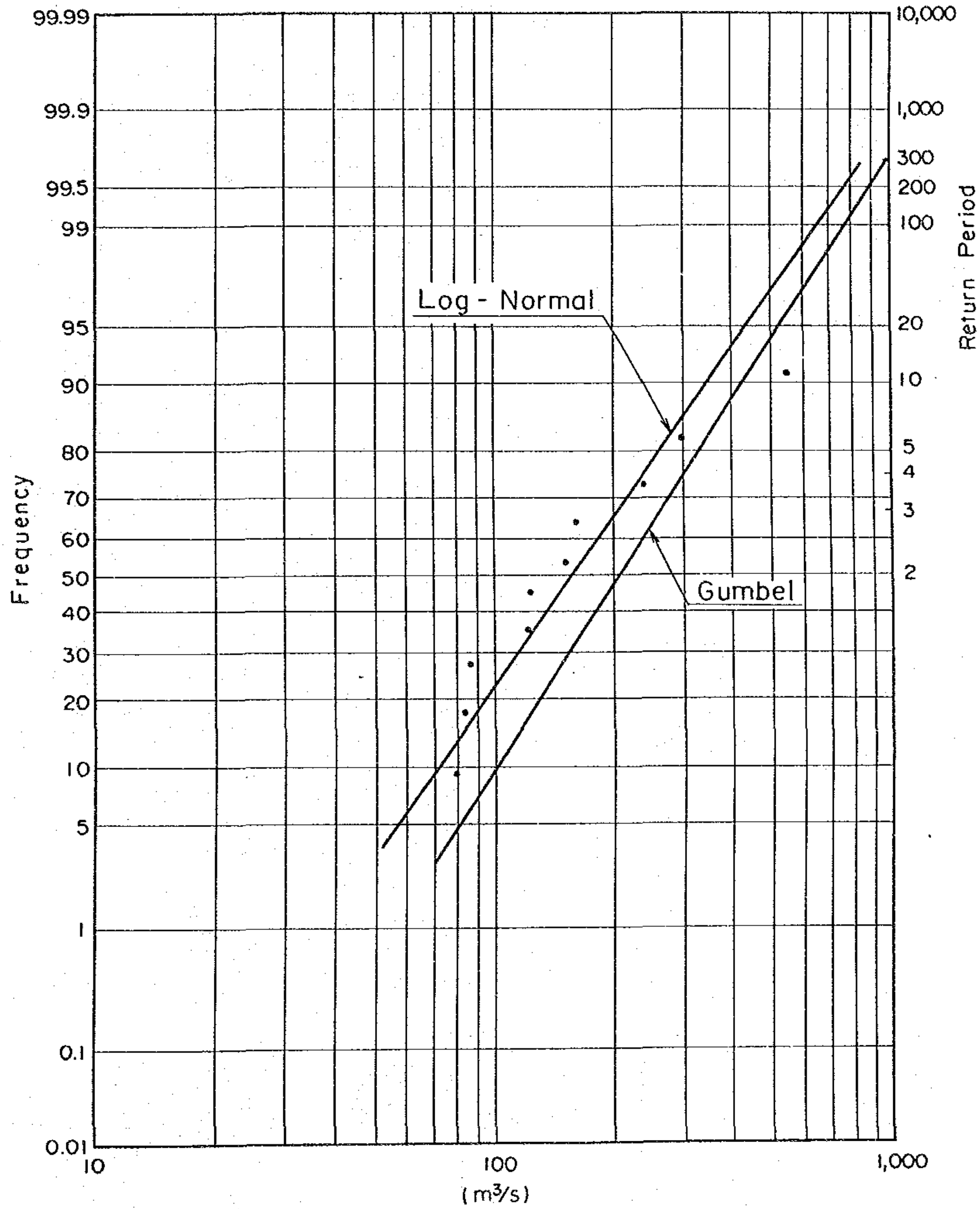


Figure 8-10 Flood Discharge at Bedukan Gauging Station



Chapter 9 SELECTION OF OPTIMUM PLAN

Chapter 9

SELECTION OF OPTIMUM PLAN

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9. SELECTION OF OPTIMUM PLAN

9.1 Outline

Selection of an optimum development plan means to select a net head, a design maximum discharge, and an installed capacity by comparing benefit and cost for alternative plans at the Naradaw site.

Two alternative plans with net heads 115 m and 170 m are compared, and it results that 170 m is the optimum net head. Then, four alternative plans having different installed capacities and design maximum discharges with the net head 170 m are compared. As the result, 1,600 kW and 1.20 m^3/s (64% flow in a year) are selected as the optimum one.

The optimum plan has the most advantage in economy among four alternative plans and also has the same economy as a diesel plant because it well meets the demand in late 1990s and early 2000s.

Benefit and cost for each alternative plan, which are estimated taking into account major characteristics in terms of river runoff (average for 10 years), water utilizations in the catchment area, environmental preservation, demand and supply in the Ranau-Kundasang system, energy and output generated from each plan, and construction cost for each plan.

The Naradaw scheme is summarized based on the optimization of a development plan in this chapter and preliminary design carried out in Chapter 11 as follows;

The Naradaw scheme, the fifth run of river plant, has a major objective to reduce diesel oil and contribute to the Ranau-Kundasang system in economy. It locates 6 km west of Ranau town, and has catchment areas of 59 km^2 which enables design maximum discharge of 1.20 m^3/s. The installed capacity is 1,600 kW with a net head of 170 m.

Two intakes located on the Liwagu River and the Mesilau River. Two ponds with an effective capacity of 1,400 m^3 in total are set just at downstream of the intakes. Two steel headraces (Liwagu: 0.7 m dia. 2,680 m length; Mesilau: 0.6 m dia. 990 m length) send water to a penstock. The penstock has a diameter of 0.8 m and length of 780 m. The powerhouse is located just upstream side of a suspension bridge across the Liwagu River near the junction of the two rivers.

Two Turgo Impulse type turbines and generators are installed for supplying power to the Ranau-Kundasang distribution system through an existing 11 kV transmission line.

The Naradaw scheme has a capability to generate firm peak power of 460 kW and annual supply capable energy of 9.5 Gwh which meet base and middle load in the system taking into consideration the minimum river maintenance water of 0.15 m^3/s which is recommended by the environmental investigation.

The Naradaw Scheme Parameter

1. Inst. Capacity	1,600 kW			
2. Supply Capable Energy	9.5 GWh			
3. Development Plan	(Tot.)	(Liwagu R.)	(Mesilau R.)
(1) Catchment Area	59 km^2	31 km^2	28 km^2
(2) Des. Flow	1.2 m^3/s	0.7 m^3/s	0.5 m^3/s
(3) Intake WL		1,050 m	1,038.5 m
(4) T'race WL	852 m		
(5) Gross Head	198 m		
(6) Head Loss	28 m		
(7) Net Head	170 m		
(8) Firm Power	290 kW	(95% in a year)	

(9)	Firm Peal Power	460 kW (95% in a year)	

4. Layout

(1)	Intake	Concrete Type	
(2)	L.P. Pipe	(Liwagu R.)	(Mesilau R.)
		L: 2,680 m	990
		D: 0.7 m	0.6
(3)	Head Pond	2,000 m^3	
(4)	Penstock	D: 0.8 m	L: 780 m
(5)	Power Station	Structural Steel Superstructure	
(6)	Turbine	Type: Turgo Impulse/3 Synchronous x 2 units	
(7)	Generator	890 kVA x 2 units, 1,000 RPM	
(8)	Transformer	890 kVA x 2 units, 3ϕ OA	
(9)	Trans. Lines	Type: Steel Post Size: HAL 0.166 sq. in 11 kV. Length 1.0 km	

9.2 Development Basic Concept

The Naradaw scheme, run of river type, has a major objective on reducing the dependence of diesel oil as a generation source as well as contributes to realization of economical system in the future. Based on the objective, basic development concept of the Naradaw scheme will be expressed as follows:

- The scheme should efficiency utilize the hydro potentials in Sabah.

- The scheme should generate much energy to meet base and middle load in the Ranau-Kundasang system because it can provide the system an advantage to be free from oil price escalation in a future. Installed capacity, however, should be appropriate taking into account demand growth and existing hydro's supply capability in the system.

- Its economy should not be less than the one of a diesel plant which might be developed if the Naradaw scheme is not done.

- It is assumed in optimization that the Naradaw scheme will be completed at the end of the year 1999 and will start to generate energy at the beginning of the year 2000. The Naradaw should be developed as soon as possible since annual energy generated from diesel plants is predicted to increase remarkably at the end of 1990s under the condition that existing hydro power stations (Carabau and Mesilau) generate full output. The year 2000 is, however, a limit for the Naradaw to realize marginal benefit cost ratio. It is considered that marginal benefit cost ratio (1.0) is sufficient for the Naradaw because the Naradaw has more advantage than the one calculated under the condition of no oil price escalation.

  The Naradaw will be developed faster if existing hydros do not generate full output, which is difficult to be predicted in present. (The commissioning year of the Naradaw is investigated in **Section 9.4** in detail.)

9.3 Optimization Method

The Optimization of the development plan, namely, net head, design maximum discharge, and installed capacity is performed by the following procedure (Refer to **Table 9-1**.)

(1) Alternative Plans

Alternative plans are prepared using the topographical map with scale of 1/2,500 provided by the Sabah State Land and Survey Department. Two plans have different net heads and layouts in terms of intake, headrace, penstock in order to compare economic difference between two net heads. After optimizing net head, four plans are made using the optimum net head to compare four different design maximum discharges, which cover fluctuation of river runoff from 45% to 77% flow in a year. (Refer to **Fig. 9-1, 9-2 and Table 9-2**)

(2) Intake Water

The following runoff data are utilized for estimating output and energy of each alternative plan. (Refer to **Fig.9-3**)

Average daily runoffs for 365 days at the Naradaw site are calculated using runoff data measured at the Bedukan Gauging Station from 1970 to 1979.

Average daily intake waters diverted from intakes are calculated taking into account water utilizations and environmental preservation along the river as follows; Mesilau Power Stations divert water from the Mesilau River to the Liwagu River; Rural Development Co. diverts water of maximum 0.1 m^3/s from Mesilau Power Stations to supply water for agriculture and daily life; River maintenance water of minimum 0.05 m^3/s is released from two intakes (total 0.1 m^3/s) to maintain the river flow between two intakes and the power house. (Minimum river maintenance water 0.1 m^3/s in total is an assumption in this optimization. On the other hand, Minimum river maintenance water 0.15 m^3/s (0.1 m^3/s in the Liwagu River and 0.05 m^3/s in the Mesilau River), recommended from the environmental investigation, is adopted to estimate annual energy in **Section 9.5** for economic evaluation conducted in **Chapter 15**.)

(3) Output and Energy

Daily outputs generated from two generators for each alternative plan are calculated by using daily intake waters mentioned above. A maximum output for each plan is assumed to decrease to 90% of an installed capacity due to corrosion of steel pipe headraces.

Annual energy generated is gotten by multiplying output by operating hours. Supply capable energy is gotten by subtracting both the energy for station service and the one lost during outages from energy generated. Supply capable energy is divided into saleable energy and diverted energy. Saleable energy means energy consumed in the system. Diverted energy means energy not consumed in the system, therefore intake water flows out of the spillway.

A firm power is defined as an output generated on 95% day in a year. A firm peak power is calculated by adding output generated from pond water to the firm power during lighting hours. (Refer to **Table 9-3**)

(4) Construction Cost

Construction cost for each plan contains all the cost for preparatory, civil works, turbines and generators, contingencies, and engineering. The construction cost is the price in 1991, and it does not contain interest during construction and any inflation after 1991.

It is estimated by an simplified method. Since most of labors, materials, and construction machinery required for civil works will be procured in Malaysia same as the Carabau hydro project, the cost of civil works is estimated by the unit prices used in planning the Carabau hydro project in 1986. The Carabau's unit prices are adjusted considering inflation from 1986 to 1991 and some improvements of the specifications. The cost of the turbines and generators, which will be imported, is approximately estimated based on international price level.

(Unit prices adopted in this Chapter are different from the ones that are estimated based on preliminary design of the optimum plan described in **Chapter 13**.)

(5) Evaluation Method

Because alternative plans have various characteristics in terms of construction cost, saleable energy, and cash flow during service life, evaluation and comparison thereof will be performed by those *benefit cost ratio*.

Benefit cost ratio is calculated by converting the Benefit and Cost to present values thereof to the beginning of first year (1997) when the investment in the Naradaw scheme is implemented at a discount rate of 10%.

The Cost on the Naradaw scheme consists of construction cost, operation and maintenance cost during the service life (25 years).

The Benefit on the Naradaw scheme is evaluated by the cost of an alternative diesel plant, which generates the same saleable energy and output (equivalent to a firm peak power) as those of the Naradaw scheme taking into account differences between a diesel plant and a hydro plant such as service life and outage rate shown in **Table 9-4**. The Benefit consists of construction cost, operation and maintenance cost, and fuel cost of the alternative diesel plant during 25 years.

9.4 Optimum Plan

9.4.1 Optimum Net Head

The Team prepared two kinds of development plans which have different net heads, 115 m and 170 m. Both powerhouses are located at a same place, while the intakes are located at different places. The plan with net head 115 m is similar to Naradaw A mentioned in Section 5.2

(Site Selection). The other plan with net head 170 m is similar to Naradaw D. **Table 9-2** shows parameters of the two plans such as output and annual energy, construction cost, and benefit cost ratio.

The comparison indicates that the net head 170 m is favorable because the bigger the net heads is, the better the benefit cost ratio is.

Judging from it's topographical condition, the plan with net head 170 m has a possibility to construct a pond of ample capacity with reasonable cost. However, it is difficult to construct an equivalent pond at a higher level than this plan. It means 170 m is the highest net head at the Naradaw site.

As the result, the net head 170 m is considered to be optimum at the Naradaw site.

9.4.2 Optimum Installed Capacity (Maximum Discharge)

Four alternative plans which have different installed capacities with an optimum neat head 170 m are prepared in order to optimize an installed capacity and a design maximum discharge. Alternative capacities are 1,200 kW, 1,600 kW, 2,000 kW, and 2,400 kW. Alternative discharges are 0.89 m^3/s, 1.20 m^3/s, 1.48 m^3/S, 1.78 m^3/S, which mean 77% flow, 64% flow, 54% flow, 45% flow in a year. **Table 2-2** shows the comparison between four plans.

Benefit cost ratios of four plans are directly affected by both those saleable energies and construction costs. Construction costs of four plans are simply proportion with those installed capacity. While, saleable energies of four plants have three characteristics as follows. These increase year by year and reach supply capable energy 8-13 years after the completion. The smaller the installed capacity is, the faster its saleable energy reaches the supply capable energy. The larger the installed capacity is, the more the supply capable energy is (Refer to **Table 9-2, Fig 9-4, 9-5**)

The three are the typical characteristics in case of developing a series of mini hydro in a small supply system isolated from the others, which should be carefully studied.

Benefit cost ratio for each plan reflects three characteristics contained in saleable energy. Benefit cost ratios result in 0.99 for the 1,200 kW plan, 1.01 for the 1,600 kW plan, 0.97 for the 2,000 kW plan 0.94 for the 2,400 kW plan. (Refer to **Table 9-2 and Fig. 9-5**)

The comparison suggests the follows:

- The 1,600 kW plan well meets the demand in early 2000s. It has the most advantage in economy among four alternative plans in case of commission in the year 2000 or early 2000s. It also has the same economy as a diesel plant.

- The 1,200 kW plan is relatively small for the hydro plant developed in 2000s.

- The 2,000 kW plan and 2,400 kW plan are relatively too large. Even if the two plans have a capability to generate much energy, the demand in early 2000s has a small room to consume only a part of those energy. It might result in that the investment for the project works partly in 2000s. (If the whole energy were immediately consumed in the system or if it were the hydro potential study without taking demand into consideration, the plans would have dominated the 1,600 kW plan.)

As the result, the installed capacity 1,600 kW and the design maximum discharge 1.20 m^3/S are selected as the optimum plan at the Naradaw site.

9.4.3 Study on Commissioning Year

(1) Assumption for Commissioning Year in the Optimization

It is assumed in optimization that the Naradaw scheme will be completed at the end of the year 1999 and will start to generate energy at the beginning of the year 2000 because of the follows:

- The Naradaw should be developed as fast as possible because reduction of diesel oil cost is first priority in the system. For instance, in case that Carabau 2 units work, if the Naradaw is not developed, energy generated from diesel plants in 1999 will exceed 6 Gwh, which is nearly equivalent to the one before completion of the Carabau and will become a burden to the system.

- Benefit cost ratio of the Naradaw should not be less than 1.0. The year 2000 is a limit for the Naradaw to realize it because the faster the Naradaw is completed, the less benefit cost ratio is as shown below:

Installed capacity	1,200 kW	1,600 kW	2,000 kW	2,400 kW
Benefit Cost Ratio Commission in 2000	0.99	1.01	0.97	0.94
Benefit Cost Ratio Commission in 2003	1.07	1.12	1.09	1.07

- It does not need for SEB to realize higher benefit cost ratio like 1.1, because the first priority for the Naradaw scheme is to reduce diesel oil and also benefit cost ratio would only be a measure of the economy under the condition that the Government of Malaysia grants all the construction cost for a mini hydro plant.

(2) Commissioning Year of Naradaw 1,600 kW Scheme in Connection with Carabau Plants

The Carabau P.S has a capability to generate much energy in a year, which meets a large portion of demand in the system. The two units of the Carabau were occasionally repaired, and it is not sure whether the two will be able to generate power stably in a future or not. As these issues affect it, the commissioning year of the Naradaw 1,600 kW scheme is studied in detail under the conditions that the Carabau two units work and the Carabau one unit work. Each saleable energy generated from hydro plants is calculated taking into account kwh balance and role of hydro and diesel plants, that is, Mesilau, Carabau, Naradaw, and diesel plants meet the demand in that order.

<u>In Case That Carabau Two Units Work</u>

The Carabau two units have a capability to generate 9.5 GWh in a year. 50% of that is estimated to be consumed in the system (as saleable energy) in 1992, and 90% of that in 1998.

The Naradaw has a possibility to generate 9.7 GWh in a year with river maintenance water of 0.10 m^3/s (9.5 GWh with river maintenance water of 0.15 m^3/s). The saleable energy, however, is estimated to be little in late 1990s, 50% of that in 2000, 90% of that in 2007. (Refer to **Table 9-5**)

Since the benefit cost ratio (B/C) of the Naradaw is proportion to an amount of saleable energy, the more lately the Naradaw is developed, the better the B/C is. On the other hand, if the Naradaw is not developed, energy generated from diesel plants is estimated to be 50% of the total energy in the system in early 2000, and it will increase year by year. As B/C of the Naradaw become marginal, the year 2000 is considered to be commissioning year for Naradaw in case that the Carabau two units work. (Refer to **Fig. 9-6**)

Benefit Cost Ratio of Naradaw Scheme
(In case of Carabau 2,000 kW)

Commissioning Year	Saleable Energy (GWh)	Construction Cost (1,000 M$)	Benefit/Cost
2000	7.4	11,500	1.01
2001	7.8	11,500	1.05
2003	8.6	11,500	1.12

Note (1) Saleable energy is an average for 10 years from each commissioning year.

(2) Minimum river maintenance water is assumed to be 0.1 m^3/s in total.

(3) Construction cost of M$11,500,000 is simply estimated for the purpose of the optimization only.

In Case that Carabau One Unit Works

The Carabau one unit have a possibility to generate 6.1 GWh in a year. Almost of that will be utilized in the system in middle 1990s. Saleable energy of the Naradaw is estimated to be 50% of its capability 9.7 GWh in 1997, and 90% in 2002. (Refer to **Table 9-6**) If the Naradaw is not developed, energy generated from diesel plants exceeds 50% of total energy in late 1990s, and it will increase year by year too. Since B/C of the Naradaw becomes marginal, the year 1997 is considered to be commissioning year for the Naradaw in case that Carabau one unit works.

Benefit Cost Ratio of Naradaw Scheme
(In case of Carabau 1,000 kW)

Commissioning Year	Saleable Energy (GWh)	Construction Cost (1,000 M$)	Benefit/Cost
1996	7.1	11,500	0.98
1997	7.7	11,500	1.03
2000	8.9	11,500	1.15

Note; same as the note mentioned above.

9.5 Characteristics of Output and Energy

Daily outputs of run-of-river plants generally fluctuate in connection with the fluctuation of daily river runoffs. It will provide SEB important information in operating both hydro and diesel plants in the system to study characteristics of output and energy of hydro.

In this study, output and energy of the Naradaw scheme is estimated based on minimum water requirement at the river between two intakes and the powerhouse of 0.15 m^3/s instead of 0.10 m^3/s assumed in the optimization, because the environmental investigation recommends that minimum water requirement should be 0.1 m^3/s at the Liwagu River and 0.05 m^3/s at the Mesilau River, 0.15 m^3/s in total after completion of the optimization study stated in **Section 9.4**.

9.5.1 Saleable Energy of the Naradaw Scheme

Supply capable energy of the Naradaw scheme in an average year is first estimated based on average daily runoffs for 365 days and river maintenance water 0.15 m^3/s. Then, Saleable energy of the Naradaw is estimated taking into account kWh balance and role of hydro and diesel plants, that is, Mesilau, Carabau, Naradaw, and diesel plants meet the demand in that order same condition as **Section 9.4**.

Annual supply capable energy of the Naradaw is 9.5 GWh under the condition of river maintenance water 0.15 m^3/s (9.7 GWh under the condition of 0.1 m^3/s).

Saleable energy of the Naradaw indicates the same characteristics as the one stated in **Section 9.4**. In case that Carabau two units work, saleable energy is estimated to increase year by year, it is little of the capability 9.5 GWh in late 1990s, around 50% of that in 2000, 90% of that in 2007.

In case that Carabau one unit works, saleable energy is estimated to increase earlier than the one in case of Carabau two units, it is around 30% of the capability 9.5 GWh in 1995, 50% of that in 1997, and 90% of that in 2003. (Refer to **Table 9-6**)

The saleable energy is utilized for the economic evaluation in **Chapter 15**.

9.5.2 Supply Capable Output of Hydro

Supply capable outputs in an average year are estimated based on average daily runoffs for 365 days and river maintenance water 0.15 m^3/s.

The Naradaw scheme has a possibility to generate its maximum output during around 56% in a year, then decreases outputs in proportion with daily river runoffs. The firm power, 95% output in a year, is estimated to be around 290 kW. The firm peak power is evaluated to be 460 kW taking into account the pond capacity 1,400 m^3/s and characteristics of peak demand.

The Mesilau three P.S can also generate their maximum outputs during around 56% in a year. The firm power is estimated to be 15 kW. The Carabau P.S, on the other hand, can generate its maximum output only during 20% in a year in case that two units work, and reduces the outputs. The firm power is estimated to be 185 kW. The firm peak power is 430 kW taking to account the pond capacity of 2,000 m^3.

9.5.3 Monthly and Annual Energy of Naradaw

Daily outputs, monthly and annual energies generated from the Naradaw scheme for 10 years are estimated by a large computer based on daily river runoffs for 3,650 days from 1970 to 1979. (Energy generated is different from supply capable energy as defined in **Table 9-3**). The estimation will provide SEB important information to operate the Naradaw for many years.

An average annual energy generated for 10 years is 10.7 GWh with maximum of 11.9 GWh in 1977 and minimum of 9.1 GWh in 1973 and 1976.

Monthly energy generated fluctuates from 1.2 GWh (full output during a month) to 0.2 GWh in March 1973. Monthly energies in February, March and April, dry season, are relatively less than those in wet season. (Refer to **Table 9-8 and Fig. 9-8**)

Table 9-1 Optimization Procedure at Naradaw Site

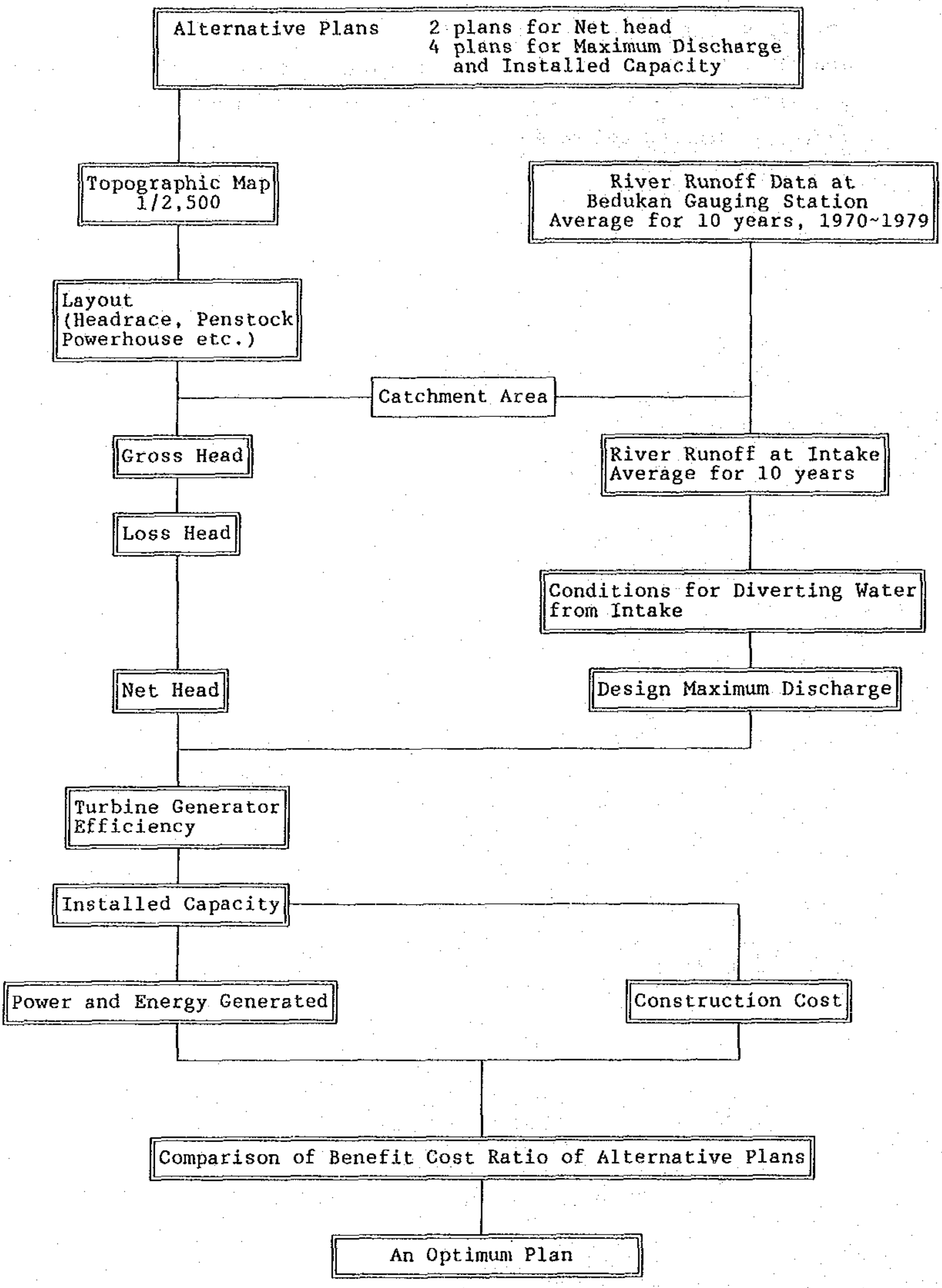


Table 9-2 Comparison of Alternative Plans

	Parameter	Unit	Net Head		Installed Capacity (Max. Discharge)			
			115m	170m	1,200kW	1,600kW	2,000kW	2,400kW
(1)	Catchment area	km^2	60.3	59	59			
(2)	Net head	m	115	170	170			
(3)	Design max. discharge	m^3/s	1.33	1.20	0.89	1.20	1.48	1.78
	(percent in a year)	%			(77)	(64)	(54)	(45)
(4)	Installed capacity	kW	1,220	1,600	1,200	1,600	2,000	2,400
(5)	Firm peak power (95%)	kW	400	560	560	560	560	560
(6)	Supply capable energy	GWh	7.2	9.7	7.8	9.7	11.3	12.8
(7)	Saleable energy	GWh	6.7	8.7	7.2	8.7	9.9	10.8
(8)	Construction cost	10^6M$	10.6	11.5	10.2	11.5	13.1	14.3
(9)	Benefit/Cost (Commission in 2000)		0.85	1.01	0.99	1.01	0.97	0.94
	Benefit/Cost (Commission in 2003)		0.92	1.12	1.07	1.12	1.09	1.07

(Note)
- Saleable energy at the parameter
- (7) is average for 25 years (2000-2024)
- Benefit/cost at the parameter (9) is calculated by (7) and (8)
- Minimum water requirement at the river between the intakes and the powerhouse is assumed to be 0.1 m^3/s in total.

Table 9-3 Calculation Conditions of Output and Energy

Parameters		Unit	Alternative Plans
Output	P	kW	9.8 x nt x ng x Q x H
Discharge	Q	m^3/s	365 days Average of 10 years
Net head	H	m	170/115
Turbine efficiency	nt		0.835
Generator efficiency	ng		0.96
Output		kW	8 x Q x H < Pmax
Installed capacity		kW	8 x Qmax x H
Maximum output	Pmax	kW	8 x Qmax x H x (1-derating rate)
Firm Power		kW	8 x Q_{95} x H
Firm Peak Power		kW	8 x (Q/0.63) x H
Annual energy generated		kWh	8 x Q x H x operating hours
Supply capable Energy		kWh	Energy generated x (1-station service rate) x (1-outage rate)
Saleable Energy		kWh	Saleable energy = supply capable energy - diverted energy from spillway

Table 9-4 Calculation Assumption for Optimization

Description		Unit	Diesel	Hydro
(1)	Service Life	year	15	25
(2)	Station Service rate for kW	%	4	1
(3)	Station service rate for kWh	%	4	1
(4)	Outage rate	%	13	6
(5)	Derating rate	%	20	10
(6)	O&M cost ratio to constuction cost (Parameter for Diesel)	%	5	1.5
(7)	Construction cost per kW	M$/kW	1,395	
(8)	Kind of Fuel		Diesel Oil	
(9)	Fuel price per liter	M$/ℓ	0.50	
(10)	Specific gravity of fuel	kg/ℓ	0.81	
(11)	Fuel price per kg	M$/kg	0.62	
(12)	Thermal efficiency	%	30	
(13)	Specific heat consumption	Kcal/kWh	2,867	
(14)	Calorific value of fuel	Kcal/kg	10,000	
(15)	Specific fuel consumption	Kg/kWh	0.287	
(16)	Fuel cost per kWh	M$/kWh	0.18	

Table 9-5 Saleable Energy of Hydro (In Case of Carabau 2,000kW, Naradaw 1,600kW)

Year	Demand		Before Naradaw Operation					After Naradaw Operation			
	Max. Demand (kW)	Annual Energy (MWh)	Energy Supplied (MWh)					Energy Supplied (MWh)			Diesel
			Mesilau Saleable 299 kW		Carabau Saleable 2,000 kW			Naradaw Saleable 1,600 kW		Hydro Total	
			Used	Disch.	Used	Disch.	Diesel	Used	Disch.	Used	
1992	1,690	7,719	1,925	0	4,796	4,690	998				
1993	1,920	8,921	1,925	0	5,589	3,896	1,407				
1994	2,200	10,208	1,925	0	6,345	3,140	1,938				
1995	2,520	11,583	1,925	0	7,036	2,449	2,622				
1996	2,740	12,943	1,925	0	7,614	1,871	3,404				
1997	3,020	14,267	1,925	0	8,115	1,371	4,227				
1998	3,320	15,715	1,925	0	8,579	906	5,211				
1999	3,640	17,201	1,925	0	8,891	594	6,385				
2000	3,930	18,958	1,925	0	9,103	383	7,930	4,632	4,841	15,660	3,298
2001	4,220	20,320	1,925	0	9,213	272	9,182	5,260	4,212	16,399	3,921
2002	4,530	21,843	1,925	0	9,311	174	10,607	5,923	3,549	17,160	4,683
2003	4,880	23,494	1,925	0	9,390	95	12,179	6,602	2,870	17,918	5,576
2004	5,230	25,204	1,925	0	9,448	37	13,831	7,232	2,241	18,605	6,599
2005	5,620	27,064	1,925	0	9,485	0	15,654	7,835	1,637	19,245	7,819
2006	5,960	28,715	1,925	0	9,485	0	17,305	8,225	1,247	19,635	9,080
2007	6,310	30,467	1,925	0	9,485	0	19,057	8,518	954	19,929	10,538
2008	6,690	32,325	1,925	0	9,485	0	20,915	8,751	721	20,162	12,163
2009	7,100	34,297	1,925	0	9,485	0	22,887	8,939	533	20,350	13,947
2010	7,590	36,585	1,925	0	9,485	0	25,175	9,118	354	20,529	16,056
2011	7,860	37,902	1,925	0	9,485	0	26,492	9,201	271	20,612	17,290
2012	8,150	39,267	1,925	0	9,485	0	27,857	9,273	199	20,684	18,583
2013	8,440	40,680	1,925	0	9,485	0	29,270	9,337	136	20,747	19,933
2014	8,740	42,145	1,925	0	9,485	0	30,735	9,389	83	20,800	21,345
2015	9,050	43,617	1,925	0	9,485	0	32,207	9,431	41	20,842	22,775
		(1)	(2)		(3)	(1)-(2)-(3)		(4)		(5)= (2)+(3)+(4)	(1)-(5)

Note (1) Maximum output of Carabau P.S is assumed to be 2,000 kW generated from 2 units.

(2) Minimum water requirement at the river between the intakes and the powerhouse is assumed to be 0.15 m^3/s in total.

Table 9-6 Saleable Energy of Hydro (In Case of Carabau 1,000kW, Naradaw 1,600kW)

Year	Demand		Before Naradaw Operation					After Naradaw Operation			
			Energy Supplied (MWh)					Energy Supplied (MWh)			Diesel
	Max. Demand (kW)	Annual Energy (MWh)	Mesilau Saleable 299 kW		Carabau Saleable 2,000 kW			Naradaw Saleable 1,600 kW		Hydro Total	
			Used	Disch.	Used	Disch.	Diesel	Used	Disch.	Used	
1992	1,690	7,719	1,925	0	4,616	1,507	1,178	794	8,678	7,335	384
1993	1,920	8,921	1,925	0	5,268	855	1,728	1,181	8,291	8,374	547
1994	2,200	10,208	1,925	0	5,656	467	2,627	1,872	7,601	9,452	756
1995	2,520	11,583	1,925	0	5,862	261	3,796	2,778	6,694	10,565	2,018
1996	2,740	12,943	1,925	0	5,992	124	5,019	3,670	5,802	11,595	1,348
1997	3,020	14,267	1,925	0	6,085	38	6,257	4,500	4,972	12,511	1,756
1998	3,320	15,715	1,925	0	6.123	0	7,667	5,365	4,107	13,413	2,302
1999	3,640	17,201	1,925	0	6,123	0	9,153	6,233	3,239	14,282	2,919
2000	3,930	18,958	1,925	0	6,123	0	10,910	7,177	2,295	15,226	3,732
2001	4,220	20,320	1,925	0	6,123	0	12,272	7,813	1,660	15,861	4,459
2002	4,530	21,843	1,925	0	6,123	0	13,795	8,276	1,196	16,325	5,518
2003	4,880	23,494	1,925	0	6,123	0	15,446	8,595	877	16,644	6,850
2004	5,230	25,204	1,925	0	6,123	0	17,156	8,839	634	16,887	8,317
2005	5,620	27,064	1,925	0	6,123	0	19,016	9,047	426	17,095	9,969
2006	5,960	28,715	1,925	0	6,123	0	20,667	9,195	277	17,243	11,472
2007	6,310	30,467	1,925	0	6,123	0	22,419	9,317	155	17,365	13,102
		(1)	(2)		(3)		(1)-(2)-(3)	(4)		(5)= (2)+(3)+(4)	(1)-(5)

Note (1) Maximum output of Carabau P.S is assumed to be 1,000 kW generated from 1 unit.

(2) Minimum water requirement at the river between the intakes and the powerhouse is assumed to be 0.15 m^3/s in total.

Table 9-7 Hydro Power Supply Capability

Average for 10 years, 1970 - 1979

Parameter		Unit	Mesilau			Carabau	Naradaw	Mesilau				Carabau	Naradaw	Total
			PH-1	PH-2	PH-3			PH-1	PH-2	PH-3	Total			
Catchment Area		km^2	11	11	11	32.1	59							
Net Head		m						72	87	161		197	170	
Maximum Discharge		m^3/s	0.198	0.140	0.098	1.27	1.18							
Installed Capacity		kW						105	85	109	299	2,000	1,600	3,899
Runoff Duration			Intake Water (m^3/s)					Supply Capable Output (kW)						
Day (th)	%	Run off m^3/s /100 km^2												
37	10	7.16	0.198	0.140	0.098	1.27	1.20	88	71	91	250	1,674	1,339	3,263
110	30	4.06	0.198	0.140	0.098	1.20	1.20	88	71	91	250	1,583	1,339	3,172
183	50	2.72	0.198	0.140	0.098	0.77	1.20	88	71	91	250	1.016	1,339	2,605
256	70	1.77	0.195	0.137	0.095	0.47	0.79	88	71	91	250	620	999	1,869
329	90	0.97	0.107	0.049	0.007	0.21	0.32	0	29	8	37	277	405	719
347	95	0.76	0.084	0.026	0	0.14	0.21	0	15	0	15	185	265	465
365	100	0.36	0.040	0	0	0.02	0.02	0	0	0	0	0	0	0

Note 1) Minimum water requirement at the river between the intakes and the powerhouse is 0.15 m^3/s at Naradaw.

2) Supply capable output: P=8x(Net head) x (Intake water) x 0.9 x 0.93

Table 9-8 Energy Generated from Naradaw Scheme

unit: MWh

Year	Jan.	Feb.	Mar.	Apr.	May	Jun.	Jul.	Aug.	Sep.	Oct.	Nov.	Dec.	Total
1970	1170	619	43	722	1029	1066	1092	1171	1030	1160	1150	1118	11764
1971	1182	1075	1190	725	606	858	483	1129	1107	1149	1150	1109	11768
1972	1154	978	1106	836	957	639	154	622	867	1036	995	863	10210
1973	370	36	206	518	939	825	900	863	1133	1089	1152	1025	9062
1974	1190	1075	1036	795	1083	585	906	909	415	647	585	858	10089
1975	1031	547	944	126	1048	875	923	1066	1152	1042	1110	1167	11036
1976	1117	901	603	436	838	413	711	699	423	835	1152	938	9071
1977	1168	1066	1009	587	815	1097	1144	913	580	1190	1152	1180	11906
1978	1101	65	621	652	1009	1118	1190	989	914	990	1152	1180	11586
1979	778	316	803	446	681	1104	1185	923	1067	1190	1152	1190	10841
Total	10266	7281	7952	5849	9009	8585	8692	9290	8693	10332	10751	10632	107338
Average	1026	728	795	584	900	858	869	929	869	1033	1075	1063	10733
Maximum	1190	1075	1190	836	1083	1118	1190	1171	1152	1190	1152	1190	1190
Minimum	370	36	206	126	606	413	154	622	415	647	585	858	36

Note 1) Installed capacity: 800 kW x 2 units
Maximum discharge : 1.2 m^3/s
Net heads : 170 m

2) Minimum water requirement at the river between the intakes and the powerhouse is 0.15 m^3/s based on the recommendation from the environmental investigation.

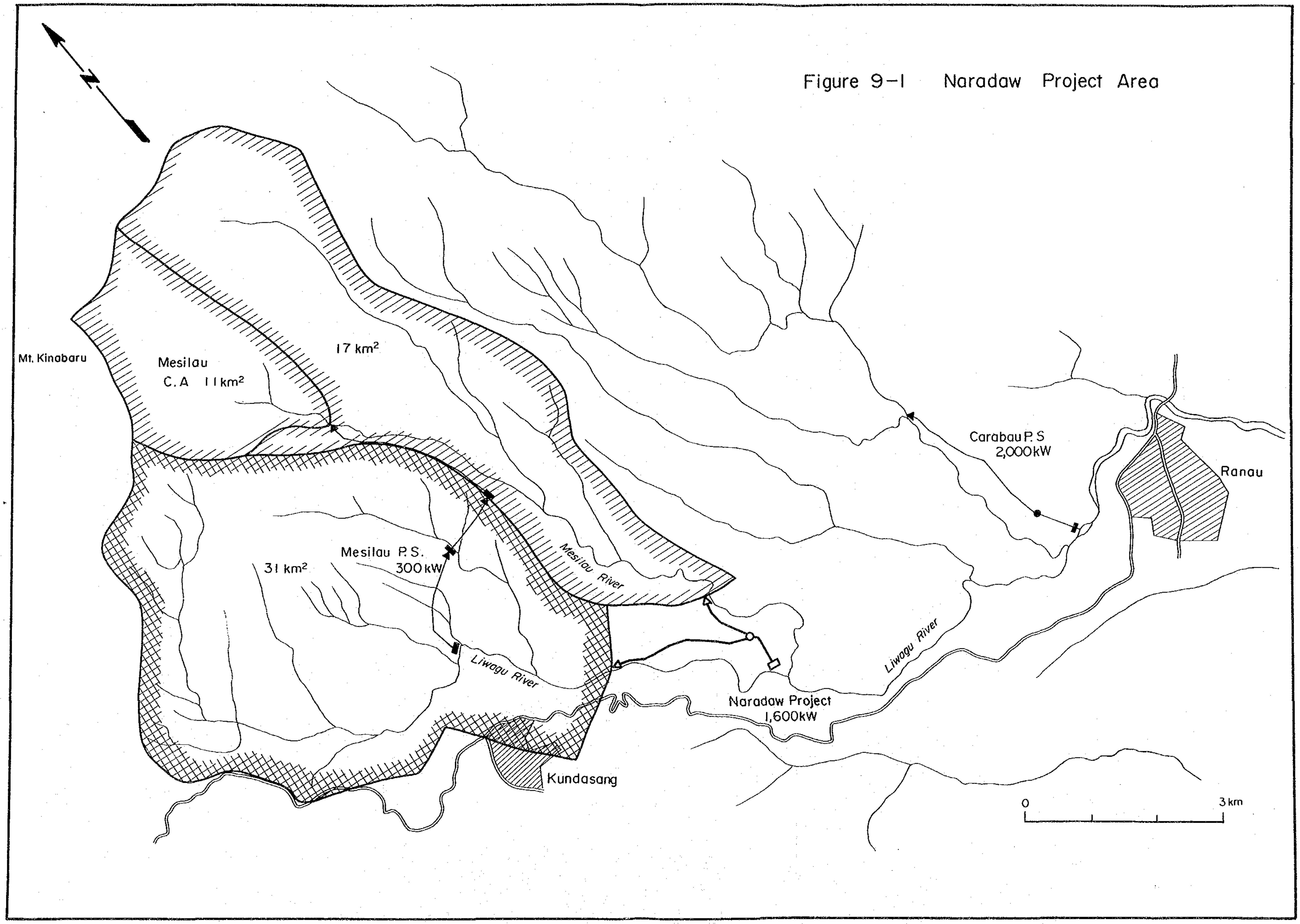
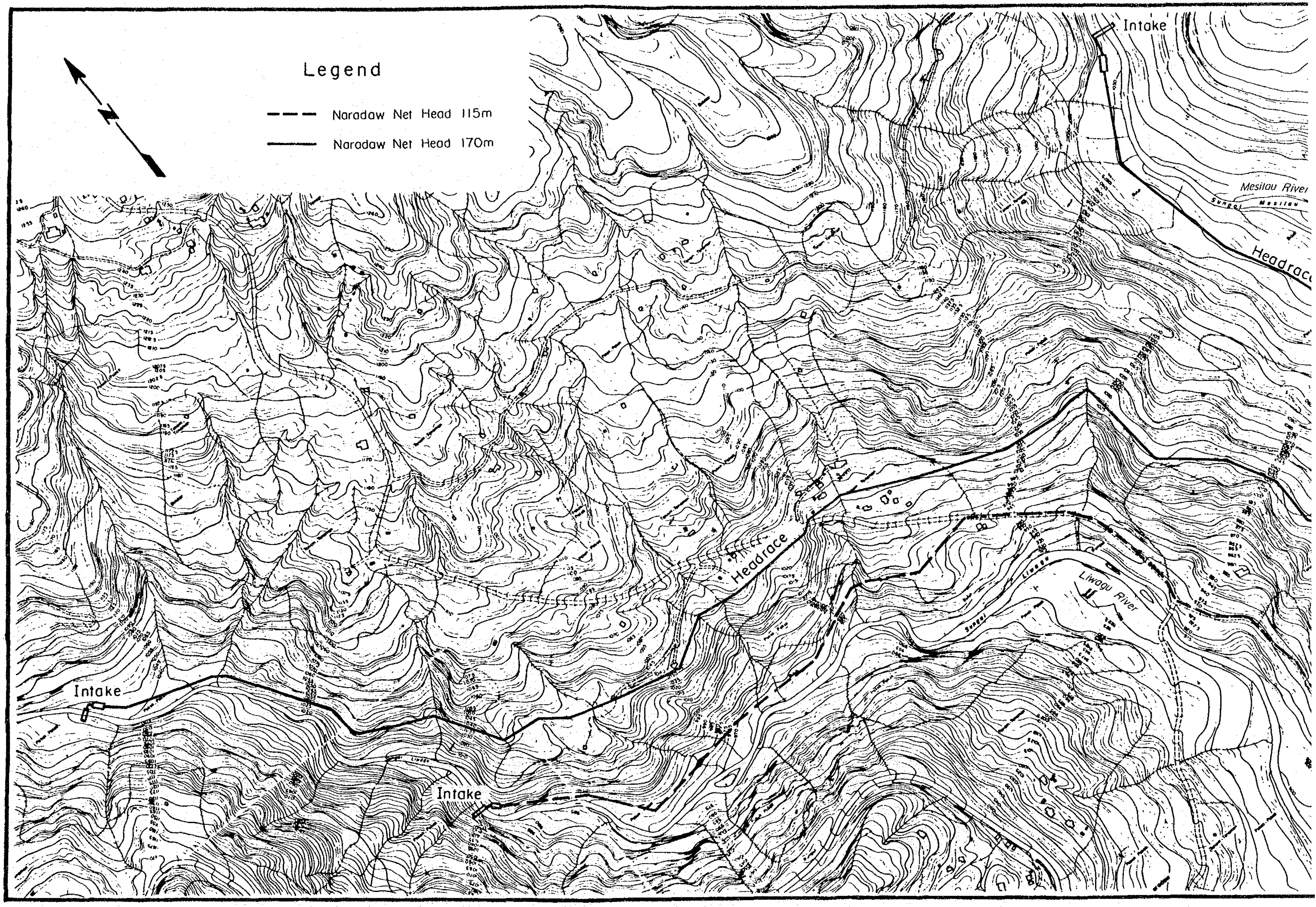


Figure 9-1 Naradaw Project Area
N
Mt. Kinabaru
Mesilau
C.A 11 km²
17 km²
31 km²
Mesilau P.S.
300 kW
Mesilau River
Liwagu River
Naradaw Project
1,600kW
Liwagu River
Carabau P.S
2,000kW
Ranau
Kundasang
0
3 km



Legend
Naradaw Net Head 115m
Naradaw Net Head 170m
N
Intake
Mesilau River
Sungai Mesilau
Headrace
Headrace
Liwagu River
Sungai Liwagu
Intake
Intake

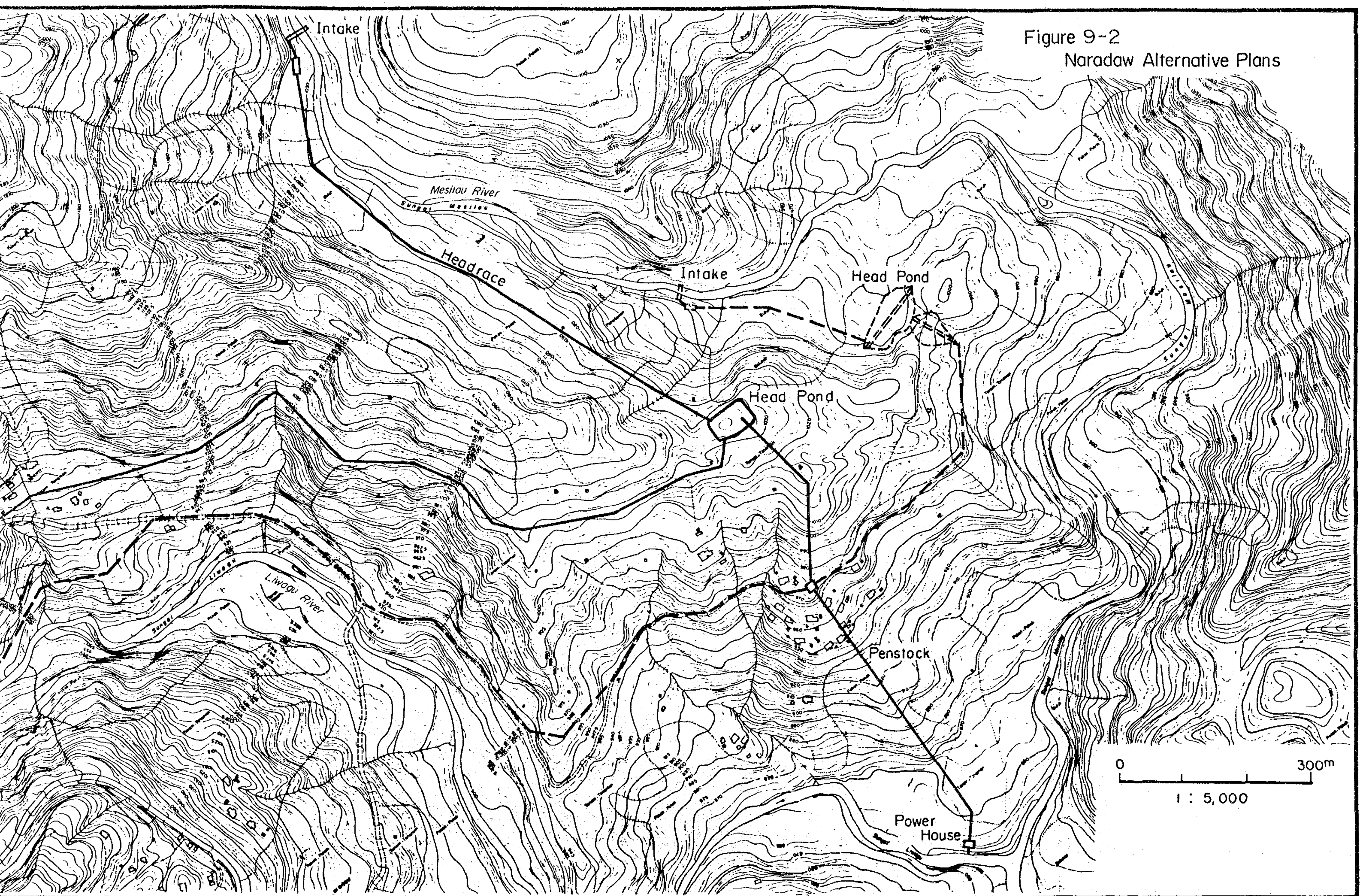


Figure 9-2
Naradaw Alternative Plans
Intake
Mesilau River
Headrace
Intake
Head Pond
Head Pond
Liwagu River
Penstock
Power House
0
300m
1 : 5,000

Figure 9-3 Discharge Duration and Intake Water at Naradaw Site

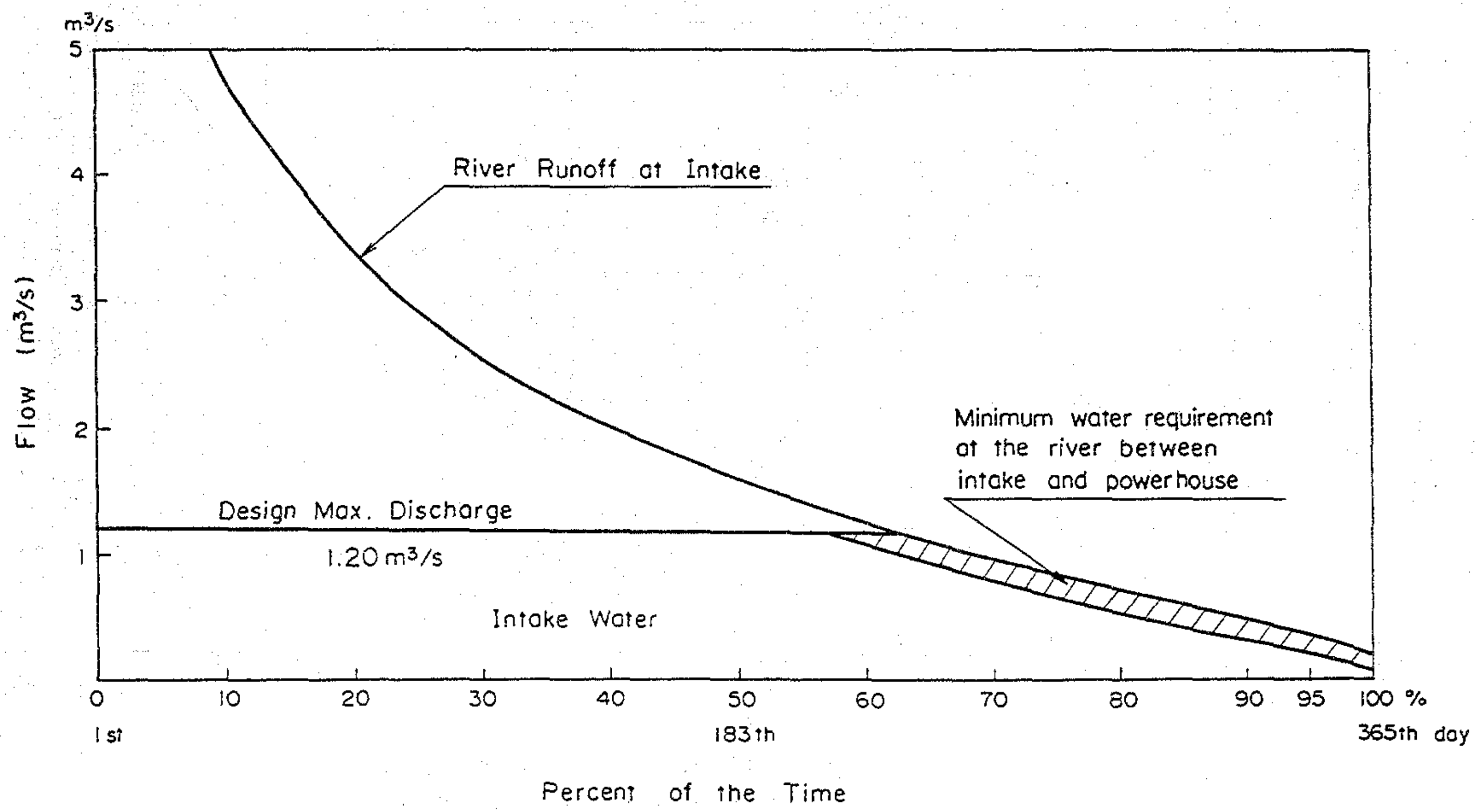
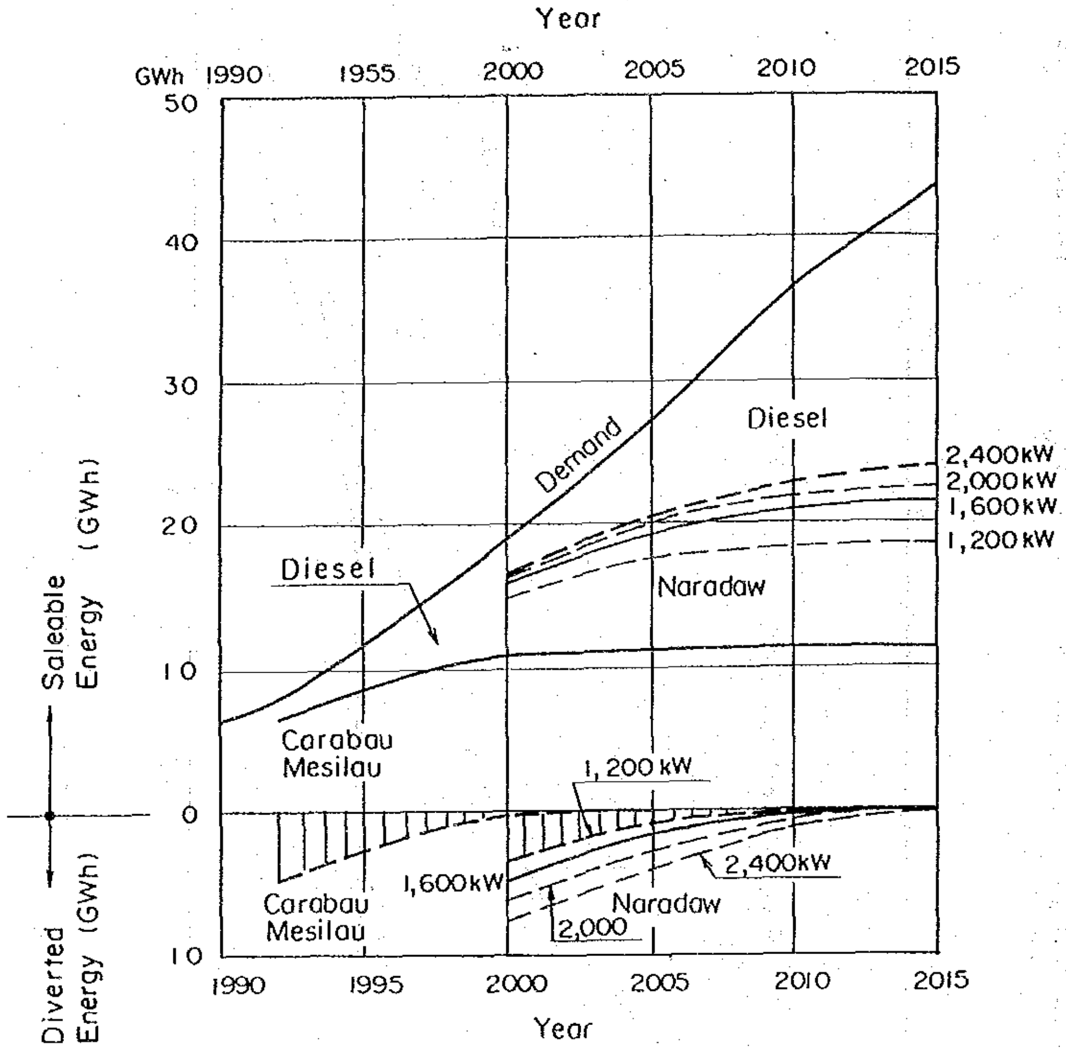


Figure 9-4 Optimization of Installed Capacity Saleable Energy

(In case that Carabau two units works)



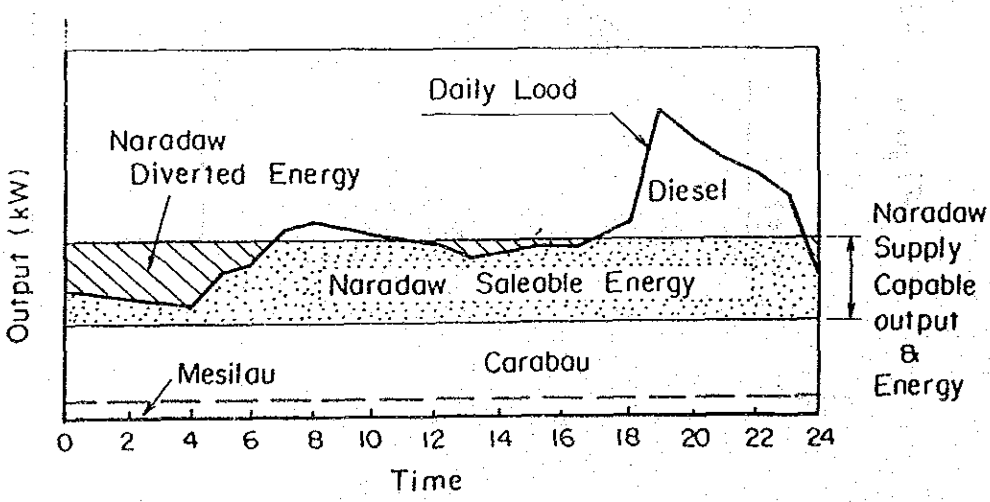
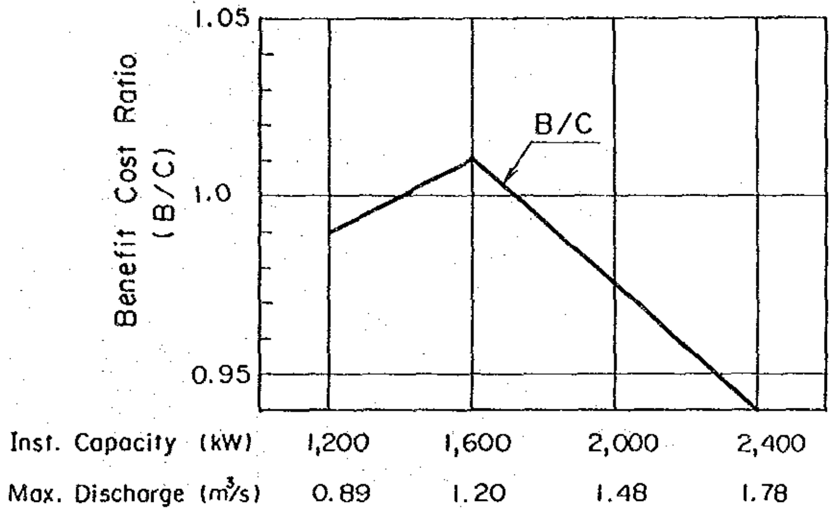


Figure 9-5 Optimization of Installed Capacity

(1) Benefit Cost Ratio (B/C)



(2) Saleable Energy and Construction Cost

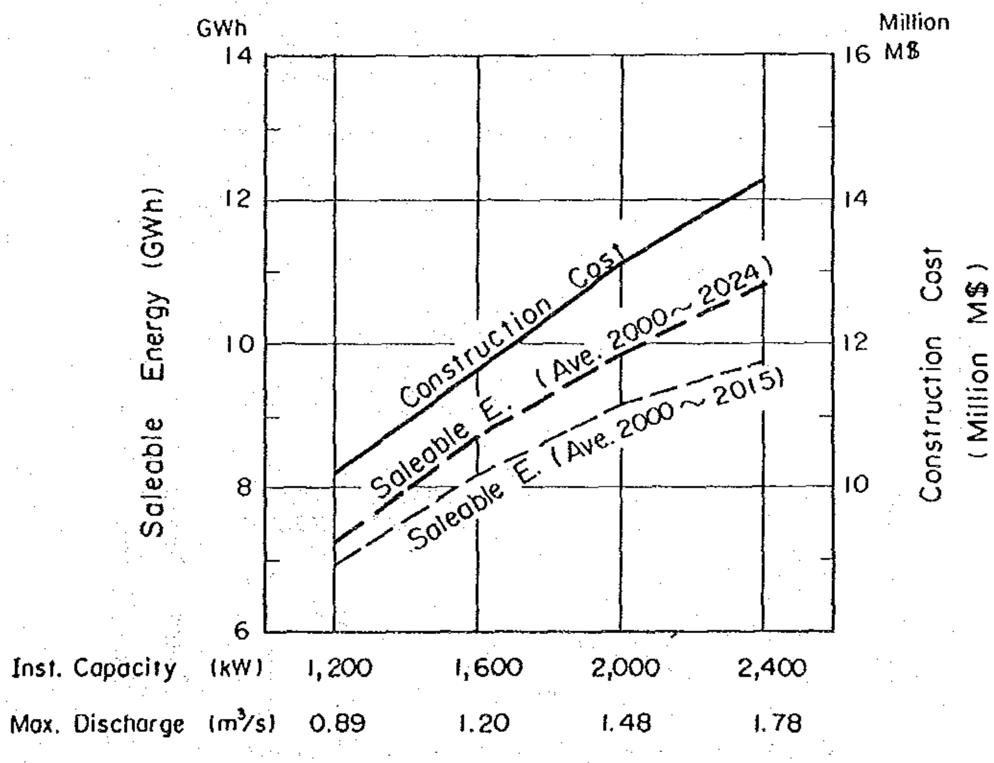
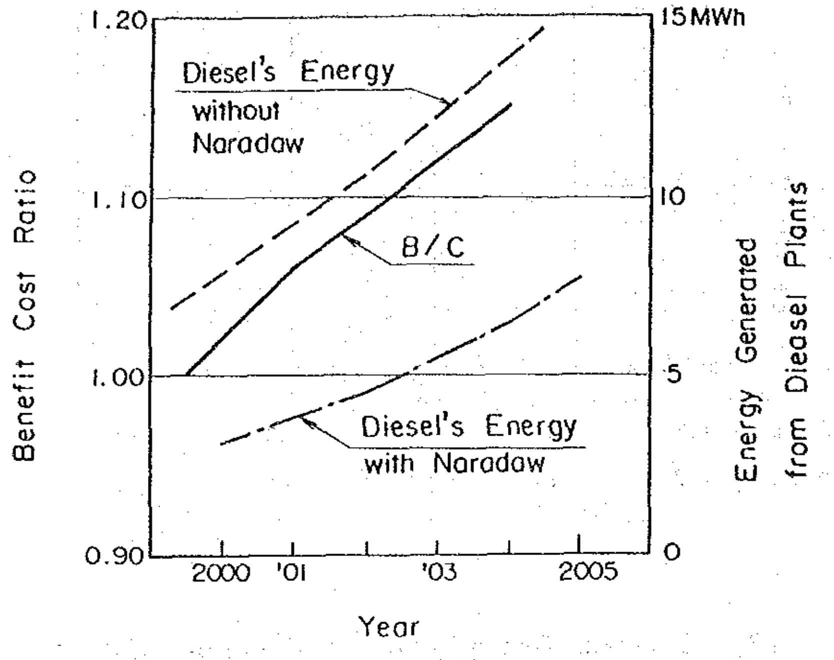


Figure 9-6 Commencement of Power Operation

(1) Benefit Cost Ratio of Naradaw
in Case That Carabau 2 units Work



(2) Benefit Cost Ratio of Naradaw
in Case That Carabau 1 unit Works

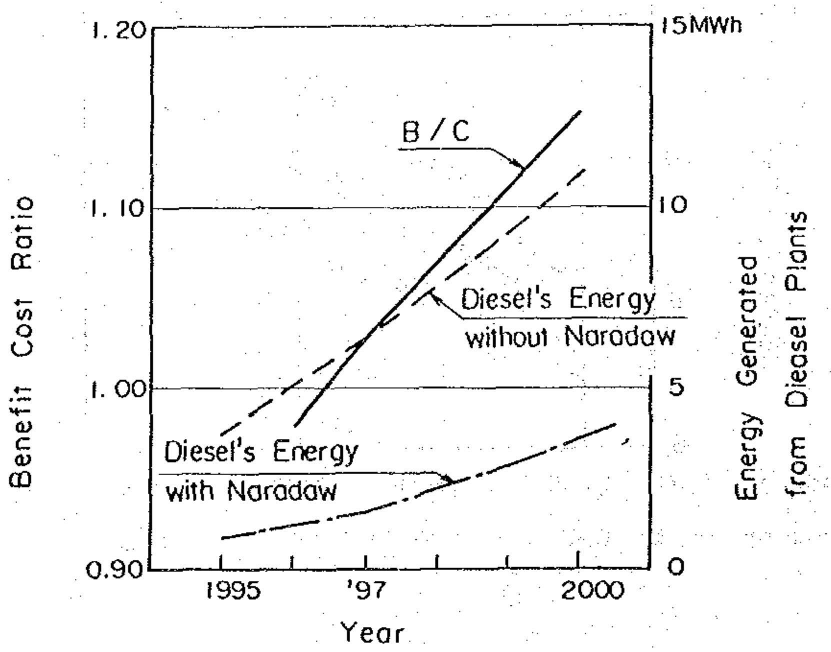


Figure 9-7 Hydro Power Supply Capability in a Year

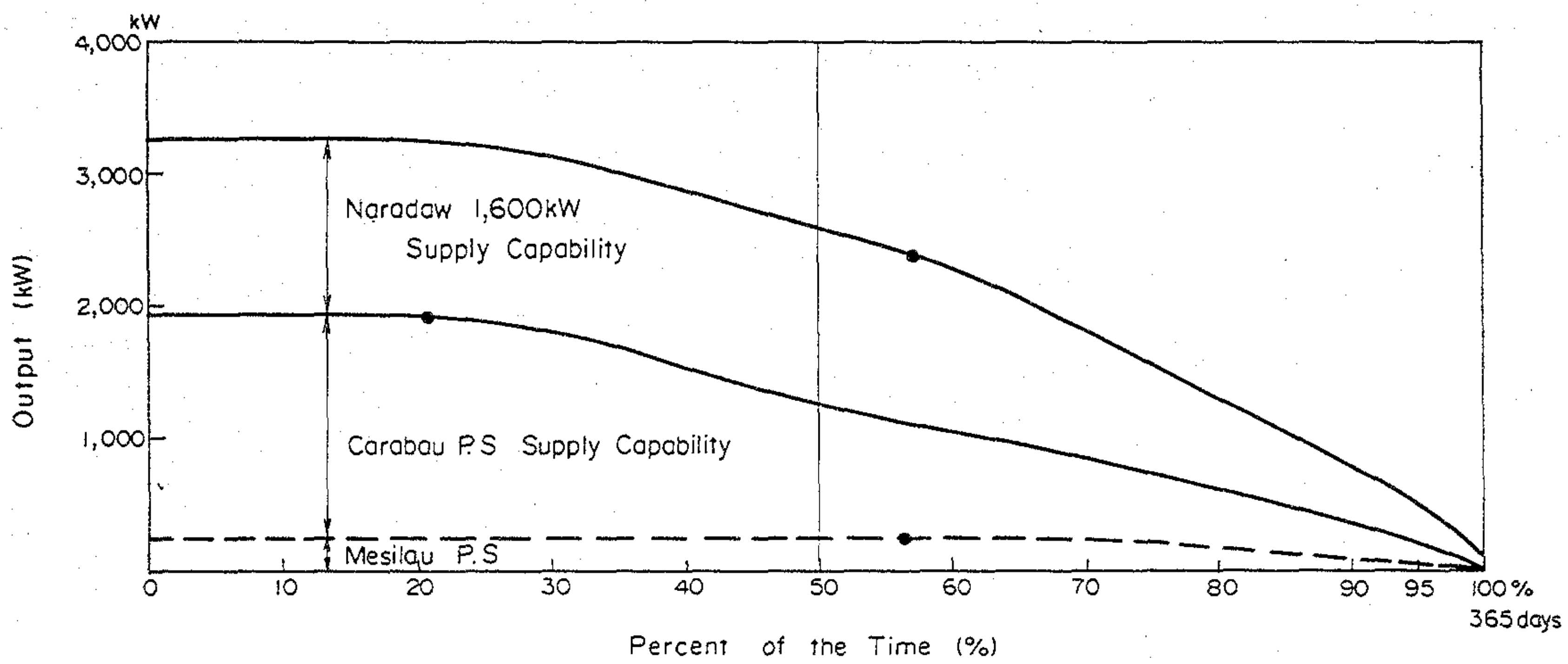
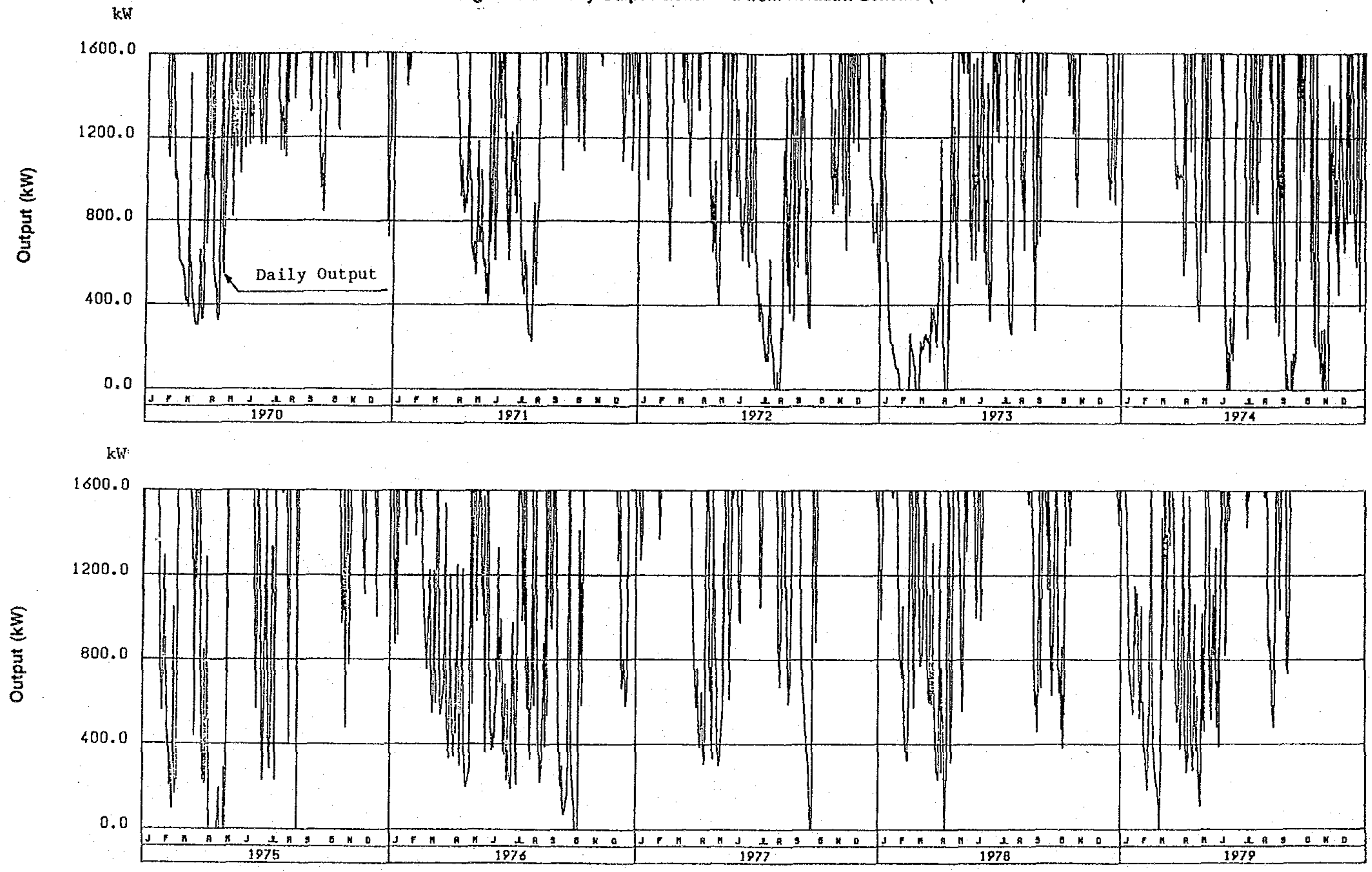


Figure 9-8 Daily Output Generated from Naradaw Scheme (1970 - 1979)



Note 1. Installed capacity 800 kW x 2 units, Max. Discharge 1.2 m^3/s Net Head 170 m.

2. Minimum water requirement at the river between the intakes and the powerhouse is 0.15 m^3/s.

Chapter 10 POWER TRANSMISSION LINE PLAN

Chapter 10

POWER TRANSMISSION LINE PLAN

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10. POWER TRANSMISSION LINE PLAN

The electric power generated at Naradaw Small Hydro Power Station will mainly be supplied to the towns of Ranau (number of customers at end of 1990: 2,296) and Kundasang (number of customers at end of 1990: 482) which are centers of electric power demand in the area. In addition, there are plans for expansion by 11-kV transmission lines in the near future to Bundu Tuhan (number of customers at end of 1990: 182) via Kinabalu National Park, and Poring (hot springs site) located 20 km northeast of Ranau Town. (See **Fig. 4-1**)

10.1 Selection of Power Transmission Pattern

The transmission voltages presently being used in Sabah State are as follows:

Nominal System Voltage	Highest Voltage for Equipment
11 kV	12.1 kV
33 kV	36.3 kV
66 kV	72.5 kV
132 kV	145 kV
275 kV (planned)	300 kV
500 kV (planned)	525 kV

The power system to be the object of Naradaw Small Hydro Power Station is of 11 kV, with total length of transmission lines, being 96 km of the 11-kV transmission lines, the 15 km between Ranau Diesel Power Station and Kundasang Diesel Power Station functions as an interconnecting transmission line connecting the two load centers by hard aluminum conductors (HAL) of cross-sectional area 0.1662 sq.in. and transmission capacity 2,000 kW.

The power generating output of Naradaw Small Hydro Power Station is planned to be 1,600 kW, and the following power transmission patterns are conceivable from the transmission capacities of the existing transmission lines and the relative locations of the abovementioned two load centers.

The technical and economic features of the abovementioned power transmission patterns are as follows:

	Transmission Pattern A	Transmission Pattern B	Transmission Pattern C
Construction cost comparison	M$140,000	M$1,701,500	M$949,000
Supply reliability	Poor	Good	Good
Transmission loss	3.0%	1.4%	3.0%
Order of selection	1	3	2

The length of the new transmission line to the power station when a T-branch is made from the existing transmission line will be 1 km. As shown in Appendix 5, the power station output for power transmission possible with voltage drop of 10% for the 10 km to the Ranau load center which is the longest transmitting distance from the Naradaw Small Hydro Power Station is 2,200 kW. In other words, this means that in case of generating power to use the existing 11-kV HV line, as the maximum output of Naradaw Power Station, up to 2,200 kW will be allowable.

The above-mentioned transmissible power from Naradaw Small Hydro Power Station means most severe condition because the total generating power is estimated to transmit only to Ranaw Town.

10.2 Principal Specifications of Transmission Line

The 11-kV HV line comprising the Ranau-Kundasang Power System is constituted of hard aluminum conductors (HAL), pin insulators, Malaysian-made steel poles and arms, etc.

Regarding the conductor to be used, when the local situation is given consideration, it will be reasonable to adopt hard aluminum conductors (No. 0000 B&S), 0.1662 sq.in. as per British Standards.

The principal specifications of steel poles which are the supports of the 11-kV HV line are shown in **Fig. 10-2**.

Principal Specifications of 11-kV HV Transmission Line

Distance :	1 km from the powerhouse to T branch point where is existing 11 kV line
Conductor:	Hard aluminum conductor (HAL) 0.1662 sq.in.
Insulator:	10-kV pin insulator, 10" anchor insulator
Support :	33-ft steel pole

Note: Guard wires of two steel wires are to be mounted below conductors arranged in horizontal plane.

10.3 Transmission Line Route

The powerhouse of Naradaw Small Hydro Power Station will be located several tens of meters upstream from the confluence of the Liwagu River and Mesilau River. An 11-kV HV line of 1 km is to be constructed from the 11,000/3,000-V step-up transformer located outdoors of the power station for a connection to be made with the existing 11-kV HV line constructed along the road between Kundasang and Ranau.

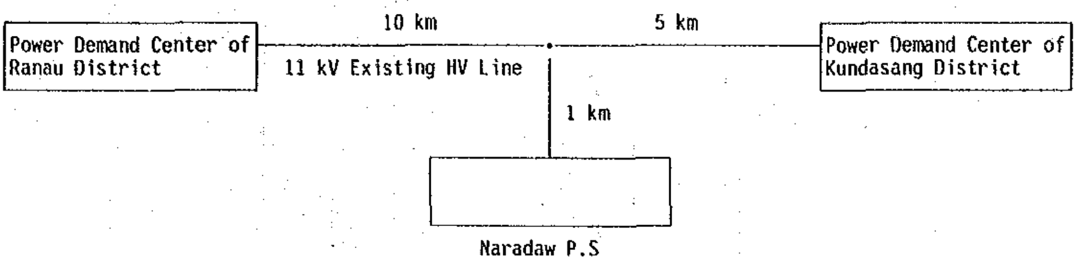
There is an 11-kV HV line along the road to Kauluan village branched from the existing 11-kV HV line along the Ranau-Kundasang road, but considered from the size of the electric power produced at Naradaw Power Station, it is not desirable for a direct connection to be made. There is a 50-kVA transformer for Besel Church installed by the road on the Ranau side of the abovementioned branch.

The 11-kV HV line from Naradaw Small Hydro Power Station is to be connected in the vicinity of the fork of the road on the Kundasang side from the point of installation of the Besel Church 50-kVA transformer.

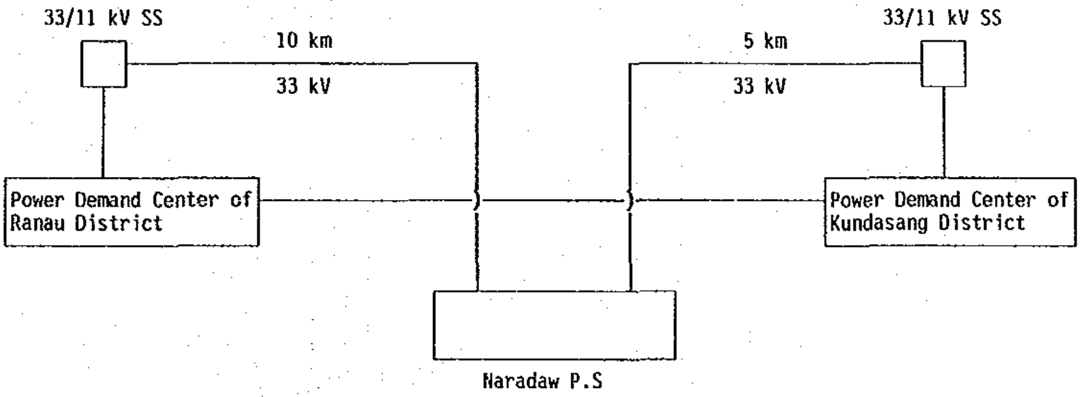
The transmission line route is shown in **Fig. 10-3**.

Figure 10-1 Comparison of Power Transmission Pattern

— Power Transmission Pattern A



— Power Transmission Pattern B



— Power Transmission Pattern C

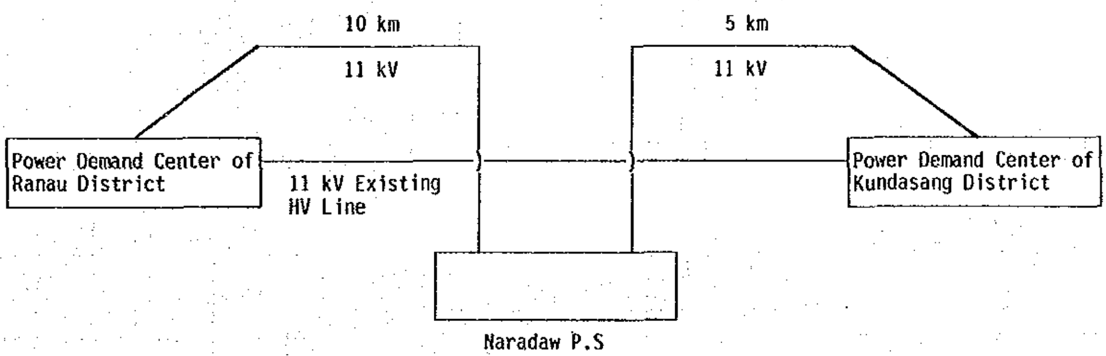
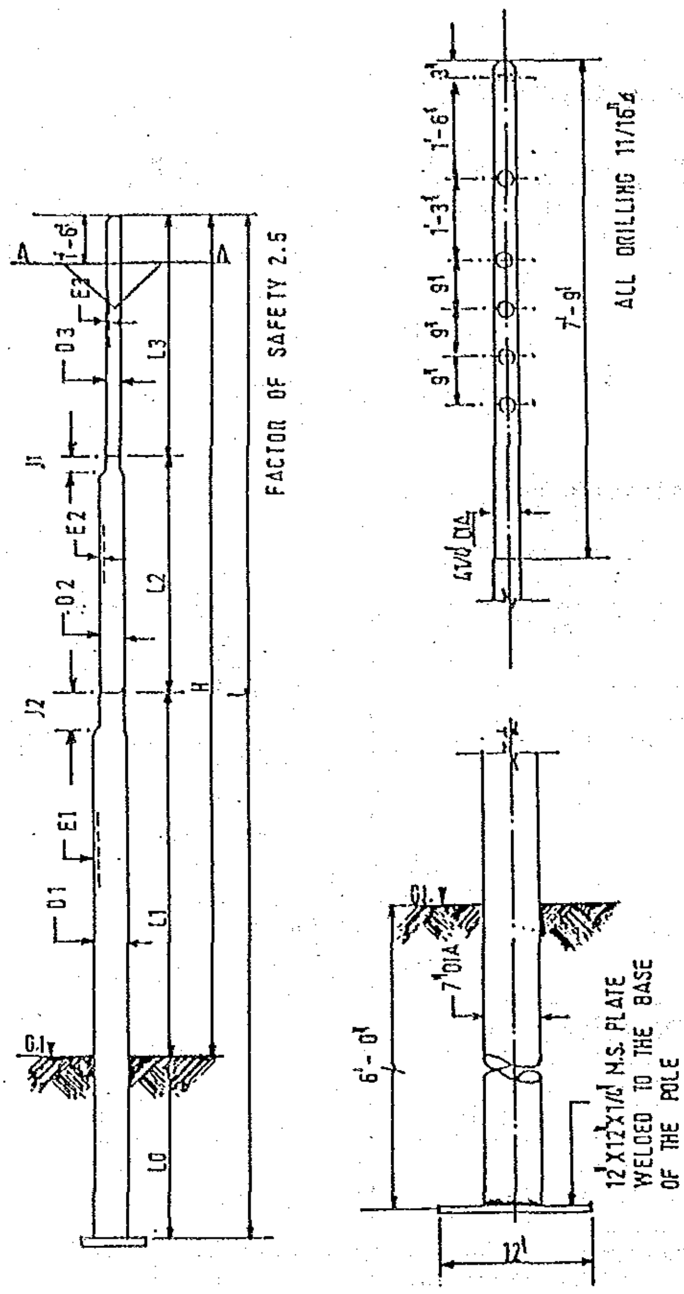


Figure 10-2 Steel Pole of 11 kV HV Line



BASE PLATE DETAIL

DRILLING DETAIL

POLES	L	H	L0	L1	L2=L3	D1	D2	D3	a	b	c	E1	E2	E3
	FT	FT	FT	FT	FT	IN	IN	IN	lb	lb	lb	IN.	IN.	IN.
INTERMEDIATE H.T.	33	27	6	11 1/2	7 3/4	7	6	5	94	704	257	.177	.157	.138
TERMINAL H.T.	33	27	6	11 1/2	7 3/4	7 1/2	6 1/4	4 3/4	103	898	428	.197	.167	.148

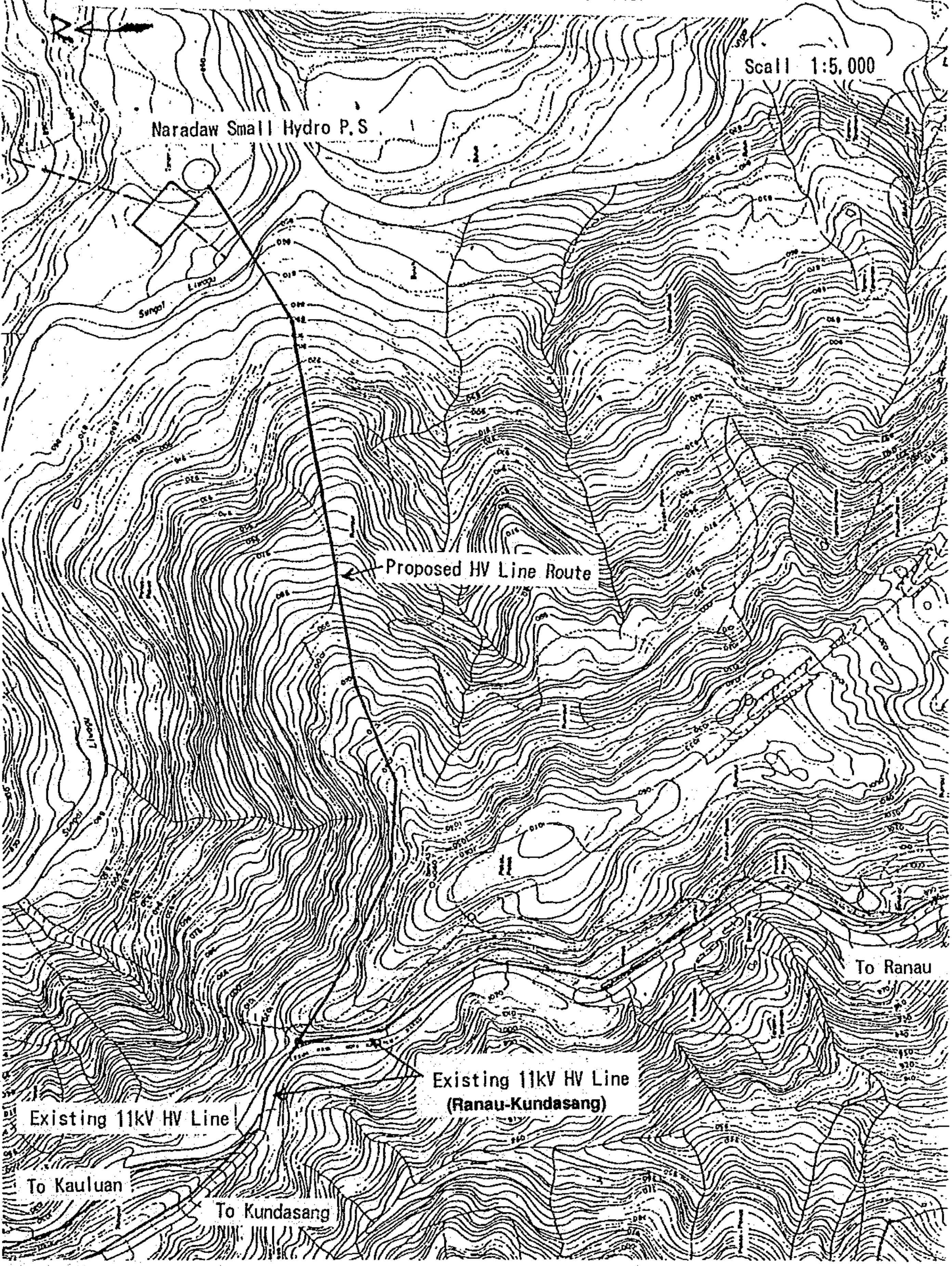
a. = EQUIVALENT WIND LOAD ON THE POLE ACTING AT A-A

b. = TOTAL , WORKING LOAD (FACTOR OF SAFETY 2.5)

c. = MINIMUM LOAD FOR TEMPORARY DEFLECTION OF NOT MORE THAN 6"

FOR INTERMEDIATE. H.T, J1 = 12" J2 = 14"

Figure 10-3 proposed 11 kV HV Line for Naradaw P.S.



Chapter 11 PRELIMINARY DESIGN

Chapter 11

PRELIMINARY DESIGN

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11. Preliminary Designs

11.1 Outline of Preliminary Designs

On the basis of the Optimum Development Plan for Naradaw Project under **Chapter 9**, preliminary designs were executed be review of many important items and selection of the optimum layout, regarding civil structures including the intake facilities, the waterways, the power station and access roads, and electro-mechanical equipment such as turbine and generator.

The Naradaw Project is designed for obtaining an electric power of 1,600 kW by utilizing a maximum discharge of 1.2 m^3/s which is 0.7 m^3/s and 0.5 m^3/s through the two intakes at the Liwagu and Mesilau Rivers. The water will be led to a hill near the confluence of both rivers through pipelines, and the penstock to the power station situated at riverside of the Liwagu River immediately upstream of the confluence with effective head of 170 m. The transmission line will be connected over a short distance to the existing line along the highway on the other side of the river.

For civil structures, reviews were made for selecting the best intake method, capacity of the desilting basin, the function and position for the headpond, and the best material, the optimum diameter and supporting methods of the pipeline, wall thickness and pipe supporting method of the penstock, etc. For electric equipment on the other hand, reviews were made in regard to selection of the types of turbine and of the generator to be used.

The following is the main features of the Naradaw Project which is finally selected, and the layout is shown on **DWG. 11-1 ~ 14**.

(1) **Liwagu Intake**

Intake dam:

Type:	Concrete dam
Dimensions:	Height: 3.50 m, Overflow crest length: 24.00 m
Maximum flood discharge:	200 m^3/s

Intake:

Stream bed type intake (Bar screen type)

Maximum discharge:	0.70 m^3/s

Desilting basin:

Type:	Concrete Culvert
Width:	4.00 m
Length:	14.00 m

Headpond:

Type:	Concrete facing
Capacity:	800 m^3

(2) **Masilau Intake**

Intake dam:

Type:	Concrete dam
Dimensions:	Height: 4.00 m, Overflow crest length: 22.00 m
Maximum flood discharge:	180 m^3/s

Intake:

Stream bed type intake (Bar screen type)

Maximum discharge:	0.50 m^3/s

Desilting basin:

Type:	Concrete Culvert
Width:	2.50 m
Length:	11.00 m

Connecting pipe (steel pipe):

Inner diameter:	0.60 m
Length:	90 m

Headpond:

Type:	Concrete facing
Capacity:	600 m^3

(3) Liwagu Pipeline

Type:	Steel pipe laid on the ground
Inner diameter:	0.70 m
Length:	2,680 m

(4) Mesilau Pipeline

Type:	Steel pipe laid on the ground
Inner diameter:	0.60 m
Length:	990 m

(5) Penstock

Type:	Steel pipe buried in the ground
Inner diameter:	0.80 m
Length:	780 m

(6) Power Station

Type:	Above ground type
Dimensions:	Width: 11.00 m; Length: 19.00 m

(7) Turbine

Type:	Turgo-Impulse Turbine
Number of units:	2
Effective head:	170 m
Maximum discharge:	0.60 m^3/s each

(8) Generator

Type:	3-phase synchronous generator
Number of units:	2
Capacity:	890 kVA

(9) Transformer

Type:	Self-cooled 3-phase transformer
Number of unit:	2
Capacity:	890 kVA

(10) Power Transmission Line

Voltage:	11 kV HV
Length:	1,000 m

(11) Access Roads

New Construction:	Length: 5,460 m
Improvement:	Length: 1,450 m

Details of designs for individual facilities are explained in the following section.

11.2 Civil Structure

11.2.1 Intake Dam

(1) Selection of Dam Site

The location where the intake dam is to be constructed must be selected at a site where the water intake performance of the dam and its structural stability can be assured for the given river conditions. This site has been selected by confirming the following requirements in the field investigation.

- The water level which is sufficient for stable intake of riverflow must be maintained even during the dry season. Particular attention must be paid on this point for Mesilau Site, because the dam site is located on Pinosuk gravel layer.

- The river bed must not be eroded at the dam site and it's condition must not be change easily by flood. Locations where many large boulders carried down must be avoided.

- The dam site must be such that collapse of river bank slopes at upstream or other adverse effect is not caused by the rise of water level after the dam is constructed.

- The dam site must be such that the structural stability can be realized, and the crest length of the dam can be made short.

- The dam site must be such a place where a suitable location for construction of the headpond is available at immediate downstream of the dam.

- The two dams must be located at such elevation that the hydraulic gradient of Liwagu pipeline and Mesilau pipeline become equal at their confluence point.

The specific reasons, based on which the locations of Liwagu Dam and Mesilau Dam have been selected at the proposed sites, are discussed below.

Liwagu

- The selected dam site is the location in this vicinity where the dam having the shortest crest length can be constructed.

- To the downstream of the selected dam site, the river gradient is so steep (1/7) that the river bed is eroded, and there are large boulders.

- To the upstream of the selected dam site, terraces spreads so widely on the right bank of the river that dam crest length becomes considerably increased.

- There is a flat terrace on the left bank to the downstream of the selected dam site, which is suitable for construction of a headpond.

Mesilau

- The selected dam site is the position in this vicinity where the crest length of the dam can be made shortest.

- Although it would be desirable to move the dam site further upstream in order to make the intake water level of Mesilau suitable for that of Liwagu, it is not possible to find a suitable dam site further upstream to the proposed position because the river route curves violently and river bed is badly eroded.

- It is not suitable either to move the dam site further downstream, because this reduces the intake water level, and there are small valley joining the main stream.

- An open river terrace exists at a location about 100 m to the downstream of the selected dam site, which is suitable for construction of headpond.

(2) Comparison Study of Intake Facilities

To select the most suitable power intake facilities for the Naradaw project, comparative studies of intake facilities are performed for Mesilau site.

The three typical types of intake facilities are examined.

- Bar Screen Type Intake (which is called stream bed type intake)
- Side Intake with Flush Gate
- Inflatable Rubber Dam

(a) Bar Screen Type Intake (Stream Bed Type Intake)

DWG. 11-7 shows a stream bed type intake which is proposed for Naradaw Project.

Bar screen type intake is constructed at the top of overflow section of the intake dam. Steel pipes of 10 cm in diameter are used for the screen.

Sand and gravel of particle size smaller than the opening in the screen flow into the intake channel with water. Gravel and boulders larger than the opening of pipe screen flows over the screen to the downstream of the dam.

Frequent flushing operation of intake channel are required especially in the flood season. This scheme has two desilting areas. They are intake channel and desilting basin.

Sediments smaller than the opening of the screen can be flushed by scouring gate installed at the downstream end of intake channel.

The intake channel and desilting basin are connected to an orifice at the upper side of intake channel, so that large size sediment can not enter into the desilting basin.

(b) Side Intake with Flush Gate

The second best alternative is side intake with flush gate as shown in **DWG. 11-15.** Intake is constructed just upstream of sand flush gate so that scouring of sediment can be performed only around the intake area.

This is the most popular type of intake for small-scale hydro-power development. During flood flows, no water is drawn for generation through the intake. Only sand flushing operation is performed through a flush gate.

Because the operation of small run-of-river type scheme during flood flows is risky, generally small scale run-of-river type power plants is not operated as other hydro-power stations such as those of reservoir controlled type have adequate inflow for generation of electricity.

If it is operated during flood time, a large quantity of sediment easily enter into the waterway and maintenance cost which is for removal of sediment in the waterway will increase.

As a results, side intake with flush gate system is not applicable to this project because the Naradaw power station must be operated during flood flows as it is connected to an isolated power supply system.

(c) Inflatable Rubber Dam

The third alternative is an inflatable rubber dam. Inflatable rubber dam is an advanced type of intake dam. full width of river bed scouring of sediment can be performed automatically during flood flows by a sensor which detects the water level at the damsite. (see **DWG. 11-16**)

In the comparative studies, this scheme is the most expensive one among the three alternatives.

(d) Costruction Cost

Comparative studies of construction costs show that the bar screen type intake is the most economical scheme, and the rubber dam cost more than 2 times the bar screen type intake.

Type of Intake	Construction Cost
Bar screen type intake (Stream bed type intake)	M$322,000
Side Intake with flush gate	M$388,000
Rubber dam scheme	M$764,000

(3) Composition of Intake Dam

In Naradaw Project the intake dam consist of the weir including the intake section, the overflow section and the non-overflow section, and the downstream apron and the gabion.

The weir is for raising water level and its sectional shape should be stable against external force. The non-over section is for preventing river side from scouring by design flood discharge.

The downstream apron is for preventing river bed from piping due to seepage and from scouring by overflow water. The gabion is also for preventing river bed from scouring by overflowed water.

(4) Design Flood Discharge

(a) Estimation of Design Flood Discharge

The design flood discharge must be estimated for the purpose of stability calculation of dams and determination of their spillway capacity, by taking account of probable flood discharge from past flood records or past rainfall records. It is popular that probable maximum flood or probable flood of 100 year return period is adopted for the design flood of large dam, considered with the damage due to failure of the dam structure.

The design flood discharge should be determined by probable discharge with the scale corresponding to the dam and the river. That of Naradaw Project is determined from 50 year return period, with due regard to largest recorded flood, though the damage of small dam failure is deference from that of large dam.

The design flood discharge of Naradaw Project is shown as below.

Liwagu Dam site	200 m^3/s
Mesilau Dam site	180 m^3/s
Tailrace site	220 m^3/s

(b) Design Flood Water Level

Design flood water level is calculated by following formula.

$$h_c = \sqrt[3]{\frac{aQ^2}{gb^2}}$$