Supporting-F Hydrological Analysis

SUPPORTING-F HYDROLOGICAL ANALYSIS

1. GENERAL

1.1 INTRODUCTION

This supporting report is a part of the report for the project entitled “The Study on Water Supply System for Tegucigalpa Urban Area in the Republic of Honduras”.

From the view point of water resource development, the particular drainage basins were selected for the hydrological study as follows:

- The Guacerique River Basin,
- The Sabacuante River Basin and
- The Tatumbla River Basin.

The above drainage basins are shown in *Figure F.1.1*.

1.2 OBJECTIVES AND SCOPE

The content of the Supporting Report F was divided into 2 parts; the Master Plan Study and the Feasibility Study.

1.2.1 MASTER PLAN STUDY

The objectives of the hydrological analysis in the Master Plan Study are to clarify the relationship of the rainfall and its return period, the runoff generated by the rainfall in each basin, and the water resources potential in that basin.

The scope of work covered necessary hydrological analyses for the Master Plan Study as follows:

- Frequency analysis,
- Rainfall-runoff analysis and
- Water balance analysis.

The frequency analysis was conducted to clarify the relationship of rainfall and its return period, and the relationship of flow during dry period and its return period by using the standard Gumbel method.

The Rainfall-runoff analysis was then conducted to reveal the runoff during flood period generated by the maximum or design rainfall. A Storage Function method with the Newton – Ralpson interpolation was employed for this analysis.

From these results, the water balance analysis in the basin with a proposed dam was calculated to clarify the maximum available yield for water supply in that basin.

1.2.2 FEASIBILITY STUDY

The objectives of the Feasibility Study are to clarify the impact of the new Los Laureles II dam to the river after the completion of the dam, and to propose the gate operation and the flood forecasting.

The scope of work covered the river modeling, the hydraulic simulation of the selected project in the Master Plan Study, the proposal for gate operation and flood forecasting. The river model was formulated from the river survey data. The hydraulic simulation was then conducted to verify the impact of the new dam to the river. From the hydraulic simulation results, the gate operation and flood forecasting were proposed for the optimum use of the storage volume.

1.3 GENERAL HYDROLOGICAL CONDITION

The Study Area covers 3 drainage basins in the southern part of Tegucigalpa, the Guacerique, the Sabacuante and the Tatumbla river basin. The climate in the Study Area is as follows:

Average maximum temperature	:	29	°C
Average minimum temperature	:	17	°C
Average temperature	:	23	°C
Evaporation	:	670	mm/year

The precipitation is slightly different in each basin as shown in the following table.

Table F.1.1 Precipitation in the Study Area

Items	Unit	Guacerique River Baisn At Guacerique II	Guacerique River Baisn At Quiebra Montes	Sabacuante River Basin	Tatumbla River Basin
Annual Precipitation	mm/ year	945	1,064	841	783
Average Precipitation from May to October	mm/ 6 months	842	957	704	655
Average Precipitation from November to April	mm/ 6 months	103	107	137	128

Source : SANAA

1.4 LITERATURE REVIEW

There are some hydrological studies conducted in the Study Area. The related studies are described in the following text briefly.

In 1989, SANAA conducted a study project entitled (in Spanish) "Actualizacion del Plan Maestro de Abastecimiento de Agua Potable de Tegucigalpa". The project estimated the maximum yields of the high potential river basins as follows:

Table F.1.2 Summary of the Proposed Dams by SANAA in 1989

Items	Unit	Proposed Dam		
		Quiebra Montes Dam	Sabacuante Dam	Tatumbla Dam
Total Storage Volume	m^3	50,000,000	24,300,000	15,700,000
Normal Water Level	m	1,147	1,122	1,165
Minimum Water Level	m	1,112	1,084	1,133
Maximum Yields at Confidential range 99%	l/s	1,110	320	250

Source : SANAA

SANAA also conducted a study project entitled (in Spanish) “Diagnostico de las Obras de Captacion del Sistema de Abastecimiento Hidrico de Tegucigalpa” funded by Banco Interamericano de Desarrollo. The project reviewed the previous study and estimated the water resource potential in the Guacerique river basin and the Sabacuante river basin.

Maximum yield of the existing Los Laureles dam was as follows:

Table F.1.3 Maximum Yield of the Existing Los Laureles Dam

Items	Unit	Existing Los Laureles Dam
Total Storage Volume	m^3	12,000,000
Normal Water Level	m	1,033
Maximum Yields for Water Supply	l/s	660
Flow rate at 1,000 years	m^3/s	920

Source : SANAA

The study recommended the improvement of the operation of the rubber gate at the existing Los Laureles dam during the flood and dry seasons and others.

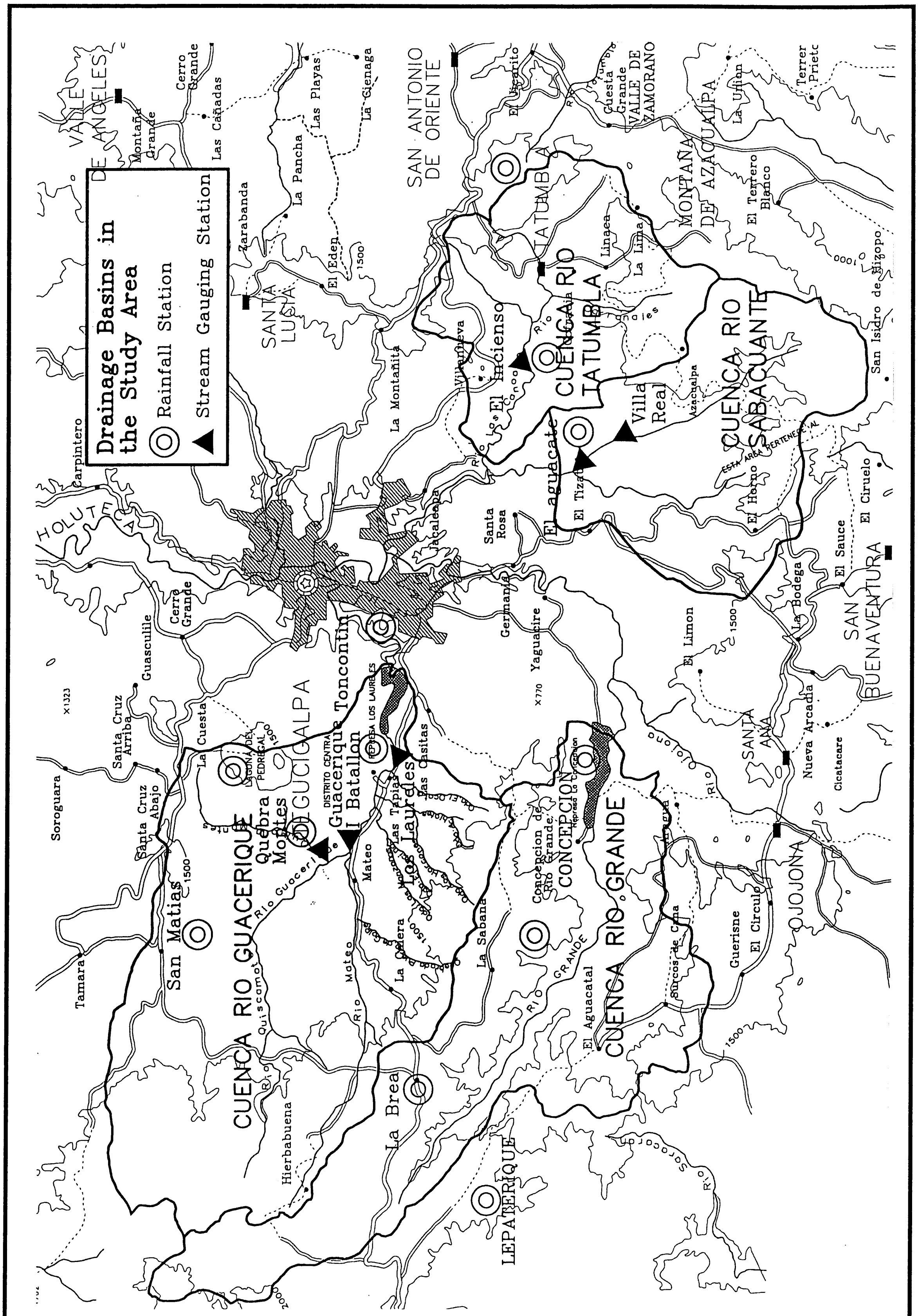


Figure F.1.1	Drainage Basins in the Study Area

2. GUACERIQUE RIVER BASIN

2.1 RIVER CONDITION

Guacerique river originates in the Rincon Dolares mountains, but with a different name, and branches into many tributaries in the upstream. The river takes its name after the confluence of the tributaries: Quebradra Quiscamnote and Quebrada Ocote Vuelto in the midstream, and then meets its main tributaries, Quiebra Montes and Mateo river at Mateo. The river flows eastwards to its end point in the Study Area at the Los Laureles dam in Los Laureles.

The overall drainage basin occupies the area in the south-western part of Tegucigalpa, in the Choluteca river basin, with a total drainage area of about 194 km^2 at the Los Laureles dam as shown in *Figure F.1.1*. The main sub-drainage basin at Guacerique II station and Quiebra Montes station occupy the area of about 174 and 125 km^2 respectively. The sub-basin areas are as follows:

Table F.2.1 Drainage Basins of Guacerique River

River/Location	Basin Area (km^2)	
	Sub-basin	Total
Guacerique Upstream	102	102
Quiebra Montes	23	125
Guacerique II Station	-	148
Mateo Bridge	-	174
Los Laureles II *	-	190
Los Laureles Dam	-	194

Source : SANAA

*Los Laureles II is the proposed location for a new dam as explained later

The Los Laureles dam was constructed with the main purpose as a water source for water supply system in Tegucigalpa during 1974 - 1976. The dam is located at the elevation of about 1,037 m above mean sea level, with the height of about 55 m, the storage capacity of about 12 millions m^3, and the maximum yield of about 660 l/s.

2.2 AVAILABLE DATA

2.2.1 RAINFALL

Rainfall data are available at the meteorological stations of SANAA in the basin as follows:

Table F.2.2 Rainfall Stations in the Guacerique River Basin

Station	Recorded Data	
	years	Range
Batallon	38	1963 - Present
Quiebra Montes	9	1992 - Present
Toncontin (Tegucigalpa)*	50	1951 - Present

Source : SANAA

*It should be noted that Toncontin station is not in the Guacerique river basin

Rainfall data are recorded regularly 4 times a day at 6:00, 12:00, 18:00 and 24:00, daily rainfall is the summation of these recorded data.

The average annual rainfall at Batallon station and Quiebra Montes station is 945 mm and 1,064 mm respectively. Annual rainfalls in the basin are shown in *Appendix F, Table AF.2.1*.

2.2.2 WATER LEVEL AND FLOW RATE

Data on water level and flow rate are available at the stream gauging stations of SANAA in the basin as follows:

Table F.2.3 Stream Gauging Stations in the Guacerique River Basin

Station	Recorded Data	
	years	Range
Daily Data		
Batallon*	10	1964 - 1973
Guacerique II	15	1982 - 1996
Quiebra Montes	7	1991 - 1997
Los Laureles	2	1999 - Present
Non-Daily Data		
Guacerique II	11	1990 - Present
Quiebra Montes	11	1990 - Present

Source : SANAA

* Data at Batallon are not complete and not in a digital format

A summary of rainfall and stream gauging stations is shown in *Table F.2.4*.

In general, data on flow rate are recorded regularly twice a day in the morning and afsternoon. The record at Batallon station was halted during the construction of the Los Laureles dam in 1974, then a new station, Guacerique II station, was set up again in 1982, a few years after the completion of the dam. Another station, Quiebra Montes station, was also set up in 1991. Although this station is named as Quiebra Montes, it is actually located in the upstream of Guacerique river just before the confluence of Guacerique river and Quiebra Montes river.

However Guacerique II station and Quiebra Montes station were severely damaged by the Hurricane Mitch in 1998 and the record was halted. In 1999, a new station, Los Laureles station, was set up at the Mateo bridge and has been the only station to record the flow rate in the basin since then.

There are also some non-daily recorded data at Guacerique II station and Quiebra Montes station after the Hurricane Mitch. These data are used as a reference in this study, but not for the analyses.

The annual maximum, average and minimum flow rates of the main stations recorded are shown in *Appendix F, Table AF.2.2* and are summarized as follows:

Table F.2.5 Average Flow Rate in the Guacerique River Basin

Station	Flow Rate (m^3/s)		
	Maximum	Minimum	Average
Guacerique II	217.0	0.011	1.393
Quiebra Montes	10.9	0.040	0.566

Source : SANAA

It should be noted that the maximum flow rate shown above was the average monthly peak flow rate at Guacerique II station and Quiebra Montes station. The flow rate of these 2 stations did not reach the peak at the same time.

2.3 FREQUENCY ANALYSIS

Frequency analysis of the recorded data was conducted to clarify its return period by using the standard Gumbel method. Theoretical background of this method is shown in the *Appendix AF.1*. The analysis was divided into 2 parts as follows:

- Frequency analysis for rainfall and
- Frequency analysis for flow rate during drought period.

2.3.1 FREQUENCY ANALYSIS FOR RAINFALL

Although the recorded rainfall range at Batallon station is sufficiently long for the analysis, data on the torrential rainfall in 1998 during the Hurricane Mitch was missing. And also the rainfall at Quiebra Montes station is not sufficiently long. The rainfall at Toncontin station in Tegucigalpa was therefore used instead.

The average annual rainfall at each station is summarized as follows:

Table F.2.6 Annual Rainfall at Batallon, Quiebra Montes and Toncontin Stations

Station	Average Rainfall (mm)
Batallon	945
Quiebra Montes	1,064
Toncontin	866

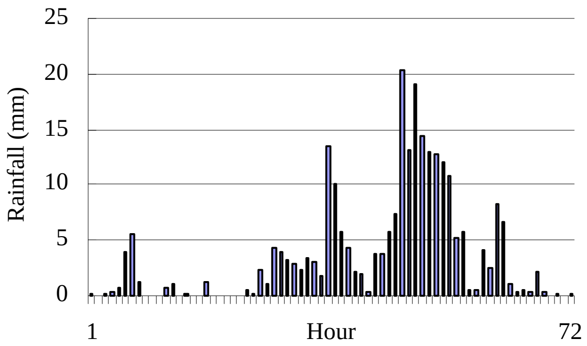
Source : SANAA

The average annual rainfalls at these 3 stations are slightly different, but it was reported that during the Hurricane Mitch, the distribution of rainfall was apparently uniform over the entire region. Therefore, the rainfall pattern in the analysis was considered similar, but different in magnitude for the entire Study Area.

At first, the frequency analysis was conducted for the daily rainfall data at Toncontin station from 1951 to 1999. The maximum daily rainfall at Toncontin station is shown in *Appendix F, Table AF.2.3*. The relationship of maximum daily rainfall and return period at Toncontin station is analyzed and shown in *Figure F.2.1*.

The hourly rainfall pattern at Toncontin station during the Hurricane Mitch had its peak at 120 mm, and the total rainfall in 72 hours was 256 mm. The rainfall pattern is as follows:

Figure F.2.2 Recorded Rainfall at Toncontin during the Hurricane Mitch



The design rainfall pattern at each return period at Toncontin station was constructed from the hourly rainfall pattern during the Hurricane Mitch.

The design rainfall pattern in the Guaecerique river basin was then constructed from this by using the ratio of the peak rainfall during the Hurricane Mitch at these 2 stations as follows:

Table F.2.7 Peak Rainfall during the Hurricane Mitch in 1998

Station	Rainfall (mm)	Ratio
Quiebra Montes	215	1.80
Toncontin	120	

From this ratio, the design rainfall pattern in the Guacerique river basin was constructed at each return period. Design maximum rainfalls at Toncontin station (calculated from the frequency analysis) and at Guacerique station (calculated from the peak rainfall ratio during the Hurricane Mitch) are as follows:

Table F.2.8 Design Maximum Daily Rainfall in the Guacerique River Basin

Return Period (Year)	Design Maximum Daily Rainfall (mm)	
	Toncontin	Guacerique river basin
10	92	165
20	104	186
During Mitch (about 50 – 60 years)	120	215*
200	142	254

Note : * This is the measured data during the Hurricane Mitch, not the calculated value

These design rainfalls, together with the synthetic rainfall pattern, were used in the Rainfall-runoff analysis for the Guacerique river basin in the latter section.

2.3.2 FREQUENCY ANALYSIS FOR FLOW RATE DURING DROUGHT PERIOD

Frequency analysis was also conducted for the minimum flow to clarify the water resources potential in drought period. The daily minimum flow rate during 1982 – 1996 at the Guacerique station was used in the analysis. The minimum flow rate is shown in *Appendix F, Table AF.2.4*. The result is shown in *Figure F.2.3* and summarized as follows:

Table F.2.9 Minimum Flow at Guacerique II Station

Probability of Non-exceedance, F(x)	Flow (m^3/s)
0.99	0.0144
0.95	0.0157
0.90	0.0174
0.85	0.0192
0.80	0.0211
0.70	0.0253

2.4 RAINFALL - RUNOFF ANALYSIS

Rainfall-runoff analysis was conducted by using a standard Storage Function method. Theoretical approach of this analysis is explained in *Appendix AF.1*.

As explained in the previous section, hourly rainfall data at Toncontin station during the Hurricane Mitch were used to construct the design rainfall pattern in the Guacerique river basin instead of using the measured data from Batallon station and Quiebra Montes station due to the data shortage. The synthetic rainfalls were then input into the Rainfall-runoff model for the calculation of runoff.

Necessary parameters in the Storage Function model shown in the following table were set up according to the actual field condition and calibrated.

Calibration was done by using the overflow at the existing Los Laureles dam as the reference for comparison. The parameters in the model were adjusted so that the simulated flow from the model had negligibly small discrepancy in comparison with the flow rate at the existing dam.

Table F.2.10 Parameters in the Rainfall-runoff Analysis

Parameter	Value
k	0.25
p	0.5
Area	190 km^2

Note : All parameters are referred in the *Appendix AF.1*

Relationship of the rainfall and simulated hydrograph from the Storage Function method is shown in *Appendix F, Figure AF.2.1*. Simulated hydrograph during the Hurricane Mitch is shown in the following figure. Relationship of the runoff (peak of the simulated hydrograph) and its return period is shown in *Figure F.2.4* and a summary is also shown in the following table.

Figure F.2.5 Simulated Hydrograph during the Hurricane Mitch in Guacerique River

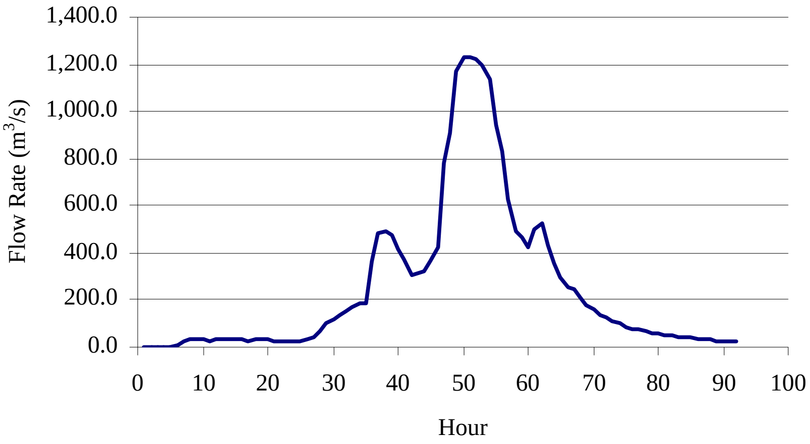


Table F.2.11 Runoff in the Guacerique River Basin

Return Period (Year)	Runoff (m^3/s)
2	558
5	762
10	900
20	1,034
During Mitch (about 50 – 60 years)	1,250
200	1,497
500	1,686

These results were used to determine the scale and normal water level of the proposed dam.

2.5 WATER BALANCE ANALYSIS

Water balance analysis was conducted to clarify the maximum yield of a dam. The maximum yield is the available maximum water quantity for water supply that would not make the reservoir dried up. The concept of water balance is as follows:

Inflow – Outflow – Evaporation = Change in the Storage Volume

Inflow in the Guacerique river basin was the monthly flow rate in the river measured at Guacerique II station between 1982 - 1996. Outflow was a combination of the excess flow over the dam and the maximum available yield.

In order to calculate the storage volume, the relationship of the water level and storage volume (H-V Curve) is necessary. This curve was prepared by using the measured water level and storage volume as the basic data, then the best-fitted curve for these data was calculated with the correlation coefficient of at least 95%. The equation of this curve is shown in *Appendix F, Table AF.2.5.*

2.5.1 EXISTING LOS LAURELES DAM

Water balance was calculated to clarify the maximum available yield of the existing Los Laureles dam.

Calculation was divided into 2 cases as follows:

- Case EL-I : Without sedimentation and upstream water intake
  (Condition in the Master Plan for the dam)
- Case EL-II : With sedimentation and upstream water intake
  (Present condition of the dam)

In case EL-I, the water level – storage volume curve (H-V curve) in the original Master Plan for the Los Laureles Dam was used. This curve was prepared without the consideration of the sedimentation and the water intake in the upstream. The normal water level of the dam was set at 1,033.0 m by considering the use of the rubber gate. This rubber gate reportedly could lift up the water level for about 3 m.

In case EL-II, the update water level – storage volume curve (H-V curve) was used. This update curve was prepared by considering the present storage volume that was less than the original storage volume due to the sedimentation and the water intake in the upstream. Although the definite quantity of water intake was not known, it was estimated from the hearing survey with the users. The normal water level of the dam was 1,033 m with the use of the rubber gate.

From the above combination, the maximum yield of the existing Los Laureles dam was calculated for 2 cases as follows:

Table F.2.12 Calculation for the Existing Los Laureles Dam

Case	Sedimentation and Upstream Water Intake	Rubber Gate	Normal Water Level (m)
EL-I	No	Yes	1,033.0
EL-II	Yes	Yes	1,033.0

Note : “EL” refers to the Existing Los Laureles Dam

The maximum yields of the existing Los Laureles dam are as follows:

Table F.2.13 Maximum Yield of the Existing Los Laureles Dam

Case	Type	Normal Water Level (m)	Total Storage Volume (m^3)	Maximum Yield (l/s)
EL-I	Concrete	1,033.0	11,927,494 (12,000,000)	725
EL-II	Concrete	1,033.0	9,171,216 (9,000,000)	540

Note : “EL” refers to the Existing Los Laureles Dam
The values in parenthesis are the rounded up values

2.5.2 PROPOSED LOS LAURELES II DAM

Water balance was also calculated to clarify the maximum available yield of the proposed Los Laureles II dam using the same methodology as mentioned above.

Sedimentation was taken into consideration by assuming the sediment volume with partial deposition for 50 years with some dredging. The expected sediment was about 2,000,000 m^3.

Variation of the storage change due to water supply intake at the maximum yield is shown in *Appendix F, Figure AF.2.2.* The result can be summarized as follows:

Table F.2.14 Maximum Yield of the Proposed Los Laureles II Dam

Proposed Los Laureles II Dam	Characteristics	Unit
Type	Concrete	-
Normal Water Level	1,053.0	m
Available Storage Volume (rounded up)	4,089,518 (4,000,000)	m^3
Sedimentation (rounded up)	2,094,798 (2,000,000)	m^3
Maximum Yield	176	l/s

Note : The values in parenthesis are the rounded up values

2.5.3 PROPOSED QUIEBRA MONTES DAM

Water balance was also calculated to clarify the maximum available yield of the Quiebra Montes dam using the same methodology as mentioned above.

Flow rate data from Quiebra Montes station are available only from 1992 to 1997 without the data in the driest year in 1988. Therefore, for comparison, the data from 1982 to 1992 was simulated from Guacerique II station using the average specific discharge in the basin.

The calculated maximum yield is as follows:

Table F.2.15 Maximum Yield of the Proposed Quiebra Montes Dam

Proposed Quiebra Montes Dam	Characteristics	Unit
Type	Concrete	-
Normal Water Level	1,147.0	m
Available Storage Volume	49,000,000	m^3
Sedimentation	-	m^3
Maximum Yield	1,134	l/s

Note : Measured Data during 1992 – 1997, and Simulated Data during 1982 – 1992
The values in parenthesis are the rounded up values

This maximum yield was calculated by combining the measured and simulated data. Thus, the calculation range was from 1982 to 1997.

2.5.4 COMBINATION TWO DAMS

(1) Combination of Los Laureles and Los Laureles II Dam

Water balance was calculated to clarify the maximum yields in case of the combination of the existing Los Laureles dam and the proposed Los Laureles II dam. The operation rule of both dams in the calculation was assumed as follows:

- Priority was put on the existing dam,
- In the flood season, water was taken from both dams. The excess water in the proposed dam was released to the existing dam in the downstream. In case that the existing dam was in its full capacity, the excess water was released over to the downstream as well, and

- In the dry season, when both dams were not in their full storage capacity. Water intake of the existing dam was kept constant at its maximum yield meanwhile water intake of the proposed dam was reduced in order not to make the storage of both dams dried up.

The normal water level of the existing Los Laureles dam and proposed Los Laureles II dam was set at 1,033.0 m and 1,053.0 respectively.

It is expected that sedimentation may be a main factor to reduce the storage volume and the maximum yield. Therefore the calculation was conducted by considering the sediment volume of about 2,000,000 m^3.

Based on this operation rule, the maximum yields of both dams were calculated as shown in the following table.

Table F.2.16 (1) Maximum Yields of Two Dams (Los Laureles and Los Laureles II Dam)

Dam	Normal Water Level (m)	Available Storage Volume (m^3)	Sedimentation (m^3)	Maximum Yield (l/s)	Total Yield (l/s)
Existing Los Laureles	1,033.0	9,171,216 (9,000,000)	-	540	670
Proposed Los Laureles II	1,053.0	4,089,518 (4,000,000)	2,094,798 (2,000,000)	130	

Note : The values in parenthesis are the rounded up values.

(2) Combination of Los Laureles and Quebra Montes Dam

Water balance was calculated to clarify the maximum yields in case of the combination of the existing Los Laureles dam and the proposed Quebra Montes dam. The operation rule of both dams was similar as those in the combination of the existing Los Laureles dam and the proposed Los Laureles II dam.

Yield of the existing Los Laureles dam was kept constant. The proposed Quebra Montes dam would supply the water to the existing Los Laureles dam to achieve its maximum yield as the first priority in the dry season, then the proposed Quebra Montes dam would take the remaining water at its maximum yield.

The normal water level of the existing Los Laureles dam and proposed Quebra Montes dam was set at 1,033.0 m and 1,147.0 respectively.

Based on this operation rule, the maximum yields of both dams were calculated as shown in the following table.

Table F.2.16(2) Maximum Yields of Two Dams (Quebra Montes and Los Laureles Dam)

Dam	Normal Water Level (m)	Available Storage Volume (m^3)	Sedimentation (m^3)	Maximum Yield (l/s)	Total Yield (l/s)
Existing Los Laureles	1,033.0	9,171,216 (9,000,000)	-	544	1,654
Quebra Montes	1,147.0	49,000,000	-	1,110	

Note : The values in parenthesis are the rounded up values.

2.5.5 COMBINATION OF THREE DAMS

Water balance was also calculated to clarify the maximum yields for the combination of the existing Los Laureles dam, the proposed Los Laureles II dam and the Quiebra Montes dam. The operation rule of all dams in the calculation was as follows:

- Priority was put on the existing Los Laureles dam and the proposed Los Laureles II dam,
- In the flood season, water was taken from all dams. The excess water in the Quiebra Montes dam was released to the proposed Los Laureles II dam and then to the existing Los Laureles dam in the downstream. In case that the existing Los Laureles dam was in its full capacity, the excess water was released over to the downstream as well, and
- In the dry season, when all dams were not in their full storage capacity. Water intakes of the existing Los Laureles dam and proposed Los Laureles II dam were kept constant at their maximum yields meanwhile water intake of the proposed Quiebra Montes dam was reduced in order not to make the storage of both dams dried up.

Calculation was also conducted by considering the sediment volume of about 2,000,000 m^3 at the proposed Los Laureles II dam.

Maximum yields for this combination are shown as follows:

Table F.2.17 Maximum Yields of Three Dams

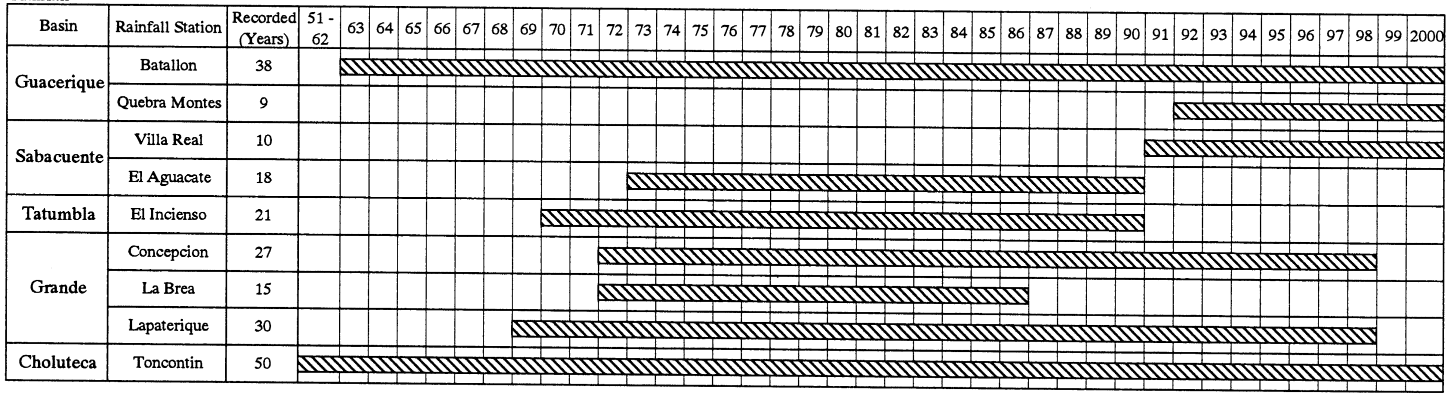
Dam	Normal Water Level (m)	Available Storage Volume (m^3)	Sedimentation (m^3)	Maximum Yield (l/s)	Total Yield (l/s)
Existing Los Laureles	1,033.0	9,171,216 (9,000,000)	-	540	1,710
Proposed Los Laureles II	1,053.0	4,089,518 (4,000,000)	2,094,798 (2,000,000)	130	
Proposed Quiebra Montes	1,147.0	49,000,000	-	1,040	

Note : The values in parenthesis are the rounded up values

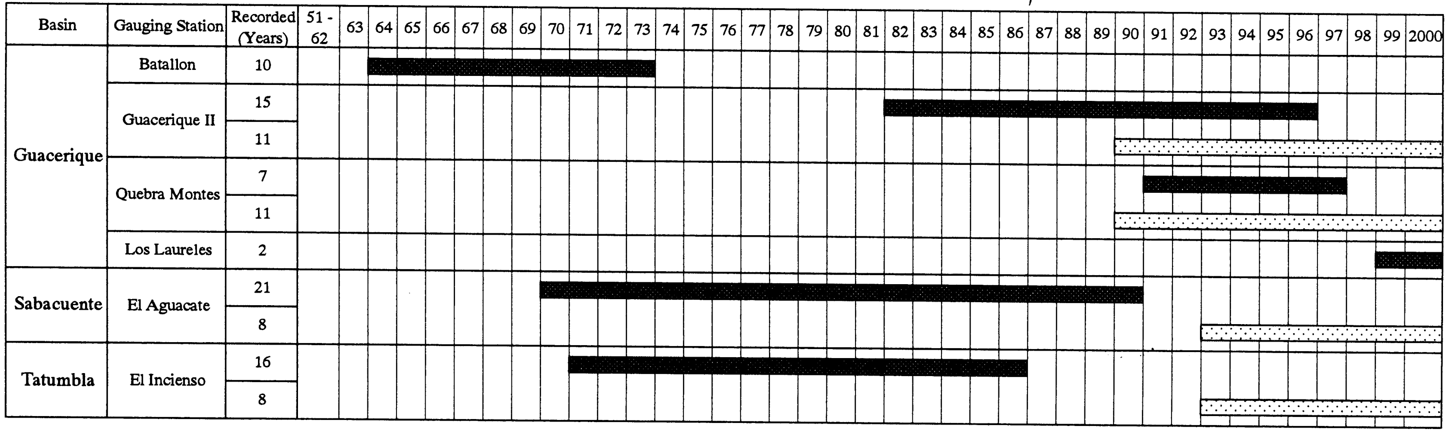
Variation of the storage change due to water supply intake at the maximum yield for each case is shown in *Appendix F, Figure AF.2.2.*

Table F.2.4 Rainfall and Stream Gauging Stations in the Study Area

Rainfall



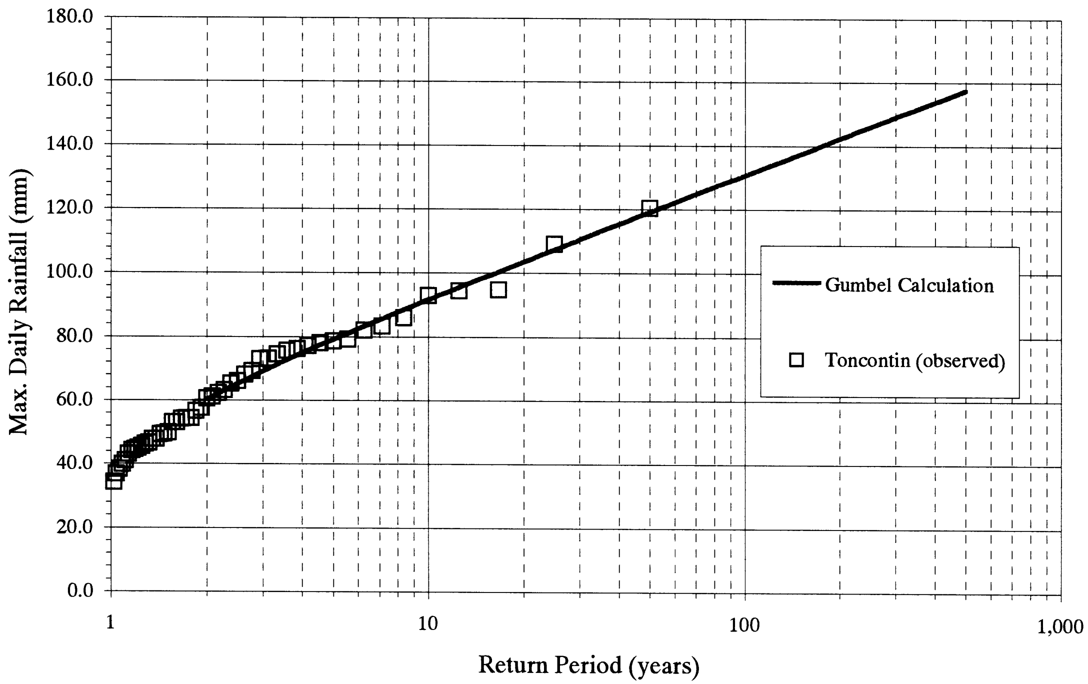
Flow Rate



Remarks : Sources of data are SANAA and SERNA

= daily rainfall data = daily flow data = non-daily flow data

Toncontin Station
(Data from 1951 to 1999)



Return Period (yr)	Daily Rainfall (mm)
500	157.5
400	153.7
300	149.0
250	145.9
200	142.2
150	137.4
100	130.6
80	126.9
60	122.1
50	119.0
40	115.2
30	110.4
25	107.3
20	103.5
15	98.6
10	91.5
9	89.7
8	87.6
7	85.2
6	82.4
5	79.0
4	74.8
3	69.1
2	60.2

Figure F.2.1 **Relationship of Maximum Daily Rainfall and Return Period at Toncontin Station**

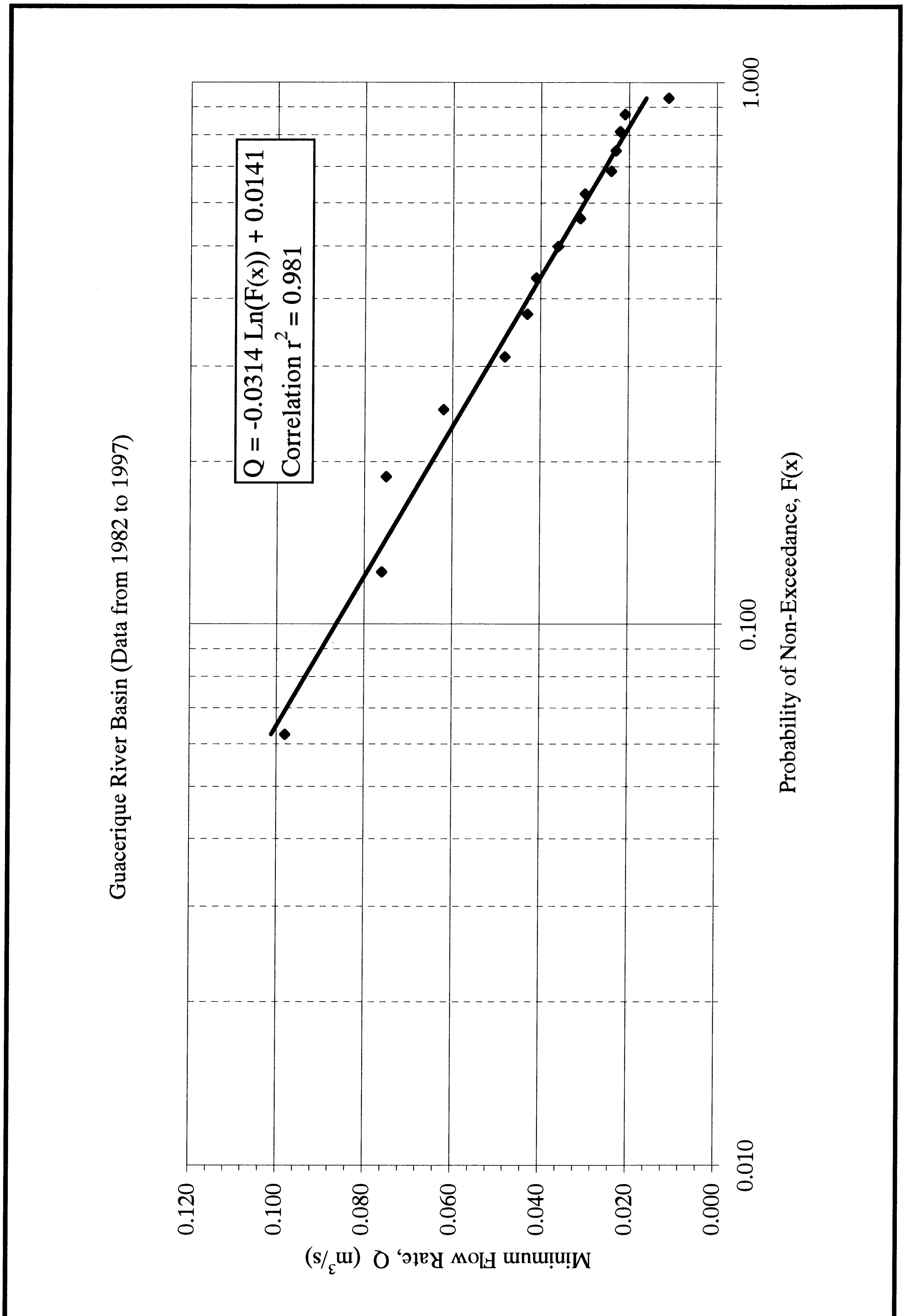
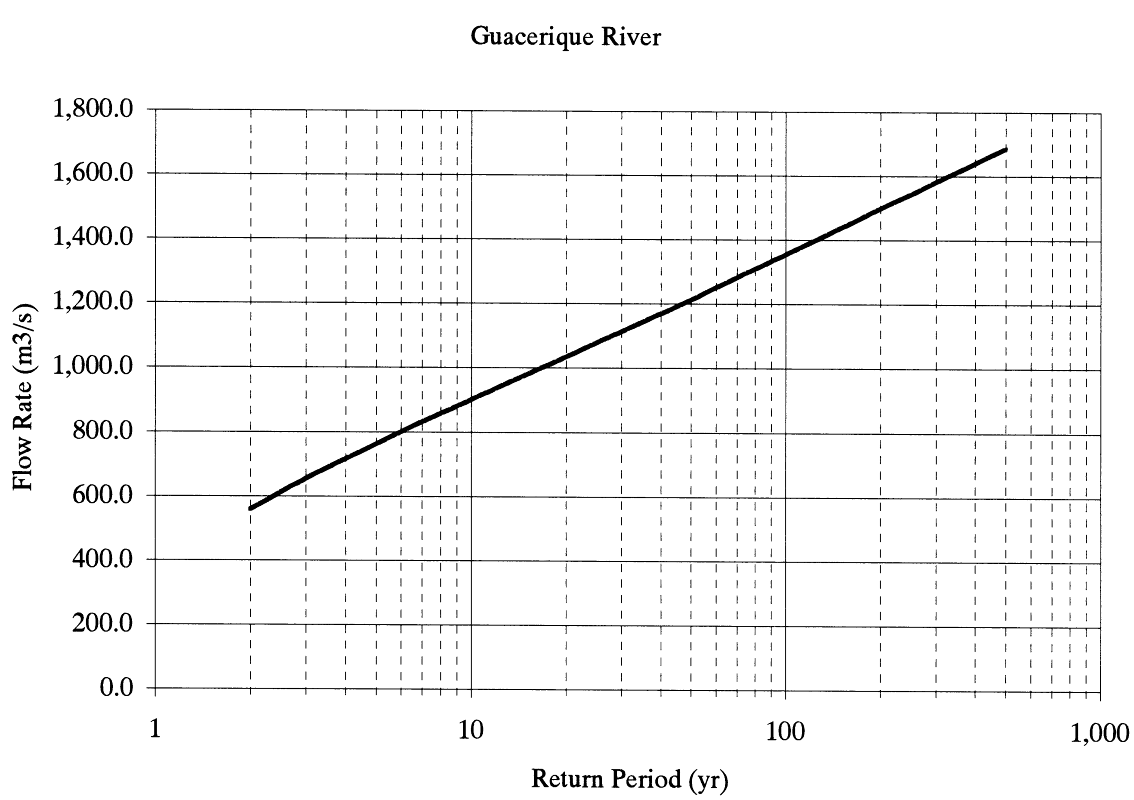


Figure F.2.3	Relationship of Minimum Flow Rate and Probability of Non-Exceedance in the Guacerique River Basin



Return Period (yr)	Flow Rate (m3/s)
500	1,686.2
400	1,639.1
300	1,580.9
250	1,542.5
200	1,496.8
150	1,437.7
100	1,354.2
80	1,308.8
60	1,250.2
50	1,212.5
40	1,168.2
30	1,112.9
25	1,077.2
20	1,033.5
15	978.1
10	899.8
9	879.9
8	856.7
7	830.2
6	799.3
5	761.7
4	715.3
3	653.0
2	558.1

Figure F.2.4 Relationships of Maximum Flow Rate and Return Period in the Guacerique River Basin

3. SABACUANTE RIVER BASIN

3.1 RIVER CONDITION

Sabacuante river originates in the Azagualpa mountains, but with a different name, and branches into many tributaries in the upstream. The river takes its name after the confluence of the tributaries: Quiebradra Potrerillos and the Quebrada El Lechero in the midstream, then flows northwards and meets sevearl tributaries, Quebra Los Robles, Quebrada Guijamanil, Quebrada Santa Elena, Quebrada El Terrero, etc. The river meets its main tributary Quebrada El Aquila (sometimes called Quebrada Grande) in the downstream and flows to its end point in the Study Area at El Aguacate.

The overall drainage basin area is shown in Chapter 1, *Figure F.1.1* and summarized as follows:

Table F.3.1 Drainage Basins of Sabacuante River

River/Location	Basin Area (km^2)	
	Sub-basin	Total
Sabacuante Upstream	-	47
Quebrada El Aguila	33	80
Total		80

Source : SANAA

3.2 AVAILABLE DATA

3.2.1 RAINFALL

Rainfall data are available at the meteorological stations of SANAA in the basin as follows:

Table F.3.2 Rainfall Stations in the Sabacuante River Basin

Station	Recorded Data	
	years	Range
Villa Real	10	1991 - Present
El Aguacate	18	1973 - 1990

Source : SANAA

The average annual rainfall at Villa Real station and El Aguacate station is 841 mm and 857 mm respectively.

3.2.2 WATER LEVEL AND FLOW RATE

Data on water level and flow rate are available at the stream gauging stations of SANAA in the basin as follows:

Table F.3.3 Stream Gauging Station in the Sabacuante River Basin

Station	Recorded Data	
	years	Range
Daily Data		
El Aguacate	21	1970 - 1990
Non-Daily Data		
El Aguacate	8	1993 - Present

A summary of the rainfall and stream gauging stations is shown in Chapter 2, *Table F.2.4.*

The measurement was conducted at El Aguacate station continuously from 1973 to 1990 then halted in 1990. From 1993 until present, the non-daily measurement was conducted again.

The annual maximum, average and minimum flow rates are shown in *Appendix F, Table AF.3.1*. The averages of these data are as follows:

Table F.3.4 Average Flow Rate in the Sabacuante River Basin

Station	Flow Rate (m^3/s)		
	Maximum	Minimum	Average
El Aguacate	88.8	0.001	0.427

Source : SANAA

3.3 FREQUENCY ANALYSIS

Frequency analysis was conducted for the rainfall in the basin, but not for the minimum flow rate due to the data shortage.

The analysis was conducted for the annual maximum rainfalls at Villa Real station and El Aguacate station by using the standard Gumbel method. The analysis is different from the Guacerique river basin because data on the rainfall during the Hurricane Mitch in this basin were available. Theoretical approach of this method is shown in *Appendix AF.1*.

The design maximum rainfalls from the analysis are shown in *Figure F.3.1* and can be summarized as follows:

Table F.3.5 Design Maximum Daily Rainfall in the Sabacuante River Basin

Return Period (Year)	Design Maximum Rainfall (mm)
10	161
20	193
During Mitch (about 50 – 60 years)	236
200	295

The hourly rainfall pattern was constructed by using the pattern at Toncontin station in similar way as for the Guacerique river basin. These design rainfalls, together with the synthetic rainfall pattern, were used in the Rainfall-runoff analysis in the latter section.

3.4 RAINFALL - RUNOFF ANALYSIS

Rainfall-runoff analysis was conducted by using a standard Storage Function method. Theoretical approach of this analysis is explained in *Appendix AF.1*.

The hourly rainfall pattern at Toncontin station during the Hurricane Mitch was also used to construct the design rainfall in the Sabacuante river basin. The synthetic rainfalls were then input into the Rainfall-runoff model.

Necessary parameters in the model were set up as follows:

Table F.3.6 Parameters in the Rainfall-runoff Analysis

Parameter	Value
k	0.25
p	0.5
Area	80 km^2

Note : All parameters are referred in the *Appendix AF.1*

Due to the data shortage, the parameters used in the Guacerique river basin were adopted in this basin based on the assumption of similarity in the hydrological and hydraulic characteristics of these basins.

Calculation was done for different return periods. Relationship of the rainfall and simulated hydrograph in the Storage Function method was found similar to that in the Guacerique river basin, but different in magnitude. Relationship of runoff (peak of the simulated hydrograph) and its return period is shown in *Figure 3.2* and summarized as follows:

Table F.3.7 Runoff in the Sabacuante River Basin

Return Period (Year)	Runoff (m^3/s)
2	161
5	286
10	373
20	457
During Mitch (about 50 – 60 years)	579
200	752

These results were used to determine the scale and normal water level of the proposed dam.

3.5 WATER BALANCE ANALYSIS

Water balance was calculated to clarify the maximum available yield of the proposed Sabacuante dam using the same concept as for the Guacerique river basin.

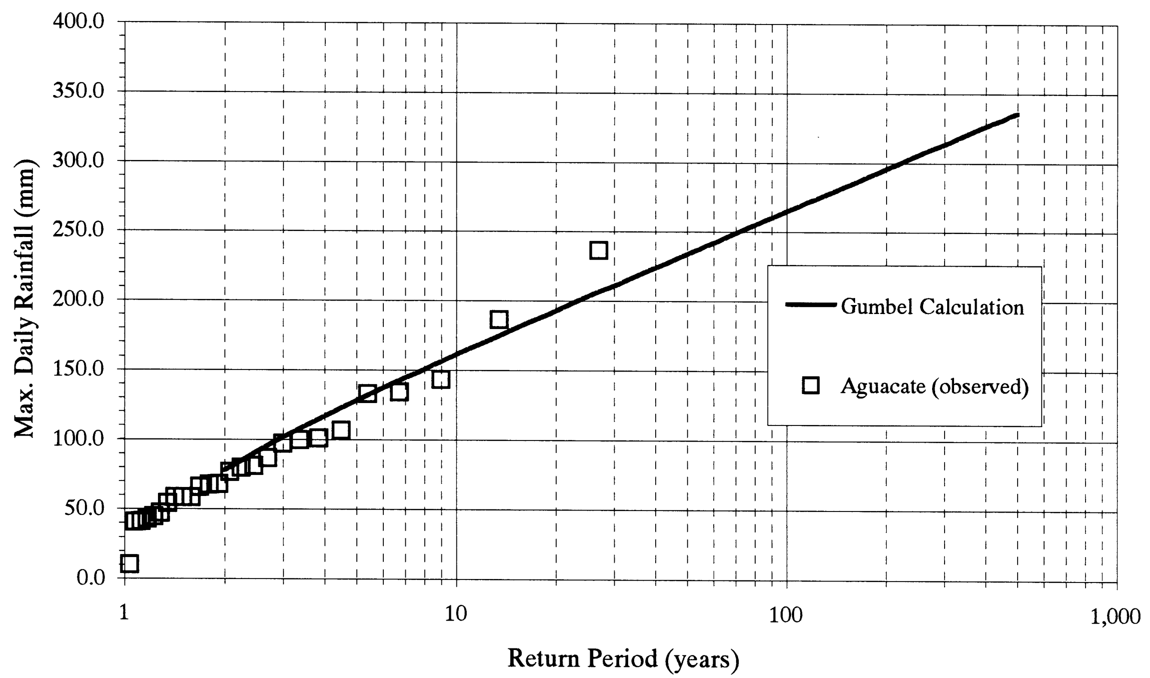
Inflow was the monthly flow rate measured at El Aguacate station between 1970 – 1990. Outflow was a combination of the excess flow over the dam and the maximum available yield for water supply that would not make the reservoir dried up.

Calculation was conducted by the same methodology as for the Guacerique river basin with the normal water level at 1,122.0 m. The H-V curve is shown in *Appendix F, Table AF.2.5.* Result of the calculation is shown in *Appendix F, Figure AF.3.1* and summarized as follows:

Table F.3.8 Maximum Yield of the Proposed Sabacuante Dam

Proposed Sabacuante Dam	Characteristics	Unit
Type	Concrete	-
Normal Water Level	1,122	m
Available Storage Volume (rounded up)	24,447,519 (25,000,000)	m^3
Sedimentation	-	m^3
Maximum Yield	276	l/s

Sabacuante River Basin
(Data from 1973 to 1998)



Return Period (yr)	Daily Rainfall (mm)
500	335.8
400	326.0
300	313.3
250	305.2
200	295.4
150	282.6
100	264.7
80	254.8
60	242.0
50	233.9
40	223.9
30	211.0
25	202.8
20	192.7
15	179.7
10	161.0
9	156.1
8	150.5
7	144.2
6	136.8
5	127.9
4	116.7
3	101.5
2	77.9

Figure F.3.1 Relationships of Maximum Daily Rainfall and Return Period in the Sabacuante River Basin

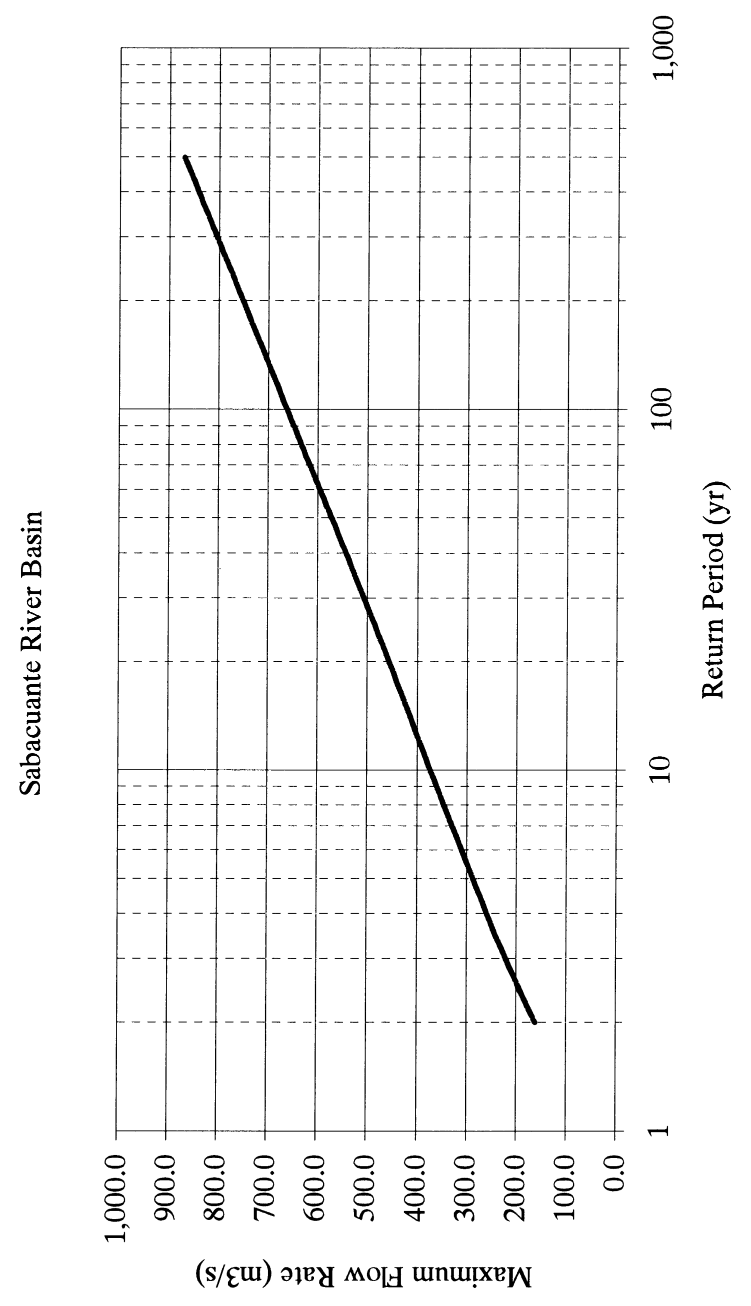


Figure F.3.2	Relationships of Maximum Flow Rate and Return Period in the Sabacuante River Basin

4. TATUMBLA RIVER BASIN

4.1 RIVER CONDITION

Tatumbla river originates from several tributaries in the La Loma mountain in the south-east and El Jicarito mountain in the south-west. The river takes its name after the confluence of the Quebrada El Chile and Chiquito river, then flows northwards and meets several tributaries, Quebrada Carrancres, Quebrada de Munuare, Quebrada La Calero. In the downstream, the river is sometimes called Las Canoas river. The river flows to its end point at the confluence with Sabacuante river in the downstream.

The overall drainage basin area is shown in Chapter 1, *Figure F.1.1* and summarized as follows:

Table F.4.1 Drainage Basin of Tatumbla River

River/Location	Basin Area (km^2)	
	Sub-basin	Total
Tatumbla	64	64

Source : SANAA

4.2 AVAILABLE DATA

4.2.1 RAINFALL

Rainfall data are available at the meteorological stations of SANAA in the basin as follows:

Table F.4.2 Rainfall Station in the Tatumbla River Basin

Station	Recorded Data	
	years	Range
El Incienso	21	1970 - 1990

Source : SANAA

The average annual rainfall at El Incienso station is 783 mm.

4.2.2 WATER LEVEL AND FLOW RATE

Data on water level and flow rate are available at the stream gauging stations of SANAA in the basin as follows:

Table F.4.3 Stream Gauging Station in the Tatumbla River Basin

Station	Recorded Data	
	years	Range
Daily Data		
El Incienso	16	1971 - 1986
Non-Daily Data		
El Incienso	8	1993 - Present

Source : SANAA

A summary of the rainfall and stream gauging stations is shown in Chapter 2, *Table F.2.4.* The annual maximum, average and minimum flow rates are shown in *Appendix F, Table AF.4.1*. The averages of these data are as follows:

Table F.4.4 Average Flow Rate in the Tatumbla River Basin

Station	Flow Rate (m^3/s)		
	Maximum	Minimum	Average
El Incienso	36.7	0.005	0.359

Source : SANAA

4.3 FREQUENCY ANALYSIS

Frequency analysis was conducted for the rainfall in the basin, but not for the minimum flow rate due to the data shortage.

Although rainfall data at El Incienso station are available, data during the Hurricane Mitch was missing. Therefore, the rainfall data at El Aguacate station and Villa Real station in the Sabacuante river basin were used in stead based on the assumption that both river basins have similar rainfalls. This assumption was verified by the rainfall data excluding the missing and incomplete data as follows:

Table F.4.5 Average Annual Rainfall at El Incienso Station and El Aguacate Station

Year	Rainfall at El Incienso (mm)	Rainfall at El Aguacate (mm)	Correlation (R^2)
1974	889.5	924.6	0.926
1975	1,033.1	1,062.9	
1976	707.1	836	
1977	812	882.1	
1980	1,273.2	1,353.1	
1981	982.9	1,010.8	
1982	764.4	643.1	
1985	571.5	485.7	
1986	729.8	667.7	
1987	792.3	786.4	
1988	1,178.6	1,187.1	
1989	873.3	858	

Correlation of these 2 stations is apparently high. Therefore, it can be summarized that rainfalls in these 2 stations are similar.

The design maximum rainfalls at El Aguacate station and Villa Real station in the Sabacuante river basin were used in the Tatumbla river basin as follows:

Table F.4.6 Design Maximum Daily Rainfall in Tatumbla River Basin

Return Period (Year)	Design Maximum Rainfall (mm)
10	161
20	193
During Mitch (about 50 – 60 years)	236
200	295

The hourly rainfall pattern was constructed by using the pattern at Toncontin station in similar way as for the Guacerique and Sabacuante river basins. These design rainfalls, together with the synthetic rainfall pattern, were used in the Rainfall-runoff analysis later.

4.4 RAINFALL - RUNOFF ANALYSIS

Rainfall-runoff analysis was conducted by using a standard Storage Function method. Theoretical approach of this analysis is explained in *Appendix AF.1*.

The hourly rainfall pattern at Toncontin station during the Hurricane Mitch was also used to construct the design rainfall for the Tatumbla river basin. The synthetic rainfalls were then input into the Rainfall-runoff model.

Necessary parameters in the model were set up as follows:

Table F.4.7 Parameters in the Rainfall-runoff Analysis

Parameter	Value
k	0.25
p	0.5
Area	64 km^2

Note : All parameters are referred in the *Appendix AF.1*

Parameters adopted for this basin were same as those in the Sabacuante river basin based on the assumption of the similarity in the hydrological and hydraulic characteristics of these basins.

Calculation was done for different return periods. A summary is shown as follows:

Table F.4.8 Runoff in the Tatumbla River Basin

Return Period (Year)	Runoff (m^3/s)
2	129
5	229
10	298
20	365
During Mitch (about 50 – 60 years)	463
200	601

These results were used to determine the scale and normal water level of the proposed dam.

4.5 WATER BALANCE ANALYSIS

Water balance was calculated to clarify the maximum available yield of the proposed Tatumbla dam with the same concept as for the Guacerique and Sabacuante river basin.

Inflow was the monthly flow rate measured at El Incienso station between 1971 – 1986. Outflow was a combination of the excess flow over the dam and the maximum available yield or maximum water quantity available for water supply that would not make the reservoir dried up.

However, flow rate data at El Incienso station was simulated until 1988 because it was evident that the driest period in the Study Area was in 1988. The simulated data were calculated from the ratio of the drainage area of the Sabacuante river basin and Tatumbla river basin based on the assumption of similarity in the rainfalls of these 2 basins.

Calculation was conducted by the same methodology as for the Guacerique river basin but only one case with the normal water level at 1,164.5 m. H-V curve is shown in *Appendix F, Table AF.2.5*. Result of the calculation is shown in *Appendix F, Figure AF.4.1* and summarized as follows:

Table F.4.9 Maximum Yield of the Proposed Tatumbla Dam

Proposed Quiebra Montes Dam	Characteristics	Unit
Type	Concrete	-
Normal Water Level	1,164.5	m
Available Storage Volume (rounded up)	17,051,886 (17,000,000)	m^3
Sedimentation	-	m^3
Maximum Yield	229	l/s

5. PROPOSED LOS LAURELES II DAM

The Guacerique river basin was considered as the highest water potential basin in the Study Area. The proposed Los Laureles II dam was selected as the priority dam for water resources development.

In this chapter, the hydraulic impacts of this proposed dam to the upstream were verified in terms of the change in water level and flooding areas.

The operation of the dam during dry and flood period was also investigated for the optimum use of the dam storage.

Hydraulic simulation was conducted by using a software so called MIKE11, a one-dimensional unsteady flow program, developed by the Danish Hydraulic Institute.

A river model of the Guacerique river basin was set up from the results of river survey in May 2000.

Flow during the Hurricane Mitch in 1998 calculated from the Rainfall-runoff analysis was used as the input data in the simulation in order to verify the change of water level and flooding areas due to the proposed dam. Details of the simulation are explained in the following text.

5.1 HYDRAULIC SIMULTION

5.1.1 RIVER NETWORK

The river network model was set up from the river coordinates and the cross sections along the river from the river survey as shown in *Appendix B.* Digitized cross section data from the river survey were input to the program.

In the river survey, the controlled sections were set up with the following criteria:

- The distance interval along the river between each section was less than 500 m,
- The measured river width was at least 200 m from the center of a cross section over the left and right bank and
- The most upstream and downstream was located at the former Batallon station and the Mateo bridge respectively.

The controlled sections are shown in *Table F.5.1* and the following table:

Table F.5.2 Coordinates of the Cross Sections in the River Model

Survery Section	Chainage in River Model (m)	River Width (m)	Center Coordinate in UTM	
			x	y
0	3,783	65	472,173	1,555,073
2	3,436	219	471,827	1,555,077
3	3,345	369	471,737	1,555,056
4	3,160	444	471,638	1,555,081
5	3,119	890	471,616	1,554,865
6	2,982	489	471,481	1,554,846
7	2,620	357	471,136	1,554,737
8	2,433	615	470,956	1,554,788
9	2,179	643	470,718	1,554,699
10	1,918	321	470,457	1,554,704
11	1,681	209	470,238	1,554,793
13	1,534	319	470,091	1,554,780
14	1,429	321	470,023	1,554,861
15	1,220	484	469,939	1,555,051
16	986	667	469,730	1,555,158
17	703	350	469,963	1,555,317
18	493	150	469,759	1,555,264
19	276	272	469,630	1,555,439
19A	146	531	469,594	1,555,314
20	0	241	469,472	1,555,395

Note : Section 19 is the bridge section, not river section
The river width was calculated from the initial point and end point of the river survey

5.1.2 CALCULATION PROCEDURE

(1) Procedure

Procedure of the calculation is as follows:

- Set up the river model using the cross sections from the survey, flow direction, nodes and branches,
- Set up the boundary condition in the upstream using the hydrograph during the Hurricane Mitch, and in the downstream using water level during the Hurricane Mitch,
- Set up the necessary hydrodynamic parameters,
- Calculate the water level and flow rate at each section along the river,
- Adjust the parameters in the model to make the least error between the simulated water level and observed water level in some sections during the Hurricane Mitch,
- Set up a dam at the downstream end,
- Set up the condition of outflow at the dam,
- Calculate the water level and flow rate at each section along the river,
- Compare the results in the case without and with a dam and
- Summarize the impact of the dam to the river.

(Theoretical consideration of the model is shown in *Appendix AF.1*).

(2) Calculation Cases

Calculation was conducted for 6 different cases as follows:

Table F.5.3 Calculation Cases in the Hydraulic Simulation

Case	Propose Los Laureles II Dam			Sediment Control Structure	
	Dam	Sediment	River Bed Elevation (m)	Structure	Top Elevation (m)
I	No	No	Original	No	-
II	Yes	No	1,044.5 (at Dam Site only)	No	-
III	Yes	Yes	1,048.0 (from Dam Site to Upstream)	No	-
IV-1	Yes	No	1,044.5 (at Dam Site only)	Yes	1,048.0
IV-2	Yes	No	1,044.5 (at Dam Site only)	Yes	1,049.0
IV-3	Yes	No	1,044.5 (at Dam Site only)	Yes	1,050.0

Note : The elevation 1,044.5 m is the crest level of the spill way when the gate is fully opened
The elevation 1,048.0 m is the design sediment level

In Case I, the present river condition without the proposed dam was used. The simulation was conducted using the hydrograph during the Hurricane Mitch as the input data in order to clarify the water level at each section along the river.

In Case II, the proposed dam was set up at the downstream end. The simulation was also conducted using the hydrograph during the Hurricane Mitch as the input data, but the outflow at the downstream end was changed to a free flow over a rectangular gate. This was the flow condition when the gates of the proposed dam were fully opened during floods and the crest level of the spill way was 1,044.5 m.

In Case III, the condition was basically same as Case II, but sedimentation was taken into consideration. Sediment was supposed to deposit on the entire river bed at the elevation of 1,048.0 m. This level was considered as the design sediment level.

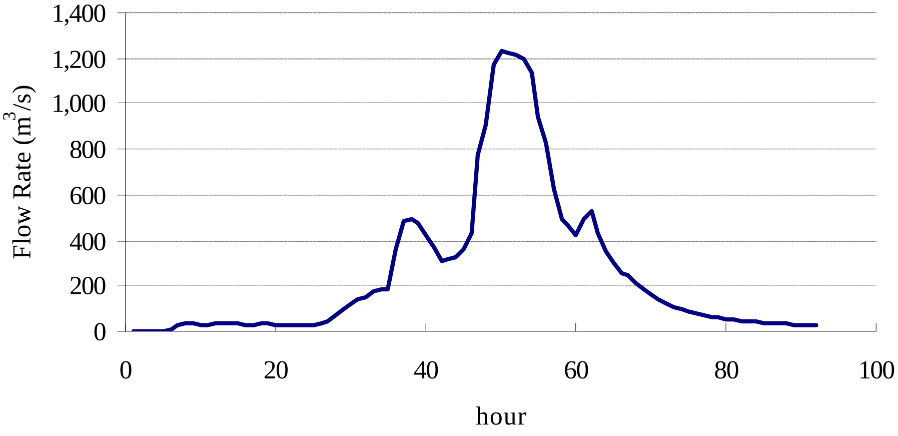
In Case IV, a sediment control structure was proposed at the section No. 15 (or at the distance of about 2.84 km from the downstream end of the survey at section No. 0) to protect the sediment deposition in the downstream or the storage pond of the dam. The structure was supposed to be simple with a constant elevation over the cross section. The verification of the impact of this structure was done for 3 cases of different height, 1,048.0 m, 1,049.0 m and 1,050.0 m.

5.1.3 PARAMETERS AND BOUNDARY CONDITION

The parameters and boundary condition in the model are:

- Manning roughness, n = 0.030 in accordance with the river bed material survey,
- At the upstream end, hydrograph during the Hurricane Mitch was used as the boundary condition as shown in the following figure,

Figure F.5.1 Hydrograph during the Hurricane Mitch



- At the downstream end, boundary condition was set up for 2 cases; without dam and with a proposed dam,
- For the case without dam, maximum water level at 1,036.76 m was set up (during the Hurricane Mitch),
- For the case with a proposed dam, water level was set up using the free overflow condition for a rectangular gate when the gates were fully opened. The relationship between flow rate and water level is as follows:

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

where Q = flow rate, m^3/s, B = gate width, m,

H = water level, m, C = constant, and

- Time step in the calculation = 5 seconds.

5.2 HYDRAULIC SIMULATION RESULTS

The simulation was done for 6 cases as explained. Variation of the water level is shown in *Figure F.5.2* and summarized as follows:

Table F.5.4 Water Level from Hydraulic Simulation

Survery Section	Chainage (m)	Water Level (m)					
		Case I	Case II	Case III	Case IV-1	Case IV-2	Case IV-3
20	0	1,056.608	1,056.653	1,056.704	1,056.657	1,056.672	1,056.715
19A	146	1,056.146	1,056.209	1,056.279	1,056.215	1,056.236	1,056.293
18	493	1,053.324	1,053.715	1,054.064	1,053.747	1,053.856	1,054.073
17	703	1,052.558	1,053.530	1,054.035	1,053.585	1,053.719	1,053.911
16	986	1,051.896	1,053.079	1,053.748	1,053.145	1,053.248	1,053.422
15	1,220	1,050.191	1,053.109	1,053.787	1,053.168	1,053.123	1,053.112
14	1,429	1,048.498	1,052.993	1,053.531	1,053.409	1,053.109	1,053.009
13	1,534	1,048.088	1,052.943	1,053.527	1,053.009	1,052.965	1,052.951
11	1,681	1,046.607	1,052.880	1,053.123	1,053.569	1,053.067	1,052.900
10	1,918	1,045.094	1,052.995	1,053.301	1,053.213	1,053.061	1,053.007
9	2,179	1,042.561	1,053.025	1,053.325	1,053.125	1,053.057	1,053.033
8	2,433	1,042.192	1,053.025	1,053.289	1,053.064	1,053.039	1,053.032
7	2,620	1,042.107	1,052.961	1,052.998	1,053.035	1,052.986	1,052.969
6	2,982	1,039.364	1,053.005	1,053.137	1,053.015	1,053.011	1,053.011
5	3,119	1,039.304	1,052.952	1,052.955	1,052.965	1,052.959	1,052.958
4	3,160	1,038.905	-	-	-	-	-
3	3,345	1,037.225	1,037.216	1,037.217	1,037.217	1,037.217	1,037.218
2	3,436	1,036.862	1,036.854	1,036.855	1,036.859	1,036.857	1,036.856
0	3,783	1,036.760	1,036.760	1,036.760	1,036.760	1,036.760	1,036.760

Note : Chainage 146 m (as shaded) is the section at the Mateo Bridge
Chainage 3,160 m is the section at the proposed Los Laureles II dam

The impacts of the proposed dam are classified as the impact to the Mateo bridge in the upstream and the water level increase in the surrounding areas.

5.2.1 IMPACT TO THE MATEO BRIDGE

The simulation was conducted for the case during the Hurricane Mitch and all the gates were fully opened. Water level at the Mateo Bridge for all cases in comparison to the bridge height can be summarized as follows:

Table F.5.5 Water Level at the Mateo Bridge

Case	Case	Elevation (m)
I	No dam	1,056.146
II	With LL II, No Sediment, No Trap Structure	1,056.209
III	With LL II, With Sediment, No Trap Structure	1,056.279
IV-1	With LL II, No Sediment, With Trap Structure at 1,048 m	1,056.215
IV-2	With LL II, No Sediment, With Trap Structure at 1,049 m	1,056.236
IV-3	With LL II, No Sediment, With Trap Structure at 1,050 m	1,056.293
Mateo Bridge Height		1,056.110

Note : "LL II" refers to the proposed Los Laureles II Dam

During the Hurricane Mitch, water level at the Mateo Bridge from the calculation was considered same as the bridge height with some negligibly small discrepancy.

It is apparent that the water level after the construction of the proposed dam would be raised up within 0.15 m in comparison with the existing condition with no dam.

The water level increase due to the construction of the dam is possibly attenuated within 1,000 m in the upstream from the section number 20 by considering the water surface gradient.

5.2.2 IMPACT TO THE SURROUNDING AREAS

In order to clarify the impact of the proposed dam to the surrounding areas along the river, water level in river was compared with the height of the left bank and right bank of the river as follows:

Table F.5.6 River Bed and Bank Height

Survey Section	Chainage (m)	Elevation of the River (m)		
		Left Bank	Bed	Right Bank
20	0	1,061.54	1,051.16	1,057.00
19A	146	1,067.65	1,050.06	1,061.19
18	493	1,058.48	1,049.23	1,061.43
17	703	1,052.78	1,047.95	1,065.60
16	986	1,069.62	1,047.33	1,059.64
15	1,220	1,070.07	1,046.29	1,068.45
14	1,429	1,064.01	1,042.81	1,075.26
13	1,534	1,057.37	1,042.82	1,068.59
11	1,681	1,057.27	1,041.14	1,078.75
10	1,918	1,071.30	1,040.65	1,090.77
9	2,179	1,102.81	1,038.02	1,092.63
8	2,433	1,117.10	1,035.86	1,082.74
7	2,620	1,105.12	1,036.19	1,092.17
6	2,982	1,059.22	1,032.66	1,067.50
5	3,119	1,081.40	1,031.91	1,089.80
4	3,160	1,084.90	1,031.05	1,097.00
3	3,345	1,078.07	1,030.92	1,092.97
2	3,436	1,061.47	1,030.46	1,063.92
0	3,783	1,038.39	1,028.23	1,034.99

Water level at each section excluding the chainage 703 (section 17) is still lower than the bank height, or the water remains confined in the river course.

Chainage 703 was verified in the topographic map from the survey. It is found that on the left bank, there is a small dried up tributary with high left and right banks, and the survey was conducted along that tributary. Therefore, it is expected that there will be no inundation at that section.

As a result, it can be summarized that there will be no flow over banks along the river during floods after the construction of the proposed dam.

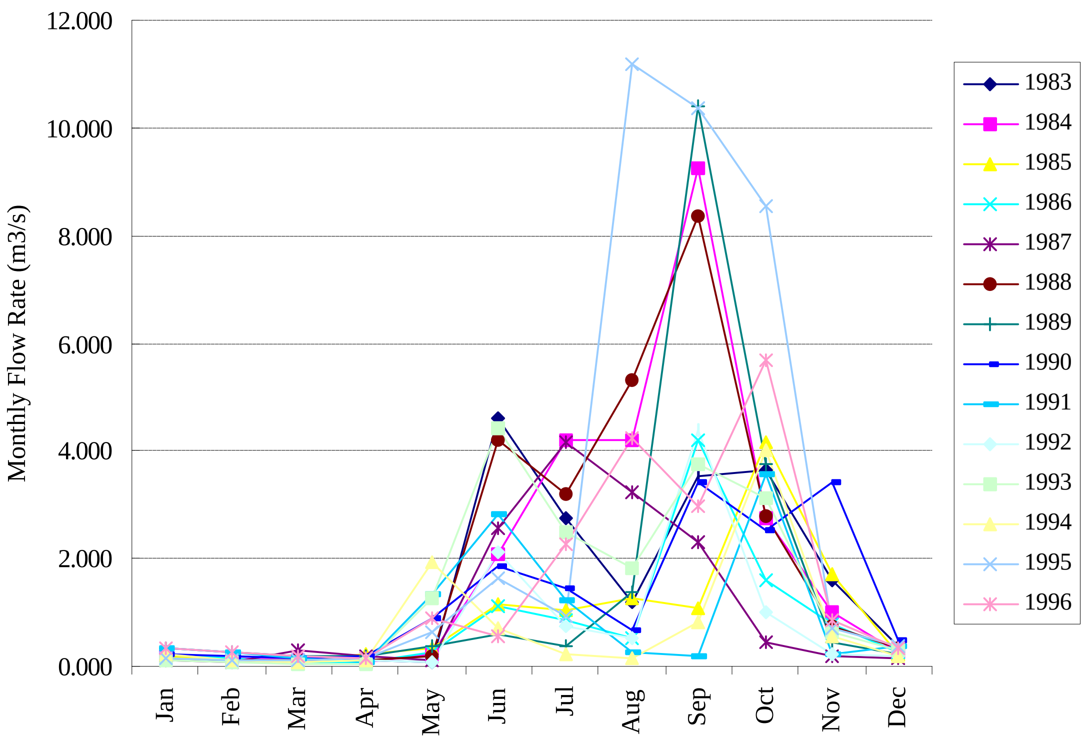
The inundated area during the Hurricane Mitch from the simulation after the completion of the dam is shown in *Figure F.5.3*.

5.3 DAM OPERATION FOR WATER SUPPLY

Dam operation for water supply was proposed herein in order to make the optimum use of the dam storage.

Period of the gate operation was divided into dry and flood season. From the variation of flow rate as shown in the following figure, the season can be divided into 3 periods; those are dry season (December to April), transition period (May and November) and flood season (June to October).

Figure F.5.4 Annual Variation of Flow Rate in Guacerique River



However, by taking into account of the rainfall, May is classified in the flood reason, while November is classified in the dry season. Therefore, the season is divided as follows:

- Dry Season : November – April
- Flood Season : May – October

From the water balance analysis, the operation for water supply is recommended as follows:

(1) In dry season, the gate should be fully closed in order to maintain the maximum yield of the existing and proposed dam for water supply.

(2) In flood season, the gate should be basically closed, the excess water can flow over the existing and proposed dam to the downstream.

However, during a severe flood, the gate should be opened before the flood arrives the dam in order to release the excess water to downstream gradually.

Variation of flow rate due to the operation of the dam to maintain the maximum yield for water supply is shown in *Appendix F, Figure AF.2.2*.

5.4 DAM OPERATION FOR FLOOD CONTROL

The main objectives of the operation during a severe flood are to discharge the excess flow and maintain the full capacity of the dam for dry season. Therefore, the operation needs the information on flood in advance from a flood forecasting system. From this information, the gate should be operated to achieve its objectives. The concerned topics are proposed herein as follows:

5.4.1 CONCEPT OF OPERATION

Based on the water balance analysis, the dam storage would always be at its full capacity during flood period. The gate should be operated to release all excess inflow and maintain the full capacity of the storage. The concept can be expressed as follows:

Inflow (flood) = Storage (full capacity) + Outflow (by gate operation)

The storage should be kept constant at its full capacity, therefore, outflow should be set same as the inflow.

Inflow due to the flood should be calculated by using the rainfall data and the Rainfall-runoff analysis (real time analysis), or measured by Los Laureles station.

Relationship between the gate height and the outflow should be established for the operation.

5.4.2 FLOOD FORECASTING

The newly constructed telemetry system at Mateo bridge, Los Laureles station, should be used for flood forecasting.

The flood forecasting system is normally established by using 3 types of data;

- Meteorological data,
- Water level data and
- Rainfall data.

Meteorological data is not available in the basin, therefore, it cannot be used for this.

Water level can possibly be used for flood forecasting although a flood at Mateo bridge would arrive the proposed dam shortly. The flood needs to be monitored closely for the gate operation.

Rainfall can possibly be used for the flood forecasting as well, but the establishment of flood forecasting system including the Rainfall-runoff real time analysis is necessary.

Establishment of the flood forecasting by using the rainfall and water level data, and the merits and demerits are described in the following text.

(1) Flood Forecasting and Gate Operation by using Water Level Data

1) Process

Process of flood forecasting by using the water level data and gate operation is proposed as follows:

Water Level, Wl	⇐	- Measure the water level in the river continuously during a severe flood.
↓		
Inflow, Qin (= Outflow, Qout)	⇐	- Calculate Flow Rate (Q) from water level by using a H-Q Curve, - Assume: Flow rate (Q) = Inflow (Qin) - In order to maintain the water level of the reservoir, this inflow should be same as outflow through gate: Inflow (Qin) = Outflow (Qout)
↓		
Gate Height, H	⇐	- Calculate the height of gate to be opened to discharge this inflow by using the relationship of gate height and outflow.

2) Flood Arriving Time

When the flood wave arrives at the end of the reservoir, the wave is transmitted downstream with a velocity equivalent to the square root of *gh (g*; gravity acceleration and *h*; average depth of water). Taking the safer side, it is assumed the flood wave comes to the dam site with no time from the end of the reservoir.

(2) Flood Forecasting and Gate Operation by using Rainfall Data

1) Process

Process of flood forecasting by using the rainfall data and gate operation is proposed as follows:

Hourly Rainfall, Rf	⇐	- Measure the rainfall in the basin by using the rainfall station in the basin, and - Input the rainfall to the Rainfall-runoff Analysis.
↓		
Runoff or Flow Rate, Q	⇐	- Calculate runoff or flow rate in the river by using the Rainfall-runoff Analysis (real time).
↓		
Inflow, Qin	⇐	- Assume: Flow rate (Q) = Inflow (Qin) - In order to maintain the water level of the reservoir, this inflow should be same as outflow through the gate: Inflow (Qin) = Outflow (Qout)
↓		
Gate Height, H	⇐	- Calculate the height of gate to be opened to discharge this inflow by using the relationship of gate height and outflow.

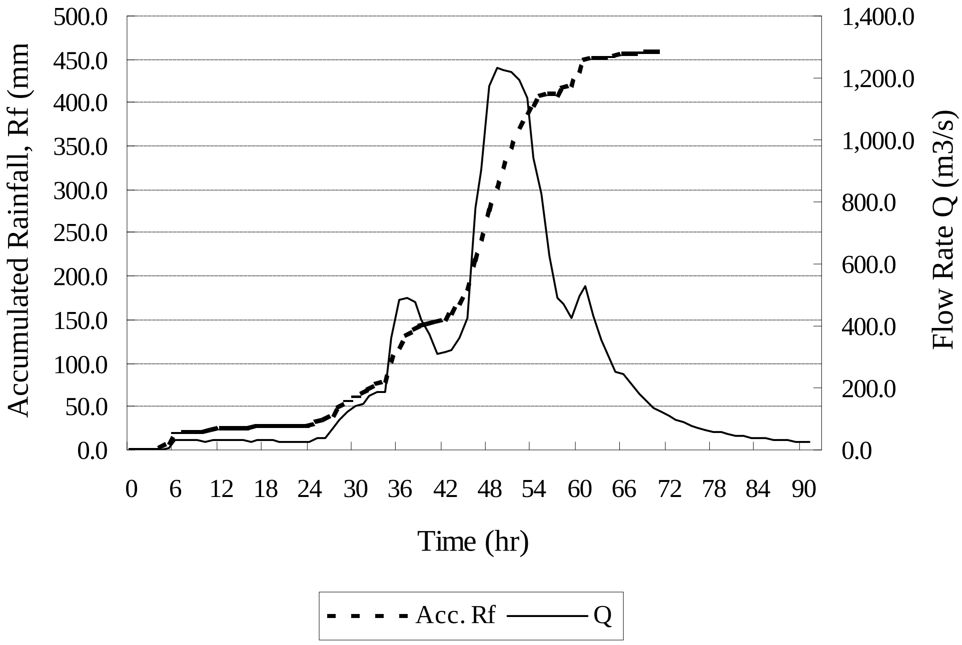
Relationship of the rainfall and flow rate is shown in Chapter 2.

2) Flow Rate and Accumulated Rainfall during the Hurricane Mitch

The relationship of the rainfall and flow rate needs to be investigated for the flood forecasting by using the rainfall data. The accumulated quantity of hourly rainfall should be used to indicate the flood scale and determine the level of flood forecasting.

Flow rate and accumulated rainfall during the Hurricane Mitch is used to determine the flood scale as shown in the following figure.

Figure F.5.5 Relationship of Flow Rate and Accumuated Rainfall



The above hydrograph shows that the accumulated rainfall with a range between 50 – 100 mm can cause a small flood, while that above 200 mm can cause a severe flood.

This can be used as a reference for flood forecasting.

3) Concentration Time of Flood

Concentration time is defined as the time difference between the peak rainfall and peak flow rate in the river. This value can be calculated by using some hypothesis. However, it should be examined from the actual measurement on hourly basis and real-time Rainfall-runoff analysis.

Based on the equation of the Ministry of Construction, Japan, relationship of concentration time and river length and slope can be express as follows:

$$t_c = 1.67 \times 10^{-3} \times \left(\frac{L}{\sqrt{S}} \right)^{0.7}$$

where t_c = concentration time, hr, L = distance (m), S = Slope

Distance, L, is defined as the longest distance from the barrier of the river basin to the checking point at Los Laureles dam. Slope, S, is the slope of the river or the basin.

Calculation of concentration time in the basin is shown in the following table.

Table F.5.7 Concentration Time in the Guacerique River Basin

Parameter	Unit	Value
L	m	2,700
S	-	1/50
tc	hour	8

It should be noted that this calculation was the rough estimation from a 1/50,000 scale

(3) Comparison

1) Flood forecasting by using water level data

Merits:

- The forecasting is accurate because the actual flood has arrived and
- No new investment is necessary if the existing Los Laureles Station at the Mateo Bridge is utilized.

Demerits:

- Time for forecasting is very short, if the existing Los Laureles Station is used,
- It is man-power consuming because the arriving time is short and the intensive attention is needed to monitor the flood and
- For the case of some condition change immediately, the operation cannot be changed in time.

2) Flood forecasting by using rainfall data

Merits:

- Time for forecasting is long with an order of several hours and
- There will be more time for the preparation of urgent countermeasure and adjustment of the gate.

Demerits:

- There is an uncertainty in the forecasting,
- A new flood forecasting system with real time analysis is necessary and
- It is cost consuming to establish a new system.

From this comparison, it is recommended that

- At present, Los Laureles station is the only telemetry station in the basin. The station should be used for the flood forecasting. Data processing and transmission system should be set up to link with the other stations and the dam,
- New stream gauging stations should be established in the upstream of the Guacerique river, Quiebra Montes and Mateo river. The data should be taken continuously from now on, and
- By considering the drainage area, new rainfall stations should be also established in the upstream of the basin. The new stations should be linked to the existing station for the data processing in the entire basin.

5.4.3 GATE OPERATION

The gate should be opened fully and consecutively one by one from the gate in the middle part (Gate No. 2 and 3) to the edge (Gate No. 1 and 4) during the flood.

Flow through the gate was calculated by using an energy equation as follows:

$$Q = C_c a B \sqrt{\frac{2g(h_0 - C_c a)}{1-(C_c a / h_0)^2}}$$

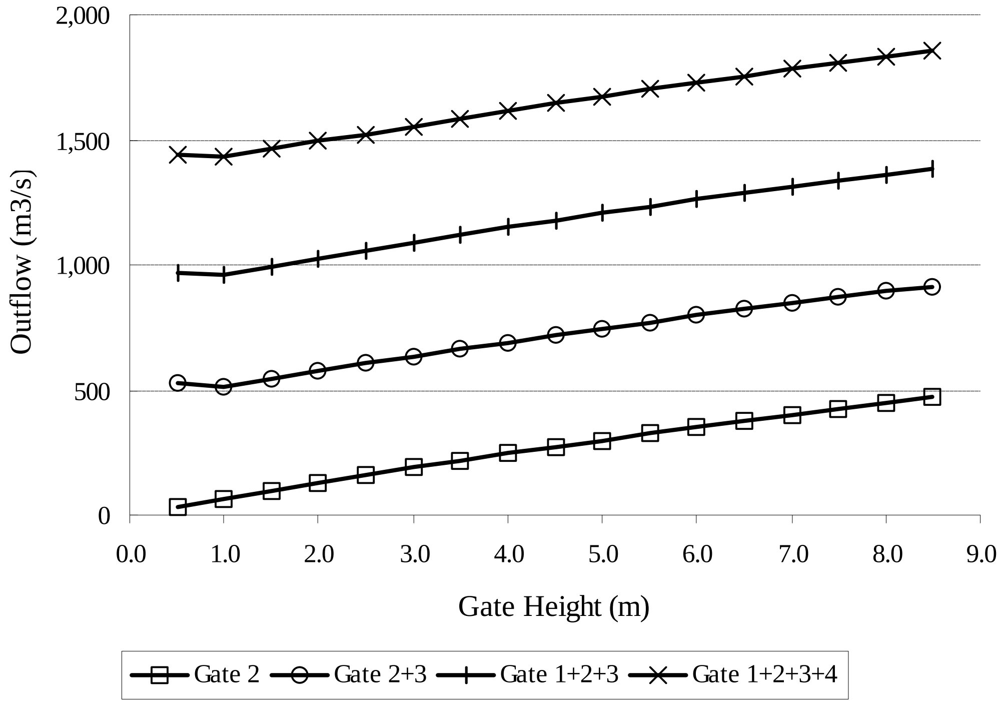
where Q = flow rate, m^3/s, B = gate width = 34.4 m,

h_0= water level from water surface to the middle of gate, m,

C_c = constant = 0.61 a = the height of gate opening, m

The relationship of the gate height and outflow is shown in the following figure.

Figure F.5.6 Relationship of Gate Height and Outflow



5.4.4 IMPACT OF GATE OPENING

(1) Impact to Water Level in the Upstream

The impact of the gate opening during a severe flood to the upstream was investigated.

The hydraulic simulation was conducted by using the same simulation model in the previous section. In order to verify the impact when the gates were moving up during the flood, boundary conditions in the upstream and downstream were set up as follows:

- The flood during the Hurricane Mitch arrived at the Mateo bridge,
- The hydrograph was measured at the Los Laureles station, nearby the bridge,
- All gates (4 gates with 8.6 m wide each) were opened simultaneously, and
- The gates were moving up at a speed of 0.3 m/min.

For comparison, the calculation cases in the previous section, Case II and Case IV-3 were selected. Each case was conducted for 2 different boundary conditions as follows:

- Gates were fully opened (F) and
- Gates were moving up (M).

The calculation cases are summarized in the following table.

Table F.5.8 Calculation Cases for the Impact of Gate Opening

Case	Gate Condition	Propose Los Laureles II Dam			Sediment Trap Structure	
		Dam	Sediment	River Bed Elevation (m)	Structure	Elevation (m)
II (F)	Fully Opened	Yes	No	1,044.5	No	-
II (M)	Moving Up					
IV-3 (F)	Fully Opened	Yes	No	1,044.5	Yes	1,050.0
IV-3 (M)	Moving Up					

Note : These 2 cases are basically same as the simulation in the previous section, but boundary condition at the proposed dam site is different.

Simulation result in comparison with the case when all gates are fully opened, is shown in the following table:

Table F.5.9 Water Level with different Gate Condition

Survery Section	Chainage (m)	Water Level (m)			
		Gates Fully Opened		Gates Moving Up	
		II (F)	IV-3 (F)	II (M)	IV-3 (M)
20	0	1,056.653	1,056.715	1,056.653	1,056.715
19A	146	1,056.209	1,056.293	1,056.209	1,056.293
18	493	1,053.715	1,054.073	1,053.716	1,054.074
17	703	1,053.530	1,053.911	1,053.532	1,053.912
16	986	1,053.079	1,053.422	1,053.081	1,053.423
15	1,220	1,053.109	1,053.112	1,053.111	1,053.115
14	1,429	1,052.993	1,053.009	1,052.995	1,053.011
13	1,534	1,052.943	1,052.951	1,052.946	1,052.954
11	1,681	1,052.880	1,052.900	1,052.882	1,052.904
10	1,918	1,052.995	1,053.007	1,052.998	1,053.010
9	2,179	1,053.025	1,053.033	1,053.028	1,053.036
8	2,433	1,053.025	1,053.032	1,053.027	1,053.034
7	2,620	1,052.961	1,052.969	1,052.964	1,052.972
6	2,982	1,053.005	1,053.011	1,053.007	1,053.013
5	3,119	1,052.952	1,052.958	1,052.955	1,052.961
4	3,160	-	-	-	-
3	3,345	1,037.216	1,037.218	1,037.216	1,037.218
2	3,436	1,036.854	1,036.856	1,036.854	1,036.856
0	3,783	1,036.760	1,036.760	1,036.760	1,036.760

Note : Chainage 146 m (as shaded) is the section at the Mateo Bridge
Chainage 3,160 m is the section at the proposed Los Laureles II dam

The result indicates that water level difference for the case when the gates were fully opened and moving up was minuscule.

This means that when the flood arrives the Mateo bridge, if the gates are opened simultaneously, there will be no major back water impact to the upstream in comparison with the case when the gates are fully opened before the flood.

(2) Impact to the Reservoir Water Level

The variation of reservoir water level was investigated in the case when a flood arrived and the gate was being opened. In this case it was assumed that the flood wave comes from the end of the reservoir to the dam site with no time. It was also assumed that the delay of gate operation is 30 minutes after getting the information on inflow discharge. The operation time unit of the gate is 10 minutes.

The calculation result is shown in *Figure F.5.7*. The reservoir water level is fluctuating and the maximum reservoir water level is 1053.1 m.

Figure F.5.7 Inflow/outflow hydrograph and reservoir water level change

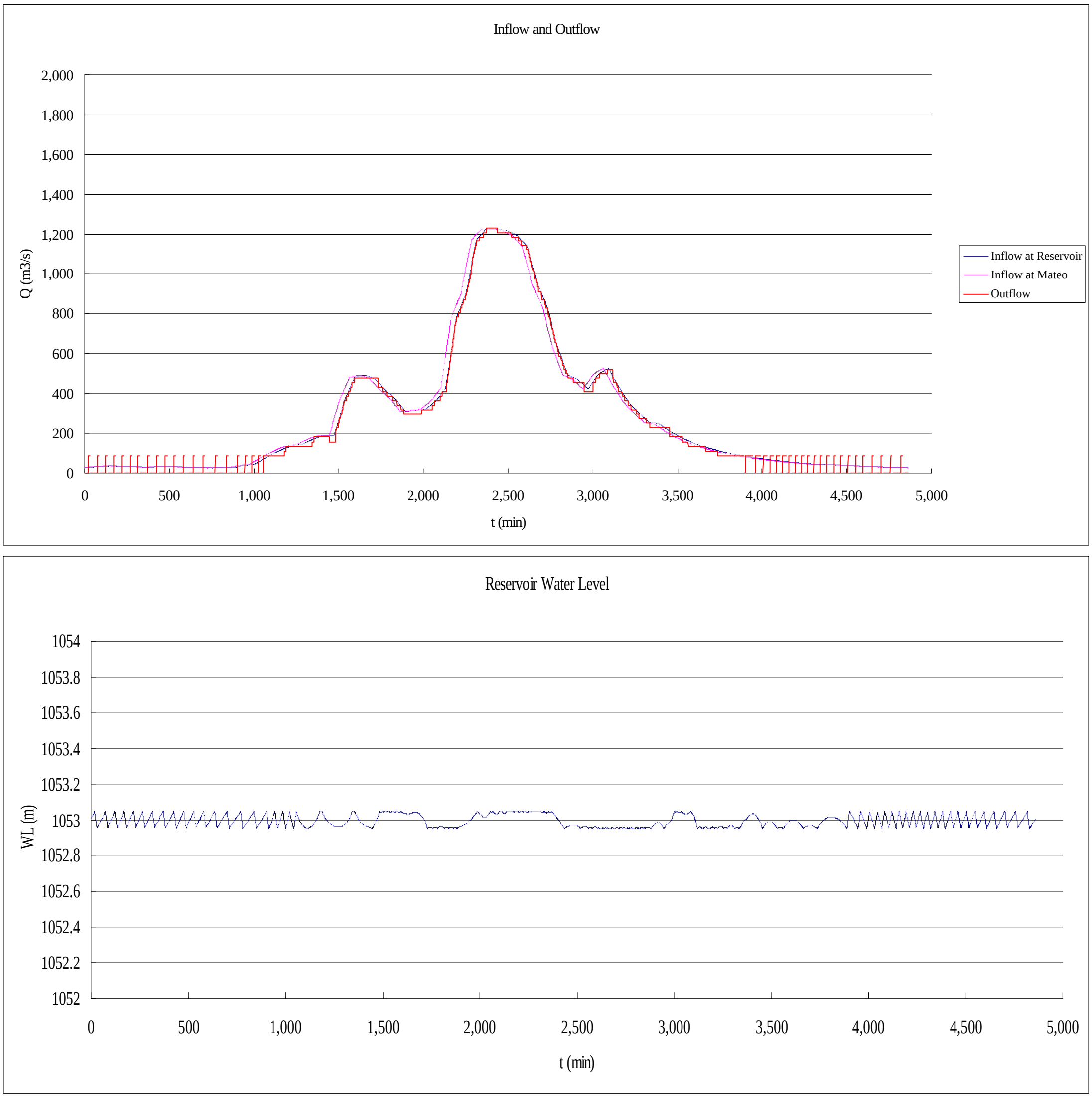


Table F.5.1 Coordinates of the Controlled Sections in Guacerique River

Section	Initial Poing			Last Point			Chainage
	X	Y	Z	X	Y	Z	km
0	472,174	1,555,105	-	472,172	1,555,041	-	3.783
2	471,858	1,555,182	-	471,796	1,554,973	-	3.436
3	471,695	1,554,877	-	471,781	1,555,235	-	3.345
4	471,596	1,554,864	2,148.65	471,682	1,555,299	2,160.45	3.160
5	471,571	1,554,423	1,038.39	471,663	1,555,308	1,063.73	3.119
6	471,397	1,555,076	1,067.50	471,566	1,554,617	1,059.22	2.982
7	471,050	1,554,894	1,092.17	471,223	1,554,582	1,105.12	2.620
8	470,826	1,555,067	1,083.47	471,086	1,554,510	1,117.10	2.433
9	470,668	1,555,018	1,092.83	470,768	1,554,382	1,102.81	2.179
10	470,539	1,554,843	1,090.83	470,375	1,554,567	1,071.23	1.918
11	470,186	1,554,703	1,057.27	470,291	1,554,884	1,057.20	1.681
13	470,192	1,554,904	1,077.13	469,990	1,554,657	1,057.37	1.534
14	470,165	1,554,938	1,075.26	469,883	1,554,785	1,064.01	1.429
15	469,757	1,554,893	1,070.07	470,122	1,555,210	1,068.45	1.220
16	469,474	1,554,945	1,069.62	469,987	1,555,371	1,059.64	0.986
17	469,836	1,555,198	1,052.78	470,090	1,555,438	1,066.33	0.703
18	469,750	1,555,190	1,058.48	469,769	1,555,339	1,060.46	0.493
19	469,599	1,555,307	1,062.27	469,662	1,555,571	1,559.85	Bridge
19A	469,532	1,555,057	1,064.58	469,657	1,555,573	1,059.85	0.146
20	469,440	1,555,280	1,057.53	469,505	1,555,511	1,057.09	0.000

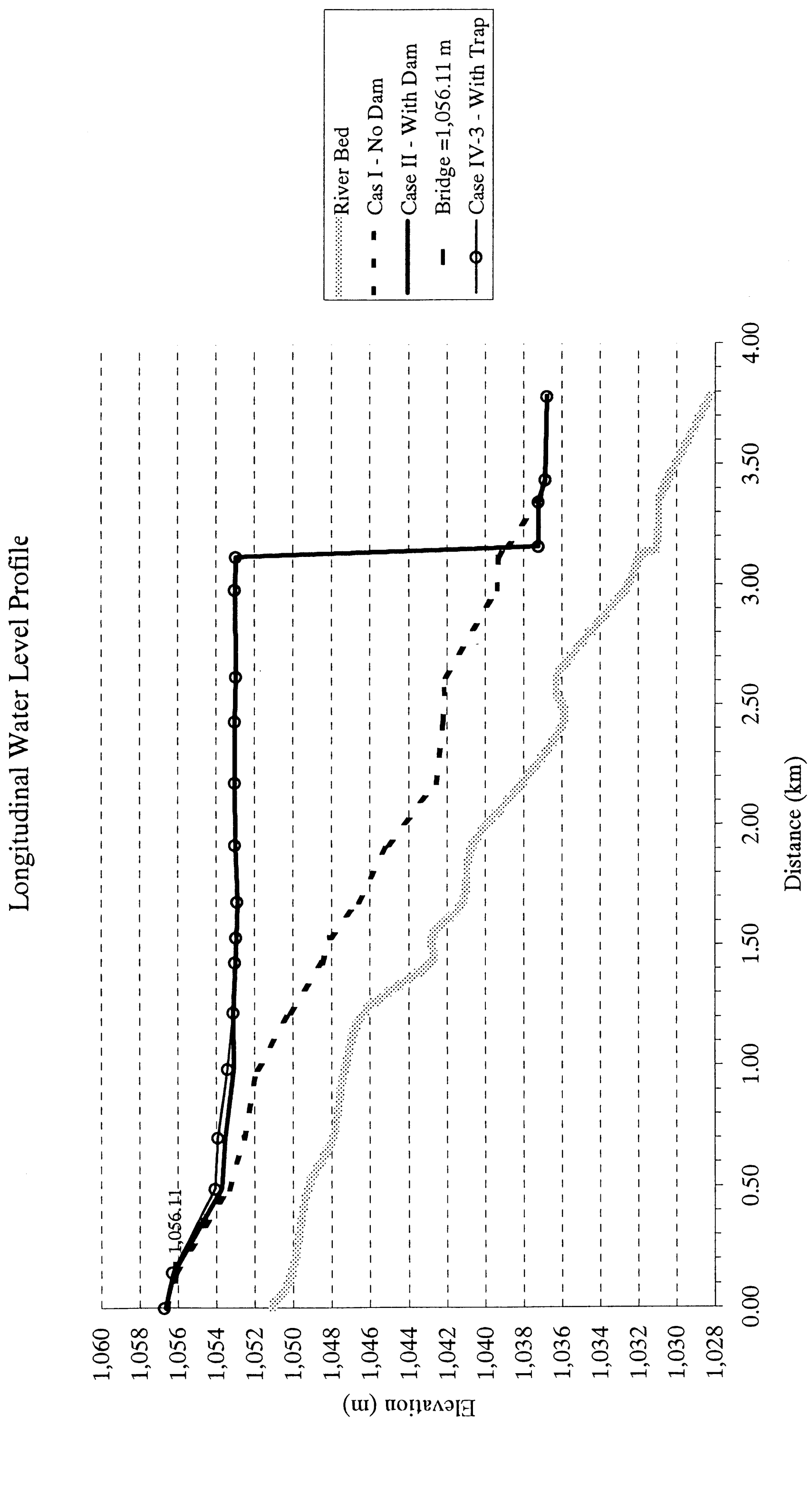


Figure F.5.2	Longitudinal Water Level Profile from Hydraulsic Simulation

Figure F.5.3	Inundated Area from Hydraulic Simulation

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SUPPORTING REPORT F

APPENDIX F

APPENDIX F.1

THEORETICAL APPROACH

APPENDIX F.1 THEORETICAL APPROACH

AF.1.1 FREQUENCY ANALYSIS

(1) Theoretical Approach

The standard Gumbel method is used to analyze the relationship of the rainfall or flow rate and its return period. The basic equations are as follows:

$$T = \frac{1}{P(x)} = \frac{1}{1-F(x)} \tag{F.1.1}$$

where T = return period, year

$P(x)$ = Probability of Exceedance

$F(x)$ = Probability of Non-exceedance

x = Maximum rainfall or flow rate each year, mm or m^3/s

From a series of data x, $F(x)$ can be calculated by using the Hazen Method or Weibull Method as follows:

$$F(x) = 1-\frac{j}{N+1} \tag{F.1.2}$$

Where j = Order of x_j from maximum

N = Total number of the data series

From the above $F(x)$, a new parameter x and y are defied as follows:

$$F(x) = 1-exp\left(-e^{-y}\right) \tag{F.1.3}$$

$$y = -ln\{-ln\,F(x)\} = a(x-x_0) \tag{F.1.4}$$

where a and x_0 can be calculated from the following equation

$$\frac{1}{a} = \frac{S_x}{S_y} \tag{F.1.5}$$

$$x_0 = \bar{x}-\left(\frac{1}{a}\right)\bar{y} \tag{F.1.6}$$

$$S_x = \sqrt{\frac{1}{N}\sum_{i=1}^{N}\left(x_i-\bar{x}\right)^2}\ ,\ S_y = \sqrt{\frac{1}{N}\sum_{i=1}^{N}\left(y_i-\bar{y}\right)^2} \tag{F.1.7}$$

$$F(x) = 1-exp\left(-e^{-y}\right) = 1-\frac{j}{N+1} \tag{F.1.8}$$

Where $\bar{x}, \bar{y}$ = Average value of the data series x and y

The relationship between rainfall or flow rate (x) and return period (T) can be converted to the following equation:

$$x = x_0 + \left(\frac{1}{a}\right) y \qquad \text{(F.1.9)}$$

$$y = -ln\{ln\,T - ln(T-1)\} \qquad \text{(F.1.10)}$$

Where x_0 and a are now know parameters

(2) Data Arrangement

The data used as the input for the model are as follows:

- Maximum rainfall (normally hourly) or flow rate each year
- The table of standard parameters for Gumbel method (the relation between the number of samples, average y and the standard deviation of y (N, $\bar{y}$ and S_y)

AF.1.2 RAINFALL – RUNOFF ANALYSIS

(1) Theoretical Approach

A storage function method is used to analyze the relationship between the rainfall and runoff. The basic equations are as follows:

$$r_e - q_l = \frac{ds_l}{dt} \tag{F.2.1}$$

$$s_l = kq_l^{\,p} \tag{F.2.2}$$

where q_l = discharge, mm

r_e = average rainfall in the basin, mm

s = storage, mm

t = time, s

The above equation can be simplified and discretized as follows:

$$q_l \rightarrow q \ , \ s_l \rightarrow s \tag{F.2.3}$$

$$r_{e,t} - \frac{q_{t-\Delta t} + q_t}{2} = \frac{s_t - s_{t-\Delta t}}{\Delta t} \tag{F.2.4}$$

$$\frac{s_t}{\Delta t} + \frac{q_t}{2} = \left(\frac{s_{t-\Delta t}}{\Delta t} - \frac{q_{t-\Delta t}}{2} \right) + r_{e,t} \tag{F.2.5}$$

The Newton – Ralpson method was employed to calculate the above equation by assuming *f(q)* as follows:

$$f(q) = aq^p + bq + C = 0 \tag{F.2.6}$$

By using 2nd order Tayler's series, the derivative of *f(q)* is

$$f'(q_1) = paq_1^{\,p-1} + b \tag{F.2.7}$$

Therefore, the Newton – Ralpson equation can be expressed as:

$$y - f(q_1) = (paq_1^{\,p-1} + b) \times (q - q_1) \tag{F.2.8}$$

$$q_i = q_{i-1} - \frac{aq_{i-1}^{\,p} + bq_{i-1} + c}{paq_{i-1}^{\,p-1} + b} \tag{F.2.9}$$

From this equation, q_i can be calculated from q_{i-1}. The program will select the best value of q_i that makes

$$y - f(q_1) = 0 \quad \text{(F.2.9)}$$

(2) Data Arrangement

The data used as the input for the model are as follows:

- The synthetic rainfall pattern at each return period and
- The necessary parameters in the model (k, p and drainage basin area).

AF.1.3 HYDRAULIC SIMULATION

The data on water level and discharge are available from the gauging stations in the basin. Hydrograph is calculated and used as a boundary condition. An unsteady flow program, MIKE11 developed by the Danish Hydraulic Institute (DHI), is used to simulate the flow along the river.

(1) Theoretical Approach

The program can be used to solve the vertically integrated equations of conservation of continuity and momentum (so called "Saint Venant equation") for incompressible and homogeneous fluid. The basic governing equations are:

$$\frac{\partial Q}{\partial x}+\frac{\partial A}{\partial t} = q \qquad \text{(F.3.1)}$$

$$\frac{\partial Q}{\partial t}+\frac{\partial\left(\alpha \frac{Q^2}{A}\right)}{\partial x}+gA\frac{\partial h}{\partial x}+\frac{gQ|Q|}{C^2AR} = 0 \qquad \text{(F3.2)}$$

where Q = discharge, m^3/s

A = flow area, m^2

q = lateral inflow, m^2/s

h = stage above datum, m

C = Chezy resistance coefficient, $m^{1/2}/s$

R = hydraulic radius, m

α = momentum distribution coefficient

g = gravity acceleration, m/s^2

t, x = The axis of time, s, and distance, m, respectively

These equations are transformed into a series of finite difference equations in a computational grid consisting of alternating Q-points (discharge) and h-points (water level). The transformed equations are as follows:

$$\frac{\partial Q}{\partial x} \approx \frac{\frac{\left(Q_{j+1}^{n+1}+Q_{j+1}^{n}\right)}{2}-\frac{\left(Q_{j-1}^{n+1}+Q_{j-1}^{n}\right)}{2}}{\Delta 2x_j} \qquad \text{(F.3.3)}$$

$$\frac{\partial A}{\partial t} = b_s\frac{\partial h}{\partial t} \approx \frac{\left(h_j^{n+1}-h_j^{n}\right)}{\Delta t} \qquad \text{(F.3.4)}$$

$$\frac{\partial Q}{\partial t} \approx \frac{\left(Q_j^{n+1} - Q_j^n\right)}{\Delta t} \qquad \text{(F.3.5)}$$

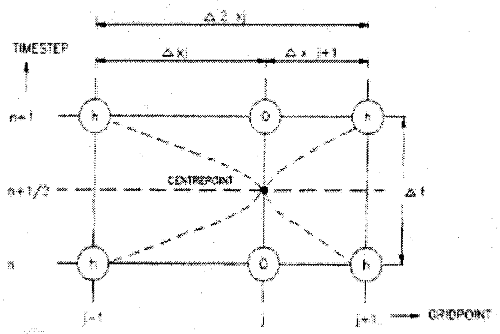
$$\frac{\partial\left(\alpha \dfrac{Q^2}{A}\right)}{\partial x} \approx \frac{\left(\left[\alpha \dfrac{Q^2}{A}\right]_{j+1}^{n+\frac{1}{2}} - \left[\alpha \dfrac{Q^2}{A}\right]_{j-1}^{n+\frac{1}{2}}\right)}{\Delta 2x_j} \qquad \text{(F.3.6)}$$

$$\frac{\partial h}{\partial x} \approx \frac{\dfrac{\left(h_{j+1}^{n+1} + h_{j+1}^{n}\right)}{2} - \dfrac{\left(h_{j-1}^{n+1} + h_{j-1}^{n}\right)}{2}}{\Delta 2x_j} \qquad \text{(F.3.7)}$$

where b_s = river width, m

n, j = time and distance step

The schematic diagram of time and distance increment is illustrated as follows:



(2) Data Arrangement

The data used as the input for the model are as follows:

- The grid set up from the river survey for river network,
- River cross sections' coordinates,
- River bed and material data, and
- The boundary condition, in this case the hydrograph at the upstream end and the water level at the downstream end.

APPENDIX F.2

SUPPLEMENTAL DATA ON

GUACERIQUE RIVER BASIN

Table AF.2.1 Annual Rainfall in the Guacerique River Basin

Year	Rainfall (mm)	
	Batallon	Q Montes
1963	315.5	
1964	1,247.7	
1965	1,099.8	
1966	1,280.7	
1967	499.8	
1968	1,075.0	
1969	1,619.5	
1970	1,296.1	
1971	1,065.3	
1972	0	
1973	977.1	
1974	915.8	
1975	1,026.0	
1976	947.0	
1977	736.4	
1978	746.6	
1979	1,005.8	
1980	1,074.5	
1981	1,057.2	
1982	733.9	
1983	882.0	
1984	1,093.5	
1985	757.6	
1986	761.3	
1987	887.9	
1988	1,218.7	
1989	893.5	
1990	847.9	
1991	834.9	
1992	876.2	935.1
1993	942.7	1,056.9
1994	758.2	866.5
1995	1,272.9	1,472.2
1996	862.2	937.0
1997	782.6	970.0

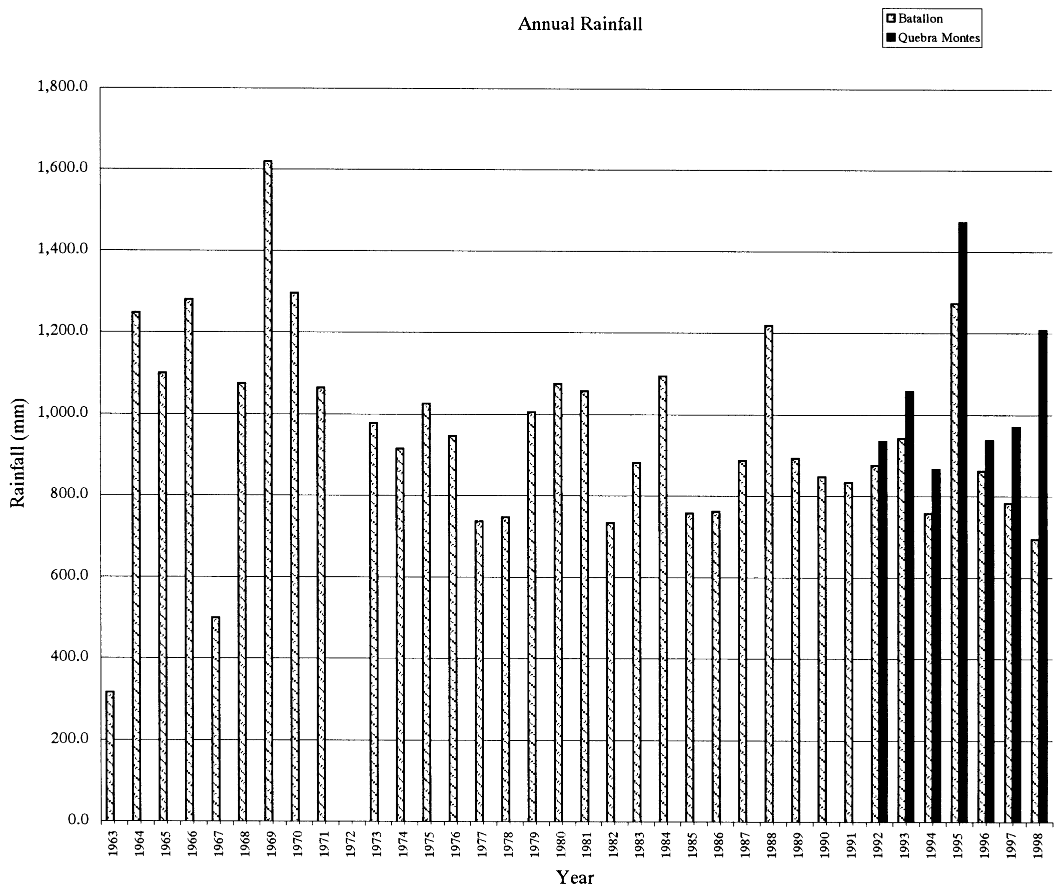


Table AF.2.2 Maximum, Minimum and Average Flow Rate in the Guacerique River Basin

Year	Guacerique River Basin							
	Guacerique II Station				Quebra Montes Station			
	Flow rate (m^3/s)			Annual (m^3/year)	Flow rate (m^3/s)			Annual (m^3/year)
	Max	Min	Average		Max	Min	Average	
1970								
1971								
1972								
1973								
1974								
1975								
1976								
1977								
1978								
1979								
1980								
1981								
1982	171.000	0.062	0.997	31,451,904				
1983	217.000	0.029	1.524	48,071,376				
1984	42.300	0.075	2.049	64,609,380				
1985	14.400	0.043	0.956	30,151,044				
1986	38.900	0.038	0.852	26,881,812				
1987	57.500	0.030	1.130	35,625,168				
1988	125.700	0.041	2.174	68,543,496				
1989	40.000	0.075	1.523	48,031,956				
1990	25.200	0.036	1.293	40,765,536				
1991	69.200	0.031	0.865	27,293,388	10.900	0.059	0.489	15,415,370
1992	72.529	0.011	0.794	25,037,627	9.270	0.040	0.436	13,754,952
1993	32.300	0.021	1.527	48,170,127	4.840	0.053	0.757	23,862,240
1994	137.000	0.048	0.754	23,783,755	4.390	0.055	0.330	10,392,545
1995	99.400	0.098	2.942	92,791,383	5.020	0.108	0.819	25,830,421
1996	39.600	0.024	1.521	47,969,510				
Average	78.802	0.044	1.393	43,945,164	6.884	0.063	0.566	17,851,106

Table AF.2.3 Maximum Daily and Annual Rainfall at Toncontin Station

Year	Rainfall (mm)	
	Max. Daily	Annual
1951	76.20	786
1952	61.20	1,146
1953	47.80	823
1954	54.40	1,173
1955	49.80	1,274
1956	44.20	689
1957	63.20	779
1958	78.70	972
1959	109.00	944
1960	45.50	962
1961	53.10	774
1962	93.00	1,066
1963	47.80	883
1964	69.30	893
1965	77.20	766
1966	79.20	1,047
1967	46.20	641
1968	83.30	1,025
1969	45.00	1,199
1970	65.20	1,003
1971	46.70	750
1972	34.30	453
1973	60.50	1,078
1974	68.10	861
1975	86.00	995
1976	44.50	750
1977	74.50	776
1978	57.60	731
1979	78.10	1,180
1980	62.30	996
1981	54.40	1,113
1982	49.20	718
1983	49.40	719
1984	94.40	1,084
1985	39.90	610
1986	41.00	503
1987	66.10	693
1988	82.00	1,264
1989	36.90	878
1990	73.10	675
1991	38.30	595
1992	54.10	728
1993	43.10	949
1994	75.70	564
1995	56.60	1,146
1996	73.00	889
1997	94.80	835
1998	120.40	1,180
1999	53.00	870

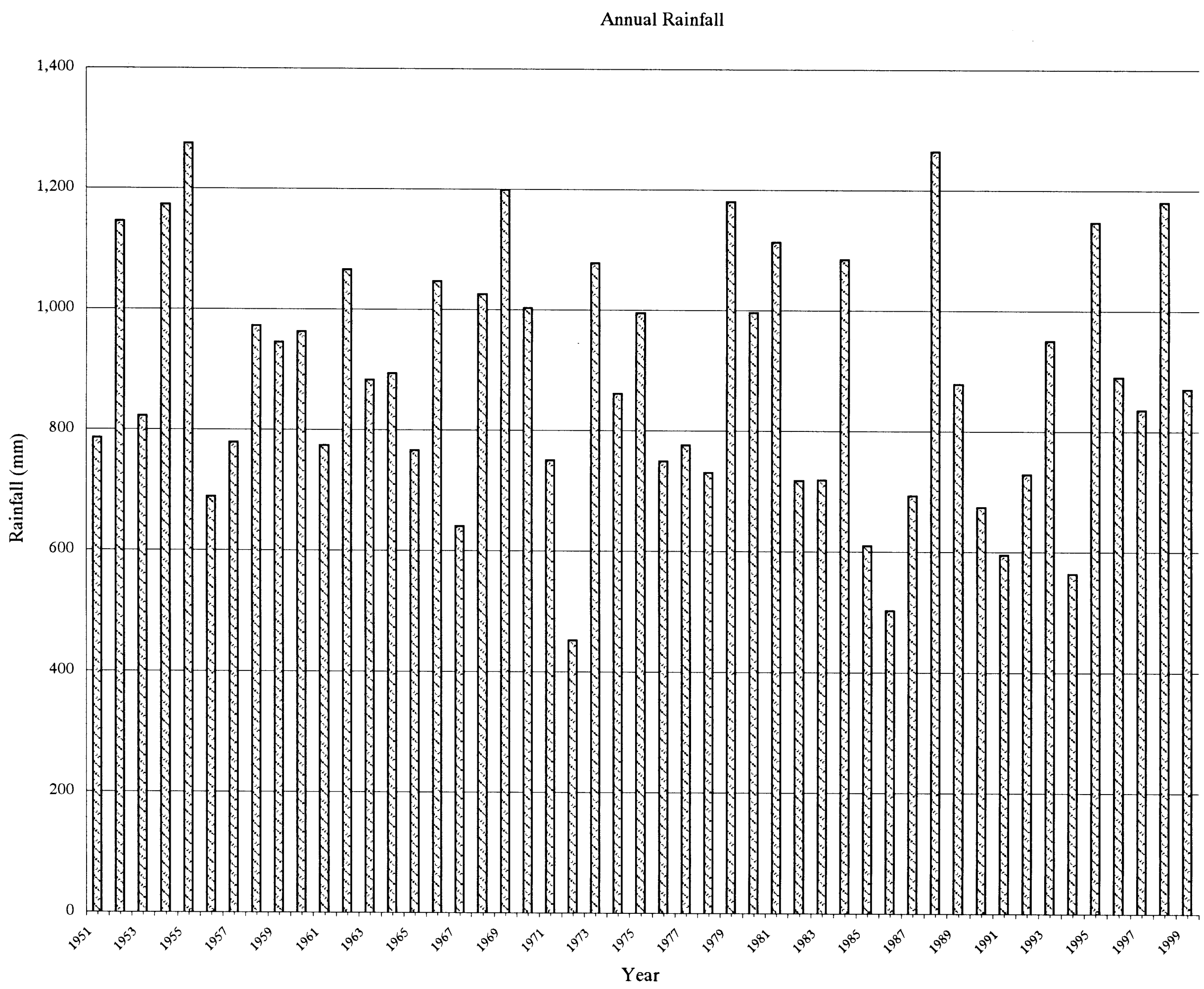


Table AF.2.4 Annual Minimum Flow Rate in Guacerique River

Year	Min. Flow
1982	0.062
1983	0.023
1984	0.076
1985	0.043
1986	0.022
1987	0.030
1988	0.041
1989	0.075
1990	0.036
1991	0.031
1992	0.011
1993	0.021
1994	0.048
1995	0.098
1996	0.024

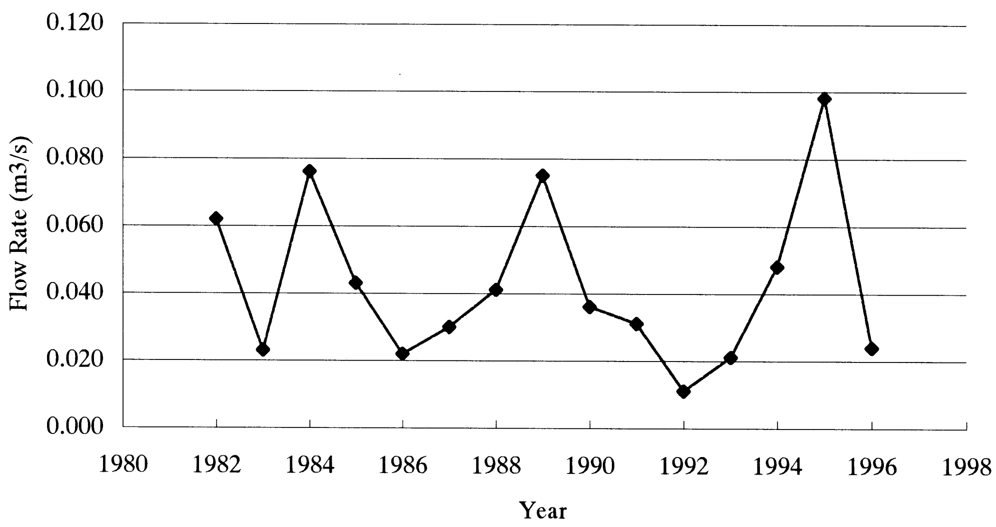


Table AF.2.5 H-V Curves for the Existing and Proposed Dams in the Study Area

Equation for H-V curve

$$V = aH^4+bH^3+cH^2+dH+e$$

V = Storage volume, m3

H = Water level, m

1. Existing Los Laureles Dam	
1. Original plan with no sedimentation	
a =	16.29537770221940
b =	-66,119.90231596630
c =	100,612,634.0234170
d =	-68,047,131,006.38210
e =	17,259,022,436,912.70
2. Modified plan with sedimentation	
a =	4.064967057776810
b =	-16,361.1722885303000
c =	24,700,283.232502100
d =	-16,576,913,699.0335000
e =	4,172,835,710,490.38000
2. Proposed Los Laureles II Dam	
a =	7.913333333388440
b =	-32,849.54814838030
c =	51,146,591.72258920
d =	-35,400,273,195.99860
e =	9,189,924,318,590.160
3. Quebra Montes Dam	
a =	-15.49055368496920
b =	70,020.80995123880
c =	-118,656,492.0031590
d =	89,341,510,978.68450
e =	-25,219,293,754,287.70
4. Sabacuante Dam	
a =	5.940680506711940
b =	-26,352.7844188190
c =	43,843,433.81146130
d =	-32,422,510,282.28960
e =	8,992,019,674,088.430
5. Tatumbla Dam	
a =	3.333333333372140
b =	-15,283.33333350810
c =	26,288,416.66696150
d =	-20,104,567,916.88750
e =	5,767,848,037,561.930

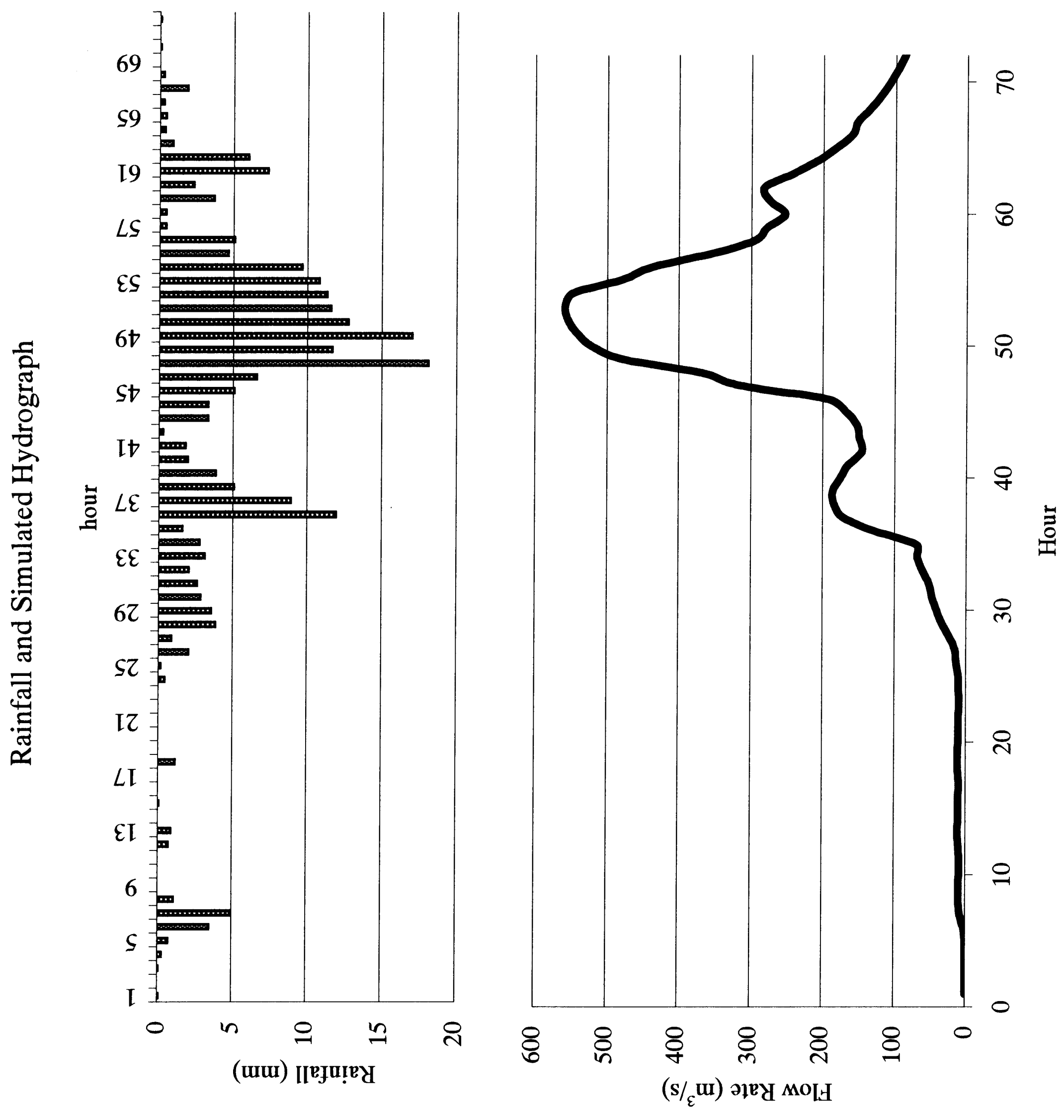


Figure AF.2.1	Relationship of Rainfall and Simulated Hydrograph

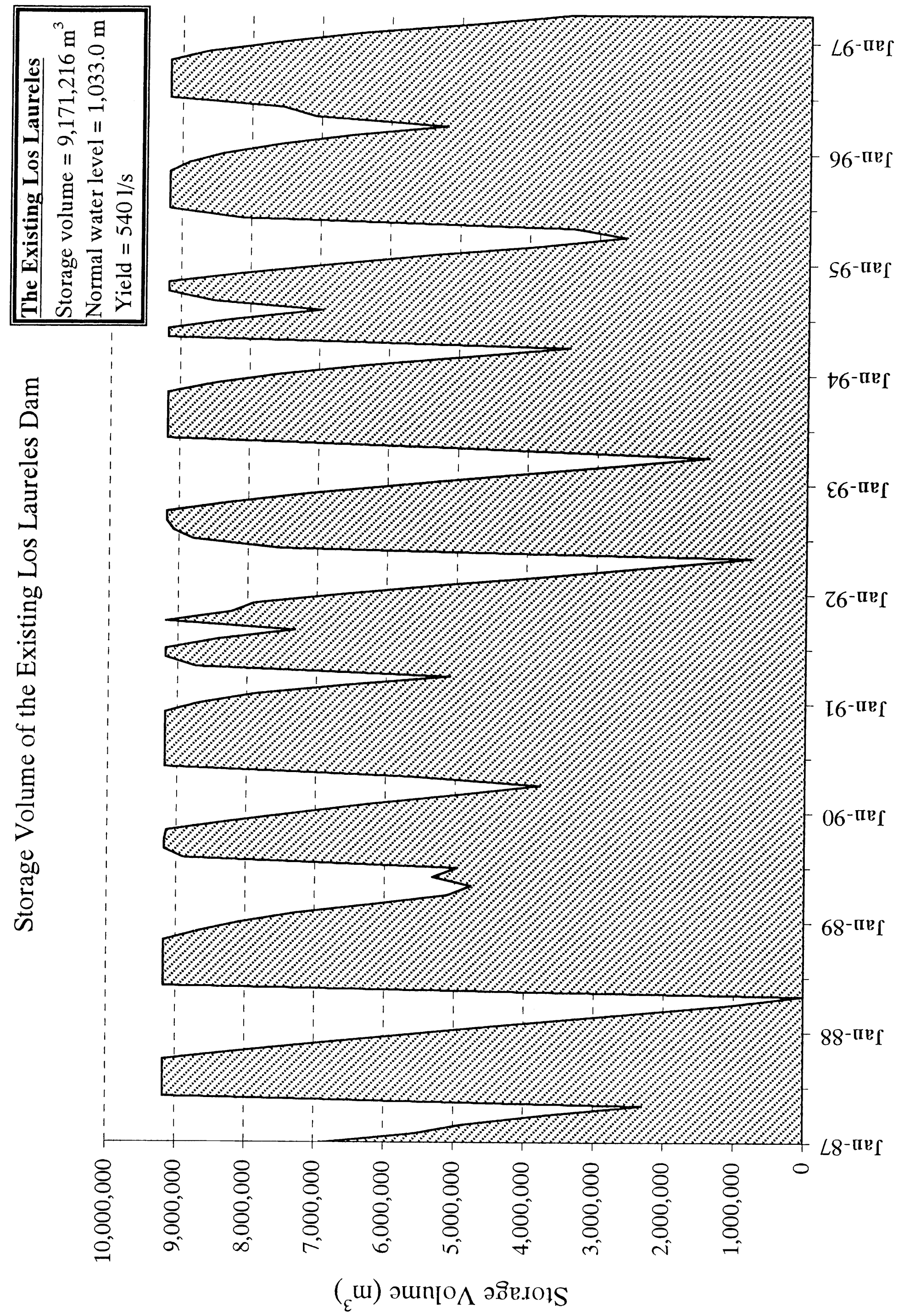


Figure AF.2.2 (1)	Storage Volume of the Existing Los Laureles Dam

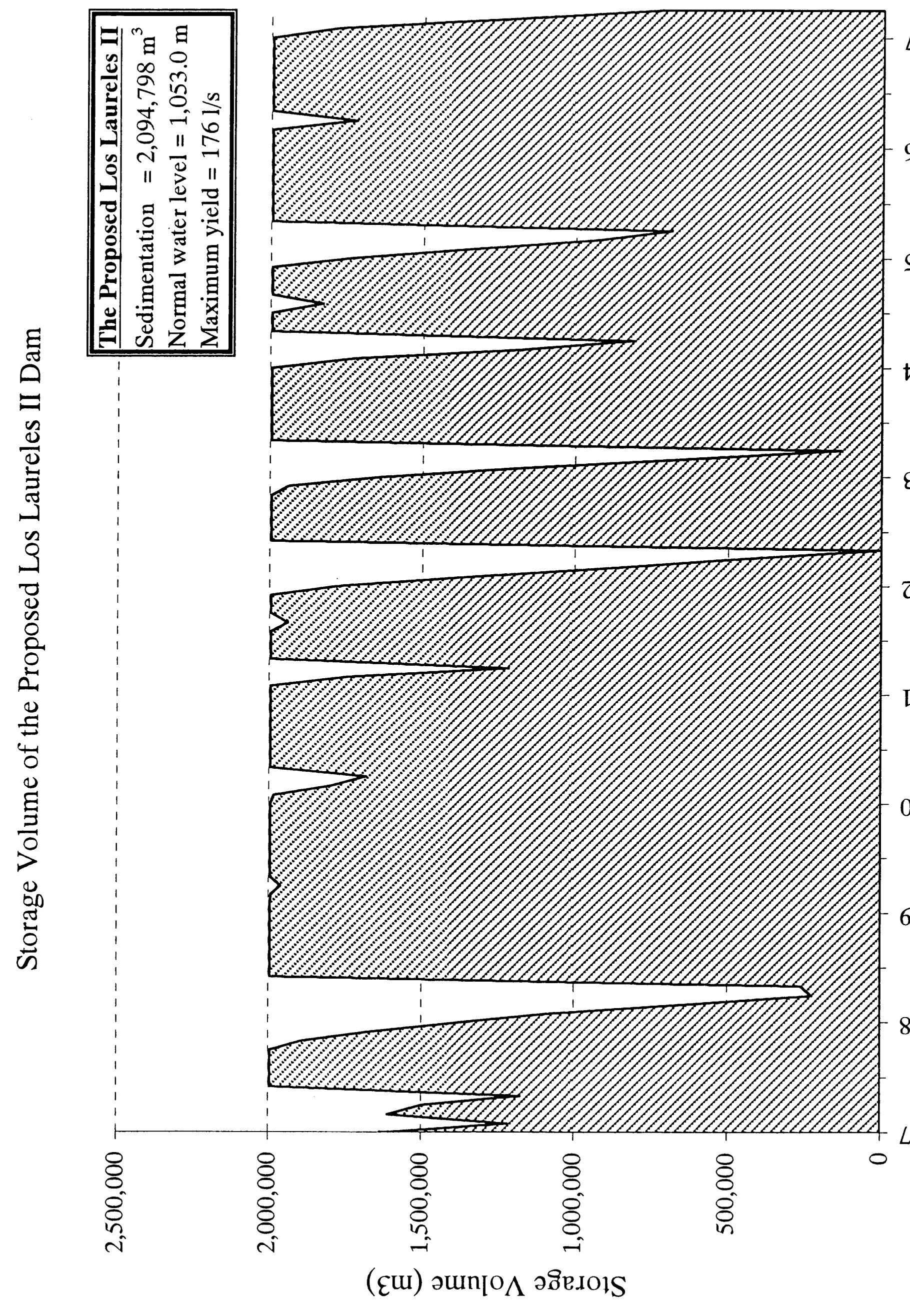


Figure AF.2.2 (2)	Storage Volume of the Proposed Los Laureles II Dam

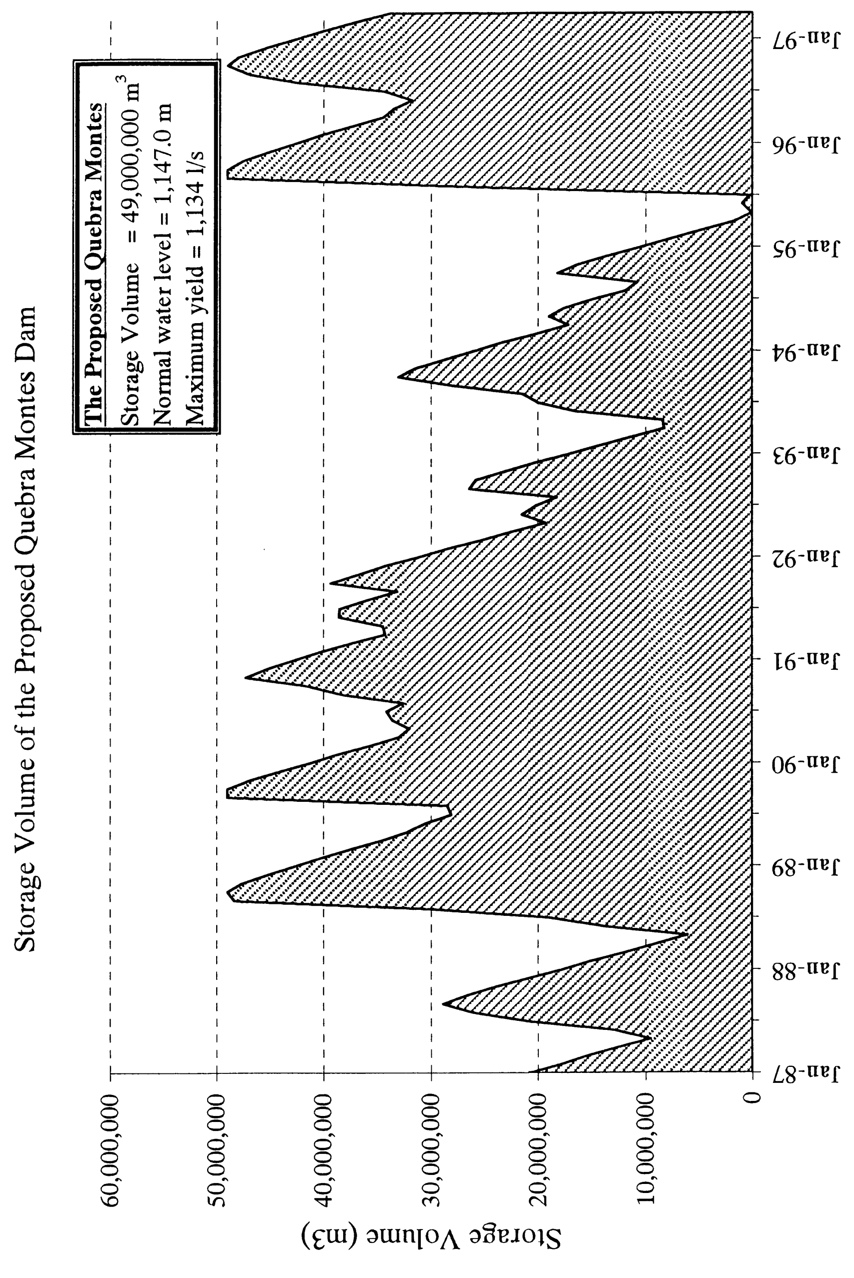
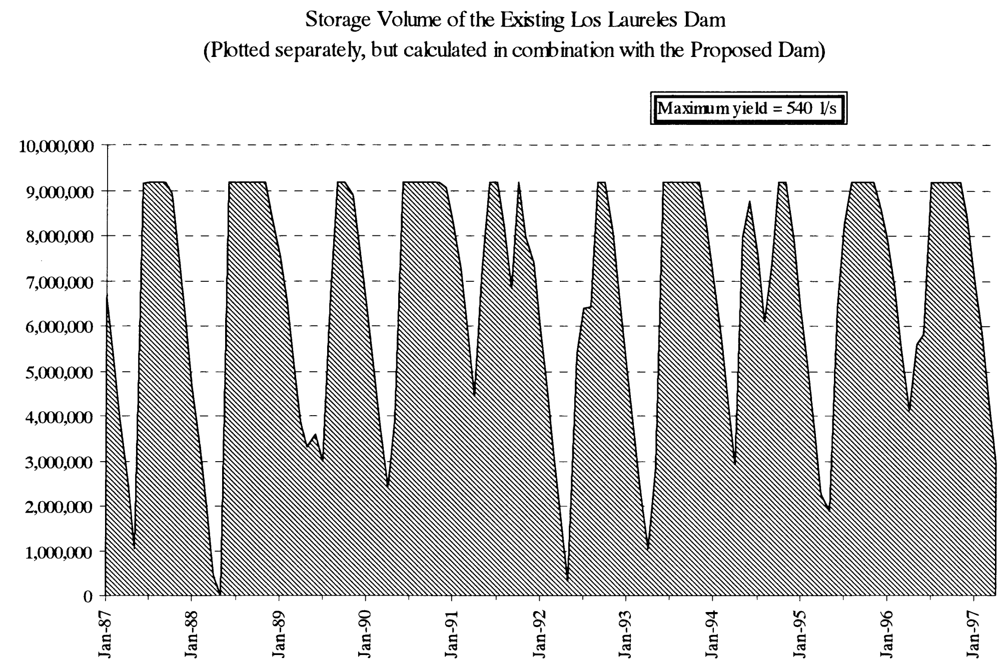


Figure AF.2.2 (3)	Storage Volume of the Proposed Quiebra Montes Dam



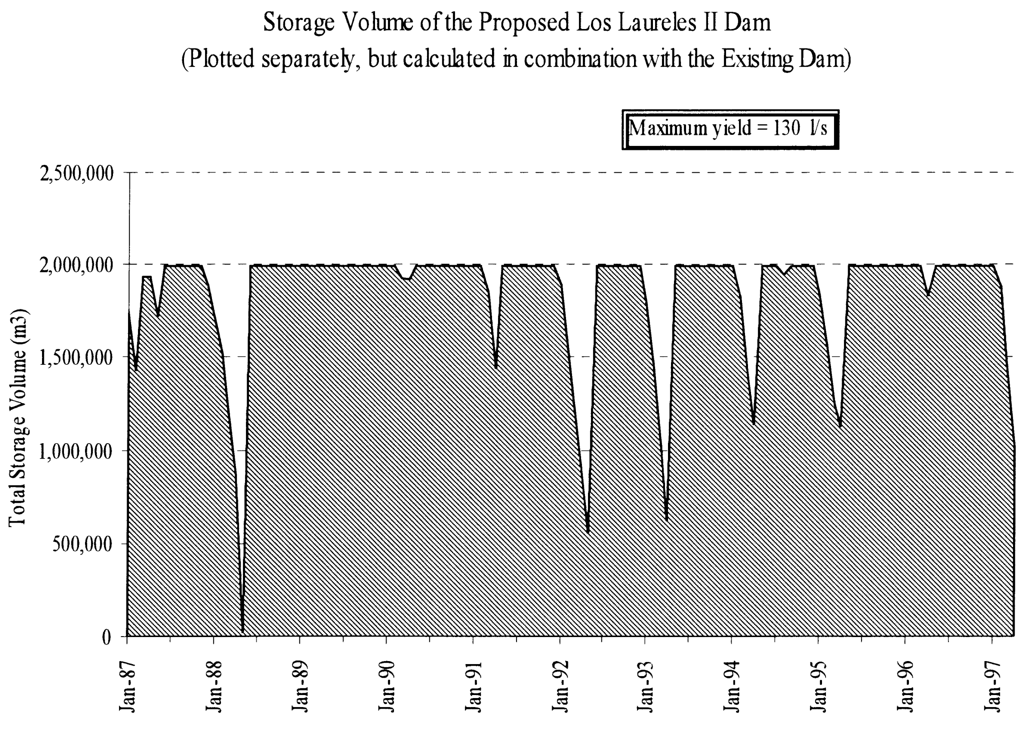


Figure AF.2.2 (4)	Storage Volume of 2 Dams (Existing Los Laurels + Proposed Los Laureles II)

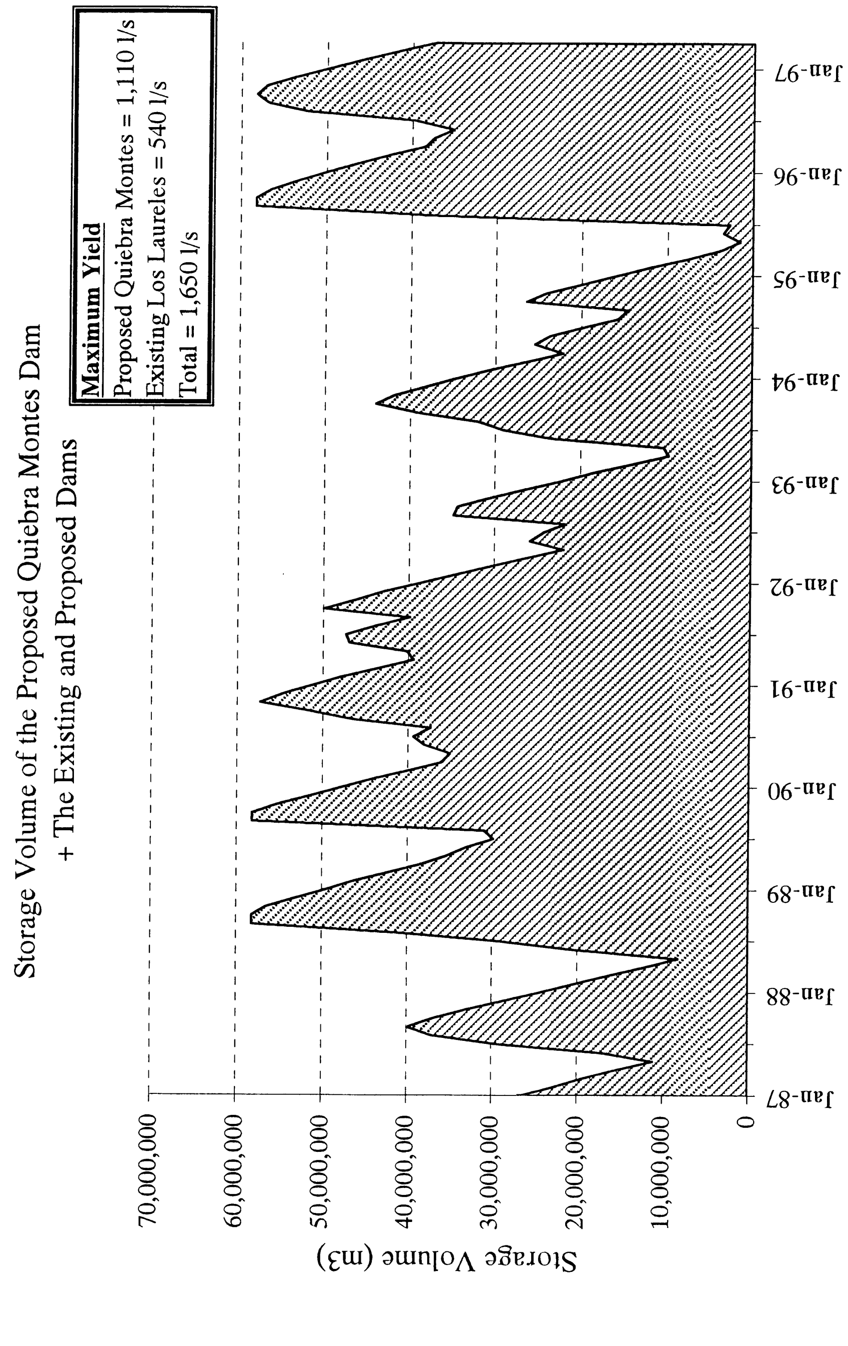


Figure AF.2.2 (5)	Storage Volume of 2 Dams (Existing Los Laureles + Proposed Quiebra Montes)

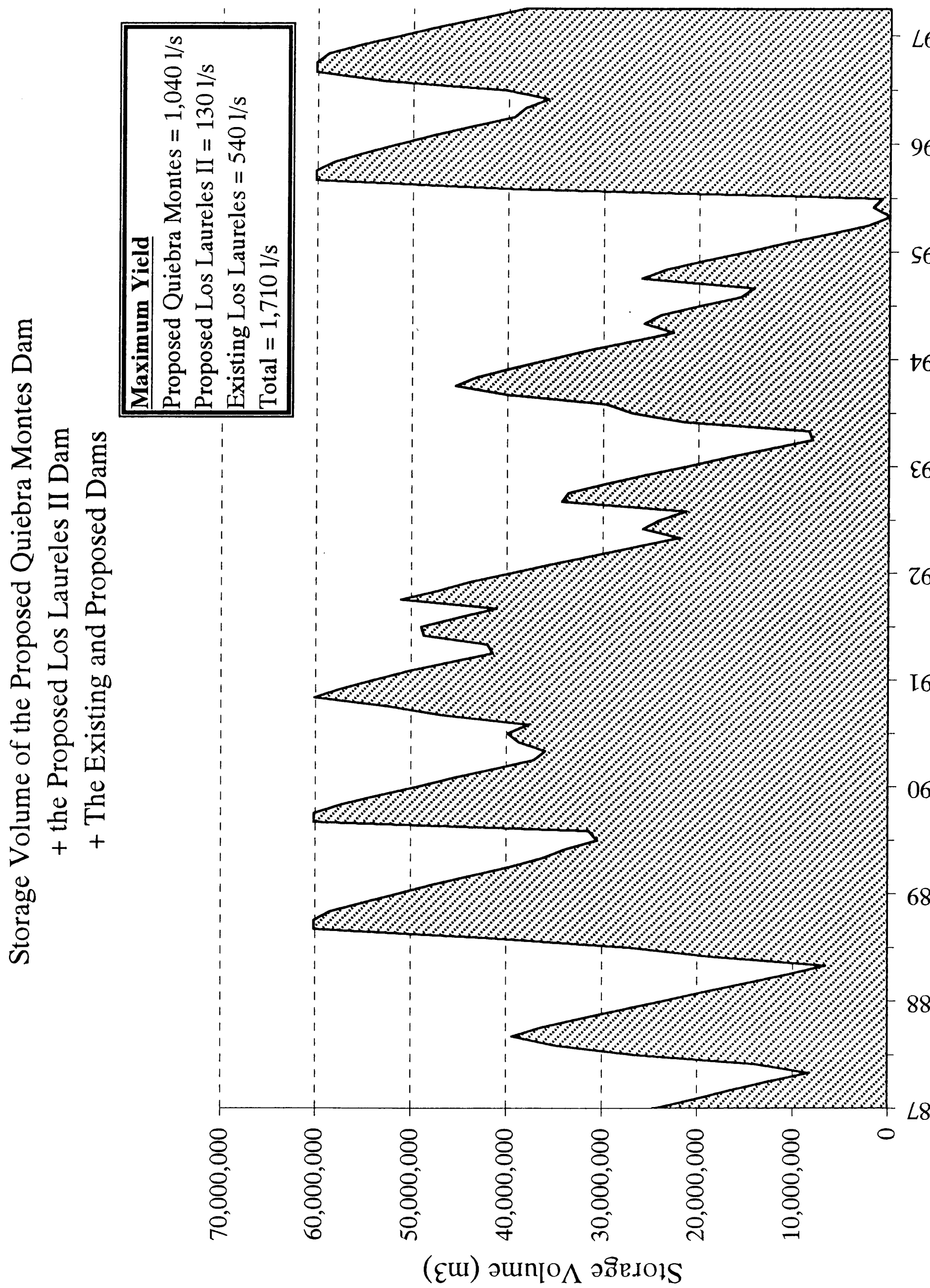


Figure AF.2.2 (6)	Storage Volume of 3 Dams (Los Laureles + Proposed Los Laureles II & Quiebra Montes)

APPENDIX F.3

SUPPLEMENTAL DATA ON

SABACUANTE RIVER BASIN

Table AF.3.1 Maximum, Minimum and Average Flow Rate in the Sabacuate River Basin

Year	Sabacuante River Basin			
	El Agucate Station			
	Flow rate (m^3/s)			Annual (m^3/year)
	Max	Min	Average	
1970	16.000	0.012	0.661	20,831,829
1971	15.800	0.013	0.426	13,420,647
1972	8.150	0.013	0.190	5,978,753
1973	7.330	0.011	0.428	13,505,270
1974	40.300	0.009	0.467	14,716,800
1975	7.940	0.017	0.387	12,196,591
1976	6.540	0.015	0.223	7,019,309
1977	11.900	0.005	0.224	7,050,586
1978	3.890	0.000	0.121	3,802,291
1979	25.100	0.001	0.894	28,200,096
1980	88.751	0.000	1.534	48,361,217
1981	14.330	0.017	1.346	42,438,334
1982	4.538	0.005	0.176	5,556,470
1983	2.490	0.007	0.114	3,599,326
1984	9.360	0.001	0.312	9,842,515
1985	1.440	0.000	0.069	2,171,750
1986	57.302	0.004	0.343	10,817,798
1987	1.543	0.005	0.058	1,824,975
1988	13.360	0.001	0.435	13,718,160
1989	10.697	0.094	0.481	15,176,730
1990	1.223	0.016	0.083	2,625,626
1991				
1992				
1993				
1994				
1995				
1996				
Average	16.571	0.012	0.427	13,469,289

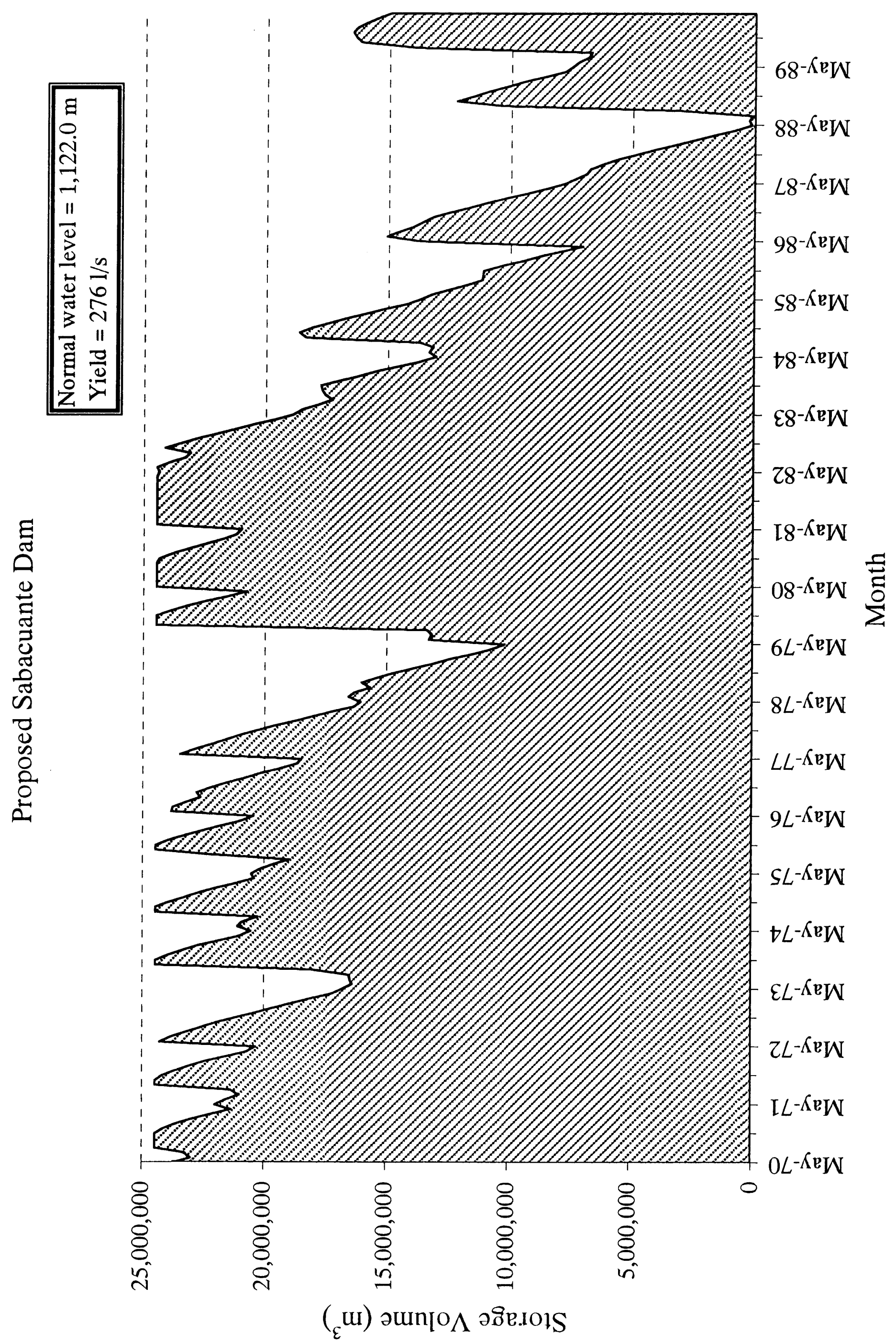


Figure AF.3.1	Storage Volume of the Proposed Sabacuante Dam

APPENDIX F.4

SUPPLEMENTAL DATA ON

TATUMBLA RIVER BASIN

Table AF.4.1 Maximum, Minimum and Average Flow Rate in the Tatumbla River Basin

Year	Tatumbla River Basin			
	El Incienso Station			
	Flow rate (m^3/s)			Annual ($m^3/year$)
	Max	Min	Average	
1970				
1971	11.100	0.007	0.303	9,569,280
1972	4.290	0.012	0.131	4,131,734
1973	6.830	0.007	0.346	10,897,546
1974	32.300	0.012	0.332	10,470,730
1975	7.370	0.012	0.443	13,966,743
1976	3.460	0.023	0.219	6,902,582
1977	7.440	0.033	0.183	5,755,882
1978	3.360	0.010	0.204	6,426,605
1979	8.140	0.022	0.535	16,868,486
1980	36.700	0.007	1.121	35,355,917
1981	7.020	0.014	0.498	15,704,755
1982	5.700	0.013	0.332	10,479,370
1983	5.160	0.009	0.225	7,106,974
1984	9.630	0.005	0.579	18,243,619
1985	1.160	0.006	0.071	2,230,243
1986	6.091	0.017	0.227	7,156,080
1987				
1988				
1989				
1990				
1991				
1992				
1993				
1994				
1995				
1996				
Average	9.734	0.013	0.359	11,329,159

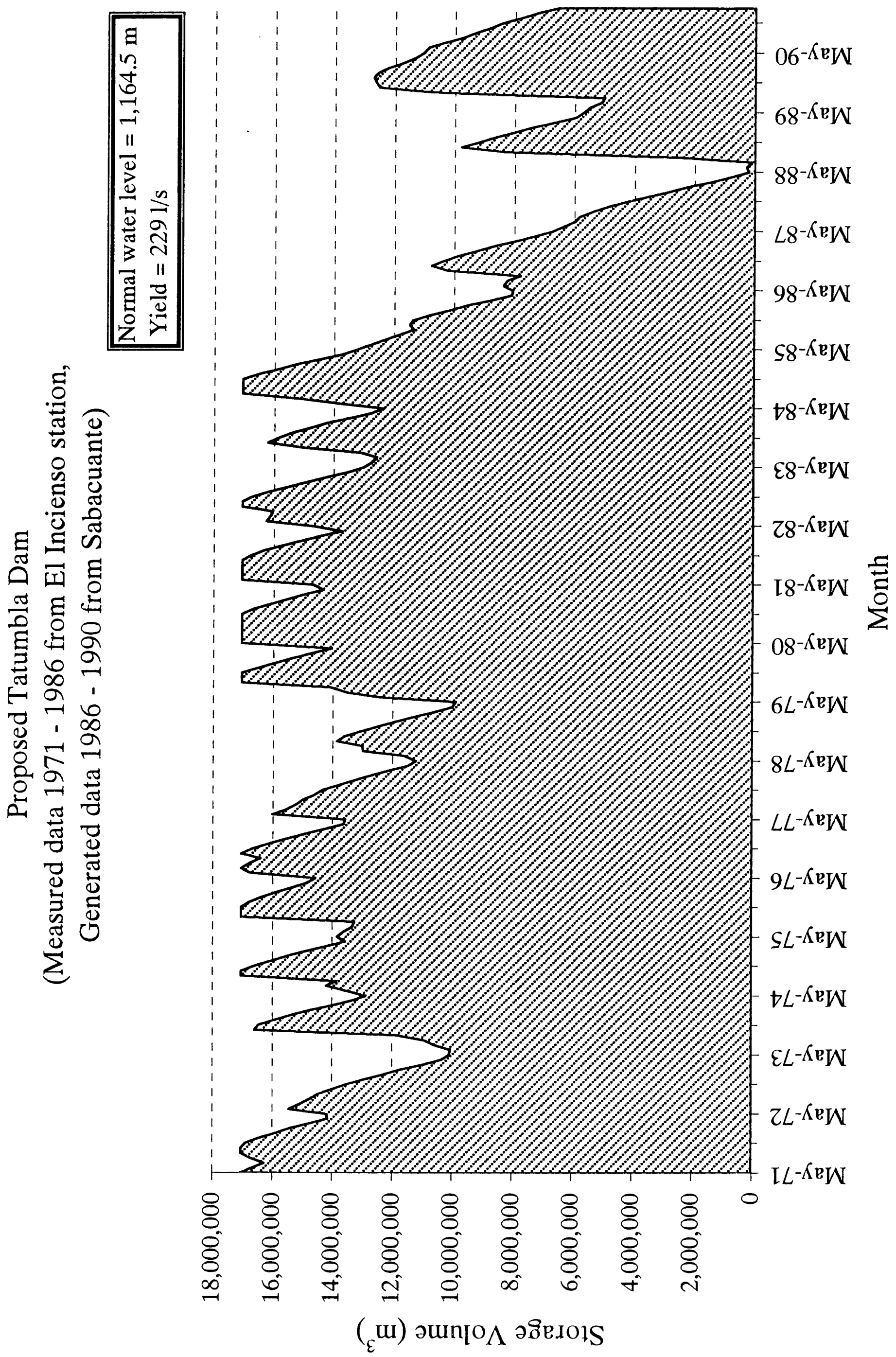


Figure AF.4.1	Storage Volume of the Proposed Tatumbla Dam