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Autoridad Del Canal De Panama
División de Proyectos de Capacidad del Canal

Work Order No.5 Feasibility Design For The Ríos Coclé Del Norte And Caño Sucio Water Supply Projects

Panama Canal

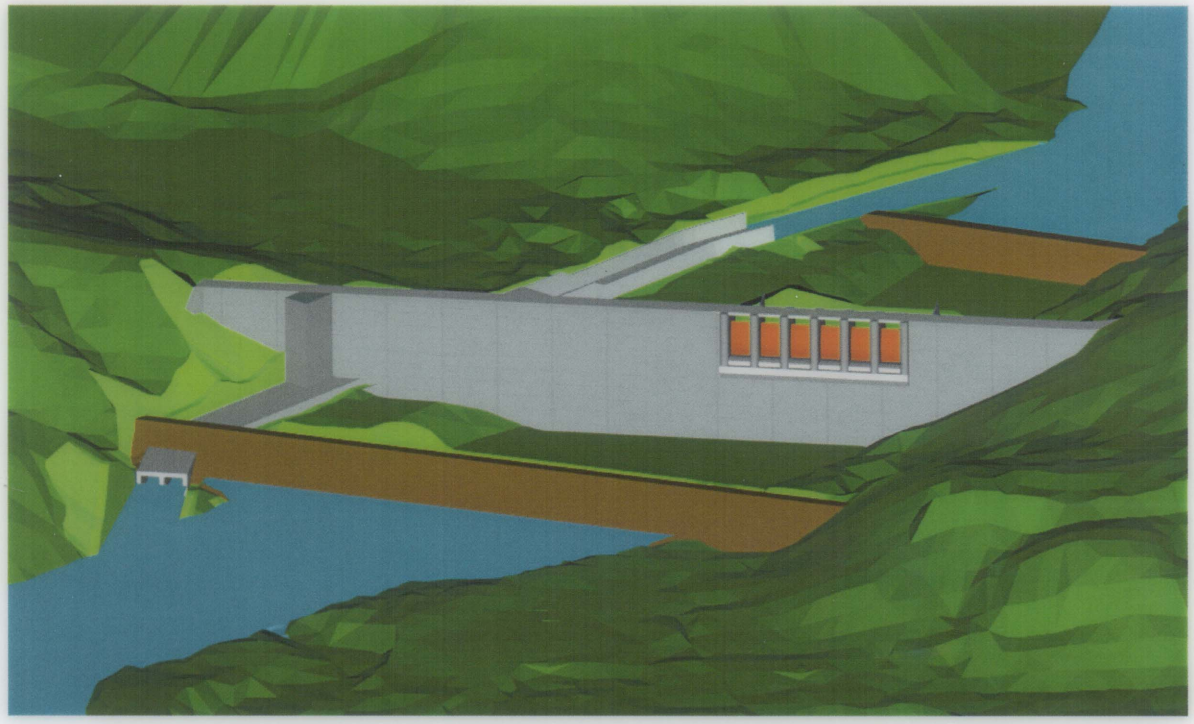
Contract Number CC-3-536

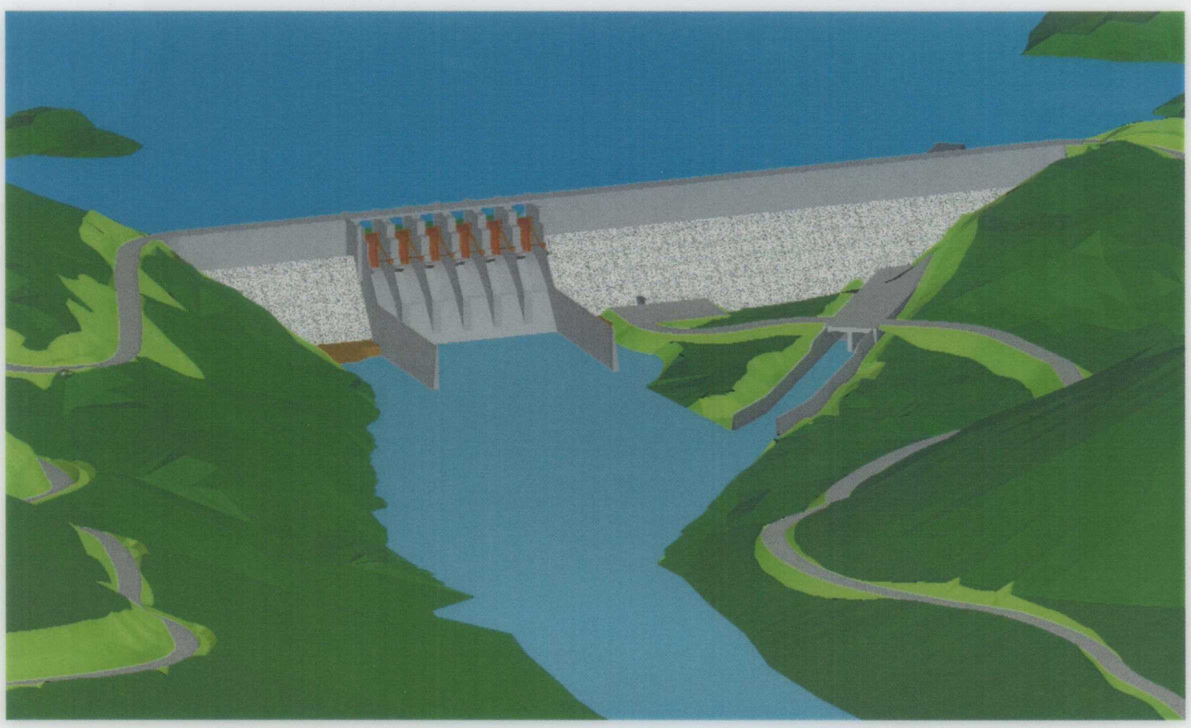
Low Coclé Del Norte Water Supply Project

December 2003

MWH

Low Coclé del Norte Water Supply Project





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AUTORIDAD DEL CANAL DE PANAMA
Division de Proyectos de Capacidad del Canal

THE PANAMA CANAL

ENGINEERING SERVICES

Work Order No. 5
The Ríos Coclé del Norte and Caño Sucio
Water Supply Projects

Preliminary Assessment
of the
Low Coclé del Norte Water Supply Project

DECEMBER 2003

MWH

Executive Summary

EXECUTIVE SUMMARY

INTRODUCTION

The ACP is undertaking a canal capacity study, which includes the evaluation of additional sources of water to augment Canal capacity. The US Army Corps of Engineers (USACE) performed a reconnaissance study for the Panama Canal Commission to identify and evaluate potential water supply projects (1). The Río Coclé del Norte Project was recommended as a feasible alternative and was studied and reported on in a separate report.

The full development of the Río Coclé del Norte water resource will have a significant impact in the Río Coclé del Norte basin due to the need for the construction of large reservoirs. While the operation of the Canal may eventually require the full development of the Río Coclé del Norte basin, the ACP is interested in examining other options that have less impact. The Low Coclé del Norte Project was identified as a potential option.

OBJECTIVE OF THE STUDY

The objectives of this study are to:

- Estimate the cost and yield of a low dam on the Río Coclé del Norte with pumping facilities to deliver the water into the Panama Canal System either at Toabré Reservoir or Caño Sucio Reservoir, and
- Compare the unit cost of water with the unit costs for the Río Indio, Upper Charges, and Río Coclé del Norte Projects.

To the extent possible, information developed for the existing feasibility-level studies was used.

PUMPING ALTERNATIVES

Two alternatives were considered to pumping water from the Low Coclé Del Norte reservoir to the Panama Canal System:

Alternative 1 - Pumping water from Low Coclé Del Norte Reservoir to Toabré Reservoir; and
Alternative 2 - Pumping water from Low Coclé Del Norte Reservoir to Caño Sucio Reservoir.

Alternative 1 is analyzed using the mean annual flow for the Coclé del Norte basin excluding the drainage above Toabré Dam. Alternative 2 uses the entire basin flow.

For Alternative 1, the following three scenarios were considered:

> Scenario 1 - Pumping against variable reservoir elevations with energy requirements supplied from the national electric grid,
>
> Scenario 2 - Pumping against a constant elevation over the top of the Toabré dam with energy requirements supplied from the national electric grid, and
>
> Scenario 3 - Pumping against variable reservoir elevations with energy supplied by a hydroelectric power station at the Low Coclé Del Norte dam.

Based on an evaluation of the pumping options, Alternative 1, Scenario 1 was selected for inclusion in the project. It was also determined that Alternative 2 would have essentially the same economic impact on the project

ELEMENTS OF THE LOW COCLÉ DEL NORTE WATER SUPPLY PROJECT

The major elements that comprise the Low Coclé del Norte Water Supply Project include:

- A roller compacted concrete dam with the top-of-structure at El. 41,
- A gated spillway located in the right-center of the dam (looking downstream) sized to protect the dam against the probable maximum flood,
- A two-barrel conduit for diversion that will be converted into the emergency drawdown facilities,
- Protection facilities for the town of Coclecito that will protect it against the operation of the reservoir, and
- A pumping station designed to pump up to the 10% exceedance flow or 100 m^3/s into the Panama Canal System through the Toabré Reservoir.

HYDROLOGY AND RIVER HYDRAULICS

Existing information was used or studies were performed to develop the long-term streamflow sequence, and to estimate the spillway design flood and anticipated reservoir sedimentation. The selected pumping alternative presumes that the Toabré Dam is in place and, therefore, the hydrologic studies were performed for the Coclé del Norte Basin excluding the Toabré Dam drainage.

The mean annual flow at the Río Coclé del Norte damsite is estimated to be 107.5 m^3/s.

The mean annual flow for the basin excluding the Toabré Dam drainage, which was used to evaluate the project, was estimated using a ratio of drainage areas for the damsite and El Torno and the completed streamflow data for El Torno. The mean annual flow at the

Río Coclé del Norte damsite excluding the Toabré Dam drainage is estimated to be 69.4 m^3/s and the monthly distribution of flow is shown below:

MONTHLY MEAN STREAMFLOW AT THE RÍO COCLÉ DEL NORTE DAMSITE EXCLUDING THE TOABRÉ DAM DRAINAGE (m^3/s)

Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Annual
60.4	39.4	29.5	41.2	65.1	69.4	67.3	80.0	85.9	96.5	102.8	95.6	69.4

The probable maximum flood (PMF), based on probable maximum precipitation (PMP) was adopted as the spillway design flood for the Project. Based on information presented in the National Weather Service publication of PMP dated 1978 and the Weather Bureau publication of depth-area-duration dated 1965, the PMP was estimated to be 714 mm.

The PMP was transformed to a PMF using the HEC-1 computer model. The probable maximum flood hydrograph has an estimated peak discharge of 9,970 m^3/s and a 3-day volume of 950 MCM.

The impact of sediment deposition on storage in the reservoir was evaluated using data from Lake Madden and other sources. After 100 years, it is expected that sediment deposition will reduce the gross storage by about 15 percent and will reach to about El. 4 at the dam.

GEOLOGIC CONDITIONS

The geologic conditions at the dam site were established as a part of the feasibility studies for the Coclé del Norte and Caño Sucio Projects (2).

In general, the foundation bedrock at the site is not expected to present any significant constraints on project development that cannot be taken care of with appropriate conventional design details and construction practices. In regard to other geological aspects, there do not appear to be any strongly adverse conditions or fatal flaws at the site.

As reported in the Executive Summary of the Toabré Dam Feasibility Study (3), either a roller-compacted concrete (RCC) dam or a concrete-faced rockfill dam (CFRD) is a viable alternative for the site, and an RCC dam was recommended for the site. If this assessment is accepted, then the location of a pumping station at the toe of the dam should not be a problem.

The construction of the Low Coclé del Norte Project will require appropriate aggregate for the roller compacted concrete (RCC) mix, concrete aggregate, and random material for the initial upstream and the downstream cofferdams.

All aggregates (including coarse and fine aggregates for concrete and the RCC mix) need to be manufactured from quarried sources. These aggregates will be manufactured from igneous rock materials from a quarry located 3 km to 5 km east of the damsite. The diversion cofferdams will be constructed from locally available random fill obtained from required excavation at the dam.

DESCRIPTION OF THE PROJECT FACILITIES

The dam will impound a reservoir with a gross storage capacity of 662 MCM at El. 35, the full supply level. The reservoir area at the full supply level will be about 51 square kilometers. A general plan of the development is presented on Exhibit 1.

Upon completion of the project facilities, the project will have the capability to deliver an average of 1,854 MCM/year into the Panama Canal system through Toabré Reservoir. The resultant incremental yield of the System at a reliability of 99.6% has not been determined. However, it is expected that, with the storage capacity available in the system, that the incremental system yield will closely approximate the amount of water supplied. This would translate into about 24.6 lockages per day in the canal system.

The Low Coclé del Norte Dam will be constructed of a low to medium cement content concrete mix with aggregate taken from a quarry located 3 km to 5 km east of the damsite. The dam will be about 51 m high from the deepest foundation excavation to the top of the dam. A precast concrete facing will act as the impermeable membrane. A site plan for the dam and appurtenant works is shown on Exhibit 2.

A gated chute spillway will be located in the right-center portion of the dam (looking downstream). The discharge under PMF conditions will be 6,520 m^3/s using a surcharge of 3.3 m above the full supply level. Control will be afforded by 6 radial gates, each 8-m wide by 12-m high. Energy from the spill will be dissipated in a stilling basin located at the toe of the dam.

The facilities for the river diversion during construction will consist of cofferdams upstream and downstream from the damsite and two 7 m by 7 m conduits located on the left side of the river channel. The conduits will serve to pass the 25-year flood during construction, control the rate of initial reservoir filling, and provide for emergency evacuation of the reservoir.

Emergency drawdown will be accomplished using the spillway and one of the diversion conduits. For emergency drawdown, an intake will deliver water into the conduit. Flow will be controlled by two 3 m by 6 m gates with sills at El. 1. The minimum release will be discharged through a valve located in the emergency drawdown gate housing.

The area around Coclecito will be protected from the operation of the reservoir by a series of levees, detention ponds, pumps, and channel improvements. A plan of the facilities is shown on Exhibit 3. The facilities will totally protect Coclecito from the 100-year flood and will prevent major flooding for up to the PMF.

The pumping facilities at the toe of the Toabré Dam will consist of a station containing 4-24 MW pumps and 4-13 MW pumps designed to deliver up to 100 m^3/s under head conditions that fluctuate with the level of the Toabré Reservoir. Each pump will deliver water to the reservoir through a steel pipe that will be located in one of the existing Toabré Dam diversion tunnels. A 230-kV switchyard will be located near the station and a 65-km, 230-kV transmission line will connect with the national grid near Penonome. A plan of the pumping station at the toe of the Toabré Dam is shown on Exhibit 4.

Operation facilities are required for the pumping station, the spillway gates, and the other gates and valves. These facilities will include a SCADA system for remote monitoring and operation of the project, instrumentation, security and lighting, and landscaping and drainage.

COST OF THE PROJECT

The estimated cost of the Low Coclé del Norte Water Supply Project has been developed on the basis of the preliminary design and construction schedule. The estimates represent the prevailing rates and prices in January 2003. The estimates are based on the assumption that there will be no restriction on sources of supplies and equipment. The unit prices have been taken from the feasibility study of the Río Coclé del Norte Project acting in full regulation with the Caño Sucio and Río Indio Reservoirs (2, Volume 2).

A summary of the construction cost is shown below.

Item	Estimated Cost
Mitigation and Compensation Costs	**$ 23,000,000**
Construction Costs	
Access Roads and Construction Camp	$6,650,000
Low Coclé del Norte Storage Facilities	$50,960,000
Pump Station and Transmission Line	$87,290,000
Reservoir Clearing	$5,920,000
Subtotal Direct Cost	*$150,820,000*
Contingency	$27,080,000
Total Direct Cost	*$177,900,000*
Engineering and Administration	$27,100,000
Construction Cost (Jan 2003 price level)	**$205,000,000**
TOTAL COST	**$228,000,000**

The annual operating costs include the costs of operation and maintenance (O&M), for the various features, the cost of replacing short-life equipment, administration by the Owner, insurance, an annual cost associated with watershed management, implementation of the environmental mitigation plan and the relocation activities, and energy for pumping. The annual operation and maintenance costs are estimated to equal $3.1 million and the annual equivalent of the variable energy cost is about $5.3 million for a total of $8.4 million.

PROJECT IMPLEMENTATION

It is estimated that implementation of the Project will required about 9.5 years including environmental studies, funding, design, contractor selection, and construction alone will require 4.8 years. An implementation schedule is shown on Exhibit 5.

CONCLUSIONS AND RECOMMENDATIONS

As a result of the studies described in the foregoing sections, it is concluded that:

- There is no readily apparent fatal flaw to the Project.
- There are no features associated with the development that would cause it to be infeasible.
- A roller compacted concrete dam is an appropriate type of dam to provide an indication of the cost of the project.
- It is our considered opinion that there are no geologic or geotechnical problems associated with the sites that cannot be accommodated using conventional solutions although the lack of subsurface investigations has increased the potential for inaccuracies in the estimate of cost.

- The Project will supply up to 1,854 MCM/year into the Toabré Reservoir.
- Construction of the project is estimated to cost about $205 million in 2003 dollars. An addition $23 million have been allowed for compensation and mitigation for a total cost of $228 million.
- The Project will provide an economically attractive source of water for the Panama Canal system if the assumed pre-construction development takes place.

Based on these conclusions, it is recommended that the Project be investigated in further detail if additional water is needed for the Panama Canal System after construction of the Río Indio Project and either the Río Toabré Project or a project on the Río Caño Sucio.

TABLE OF SIGNIFICANT DATA

Project Setting

The Río Coclé del Norte Basin drains into the Atlantic Ocean and is located in the Districts of Donoso, Penonome, and Pintada about 90 km west of Panama City.

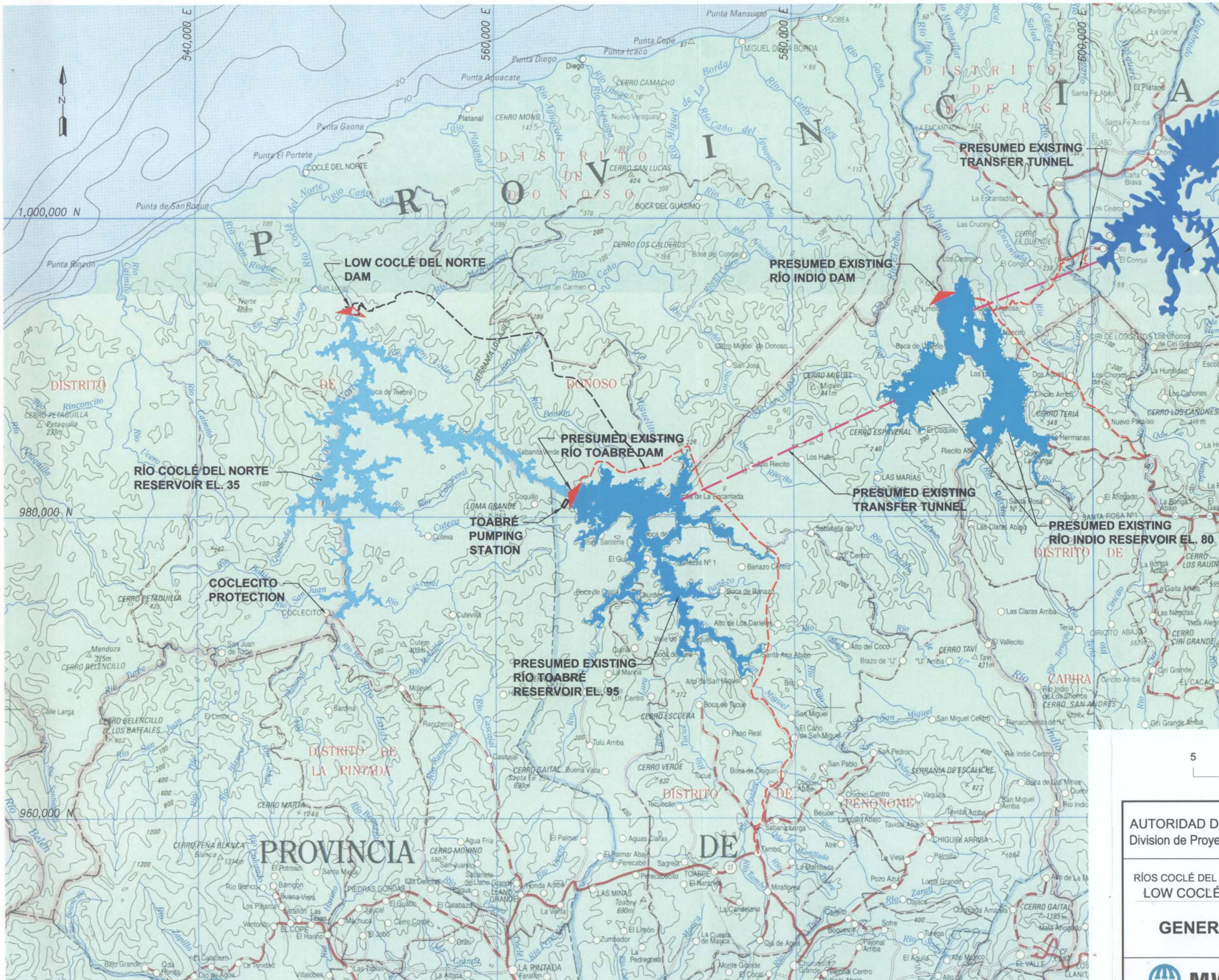
Hydrology		
Average Annual Precipitation	2,800	mm
Average Annual Streamflow (excluding drainage upstream of the proposed Toabré Dam)	69.4	m^3/s
Storage Facilities		
Reservoir		
Drainage Area	865	km^2
Normal Maximum Water Level	El. 35	msl
Volume	662	MCM
Surface Area	51	km^2
Dam		
Type of Dam	Roller compacted concrete	
Crest Elevation	41	msl
Minimum Foundation Elevation	-10	msl
Maximum Height	51	m
Fill Volume	180,000	m^3
Upstream and Downstream Slope	Vertical, 0.75H:1V	
Spillway		
Type of Spillway	Gated ogee	
Spillway Gates	Six 12 m high by 8 m wide	
Spillway Crest Length	63	m
Spillway Crest Elevation	El 23	m
Spillway Design Flood		
Peak Inflow	9,970	m^3/s
3-day Volume	950	MCM
Peak Outflow	6,520	m^3/s
Surcharged Reservoir Level	38.3	msl
Diversion During Construction		
Section Shape	7 m by 7 m concrete culvert	
Number of Culverts	2	
Length	230	m
Diversion Flood (25-year)	2,100	m^3/s
Discharge Capacity	770	m^3/s
Upstream Cofferdam Height (maximum)	20.5	m
Downstream Cofferdam Height (maximum)	9	m

TABLE OF SIGNIFICANT DATA, cont.

Storage Facilities, cont.		
Emergency Drawdown Facility		
Description	Located in one of diversion conduits. Flow controlled by two 3 m by 6 m gates	
Capacity at full supply level	5,000 m^3/s	
Minimum Release Facility		
Description	Valve in wall of diversion conduit	
Capacity	10.7 m^3/s	
Pumping Facilities		
Pumps		
Number of Units and Capacity	4-24 MW	4-13 MW
Design Flow	38.5 m^3/s	38.5 m^3/s
Total Dynamic Head	53 m	28 m
Rotational Speed	257 rpm	225 rpm
Estimated Project Cost		
Project Cost	$228,000,000	
Annual Cost	$8,400,000	
Estimated Project Schedule		
Implementation Period	9.5 Years	
Construction Period	4.8 Years	
Estimated Project Yield		
Average Pumping Capability	1,854 MCM/yr	

EXHIBITS

No.	Title
1	General Plan of Development
2	Low Coclé del Norte Dam, Plan
3	Low Coclé del Norte Project, Coclecito Protection Plan
4	Toabré Pumping Station, Plan and Elevation
5	Implementation Schedule



LEGEND:

- PRESUMED EXISTING PAVED ROAD
- PERMANENT ACCESS
- PRESUMED EXISTING DAM
- PROPOSED DAM
- PRESUMED EXISTING LAKE / RESERVOIR
- PROJECT LAKE / RESERVOIR

5 0 5 10 km

GRAPHIC SCALE

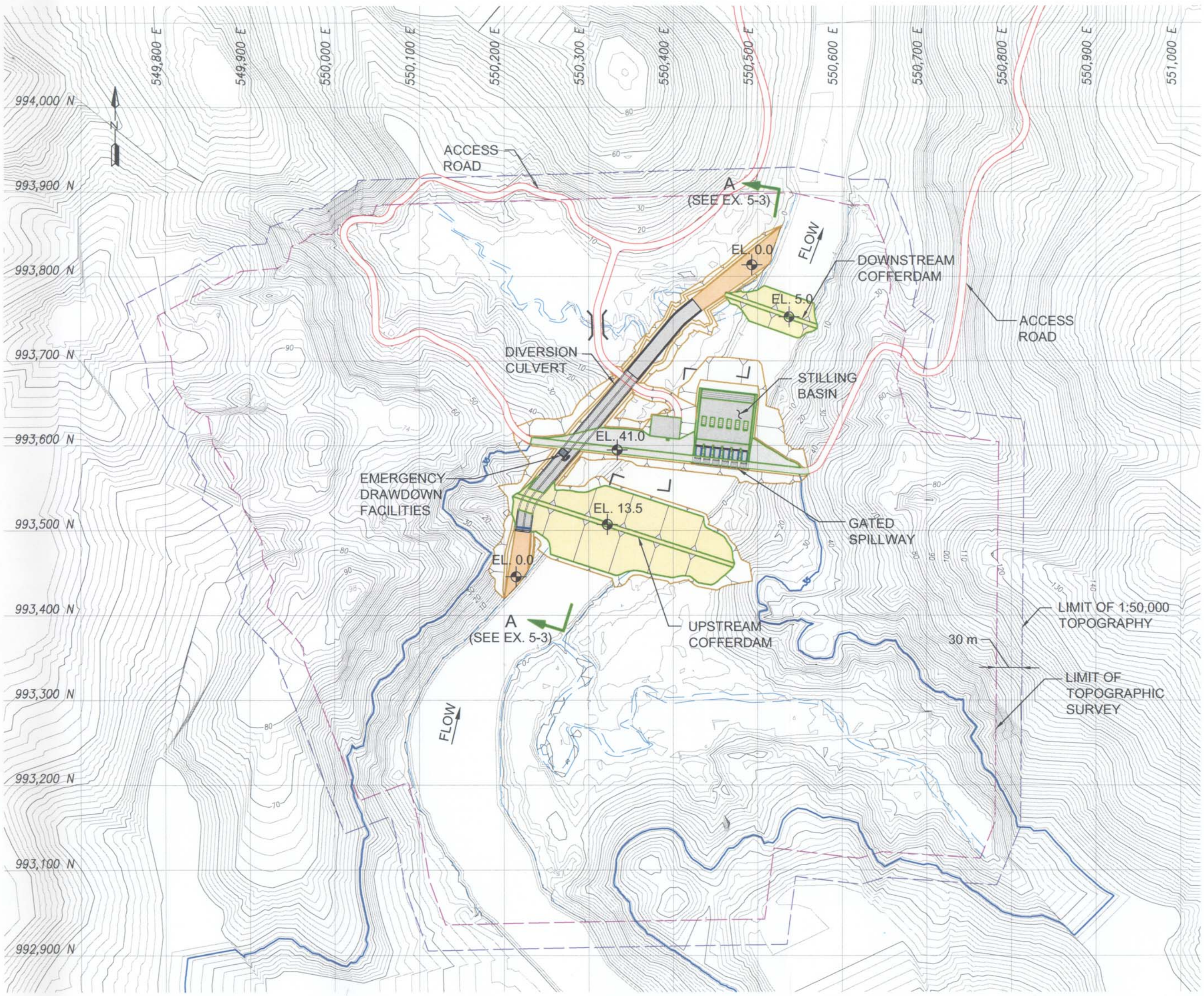
AUTORIDAD DEL CANAL DE PANAMA
Division de Proyectos de Capacidad del Canal

ACP

CONTRACT NO. CC-3-536
RÍOS COCLÉ DEL NORTE AND CAÑO SUCIO WATER SUPPLY PROJECTS
LOW COCLÉ DEL NORTE WATER SUPPLY PROJECT

GENERAL PLAN OF DEVELOPMENT

MWH	DATE: DECEMBER, 2003	EXHIBIT: 1



SITE PLAN

AUTORIDAD DEL CANAL DE PANAMA
Division de Proyectos de Capacidad del Canal

CONTRACT NO. CC-3-536
RÍOS COCLÉ DEL NORTE AND CAÑO SUCIO WATER SUPPLY PROJECTS
LOW COCLÉ DEL NORTE WATER SUPPLY PROJECT

LOW COCLÉ DEL NORTE DAM PLAN

MWH

DATE: DECEMBER, 2003

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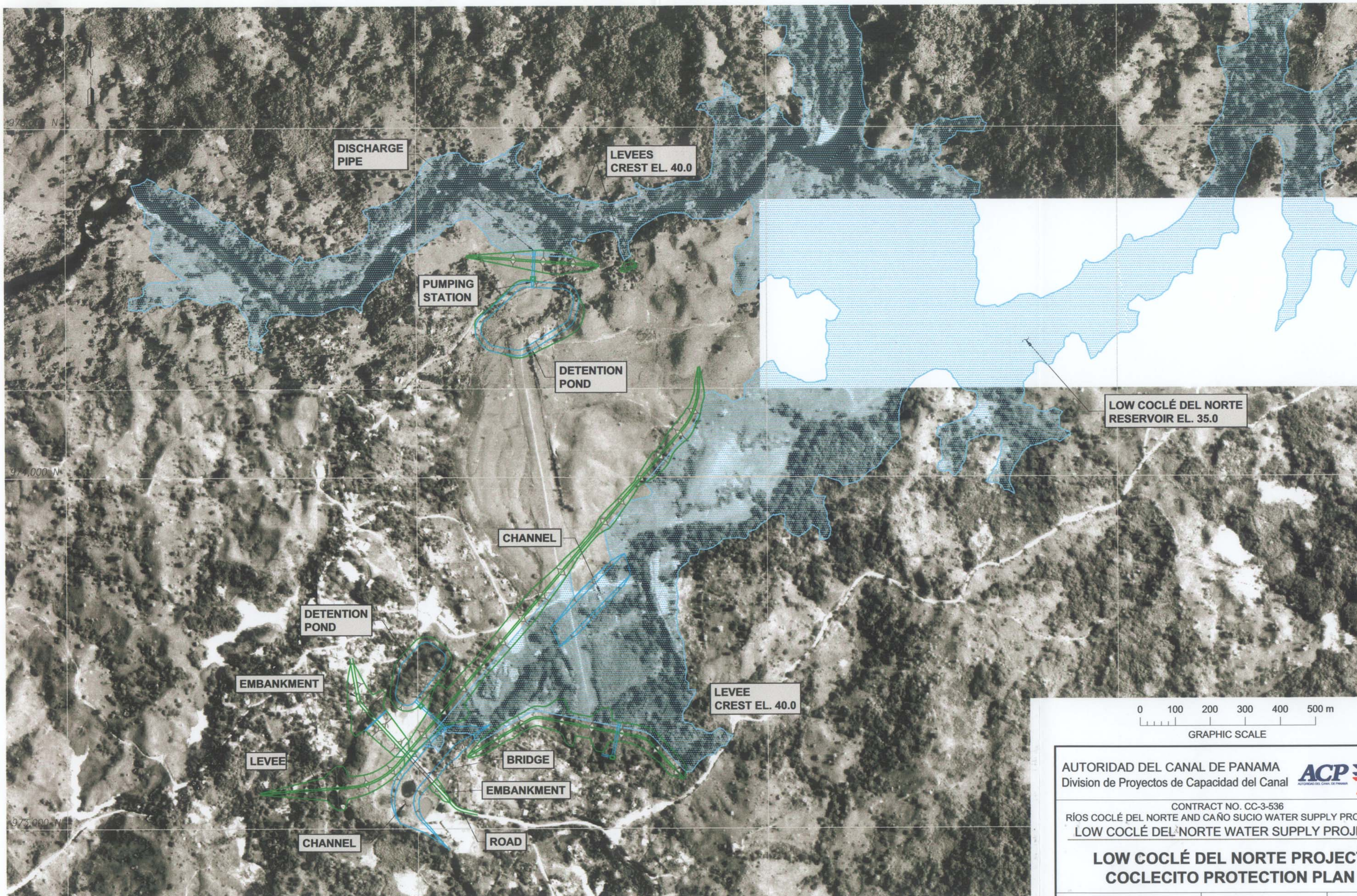


SITE PLAN

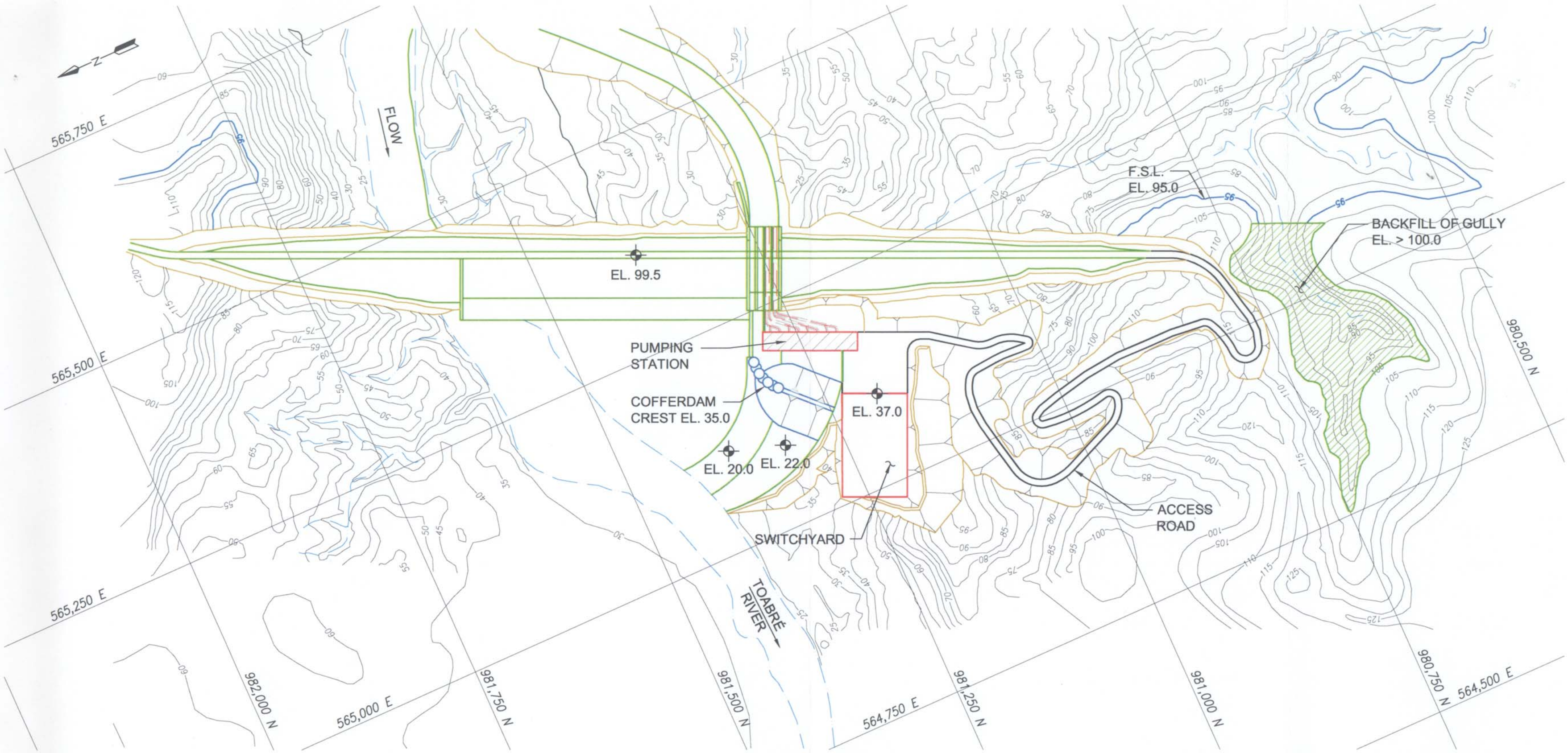
AUTORIDAD DEL CANAL DE PANAMA
División de Proyectos de Capacidad del Canal

CONTRACT NO. CC-3-536
RÍOS COCLÉ DEL NORTE AND CAÑO SUCIO WATER SUPPLY PROJECTS
LOW COCLÉ DEL NORTE WATER SUPPLY PROJECT

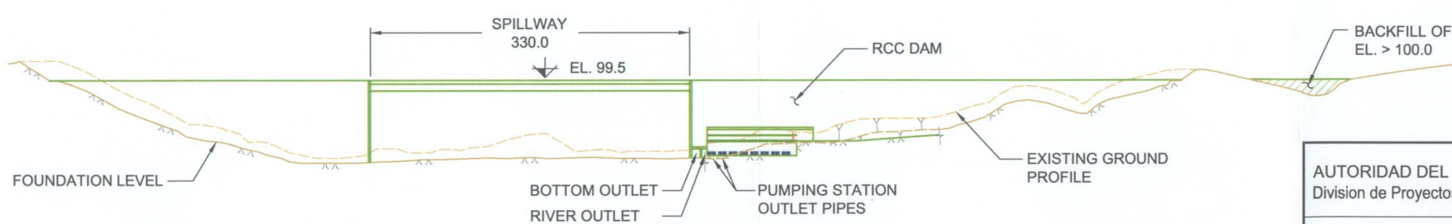
LOW COCLÉ DEL NORTE PROJECT
COCLECITO PROTECTION PLAN

MWH

DATE:	EXHIBIT:
DECEMBER, 2003	3



SITE PLAN



DOWNSTREAM ELEVATION

AUTORIDAD DEL CANAL DE PANAMA
División de Proyectos de Capacidad del Canal

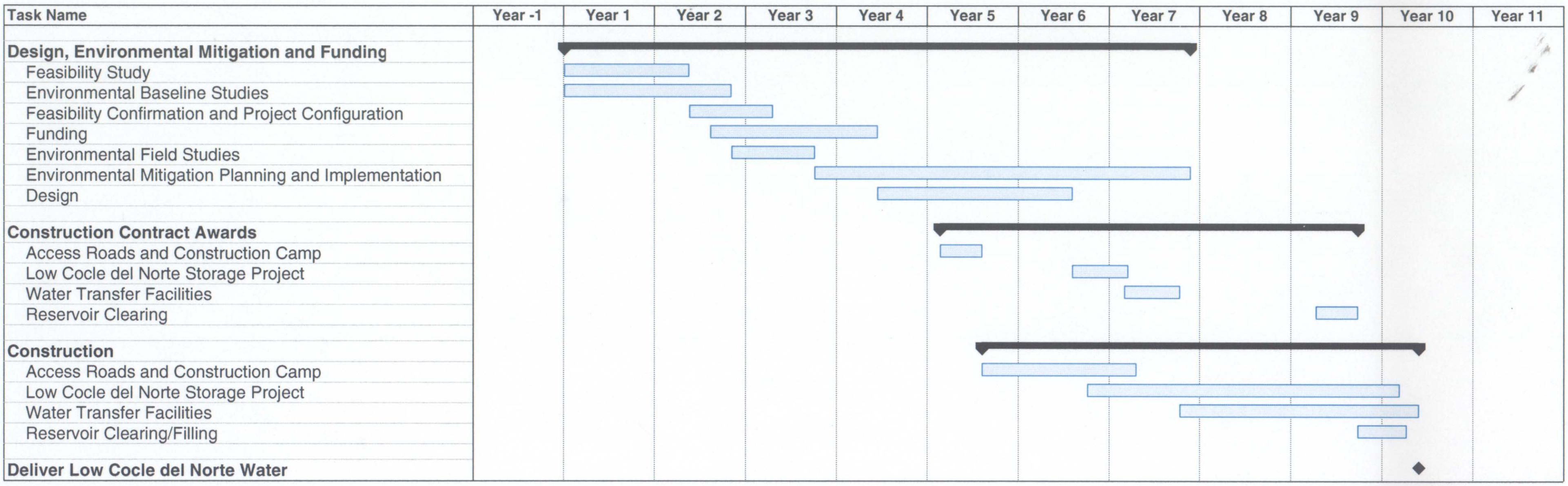
CONTRACT NO. CC-3-536
RÍOS COCLÉ DEL NORTE AND CAÑO SUCIO WATER SUPPLY PROJECTS
LOW COCLÉ DEL NORTE WATER SUPPLY PROJECT

TOABRÉ PUMPING STATION
PLAN AND ELEVATION

MWH	DATE: DECEMBER, 2003	EXHIBIT: 4

Low Cocle del Norte Water Supply Project

Implementation Schedule



AUTORIDAD DEL CANAL DE PANAMA
Division de Proyectos de Capacidad del Canal

CONTRACT NO. CC-3-536
RÍOS COCLÉ DEL NORTE AND CAÑO SUCIO WATER SUPPLY PROJECTS
LOW COCLÉ DEL NORTE WATER SUPPLY PROJECT

IMPLEMENTATION SCHEDULE

MWH	DATE: DECEMBER, 2003	EXHIBIT: 5

Main Report

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1. INTRODUCTION

1.1 Authorization

The Autoridad del Canal de Panama (ACP), formerly the Panama Canal Commission, has authorized Montgomery Watson.Harza (MWH), formerly Harza Engineering Company, to perform an engineering feasibility study of the Ríos Coclé del Norte and Caño Sucio Water Supply Projects (Project) under Contract CC-3-536, Work Order 0005, dated June 21, 2000. As a part of this study, MWH was asked to evaluate, on a preliminary basis, the option of developing the Río Coclé del Norte water resource with a low dam at the site identified in the Corps of Engineers' Reconnaissance Report (1).

1.2 Background

The full development of the Río Coclé del Norte basin requires a relatively high dam on the Río Coclé del Norte and the transfer of water to the Río Indio Reservoir, either directly by tunnel, or by diversion through the Río Caño Sucio Reservoir. This type of development will have a significant impact in the Río Coclé del Norte basin due to the need for the construction of a large reservoir. The ACP is interested in examining other options that have less impact. Two options have been identified:

1. The full development of the Río Miguel de la Borda and its tributaries, one of which is the Río Caño Sucio, and
2. A low dam on the Río Coclé del Norte and pumping facilities to deliver water to the Río Toabré Reservoir or to the Caño Sucio Reservoir. (The ACP has authorized studies of a project on the Río Toabré outside the scope of this contract.)

This report presents the results of studies associated with the second option.

The funds to perform this preliminary evaluation were reallocated from the original work order when it became evident that neither the refraction studies nor the economic studies would be performed for the large projects in the Río Coclé del Norte basin.

1.3 Objective

The objective of this study is to estimate the cost and yield of a project consisting of a low dam on the Río Coclé del Norte, pumping facilities to deliver the water to either the proposed Toabré or Caño Sucio Reservoirs, and protection of the town of Coclecito.

1.4 Scope of Services

Due to the limited funds available for the study, certain simplifying assumptions were made for the scope of services based on preliminary studies done during the preparation of the scope. These assumptions dealt with the location of the project and the approximate size of the development. In addition, to the extent possible, information collected and analyzed as a part of the Río Coclé del Norte feasibility studies (2) will be used and reported in the documentation for this study.

The project on the Río Coclé del Norte will consist of a dam with a full supply level at El. 35, located at the site identified in the Reconnaissance Study (1) and also used for the feasibility studies of the *Río Coclé del Norte acting in full regulation with the Río Caño Sucio and Río Indio Reservoirs* and the *Río Coclé del Norte acting in full regulation with the Río Indio Reservoir* (2). The quantity of water pumped into the canal system will be developed as a part of the study.

Hydrologic and geologic information for the project will be based, to the extent possible, on studies performed for the feasibility studies. No new geologic investigations will be performed. New hydrology analyses will be made for basin conditions that include a dam on the Río Toabré.

The specific tasks are as follows:

Task 1. Assess hydrologic conditions with a dam at the Río Toabré site (3) including:
  a. Long-term stream flow at the damsite
  b. Construction period floods
  c. Probable maximum flood
  d. Sediment yield and deposition at the dam

Task 2. Estimate the yield of the pumping operation.
Task 3. Develop features for a low dam and its appurtenant works.
Task 4. Investigate alternative pumping schemes.
Task 5. Develop protective measures for the town of Coclecito.
Task 6. Estimate the cost of project.
Task 7. Develop a construction schedule.
Task 8. Prepare a report to document the studies.

1.5 Acknowledgements

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- José Pascal, Water Projects Team Leader, Canal Capacity Projects Division, Autoridad del Canal de Panama;
- The Environmental and Safety Group;
- The Supporting Staff of the Canal Capacity Projects Division; and
- The Department of Meteorology and Hydrology, Autoridad del Canal de Panama.

2. PROJECT SETTING

The Low Coclé del Norte Water Supply Project consists of a small storage facility in the Río Coclé del Norte basin, pumping facilities to deliver water to either the (presumed existing) Río Toabré Reservoir or the Río Caño Sucio Reservoir, and protective measures for the town of Coclecito. It is located essentially in the middle of the Republic of Panama in the Districts of Donoso, Penonome, and La Pintada. A location map is presented on Exhibit 2-1.

2.1 Climate

The general climate of Panama is tropical with distinct wet and dry seasons induced by the movement of the inter-tropical convergence zone (ITCZ). When the ITCZ is located to the south of Panama, it causes a dry season in Panama. When it travels over Panama either moving northward or southward, its passage results in heavy rainfall; and when it is to the north, the strength of the rainy season decreases somewhat. This movement generally results in a dry season from January through April, a moderated wet season from May to mid-September, and a wet season for the rest of the year.

The average annual rainfall over the Río Coclé del Norte basin above the dam site is estimated to be 2,800 mm. A map of mean annual rainfall, taken from *Atlas Nacional de la República de Panamá* (4), is presented on Exhibit 2-2. The map shows that mean annual rainfall is higher in the coastal area and decreases inland.

Based on extended records for the Coclecito station, the mean monthly rainfall over the Río Coclé del Norte basin is estimated as the ratio of the basin annual rainfall and the station annual rainfall times the station monthly rainfall. Mean monthly rainfall values are shown in Table 2-1.

TABLE 2-1 MEAN MONTHLY RAINFALL, RÍO COCLÉ DEL NORTE BASIN (mm)

Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Annual
192	128	119	203	287	258	215	279	241	303	265	310	2,800

The mean monthly rainfall varies from a low of 119 mm in March to a high of 310 mm in December.

Mean monthly temperatures vary about 2° C throughout the year around 26° C near the dam to about 24° C in the head reach. The lowest temperature occurs in September and

October, and the highest occurs in March and April at lower altitudes and in June at higher altitudes.

2.2 Description of the Río Coclé del Norte Basin

The drainage configuration of the Río Coclé del Norte basin is presented on Exhibit 2-3. The Río Coclé del Norte is formed downstream from the confluence of the Río San Juan, Río Coclecito, and Río Cascajal near the town of Coclecito. The three rivers drain the northern slopes of the Cordillera Central (Continental Divide) and flow northward to the Atlantic Ocean. About halfway to the damsite, the Río Coclé del Norte is joined by the major basin tributary, the Río Toabré. At its mouth, the drainage area of the Río Coclé del Norte is about 1,730 km^2. The damsite is located about 15 km upstream from the Atlantic Ocean. Above the damsite, the river drains an area of about 1,600 km^2. The river basin is fan-shaped with a maximum length of about 58 km and a width of about 55 km.

The Río Toabré drains an area of about 810 km^2 at the confluence. The Toabré system is comprised of the main stem of the Río Toabré, the Río San Miguel and the Río de U on the right bank, and the Río Lura and Río Tulu on the left bank. The Río San Miguel is the major and longest tributary of the Río Toabré. It rises at about El. 900 and flows in a general northwesterly direction to join the Río Toabré. The slope of the Río San Miguel/Río Toabré is about 10 percent in the 4-km long head reach, decreases to 3.3 percent in the next 6 km, and gradually flattens to about 0.06 percent near the confluence. The Río de U is the closest connection to the Río Caño Sucio watershed.

The three rivers above Coclecito drain an area of about 520 km^2. The Río San Juan is the largest and longest of the three rivers. It drains an area of 270 km^2, and rises at an elevation of 1,300 meters above mean sea level (El. 1300). The river is very steep in the head reach, dropping about 900 meters in a distance of about 5 km (about 18 percent slope). The slope decreases downstream to about 6 percent in about 4 km. From Coclecito, the Río Coclé del Norte flattens to a slope of 0.3 percent to the damsite.

The Río Cuatro Calles is another right bank tributary, joining the Río Coclé del Norte about 2 km upstream from the dam site. The drainage area is about 140 km^2. Except for the most upstream 1,200 m, the river slope is about 0.3 percent.

The drainage areas of the basins are presented below:

Basin	Drainage Area (km^2)	Accumulated Drainage Area (km^2)
Río San Juan	270	
Río Coclecito	110	
Río Cascajal	140	
Area above Coclecito		*520*
Intervening local drainage	260	
Río Cuatro Calles	140	
Río Toabré	700	
Area above damsite		*1,600*
Intervening local drainage	130	
Total Basin Area		**1,730**

There is no access to the basin in the vicinity of the dam.

Slightly less than 40% of the basin is in forest. The remaining area has been deforested and is now comprised of shrubs, pasture, and annual crops. Within the inundated area, there are about 1,200 persons. Agriculture and cattle ranching are the main economic activities. There are also mineral and ore resources reported in the area.

2.3 Socio-Economic Conditions

The population of the Río Coclé del Norte basin was estimated in year 2000 to be about 26,000 persons. However, there were no indigenous populations reported in the area.

Agriculture and cattle ranching are the main economic activities, but per capita income falls below the extreme poverty level. Cattle ranching occupies about 22% of the Río Coclé del Norte basin and, based on a count of animals, low productivity and inefficiency is indicated. Basic crops are rice, corn, and beans, and production rates of these staples are considered to be low when compared to national averages. There are mining metallic concession contracts and requests in the basin according to the Ministry of Industry and Commerce, however no current mining activity.

The literacy rate is around 90 percent, although this is slightly lower than the national average. Most homes do not have access to potable water and over 90 percent of the homes do not have electricity. Except for the road into Coclecito, there is a lack of infrastructure. Medical services are more than an hour away for a high percentage of the population.

The six largest towns in the Río Coclé del Norte Reservoir are Coclecito (population 683), Villa de Carmen (495), Cutevilla (372), Valle de San Miguel (256), Boca de la Encantada (193), and Sabanita Verde (193). It is expected that most of the people residing in the basin live along the river. While major relocation is not expected as a result of the low dam, there will be some disruption to these communities.

According to the Reconnaissance Report (1) there are few public services. The town of Coclecito is the most developed and the largest of the towns. It has several schools and a hospital as well as access by road to La Pintada and Penonome. For these reasons, it has been decided to maintain the entire town in place by protecting in from encroachment with levees.

There is essentially no power except from small, local generating sources and telephone coverage is limited. All of the towns obtain water from the rivers or from groundwater. There is no treatment of community waste and most finds its way into the environment. As a result, there are known health problems such as hepatitis, dysentery, dermatitis, intestinal parasites, and respiratory illnesses associated with the waste disposal methods. A lack of good quality all-weather roads is probably one of the most pressing needs. The only roads are rarely graded and receive limited attention from the Ministry of Public Works or local government.

2.4 Agricultural Sector

The Río Coclé del Norte basin is largely undeveloped. The vegetation consists mostly of shrubs, forest, and pasture. Croplands are included within the shrub category, mostly because of the nature of the area under cultivation, the landscape position, and the size of the farm holdings. The distribution of vegetation is shown in Table 2-2:

TABLE 2-2 VEGETATION DISTRIBUTION

Habitat	Río Coclé del Norte Basin	
	Ha (rounded)	% of Total
Shrub	65,000	40
Forest	63,000	39
Pasture	35,000	21
Total	**163,000**	**100**

Farms and ranches of various sizes occupy approximately 60% of the land in the project area. Farm crops include manioc, maize, rice, beans, sugar, coffee, and tobacco. Ranchers raise cows, horses, chickens, and hogs. Most of the farmers and ranchers are small commercial enterprises although there is also some cash crop and subsistence farming (1).

2.5 Geologic Setting

A regional geologic map is presented as Exhibits 2-4.

2.5.1 Regional Geology

Bedrock in the region of the proposed Río Coclé del Norte project consists mostly of volcanic igneous rocks belonging to the Tucue Formation. These include basic and intermediate (basaltic and andesitic) lava flows, breccias, tuffs, and agglomerates. Reportedly, other rock types are intrusive igneous rocks classified as granodiorites, quartz monzonites, gabrodiorites, diorites, or dacites.

Although little information on the engineering characteristics of these rocks exists, it is anticipated that they may exhibit a wide variety in quality (ranging from high quality intrusive rocks and extrusive lava flows to weathered and lesser quality volcanic tuffs and epiclastics).

2.5.2 Regional Tectonics

The tectonics in the Central American region are described in Appendix B to the feasibility studies for Río Coclé del Norte (2), and are not repeated here.

2.6 Environmental Setting

The information presented has been supplied by the ACP or was taken from the Reconnaissance Report (1).

2.6.1 Terrestrial Habitat

Forests cover about 39 % of the land in the basin. The remaining areas are categorized as shrub and pasture.

2.6.2 Fish and Wildlife

The Río Coclé del Norte is typical of a river in mountainous country. The water is clear and cool and the river bottom ranges from sand to boulders with numerous riffles, rapids, and pools. The river and its tributaries support some fish and benthic communities. Currently, no fish species information is available. The snail *Melanoides tuburculata* is present and is considered a vector of some parasites. *Corbicula fluminea* (clam) are abundant in the Río Coclé del Norte.

The biological diversity of the Río Coclé del Norte basin includes 317 species of birds, 71 species of amphibians, 46 species of reptiles, and 75 species of mammals. The endangered species include 143 species of birds, 32 species of amphibians, 22 species of reptiles, and 35 species of mammals or about one-half of all species.

2.6.3 Wetlands

According to the Reconnaissance Report (1), the wetlands in the project area consist of forested riparian habitat along the immediate stream bank area. The width of the riparian habitat within the impoundments varies from 10 m to 75 m. About 80% to 90% of the streams above the dam site are bordered by forested riparian habitat.

2.6.4 Air Quality

Air quality in the project area is generally good except during the slash and burn period. At the end of the dry season, during March and early April, sizable areas of forest and secondary growth are burned and cleared to prepare the land for agricultural use.

2.6.5 Cultural and Historic Resources

In the Río Coclé del Norte basin, below El. 80, there are about 70 reported archaeological sites. These sites are located and consist of indigenous villages, hamlets, funeral sites, mines, and miscellaneous sites. It is likely that many sites are located within the proposed impoundment (full supply level at El. 35).

3. PROJECT DEFINITION STUDIES

The project definition studies are the basic studies that support the selection of the project facilities and the final project arrangement. These studies include topography, hydrology, geology and geotechnical investigations, and project operation studies.

3.1 Topography

Ingenieria Avanzada, S.A. prepared topographic mapping of the proposed dam site area under subcontract to MWH as a part of the feasibility studies. The services were completed and submitted to the ACP under Contract CC-3-536, Task Order 2. The extent of the topographic coverage is shown on the plan views of the proposed project facilities.

3.2 Hydrology Studies

Hydrologic analyses were performed to estimate reservoir evaporation, the long-term streamflow sequence, construction-period floods, the spillway design flood, and sediment deposition in the reservoir. These studies were done to a feasibility level and their results are presented in more detail in Appendix A, Hydrology and River Hydraulics.

To analyze the options for pumping water to the System Reservoirs, however they may be configured at the time of development, it is necessary to have two estimates of hydrologic information. The first, for the entire Coclé del Norte drainage, was done as a part of the feasibility-level studies (2). The second, which excludes the drainage behind Toabré Dam, was also done at feasibility level for this study.

3.2.1 Net Reservoir Evaporation

Net reservoir evaporation is estimated to total 1,134 mm/year and is based on historic reservoir evaporation data from Gatun Lake over the period 1993 to 1998. The estimate, developed by the ACP, was judged to be reasonable and was used in this study. The monthly distribution of net reservoir evaporation is presented in Table 3-1.

TABLE 3-1 MEAN MONTHLY NET RESERVOIR EVAPORATION (MM)

Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Annual
112	117	133	123	91	80	84	80	78	80	72	84	1,134

3.2.2 Long-Term Streamflow at the Río Coclé del Norte Damsite

The long-term flow sequence was developed by the APC and reviewed by MWH. There are three gages located in the Río Coclé del Norte basin. Two stations, Batatilla and El Torno were used with three other gages located outside the basin to derive the long-term streamflow. The location of these gages is shown on Exhibit 3-1 and pertinent data for each gage is presented in Table 3-2.

TABLE 3-2 STREAM GAGES USED IN ANALYSIS OF LONG TERM FLOW RECORD

Station	Watershed	Location	Period of Record	Drainage Area
Río Coclé del Norte at Canoas	Río Coclé del Norte	Half-way between Coclecito and the Río Toabré	1983-date	571 km^2
Río Coclé del Norte at El Torno	Río Coclé del Norte	Slightly upstream from Río Toabré confluence	1958-1986	672 km^2
Río Toabré at Batatilla	Río Coclé del Norte	5 km upstream from the mouth	1958-date	788 km^2
Río Trinidad at El Chorro	Gatun	3 km upstream from Lake Gatun	1948-date	172 km^2
Río Ciri Grande at Los Canones	Gatun	9 km upstream from Lake Gatun	1948-date	186 km^2
Boca de Uracillo	Río Indio	5 km upstream from Río Indio damsite	1979-date	376 km^2

The long-term flow sequence (1948-1999) at the damsite was generated by adding the data from El Torno and Batatilla, adjusted for drainage area and rainfall. The flows at El Torno and Batatilla were completed using correlations with El Chorro, Los Canones and Boca de Uracillo. MWH reviewed the correlation and double mass curve analyses and determined that the approach is logical and results are acceptable.

The mean annual flow at the Río Coclé del Norte damsite is estimated to be 107.5 m^3/s and the monthly distribution of flow is shown in Table 3-3. The complete monthly flow at the damsite is presented in Table 1 (note tables numbered without the section number are located at the end of the report text).

TABLE 3-3 MONTHLY MEAN STREAMFLOW AT THE RÍO COCLÉ DEL NORTE DAMSITE (m^3/s)

Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Annual
89.5	52.1	37.7	53.5	91.8	108.8	107.7	125.3	139.2	162.7	173.0	149.2	107.5

The mean annual flow for the basin excluding the Toabré Dam drainage was estimated using a ratio of drainage areas for the damsite and El Torno and the completed streamflow data for El Torno. The mean annual flow at the Río Coclé del Norte damsite excluding the Toabré Dam drainage is estimated to be 69.4 m^3/s and the monthly distribution of flow is shown in Table 3-4. The complete monthly flow at the damsite is presented in Table 2.

TABLE 3-4 MONTHLY MEAN STREAMFLOW AT THE RÍO COCLÉ DEL NORTE DAMSITE EXCLUDING THE TOABRÉ DAM DRAINAGE (m^3/s)

Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Annual
60.4	39.4	29.5	41.2	65.1	69.4	67.3	80.0	85.9	96.5	102.8	95.6	69.4

Mass curve and time series analyses indicate that the annual flows are consistent, and homogeneous. There are no apparent trends for the basin data; however, there apparently is a downward trend for the data excluding the Toabré Dam drainage.

On an annual basis, the lowest 1, 2, 3, and 4 calendar year flow sequences occurred in 1997, 1976-77, 1976-1978, and 1976-79, respectively. The 2, 3, and 4-year flows over the period including 1997 and 1998 were very close. The average runoff in these periods amounted to 67%, 72%, 82%, and 85% of normal, respectively. For reference, flows at the dam site exceeded 90% and 95% of the time are estimated to be 26.1 m^3/s and 19.3 m^3/s for the entire basin and 20.8 m^3/s and 15.0 m^3/s for the basin excluding the Toabré Dam drainage.

3.2.3 Construction Period Floods

Construction period floods at the Coclé del Norte damsite were evaluated using both available regional flood frequency data and annual maximum instantaneous flood peaks at representative stations. Use of the site-specific instantaneous peak data was considered more appropriate than using the regional analysis.

Instantaneous flood peak data are available at the three gages in the Río Coclé del Norte basin, at Canoas, at El Torno, and on the Río Toabré at Batatilla. Both Canoas and El Torno measure essentially the same contributing watershed and, since the record at El Torno is longer, the flood peak data for El Torno was used to represent the Río Coclé del Norte basin above Río Toabré. The Generalized Extreme Value (GEV) distribution was adopted to estimate the relationship between return period and flood peak for the two stations.

A general procedure for transposition of flood peaks from a gaged location to an ungaged location is to use coefficients of empirical relationships assuming that these coefficients remain constant for hydrologically and meteorologically similar drainage basins.

The flood peaks at the dam site are presented in Table 3-5.

TABLE 3-5 FLOOD PEAKS AT THE DAM SITE

Return Period (years)	Entire Basin		Excluding Toabré Dam Drainage	
	Flood Peak (m^3/s)	Dry Period Flood Peak (m^3/s)	Flood Peak (m^3/s)	Dry Period Flood Peak (m^3/s)
5	1,925	288	1,379	299
10	2,430	458	1,701	445
20	2,995	697	2,026	639
25	3,050	737	2,100	779
50	3,860	1,171	2,528	1,007
100	4,610	1,705	2,976	1,379

3.2.4 Spillway Design Flood

The probable maximum flood (PMF), based on probable maximum precipitation (PMP), was adopted as the spillway design flood.

3.2.4.1 Probable Maximum Precipitation

Three procedures were used to evaluate and select the PMP.

1. The first consisted of transposing the most severe storms listed in the 1965 US Weather Bureau (WB1965) and 1978 National Weather Service (1978NWS) reports (5,6). These reports covered storms up to 1976.

2. The second procedure was to develop and evaluate storm isohyetal patterns of major storms that occurred over the basins since 1976, and
3. The third was to use the PMP estimates and depth-area duration curves developed as a part of the NWS1978 and WB1965 reports.

The PMP was based on the 24-hour, 10-mi^2 PMP estimates, and depth-area duration curves developed as a part of the NWS1978 (5) and WB1965 (6) reports. This is the same procedure that was selected for the Río Coclé del Norte feasibility level studies.

A storm duration of 48 hours was adopted for the PMF based on the size of the basin. For the entire basin and for the basin excluding the Toabré Dam drainage, the 48-hour average PMP is estimated to be 714 mm and 761 mm, respectively.

3.2.4.2 Probable Maximum Flood

The PMP was transformed to a PMF using the HEC-1 computer model. A one-hour duration of PMP increments was selected based on the lag time of the basin above the damsite. PMP was distributed into one-hour increments using the depth duration curve developed for the Río Coclé del Norte and Caño Sucio Feasibility Study. The one-hour increments were arranged sequentially using the "alternating block method (7), adjusted for losses, and applied to a unit hydrograph.

A unit hydrograph was developed for both basin conditions using Clark's method (8). The watershed parameters, length of mainstream, overall stream slope, and time-area histogram were calculated from the topographic maps of 1:50,000 scale.

A uniform infiltration rate of 3 mm per hour was used. No initial loss was used because antecedent rainfall would saturate the soil moisture. No infiltration loss was considered for the reservoir area at maximum operating pool level.

For the entire basin, the probable maximum flood hydrograph would have a peak of 10,000 m^3/s and a 5-day volume of about 950 MCM. For the basin excluding the Toabré Dam drainage, the probable maximum flood hydrograph, including the routed outflow from Toabré Dam, would have a peak of 9,970 m^3/s and a 3-day volume of about 950 MCM. The PMF hydrographs are shown on Exhibits 3-2 and 3-3 for the entire basin and for the basin excluding the Toabré Dam drainage respectively.

3.2.5 Reservoir Sedimentation

The analysis of reservoir sedimentation for the Río Indio and Río Coclé del Norte studies resulted in an assumption of a unit yield of 1.4 mm/year. One this basis, after 100 years, the sediment deposition at the Low Coclé del Norte dam will have reached to about El. 6

and El. 4 for sediment contributions from the entire basin and from the basin excluding the Toabré Dam drainage respectively. The storage behind the dam will be reduced by 30% and 15% respectively after 100 years.

3.3 Engineering Geology

The investigation program conducted for the original feasibility studies and used for this prefeasibility study incorporated the following activities:

- Reconnaissance of dam and powerhouse sites; establish exploration program and investigation requirements,
- Reconnaissance geologic mapping, including geomorphological analysis and photo-geologic studies,
- Outcrop geologic mapping at the dam site,
- Construction materials investigation,
- Identification of principal geologic factors governing alternative tunnel routes,
- Development of preliminary geologic and geotechnical criteria for use in the selection of recommended project concepts and features/structures,
- Seismic hazard assessment of project region,
- Laboratory testing and analyses of test pit samples, and
- Development of geologic and geotechnical parameters for use in design of selected project and estimation of construction costs.

Reconnaissance geologic mapping was performed along the Río Coclé del Norte from the reservoir area to immediately downstream of the dam site. A general reconnaissance of some of the proposed reservoir area (up to the confluence with the Río Toabré) was performed by helicopter to identify and evaluate any geologic features relevant to reservoir rim stability and watertightness. Available aerial photographic coverage was obtained from *Instituto Geográfico Nacional.* Conventional photogeologic methods were followed using a mirror stereoscope and photo-comparator.

Samples of rock and soil samples from test pits were collected for subsequent laboratory testing and analysis through the services of Tecnilab S.A. in Panama City. The laboratory testing was performed in order to establish their potential use as construction materials. Tests performed include gradation, specific gravity, absorption, soundness, and abrasion resistance. In addition, preliminary petrologic determinations were made from hand samples collected during geologic mapping.

3.3.1 Geology of the Damsite

In general, the foundation bedrock at the site is not expected to present any significant constraints on project development that cannot be taken care of with appropriate conventional design details and construction practices. In regard to other geological aspects, there do not appear to be any strongly adverse conditions or fatal flaws at the site.

3.3.2 Geotechnical Design Parameters

Geotechnical design parameters and criteria used for developing project layouts and cost estimates are presented in Table 3-6.

TABLE 3-6 SUMMARY OF GEOTECHNICAL DESIGN PARAMETERS FOR RÍO COCLÉ DEL NORTE DAM

Parameter	Selected Design Criteria
Thickness of overburden (top of weathered rock)	3 m
Depth to top of competent work	6 m
Excavation depth under main dam body	6 m
Excavation depth under spillway headworks	6 m
Rock Excavation Slopes	1H:5V, 3-m-wide benches every 10 m vertically
Soil Excavation Slopes, Permanent	2H:1V, 3-m-wide benches every 10 m vertically. Bench at soil-rock contact
Soil Excavation Slopes, Temporary	1.5H:1V, 3-m-wide benches every 10 m vertically. Bench at soil-rock contact

3.3.3 Foundation Treatment

Dental excavation and concrete will be used to treat local zones of highly weathered, sheared, or otherwise unacceptable rock encountered in the foundation. Required dental treatment should be nominal and only local. Consolidation grouting is not envisaged except in limited areas (e.g. fault or fracture zones) should they become exposed during excavation.

Curtain grouting will be used to reduce seepage through joints and fractures under the dam and in the abutments. For estimating purposes, a single row, staged grout curtain constructed by the split-spacing method is assumed.

3.3.4 Diversion and Cofferdams

River diversion during construction will be accomplished through a culvert located under the dam at the left abutment in a trench excavated in bedrock. This bedrock should be of moderate strength and unweathered over most of the culvert length. The upstream and downstream cofferdams will be founded on weathered rock and the natural ground surface, respectively.

3.3.5 Geology at the Pumping Station

As reported in the Executive Summary of the Toabré Dam Feasibility Study (3), either an RCC dam or a CFRD are viable alternatives for the site. If this assessment is accepted, then the location of a pumping station at the toe of the dam should not be a problem.

3.3.6 Seismicity

Several major historical earthquakes have occurred in the study region. Most notably, earthquakes occurred in 1822 and 1916 in Northwest Panama along the border of the North Panama Deformed Belt, while two earthquakes occurred nearly 25 km off the northern coast near Colon in 1621 and 1882. An additional earthquake event is noted in 1914 on the northeastern coast in the San Blas region. As a part of the feasibility studies for the projects in the Western Watershed, the Global Hypocenter Database prepared by the U.S. Geological Survey/National Earthquake Information Center (USGS/NEIC) of Denver, CO, was used to search for all historical (non-instrumented) and modern (instrumented) seismicity data within the region bounded by latitudes 5°N and 11°N and longitudes 75°W and 85°W. The database contains over 900,000 earthquakes from 2100 B.C. through 2002 and draws on information from 53 separate regional and worldwide catalogs. Within the defined region, nearly 2,150 earthquakes were identified. A map of the location and magnitude of many of these earthquakes is shown on Exhibit 3-4.

Feasibility–level estimates were made of the maximum design earthquake (MDE) and the operating basis earthquake (OBE) for the Río Coclé del Norte Project The recommended seismic design parameters for the project are 0.27 for the MDE and 0.14 for the OBE.

3.3.7 Construction Materials

As most of the facilities are comprised of concrete or roller compacted concrete, the required construction material is concrete aggregate.

All aggregates (including coarse and fine aggregates for concrete, saddle dam filters, drains, and riprap) need to be manufactured from quarried sources. Coarse and fine aggregates for concrete will be processed from quarried igneous rock materials, i.e. basalt or andesite. Aggregates for filters and drains will be obtained by processing of the same quarry sources as exploited for concrete aggregates. The area three to five kilometers east of the damsite could be stripped and opened as a quarry. The location of these sources is shown on Exhibit 2-4. Quarries identified for the construction of the Toabré dam offer an alternative material sources for construction of the pumping station.

3.4 Dam Site and Dam Type Selection Studies

The dam site selected for the feasibility studies in the Coclé del Norte basin was adopted because no better site was identified and feasibility-level mapping was available. The selected site can regulate almost the entire water resource of the basin.

A roller compacted concrete (RCC) dam with a gated spillway was selected to minimize cost and the maximum water surface elevation and, therefore, the impact in the basin. Although no subsurface investigations were performed and, in earlier studies, this was a basis for not selecting a high RCC dam, foundation conditions at the site for a low dam were determined to be acceptable for this type of dam.

3.5 Reservoir Operation Level and Yield

3.5.1 Area/Volume Relationship

Area and volume data versus elevation were developed for reservoir based on the 1:50,000 scale maps. The "zero" area and volume point was determined from a terrestrial survey. A curve of area and volume versus elevation is shown on Exhibit 3-5.

3.5.2 Reservoir Operating Level

Three reservoir operating levels were considered: El.30, El. 35 and El. 40. A reservoir at El. 40 would require a substantial and intrusive protection scheme for Coclecito and would impact several additional towns compared with lower reservoir levels. The reservoir at El. 30 required more pumping against a wider range of head conditions as well as additional work to connect the reservoir to Toabré or a location suitable for pumping to Caño Sucio. The reservoir operating level was selected subjectively at El. 35. This level provides sufficient storage for daily regulation of inflow, a reservoir that extends to the Toabré site to minimize pumping requirements, and permits a relatively inexpensive protection scheme for Coclecito.

3.5.3 Project Yield

The project pumping facilities were sized to deliver all flows up to the flow equivalent to the 10% probability of exceedance, adjusted for mandatory releases and evaporation. For Alternative 1, which assumes that a dam exists at the Toabré site, all flows up to 100 m^3/s will be pumped. For Alternative 2, which assumes that there are no other storage facilities in the Coclé del Norte basin, all adjusted flow up to 170 m^3/s will be pumped into the system.

Under these two conditions, the estimated average annual water supply for the four scenarios is presented in Table 3-7.

TABLE 3-7 AVERAGE ANNUAL WATER SUPPLY

Pumping Alternative	Average Annual Water Supply
Alternative 1	
Scenario 1	58.8 m^3/s
Scenario 2	58.8 m^3/s
Scenario 3	24.2 m^3/s
Alternative 2	90.4 m^3/s

The estimated average annual water supply is not a system yield. To estimate the impact on the system yield the monthly pumped flows would have to be input to the HEC-5 system studies. It is expected that the yield of the system would increase by essentially the same amount; however, this would probably require modification to the operating rules and perhaps the addition of some transfer facilities.

4. PROJECT CONFIGURATION STUDIES

These project configuration studies considered the pumping scheme, the spillway and diversion flood, the spillway size, and the diversion arrangement. The studies are summarized below.

4.1 Pumping Scheme

Two alternatives were considered to pump water from the Low Coclé Del Norte reservoir to the Panama Canal system:

- Alternative 1 - Pumping water from Low Coclé Del Norte Reservoir to Toabré Reservoir, and
- Alternative 2 - Pumping water from Low Coclé Del Norte Reservoir to Caño Sucio Reservoir.

Alternative 1 is analyzed using the mean annual flow for the Coclé del Norte basin excluding the drainage above Toabré Dam. Alternative 2 uses the entire basin flow.

For Alternative 1, the following three scenarios were considered:

- Scenario 1 - Pumping against variable reservoir elevations with energy requirements supplied from the national electric grid,
- Scenario 2 - Pumping against a constant elevation over the top of the Toabré dam with energy requirements supplied from the national electric grid, and
- Scenario 3 - Pumping against variable reservoir elevations with energy supplied by a hydroelectric power station at the Low Coclé Del Norte dam.

The studies are described in detail in Appendix B and are summarized below.

The design flow for the pumping stations was selected based on the criterion of 10% probability of exceedance of the monthly average flows for the period 1948-1999 at the dam site. A duration curve of flows available for pumping was derived based on the average monthly flows, flow releases, and evaporation losses. For Alternative 1, the flow data for the basin excluding the Toabré Dam drainage was used (Table 2). For Alternative 2, flow data for the entire basin was used (Table 1). The selected design flows for the pumping station for Alternatives 1 and 2 are shown below:

SELECTED DESIGN FLOWS FOR PUMPING STATIONS – ALTERNATIVES 1 AND 2

Description	Alternative 1 Scenarios 1 and 2	Alternative 2
Selected design flow for pumping station	100 m^3/s	170 m^3/s

For Scenario 3 of Alternative 1 the design flows for the pumping station and hydropower station were selected to balance energy generation with energy required for pumping water into the Toabré dam. The selected design flows are shown below:

SELECTED FLOWS FOR PUMPING AND HYDROPOWER STATIONS - SCENARIO 3 OF ALTERNATIVE 1

Description	Alternative 1 Scenario 3
Selected design flow for pumping station	37 m^3/s
Selected design flow for hydropower station	80 m^3/s

It was assumed that 4 equal-sized pumps would be used for Alternative 1, Scenario 2 and Alternative 2 and two different sized sets of 4 pumps for Scenarios 1 and 3 of Alternative 1 to handle the wide range of head conditions.

A project concept was developed for each alternative and scenario. Estimates of cost were developed on a preliminary basis and the various concepts were compared on the basis of an economic cost of water. Costs were estimated on a parametric basis at a 2003 price level. Energy costs were estimated at $0.07/kWh based on information available in Panama. Operation and maintenance of the pumping schemes was estimated at 2% of the construction cost. The economic cost of water is based on an assumed demand schedule that assumes that the year 2000 demand is 38 lockages per day (L/d) and that the demand increases at a rate of 0.75 L/d/year. This is consistent with the assumptions for the feasibility-level studies of project in the Western Watershed. It is assumed that the pumping scheme comes on line in the first year of deficit and the economic cost is the present value of costs divided by the discounted stream of water supply.

4.1.1 Alternative 1, Scenario 1 Pumping into Toabré Reservoir

Scenario 1 includes a pumping station located at the toe of Toabré Dam and steel pipes to discharge water into the Toabré Reservoir, and a 50-km long, 230 kV, single circuit transmission line connecting to the national grid[1]. These discharge pipes would be about 150 m long and installed in the Toabré Dam temporary diversion facilities. The scenario requires two sets of 4 pumps due to the large operating range. The larger pumps would be rated at 23,900 kW and the smaller set at about 12,600 kW. The scenario would deliver an average of 59 m^3/s and require an energy input of about 301 GWh/year. The civil and equipment costs are estimated to be about $100 million.

4.1.2 Alternative 1, Scenario 2 Pumping over Toabré Dam

Scenario 2 includes a pumping station located at the toe of Toabré Dam and steel pipes to discharge water into the Toabré Reservoir over the top of the dam, and a 50-km long, 230 kV, single circuit transmission line connecting to the national grid[2]. The discharge pipes would be about 150 m long and installed on the downstream slope of the RCC dam and over the crest. An energy dissipation structure would be required on the upstream face of the temporary diversion facilities. The scenario requires a set of 4 pumps rated at about 20,000 kW. The scenario would deliver an average of 59 m^3/s and require an energy input of about 441 GWh/year. The civil and equipment costs are estimated to be about $53 million.

4.1.3 Alternative 1, Scenario 3 Balanced Pumping into Toabré Reservoir

Scenario 3 includes a hydroelectric station at the toe of Coclé del Norte Dam, a pumping station located at the toe of Toabré Dam, steel pipes to discharge water into the Toabré Reservoir, and a 21-km long, 115 kV, single circuit transmission line between the hydroelectric plant and the pumping plant. The distribution of water between the hydro plant and the pumping plant was designed to balance the available and required energy. The hydro plant would contain 4 equal-sized units with an aggregate capacity of 25 MW. The pumping station will contain two sets of 4 pumps due to the large operating range. The larger pumps would be rated at 8,900 kW and the smaller set at about 4,700 kW. The scenario would deliver an average of 25 m^3/s and require no energy input from the national grid. The civil and equipment costs are estimated to be about $75 million.

4.1.4 Alternative 2 Pumping into Caño Sucio Reservoir

Alternative 2 includes a pumping station located on the Low Coclé del Norte Reservoir near to the Caño Sucio Reservoir, a 3.6-km long, concrete lined tunnel, an underground pumping station, a shaft and steel pipes connecting the pumping station to the Caño Sucio Reservoir, safety valves on the pump discharge line and an energy dissipation structure in

[1] Note that the final transmission line length is 65 km. This change in length would have no impact on the selection of an alternative pumping scheme.

[2] Ibid.

Caño Sucio Reservoir, and a 50-km long, 230 kV, single circuit transmission line connecting to the national grid[1]. The scenario requires one set of 4 pumps rated at 36,800 kW. The scenario would deliver an average of 90 m^3/s and require an energy input of about 756 GWh/year. The civil and equipment costs are estimated to be about $100 million.

4.1.5 Evaluation of Alternatives

In addition to the costs of the pumping facilities identified for each scenario, a cost has been added for Low Coclé del Norte Dam, the protection facilities at Coclecito, and for compensation and mitigation of the environmental impacts. The economic costs of water, estimated as described above, are shown in Table 4-1.

TABLE 4-1 EVALUATION OF ALTERNATIVE PUMPING SCENARIOS

Item	Alternative 1			Alternative 2
	Scenario 1	Scenario 2	Scenario 3	
Construction Cost (million)	$213.9	160.3	$185.6	$215.3
Annual O&M Cost (million)	$1.97	$1.19	$1.56	$1.99
Unit Energy Cost	$0.011/m^3$	$0.017/m^3$	-0-	$0.019/m^3$
Average Supply, MCM/yr	1,854	1,854	762	2,852
Economic Cost of Water	$0.08/m^3$	$0.07/m^3$	$0.07/m^3$	$0.09/m^3$

Within the limits of estimated costs and the assumptions for future demand, all cases are considered to have about the same economic cost of water.

In the scope of services, the arrangement delivering water to Caño Sucio Reservoir was included to determine if it had any cost advantage. From Table 4-1, it is evident that there is no cost advantage for supplying water directly to Caño Sucio Reservoir. However, based on the information presented in the Executive Summary for the Toabré Study (3), it can be estimated that the economic cost of water for the Toabré Project (computed using the same assumptions as for this study) is significantly higher than Alternative 2. Therefore, it can be postulated that Alternative 2 should be used in place of the Toabré Project. It was decided that judging the viability of the Toabré Project on the basis of a draft executive summary was not appropriate. Therefore, Alternative 1, Scenario 1 is adopted for inclusion in the project description for the following reasons:

1. It was assumed that a development of this nature should be sized to near fully develop resource and, therefore, Alternative 1, Scenario 3 was not selected.

[1] Ibid.

2. The construction cost is high enough to represent any alternative that might be implemented.
3. There are financial disadvantages to high operating costs and, therefore, Alternative 1, Scenario 2 and Alternative 2 were not selected.
4. The 8 required pumps required for Scenario 1 could more easily be staged thus improving the economic cost of water to some degree.
5. In the adoption of Scenario 1, it is implicitly assumed that the Toabré Project will exist. If this assumption were not correct, the inclusion of Alternative 2 into the Panama Canal System would have about the same economic impact.

Although not a part of the scope of services, a preliminary evaluation was performed to determine if the addition of hydropower at the dam is attractive for Alternative 1 Scenario 1 and Alternative 2. The power plant was sized for the pumping station design flow plus the minimum release. Generation was computed for a flow equal to the average annual supply less the amount actually supplied in any year. Allowing for the costs and benefits associated with the hydro installation, the economic cost of water was reduced by $0.01/m^3 for both arrangements.

4.2 Spillway and Diversion Flood

The spillway is designed for the probable maximum flood. For a project whose failure could result in loss of human life and economic endeavor, it is customary to design the project for the worst conditions that could reasonably be postulated. The adopted pumping alternative presumes construction of Toabré Dam, therefore, the maximum peak inflow of the PMF is estimated to be about 9,970 m^3/s and the 3-day volume is about 950 MCM.

A flood with a return period of 25 years was selected for the construction diversion flood for the RCC dam. This flood would have a peak discharge of 2,100 m^3/s and a 2-day volume of about 50 MCM.

4.3 Spillway Size

The spillway was sized to limit the flood surcharge to between 3 m and 5 m to minimize impacts in the valley upstream from the dam, most notably in Coclecito. Both ungated and gated spillways were evaluated. A 100-m wide ungated spillway with its crest at El. 35 resulted in a flood surcharge of 7.8 m. This surcharge exceeds the imposed limit. As a result, ungated spillways were rejected because wider spillways are not suitable for the channel conditions below the dam. A six-gate spillway with a clear opening of 48 m and a crest at El. 23 resulted in a surcharge of 3.3 m using spillway rating information developed for the Upper Chagres Project. This surcharge was within the adopted limits and the configuration was accepted. Each of the spillway gates will be 6 m wide by 12 m high. The peak outflow under PMF conditions will be about 6,520 m^3/s

4.4 Diversion Facilities

The diversion tunnel will be sized to pass the 25-year flood, which was estimated to be 2,100 m^3/s. This is coincidentally the same magnitude of flood that was used in the Upper Chagres Feasibility Study (9) and, therefore, the same conduit arrangement was used. The diversion facilities will consist of to 7 m by 7 m box culverts and the upstream water surface will reach El. 13.2. The diversion facilities also will be used for emergency drawdown and minimum release.

5. DESCRIPTION OF THE LOW COCLÉ DEL NORTE PROJECT

The major elements that comprise the Low Coclé del Norte Water Supply Project include:

- Storage facilities on the Río Coclé del Norte consisting of:
  - A roller compacted concrete dam with the top-of-structure at El. 41,
  - A 6-bay gated spillway sized to protect the dam against the PMF,
  - A twin-box culvert for diversion during construction, and
  - Access roads.
- A pumping station at the toe of Toabré Dam that includes two sets of 4 pumps and a transmission line to connect to the national grid.
- A series of dikes, channels, and bridges to protect the town of Coclecito against inundation from the operation of the reservoir.

Prior to the implementation of the Low Coclé del Norte Project, it is presumed that the Río Indio Water Supply Project and the Río Toabré Water Supply Project have been implemented and that the transfer tunnels from Toabré to Indio and Indio to Gatun have the capacity to accommodate water from the project.

The Low Coclé del Norte Dam will impound a reservoir with a gross storage capacity of 662 MCM at El. 35, the full supply level. The reservoir area at the full supply level is approximately 51 square kilometers.

A general plan of the development, showing the location of the dam and pumping station is shown on Exhibit 5-1.

Upon completion of the dam and pumping station, the project will have the capability of supplying an average of 1,854 MCM or the equivalent of 24.6 lockages per day into the Panama Canal System. The demand on the Canal's water supply system will depend on the future configuration of the Canal System, the adopted reliability of supply, and continued supply of M&I water from the system to the Panama Canal Watershed through IDAAN. The implementation of the Low Coclé del Norte Project will provide a significant quantity of water to assist the ACP in meeting water supply demands for foreseeable future conditions.

5.1 Description and Preliminary Design of Project Features

This section presents a more complete description of the project features and the design assumptions that were adopted. The project hydrology and details of the pumping scheme are presented in detail in Appendices A and B.

5.1.1 Low Coclé del Norte Dam

The Low Coclé del Norte Dam is proposed as a roller compacted concrete (RCC) dam. A plan of the dam is shown on Exhibit 5-2 and a profile, sections, and details are presented on Exhibit 5-3. The dam will be made of a low to medium cement content concrete mix with aggregate taken from a quarry located to the east of the site. A grout-enriched RCC mix will be placed on the foundation to provide a good contact with the foundation and act as a water barrier. The upstream face will be vertical and downstream face will have a slope of 0.75H:1V. The dam will have an in-place volume of about 180,000 cubic meters.

The axis of the dam will be essentially in an east-west direction about 15 degrees off of the perpendicular across the river. The axis crosses the main channel of the Río Coclé del Norte at about 993,600N, 550,400E (UTM Coordinate System). The crest of the dam will be at El. 41 and the crest width will be 8.0 m. The dam will be about 51 m high from the deepest foundation excavation to the top of the dam.

Precast concrete panels, 0.3 m thick, and an impermeable membrane will act as the principal water barrier and as a facing for the upstream vertical face and the vertical section of the downstream face. The sloped downstream face of the dam will be stepped. A gallery will be located at about El. 5 near the face of the dam. It will serve as access to the interior of the dam for installation of the grout curtain and inspections, as a collector of seepage, a terminal point for foundation drains, and access for instrumentation. The galleries will be 3-m high by 2.5-m wide and traverse the entire length of the dam.

5.1.2 Foundation Treatment

The proposed treatment programs for the dam foundation will include surface treatment, curtain grouting, and drainage.

Dental excavation and concrete will be used to treat local zones of highly weathered, sheared, or otherwise unacceptable rock encountered in the foundation. Required dental treatment should be nominal and only local. Contingency quantities for backfill concrete have been included to reflect the potential for unforeseen conditions.

Consolidation grouting is not envisaged except in limited areas, or in fault or fracture zones. Low pressure cement grouting will be used in such limited zones to fill open cracks or joints in the rock zone immediately beneath the dam foundation. In general, grout takes should be low.

Curtain grouting will be performed from the gallery constructed in the dam. It is assumed that the spacing of the primary holes will be 10-m on center and that split spacing will be performed down to 2.5 m over the entire width of the dam and to 1.25 m over 75% of the width of the dam. The grouting will be to a depth of one-half the height of the PMF above the dam foundation or a maximum depth of about 25 m below the structural foundation level. The average grout consumption was assumed, for estimating purposes, to be about 30 kg/m.

Foundation drainage will be provided to control seepage to reduce pore pressures in the rock mass, and hence uplift. For estimating purposes, a drain hole spacing of 3 m was assumed with depths extending to about half the depth of the grout curtain from the crest of the dam. The holes would be appropriately inclined in order to maximize the number of joint/fracture interceptions.

5.1.3 Spillway

A gated chute spillway will be located in the right-center portion of the dam (looking downstream). A plan and sections are shown on Exhibit 5-4. The spillway has been designed to pass the PMF without overtopping the dam. The discharge under PMF conditions will be 6,520 m^3/s using a surcharge of 3.3 m above the full supply level.

The spillway will consist of an ogee control structure, a straight chute, and a stilling basin to dissipate energy. The ogee section will consist of six bays with a clear opening of 48 m. Control will be afforded by 6 radial gates, each 8-m wide by 12-m high. Seven piers will support an 8-m wide bridge to connect the left and right abutments. The piers will be high enough to provide control over one-half the gate at the full open position.

The chute will be aligned down the downstream face of the dam and will terminate in the stilling basin. The width of the spillway chute will be constant at 63 m. The chute will be comprised of conventional concrete placed over the stair-stepped downstream face of the RCC dam.

The stilling basin will be founded on and anchored to competent rock and will be about 60 m long. It will contain a single row of baffle blocks and will contain the jump for all discharge conditions. The lip of the end sill will be at El. –5 and the walls of the basin will be at El. 15. The channel downstream from the basin will be sloped at 1V:5H back to original grade.

5.1.4 Diversion During Construction

During construction, the Río Coclé del Norte will be diverted through a twin-barrel conduit and cofferdams located upstream and downstream from the damsite will protect the work site. The conduits will serve to:

- Pass the 25-year flood during construction,
- Control the rate of initial reservoir filling,
- Provide for emergency evacuation of the reservoir, and
- House the minimum release facility.,

The river diversion facilities plan, profile, sections, and details are presented on Exhibits 5-5 and 5-6.

The two-barrel conduit, 230 m long, will be trenched into rock on the left (west) side of the river. Each barrel of the conduit will have a 7-m wide by 7-m high opening. The maximum discharge will be under diversion conditions and, for the 25-year flood event, will equal about 770 m^3/s. The excavation for the conduit will be located off-channel and protected so that it will remain dry under the 20-year flood conditions. The excavation within the footprint of the dam will be backfilled around the conduit with conventional concrete.

The approach channel will be about 60 m long and will be excavated to El. 0. The first 115 m of the discharge channel will be concrete lined and the remaining channel back to the river will be about 120 m long.

A 20.5 m high cofferdam founded on weathered rock will be located across the river channel about 80 m upstream from the upstream face of the dam. It will be constructed of relatively impervious random fill with its crest at El. 13.5. A 9-m high cofferdam will be located about 50 m downstream from the lip of the spillway stilling basin.

5.1.5 Emergency Drawdown Facilities

Emergency drawdown will be accomplished using the spillway and one of the diversion conduits. To accomplish the drawdown, a gate tower 10 m by 6 m will be constructed on the face of the dam. Two sets of two 3-m wide by 6-m high gates will be installed with their sills at the bottom of the culvert (El. 0). The gate sets will include emergency gates and operating gates. The gates were set at a level required to maintain the reservoir at about the 25% level while passing the design inflow even though this resulted in drawdown times much shorter than required. The sediment deposition is estimated to reach El. 4 at the dam after 100 years and, therefore, should not adversely impact the operation of the emergency drawdown facilities. The design inflow was assumed to equal the average of the two highest mean monthly flow values or about 126 m^3/s. A profile, sections, and details of the emergency drawdown facilities are shown on Exhibit 5-6.

The drawdown requirements and time to empty the reservoir are shown in Table 5-1.

TABLE 5-1 EMERGENCY DRAWDOWN

Reservoir Level (% of FSL)	Reservoir Elevation	Allowable Time to Reach Elevation (days)	Computed Time to Reach Required Elevation (days)
75	26.0	30-40	2
50	17.5	50-60	11
25	9.0	80-100	36

5.1.6 Minimum Release Facility

The minimum release has been assumed to equal 10% of the average flow or about 10.7 m^3/s. The minimum release will be discharged through a valve into the second conduit. It should be attractive to add a penstock and hydro plant to this minimum release facility.

5.1.7 Coclecito Protection Plan

The Coclecito Protection Plan will protect almost the entire area of Coclecito together with the developed area east of Coclecito from flooding after the construction of the Coclé del Norte dam and the impoundment of a reservoir with its full supply level at El. 35. A plan and sections of the facilities are shown on Exhibit 5-7 and 5-8. The maximum flood level of the reservoir is estimated at El. 38.3. The protection system includes a series of levees around the low areas that will have a crest at El. 40. The levees have been located to avoid existing properties while protecting as many as possible. To minimize impact to the town, the Río Coclecito will need to be relocated along with the main access road and bridge to provide room for the protection levees as shown on Exhibit 5-7. The relocated river channel and bridge have been designed to safely pass the100 year flood. In addition, detention ponds and pumping facilities will be included in the protection works to prevent flooding inside the protected area for storms with a return period of 100 years.

The levees will be constructed of locally excavated impervious materials. The crest width is 5 m, slopes are 3H:1V, and a minimum of 1 m of overburden will be removed to provide underseepage control. No foundation improvements are planned. Three levees totaling 1,500 m are required. As the grass runway or landing strip is in a state of disrepair and is currently unusable, it has not been replaced. Two storm runoff detention ponds and two pumping stations are required to prevent flooding behind the levees on the Coclecito side, and one small pumping station east of the town. They will be provided with standby generators as well as capability to connect to the local power supply. They will operate automatically when the level of water in the detention basins rises over a preset level.

5.1.8 Pumping Station and Associated Facilities

The pumping facilities consists of a pumping station, pipes to deliver water through the Toabré Dam diversion tunnel into the reservoir, and an energy connection to the national grid. The pumping station will house four 23.9 MW and four 12.6 MW centrifugal pumps.

5.1.8.1 Pumping Station

The pumping station will be located at the toe of Toabré Dam on the south bank of the Río Toabré. Plans and sections of the station are shown on Exhibit 5-9 and 5-10.

The pumping station will be a conventional reinforced concrete structure, 110 m long by 22 m wide. It will contain eight pump bays and one service bay. The foundation will be excavated in competent rock at El. 20. The centerline of the pumps will be at El. 29. The general characteristics of the pumps and motors are described below.

	24-MW Units	13-MW Units
Pumps		
Design flow (m^3/s)	38.5	38.5
Design Dynamic Head (m)	52.6	27.7
Peak Efficiency %	83	83
Rotational Speed (rpm)	257	225
Motors		
Rated Output (kVA)	28,600	15,060
Rated Installed Capacity (MW)	25.7	13.3
Frequency (Hz)	60	60
Power Factor	0.90	0.90

One each of the large and smaller pumps will connect to an 85-m long pipe constructed in the Toabré Dam diversion conduit. The connection will consist of a manifold and pipe. The length of the pipe will vary with the location of the pumps in the pumping station. The 2.75-m diameter discharge lines will average about 100 m long and sized for a maximum velocity of 4.5 m/s.

5.1.9 Power Supply

The energy requirements for the pumping scheme, which include an allowance for pumping, station services, and transmission line losses, are estimated to be about 300 GWh/year at full supply. To provide this energy, a 65-km long, 230 kV, single circuit transmission line will connect to the nation grid near Penonome. A new substation will be required for the connection at Toabré Dam and a new switchyard will be required at Penonome.

To provide power to the spillway gates and other gates and valves at the dam, a 13.8 kV line will be installed from the pumping station to the dam over a distance of 20 km. In addition, a portable emergency generator will be provided.

The alignments of the 230 kV and the 13.8 kV line are shown on Exhibit 5-11.

5.1.10 Additional System Modifications and Transfer Facilities

As was mentioned in Section 3.5.3, the average flows pumped into the system do not represent additional system yield. For the scenarios associated with Alternative 1, the respective monthly flows would be added to the inflow to Toabré Reservoir and operated through the system to determine yield. Optimization of the yield of the system with these additional flows may require: a larger or second transfer tunnel to the Río Indio Reservoir than is currently planned under the Toabré Reservoir studies (3), a larger or second transfer tunnel from the Río Indio Reservoir to Lake Gatun, a higher dam at Toabré, or a new system rule curves. For Alternative 2, similar adjustments may be necessary for: the Río Caño Sucio Reservoir, the transfer tunnel to the Río Indio reservoir, or the transfer tunnels from Río Indio Reservoir to Lake Gatun.

Studies to evaluate these potential changes are outside of the scope of this study and the current estimate of cost does not include any allowance for these potential modifications.

6. CONSTRUCTION PLAN AND ESTIMATE OF COST

A project implementation schedule, a construction schedule, and a cost estimate have been prepared for the project as described in Section 5. Implementation is expected to require about 9.5 years with construction taking about 4.8 years.

6.1 Implementation

The major steps required for project implementation following this engineering study are as follows:

- Feasibility Study
- Environmental Base Line Studies
- Feasibility Confirmation and Project Configuration
- Funding
- Environmental Field Studies
- Environmental Mitigation Planning and Implementation
- Design
- Contractor Procurement and Construction Contract Awards
- Construction

The implementation schedule is presented on Exhibit 6-1.

If the project moves forward, one of the first activities will be to perform a technical, economic, and financial feasibility study. The study will include subsurface investigations to evaluate the foundation conditions, economic studies to optimize the project facilities and establish the economic justification, and financial analyses to determine the cost of water. As a part of the feasibility assessment and to support a funding request, environmental base-line studies will be required. Upon completion of the baseline studies, the feasibility of the project will be confirmed and the final project configuration studies will be completed.

Once the decision to obtain funding has been taken, a mitigation plan will be required. This can be prepared while funding is being secured. This plan may cover relocation or resettlement of people from the villages along the river, habitat mitigation, evaluation of cultural resources, development of a construction plan, and any other mitigation activities identified during the base-line studies. The field studies may include protection or removal of significant archaeological sites, habitat replacement, or additional detailed studies of the flora and fauna in the reservoir area.

With funding in place, the design-level investigation program and final designs can commence.

Construction of the project is envisaged to consist of four general contracts: 1) Access Roads and the Construction Camp, 2) The Storage Facilities, 3) The Pumping Station, and 4) Reservoir Clearing. Contractor selection and award will be performed for each contract and scheduled to meet the construction schedule.

It is assumed that the ACP has the capability to manage the construction contracts and also to perform the operation and maintenance. It is suggested that the ACP hire a Project Management Team responsible for the consulting engineering services relating to the works required for implementation. The operation and maintenance can be accomplished by adding staff to the ACP's existing organization. Operation and maintenance of the facilities required for the water supply project, including any pumping facilities, are well within the existing capability of the current O&M organization.

6.2 Construction Plan and Schedule

A plan and schedule have been developed for the construction of the Low Coclé del Norte Project. The construction of the project, including the access roads and construction camp, the dam and appurtenant works and the pumping facilities is anticipated to take approximately 4.8 years. Filling the reservoir to the full supply level, El. 35, is estimated to require about one wet season, assuming average hydrologic conditions and the continuous release of 10% of the flow downstream. A construction schedule is shown of Exhibit 6-2.

The project has been divided into four major contracts:

- Access Roads and Construction Camps,
- Low Coclé del Norte Storage Facilities,
- Pumping Facilities, and
- Reservoir Clearing.

The durations of the construction of each component are estimated as 1.7 years for the access roads, 3.4 years for the Low Coclé del Norte Storage Facilities, 2.7 years for the Pumping Station including the transmission line, and 0.5 years for reservoir mapping and clearing.

6.2.1 Access Road and Construction Camp, Contract 1

Contract 1 requires the construction of 25 km of permanent roads, 4 km of construction roads, two bridges, and the construction camp for the storage facilities.

6.2.1.1 Mobilization

Upon receiving notice to proceed, the contractor will initiate mobilization of staff and equipment. The mobilization of personnel and equipment for road construction is expected to be rapid as it is entirely available locally.

6.2.1.2 Access Roads

Access to the Low Coclé del Norte Project will be provided from the permanent access for the Río Toabré dam. The access road will be 25 km long and will require a major bridge over the Río Coclé del Norte, and smaller bridges along the route. The road alignment is shown on Exhibit 5-11.

Construction of the access roads and bridges is expected to take about 1.7 years and the main access road is on the critical path. Temporary construction access will be constructed as a part of the individual contracts.

6.2.1.3 Construction Camp

It is anticipated that two construction camps will be required: one for the construction of the Low Coclé del Norte Storage Facilities, and one for the Pumping Station. A smaller camp may be required for the transmission line.

The camp for the Low Coclé del Norte Storage Facilities will be located in an area of 10 hectares approximately 2.5 kilometer east of the damsite along the access road. The construction will be started so that its substantial completion coincides with the completion of the Main Access and the beginning of the construction of the main features of the project. Ten weeks of construction will be sufficient to provide housing for the initial crews working at the dam site.

The camp for the Pumping Station will be located at the toe of Toabré Dam and will be a part of the Pumping Station Facilities contract.

6.2.2 Low Coclé del Norte Storage Facilities – Contract 2

Construction of the dam and its appurtenant works will take about 3.8 years and falls on the critical path.

6.2.2.1 Mobilization

The effort is anticipated to extend over a three-month period, as the contractor's effort will build-up during the execution of the preliminary works including construction access and a power supply. Most of the heavier pieces of equipment will be mobilized towards the end of that period when the main access road and the camp have been completed and when the construction of the main features of the project starts.

6.2.2.2 Rock and Aggregate Quarries

The quarry is located about three kilometers east of the damsite. A crushing plant and an aggregate stockpile area will be established. The establishment of the quarries is anticipated to take approximately 7 months including geotechnical investigations. As it is not on the schedule critical path, investigations can be initiated at any time during the first year of activities before aggregates are needed.

6.2.2.3 Low Coclé del Norte Dam and Appurtenant Works

A twin-barrel diversion culvert approximately 230 meters long will be used to pass the river flows during the dam construction. One culvert will be ultimately used as the low-level outlet for emergency drawdown. The diversion works, including excavation and concreting, is estimated to take approximately 1 year. The main upstream and downstream cofferdams, including excavation and foundation preparation, will be completed in 3 months.

Construction of the dam and appurtenant works requires the placement of approximately 180,000 m^3 of roller-compacted concrete. Aggregates will be obtained from the quarry located slightly downstream from the damsite on the right side of the river. A rapid placement of the RCC will be followed by the construction of the spillway with conventional concrete. A conveyor or crane system will be used to transport the RCC to the dam to avoid contamination that might be caused by trucks during the wet season. Overall construction of the dam, including the gravity section and the overflow spillway is expected to take approximately 2 years. The installation of the six tainter gates and hoisting mechanisms will require another 12 months to complete.

One of the diversion culverts will be transformed into the low level outlet. The construction of the gate tower on the upstream face of the dam will approximately follow the progress of the dam construction. Upon completion of the RCC placement, the embedded parts and the gates will be installed in the conduit at the bottom of the tower. The installation is expected to take approximately three months. Upon completion of the gate installation one culvert will be plugged at the upstream end.

The protection facilities at Coclecito will require about 18 months to complete. The construction sequence calls for ordering the pumps and beginning the bridge construction. The supply of pumps is estimated to take 12 months to be followed by 6 months to construct the pumping stations and install the pumps. In the interim, the detention ponds and levees will be constructed. The detention ponds required the excavation of about 855,000 m^3 and the levees will require the placement of about 1,050,000 m^3.

6.2.3 Pumping Facilities- Contract 3

This contract will include the construction of a camp, the pumping station, and the transmission lines. It is assumed that the facilities required to convey the water through the diversion facilities of Toabré Dam were put in place at the time the dam was constructed although the cost of these facilities is included in this estimate.

6.2.3.1 *Mobilization*

Mobilization will include construction of temporary access roads and the construction camp for the pumping facilities. The camp will be located in the vicinity of the access road to the construction site.

6.2.3.2 Pumping Station

The pumping station will be built at the toe of Toabré Dam. The construction will be started with a cellular and embankment cofferdam to protect the work area. The civil works of the pumping station will take approximately 16 months to complete. Furnishing and installing the pumps and motors, which is on the critical path, will require about 2.5 years.

6.2.3.3 Transmission Facilities

Two substations and a switchyard will be required at Coclé del Norte, Toabré and Penonome respectively. Construction, including the installation of the equipment, will require about 6 months for each switchyard and they have been scheduled concurrently with construction of the transmission line.

Construction of the power supply for the pumping station and the gates at Coclé del Norte Dam is expected to take about 1.8 years of which 11 months will be for the construction of the 65-km 230-kV transmission line connecting to the national grid at Penonome. Start-up and commissioning of the pumping station and power system will take place over 10 months, of which 2 months is on the critical path.

6.2.4 Reservoir Clearing and Filling – Contract 4

The reservoir will be cleared to El. 40. At El. 40, the surface area is approximately 65 km^2. It is assumed that about 40% of the area, or about 2,600 ha, is in forest and needs to be cleared. The clearing is expected to take approximately 6 months. With mapping, the contract will take about 7 months.

The Low Coclé del Norte reservoir can be filled in one wet season using its own average inflow and allowing for a continuous minimum release of 10% of the inflow.

6.3 Cost Estimate

The cost estimate for the construction of the Low Coclé del Norte Water Supply Project is based on the feasibility-level estimate prepared for the Río Coclé del Norte Project at El. 100. The unit prices, which include the direct costs of labor, equipment and material, the Contractor indirect costs and engineering and administration costs, were used without adjustment. The costs were estimated in US Dollars at a January-2003 level.

For each project feature, quantities were calculated from the drawings and exhibits. Unit prices were estimated for excavation (common and rock), fill placement, quarrying, concrete fabrication and placement, formwork, steel reinforcement and tunnel excavation and lining. Other unit costs were estimated from experience on other projects of similar nature.

6.3.1 Cost of Labor and Materials

The cost of local labor was estimated based on the "Convención Colectiva de Trabajo de Panamá" dated July 1998. This document indicates the minimum applicable wages to be paid to workers in the construction industry by profession and region, for every year from July 1998 to June 2002. Beyond that date, labor rates were adjusted according to the local CPI. These rates were increased by 30% to reflect the fact they are mandatory minimum wages. An average across the professions was taken to derive four main categories: unskilled labor, skilled labor, equipment operator and truck driver. The wages were also increased to reflect the expected 60-hour workweek: an overtime premium of 16.7% was assumed. Labor rates were then calculated by adding 50% for social costs. This resulted in the following hourly cost of salary.

TABLE 6-1 LOCAL LABOR COST OF SALARY

Category	Labor Rate
Unskilled labor	\$5.60/hr
Skilled labor	\$6.70/hr
Equipment operator	\$8.00/hr
Truck driver	\$6.30/hr

In addition to the local labor a crew leader was generally included at the rate of $10.00/hr.

Equipment rates were generally obtained from the publication of the US Army Corps of Engineers entitled "Construction Equipment Ownership and Operating Expense" (EP 1110-1-8), dated August 31, 2001. These rates have been increased at a 0.8% p.a. factor, reflecting the construction equipment cost escalation for the period. Equipment requirements and production rates were developed based on experience in similar type of project in tropical countries.

It is anticipated that materials including explosives, cement, and reinforcement steel will be imported for the most part. International unit prices were used: Table 6-2 below shows estimated unit costs of materials delivered at the site.

TABLE 6-2 MATERIAL UNIT COST

Material	Unit	Cost
Cement	MT	$122
Explosive	Kg	$1.50
Reinforcing Steel	MT	$725

The operational costs for the international contractor were also itemized for the purpose of this estimate. These costs include a management and engineering crew of eight, including a project manager, a superintendent, three staff engineers, a purchasing agent, a scheduler (coordinator) and an accountant. The crew will be fully mobilized on site for the duration of construction, after completion of the preliminary works such as access road, construction camp, establishing quarries, etc. A supporting crew of administrative personnel and drivers was also itemized. Other operational costs accounted for include items such as a maintenance crew, vehicles for staff transportation and telephone.

Overall the contractor operating costs were estimated at approximately 7% of the total direct construction cost. In addition, the following indirect costs were also added:

Contractor home office charges	7.0%
Bond	1.5%
Insurance	2.5%
Margin for risk	2.0%
Margin for profit	10.0%

As a result, unit rates calculated on the basis of the costs of labor, equipment and materials have been increased by a margin of 30% to reflect these items.

The resulting unit prices were compared with those obtained through the bidding process on other international water resources projects in Central and South America and appeared to be reasonable estimate for this type of construction.

The unit prices shown in Table 6-3 were used.

TABLE 6-3 ESTIMATED UNIT PRICES (JAN-2003 LEVEL)

Material	Unit	Price
Clearing	Ha	$2,200
Overburden Excavation	m^3	$3.70
Rock Excavation	m^3	$9.20
Cofferdam Selected Fill	m^3	$7.30
Drain Material	m^3	$16.10
Mass RCC	m^3	$56.00
Mass Concrete	m^3	$116.00
Structural Concrete	m^3	$145.00
Backfill concrete	m^3	$105.00
Precast Concrete Panels	m^3	$282.00
Formwork	m^2	$46.80
Reinforcing Steel	Kg	$1.32
Temporary Access Road	Km	$115,200
Permanent Access Road	Km	$147,600

A detailed estimate of the Construction Cost is presented in Table 2 (at the end of the text) and a summary is shown in Table 6-4 below.

TABLE 6-4 SUMMARY COST OF THE LOW COCLÉ DEL NORTE PROJECT

Item	Estimated Cost
Mitigation and Compensation Costs	**$23,000,000**
Construction Costs	
Access Roads and Construction Camp	$6,650,000
Low Coclé del Norte Storage Facilities	$50,960,000
Pumping Facilities	$87,290,000
Reservoir Clearing	$5,920,000
Subtotal Direct Cost	*$150,820,000*
Contingency	$27,080,000
Direct Cost	*$177,900,000*
Engineering and Administration	$27,100,000
Construction Cost (Jan 2003 price level)	**$205,000,000**
TOTAL COST	**$228,000,000**

A detailed estimate of cost is presented in Table 3 at the end of the text.

6.3.2 Social, Economic and Environmental Mitigation and Compensation Costs

The ACP has not estimated mitigation and compensation costs for the Low Coclé del Norte Project. However, these costs have been estimated for the Western Watershed projects that have been done at the feasibility level. A curve of compensation and mitigation costs versus reservoir area at full supply level has been prepared. From this curve it is estimated that the compensation and mitigation costs for the Low Coclé del Norte Project are about $23,000,000. The allocation for land acquisition has been included in these costs. These costs are in addition to the construction cost of the Coclecito protection works.

6.3.3 Contingencies

A contingency allowance is included in the cost estimate for unforeseen site conditions, approximations, and the potential for future design changes. For these estimates, an allowance of 20% was used for the pumping facilities, 15% was for all other civil items and 10% for all other equipment. Overall, the contingency is approximately 13% of the project construction cost or about 18% of the subtotal of the direct costs.

6.3.4 Engineering and Administration

Indirect costs for final design, engineering services during construction, and for administration costs of the APC chargeable to the project are based on previous experience for similar projects. It has been estimated that about 15 percent of the total direct costs will be adequate for engineering and administration.

6.3.5 Disbursement Schedule

A disbursement schedule has been estimated beginning when funding is secured according to the implementation schedule. The disbursement schedule has been estimated on the basis of the cost estimate, the implementation schedule, and the construction schedule. The disbursement schedule, presented in Table 6-5 shows a distribution for the construction cost. Based on outdated demand data, the project could be required around 2030. The disbursement schedule has been tied to that date.

TABLE 6-5 DISBURSEMENT SCHEDULE
($1,000)

Year	Disbursement of Construction Costs
1(2026)	$15,000
2(2027)	$28,000
3(2028)	$58,000
4(2029)	$68,000
5(2030)	$59,000
Total	**$228,000**

6.4 Annual Operating Cost

The annual operating costs include the costs of operation and maintenance (O&M), for the various features, the cost of replacing short-life equipment, administration by the Owner, and insurance. For this estimate, it was assumed that the operating costs would be the equivalent of 1% of the construction cost for the access roads and storage facilities plus 2% of the construction cost of the pumping facilities plus the energy cost.

O&M for the roads, dam and pumping facilities will be performed by an O&M group that will be a part of the ACP's much larger Canal Operation Group. The estimate also includes an allowance for vehicles, spare parts, and maintenance equipment.

The cost of replacing short-life equipment normally is included in the annual cost as a sinking fund. The allowance would permit replacing all equipment in 25 years.

Administration and general expenses of the owner is for salaries, outside services, injuries and damages, welfare, pensions and miscellaneous expenses. These costs are typically assumed to equal about 40% of the labor cost of the O&M personnel.

The annual cost of insurance is usually about 0.1% of the construction cost.

In addition, the percent allowance for operation is also meant to include an annual expenditure for watershed management, implementation of the environmental mitigation plan and the relocation activities is included.

The average energy requirement at full supply is estimated to be about 300 GWh/year and is comprised of 289GWh for pumping, 4 GWh/year for station services, and 7 GWh/year for transmission line losses. The actual pumping cost varies each year depending on the demand. Valued at $0.07/kWh, the average annual cost of providing energy was computed as the equal annual equivalent of the present worth of the required generation times the value of energy. The annual cost varies from about $200,000/year in the early years of operation in a dry year to about $40 million/year at full supply in a wet year and is estimated to be about $5.3 million per year.

The average annual operating cost is, therefore, estimated to be $8,400,000.

7. ECONOMIC COST OF WATER

As was the case for the feasibility-level studies performed under this work order, there is not sufficient information available to perform an economic feasibility assessment. As for the other projects, an estimate of economic cost of water has been developed as a means of comparing all the projects. The estimate for the Low Coclé del Norte Water Supply Project is derived by dividing the present worth of the total project cost (construction, compensation and mitigation cost) and the annual costs by the present worth of the supply. In all estimates, a discount rate of 12% was used. For these studies, the supply, which was taken equal to the average annual flow pumped into the system, was used as no estimate of the incremental system yield was made.

If conditions existed where the full amount of the water supply from the implementation of the Project could be beneficially used immediately upon the completion of construction, the economic cost of water would be about $0.02/m^3$. This condition is highly unlikely and only serves to indicate a minimum cost of water.

A more likely condition assumes that only a portion of the supply can be used when the project comes on line and that the usable supply increase at some reasonable rate. For example, if the year 2000 demand were assumed to be 38 L/d, the demand increased at 0.75 L/d/yr, the existing system, which is presumed to include the Río Indio Reservoir, could yield 60.3 L/d, the supply translates directly to yield, and the project came on line in the first year that there was a system deficit (year 2030), the economic cost of water would be about $0.09/m^3$.

Under the same assumptions and a rate of increase of 0.6 L/d/yr, the cost of water would be about $0.10/m^3$.

It must be noted that the Low Coclé del Norte Water Supply Project cannot be implemented on its own. It has been presumed that the Toabré Water Supply Project is already in existence; however, it can be implemented at about the same economic cost with a slightly different configuration and the Río Caño Sucio Project. The indicated economic costs represent only the Project facilities and do not include any costs for the Toabré Water Supply Project.

For comparison purposes, the following table presents historic demands with growth rates of 0.75 L/d and 0.6 L/d assuming that the year 2000 demand for navigation and M&I water is about 38 L/d.

TABLE 7-1 COMPARISON OF DEMANDS AND COST OF WATER

Year	Base Demand Increased by 0.75 L/d/yr	Base Demand Increased by 0.6 L/d/yr
2000	38.0 L/d	38.0 L/d
2020	53.0 L/d	50.0 L/d
2040	68.0 L/d	62.0 L/d
2060	83.0 L/d	74.0 L/d
Cost of Water	**$0.09/m^3**	**$0.10/m^3**

A comparison of the Western Watershed Projects studied to date, ranked by cost of water, is shown below.

TABLE 7-2 COMPARISON OF PROJECTS AND COST OF WATER

Project	Yield	Base Demand Increased by 0.75 L/d/yr	Base Demand Increased by 0.6 L/d/yr
Río Indio	15.8 L/d	\$0.07/m^3	\$0.10/m^3
Low Coclé del Norte	**24.6 L/d**	**\$0.09/m^3**	**\$0.10/m^3**
Upper Chagres	5.3 L/d	\$0.14/m^3	\$0.19/m^3
Coclé del Norte FSL @ El. 71 (acting in full regulation with Indio Reservoir)	45.8 L/d	\$0.19/m^3	\$0.22/m^3
Coclé del Norte FSL @ El. 100 (acting in full regulation with Sucio and Indio Reservoirs)	47.0 L/d	\$0.23/m^3	\$0.27/m^3

The economic cost of water for the Project indicates that the Low Coclé del Norte Project provides an attractive source of water when compared to the other projects in the western watershed and in the Upper Chagres basin.

However, the economic cost of water presented in Table 7-2 does not necessarily provide a basis for ranking because there are development assumptions associated with each project. For the Low Coclé del Norte Project, these assumptions are as follows:

- The economic cost of water for the Low Coclé del Norte Project is based on the assumption that the Río Indio Project and the Río Toabré Project have been completed. (As an alternative, a project with a similar economic cost can be developed if the Caño Sucio Project has been completed.) It was also assumed that the incremental system yield is the amount of water pumped through the project.

- The demand assumptions, which form the basis for the economic cost of water, may not be correct.

For the other projects, the same demand assumption was made plus:

- The economic cost of water for the Río Indio Project assumes that the Lake Gatun deepening, that is now underway, is completed.

- The yield for Upper Chagres will only be realized if it is the first new water supply project constructed. If the Upper Chagres Project is constructed first, then the yield of all subsequent projects will be slightly reduced.

- The economic cost of water for the storage projects on the Río Coclé del Norte at full supply levels at El. 100 and El. 71 is based on the assumption that the Río Indio Project has been completed.

8. CONCLUSIONS AND RECOMMENDATIONS

As a result of the studies described in the foregoing sections, it is concluded that:

- There is no readily apparent fatal flaw to the Project.
- There are no features associated with the development that would cause it to be infeasible.
- A roller compacted concrete dam is an appropriate type of dam to provide an indication of the cost of the project.
- It is our considered opinion that there are no geologic or geotechnical problems associated with the sites that cannot be accommodated using conventional solutions although the lack of subsurface investigations has increased the potential for inaccuracies in the estimate of cost.
- The Project will supply up to 1,854 MCM/year into the Toabré Reservoir.
- Construction of the project is estimated to cost about $205 million in 2003 dollars. An addition $23 million have been allowed for compensation and mitigation for a total cost of $228 million.
- The Project will provide an economically attractive source of water for the Panama Canal system if the assumed pre-construction development takes place.

Based on these conclusions, it is recommended that the Project be investigated in further detail if additional water is needed for the Panama Canal System after construction of the Río Indio Project and either the Río Toabré Project or a project on the Río Caño Sucio.

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Tables

TABLES

No.	Title
1	Monthly Streamflow at the Dam Site
2	Monthly Streamflow at the Dam Site Excluding the Toabré Dam Drainage
3	Detailed Cost Estimate

Table 1

MONTHLY MEAN DISCHARGES
RÍO COCLÉ DEL NORTE AT DAM SITE
(m^3/s)

Drainage Area: 1,594 km^2

YEAR	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP	OCT	NOV	DEC	ANNUAL
1948	67.5	37.1	30.1	25.0	40.2	50.3	120.0	133.4	128.8	122.3	198.1	82.5	**86.3**
1949	46.6	33.9	25.9	24.5	49.0	152.0	119.4	139.4	176.5	169.9	250.9	212.4	**116.7**
1950	64.7	44.5	32.3	26.4	92.2	139.6	143.6	186.5	140.4	173.9	208.8	209.3	**121.9**
1951	95.0	64.1	44.5	34.3	93.8	107.5	102.3	108.2	155.8	138.8	184.4	126.1	**104.6**
1952	72.5	44.9	30.1	27.8	64.7	120.9	101.6	105.7	151.0	193.9	140.2	178.6	**102.7**
1953	145.8	76.8	50.9	40.9	105.0	97.6	95.7	84.5	97.4	200.5	200.6	132.1	**110.7**
1954	83.2	50.9	37.9	33.1	90.2	93.6	168.8	139.0	172.1	153.9	240.1	153.7	**118.1**
1955	161.3	73.6	46.6	40.2	62.6	154.5	122.1	172.5	198.4	179.0	241.6	164.5	**134.7**
1956	165.8	74.9	52.6	49.9	120.2	155.1	148.5	121.5	168.0	223.4	175.2	120.2	**131.3**
1957	65.0	42.4	32.3	26.4	66.1	66.4	65.8	103.0	109.3	193.7	144.4	124.5	**86.6**
1958	90.7	76.3	50.6	40.9	85.5	96.5	80.9	135.8	121.9	123.8	108.0	108.5	**93.3**
1959	64.6	31.2	15.4	39.9	40.2	197.5	211.1	131.9	119.5	151.7	146.1	271.0	**118.3**
1960	128.9	61.6	50.8	77.4	71.7	100.6	83.5	79.8	110.3	152.0	191.5	275.9	**115.3**
1961	80.3	52.3	36.4	37.0	70.4	92.3	103.1	172.3	115.7	164.6	191.0	199.3	**109.5**
1962	76.2	42.2	26.9	42.6	67.8	92.2	85.2	115.6	105.6	137.3	183.3	131.2	**92.2**
1963	64.0	52.3	37.0	156.2	124.9	114.5	112.9	120.5	100.3	115.6	194.4	97.3	**107.5**
1964	72.8	24.8	20.0	129.5	121.8	157.3	144.6	142.2	141.2	187.3	178.2	67.8	**115.6**
1965	125.4	47.8	23.9	14.1	88.0	55.8	59.8	75.2	73.2	117.0	133.4	160.7	**81.2**
1966	71.7	38.6	29.9	38.5	112.0	137.2	110.6	106.8	86.2	156.6	279.2	223.0	**115.9**
1967	95.5	60.9	29.2	85.4	132.1	181.5	128.1	154.6	159.8	176.6	150.3	113.1	**122.3**
1968	61.9	64.1	64.7	45.2	90.9	129.0	95.9	110.9	125.9	156.0	136.9	143.8	**102.1**
1969	44.6	40.7	18.8	27.7	54.6	84.8	59.6	106.5	115.0	163.7	176.0	164.2	**88.0**
1970	313.0	114.8	55.9	149.3	249.7	86.2	116.4	171.8	191.6	189.4	268.0	353.8	**188.3**
1971	138.2	68.6	96.8	56.7	106.8	139.9	133.9	144.9	168.6	177.1	114.2	59.1	**117.1**
1972	105.3	54.7	39.1	90.0	92.2	70.2	75.6	66.8	123.1	113.5	108.0	71.1	**84.1**
1973	62.4	40.6	19.9	20.7	84.9	125.2	123.6	130.8	176.4	201.9	245.5	200.3	**119.4**
1974	91.6	73.9	46.7	55.8	94.7	90.3	106.5	114.9	104.6	214.5	155.7	103.4	**104.4**
1975	62.6	40.7	23.2	16.4	68.9	72.4	97.3	174.0	215.5	235.9	325.0	283.1	**134.6**
1976	101.5	60.8	37.9	26.2	44.8	44.3	46.0	77.4	137.4	127.5	117.6	49.3	**72.5**
1977	40.1	29.2	17.5	23.5	42.9	80.9	85.1	172.2	130.4	164.6	123.9	72.8	**81.9**
1978	47.0	41.8	28.0	179.2	224.6	106.3	66.7	75.0	137.8	149.5	159.5	93.0	**109.0**
1979	31.3	28.3	21.9	97.5	89.8	143.5	110.3	177.5	144.0	106.2	108.1	157.9	**101.4**
1980	144.8	49.4	24.5	38.8	56.5	86.4	84.2	146.9	109.8	146.8	155.9	161.7	**100.5**
1981	127.0	87.8	68.8	185.8	178.7	149.6	140.8	146.6	95.3	132.2	244.5	341.1	**158.2**
1982	77.8	45.5	31.1	32.8	59.7	85.2	111.5	89.9	100.4	145.2	121.3	61.2	**80.2**
1983	51.3	23.3	16.1	18.2	131.6	81.5	51.9	69.8	154.9	116.3	106.0	131.0	**79.3**
1984	90.2	94.7	68.3	24.5	68.3	112.0	145.2	206.9	164.7	163.8	164.5	79.2	**115.2**
1985	79.6	42.2	38.7	24.1	36.4	141.6	84.6	103.2	99.1	128.5	147.5	169.3	**91.2**
1986	102.6	55.1	37.1	107.0	113.3	114.1	117.3	123.2	134.1	236.2	226.3	83.0	**120.8**
1987	48.1	35.4	24.5	59.4	68.3	65.4	85.9	101.6	115.3	229.6	143.3	96.4	**89.4**
1988	51.3	46.1	23.6	19.9	90.9	89.9	127.5	136.8	137.3	189.1	175.5	96.2	**98.7**
1989	71.1	44.1	34.6	26.6	83.4	106.4	137.2	147.7	137.2	146.1	192.2	126.6	**104.4**
1990	76.8	45.1	30.2	25.1	64.2	66.5	106.2	109.1	160.0	207.1	156.6	205.1	**104.3**
1991	56.6	36.6	79.6	29.0	76.1	96.6	78.7	91.0	152.2	198.5	139.5	132.8	**97.3**
1992	48.6	30.8	21.0	71.1	118.4	125.3	102.6	122.6	176.9	140.6	118.2	85.1	**96.8**
1993	78.3	42.0	47.1	48.6	63.5	128.8	108.5	83.6	204.8	151.5	242.8	165.5	**113.8**
1994	59.2	40.7	35.1	46.9	96.6	150.4	94.4	133.1	162.3	164.6	149.7	75.3	**100.7**
1995	49.8	37.2	30.0	43.3	113.8	108.1	132.2	138.3	163.3	106.6	133.0	180.2	**103.0**
1996	297.4	118.0	74.9	50.0	170.1	167.2	185.4	210.7	232.9	181.3	216.2	319.4	**185.3**
1997	77.0	57.5	36.1	30.6	75.5	63.5	70.9	87.0	81.3	125.8	103.4	53.5	**71.8**
1998	38.2	35.1	27.5	67.4	74.8	71.3	104.0	90.8	113.4	154.3	107.2	140.4	**85.4**
1999	89.5	55.2	38.1	53.4	92.3	110.4	108.8	127.8	141.2	162.8	172.9	149.2	**108.5**
Mean	**89.5**	**52.1**	**37.7**	**53.5**	**91.8**	**108.8**	**107.7**	**125.3**	**139.2**	**162.7**	**173.0**	**149.2**	**107.5**
Maximum	**313.0**	**118.0**	**96.8**	**185.8**	**249.7**	**197.5**	**211.1**	**210.7**	**232.9**	**236.2**	**325.0**	**353.8**	**188.3**
Minimum	**31.3**	**23.3**	**15.4**	**14.1**	**36.4**	**44.3**	**46.0**	**66.8**	**73.2**	**106.2**	**103.4**	**49.3**	**71.8**

MWH

Table 2

MONTHLY MEAN DISCHARGES
RÍO COCLÉ DEL NORTE DAM SITE EXCLUDING TOABRÉ DAM DRAINAGE
(m^3/s)

Drainage Area: 865.4 km^2

Year	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Mean
1948	49.4	30.1	25.2	21.4	32.2	38.8	78.2	85.0	82.7	79.4	115.0	58.1	**58.0**
1949	36.4	27.9	22.1	21.0	38.0	94.0	77.9	87.9	105.4	102.4	137.4	121.3	**72.6**
1950	47.7	35.0	26.7	22.5	63.5	88.0	90.0	109.9	88.4	104.2	119.7	120.0	**76.3**
1951	65.1	47.4	35.0	28.2	64.4	71.8	69.0	72.1	95.8	87.6	109.0	81.4	**68.9**
1952	52.4	35.3	25.2	23.5	47.7	78.7	68.6	70.8	93.5	113.2	88.3	106.4	**67.0**
1953	91.0	54.9	39.2	32.7	70.5	66.5	65.5	59.3	66.4	116.1	116.2	84.4	**71.9**
1954	58.6	39.2	30.6	27.3	62.5	64.3	101.9	87.7	103.4	94.9	133.0	94.8	**74.8**
1955	98.4	53.0	36.4	32.2	46.5	95.2	79.3	103.6	115.2	106.5	133.6	99.9	**83.3**
1956	100.5	53.8	40.3	38.6	78.4	95.5	92.3	79.0	101.5	126.0	104.8	78.4	**82.4**
1967	47.9	33.6	26.7	22.5	48.6	48.8	48.4	69.4	72.7	113.1	90.4	80.5	**58.6**
1958	62.7	54.6	39.0	32.7	59.8	65.9	50.0	81.9	66.8	93.0	58.5	76.9	**61.8**
1959	47.3	23.4	11.2	22.6	32.2	75.3	75.3	88.2	72.5	93.0	90.4	202.2	**69.5**
1960	98.5	50.4	43.9	63.9	44.7	61.8	43.5	78.4	73.2	94.0	112.2	147.4	**76.0**
1961	56.9	40.1	29.6	30.0	56.3	59.8	62.3	106.4	66.6	91.2	130.1	153.2	**73.5**
1962	61.8	36.1	23.2	38.0	53.3	60.9	57.4	69.8	60.4	79.3	122.9	96.5	**63.3**
1963	44.4	41.3	33.1	132.6	87.2	66.5	64.0	65.7	57.8	63.2	131.4	66.8	**71.2**
1964	56.5	16.7	16.6	108.4	83.6	107.1	83.4	75.6	69.8	107.7	82.3	35.0	**70.2**
1965	106.8	35.9	16.2	9.3	87.6	44.2	50.2	50.1	52.8	77.8	84.9	111.5	**60.6**
1966	53.3	28.8	23.3	32.3	74.6	86.3	68.4	70.3	58.7	78.3	185.4	149.4	**75.8**
1967	70.4	48.7	21.9	78.7	104.1	110.5	71.7	94.8	78.3	83.3	88.2	85.3	**78.0**
1968	48.2	55.6	61.2	40.7	77.7	85.1	61.6	62.2	76.1	72.0	54.0	108.0	**66.9**
1969	25.6	30.3	12.7	18.8	38.1	57.3	35.2	66.7	64.0	110.5	115.6	115.9	**57.6**
1970	71.3	46.2	42.3	43.7	103.1	57.3	68.3	103.8	139.1	105.5	117.8	149.7	**87.3**
1971	74.6	55.2	40.7	30.0	82.0	84.9	79.3	84.7	103.7	106.9	67.9	39.5	**70.8**
1972	72.8	42.9	31.4	69.0	76.2	41.5	55.4	48.8	80.9	74.2	65.5	52.5	**59.3**
1973	52.9	34.9	16.0	17.0	68.8	77.7	89.3	91.4	110.1	110.8	132.1	102.0	**75.2**
1974	70.1	38.4	29.0	29.3	85.6	62.2	64.9	75.9	66.2	130.1	101.7	65.4	**68.2**
1975	50.4	29.7	14.9	13.8	55.9	48.6	63.1	112.8	131.4	128.8	191.9	184.2	**85.4**
1976	70.8	50.2	32.6	21.6	33.6	27.7	30.6	46.0	88.4	91.3	68.6	27.9	**49.1**
1977	30.6	23.6	13.7	20.7	33.5	51.5	53.2	106.6	82.4	90.8	77.5	48.0	**52.7**
1978	35.7	35.0	21.4	140.4	145.5	69.2	34.9	37.7	80.6	87.3	84.7	66.6	**69.9**
1979	19.8	22.3	18.7	87.4	61.4	82.2	78.9	107.8	92.4	80.5	77.3	109.3	**69.8**
1980	106.6	37.1	17.6	30.0	36.4	55.1	50.2	83.1	63.7	76.8	95.0	123.5	**64.6**
1981	99.2	69.2	50.6	148.1	116.3	92.1	65.9	107.4	80.0	94.6	157.1	235.7	**109.7**
1982	49.5	33.2	23.8	26.0	46.2	50.9	74.8	56.5	58.7	80.0	74.6	46.2	**51.7**
1983	43.7	18.9	13.7	15.7	98.4	48.0	31.2	44.0	89.9	86.5	78.1	82.0	**54.2**
1984	51.9	78.9	56.5	20.1	51.4	69.7	83.2	140.4	93.4	89.6	98.4	56.5	**74.2**
1985	65.3	34.1	33.7	21.0	29.6	88.9	47.0	52.2	48.2	74.4	77.1	126.5	**58.2**
1986	89.4	47.6	32.3	87.3	86.8	69.9	85.9	79.9	85.3	131.3	127.2	58.4	**81.8**
1987	37.4	28.8	20.9	44.5	49.9	48.2	60.1	68.6	75.9	128.6	89.8	65.8	**59.9**
1988	39.5	36.1	20.4	17.6	62.8	62.3	82.1	86.6	86.9	111.1	105.0	65.7	**64.7**
1989	51.6	34.8	28.4	22.6	58.7	71.2	86.9	91.9	86.9	91.2	112.5	81.6	**68.2**
1990	54.9	35.4	25.2	21.4	47.5	48.8	71.1	72.6	97.8	119.0	96.2	118.1	**67.3**
1991	42.8	29.8	56.5	24.4	54.5	66.0	56.0	62.9	94.1	115.2	88.0	84.7	**64.6**
1992	37.7	25.6	18.4	51.6	77.4	80.9	69.2	79.6	105.6	88.5	77.3	59.6	**64.3**
1993	55.8	33.4	36.8	37.7	47.1	82.7	72.3	58.8	118.0	93.8	134.1	100.3	**72.6**
1994	44.4	32.5	28.8	36.6	65.9	93.2	64.7	84.9	98.8	99.9	92.9	54.0	**66.4**
1995	38.5	30.1	25.2	34.3	75.1	72.1	84.4	87.4	99.3	71.3	84.8	107.1	**67.5**
1996	155.8	77.2	53.8	38.6	102.5	101.2	109.4	120.5	130.0	107.6	122.9	132.7	**104.4**
1997	55.0	43.4	29.4	25.6	54.1	47.1	51.5	60.6	57.5	81.2	69.6	40.8	**51.3**
1998	30.9	28.7	23.3	49.4	53.7	51.7	69.9	62.8	74.8	95.1	71.6	88.4	**58.4**
1999	62.0	44.8	38.6	41.4	72.1	85.6	70.9	109.3	122.8	87.3	106.1	123.9	**80.4**
Mean	**60.4**	**39.4**	**29.5**	**41.2**	**65.1**	**69.4**	**67.3**	**80.0**	**85.9**	**96.5**	**102.8**	**95.6**	**69.4**
Maximum	**155.8**	**78.9**	**61.2**	**148.1**	**145.5**	**110.5**	**109.4**	**140.4**	**139.1**	**131.3**	**191.9**	**235.7**	**109.7**
Minimum	**19.8**	**16.7**	**11.2**	**9.3**	**29.6**	**27.7**	**30.6**	**37.7**	**48.2**	**63.2**	**54.0**	**27.9**	**49.1**

Table 3

LOW COCLE DEL NORTE WATER SUPPLY PROJECT

SUMMARY COST ESTIMATE

	US$ million
1. ACCESS ROADS AND CONSTRUCTION CAMP	$ 6.65
2. LOWER COCLE DEL NORTE STORAGE PROJECT	$ 50.96
3. PUMPING FACILITIES	$ 87.29
4. RESERVOIR CLEARING	$ 5.92
SUBTOTAL DIRECT COSTS	$ 150.8
Contingency	$ 27.1
DIRECT COST	$ 177.9
Engineering and Administration	$ 27.1
CONSTRUCTION COST (Jan 2003 price level)	$ 205.0
COMPENSATION AND MITIGATION COST	$ 23.0
TOTAL COST	$ 228.0

LOW COCLE DEL NORTE WATER SUPPLY PROJECT

CONSTRUCTION COST ESTIMATE SUMMARY

1. ACCESS ROADS AND CONSTRUCTION CAMP		
1 General	$	360,000
2 Access Roads	$	4,790,000
3 Construction Camps	$	1,500,000
Subtotal	$	6,650,000
2. LOWER COCLE DEL NORTE STORAGE PROJECT		
1 General	$	2,150,000
2 Diversion	$	8,620,000
3 Dam	$	13,830,000
4 Spillway	$	10,910,000
5 Drawdown Facilities	$	2,650,000
6 Operation Facilities	$	1,120,000
7 Coclecito Protection Works	$	11,680,000
Subtotal	$	50,960,000
3. PUMPING FACILITIES		
1 General	$	2,660,000
2 Civil Works	$	11,510,000
3 Electro-mechanical Equipment	$	56,220,000
4 Transmission System	$	16,900,000
Subtotal	$	87,290,000
4. RESERVOIR CLEARING		
1 General	$	200,000
2 Clearing	$	5,720,000
Subtotal	$	5,920,000
SUBTOTAL DIRECT COSTS	**$**	**150,820,000**
Contingency	$	27,080,000
DIRECT COST	**$**	**177,900,000**
Engineering and Administration	$	27,100,000
CONSTRUCTION COST (Jan 2003 price level)	**$**	**205,000,000**

LOW COCLE DEL NORTE WATER SUPPLY PROJECT

ACCESS ROADS AND CONSTRUCTION CAMPS

Description		Unit	Unit Cost	Quantity	Amount
1 GENERAL					
1.1 Mobilization and Demobilization		LS	$250,000	1	$250,000
1.2 General Maintenance		LS			$110,000
	Subtotal 1				**$360,000**
2 ACCESS ROADS					
2.1 Permanent Roads		km	$147,600	25	$3,690,000
2.3 Bridge over Rio Miguelito		LS	$350,000	1	$350,000
2.3 Bridge over Cocle del Norte		LS	$750,000	1	$750,000
	Subtotal 2				**$4,790,000**
3 CONSTRUCTION CAMPS					
3.1 Cocle del Norte		LS	$1,500,000	1	$1,500,000
3.2 Pumping Station		Included in Pumping Facilities Contract			
	Subtotal 4				**$1,500,000**
Subtotal Direct Cost (rounded)		LS			**$6,650,000**
Contingency		LS			**$1,000,000**
Direct Cost		LS			**$7,650,000**

LOW COCLE DEL NORTE WATER SUPPLY PROJECT

Description	Unit	Unit Cost	Quantity	Amount
1 GENERAL				
1.1 Mobilization and Demobilization	LS	$1,500,000	1	$1,500,000
1.2 Temporary Access	km	$115,200	3	$345,600
1.2 Temporary Facilities	LS	$300,000	1	$300,000
Subtotal 1				**$2,145,600**
2 DIVERSION				
2.1 Site Preparation	m^2	$0.55	40,000	$22,000
2.2 Diversion Culverts				
2.2.1 Overburden Excavation	m^3	$3.70	61,800	$228,660
2.2.2 Rock Excavation	m^3	$9.20	61,800	$568,560
2.2.3 Concrete	m^3	$116.00	22,428	$2,601,648
2.2.4 Concrete (High Strength)	m^3	$145.00	2,243	$325,206
2.2.5 Backfill Concrete	m^3	$105.00	3,220	$338,100
2.2.6 Formwork	m^2	$46.80	17,200	$804,960
2.2.7 Reinforcement	kg	$1.32	1,400,000	$1,848,000
2.3 Cofferdams				
2.3.1 Overburden Excavation	m^3	$3.20	40,800	$130,560
2.3.2 Fill	m^3	$7.30	220,715	$1,611,220
2.3.3 Filter/Drain	m^3	$16.10	8,785	$141,439
Subtotal 2				**$8,620,352**
3 DAM				
3.1 Site Preparation	m^2	$0.55	20,000	$11,000
3.2 Foundation Excavation				
3.2.1 Overburden Excavation	m^3	$3.70	80,000	$296,000
3.2.2 Rock Excavation	m^3	$9.20	39,600	$364,320
3.3 Foundation Treatment				
3.3.1 Surface Treatment	LS	$150,000.00	1	$150,000
3.3.2 Consolidation Grouting	m	$69.20	2,750	$190,300
3.3.1 Curtain Grouting	m^2	$46.00	7,000	$322,000
3.3.2 Drainage	m	$50.00	1,400	$70,000
3.4 Dam Section				
3.4.1 Roller Compacted Concrete	m^3	$56.00	170,000	$9,520,000
3.4.2 Uncompacted Concrete	m^3	$56.00	10,400	$582,400
3.4.3 U\S, D\S Face Precast Panels and Membrane	m^3	$282.00	5,200	$1,466,400
3.4.4 Bedding Mix (grout enriched RCC)	m^3	$70.00	3,720	$260,400
3.4.5 Gallery Concrete	m^3	$210.00	2,850	$598,500
Subtotal 3				**$13,831,320**

LOW COCLE DEL NORTE WATER SUPPLY PROJECT

Description	Unit	Unit Cost	Quantity	Amount
4 SPILLWAY				
4.1 Headworks				
4.1.1 Concrete	m^3	$116.00	14,600	$1,693,600
4.1.2 Formwork	m^2	$46.80	9,010	$421,668
4.1.3 Reinforcement	kg	$1.32	745,000	$983,400
4.2 Chute and Stilling Basin				
4.2.1 Concrete	m^3	$116.00	20,300	$2,354,800
4.2.2 Formwork	m^2	$46.80	10,800	$505,440
4.2.3 Reinforcement	kg	$1.32	980,500	$1,294,260
4.3 Bridge				
4.3.1 Concrete	m^3	$145.00	550	$79,750
4.3.2 Formwork	m^2	$46.80	690	$32,292
4.3.3 Reinforcement	kg	$1.32	86,600	$114,312
4.4 Gates				
4.4.1 Radial Gates and Embedded Parts	Each	$270,000	6	$1,620,000
4.4.2 Gate Hoists	Each	$230,000	6	$1,380,000
4.4.3 Electrical Supply	LS	$100,000	1	$100,000
4.4.4 Stoplogs	Each	$332,500	1	$332,500
4.5 Stilling Basin Excavation				
4.5.1 Overburden Excavation	m^3	$3.70	Included in	$0
4.5.2 Rock Excavation	m^3	$9.20	2.2.1 and 2.2.2	$0
Subtotal 4				**$10,912,022**
5 EMERGENCY DRAWDOWN FACILITY				
5.1 Tower and Gate Structure				
5.1.1 Concrete	m^3	$116.00	936	$108,576
5.1.2 Concrete (High Strength)	m^3	$145.00	90	$13,050
5.1.5 Formwork	m^2	$46.80	2,300	$107,640
5.1.6 Reinforcement	kg	$1.32	81,700	$107,844
5.2 Culvert				
5.2.1 Concrete Plug	m^3	$116.00	980	$113,680
5.2.2 Stop Logs and Embeds	LS	$100,000	1	$100,000
5.3 Gates and Valves				
5.3.1 Wheeled Gate and Hoist (3.0 m x 6.0 m)	Each	$440,000	4	$1,760,000
5.3.2 Valve 1.6-m diameter	Each	$240,000	1	$240,000
5.3.2 Power and Controls	LS	$100,000	1	$100,000
Subtotal 5				**$2,650,790**

Description	Unit	Unit Cost	Quantity	Amount
6 OPERATION FACILITIES				
6.1 Emergency Generator	Each	$35,000	2	$70,000
6.2 SCADA	LS	$200,000	1	$200,000
6.3 Other Communication Facilities	LS	$150,000	1	$150,000
6.4 Instrumentation	LS	$200,000	1	$200,000
6.5 Lighting, Landscaping, and Drainage	LS	$200,000	1	$200,000
6.6 Unspecified Facilities	LS	$300,000	1	$300,000
Subtotal 6				**$1,120,000**
7 COCLECITO PROTECTION WORKS				
7.1 Temporary Facilities	L.S.	$50,000	1	$50,000
7.2 Excavation	m^3	$3.20	855,000	$2,736,000
7.3 Berms	m^3	$7.30	1,050,000	$7,665,000
7.4 Bridge	L.S.	$250,000	1	$250,000
7.4 Pumping Stations (3)	L.S.	$350,000	1	$350,000
7.5 Road	km	$147,600	0.5	$73,800
7.6 Miscellaneous	%		5%	$555,000
Subtotal 7				**$11,679,800**
Subtotal Direct Cost (rounded)				**$50,960,000**
Contingency				**$7,800,000**
Direct Cost				**$58,760,000**

PUMPING FACILITIES

Description	Unit	Unit Cost	Quantity	Amount
1 GENERAL				
1.1 Mobilization and Demobilization	LS			$1,500,000
1.2 Temporary Access	km	$115,200	1	$115,200
1.3 Construction Camp	Each	$750,000	1	$750,000
1.4 Temporary Facilities	Each	$295,000	1	$295,000
Subtotal 1				**$2,660,200**
1 CIVIL WORKS				
1.1 Excavation				
1.1.1 Cellular Cofferdam	LS	$1,000,000	1	$1,000,000
1.1.2 Overburden Excavation	m^3	$3.70	160,000	$592,000
1.1.3 Rock Excavation	m^3	$9.20	240,000	$2,208,000
1.2 Pumping Station				
1.2.1 Mass Concrete	m^3	$116.00	20,900	$2,424,400
1.2.2 Structural Concrete	m^3	$145.00	2,000	$290,000
1.2.3 Formwork		$46.80	22,900	$1,071,720
1.2.4 Steel Reinforcement		$1,320	916.0	$1,209,120
1.2.5 Roof, siding, windows, doors, etc		$310	5,900	$1,829,000
1.2.6 Miscellaneous	%	5%		$340,000
1.3 Ancillary Facilities				
1.3.1 Access Road	km	$145,000	1	$145,000
1.3.2 Parking Area	LS	$50,000	1	$50,000
1.3.3 Lighting, Landscaping, Drainage etc.	LS	$100,000	1	$100,000
1.3.4 Modifications to Low Level Outlet	LS	$250,000	1	$250,000
Subtotal 2				**$11,509,240**
3 PUMPING EQUIPMENT				
3.1 Pumping Equipment				
3.1.1 Trashracks	Each	$25,000	16	$400,000
3.1.2 Draft Tube Gates (3m x 6 m)	Each	$125,000	4	$500,000
3.1.3 Upper Stage Pump/Motor Unit (26 MW)	Each	$5,800,000	4	$23,200,000
3.1.4 Lower Stage Pump/Motor Unit (13.5 MW)	Each	$3,700,000	4	$14,800,000
3.1.5 Main Valves (2.30 m dia)	Each	$450,000	4	$1,800,000
3.1.6 Main Valves (2.75 m dia)	Each	$550,000	4	$2,200,000
3.1.7 Penstocks	Ton	$6,000	600	$3,600,000
3.1.8 Guard Valves (2.75 m dia)	Each	$350,000	4	$1,400,000

Description	Unit	Unit Cost	Quantity	Amount
3.2 Miscellaneous Mechanical				
3.2.1 Gantry Crane	Each	$750,000	1	$750,000
3.2.2 Draft Tube Gate Crane	Each	$250,000	1	$250,000
3.2.3 General	LS	$2,500,000	1	$2,500,000
3.3 Miscellaneous Electrical				
3.3.1 Main Power Transformer -30 MVA	Each	$400,000	4	$1,600,000
3.3.2 Take-off Structures & OH lines to Switchyard	LS	$250,000	1	$250,000
3.3.3 Switchgear - 13.8 kV	LS	$120,000	1	$120,000
3.3.4 Station Service Transformer	Each	$50,000	4	$200,000
3.3.5 Stand-by Diesel Generator	Each	$250,000	2	$500,000
3.3.6 Station Auxiliaries (light, HVAC, etc.)	LS	$750,000	1	$750,000
3.3.7 Control and Communication Equip	LS	$1,000,000	1	$1,000,000
3.3.8 Cabling, MV & LV Power, Cont/Comm	LS	$400,000	1	$400,000
Subtotal 3				**$56,220,000**
4 TRANSMISSION SYSTEM				
4.1 Toabre Substation	LS	$3,500,000	1	$3,500,000
4.2 Penonome Switchyard	LS	$2,500,000	1	$2,500,000
4.3 230-kV Transmission Line				
4.3.1 Civil Works (survey, Found., Struc.)	km	$62,500	65	$4,062,500
4.3.2 Conductors and Shield Wire	km	$62,500	65	$4,062,500
4.3.3 Insulators and Accessories	km	$31,250	65	$2,031,250
4.3.4 Grounding and Miscellaneous	LS	$400,000	1	$400,000
4.4 13.8 kV Transmission Line to Cocle del Norte	km	$17,000	20	$340,000
Subtotal 4				**$16,896,250**

Subtotal Direct Cost (rounded)	**$87,290,000**
Contingency	**$17,390,000**
Direct Cost	**$104,680,000**

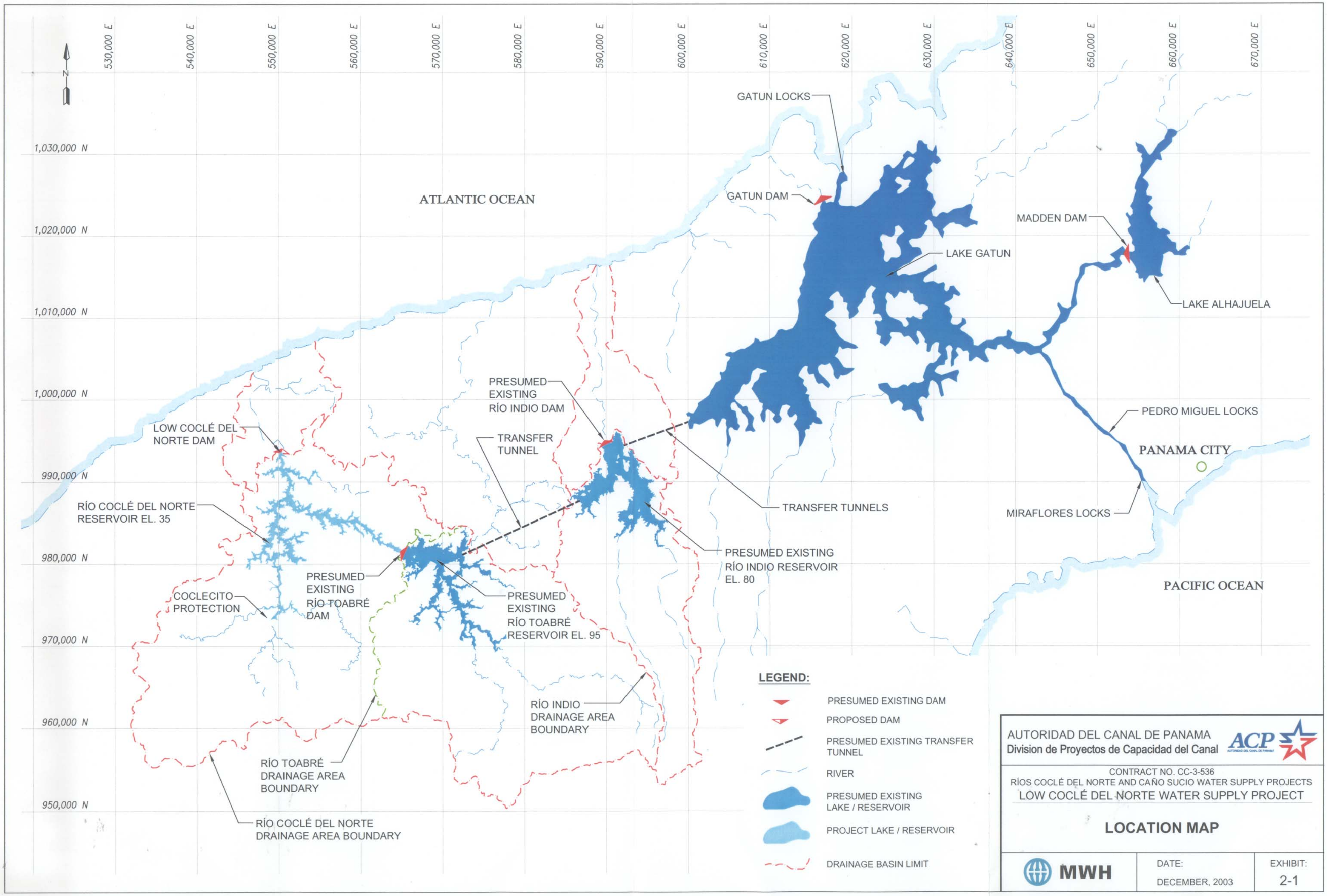
RESERVOIR CLEARING

Description	Unit	Unit Cost	Quantity	Amount
1 GENERAL				
1.1 Mobilization and Demobilization	LS			$150,000
1.2 Temporary Facilities	LS			$50,000
Subtotal 1				**$200,000**
2 RESERVOIR CLEARING				
2.1 Cocle del Norte Reservoir FSL 35 (to El. 40)	ha	$2,200	2,600	$5,720,000
Subtotal Direct Cost (rounded)				**$5,920,000**
Contingency				**$890,000**
Direct Cost				**$6,810,000**

Exhibits

EXHIBITS

No.	Title
2-1	Project Location Map
2-2	Regional Isohyetal Map
2-3	Drainage Configuration of the Coclé del Norte Basin
2-4	Regional Geologic Map
3-1	Location of Stream Gages and Rainfall Stations
3-2	Probable Maximum Flood Hydrograph, Entire Basin
3-3	Probable Maximum Flood Hydrograph, Excluding Toabré Dam Drainage
3-4	Seismicity of Panama
3-5	Area and Volume vs. Elevation
5-1	General Plan of Development
5-2	Low Coclé del Norte Dam, Plan
5-3	Low Coclé del Norte Dam, Profile, Typical Cross Sections and Details
5-4	Low Coclé del Norte Spillway, Plan, Sections and Upstream Elevation
5-5	Low Coclé del Norte River Diversion Facilities, Plan and Typical Cross Sections
5-6	Low Coclé del Norte Diversion and Emergency Drawdown Facilities, Profile and Details
5-7	Low Coclé del Norte Project, Coclecito Protection, Plan
5-8	Low Coclé del Norte Project, Coclecito Protection, Plan and Sections
5-9	Toabré Pumping Station, Plan and Elevation
5-10	Toabré Pumping Station, Typical Sections and Plan at El. 29
5-11	Low Coclé del Norte Project, Access Roads and Transmission Line Alignment
6-1	Implementation Schedule
6-2	Construction Schedule (2 Sheets)



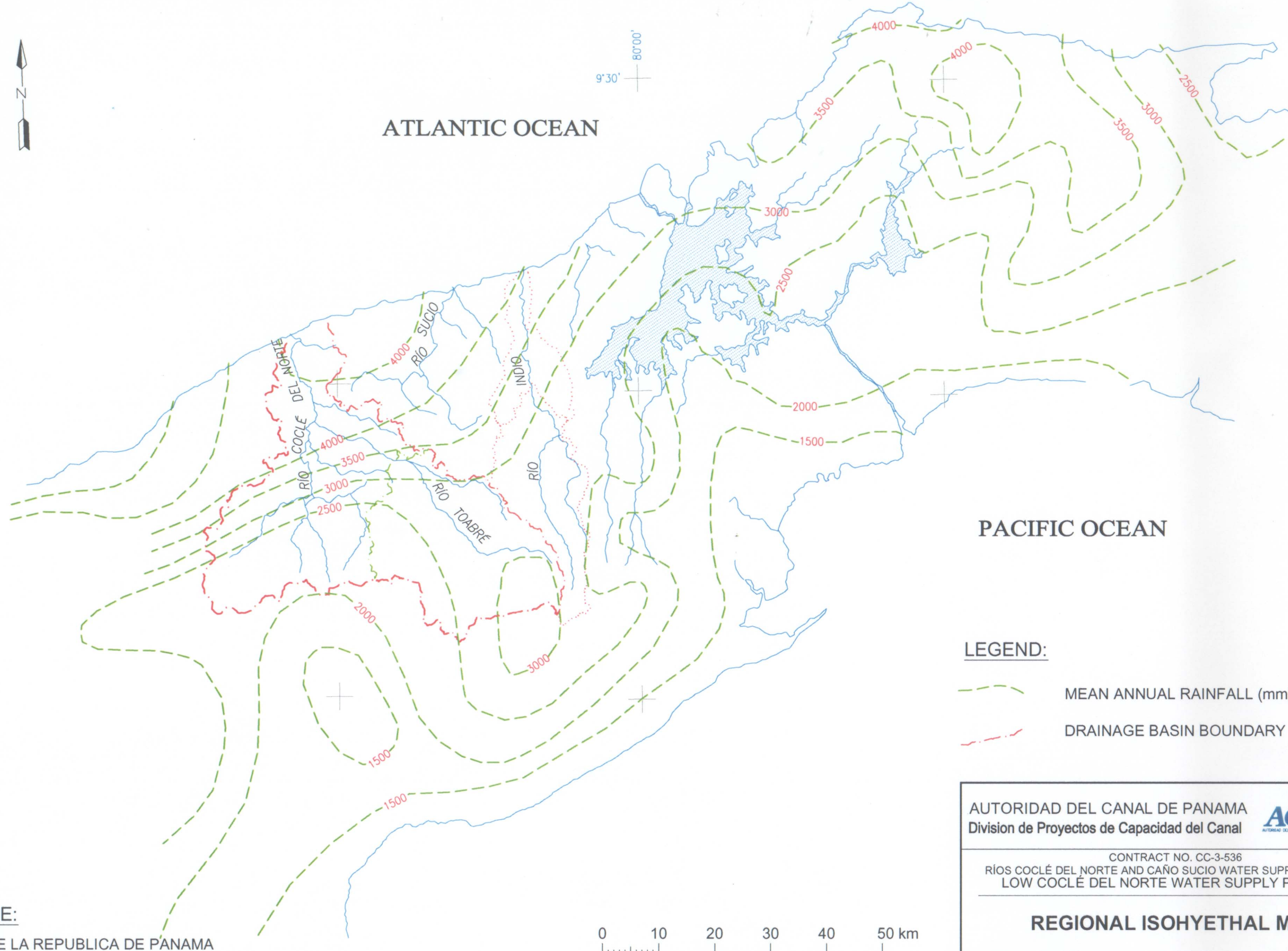
AUTORIDAD DEL CANAL DE PANAMA
Division de Proyectos de Capacidad del Canal
ACP

CONTRACT NO. CC-3-536
RÍOS COCLÉ DEL NORTE AND CAÑO SUCIO WATER SUPPLY PROJECTS
LOW COCLÉ DEL NORTE WATER SUPPLY PROJECT

LOCATION MAP

MWH

DATE:	EXHIBIT:
DECEMBER, 2003	2-1



LEGEND:

MEAN ANNUAL RAINFALL (mm)

DRAINAGE BASIN BOUNDARY

SOURCE:

ATLAS DE LA REPUBLICA DE PANAMA
INSTITUTO GEOGRAFICO NACIONAL, "TOMMY GUARDIA"

AUTORIDAD DEL CANAL DE PANAMA
Division de Proyectos de Capacidad del Canal

CONTRACT NO. CC-3-536
RÍOS COCLÉ DEL NORTE AND CAÑO SUCIO WATER SUPPLY PROJECTS
LOW COCLÉ DEL NORTE WATER SUPPLY PROJECT

REGIONAL ISOHYETHAL MAP

MWH

DATE:	EXHIBIT:
DECEMBER, 2003	2-2

LEGEND:

▲ RAIN GAGE

⊙ STREAM GAGE

— · — DRAINAGE BASIN BOUNDARY

■ VILLAGE

AUTORIDAD DEL CANAL DE PANAMA
Division de Proyectos de Capacidad del Canal

ACP
AUTORIDAD DEL CANAL DE PANAMA

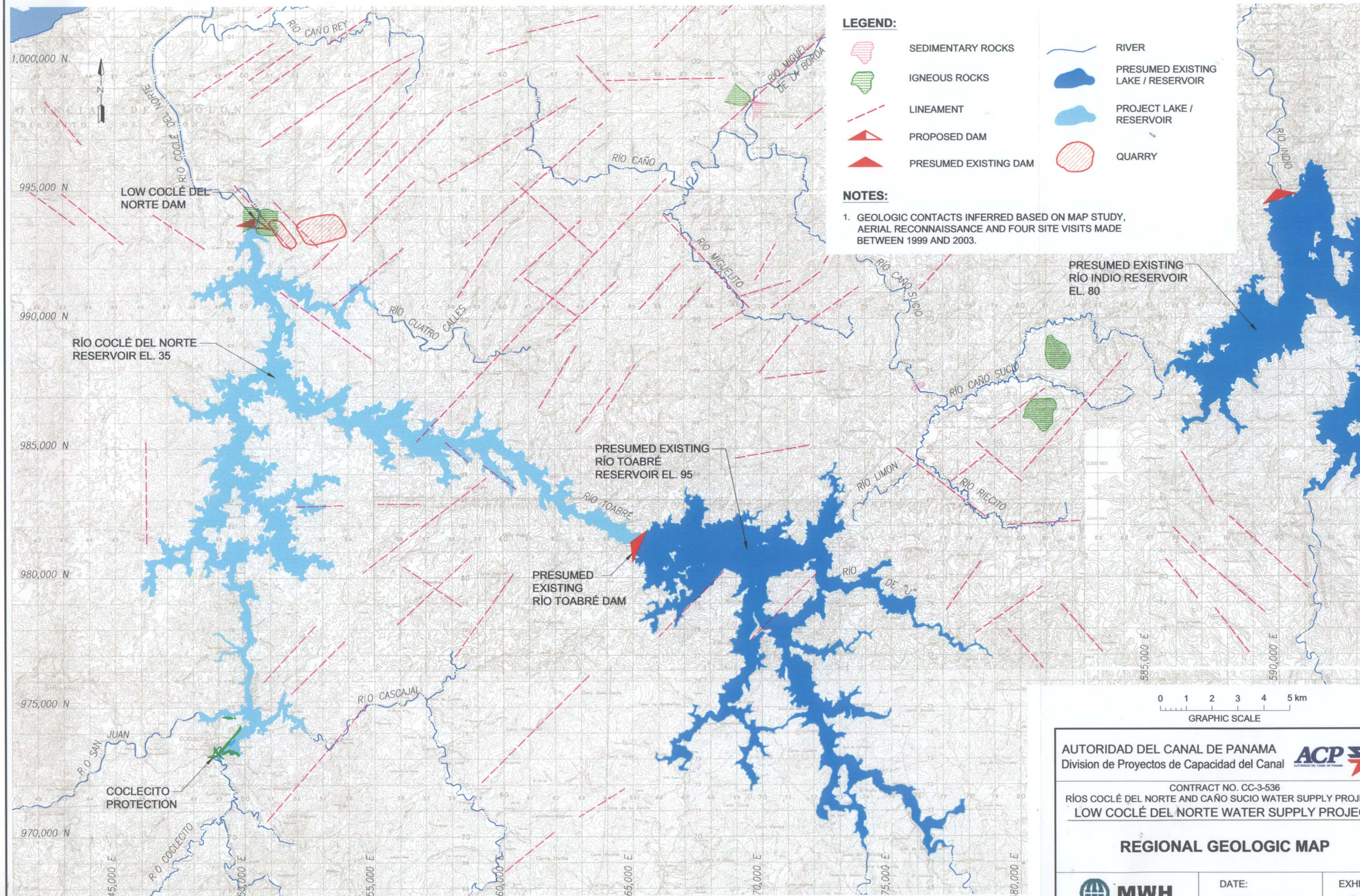
CONTRACT NO. CC-3-536
RÍOS COCLÉ DEL NORTE AND CAÑO SUCIO WATER SUPPLY PROJECTS
LOW COCLÉ DEL NORTE WATER SUPPLY PROJECT

DRAINAGE CONFIGURATION OF THE COCLÉ DEL NORTE BASIN

MWH

DATE: DECEMBER, 2003

EXHIBIT: 2-3



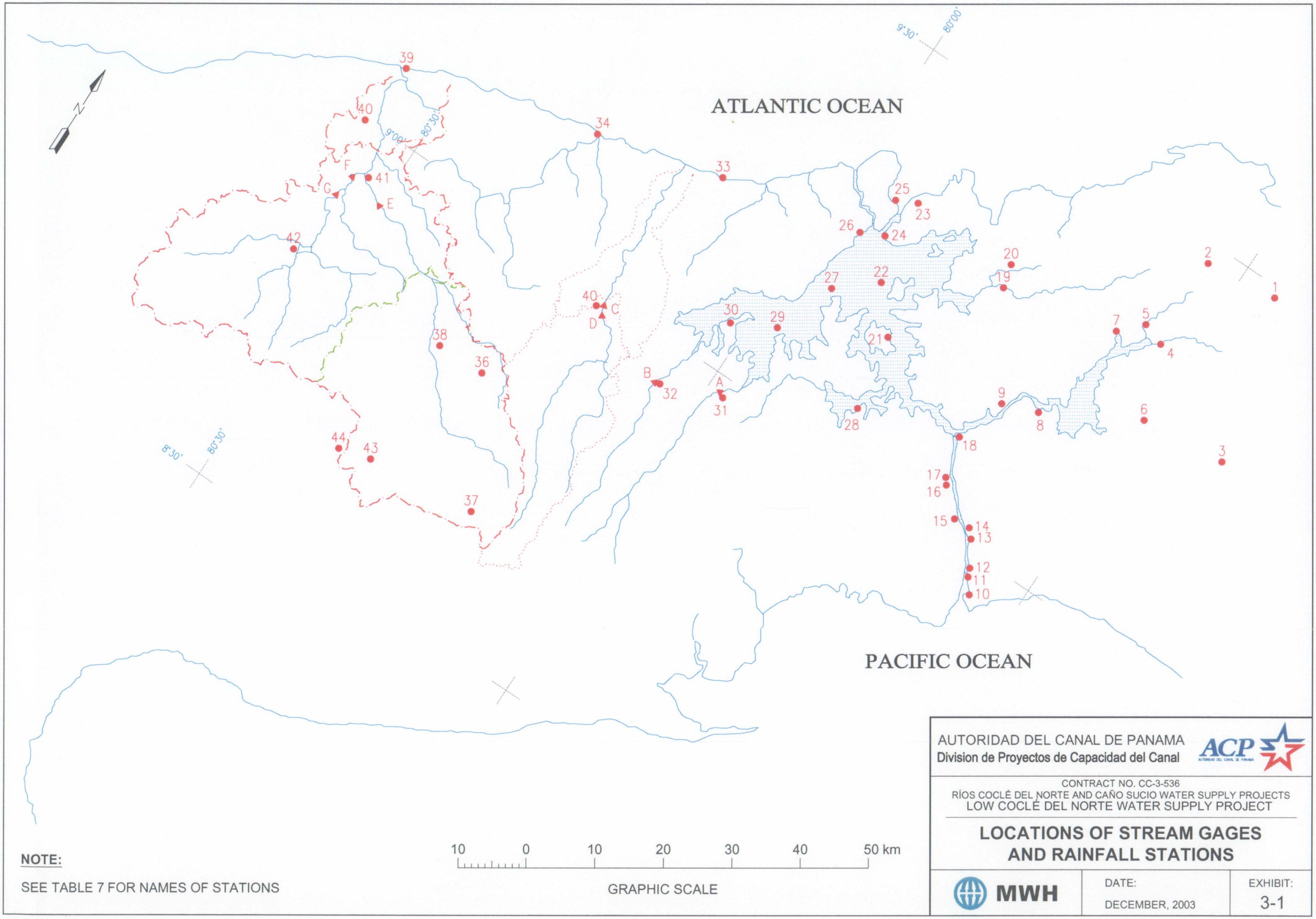
AUTORIDAD DEL CANAL DE PANAMA
Division de Proyectos de Capacidad del Canal

CONTRACT NO. CC-3-536
RÍOS COCLÉ DEL NORTE AND CAÑO SUCIO WATER SUPPLY PROJECTS
LOW COCLÉ DEL NORTE WATER SUPPLY PROJECT

REGIONAL GEOLOGIC MAP

MWH

DATE:	EXHIBIT:
DECEMBER, 2003	2-4



NOTE:

SEE TABLE 7 FOR NAMES OF STATIONS

AUTORIDAD DEL CANAL DE PANAMA
Division de Proyectos de Capacidad del Canal

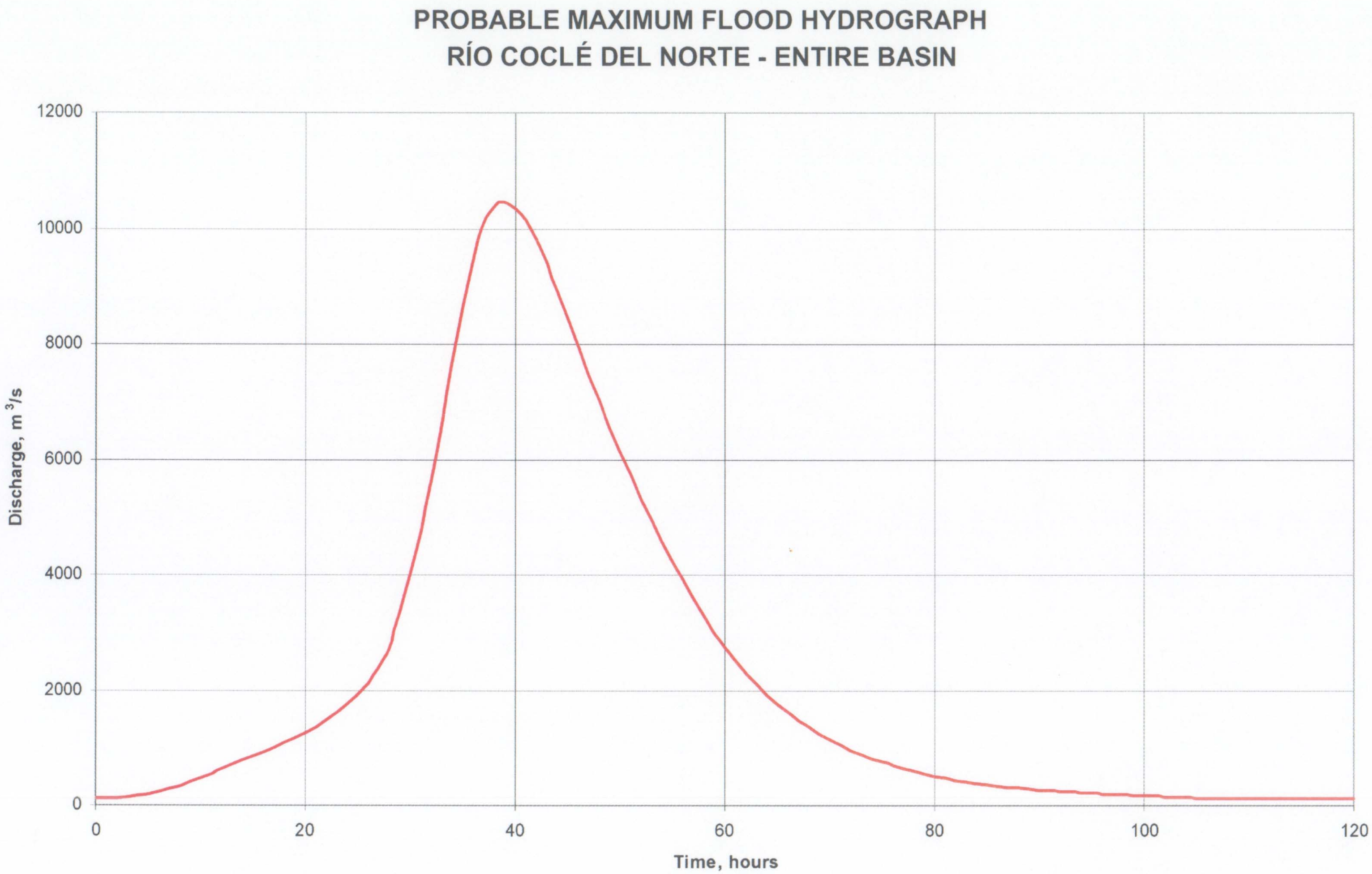
CONTRACT NO. CC-3-536
RÍOS COCLÉ DEL NORTE AND CAÑO SUCIO WATER SUPPLY PROJECTS
LOW COCLÉ DEL NORTE WATER SUPPLY PROJECT

LOCATIONS OF STREAM GAGES AND RAINFALL STATIONS

MWH

DATE: DECEMBER, 2003

EXHIBIT: 3-1



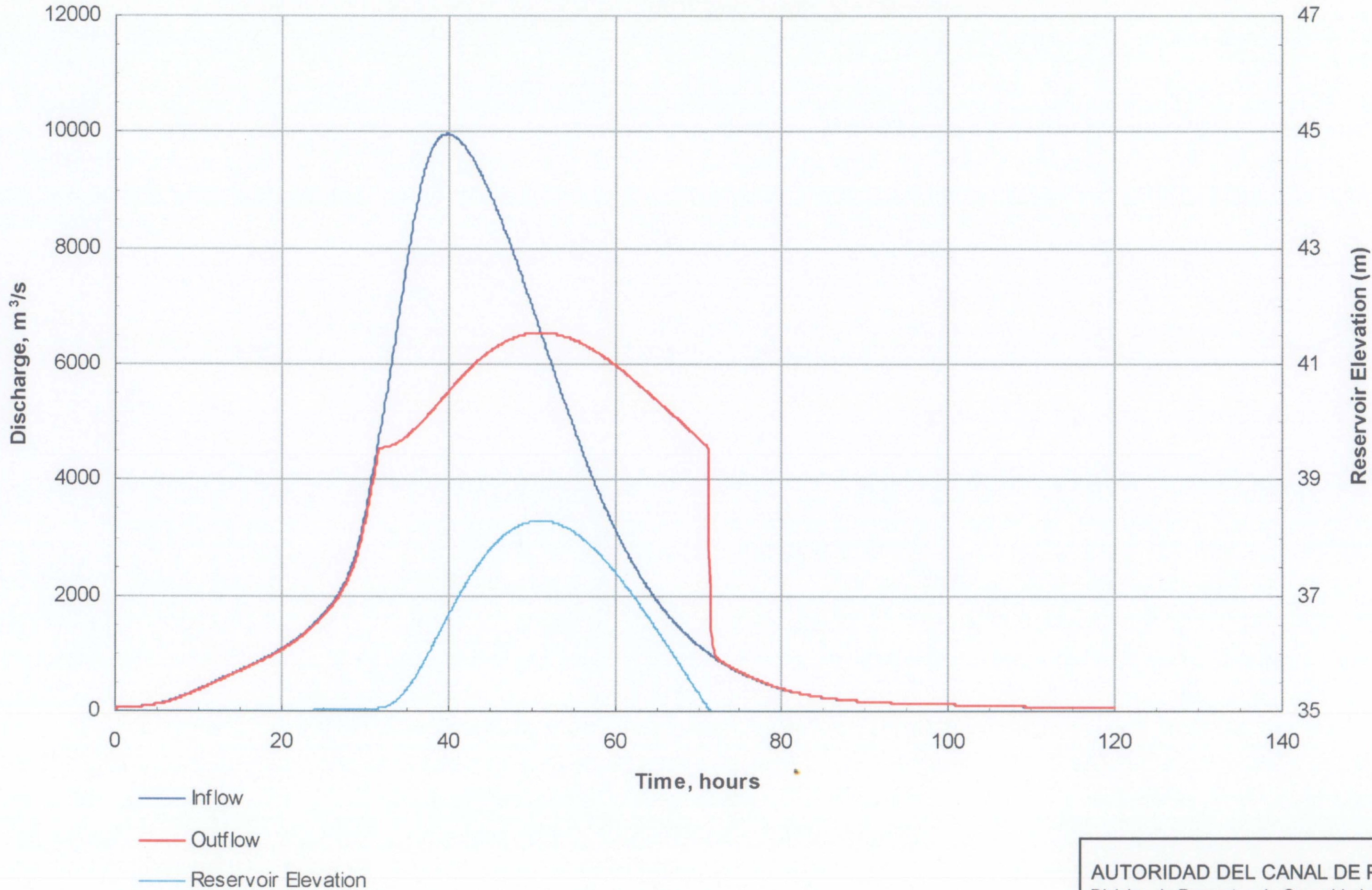
AUTORIDAD DEL CANAL DE PANAMA
Division de Proyectos de Capacidad del Canal

CONTRACT NO. CC-3-536
RÍOS COCLÉ DEL NORTE AND CAÑO SUCIO WATER SUPPLY PROJECTS
LOW COCLÉ DEL NORTE WATER SUPPLY PROJECT

PROBABLE MAXIMUM FLOOD HYDROGRAPH - ENTIRE BASIN

MWH	DATE: DECEMBER, 2003	EXHIBIT: 3-2

PROBABLE MAXIMUM FLOOD HYDROGRAPH
RÍO COCLÉ DEL NORTE - EXCLUDING TOABRÉ DAM DRAINAGE



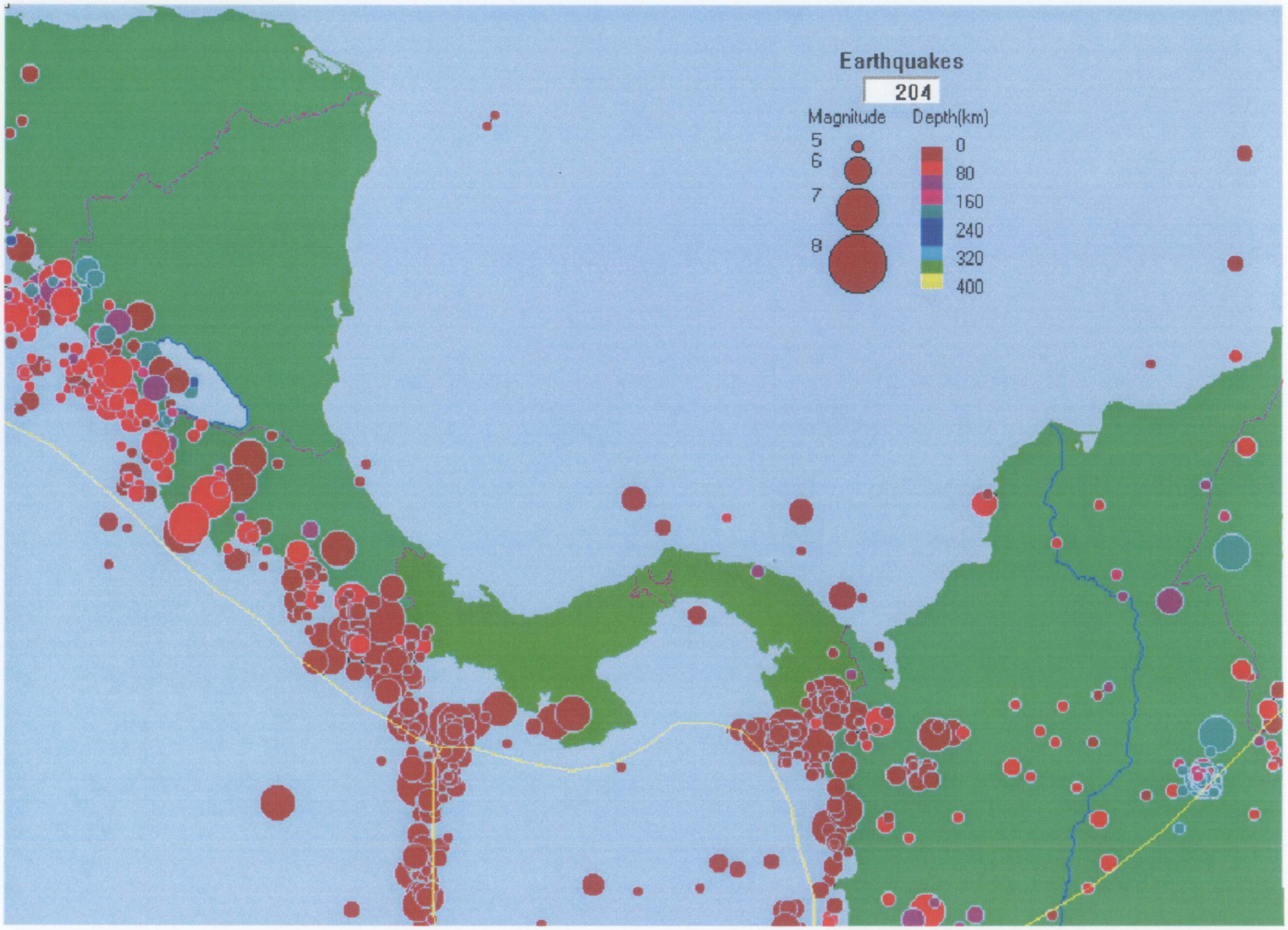
AUTORIDAD DEL CANAL DE PANAMA
Division de Proyectos de Capacidad del Canal

ACP

CONTRACT NO. CC-3-536
RÍOS COCLÉ DEL NORTE AND CAÑO SUCIO WATER SUPPLY PROJECTS
LOW COCLÉ DEL NORTE WATER SUPPLY PROJECT

PROBABLE MAXIMUM FLOOD HYDROGRAPH EXCLUDING TOABRÉ DAM DRAINAGE

MWH	DATE: DECEMBER, 2003	EXHIBIT: 3-3



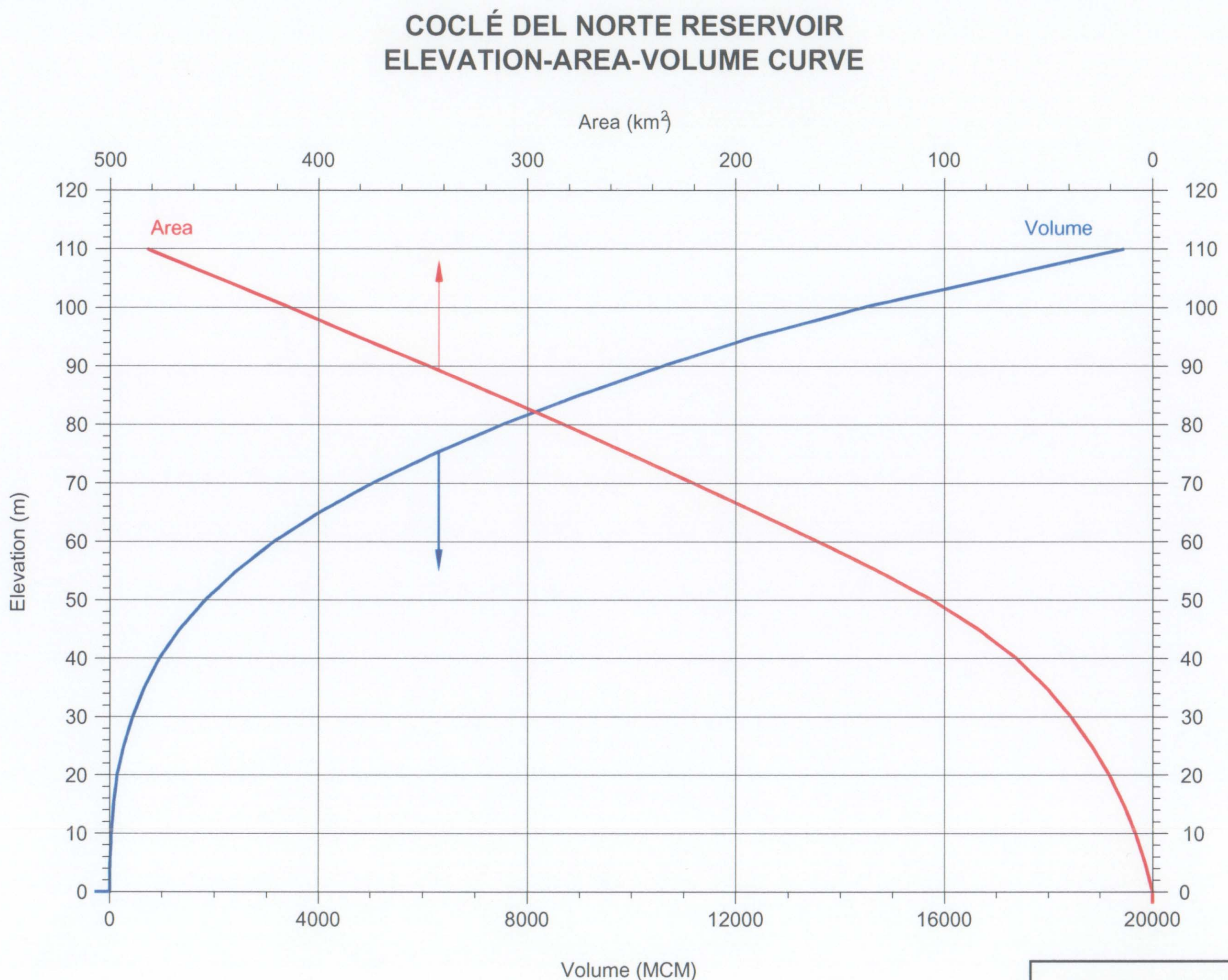
AUTORIDAD DEL CANAL DE PANAMA
Division de Proyectos de Capacidad del Canal

ACP

CONTRACT NO. CC-3-536
RÍOS COCLÉ DEL NORTE AND CAÑO SUCIO WATER SUPPLY PROJECTS
LOW COCLÉ DEL NORTE WATER SUPPLY PROJECT

SEISMICITY OF PANAMA

MWH	DATE: DECEMBER, 2003	EXHIBIT: 3-4

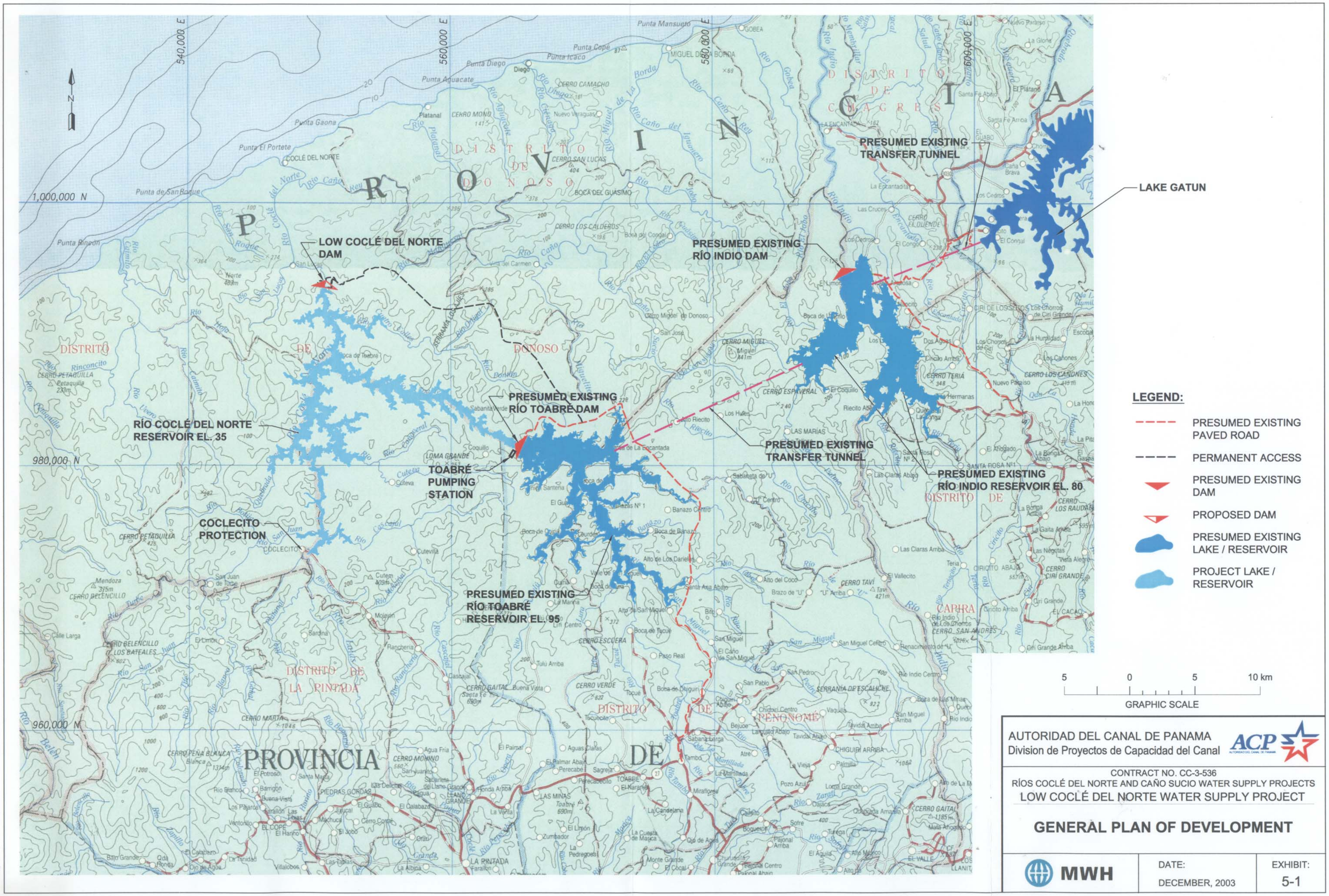


AUTORIDAD DEL CANAL DE PANAMA
Division de Proyectos de Capacidad del Canal

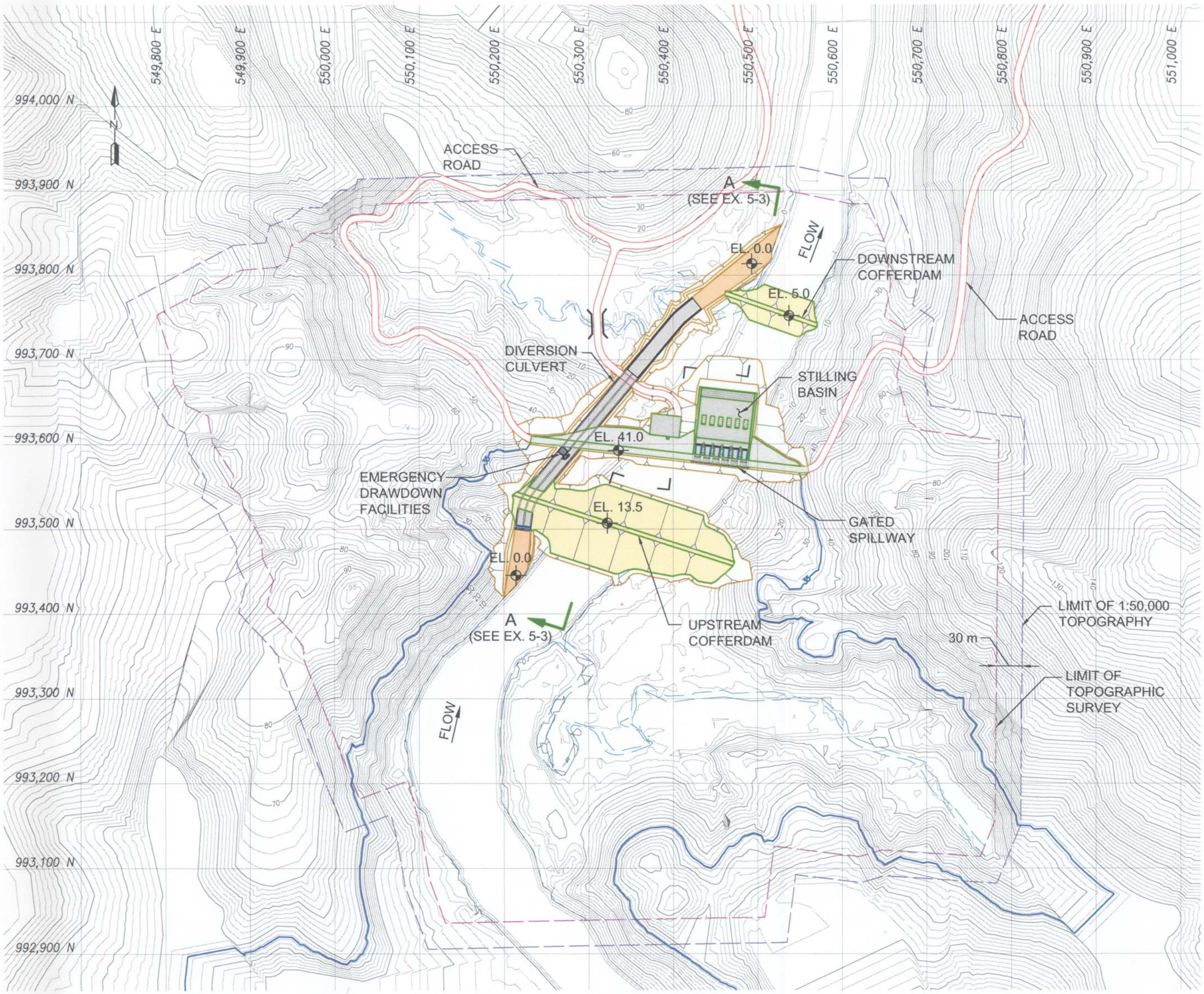
CONTRACT NO. CC-3-536
RÍOS COCLÉ DEL NORTE AND CAÑO SUCIO WATER SUPPLY PROJECTS
LOW COCLÉ DEL NORTE WATER SUPPLY PROJECT

AREA AND VOLUME VS. ELEVATION

MWH	DATE: DECEMBER, 2003	EXHIBIT: 3-5



LOW COCLÉ DEL NORTE DAM
PRESUMED EXISTING TRANSFER TUNNEL
LAKE GATUN
PRESUMED EXISTING RÍO INDIO DAM
PRESUMED EXISTING RÍO TOABRE DAM
RÍO COCLÉ DEL NORTE RESERVOIR EL. 35
TOABRE PUMPING STATION
PRESUMED EXISTING TRANSFER TUNNEL
PRESUMED EXISTING RÍO INDIO RESERVOIR EL. 80
COCLECITO PROTECTION
PRESUMED EXISTING RÍO TOABRE RESERVOIR EL. 95
PROVINCIA
PROVINCIA DE
540,000 E
560,000 E
580,000 E
600,000 E
1,000,000 N
980,000 N
960,000 N
N
LEGEND:
PRESUMED EXISTING PAVED ROAD
PERMANENT ACCESS
PRESUMED EXISTING DAM
PROPOSED DAM
PRESUMED EXISTING LAKE / RESERVOIR
PROJECT LAKE / RESERVOIR
5 0 5 10 km
GRAPHIC SCALE
AUTORIDAD DEL CANAL DE PANAMA
Division de Proyectos de Capacidad del Canal
ACP
CONTRACT NO. CC-3-536
RÍOS COCLÉ DEL NORTE AND CAÑO SUCIO WATER SUPPLY PROJECTS
LOW COCLÉ DEL NORTE WATER SUPPLY PROJECT
GENERAL PLAN OF DEVELOPMENT
MWH
DATE:
DECEMBER, 2003
EXHIBIT:
5-1



SITE PLAN

AUTORIDAD DEL CANAL DE PANAMA
Division de Proyectos de Capacidad del Canal

CONTRACT NO. CC-3-536
RÍOS COCLÉ DEL NORTE AND CAÑO SUCIO WATER SUPPLY PROJECTS
LOW COCLÉ DEL NORTE WATER SUPPLY PROJECT

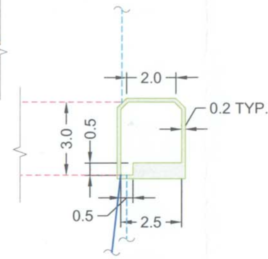
LOW COCLÉ DEL NORTE DAM PLAN

MWH

DATE:	EXHIBIT:
DECEMBER, 2003	5-2

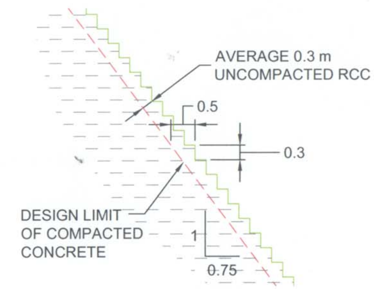
UPSTREAM ELEVATION

0 25 50
GRAPHIC SCALE



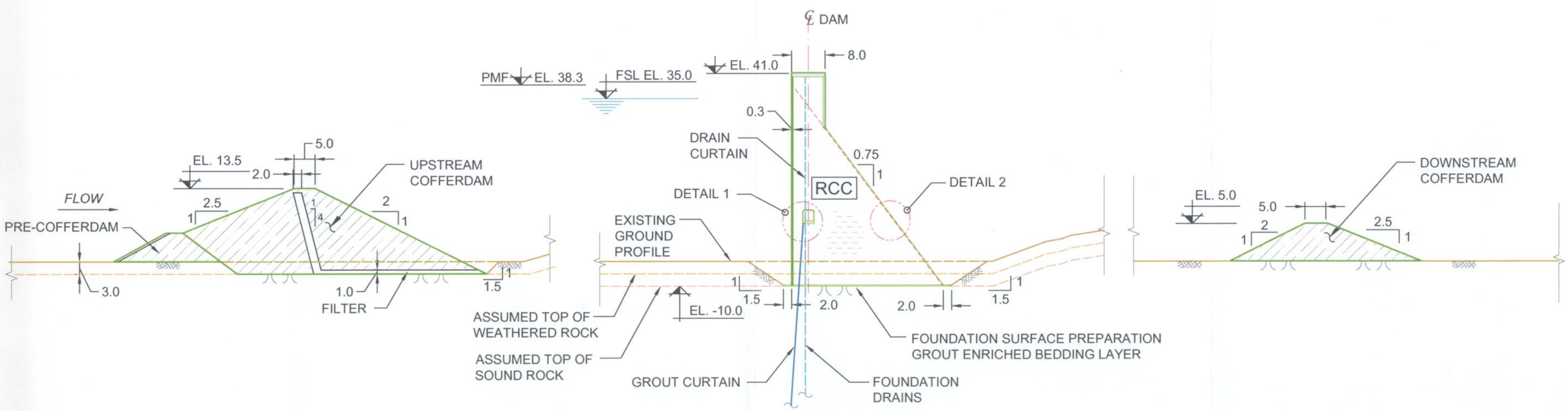
DETAIL 1

0 2 4
GRAPHIC SCALE



DETAIL 2

0 1 2
GRAPHIC SCALE



CROSS SECTION A - A

(SEE EXHIBIT 5-2)

0 10 20
GRAPHIC SCALE

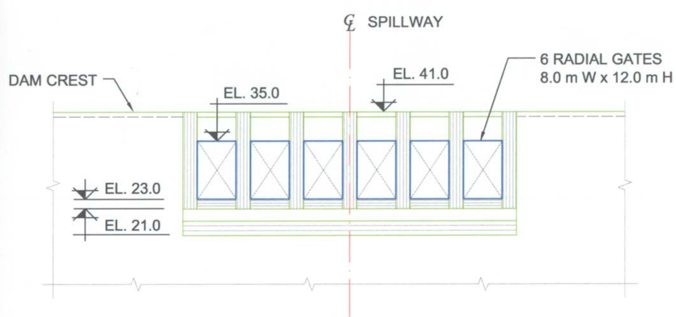
AUTORIDAD DEL CANAL DE PANAMA
Division de Proyectos de Capacidad del Canal

CONTRACT NO. CC-3-536
RÍOS COCLÉ DEL NORTE AND CAÑO SUCIO WATER SUPPLY PROJECTS
LOW COCLÉ DEL NORTE WATER SUPPLY PROJECT

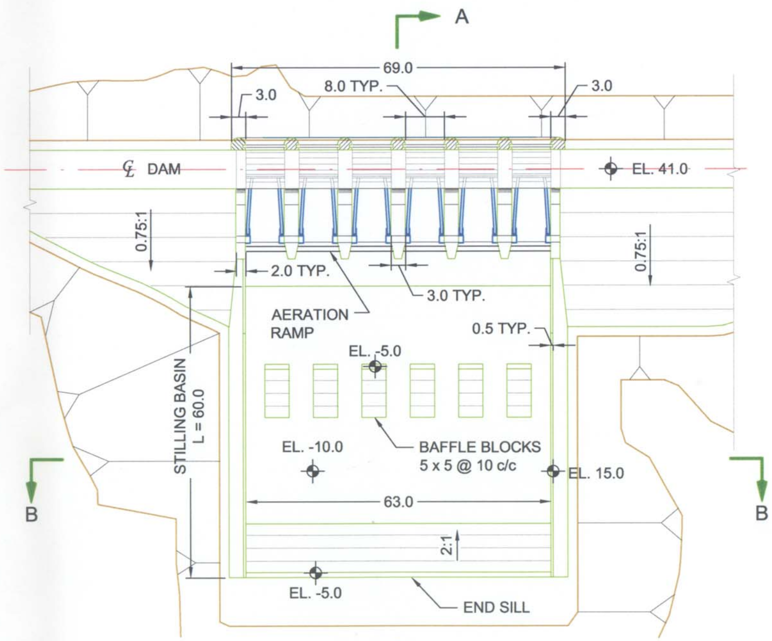
LOW COCLÉ DEL NORTE DAM PROFILE, TYPICAL CROSS SECTIONS AND DETAILS

MWH

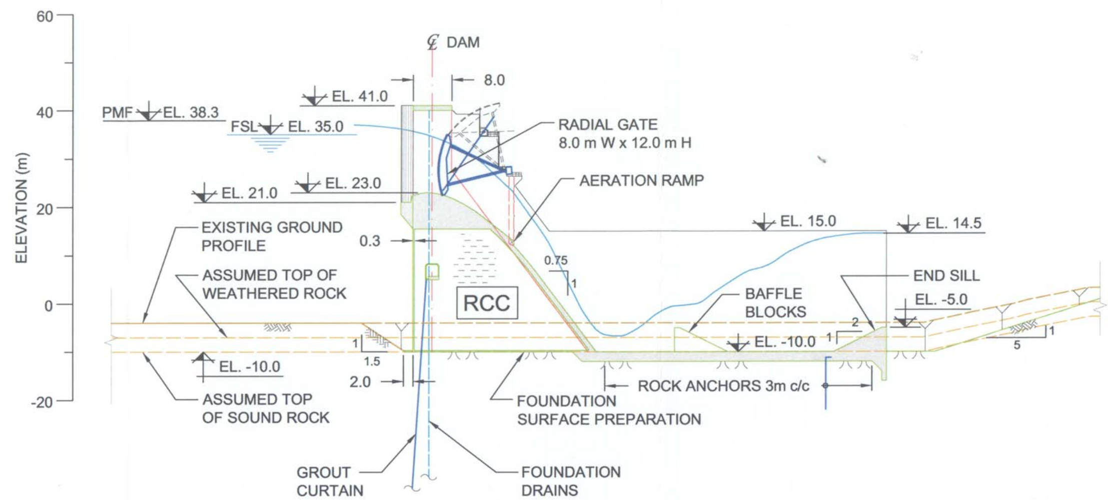
DATE:	EXHIBIT:
DECEMBER, 2003	5-3



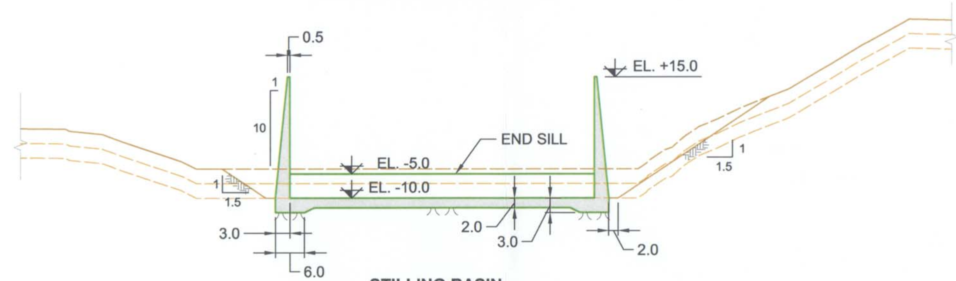
UPSTREAM ELEVATION



PLAN



SPILLWAY AND STILLING BASIN
TYPICAL CROSS SECTION A - A



STILLING BASIN
TYPICAL CROSS SECTION B - B

GRAPHIC SCALE

AUTORIDAD DEL CANAL DE PANAMA
División de Proyectos de Capacidad del Canal
ACP

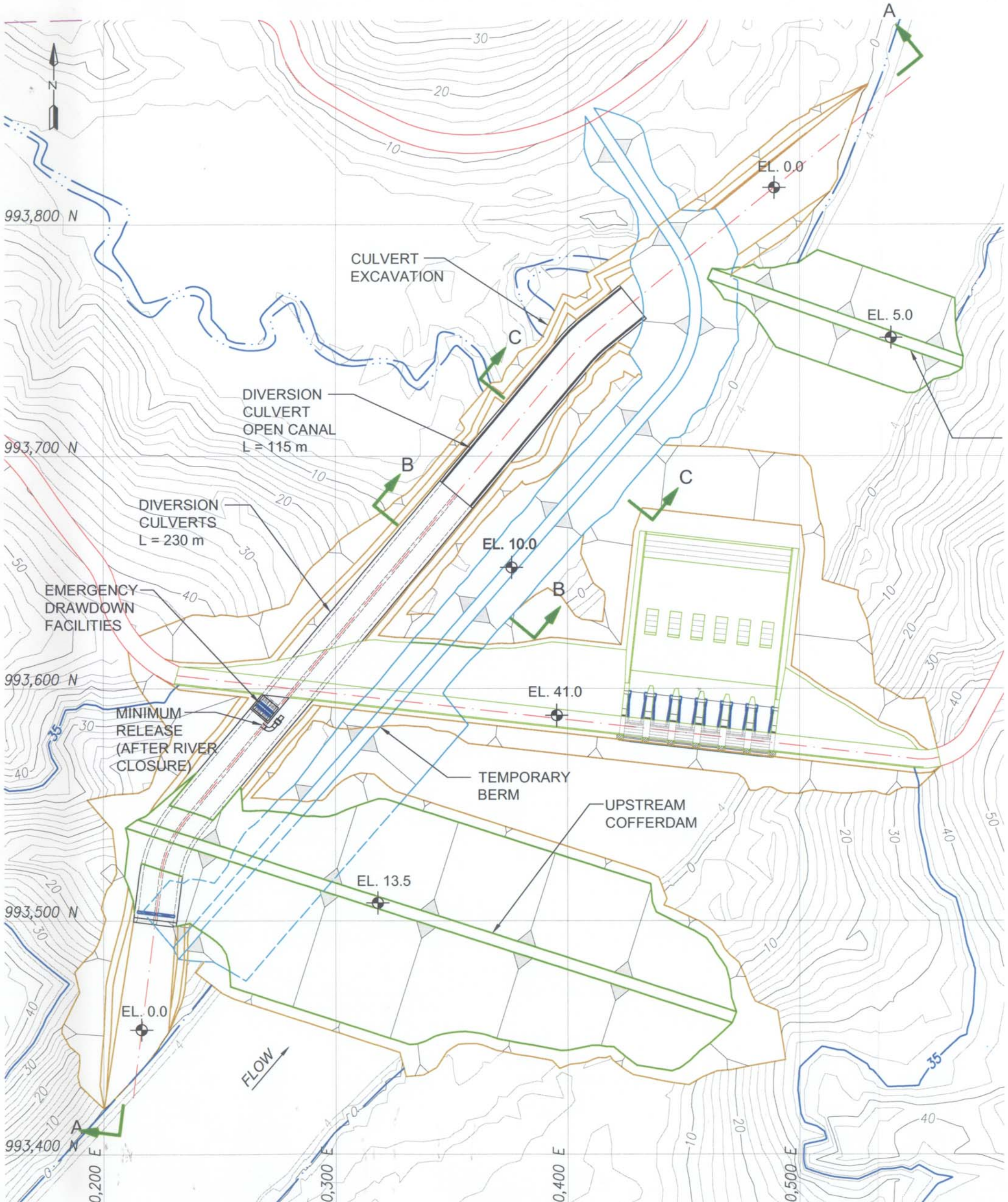
CONTRACT NO. CC-3-536
RÍOS COCLÉ DEL NORTE AND CAÑO SUCIO WATER SUPPLY PROJECTS
LOW COCLÉ DEL NORTE WATER SUPPLY PROJECT

LOW COCLÉ DEL NORTE SPILLWAY PLAN, SECTIONS AND UPSTREAM ELEVATION

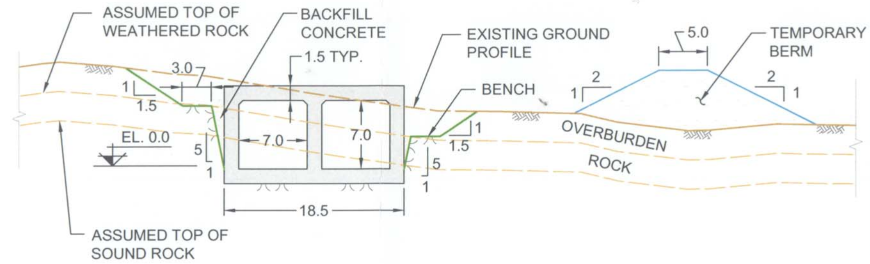
MWH

DATE: DECEMBER, 2003

EXHIBIT: 5-4

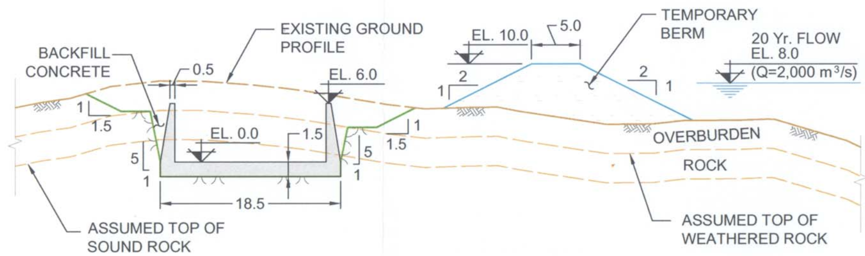


PLAN



TYPICAL CROSS SECTION B - B

0 5 10
GRAPHIC SCALE



TYPICAL CROSS SECTION C - C

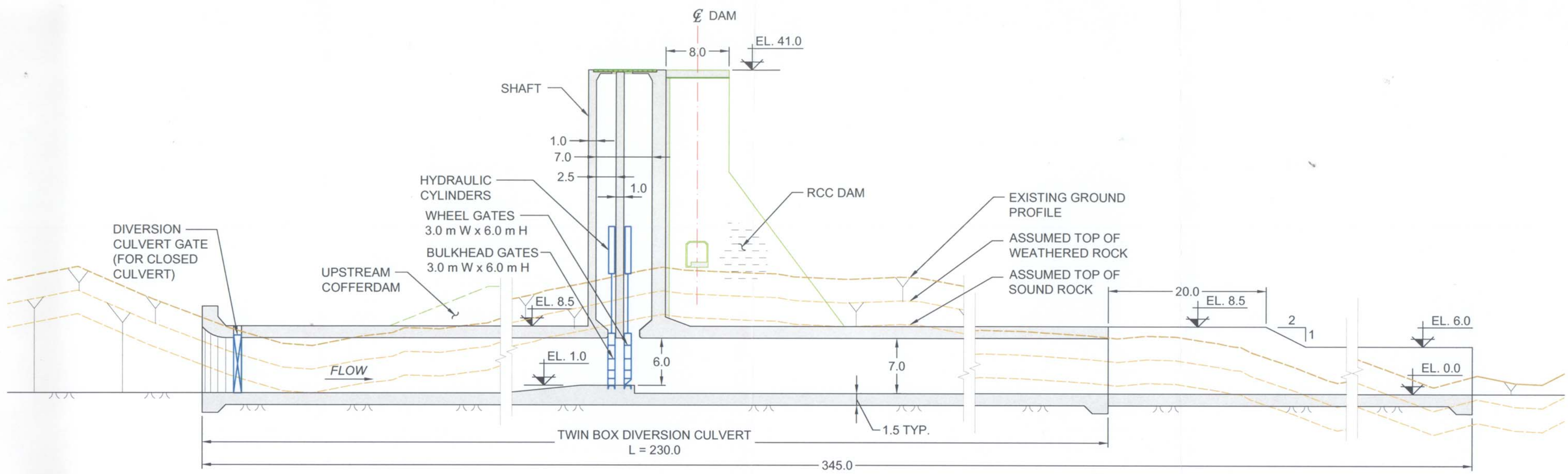
0 5 10
GRAPHIC SCALE

AUTORIDAD DEL CANAL DE PANAMA
División de Proyectos de Capacidad del Canal
ACP

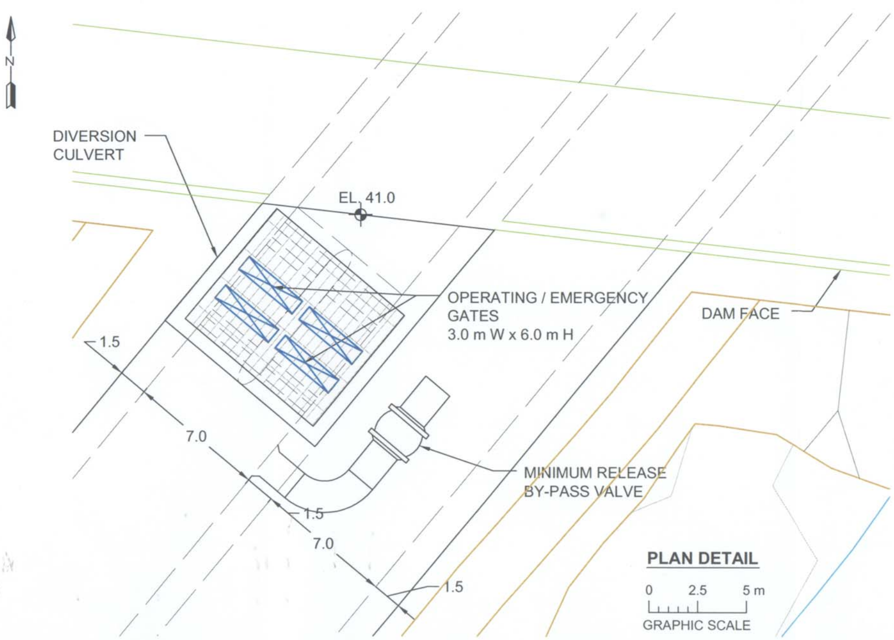
CONTRACT NO. CC-3-536
RÍOS COCLÉ DEL NORTE AND CAÑO SUCIO WATER SUPPLY PROJECTS
LOW COCLÉ DEL NORTE WATER SUPPLY PROJECT

LOW COCLÉ DEL NORTE
RIVER DIVERSION FACILITIES
PLAN AND TYPICAL CROSS SECTIONS

MWH	DATE: DECEMBER, 2003	EXHIBIT: 5-5



SECTION A - A



PLAN DETAIL

0 2.5 5 m
GRAPHIC SCALE

0 5 10 15 20 25 m
GRAPHIC SCALE

AUTORIDAD DEL CANAL DE PANAMA
Division de Proyectos de Capacidad del Canal

ACP

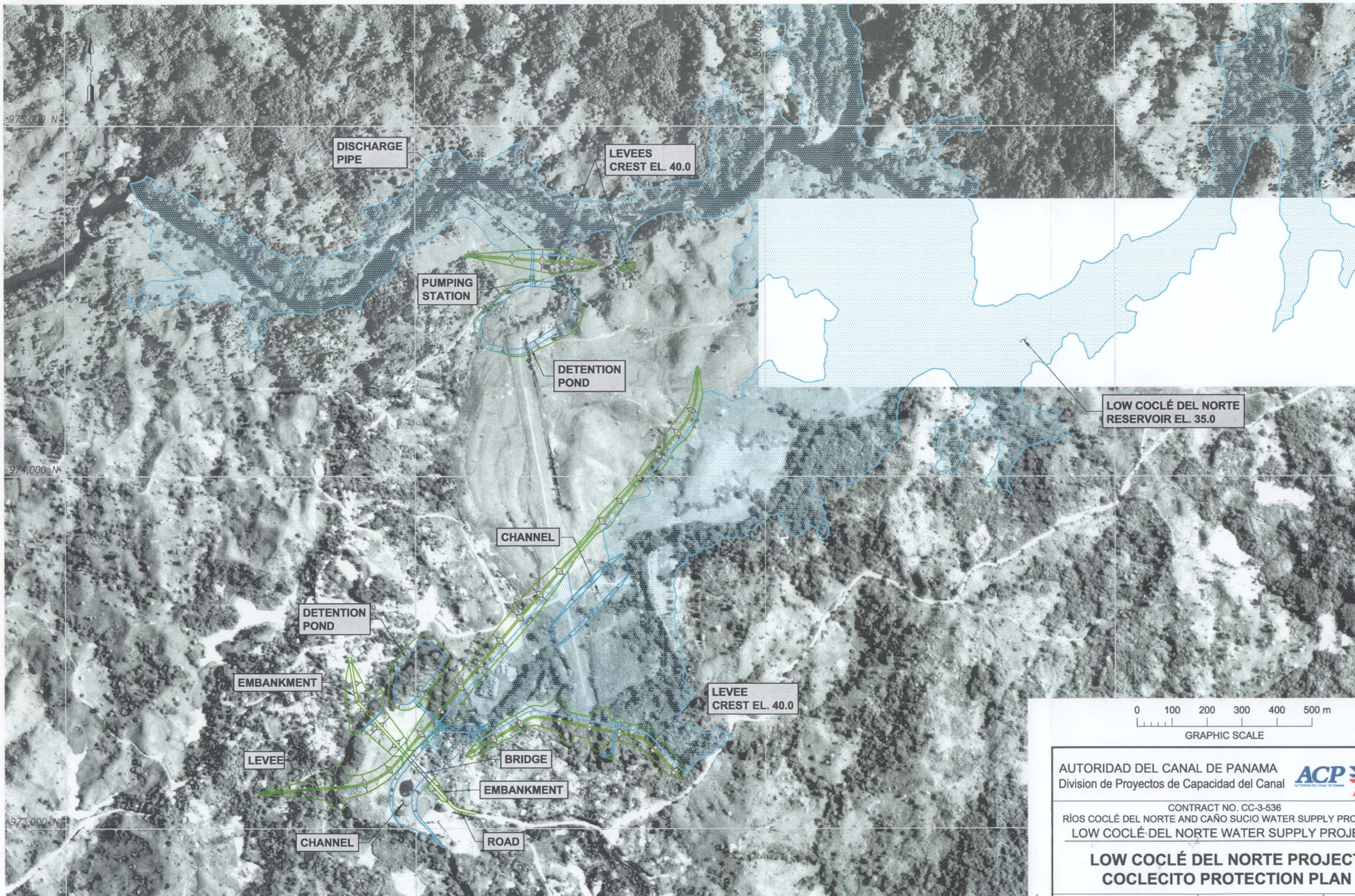
CONTRACT NO. CC-3-536
RÍOS COCLÉ DEL NORTE AND CAÑO SUCIO WATER SUPPLY PROJECTS
LOW COCLÉ DEL NORTE WATER SUPPLY PROJECT

LOW COCLÉ DEL NORTE DIVERSION AND EMERGENCY DRAWDOWN FACILITIES PROFILE AND DETAIL

MWH

DATE: DECEMBER, 2003

EXHIBIT: 5-6



SITE PLAN

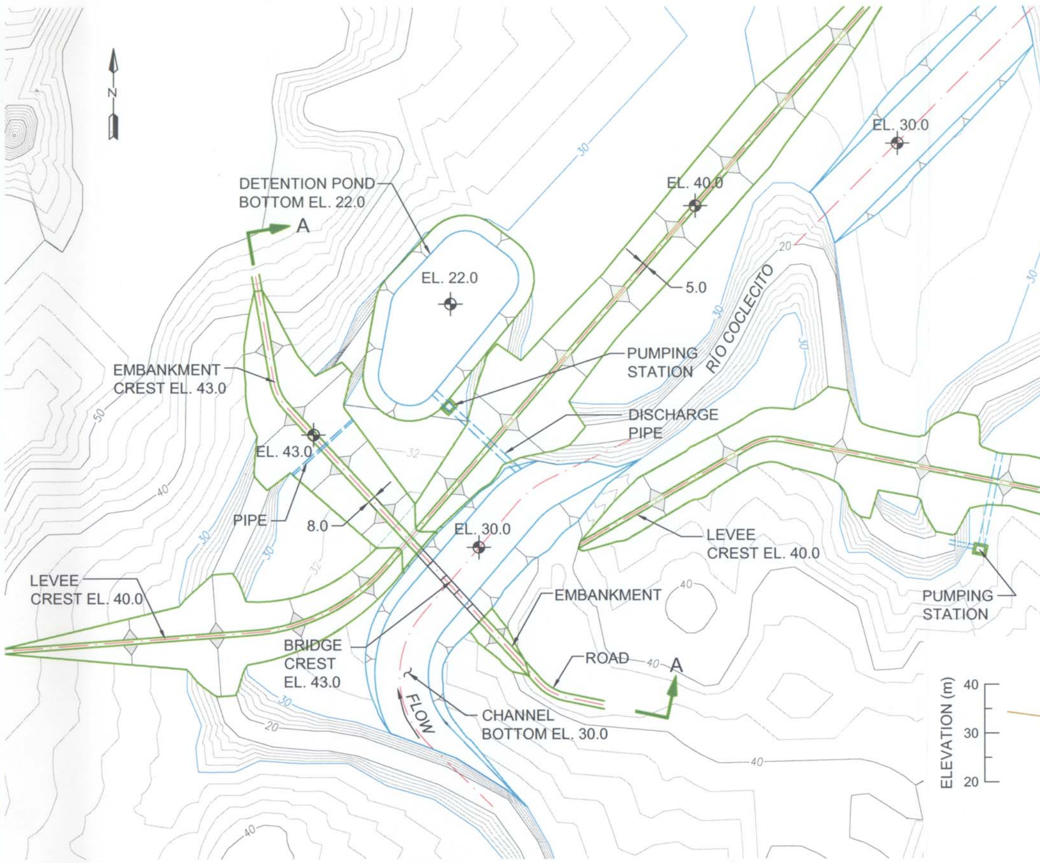
AUTORIDAD DEL CANAL DE PANAMA
Division de Proyectos de Capacidad del Canal

CONTRACT NO. CC-3-536
RÍOS COCLÉ DEL NORTE AND CAÑO SUCIO WATER SUPPLY PROJECTS
LOW COCLÉ DEL NORTE WATER SUPPLY PROJECT

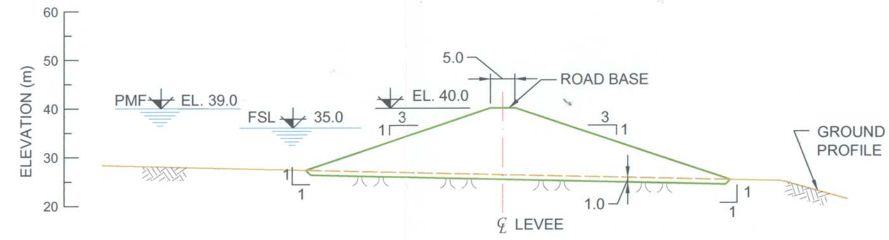
LOW COCLÉ DEL NORTE PROJECT
COCLECITO PROTECTION PLAN

MWH

DATE:	EXHIBIT:
DECEMBER, 2003	5-7

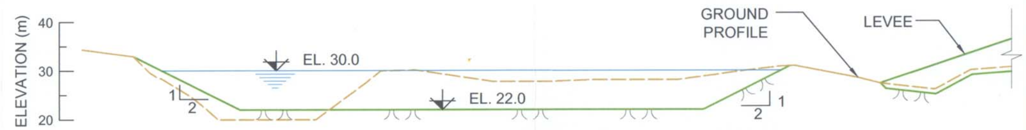


PLAN DETAIL



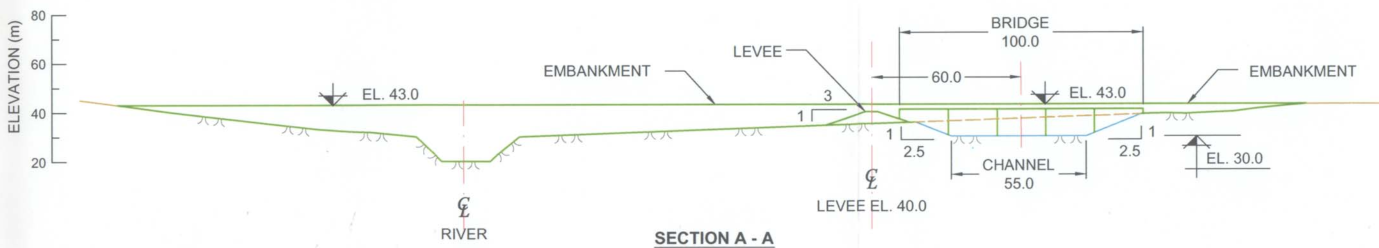
LEVEE
TYPICAL CROSS SECTION

0 10 20 m
GRAPHIC SCALE



DETENTION POND
TYPICAL CROSS SECTION

0 10 20 m
GRAPHIC SCALE



SECTION A - A

0 20 40 m
GRAPHIC SCALE

0 50 100 150 200 250 m
GRAPHIC SCALE

AUTORIDAD DEL CANAL DE PANAMA
Division de Proyectos de Capacidad del Canal

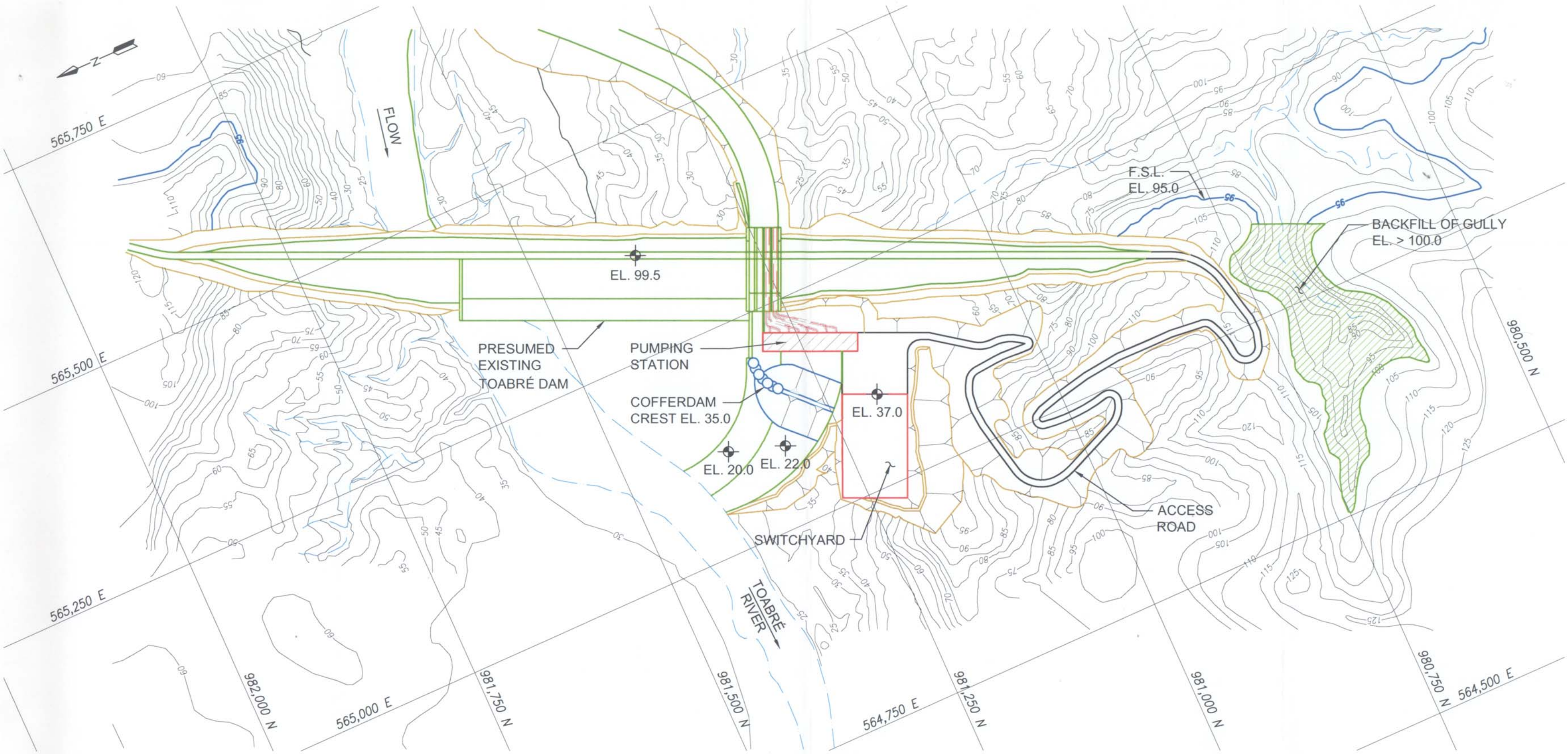
CONTRACT NO. CC-3-536
RÍOS COCLÉ DEL NORTE AND CAÑO SUCIO WATER SUPPLY PROJECTS
LOW COCLÉ DEL NORTE WATER SUPPLY PROJECT

LOW COCLÉ DEL NORTE PROJECT
COCLECITO PROTECTION
PLAN AND SECTIONS

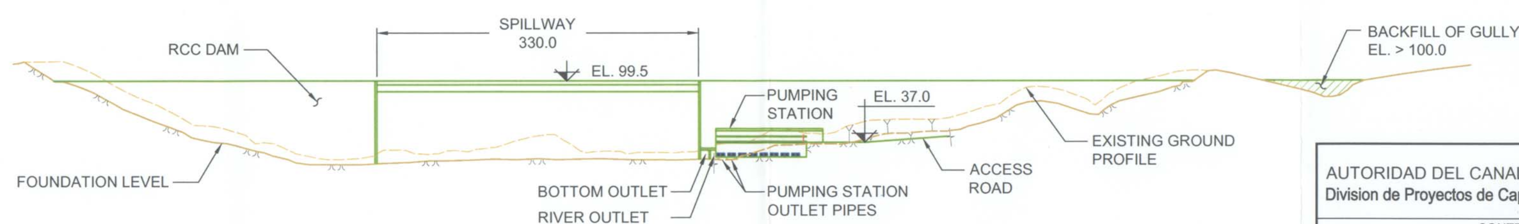
MWH

DATE: DECEMBER, 2003

EXHIBIT: 5-8



SITE PLAN



DOWNSTREAM ELEVATION
OF PRESUMED EXISTING DAM

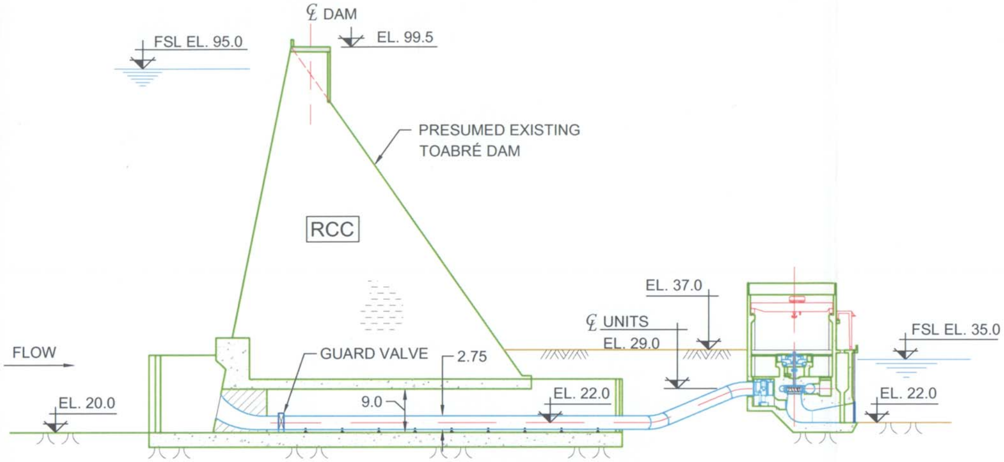
AUTORIDAD DEL CANAL DE PANAMA
División de Proyectos de Capacidad del Canal

CONTRACT NO. CC-3-536
RÍOS COCLÉ DEL NORTE AND CAÑO SUCIO WATER SUPPLY PROJECTS
LOW COCLÉ DEL NORTE WATER SUPPLY PROJECT

TOABRÉ PUMPING STATION
PLAN AND ELEVATION

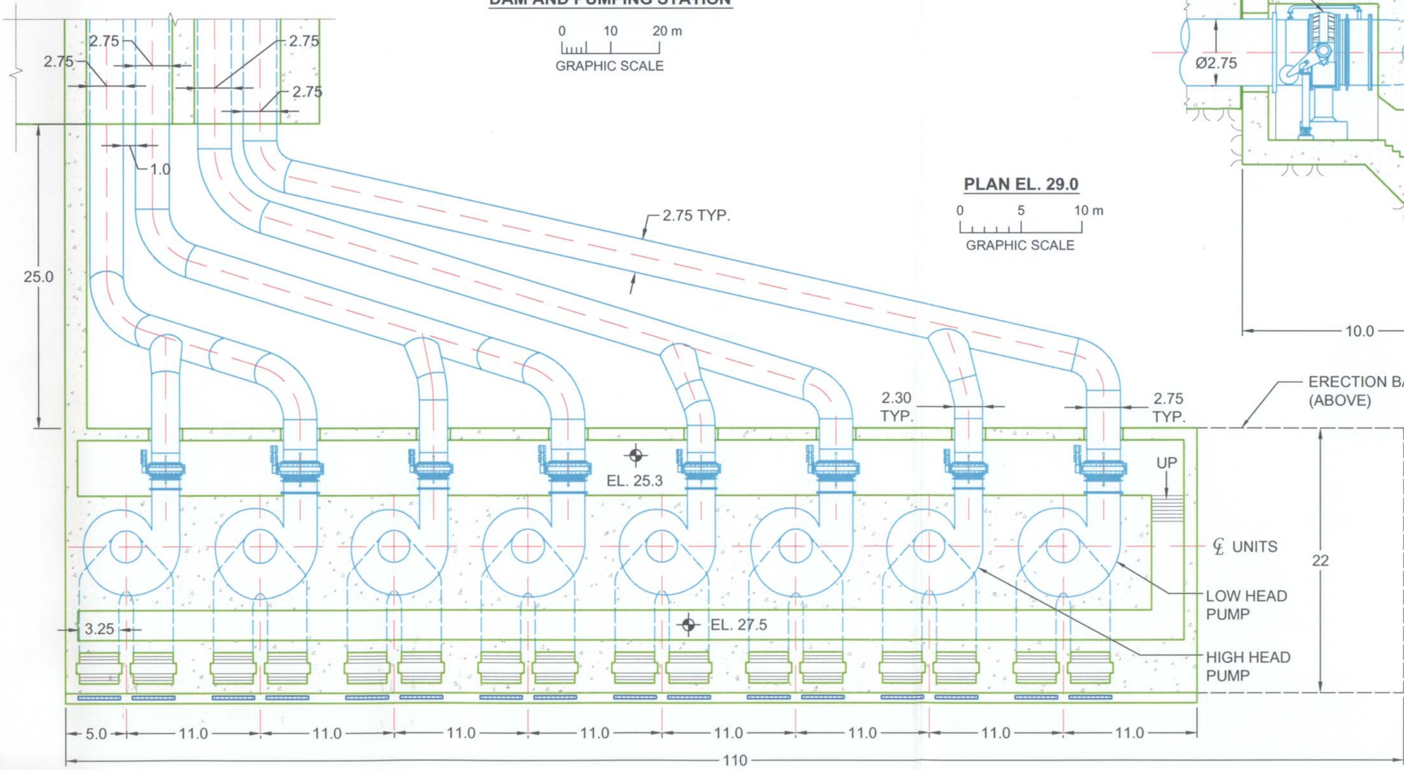
MWH

DATE:	EXHIBIT:
DECEMBER, 2003	5-9



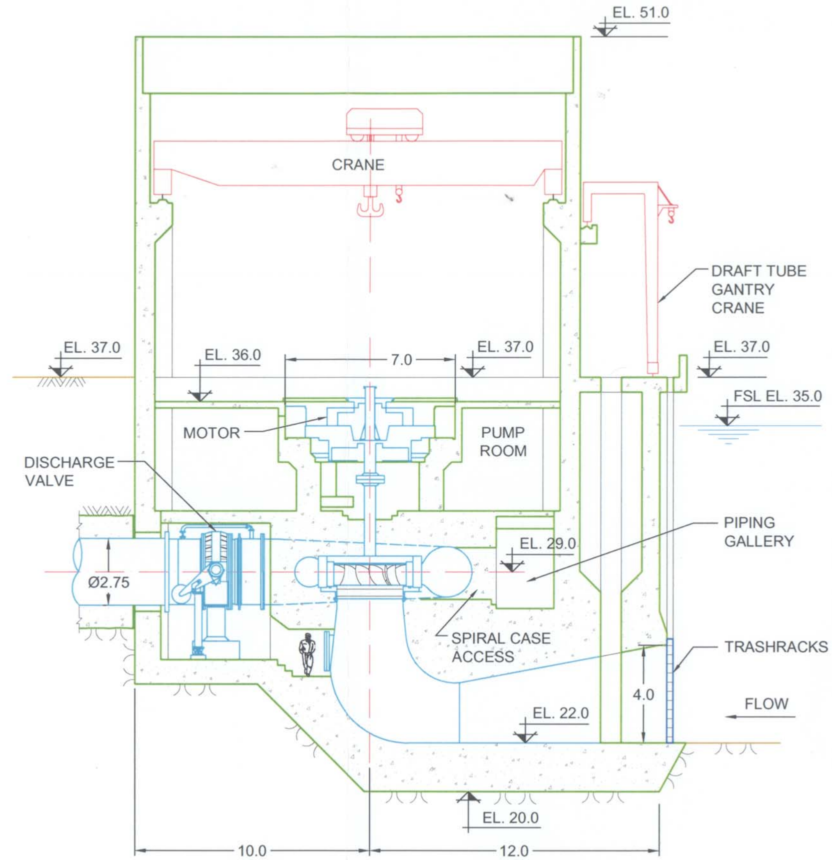
TYPICAL CROSS SECTION
DAM AND PUMPING STATION

0 10 20 m
GRAPHIC SCALE



PLAN EL. 29.0

0 5 10 m
GRAPHIC SCALE



TYPICAL CROSS SECTION

0 2 4 m
GRAPHIC SCALE

AUTORIDAD DEL CANAL DE PANAMA
División de Proyectos de Capacidad del Canal

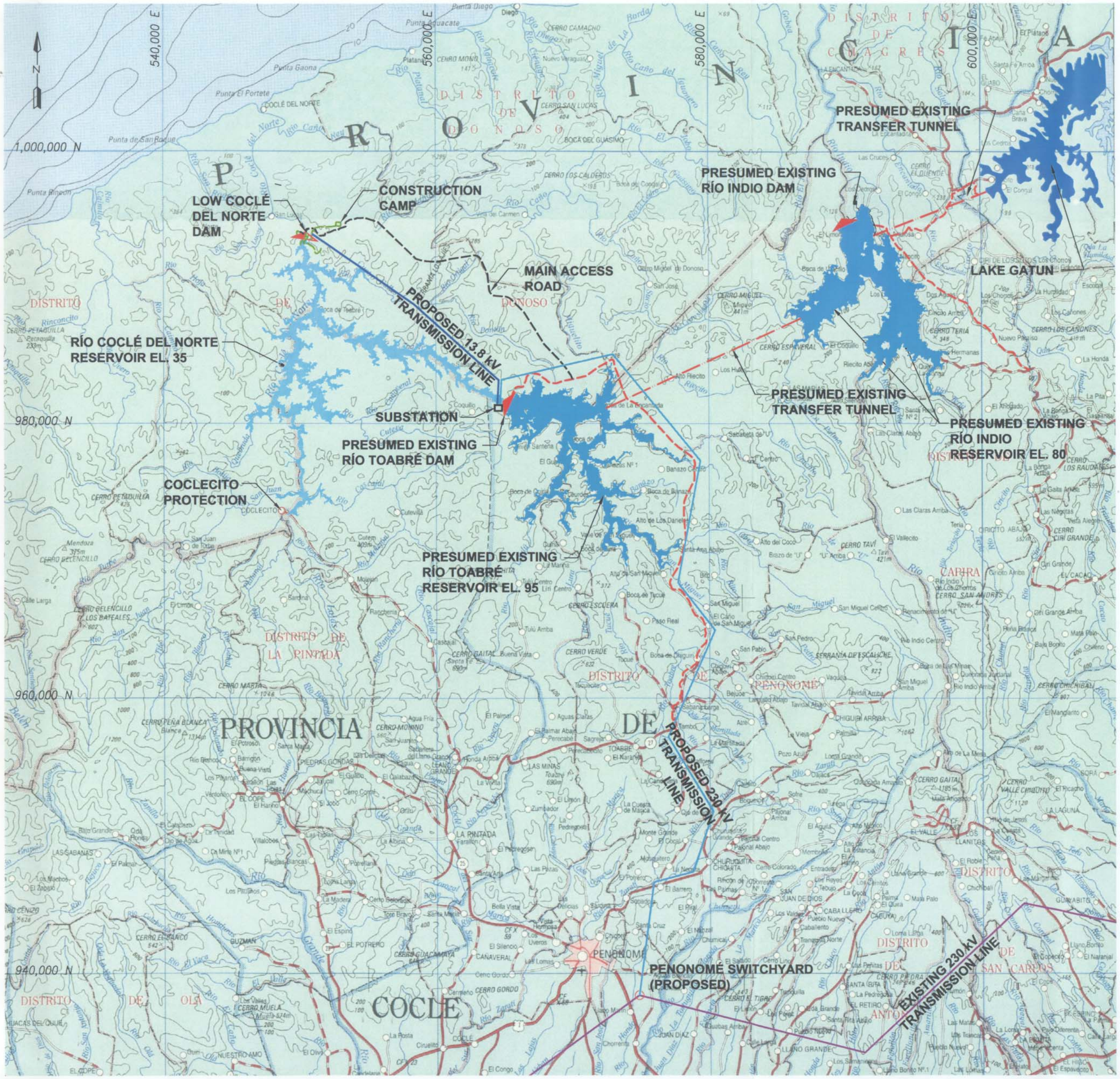
CONTRACT NO. CC-3-536
RÍOS COCLÉ DEL NORTE AND CAÑO SUCIO WATER SUPPLY PROJECTS
LOW COCLÉ DEL NORTE WATER SUPPLY PROJECT

TOABRÉ PUMPING STATION
TYPICAL CROSS SECTIONS AND
PLAN AT EL. 29.0

MWH

DATE: DECEMBER, 2003

EXHIBIT: 5-10



LEGEND:

- EXISTING 230 kV TRANSMISSION LINE
- PROPOSED 230 kV TRANSMISSION LINE
- PROPOSED 13.8 kV TRANSMISSION LINE
- PRESUMED EXISTING PAVED ROAD
- PERMANENT ACCESS
- CONSTRUCTION ACCESS

5 0 5 10 km

GRAPHIC SCALE

AUTORIDAD DEL CANAL DE PANAMA
División de Proyectos de Capacidad del Canal

CONTRACT NO. CC-3-536
RÍOS COCLÉ DEL NORTE AND CAÑO SUCIO WATER SUPPLY PROJECTS
LOW COCLÉ DEL NORTE WATER SUPPLY PROJECT

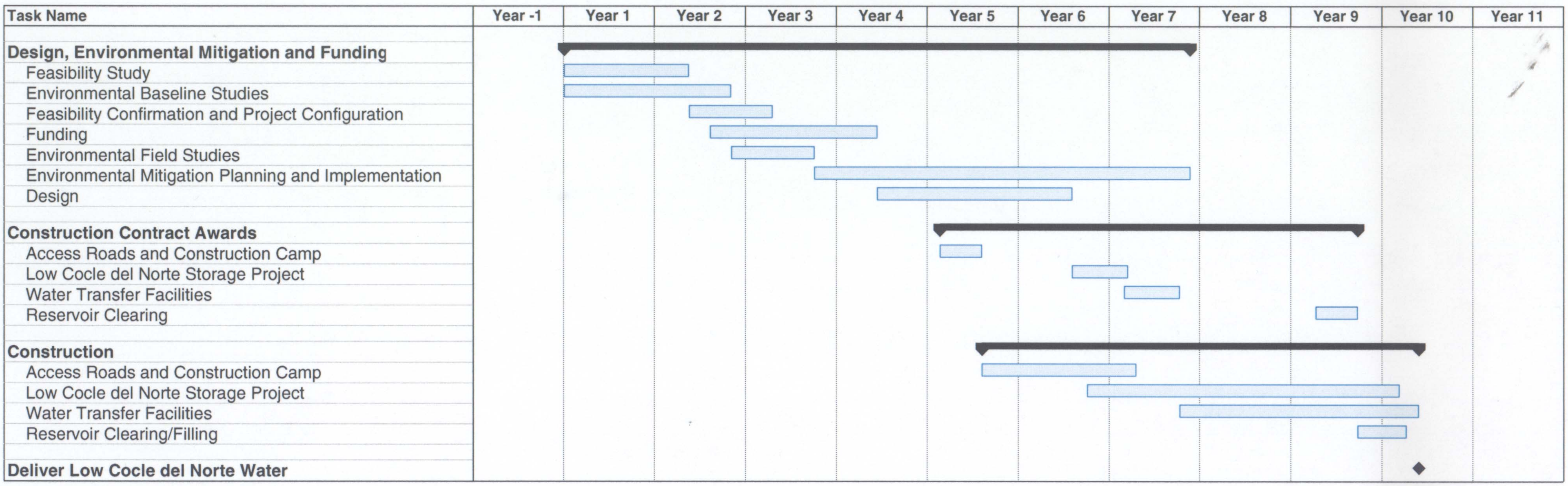
LOW COCLÉ DEL NORTE PROJECT ACCESS ROADS AND TRANSMISSION LINE ALIGNMENT

MWH

DATE:	EXHIBIT:
DECEMBER, 2003	5-11

Low Cocle del Norte Water Supply Project

Implementation Schedule



AUTORIDAD DEL CANAL DE PANAMA
División de Proyectos de Capacidad del Canal

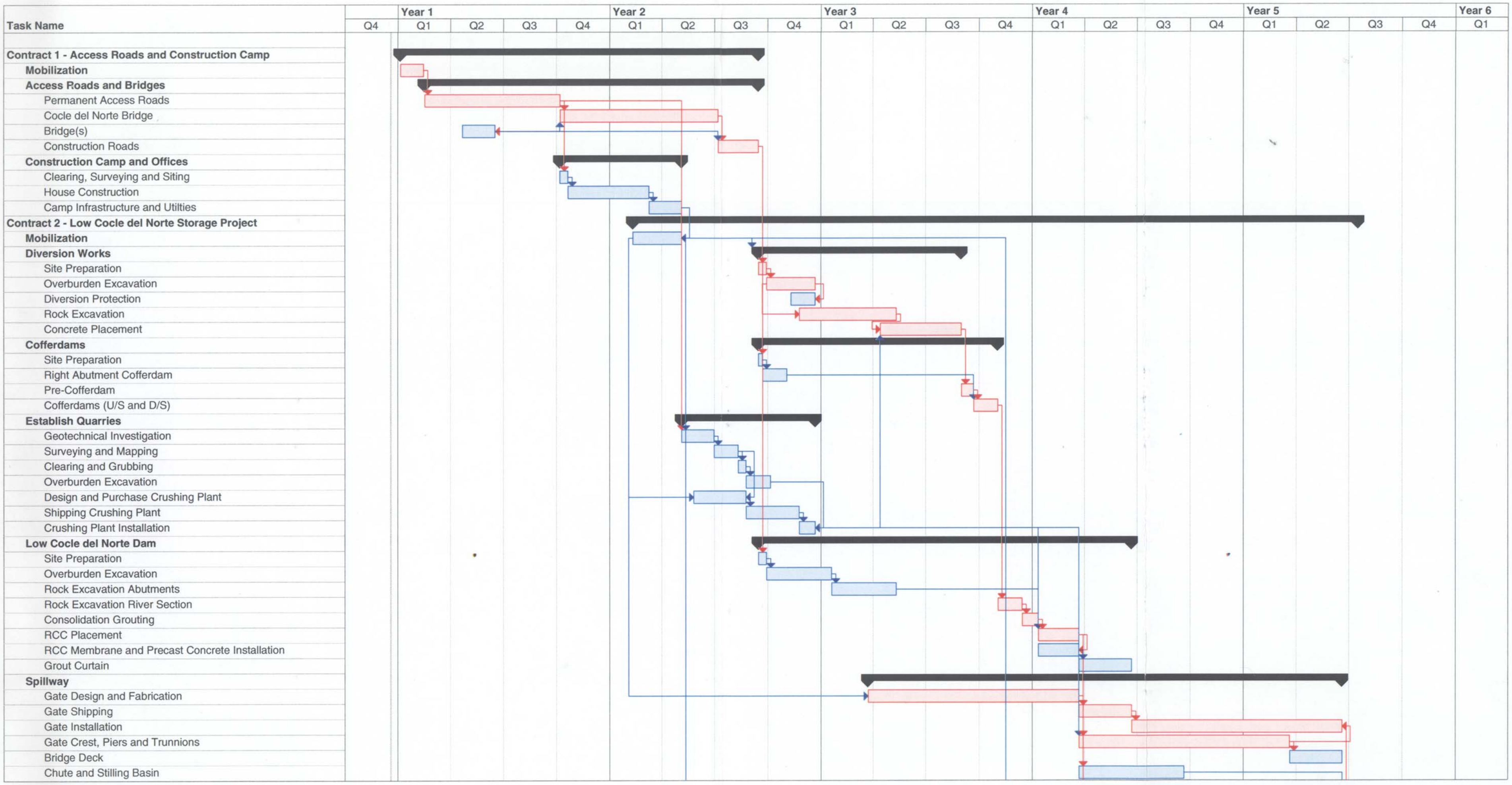
CONTRACT NO. CC-3-536
RÍOS COCLÉ DEL NORTE AND CAÑO SUCIO WATER SUPPLY PROJECTS
LOW COCLÉ DEL NORTE WATER SUPPLY PROJECT

IMPLEMENTATION SCHEDULE

MWH

DATE: DECEMBER, 2003

EXHIBIT: 6-1



AUTORIDAD DEL CANAL DE PANAMA
División de Proyectos de Capacidad del Canal

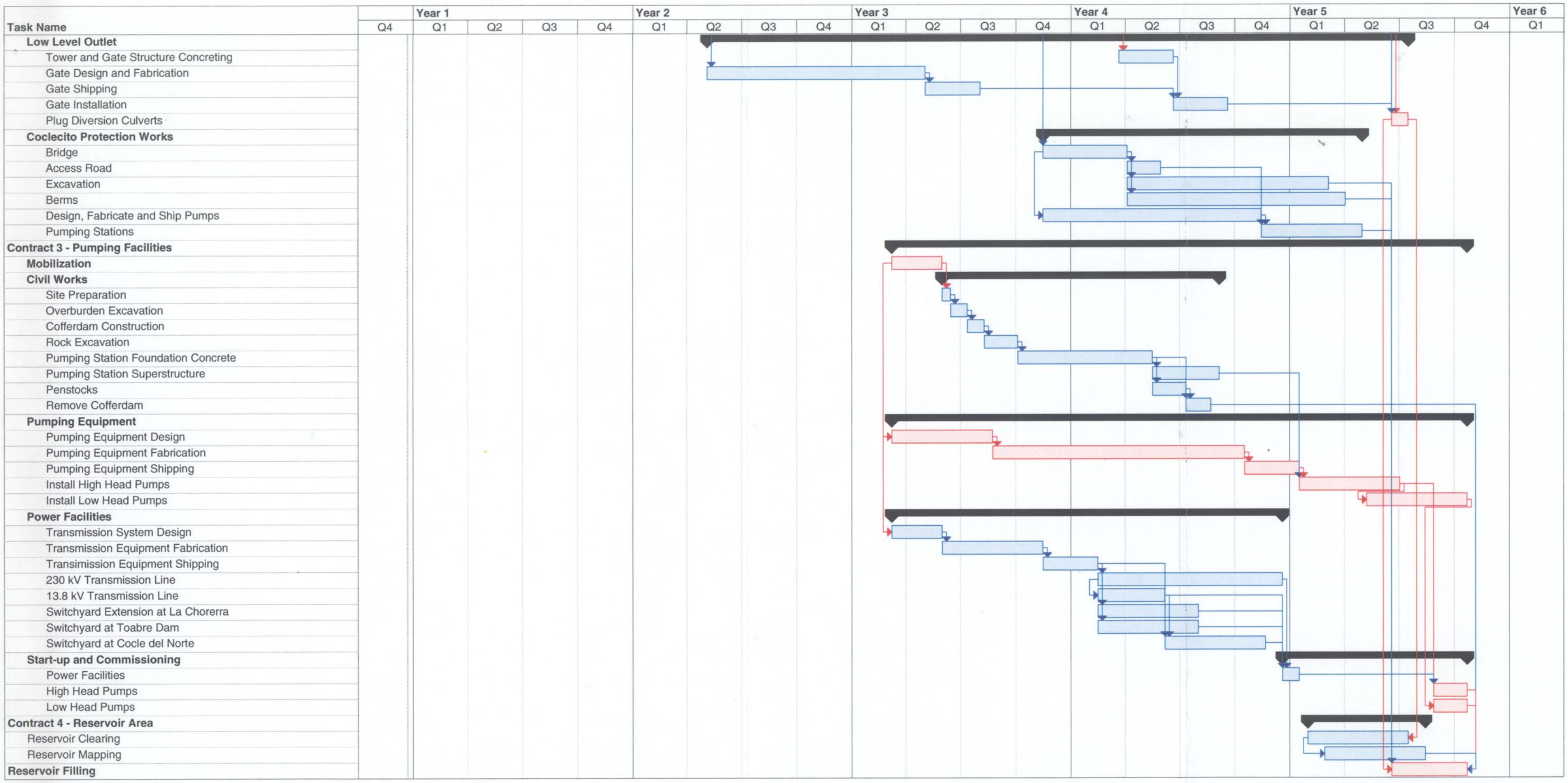
CONTRACT NO. CC-3-536
RÍOS COCLÉ DEL NORTE AND CAÑO SUCIO WATER SUPPLY PROJECTS
LOW COCLÉ DEL NORTE WATER SUPPLY PROJECT

CONSTRUCTION SCHEDULE
SHEET 1 OF 2

MWH

DATE: DECEMBER, 2003

EXHIBIT: 6-2



AUTORIDAD DEL CANAL DE PANAMA
Division de Proyectos de Capacidad del Canal

CONTRACT NO. CC-3-536
RÍOS COCLÉ DEL NORTE AND CAÑO SUCIO WATER SUPPLY PROJECTS
LOW COCLÉ DEL NORTE WATER SUPPLY PROJECT

CONSTRUCTION SCHEDULE
SHEET 2 OF 2

MWH

DATE: DECEMBER, 2003

EXHIBIT: 6-2

Appendix A
Hydrology

PRELIMINARY ASSESSMENT OF THE LOW COCLÉ DEL NORTE WATER SUPPLY PROJECT

APPENDIX A

HYDROLOGY, METEOROLOGY AND RIVER HYDRAULICS

Prepared by

MWH

PRELIMINARY ASSESSMENT OF THE LOW COCLÉ DEL NORTE WATER SUPPLY PROJECT

APPENDIX A – HYDROLOGY, METEOROLOGY AND RIVER HYDRAULICS

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1 INTRODUCTION

A project is proposed to pump water from a small reservoir on the Río Coclé del Norte to either a reservoir on the Río Toabre or in the Río Caño Sucio basin.

1.1 Objective and Scope

This Appendix draws on the hydrologic and river hydraulic analyses performed at a feasibility level for the 100-90 development in the Río Coclé del Norte basin (1) for representations of the hydrology for the entire basin, and describes additional studies to reflect the conditions that would exist if a reservoir were constructed to El. 95 on the Río Toabre. The procedures and basic data used in the determination of the following hydrologic parameters are discussed.

- Climate,
- Topography and Drainage
- Reservoir Evaporation,
- Streamflow Availability,
- Construction Period Flood,
- Spillway Design Flood, and
- Reservoir Sedimentation.

Coyne et Bellier, Bureau d'Ingenieurs Conseils (CEB) investigated the Río Toabre project at a feasibility level (2). The hydrologic parameters determined by CEB and pertinent to these analyses are given below.

Mean annual rainfall over the proposed lake:	3,411 mm
Basin annual rainfall:	2,761 mm
Mean annual flow:	40.8 m^3/s
All season 100-year flood (single peak):	2,780 m^3/s
Single peak PMF, peak:	9,510 m^3/s
Double peaks PMF, peak:	6,490 m^3/s

1.2 Location and Accessibility

The drainage basin of the Río Coclé del Norte is located to the west of the Panama Canal Area (see Exhibit 1). As proposed by the U.S. Army Corps of Engineers (USACE) in their Reconnaissance Report, the dam on the Río Coclé del Norte will be located approximately at latitude about 8° 59' north and longitude about 80° 32' west. The location is about 15 kilometers (km) inland from the Caribbean Sea (Atlantic Ocean) and about 7 km downstream from the confluence with the Río Toabre, near the mountain named Cerro Pelado. The drainage area at the dam site is about 1,594 km^2 (615.4 mi^2).

2 CLIMATE

2.1 General

The general climate of Panama is tropical with wet and dry seasons induced by the annual movement of the intertropical convergence zone (ITCZ). During the dry season, generally the months of February, March and April, the ITCZ is located south of Panama near the equator. In March or April, the ITCZ moves northward generally reaching Panama in late May or early June. Its passage results in heavy rainfall over a major portion of Panama. When the ITCZ is well north of Panama, occasionally the strength of the rainy season subsides and the months of July or August or both become a secondary dry season. In late summer or early autumn, the ITCZ starts its southward migration and it passes over Panama in late October or early November. During the months of October through December and occasionally in January, heavy rainfall occurs over Panama. When the ITCZ has moved well south of Panama, the dry season is established again. In general, the wet season is characterized by mild humid winds from a southerly direction while less humid, but somewhat stronger, northerly winds are more typical of the dry season (La Fortuna Project, 1976)

2.2 Average Annual Rainfall

The average annual rainfall over the Río Coclé del Norte basin above the damsites is estimated to be 2,800 mm. Exhibit 2 shows a mean annual rainfall map taken from *Atlas Nacional de la República de Panamá* (7). The map shows that mean annual rainfall is higher in the coastal area and decreases inland.

There are nine rainfall stations in the Río Coclé del Norte basin for which historic rainfall data are available (see Exhibit 3). Daily rainfall data are available from September 1974 to December 1998. Monthly rainfall data at these stations and the stations in the vicinity towards east were generated for a 30-year period (1966 to 1995) through correlation with nearby stations in the Canal Area. The mean monthly rainfall amounts are given in Attachment 1.

The mean annual rainfall over the contributing watershed is estimated to be about 2,800 mm. Based on extended records for the Coclecito station, the mean monthly rainfall is estimated based on a ratio of the basin annual rainfall and the station annual rainfall times the station monthly rainfall. Mean monthly rainfall values are shown below.

Mean Monthly Rainfall, Coclé del Norte Basin
(mm)

Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Annual
192	128	119	203	287	258	215	279	241	303	265	310	2,800

The mean monthly rainfall varies from a low of 119 mm in March to a high of 310 mm in December.

Through verbal discussion with the climatologists of ACP, it was determined that in the Canal Area and neighboring basins, including the Río Coclé del Norte basin, the effect of El Niño has been a slight to significant decrease in rainfall during the episodes. During the 1976 and 1982 episodes, the annual rainfalls at Boca De Toabre were about 3,260 and 3,891 mm, respectively, compared to mean annual rainfall of about 4,393 mm. This indicates a decrease of about 26 and 11 percent, respectively. The 1997-98 El Niño significantly decreased the rainfall in the Canal Area and over the Río Coclé del Norte basin. In 1997 the annual rainfall at Boca de Toabre was about 79 percent of the mean annual rainfall (period 1966 to 1995).

2.3 Temperature

Mean monthly temperatures vary within about 2° C throughout the year. Mean annual temperature varies from about 26° C near the dam to about 24° C in the head reach. The lowest temperature is in September-October and highest in March-April at lower altitudes. At higher altitudes, maximum temperature usually occurs in June.

3 TOPOGRAPHY AND DRAINAGE

The Río Coclé del Norte is formed downstream from the confluence of the Río San Juan and the Río Coclecito near the town of Coclecito. Both rivers drain the northern slopes of the Cordillera Central (Continental Divide) and flow northward. The Río San Juan is larger and longer of the two rivers. It rises at an elevation of 1,300 meters above mean sea level (El. 1300). The river is very steep in the head reach, dropping about 900 meters in a distance of about 5 km (about 18 percent slope). The slope decreases downstream to about 6 percent in about 4 km. Further downstream up to the confluence with the Río Toabre, the slope is about 0.3 percent. The drainage area of the Río Coclé del Notre at the confluence is about 674 km^2 (about 260.2 mi^2). The river basin is fan-shaped with a maximum length of about 58 km and a width of about 55 km.

The Río Toabre is the major right bank tributary of the Río Coclé del Norte and drains an area of about 805 km^2 (310.8 mi^2) at the confluence. The Río San Miguel is the major and longest tributary of the Río Toabre. It rises at about El. 900 and flows in a general northwestern direction to join the Río Toabre. The slope is about 10 percent in the 4-km long head reach, decreases to 3.3 percent in the next 6 km, and flattens to about 0.06 percent near the confluence.

The Río Cuatro Calles is another major right bank tributary, joining the Río Coclé del Norte about 2 km upstream from the dam site. The drainage area, including the area of the Río Coclé del Norte from confluence of the Río Toabre and Coclé del Norte to the dam site, is about 115 km^2 ((about 44.4 mi^2). Except for the most upstream distance of about 1.2 km, the river slope is about 0.3 percent.

The drainage area at the dam is 1,594 km^2, the sum of the three components.

4 NET RESERVOIR EVAPORATION

Monthly net reservoir evaporation for a reservoir is generally computed using the following relationship:

$$NRE = A(PE) - (PPT\text{-}RO)$$

in which

NRE = monthly net reservoir evaporation
A = pan coefficient
PE = monthly pan evaporation
PPT = monthly precipitation over the reservoir
RO = runoff presently contributed by the area that will be inundated by the reservoir

Since ACP with the help of COE had computed the net reservoir evaporation rates, the above procedure was not used. ACP derived the net reservoir evaporation rates using the historic evaporation data of Gatun Lake. The data was judged to be reasonable. Due to proximity of Gatun Lake to the Río Coclé del Norte reservoir, the net evaporation data derived for Gatun Lake was used for this study. The total net reservoir evaporation is estimated to be 1,134 mm/yr, and the monthly rates are given in Table 1.

5 STREAMFLOW ANALYSIS

5.1 Entire Coclé del Norte Basin

5.1.1 Data Sources

Monthly streamflow data for stream gaging stations pertinent to this study were obtained from ACP. The data included: measured flows, estimated flows identified with asterisks, and the flows filled-in through correlation/transposition with other stations. The period of record considered in the analysis was from January 1948 through December 1999. Table 2 provides names of the stream gaging stations and the rainfall stations used in this study. Exhibit 4 shows the locations of stream gaging stations. The exhibit also shows the locations of rainfall stations as per list given in Table 2. The rainfall stations were used for the design flood study as discussed under "Spillway Design Flood".

There are three stream gaging stations in the Río Coclé del Norte basin. These include Río Coclé del Norte at Canoas (drainage area 571 km^2, or about 220.5 mi^2), Río Coclé del Norte at El Torno (drainage area 672 km^2 or about 259.5 mi^2) and Río Toabre at Batatilla (drainage area 788 km^2) or about 304.2 mi^2). The station at El Torno was discontinued in 1986. The locations of these stations are shown on Exhibit 3. Mean annual flows for the period of record are about 39.5, 53.0 and 41.5 m^3/s, respectively (about 1,95, 1,72 and 1,66 ft^3/s, respectively).

5.1.2 ACP Analysis

The ACP performed analyses to generate a long-term monthly flow sequence on the Río Coclé del Norte at the dam site for the period from January 1948 to December 1999. The monthly flows were generated as follows:

Monthly flow data for the period from January 1948 to December 1999 (flow missing for a few months) was available for the Río Trinidad at El Chorro (drainage area 172 km^2 or 66.4 mi^2). The data for the missing months was estimated either using gage height data from a staff gage installed at the station or based on the general trend in the monthly flows. A correlation was developed between the monthly flows of the Río Ciri Grande at

Los Canones (drainage area 186 km^2 or 71.8 mi^2) and the Río Trinidad at El Chorro using the concurrent period of record.

Using concurrent monthly flows of the Río Indio at Boca de Uracillo (drainage area 365 km^2 or 140.9 mi^2) and of the Río Ciri Grande at Los Canones, a regression equation was developed to generate the monthly flow data at Boca de Uracillo for the period from January 1948 to July 1979 and June to December 1999.

Using concurrent monthly flows of the Río Indio at Boca de Uracillo and Río Toabre at Batatilla, a regression equation was developed to extend the monthly flows of the Río Toabre at Batatilla.

Using concurrent monthly flows of the Río Coclé del Norte at El Torno and the Río Toabre at Batatilla, a regression equation was developed to fill-in and extend the monthly flows of the Río Coclé del Norte at El Torno.

Monthly flows at the dam site were developed using the following equation:

Dam site flows = ((Batatilla flows) + 1.1384(El Torno flows)) * 1.0264

MWH made an independent check of the equation used to compute monthly flows at the damsite. Using conventional procedure of transposing flows by a combined ratio of drainage area and mean annual rainfall, the estimated flows were about 1 to 3 percent less than the flows derived by ACP. This was considered to be an insignificant difference. Therefore, the monthly flows estimated by ACP were adopted.

5.1.3 Annual Streamflow and Monthly Streamflow Sequence

The estimated mean annual flow of the Río Coclé del Norte at the damsite, as determined by the foregoing analysis, is estimated to be 107.5 m^3/s. Long-term monthly discharges (in m^3/s and ft^3/s units) for the Río Coclé del Norte at the dam site are given in Table 3. Exhibits 5 and 6 show the mass curve and the time series of annual flows for the Río Coclé del Norte at the dam site. These exhibits show that the annual flows are consistent, homogeneous and there is no apparent trend in the data. However, there are significant variations in flows from year to year. The highest flow occurred in 1970 and the lowest in 1997. The low flow was due to the El Niño episode recorded in 1997-1998.

5.1.4 Streamflow Characteristics

The wet period is generally from October through December but quite often, high flows can occur in the months of January and September. The months of low flows are from February to April.

Generally, floods occur during the months of September through January due to general type of storms. The floods due to thunderstorms can occur during any time of the year but generally in the months of June through August. The highest floods of record at Canoas (period from 1983 to 1999), El Torno (period from 1958 to 1986) and Batatilla (period from 1958 to 1999) were about 1,356 m^3/s (47,900 ft^3/s) in June 1994, 3,116 m^3/s (110,000 ft^3/s) in January 1990 and 2,633 m^3/s (93,000 ft^3/s) in January 1970, respectively.

5.1.5 Flow Duration Curves

Daily flow data were available for the Río Toabre at Batatilla (from 1968 to 1998) and the Río Coclé del Norte (from 1984 to 1998). These data were used to develop flow duration curves for the two stations. The curves are shown on Exhibits 7 and 8, respectively. The minimum observed daily flows were about 0.1 and 3.1 m^3/s, respectively (3.5 and 109.5 ft^3/s, respectively). Flows exceeding 90 and 95 percent of the time were estimated to be 7.3 and 5.4 m^3/s, respectively (257.5 and 190.7 ft^3/s, respectively) at Batatilla, and about 13.7 and 9.9 m^3/s, respectively (483.8 and 349.6 ft^3/s, respectively) at Canoas.

Daily flow data were not generated at the dam site. The flow duration curve based on monthly data does not provide a good indication of low flows. To estimate the flows exceeding 90 and 95 percent of the time at the dam site, the flows corresponding to these percentages were transposed from Canoas and Batatilla and combined. The flows at Canoas were transposed to the dam site using drainage area ratio and to this the flows at

Batatilla were added. The resulting flows were about 26.1 and 19.3 m^3/s, respectively (921.7 and 681.6 ft^3/s, respectively). These flows were judged to be reasonable.

5.1.6 Drought-Duration-Frequency Analysis

Monthly flows of the Río Coclé del Norte at dam site were arrayed in one column. Running totals of 6-, 12-, 18- and 24-month periods were computed. For the flows in each period, the following procedure, illustrated for the 12-month period, was used.

- Select the lowest 12-month value.
- To avoid overlapping, exclude the 11 totals prior and subsequent to the selected lowest value.
- After excluding the values, select the next lowest value and again exclude the 11 totals prior and subsequent to the selected value.
- Continue until all totals have been used either by selecting or excluding.
- Array the selected values from lowest to highest and assign 1 to the lowest value.
- Compute the return period of the lowest value as "number of years of record divided by the order", that is, "52/1" (Stall, 1964). The return period for the second lowest value will be "52/2 = 26."

Table 4 shows the 6-, 12-, 18- and 24-month flows and their assigned recurrence intervals. These data show estimates of the average length of time in years that can be expected to elapse between the beginnings of the various events. For example, the third ranking event in the 12-month series has a recurrence interval of 52/3 = 17.3 years. Thus, it can be said that in any year the probability is 1 in 17.3 for the start of a 12-month period during which the total flow would be as low as 2,220 MCM.

5.2 Coclé del Norte Basin Excluding the Toabre Reservoir Basin

This section presents the hydrologic analyses performed for the Río Coclé del Norte reservoir considering that a reservoir on the Río Toabre, located upstream from its confluence with the Río Coclé del Norte, would be operative.

5.2.1 Annual Streamflow and Monthly Streamflow Sequence

Long-term monthly discharges at the Río Coclé del Norte Dam site (drainage area about 865.4 km^2 excluding an area of about 728.6 km^2 upstream from proposed Toabre dam) were estimated using the following procedure.

1. Monthly flows for the Río Coclé del Norte at El Torno gaging station were estimated for the period from January 1948 to December 1999 for the feasibility study of the Río Coclé del Norte development. These included observed and filled-in and/or generated flows.
2. Based on a review of Exhibit 2, it was determined that the mean annual rainfall over the drainage basin between El Torno gaging station–Toabre dam and Coclé del Norte dam (drainage area about 193.4 km^2) could be higher than that over the basin above El Torno gage site. However, the mean annual rainfall over the total basin, about 865.4 km^2, and over the 672-km^2 area above El Torno gage may not be significantly different. Based on this assumption, the monthly flows at El Torno were transposed to the dam site using drainage area ratio (865.4/672).
3. The resulting transposition equation was: Coclé del Norte Flow at dam site = (El Torno Flow) * (1.2878). This equation was used to generate the data at the dam site.

The flow at the damsite for hydrologic conditions excluding the Toabre Dam drainage are shown in Table 5. The mass curve of annual flows and 5-year moving average are shown on Exhibits 9 and 10, respectively. These exhibits indicate that the generated flow data is consistent but there is a downward trend in the annual flows.

The effect of El Niño on the project flows was investigated. It was found that during the episodes of 1965, 1976, 1982, 1997 and 1998, the mean annual flows were about 88.0, 72.4, 75.9, 75.1 and 85.0 percent of the long-term average. The most affected month was of December 1976, when the flow was only 32.1 percent of long-term monthly flow for December.

5.2.2 Flow Duration Curve

The annual flow duration curve derived for the Río Coclé del Norte at Canoas (drainage area about 571 km) based on daily data was used. The daily flows at Canoas exceeding 90 and 95 percent of time were about 13.7 and 9.9 m^3/s, respectively. Transposed by drainage area ratio (865.4/571), the flows at dam site excluding the Toabre Dam drainage would be about 20.8 and 15.0 m^3/s, respectively.

5.2.3 Drought-Duration-Frequency Analysis

The method of analyzing drought frequencies and duration is based the procedures described in Section 5.1.7. Table 6 shows the drought-duration-frequency data.

The 6-, 12-, 18-, 24-month flows and their assigned recurrence intervals furnish estimates of average length of time in years that can be expected to elapse between the beginnings of the various events. For example, the fourth ranking event in the 12-month series has a recurrence interval of 53/4 = 13.3 years. Thus, it can be said that in any year the probability is about 1 in 13 for the start of a 12-month period during which the total flow would be as low as 1,638 MCM.

6 CONSTRUCTION PERIOD FLOODS

6.1 Available Flood Data

The *Instituto de Recursos Hidraulicos y Electrificacion* (IRHE) performed a regional flood frequency analysis in June 1986 for the river basins west of about 79° west longitude in the Republic of Panama. This study is discussed in Attachment 2.

Annual maximum instantaneous peaks are available for the Río Coclé del Norte at El Torno and Canoas, and the Río Toabre at Batatilla for 16 years (1970 to 1985). The data for Canoas was partly estimated.

Analysis of extreme flood events involves the selection of the largest events from a set of flow data. The flood frequency analysis uses the annual largest recorded floods at a representative stream gaging station. For the present analysis, the monthly instantaneous flood peaks for the Río Coclé del Norte at Canoas and El Torno, and Río Toabre at Batatilla were obtained from ACP. The data are given in Table 7 to 9. The data for some months and/or years are missing. The annual peaks derived from these data are for 16 years at Canoas, 26 years at El Torno and 35 years at Batatilla. The maximum instantaneous observed peaks were 1,356 m^3/s or 47,900 ft^3/s (June 1994) at Canoas, about 3,116 m^3/s or 110,000 ft^3/s (January 1907) at El Torno and about 2,633 m^3/s or 93,000 ft^3/s at Batatilla.

6.2 Analysis for the Entire Coclé del Norte Basin

Log-Pearson type III (LP III) distribution, recommended by the Hydrology subcommittee, United States Geological Survey (March 1982) was first fitted to the annual peaks of the Río Coclé del Norte at Canoas. A computer program developed by the United States Army Corps of Engineers, Hydrologic Engineering Center was used.

The generalized extreme value (GEV) distribution was also fitted to the data to compare the results from this distribution with that obtained by using LP III distribution. A computer program developed by Environment Canada (1994) was used.

The skew coefficient of the flood peaks was about 0.52 and that of log-transferred values was about –0.02. The flood peaks estimated by LP III were low compared to those obtained by using GEV. However, based on a visual judgment of a plot of the results, both the distributions indicated reasonable goodness of fit. For a conservative estimate of the flood peaks, the values resulting from GEV distribution were adopted.

The above analysis was repeated using the flood peaks of the Río Coclé del Norte at El Torno and Río Toabre at Batatilla. For both the stations, the flood frequency data based on GEV distribution was adopted.

The flood estimates by the IRHE (based on regional analysis) and site-specific estimates discussed above were compared. For the Río Coclé del Norte at Canoas and El Torno, and the Río Toabre at Batatilla, the 20-year flood peaks estimated by the IRHE were 2,144, 2,357, and 2,525 m^3/s respectively, compared to the site-specific estimates of 1,320, 2,160, and 1,810 m^3/s respectively. The IRHE estimates are higher for all sites and return periods except for El Torno above a return period of 20 years. It was concluded that the site specific estimates were a better representation of the construction period floods and MWH's estimates by the GEV distribution were adopted for the subsequent studies.

Realizing that flood protection works might be designed for protection during dry season, flood frequency analysis was also performed for the dry period. From a review of the monthly flood peak data, the dry season was judged to be the months of January, February and March.

6.2.1 Transposition of Flood Peaks to Dam Site

The dam site will be located downstream from the confluence of the Río Coclé del Norte and Río Toabre. The rainfall over the Río Coclé del Norte basin is higher than that over the Río Toabre basin. Therefore, flood peaks at the dam site should include effect of the variation of extreme rainfall over both basins. Since the period of record at El Torno was longer than that at Canoas, the flood peak data at El Torno was used. The following procedure was used to derive the flood peaks at dam site using the estimated flood peaks at El Torno and Batatilla.

A general procedure for transposition of flood peaks from a gaged location to an ungaged location is to use coefficients of empirical relationships assuming that these coefficients remain constant for hydrologically and meteorologically similar drainage basins. The relationship used is given below.

Rodier's Formula

$$K = 10 * (1- ((\log^{Q} -6) / (\log^{A} -8))) \quad \text{(Rodier, 1985)}$$

'K' is a coefficient. A is the drainage area in km^2 and Q is flood peak in m^3/s. The value of 'K' was computed for floods of various return periods. Table 11 shows the values. For the El Torno station (draining an area of relatively high rainfall), the values of K were greater than that for Batatilla (draining an area of relatively low rainfall). Considering the rainfall variation, the mean of two values for a given return period was used for transposition. Table 10 shows the mean values of K and the flood peaks derived at the dam site.

6.2.2 Flood Hydrographs

Flood hydrographs of 20- and 50-year return periods were developed for the Río Coclé del Norte at dam site using the following procedure:

- The historic floods (hourly discharge data) of the Río Coclé del Norte at Canoas and at El Torno, and the Río Toabre at Batatilla were reviewed. The data showed that the duration of floods could vary from about 1 to 2 days.
- The annual maximum one- and two-day flood-volumes were determined for the three stations.
- A volume-frequency analysis was performed.
- The 1-day and 2-day flood volumes at the dam site were estimated as sum of flood volumes at El Torno and Batatilla adjusted for difference in drainage area.

The 20- and 50-year flood volumes for 1-day duration were about 1,490 and 1,950 cubic meters per second – day (cms-day), respectively. The 2-day volumes were about 2,370 and 3,100 cms-day.

- The observed hydrographs at the two stations were plotted. The shape of the hydrographs was reviewed. Although the peaks and volumes of the hydrographs were quite different yet the shape (rising and falling limbs) had a reasonable similarity. The single peaked historic flood of December 1995 was selected to shape the flood hydrographs of 20- and 50-year return periods.

- The historic flood was adjusted to represent flood peaks and volumes equal to the 20- and 50-year floods.

The derived floods are shown on Exhibits 11 and 12.

6.3 Analysis for the Coclé del Norte Basin Excluding the Toabre Reservoir Basin

6.3.1 Method of Analysis

The "K" factors used to compute the flood frequency data were derived using the flood frequency data estimated for the Río Coclé del Norte at Canoas and Río Coclé del Norte at El Torno. The adopted "K" factors, for all season and dry period floods are given below.

'K' FACTORS FOR COMPUTATION OF FLOOD PEAKS

Return Period, years	2	5	10	20	50	100
All season K factor	4.06	4.35	4.53	4.68	4.87	5.01
Dry season K factor	2.46	3.04	3.38	3.69	4.08	4.35

Table 11 shows the all season and dry season flood peaks for 2-, 5-, 10-, 20-, 50- and 100-year return periods based on the above factors. This procedure of flood peak computation implies that there would be no significant flood releases from Toabre dam that would increase the estimated peaks.

6.3.2 Flood Hydrographs

Flood hydrographs for 20- and 50-year return periods were developed based on the corresponding flood hydrographs derived at the Río Coclé del Norte dam site during the feasibility study. The previous hydrographs were scaled down using ratios of the corresponding peaks. The flood volumes were not adjusted. This assumption could result in higher flood volumes but is conservative in case there would be releases from Toabre. The derived flood hydrographs are shown on Exhibits 13 and 14, respectively.

7 SPILLWAY DESIGN FLOOD

The probable maximum flood (PMF) based on the probable maximum precipitation (PMP) was used as the spillway design flood for the Río Coclé del Norte dam. The derivation of the PMF involved the following sub-tasks:

- Estimation of PMP, its duration and time distribution
- Estimation of Retention Losses
- Development of a Unit Hydrograph
- Estimation of Base Flow
- Transformation of the PMP to a PMF
- Evaluation of the PMF

7.1 Probable Maximum Precipitation (PMP)

7.1.1 Rainfall Regime

In Panama, October and November are the heaviest rainfall months. This period of heavy rainfall is associated with the southward traverse of the inter-tropical convergence zone (ITCZ). November dominates high values on the Atlantic side. Higher values occur in October than in November on the Pacific side because of more frequent southerly winds in October.

Reports on PMP by the United States Weather Bureau (WS, 1965) and National Weather Service (NWS, 1978) discussed the possibility of hurricanes in Panama. A necessary condition for a hurricane is a coriolis force sufficiently strong to cause the winds to spin around the center of a low-pressure area. On the equator, the Coriolis force is zero and still relatively weak within 10° of the equator. Therefore, only rarely there are hurricanes within 10° of the equator. Thus, hurricanes generally do not occur over Panama. (The exception was Hurricane Martha. The track of this hurricane is discussed in the 1978 report by the NWS.) However, the influences of peripheral circulation, both direct and indirect, cannot be ruled out. Heavy rainfalls have occurred in southwest Panama because of peripheral circulation.

Both general type and local storms have been recorded in Panama. Local storms are of relatively small aerial extent, covering from about 200 to 500 mi^2. General storms can cover larger areas. The months of October through December are the season of large-area rainfalls. Nearly all-major storms reported in the 1965 and 1978 reports occurred in this period. Of the 22 storms analyzed in the 1965 and 1978 reports, 15 occurred during these months.

During the months of October through December, strong air outflows come from the northern latitudes. This implies northerly winds, at least for some times during major storms, which impinge on the mountains, and augment the rainfall through stimulation, triggering of convergence, or otherwise giving additional lift to saturated air. Generally, most intense rainfall occurs over the northern slopes of the Continental Divide.

The northerly winds, coming from Atlantic Ocean, pass over Panama and have their first encounter with the coastal hills. These hills trigger convergence and heavy rainfall occurs over the coastal area. The rainfall amount and intensity decrease further inland but are increased near the Continental Divide. This pattern is clear from the mean annual rainfall map shown on Exhibit 2. The pattern is controlled by the local topography.

7.1.2 Methods for Estimating PMP

The adopted approach for estimating the PMP was taken from the feasibility-level studies and consists of extending the 24-hour, 10-mi^2 PMP developed in the NWS 1978 Report over the Río Coclé del Norte basin, use depth-area-duration curves of WS 1965 Report and estimate basin average PMP.

7.1.3 Duration of PMP

All available storm isohyetal maps were for three-day rainfall. For the storms since 1976, three-day rainfall amounts were also used. However, the hourly rainfall data for the stations at Chorro and Los Canones, located east of the Río Coclé del Norte basin, indicated that the actual maximum rainfall duration in the major three-day storms (based on daily observations taken at 07-09 hours in the morning) was about 48 hours. For this reason, a duration of 48 hours was considered appropriate for the PMP.

7.1.4 PMP Estimate

The 24-hour, 10 mi^2 PMP map given in the NWS 1978 Report (Exhibit 15) was extended towards west to cover the Río Coclé del Norte basin. The following procedure was used for the extension.

For about ten rainfall stations in the Río Coclé del Norte basin and its vicinity, annual maximum daily rainfall data were obtained. The stations included: Boca de Toabre, Chiguiri Arriba, Coclé del Norte, Toabre, San Lucas, Sabanita Verde, Coclecito, Santa Ana, Miguel de la Borda and Boca de Uracillo. The stations are shown on Exhibit 4 with names given in Table 2. The period of record varied from about 18 to 41 years.

Point PMP at each station was determined using Hershfield's method (1963). The values of the point PMP varied from station to station. Some values were quite consistent with the values from the NWS point PMP map. The value at Toabre, located at relatively high altitude, was high compared to other values. The value was retained considering orographic effect due to high altitude of the station.

Keeping in mind the local topography and the trend of the point PMP lines on the NWS map, the point PMP lines were extended over the Río Coclé del Norte basin. It should be realized that the extension was based on the trend of the lines on the NWS map, and estimated point PMP values. Some of the values considered to be inconsistent (especially based on 20 years or less data) were given less weight. No meteorological factors were used in the estimation of point PMP.

Exhibit 15 was used to derive the basin average PMP for the Río Coclé del Norte basin. Because of variation in the point PMP, the drainage basin upstream from the dam was divided into three sub-basins as shown on Exhibit 16. The three sub basins are:

1. Area between the confluence of the Río Coclé del Norte and Río Toabre, and dam site including the Río Caño Rey (115 km^2),
2. Río Toabre above its confluence with the Río Coclé del Norte (805 km^2)
3. Río Coclé del Norte above its confluence with the Río Toabre (674 km^2)

The derived 24-hour, 10-mi^2 PMP were about 30.60 (777), 26.25 (667) and 26.89 (683) inches (mm) for sub-basins 1, 2 and 3, respectively. To obtain the sub-basin average PMP for duration of 48 hours, the depth-area-duration curves shown on Exhibit 17 were used. For sub-basins 1 (drainage area about 115 km^2 or 44.4 mi^2), 2 (drainage area about 805 km^2 or 310.8 mi^2) and 3 (drainage area about 674 km^2 or 260.2 mi^2), the factors were about 1.23, 1.02 and 1.04, respectively. The sub-basin average PMP were about 37.65 (956), 26.75 (679) and 27.95 (710) inches (mm), respectively. Based on the sub-basin PMPs, the basin 48-hour PMP was about 28.1 inches (714 mm). The 48-hour PMP based on the maximized and transposed storm was about 24 inches (610 mm) which is lower than the PMP based on NWS point PMP map. Therefore, a 48-hour basin average PMP of 28.1 inches (714 mm) was used.

With the presence of Toabre dam on the Río Toabre, the basin was divided into four sub-basins.

1. Area between the confluence of the Río Coclé del Norte and Río Toabre, and dam site including the Río Caño Rey (115 km^2)
2. Río Coclé del Norte above its confluence with the Río Toabre (674 km^2)
3. Area below Toabre dam and up to the confluence of the Río Toabre and Río Coclé del Norte (76.4 km^2),
4. Río Toabre at proposed dam site (728.6 km^2)

It was concluded above that the PMP based on 24-hour, 10-mi^2 PMP given in the NWS 1978 Report was more critical than that developed by using the transposition and maximization of historic storms. Therefore, the 24-hour, 10-mi^2 PMP values for sub-basins 1 and 2 were as derived above and for 3 and 4 were derived from the NWS report. Using the depth-area-duration curves shown on Exhibit 17, the 48-hour PMP values for basins 1-4 were estimated to be 956 mm, 710 mm, 928 mm and 685 mm, respectively. The average basin PMP excluding the Toabre Dam drainage is 762 mm.

7.1.5 Depth-Duration Curve

Depth-duration data for the size of each sub-basin was obtained from Exhibit 17. There was not much variation of percentages of 48-hour derived for each sub-basin from the exhibit. Therefore, same percentages were used for all sub-basins. The estimated percentages were 45.9, 61.1, 70.5 79.1 and 100.0 for 6, 12, 18, 24 and 48 hours. These were plotted and a smooth curve was drawn as shown on Exhibit 18. This data was for duration of six hours and greater. Because of small sizes of the sub-basins, the PMP amounts for duration less than six hours were required. To extend the depth-duration curve for duration less than 6 hours, guidance was obtained from the hourly rainfall data recorded at El Chorro. The curve on Exhibit 18 also shows extrapolation to a one-hour duration.

7.1.6 Sequential Arrangement of PMP Increments

A unit duration of one hour was selected considering the size of the sub-basins. The hourly PMP increments were obtained from Exhibit 18. There are a number of methods available to sequentially arrange the PMP increments to produce critical flood conditions. For this study, the "alternating block method" (Ven Te Chow, et al 1988) was used. This method provides reasonable critical flood conditions. The highest hourly increment was placed at 28th hour and the remaining increments were arranged in descending order alternately to the right and left of the maximum increment to form PMP hyetograph. Table 12 shows the arrangement of the increments.

7.2 Retention Losses

For this study, initial loss and uniform loss rate method was used. The derivation of these losses is discussed below.

7.2.1 Initial Loss

A review of the daily rainfall data at various stations in the basin indicated that during the months of October through December, the rainfall occurred quite frequently. Therefore, during these months when the PMP is most likely to occur, there is a strong likelihood of significant storms prior to the PMP storm. The antecedent rainfall could be substantial. Therefore, the initial retention was considered negligible on the assumption that the soil moisture deficiency and other abstractions would be satisfied by an antecedent storm.

7.2.2 Uniform Loss

This loss represents the rate at which the soils in the basin will allow the rainfall to percolate through during the storm period. From the study of soils and geology from the Atlas (1988), and based on the field reconnaissance, the soils in the basin were judged to be predominantly of SCS soil group C. The recommended minimum infiltration rate for this group varies from 0.05 to 0.15 inches (1.3 to 3.8 mm) per hour. A rate of 3 mm (0.12 inches) per hour was used. No infiltration loss was considered from the reservoir area.

7.3 Unit Hydrograph

Synthetic unit hydrographs were developed for the sub-basins using Clark's method (Clark, 1945). The Clark's method translates incremental runoff from the sub-areas within a basin to the outlet of the basin according to the travel time (time of concentration) and then routes the runoff through a linear reservoir to account for the storage effect of the basin size and channel system. The method requires estimates of time of concentration and storage routing coefficient, and a time-area curve defining the cumulated area of the basin contributing runoff to the outlet of the basin as a function of time, expressed as ratio or percent of the time of concentration.

7.4 Base Flow

Base flow was estimated to be 50 m^3/s, 50 m^3/s, and 10 m^3/s for the three basins used to develop the PMF for the entire basin. When excluding the area above Toabre, the base flow was estimated to be 10 m^3/s, 5 m^3/s, 50 m^3/s, and 50 m^3/s for basins 1-4 respectively.

7.5 Probable Maximum Flood

In the feasibility-level studies (2), the HEC-1 computer model was used to develop flood hydrograph from each sub-basin and the PMF at the dam site resulting from the 48-hour PMP. The input to the model included: drainage area, base flow, 48-hour PMP, time distribution of the PMP, retention losses and the percentage of the drainage area under reservoir at maximum operating pool elevation, values of Tc and R, and the time area curve for each sub-basin. Since two operating levels of 100 and 80 meters were used, the percentages of the drainage areas under reservoir varied. For the 80 meters case, the resulting flood hydrograph at the dam site had a peak of about 10,460 m^3/s and a 5-day volume of about 988 MCM. If the maximum operating pool will be at 100 meters, the PMF peak would be about 10,550 m^3/s and a 5-day volume of 1,005 MCM. Exhibit 19 shows the PMF inflow hydrograph.

7.5.1 PMF for Entire Basin

For a reservoir full supply level at El. 35, the PMF would be further reduced. The peak inflow adapted for a full supply level at El. 35 was 10,000 m^3/s and the 3-day volume was 950 MCM.

7.5.2 PMF for Basin Excluding Toabre Drainage

For the basin excluding the Toabre Dam drainage, the runoff from sub-basin 3, draining into Toabre dam, was routed through the Toabre reservoir. Elevation-volume and spillway discharge data were derived from the figures provided by ACP from the feasibility report of Toabre dam prepared by CEB. These data are presented below and should be considered approximate.

ELEVATION-VOLUME AND SPILLWAY DISCHARGES
TOABRE DAM

Elevation (m)	Storage Volume (MCM)	Elevation (m)	Spillway Discharge (m^3/s)
90	750	95.0	0
95	940	95.2	400
100	1,140	95.5	1,000
105	1,400	96.0	2,000
110	1,690	97.0	4,000
115	2,040	98.0	5,800

The resulting PMF hydrograph has a peak of about 9,970 m^3/s and a 3-day volume of about 950 MCM. Exhibit 20 shows the PMF hydrograph.

7.5.3 Evaluation of PMF

Generally, a PMF estimate is compared with the historic floods and 100-year flood at the site. Also based experience, the value of coefficient C in the Creager's formula (Creager, 1950), is compared with the values obtained for PMFs in hydrologically similar drainage basins.

For the entire basin, the 100-year flood at the dam site was estimated to be 4,610 m^3/s. The ratio between the PMF peak and the 100-year flood peak was about 2.3, which is reasonable for the hydrologic conditions in the basin. The value of Creager's C was about 116. The value is in the range of the values expected in similar areas and, therefore, the estimated PMF is considered to be reasonable.

For the basin excluding Toabre, the 100-year flood at site was estimated to be 2,976 m^3/s. The ratio between the PMF peak and 100-year flood peak is about 3.4, which is reasonable for the hydrologic conditions in the basin. The value of Creager's C was about 113. The value is also in the range of the values expected and the estimated PMF is considered to be reasonable.

8 RESERVOIR SEDIMENTATION

8.1 Data Sources

Suspended sediment data were collected by *Empresa de Transmision Electrica, S.A., Departmento de Hidrometeorologia, Sección de Hidrología* (ETESA). At Canoas, a total of 46 suspended samples with corresponding discharge measurements were collected from November 1983 to August 1998. The maximum observed concentration was about 33.6 milligram per liter (mg/l) corresponding to a measured flow of about 25.9 m^3/s on September 04, 1991. The maximum measured flow was about 58.5 m^3/s with a corresponding concentration of about 9.7 mg/l on November 16, 1996.

At Batatilla, a total of 56 suspended sediment samples were collected from February 03, 1982 through August 12, 1998. The maximum measured concentration was about 282 mg/l corresponding to a flow of about 73.6 m^3/s. A concentration of 120 mg/l was measured corresponding to the maximum measured flow of about 94.9 m^3/s.

During the field visit, the methods of collection of suspended sediment samples and sample analysis were discussed with ETESA. The agency is using standard methods of United States Geological Survey (USGS) for the collection and analysis of the samples.

ACP is collecting suspended sediment samples on the streams entering Lake Madden and Lake Gatun. These include:

- Stations on Streams Entering Lake Madden
  - Río Chagres at Chico
  - Río Pequeni at Candelaria
  - Río Boqueron at Peluca
- Stations on Streams Entering Lake Gatun
  - Río Gatun at Ciento
  - Río Trinidad at El Chorro
  - Río Ciri Grande at Los Canones

The ACP also conducted a sedimentation survey of Lake Madden in 1983 when the Lake was at an elevation of 235 feet (PCC 1987). Jack R. Tutzauer of the ACP revised this report in March 1990 (Tutzauer, March 1990). He estimated the sediment deposited between elevations, 235 feet and 252 feet (normal pool elevation), which was not surveyed in 1983.

8.2 Current Analyses

After a careful review of the analysis performed by ACP, a yield of 1.4 mm/year was considered to be reasonable for the Río Coclé del Norte watershed.

Additional analysis was performed for the Río Coclé del Norte and Río Toabre. The suspended sediment rating curves fitted by ETESA to the observed data points were judged to be reasonable. However, a limiting concentration of about 10,000 mg/l was adopted. The suspended sediment rating curve was revised for the high flows and is shown on Exhibits 21 and 22.

Flow duration curves were developed for the Río Coclé del Norte and the Río Toabre based on daily flows for the period of record. These curves were used with the suspended sediment rating curves to estimate mean annual suspended sediment load. The estimated loads were about 676,330 and 873,800 metric tons per year (mt/yr), respectively. Assuming 15 percent as bed load and a specific weight of about 1.04 mt/m^3 (about 65 pounds per cubic feet, estimated by ACP), the total volumes were about 747,900 and 966,200 m^3/yr, respectively. These are equivalent to about 1.31 and 1.23 mm/yr.

8.3 Sediment Yield

The above analysis indicated that the sediment yield (including bed load) could vary from 1.23 to 1.4 mm/yr from the drainage basin of the Río Coclé del Norte. For a conservative estimate of the reservoir sedimentation analysis, a unit yield of 1.4 mm/yr was adopted. However, it should be realized that this yield is indicative of the current land use in the basin. If deforestation and increased agriculture occur in future, the yield could increase significantly. Therefore, the land use conditions in the basin should be monitored periodically to assess any increase in the sediment yield.

The drainage area at the dam site is about 1,594 km^2. Using a yield of 1.4 mm/yr, the mean annual total sediment inflow in the reservoir would be about 2,232 MCM. In their computations, ACP used a specific weight of 1.04 mt/m^3. Therefore, the annual yield would be about 2.321 million metric tons.

8.4 Analysis of Storage Depletion

Depletion in the reservoir storage was estimated using the methods developed by the United States Bureau of Reclamation (USBR, 1987).

8.4.1 Specific Weight of Sediment

The reservoir operation was assumed as Type II (USBR classification, normally moderate to considerable drawdown). The particle size distribution of the deposit was not available. For the purpose of estimating specific weight of fresh deposit and the weights after a period of reservoir operation, the particle size distribution of the sediment was assumed to be about 15 percent clay, 45 percent silt and 40 percent sand based on MWH experience on similar streams. Using the USBR procedure (USBR, 1987), the specific weight of the fresh deposit was about 75 pounds per cubic feet (about 1.2 mt/m^3). The average specific weights for 5-, 10-, 20-, 25-, 50- and 100-year of operation were about 1.21, 1.22, 1.23, 1.23, 1.24 and 1.25 mt/m^3, respectively. These values were used to compute the volume of deposit at the end each period.

8.4.2 Estimate of Storage Depletion and Deposition Level at the Dam

Empirical methods for sediment distribution were adopted. The USBR has developed empirical procedures to distribute sediment in a reservoir and estimate the elevation of the deposit at the dam site. The empirical area-reduction (EAR) method was developed based on the survey of about 30 reservoirs. The method recognizes that the distribution depends upon (1) the manner in which reservoir is to be operated, (2) the texture and size of the deposit, (3) the shape of the reservoir, and (4) the amount of sediment deposited. However, the shape factor was adopted to be the major criteria and the least important factor was the texture and size. The USBR has provided the following criteria to classify a reservoir.

CLASSIFICATION OF RESERVOIR

Classification on Operation	Type	Classification on Shape	Type	Weighted Type
Sediment submerged	I	Lake	I	I
		Flood plain – foot hill	II	I or II
		Hill and gorge	III	II
Moderate draw down	II	Lake	I	I or II
		Flood plain – foot hill	II	II
		Hill and gorge	III	II or III
Considerable draw down	III	Lake	I	II
		Flood plain – foot hill	II	II or III
		Hill and gorge	III	III
Normally empty	IV	All shapes	IV	IV
Predominant Size				
Sand or coarse	I			
Silt	II			
Clay	III			

The USBR has developed sediment distribution curves for Types I, II, III and IV reservoirs, which are used with EAR method. The depth to capacity relationship defines the classification of a reservoir. A moderate to considerable draw down could be expected, therefore, the weighted type for the reservoir was judged to be Type II.

The USBR also uses alternate area increment (AAI) method, which is based on the assumption that the area of sediment deposition remains constant throughout the reservoir depth. This method is almost identical to Type II design curve. The method is often used to estimate the new zero capacity elevation at the dam. Since the objective of this analysis was to define the sediment distribution and also estimate the sediment deposit elevation at the dam site, the AAI method was used.

8.4.3 Results

The USBR's alternate area increment (AAI) method was applied to determine the loss in live capacity and the sediment level at the dam. A steady state reservoir elevation of 35 meters was assumed. With the full supply level at El. 35, for the total basin the deposition in the reservoir would amount to about 200 MCM. Excluding the area above the Toabre Dam, the depositions would be about 100 MCM. This would amount to about 15% and 30% of the gross storage behind the dam. After 100 years of operation, the sediment level at the dam site would be about El. 6 and El. 4 for sediment contributions from the entire basin and the basin without the Toabre Dam drainage respectively, assuming that the current river bed is at elevation 0.0 meters.

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TABLES

Table 1

MEAN MONTHLY NET RESERVOIR EVAPORATION
RÍO COCLÉ DEL NORTE RESERVOIR

Month	Evaporation (mm)
January	112
February	117
March	133
April	123
May	91
June	80
July	84
August	80
September	78
October	80
November	72
December	84
Annual	**1,134**

Table 2

LIST OF RAINFALL AND STREAM GAGING STATIONS
(shown on Exhibit 4)

Rainfall Station

01. San Miguel
02. Escandalosa
03. Río Piedras
04. Candelaria
05. Peluca
06. Chico
07. Salamanca
08. Alhajuela
09. Santa Rosa
10. Balboa Heights
11. FAA
12. Diablo Heights
13. Miraflores
14. Pedro Miguel
15. Hodges Hills
16. Empire
17. Cascadas
18. Gamboa
19. Ciento
20. Agua Clara
21. Boro Colorado
22. Monte Lirio
23. Coco Solo
24. Gatun
25. Limon Bay
26. Gatun West
27. Guacha
28. Cano
29. Raises
30. Humedad
31. Chorro
32. Canones
33. Icacal
34. Miguel de la Borda
35. Boca de Uracillo
36. Santa Ana
37. Chiguiri Arriba
38. Sabanita Verde
39. Cocle del Norte
40. San Lucas
41. Boca de Toabre
42. Coclesito
43. Tambo
44. Toabre

Stream Gaging Stations

A. Río Trinidad at el Chorro (drainage area 172 km^2)
B. Río Ciri Grande at Los Canones (drainage area 186 km^2)
C. Río Indio at Limon (drainage area 376 km^2)
D. Río Indio at Boca de Uracillo (drainage area 365 km^2)
E. Río Toabre at Batatilla (drainage area 788 km^2)
F. Río Cocle del Norte at El Torno (drainage area 672 km^2)
G. Río Cocle del Norte at Canoas (drainage area 571 km^2)

Table 3

MONTHLY MEAN DISCHARGES
RÍO COCLÉ DEL NORTE AT DAM SITE
(m^3/s)

Drainage Area 1594 km^2

YEAR	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP	OCT	NOV	DEC	ANNUAL
1948	67.5	37.1	30.1	25.0	40.2	50.3	120.0	133.4	128.8	122.3	198.1	82.5	86.3
1949	46.6	33.9	25.9	24.5	49.0	152.0	119.4	139.4	176.5	169.9	250.9	212.4	116.7
1950	64.7	44.5	32.3	26.4	92.2	139.6	143.6	186.5	140.4	173.9	208.8	209.3	121.9
1951	95.0	64.1	44.5	34.3	93.8	107.5	102.3	108.2	155.8	138.8	184.4	126.1	104.6
1952	72.5	44.9	30.1	27.8	64.7	120.9	101.6	105.7	151.0	193.9	140.2	178.6	102.7
1953	145.8	76.8	50.9	40.9	105.0	97.6	95.7	84.5	97.4	200.5	200.6	132.1	110.7
1954	83.2	50.9	37.9	33.1	90.2	93.6	168.8	139.0	172.1	153.9	240.1	153.7	118.1
1955	161.3	73.6	46.6	40.2	62.6	154.5	122.1	172.5	198.4	179.0	241.6	164.5	134.7
1956	165.8	74.9	52.6	49.9	120.2	155.1	148.5	121.5	168.0	223.4	175.2	120.2	131.3
1957	65.0	42.4	32.3	26.4	66.1	66.4	65.8	103.0	109.3	193.7	144.4	124.5	86.6
1958	90.7	76.3	50.6	40.9	85.5	96.5	80.9	135.8	121.9	123.8	108.0	108.5	93.3
1959	64.6	31.2	15.4	39.9	40.2	197.5	211.1	131.9	119.5	151.7	146.1	271.0	118.3
1960	128.9	61.6	50.8	77.4	71.7	100.6	83.5	79.8	110.3	152.0	191.5	275.9	115.3
1961	80.3	52.3	36.4	37.0	70.4	92.3	103.1	172.3	115.7	164.6	191.0	199.3	109.5
1962	76.2	42.2	26.9	42.6	67.8	92.2	85.2	115.6	105.6	137.3	183.3	131.2	92.2
1963	64.0	52.3	37.0	156.2	124.9	114.5	112.9	120.5	100.3	115.6	194.4	97.3	107.5
1964	72.8	24.8	20.0	129.5	121.8	157.3	144.6	142.2	141.2	187.3	178.2	67.8	115.6
1965	125.4	47.8	23.9	14.1	88.0	55.8	59.8	75.2	73.2	117.0	133.4	160.7	81.2
1966	71.7	38.6	29.9	38.5	112.0	137.2	110.6	106.8	86.2	156.6	279.2	223.0	115.9
1967	95.5	60.9	29.2	85.4	132.1	181.5	128.1	154.6	159.8	176.6	150.3	113.1	122.3
1968	61.9	64.1	64.7	45.2	90.9	129.0	95.9	110.9	125.9	156.0	136.9	143.8	102.1
1969	44.6	40.7	18.8	27.7	54.6	84.8	59.6	106.5	115.0	163.7	176.0	164.2	88.0
1970	313.0	114.8	55.9	149.3	249.7	86.2	116.4	171.8	191.6	189.4	268.0	353.8	188.3
1971	138.2	68.6	96.8	56.7	106.8	139.9	133.9	144.9	168.6	177.1	114.2	59.1	117.1
1972	105.3	54.7	39.1	90.0	92.2	70.2	75.6	66.8	123.1	113.5	108.0	71.1	84.1
1973	62.4	40.6	19.9	20.7	84.9	125.2	123.6	130.8	176.4	201.9	245.5	200.3	119.4
1974	91.6	73.9	46.7	55.8	94.7	90.3	106.5	114.9	104.6	214.5	155.7	103.4	104.4
1975	62.6	40.7	23.2	16.4	68.9	72.4	97.3	174.0	215.5	235.9	325.0	283.1	134.6
1976	101.5	60.8	37.9	26.2	44.8	44.3	46.0	77.4	137.4	127.5	117.6	49.3	72.5
1977	40.1	29.2	17.5	23.5	42.9	80.9	85.1	172.2	130.4	164.6	123.9	72.8	81.9
1978	47.0	41.8	28.0	179.2	224.6	106.3	66.7	75.0	137.8	149.5	159.5	93.0	109.0
1979	31.3	28.3	21.9	97.5	89.8	143.5	110.3	177.5	144.0	106.2	108.1	157.9	101.4
1980	144.8	49.4	24.5	38.8	56.5	86.4	84.2	146.9	109.8	146.8	155.9	161.7	100.5
1981	127.0	87.8	68.8	185.8	178.7	149.6	140.8	146.6	95.3	132.2	244.5	341.1	158.2
1982	77.8	45.5	31.1	32.8	59.7	85.2	111.5	89.9	100.4	145.2	121.3	61.2	80.2
1983	51.3	23.3	16.1	18.2	131.6	81.5	51.9	69.8	154.9	116.3	106.0	131.0	79.3
1984	90.2	94.7	68.3	24.5	68.3	112.0	145.2	206.9	164.7	163.8	164.5	79.2	115.2
1985	79.6	42.2	38.7	24.1	36.4	141.6	84.6	103.2	99.1	128.5	147.5	169.3	91.2
1986	102.6	55.1	37.1	107.0	113.3	114.1	117.3	123.2	134.1	236.2	226.3	83.0	120.8
1987	48.1	35.4	24.5	59.4	68.3	65.4	85.9	101.6	115.3	229.6	143.3	96.4	89.4
1988	51.3	46.1	23.6	19.9	90.9	89.9	127.5	136.8	137.3	189.1	175.5	96.2	98.7
1989	71.1	44.1	34.6	26.6	83.4	106.4	137.2	147.7	137.2	146.1	192.2	126.6	104.4
1990	76.8	45.1	30.2	25.1	64.2	66.5	106.2	109.1	160.0	207.1	156.6	205.1	104.3
1991	56.6	36.6	79.6	29.0	76.1	96.6	78.7	91.0	152.2	198.5	139.5	132.8	97.3
1992	48.6	30.8	21.0	71.1	118.4	125.3	102.6	122.6	176.9	140.6	118.2	85.1	96.8
1993	78.3	42.0	47.1	48.6	63.5	128.8	108.5	83.6	204.8	151.5	242.8	165.5	113.8
1994	59.2	40.7	35.1	46.9	96.6	150.4	94.4	133.1	162.3	164.6	149.7	75.3	100.7
1995	49.8	37.2	30.0	43.3	113.8	108.1	132.2	138.3	163.3	106.6	133.0	180.2	103.0
1996	297.4	118.0	74.9	50.0	170.1	167.2	185.4	210.7	232.9	181.3	216.2	319.4	185.3
1997	77.0	57.5	36.1	30.6	75.5	63.5	70.9	87.0	81.3	125.8	103.4	53.5	71.8
1998	38.2	35.1	27.5	67.4	74.8	71.3	104.0	90.8	113.4	154.3	107.2	140.4	85.4
1999	89.5	55.2	38.1	53.4	92.3	110.4	108.8	127.8	141.2	162.8	172.9	149.2	108.5
Mean	89.5	52.1	37.7	53.5	91.8	108.8	107.7	125.3	139.2	162.7	173.0	149.2	107.5
Maximum	313.0	118.0	96.8	185.8	249.7	197.5	211.1	210.7	232.9	236.2	325.0	353.8	188.3
Minimum	31.3	23.3	15.4	14.1	36.4	44.3	46.0	66.8	73.2	106.2	103.4	49.3	71.8

MWH

Table 4

DROUGHT-DURATION-FREQUENCY ANALYSIS
RÍO COCLÉ DEL NORTE AT DAM SITE

ACCUMULATED 6-MONTH FLOWS

Rank of Event	Return Period (Years)	6-Month Flow (MCM)	Date of Occurrence	
			From	To
1	52.00	525	Dec 76	May 77
2	26.00	648	Jan 48	Jun 48
3	17.33	674	Feb 76	Jul 76
4	13.00	680	Dec 48	May 49
5	10.40	703	Jan 69	Jun 69
6	8.67	737	Jan 75	Jun 75
7	7.43	750	Feb 65	Jul 65
8	6.50	756	Nov 82	Apr 83
9	5.78	768	Dec 97	May 98
10	5.20	774	Jan 57	Jun 57
11	4.73	776	Dec 72	May 73
12	4.33	777	Dec 58	May 59
13	4.00	778	Dec 84	May 85
14	3.71	780	Jan 87	Jun 87
15	3.47	798	Jan 90	Jun 90
16	3.25	834	Jan 88	Jun 88
17	3.06	861	Jan 82	Jun 82
18	2.89	866	Feb 97	May 97
19	2.74	880	Feb 80	Jul 80
20	2.60	902	Jan 62	Jun 62
21	2.48	906	Dec 94	May 95
22	2.36	923	Dec 88	May 89
23	2.26	935	Jan 52	Jun 52
24	2.17	938	Dec 78	May 79
25	2.08	945	Dec 92	May 93
26	2.00	956	Jan 61	Jun 61
27	1.93	971	Jan 91	Jun 91
28	1.86	1,008	Jan 54	Jun 54
29	1.79	1,036	Jan 50	Jun 50
30	1.73	1,076	Jan 92	Jun 92
31	1.68	1,093	Feb 72	Jul 72
32	1.63	1,109	Jan 66	Jun 66
33	1.58	1,111	Jan 94	Jun 94
34	1.53	1,116	Feb 58	Jul 58
35	1.49	1,138	Jan 99	Jun 99
36	1.44	1,139	Jan 51	Jun 51
37	1.41	1,140	Dec 67	May 68
38	1.37	1,155	Feb 60	Jul 60
39	1.33	1,174	Jan 74	Jun 74
40	1.30	1,187	Jan 84	Jun 84
41	1.27	1,208	Dec 63	May 64
42	1.24	1,210	Feb 52	Jul 53
43	1.21	1,239	Oct 77	Mar 78
44	1.18	1,295	Feb 55	Jul 55
45	1.16	1,371	Jan 86	Jun 86
46	1.13	1,423	Jan 63	Jun 63
47	1.11	1,504	Jun 83	Nov 83
48	1.08	1,515	Jan 67	Jun 67
49	1.06	1,558	Feb 56	Jul 56
50	1.04	1,562	Feb 71	Jul 71
51	1.02	1,661	Jun 98	Nov 98
52	1.00	1,801	Jun 78	Nov 78

Table 4, cont.

DROUGHT-DURATION-FREQUENCY ANALYSIS
RÍO COCLÉ DEL NORTE AT DAM SITE

ACCUMULATED 12-MONTH FLOWS

Rank of Event	Return Period (Years)	12-Month Flow (MCM)	Date of Occurrence	
			From	To
1	52.00	1,951	Jun 76	May 77
2	26.00	2,054	Apr 97	Mar 98
3	17.33	2,220	Jun 72	May 73
4	13.00	2,260	Sep 82	Aug 83
5	10.40	2,284	Dec 64	Nov 65
6	8.67	2,505	Jul 58	Jun 59
7	7.43	2,564	Oct 68	Sep 69
8	6.50	2,604	Dec 84	Nov 85
9	5.78	2,609	Jun 48	May 49
10	5.20	2,682	Dec 56	Nov 57
11	4.73	2,713	May 87	Apr 88
12	4.33	2,739	Jun 78	May 79
13	4.00	2,785	Aug 74	Jul 75
14	3.71	2,831	May 98	Apr 99
15	3.47	2,835	Feb 62	Jan 63
16	3.25	2,837	Apr 91	Mar 92
17	3.06	2,886	Oct 79	Sep 80
18	2.89	2,906	Sep 92	Aug 93
19	2.74	2,917	Sep 89	Aug 90
20	2.60	2,931	Dec 94	Nov 95
21	2.48	3,015	Jun 51	May 52
22	2.36	3,110	Jul 53	Jun 54
23	2.26	3,141	Jun 88	May 89
24	2.17	3,173	Jun 63	May 64
25	2.08	3,184	Oct 67	Sep 68
26	2.00	3,270	Jul 60	Jun 61
27	1.93	3,365	Dec 93	Nov 94
28	1.86	3,370	Apr 71	Mar 72
29	1.79	3,407	May 86	Apr 87
30	1.73	3,417	Sep 83	Aug 84
31	1.68	3,442	Dec 65	Nov 66
32	1.63	3,490	Sep 81	Aug 82
33	1.58	3,501	Jun 77	May 78
34	1.53	3,598	Jul 52	Jun 53
35	1.49	3,725	May 49	Apr 50
36	1.44	3,925	Aug 73	Jul 74
37	1.41	3,939	Aug 54	Jul 55
38	1.37	3,945	Jul 59	Jun 60
39	1.33	3,972	May 50	Apr 51
40	1.30	4,198	Dec 55	Nov 56
41	1.27	4,934	Apr 96	Mar 97
42	1.24	5,060	Oct 69	Sep 70

ACCUMULATED 18-MONTH FLOWS

Rank of Event	Return Period (Years)	18-Month Flow (MCM)	Date of Occurrence	
			From	To
1	52.00	2,821	Feb 76	Jul 77
2	26.00	3,049	Jan 97	Jun 98
3	17.33	3,260	Feb 82	May 83
4	13.00	3,362	Dec 71	May 73
5	10.40	3,454	Dec 64	May 66
6	8.67	3,543	Jan 48	Jun 49
7	7.43	3,597	Dec 86	May 88
8	6.50	3,719	Dec 57	May 59
9	5.78	3,872	Feb 68	Jul 69
10	5.20	3,983	Jan 74	Jun 75
11	4.73	4,046	Jan 89	Jun 90
12	4.33	4,068	Jan 92	Jun 93
13	4.00	4,118	Dec 84	May 86
14	3.71	4,122	Jan 94	Jun 95
15	3.47	4,188	Jul 51	Jun 52
16	3.25	4,190	Jan 79	Jun 80
17	3.06	4,289	Jan 62	Jun 63
18	2.89	4,450	Jan 53	Jun 54
19	2.74	4,476	Feb 60	Jul 61
20	2.60	4,598	Feb 56	Jul 57
21	2.48	5,214	Jul 98	Dec 99
22	2.36	5,472	Jul 90	Dec 91
23	2.26	6,171	Aug 66	Jan 68
24	2.17	6,559	Jul 49	Dec 50
25	2.08	6,847	Aug 54	Jan 56
26	2.00	6,966	Feb 70	Jul 71
27	1.93	6,991	Aug 80	Jan 82
28	1.86	7,975	Jul 95	Dec 96

Table 4, cont.

DROUGHT-DURATION-FREQUENCY ANALYSIS
RÍO COCLÉ DEL NORTE AT DAM SITE

ACCUMULATED 24-MONTH FLOWS

Rank of Event	Return Period (Years)	24-Month Flow (MCM)	Date of Occurrence	
			From	To
1	52.00	4,589	Apr 76	Mar 78
2	26.00	4,890	Jan 97	Dec 98
3	17.33	4,960	Jan 82	Dec 83
4	13.00	5,464	Nov 64	Oct 66
5	10.40	5,488	Jun 57	May 59
6	8.67	5,538	Sep 71	Aug 73
7	7.43	5,714	Nov 67	Oct 69
8	6.50	5,816	Dec 86	Nov 88
9	5.78	5,956	Jul 78	Jun 80
10	5.20	6,021	Apr 91	Mar 93
11	4.73	6,138	Apr 62	Mar 64
12	4.33	6,211	Dec 88	Nov 90
13	4.00	6,236	Oct 84	Sep 86
14	3.71	6,296	Dec 93	Nov 95
15	3.47	6,305	Feb 48	Jan 50
16	3.25	6,417	Oct 51	Sep 53
17	3.06	6,746	Apr 60	Mar 62
18	2.89	6,823	Sep 73	Aug 75
19	2.74	7,672	Nov 53	Oct 55

Table 5

MONTHLY MEAN DISCHARGES
RÍO COCLÉ DEL NORTE DAM SITE EXCLUDING TOABRE DAM DRAINAGE
(m^3/s)

Drainage Area: 865.4 km^2

Year	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Mean
1948	49.4	30.1	25.2	21.4	32.2	38.8	78.2	85.0	82.7	79.4	115.0	58.1	**58.0**
1949	36.4	27.9	22.1	21.0	38.0	94.0	77.9	87.9	105.4	102.4	137.4	121.3	**72.6**
1950	47.7	35.0	26.7	22.5	63.5	88.0	90.0	109.9	88.4	104.2	119.7	120.0	**76.3**
1951	65.1	47.4	35.0	28.2	64.4	71.8	69.0	72.1	95.8	87.6	109.0	81.4	**68.9**
1952	52.4	35.3	25.2	23.5	47.7	78.7	68.6	70.8	93.5	113.2	88.3	106.4	**67.0**
1953	91.0	54.9	39.2	32.7	70.5	66.5	65.5	59.3	66.4	116.1	116.2	84.4	**71.9**
1954	58.6	39.2	30.6	27.3	62.5	64.3	101.9	87.7	103.4	94.9	133.0	94.8	**74.8**
1955	98.4	53.0	36.4	32.2	46.5	95.2	79.3	103.6	115.2	106.5	133.6	99.9	**83.3**
1956	100.5	53.8	40.3	38.6	78.4	95.5	92.3	79.0	101.5	126.0	104.8	78.4	**82.4**
1967	47.9	33.6	26.7	22.5	48.6	48.8	48.4	69.4	72.7	113.1	90.4	80.5	**58.6**
1958	62.7	54.6	39.0	32.7	59.8	65.9	50.0	81.9	66.8	93.0	58.5	76.9	**61.8**
1959	47.3	23.4	11.2	22.6	32.2	75.3	75.3	88.2	72.5	93.0	90.4	202.2	**69.5**
1960	98.5	50.4	43.9	63.9	44.7	61.8	43.5	78.4	73.2	94.0	112.2	147.4	**76.0**
1961	56.9	40.1	29.6	30.0	56.3	59.8	62.3	106.4	66.6	91.2	130.1	153.2	**73.5**
1962	61.8	36.1	23.2	38.0	53.3	60.9	57.4	69.8	60.4	79.3	122.9	96.5	**63.3**
1963	44.4	41.3	33.1	132.6	87.2	66.5	64.0	65.7	57.8	63.2	131.4	66.8	**71.2**
1964	56.5	16.7	16.6	108.4	83.6	107.1	83.4	75.6	69.8	107.7	82.3	35.0	**70.2**
1965	106.8	35.9	16.2	9.3	87.6	44.2	50.2	50.1	52.8	77.8	84.9	111.5	**60.6**
1966	53.3	28.8	23.3	32.3	74.6	86.3	68.4	70.3	58.7	78.3	185.4	149.4	**75.8**
1967	70.4	48.7	21.9	78.7	104.1	110.5	71.7	94.8	78.3	83.3	88.2	85.3	**78.0**
1968	48.2	55.6	61.2	40.7	77.7	85.1	61.6	62.2	76.1	72.0	54.0	108.0	**66.9**
1969	25.6	30.3	12.7	18.8	38.1	57.3	35.2	66.7	64.0	110.5	115.6	115.9	**57.6**
1970	71.3	46.2	42.3	43.7	103.1	57.3	68.3	103.8	139.1	105.5	117.8	149.7	**87.3**
1971	74.6	55.2	40.7	30.0	82.0	84.9	79.3	84.7	103.7	106.9	67.9	39.5	**70.8**
1972	72.8	42.9	31.4	69.0	76.2	41.5	55.4	48.8	80.9	74.2	65.5	52.5	**59.3**
1973	52.9	34.9	16.0	17.0	68.8	77.7	89.3	91.4	110.1	110.8	132.1	102.0	**75.2**
1974	70.1	38.4	29.0	29.3	85.6	62.2	64.9	75.9	66.2	130.1	101.7	65.4	**68.2**
1975	50.4	29.7	14.9	13.8	55.9	48.6	63.1	112.8	131.4	128.8	191.9	184.2	**85.4**
1976	70.8	50.2	32.6	21.6	33.6	27.7	30.6	46.0	88.4	91.3	68.6	27.9	**49.1**
1977	30.6	23.6	13.7	20.7	33.5	51.5	53.2	106.6	82.4	90.8	77.5	48.0	**52.7**
1978	35.7	35.0	21.4	140.4	145.5	69.2	34.9	37.7	80.6	87.3	84.7	66.6	**69.9**
1979	19.8	22.3	18.7	87.4	61.4	82.2	78.9	107.8	92.4	80.5	77.3	109.3	**69.8**
1980	106.6	37.1	17.6	30.0	36.4	55.1	50.2	83.1	63.7	76.8	95.0	123.5	**64.6**
1981	99.2	69.2	50.6	148.1	116.3	92.1	65.9	107.4	80.0	94.6	157.1	235.7	**109.7**
1982	49.5	33.2	23.8	26.0	46.2	50.9	74.8	56.5	58.7	80.0	74.6	46.2	**51.7**
1983	43.7	18.9	13.7	15.7	98.4	48.0	31.2	44.0	89.9	86.5	78.1	82.0	**54.2**
1984	51.9	78.9	56.5	20.1	51.4	69.7	83.2	140.4	93.4	89.6	98.4	56.5	**74.2**
1985	65.3	34.1	33.7	21.0	29.6	88.9	47.0	52.2	48.2	74.4	77.1	126.5	**58.2**
1986	89.4	47.6	32.3	87.3	86.8	69.9	85.9	79.9	85.3	131.3	127.2	58.4	**81.8**
1987	37.4	28.8	20.9	44.5	49.9	48.2	60.1	68.6	75.9	128.6	89.8	65.8	**59.9**
1988	39.5	36.1	20.4	17.6	62.8	62.3	82.1	86.6	86.9	111.1	105.0	65.7	**64.7**
1989	51.6	34.8	28.4	22.6	58.7	71.2	86.9	91.9	86.9	91.2	112.5	81.6	**68.2**
1990	54.9	35.4	25.2	21.4	47.5	48.8	71.1	72.6	97.8	119.0	96.2	118.1	**67.3**
1991	42.8	29.8	56.5	24.4	54.5	66.0	56.0	62.9	94.1	115.2	88.0	84.7	**64.6**
1992	37.7	25.6	18.4	51.6	77.4	80.9	69.2	79.6	105.6	88.5	77.3	59.6	**64.3**
1993	55.8	33.4	36.8	37.7	47.1	82.7	72.3	58.8	118.0	93.8	134.1	100.3	**72.6**
1994	44.4	32.5	28.8	36.6	65.9	93.2	64.7	84.9	98.8	99.9	92.9	54.0	**66.4**
1995	38.5	30.1	25.2	34.3	75.1	72.1	84.4	87.4	99.3	71.3	84.8	107.1	**67.5**
1996	155.8	77.2	53.8	38.6	102.5	101.2	109.4	120.5	130.0	107.6	122.9	132.7	**104.4**
1997	55.0	43.4	29.4	25.6	54.1	47.1	51.5	60.6	57.5	81.2	69.6	40.8	**51.3**
1998	30.9	28.7	23.3	49.4	53.7	51.7	69.9	62.8	74.8	95.1	71.6	88.4	**58.4**
1999	62.0	44.8	38.6	41.4	72.1	85.6	70.9	109.3	122.8	87.3	106.1	123.9	**80.4**
Mean	**60.4**	**39.4**	**29.5**	**41.2**	**65.1**	**69.4**	**67.3**	**80.0**	**85.9**	**96.5**	**102.8**	**95.6**	**69.4**
Maximum	**155.8**	**78.9**	**61.2**	**148.1**	**145.5**	**110.5**	**109.4**	**140.4**	**139.1**	**131.3**	**191.9**	**235.7**	**109.7**
Minimum	**19.8**	**16.7**	**11.2**	**9.3**	**29.6**	**27.7**	**30.6**	**37.7**	**48.2**	**63.2**	**54.0**	**27.9**	**49.1**

MWH

Table 6

DROUGHT-DURATION-FREQUENCY ANALYSIS EXCLUDING TOABRE DAM DRAINAGE

Accumulated 6-Month Flows

Rank of Event	Return Period (Years)	6-Month Flow (mcm)
1	53.00	453
2	26.50	538
3	17.67	562
4	13.25	575
5	10.60	591
6	8.83	613
7	7.57	615
8	6.63	617
9	5.89	651
10	5.30	652
11	4.82	655
12	4.42	659
13	4.08	659
14	3.79	668
15	3.53	682
16	3.31	683
17	3.12	687
18	2.94	695
19	2.79	714
20	2.65	730
21	2.52	742
22	2.41	745
23	2.30	761
24	2.21	764
25	2.12	770
26	2.04	774
27	1.96	780
28	1.89	796
29	1.83	799
30	1.77	820
31	1.71	838
32	1.66	845
33	1.61	846
34	1.56	863
35	1.51	863
36	1.47	866
37	1.43	872
38	1.39	884
39	1.36	915
40	1.33	917
41	1.29	952
42	1.26	957
43	1.23	968
44	1.20	999
45	1.18	1016
46	1.15	1019
47	1.13	1043
48	1.10	1069
49	1.08	1086
50	1.06	1097
51	1.04	1112
52	1.02	1126

Accumulated 12-Month Flows

Rank of Event	Return Period (Years)	12-Month Flow (mcm)
1	53.00	1431
2	26.50	1607
3	17.67	1613
4	13.25	1638
5	10.60	1704
6	8.83	1756
7	7.57	1759
8	6.63	1807
9	5.89	1866
10	5.30	1882
11	4.82	1943
12	4.42	1943
13	4.08	1982
14	3.79	2013
15	3.53	2051
16	3.31	2054
17	3.12	2069
18	2.94	2084
19	2.79	2086
20	2.65	2088
21	2.52	2125
22	2.41	2125
23	2.30	2176
24	2.21	2190
25	2.12	2194
26	2.04	2232
27	1.96	2266
28	1.89	2268
29	1.83	2313
30	1.77	2347
31	1.71	2429
32	1.66	2431
33	1.61	2449
34	1.56	2450
35	1.51	2527
36	1.47	2534
37	1.43	2603
38	1.39	2615
39	1.36	2747
40	1.33	2751
41	1.29	2961

MWH

Table 6 (cont.)

DROUGHT-DURATION-FREQUENCY ANALYSIS EXCLUDING TOABRE DAM DRAINAGE

Accumulated 18-Month Flows

Rank of Event	Return Period (Years)	18-Month Flow (mcm)
1	53.00	2124
2	26.50	2370
3	17.67	2404
4	13.25	2615
5	10.60	2626
6	8.83	2663
7	7.57	2663
8	6.63	2711
9	5.89	2717
10	5.30	2848
11	4.82	2916
12	4.42	2952
13	4.08	2970
14	3.79	3016
15	3.53	3034
16	3.31	3037
17	3.12	3159
18	2.94	3168
19	2.79	3210
20	2.65	3211
21	2.52	3461
22	2.41	3689
23	2.30	3859
24	2.21	3892
25	2.12	4147
26	2.04	4204
27	1.96	4375
28	1.89	4822

Accumulated 24-Month Flows

Rank of Event	Return Period (Years)	24-Month Flow (mcm)
1	53.00	3262
2	26.50	3548
3	17.67	3667
4	13.25	3888
5	10.60	3904
6	8.83	3933
7	7.57	4040
8	6.63	4110
9	5.89	4176
10	5.30	4255
11	4.82	4313
12	4.42	4324
13	4.08	4334
14	3.79	4357
15	3.53	4401
16	3.31	4474
17	3.12	4510
18	2.94	4720
19	2.79	5089

Table 7

RÍO COCLÉ DEL NORTE AT CANOAS
MAXIMUM INSTANTANEOUS FLOOD PEAKS
(m^3/s)

Year	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Annual
1983										218	475	392	
1984	398	532	354	36.9	462	366	551	658	315	396	524	179	**658**
1985	468	81	121	82.1	157	625	256	180	131	347	416	1,073	**1,073**
1986	199	123	99	350	546	235	155	458	617	721	815	65.2	**815**
1987	179	128	22	610	343	280	217	238	215	843	200	228	**843**
1988	105	151	38	47.8	617	311					200	145	**617**
1989	152	121	122	59.2	378	503	401	357	283			519	**519**
1990	309	104	223	87.2	561	179						604	**604**
1991	94.5	62.1	1,013	447	435	259	177	430	931	476	258	322	**1,013**
1992	74.7	46.4	37	798	419	285	146	463	229	351	224	139	**798**
1993	451	255	202	338	298	443	296	239	692	527	359		**692**
1994				479	389	1,356	239	473	317	519	442	302	**1,356**
1995	99	42.2	93	569	91.8	268	723	443	272	138	352	940	**940**
1996			908	470	429		147				899	1,019	**1,019**
1997	225	142		176	1,109	156	265	268	430	205	154	60.3	**1,109**
1998	133	113	181	822	713	374	166	168	374	779	475	287	**822**
1999	156	120	515	449	446	731	867						**867**

Table 8

RÍO COCLÉ DEL NORTE AT EL TORNO
MAXIMUM INSTANTANEOUS FLOOD PEAKS
(m^3/s)

Year	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Annual
1958							256	186	251	157	120	150	
1959	133	35.7	16.1	318		151	401	177	203	157	197	662	**662**
1960	299	233	271	145	99.3	138	116	84.7					
1961				108	282	111	115	531	150	236	526	659	**659**
1962	168	114	37.7	146	267	204	154	206	168	225	319	372	**372**
1963	118	139	51.7	581	312	157	320	171	274	226	299	338	**581**
1964	263	48.9	104	482	237	277	213	247	183	566	220	57.4	**566**
1965	424	60	22	16.2	461	231	139	167	83.7	326	161	643	**643**
1966	85.7	45.6	110	136	200	176	197	150	135	266	482	437	**482**
1967	277	165	29.2	226	255	367	259	233	214	264	362	294	**367**
1968	43.5	266	507	340	627	366	575	143	276	201	123	439	**627**
1969	63.8	326	115	72.9	381	556	152	325	457	1070	657	1150	**1150**
1970	3116	782	157	2357	782	689	651	601	787	601	690	542	**3116**
1971	491	45.6	826	105	631	478	548	478	778	636	258	83.9	**826**
1972	264	129	100	379	599	274	627	287	581	439	399	179	**627**
1973	714	42.7	38.1	67.2	395	158	338	303	652	537	590	343	**714**
1974	174	117	87.9	534	442	358	800	415	350	560	392	255	**800**
1975	182	64.7	16.5	36.1	299	251	474	643	1090	548	893	770	**1090**
1976	171	114	55.2	116	85.8	85.8	113	264	943	298	291	79.4	**943**
1977	196	34.4	18.2	96.2	663	1019	291	726	532	511	396	150	**1019**
1978	109	100	198	2599	1645	577	234	176	531	659	495	366	**2599**
1979	36.4	133	41.9	1164	368	638	259	445	443	216	271	1170	**1170**
1980	867	97	26.4	925	433	770	517	793	588	375	261	704	**925**
1981	415	168	227	1072	677	593	576	428	194	461	753	1645	**1645**
1982	364	99	56	219	622	293	357	408	177	524	439	100	**622**
1983	263	47.1	29.3	158	699	354	67.9	402	503	212	537	447	**699**
1984	443	608	474	34.9	566	548	599	890	502	593	572	171	**890**
1985	579	65.6	94.9	59.8	128	1030	242	284	147	355	773	1752	**1752**
1986	487	126	114	445	864	624	234						

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Table 9

RÍO TOABRE AT BATATILLA
MAXIMUM INSTANTANEOUS FLOOD PEAKS
(m^3/s)

Year	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Annual
1958							101	139	422	151	144	121	**422**
1959	42.3	17.7	10.3	26.3	120	139		152	153	153	164	422	**422**
1960	114	33.8	43	153	106	136	104						
1961					82.2	78.7	96.5	152					
1962	132	61.7	57.2		187	523	139	167	209	209	371	185	**523**
1963	62.6	59.9	54.5	267	161	142	136	155	146	150	251	78.1	**267**
1964	69.8	12.8	9.65	375	192	203	150	323	230	357			**375**
1965													
1966													
1967													
1968													
1969					216	462	168	224	751	775	747	442	**775**
1970	2633			929	806	229	414	636	808		1025	1368	**2633**
1971	687		251	22.9	367	557	378	498	842	465	263	105	**842**
1972	751	57.5			141	184	207	175	367	237	329	131	**751**
1973	149	13.7	11.9	11.9	213	369	597	354	765	536		424	**765**
1974	86.7	39.8	20.7				1230	370	232	628	386	312	**1230**
1975	33.2			4.44	75.3					504	842	1044	**1044**
1976	35.3	24	11.9	43	77.9	99.2	88.4	275	403	257	231	79.6	**403**
1977	16.2	9.7	5.82	12.8	166	296	315	393	331	306	185	108	**393**
1978	24.8	18.6	70.9	1817	727	155	247	206	296	522	469	84	**1817**
1979	18.6	11.6	9.59	308	231	293	286	806			108	1108	**1108**
1980	450	24	11.9	308	132	222	299	954	414	398	511		**954**
1981				474	632	523	737	794	412	397	800	1390	**1390**
1982	57.4	26.3	12.9	43	178	511	296	296	328	422	544	36.8	**544**
1983	27.8	8.05	6.04	42.3	437	224	87.6	149	612	555	203	232	**612**
1984	84	136	117	9.39	255	819	389	507	583	1111	386	50.1	**1111**
1985	186	15.8	11.4	8.47	78.9	1276	372	829	247	397	840	757	**1276**
1986	89.9	19.5	26	218	608	1266	261	608	414	943	894	72.6	**1266**
1987	22.4	12.4	6.77	255	218	203	369	363	422	618	236	145	**618**
1988	20.9	18.4	9.24	9.24	804	188	311				354	120	**804**
1989	52.4	14.3	12.4	6.77	570	255	1053	231	306			376	**1053**
1990	72.6	14.6	11.9	11.4	326	168						1080	**1080**
1991	30.5	12.4	920							794	211	381	**920**
1992	20.9	16.4	6.47	513	568	360	392	375	638	369	367	90.9	**638**
1993	153	24.6	274	363	121	585	450	160	727	361	1116	703	**1116**
1994	31.1	17	51.3	242	422	802	108	593	664	424	810	72.2	**810**
1995	17.5	9.5	40.9	349	311	313		603	429	142	369	1355	**1355**
1996	2438	577	568	149	859	723	767	1253	1133	1077	755	898	**2438**
1997	60.7	274	69.7	79	429	145	147	123	236	488	120	31.1	**488**
1998	14.5			488	384	559	422	442	486	759	731	672	**759**
1999		36.6	182	173	392	634	1382	777	429				

Table 10

FLOOD PEAKS FOR SELECTED RETURN PERIODS AT RÍO COCLÉ DEL NORTE DAM SITE

Drainage Area: 1,594 sq.km

All Season

Return Period (years)	Values of K Factor		Mean Value	Flood Peak (m^3/s)
	El Torno	Batatilla		
2	3.99	3.96	3.98	1,295
5	4.37	4.30	4.34	1,925
10	4.62	4.48	4.55	2,430
20	4.85	4.63	4.74	2,995
50	5.14	4.79	4.97	3,860
100	5.36	4.90	5.13	4,610

Dry Season (February and March)

Return Period (years)	Values of K Factor		Mean Value	Flood Peak (m^3/s)
	El Torno	Batatilla		
2	2.46	1.33	1.90	130
5	3.04	2.20	2.62	288
10	3.38	2.69	3.04	458
20	3.69	3.14	3.42	697
50	4.08	3.69	3.89	1,171
100	4.35	4.10	4.23	1,705

Table 11

FLOOD PEAKS FOR SELECTED RETURN PERIODS
AT RÍO COCLÉ DEL NORTE DAM SITE EXCLUDING TOABRE DAM DRAINAGE

Drainage Area = 865.4 km^2

ALL SEASON

Return Period (Years)	Value of "K"	Flood Peak (cms)
2	4.06	983
5	4.35	1379
10	4.53	1701
20	4.68	2026
50	4.87	2528
100	5.01	2976

DRY SEASON

Return Period (Years)	Value of "K"	Flood Peak (cms)
2	2.46	152
5	3.04	299
10	3.38	445
20	3.69	639
50	4.08	1007
100	4.35	1379

A = drainage area in sq.km
Q = flood peak in cubic meters per second (cms)

Table 12

SEQUENTIAL ARRANGEMENT OF PMP INCREMENTS
RÍO COCLÉ DEL NORTE
(Increments as percentages of 48-hour PMP)

Hour	Increment	Hour	Increment
1	0.7	25	3.0
2	0.7	26	4.0
3	0.7	27	4.0
4	0.7	28	13.5
5	0.8	29	14.0
6	0.8	30	6.5
7	0.8	31	4.0
8	0.8	32	3.0
9	0.8	33	3.0
10	0.8	34	2.0
11	0.8	35	2.0
12	0.9	36	1.5
13	0.9	37	1.5
14	1.0	38	1.5
15	1.0	39	1.5
16	1.0	40	1.5
17	1.0	41	1.0
18	1.5	42	1.0
19	1.5	43	1.0
20	1.5	44	0.9
21	1.5	45	0.9
22	2.0	46	0.9
23	2.0	47	0.8
24	2.0	48	0.8

EXHIBITS

MWH

5 0 5 10 km

GRAPHIC SCALE

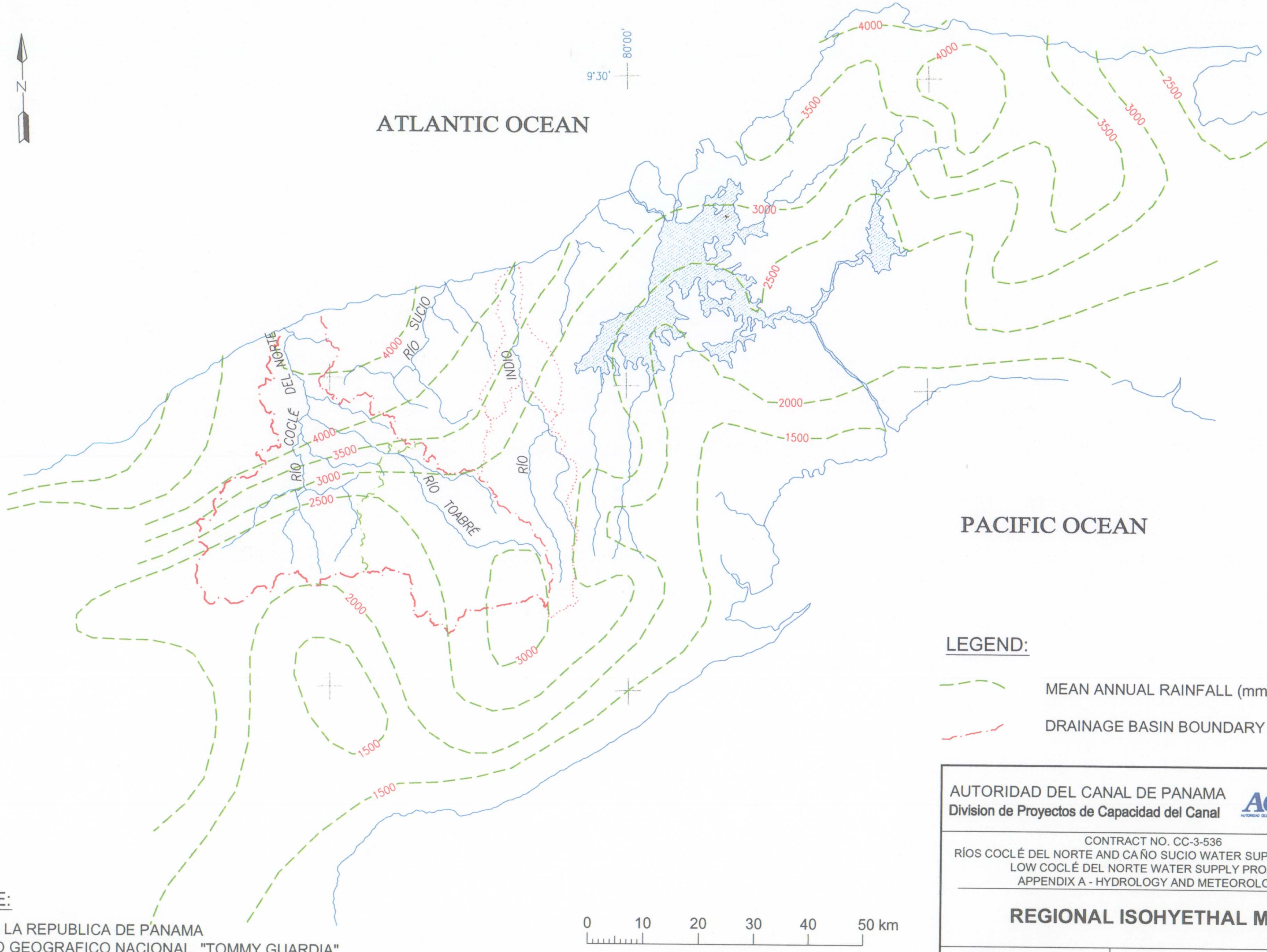
AUTORIDAD DEL CANAL DE PANAMA
Division de Proyectos de Capacidad del Canal

ACP

CONTRACT NO. CC-3-536
RÍOS COCLÉ DEL NORTE AND CAÑO SUCIO WATER SUPPLY PROJECTS
LOW COCLÉ DEL NORTE WATER SUPPLY PROJECT
APPENDIX A - HYDROLOGY AND METEOROLOGY

LOW COCLÉ DEL NORTE LOCATION MAP

MWH	DATE: DECEMBER, 2003	EXHIBIT: 1



SOURCE:
ATLAS DE LA REPUBLICA DE PANAMA
INSTITUTO GEOGRAFICO NACIONAL, "TOMMY GUARDIA"

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ACP

CONTRACT NO. CC-3-536
RÍOS COCLÉ DEL NORTE AND CAÑO SUCIO WATER SUPPLY PROJECTS
LOW COCLÉ DEL NORTE WATER SUPPLY PROJECT
APPENDIX A - HYDROLOGY AND METEOROLOGY

REGIONAL ISOHYETHAL MAP

MWH	DATE: DECEMBER, 2003	EXHIBIT: 2

0 5 10 15 20 25 km

GRAPHIC SCALE

LEGEND:

▲ RAIN GAGE

⊙ STREAM GAGE

— · — DRAINAGE BASIN BOUNDARY

■ VILLAGE

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ACP AUTORIDAD DEL CANAL DE PANAMÁ

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RÍOS COCLÉ DEL NORTE AND CAÑO SUCIO WATER SUPPLY PROJECTS
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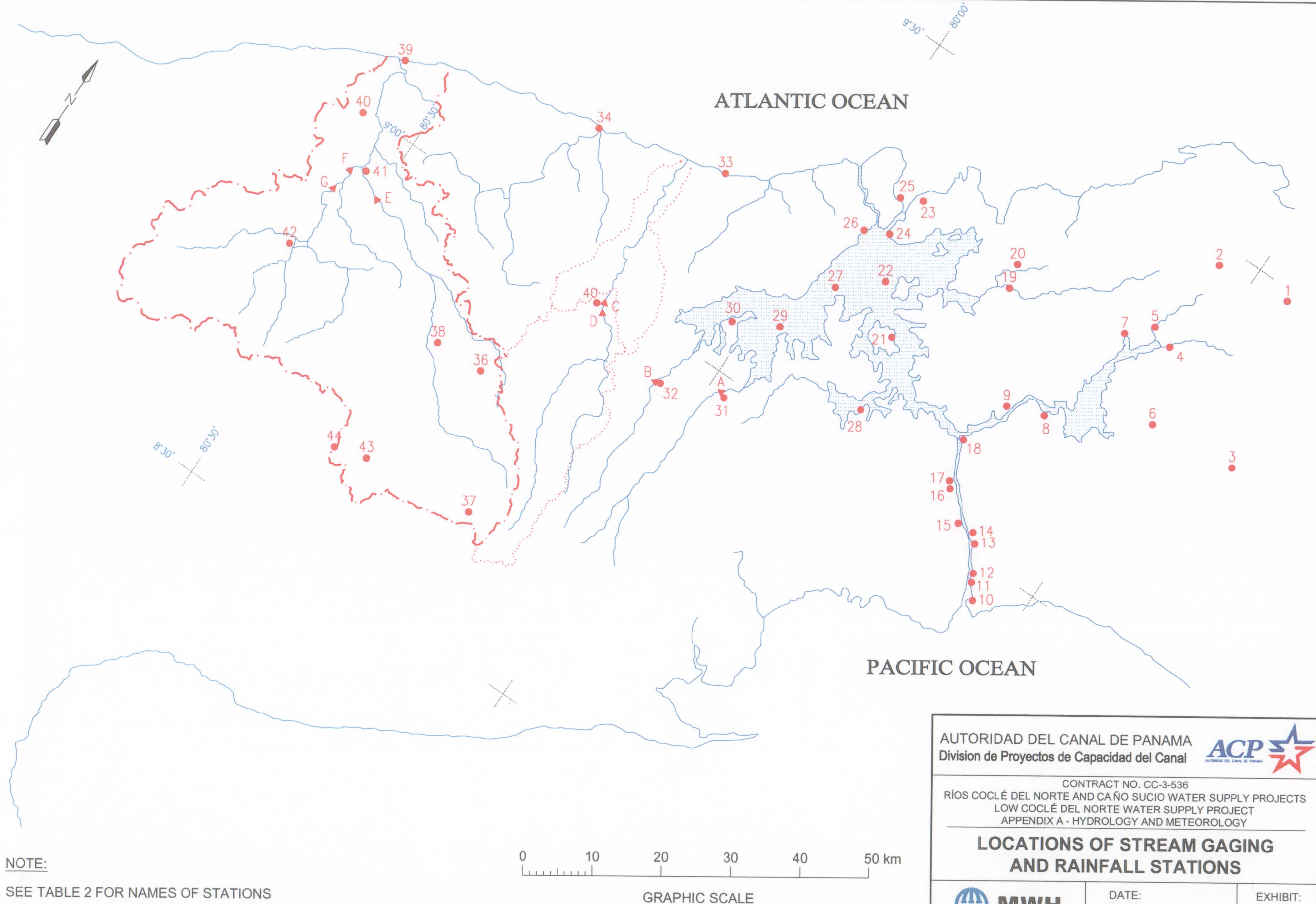
DRAINAGE CONFIGURATION OF THE COCLÉ DEL NORTE BASIN

MWH

DATE: DECEMBER, 2003

EXHIBIT: 3



NOTE:

SEE TABLE 2 FOR NAMES OF STATIONS

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RÍOS COCLÉ DEL NORTE AND CAÑO SUCIO WATER SUPPLY PROJECTS
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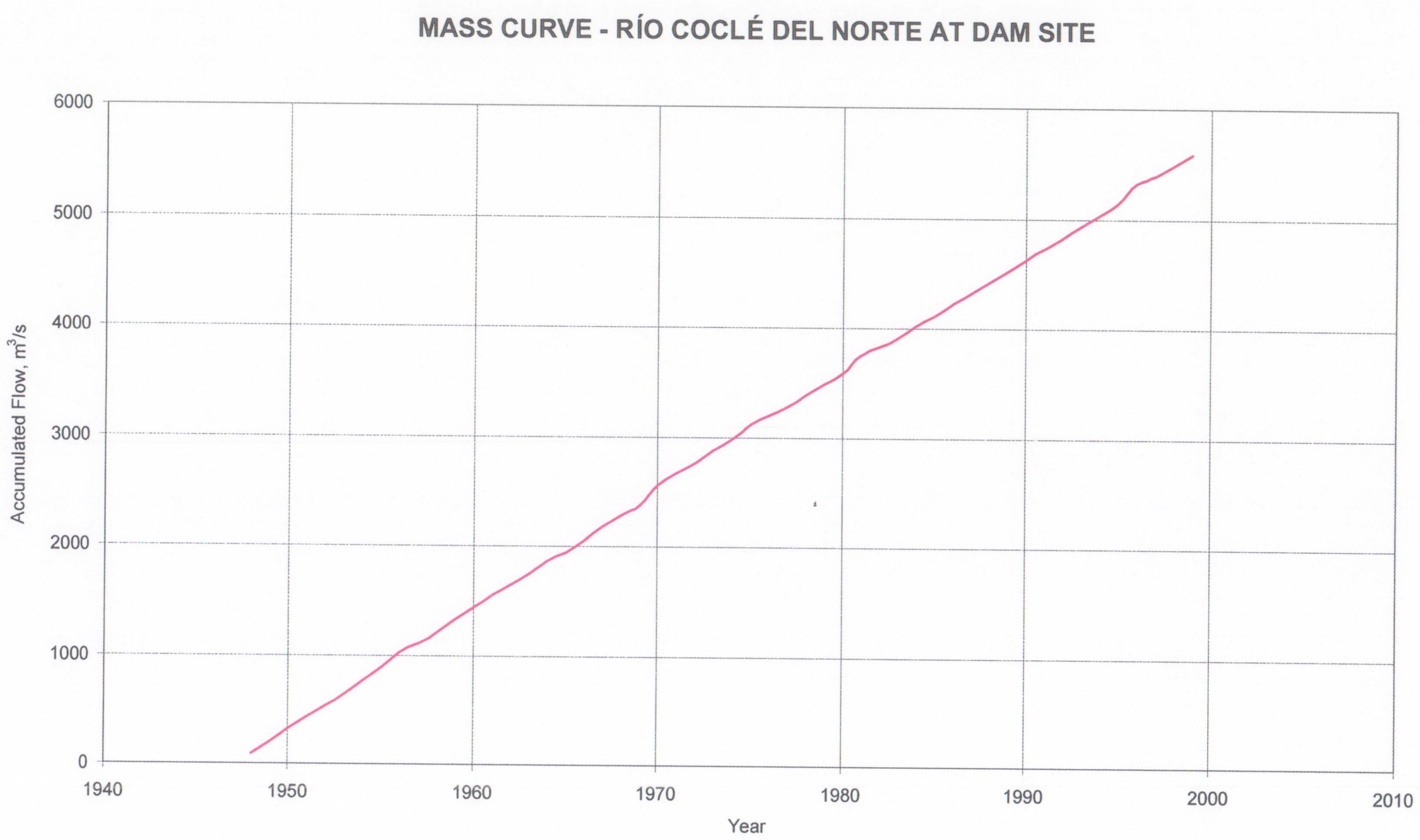
APPENDIX A - HYDROLOGY AND METEOROLOGY

LOCATIONS OF STREAM GAGING AND RAINFALL STATIONS

MWH

DATE: DECEMBER, 2003

EXHIBIT: 4



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RÍOS COCLÉ DEL NORTE AND CAÑO SUCIO WATER SUPPLY PROJECTS
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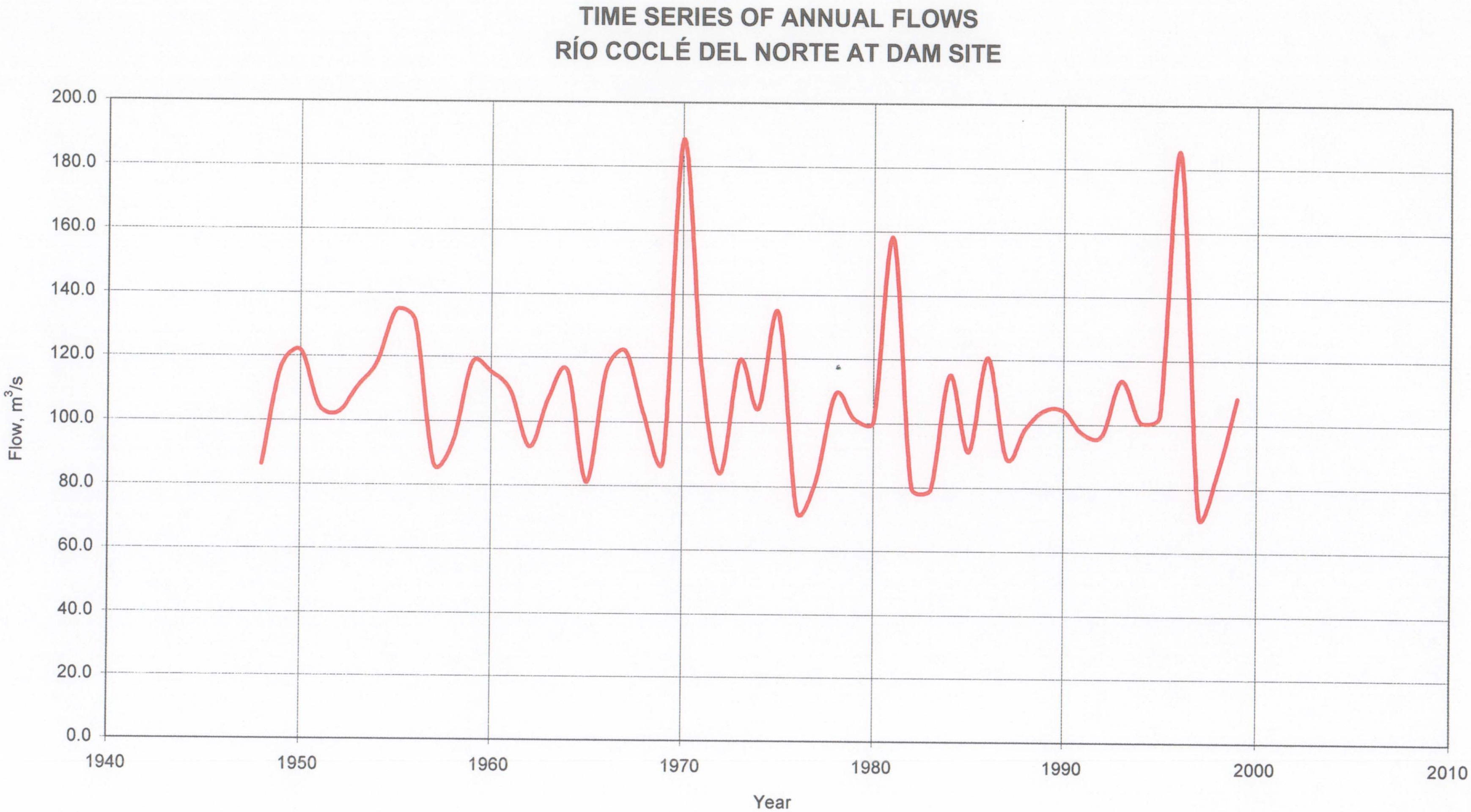
MASS CURVE
RÍO COCLÉ DEL NORTE AT DAM SITE

MWH

DATE: DECEMBER, 2003

EXHIBIT: 5



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Division de Proyectos de Capacidad del Canal

CONTRACT NO. CC-3-536
RÍOS COCLÉ DEL NORTE AND CAÑO SUCIO WATER SUPPLY PROJECTS
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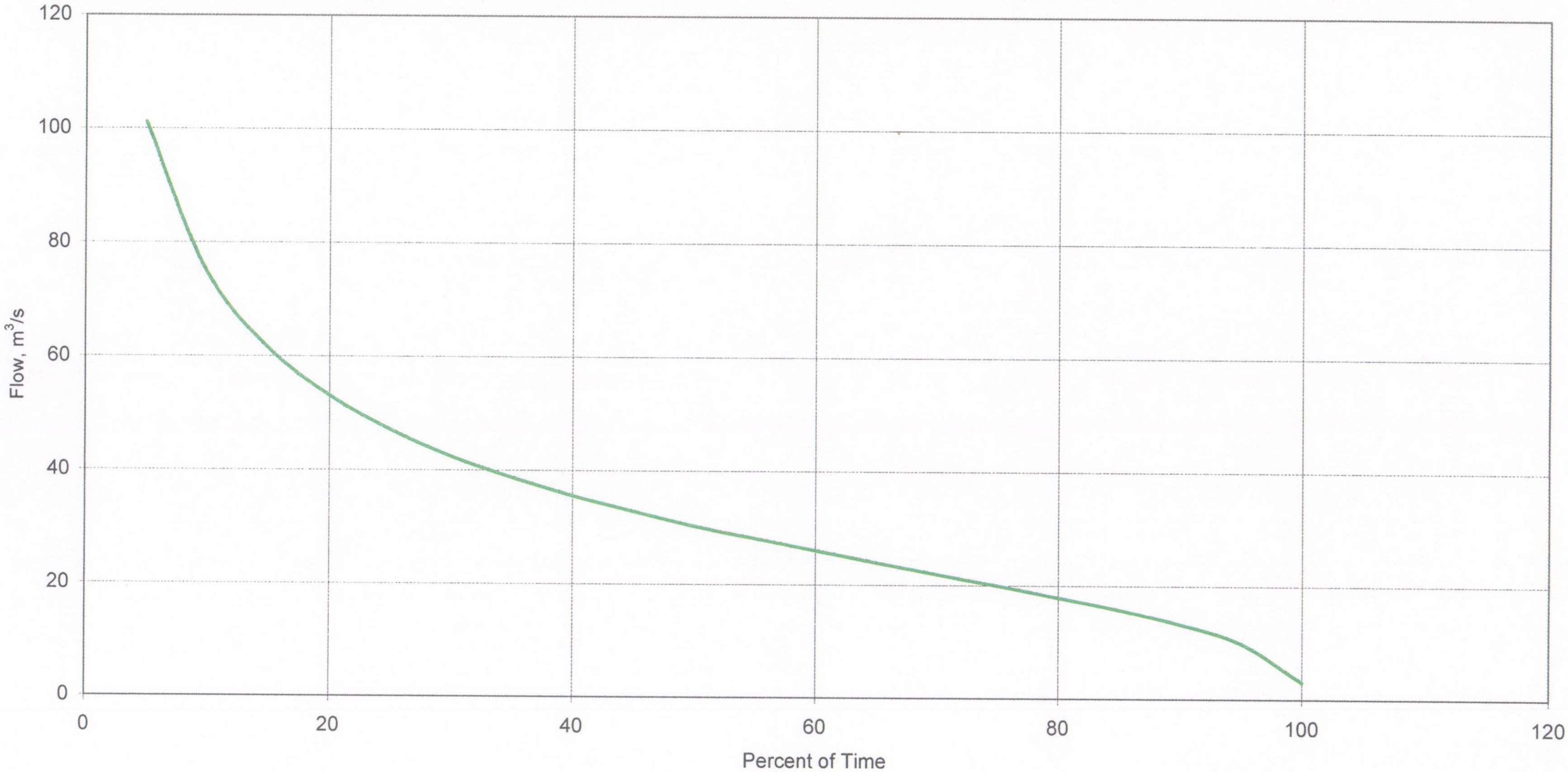
TIME SERIES OF ANNUAL FLOWS
RÍO COCLÉ DEL NORTE AT DAM SITE

MWH	DATE: DECEMBER, 2003	EXHIBIT: 6

FLOW DURATION CURVE - RÍO COCLÉ DEL NORTE AT CANOAS



AUTORIDAD DEL CANAL DE PANAMA
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CONTRACT NO. CC-3-536
RÍOS COCLÉ DEL NORTE AND CAÑO SUCIO WATER SUPPLY PROJECTS
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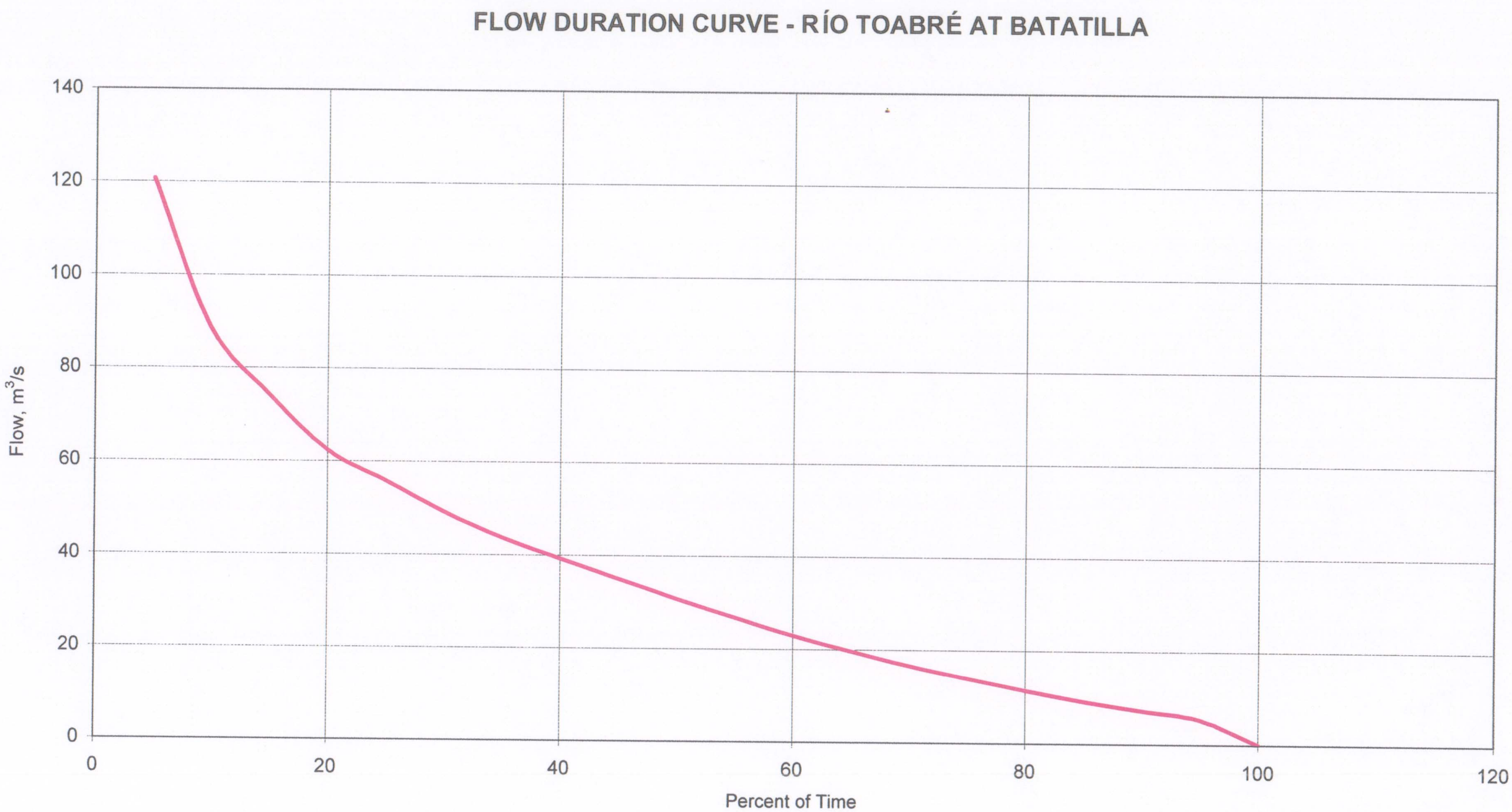
FLOW DURATION CURVE
RÍO COCLÉ DEL NORTE AT CANOAS

MWH	DATE: DECEMBER, 2003	EXHIBIT: 7



AUTORIDAD DEL CANAL DE PANAMA
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CONTRACT NO. CC-3-536
RÍOS COCLÉ DEL NORTE AND CAÑO SUCIO WATER SUPPLY PROJECTS
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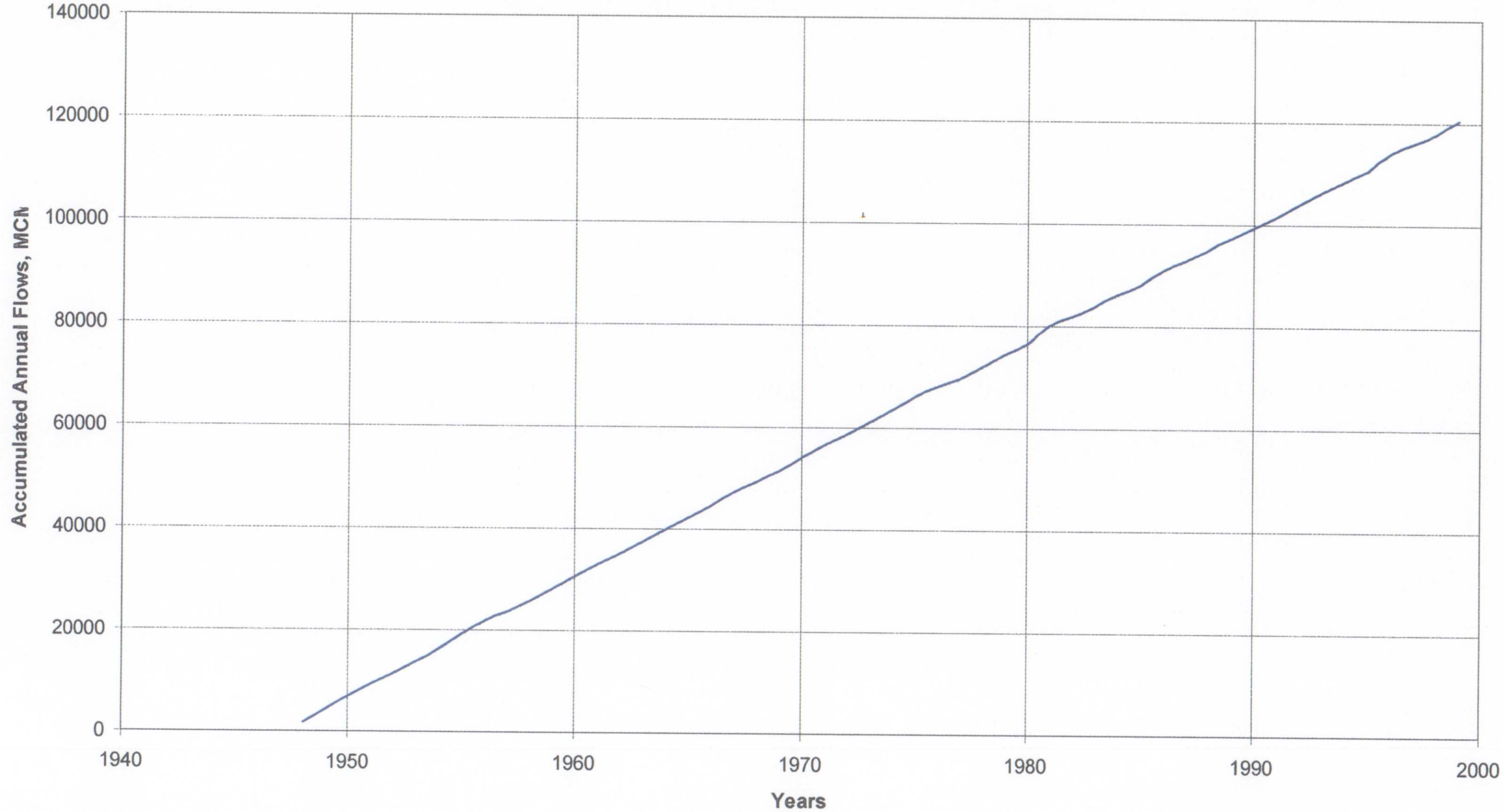
FLOW DURATION CURVE
RÍO TOABRÉ AT BATATILLA

MWH	DATE: DECEMBER, 2003	EXHIBIT: 8

MASS CURVE OF ANNUAL FLOWS
DAM SITE EXCLUDING TOABRÉ DAM DRAINAGE



AUTORIDAD DEL CANAL DE PANAMA
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RÍOS COCLÉ DEL NORTE AND CAÑO SUCIO WATER SUPPLY PROJECTS
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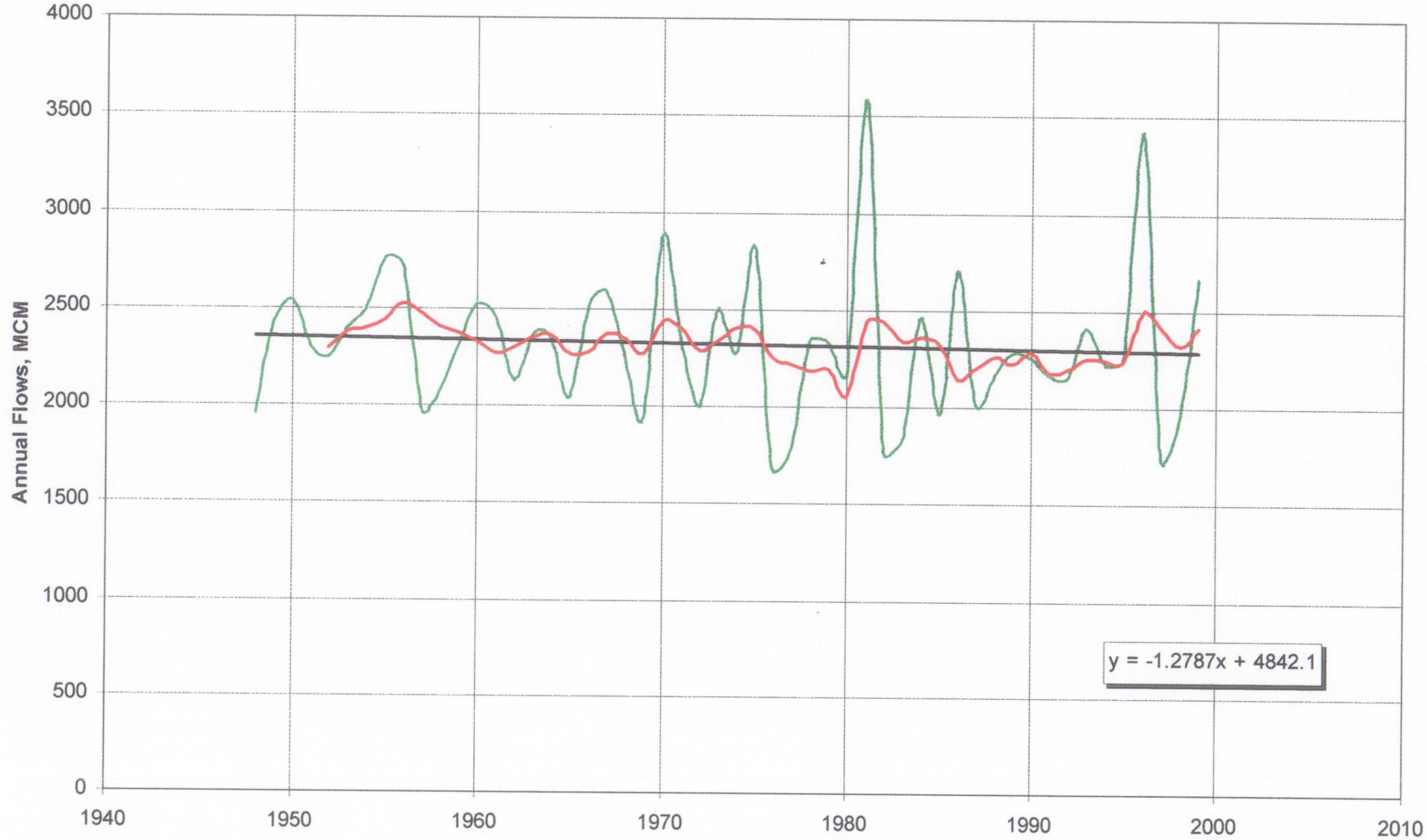
MASS CURVE - DAM SITE
EXCLUDING TOABRÉ DAM DRAINAGE

MWH	DATE: DECEMBER, 2003	EXHIBIT: 9

TIME SERIES OF ANNUAL FLOWS
DAM SITE EXCLUDING TOABRÉ DAM DRAINAGE



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RÍOS COCLÉ DEL NORTE AND CAÑO SUCIO WATER SUPPLY PROJECTS
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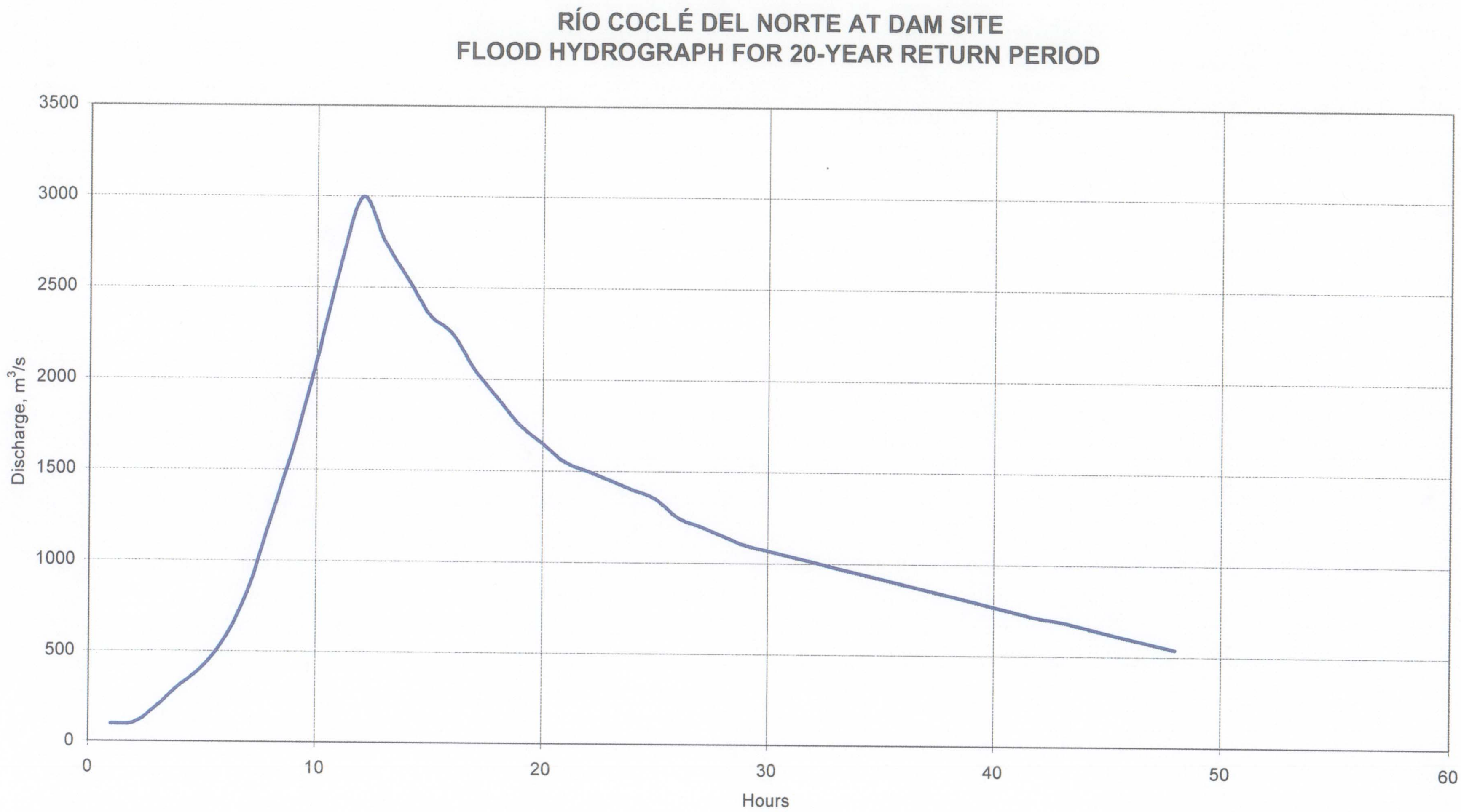
TIME SERIES OF ANNUAL FLOWS DAM SITE EXCLUDING TOABRÉ DAM DRAINAGE

MWH	DATE: DECEMBER, 2003	EXHIBIT: 10



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ACP

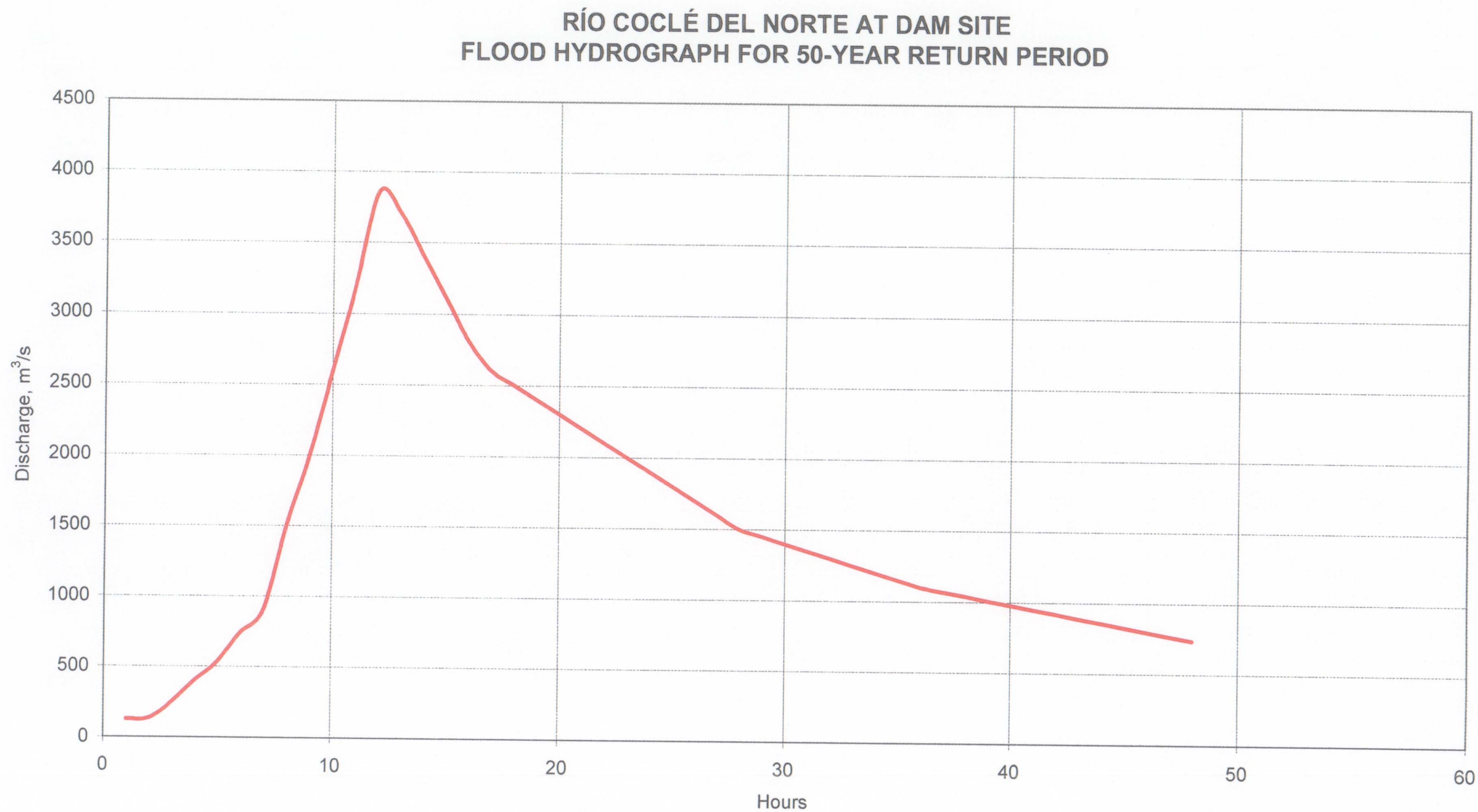
CONTRACT NO. CC-3-536
RÍOS COCLÉ DEL NORTE AND CAÑO SUCIO WATER SUPPLY PROJECTS
LOW COCLÉ DEL NORTE WATER SUPPLY PROJECT
APPENDIX A - HYDROLOGY AND METEOROLOGY

FLOOD HYDROGRAPH FOR 20-YEAR RETURN PERIOD

MWH

DATE: DECEMBER, 2003

EXHIBIT: 11



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ACP

CONTRACT NO. CC-3-536
RÍOS COCLÉ DEL NORTE AND CAÑO SUCIO WATER SUPPLY PROJECTS
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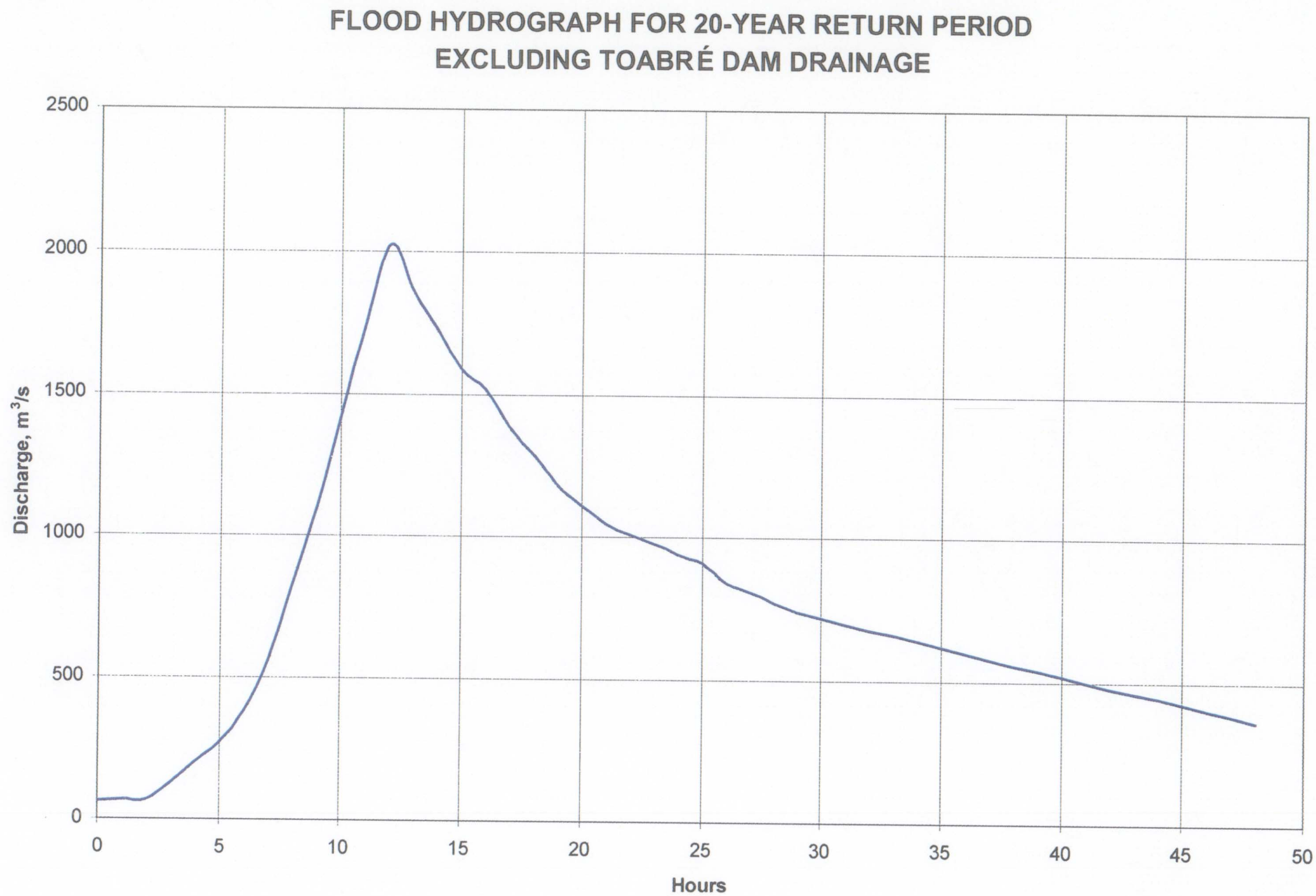
APPENDIX A - HYDROLOGY AND METEOROLOGY

FLOOD HYDROGRAPH FOR 50-YEAR RETURN PERIOD

MWH

DATE: DECEMBER, 2003

EXHIBIT: 12



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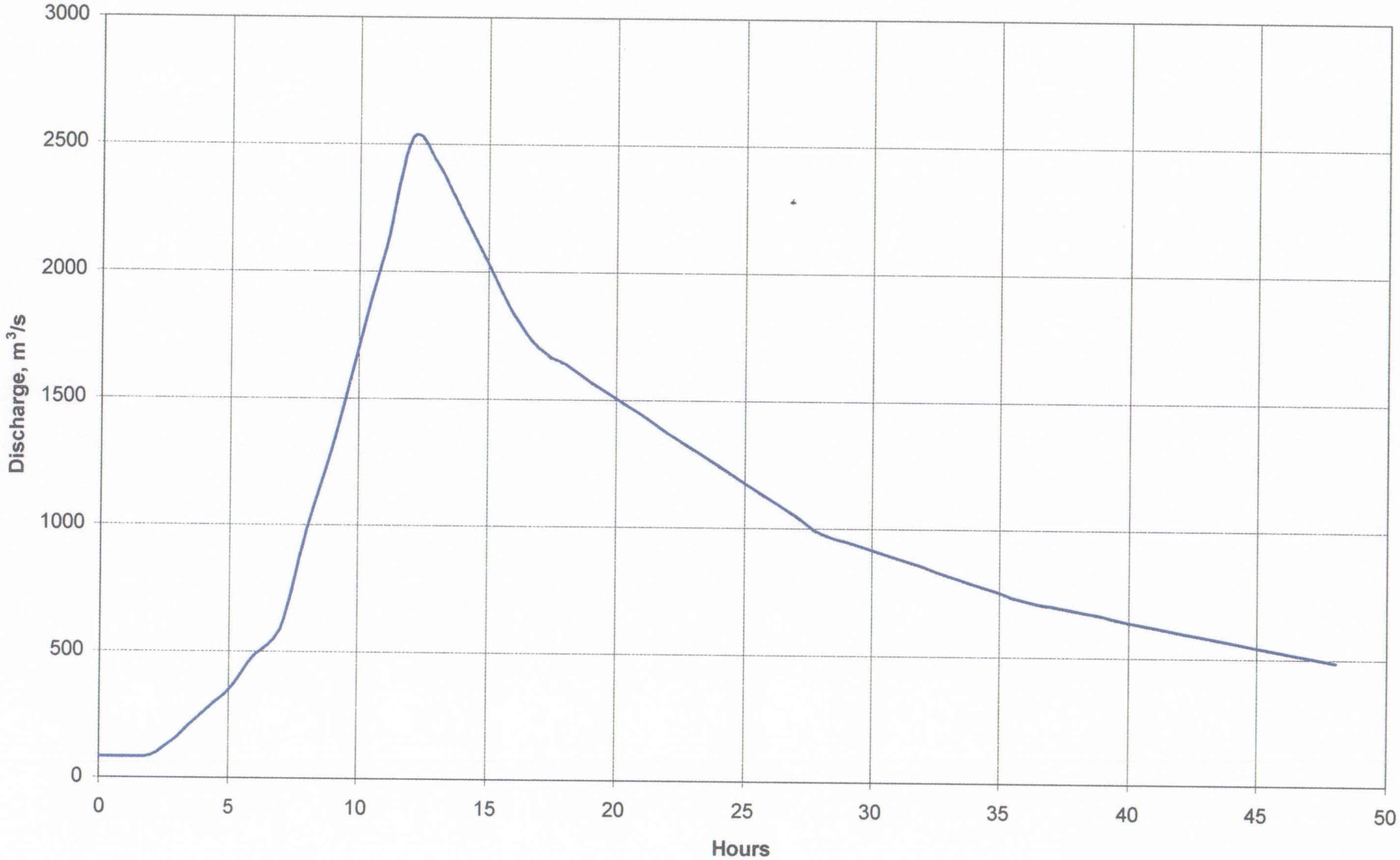
CONTRACT NO. CC-3-536
RÍOS COCLÉ DEL NORTE AND CAÑO SUCIO WATER SUPPLY PROJECTS
LOW COCLÉ DEL NORTE WATER SUPPLY PROJECT
APPENDIX A - HYDROLOGY AND METEOROLOGY

FLOOD HYDROGRAPH FOR 20-YEAR RETURN PERIOD EXCLUDING TOABRÉ DAM DRAINAGE

MWH	DATE: DECEMBER, 2003	EXHIBIT: 13

FLOOD HYDROGRAPH FOR 20-YEAR RETURN PERIOD
EXCLUDING TOABRÉ DAM DRAINAGE



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RÍOS COCLÉ DEL NORTE AND CAÑO SUCIO WATER SUPPLY PROJECTS
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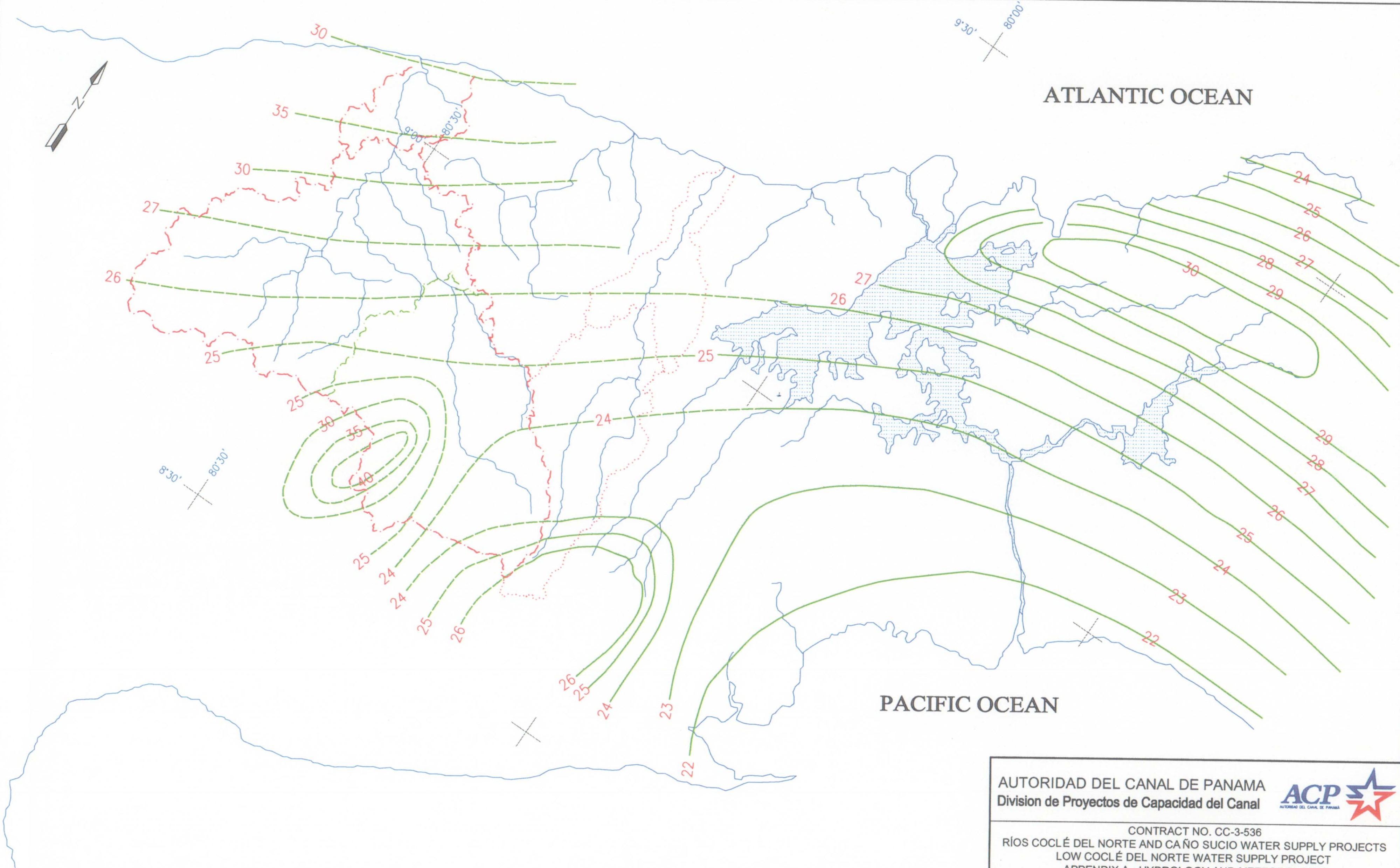
FLOOD HYDROGRAPH FOR 20-YEAR RETURN PERIOD EXCLUDING TOABRÉ DAM DRAINAGE

MWH	DATE: DECEMBER, 2003	EXHIBIT: 14



SOURCE:

US NWS, FEBRUARY 1978 REPORT

0 10 20 30 40 50 km

GRAPHIC SCALE

AUTORIDAD DEL CANAL DE PANAMA
Division de Proyectos de Capacidad del Canal

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CONTRACT NO. CC-3-536
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LOW COCLÉ DEL NORTE WATER SUPPLY PROJECT
APPENDIX A - HYDROLOGY AND METEOROLOGY

24-HOUR, 10-MI^2 PMP

(inches)

MWH

DATE: DECEMBER, 2003

EXHIBIT: 15

SOURCE:

IRHE, DEPARTAMENTO DE HIDROMETEOROLOGIA

AUTORIDAD DEL CANAL DE PANAMA
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RÍOS COCLÉ DEL NORTE AND CAÑO SUCIO WATER SUPPLY PROJECTS
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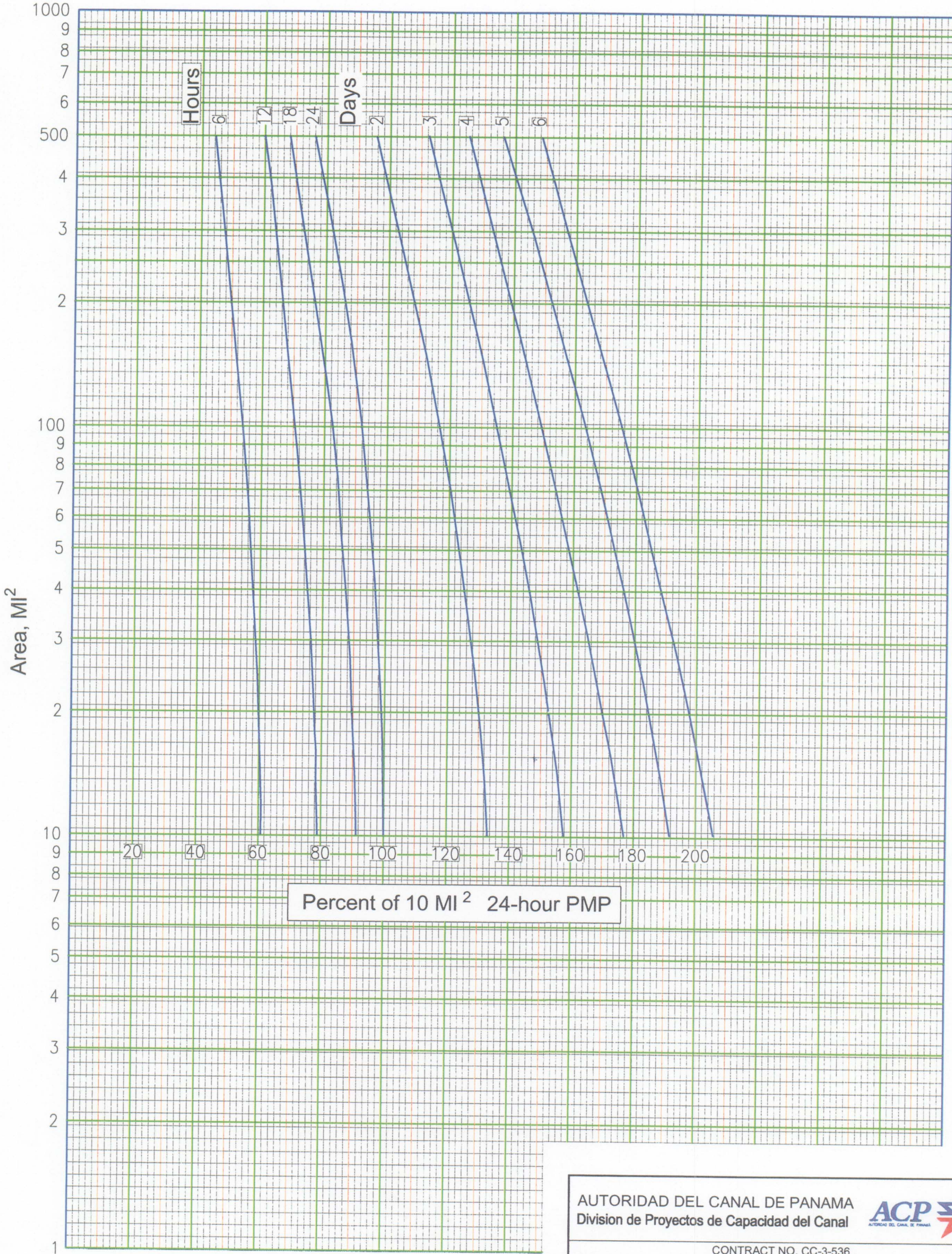
SUB-BASINS OF RIO COCLÉ DEL NORTE BASIN ABOVE THE DAM SITE

MWH

DATE:	EXHIBIT:
DECEMBER, 2003	16



SOURCE:

US WEATHER BUREAU, 1965 REPORT

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RÍOS COCLÉ DEL NORTE AND CAÑO SUCIO WATER SUPPLY PROJECTS
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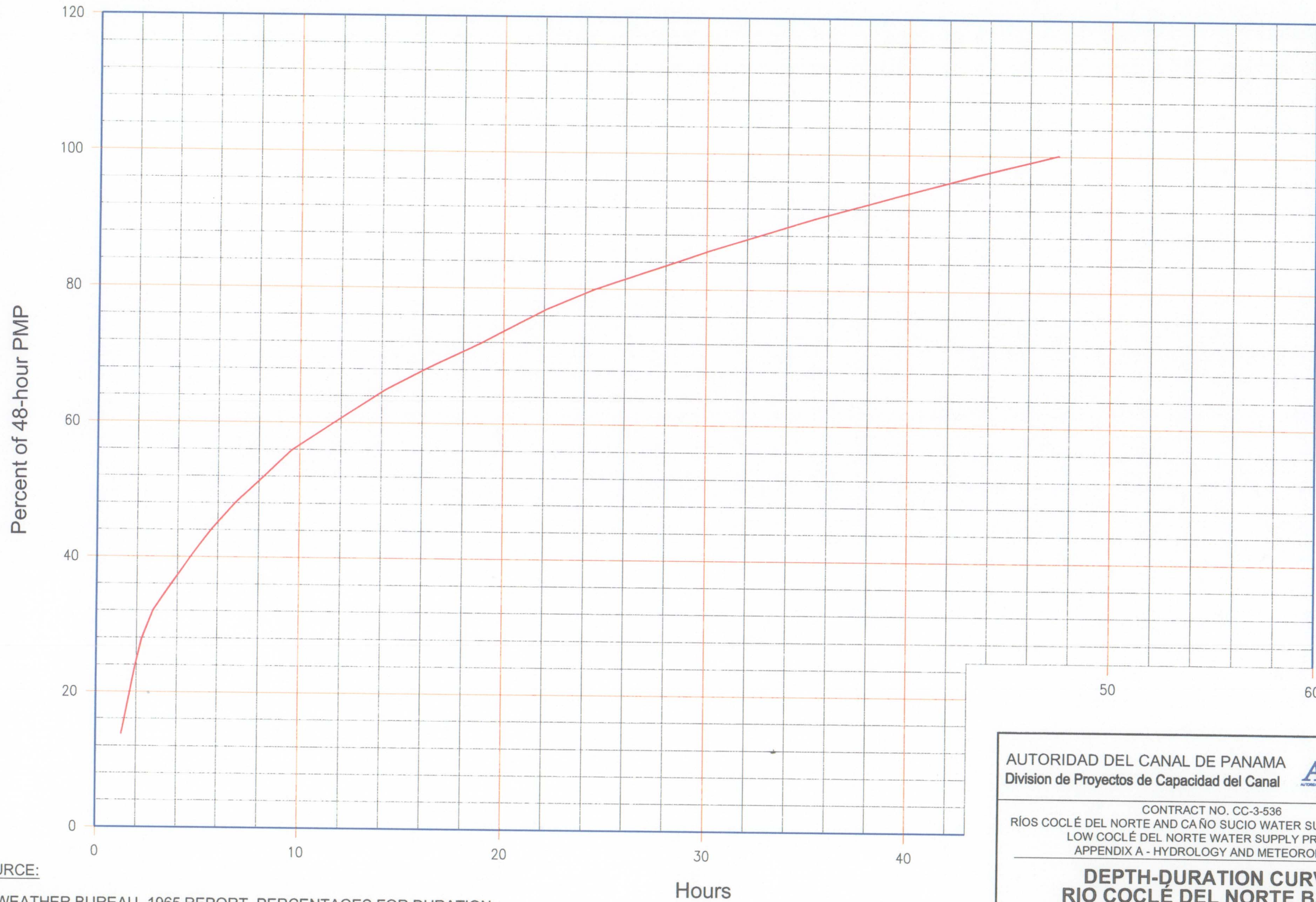
APPENDIX A - HYDROLOGY AND METEOROLOGY

DEPTH-AREA-DURATION CURVES

MWH | DATE: DECEMBER, 2003 | EXHIBIT: 17



SOURCE:

US WEATHER BUREAU, 1965 REPORT, PERCENTAGES FOR DURATION LESS THAN 6 HOURS FROM HOURLY DATA

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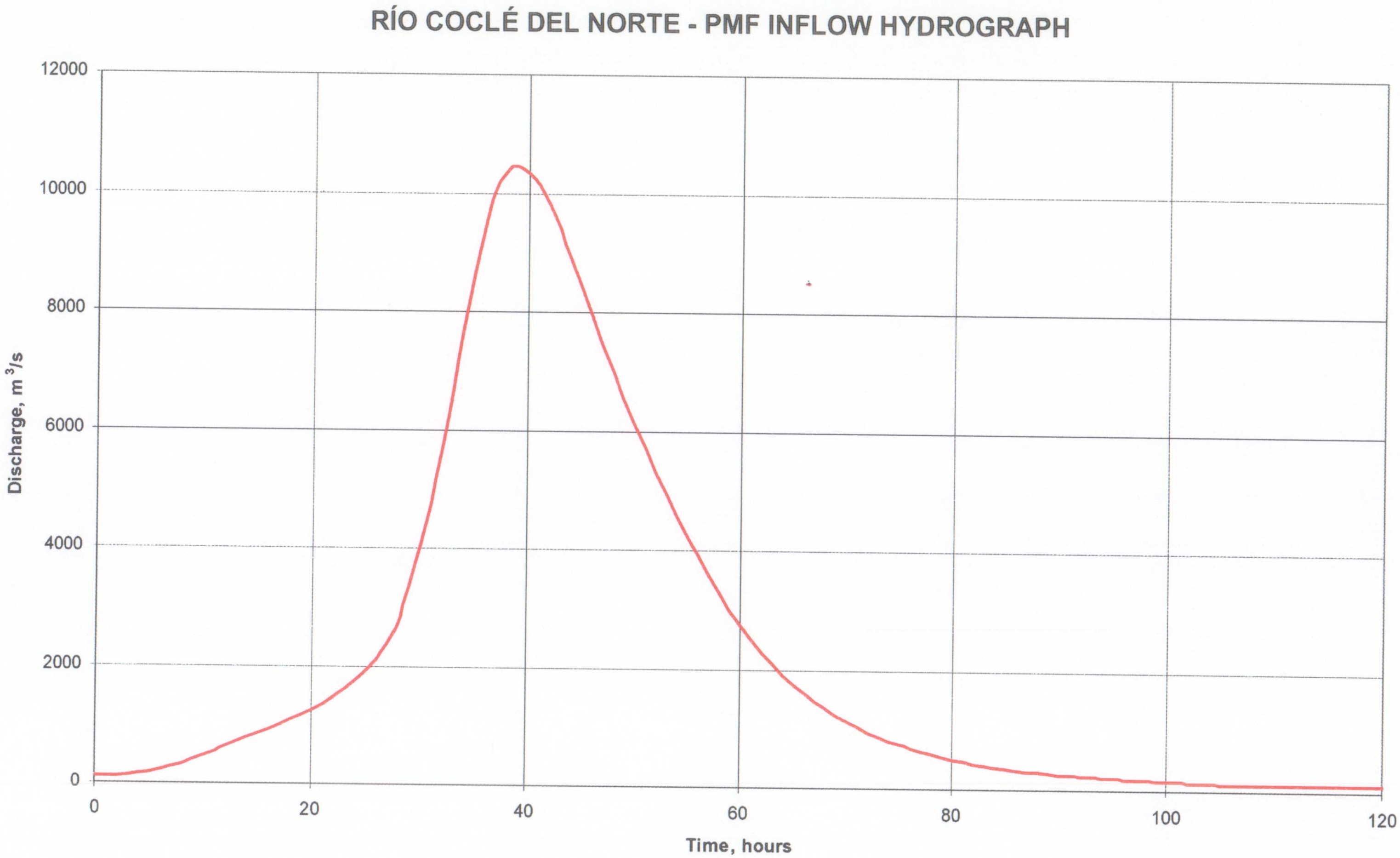
DEPTH-DURATION CURVE
RIO COCLÉ DEL NORTE BASIN

MWH	DATE: DECEMBER, 2003	EXHIBIT: 18



AUTORIDAD DEL CANAL DE PANAMA
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CONTRACT NO. CC-3-536
RÍOS COCLÉ DEL NORTE AND CAÑO SUCIO WATER SUPPLY PROJECTS
LOW COCLÉ DEL NORTE WATER SUPPLY PROJECT
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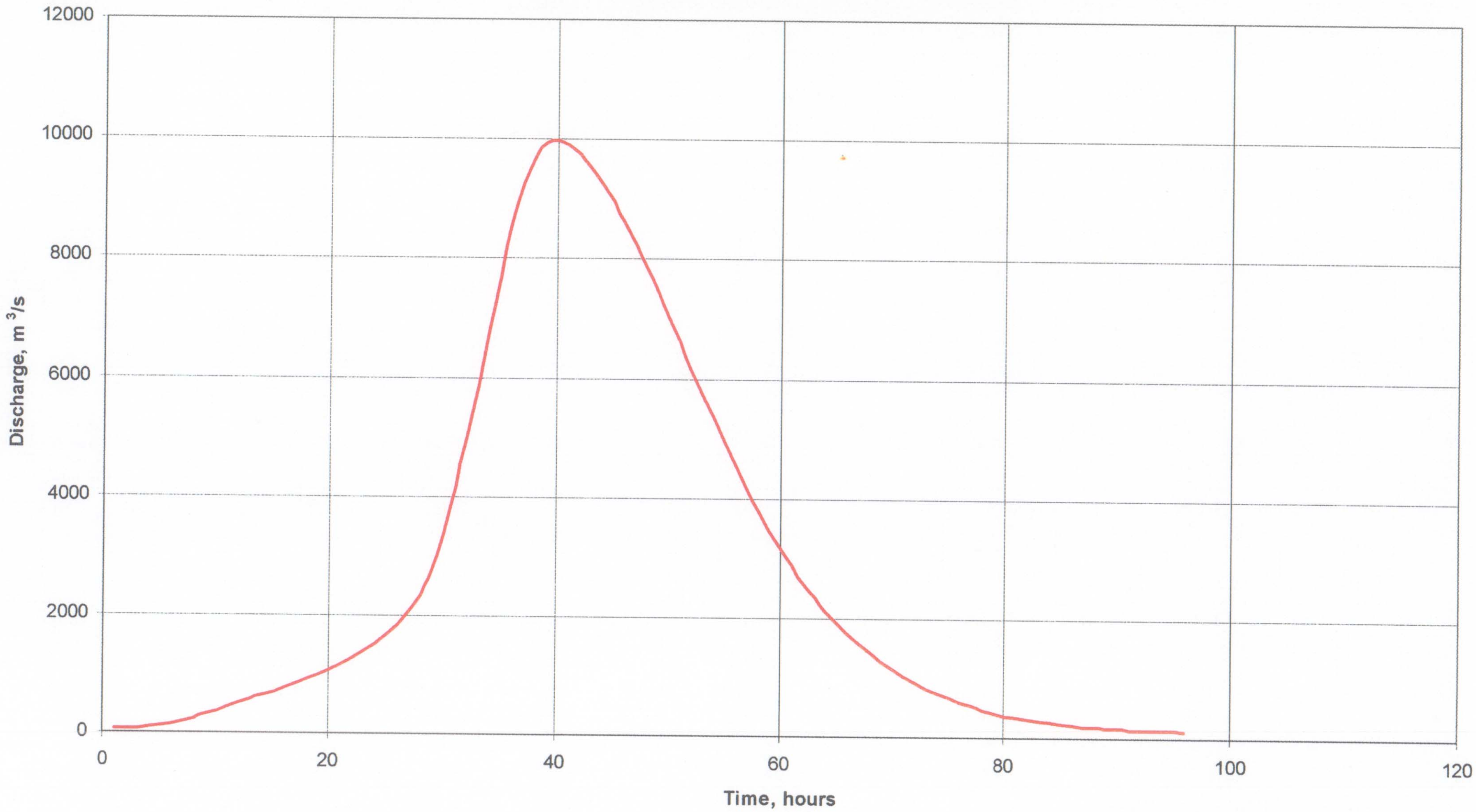
RÍO COCLÉ DEL NORTE
PMF INFLOW HYDROGRAPH

MWH

DATE: DECEMBER, 2003

EXHIBIT: 19

RÍO COCLÉ DEL NORTE - PMF INFLOW HYDROGRAPH
EXCLUDING TOABRÉ DAM DRAINAGE



AUTORIDAD DEL CANAL DE PANAMA
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RÍOS COCLÉ DEL NORTE AND CAÑO SUCIO WATER SUPPLY PROJECTS
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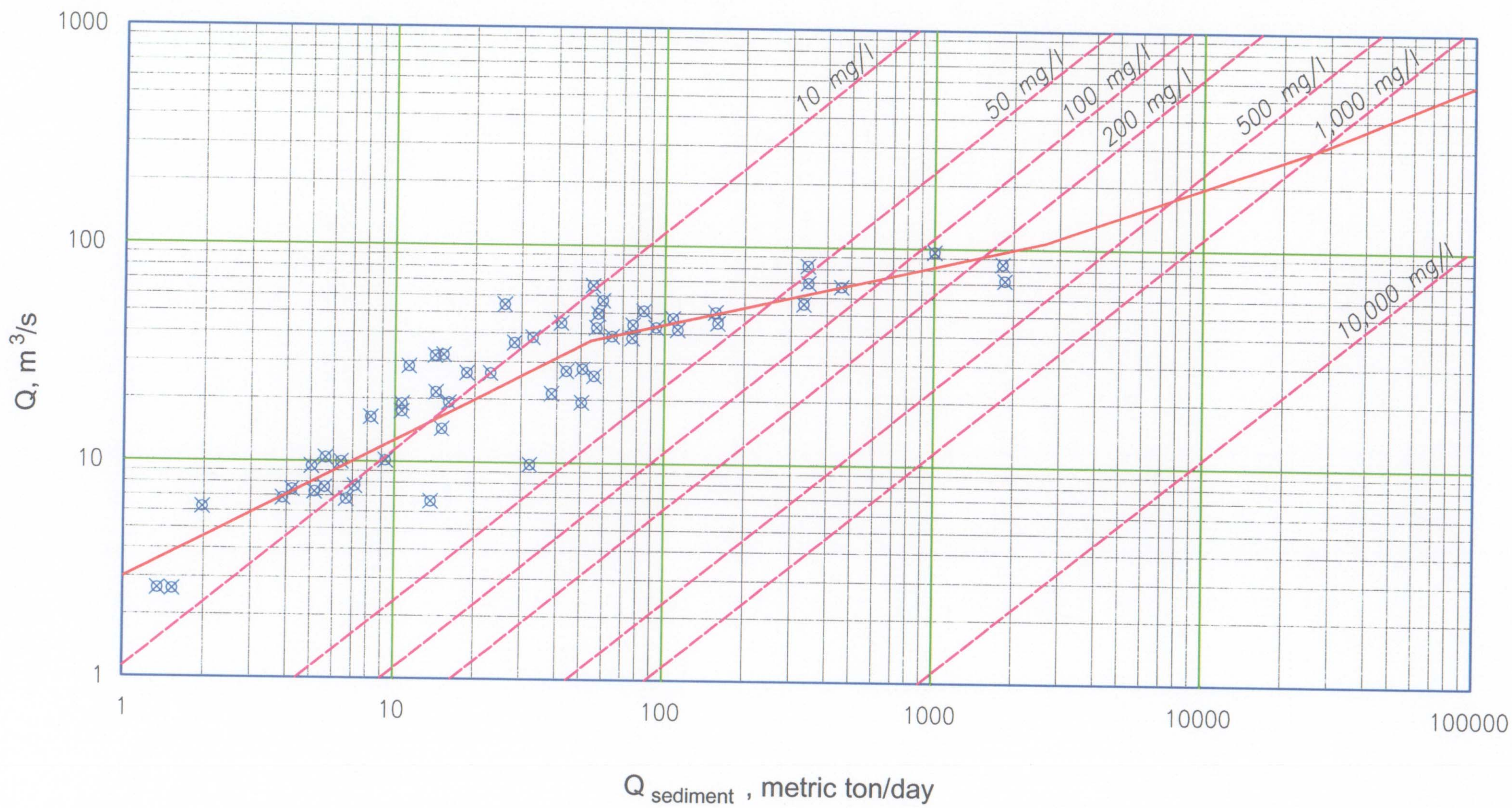
RÍO COCLÉ DEL NORTE
PMF INFLOW HYDROGRAPH
EXCLUDING TOABRÉ DAM DRAINAGE

MWH	DATE: DECEMBER, 2003	EXHIBIT: 20



AUTORIDAD DEL CANAL DE PANAMA

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CONTRACT NO. CC-3-536

RÍOS COCLÉ DEL NORTE AND CAÑO SUCIO WATER SUPPLY PROJECTS

LOW COCLÉ DEL NORTE WATER SUPPLY PROJECT

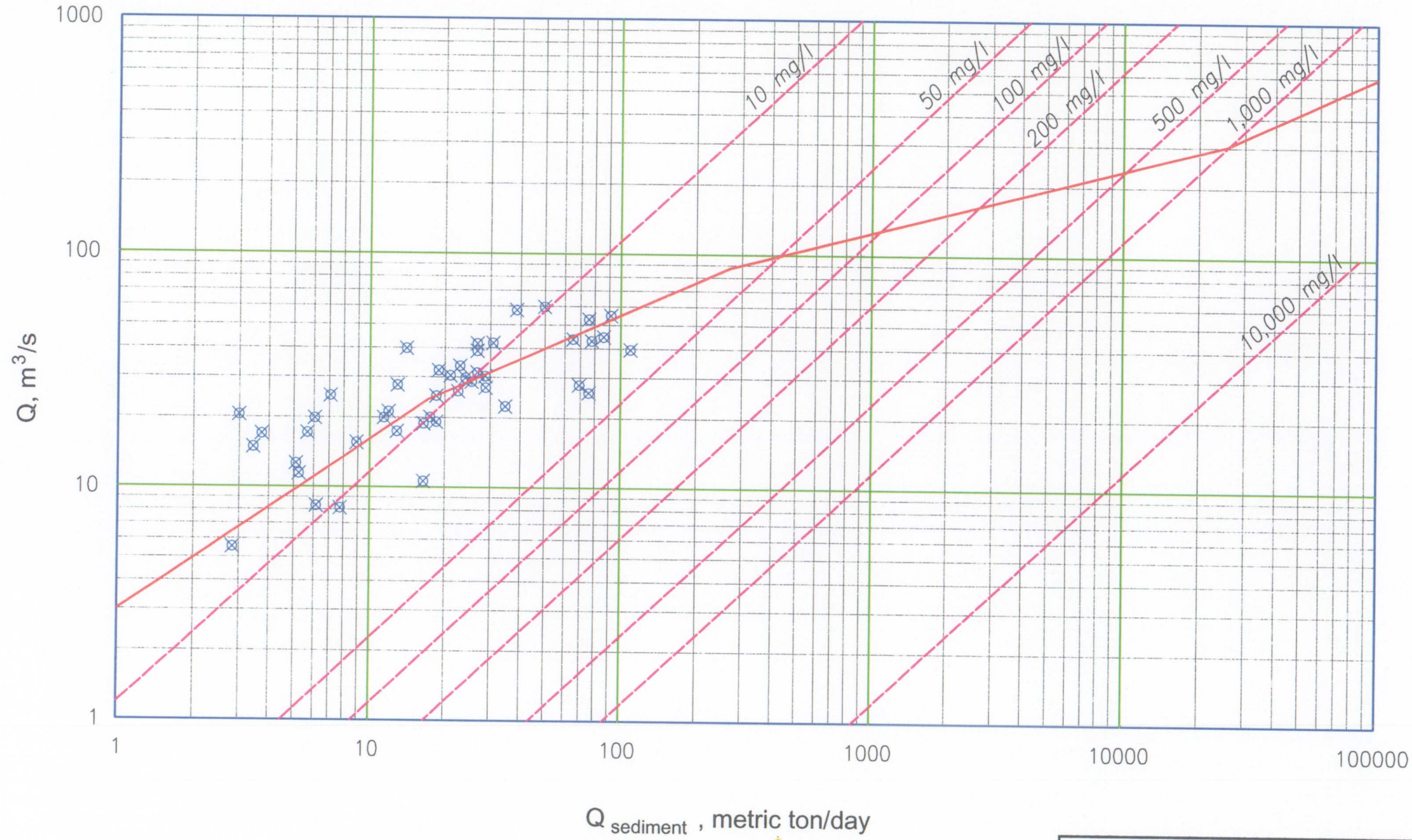
APPENDIX A - HYDROLOGY AND METEOROLOGY

SUSPENDED SEDIMENT RATING CURVE
RIO TOABRÉ

MWH

DATE: DECEMBER, 2003

EXHIBIT: 21



AUTORIDAD DEL CANAL DE PANAMA
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LOW COCLÉ DEL NORTE WATER SUPPLY PROJECT
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SUSPENDED SEDIMENT RATING CURVE
RIO COCLÉ DEL NORTE

MWH	DATE: DECEMBER, 2003	EXHIBIT: 22

ATTACHMENTS

Attachment 1 – Monthly Rainfall Data

MONTHLY RAINFALL DATA
BOCA DE TOABRE
(mm)

Year	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Annual
1966	293	66	147	286	383	465	435	253	236	403	950	723	4635
1967	326	136	163	454	722	425	315	510	221	450	363	513	4594
1968	147	206	483	147	383	381	408	396	329	259	348	434	3919
1969	185	191	98	265	268	176	237	457	365	296	424	609	3567
1970	491	245	166	668	970	407	400	496	297	371	914	817	6239
1971	433	270	430	115	595	530	360	499	260	316	299	236	4341
1972	680	264	197	600	419	294	503	231	480	364	359	367	4756
1973	273	131	61	185	538	257	299	366	234	382	920	504	4148
1974	431	242	212	215	417	296	414	451	343	515	639	321	4493
1975	338	109	109	73	777	260	362	419	387	468	622	598	4518
1976	285	227	55	284	206	269	280	454	328	320	345	211	3260
1977	113	117	130	317	245	233	437	651	322	447	340	251	3600
1978	190	123	264	650	568	323	280	295	364	249	532	231	4067
1979	77	202	102	518	429	495	382	431	327	417	433	548	4360
1980	369	245	78	303	274	366	429	365	307	354	450	628	4166
1981	379	372	174	889	526	338	526	484	211	394	862	955	6108
1982	355	213	158	303	252	238	536	526	362	325	368	257	3891
1983	221	59	89	236	720	254	330	444	395	219	485	574	4026
1984	393	649	128	90	526	428	292	403	315	345	373	303	4244
1985	350	118	193	109	319	383	357	281	180	412	348	476	3525
1986	517	150	274	707	485	397	451	636	389	467	713	237	5420
1987	184	130	73	868	295	241	364	345	298	361	528	485	4171
1988	193	233	105	150	602	345	282	452	409	348	453	428	3998
1989	246	225	217	171	516	344	473	606	327	395	588	361	4467
1990	403	137	250	357	506	242	306	553	397	636	486	567	4840
1991	189	206	316	242	550	433	434	360	701	400	584	661	5075
1992	154	215	66	633	657	283	439	574	400	417	375	305	4517
1993	314	129	200	411	281	271	327	267	462	353	816	710	4540
1994	200	168	201	382	526	475	383	451	244	396	511	373	4308
1995	213	102	186	297	304	353	371	308	264	309	422	872	4001
Mean	298	196	177	364	475	340	380	432	338	379	528	485	4393
Maximum	680	649	483	889	970	530	536	651	701	636	950	955	6239
Minium	77	59	55	73	206	176	237	231	180	219	299	211	3260

MONTHLY RAINFALL DATA
CHIGUIRI ARRIBA
(mm)

Year	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Annual
1966	123	34	32	196	489	507	614	412	519	430	545	516	4414
1967	96	48	25	193	298	623	417	430	536	554	475	308	4001
1968	13	105	102	85	350	489	454	255	399	540	418	241	3448
1969	85	41	23	84	310	370	351	364	503	290	506	196	3120
1970	215	85	137	198	551	271	619	353	493	475	343	577	4315
1971	145	160	264	72	492	520	600	668	412	485	453	94	4363
1972	193	49	34	348	287	377	290	261	370	370	185	87	2847
1973	30	38	14	79	390	635	380	469	400	526	689	176	3824
1974	116	32	30	48	264	285	342	334	432	613	365	54	2912
1975	40	20	22	25	336	436	290	591	343	491	807	384	3782
1976	82	35	12	81	318	247	172	382	348	426	189	66	2355
1977	28	29	19	78	408	394	356	409	344	504	211	102	2878
1978	68	61	215	203	486	349	317	422	293	458	624	68	3562
1979	16	32	15	289	311	303	324	442	390	338	263	405	3125
1980	119	51	12	52	415	362	373	482	315	517	319	171	3186
1981	131	99	82	303	557	328	462	510	459	407	536	462	4335
1982	113	18	25	270	372	363	321	361	207	426	306	69	2848
1983	41	10	15	148	744	566	278	509	670	402	242	350	3975
1984	78	229	32	2	302	382	405	424	497	487	238	66	3142
1985	69	22	18	14	142	487	194	412	353	488	353	203	2754
1986	30	4	15	158	124	525	345	386	318	427	393	59	2784
1987	56	48	6	159	168	237	215	241	556	479	145	96	2407
1988	31	64	13	113	409	421	434	396	430	485	354	226	3376
1989	72	57	40	24	424	328	428	574	314	374	447	176	3259
1990	120	93	55	37	398	261	354	395	468	467	308	241	3196
1991	31	35	125	58	382	311	326	464	412	442	208	144	2938
1992	35	33	11	191	379	423	293	324	482	349	306	153	2978
1993	125	31	126	101	164	517	350	359	413	435	716	244	3580
1994	32	23	26	182	636	520	612	433	605	685	378	172	4304
1995	60	15	87	218	750	700	488	724	368	606	693	414	5122
Mean	80	53	54	133	388	418	380	426	421	466	400	217	3438
Maximum	215	229	264	348	750	700	619	724	670	685	807	577	5122
Minimum	13	4	6	2	124	237	172	241	207	290	145	54	2355

MONTHLY RAINFALL DATA
COCLE DEL NORTE
(mm)

Year	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Annual
1966	214	22	36	298	576	615	696	563	201	427	740	459	4847
1967	322	205	46	839	284	435	519	693	483	443	1238	1116	6623
1968	126	56	98	160	215	384	1319	701	400	560	530	217	4766
1969	621	249	19	114	37	721	746	549	330	637	675	669	5367
1970	340	426	935	895	1098	642	791	536	463	511	1288	911	8836
1971	472	253	467	134	558	745	632	518	175	252	299	241	4746
1972	496	133	100	286	357	351	453	398	718	462	520	396	4670
1973	814	139	38	104	754	468	668	303	217	362	1065	777	5709
1974	423	131	144	171	583	674	937	404	100	370	563	273	4771
1975	320	146	190	78	762	620	656	353	109	410	440	811	4894
1976	283	160	39	226	512	325	535	421	578	485	598	456	4617
1977	86	114	65	232	328	450	804	501	193	563	558	318	4211
1978	114	103	393	808	576	304	567	372	276	182	603	213	4510
1979	90	204	77	252	198	267	420	400	217	274	290	475	3164
1980	200	226	60	104	223	391	390	287	305	380	556	625	3745
1981	226	234	135	775	372	395	519	322	173	473	973	880	5476
1982	526	171	191	298	334	465	836	615	286	486	488	260	4955
1983	176	49	50	245	563	646	744	464	736	616	506	1004	5799
1984	361	624	72	113	609	576	371	476	323	495	654	262	4935
1985	173	95	95	18	561	551	660	520	434	465	772	396	4741
1986	390	245	130	518	308	554	511	458	408	625	1101	227	5473
1987	127	20	52	422	438	331	771	517	370	820	517	336	4721
1988	91	88	59	162	427	248	306	324	244	413	560	544	3465
1989	150	142	291	214	589	558	518	503	219	596	519	256	4553
1990	307	28	66	146	552	380	567	528	393	504	535	491	4496
1991	157	90	68	226	731	460	672	340	528	484	717	402	4875
1992	184	239	60	335	646	284	862	499	436	602	539	316	5001
1993	153	44	259	391	331	549	638	434	425	398	1313	887	5822
1994	167	95	290	433	567	709	897	796	413	240	799	538	5944
1995	312	91	218	434	492	507	647	302	305	211	638	891	5049
Mean	281	161	158	314	486	487	655	470	349	458	686	522	5026
Maximum	814	624	935	895	1098	745	1319	796	736	820	1313	1116	8836
Minimum	86	20	19	18	37	248	306	287	100	182	290	213	3164

MONTHLY RAINFALL DATA
TAMBO
(mm)

Year	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Annual
1966	70	6	30	62	311	152	211	213	174	212	126	48	1615
1967	62	8	24	49	69	117	195	321	265	311	296	130	1847
1968	19	22	31	27	186	214	228	174	193	436	171	13	1714
1969	40	7	1	96	84	157	279	108	184	150	18	48	1172
1970	69	197	258	380	765	478	230	141	244	545	261	224	3792
1971	90	26	34	13	65	218	224	228	299	192	217	79	1682
1972	58	25	32	98	81	78	141	201	227	164	77	45	1227
1973	8	11	2	29	191	272	329	261	351	403	292	85	2233
1974	15	32	21	31	125	211	254	222	299	359	164	22	1755
1975	8	5	1	3	221	160	244	373	382	445	513	267	2621
1976	24	9	0	66	95	145	152	230	261	266	24	29	1302
1977	3	6	0	8	185	209	168	354	181	161	160	49	1484
1978	27	10	1	122	313	238	292	265	396	303	300	124	2392
1979	22	15	6	142	211	261	186	333	280	287	190	84	2017
1980	42	18	4	17	323	279	116	229	169	384	358	124	2063
1981	78	34	51	125	252	116	224	287	169	222	254	189	1999
1982	69	4	13	100	252	208	361	207	204	376	40	19	1850
1983	11	1	15	66	299	172	145	102	248	283	124	151	1616
1984	24	66	30	33	242	356	387	454	201	322	210	51	2376
1985	64	12	24	4	161	445	194	365	252	249	326	118	2213
1986	49	5	16	128	86	316	279	191	234	332	309	24	1969
1987	16	14	2	62	104	173	212	184	285	386	146	46	1628
1988	5	18	0	71	322	280	255	350	449	406	304	110	2571
1989	43	32	23	11	266	272	238	172	117	166	263	138	1740
1990	53	0	41	18	159	98	212	353	366	317	158	160	1935
1991	7	20	87	0	206	173	199	148	363	214	95	138	1649
1992	5	16	1	53	229	334	233	130	208	350	84	35	1677
1993	82	4	64	38	117	255	81	169	232	232	321	101	1695
1994	0	0	72	365	357	160	231	227	235	396	117	24	2182
1995	16	6	10	86	269	180	180	343	426	205	157	151	2028
Mean	36	21	30	77	218	224	223	244	263	302	202	94	1935
Maximum	90	197	258	380	765	478	387	454	449	545	513	267	3792
Minimum	0	0	0	0	65	78	81	102	117	150	18	13	1172

MONTHLY RAINFALL DATA
TOABRE
(mm)

Year	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Annual
1966	123	17	52	96	265	167	183	154	142	261	163	61	1684
1967	52	12	2	45	106	169	387	376	206	375	221	293	2244
1968	42	23	22	36	222	169	144	154	176	236	265	11	1500
1969	37	9	9	49	155	162	178	66	183	157	13	85	1103
1970	88	61	144	176	689	215	224	179	395	439	213	289	3112
1971	166	11	56	14	188	245	187	252	325	290	288	36	2060
1972	105	27	63	176	113	76	133	195	203	135	107	74	1408
1973	6	33	1	46	181	266	370	299	399	501	338	100	2540
1974	22	28	14	36	100	269	231	190	356	266	170	30	1711
1975	10	6	7	10	203	186	178	413	246	454	551	164	2426
1976	42	15	12	73	86	94	139	222	266	246	46	34	1273
1977	17	13	3	11	331	219	174	371	146	256	194	106	1839
1978	30	15	100	171	243	190	181	247	296	408	339	72	2292
1979	10	17	4	247	303	253	361	338	186	257	137	168	2281
1980	59	29	3	24	264	306	158	302	214	416	307	119	2201
1981	76	26	44	173	391	213	271	419	252	335	310	234	2744
1982	78	6	14	125	307	164	215	173	148	330	38	10	1607
1983	19	0	8	41	146	132	153	91	267	267	140	174	1439
1984	22	48	42	34	210	315	436	398	316	304	300	82	2506
1985	70	10	18	12	170	376	145	373	282	250	233	81	2021
1986	52	6	28	103	130	335	210	210	291	381	305	12	2062
1987	19	11	2	91	54	129	260	124	224	364	110	47	1435
1988	11	20	4	52	285	235	294	438	368	384	303	96	2490
1989	28	29	9	8	259	243	226	156	143	160	309	150	1719
1990	32	11	44	17	213	133	179	228	361	276	197	149	1840
1991	7	12	81	2	216	165	173	134	338	140	73	64	1407
1992	10	3	0	72	153	346	126	218	187	230	92	61	1497
1993	133	14	65	31	160	263	63	155	295	289	240	90	1799
1994	11	10	37	63	358	185	116	134	141	251	111	15	1431
1995	7	5	6	76	186	161	136	126	347	153	133	128	1465
Mean	46	18	30	70	223	213	208	238	257	294	208	101	1904
Maximum	166	61	144	247	689	376	436	438	399	501	551	293	3112
Minimum	6	0	0	2	54	76	63	66	141	135	13	10	1103

MONTHLY RAINFALL DATA
SAN LUCAS
(mm)

Year	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Annual
1966	485	38	130	138	353	460	272	331	273	495	607	508	4090
1967	307	154	48	240	268	384	533	544	189	361	850	629	4507
1968	252	244	184	139	274	286	463	368	372	396	448	132	3558
1969	566	173	101	147	197	259	215	226	196	488	525	944	4037
1970	233	199	231	389	451	316	289	276	363	621	796	851	5015
1971	302	205	200	315	458	386	414	495	190	329	344	297	3935
1972	1235	297	142	341	255	202	240	297	313	379	457	305	4463
1973	651	93	97	132	408	457	380	398	484	412	779	1077	5368
1974	188	59	239	250	439	425	549	576	265	401	845	408	4644
1975	510	172	139	65	854	466	450	448	208	453	590	758	5111
1976	311	217	45	376	398	255	429	564	347	354	476	413	4184
1977	122	161	96	387	295	467	610	615	392	585	501	274	4504
1978	139	179	286	756	610	372	350	367	387	235	632	299	4610
1979	143	206	199	380	397	505	346	556	192	439	517	724	4605
1980	394	264	78	213	292	487	449	396	211	365	529	853	4532
1981	340	383	201	1117	415	327	537	427	131	516	841	824	6058
1982	437	211	237	262	356	322	717	406	321	323	349	347	4288
1983	209	72	119	104	599	220	427	477	433	263	382	774	4079
1984	400	678	135	126	549	490	291	429	335	395	402	270	4500
1985	283	106	196	84	466	449	273	272	309	323	352	324	3434
1986	152	87	129	308	385	438	220	320	295	426	687	103	3550
1987	145	57	44	81	285	230	254	246	415	441	334	297	2829
1988	88	108	45	97	430	302	328	286	318	482	567	250	3301
1989	165	138	134	94	366	294	308	372	258	293	373	291	3084
1990	175	83	153	182	347	209	294	314	208	526	353	286	3129
1991	110	93	75	154	294	253	237	282	406	310	352	581	3146
1992	107	182	69	263	325	244	261	300	331	233	335	227	2876
1993	305	78	134	225	263	330	274	275	499	349	440	274	3446
1994	139	93	229	292	367	419	256	274	286	263	429	171	3218
1995	143	109	163	256	240	244	246	290	331	319	411	336	3088
Mean	301	171	143	264	388	350	364	381	309	393	517	461	4040
Maximum	1235	678	286	1117	854	505	717	615	499	621	850	1077	6058
Minimum	88	38	44	65	197	202	215	226	131	233	334	103	2829

MONTHLY RAINFALL DATA
SABANITA VERDE
(mm)

Year	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Annual
1966	496	70	192	211	311	325	241	282	247	473	447	298	3593
1967	187	103	37	407	303	336	352	466	214	285	482	698	3870
1968	125	181	181	95	248	160	252	265	272	339	439	88	2645
1969	289	101	61	146	261	262	255	207	176	344	351	612	3065
1970	169	164	225	322	345	432	294	234	320	520	509	776	4310
1971	316	140	155	214	421	325	332	465	273	356	324	254	3575
1972	873	187	75	332	263	177	241	248	330	312	325	171	3534
1973	455	103	43	116	354	291	307	311	348	366	417	671	3782
1974	110	34	176	407	416	344	319	302	236	320	520	166	3350
1975	185	115	95	38	550	393	269	409	231	336	416	441	3478
1976	394	305	35	194	306	244	264	371	342	287	461	506	3709
1977	121	198	112	190	279	369	287	452	303	595	443	287	3636
1978	97	110	172	358	465	383	411	358	379	235	370	219	3557
1979	44	127	111	303	337	558	263	442	174	377	330	448	3512
1980	211	120	12	144	397	447	342	293	294	429	351	548	3586
1981	336	183	148	650	364	296	387	452	152	361	458	620	4406
1982	262	97	87	245	359	245	504	329	278	392	289	176	3264
1983	209	36	22	184	597	240	275	320	319	211	394	398	3204
1984	229	339	101	49	370	268	228	471	274	291	402	171	3192
1985	219	70	135	52	222	413	293	160	271	479	374	220	2909
1986	248	48	144	452	335	318	251	421	267	361	489	145	3479
1987	105	198	65	399	244	194	250	292	260	175	371	288	2841
1988	88	108	45	97	430	302	328	286	318	482	567	250	3301
1989	165	138	134	94	366	294	308	372	258	293	373	291	3084
1990	175	83	153	182	347	209	294	314	208	526	353	286	3129
1991	110	93	75	154	294	253	237	282	406	310	352	581	3146
1992	107	182	69	263	325	244	261	300	331	233	335	227	2876
1993	305	78	134	225	263	330	274	275	499	349	440	274	3446
1994	139	93	229	292	367	419	256	274	286	320	478	171	3324
1995	143	109	163	256	240	244	246	290	331	319	411	336	3088
Mean	230	130	113	236	346	311	294	331	286	356	409	354	3396
Maximum	873	339	229	650	597	558	504	471	499	595	567	776	4406
Minimum	44	34	12	38	222	160	228	160	152	175	289	88	2645

MONTHLY RAINFALL DATA
COCLECITO
(mm)

Year	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Annual
1966	322	92	114	105	314	292	210	222	261	293	326	486	3037
1967	198	136	33	172	200	285	245	444	257	330	277	564	3141
1968	175	118	185	37	217	198	275	325	336	573	257	181	2877
1969	196	91	63	83	231	447	189	206	272	312	216	438	2744
1970	210	245	366	1003	413	379	320	299	343	528	393	696	5195
1971	420	295	279	50	248	444	209	485	233	201	243	257	3364
1972	356	178	152	288	221	153	145	206	285	146	221	261	2612
1973	336	110	71	107	263	248	272	216	304	399	459	523	3308
1974	129	135	176	240	350	378	371	281	192	314	399	235	3200
1975	255	107	128	57	518	349	215	283	190	343	366	559	3370
1976	272	141	18	188	351	233	161	343	315	300	245	255	2822
1977	128	173	110	248	316	255	237	330	253	329	247	189	2815
1978	147	165	174	412	413	216	235	364	290	372	311	308	3407
1979	130	170	184	157	279	263	189	416	142	219	191	266	2606
1980	197	188	44	131	259	166	205	292	325	404	387	406	3002
1981	346	253	143	653	378	296	233	308	194	332	456	503	4095
1982	193	147	106	261	266	334	439	240	167	293	224	194	2862
1983	176	36	61	166	527	163	152	371	177	353	224	344	2747
1984	301	395	87	78	396	280	190	437	296	303	370	202	3336
1985	262	98	111	62	265	411	205	198	250	286	239	232	2619
1986	262	98	111	64	265	411	205	198	250	286	239	232	2621
1987	122	47	35	573	223	201	311	166	391	625	247	238	3179
1988	105	160	50	74	323	262	207	454	347	379	235	275	2871
1989	208	69	120	137	400	397	425	370	243	153	325	262	3109
1990	254	101	248	150	354	185	302	446	227	436	326	371	3399
1991	93	134	252	144	380	295	246	327	380	341	310	495	3396
1992	143	127	37	399	374	263	211	272	269	217	238	275	2823
1993	253	123	188	163	308	244	192	221	437	354	360	559	3402
1994	122	71	178	388	381	392	191	307	197	529	291	245	3292
1995	131	102	152	216	209	217	221	320	240	207	256	332	2604
Mean	215	143	133	227	321	289	240	312	269	339	296	346	3129
Maximum	420	395	366	1003	527	447	439	485	437	625	459	696	5195
Minimum	93	36	18	37	200	153	145	166	142	146	191	181	2604

MONTHLY RAINFALL DATA
SANTA ANA
(mm)

Year	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Annual
1966	192	33	33	108	201	207	257	159	162	359	351	114	2176
1967	123	48	34	55	156	169	217	324	182	346	492	401	2547
1968	132	182	135	54	160	134	233	246	358	379	251	45	2309
1969	80	46	42	174	214	187	290	131	200	248	137	258	2007
1970	130	114	227	222	341	364	458	205	253	433	306	323	3376
1971	332	52	71	19	148	298	235	288	239	327	246	77	2332
1972	313	129	183	207	112	156	101	163	189	193	174	123	2043
1973	99	57	33	48	346	299	449	378	419	457	482	166	3233
1974	114	56	293	146	135	252	338	167	201	321	396	74	2493
1975	57	39	31	47	315	321	242	452	250	539	558	411	3262
1976	159	63	23	104	157	198	140	211	398	340	192	78	2063
1977	58	41	13	61	277	242	246	327	205	545	292	92	2399
1978	177	74	241	219	294	175	404	277	302	246	335	122	2866
1979	76	51	71	265	320	342	329	316	208	323	286	137	2724
1980	146	86	30	38	312	234	339	276	215	294	381	377	2728
1981	188	115	95	261	457	245	402	412	119	414	438	404	3549
1982	174	35	38	153	161	258	249	203	208	424	107	95	2103
1983	60	12	28	87	381	178	104	255	311	316	250	251	2233
1984	70	149	47	21	305	345	359	349	358	339	238	77	2655
1985	142	32	72	41	215	308	192	348	199	228	287	98	2162
1986	103	15	70	301	132	415	277	273	207	489	403	50	2733
1987	49	37	12	187	189	147	249	137	233	418	184	120	1961
1988	46	62	12	89	319	180	292	303	188	356	309	109	2265
1989	67	51	79	51	248	350	296	248	220	280	349	139	2376
1990	67	23	84	71	207	192	133	158	387	442	408	225	2397
1991	56	47	332	62	309	227	234	209	369	424	266	133	2668
1992	43	51	16	123	308	238	157	200	313	225	229	117	2021
1993	162	36	88	91	80	151	108	163	200	321	274	168	1843
1994	62	29	133	133	266	205	210	236	345	291	185	71	2165
1995	71	23	95	111	256	275	243	214	258	264	224	136	2170
Mean	118	60	89	118	244	243	259	254	257	353	301	166	2462
Maximum	332	182	332	301	457	415	458	452	419	545	558	411	3549
Minimum	43	12	12	19	80	134	101	131	119	193	107	45	1843

MONTHLY RAINFALL DATA
MIGUEL DE LA BORDA
(mm)

Year	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Annual
1966	358	25	49	79	519	381	482	604	289	667	697	345	4495
1967	372	76	70	207	426	319	390	487	418	342	801	739	4647
1968	89	254	160	79	271	175	601	427	403	464	500	97	3520
1969	122	137	16	111	239	183	421	348	206	397	422	768	3370
1970	188	112	216	533	474	261	335	363	426	465	860	1379	5612
1971	147	167	227	67	284	510	315	450	305	268	331	331	3402
1972	489	151	103	171	381	135	233	401	473	576	380	476	3969
1973	157	77	14	50	264	445	368	411	192	299	489	572	3338
1974	210	14	133	313	518	340	468	556	62	277	481	182	3554
1975	290	151	148	28	398	576	402	481	194	316	461	643	4090
1976	126	91	11	299	351	128	325	542	743	561	555	491	4223
1977	52	95	47	44	484	490	356	562	357	737	733	442	4400
1978	118	39	218	415	200	393	698	532	395	472	473	125	4077
1979	47	162	45	174	478	467	235	314	286	309	492	462	3471
1980	188	143	19	37	335	388	336	423	430	425	290	524	3536
1981	261	244	118	750	415	237	377	397	272	456	746	747	5019
1982	171	100	114	184	168	432	496	537	423	536	421	201	3782
1983	93	24	17	215	433	311	477	350	588	341	362	711	3921
1984	314	219	79	79	444	459	366	380	255	310	432	176	3513
1985	145	67	96	39	420	279	402	309	537	491	556	408	3749
1986	102	29	81	491	337	317	487	401	229	327	321	80	3202
1987	44	31	27	226	279	242	298	261	256	501	407	334	2906
1988	46	25	19	116	236	304	360	523	422	244	478	381	3154
1989	54	60	82	115	340	463	373	504	246	658	439	184	3519
1990	126	55	48	185	430	255	327	500	297	422	338	462	3444
1991	158	52	183	99	474	511	456	341	559	391	867	196	4285
1992	134	112	20	460	324	119	572	436	283	310	504	168	3442
1993	104	68	138	475	122	405	260	516	259	334	536	438	3653
1994	165	65	113	203	496	418	358	647	145	269	421	142	3441
1995	129	6	111	308	330	328	322	264	328	265	523	588	3503
Mean	167	95	91	218	362	342	397	442	343	414	510	426	3808
Maximum	489	254	227	750	519	576	698	647	743	737	867	1379	5612
Minimum	44	6	11	28	122	119	233	261	62	244	290	80	2906

MONTHLY RAINFALL DATA
BOCA DE URACILLO
(mm)

Year	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Annual
1966	168	34	169	432	432	338	215	269	207	513	492	203	3472
1967	226	47	80	282	397	306	477	502	175	488	482	530	3992
1968	49	108	160	92	207	180	311	343	346	457	383	100	2736
1969	159	103	21	76	336	262	227	218	223	316	225	245	2411
1970	137	207	289	326	493	567	357	237	391	431	453	523	4411
1971	138	74	187	34	276	399	192	310	347	329	281	133	2700
1972	513	93	57	325	303	217	172	193	321	294	276	231	2995
1973	227	86	12	56	314	281	375	319	272	364	414	395	3115
1974	83	62	53	51	456	373	316	170	196	354	536	120	2770
1975	65	35	60	27	418	247	241	451	364	567	610	616	3700
1976	101	74	32	126	310	243	142	198	431	365	303	166	2490
1977	59	64	25	77	318	309	229	337	166	477	337	222	2618
1978	196	110	185	388	404	222	374	287	306	380	363	97	3309
1979	36	134	31	216	416	433	471	533	327	323	258	219	3396
1980	195	125	19	68	438	371	391	246	170	291	329	310	2954
1981	222	122	169	396	352	386	407	378	190	330	520	466	3939
1982	163	41	56	139	183	216	347	366	244	514	197	74	2540
1983	58	23	10	105	413	313	251	248	395	411	314	320	2860
1984	144	126	73	63	608	358	263	431	323	406	358	111	3263
1985	201	44	79	37	306	435	315	311	286	411	349	248	3021
1986	105	21	54	517	257	297	196	249	288	462	405	77	2928
1987	74	61	14	234	319	253	353	287	338	473	307	187	2900
1988	38	87	20	126	325	250	313	495	390	472	446	154	3114
1989	55	103	40	77	232	258	236	438	193	475	391	168	2665
1990	160	22	115	116	561	235	235	305	566	587	282	408	3591
1991	60	60	140	106	295	225	180	131	440	355	288	268	2547
1992	70	56	22	200	396	328	301	303	254	315	327	164	2734
1993	109	59	122	323	212	332	181	173	343	432	397	282	2964
1994	78	39	161	148	526	389	212	186	394	366	368	128	2995
1995	151	28	79	257	319	509	345	257	320	312	344	260	3181
Mean	135	75	84	181	361	318	287	306	307	409	368	247	3077
Maximum	513	207	289	517	608	567	477	533	566	587	610	616	4411
Minimum	36	21	10	27	183	180	142	131	166	291	197	74	2411

MONTHLY RAINFALL DATA
ICACAL
(mm)

Year	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Annual
1966	75	7	70	225	665	286	461	652	290	615	1081	468	4896
1967	167	35	71	160	338	527	497	671	257	272	620	304	3919
1968	10	205	115	48	409	266	580	510	308	668	515	83	3717
1969	133	107	36	156	516	222	493	327	313	421	384	612	3720
1970	284	51	134	509	572	278	380	252	311	423	1005	757	4956
1971	156	112	146	9	457	602	587	367	227	315	398	167	3543
1972	408	73	112	627	454	196	341	393	325	673	273	236	4112
1973	46	29	20	104	424	412	376	466	292	343	516	417	3445
1974	63	67	72	40	461	348	699	484	117	284	955	130	3719
1975	43	37	137	34	418	770	669	342	353	438	451	668	4359
1976	64	60	9	155	355	290	343	393	502	502	329	207	3210
1977	23	31	8	40	339	426	313	568	416	728	601	480	3972
1978	102	72	147	369	250	556	464	375	269	401	473	109	3586
1979	22	137	7	214	565	512	364	358	354	359	550	399	3839
1980	206	79	18	14	431	144	374	323	457	548	283	449	3324
1981	158	96	81	677	553	334	534	389	352	416	926	656	5171
1982	271	52	220	114	106	291	565	373	418	490	560	167	3627
1983	63	30	74	240	551	299	410	473	346	199	237	547	3468
1984	115	125	78	71	423	386	405	339	229	346	547	142	3206
1985	183	56	52	25	439	393	444	171	367	407	500	353	3389
1986	115	40	89	413	403	478	783	546	455	575	505	185	4587
1987	73	47	33	522	608	324	510	373	572	840	366	423	4689
1988	48	67	32	77	411	295	446	356	218	373	590	314	3227
1989	28	55	70	18	207	425	288	740	481	677	484	220	3693
1990	79	10	59	170	412	246	432	425	496	587	433	467	3816
1991	119	60	126	190	816	442	395	309	435	552	755	196	4394
1992	49	61	82	195	249	374	454	472	366	343	524	278	3447
1993	94	32	203	278	290	30	86	226	236	354	582	373	2785
1994	72	46	62	78	322	565	439	364	405	288	597	149	3386
1995	178	34	93	93	498	528	490	137	152	227	707	645	3781
Mean	115	64	82	195	431	375	454	406	344	455	558	353	3833
Maximum	408	205	220	677	816	770	783	740	572	840	1081	757	5171
Minimum	10	7	7	9	106	30	86	137	117	199	237	83	2785

MONTHLY RAINFALL DATA
CIRI GRANDE
(mm)

Year	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Annual
1966	79	10	30	65	201	168	299	339	297	184	292	51	2015
1967	46	30	66	171	185	378	189	747	428	372	241	100	2953
1968	40	41	10	30	260	120	195	244	348	321	286	48	1943
1969	129	32	0	36	237	244	101	400	233	367	317	124	2220
1970	357	93	137	350	305	302	210	435	240	302	270	148	3149
1971	39	25	18	83	295	350	178	140	143	286	286	97	1940
1972	149	52	35	180	270	298	132	205	350	250	171	39	2131
1973	74	16	0	89	192	190	228	125	217	555	198	102	1986
1974	0	0	2	56	240	136	238	88	147	399	279	178	1763
1975	38	46	16	31	267	250	241	494	228	813	290	173	2887
1976	20	62	29	164	342	334	134	33	270	314	154	67	1923
1977	220	62	20	59	219	124	352	342	161	388	272	228	2447
1978	35	0	84	300	300	145	368	391	345	282	277	36	2562
1979	20	41	5	274	231	239	325	269	251	165	206	193	2220
1980	165	64	10	51	318	351	315	310	119	358	277	224	2560
1981	160	69	163	312	323	358	302	251	198	533	361	249	3279
1982	160	30	20	84	191	259	145	196	206	409	147	36	1882
1983	20	8	5	46	231	180	249	224	351	231	338	262	2144
1984	81	137	20	56	231	279	145	422	401	452	221	71	2517
1985	127	20	25	15	173	267	203	259	300	170	193	124	1877
1986	28	8	10	196	231	86	155	36	277	213	173	23	1435
1987	13	13	5	102	257	81	127	188	671	668	343	345	2812
1988	10	30	3	43	310	277	226	386	368	310	264	178	2405
1989	15	66	51	36	175	203	310	356	330	376	259	208	2385
1990	130	8	58	79	318	198	191	208	422	371	264	229	2474
1991	20	30	86	56	343	168	208	122	297	224	282	147	1984
1992	48	23	10	155	409	244	163	305	231	244	358	140	2329
1993	76	20	89	201	384	343	114	130	401	378	378	155	2670
1994	15	3	5	0	150	335	69	206	180	376	312	41	1692
1995	109	15	23	137	315	394	221	373	290	269	290	267	2703
Mean	81	35	35	115	263	243	211	274	290	353	267	143	2310
Maximum	357	137	163	350	409	394	368	747	671	813	378	345	3279
Minimum	0	0	0	0	150	81	69	33	119	165	147	23	1435

MONTHLY RAINFALL DATA
TRINIDAD
(mm)

Year	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Annual
1966	47	1	22	62	271	271	201	328	279	135	300	309	2226
1967	37	10	54	112	214	259	185	316	270	385	269	49	2160
1968	5	129	66	13	389	468	80	288	386	399	268	39	2530
1969	63	36	4	174	229	245	132	307	173	421	383	176	2343
1970	155	53	108	115	325	217	163	306	330	269	398	430	2869
1971	108	55	68	5	449	323	153	323	196	264	310	41	2295
1972	160	51	28	257	211	305	102	173	295	267	198	51	2096
1973	23	3	0	69	229	284	124	132	269	480	495	206	2314
1974	8	25	18	30	122	165	246	183	183	457	394	43	1875
1975	8	30	25	18	315	320	201	318	312	681	472	373	3073
1976	33	23	0	119	213	119	89	102	315	358	124	61	1557
1977	18	10	3	23	251	109	165	272	211	409	305	91	1867
1978	84	13	79	302	356	356	246	272	198	373	173	25	2477
1979	13	0	3	132	203	305	335	206	259	216	155	102	1928
1980	86	48	3	43	376	244	389	226	198	310	429	178	2530
1981	170	30	38	307	373	262	262	325	145	211	373	246	2743
1982	147	13	20	69	267	191	132	264	218	384	140	15	1859
1983	28	10	18	28	218	137	165	259	277	246	249	152	1788
1984	91	97	20	25	274	30	122	417	381	318	399	43	2217
1985	74	23	18	38	163	325	191	163	221	107	180	203	1704
1986	58	10	10	211	269	180	117	241	191	366	386	61	2101
1987	33	20	3	91	330	155	226	234	538	231	64	99	2024
1988	5	20	5	53	259	290	272	234	279	213	295	160	2085
1989	33	36	13	28	173	206	218	257	239	236	318	165	1920
1990	15	3	33	58	483	175	221	254	404	414	335	84	2479
1991	33	8	208	5	381	140	130	132	333	216	287	117	1989
1992	0	10	3	163	274	249	178	231	429	183	236	122	2078
1993	38	18	79	183	231	284	124	130	399	358	427	142	2413
1994	56	8	71	38	196	180	89	157	183	404	409	3	1793
1995	102	3	10	109	272	234	221	320	239	300	249	249	2306
Mean	58	26	34	96	277	234	183	246	278	320	301	135	2188
Maximum	170	129	208	307	483	468	389	417	538	681	495	430	3073
Minimum	0	0	0	5	122	30	80	102	145	107	64	3	1557

Attachment 2 – Review of Previous Hydrologic Reports and Analyses and List of Hydrologic Data Obtained from PCC

ATTACHMENT 2

INVENTORY OF HYDROLOGIC DATA OBTAINED FROM THE ACP AND REVIEW OF PREVIOUS HYDROLOGIC REPORTS AND ANALYSES

Introduction

This report presents an inventory of hydrologic data and previous reports on hydrologic analysis obtained through the ACP. The previous reports were reviewed and the analyses presented in the reports are summarized.

Basic Data Inventory

Streamflows

Streamflow data, historic and extended using statistical techniques, was obtained from the ACP. The period of record included a few months of missing data for each station. Some of the months with asterisk marks were estimated from general monthly trend in the data or from staff gage readings. A list of stream gaging stations operated by Instituto de Recursos Hidraulicos y Electrificacion (IRHE) was also provided by the ACP. The list showed station numbers, river names, locations of gages, names of provinces, type of stations, drainage areas and elevations at the gages, latitudes and longitudes, and dates of installation and suspension. The data supplied by the ACP are summarized below.

- Summary of Discharge Measurements (from IRHE)
  Río Coclé Del Norte at El Torno:
  272 measurements from April 1958 to June 1986
  Río Toabre at Batatilla:
  267 measurements from September 1968 to February 1995
  Río Coclé Del Norte at Canoas:
  124 measurements from November 1983 to February 1995

- Measured Daily Flow Data (from IRHE)
  Río Toabre at Batatilla, July 1958 to April 1999
  Río Coclé Del Norte at Canoas, October 1983 to July 1999
  Río Indio at Boca de Uracillo, August 1979 to May 1998

- Measured Monthly Flow Data (from IRHE)
  Río Toabre at Batatilla, July 1958 to April 1999
  Río Coclé Del Norte at Canoas, October 1983 to July 1999
  Río Indio at Boca de Uracillo, August 1979 to May 1998
  Río Trinidad at El Chorro, January 1948 to December 1998
  Río Coclé Del Norte at El Torno, July 1958 to June 1986

Río Ciri Grande at Los Canones, January 1948 to May 1959, August 1978 to December 1998
Río Toabre at Batatilla, July 1958 to September 1964, June 1969 to April 1998

- Filled-in and Extended Monthly Flows (the ACP), January 1948 to December 1998
  Río Coclé Del Norte at Dam site plus Río Caño Sucio
  Río Caño Sucio
  Río Coclé Del Norte at Dam site
  Río Coclé Del Norte at El Torno
  Río Toabre at Batatilla
  Río Coclé Del Norte at Canoas
  Río Indio at Boca de Uracillo
  Río Ciri Grande at Los Canones
  Río Trinidad at El Chorro
  Río Indio at Dam Site

- Miscellaneous Data
  1. Exhibits showing double mass curves for Río Trinidad at ElChorro versus Río Ciri Grande at Los Canones, Río Toabre at Batatilla versus Río Indio at Boca de Uracillo, Río Coclé Del Norte at Canoas versus Río Coclé Del Norte at El Torno, Río Indio at Boca de Uracillo versus Río Ciri Grande at Los Canones, and Coclé Del Norte at El Torno versus Río Toabre at Batatilla.

  2. Exhibits showing correlation between Río Indio at Boca de Uracillo and Río Ciri Grande at Los Canones, Río Coclé Del Norte at Canoas and Río Coclé Del Norte at El Torno, and Río Coclé Del Norte and Río Toabre at Batatilla.

  3. Río Indio at Limon: monthly maximum and minimum daily observed discharges, May 1958 to October 1980.

  4. Río Indio at Boca de Uracillo: monthly minimum observed discharges, August 1979 to April 1990, and monthly maximum instantaneous discharge July 1979 to April 1990.

  5. Map showing hydrologic units in Panama

  6. Maps showing locations of stream gaging and meteorological stations in the Río Indio and Río Coclé Del Norte basins.

Rainfall

Historic rainfall data were obtained for the following stations from the ACP. A list of meteorological stations, showing station numbers, names, provinces, latitudes and longitudes, elevations, type of stations, and date of installation, was also provided by the ACP. The data with period of record are listed below. The indicated period includes a few months of missing data for some stations.

Monthly Rainfall

111001 Boca de Uracillo	September 1974 – September 1998
109001 Miguel de la Borda	February 1975 – October 1998
105010 Santa Ana	November 1980 – August 1998
105009 Coclecito	January 1980 – February 1998
105008 Sabanita Verde	January 1979 – August 1998
105007 San Lucas	February 1974 – April 1998
105005 Toabre	June 1970 – September 1998
105004 Tambo	February 1970 – October 1998
105003 Coclé del Norte	May 1969 – August 1998
105002 Chiguiri Arriba	July 1958 – October 1998
105001 Boca de Toabre	May 1958 – August 1998

Daily Rainfall

105001 Boca de Toabre	May 1958 – July 1999
105002 Chiguiri Arriba	July 1958 – July 1999
105003 Coclé Del Norte	May 1969 – July 1999
105005 Toabre	June 1970 – January 1999
105007 San Lucas	January 1974 – July 1999
105008 Sabanita Verde	January 1979 – June 1999
105009 Coclecito	January 1980 – December 1998
105010 Santa Ana (Obre)	November 1980 – June 1999
109001 Miguel de la Borda	February 1975 – January 1999
111001 Boca de Uracillo	September 1974 – June 1999

Daily Rainfall for all station in Canal Zone for the following storms (from the ACP)

December 13, 1981
December 4 to 7, 1985
May 8 to 9, 1987
January 7 to 8, 1996
January 13 to 15, 1996
November 27 to 29, 1996

Hourly Rainfall Data for the above storms for all stations in the Canal Zone from the ACP.

Meteorological Data

These data included: wind speed – average, maximum and minimum; average wind direction; air temperatures – average, maximum and minimum; dew points – average, maximum and minimum; barometric pressure – average, maximum and minimum; and total solar radiation.

Station No	Station ID	Station Name
55	ALH	Alhajuela
50	ACL	Agua Clara
60	BHT	Balboa Heights
04	BCI	Barro Colorado
59	CNO	Cano
53	CHI	Chico
51	CDL	Candelaria
52	CNT	Ciento
48	CHR	Chorro
21	CAN	Canones
30	CAS	Cascadas
65	CSO	Coco Solo
06	DHT	Diablo Heights
14	ESC	Escandalosa
64	EMH	Empire
63	FAA	FAA (Balboa)
54	GAT	Gatun
46	GUA	Guacha
16	GAM	Gamboa
09	GTW	Gatun West
43	HUM	Humedad
41	HHI	Hodges Hill
70	LMB	Limon Bax
42	MLR	Monte Lirio
58	MIR	Miraflores
45	PEL	Peluca
61	PMG	Pedro Miguel
44	RAI	Raices
66	RPD	Río Piedras
47	SAL	Salamanca
49	SMG	San Miguel
08	SRO	Santa Rosa

Floods

Monthly maximum instantaneous flood peaks for the Río Indio at Boca de Uracillo, Río Toabre at Batatilla and Río Coclé del Norte at Canoas for the available period of record.

Sediment

Suspended sediment sampling results including: date of sampling, water discharge in cubic meters per second, sediment concentration in milligram per liter, suspended sediment load in tons per day and water temperature. The data was made available by IRHE for the following sites.

- Río Toabre at Batatilla, 56 samples taken during the period from March 1982 through August 1998.

- Río Coclé del Norte at Canoas, 46 samples taken during the period from November 1983 through August 1998.

- Suspended sediment loads on monthly basis were obtained from the ACP for three rivers (Río Chagres, Río Pequeni and Río Boqueron) entering Madden Lake and three rivers (Río Gatun, Río Trinidad and Río Ciri Grande) entering Gatun Lake. the ACP also provided two reports on sedimentation survey of Lake Madden.

Water Quality

The water quality parameters included: physical quality (conductivity, temperature, turbidity, dissolved solids, total solids), inorganic metals (Ca, Mg, total hardness as CaCo3, Fe, Mn, K, Na), inorganic non-metal (pH, alkalinity, OH, CO3, HCO3, NH3, B, Cl, F, PO4, NO3, NO2, SiO2, SO4) and organic matter. The data for the following stations were obtained.

105000102 Coclé del Norte at Canoas
One sample in each 1991, 1992 and 1993, and four samples in 1994.

115000802 Ciri Grande at Los Canones
Three samples in 1990, four in 1991, three in 1992, and three in 1993.

105000201 Toabre at Batatilla
One sample in 1990, one in 1991, two in 1992 one in 1993 and four in 1994.

Previous Reports

Atlas Nacional de la Republica de Panama by Instituto Geografico Nacional "Tommy Guardia," 1988.

The Atlas has a number of maps showing topographical, hydrological, meteorological, soil, land use, etc., features of Panama. The following information was obtained from the Atlas.

About 10 percent of the Río Indio Basin (mostly in the head reach) is covered with forest and the land is subjected to inundation. The remaining downstream area is covered with tropical forest with perennial foliage.

Mean monthly temperature varies within about 2° C through the year. Mean annual temperature varies from about 26° C at the dam site to about 24° C in the head reach. At lower elevations, the lowest temperatures occur in the months of September-October where as the months of March-April have highest temperatures. High temperatures occur during the month of June at relatively higher locations (above about 2,300 meters) in the basin. Generally, temperatures during cold months are less than about 18° C.

The Río Indio basin is humid, with a tropical climate. Mean annual precipitation is higher near the coastal area of the Río Indio (about 4,000 mm) and decreases inland (about 3,000 mm). A few months could be significantly dry. There is a slight increase in precipitation near the watershed divide.

"Development of Probable Maximum Flood (PMF) and Review of Flood Routing Procedures, Phase III and Phase IV Studies," by U.S. Army District, Mobile, Alabama, February 1979.

This report presents methodology and results of studies of the probable maximum floods for Madden and Gatun dams, PMF routings, flood control operation, canal surges, wave run-up, and wind setup.

The PMF for each dam is based on the probable maximum precipitation (PMP). The PMP, and its aerial and temporal distributions were derived using the February 1978 study by F.K. Schwarz and J.T. Riedel entitled, "Probable Maximum Precipitation Estimates for Drainages above Gatun and Madden Dams."

The drainage area above Gatun Dam was divided into 12 sub-basins, mostly small streams directly entering Gatun Reservoir. A detailed study was made to derive unit hydrographs using Snyder's method. The following equations were developed:

$$t_p = 0.21\ (AL)^{0.38}$$

$$q_p = 450/t_p$$

where

L = length of main stream, mi
t_p = time to peak, hours
q_p = rate of runoff cubic feet per second per square mile (cfs/sq mi)
A = drainage area in square miles

For the two major tributaries, Río Ciri Grande and Río Trinidad, the unit hydrographs were taken from a 1968 study entitled, "Climatology and Hydrology of the Panama Canal Watershed, 1968, IOCS Memorandum Jax – 50 prepared by Jacksonville District, Corps of Engineers." The equations used were:

$$t_p = 0.21\ (LLc)^{0.50}$$

$$q_p = 2200\ (1/AL)^{0.38}$$

The parameters are as defined above except that Lc is the main stream length from the outlet of the basin to a point opposite to the centroid of the area.

The estimated times to peak for all sub-basins were reduced by 20 percent to obtain conservatively high unit hydrograph peaks. A uniform infiltration rate of 0.05 inches per hour was used for the duration of the PMP. Channel routing was performed using Muskingum method. The coefficient K was assumed equal to travel time through the reach and the coefficient X was set equal to 0.2. The report does not clearly show the estimated PMF peak inflows for Madden and Gatun reservoirs.

"Análisis Regional De Crecidas Maximas," by Instituto de Recursos Hidraulicos y Electrificacion, Departamento de Hidrometeorologia, Sección de Hidrología, June 1986.

This study presents basic data, methodology and results of a regional flood frequency analysis for the river basins in the Republic of Panama west of about 79° west longitude. The area was divided into seven zones based on the characteristics of maximum floods observed in the various river basins. The analysis included:

1. Selection of common period for sets of groups of stations: Missing peaks in each common period were estimated using either drainage area ratio raised to an exponent, exponential correlation or adjustment factor. The

parameters for each method were developed from the concurrent instantaneous flood peaks.

2. Estimation of mean annual flood: Mean annual flood was considered to be the mean of selected flood peak series (including estimated flood peaks, that is, for the selected common period of record). Computations were not made to derive mean annual flood with a return period of 2.3 years. However, it was mentioned in the report that the mean of the annual peaks was assumed equivalent to the flood of 2.3-year return period.

3. Development of relationship between mean annual floods and drainage areas: Exponential relationships were developed as given below.

$Q_{mean\ annual} = 34\ A^{0.58}$ equation 1, for zones I and II
$Q_{mean\ annual} = 27\ A^{0.58}$ equation 2, for zones III and IV
$Q_{mean\ annual} = 13\ A^{0.58}$ equation 3, for zones V and VI
$Q_{mean\ annual} = 10\ A^{0.58}$ equation 4, for zone VII

4. Development of dimensionless flood frequency curves: Dimensionless ratios "maximum instantaneous peak divided by mean annual flood" were computed for all stations. These ratios were plotted on a probability paper using the Weibull plotting position formula. The stations with similar flood characteristics, were grouped. The best fit through the plotted points was achieved by eye-ball fitting. A set of four curves was developed. The ratios corresponding to select return periods for each curve are given below.

Return Period	**Ratios (Qmax/Qmean annual)**			
	A	**B**	**C**	**D**
2	0.92	0.93	0.95	0.93
5	1.38	1.35	1.32	1.20
10	1.68	1.62	1.57	1.45
20	2.00	1.90	1.57	1.65
25	2.10	2.00	1.80	1.75
50	2.40	2.25	1.90	1.95
100	2.75	2.55	2.15	2.10
1,000	3.95	3.55	3.25	2.75
10,000	5.3	4.60	4.10	3.40

5. The following table was recommended for estimating flood frequency data.

Zone	Applicable Equation	Ratio (Qmax/Qmean annual)
I	1	A
II	1	C
III	2	A
IV	2	D
V	3	B
VI	3	A
VII	4	C

The drainage area of the Río Indio at Boca de Uracillo is about 365 km^2 and is located in zone III. The report showed the following flood frequency data for this station.

Return Period (years)	Flood Peak (m^3/s)
2	761
5	1,141
10	1,389
20	1,654
25	1,737
50	1,985
100	2,274
1,000	3,267
10,000	4,383

"Probable Maximum Precipitation over Eastern Panama and Northwest Colombia," prepared by Hydrometeorological Branch, Office of Hydrology, Weather Bureau, and September, 1965.

This study presented a good description of the meteorology of the major storms in the canal zone area. A 10-mi^2, 24-hour PMP map was prepared for the canal zone and northwest Colombia. The starting point for the estimation of the PMP was five storms: October 21-24, 1923; November 6-9, 1931; November 26-29, 1932; November 2-4, 1935; and December 2-4, 1937. A range of estimates was made which involved:

1. Moisture maximization of maximum 24-hour rainfall.

2. Adaptation of 1-hour rainfall amount.

3. Adjustment of the value from HMR 4 to 10-mi^2 PMP (U.S Weather Bureau, Possible Precipitation over the Panama Canal Basin, Hydrometeorological Report No. 4, 1943)

4. Adjustment of the canal zone stations 100-year values to the PMP by appropriate ratios from other "similar climatic region where comprehensive PMP studies have been made.

Conclusions drawn from these four approaches as to the magnitude of 24-hour PMP in the canal zone indicated that a value of 28 inches applies to the sea level Atlantic side. Extracts of the reasoning presented in the report are presented below.

"Moisture maximization of the largest storm rainfall in Panama is less meaningful in estimating the PMP because the variation in precipitation intensity from storm to storm depends mostly on the variation in the mechanism which lifts the moist air in cloud masses and less on the availability of the moisture. However, the factor was computed. Based on observed dew points and sea surface temperature (U.S. Navy Hydrographic Office, "World Atlas of Sea Surface Temperatures, H.O. No. 225, 1944), an estimated upper limit to the 12-hour dew point for Panama in November or December was 77° F. The maximum 12-hour persisting dew point on the Gulf of Mexico Coast of the United States ranges about 3° F to 4° F below sea temperatures within a few hundred miles." "Based on the seasonal variation of sea surface temperature and a dew point 3° F below the sea surface temperature, the seasonal variation of the maximum 12-hour persisting dew point was estimated as:

November – February	77° F
March	77.5° F
April – August	78° F
September – October	79° F

The 12-hour dew point for the December 14-15, 1944 storm was estimated to be 72° F based on prevailing dew points on the Northern Hemisphere surface maps (U.S. Weather Bureau, "Daily Series Synoptic Weather Maps"). The December 1944 observed 24-hour rainfall could occur as early as October. Hence, the sea level value of 13.5 inches was adjusted a the dew point of 79° F, resulting in a value of 19.0 inches (factor 19/13.5 = 1.407)."

"Maximum 1-hour observed rainfall of 7.54 inches was adopted to be that of October 7, 1957 at Moran on the Pacific side of Panama during a local storm typical of the summer season in its aerial extent. An upward adjustment of this observed value for moisture and for small sampling area suggested an adopted 1-hour all season PMP of 11.5 inches or a 3-hour value of 15.0 inches, based on the depth-duration relationship typical for the Northwest United States."

"Giving some reasons on the observation and experience basis, the observed 7.54 inches value on the Pacific side, was adopted as 1-hour point PMP on the Atlantic side (near Cristobal). This value was extrapolated to 24-hour 10-mi^2 PMP using the following ratios:

	Ratio 1-hour to 24-hour
Hawaiian	0.25
TP 40 for Gulf Coast	0.33
100-year rainfall (4.4/13.7)	0.32
Design Storm Panama	0.24

(Brod, Howard, W., "Hydrology of the Panama Canal," Part I, Flood Control, Department of Operation and Maintenance, Balboa Heights, Canal Zone, 1941)

The above ratios resulted in a range of 23 to 31 inches for the 24-hour 10-mi^2 PMP. Adjustment to HMR 4 values resulted in a range of 25.3 to 32.1 inches. For adjustment of 100-year rainfall to PMP, ratios were derived for Hawaii and Gulf Coast. The ratio was estimated to range from 1.7 to 2.3. This range multiplied with the 100-year value of 13.2 at Cristobal, resulted into a PMP range of 22.4 to 30.4 inches."

"Based on the above analyses, a value of 28 inches was adopted for the 24-hour 10-mi^2 PMP on the Atlantic side of the canal zone. For extension of this point rainfall to other area, four factors were considered.

1. Latitude trends
2. Atlantic to Pacific trends
3. Terrain relationships
4. Comparison with equatorial rain data."

"100-year daily rainfall values were assumed to represent latitudinal trend. Representative sea level values in the Canal Zone were 11.7 inches at Cristobal, 7.9 inches at Balboa Heights and 9.6 inches at La Palma. However, these values were affected by unequal period of record."

"There are variations from Atlantic to Pacific. In the Canal Zone, rains were extreme for one day and longer duration on the Atlantic side than on the Pacific side. This was based on the observed maximum and 100-year values."

"Experience of extreme events at coastal relative to foothill or mountain ridge locations suggests that the manner in which the 24-hour PMP index map should vary with terrain. There are apparent topographic effects in the highest observed and 100-year rainfall values. The data suggested that 100-year values are highest on slopes near to the coasts and on windward foothill areas but decrease on higher slopes and on the lee slopes in response to decreasing moisture. Compared to mean annual values, the 100-year values of daily rainfall show less areal variation in areas where data are available, because of greater effect of rain frequency on the mean annual values than on 100-year values."

"The 100-year data suggest a triggering effect along coast lines and in foothills areas which readily stimulates instability release with less lifting required than in middle altitudes. Thus, light on shore winds and diurnal heating (or both) can trigger extreme convective moisture release when the temperature lapse rate and moisture conditions are right. Early release thus robs unstable air of rain that otherwise would fall further inland or on higher slopes. Combined with this, the effect of distance from the coast is that of decrease of moisture with elevation. With little orographic lifting involved, the net effect is considered to be a decrease in rain potential above low elevations in general long-duration storms. Trends in lee areas evident from the mean annual maps are considered valid in a limited sense to PMP."

After all the above discussion, the report does not say how the 24-hour 10-mi² isohyetal map was developed. Probably, it was sketched as the best judgment.

"About 10 storms from 1923 to 1959 were initially selected to develop depth-duration-area relationships. Separate isohyetal maps were plotted for the day of heaviest rain in five of the ten selected storms. Aerial average rainfall for each storm was expressed in percent of 10-mi² values. Values with least decrease with increasing area were adopted as:

Area (mi²)	Percent of 10-mi² Rain
10	100
50	95.2
100	91.5
150	88.5
200	86.0
250	84.0
300	82.0
350	80.0
400	78.5
450	77.0
500	75.2"

"Station rainfalls for locations in Panama and Colombia were analyzed to define the durational variation of 10-mi² PMP for 1 to 6 days. Highest ratios (ratio to one-day rainfall) adopted were 100, 133, 157, 175, 190 and 204 percent for 1, 2, 3, 4, 5, and 6 days, respectively."

"Using the depth-area and depth-duration data, smooth depth-area duration curves were drawn. These curves were evaluated and judged to be applicable for the Canal Zone area."

"Estimates of local storm PMP were made for the areas up to 100 mi². The values are given in the table below:

Hours	Point PMP (inches)	Hour	Point PMP (inches)
.25	6.5	3.0	15.0
.50	9.0	3.5	15.4
11.5	4.0	15.8	
12.9	5.0	16.2	
13.8	6.0	16.5	
2.5	14.5		

"Probable Maximum Precipitation Estimates for Drainages above Gatun and Madden Dams, Panama Canal Zone," by F.K. Schwarz and J.T. Riedel, Hydrometeorological Branch, Office of Hydrology, National Weather Service, February 1978.

This study was an extension of the 1965 study by Weather Bureau. The report presented additional storm data since 1965. The following analyses were made and presented:

1. From the tracks of major hurricanes, it was concluded that hurricanes do not affect canal-zone watershed. The track of hurricane Martha that affected Panama was also given.

2. Mean October to December isohyetal map was developed using data for the period of 1941 to 1970. The map was based on the rainfall data at the stations and extrapolated data for higher elevations using a relationship between the mean October to December rainfalls and station elevations.

3. Three-day rainfall isohyetal maps were drawn for the selected storms of November 17-19, 1909; October 22-24, 1923; November 7-9, 1931; November 27-29, 1932; November 5-7, 1939; November 12-14, 1941; December 18-20, 1943; December 12-14, 1944; November 3-5, 1966 and April 7-9, 1970.

4. The procedure for developing the isohyetal maps included the following steps:

   i. Plot three-day rainfall at each station
   ii. Express station rainfall as percent of October-December rainfall
   iii. Draw smooth lines to cover Gatun catchment
   iv. Put back the percentage map on the October-December map
   v. Multiply the percentage with October-December rainfall and draw smooth isohyetal

5. Depth-area relationship was developed for each storm as percent of 3-day, 10-mi^2 rainfall

6. Depth area curves were not enveloped, 75 percentile values were determined at 1,285 mi^2 (area draining into Gatun Lake) and 393 mi^2 (area draining into Madden Lake). The area reduction factor for Gatun Lake was about 0.65.

7. One-day 10-mi^2 PMP map developed in 1965 was extended over the catchment for Gatun Lake. The difference was a 2-inch increase in the PMP in the extreme southwest portion of the drainage area.

8. For the Gatun drainage area, the one-day, 10-mi^2 was about 26.1 inches from the PMP isohyetal map. The ratio of 3-day to one-day rainfall was 1.56. This resulted into 3-day, 10-mi^2 rainfall of 40.7 inches.

9. The depth duration data (inches) for Madden and Gatun was reported as:

Hrs	Madden	Gatun
6	12.6	10.0
12	17.0	13.8
18	19.8	16.4
24	22.0	18.6
36	25.0	21.1
48	27.6	23.0
60	29.8	24.9
72	31.8	26.4
84	33.8	27.9
96	35.6	29.4
108	37.3	30.7
120	39.0	32.0
132	40.5	33.2
144	42.0	34.3

10. With sixth increment being the lowest and first as the highest, the time distribution of six-hour increments was suggested as 6, 4, 2, 1, 3, and 5.

"Sedimentation in Madden Reservoir," Meteorological and Hydrographic Branch, Engineering Division, Engineering and Construction Bureau, Panama Canal Commission, Balboa, Panama, June 1985, Revised January 1987.

The drainage area contributing to the Madden Lake is about 376.6 mi^2. Of this, the most important and basically unaltered is the Chagres forest reserve with about 301.3-mi2 area. The first detailed survey of the lake was made in 1983 at lake elevation of 235 feet. As of December 1983, the volume of accumulated sediment in Madden Lake since impoundment was estimated to be 30,700 acre-feet (49,469,319 cubic yards). This is about 4.7 percent of the total storage capacity of about 648,000 acre-feet. Active storage of about 470,000 acre-feet is between elevation 200 and 252 feet. The inactive storage below elevation 200 feet is about 178,000 acre-feet.

Using the total drainage of 376.6 mi^2 as erosion-susceptible, the sediment deposition rate would be about 113.11 cubic feet/acre/year during the 49 years of operation. With trap efficiency of 93 percent (Brune's diagram), the average sediment yield would be about 121.03 cubic feet/acre/year. However, the major erosion in the watershed started in about 1958 when farmers and cattlemen became more active in the watershed. Using a weighted calculation, a more accurate yield was considered to be about 177.56 cubic feet/acre/year.

A comparison was made of the suspended sediment measured on the three rivers and the measured deposit in the lake. This did not consider the sediment contributed by about 15 small tributaries directly entering the lake. A bed load of about 15 percent was added to the measured suspended loads. To obtain a volumetric estimate, a specific weight of 65 pounds/cubic feet was used. The three-year sediment inflow was about 454 acre-feet/year compared to 49-year deposit of 628 acre-feet/year.

"Madden Reservoir Sedimentation, 1984-1986," by Jack R. Tutzauer, Meteorological and Hydrographic Branch, Engineering Division, Engineering and Construction Bureau, Panama Canal Commission, Balboa, Panama, March 1990.

In this report, the sediment volumes measured by the hydrographic survey of 1983 were adjusted for sediment volumes contained between 235 feet and 252 feet, which were not measured by the survey.

The 1981-86 suspended sediment data measured at the three tributaries were used to establish a correlation between rainfall and suspended sediment. The relationship is:

$$Ss = 296.66\ Rm^2 - 50516.23\ Rm + 22383608$$

Se = estimated total annual suspended sediment in tons transported by three rivers
Rm = annual Madden watershed rainfall in inches

Based on the above relationship and 51 annual Madden watershed rainfall values from 1933 to 1983, the estimated total amount of suspended sediment transported by three rivers was 736 million cubic feet (mcf). Increasing this by 15 percent for bed load, the yield was 866 mcf. Using a trap efficiency of 99 percent, the deposition in the lake Madden based on inflow was about 857 mcf.

The drainage area of the three rivers at the gaging stations is about 247 mi^2. The total area contributing to the lake is about 393 mi^2 minus an area of about 11 mi^2 covered by the lake. This gave an adjusted deposit of 1,328 mcf (1.55*857). The factor of 1.55 was computed as (857)*((393-11)/247). This volume was about 15 percent less than the 1,572 mcf estimate based on the 1983 hydrographic survey.

The amount of sediment in Madden Lake at the end of 1983, below elevation 235 was about 1,353 mcf. The maximum usable elevation is about 252 feet. The lake area between 235 and 252 feet is about 3 mi. Adjusting for this area, the total sediment deposit was 1,572 (1353+219) mcf.

Based on the adjusted hydrographic survey of Madden Lake, a correlation was developed between Madden watershed rainfall and sediment deposited in the lake.

$$Sd = (296.66\ Rm^2 - 50516.23\ Rm + 22383608)\ 65.723 * 10^6$$

Sd = estimated total annual sediment in mcf deposited in the lake,
Rm = annual Madden watershed rainfall in inches

To develop the relationship, the annual sediment deposit rates were computed as 3-river suspended sediment volume multiplied by a factor of 2.136. The factor was estimated to compensate for unmeasured area, bed load, trap efficiency, land use, etc.

Reconnaissance Report, Section 5 – Río Indio

This section provides the development plan for the Río Indio. The plan would include a dam and lake on the Río Indio connected by a tunnel to the Panama Canal watershed above Gatun Lake. There would be two power plants, one of 5 MW at the end of the diversion tunnel and the second of 25 MW on the Río Indio downstream from the dam.

The section includes brief discussion on hydrology, geology, lake operation, project features, construction material and development sequences. Hydrologic reliability and project cost-benefit are also discussed.

Hydrologic reliability was derived by simulating the lake operation using HEC-5 computer model. Two operating options for transfer of water from the Río Indio Lake to Panama Canal were considered. Under first option, the lake would fluctuate from normal operating level of 80 meters to a minimum of 70 meters, with 359 MCM of usable storage. For the second option, the lake would fluctuate between 50 and 80 meters with a usable storage of 993 MCM. The maximum flood level would be about 82.5 meters.

An uncontrolled spillway with a crest elevation at 80 meters is provided. The maximum surcharge level is 82.5 meters. The spillway is designed for a maximum discharge of about 920 m^3/s, estimated to be a flood of 1,000-year return period.

Appendix B
Pumping Alternatives

PRELIMINARY ASSESSMENT OF THE LOW COCLÉ DEL NORTE WATER SUPPLY PROJECT

APPENDIX B

EVALUATION OF PUMPING ALTERNATIVES

Prepared by

MWH

PRELIMINARY ASSESSMENT OF THE LOW COCLÉ DEL NORTE WATER SUPPLY PROJECT

APPENDIX B – EVALUATION OF PUMPING ALTERNATIVES

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No.	Title
1	Low Coclé del Norte Pump/Storage Project, Site Visit Report

1 INTRODUCTION

The Autoridad del Canal de Panama (ACP) is undertaking a canal capacity study, which includes the evaluation of additional sources of water to augment the Canal capacity. Currently, ACP has several water supply projects at the planning level. Furthermore, ACP has authorized MWH to undertake a pre-feasibility study of a project to supply water from the Low Coclé del Norte reservoir to the Panama Canal. This Appendix supports the main report, which documents the findings and conclusions of the Low Coclé del Norte water supply pre-feasibility study.

1.1 Alternatives Considered

Two alternatives were considered to supply water from the Low Coclé del Norte reservoir to the Panama Canal:

- Alternative 1 – Pumping from Low Coclé del Norte reservoir to Toabré reservoir; and
- Alternative 2 – Pumping from Low Coclé del Norte reservoir to Caño Sucio reservoir.

Furthermore, the following three scenarios were considered for Alternative 1:

- Scenario 1 – Pumping against variable reservoir elevation with energy requirements supplied from the national electric grid;
- Scenario 2 – Pumping against a constant elevation over the top of the Toabré dam with energy requirements supplied from the national electric grid;
- Scenario 3 – Pumping against variable reservoir elevation with energy supplied by a hydroelectric power station on the Low Coclé del Norte dam.

The Low Coclé del Norte reservoir would be impounded by a roller compacted concrete (RCC) gravity dam. The reservoir water level would be kept constant at elevation of 35.0 m.

This Appendix describes the scope, approach and methodology of the studies carried out to evaluate the technical requirements of pumping water from Low Coclé del Norte reservoir to the Panama Canal.

1.2 Objective and Scope of Study

The objective of the desk study was to evaluate the technical requirements and cost of pumping water from the Low Coclé del Norte reservoir either to the Toabré reservoir or to the Caño Sucio reservoir.

The study included:

- Selection of pumping equipment, generating equipment, and transmission line voltages;
- Estimation of construction costs, operation and maintenance costs associated with the pumping facilities, and operation and maintenance costs associated with the hydroelectric power station at Low Coclé del Norte;
- Estimation of energy requirements for pumping;
- Estimation of net energy production by the hydropower station at Low Coclé del Norte (Scenario 3 of Alternative 1); and
- Estimation of volume of water supplied to the Panama Canal System.

1.3 Supporting Investigation

To assess impacts to the town of Coclecito, a site visit was made on July 29, 2003, and additional topographic information was obtained from ACP. A summary of the site visit, including photographs, is presented in an Attachment at the end of this appendix. Development of protection works for Coclecito is described in Section 5.1.7 of the Main Report.

2 DESCRIPTION OF ALTERNATIVES

2.1 Introduction

Two alternatives were considered to supply water from the Low Coclé del Norte reservoir to the Panama Canal:

- Alternative 1 – Pumping from Low Coclé del Norte Reservoir to Toabré reservoir; and
- Alternative 2 – Pumping from Low Coclé del Norte Reservoir to Caño Sucio reservoir.

2.2 Alternative 1

Alternative 1 would include a surface pumping station at the toe of the Toabré dam and steel pipes to discharge the water into the Toabré Reservoir. The pumps would be of the centrifugal type and a safety valve would be provided on each pump discharge line

The following three scenarios were considered for this alternative:

- Scenario 1 – Pumping against variable reservoir elevation with energy required for pumping being supplied from the national electric grid;
- Scenario 2 – Pumping against a constant elevation (over the top of the Toabré dam) with energy required for pumping being supplied from the national electric grid;
- Scenario 3 – Pumping against variable reservoir elevation with energy required for pumping being supplied by a hydroelectric power station on the Low Coclé del Norte dam.

The pumping station discharge pipes for Scenarios 1 and 3 would be approximately 150 m long each and would be installed in the Toabré dam openings for temporary river diversion (plugged after diver diversion). For Scenario 2, pipes would have the same length and would be installed on the downstream slope of the Toabré RCC gravity dam and over the crest. An energy dissipation structure (skip jump type) would be required on the upstream face of the dam to dissipate the kinetic energy of the water plunging into the reservoir.

The hydropower station for Scenario 3 would of the surface type and would be located at the toe of the Low Coclé del Norte dam. Steel penstocks would convey the power flows from the reservoir to the turbines in the powerhouse. The turbines would be of the Francis type and safety valves would be provided at the inlet of each the turbine spiral case.

The energy required for pumping for Scenarios 1 and 2 would the supplied from the electric grid by a 230 kV, single circuit, steel tower transmission line approximately 50 km long. In the case of Scenario 3, the energy required for pumping would be supplied by the Low Coclé del Norte hydropower station through a 115 kV, single circuit, steel pole transmission line approximately 21 km long.

2.3 Alternative 2

Alternative 2 would include a surface pumping station on the Low Coclé del Norte reservoir and a concrete lined tunnel, approximately 3.6 km long, connecting the pumping station with Caño Sucio reservoir. The pumps would be of the centrifugal type and a safety valve would be provided on each pump discharge line. An energy dissipation structure would be required at the discharge point in the Caño Sucio Reservoir to dissipate the kinetic energy in the water plunging into the reservoir.

The energy required for pumping would the supplied from the electric grid by a 230 kV, single circuit, steel tower transmission line approximately 50 km long.

3 STUDY OF ALTERNATIVES

3.1 Selection of Pumping Design Flow

The design flow for the pumping station was selected based on the criterion of 10% probability of exceedence of the monthly average flows for the period 1948-1999 at the proposed Low Coclé del Norte dam site. A duration curve of flows available for pumping was derived based on the average monthly flows, flow releases, and evaporation losses in accordance with the following formula:

$$Q_{pumping} := Q_{monthly\_average} - \left(Q_{release\_Toabre} + Q_{release\_Cocle} + Q_{evaporation}\right)$$

where:

$Q_{pumping}$ = flow available for pumping;
$Q_{monthly\_average}$ = synthesized monthly average flows for the period 1948-1999;
$Q_{release\_Toabre}$ = minimum release at Toabré dam;
$Q_{release\_Cocle}$ = minimum release at Low Coclé del Norte dam; and
$Q_{evaporation}$ = evaporation losses at Low Coclé del Norte Reservoir.

The following stable summarizes the minimum releases at Toabré and Low Coclé del Norte dams, and the evaporation losses at Low Coclé del Norte reservoir considered for Alternatives 1 and 2:

TABLE 1 MINIMUM RELEASES AND EVAPORATION LOSSES

Description	Alternative 1	Alternative 2
Minimum release at Toabré dam (m^3/s)	4.10	0.00
Minimum release at Low Coclé del Norte dam (m^3/s)	10.80	10.80
Evaporation losses at Low Coclé del Norte reservoir (m^3/s)	1.83	1.83
Total releases and evaporation losses (m^3/s)	**16.73**	**12.63**

The duration curves of the flows available for pumping for Alternatives 1 and 2 are presented in Exhibits 1 and 2, respectively. The selected design flows for the pumping station for Alternatives 1 and 2 are summarized in the following table:

Selected Design Flows for Pumping Stations – Alternatives 1 & 2

Description	Alternative 1	Alternative 2
Selected design flow for pumping station (m^3/s)	100.0	168.8

The criterion used for selecting the design flow for the pumping station for Scenario 3 of Alternative 1 was different from that indicated above. For this scenario, the design flows for the pumping station and hydropower station at Low Coclé del Norte were selected to balance energy generation with energy required for pumping water into the Toabré dam. The selected design flows are summarized in the following table:

Selected Flows for Pumping and Hydropower Stations – Scenario 3 of Alternative 1

Description	Alternative 1 – Scenario 3
Selected design flow for pumping station (m^3/s)	36.8
Selected design flow for hydropower station (m^3/s)	80.0

3.2 Pumping Operating Range

The criteria used to select the pumps operating range for Scenarios 1 and 3 of Alternative 1 were derived from equipment manufacturer's curves (Hitachi, LTD) for the Mainstream Tunnel System Pumping Station of the Metropolitan Sanitary District of Greater Chicago as are summarized in the following table:

TABLE 2 CRITERIA FOR PUMPS OPERATING RANGE

Description	Criteria
Head	
maximum dynamic head	115% of dynamic design head
minimum dynamic head	60% of dynamic design head
Flow	
maximum flow (under minimum dynamic head)	130% of design flow
minimum flow (under maximum dynamic head)	65% of design flow

The Toabré reservoir would vary between the full supply level (FSL) of 95 m and the low supply level (LSL) of 50 m. Under these conditions, two sets of pumps would be required to handle the flows in accordance with the operating criteria described above. One set of pumps would operate for reservoir elevations varying from el. 95 m to 66.3 m and the other set of pumps would operate for reservoir elevations between 66.3 m and 50.0 m.

In the case of Scenario 2 of Alternative 1 (pumping over the top of Toabré dam) and Alternative 2 (free discharge at an elevation corresponding to the FSL of 100 m on the Caño Sucio reservoir), the static head would be constant and the pumps would operate at essentially the dynamic design head with small variations in head for pumped flows other than the design flow.

The design flows indicated above in Section 4.1 are the minimum flows to be pumped under the most unfavorable conditions, i.e., under the maximum dynamic head. In the case of Scenario 2 of Alternative 1 and Alternative 2 the pump minimum flow and design flow are the same given that the dynamic head is essentially constant (only minor variations for other flows than the design flow). The actual flows used to size the pumps are summarized in the following table:

Flows Used for Sizing Pumps

Description	**Alternative 1**			**Alternative 2**
	Scenario 1	**Scenario 2**	**Scenario 3**	
Flow used for sizing pumps (m^3/s)	154.0	100.0	56.8	168.8

The pump operating conditions in terms of flow and dynamic head for Alternatives 1 and 2 are summarized in the following table:

TABLE 3 PUMP OPERATING CONDITIONS

Description	Units	Alternative 1			Alternative 2
		Scenario 1	Scenario 2	Scenario 3	
Low Coclé del Norte Reservoir El.	m	35.0			
Upper Reservoir	-	Toabré			Caño Sucio
Full supply level (FSL)	m	95.0			100.0
Low supply level (LSL)	m	50.0			90.0
Set of pumps required	-	2	1	2	1
Pumps operating condition	-				
- Set for upper reservoir elevations	-				
Maximum reservoir elevation	m	95.0	101.5	95.0	100.0
Minimum reservoir elevation	m	66.3	101.5	66.3	100.0
Static head	-				
Maximum	m	60.0	66.5	60.0	65.0
Minimum	m	31.3	66.5	31.3	65.0
Dynamic head	-				
Maximum	m	60.6	67.1	60.6	73.8
Design	m	52.6	67.1	52.6	73.8
Minimum	m	31.9	67.1	31.9	73.8
Flow	-				
Maximum (under minimum head)	m^3/s	200.2	100.0	73.84	168.8
Rated	m^3/s	154.0	100.0	56.8	168.8
Minimum (under maximum head)	m^3/s	100.0	100.0	36.8	168.8
- Set for lower reservoir elevations	-				
Maximum reservoir elevation	m	66.3	N/A	66.3	N/A
Minimum reservoir elevation	m	50.0	N/A	50.0	N/A
Static head					
Maximum	m	31.3	N/A	31.3	N/A
Minimum	m	15.0	N/A	15.0	N/A

Description	Units	Alternative 1			Alternative 2
		Scenario 1	Scenario 2	Scenario 3	
Dynamic head	-				
Maximum	m	31.9	N/A	31.9	N/A
Design	m	27.6	N/A	27.6	N/A
Minimum	m	15.6	N/A	15.6	N/A
Flow	-				
Maximum (under minimum head)	m^3/s	200.2	N/A	73.84	N/A
Rated	m^3/s	154.0	N/A	56.8	N/A
Minimum (under maximum head)	m^3/s	100.0	N/A	36.8	N/A

3.3 Number of Pumps

The number of pumps that would be contained in the pumping station was arbitrarily assumed at 4 for the two alternatives. However, for Scenarios 1 and 3 of Alternative 1, two sets of 4 pumps each were selected to handle the wide range of head conditions. Therefore, a total of 8 pumps were selected for each of these two scenarios. The number of pumps should be optimized in subsequent studies.

3.4 Pump and Motor Installed Capacities

The pump installed capacity, also called pump output power or "water horsepower", is the power imparted to the liquid and was calculated using the following formula:

$$\text{Pump}_{\text{installed\_capacity}} := \frac{9.81 \text{Flow} \cdot \text{Head}}{\text{pump}_{\text{eff}}}$$

where:

$\text{Pump}_{\text{installed\_capacity}}$ = power imparted to the liquid in kW;
Flow = pump design flow in m^3/s;
Head = design dynamic head in m;
Pump_{eff} = pump peak efficiency (assumed at 83%).

The motor installed capacity, also called pump input power or "brake horsepower", is the power delivered by the electric motor to the pump shaft and was calculated by the following formula:

$$Motor_{installed\_capacity} := \frac{Pump_{installed\_capacity}}{motor_{eff}}$$

where:

$Motor_{installed\_capacity}$ = power delivered by the electric motor to the pump shaft ("brake horsepower") in kW;

$Pump_{installed\_capacity}$ = power imparted to the liquid in kW;

$Motor_{eff}$ = motor peak efficiency (assumed at 93%).

The following table summarizes the characteristics of the pumps and motors for Alternatives 1 and 2:

TABLE 4 CHARACTERISTICS OF PUMPS AND MOTORS

Description	Units	Alternative 1			Alternative 2
		Scenario 1	Scenario 2	Scenario 3	
Pump					
- Sets of pumps required	-	2	1	2	1
- Number of pumps per set	-	4			
- Upper stage set	-				
Design flow (each pump)	m^3/s	38.5	25.0	14.2	42.2
Design dynamic head	m	52.6	67.1	52.6	73.8
Peak efficiency	%	83			
Rated installed capacity	kW	23,900	20,000	8,825	36,797
Rotational speed	rpm	257.14	327.3	400.0	276.9
- Lower stage set	-				
Design flow	m^3/s	38.5	N/A	14.2	N/A
Design dynamic head	m	27.7	N/A	27.7	N/A
Peak efficiency	%	83	N/A	83	N/A

Description	Units	Alternative 1			Alternative 2
		Scenario 1	Scenario 2	Scenario 3	
Installed capacity	kW	12,605	N/A	4,654	N/A
Rotational speed	rpm	225.0	N/A	327.3	N/A
Motor					
- Upper stage set	-				
Peak efficiency	%	93			
Rated installed capacity	kW	25,700	21,500	9,490	39,567
Rotational speed	rpm	257.14	327.3	400.0	276.9
Frequency	-	60			
Power factor	-	0.9			
Rated continuous output in kVA	kVA	28,600	23,900	10,550	43,970
- Lower stage set	-				
Peak efficiency	%	93			
Rated installed capacity	kW	13,354	N/A	5,005	N/A
Rotational speed	rpm	225.0	N/A	327.3	N/A
Frequency	-	60	N/A	60	N/A
Power factor	-	0.9	N/A	0.9	N/A
Rated continuous output in kVA	kVA	15,060	N/A	5,560	N/A

3.5 Number of Hydroelectric Generating Units

The number of generating units that would be contained in the Low Coclé del Norte powerhouse was arbitrarily assumed at 4. The number of units should be optimized in subsequent studies.

3.6 Hydroelectric Power Station – Plant Installed Capacity

The installed capacity of the hydroelectric power station at Low Coclé del Norte dam (Scenario 3 of Alternative 1) was estimated at 24,280 kW, at the high voltage side of the transformers, based on the following assumptions and parameters:

Low Coclé del Norte reservoir elevation (constant):	35.0 m
Tailwater elevation (constant):	0.0 m
Gross head:	35.0 m
Hydraulic headlosses (for plant design flow of 80 m^3/s):	0.44 m
Plant rated net head:	34.56 m
Plant rated flow:	80.0 m^3/s
Turbine rated efficiency:	92%
Generator rated efficiency:	98%
Transformer rated efficiency:	99.5%

The criterion used in selecting the installed capacity of the hydropower station was to match energy production with energy requirements to pump water from the Low del Coclé reservoir to the Toabré reservoir. The characteristics of the hydroelectric generating equipment are summarized in the following table:

TABLE 5 CHARACTERISTICS OF GENERATING EQUIPMENT

Description	Characteristics
Turbine	
Number	4
Type	Francis
Rated Capacity, kW	6,225
Rated flow, m^3/s	20
Rated head, m	34.56
Speed, rpm	300
Efficiency at rated conditions, %	92.0
Generator	
Number	4
Rated output, kVA	6,800
Power Factor	0.9
Speed, rpm	300
Frequency, Hz	60
Efficiency at rated conditions, %	98.0

3.7 Characteristics of Water Passages

The water passages associated with the pumping facilities, i.e, from the pump to the upper reservoir ("discharge line"), were sized assuming a maximum water velocity of 4.5 m/s (approximately 15 ft/s) for the rated flow condition. In the case of the hydroelectric power station (Scenario 3 of Alternative 1), the water passages were also sized assuming a maximum velocity of 4.5 m/s for the rated flow condition.

The following table summarizes the characteristics of the water passages associated with the pumping and hydroelectric generating facilities:

TABLE 6 CHARACTERISTICS OF WATER PASSAGES

Description	Units	Alternative 1			Alternative 2
		Scenario 1	Scenario 2	Scenario 3	
Pumping facilities – water passages					
type	-	Surface steel pipe			Tunnel
number	-	4	4	2	1
lining	-	steel			concrete
cross section type	-	circular			"D" shape
internal diameter	m	2.75		2.35	6.5
design flow (each water passage)	m^3/s	25		18.4	168.8
water velocity (at design flow)	m/s	4.21		4.24	4.57
length	m	150			3,600
Hydroelectric power station – water passages					
type	-	N/A	N/A	penstock	N/A
number	-	N/A	N/A	4	N/A
lining	-	N/A	N/A	steel	N/A
cross section type	-	N/A	N/A	circular	N/A
internal diameter	m	N/A	N/A	2.45	N/A
design flow (each water passage)	m^3/s	N/A	N/A	20.0	N/A
water velocity (at design flow)	m/s	N/A	N/A	5.0	N/A
length	m	N/A	N/A	100.0	N/A

3.8 Hydraulic Headlosses

Hydraulic headlosses were estimated for the water passages associated with pumping and hydroelectric generating facilities. The hydraulic headlosses comprise friction and form losses.

Friction losses were estimated for the design flow condition using the following formula (Darcy-Weisbach):

$$\text{FrictionLoss} = f \cdot \frac{L}{D} \cdot \frac{V^2}{2 \cdot g}$$

where:

FrictionLoss = hydraulic friction headloss (in pipeline, tunnel, etc);
f = Darcy-Weisback friction factor (see formula below);
V = water velocity (in pipeline, tunnel, etc.);
D = Internal diameter (pipeline, tunnel, etc.); and
g = acceleration of gravity (assumed at 9.81 m/sec^2).

The Darcy-Weisbach friction factor (f) was calculated using the following formula:

$$f := \frac{1.325}{\left[\ln\left(\frac{\frac{\varepsilon}{D}}{3.7} + \frac{5.74}{(Re)^{.9}}\right)\right]^2}$$

where:

f = Darcy-Weisback friction factor;
ε = wall roughness (pipeline, tunnel, etc.);
D = internal diameter (pipeline, tunnel, etc.); and
Re = Reynolds number (see formula below).

$$Re := \frac{V \cdot D}{\nu}$$

where:

Re	= Reynolds number;
V	= water velocity (pipeline, tunnel, etc.);
D	= internal diameter (pipeline, tunnel, etc.); and
ν	= water viscosity.

The following parameters and assumptions were used in estimating friction headlosses:

- Wall roughness (ε) for use in the Darcy-Weisbach formula
    - Steel pipes and penstock (coated steel): 0.06 mm
    - Concrete lined tunnel: 0.17 mm
- Design flow used to estimate headlosses
    - Pumping facilities
        - Alternative 1 – Scenarios 1 & 2 (each steel pipe): 25.0 m^3/s
        - Alternative 1 – Scenarios 3 (each steel pipe): 18.4 m^3/s
        - Alternative 2 (tunnel): 168.8 m^3/s
    - Hydroelectric generation facility (each steel penstock): 20.0 m^3/s
- Length of water passages
    - Pumping facilities
        - Alternative 1 (each steel pipe): 150 m
        - Alternative 2 (tunnel): 3,600m
    - Hydroelectric generation facility (each steel penstock): 100 m
- Water viscosity (at 20 °C) : $1.01x10^{-6} m^2/sec$

Form losses include the hydraulic headlosses in singularities such as trashracks, water intakes, bends, transitions, bifurcations, valves, etc., and were assumed to be 15% of the friction headlosses.

The hydraulic headlosses estimated for the design flow condition are summarized in the following table:

TABLE 7 HYDRAULIC HEADLOSSES

Description	Units	Alternative 1			Alternative 2
		Scenario 1	Scenario 2	Scenario 3	
Pumping facilities - water passages					
design flow	m^3/s	25.0		18.4	168.8
friction losses	m	0.49		0.59	7.62
form losses (15% of friction losses)	m	0.07		0.09	1.14
total hydraulic headlosses	*m*	*0.56*		*0.68*	*8.76*
Hydroelectric power station - water passages					
design flow	m^3/s	N/A	N/A	20.0	N/A
friction losses	m	N/A	N/A	0.38	N/A
form losses (15% of friction losses)	m	N/A	N/A	0.06	N/A
total hydraulic headlosses	*m*	*N/A*	*N/A*	*0.44*	*N/A*

3.9 Water Supply to Panama Canal

The estimated volume of water that would be supplied to the Panama Canal with the construction of the Low Coclé del Norte Reservoir and pumping station is summarized in the following table:

TABLE 8 WATER SUPPLY TO PANAMA CANAL - SUMMARY

Description	Units	Alternative 1			Alternative 2
		Scenario 1	Scenario 2	Scenario 3	
Annual average flow	m^3/s	58.8		24.2	90.4
Annual average volume of water	m^3 x million	1,854.3		762.1	2,852.4
Average number of lockages per day	Locks/day	24.6		10.1	37.8

4 ESTIMATION OF ENERGY REQUIREMENTS FOR PUMPING

4.1 Introduction

The energy requirements for pumping were estimated for Alternatives 1 and 2 and consists of the energy at the high voltage side of the transformers (substation associated with pumping station) plus the following losses:

- Station services; and
- Transmission losses.

4.2 Energy Estimate at the High Voltage Side of Step Down Transformers

An Excel spreadsheet energy simulation model was developed to estimate the power and energy requirements for pumping. The model uses the synthesized monthly average flow data at the Low Coclé del Norte dam site for the period 1948-1999 adjusted for the minimum releases at Toabré and Low Coclé del Norte dams, and evaporation losses at the Low Coclé del Norte reservoir as described above in Section 4.1. The model assumes that the flows are constant and uniform throughout each day of the month.

For each month of the period of analysis (1948 to 1999), the model determines the following parameters as a function of both the flow available for pumping for that month and the pumping station rated flow (for pump station rated flows refer to Section 4.1):

- Hydraulic headlosses;
- Dynamic head;
- Monthly power and energy at the high voltage side of the transformers.

Equipment efficiencies were assumed constant for all operating conditions and are as follows:

Pump efficiency:	80%
Motor efficiency:	93%
Step down transformer:	99.5%
Pumping station overall efficiency:	74.03%

The static head for Scenarios 1 and 3 of Alternative 1 (pumping against variable head) was calculated assuming an average elevation of 80.0 m at the Toabré reservoir, which resulted in a constant static head of 45.0 m

The energy required for pumping estimated at the high voltage side of the step down transformers is summarized in the following table.

Energy Required for Pumping – Estimate at High Voltage Side of Transformers

Description	Units	Alternative 1			Alternative 2
		Scenario 1	Scenario 2	Scenario 3	
Annual avg energy required for pumping	GWh/yr	289.10	426.22	107.06	735.95

4.3 Energy Requirements for Station Services

Station service consumption losses such as pumping for cooling of bearings, heating, cooling, ventilation, lighting, etc. are in addition to the energy required for pumping. The following loads were assumed for estimating station service energy consumption:

- Alternative 1
  - Scenarios 1 and 2 500 kW
  - Scenario 3 250 kW
- Alternative 2 500 kW

The energy required for station services is summarized in the following table:

Energy Required for Station Services

Description	Units	Alternative 1			Alternative 2
		Scenario 1	Scenario 2	Scenario 3	
Annual energy required for station services	GWh/yr	4.38	4.38	2.19	4.38

4.4 Transmission Line Losses

Transmission line losses were assumed at 2.5% of the sum of the energy required for pumping and station services. However, the transmission line losses considered for Scenario 3 of Alternative 1 was 1.5% as a result of the shorter line length. The following table summarizes the estimated transmission energy losses:

Transmission Line Losses

Description	Units	Alternative 1			Alternative 2
		Scenario 1	Scenario 2	Scenario 3	
Annual transmission line losses	GWh/yr	7.24	10.77	1.67	18.51

4.5 Summary

The following table summarizes the estimated energy requirements for pumping which include the energy at the high voltage side of the step down transformers (substation associated with pumping station), station service losses, and transmission line losses:

TABLE 9 ENERGY REQUIREMENTS FOR PUMPING - SUMMARY

Description	Units	Alternative 1			Alternative 2
		Scenario 1	Scenario 2	Scenario 3	
Annual avg energy required for pumping	GWh/yr	289.10	426.22	107.06	735.95
Annual energy required for station services	GWh/yr	4.38	4.38	2.19	4.38
Annual transmission line losses	GWh/yr	7.24	10.77	1.67	18.51
Total	**GWh/yr**	**300.72**	**441.37**	**110.92**	**758.84**

5 ESTIMATION OF HYDROELECTRIC ENERGY PRODUCTION

5.1 Introduction

The energy required for pumping water from the Low Coclé del Norte reservoir to the Toabré reservoir under Scenario 3 of Alternative 1 would be provided by a hydroelectric power station located at the toe of the Low Coclé del Norte dam. As mentioned above, the hydropower station was sized to generate enough energy to satisfy the pumping energy requirements plus the following losses:

- Parasitic load (internal consumption of the plant);
- Transmission losses; and
- Station services at the pumping station.

5.2 Estimate of Gross Energy Production

An Excel spreadsheet energy simulation model was developed to estimate the power and energy production of the hydropower station. The model uses the synthesized monthly average flow data at the Low Coclé del Norte dam site for the period 1948-1999 adjusted for the minimum release at Toabré dam, evaporation losses at the Low Coclé del Norte, and the flows pumped into the Toabré reservoir. The minimum release at Low Coclé del Norte dam was assumed to pass through the turbines to increase energy production. The model assumes that the flows are constant and uniform throughout each day of the month.

For each month of the period of analysis (1948 to 1999), the model determines the following parameters as a function of both the flow available for generation for that month and the hydropower station rated flow (36.8 m^3/s):

- Hydraulic headlosses;
- Net head;
- Monthly power and energy at the high voltage side of the transformers.

Equipment efficiencies were assumed constant for all operating conditions and are as follows:

Turbine efficiency:	85%
Motor efficiency:	97%
Step up transformer:	99.5%
Plant overall efficiency (equipment):	82.04%

Both the reservoir elevation and tailwater level were assumed constant for all operating conditions, which resulted in a constant static head of 35.0 m.

The gross energy production at the high voltage side of the transformers was estimated at 113.11 GWh/yr.

5.3 Parasitic Load

Generator excitation and other station service consumption losses such as pumping for cooling of bearings, heating, cooling, ventilation, lighting, etc. are drawn from the energy generated by the plant and are generally known as parasitic load. A load of 250 kW was assumed for the station services and the annual energy losses were estimated at 2.19 GWh/yr.

5.4 Transmission Line Losses

Transmission line losses between the hydropower station and the pumping station (approximately 21 km long line) were assumed at 1.5% of the gross energy production at the high voltage side of the main step up transformers. Transmission losses were estimated at 1.67 GWh/yr.

5.5 Station Services at Pumping Station

Station service consumption losses at the pumping station include pumping for cooling of bearings, heating, cooling, ventilation, lighting, etc. A load of 250 kW was assumed for the station services and the annual energy losses were estimated at 2.19 GWh/yr.

5.6 Summary

The following table summarizes the estimated gross energy production at the high voltage side of the main step up transformers and the energy losses associated with parasitic load at the hydropower plant, transmission line, and station services at the pumping station. The net energy is the energy at the high voltage side of the step down transformers at the substation associated with the pumping station:

TABLE 10 ENERGY PRODUCTION AND LOSSES

Description	Energy (GWh/yr)
Gross Energy Production	*113.11*
Energy Losses	
Parasitic load (hydro power station)	2.19
Transmission losses	1.67
Station services (pumping station)	2.19
Total losses	*6.05*
Net Energy (used for pumping)	**107.06**

6 COST ESTIMATE

6.1 Construction Costs

Construction costs were estimated based on parametric cost analysis and include engineering and construction of the civil works, and procurement, installation, testing, and commissioning of the electrical and mechanical equipment. Construction costs were broken down into the following items:

- Civil works;
- Electrical and mechanical equipment; and
- Transmission line and interconnection.

The cost of the civil works pertains to the pumping station and associated substation, and water conveyance system from the pumping station to the upper reservoir. In the case of Scenario 3 of Alternative 1, the civil works also include the hydropower station and associated substation, and steel penstocks. The cost of the civil works was estimated assuming the following parametric relationship for the pumping and hydropower stations:

Description	Cost of Civil Works
Pumping station and associated substation	US$48/kW
Hydropower station and associated substation	

Note: The parametric cost is per kW of installed capacity

The cost of the water conveyance system was estimated based on quantities take-off and the following unit prices and assumptions:

- Penstock and steel pipes: US$5.5/kg
- Tunnel
  - Rock excavation: US$70.0/m^3
  - Rock support: 20% of the cost of rock excavation
  - Concrete lining: US$240/m^3

- Vertical shaft/Surge shaft
  - Rock excavation: US$120.0/$m^3$
  - Rock support: 15% of the cost of rock excavation
  - Concrete lining: US$300/$m^3$

The following lump sum amounts were considered for the energy dissipation structures that would be required at Toabré and Caño Sucio reservoirs for Scenario 2 of Alternative 1 and Alternative 2, respectively:

- Energy dissipation structure – Toabré reservoir: US$10.0 million
- Energy dissipation structure – Caño Sucio reservoir: US$ 1.5 million

The cost of the electrical and mechanical equipment for the pumping station was estimated as a water to wire package and includes all the electrical and mechanical equipment from the pump discharge valve to the substation (pumping equipment, auxiliary equipment, controls, main step down transformers, high voltage switchgear, etc). The cost estimates include supply, transport, installation, testing, and commissioning. Similarly, the cost of the electrical and mechanical equipment for the hydropower station (Scenario 3 of Alternative 1) was also estimated as a water to wire package. The cost of the water to wire package was estimated based on the following parametric relationship derived from regression analysis of cost data obtained from recent bids for hydropower stations.

Description	Cost of Water to Wire Package
Pumping station including substation	US\$653,597*$(kVA/rpm)^{0.5537}$
Hydropower station including substation	US\$687,997*$(kVA/rpm)^{0.5537}$

Note: kVA and rpm are the motor/generator rated output and rotational speed, respectively

The cost of the water to wire package for the pumping station was assumed as 95% of the cost for a hydropower station with the same installed capacity.

The cost of the interconnection with the grid includes the transmission line from the point of interconnection substation to the pumping station and the upgrade/modifications to the interconnection substation to accommodate the new line. The transmission voltage was assumed to be 230 kV. In the case of Scenario 3 of Alternative 1, the transmission line would connect the hydropower station with the pumping station and the transmission voltage was assumed to be 115 kV. The following units prices were considered to estimate the transmission line construction cost:

- Transmission line cost
  - Transmission voltage - 230 kV: US$135/m
  - Transmission voltage - 115 kV: US$90/m

The cost of the upgrade to the interconnection substation was assumed at US$ 800,000.

A 25% contingency was added to the construction costs to reflect the uncertainty of the estimates. The construction cost estimate is summarized in the following table:

TABLE 11 CONSTRUCTION COST ESTIMATE – SUMMARY
(US$ x million)

Description	Alternative 1			Alternative 2
	Scenario 1	Scenario 2	Scenario 3	
Civil Works				
Pumping station and substation	8.76	4.80	2.70	8.83
Steel pipes & manifold	3.38	3.38	1.33	0.8
Energy dissipation structure – Toabré	N/A	N/A	10.00	N/A
Tunnel	N/A			26.39
Vertical shaft	N/A			0.80
Surge Shaft	N/A			1.50
Energy dissipation structure – Caño Sucio	N/A			1.50
Hydropower station and substation	N/A	N/A	1.52	N/A
Steel penstocks	N/A	N/A	1.84	N/A
Subtotal Civil Works	*12.14*	*8.18*	*17.39*	*39.82*

Description	Alternative 1			Alternative 2
	Scenario 1	Scenario 2	Scenario 3	
Electrical and Mechanical Equipment				
Pumping station and substation	77.86	35.16	35.68	51.35
Hydropower station and substation	N/A	N/A	19.33	N/A
Subtotal E&M Equipment	*77.86*	*35.16*	*55.01*	*51.35*
Transmission				
Transmission line				
From electric grid to pumping station	8.44	8.44	N/A	8.44
From hydro and pumping stations	N/A	N/A	2.36	N/A
Upgrade interconnection substation	1.00	1.00	N/A	1.00
Subtotal Transmission	*9.44*	*9.44*	*2.36*	*9.44*
Total Cost	**99.44**	**52.78**	**74.76**	**100.61**

6.2 Cost of Energy for Pumping

The energy required for pumping for Scenarios 1 and 2 of Alternative 1 and for Alternative 2 would be purchased from the grid while the hydropower station at Low Coclé del Norte dam would supply the energy required for pumping under Scenario 3. The annual average cost of energy purchased from the grid to meet the requirements for pumping was estimated assuming an energy tariff of US$0.07/kWh and is summarized in the following table:

TABLE 12 COST OF ENERGY FOR PUMPING

Alternative	Estimated Pumping Energy Requirements (GWh/yr)	Estimated Cost of Energy Purchased from Grid (US$/yr x million)
Alternative 1		
Scenario 1	300.72	21.05
Scenario 2	441.37	30.90
Scenario 3	110.92	N/A
Alternative 2	758.84	53.12

Note: The energy required for pumping includes transmission losses and station services at the pump station in addition to the energy required to drive the pumps.

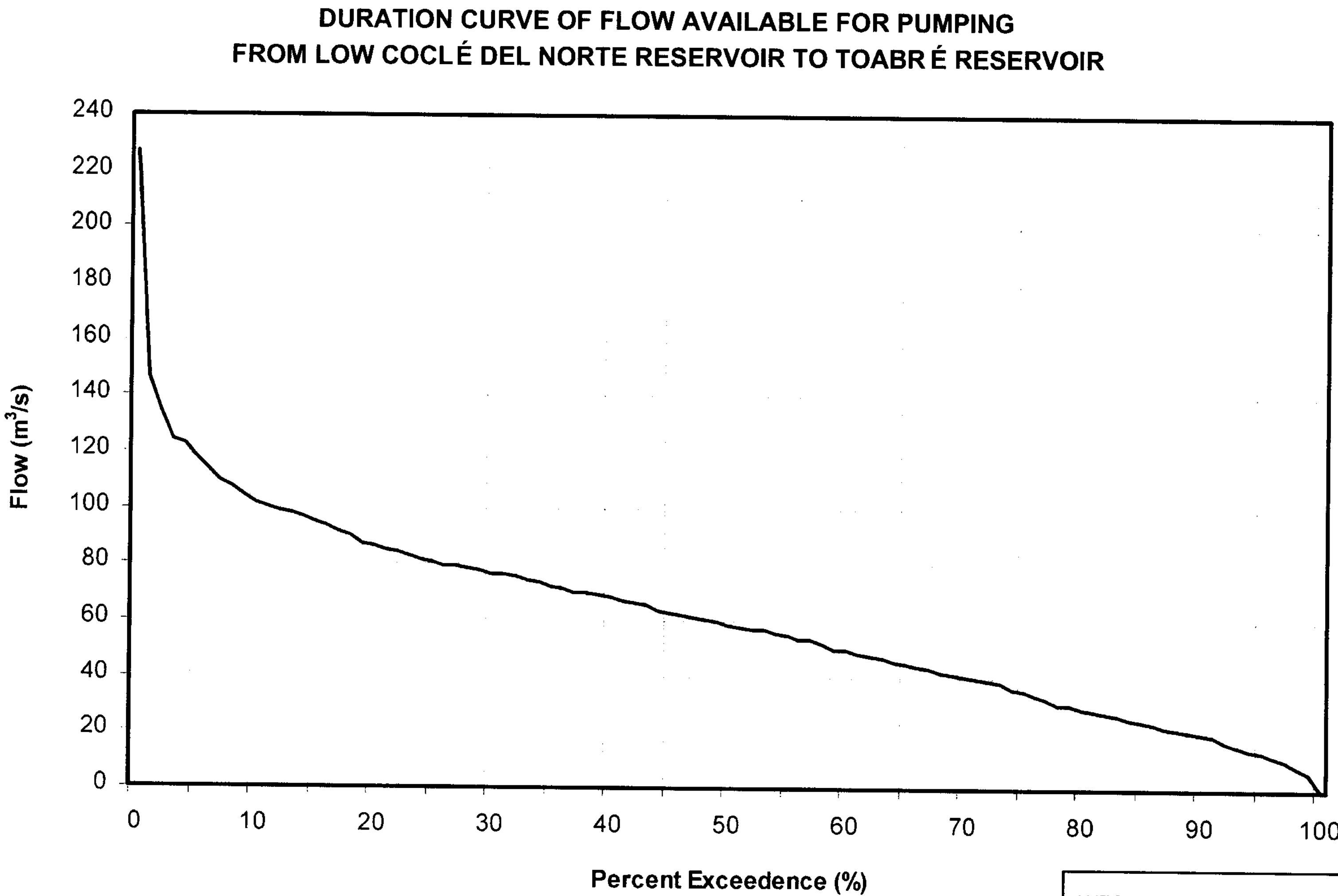
6.3 Annual Operation and Maintenance Costs

Annual operation and maintenance costs (O&M) were assumed at 2% of the construction cost of the pumping facilities, including the transmission line. In the case of Scenario 3 of Alternative 1, the construction cost of the hydropower station was also included. The O&M costs include administration, insurance, routine maintenance, breakdown or emergency maintenance, major repairs and overhauls, spare parts, and capital expenditures throughout the life of the project. The following table summarizes the estimated annual O&M costs:

TABLE 13 ANNUAL O&M COST ESTIMATE
(US$ x million)

Description	Alternative 1			Alternative 2
	Scenario 1	Scenario 2	Scenario 3	
Annual O&M Cost	2.0	1.1	1.5	2.0

EXHIBITS



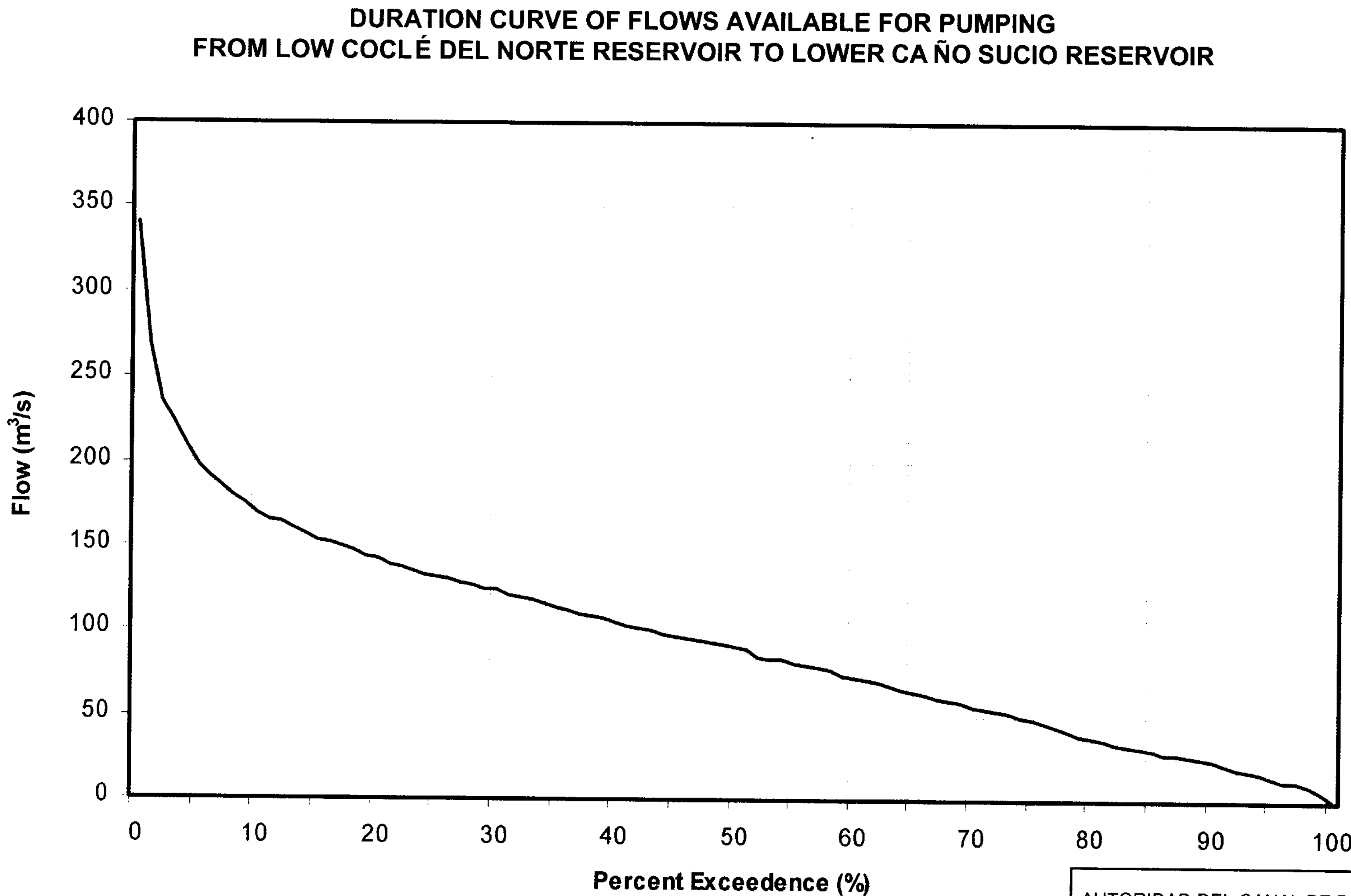
AUTORIDAD DEL CANAL DE PANAMA
Division de Proyectos de Capacidad del Canal

CONTRACT NO. CC-3-536
RÍOS COCLÉ DEL NORTE AND CAÑO SUCIO WATER SUPPLY PROJECTS
LOW COCLÉ DEL NORTE WATER SUPPLY PROJECT
APPENDIX B - EVALUATION OF PUMPING ALTERNATIVES

DURATION CURVE OF FLOWS AVAILABLE FOR PUMPING FROM LOW COCLE DEL NORTE RESERVOIR TO TOABRE RESERVOIR

MWH

DATE:	EXHIBIT:
DECEMBER, 2003	1



AUTORIDAD DEL CANAL DE PANAMA
Division de Proyectos de Capacidad del Canal

ACP

CONTRACT NO. CC-3-536
RÍOS COCLÉ DEL NORTE AND CA ÑO SUCIO WATER SUPPLY PROJECTS
LOW COCLÉ DEL NORTE WATER SUPPLY PROJECT
APPENDIX B - EVALUATION OF PUMPING ALTERNATIVES

DURATION CURVE OF FLOWS AVAILABLE FOR PUMPING FROM LOW COCLÉ DEL NORTE RESERVOIR TO LOWER CAÑO SUCIO RESERVOIR

MWH	DATE: DECEMBER, 2003	EXHIBIT: 2

ATTACHMENT

Location: Balboa, República de Panamá
Chicago, USA

Date: August 2, 2003

To: Michael Newbery

From: Rori Green

Subject: Low Coclé del Norte Pump/Storage Project
Site Visit (28 July 2003 to 1 August 2003)

INTRODUCTION

This Site Visit Memorandum documents preliminary reconnaissance activities conducted in support of the Low Coclé del Norte Pump/Storage Project. Activities performed include a general reconnaissance of Coclecito to evaluate the needs for flood protection measures and to generally assess what structures in the town might be affected by construction of the project.

During this trip, visits were also made to the proposed Lower Caño Sucio damsite and to the proposed upstream tunnel portal of the Caño Sucio to Río Indio Transfer tunnel. Observations made at these two sites are described in a separate memo.

Itinerary and Visit Activities

Date and Location	Activity
Monday, 28 July 2003 Chicago – Panama	Travel day
Tuesday, 29 July 2003 Coclecito and Caño Sucio Tunnel Intake Portal	At Coclecito, assess flood protection needs, take photos, and take GPS readings. At Caño Sucio, view proposed tunnel intake portal. R. Green (MWH) and D. Irving (ACP).
Wednesday, 30 July 2003 Lower Caño Sucio Site	Geologic reconnaissance and confirmation of abutment elevations. R. Green (MWH) and D. Irving (ACP)
Thursday, 31 July 2003 Panama City – ACP offices	Visit Tommy Guardia to obtain aerial photos of Coclecito and the Lower Caño Sucio site
Friday, 1 August 2003 Panama City – ACP offices	Meeting with Geocart Grafos to discuss available topographic information for Coclecito. Discussions with R. Lee (ACP) about the Caño Sucio Project.

Date and Location	Activity
Saturday, 2 August 2003 Panama – Chicago	Travel day (R. Green)

General Information

GPS base station data was collected by ACP surveyors at three sites on July 29 and July 30 for the purpose of making differential corrections. The base station data was provided on July 31 and the corrections made. The three base stations are all located within about 40 to 110 km from the Coclecito and Lower Caño Sucio sites. The base stations are designated UTPC, NEAB and 9212.

Project Description

The Coclé del Norte Pump/Storage project was identified by ACP and MWH in early 2003 as a potential alternative to a larger projected located on the Río Coclé del Norte. The project would consist of an RCC dam, approximately 45 to 50 m high, located on the Río Coclé del Norte at the same site that is currently being considered for the two larger Coclé del Norte projects. Water impounded by the lower dam would be pumped into either the Río Toabre or Caño Sucio reservoirs. The full supply level would be between El. 30 and El. 40.

The town of Coclecito, with about 700 to 800 inhabitants, would be partially inundated by the proposed project. Some form of flood protection, either walls or dikes, would be required in order to prevent inundation in certain areas of the town. Some structures located in low-lying areas would have to be relocated. The purpose of the present site visit was to make a general reconnaissance of Coclecito in order to assess possible flood protection options and to confirm ground elevations.

Photographs of Coclecito taken during the site visit are included at the end of this memorandum. The location of the bridge is indicated in the photos as a reference point.

Site Description - Coclecito

Coclecito is situated on the Río Coclecito between Quebrada La Mona and Quebrada El Fraile, and about 2 km downstream of the confluence of the Río Coclecito and Río Cascajal. The town is spread out over about 4 square km, about half of which is relatively flat terraces. The most densely populated area of the town covers a smaller area (about one square km), much of which is located on a series of hills right above an entrenched section on the west side of the Río Coclecito. Near this section of the town, a one-lane bridge crosses over the Río Coclecito. The land on the immediate east side of the bridge is sparsely populated and is predominantly flat pastureland.

Geology

Although the intent of the visit to Coclecito was not for geologic reconnaissance, the following observations were made:

- The Río Coclecito is entrenched where it is near the most densely populated portion of the town. The river channel is approximately 15 to 20 meters wide below the bridge. No depth measurements were made, however the river bottom could be seen and it is estimated to have a maximum depth of about 3 to 4 m.
- The river bottom is composed of rock outcrops and coarse-grained sediments consisting of sands and gravels. The rock outcrops were observed at one location and were found to be composed of volcanic agglomerate with a dark grey crystalline matrix
- There are at least two terrace levels developed in the entrenched section of the river, one about 2 to 4 m about the water surface and the other about 5 to 10 m above the first terrace. The width of the lower terrace varies and is up to about 10 to 15 m wide in some areas. The terrace deposits are primarily composed of silty sands, with some clays.
- Rock outcrops were observed in the river farther upstream from the helicopter, about 2-3 km u/s of airstrip, however we did not stop to observe the outcrops.

Construction Materials

A relatively small amount of materials would be required for construction of flood protection berms and/or walls. Fill material for berms could be obtained from overburden material excavated from near-by sources. Most of the ground surface in the area is covered by a layer of residual soil deposits consisting of silty sands and clays, which would be suitable.

Presumably there is a quarry or other location near Coclecito that the local inhabitants use for construction materials, however this was not confirmed during the visit. Otherwise, if required for construction, rockfill could be quarried from nearby hills such as Cerro Moreno, which is located about 3 to 4 km southeast of Coclecito.

Conclusions

The part of town that would be most significantly impacted by the construction of flood protection works is located on the west side of the Río Coclecito, just upstream and downstream of the bridge. This is the most densely populated section of town that lies within the proposed area of inundation.

Additional Information

As a result of land titling efforts, additional survey information has been collected at Coclecito by Geocart Grafos, a contractor to ACP. A meeting was held with Roderick Lee (ACP) and Gabriel Gomez (Geocart Grafos) to determine what information is available and whether or not it would be available to MWH for use in the present study. The information consists of ortho-rectified aerial photos, ground GPS surveys and an electronic file (CAD/GIS) delineating the infrastructure (roads, houses, schools...etc.) in and around Coclecito.

A request was made to ACP to provide MWH with available ground survey information and orthophotos in support of the present study. If these items are made available in a timely manner, then they will most likely be helpful in completing work on this part of the project.

Photographs of Coclecito

Photo 1 Looking Upstream to Bridge

Photo 2 Looking Downstream from Bridge

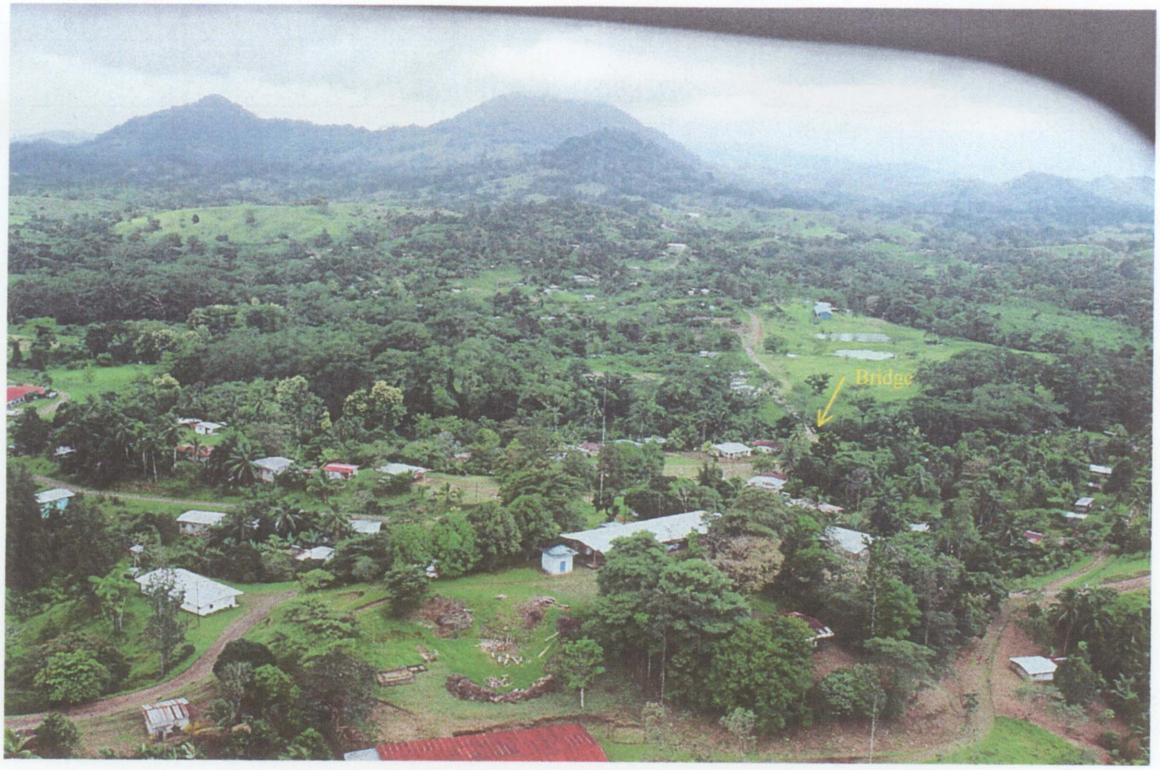


Photo 3 Coclecito - Looking East

Photo 4 Coclecito - Looking East-Southeast

Photo 5 Coclecito - Looking North

Photo 6 Coclecito - Looking Northwest

Photo 7 Coclecito - Looking Northwest along Airstrip

Photo 8 Coclecito - Looking Southeast