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REPUBLIC OF THE PHILIPPINES
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THE PANAY RIVER BASIN WIDE FLOOD CONTROL STUDY

SUPPORTING REPORT

APPENDIX I METEOROLOGICAL AND HYDROLOGICAL STUDY

NOVEMBER 19[illegible]

JAPAN INTERNATIONAL COOPERATION AGENCY

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MINISTRY OF PUBLIC WORKS AND HIGHWAYS

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FOR

FINAL REPORT

ON

THE PANAY RIVER BASIN-WIDE

FLOOD CONTROL STUDY

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APPENDIX I METEOROLOGICAL AND HYDROLOGICAL STUDY

TABLE OF CONTENTS

Page

APPENDIX I

LIST OF TABLES

APPENDIX I

LIST OF FIGURES

1. Meteorological Condition

1.1 General

There are the following bodies concerned in the meteorological observation in the basin including the surrounding area.

(A) PAGASA (Philippine Atmospheric, Geophysical and Astronominal Services Administration)

There are three PAGASA observatories (i.e., Roxas City, Matec and Astorga) in the basin and about 13 observatories in the outside of the basin but in Panay island. The observatory at Roxas City is classified as SYNOP (Synoptic) station where observation of almost all meteorological elements are made at fixed observation time and are transmitted to the central office in Manila.

The other two station at Matec and Astorga are classified as AGRO (Agro-meteorological) stations which gather and provide on a routine basis simultaneous meteorological and biological information.

(B) NIA (National Irrigation Administration)

There is one project office of NIA in the basin, i.e., Mambusao irrigation system which has three raingage stations in the project area. But, there is a proposed irrigation project at Asue (Sara) which is located in the outside area neighboring on eastern boundary of the basin and observes not only rainfall but also evaporation.

(C) NPC (National Power Corporation)

NPC has one rainfall gaging station at Aglinab, Tapaz. But, it is informed that the observation was suspended in April 1985.

(D) PHILSUMA (Philippine Sugar Marketing Arms)

PHILSUMA (PHILSUCOM changed the name on May 18, 1985) has one sugar mill in the basin, i.e., Asturias mill which has 3 rainfall gaging stations in the vicinity of the mill. The rainfall data are also available from Pilar mill of which rainfall gages are located in the outskirt area of northeastern boundary of the basin.

(E) MPWH (Ministry of Public Works and Highways)

MPWH is generally not in charge of the meteorological observation. But they sometimes establish the observatories and keep observation for a Project if necessary. For the Panay river study, MPWH established 4 rainfall gaging stations in accordance with the request from JICA study team.

In the climate classification commonly used by PAGASA, the climate of Panay river basin is generally classified as Type III among four types, except that of the western mountainous area which is classified as Type I.

Type I has two pronounced seasons, that is, the wet season from May to October and the dry season from November to April. On the other hand, type III has no pronounced season but it is relatively wet from May to October and dry from November to April.

It is generally true that the Panay river basin has more or less such climatic characteristic. But, the actual climate in the basin cannot be shown exactly by only such types of rainfall.

In the following sections, summarized are the principal meteorological information in the basin such as data of rainfall, humidity, temperature, wind, evaporation and typhoon and their characteristics.

1.2 Rainfall

1.2.1 Gaging Stations

In the Panay river basin, 14 rainfall gaging stations exist at present (as of the end of December 1984) and another 4 stations existed in the past. In addition, there are some other rainfall gaging stations located outside of but close to Panay river basin.

However, the records of above mentioned gaging stations are generally not sufficient for the detailed rainfall analysis of the basin due to the short observation period and the location of the stations which are concentrated almost in the plain area except some newly established stations. The rainfall data during the floods caused by the typhoon "Openg" in November 1973 and the typhoon "Undang" in November 1984 seem to be most available among all the data for the flood runoff analysis.

The list of rainfall gaging station is shown in Table I.1-1. The period of record at rainfall gaging stations and the location of rainfall gaging stations in the basin are respectively shown in Fig. I.1-1 and Fig. I.1-2. In regard to the location of rainfall stations located outside of the basin, Fig. I.1-6 is to be referred.

1.2.2 Records

The daily rainfall record of the stations listed in Table I.1-1 are collected from the bodies concerned. Then the monthly rainfall records are compiled from the daily records. The only Roxas City station has the records of long period (from 1949 to present) in the basin. The other stations have the records of less than 10 years. The monthly rainfall records at Roxas City are representatively shown in Table I.1-2.

The records of monthly mean rainfall as well as yearly maximum and yearly minimum rainfall at the gaging stations not only in the basin but also outside of the basin (but in Panay island) are summarized in Table I.1-3. And the monthly mean rainfall at Roxas City is representatively shown in Fig. I.1-3.

The hourly rainfall records only for heavy rainfall periods at Roxas City, Iloilo, Balete, Pototan and Libacao are collected at PAGASA for studying the nourly distribution of neavy rainfall, though the periods observed by an automatic recorder are comparatively short except those at Roxas City and Iloilo. The hourly rainfall records of more than 30 mm at Roxas City from 1972-1979 are shown in Table I.1-4. The hourly distributions at Roxas City and Iloilo during the flood period in November 1973 are shown in Fig. I.1-4 and I.1-5 respectively.

The daily (2 times a day) rainfall records at the time of the past too biggest floods, that is, in November 1973 and in November 1984, at not only the stations in the basin but also at the other stations in the island are summarized in Table I.1-5 and I.1-6.

In regard to the heavy rainfall, the recorded extreme rainfalls of 1 day, 2 days and 3 days at 15 gaging stations in and around the basin are summarized in Table I.1-5 to I.1-7.

Additionally, the annual maximum rainfalls of 3 days at Roxas City and at five stations (Libacao, Balete, Barbaza, Valderrama, and Culasi) located outside of the basin are compiled in Table I.1-10 to I.1-15.

1.2.3 Isohyetal Map

Based on the annual mean rainfall at more than 25 gaging stations in the Panay island, the isohyetal map of Panay island is made as shown in Fig. I.1-6. The isohyetal maps made by PAGASA (for whole Philippines), NIA (for Panay island, shown in feasibility report on Jalaur river multi-prupose project) and NWRC (for Panay river basin, shown in Frame-works plan for Panay river basin) are referred for confirming the reliability of the estimated distribution of rainfall.

The isohyetal map of Panay river basin is made by just enlarging the map of Panay island as shown in Fig. I.1-7.

1.2.4 Features

The basin mean annual rainfall is calculated at 2,550 mm based on the isohyetal map. But the distribution of rainfall in the basin has a clear inclination, that is, an annual mean rainfall inclines from high in the west toward low in the east. In other words, annual rain-fall is as high as 3,500 mm at the westend area but it decreases gradually to the east and shows less than 2,000 mm in the south-eastern area of the basin.

In regard to monthly variability of rainfall, there is no apparent difference between dry season and wet season. But, it can be said as follows.

(a) From February to April, it is comparatively dry. The monthly rainfall in these periods is generally below 100 mm.

(b) From June to November, it is comparatively wet. The monthly rainfall in these periods is generally more than 200 mm.

(c) May, December and January are the transitional period. The monthly rainfall is generally between 100 mm and 200 mm.

The rainfall records at the time of the past biggest two floods, shown in Table I.1-8 and I.1-9, represent the following.

(a) Rainfall in November 1973 is very heavy. For example, the 3 days rainfall is over 250 mm in the plain and possibly over 500 mm in the western mountain area, though the rainfall in the western mountain area is estimated by the records of stations located outside of the basin.

(b) Rainfall in November 1984 is not so heavy in comparison with that in 1973. For example, the 3 days rainfall is about 100 mm in the plain area and less than 300 mm in the mountaneous area.

(c) The rainfall period is comparatively short in November 1984, while the rainfall in November 1973 continued more than a week, especially in the mountaneous area.

The annual maximum rainfall records of 3 days at 7 stations, shown in Table I.1-10 to I.1-16, are summarized as follows.

		Heaviest Rainfall (3 days)		
Station	Period	No. 1	No. 2	No. 3
Roxas City	35 years	396.8 mm	372.5 mm	305.5 mm
Libacao	11 years	427.9	356.1	298.9
Balete	28 years	445.8	344.4	322.3
Barbaza	27 years	625.1	593.1	554.6
Valderrama	22 years	665.4	450.9	398.7
Culasi	19 years	673.1	665.5	561.4
Iloilo	30 years	393.4	276.9	274.5

According to Table I.1-4, the maximum hourly rainfall in Roxas City is recorded at 93.1 mm (and the second, 75.7 mm) in 8 years records of 1972-1979. But, it is noticed that the daily rainfall is not always much when hourly rainfall is heavy in the day. For example, the maximum hourly rainfall during November 1973 flood is only 29.8 mm but the daily total of the day is much more than the daily total of other days which has heavier hourly rainfall.

1.2.5 Additional Records

During the study in the field (office in the Panay river basin), the study team collected the additional rainfall records besides the records directly used for the analysis. The additional records are the new records observed in 1984 at the new rainfall gaging stations established by the recommendation from JICA team. These new data are collected for the further study in the future and also for the reference to the study based on the analysis by the past records before 1984.

The following are the stations of the additional rainfall records.

(a) Lemery
(b) Jamindan
(c) West Villaflores
(d) Tapaz

The locations are shown in Fig. I.1-2.

The observation was carried out two times a day but in case of heavy and continuous rainfall it was requested to observe every one or two hours. The records are to be attached in Data book.

JICA study team requested MPWH (Roxas) to continue the observation and MPWH agreed to do so for the study of the next stage.

1.3 Temperature

In the basin, the temperature is recorded at 3 PAGASA stations, that is, Roxas City, Matec and Astorga. Among three stations, the records at Roxas City station are available for long period of 35 years but the other two stations have the records of only 3 or 4 years.

In Table I.1-17 and Fig. I.1-8, the records (1971-1983) of air-temperature at Roxas City are summarized. According to the table, the temperature at Roxas City has the following features.

(a) Annual mean temperature is 27.14°C

(b) Monthly variation in a year is not remarkable. But, it is relatively high from May to June (28 - 28.5°C) and relatively low in January and February (around 26.0°C).

(c) Daily variation is as small as 5 - 8°C on an average. That is, the daily temperature is changed between about 23.5°C and 29°C in cooler season and about 24°C and 32°C in hotter season.

(d) Any abnormal high and low temperature is not recorded in Roxas City. The highest is 38.5°C and the lowest is 16.5°C.

The meteorological records including the air temperature at Iloilo City are shown for the reference in Table I.1-18. And additionally, the meteorological data at Roxas City in 1983 (January - December) are separately compiled in Table I.1-19.

1.4 Humidity

The relative humidity in the basin is recorded also at three PAGASA stations. The monthly humidity at Roxas City is shown in Table I.1-17 and Fig. I.1-9.

The features of humidity at Roxas City is summarized as follows.

(a) The annual mean relative humidity is 80.85%.

(b) Monthly variation is not remarkable. But, it is relatively high (82-82%) from July to October and relatively low (77-79%) from March to May.

(c) The highest humidity is recorded at 100% and the lowest at 58%.

1.5 Evaporation

There is only one observatory for the observation of evaporation in the basin, that is, Matec (PAGASA) station. But, some other observatories for evaporation exist in Panay island.

In Table I.1-20 to I.1-22, the monthly evaporation records at Matec (Mambusao), Aguirre (Sara) and Cato-ogan (Pototan) are shown. The daily records at Matec and Aguirre are collected at PAGASA and NIA in Manila and are compiled to monthly records. But, there are not-a-few missing records in the original. Therefore, the revision of monthly evaporation is carried out by increasing the value in proportion to the ratio between the recorded days of the month. But, it seems that the records at Cato-ogan (Pototan) are more reliable for the study as the observation period is as long as 18 years and there is no doubtful value as far as checking the monthly records, though the location is outside of the basin. The records at Matec and Aguirre may be available for the reference. The graph of monthly mean evaporation at Pototan is shown in Fig. I.1-10.

The open-rim pan is used for the observation.

The features of evaporation at Cato-ogan is described as follows:

(a) Mean annual evaporation is 2,090 mm and the yearly variation is not so much, that is, within 10% from the mean (2,090 mm).

(b) Monthly variation is not remarkable. But, it is high (210 - 220 mm) from March to May and low (145 - 155 mm) from September to November.

The following are the reference description for estimating the reasonable runoff coefficient of the basin from the view point of evaporation.

(a) The actual evaporation from the water surface in the basin is estimated as follows.

$$2{,}090 \text{ mm} \times 0.7 \doteqdot 1{,}460 \text{ mm}$$

where; 0.7 is a coefficient generally used for a large open-rim pan.

While, the actual evapo-transpiration from the basin is estimated as follows.

$$2,090 \times 0.5 \doteqdot 1,050 \text{ mm}$$

where; 0.5 is roughly estimated coefficent. The evapotranspiration is generally 400 - 900 mm and the evaporation by open-rim pan is generally 900 - 1,500 mm in Japan.

(b) The annual runoff in the basin is estimated as follows.

$$2,600 - 1,050 = 1,550 \text{ mm}$$

where; 2,600 mm is annual mean basin rainfall.

(c) The reasonable runoff coefficient estimated from the evaporation records is calculated as follows.

$$1,550/2,600 \doteqdot 0.6 \ (60\%)$$

1.6 Wind

There are two observatories for wind in the basin, that is, at Roxas City (PAGASA) and Matec (PAGASA).

As the observation period at Matec is as short as 4 years, the summarized records at Roxas City from 1971 to September 1983 are compiled and shown in Table I.1-17 together with the other meteorological records. And additionally, the wind records together with the other meteorological records at Roxas City in 1983 are compiled separately in Table I.1-19 which is made from the original daily records.

The features of wind at Roxas City are described below.

(a) The maximum velocity of recent 13 years (until the end of 1983) at Roxas City is 83 knot/hour (43 m/sec).

Note: The wind velocity at the time of typhoon "Undang" passing by the Panay river basin on Nov. 5, 1984 is the strongest. In Roxas City, about 50 m/sec was recorded before the wind velocity meter was broken by the wind.

(b) The average wind velocity is 3 - 3.5 m/sec. The velocity is relatively high (4 - 4.5 m/sec) in December and November and low (2 - 3 m/sec) from July to October.

(c) There is no remarkable prevailing direction of wind as far as checking the original daily records.

The maximum wind velocity in Iloilo shown in Table I.1-18 is also available for the reference.

1.7 Typhoon

Typhoon records including tropical storm and depression are collected from PAGASA in Manila and compiled in the following tables and figures.

(a) Records of typhoon passing by Panay from 1948 to 1983, describing the date, name, maximum wind velocity, minimum atmospheric pressure, maximum 24 hours rainfall and daily rainfall at Roxas City during the period (Table I.1-23)

(b) Tracks of typhoon passing by Panay from 1949 to 1983 (Fig. I.1-11)

(c) Tracks of typhoon "Undang" (Fig. I.1-12)

(d) Number of typhoon from 1948 to 1983 (Table I.1-24)

(e) Monthly frequency of tropical cyclones crossing the Philippines (Table I.1-25)

(f) Monthly frequency of tropical cyclones crossing the Panay island (Table I.1-26)

The tropical cyclone (or typhoon as another general expression) are classified into the following three categories though the actual judgement of the categories is done by the forecaster of PAGASA, especially for tropical depression.

(A) Tropical Depression

The maximum wind speed within the center of the disturbance up to 63 km hour.

(B) Tropical Storm

The maximum wind speed within the disturbance ranges of 64 - 118 km per hour.

(C) Typhoon

The maximum wind speed within the distrubance exceeds 118 km per hour.

The features of typhoon especially passing by Panay island are described below.

(a) During the year 1948 - 1983, a total of 723 tropical cyclones developed in or entered the Philippines area, that is, almost 20 numbers every year.

(b) During the year 1948 - 1983, a total of 33 tropical cyclones passed by Panay island. That is, only 0.9 times a year. Among 33, typhoon is 19 times, tropical storm is 11 times and tropical depression is 3 times. It can be said that the real typhoon attacks Panay only once two years.

(c) In Philippines, the typhoon season is generally from July to November. More than 70% of typhoon happens during these 5 months. In Panay island, it was attacked by typhoon at any month of a year in the past, but, almost 50% of typhoon came in November or December, in the late period of wet season.

(d) Most of typhoon passing by Panay originate in the eastern ocean of Philippines islands and move to generally the north-west by west direction. That is, all the past typhoon passed by Panay island from the east to west.

(e) During the period of typhoon, the rainfall in Panay island is generally pretty high and it seems that the rainfall distribution covers whole the Panay island or at least the Panay river basin as far as checking the limited rainfall records at the time of typhoon. But, it should be mentioned that the heavy rainfall at least at a point or in a certain limited area occurs without relation to typhoon as far as checking the past records of heavy rainfall at Roxas City and at the other stations.

(f) The maximum flood in the basin in these more than 30 years happened in November 1973 when the typhoon "Openg" passed by the northern part of Panay island, that is, the Panay river basin. The second biggest flood in the basin happened in November 1984 by the typhoon "Undang" of which track is almost same as that of "Openg".

2. Hydrological Condition

2.1 General

There are the following hydrological items which are necessary for the study of the Project.

(a) Streamflow
(b) Sediment
(c) Tidal water
(d) Water quality
(e) Ground water

The study of streamflow is the most essential for the river improvement plan. The following items are described in the succeeding sub-section.

(a) Observatories for the streamflow

(b) Catchment area at the gaging stations

(c) Streamflow records collected for the study

(d) Specific discharge at the gaging stations

(e) Double mass curve (used for checking the availability of original discharge records at each gaging station)

(f) Discharge duration curve at the gaging stations (to see the dependable discharge at any percentage of a year)

(g) Conversion rate (to get the discharge at strategic point from that of gaging station)

(h) Dependable discharge at strategic points

(i) Monthly discharge at strategic points

The analyses of probable flood and flood runoff are included in Section 3 "Flood Analysis".

In regard to the sediment, the sediment sampling was performed at more than 20 locations and the samples are tested in the laboratory, the results are summarized in this section and the back data are attached in the Data book. The sediment analysis using the results of test are shown in Section 4 "Sediment Analysis".

The study of tidal water level is performed for deciding the reasonable water level at the river mouth which will be used for the study of river improvement plan. The saline water analysis are shown in Section 5 "Saline water Analysis".

In regard to the water quality, the river water was sampled at 16 places and tested in the laboratory. The summary of test results and its evaluation are shown in this section.

The investigation of ground water was performed by a geologist and the results are shown in Appendix II "Geology and Ground Water.

2.2 Streamflow

2.2.1 Observatories

There are the following bodies concerned in the streamflow observation in the basin.

(A) NWRC

There are four existing streamflow gaging stations under the control of NWRC, that is, Dumalag, Dumarao, Cuartero and Panitan. But these stations are newly established in 1984 according to the request from JICA team.

On the otherhand, there are the records of three gaging stations which do not exist at present but have the records of more than 20 years. They are located at Cuartero, Tumalalud, (Mambusao) and Rallano (Maayon). These records are compiled in books titled "Surface water supply bulletin" prepared by NWRC.

(B) NIA

NIA has Mambusao irrigation system of which service has started in 1975. The observation of streamflow is carried out at the weir crossing Mambusao river and the canal which takes water from the upstream of the weir. Therefore, the total of both discharge becomes the discharge at Mambusao river.

But, it is judged that the records are not reliable to be used for the study, because, after all the original records are collected and compiled, many doubtful records are found among them.

(C) NPC

NPC has Aglinab (Tapaz) streamflow gaging station. NPC started the observation in 1959 but the observation was suspended from March 1971 to August 1979 and it seems that the discharge is generally too big even if it is considered that the rainfall in this sub-basin is more than the other stations. Possibly, the method and range (only for low water) of discharge measurement is not sufficient. Additionally, the discharge record compiled by NPC is available only for 8 years and the discharge data of the other years are estimated by using the gage height records and the same rating curve of the period with the discharge records.

(D) MPWH

MPWH is usually not in charge of hydrological observation. But, they perform the establishment of gages and also their observation and maintenance for the necessity of a Project. In case of Panay river project, MPWH (Capiz district) already established the staff gages at Pontevedra town (Pontevedra river) and Salocon (Lower Panay river) and Sigma (Mambusao river) at the request of JICA Engineers in 1984.

The list of gaging stations is shown in Table 1.2-1. There are nine existing stations in the basin. And three stations existed in the past.

Among 12 gaging stations in total, the past three stations present more available data in comparison with the others of which observation period is too short or the records has not-a-few unreliable points. The period of records is shown in Fig.1.2-1. The location of stream gaging station is shown in Fig.1.2-2.

2.2.2 Catchment Area

The following catchment areas at the point of streamflow gaging station are used in the reference books of streamflow records.

(a)	Cuartero (NWRC)	880 km^2
(b)	Tumalalud (NWRC)	307 km^2
(c)	Rallano (NWRC)	265 km^2
(d)	Aglinab (NWRC)	312 km^2

The catchment areas are newly checked by using a map of 1/50000 in scale. Then, it becomes clear that there are serious errors of the catchment areas at Cuartero and Aglinab stations.

The catchment areas used for the study are decided to be as follows.

(a)	Cuartero	930 km^2
(b)	Tumalalud	307 km^2
(c)	Rallano	265 km^2
(d)	Aglinab	230 km^2

2.2.3 Record

The daily mean discharge records of all the stations in the basin are collected and compiled in the recording forms. But only four stations of Cuartero, Tumalalud, Rallano and Aglinab have the records of comparatively long period.

The records at Cuartero, Tumalalud and Rallano are obtained from the "Surface water supply bulletin" officially published by NWRC. But, the following questions are found for the discharge records at Cuartero after compiling the records and checking them.

(a) The annual runoff coefficient at Cuartero is comparatively high as 73%; about 54% at Tumalalud and Rallano.

(b) At the time of flood by "Openg" in November 1973, the flood volume at Cuartero is calculated to be more than 8×10^8 m^3 in accordance with the hydrograph made by the discharge records of NWRC. In this case, the average rainfall in the basin of Cuartero has to be more than 1,000 mm even if the runoff coefficient of the basin is 100%. Though thre is no available rainfall records in the basin, the rainfall of more than 1,000 mm seems to be not realistic as far as checking the records at the rainfall gaging stations in the Panay island.

On the other hand, it was found that the rating curve at Cuartero was prepared by the records of discharge measurement which has been carried out only at the time of low water level. That is, the rating curve at high water level is made by just extending the curve for the low water level without verification of the curve.

Therefore, it is decided to revise the rating curve at Cuartero for obtaining the reasonable discharge.

The revised rating curve is made by the results of uniform flow calculation in the river course including the flooded area. It is assumed that the roughness coefficient of river as 0.03, the roughness coefficient of flood plain as 0.1 and the river surface slope as 1:3000. The rating curve at Cuartero is shown in Fig. I.2-3.

The monthly mean discharge records of the four stations are shown in Table I.2-2 to I.2-5. There are missing data in the original, therefore, the following methods are taken for covering the deficit of discharge records for some periods.

(a) If there are some (more than 10-15 days) daily records in the month, the mean of them is taken as the monthly discharge.

(b) The correlation constants of discharge to another station are made among three gaging stations by using rate of annual mean discharge of each station. The rate of correlation between the closer gaging stations is used for estimating the records of deficit. (Note: For Aglinab, this method is not taken.)

(c) If there is no available station for the correlation, the mean of the monthly discharge of all the other years in the station is applied for the deficit. (Note: For Aglinab, this method is not taken.)

The annual extreme daily discharge records at four stations are also compiled in Table I.2-6 to I.2-9 for the probability analysis.

Besides, the variation of water level at the stream gaging stations during the two extreme floods; in November 1973 and in November 1984, are collected and shown in the tables and figures listed below.

(a) Table of water level at Cuartero, November 1973 Flood (Table I.2-10)
Note; The streamflow observation at Tumalalud and Rallano was not carried out due to the over-flood at the area of these two stations.

(b) Water stage hydrograph at Cuartero, Tumalalud and Rallano and Discharge hydrograph at Cuartero, November 1973 Flood (Fig. I.2-4)

(c) Table of water level at Dumalag, Cuartero, Sigma, Panitan, Solocon, Pontevedra and Aglinab, November 1983 Flood (Table I.2-11) (The records of water level and discharge at Mambusao Weir are attached for the reference in Table I.2-11)

(d) Water stage hydrograph at Dumalag, Cuartero, Sigma, Panitan, Salocon and Pontevedra, November 1984 Flood (Fig. I.2-5)

2.2.4 Specific Discharge

The specific discharges at four gaging stations are summarized as follows.

Station	C.A. (km^2)	Annual Mean Discharge (m^3/sec)	Specific Discharge ($m^3/sec/km^2$)	Annual depth of Runoff (mm)
Rallano (Maayon)	265	10.3	0.039	1,224
Tumalalud (Mambusao)	307	16.9	0.055	1,734
Sto. Nino (Cuartero)	930	43.5	0.048	1,473
Aglinab (Tapaz)	230	22.7	0.099	3,109

As shown in the above, the specific discharge is different by the stations. As already mentioned before, the discharge records at Aglinab seem to be not reliable. The comparison of discharge features is to be done by the data of remaining three stations.

The specific discharge is the biggest at Tumalalud the second at Cuartero, and the smallest at Rallano. This feature represents the similar tendency of the annual rainfall distribution, that is, the rainfall is as much as 3000-3500 mm in the west and as small as 2000 mm in the east.

2.2.5 Double Mass Curve

The double mass curves are made by using the monthly discharge records of three gaging stations (Tumalalud, Rallano and Cuartero) for checking the necessity for the modification of monthly discharge at a station. The record of the Aglinab is checked separately by double mass curve to the three stations by using the records of the same available period. As a results, it is decided that the modification of monthly discharge is not necessary to be carried out as the double mass curves of four stations show almost straight line without remarkable bend, which means that no remarkable change of observation condition has not happened at any gaging stations.

The procedure for making double mass curves is shown below.

(a) The annual mean discharges at four stations are compiled as shown in Table I.2-12.

(b) The annual mean discharge records are converted to the annual depth of runoff as shown in Table I.2-13.

(c) The calculation for double mass curve for each station is carried out by accumulating the annual depth of runoff at the station and also total of the other stations respectively as shown in Table I.2-14 to 17.

(d) The double mass curves are made by using these accumulated annual depth of runoff as shown in Fig. I.2-6 to 9.

2.2.6 Duration Curve

The discharge duration curve at three gaging stations (Tumalalud, Rallano and Cuartero are made by using the daily mean discharge records at each station.

The duration curve at each station is made by using all the available daily discharge records. That is, the records are arranged from the biggest to the smallest. The results are summarized as shown in Table I.2-18 and Fig. I.2-10 to 12.

Additionally, the dimensionless duration curves at three stations are drawn together on the same sheet as shown in Fig. I.2-13. It is judged that the curves show almost similar shape and therefore it is not necessary to revise the daily discharge records further.

2.2.7 Conversion Rate

The estimation of conversion rate of discharge between the gaging station and the strategic points is carried out by using the mean annual rainfall and catchment area at each sub-basin.

First, the strategic points (which are proposed or possible sites for the dam and irrigation intake and also the confluence or bifurcation of rivers) are decided as shown in Fig. I.2-14.

Then the catchment areas of each strategic points including the stream gaging stations are checked by using the map of 1/50000 in scale.

The annual rainfall in each basin is estimated by using the figure showing the basin area on the isohyetal map as shown in Fig. I.2-15.

The conversion rate (k) is decided by the following.

$$K = \frac{Cs \cdot Rs}{Cg \cdot Rg}$$

Where, Cs: Catchment area of a strategic point

Rs: Annual mean rainfall in the basin of a strategic point

Cg: Catchment area of a gaging stations

Rg: Annual mean rainfall in the basin of a gaging station

The gaging station related to conversion rate of a strategic point is decided to be in the same river (tributary) and the nearest one along the stream.

The conversion rate for the estimation of discharge at the strategic points are calculated and summarized in Table I.2-19.

2.2.8 Dependable Discharge

The dependable discharge at strategic points are calculated by multiplying the discharge at the gaging station by the conversion rate.

The discharge of dependable rate at 95%, 90%, 50%, 10% and 5% at the strategic points as well as at the gaging stations are shown in Table I.2-20.

2.2.9 Monthly Discharge at Strategic Points

The monthly mean discharge at strategic points is calculated by multiplying the mean monthly discharge at the nearby gaging station by the conversion rate. The results are shown in Table I.2-21. The monthly discharge at strategic points (damsites) are also calculated by the same method and are shown in Table I.2-22 to 25.

2.2.10 Additional Records

During the study in the field (Capiz province), the study team collected the additional records besides the records directly used for the analyses. The additional records are the new records observed in 1984 which are collected for the further study in the future and also for the reference to the study based on the analyses by the past records before 1984.

The following are the stations of the additional records of stream flow observation.

(a) Dumalag (Panay river)
(b) Damarao (Badbaran river)
(c) Cuartero (Panay river)
(d) Sigma (Mambusao river)
(e) Panitan (Panay river)
(f) Salocon (Lower Panay river)
(g) Pontevedra (Pontevedra river)

Besides the daily (3 times a day) observation of water level, the discharge measurements are also carried out at each gaging station though the frequency and range of discharge are not sufficient for making the reliable rating curve. It is requested for MPWH (Roxas city) and NWRC (Iloilo) to continue the observation.

The additional records are to be attached in Data book.

JICA study team requested MPWH to continue the observation in association with NWRC. MPWH agreed to do it for obtaining the available data for the next stage of study.

2.3 Sediment

2.3.1 Investigation Items

It is decided that the following direct investigations of river bed fluctuation are not performed at this study stage due to the limit of time.

(a) Detailed periodical survey of longitudinal and cross sections in the selected river stretches.

(b) River bed fluctuation before and after the flood.

The estimate of river bed fluctuation is generally carried out by using the results of following sediment investigation.

(a) Sediment sampling observation

(b) River bed excavation

(c) Sediment volume in reservoirs

Among above three methods, it is decided to perform the investigation by sediment sampling observation which is the most common investigation. Though theinvestigation of sediment volume in reservoirs is also useful method, no adequeate reservoir is found in the Panay river basin.

For the sediment sampling, the following two different types of sediment have to be considered.

(a) Tractive sediments along the river bed

(b) Suspended sediments

In regard to the tractive sediments, the sampling device could not be obtained and the manufacturing is also difficult. Therefore, it is decided to perform the sampling of river bed materials substituting for the sampling of tractive sediments. Some empirical formulas are available for the estimation of not only tractive sediments but also suspended sediments.

In regard to the suspended sediments, it is decided to perform the sampling by using a device. But, the results of sampling will be used only for the reference as it is difficult to get sufficient number of data in the limited period of study in the field and additionally the suspended sediments can be estimated from the data of river bed materials by using the formulas.

As the conclusion for selecting investigation items, the following are to be carried out.

(a) Suspended materials investigation

(b) River bed materials investigation

2.3.2 Investigation Method

(1) River bed materials

The following methods are taken for the sampling of river bed materials.

(A) Location

At least one place in all the main tributaries and the river stretches between the confluences, confluence/bifurcation, or bifurcation/river mouth. Strategic points such as the location of stream flow gaging stations are desirable.

(B) Sampling number in a section

Though three places (center and both sides) in the same cross section are desirable, it is decided to accept only one or two places when the difficulty of sampling is found or the grain size looks almost same.

(C) Depth of sampling

River bed surface materials are sampled. But, the underlain materials are also sampled if the material size remarkably changes by the depth.

(D) Frequency of sampling

One time only at a place.

(E) Sampling volume

2 - 3 kg.

After the sampling, the materials are tested in the laboratory of MPWH in Roxas City. The following items are to be obtained by the test.

(A) Grain size distribution

The materials are dried out and sifted out by the different meshes of screens.

(B) Specific gravity

The other tests such as the tests for sinking velocity and aerial void ratio are not performed as the items are not necessary for using the sediment formulas and the test equipment are not available in Roxas City.

(2) Suspended materials

The following methods are taken for the sampling of suspended materials.

(A) Location

At or around the stream gaging stations.

(B) Sampling number in a river section

Though three places (center and both sides) in the same cross section are deriable, it is decided to accept only one or two places when the difficulty of sampling is found or the distribution seems to be not different.

(C) Sampling number in vertical direction

Though three places (surface, middle, and bottom) are desirable, it is decided to accept only one place in a depth direction when the velocity distribution seems to be almost uniform.

(D) Frequency of sampling

As many times as possible in the same location.

After the sampling, the materials are tested in the laboratory of MPWH in Roxas City to get the sediment ratio in the samples. As the equipment for the test is not sufficient in the laboratory, it is considered that the frequency of test in a period is limited and some errors are included in the results.

2.3.3 Results of Sampling and Tests

The following are the results of sampling and test. The most of the data are to be attached in the Data book and the summaries with the figure number are to be shown in this Appendix 1.

(A) River bed materials

(a) Location map of sampling

The locations of sampling river bed materials are shown in Fig. I.2-16.

(b) List of sampling places

The date and locations of river bed material sampling are listed in Table I.2-26.

(c) River bed material sampling record

The sample No., sampling date and time, location (with sketch), sampling depth, etc. are shown. (Attached in Data book)

(d) Work sheets for specific gravity test

The weight of sample obtained in the procedure of test and the calculation for specific gravity is shown. (Attached in Data book)

(e) Work sheets for seive analysis

The results of seiving test are recorded in the sheets. (Attached in Data book)

(f) Grain size distribution curve

The distribution curve which shows the relation between the grain size and the accumulated passing rate of sieves. (Attached in Data book)

(g) Calculation sheet of a sample diameter

The calculation sheets for obtaining the mean diameter of each sample. The diameter at each percentage of 10% interval is also described in the calculation sheet. (Attached in Data book)

(h) Summary of sample diameters

The results of calculation for diameters at mean and some representative percentages are summarized in Table I.2-27. The uniformity coefficient is also shown in the same table.

(i) Variation of material composition

The river bed materials are classified as clay, wilt, very fine sand, fine sand, medium sand, course sand, fine gravel, medium gravel and large gravel by its size. The percentages of composition in each sample are shown in Table I.2-28.

The Variations of material composition in each river course are shown in Fig. I.2-17 though the variations are available only for the reference as the sampling number seems to be not sufficient for fixing the material size in each river course.

(B) Suspended materials

(a) Location map of sampling

The locations of sampling river water for suspended materials are shown in Fig. I.2-18.

(b) List of sampling places

The sampling date and locations for suspended materials are listed in Table I.2-29.

(c) Sampling records

The sample No., sampling date and time, location (with sketch), gage height, etc. are shown in the sheets attached in Data book.

(d) Test results for material content

The data obtained on the way of test for material content are shown in the sheets attached in Data book. The test results are summarized in Table I.2-29.

2.4 Tidal Water

2.4.1 Observation

The staff gage for tidal water level observation was established in the beginning of April 1984 at Culasi port and since then the observation has been carried out every 2 hours but only in day time due to the difficulty for finding a gage keeper at night. Though the observation at night time has been also carried out for about a week to get the continuous variation.

2.4.2 Tide and Current Tables

The Bureau of Coast and Geodetic Survey (BCGS) publishes the tide and current tables at the principal ports in Philippines every year. There is no official observatory for tidal water in the coast of Panay river basin. But, it is described that the tidal water level at Culasi port, located close to the river mouth of lower Panay river, can be estimated from the table of tidal water at Cebu port by using the correlation numbers. In the table, the high and low water levels with the time at Cebu port are shown. But the relation between the elevation and the water level is not known.

2.4.3 Availability for Using the Tidal Data at Cebu

The variation of tidal water levels of both the actual observation records and the correlated values from those at Cebu port are drawn in the same chart for comparing the difference. A part of water level variations are shown in Fig.I.2-19 as an example (In the figure, the water levels at Cebu and Culasi are adjusted to see the difference clearly). As the results, it is judged that the correlated values are available to use for the study due to the following.

(a) The time of high and low water levels coincide each other.

(b) It is seen that the difference at the high and low water levels is as small as within 15 cm. And the mean difference at high water levels in the figure is almost nil.

Based on the figure, it is decided that the water levels (elevation) at Culasi are obtained by the following equations.

(a) High water level (EL.) at Culasi: W.L. at Cebu-0.28 m

(b) Low water level (EL.) at Culasi: W.L. at Cebu-0.32 m

2.4.4 Decision of Water Level at Culasi

The following are the highest and lowest water levels at Cebu in these four years.

	Highest	Lowest
1981	W.L. 2.02 m	W.L. – 0.46 m
1982	2.07	– 0.49
1983	2.09	– 0.51
1984	2.09	– 0.48

Note: W.L. 0^m is the mean water level of low water.

The above tidal water levels are converted to the water level (elevation) at Culasi as shown below.

	Highest	Lowest
1981	EL. 1.74 m	EL. – 0.78 m
1982	1.79	– 0.81
1983	1.81	– 0.83
1984	1.77	– 0.80

Besides the above, the average high and low water levels are calculated by using the data of 1984 as shown in Table I.2-30.

The tidal water levels at Culasi port used for the study are finally decided as follows.

Annual Highest High WL	EL. 1.78 m
Monthly Highest High WL	EL. 1.62 m
Mean Highest	EL. 1.00 m
Mean WL	EL. 0.44 m
Mean Lowest WL	EL.-0.12 m
Monthly Lowest Low WL	EL.-0.66 m
Annual Lowest Low WL	EL.-0.81 m

2.4.5 Verification of Tidal Water Level

It is found that there is discrepancy between the elevation of tidal water level and the elevation in the estuary area on the map. That is, it is shown on the map that the elevations on the top of dikes in the fishpond area are lower than EL.1.0 m, for example EL.0.5 m, EL.0.7 m etc., at many points. In this case, the dikes of fishponds have to be overflowed everyday as the tidal water level is higher than the hight of dikes.

In regard to the discrepancy mentioned above, it is concluded that the elevation of tidal water level is reasonable to be used for the study due to the following points.

(a) The monthly highest water level at Pontevedra gaging station for 4 months in 1984 was as follows.

September	EL.1.93 m
October	EL.1.85 m
November	EL.3.03 m
December	EL.1.18 m

EL.3.03 m in November is the record of flood caused by the typhoon "Undang". At that time, the town of Pontevedra was inundated and the water depth was almost 1 m on the river side road. On the other hand, the inundation of town area did not happen in the other months. Therefore, it is considered that the elevation of banks along the river is generally higher than about EL.2.0 m. The fishpond area is located downstream of Pontevedra town, however, the water level between the river month and Pontevedra town will be not so much different.

(b) It is informed about the accuracy of elevations on the map as follows.

The aerial photographs were taken in scale of 1/20,000 from the height of EL.3,000 m. In this case, it is probable that the error of elevation is about 1 m even on the flat area.

Further, it is difficult to read a point in narrow portion like the dike of fishpond. The elevations in fishpond area were not checked by the ground survey.

2.5 Water Quality

2.5.1 General

As far as inspecting the basin, it seems that no serious problem exists except the saline water in the lower Panay river. But, it is necessary to investigate the river water quality for confirming the appropriateness of water for irrigation, municipal and industrial uses.

The problem of saltwater intrusion is to be discussed in Appendix VIII "Water Supply Plan" and the analysis of saltwater intrusion is to be performed in Section 5 "Saline Water Analysis" of this Appendix I. Therefore, the water quality in this Section 2.5 is to be shown without putting emphasis on the saline water.

2.5.2 Data from the Previous Report

The Nation-wide Dredging II Report shows some data of water quality as described below.

The National Pollution Control Commission (NPCC) has performed the water quality investigation and analysis and made the classification and criteria of fresh surface water for the water of the Philippines.

The classification and their description for usage are as follows.

(a) Class AA: For source of public water supply. This class is intended primarily for waters having watersheds which are uninhabited and otherwise protected and which require only approved disinfection in order to meet the National Standards for Drinking Water (NSDW) of the Phillippines.

(b) Class A: For source of water supply that will require complete treatment (coagulation, sedimentation, filtration and disinfection) in order to meet the NSDW.

(c) Class B: For primary contact recreation

(d) Class C: For the propagation and growth of fish and other aquatic resources

(e) Class D: For agriculture, irrigation, livestock watering and industrial cooling and processing

(f) Class E: For navigational use

The water quality criteria used for the classification are shown in Table I.2-31 and the test results of water sampled in the Panay river in 1975 and 1976 are shown in Table I.2-32.

The water in the Panay river is classified as Class A. That is, the water is generally usable for irrigation, municipal and industrial purposes though the total hardness and coliform density are high.

2.5.3 Data Collected in This Time

The sampling of river water is performed at about 15 places which covers all the main tributaries, branch river, and the main river. The locations of sampling are shown in Table I.2-18 together with the sampling for suspended material content.

After sampling, the water is tested in the laboratory of MPWH in Roxas city though the test items are limited due to the kind and capacity of testing devices. The results of water quality tests are seen in the sheets attached in Data book and the summary of results is shown in Table I.2-33.

As far as checking the water quality standard of MPWH shown in Table I.2-34 and that of Japan show as a reference in Table I.2-33, it is concluded that the water in Panay river is sufficiently usable for irrigation as well as muncipal and industrial purposes.

The water quality affected by the river improvement works will be described in Appendix XI "Environment Study".

3. Flood Analysis

3.1 Flood Analysis in Present River Condition

3.1.1 General

The basic hydrological data for flood analysis generally consist of rainfall, water level, and discharge. The flood analysis in present river condition is performed for deciding the probable rainfall in the basin and the probable flood runoff with different return periods. In other words, the flood condition under the present river condition is made being based on the verification of the rainfall, water level, and discharge at some strategic points in the basin at the time of "Undang" typhoon.

The typhoon "Undang" passed by the Panay river basin in November 1984 and caused the serious damage due to flood which is the second biggest as far as the inhabitants can remember the past. The JICA study team could collet more reliable and detailed data than those for any other floods in the past. The flood happened in November 1973 by the typhoon "Openg" is the biggest one for these 40 to 50 years. But, it is considered that the accuracy of verification is low as the number of hydrological data is not sufficient. Especially, there is no other station except Roxas city that has the rainfall records in the basin in November 1973.

Therefore, it is decided to estimate the probability of rainfall, water level, and discharge in present river condition based on the data at the time of "Undang" flood. But, the verification for "Openg" flood is to be carried out by the hydrograph at Cuartero for confirming the adequecy of the verification based on the "Undang" flood.

3.1.2 Flow Chart of Flood Analysis in Present River Condition

The flow chart of flood analysis in the present river condition, for deciding the probable rainfall in sub-basins, is shown in Fig. I.3-1. The brief procedure of the analysis is as follows.

(a) Construction of river system model and decision of constants K, P of sub-basins and river channels for storage function method.

(b) Preparation of rating curve and flood hydrograph at base points. (The reasonable runoff coefficient has to be obtained by the hydrographs)

(c) Decision of rainfall and its distribution in each sub-basins. (The reasonable runoff coefficient has to be obtained by the basin rainfall)

(d) Calculation of flood runoff. Then, the preparation of hydrograph at each base point. (The verfication of discharge and water level at the base points is to be carried out)

The above calculation and analysis are to be performed to the data of "Undang" flood, though a part of the above procedure is done to the data of "Openg" flood too.

Additionally, the calculations at each procedure are to be repeated by revising the first conditions such as rating curve, rainfall, etc. until the resonable results are obtained by checking the runoff coefficient, water levels, discharge, etc.

The common base points for the procedure (b), (c) and (d) are to be at Panitan and Cuartero.

3.1.3 River System Model

The river system model is constructed and the flood runoff calculation of the Panay river is made using the model with the aid of an electronic computor. In the model, the flood flow mechanism of the Panay river is simplified by dividing the basin to the sub-basins at the strategic points for the river improvement plan. The division of basin is shown in Fig. I.3-2. The river system models are made according to the division as shown in Fig. I.3-3. The model is used for the present condition as well as for the improved condition.

In case of the model shown in Fig. I.3-3, the channel function of the two rivers at the junctions of major tributaries with river are cosidered to be different channel to see the inflow discharge of the two rivers separately. The model comprises 28 sub-basins and 17 channels.

Base points, which are selected among the sub-base points, are the principal points for estimating the flood runoff and for determining the flood distribution along the river. The base points are selected at the location listed below.

BP-1	Proposed site of Panay B dam, in Panay river
BP-2	Proposed site of Panay C dam, in Panay river
BP-3	Just upstream of confluence with Badbaran river, in Panay river
BP-4	Dumarao town, in Badbaran river
BP-5	Middle of Dumarao town and conflucence with Panay river, in Badbaran river
BP-6	Just upstream of confluence with Panay main river, in Badbaran river
BP-7	Cuartero town, in Panay river
BP-8	Just upstream of confluence with Mambusao river, in Panay river
BP-9	Mambusao weir, in Mambusao river
BP-10	Middle of Sigma town and Mambusao town, in Mambusao river
BP-11	Just downstream of Sigma town, in Mambusao river
BP-12	Just upstream of confluence with Panay main river, in Mambusao river
BP-13	Just upstream of confluence with Maayon river, in Panay river
BP-14	Maayon town, in Maayon river
BP-15	Just upstream of confluence with Panay main river, in Maayon river

BP-16 Just upstream of confluence with Maayon river, in Ilas river

BP-17 Just upstream of confluence with Mambusao river, in Balacuan river

3.1.4 Constants K, P of Sub-basins and River Channels in Present River Condition

In the flood flow analysis, the storage function method developed by Dr. Kimura, Japan, is used. The storage function method can express both basin storage and river channel storage including inundation by simple equation and suitable for calculation by electronic computor.

Flood runoff from each sub-basin is estimated by means of storage function method. The basic formula of storage function method is expressed by the following equation.

$$S = KQ^P$$

$$\frac{dS}{dt} = \frac{1}{3.6} f.r.A - Q \quad \text{(Continuity equation for basin)}$$

Where,
- S : Basin storage (m^3)
- Q : Runoff from basin (m^3/sec)
- K, P : Constants
- dt : Unit time (sec)
- f : Runoff ratio
- r : Basin average rainfall (mm/hr)
- A : Catchment area (km^2)
- dS : Incremental basin storage corresponding to dt (m^3)

In case where the method is used to determine flood runoff from a river basin, the storage function (K and P) which show the relationship between storage and runoff is determined usually by comparing storm rainfall records and flood runoff records actually observed. Since such observation records are not available for this study, K and P values are determined by the empirical formula shown below.

$K = 118.84 . i^{0.3}$

$P = 0.175 . i^{-0.235}$

Where, i : Average bed slope of the river in the basin

The formula is prepared on the basis of many observed storm rainfall and flood runoff records in the mountainous river basins of the Tone river basin, one of major river basins in Japan.

In addition to K and P values, it is necessary to decide also lag time (Tl) to obtain flood outflow. The lag time is roughly estimated by the following empirical formula:

$Tl = 0.0470L - 0.56$ (hour) $(L > 11.9$ km)

$Tl = 0$ $(L \leq 11.9$ km)

Where, L : Length of river in the basin (km)

The constants for each sub-basin are shown in Table I.3-1.

In regard to the Constant K, P of river channels, it is estimated according to the following procedure.

(a) Rating Curves of water level (H) and discharge (Q) at the outlet point of each river channel are made by the uniform flow calculations.

(b) The rating curves of water level (H) and storage volume (S) in each river channel are made by assuming that the longitudinal slope of flood plain is almost parallel to that of river channel.

(c) The rating curves of Storage (S) and Discharge (Q) are made by combining the curves of H-Q and H-S.

The constants for each river channels are shown in Table I.3-2.

3.1.5 Rainfall Analysis for "Undang" Flood

3.1.5.1 Rainfall in Sub-basins

The basin mean rainfall at Cuartero and Panitan is calculated by using Thiessen Polygon method of which figure is shown in Fig. I.3-4.

The daily mean rainfall records at the time of "Undang" flood are summarized in Table I.1-6. The basin mean rainfall of 3 days (Nov. 4, 5 and 6) at Cuartero and Panitan as well as in each sub-basins, used for the runoff analysis by storage function method, are calculated as shown in Table I.3-3.

The basin rainfall at Cuartero and Panitan is obtained as below.

Station	3 days-rainfall	6 days-rainfall
Cuartero	220 mm	274 mm
Panitan	229 mm	262 mm

3.1.5.2 Hourly Rainfall Distribusion

The accumulated rainfall curves from Nov. 3 to Nov. 6 at each rainfall gaging station are made first. Then, it is decided to select the curves of 6 stations for making the rainfall pattern, after comparing the curves. They are Astorga, Brgy Roxas Tapaz, Mambusao, Matec, Jamindan and Villaflores as shown in Fig. I.3-5 to I.3-10.

Then, the hourly rainfall distribusion at these 6 stations are made as shown in Fig. I.3-11 to I.3-13.

In the simulation, the pattern of hourly rainfall distribusion at a station is to be used for making the rainfall distribusion in each sub-basin. One of six stations located nearby the sub-basin is selected for the simulation. In the simulation, the hourly rainfall quantity is to be enlarged or shortened proportionally to the ratio between the total rainfall at the station and that in the sub-basin.

3.1.6 Streamflow Analysis for "Undang" Flood

3.1.6.1 Rating Curve at Panitan and Cuartero

(1) Rating Curve at Panitan

The rating curve at Panitan was made in accordance with the following procedure.

(a) The rating curve based on the results of discharge measurement is prepared.

(b) For checking the reliablity of the rating curve, the following points are examined.

(c) First, the mean velocity at the time of flood was obtained from the velocity at a point within 1 m from the surface without considering the decrease by the depth. In regard to this point, it is judged that the velocity obtained in the range of 1 m from the surface is assumed to be the mean velocity at the section in the Panay river after considering the matter from some aspects.

(d) Second, the rating curve is made by the results of discharge measurement including by float. So, the necessity of adjustment of velocity by float has to be considered. In regard to this point, the comparison curve of velocity by float and current meter is prepared as shown in Fig. I.3-14 where the correlation coefficient is almost 1 each other. Therefore, it is decided that no adjustment for the velocity by float is required.

(e) Third, the uniform flow calculation is carried out in the stretch of Panitan on the conditions below.

roughness coefficient	(river channel)	$n = 0.03$
"	(flooded plain)	$n = 0.1$
water surface slope		$i = 1/5,000$

For making a rating curve based on the uniform flow calculation, the overflooded discharge from both banks at the upstream stretch of Panitan is roughly estimated and added to the discharge in the river channel.

The results are shown in Fig. I.3-15 together with the rating curve based on the discharge measurement.

As a conclusion, it is decided to use the rating curve drawn by thick line shown in Fig. I.3-15. That is, the rating curve based on the discharge measurement is modified in the range of high water in consideration of the overflood from the upstream banks.

(2) Rating Curve at Cuartero

At Cuartero gaging station, the number and the range of discharge measurement is not sufficient for making the rating curve. There is no measurement records at the time of flood.

Therefore, it is decided to make the curve based on the uniform flow calculation. The conditions for the calculation are decided as follows.

Roughness coefficient	(river channel)	$n = 0.03$
"	(flooded plain)	$n = 0.1$
Water surface slope		$i = 1/3,000$

The calculation is performed for the both old and new sections. The old section is a standard section available for the records from 1950's to 1970's, though the fluctuation of river bed during those more than 20 years actually might happen. While, the new section is a standard section available for the record since 1984 when the new gaging station was established. The both standard sections are shown in Fig. I.3-16.

The rating curves are shown in Fig. I.3-17.

The adequacy of the rating curves are checked by the runoff coefficient of the hydrographs at the time of "Undang" flood and "Openg" flood shown in Section 3.1.6.2.

3.1.6.2 Hydrograph at Panitan and Cuartero

The hydrographs at Panitan and Cuartero at the time of "Undang" typhoon are prepared by using the following data.

(a) The rating curves (See: Section 3.1.6.1)

(b) Records of water level

The records of water levels as well as the record converted discharge from the rating curves are shown in Table I.3-4.

The hydrographs are shown in Fig. I.3-18.

3.1.7 Verification of Flood Analysis for "Undang" flood

The verification of flood analysis for "Undang flood" in present river condition is made by checking the hydrograph at Panitan.

The rainfall in each sub-basin including the hourly distribution is already decided in Section 3.1.5. The effective rainfall for runoff to the river is decided by the primary runoff coefficient (f_1) in the condition up to the saturated rainfall (Rsa). As there is no sufficient records for estimating f_1 and Rsa, the experimental values are adopted.

The following two cases are used for the simulation of runoff analysis.

(a) Case 1 $f_1 = 0.6$, Rsa = 160 mm

(b) Case 2 $f_1 = 0.5$, Rsa = 160 mm

The saturated rainfall; 160 mm, is estimated for the volume of actual hydrograph to coincide with that of calculated hydrograph, in Case 1. On the other hand, the rainfall in Case 2 is proportionally extended for making the volume same as Case 1. The results are shown below and also in Fig. I.3-19.

Case 1	6 days rainfall	262 mm
	runoff (at Panitan)	187 mm
	runoff coefficient	0.71
Case 2	6 days rainfall	279 mm
	runoff (at Panitan)	187 mm
	runoff coefficient	0.67

The both results show comparatively good coincidence with the observed records. Therefore, it is decided to take Case 1 as the rainfall used for the simulation. Case 1 is based on the actually observed records without modification.

The water stage hydrographs at 4 strategic points; Panitan, Signa, Cuartero, and Dumalag, are prepared as shown in Fig. I.3-20. The hydrographs made by the calculation show the similar shape though there are some differences at some points. It is very hard to completely coincide the actual records with the resultsof calculation at all the points.

It is concluded that the values estimated or assumed for the flood runoff analysis in the present river condition, from Section 3.1.1 to 3.1.7, are verified as reasonable ones.

3.1.8 Flood Storage Volume of "Undang" Flood

The flood storage volume means the accumulated volume of the difference between Inflow volume and Outflow volume.

The flood storage volume of "Undang" flood is estimated by the flood storage hydrographs shown in Fig. I.3-21 and I.3-22. The results are as follows.

(A) Panay river (total volume)

(a) at Panitan: 156.8×10^6 m^3

(b) at just downstream of the confluence with the Mambusao river: 124.0×10^6 m^3

(c) at Cuartero: 68.1×10^6 m^3

(d) at just upstream of the confluence with the Badbaran river: 40.7×10^6 m^3

(B) Panay river (volume in each stretch)

(a) between the confluences with the Badbaran river and the Mambusao river: 38.9×10^6 m^3

(b) between the confluences with the Mambusao river and the Maayon river: 34.3×10^6 m^3

(c) between the confluence with the Maayon river and the Panitan: 12.7×10^6 m^3

(C) Badbaran river: 7.6×10^6 m^3

(D) Mamusao river: 47.6×10^6 m^3

(E) Maayon river: 4.9×10^6 m^3

3.1.9 Probability Analysis

3.1.9.1 Probability of "Undang" Flood

The probability of flood is usually evaluated by the basin mean rainfall. However, there is not sufficient records to estimate the probability of basin rainfall.

The probability of "Undang" flood is estimated first by the annual maximum daily mean discharge records at Cuartero where the daily discharge records of 24 years are available. The annual maximum discharge records are plotted by Hazen method on the logarithmic probability paper shown in Fig. I.3-23. The line for the probability is drawn for the records of bigger discharge. The results are obtained as follows.

Probability (year)	Discharge (m^3/s)
2	400
5	720
10	1,000
20	1,350
50	1,800
100	2,200

The peak discharge at the time of "Undang" typhoon is estimated at 1,078 m^3/s from the observed peak water level and the rating curve at Cuartero. This discharge is evaluated as about 10 year probable flood from the probability curve of flood discharge.

3.1.9.2 Probable Rainfall at Gaging Stations

The probable point rainfall are calculated at the following rainfall gaging stations which has the records of comparatively long period though they are located outside of the Panay river basin.

(a) Valderama, Antique
(b) Balete, Aklan
(c) Libacao, Aklan
(d) Barbaza, Antique
(e) Culasi, Antique
(f) Iloilo, Iloilo
(g) Roxas city, Capiz

The annual maximum 1 day, 2 days, and 3 days rainfall records are shown in Table I.1-11 to I.1-16. The probability is calculated by Piason Type III distribution method which is commonly used for the probability analysis of rainfall. The results are shown in Table I.3-5 and Fig. I.3-24 to I.3-26.

3.1.9.3 Probable Rainfall in Sub-basins

The probable rainfall in Sub-basins is obtained by the following procedure.

(a) The probable rainfall at Culasi is selected as the representative probable point rainfall in the basin. Though Culasi gaging station is located outside of the Panay river basin, it is considered to be reasonable to use the records of a nearby station when there is no available rainfall gaging station which has the records of sufficiently long period. Additionally, it is considered to be conservative for making flood control plan based on the results of flood runoff analyses using the records of a station which has the heaviest rainfall records; Culasi gaging station.

Roxas station has the records of more than 30 years, however, it is located in the lower plain area where the rainfall intensity is much wedk in comparison with that in the mountain area in the basin. The probable rainfall at Culasi is calculated in the preceding Section 3.1.9.2.

(b) The reduction of rainfall due to the basin area is studied. The areas for estimating rainfall reduction by area are decided by making circles of each representative area which encloses as many stations as possible. The areas are shown in Fig. I.3-27. The rainfall reduction rate of each representative area are calculated by using the past heavy rainfall records in the basin as shown in Table I.3-6 to I.3-9. The results are plotted in the logrithm paper shown in Fig. I.3-28 for deciding constants of Horton's formula shown below.

$$P = Po.exp^{-k(0.386A)^n}$$

Where, P : Average depth of rainfall in a basin (mm)

Po: Largest point rainfall amount in a basin (mm)

A : Area (km^2)

k : Constant k, is usually taken at 0.1

n : To be decided by the study of observed heavy rainfall record in the basin

The Horton's formula is widely used in Japan for deciding the reduction rate by areas. The formula is represented below in accordance with the results shown in Fig. I.3-28.

$$P = Po.exp^{-0.1(0.386A)^{0.31}}$$

The probable rainfall in each subbasin is calculated by applying the area (A) of the sub-basin into the above formula.

The Probable basin rainfall at Panitan and Cuartero is calculated as shown below.

Return Period (yr)	Probable Rainfall at Culasi (mm)			Probable Basin Rainfall at Panitan (mm)		
	1-day	2-day	3-day	1-day	2-day	3-day
1.01	131	172	214	60	79	98
2	170	265	321	78	121	147
5	250	376	440	114	172	201
10	325	471	538	149	215	246
25	451	618	684	206	283	313
50	571	749	811	261	342	371
100	718	901	955	328	412	437

Return Period (yr)	Probable Basin Rainfall at Cuartero (mm)		
	1-day	2-day	3-day
1.01	71	93	116
2	92	143	173
5	135	203	238
10	176	254	290
25	244	334	369
50	308	404	438
100	388	486	516

3.1.10 Hourly Rainfall Distribution

In the previous Section 3.1.5.2, the rainfall distributions for the analysis of "Undang" flood were decided and it was verified that the distributions give the similar results as the actually observed ones. Moreover, it is considered that the rainfall distributions obtained in Section 3.1.5.2 are made by only the records at the time of "Undang" flood, that is, no other records can support the distributions. Therefore, it is decided to construct the common pattern of rainfall distribution in each sub-basin.

The rainfall intensity-duration curve shown in Fig. 1.3-29 was prepared from hourly rainfall data at Iloilo and Roxas City gauging stations. The hourly rainfall distribution is prepared based on above rainfall intensity-duration curve assuming a center-concentrated pattern. The hourly rainfall distribution expressed in term of percentage of the total 3-day rainfall is shown in Fig. 1.3-30.

The verification of the hourly rainfall distribution was performed on the following conditions.

(a) 10 year probable, basin rainfall at Cuartero decided in the previous Section 3.1.9.3

1 day	176 mm
2 day	254 mm
3 day	290 mm

(b) River system model decided in Section 3.1.3

(c) Constants K, P and Tℓ of sub-basins and river channel decided in Section 3.1.4

(d) Primary runoff coefficient (f_1) and saturated rainfall (Rsa) decided in Section 3.1.7

$f_1 = 0.6$

Rsa = 160 mm

The flood runoff analysis performed on the above conditions show that the peak discharge at Cuartero is 1,130 m^3/sec which is comparatively close to the peak discharge during "Undang" flood that is, 1,078 m^3/sec. As the "Undang" flood was evaluated at about 10 year probable flood in Section 3.1.9.1, it can be said that the hourly rainfall distribution made in this section is established to be available to use for the further analyses.

3.1.11 Flood Runoff Analysis in Present River Condition

Based on the probable rainfall decided in the foregoing Subsections 3.1.9 and 3.1.10, the probable flood runoff at each base point under the present river condition is calculated as shown in Fig. I.3-31.

The following table is the summary of the probable discharge calculation at each representative point.

Base Point	Basin Area (km^2)	Peak Flow (m^3/sec) 100-yr Flood	25-yr Flood	10-yr Flood
Panay river				
(BP-3)	553	1,530	890	610
(BP-8)	983	2,040	1,310	920
(BP-13)	1,533	2,570	1,700	1,220
(BP-17)	1,987	2,670	1,830	1,370
Badbaran river				
(BP-6)	348	1,020	500	390
Mambusao river				
(BP-15)+(BP-16)	373	710	520	400
Maayon river				
(BP-12)+(BP-17)	513	1,060	610	390

Note; The location of each base point is described in Section 3.1.3.

The flood water level and its duration can be estimated from stage hydrograph at each base point converted from discharge hydrograph by using discharge rating curve and elevation of each point in the inundated area. The hydrograph of 100 year flood at Cuartero and Panitan is shown in Fig. I.3-32. Therefore the conceivable inundation water level in the flood plain under the present condition are assessed by using the probable flood runoff shown in Fig. I.3-31 and the rating curve shown in Fig. I.3-33. The results of calculation; water levels and duration at the representative points in the basin, are used for estimation of flood damage shown in Appendix III. The discharge diagram and water level diagram under present river condition are shown in Figs. I.3-34 and I.3-35.

3.2 Flood Analysis for Flood Control Plans

The flood analyses for flood control plans are to be performed to the following alternative cases. The details of each alternative plan are to be referred to Appendix IV.

(A) Alternatives for Protection Areas

(a) Alternative 1
(b) " 2
(c) " 3
(d) " 4
(e) " 5
(f) " 6

Note; (1) The division of protective areas at each alternative is shown in Table I.3-10. But, the detailed plans and descriptions are to be referred to Appendix IV.

(2) Flood analysis is carried out for the case of probable discharge/water level at each base point.

(B) Alternative Dam plans

(a) Panay B dam
(b) " C dam
(c) " B dam + Panay C dam
(d) Badbaran dam
(e) Mambusao dam

Note; (1) In these cases, the river condition is to be under present one.

(2) Flood analysis is carried out for the case of probable discharge/water level at Panitan base, that is, the passing discharge at each base point except Panitan is obtained.

(C) Long-term Plans (LP)

(a) Riber improvement + Panay B dam
(b) " + Panay C dam
(c) " + Panay B dam + Panay C dam
(e) " + Panay B dam + Panay C dam + Badbaran A dam + Mambusao B dam

Note; (1) River improvement has a protective area alternative selected by the alternative study; Alternative 4 of 6 alternatives in (A).

(2) Flood analysis is carried out for the following two cases.

(a) Probable discharge (or Water level) at Panitan base
(b) Probable discharge (or water level) at each base point

(D) Mid-term Plan (MP)

(a) River improvement* + Panay B dam
(b) River improvement* + Panay B dam + Panay C dam

Note; (1) * Some selected areas of protection areas for LP
(2) Probablity of Discharge/water level at each base point

(E) Short-term Plan (SP)

(a) River improvement*** + Panay B dam (10 year flood)
(b) " *** + Panay B dam (2 year flood)

Note; (1) *** Some further selected areas of protection area for MP

(2) Probability of Discharge/Water level at each base point

The results of flood analysis of each alternatives in each plan; (A) - (E) above, are summarized in the following figures.

(A) Fig. I.3-34 to I.3-39

Discharge Distribution Diagram for:

(a) Under present river condition
(b) dam plans under present river condition
(c) protective areas alternatives
(d) LP alternatives
(e) MP alternatives
(f) SP alternative

(B) Fig. I.3-40 to I.3-45

Water Level Diagram for:

(a) under present river condition
(b) protective area alternatives
(c) dam plans alternatives under present river condition
(d) LP alternatives
(e) MP alternatives
(f) SP alternatives

(C) Fig. I.3-46 to I.3-55

Peak Discharge and WL at representative base points for:

(a) Protective areas alternatives
(b) Dam plan alternatives under present river condition (100 year flood)
(c) LP (100 year flood)
(d) MP (25 year flood)
(e) SP (10 year flood)

The followings are some descriptions about the discharge and water level obtained from the figures listed above.

(A) Probable Discharge and Water Level at Each Base Point

For the above figures, it is noted that the probable discharge and water level at each base point on the following diagrams are those in the case that BP-17 (at Panitan) has the equivalent probability. In other words, the discharge and water level at the other base point don't have the same probability as those at BP-17. For exsample, the discharges at BP-3, BP-8, BP-13 etc. for 100 year flood are the passing discharges when the discharge at BP-17 has 100-year probability.

(a) Under present river condition
(b) Dam plans under present river condition
(c) LP alternatives (Case 1)

On the other hand, the probable discharge and water level at each base point are shown in case of the following diagrams.

(a) Protective areas alternatives
(b) LP alternatives (Case 2)
(c) MP alternatives
(d) SP alternatives

For comparing the difference of the two cases, the 100 year flood discharge in Fig. I.3-34 and the 100 year flood discharge of Alternative 1 in Fig. I.3-35 are shown below. The Alternative 1 in Fig. I.3-35 shows the probable discharge at each base point under the present river condition as the Alt. 1 has the protection areas only at the downstream from BP-17.

	100 year flood discharge (m^3/sec)			
	BP-3	BP-8	BP-13	BP-17
Fig. I.3-34	1,520	2,032	2,570	2,668
Alt. 1 in Fig. I.3-35	2,048	2,382	2,684	2,668

As shown above, the discharge at BP-17 show the same quantity, while the discharge at the other base points is different each other and the difference is more remarkable at the upstream base point where the drainage basin area is much smaller than that at BP-17.

The discharge and water level at each base point in case that the probability is placed at only BP-17 are used for estimating the flood damage and those in case that probability is placed at all the base points are used for the plan of flood control structures.

(B) Discharge and Water Level for Protective Area Alternatives

The effectiveness of flood control plan is examined for 6 river-improvement-alternatives with different protection areas. The results in case of 100 year flood are summarized below.

	BP-3	BP-8	BP-13	BP-17
Discharge (m^3/s)				
Alt. 1	2,048	2,382	2,684	2,668
" 2	2,048	2,382	2,896	3,121
" 3	2,048	3,193	4,133	4,319
" 4	2,244	3,346	4,457	4,524
" 5	2,244	3,346	4,497	5,401
" 6	2,449	4,106	5,407	6,347
Water level (El. m)				
Alt. 1	22.56	16.46	12.84	10.29
" 2	22.56	16.46	11.67	10.03
" 3	22.56	18.16	13.14	11.73
" 4	22.57	18.40	13.48	11.99
" 5	22.57	18.40	13.53	12.49
" 6	22.92	18.15	14.40	13.42

As seen above, the discharge and the water level is much different between each alternatives, as the areas with river improvement is different each other. For example, at BP-17, the discharge of Alt. 1 (the same as the present river condition) of 2,668 m^3/s is increased to 6,347 m^3/s in case of Alt. 6 of which protection areas cover almost all the stretches in the flood prone areas. That is, the retarding function at each alternative show the effectiveness for reducing the peak discharge and the peak water level.

(C) Discharge and Water Level for Dam Plans

The effectiveness of flood control for dam plans is examined under the present river condition. The following are the summary of results.

100 year flood for Dam Plans (m^3/sec)

	BP-3	BP-8	BP-13	BP-17
Present river	1,520	2,032	2,570	2,668
Panay B dam	1,366	1,931	2,481	2,612
Panay C dam	686	1,523	2,099	2,302
Badbanan dam	1,520	1,907	2,461	2,599
Mambusao dam	1,520	2,032	2,540	2,645
Panay B + Panay C	581	1,433	2,032	2,242
4 dams	581	1,294	1,872	2,111

As seen above, the effectiveness of flood discharge control in case of Panay C dam is the highest among for sole dam cases though the effectiveness of combined dam plan with Panay C dam has higher than the sole dam plan. Additionally, the reduction rate of discharge due to dam regulation is generally higher at the upstream base point and the reduction effect is decreased at the downstream. For example, Panay C dam can reduce the peak discharge from 1,520 m^3/s to 686 m^3/s at BP-1 but from 2,668 m^3/s to 2,302 m^3/s at BP-17.

(D) Discharge and Water Level for LP

The calculation of flood runoff is carried out for 5 alternatives; the river improvement with protection area selected as Alt. 4 and dam alternatives. The results of 100 year flood discharge at each base point (Case 2) are shown below.

100 year Discharge for LP (m^3/sec)

	BP-3	BP-8	BP-13	BP-17
LP without dam	2,244	3,346	4,457	4,524
LP with Panay B dam	1,940	3,110	4,230	4,380
LP with Panay C dam	1,070	2,260	3,680	3,960
LP with Panay B + Panay C	920	2,110	3,570	3,870
LP with 4 dams	920	1,850	3,030	3,520

As seen above, the effectiveness with Panay C dam and the combination dams including Panay C dam show the remarkable reduction of peak discharge at the upper reach but the effect is rather small at the downstream reach.

(E) Discharge and Water Level for MP and SP

The results of runoff analysis in case of MP and SP and summarized below.

25 year flood discharge for MP (m^3/sec)

	BP-3	BP-8	BP-13	BP-17
MP with Panay B dam	1,160	1,880	2,440	2,670
MP with Panay B + Panay C	530	1,280	1,950	2,270

	Flood discharge for SP (m^3/sec)			
	BP-3	BP-8	BP-13	BP-17
10 year flood				
(SP-1A) with Panay B dam	790	1,090	1,270	1,360
(SP-1B) with Panay B dam	760	1,080	1,360	1,490
(SP-1A) with Panay B dam + Panay C dam	350	770	1,030	1,140
2 year flood				
(SP-2) with Panay B dam	400	530	690	800

Note; Protection area of SP-1A and SP-1B is different.

4. Sediment Analysis

4.1 General

For river improvement plan including the construction of dam, it is required to make a stable river channel so that the maintenance and repair in the future will be not frequent.

It is a basic point to estimate the sediment volume as accurate as possible for understanding the river bed fluctnation system. In other words, it is necessary to estimate the following items for preventing the any serious problems caused by rising and lowering of river bed.

(A) Sediment volume in the proposed reservoir

(B) Sediment transport capacity in the river channels

The data collected by the investigation in regard to the sediment are available in Data book and the summary is shown in Section 2.3 of this Appendix I.

4.2 Sediment volume in Reservoirs

In the Panay river basin, it is expected that the sediment yield from the upper river basins would be remarkably big in comparing with that of the ordinary river basins due to the following.

(A) In the upper river basins, the annual rainfall is as much as over 3000 mm in the western basin.

(B) The condition of land conservation especially in the upper river basins is poor, that is, the forest is scarce and only the shrubs and grasses cover the upper basin lands.

For estimating the sediment volume in the proposed reservoirs in the Panay river basin, it is decided to use the sediment data recorded in the Jalaur river due to the following reasons.

(A) For estimating the sediment yield, there are so many factors to be considered. They are the basin area, rainfall, elevation, basin relief, soils, geology, land use and etc. And not-a-few empirical or theoretical formulas have been made by the engineers. It is necessary to use the formula which is in conformity with the actual condition of the basin. However, the results using these formula are usually much different. Additionally, no adequate existing formula was found for the Panay river basin as far as using the data which are available at present.

(B) The Jalaur river is located in the Panay island and the source of main stream is in the same mountain range of that of the Panay river though the Panay river flows down to the northeastern direction, while the Jalaur river to the southeastern direction. The basin rainfall is considered to be not different so much.

The Jalaur river has the observation records of sediments at three stations and the rating curves of sediment and discharge are available.

It seems that the records of specific sediment yield of the Jalaur river are the most applicable method for estimating the sediment yield of the Panay river basin.

There are three sediment sampling stations in Jalaur river and the rating curve of annual sediment volume and drainage area is made as shown in Fig.I. 4 - 1.

But, it is not suitable to use this curve directly for estimating the sediment in the Panay river due to the following reasons.

(A) The number of sampling station is only three. It seems to be not sufficient to make such rating curve.

(B) According to the curve between Alibunan and Passi, the sediment volume per unit drainage area is rather increasing. This tendency is not reasonable.

Therefore, it is decided to take the mean of specific sediments at Alibunan and Passi due to the following considerations.

(A) There is no reliable data except the records in the Jalaur river.

(B) Drainage area at Pototan is too big comparing with that of dam sites in the Panay river basin.

(C) Though some errors are expected, it is considered that the recorded data at both stations are more or less reliable.

The specific sediment volume to be applied for the Panay river basin is calculated as below.

Alibunan

$$\frac{0.185 \times 10^6}{120} = 1541 \ m^3/km^2/year$$

Passi

$$\frac{0.715 \times 10^6}{534} = 1339 \ m^3/km^2/year$$

Mean 1440 $m^3/km^2/year$

For the trapping percent in the reservoir, the rating curve (shown in Fig.1.4 - 2) prepared by G.M. Brune is applied and the results are shown as follows.

Damsite	Annual inflow (VQ)	Storage (VR)	VR/VQ	Trapping Rate
Panay A	529.7 x 10^6 m^3	134 x 10^6 m^3	0.253	0.94
B	598.1	119	0.199	0.92
C	1179.1	323	0.273	0.94
Mambusao A	127.6	38	0.298	0.95
B	342.6	70	0.204	0.92
Badbaran	393.0	102	0.260	0.94

Note; The trapping rate will be changed if the storage volume is changed.

Then the sediment in flow-volume to the reservoir is estimated as follows.

Damsite	Catchment area	Specific volume	Annual sed inflow
Panay A	211.9	1440 $m^3/km^2/year$	0.305 MCM
B	238.8	"	0.344
C	509.2	"	0.733
Mambusao A	72.9	"	0.105
B	216.6	"	0.312
Badbaran	277.3	"	0.399

The sediment volume trapped in the reservoir is estimated as follows.

Damsite	Sediment inflow	Trapping rate	Sediment volume	Specific sediment volume
Panay A	0.305 MCM	0.94	0.287 MCM	1353 $m^3/km^2/year$
B	0.344	0.92	0.317	1325
C	0.733	0.94	0.689	1353
Mambusao A	0.105	0.95	0.100	1368
B	0.312	0.92	0.287	1325
Badbaran	0.399	0.94	0.375	1352

As the results of the above study, the sediment volume in the reservoir is calculated as shown below.

Dam site	Annual volume	Design sediment Vol.1	Total storage Vol.1	Effective storage Vol.1
Panay A	0.287×10^6	28.7×10^6	134×10^6	105.3×10^6 m^3
B	0.317	31.7	73	41.3
C	0.689	68.9	229	160.1
Mambusao A	0.100	10.0	38	28.0
B	0.287	28.7	72	43.3
Badbaran	0.375	37.5	97	59.5

It is noted that sediment volume as well as effective storage volume shall be changed if the total storage volume at each dam is changed.

4.3 Sediment Transport Capacity

4.3.1 General

The riverbed materials start to move when the tractive force of flow or friction velocity is over a critical point of the materials. There are generally two types of materials by the movement as described below.

(A) Tractive flow materials which move by rolling, sliding, or leaping along the riverbed with the direct resistance of stream flow.

(B) Suspended materials which move in the floating condition by the dispersing phenomenon of stream flow disturvance.

It is desirable that the sediment transport capacities are balanced as much as possible from the upstream course to downstream course for the stable river channel. Therefore, it is necessary to estimate the sediment transport capacities in the river courses of the improved condition as well as the present condition.

The annual sediment transport capacities of the river channels are estimated by using the estimated capacities in all the ranges of discharge and the discharge duration of the objective river channels. But, the discharge records for sediment calculations are limited at three stations and it is difficult to estimate the discharge duration in the improved river channels as the runoff condition is much changed especially at the time of flood when the sediment transport volume is remarkable. That is, it is considered to be difficult to get the accurate or reliable results of annual sediment transport even if the detailed calculations are carried out. But, it is also considered to be necessary to see the balance of sediment in the river channels not only in case of the present river condition but also in case of the improved river conditions. Therefore, it is decided to take the following procedure for estimating the sediment transport capacity, though the procedure from (C) is to be carried out in the next stage of study, that is, after deciding the final selected plan.

(A) The standard section with a constant channel inclination is established at each river channel in the present condition and also in the improved conditions.

(B) Sediment transport capacity in each river channel at the different case of discharge is calculated in the present river condition and the improved river conditions by using the selected sediment transport formula.

(C) The rating curves of sediment transport capacity and discharge are made.

(D) The sediment transport capacity at the discharge of 1.1 year return period is obtained from the rating curve. The discharge of 1.1 year return period is considered to be almost the capacity of present river channel or the capacity of low water channel of the improved river channel. The conversion rate between the annual transport capacity and the capacity of 1.05 year runoff in regard to the sediment is obtained in the Nationwide Dredging Report.

(E) The sediment transport capacity is estimated by multiplying the sediment transport capacity of 1.1 year discharge by the conversion rate.

4.3.2 Selection of Sediment Formula

For obtaining the sediment transport capacity, there are not-a-few formulas which are commonly used. Most of the formulas are named by the name of the researcher(s) who made the formula as shown below.

(A) Total sediment transport capacity
- (a) Brown
- (b) Engeland-Hansen
- (c) Einstein-Brown
- (d) Laursen
- (e) Einstein (Modified)

(B) Tractive sediment transport capacity
- (a) Einstein
- (b) Meyer-Peter·Muller
- (c) Sato-Kikkawa-Ashida
- (d) Kalinske-(Brown)
- (e) Isubaki-Shimohara

(C) Suspended sediment transport capacity

(a) Lane-Kalinske

(b) Einstein

(c) Practical (formula)

These formulas are established on the basis of the measurement records at a model channel. The results of calculations by these formulas are usually much different, that is, it is hard to find a representative formula. Therefore, it is decided to use several formulas and to find the reasonable sediment transport capacity from the formula by synthetic consideration.

The following formulas are selected to use. They are comparatively common formulas and the procedures of calculation are comparatively simple.

(A) Brown formula (Total sediment)

$$Qs = 2.633 \times 10^6 B\ (RI)^{2.5}/d$$

(B) Engeland-Hansen formula (Total sediment)

$$Qs = 1.096 \times 10^6\ BR^{2.83}\ I^{2.5}/d$$

(C) Sato-Kikkawa-Ashida formula (Tractive sediments)

$$Qs = 2.705 \times 10^5\ BF\ (t_o/t_c)(RI)^{1.5}$$

(D) Tsubaki-Shinohara formula (Tractive sediments)

$$Qs = Ue \cdot d \cdot 25\ \phi e^{0.8}(\phi e - 0.8\phi c) \times Ws \times 8.64 \times 10^4 \times B$$

(E) Meyen-Peter·Muller formula (Tractive sediment)

$$Qs = \sqrt{(\frac{Ws}{Ww} - 1)gd^3} \times 8(\phi e' - \phi c')^{3/2} \times Ws \times 8.64 \times 10^4\ B$$

(F) Lane-Kalinske formula (Suspended sediment)

$$Qs = q.Ca.P\ \exp(\frac{15\ a\ Wo}{h\ U}).B \times 8.64 \times 10^4$$

$$= q\ Co\ P\ B \times 8.64 \times 10^4$$

where,

- Qs : Sediment discharge (t/day)
- U : Friction velocity (m/s); $U = (g\ R\ I)^{0.5}$
- d : Grain size of bed materials (m)
- B : River width (m)
- Ws : Unit weight of sediment (t/m^3); $Ws = 2.65\ t/m^3$
- Ww : Unit weight of water (t/m^3); $Ww = 1.00\ t/m^3$
- V : Mean velocity (m/s)
- R : Hydraulic mean depth
- I : Energy gradient of flow
- t_o : Tractive force of flow (t/m^2); t_o = Ww R I
- P : Sediment function given in Fig.1.4 - 3
- Ps : Factor related to Manning's roughness
  $Ps = 0.623$ for $n = 0.025$
- t_c : Critical tractive force of bed materials (t/m^2) which is given in Fig. 4 - 3 by Iwagaki
- $F(t_o/t_c)$: Function of the ratio t_o/t_c as shown in Fig. 4 - 3
- n_w : Kinematic viscosity of water; $n_w = 8.5 \times 10^{-7}\ m^2/s$ for temperature 27^oC
- g : Acceleration of gravity; $g = 9.8\ m/sec^2$

$$\varphi = V/U$$

$$\varphi_o = 6.0 + 5.75 \log_{10} (R/d_{65})$$

$$n = \frac{1}{v} R^{2/3}\ I^{1/2}$$

$$\psi = t_o/(\frac{Ws}{Ww} - 1)\ g \times d$$

$$\psi e = \psi \times \frac{\varphi}{\varphi_o}$$

$$Ue = U \times \sqrt{\varphi/\varphi_o}$$

$$\psi c = t_c^{\ 2}/(\frac{Ws}{Ww} - 1)\ g\ d$$

$$\psi c' = 0.047$$

$$\psi e' = \psi(n_6/n)^{3/2}$$

$$n_b = 0.0192\ d_{90}^{\ 1/6}$$

$$Co = 5.55 \Delta F(Wo) \frac{1}{2} (\frac{U}{Wo}) \exp -(\frac{Wo}{U})^2 \quad 1.61$$

q : Discharge per unit width ($m^3/s/m$)

Ca : Density of suspended load at z = a (PPM)

Co : Density of suspended load near river bed (PPM)

P : Function of Wo/U and $n/h^{1/6}$ ----> See Fig. 4-3

n : Manning's coefficient of roughness

$\Delta F(Wo)$: Percentage of content of the sand particles in the river bed of which settling velocity is Wo (%)

Wo : Settling velocity of a particle (cm/s)

$$Wo = \left\{ \sqrt{\frac{2}{3} + \frac{36\nu^2}{Sgd^3}} - \sqrt{\frac{36\nu^2}{Sgd^3}} \right\} \sqrt{S g d}$$

$$S = \frac{Ws}{Ww} - 1$$

ν : Kinematic velocity of water = 0.01 cm^3/s

4.3.3 Analysis of Sediment Transport Capacity

The analyses of sediment transport capacity are carried out by using the formulas selected in the previous subsection and the results of sieving test for the river bed materials sampled during the study in the field. The calculation sheets are attached in Data Book. It is confirmed that the results of calculation are too much different though such results are more or less expected.

It is decided that the results of following formulas are to be used for the further study as these formulas are comparatively adequate for the river conditions of the Panay river. For example the Brown formula and Engeland-Hansen formula re available for the flat river bed where no flow resistance due to the fluctuation of river bed is expected.

(A) Tractive sediment
  (a) Sato-Kikkawa-Ashida formula
  (b) Meyer-Peter Muller formula

(B) Suspended sediment
  (a) Lane-Kalinske formula

The results of calculation by the above formulas are summarized in Table I.4-1.

The results will be available for the further study of river improvement plans.

5. Saline Water Analysis

5.1 General

It is considered that the saltwater intrudes into the Panay river from two courses, that is, the Pontevedra river and the lower Panay river.

In the lower Panay river, the flow from the main Panay river is regulated or limited due to the sedimentation at the upstream courses, that is, at the just downstream from the bifurcation with the Pontevedra river. Especially in the dry season (generally from February to April), the inflow from the main Panay river becomes smaller as the water level at the main Panay river is low. In this condition, the water for irrigation is pumped up from the lower Panay river and the equivalent volume is usually supplied from the downstream, that is, from river mouth. Especially at the high tide, the saline water intrusion reaches to the upstream course as the adverse flow against the saltwater intrusion is not sufficient. This present situation causes the serious problems especially for the domestic water supply to Roxas city.

There are the following two methods to prevent the present problems in regard to the saltwater intrusion to the lower Panay river.

(A) Sufficient fresh water is supplied from the main Panay river by dredging or the other methods.

(B) Checking the saltwater intrusion at the downstream course of lower Panay river by constructing a barrier.

In the Pontevedra river, the saltwater intrusion is not remarkable, that is, the saltwater usually does not reach to the town of Pontevedra at present, possibly due to the much sedimentation in the river mouth. But, this condition will be changed when the river channel improvement works are realized as the river bed is much lowered by dredging. If the saltwater intrusion reaches to the bifurcation point of Pontevedra river and lower Panay river, the saltwater will flow into the lower Panay river and makes the serious problem for the water supply to Roxas city. Therefore, it is necessary to estimate the conditions of saltwater intrusion to the Pontevedra river in case of river improvement plan.

The saline water analysis from the lower Panay river is to be not carried out as there is no plan of river improvement in the lower Panay river except the dredging at the upstream course. In this section, the analysis of saltwater intrusion in Pontevedra is to be studied.

5.2 Saline Water Intrusion Analysis

5.2.1 General

The analysis of saline water intrusion in the Pontevedra river in the improved river condition is carried out by the following procedure.

(a) Judgement of intrusion type

(b) Decision of analysis method

(c) Analysis and conclusion

5.2.2 Type of Saline water Intrusion

The type of saline water intrusion is to be decided by the following.

$C > 0.7$	Weak mixture
$0.2 < C < 0.5$	Moderate mixture
$C < 0.1$	Strong mixture

where, $C = \dfrac{QT}{2\ Pt}$

Q : Mean inflow discharge of river (m^3/s)

T : Cycle time of tide; 12 hours

Pt: Tidal prism, volume of water between MLLWL and MHHWL (m^3)

Q is assumed to be 10 m^3/s which is almost the 95 % dependable discharge of present natural flow and the 80% dependable discharge after taking the max. water requirement for irrigation and M&I water. The back data are to be referred to Subsection 2.2.8 "Dependable Discharge" and Section 2 "Water Budget".

For calculating P, MHHWL and MLLWL are decided at respectively El.1.00 m and El.-0.12 m (to be referred to Subsection 2.4 "Tidal Water") and the improved river-width is assumed to be 80 m which is the river bed width of B1 and B2 plans.

C is calculated as below.

$$C = \frac{10.0 \times 12 \times 3,600}{2 \times 1.12 \times 80 \times 23,000} = 0.105$$

It is judged that the type of saline water intrusion is almost a strong mixture.

5.2.3 Analyses of Saline water intrusion

In case that saline water intrusion is classified as the type of strong mixture, the method of tidal prism is available.

The model of this method is shown in Fig.I.5-1. In this model, the following conditions are assumed.

(a) River cross section is rectangle and the river bed width is 80 m.

(b) River bed elevation is that of improved river channel.

(c) MHHWL is El.1.00 m and MLLWL is El.-0.12 m.

(d) Tidal cycle is 12 hours.

(e) The mixture zone of a concentration is not changed.

(f) The volume of mixture zone is not changed at MLLWL and MHHWL.

(g) The upstream end of mixture zone at MLLWL and the downstream end of mixture zone at MHHWL is the same.

The method and procedure for calculating the saline water concentration are as follows.

(A) Assumption of Discharge

The inflow discharge is to be assumed first. 10 m^3/sec and the other discharges are to be assumed. The following procedure are to be done for each discharge.

(B) Assumption of the upstream end of mixture zone at MLLWL

Several locations for each discharge are to be assumed and the following procedure is to be done for each assumption of the upstream end.

(C) Calculation of the extreme end of saline water

The extreme end is the point where the accumulated volume of river flow during the water level rising from MLLWL to MHHWL is stationed at the upstream end zone. The volume is calculated as below.

$$Po = Q \times 60 \times 60 \times 6$$
$$= 21{,}600\ Q \quad (m^3)$$

(D) Calculation of the downstream end of mixture zone at MLLWL

The downstream end is calculated by assuming the volume of zone between the upstream end of mixture zone (assumed in (B)) and the extreme end of saline water (obtained in (C)) above MLLWL is the same as the mixture zone at MLLWL.

(E) Calculation of the upstream end of saline water intrusion

The upstream end of saline water intrusion is calculated by assuming the volume of mixture zone at MLLWL and that of MHHWL is the same.

(F) Calculation of saline concentration

For the calculation of saline water density, the following calculations are to be performed. Fig.I.6-1 is to be referred to the following.

(a) $r_n = \dfrac{Pn}{Pn + Vn}$

(b) $Qn = \dfrac{R}{r_n}$

where, Qn : Volume of fresh water

R : River inflow discharge volume during a tidal cycle (12 hours)

(c) Saline density

$$Ds = \frac{Pn + Vn - Qn}{Pn + Vn} \times 18{,}000 \ (mg/\ell)$$

The results of calculation are shown in Fig. I.5-2 as relation curves of distance from the river mouth and density (concentration) at some cases of river discharge.

5.3 Intake Period

The intake period of time at the intake in the lower Panay river for M & I water supply to Roxas city is estimated in this Section.

(1) Intake hours

Fig. I.5-2 shows distribution of salinity concentration on a condition of MHHWL. And the distances from river mouth where the saline concentration become 200 mg/l are shown in Fig. I.5-3. 200 mg/l is the limit for domestic water supply in the Philippines. If the water level lowers from MHHWL to MLLWL, salt water moves to downstream. Assuming that salt water moves in proportion to water level, the intake hour is calculated by the following equation. Fig. I.5-4 will be available for understanding the equation.

$$T = (180 + 2\sin^{-1} (h/0.56))/360 \times 24 \text{ (hour)}$$

where, T (hours): Intake hours

h (m) : H - 0.44

H (m) : H = 1.00 - 1.12 x L/Lo
Upper limit water level not to takein salt water

L (km) : Distance from the junction to the point where concentration is 200 mg/l on a condition of MHHWL

Lo (km) : Moving distance of saltwater zone when the water level lowers from MHHWL to MLLWL.
Lo = (1.0-(-0.12)) i = 5,600 m

1.00 (m) : MHHWL

1.12 (m) : Defference of elevation between MHHWL and MLLWL

0.44 (m) : MSL

0.56 (m) : Half of 1.12

Results are shown in the following Table.

Discharge (m^3/sec)	L (km)	H (m in EL)	T (hours)
10.0	-	-	24
9.0	-4.2	-	24
8.5	-1.7	-	24
8.0	0.0	1.0	24
7.5	1.5	0.7	19.1
7.0	3.1	0.38	17.7
6.5	5.0	0.00	8.5
6.0	7.1	-0.42	0

(2) Flow in the Lower Panay River

The type of flow in the Lower Panay river is actually unsteady flow as water level varies at the junction and accordingly the inflow varies. However, it is approximately analyzed as non-uniform-flow on a condition of water levels.

The flow analysis is to be performed on the following conditions.

(a) The flow dereases in proportion to the distance from the junction as the water is pumped up by pump facilities along the river (refer to Fig. I.5-5).

(b) Initial depth at ROX-WD pump station is 1.0 m.

(c) Coefficient of roughness (n) is 0.04 in Manning formula.

Fig. I.5-6 shows the inflow and water level at the junction. The calculation sheets of non-uniform-flow are to be attached in Data Book.

(3) Intake Volume

On a condition of each discharge in the main Panay river, maximum, minimum and average inflow to the Lower Panay river are shown in the following table, where the maximum is the flow when water level at the junction is H (upper limit water level not to take in salt water) and the minimum is the flow when the water level is MLLWL.

Discharge	Inflow (m^3/sec)		
	Maximum	Minimum	Average
8.0	6.0	1.8	3.9
7.5	4.4	1.8	3.1
7.0	3.1	1.8	2.4
6.5	2.0	1.8	1.9
6.0	1.1	1.8	1.4

The intake volume is the average rate of flow multiplied by intake hours as follows.

Intake Volume (m^3/day)

Discharge (m^3/sec)	Average rate of flow (m^3/sec)	Intake hours (hour)	Intake volume (m^3/day)
8.0	3.9	24	336,960
7.5	3.1	19.1	213,156
7.0	2.4	17.7	152,928
6.5	1.9	8.5	58,140
6.0	1.4	0	0

(4) Consideration

The total capacity of pump facilities along the upstream reach of the proposed tide gate is about 1.07 m^3/sec, or 92,000 m^3/day. If inflow volume is less than 92,000 m^3/s, the water flowing into the river will be used up for irrigation purpose and does not reaches up to the intake of pump station for M & I water supply.

Therefore, the inflow volume required is more than 92,000 m^3/day, which is equivalent to more than 6.7 m^3/sec of discharge as shown in Fig. I.5-7.

6. Water Budget

6.1 General

The water budget calculation for the Panay river basin is done on the following conditions.

(A) Irrigation water already taken at present by National irrigation systems (NIS), Communal irrigation system (CIS), Pump irrigation system (PIS), and other private irrigation systems shall be not considered for the water budget calculation. That is, it is assumed that the present use for irrigation have been continued in the past and would be not changed in the future, therefore, the observation of stream flow have been carried out in the condition of the present water use for irrigation.

(B) It is assumed that the intake loss is negligibly small. That is, the intake efficiency can be neglected.

(C) It is assumed that the water can be taken in prior to the river maintenance flow.

(D) It is assumed that the municipal and industrial water can be taken in prior to the irrigation water.

(E) It is assumed that the return flow of used water happens in the same month and the rate of return flow is 50% of used water.

(F) It is decided that 95% dependable flow shall be sufficient for the M & I water requirement and 80% dependable flow after taking M & I water preferencially shall be sufficient for the irrigation water requirement.

6.2 Diagram

The schematic diagram for water budget calculation is shown in Fig. I.6-1.

For making the diagram and calculations as simple as possible, the following three locations are decided for the representative intake.

(A) Intake 1 is located in the Panay river at just downstream of confluence with the Badbaran river for the municipal and industrial (M & I) water use of Cuartero, Dao, Dumalag, and Dumarao.

(B) Intake 2 is located at Mambusao weir in the Mambusao river for the M & I water use of Mambusao and Sigma and the irrigation water use of Mambusao area.

(C) Intake 3 is located in the Panay river at just downstream of confluence with the Maayon river for the M & I water of Panitan, Maayon, Panay, Pontevedra and Roxas city and the irrigation water of Panitan-Panay area.

At Intake 3, the actual intake for M & I water of Panay and Roxas city is located in the lower Panay river. But, it is assumed that the water can be taken at the intake in the Panay river at just downstream of confluence with the Maayon river for making the water budget calculation simple.

6.3 Water Requirement

The water requirements at present (1983) and in the future (1990, 2000, 2010, 2020 and 2030) are estimated. The requirements at the following purposes and locations are shown in Table I.6-1.

(A) at Intake 1

U1 : M & I for Cuartiro, Dao, Dumalag and Dumarao

(B) at Intake 2

U2A: M & I for Mambusao and Sigma
U2B: Irrigation for Mambusao Project area

(C) at Intake 3

U3A: M & I for Panitan and Maayon
U3B: M & I for Roxas city, Panay and Pontevedra
U3C: Irrigation for Panitan-Panay project area

For the monthly water requirement of irrigation, the requirement in the year with 5-year return period is used.

The water requirement for irrigation is monthly variable. Therefore, the month of which water balance seems to be most severe is selected by comparing the requirement and the monthly average discharge. It can be said that the water in the other months is sufficient if the dependable discharge in the month is more than the requirement.

The month used for the water budget calculation of irrigation use at Intake 2 and 3 is decided as follows.

(A) at Intake 2

February and June

(B) at Intake 3

April and May

6.4 Dependable Discharge

The dependable discharge is a possible intake discharge with a certain dependability such as 95% and 80% of the period.

For obtaining the dependable discharge, the daily average discharge records at the following gaging stations are available.

(A) Cuartero gaging station (1956 - 1978)

(B) Mambusao gaging station (1950 - 1977)

The following procedures are taken for obtaining the dependable discharge at the intake.

(A) The dependable discharge at these two stations are calculated from the daily discharge records.

(B) The conversion rates from the records at a gaging station to the records at a intake point are calculated by using the proportion of rainfall and catchment area between the two points.

(C) The dependable discharge at the intakes is obtained by the dependable discharge at a gaging station multiplying by the conversion rate.

The results to get the dependable discharge at the intakes are shown in Table I.6-2.

6.5 Water Budget

The summary for comparing the water requirement and the dependable discharge is shown in Table I.6-3.

It is concluded that the dependable discharge fulfills the water requirement at all the intakes. That is, 95% dependable flow fulfills the requirement of M & I and 80% dependable flow fulfills the requirement of irrigation even in the month when the water budget seems to be most tight.

Further, it is noted that the incremental discharge would be expected if the Panay B dam is constructed. That is, the effect of lowflow augmentation by the flow regulation for hydropower generation of dam may increased the discharge in dry seasons.