THE REPUBLIC OF THE PHILIPPINES

MASTER PLAN FOR MAYON VOLCANO SABO AND FLOOD CONTROL PROJECT

SUPPORTING REPORT

MARCH 1981

JAPAN INTERNATIONAL COOPERATION AGENCY

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1. MAIN REPORT

2. SUPPORTING REPORT

MASTER PLAN

FOR

MAYON VOLCANO SABO AND FLOOD CONTROL PROJECT

SUPPORTING REPORT

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I. HYDROLOGICAL ANALYSIS

1.1 Available Data

1.1.1 Rainfall

There are ten (10) ordinary rainfall gaging stations within the project area. The locaton and record length of the stations are shown in FIG.-I.1 and FIG.-I.2. Among these stations, three stations of Guinobatan, Legaspi and Sto. Domingo have daily records over more than 20 years, but other stations have records of less than 10 years. No hourly record is available at any stations. The 6-hour intervals records are Legaspi is one with the shortest duration, which is available since 1970.

As seen in the location map, all stations are located along the major roads and at low altitude of less than 130 m. As no rainfall station exists in the mountainous region, two self-recording type rain gages were installed on the slope of Mayon Volcano in October 1979 by the Study Team in order to grasp the rainfall amount in the mountainous area, that is, at high altitude.

Both rain gages produced hourly records for three months. However, they are no longer operational due to a failure of management.

1.1.2 Stream Flow

Though the project area has many streams, the stream flow observation network is poor. Only the Quinali (A) River ands the Talisay River have gaging stations. Eight (8) ordinary gaging stations are located in the Quinali (A) River basin and one is in the Talisay River basin. The location and record length of the stations are indicated in FIG.-I.1 and FIG.-I.3 respectively. Other basins however have no gaging stations.

Measurement is conducted with a stuff gage and a gage reading is recently made three times a day. During flood time, it becomes more frequent, but due to a failure of river dikes and a bad accessibility to the stations the measurement is often interrupted. For this, the flood peak runoff recording is apt to be missed and thus the availability of flood record is low. Also, the existence of irrigation dams has apparently affected the ordinary runoff record especially during dry months.

No hourly record is available except Banao bridge station established by the Study Team. Banao gage is still working as of August 1980, but as the major floods have not yet been recorded and also the discharge rating curve is not prepared so far, the gage height records obtained are not available for the present study.

As for Lake Bato, gage height records are available since 1960 with some interruptions.

1.1.3 Sediment Load

Suspended load records are available at two points. One is Busac of the Quinali (A) River, another is allang of the Talisay River. Both sites coincide with the stream flow gaging stations shown in FIG.-I.1. The measurement was started in April 1974 and has been conducted together with discharge measurement. The measurement frequency is almost once a month. FIG.-I.31 shows the actual suspended load records plotted against water discharge on the logarithmic paper.

On the other hand, as no bed load measurement records are available in the area, the sieve analysis of river bed materials was performed in order to obtain the data for estimation of bed load at various points in the area during the field investigation periods in 1979 and 1980. Sampling sites are shown in FIG.-I.21.

1.2 Flood Analysis

1.2.1 Basic Principle

The present flood analysis aims to estimate only probable flood peak runoff for the reasons below.

(1) The existence of flood record is limited within the Quinali (A) River basin including the Talisay River and also the reliability of flood peak record is low due to the poor measurement manner.

(2) No hourly records of both rainfall and stream flow are available within the area. This means the actual hyetograph and hydrograph do not exist and thus the flood hydrograph analysis cannot be made.

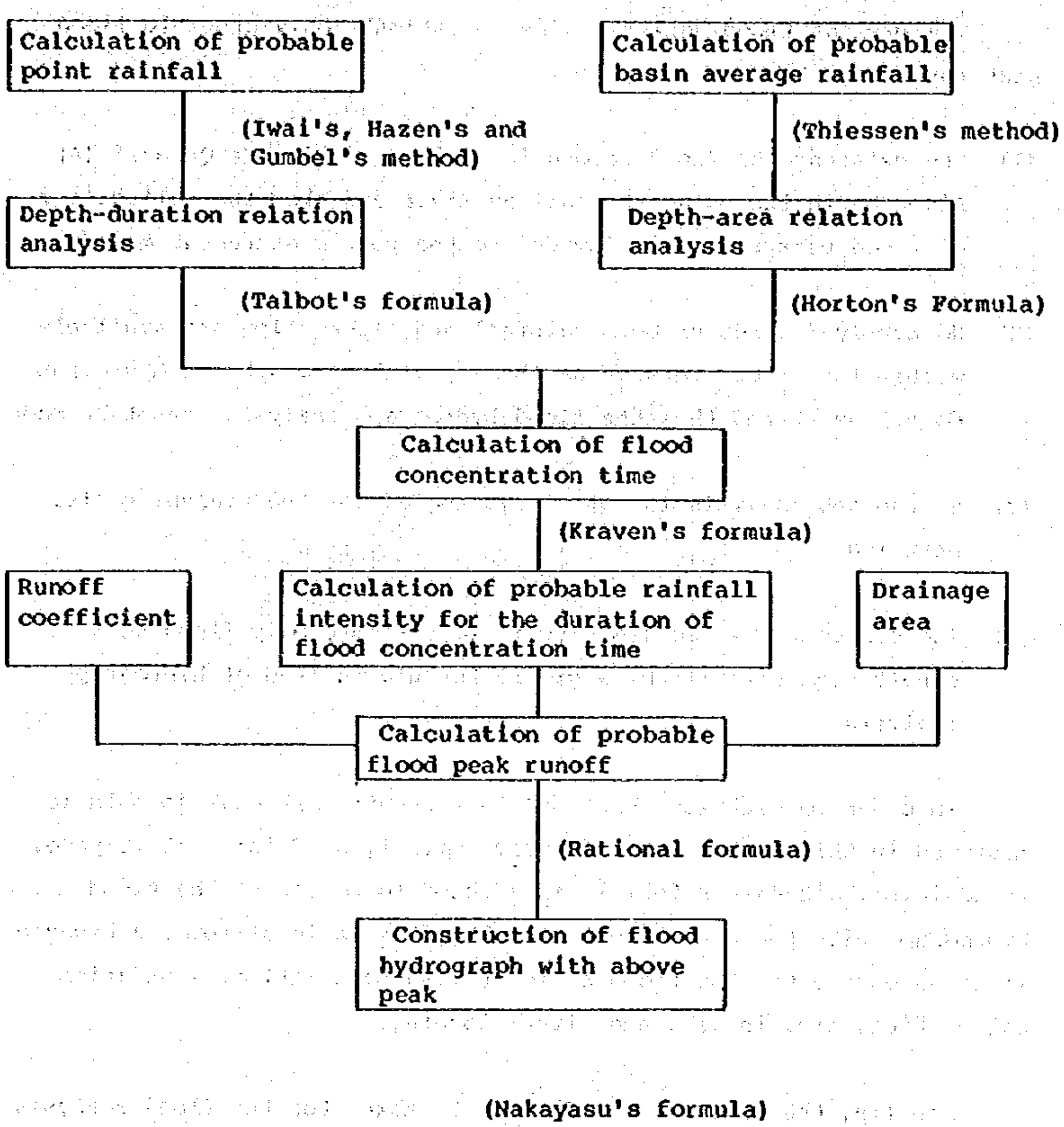
(3) Hydrograph analysis is not always needed for the present master planning.

(4) The common way in such a case is to estimate only flood peak runoff from rainfall by a simple formula instead of hydrograph analysis.

In order to estimate the flood peak runoff, Rational formula is employed in this study, which is very commonly used for that purpose. In addition, Nakayasu's formula is adopted to construct the flood hydrograph with the peak runoff calculated by Rational formula because it is necessary for the flood sediment load calculation, inundation calculation, etc. in Sabo and river planning.

Finally, the general procedure to be taken for the flood analysis may be explained by the following flow chart with employing Rational formula.

GENERAL PROCEDURE FOR FLOOD ANALYSIS



1.2.2 Probable Rainfall

1.2.2.1 Calculation Method

The calculation of probable rainfall is made by three methods of Iwai's, Hazen's and Gumbel's, and the method which gives the largest values for higher return periods is adopted. Plotting position after Hazen is employed to plot the data and examine the frequency curve fitting.

1.2.2.2 Probable Point Rainfall

In general, the record length of more than 20 years is preferably needed for the reliable statistical analysis. Nevertheless, only three out of ten stations in the area meet the above condition, so that the stations with records of more than 5 years are taken up for the study relaxing the condition said above. Eight stations can be analyzed.

The calculation is made for the annual maximum 1-day and 3-day rainfall of the qualified stations and also the annual maximum 6-hour, 12-hour, 18-hour and 24-hour rainfall of Legazpi station. FIG.-I.4 throiugh FIG.-I.13 give the results of calculation graphically.

1.2.2.3 Probable Basin Average Rainfall

As shown in the flow chart of analysis procedure in the previous section, the probable basin average rainfall should be estimated to check the depth-duration relation analysis mentioned later.

The Quinali (A) River basin including the Talisay River is taken up as a representative basin taking into account the size of basin and the number of rainfall stations in it.

TABLE-I.10 lists the stations concerned and the annual maximum basin average 1-day rainfalls estimated by Thiessen's method. These rainfalls are analyzed statistically and shown in FIG.-I.14.

1.2.3 Depth-Area-Duration Analysis

1.2.3.1 Depth-Area Relation

The average depth of rainfall for a given duration in a certain area generally tends to decrease conversely with increasing of area. This relation is usually derived from the isohyetal map with a given duration. However, as the available data are inadequate to prepare the isohyetal map, Horton's formula is employed for the present study and it is expressed in the form of

$$P = Po \exp(-kA^n)$$

where,

- P : average depth of rainfall for a given duration over a certain area (in.)
- Po: largest rainfall amount in the area (in.)
- A : area ($mile^2$)
- k, n: constants

Po, k and n are determined in the manner described below.

Po is usually given on the isohyetal map, although it is not available in this study. The rainfall depth generally tends to increase with altitude. All stations in the area are located at low altitude of around 130 m or less, while the highest point in the area is the top of Mayon Volcano with an altitude of 2,469 m. Therefore, Po is obliged to be roughly estimated through the rainfall of stations at low altitude.

FIG.-I.16 and FIG.-I.17 show the rainfall amount correlation among Legazpi, Guinobatan and Masarawag station which was established by the Study Team at high altitude of about 450 m to grasp the difference of rainfall amount due to altitude.

As seen in the figures, the rainfall amount of Masarawag with high altitude is obviously greater than that of the stations at low altitude such as Legaspi (EL. 19 m) and Guinobatan (EL. 123 m). The increase rate is more than 100 percent. The rate however is not definite because the data include no large values occurred concurrently at both stations concerned and also the record length is only three months. Accordingly, an increase rate of 50 percent is adopted tentatively and therefore Po is assumed to be indiscriminately given as 1.5 times of the rainfall of the representative station in the basin to be studied.

As for k and n, they are taken as approximately 0.1 and 0.2 respectively for 24-hour rainfall in the United States according to Horton. In the study, n is determined with fixing k as 0.1 constantly. As shown in FIG.-I.18, n should be taken as 0.3 for the probable depth-area curve to pass through or near the probable basin average rainfalls plotted which were calculated taking up the Quinali (A) River basin as a representative area.

1.2.3.2 Depth-Duration Relation

The average rainfall intensity (i.e. depth) in a certain area decreases with increase of duration. This relation is obtained from analyzing the rainfall intensities for the various durations.

The depth-duration relation for the short duration such as 1-hour, 2-hour, etc., is needed for the present study sine the flood concentration time will range from about 20 minutes to around 6 hours. On the other hand, the shortest duration of available rainfall records is 6-hour at Legazpi, and therefore the rainfall for the duration of less than 6 hours is obliged to be estimated by the appropriate formula.

Talbot's formula is commonly applied for the rainfall intensity-duration relation for the shorter duration. It is expressed in the form of

$$I = \frac{b}{t + a}$$

where,

I : rainfall intensity (mm/hr)

t : duration (hrs)

a, b: constants

FIG.-I,20 shows the probable rainfall intensity - duration curves defined by Talbot's formula for Legazpi station, which are prepared with determining the above constants by the least square method for the probable rainfall intensities with the duration of 6-, 12-, 18- and 24-hours.

The intensity-duration relation of Legazpi station is assumed to be applicable to other basins multiplying by the ratio of the probable basin average 1-day rainfall of the basin to be studied and the probable 24-hour rainfall given by the said curve.

1.2.4 Runoff Coefficient

No data on the runoff coefficient related to flood peak runoff are available in the area and also no standards on that matter are provided in the Philippines so far. Consequently, the Japanese Standard may be referred to, because the hydrological conditions of the area is considered to be similar to those in Japan.

Japanese Standard indicates the following runoff coefficients for Sabo and river planning depending on the ground condition.

RUNOFF COEFFICIENT IN JAPANESE STANDARD

Ground Condition	Runoff Coefficient
Field, Farm	0.6
Paddy Field	0.7
Mountainous Area	0.7

As the project area is composed of almost the mountainous area and paddy field, the runoff coefficient to be applied for Rational formula can be taken as 0.7 uniformly over the area.

1.2.5 Flood Concentration Time

The flood concentration time is given by the summation of the time required for flood to flow out into the river course from the forest point in the basin and the time required for flood to run down through the river course up to the point under consideration.

The various empirical formulas have been proposed for the estimation of flood concentration time. Among them, Kraven's formula is simple and applicable for the calculation of both times defined above. It is therefore adopted for the present study.

Kraven gave the following empirical values on the flood velocity depending on the river bed slope.

FLOOD VELOCITY AFTER KRAVEN

River Bed Slope	1/100	1/100 - 1/200	1/200
Flood Velocity (m/sec)	3.5	3.0	2.1

Finally, the flood concentration time can be calculated by the following equation.

$$T = L/60V$$

where,

- T : time of flood concentration (min)
- L : length of river course from the farthest point in the basin to the point under consideration (m)
- W : flood velocity after Kraven (m/sec)

1.2.6 Probable Flood Peak Runoff

1.2.6.1 Base Point

The base point for Sabo and river plannings is usually set at the point to be studied hydrologically and hydraulically and which is closely related to the plan under consideration.

The project area has many rivers and their tributaries and also each of them is concerned to the present Sabo and river master planning more or less. Consequently, the confluences of the rivers are mainly selected as a base point as well as the important structure sites such as road bridges, railway bridges, etc.

These base points are indicated in FIG.-I.21. The project area is divided into sub-basins according to the base points selected.

1.2.6.2 Representative Rainfall Station

The representative rainfall station should be determined for each basin, which definites the meteorological conditions of the basin. In general, the station having the long and reliable record is selected as a representative one. In the project area, four stations of Guinobatan, Legazpi, Sto. Domingo and Malinao which are located around Mayon Volcato have a qualification. The project area therefore is divided into four large parts round Mayon Volcano as a center corresponding to the above four stations. They are the Quinali (A) River basin, the Yawa River basin, the Quinali (B) River basin and others.

As for the Talisay River basin, Allang is selected as a representative station because the Talisay River originates in the mountains other than Mayon Volcano.

TABLE-I.12 lists the representative rainfall stations and the basins subject to them.

1.2.6.3 Probable Flood Peak Runoff

As mentioned before, Rational formula is employed for the calculation of probable flood peak runoff. It is expressed in the form of:

$$Q = \frac{1}{3.6} A \cdot f \cdot I$$

where,

Q : flood peak runoff (m^3/sec)

A : drainage area (km^2)

f : runoff coefficient (= 0.7)

I : rainfall intensity for the duration equal to the flood concentration time (mm/hr)

In the equation, f and I are defined as described in the previous sections and A is given. The calculation procedure is itemized below in rather detail.

(1) The drainage area, river length, slope and runoff coefficient are given at first.

(2) The flood concentration time is calculated by Kraven's formula.

(3) The probable basin average 1-day rainfall is given by converting the probable 1-day point rainfall of the representative station into the areal rainfall through Horton's formula against the drainage area of the subject basin.

(4) The probable rainfall intensity - duration curves of Legaspi are revised with the ratio of the probable basin average 1-day rainfall obtained above to the probable 24-hour rainfall given by the original curves in order to apply to the basin concerned.

(5) The probable average rainfall intensity during the flood concentration time to be substituted to Rational formula is given by the above revised rainfall intensity - duration curve against the flood concentration time.

(6) The probable flood peak runoff is calculated by substituting the drainage area, runoff coefficient and the rainfall intensity given above to Rational formula.

(7) Above calculation is made for every return period and every base point.

The results of calculation are summarized in TABLE-I.13.

1.2.6.4 Examination of Results

The probable peak runoff is derived from the probable rainfall and also some assumptions such as the depth-area-duration are accepted. The results therefore should be examined by proper way.

FIG.-I.22 shows the specific discharge diagram of the Philippines, of which data plotted are the recorded maximum specific discharges of the rivers all over the Philippines.

The figure illustrates the comparison between the recorded maximum specific discharge and the 50-year probable specific discharge calculated in this study. The calculated values are expected to be within a reasonable range because the recorded data have a return period of about 30 to 60-year after Hazen's and Weibull's plotting position and also the constant of Creager's enveloping curve is 60, which is within the reasonable range from 30 to 100.

1.2.7 Construction of Flood Hydrograph

For the estimation of flood sediment load, inundation depth and area, etc., flood hydrograph is required. As the probable flood peak runoff has been already estimated, the shape of flood hydrograph can be defined by giving functions of rising and falling limbs. Nakayasu's unit hydrograph formula is employed for this study, which is commonly applied for the small scale basin such as the present study basin with no data on flood hydrograph. Nakayasu's formula is expressed by the following form.

Rising limb: $Q/Qmax = (t/T_1)^{2.4}$

Falling limb:

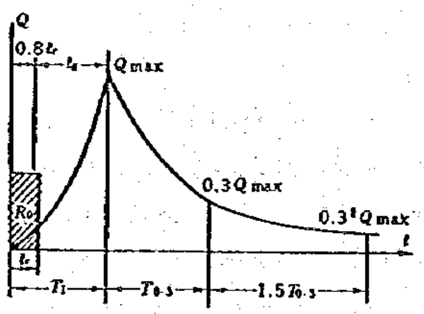
$1 > Q/Qmax > 0.3 \qquad Q/Qmax = 0.3^{(t-T_1)/T_{0.3}}$

$0.3 \geq Q/Qmax \geq 0.3^2 \qquad Q/Qmax = 0.3^{(t-T_1) + 0.5\ T_{0.3}/1.5T_{0.3}}$

$0.3^2 > Q/Qmax \qquad Q/Qmax = 0.3^{(t-T_1) + 0.5\ T_{0.3}/2.0T_{0.3}}$

where, Qmax is flood peak runoff, Q is flood runoff, T_1 is flood concentration time, $T_{0.3}$ is time required for flood runoff to be equal to 0.3 times of peak runoff, which is usually taken as the time equal to T_1.

The shape of unit hydrograph defined by Nakayasu's formula is schematically illustrated below.



Regarding to the rainfall, the probable hyetograph is produced from the probable rainfall depth - duration curves at Legazpi constructed in the previous sections, by using a critical arrangement method.

The probable hyetographs are shown in FIG.-I.23.

The flood hydrograph is generated using Nakayasu's formula and probable hyetograph so as to have the required probable flood peak runoff by a trial and error method.

Flood hydrographs constructed are shown in the respective chapter concerned as required and not presented here.

1.3 Draught and Low Flow Analysis

1.3.1 Draught Frequency Analysis

The draught analysis is essential for the irrigation development scheme. In this study, low rainfall amount and low flow in the area are statistically analyzed with Weibull's extreme value frequency analysis method which is applied for the statistical analysis of annual minimum events.

As for low rainfall, annual rainfall and annual minimum monthly rainfall are analyzed for three stations of Guinobatan, Legaspi and Malinao which will be the meteorological key station for irrigation development scheme. FIG.-I.24 and FIG.-I.25 show the draught frequency curves of low rainfall amount.

Then, low flow is analyzed on the annual minimum monthly mean. FIG.-I.26 through FIG.-I.28 show the results.

1.3.2 Regional Draught Frequency Analysis

In the irrigation development scheme, some base points are located in the ungaged basin and that the draught analysis is required. One procedure designed to overcome this problem is regional draught frequency analysis. Records from a number of streams are assembled. Flow data are made nondimensional by dividing by the mean annual draught runoff with a return period of 2.33-yr for the station. Median ratios for each order number are then plotted and a regional curve fitted. FIG.-I.29 shows the regional draught frequency curve of annual minimum monthly runoff constructed in the Quinali (A) River basin employing the Nasisi, the San Francisco and the Cabilogan River gaging stations which have appropriate records. Then, a relation defining the mean annual draught runoff is required, and this is often given by a simple relation between mean annual draught runoff and drainage area. FIG.-I.30 shows this relation constructed for the Quinali (A) River basin.

Finally, the frequency curve for an ungaged basin is constructed by first determining the mean annual flood and the converting the ratio scale of the regional frequency curve to runoff by multiplying the ratios by the estimated mean annual draught runoff.

But, it should be noted that the weakness of the regional frequency approach lies in the assumption that all streams will show the same variance of the ratio to mean annual draught runoff.

1.3.3 Drainage Area-Runoff Relation

In general, the low flow rate is roughly defined with a function of drainage area. The construction of drainage area - runoff relation is useful for the estmation of runoff in the ungaged basin. The relation is usually expressed by the equation of

$$Q = a \cdot A^n$$

where,

Q : runoff (m^3/sec)

A : drainage area (km^2)

a, n: constants

FIG.-I.31 shows the drainage area - annual mean runoff relation curve constructed for the Quinali (A) River basin. TABLE-I.19 lists the monthly mean runoff at each station.

1.4 Sediment Analysis

1.4.1 Suspended Load

As stated before, the measurement records of suspended load are available at two stream flow gaging stations, that is, the Quinali (A) River at Busac and the Talisay River at Allang since April 1974. The measurement is conducted almost once a month. The records obtained are plotted in FIG.-I.32 and FIG.-I.33.

In general, the sediment discharge is correlated with the water discharge by the equation.

$$Qs = K \cdot Q^n$$

where,

- Qs : sediment discharge (tons/day)
- Q : water discharge (m^3/sec)
- K, n : constants

The constants, K and n, are defined by the observed data. n is known to be 2 to 3 statistically. In this study, K is determined taking n as the value between 2 and 3 because the data are dispersed on the log-log paper and thus n, that is, slope of the straight line is difficult to be difined without some instruction. The determined K and n are shown in FIG.-I.32 and FIG.-I.33.

The annual suspended load amount is calculated by converting the daily water discharge into the daily suspended load using the equation explained above, that is, suspended load rating curve.

TABLE-I.20 shows the annual suspended loads calculated under the present river conditions for the Quinali (A) River at Busac and for the Talisay River at Allang. The mean annual suspended loads are estimated at 86,400 m^3 at Busac and 131,000 m^3 at Allang. The specific suspended loads are 370 m^3/year/km^2 and 1,450 m^3/year km^2 respectively. But, it should be taken account of that the used suspended load rating curves are made based on the data below 13 m^3/sec in water discharge, while the recorded runoff sometimes exceeds the limit, especially at Allang. Therefore, the suspended load estimated for the Talisay River at Allang may be very rough, while that for the Quinali River at Busac is considered to be correct.

1.4.2 Bed Load

No bed load measurements have been performed in the project area so far. The bed load therefore is obliged to be estimated by the appropriate formula using the sieve analysis results of river bed materials.

The estimation is made for the Quinali River at Busac, the San Agustin River at San Agustin and the Talisay River at Bacolod to grasp the bed load transport amount under the present river conditions.

Einstein's formula is one of the most popular formulas to calculate the sediment load using the given grain size distribution curve, and which is adopted for the present study.

FIG.-I.34 shows the bed loads calculated by Einstein's formula under the present conditions for the different water discharges and the bed load rating curves fitted assuming that the same equation as the suspended load can be applied to the bed load. The annual bed load amount is calculated in the same way as the suspended load. The grain size distribution of QA-4 sieve analysis site is applied to the Quinali River at Busac, QA-2 site to the San Agustin River at San Agustin and T-3 site to the Talisay River at Bacolod. The grain size distribution curves are shown in FIG.-I.35.

The daily runoff record is available at Busac and at San Agustin, but the Talisay River at Bacolod has no runoff records so that the runoff at that site is produced by multiplying the runoff at Allang by the ratio of annual mean runoff read out from FIG.-I.39 to the respective drainage area.

The results are arranged in TABLE-I.21. Mean annual bed loads are estimated at 16,600 m^3 at Busac, 1,800 m^3 at San Agustin and 1,100 m^3 at Bacolod. The differences among these values may result from the difference of the mean grain size used in calculation.

Compared with the suspended load amount, the value for Busac may be reasonable because the bed load amount is usually estimated at 10 to 20 percent of the suspended load amount statistically.

1.4.3 Comparison with Other Rivers in Luzon

In order to examine the results of sediment load calculations, information provided by the National Irrigation Administration on sediment yields is given below as specific annual sediment yields for rivers in Luzon.

SPECIFIC ANNUAL SEDIMENT YIELD IN LUZON

Name of River	Drainage Area (km^2)	Yield ($m^3/yr/km^2$)	Record
Ibulao R., Hapid	606	292	1972-73
Siffu R., Munoz	686	292	1965-70
Chico R., Pasonglao	1,987	580	1963-70
Bokod R., Bokod	48	274	1963-69
Ambayoan R., Sta. Maria	281	379	1963-71
Agno R., Carmen Rosales	2,209	38	1964-71
Camiling R., Nambalan	142	131	1963-69
Pila R., Pacalay	126	31	1963-71
Purac R., Valdez	118	669	1963-71
Santor R., Cuyago	89	15	1965-72
Cabu R., Cabu	143	204	1963-72
Pampanga R., San Antonio	2,851	798	1963-72
Pampanga R., San Vicente	3,467	441	1963-72
Rio Chico R., Sto. Rosario	1,177	167	1963-72
Pampanga R., San Augstin	6,487	317	1963-71
Quinali R., Busac	233	442	1954-78

As seen in the above table, the sediment load of the Quinali River at Busac is estimated within the reasonable range comparing with other rivers in Luzon.

II. GEOLOGY

Geology of Mayon Volcano and its surroundings is largely classified into Pre-Mayon's rocks and sedimentary rocks, volcanic rocks and alluvium.

They are sub-classified as follows and explained individually in rather detail. (Refer to "Geological Map" in the main report)

2.1 Pre-Mayon's Rocks and Sedimentary Rocks

(a) Limestone (I)

Grey or dark-grey limestone, sometimes intercalated with sandstone are developed on the low hill area of elevation about 200 m in south of San Vicente. They were formed in Miocene-Pleiocene period of Tertiary. Their general strike is WNW-ESE, dipping 30^{o} to south.

(b) Andesites (I)

These are extruded in Oligocene to Miocene Epoch of Tertiary and named as "Daraga Formation" by Corby (1951). They are mostly of andesitic or dacitic lava (partly including rhyolite), agglomerate and pyroclastic material, sometimes intercalating conglomerate, sandstone and shale. They are distributed on the hill area in south of Legaspi City, hills in north of Sto Domingo and on Mt. Bulakawan in south of Malilipot. They are moderately consolidated and are cropped out as soft or hard rock.

(c) Limestone (II)

Limestone formed in Pleicene Epoch of Tertiary is named as "Ligao limestone". Foraminifera, calcareous Algae and Echinodermata remnants are embedded in caleareous silt under microscopic observation. Nephrolepidina, Discocyclina or Amphystegma is sometimes observed in poorly reserved state. They are found in the hill area, south of Ligao and southwest of Camalig.

(d) Sandstone shale alternation

A group of sandstone shale alternation was deposited in Oligocene to Pliocene Epoch of Tertiary. Those distributed underneath limestone in the hiills in south of San Vicente was formed in Oligocene to Miocene of Tertiary. The others was in Pliocene Epoch. These alternation are developed in hill area from San Vicente to southern hills of Lake Bato and on the southern hills of Camalig. The latter contains limestone and calcareous siltstone intercalation.

(e) Andesite rocks (II)

These are group name of volcanic rocks formed in Miocene to Pleistocene Epoch of Tertiary, comprising andesite-, dacite-lava, agglomerate and tuff.

Low hills of Kilicao, Maipon, Bubulusan and Amtic are covered with ash fall deposit from Mayon Volcano for its greater part. An altered volcanic rocks of unknown age are underlying. Rhyolite is found on the Tancalao hill near Bantayan. It possibly forms volcanic spine. Andesite is exposed on both sides of the Quinali (B) River, close to Bantayan and southwest of Tabiguian, providing favourable construction site for Sabo dam.

Microscopic study on the lava rock, south of Polangui reveals that rock forming minerals are plagioclase, augite and other opaque minerals and that secondary minerals are chlorite, actinolite, sericite, calcite and serpentive. The rock can be identified as augite-andesite, as same as the andesite of Mayon Volcano, though partly altered.

2.2 Volcanic Rocks of Mayon Volcano

(a) Lava

Lava rocks of Mayon Volcano is A-A lava with rugged surface and coke-like texture. They are andesite or andesitic basalt. They can be identified as augite andesite of hypersthene-augite andesite. Most of the lava flowed out from the central crater, some of them flowed out from fissures at elevation 1,300 m on the eastern slope. The lowest end of the existing lava is at an elevation of more or less 200 m. Most of the lava flow terminated at an elevation of 400 m - 500 m. Lava, extruded in historic age was mostly flowing to east, south or southwest directions.

These lava flows are hard and solid rock and mostly filling up old gully channels. There are much debris produced by chilled effects in the edge of lava flow on both sides of flow channels. Mud flow or debris avalanche are originated in these debris. Especially, in the lower reach of the southwestern lava flow, extruded in 1978's eruption, may provide new debris flow in near future.

(b) Pyroclastic flow

Pyroclastic flows in Mayon Volcano were produced by glowing cloud, ash flow and scoria flow. They are neither stratified nor welded sediments, comprising breadcrust bomb, lappili and blocks inbedded in ash matrix, sometimes include carbonized wood fragments.

These pyroclastic flows are developing in the range of elevation 1,300 m down to the skirts of the slope. They flowed in strip shape on the upper slope and in fan shape on the lower slope. Their thickness is variable between one meter and several meters, thinner on a steep slope and thicker on a gentle slope of lower skirt.

Most of pyroclastic flows in historic age are flowing toward east, south or southwest, the same directions as in the case of lava flows. Glowing cloud were often recorded in the past eruptions. Especially, emission of glowing cloud in 1897 caused a damage of 212 death in Libog (Sto. Domingo). Pyroclastic flow deposits newly formed are very loose and non-resistive against erosion, and those in the upper slope of higher than elevation 600 m will possibly cause breaking and debris flows under heavy rain in future.

(c) Ash fall deposits

These are well-bedded sediments of breadcrust bomb and ejectamenta materials. Volcanic ejecta of bigger size are usually deposited within 1,000 m range around crater. It is reported that volcanic bomb of 10 - 20 cm in diameter fell on Budiao village, 8 km apart from the crater in 1914's eruption. Lappili is usually ejected within 8 - 10 km range from the crater. Volcanic ash is sometimes spread over as far as 100 km or more in distance.

Prevailing direction of volcanic ejecta (especially volcanic ash) can be classified into following seasons.

June to November: Ash is transported to northeast (Tabaco site) by southwest Monsoon. In case the monsoon is dropped, the ash is flown to south westward.

November to March: Ash is transported to southwest by northeast Monsoon and northeast Trade Wind.

April-May: Monsoon wind drops in these months. The ash is transported to southwestward.

Airfall ejectamenta from Mayon eruption is usually deposited as an alternation of volcanic ash and lappili in 1 - 2 m thick. These airfall deposits on both banks of gully below elevation of 600 m becomes thinner in lower slope and thickner and more compacted in upper slope. These airfall deposits of well-consolidated and older one, can be used as basement rocks for small-scale Sabo dam, though less resistive against erosion. These well-consolidated airfall deposits are exposed at high cliff of 10 - 20 m high on the riverside. Around Balaigang, east of Mayon Volcano, there developed airfall deposits of well-consolidated all along surface to bottom.

(d) Talus

The talus comprising ejected talus, airfall coarse deposits and small size lava flow, develops over the slope above elevation 1,000 m. These areas are mostly barren ground and naked ground between summit and an elevation of 1,850 m and grass land in the lower elevation.

(e) Mud flow deposits

Volcanic mud flow is formed in various modes of occurrence: (1) muddy materials are flowing out directly from crater; (2) eruption in the bottom of crater lake causes overtopping at the crater wall; (3) volcanic ash deposited once on the slope and produced mud flow under heavy rain; (4) Pyroclastic flow debauches in swamp, gully or lake and spills out mud flow.

In case of Mayon Volcano, mud flow of above item (3) takes place. Pyroclastic materials ejected from Mayon Volcano are deposited on the slope, and then moved again as a mud flow by torential rain. Occurrence of mud flow is not always associate directly with eruption.

Mud flow sometimes contains well-bedded deposits but pyroclastic flow does not contains such a deposit. Mud flow deposits are developed on the slope of lower than elevation 800 m, usually below elevation 500 m, covering most of the skirt of volcano. Thickness of a sheet of mud flow is usually between 1 m and 10 m, thinner on upper slope and thicker on lower slope. Mud flow in 1766 had a thickness as high as top of coconut tree; Mud flow in 1814 burried Bubulusan, Cagsawa and Budiao villages with a depth of 10 - 12 m.

2.3 Alluvium

These are sand and gravel bed which was removed from old mud flow deposits, transported by torrent and redeposited on the downstream riverbed. Major components are subanguler or subrounded gravels in the upstream reach and sandy material with volcanic ash in the downstream reach of the rivers in alluvial plain. Flat plain is covered with ash and silt layer of several to 20 cm thick.

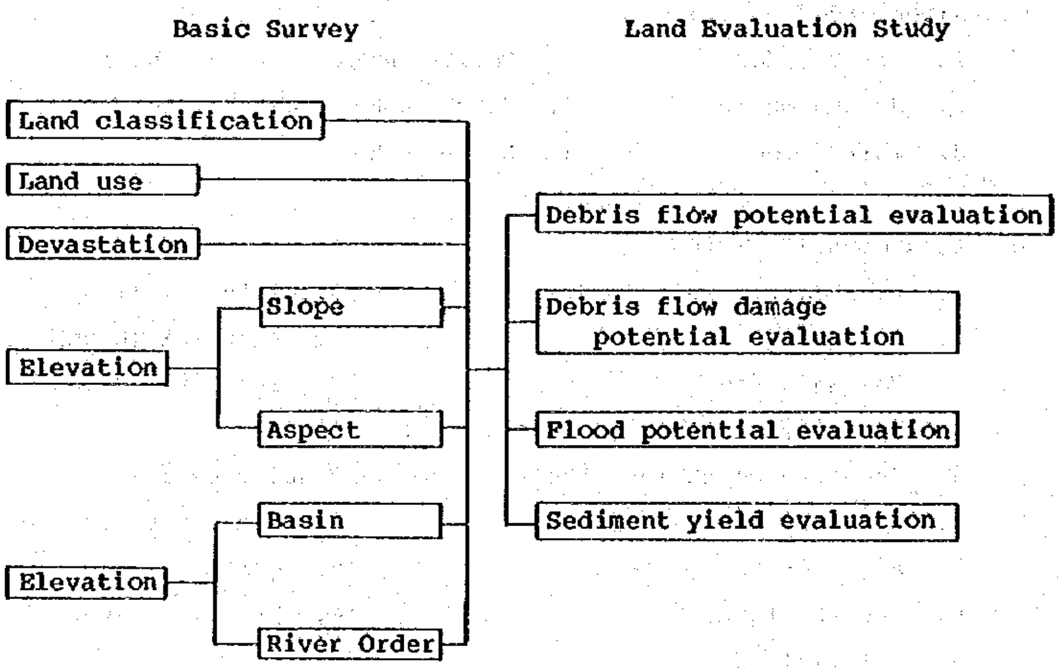
III. LAND EVALUATION

3.1 Methodology

A series of evaluation maps are developed by overlaying the topographic analysis data on the basic data such as land classification, land use and devastation.

The flow chart of the land evaluation study is shown below.

FLOW CHART OF THE LAND EVALUATION STUDY



The basic environmental variables applied for the land evaluation study are listed as follows:

BASIC ENVIRONMENTAL VARIABLES FOR LAND EVALUATION

A. Topographic elevation

B. Slope

1. 0 - 2%	5. 21 - 30 %	9. 76 - 100 %
2. 3 - 5	6. 31 - 40	10. more than 101
3. 6 - 10	7. 41 - 50	
4. 11 - 20	8. 51 - 75	

C. River Order

1. First order	4. Fourth order	7. None
2. Second order	5. Fifth order	
3. Third order	6. Sixth order	

D. Land classification

1. Lava (new)
2. Lava (old)
3. Volcanic fan
4. Tertiary mountain (Volcanics)
5. Tertiary mountain (Limestone, standstone,volcanics)
6. Tertiary mountain (Dirite)
7. High Terrace plain
8. Natural levee
9. Alluvial (fan)
10. Alluvial (low)
11. Alluvial (high)
12. Fan-like lowland
13. Bar
14. Old river terrace
15. Marsh
16. Existing river, lake

E. Devastation

1. Collapse (Productive)
2. Collapse (Less productive)
3. Bare land
4. Lava flow
5. Unstable sediment in transport zone
6. Moderately unstable sediment in transport zone
7. Stable sediment in transport zone
8. Unstable sediment in sedimentation zone
9. Moderately unstable sediment in sedimentation zone
10. Stable sediment in sedimentation zone
11. Debris flow (old)
12. Debris flow (new)

3.2 Debris Flow Potential Evaluation

As environmental variables concerned with mud flow potentials, River order, devastation, land classification and slope are considered. Each component of the above variables is weighted as follows depending on the magnitude of debris flow occurrence potential expected.

The next table shows the weights given to the respective variables as applied in the debris flow potential evaluation.

WEIGHTS FOR DEBRIS FLOW POTENTIAL EVALUATION

Weight / Environmental variable	5	4	3	2	1	Off
River Order	1,2	3			4,5,6	
Land classification	1	2,4			3,8,9, 10,11,12, 13,14,15, 16	
Devastation	1,5,8	3,6,9	2,4 7,10		11,12	
Slope						Under 9%
	High		Debris flow potential			Low

The debris flow potential is defined by 5 levels and its areal distribution in the project area is summarized as shown below.

AREAL DISTRIBUTION OF DEBRIS FLOW POTENTIAL

Debris Flow Potential	Number of cells (nos.)	Area (km2)	Percent (%)
I (low)	44,757	447.57	62.1
II	17,432	174.32	24.2
III	5,679	56.79	7.9
IV	2,439	24.39	3.4
V (high)	1,733	17.33	2.4
Total	72,040	720.40	100.0

3.3 Debris Flow Damage Potential Evaluation

River order, devastation, land classification and slopes are considered as environmental variables related to debris flow damage potentials. Each component of the above variables is weighted as follows depending on the magnitude of debris flow damage potential expected.

The table below shows the weights given to the respective variables as applied in the debris flow damage potential evaluation.

WEIGHTS FOR DEBRIS FLOW DAMAGE POTENTIAL EVALUATION

Weight / Environmental Variable	5	4	3	2	1	Off
River Order	1,2	3,4		5	6, 7	
Land classification	1,2,3	4,5,6	9	7,8,12	10,11,13 14,15,16	
Devastation	1,5,8 11,12	3,6,9	2,4 7,10			
Slope						Under 5%

High Debris flow Damage potential Low

The debris flow potential is defined by 5 levels and its areal distribution in the project area is summarized as shown below.

AREAL DISTRIBUTION OF DEBRIS FLOW POTENTIAL

Debris Flow Potential	Number of cells (nos.)	Area (km2)	Percent (%)
I (low)	40,384	403.84	56.1
II	20,140	201.40	27.9
III	5,092	50.92	7.1
IV	4,232	42.32	5.9
V (high)	2,192	21.92	3.0
Total	72,040	720.40	100.0

3.4 Flood Potential Evaluation

As environmental variables that relate to the flood potential evaluation, river order, land classification and slope are considered. Each component of these variables is weighted as follows depending on the magnitude of flood occurrence potential expected.

The table below shows the weights given to the respective variables as applied in the flood potential evaluation.

WEIGHTS FOR FLOOD POTENTIAL EVALUATION

Weight / Environmental Variable	5	4	3	2	1	Off
River Order	6,5	4		3	1,2,7	
Land classification	10,14 15,16	3, 9 11,12	8,13	7	1,2,4 5,6	
Slope	0-2%	3-5%			6% -	
	High		Flood potential			Low

The flood potential evaluation is defined by 5 levels and its areal distribution is summarized as shown below.

AREAL DISTRIBUTION OF DEBRIS FLOW POTENTIAL

Debris Flow Potential	Number of cells (nos.)	Area (km2)	Percent (%)
I (low)	30,781	307.81	42.7
II	15,585	155.85	21.7
III	19,320	193.20	26.8
IV	5,041	50.41	7.0
V (high)	1,313	13.13	1.8
Total	72,040	720.40	100.0

3.5 Sediment Yield Evaluation

From devastation map, the sediments yield is calculated for each major basin. The sediment yield is defined as the amount of sediment expected to runoff from land collapse plus the amount of unstable sediment in the stream.

The depth of collapse and sediment are estimated from the field survey and aerial photo interpretation. Sediment yield are the amounts expected to be produced over a period of 50 years.

Sediments from land collapse and unstable sediments in the stream are given in TABLE-III.1 and III.2 respectively. Estimated sediment yield is summarized in TABLE-III.3 and the basins evaluated are listed in TABLE-III.4.

IV. SABO PLANNING

4.1 Base Point and Sub-base Point

Base point and sub-base point are the points where the sediment run-off volume, excess sediment volume, allowable sediment volume, etc. are to be estimated for Sabo planning. The type, scale and location of Sabo facilities are determined based on the sediment amounts at base points or sub-base points.

In the Quinali (A) River basin, the Quirangai, the Tumpa, the Maninila, the Masarawag, the Nasisi and the Ogson (Nabonton Creek) Rivers are selected as a river to be designed. The base points and sub-base points of the respective rivers are shown in FIG.-IV.1.

In the Quinali (B) River basin, the upstream reach from Tabigyan is taken up for sabo planning and the base point and sub-base points are set at the locations as shown in FIG.-IV.2.

Also, in the Yawa River basin, three rivers of the Anuling, the Budiao and the Pawa-Burabod Rivers are subject to sabo planning. Their base points and sub-base points are shown in FIG.-IV.3

4.2 Sediment Run-off Volume

No sediment observation record for Sabo planning is available within the project area and also other river basins in the Philippines do not have any useful data on sediment run-off for Sabo planning. Therefore the empirical formula proposed by Drs. Ashida and Okumura is employed in this study. The formula was derived from analyzing the data over about 40 years in Japan.

It is expressed in the form of

$$D = C \cdot (A \cdot RD \cdot I_{200})^2$$

where,

D : sediment run-off volume in m^3

A : drainage area in km^2

Rd: daily rainfall in mm

I_{200}: average river bed slope from the point concerned up to the point 200 m higher in altitude.

C : coefficient

The coefficient is usually taken as 10, but it tends to give the excessive sediment volume. Therefore, the values of 10, 7 and 4 are properly used depending on the basin conditions in this study.

The sediment run-off volume calculated by the above formula for the respective rivers are summarized in TABLE-IV.1 and TABLE-IV.2 and shown in FIG.-IV.4 through FIG.-IV.8 diagramatically.

4.3 Excess Sediment Volume

Excess sediment volume is defined as the sediment amount given by subtracting allowable sediment volume from the sediment run-off volume. The allowable sediment volume means the sediment amount to be released to maintain the lower river channel equilibrium or to be able to release downstream without no problems.

The excess sediment volume is the objective sediment volume for Sabo planning and it is defined by the allowable sediment volume as mentioned above. It is however very difficult to determine the allowable sediment volume right fully by calculation because it is closely concerned to the sedimentation in the downstream river course.

In this study, the bed load at base point under the present river conditions is taken as an allowable sediment volume in Sabo planning supposing that the bed load said above is allowable to the downstream.

Drs. Ashida, Takahashi and Mizuyama's formula is employed to calculate the bed load amount. It is expressed by the form of

$$\frac{q_B}{\sqrt{(\sigma/\rho-1)\cdot g\cdot d^3}}=\frac{12-24\sqrt{I}}{\cos\theta}\cdot\tau_*^{\frac{3}{2}-\sqrt{I}}\cdot\left(1-\alpha^2\frac{\tau_{*c}}{\tau_*}\right)\cdot\left(1-\alpha\sqrt{\frac{\tau_{*c}}{\tau_*}}\right)$$

where,

- τ_* : non-dimensional tractive force
- τ_{*c}: non-dimensional critical tractive force
- I : river bed slope
- σ: specific gravity of grain (=2.6)
- ρ: water density (=1.0)
- g : gravity acceleration
- θ: river bed gradient
- d : grain diameter
- α : coefficient (=0.92)

The grain size distribution of river bed materials is made based on the sieve analysis of the grain of less than 50.8 mm in diameter and supplementary the cross line method survey of the grain between 50.8 mm and 1,000 mm in diameter.

The bed load amount for the design flood with a return period of 50-yr is estimated for the subject rivers and the results are listed in TABLE-IV.2.

Finally, the excess sediment volume, that is, the objective sediment volume for Sabo planning are determined as shown in TABLE-IV.3.

4.4 Sediment Control Plan

The sediment control plan should be established so as to control and check the sediment yield and runoff with arranging the proper Sabo facilities in order to redsuce the excess sediment volume at base point as much as possible.

4.4.1 Quinali (A) River Basin

The sediment control plans for six (6) tributaries are explained as below. FIG.-IV.4 and FIG.-IV.5 show the sediment balance diagrams for without- and with-project conditions.

(1) Quirangay River

The excess sediment volume of the Quirangay River is 1,77,500 m^3 at base point. The natural sand retarding capacity is 28,800 m^3 in the No.3 basin(*) and 65,600 m^3 in the No.4 basin as shown in FIG.-IV.1.

Arranging six (6) spur dikes and three (3) jetties in the No.4 basin, the sand retardsing capacity is increased from the present 65,600 m^3 to 247,500 m^3 with a retarding area of about 165,000 m^2 and an average retarding depth of 1.5 m.

Consequently, the sediment run-off at base point No.4 will be reduced from 260,100 m^3 in without-project condition to 78,200 m^3 (260,100 - (247,500 - 65,600) = 78,200), which is lower than the allowable sediment volume of 82,600 m^3.

(2) Tumpa River

The excess sediment volume of the Tumpa River is 8,500 m^3 at base point and it is not too large to control. With executing fifteen (15) consolidation works using coconut trunk in the No.2 basin, the longitudinal and lateral erosions in the river course are checked, of which volume will amounted about 16,800 m^3.

(*) Drainage basin between sub-base point No.2 and sub-base point No.3

Finally, the sediment run-off at base point will be redsuced from 43,700 m^3 in without-project condition to 26,900 m^3 which is lower than the allowable sediment volume of 35,200 m^3.

(3) Maninila River

The excess sediment volume of the Maninila River is 57,300 m^3 at base point and it is due to the secondary erosion in the downstream reach. Accordingly, nine (9) consolidation works with crib are planned in the No.4 basin to mitigate the longitudal and lateral erosion. The sediment yield of 57,300 m^3 will be checked with these facilities.

The sediment run-off at base point will be redsuced from 94,000 m^3 in without-project condition to 42,600 m^3 which is more than the allowable sediment volume of 36,700 m^3. However, as the excess of 5,900 m^3 will be disappeared by the sand retarding function of consolidation works.

(4) Masarawag River

The excess sediment volume of the Masarawag River is 199,200 m^3 and it is relatively large. The Masarawag River branches into two streams at EL. 280 m and those two streams confluence again at EL. 140 m.

For Sabo plan, the main stream presently in the No.3 basin is led into the No.2 basin having wider sand retarding area. The sand retarding capacity of the No.2 basin is satisfactorily enough to retard almost all the sediment run-off from the upper reach and thus only the bed load will through the No.2 basin. Its amount is estimated at 58,300 m^3 assuming 2% of bed load concentration ($5.7\ km^2 \times 511\ mm \times 10^3 \times 0.02 = 58,524\ m^3$). Accordingly, the sand retarding volume in the No.2 basin is expected to be 300,100 m^3 (358,400 - 58,300 = 300,100) providing the proper Sabo facilities. Then, the river channel in the No.4 basin is unstable and partialy scoured. It should be stabilized.

For the No.2 and No.3 basin, twelve (12) spur dikes and three (3) jetties are arranged and for the No.4 basin the consolidation works with crib are planned at nine (9) sites. With these Sabo facilities, the sediment run-off at base point will be reduced from 276,800 m^3 in without-project condition to 65,300 m^3 which is lower than the allowable sediment volume of 77,600 m^3.

(5) Ogsong River

The excess sediment volume of the Ogsong River is 107,800 m^3 at base point. Only the No.2 basin has an enough sand retarding area, but the natural sand retarding can not be expected so much under the present river condition. The above sand retarding area is about 700,000 m^2 and a large sand retarding capacity will be created with the proper arrangement of spur dikes and jetties. In this plan, six (6) spur dikes and nine (9) jetties are arranged in the No.2 basin.

The sediment passing through the No.2 basin may be a bed load and its amount is estimated at 41,400 m^3 assuming 1.5% of bed load concentration (5.4 km^2 x 511 mm x 10^3 x 0.015 = 41,391).

Then, the natural sand retarding amount in the No.3 basin in 3,900 m^3 and also the amount of about 9,000 m^3 will be retarded with two (2) consolidation works with cribs and three (3) ground-sills.

Finally, the sediment run-off at base point will be reduced from 140,500 m^3 in without-project condition to 28,500 m^3 (41,400 - 3,900 - 9,000 = 28,500) which is lower than the allowable sediment volume of 32,700 m^3.

(6) Nasisi River

The excess sediment volume of the Nasisi River is 1,042,800 m^3 at base point and it is very large compared with other rivers. The sediment control plan for this river is described below from upstream to downstream.

Two (2) consolidation dams combined with spur dike are constructed in the No.3 basin because it is too difficult to constructed the Sabo facilities in the No.1 and No.2 basins. With these dams, the river channel erosion of 184,900 m^3 is checked and controlled and the sand retarding of 485,000 m^3 is expected. Consequently, the sediment run-off at sub-base point No.3 will be reduced from 1,001,400 m^3 in without-project condition to 331,500 m^3.

Then, three (3) spur dikes and ten (10) groins are arranged in the river course between sub-base point No.3 and No.7, and also the levee of 2,350 m long is constructed on the right bank. With these works, a big sand retarding basin with an area of more than 400,000 m^2 is created and the sand retarding of 506,200 m^3 is expected as well as checking of erosion. The sediment run-off at sub-base point No.7 will be sharply reduced from 1,528,600 m^3 in without-project condition to 21,000 m^3.

The intake weir of National Irrigation System about 500 m downstream from sub-base point No.7 is functioning as a consolidation work and then the ovaerflow culvert of Ligao-Tabaco National Highway is also functioning as a consolidation work. Accordingly, the sediment control of 29,700 m^3 is expected by reinforcing the said culvert.

The sediment run-off at base point will be finally reduced from 1,128,700 m^3 in without-project condition to 107,400 m^3 which is a little over the allowable sediment volume of 85,900 m^3. However the excess can be liquidated by the consolidation works just downstream from base point.

4.4.2 Quinali (B) River Basin

In the Quinali (B) River basin, the rivers subject to the plan are the Buang River and the Quinli (B) River, main course, downstream from junction with the Buang River.

The sediment control plans are prepared as explained below and the sediment balance diagram is shown in FIG.-IV.6 for without- and with-project conditions.

The excess sediment volume of the Quinali (B) River is 176,000 m^3 at base point. With arranging four (4) consolidation works in the Buang River, the sediment of 144,000 m^3 will be controlled and thus the sediment inflow to the Quinali (B) River, main course, is estimated at 67,800 m^3. Also the Sabo dam of about 8 m high proposed on the main course will retard 56,000 m^3 without taking into account the sediment yield from river bed because the upstream reach from dam is in the sediment deposition zone.

After all, the sediment run-off at base point will be redsuced from 319,700 m^3 in without-project condition to 119,700 m^3 which is lower than the allowable sediment volume of 176,000 m^3.

4.4.3 Yawa River Basin

The sediment control plans are established for three (3) tributaries, that is, the Anuling River, the Budiao River and Pawa-Burabod River. The proposed plans are explained below and the sediment balance diagrams are shown in FIG.-IV.7 and FIG.-IV.8 for without- and with-project conditions.

(1) Anuling River

The excess sediment volume of the Anuling River is 329,800 m^3 at base point. Five (5) consolidation works with crib and spur dike are constructed in the narrow river channel and the sediment yield of 221,000 m^3 is checked in the No.1 basin and 126,000 m^3 in the No.2 basin.

Consequently, the sediment run-off at base point will be reduced from 415,600 m^3 in without-project condition to 68,600 m^3 which is lower than the allowable sediment volume of 85,800 m^3.

(2) Budiao River

The excess sediment volume of the Budiao River is 176,500 m^3 at base point. In the No.2 basin, there is a vast natural sand retarding basin formed by debris flow. Eight (8) spur dikes and six (6) jetties are arranged there in order to retard the sediment more widely and more thinly.

As the area of retarding basin is 531,000 m^2 and it is enough large to control the sediment run-off, the sediment passing through the No.2 basin will be almost a bed load. Assuming 1.5% of bed load concentration the amount is estimated at 38,000 m^3 ($3.2\ km^2 \times 793\ mm \times 10^3 \times 0.015 = 38,064$).

The sediment run-off at base point will be 51,800 m^3 adding the erosion of 13,800 m^3 in the No.3 basin.

Namely, the sediment run-off at base point will be reduced from 234,600 m^3 in without-project condition to 51,800 m^3 which is lower than the allowable sediment volume of 58,100 m^3.

(3) Pawa-Burabod River

The excess sediment volume of the Pawa-Burabod River is 371,400 m^3 at base point.

One (1) sabo dam and four (4) consolidation works are planned in the No.1 basin to reduce the sediment run-off and the amounts to be controlled are 63,800 m^3 and 140,000 m^3 respectively. With these facilities, the sediment run-off at sub-base point No.1 is decreased to 913,800 m^3.

Then, seven (7) spur dikes are arranged in the No.2 basin and the sediment of 855,000 m^3 is retarded there at a retarding area of 712,500 m^2 and an average depth of 1.2 m. With this, the sediment run-off at sub-base point No.2 will come to 58,800 m^3 and that at base point will be reduced from 440,900 m^3 in without-project condition to 58,800 m^3 which is lower than the allowable sediment volume of 69,500 m^3.

V. RIVER PLANNING

5.1 Estimate of Inundation Area, Depth and Duration

In order to assess the damages caused by flood, the inundation area, depth and duration are roughly estimated for the different flood magnitudes. The estimation is made for both of the case without and with project. The estimation methods for the respective basins are explained below.

5.1.1 Quinali (A) River basin including the Talisay River

The present inundation area, depth and duration in the basin are summarized in FIG.-V.1. As seen in the figure, the inundation in this area extends over almost the whole plain regardless of the ground elevation. This means that the flood water overflew the levee due to a lack of discharge capacity or flushed out due to a failure of levee is dammed up with roads, railway, etc. at various sites and thus the inundation occurs even at the plain with relatively high altitude as well as the low plain. On the other hand, the inundation area directly affected by Lake Bato is not so large in area.

Accordingly, the criteria for calculation are constructed as follows considering the topographic conditions, the present situation of inundation, etc.

(1) Inundation extends over the present inundation area whenever the inundation occurred.

(2) For the present river conditions, the flood water drained through the Quinali (A) River does not contribute the inundation.

(3) For the improved river conditions, the flood water drained through the Quinali (A) and the Talisay Rivers does not contribute the inundation.

(4) Inundation is not caused by the flood with peak discharge of less than the design discharge for river planning.

(5) Inundation depth is calculated assuming that the flood water inundates uniformly over the inundation area.

(6) Duration of the inundation is roughly estimated based on the actual inundation depth and duration without calculation.

(7) Estimation is made based on the volumetric water balance instead of hydraulic calculation.

(8) Runoff coefficient of flood is 0.7 in volume.

(9) Duration of rainfall is 3-day.

The results are summarized in TABLE-V.1.

5.1.2 Quinali (B) River Basin

The present inundation, depth and duration in the basin are shown in FIG.-V.2. The inundation in this area occurs under the conditions basically similar to the Quinali (A) River basin, but the inundation depth is expected not to increase proportionally with increasing of rainfall amount. The 1-day rainfall will be dominant for the inundation.

In addition, this area has two flooding area as seen in the figure. The flooding occurs due to a lack of discharge capacity of channel.

The criteria for calculation are determined as follows.

(1) Inundation extends over the present inundation area whenever the inundation occurred.

(2) The flood water drained through the Quinali (B) River does not contribute the inundation.

(3) The inundation is assumed to be not caused by the flood with peak discharge of less than the design discharge for river planning.

(4) Inundation depth is calculated assuming that the flood water inundates uniformly over the inundation area.

(5) Duration of the inundation is roughly estimated based on the actual inundation depth and duration without calculation.

(6) Estimation is made based on the volumetric water balance instead of hydraulic calculation.

(7) Runoff Coefficient of flood is 0.7 in volume.

(8) Duration of rainfall is 1-day.

(9) Flooding depth is calculated by Manning's formula for the peak runoff.

The results are summarized in TABLE-V.2 through TABLE-V.4.

5.1.3 Yawa River Basin

The Yawa River Basin has not so inundated so far. However, the flood runoff will increase with development of the basin and the heavy damages may be caused by a failure of levee in the future. In this study, supposing a failure of levee as shown in FIG.-V.3 the flooding area and depth are estimated with hydraulic calculation by Manning's formula. But, the levee failure is not caused by the runoff of less than the design discharge for river planning. Also, the flooding duration is estimated at less than one day considering the topographic conditions in the basin, etc.

The results are summarized in TABLE-V.7 and TABLE-V.8.

5.2 Study of Sedimentation

5.2.1 Flood Sediment Volume

Flood sediment volume means a sediment volume transported by one flood, which is very closely concerned to Sabo planning.

In this study, the flood sediment volume in the river work section is calculated by Einstein's formula for the flood hydrograph with a return period of 50-year which is prepared by Nakayasu's formula. The grain size is given by the grain size distribution curve based on the sieve analysis results of river bed materials.

The results are as follows.

FLOOD SEDIMENT VOLUME FOR 50-YEAR PROBABLE FLOOD

Name of River	Station	Flood Peak Runoff (m3/sec)	Flood Sediment Volume (m3)
Quinali (A)	Sta. 6 + 000	3,013	9,000
"	Sta.23 + 000	1,725	8,000
Nasisi	Sta. 1 + 000	840	3,400
Talisay	Sta. 2 + 000	1,938	4,900
Quinali (B)	Sta. 1 + 000	2,383	2,800
Yawa	Sta. 0 + 000	2,142	2,300

5.2.2 Sediment Balance

It is important to examine the sediment balance in the river course of the rivers which need Sabo works in their upper reaches.

In this study, the Quinali (A) River, main course, the Nasisi River, the Talisay River, the Quinali (B) River and the Yawa River are examined for the probable floods with return periods of 50-year, 10-year and 1.01-year by using Einstein's formula and sieve analysis results of river bed materials in the river work sections. The results are shown in TABLE-I.6 to TABLE-I.10.

From the tables, the followings are noted. The sediment transportations in the rivers except the Nasisi River and the Quinali (B) River are roughly balanced taking into account the acuracy of the present sediment calculation.

The Nasisi River and the Quinali (B) River have sediment depositions in their upper reaches according to calculation and this fact is substantiated with the field survey.

VI. AGRICULTURAL BACKGROUND

6.1 General

To investigate the present agricultural conditions in the project area, sample survey was conducted 3 times, from September to October in 1979, from November to December in 1979 and from July to August in 1980. Major parts of the agricultural information are obtained through the questionnaires distributed to each barangay situated in the project area. Out of 295 questionnaires distributed, 239 questionnaires corresponding to 81% were collected. All the survey items in the questionnaires were not filled out. The information obtained about farm household, land holding, land tenure, farm input and rice yield is processed statistically.

6.2 Natural Condition for Agriculture

6.2.1 Soil

The interior alluvial plain is predominated by rice field. Most of the steep lands are planted with coconuts, citrus and abaca. Lava areas are occupied by tall grasses.

Most of the soils in the alluvial plain except those developed under the influence of the Talisay River are formed in a recent alluvium derived mainly from pyroclastic materials. The soils developed under the influence of the Talisay River is derived from the clastic deposit and limestone and is often calicaneons.

The soils in the alluvial plain are mostly well suited for irrigated rice cultivation. They are deep and have moderate to high natural fertility. Surface soils are silty clay to loam and are easily puddled. The soils in the south of the Talisay River are very clayey and are marginally or not suited for upland crops because of soil tillage problems and relatively high groundwater level. The soils in the piedmont area are sandy loam in the surface soil and have moderate to locally low fertility. The soils in or near valleys of Mayon Volcano are very acid with pH values of 4.5 to 6.2 in the surface soil and 3.3 to 4.1 below about 60 cm deep.

6.2.2 Water Resources for Irrigation

The water resources of existing irrigation systems almost depend on surface water of the major rivers and their tributaries in the project area and are summarized in TABLE-VI.1.

The average seasonal stream flow at the Quinali Gaging Station installed on the middle reach of the Quinali (A) River is estimated to be 8.10 m^3/sec in the wet season (June - January) and 5.75 m^3/sec in the dry season (February - May). There is no available data on stream flow for the Quinali (B) river. On the basis of the data on stream flow for the Quinali (A) River, therefore, the average seasonal stream flow at Bantayan is estimated at 1.78 m^3/sec in the wet season and 1.03 m^3/sec in the dry season. The major water resources of existing irrigation systems in the Yawa River basin and the East and North-East area of Mayon Volcano are spring water produced constantly in the slope of Mayon Volcano.

6.2.3 Water Quality

As for the water quality of the Quinali (A) River and its tributaries, there are available data such as results of chemical analysis and electric conductivity tests, as shown in TABLE-VI.2, although these data have been taken at a few sites on the Quinali (A) River and its tributaries.

The evaluation of water quality is carried out by referring these data to the standard of the U.S Salinity Laboratory, especially, in view of total concentration of soluble salts, sodium adsorption ratio (SAR), and concentration of boron. In result, the surface water can be used for irrigation without any harm.

6.3 Socio-Economic Background

6.3.1 Farm Household

The total number of household in the barangay is obtained from the sensus in 1980. The ratio of the number of farm household to the number of the total household is obtained through or estimated based on the questionnaires. As shown in TABLE-VI.3, the total number of the farm household in the project area in 1980 is estimated at 39,190.

6.3.2 Land Holding and Land Tenure

Land holding and land tenure of the agricultural land in the project area are estimated in the same way as that for the farm household. The results are presented in TABLE-VI.4 and VI.5.

The average farm size is about 1.2 ha including rice field, upland field, coconut field, abaca field and citrus field, while about 60% of the total farm household has an area of less than 0.9 ha. In respect to land tenure, only 18% of the farm household are ownercultivator. The remaining 17% is amortizing farmer, 45% is leaseholder, 16% is sharetenants and 4% is mixed tenure.

The share-cropping arrangement or rent for rice cultivation in the project area can be divided into two systems, i.e. sharing system and leaseholder system. In the sharing system, tenants share normally about 50-60% of the total farm products and 100% of the production costs. The leaseholder system has the fixed rent which is assessed according to the law at less than 25% of the average normal harvest for the past three crop years preceding the date that the leasehold is established. The rent is paid in cash or kind after deducting the amount used for seeds and the costs of harvesting, threshing, loading, hauling and processing whichever is applicable.

6.4 Agricultural Production

6.4.1 Land Use

The present land use in the project area was surveyed by interpretation of aerial photography taken in 1980 with a scale of 1:25,000 and by ground check. The results are presented in TABLE-VI.6.

The agricultural land is 44,712 ha consisting of 21,714 ha of rice field, 1,879 ha of upland, 20,866 ha of coconut field, 131 ha of abaca field and 122 ha of citrus field. The national and communal irrigation systems cover about 2,400 and 10,000 ha of rice field respectively. The remaining rice fields are mostly cultivated under rainfed condition.

The non-agricultural land totals 25,190 ha including 10,182 ha of forest and 7,720 ha of grass land. Due to the limitations such as slope and soils, however, these areas are mostly not suitable for further reclamation to arable land.

6.4.2 Cropping Pattern of Rice

The cropping pattern of rice in the project area is restricted by the rainfall pattern. Normally, the start of the land preparation such as puddling coincides with the start of the wet season. Typhoon and flood seasons also influence the cropping patterns of rice. The rice cultivation areas are grouped into three areas according to the cropping patterns, i.e., Quinali (A) River basin, Quinali (B) River basin and Lake Bato flood areas. The Quinali (A) River basin covers the Eastern part of Mayon Volcano and the Quinali (A) River flood plain. In this area, double cropping of rice, as shown in FIG.-VI.1, is practiced. The sowing of wet season rice starts in April and ends in June. The harvest starts in August and ends in October. The sowing for dry season rice cultivation starts in November and ends in January. The harvest season is from March to early May.

The Quinali (B) River basin covers the Southern and Eastern parts of Mayon Volcano and the Quinali (B) River flood plain. In this area, the rice cultivation starts one month earlier than in the Quinali (A) River basin because of more rainfall in the dry season. The double cropping of rice, as shown in FIG.-VI.2, is commonly practiced in this area. The sowing of wet season rice starts in late March and ends in May. The harvesting starts in August and ends in September. The dry season rice is sown from October to November and is harvested from late February to early April.

About 1,000 ha adjacent to Lake Bato is severely flooded every year, where only single rice cultivation is practiced. The sowing of rice is done in April and harvesting is from late July to August.

6.4.3 Farming Practices and Farm Input in Rice Cultivation

Farming practices are partially mechanized. Plowing is done for most cases by carabao. Puddling is done by carabao or hand-tractor attached with puddling wheels. According to the information from the provincial BAEx (Bureau of Agriculturl Extension) officials, the number of the hand-tractors in Albay Province in 1980 is 769, of which 622 are possessed in 4 municipalities situated in Quinali (A) River basin, namely, Libon, Oas, Polangui and Ligao. Weeding is done mostly by hands. Herbicides and rotary hand weeders are used subsidiary.

The compressed air-type sprayer is mostly used for the pest and disease control. The number of the sprayer in Albay Province in 1980 is 3,111, of which 1,313 are in the said 4 municipalities. Threshing is done for most cases by thresher. The number of the threshers in Albay Province in 1980 is 183, of which 114 are in the said 4 municipalities.

Improved high-yielding varieties such as IR-36, IR-42 are the major varieties of rice in the project area. Planting density is 20 cm x 20 cm in most cases.

The survey of the amount of farm input is made on the rice, because fertilizer application is the common agricultural practices for rice in the project area. The amounts of seed and fertilizers were surveyed by the questionnaires. Other items were surveyed by interviews with extension workers of the BAEx. The results of the survey by the questionnaires are presented in TABLE-VI.7. The average uses of fertilizers in the Quinali (A) River basin are 76 kg N/ha, 16 kg P_2O_5/ha and 8 kg K_2O/ha, and those in the Quinali (B) River basin are 67 kg N/ha, 19 kg P_2O_5/ha and 6 kg K_2O/ha.

6.4.4 Rice Yield and Production

Irrigated rice yields in 1979 and 1980 were surveyed by the questionnaires in the project area. The sampled area, their production and the yield of each municipality are presented in TABLE-VI.8. The average yield of palay in the sample area situated in the Quinali (A) River basin was 4.0 tons/ha in the dry season in 1979, 3.5 tons/ha in the wet season in 1979 and 3.6 tons/ha in the dry season in 1980. That in the Quinali (B) River basin is 3.4 tons/ha in the wet season, 1979 and 3.3 tons/ha in the dry season, 1980. The yield in the dry season in 1979 was 3.6 tons/ha according to the report on Masagana-99, Phase XII (November, 1978 to June, 1979). The average yield of palay in all sample areas is estimated at 3.5 tons/ha in the dry season and 3.2 tons/ha in the wet season. It is considered that the high yield of palay in 1979 can attribute to high dosage of fertilizers, the well extension services and flood-free condition.

Taking into consideration the actual harvested area, annual production of palay obtained from the project area is estimated at about 70,900 tons, which is 1.3 times of the estimated consumption of palay in the project area. The results of estimation are summarized in TABLE-VI.9.

Damages to rice by pests and diseases are slight in the project area. The affected rice field by pests in Albay province in 1979 were 2,400 ha by rodents, 300 ha by stem borer, 240 ha by whorl maggot and 80 ha by case worm out of 26,000 ha rice field. Damages by diseases were very little. Major rice diseases are tungro, leaf spot and bacterial leaf blight diseases. Farmers apply insecticides normally 3 to 4 times against stem borer and whorl maggot for a preventive measure.

6.4.5 Other Crop Production

Coconut is also the most important crops in the project area. Coconut is planted in upland field with deep soil and good drainability. Major upland crops in the project area are maize, cassava, sweet potato, vegetables, banana and citrus.