8.5. Earthquake Records

8.5.1. General

The Ambuklao Dam is located in an area with high earthquake sensitivity and frequency. Despite the above, it appears that, until recently, not much concern was paid to this matter. Until the earthquake of April 1985, no observation device had been installed at the damsite. After the earthquake, a seismograph with one vertical component was installed at the site office on the abutment of the dam. It has been observed, however, that, since its installation, the maintenance of this instrument was inadequately done.

Several years ago, a seismic observation station was built in the San Manuel Substation of NAPOCOR, located in the downstream section of the Agno River, about 50 km south of the Ambuklao Dam. This station is a control center for recording the seismic waves transmitted by feeder sensors distributed over various locations. This seismic observation network was completed in 1982. Its main purpose was collection of data. There is, however, lack of observation records due to malfunction of the instruments and, apparently, inadequate information.

The results of the analysis of seismic waves, recorded at the Kanoong and Bongel Stations, located near the Ambuklao Dam, are presented below. These records, as compared to the others, were rather well aranged and preserved.

Kanoong Station is located about 10 km north of the San Manuel Substation, the Bongel station is 8 km northeast of the same (see Fig.-8.20). The recording system in the Control Center consists of an oscillograph with one vertical component, a portable seismic system MEQ-800 manufactured by SPRENGENETHER INSTRUMENT CO., INC.

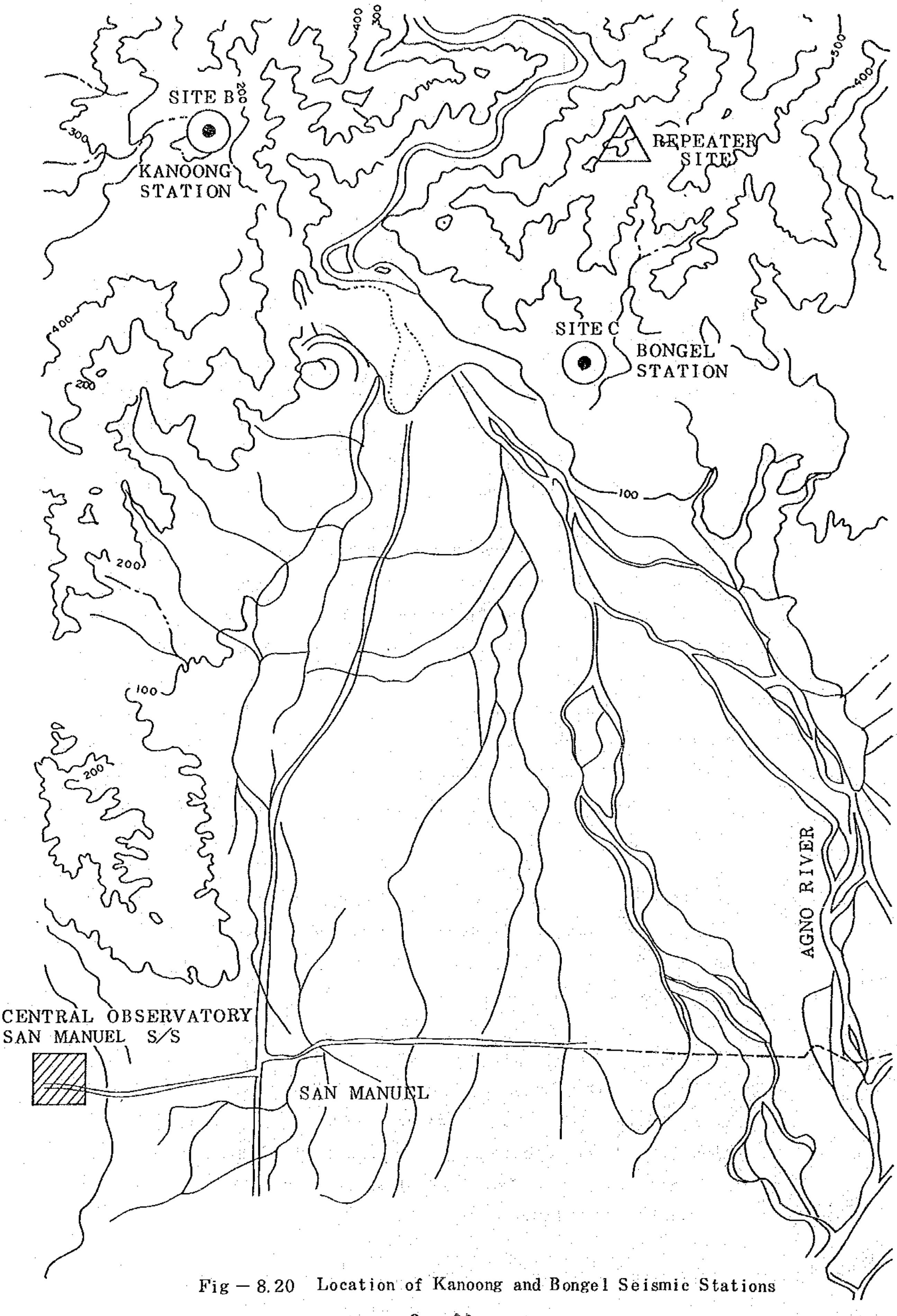


Fig — 8.20 Location of Kanoong and Bongel Seismic Stations

8.5.2. Seismic Waves

As mentioned above the earthquake records are of one vertical component. They include some improper functioning of the multiplier of the oscillograph, as compared to the magnitude of earthquakes, and overruns of the recording pen for most data. It is, therefore, not possible, from these records, to read the maximum acceleration of earthquakes.

The most important factors of an earthquake which affect safety of structures are three elements, maximum acceleration, period and duration, in addition to the predominant period characteristics of the structure itself. It is possible to read at least two earthquake factors from the above cited earthquake records, those of frequency and duration. Because of this, the analysis of these data is considered to be significant as will give reference data for planning the specifications for the new seismographs to be installed in the future. As the seismic waves have been recorded at the speed of a 6 cm per minute, they were enlarged four times and analysed with eye sights, examples of which, for reference, are presented in Fig.-8.21.

The total number of seismic waves used for analysis were 62, i.e., 16 from the records from the Kanoong Station, and 46 from those from the Bongel Station.

8.5.3. Analysis of Recorded Data

The results of the analyses of the seismic records obtained from the Kanoong and Bongel Stations are summarized in Tables-8.12 and 8.13, respectively.

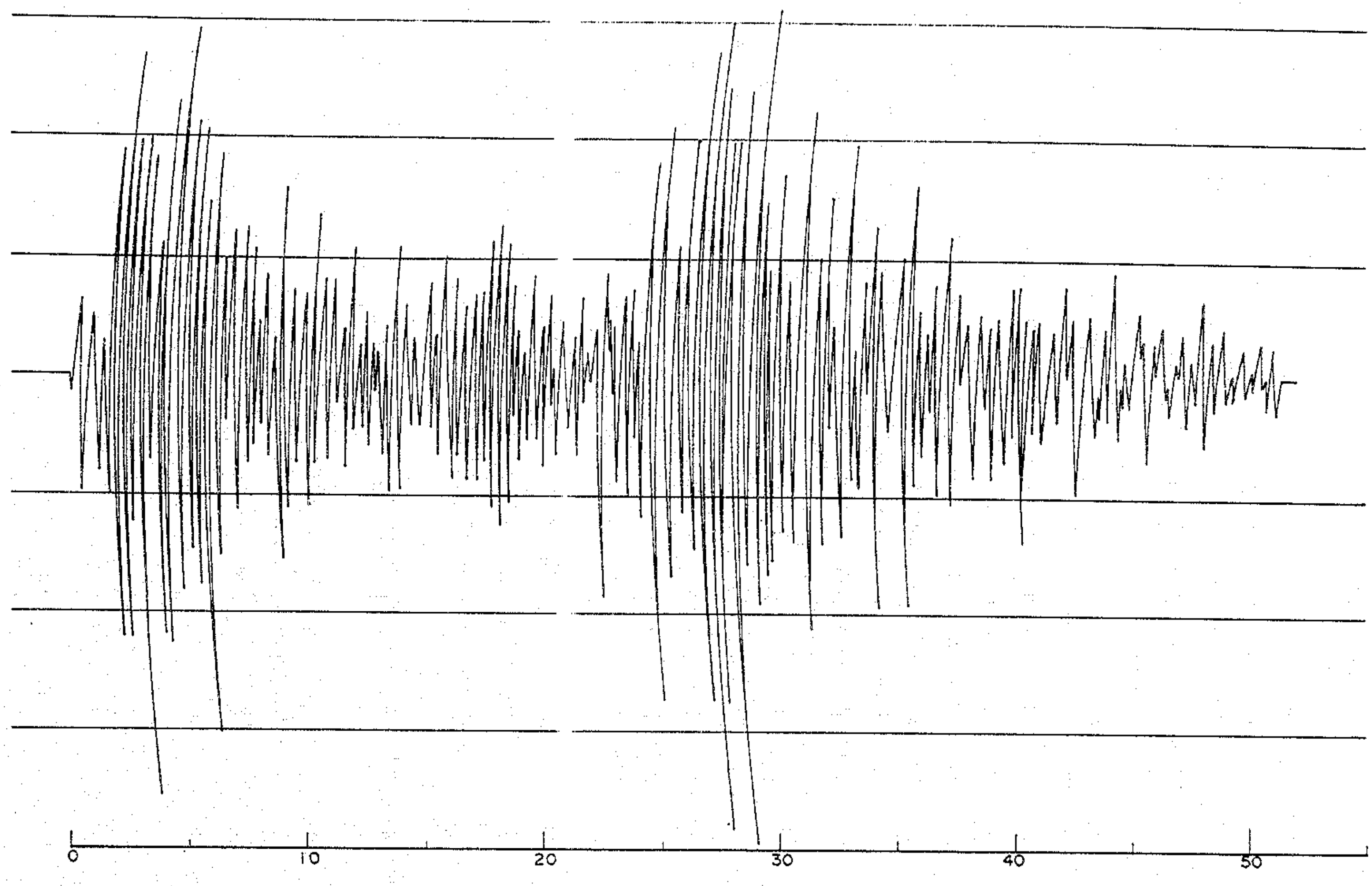


Fig — 8.21 Example of Seismic Waves Recorded at Kanoong St.
(K—1 Wave, Enlarged by 4 Times)

Table − 8.12 Characteristic Data of Seismic Records at Kanoong Station

Record of Seismograph	Data Recorded Year Month Day	Time Taken for Analysis (sec)	Number of Waves	Predominant Period Pd_1 (sec)	Remark
K − 1	'85. 1. 15	48.3	105	0.25	
K − 2	'85. 1. 15	40.5	98	0.375	
K − 3	'85. 2. 12	53.0	140	0.25	
K − 4	'85. 11. 21	43.5	99	0.375	
K − 5	'85. 11. 27	58.5	150	0.375	
K − 6	'86. 8. 9	124.3	211	0.375	
K − 7	'86. 8. 11	100.0	206	0.375	
K − 8	'86. 8. 12	27.0	78	0.375	
K − 9	'86. 8. 31	84.8	197	0.375	
K − 10	'85. 1. 21	102.3	205	0.375	
K − 11	'85. 1. 21	148.8	254	0.375	
K − 12	'84. 1. 3	111.5	197	0.375	
K − 13	'85. 1. 12	97.0	182	0.25	
K − 14	'85. 1. 12	144.3	254	0.375	
K − 15	'85. 1. 31	97.3	184	0.375	
K − 16	'85. 2. 8	38.5	117	0.25	

Table − 8.13 Characteristic Date of Seismic Records Bongel Station

Record of Seismograph	Data Recorded Year Month Day	Time Taken for Analysis (sec)	Number of Waves	Predominant Period Pd_1 (sec)	Remark
B − 1	'85. 5. 17	154.3	279	0.5	
B − 2	'85. 5. 25	30.8	64	0.375	
B − 2'	'85. 5. 25	55.3	97	0.5	
B − 3	'85. 5. 20	46.3	107	0.375	
B − 4	'85. 5. 16	52.0	104	0.375	
B − 5	'85. 5. 15	44.0	97	0.375	
B − 6	'85. 5. 15	54.3	124	0.375	
B − 7 − 1	'85. 5. 15	69.8	142	0.5	
B − 7 − 2	'85. 5. 15	40.5	89	0.375, 0.5	
B − 8	'85. 5. 15	72.3	141	0.5	
B − 9	'85. 5. 14	70.8	144	0.375	
B − 9'	'85. 5. 14	43.3	89	0.5	
B −10	'85. 4. 25	39.5	83	0.375	
B −11	'85. 4. 17	70.8	138	0.5	
B −12	'85. 3. 5	100.0	182	0.5	
B −13	'85. 2. 16	113.3	210	0.5	
B −14	'85. 2. 5	46.5	98	0.375	
B −15	'85. 1. 31	99.0	190	0.375	
B −16	'85. 1. 12	95.8	189	0.375	
B −17	'85. 1. 12	170.8	276	0.5	
B −18	'85. 5. 13	40.4	88	0.375	
B −20	'85. 5. 13	114.0	182	0.375	
B −22	'85. 5. 13	53.3	113	0.375, 0.5	
B −23	'85. 5. 13	28.1	50	0.5	
B −24	'85. 4. 23	234.5	327	0.375	
B −25	'85. 4. 24	14.0	33	0.375	
B −26− 1	'85. 4. 24	42.8	93	0.375	
B −26− 2	'85. 4. 24	16.0	33	0.375	
B −26− 3	'85. 4. 24	28.0	58	0.375	
B −26− 4	'85. 4. 24	76.5	158	0.375	
B −26− 5	'85. 4. 24	43.4	82	0.5	
B −27	'85. 4. 24	38.3	74	0.375	
B −28	'85. 4. 24	36.0	77	0.375	
B −29	'85. 4. 24	45.5	98	0.375, 0.5	
B −30	'85. 4. 24	30.5	70	0.375	
B −31	'85. 4. 24	46.8	96	0.5	
B −32	'85. 4. 24	47.0	99	0.375	
B −33	'85. 4. 24	37.8	76	0.375	
B −34	'85. 4. 24	51.3	102	0.375	
B −35'	'85. 4. 24	77.5	160	0.375	
B −36	'85. 4. 24	31.8	77	0.25	
B −37	'85. 4. 24	61.5	125	0.375	
B −38	'85. 4. 24	27.0	61	0.375	
B −39	'85. 4. 24	37.3	86	0.375	
B −40	'85. 4. 24	40.5	81	0.375, 0.5	
B −41	'85. 4. 24	14.5	32	0.375, 0.5	

(1) Earthquakes recorded at Kanoong Station

Sixteen waves of relatively large earthquakes from the records for the period of 1984 to 1986 were selected from this station.

As shown in Table 8-12, the time span taken for analysis, namely, the duration of major seismic vibrations, have the characteristics of a rather long period, as recorded for K-11 and K-14 (140 - 150 sec.) and K-6 (120 sec.).

Predominant periods of seismic vibrations are mostly 0.375 sec., while the shortest period is 0.25 sec. The above means that the hypocenter of the earthquakes were located comparatively near to the observation point.

(2) Earthquakes recorded at Bongel Station

The seismic data used from the Bongel Station were for the earthquakes which occurred during the period of January to May 1985. Most of these occurred during the period of April and May 1985. The frequency of occurrences of the earthquake, particularly during this period, could not be defined easily due to the short period of observation at the Bongel Station which commenced 1982. Looking at the durations of the earthquakes shown in Table-8.13, they can be characterized to be of a long period, similar to those observed and recorded at the Kanoong Station.

The predominant periods range from 0.375 sec., to 0.5 sec., as indicated in Table-8.13, which include periods which are rather longer than those recorded at the Kanoong Station.

The Kanoong and Bongel Stations are situated about 4.5 km from each other putting the Agno River in the middle. Records on the same earthquakes were collected from both stations. They were the earthquakes of January 12 and 31, 1985. For the former earthquake, K-13 and K-14 at Kanoong corresponds to B-16 and B-17 at Bongel, respectively. And for the latter, K-15 at Kanoong corresponds to B-15 at Bongel. Since the information is for the same earthquakes, the duration recorded by both stations is almost same. There are, however, some discrepancies in the characteristics of frequency. The predominant periods recorded at Bongel are larger.

Generally, rivers like Agno often form geologic tectonic lines with faults. Then, owing to such geologic structure, propagation of seismic acceleration takes different character at each bank when crossing the river.

It is well known that seismic acceleration characteristics do not change much for a distance of 40 to 50 km from the epicenter. The larger the magnitude of the earthquake is, the larger the predominant period is. When the magnitude of an earthquake is 5.0 to 8.0 on the Magnitude, the predominant period within 40 to 50 km from the hypocenter, is in the range of 0.2 - 0.4 sec.

The selected earthquakes from the records from Kanoong and Bongel are of the same magnitude. The small differences in the predominant periods between the two stations are attributed to the differences in the geologic structure.

The earthquake of April 24, 1985 recorded at Bongel Station, caused serious damages to the Ambuklao Dam and its appurtenant structures.

The records indicated that the hypocenter of this earthquake was 26 km east of the Ambuklao Dam and 33 km depth. The epicenter had a latitude of 16.48N and longitude, 120.99E. The magnitude was 5.9. Its predominant period at Bongel station was 0.375, but was estimated to be less, about 0.2 - 0.25, at the Ambuklao Dam. Fig.-8.22 indicates the relation between distance from hypocenter and predominant period. As the earthquake occurred in April 1985 is at the magnitude of 5.9, the figure shows that the earthquake was of shorter period at the dam site. This fact indicates a good coincidence with the result of analyses for measurement records at Bongel Station. Kanoong Station could not record this earthquake due to the malfunctioning of the instruments.

8.5.4. Conclusions

Both, Kanoong and Bongel Stations recorded a number of earthquakes. Although detailed data, such as magnitudes and location of epicenters were not defined properly, the epicenters were recorded to have been near to these stations, namely, within 40 to 50 km from their locations. The above means that the hypocenters were not far from the Ambuklao Dam.

The natural free period (predominent period) of the Ambuklao Dam was estimated to be about 0.65 sec., while, the predominant periods of all the earthquakes which have occurred in the area and were analysed, were less.

Based on the above, the following is concluded.

The Ambuklao Dam was constructed in an earthquake area with high sensitivity and frequency. Earthquakes which may have any serious effect on the stability of the dam are expected to occur within 40 to 50 km from the dam site. The Predominant periods of such earthquake vibrations will be small, in the range of 0.2 sec. to 0.4 sec.

For examination of stability against earthquake, seismic forces corresponding to small periods should be considered.

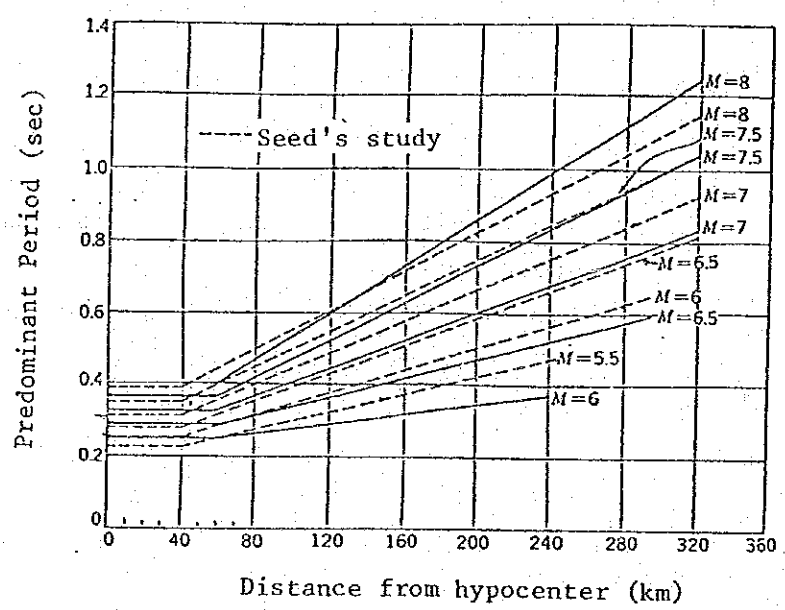


Fig - 8.22 Relation between Predominant Period and Hypocenter of Earthquake

9. Safety of Structures

9. Safety of Structures

9.1. Spillway Capacity

9.1.1. Sequence of Analysis

The required spillway capacity was calculated using the following sequence:

(1) Evaluation of the probable daily rainfall

(2) Conversion of the daily rainfall to the hourly rainfall

(3) Calculation of time for flood concentration

(4) Confirmation of the appropriateness of the runoff model by comparison of the calculated runoff with the actual runoff when the actual rainfall is applied to the runoff model

(5) Conversion of the probable daily rainfall to the hourly rainfall

(6) Application of the probable rainfall to the runoff model and computation of the probable flood inflow.

(7) Selection of the design flood inflow

(8) Calculation of reservoir water level rises for various conditions of gate operation for the selected maximum design flood inflow

(9) Safety analysis of spillway capacity

9.1.2. Probable Daily Rainfall

The rainfall record period at the Ambuklao Dam site covered a very short period. Therefore, it was decided to develop the required data for the flood studies by the use of the following four methods (A, B1, B2, and B3):

Method A: It comprises the data of daily rainfall based on the actual record from 1964 to 1986.

Method B: It is based on the estimated daily rainfall data (1902 - 1963) in addition to the above actually recorded data. For making the estimates, the relation between the monthly rainfall records for the Baguio basin for the period 1902 to 1984 and those of the Ambuklao Dam for the period 1957 to 1986 were obtained. In addition, a relation was also obtained between the peak daily rainfall and the monthly rainfall for the Ambuklao for the period 1964 to 1987. These data are presented in Figs.-9.1 and 9.2, respectively.

Using the above, the rainfall data for the period 1902 to 1963 were estimated as follows:

The monthly rainfall for the Ambuklao was obtained from monthly rainfall data for Baguio applying Fig.-9.1. The Ambuklao daily rainfall data were obtained from the monthly rainfall for the Ambuklao applying Fig.-9.2. In applying Fig.-9.2, the following three methods, (B1, B2, B3), were adopted to estimate the daily rainfall for the Ambuklao (Period 1902 to 1963):

Method B1: It uses the envelope curve of the confidence zone (D-curve in Fig.-9.2), assuming the rainfall is maximum at the Ambuklao.

Method B2: The average line (B-line in Fig.-9.2) is used. The B-line is obtained by the least square method, excluding data of extreme values.

Method B3: The F-line is used. The F-line has a 95% confidence zone of the average line.

Using the above four methods, the probable daily rainfall was obtained by the use of the Gumbel-Chow method and is summarized in Table-9.1. The Hazen-plot of the calculated data is shown in Figs.-9.3, 9.4, 9.5, 9.6.

The computated values for the probable daily rainfall vary in accordance with the approach used. The probable daily rainfalls for a 200-year frequency, for each method used above were:

Method		Rainfall
A :	(Actual records only)	560 mm/day
B1:	(Envelope curve of confidence zone)	903 mm/day
B2:	(Average line)	676 mm/day
B3:	(Line having 95% confidence zone of the average line)	759 mm/day

If the 903 mm/day rainfall, which by using method B1 produces a 200-year return period, is adopted for computation of corresponding frequences by the other three methods, the following return periods would be obtained:

Method	Years
A :	19,680
B1 :	200
B2 :	2,360
B3 :	800

If a rainfall of 903 mm/day is adopted to study the spillway capacity, depending on the method used, it could have a range of 200 to 19680-year return period. In other words, the spillway capacity could be good for 200 to 19,680-year flood.

The above tremendous difference between the return periods results from a very short period of rainfall records for the Agno River basin. Therefore, it is very important that continuous observation and hydrological data collection for the basin be established as soon as possible.

The daily rainfall was converted to hourly rainfall using the wellknown formula of Dr. Mononobe:

$$\gamma T = R_{24} (T/24)^{1/3}$$

where, γT : rainfall for T hours

R_{24} : daily rainfall (unit: mm)

To compute the runoff, this hourly rainfall was arranged and distributed as center-concentration in a day.

9.1.3. Probable Flood Discharge

To compute the inflow to the reservoir, the time of flood concentration must be determined first. The formula of Rziha, which is being used in Bayern, Western Germany, was also used here:

$$W = 20(h/L)^{0.6}$$

where,

W : flood velocity to the reservoir (m/s) h : difference in elevation between the upstream end of the basin and the reservoir (m)

L : distance from the upstream end of the basin to the reservoir (m)

Then, the time of flood concentration was calculated using the following formula:

$$Ta = L/W$$

where,

L : 45 km
h : 1048 m
W : 2.1 m/sec
Ta : 6.0 hr.

For computation of runoff, the runoff function method was used. The corresponding formula was:

$$Q = (r/3.6)\ Af\alpha^2 t\ \ Exp\ (-\alpha t)$$

where,

A : catchment area (km^2)
f : runoff coefficient (=0.8)
α : 1/ta, r: hourly rainfall
ta : time lag until the peak runoff appears from the start of rainfall
t : time (hr)

Using this formula, reproduction by model is confirmed with the heaviest flood record after completion of the dam and in comparison with the values of runoff computation.

The flood used was the one which was recorded between May 23 and 26, 1976. The daily rainfall for these four days was 123.2 mm on May 23, 218.4 mm on May 24, 406.1 mm on May 25 and 91.7 mm on May 26.

The peak inflow into the reservoir was estimated at 3,000 m^3/s from the records of hourly water level rise in the reservoir. Fig.-9.7 shows the hourly curves of the recorded water levels and estimated inflows.

Fig.-9.8 shows the inflow computed from the simulation made by using the above rainfall records.

The peak inflow shown in Fig.-9.8 is 3,687 m^3/s which is in fairly good agreement with the record of 3,000 m^3/s.

The runoff corresponding to the probable daily rainfall was computed next. The distribution of rainfall was determined based on the records of May 23 to 26, 1976, which was the heaviest continued rainfall and flood record since the beginning of observations. Firstly, the actual peak daily rainfall was converted into a probable daily rainfall. Multiplying another day's daily rainfall by (probable daily rainfall)/(actual peak daily rainfall), a distribution similar to that of actual daily rainfall was prepared. This was used as an input value for the simulation system. This method is normally used in Japan for river and dam projects.

By the above method, the peak inflow for each probable daily rainfall was obtained and summarized in Table-9.2. The peak inflow thus obtained from each probable daily rainfall is defined as probable flood inflow.

The relation between time and inflow to the reservoir for the 200-year return period rainfall computed by Method B1 is shown in Fig.-9.9.

9.1.4. Spillway Capacity

The required capacity for the spillway was studied for an inflow of 9,840 m^3/s. This was a 200-year return period flood computed on the basis of Method B1 and multiplied by a coefficient of 1.2, i.e., 8,200 x 1.2 = 9,840 m^3/sec.

The 200-year return period flood (inflow design flood) was obtained using the envelope curve which gives the maximum rainfall in estimating daily rainfall data. However, it is pointed out that the flood inflow of 8,200 m^3/s computed on the basis of Method B1 has a return period of 200 years. If, this flood is computed using actually recorded data for the Ambuklao basin, its return period will be 19,680 years (Method A).

The main characteristics of the spillway gates are as follows:

Spillway Type	:	Open chute gate
Spillway Length	:	127 m
No. of Gates	:	8
Type of Gate	:	Tainter
Size of Gate	:	12.5 m (W) x 12.5 m (H)
Design Flood	:	7,300 m^3/s
Elevation of Crest Overflow	:	EL 740 m
Elevation of Top of Parapet	:	EL 758 m
Maximum Storage Level	:	EL 752 m

The spillway discharge through one gate opening when the gate is fully open can be roughly calculated by formula $Q = CBh^{3/2}$. The coefficient of discharge (C) was calculated back from the design discharge of 7,300 m^3/s (with 8 gates fully open) as follows:

$$C = Q/Bh^{3/2}N$$
$$= 7,300\ (12.5 \times 12^{1.5} \times 8)$$
$$= 1,756$$

The following gate operation was assumed:

- Three gates are opened when the reservoir water level reaches EL 752.3 m.
- Two gates are additionally opened at EL 752.5 m, and another three gates are fully opened at EL 752.7 m.

  Thus, total of 8 gates are fully open when the reservoir water level is at EL 752.7 m.

  Once the gates are opened, they are not to be closed until the reservoir level starts receding and the normal maximum operating level is reached.

The water level rise versus time, inflow (to reservoir) versus time, and discharge (through the gate opening) versus time curves, are all shown in Fig.-9.10.

The flood routing results for the inflow flood hydrograph with an inflow of 9,840 m^3/s are as follows:

HWL*	EL 754.290 m
Allowance	3.710 m
Total	758.000 m

* HWL = High water level.

The top of the parapet wall is at EL 758.0 m, resulting in a freeboard of 3.710 m. The spillway discharge, corresponding to the above, is 9,486 m^3/s.

In order to avoid a sudden increase in the discharge downstream, each gate should be opened individually and in a sequence as proposed below:

Elevation	Number of gates fully open
752.30	one
752.32	two
752.34	three
752.45	four
752.50	five
752.55	six
752.60	seven
752.65	eight

The above sequence will produce:

HWL	EL 754.372 m
Allowance	3.628 m
Total	EL 758.000 m

The maximum spillway discharge for the above is 9,568 m^3/s. Fig.-9.11 shows the water level rise in the reservoir and the corresponding spillway discharges.

The height of wave due to earthquake was calculated by Dr. Satou's formula which gives 0.63 m in case of a design seismic coefficient K = 0.15 for a seismic wave period of 1.0 sec. The height of wave due to wind was found to be 3.0 m according to Bretsehneider - Sarage's figure in case of wind velocity of 50 m/s, fetch of 2,831 m, and a dam slope gradient of 1 to 1.75.

Based on the results of the above study and consideration of various relevant matters existing at the Ambuklao Project, the spillway capacity, at present, is considered to be sufficient.

In the light of the inadequate rainfall and streamflow data, it is imperative that, as soon as possible, measures should be taken to record and collect the necessary hydrometeorological data so that more accurate methods could be used in calculating and estimating the floods. By continuing the rainfall observations and conducting additional studies for the maximum design flood, the results of the studies, presented in this Report, will be reevaluated. Finer adjustment to the spillway gate operation should be studied in the future as affected by the water level rise in the reservoir and downstream of the Project, and installation of the flood warning system.

9.2. Seepage at the Powerplant Facilities

9.2.1. Seepage at and around the Powerhouse

The Ambuklao Powerstation is of an underground type and is located immediately underneath the dam. Therefore it would appear that a considerable amount of water would seep from the reservoir through the bedrock to the powerhouse, access tunnel, bus duct tunnel, etc.

To investigate the causes of seepage, gauging of water seeping through the bedrock has been carried out since March 1, 1987, at 13 gauging stations situated in various locations along the tunnels around the powerhouse.

The gauging stations near the ground surface (SW-1, SW-2A, SW-8, SW-9) were very much affected by the rainfall. Thus, the readings of these stations included the reservoir seepage plus rainfall. However, the readings of the other gauging stations reflected the reservoir water level fluctuations, in general, and they indicated only the amount of water losses from the reservoir.

Except for the gauging station SW-7 at the access to the surge tank and SW-12 at the powerhouse, all other stations have indicated an increase of 20 to 550 ltr./min. of flow when the reservoir reaches the high water level, as compared to only 100 ltr./min. when the reservoir is at the minimum water level, EL.700 meters.

As the above seepage and changes to it were observed and recorded only from March 1, 1987, to the middle of November 1987, it is very important that continuous monitoring of the seepage be carried out in the future.

The observed data to date do not indicate any serious problems to develop with regard to seepage.

As SW-7 and SW-12 stations show changes which may be caused by pumping, the cause of the above must be confirmed by correlation with the plant operation.

9.2.2. Turbine Leakage

During the site inspection conducted in November 1986, considerable amount of water leakage was noticed to come out from the water sealing equipment of Nos. 1 and 2 turbines.

Cooling water for the Ambuklao power units comes from the water used for generation which contains silts and sands from the reservoir sediments. The sands wear out the carbon packing of the water sealing equipment and causes the leakage.

If the cooling water is not purified and the sand prevented from mixing with the water at the intake, the amount of the required repairs for the turbines would further increase in the future.

This problem will be greatly reduced or eliminated by modifying or replacing the intake structure. In addition, however, the cooling water supply system should be improved as required to purify the water and prevent further damaging of the water sealing system.

9.2.3. Turbine Inlet Valve Leakage

For protection of each turbine, two valves are provided. One is a butterfly valve, 2,600 mm diameter, for use in emergency, and the other is a spherical valve, 2,100 mm diameter, to be used for normal operation. The valves are installed in a special valve chamber located upstream of the powerhouse.

At the time of the site visit, practically, no water leakage was observed. This was because the turbines were in operation. However, as reported by NAPOCOR, there is a leakage of about 3,000 ltr./min., when the valves are closed.

Although an inspection of the valves at the time of the above visit was not possible, it is assumed that the valves could not be closed anyway due to the heavy damage to the valve sheet by scouring.

The valves should be inspected when inoperative in order to determine whether they should be repaired or replaced.

The turbine valves play an important role in maintaining the turbines in good operating condition.

In the rehabilitation plan, considering the worst case, replacement of each valve (spherical) by a biplane valve is proposed. Assuming that the valves are seriously damaged, it is proposed, as part of the rehabilitation plan, that the spherical valves should be replaced by biplane valves.

Table − 9.1 Probable Daily Rainfall

Return Period	Group A	Group B_1	Group B_2	Group B_3
	Peak rainfall estimation from actual data only *(observed from 1964 through 1986)	Peak rainfall estimation by envelope curve *(observed from 1902 through 1986)	Peak rainfall estimation by average line	Peak rainfall estimation by 95% confidence zone curve of average line
year	mm/day	mm/day	mm/day	mm/day
10,000	852	1,368	1,036	1,168
1,000	680	1,093	824	927
200	560	903	676	759
100	508	822	612	686
50	456	740	548	613
25	403	657	484	540
20	386	630	463	516
10	332	546	397	441
5	276	458	328	363
2	191	325	224	244
1.4	147	255	170	182

Note : Data from 1902 to 1986

1902 ～ 1963 : Fitted value

1964 ～ 1986 : Observed value

Data from 1964 to 1986

All data were observed.

Table — 9.2 Probable Peak Discharge

Return Period \ Data on Daily Rainfall	Group A (actual record)	Group B_1 (envelope curve of confidence zone)	Group B_2 (average line)	Group B_3 (95% confidence zone curve of average line)
10,000	m^3/s 7,736	m^3/s 12,419	m^3/s 9,407	m^3/s 10,605
1,000	6,173	9,919	7,482	8,415
200	5,085	8,201	6,136	6,892
100	—	7,460	—	—
50	—	6,715	—	—
20 (observed June only)	1,720	—	1,654	—
20 (observed July only)	2,001	—	2,201	—
20 (observed August only)	1,873	—	2,221	—

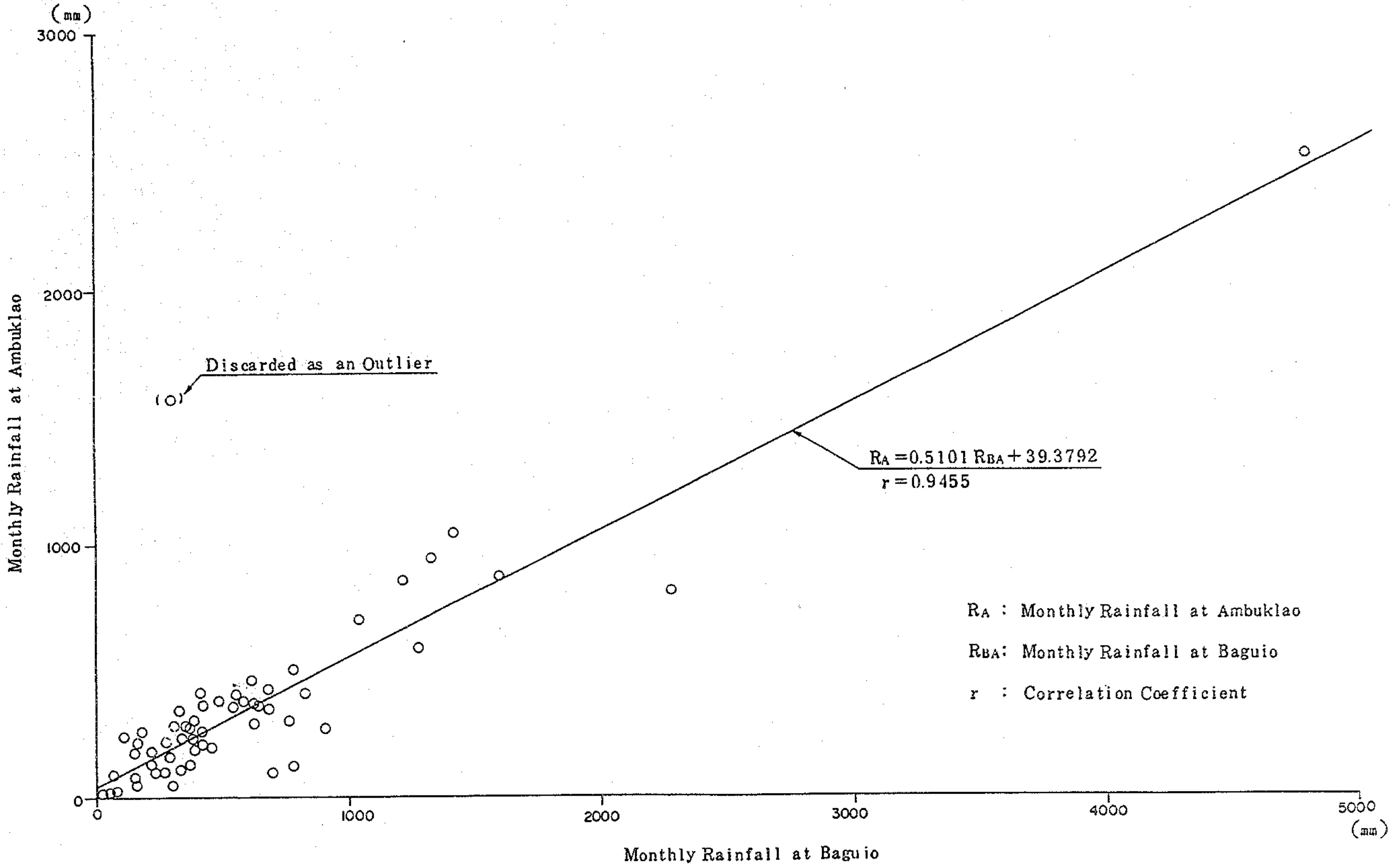


Fig – 9.1 Relation of Monthly Rainfall Between Ambuklao and Baguio (Data period : May~Nov.)

Relation of monthly rainfall and daily rainfall		
	Average line of all data	: (A) line
	Average line excluding an outlier $y = 44.3 + 2.843x$	: (B) line
	Envelope curve of confidence zone of (B) line	: (C) line(upper)
	ditto	: (D) line(lower)
	$y = 44.3+2.843x \pm 1.96\sqrt{0.0325x^2 - 3.046x + 42{,}526}$	
	95% confidence zone of (B) line	: (E) line(upper)
	ditto	: (F) line(lower)
	$y = 44.3+2.843x \pm 1.96\sqrt{0.0325x^2 - 3.046x + 423.0}$	

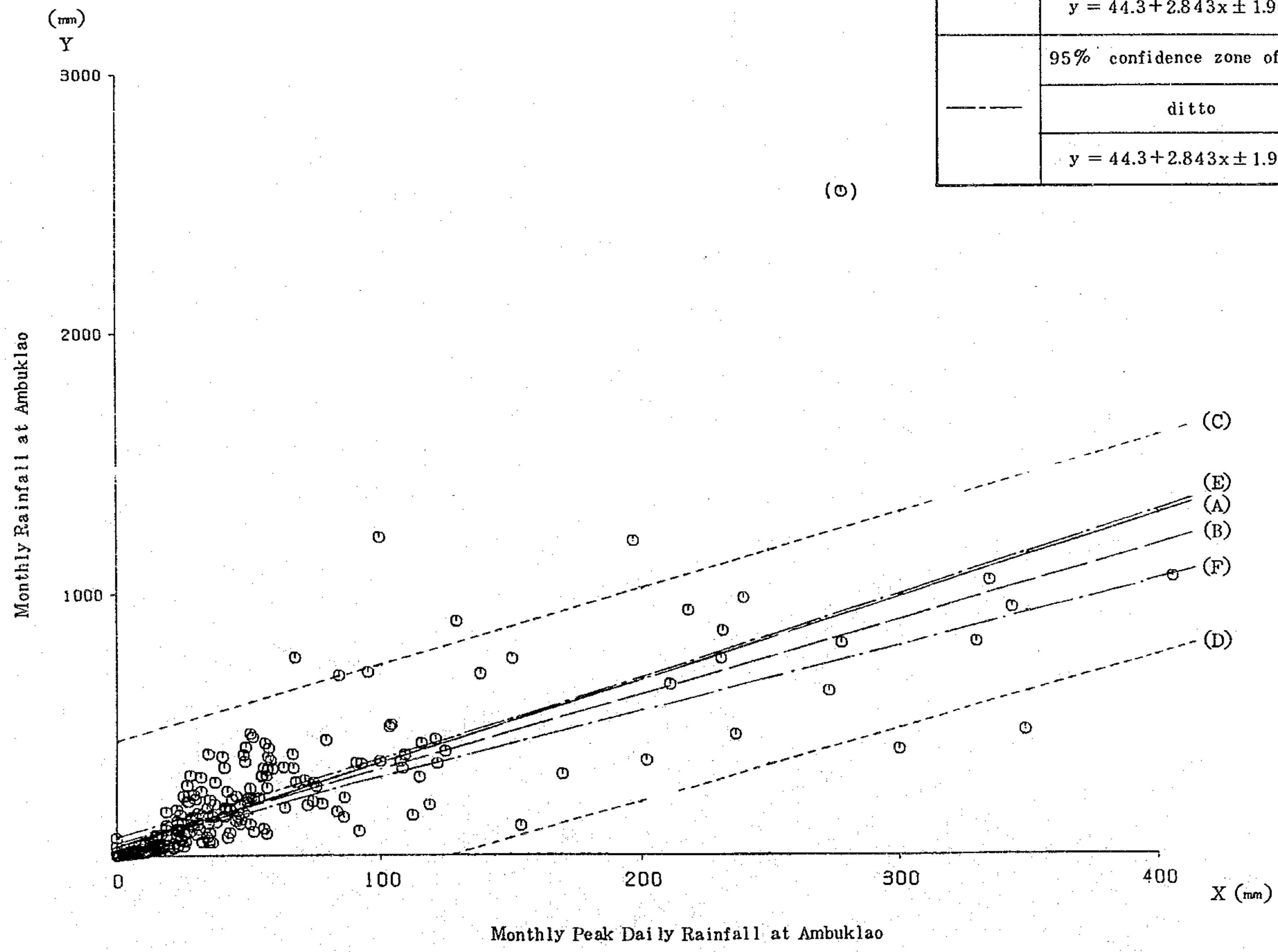
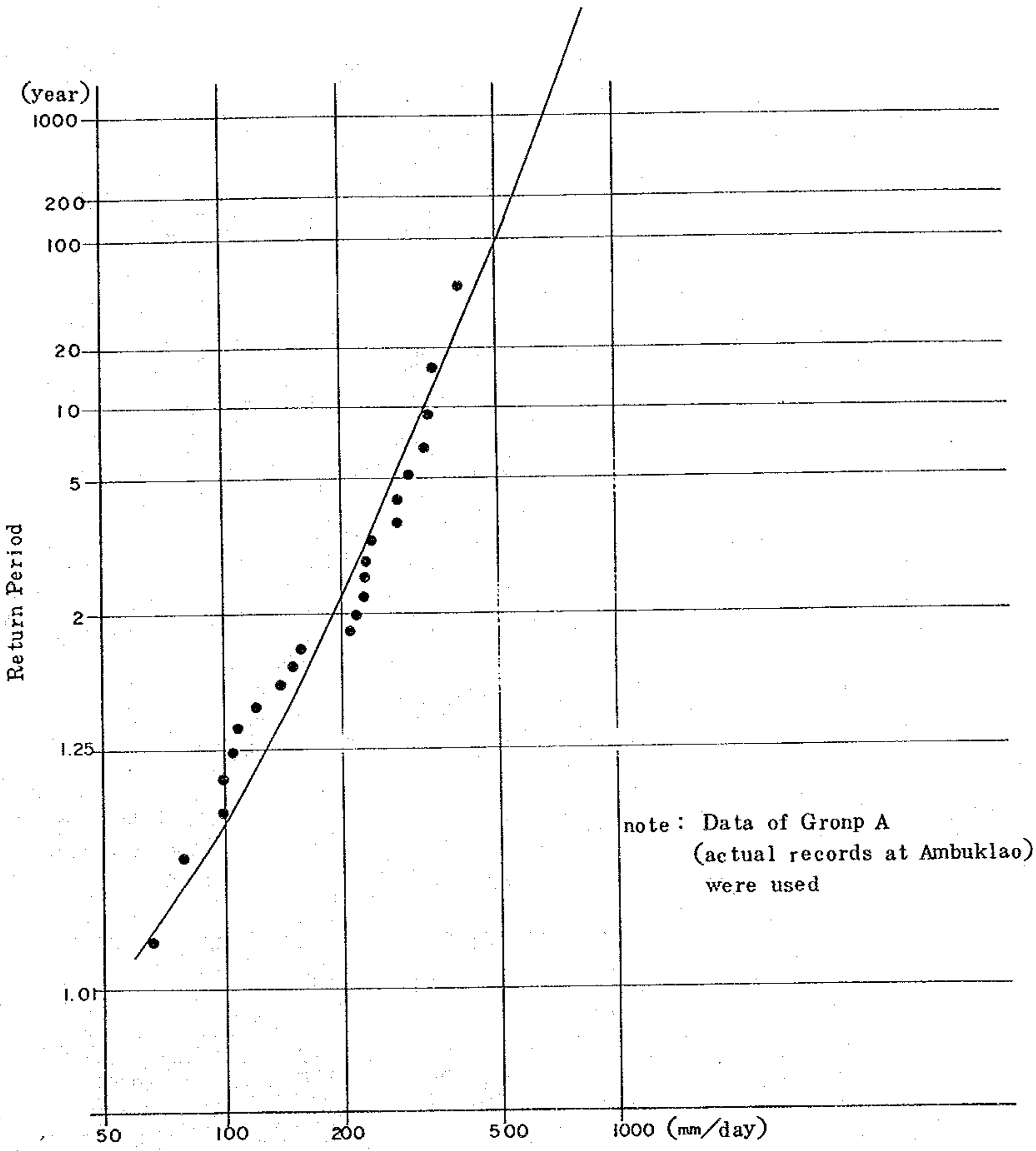


Fig — 9.2 Relation of Monthly Peak Daily Rainfall and Monthly Rainfall at Ambuklao



Peak Daily Rainfall at Ambuklao Dam

Fig — 9.3 Probable daily rainfall at Ambuklao dam

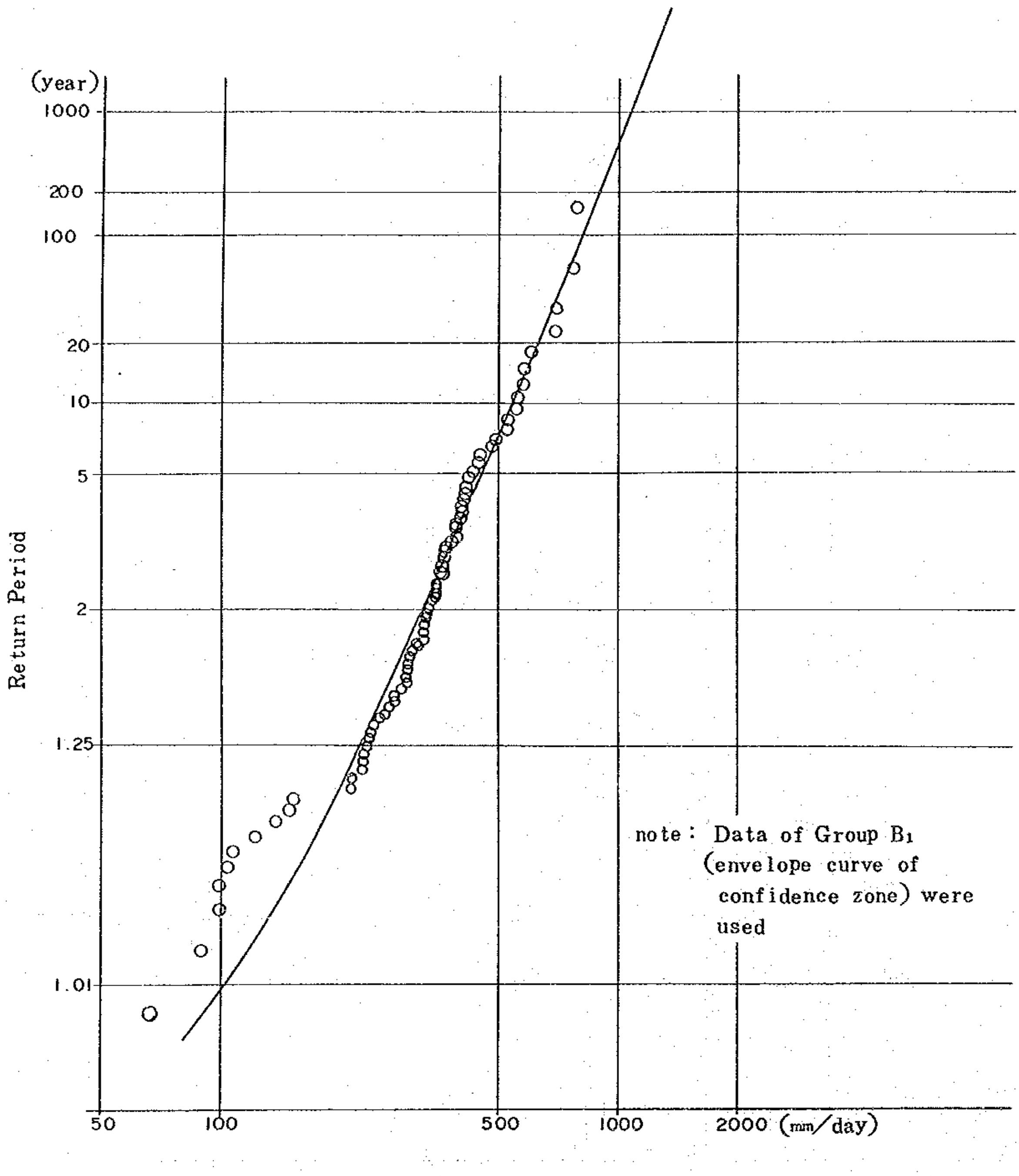


Fig — 9.4 Probable Daily Rainfall at Ambuklao Dam

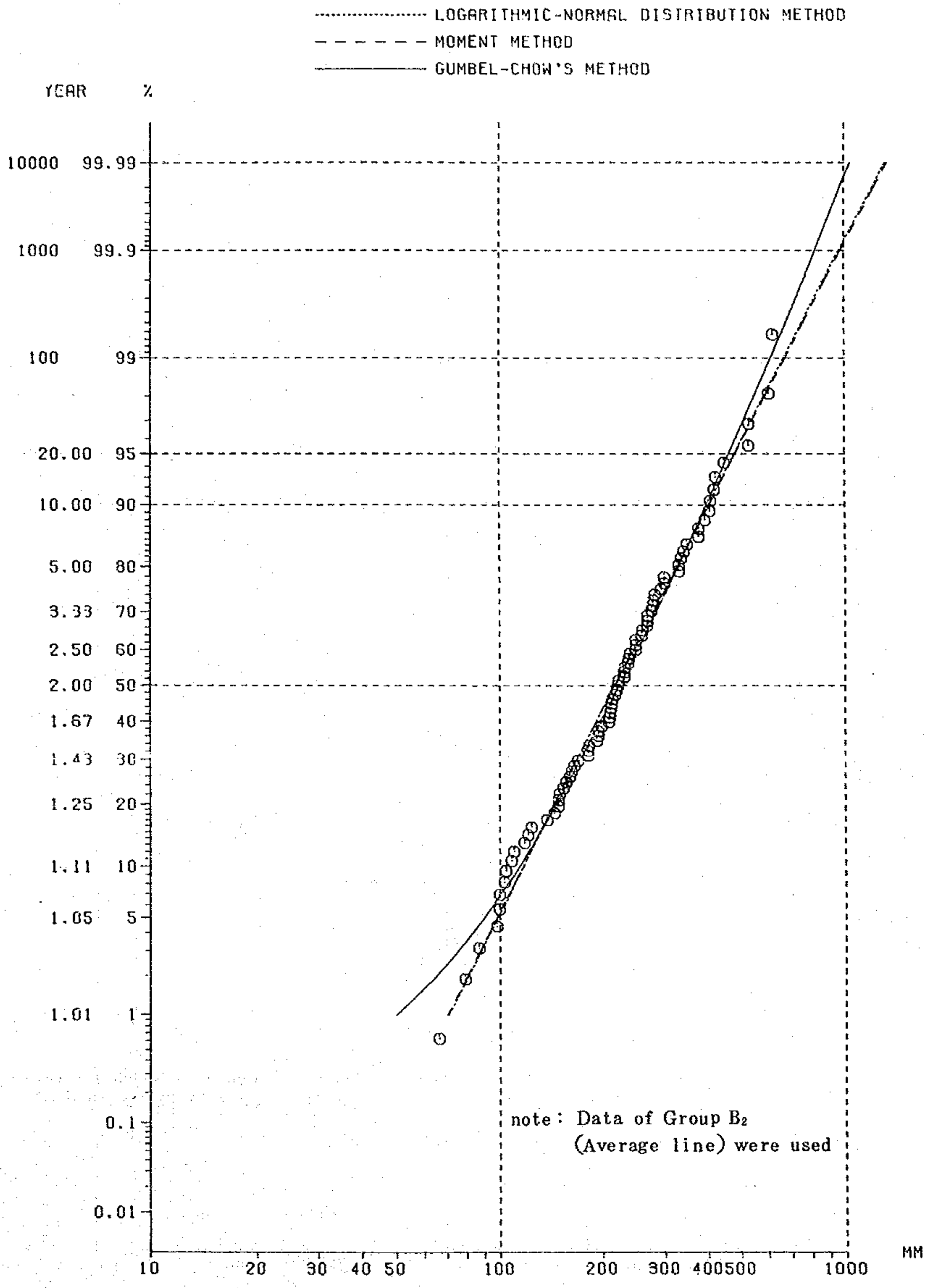


Fig — 9.5 Ambuklao Dam Peak Daily Rainfall

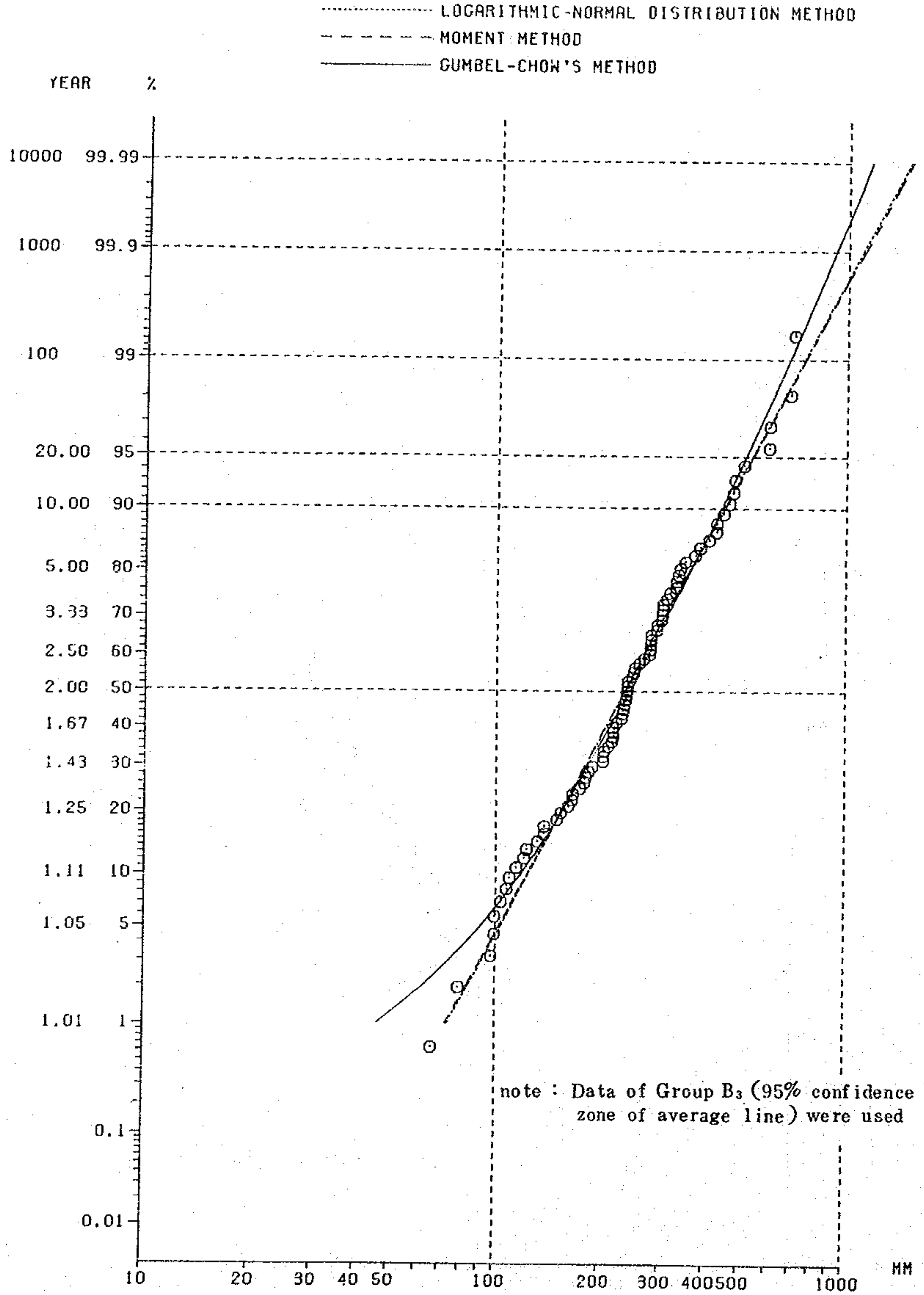


Fig — 9.6 Ambuklao Dam Peak Daily Rainfall

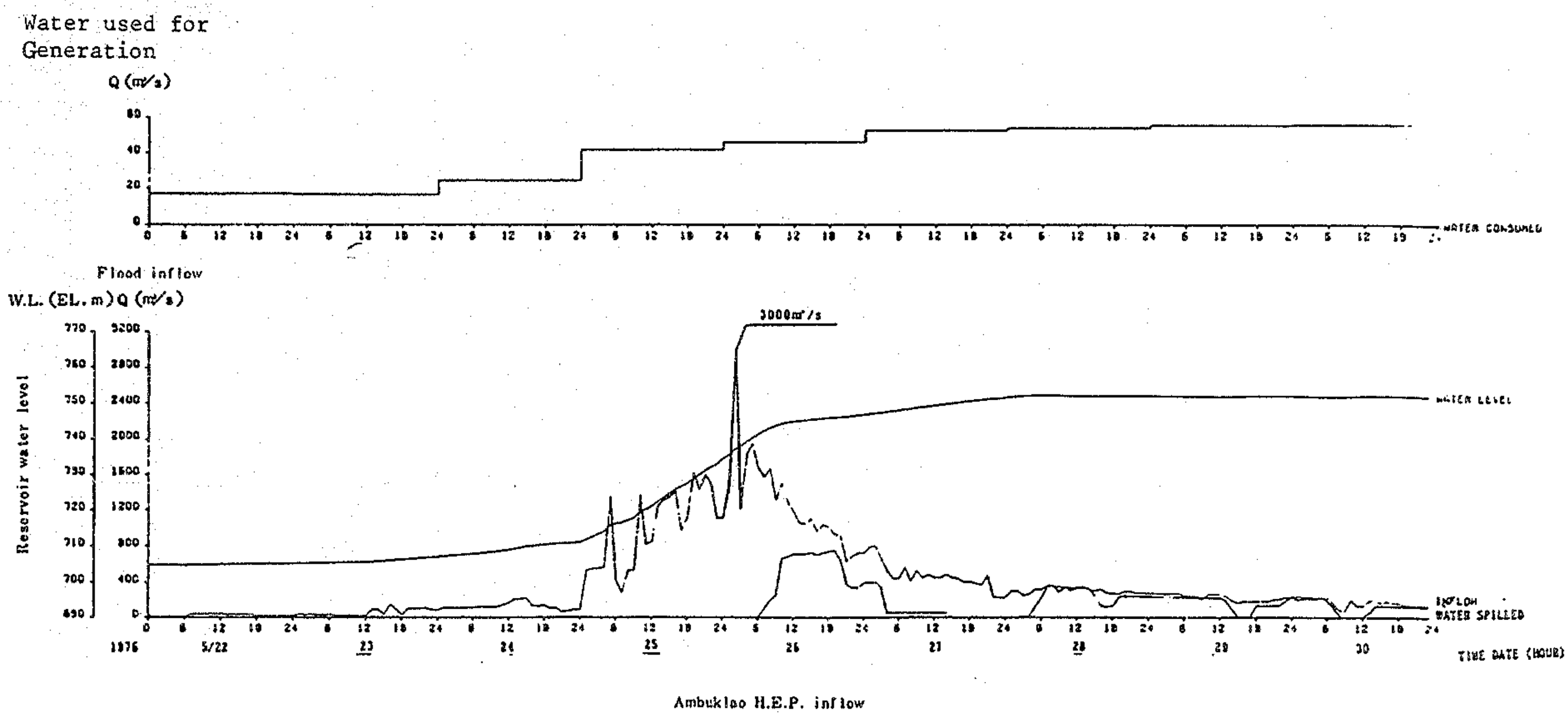


Fig — 9.7 Variation of Water Level, Water Consumed, Inflow and Spilled Water

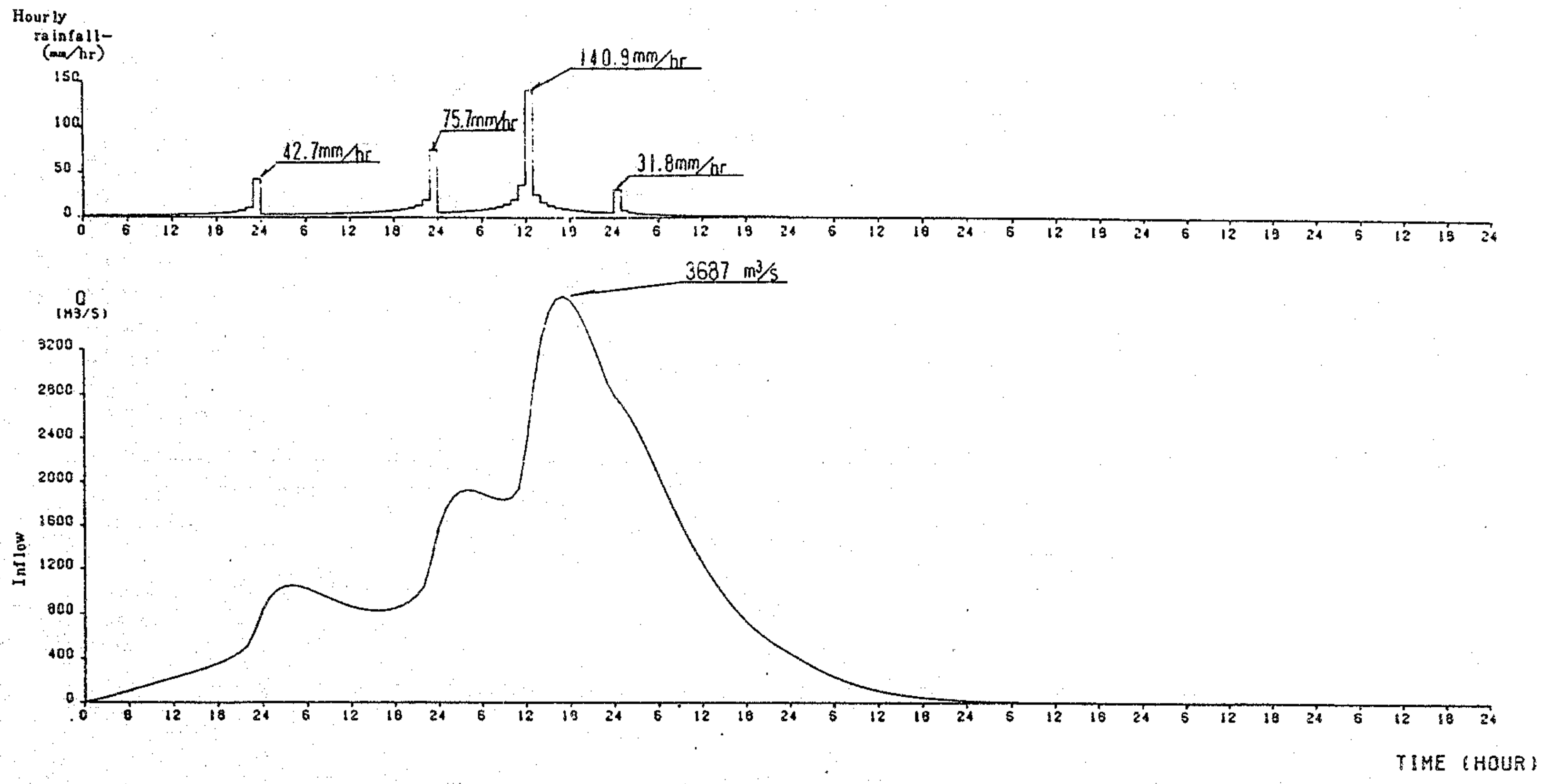


Fig — 9.8 Runoff Computation (f = 0.8) with Peak Daily Rainfall of 406mm at May, 1976

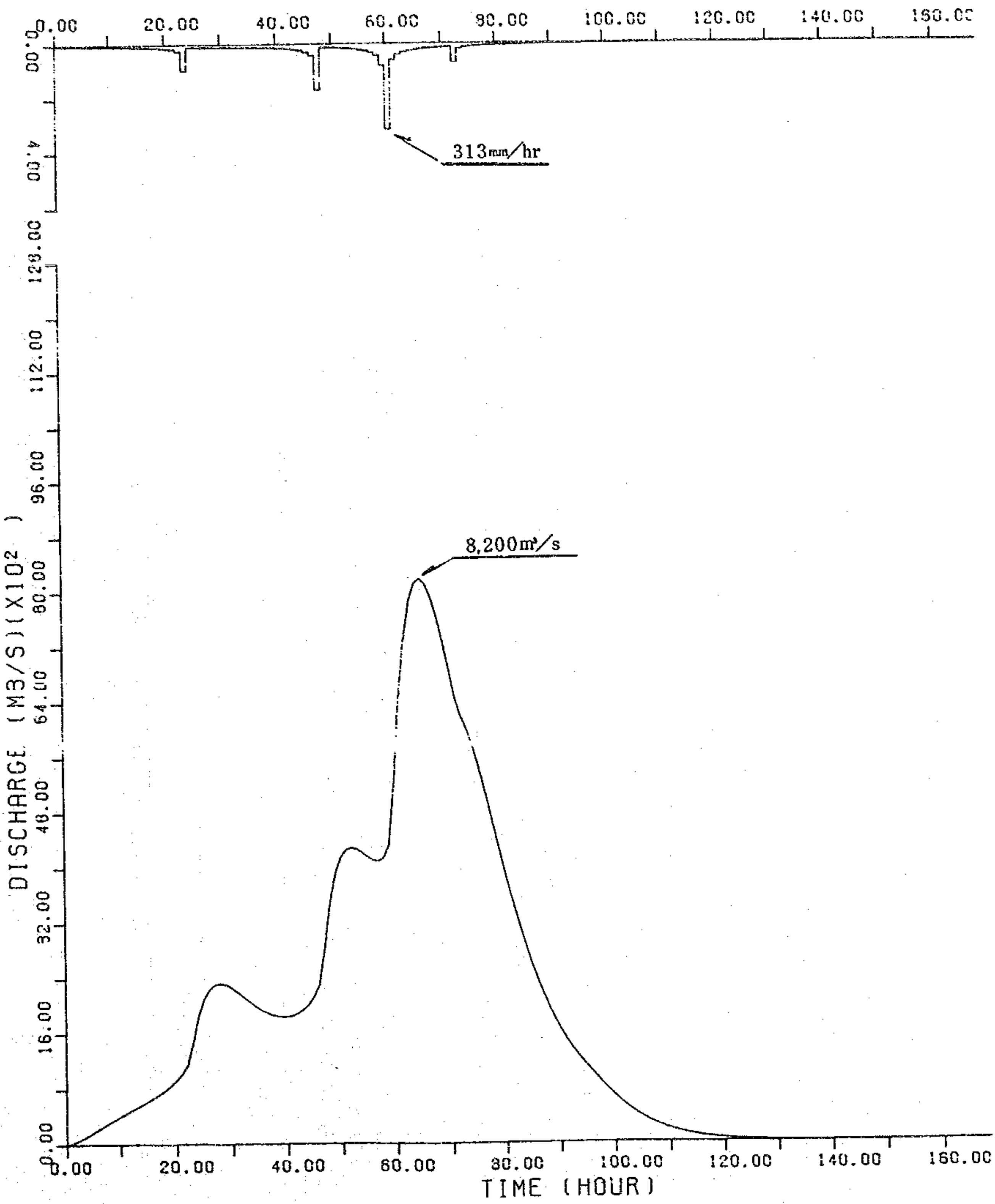


Fig — 9.9 Curve of Flood Inflow (200 — year return period)

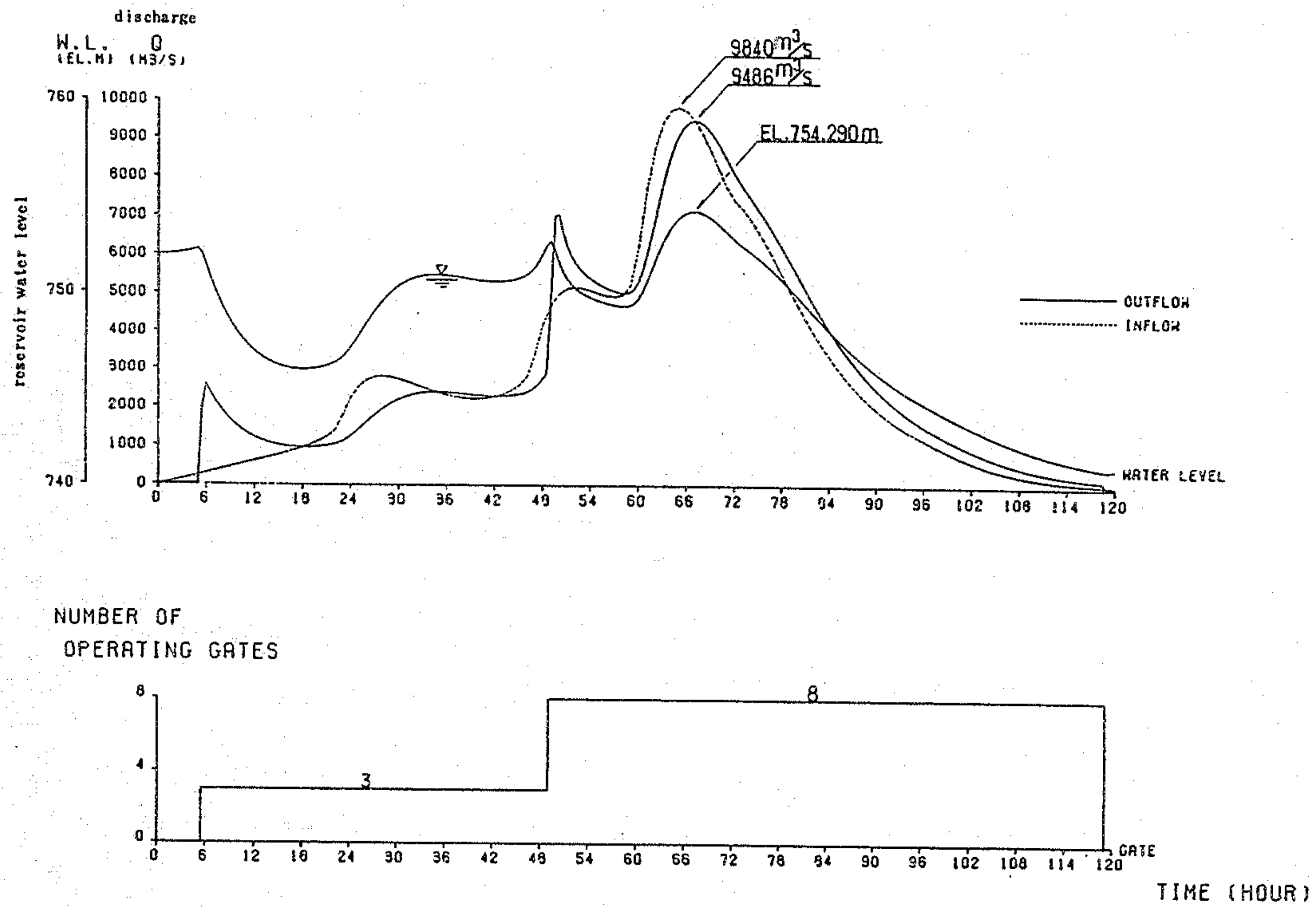


Fig — 9.10 Gate Oreration under Flood Inflow (200 - year return period * 1.2)
& Time Series Cunves of Dam Water Level and Outflow

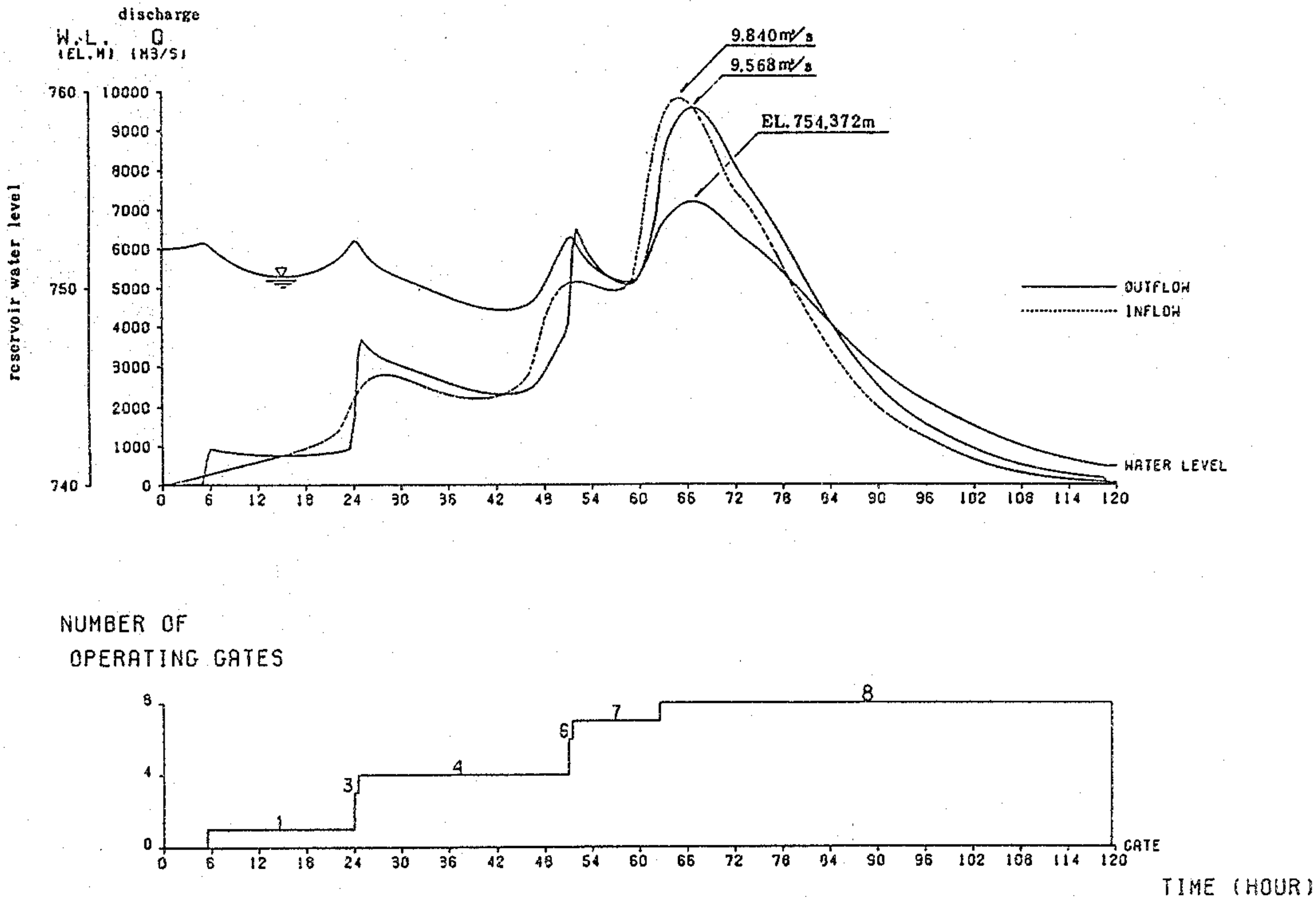


Fig − 9.11 Gate Operation under Flood Inflow (200−year return period＊1.2)
& Time Series Curves of Dam Water Level and Outflow

10. Safety Control System

10. Safety Control System

10.1. General

Continuous inspection and surveillance are essential in maintaining the project facilities in good operating condition. This will make the facilities to act to the best of their functions, stably and efficiently, and to obtain the desired results as plant investment.

The JICA Study Team had an opportunity to investigate the present conditions of the existing power facilities in the Philippines. As a result of the investigation, it was concluded that the operation of these facilities did not appear to be quite satisfactory. It was found that there was a big possibility for improvement in maintenance and operation of the power system of the Philippines.

There are big differences between Japan and the Philippines in terms of power consumption, weather conditions, and scale and organization of power generation facilities. Taking into account the above differences, it was realized that there were big possibilities for more effective utilization of the facilities by improving the maintenance and operation organizations and by investing money in operations.

If the above would lead to a certain increase in the production of the facilities, this could be equal to the investment required for installation of hundreds of additional power facilities.

The improvement of the existing facilities is financially justified as the planned maintenance and operation costs, on a capital basis, would be one tenth of the investment required for provision of additional power facilities.

Although power demand increases for the last few years have been low, the potential for power demand in the Philippines is still large, and it is obvious that the power demand in the future will increase considerably.

In anticipation of the expansion of the power facilities in the future, it is indispensable to improve and expand on the maintenance and operation of the existing power generating facilities. This will add to the efficient operation not only of the current but also of future power facilities which would be added to the system.

10.2. Northern Luzon Regional Center

The Northern Luzon Regional Center of NAPOCOR, of which the Ambuklao powerplant is a part, is the center of the Luzon Grid. In this capacity, the Center is responsible for operation and maintenance of five hydro powerplants of total capacity of 863 MW, and it includes such plants as Binga, Angat and Magat.

The Center comprise about 1,590 NAPOCOR staff. The five hydro powerplants managed by the Center are located in four regions. The Center is staffed mostly with electrical and mechanical engineers. The center is maintained and operated by 1,120 operational staff (operation 455, maintenance 665) including engineers assigned to the power facilities over 4 regions.

The Center includes no civil engineers except for the three hydrologists posted at the Angat powerplant. The Center has under its juridiction several large dams and other civil facilities. As such, it is responsible for surveillance and maintenance of these facilities. However, it is surprising that it does not include in its organization civil engineers with experience and background in surveillance and maintenance of such structures. The above means that such serveillance and maintenance is not done by engineers who are familiar with this type of work.

If the dams and other related civil structures are not properly surveyed and maintained, any serious problems which could certainly develop in the future, could be taken care of only at great expense.

Therefore, it is recommended that at least two civil engineers and three assistant engineers be detailed for maintenance, surveillance and inspection of the civil structures of the hydroelectric projects under the jurisdiction of the Northern Luzon Regional Center, including Ambuklao. (Fig.-10.2)

10.3. NAPOCOR Head Office

The current situation of the power facilities is complicated. Many of the existing plants have been in service for many years and, thus, are considered to be very old from operational point of view, others are newly constructed and, thus, in service for relatively few years and, apparently without serious problems.

The present conditions of the civil structures are the same or similar to those of the power facilities.

For smooth operation of these facilities it is necessary to regularly analyze the maintenance, inspection and surveillance results, and to establish yearly and longterm maintenance programs. Budgeting and maintenance work including inspection, surveillance and repair work, should be carried out in accordance with the maintenance programs.

Besides the above, continuous surveillance by monitoring the behaviour of large and important civil structures should be made. The results should be analyzed regularly by specialized civil engineers.

For efficient execution of these tasks, it is recommended that a maintenance division be established in addition to the design division of the Hydro Power Projects Department of the Head Office. A number of civil engineers as required should be assigned to the above maintenance division. (Fig.-10.1)

Certain power companies use a system of analysis which allows them to evaluate the conditions of the structures and foresee their future behaviour. By diagnosing the conditions of old and new structures, the most effective way of operating the facilities at a minimum cost, are developed.

10.4. Ambuklao Powerstation

With regard to the Ambuklao Project, it is recommended that the existing conditions of all civil structures of the project be clealy established and understood. This can be done by collecting all available data from the project completion and making a survey of the existing conditions of the structures.

As many important design and construction data on the project have been lost with the passage of time, understanding of the current conditions of the structures would be more difficult to accomplish. Therefore, to properly understand conditions of the structures, a serious study of all the existing data should be made. It is understood that many important data on the design and construction of the project are not available any more. Some of these data could be obtained by surveying the structures in the field with a purpose of obtaining the information required for evaluation of the existing conditions.

10.5. General Remarks

Implementation of the Safety Control System for each project should be carried out in accordance with the importance of the project or year of construction. Once the program has been implemented, the work should be performed systematically and at a minimum cost.

Inspection, surveillance and maintenance of civil structures must be conducted regularly. The Northern Luzon Regional Center which has projects with large and important civil structures under its jurisdiction, should assign to each project at least one civil engineer with the responsibility for maintenance and inspection. In addition to the above duties, this engineer should also supervise the operation of the spillway gates, observe reservoir water levels, etc.

Resident civil engineers at each project could be given the necessary authority to carry out small repair or maintenance work, efficiently and smoothly, without asking special permission for the Head Office.

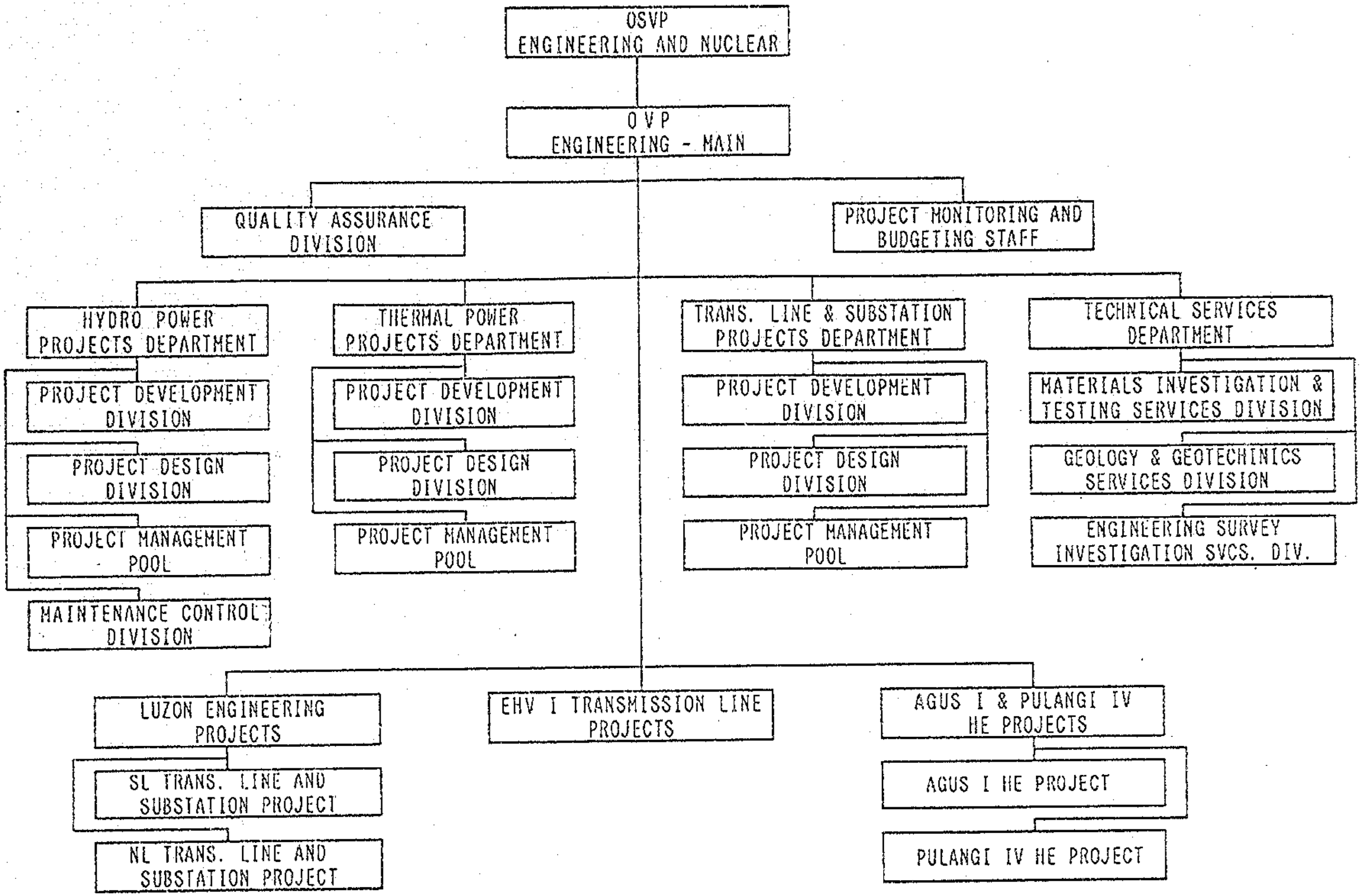


Fig - 10.1 Organization Chart for NAPOCOR Head Office

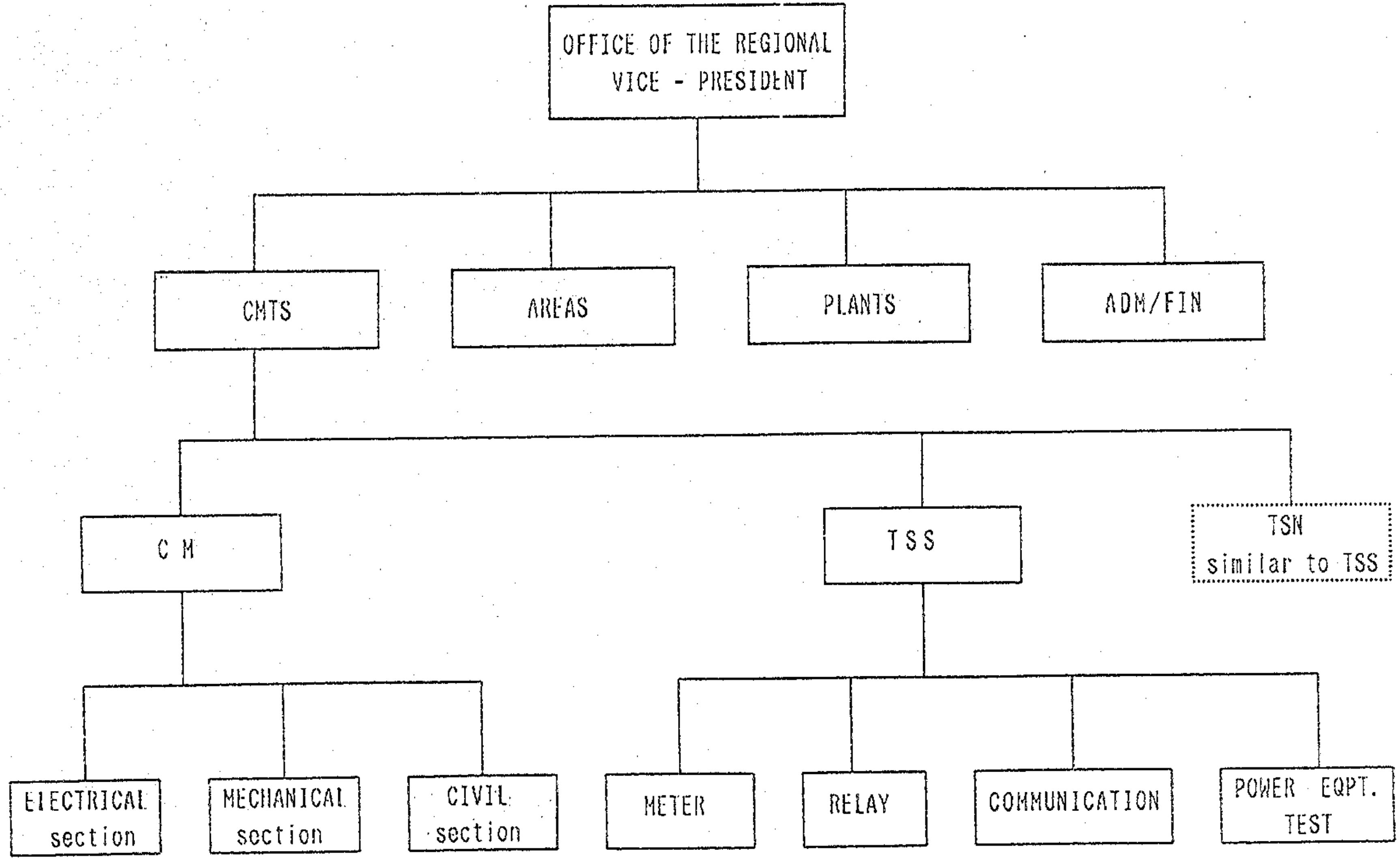
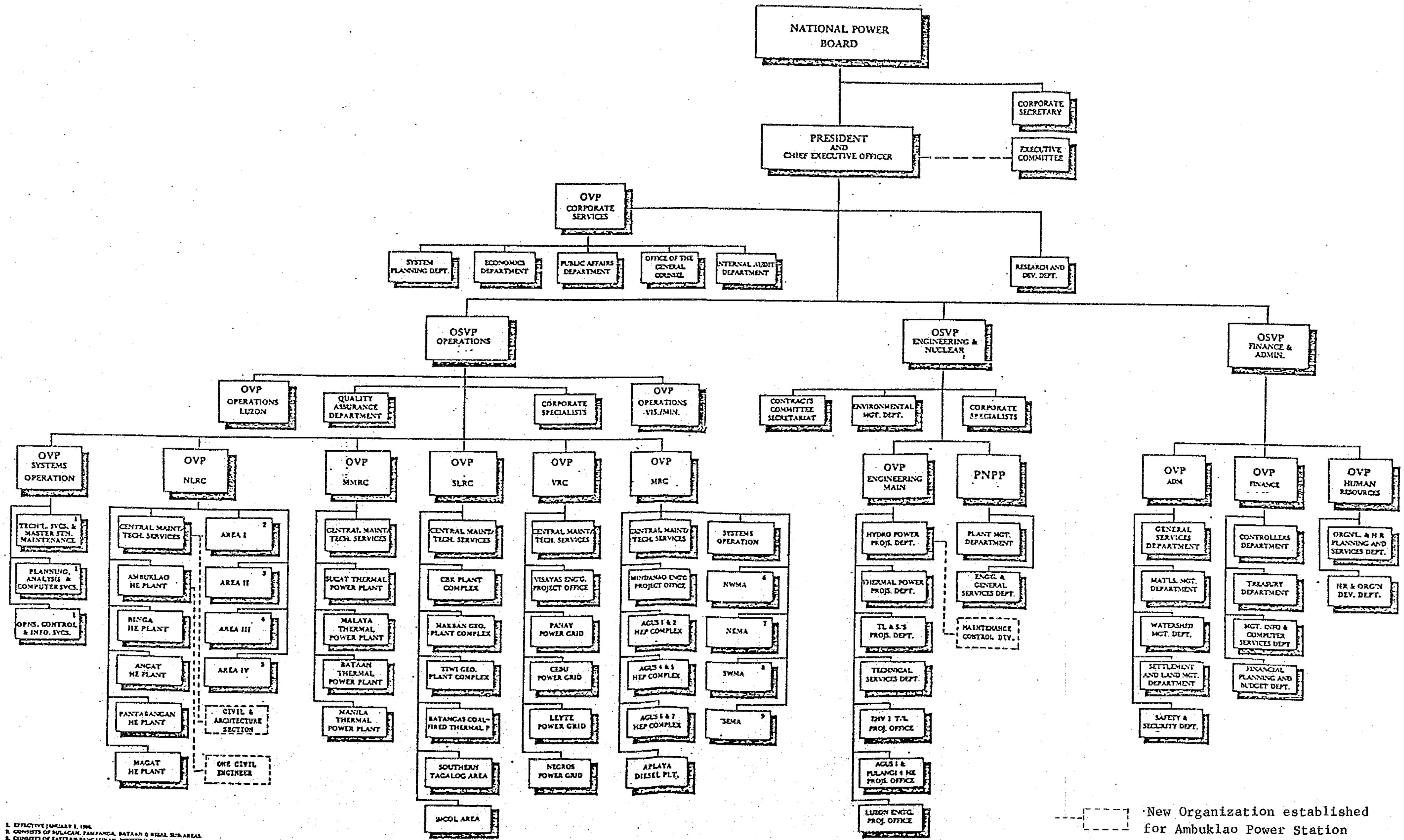


Fig - 10.2 Organization Chart for Northern Luzon Regional Center (NLRC)



1. EFFECTIVE JANUARY 1, [illegible]
2. CONSISTS OF BULACAN, PAMPANGA, BATAAN & RIZAL SUB-AREAS
3. CONSISTS OF EASTERN PANGASINAN, WESTERN PANGASINAN, ZAMBALES & TARLAC SUB-AREAS
4. CONSISTS OF ILOCOS NORTE, ILOCOS SUR, NORTHERN BENGUET, SOUTHERN BENGUET & LA UNION SUB-AREAS
5. CONSISTS OF CAGAYAN, ISABELA, NUEVA VIZCAYA-QUIRINO & NUEVA ECIJA-AURORA SUB-AREAS
6. CONSISTS OF ILIGAN, LANAO SUR, AURORA & ZAMBOANGA SUB-AREAS
7. CONSISTS OF CAGAYAN DE ORO, BUTUAN, SURIGAO & AGUSAN SUB-AREAS
8. CONSISTS OF GENERAL SANTOS & KABACAN SUB-AREAS
9. CONSISTS OF SOUTH DAVAO, NORTH DAVAO & RIZAWE SUB-AREAS

* Down to Department/Grid level.

Fig. – 10.3 Organization Chart for NAPOCOR

11. Sedimentation

11.1. Reservoir Sedimentation

11.1.1. Hydraulic Analysis

To forecast the sediment inflow and deposition progress in the future, the following sequence in the study of the sedimentation was used:

(1) Determine sediment quantity and sedimentation shape judging from sedimentation progress through years using the contour survey map of the reservoir.

(2) Evaluate the yearly sediment yield by studying the relation of sedimentation by year and runoff.

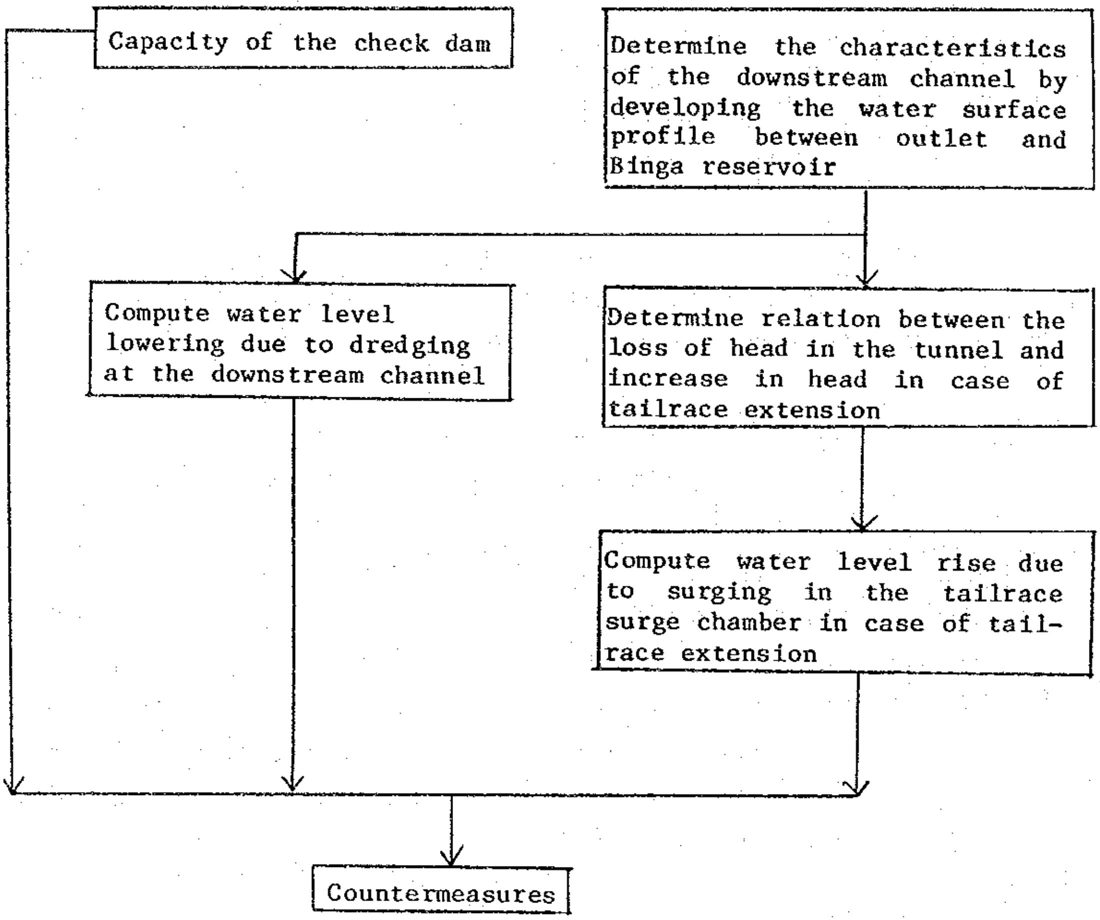
(3) (a) Estimate the level of sediment from recent sedimentation records.

(b) Forecast future sedimentation progress by simulation of sedimentation in the reservoir and calculate water level - capacity curve and sediment elevation at the intake.

(4) Hydraulic study of temporary measures.

(5) Hydraulic study of permanent measures.

The hydraulic study for the required countermeasures at the tailrace outlet was made as follows:



11.1.2. Characteristics of the Reservoir Sediment

The Ambuklao Dam was completed in 1956. The volume of sediment in the reservoir during the 30 years since its completion amounts to about $110.5 \times 10^6 m^3$ ($3.68 \times 10^6 m^3$ per annum). The quantity of the sediment in the Ambuklao reservoir is compared with that of other reservoirs in Figs.-11.1 and 11.2.

Fig.-11.1 shows the relation between the annual sediment yield per unit catchment area for the Ambuklao reservoir: $q = 3.68 \times 10^6/690 = 5,337\ m^3/km^2/year$, and catchment area of Ambuklao ($A = 690\ km^2$), as compared to that of Japan, Taiwan and Indonesia.

Fig.-11.2 shows relation between the above sediment (q) and the ratio ($C/I = 0.25$) of the gross storage capacity at the time of dam completion ($C = 327.17 \times 10^6 m^3$) to the mean annual inflow ($I = 1,320 \times 10^6 m^3$), as compared to that of Japan (157 reservoirs), USA (58), India (47), and other countries (14).

The data in Fig.-11.2 indicate that the sediment deposition in the Ambuklao reservoir is one of the largest among the reservoirs of a comparable size.

The sediment level around the intake tower is at about the intake sill, EL 686.0 meters. This was established by measurements taken in 1986. Therefore, studies for countermeasures against sediment inflows into the intake must be performed urgently.

The inflow of sediment into the power intake was first reported around 1980. At that time, it was observed that the amount of siltation in the turbine cooling water system became excessive. Since 1984, in particular, during every typhoon season, blocking of the cooling water pipe and abrasion of turbines due to sediments of sands and silts have been reported.

For the investigation of the sedimentation progress, cross sections at 200 meters intervals were prepared using the contour maps of the reservoir surveyed in 1986, 1980, 1967 and 1956. Then the profile along the lowest levels of each cross section was prepared (For illustration, see Figs.-11.3 through 11.5).

The above data and other hydrologic data were used for the study described in the following paragraphs.

(1) Relation between Sedimentation Shape, Grain Size and Reservoir Water Level

It was observed from the sediment profile along the riverbed of the Ambuklao reservoir (Figs.-11.3 and 11.4) that EL 740 (Point A in Fig.-11.3) and EL 705 (Point C in Fig.-11.3) are progressively moving toward the dam while maintaining the same level.

Point A, EL 740 m is 12 m lower than the high water level at EL.752 m. The level of point C is almost the same as that of the lowest reservoir water level which occurs every year (the mean lowest reservoir water level at EL 708.7 m.). The slope of the line connecting Point B to C is about half the gradient of the Agno River which existed at that location before construction of the dam.

It is judged from the above that the section between Points B and C is formed by the flood when the reservoir water level begins to rise from its lowest level. On the other hand, the sedimentation at Point A is assumed to be formed by the flood when the water level reaches the high reservoir water level (HWL).

The grain size of the sediment was established to be as follows:

- Upstream of Point B : sand and gravel
- From Point B to D : gravel mixed with sand
- From Point D to E : large amount of silt

The gradient from EL 740 to EL 720 meters of the tributaries Labey, Bantey and Pesack, is very steep (1/30 - 1/20 : See Fig.-11.5). Transport of large grain size sediment from these tributaries is anticipated at the time of flood occurring at low reservoir levels. These three tributaries, therefore, present a rather serious threat to the intake structure.

After classification of sections by elevations, based on the reservoir profile along the river, the sediment volume of each section was computed to be as shown in Table-11.1.

(2) Relation between Sediment Volume and Inflow

The mean sediment volumes per year (annual sediment yield) for each period of survey are as given below.

Period	Volume
1956 - 1967	$3.008 \times 10^6 m^3$/year (= 33.09×10^6/11)
1968 - 1980	$5.307 \times 10^6 m^3$/year (= 68.991×10^6/13)
1981 - 1986	$1.399 \times 10^6 m^3$/year (= 8.394×10^6/6)
1956 - 1986	$3.683 \times 10^6 m^3$/year (= 110.475×10^6/30)

The sediment volume varies according to the period. This is due to the scale of the flood discharge for each particular period. Fig.-11.6 shows the relation between the total annual inflow and the sediment for each period since 1956.

The sediment is generally in proportion to the 2nd to 5th power of the inflow, The quantity of fine grain-size silt is in proportion to the square of the discharge. On the basis of the above, the relation between the sediment volume and the inflow shown in Fig.-11.6 is regarded to be appropriate.

According to C.C. Inglis, M.G. Wolman and L.B. Leopold's study of the relation between the probability of the mean sediment and the inflow, the mean sediment is found tohave a probability of occurrence of 1.4 to 2.0 years.

The runoff frequency of 1985 which corresponds to a 2-year flood was selected as a representative frequency and was used in sediment simulation.

11.1.3. Forecast of Sedimentation Progress

(1) Forecast based on Past Sedimentation Progress

Using past sedimentation progress shown in Fig.-11.3, the tendency of sedimentation was classified for the following three reservoir sections:

a) Mean sediment volume - upperstream from Point B
$0.463 \times 10^6 m^3$/year

b) Same - Point B to D
$1.844 \times 10^6 m^3$/year

c) Same - Point D to E
$1.376 \times 10^6 m^3$/year

It is estimated that Point D will reach the intake in about 6.2 years (1992) assuming that the above sediment will be accumulated on the top of the sediments surveyed in 1986. (Computation is shown in Fig.-11.7).

(2) Forecast of Sediment Shape using Simulation Model

The calculation methods and formulae used for this forecast were as follows:

Flows and sediment movements	- One dimensional analysis method
Flows	- Unsteady flow calculation method
Bed load	- Shinohara-Tsubaki's formula
Suspended load	- Lane-Kalinske's formula
Wash load	- Odd-Owen's formula

a) Simulation was made on the basis of river bed levels (top of sediments) surveyed in 1980 and 1986. The initial river bed elevation simulated was the actual value surveyed in 1980. Assuming this as the initial condition, the level rise from 1980 to 1986 was simulated and the simulated value for 1986 was compared with the actual value of 1986. The result is shown in Fig.-11.10 and was regarded to be satisfactory.

The following assumptions were used in the above computations:

Flow pattern
The actual values of daily discharges for 1985 corresponding to the average year, were applied in the calculations. (For illustration, see Fig.-11.8). As a peak flood discharge, the inflow of 1,735 m^3/s (which occurred in June, 1985) was applied.

Only the data when the discharge exceeded 100 m^3/s after June 21 and lasted 52 days were included. The same flow pattern was repeated for every year.

Reservoir water level

The initial water level in every year for the calculation was assumed to be at EL 708.7 m. This was the average lowest water level from 1957 to 1986. The reservoir water level was determined on the basis of the relation between the inflows and the water quantities used for generation (61.4 m^3/s). For reservoir water levels above EL 752 m, the water was assumed to be released through the spillway so that EL 752 m, as a maximum reservoir water level, was maintained.

Number of sections

Sixteen sections in the Agno River and seven sections in the Bokod River were used. The three tributaries (Labey, Bantey and Desack) were disregarded.

Sediment volume

The mean sediment inflow of 1.399 x $10^6 m^3$ from 1981 to 1986 was applied as the annual sediment inflow. The breakdown of the annual sediment inflow was assumed as follows:

Silt = 0.977 x $10^6 m^3$/year

Gravel and sand = 0.422 x $10^6 m^3$/year

For the input, this was devided in proportion to the catchment areas of the Agno and Bokod Rivers.

Distribution of grain size of sand and gravel

The distribution of the grain size of sand and gravel was done in accordance with the information presented in Fig.-11.9. This information was based on the actual survey data.

b) The calculation results for the forecast of the sedimentation progress from 1986 to 2035 are shown in Fig.-11.10. Table-11.2 shows the changes in the reservoir capacity. It was assumed that after 2035, a part of the sediment will travel downstream of the dam through the spillway. The remaining part will be trapped in the reservoir and deposited there.

The assumptions used in the above calculations, period 1986 to 2035, were as follows:

Flow pattern, distribution of grain size of sand and gravel, number of sections:

The assumptions for all above items were the same as those for the period of 1980 to 1986.

Reservoir water levels

Comparing the sediment level around the intake tower in every year plus 14.65 m (= EL 695.65 *1 - 681 *2) with EL 708.7, the higher water level was adopted.

*1 = elevation of the intake top

*2 = elevation of sediment in 1980 around the intake tower

This was to maintain the distance between the sediment level and the water level, no smaller than 14.65 m.

Input Sediment Volume

The volume of 3.683 x $10^6 m^3$/year, the mean sediment for 30 years from 1956 to 1986, was applied.

The following breakdown of the sediment was used:

Silt: 1.376 x $10^6 m^3$/year
Sand/gravel: 2.307 x $10^6 m^3$/year

11.1.4. Dredging around the Intake Tower

To investigate the conditions of sediment inflow into the intake tower which occurs due to scouring of the sediment surface around it, flow simulation studies were conducted using the following parameters:

- Reservoir sediment levels: Data measured in 1980 and 1986.
- Topographic range: Between the dam and a section 300 m upstream from the center of the intake tower.
- Reservoir water level: L.W.L. 694.0 m.
- Intake flow: 61.4 m^3/s
- Number of computer outputs for flow calculation: 7 layers in depth and 198 points on a plane, totally 1,386 points

The sediment inflow into the intake was reported for the first time around 1980. The sediment level in 1980 was about 5.4 m lower than the level of the intake sill, EL.686.0 m. The sediment level reached EL 686.0 m in 1986. These conditions are illustrated in Figs.-11.11 and 11.12.

The flow velocities near the bottom which cause sediment movement are shown in the following table.

Sediment levels at	Year	
	1980	1986
A point 300 m upstream of the intake tower	1.11 cm/s (0.55 mm/s)	1.44 cm/s (0.78 mm/s)
Around the intake tower	3.43 cm/s (1.52 mm/s)	34.99 cm/s (5.42 mm/s)
Near the dam	1.74 cm/s (0.77 mm/s)	2.90 cm/s (1.45 mm/s)

The values in parenthesis are friction velocities. The limiting conditions at which the sediments start moving and are carried by the flow are expressed, as commonly used, by friction velocity (U*c).

According to Dr. Muraoka who measured such friction velocities in the rivers and lakes of Japan, the critical friction velocity U*c can be from 6 to 30 mm/s. In case the critical friction velocity of 6 mm/s is applied to the wash load around the intake tower, floating of such materials should occur to within a radius of 10 m. However, no such floating was observed in 1980.

At the time of flood and for some weeks after the flood, high density wash load movements occur in the reservoir. This becomes a density current and piles up between Points E and D as shown in Fig.-11.3, and forms a density current bed. The abrasion of the turbine is accelerated by the sand flowing into the intake at the time of typhoon.

Assuming that the sediments along the 300 m length of the reservoir immediately upstream of the intake tower consist of sands, the limit flood discharge which causes floating was calculated by Dr. Iwazaki's formula at respective reservoir water levels, and summarized in the following table.

Grain Size	Reservoir water level	Flood discharge at the floating 1980 topo*	1986 topo*
		m^3/s	m^3/s
Wash load	EL 694	870	364
	EL 720	3417	2800
Sand 0.56 mm	EL 694	2437	1020
	EL 720	9570	7830
	EL 740	16500	14650
	EL 752	21220	19320
Sand 1 mm	EL 694	3228	1350
	EL 720	12680	10380
	EL 740	21860	19412
	EL 752	28110	25600

* Topographic Data.

The flood discharges which occur at the end of a dry season (May-Middle of July), when the reservoir water level is low, are 700 to 3,000 m^3/s. On the other hand, the flood discharges during a rainy season, at high reservoir water levels, are 800 to 2,083 m^3/s.

Thus, for sediment levels recorded in 1986, when the reservoir water is at low level, the flood has a potential of transporting to the intake a sediment with a grain size of 0.56 mm. To prevent the above from occurring, dredging of the sediments down to the 1980 sediment level, would be required.

The proposed plane range for dredging is as follows:

If dredging is carried out by starting at the intake tower and proceeding in the upstream direction, with stability gradient for the silt assumed at 1/75, the sediment inflow into the intake could occur around 1995. This is based on the results of simulation given in Fig.11.10(4). The stability gradient of 1/75 was adopted from the records of stability gradient studies made after dredging of the mouth of the Yamato-gawa River in Japan.

11.2. Tailrace Sedimentation

11.2.1. Tailrace Tunnel Extension

Extension of the tunnel was considered by construction of a conduit along the river downstream of the tunnel portal. This scheme also requires construction of an open channel about 110 meters long downstream of the conduit.

Countermeasures for prevention of sedimentation in the area downstream of the existing tailrace tunnel outlet from the Besar River were also studied. The hydraulic calculations for the check dams are described further below.

The increase in power generation due to tunnel extension was computed as required. The tunnel extension will cause increase in head due to a greater difference in elevation, between the reservoir and tailwater levels, however, the friction losses inside the tunnel will also be increased.

Also, the relation between the maximum upsurge in the surge tank at the time of load increase and the time required for load increase (turbine gate opening time) was studied.

To compute the increase in head resulting from tunnel extension, calculation for a non-uniform flow between the outlet and the Binga reservoir was made for turbine discharges of 61.4 m^3/s. The water surface profile obtained for these conditions is shown in Fig.-11.13.

The nomal water level at the existing outlet was assumed at EL 579.061 m for these calculations. The other conditions used for these computations were as follows:

Length	: 3,065 m
Number of cross sections	: 27 (1987 survey map)
Binga reservoir water level	: NWL 575.0

Manning's coefficient of roughness: n = 0.03

The next step was to compute the hydraulic effects of a 1,500 m extension of the tailrace tunnel by construction of the conduit. For this purpose, the water surface profile along the conduit was computed. The effects of the topography on the conduit arrangement were taken into account.

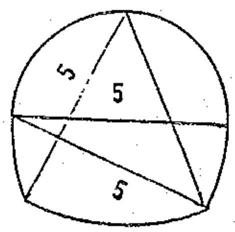
The physical characteristics of the extension were as follows:

(a) Conduit

Conduit shape: Standard horseshoe, 5m in diameter

$A = 20.733\ m^2$, $R = 20.733/16.345$

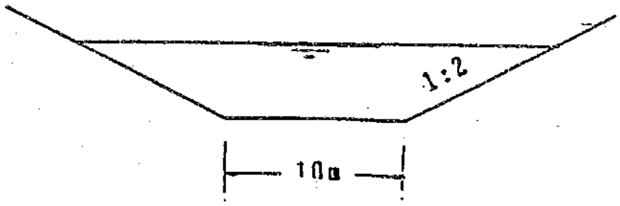
$= 1.268$ m



Conduit Gradient: 1/900

(b) Channel

Channel length 110m, Channel width 10 m, side slopes gradient 1:2



The head loss in the channel was calculated using the Manning's coefficient of roughness n = 0013, as follows:

Head loss due to friction	:	1.579 m
Head loss due to bends	:	0.305 m
Allowance	:	0.186 m
Total		2.070 m

The water level at the outlet of the extension structure was assumed at EL 576.4 m (Fig.-11.13). Therefore, the increase in head due to extension was found to be equal to EL 579.06 - El 576.4 = 2.661 m. The increase in loss of head due to the extension as stated above was 2.070 m. Thus, the net increase in head was found to be 2.661 - 2.070 = 0.591 m.

To obtain the contribution to the power generation on a yearly basis of the 0.591 m increase in head, a detail study, as affected by the changes in the reservoir water levels, would be required.

Approximate calculations, therefore, were made assuming the following operating conditions:

Reservoir water level : EL 723 m intermediate water level of NWL 752 and LWL 694 m

Water level in the tailrace: EL 579.061 m

Rate of generation increase: ratio of the head (143.939m = EL 723 - EL 579.061) to the increase in effective head (0.591 m)

$$0.591/143.939 = 0.0042 \quad \text{(about 0.4\% increase)}$$

Another hydraulic problem connected with the tailrace extension was the water level increase in the tailrace surge chamber at the time of the increase of load (power demand). The water level increases in the surge chamber can be maintained below certain levels by lengthening the time of turbine gate opening during load increase. Fig.11.14 shows the time required for load increase and highest water level.

The parameters given in the above figure are (1) existing tunnel and (2) existing tunnel plus extension. The water level at the outlet was assumed at the time of flood and normal level. At the time of flood, with the spillway discharging 9,490 m^3/sec., the water level at the tailrace was assumed at EL 582.85 m. The time required for the turbine gate opening (load increase) for this condition should be increased to 180 seconds in order to maintain the water level in the surge chamber below the level of its ceiling, EL.604.0 m. Even in case of using the existing tailrace, required time for load increasing should be taken more than 30 sec. for the protection measure against inflow into valve chamber by surging.

11.2.2. Dredging of the Area Downstream of the Existing Tailrace Tunnel

The sedimentation of the river along the first 1,500 meters downstream of the portal of the existing tailrace tunnel is seriously affecting the water levels of this section of the river. There is an increase in the water level of about 4 meters as compared to the original conditions (see Fig.-11.13). Thus, studies were made to determine whether by dredging the water level could be lowered to its original conditions.

Eight different schemes were studied for doing the above. The characteristics of the schemes and the results of the studies are shown in the following tabulation.

	Dredging gradient Channel gradient	Riverbed EL after dredging		Width of dredging		Q'ty of dredging $\times 10^3$	Outlet water level	Difference from EL 579.061
		At Outlet	At 1,390m downstream	Outlet up to 735m downstream	from 750m to 1500m			
		EL	EL	m	m	m^3	EL	
A	1/1130	576.23	575	25	25	48.5	578.050	1.011
B	"	"	"	25	50	79.7	577.780	1.281
C	"	"	"	50	50	122.2	577.435	1.626
D	"	"	"	50	100	231.2	577.399	1.662
E	1/1675	575.83	575	25	25	54.6	577.791	1.270
F	"	"	"	25	50	93.6	577.537	1.524
G	"	"	"	50	50	136.8	577.204	1.857
H	"	"	"	50	100	248.8	577.110	1.951

The increase in power generation is approximately calculated using the same approach as that in Paragraph 11.4.2. The results are tabled below (See also Fig.-11.15).

As shown in the table, Scheme E is more advantageous than Scheme A. The water surface profile and cross sections for dredging required for Scheme E are shown in Figs.-11.16 and 11.17, respectively.

Scheme	Amount of dredging in $10^3 m^3$	Percentage of output increase (%)
A	48.5	0.70
B	79.2	0.89
C	122.2	1.13
D	231.2	1.15
E	54.6	0.88
F	93.6	1.06
G	136.8	1.29
H	248.8	1.36

The mean annual sediment inflow from the Besal River, a tributary between the Ambuklao Dam and the tailrace outlet, should be estimated from the sediment in the Binga reservoir. A rough estimation from the sediment data for the Bokod, Labey, Bantey and Pesack Rivers was made for these studies. The mean annual sediment inflow from the Besal River was thus estimated at 456,000 m^3/year, excluding silt.

The sediment coming from the Besal River is 8.3 times the dredging amount assumed for Scheme E (54.6 x $10^3 m^3$).

The Besal River annual sediment yield per km^2 was calculated by first determining the annual yield of the Bokod, Labey, Bantey and Pesack Rivers, using the sedimentation data of 24 years (total sediment inflow) for the period of 1957 to 1980. Then, the sediment annual yields for the Labey, Bantey and Pesak Rivers were lumped together and the average of the above and the Bokod River yield was considered to be the annual sediment yield per km^2 for the Besal River.

The calculations prepared on the basis of the above, follow:

Period	River	Sediment	Mean annual sediment
		m^3	m^3/year
1957 - 1980	Bokod	4.095×10^6	170600
	Labey	4.088×10^6	170300
	Bantey	2.447×10^6	102000
	Pesack	3.508×10^6	146200

(1) Sediment/km^2 from the Bokod river:

$$170600/142.5 = 1,197.2 \ m^3/km^2/year$$

(2) Total sediment/km^2 from Labey, Bantey, Pesack

$$(170300 + 102000 + 146200)/80.7$$
$$= 5,185.9 \ m^3/km^2/year$$

The estimated value of mean annual sediment from the Besal river is:

$$143 \times (1,197.2 + 5,185.9)/2 = 456,400 \ m^3/year$$

11.3. Check Dam

11.3.1. Design Discharge and Sediment for the Check Dams

A plan for check dams was studied to prevent or delay the sediment from the Labey, Bantey and Pesack Rivers to reach the area of the intake tower. Countermeasures for prevention of sedimentation of the area downstream of the tailrace tunnel outlet from the Besar River were also studied. The hydraulic calculations for the check dams are described below.

Topographic map of the proposed locations for the check dams were not available. Thus, a map in scale of 1/50,000 was used by enlarging it to 1/4,000. Additional studies will be required when the check dam locations are properly surveyed and the estimated sediment volumes are confirmed.

The design parameters for the check dams were as follows:

River	Pesack	Bantey	Labey	Besal
Catchment area (km^2)	30.3	30.2	20.2	143
Mean annual sediment from actual records, 1957-1980 ($x\ 10^3 m^3$)	170.3	102	146.2	(456.4) Estimated
Planned sediment ($x\ 10^3 m^3$) (for sediment load)	1390 (for 8.2 years)	1160 (for 11.4 years)	217 (for 1.5 years)	907 (for 2.0 years)
Dam height (m)	21 (=14+7)	19.8 (=14+5.8)	20.8 (=14+6.8)	25.8 (=14+11.8)
Dam width (m)	172	258	82	90
Overflow length (m)	34	49	23	40

The design overflow discharge was calculated based on the probable daily rainfall for a 100-year return period (508 mm/day) and using the formula $Q = rfA/3.6$.

The runoff coefficient of $f = 0.7$ was applied. The other parameters used in the calculations were as follows:

River	Time of flood concentration	Peak hourly rainfall r	Catchment area A	Peak discharge Q
		mm/hr	km^2	m^3/s
Pesack	Within one hour	176	30.3	1037
Bantey	"	"	30.2	1034
Labey	"	"	20.2	691
Besal	Within 2 hours	110	143	3059

The design formula for overflow was as follows:

$$Q = (2/15)C(2g)^{1/2}(3B_1 + 2B_2)h^{3/2}$$

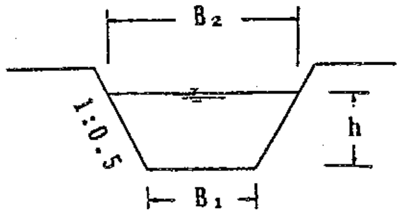
C : discharge coefficient = 0.66

B_1 : bottom width

B_2 : water surface width (side slope gradient 1:0.5)

h : water depth

Q : overflow discharge



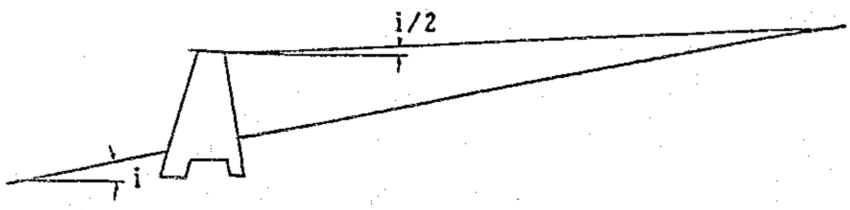
B_1 was assumed to be narrower than the riverbed at the location of the dam. B_2 and h were computed using above relationship. One meter freeboard was included in the design of h.

River	$Q(m^3/s)$	B_1 (m)	B_2 (m)	h (m)
Pesack	1037	34	39.98	5.98
Bantey	1034	49	53.78	4.78
Labey	691	23	28.81	5.81
Besal	3059	40	50.79	10.79

The total sediment retained by the check dam was obtained by considering the existing topography of the river. The gradient of the sediment surface was assumed to be half of the actual river slope.

River	Riverbed Gradient	Gradient of the Sediment Surface
Pesack	1 / 27	1 / 54
Bantey	1 / 23	1 / 46
Labey	1 / 19	1 / 38
Besal	1 / 50	1 / 100

The gradient of the Besal River was estimated using as a basis the gradient of the Bokod river (1/53).



11.3.2. Estimated Construction Cost for Check Dams

In this Study, 14.0 m basic height (from riverbed to design flood water level) was adopted since the dams with less than 15 m height do not have a big influence on the up-and down-stream of the dam. The influence of each check dam on the river will be studied on a case-by-case basis.

In Japan, about 14 m high check dams having the maximum sediment deposit capacity while satisfying above conditions are commonly being constructed. It is also common that sediment ponds of the check dams are filled within 1.5 to 2 years. Therefore, in Japan several check dams are constructed in the same river. Afforestation in the upstream area and protection work against landslides are also considered.

The effects of check dams generally include protection for riverbed scouring by slowing the flow velocity besides trapping of the river sediments. However our study is focusing on the pooling effect of the check dams, because protection against sediment inflow from the river is urgently required for this rehabilitation project.

The construction costs of planned check dams shown below:

Estimated Construction Cost of Check Dam

Unit: US$

Name of Dam	F.C.	L.C.	Total
Pesack	981,800	2,794,600	3,776,400
Bantey	855,500	2,435,000	3,290,500
Labey	523,600	1,490,100	2,013,700
Besal	1,018,100	2,897,700	3,915,800
Total	3,379,000	9,617,400	12,996,400

F.C.: Foreign Currency
L.C.: Local Currency

The Table below shows the unit construction cost per estimated sediment volume (m^3) of each check dam which was calculated based on the above construction cost.

Name of Dam	Pesack	Labey	Bantey	Besal
Construction Cost 10^3/$	3,776.4	2,103.7	3,290.5	3,915.8
Estimated Sediment Volume $10^3 m^3$	1,390.0	1,160.0	217.0	907.0
Unit Cost per Estimated Sediment Volume	2.7	1.7	15.6	4.3

The above unit costs of construction of check dams in three tributaries (Pesack, Labey and Bantey) flowing into the reservoir seem extremely high ranging from $1.70 to $15.60/$m^3$ (average of $3.30/$m^3$).

The cost for sediment countermeasures (dredging work at Intake Tower and its surroundings) among the rehabilitation costs amounts to US$20,168,000 and the reservoir after rehabilitation will stand for 40 years against the sediment discharge of 3,683,000 m^3 per anum.

Therefore, it can be said that the unit cost of $0.14/$m^3$ per sediment volume is reasonable.

Considering the above result, and as stated in the Chapter 12.1.1., since the erosion control in the Agno River should be considered through the whole catchment area, the construction of check dams would be uneconomical. Therefore, it was not adopted in this Study.

11.4. Other Calculations

11.4.1. Calculation of the Size of the Stilling Basin in Case Large Sediment Scour Gates are used.

The design parameters used were as follows:

Number of scour gates	:	2
Width of gate	:	5 m (total 10 m)
Discharge	:	500 m^3/s/gate (total 1,000 m^3/s)
Gradient of scouring channel	:	1/5.714 =

Manning's coefficient of roughness: n = 0.025

The water depth of the scouring channel h_1 was obtained using the uniform flow formula $\sin = n^2Q^2/R^{4/3}A^2$, as follows:

$h_1 = 4.4$ m

Froude number Fr was computed as follows:

$Fr = V/(gh_1)^{1/2} = 22.727/6.566 = 3.461$

The water depth in the stilling basin h_2 was calculated from

$h_2/h_1 = \left\{(1 + 8Fr^2)^{1/2} - 1\right\}/2$:
$h_2 = 4.42 \times 4.4 = 19.45$ or about 20 m

The length of the stilling basin L was obtained from

$L = 4.5\ h_2$, as follows:
$L = 4.5 \times 20 = 90$ m

The water depth in the downstream channel (riverbed) was obtained by assuming the width of the river to be 30 meters, the riverbed gradient 1/1,000, and Manning's n equal to 0.025. The calculated water depth was 7.12 meters.

Thus, the following dimensions were found to be required for the stilling basin:

Length: 90 m

Depth from the present river bed:
13 m (≒ 20 m - 7.12 m)

Width : over 10 m.

11.4.2. For Design of the Intake Tower Improvement

It is important to know the extent to which the reservoir water level will rise in case of a flood which occurs at the beginning of a rainy season.

Using the records of the water level rise during May and June of 1987, the increase in storage which was affected by the water level rise was calculated and followed by calculation of a monthly probability.

Table-11.3 indicates the increase in storage obtained by the highest rise in a month. This was used as input data for probability calculations. As a result, the increase in probable storage for 2 to 20-year return periods was calculated to be as follows:

Frequency	May	June
20 years	$83 \times 10^6 m^3$	$90 \times 10^6 m^3$
10 years	$62 \times 10^6 m^3$	$70 \times 10^6 m^3$
5 years	$40 \times 10^6 m^3$	$49 \times 10^6 m$
2 years	$7 \times 10^6 m^3$	$18 \times 10^6 m^3$

The increase in storage was converted to water level rise for which these corresponding water levels were calculated and shown in the table below provided the reservoir water level before occurrence of the flood was at EL 694 m.

Frequency	May	June
20 years	EL 728.29	EL 729.91
10 years	722.89	725.07
5 years	717.61	719.04
2 years	698.99	705.77

11.4.3. Low Level Outlet

There is an idea to remove silts in the vicinity of the intake by operating the low level outlet. This idea is, however, not discussed in the report for the following reason.

It is too risky to consider from the engineering point of view. A screen is mounted on the intake of the low level outlet, but the actual state of things is that it is covered by piles of silts in some 40 m thickness. When the intake valves are opened under such state, a volume of silts flowing into the intake may crash the rotten screen, and if any broken pieces of the screen hit the valves, particularly the Howell Bunger Valve, and break them down, then it becomes in no way possible to close the valves. The same trouble may take place, if sunken driftwoods run into the intake and are caught in the valves.

The hardness of lower layers of silts is not known, but if they have become hard and solid, it may be necessary to devise a method of manipulating the intake valves to remove silts, though there has been no experience in removing silts in such manner.

There may be another possibility of damaging the tunnel itself. In the process of removing a portion of piled silts, a sliding phenomenon may happen on the surrounding layers of silts, and this may cause the intake to clog momentarily. Should this situation happen, the internal pressure of the tunnel would suddenly turn to negative, and this water hammering phenomenon may damage the tunnel.

As mentioned, the idea is charged with a very high risk of damages on the intake valves, and any discussion on the idea may not be considered necessary.

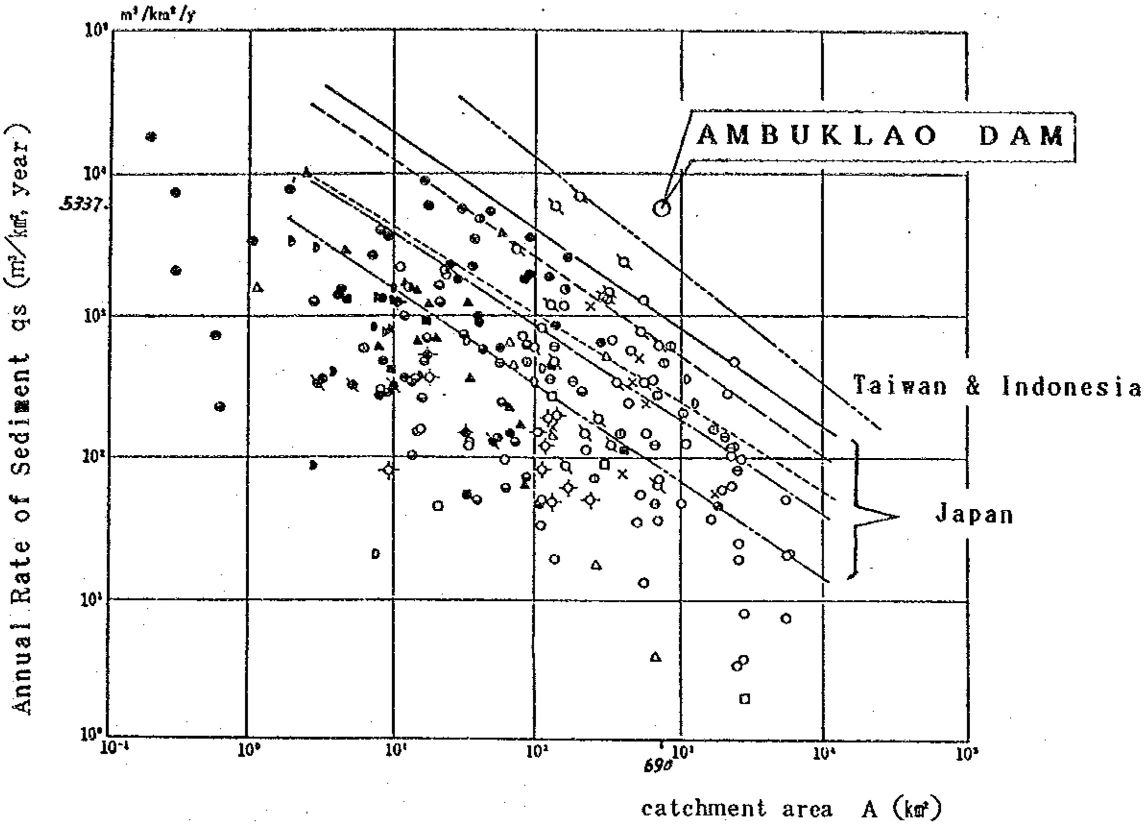
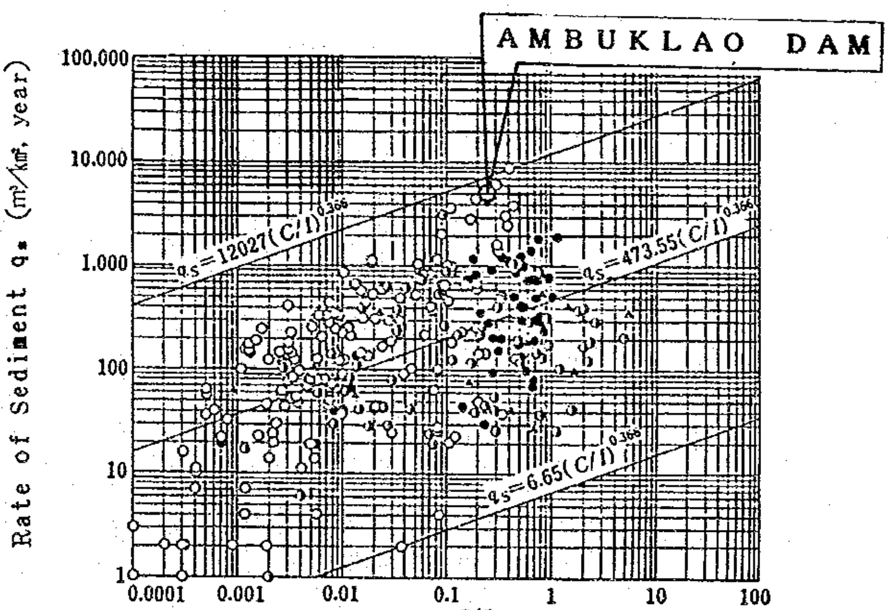


Fig — 11.1 Relationship between Annual Rate of Sediment and Catchment Area



$q_s = 12027(C/I)^{0.366}$

$q_s = 473.55(C/I)^{0.366}$

$q_s = 6.65(C/I)^{0.366}$

Fig — 11.2 Relationship between C/I Ratio and Sediment Rate q_s

C : Total Capacity of Reservoir

I : Annual Average of Inflow

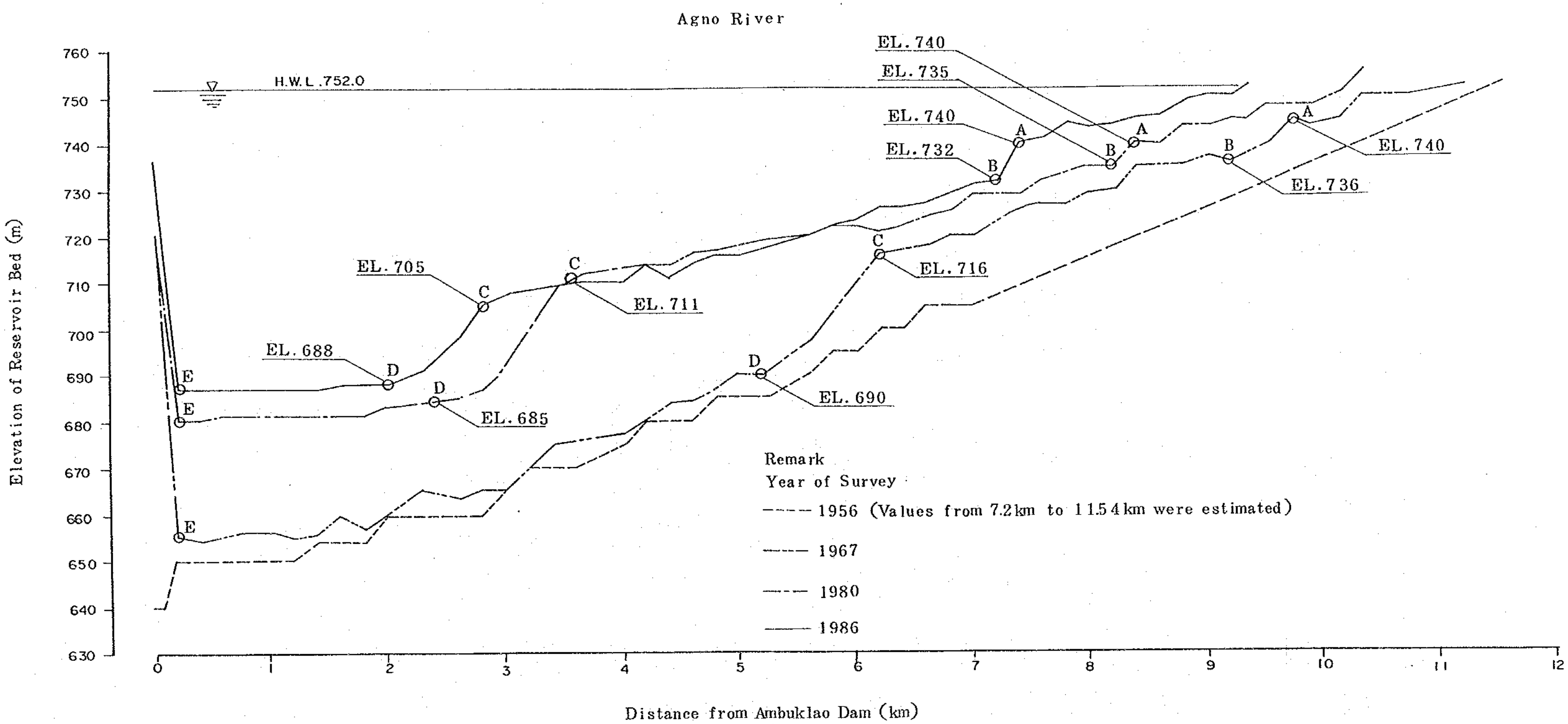


Fig - 11.3 Profile of The Reservoir Bed

Bokod River

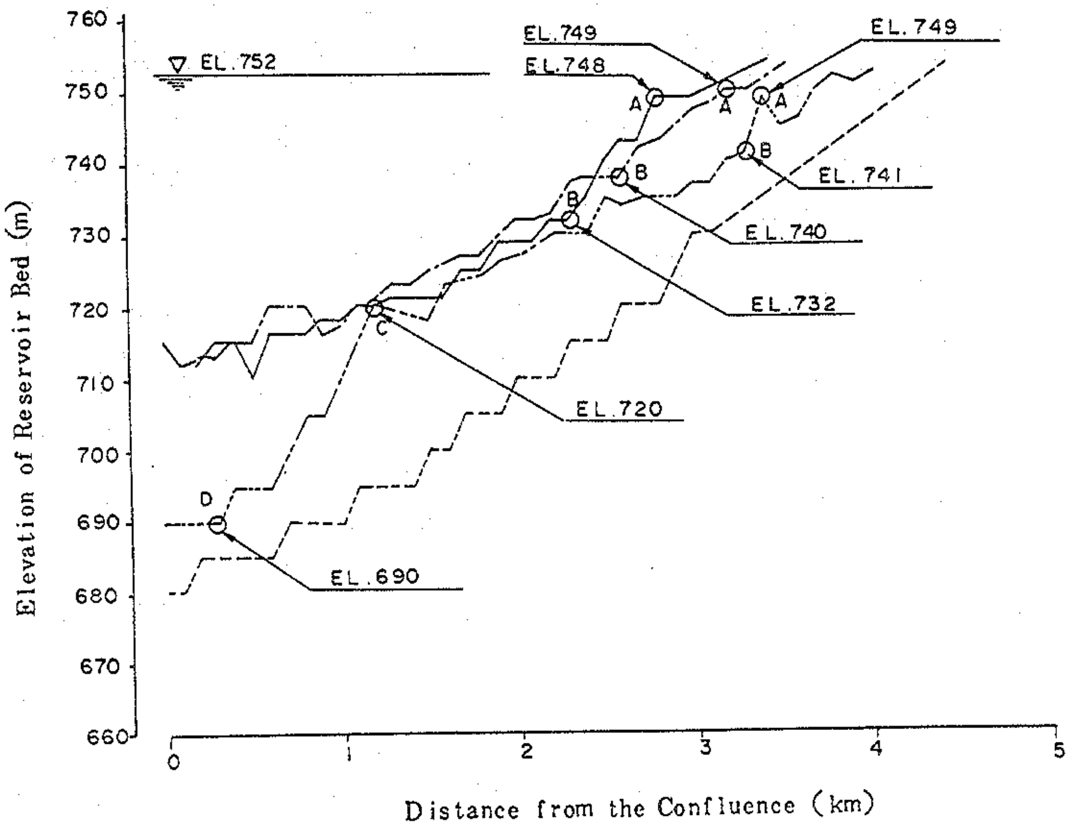


Fig — 11.4 Profile of The Reservoir Bed

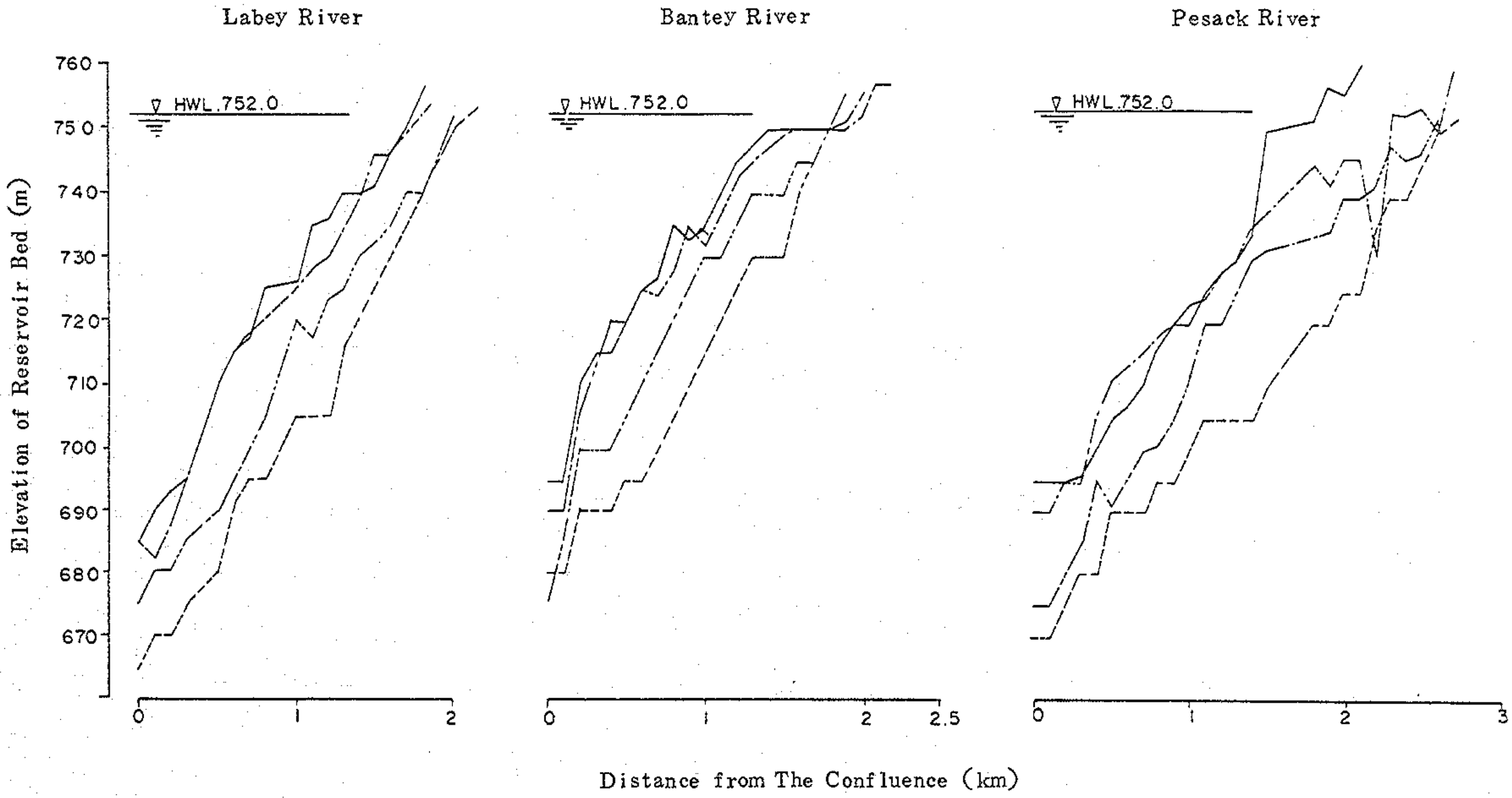


Fig — 11.5 Profile of The Reservoir Bed

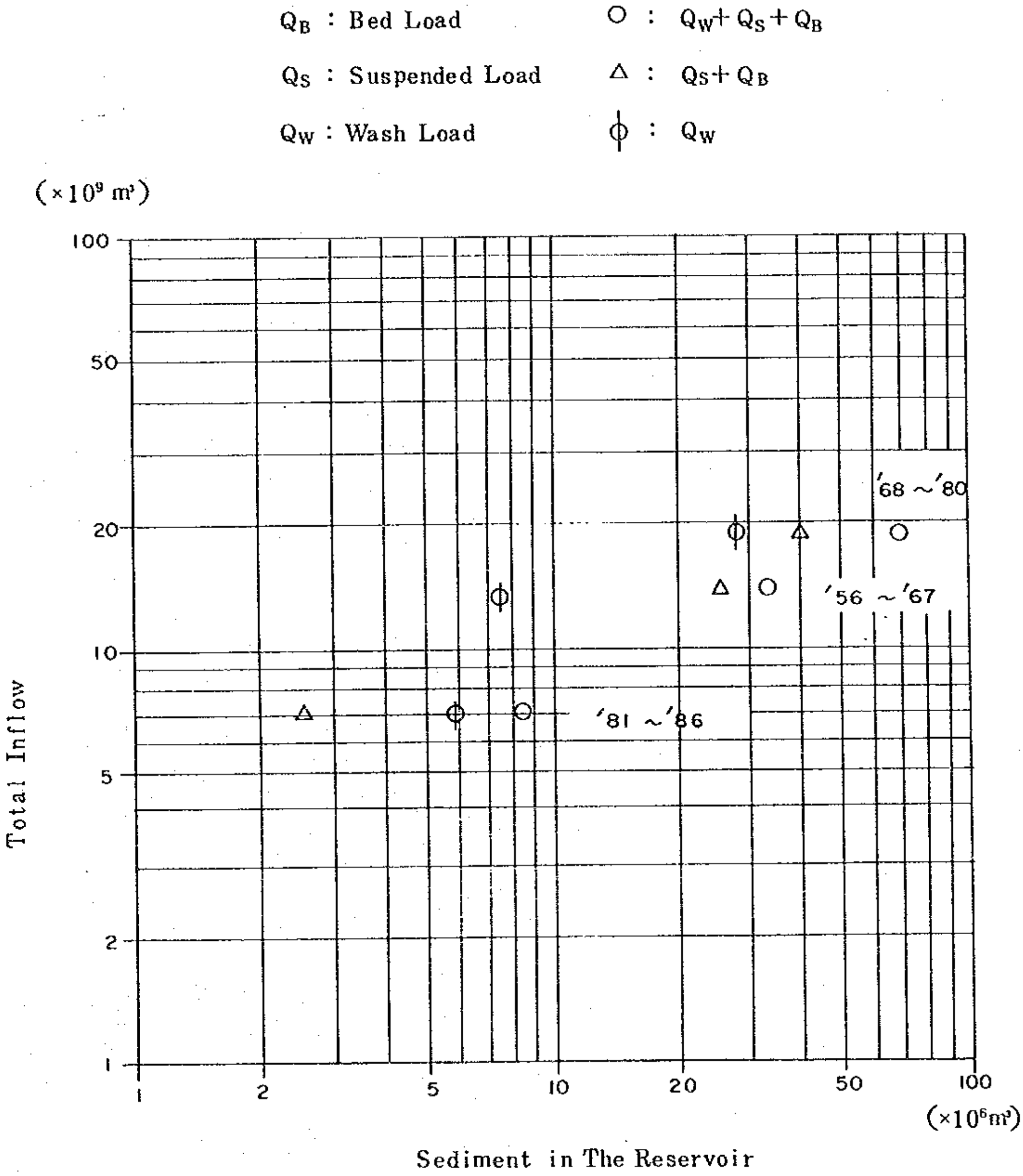
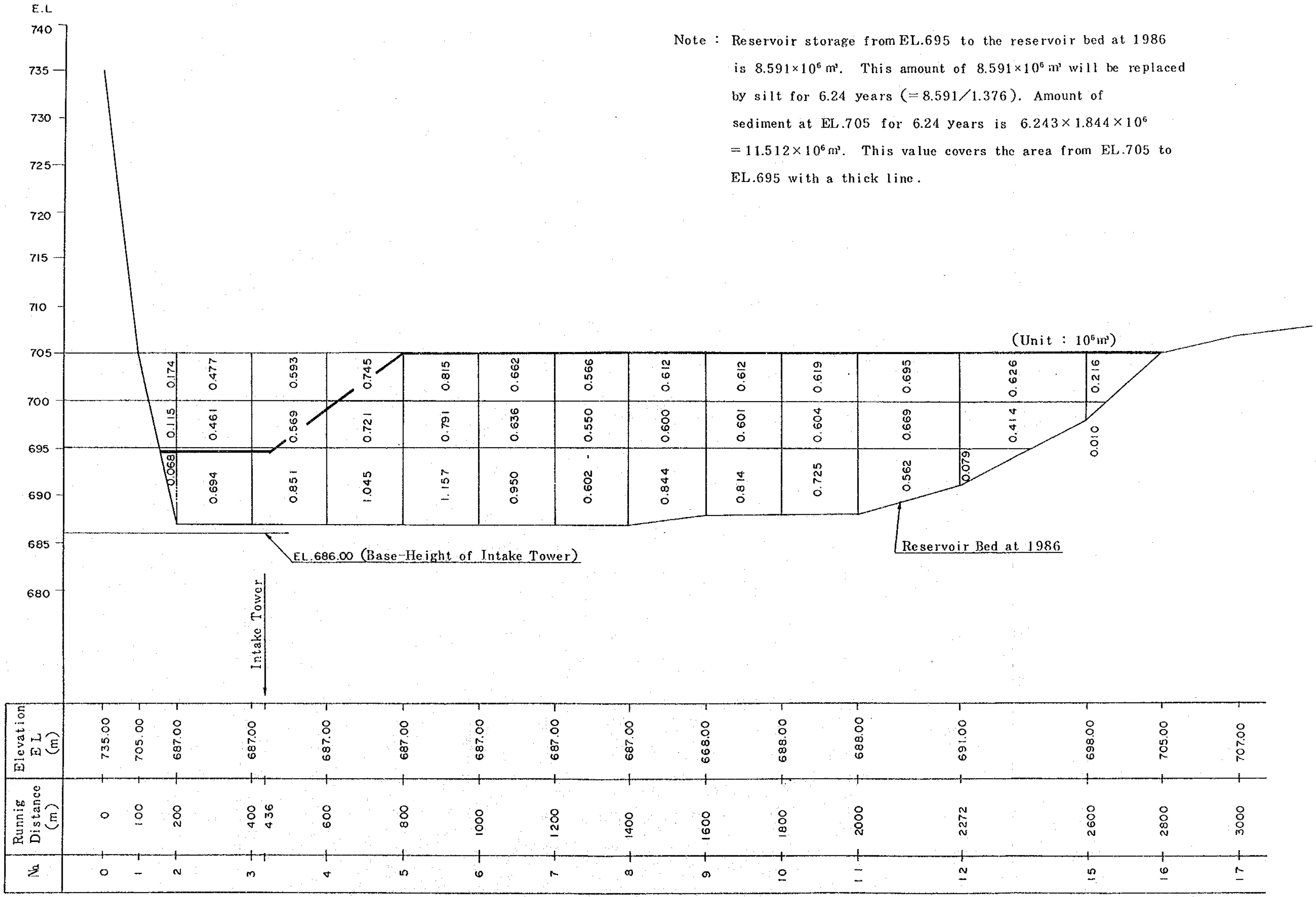


Fig — 11.6 Relation between Total Inflow and Sediment in The Reservoir at The Period of 1956 - 1967, 1968 - 1980 and 1981 - 1986.

Fig − 11.7 Reservoir Storage by Each Section and Elevation used for Estimation of Progressive Sedimentation after 1968



Note : Reservoir storage from EL.695 to the reservoir bed at 1986 is $8.591 \times 10^6 m^3$. This amount of $8.591 \times 10^6 m^3$ will be replaced by silt for 6.24 years ($= 8.591/1.376$). Amount of sediment at EL.705 for 6.24 years is $6.243 \times 1.844 \times 10^6 = 11.512 \times 10^6 m^3$. This value covers the area from EL.705 to EL.695 with a thick line.

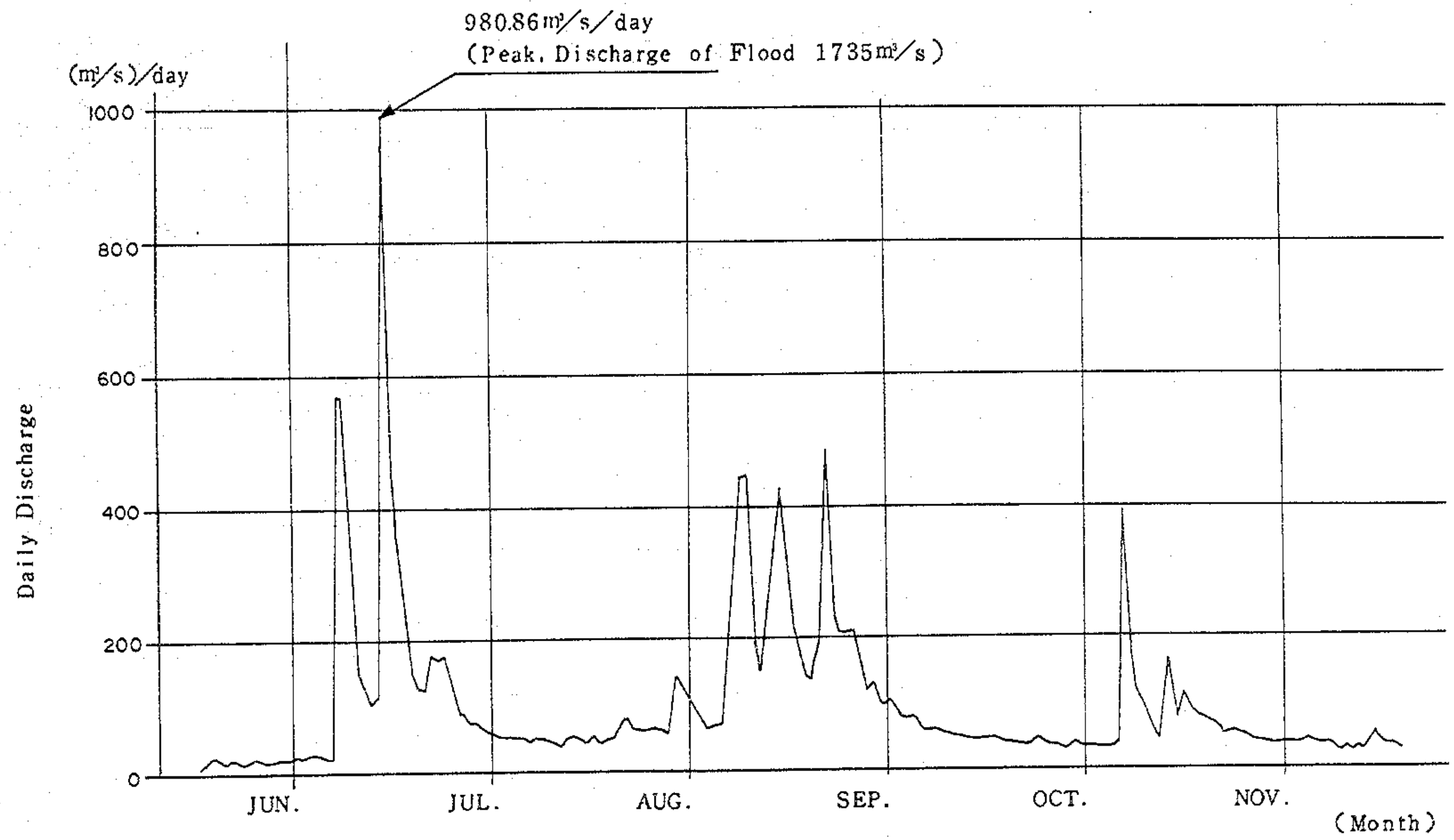


Fig-11.8 Daily Disharge Record (1985)

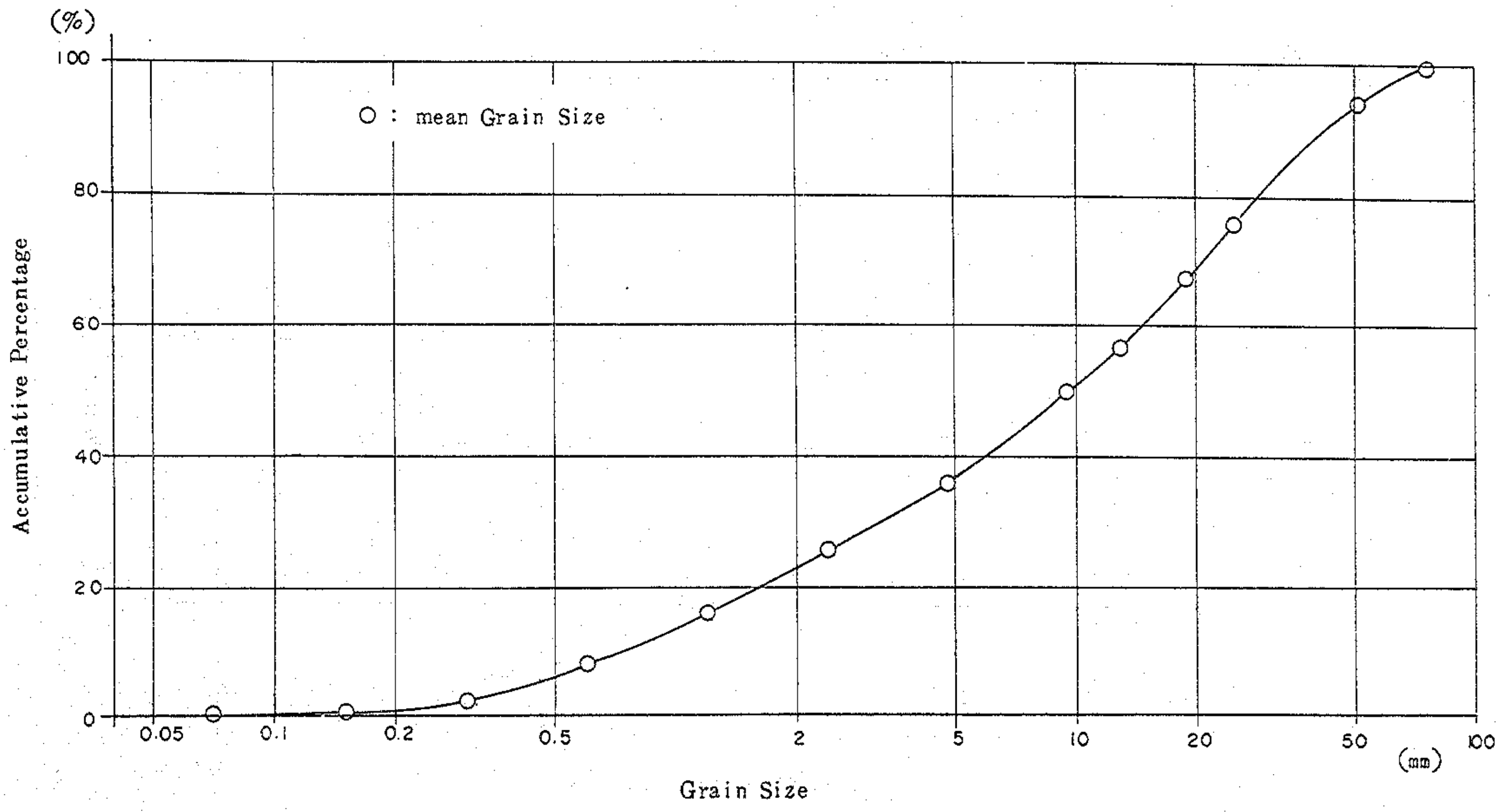


Fig — 11.9 Grain-Size Distribution Curve

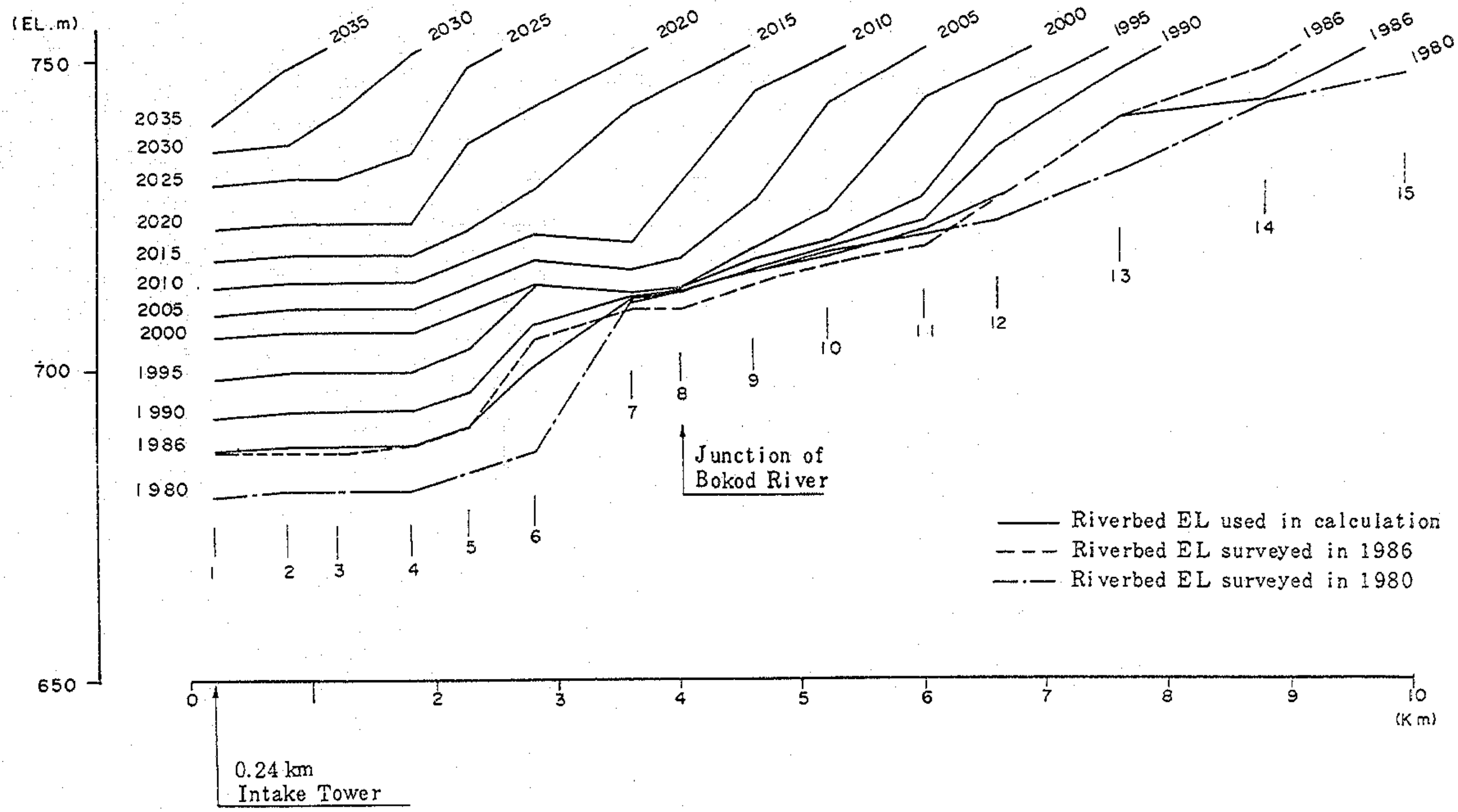


Fig — 11.10(1) Riverbed Profile by Simulation for Forecast of Sedimentation Progress in The Reservoir (Agno River)

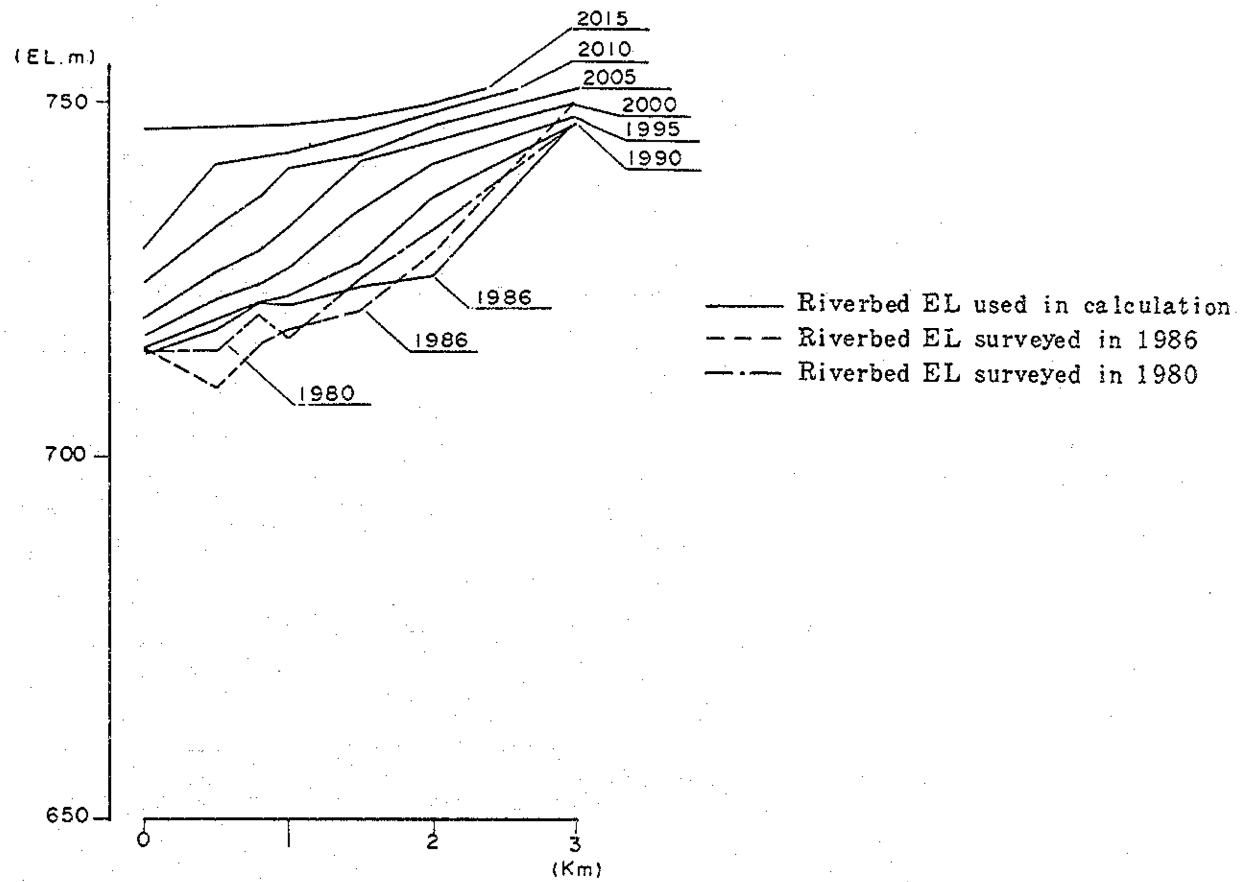


Fig − 11.10(2) Riverbed Profile by Simulation for Forecast of Sedimentaion Progress in The Reservoir (Bokod River)

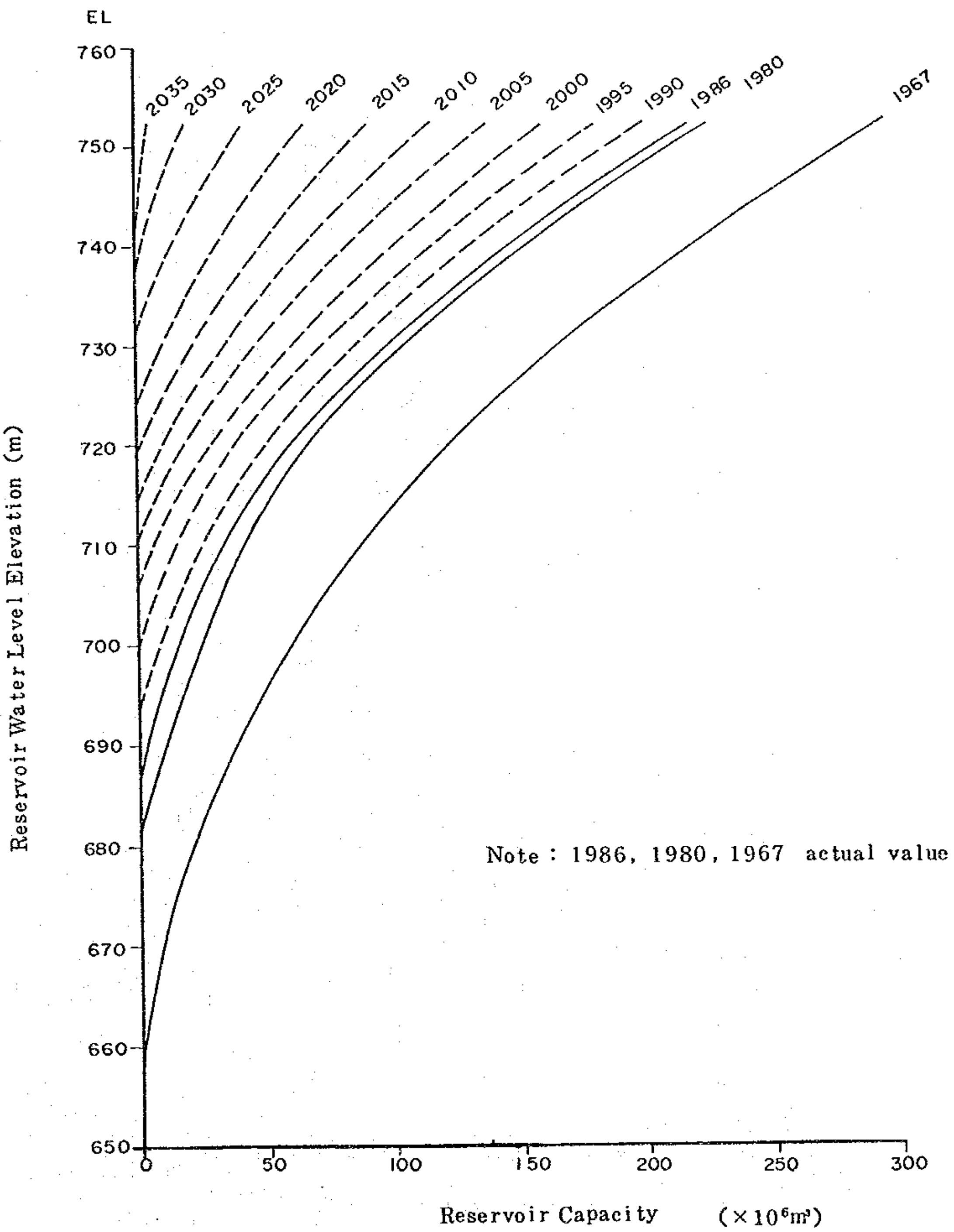


Fig — 11.10(3) Relation of Future Reservoir Capacity and Water Level by Sediment Progress Simulation

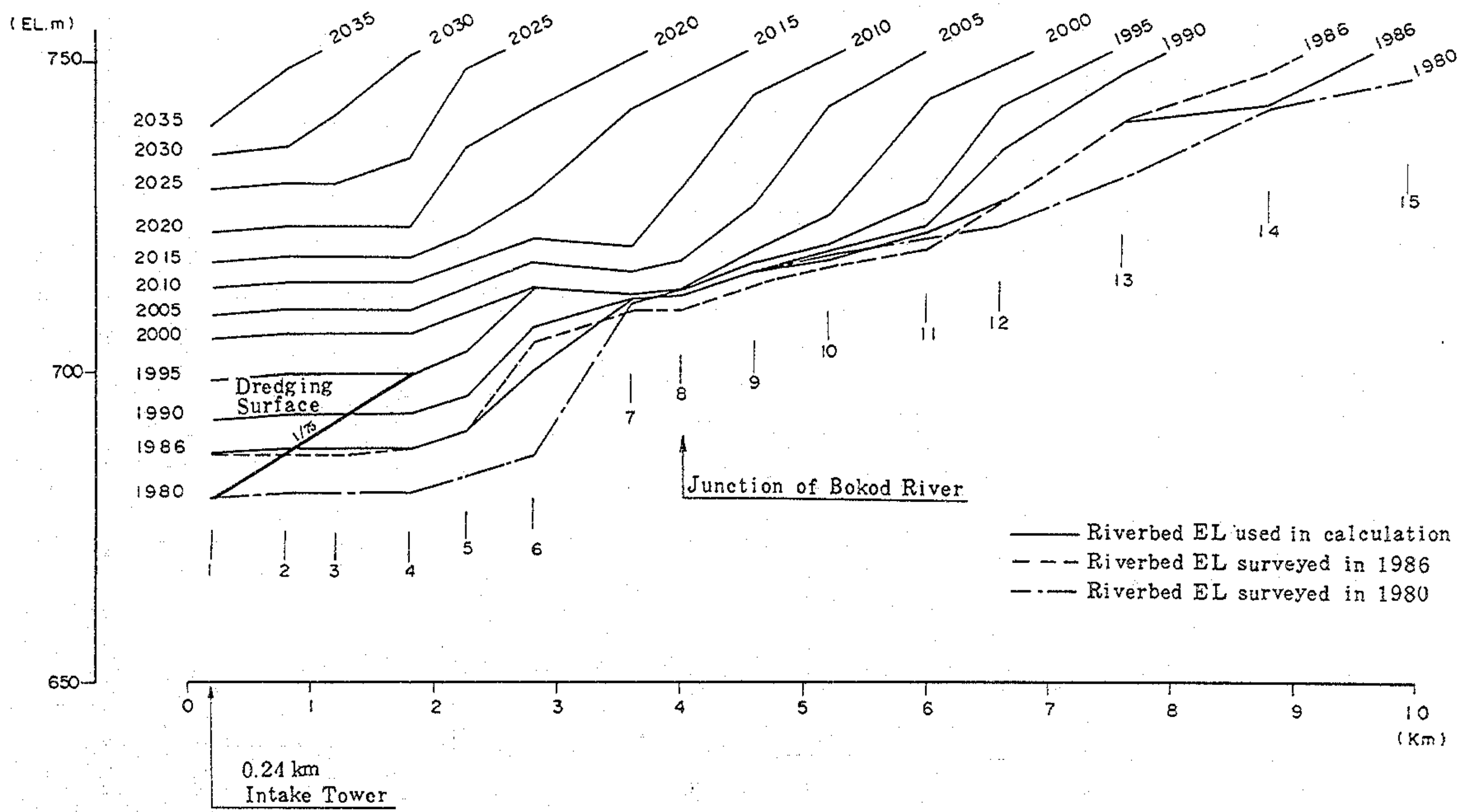


Fig − 11.10(4) Riverbed Profile in Case of Dredging with 1/75 Gradient for Simulation of Sedimentation Progress Forecast

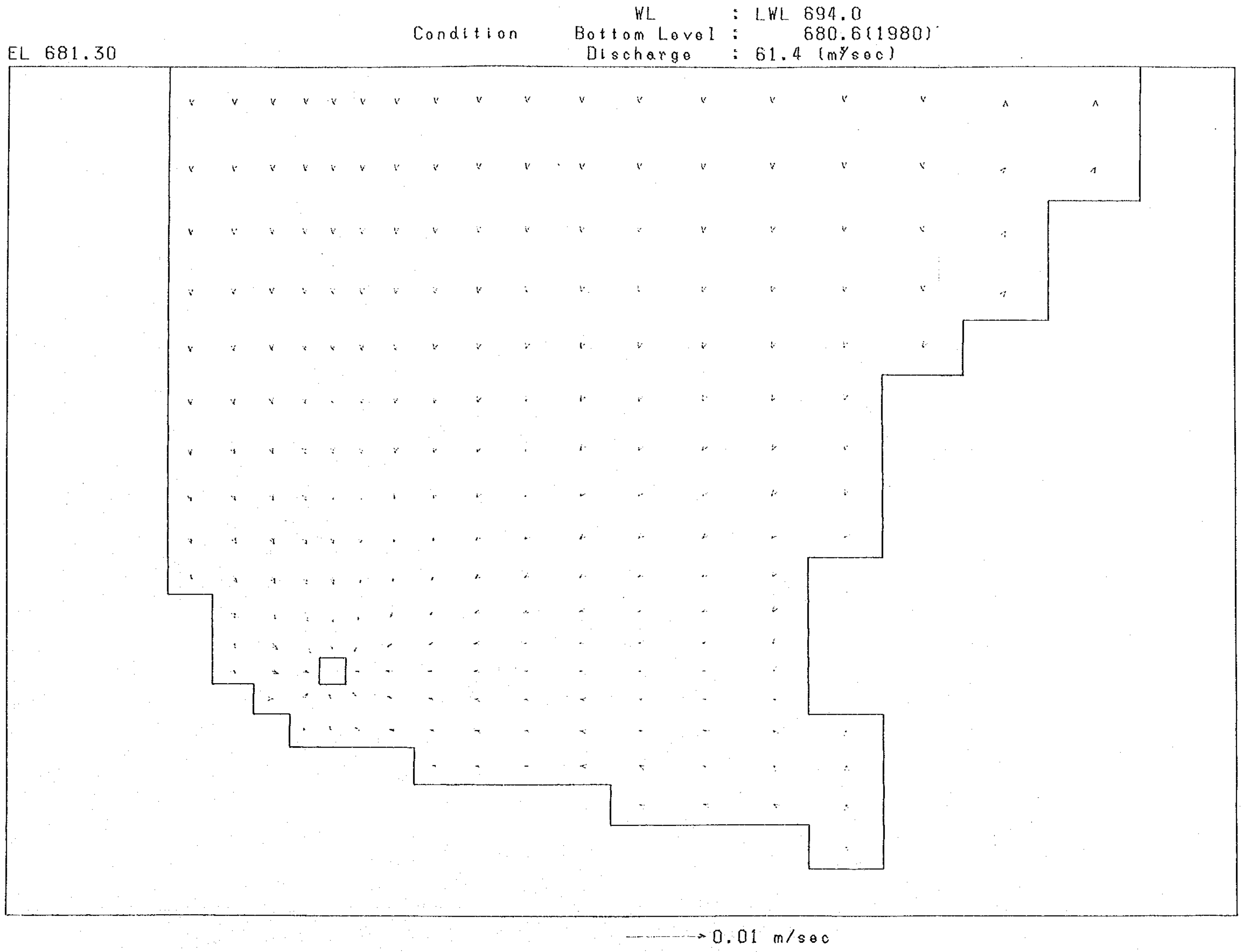


Fig — 11.11 Flow Pattern in X—Y Plane (EL.681.3)
(Near Field of Intake Tower)

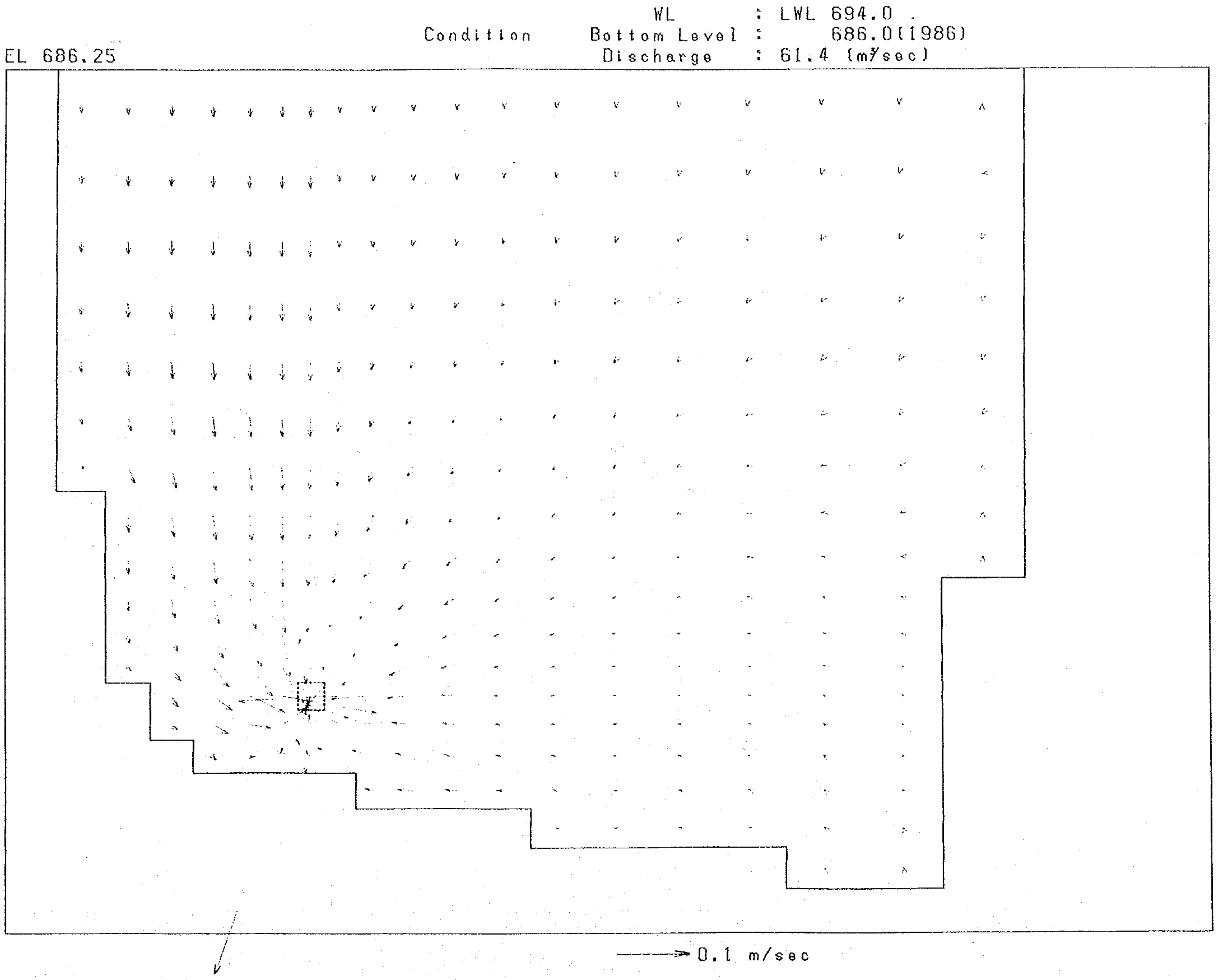


Fig — 11.12 Flow pattern in X—Y plane (EL.686.25)
(Near Field of Intake Tower)

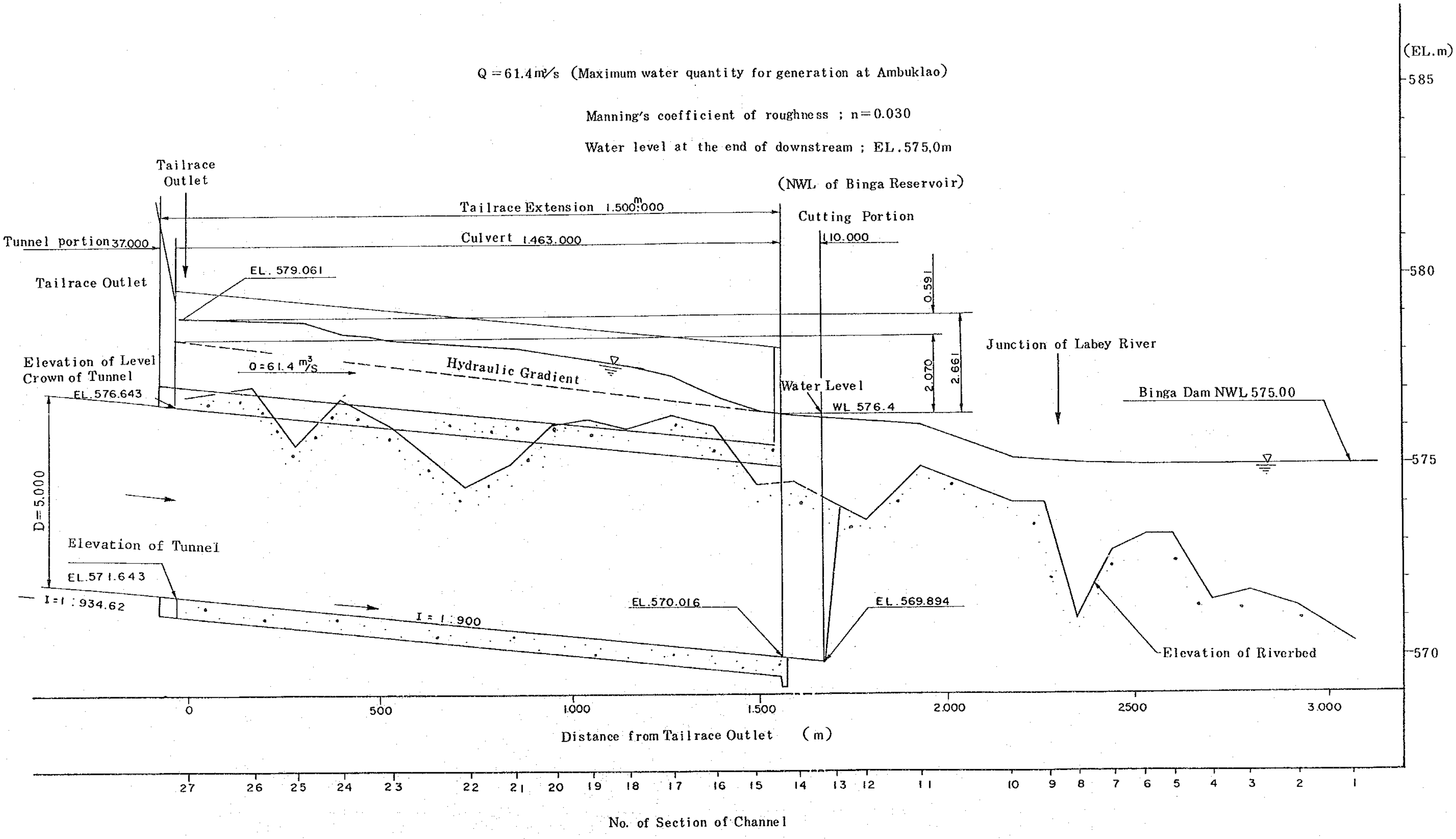


Fig — 11.13 Water Surface Profile at Ambuklao Tailrace Outlet & Tailrace Extension Plan

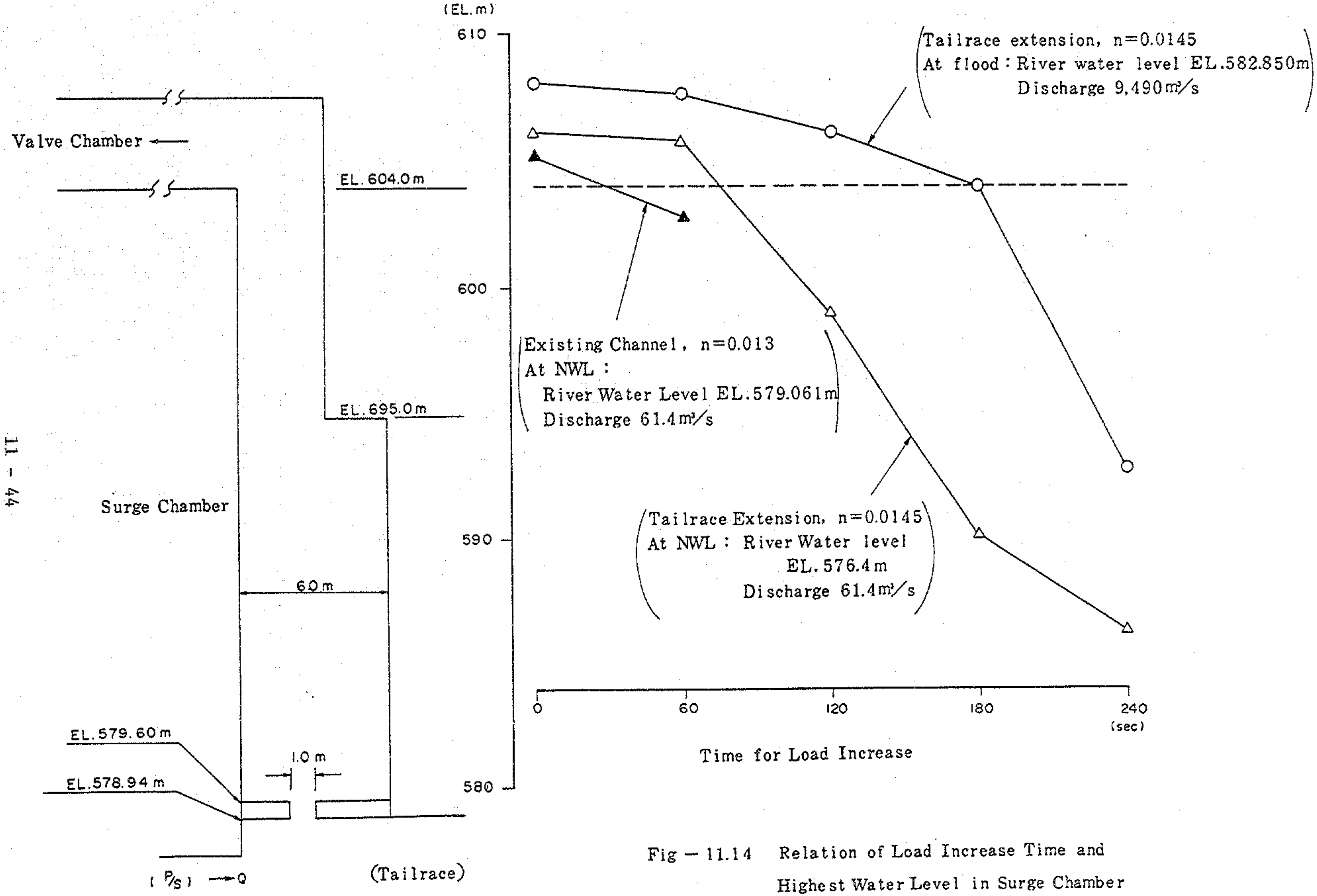
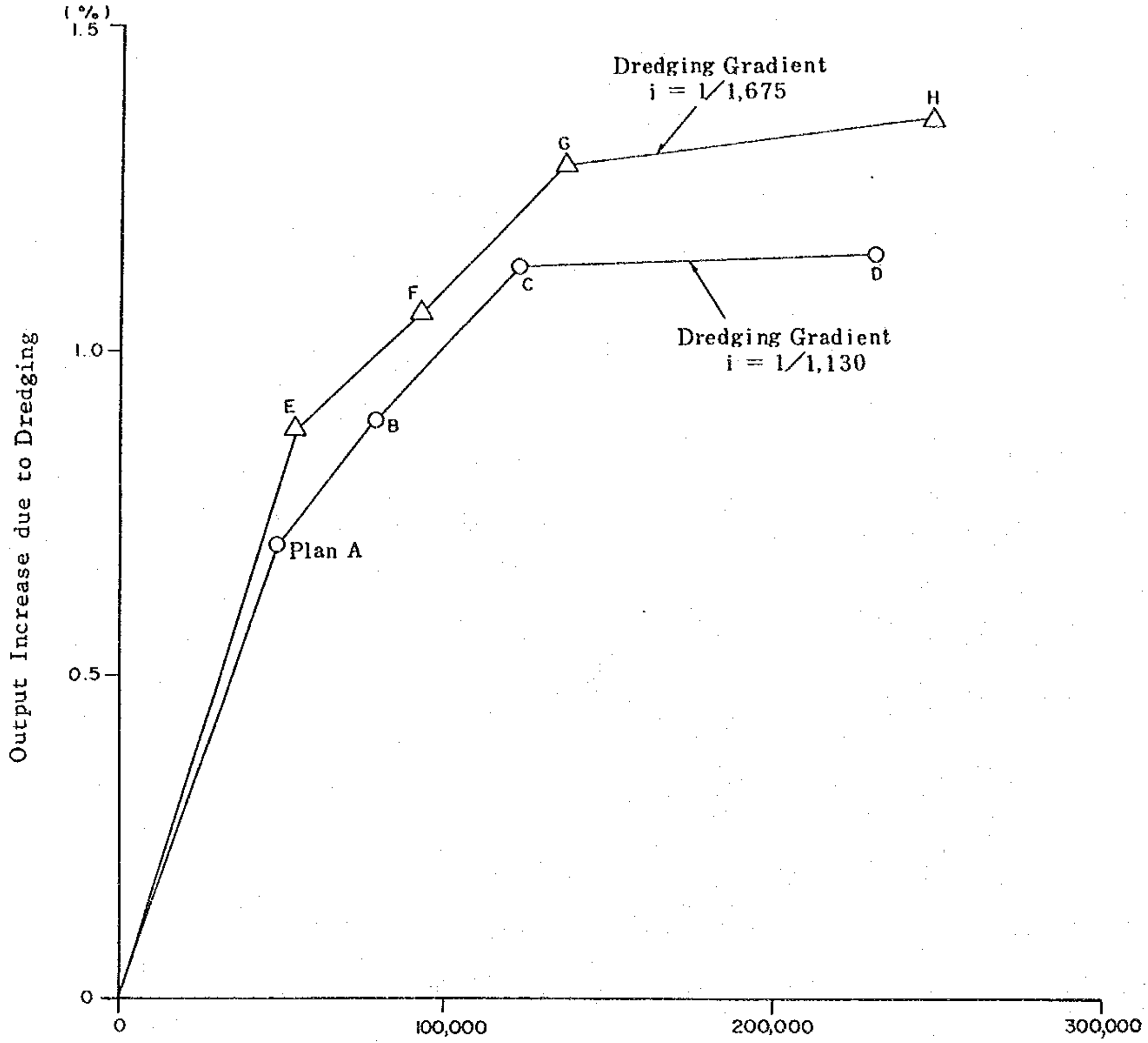


Fig — 11.14 Relation of Load Increase Time and Highest Water Level in Surge Chamber



Dredged Sediment from Tailrace Outlet
to 1500m Downstream (m³)

Fig — 11.15 Relation of Dredged Sediment Volume and Output Increase in Respective Plans

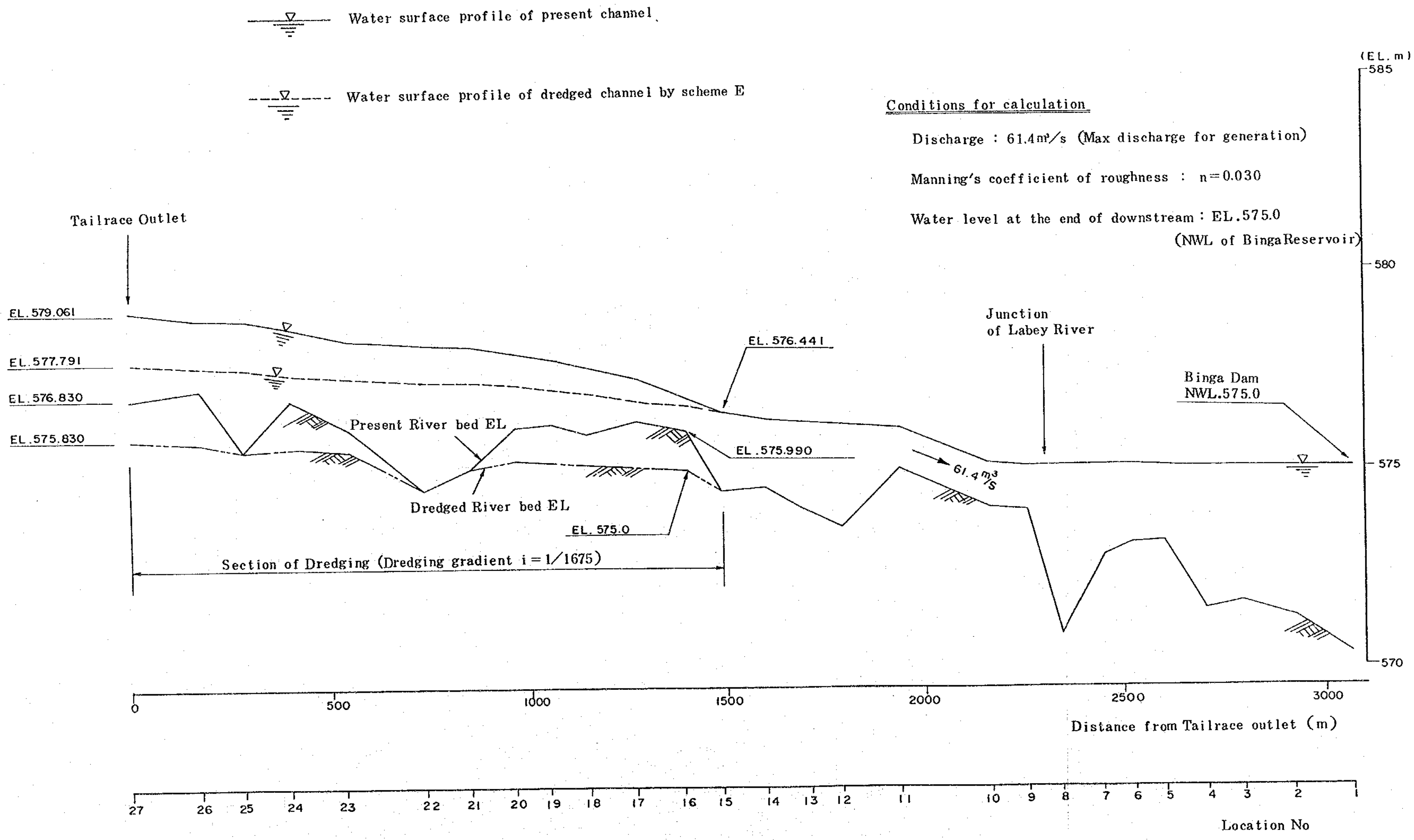


Fig — 11.16 Comparison in Water Surface Profile of Present Downstream Channel with Dredged Channel by Scheme E

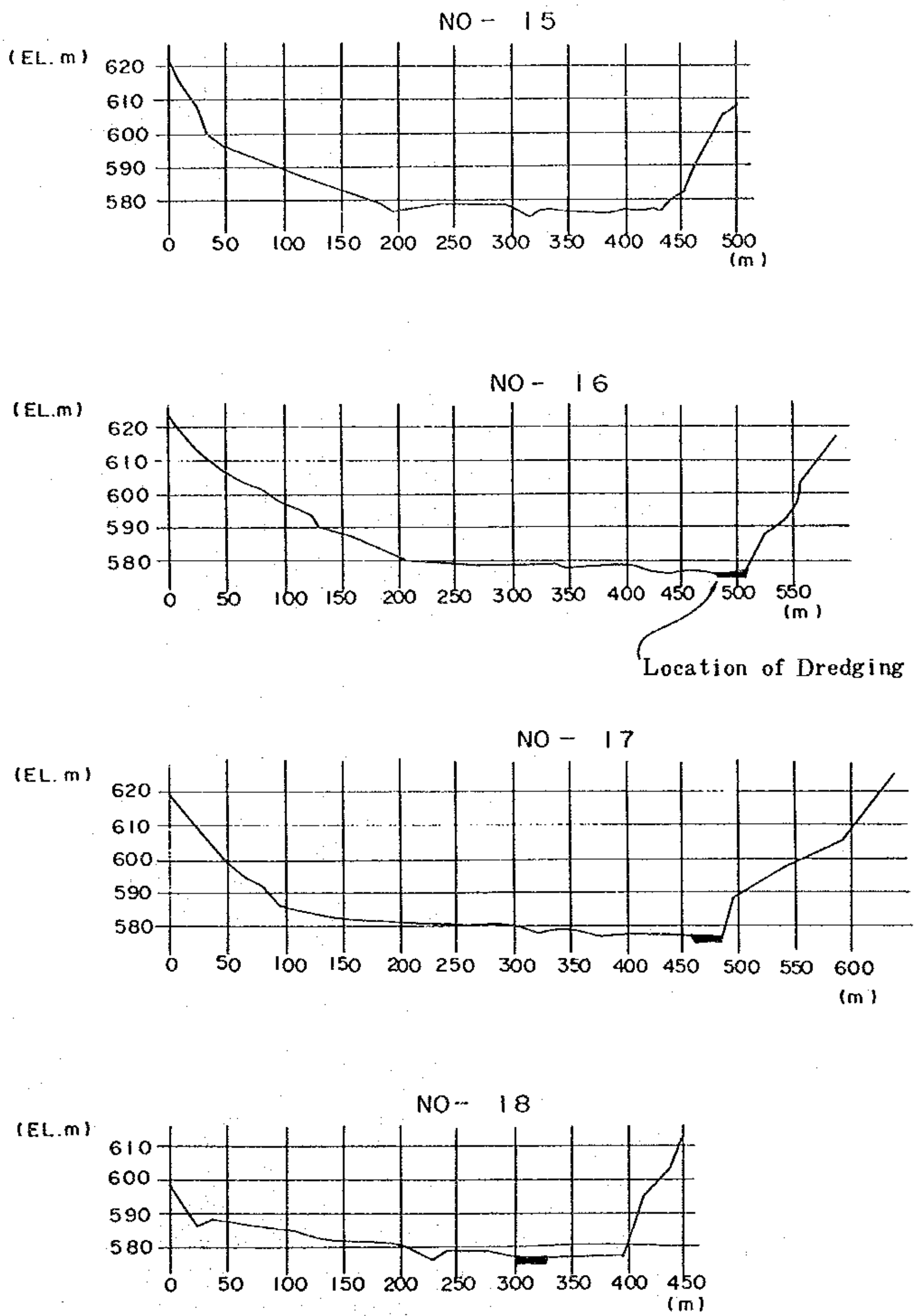
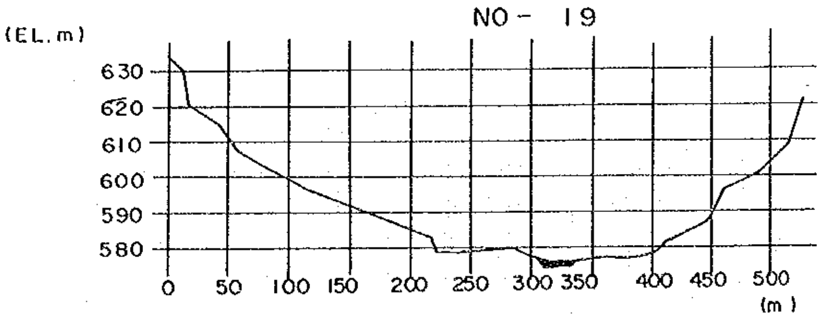
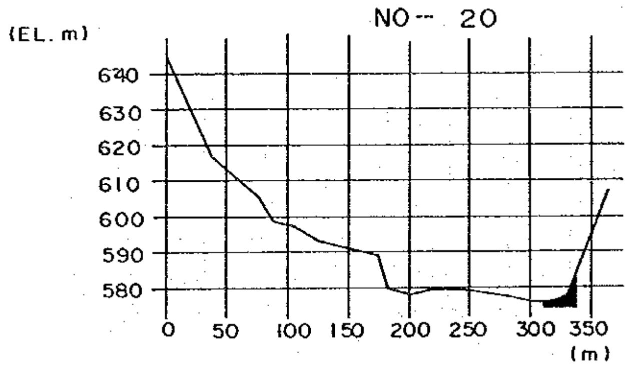
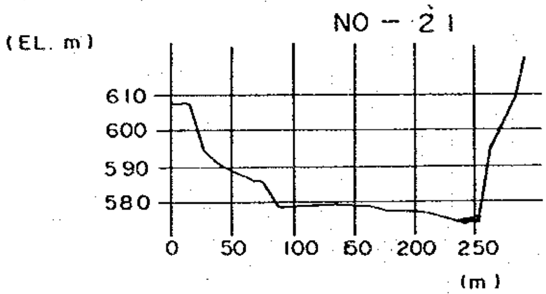


Fig − 11.17(1) Cross Section of The River in Dredging Scheme E and Location
(Section No is same as location No in Fig − 11.16)







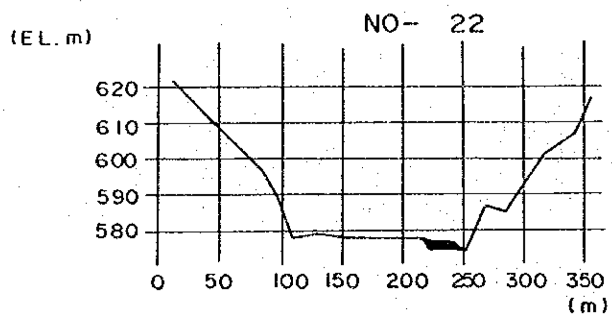
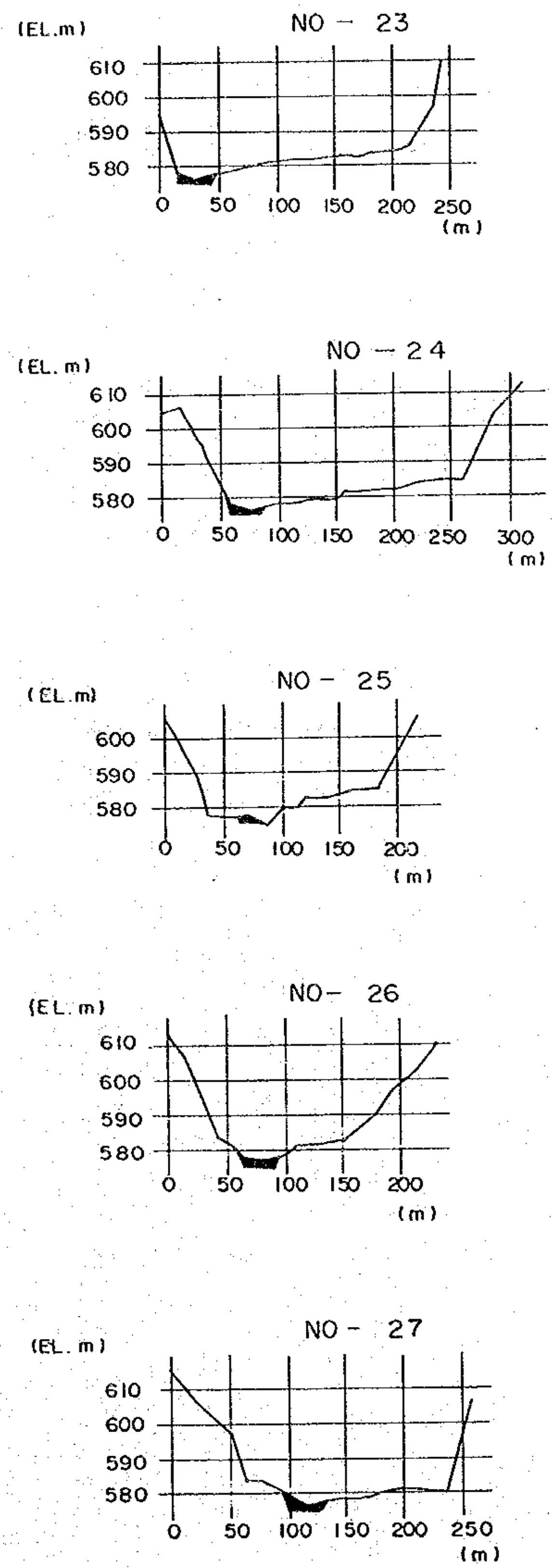


Fig — 11.17(2) Cross Section of The River in Dredging Scheme E and Location
(Section No is same as Location No in Fig — 11.16)



Tailrace Outlet

Fig − 11.17(3) Cross Section of The River in Dredging Scheme E and Location
(Section No is same as Location No in Fig − 11.16)

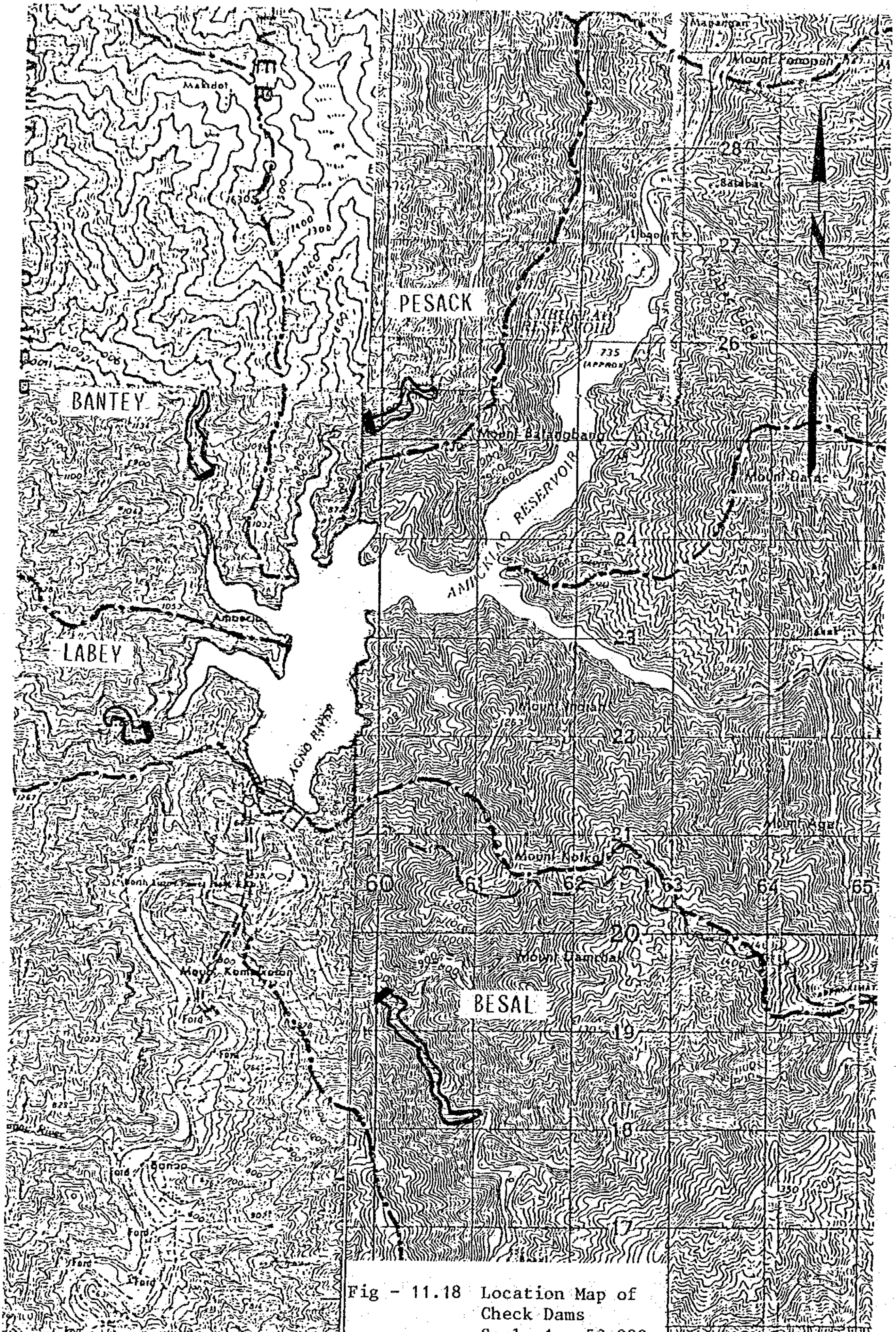


Fig - 11.18 Location Map of Check Dams
Scale 1 : 50,000

Table − 11.1 Volume of Sediment Deposited at Ambuklao Reservoir

<table>
<tr><th>Year</th><th>Agno River</th><th>Bokod River</th><th>Labey River</th><th>Bantey River</th><th>Pesack River</th><th colspan="2">Total Volume</th></tr>
<tr><td rowspan="3">1957
~
1967</td><td>10.015×10^6 m^3
(EL752~736)</td><td>0.233×10^6 m^3
(EL752~741)</td><td rowspan="2">1.193×10^6 m^3</td><td rowspan="2">0.618×10^6 m^3</td><td rowspan="2">1.86×10^6 m^3</td><td rowspan="2">Gravel m^3
25.632×10^6</td><td rowspan="3">Total m^3
33.09×10^6</td></tr>
<tr><td>9.043×10^6
(EL736~690)</td><td>2.67×10^6
(EL741~695)</td></tr>
<tr><td>7.132×10^6
(EL690~)</td><td>0.326×10^6
(EL695~)</td><td></td><td></td><td></td><td>Silt
7.458×10^6</td></tr>
<tr><td rowspan="3">1968
~
1980</td><td>2.77×10^6
(EL752~735)</td><td rowspan="2">1.192×10^6
(EL752~738)</td><td rowspan="2">2.895×10^6</td><td rowspan="2">1.829×10^6</td><td rowspan="2">1.648×10^6</td><td rowspan="2">Gravel
41.037×10^6</td><td rowspan="3">Total
68.991×10^6</td></tr>
<tr><td>30.703×10^6
(EL735~685)</td></tr>
<tr><td>24.476×10^6
(EL685~)</td><td>3.478×10^6
(EL738~)</td><td></td><td></td><td></td><td>Silt
27.954×10^6</td></tr>
<tr><td rowspan="3">1981
~
1986</td><td>1.098×10^6
(EL752~732)</td><td rowspan="2">-0.032×10^6
(EL752~738)</td><td rowspan="2">0.048×10^6</td><td rowspan="2">-0.153×10^6</td><td rowspan="2">0.161×10^6</td><td rowspan="2">Gravel
2.532×10^6</td><td rowspan="3">Total
8.394×10^6</td></tr>
<tr><td>1.41×10^6
(EL732~688)</td></tr>
<tr><td>6.151×10^6
(EL688~)</td><td>-0.289×10^6
(EL738~)</td><td></td><td></td><td></td><td>Silt
5.862×10^6</td></tr>
<tr><td rowspan="2">1957
~
1986</td><td colspan="5" rowspan="2"></td><td>Gravel
69.201×10^6</td><td rowspan="2">Total
110.475×10^6</td></tr>
<tr><td>Silt
41.274×10^6</td></tr>
</table>

Table − 11.2 Estimation of Reservoir Capacity / Water Level by Sediment Simulation (Ambuklao Dam)

Reservoir Water Level (EL. m)	Reservoir Capacity ($\times 10^6$ m³)											
	Year 1980	1986	1990	1995	2000	2005	2010	2015	2020	2025	2030	2035
752	225.09	216.70	199.08	180.00	159.39	137.00	115.16	90.90	65.57	41.01	18.59	5.01
750	212.30	204.07	187.10	168.60	148.75	126.77	105.74	82.57	59.06	35.46	14.97	3.20
745	182.13	174.24	158.68	141.65	123.81	103.30	84.45	64.20	43.99	24.04	7.65	0.90
740	154.26	146.82	132.32	116.87	100.98	82.44	66.05	48.68	30.55	14.02	1.80	0.00at EL.740
735	128.40	121.31	108.24	94.04	80.04	63.55	49.53	34.57	19.17	5.11	0.06at EL.737	
730	104.73	97.78	86.55	73.14	60.90	46.55	34.59	21.92	9.91	0.23at EL.731		
725	83.53	77.14	66.92	54.22	43.58	31.41	20.96	10.80	1.82			
720	65.79	59.49	49.78	37.99	28.41	18.22	9.18	1.82	0.01at EL.723			
715	52.15	44.79	36.09	24.91	15.80	7.33	1.00	0.01at EL.718				
710	41.43	33.31	25.17	15.08	6.31	0.13at EL.710	0.08at EL.714					
705	32.19	24.09	16.22	6.99	0.83at EL.706							
700	23.72	15.94	8.38	0.32								
695	16.23	8.71	1.64	0.00at EL.699								
690	9.36	2.66	0.02at EL.693									
685	3.44	0.00at EL686										
681	0.10											
Note	Actual Record (Calculated from topo map)		1986 ~ 2035 : Estimation from Riverbed Movement Simulation									

Table – 11.3 Highest Water Level and Increase in Storage in May and June at Ambuklao Reservoir

Year	May					June				
	Highest Water Level		Storage at Left WL		Storage Increase (= $V_2 - V_1$) ($\times 10^6$ m³)	Highest Water Level		Storage at Left WL		Storage Increase (= $V_2 - V_1$) ($\times 10^6$ m³)
	W_1 (EL.m)	W_2 (EL.m)	V_1 ($\times 10^6$ m³)	V_2 ($\times 10^6$ m³)		W_1 (EL.m)	W_2 (EL.m)	V_1 ($\times 10^6$ m³)	V_2 ($\times 10^6$ m³)	
1957	–	–	–	–	–	723.90	730.00	73.060	97.777	24.717
1958	706.60	707.33	26.853	28.162	1.309	706.07	717.10	25.913	50.566	24.653
1959	717.04	717.65	50.389	52.186	1.797	707.45	708.40	28.379	30.141	1.762
1960	–	–	–	–	–	735.00	735.78	121.310	125.194	3.884
1961	721.30	722.12	63.860	66.693	2.833	720.22	729.53	60.220	95.721	35.501
1962	–	–	–	–	–	710.82	711.50	35.079	36.566	1.487
1963	710.61	710.81	34.626	35.057	0.431	709.34	729.60	31.974	96.027	64.053
1964	710.72	711.71	34.863	37.027	2.164	711.81	723.82	37.247	72.770	35.523
1965	721.51	724.98	64.583	77.068	12.485	723.68	725.53	72.263	79.185	6.922
1966	713.55	736.89	41.225	130.785	89.560	737.14	737.30	132.054	132.870	0.816
1967	–	–	–	–	–	709.34	741.98	31.974	157.402	125.428
1968	702.20	702.32	19.425	19.621	0.196	696.75	698.76	11.148	14.069	2.921
1969	–	–	–	–	–	712.80	714.02	39.474	42.337	2.863
1970	–	–	–	–	–	710.96	719.53	35.381	57.981	22.600
1971	709.14	718.60	31.569	55.058	23.489	700.78	711.92	17.156	37.488	20.332
1972	717.95	721.69	53.070	65.202	12.132	719.33	723.00	57.342	69.799	12.457
1973	705.09	706.03	24.240	25.842	1.602	703.20	704.16	21.062	22.663	1.601
1974	708.71	709.10	30.732	31.488	0.756	707.56	725.05	28.579	77.335	48.756
1975	722.97	724.51	69.693	75.319	5.626	723.84	724.72	72.842	76.100	3.258
1976	704.89	752.23*	23.901	218.156	194.255	751.37	752.17*	212.712	217.776	5.064
1977	702.82	704.99	20.436	24.070	3.634	702.04	702.97	19.164	20.681	1.517
1978	–	–	–	–	–	708.79	712.82	30.885	39.520	8.635
1979	719.93	724.50	59.261	75.282	16.021	724.70	726.14	76.026	81.559	5.533
1980	716.28	718.52	48.231	54.813	6.582	701.75	702.65	18.700	20.159	1.459
1981	734.91	735.12	120.869	121.908	1.039	722.92	740.54	69.517	149.687	80.170
1982	708.45	709.48	30.236	32.258	2.022	699.95	703.51	15.859	21.578	5.719
1983	718.09	718.89	53.493	55.948	2.455	702.10	704.95	19.262	24.002	4.740
1984	727.88	731.66	88.651	105.329	16.678	719.35	721.15	57.406	63.344	5.938
1985	714.90	715.10	44.542	45.057	0.515	702.36	743.45	19.686	165.472	145.786
1986	709.67	710.90	32.642	35.251	2.609	696.64	698.58	10.992	13.807	2.815

Note. *:Overflow from Spillway.　　　– : No water level increase.

12. Rehabilitation Program

12. Rehabilitation Program

Based on the studies described in the preceding Chapters, such as safety of structures and sedimentation, it was concluded that the rehabilitation of the following four items was urgently required:

(1) Changes to the intake structure as required to prevent sediment inflow and maintain the operation of the power-plant at maximum efficiency.
(2) Measures against failure of the upstream face of the dam.
(3) Measures against sedimentation at the tailrace outlet.
(4) Repair or replacement of the turbine inlet valves.

For the intake structure, item (1) above, several alternatives were studied and compared with each other. Economic analysis was made as required to facilitate the selection of the optimum solution.

In the following Sections the results of the studies made in connection with the above are described.

The execution of the rehabilitation work would be possible to commence, at the earliest, in Autumn 1992. Until then, sediment accumulation in the reservoir will continue. Therefore, dredging around Intake Tower should be started immediately, so that there should be no sediment inflow into the powerplant which causes serious problems with regard to maintenance and operation of the units.

12.1. Rehabilitation Planning

12.1.1. Reservoir Sedimentation

As discussed in Chapter 11, if the sedimentation process is not dealt with immediately, the flow into the intake tower will gradually be reduced and, in several years, be stopped completely. As a result, the powerplant will stop producing energy.

The function of the dam and the reservoir are expected to continue for at least another 40 years, despite the inflow of sediment into the reservoir.

Therefore, to maintain the function of the powerplant, the following possibilities were studied:

- Eliminate or slow down the sedimentation process, or
- Defend the intake tower against the sediments, or
- Modify or provide a new intake tower as required for continued operation despite the progress of sedimentation.

In studying the above, five rehabilitation schemes were developed and compared. They were as follows:

<u>Scheme</u>	<u>Description</u>
A	Removal of sediments by big dredging boat.
B	Heightening of the existing intake tower.
C	Provision of a large capacity sediment removal facility.
D	Provision of a new intake tower (inclined type).
E	Provision of a new intake tower (vertical type).

As additional measure, erosion control and flood improvement of the river basin should be planned to prevent sediment inflow into the reservoir. However, this type of large undertakings cannot be implemented only by a power corporation, but are regarded to be a national enterprise. Therefore, the erosion control and flood improvements were, in this study, limited only to the rivers (tributaries) which were regarded to have an influence on the operation of the intake tower.

The outlines and specifications of the said five schemes are summarized in Table-12.1.

The main characteristics and the associated problems of the above five schemes are as follows:

Schemes	Main Characteristics	Problems
A	Reservoir capacity is maintained	Annual removal of 3.6 x $10^6 m^3$ sediment
B	Current intake tower facility is utilized	High construction cost
C	Same as above	Sediment flow into the Binga reservoir
D	Major portion of existing tower is utilized.	Critical foundation problems
E	New tower is connected to headrace tunnel with a shaft.	Construction of the connecting shaft is regarded to be difficult.

Of the above five schemes, Scheme E was found to be the most advantageous. This scheme is easier to construct, requires shorter construction time, and results in less cost than any of the other four schemes considered.

As a plan for countermeasures against sedimentation in the reservoir, Scheme E together with Schemes B and D are studied and compared further below in this Report.

Table - 12.1

SUMMARY OF REHABILITATION SCHEMES

Scheme	Brief Description	Outline of facility	Approx. cost (US$)	Approx. Time Required & Remarks.
(O) Dredging	- Dredging around intake by dredger - Annual dredging of 250,000 m^3 - Lower silt level	Dredger - 360 kw/6" x 1 Pipeline - 6" x 1800 m Silt basin - 100,000 m^2	Equipment - 3,746 x 10^3 Civil work - 374 x 10^3 Total 4,120 x 10^3 Annual OM 334 x 10^3	15 months Manufacture , delivery, installation and civil work
(A) Large Scale Dredging	- Dredging about 62% of annual sediment inflow and discharge to downstream	Dredger - 1100 kw/12" x 3 Pipeline - 12" x 5300 m x 3 Silt basin - 314,000 m^2	Equipment - 22,640 x 10^3 Civil work - 1,400 x 10^3 Total 24,040 x 10^3 Annual OM 4,500 x 10^3	2 years Manufacture, delivery, installation and civil work
(B) Raising of Intake Tower	- Raised by 1.7 m high Ring in order to prevent sediment inflow to inlet.		Metal Work - 10,320 x 10^3 Civil Work - 25,550 x 10^3 Total 35,870 x 10^3	5 years ° Fabrication 2 seasons 9 months Installation 2 seasons 5 months ° Power Interruption 5 months x 5 seasons ° No Intake Gate ° Reinforcement around intake tower by steel pipe pile ° Much underwater work
(C) Sediment Discharge Tunnel	- Construct new sand discharge tunnel	- Tunnel ø5 m x 410 m x 2 - Downstream river Channel protection work 1,200m - Roller gate 5 m x 15 m x 2 - Radial gate 5 m x 7 m x 2	Civil work 32,550 x 10^3 Metal work 17,150 x 10^3 Total 49,700 x 10^3	5 years ° Underwater work is necessary at the reservoir side ° Difficulty in connection work of intake portion and tunnel

SUMMARY OF REHABILITATION SCHEMES

Scheme	Brief Description	Outline of facility	Approx. cost (US$)	Approx. Time Required & Remarks
(D) Inclined Intake Tower	Inclined steel intake tower to be constructed on the slope of bedrock and to be connected to the existing tower	- Foundation work - Horizontal steel pipe ø7 m x 70 m - Inclined steel pipe ø7 m x 78 m	Civil work 11,870 x 10^3 Metal work 14,240 x 10^3 Total 26,110 x 10^3	4 years ° Execution of foundation work ° Power interruption - 5 months x 4 seasons ° Connection with existing tower ° Much underwater work
(E) Vertical Intake Tower	Vertical intake tower to be constructed above the existing headrace and to be connected thereto	- Shaft work ø7 m x 86.7m	Civil work 7,650 x 10^3 Metal work 11,700 x 10^3 Total 19,350 x 10^3	4 years ° Power interruption - 5 months x 2 seasons ° 80m vertical shaft excavation ° Blockade work of existing waterway
(F) River bed Excavation	Lower water level at tailrace outlet by excavating the river-bed	- Waterway Length 1,500 m	Civil work 1,333 x 10^3 Total 1,333 x 10^3 Annual Civil Work 60 x 10^3	
(H) Headrace Tunnel Extension	Extend tailrace to the point where no deposit is expected.	- Waterway Length 1,610 m - Dia meter 5 m - Tunnel length 37 m - Culvert length 1,463 m - Open-cut length 110 m	Civil work 11,900 x 10^3 Total 11,900 x 10^3	3 years ° Power interruption 5 month ° Connection with existing Tunnel

12.1.2. Upstream Face of the Dam

Based on the survey data provided by NAPOCOR, the current conditions of the upstream face of the dam are clearly understood. The damage to the face of the dam is worse than expected and the conditions are deteriorating every year. As indicated in Chapter 8, the stability of the upstream face of the dam is now regarded to be in a critical condition.

Therefore, the deteriorating portion of the upstream face should be repaired as soon as possible and the slope gradient of 1 to 1.80, as originally designed, should be reestablished.

It has been reported in 1986 that the sediments of silt in the reservoir have reached the level at EL 686.00 m.

Since this may affect adversely the repair work for the upstream face, the profile, conditions and properties of the silt should be investigated as soon as possible.

The necessary quantities of rockfill materials for the rehabilitation work were calculated by considering a quarry site downstream of the dam on the basis of the information presently available. This, however, should be studied further as necessary when more information becomes available.

12.1.3. Sedimentation at the Tailrace Outlet

As described in Chapter 11, the sediment deposition at the tailrace outlet area was caused by the sediment from the Besal River, a tributary of the Agno River in the area between the Ambuklao Dam and its tailrace tunnel.

As the tailrace of the Ambuklao Project is at the upstream and of the Binga reservoir, it is natural to assume that the

Binga reservoir had a big influence on the sedimentation at the above tailrace area. Thus, the location and the shape of the sediment deposits immediately downstream of the tailrace tunnel outlet were greatly affected by the fluctuations of the water levels in the Binga reservoir.

The sediment at the tailrace outlet area coming mainly from the Besal River with a drainage area of 143 km^2, is estimated to have a volume of 456, 400 m^3/year. A portion of this sediment is accumulated in the stretch of the river between the tailrace outlet and the the upstream end of the Binga reservoir. This has caused a water level rise at the tailrace area of about 4 meters.

Two alternative solutions to the problem of the tailrace outlet sedimentation were studied, Scheme H involving extension of the tunnel by construction of a conduit about 1,500 m. long, and Scheme F consisting of on open channel involving dredging of sediments and riverbed excavation downstream of the tunnel.

These two schemes were thoroughly analysed by making hydraulic and economic studies. It was found that Scheme H was very expensive, while Scheme F was considerably cheaper. However, the benefits from both schemes were small.

It was concluded that this problem could be best resolved in conjunction with the general solution for the problems of the sedimentation of both, Ambuklao and Binga reservoirs. As for the above some time will be required before such work is implemented, it is suggested that temporary measures, such as localized dredging, be currently provided. The above should be implemented as soon as possible, as no further reduction of the head for the Ambuklao units should be tolerated.

For final measures to be taken some time in the future, further studies of the sedimentation problem at the tailrace area, with future predictions, should be made. In addition, the sedimentation problem of the Binga reservoir and its influence on the Ambuklao tailrace and powerplant must be taken into consideration in studying the above measures.

For the current rehabilitation plan, it is recommended that channel construction downstream of the tailrace tunnel be adopted.

The disposal banks are proposed each at the right upstream and right downstream areas of the trailrace outlet. These points are tentatively illustrated on the Fig.-12.29. Besides, should there be a fear that the disposed materials would flow into the Binga Reservoir, the solidification agents are mixed and they should be and disposed of in solidified lumps. The best way of execution has to be selected based on the results of trial tests.

12.1.4. Turbine Inlet Valves

Two turbine inlet valves are used for protection of each turbine. They consist of one butterfly and one spherical valve. The total number of valves for the powerplant is six. The butterfly valves are used in emergency. At the time of the site visit by the JICA Study Team the valves could not be inspected as the flow could not be stopped. However, NAPOCOR has reported that, when the gates are closed, the leakage through the valves is of the order of about 3,000 ltr./min. As the valves have been in use for over 30 years, their economic life is assumed to have ended.

As there are no gates in the intake structure, the valves are the only protection for the turbines. Therefore, the valves are very important for maintaining the function and the operation of the plant in good order.

For the rehabilitation plan, it is recommended that gates be installed at the intake so that the repair work for the valves could proceed easily. Restoration of the original conditions of the spherical valves is absolutely required.

On the basis of the available data it is considered that all six valves have been seriously damaged. Since the new intake is to be provided with gates, the butterfly valves (for emergency use) will be of no use after the gates are installed. Besides, the existing of those valves themselves causes head losses. It is therefore recommended that the butterfly valves be all removed at the time of replacing the spherical valves with new ones.

12.2. Design and Construction Methods and Schedules

As discussed in Section 12.1, above, various schemes for the rehabilitation of the Ambuklao Project were studied. In this Section, the proposed design and construction methods and schedule for the above schemes are described.

12.2.1. Scheme A : Removal of Sediments by Large Dredging Boat

(1) Design

Annual inflow of sediments to the reservoir was discussed in the previous section of 3.1.2. and its average volume of inflow was estimated to amount 3,683 x $10^6 m^3$. This Scheme is designed to remove about 60% of abovementioned estimated volume of sediments by means of dredging. For the scheme, the rate of silt containing is assumed at 15%.

Main features of this Scheme are described below:

1) Dredge plan

a. Gross dredging volume 2,250,000 m^3/year

b. Hourly dredging volume

Annual operating hours

20 hrs/day x 25 days/ month x 10 months/year = 5,000 hrs/year

Volume $\frac{2,250,000 m^3}{5,000 \text{ hrs}} = 450 \text{ m}^3/\text{hrs}$

2) Dredging facilities

a. No. of dredgers

Pump dia. 12'

Capacity - 150 m^3/hr

$\frac{450 \text{ m}^3/\text{hr}}{150 \text{ m}^3/\text{hr/barge}} = 3$ dredger

b. Dredger barge

Size L48m x W8m x D2.25m

Dredging depth max. 77m

Pump dia 12'(300 mm)

c. Necessary pumping Capacity

Soil 150 m^3/hr

Silt 1,000 m^3/hr

Suction pipe length 4,000 m

Overland pipe length 1,300 m

d. Pump equipment

Main pump

Total head 75m/560kW/4,160V - 3 units

Underwater pump	Total head 25m/170kW/4,160V - 3 units
Booster pump	Total head 75m/560kW/4,160V - 6 units

3) Power consumption

a. Equipment Dredger 1,100kW/barge x 3 = 3,300 kW

Booster 600kW/unit x 2 units/ barge x 3 = 3,600 kW

Total 6,900 kW

b. Power load Barge 800kW x 3 barges = 2,400kW

Booster 600kW x 6 units = 3,600kW

Total 5,400kW

c. Annual energy consumption

5,400kW x 5,000 hrs = 27 mil.kWh

d. Switching Equipment

Tr. 1,400kVA/13.2kV - 4.16kV 3 units

800kVA/13.2kV - 4.16kV 6 units

e. Power supply (Power plant - Dam - Upstream Left Bank)

3,600m long overhead line

As shown on Fig.-12.1, dredgers are mainly located at the middle to upstream area of the reservoir for removal of sediment loads transported. One of the dredger placed in the middle portion of the reservoir will be used also for dredging around the intake tower in order to maintain the sediment height at the level of 1980.

In dredging operation, the sediments are first excavated by a jet flow excavator and sucked by underwater pumps for transport to a main pump on the barge. The main pump discharges sediment materials to a stilling basin provided immediately downstream of the dam through pipe line.

As indicated in Fig.-12.1, the stilling basin is located at the curved portion of the original river channel. This yard has an estimated storage capacity of 10 to 15 million m^3, which is capable of containing the discharged sediments only for 5 to 8 years considering annual sediment discharge by dredgers is approx. 2 million m^3. Therefore, the next disposal area should be sought along the Besal river or the main Agno river. This would, however, require costs equivalent to construction of about 90 m class rockfill dam every year. Such high cost is the most objectionable factor of this Scheme.

Besides, when typhoon or flood is anticipated to attack, these dredging facilities have to be protected. Dredging barge will be pulled by a tugboat and anchored with steel wire ropes at the right bank. Other equipment such as off-shore pipe lines, booster and dredger cables will be separated from the barge and anchored to the reservoir banks.

(2) Construction method

This dredging work is carried out at the inland mountain area. The dredger barges are designed to be non-self-navigated, steel box type, and of a structure consisting of several factory-fabricated sections which permit transportation by trucks or trailers. These sections are assembled into a barge at an available place at the reservoir banks in use of crane cars. Operation of a barge in forward/backward or left/right directions is done by 4 units of power winches installed on the deck through steel wire ropes which are fixed at their respective ends.

The dredging depth is adjusted by reeling device of rubber hose connected to the underwater pump.

Sucked sediment materials are disposed to the designated area through pipe lines pressurized with a main pump on the barge.

Off-shore pipe lines are equipped with floats and its on-land portions are supported by concrete anchors. Pressure boosters are arranged for pipe lines at proper intervals in order to raise the pressure in pipes.

Discharged sediment debris are left for a certain time and disposed at the designated area after dried up.

(3) Construction schedule

It would take about 15 months from bidding, design, manufacture, delivery and field assembly. Outlined construction schedule is given in Fig.-12.3.

12.2.2. Scheme B : Raising of the Intake Tower

(1) Design

By modifying the top and the bottom of the existing intake tower, the tower could be raised. As the sedimentation is advancing, the tower is raised by installing rings, one on another, in stages. The rings have a diameter of 7 meters and a height of 1.7 meters. The design is shown in Figs.-12.4, and 12.5.

The base will be reinforced by a shell, 30 meters in diameter. The shell will be constructed around the intake tower with tubular steel piles. The silt inside the shell will be removed by dredging and replaced by crusher-run material for stability of the intake tower.

For modifying the top, the existing intake will be demolished. Then, a steel ring, 7 m in diameter and 1.3 m in height, will be installed and secured in place. In accordance with the design, a second ring 1.7 m high will be installed on the top of the first, and other rings will follow.

As the rings are made of steel, their maximum height was determined to be 1.7 m, considering the transportation and installation requirements.

The rings will be interconnected and attached firmly to each other. After completion of the installation, they will be concreted in to form one solid unit.

The hydraulic design of the intake will remain the same as before and no gates will be provided.

(2) Construction Method

For reinforcing the base, the sheet pile driving is probably the largest problem. It is estimated that four 300 tons (26 m x 11 m x 2.2 m) barges equipped with reversible excavators (R.R.C-15) will be required for excavating the base of the piles and their driving.

As the working area around the intake tower is small, only one working group could work at a time. Therefore, the construction period for heightening the intake tower will depend on the time required for construction of the shell. It must be stated that construction could be carried out only during dry season. It is estimated that the yearly construction period for the modification of the intake tower will be less than six months. This will be mainly affected by the water levels in the reservoir as required for the operation of the units. Occasional suspension of the flow during this periods should also be taken into account.

Modification of the top of the intake tower will be done underwater. The cover of the intake tower will be cut into small blocks by divers using boring machines. After the above is done, the cover will be removed by a crane and discarded.

The concrete base (finishing of the top of the intake) for the ring to be used for raising the tower will be made underwater. After the first ring is installed and fastened to the base, underwater concrete will be poured into the ring. Installation of the next ring will follow after the above work is finished.

Since the above work will be done underwater, the capability of the divers will be very important for the efficiency of the work.

(3) Construction Schedule

As described above, construction on the modification of the intake tower will be carried out in a small area around the intake tower and, most of the time, underwater. In addition, with occasional suspensions of flows and the control of the reservoir water level, the required construction time for the above work will be rather long. The key item of the above construction will be the pile driving work.

It is estimated that it will take four seasons (four years) to modify the base of the intake tower. In the fourth year, the work on the modification of the top of the intake tower will be started. This work will be carried out in two stages. During Stage I (the first two seasons), the intake tower will be raised to EL 717.0 m. Stage II (the second two seasons) construction will be started when the sediment surface reaches the level at EL 713.0 m. This will occur, in accordance with the simulation studies, in or after the year 2010. Fig.-12.6 shows the construction schedule for Stage I.

12.2.3. Scheme C: Provision of Large Capacity Sediment Removal Facility

(1) Design

Sediment removal facility of a large capacity is designed to be located nearby the existing intake tower. Inlet of discharge tunnel is set at the lowest possible level from the viewpoint of construction execution (EL. 700.00 m). Gates are opened at the time of flood in order to discharge sediment loads.

Planned discharge capacity is approx. 1,000 m^3/sec, with two tunnels each having 5 m inner diameters. Discharge outlet is set at the river bank downstream of the dam. Sediments are disposed to the Agno river via a stilling basin.

Gate facilities are consisting of two main gates (radial gates) at the outlet and two auxiliary gates (roller gates) at the inlet (Refer to Fig.-12.7).

By this scheme, sediment deposit level around the intake tower is considered to settle down stably at the level of EL.704.00 m which is a little higher than the bottom of tunnel inlet. Accordingly, the intake sill level will be raised up to EL.710.55 m. Deficiencies of this scheme are that there is no intake gate and that disposed sediments will be directly discharged into the Binga reservoir.

(2) Construction method

Construction of inlet and inclined portions for gates is planned to be executed after dewatering by controlling the reservoir level. Tunnels will be excavated from the downstream side and connected to the inlet portion after roller gates have been installed. Other works for stilling basin and disposal area are implemented in the same manner as mentioned in Scheme B.

(3) Construction schedule

Dominant controlling factors in the construction schedule are tunnelling work, stilling basin and river bank disposal area works. Besides, the works for inlet of large capacity discharge tunnels will be done within the reservoir area. Therefore, the inlet works will be carried out in advance by controlling the reservoir water level and be completed in coordination with the tunnelling works. The raising work of intake tower will be finished in the latter 2 years of the construction period.