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

Work Order No.3 Feasibility Design For The Río Indio Water Supply Project

Contract Number CC-3-536

Panama Canal

VOLUME 2: APPENDICES

MWH
In association with
TAMS
AN EARTH TECH COMPANY

April 2003

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

THE PANAMA CANAL

ENGINEERING SERVICES

Work Order No. 3
Río Indio Water Supply Project

Feasibility Study

Volume 2 APPENDICES A-D

APRIL 2003

MWH

In association with
TAMS Consultants, Inc.
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Feasibility Design For
The Río Indio Water Supply Project

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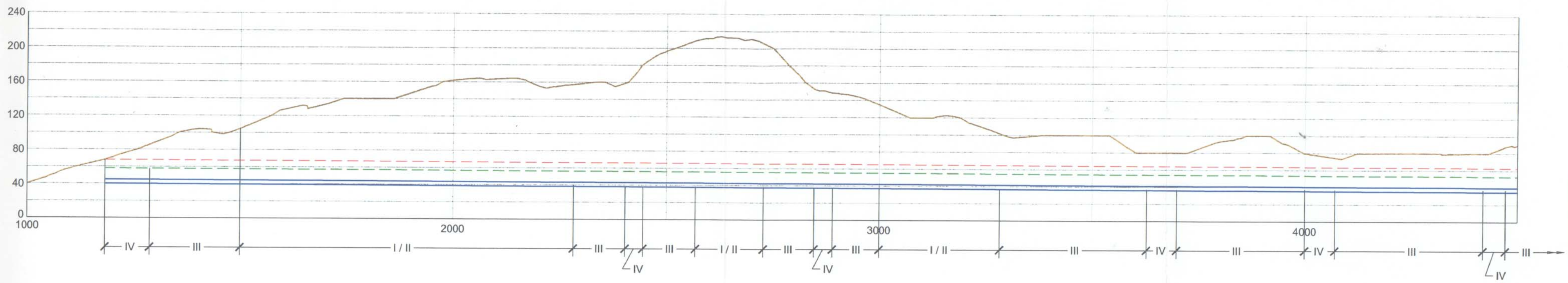

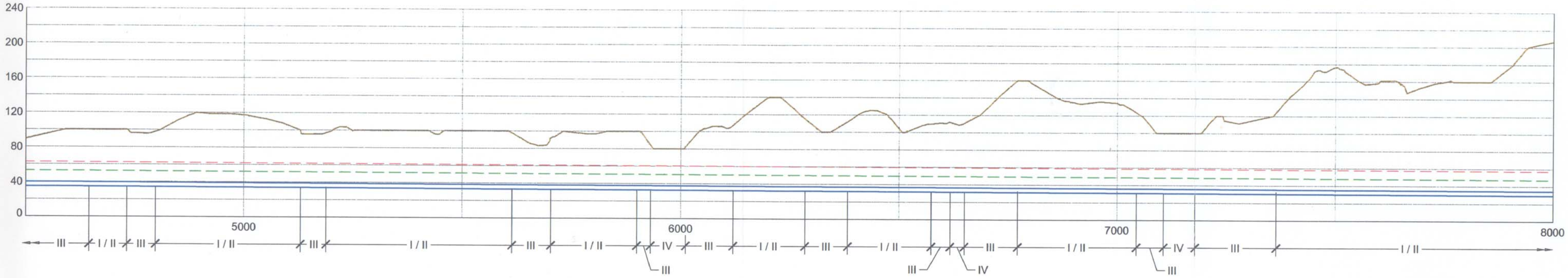

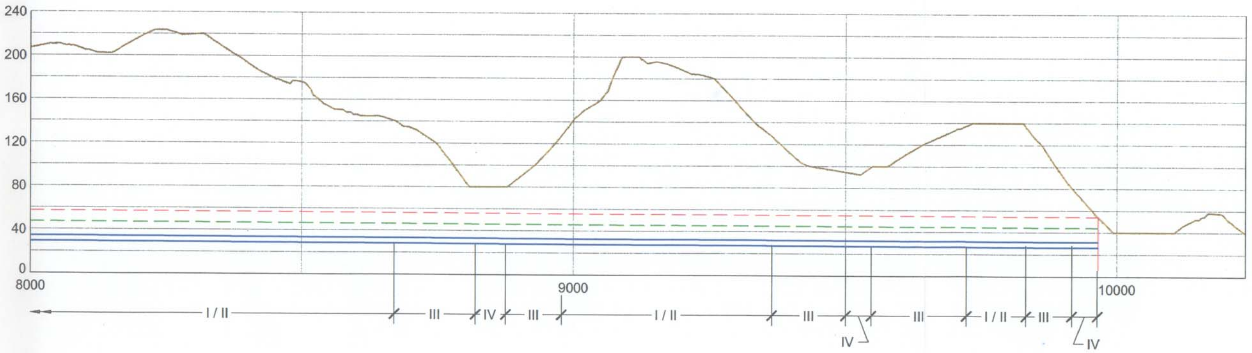

ROCK TYPE	LENGTH	%
I	1757	20
II	3210	35
III	3075	35
IV	743	10
TOTAL:	8785	

0 100 200 300 400 500 m

HORIZONTAL SCALE: 1:10,000
VERTICAL SCALE: 1:5,000

AUTORIDAD DEL CANAL DE PANAMA
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ACP
AUTORIDAD DEL CANAL DE PANAMA

CONTRACT NO. CC-3-536
RIO INDIO WATER SUPPLY PROJECT
APPENDIX D, PART 4 - INDIO - GATUN TRANSFER TUNNEL

RIO INDIO TRANSFER TUNNEL
ALIGNMENT 6

MWH TAMS

DATE:
APRIL, 2003

EXHIBIT:
4

Appendix A
Hydrology

FEASIBILITY DESIGN FOR THE RÍO INDIO WATER SUPPLY PROJECT

APPENDIX A

HYDROLOGY, METEOROLOGY AND RIVER HYDRAULICS

Prepared by

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In association with

TAMS

AN EARTH TECH COMPANY

FEASIBILITY DESIGN FOR THE RÍO INDIO WATER SUPPLY PROJECT

APPENDIX A – HYDROLOGY, METEOROLOGY AND RIVER HYDRAULICS

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EXECUTIVE SUMMARY

The hydrology and meteorology analyses performed for the Río Indio development consisted of the determination of five hydrologic parameters including:

- Develop long-term monthly flow sequence,
- Estimate net reservoir evaporation,
- Determine construction period floods,
- Develop spillway design flood using PMP/PMF techniques, and
- Assess erosion potential in the watershed and estimate the depletion of reservoir storage due to sedimentation.

The long-term monthly flow sequence at the proposed dam site was developed by ACP for the period from January 1948 to December 1998. The data for 1999 was included later. The sequence was reviewed, checked and considered to be reasonable for representation of long-term future conditions. The mean annual flow was about 25.8 m^3/s (about 911 ft^3/s). The drainage area at the dam site is about 381.1 km^2.

The rainfall and consequently the flows in the Río Indio basin are affected by El Nino. During the El Niño episodes of 1976, 1982, 1997 and 1998, the annual flows were about 72, 75, 43 and 69 percent of the long-term mean annual flow.

Net reservoir evaporation was estimated by ACP using the evaporation data from Gatun Lake. Annual net reservoir evaporation was about 1,134 mm (about 44.6 inches).

Construction period floods were estimated on an annual basis and for the dry season of January through March. Monthly maximum instantaneous flood peak data was used. Two probability distributions, log Pearson type III and generalized extreme value (GEV) were tested. Based on a visual judgment, both distributions resulted in a reasonably good fit. The GEV distribution provided slightly higher results and was adopted for conservatism. The estimated 10-, 20-, 50- and 100-year flood peaks were about 713, 762, 820 and 859 m^3/s, respectively. For the dry months, the peaks were about 94, 133, 202 and 274 m^3/s, respectively.

The probable maximum flood (PMF) was adopted a the design flood for the spillway. It was based on a 48-hour probable maximum precipitation (PMP) of about 711mm. The unit hydrograph for the basin was derived using Clark's method. The resulting PMF had a peak of 4,345 m^3/s and 5-day volume of 242.9 million cubic meters. The PMF hydrograph was routed through the reservoir assuming an initial reservoir elevation equal to the normal pool level (80.0 m). The elevation-volume and spillway rating curves were computed by ACP and the United States Army Corps of Engineers. These were judged to be reasonable and used in this study. As a result of the routing effect of the reservoir, the reservoir reached an elevation of 83.3 m corresponding to a spillway discharge of about 1,390 m^3/s. A revised routing will be performed when the spillway is optimized as a part of future studies.

No sediment data has been collected for the Río Indio basin and, therefore, the estimates of sediment yield were based on data and information obtained from adjacent basins. The mean annual sediment inflow for the Río Indio Reservoir was estimated to be about 554,880 metric tons. Analysis for the depletion of the live storage of the reservoir was made using the methods developed by the United States Bureau of Reclamation. As per current project investigations being performed by ACP and MWH, the live storage would be between elevations of 40 and 80 meters. The live storage between these elevations is about 1,294 million cubic meters. The analysis of live-storage depletion indicated that this storage will be reduced by about one and two percent after about 50 and 100 years of operation, respectively.

The river hydraulics component of this study included the following major tasks:

- Review water quality and sediment sampling and analysis methods, and make recommendations to improve the methods.
- Assess the project impact on water quality in the river downstream from the dam.
- Assess channel stability downstream from the dam.

At this stage of the report, water quality data for the Río Indio was not available. Also the methods of sampling and analysis could not be obtained.

Empresa de Transmision Electrica, S.A., Departmento de Hidrometeorologia, Section Hidrologia (ETESA) has collected suspended sediment data on the Río Indio. The methods of sampling and analysis are the same as recommended by the Water Resources Division of the United States Geological Survey (USGS).

ACP is collecting suspended sediment samples on the rivers draining into Gatun Lake and Madden Lake. The sampling techniques should be improved as discussed

During the construction period, the sediment production would be increased due to construction activities. The increased erosion could make the water turbid and, perhaps, unsuitable for drinking or other uses. A water quality management plan should be prepared prior to the construction and implemented for the construction period. Guidelines for the preparation of such a plan are provided.

During project operation, erosion of the reservoir banks due to water level fluctuations in the reservoir. The reservoir banks should be stabilized as needed.

The water temperature in the reservoir may show stratification. Therefore, the water released through the low level outlets may be cooler than normal.

Channel Stability analyses were made for "pre-project" and "post-project" conditions. Flood frequency data were developed for the two conditions. The analyses confirmed that under current condition, the bed of the Río Indio scours significantly. Under post-project condition, the flood peaks would be significantly reduced. The degradation below the dam would be insignificant. However, the bed material transported by the tributaries would cause aggradation near the mouths of these tributaries because there would not be sufficiently high floods in the main channel to remove these deposits.

1 INTRODUCTION

The Panama Canal Authority (ACP) has proposed a dam on the Río Indio to form a reservoir that will provide water to the Panama Canal watershed through a tunnel discharging into Gatun Lake. The transfer of water from the Río Indio to the Panama Canal would support navigation, municipal and industrial (M&I) water supply, and the generation of electricity. The proposed transfer would contribute significantly to the hydrologic reliability of the Panama Canal operation to serve its customers and it would reduce the need for imposing draft restrictions and resulting light loading of vessels during traditional periods of low water availability.

1.1 Objective and Scope

This Appendix describes the hydrologic and river hydraulic analyses performed at a feasibility level for the development on the Río Indio. The procedures and basic data used in the determination of the following hydrologic parameters are discussed.

- Streamflow Availability
- Reservoir Evaporation
- Construction Period Flood
- Spillway Design Flood
- Reservoir Sedimentation
- Impact on Water quality
- Channel Stability Downstream of the Dam

A summary of previous reports pertinent to the development on the Río Indio was also made and review comments are given as Attachment I.

1.2 Location and Accessibility

The Río Indio watershed is located to the west of the Panama Canal Area (see Exhibit 1 located at the end of the report). The dam will be located about 25 kilometers (km) inland from the Caribbean Sea Atlantic Ocean, near the mountain named Cerro Tres Hermanas, in the vicinity of latitude 9^0 $00'$ north and longitude 80^0 $11'$ west. The drainage area of the river at the dam site is about 381.1 km^2 (147 mi^2).

Presently the dam site is not accessible by road. Access is by boat or helicopter.

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 Rainfall

There is only one rainfall station in the Río Indio basin for which historic rainfall data is available. The station is located at Boca de Uracillo and it is a non-recording station. Daily rainfall data is available from September 1974 to December 1998. Monthly rainfall data at the station was generated for a 30-year period (1966 to 1995) through correlation with nearby stations in the Canal Area for this study. The mean monthly rainfall amounts are given in Table 1.

A mean annual rainfall map, taken from Atlas (1988), is shown on Exhibit 2. The map shows that mean annual rainfall is higher in the coastal area and decreases inland. There is a slight increase in rainfall with elevation in the head reaches of the Río Indio basin.

Estoque in 1985 (Estoque, ET al., 1985) studied the effect of El Niño on the rainfall in Panama. He listed thirteen episodes of El Niño for the period from 1920 to 1983, and compared the annual rainfall during an El Niño with the long-term mean annual rainfall. The results indicated that El Niño produced below normal rainfall in almost all regions of Panama. The average annual rainfall anomaly, based on all El Niño episodes was about 8 percent below normal. In case of the strong El Niño episodes of 1976 and 1982, the corresponding anomalies were about 28 percent and 24 percent below normal, respectively. The driest month of the year 1982 (December) had a rainfall anomaly of about 60 percent below normal. The study also indicated that there was a considerable geographic variation in the rainfall anomalies. In case of the 1976 El Niño, the largest magnitudes of negative anomalies were located in the southwestern part of Panama, just south of the central cordillera on the Pacific side. On the other hand, El Niño had the opposite effect (positive anomalies) in some basins north of the cordillera in the Atlantic coastal region. The 1997-1998 El Niño episode also resulted in low rainfall in Panama, particularly in Panama Canal Area.

Through verbal discussion with the climatologists of ACP, it was determined that in the Canal Area and neighboring basins, 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 rainfall at Boca De Uracillo were about 2,490 and 2,540 mm, respectively, compared to mean annual rainfall of about 3,078 mm. This indicates a decrease of about 19 and 17 percent, respectively. The 1997-98 El Niño significantly decreased the rainfall in the Canal Area and over the basins of the Río Indio and the Río Coclé del Norte. In 1997 the annual rainfall at Boca de Uracillo was about 50 percent of the mean annual rainfall (period 1975 to 1997).

2.3 Temperature

Mean monthly temperatures vary within about 2^0 C throughout the year. Mean annual temperature varies from about 26^0 C near the dam to about 24^0 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 Indio flows northward from the Continental Divide. The river rises on the northern slopes of Cerro Gaitel at about 1,180 meters above mean sea level. The river is quite steep in the head reach and gradually becomes relatively flat near the dam site. The river slope is about 6.5 percent in the first 5 km of the head reach. For the next 10- and 25-km reaches, the slopes are about 2.5 and 0.8 percent, respectively. Near the dam site slope is about 0.25 percent. The riverbed profile is shown on Exhibit 3.

The river basin is elongated in shape with a length of about 42 km. The basin is about 5-km wide in the head reach. In the middle reach, with the Río Uracillo on the left and the Río Teria on the right, the basin width is about 16 km.

The Río Uracillo, which joins the Río Indio at about 5.7 km upstream from the dam site, is the major left bank tributary. The drainage area of the tributary river is about 104 km^2 at the confluence with the Río Indio. The river slope is about 3.5 percent in the first 5 km of the head reach and gradually decreases downstream to about 0.3 percent near the confluence.

The major right bank tributary is the Río Teria. A number of other small tributaries join the Río Indio along its 71-km (river distance) traverse to the dam site.

4 STREAMFLOW ANALYSIS

4.1 Data Sources

There are 44 rainfall stations and 7 stream gaging stations in the vicinity. A list of these stations is presented in Table 2 and the locations of the stations are shown on Exhibit 4. Monthly streamflow data for stream gaging stations pertinent to this study were obtained from the 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 3 shows a list of the stream gaging stations and the period of record for measured flows. The rainfall stations were used for the design flood study as discussed under "Spillway Design Flood."

A stream gaging station was established on the Río Indio at Limon (drainage area about 376 km^2) in March 1958 by the Instituto De Recursos Hidraulicos y Electrificacion, Departamento De Hidrometeorologia (DMH). The station was equipped with a staff gage to record river stages and an overhead cable for discharge measurements. This station was discontinued in October 1980.

A new station equipped with an automatic water level recorder and an overhead cable was established in July 1979 at Boca de Uracillo about 4.3 km upstream from Limon site (see Exhibit 4). The drainage area at the station is about 365 km^2. The two stations were operated concurrently for about 15 months.

4.2 ACP Analysis

The ACP performed an analysis to generate a long-term monthly flow sequence on the Río Indio at Boca de Uracillo for the period from January 1948 to December 1999. This data was transposed to the dam site using drainage area ratio of 1.044 (381.1/365). The monthly flows at Boca de Uracillo were generated using the following steps:

1. Measured monthly flow data at Boca de Uracillo was available from August 1979 to May 1998 with about 28 months with missing data. The station is equipped with a water level recorder. The data for the missing

months was estimated either using gage height data from the staff gage at the station or based on general trend in the monthly flows.

2. Monthly flow data for the period from January 1948 to December 1999 were available for the Río Trinidad at El Choro (drainage area 172 km^2). A correlation was developed between the monthly flows of the Río Ciri Grande at Los Canones (drainage area 186 km^2) and the Río Trinidad at El Choro using the concurrent period of record. The regression equation is given below:

$$\text{Los Canones} = 1.4075\ (\text{El Chorro})^{0.9983}$$

3. The above equation was used to fill-in monthly flows at Los Canones for the missing period from January 1959 to July 1978 plus a few other missing months in the remaining period. This resulted in monthly flows for the period from January 1948 to December 1999.

4. Using concurrent monthly flows of the Río Indio at Boca de Uracillo and of the Río Ciri Grande at Los Canones, the following regression equation was developed.

$$\text{Boca de Uracillo} = 2.865\ (\text{Los Canones})^{0.9606}$$

5. The above equation was used to generate the monthly flow data for the period from January 1948 to July 1979 and June to December 1999.

ACP performed mass curve and double mass curve analyses using the monthly flow series at various stations to check the consistency of the data.

4.3 Review of the ACP Analysis

The streamflow analysis performed by ACP was reviewed. The correlation coefficients for the both regression equations were above about 0.95. The mass curves of the monthly flows indicated consistency in the data. The two-step procedure, first to develop a correlation between Los Canones and El Chorro and then between Los Canones and Boca de Uracillo, was reasonable because the Río Trinidad is east of the Río Ciri Grande and the Río Indio is on the west of the Río Ciri Grande. The rainfall decreases from east to west. Therefore, the ACP first generated the data for the Río Ciri Grande and then for the Río Indio using the data of the Río Ciri Grande. This approach is logical and acceptable.

4.4 Monthly Streamflow Sequence

Long-term monthly discharges (in m^3/s and cfs units) for the Río Indio at Boca de Uracillo are given in Table 4. Exhibits 5 and 6 show the mass curve and the time series of annual flows for the Río Indio at Boca de Uracillo, respectively. These exhibits show that the annual flows are consistent and homogeneous. In addition, there is no statistically significant trend in the data. However, there are significant variations in flows from year to year. The highest flow occurred in 1996 followed by lowest in 1997. These were the measured flows at Boca de Uracillo. The El Niño episode was recorded in 1997-1998.

The monthly discharges (in m^3/s and cfs) at the dam site for the period from January 1948 through December 1999 were developed by ACP by multiplying the flows at Boca de Uracillo with the drainage area ratio (381.1/365 = 1.044). These data are given in Table 5. The estimated flows were judged to be reasonably accurate and adopted for this study.

4.5 Streamflow Characteristics

The estimated mean annual flow at the damsite for the period 1948 through 1999 is 25.8 m^3/s. For the same period, the mean annual flow at Boca de Uracillo was about 24.7 m^3/s.

The wet period is generally from October through December but quite often, high flows can occur in the month of 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 occur in the months of June through August. The highest flood of record (from 1979 to 1996), about 772 m^3/s, occurred in December 1991.

The potential firm yield of the Río Indio at the dam site was estimated using mass curve analysis (as a spreadsheet) and the monthly flow data from 1948 to 1998. For selected yield rates, the required active storage was determined for each rate. The yield curve is shown on Exhibit 7. This curve shows that the firm yield at the dam site would be about 24.1 m^3/s, or about 98 percent of the mean annual flow, and would result from an active storage of about 1,127 million cubic meters (MCM)). Using the elevation-volume relationship given in Table 6, and assuming a minimum operating level of 50 meters, which corresponds to the preliminary studies reported in the USACE Reconnaissance Report (1999), a yield-reservoir elevation curve was developed as shown on Exhibit 8. This curve shows that for a firm yield of about 24.1 m/s, the maximum reservoir elevation would be about 72.5 meters. By increasing the maximum reservoir elevation, the firm yield will not increase significantly.

4.6 Flow Duration Curves

Daily flow data for the Río Indio at Boca de Uracillo was available for the period from 1980 to 1996. The data was used to develop a flow duration curve (Exhibit 9). The minimum observed flow was about 0.8 m^3/s. Flows exceeding 90 and 95 percent of the time were estimated to be 3.6 and 2.6 m^3/s, respectively. Transposed to the dam site, the flows would be about 3.8 and 2.7 m^3/s, respectively. A flow duration curve based on the monthly flows at the damsite is shown on Exhibit 10.

4.7 Drought-Duration-Frequency Analysis

The method of analyzing drought frequencies and duration is based on the assumption that meteorological conditions recorded in the past would be repeated. In most cases, the absence of long records, potential long-term variation in rainfall and runoff, and topographic changes brought by man make it rather difficult to make precise forecasts.

For drought analysis, data is selected by one or two methods: for either one, an extreme value is chosen for each time unit, such as the lowest monthly flow in a year or the lowest monthly flows for selected durations in the period of record regardless of when they occurred. With the latter method, the number of values chosen need not equal the number of years of record.

The first method is not very useful since this deals with a discrete value of flow and reveals nothing about the sequence of low flows. The second method is more useful. In this case, the analysis is made by determining the flows over a given period of consecutive days, months or years. A difficulty encountered in this frequency analysis of sequential events is overlapping of data and repeated appearance of extreme values.

Thus, in the analysis of droughts lasting 24-months (or two years), certain low flow months might appear twice. The overlapping is eliminated as illustrated by the following example of monthly flows for the Río Indio.

Monthly flows of the Río Indio 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." The return period for the second lowest value was "52/2 = 26."

In case of 6-month period, there were more than 52 values or more than the years of record. In this case, only 52 lowest values were used. The lowest values for the selected duration are given in Table 7. The table also shows the recurrence intervals. Exhibits 11 to 14 show the frequency curves.

The 6-, 12-, 18- and 24-month flows and their assigned recurrence intervals furnish estimates of average length of time in years, which 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 490.1 MCM.

5 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 and used for this study. The total net reservoir evaporation is estimated to be 1,134 mm, and the monthly rates are given in Table 8.

6 CONSTRUCTION PERIOD FLOODS

6.1 Previous Regional Flood Analysis

Instituto de Recursos Hidraulicos y Electrificacion (IRHE) performed a regional flood frequency analysis in June 1986 for the river basins west of about 79^0 west longitude in the Republic of Panama. The Río Indio basin was included in this study. This study is discussed in Attachment I.

Annual maximum instantaneous peaks for the Río Indio at Boca de Uracillo were available for six years (1980 to 1985). The station was not used in the analysis, probably because only 6 years of data were available.

The study region was divided into seven zones. For each zone, relationships between drainage areas and mean annual floods, and between ratios of flood peak and mean annual flood and return periods were developed. According to these relationships, the Río Indio was in zone III. The flood peaks corresponding to various return periods listed in this report are shown in Table 9. The flood peaks were estimated up to return period of 10,000 years.

6.2 Available Flood Data

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 Indio at Boca de Uracillo were obtained from ACP. The data are given in Table 10. The data is available from 1979 to 1998 with the values missing for a number of months in 1988 and from 1996 to 1998. The annual peaks are available for 16 years. The maximum instantaneous peak, which occurred in 1991, is about 772 m^3/s.

The IRHE estimated a 10-year flood of 1,390 m^3/s (see Table 9), which was judged to be too high and not probable for the Río Indio. A possible reason for the high estimate could be that the rainfall storms over the Río Indio basin are of relatively lower magnitudes compared to the storms over the adjacent basin because of local topography. Since the regional analysis was made based on the data from other stations and the short-period data for the Río Indio was not used, this could result in an over-estimation of flood peaks for the Río Indio. Therefore, the flood peaks estimated by the IRHE were not considered for this study.

6.3 Current Analysis

The Log-Pearson type III (LP III) distribution, recommended by the Hydrology subcommittee, United States Geological Survey (March 1982) was fitted to the annual peaks from Boca de Uracillo. A computer program developed by the United States Army Corps of Engineers, Hydrologic Engineering Center was used. The results are given in Table 9.

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 results are also given in Table 9. Exhibits 15 and 16 show the frequency curves based on the LP III and GEV distributions, respectively.

The skew coefficient of the flood peaks is about 0.6 and that of log-transferred values is about –0.5. Because of the negative skew of the logarithms, the flood peaks estimated by LP III were low compared to those obtained by using GEV. However, based on a visual judgment of Exhibits 15 and 16, both the distributions indicated reasonable goodness of fit. For a conservative estimate of the flood peaks, the values resulting from GEV were adopted.

Realizing that flood protection works could be designed for protection during the dry season, flood frequency analysis was also performed for the dry period. From a preliminary flood frequency analysis of monthly flood peaks, the dry period was determined to be the months of January through March. The flood frequency data based on GEV distribution was estimated and is given in Table 9.

6.4 Flood Hydrographs

Flood hydrographs of 20- and 50-year return periods were developed for the Río Indio at Boca de Uracillo using the following procedure:

- The historic floods (hourly discharge data) at Boca de Uracillo were reviewed and the duration of floods were determined to be about 2 days.

- The annual maximum one- and two-day flood-volumes were determined.
- Historic floods with peak discharges near to the 20- and 50-year flood peaks were selected.
- The hydrographs of these historic floods were adjusted to represent flood peaks and volumes equal to the 20- and 50-year floods.

The derived floods are shown on Exhibits 17 and 18. Table 11 shows the ordinates of the hydrographs.

6.5 Transposition to Dam Site

The above analysis provided flood frequency data and the hydrographs at Boca de Uracillo. In principle, these should have been transposed to the dam site. Transposition is generally performed by drainage ratio raised to a power varying from 0.5 to 0.75. This could increase the flood peaks by about 2 to 3 percent. This refinement was considered inappropriate because of basic data that was judged to be fair quality only. Therefore, the hydrographs at Boca de Uracillo were assumed to represent the hydrographs at the damsite.

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 Indio 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

The above sub-tasks are discussed in the subsequent paragraphs.

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 10^0 of the equator. Therefore, only rarely there are hurricanes within 10^0 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. A generalized map of the topography in the Canal Area and in the drainage basins of the Río Indio and Río Coclé del Norte is shown on Exhibit 19.

7.1.2 Methods for Estimating PMP

Two approaches were used to estimate the PMP for the Río Indio Basin:

- Study the storm patterns of major storms listed (up to 1976) in WS 1965 and NWS 1978 reports that occurred over the Canal Area. Also develop storm isohyetal patterns of major storms (since 1976) that occurred over the Río Indio and Río Coclé del Norte basins including the Canal Area. Maximize, transpose and locate the most severe storm over the Río Indio basin to produce critical flood conditions.
- Use the 24-hour, 10-mi^2 PMP developed in the NWS 1978 Report and depth-area-duration curves of WS 1965 Report and estimate basin average PMP.

For the above approaches, a relationship between elevation and mean October – December rainfall, and a mean October-December rainfall map were required to develop the storm isohyetal patterns and to transpose the storms. These maps were developed using the following procedures.

7.1.3 Relationship between Elevation and Mean October-December Rainfall

A relationship between elevation and mean October-December rainfall was developed by the NWS and presented in their 1978 Report. The relationship was based on the rainfall for the period 1941-70. Exhibit 20 shows the relationship.

For the present study, the mean October-December rainfall amounts were computed for the period 1966 through 1995 at a number of stations in the Canal Area and the drainage basins of the Río Indio and The Río Coclé del Norte. Exhibit 4 shows the locations of the stations as per Table 2. The new data points, when checked with Exhibit 20, did not show a need for revising the relationship. Therefore, the previous relationship was adopted for this study. The purpose of this relationship was to extrapolate the mean October-December rainfall at higher altitudes (where no rainfall stations exit) for preparing the October-November isohyetal map.

7.1.4 Mean October-December Rainfall Map

The mean October-December rainfall developed by the NWS for their 1978 Report was checked in two steps.

First, the latest 30-year mean October-December rainfall amounts for the period 1966-95 were calculated. The values in the Canal Area confirmed the shape of the isohyets in that zone. Therefore, the isohyetal pattern was not changed. Secondly, the 30-year mean rainfall amounts in the Río Indio and the Río Coclé del Norte were plotted. The data points were used to extend the previous map towards the west. The relationship between elevation and mean October-December rainfall and the general topography map were used to guide this extension. The derived map is shown on Exhibit 21.

7.1.5 Selection of Major Storms and Storm Analysis

In the NWS 1978 Report, a detailed discussion is presented for the criteria used for the selection of major storms. These criteria were also used for this study and are discussed below.

The 1978 Report concluded that storm rainfall of the late fall and early winter cold-outbreak would be the prototype to the PMP for the Gatun Lake watershed. Three-day rainfall amounts were added to represent the storm period. About 24 three-day storms up to 1976 were reviewed. The storms with more than 150 mm rainfall in a day or 254 mm in three days were considered as the major storms. This resulted in the selection of 10 storms. The selected storms are listed below and their isohyetal patterns are shown on Exhibits 22 to 31.

November 17-19, 1909
October 22-24, 1923
November 7-9, 1931
November 27-29, 1932
November 5-7, 1939
October 12-14, 1941
December 18-20, 1943
December 12-14, 1944
November 3-5, 1966
April 7-9, 1970

Exhibits 22 to 31 were reviewed and compared with the general topography (Exhibit 19). The centers of most of the storms were near Lake Madden. The heavy rainfall was caused by the high-elevation land masses on the east and north of the lake. Five of the storms had their centers near or at elevations varying from 150 m to 300 m near Lake Gatun. These storms were judged to be transposable to the Río Indio basin where a major part of the basin is at altitudes from 150 m to 300 m. The selected storms included:

November 7-9, 1931
November 27-29, 1932
November 5-7, 1939
December 18-20, 1943
December 12-14, 1944

These storms were carefully reviewed and the storm of November 7-9, 1931 was judged to be critical in respect of rainfall amount and aerial extent. This storm was selected and transposed to the middle of the Río Indio basin where the altitudes vary from about 150 m to 300 m. The transposition is discussed under "PMP Estimate."

For the major storms that occurred over the Río Indio and the Río Coclé del Norte since 1976, daily rainfall data were obtained for the stations in the Canal Area, Río Indio basin and Río Coclé del Norte basin. The following five storms, centered over the Río Indio and/or the Río Coclé del Norte basin, were selected.

December 4-6, 1981
December 10-12, 1981
December 4-6, 1985
January 13-15, 1996
November 27-29, 1996

The rainfall amounts associated with these storms and recorded at various stations are given on Exhibits 32 to 36. The storm of December 4-6, 1981 had heavy rainfall over the Río Indio basin with the storm center at Boca de Toabre (three-day rainfall about 362 mm) in the Río Coclé del Norte. This storm produced heaviest rainfall over the Río Indio and Río Coclé del Norte, compared to all storms reported in the NWS 1978 Report. An isohyetal map of this storm was prepared and is shown on Exhibit 37.

7.1.6 Duration of PMP

All storm isohytal maps from the NWS 1978 Report 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 Las Canones, located east of the Río Indio basin, indicated that the actual maximum rainfall duration in the major three-day storms was about 48 hours. For this reason, a duration of 48 hours was considered appropriate for the PMP.

7.1.7 PMP Estimate

The isohyetal maps of the November 7-9, 1931 (Exhibit 24, the most critical storm transposable to the Río Indio basin from NWS 1978 Report) and the storm of December 4-6, 1981 (Exhibit 37, the critical storm near the Río Indio basin) were compared. The December 4-6, 1981 storm was centered over the Río Coclé del Norte basin and it was necessary to transpose it to the Río Indio basin. The three-day rainfall at the center of the storm was about 350 mm. The storm center covered a small area.

For the November 7-9, 1931 storm, the three-day rainfall at the center of the storm was about 559 mm and the center covered a relatively large area. The next lower isohyet of 508 mm covered significantly large area (Exhibit 24). Thus, the storm of 7-9, 1931 was the most critical and, therefore, was transposed to the Río Indio basin as shown on Exhibit 38. The storm center was placed approximately at an altitude of 150 m with nearly same orientation as at the place of occurrence of the storm. This resulted in a basin average rainfall of about 439 mm.

As stated by the Weather Bureau (WB, 1965), moisture maximization of the largest storm rainfall in Panama is less meaningful in estimating PMP than in the United States because the variation in precipitation intensity from storm to storm depends mostly on the variation in the mechanism that lifts the moist air in cloud masses and less on the availability of the moisture. However, the Weather Bureau considered storm maximization in their 1965 study. Therefore, for this study the storm of November 7-9, 1931 was maximized in place as discussed below.

U.S. Weather Bureau (1965) estimated the seasonal variation of maximum 12-hour persisting dew points. This variation is given in Table 12. This table was adopted for this study in the absence of any additional data. Thus, the maximum dew point of 77^0 F was selected for the month of November. The elevation at the place of occurrence is about 150 m. Using this elevation, the 1kg/cm^2 dew point was about 25.6^0 C. The corresponding precipitable water was estimated to be 82 mm.

The dew point data during the storm was not available. From the dew point data available for the station at FAA in the Canal Area, it was determined that the dew points could vary from 21.5^0 to 20.5^0 C during the November storms. For a conservative estimate of maximization factor, a dew point of 20.5^0 C was adopted. Using an elevation of 150 m, the 1kg/cm^2 temperature was about 21.2^0 C. The precipitable water corresponding to this was about 55 mm.

The resulting maximization factor (82/55=1.5 (rounded)) was used for maximization.
The transposition factor was based on the mean October-December isohyetal map. The basin average October-December rainfall for the Río Indio basin, with the basin oriented

over the place of occurrence of the storm, was about 1,092 mm. For the Río Indio at its own location, the basin average October-December rainfall was about 1,003 mm. This resulted in a transposition ratio of 0.92 (1003/1092). Thus the maximized and transposed PMP for the Río Indio Basin was about 607 mm.

For the second approach, the 24-hour, 10 mi^2 PMP map given in the NWS 1978 Report (Exhibit 39) was used. This map gave a 24-hour 10 mi^2 PMP of about 24.7 inches over the Río Indio basin. To obtain the basin average PMP, the depth-area-duration curves (shown on Exhibit 40) given in the WS 1965 Report were used. For a duration of 48 hours and a drainage area of 147 mi^2 at the dam site, the PMP was about 711 mm. This PMP is greater than 607 mm obtained by maximization and transposition of the November 7-9, 1931 storm. Therefore, a PMP of 711mm was adopted for the Río Indio basin.

7.1.8 Depth-Duration Curve

Depth-duration data for the size of the basin was obtained from Exhibit 40 and are plotted on Exhibit 41 as percentages of 48-hour PMP. This data was available for durations greater than six hours. Because of relatively small size of the Río Indio basin, 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 Chorro. A smooth curve drawn through the points is shown on Exhibit 41.

7.1.9 Sequential Arrangement of PMP Increments:

A unit duration of one hour was estimated based on the lag time of the basin. This is discussed under the section entitled "Unit Hydrograph." The hourly PMP increments were obtained from Exhibit 41. 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 13 shows the arrangement of the increments.

7.2 Retention Losses

In a rainfall-runoff process, two types of retention losses are considered. First is the initial loss to satisfy interception and depression storage and soil moisture deficiency. The second is the uniform loss during the duration of the storm that occurs once the initial loss has been satisfied. In a single event-oriented rainfall-runoff model, these losses do not contribute to the flood.

A preferred method is to estimate these losses through calibration of a hydrologic model like HEC-1 (COE, 1981) using concurrent observed hourly rainfall and flood discharge data. An attempt was made to use this method. Hourly flood stages and rating curve (river stage and discharge relationship) were obtained for five major floods recorded on the Río Indio at Boca de Uracillo listed below and plotted on Exhibits 42 to 46.

August 15-17, 1980
October 19-21, 1987
December 4-6, 1991
August 14-15, 1992
September 12-14, 1994

Contacts were made with the Empresa de Transmision Electrica, S.A. (ETESA), and ACP to obtain the daily rainfall data and hourly time distribution of the rainfall corresponding to each of the above floods. There is only one rainfall station at Boca de Uracillo in the basin and there is no recording rainfall station. Therefore, the calibration approach was not feasible.

The COE (1981) has discussed four methods – initial loss and uniform loss rate, exponential loss rate, United States Soil Conservation Service (SCS) curve number and Holtan loss rate, to compute retention losses. The exponential loss rate and Holtan loss rate require calibration of HEC-1 model, which was not feasible because no rainfall data is available. The SCS method also requires either calibration or a detailed knowledge of the soils and land use in the basin. For this study, the 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 Boca de Uracillo 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

The uniform 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

The derivation of the unit hydrograph for the basin using historic floods was not feasible because the basin average rainfall amounts and their hyetographs could not be determined for the five floods listed under the section entitled "Retention Losses." Therefore, a synthetic unit hydrograph was developed for the Río Indio at dam site as discussed below.

There are a number of methods available to develop a synthetic unit hydrograph. MWH has tested these methods on various projects and determined that Clark's method (Clark, 1945) provides a better definition of watershed characteristics that transform rainfall to runoff. This method was used in this study.

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.

The time of concentration (Tc) is defined as the travel time of water particles from the most upstream point (time wise) in the basin to the outflow location. This time may be estimated by measuring the time between the end of effective basin average rainfall over the basin and the inflection point on the recession limb of the surface runoff hydrograph resulting from that rainfall. The storage routing coefficient (R), also called the attenuation coefficient has the dimension of time. The coefficient can be defined by the following equation when the inflow into a storage reach has ceased (Muskingum X=0):

$$R = - (Q/(dQ/dt)$$

The magnitude of R can be approximately calculated at the point of inflection of the recession limb of the observed direct runoff hydrograph. The above ratio decreases to a minimum at the point of inflection and, in theory, remains constant thereafter. Therefore, R may be estimated by dividing the ordinate of the surface runoff hydrograph at the point of inflection by the rate of change of discharge (slope) at the same point. An average value of R from a number of hydrographs is adopted.

The hydrographs of the five floods listed under "Retention Losses" were plotted. From the hourly rainfall data at Los Canones and Chorro, located in the catchment area of Lake Gatun, it was approximated that the duration of rainfall excess for each storm causing the first peak of each of the flood could be about six hours.

The time from the rise of the hydrograph to half the volume of direct runoff (after separating the base flow) was estimated for each flood hydrograph. This time represents the lag time of the basin plus half the duration of the rainfall excess (USBR, 1987). The lag time is defined as the time from the center of the rainfall excess to half the volume of the direct runoff. The half duration of three hours was subtracted from the time from the rise to half the volume. This resulted in lag times varying from about 4.0 to 6.5 hours with an average of 5.0 hours.

The SCS (1972) presented a relationship between lag time and time of concentration as:

$$\text{Lag time} = 0.6 * \text{Time of concentration}$$

Using the above relationship, the time of concentration was about 8 hours.

As an alternate approach, Kirpich formula (1940, also presented in the SCS Handbook) was used to compute the time of concentration.

$$Tc = (11.9 * L^3 / H)^{0.385}$$

in which,

L = length of main channel, miles
H = difference between the elevations at the upstream end of the main channel and that at the outlet of the basin, feet

The above formula resulted in a value of about 9 hours. For conservatism, a value of 8 hours was adopted.

To compute the value of R, out of the selected five hydrographs, three were judged to be resulting from relatively isolated bursts. The R value for each of these was computed as the ratio between the discharge ordinate at the point of inflection and the rate of change of discharge at the same point. The values ranged from 3.4 to 3.6, with an average of 3.5 hour. This value was adopted.

A time of concentration of 8 hours and routing coefficient of 3.5 hours were used in the HEC-1 computer model to compute the unit hydrograph for the basin. The time area curve was derived using the topographic maps of 1: 50,000 scale. The areas contributing to runoff at the outlet of the basin at equally spaced intervals are given in Table 14 and were used in the HEC-1 model.

7.4 Base Flow

The base flow was estimated from the selected five flood hydrographs. The flow varied from about 30 to 50 m^3/s. A flow of 50 m^3/s was adopted.

7.5 Transformation of PMP to PMF

HEC-1 computer model was used to develop the PMF 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, values of Tc and R, and the time area curve. The resulting flood hydrograph, shown on Exhibit 47, has a peak of about 4,345 m^3/s and a 5-day volume of about 242.9 MCM. The HEC-1 output is presented in Attachment II to this Appendix.

7.6 Evaluation of PMF

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

$$Q = 1.303 * C * (0.386\ A)^{0.936A^{\wedge}(-0.048)}$$

in which

Q = flood peak, m^3/s
A = drainage area, km^2

The 100-year flood at Boca de Uracillo was estimated to be 859 m^3/s (see Table 9). The maximum historic flood at Boca de Uracillo occurred in 1991 and the peak was about 772 m^3/s (see Table 10). The drainage areas at Boca de Uracillo and dam site are about 365 and 381.1 km^2, respectively. The flood peaks at the dam site were assumed to be same as that at Boca de Uracillo. The ratios between the PMF peak and these flood peaks are about 5 and 6, respectively which are reasonable for the hydrologic conditions in the basin.

The value of Creager'C was about 99. This is in the range of the values expected in similar areas. Therefore, the estimated PMF is reasonable.

7.7 PMF Routing

The PMF hydrograph was routed through the reservoir using HEC-1 computer model. The elevation-volume and spillway capacity data provided by ACP were used. The reservoir starting elevation was set at 80 meters (maximum operation level). The results indicated a maximum outflow of about 950 m^3/s with the reservoir at elevation 84 meters. HEC-1 output (Attachment II) includes the routing calculations. Exhibit 47 also shows the PMF outflow hydrograph.

8 RESERVOIR SEDIMENTATION

8.1 Data Sources

Neither suspended sediment sampling data nor water quality data were available for the Río Indio. In the vicinity of the Río Indio, the data collected by Empresa De Transmision Electrica, S.A., Departamento De Hidrometeorologia, Section Hidrologic (ETESA) were available for the following gaging stations.

- Río Coclé del Norte at Canoas
- Río Toabre at Batatilla

At Canoas, a total of 46 suspended samples with corresponding discharge measurements were collected from November 1983 to August 1998 (see Table 15). 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 (see Table 16). 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.

The 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 Choro
  Río Ciri Grande at Los Canones

Monthly suspended sediment transport data estimated by ACP for the above stations are given in Tables 17 to 22.

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

8.2 Suspended and Bed Load Material Sampling Protocol

The methods of sampling and analysis were discussed with the hydrologists of the ACP.

8.2.1 Existing Method

ETESA is using standard United States Geological Survey, Water Resources Division (USGS) methods and instruments to collect and analyze the suspended sediment samples. ACP is also using the equipment recommended by the USGS, but the method of sampling is incorrect. Suspended sediment samples are collected from the riverbanks using hand-held USGS DH-48 depth-integrating samplers. However, as per verbal communication with Ing. Jaime Massot, head of the field data collection unit, ACP has started using US D-74 depth-integrating samplers from overhead cableways. One sample is taken in the middle of the stream.

8.2.2 Recommended Method

ACP should revise its suspended sediment sampling and analysis program following the USGS guidelines. These guidelines are given in the following USGS publications.

Techniques of Water Resources Investigations of the USGS
Book 3, Chapter C2, Field Methods for Measurements of Fluvial Discharge

Book 3, Chapter C3, Computations of Fluvial Sediment Discharge
Book 5, Chapter C1, Laboratory Theory and Sediment Analysis

National Handbook of Recommended Methods for Water Data Acquisition, USGS, 1978, "Field Sampling Procedures and Methods for Analyzing Sediment Concentration and Particle Size Distribution, Chapter 3, Sediment."

During low flows, one sample in the middle of the stream is sufficient. But during medium to high flows, three samples should be collected using equal-discharge increment (EDI) or equal-width-increment (EWI), also called equal-transit-rate (ETR). Details of these methods are given in Book 3, Chapter C2 or Chapter 3 – Sediment. The three samples may be combined to form a composite sample, representative for the cross section, or each sample may be analyzed separately and results averaged. All samples with concentration greater than 200 milligram per liter should be analyzed for particle size distributions.

Efforts should be made to collect bed material samples after each major flood. The samples should be collected at ¼, ½ and ¾ of the width from either bank. An appropriate sampler should be used. A description of the bed material samplers with their limitations is given in Chapter 3, Sediment. All bed material samples should be analyzed for particle size distribution.

8.3 Previous Analysis

The ACP Report of 1983 indicated a unit yield (including bed load) of about 177.6 cubic feet per acre per year (cuft/ac/yr) from the total drainage area contributing to Lake Madden. Tatzauer (March 1990) added the sediment deposited between elevations 235 to 252 feet and also revised the suspended sediment estimates of the three rivers – Chagres, Pequeni and Boqueron. The revised estimate (including bed load) was about 203 cuft/ac/yr or about 1.4 mm/year.

For the three rivers (Gatun, Trinidad and Ciri Grande) entering Lake Gatun, the unit suspended sediment yield varied from about 0.10 to 0.27 mm/year. These values were judged to be too low.

ETESA prepared sediment rating curves for the Río Coclé del Norte and Río Toabre as shown on Exhibits 48 and 49, respectively. Best-fit curves were drawn from the data points. For the Río Toabre, for all flows greater than 107.2 m^3/s, the suspended sediment concentration was assumed to be the maximum observed concentration. Similarly, for the Río Coclé del Norte, the suspended sediment concentration was assumed to be the observed maximum concentration for all flows greater than 84.7 m^3/s. Total sediment transports at the two stream gaging stations were not available from ETESA.

8.4 Current Analysis

After a careful review of the analysis performed by the ACP, the estimate of 1.4 mm/year for the Madden basin was considered to be reasonable. This estimate was also considered applicable for the Río Indio watershed because of similar hydrological conditions.

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 (Exhibits 48 and 49) were judged to be reasonable. However, the use of maximum observed concentration as the limiting concentration for the high discharges was not a correct assumption. The limiting concentration could be significantly higher than the observed value. From the field reconnaissance and the experience of the MWH hydrologist, a limiting concentration of 10,000 mg/l was adopted for both the rivers. The suspended sediment rating curves for the two rivers were revised for the high flows. The revised curves are shown on Exhibits 50 and 51. The equations for the curves are given below.

- Río Coclé del Norte (drainage area 571 km^2)

Qw < or = 24.54 m^3/s	$Qs = 0.2464 (Qw)^{1.301}$
Qw > 24.54 and < or = 84.73 m^3/s	$Qs = 0.0114 (Qw)^{2.2613}$
Qw > 84.73 and < or = 300 m^3/s	$Qs = 0.000025 (Qw)^{3.637}$
Qw > 300 and < or = 2,000 m^3/s	$Qs = 0.0773 (Qw)^{2.228}$
Qw >2,000 m^3/s	Qs = 864.0 Qw

- Río Toabre (drainage area 786 km^2)

Qw < or = 38.97 m^3/s	Qs = 0.3455 (Qw)^$^{1.3813}$
Qw > 38.97 and < or = 107.23 m^3/s	Qs = 0.0000448 (Qw)^$^{3.825}$
Qw > 107.23 and < or = 300 m^3/s	Qs = 0.0772 (Qw)^$^{2.231}$
Qw > 300 and < or = 3,000 m^3/s	Qs = 0.2867 (Qw)^$^{2.001}$
Qw >3,000 m^3/s	Qs = 864.0 Qw

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 (see Tables 23 and 24). 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 the 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.

8.5 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 basins adjacent to the Río Indio. From the field reconnaissance, the physical and hydrologic conditions in the Río Indio basin and the adjacent basins were judged to be similar. For a conservative estimate of the reservoir sedimentation analysis, a unit yield of 1.4 mm/yr was adopted for the Río Indio basin. However, it should be realized that this yield is indicative of the current land use in the Río Indio basin. If deforestation and increased agriculture occur in future, the yield could increase significantly. Therefore, the land use conditions in the basin should be reviewed periodically to assess any increase in the sediment yield.

The drainage area at the Río Indio dam site is about 381.1 km^2. Using a yield of 1.4 mm/yr, the mean annual total sediment inflow in the reservoir would be about 533,540

m^3/yr. In their computations, the ACP used a specific weight of 1.04 mt/m^3. Therefore, the annual yield would be about 554,900 mt/yr.

A yield of 1.4 mm/yr appears to be somewhat conservative on the high side. However, it should be realized that this yield is indicative of current or near future land use conditions in the watershed. If deforestation, increased agriculture, and new settlements occur in future, the sediment yield could increase significantly. Therefore, the land use conditions should be monitored periodically to assess any potential increase in sediment production

8.5.1 Specific Weight of Sediment

Using the procedure given in Design of Small Dams (United States Bureau of Reclamation, Third Edition, 1987), the specific weight of the fresh deposit was about 75 pounds per cubic feet (1.2 mt/m^3) as follows:

$$W = WcPc + WmPm + WsPs$$

in which

W = unit weight of fresh deposit, pounds per cubic feet
Pc, Pm, Ps = percentages of clay, silt and sand, respectively of inflowing sediment
Wc, Wm, Ws = unit weights for clay, silt and sand

A normally moderate to considerable reservoir draw down (type 2) was assumed. The values of Wc, Wm and Ws, from the Design of Small Dams, were about 35, 71 and 97 pounds per cubic feet, respectively. The percentages of clay, silt and sand were assumed to be about 15, 45, and 40, respectively. Therefore, the annual yield would be about 640,300 mt/yr.

To determine the density of deposits after a period of reservoir operation, the following equation given in the Design of Small Dams was used.

$$Wt = Wi + 0.4343K\ ((T/(T-1))(LnT) - 1)$$

in which

Wt = average weight after T years of operation
Wi = initial weight

K = a constant based on type of reservoir operation and sediment sizes and

$$K = KcPc + KmPm + KsPs$$

For type 2 operation, the values of Kc, Km and Ks were 8.4, 1.8 and 0.0, respectively. This resulted in a value of 2.07 for K. Using this value in the above equation and an initial weight of 75 pounds per cubic feet, the specific weights after 50 and 100 years of operation were about 77.7 and 78.3 pounds per cubic feet (1.24 and 1.25 mt/m^3), respectively. These values were used to compute the volumes of deposit after 50 and 100 years.

8.6 Trap Efficiency

The maximum normal pool elevation for the Río Indio reservoir is planned to be about 80 meters. The reservoir volume at this elevation is about 1,577 MCM. Long-term mean annual flow is about 25.8 m^3/s. Thus, the capacity-inflow ratio is about 1.94.

To estimate the trap efficiency of the reservoir, Brune's method (USBR, 1987) was used. From the sediment deposited along the banks of the Río Indio and the general soils in the watershed, it was judged that the sediment entering the reservoir would be of medium sizes. Therefore, Brune's curve for medium sediment sizes was used. For a capacity-inflow ratio of 1.94, the trap efficiency would be about 99 percent. This resulted in a mean annual deposit of about 549,400 metric tons.

8.7 Analysis of Storage Depletion

Depletion in the reservoir storage was estimated using the methods developed by the United States Bureau of Reclamation (USBR, 1987). An annual sediment deposit of 549,300 metric tons was used.

The reservoir operation was assumed as type 2 (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. 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-years 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.

Sediment distribution in the reservoir was determined using a computer program obtained from USBR. Both the area-increment and empirical area-reduction methods were tested. The empirical area-reduction method could not be used because the data and information about the Río Indio reservoir did not fit any of the sediment distribution design curves developed by USBR. Therefore, area-increment method was used.

As per current investigations of the project, the maximum and minimum operating reservoir elevations would be 80 and 40 meters, respectively. The incoming sediment will partly deposit in the dead storage (below elevation of 40 meters) and partly in the operating volume (live storage, above elevation of 40 meters). Using these operating limits, the operating volume and the loss in the original volume after a given period of operation are given in Table 25. The table shows that reservoir sedimentation will not be a major problem for the project. Even after 100-year of reservoir operation, loss in live storage capacity would only be about 2 percent.

9 PROJECT IMPACT ON WATER QUALITY

9.1 Impact During Construction Period

Sediment production (erosion) will increase due to construction activities such as excavation, construction of dirt roads, construction of temporary buildings for construction staff, etc. The increased erosion, if not properly controlled, will make river water turbid and unsuitable for drinking or other uses.

9.1.1 General Concepts

The erosion process is influenced primarily by climate, topography, soils and vegetative cover. The climatic factors influencing the erosion include the frequency, intensity and duration of rainfall and temperature extremes. These factors will be unchanged due to construction activities. However, the size, shape and slope characteristics of the disturbed area will be changed by construction activities and will influence the erosion. .

Properties determining the erodibility of a soil are texture, structure, organic matter content and permeability. Soils containing high percentages of fine sands and silt are normally the most erodible. The erodibility decreases as the clay and organic matter contents increase. The soil horizon exposed at a particular location during construction will determine the severity of erosion.

A general procedure is to estimate the soil erosion rates using the universal soil loss equation or modified universal soil loss equation. The various factors in the equation include: rainfall intensity, soil-erodibility, length and steepness of slopes, cropping management, and erosion control practice. The equation should be used to estimate potential erosion rates from disturbed lands during construction.

Construction sites or borrow areas will be in their most vulnerable bare conditions for only part of a year, when erosion potential will be high. In the Río Indio basin, the general type storms occur during October through December, but local thunderstorm with intense rainfall could occur any time during the year. However, during the dry months of February through April, the erosion may be minimum.

9.1.2 Mitigation Measures During Construction

A water quality management plan (WQMP) should be prepared by the contractor and submitted for approval. The plan should ensure that unclean water or foreign material would not enter any surface waters or water courses in the area.

Mitigation measures for erosion and sedimentation will be in full compliance with the local standards and requirements. The contractor should ensure that the bare slopes are exposed for a minimum period and are protected from the erosive forces of wind, rain and runoff as soon as possible. The eroded soil will be captured on-site and not allowed to enter the water bodies. Major land clearing and grading should be scheduled during season of relatively low runoff potential. A combination of both vegetative and structural measures should be employed.

The plan for controlling sediment should apply to all aspects of construction activities including, but not limited to: clearing, operation, all excavation spoil area, drilling and grouting, fills, and roadwork. Sediment control methods such as silt fences, sediment barriers, sediment ponds, ditches, interceptor dikes, perimeter dikes, leaving of buffer zones, graveling after grading, and other such devices or actions will be constructed and maintained, or performed, as necessary to comply with the local requirements.

The contractor should ensure that the following principles guide the construction activities and these principles are integrated with the mitigation measures to prevent off-site sedimentation:

- <u>Fit the Activities to Existing Site Conditions</u>
  Construction activities should follow the existing topography, especially the cutting of borrow areas and road grading should follow the natural contours. Steep slopes, areas subjected to flooding and highly erodible soils should be avoided to the extent feasible.

- <u>Minimize the Extent and Duration of Exposure</u>
  The construction activities should be scheduled such that the exposed areas are stabilized as quickly as possible.

- Protect Disturbed Areas from Runoff

  Measures should be taken to intercept runoff and divert it away from cut-and fill slopes or other disturbed area. The selected measures should be installed before clearing and grading.

- Stabilize Disturbed Areas

  After the land is disturbed, temporary or permanent vegetation, mulches or other protective measures should be implemented as quickly as possible.

- Keep Runoff Velocities Low

  This should be achieved by conveying the storm water runoff away from the steep slopes, preserving natural vegetation where possible and mulching and vegetating exposed areas immediately.

- Retain Sediment On-Site

  Some erosion would occur in spite of well planned mitigation measures. These sediments should be retained on-site using sediment basins, sediment barriers and related structures. If on-site sedimentation is required, the sediment traps or basins should be constructed prior to land disturbing activities.

- Do Not Encroach Upon Water Courses

  Where feasible, the project related buildings, access roads and borrow areas should not be constructed in flood-prone areas. If unavoidable, temporary bridges and culverts should be employed to permit passage of selected peak discharges.

9.2 Potential Long-Term Impact

The project operation may have three longer-term impacts on water quality. First, normal daily fluctuations in the reservoir may cause reservoir bank erosion or landslides. Second, the flow released from the reservoir may cause bank and bed erosion in the Río Indio, and third, the water released through the low level outlets for in-stream flow requirements may be cooler than normal.

Based on an inspection of the reservoir area, the reservoir rim is not expected to exhibit much erosion except in very limited areas. It is not expected that this local erosion will have any significant impact. The most likely areas will be in the vicinity of the tunnel portal, however, as a part of construction, all disturbed areas will be stabilized and restored to natural conditions as far as feasible.

The stability of the river channel downstream from the dam is addressed in the next section. The low-level outlet and spillway, to the extent possible, will be operated to control the maximum hourly increase or decrease in flow so that it would be comparable with natural conditions. This would check the erosion of river banks and bed.

10 STABILITY OF THE ALLUVIAL CHANNELS

The assessment of the Río Indio channel stability consisted of the estimation of flood conditions without and with the project, a determination of the hydraulic and bed material characteristics of the channel downstream from the dam, and an evaluation of channel stability.

10.1 Flood Regime Downstream from the Dam

Pre-project flood conditions are discussed in Section 6. The pre-project flood peaks and return intervals are presented in Table 9.

Flood hydrographs were developed for the 2-, 5-, 10-, 25-, 50- and 100-year return periods using flood peak, and 1-day and 2-day flood volumes of the selected return period. The hydrographs were shaped after the historic flood of December 4-6, 1991, and adjusted for the flood peak and volumes. The six flood hydrographs were routed through Indio reservoir and the resulting peaks are given in Table 26.

10.2 Hydraulic Characteristics

The ACP surveyed six cross sections downstream from the proposed dam site. The cross sections are plotted on Exhibits 52 to 57. The cross sections were located about 400, 1,500, 1,900, 3,000, 5,300 and 7,800 meters downstream from the dam axis.

A review of the cross sections indicated that the river channel shapes (within the banks) were nearly same for all cross sections. Cross sections number 3, 4 and 7 indicated relatively steep slopes on the right bank and mildly sloping left bank. In case of cross section numbers 2, 5 and 6, the left banks were relatively steep and the right banks were mildly sloping.

The HEC-2 computer model was used to determine the hydraulic properties of all cross sections. The properties included: area, depth, top width and channel mean velocity. Uniform discharge rates of 50,100, 200, 300, 500, 700, 1000, 1200 and 1500 m^3/s were used.

Because of one bank being mildly sloping or relatively steep, the hydraulic properties were not significantly different from one cross section to another. This suggested that a representative cross section can be used for the whole 7,600 meters long reach.

The HEC-2 model was used to determine the properties of the representative cross section corresponding to uniform discharge rates of 50, 100, 200, 300, 500, 700, 1000, 1200 and 1500 m/s. For other discharges shown in Table 26 (flood peaks with and without project), the properties were interpolated. These hydraulic properties are shown in Table 27

10.3 Bed Material Characterisitcs

ACP took six bed material samples at the locations of the surveyed cross sections and analyzed the samples for particle size distribution. Exhibit 59 shows the particle size distribution curves and an average, representative curve is shown on Exhibit 60. Based on this curve, the bed material characteristics are given in Table 28.

10.4 River Channel Stability

The sediment transport capability of a river depends upon flood flows. A natural flowing river transporting sediment is usually in a state of regime or quasi-equilibrium with no long-term trend toward aggradation or degradation. When a dam is constructed on the river, three potential effects could be experienced – downstream effects due to changed time distribution of flow, reduction in sediment load, and reduced competence to transport sediment. These effects are discussed below.

Downstream effects due to changed time distribution of flow are generally manifested as degradation at the mouths of the tributaries. At the time of flood in the tributary, the water level in the main river could be much lower than that under without/dam conditions. This would cause relatively steep water surface slope at the mouth of the tributary providing the potential for scour at the mouth.

A reduction in the sediment load occurs as the sediment is trapped in a reservoir. The downstream effects are generally degradation of the channel and banks as the sediment-free reservoir releases pick up sediment from the bed. The degradation continues until a stable, gravel-armored bed is formed or until the slope is reduced to a value that prevents further sediment removal from the bed. However, if a reservoir is designed to pass sediment through low-level outlets, most of the sediment passes to the downstream channel and there is no degradation downstream. The trap efficiency of this type of reservoir is quite low. For reservoirs with no low-level sediment excluders, the trap efficiency is high. The released water is sediment-free and is capable of picking up bed material.

The reduced transport capability is due to the storage effect of a reservoir, *i.e.*, flood flows are significantly reduced. A river may no longer be able to move the bed loads carried by its tributaries. This could cause extensive aggradation at the mouths of some tributaries. In some cases, the main channel may not show any degradation.

10.4.1 Computation of Degradation Potential

The techniques for computing degradation below a dam vary considerably depending on the size of sediments in the riverbed and banks, the magnitudes of release discharges at the dam, and sophistication desired in the results. Sophisticated mathematical computer models have become available for computing degradation. Such models simulate the behavior of an alluvial channel by combining a steady-state backwater computation for defining channel hydraulics with a sediment transport model. The models need detailed hydraulic properties of the river channel, sediment characteristics of river bed and suspended sediment in the releases, and flow pattern of the releases. These data are not available for the Río Indio and Río Coclé del Norte. Therefore, a mathematical computer model was not used.

In Design of Small Dams, the United States Bureau of Reclamation (the Bureau) recommends two approaches, each specific to the type of bed material composing the downstream channel. If the streambed is composed of sufficient quantity of large and coarse material that cannot be transported by normal river discharges, an armor layer will develop. The smaller particles in the riverbed are picked up and transported further downstream. Large particles that cannot be transported by the flood releases, remain on the river bed and gradually form an armor layer that stops further degradation below a dam. The armor layer is formed for a certain magnitude of flood. If this flood release is exceeded, the layer is disturbed and a new layer is formed. If the conditions required to for an armoring are not present, then a second approach can be used. It is used when the stream bed is composed of fine transportable (usually sand and small gravels) material and the depth of this material is greater than expected depth of degradation, stable channel slope (or limiting slope) method is used. The method consists of computing the limiting slope, estimating the volume of expected degradation and then determining a three-slope channel profile that fits these values. Since the Río Indio has sufficient coarse material, this later approach was not used.

As discussed above, this method is applicable if there is large or coarse material in the channel bottom that cannot be transported by normal releases and there is enough of this material to develop an armor layer. In the armoring process, transportable material is sorted out, and vertical degradation proceeds at a progressively slower rate until the armor is deep enough to control further degradation. Usually, an armoring layer should be expected below a dam if 10 percent or more of the bed material is larger than the armoring size corresponding to the flow magnitude (the Bureau).

The armoring layer is assumed to form as follows:

$$Y_A = Y - Y_D$$

in which

Y_A = thickness of the armoring layer,
Y = depth from original stream bed to bottom of the armoring layer
Y_D = depth from the original streambed to top of armoring layer (depth of degradation)

By definition

$$Y_A = p * Y$$

where 'p' is decimal percentage of material larger than the armoring size.

By combining the above two relationships, the depth of degradation can be computed as:

$$Y_D = Y_A\,((1/p) - 1)$$

As per Bureau, the thickness of an armoring layer (Y_A) is usually three times the armoring particle diameter or 0.15 meters, whichever is smaller. Therefore, if the armoring size and the percentage of streambed material larger than that size are available, the depth of degradation (Y_D) can be computed.

10.4.2 Required Armor Sizes

The sediment particle sizes required for armoring can be computed by several methods, and each is regarded as a check on the others. Each method indicates a different size and, therefore, experience of the investigator and judgment are required to select the most appropriate size. The basic data required to compute particle size includes:

1. Samples of streambed material in the reach selected for degradation to a depth anticipated to the scour zone.
2. A discharge rate that will cause degradation equivalent to long-term degradation below a dam, defined as dominant discharge and is equivalent to mean annual flood (approximately a flood of 2-year return period).
3. Average hydraulic properties of the channel reach – width, depth, velocity and gradient.

The Bureau has recommended the following methods:

Meyer-Peter, Muller (Meyer 1948, Sheppard 1960)
Competent bottom velocity (Mavis 1948)
Shield diagram (Pemberton and Lara 1982, ASCE 1975)
Yang incipient motion (Yang 1973)
Critical tractive force (Bureau 1952)

Meyer-Peter, Muller:

$$D = (S * d) / (K * ((N_s/(D_{90})^{1/6})^{1.5})$$

in which

$K = 0.19$ English units (0.058 SI units)
N_s = Manning's roughness coefficient for streambed
D_{90} = bed material diameter in mm, 90 percent material finer than the diameter
D = armoring size, mm
S = stream gradient, ft/ft or m/m
d = channel depth, ft or m

Competent Bottom Velocity:

$V_B = 0.7\ V_M$
$D = 3.84\ (V_B)^2$ (English units)
$D = 41.6\ (V_B)^2$ (SI units)

in which
V_M = mean velocity, ft/sec or m/sec
V_B = competent bottom velocity, ft/sec or m/sec
D = armoring size, mm

Shield's Method:

The following method is used for material >1.0 mm and Reynolds's number R* >500

$Tc / ((Ws - Ww)*D) = 0.06$

or $Tc/0.06 = (Ws-Ww)*D$

or $D = Tc/((0.06 (Ws-Ww))$

in which
Tc = Ww * d * S, critical shear stress, lb/ft^2 or gm/m^2
Ws = unit weight (mass) of particle, 165 lb/ft^3 or 2.65 mt/m^3
Ww = unit weight (mass) of water, 62.4 lb/ft^3 or 1.0 mt/m^3
d = depth of water, ft or m
S = stream gradient, ft/ft or m/m
D = armoring size, mm

Yang Incipient Motion

$Vcr / w = 2.05$
$w = 6.01\ D^{1/2}$ (English units)
$w = 3.32\ D^{1/2}$ (SI units)
$D = 0.00659\ (Vcr)^2$ (English units)
$D = 0.0216\ (Vcr)^2$ (SI units)

in which

Vcr = critical average velocity, ft/sec or m/sec

w = terminal fall velocity, ft/sec or m/sec

D = armoring size, mm

Critical Tractive Force:

$$t.f. = Ww * d * S$$

in which

t.f. = tractive force, lb/ft^2 or gm/m^2

Ww = unit weight (mass) of water, 62.4 lb/ft^3 or 1.0 mt/m^3

d = water depth, ft or m

S = stream gradient, ft/ft or m/m

After computing the tractive force, Figure 4 given by Pemberton and Lara (1984), should be used to find armoring size in mm. Usually the recommended set of "curves for clear water in coarse non-cohesive material" gives the lower size limit of the non-transportable material corresponding to a critical tractive force. Because a number of curves are provided and sufficient field data were not available to select an appropriate curve, this method was not considered.

Four methods (Meyer-Peter, Muller, Competent Bottom Velocity, Shield and Yang Incipient Motion) were applied to compute the armoring size (the particle size that would not be removed or eroded from the bed under given hydraulic conditions). The armoring sizes derived by using the Competent Bottom Velocity and Yang methods were judged to be too large and were not considered representative. The estimated armoring sizes using Meyer-Pete, Muller and Shield methods are given in Table 29 for both the pre- and post-project flooding conditions.

10.4.3 Potential for Degradation and Aggradation

A comparison of pre-project armoring sizes with the median diameter of 16.5 mm (Table 28), indicates the potential for degradation in the Río Indio. Under pre-project conditions, the required armor size is always greater that the median diameter of the available bed-load material and degradation will occur. Under post-project conditions, the required armor size is less than the median diameter of the available bed load material and degradation will not occur. Aggradation will occur at the mouths of the tributaries downstream from the dam because the reduced flood peaks will not be able to transport the bed load deposited by the tributaries.

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TABLES

Table 1

MEAN MONTHLY RAINFALL
BOCA DE URACILLO

Month	Rainfall (mm)
January	135
February	75
March	84
April	181
May	361
June	318
July	287
August	306
September	307
October	409
November	368
December	247
Annual	**3,078 mm**

Table 2

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

Rainfall Station

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

Table 3

STREAM GAGING STATIONS WITH PERIOD OF RECORD

River	Station	Latitude (N)	Longitude (W)	Elevation (m)	D.A. (sq. km.)	Period of Record
Indio	Boca de Uracillo	08-58	80-11	8	365	Aug 79-May 81,Aug 81-Apr 82 Jul 82-Feb 84,Apr 84-May 86 Sep 86-Mar 87,Jul 87-Apr 88 Nov 88-Jan 89,Apr 89-May 90 Dec 90-Dec 94,Mar 95-Jul 95 Sep 95-Dec 95,Feb, May, Jun, 96 Sep 96-Feb 97,Jun 97 Jan 98-May98
Toabre	Batatilla	08-55	80-30	20	788	Jul 58-Apr 59,Aug 59-Jul 60 May 61-Aug 61,Jan 62-Mar 62 May 62-Sep 64,Jun 69-Dec 69 May, Jul, Aug 70 Nov 70-Jan 71,Apr 71-Apr 72 Jun 72-Feb 73,Apr 73-Sep 73 Dec 73-Mar 74,Jul 74-Jan 75 Apr, May 75,Oct 75-Apr 79 Jun, Jul, Aug 79 Oct 79-Nov 80,May 81-Jul 88 Nov 88-Sep 89,Dec 89-Jun 90 Dec 90-Mar 91,Oct 91-Jun 95 Aug 95-Jan 96,May 96-Jan 98 Apr 98
Cocle del Norte	El Torno	08-56	80-33	15	672	Jul 58-Mar 59,Jun 59-Jul 60 May 61-Dec 69,Jun 70-Oct 70 May 71-Jun 73,Oct 73, Jan 74 May 74-Aug 76,Nov 76-Apr 79 Dec 79-Mar 80,May 80-Jul 81 Nov 81-Sep 83,Feb 84-Jun86
Ciri Grande	Los Canones	08-57	80-04	87	186	Jan 48-Dec 58 Aug 78-Dec 83,Mar 84-Oct 84 Dec 84,Mar 85-Oct 85 Jan 86-Oct 86,Jan87-Dec98
Trinidad	El Choro	08-59	79-59	43	172	Jan 48-Jul 66,Oct 66-Jun 69 Oct 69-Jul 85, Sep 85-Dec98
Cocle del Norte	Canoas	08-53	80-34	20	571	Oct 83-Nov 85,Jan 86-Jun 88 Nov 88-Sep 89,Dec 89-Jun 90 Dec 90-Mar 91, Jul 91-Nov 93 Apr 94-Dec 95,Mar 96-May 96 Jul 96,Dec 96-Jan 97 Apr 97-Jul 98
Rio Indio	Limon	08-56	80-11	10	376	March 58- Oct 80

MWH / TAMS

Table 4-1

MONTHLY MEAN DISCHARGE (cubic feet per second)
RIO INDIO AT BOCA DE URACILLO

Drainage Area: 141 mi^2

YEAR	JAN	FEB	MAR	APRIL	MAY	JUNE	JULY	AUG	SEPT	OCT	NOV	DEC	AVE
1948	411	163	117	87	185	262	967	1127	1072	995	1982	556	660
1949	233	141	92	84	251	1361	960	1202	1684	1595	2759	2186	1046
1950	385	217	131	95	656	1204	1254	1821	1214	1649	2135	2143	1075
1951	687	380	217	144	674	823	765	831	1410	1195	1792	1040	830
1952	458	219	117	103	385	977	758	803	1348	1924	1212	1713	835
1953	1282	500	267	190	796	715	694	577	712	2016	2019	1112	907
1954	564	267	168	136	636	671	1580	1197	1625	1386	2595	1383	1017
1955	1482	469	233	185	367	1393	992	1630	1987	1718	2617	1524	1216
1956	1541	481	280	259	970	1400	1316	985	1571	2346	1667	970	1149
1967	388	201	131	95	398	401	396	773	844	1921	1264	1020	653
1958	641	494	265	190	587	702	965	1296	1257	1642	1334	709	840
1959	319	195	139	122	185	2830	3161	480	563	1696	1174	1486	1029
1960	713	297	308	381	900	953	943	971	855	1361	1891	3146	1060
1961	534	278	158	162	282	706	624	692	1105	1707	1504	1216	747
1962	447	241	143	132	196	297	407	1014	873	1171	1216	926	589
1963	374	252	139	282	610	731	880	1139	1153	1638	1834	613	804
1964	308	166	109	124	451	1455	1435	1462	1755	1888	2215	688	1005
1965	530	264	151	89	139	275	245	556	443	912	1129	1199	494
1966	425	211	143	151	866	1185	957	853	623	1792	2391	1837	953
1967	595	293	155	241	724	1693	1279	1404	1868	2154	1442	681	1044
1968	323	234	151	136	367	1028	774	1087	1139	1912	1847	897	825
1969	385	226	116	177	360	731	1879	2981	1746	1490	1624	989	1059
1970	813	363	308	326	964	649	727	1569	1331	1984	1566	3238	1153
1971	1411	505	286	215	844	1251	1066	1551	1707	1892	2239	656	1135
1972	447	297	184	491	418	756	336	483	1105	1115	1169	480	607
1973	276	173	84	92	328	1320	1366	1098	1946	2164	2563	1265	1056
1974	574	354	258	155	299	642	781	901	1059	2957	1696	897	881
1975	400	236	160	101	268	527	774	1473	2045	2259	3441	1837	1127
1976	720	385	230	202	469	433	226	277	809	1885	1310	523	622
1977	321	202	127	101	274	407	389	869	999	1857	1386	766	642
1978	377	249	166	912	795	1112	1073	1447	1516	1730	1475	866	977
1979	484	375	341	385	615	990	980	1232	1317	1240	915	879	813
1980	819	295	139	94	477	646	710	1667	837	1271	1144	982	757
1981	710	459	417	1021	1045	1169	1405	1328	1014	1695	2069	2670	1250
1982	657	204	135	185	322	749	953	611	982	1787	922	313	651
1983	206	107	64	54	551	823	583	653	1593	1243	1056	1356	691
1984	554	351	228	125	597	968	1088	1713	1405	1575	1331	452	866
1985	303	189	146	94	296	1066	611	1377	1328	1236	1430	936	751
1986	332	173	106	565	671	1049	784	664	971	1981	2165	530	833
1987	261	176	102	177	505	809	908	1098	1413	2546	1130	625	812
1988	265	179	101	92	417	833	890	1169	1176	1858	1444	780	767
1989	445	246	170	99	466	653	982	1261	1222	1285	1900	1059	816
1990	537	240	164	112	872	632	809	842	1465	2110	1420	2190	949
1991	363	204	248	110	493	703	518	643	1364	1724	1177	1558	759
1992	323	120	130	194	943	983	946	1629	1521	1326	1114	684	826
1993	414	236	192	307	393	1055	771	597	1248	1740	2219	1169	862
1994	427	248	202	219	474	742	551	427	899	1272	1304	471	603
1995	290	154	102	142	812	1269	1113	1018	1292	1087	1477	1043	817
1996	2026	943	481	260	740	1472	1622	1997	1448	2517	1963	1820	1441
1997	399	249	164	120	199	242	283	232	563	733	887	390	372
1998	180	149	101	139	480	447	784	642	890	1390	819	1215	603
1999	627	343	260	296	830	1144	805	1802	2237	1186	1704	2276	1126
AVE	557	281	183	211	535	910	924	1099	1261	1668	1655	1192	873

MWH / TAMS

Table 4-2

MONTHLY MEAN DISCHARGE (cubic meters per second)
RIO INDIO AT BOCA DE URACILLO

Drainage Area: 365 km^2

YEAR	JAN	FEB	MAR	APRIL	MAY	JUNE	JULY	AUG	SEPT	OCT	NOV	DEC	AVE
1948	11.7	4.6	3.3	2.5	5.2	7.4	27.4	31.9	30.4	28.2	56.1	15.7	18.7
1949	6.6	4.0	2.6	2.4	7.1	38.5	27.2	34.0	47.7	45.2	78.1	61.9	29.6
1950	10.9	6.1	3.7	2.7	18.6	34.1	35.5	51.6	34.4	46.7	60.5	60.7	30.5
1951	19.4	10.8	6.1	4.1	19.1	23.3	21.7	23.5	39.9	33.8	50.7	29.4	23.5
1952	13.0	6.2	3.3	2.9	10.9	27.7	21.5	22.7	38.2	54.5	34.3	48.5	23.6
1953	36.3	14.1	7.6	5.4	22.5	20.2	19.7	16.3	20.2	57.1	57.2	31.5	25.7
1954	16.0	7.6	4.8	3.9	18.0	19.0	44.8	33.9	46.0	39.2	73.5	39.2	28.8
1955	42.0	13.3	6.6	5.2	10.4	39.4	28.1	46.1	56.3	48.6	74.1	43.2	34.4
1956	43.6	13.6	7.9	7.3	27.5	39.7	37.3	27.9	44.5	66.4	47.2	27.5	32.5
1967	11.0	5.7	3.7	2.7	11.3	11.4	11.2	21.9	23.9	54.4	35.8	28.9	18.5
1958	18.1	14.0	7.5	5.4	16.6	19.9	27.3	36.7	35.6	46.5	37.8	20.1	23.8
1959	9.0	5.5	3.9	3.5	5.2	80.1	89.5	13.6	15.9	48.0	33.2	42.1	29.1
1960	20.2	8.4	8.7	10.8	25.5	27.0	26.7	27.5	24.2	38.5	53.5	89.1	30.0
1961	15.1	7.9	4.5	4.6	8.0	20.0	17.7	19.6	31.3	48.3	42.6	34.4	21.2
1962	12.7	6.8	4.1	3.7	5.6	8.4	11.5	28.7	24.7	33.2	34.4	26.2	16.7
1963	10.6	7.1	3.9	8.0	17.3	20.7	24.9	32.3	32.7	46.4	51.9	17.4	22.8
1964	8.7	4.7	3.1	3.5	12.8	41.2	40.6	41.4	49.7	53.5	62.7	19.5	28.4
1965	15.0	7.5	4.3	2.5	3.9	7.8	6.9	15.7	12.6	25.8	32.0	33.9	14.0
1966	12.0	6.0	4.1	4.3	24.5	33.6	27.1	24.2	17.6	50.8	67.7	52.0	27.0
1967	16.9	8.3	4.4	6.8	20.5	47.9	36.2	39.7	52.9	61.0	40.8	19.3	29.6
1968	9.1	6.6	4.3	3.8	10.4	29.1	21.9	30.8	32.3	54.1	52.3	25.4	23.3
1969	10.9	6.4	3.3	5.0	10.2	20.7	53.2	84.4	49.4	42.2	46.0	28.0	30.0
1970	23.0	10.3	8.7	9.2	27.3	18.4	20.6	44.4	37.7	56.2	44.3	91.7	32.7
1971	39.9	14.3	8.1	6.1	23.9	35.4	30.2	43.9	48.3	53.6	63.4	18.6	32.1
1972	12.7	8.4	5.2	13.9	11.8	21.4	9.5	13.7	31.3	31.6	33.1	13.6	17.2
1973	7.8	4.9	2.4	2.6	9.3	37.4	38.7	31.1	55.1	61.3	72.6	35.8	29.9
1974	16.2	10.0	7.3	4.4	8.5	18.2	22.1	25.5	30.0	83.7	48.0	25.4	24.9
1975	11.3	6.7	4.5	2.9	7.6	14.9	21.9	41.7	57.9	64.0	97.4	52.0	31.9
1976	20.4	10.9	6.5	5.7	13.3	12.2	6.4	7.8	22.9	53.4	37.1	14.8	17.6
1977	9.1	5.7	3.6	2.8	7.8	11.5	11.0	24.6	28.3	52.6	39.3	21.7	18.2
1978	10.7	7.1	4.7	25.8	22.5	31.5	30.4	41.0	42.9	49.0	41.8	24.5	27.7
1979	13.7	10.6	9.7	10.9	17.4	28.0	27.7	34.9	37.3	35.1	25.9	24.9	23.0
1980	23.2	8.3	3.9	2.7	13.5	18.3	20.1	47.2	23.7	36.0	32.4	27.8	21.4
1981	20.1	13.0	11.8	28.9	29.6	33.1	39.8	37.6	28.7	48.0	58.6	75.6	35.4
1982	18.6	5.8	3.8	5.2	9.1	21.2	27.0	17.3	27.8	50.6	26.1	8.8	18.4
1983	5.8	3.0	1.8	1.5	15.6	23.3	16.5	18.5	45.1	35.2	29.9	38.4	19.6
1984	15.7	9.9	6.4	3.5	16.9	27.4	30.8	48.5	39.8	44.6	37.7	12.8	24.5
1985	8.6	5.3	4.1	2.6	8.4	30.2	17.3	39.0	37.6	35.0	40.5	26.5	21.3
1986	9.4	4.9	3.0	16.0	19.0	29.7	22.2	18.8	27.5	56.1	61.3	15.0	23.6
1987	7.4	5.0	2.9	5.0	14.3	22.9	25.7	31.1	40.0	72.1	32.0	17.7	23.0
1988	7.5	5.1	2.9	2.6	11.8	23.6	25.2	33.1	33.3	52.6	40.9	22.1	21.7
1989	12.6	7.0	4.8	2.8	13.2	18.5	27.8	35.7	34.6	36.4	53.8	30.0	23.1
1990	15.2	6.8	4.6	3.2	24.7	17.9	22.9	23.8	41.5	59.7	40.2	62.0	26.9
1991	10.3	5.8	7.0	3.1	14.0	19.9	14.7	18.2	38.6	48.8	33.3	44.1	21.5
1992	9.1	3.4	3.7	5.5	26.7	27.8	26.8	46.1	43.1	37.5	31.5	19.4	23.4
1993	11.7	6.7	5.4	8.7	11.1	29.9	21.8	16.9	35.3	49.3	62.8	33.1	24.4
1994	12.1	7.0	5.7	6.2	13.4	21.0	15.6	12.1	25.5	36.0	36.9	13.3	17.1
1995	8.2	4.4	2.9	4.0	23.0	35.9	31.5	28.8	36.6	30.8	41.8	29.5	23.1
1996	57.4	26.7	13.6	7.4	21.0	41.7	45.9	56.5	41.0	71.3	55.6	51.5	40.8
1997	11.3	7.1	4.6	3.4	5.6	6.9	8.0	6.6	15.9	20.8	25.1	11.0	10.5
1998	5.1	4.2	2.9	3.9	13.6	12.7	22.2	18.2	25.2	39.4	23.2	34.4	17.1
1999	17.8	9.7	7.4	8.4	23.5	32.4	22.8	51.0	63.3	33.6	48.3	64.4	31.9
AVE	15.8	7.9	5.2	6.0	15.2	25.8	26.2	31.1	35.7	47.2	46.9	33.7	24.7

Table 5-1

MONTHLY MEAN DISCHARGE (cubic feet per second)
RIO INDIO AT THE DAMSITE

Drainage Area: 147 mi^2

YEAR	JAN	FEB	MAR	APRIL	MAY	JUNE	JULY	AUG	SEPT	OCT	NOV	DEC	AVE
1948	430	170	122	90	193	274	1010	1177	1120	1039	2070	581	689
1949	243	148	96.2	87.5	262	1421	1002	1255	1758	1666	2881	2283	1092
1950	402	226	136	99	685	1258	1310	1901	1268	1722	2230	2237	1123
1951	717	397	226	150	704	860	799	868	1473	1247	1871	1086	866
1952	478	229	122	108	402	1020	791	839	1408	2008	1265	1789	872
1953	1338	522	279	198	831	746	725	602	743	2105	2108	1161	947
1954	589	279	176	142	664	701	1650	1250	1696	1447	2710	1444	1062
1955	1547	489	243	193	383	1454	1036	1702	2075	1794	2732	1591	1270
1956	1609	503	293	271	1012	1462	1374	1028	1640	2450	1740	1012	1200
1967	405	210	136	99	416	419	413	807	881	2006	1320	1065	681
1958	669	516	276	198	613	733	1007	1354	1312	1714	1392	741	877
1959	333	204	145	128	193	2955	3300	501	588	1771	1226	1552	1075
1960	744	310	321	398	940	995	984	1014	892	1421	1974	3285	1107
1961	558	291	165	169	295	737	651	722	1153	1782	1570	1270	780
1962	467	252	150	138	205	310	425	1058	911	1223	1270	966	615
1963	391	264	146	295	636	763	919	1190	1204	1710	1914	640	839
1964	322	173	113	130	471	1520	1498	1527	1832	1972	2313	719	1049
1965	554	275	157	93	146	287	256	580	463	952	1179	1252	516
1966	444	221	150	157	904	1237	1000	891	650	1872	2497	1918	995
1967	621	306	161	252	756	1768	1335	1466	1950	2249	1505	711	1090
1968	337	244	157	142	383	1073	808	1135	1190	1997	1929	937	861
1969	402	236	122	185	375	763	1962	3113	1823	1556	1696	1033	1105
1970	848	379	322	341	1007	678	759	1639	1390	2071	1635	3381	1204
1971	1473	527	298	225	882	1306	1113	1620	1782	1975	2338	685	1185
1972	467	310	192	512	436	789	351	505	1153	1164	1221	501	633
1973	288	180	88	96	343	1379	1426	1146	2032	2260	2677	1321	1103
1974	599	369	269	162	312	670	815	941	1106	3087	1771	937	920
1975	417	247	167	106	279	550	808	1538	2135	2359	3593	1918	1176
1976	752	402	240	210	490	452	236	289	845	1968	1368	546	650
1977	335	211	132	105	286	425	406	908	1044	1939	1447	800	670
1978	394	260	174	952	830	1161	1120	1511	1583	1807	1540	905	1020
1979	505	392	356	402	642	1033	1023	1287	1375	1294	955	918	849
1980	855	308	145	98	498	675	741	1740	874	1327	1195	1025	790
1981	741	479	435	1066	1091	1220	1467	1386	1058	1770	2161	2787	1305
1982	686	213	140	193	336	782	996	638	1025	1866	962	326	680
1983	215	112	67	56	575	859	608	682	1663	1298	1102	1416	721
1984	579	367	238	131	623	1010	1136	1788	1467	1644	1390	472	904
1985	316	197	152	98	309	1114	638	1438	1386	1291	1493	977	784
1986	347	181	111	590	701	1095	819	693	1014	2069	2260	553	869
1987	272	184	107	184	527	844	948	1147	1475	2658	1180	653	848
1988	277	187	106	96	435	870	929	1220	1228	1939	1508	815	801
1989	465	257	177	104	487	682	1025	1316	1276	1342	1984	1106	852
1990	560	250	171	117	910	660	845	879	1529	2203	1483	2286	991
1991	379	213	258	115	515	734	541	672	1424	1800	1229	1627	792
1992	337	125	136	203	985	1027	988	1701	1588	1384	1163	714	863
1993	433	246	201	320	410	1102	805	623	1303	1816	2317	1220	900
1994	445	259	211	229	495	775	575	446	939	1328	1361	492	630
1995	303	161	107	148	848	1325	1162	1063	1350	1135	1542	1089	853
1996	2115	985	503	271	773	1536	1694	2085	1511	2628	2050	1900	1504
1997	417	260	171	126	208	253	295	242	588	766	927	407	388
1998	188	156	106	145	501	467	819	670	929	1452	856	1268	630
1999	655	358	271	309	867	1194	840	1881	2336	1238	1779	2376	1175
AVE	582	293	191	220	559	950	965	1148	1316	1742	1728	1244	912

(Monthly flows for Indio at Damsite were obtained from flows of Indio at Uracillo adjusted with the ratio of the drainage areas of Indio at Damsite = 381.1 Km^2 and Indio at Uracillo = 365 Km)
Indio Damsite = Indio at Uracillo * (381.1/365)

MWH / TAMS

Table 5-2

MONTHLY MEAN DISCHARGE (cubic meters per second)
RIO INDIO AT THE DAMSITE

Drainage Area: 381.1 km^2

YEAR	JAN	FEB	MAR	APRIL	MAY	JUNE	JULY	AUG	SEPT	OCT	NOV	DEC	AVE
1948	12.16	4.82	3.46	2.56	5.46	7.75	28.59	33.33	31.70	29.41	58.60	16.44	19.52
1949	6.88	4.18	2.72	2.48	7.43	40.23	28.37	35.54	49.78	47.16	81.57	64.64	30.92
1950	11.40	6.41	3.86	2.80	19.39	35.61	37.08	53.84	35.91	48.76	63.14	63.35	31.80
1951	20.30	11.24	6.41	4.26	19.92	24.35	22.63	24.57	41.70	35.32	52.97	30.74	24.53
1952	13.55	6.49	3.46	3.05	11.40	28.89	22.40	23.75	39.87	56.87	35.83	50.65	24.68
1953	37.89	14.77	7.90	5.62	23.52	21.13	20.52	17.05	21.05	59.61	59.68	32.88	26.80
1954	16.67	7.90	4.98	4.02	18.79	19.85	46.73	35.39	48.04	40.97	76.73	40.89	30.08
1955	43.81	13.85	6.88	5.46	10.85	41.18	29.33	48.18	58.75	50.80	77.37	45.05	35.96
1956	45.56	14.23	8.29	7.67	28.67	41.40	38.92	29.11	46.44	69.37	49.27	28.67	33.97
1967	11.47	5.93	3.86	2.80	11.78	11.86	11.70	22.85	24.94	56.80	37.37	30.15	19.29
1958	18.94	14.62	7.82	5.62	17.35	20.75	28.52	38.33	37.15	48.55	39.43	20.97	24.84
1959	9.43	5.78	4.10	3.62	5.46	83.66	93.45	14.19	16.64	50.15	34.71	43.94	30.43
1960	21.07	8.77	9.10	11.27	26.62	28.18	27.87	28.70	25.27	40.25	55.90	93.01	31.34
1961	15.79	8.23	4.68	4.80	8.34	20.87	18.44	20.45	32.66	50.46	44.46	35.96	22.10
1962	13.22	7.13	4.23	3.90	5.80	8.78	12.04	29.96	25.80	34.62	35.96	27.37	17.40
1963	11.06	7.46	4.12	8.34	18.02	21.61	26.01	33.69	34.10	48.43	54.21	18.13	23.77
1964	9.11	4.91	3.21	3.67	13.33	43.03	42.42	43.24	51.88	55.83	65.49	20.35	29.70
1965	15.68	7.79	4.46	2.64	4.12	8.12	7.24	16.43	13.11	26.95	33.38	35.44	14.61
1966	12.57	6.25	4.23	4.46	25.59	35.03	28.30	25.22	18.41	53.00	70.70	54.31	28.17
1967	17.60	8.67	4.57	7.13	21.40	50.06	37.81	41.50	55.22	63.68	42.62	20.14	30.87
1968	9.54	6.91	4.46	4.01	10.85	30.38	22.87	32.14	33.69	56.53	54.61	26.53	24.38
1969	11.39	6.69	3.44	5.24	10.63	21.61	55.55	88.14	51.62	44.05	48.02	29.24	31.30
1970	24.02	10.74	9.11	9.65	28.51	19.19	21.51	46.40	39.35	58.65	46.30	95.73	34.10
1971	41.70	14.93	8.45	6.36	24.97	36.99	31.52	45.87	50.46	55.93	66.19	19.40	33.56
1972	13.22	8.79	5.43	14.51	12.36	22.35	9.93	14.29	32.66	32.97	34.57	14.19	17.94
1973	8.17	5.11	2.49	2.72	9.71	39.04	40.37	32.45	57.54	63.98	75.79	37.40	31.23
1974	16.96	10.46	7.63	4.58	8.84	18.97	23.08	26.64	31.31	87.42	50.16	26.53	26.05
1975	11.82	6.99	4.74	3.00	7.91	15.57	22.87	43.54	60.47	66.79	101.74	54.31	33.31
1976	21.30	11.39	6.79	5.96	13.86	12.79	6.68	8.18	23.92	55.73	38.73	15.47	18.40
1977	9.48	5.98	3.75	2.97	8.11	12.04	11.50	25.70	29.55	54.92	40.99	22.66	18.97
1978	11.16	7.36	4.92	26.95	23.50	32.86	31.73	42.79	44.83	51.16	43.60	25.62	28.87
1979	14.31	11.09	10.08	11.40	18.19	29.26	28.96	36.44	38.94	36.65	27.04	26.00	24.03
1980	24.22	8.72	4.10	2.78	14.10	19.11	20.99	49.28	24.74	37.59	33.83	29.03	22.37
1981	20.99	13.57	12.32	30.17	30.90	34.56	41.55	39.26	29.97	50.12	61.18	78.93	36.96
1982	19.42	6.02	3.98	5.46	9.52	22.13	28.19	18.06	29.03	52.83	27.25	9.24	19.26
1983	6.08	3.16	1.89	1.59	16.29	24.33	17.23	19.32	47.09	36.75	31.22	40.09	20.42
1984	16.39	10.39	6.73	3.71	17.64	28.61	32.16	50.64	41.55	46.57	39.36	13.36	25.59
1985	8.95	5.58	4.31	2.77	8.76	31.53	18.06	40.72	39.26	36.54	42.29	27.67	22.20
1986	9.82	5.13	3.13	16.71	19.84	31.01	23.18	19.63	28.71	58.57	64.00	15.66	24.62
1987	7.72	5.20	3.02	5.22	14.93	23.91	26.83	32.47	41.76	75.28	33.41	18.48	24.02
1988	7.83	5.30	3.00	2.71	12.32	24.64	26.31	34.56	34.77	54.92	42.70	23.07	22.68
1989	13.16	7.29	5.01	2.93	13.78	19.32	29.03	37.27	36.13	38.00	56.17	31.32	24.12
1990	15.87	7.09	4.85	3.30	25.78	18.70	23.92	24.89	43.30	62.38	41.99	64.74	28.07
1991	10.74	6.02	7.32	3.26	14.59	20.80	15.33	19.02	40.32	50.98	34.79	46.08	22.44
1992	9.54	3.54	3.85	5.74	27.89	29.08	27.98	48.16	44.98	39.19	32.94	20.22	24.43
1993	12.25	6.96	5.68	9.07	11.62	31.20	22.78	17.64	36.91	51.43	65.62	34.56	25.48
1994	12.61	7.33	5.97	6.48	14.00	21.95	16.29	12.63	26.58	37.60	38.55	13.93	17.83
1995	8.57	4.56	3.02	4.20	24.00	37.51	32.90	30.10	38.21	32.13	43.66	30.84	24.14
1996	59.90	27.89	14.23	7.68	21.88	43.51	47.96	59.03	42.80	74.41	58.04	53.80	42.59
1997	11.80	7.37	4.85	3.55	5.88	7.16	8.36	6.86	16.64	21.69	26.24	11.53	10.99
1998	5.34	4.42	3.00	4.10	14.20	13.22	23.18	18.98	26.31	41.11	24.23	35.91	17.83
1999	18.55	10.14	7.67	8.75	24.55	33.81	23.79	53.26	66.15	35.06	50.38	67.28	33.28
AVE	16.48	8.30	5.42	6.22	15.83	26.91	27.33	32.49	37.27	49.33	48.94	35.23	25.81

(Monthly flows for Indio at Damsite were obtained from flows of Indio at Uracillo adjusted with the ratio of the drainage areas of Indio at Damsite = 381.1 Km^2 and Indio at Uracillo = 365 Km)
Indio Damsite = Indio at Uracillo * (381.1/365)

MWH / TAMS

Table 6

ELEVATION – VOLUME RELATIONSHIP
RIO INDIO RESERVOIR

Elevation (m)	Area (km^2)	Volume (MCM)
5	0.0	0
10	1.2	3
20	6.4	41
40	17.7	283
42	19.4	327
44	21.0	370
46	22.7	414
48	24.3	458
50	26.0	502
52	27.0	562
54	27.9	622
56	28.9	682
58	29.9	743
60	30.9	803
62	32.4	873
64	34.0	943
66	35.6	1,013
68	37.2	1,084
70	38.7	1,154
72	40.1	1,238
74	41.5	1,323
76	42.9	1,408
78	44.3	1,492
80	45.6	1,577
85	48.8	1,837
90	52.0	2,096
95	55.1	2,356
100	58.3	2,616

Table 7

RIO INDIO DAMSITE
DROUGHT-DURATION-FREQUENCY ANALYSIS

Accumulated 6-Month Flows				
Rank of Event (m)	Return Period (52/m)	6-Month Flow (mcm)	Date of Occurrence	
			From	To
1	52.00	89.2	Feb 65	Jul 65
2	26.00	93.8	Jan 48	Jun 48
3	17.33	95.1	Mar 97	Aug 97
4	13.00	99.0	Dec 82	May 83
5	10.40	103.9	Dec 48	May 49
6	8.67	108.6	Feb 62	Jul 62
7	7.43	109.6	Jan 77	Jun 77
8	6.50	109.9	Dec 72	May 73
9	5.78	110.4	Dec 97	May 98
10	5.20	113.3	Dec 84	May 85
11	4.73	123.6	Jan 57	Jun 57
12	4.33	128.0	Dec 58	May 59
13	4.00	128.6	Dec 87	May 88
14	3.71	129.6	Jan 75	Jun 75
15	3.47	134.0	Dec 86	May 87
16	3.25	135.8	Dec 63	May 64
17	3.06	140.7	Mar 76	Aug 76
18	2.89	144.9	Dec 67	May 68
19	2.74	151.1	Dec 94	May 95
20	2.60	152.9	Jan 69	Jun 69
21	2.48	159.4	Jan 89	Jun 89
22	2.36	162.5	Jan 91	Jun 91
23	2.26	162.5	Jan 61	Jun 61
24	2.17	170.6	Dec 92	May 93
25	2.08	172.4	Jan 82	Jun 82
26	2.00	173.1	Jan 52	Jun 52
27	1.93	174.7	Jan 74	Jun 74
28	1.86	177.3	Jan 94	Jun 94
29	1.79	180.9	Feb 80	Jul 80
30	1.73	183.0	Jan 63	Jun 63
31	1.68	187.1	Jan 54	Jun 54
32	1.63	190.3	Feb 72	Jul 72
33	1.58	196.0	Jan 90	Jun 90
34	1.53	206.1	Jan 50	Jun 50
35	1.49	206.3	Jan 92	Jun 92
36	1.44	213.3	Dec 85	May 86
37	1.41	216.4	Jan 84	Jun 84
38	1.37	220.6	Jan 58	Jun 58
39	1.33	224.2	Jan 51	Jun 51
40	1.30	228.4	Jan 66	Jun 66
41	1.27	235.1	Dec 78	May 79
42	1.24	242.4	Feb 53	Jul 53
43	1.21	250.4	Dec 77	May 78
44	1.18	255.8	Feb 70	Jul 70
45	1.16	268.3	Jan 99	Jun 99
46	1.13	272.2	Jan 60	Jun 60
47	1.11	278.9	Feb 55	Jul 55
48	1.08	283.6	Jan 67	Jun 67
49	1.06	319.3	Feb 71	Jul 71
50	1.04	355.1	Dec 80	May 81
51	1.02	360.8	Feb 56	Jul 56
52	1.00	381.0	Jun 98	Nov 98

Accumulated 8-Month Flows				
Rank of Event (m)	Return Period (52/m)	8-Month Flow (mcm)	Date of Occurrence	
			From	To
1	52.00	312.9	Apr 97	Mar 98
2	26.00	415.5	Dec 64	Nov 65
3	17.33	490.1	Jun 72	May 73
4	13.00	495.1	Jul 76	Jun 77
5	10.40	523.3	May 94	Apr 95
6	8.67	535.8	Feb 62	Jan 63
7	7.43	536.5	Aug 82	Jul 83
8	6.50	589.7	May 48	Apr 49
9	5.78	596.2	Dec 56	Nov 57
10	5.20	615.9	Apr 98	Mar 99
11	4.73	638.9	Oct 92	Sep 93
12	4.33	645.9	Nov 84	Oct 85
13	4.00	668.0	Oct 79	Sep 80
14	3.71	669.5	Nov 87	Oct 88
15	3.47	679.4	Jun 58	May 59
16	3.25	679.4	Apr 91	Mar 92
17	3.06	680.7	Feb 61	Jan 62
18	2.89	693.9	Nov 88	Oct 89
19	2.74	699.1	Jul 86	Jun 87
20	2.60	700.4	Jun 51	May 52
21	2.48	700.9	Jun 63	May 64
22	2.36	710.5	Nov 67	Oct 68
23	2.26	733.5	Jul 53	Jun 54
24	2.17	751.9	Aug 83	Jul 84
25	2.08	757.1	Jul 77	Jun 78
26	2.00	758.2	Aug 59	Jul 60
27	1.93	764.6	Aug 74	Jul 75
28	1.86	786.4	Dec 89	Nov 90
29	1.79	826.8	Oct 78	Sep 79
30	1.73	827.4	Dec 65	Nov 66
31	1.68	854.8	Oct 69	Sep 70
32	1.63	881.8	Jul 52	Jun 53
33	1.58	916.8	Oct 80	Sep 81
34	1.53	927.2	Aug 73	Jul 74
35	1.49	934.9	Jul 71	May 72
36	1.44	976.9	Jun 95	May 96
37	1.41	982.9	May 49	Apr 50
38	1.37	1019.7	Aug 54	Jul 55
39	1.33	1035.0	May 50	Apr 51
40	1.30	1099.0	Dec 55	Nov 56

Table 7 (cont.)

RIO INDIO DAMSITE
DROUGHT-DURATION-FREQUENCY ANALYSIS

Accumulated 18-Month Flows				
Rank of Event (m)	Return Period (52/m)	6-Month Flow (mcm)	Date of Occurrence From	To
1	52.00	456.7	Jan 97	Jun 98
2	26.00	644.9	Dec 64	May 66
3	17.33	656.6	Feb 76	Jul 77
4	13.00	681.4	Dec 71	May 73
5	10.40	724.2	Jan 62	Jun 63
6	8.67	731.7	Feb 82	Jul 83
7	7.43	758.9	Dec 93	May 95
8	6.50	772.9	Jan 48	Jun 49
9	5.78	820.6	Jan 57	Jun 58
10	5.20	862.6	Dec 87	May 89
11	4.73	866.8	Dec 84	May 86
12	4.33	904.3	Jan 91	Jun 92
13	4.00	907.5	Dec 67	May 69
14	3.71	936.2	Jan 51	Jun 52
15	3.47	936.7	Jan 79	Jun 80
16	3.25	939.9	Jan 74	Jun 75
17	3.06	1020.7	Jan 53	Jun 54
18	2.89	1130.4	Feb 60	Jul 61
19	2.74	1167.7	Jan 49	Jun 50
20	2.60	1323.2	Jun 86	Nov 87
21	2.48	1346.0	Jun 89	Nov 90
22	2.36	1386.2	Dec 69	May 71
23	2.26	1475.1	Jul 98	Dec 99
24	2.17	1479.0	Aug 58	Jan 60
25	2.08	1483.9	Feb 55	Jul 56
26	2.00	1646.4	Jun 66	Nov 67
27	1.93	1652.1	Aug 80	Jan 82
28	1.86	1821.4	Jun 95	Nov 96

Accumulated 24-Month Flows				
Rank of Event (m)	Return Period (52/m)	8-Month Flow (mcm)	Date of Occurrence From	To
1	52.00	896.6	Jan 97	Dec 98
2	26.00	1120.8	Apr 76	Mar 78
3	17.33	1212.8	Apr 61	Mar 63
4	13.00	1226.3	Feb 82	Jan 84
5	10.40	1229.4	Nov 64	Oct 66
6	8.67	1305.3	Jan 94	Dec 95
7	7.43	1353.3	Jun 57	May 59
8	6.50	1355.4	Oct 84	Sep 86
9	5.78	1363.1	Nov 87	Oct 89
10	5.20	1418.6	Sep 71	Aug 73
11	4.73	1434.4	Dec 78	Nov 80
12	4.33	1457.7	Jan 91	Dec 92
13	4.00	1515.5	Oct 51	Sep 53
14	3.71	1566.9	Feb 48	Jan 50
15	3.47	1587.6	Jul 67	Jun 69
16	3.25	1720.6	Sep 73	Aug 75
17	3.06	1976.7	Nov 53	Oct 55
18	2.89	2055.2	Sep 69	Aug 71

Table 8

MEAN MONTHLY NET RESERVOIR EVAPORATION
RIO INDIO RESERVOIR

Month	Evaporation (inches)
January	4.41
February	4.62
March	5.23
April	4.86
May	3.59
June	3.13
July	3.29
August	3.15
September	3.08
October	3.13
November	2.83
December	3.31
Annual	44.63 (1,134 mm)

Table 9

FLOOD PEAKS FOR SELECTED RETURN PERIODS
RIO INDIO AT BOCA DE URACILLO
(annual peaks)

Return Period (years)	Flood Peaks (cubic meters/second)		
	IHRE Estimates	MWH Estimates	
2	761	LP III	GEV
5	1141	569	562
10	1389	652	657
20	1654	695	713
25	1737	730	762
50	1985	739	780
100	2274	767	820
1000	3267	792	859
10000	4383		
Dry Period (January through March) Flood Peaks (cubic meters per second) Based on Generalized Extreme Value Distribution			
2			32
5			63
10			94
20			133
50			202
100			274

Table 10

RIO INDIO AT BOCA DE URACILLO
MAXIMUM INSTANTANEOUS FLOOD PEAKS (CMS)

Year	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Annual
1979							107.0	515.0	166.0	178.0	82.2	186.0	515.0
1980	225.0	20.1	5.3	7.2	146.0	176.0	58.4	610.0	67.5	325.0	166.0	89.2	610.0
1981	32.4	28.8	49.8	316.0	162.0	301.0		213.0	96.8	324.0	489.0	583.0	583.0
1982	80.9	10.0	7.1	49.1	43.9	169.0	386.0	68.3	198.0	445.0	213.0	13.9	445.0
1983	9.0	3.8	3.5	9.8	132.0	427.0	81.7	190.0	404.0	581.0	195.0	270.0	581.0
1984	49.1	22.9		4.7	167.0	197.0	186.0	375.0	237.0	526.0	190.0	20.1	526.0
1985	43.5	6.7	6.7	5.3	145.0	256.0	51.2	603.0	210.0	229.0	401.0	253.0	603.0
1986	15.9	8.2	4.7	242.0	380.0	375.0		182.0	171.0	261.0	504.0	25.2	504.0
1987	11.4	9.0	3.8			319.0	247.0	227.0	443.0	709.0	151.0	89.2	709.0
1988	10.3	8.2	4.1	14.9							187.0	97.6	
1989	25.2			4.7	481.0	346.0	474.0	289.0	360.0	156.0	459.0	297.0	481.0
1990	80.9	9.0	9.4	6.0	595.0	122.0				435.0	345.0	597.0	597.0
1991	22.6	9.0	99.3	10.7	138.0	128.0	54.8	115.0	316.0	336.0	207.0	772.0	772.0
1992	15.1	8.2	6.7	114.0	269.0	199.0	131.0	619.0	286.0	546.0	230.0	41.5	619.0
1993	28.8	9.8	27.0	229.0	48.3	352.0	165.0	57.0	337.0	337.0	391.0	233.0	546.0
1994	22.5	9.4	33.7	25.8	390.0	521.0	124.0	83.4	632.0	133.0	626.0	193.0	632.0
1995	38.8	6.7	5.3	13.4	355.0	296.0	289.0	198.0	366.0	550.0	372.0	153.0	372.0
1996	20.1	34.3			194.0	522.0			515.0		372.0	512.0	
1997	20.1	34.3				126.0							
1998	10.3	6.7	6.0	39.5	256.0								

Table 11

RIO INDIO AT BOCA DE URACILLO FLOOD HYDROGRAPHS

Time (hours)	20-Year Return Period (cms)	50-Year Return Period (cms)
0	25	27
1	48	30
2	51	33
3	109	96
4	122	185
5	229	301
6	246	426
7	326	404
8	363	391
9	414	445
10	466	502
11	555	651
12	673	778
13	703	810
14	762	820
15	733	794
16	718	778
17	658	713
18	629	682
19	600	651
20	543	590
21	475	516
22	461	502
23	422	459
24	383	418
25	334	364
26	298	326
27	263	288
28	230	252
29	197	218
30	167	185
31	133	148
32	101	114
33	88	100
34	77	89
35	68	78
36	60	70
37	57	66
38	51	61
39	50	59
40	49	57
41	48	57
42	47	56
43	46	55
44	45	53
45	44	52
46	43	51
47	42	51
48	41	50

Table 12

SEASONAL VARIATION OF ESTIMATED MAXIMUM 12-HOUR PERSISTING DEW POINTS IN PANAMA

Month	12-Hour Dew Point (°F)
November- February	77.0
March	77.5
April – August	78.0
September – October	79.0

Table 13

SEQUENTIAL ARRANGEMENT OF PMP INCREMENTS
(increments as percentages of 48-hour PMP)

Hour	Increment	Hour	Increment	Hour	Increment	Hour	Increment
1	0.8	13	1.0	25	3.0	37	1.3
2	0.8	14	1.0	26	4.0	38	1.2
3	0.8	15	1.0	27	14.0	39	1.1
4	0.8	16	1.1	28	15.0	40	1.1
5	0.8	17	1.1	29	6.0	41	1.0
6	0.8	18	1.2	30	4.0	42	1.0
7	0.9	19	1.4	31	3.0	43	1.0
8	0.9	20	1.6	32	2.5	44	0.9
9	0.9	21	1.8	33	2.3	45	0.9
10	0.9	22	2.0	34	1.9	46	0.9
11	0.9	23	2.5	35	1.7	47	0.9
12	1.0	24	3.0	36	1.4	48	0.9

Table 14

TIME AREA CURVE FOR RIO INDIO BASIN

Interval (hours)	**Contibuting Area** (sq. km)
1	21.4
2	32.1
3	42.8
4	69.8
5	124.8
6	179.9
7	244.6
8	300.6
9	346.5
10	381.1

Table 15

EMPRESA DE TRANSMISION ELECTRICA, S.A.
DEPARTMENTO DE HIDROMETEROLOGIA, SECCION HIDROLOGIA
MUESTROS DE SEDIMENTOS
COCLE DEL NORTE, CANOAS. 105-0102

Número de muestra	Fecha (Date)	Q. Liquido (Streamflow) m^3/s	Concentración (Concentration) mg/l	Q. Sólido (Sediment Discharge) ton/dia (tons/day)	Temperatura (Temperature) °C
1	23-11-83	18.98	11.14	18.30	26
2	28-1-84	27.10	5.40	12.60	25.5
3	1-4-84	11.58	5.16	5.16	25.5
4	20-5-84	27.50	28.55	67.83	26
5	9-7-84	38.45	8.07	26.81	23
6	29-8-84	42.77	17.30	63.90	24
7	21-10-84	33.01	7.94	22.60	
8	1-3-85	19.42	10.40	17.40	
9	21-6-85	26.71	12.50	28.80	
10	20-9-85	20.91	6.50	11.70	
11	12-12-85	56.85	7.77	38.20	
12	14-3-86	12.60	4.85	5.28	
13	9-10-86	43.45	22.40	84.10	
14	9-10-86	43.45	22.40	84.10	
15	27-11-86	38.99	4.12	13.90	
16	25-2-87	8.36	8.43	6.09	26
17	20-5-87	25.77	9.97	22.20	25
18	12-8-87	24.96	8.52	18.40	24
19	7-10-87	28.32	10.30	25.20	27
20	24-2-88	28.88	9.74	24.31	24
21	26-10-88	29.94	8.09	20.90	25
22	16-3-89	14.96	2.64	3.41	25
23	30-6-89	19.81	3.49	5.97	
24	23-11-89	53.52	19.50	90.20	
25	21-11-90	15.64	6.51	8.79	
26	15-11-90	40.49	7.66	26.80	
27	4-9-91	25.94	33.56	75.22	
28	2-9-92	20.16	1.72	3.00	
29	17-12-92	24.52	3.25	6.89	
30	19-8-93	42.29	20.66	75.48	
31	17-3-94	8.14	10.77	7.57	27
32	8-6-94	29.48	11.26	28.68	28
33	8-sep-94	19.96	6.64	11.45	27
34	30-nov-94	31.51	6.89	18.75	27
35	17-2-95	10.60	17.73	16.24	26
36	1-may-95	17.41	8.50	12.76	
37	08-jul-95	18.78	10.10	1.78	
38	29-sep-95	38.89	32.18	108.13	
39	01-dic-95	22.20	17.80	34.13	
40	22-AGP-96	51.28	16.50	73.13	
41	16-nov.96	58.53	9.73	49.19	26
42	14 MAR. 97	16.95	3.79	5.55	--
43	30 jul. 97	30.41	10.47	27.51	--
44	13 nov. 97	32.65	8.19	23.04	--
45	21 abr. 98	5.56	5.89	2.83	--
46	11 ag0. 98	16.92	2.52	3.68	--

Table 16

EMPRESA DE TRANSMISION ELECTRICA, S.A.
DEPARTMENTO DE HIDROMETEROLOGIA, SECCION HIDROLOGIA
MUESTROS DE SEDIMENTOS
TOABRE, BATATILLA. 1050-0201

Número de muestra	Fecha (Date)	Q. Liquido (Streamflow) m^3/s	Concentración (Concentration) mg/l	Q. Sólido (Sediment Discharge) ton/dia (tons/day)	Temperatura (Temperature) °C
1	3-2-82	18.44	6.60	10.50	
2	10-3-82	10.50	6.02	5.46	28.0
3	28-4-82	7.53	6.27	4.08	28.0
4	29-7-82	46.30	38.50	154	26.0
5	8-9-82	42.40	15.60	57.1	27.0
6	27-10-82	82.3	46.80	33.3	26.0
7	24-11-82	26.40	19.10	43.6	
8	26-1-83	7.96	10.27	7.06	26.0
9	6-4-83	2.72	6.44	1.51	26.0
10	26-5-83	26.28	8.04	18.25	27.0
11	24-8-83	38.54	22.4	74.59	26.0
12	23-11-83	43.79	19.81	75	
13	26-1-84	17.15	6.91	10.25	24.5
14	30-3-84	14.78	11.71	15	27.0
15	18-5-84	10.21	7.15	6.31	26.0
16	6-7-84	94.89	120.06	984.3	24.0
17	30-8-84	50.61	19.10	83.5	26.0
18	19-10-84	66.70	78.80	454	25.0
19	29-11-84	49.87	13.12	56.6	
20	5-3-85	10.41	10.10	9.08	
21	23-6-85	25.28	25.00	54.6	
22	21-9-85	43.20	24.70	92.2	
23	11-12-85	46.70	26.00	105	
24	13-3-86	7.06	6.34	3.87	
25	3-6-86	9.97	5.84	5.03	
26	8-10-86	86.20	239.30	1782	
27	26-11-86	65.20	9.54	53.7	
28	24-2-87	7.04	10.73	6.53	
29	19-5-87	19.20	9.55	15.8	26.0
30	11-8-87	39.10	18.58	62.8	25.0
31	6-10-87	73.60	282.10	1794	25.0
32	23-2-88	6.32	3.57	1.95	24.0
33	27-10-88	55.55	66.86	320.87	25.0
34	17-3-89	7.97	7.91	5.45	26.5
35	1-7-89	31.12	5.27	14.17	
36	24-11-89	53.11	5.56	25.5	
37	18-11-90	56.42	12.00	58.5	
38	5-9-91	25.90	10.11	22.62	
39	1-9-92	40.79	16.90	59.5	
40	15-12-92	31.56	5.53	15.08	
41	17-8-93	21.50	7.59	14.1	
42	19-3-94	6.89	23.10	13.75	27.0
43	10-6-94	19.61	28.70	48.63	
44	9-sep-94	42.43	29.98	109.92	27.0
45	1-dic-94	35.89	8.83	27.38	--
46	19-2-95	7.63	7.67	5.05	27.0

Table 16 (cont.)

EMPRESA DE TRANSMISION ELECTRICA, S.A.
DEPARTMENTO DE HIDROMETEROLOGIA, SECCION HIDROLOGIA
MUESTROS DE SEDIMENTOS
TOABRE, BATATILLA. 1050-0201

Número de muestra	Fecha (Date)	Q. Liquido (Streamflow) m^3/s	Concentración (Concentration) mg/l	Q. Sólido (Sediment Discharge) ton/dia (tons/day)	Temperatura (Temperature) °C
47	09-may-95	16.40	5.78	8.18	28.0
48	10-jul-95	28.09	4.57	11.08	
49	28-sep-95	49.22	35.61	151.44	27.0
50	04-dic-95	69.23	55.99	334.92	
51	19-nov-96	44.04	10.82	41.17	26
52	15-mar-97	9.99	36.47	31.47	--
53	31-jul.	20.93	20.79	37.6	--
54	14-	38.13	9.8	32.29	--
55	22-	2.7	5.78	1.35	--
56	12	27.46	20.83	49.415	--

Table 17

MONTHLY SUSPENDED SEDIMENT DISCHARGE
RIO CHAGRES AT CHICO
(Metric Tons)

Drainage Area: 415 km^2

Year	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Annual
1980													
1981	762	981	367	73,458	16,517	7,687	31,683	44,578	18,956	26,299	27,001	51,198	299,489
1982	677	125	73	226	526	280	8,607	3,724	4,326	9,457	2,207	993	31,220
1983	439	161	101	2,148	34,182	5,040	4,618	10,474	8,859	10,158	12,553	41,637	130,370
1984	271	32	30	42	9,510	11,156	6,866	27,779	8,747	42,323	14,020	2,932	123,708
1985	280	224	239	149	3,832	16,072	4,919	5,636	14,475	6,223	886	47,053	99,987
1986	216	86	1,365	3,067	55,169	3,471	2,759	4,505	11,289	10,918	24,437	1,037	118,319
1987	72	55	49	13,452	60,093	3,160	14,173	16,010	52,045	10,314	65,068	2,286	236,778
1988	48	92	40	27	7,117	1,602	12,057	7,762	3,429	5,427	8,530	1,324	47,454
1989	353	1,847	194	377	3,517	6,146	9,140	2,769	2,854	20,496	9,413	2,894	59,999
1990	550	302	275	1,520	41,260	1,223	1,964	5,541	5,885	12,230	11,296	18,557	100,601
1991	168	155	3,270	1,542	7,168	1,892	2,948	1,885	11,397	4,639	24,619	790	60,474
1992	132	227	82	1,247	5,033	9,745	5,090	16,957	9,539	2,486	14,108	374	65,022
1993	1,388	57	3,067	6,365	6,094	18,594	2,826	1,745	5,600	13,872	1,642	591	61,842
1994	126	155	183	53	4,713	9,303	7,583	3,051	3,100	6,391	16,728	716	52,103
1995	273	12	-	439	24,706	12,492	12,881	7,547	1,994	2,790	7,227	23,238	93,599
1996	13,781	357	1,078	1,210	11,776	3,756	2,371	8,819	3,234	9,252	119,678	23,109	198,423
1997	69	43	2	208	19,841	1,894	762	1,825	1,046	1,394	1,033	768	28,884
1998													
Mean	1,153	289	613	6,208	18,297	6,677	7,720	10,036	9,810	11,451	21,203	12,912	106,369

Table 18

MONTHLY SUSPENDED SEDIMENT DISCHARGE RIO PEQUENI AT CANDELARIA (Metric Tons)

Drainage Area: 135 km^2

Year	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Annual
1980													
1981	1,548	461	512	289,267	40,078	10,710	35,101	12,365	5,818	3,981	3,463	14,661	417,964
1982	377	107	71	119	234	198	733	5,522	4,780	37,676	1,653	343	51,812
1983	146	65	56	10,652	26,589	26,012	7,267	3,874	5,874	11,139	13,975	124,654	230,302
1984	159	144	52	21	930	4,593	3,264	6,248	1,716	2,404	3,561	1,575	24,667
1985	74	16	64	127	3,259	5,046	1,388	294	3,704	2,161	1,581	8,908	26,621
1986	280	59	156	3,558	33,621	6,406	5,652	4,003	10,318	1,111	6,242	250	71,656
1987	49	53	17	4,095	38,666	17,912	5,839	12,933	12,806	6,164	13,175	2,228	113,936
1988	46	118	71	74	2,357	1,758	17,269	9,568	800	7,790	1,828	1,456	43,134
1989	237	1,881	49	25	5,251	5,988	14,774	5,745	2,693	19,416	11,614	8,470	76,142
1990	668	38	103	202	7,188	1,318	1,834	8,067	2,994	7,788	3,397	1,901	35,498
1991	95	38	473	464	6,037	1,331	1,069	3,340	18,429	1,432	17,751	479	50,938
1992	99	31	65	1,894	11,770	1,956	3,067	10,770	3,731	539	6,084	2,196	42,201
1993	824	26	393	10,433	1,768	3,193	1,173	833	3,236	8,436	1,719	5,651	37,684
1994	74	51	77	43	2,386	8,828	1,270	9,706	972	2,510	7,900	186	34,002
1995	448	26	17	838	2,491	1,730	1,835	698	683	1,020	2,421	21,627	33,833
1996	10,534	52	1,325	738	6,652	4,434	1,123	2,816	2,035	1,248	65,551	4,510	101,018
1997	68	417	15	122	5,901	1,754	1,332	4,804	2,177	1,543	440	24	18,597
1998													
Mean	925	211	207	18,981	11,481	6,069	6,117	5,976	4,868	6,845	9,550	11,713	82,942

Table 19

MONTHLY SUSPENDED SEDIMENT DISCHARGE
RIO BOQUERON AT PELUCA
(Metric Tons)

Drainage Area: 91 km^2

Year	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Annual
1980													
1981	948	143	722	66,910	15,804	18,587	46,485	9,464	3,422	7,204	51,819	111,843	333,351
1982	1,437	2,018	437	2,954	2,310	3,948	7,082	23,462	8,835	25,567	2,345	552	80,947
1983	192	25	23	5,080	9,711	6,598	13,583	8,298	9,104	11,826	19,483	136,506	220,427
1984	351	61	29	17	1,866	31,857	8,719	11,414	1,696	2,079	9,304	2,909	70,301
1985	28	7	29	26	1,278	7,986	4,279	1,227	3,048	3,104	2,417	5,660	29,087
1986	161	70	32	4,610	8,963	3,793	1,889	1,879	11,684	1,308	5,081	118	39,589
1987	21	79	25	14,670	51,760	5,174	2,523	6,099	8,830	3,611	5,771	831	99,393
1988	45	127	49	41	2,465	742	18,947	8,217	891	8,932	1,598	1,679	43,733
1989	84	453	17	5	6,337	2,430	5,447	4,517	1,090	6,606	4,348	2,741	34,075
1990	99	69	205	171	5,346	892	1,171	5,573	2,067	4,021	2,459	883	22,956
1991	87	15	493	441	8,881	666	606	1,434	13,660	563	14,593	410	41,849
1992	3	3	3	1,463	6,890	1,421	2,049	13,484	2,731	712	5,175	1,715	35,648
1993	547	1	211	7,176	708	3,924	3,295	2,098	3,395	5,144	2,645	3,688	32,831
1994	14	6	45	4	2,514	3,645	1,406	6,267	1,633	599	8,385	825	25,342
1995	1,284	0	0	173	519	3,544	4,103	166	1,255	403	3,653	19,395	34,495
1996	5,324	123	819	1,014	7,599	1,058	1,035	1,764	1,262	1,795	73,898	20,589	116,280
1997	19	365	7	3	5,422	5,024	865	3,168	1,792	1,660	198	2	18,523
1998													
Mean	626	210	185	6,162	8,140	5,958	7,264	6,384	4,494	5,008	12,539	18,256	75,225

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Table 20

MONTHLY SUSPENDED SEDIMENT DISCHARGE
RIO GATUN AT CIENTO
(Metric Tons)

Drainage Area: 117 km^2

Year	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Annual
1980													
1981													
1982													
1983													
1984													
1985													
1986													
1987	23	25	13	14,193	2,666	906	1,466	2,251	5,317	24,423	20,392	1,905	73,580
1988	39	20	11	11	604	316	8,358	10,995	6,246	32,806	3,526	2,642	65,573
1989	41	124	15	17	845	430	3,810	1,695	1,206	4,737	17,508	2,224	32,653
1990	67	6	19	6	1,808	309	839	4,049	3,612	6,541	2,638	809	20,704
1991	15	4	15	81	1,866	116	479	554	4,053	2,287	12,454	304	22,226
1992	22	26	13	138	7,458	2,298	743	4,855	3,821	2,281	1,387	1,177	24,218
1993	329	11	80	652	601	1,339	3,624	968	4,091	4,245	4,103	484	20,526
1994	688	-	4	3	188	2,061	1,052	1,040	425	2,270	7,502	68	15,300
1995	344	23	5	21	333	1,783	1,762	2,803	1,007	1,502	6,872	3,863	20,318
1996	7,423	22	43	151	2,377	2,096	1,087	1,697	901	1,296	39,636	4,237	60,966
1997	46	32	6	9	790	874	268	95	1,562	763	2,084	61	6,589
1998													
Mean	822	26	20	1,389	1,776	1,139	2,135	2,818	2,931	7,559	10,737	1,616	32,968

Table 21

MONTHLY SUSPENDED SEDIMENT DISCHARGE
RIO TRINIDAD AT CHORRO
(Metric Tons)

Drainage Area: 174 km^2

Year	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Annual
1980													
1981													
1982													
1983													
1984													
1985													
1986													
1987	16	25	28	99	696	502	1,085	2,774	4,042	5,328	793	838	16,226
1988	135	6	1	77	2,221	924	1,366	4,756	4,639	2,916	1,707	1,722	20,470
1989	199	37	19	8	968	727	2,053	2,935	3,047	2,529	1,487	3,377	17,387
1990	180	21	5	5	450	1,829	2,492	1,667	9,023	7,462	4,318	896	28,347
1991	32	6	29	6	767	646	197	391	1,386	2,246	1,689	795	8,189
1992	14	5	2	91	501	1,794	352	1,232	3,741	1,343	2,009	462	11,545
1993	41	12	11	184	714	2,243	467	563	3,307	2,163	4,234	941	14,879
1994	3	-	22	10	790	505	226	176	3,110	3,165	1,715	182	9,902
1995	9	1	-	20	1,179	2,345	1,939	3,320	2,832	5,818	2,342	470	20,274
1996	13,475	63	134	13	2,896	5,388	3,352	5,367	4,534	6,586	3,374	1,997	47,178
1997	416	1	-	22	13	389	67	28	285	1,557	840	7	3,625
1998													
Mean	1,320	16	23	49	1,018	1,572	1,236	2,110	3,631	3,737	2,228	1,062	18,002

Table 22

MONTHLY SUSPENDED SEDIMENT DISCHARGE
RIO CIRI GRANDE AT LOS CANONES
(Metric Tons)

Drainage Area: 187 km^2

Year	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Annual
1980													
1981													
1982													
1983													
1984													
1985													
1986													
1987	47	32	12	57	2,812	1,094	1,437	4,412	6,187	8,753	3,255	3,758	31,854
1988	49	22	7	29	728	1,551	1,469	4,645	4,173	5,040	3,939	1,801	23,451
1989	364	87	20	5	1,135	844	2,180	4,607	4,737	9,667	2,846	2,095	28,588
1990	596	12	26	7	1,042	1,062	1,494	1,074	7,588	5,774	2,740	2,929	24,345
1991	109	48	184	31	591	537	158	428	1,836	3,041	1,455	876	9,293
1992	46	13	6	286	1,047	1,267	1,268	4,005	3,686	1,974	2,977	357	16,932
1993	154	46	39	272	485	2,461	467	582	3,883	3,449	8,735	1,634	22,206
1994	15	8	39	33	859	1,348	391	141	1,957	3,960	3,224	224	12,198
1995	27	0	18	163	4,297	3,246	2,299	3,217	3,896	4,584	2,339	1,892	25,980
1996	93,184	150	216	31	3,695	6,064	9,620	14,255	8,190	13,264	7,511	7,214	163,392
1997	27	33	-	44	119	533	253	94	1,165	1,551	2,172	67	6,057
1998													
Mean	8,602	41	52	87	1,528	1,819	1,912	3,405	4,300	5,551	3,745	2,077	33,118

Table 23

SUSPENDED SEDIMENT DISCHARGE – COCLE DEL NORTE AT CANOAS USING SEDIMENT RATING AND FLOW DURATION CURVES

Limits (%)	Interval (%)	Middle Ordinate (%)	Qw (cms)	qs (mt/day)	Qs (mt/day)
0-0.01	0.01	0.005	1,350.0	728,724.1	7,287.2
0.01-0.02	0.01	0.015	1,300.0	669,954.5	6,699.5
0.02-0.05	0.03	0.035	1,200.0	560,525.4	16,815.8
0.05-0.1	0.05	0.075	1,100.0	461,745.2	23,087.3
0.1-0.5	0.4	0.3	800.0	227,124.5	90,849.8
0.5-1.0	0.5	0.75	450.0	63,028.5	31,514.3
1.0-2.0	1	1.5	145.0	1,814.8	1,814.8
2.0-5.0	3	3.5	115.0	781.1	2,343.3
5-10	5	7.5	87.0	283.1	1,415.6
10-15	5	12.5	68.0	158.8	793.8
15-20	5	17.5	57.0	106.5	532.6
20-25	5	22.5	50.0	79.2	396.1
25-30	5	27.5	45.0	62.4	312.1
30-35	5	32.5	41.5	52.0	259.9
35-40	5	37.5	38.0	42.6	212.9
40-45	5	42.5	35.0	35.4	176.8
45-50	5	47.5	32.0	28.9	144.4
50-55	5	52.5	30.0	25.0	124.8
55-60	5	57.5	27.0	19.7	98.3
60-65	5	62.5	25.5	17.3	86.4
65-70	5	67.5	23.0	14.6	72.8
70-75	5	72.5	21.0	12.9	64.7
75-80	5	77.5	19.0	11.4	56.8
80-85	5	82.5	17.0	9.8	49.1
85-90	5	87.5	15.0	8.4	41.8
90-95	5	92.5	12.0	6.2	31.2
95-97	2	96	8.8	4.2	8.3
97-99	2	98	5.6	2.3	4.6
99-100	1	99.5	3.5	1.3	1.3
				Sum =	185,296
				mt/yr	676,331

Qw = discharge at mid-ordinate of the interval
qs = sediment discharge corresponding to Qw
Qs = qs* interval

Table 24

SUSPENDED SEDIMENT DISCHARGE – RIO TOABRE AT BATATILLA USING SEDIMENT RATING AND FLOW DURATION CURVES

Limits (%)	Interval (%)	Middle Ordinate (%)	Qw (cms)	qs (mt/day)	Qs (mt/day)
0-0.01	0.01	0.005	2,400.0	1,664,295.3	16,643.0
0.01-0.02	0.01	0.015	2,100.0	1,274,056.0	12,740.6
0.02-0.05	0.03	0.035	1,900.0	1,042,830.3	31,284.9
0.05-0.1	0.05	0.075	1,400.0	566,017.5	28,300.9
0.1-0.5	0.4	0.3	900.0	233,812.1	93,524.8
0.5-1.0	0.5	0.75	300.0	25,946.3	12,973.1
1.0-2.0	1	1.5	180.0	8,301.0	8,301.0
2.0-5.0	3	3.5	135.0	4,369.1	13,107.3
5-10	5	7.5	104.0	2,441.3	12,206.4
10-15	5	12.5	80.0	852.3	4,261.5
15-20	5	17.5	66.0	408.4	2,041.8
20-25	5	22.5	58.0	249.1	1,245.5
25-30	5	27.5	51.0	152.3	761.6
30-35	5	32.5	45.0	94.4	471.8
35-40	5	37.5	40.5	63.1	315.3
40-45	5	42.5	37.0	50.7	253.3
45-50	5	47.5	32.5	42.3	211.7
50-55	5	52.5	28.5	35.3	176.6
55-60	5	57.5	24.0	27.9	139.3
60-65	5	62.5	20.5	22.4	112.0
65-70	5	67.5	17.5	18.0	90.0
70-75	5	72.5	14.5	13.9	69.4
75-80	5	77.5	12.4	11.2	55.9
80-85	5	82.5	10.3	8.7	43.3
85-90	5	87.5	8.4	6.5	32.7
90-95	5	92.5	6.4	4.5	22.4
95-97	2	96	4.6	2.8	5.7
97-99	2	98	3.6	2.0	4.1
99-100	1	99.5	2.9	1.5	1.5

Qw = discharge at mid-ordinate of the interval — Sum = 239,398

qs = sediment discharge corresponding to Qw

Qs = qs* interval — mt/yr 873,801

Table 25

RIO INDIO RESERVOIR
LOSS IN LIVE STORAGE

Period of Operation (years)	Live Reservoir Volume (MCM)	Loss (MCM)	Loss (%)
0	993.27		
5	992.29	0.98	0.10
10	991.32	1.95	0.20
20	989.40	3.87	0.39
25	988.42	4.85	0.49
50	983.63	9.64	0.97
100	974.03	19.24	1.94

Table 26

FLOOD FREQUENCY DATA
RIO INDIO AT BOCA DE URACILLO

Return Period (years)	Pre-Project Flood Peak (cms)	Post-Project Flood Peak (cms)
2	562	57
5	657	71
10	713	79
25	780	89
50	820	100
100	859	108

Table 27

HYDRAULIC CHARACTERISTICS OF REPRESENTATIVE CROSS SECTION RIO INDIO DOWNSTREAM FROM DAM

Discharge (cms)	Depth (m)	Area (sq m)	Top Width (m)	Channel Velocity (m/s)	Channel 'n' Value	Overbank 'n' Value	Slope
50	2.6	51.1	23.8	1.0	0.035	0.050	0.00046
57	2.8	55.6	24.4	1.0	0.035	0.050	0.00046
71	3.1	64.5	25.5	1.1	0.035	0.050	0.00046
79	3.4	69.5	26.2	1.1	0.035	0.050	0.00046
89	3.6	5.9	27.0	1.2	0.035	0.050	0.00046
100	3.9	82.9	27.9	1.2	0.035	0.050	0.00046
108	4.0	87.2	28.4	1.2	0.035	0.050	0.00046
200	5.6	136.2	33.7	1.5	0.035	0.050	0.00046
300	8.0	232.5	74	1.3	0.035	0.050	0.00046
500	8.6	296.4	115.1	1.9	0.035	0.050	0.00046
562	8.9	334.7	118.1	2.0	0.035	0.050	0.00046
657	9.4	393.3	122.7	2.1	0.035	0.050	0.00046
700	9.6	419.8	124.8	2.2	0.035	0.050	0.00046
713	9.7	426.7	125.3	2.2	0.035	0.050	0.00046
780	9.9	462.0	127.9	2.3	0.035	0.050	0.00046
820	10.1	483.0	129.4	2.3	0.035	0.050	0.00046
859	10.3	503.6	130.9	2.3	0.035	0.050	0.00046
1000	10.9	577.9	136.3	2.4	0.035	0.050	0.00046
1200	11.5	672.2	142.7	2.5	0.035	0.050	0.00046
1500	12.4	804.7	151.2	2.7	0.035	0.050	0.00046

Table 28

CHARACTERISTICS OF BED MATERIAL RIO INDIO DOWNSTREAM FROM INDIO DAM

Size Designation	Particle Size (mm)
D35	1.6
D50	6.0
D65	14.0
D90	50.0
Median D	16.5

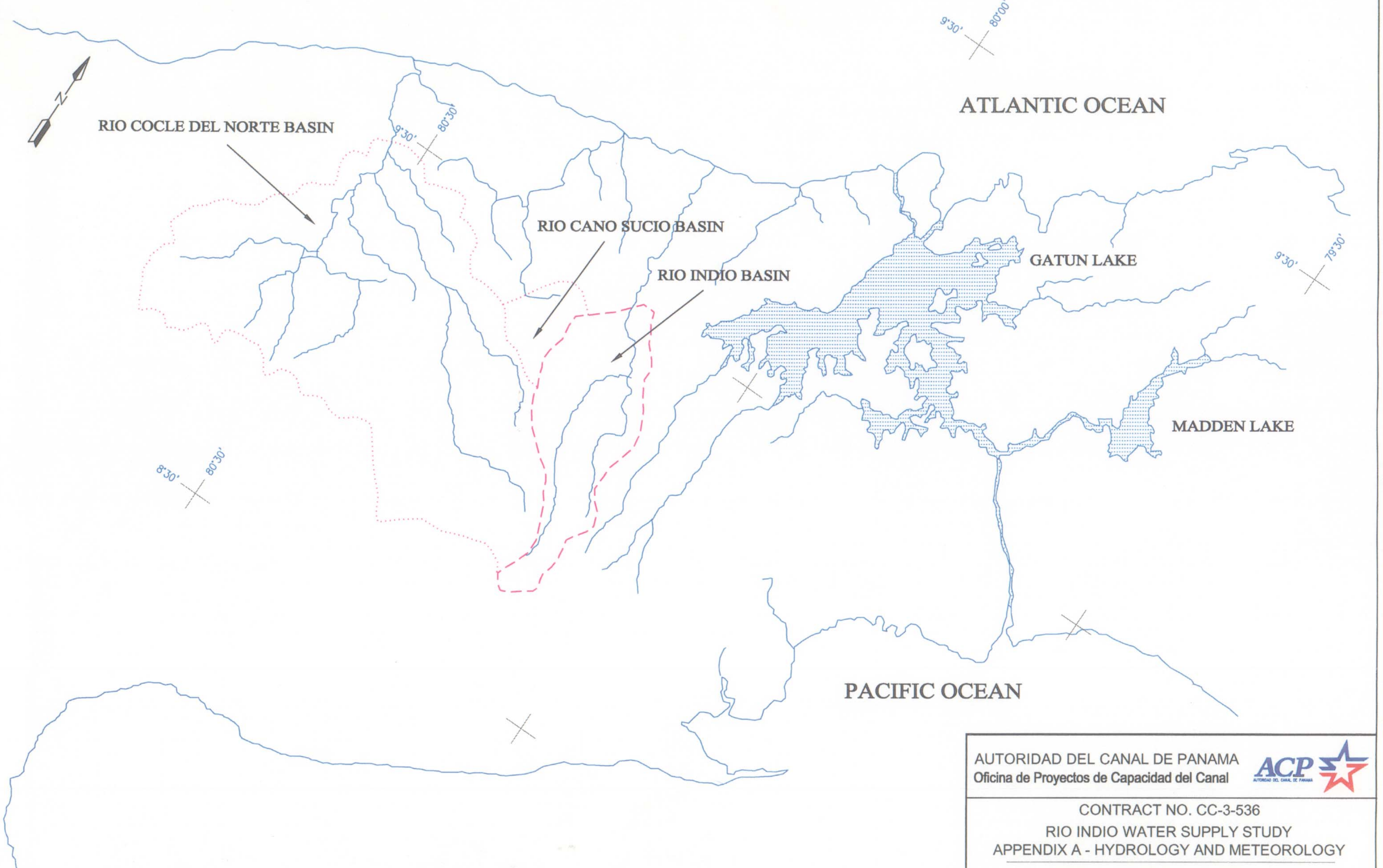
Note: The median diameter is estimated using United States Bureau of Reclamation procedure.

Table 29

ARMORING SIZES – RIO INDIO DOWNSTREAM FROM DAM

	Armoring Size (mm) For Indicated Flood					
	2-Yr	5-Yr	10-Yr	25-Yr	50-Yr	100-Yr
PRE-PROJECT						
Meyer-Peter, Muller	29	30	31	32	33	33
Shield's	41	44	45	46	47	48
POST-PROJECT						
Meyer-Peter, Muller	9	10	11	12	13	13
Shield's	13	14	16	17	18	19

EXHIBITS



Source:
US NWS, February 1978 Report

10 0 10 20 30 40 50 km
GRAPHIC SCALE

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

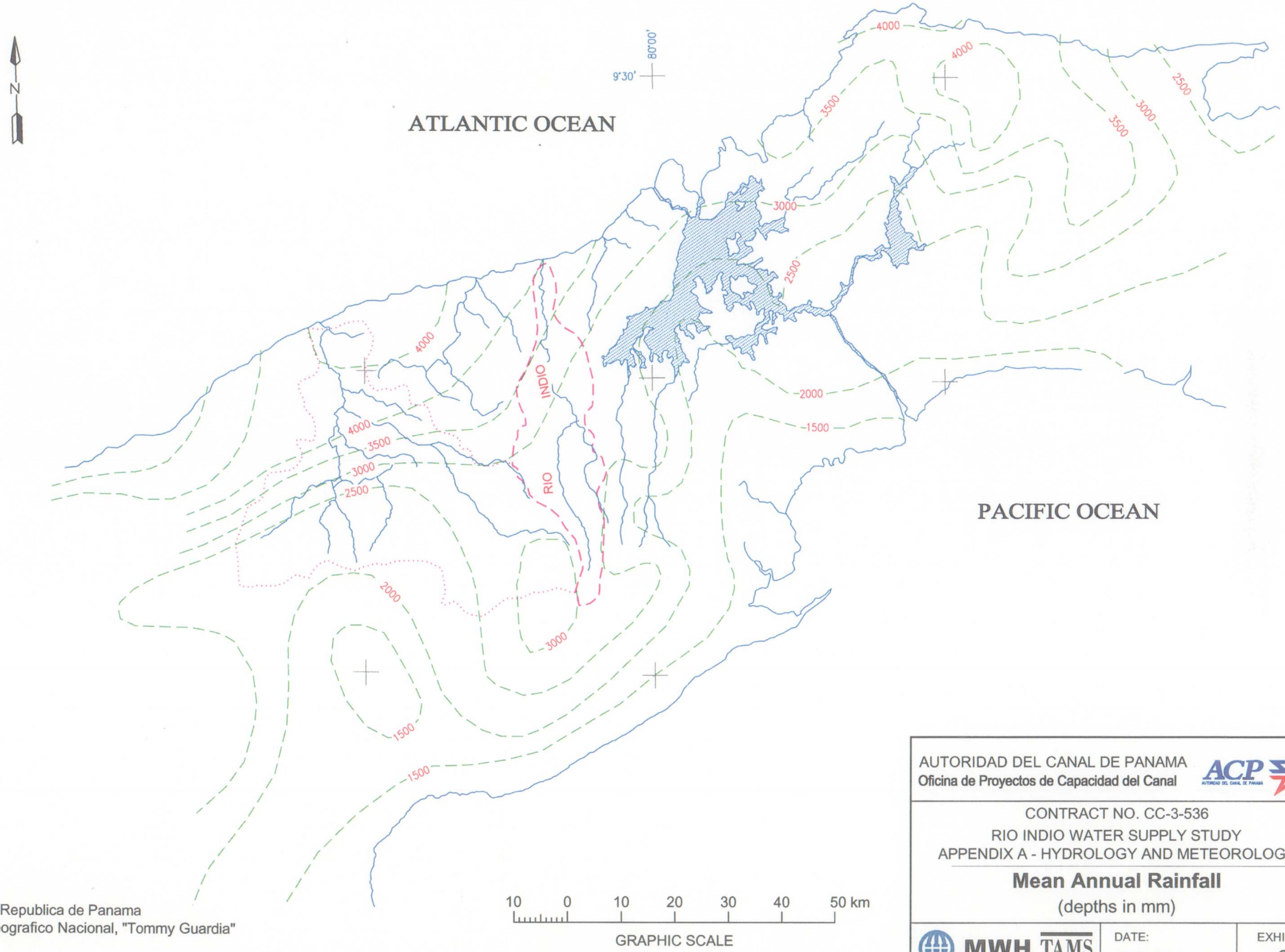
CONTRACT NO. CC-3-536
RIO INDIO WATER SUPPLY STUDY
APPENDIX A - HYDROLOGY AND METEOROLOGY

Location Map

MWH TAMS

DATE: APRIL, 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
RIO INDIO WATER SUPPLY STUDY
APPENDIX A - HYDROLOGY AND METEOROLOGY

Mean Annual Rainfall
(depths in mm)

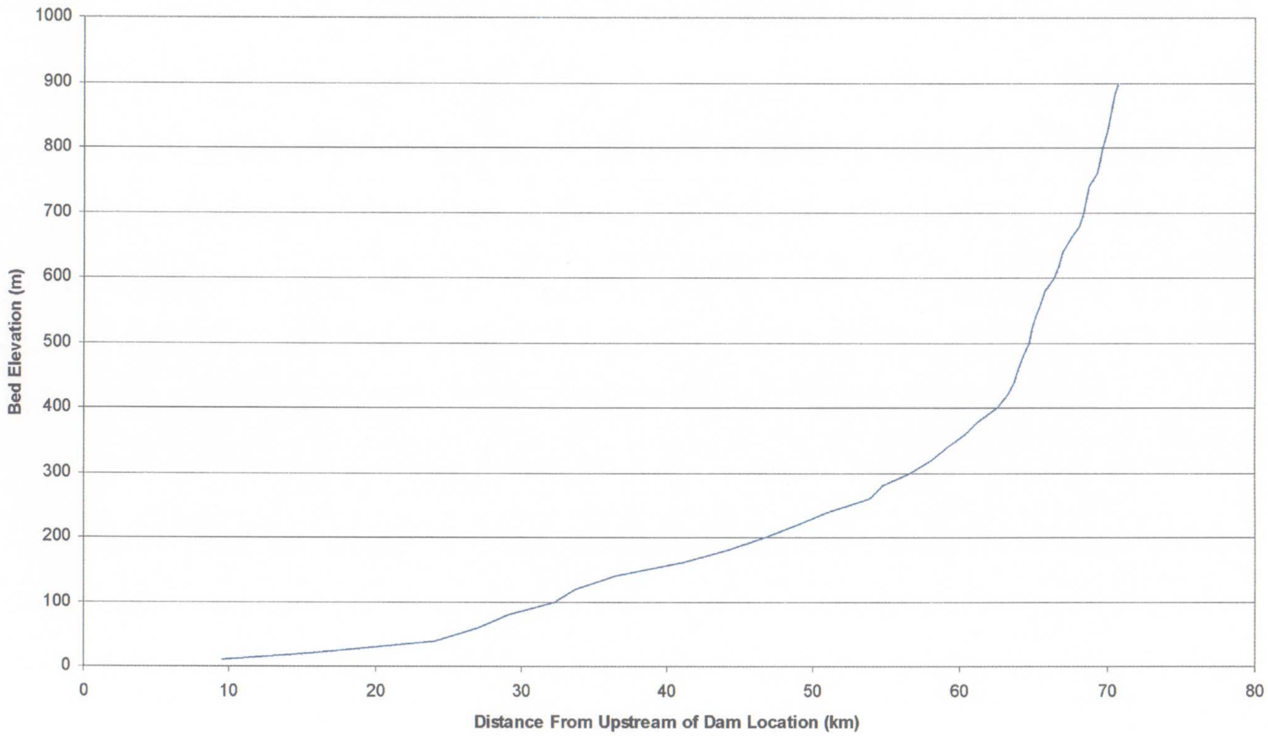
MWH TAMS

DATE: APRIL, 2003

EXHIBIT: 2

No.	Distance from U/S of Dam Location (km)	Elevation (m)
1	9.5	10
2	15.6	20
3	24.1	40
4	27.1	60
5	29.1	80
6	32.3	100
7	33.7	120
8	36.5	140
9	41.0	160
10	44.0	180
11	46.7	200
12	49.1	220
13	51.1	240
14	53.9	260
15	54.8	280
16	56.6	300
17	58.2	320
18	59.2	340
19	60.5	360
20	61.3	380
21	62.6	400
22	63.3	420
23	63.7	440
24	64.0	460
25	64.4	480
26	64.7	500
27	64.9	520
28	65.2	540
29	65.5	560
30	65.8	580
31	66.4	600
32	66.8	620
33	67.0	640
34	67.5	660
35	68.1	680
36	68.4	700
37	68.6	720
38	68.8	740
39	69.3	760
40	69.5	780
41	69.7	800
42	69.9	820
43	70.1	840
44	70.3	860
45	70.5	880
46	70.7	900

RIVER BED PROFILE FOR RIO INDIO



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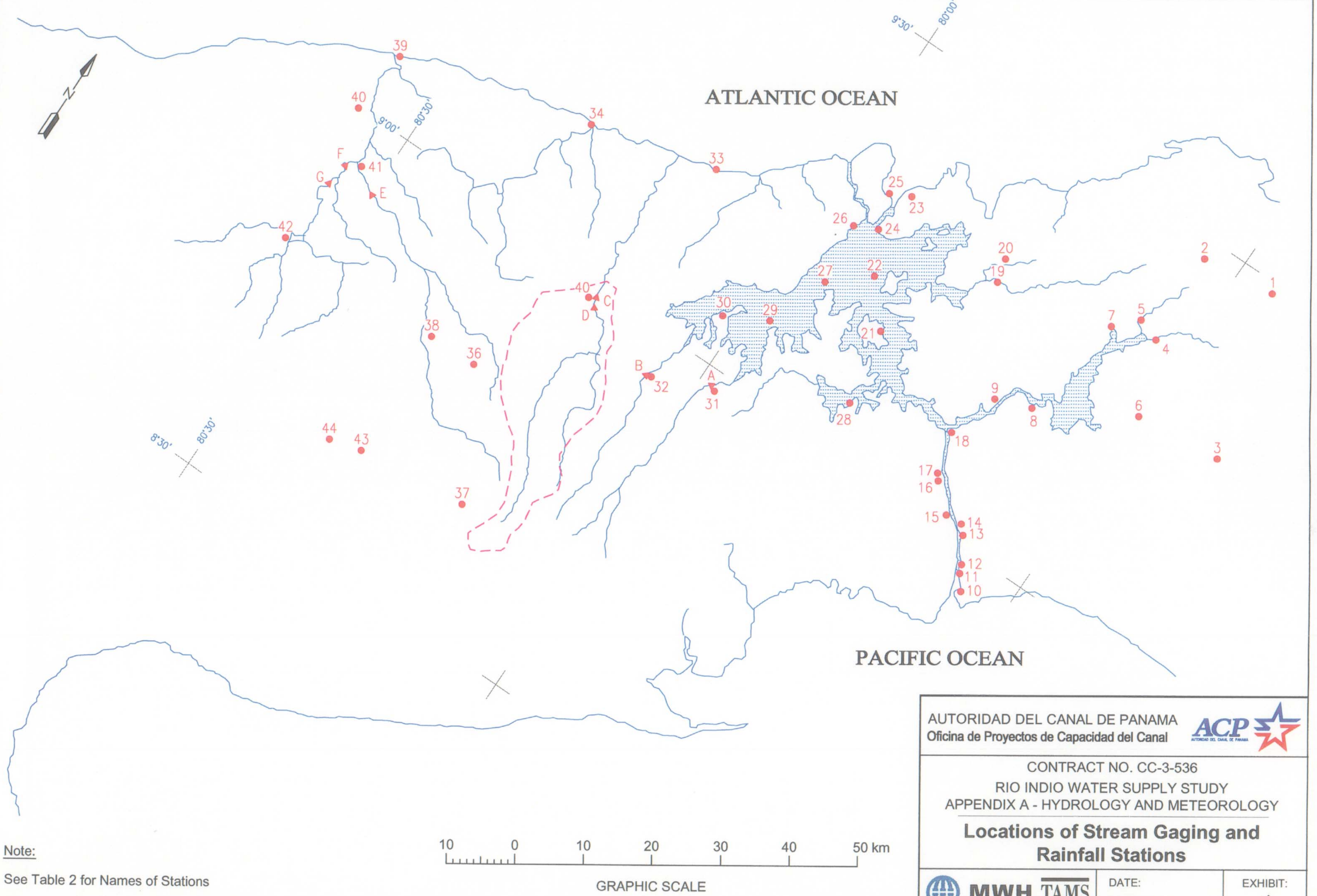
APPENDIX A - HYDROLOGY AND METEOROLOGY

River Bed Profile of Rio Indio

MWH TAMS

DATE: APRIL, 2003

EXHIBIT: 3



Note:

See Table 2 for Names of Stations

AUTORIDAD DEL CANAL DE PANAMA

Oficina de Proyectos de Capacidad del Canal

CONTRACT NO. CC-3-536

RIO INDIO WATER SUPPLY STUDY

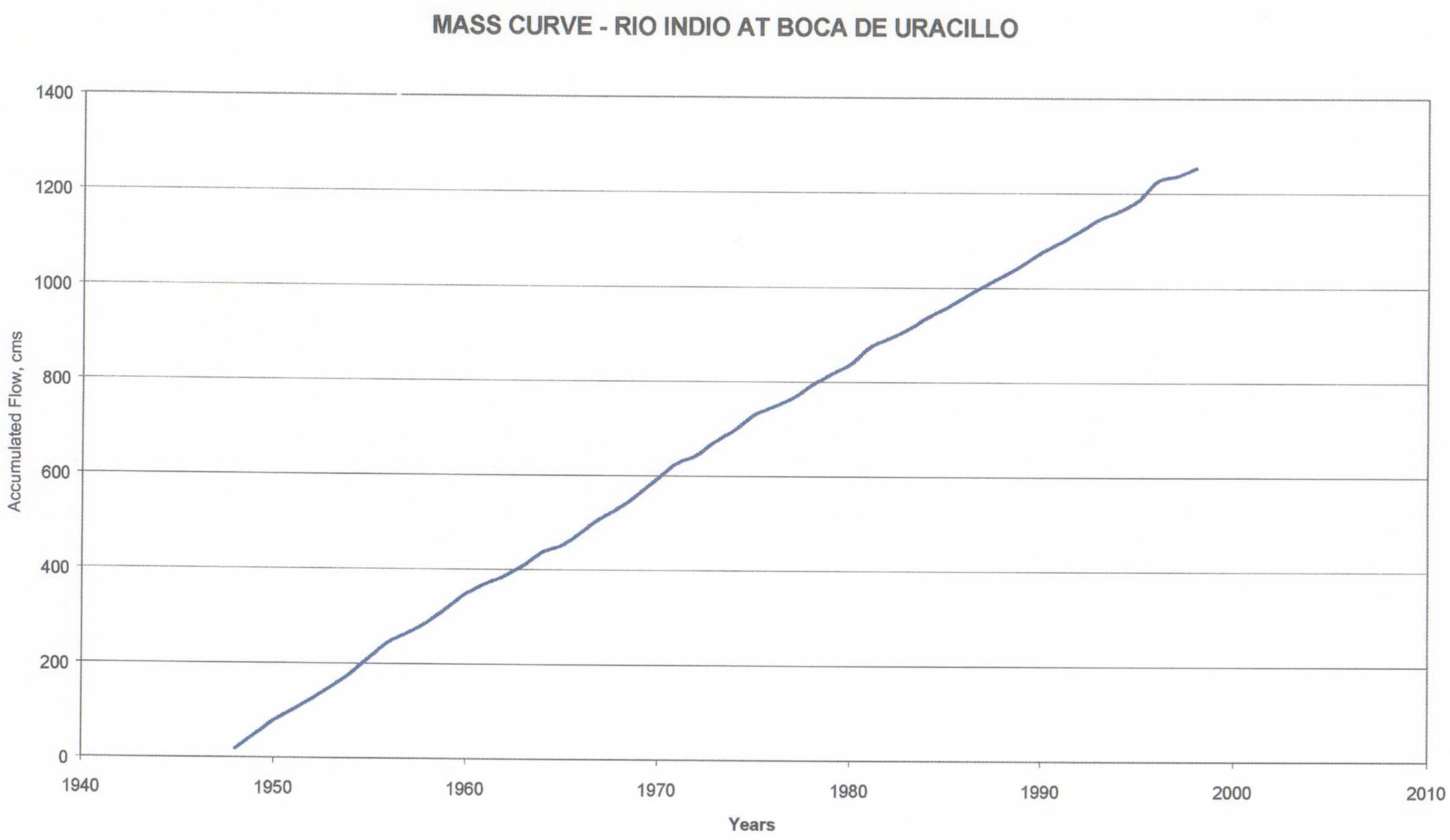
APPENDIX A - HYDROLOGY AND METEOROLOGY

Locations of Stream Gaging and Rainfall Stations

MWH TAMS

DATE: APRIL, 2003

EXHIBIT: 4



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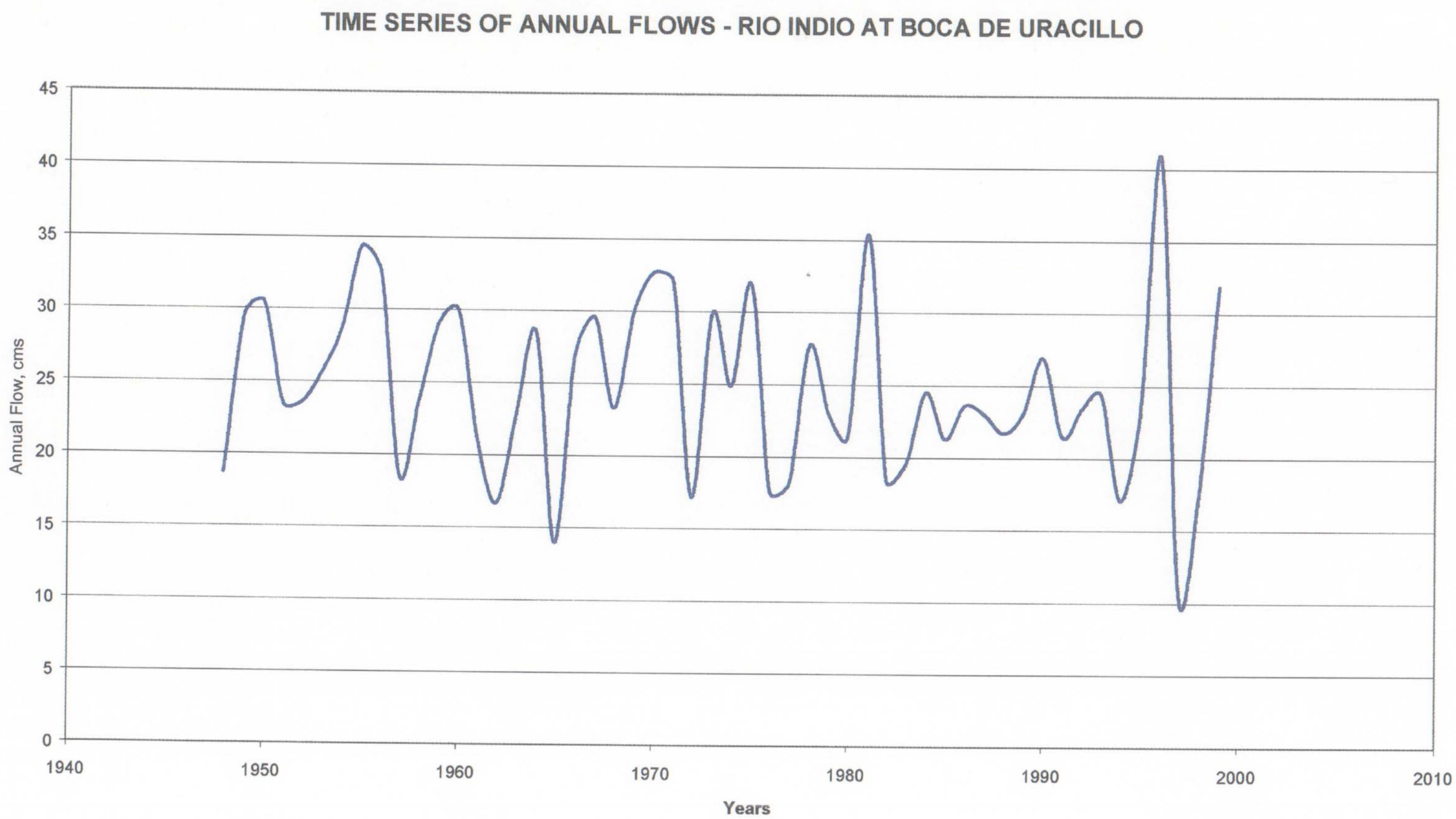
CONTRACT NO. CC-3-536
RIO INDIO WATER SUPPLY STUDY
APPENDIX A - HYDROLOGY AND METEOROLOGY

Mass Curve
Rio Indio at Boca de Uracillo

MWH TAMS

DATE:	EXHIBIT:
APRIL, 2003	5



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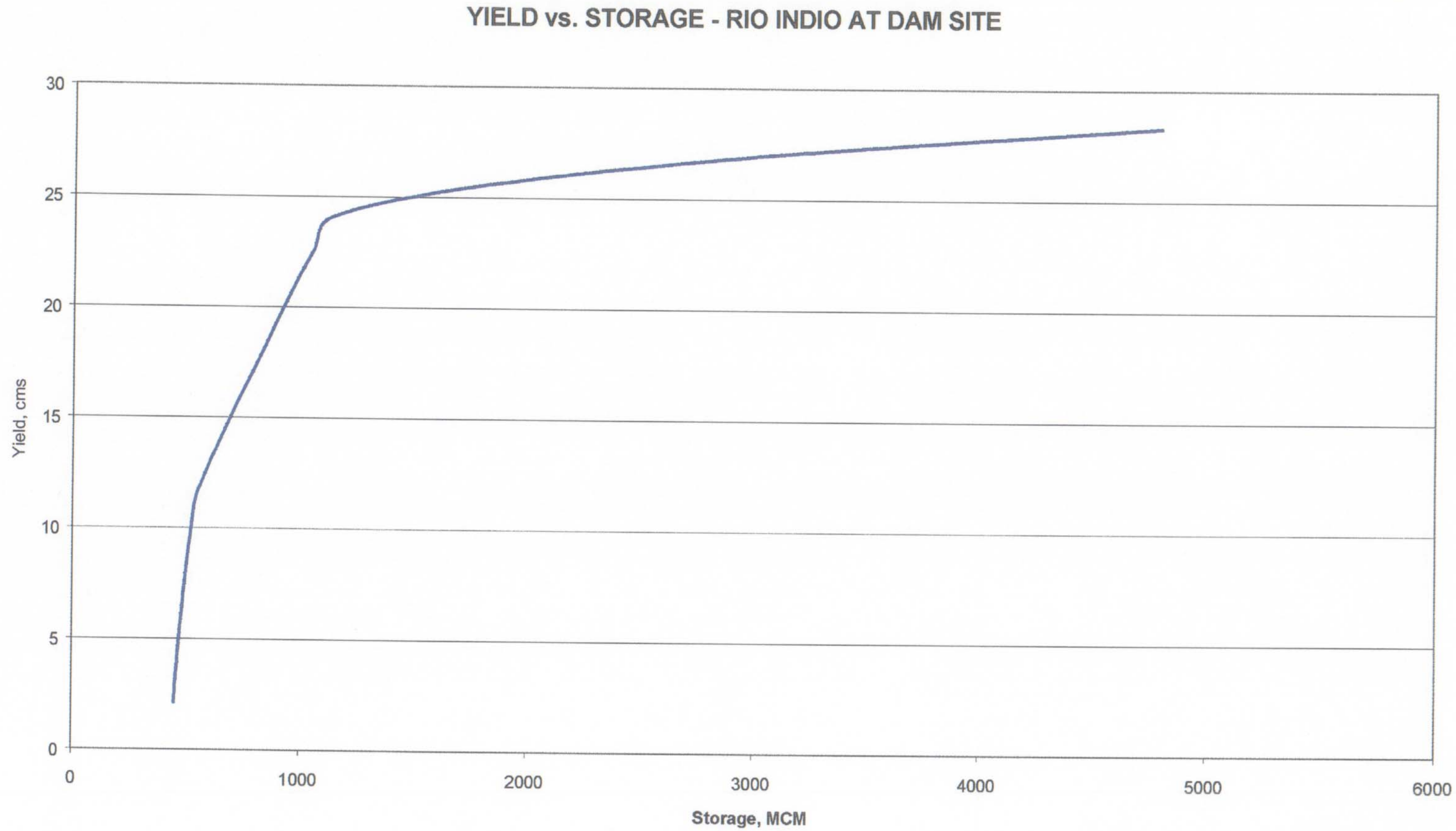
CONTRACT NO. CC-3-536
RIO INDIO WATER SUPPLY STUDY
APPENDIX A - HYDROLOGY AND METEOROLOGY

Time Series of Annual Flows
Rio Indio at Boca de Uracillo

MWH TAMS

DATE: APRIL, 2003

EXHIBIT: 6



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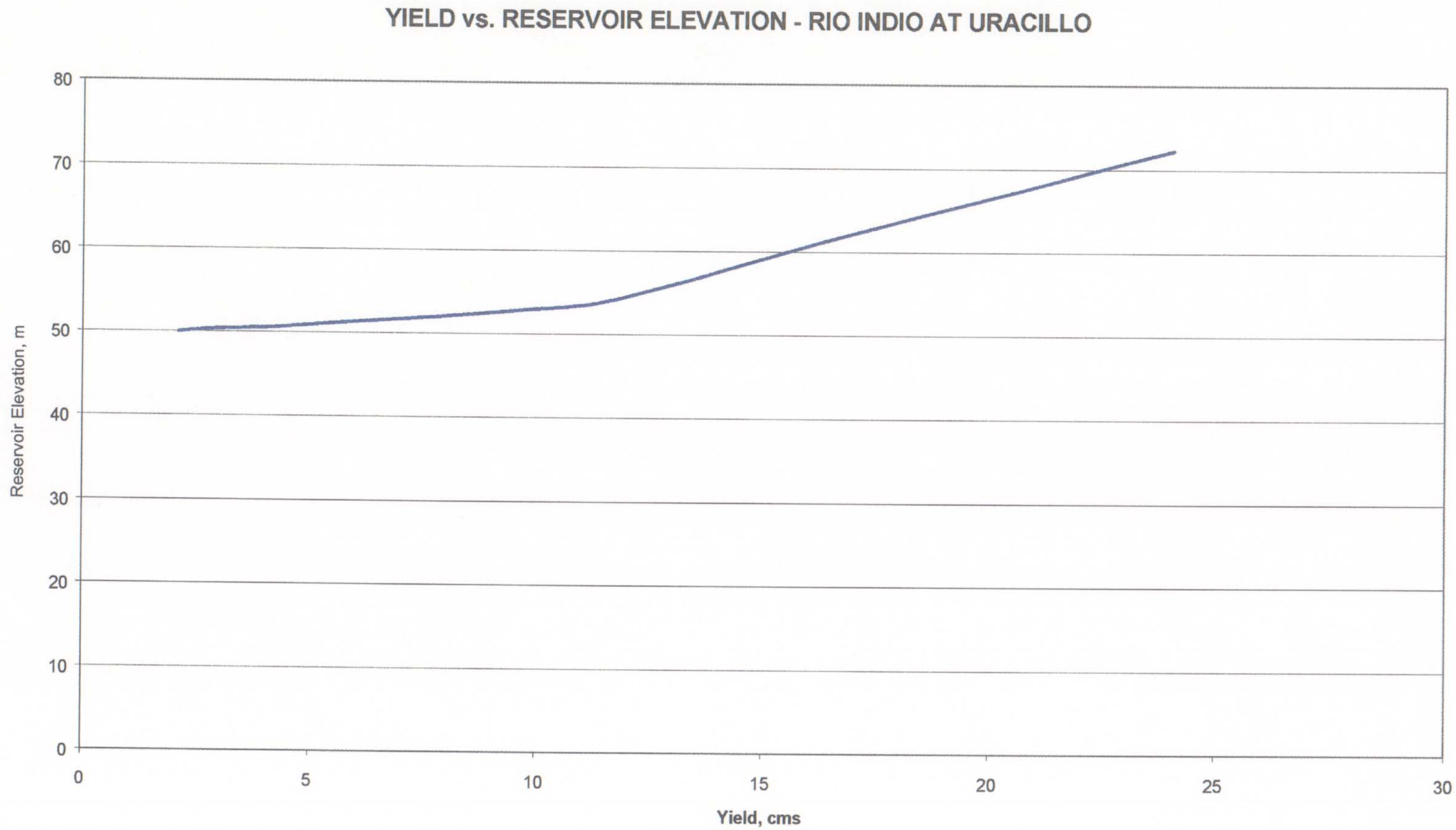
CONTRACT NO. CC-3-536
RIO INDIO WATER SUPPLY STUDY
APPENDIX A - HYDROLOGY AND METEOROLOGY

Yield vs. Storage
Rio Indio at Dam Site

MWH TAMS	DATE: APRIL, 2003	EXHIBIT: 7



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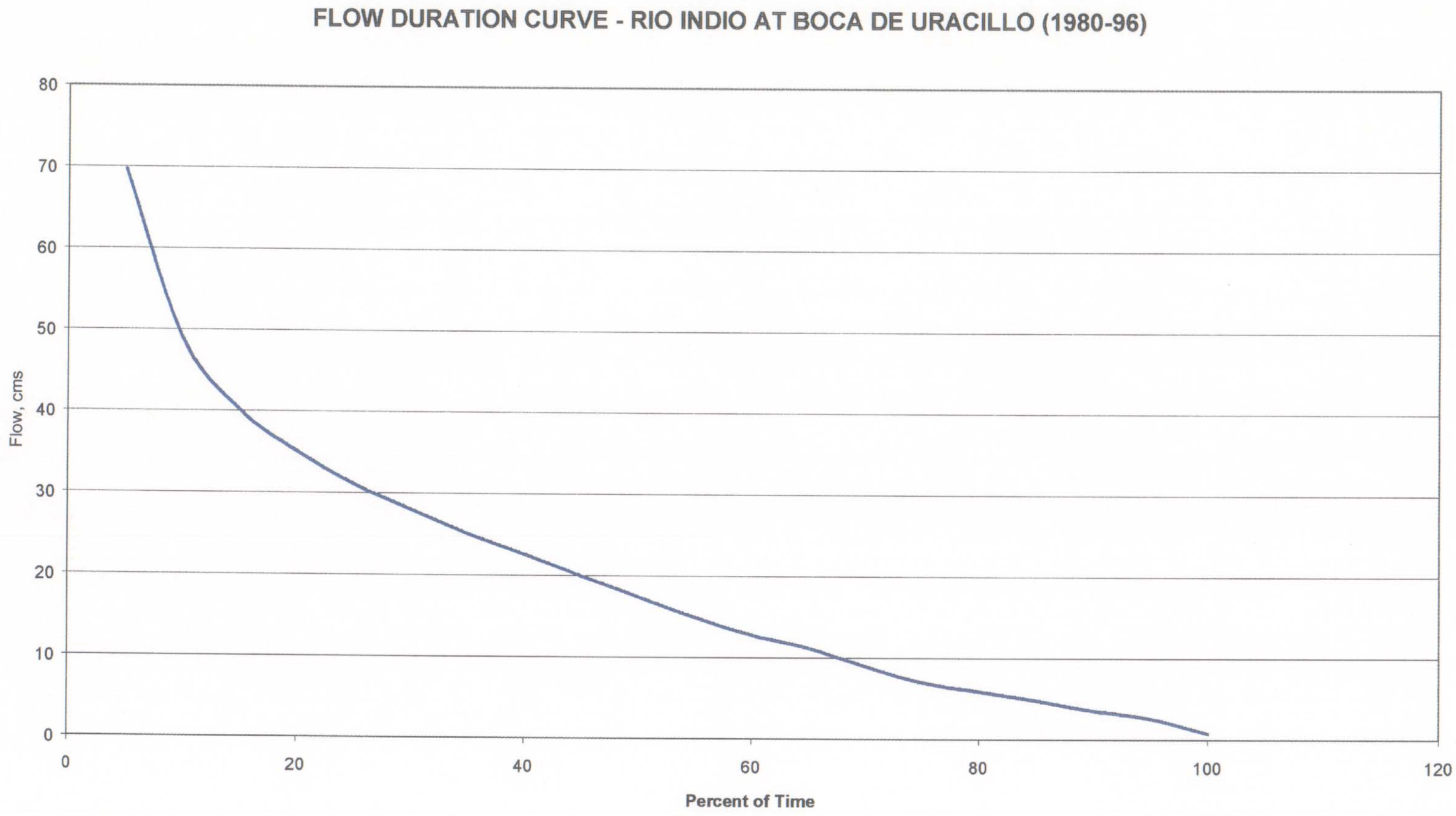
CONTRACT NO. CC-3-536
RIO INDIO WATER SUPPLY STUDY
APPENDIX A - HYDROLOGY AND METEOROLOGY

Yield vs. Reservoir Elevation
Rio Indio at Dam Site

MWH TAMS

DATE:	EXHIBIT:
APRIL, 2003	8



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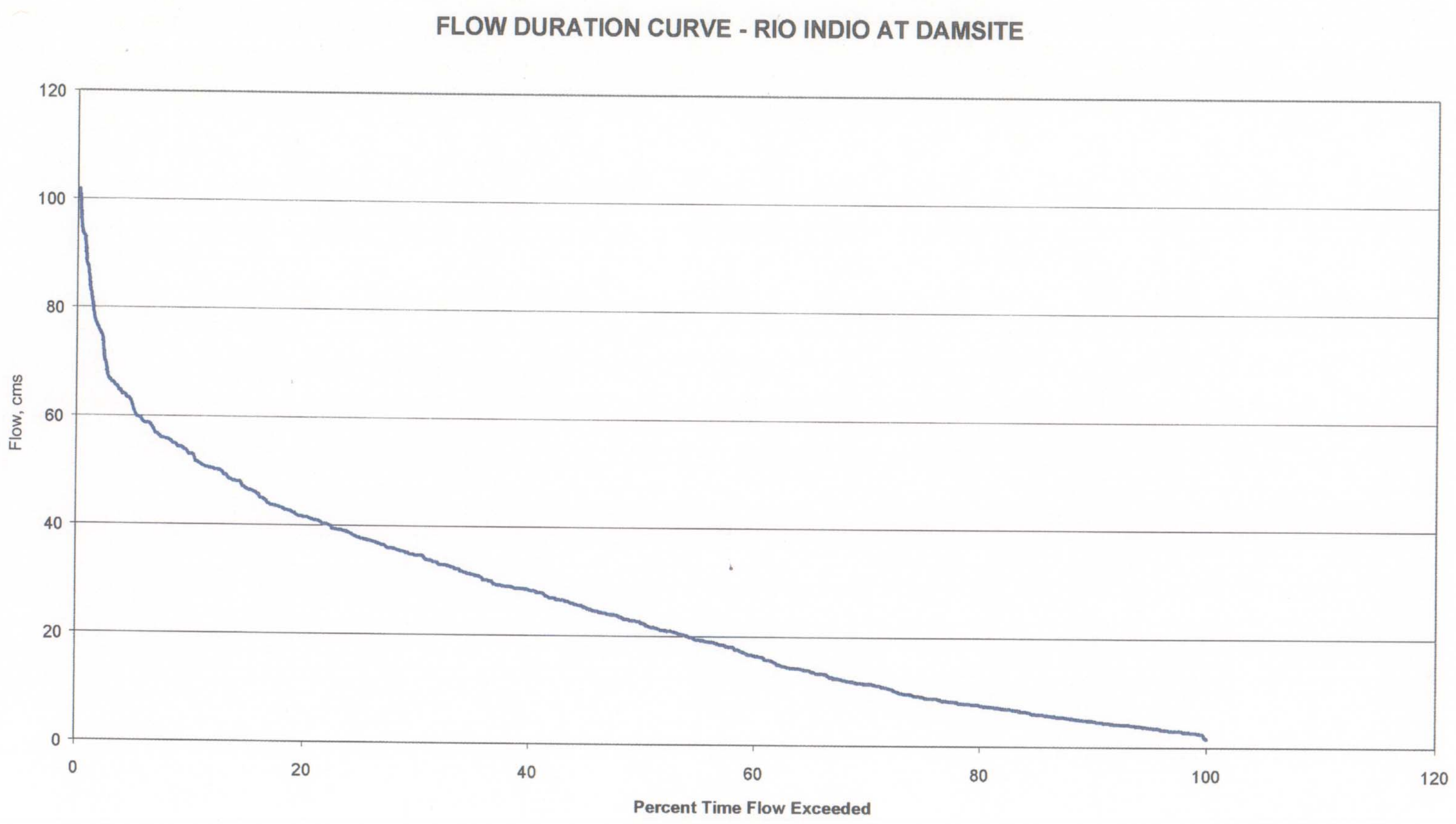
CONTRACT NO. CC-3-536
RIO INDIO WATER SUPPLY STUDY
APPENDIX A - HYDROLOGY AND METEOROLOGY

Flow Duration Curve - Rio Indio at Boca de Uracillo (Daily Flows)

MWH TAMS

DATE:	EXHIBIT:
APRIL, 2003	9



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Oficina de Proyectos de Capacidad del Canal

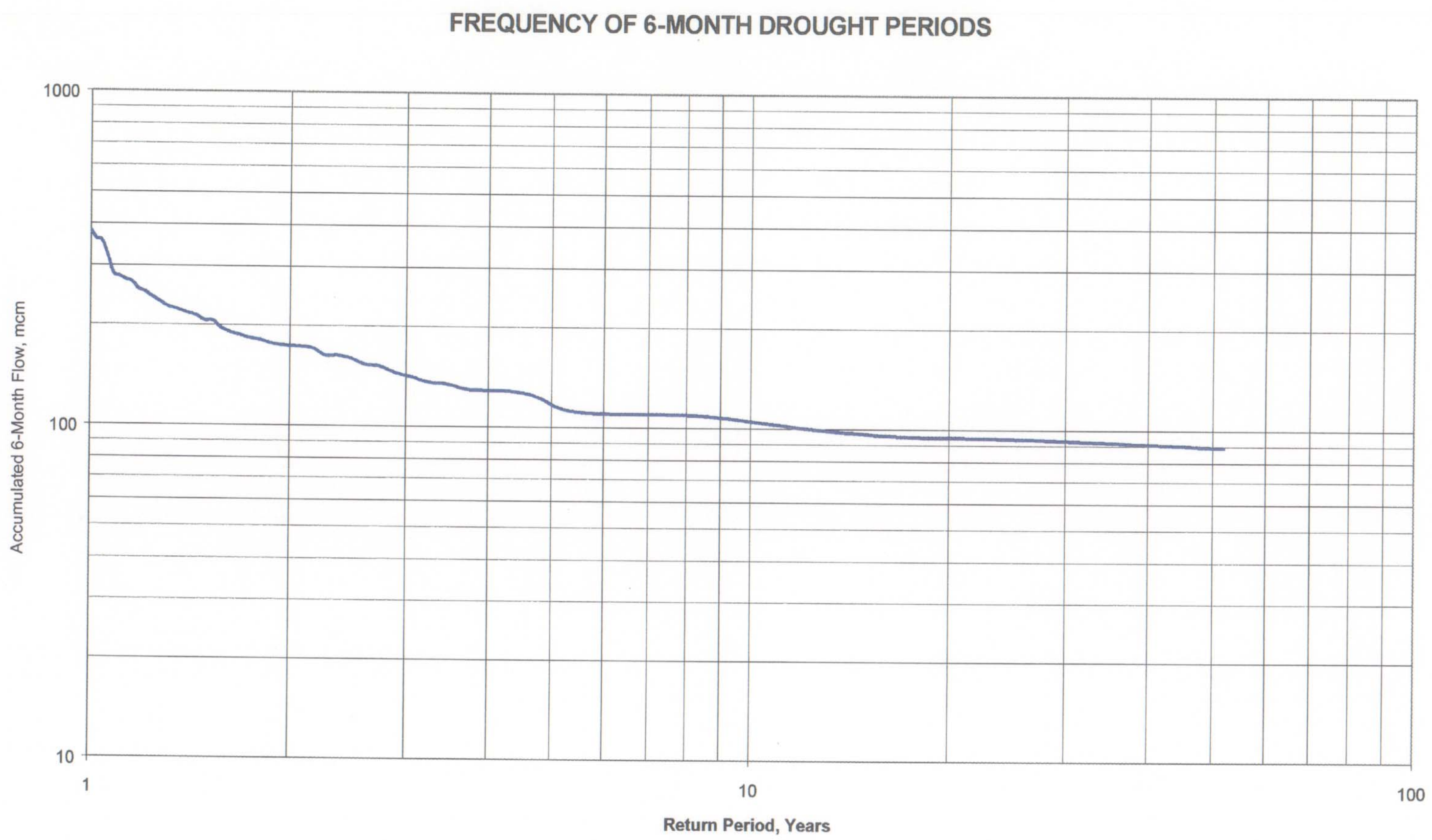
CONTRACT NO. CC-3-536
RIO INDIO WATER SUPPLY STUDY
APPENDIX A - HYDROLOGY AND METEOROLOGY

Flow Duration Curve - Rio Indio at Dam Site (Monthly Flows)

MWH TAMS

DATE: APRIL, 2003

EXHIBIT: 10



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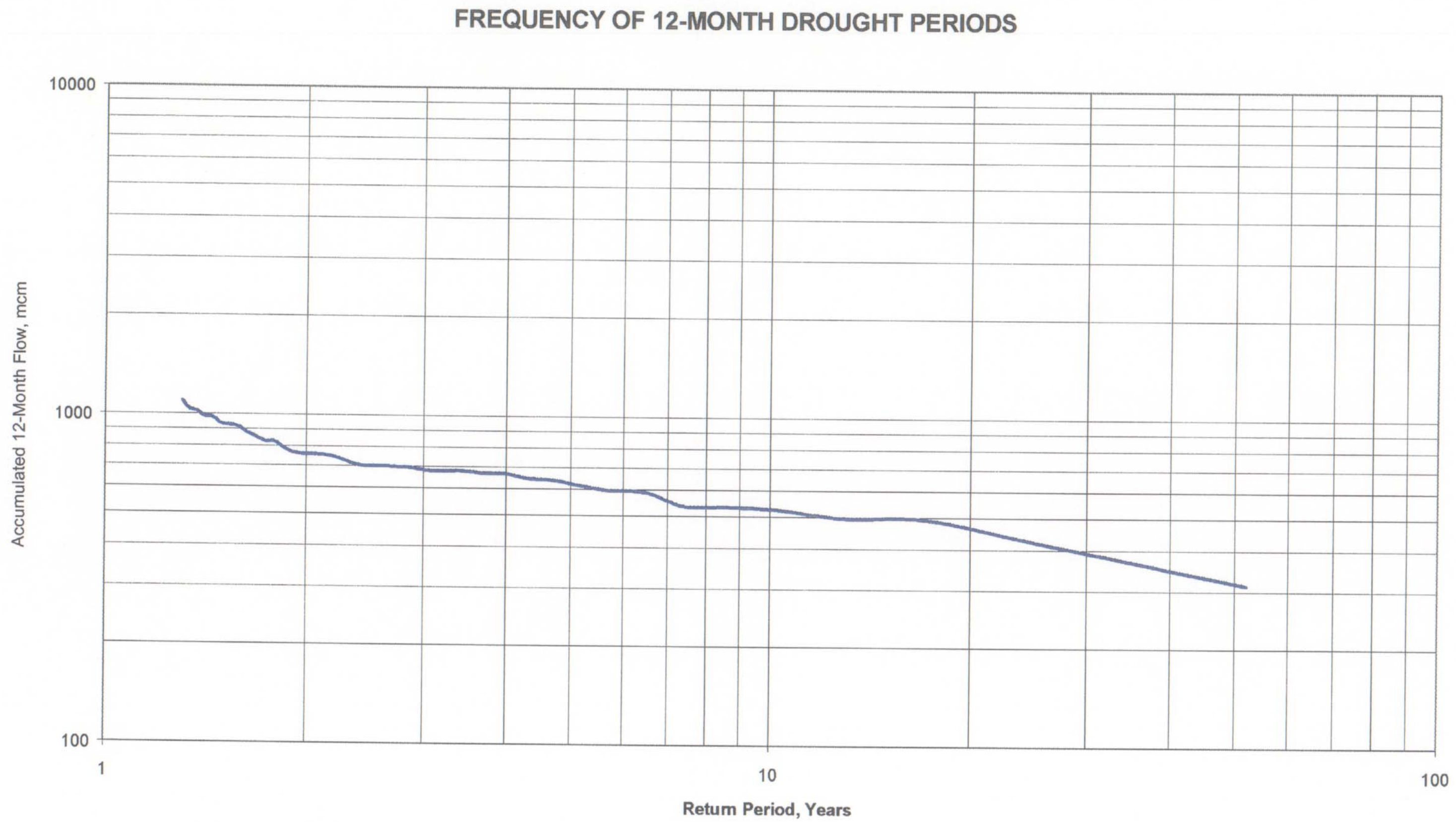
CONTRACT NO. CC-3-536
RIO INDIO WATER SUPPLY STUDY
APPENDIX A - HYDROLOGY AND METEOROLOGY

Frequency of 6-Month Drought Periods

MWH TAMS

DATE: APRIL, 2003

EXHIBIT: 11



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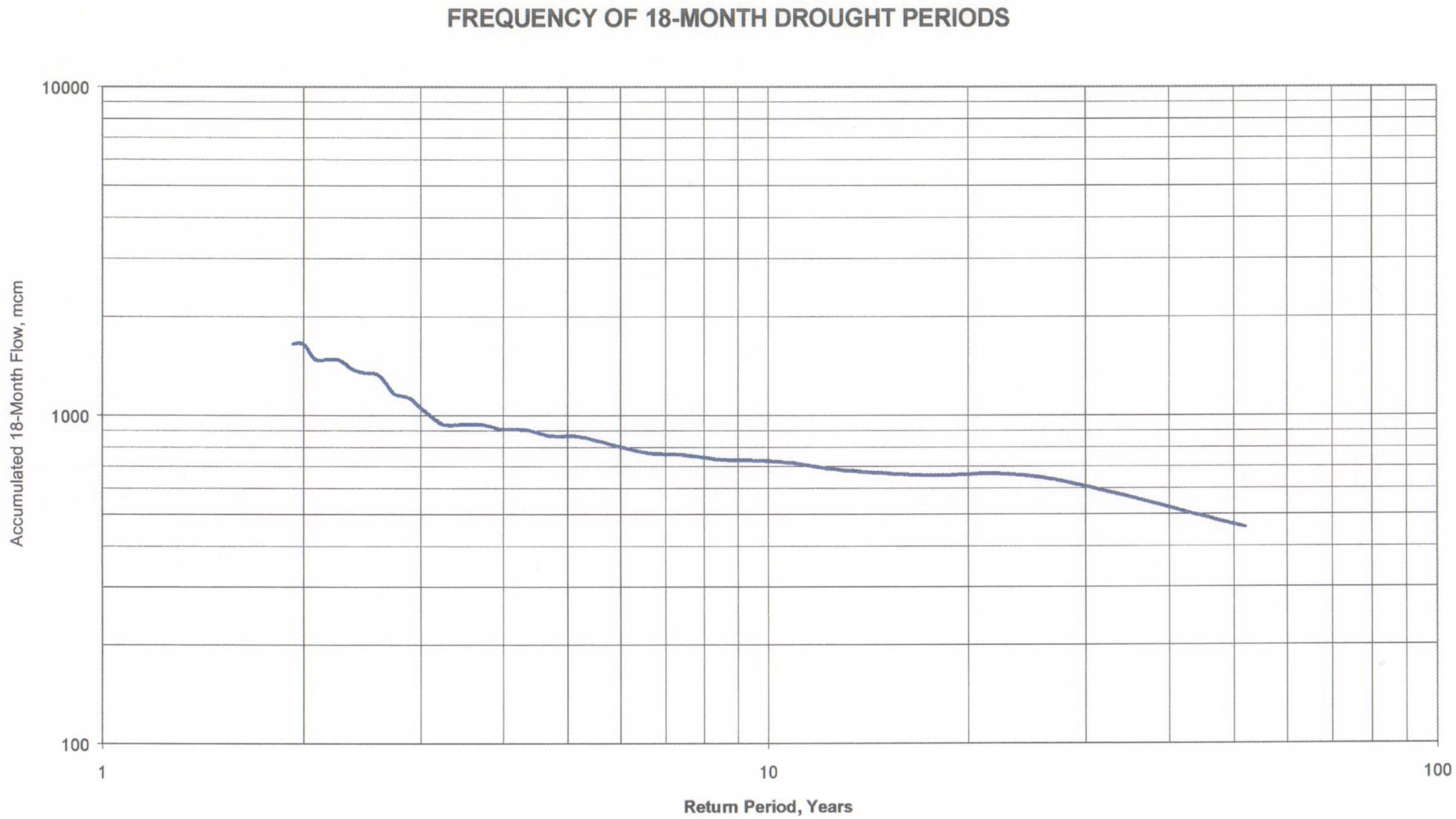
CONTRACT NO. CC-3-536
RIO INDIO WATER SUPPLY STUDY
APPENDIX A - HYDROLOGY AND METEOROLOGY

Frequency of 12-Month Drought Periods

MWH TAMS

DATE: APRIL, 2003

EXHIBIT: 12



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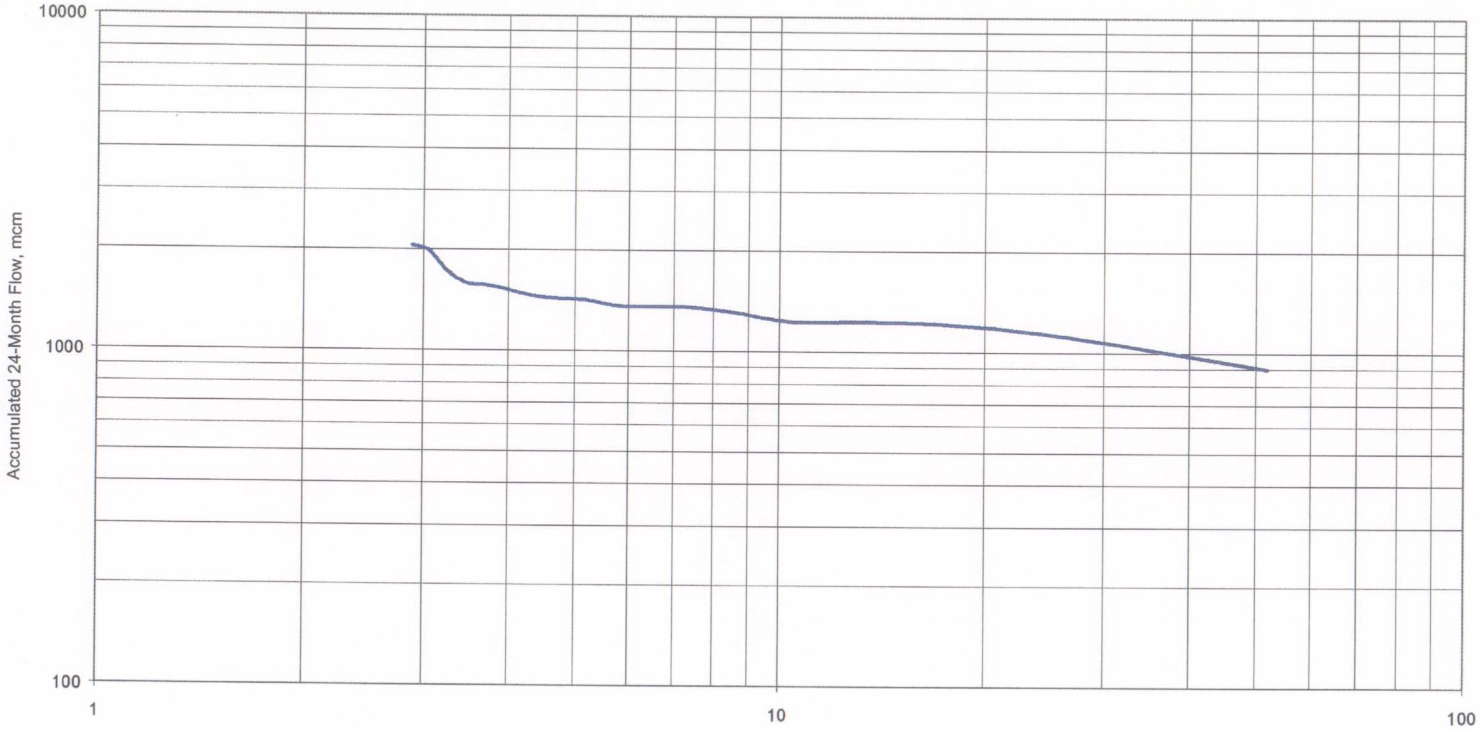
RIO INDIO WATER SUPPLY STUDY
APPENDIX A - HYDROLOGY AND METEOROLOGY

Frequency of 18-Month Drought Periods

MWH TAMS	DATE: APRIL, 2003	EXHIBIT: 13

FREQUENCY OF 24-MONTH DROUGHT PERIODS



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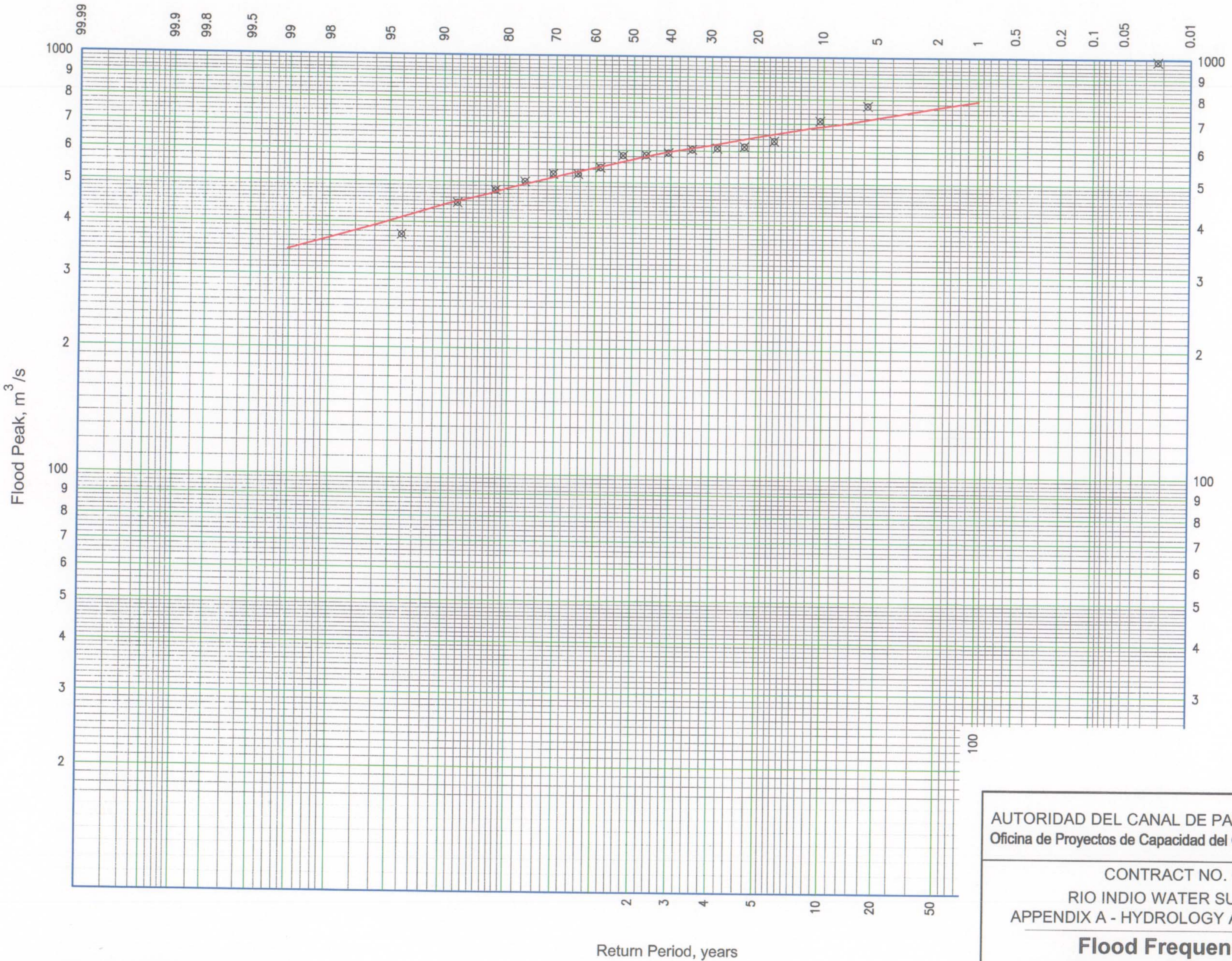
RIO INDIO WATER SUPPLY STUDY
APPENDIX A - HYDROLOGY AND METEOROLOGY

Frequency of 24-Month Drought Periods

MWH TAMS

DATE: APRIL, 2003

EXHIBIT: 14



Period: 1979-86
1988-95

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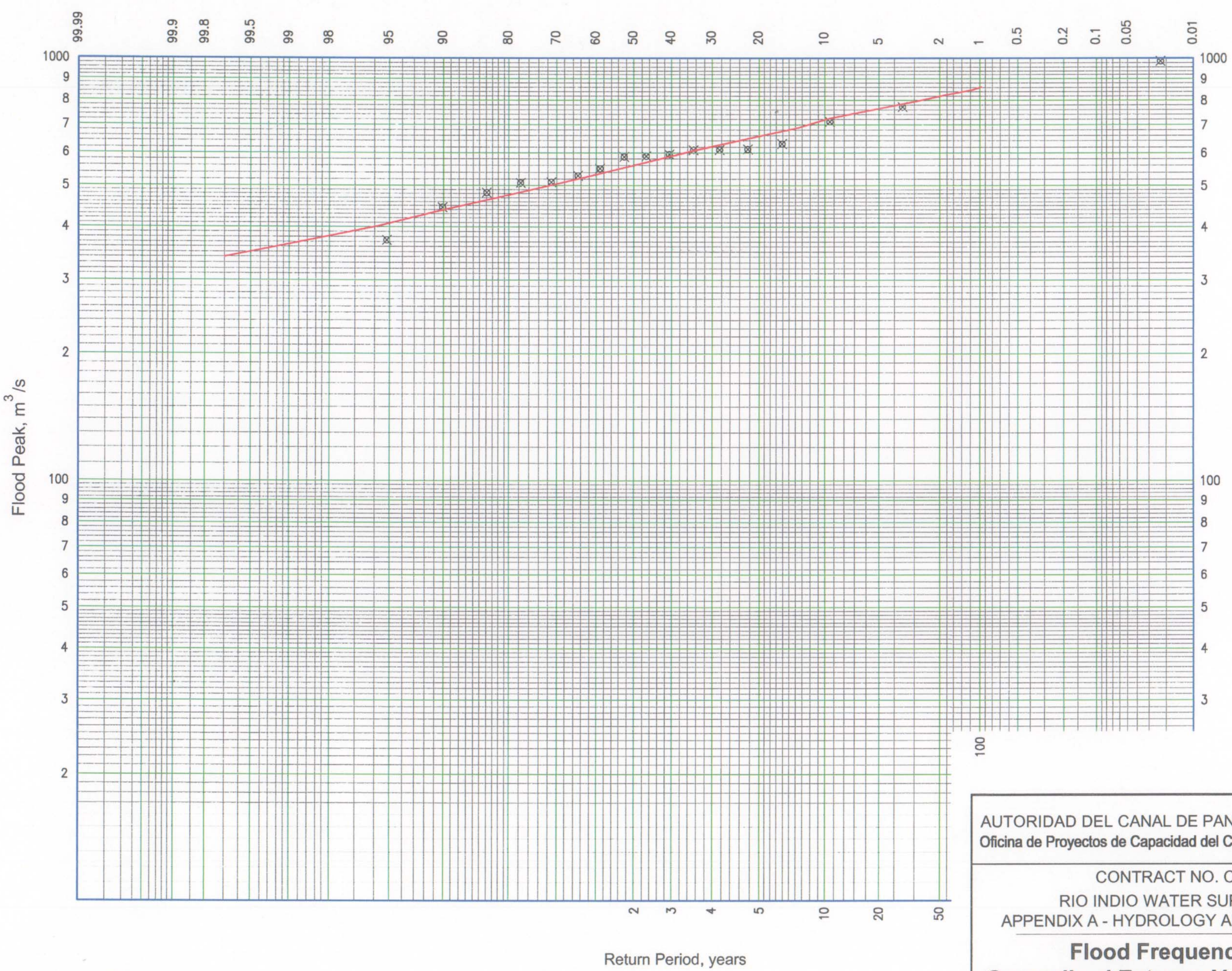
CONTRACT NO. CC-3-536
RIO INDIO WATER SUPPLY STUDY
APPENDIX A - HYDROLOGY AND METEOROLOGY

Flood Frequency Curve
Log Pearson Type III Distribution

MWH TAMS

DATE: APRIL, 2003

EXHIBIT: 15



Period: 1979-86
1988-95

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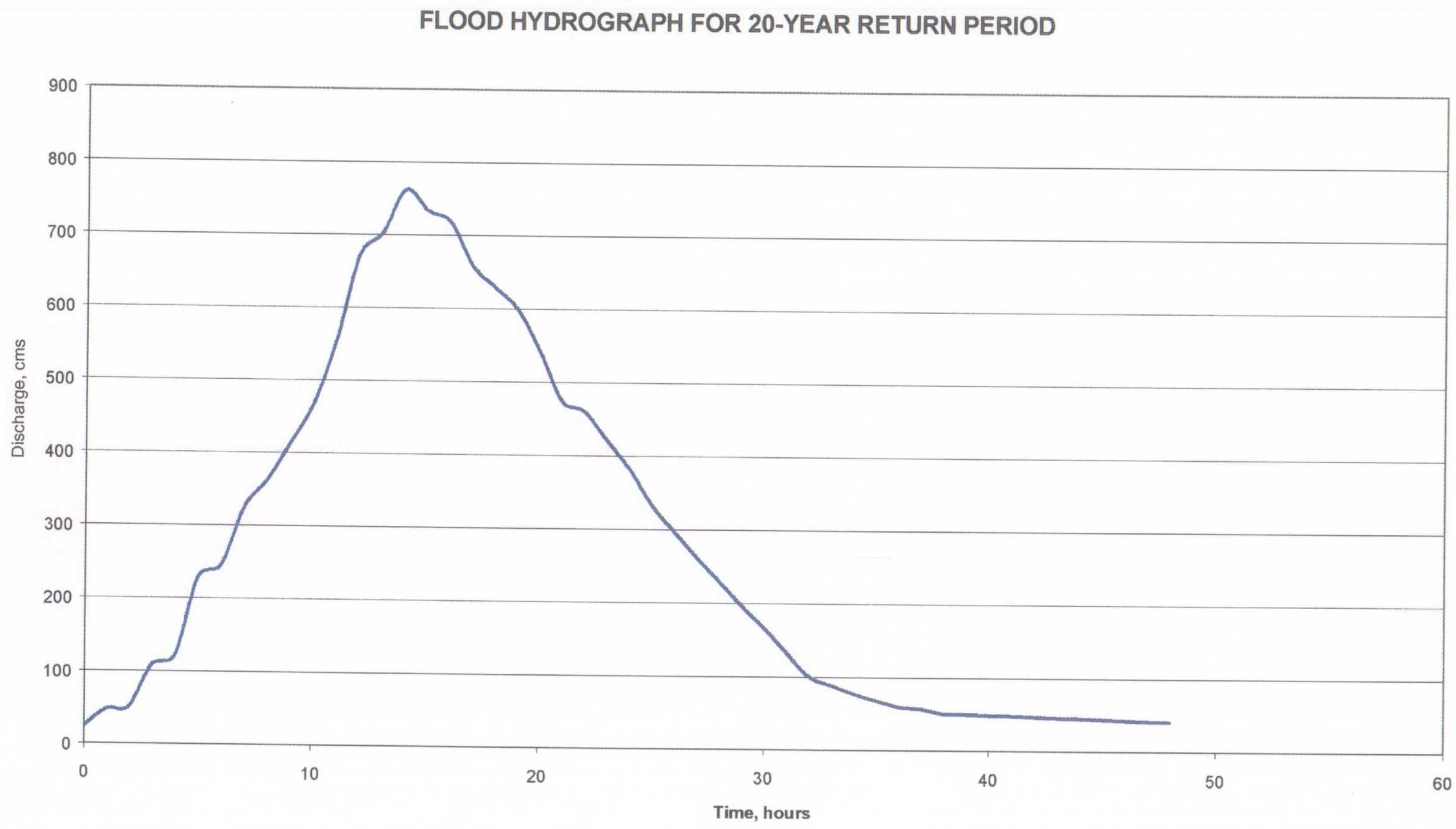
CONTRACT NO. CC-3-536
RIO INDIO WATER SUPPLY STUDY
APPENDIX A - HYDROLOGY AND METEOROLOGY

Flood Frequency Curve
Generalized Extreme Value Distribution

MWH TAMS

DATE: APRIL, 2003

EXHIBIT: 16



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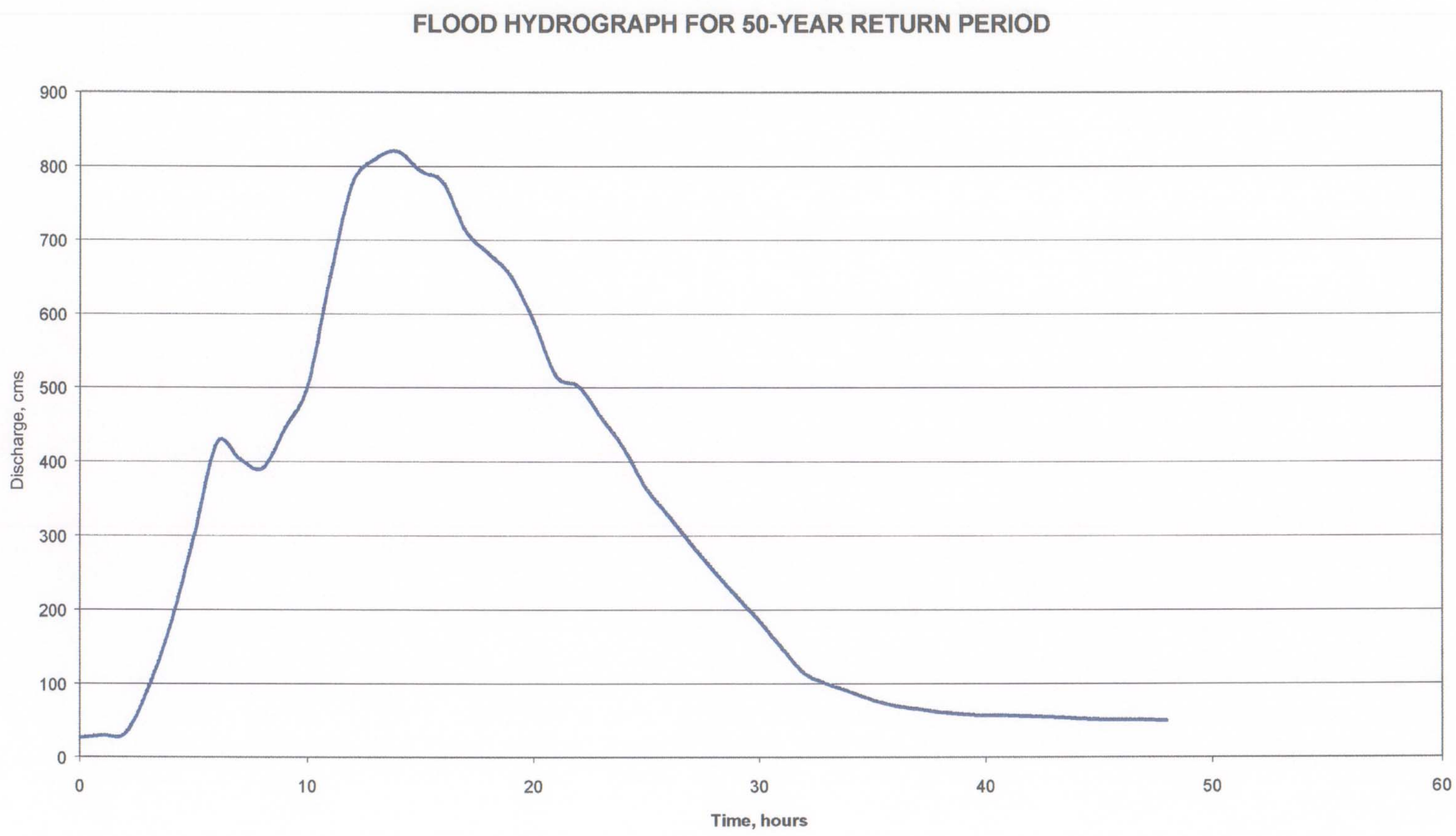
RIO INDIO WATER SUPPLY STUDY
APPENDIX A - HYDROLOGY AND METEOROLOGY

Flood Hydrograph for 20-Year Return Period

MWH TAMS

DATE: APRIL, 2003

EXHIBIT: 17



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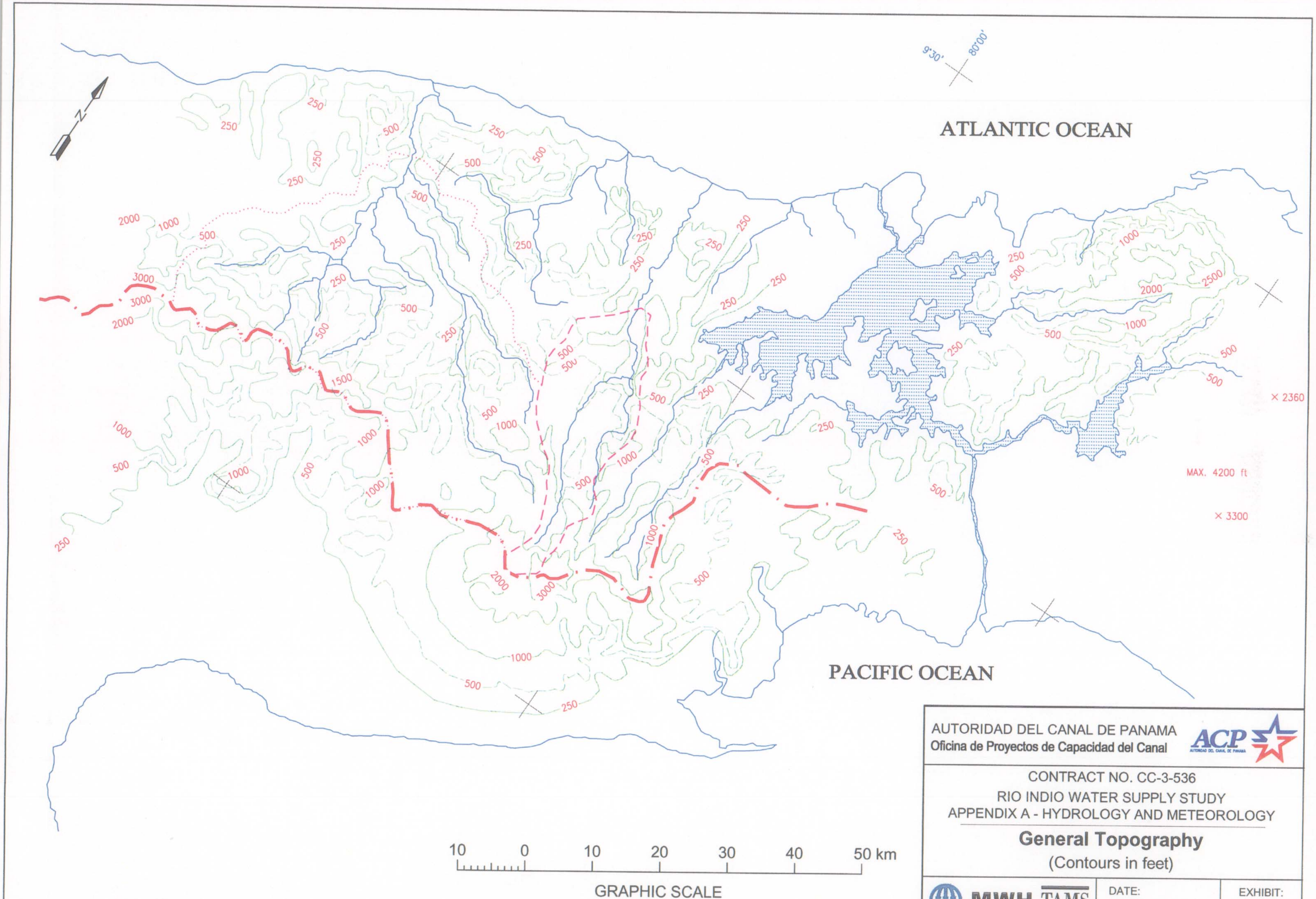
CONTRACT NO. CC-3-536
RIO INDIO WATER SUPPLY STUDY
APPENDIX A - HYDROLOGY AND METEOROLOGY

Flood Hydrograph for 50-Year Return Period

MWH TAMS

DATE:	EXHIBIT:
APRIL, 2003	18



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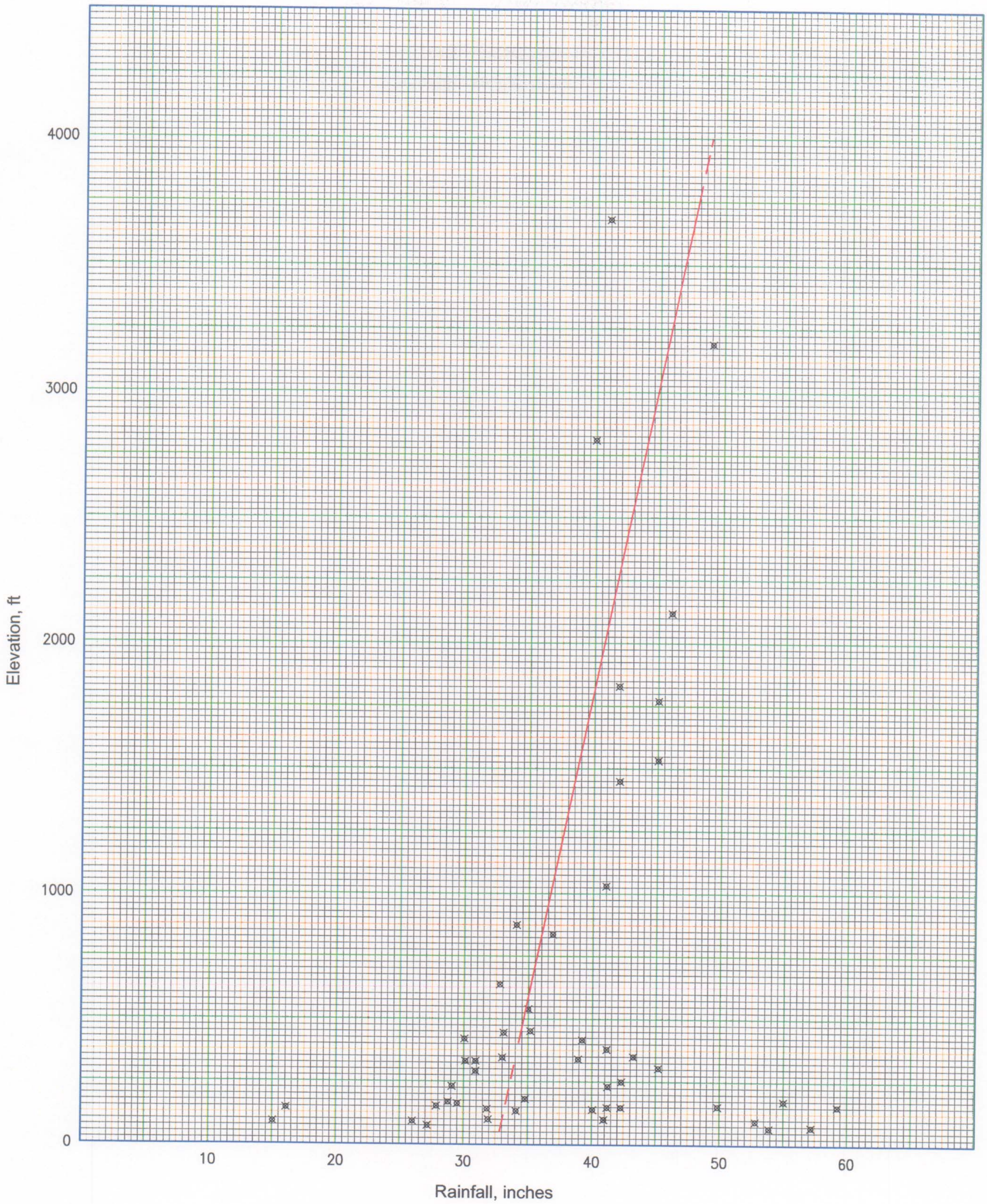
CONTRACT NO. CC-3-536
RIO INDIO WATER SUPPLY STUDY
APPENDIX A - HYDROLOGY AND METEOROLOGY

General Topography
(Contours in feet)

MWH TAMS

DATE:	EXHIBIT:
APRIL, 2003	19



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CONTRACT NO. CC-3-536
RIO INDIO WATER SUPPLY STUDY
APPENDIX A - HYDROLOGY AND METEOROLOGY

Elevation vs. Mean Oct-Dec Rainfall

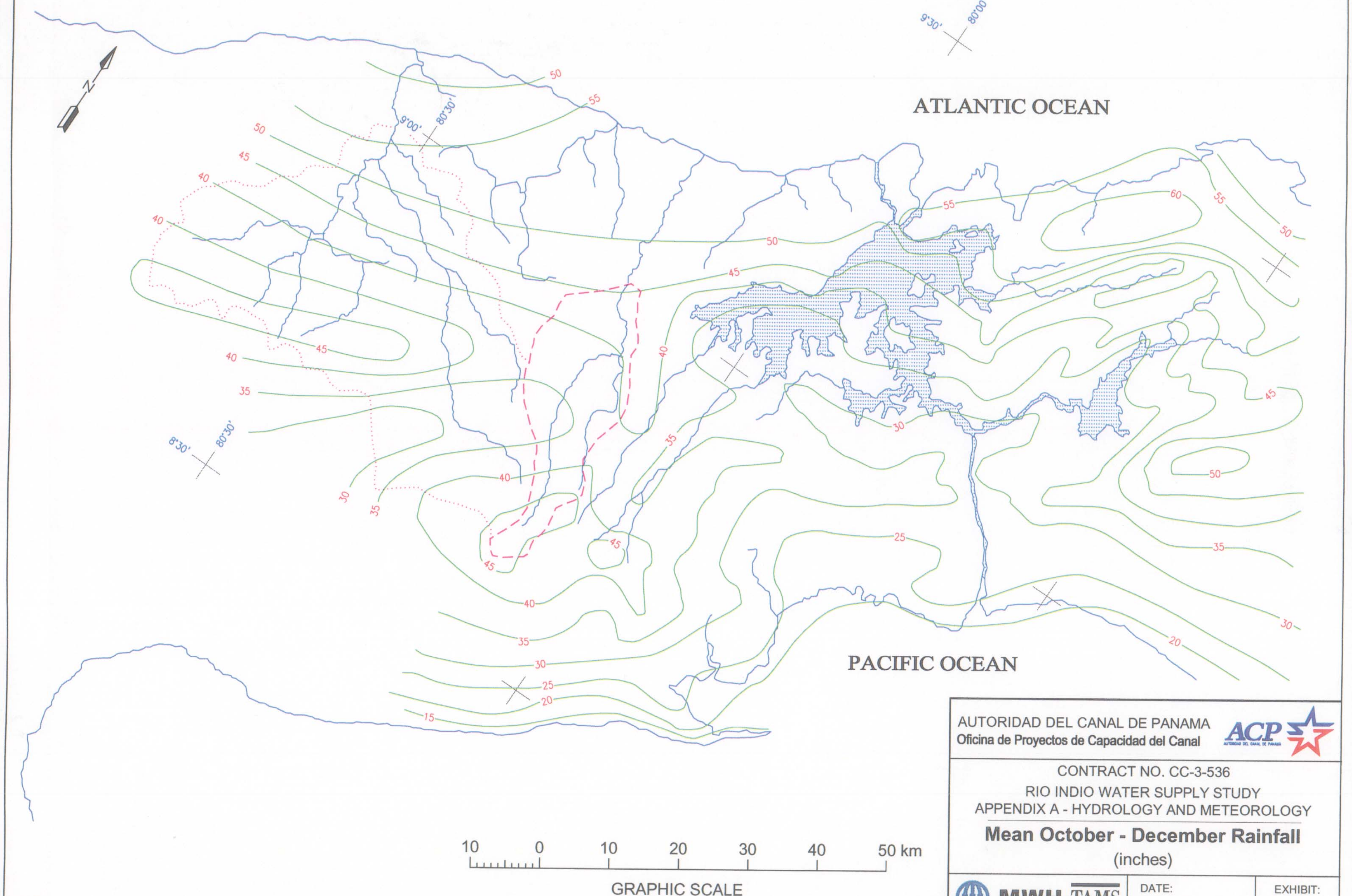
MWH TAMS

DATE: APRIL, 2003

EXHIBIT: 20

Source:

US NWS, February 1978 Report



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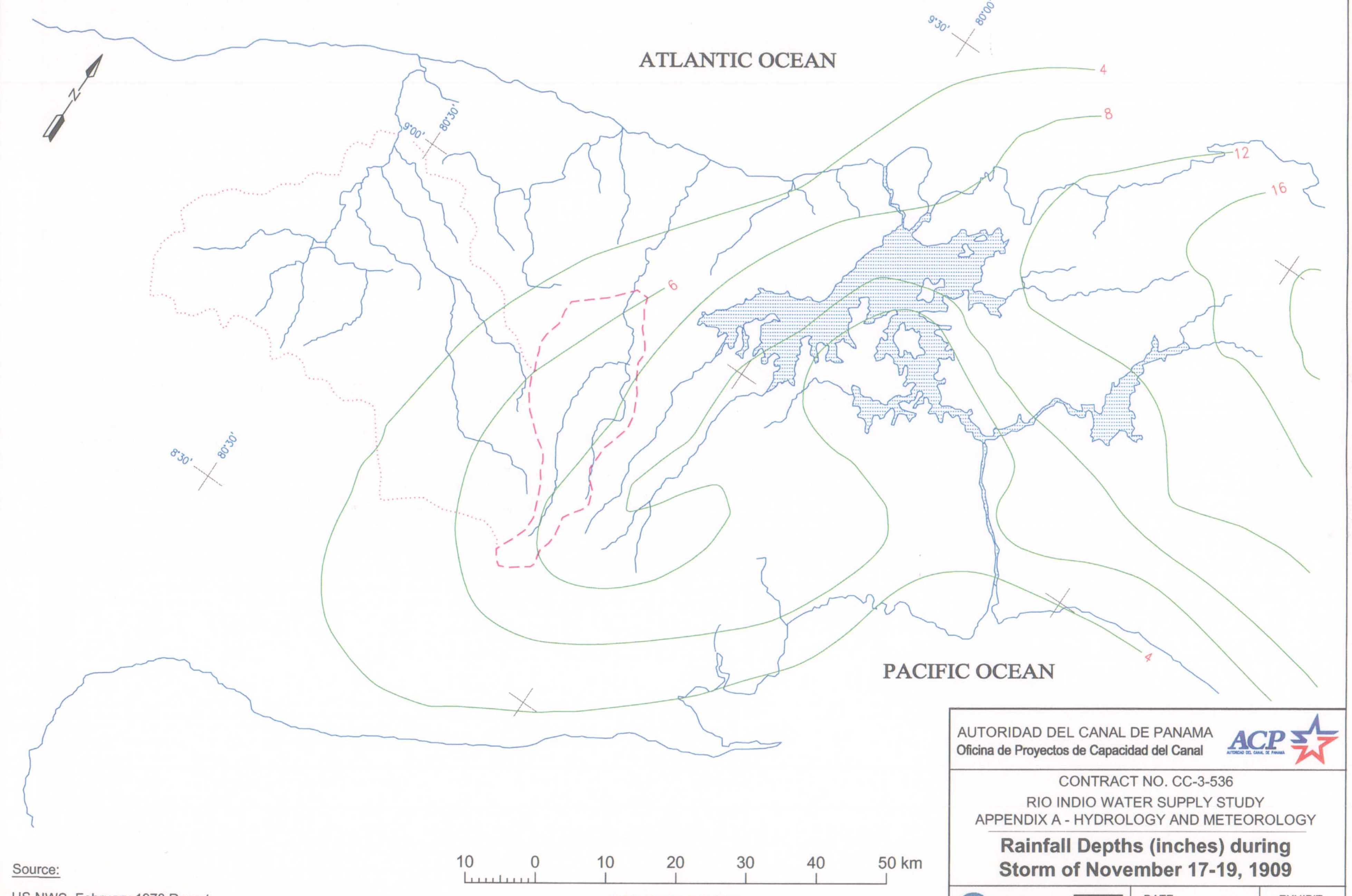
CONTRACT NO. CC-3-536
RIO INDIO WATER SUPPLY STUDY
APPENDIX A - HYDROLOGY AND METEOROLOGY

Mean October - December Rainfall
(inches)

MWH TAMS

DATE: APRIL, 2003

EXHIBIT: 21



Source:

US NWS, February 1978 Report

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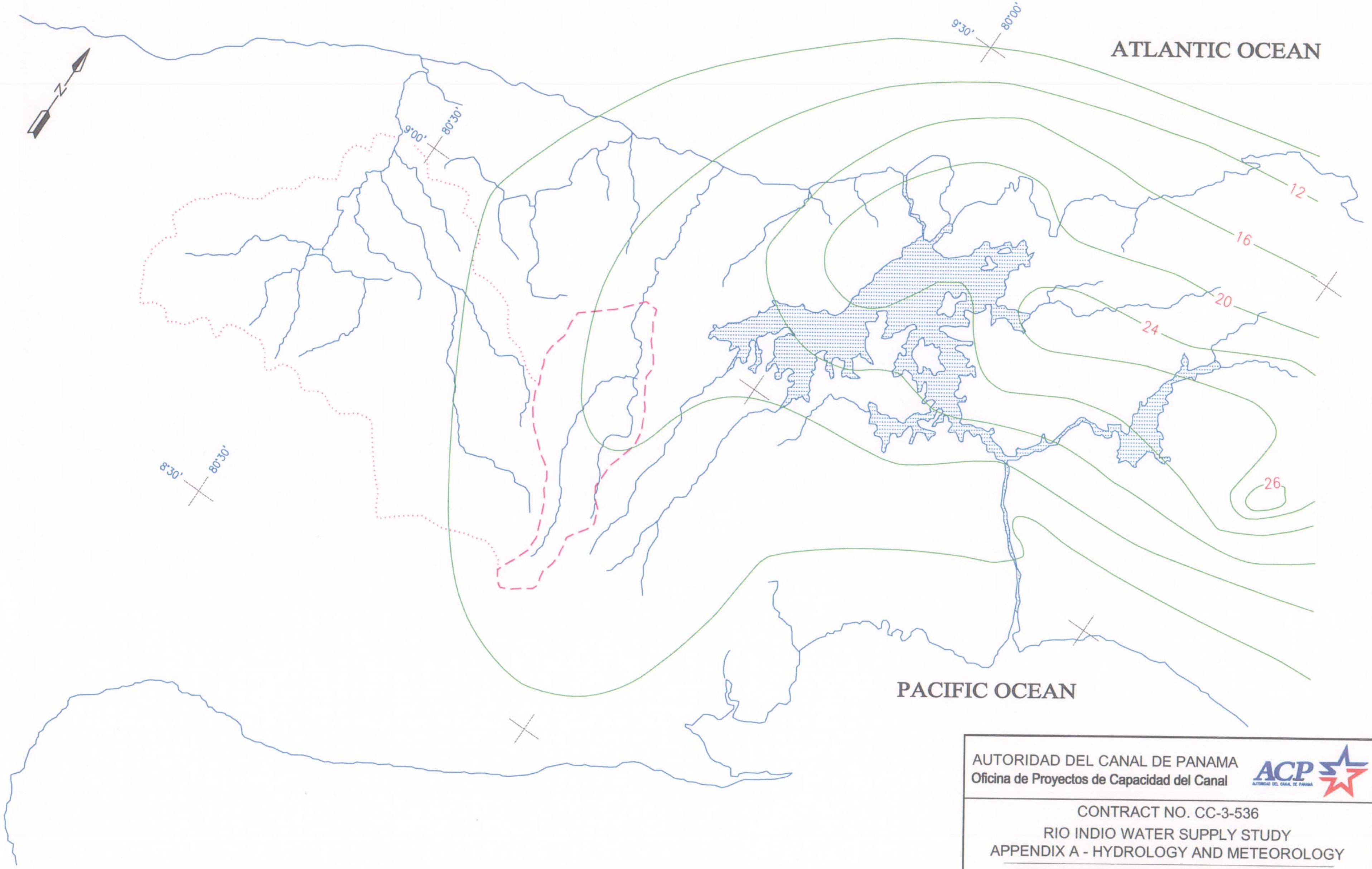
APPENDIX A - HYDROLOGY AND METEOROLOGY

Rainfall Depths (inches) during Storm of November 17-19, 1909

MWH TAMS

DATE: APRIL, 2003

EXHIBIT: 22



Source:

US NWS, February 1978 Report

10 0 10 20 30 40 50 km

GRAPHIC SCALE

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

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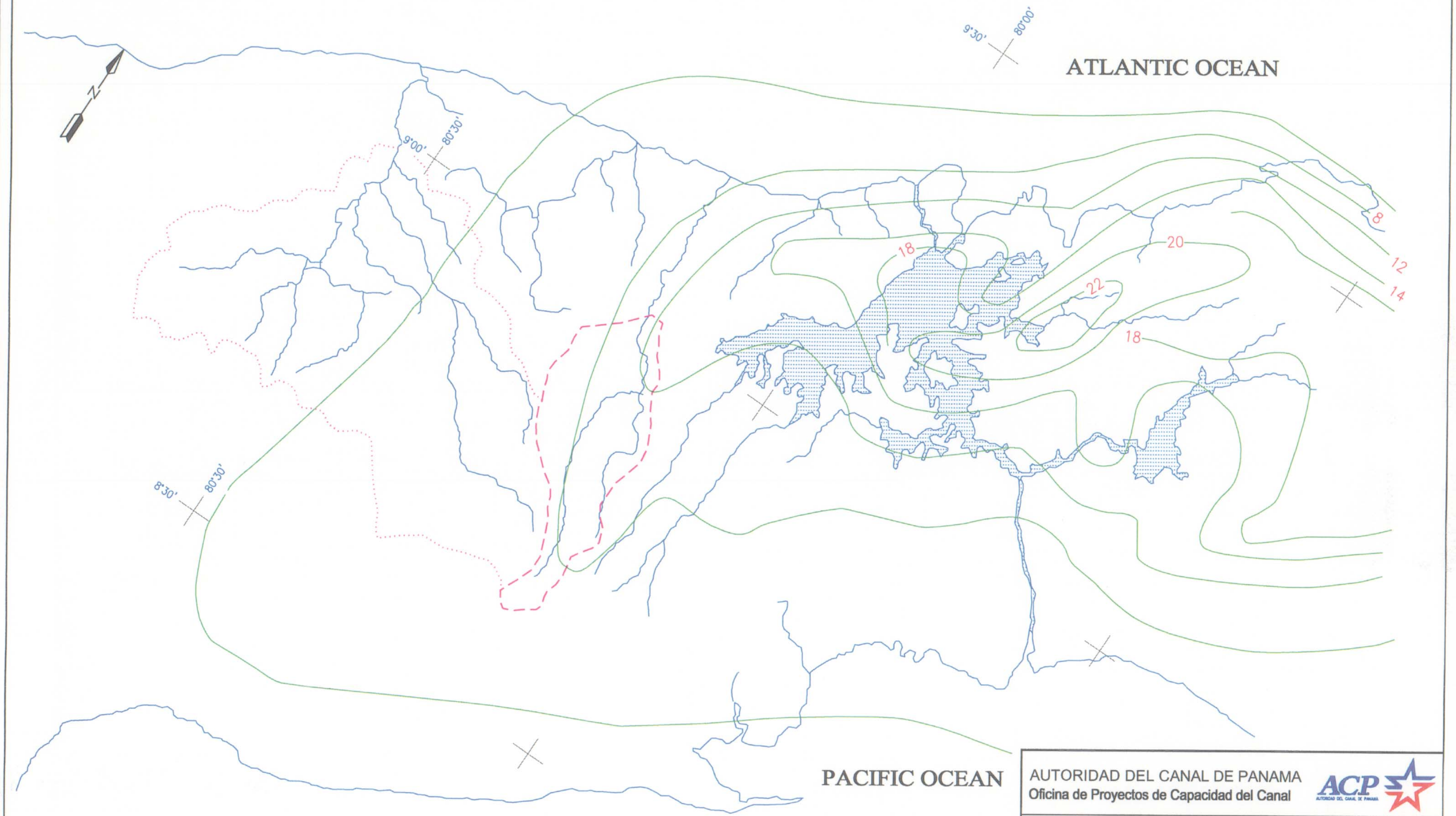
APPENDIX A - HYDROLOGY AND METEOROLOGY

Rainfall Depths (inches) during Storm of October 22-24, 1923

MWH TAMS

DATE: APRIL, 2003

EXHIBIT: 23



Source:

US NWS, February 1978 Report

10 0 10 20 30 40 50 km

GRAPHIC SCALE

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

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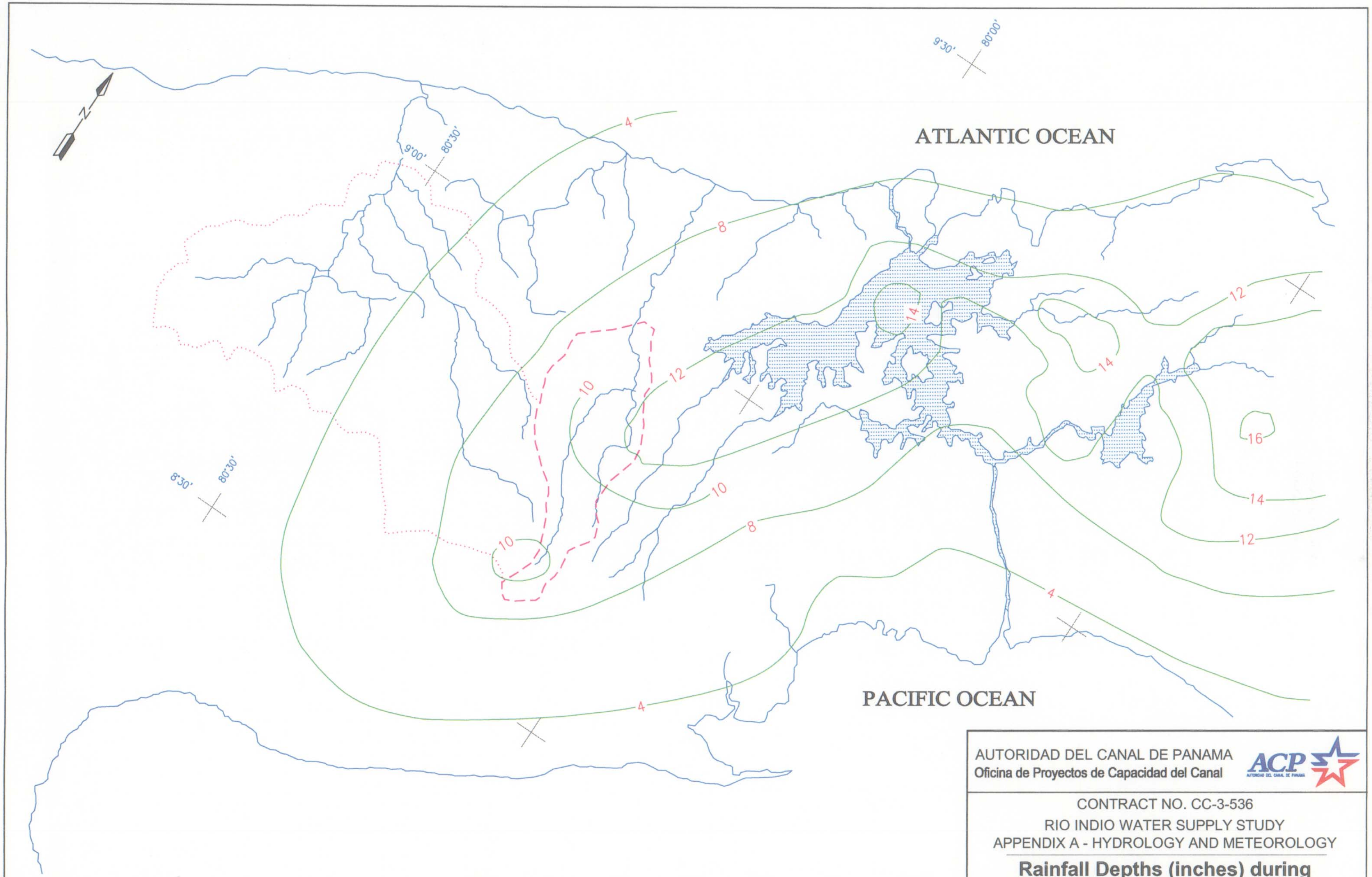
CONTRACT NO. CC-3-536
RIO INDIO WATER SUPPLY STUDY
APPENDIX A - HYDROLOGY AND METEOROLOGY

Rainfall Depths (inches) during Storm of November 7-9, 1931

MWH TAMS

DATE: APRIL, 2003

EXHIBIT: 24



Source:

US NWS, February 1978 Report

AUTORIDAD DEL CANAL DE PANAMA
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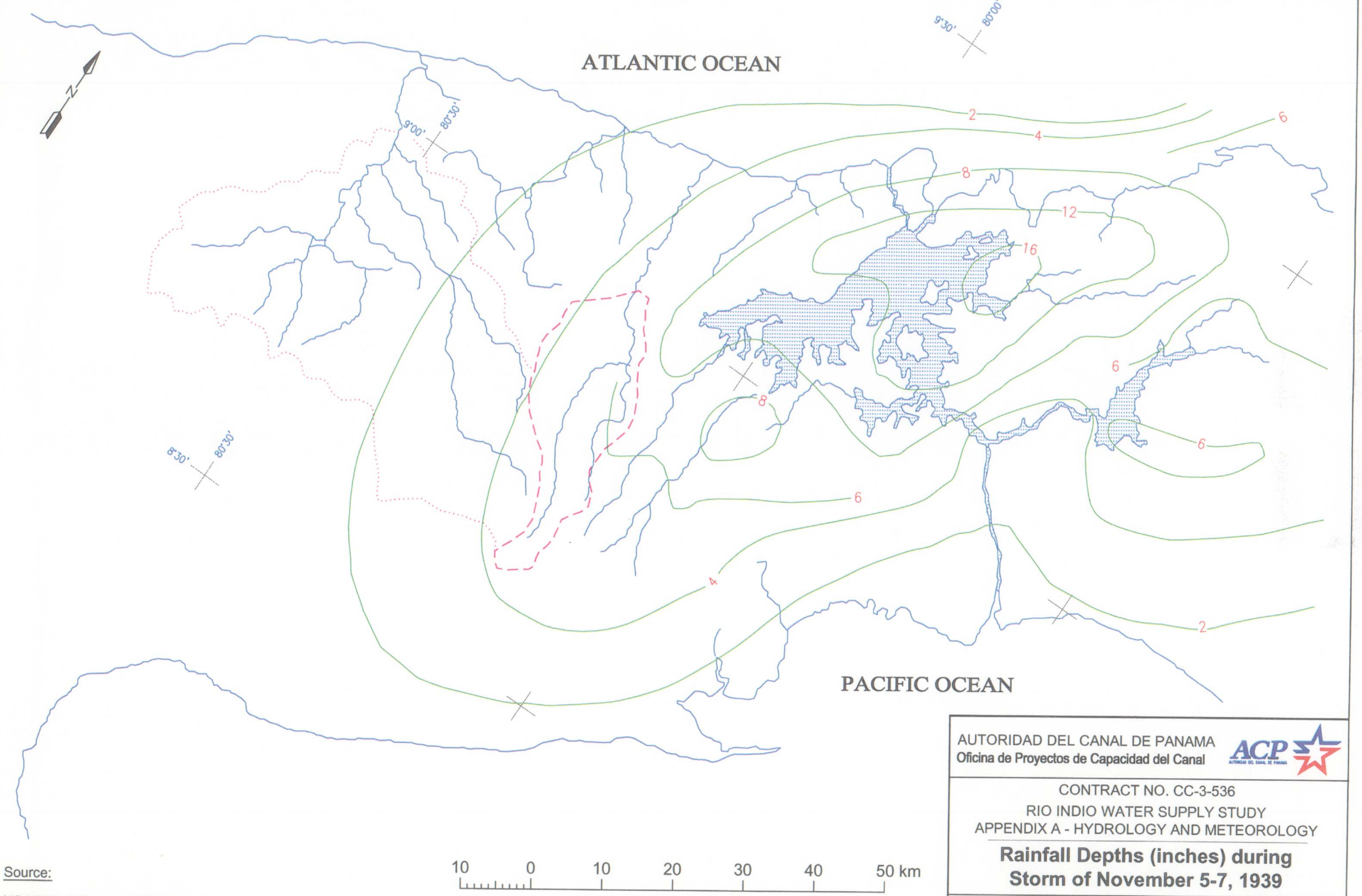
CONTRACT NO. CC-3-536
RIO INDIO WATER SUPPLY STUDY
APPENDIX A - HYDROLOGY AND METEOROLOGY

Rainfall Depths (inches) during Storm of November 27-29, 1932

MWH TAMS

DATE: APRIL, 2003

EXHIBIT: 25



Source:

US NWS, February 1978 Report

AUTORIDAD DEL CANAL DE PANAMA
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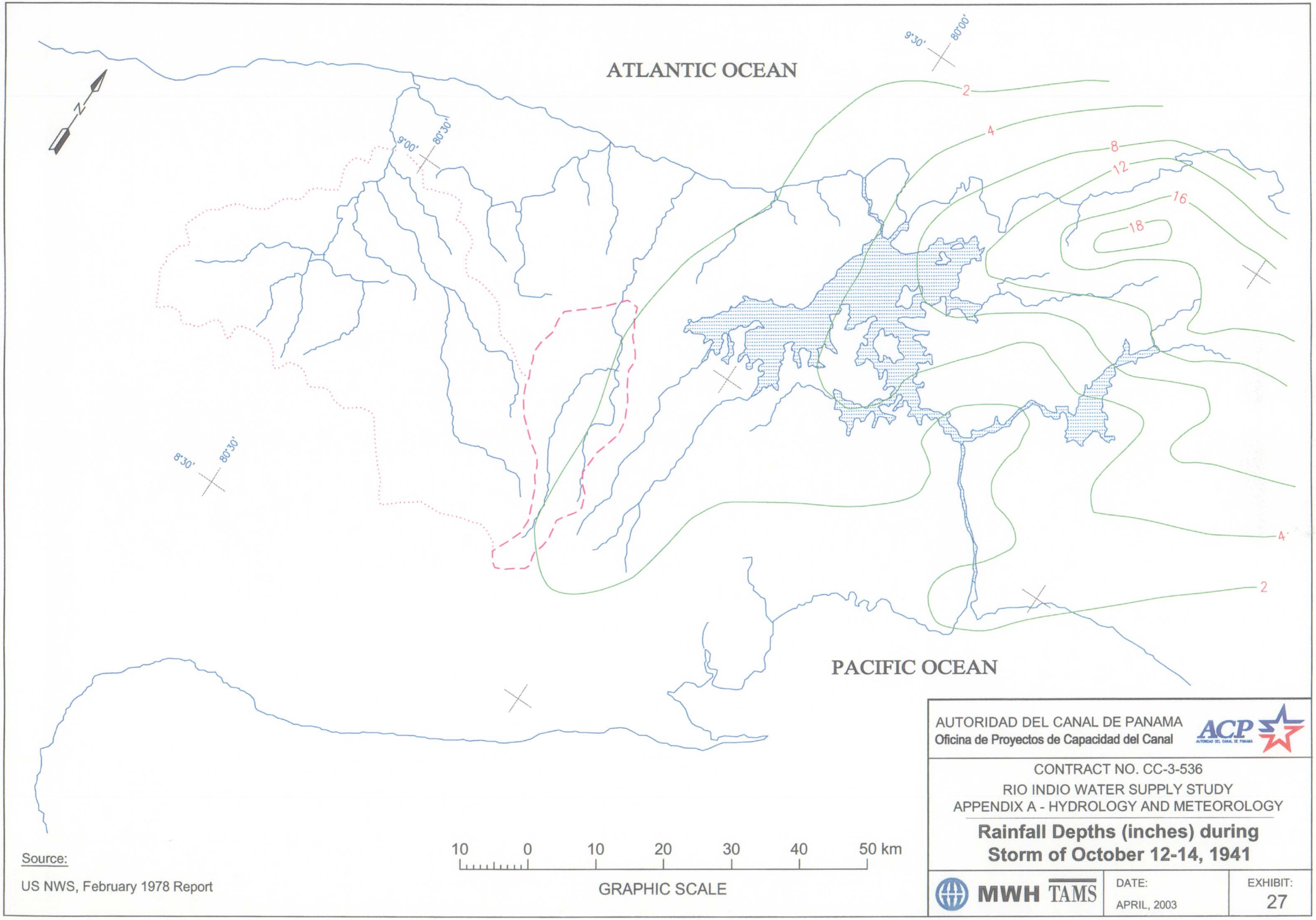
CONTRACT NO. CC-3-536
RIO INDIO WATER SUPPLY STUDY
APPENDIX A - HYDROLOGY AND METEOROLOGY

Rainfall Depths (inches) during Storm of November 5-7, 1939

MWH TAMS

DATE: APRIL, 2003

EXHIBIT: 26



Source:
US NWS, February 1978 Report

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Oficina de Proyectos de Capacidad del Canal

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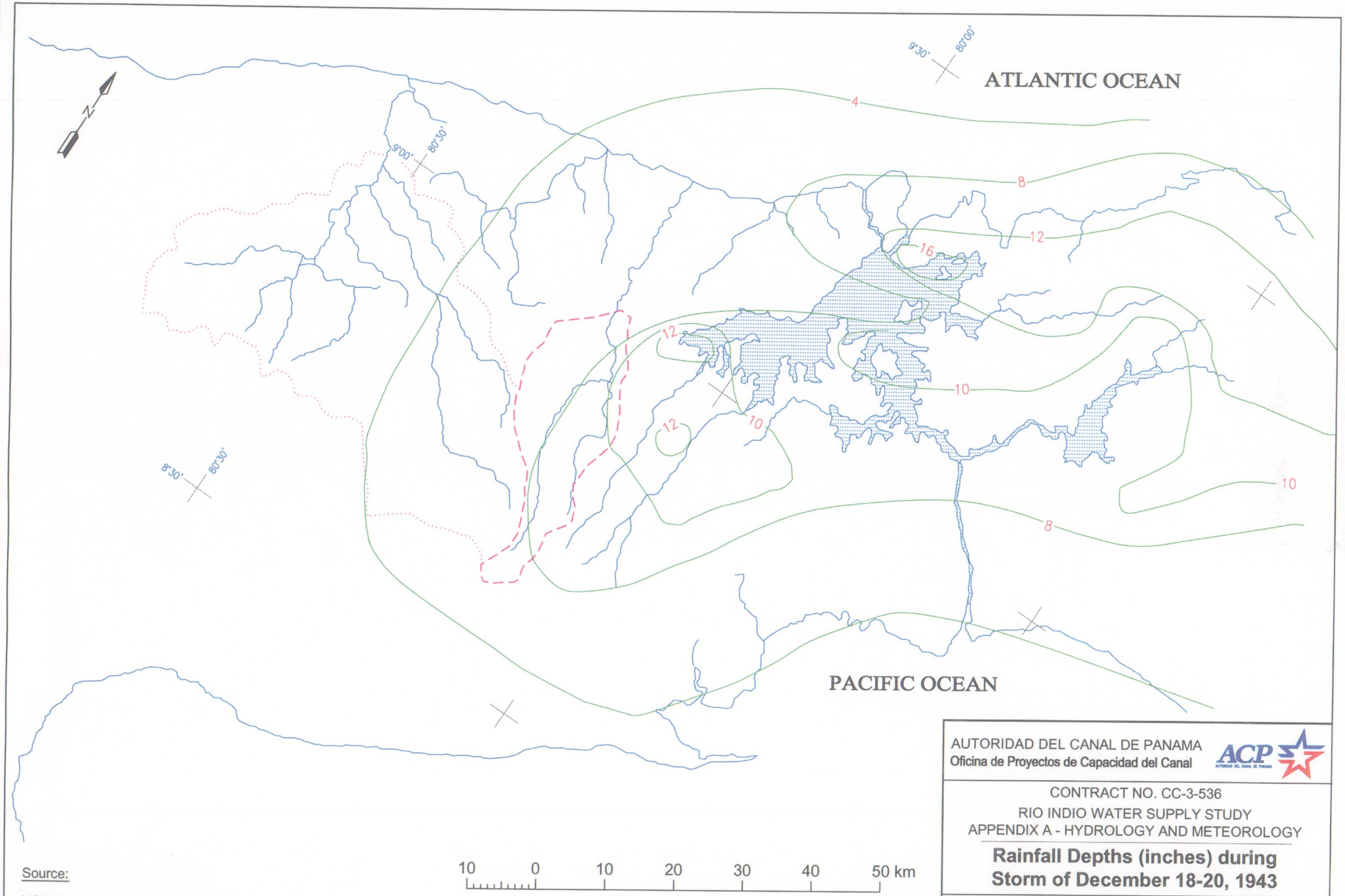
RIO INDIO WATER SUPPLY STUDY
APPENDIX A - HYDROLOGY AND METEOROLOGY

Rainfall Depths (inches) during Storm of October 12-14, 1941

MWH TAMS

DATE: APRIL, 2003

EXHIBIT: 27



Source:
US NWS, February 1978 Report

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Oficina de Proyectos de Capacidad del Canal

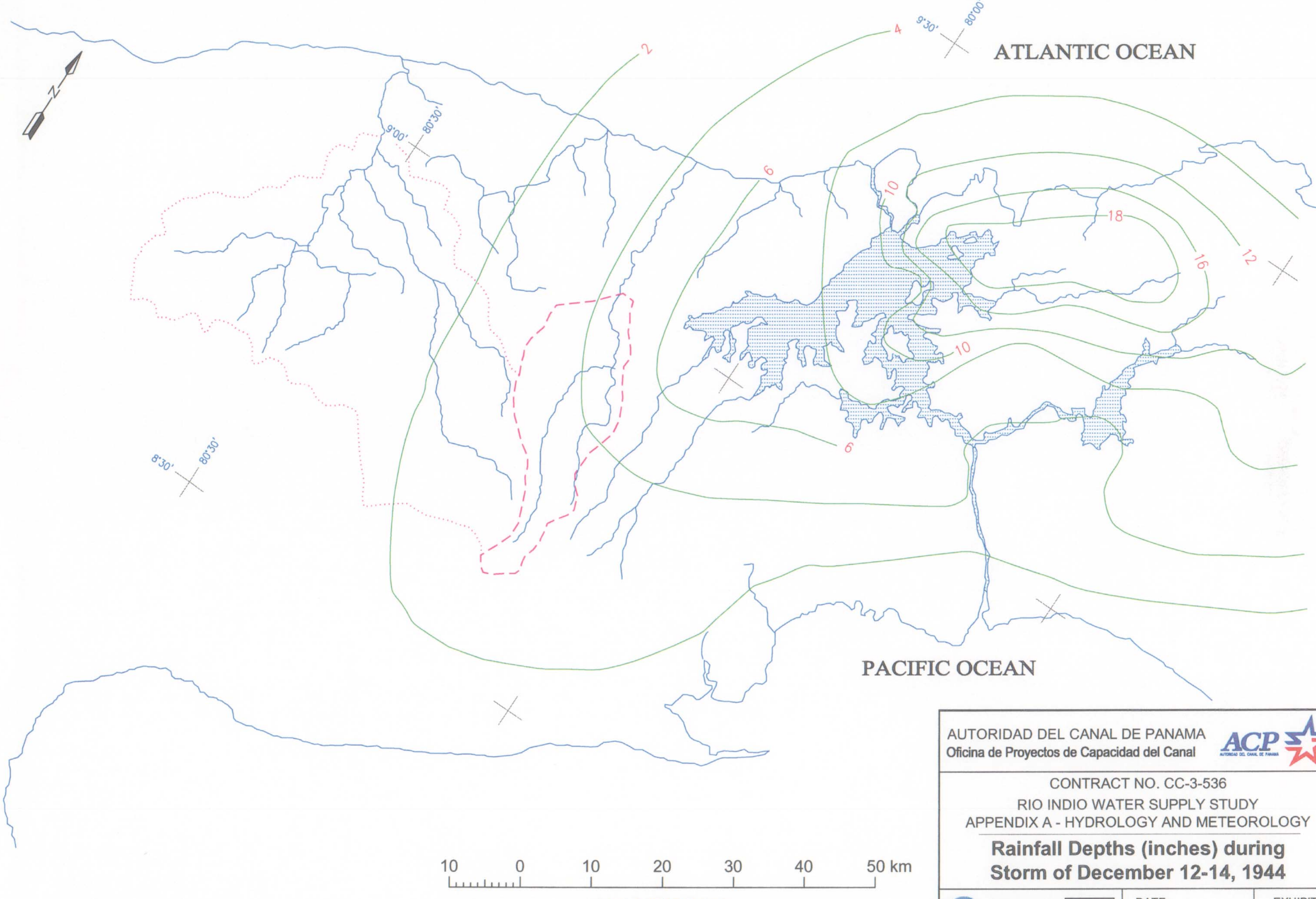
CONTRACT NO. CC-3-536
RIO INDIO WATER SUPPLY STUDY
APPENDIX A - HYDROLOGY AND METEOROLOGY

Rainfall Depths (inches) during Storm of December 18-20, 1943

MWH TAMS

DATE: APRIL, 2003

EXHIBIT: 28



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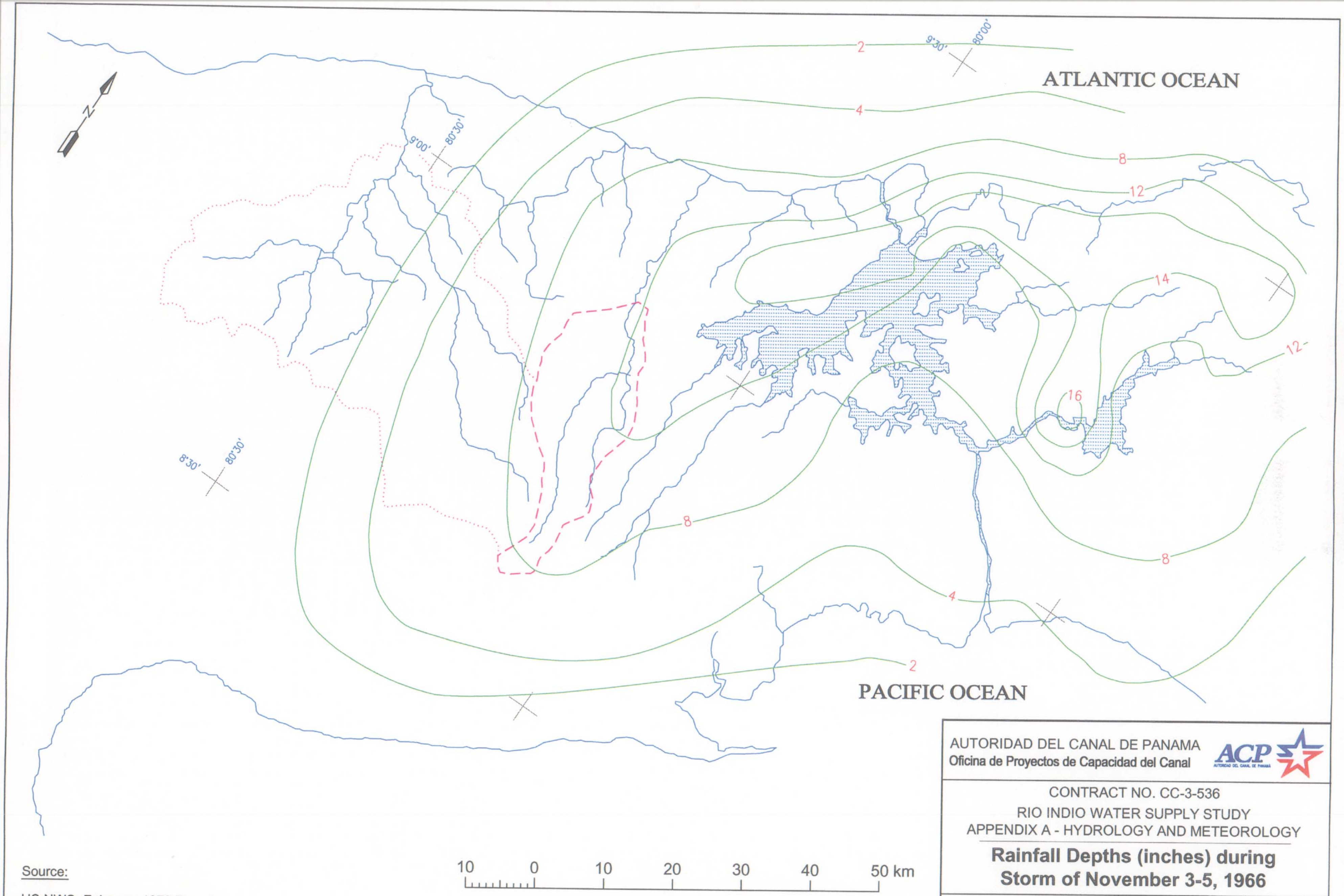
CONTRACT NO. CC-3-536
RIO INDIO WATER SUPPLY STUDY
APPENDIX A - HYDROLOGY AND METEOROLOGY

Rainfall Depths (inches) during Storm of December 12-14, 1944

MWH TAMS

DATE: APRIL, 2003

EXHIBIT: 29



Source:

US NWS, February 1978 Report

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

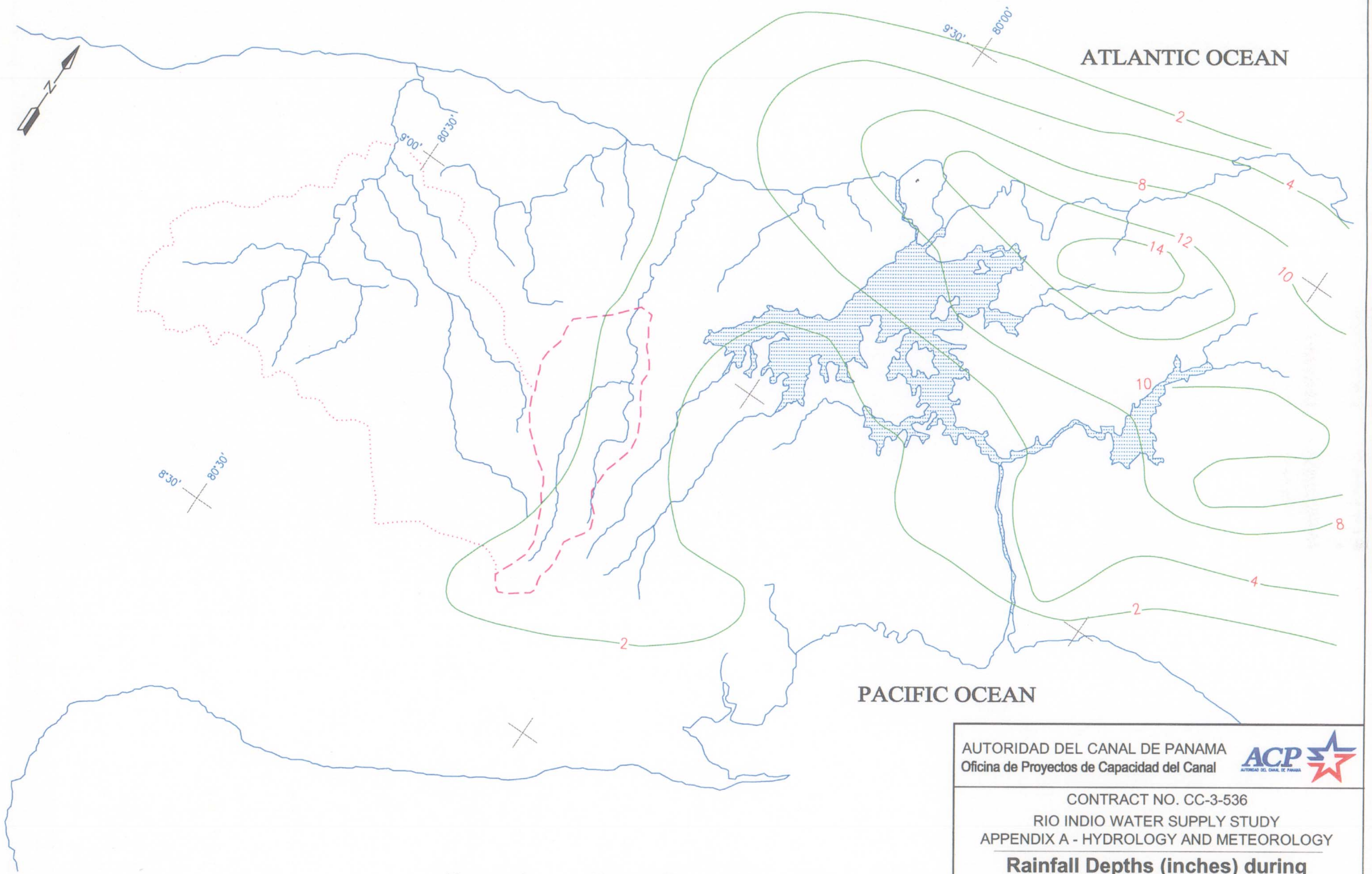
CONTRACT NO. CC-3-536
RIO INDIO WATER SUPPLY STUDY
APPENDIX A - HYDROLOGY AND METEOROLOGY

Rainfall Depths (inches) during Storm of November 3-5, 1966

MWH TAMS

DATE: APRIL, 2003

EXHIBIT: 30



Source:

US NWS, February 1978 Report

10 0 10 20 30 40 50 km

GRAPHIC SCALE

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

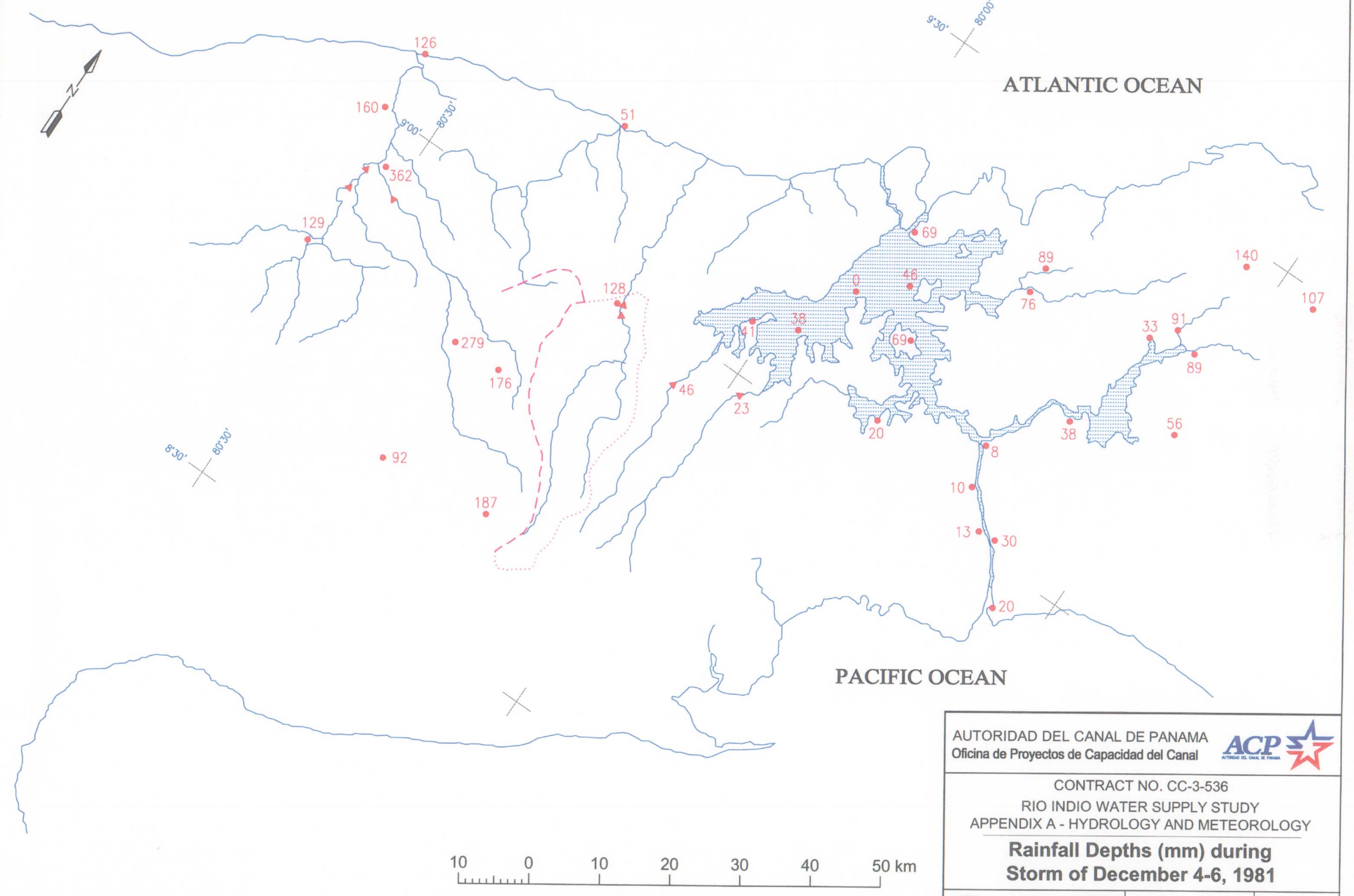
CONTRACT NO. CC-3-536
RIO INDIO WATER SUPPLY STUDY
APPENDIX A - HYDROLOGY AND METEOROLOGY

Rainfall Depths (inches) during Storm of April 7-9, 1970

MWH TAMS

DATE: APRIL, 2003

EXHIBIT: 31



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

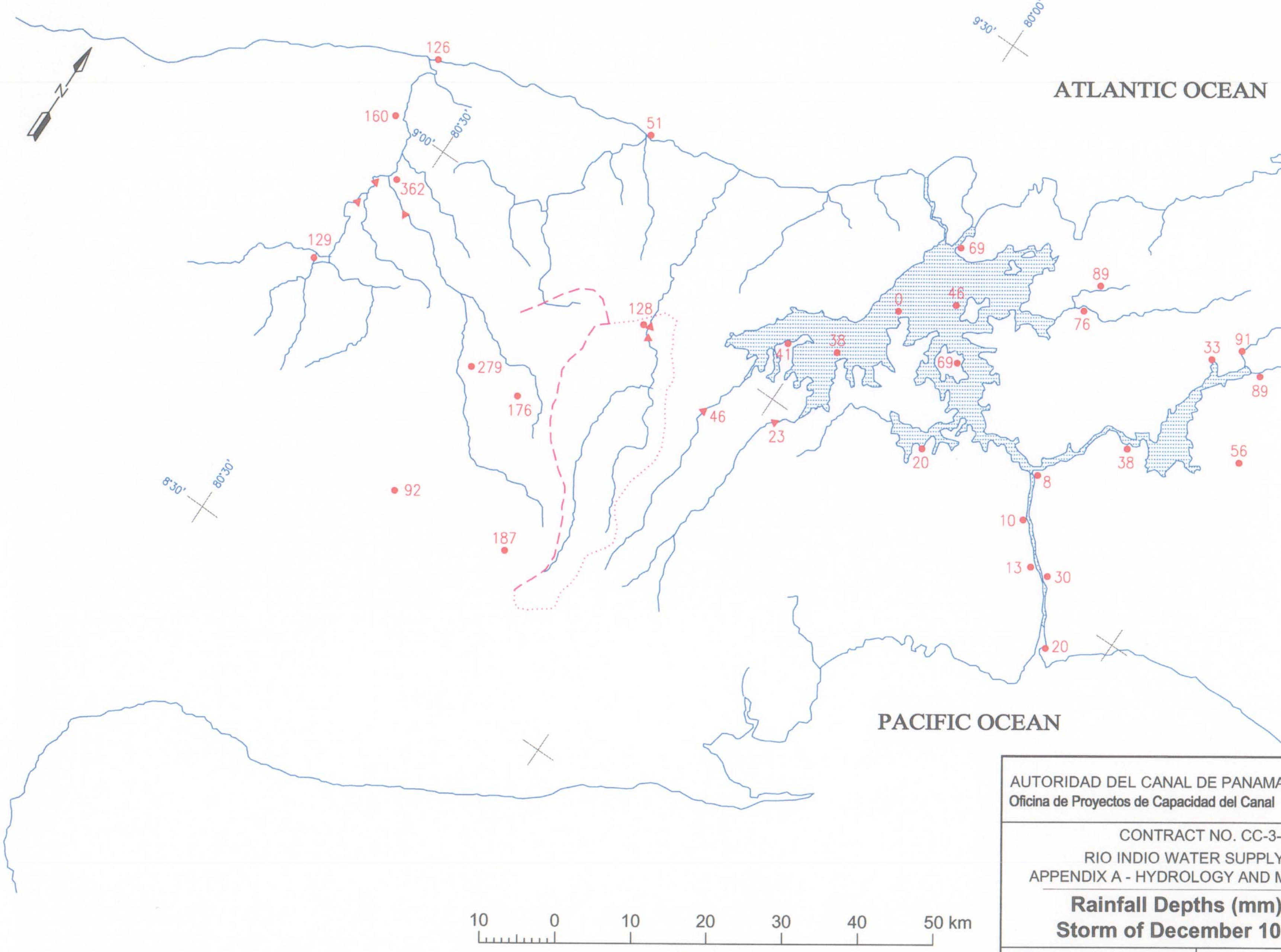
CONTRACT NO. CC-3-536
RIO INDIO WATER SUPPLY STUDY
APPENDIX A - HYDROLOGY AND METEOROLOGY

Rainfall Depths (mm) during Storm of December 4-6, 1981

MWH TAMS

DATE: APRIL, 2003

EXHIBIT: 32



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

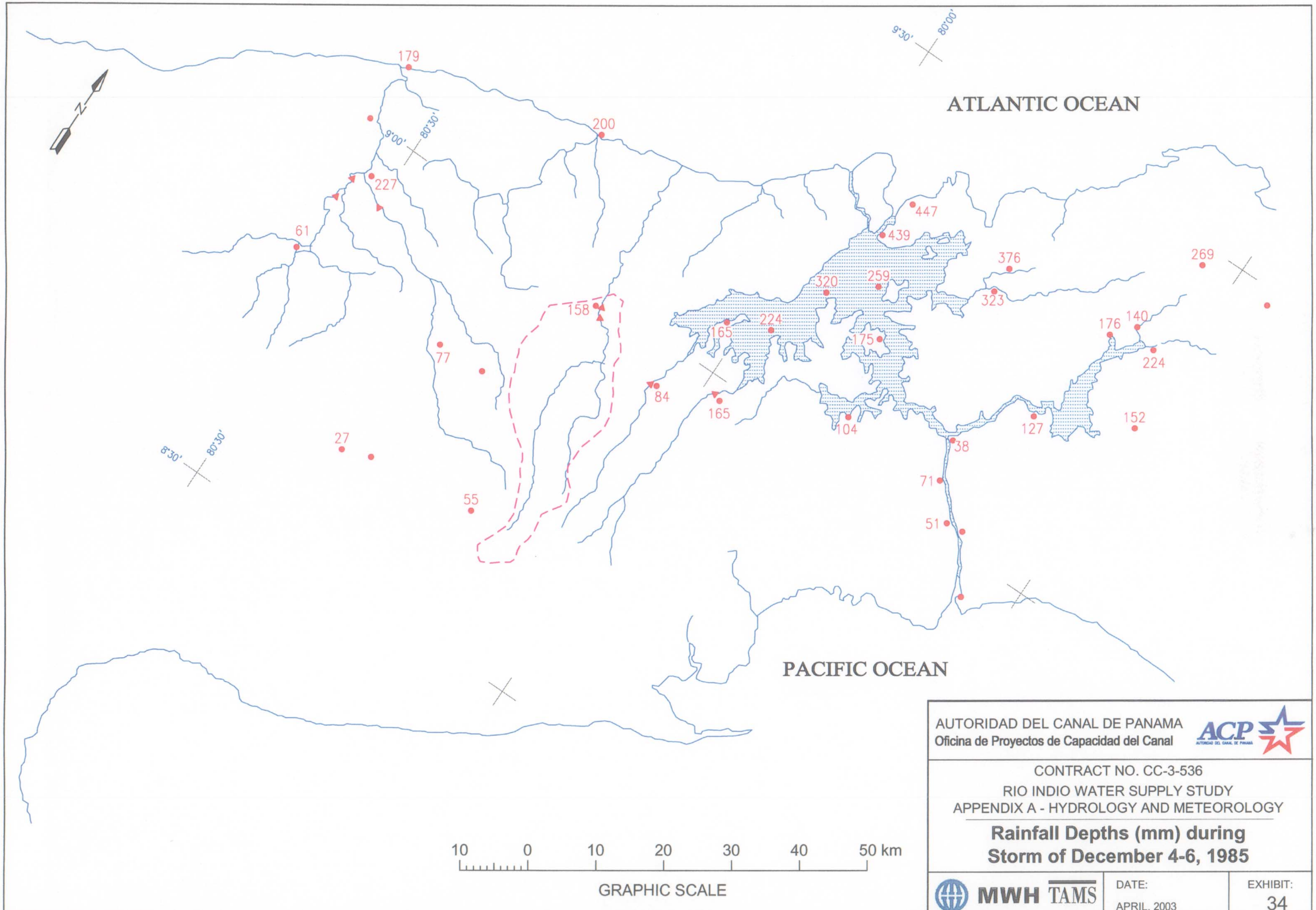
CONTRACT NO. CC-3-536
RIO INDIO WATER SUPPLY STUDY
APPENDIX A - HYDROLOGY AND METEOROLOGY

Rainfall Depths (mm) during Storm of December 10-12, 1981

MWH TAMS

DATE: APRIL, 2003

EXHIBIT: 33



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

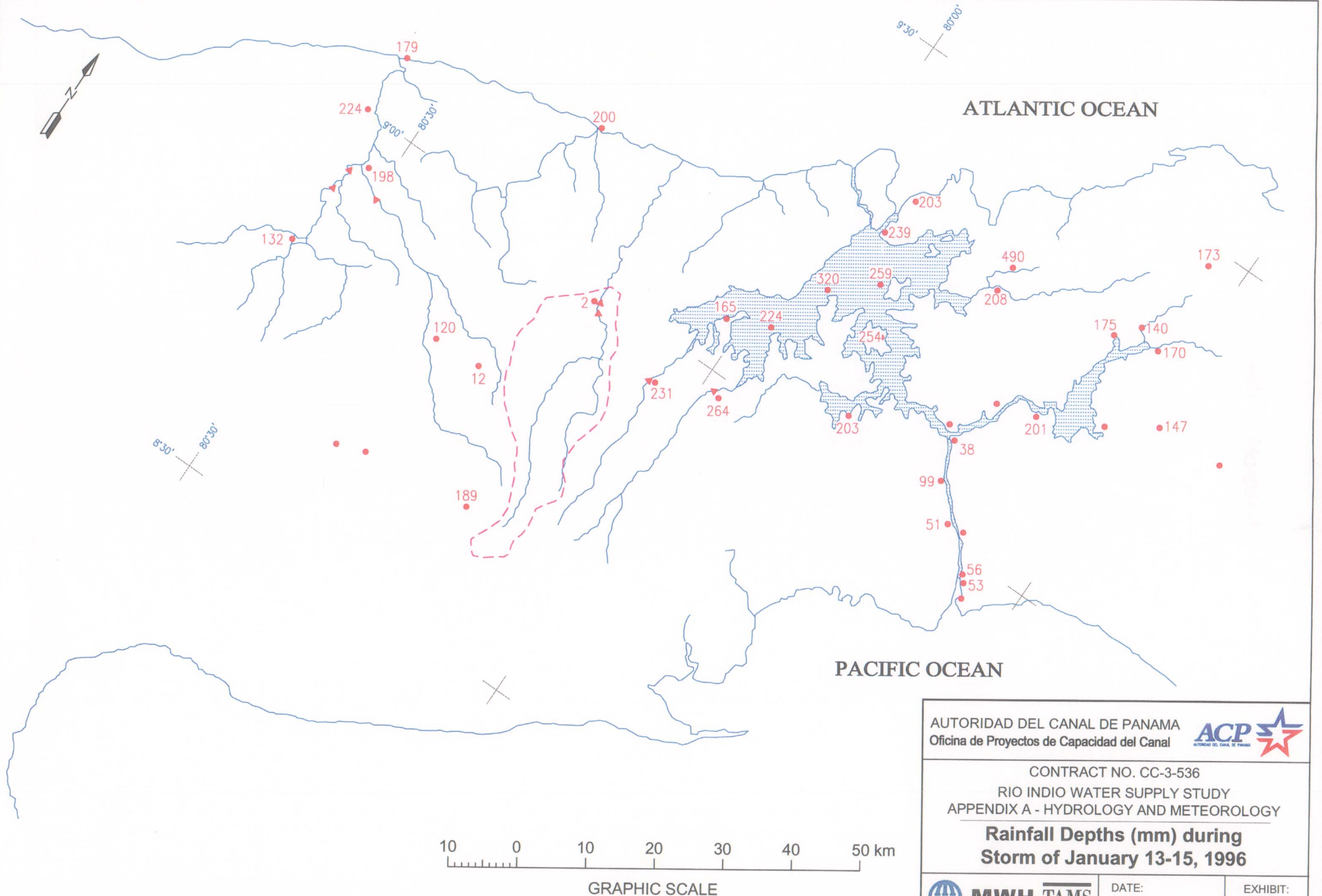
CONTRACT NO. CC-3-536
RIO INDIO WATER SUPPLY STUDY
APPENDIX A - HYDROLOGY AND METEOROLOGY

Rainfall Depths (mm) during Storm of December 4-6, 1985

MWH TAMS

DATE: APRIL, 2003

EXHIBIT: 34



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

ACP

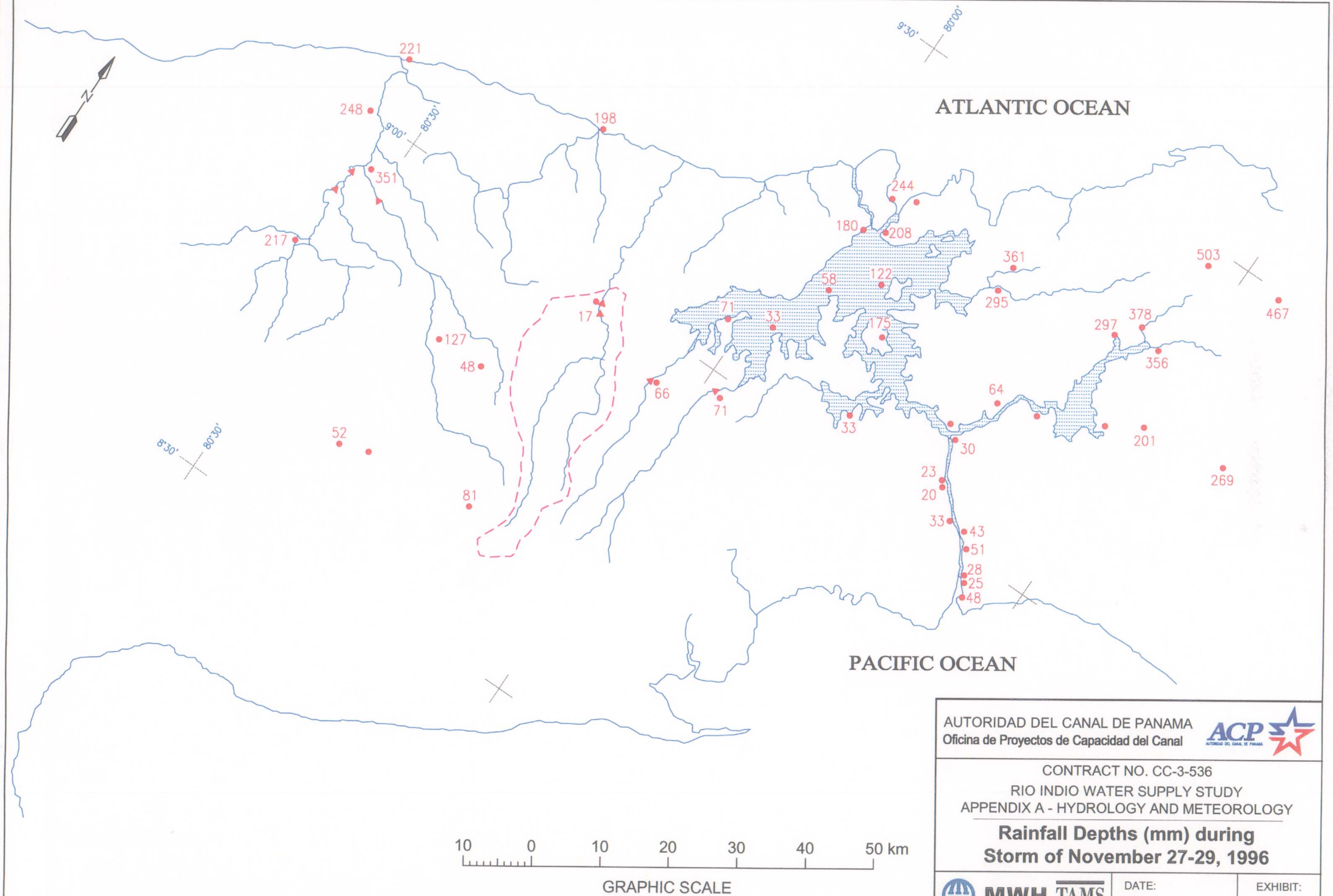
CONTRACT NO. CC-3-536
RIO INDIO WATER SUPPLY STUDY
APPENDIX A - HYDROLOGY AND METEOROLOGY

Rainfall Depths (mm) during Storm of January 13-15, 1996

MWH TAMS

DATE: APRIL, 2003

EXHIBIT: 35



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

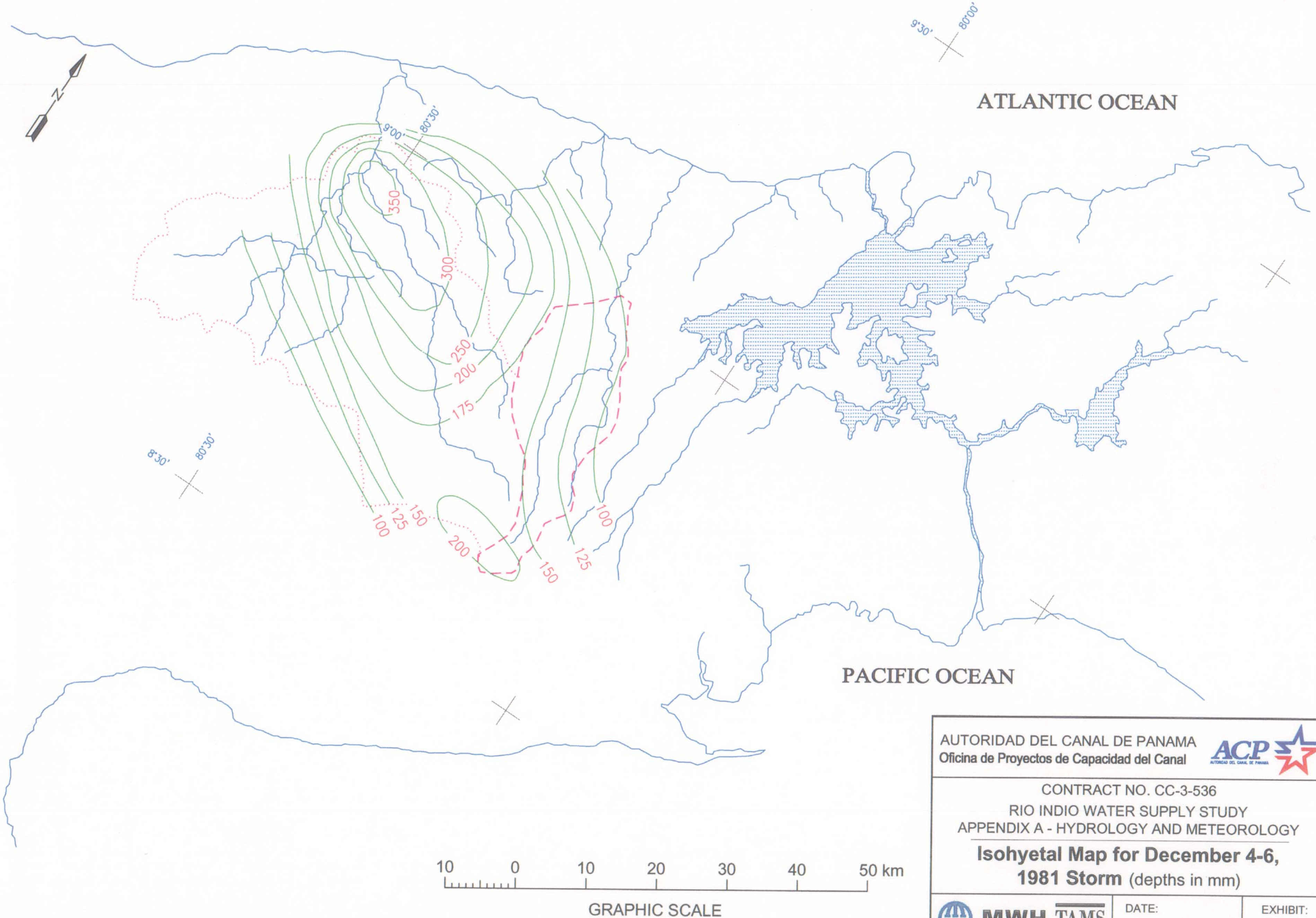
CONTRACT NO. CC-3-536
RIO INDIO WATER SUPPLY STUDY
APPENDIX A - HYDROLOGY AND METEOROLOGY

Rainfall Depths (mm) during Storm of November 27-29, 1996

MWH TAMS

DATE: APRIL, 2003

EXHIBIT: 36



AUTORIDAD DEL CANAL DE PANAMA

Oficina de Proyectos de Capacidad del Canal

ACP

CONTRACT NO. CC-3-536

RIO INDIO WATER SUPPLY STUDY

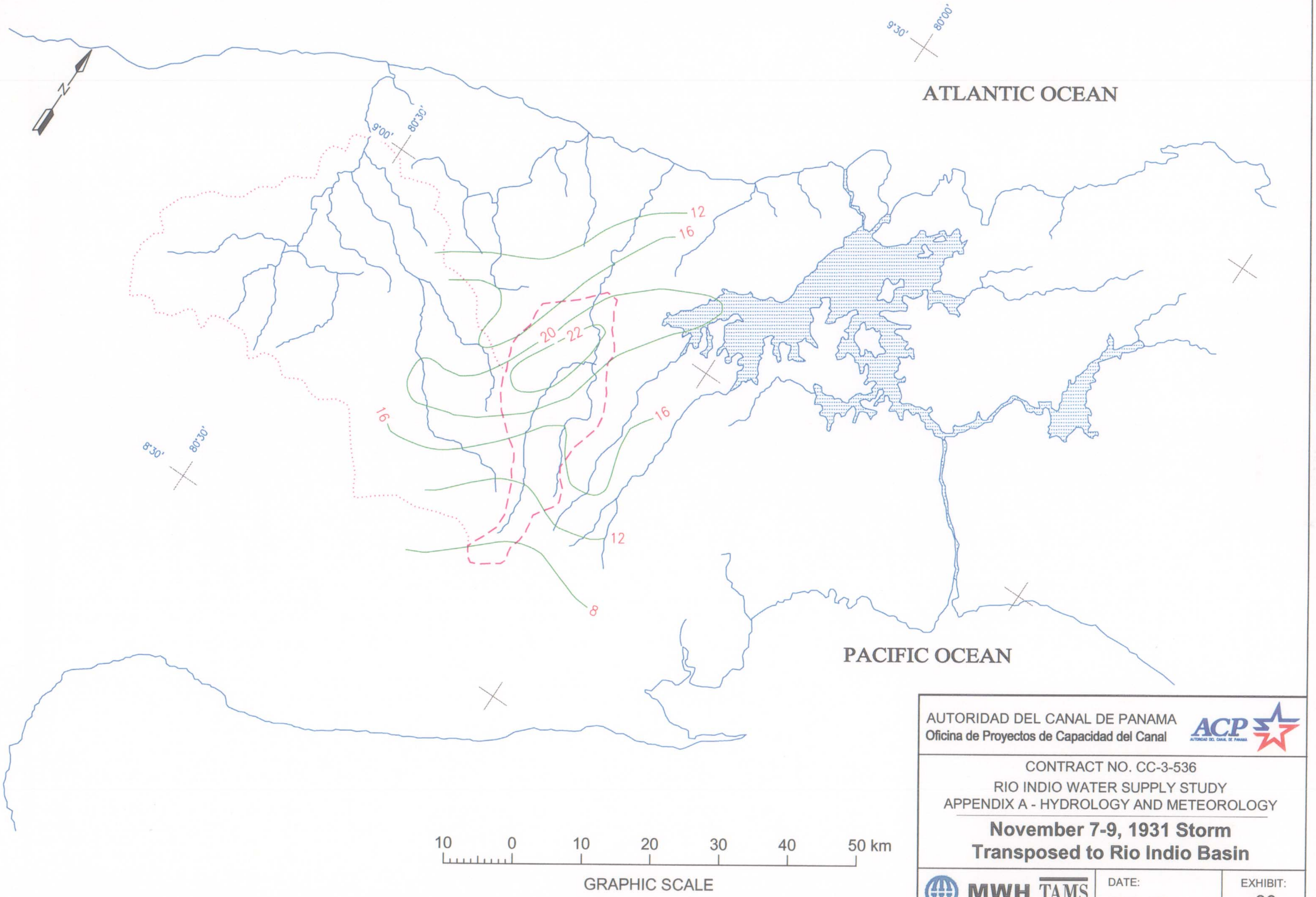
APPENDIX A - HYDROLOGY AND METEOROLOGY

Isohyetal Map for December 4-6, 1981 Storm (depths in mm)

MWH TAMS

DATE: APRIL, 2003

EXHIBIT: 37



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

ACP

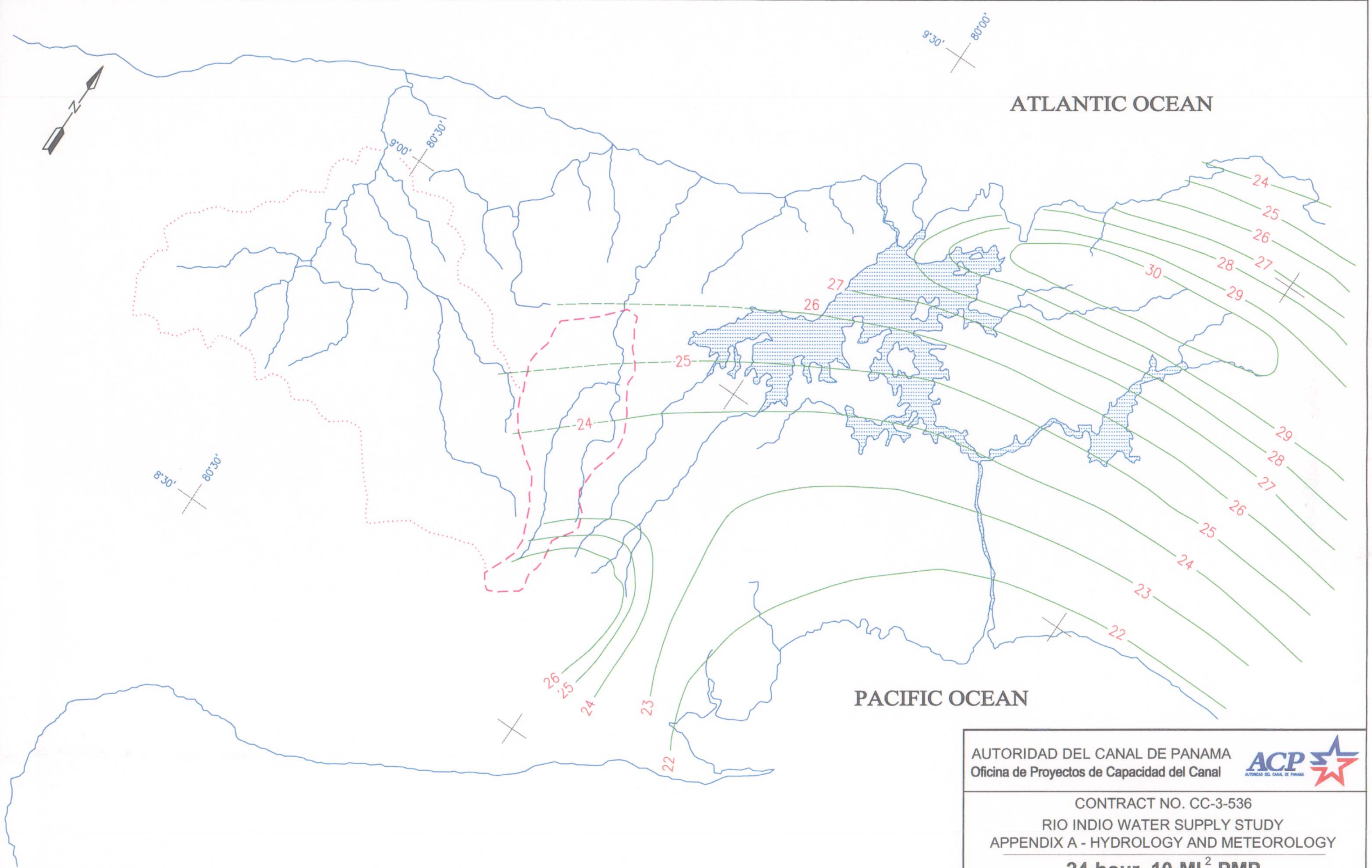
CONTRACT NO. CC-3-536
RIO INDIO WATER SUPPLY STUDY
APPENDIX A - HYDROLOGY AND METEOROLOGY

November 7-9, 1931 Storm
Transposed to Rio Indio Basin

MWH TAMS

DATE: APRIL, 2003

EXHIBIT: 38



10 0 10 20 30 40 50 km

GRAPHIC SCALE

Source:

US NWS, February 1978 Report

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

CONTRACT NO. CC-3-536

RIO INDIO WATER SUPPLY STUDY

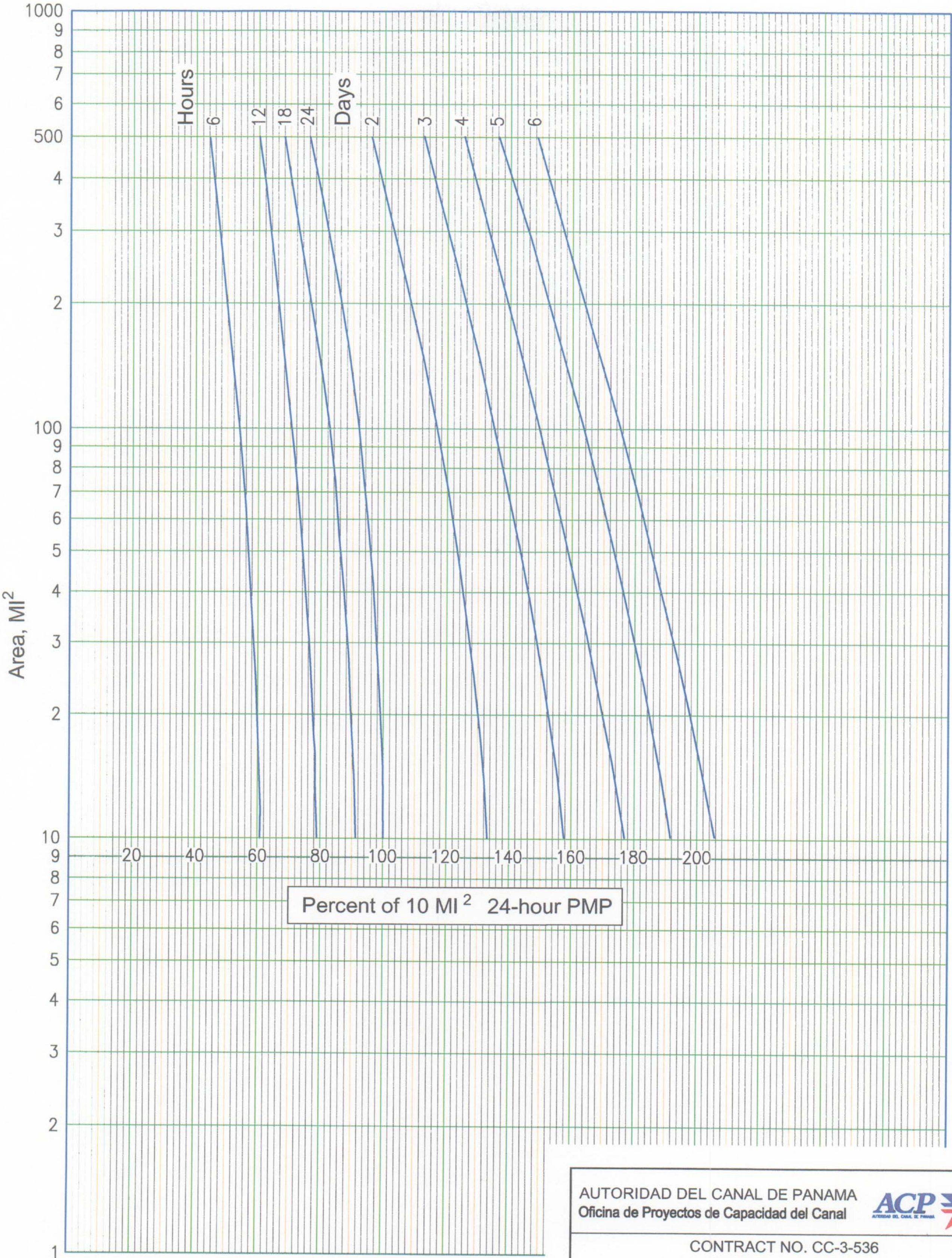
APPENDIX A - HYDROLOGY AND METEOROLOGY

24-hour, 10-MI² PMP

(inches)

MWH
MONTGOMERY WATSON HARZA

FECHA:	EXHIBIT:
DECEMBER, 2001	39



Source:

US Weather Bureau, 1965 Report

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

ACP

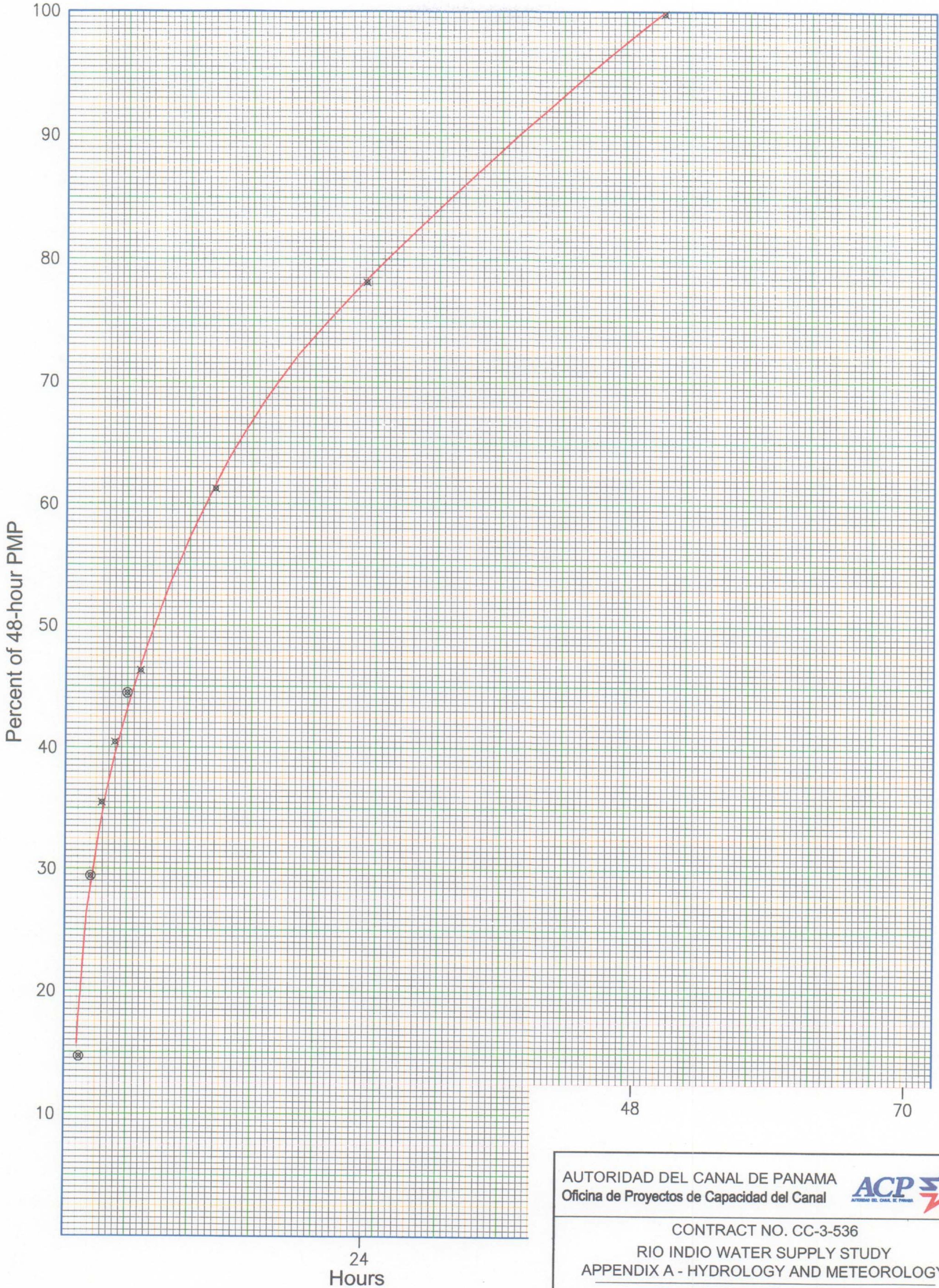
CONTRACT NO. CC-3-536
RIO INDIO WATER SUPPLY STUDY
APPENDIX A - HYDROLOGY AND METEOROLOGY

Depth-Area-Duration Curves

MWH TAMS

DATE: APRIL, 2003

EXHIBIT: 40



Source:

US Weather Bureau, 1965 Report

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

CONTRACT NO. CC-3-536
RIO INDIO WATER SUPPLY STUDY
APPENDIX A - HYDROLOGY AND METEOROLOGY

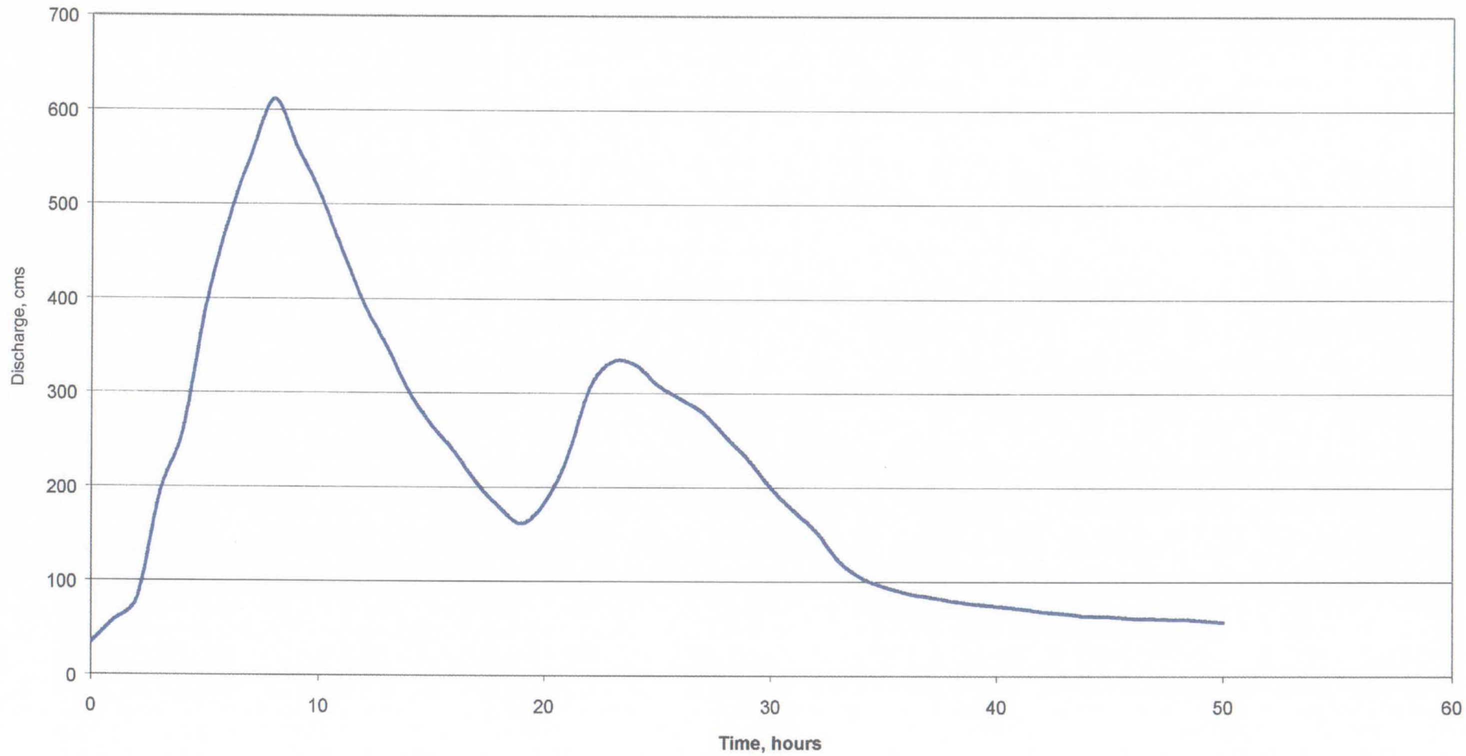
Depth-Duration Curve
Rio Indio Basin (147 MI^2)

MWH TAMS

DATE: APRIL, 2003

EXHIBIT: 41

RIO INDIO AT BOCA DE URACILLO - FLOOD OF AUGUST 15-17, 1980



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

CONTRACT NO. CC-3-536
RIO INDIO WATER SUPPLY STUDY
APPENDIX A - HYDROLOGY AND METEOROLOGY

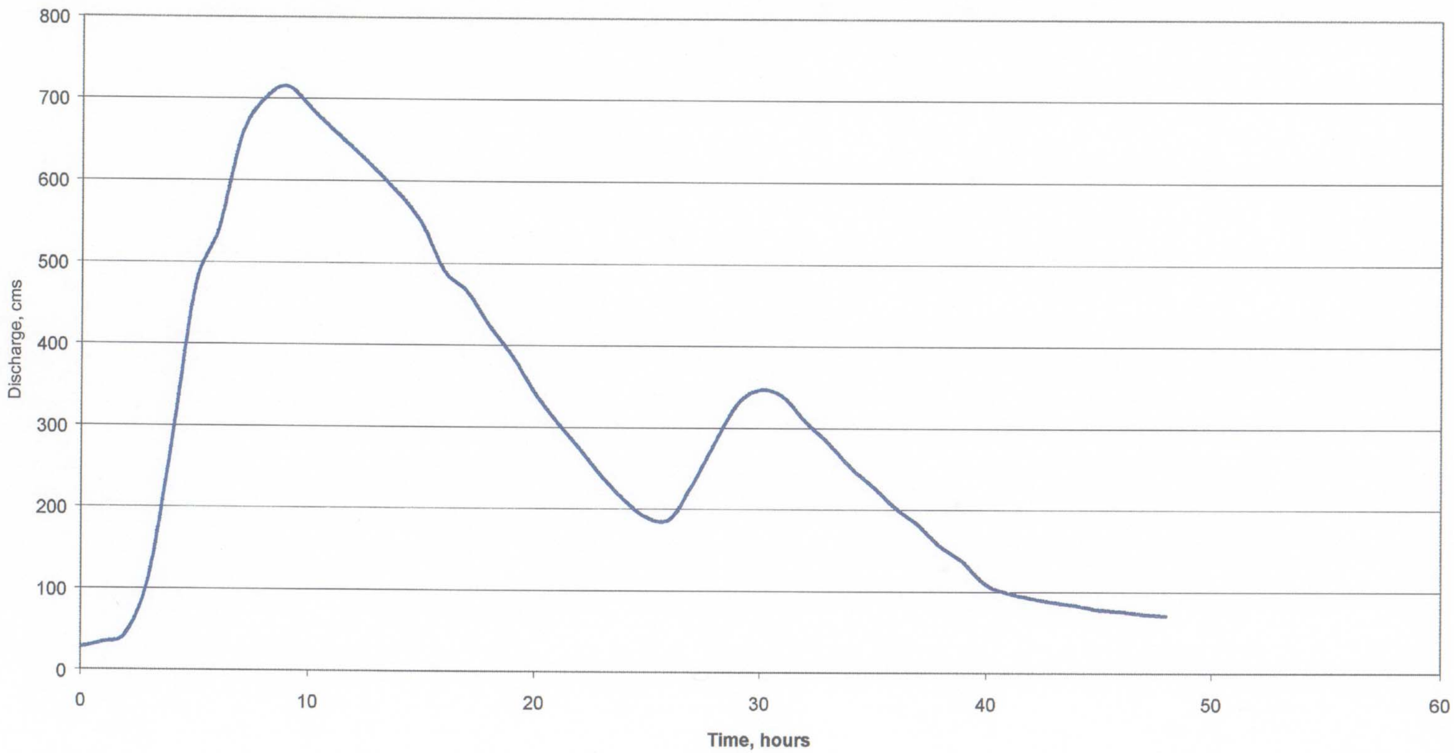
Rio Indio at Boca de Uracillo
Flood of August 15-17, 1980

MWH TAMS

DATE: APRIL, 2003

EXHIBIT: 42

RIO INDIO AT BOCA DE URACILLO - FLOOD OF OCTOBER 19-21, 1987



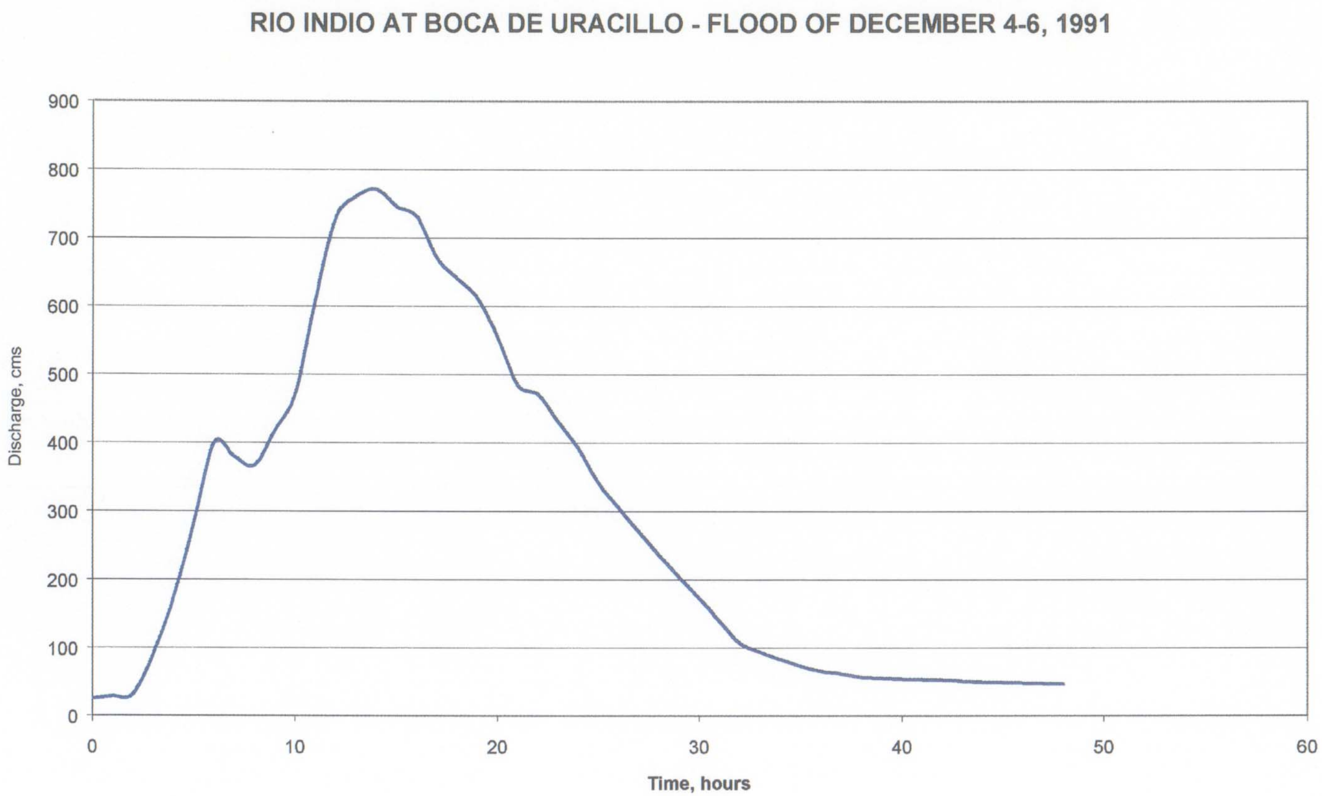
AUTORIDAD DEL CANAL DE PANAMA
Oficina de Proyectos de Capacidad del Canal

CONTRACT NO. CC-3-536
RIO INDIO WATER SUPPLY STUDY
APPENDIX A - HYDROLOGY AND METEOROLOGY

Rio Indio at Boca de Uracillo
Flood of October 19-21, 1987

MWH TAMS

DATE:	EXHIBIT:
APRIL, 2003	43



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

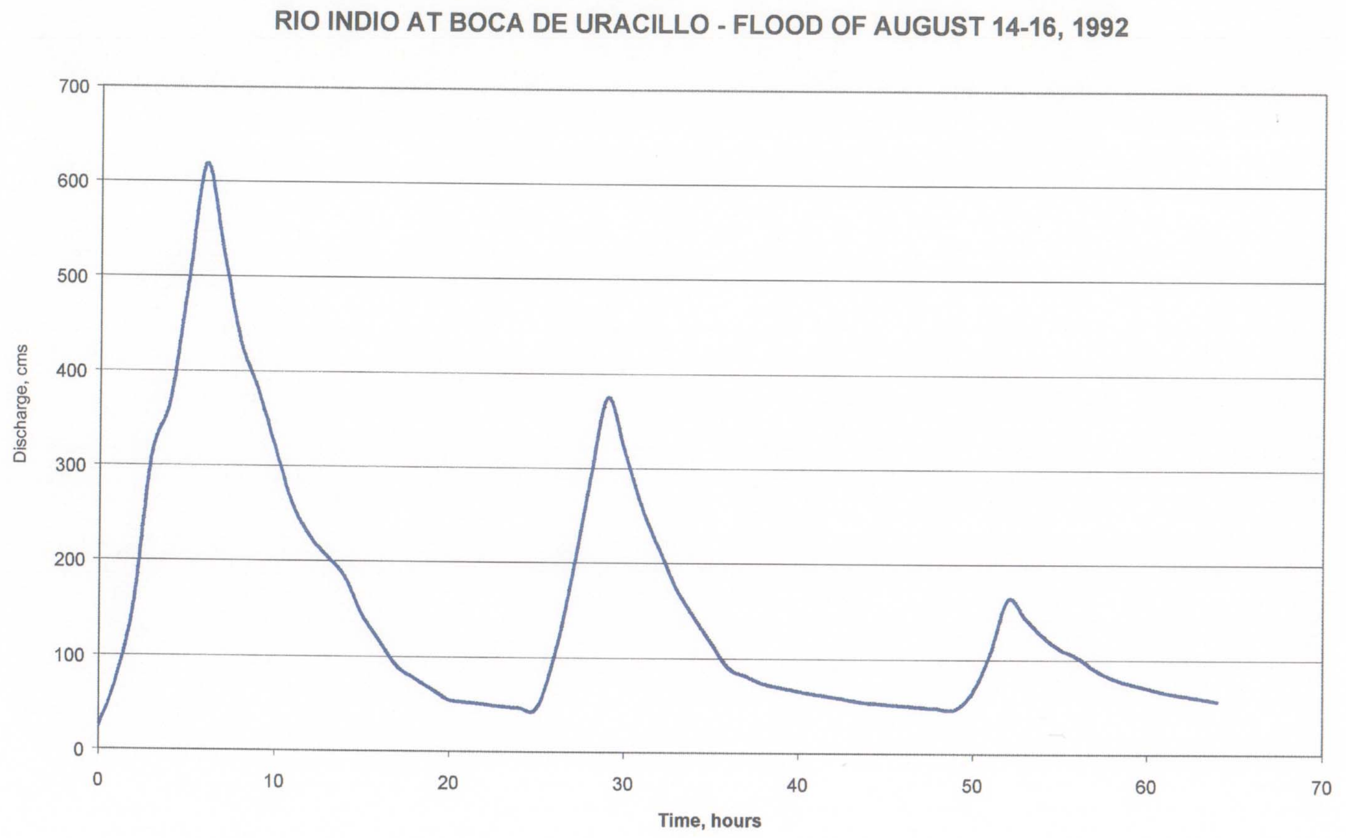
CONTRACT NO. CC-3-536
RIO INDIO WATER SUPPLY STUDY
APPENDIX A - HYDROLOGY AND METEOROLOGY

Rio Indio at Boca de Uracillo
Flood of December 4-6, 1991

MWH TAMS

DATE: APRIL, 2003

EXHIBIT: 44



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

CONTRACT NO. CC-3-536
RIO INDIO WATER SUPPLY STUDY
APPENDIX A - HYDROLOGY AND METEOROLOGY

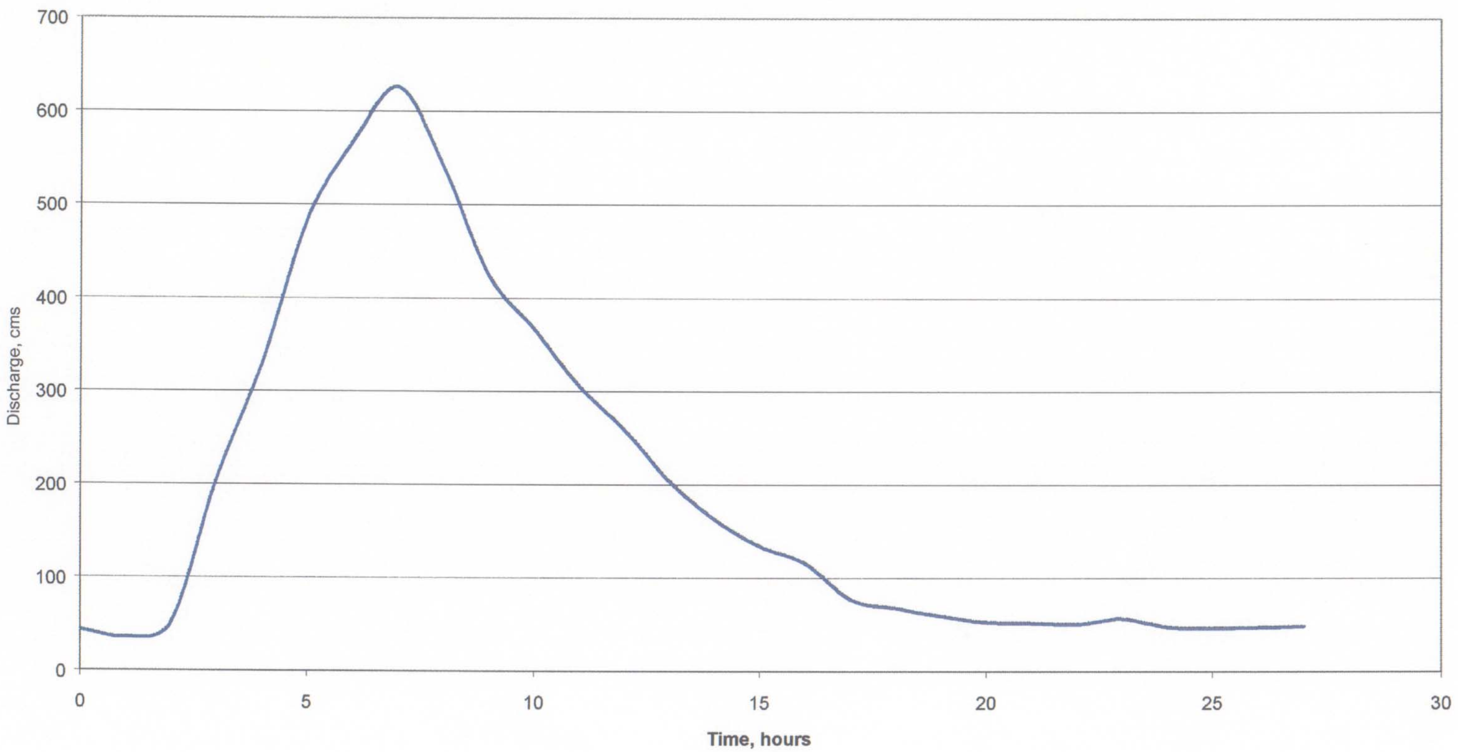
Rio Indio at Boca de Uracillo
Flood of August 14-15, 1992

MWH TAMS

DATE: APRIL, 2003

EXHIBIT: 45

RIO INDIO AT BOCA DE URACILLO - FLOOD OF SEPTEMBER 12-14, 1994



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

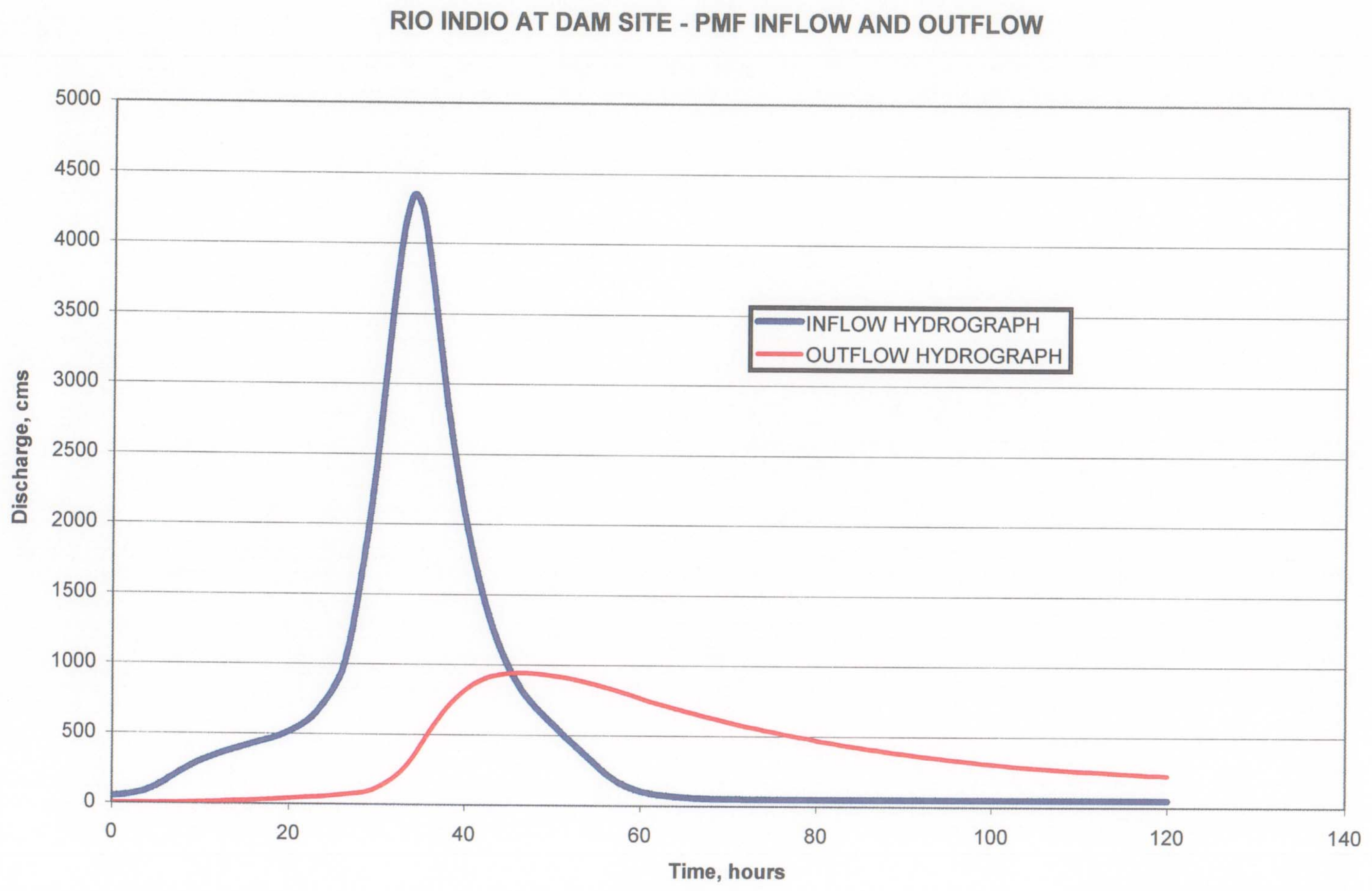
CONTRACT NO. CC-3-536
RIO INDIO WATER SUPPLY STUDY
APPENDIX A - HYDROLOGY AND METEOROLOGY

Rio Indio at Boca de Uracillo
Flood of September 12-14, 1994

MWH TAMS

DATE: APRIL, 2003

EXHIBIT: 46



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

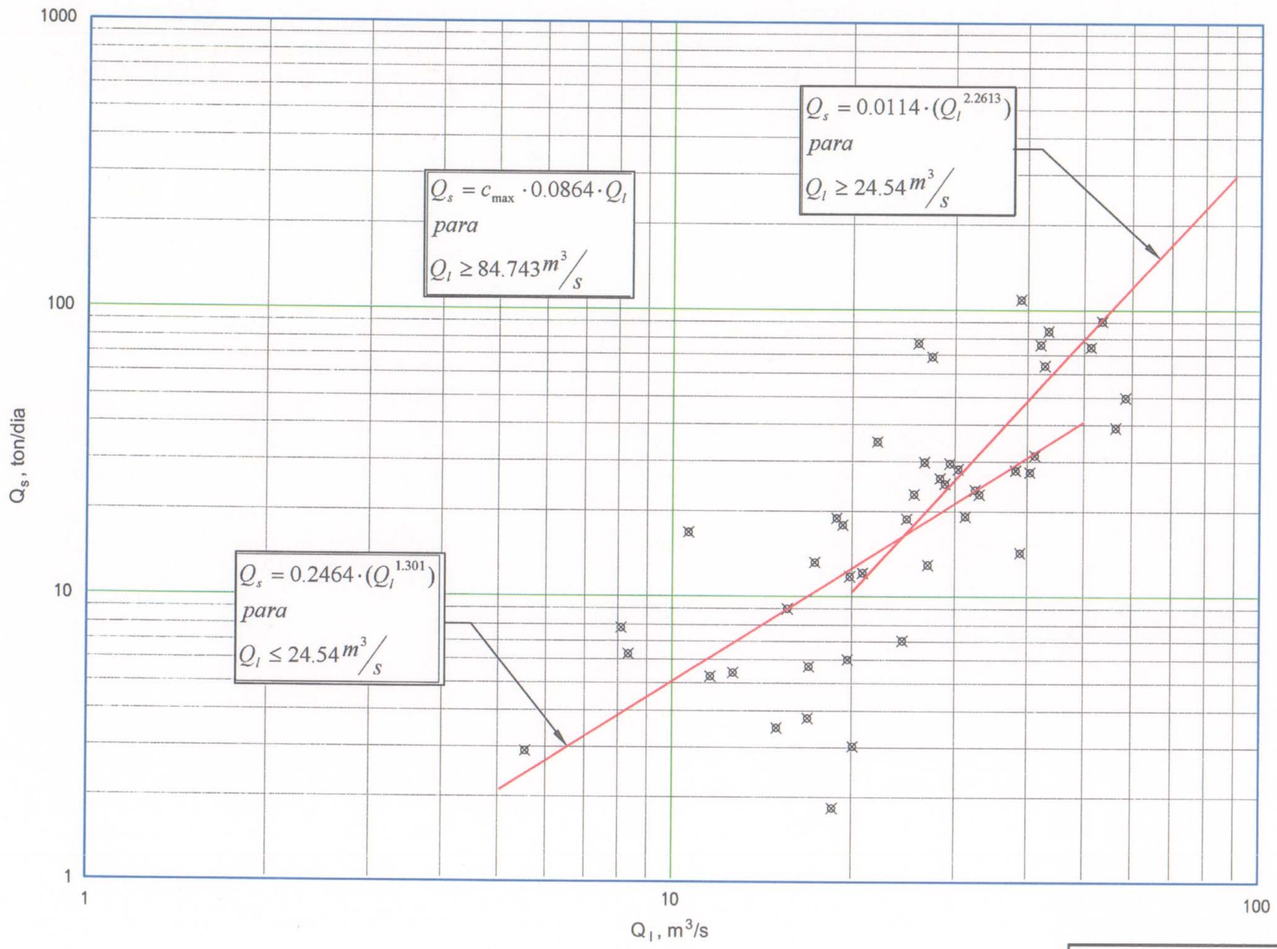
CONTRACT NO. CC-3-536
RIO INDIO WATER SUPPLY STUDY
APPENDIX A - HYDROLOGY AND METEOROLOGY

Rio Indio at Dam Site
PMF Inflow and Outflow Hydrographs

MWH TAMS

DATE: APRIL, 2003

EXHIBIT: 47



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

CONTRACT NO. CC-3-536
RIO INDIO WATER SUPPLY STUDY
APPENDIX A - HYDROLOGY AND METEOROLOGY

Curva de Sedimento en Suspension
Cocle del Norte, Canoas - 105-0102

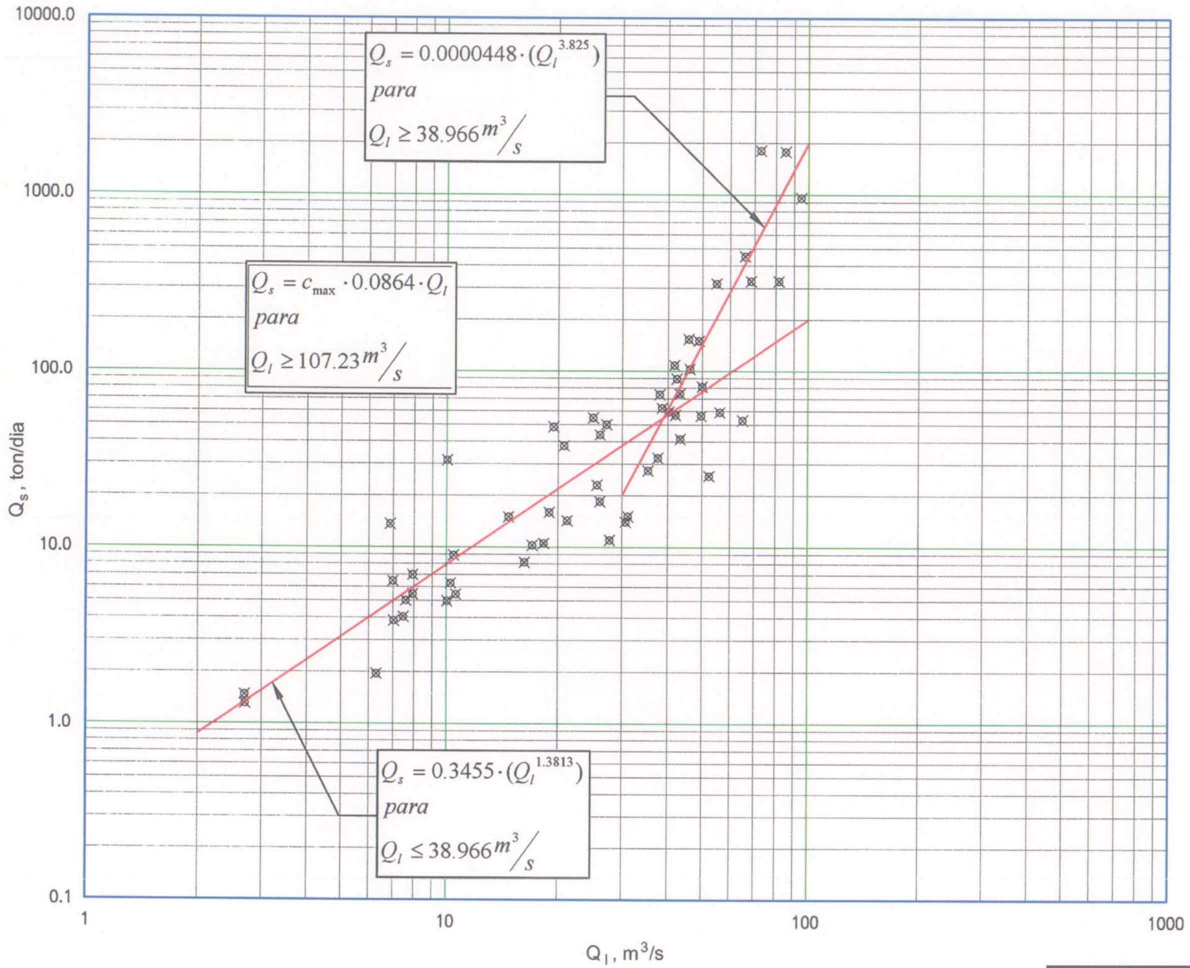
MWH TAMS

DATE: APRIL, 2003

EXHIBIT: 48

Source:

Departamento de Hidrometeorologia



Source:

Departamento de Hidrometeorologia

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

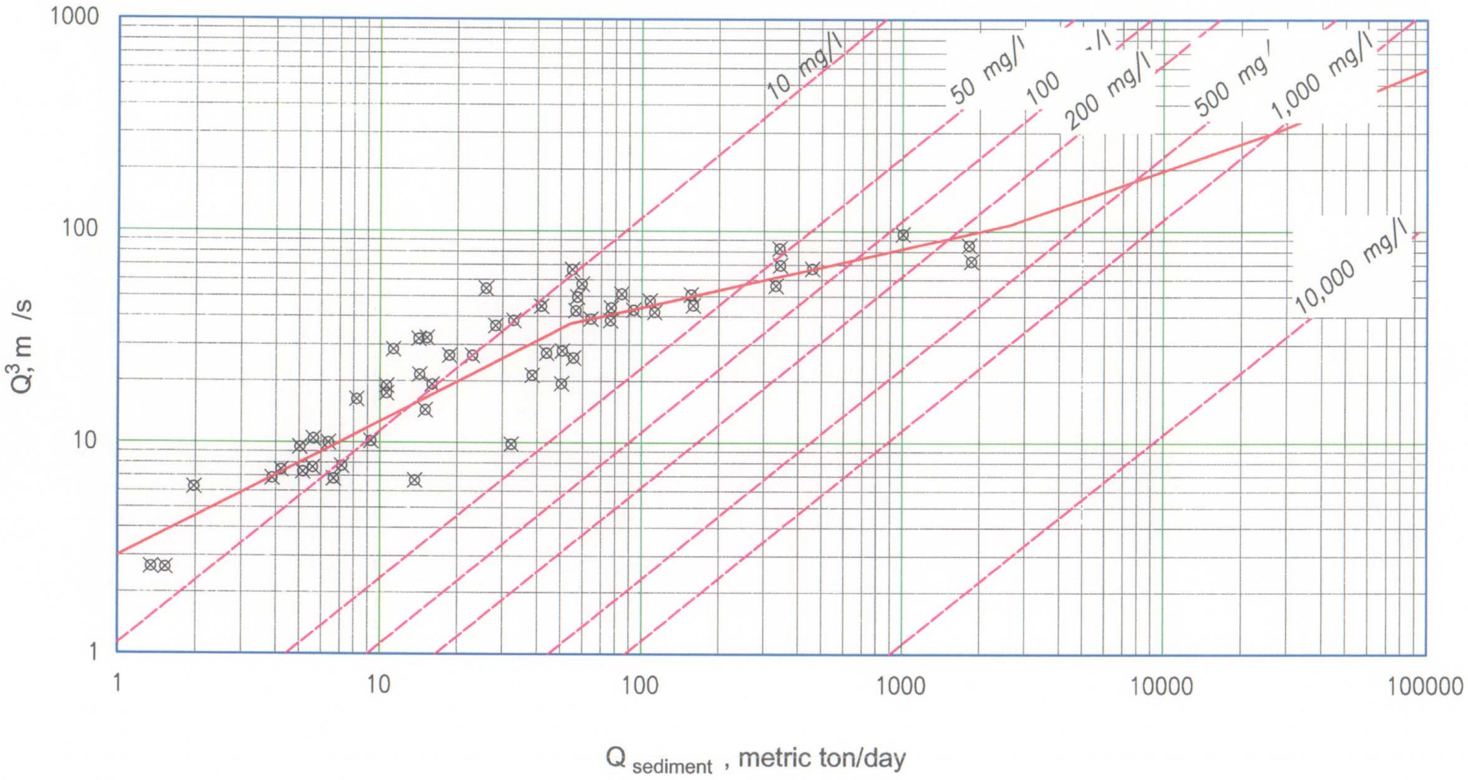
CONTRACT NO. CC-3-536
RIO INDIO WATER SUPPLY STUDY
APPENDIX A - HYDROLOGY AND METEOROLOGY

Curva de Sedimento en Suspension
Toabre, Batatilla - 105-0201

MWH TAMS

DATE: APRIL, 2003

EXHIBIT: 49



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

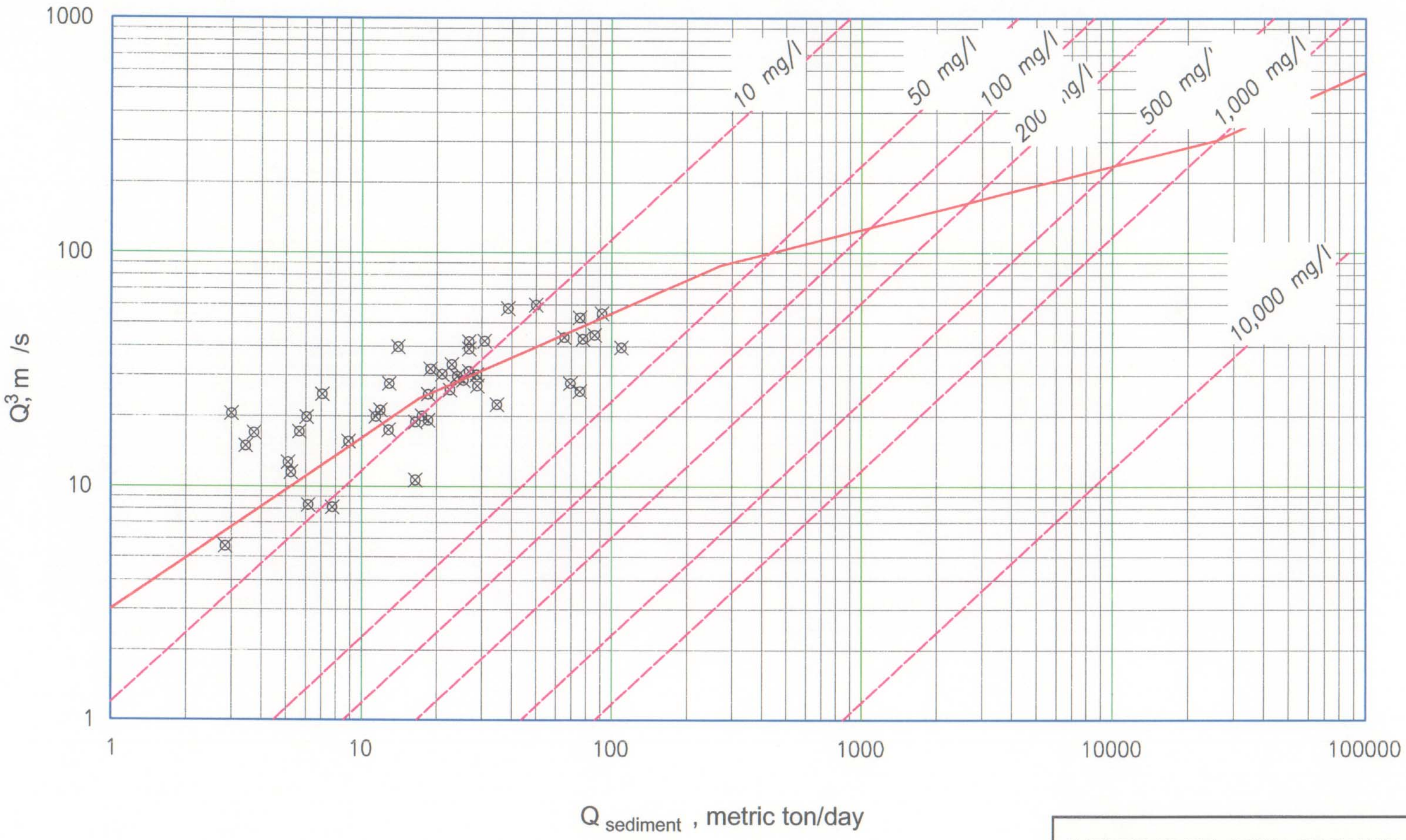
ACP

CONTRACT NO. CC-3-536
RIO INDIO WATER SUPPLY STUDY
APPENDIX A - HYDROLOGY AND METEOROLOGY

Suspended Sediment Rating Curve
Rio Toabre

MWH TAMS

DATE:	EXHIBIT:
APRIL, 2003	50



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

ACP

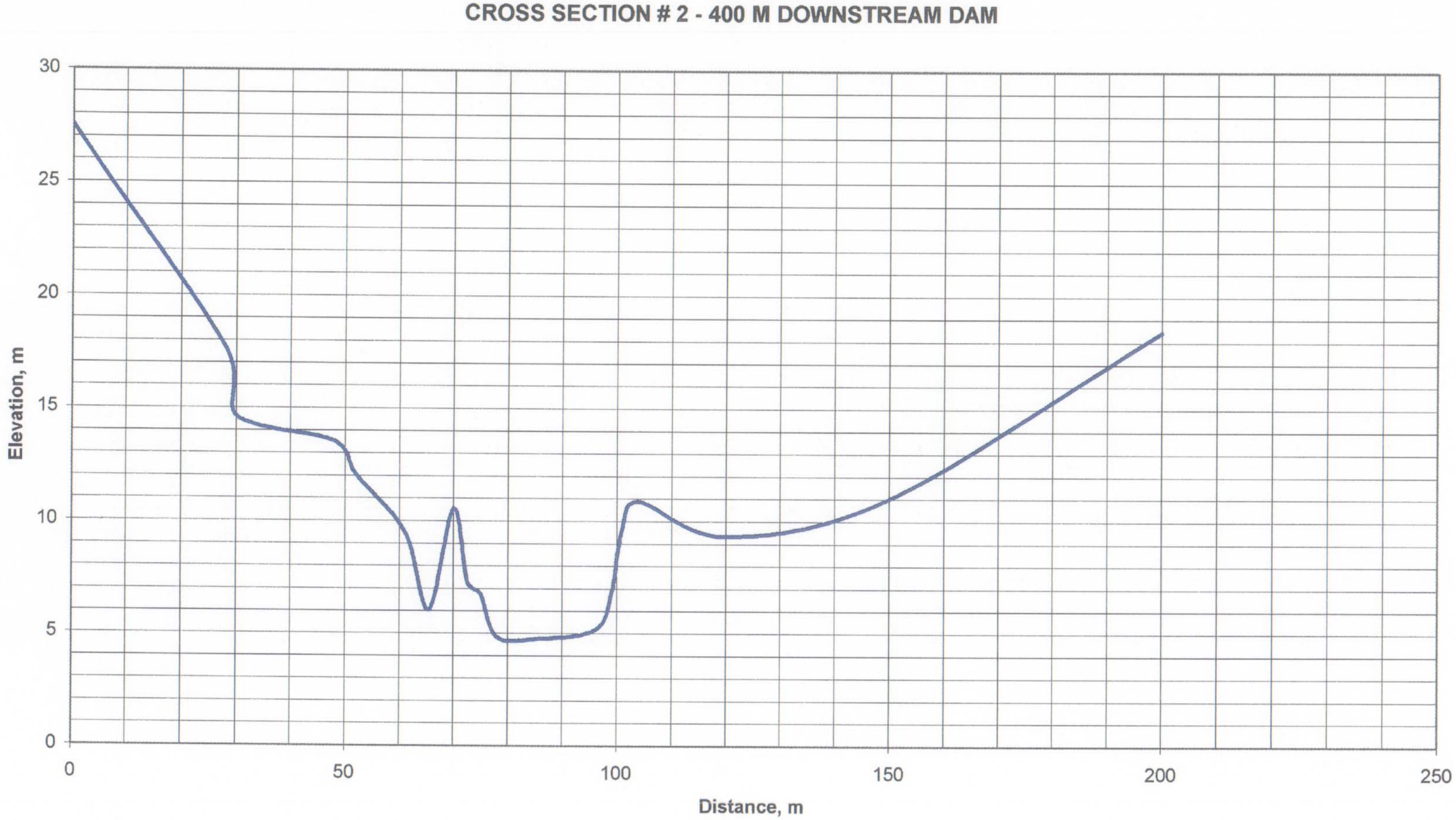
CONTRACT NO. CC-3-536
RIO INDIO WATER SUPPLY STUDY
APPENDIX A - HYDROLOGY AND METEOROLOGY

Suspended Sediment Rating Curve
Rio Cocle del Norte

MWH TAMS

DATE: APRIL, 2003

EXHIBIT: 51



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

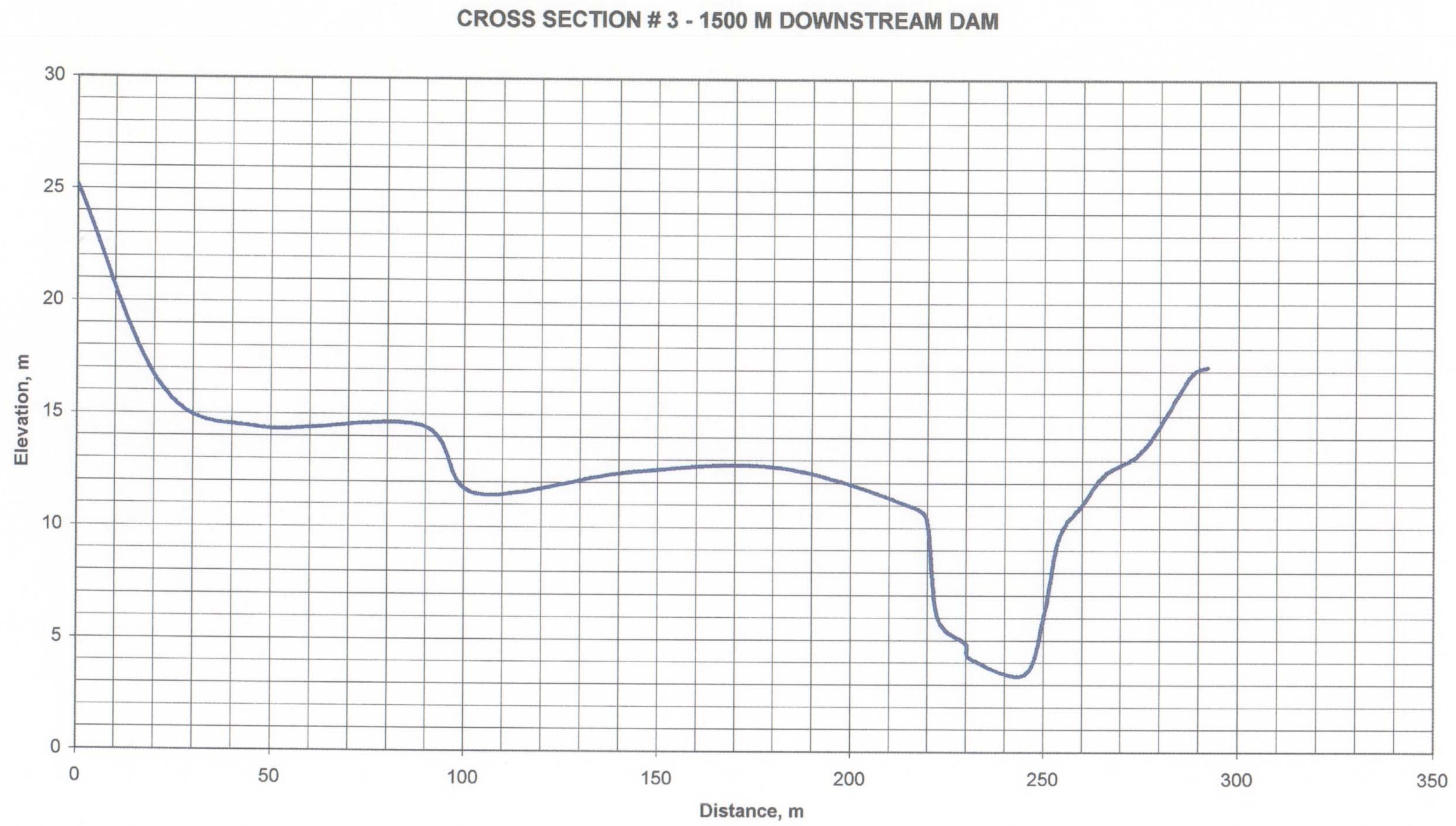
CONTRACT NO. CC-3-536
RIO INDIO WATER SUPPLY STUDY
APPENDIX A - HYDROLOGY AND METEOROLOGY

Cross Section No. 2
400 m Downstream from Dam

MWH TAMS

DATE: APRIL, 2003

EXHIBIT: 52



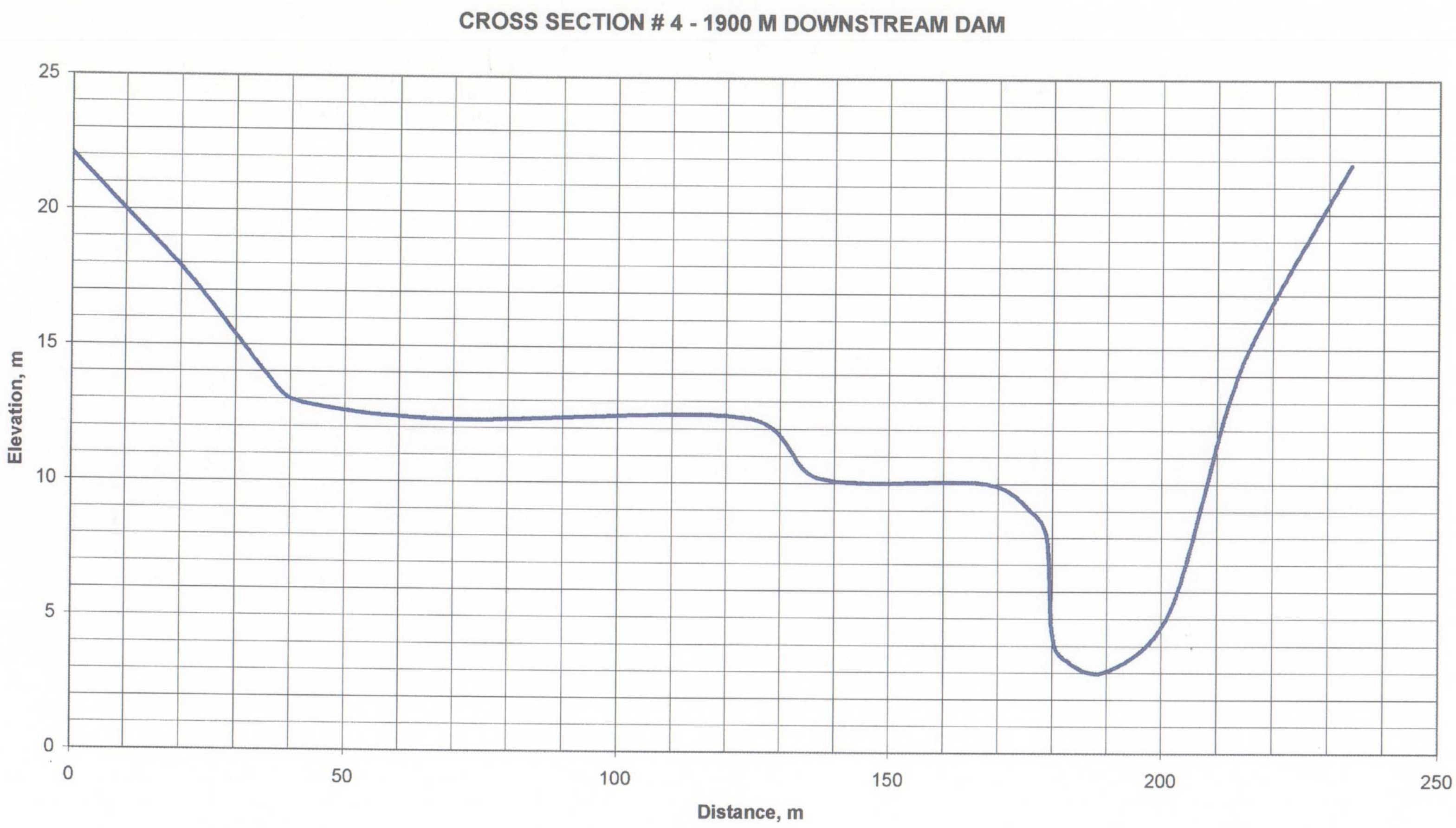
AUTORIDAD DEL CANAL DE PANAMA
Oficina de Proyectos de Capacidad del Canal

CONTRACT NO. CC-3-536
RIO INDIO WATER SUPPLY STUDY
APPENDIX A - HYDROLOGY AND METEOROLOGY

Cross Section No. 3
1500 m Downstream from Dam

MWH TAMS

DATE:	EXHIBIT:
APRIL, 2003	53

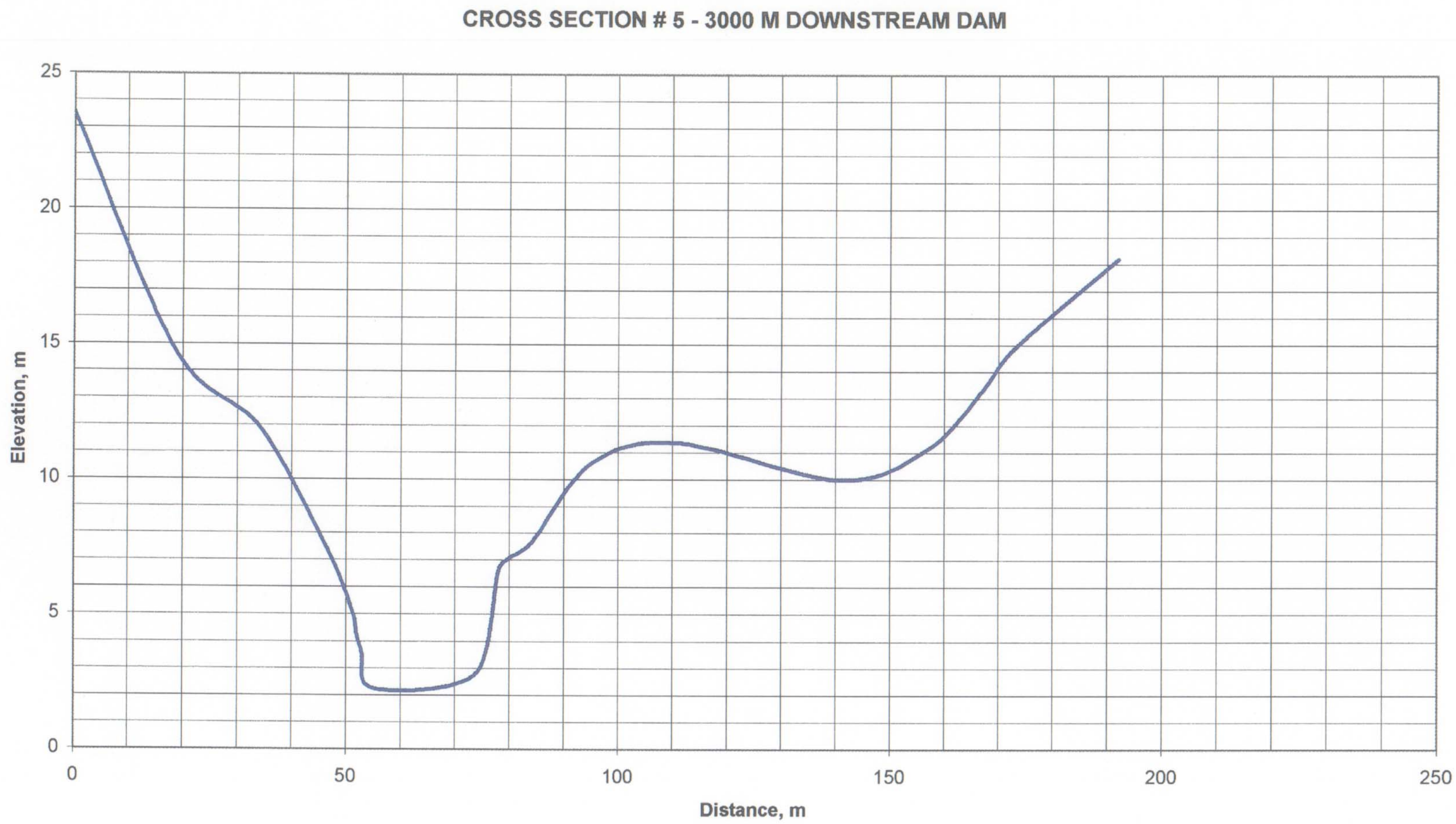


AUTORIDAD DEL CANAL DE PANAMA
Oficina de Proyectos de Capacidad del Canal

CONTRACT NO. CC-3-536
RIO INDIO WATER SUPPLY STUDY
APPENDIX A - HYDROLOGY AND METEOROLOGY

Cross Section No. 4
1900 m Downstream from Dam

MWH TAMS	DATE: APRIL, 2003	EXHIBIT: 54



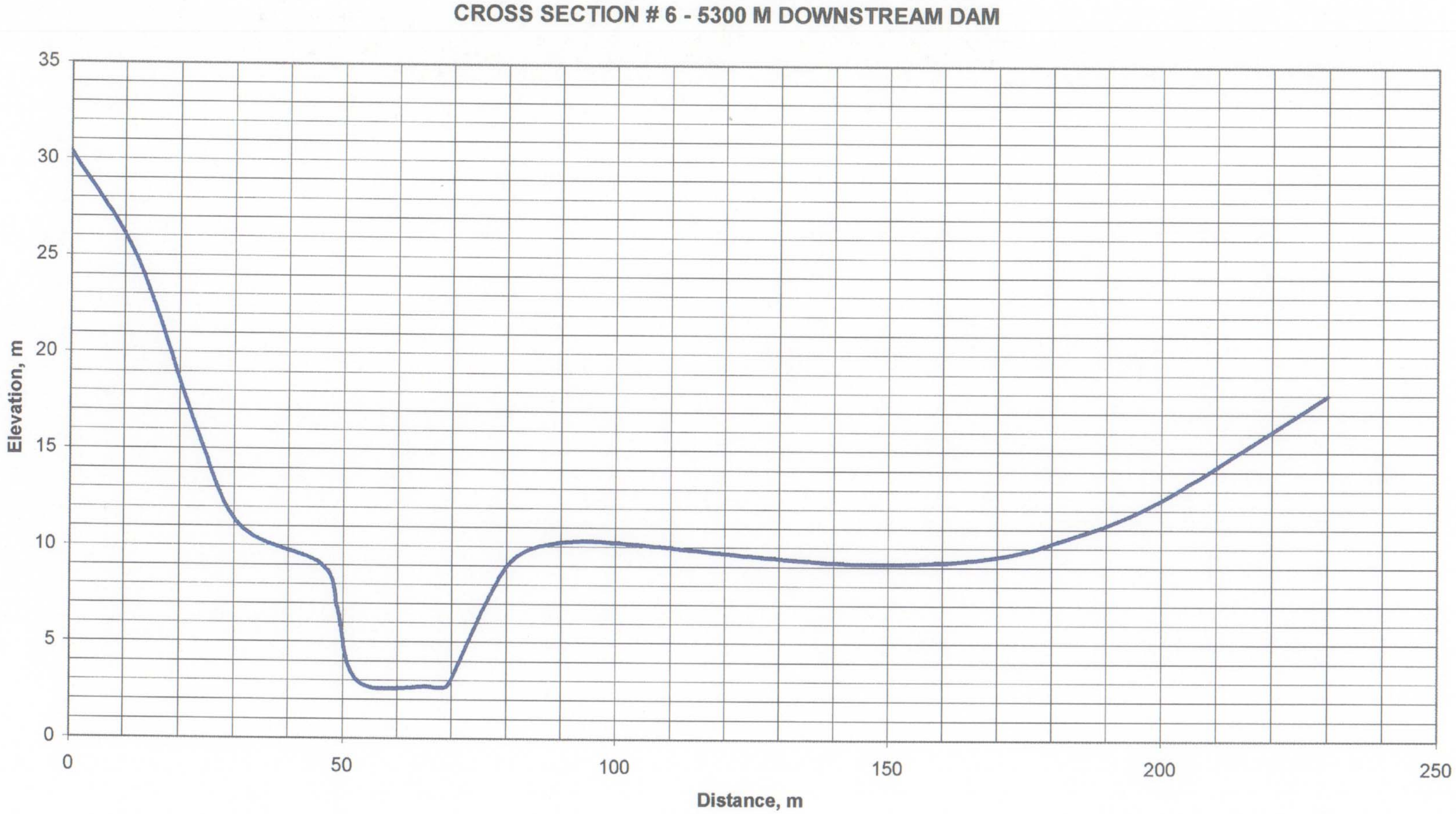
AUTORIDAD DEL CANAL DE PANAMA
Oficina de Proyectos de Capacidad del Canal

CONTRACT NO. CC-3-536
RIO INDIO WATER SUPPLY STUDY
APPENDIX A - HYDROLOGY AND METEOROLOGY

Cross Section No. 5
3000 m Downstream from Dam

MWH TAMS

DATE:	EXHIBIT:
APRIL, 2003	55



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

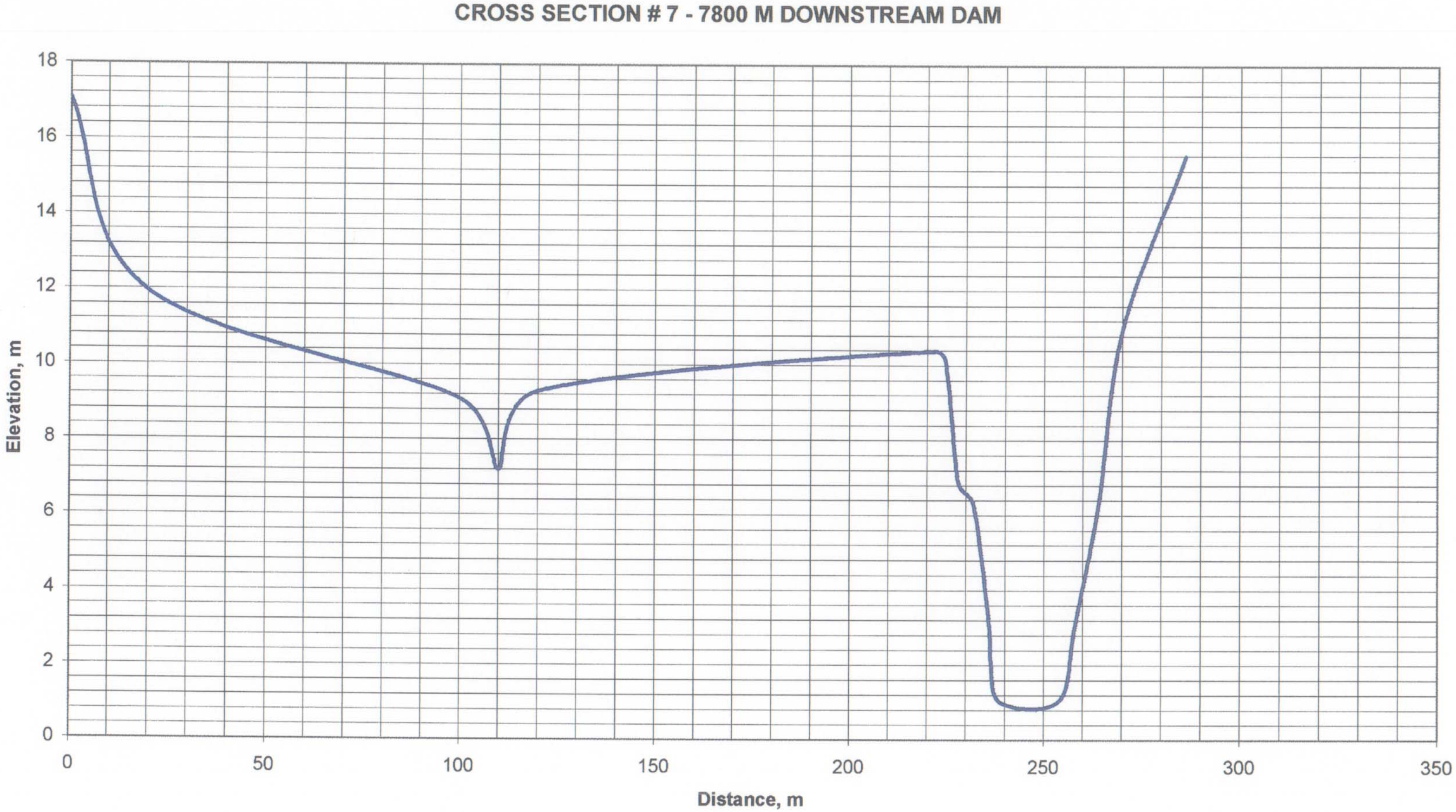
CONTRACT NO. CC-3-536
RIO INDIO WATER SUPPLY STUDY
APPENDIX A - HYDROLOGY AND METEOROLOGY

Cross Section No. 6
5300 m Downstream from Dam

MWH TAMS

DATE: APRIL, 2003

EXHIBIT: 56



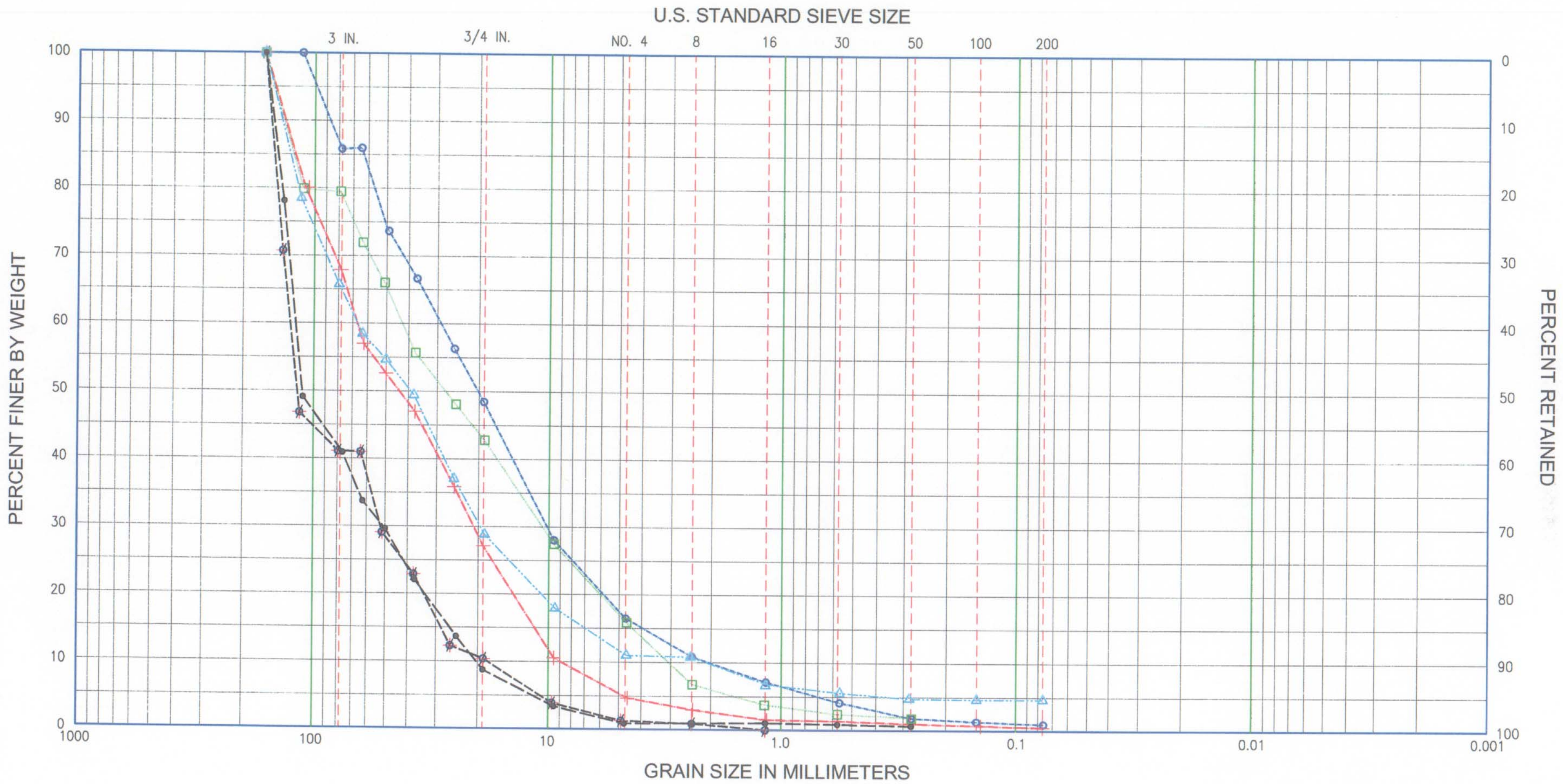
AUTORIDAD DEL CANAL DE PANAMA
Oficina de Proyectos de Capacidad del Canal

CONTRACT NO. CC-3-536
RIO INDIO WATER SUPPLY STUDY
APPENDIX A - HYDROLOGY AND METEOROLOGY

Cross Section No. 2
400 m Downstream from Dam

MWH TAMS

DATE:	EXHIBIT:
APRIL, 2003	57



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

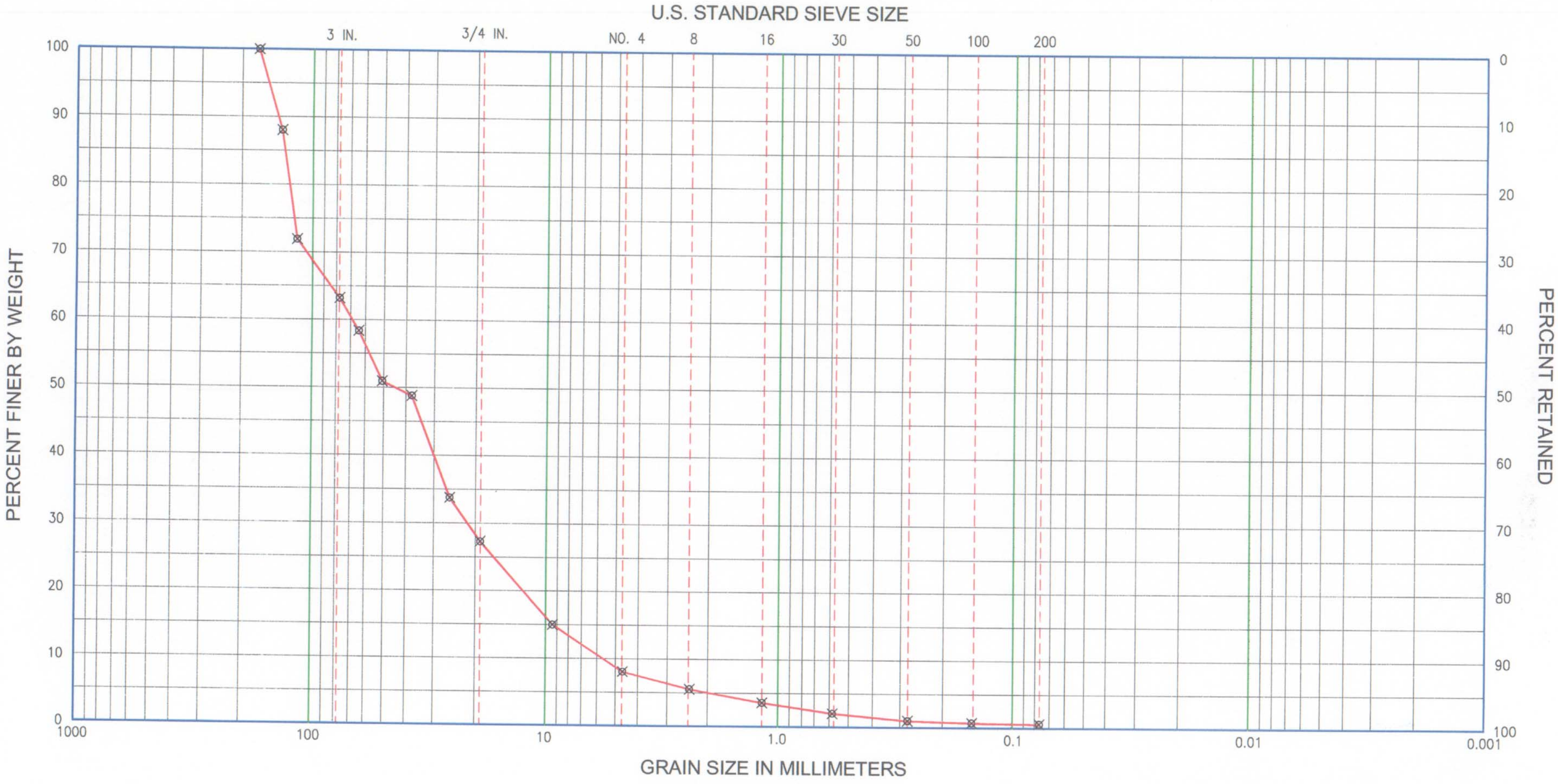
CONTRACT NO. CC-3-536
RIO INDIO WATER SUPPLY STUDY
APPENDIX A - HYDROLOGY AND METEOROLOGY

Bed Material Samples
Particle Size Distribution

MWH TAMS

DATE: APRIL, 2003

EXHIBIT: 58



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

CONTRACT NO. CC-3-536
RIO INDIO WATER SUPPLY STUDY
APPENDIX A - HYDROLOGY AND METEOROLOGY

Bed Material Average
Particle Size Distribution

MWH TAMS

DATE: APRIL, 2003

EXHIBIT: 59

ATTACHMENTS

Attachment 1 – Inventory of Hydrologic Data Obtained from the ACP and Review of Previous Hydrologic Analyses

ATTACHMENT I

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 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 Chorro 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 Coclécito	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 Coclécito	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 2^0 C through the year. Mean annual temperature varies from about 26^0 C at the dam site to about 24^0 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 18^0 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 areal 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, Seccion de Hidrologia, 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 79^0 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.

Qmean annual = 34 $A^{0.58}$ equation 1, for zones I and II
Qmean annual = 27 $A^{0.58}$ equation 2, for zones III and IV
Qmean annual = 13 $A^{0.58}$ equation 3, for zones V and VI
Qmean 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	**A**	**B**	**C**	**D**
	Ratios (Qmax/Qmean annual)			
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

Zone	Applicable Equation	Ratio (Qmax/Qmean annual)
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 77^0 F. The maximum 12-hour persisting dew point on the Gulf of Mexico Coast of the United States ranges about 3^0 F to 4^0 F below sea temperatures within a few hundred miles." "Based on the seasonal variation of sea surface temperature and a dew point 3^0 F below the sea surface temperature, the seasonal variation of the maximum 12-hour persisting dew point was estimated as:

November – February	77^0 F
March	77.5^0 F
April – August	78^0 F
September – October	79^0 F

The 12-hour dew point for the December 14-15, 1944 storm was estimated to be 72^0 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 79^0 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 areal 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. Areal average rainfall for each storm was expressed in percent of 10-mi^2 values. Values with least decrease with increasing area were adopted as:

Area (mi^2)	Percent of 10-mi^2 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-mi^2 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 mi^2. 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 develop for each storm as percent of 3-day, 10-mi^2 rainfall

6. Depth area curves were not enveloped, 75 percentile values were determine 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.

Attachment 2 – HEC-1 Output

Rio Indio – HEC-1 Input File

```
ID       PROBABLE MAXIMUM FLOOD, RIO INDIO AT DAM SITE
ID       MAXIMUM NORMAL POOL ELEVATION 80 METERS
ID       PMP AND TIME DISTRIBUTION FROM PREVIOUS REPORTS
IT     60 01NOV02      600          06NOV02      600
IN     60 01NOV02      600
IM
IO      1       2
KK BASIN
KM       RIO INDIO AT DAM SITE - PMF COMPUTATIONS
BA 381.1
BF     50
PB    711
PI    0.8     0.8     0.8     0.8     0.8     0.8     0.9     0.9     0.9     0.9
PI    0.9     1.0     1.0     1.0     1.0     1.1     1.1     1.2     1.4     1.6
PI    1.8     2.0     2.5     3.0     3.0     4.0    14.0    15.0     6.0     4.0
PI    3.0     2.5     2.3     1.9     1.7     1.4     1.3     1.2     1.1     1.1
PI    1.0     1.0     1.0     0.9     0.9     0.9     0.9     0.9
LU    0.0     3.0    11.2
UC    8.0     3.5
UA   21.4    32.1    42.8    69.8   124.8   179.9   244.6   300.6   346.5   381.1
KK    DAM
KM       ROUTE THROUGH RESERVOIR
RS      1    ELEV    80.0
SV473300  597600  747100  927100 1116500 1306800 1498400 1703900 1944000 2213000
SE   50.0    55.0    60.0    65.0    70.0    75.0    80.0    85.0    90.0    95.0
SQ    0.0      94     184     288     429     579     733     912    1088    1275
SE   80.0    81.0    81.5    82.0    82.5    83.0    83.5    84.0    84.5    85.0
ZZ
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1***************************************                                       ***************************************
 *                                     *                                       *                                     *
 *   FLOOD HYDROGRAPH PACKAGE  (HEC-1)  *                                      *   U.S. ARMY CORPS OF ENGINEERERS    *
 *            SEPTEMBER 1990            *                                      *   HYDROLOGIC ENGINEERING CENTER     *
 *             VERSION 4.0              *                                      *        609 SECOND STREET            *
 *                                     *                                       *      DAVIS, CALIFORNIA 95616        *
 *  RUN DATE  10/30/2002  TIME  12:22:39 *                                     *          (916) 756-1104             *
 *                                     *                                       *                                     *
 ***************************************                                       ***************************************


                              X     X  XXXXXXX   XXXXX          X
                              X     X  X        X     X        XX
                              X     X  X        X               X
                              XXXXXXX  XXXX     X       XXXXX   X
                              X     X  X        X               X
                              X     X  X        X     X         X
                              X     X  XXXXXXX   XXXXX         XXX


         THIS PROGRAM REPLACES ALL PREVIOUS VERSIONS OF HEC-1 KNOWN AS HEC1 (JAN 73), HEC1GS, HEC1DB, AND HEC1KW.

         THE DEFINITIONS OF VARIABLES -RTIMP- AND -RTIOR- HAVE CHANGED FROM THOSE USED WITH THE 1973-STYLE INPUT STRUCTURE.
         THE DEFINITION OF -AMSKK- ON RM-CARD WAS CHANGED WITH REVISIONS DATED 28 SEP 81. THIS IS THE FORTRAN77 VERSION
         NEW OPTIONS: DAMBREAK OUTFLOW SUBMERGENCE , SINGLE EVENT DAMAGE CALCULATION, DSS:WRITE STAGE FREQUENCY,
         DSS:READ TIME SERIES AT DESIRED CALCULATION INTERVAL   LOSS RATE:GREEN AND AMPT INFILTRATION
         KINEMATIC WAVE: NEW FINITE DIFFERENCE ALGORITHM

1                                    HEC-1 INPUT                                            PAGE  1

        LINE        ID.......1.......2.......3.......4.......5.......6.......7.......8.......9......10

          1         ID        PROBABLE MAXIMUM FLOOD, RIO INDIO AT DAM SITE
          2         ID        MAXIMUM NORMAL POOL ELEVATION 80 METERS
          3         ID        PMP AND TIME DISTRIBUTION FROM PREVIOUS REPORTS
          4         IT      60 01NOV02     600           06NOV02     600
          5         IN      60 01NOV02     600
          6         IM
          7         IO       1       2

          8         KK   BASIN
          9         KM        RIO INDIO AT DAM SITE - PMF COMPUTATIONS
         10         BA   381.1
         11         BF      50
```

```
          12           PB     711
          13           PI     0.8      0.8      0.8      0.8      0.8      0.8      0.9      0.9      0.9      0.9
          14           PI     0.9      1.0      1.0      1.0      1.0      1.1      1.1      1.2      1.4      1.6
          15           PI     1.8      2.0      2.5      3.0      3.0      4.0     14.0     15.0      6.0      4.0
          16           PI     3.0      2.5      2.3      1.9      1.7      1.4      1.3      1.2      1.1      1.1
          17           PI     1.0      1.0      1.0      0.9      0.9      0.9      0.9      0.9
          18           LU     0.0      3.0     11.2
          19           UC     8.0      3.5
          20           UA    21.4     32.1     42.8     69.8    124.8    179.9    244.6    300.6    346.5    381.1

          21           KK     DAM
          22           KM        ROUTE THROUGH RESERVOIR
          23           RS       1     ELEV     80.0
          24           SV  473300   597600   747100   927100  1116500  1306800  1498400  1703900  1944000  2213000
          25           SE    50.0     55.0     60.0     65.0     70.0     75.0     80.0     85.0     90.0     95.0
          26           SQ     0.0       94      184      288      429      579      733      912     1088     1275
          27           SE    80.0     81.0     81.5     82.0     82.5     83.0     83.5     84.0     84.5     85.0
          28           ZZ
1****************************************                                                   ***************************************
*                                       *                                                   *                                     *
*   FLOOD HYDROGRAPH PACKAGE  (HEC-1)   *                                                   *   U.S. ARMY CORPS OF ENGINEERS      *
*           SEPTEMBER 1990              *                                                   *   HYDROLOGIC ENGINEERING CENTER     *
*             VERSION 4.0               *                                                   *         609 SECOND STREET           *
*                                       *                                                   *      DAVIS, CALIFORNIA 95616        *
*  RUN DATE  10/30/2002  TIME  12:22:39 *                                                   *          (916) 756-1104             *
*                                       *                                                   *                                     *
*****************************************                                                   ***************************************


                          PROBABLE MAXIMUM FLOOD, RIO INDIO AT DAM SITE
                          MAXIMUM NORMAL POOL ELEVATION 80 METERS
                          PMP AND TIME DISTRIBUTION FROM PREVIOUS REPORTS

   7 IO          OUTPUT CONTROL VARIABLES
                       IPRNT           1  PRINT CONTROL
                       IPLOT           2  PLOT CONTROL
                       QSCAL          0.  HYDROGRAPH PLOT SCALE

     IT          HYDROGRAPH TIME DATA
                        NMIN          60  MINUTES IN COMPUTATION INTERVAL
                       IDATE      1NOV 2  STARTING DATE
                       ITIME        0600  STARTING TIME
                          NQ         121  NUMBER OF HYDROGRAPH ORDINATES
                      NDDATE      6NOV 2  ENDING DATE
                      NDTIME        0600  ENDING TIME
                      ICENT           19  CENTURY MARK
```

```
          COMPUTATION INTERVAL    1.00 HOURS
               TOTAL TIME BASE  120.00 HOURS

 METRIC UNITS
      DRAINAGE AREA         SQUARE KILOMETERS
      PRECIPITATION DEPTH   MILLIMETERS
      LENGTH, ELEVATION     METERS
      FLOW                  CUBIC METERS PER SECOND
      STORAGE VOLUME        CUBIC METERS
      SURFACE AREA          SQUARE METERS
      TEMPERATURE           DEGREES CELSIUS

*** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** ***

             **************
             *            *
     8 KK    *    BASIN   *
             *            *
             **************
                            RIO INDIO AT DAM SITE - PMF COMPUTATIONS

     5 IN          TIME DATA FOR INPUT TIME SERIES
                         JXMIN          60  TIME INTERVAL IN MINUTES
                        JXDATE      1NOV 2  STARTING DATE
                        JXTIME         600  STARTING TIME

                 SUBBASIN RUNOFF DATA

    10 BA          SUBBASIN CHARACTERISTICS
                         TAREA      381.10  SUBBASIN AREA

    11 BF          BASE FLOW CHARACTERISTICS
                         STRTQ       50.00  INITIAL FLOW
                         QRCSN         .00  BEGIN BASE FLOW RECESSION
                         RTIOR     1.00000  RECESSION CONSTANT

                   PRECIPITATION DATA

    12 PB                STORM      711.00  BASIN TOTAL PRECIPITATION

    13 PI            INCREMENTAL PRECIPITATION PATTERN
                       .80        .80        .80        .80        .80        .80        .90        .90        .90        .90
                       .90       1.00       1.00       1.00       1.00       1.10       1.10       1.20       1.40       1.60
                      1.80       2.00       2.50       3.00       3.00       4.00      14.00      15.00       6.00       4.00
                      3.00       2.50       2.30       1.90       1.70       1.40       1.30       1.20       1.10       1.10
                      1.00       1.00       1.00        .90        .90        .90        .90        .90
```

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18 LU          UNIFORM LOSS RATE
                     STRTL           .00  INITIAL LOSS
                     CNSTL          3.00  UNIFORM LOSS RATE
                     RTIMP         11.20  PERCENT IMPERVIOUS AREA

19 UC          CLARK UNITGRAPH
                        TC          8.00  TIME OF CONCENTRATION
                         R          3.50  STORAGE COEFFICIENT

20 UA          ACCUMULATED-AREA VS. TIME,  10 ORDINATES
                    21.4      32.1      42.8      69.8     124.8     179.9     244.6     300.6     346.5     381.1

                                                      ***

                                             UNIT HYDROGRAPH PARAMETERS
                                       CLARK   TC=  8.00 HR,        R=  3.50 HR
                                      SNYDER   TP=  7.09 HR,       CP=   .86

                                                  UNIT HYDROGRAPH
                                             24 END-OF-PERIOD ORDINATES
               1.        3.        4.        7.        9.       12.       13.       13.       11.        8.
               6.        5.        4.        3.        2.        1.        1.        1.        1.        0.
               0.        0.        0.        0.
```

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HYDROGRAPH AT STATION BASIN

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DA	MON	HRMN	ORD	RAIN	LOSS	EXCESS	COMP Q	*	DA	MON	HRMN	ORD	RAIN	LOSS	EXCESS	COMP Q
1	NOV	0600	1	.00	.00	.00	50.	*	3	NOV	1900	62	.00	.00	.00	90.
1	NOV	0700	2	5.69	2.66	3.02	54.	*	3	NOV	2000	63	.00	.00	.00	79.
1	NOV	0800	3	5.69	2.66	3.02	61.	*	3	NOV	2100	64	.00	.00	.00	71.
1	NOV	0900	4	5.69	2.66	3.02	73.	*	3	NOV	2200	65	.00	.00	.00	65.
1	NOV	1000	5	5.69	2.66	3.02	93.	*	3	NOV	2300	66	.00	.00	.00	61.
1	NOV	1100	6	5.69	2.66	3.02	121.	*	4	NOV	0000	67	.00	.00	.00	58.
1	NOV	1200	7	5.69	2.66	3.02	157.	*	4	NOV	0100	68	.00	.00	.00	55.
1	NOV	1300	8	6.40	2.66	3.74	197.	*	4	NOV	0200	69	.00	.00	.00	54.
1	NOV	1400	9	6.40	2.66	3.74	238.	*	4	NOV	0300	70	.00	.00	.00	52.
1	NOV	1500	10	6.40	2.66	3.74	274.	*	4	NOV	0400	71	.00	.00	.00	51.
1	NOV	1600	11	6.40	2.66	3.74	304.	*	4	NOV	0500	72	.00	.00	.00	51.
1	NOV	1700	12	6.40	2.66	3.73	330.	*	4	NOV	0600	73	.00	.00	.00	50.
1	NOV	1800	13	7.11	2.66	4.45	353.	*	4	NOV	0700	74	.00	.00	.00	50.
1	NOV	1900	14	7.11	2.66	4.45	375.	*	4	NOV	0800	75	.00	.00	.00	50.
1	NOV	2000	15	7.11	2.66	4.45	395.	*	4	NOV	0900	76	.00	.00	.00	50.
1	NOV	2100	16	7.11	2.66	4.45	414.	*	4	NOV	1000	77	.00	.00	.00	50.
1	NOV	2200	17	7.82	2.66	5.16	432.	*	4	NOV	1100	78	.00	.00	.00	50.
1	NOV	2300	18	7.82	2.66	5.16	450.	*	4	NOV	1200	79	.00	.00	.00	50.

2 NOV 0000	19	8.53	2.66	5.87	468.	*	4 NOV 1300	80	.00	.00	.00	50.
2 NOV 0100	20	9.95	2.66	7.29	490.	*	4 NOV 1400	81	.00	.00	.00	50.
2 NOV 0200	21	11.38	2.66	8.71	516.	*	4 NOV 1500	82	.00	.00	.00	50.
2 NOV 0300	22	12.80	2.66	10.13	548.	*	4 NOV 1600	83	.00	.00	.00	50.
2 NOV 0400	23	14.22	2.66	11.56	591.	*	4 NOV 1700	84	.00	.00	.00	50.
2 NOV 0500	24	17.77	2.66	15.11	649.	*	4 NOV 1800	85	.00	.00	.00	50.
2 NOV 0600	25	21.33	2.66	18.67	728.	*	4 NOV 1900	86	.00	.00	.00	50.
2 NOV 0700	26	21.33	2.66	18.67	826.	*	4 NOV 2000	87	.00	.00	.00	50.
2 NOV 0800	27	28.44	2.66	25.78	953.	*	4 NOV 2100	88	.00	.00	.00	50.
2 NOV 0900	28	99.54	2.66	96.88	1190.	*	4 NOV 2200	89	.00	.00	.00	50.
2 NOV 1000	29	106.65	2.66	103.99	1558.	*	4 NOV 2300	90	.00	.00	.00	50.
2 NOV 1100	30	42.66	2.66	40.00	1976.	*	5 NOV 0000	91	.00	.00	.00	50.
2 NOV 1200	31	28.44	2.66	25.78	2492.	*	5 NOV 0100	92	.00	.00	.00	50.
2 NOV 1300	32	21.33	2.66	18.67	3115.	*	5 NOV 0200	93	.00	.00	.00	50.
2 NOV 1400	33	17.77	2.66	15.11	3708.	*	5 NOV 0300	94	.00	.00	.00	50.
2 NOV 1500	34	16.35	2.66	13.69	4145.	*	5 NOV 0400	95	.00	.00	.00	50.
2 NOV 1600	35	13.51	2.66	10.85	4345.	*	5 NOV 0500	96	.00	.00	.00	50.
2 NOV 1700	36	12.09	2.66	9.42	4230.	*	5 NOV 0600	97	.00	.00	.00	50.
2 NOV 1800	37	9.95	2.66	7.29	3819.	*	5 NOV 0700	98	.00	.00	.00	50.
2 NOV 1900	38	9.24	2.66	6.58	3305.	*	5 NOV 0800	99	.00	.00	.00	50.
2 NOV 2000	39	8.53	2.66	5.87	2830.	*	5 NOV 0900	100	.00	.00	.00	50.
2 NOV 2100	40	7.82	2.66	5.16	2416.	*	5 NOV 1000	101	.00	.00	.00	50.
2 NOV 2200	41	7.82	2.66	5.16	2062.	*	5 NOV 1100	102	.00	.00	.00	50.
2 NOV 2300	42	7.11	2.66	4.45	1761.	*	5 NOV 1200	103	.00	.00	.00	50.
3 NOV 0000	43	7.11	2.66	4.45	1508.	*	5 NOV 1300	104	.00	.00	.00	50.
3 NOV 0100	44	7.11	2.66	4.45	1298.	*	5 NOV 1400	105	.00	.00	.00	50.
3 NOV 0200	45	6.40	2.66	3.74	1125.	*	5 NOV 1500	106	.00	.00	.00	50.
3 NOV 0300	46	6.40	2.66	3.74	985.	*	5 NOV 1600	107	.00	.00	.00	50.
3 NOV 0400	47	6.40	2.66	3.74	871.	*	5 NOV 1700	108	.00	.00	.00	50.
3 NOV 0500	48	6.40	2.66	3.74	779.	*	5 NOV 1800	109	.00	.00	.00	50.
3 NOV 0600	49	6.40	2.66	3.74	704.	*	5 NOV 1900	110	.00	.00	.00	50.
3 NOV 0700	50	.00	.00	.00	640.	*	5 NOV 2000	111	.00	.00	.00	50.
3 NOV 0800	51	.00	.00	.00	581.	*	5 NOV 2100	112	.00	.00	.00	50.
3 NOV 0900	52	.00	.00	.00	519.	*	5 NOV 2200	113	.00	.00	.00	50.
3 NOV 1000	53	.00	.00	.00	457.	*	5 NOV 2300	114	.00	.00	.00	50.
3 NOV 1100	54	.00	.00	.00	402.	*	6 NOV 0000	115	.00	.00	.00	50.
3 NOV 1200	55	.00	.00	.00	344.	*	6 NOV 0100	116	.00	.00	.00	50.
3 NOV 1300	56	.00	.00	.00	286.	*	6 NOV 0200	117	.00	.00	.00	50.
3 NOV 1400	57	.00	.00	.00	230.	*	6 NOV 0300	118	.00	.00	.00	50.
3 NOV 1500	58	.00	.00	.00	184.	*	6 NOV 0400	119	.00	.00	.00	50.
3 NOV 1600	59	.00	.00	.00	149.	*	6 NOV 0500	120	.00	.00	.00	50.
3 NOV 1700	60	.00	.00	.00	123.	*	6 NOV 0600	121	.00	.00	.00	50.
3 NOV 1800	61	.00	.00	.00	104.	*						
						*						

****

TOTAL RAINFALL = 711.00, TOTAL LOSS = 127.87, TOTAL EXCESS = 583.13

PEAK FLOW	TIME	MAXIMUM AVERAGE FLOW			
		6-HR	24-HR	72-HR	120.00-HR

```
+  (CU M/S)     (HR)
                         (CU M/S)
+    4345.     34.00                3909.      2167.       904.        562.
                             (MM)  221.567    491.335    614.672     637.343
                      (1000 CU M)   84439.    187248.    234251.     242892.

                         CUMULATIVE AREA =  381.10 SQ KM

1                                                        STATION    BASIN

                          (O) OUTFLOW
          0.       500.     1000.     1500.     2000.     2500.     3000.     3500.     4000.     4500.       0.        0.        0.
                                                                                                      (L) PRECIP,   (X) EXCESS
          0.         0.         0.         0.         0.         0.         0.         0.         0.       120.       80.       40.        0.
 DAHRMN PER
  10600   1.O--------.---------.---------.---------.---------.---------.---------.---------.---------.---------.---------.---------.
  10700   2.O        .         .         .         .         .         .         .         .         .         .         .        X.
  10800   3.O        .         .         .         .         .         .         .         .         .         .         .        X.
  10900   4.O        .         .         .         .         .         .         .         .         .         .         .        X.
  11000   5. O       .         .         .         .         .         .         .         .         .         .         .        X.
  11100   6. O       .         .         .         .         .         .         .         .         .         .         .        X.
  11200   7.  O      .         .         .         .         .         .         .         .         .         .         .        X.
  11300   8.   O     .         .         .         .         .         .         .         .         .         .         .       LX.
  11400   9.    O    .         .         .         .         .         .         .         .         .         .         .       LX.
  11500  10.    O    .         .         .         .         .         .         .         .         .         .         .       LX.
  11600  11. . . O . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . LX.
  11700  12.      O  .         .         .         .         .         .         .         .         .         .         .       LX.
  11800  13.      O  .         .         .         .         .         .         .         .         .         .         .       LX.
  11900  14.       O .         .         .         .         .         .         .         .         .         .         .       LX.
  12000  15.       O .         .         .         .         .         .         .         .         .         .         .       LX.
  12100  16.       O .         .         .         .         .         .         .         .         .         .         .       LX.
  12200  17.        O.         .         .         .         .         .         .         .         .         .         .       LX.
  12300  18.        O.         .         .         .         .         .         .         .         .         .         .       LX.
  20000  19.        O.         .         .         .         .         .         .         .         .         .         .       LX.
  20100  20.         O         .         .         .         .         .         .         .         .         .         .       XX.
  20200  21. . . . . O . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . .LXX.
  20300  22.         .O        .         .         .         .         .         .         .         .         .         .      XXX.
  20400  23.         . O       .         .         .         .         .         .         .         .         .         .     LXXX.
  20500  24.         .  O      .         .         .         .         .         .         .         .         .         .     XXXX.
  20600  25.         .    O    .         .         .         .         .         .         .         .         .         .    XXXXX.
  20700  26.         .      O  .         .         .         .         .         .         .         .         .         .    XXXXX.
  20800  27.         .        O.         .         .         .         .         .         .         .         .         .  LXXXXXX.
  20900  28.         .         .    O    .         .         .         .         .         .         .  LXXXXXXXXXXXXXXXXXXXXXXXXX.
  21000  29.         .         .         .O        .         .         .         .         .         . LXXXXXXXXXXXXXXXXXXXXXXXXXXX.
  21100  30.         .         .         .         O         .         .         .         .         .         .   LXXXXXXXXXXX.
  21200  31. . . . . . . . . . . . . . . . . . . . . . . . . O . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . .LXXXXXXX.
  21300  32.         .         .         .         .         .         . O       .         .         .         .         .    XXXXX.
  21400  33.         .         .         .         .         .         .         .    O    .         .         .         .     XXXX.
  21500  34.         .         .         .         .         .         .         .         . O       .         .         .     LXXX.
  21600  35.         .         .         .         .         .         .         .         .     O   .         .         .      XXX.
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```
21700  36.         .         .         .         .         .         .         .         .    O    .         .         .      LXX.
21800  37.         .         .         .         .         .         .         .     O   .         .         .         .       XX.
21900  38.         .         .         .         .         .         .     O   .         .         .         .         .       XX.
22000  39.         .         .         .         .         .      O  .         .         .         .         .         .       LX.
22100  40.         .         .         .         .       O .         .         .         .         .         .         .       LX.
22200  41. . . . . . . . . . . . . . . . . . . . O. . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . LX.
22300  42.         .         .         .    O    .         .         .         .         .         .         .         .       LX.
30000  43.         .         .         O         .         .         .         .         .         .         .         .       LX.
30100  44.         .         .     O   .         .         .         .         .         .         .         .         .       LX.
30200  45.         .         .  O      .         .         .         .         .         .         .         .         .       LX.
30300  46.         .         O         .         .         .         .         .         .         .         .         .       LX.
30400  47.         .      O  .         .         .         .         .         .         .         .         .         .       LX.
30500  48.         .     O   .         .         .         .         .         .         .         .         .         .       LX.
30600  49.         .   O     .         .         .         .         .         .         .         .         .         .       LX.
30700  50.         .  O      .         .         .         .         .         .         .         .         .         .         .
30800  51. . . . . O . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . .
30900  52.         O         .         .         .         .         .         .         .         .         .         .         .
31000  53.        O.         .         .         .         .         .         .         .         .         .         .         .
31100  54.       O .         .         .         .         .         .         .         .         .         .         .         .
31200  55.      O  .         .         .         .         .         .         .         .         .         .         .         .
31300  56.     O   .         .         .         .         .         .         .         .         .         .         .         .
31400  57.    O    .         .         .         .         .         .         .         .         .         .         .         .
31500  58.   O     .         .         .         .         .         .         .         .         .         .         .         .
31600  59.  O      .         .         .         .         .         .         .         .         .         .         .         .
31700  60. O       .         .         .         .         .         .         .         .         .         .         .         .
31800  61. O . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . .
31900  62. O       .         .         .         .         .         .         .         .         .         .         .         .
32000  63. O       .         .         .         .         .         .         .         .         .         .         .         .
32100  64.O        .         .         .         .         .         .         .         .         .         .         .         .
32200  65.O        .         .         .         .         .         .         .         .         .         .         .         .
32300  66.O        .         .         .         .         .         .         .         .         .         .         .         .
40000  67.O        .         .         .         .         .         .         .         .         .         .         .         .
40100  68.O        .         .         .         .         .         .         .         .         .         .         .         .
40200  69.O        .         .         .         .         .         .         .         .         .         .         .         .
40300  70.O        .         .         .         .         .         .         .         .         .         .         .         .
40400  71.O. . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . .
40500  72.O        .         .         .         .         .         .         .         .         .         .         .         .
40600  73.O        .         .         .         .         .         .         .         .         .         .         .         .
40700  74.O        .         .         .         .         .         .         .         .         .         .         .         .
40800  75.O        .         .         .         .         .         .         .         .         .         .         .         .
40900  76.O        .         .         .         .         .         .         .         .         .         .         .         .
41000  77.O        .         .         .         .         .         .         .         .         .         .         .         .
41100  78.O        .         .         .         .         .         .         .         .         .         .         .         .
41200  79.O        .         .         .         .         .         .         .         .         .         .         .         .
41300  80.O        .         .         .         .         .         .         .         .         .         .         .         .
41400  81.O. . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . .
41500  82.O        .         .         .         .         .         .         .         .         .         .         .         .
41600  83.O        .         .         .         .         .         .         .         .         .         .         .         .
41700  84.O        .         .         .         .         .         .         .         .         .         .         .         .
41800  85.O        .         .         .         .         .         .         .         .         .         .         .         .
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 41900  86.0        .         .         .         .         .         .         .         .         .         .         .         .
 42000  87.0        .         .         .         .         .         .         .         .         .         .         .         .
 42100  88.0        .         .         .         .         .         .         .         .         .         .         .         .
 42200  89.0        .         .         .         .         .         .         .         .         .         .         .         .
 42300  90.0        .         .         .         .         .         .         .         .         .         .         .         .
 50000  91.0. . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . .
 50100  92.0        .         .         .         .         .         .         .         .         .         .         .         .
 50200  93.0        .         .         .         .         .         .         .         .         .         .         .         .
 50300  94.0        .         .         .         .         .         .         .         .         .         .         .         .
 50400  95.0        .         .         .         .         .         .         .         .         .         .         .         .
 50500  96.0        .         .         .         .         .         .         .         .         .         .         .         .
 50600  97.0        .         .         .         .         .         .         .         .         .         .         .         .
 50700  98.0        .         .         .         .         .         .         .         .         .         .         .         .
 50800  99.0        .         .         .         .         .         .         .         .         .         .         .         .
 50900 100.0        .         .         .         .         .         .         .         .         .         .         .         .
 51000 101.0. . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . .
 51100 102.0        .         .         .         .         .         .         .         .         .         .         .         .
 51200 103.0        .         .         .         .         .         .         .         .         .         .         .         .
 51300 104.0        .         .         .         .         .         .         .         .         .         .         .         .
 51400 105.0        .         .         .         .         .         .         .         .         .         .         .         .
 51500 106.0        .         .         .         .         .         .         .         .         .         .         .         .
 51600 107.0        .         .         .         .         .         .         .         .         .         .         .         .
 51700 108.0        .         .         .         .         .         .         .         .         .         .         .         .
 51800 109.0        .         .         .         .         .         .         .         .         .         .         .         .
 51900 110.0        .         .         .         .         .         .         .         .         .         .         .         .
 52000 111.0. . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . .
 52100 112.0        .         .         .         .         .         .         .         .         .         .         .         .
 52200 113.0        .         .         .         .         .         .         .         .         .         .         .         .
 52300 114.0        .         .         .         .         .         .         .         .         .         .         .         .
 60000 115.0        .         .         .         .         .         .         .         .         .         .         .         .
 60100 116.0        .         .         .         .         .         .         .         .         .         .         .         .
 60200 117.0        .         .         .         .         .         .         .         .         .         .         .         .
 60300 118.0        .         .         .         .         .         .         .         .         .         .         .         .
 60400 119.0        .         .         .         .         .         .         .         .         .         .         .         .
 60500 120.0        .         .         .         .         .         .         .         .         .         .         .         .
 60600 121.0--------.---------.---------.---------.---------.---------.---------.---------.---------.---------.---------.---------.
1


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              *            *
   21 KK      *       DAM  *
              *            *
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                                  ROUTE THROUGH RESERVOIR
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HYDROGRAPH ROUTING DATA

23 RS STORAGE ROUTING

NSTPS	1	NUMBER OF SUBREACHES
ITYP	ELEV	TYPE OF INITIAL CONDITION
RSVRIC	80.00	INITIAL CONDITION
X	.00	WORKING R AND D COEFFICIENT

24 SV	STORAGE	473300.0	597600.0	747100.0	927100.0	1116500.0	1306800.0	1498400.0	1703900.0	1944000.0	2213000.0
25 SE	ELEVATION	50.00	55.00	60.00	65.00	70.00	75.00	80.00	85.00	90.00	95.00
26 SQ	DISCHARGE	0.	94.	184.	288.	429.	579.	733.	912.	1088.	1275.
27 SE	ELEVATION	80.00	81.00	81.50	82.00	82.50	83.00	83.50	84.00	84.50	85.00

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COMPUTED STORAGE-OUTFLOW-ELEVATION DATA

STORAGE	473300.00	597600.00	747100.00	927100.00	1116500.00	1306800.00	1498400.00	1539500.00	1560050.00	1580600.00
OUTFLOW	.00	.00	.00	.00	.00	.00	.00	94.00	184.00	288.00
ELEVATION	50.00	55.00	60.00	65.00	70.00	75.00	80.00	81.00	81.50	82.00

STORAGE	1601150.00	1621700.00	1642250.00	1662800.00	1683350.00	1703900.00	1944000.00	2213000.00
OUTFLOW	429.00	579.00	733.00	912.00	1088.00	1275.00	3145.00	5015.00
ELEVATION	82.50	83.00	83.50	84.00	84.50	85.00	90.00	95.00

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HYDROGRAPH AT STATION DAM

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DA	MON	HRMN	ORD	OUTFLOW	STORAGE	STAGE	*	DA	MON	HRMN	ORD	OUTFLOW	STORAGE	STAGE	*	DA	MON	HRMN	ORD	OUTFLOW	STORAGE	STAGE
1	NOV	0600	1	0.	1498400.0	80.0	*	2	NOV	2300	42	895.	1660884.0	84.0	*	4	NOV	1600	83	440.	1602711.0	82.5
1	NOV	0700	2	0.	1498586.0	80.0	*	3	NOV	0000	43	918.	1663505.0	84.0	*	4	NOV	1700	84	430.	1601324.0	82.5
1	NOV	0800	3	1.	1498790.0	80.0	*	3	NOV	0100	44	933.	1665224.0	84.1	*	4	NOV	1800	85	421.	1599972.0	82.5
1	NOV	0900	4	1.	1499028.0	80.0	*	3	NOV	0200	45	941.	1666213.0	84.1	*	4	NOV	1900	86	412.	1598653.0	82.4
1	NOV	1000	5	2.	1499321.0	80.0	*	3	NOV	0300	46	945.	1666616.0	84.1	*	4	NOV	2000	87	403.	1597366.0	82.4
1	NOV	1100	6	3.	1499697.0	80.0	*	3	NOV	0400	47	944.	1666558.0	84.1	*	4	NOV	2100	88	394.	1596110.0	82.4
1	NOV	1200	7	4.	1500185.0	80.0	*	3	NOV	0500	48	941.	1666136.0	84.1	*	4	NOV	2200	89	386.	1594886.0	82.3
1	NOV	1300	8	5.	1500804.0	80.1	*	3	NOV	0600	49	935.	1665432.0	84.1	*	4	NOV	2300	90	378.	1593691.0	82.3
1	NOV	1400	9	7.	1501563.0	80.1	*	3	NOV	0700	50	927.	1664501.0	84.0	*	5	NOV	0000	91	370.	1592525.0	82.3
1	NOV	1500	10	9.	1502455.0	80.1	*	3	NOV	0800	51	917.	1663379.0	84.0	*	5	NOV	0100	92	362.	1591388.0	82.3

1 NOV 1600	11	12.1503460.0	80.1 *	3 NOV 0900	52	906.1662077.0	84.0 *	5 NOV 0200	93	354.1590278.0	82.2
1 NOV 1700	12	14.1504555.0	80.1 *	3 NOV 1000	53	893.1660597.0	83.9 *	5 NOV 0300	94	347.1589196.0	82.2
1 NOV 1800	13	17.1505730.0	80.2 *	3 NOV 1100	54	879.1658955.0	83.9 *	5 NOV 0400	95	340.1588140.0	82.2
1 NOV 1900	14	20.1506976.0	80.2 *	3 NOV 1200	55	863.1657162.0	83.9 *	5 NOV 0500	96	333.1587109.0	82.2
1 NOV 2000	15	23.1508286.0	80.2 *	3 NOV 1300	56	846.1655220.0	83.8 *	5 NOV 0600	97	326.1586104.0	82.1
1 NOV 2100	16	26.1509655.0	80.3 *	3 NOV 1400	57	828.1653136.0	83.8 *	5 NOV 0700	98	319.1585124.0	82.1
1 NOV 2200	17	29.1511078.0	80.3 *	3 NOV 1500	58	809.1650936.0	83.7 *	5 NOV 0800	99	312.1584167.0	82.1
1 NOV 2300	18	32.1512554.0	80.3 *	3 NOV 1600	59	789.1648660.0	83.7 *	5 NOV 0900	100	306.1583234.0	82.1
2 NOV 0000	19	36.1514083.0	80.4 *	3 NOV 1700	60	769.1646347.0	83.6 *	5 NOV 1000	101	300.1582323.0	82.0
2 NOV 0100	20	40.1515673.0	80.4 *	3 NOV 1800	61	748.1644025.0	83.5 *	5 NOV 1100	102	294.1581435.0	82.0
2 NOV 0200	21	43.1517335.0	80.5 *	3 NOV 1900	62	729.1641715.0	83.5 *	5 NOV 1200	103	288.1580568.0	82.0
2 NOV 0300	22	47.1519088.0	80.5 *	3 NOV 2000	63	712.1639426.0	83.4 *	5 NOV 1300	104	284.1579719.0	82.0
2 NOV 0400	23	52.1520961.0	80.5 *	3 NOV 2100	64	695.1637165.0	83.4 *	5 NOV 1400	105	279.1578886.0	82.0
2 NOV 0500	24	56.1522999.0	80.6 *	3 NOV 2200	65	678.1634939.0	83.3 *	5 NOV 1500	106	275.1578068.0	81.9
2 NOV 0600	25	61.1525266.0	80.7 *	3 NOV 2300	66	662.1632755.0	83.3 *	5 NOV 1600	107	271.1577265.0	81.9
2 NOV 0700	26	67.1527832.0	80.7 *	4 NOV 0000	67	646.1630615.0	83.2 *	5 NOV 1700	108	267.1576476.0	81.9
2 NOV 0800	27	74.1530780.0	80.8 *	4 NOV 0100	68	630.1628522.0	83.2 *	5 NOV 1800	109	263.1575701.0	81.9
2 NOV 0900	28	82.1534357.0	80.9 *	4 NOV 0200	69	615.1626478.0	83.1 *	5 NOV 1900	110	259.1574941.0	81.9
2 NOV 1000	29	93.1538989.0	81.0 *	4 NOV 0300	70	600.1624482.0	83.1 *	5 NOV 2000	111	256.1574194.0	81.8
2 NOV 1100	30	118.1544973.0	81.1 *	4 NOV 0400	71	585.1622535.0	83.0 *	5 NOV 2100	112	252.1573460.0	81.8
2 NOV 1200	31	151.1552531.0	81.3 *	4 NOV 0500	72	571.1620637.0	83.0 *	5 NOV 2200	113	248.1572740.0	81.8
2 NOV 1300	32	194.1562003.0	81.5 *	4 NOV 0600	73	558.1618786.0	82.9 *	5 NOV 2300	114	245.1572033.0	81.8
2 NOV 1400	33	252.1573481.0	81.8 *	4 NOV 0700	74	545.1616982.0	82.9 *	6 NOV 0000	115	241.1571339.0	81.8
2 NOV 1500	34	329.1586569.0	82.1 *	4 NOV 0800	75	532.1615224.0	82.8 *	6 NOV 0100	116	238.1570657.0	81.8
2 NOV 1600	35	424.1600493.0	82.5 *	4 NOV 0900	76	519.1613513.0	82.8 *	6 NOV 0200	117	234.1569987.0	81.7
2 NOV 1700	36	524.1614219.0	82.8 *	4 NOV 1000	77	507.1611845.0	82.8 *	6 NOV 0300	118	231.1569330.0	81.7
2 NOV 1800	37	616.1626654.0	83.1 *	4 NOV 1100	78	495.1610221.0	82.7 *	6 NOV 0400	119	228.1568684.0	81.7
2 NOV 1900	38	695.1637118.0	83.4 *	4 NOV 1200	79	484.1608639.0	82.7 *	6 NOV 0500	120	224.1568050.0	81.7
2 NOV 2000	39	762.1645539.0	83.6 *	4 NOV 1300	80	472.1607098.0	82.6 *	6 NOV 0600	121	221.1567428.0	81.7
2 NOV 2100	40	819.1652136.0	83.7 *	4 NOV 1400	81	461.1605597.0	82.6 *				
2 NOV 2200	41	863.1657167.0	83.9 *	4 NOV 1500	82	451.1604135.0	82.6 *				
			*				*				

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PEAK FLOW	TIME		MAXIMUM AVERAGE FLOW			
			6-HR	24-HR	72-HR	120.00-HR
+ (CU M/S)	(HR)					
		(CU M/S)				
+ 945.	45.00		939.	866.	595.	402.
		(MM)	53.229	196.282	404.930	456.216
		(1000 CU M)	20286.	74803.	154319.	173864.

PEAK STORAGE	TIME	MAXIMUM AVERAGE STORAGE			
		6-HR	24-HR	72-HR	120.00-HR
+ (1000 CU M)	(HR)				
1666616.	45.00	1665970.	1657492.	1622485.	1587588.

PEAK STAGE	TIME	MAXIMUM AVERAGE STAGE			
		6-HR	24-HR	72-HR	120.00-HR
+ (METERS)	(HR)				

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    84.09     45.00                84.08       83.87       83.02        82.17

                        CUMULATIVE AREA =  381.10 SQ KM

1                                                         STATION      DAM

                         (I) INFLOW,   (O) OUTFLOW
         0.      500.     1000.     1500.     2000.     2500.     3000.     3500.     4000.     4500.        0.        0.        0.
                                                                                (S) STORAGE
         0.        0.        0.        0.        0.        0.  1450000.  1500000.  1550000.  1600000.  1650000.  1700000.        0.
DAHRMN PER
 10600   10I--------.---------.---------.---------.---------.---------.---------S---------.---------.---------.---------.---------.
 10700   20I        .         .         .         .         .         .         S         .         .         .         .         .
 10800   30I        .         .         .         .         .         .         S         .         .         .         .         .
 10900   40I        .         .         .         .         .         .         S         .         .         .         .         .
 11000   50 I       .         .         .         .         .         .         S         .         .         .         .         .
 11100   60 I       .         .         .         .         .         .         S         .         .         .         .         .
 11200   70  I      .         .         .         .         .         .         S         .         .         .         .         .
 11300   80   I     .         .         .         .         .         .         S         .         .         .         .         .
 11400   90    I    .         .         .         .         .         .         S         .         .         .         .         .
 11500  100    I    .         .         .         .         .         .         S         .         .         .         .         .
 11600  110 . . I . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . .S. . . . . . . . . . . . . . . . . . . . . . . . .
 11700  120      I  .         .         .         .         .         .         .S        .         .         .         .         .
 11800  130      I  .         .         .         .         .         .         .S        .         .         .         .         .
 11900  140       I .         .         .         .         .         .         .S        .         .         .         .         .
 12000  150       I .         .         .         .         .         .         . S       .         .         .         .         .
 12100  16.O      I .         .         .         .         .         .         . S       .         .         .         .         .
 12200  17.O       I.         .         .         .         .         .         . S       .         .         .         .         .
 12300  18.O       I.         .         .         .         .         .         .  S      .         .         .         .         .
 20000  19.O       I.         .         .         .         .         .         .  S      .         .         .         .         .
 20100  20.O        I         .         .         .         .         .         .  S      .         .         .         .         .
 20200  21.O. . . . I . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . .S. . . . . . . . . . . . . . . . . . . . . . . .
 20300  22.O        .I        .         .         .         .         .         .   S     .         .         .         .         .
 20400  23.O        . I       .         .         .         .         .         .   S     .         .         .         .         .
 20500  24.O        .  I      .         .         .         .         .         .    S    .         .         .         .         .
 20600  25.O        .    I    .         .         .         .         .         .    S    .         .         .         .         .
 20700  26.O        .      I  .         .         .         .         .         .     S   .         .         .         .         .
 20800  27.O        .        I.         .         .         .         .         .     S   .         .         .         .         .
 20900  28. O       .         .   I     .         .         .         .         .      S  .         .         .         .         .
 21000  29. O       .         .         .I        .         .         .         .       S .         .         .         .         .
 21100  30. O       .         .         .         I         .         .         .        S.         .         .         .         .
 21200  31. .O. . . . . . . . . . . . . . . . . . . . . . . I . . . . . . . . . . . . . . . .S. . . . . . . . . . . . . . . . . . . . .
 21300  32.   O     .         .         .         .         .          I        .         . S       .         .         .         .
 21400  33.    O    .         .         .         .         .         .         .   I     .    S    .         .         .         .
 21500  34.      O  .         .         .         .         .         .         .         .  I   S  .         .         .         .
 21600  35.       O .         .         .         .         .         .         .         .      I  S         .         .         .
 21700  36.         O         .         .         .         .         .         .         .    I    .  S      .         .         .
 21800  37.         . O       .         .         .         .         .         .     I   .         .    S    .         .         .
 21900  38.         .   O     .         .         .         .         .     I   .         .         .      S  .         .         .
 22000  39.         .    O    .         .         .         .      I  .         .         .         .        S.         .         .
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22100  40.         .     O   .         .         .       I .         .         .         .         .         S         .         .
22200  41. . . . . . . . .O. . . . . . . . . . . .I. . . . . . . . . . . . . . . . . . . . . . . . . . . . . .S. . . . . . . . . .
22300  42.         .       O .         .    I    .         .         .         .         .         . S       .         .
30000  43.         .       O .         I         .         .         .         .         .         .  S      .         .
30100  44.         .        O.     I   .         .         .         .         .         .         .  S      .         .
30200  45.         .        O.  I      .         .         .         .         .         .         .  S      .         .
30300  46.         .        OI         .         .         .         .         .         .         .  S      .         .
30400  47.         .      I O.         .         .         .         .         .         .         .  S      .         .
30500  48.         .     I  O.         .         .         .         .         .         .         .  S      .         .
30600  49.         .   I    O.         .         .         .         .         .         .         .  S      .         .
30700  50.         .  I     O.         .         .         .         .         .         .         .  S      .         .
30800  51. . . . . . I . . O . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . .S. . . . . . . . .
30900  52.         I       O .         .         .         .         .         .         .         . S       .         .
31000  53.        I.       O .         .         .         .         .         .         .         . S       .         .
31100  54.       I .       O .         .         .         .         .         .         .         . S       .         .
31200  55.      I  .      O  .         .         .         .         .         .         .         .S        .         .
31300  56.     I   .      O  .         .         .         .         .         .         .         .S        .         .
31400  57.    I    .      O  .         .         .         .         .         .         .         .S        .         .
31500  58.   I     .     O   .         .         .         .         .         .         .         S         .         .
31600  59.  I      .     O   .         .         .         .         .         .         .         S         .         .
31700  60. I       .    O    .         .         .         .         .         .         .        S.         .         .
31800  61. I . . . . . .O. . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . .S. . . . . . . . . . .
31900  62. I       .    O    .         .         .         .         .         .         .       S .         .         .
32000  63. I       .   O     .         .         .         .         .         .         .       S .         .         .
32100  64.I        .   O     .         .         .         .         .         .         .      S  .         .         .
32200  65.I        .   O     .         .         .         .         .         .         .      S  .         .         .
32300  66.I        .  O      .         .         .         .         .         .         .      S  .         .         .
40000  67.I        .  O      .         .         .         .         .         .         .     S   .         .         .
40100  68.I        .  O      .         .         .         .         .         .         .     S   .         .         .
40200  69.I        . O       .         .         .         .         .         .         .    S    .         .         .
40300  70.I        . O       .         .         .         .         .         .         .    S    .         .         .
40400  71.I. . . . . O . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . .S. . . . . . . . . . . . .
40500  72.I        .O        .         .         .         .         .         .         .   S     .         .         .
40600  73.I        .O        .         .         .         .         .         .         .   S     .         .         .
40700  74.I        .O        .         .         .         .         .         .         .  S      .         .         .
40800  75.I        .O        .         .         .         .         .         .         .  S      .         .         .
40900  76.I        O         .         .         .         .         .         .         .  S      .         .         .
41000  77.I        O         .         .         .         .         .         .         . S       .         .         .
41100  78.I        O         .         .         .         .         .         .         . S       .         .         .
41200  79.I        O         .         .         .         .         .         .         . S       .         .         .
41300  80.I       O.         .         .         .         .         .         .         .S        .         .         .
41400  81.I. . . .O. . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . .S. . . . . . . . . . . . . . .
41500  82.I       O.         .         .         .         .         .         .         .S        .         .         .
41600  83.I       O.         .         .         .         .         .         .         .S        .         .         .
41700  84.I       O.         .         .         .         .         .         .         .S        .         .         .
41800  85.I      O .         .         .         .         .         .         .         S         .         .         .
41900  86.I      O .         .         .         .         .         .         .         S         .         .         .
42000  87.I      O .         .         .         .         .         .         .        S.         .         .         .
42100  88.I      O .         .         .         .         .         .         .        S.         .         .         .
42200  89.I      O .         .         .         .         .         .         .        S.         .         .         .
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42300  90.I      O .         .         .         .         .         .         .         .        S.         .         .         .
50000  91.I. . .O. . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . .S. . . . . . . . . . . . . . . .
50100  92.I     O  .         .         .         .         .         .         .         .       S .         .         .         .
50200  93.I     O  .         .         .         .         .         .         .         .       S .         .         .         .
50300  94.I     O  .         .         .         .         .         .         .         .       S .         .         .         .
50400  95.I     O  .         .         .         .         .         .         .         .       S .         .         .         .
50500  96.I     O  .         .         .         .         .         .         .         .      S  .         .         .         .
50600  97.I     O  .         .         .         .         .         .         .         .      S  .         .         .         .
50700  98.I    O   .         .         .         .         .         .         .         .      S  .         .         .         .
50800  99.I    O   .         .         .         .         .         .         .         .      S  .         .         .         .
50900 100.I    O   .         .         .         .         .         .         .         .      S  .         .         .         .
51000 101.I. . O . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . S . . . . . . . . . . . . . . . . .
51100 102.I    O   .         .         .         .         .         .         .         .     S   .         .         .         .
51200 103.I    O   .         .         .         .         .         .         .         .     S   .         .         .         .
51300 104.I    O   .         .         .         .         .         .         .         .     S   .         .         .         .
51400 105.I    O   .         .         .         .         .         .         .         .     S   .         .         .         .
51500 106.I    O   .         .         .         .         .         .         .         .     S   .         .         .         .
51600 107.I   O    .         .         .         .         .         .         .         .    S    .         .         .         .
51700 108.I   O    .         .         .         .         .         .         .         .    S    .         .         .         .
51800 109.I   O    .         .         .         .         .         .         .         .    S    .         .         .         .
51900 110.I   O    .         .         .         .         .         .         .         .    S    .         .         .         .
52000 111.I. .O. . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . .S. . . . . . . . . . . . . . . . . .
52100 112.I   O    .         .         .         .         .         .         .         .    S    .         .         .         .
52200 113.I   O    .         .         .         .         .         .         .         .    S    .         .         .         .
52300 114.I   O    .         .         .         .         .         .         .         .   S     .         .         .         .
60000 115.I   O    .         .         .         .         .         .         .         .   S     .         .         .         .
60100 116.I   O    .         .         .         .         .         .         .         .   S     .         .         .         .
60200 117.I   O    .         .         .         .         .         .         .         .   S     .         .         .         .
60300 118.I   O    .         .         .         .         .         .         .         .   S     .         .         .         .
60400 119.I   O    .         .         .         .         .         .         .         .   S     .         .         .         .
60500 120.I  O     .         .         .         .         .         .         .         .   S     .         .         .         .
60600 121.I--O-----.---------.---------.---------.---------.---------.---------.---------.---S-----.---------.---------.---------.
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RUNOFF SUMMARY, AVERAGE FLOW IN CUBIC METERS PER SECOND
AREA IN SQUARE KILOMETERS

	OPERATION	STATION	PEAK FLOW	TIME OF PEAK	AVERAGE FLOW FOR MAXIMUM PERIOD 6-HOUR	24-HOUR	72-HOUR	BASIN AREA	MAXIMUM STAGE	TIME OF MAX STAGE
	HYDROGRAPH AT									
+		BASIN	4344.65	34.00	3909.22	2167.22	903.75	381.10		
	ROUTED TO									
+		DAM	944.68	45.00	939.14	865.78	595.37	381.10		
+									84.09	45.00

*** NORMAL END OF HEC-1 ***

Appendix B
Geology

FEASIBILITY DESIGN FOR THE RÍO INDIO WATER SUPPLY PROJECT

APPENDIX B

GEOLOGY, GEOTECHNICAL AND SEISMOLOGICAL STUDIES

Prepared by

MWH

In association with

TAMS

AN EARTH TECH COMPANY

FEASIBILITY DESIGN FOR THE RÍO INDIO WATER SUPPLY PROJECT

APPENDIX B – GEOLOGY, GEOTECHNICAL AND SEISMOLOGICAL STUDIES

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FOREWORD

The studies described in this appendix have been performed in accordance with the scope of services for Contract CC3-5-536 - Work Order 003, Feasibility Design and Related Services for the Río Indio Water Supply Project entered into on September 30, 1999. This appendix presents the results of the investigations and studies related to services addressing **Task 5 - Geology**, and **Task 6 - Geotechnical and Seismological Studies**. The original scope of work anticipated that access to the site for drilling and other site investigations would be available. However, except for site reconnaissance, access has not been obtained.

Appendix B has been prepared using the following basic information:

- Reconnaissance Study: Identification, Definition, and Evaluation of Water Supply Projects, prepared by the U.S. Army Corps of Engineers, Mobile District, dated August 1999;
- Topographic mapping of areas of the proposed dam site prepared by Ingenieria Avanzada, S.A. under subcontract to MWH. Services were completed and submitted to the ACP under Contract CC-3-536, Task Order 2, Altimetric and Planimetric Surveys of 13 sites located on the Western Side of Lake Gatun;
- Additional topographic mapping of the dam site developed by digitizing 1:50,000 scale maps obtained from Instituto Geografico Nacional (Instituto "Tommy Guardia");
- Geological and geotechnical information obtained from two dam site exploration and mapping field programs, and a construction materials investigation program, including both test pit sampling and laboratory testing;
- Geological reconnaissance at selected water transfer intake and outlet portals and along the selected tunnel route; and
- The results of hydrology and meteorology studies presented in the companion Appendix A.

1 INTRODUCTION

1.1 Objectives and Scope of Study

This report presents a summary of the results and interpretations of geologic and geotechnical investigations and studies carried out for the proposed Río Indio Water Supply Project, Panama. The purpose of the work was to identify and evaluate geologic conditions of the proposed project and to develop feasibility level geotechnical design criteria and parameters for preparation of layouts and support of cost estimates.

The original scope of the work included substantial site investigations involving field geologic mapping, drilling, geophysical exploration, laboratory testing and technical evaluations. The results of these investigations were to be used to support the development of feasibility level geotechnical design criteria and cost parameters. Because access for site exploration could not be obtained, as explained in the next chapter, the present feasibility study is supported by data that is not as extensive as originally anticipated. On the basis of the limited fieldwork that was permitted, technical feasibility is demonstrated through description of site conditions and how these could be dealt with in the layout and design of project features.

Information from the geologic and geotechnical activities was channeled into concurrent planning studies as part of associated tasks, starting with site reconnaissance (Task 2), dam type selection, water transfer tunnel alignment selection, project optimization and preparation of final costing information for the preferred arrangements. The project features and structures are described in detail in the main volume of the feasibility report and are included herein only to the degree necessary for the geologic evaluations.

1.2 Location and Description of Project

The location of project and sites investigated is shown on Exhibit 1 of this Appendix. The project and zone of study is found on Sheets 4142-I and 4143-II of the 1:50,000 scale topographic map series.

The ACP identified the Río Indio Project as one of several potential projects to provide additional water supply, storage and hydropower generation to augment capacity and capability improvements for the Panama Canal. The project would impound water from the Río Indio drainage basin and divert it to the Panama Canal through an inter-basin transfer tunnel from the Río Indio basin to Lake Gatun. Hydropower would be generated at the damsite on the Río Indio and/or at the outlet of the inter-basin transfer tunnel on Lake Gatun.

The project area is located west of the Canal within the Río Indio watershed, approximately 25 km inland from the Caribbean Sea and 10 km west of Lake Gatun (Exhibit 1). The Río Indio dam site is located about 70 km west of Panama City and the Canal. The Río Indio is one the first north-flowing Atlantic drainages west of Lake Gatun. The dam site is about 1.2 km downstream from the community of Limon.

Structures for the project would include a main dam on the Río Indio (approximately 75 m high), a spillway, two saddle dams, inter-basin transfer tunnels, hydropower facilities, and other outlet works. The reservoir formed by the dam would have a normal operating level about 80 m above mean sea level with a surface area of approximately 4,440 hectares. Water from the reservoir will be diverted from the Río Indio basin to Lake Gatun through an inter-basin transfer tunnel about 8.4 km in length.

2 GEOLOGIC AND GEOTECHNICAL INVESTIGATIONS

This chapter summarizes the sequence and scope of geologic investigations relevant to the proposed Río Indio Water Supply Project.

2.1 Scope of Investigations

Geologic investigations have not been performed previously specifically for the Río Indio Water Supply Project. A scoping and reconnaissance study, which included the Río Indio project as well as other western watershed projects, was carried out by the US Army Corps of Engineers (USACE, Mobile District) in 1999 but this initial study did not include any geologic investigation. Regional geologic mapping for this part of the country consists of the 1:1,000,000 scale national map and limited coverage at 1:250,000 and 1:75,000 scales (Woodring, 1955). Although very informative for the immediate area of the Canal, these maps provide insufficient detail for project engineering purposes within the project area.

As part of the present scope of work, a reconnaissance visit was made in September 1999 that included identification and examination of bedrock types at various project locations. This was followed in January 2000 by a visit that involved dam site geologic mapping and construction materials studies. In August 2002, another field visit was made to carry out reconnaissance in the intake and outlet portal areas of the water transfer tunnel.

The original scope of the feasibility investigation program was quite extensive and included reconnaissance geologic mapping, core drilling at locations of principal project elements, geophysical surveys, test pit excavation, and laboratory testing. Because there were problems in obtaining access to the project site, the drilling program and associated activities (sampling, permeability testing) have been indefinitely postponed. The proposed seismic refraction program was also postponed.

The final program 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; and
- Development of geologic and geotechnical parameters for use in design of selected project and estimation of construction costs.

2.1.1 Geologic Mapping

The objectives of geologic mapping performed during these investigations included identifying, interpreting, and documenting the following aspects:

- Geomorphic conditions at the project sites,
- Occurrence and general nature of overburden units,
- Location and conditions of rock outcrops,
- Lithologic and surficial properties of rock units,
- Surficial extent and characteristics of rock weathering, and
- Orientation and condition of joints, shears, and faults.

Particular attention was paid to possible faults identified as photolinears in photogeologic studies and on regional geologic maps (Exhibit 2).

Available topographic maps for the study region were obtained from the Instituto Geografico Nacional. These included maps at 1:50:000 scale (contour interval 20 m). During the course of the work, new detailed topographic maps became available for selected areas at the dam site.

Reconnaissance geologic mapping was performed along the Río Indio from the reservoir area to immediately downstream of the dam site (Cerro Tres Hermanas). Geologic mapping was also carried out at selected locations to help identify conditions along prospective tunnel alignments, tunnel portals and intake locations, and possible powerhouse sites. Reconnaissance was also carried out at selected intake and outlet portal locations for the water transfer tunnel and along the selected tunnel route. A general reconnaissance of the proposed reservoir area was performed by helicopter to identify and evaluate any geologic features relevant to reservoir rim stability and watertightness.

2.1.2 Photogeology

Available aerial photographic coverage for the study region was limited in scope and quality. The only source from which aerial photographs are available in Panama is the Instituto Geográfico Nacional. A limited number of black-and-white photographs are available for the project area. The quality, age, and scale of the available aerial photographic coverage was a limiting factor in performing detailed examination of key areas and accurate studies for photogeologic interpretations. Conventional photogeologic methods were followed using a mirror stereoscope and photo-comparator.

2.1.3 Drilling and *In Situ* Testing

ACP contracted with Swissboring Overseas Corporation, Ltd, of Guatemala City, Guatemala, for drilling services but the work was indefinitely postponed before it even started. The scope of the drilling program was formulated by the USACE Pittsburgh District and would have included core drilling and *in situ* testing at the dam site, at saddle dam locations, and at proposed tunnel intake/outlet locations.

2.1.4 Geophysical Profiling

A program of seismic refraction profiling was developed by MWH to be performed in conjunction with and as an adjunct to the drilling program. This too has been postponed in view of difficulties in obtaining access to the sites.

2.1.5 Laboratory Testing and Analyses

Samples of rock and soil samples from test pits were collected for subsequent laboratory testing and analysis through the services of Tecnilab in Panama City.

- Construction Materials Testing. Laboratory tests for gradation, specific gravity, absorption, soundness, and abrasion resistance were performed on samples collected from test pits in order to establish their potential use as construction materials. Results are discussed later in this report, Section 10, Construction Materials.
- Petrographic Analyses. These were to have been performed on selected core samples. Since the drilling was cancelled, only preliminary petrologic determinations were made from hand samples collected during geologic mapping.
- Rock Mechanics and Index Testing. No rock testing was performed because of the cancellation of the drilling work.

Laboratory test results and test pit logs are presented in Attachment 1.

3 REGIONAL GEOLOGIC AND TECTONIC SETTING

3.1 General

This chapter presents the basic geologic and tectonic setting of the Panamá Canal western watershed water supply projects including the Río Indio project area. Information on this basic setting was derived from interpretations made during the course of these studies and from published geologic reports and maps cited in the text and in the bibliography.

Seismic design criteria developed as part of this study include determination of the hazard rating and associated return periods. Recommendations for design earthquake magnitudes and peak horizontal bedrock accelerations are also made. More detailed analyses may be desired in the future to either update or complement the recommendations provided. Recommendations made in this study are based on a review of existing information on the seismic hazard of the area and its application to the Panama Canal projects.

Due to the relative proximity of the water supply projects currently under investigation in the western watershed region (namely the Río Indio, Río Cocle del Norte, and the Caño Sucio sites), the study was expanded to cover an area encompassing all three sites. These projects lie in the area bounded by 80.1°W and 80.6°W and 8.6°N and 9°N.

3.2 General Regional Geology of Western Watershed Projects

The proposed Río Indio and Caño Sucio projects are located in an area underlain by Oligocene-aged sedimentary rocks of the three-membered Caimito Formation of Oligocene age (Woodring, 1982 a, 1982 b). A general stratigraphic column is presented as Table 1 and a regional geologic map is presented as Exhibit 2. The lower member of the Caimito Formation is composed of conglomerate, greywacke, and tuffaceous sandstone while the middle member consists of tuffaceous sandstone, greywacke, and lenticular foraminiferal limestone. The upper principal member consists of tuff, agglomeratic tuff, tuffaceous siltstone, and discontinuous sandy tuffaceous foraminiferal limestone. The deposits are primarily marine, but lithologically heterogeneous and the rocks of all members are hard, thinly to thickly bedded, and closely to moderately jointed.

The sedimentary units at the Río Indio and Caño Sucio sites comprise tuffaceous siltstones and sandstones, conglomerates and agglomerates. These are interbedded with lavas and in some parts of the reservoir area, the sedimentary rocks are stratigraphically overlain or are intruded by andesite and basalt flows, sills, and dikes. The volcanic units form many of the steep hills and high plateaus that are readily apparent on topographic maps and aerial photographs. Some of the volcanic formations might represent older units cropping out as erosional inliers. More recent volcanic sequences are found south of the project area.

Bedrock in region of the proposed Río Cocle 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, quartzmonzonites, gabrodiorites, diorites, or dacites. The published regional geologic map indicates bedrock in the site area to be of an intrusive igneous nature, possibly granodiorite or quartz monzonite (Tertiary age Petaquilla Formation). These rock types were not found during reconnaissance visits to the dam site and surrounding areas.

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).

The general pattern and distribution of major faulting in the region is depicted on Exhibit 3.

Table 1: Río Indio Area – Stratigraphic Column

Adapted from: Woodring and Thompson, 1949

ERA	PERIOD	EPOCH	AGE	FORMATION	DESCRIPTION
CENOZOIC ERA	QUATERNARY	($3x10^6$ to present)			Rio Indio alluvial deposits
				Chagres	Sandstone
	TERTIARY	Pliocene ($3x10^6$)		Toro	Limestone, lime-cemented coquina
		Miocene ($11x10^6$)	Late	Dacite, Andesite & Basalt flows and intrusions; Gatun	Massive sandstone, siltstone, conglomerate and tuff
			Middle	No deposition	
			Early	No deposition; La Boca	Predominantly silty or sandy tuffaceous mudstone with sandstone, limestone and conglomerate
		Oligocene ($26x10^6$)	Late	Caimito	Tuffaceous sandstone, siltstone and agglomeratic tuff
					Tuffaceous sandstone, tuff and thin limestone beds
					Conglomerate and tuffaceous sandstone
			Early	Las Cascadas; Bohio	Las Cascadas – agglomerate and intercalated lava flows Bohio - conglomerate, tuffaceous sandstone, tuffaceous siltstone with much pyroclastic material, largely non-marine
		Eocene ($38x10^6$)	Late	Gatuncillo	Mudstone, siltstone, impure bentonite and limestone lenses
			Middle		
			Early	No deposition	
		Paleocene ($48x10^6$)			
MESOZOIC		Cretaceous ($71x10^6$)		Pre-Tertiary basement complex	Indurated sedimentary rocks, intrusive and extrusive igneous rocks and metamorphic rocks

3.3 Regional Tectonics

The tectonics in the Central American region is predominantly governed by the interaction of the Nazca, Cocos, South American, and Caribbean Plates. Geologic processes in the Republic of Panama, including tectonics, sedimentation, volcanism, seismicity, and epeirogenesis, are all strongly influenced by the relative movements of these plates, Exhibit 4. Although the country is located on the southwest edge of the Caribbean Plate, Panama itself is located on a tectonic microplate called the Panama Block, which is a fairly rigid, yet seismically active segment of crust.

Plate movement in Central America is typically generalized as subduction zone tectonics. However, based on a review of the tectonics, the limit of the strongest influence of the subduction zone appears to cease near the border between Panama and Costa Rica and begins again on the eastern side of Panama and runs along the west coast of South America (Bodare, 2001).

3.3.1 Tectonic Boundaries

The Panama Block was formed over a period of 12 million years, largely as a result of the north to south spreading at the Galapagos Rift boundary between the Cocos and Nazca plates. Newly created crust at this boundary is being subducted beneath Costa Rica and regions further north. This action contributes to seismic activity extending from Costa Rica all the way to the western coast of Mexico. Four major tectonic regions define the boundaries of the Panama Block (Camacho et al., 1994):

- Panama Block-Caribbean Plate Boundary,
- Panama Block-Nazca Plate Boundary,
- Eastern Panama-Columbia Collision Zone, and
- Panama Block-Cocos Plate Boundary

Most historical seismicity within a 400-km-radius of the Panama Canal watershed can be attributed to collision and shear deformation at each of these neighboring plate boundaries (Cowan 1995). The junction of the Cocos, Nazca, and Caribbean Plates occurs near what is termed Punta Burica, or Burica Peninsula. The junction of the Cocos and Nazca Plates is termed the Panama Fracture Zone (Acres, 1981).

The north edge of the Cocos Plate is being subducted under the Caribbean Plate resulting in a reverse fault structure termed the Middle American Trench. The Nazca plate is being subducted obliquely in the northeast direction beneath the southwest margin of Panama creating the Southern Panama Deformed Belt, while the eastern portion of the Nazca plate is being subducted under South America (Cowan 1995). The thrust of the Caribbean Plate beneath the northern margin of the Panama Block has produced some large earthquakes in the past. The provinces and adjoining offshore regions of Bocas del Toro, Chiriqui, Los Santos in western and southern Panama, and San Blas and Darien in the east are also seismically active regions of Panama located along the margins of the Panama Block (Cowan 1995).

A detailed description of the significant tectonic features of Panama follows.

3.3.1.1 Panama Block-Caribbean Plate Zone

The North Panama Deformed Belt (NPDB) is a wide and shallow belt of folds and thrust faults that extends offshore from the Gulf of Uraba in eastern Panama to near the port of Limon in Costa Rica as seen in Exhibit 3 (Bodare, 2001). The NPDB is the most significant fault zone located in the Panama Block-Caribbean Plate Zone (Cowan 1995) and is one of the two largest seismic sources of engineering interest in the Panama Canal watershed. The other source is the interface between the subducted Caribbean plate beneath the Panama Block (Cowan 1995). Some discrepancy exists in identification of the NPDB as a subduction zone. Studies performed by Camacho and Viquez 1993 indicate that the overthrust boundary due to the convergence of the Caribbean Plate and the Panama Block is not a subduction. However, in his report, Joyner addresses the uncertainty in identifying the NPDB as a subduction zone or shallow crustal zone (Joyner, 1999). Subduction earthquakes give significantly larger ground motions for a specified magnitude and distance. Therefore, Joyner recommended that the NPDB, although not a typical subduction zone, should be considered a subduction zone so that ground motions selected to represent this zone are not too small.

3.3.1.2 Panama Block-Cocos Plate Boundary Zone

The Panama Fracture Zone (also known as the Longitudinal Fault Zone) and the Cocos Ridge are the largest faults in the Panama Block-Cocos Plate Boundary Zone (Cowan 1995). The Panama Fracture Zone as seen on Exhibit 3 separates the Cocos and Nazca plates south of the Costa Rica-Panama border (Cowan 1995).

There is evidence of rapid horizontal movement during the last 10,000 years with strong surface expressions that cut through the overriding Panama Block in the Medial Fault Zone and Canoas Fault Zones (Cowan 1995). The Panama Fracture Zone exhibits right lateral strike-slip motion at the rate of 50 to 70 mm/year and is subducted beneath the Panama Block near Cocos Ridge (Cowan 1995). The right-side north-south striking oceanic transform fault that makes the boundary between the Cocos and the Nazca plates extends between 82°W and 83°W and from the equator to 6°N splaying in a series of parallel northwest trending strike-slip faults (Bodare, 2001). The Cocos Ridge, a thickened area of oceanic crust created due to the subduction of the Cocos plate west of the Panama Fracture Zone beneath the southwest margin of the Caribbean plate and the Panama Block, appears to be moving nearly 90 mm/year (Cowan 1995). The Cocos Ridge can be defined by seismicity depth of nearly 50 km beneath southern Costa Rica (Cowan 1995).

3.3.1.3 Panama Block-Nazca Plate Boundary

The South Panama Deformed Belt (SPDB), along with the Azuero-Sona Fault Zone, and the Southern Panama Fault Zone, are the largest faults in the Panama Block-Nazca Plate Boundary (Cowan 1995). The SPDB is located at the southern border of Panama along the Pacific continental margin where the Nazca plate under-rides the Panama Block at a shallow oblique angle (Exhibit 4). This reverse fault structure may be the cause of volcanic activity in western Panama (Bodare, 2001). The SPDB is a zone of recent subduction of the oceanic plates to a depth of 100-120 km beneath Cordillera Central and represents a slice of the Cocos oceanic lithosphere between Coiba, Balboa, and the PFZ due to the migration of the Cocos-Nazca Boundary. East of the PFZ, the Nazca plate is moving E-NE oblique to the southern margin of Panama. Associated deformation is caused by thrust faulting along the continental margin and oblique normal faulting within the Panama Block west of the Azuero Peninsula and Coiba Island.

The Azuero-Sona peninsula in the southwestern portion of Panama contains a series of active parallel left-lateral strike-slip faults (Exhibit 3). Between 78°50'W and 80°30'W the fault bends landward and continues onshore with a northwest strike as the Azuero-Sona Fault Zone (Exhibit 3). The Azuero-Sona Fault Zone is a large strike-slip fault within the Panama Block that strikes NW-SE, defines a linear valley across the peninsulas and extends offshore to the southeast and merges with the Southern Panama Fault Zone (Camacho et al. 1994). The Southern Panama Fault Zone forms the left-lateral, strike-slip margin of the Nazca plate, offshore of the Azuero Peninsula and extends east across the Gulf of Panama (Exhibits 3 and 4).

3.3.1.4 Central Panama

Within Central Panama lies the Río Gatun Fault, the principal shallow crustal fault in the Canal Basin (Exhibit 3). This tectonic structure, also labeled the Gatun Fracture Zone, bisects Panama in a NNW-SSE direction and bounds the northwest margin of the Madden Basin and the slopes of Sierra Maestra (Schweig et al., 1999). The Río Gatun Fault is a shallow crustal fault with predominantly dip-slip displacement and is thought to be capable of producing moderate to large earthquakes (Cowan 1995). The fault length is estimated at 30 km with the northeast extent of the fault limited by the Río Boqueron valley and the western extent in Lake Gatun (Schweig et al., 1999).

Mainly intermediate depth earthquakes have been detected beneath Central Panama with a plate interface 35 km beneath Gatun, which deepens farther southeast to 60 km (Schweig et al., 1999). There is also the possibility of an active microseismic zone parallel to the Panama Canal (Camacho et al. 1994). The complexity of the Río Gatun Fault and related faults in the Sierra Maestra and the Madden Basin may not be continuous features but small segments, which collectively can produce significant events (Schweig et al., 1999). There is a high density of shallow faults north of Gatun Dam about 250 meters apart and up to 13 km long with varying orientations. In addition to the Río Gatun fault, a system of N-S trending predominantly east-facing normal faults are located west and south of the west end of the Río Gatun fault with no evidence of recent movement (Cowan 1995). Research has determined that all faults in this region are younger than 8.3 million years old and many are more than 6,000 years old based on the results of field studies (Schweig et al., 1999).

Field investigations in the area of the Río Gatun fault also indicate that alluvial deposits that overlie the Río Gatun fault are highly weathered to a depth of at least 5 meters; similar alluvial deposits in Western Panama indicate that these deposits are dated at 10,000 years. Therefore, the rate of deformation has slowed dramatically (Schweig et al., 1999).

3.3.1.5 Eastern Panama – Colombia Collision Zone

Another tectonic region identified on the east coast of Panama along the border with Colombia is the Eastern Panama Deformed Belt, also termed the Eastern Panama-Colombia Collision zone. This structure is located in the eastern portion of Panama and extends to northwestern Colombia between 79°W and the Atrato River Basin. Faults located in this region are shown on Exhibit 3 and include:

- Chucunaque and Atrato Faults (onshore and offshore normal faults)
- Ungia and Pirre Faults (thrust faults along the Panama-Colombia border)
- Utria Fault (extends from Colombia into Panama along the Pacific Coast)
- Uramita Fault (west-dipping left-lateral strike-slip fault at tectonic boundary between Panama and the North Andes blocks)
- Gulf of Panama and Pearl Islands contain nine NW striking thrust faults that deform the seafloor in the eastern Gulf of Panama
- Sanson Hills Fault Zone (includes active left-lateral strike-slip faults)
- Pirre Hills Fault Zone (includes reverse or thrust faults that bound the northeast side of the Serrania de Pirre and is defined topographically with the most recent movement in 1974)
- Sambu Fault Zone (contains NW striking left-lateral strike-slip faults that bound the southeast margin of the Sambu Basin)
- Jaque River Fault Zone (contains NW striking left-lateral strike-slip that bound the SW margin of the Serrania de Sapo)

- Colombian Trench (zone of westward and eastward verging folded thrust slices with near surface deformation associated with the collision of the Nazca plate subducting beneath South America)

4 SEISMICITY

4.1 Distribution of Earthquakes

4.1.1 Historical Earthquakes

As indicated on Exhibit 5, 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 NPDB, while two earthquakes in 1621 and 1882 occurred nearly 25 km off the northern coast near Colon. An additional earthquake event is noted in 1914 on the northeastern coast in the San Blas region.

4.1.2 Instrumentally Recorded Earthquakes

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. The general distribution of these earthquakes plotted as function of their depth below the surface is presented in Exhibit 5.

A discussion of the overall seismicity of the area in each of the identified tectonic zones follows.

4.1.3 Panama Block-Caribbean Plate Zone

The North Panama Deformed Belt (NPDB) results from the compression of the Caribbean plate and the Panama Block and is thus the source of several large earthquakes (Schweig et al., 1999). The eastern portion of the NPDB is characterized with thrust-type mechanisms, some strike-slip components, and some normal faulting. This area is the source zone for the largest earthquake on the Caribbean coast of Central America, an 1882 event with M_s = 7.7 (Bodare, 2001). The northwest portion of the NPDB along western Panama has experienced fairly low seismicity recently with some large earthquake events in the past (Bodare, 2001). The eastern portion of the NPDB exhibits a more regular pattern of thrust deformation than the western portion with variations in sediment thicknesses and crustal structure (Camacho and Viquez 1993). In the western portion, the NPDB reflects a lateral transmission of stress from between the Cocos Ridge collision zone to the area of overthrusting of the Panama Block to the Caribbean Plate. Other earthquakes occurring at greater depths beneath northeast Panama are attributed to the weak southwest dipping zone beneath the Caribbean coast with the Caribbean plate underthrusting the Panama Block (Camacho et al., 1994).

Nearly all of the seismicity seems to occur below the upper plate at crustal depths; thus the area can be described loosely as a dipping subduction zone. The deformed belt can be explained by movement of blocks with dip-slip, mainly thrust-type movements in the Caribbean plate (Camacho and Viquez 1993). The convergence rate between the Caribbean Plate and the Panama Block is nearly 15 mm/year with the Caribbean Plate underthrusting the Panama block (Bodare, 2001). The central portion of the North Panama Deformed Belt between 80° and 81.5°W, near the area of the focus of this report, has no evidence of major seismic activity with hardly any instrumentally recorded seismicity originating from this location. (Camacho and Viquez 1993).

4.1.4 Panama Block-Cocos Plate

This area has experienced high seismic activity including earthquake events with magnitudes greater than 7.0 (Bodare, 2001). The epicenter of the 1934 $M_S7.5$ event in the Gulf of Chiriqui was the largest Panamanian event recorded instrumentally (Camacho et al., 1994). The Panama Fracture Zone is marked by a number of earthquakes west of 83° W (Acres, 1981). Based on evaluations performed by USGS, the highest seismic hazard was found along the west coast of Central America coincident with the subduction of the Cocos Plate under the Caribbean (Shedlock, 2001).

4.1.5 Panama Block – Nazca Boundary

This region has historically experienced several moderate earthquakes between M4.0 and M6.0 (Camacho et al., 1994).

4.1.6 Central Panama

Studies performed at Gatun Dam to develop the seismic hazard of the area included the deployment of a seismic network as well as a search for evidence of prehistoric liquefaction, or paleoliquefaction (Schweig et al., 1999). Over a six-month period, nearly 64 events were recorded with usable data and captured nearly 75% of the seismic events above magnitude 2.5 within 50 km of the network. It was noted that a lack of events were recorded within the network region, while a number of events were recorded at farther distances. Most of the events occurred at depths well below that typical of shallow crustal seismicity and were most likely associated with under-thrusting of the Caribbean Plate beneath Panama. Numerous faulting mechanisms were also identified and a mixture of faulting types for the coastal earthquakes were deep and associated with subduction (Schweig et al., 1999).

4.1.7 Eastern Panama-Colombia Collision Zone

As seen on Exhibit 4, the Eastern Panama-Colombia Collision Zone is a complex zone with diffuse seismicity. Within this region, the Sambu Fault Zone and Jaque River Fault Zone are inferred to be active, while the Colombian Trench is the source of several large historical earthquakes (Cowan 1995).

4.2 Focal Mechanisms

The Panama Canal watershed region is characterized by several types of faulting, normal fault, thrust or reverse fault and right-lateral/left-lateral strike slip faulting. This, combined with the observation (and relative lack of information to contradict) that the subduction characteristics prevalent in the rest of Central America are not apparent in the Panama Basin area contributes to the complexity of the region. Studies have been performed which indicate that although recorded events have included shallow crustal earthquakes, many intermediate and deeper events have also been recorded thus classifying the area as not purely a subduction zone.

4.3 Seismic Source Zonation

Seismic source zonation involves the following:

- Identification of individual source zones
- Definition of a maximum magnitude for each zone, and
- Estimation of the rate of seismic activity for each zone

Once these have been determined, the appropriate attenuation relationships for the area can be developed as discussed in subsequent sections.

4.3.1 Identification and Distribution of Seismic Source Zones

Superposition of the seismicity over the seismotectonic provinces discussed earlier permits delineation of seismic source zones of equal activity. Exhibit 6 graphically identifies some of these areas. The highest concentration of seismic activity appears to occur along the west coast of Panama at its boundary with Costa Rica, along the Panama Fracture Zone, as well as on the east coast at Panama's border with Colombia. The North and South Panama Deformed Belts are also areas of high seismic activity. As seen on Exhibit 5, the central portion of Panama near Panama City is a zone of moderate concentration of seismic activity. The Azuero-Sona region can be delineated as an additional seismic source zone. Based on an evaluation of this information, the following seismic source zones are significant source zones:

- North Panama Deformed Belt
- Río Gatun Fault Zone
- Azuero-Sona Fault Zone
- Panama Fracture Zone

4.4 Maximum Magnitudes

Selection of the maximum magnitude, also termed maximum or controlling earthquake, for the area in question depends on the definition of the maximum earthquake for the specific application. Several types of maximum earthquakes exist as follows (Reiter, 1990):

- *Maximum Possible Earthquake*: Defines the upper bound earthquake event related to a specific earthquake source no matter how improbable the event, and is used in probabilistic analyses.
- *Maximum Credible Earthquake (MCE):* Defines the upper bound earthquake event related to a specific earthquake source that is reasonably expected to occur. This is commonly used in deterministic analyses.
- *Maximum historic earthquake:* Defines the lower bound of the maximum credible earthquake but does not define an earthquake size that will always be exceeded in the future.

Available and accessible historical records have been researched by others and were reviewed to determine maximum historical earthquakes for Panama. History has shown that the largest earthquakes in the Panama Block have occurred at or near the Block boundaries. A review of the research performed and summarized by Camacho et al., Acres Int'l, and Schweig et al. indicates maximum historical magnitudes for each of the main seismotectonic structures presented above as follows:

Structure	Maximum Magnitude, M_w
North Panama Deformed Belt	$M_w = 7.0$ to 7.2 [1](Acres, 1981)
	$M_w = 7.7$ (Schweig, 1999)
Río Gatun	M_w=6.8 (Schweig, 1999)
Azuero-Sona	M_w=7.5 (Schweig, 1999)
Panama Fracture Zone	$M_w > 6.8$ to 7.2 [1](Acres, 1981)
Central Panama	$M_w > 5.0$ [2] (Acres, 1981)
Eastern Panama Deformed Belt	$M_w = 6.8$ to 7.0 [1] (Acres, 1981)
Western Panama Block	$M_w = 5.0$ to 5.7 [2] (Camacho et al., 1994)
Eastern Panama Block	$M_w = 6.3$ [2] (Camacho et al., 1994)

Notes:
(1) Originally recorded as M_s and converted to M_w using $M_w= 2.251+0.655M_s$.
(2) Originally recorded as Modified Mercalli Index (MMI) and converted to M_L using $M_L= ⅔ (I) +1$. M_L converted to M_w using Reiter 1990.
(3) M_w = moment magnitude, M_s = surface wave magnitude, M_L = Richter (local) magnitude, m_b = body wave magnitude

Using this information, the maximum earthquakes selected for this study ranged from M_w=6.8 to 7.7 for the different zones. Maximum magnitudes reported for Central Panama and the Western and Eastern Panama Blocks may be categorized as background seismicity.

5 SEISMIC HAZARD EVALUATION

Determination of the seismic design criteria can be based on deterministic methods that involve evaluation of the affect of ground motions from individual seismic sources attenuated to sites under investigation. Alternatively, seismic design criteria can be based on probabilistic methods involving selection of the allowable risk and associated return period selected for the area. Such methodology typically involves:

1. Selection of the allowable risk, or frequency of exceedance, and associated period of return for the projects.
2. Use of seismic hazard curves and magnitude recurrence relationships to determine the largest magnitude and peak ground acceleration (PGA) based on the associated period of return.

Development of the magnitude-recurrence relationship and the appropriate attenuation relationships are discussed below.

For the water supply projects, seismic design criteria were developed from probabilistic approaches described in the technical literature that were carried out for other projects and civil works in the region. The conclusions drawn from review of such previous work were then compared to results obtained from deterministic methods.

5.1 Earthquake Recurrence Relations

The fundamental relationship characterizing the distribution of earthquakes as a function of magnitude in each source zone described above is the Gutenberg-Richter relationship:

$$\log N = a - bM$$

Earthquake events from the Global Hypocenter Database were evaluated for this study to determine the appropriate magnitude-recurrence relationship. Exhibit 7 presents a plot of the cumulative number of earthquake events of a certain magnitude per year. The apparent drop in number of events recorded below M5.5 may be due to the fact that seismic events below M5.5 are typically difficult to record and therefore records of these events are limited. A regression analysis was used for the events greater than M 5.5 to develop the *a* and *b* parameters in the Gutenberg-Richter relationship:

$$\log N_c = 7.90 - 1.27M \text{, where M is } m_b$$

The following conversions to standardize magnitude scales for comparison were required to effectively compare magnitudes reported by each of the references (Camacho et al., 1994):

$$M_w = 2.251(\pm 0.19) + 0.655(\pm 0.04) \times M_s$$

$$m_b = 2.64 + 0.50M_s$$

Using these relationships, the magnitude-recurrence relationship can be converted for the M_s and M_w magnitude scales to yield the following equations:

$$\log N_c = 4.63 - 0.63M_s$$

$$\log N_c = 6.79 - 0.96M_w$$

Exhibit 7 presents a preliminary magnitude-recurrence curve developed for this study using the above relationship.

5.2 Ground Motion Attenuation

Attenuation relations are probabilistic descriptions of the level of ground shaking as a function of the earthquake and site parameters including earthquake magnitude (moment magnitude is preferred), type of faulting, site-to-source distance, and local site conditions. Differences in tectonic regimes result in different tectonic relationships: shallow crustal earthquakes in active regions, shallow crustal earthquakes in stable regions, and subduction zone earthquakes. Attenuation relationships for Panama typically recognize subduction zone earthquakes and shallow crustal earthquakes in tectonically active regions.

The most preferable attenuation relationship for use in seismic risk is derived with local data. Although very few seismicity studies have been performed for Panama due to the lack of sufficient strong motion accelerograph data and isoseismal maps, attenuation relationships have been developed by others to describe the seismicity of Panama.

Attenuation relationships developed to describe the seismicity of Panama typically incorporate records from the Guerrero, Mexico earthquake with available Central American data to strengthen the magnitude-distance distribution of the data at larger magnitudes. The differences in the two data sets have been explored and based on comparative plots for soil and rock observations at various magnitudes, no significant differences were observed (Dahle et al., 1995).

Exhibit 8 presents the attenuation curves as prepared by Climent et al. for the Central American sites only and for that including the Mexico earthquake data. Also, the differences between shallow earthquakes and subduction zone earthquakes in the database were explored by Dahle and Climent. Their results indicate that no clear difference exists between the two databases and therefore, their further analyses eliminated the distinction between shallow crustal and subduction zone events.

Exhibit 9 presents the Central American data plotted in comparison with curves developed by Boore et al. (Climent et al, 1994). The relationship developed by Boore et al. depicts the attenuation of events in the Western U. S.; the Western U. S. relations typically appear slightly lower than those for Central America.

Exhibit 10 presents the Central American attenuation curves in comparison with that developed for Japan by Fukushima and Tanaka; such a comparison is made due to the similarity in the geology of Central America and Japan (Bodare, 2001).

The Central American attenuation relationship for rock outcrops as presented in Exhibits 8 through 11 is represented by:

$$\ln a_{\max}(m/s^2) = -1.687 + 0.553M - 0.537\ln R - 0.0032R$$

where R is the hypocentral distance in km (Bodare, 2001).

Exhibit 11 presents a composite comparison of various peak ground acceleration attenuation curves. In addition to the Central American attenuation curve developed by Dahle et al., the plot presents attenuation curves for subduction zone earthquakes as developed by Kawashima et al., Fukushima and Tanaka, Youngs et al., and Joyner, and for shallow crustal earthquakes by Campbell and Joyner and Boore (Cowan). For this study, this Central American attenuation relationship is used with knowledge that recommendations will be fairly conservative.

5.3 Seismic Risk

Ground motions for analysis are selected based on the assumed risk of the project. Ground motions selected corresponding to a return period of 10,000 years are typically recommended for analysis of high-hazard structures under the maximum design earthquake event (USCOLD, 1999). Return periods between 1,000 years and 10,000 years are typically used for dams where a return period of 10,000 years is approximately equal to a 1 to 2 percent chance of exceedance during a 100-year project life, while a 1,000 year return period is approximately equal to a ten percent chance of exceedance during a 100-year project life.

Design earthquakes representing the assumed risk of the projects must be identified so that seismic design parameters can be developed for each of the design earthquakes identified. Terminology typically used in analyses to define the design earthquakes is presented in USCOLD, 1999. The following paragraphs summarize pertinent information from this document.

5.3.1 Maximum Earthquakes Considered

Maximum Credible Earthquake (MCE). The maximum credible earthquake (MCE) is defined as the "largest reasonable conceivable earthquake that appears possible along a recognized fault or within a geographically defined tectonic province, under the presently known or presumed tectonic framework." In the process of selecting design earthquakes and ground motions, several different

MCE's may be selected based on their generating source, fault mechanisms, and distance from the site. One may be selected as being the controlling event, typically the most severe.

Maximum Design Earthquake (MDE). The maximum design earthquake (MDE) is defined as that which will "produce the maximum level of ground motion for which the dam should be designed or analyzed." Following an MDE event, the project should perform without catastrophic failure, with allowance for severe damage and/or economic loss. Uncontrolled reservoir releases are characterized as a catastrophic failure and would be unacceptable. The MDE can be less than the MCE but, in many cases is taken to be equal to the MCE, in particular for high-hazard, reservoir-retaining structures.

Operating Basis Earthquake (OBE). The operating basis earthquake (OBE) is defined as "the level of ground motion at the dam site with a 50 percent probability of being exceeded in 100 years." Following an OBE event, the structure should function with little to no damage and without interruption towards the true functionality of the project.

5.3.2 Design Earthquakes for the Projects

An evaluation of the project seismicity as well as the economic and life-safety issues associated with the western watershed projects indicates that these projects can most likely be classified as significant rather than high hazard projects. No fault movement, or ground breakage due to tectonic offset, has been recorded in the area over the last 10,000 years and the project region has no potential for the development of seiches or earthquake-triggered tsunamis.

Because the projects will be newly designed and constructed, the most up-to-date seismic design guidelines will be used and seismic resistant design features adopted where needed. The projects will not be constructed of or founded on liquefiable or potentially liquefiable materials and the projects will not be constructed on any known active or potentially active faults.

Based on the density plot of earthquakes (Exhibit 5), it is apparent that the greater percentage of earthquakes occurs on the borders of the Panama Block, away from the location of the projects. Although the occurrence of a large event affecting the project area is possible, it is more likely to affect the plate boundaries.

On the basis of the above, therefore, it is recommended to analyze the projects with a return period near 2,000 years, *i.e.*, a five percent probability of exceedance over a project life of 100 years. In this respect, it is suggested that a level of motion less than the controlling MCE can be acceptable to represent the MDE for these projects when using probabilistic methods.

The recommended OBE for the projects shall be as recommended by USCOLD at 50 percent probability of exceedance over a project life of 100 years, or a return period of 144 years.

5.3.3 Seismic Hazard Curves

The seismic hazard curve showing the peak ground accelerations (PGA) versus the annual probability of exceedance as developed by others for the Central American attenuation is presented in Exhibit 12 (Bodare, 2001).

In addition, Camacho et al. developed hazard curves for soil conditions using the Central American attenuation for seventeen different localities in Panama. The catalog used for the development of the seismic hazard curves by Camacho et al. was complete from 1910 to 1992 for magnitudes above M_w 5.8 and for magnitudes above M_w 4.7 from 1963 to 1992. The seismic source zones were defined at two depth levels, shallow sources less than 50 km deep and deep sources between 51 and 100 km deep and sources with depths greater than 100 km were assumed to have minor impact on Panama were not included. Of the hazard curves developed by Camacho et al., the curves for the following cities within the pre-defined boundary of the projects were evaluated for this study and are presented in Exhibits 13 through 21:

Aguadulce, Cocle	Chitre, Herrera
Chorrera, Panama	Coronado, Panama
Panama City, Panama	Penonome, Cocle
Santiago, Veraguas	Sona, Veraguas

5.4 Results

5.4.1 MDE Event – Probabilistic Basis

The recommended frequency of exceedance (or return period) for the MDE event for these projects is five percent over a 100-year project life, corresponding to a period of return of approximately 2,000 years. Based on the seismic hazard curve for Central America shown in Exhibit 12, the PGA due to this earthquake is 0.42g. Such a value is thought to be disproportionately large, possibly because this peak ground acceleration is based on relationships developed from data collected outside Panama and mostly from subduction zone events, which are not characteristic of the project region.

It is noted that the seismic parameters resulting from the referenced studies do not seem to be in keeping with apparent distribution or incidence of earthquakes in the project region as shown on Exhibit 5. Additional study should be made of this seeming paradox but is outside the current scope of work. It is recommended that any future investigations for the water supply projects should include a site-specific seismic hazard analysis with an attenuation model that is more in keeping with the region.

5.4.2 MDE Event – Deterministic Basis

The closest active and most important seismic source zone to the proposed water supply projects is the North Panama Deformed Belt. The maximum magnitude assigned to this source is between $M_w = 7.0$ and 7.7

$M_w = 7.0$ to 7.2 (Acres, 1981)

$M_w = 7.7$ (Schweig, 1999)

With a deterministic approach and applying various Central American attenuation models described above, the resulting peak bedrock acceleration at the various water supply projects is as follows:

Name of Project	Closest Distance to N. Panama Deformed Belt (km)	PGA[1] (g)	PGA[2] (g)	PGA[3] (g)
Río Coclé del Norte	75	**0.27**	0.16	0.13
Caño Sucio	90	**0.21**	0.11	0.12
Río Indio	95	**0.21**	0.11	0.11
Río Chagres	100	**0.20**	0.11	0.10

Notes: (1) based on Joyner (1999) subduction zone earthquakes. (2) based on Dahle et al. (1995), subduction and shallow crustal earthquakes in Central America and Mexico. Both curves for rock sites. (3) based on Camacho et al. (1994).

The peak bedrock acceleration values based on Joyner (1999), presented in the first column of the table above, are recommended for use in preliminary design. The values based on the attenuation models of Dahle et al. (1995) and Camacho et al. are considered rather low. In contrast, those calculated based on Joyner (1999) are considered to be appropriate and more in keeping with the regional tectonic framework and known rates of seismicity compared with those developed by probabilistic methods described earlier.

5.4.3 OBE Event

The recommended frequency of exceedance (or return period) for the OBE event for the projects is 50% over 100 years, corresponding to a period of return of approximately 144 years. For the western watershed projects, this hazard is determined from curves developed by Camacho for specific cities within Panama. These were developed for a return period of 1,000 years but can be used to determine the average PGA for the OBE event. The following table presents the maximum peak ground acceleration on soil sites for each of the cities for a return period of 144 years, as defined by the OBE. Equivalent bedrock accelerations are obtained by using a factor of 0.76 (Camacho et al., 1994).

City	Return Period	PGA[1] (m/s^2)	PGA[1] (g)	PGA[2] (g)
Aguadulce, Cocle	144	1.75	0.18	0.14
Chitre, Herrera	144	1.85	0.19	0.14
Chorrera, Panama	144	1.75	0.18	0.14
Coronado, Panama	144	1.65	0.17	0.13
Panama City, Panama	144	1.85	0.19	0.14
Penonome, Cocle	144	1.65	0.17	0.13
Santiago, Veraguas	144	2.00	0.20	0.15
Sona, Veraguas	144	2.15	0.22	0.17
Average				**0.14**

Notes:

(1) Peak ground acceleration reported for soil sites.

(2) Peak ground acceleration converted to peak bedrock acceleration by multiplying factor of 0.76

The seismic hazard curves shown in Exhibits 12 through 21 were used to determine that the PGA (in bedrock) for the OBE event is 0.14g. Further investigation, as recommended above, might result in lower ground accelerations.

5.5 Recommended Seismic Design Parameters

The recommended seismic design parameters for the Río Indio Project are as follows:

Maximum Design Earthquake (MDE) = 0.21 g
Operating Basis Earthquake (OBE) = 0.14 g

6 GEOLOGY OF RÍO INDIO DAM SITE AREA

This chapter provides descriptions of the geology of the Río Indio dam site and reservoir areas and of the region through which the water transfer tunnel would pass. These descriptions and interpretations are based on the results of studies and investigations described above.

6.1 Topographic Conditions and Physiography

At the damsite, the Río Indio flows northwest forming an asymmetrically shaped valley that exhibits nearly 100 m of relief. The relatively steep slopes on the right abutment are formed by Cerro Tres Hermanas, while the slopes on the left abutment have a more gradual slope. Both abutments are almost entirely covered with colluvial and residual soils, and are moderately heavily vegetated. Although a few small, scattered rock outcrops can be observed, most of the project area is characterized by a moderate to deep weathered profile and thick soil cover typical of the sub-tropical climate.

The valley floor at the damsite is approximately 200 m wide and is filled with alluvial terrace deposits consisting primarily of silts and clays. The river, which is about 20-50 m wide at the site, has eroded steep banks up to 5 m high. Terraces are seen on both sides about 3 to 4 m above the present river elevation. There is evidence of flood debris/trash in trees up to at least 5 m above the river. Bedrock is exposed in parts of the riverbed and along the cut banks of the lower right abutment. The valley broadens upstream of the dam site and large areas have been cleared for pasture and local arable cultivation.

The valley is somewhat asymmetrical at the dam site with the right abutment noticeably steeper than the left side. The slopes on the right side rise up rapidly to over 100 m. On the left side of the valley, the slopes are gentler and dissected by small drainages. At least one saddle dam will be required to contain the reservoir at the proposed maximum flood elevation of 82.5 m.

The area under which the water transfer tunnel would pass is characterized by a rolling topography and pronounced dendritic drainage with several small streams. Rock outcrops are rare and difficult to locate. A few isolated hills rise above the others, presumably formed by more resistant rock than surrounding areas. The region has mixed forest and cleared pastureland with small-holdings.

6.2 Unconsolidated Deposits

Overburden at the dam site mostly consists deposits of talus, colluvium, and residual soils above about elevation 15m on the valley slopes and alluvial deposits found in the riverbed or as terraces on the valley sides.

6.2.1 Quaternary Alluvium

Alluvial deposits of the Río Indio valley are not extensive compared to many rivers and drainage basin of comparable size elsewhere in Central America. Sand and gravel deposits in the riverbed are thin and only localized in occurrence and found as lag deposits on the inside of meanders and as occasional bar deposits. The gravel-sized fractions originate from nearby outcrops of tuffaceous sandstone, andesite, basalt, and a few fragments of slabby siltstone.

The most significant sources of alluvial deposits located in the vicinity of the dam site form 4- to 5-m-high terraces along the banks of the Río Indio approximately 2- to 3-km upstream of the dam site. These, however, are also localized in occurrence and consist predominantly of clayey silt (MH) with some layers of clayey sand (SC) and inorganic clay (CH). No deposits of clean alluvial sands and gravels have yet been observed.

6.2.2 Colluvium and Residual Soils

Most of the bedrock in the project area is mantled by well-developed horizons of colluvial and residual soils. In the dam site area, these materials were found to consist of clayey silt (MH) to a depth of between one and two meters on top of weathered tuffaceous sandstone and siltstone bedrock. During reconnaissance, it was noted that on some steeper slopes, the colluvium tends to contain rock float (fragments of weathered bedrock) in a soil matrix. Nowhere is the content of rock fragments sufficient for the deposit to be called talus. Most of the overburden in the project site area has a moderate to high clay content due to the tuffaceous nature and volcanic origin of the bedrock.

6.3 Rock Units

Bedrock at the dam site and along the headrace tunnel route consists almost entirely of Tertiary sedimentary and volcanic rocks. Based on observation made during geologic reconnaissance mapping, the sedimentary formations are found to be comprised of tuffaceous siltstones and sandstones, conglomerates and agglomerates thought to belong to the Caimito Formation or its age equivalent (Table 1: Regional Stratigraphic Column).

It is suspected these units are interbedded with lavas in the vicinity of the larger (western) saddle dam. In some parts of the reservoir area, and in the area of the village of Limon, the sedimentary rocks are stratigraphically overlain by andesite and basalt flows. The volcanic units form many of the steep hills and high plateaus that are readily apparent on topographic maps and aerial photographs. Some of the volcanic formations might represent older units cropping out as erosional inliers. More recent volcanic sequences are found south of the project area.

6.3.1 Sandstone and Siltstone

Tuffaceous sandstones and siltstones form the uppermost bedrock unit at the dam site and are widespread throughout the project area. Most of the bedrock is covered by overburden, however in some areas shallow landslides/slumps have removed the overburden to expose underlying *in situ* bedrock. Residual soils developed from the weathering of the sandstones and siltstones at the dam site tend to be clay-silts, as described in previous paragraphs.

Outcrops of sandstone approximately 3 to 4 km south-east of the proposed dam site, and thought to be stratigraphically equivalent to units at the dam site, were investigated as a potential source of quarry construction material. Laboratory testing conducted on these tuffaceous sandstones and siltstones indicates that the material is of sufficient strength to be used as rockfill, however its durability is such that it most likely would not be suitable for use as concrete aggregate or for select processed fills (i.e. filters, drains, riprap).

6.3.2 Andesite and Basalt

Andesite and basalt rock units form many of the steep hills and high plateaus in the project area. During field reconnaissance, several potential quarry sources were identified, as described later in this report in the section on construction materials. No such materials are known to occur at the dam site. Rock samples collected from the potential quarry sites ENE of the dam site were tested in the laboratory. Results indicate that the material would be suitable for use as rockfill, processed select fills, and concrete aggregate.

6.4 Rock Properties for Preliminary Design

Table 3 lists typical ranges for compressive strengths and unit weights for the different rock types that may be encountered at the dam site and in construction of the water transfer tunnel. In the absence of data from subsurface investigations performed specifically for this project, these data are from published bibliographic sources and experience with similar rock types.

6.5 Structural Geology

6.5.1 General

The principal geologic structures at the Río Indio dam site are joints and bedding. Until subsurface investigations are performed, the existence and extent of other features, such as shear zones and faults, are unknown.

6.5.2 Faults and Shear Zones

Based upon experience with geological investigations and construction in the Canal area, it is likely that several small faults and shear zones could exist at the dam site. Such structures can locally influence the pattern and degree of weathering in the rock mass, and probably have exerted a minor control over the morphological development of the site, and either singly or in combination significantly affect the strength and deformation modulus of the rock in local areas.

From regional geologic mapping and photogeologic studies, the presence of major faults is not expected at the dam site. Some photogeologic linears have been interpreted parallel to the river valley at the site and trending northwest but these are not thought to be caused by significant faulting in this direction, rather are more likely related to fold structures in the sedimentary rock cover.

6.5.3 Rock Joints and Bedding

Based on limited observations at rock outcrops in the dam site area, bedding in the tuffaceous sandstone is found to more or less horizontal to slightly inclined. The strata are intersected by a well-developed pattern of systematic, near-vertical joints. At this time, few details have been gathered on the characteristics and properties of such jointing but two distinct sets are thought to present, one trending northwest and the other northeast. Currently, there are insufficient data to develop typical stereographic plots. Joints of the major sets are probably mostly planar. Joint fillings or coatings are probably common up to the limits of weathering. Clay infillings should be expected in the upper part of the weathering profile.

6.6 Hydrogeology

Groundwater data and hydrogeologic properties of the geologic units at the dam site are not known. It is suspected that groundwater levels are at relatively shallow depths in the abutments. During reconnaissance, a small spring was observed in the thalweg close to the western saddle dam, indicating shallow groundwater conditions.

7 ENGINEERING GEOLOGY OF RÍO INDIO DAM SITE

This chapter presents engineering geologic evaluations of various proposed project elements at the Río Indio dam site. This is preceded by a brief evaluation of factors influencing selection of the site and interim dam axis and those aspects influencing dam type selection.

The proposed project will consist of a concrete faced rockfill dam with an ungated chute spillway located on the right abutment. Diversion during construction will be achieved by a diversion tunnel in the right bank and cofferdams.

7.1 Factors Influencing Dam Type Selection and Preliminary Layouts

Generally, the geologic and geotechnical factors that most influence selection of dam type fall into the following categories:

- General foundation bedrock acceptability;
- Sliding resistance and deformation characteristics of foundation;
- Required excavation depths to achieve acceptable foundation materials;
- Measures required to treat the foundation to improve physical properties and control leakage;
- Long-term performance of the foundation under normal operation conditions and extreme events, especially earthquake; and
- Availability of suitable construction materials.

Such geological and geotechnical factors can have direct influence on the development of comparative construction costs and were taken into consideration during the study of dam type alternatives. However, in the absence of subsurface investigation data, the process was based on qualitative evaluations involving engineering judgment and previous experience in similar geological environments including comparisons with other projects in Panama, such as the Esti Project currently under construction.

7.1.1 Dam Type Selection

Following layout comparisons based on hydrologic, geologic/geotechnical, and cost considerations, the final recommended arrangement involves a concrete faced rock fill dam (CFRD) with an ungated spillway located on the right abutment (Appendix D-1, Dam Type Selection). Geologic and geotechnical studies were concurrent with or completed slightly ahead of the dam type selection study. The initial focus of the Appendix B studies was on the dam site and dam type selection, as well as input to formulation of the subsurface investigation program.

7.2 Geotechnical Design Parameters

Geotechnical design parameters and criteria used for developing preliminary layouts and cost estimates for dam type selection are presented in Table 2 and are described in the following paragraphs. In the absence of additional field data these were used for the remainder of the feasibility design.

Table 2: Summary of Geotechnical Design Parameters for Layouts

Parameter	Selected Design Criteria	
Thickness of overburden (top of weathered rock)	4 m	
Depth to top of competent rock	8 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
	Temporary	1.5H : 1V, 3-m-wide benches every 10 m vertically. Bench at soil-rock contact

A specific seismic design criterion was not used in the dam type selection stage since it was assumed that whatever dam type was selected would be designed to include seismic design features appropriate for the site. It was already understood that the project is located in a region known to be subject to earthquake activity. The selected dam type for Río Indio (CFRD) was analyzed for deformation of the rock fill due to ground motions associated with the Maximum Design Earthquake (MDE) as described earlier in this Appendix. The analytical methods used and results are presented in Appendix D of this report.

7.3 Foundation Bedrock Characteristics

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. This is in contrast to some sedimentary rock units and geotechnical conditions known from the immediate area of the Canal (e.g. Cucaracha Formation), where sliding and foundation failures have been common and presented serious problems.

- *Bearing Capacity.* The tuffaceous siltstones and sandstones are relatively soft rocks but are expected to present adequate bearing capacity to support any of the structures being considered. However, their moderately low modulus of deformation may result in settlement and/or displacements between adjacent structures of different size and weight. Data from subsurface investigation and testing will be needed to develop design and construction details to deal with such behavior.

- *Resistance to Sliding.* The sandstone should provide adequate resistance to sliding along bedding planes or other planes of weakness provided excavation depths are sufficient to achieve fresh sound bedrock.

- *Material Strength Parameters.* The estimated range of material strength parameters recommended for use in preliminary stability checks are provided in Table 3. These are based on published data for the respective lithologies and experience with similar materials elsewhere.

Table 3: Estimated Material Strength Parameters

Rock Type	UCS (MPa)	ϕ (deg)	c (kg/cm^2)	Unit Weight (kN/m^3)
Fresh sandstone	35-120	45°	5	22
Weathered sandstone (and along bedding)	20-70	30°	1	21
Basalt (quarry)	150-250	50°	10	26

7.4 Excavation Depths

Based upon observations made at the site and comparison with rock types elsewhere in similar environments, an average overburden thickness of 4 m was assumed, i.e. depth to top of weathered rock. An average depth to the top of competent rock was assumed to be about 8 m. These values were used in the development of layouts and in the computation of quantity takeoffs for cost estimates. Actual depths and characteristics of weathering need to be investigated by drilling and geophysical exploration since these can have significant impact on the cost estimates.

Excavation for the dam and spillway was estimated as indicated in Table 4.

Table 4: Estimated Excavation Depths

Feature	Estimated Excavation Depth (m)
Plinth and 25 m downstream of grout curtain	8
Under main dam body	4
Spillway headworks and chute	8

Excavation of test pits could not penetrate deep into the residual soils and saprolites covering bedrock at the site, but a fully developed weathering profile several meters thick is expected. Particular concern and attention during any future subsurface exploration program should be given to investigate the nature of the two differently sloping abutments. Compressibility of the soils and saprolites left under the downstream part of a rock fill embankment would impact dam behavior and the effect of the narrow ridge on the left abutment should be evaluated if it is left under a rockfill dam.

Because of the absence of subsurface data, a detailed foundation excavation plan has not been developed. Instead, a general concept has been reached in which proposed excavation depths are equal to or deeper than the estimated minimum depths indicated on Table 4. It should be understood that, in places, deeper excavation might be required where localized geologic conditions require. Also, in some places additional excavation may be required just for foundation shaping.

The foundation excavation depth requirements for the dam plinth and spillway are similar. In both cases the excavation should go down to uncompressible, sound bedrock that can be readily grouted. It should be sufficiently unweathered such that rock strengths are not seriously impaired and joints should be tight, or, if open, should readily accept grout and not be filled with any soft or erodible materials. The rock mass should not be compromised by any pervasive or major through-going discontinuities that could affect sliding stability or uplift of the structure.

The deformation modulus of the foundation should not differ radically from one location to another such that unacceptable stresses may result in the structure itself.

7.5 Foundation Improvement, Treatment, and Long-Term Performance

No special foundation improvement or treatment measures are expected for the Río Indio site that would influence selection of one dam type over another. Similarly, the sandstone bedrock is expected to perform satisfactorily over the lifetime of the project without adverse deterioration. During subsequent investigations, the potential for internal erosion of the sandstone under high seepage pressures and flows should be studied to determine appropriate design and construction details.

Although small landslip and slumps are evident at many locations in the reservoir and even at the dam site, no large mass movements are expected to affect the reservoir, but the effect of saturation, say after intense rainfall, on the stability of residual soils and saprolites needs to be properly evaluated.

7.5.1 Foundation Treatment

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

Surface Treatment. For the plinth slab excavation at the dam toe and under the spillway headworks, 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. However, contingency quantities for backfill concrete should reflect the potential for surprises as found on some other CFRD projects. For example, it is understood that during construction at the Esti Hydroelectric Project in Panama considerable over-excavation and additional treatment were required along parts of the plinth slab. Geologic conditions at this project are similar to what is interpreted for the Río Indio area with sedimentary and volcanic rock types.

Shallow Foundation Grouting (Consolidation). Consolidation grouting is not envisaged except in limited areas (e.g. fault or fracture zones) should they become exposed during excavation for the plinth slab and under the spillway headworks. 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. Curtain grouting will be used to reduce seepage through joints and fractures under the dam and in the abutments. Because of the lack of subsurface investigation data it is difficult to estimate depths and extent of grouting. For initial estimating purposes, a single row, staged grout curtain constructed by the split-spacing method is assumed. Final design might require grout holes to be inclined to intercept the maximum number of open joints, fractures, and faults. The initial spacing of primary grout holes is taken to be 10 m and it is assumed that procedures will entail split-spacing down to 2.5 m (tertiary holes) over the entire curtain, and to 1.25 m (quaternary holes) over 75% of the curtain. In some more permeable locations, such as shear zones or faulted areas, grout holes might have to be staggered upstream and downstream.

Grouting will be performed from the toe slab of the CFRD and through the spillway concrete (or from a grout slab prior to placing first stage spillway concrete). Grout takes should be low to moderate through most of the curtain. The average grout consumption was assumed for estimating purposes to be about 30 kg/m.

Foundation Drainage. Foundation drainage will be provided for the spillway 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. Holes would be appropriately inclined in order to maximize the number of joint/fracture interceptions.

7.6 Construction Materials

Proposed sources and uses of available materials for construction, included required excavation, are discussed later in this report, Section 10, Construction Materials.

7.7 Plunge Pool Basin

Future assessments should examine rock erosion and stability conditions in areas where spillway discharge will impact. Currently, it is expected that spillway discharge will not result in unfavorable progressive erosion or instability of individual blocks.

7.8 Diversion and Cofferdams

7.8.1 General

River diversion during construction will be accomplished by a diversion tunnel and cofferdams. The optimum size and location of these structures will be established during later phases of design with the benefit of subsurface investigations.

For preliminary design and estimating purposes, a tunnel located on the right side has been assumed with a length of about 650 m and finished diameter of 5.0 m. The river elevation of the upstream portal will be about 7m and at the downstream portal, about 6m. The tunnel will be horse-shoe (or modified horseshoe) in shape and will be concrete-lined.

The upstream cofferdam will have a crest elevation of 26 m and will be about 20 m high. The cofferdams are proposed as random fill structures constructed with materials excavated at the dam site. In the weathered state, these would be relatively impermeable upon compaction.

7.8.2 Diversion Tunnel Conditions

The diversion tunnel for construction of a CFRD dam will be excavated entirely in tuffaceous sandstone. This bedrock should be of moderate strength and unweathered over most of the tunnel length except at the tunnel portals. Sandstone should provide favorable tunneling conditions using conventional drill and blast methods. Poor conditions may be encountered locally in the tunnel associated with deeply weathered areas, fracture zones, minor faults, or shear zones. Groundwater is contained almost entirely in open joints. Although local inflows may occur from individual open planes, these flows should be minor and temporary and will not pose any problems during construction. The tunnel, as currently aligned provides adequate cover below the assumed limits of the dam foundation excavation.

It is assumed that rock support required in the construction of the diversion tunnel would consist of steel sets, pattern bolting, and shotcrete near the portals, and pattern bolts, with or without shotcrete elsewhere. Four tunnel support categories have been identified as indicated on the following table.

Table 5: Diversion Tunnel Rock Support Categories

Tunnel Support Category	Rock Condition	Rock Support
I	Good	0-5 cm shotcrete; spot bolts as required
II	Good to Fair	0-5 cm shotcrete with wiremesh; pattern bolts, 5 per section
III	Poor	5 cm shotcrete with wiremesh; pattern bolts, 7 per section
IV	Poor to Worst	5-10 cm shotcrete; steel sets (or lattice girders); bolts as required

For construction cost estimating purposes, the distribution of these rock support categories has been conservatively estimated as indicated on Table 6 below.

Table 6: Estimated Rock Conditions in Construction Diversion Tunnel

Type of Rock	I	II	III	IV
Estimated % of Type encountered	10	25	40	25

It is pointed out that the percentages indicated in Table 6 reflect estimated tunneling conditions, i.e. weathering state plus water plus joint condition. Although, severe or pronounced weathering conditions are not expected over long distances, closely spaced joints (associated with valley stress relief) could impact this diversion tunnel over relatively longer distances than a tunnel that is entirely at right angles to the river valley.

7.8.3 Diversion Cofferdams

Geologic data currently available for the diversion cofferdam areas are limited to surface mapping and the foundation areas are underlain by an unknown thickness of overburden. Construction of the upstream cofferdam may present some problems. The structure will be founded only partly on bedrock; the majority will be on channel fill and terrace deposits. Cut-off will involve excavation through varied overburden materials of unknown thickness (possibly 3-5 m to top of rock).

7.9 Summary and Conclusions on Dam Site Geology

The Río Indio dam site has been well selected from a topographic viewpoint and appears to be the most suitable site in the area.

In regard to geological aspects, there do not appear to be any strongly adverse conditions or fatal flaws at the site that would seriously hinder or prevent development or make it too costly to construct. Site conditions are interpreted to be generally favorable and can be readily handled through conventional design and construction practices, without having to resort to special or unprecedented methods. Subsurface investigations have yet to be done to confirm technical feasibility, but at this stage it is thought that geologic conditions do not impose any significant development impediment.

Factors that appear favorable include:

- No deep alluvium is present at the site that would require deep seepage cutoff;
- Foundation bedrock at the dam site is the same on both sides of the river with no indication of major faulting in the river channel;
- Bedrock consists of medium to fine-grained sandstone that would serve as a suitable foundation for the types of dam and appurtenant structures being considered;
- Based on outcrop data, a competent foundation for the dam can be obtained at reasonable depths;
- Given appropriate engineering design and construction methods, the foundation could be readily treated to control seepage, and would not pose hazards with respect to stability or liquefaction;
- A construction scheme can be developed that will permit access to the foundations in the dry, allowing inspection and appropriate treatment;
- Suitable materials for aggregates, rockfill, and other construction requirements are available.

8 ENGINEERING GEOLOGY OF WATER TRANSFER TUNNEL

8.1 Tunnel Arrangements, Alternatives, and Alignment

Several alternative tunnel alignments were investigated for the Río Indio Project culminating in the selection of a recommended alignment to be examined in the Feasibility Study. Documentation of a tunnel alignment selection study is presented in Appendix D, Part 4. The following includes description of the criteria used for laying out the various alternatives and the basis for selection of a preferred arrangement for transferring water from the Río Indio basin to Lake Gatun.

The alternative tunnel alignments that were studied are indicated on Exhibit 23. The recommended alignment is one designated IP-1/OP-1 with a total length of about 8,400 m.

Typically, the principal concern during preparation of initial layouts is to avoid or minimize areas where geologic conditions would be suspected to be unfavorable for the construction of long tunnels. Such areas would include zones of faulted rock, exceptionally weak rock, or where the tunnel alignment could parallel fracture zones. Ideally, it would be most favorable to maintain the tunnel excavation in uniform rock type and conditions.

Factors affecting the alignment and construction cost of the water transfer tunnel include:

- Length of tunnel
- Effective tunnel diameter
- Ground cover
- Geology and rock support requirements
- Excavation method
- Tunnel lining

All other features and considerations of the Indio-Gatun water transfer tunnel being equal, the length and diameter of the tunnel can have the most direct impact on construction schedule and construction costs. Geologic factors influencing design and construction are described below.

8.2 Principal Lithologies along Tunnel Alignments

As explained earlier in this report (Section 3, Regional Geologic Setting), existing geologic maps of the region show bedrock in the region as belonging to 'undifferentiated Tertiary volcanics' or alternatively as belonging to the Tertiary age Caimito Formation (tuffaceous sandstone, tuffaceous siltstone, tuffs, dacitic agglomerate, conglomerate, sandstone, and limestone). There is debate as to whether the rock sequence found in this region is the same as the Caimito Formation best known in the immediate area of the Canal. Based on discussions with ACP geologists in the field, it is conjectured that the sandstone and tuffaceous sequence observed in the Río Indio area could in fact be younger than the Caimito known from elsewhere and overlay older, possibly pre-Tertiary volcanics.

Geologic investigations at the Río Indio dam site indicate that bedrock units in the Río Indio project area consist of Tertiary sedimentary and volcanic rocks. The sedimentary formations comprise tuffaceous siltstones and sandstones, conglomerates and agglomerates. These are interbedded with lavas and in some parts of the reservoir area, the sedimentary rocks are stratigraphically overlain by andesite and basalt flows. The volcanic units form many of the steep hills and high plateaus that are readily apparent on topographic maps and aerial photographs, while the sedimentary units tend to occur in the lower ground. Some of the volcanic formations might represent older units cropping out as erosional inliers.

During a reconnaissance visit in 1999, a potential location for the tunnel outlet works was visited close to Isla Pablon on Lake Gatun. Outcrops of a very hard and strong andesite and basalt were found in the vicinity. These materials are quarried locally and would provide useful sources of construction material for the project. Sedimentary rocks, possibly tuffaceous and foraminiferal sandstone belonging to the marine phase of the Caimito Formation, were found nearby.

In August 2002, further geologic reconnaissance was carried out at the proposed intake and outlet portal locations for the selected water transfer tunnel route. It was confirmed that the Gatun outlet works could be founded on sound igneous bedrock, however, design details, such as the extent of tailrace channel excavation and the extent of tunnel steel lining would depend on final arrangements with respect to local topography. At the intake end, reconnaissance revealed that the topography in the portal area is favorable and provides a range of options for detailed design, i.e. flexibility in vertical and horizontal location. The bedrock geology consists of thick-bedded sandstone units, such as found at the dam site, which crop out or are mantled with a thin layer of cobbly/bouldery colluvium. Based on the presence of basalt float in the area, an igneous unit also occurs in the area, probably a local sill or dike, but its exact location with respect to the portal is not known.

It is probable that tunnel construction for the inter-basin transfer will encounter a wide range of rock types and tunneling conditions. The range and relative persistence of various conditions will depend on final alignment selection. Rock types could include sandstone and softer epiclastics of the Caimito Formation as well as hard, strong lavas (andesites, dacites, and basalts) and agglomerates. There is potential for transition over short distances from very hard strong rock (such as andesite or basalt) to soft, weak almost clay-like materials. Differing ground conditions can be expected for any of the tunnel alignments considered in this study. Such aspects would need to be examined in more detail to establish their impact on construction method as well as cost parameters, including support and lining requirements.

8.3 Geologic Structures Influencing Alignment Selection

The principal geologic structures that can influence the siting, design, and construction of the tunnel system include faults, shear zones, joints, bedding, and contacts between different rock types. The nature and distribution of these geologic structures is little known in the region but assumptions used in this study are described below.

8.3.1 Major Regional Structures Affecting Tunnel Alignments

There are no known major features that would seriously impact tunnel alignment selection. As interpreted from aerial photographs and topographic maps, the general trend of the principal structures, including faults and principal lithologic contacts, is northwest-southeast (Exhibit 2). Local northeast-southwest lineaments are also observed. In this regard, the overall alignment of the alternative tunnel alignments is favorable with respect to the strike of features that will be intercepted by underground construction. Most of the significant discontinuities and contacts will be intersected more or less at favorable angles that will reduce the amount of rock support required. The alignment of some smaller sections of the tunnel system may be less favorable with respect to the orientation of the major geologic features.

Review of the regional geology with respect to tunnel alignments has also taken into consideration the depth of cover, depth of overburden and weathered bedrock under stream crossings, potential for faults or shear zones under stream crossings, and stability of hillsides and slopes over or adjacent to the tunnel alignment.

8.3.2 Local Geologic Structures Affecting Tunnels

The stability of the underground openings (tunnels and shafts) will depend on the orientation, frequency, and characteristics of rock discontinuities, rock stresses, and on groundwater conditions encountered during construction. At this stage of study, it is not possible to predict the location or accurately estimate the extent of local geologic conditions that will impact tunnel stability. Anticipated tunneling conditions are described below based on interpretation of available geologic information, review of other tunnel projects, and experience from elsewhere.

Geologic factors affecting the design and construction costs of the Indio-Gatun water transfer tunnel involve two basic areas:

- Anticipated tunneling (excavation) conditions along selected alignments, including potential for water inflow during construction, potential for hazardous gases and hydrothermal water, and potential for squeezing ground, slabbing rock and rock bursts, and
- Anticipated support requirements

Generally, it is desirable to locate tunnels in as geologically competent material as possible to minimize lengths requiring heavy tunnel support, especially near to the portals. Since there is little information on geologic conditions along the various alignments and at any of the tunnel portal locations, evaluation relied on the estimated geologic conditions interpreted from topographic and photogeologic features. Preference was given to portals located where relatively steep topographic rises indicate the potential to encounter comparatively competent bedrock within a short distance.

Approximate estimates were made of anticipated tunneling conditions and support requirements along potential alignments under consideration. Geologic criteria and their impacts upon the development and design of various tunnel features are discussed below.

8.4 Cover Criteria

The tunnel cover criterion (the minimum vertical distance between the ground surface and the crown of the tunnel excavation) used in developing various potential alignments was:

$$H = 2D + 10 \text{ meters}$$

Where H equals the distance between the ground surface and the crown of the excavated tunnel and D equals the tunnel diameter. The requirement for two effective tunnel diameters between the top of ground and the crown of the tunnel is based on previous experience with the construction of tunnels of this type in similar environments. Ten meters was added to this value to account for topographic uncertainty (including the 20-m-contour interval of the topography available for this study).

In addition to influencing factors relating to excavation and support, ground cover becomes an important consideration for selection of tunnel lining requirements.

8.5 Tunnel Diameter

For the initial tunnel alignment studies an effective finished tunnel diameter of 5 meters was used, based on hydraulic criteria. The final finished tunnel diameter of the selected tunnel arrangement is 4.5 meters. Depending on rock conditions, the excavated diameter will range from 5.0 to 5.5 meters.

8.6 Excavation Methods

Alternative methods of excavation can be considered for the water transfer tunnel, involving either conventional drill-and-blast or mechanical excavation by tunnel boring machine (TBM), or both. These methods would entail different excavation and finished tunnel profiles, as well as differing construction schedules and associated risks and costs. Selection of a preferred method would involve consideration of various factors, including geologic conditions, construction schedule, risk, and cost.

For drill-and-blast methods, the excavated shape for the tunnel will be of a horseshoe, modified horseshoe, or inverted U (D-shaped). The finished tunnel profile would be in shape of a horseshoe or possibly circular, depending on internal design pressures. For mechanical excavation by TBM, the excavated shape for the tunnel will be circular. The finished tunnel profile would also be circular.

For comparative costing purposes, it was assumed that tunnel construction would utilize drill-and-blast techniques from multiple headings. A typical tunnel cross-section is shown in Exhibit 22.

Intermediate access locations have yet to be established but at this time, it is assumed that two construction shafts (each about 25 m deep) would be used at third points along the alignment. Shafts are considered better options than construction adits because of the relatively flat topography that would require long access adits (each at least 250-300 m long).

8.7 Estimated Excavation Conditions

Conditions at Contacts and in other Geologic Structures (Faults, Shear Zones, etc.). Most of the principal rock formations are expected to be in normal contact with one another. In some places lithologic contacts could be faulted and in such areas there will be zones of highly fractured rock, breccia, gouge, and probable water inflow. These conditions will pose special problems for excavation and support. The width of contact zones and faults will vary and range up to one tunnel diameter (<1 m to > 5 m).

Most lithologic contacts and faults are interpreted to be more or less vertical, or steeply inclined, and therefore the tunnels should be affected over only relatively short distances. Some contacts and faults, however, could be gently inclined or subparallel to the alignments. Any conditions or characteristics associated with them would be experienced over longer distances. These would have the most severity where mixed ground or lithologies of strongly contrasting character could be encountered.

Potential for Water Inflow. Experience indicates that groundwater inflow should be expected at various points along the proposed tunnel alignment. However, the location and quantity of water inflow are not known and cannot be predicted with any certainty. It should be assumed that inflow potential will be greatest in zones of faulted or highly fractured rock, in zones of deep weathering, and in low cover zones at stream crossings (which are probably characterized anyway by faulted, fractured, or weathered bedrock). Water inflow potential should also be assumed at contacts between formations, especially at contacts with igneous units. Water encountered in the tunnel excavations should not be at high very pressures, only up to the head equivalent of the overlying ground cover.

Potential for Hazardous Gases and Hydrothermal Water. The potential for encountering hazardous gases is considered remote. Ventilation and shotcrete have been used successfully to control gas occurrences, should they occur.

Potential for Squeezing Ground, Slabbing Rock, Rock Bursts. The tunnels are not likely to encounter stress-related problems (popping rock, slabbing rock, or rock burst in competent rock, squeezing ground in weak/fractured rock) because the rock cover is not that great.

8.8 Anticipated Tunneling Conditions and Rock Support Classification

Rock support requirements estimated to be encountered in the tunnels during construction are listed in Tables 7 and 8. Geology and rock support requirements have a direct impact on the tunnel alignment, and therefore directly impact construction costs and the selection of a preferred tunnel alignment. Rock support requirements are determined based upon rock conditions encountered along the tunnel alignment, which are typically divided into four different types. The four rock conditions types used for this study are listed in Table 7 below.

Table 7: Estimated Rock Support Classes for Water Transfer Tunnel

Rock Type	Description
Type I	**Excellent** - Best Rock Conditions; typically minimal overbreak and generally self-supporting or requiring minimal support with shotcrete and spot rock bolting of localized rock wedges, full face excavation with normal advance.
Type II	**Good to Fair** - Good to Fair Rock Conditions; moderate overbreak, generally requiring systematic support with shotcrete and rock bolts to within 10-20 m of face. Full-face excavation with normal advance. Prompt, systematic rock bolting with shotcrete (fiber-reinforced shotcrete) to within 2-5 m of the face required to control loosening of rock.
Type III	**Fair to Poor** - Fair to Poor Rock Conditions; weathered or weak rock, fractured rock, contacts between different rock units, closely jointed rock. Possible local inflows of groundwater. Full-face excavation, with slower, shorter advance and larger amounts of overbreak. Requires prompt support with shotcrete after excavation, with systematic pattern rock bolting.
Type IV	**Very Poor** – Worst Conditions; fault zones and shear zones containing crushed, altered rock, or at shallow depths, highly weathered, disaggregated rock, potentially squeezing or running/ flowing ground. High inflows of groundwater possible. Requires shortened excavation rounds, possibly top heading and bench in worst areas. Prompt support to within 1-2 m of the face with steel ribs (or lattice girders) with steel lagging and rock backpacking or shotcrete (fiber-reinforced, or with welded wire mesh). Grouting may be necessary to reduce water inflows.

Tunnel lengths associated with the rock support classes, I - IV, listed in Table 7, were estimated for the different alignments considered. These were based on the general knowledge of the geology of the area, geologic mapping, and judgment to account for 1) potential widths and degree of rock fracturing in faults, 2) depth of cover and potential for development of weathered ground, 3) effects of tectonic shears commonly associated with folding of rocks, and 4) other factors, such as presence of water or proximity to a major stream crossing. Also taken into consideration was the extent to which a tunnel alignment could encounter mixed face conditions or rock types of markedly different character (strength, fracture type, etc.) alternating over short distances (Type III condition).

For the selected tunnel alignment (IP-1 – OP-1), the proportional distribution of the different rock support classes are estimated as shown in Table 8 and Exhibit 24. On Table 8 is also shown a suggested comparison, based on experience, between rock support classification and conventional rock mass classifications, Q-system and RMR.

Table 8: Estimated Rock Conditions for Selected Water Transfer Tunnel Route

Rock Support Class	Symbol	Q Values	RMR Values	Estimated Percent of Total Tunnel Length
Excellent	I	> 7.0	> 60	25%
Good to Fair	II	1.0 - 7.0	40 - 60	40%
Fair to Poor	III	0.4 - 1.0	35 - 40	30%
Very Poor	IV	< 0.4	< 35	5%

8.9 Tunnel Lining

For this study, it is assumed that the tunnel will be fully lined from portal to portal. The lining will be required mostly for hydraulic reasons, but will also be to control water loss in low cover zones and areas of severely fractured rock and to prevent erosion and deterioration of the rock in areas of soft or highly fractured rock.

A cast-in-place concrete lining has been assumed in all rock conditions (I-IV) described above. Minimum lining thickness will be 25 cm but will be thicker, 50 cm, in areas of low cover (inadequate confinement) or poor ground.

Nominal reinforcement (for temperature/shrinkage crack control) will be provided for lining in Types III and IV rock. In areas where the rock cover is low and there is inadequate confinement (estimated to be where the hydraulic grade line exceeds 40 m of total cover), more heavily reinforced concrete linings will be designed to accept internal tunnel pressures with some interaction with the surrounding rock.

Reinforced concrete lining and a steel-lined section are included in the tunnel at the downstream end approaching Lake Gatun where internal pressures would be greatest. Currently it is assumed that steel lining is necessary because the rock cover over and around the pressure tunnels along the present alignment does not provide adequate confinement for the tunnels. If later studies determine that geologic conditions are favorable or find alternative alignments that provide greater cover and better rock conditions, the lengths of steel lining may be shortened.

Final design of the tunnel lining will require information on geologic and groundwater conditions obtained from appropriate investigations. The steel-lined sections for the tunnel should be designed to withstand, when empty, an external hydrostatic pressure equal to the groundwater table in the surrounding rock mass. External pressures acting on the concrete - lined sections should not be a concern since the cover depth is not large and groundwater pressures would rapidly dissipate through cracks in the concrete. Final design optimizations might consider benefits offered by alternative tunnel shapes (horseshoe or circular) with respect to savings in concrete volumes and structural stability of the lining.

The design of steel lining for internal pressures, as well as reinforced concrete-lined sections, should consider some interaction and participation with the rock mass surrounding the lining. The extent of such interaction is a function of the quality and properties of the surrounding rock mass and the ability to perform and effectiveness of consolidation grouting.

Concrete lining, consisting of cast-in-place concrete or fiber-reinforced shotcrete, would be required in fault zones and in areas where the rock mass is severely fractured or deeply weathered. The potential for substituting steel-fiber reinforced shotcrete lining for cast-in-place concrete should be investigated in a later phase. For cost estimating purposes, it was assumed in this study that fiber-reinforced shotcrete would constitute the primary method of support. It might be possible to optimize its use as permanent lining.

Since proportionally more tunnel concrete lining quantities would be required in a drill-and-blast excavation than with TBM, this is another topic that should be examined in more detail during the next phase of study.

8.10 Tunnel Muck

It is estimated that about 200,000 m^3 of excavated material (tunnel muck) will be removed from the underground works. Initially, some of the material (such as the muck from excavation of the construction adits) might be used immediately or for other construction purposes (e.g. road bedding, fills, etc.) though the majority will probably be hauled to disposal areas.

8.11 Excavation Advance Rate

The anticipated geologic and tunneling conditions strongly influenced the estimate of excavation advance rate and of course construction cost.

The daily advance rate estimates per excavation face for drill-and-blast excavation sections of the headrace tunnel range from about 1 m/day in the Type IV ground to about 5 m/day in the best ground. The average is just under 4 m/day, which is considered realistic. Limiting factors on the production rates will probably not be geologic but rather other aspects such as resource availability and intermediate access.

An estimate of the advance rate that could be achieved by a TBM-mined alternative has not been made. As indicated earlier, a TBM excavation option is not being considered at this time. Design of a TBM system for the Río Indio project would probably be able to address most, if not all the rock mass parameters that influence TBM excavation advance rate. Nevertheless, there can remain important risk elements associated with certain geologic conditions, such as potential for water inflow or squeezing conditions, though for this project squeezing ground is considered to be only a remote possibility because of relatively low cover.

9 GEOLOGY OF POWERHOUSE AREA

Geologic investigation in the proposed powerhouse area at the outlet of the water transfer tunnel (i.e. Lake Gatun end) was limited to reconnaissance visits only. Based on observations made during this reconnaissance, the powerhouse will be entirely located on bedrock consisting of relatively strong, sound igneous rock units that are probably andesitic in composition.

The backslope for the powerhouse should be benched and cut to a suitable slope to maintain stability. The exposed surface of the excavation should be rock-bolted and covered with chain-link to control loosening and falling of small blocks. Areas on permanent slopes that are more weathered or closely fractured would require, in addition to the chain-link, a layer of shotcrete provided with drain relief holes. A complete subsurface and surface drainage system would be required to intercept and control surface runoff, and prevent erosion damage.

A location for a powerhouse facility at immediately downstream of the dam was not specifically investigated in the field as its size and arrangement were not known at the time of the investigations. However, no unusual or exceptional problems are envisaged with respect to geology or geotechnical conditions.

10 CONSTRUCTION MATERIALS

The types of required construction materials for the project are:

- Materials for cofferdams,
- Concrete aggregates,
- Filters and drains,
- Rock fill for the dam, backfill materials and other structural fills, and
- Rock for riprap and slope protection.

10.1 Sources

Four primary construction material sources were identified based on field reconnaissance, aerial photograph analysis and study of topographic maps. Each of the four material types are discussed below; their locations are indicated on Exhibit 25.

10.1.1 Alluvial Deposits

Several alluvial deposits were identified on aerial photographs and topographic maps as potential sources of construction materials. The most significant sources of alluvial deposits located in the vicinity of the dam site form 4- to 5-m-high terraces along the banks of the Río Indio approximately 2- to 3-km upstream of the dam site. The alluvial terraces were investigated through excavation of test pits and were found to be composed predominantly of clayey silt with some layers of clayey sand and inorganic clay. These materials were tested in the laboratory and found to be suitable for use as impervious fill for cofferdams and secondary fills (results attached to this Appendix). Their use in the core of an earth (clay) core rockfill dam (ECRD) is not considered feasible because of their heterogeneity, limited available volume, and location.

No deposits of clean alluvial sands and gravels were encountered in the test pits nor were observed elsewhere in the field. Alluvial sands and gravels that are used locally as aggregate are dredged from the river channel or mined from small gravel bars along the river. These deposits are relatively small and limited in extent, and therefore are not considered a suitable or economic material resource for the construction of the project.

10.1.2 Colluvial and Residual Soils

Most of the bedrock in the project area is covered by well-developed horizons of colluvial and residual soils. A test pit was excavated in the slope of the left abutment to investigate the depth and type of overburden. Materials in the pit were found to consist of about 1-2 m of clayey silt on top of weathered tuffaceous sandstone and siltstone bedrock. It is presumed that most of the overburden in the project area is clay-rich due to the tuffaceous nature and volcanic origin of the bedrock. This material was tested in the laboratory and was found to be suitable for use as impervious fill. However, its exploitation and use as core material in ECRD construction is not considered feasible because of its expected high moisture content, and the difficulty of conditioning the material. Available quantities are limited and production rates would not be economic.

10.1.3 Sandstone and Siltstone

Tuffaceous sandstones and siltstones form the uppermost bedrock unit and are widespread throughout the project area. Most of the bedrock is covered by overburden, however in many areas shallow landslides have removed the overburden to expose underlying in situ bedrock. Sandstone from required excavation, provided it is not entirely decomposed, could be used as rockfill material for a rockfill dam. It is expected, however, that handling, placement, and compaction of the relatively weak sandstone would result in production of fines. This can be handled through appropriate design of the zoning of the dam and construction specifications for material handling. Laboratory testing conducted on tuffaceous sandstones and siltstones indicates that the material is of sufficient strength to be used as rockfill, however its durability is such that it most likely would not be suitable for use as concrete aggregate or for select processed fills (i.e. filters, drains, riprap).

A supplementary sandstone quarry could be opened closer to the site than the proposed basalt quarry described below. A potential sandstone quarry source was reconnoitered approximately 3 to 4 km southeast of the proposed dam site, but other even closer locations might exist.

10.1.4 Andesite and Basalt

Andesite and basalt rock units form many of the steep hills and high plateaus in the project area. Several potential quarry sources were identified:

- Cerro del Barrero, located about 4 km SSE of the dam site; and
- Cerro La Jota, Cerro del Duende, and Palmira, located between 4 and 9 km ENE of the dam site.

Rock samples collected from the potential quarry sites ENE of the dam site were tested in the laboratory. Results indicate that the material would be suitable for use as rockfill, processed select fills, and concrete aggregate.

10.2 Use of Materials

Diversion cofferdams will consist of temporary dikes designed to divert the river, in combination with channel excavation. Currently, it is assumed that these could be constructed from locally available random fill obtained from the immediate area of the dam site.

All aggregates (including coarse and fine aggregates for concrete, 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. The tuffaceous sandstone occurring at the dam site would not be suitable as a source of aggregates. Aggregates for filters and drains will be obtained by processing of the same quarry sources as exploited for concrete aggregates.

Rockfill for the dam will be obtained from required excavation, mostly sandstone units from the right bank spillway excavation, and from quarry run material. Since the local sandstone appears to be suitable for rockfill, there is a possibility of opening a sandstone quarry closer to the site than the igneous rock quarry indicated above. Materials for backfill will come from the required excavations, including use of tunnel excavation spoil.

Riprap for channel and slope protection, where required, will consist of over-sized material obtained from the rock quarries.

11 RECOMMENDED ADDITIONAL INVESTIGATIONS

11.1 Dam Site

In the next phase of study of the Río Indio Water Supply Project, investigations will continue with site characterization activities and development of geologic and engineering parameters required for detailed design and preparation of more accurate construction cost estimates. These will have to include the types of subsurface investigations that were cancelled in the present study. As engineering studies continue to optimize and finalize structure locations, axes, and alignments, the investigations will focus on developing information on foundation conditions for specific structures and areas.

11.1.1 Dam Site Drilling

The additional exploration drilling shown in Table 9 is recommended as a minimum, assuming that the type of dam will be a concrete faced rockfill dam. Another program of investigations may be required to support detailed design. Approximate hole locations are indicated on Exhibit 26.

Table 9: Proposed Drilling for Río Indio Dam

Location	Number of Drill Holes	Total Depth (m)
Plinth foundation, left side	4	200
Plinth foundation, right side	2	100
Dam foundation, downstream left side	3	150
Dam foundation, downstream right side	1	50
River valley section (angle holes under river)	2	200
Spillway	4	200
Diversion tunnel	2	150
Upstream Cofferdam	2	50

Location		Number of Drill Holes	Total Depth (m)
Saddle dam 1		4	200
Saddle dam 2		3	150
Quarry		4	200
Contingency		3	200
	Total	34	1,850

11.1.2 Dam Site Geologic Mapping

Detailed geologic mapping would be performed for the entire site area. The detailed geologic mapping will be performed to illustrate the detailed surface geology of proposed structure sites, usually in conjunction with subsurface exploration that will be performed in the same area. Detailed maps will be required for all major structure sites. This mapping should include:

- The surface extent and properties of all overburden units;
- Extent and lithology of all rock units including major interbeds or facies;
- All geologic contacts;
- Rock outcrop areas;
- Patterns of jointing, bedding, foliation and schistosity throughout the map area;
- Surface trace, attitude and width of faults and shear zones;
- Evidence of slope instability; and
- Springs, seeps, and any other relevant geologic condition.

The completed detailed geologic maps, in addition, should show the location of major project structures, the location of holes, adits, pits, trenches, seismic lines or other exploration features, and the location of geologic sections used to illustrate the subsurface geology. The maps should illustrate both the general, geology of the site, and the detailed, local geology.

11.1.3 Adits

It is recommended that the next phase of investigation include an exploration adit to assist in the evaluation of dam foundation conditions. The total length of adit excavation should be about 100 m.

11.1.4 *In Situ* Testing and Geophysics

Conventional water pressure testing will be performed in all exploration drill holes. All drill holes, wherever practical, will be converted to permanent groundwater monitoring holes.

The possible need for and practicality of performing dilatometer tests in selected drill holes should be also examined in order to obtain additional information on the range of rock mechanics properties of *in situ* bedrock.

Geophysical testing should be conducted throughout the dam site. In addition to surface seismic refraction, consideration should be given to downhole seismic (cross hole seismic) and possibly resistivity surveys. Collection of shear wave velocity data in exploration adit(s) and elsewhere is recommended as it is a valuable method of obtaining information on rock mass properties (modulus of deformation).

11.1.5 Construction Materials for Dam

Investigation of potential quarry sources should include core drilling, sampling, and testing. The number of drill holes and total length of drilling will depend on the location of quarry sites selected for investigation. For a basalt rock quarry, it is estimated that 4 drill holes will be required for a total of about 300 m. If a sandstone rock quarry is needed as well, additional drilling will be required. Rock core samples will be collected for laboratory testing which should include compression testing, soundness, LA abrasion, and concrete petrography, etc. as follows:

Bulk Specific Gravity of Rock	ASTM C 97
Unconfined Compressive Strength of Rock Core	ASTM D 2938
Petrography	ASTM C 295
Potential Reactivity of Aggregates	ASTM C 1260
Potential Alkali Reactivity	ASTM C 227
LA Abrasion	ASTM C 131 and ASTM C 535

A source of natural sand for blending in the concrete mix might need to be investigated other than using sand processed from quarry rock crushing.

Additional tests required on materials other than rock quarry, would include:

Visual classification of Soils	ASTM D 2488
Water Content	ASTM D 2216
Gradation	ASTM D 422
Specific Gravity	ASTM D 854
Liquid/ Plastic Limits and PI	ASTM D 4318
Unit weight	ASTM D 1188 (applicable sections) and D 2216
Moisture Density	ASTM D 698
Specific Gravity (rock)	ASTM C 97
Compressive strength	ASTM D 2938
Petrography	ASTM C 295
Sulfate Resistance	ASTM 88
Reactivity	ASTM 289 ASTM C 227
LA Abrasion	ASTM C131 and ASTM C 535
Point Load Test	ISRM

11.2 Water Transfer Tunnel

In the next phase, investigations for the water transfer tunnel will focus on a specific alignment and locations for the tunnel and other structures (i.e. Lake Gatun powerhouse). Activities will cover the following features:

- Water conveyance tunnel, intake, and outlet portal; information will be required to develop new or confirm previous data for estimating excavation and support requirements, construction technology, portal stabilization, groundwater inflows, etc.
- Penstock arrangement; information will be required to develop appropriate excavation and support requirements for the penstock including anchor blocks and saddle supports.
- Powerhouse; information will be required to develop an appropriate design for the foundation of the powerhouse. This will include data needed to estimate excavation depths to foundation grade, foundation conditions, groundwater control requirements, excavation stability, and other construction parameters.
- Tailrace; information will be obtained that will be used to develop excavation stability parameters and selection of suitable construction methodology.

Investigation activities will include geologic mapping, core drilling, and testing.

11.2.1 Core Drilling Investigation for Tunnel

The approximate scope of recommended core drilling includes:

Table 10: Additional Investigations for Water Transfer Tunnel

Location	Number of Drill Holes	Total Depth (m)
Transfer tunnel portals, backslopes	4	250
Penstock slope, key anchor blocks	2	70
Powerhouse	3	100
Tailrace	1	30
Total	10	450

Depending on the number and location of intermediate points of access for construction, additional drilling may be advisable at points along the transfer tunnel route.

11.2.2 Geologic Mapping

Detailed geologic mapping will be performed at the locations of tunnel portals and detailed geologic reconnaissance will be performed along the entire length of the tunnel corridor. The latter will be combined with revised photogeologic interpretations carried out on new aerial photography that should be flown as part of the next phase of study. The primary objective will be to revise and improve the current estimate of conditions that would be encountered during construction of the proposed tunnel.

11.2.3 *In Situ* Testing

Permeability testing will be performed in all exploration drill holes (Lugeon testing in rock, Lefranc testing in soil and overburden). Dilatometer and pressuremeter tests might be considered.

11.2.4 Construction Materials

Construction materials investigation for the water transfer tunnel facilities will be linked to those being carried out for the dam.

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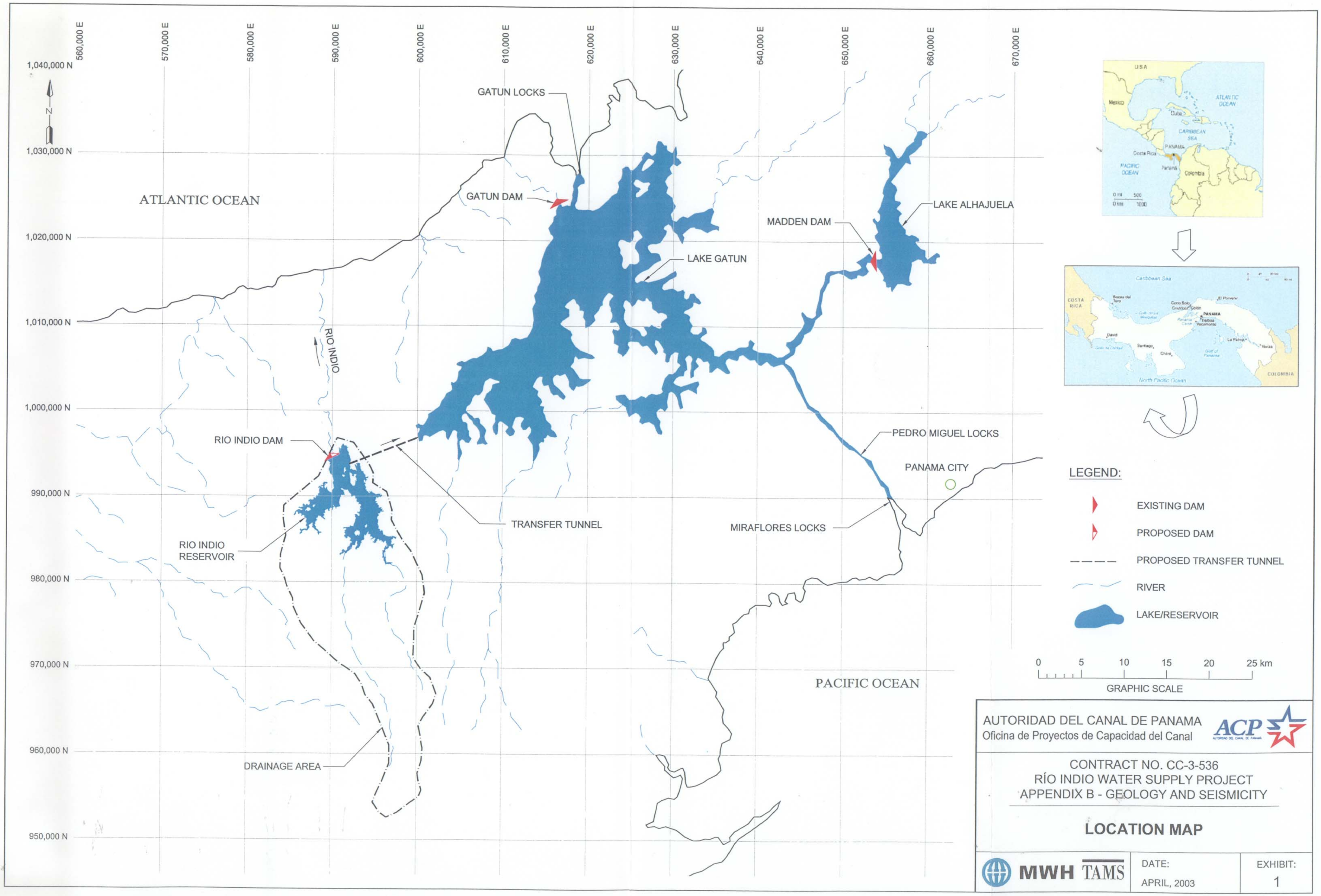
EXHIBITS



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LAKE ALHAJUELA
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TRANSFER TUNNEL
RIO INDIO
RESERVOIR
DRAINAGE AREA
PEDRO MIGUEL LOCKS
PANAMA CITY
MIRAFLORES LOCKS
PACIFIC OCEAN
USA
Mexico
Cuba
ATLANTIC OCEAN
CARIBBEAN SEA
PANAMA
Costa Rica
PACIFIC OCEAN
Colombia
Caribbean Sea
COSTA RICA
PANAMA
COLOMBIA
LEGEND:
EXISTING DAM
PROPOSED DAM
PROPOSED TRANSFER TUNNEL
RIVER
LAKE/RESERVOIR
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GRAPHIC SCALE
AUTORIDAD DEL CANAL DE PANAMA
Oficina de Proyectos de Capacidad del Canal
ACP
CONTRACT NO. CC-3-536
RÍO INDIO WATER SUPPLY PROJECT
APPENDIX B - GEOLOGY AND SEISMICITY
LOCATION MAP
MWH TAMS
DATE:
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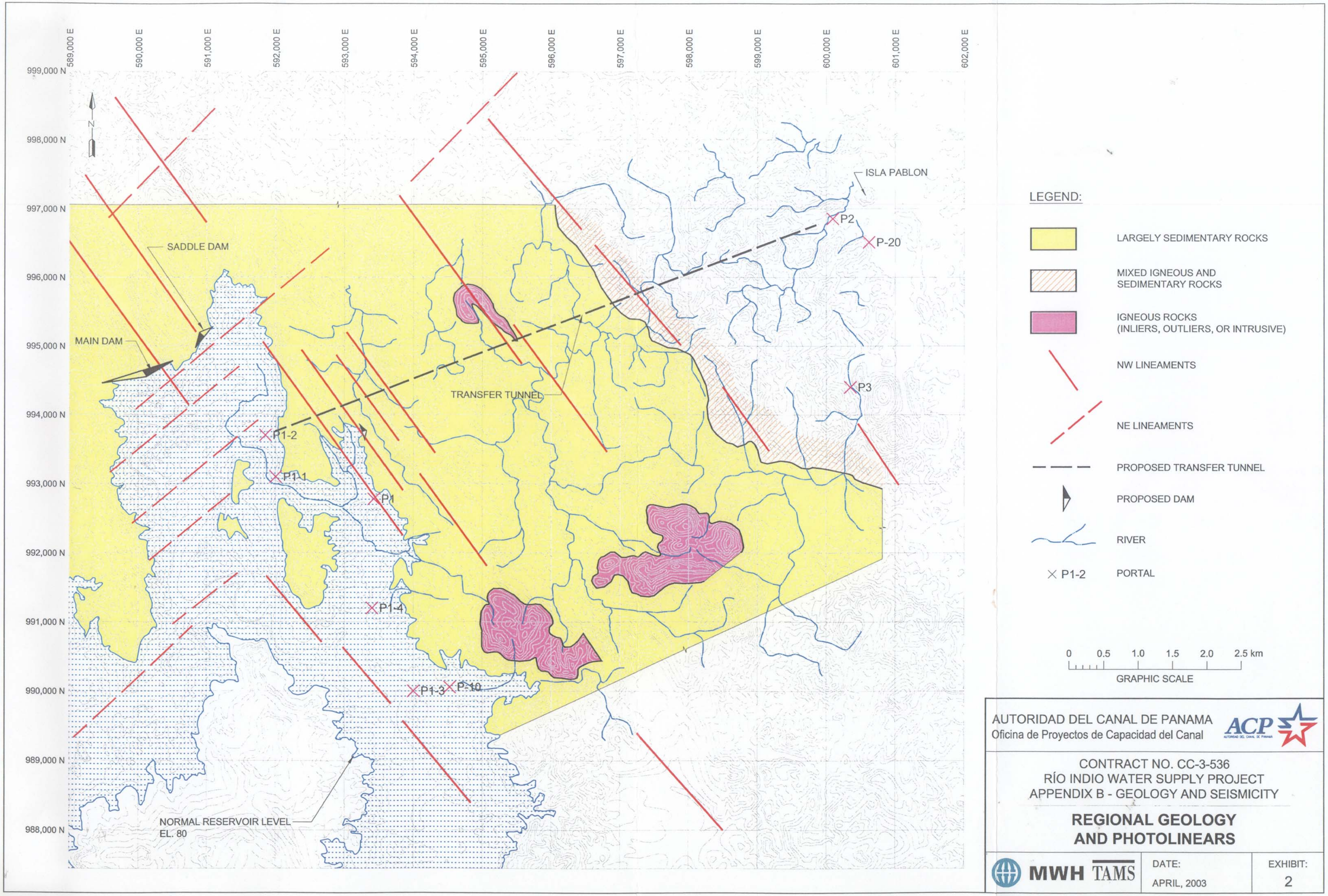

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RÍO INDIO WATER SUPPLY PROJECT
APPENDIX B - GEOLOGY AND SEISMICITY

REGIONAL GEOLOGY AND PHOTOLINEARS

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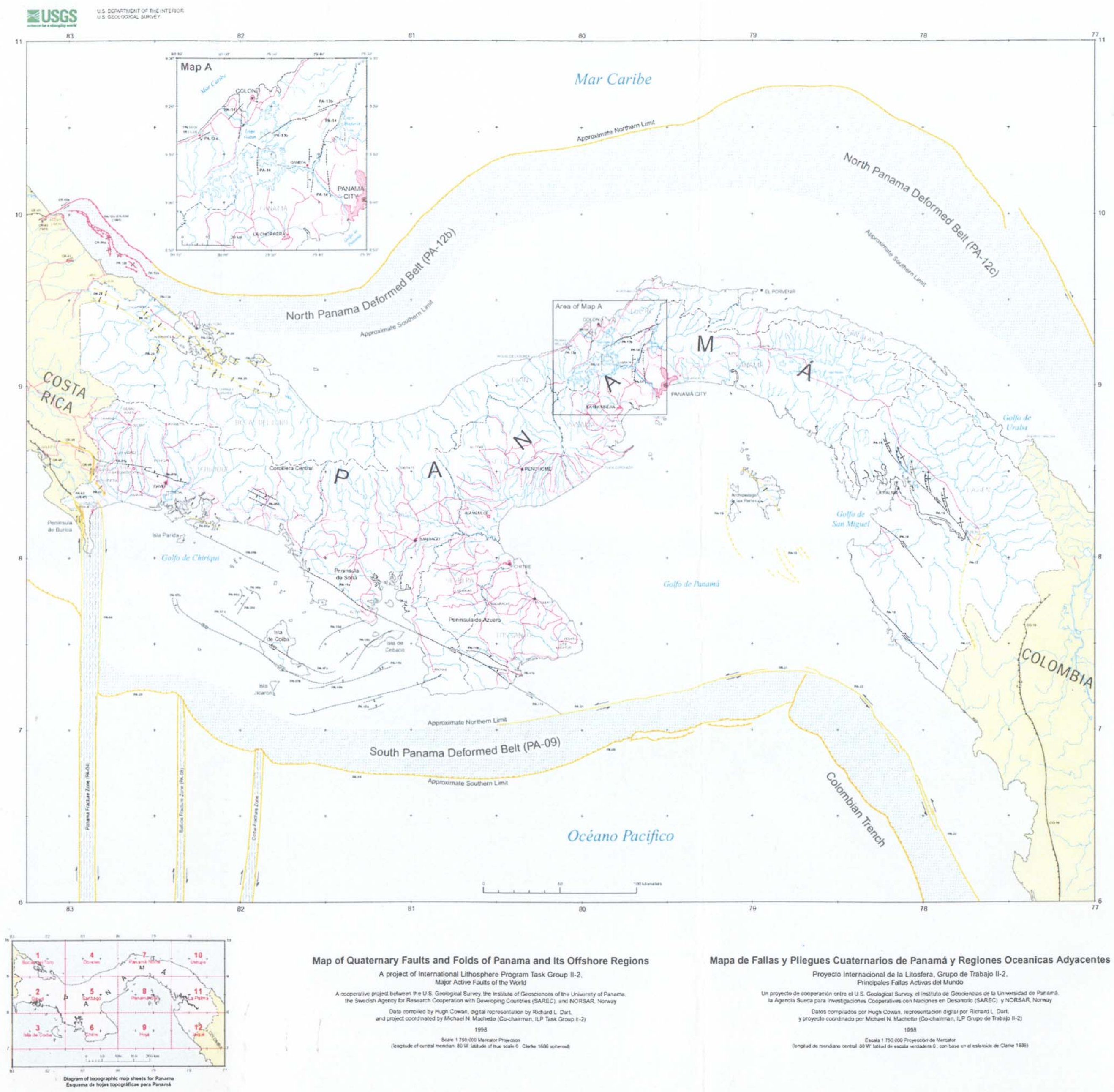
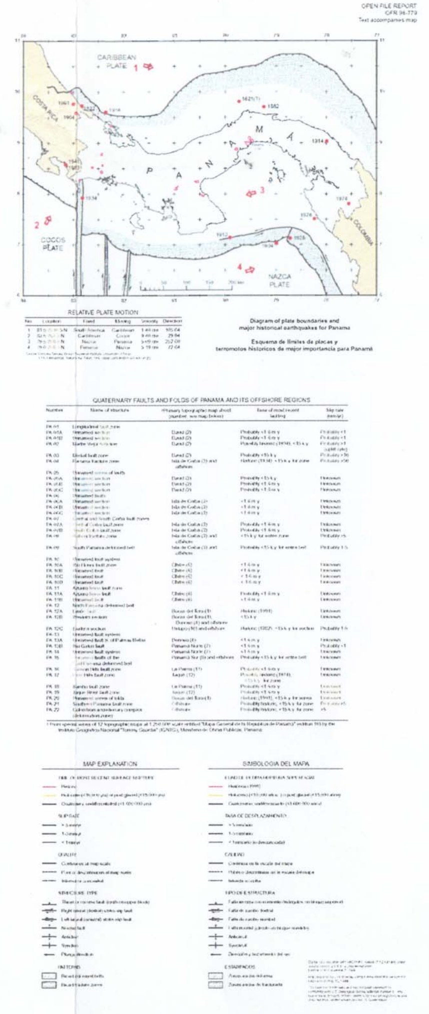
DATE:	EXHIBIT:
APRIL, 2003	2





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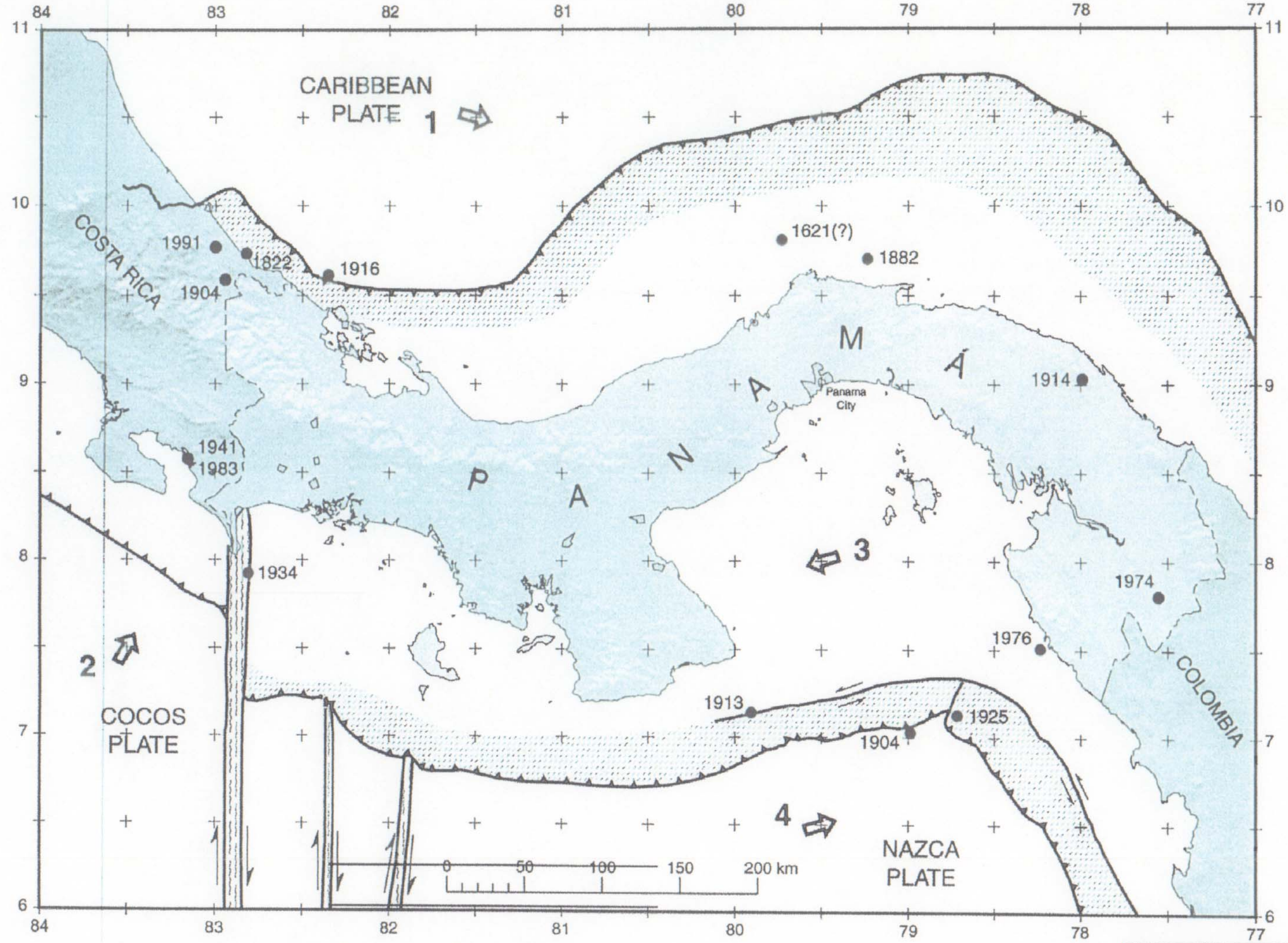
RÍO INDIO WATER SUPPLY PROJECT
APPENDIX B - GEOLOGY AND SEISMICITY

MAP OF FAULTS AND FOLDS IN PANAMA

MWH TAMS	DATE: APRIL, 2003	EXHIBIT: 3



RELATIVE PLATE MOTION

No.	Location	Fixed	Moving	Velocity	Direction
1	81.5 W/10.5 N	South America	Caribbean	1.40 cm	105.64
2	83.5 W/7.5 N	Caribbean	Cocos	9.40 cm	29.94
3	79.5 W/8.0 N	Nazca	Panama	5.09 cm	252.60
4	79.0 W/6.5 N	Panama	Nazca	5.19 cm	72.64

Source: Kensaku Tamaki, Ocean Research Institute, University of Tokyo
1-15-1 Minamidai, Nakano-ku, Tokyo, 164, Japan (tamaki@ori.u-tokyo.ac.jp)

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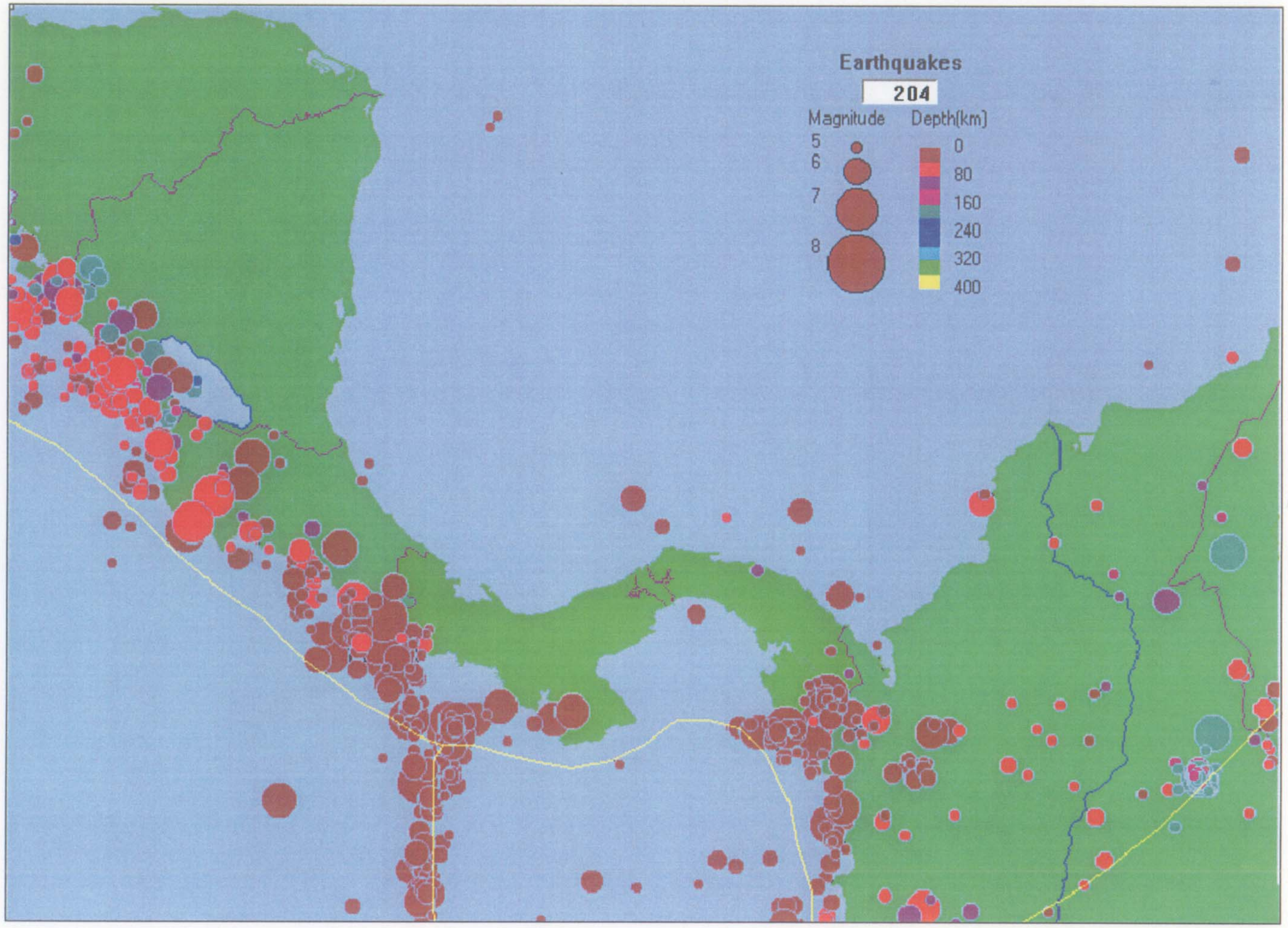
APPENDIX B - GEOLOGY AND SEISMICITY

DIAGRAM OF PLATE BOUNDARIES AND MAJOR HISTORICAL EARTHQUAKES

MWH TAMS

DATE: APRIL, 2003

EXHIBIT: 4



NOTE:

PLOT OF ALL EARTHQUAKES >M = 3.0 IN 30-YEAR PERIOD JANUARY 1960 TO JANUARY 1990.
YELLOW LINES INDICATE PLATE MARGINS.

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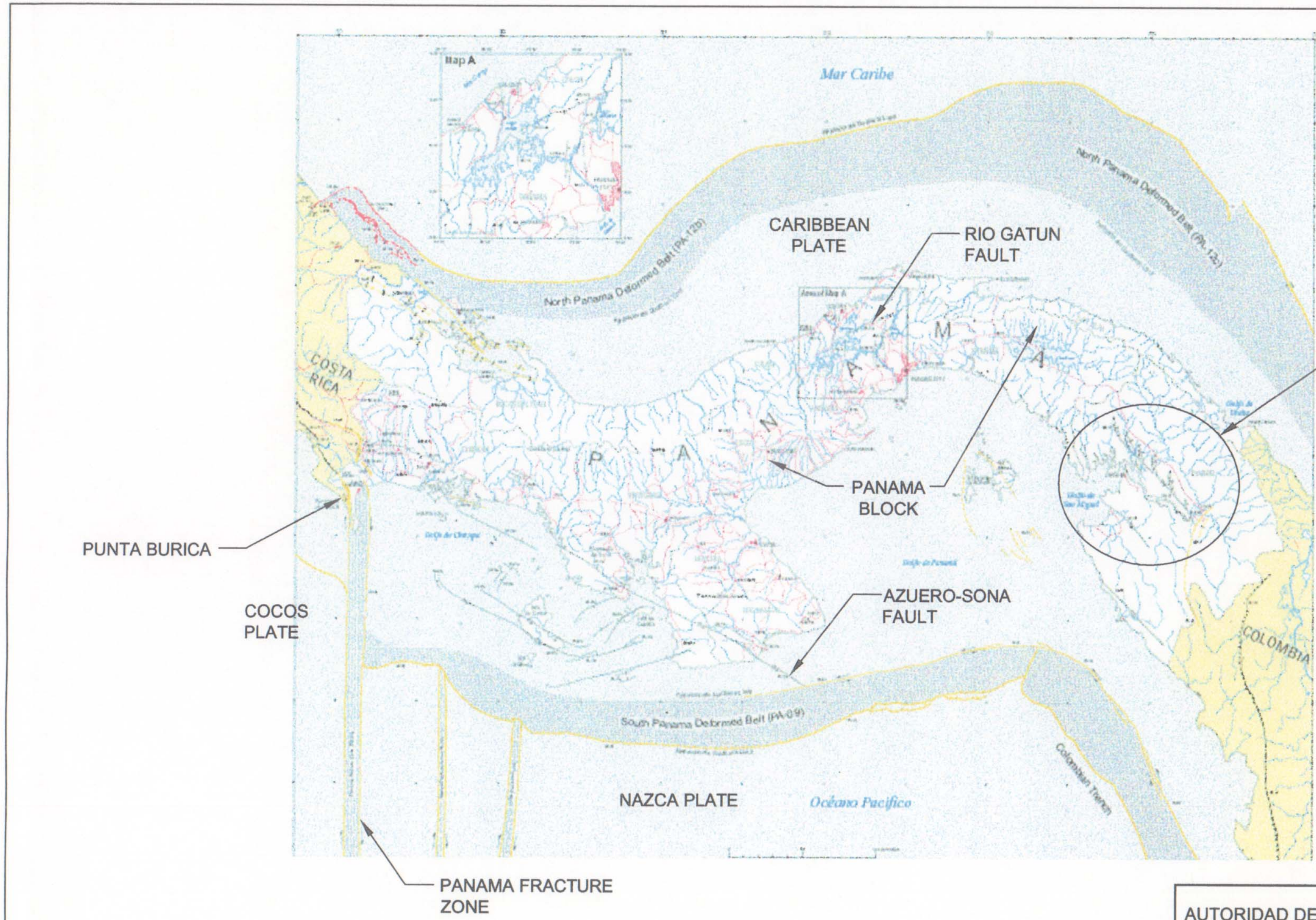
APPENDIX B - GEOLOGY AND SEISMICITY

SEISMICITY OF PANAMA

MWH TAMS

DATE: APRIL, 2003

EXHIBIT: 5



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APPENDIX B - GEOLOGY AND SEISMICITY

SEISMIC SOURCE ZONES

MWH TAMS

DATE: APRIL, 2003

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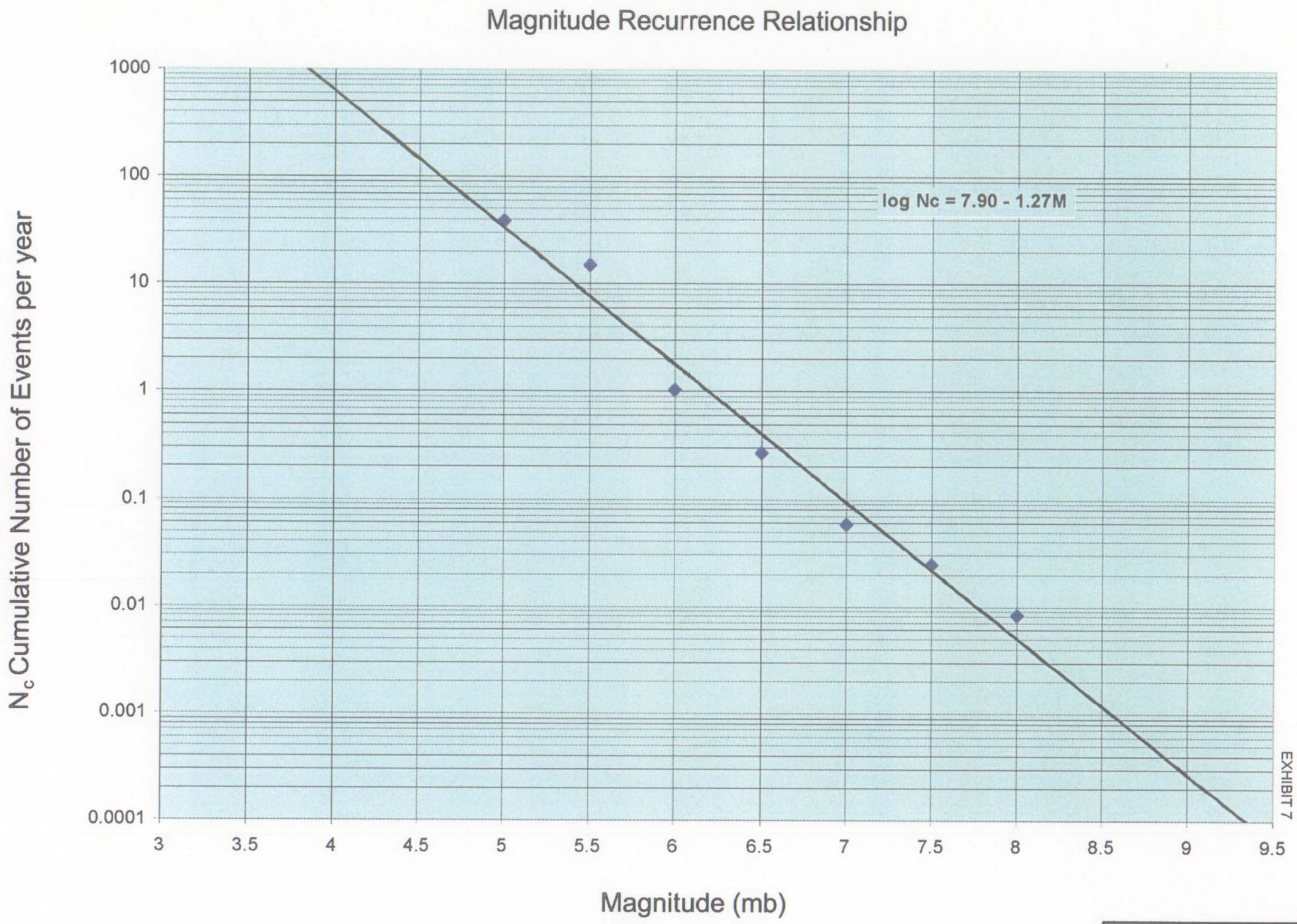

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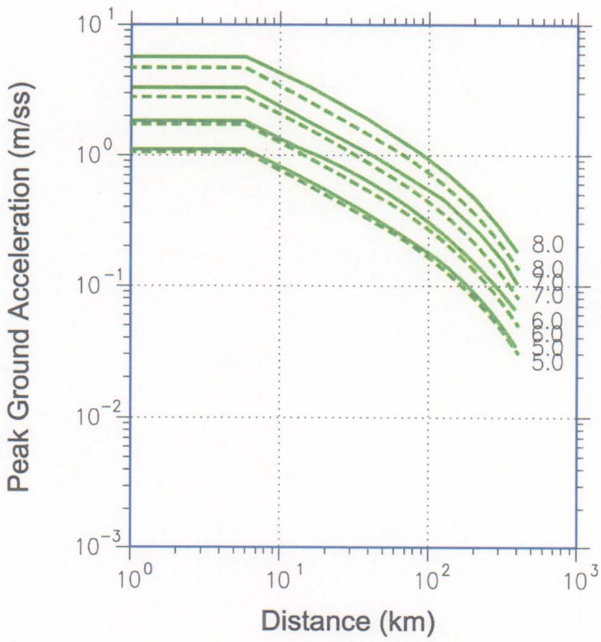
APPENDIX B - GEOLOGY AND SEISMICITY

PRELIMINARY MAGNITUDE RECURRENCE CURVE

MWH TAMS

DATE: APRIL, 2003

EXHIBIT: 7



Peak Ground Acceleration Curve for Central American Attenuation with (solid line) and without (dashed line) Guerrero Mexico Data (Climent et al., 1994)

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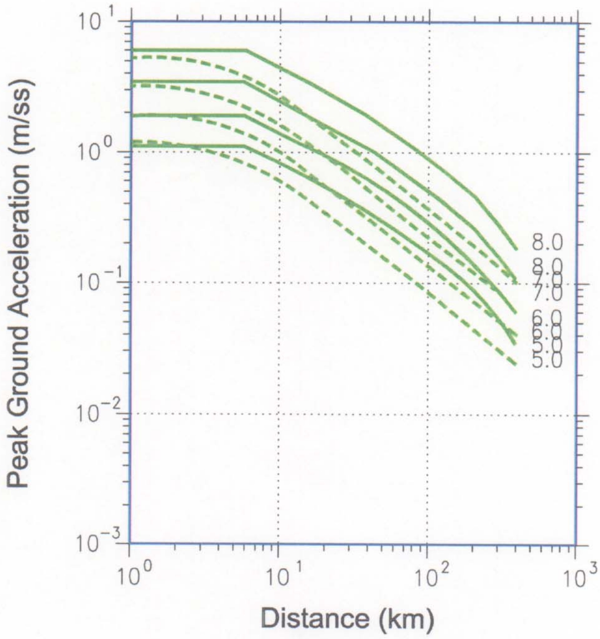
APPENDIX B - GEOLOGY AND SEISMICITY

ATTENUATION CURVES
SHEET 1 OF 4

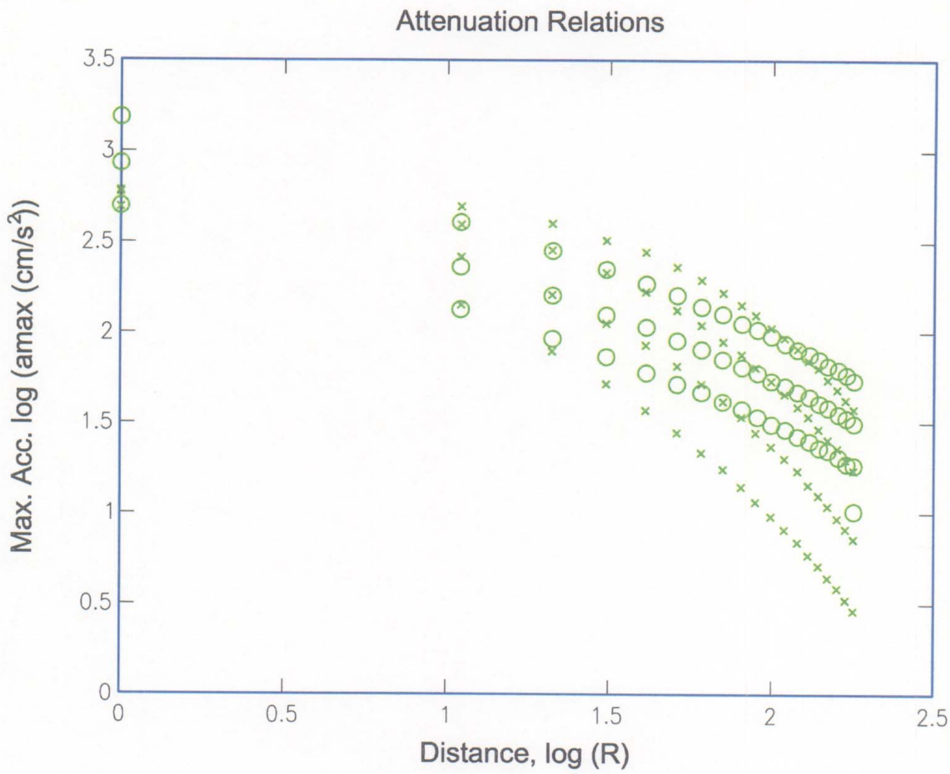
MWH TAMS

DATE: APRIL, 2003

EXHIBIT: 8



Peak Ground Acceleration Curve for Central American Attenuation with (solid line) compared to Boore et al. (dashed line) (Climent et al., 1994)



Peak ground Acceleration Curve for Central American Attenuation (circles) compared to Fukushima-Tanaka Attenuation Relationship (plus marks) (Bodare, 2001)

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

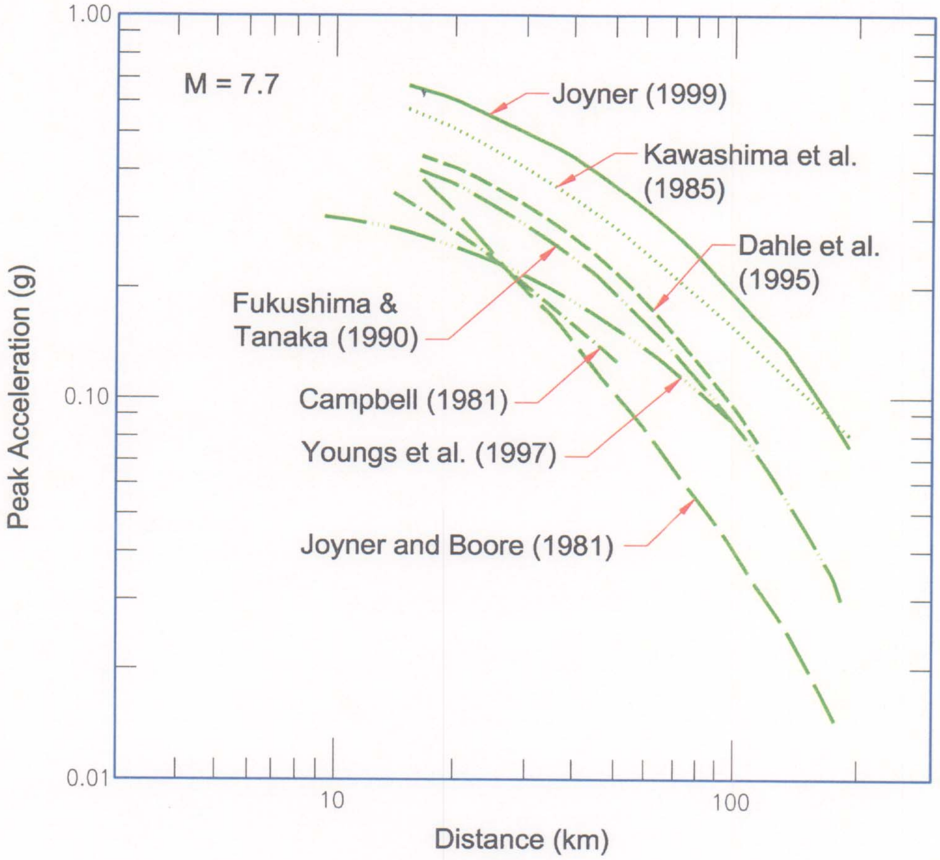
CONTRACT NO. CC-3-536
RÍO INDIO WATER SUPPLY PROJECT
APPENDIX B - GEOLOGY AND SEISMICITY

ATTENUATION CURVES
SHEET 3 OF 4

MWH TAMS

DATE: APRIL, 2003

EXHIBIT: 10



Comparison of Peak Ground Acceleration Attenuation Relationships for Subduction Zone Earthquakes (Kawashima et al, 1985; Fukushima and Tanaka, 1990; Youngs et al. 1997; Joyner, 1999) and Shallow Crustal Earthquakes (Campbell, 1981; Joyner and Boore, 1981) compared to Central American Attenuation Relationship (Dahle et al, 1995), (Exhibit from Cowan)

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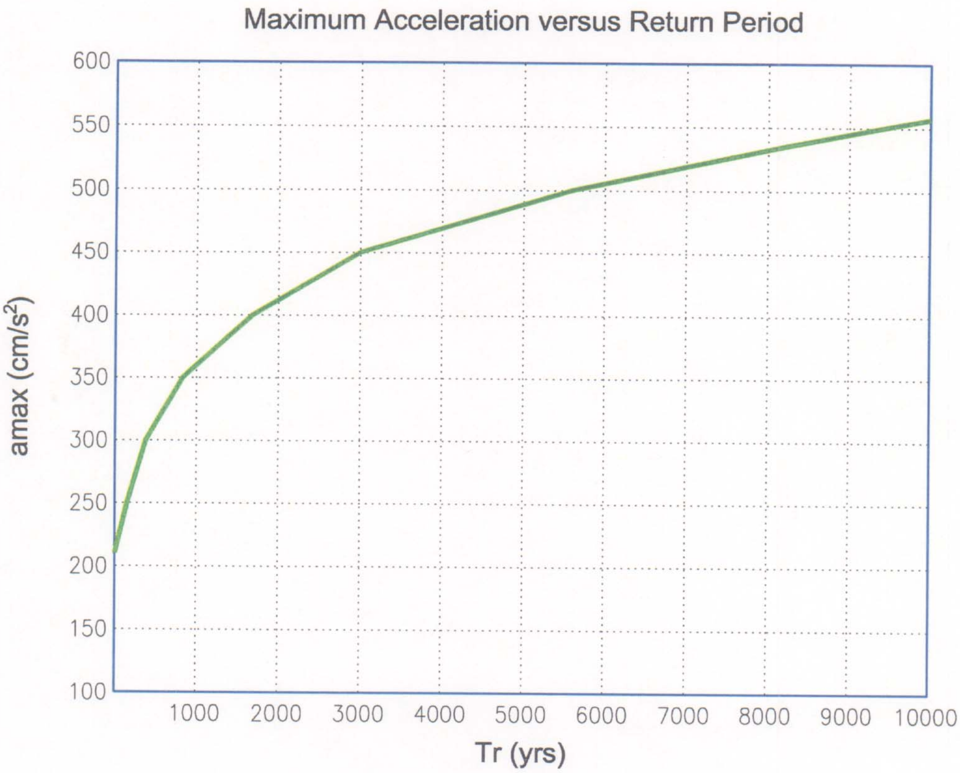
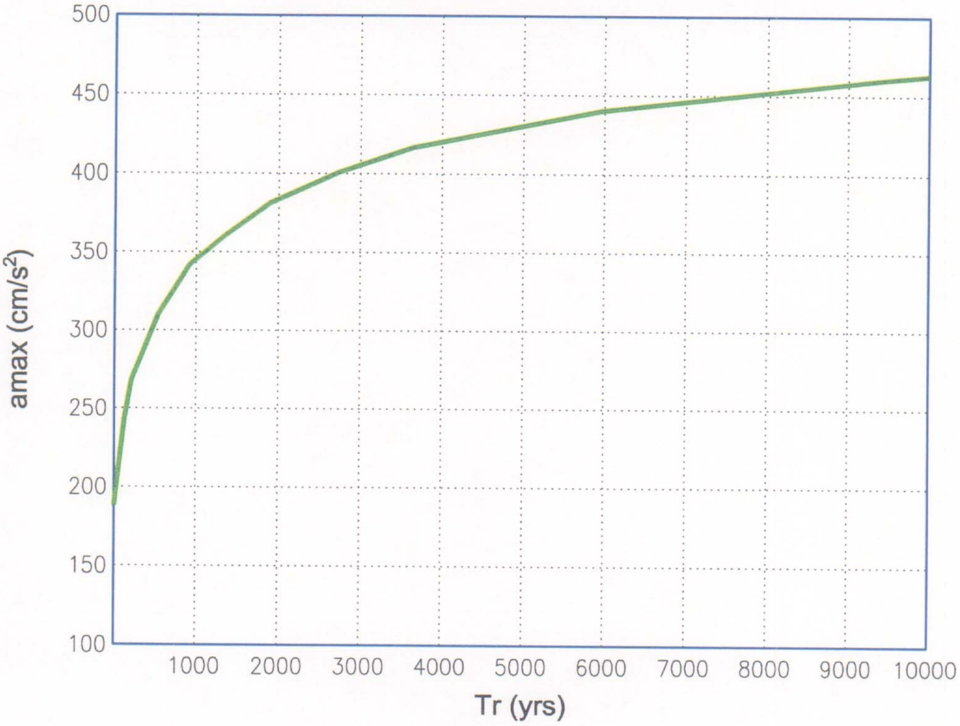
CONTRACT NO. CC-3-536 RÍO INDIO WATER SUPPLY PROJECT APPENDIX B - GEOLOGY AND SEISMICITY **ATTENUATION CURVES** **SHEET 4 OF 4**	
MWH TAMS	DATE: APRIL, 2003 — EXHIBIT: 11



Seismic Hazard Curve based on Central American Attenuation Curve (Bodare, 2001)

AUTORIDAD DEL CANAL DE PANAMA Oficina de Proyectos de Capacidad del Canal	ACP
CONTRACT NO. CC-3-536 RÍO INDIO WATER SUPPLY PROJECT APPENDIX B - GEOLOGY AND SEISMICITY **SEISMIC HAZARD CURVES SHEET 1 OF 10**	
MWH TAMS	DATE: APRIL, 2003 — EXHIBIT: 12

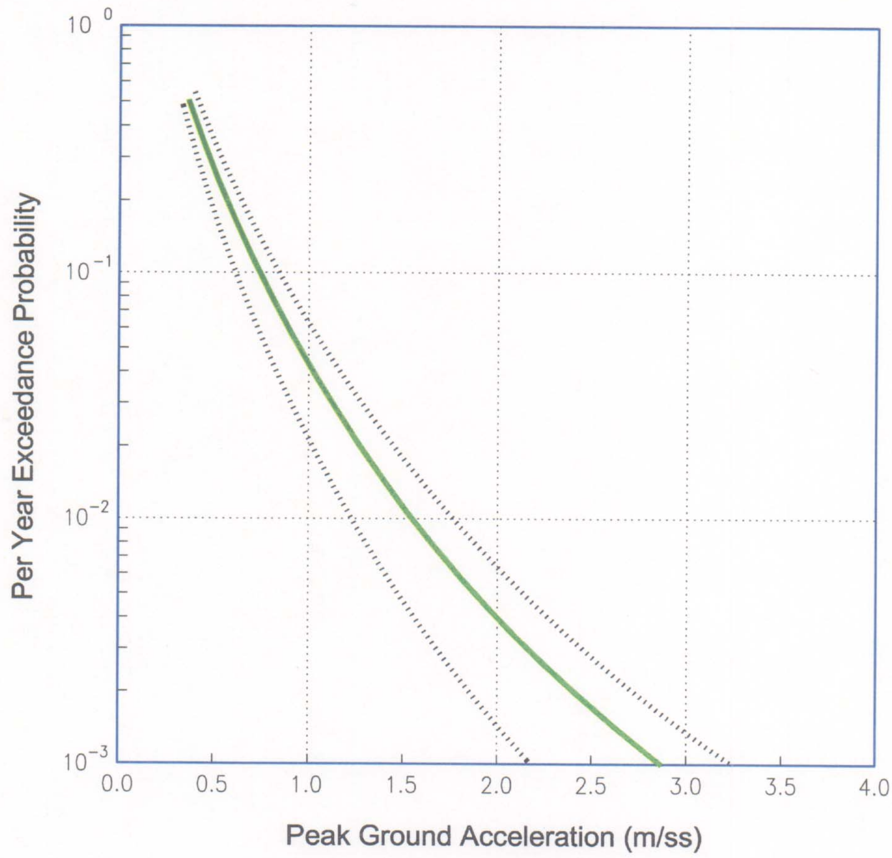
Maximum Acceleration versus Return Period



Seismic Hazard Curve based on Fukushima-Tanaka Attenuation Curve (Bodare, 2001)

AUTORIDAD DEL CANAL DE PANAMA Oficina de Proyectos de Capacidad del Canal	ACP
CONTRACT NO. CC-3-536 RÍO INDIO WATER SUPPLY PROJECT APPENDIX B - GEOLOGY AND SEISMICITY **SEISMIC HAZARD CURVES** **SHEET 2 OF 10**	
MWH TAMS	DATE: APRIL, 2003 — EXHIBIT: 13

Results for Site Location -80.544 8.241



Seismic Hazard Curve for Aguadulce, Cocle (Camacho et al., 1994)

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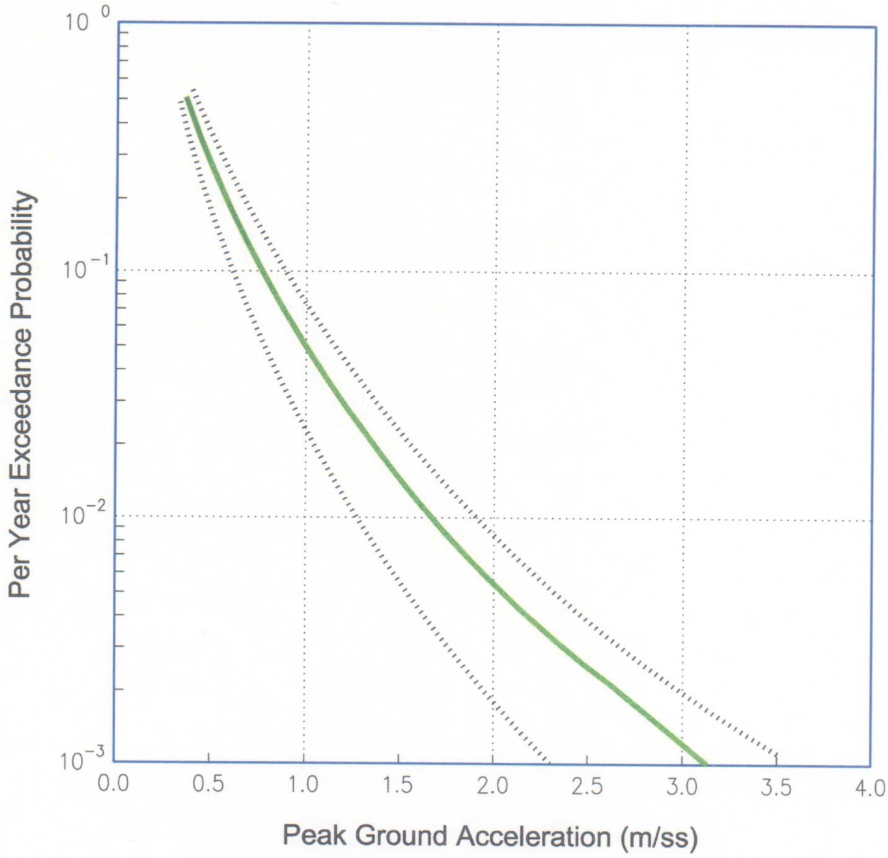
SEISMIC HAZARD CURVES
SHEET 3 OF 10

MWH TAMS

DATE: APRIL, 2003

EXHIBIT: 14

Results for Site Location -80.433 7.961



Seismic Hazard Curve for Chitre, Herrera (Camacho et al., 1994)

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

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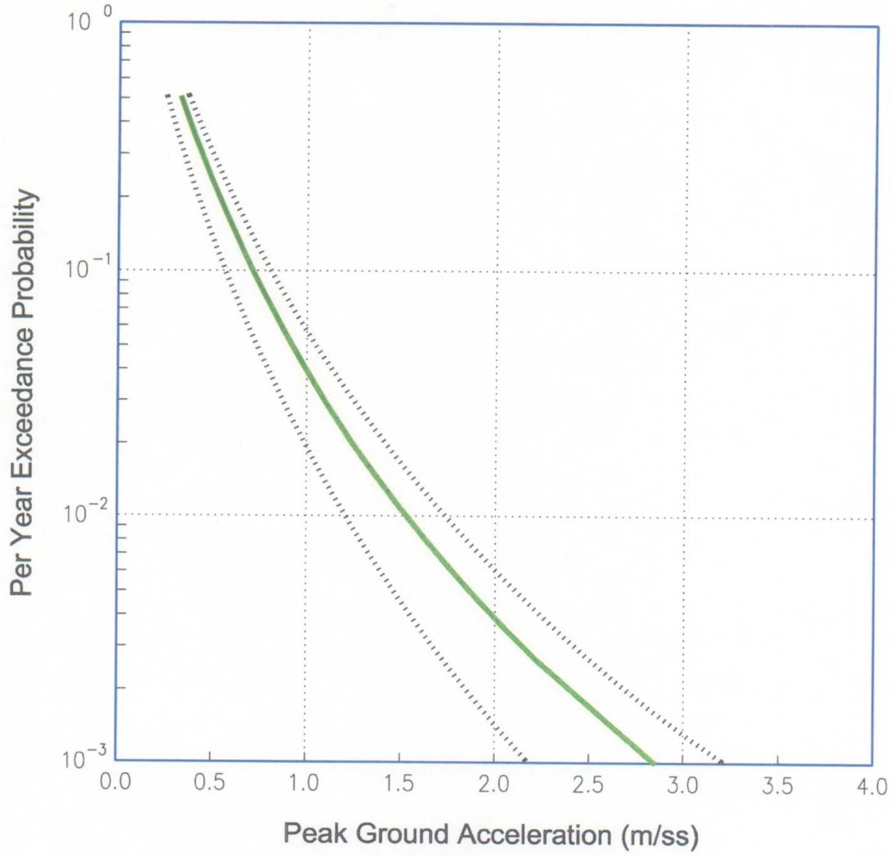
SEISMIC HAZARD CURVES
SHEET 4 OF 10

MWH TAMS	DATE: APRIL, 2003	EXHIBIT: 15

Results for Site Location -79.782 8.879



Seismic Hazard Curve for Chorrera, Panama (Camacho et al., 1994)

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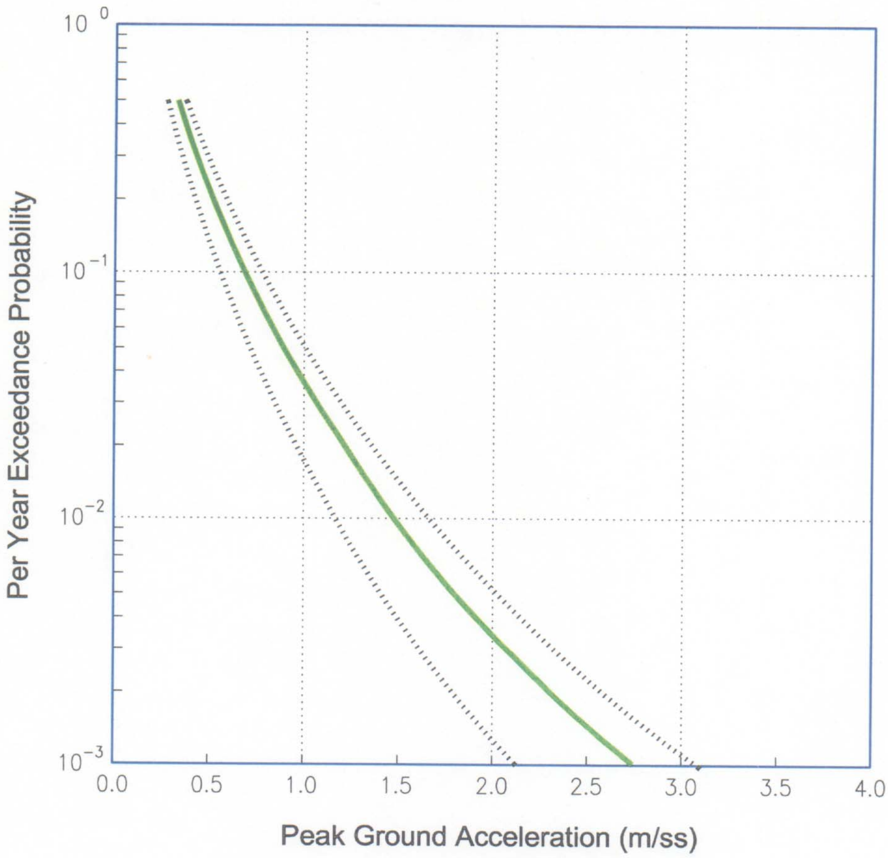
SEISMIC HAZARD CURVES
SHEET 5 OF 10

MWH TAMS

DATE: APRIL, 2003

EXHIBIT: 16

Results for Site Location -79.883 8.525



Seismic Hazard Curve for Coronado, Panama (Camacho et al., 1994)

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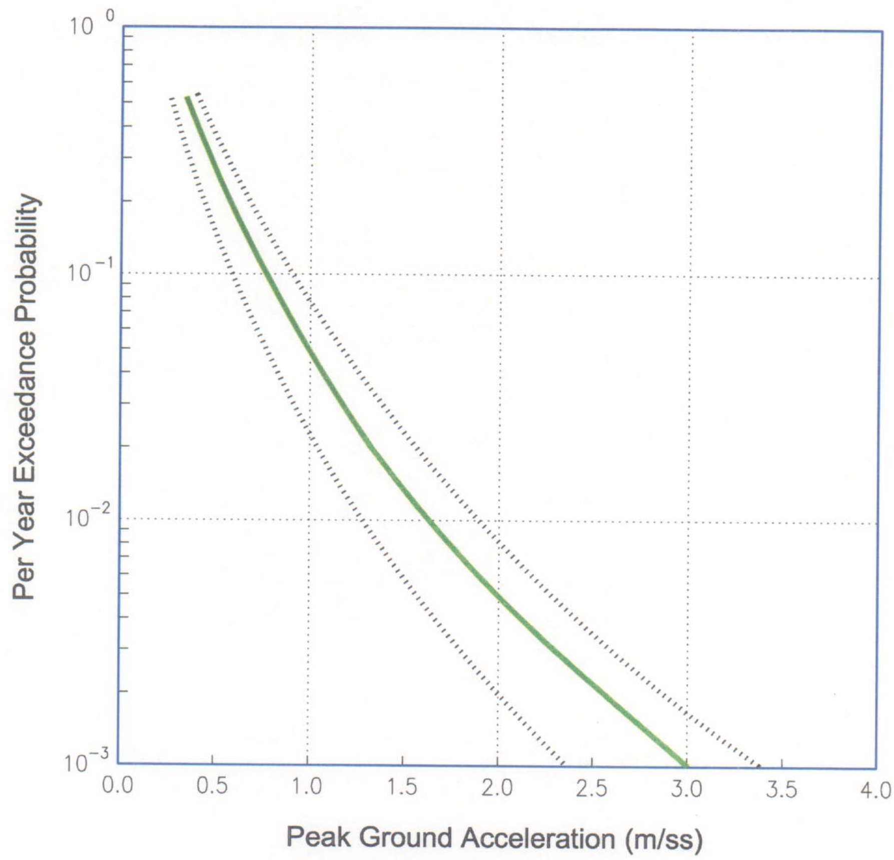
SEISMIC HAZARD CURVES
SHEET 6 OF 10

MWH TAMS

DATE:	EXHIBIT:
APRIL, 2003	17

Results for Site Location -79.534 8.984



Seismic Hazard Curve for Panama City, Panama (Camacho et al., 1994)

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RÍO INDIO WATER SUPPLY PROJECT
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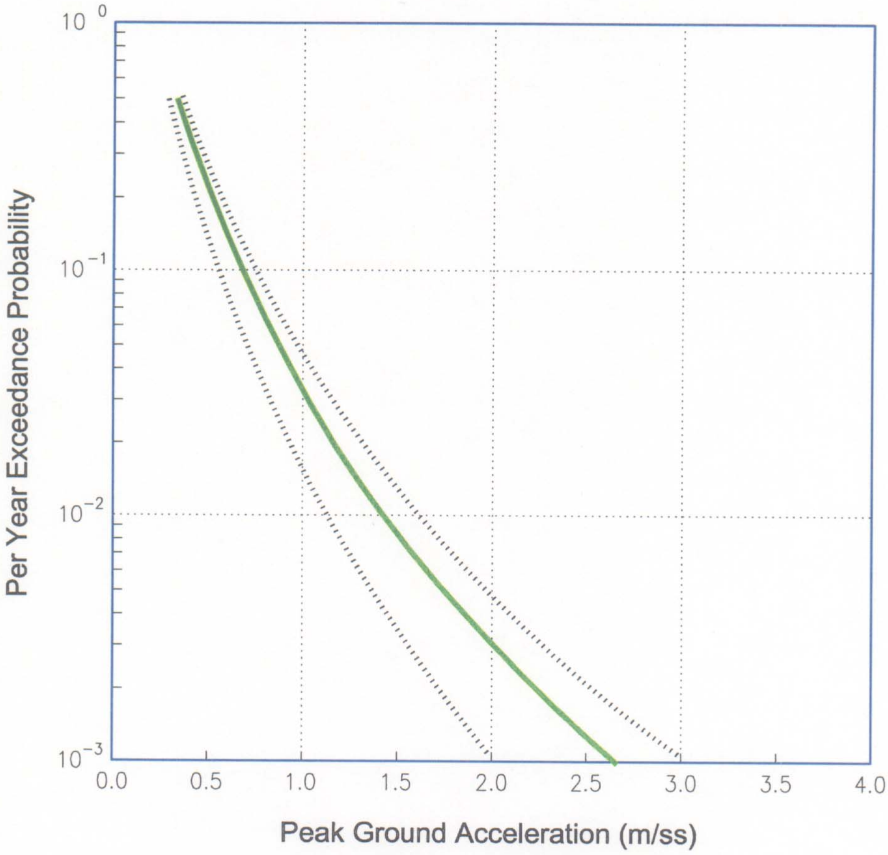
SEISMIC HAZARD CURVES
SHEET 7 OF 10

MWH TAMS

DATE: APRIL, 2003

EXHIBIT: 18

Results for Site Location -80.358 8.518



Seismic Hazard Curve for Penonome, Cocle (Camacho et al., 1994)

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Oficina de Proyectos de Capacidad del Canal

CONTRACT NO. CC-3-536
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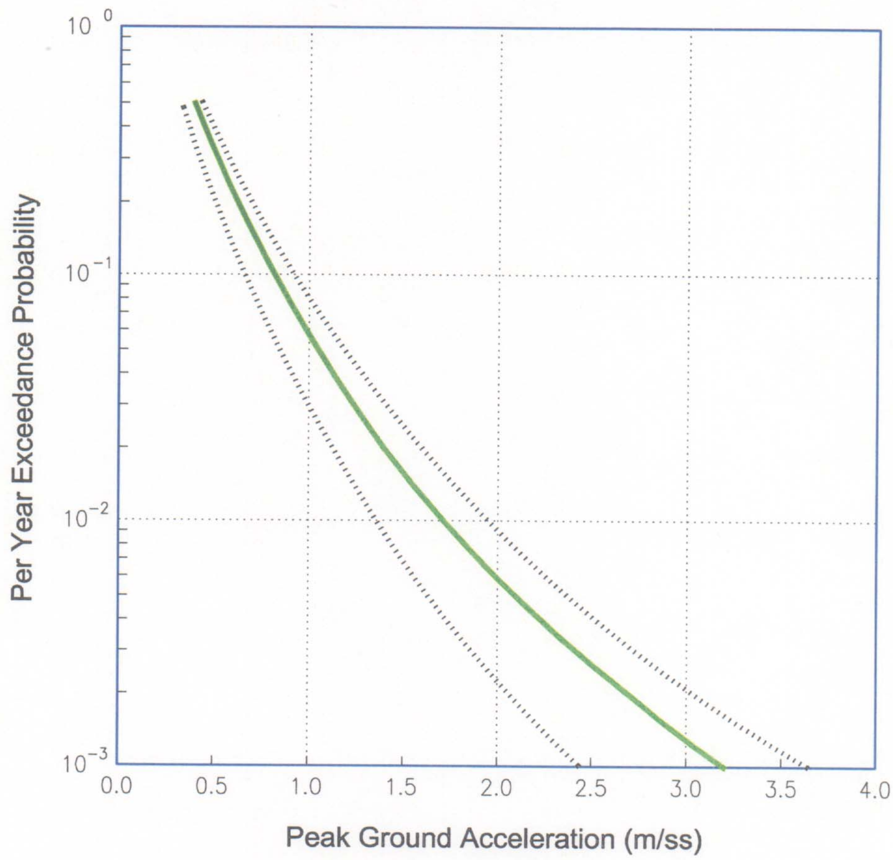
SEISMIC HAZARD CURVES
SHEET 8 OF 10

MWH TAMS

DATE: APRIL, 2003

EXHIBIT: 19

Results for Site Location -80.979 8.101



Seismic Hazard Curve for Santiago, Veraguas (Camacho et al., 1994)

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CONTRACT NO. CC-3-536
RÍO INDIO WATER SUPPLY PROJECT
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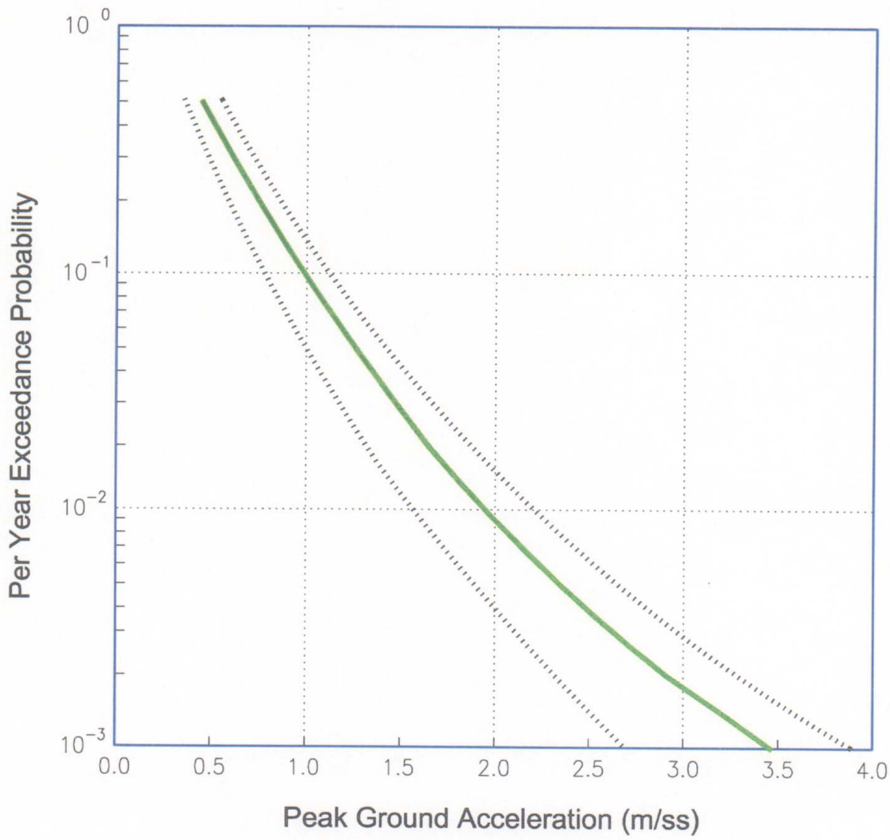
SEISMIC HAZARD CURVES
SHEET 9 OF 10

MWH TAMS

DATE:	EXHIBIT:
APRIL, 2003	20

Results for Site Location -81.319 8.008



Seismic Hazard Curve for Sona, Veraguas (Camacho et al., 1994)

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ACP

CONTRACT NO. CC-3-536
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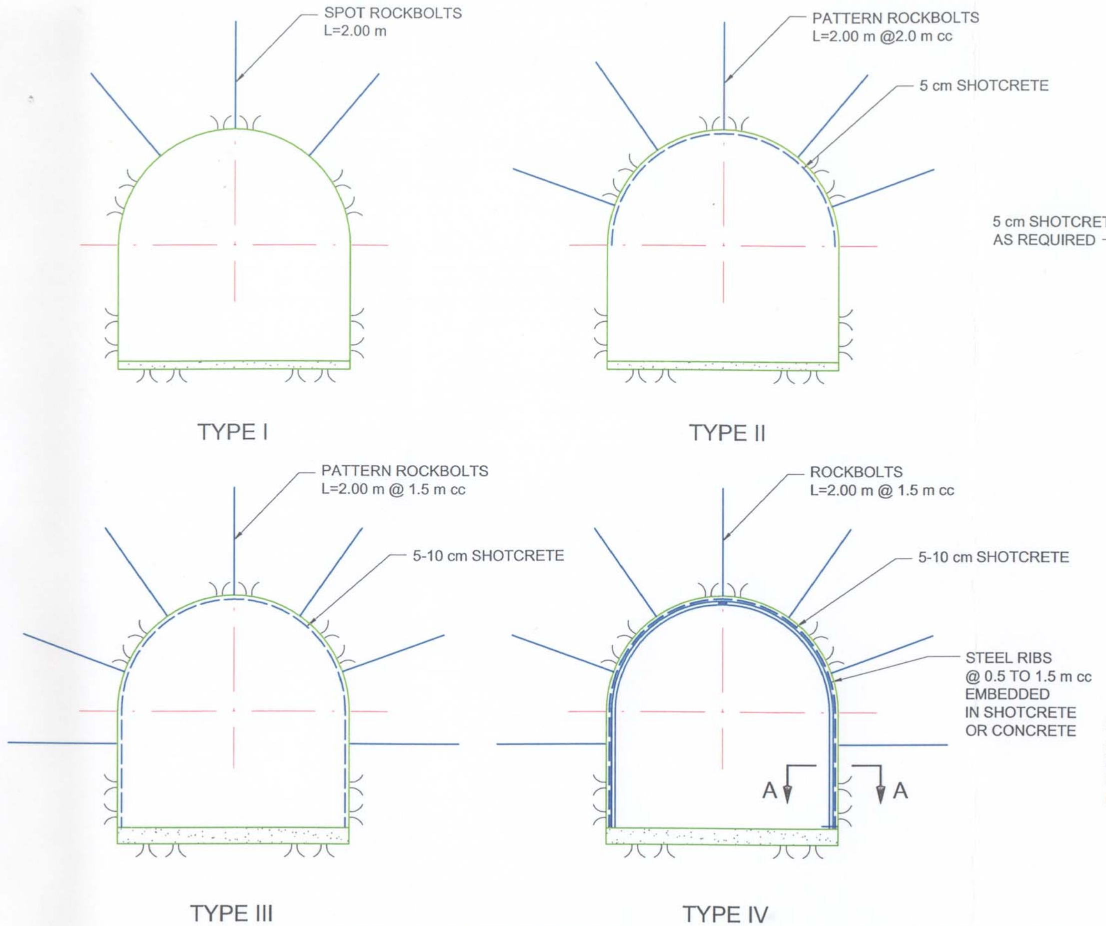
APPENDIX B - GEOLOGY AND SEISMICITY

SEISMIC HAZARD CURVES
SHEET 10 OF 10

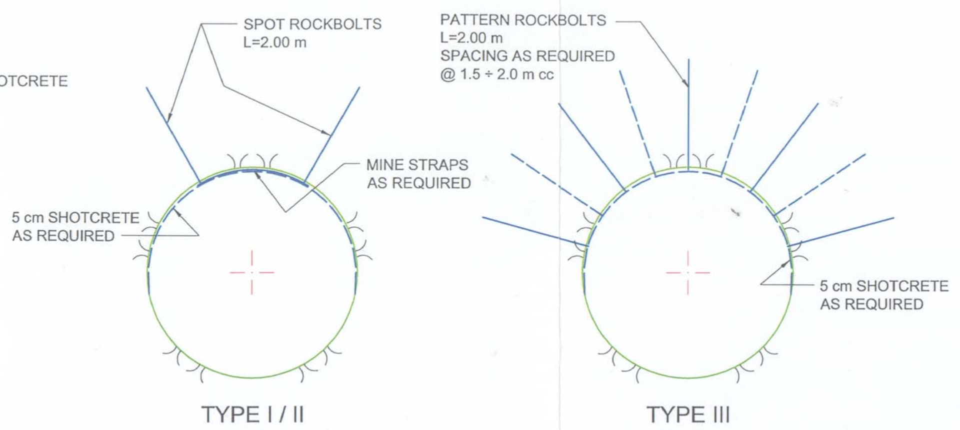
MWH TAMS

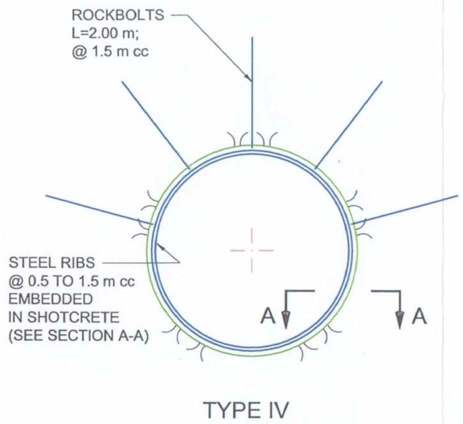
DATE: APRIL, 2003

EXHIBIT: 21



DRILL & BLAST SECTIONS





TBM SECTIONS

ROCK CONDITION CATEGORIES:

TYPE I - BEST ROCK CONDITIONS, MINIMAL OVERBREAK, GENERALLY SELF-SUPPORTING OR REQUIRING MINIMAL SUPPORT WITH SHOTCRETE OR SPOT BOLTING, FULL FACE EXCAVATION WITH NORMAL ADVANCE.

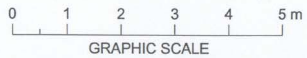
TYPE II - GOOD TO FAIR ROCK CONDITIONS, MODERATE OVERBREAK WITH ROCKBOLTS AND SHOTCRETE; NORMAL ADVANCE POSSIBLE WITH PROPER BOLTING AND SHOTCRETING.

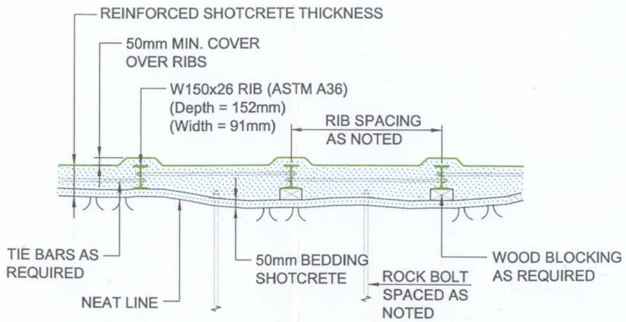
TYPE III - POOR ROCK CONDITIONS, WEATHERED OR WEAK ROCK, LOOSELY JOINTED, FULL FACE EXCAVATION WITH SLOWER SHORT ADVANCE AND LARGE OVERBREAKS. REQUIRES PROMPT SUPPORT WITH PATTERN ROCKBOLTING AND SHOTCRETE.

TYPE IV - VERY POOR ROCK CONDITIONS, FAULT AND SHEAR ZONES HIGHLY WEATHERED; PROMPT SUPPORT WITHIN THE OPEN FACE WITH STEEL RIBS AND LAGGING, BACKPACKING, REINFORCED SHOTCRETE; GROUTING MAY BE NECESSARY TO CONTROL WATER.

SHOTCRETE TO BE STEEL-FIBER REINFORCED OR INSTALLED WITH WIREMESH.

ALL ROCKBOLTS FULLY GROUTED, Ø 25 mm.





SECTION A-A
N.T.S.

AUTORIDAD DEL CANAL DE PANAMA
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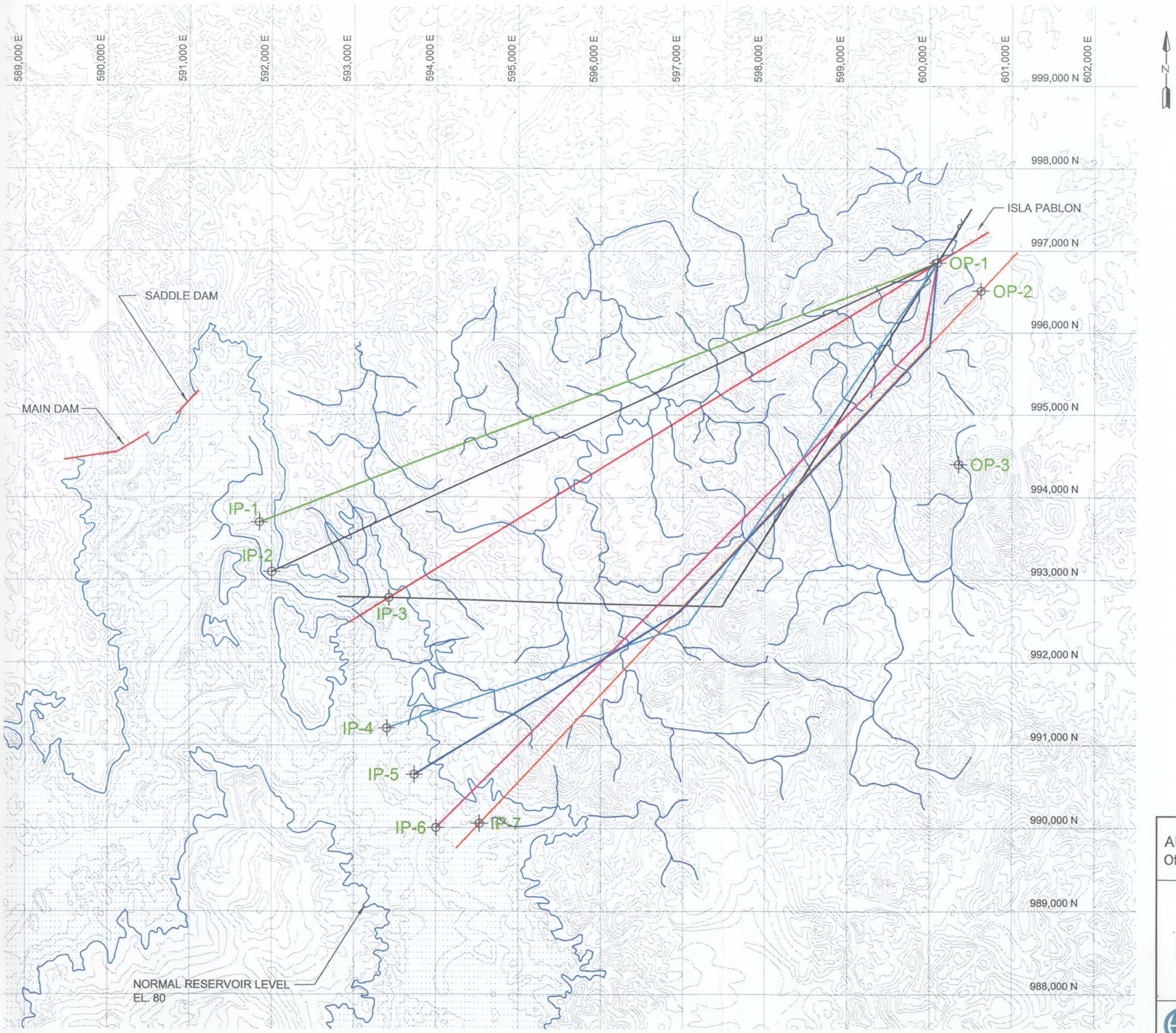
RÍO INDIO WATER SUPPLY PROJECT
APPENDIX B - GEOLOGY AND SEISMICITY

TYPICAL TUNNEL CROSS SECTIONS

MWH TAMS	DATE: APRIL, 2003	EXHIBIT: 22



LEGEND:

QUEBRADAS

IP-1 — OP-1 TUNNEL ALIGNMENT

TRANS-BASIN TRANSFER TUNNEL

0 0.5 1.0 1.5 2.0 2.5 km

GRAPHIC SCALE

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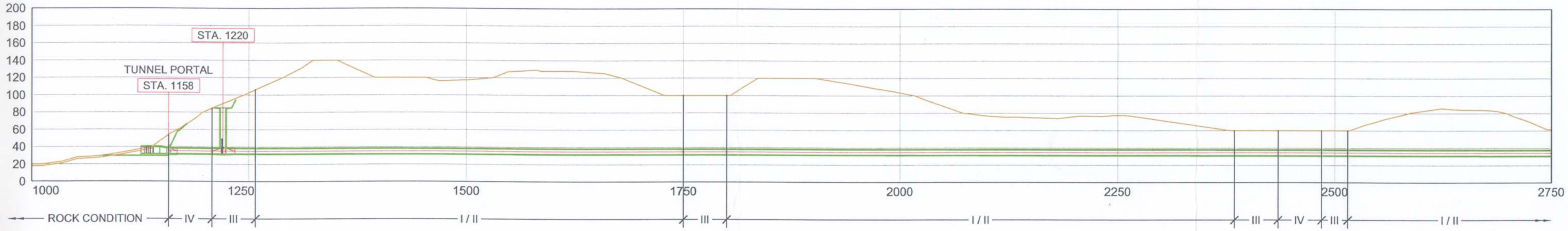
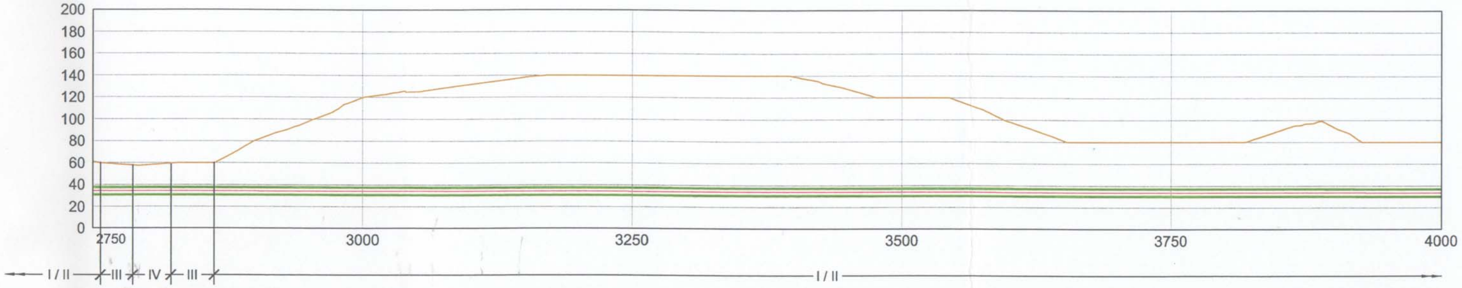
TUNNEL ALIGNMENT ALTERNATIVES

MWH TAMS

DATE:	EXHIBIT:
APRIL, 2003	23





NOTE:

SEE EXHIBIT 22 FOR THE DESCRIPTION OF THE ROCK CONDITION CATEGORIES.

0 50 100 150 200 250 m

GRAPHIC SCALE

AUTORIDAD DEL CANAL DE PANAMA

Oficina de Proyectos de Capacidad del Canal

CONTRACT NO. CC-3-536

RÍO INDIO WATER SUPPLY PROJECT

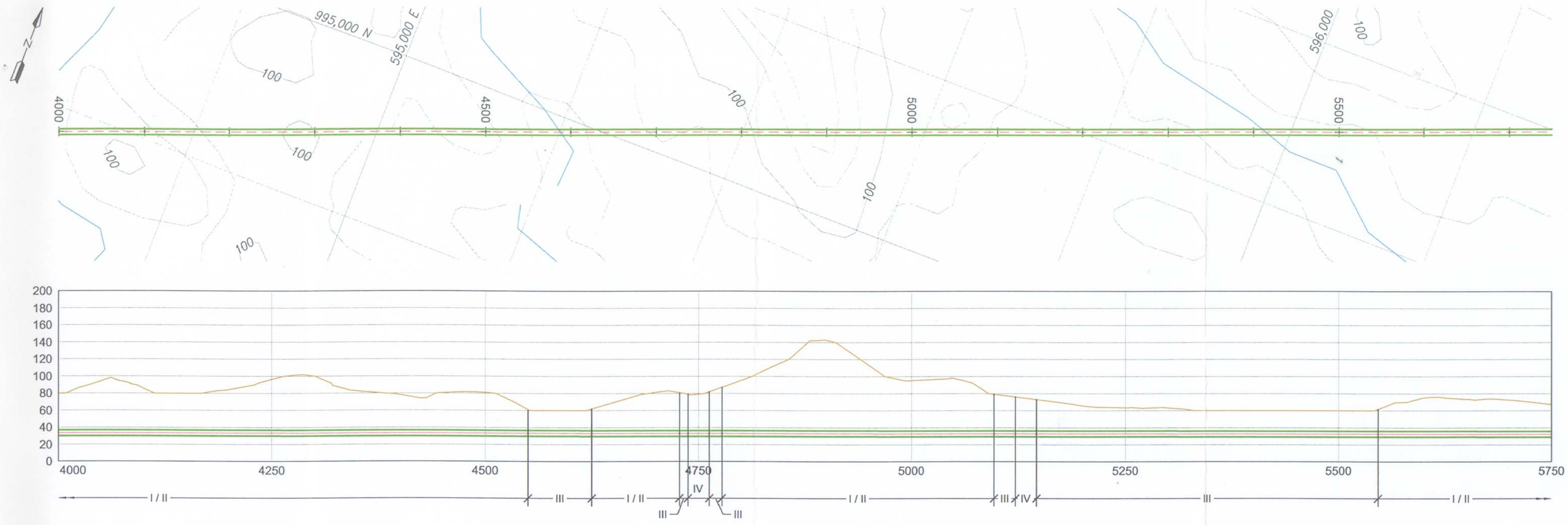
APPENDIX B - GEOLOGY AND SEISMICITY

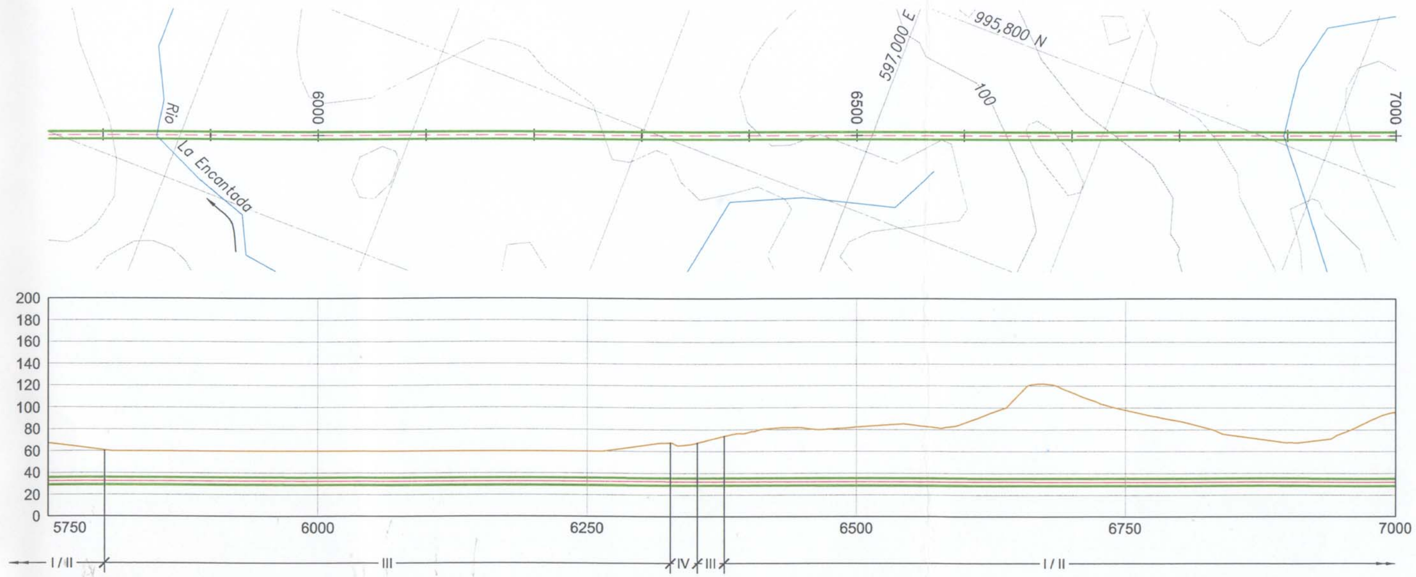
SELECTED WATER TRANSFER TUNNEL PROFILE - SHEET 1 OF 3

MWH TAMS

DATE: APRIL, 2003

EXHIBIT: 24





NOTE:

SEE EXHIBIT 22 FOR THE DESCRIPTION OF THE ROCK CONDITION CATEGORIES.

0 50 100 150 200 250 m

GRAPHIC SCALE

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

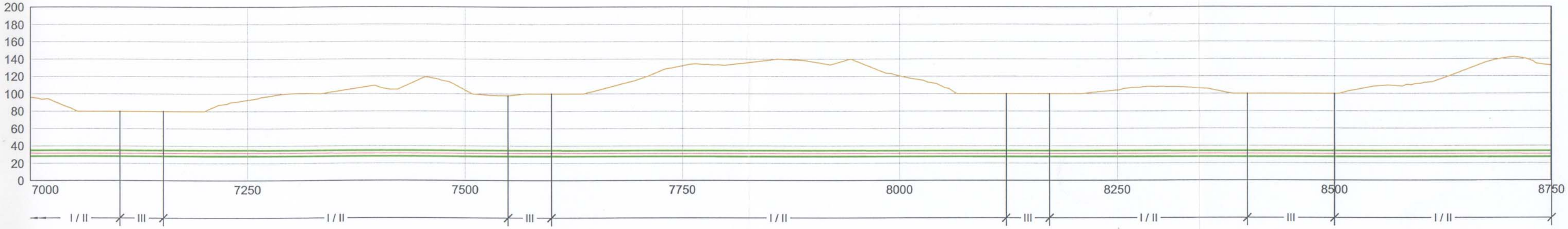
ACP

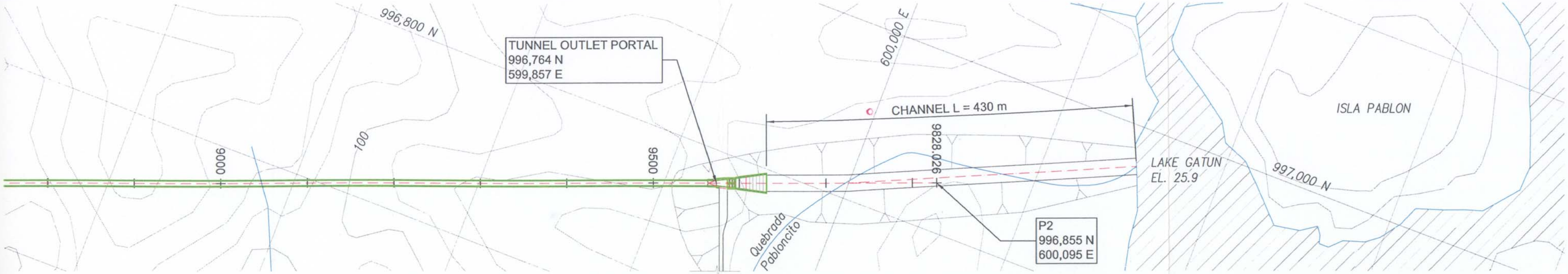
CONTRACT NO. CC-3-536
RÍO INDIO WATER SUPPLY PROJECT
APPENDIX B - GEOLOGY AND SEISMICITY

SELECTED WATER TRANSFER TUNNEL PROFILE - SHEET 2 OF 3

MWH TAMS

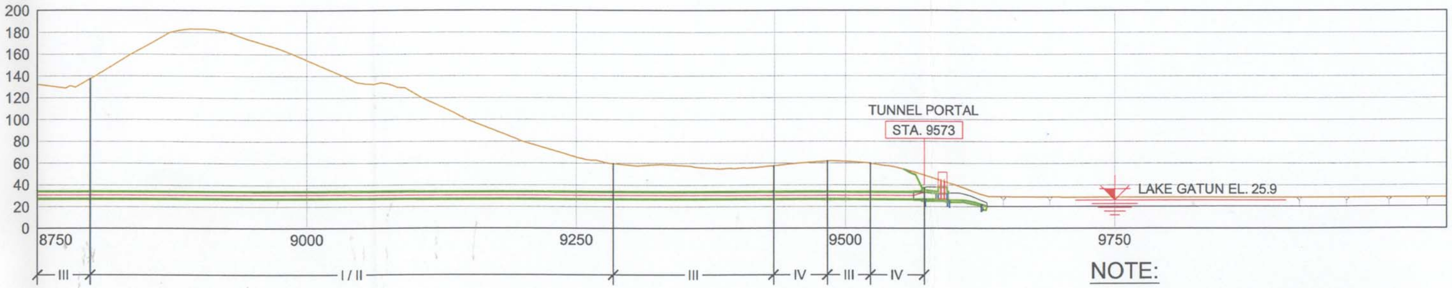
DATE:	EXHIBIT:
APRIL, 2003	24





0 50 100 150 200 250 m

GRAPHIC SCALE



NOTE:

SEE EXHIBIT 22 FOR THE DESCRIPTION OF THE ROCK CONDITION CATEGORIES.

AUTORIDAD DEL CANAL DE PANAMA
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CONTRACT NO. CC-3-536
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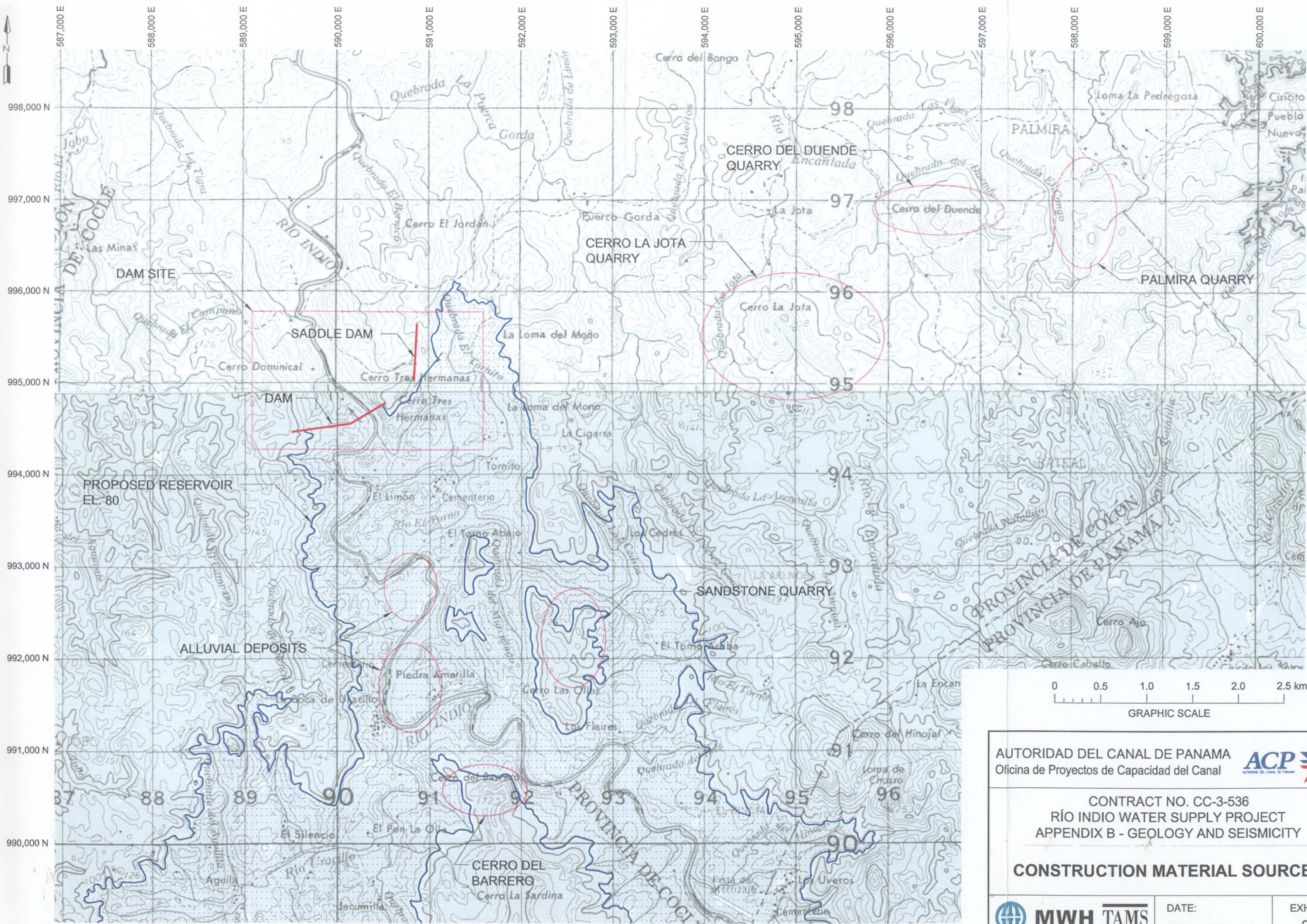
APPENDIX B - GEOLOGY AND SEISMICITY

SELECTED WATER TRANSFER TUNNEL PROFILE - SHEET 3 OF 3

MWH TAMS

DATE:	EXHIBIT:
APRIL, 2003	24



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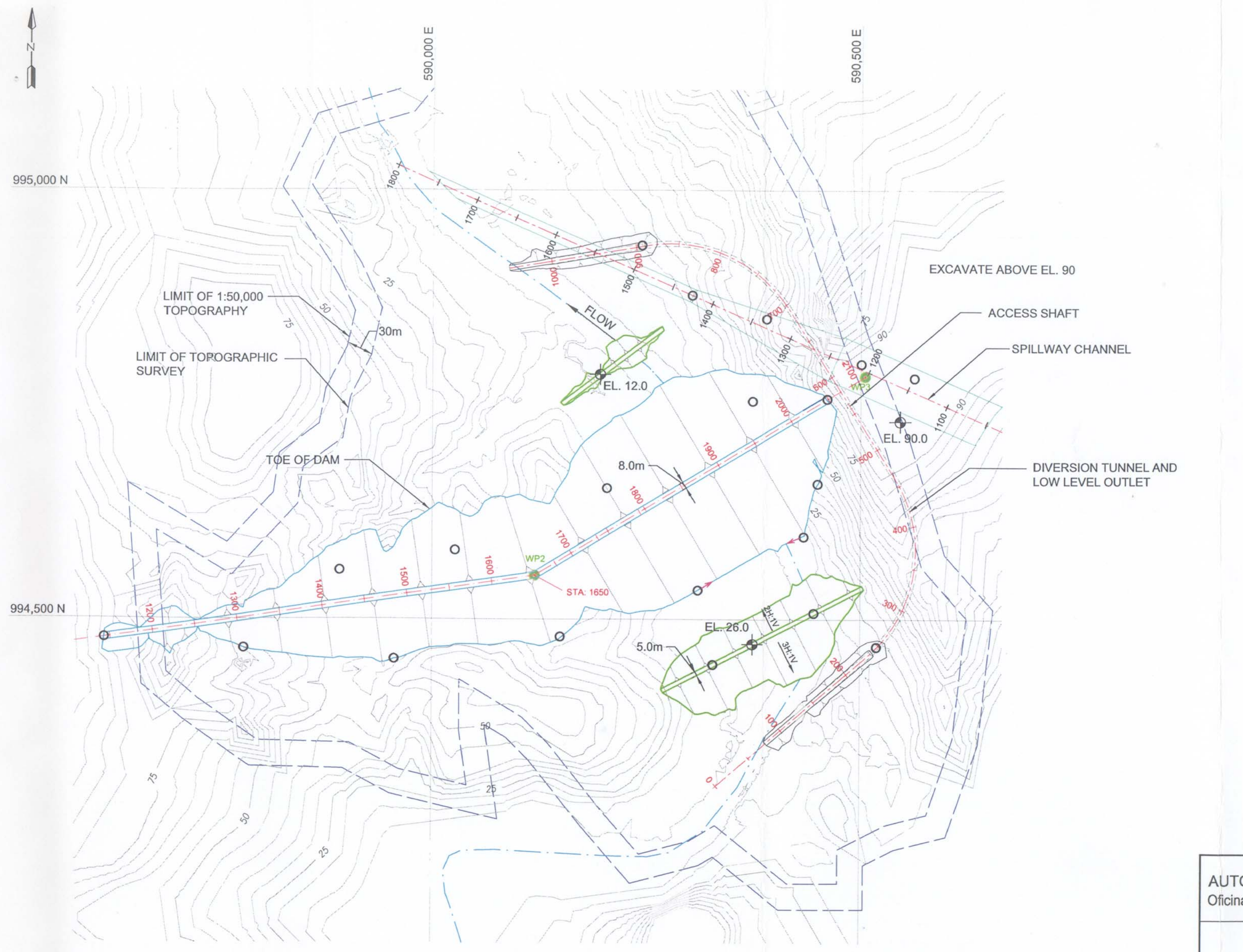
APPENDIX B - GEOLOGY AND SEISMICITY

CONSTRUCTION MATERIAL SOURCES

MWH TAMS

DATE: APRIL, 2003

EXHIBIT: 25



SITE PLAN

LEGEND:

- O PROPOSED DRILLHOLE (VERTICAL)
- O→ PROPOSED DRILLHOLE (INCLINED)

COORDINATES (UTM):

POINT	STATION	NORTH	EAST
WP1	1000	994456.9	589476.2
WP2	1650	994551.0	590119.4
WP3	2100	994782.8	590505.1

0 50 100 150 200 250 meters

GRAPHIC SCALE

AUTORIDAD DEL CANAL DE PANAMA
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APPENDIX B - GEOLOGY AND SEISMICITY

ADDITIONAL INVESTIGATIONS

MWH TAMS

DATE: APRIL, 2003

EXHIBIT: 26

ATTACHMENTS

Attachment 1 – Laboratory Test Results (from Tecnilab, S.A.)

REPUBLICA DE PANAMA

AUTORIDAD DEL CANAL DE PANAMA

INVESTIGACION PRELIMINAR DE SUELOS

PROYECTO

ABASTECIMIENTO DE AGUA DEL RIO INDIO

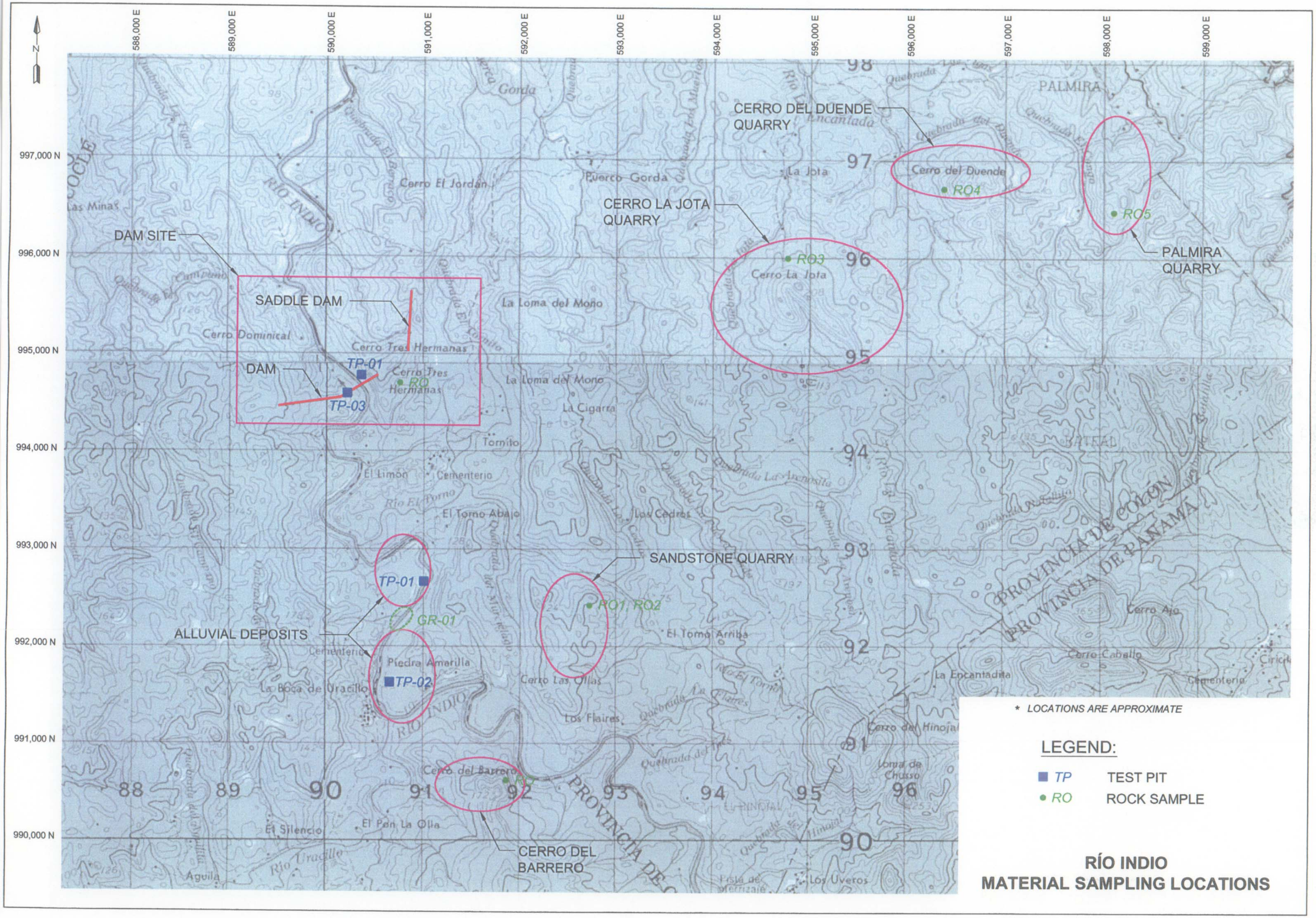
HARZA ENGINEERING COMPANY

PRESENTADO POR

TECNILAB, S. A.

LABORATORIO DE SUELOS Y MATERIALES

FEBRERO DEL 2000



RÍO INDIO
MATERIAL SAMPLING LOCATIONS

INVESTIGACIÓN DE SUELOS
ABASTECIMIENTO DE AGUA DEL RIO INDIO

DETALLES DE CALICATAS

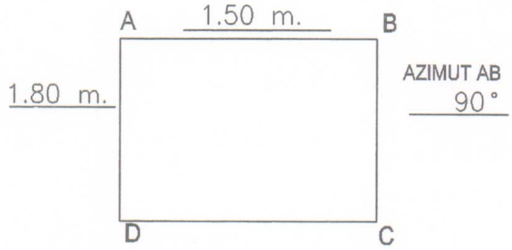
Y FOTOGRAFIAS

TECNILAB, S. A.

TECNILAB, S.A.
LABORATORIO DE SUELOS Y MATERIALES

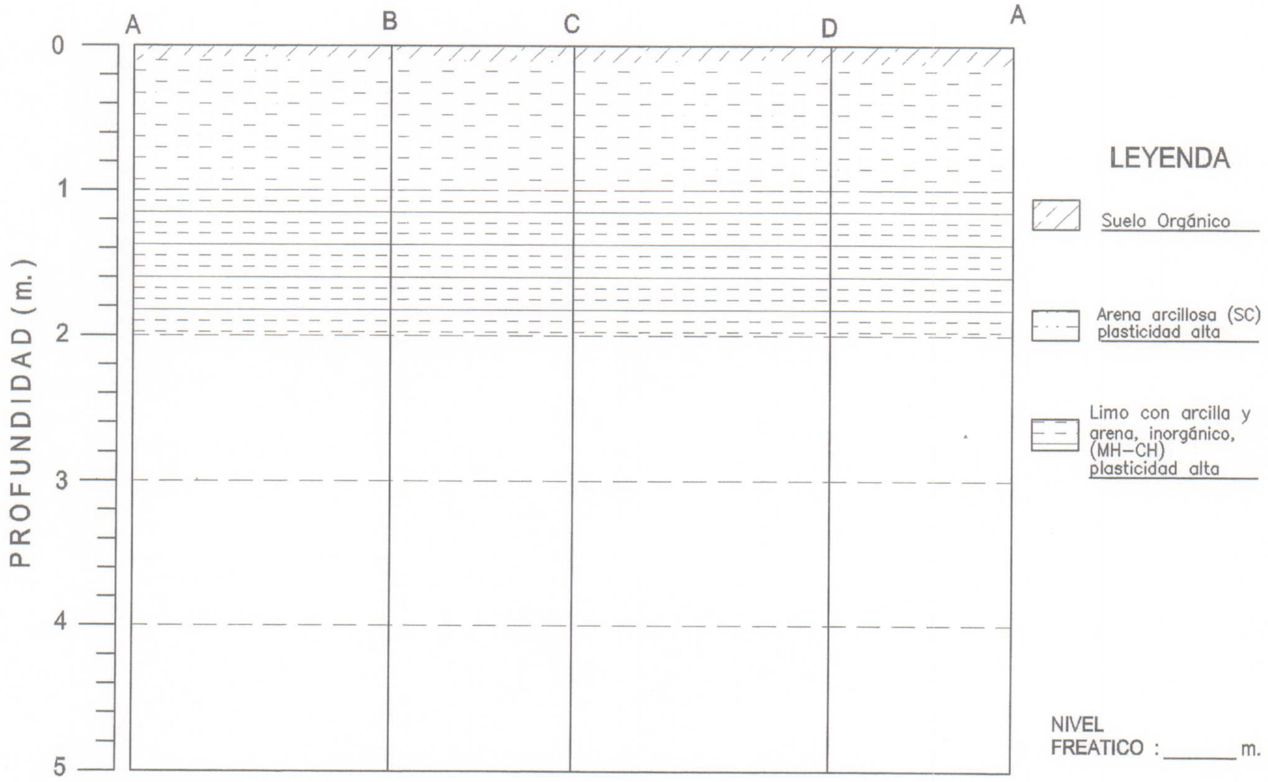
DETALLES DE CALICATA

Trabajo No. TP-01 Fecha : 25-01-00
Proyecto : 4-194 Río Indio
Cliente : HARZA ENGINEERING COMPANY



UBICACION : Aproximadamente
590270-994750

REFERENCIAS :



OBSERVACIONES : El material es color pardo. Dos muestra (TP-01-S1 [0-1m.] y y TP-01-S2 [1-2 m.]). El material es arena arcillosa de (0-1 m), y de (1-2 m) es limo con arcilla y arena (33.8%), inorgánico, de alta plasticidad.

FECHA INICIO : 25-01-00 HORA : 11:30 a.m.
TERMINACION : 25-01-00 HORA : 3:00 p.m.

RESPONSABLE

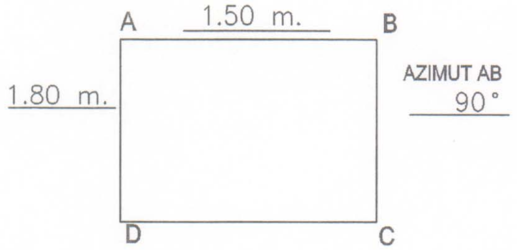
TECNILAB, S.A.
LABORATORIO DE SUELOS Y MATERIALES

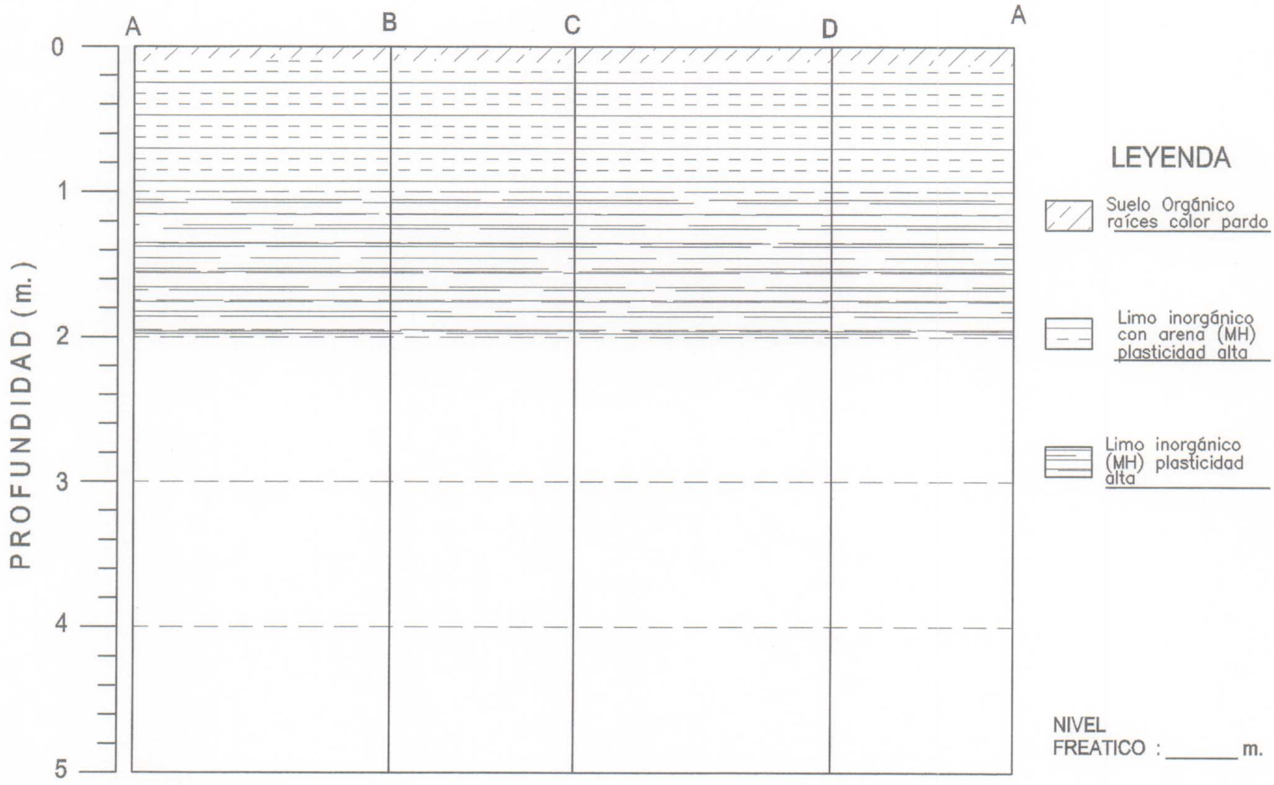
DETALLES DE CALICATA

Trabajo No. TP-02 Fecha : 26-01-00
Proyecto : 4-194 Río Indio
Cliente : HARZA ENGINEERING COMPANY

UBICACION : 590750-991600
terraza frente Boca de Uracillo

REFERENCIAS : Calicata en plan aluvional
cerca de Río Indio





OBSERVACIONES : Pocos rodados relativamente, se hace dentro de ciertos límites más compacta con la profundidad. Pero en general es el mismo materia y la variación es poca. Muestras TP-02-S1 (0-1 m.) y TP-02-S2 (1-2m.)

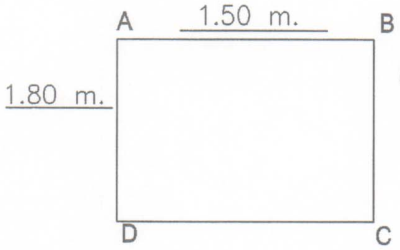
FECHA INICIO : 26-01-00 HORA : 9:30 a.m.
TERMINACION : 26-01-00 HORA : 5:00 p.m.

RESPONSABLE

TECNILAB, S.A.
LABORATORIO DE SUELOS Y MATERIALES

DETALLES DE CALICATA

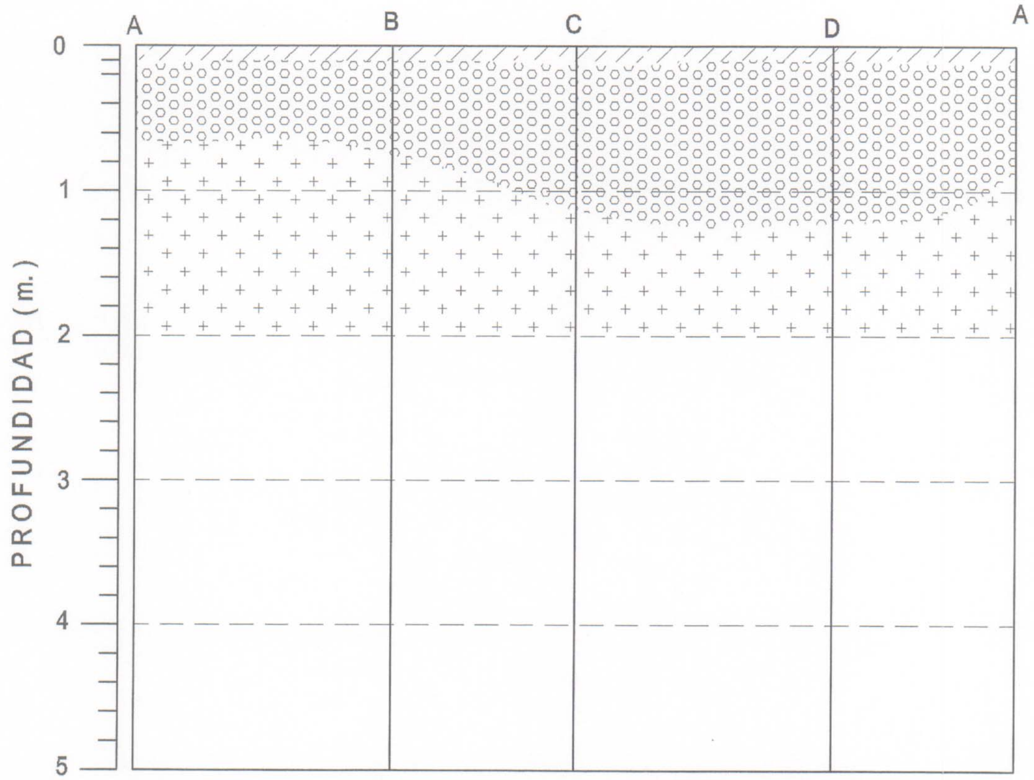
Trabajo No. TP-03 Fecha : 27-01-00
Proyecto : 4-194 Río Indio
Cliente : HARZA ENGINEERING COMPANY



AZIMUT AB 90°

UBICACION : Eje de Presa
590750-99600

REFERENCIAS : Calicata en "Cresta de Presa'
Tratar de topar roca adyacente.



LEYENDA

- Suelo Orgãnico
- Regolito color rojo
- Piedra ± fresca color verde

NIVEL FREATICO : ______ m.

OBSERVACIONES : Se trata de material de meteorización de arenisca (color rojo de hierro). A los 0.61 m. de profundidad Roca± fresca (de la cara AB). La cara CD 1.40 → roca ± fresca roja meteorizada.

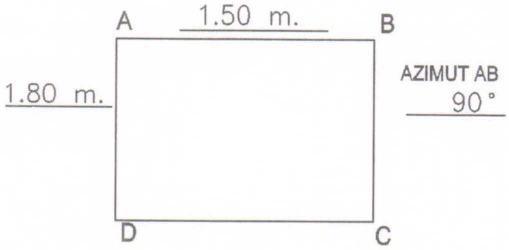
FECHA INICIO : 27-01-00 HORA : 9:30 a.m.
TERMINACION : 27-01-00 HORA : 5:00 p.m.

RESPONSABLE

TECNILAB, S.A.
LABORATORIO DE SUELOS Y MATERIALES

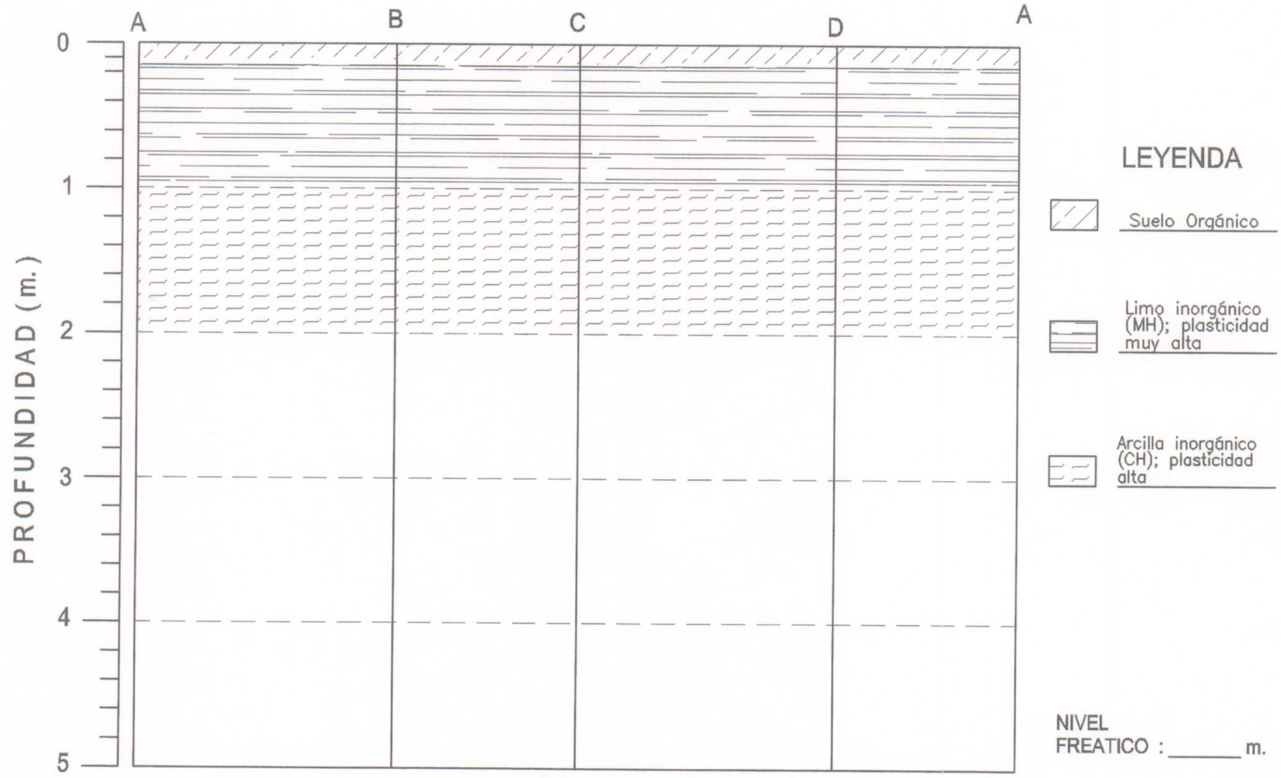
DETALLES DE CALICATA

Trabajo No. TP-04 **Fecha :** 28-01-00
Proyecto : 4-194 Río Indio
Cliente : HARZA ENGINEERING COMPANY



UBICACION : 590990-992700

REFERENCIAS : Plan aluvional, terraza + laja + nueva



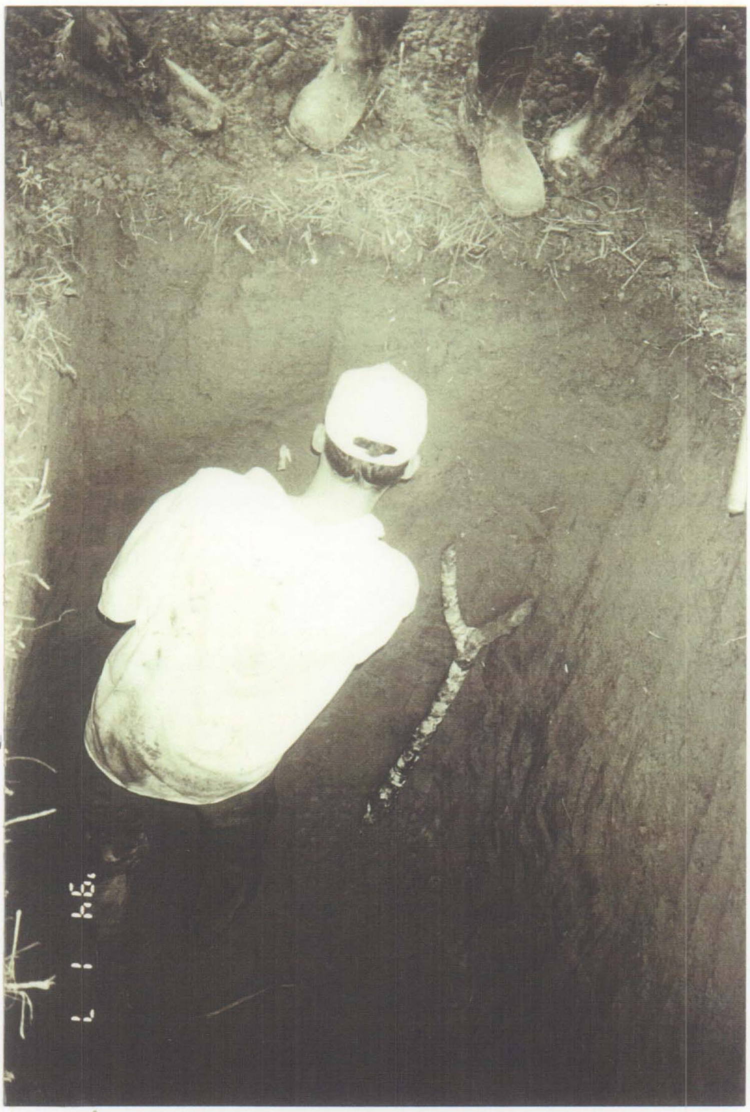
NIVEL FREATICO : _____ m.

OBSERVACIONES : Muestras TP 04-S1 (0-1 m) y TP 04-S2 (1-2 m) .

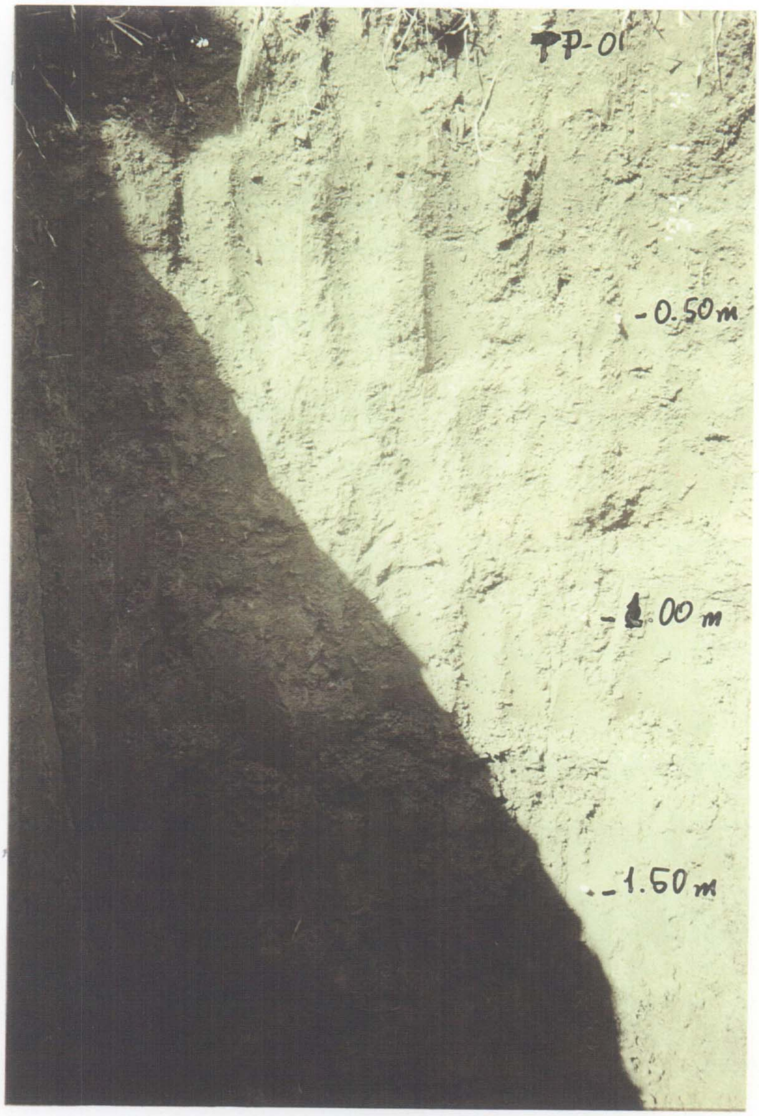
FECHA INICIO : 28-01-00 **HORA :** 9:30 a.m.
TERMINACION : 28-01-00 **HORA :** 5:00 p.m.

RESPONSABLE

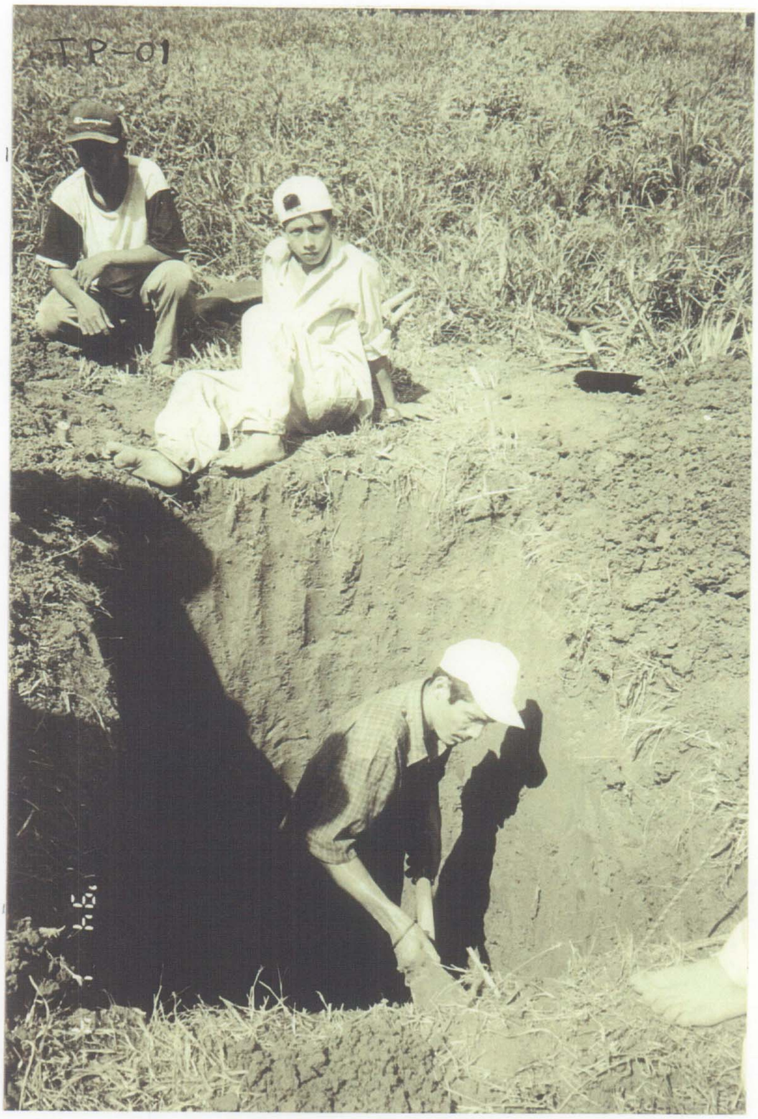
FOTOS



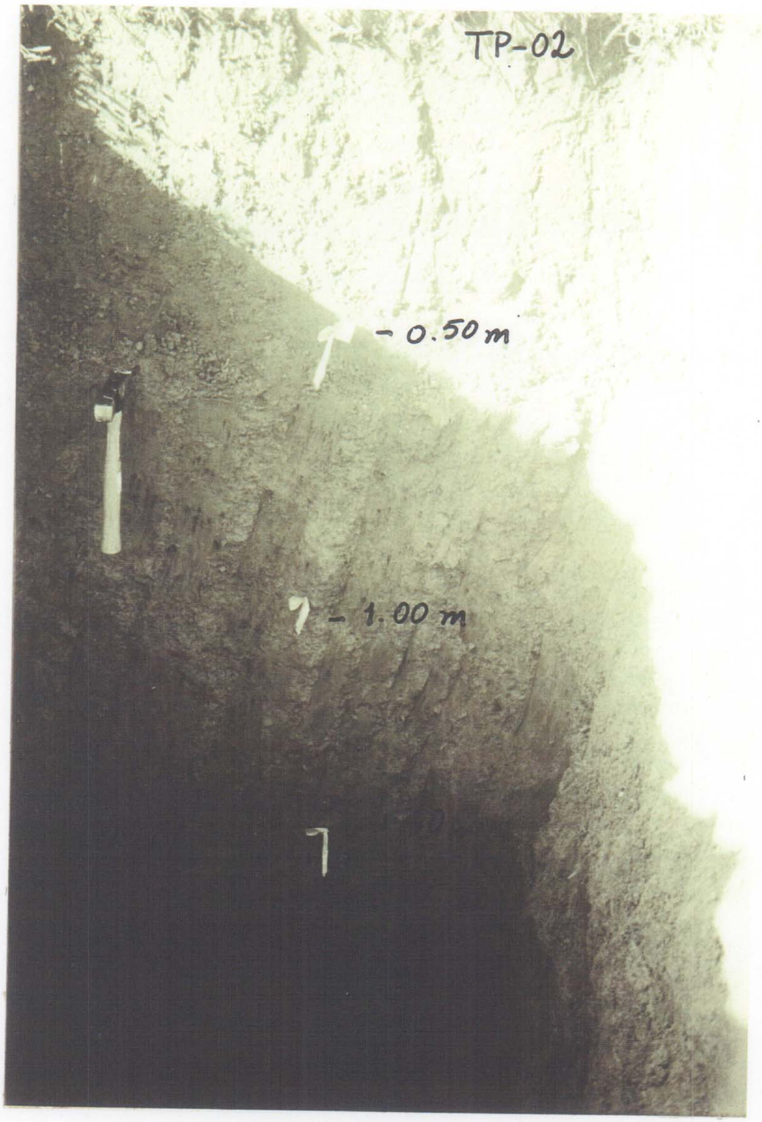
TP-04



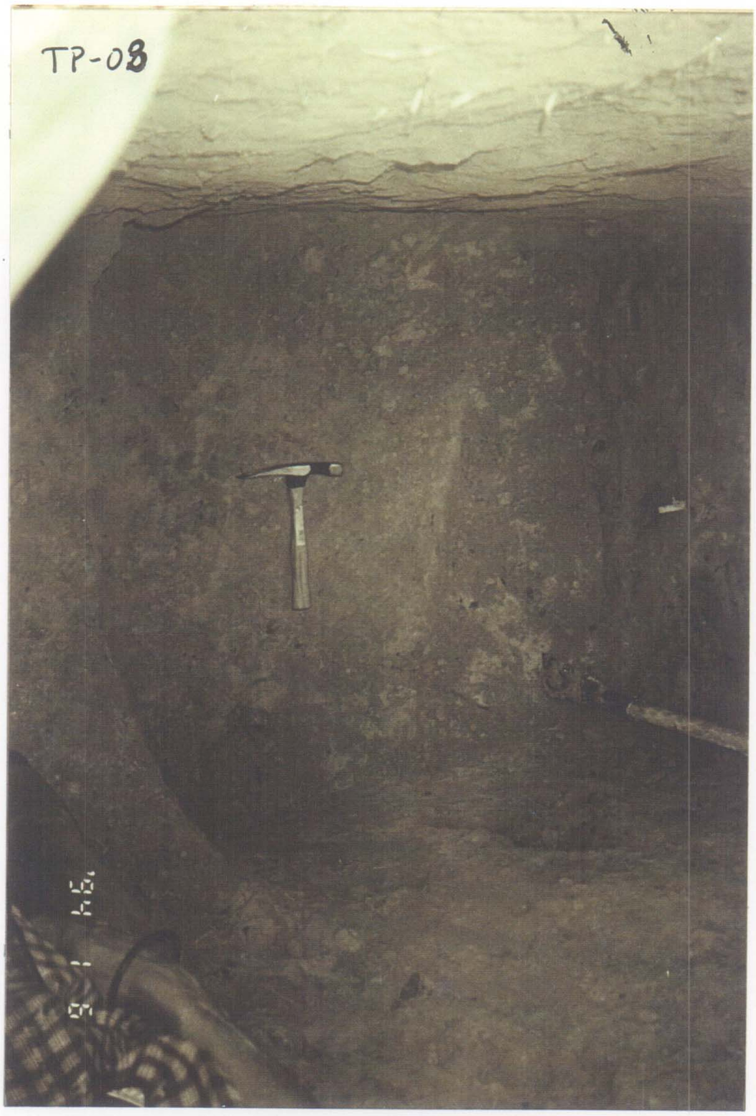
TP-01



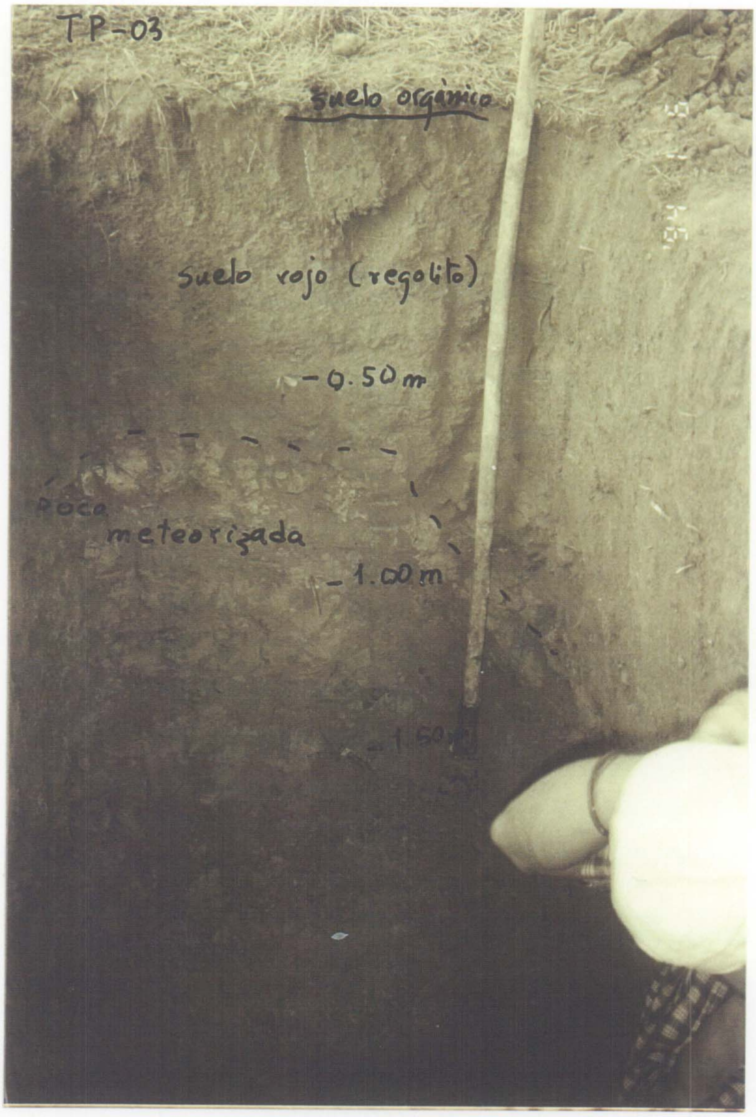
TP-01



TP-02

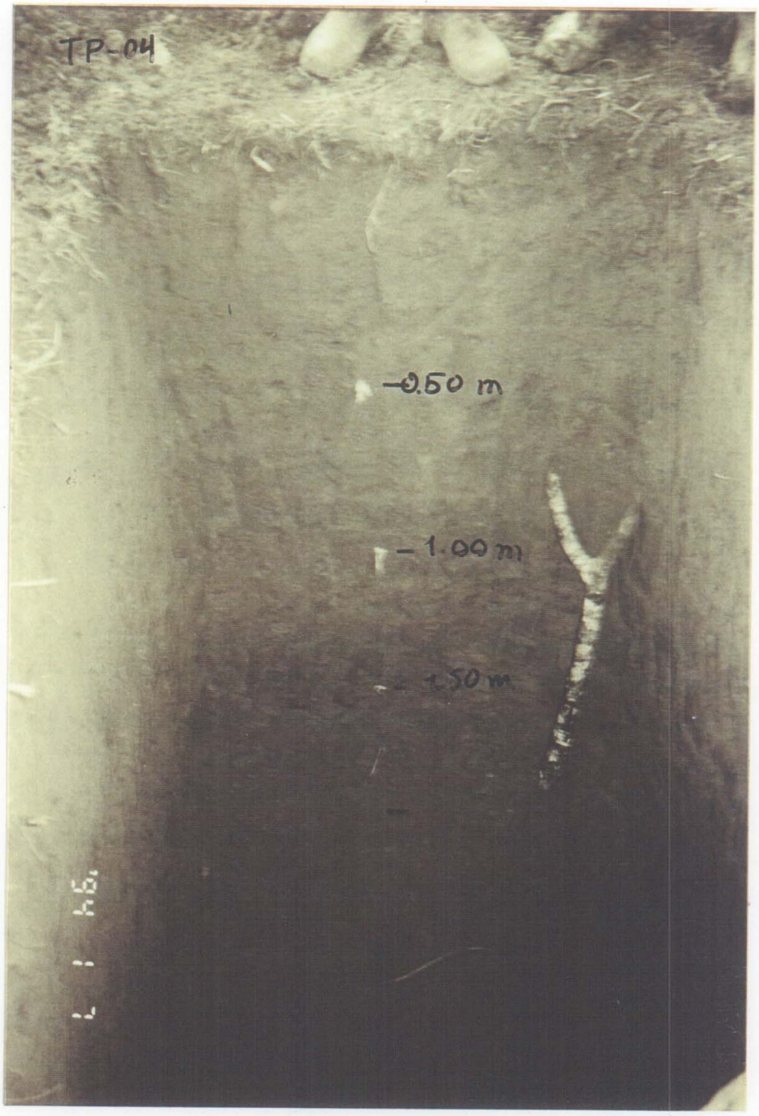


TP-03


TP-03
suelo orgánico
suelo rojo (regolito)
-0.50m
Roca meteorizada
-1.00m

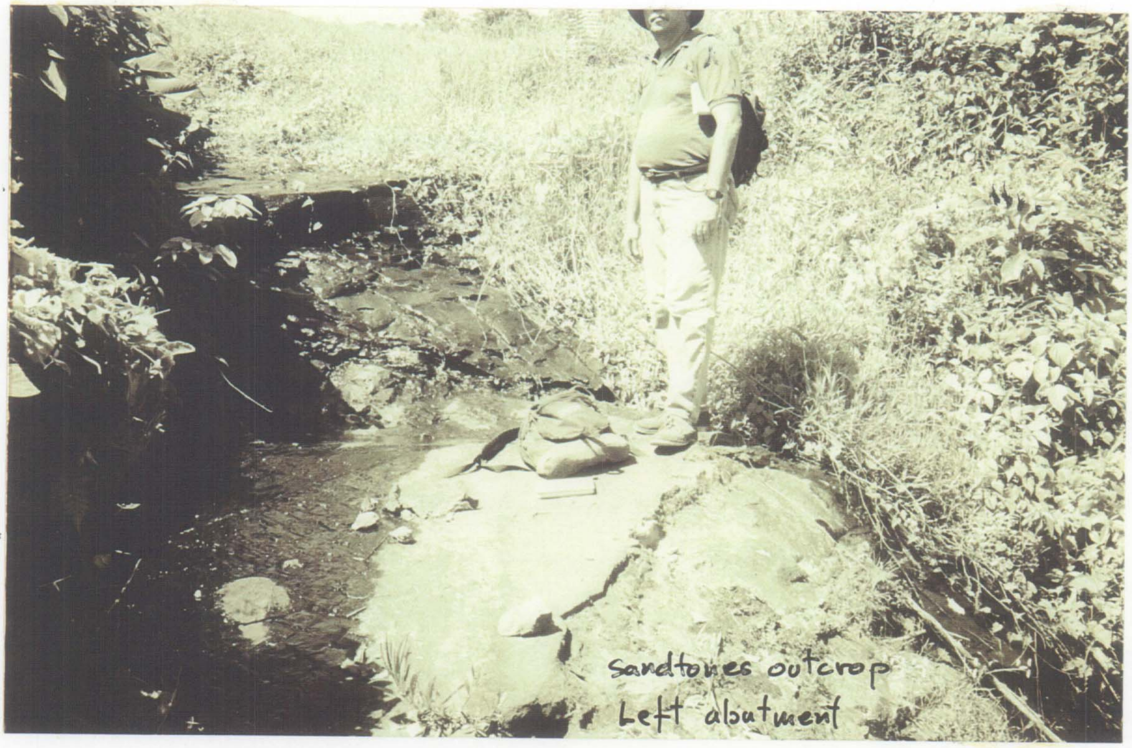
TP-03

LEFT ABUTMENT, CERRO TRES HERMANAS



TP-04

TP-04



AFLORAMIENTO DE ARENISCA CERCA DEL SITIO LA PRESA

TERRAZA DEL TP-04

TERRAZA DEL TP-04

TERRAZA DEL TP-04

TERRAZA DEL TP-04

INVESTIGACIÓN DE SUELOS
ABASTECIMIENTO DE AGUA DE RIO INDIO

RESULTADOS DE PRUEBAS REALIZADAS A MUESTRAS DE SUELOS

TECNILAB, S. A.

TECNILAB, S. A. FUNDADA EN 1973
UNA EMPRESA E. BARRANCO Y ASOC., S. A.
LABORATORIO DE SUELOS Y MATERIALES

CLASIFICACION VISUAL

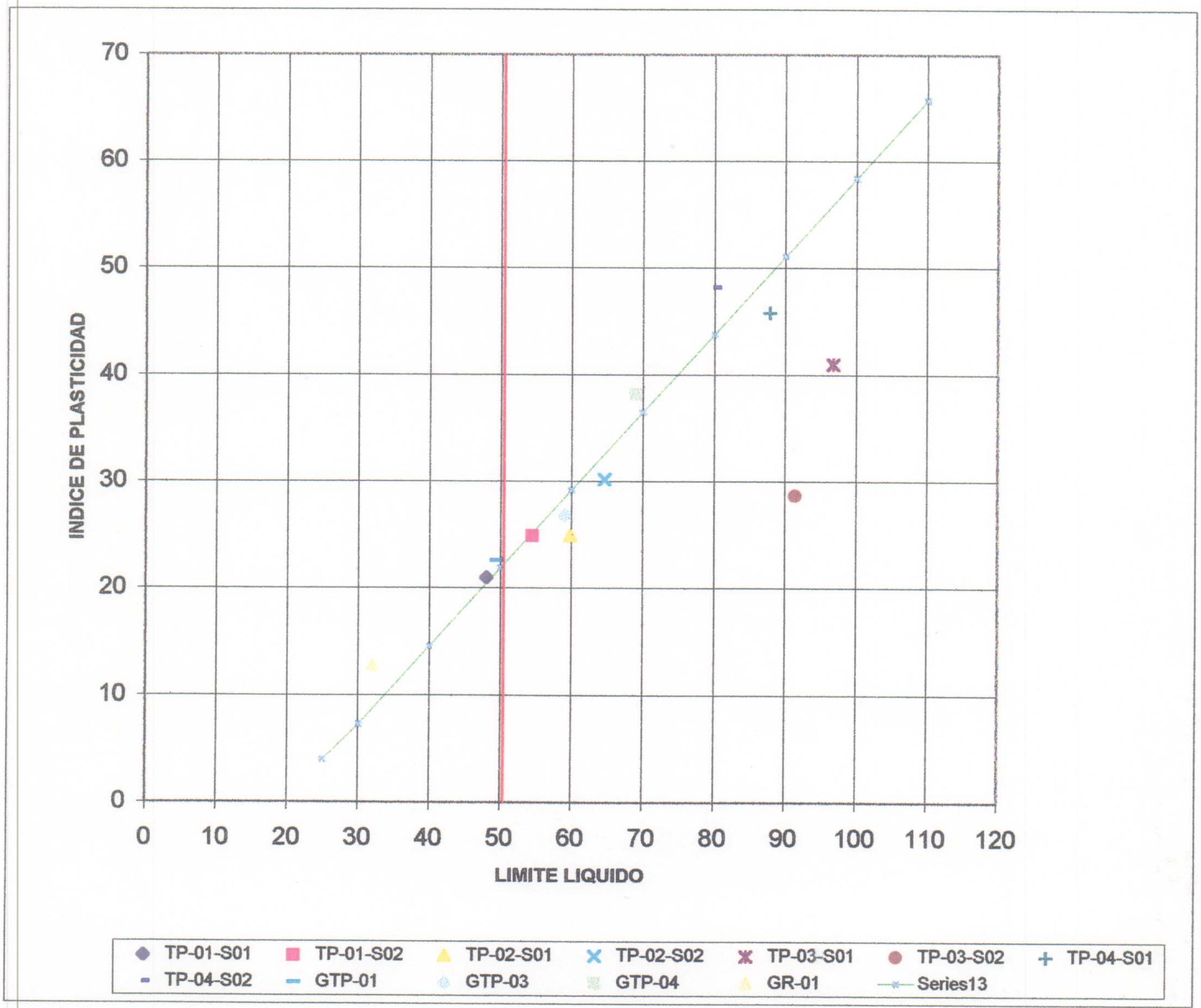
TRABAJO No. : 4-194 CLIENTE HARZA Engineering Company HUECO No. ____ MUESTRA No. 1
PROYECTO: Cuenca del Canal. LOCALIZACION Río Indio
MUESTREADO POR Tecnilab, S. A. FECHA Feb.00
PREPARADO POR Tecnilab, S. A. FECHA Feb.00 LABORATORISTA J.Vergara

1	Muestra No.	CLASIFICACION	
		VISUAL	S.U.C.S
2	TP-01-S01	Arcilla arenosa con poco limo, color café claro	SC
3	TP-01-S02	Arcilla con poco limo, color café claro	MH-CH
4	GTP-01	Limo arenoso, color café claro	SC
5	TP-02-S01	Arcilla, color café claro	MH
6	TP-02-S02	Arcilla con poco limo, color café claro	MH
7	TP-03-S01	Limo arcilloso, color café rojizo a ocre claro	MH
8	TP-03-S02	Limo granular, color ocre con vetas rojizas	MH
9	GTP-03	Limo con poca arena, contiene raices (materia orgánica), color café claro	OH
10	TP-04-S01	Arcilla con poco limo, color café claro	MH
11	TP-04-S02	Arcilla con poco limo, color café claro	CH
12	GTP-04	Arcilla con poco limo, Color café claro con ocre	CH
13	GR-01	Mezcla de grava de río con limo y arcilla, color café claro	GW-GC

TECNILAB, S. A.
FUNDADA EN 1973
UNA EMPRESA E. BARRANCO Y ASOC., S. A.
LABORATORIO DE SUELOS Y MATERIALES

GRAFICO DE LA LINEA "A"

TRABAJO No. : CLIENTE HARZA Engineering Company
PROYECTO: Cuenca del Canal (4-194 Río Indio)
LOCALIZACION Rio Indio
MUESTREADO POR Tecnilab, S. A. FECHA Feb-00
PREPARADO POR Tecnilab, S. A. FECHA Feb-00 LABORATORISTA J.V



TECNILAB, S. A.
FUNDADA EN 1973
UNA EMPRESA E. BARRANCO Y ASOC., S. A.
LABORATORIO DE SUELOS Y MATERIALES

ANALISIS MECANICO Y LIMITES DE ATTERBERG

TRABAJO No. : 4-194 CLIENTE HARZA Engineering Company MUESTRA No. TP-01-S01
PROYECTO: Cuenca del Canal. LOCALIZACION Río Indio
MUESTREADO POR TECNILAB, S.A. FECHA Feb. 00 PROFUNDIDAD
PREPARADO POR TECNILAB, S.A. FECHA Feb. 00 LABORATORISTA J. VERGARA

ANALISIS MECANICO

TAMIZ	RETENIDO ACUMULADO	% RETENIDO	% QUE PASA
3"			
$2^1/_2$"			
2"			
$^3/_4$"			
$^1/_2$"			
$^3/_8$"			
#4			
FONDO			
TOTAL			

AGREGADO GRUESO
Peso Muestra Total Secada al Aire ______ gr

TAMIZ	RETENIDO ACUMULADO	% RETENIDO	% QUE PASA
#4	0.00	0.00	100.00
#10	0.00	0.00	100.00
#40	9.00	5.10	94.90
#200	104.00	58.50	41.50
FONDO	104.20	58.60	41.40
TOTAL			

AGREGADO FINO
Peso Muestra Total Secada al Aire 220.00 gr
Peso Muestra Total Seca 177.80 gr
Peso Seco Depués de Lavado 104.20 gr

LIMITES DE ATTERBERG

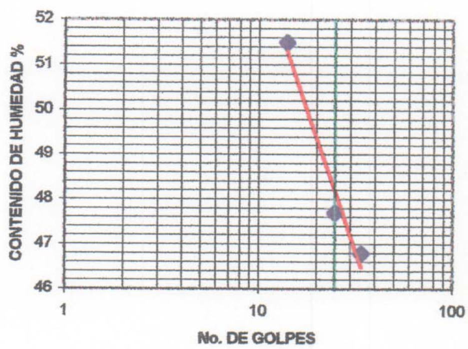
LIMITE LIQUIDO

CAPSULA No.	PESO CAPSULA (gr)	CAPSULA + SUELO HUM. (gr)	CAPSULA + SUELO SECO (gr)	AGUA (gr)	SUELO SECO (gr)	CONTENIDO DE HUMEDAD (%)	NUMERO DE GOLPES
T-10	10.70	30.70	23.90	6.80	13.20	51.50	14
T-4	10.70	30.20	23.90	6.30	13.20	47.70	25
T-2	10.50	26.50	21.40	5.10	10.90	46.80	34

LIMITE PLASTICO

CAPSULA No.	PESO CAPSULA (gr)	CAPSULA + SUELO HUM. (gr)	CAPSULA + SUELO SECO (gr)	AGUA (gr)	SUELO SECO (gr)	CONTENIDO DE HUMEDAD (%)	PROM.
25	11.80	19.70	17.90	1.80	6.10	29.50	27.30
75	11.90	19.40	17.90	1.50	6.00	25.00	

CLASIFICACION Arena arcillosa (SC), plásticidad alta, color café claro.



CLASIFICACION S.U.C.S. SC

L.L. = 48.00
L.P. = 27.30
I.p. = 21.00

CLASIFICACION A.A.S.H.T.O.
A-7-6 (IG: 4.0)

TECNILAB, S. A.
UNA EMPRESA E. BARRANCO Y ASOC., S. A.
LABORATORIO DE SUELOS Y MATERIALES
FUNDADA EN 1973

ANALISIS MECANICO Y LIMITES DE ATTERBERG

TRABAJO No. : 4-194 CLIENTE HARZA Engineering Company MUESTRA No. TP-01-S02
PROYECTO: Cuenca del Canal. LOCALIZACION Río Indio
MUESTREADO POR TECNILAB, S.A. FECHA Feb. 00 PROFUNDIDAD
PREPARADO POR TECNILAB, S.A. FECHA Feb. 00 LABORATORISTA J. VERGARA

ANALISIS MECANICO

TAMIZ	RETENIDO ACUMULADO	% RETENIDO	% QUE PASA
3"			
$2^1/_2$"			
2"			
$^3/_4$"			
$^1/_2$"			
$^3/_8$"			
#4			
FONDO			
TOTAL			

AGREGADO GRUESO
Peso Muestra Total Secada al Aire ________ gr

TAMIZ	RETENIDO ACUMULADO	% RETENIDO	% QUE PASA
#4	0.00	0.00	100.00
#10	0.00	0.00	100.00
#40	4.20	3.40	96.60
#200	41.80	33.80	66.20
FONDO	2.30	1.90	98.10
TOTAL			

AGREGADO FINO
Peso Muestra Total Secada al Aire 141.50 gr
Peso Muestra Total Seca 123.80 gr
Peso Seco Depués de Lavado 42.30 gr

LIMITES DE ATTERBERG

LIMITE LIQUIDO

CAPSULA No.	PESO CAPSULA (gr)	CAPSULA + SUELO HUM. (gr)	CAPSULA + SUELO SECO (gr)	AGUA (gr)	SUELO SECO (gr)	CONTENIDO DE HUMEDAD (%)	NUMERO DE GOLPES
T-9	4.00	23.90	16.60	7.30	12.60	57.90	16
T-3	4.20	21.90	15.50	6.40	11.30	56.60	26
T-6	4.20	23.50	17.10	6.40	12.90	49.60	33

LIMITE PLASTICO

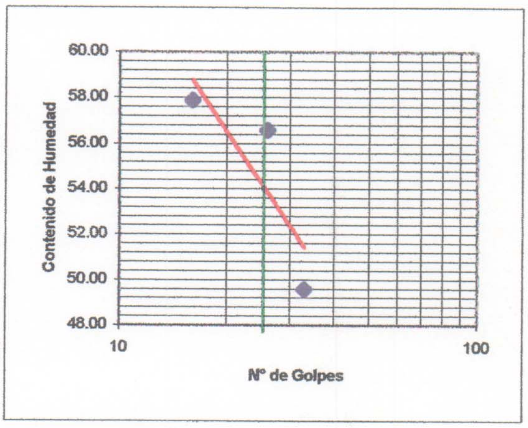
CAPSULA No.	PESO CAPSULA (gr)	CAPSULA + SUELO HUM. (gr)	CAPSULA + SUELO SECO (gr)	AGUA (gr)	SUELO SECO (gr)	CONTENIDO DE HUMEDAD (%)	PROM.
50	11.10	18.30	16.70	1.60	5.60	28.60	29.60
1	15.30	23.40	21.50	1.90	6.20	30.60	

CLASIFICACION Limo con arcilla y arena (33.80%), inorgánico, plásticidad alta, color café claro

CLASIFICACION S.U.C.S. MH-CH

L.L. = 54.5
L.P. = 29.60
I.p. = 24.90

CLASIFICACION A.A.S.H.T.O.
A-7-6 (IG: 14)



TECNILAB, S. A.

FUNDADA EN 1973

UNA EMPRESA E. BARRANCO Y ASOC., S. A.

LABORATORIO DE SUELOS Y MATERIALES

ANALISIS MECANICO Y LIMITES DE ATTERBERG

TRABAJO No. : 4-194 CLIENTE HARZA Engineering Company MUESTRA No. TP-02-S02

PROYECTO: Cuenca del Canal. LOCALIZACION Río Indio

MUESTREADO POR TECNILAB, S.A. FECHA Feb. 00 PROFUNDIDAD

PREPARADO POR TECNILAB, S.A. FECHA Feb. 00 LABORATORISTA J. VERGARA

ANALISIS MECANICO

TAMIZ	RETENIDO ACUMULADO	% RETENIDO	% QUE PASA
3"			
$2^1/_2$"			
2"			
$^3/_4$"			
$^1/_2$"			
$^3/_8$"			
#4			
FONDO			
TOTAL			

AGREGADO GRUESO

Peso Muestra Total Secada al Aire ________ gr

TAMIZ	RETENIDO ACUMULADO	% RETENIDO	% QUE PASA
#4	0.00	0.00	100.00
#10	0.50	0.30	99.70
#40	3.70	2.30	97.70
#200	23.50	14.60	85.40
FONDO	24.10		
TOTAL			

AGREGADO FINO

Peso Muestra Total Secada al Aire 170.00 gr

Peso Muestra Total Seca 160.50 gr

Peso Seco Depués de Lavado 24.10 gr

LIMITES DE ATTERBERG

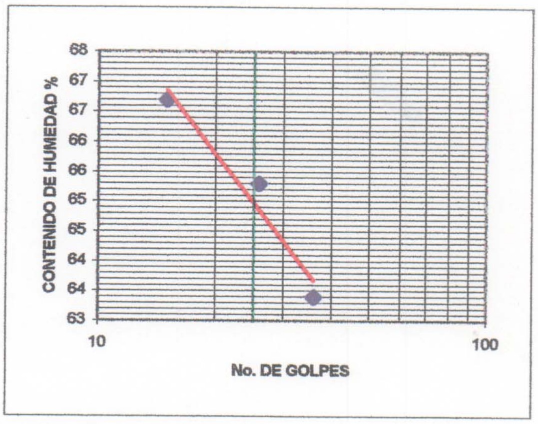
LIMITE LIQUIDO

CAPSULA No.	PESO CAPSULA (gr)	CAPSULA + SUELO HUM. (gr)	CAPSULA + SUELO SECO (gr)	AGUA (gr)	SUELO SECO (gr)	CONTENIDO DE HUMEDAD (%)	NUMERO DE GOLPES
T-4	4.50	21.50	14.70	6.80	10.20	66.70	15
T-5	4.40	23.90	16.20	7.70	11.80	65.30	26
T-1	4.50	24.60	16.80	7.80	12.30	63.40	36

LIMITE PLASTICO

CAPSULA No.	PESO CAPSULA (gr)	CAPSULA + SUELO HUM. (gr)	CAPSULA + SUELO SECO (gr)	AGUA (gr)	SUELO SECO (gr)	CONTENIDO DE HUMEDAD (%)	PROM.
1	24.40	32.60	30.50	2.10	6.10	34.40	34.40
2	26.00	35.00	32.70	2.30	6.70	34.30	

CLASIFICACION Limo inorgánico, plásticidad alta color café.



CLASIFICACION S.U.C.S. MH

L.L. = 64.60

L.P. = 34.40

I.p. = 30.20

CLASIFICACION A.A.S.H.T.O.

A-7-5 (IG: 20)

TECNILAB, S. A.
FUNDADA EN 1973
UNA EMPRESA E. BARRANCO Y ASOC., S. A.
LABORATORIO DE SUELOS Y MATERIALES

ANALISIS MECANICO Y LIMITES DE ATTERBERG

TRABAJO No. : 4-194 CLIENTE HARZA Engineering Company MUESTRA No. TP-03-S01
PROYECTO: Cuenca del Canal. LOCALIZACION Río Indio
MUESTREADO POR TECNILAB, S.A. FECHA Feb. 00 PROFUNDIDAD
PREPARADO POR TECNILAB, S.A. FECHA Feb. 00 LABORATORISTA J. VERGARA

ANALISIS MECANICO

TAMIZ	RETENIDO ACUMULADO	% RETENIDO	% QUE PASA
3"			
$2^1/_2$"			
2"			
$^3/_4$"			
$^1/_2$"			
$^3/_8$"			
#4			
FONDO			
TOTAL			

AGREGADO GRUESO
Peso Muestra Total Secada al Aire ______ gr

TAMIZ	RETENIDO ACUMULADO	% RETENIDO	% QUE PASA
#4	2.80	2.33	97.67
#10	8.30	6.89	93.11
#40	20.10	16.70	83.30
#200	37.20	30.90	69.10
FONDO	37.60	31.20	68.80
TOTAL			

AGREGADO FINO
Peso Muestra Total Secada al Aire 170.00 gr
Peso Muestra Total Seca 120.40 gr
Peso Seco Depués de Lavado 37.60 gr

LIMITES DE ATTERBERG

LIMITE LIQUIDO

CAPSULA No.	PESO CAPSULA (gr)	CAPSULA + SUELO HUM. (gr)	CAPSULA + SUELO SECO (gr)	AGUA (gr)	SUELO SECO (gr)	CONTENIDO DE HUMEDAD (%)	NUMERO DE GOLPES
T-1	4.00	21.10	12.50	8.60	8.50	101.20	14
T-11	4.10	20.60	12.50	8.10	8.40	96.40	25
T-7	4.10	20.10	12.30	7.80	8.20	95.10	35

LIMITE PLASTICO

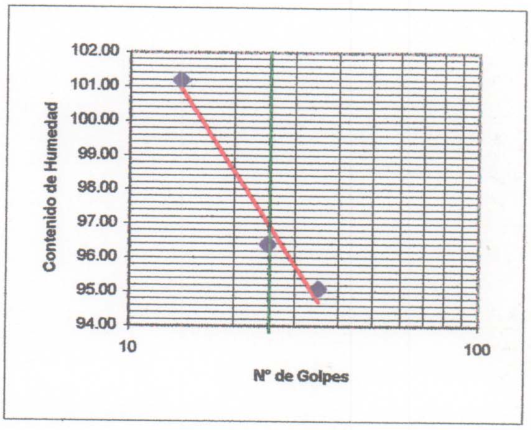
CAPSULA No.	PESO CAPSULA (gr)	CAPSULA + SUELO HUM. (gr)	CAPSULA + SUELO SECO (gr)	AGUA (gr)	SUELO SECO (gr)	CONTENIDO DE HUMEDAD (%)	PROM.
50	25.00	31.00	28.90	2.10	3.90	53.80	55.70
27	25.90	32.20	29.90	2.30	4.00	57.50	

CLASIFICACION Limo inorgánico, plásticidad alta, color rojizo a ocre claro

CLASIFICACION S.U.C.S. MH

L.L. = 96.70
L.P. = 55.70
I.p. = 41.00

CLASIFICACION A.A.S.H.T.O.
A-7-5 (I.G: 18)



TECNILAB, S. A.
FUNDADA EN 1973
UNA EMPRESA E. BARRANCO Y ASOC., S. A.
LABORATORIO DE SUELOS Y MATERIALES

ANALISIS MECANICO Y LIMITES DE ATTERBERG

TRABAJO No. : 4-194 CLIENTE HARZA Engineering Company MUESTRA No. TP-03-S02
PROYECTO: Cuenca del Canal. LOCALIZACION Río Indio
MUESTREADO POR TECNILAB, S.A. FECHA Feb. 00 PROFUNDIDAD
PREPARADO POR TECNILAB, S.A. FECHA Feb. 00 LABORATORISTA J. VERGARA

ANALISIS MECANICO

TAMIZ	RETENIDO ACUMULADO	% RETENIDO	% QUE PASA
3"			
$2^1/_2$"			
2"			
$^3/_4$"			
$^1/_2$"			
$^3/_8$"			
#4			
FONDO			
TOTAL			

AGREGADO GRUESO
Peso Muestra Total Secada al Aire ________ gr

TAMIZ	RETENIDO ACUMULADO	% RETENIDO	% QUE PASA
#4	9.60	4.90	95.10
#10	27.20	13.90	86.10
#40	47.80	24.50	75.50
#200	69.30	35.40	64.60
FONDO	70.00		
TOTAL			

AGREGADO FINO
Peso Muestra Total Secada al Aire 250.00 gr
Peso Muestra Total Seca 195.50 gr
Peso Seco Depués de Lavado 70.00 gr

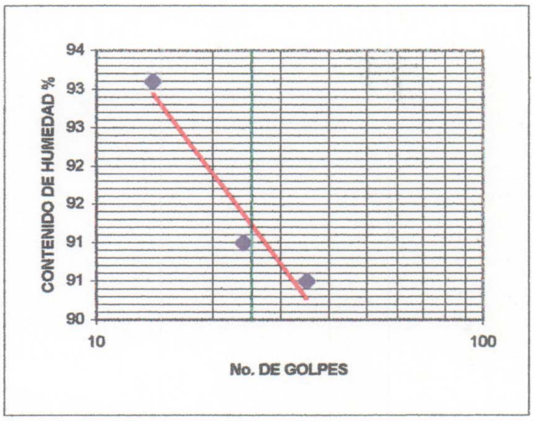
LIMITES DE ATTERBERG

LIMITE LIQUIDO

CAPSULA No.	PESO CAPSULA (gr)	CAPSULA + SUELO HUM. (gr)	CAPSULA + SUELO SECO (gr)	AGUA (gr)	SUELO SECO (gr)	CONTENIDO DE HUMEDAD (%)	NUMERO DE GOLPES
48	11.10	30.80	21.30	9.50	10.20	93.10	14
T-8	4.50	21.50	13.40	8.10	8.90	91.00	24
T-12	4.00	18.10	11.40	6.70	7.40	90.50	35

LIMITE PLASTICO

CAPSULA No.	PESO CAPSULA (gr)	CAPSULA + SUELO HUM. (gr)	CAPSULA + SUELO SECO (gr)	AGUA (gr)	SUELO SECO (gr)	CONTENIDO DE HUMEDAD (%)	PROM.
76	25.70	32.00	29.60	2.40	3.90	61.50	62.70
24	26.30	32.20	29.90	2.30	3.60	63.90	



CLASIFICACION Limo inorganico con arena(35.40%), plásticidad alta, color ocre con vetas rojizas

CLASIFICACION S.U.C.S. MH

L.L. = 91.40
L.P. = 62.70
I.p. = 28.70

CLASIFICACION A.A.S.H.T.O.
A-7-5 (IG: 16)

TECNILAB, S. A.

FUNDADA EN 1973

UNA EMPRESA E. BARRANCO Y ASOC., S. A.

LABORATORIO DE SUELOS Y MATERIALES

ANALISIS MECANICO Y LIMITES DE ATTERBERG

TRABAJO No. : 4-194 CLIENTE HARZA Engineering Company MUESTRA No. GTP-03

PROYECTO: Cuenca del Canal. LOCALIZACION Río Indio

MUESTREADO POR TECNILAB, S.A. FECHA Feb. 00 PROFUNDIDAD

PREPARADO POR TECNILAB, S.A. FECHA Feb. 00 LABORATORISTA J. VERGARA

ANALISIS MECANICO

TAMIZ	RETENIDO ACUMULADO	% RETENIDO	% QUE PASA
3"			
$2^1/_2$"			
2"			
$^3/_4$"			
$^1/_2$"			
$^3/_8$"			
#4			
FONDO			
TOTAL			

AGREGADO GRUESO

Peso Muestra Total Secada al Aire ________ gr

TAMIZ	RETENIDO ACUMULADO	% RETENIDO	% QUE PASA
#4	0.00	0.00	100.00
#10	0.00	0.00	100.00
#40	3.40	2.70	97.30
#200	38.00	30.10	69.90
FONDO	38.30		
TOTAL			

AGREGADO FINO

Peso Muestra Total Secada al Aire 170.00 gr

Peso Muestra Total Seca 126.30 gr

Peso Seco Depués de Lavado 38.30 gr

LIMITES DE ATTERBERG

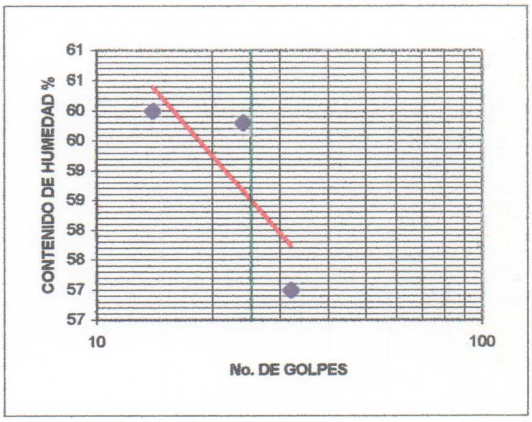
LIMITE LIQUIDO

CAPSULA No.	PESO CAPSULA (gr)	CAPSULA + SUELO HUM. (gr)	CAPSULA + SUELO SECO (gr)	AGUA (gr)	SUELO SECO (gr)	CONTENIDO DE HUMEDAD (%)	NUMERO DE GOLPES
51	12.00	31.20	24.00	7.20	12.00	60.00	14
18	11.40	30.10	23.10	7.00	11.70	59.80	24
54	11.10	29.00	22.50	6.50	11.40	57.00	32

LIMITE PLASTICO

CAPSULA No.	PESO CAPSULA (gr)	CAPSULA + SUELO HUM. (gr)	CAPSULA + SUELO SECO (gr)	AGUA (gr)	SUELO SECO (gr)	CONTENIDO DE HUMEDAD (%)	PROM.
51	25.40	32.80	31.00	1.80	5.60	32.10	32.20
75	26.50	34.70	32.70	2.00	6.20	32.30	

CLASIFICACION Limo orgánico, plásticidad alta, color café claro



CLASIFICACION S.U.C.S. OH

L.L. = 59.00

L.P. = 32.20

I.p. = 26.80

CLASIFICACION A.A.S.H.T.O.

A-7-5 (IG: 17)

FUNDADA EN 1973

TECNILAB, S. A.
UNA EMPRESA E. BARRANCO Y ASOC., S. A.
LABORATORIO DE SUELOS Y MATERIALES

ANALISIS MECANICO Y LIMITES DE ATTERBERG

TRABAJO No. : 4-194 CLIENTE HARZA Engineering Company MUESTRA No. TP-04-S01
PROYECTO: Cuenca del Canal. LOCALIZACION Río Indio
MUESTREADO POR TECNILAB, S.A. FECHA Feb. 00 PROFUNDIDAD
PREPARADO POR TECNILAB, S.A. FECHA Feb. 00 LABORATORISTA J. VERGARA

ANALISIS MECANICO

TAMIZ	RETENIDO ACUMULADO	% RETENIDO	% QUE PASA
3"			
$2^1/_2$"			
2"			
$^3/_4$"			
$^1/_2$"			
$^3/_8$"			
#4			
FONDO			
TOTAL			

AGREGADO GRUESO
Peso Muestra Total Secada al Aire ________ gr

TAMIZ	RETENIDO ACUMULADO	% RETENIDO	% QUE PASA
#4	4.30	2.80	97.20
#10	12.40	8.20	91.80
#40	22.40	14.80	85.20
#200	33.90	22.30	77.70
FONDO	34.40		
TOTAL			

AGREGADO FINO
Peso Muestra Total Secada al Aire 170.00 gr
Peso Muestra Total Seca 151.80 gr
Peso Seco Depués de Lavado 34.40 gr

LIMITES DE ATTERBERG

LIMITE LIQUIDO

CAPSULA No.	PESO CAPSULA (gr)	CAPSULA + SUELO HUM. (gr)	CAPSULA + SUELO SECO (gr)	AGUA (gr)	SUELO SECO (gr)	CONTENIDO DE HUMEDAD (%)	NUMERO DE GOLPES
T-6	10.60	32.90	22.20	10.70	11.60	92.20	15
T-3	10.60	31.30	21.70	9.60	11.10	86.50	26
T-2	10.50	30.70	21.40	9.30	10.90	85.30	37

LIMITE PLASTICO

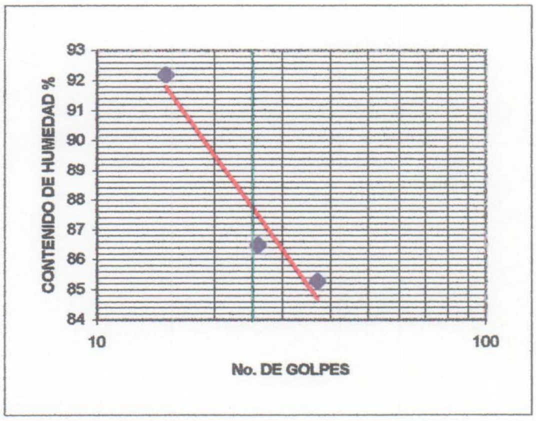
CAPSULA No.	PESO CAPSULA (gr)	CAPSULA + SUELO HUM. (gr)	CAPSULA + SUELO SECO (gr)	AGUA (gr)	SUELO SECO (gr)	CONTENIDO DE HUMEDAD (%)	PROM.
75	11.90	20.20	17.90	2.30	6.00	38.30	42.00
25	11.80	18.50	16.40	2.10	4.60	45.70	

CLASIFICACION Limo inorgánico con arena(22.30%), plásticidad muy alta, color café claro

CLASIFICACION S.U.C.S. MH

L.L. = 87.80
L.P. = 42.00
I.p. = 45.80

CLASIFICACION A.A.S.H.T.O.
A-7-5 (IG: 20)



TECNILAB, S. A.

FUNDADA EN 1973

UNA EMPRESA E. BARRANCO Y ASOC., S. A.

LABORATORIO DE SUELOS Y MATERIALES

ANALISIS MECANICO Y LIMITES DE ATTERBERG

TRABAJO No. : 4-194 CLIENTE HARZA Engineering Company MUESTRA No. TP-04-S02

PROYECTO: Cuenca del Canal. LOCALIZACION Río Indio

MUESTREADO POR TECNILAB, S.A. FECHA Feb. 00 PROFUNDIDAD

PREPARADO POR TECNILAB, S.A. FECHA Feb. 00 LABORATORISTA J. VERGARA

ANALISIS MECANICO

TAMIZ	RETENIDO ACUMULADO	% RETENIDO	% QUE PASA
3"			
$2^1/_2$"			
2"			
$^3/_4$"			
$^1/_2$"			
$^3/_8$"			
#4			
FONDO			
TOTAL			

AGREGADO GRUESO

Peso Muestra Total Secada al Aire ________ gr

TAMIZ	RETENIDO ACUMULADO	% RETENIDO	% QUE PASA
#4	0.00	0.00	100.00
#10	0.00	0.00	100.00
#40	0.50	0.40	99.60
#200	8.20	5.90	94.10
FONDO	8.40		
TOTAL			

AGREGADO FINO

Peso Muestra Total Secada al Aire 170.00 gr

Peso Muestra Total Seca 138.50 gr

Peso Seco Depués de Lavado 8.40 gr

LIMITES DE ATTERBERG

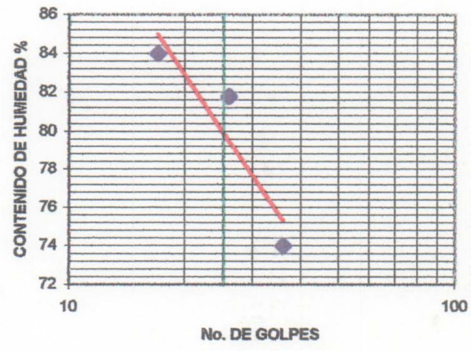
LIMITE LIQUIDO

CAPSULA No.	PESO CAPSULA (gr)	CAPSULA + SUELO HUM. (gr)	CAPSULA + SUELO SECO (gr)	AGUA (gr)	SUELO SECO (gr)	CONTENIDO DE HUMEDAD (%)	NUMERO DE GOLPES
T-5	10.90	35.00	24.00	11.00	13.10	84.00	17
T-10	10.70	32.70	22.80	9.90	12.10	81.80	26
T-10	10.70	32.10	23.00	9.10	12.30	74.00	36

LIMITE PLASTICO

CAPSULA No.	PESO CAPSULA (gr)	CAPSULA + SUELO HUM. (gr)	CAPSULA + SUELO SECO (gr)	AGUA (gr)	SUELO SECO (gr)	CONTENIDO DE HUMEDAD (%)	PROM.
51	12.00	16.80	15.60	1.20	3.60	33.30	31.80
18	11.40	15.70	14.70	1.00	3.30	30.30	

CLASIFICACION Arcilla inorgánica , plásticidad alta color café claro.



CLASIFICACION S.U.C.S. CH

L.L. = 80.00

L.P. = 31.80

I.p. = 48.20

CLASIFICACION A.A.S.H.T.O.

A-7-5 (IG: 20)

TECNILAB, S. A. FUNDADA EN 1973
UNA EMPRESA E. BARRANCO Y ASOC., S. A.
LABORATORIO DE SUELOS Y MATERIALES

ANALISIS MECANICO Y LIMITES DE ATTERBERG

TRABAJO No. : 4-194 CLIENTE HARZA Engineering Company MUESTRA No. GTP-04
PROYECTO: Cuenca del Canal. LOCALIZACION Río Indio
MUESTREADO POR TECNILAB, S.A. FECHA Feb. 00 PROFUNDIDAD
PREPARADO POR TECNILAB, S.A. FECHA Feb. 00 LABORATORISTA J. VERGARA

ANALISIS MECANICO

TAMIZ	RETENIDO ACUMULADO	% RETENIDO	% QUE PASA
3"			
$2^1/_2$"			
2"			
$^3/_4$"			
$^1/_2$"			
$^3/_8$"			
#4			
FONDO			
TOTAL			

AGREGADO GRUESO
Peso Muestra Total Secada al Aire ________ gr

TAMIZ	RETENIDO ACUMULADO	% RETENIDO	% QUE PASA
#4	0.00	0.00	100.00
#10	2.90	2.10	97.90
#40	15.80	11.60	88.40
#200	37.80	27.80	72.20
FONDO	38.00		
TOTAL			

AGREGADO FINO
Peso Muestra Total Secada al Aire 170.00 gr
Peso Muestra Total Seca 135.80 gr
Peso Seco Depués de Lavado 38.00 gr

LIMITES DE ATTERBERG

LIMITE LIQUIDO

CAPSULA No.	PESO CAPSULA (gr)	CAPSULA + SUELO HUM. (gr)	CAPSULA + SUELO SECO (gr)	AGUA (gr)	SUELO SECO (gr)	CONTENIDO DE HUMEDAD (%)	NUMERO DE GOLPES
T-7	10.70	36.70	25.80	10.90	15.10	72.20	13
T-3	10.60	34.20	24.50	9.70	13.90	69.80	24
T-4	10.70	32.70	23.90	8.80	13.20	66.70	36

LIMITE PLASTICO

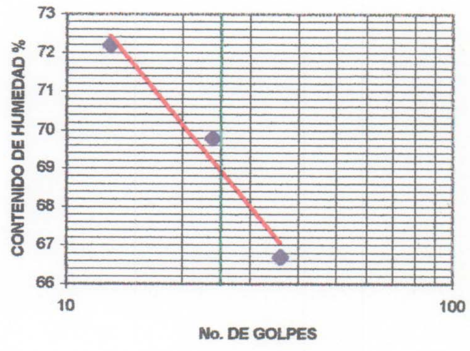
CAPSULA No.	PESO CAPSULA (gr)	CAPSULA + SUELO HUM. (gr)	CAPSULA + SUELO SECO (gr)	AGUA (gr)	SUELO SECO (gr)	CONTENIDO DE HUMEDAD (%)	PROM.
T-9	4.00	12.50	10.40	2.10	6.40	32.80	30.80
T-1	4.00	11.60	9.90	1.70	5.90	28.80	

CLASIFICACION Arcilla inorgánica con arena (27.80%), plásticidad alta, color café claro.

CLASIFICACION S.U.C.S. CH

L.L. = 69.00
L.P. = 30.80
I.p. = 38.20

CLASIFICACION A.A.S.H.T.O.
A-7-5 (IG: 19)



TECNILAB, S. A.
FUNDADA EN 1973
UNA EMPRESA E. BARRANCO Y ASOC., S. A.
LABORATORIO DE SUELOS Y MATERIALES

ANALISIS MECANICO Y LIMITES DE ATTERBERG

TRABAJO No. : 4-194 CLIENTE HARZA Engineering Company MUESTRA No. GR-01
PROYECTO: Cuenca del Canal. LOCALIZACION Río Indio
MUESTREADO POR TECNILAB, S.A. FECHA Feb. 00 PROFUNDIDAD
PREPARADO POR TECNILAB, S.A. FECHA Feb. 00 LABORATORISTA J. VERGARA

ANALISIS MECANICO

TAMIZ	RETENIDO ACUMULADO	% RETENIDO	% QUE PASA
$2^1/_2$"			
2"	0.00		
11/2"	131.00	4.00	96.00
1"	1,220.00	37.00	63.00
$^1/_2$"	1,361.00	41.30	58.70
$^3/_8$"	1,673.00	50.80	49.20
#4	1,760.00	53.40	46.60
FONDO	1,991.00	60.50	39.50
TOTAL			

AGREGADO GRUESO
Peso Muestra Total Secada al Aire 3,293.00 gr

TAMIZ	RETENIDO ACUMULADO	% RETENIDO	% QUE PASA	%CORR. QUE PASA
#4	19.91	4.90	95.10	39.50
#10	38.20	9.40	90.60	35.80
#40	188.50	46.30	53.70	21.20
#200	314.30	77.10	22.90	9.00
FONDO	314.80			
TOTAL				

AGREGADO FINO
Peso Muestra Total Secada al Aire 170.00 gr
Peso Muestra Total Seca 407.40 gr
Peso Seco Depués de Lavado 314.80 gr

LIMITES DE ATTERBERG

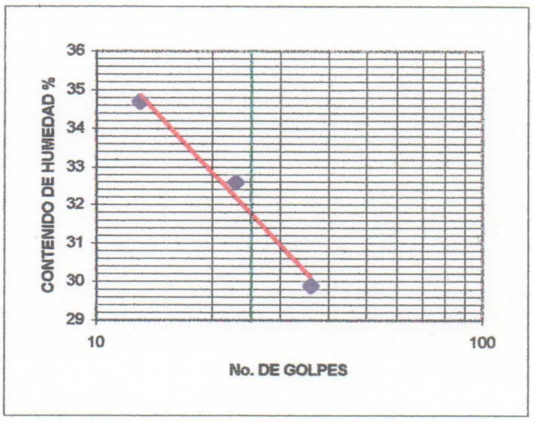
LIMITE LIQUIDO

CAPSULA No.	PESO CAPSULA (gr)	CAPSULA + SUELO HUM. (gr)	CAPSULA + SUELO SECO (gr)	AGUA (gr)	SUELO SECO (gr)	CONTENIDO DE HUMEDAD (%)	NUMERO DE GOLPES
T-8	4.50	31.30	24.40	6.90	19.90	34.70	13
T-11	4.10	29.30	23.10	6.20	19.00	32.60	23
T-4	10.80	29.50	25.20	4.30	14.40	29.90	36

LIMITE PLASTICO

CAPSULA No.	PESO CAPSULA (gr)	CAPSULA + SUELO HUM. (gr)	CAPSULA + SUELO SECO (gr)	AGUA (gr)	SUELO SECO (gr)	CONTENIDO DE HUMEDAD (%)	PROM.
48	11.10	24.90	22.60	2.30	11.50	20.00	19.20
54	11.10	24.60	22.50	2.10	11.40	18.40	

CLASIFICACION Grava mal graduada con limo, de plásticidad baja.



CLASIFICACION S.U.C.S. GW-GC

L.L. = 31.80
L.P. = 19.20
I.p. = 12.60

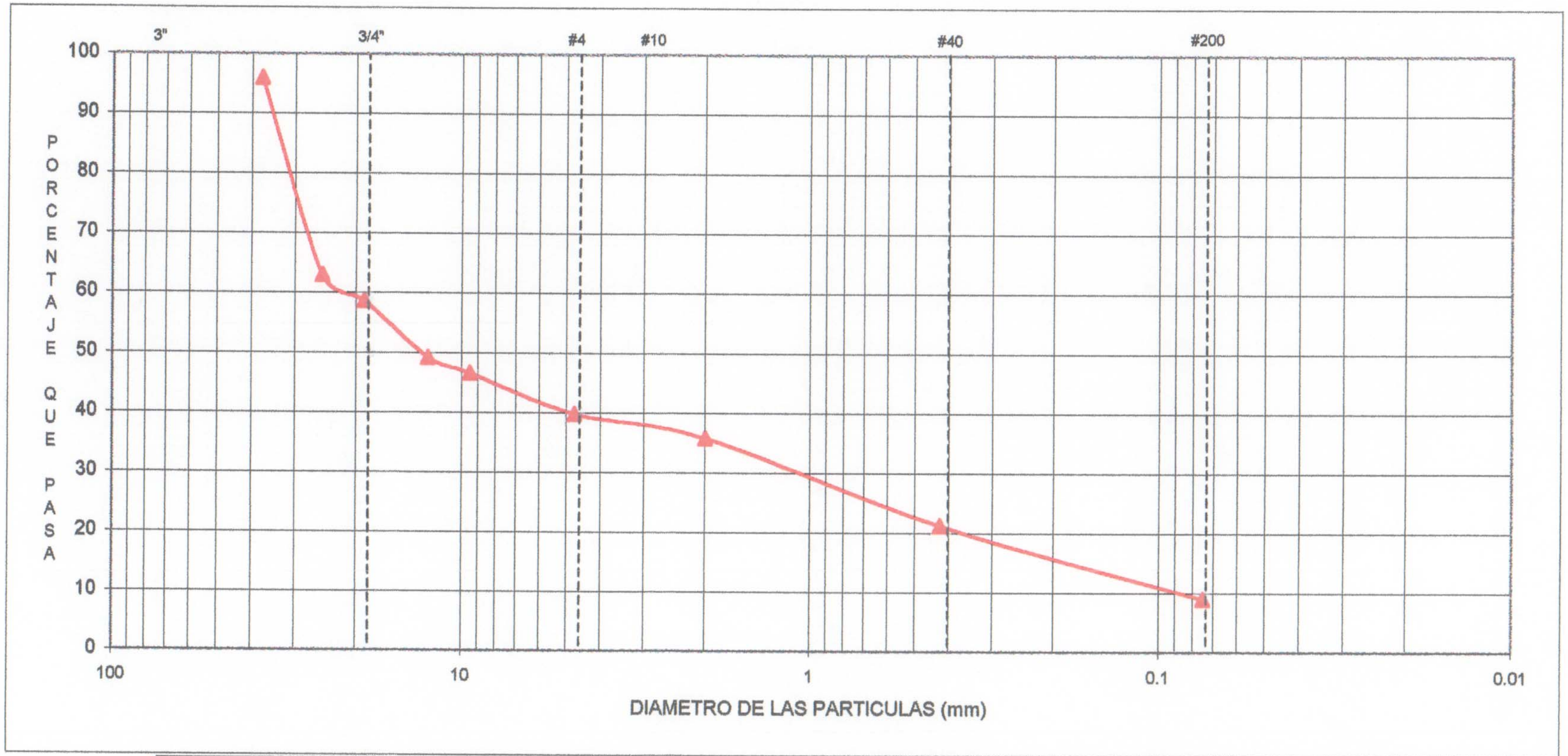
CLASIFICACION A.A.S.H.T.O.
A-2-6 (I.G:0)

TECNILAB, S. A.
UNA EMPRESA E. BARRANCO Y ASOC., S. A.
LABORATORIO DE SUELOS Y MATERIALES

FUNDADA EN 1973

ANALISIS GRANULOMETRICO

TRABAJO No. : 4-194 CLIENTE HARZA Engineering Company MUESTRA No. GR-01
PROYECTO: Cuenca del Canal. LOCALIZACION Río Indio
MUESTREADO POR Tecnilab, S. A. FECHA Feb. 2000 PROFUNDIDAD
PREPARADO POR Tecnilab, S. A. FECHA Feb. 2000 LABORATORISTA J.V



GRAVA		ARENA			LIMO Y/O ARCILLA
GRUESA	FINA	GRUESA	MEDIA	FINA	

TECNILAB, S. A.
FUNDADA EN 1973
UNA EMPRESA E. BARRANCO Y ASOC., S. A.
LABORATORIO DE SUELOS Y MATERIALES

GRAVEDAD ESPECIFICA EN LOS SUELOS

TRABAJO No. : 4-194 CLIENTE HARZA Engeenring Company

PROYECTO: Cuenca del Canal. LOCALIZACION Río Indio

MUESTREADO POR Tecnilab, S. A. FECHA Feb. 00 PROFUNDIDAD

PREPARADO POR Tecnilab, S. A. FECHA Feb. 00 LABORATORISTA J. Vergara

DETALLE	UNIDAD	NUMERO DE ENSAYO			
		1	2	3	4
PROFUNDIDAD	m				
MUESTRA No.		TP-01-S01	TP-01-S02	GTP-01	TP-02-S01
PICNOMETRO No.		1	1	3	3
PESO DE LA TARA	g	97.20	95.40	95.70	98.20
PESO DE LA TARA + SUELO SECO	g	197.20	196.70	196.00	194.00
PESO DEL SUELO SECO (W_0)	g	100.00	101.30	100.30	95.80
PICNOMETRO + AGUA + SUELO (W_1)	g	731.40	732.40	736.50	737.70
PICNOMETRO + AGUA A CAPACIDAD TOTAL (W_2)	g	668.70	668.70	674.20	674.20
TEMPERATURA DE ENSAYO	^0C	26	26	26	26
GRAVEDAD ESPECIFICA (G_S)		2.68	2.69	2.64	2.97

$$G_S = \frac{W_0}{W_0 + W_2 - W_1}$$

OBSERVACIONES:

REVISADO POR: N.R.C

TECNILAB, S. A.
FUNDADA EN 1973
UNA EMPRESA E. BARRANCO Y ASOC., S. A.
LABORATORIO DE SUELOS Y MATERIALES

GRAVEDAD ESPECIFICA EN LOS SUELOS

TRABAJO No. : 4-194 CLIENTE HARZA Engeenring Company

PROYECTO: Cuenca del Canal. LOCALIZACION Río Indio

MUESTREADO POR Tecnilab, S. A. FECHA Feb. 00 PROFUNDIDAD

PREPARADO POR Tecnilab, S. A. FECHA Feb. 00 LABORATORISTA J. Vergara

DETALLE	UNIDAD	NUMERO DE ENSAYO			
		5	6	7	8
PROFUNDIDAD	m				
MUESTRA No.		TP-02-S02	TP-03-S01	TP-03--S02	GTP-03
PICNOMETRO No.		1	3	1	3
PESO DE LA TARA	g	73.20	74.00	94.10	73.40
PESO DE LA TARA + SUELO SECO	g	174.60	175.00	194.70	173.50
PESO DEL SUELO SECO (W_0)	g	101.40	101.00	100.60	100.10
PICNOMETRO + AGUA + SUELO (W_1)	g	733.40	738.60	733.60	736.80
PICNOMETRO + AGUA A CAPACIDAD TOTAL (W_2)	g	668.70	674.20	668.70	674.20
TEMPERATURA DE ENSAYO	^0C	26	26	26	26
GRAVEDAD ESPECIFICA (G_S)		2.76	2.76	2.82	2.67

$$G_S = \frac{W_0}{W_0 + W_2 - W_1}$$

OBSERVACIONES:

REVISADO POR: N.R.C

TECNILAB, S. A.
UNA EMPRESA E. BARRANCO Y ASOC., S. A.
LABORATORIO DE SUELOS Y MATERIALES
FUNDADA EN 1973

GRAVEDAD ESPECIFICA EN LOS SUELOS

TRABAJO No. : 4-194 CLIENTE HARZA Engeenring Company
PROYECTO: Cuenca del Canal. LOCALIZACION Río Indio
MUESTREADO POR Tecnilab, S. A. FECHA Feb. 00 PROFUNDIDAD
PREPARADO POR Tecnilab, S. A. FECHA Feb. 00 LABORATORISTA J. Vergara

DETALLE	UNIDAD	NUMERO DE ENSAYO			
		9	10	11	12
PROFUNDIDAD	m				
MUESTRA No.		TP-04-S01	TP-04-S02	GTP-04	GR-01
PICNOMETRO No.		1	3	1	3
PESO DE LA TARA	g	98.00	97.00	96.30	98.40
PESO DE LA TARA + SUELO SECO	g	199.80	198.10	195.20	197.50
PESO DEL SUELO SECO (W_0)	g	101.80	101.10	98.90	99.10
PICNOMETRO + AGUA + SUELO (W_1)	g	732.00	737.40	730.90	737.20
PICNOMETRO + AGUA A CAPACIDAD TOTAL (W_2)	g	668.70	674.20	668.70	674.20
TEMPERATURA DE ENSAYO	^0C	26	26	26	26
GRAVEDAD ESPECIFICA (G_S)		2.64	2.67	2.69	2.75

$$G_S = \frac{W_0}{W_0 + W_2 - W_1}$$

OBSERVACIONES:

REVISADO POR: N.R.C

TECNILAB, S. A.

FUNDADA EN 1973

UNA EMPRESA E. BARRANCO Y ASOC., S. A.

LABORATORIO DE SUELOS Y MATERIALES

GRAVEDAD ESPECIFICA Y ABSORCION DE AGREGADOS GRUESOS Y FINOS

TRABAJO No. : 4-194 CLIENTE HARZA Engineering Company HUECO No. ______ MUESTRA No. GR-01

PROYECTO: Cuenca del Canal. LOCALIZACION Río Indio

MUESTREADO POR Tecnilab, S. A. FECHA Feb.00 PROFUNDIDAD ______

PREPARADO POR Tecnilab, S. A. FECHA Feb. 00 LABORATORISTA J.Vergara

MATERIAL ______

AGREGADO GRUESO

DETALLE	UNIDAD	NUMERO DE ENSAYO			
		1	2	3	4
(A) PESO DE LA GRAVA SECADA AL HORNO	g	5,761.0			
PESO DE LA GRAVA SATURADA EN AGUA + TARA	g	3,830.0			
(B) PESO DE LA GRAVA SATURADA	g	6,126.0			
PESO DE LA TARA	g	173.0			
(C) PESO DE LA GRAVA SATURADA EN AGUA	g	3,657.0			
GRAVEDAD ESPECIFICA BRUTA A / B-C		2.33			
GRAVEDAD ESPECIFICA BRUTA (S.S.S.) B / B-C		2.48			
GRAVEDAD ESPECIFICA APARENTE A / A-C		2.74			
PORCENTAJE DE ABSORCION (B-A x 100) / A		6.34			

AGREGADO FINO

DETALLE	UNIDAD	NUMERO DE ENSAYO			
		1	2	3	4
(A) PESO DE LA ARENA SECADA AL HORNO	g				
PESO DE LA ARENA SATURADA EN AGUA + TARA	g				
(B) PESO DE LA ARENA SATURADA	g				
PESO DE LA TARA	g				
(C) PESO DE LA ARENA SATURADA EN AGUA	g				
GRAVEDAD ESPECIFICA BRUTA A / B-C					
GRAVEDAD ESPECIFICA BRUTA (S.S.S.) B / B-C					
GRAVEDAD ESPECIFICA APARENTE A / A-C					
PORCENTAJE DE ABSORCION (B-A x 100) / A					

OBSERVACIONES: ______

REVISADO POR: ______

TECNILAB, S. A.
FUNDADA EN 1973
UNA EMPRESA E. BARRANCO Y ASOC., S. A.
LABORATORIO DE SUELOS Y MATERIALES

PESO VOLUMETRICO SECO SUELTO

TRABAJO No. : 4-194 CLIENTE HARZA Engineering Company
PROYECTO: Cuenca del Canal. LOCALIZACION
MUESTREADO POR Tecnilab, S. A. FECHA Feb. 00 PROFUNDIDAD
PREPARADO POR Tecnilab, S. A. FECHA Feb. 00 LABORATORISTA J.Vergara

MUESTRA	NUMERO DE MOLDE	PESO DE MATERIAL SECO + VANDEJA	PESO DE VANDEJA (gr)	VOLUMEN DE MOLDE (cm^3)	PESO UNITARIO SECO SUELTO (kg./m^3)	PESO UNITARIO SECO SUELTO (kg./m^3)
TP-01-S01	C1	442.3	241.8	214.3	935.6	936.4
TP-01-S01	C2	441.8	241.8	213.4	937.1	936.4
TP-01-S02	C2	451.1	241.8	213.4	980.6	975.8
TP-01-S02	C2	450.0	241.8	214.4	970.9	975.8
GTP-01	C1	423.2	241.8	214.3	846.5	845.9
GTP-01	C2	422.2	241.8	213.4	845.2	845.9
TP-02-S01	C1	435.0	241.8	214.3	901.6	893.3
TP-02-S01	C2	430.7	241.8	213.4	885.1	893.3
TP-02-S02	C1	433.0	241.8	214.3	892.2	891.9
TP-02-S02	C2	432.1	241.8	213.4	891.6	891.9
TP-03-S01	C1	419.3	241.8	214.3	828.3	826.6
TP-03-S01	C1	418.6	241.8	214.3	825.0	826.6
TP-03-S02	C1	407.8	241.8	214.3	774.7	775.7
TP-03-S02	C2	407.6	241.8	213.4	776.8	775.7
GTP-03	C1	415.6	241.8	214.3	811.1	808.9
GTP-03	C2	414.0	241.8	213.4	806.8	808.9
TP-04-S01	C1	442.4	241.8	214.3	936.1	934.3
TP-04-S01	C2	440.8	241.8	213.4	932.4	934.3
TP-04-S02	C1	428.5	241.8	214.3	871.2	870.2
TP-04-S02	C2	427.3	241.8	213.4	869.1	870.2
GTP-04	C2	375.7	131.1	213.4	1146.2	1145.3
GTP-04	C2	375.3	131.1	213.4	1144.3	1145.3
GR-01	C1	486.8	241.8	214.3	1143.3	1140.7
GR-01	C2	484.7	241.8	213.4	1138.2	1140.7

OBSERVACIONES

PRESENTADO POR

TECNILAB, S. A. FUNDADA EN 1973
UNA EMPRESA E. BARRANCO Y ASOC., S. A.
LABORATORIO DE SUELOS Y MATERIALES

CONTENIDO DE HUMEDAD NATURAL

TRABAJO No. : 4-194 CLIENTE HARZA Engineering Company HUECO No. ____ MUESTRA No. ____
PROYECTO: Cuenca del Canal. LOCALIZACION Río Indio
MUESTREADO POR Tecnilab, S. A. FECHA Feb. 00 PROFUNDIDAD ____
PREPARADO POR Tecnilab, S. A. FECHA Feb. 00 LABORATORISTA J. Vergara

1	Muestra de Laboratorio No.	1	2	3	4	5	6	7	8		
2	Localización	TP-01-S01	TP-01-S02	TP-02-S01	TP-02-S02	TP-03-S01	TP-03-S02	TP-04-S01	TP-04-S02		
3	Elevación (m)										
4	Profundidad (m)	0.50-1.50	1.5	0.00-1.00	1.00-2.00	0.00-1.00	1.00-2.00	1.00-2.00	2.00		
5	Tara No.	16	1	B-40	E-71	99	E-46	R-22	A-37		
6	Tara + Suelo Húmedo (g)	191.50	206.40	217.40	241.50	190.00	204.30	230.50	209.10		
7	Tara + Suelo Seco (g)	165.70	173.30	183.50	198.10	153.30	164.40	187.90	172.40		
8	Pérdida de Humedad (g)	25.80	33.10	33.90	43.40	36.70	39.90	42.60	36.70		
9	Peso de la Tara (g)	111.80	105.00	105.90	104.50	101.50	109.10	105.30	107.60		
10	Peso del Suelo Seco (g)	53.90	68.30	77.60	93.60	51.80	55.30	82.60	64.80		
11	Contenido de Humedad (%)	47.90	48.50	43.70	46.40	70.80	72.20	51.60	56.60		

TECNILAB, S. A.

FUNDADA EN 1973

UNA EMPRESA E. BARRANCO Y ASOC., S. A.

LABORATORIO DE SUELOS Y MATERIALES

PRUEBA DE COMPACTACION

TRABAJO No. : 4-194 CLIENTE HARZA Engineering Company.

PROYECTO: Cuenca del Canal.

LOCALIZACION Río Indio

MUESTREADO POR Tecnilab, S. A. FECHA Feb. 00 MUESTRA No. TP-01-S01

PREPARADO POR Tecnilab, S. A. FECHA Feb. 00 LABORATORISTA J. Vergara

Volumen del Molde 0.03405 pie^3 Peso del Molde 4.20 lb

DETERMINACION DEL CONTENIDO DE HUMEDAD

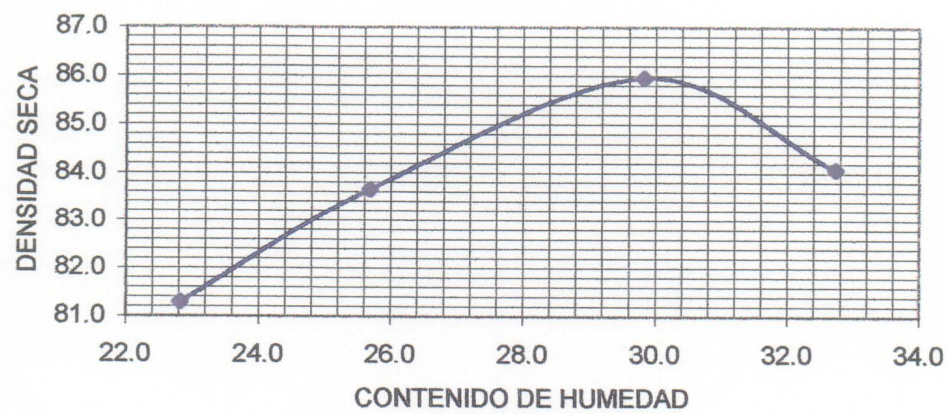
Prueba No.	1		2		3		4		5		6	
Recipiente No.	R-68	A-16	O-19	O-A	E-39	A-C	M-3	A-88				
Peso Recipiente	105.5	105.5	104.4	102.4	108.2	108.5	108.9	112.8				
Peso Recipiente + Suelo Húmedo	211.2	214.0	217.3	219.0	211.9	208.1	218.0	217.0				
Peso Recipiente + Suelo Seco	191.4	194.0	194.3	195.1	188.3	185.0	191.1	191.3				
Peso de Agua	19.8	20.0	23.0	23.9	23.6	23.1	26.9	25.7				
Peso de Suelo Seco	85.9	88.5	89.9	92.7	80.1	76.5	82.2	78.5				
% de Humedad	23.1	22.6	25.6	25.8	29.5	30.2	32.7	32.7				
% Humedad Promedio	22.8		25.7		29.8		32.7					

DETERMINACION DE LA DENSIDAD

Prueba No.	1	2	3	4	5	6
% Humedad Promedio	22.8	25.7	29.8	32.7		
Cantidad de Agua Añadida	200	100	100	100		
Peso del Molde + Agua	7.60	7.78	8.00	8.00		
Peso del Molde	4.20	4.20	4.20	4.20		
Peso del Suelo Húmedo	3.40	3.58	3.80	3.80		
Densidad Húmeda	99.9	105.1	111.6	111.6		
Densidad Seca	81.3	83.7	86.0	84.1		

D.M. 86.0 lb/pie^3 H.O. 30 %

1379.4 kg/m^3



TECNILAB, S. A.
UNA EMPRESA E. BARRANCO Y ASOC., S. A.
LABORATORIO DE SUELOS Y MATERIALES

FUNDADA EN 1973

PRUEBA DE COMPACTACION

TRABAJO No. : 4-194 CLIENTE HARZA Engineering Company.
PROYECTO: Cuenca del Canal.
LOCALIZACION Río Indio
MUESTREADO POR Tecnilab, S. A. FECHA Feb.00 MUESTRA No. TP-03-S01
PREPARADO POR Tecnilab, S. A. FECHA Feb.00 LABORATORISTA

Volumen del Molde 0.03405 pie^3 Peso del Molde 4.20 lb

DETERMINACION DEL CONTENIDO DE HUMEDAD

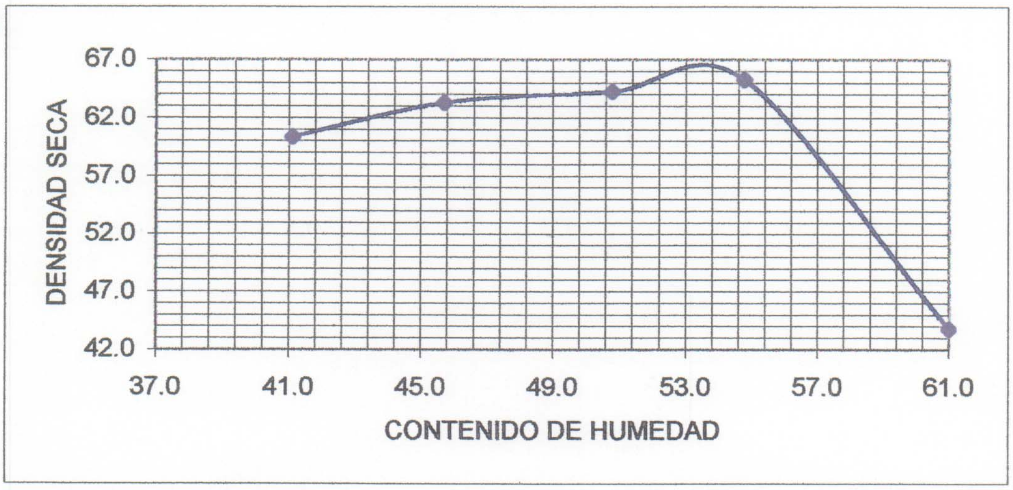
Prueba No.	1		2		3		4		5		6	
Recipiente No.	R-78	J-18	B1	R-4	N-1	R-22	10	A-37	A-38	L-20		
Peso Recipiente	109.4	111.8	105.0	105.3	108.7	108.4	101.7	102.6	107.6	109.2		
Peso Recipiente + Suelo Húmedo	212.5	211.5	172.1	169.6	173.6	162.5	161.5	169.1	204.1	204.3		
Peso Recipiente + Suelo Seco	182.2	182.7	151.3	149.2	151.6	144.4	140.4	145.5	167.4	168.4		
Peso de Agua	30.3	28.8	20.8	20.4	22.0	18.1	21.1	23.6	36.7	35.9		
Peso de Suelo Seco	72.8	70.9	46.3	43.9	42.9	36.0	38.7	42.9	59.8	59.2		
% de Humedad	41.6	40.6	44.9	46.5	51.3	50.3	54.5	55.0	61.4	60.6		
% Humedad Promedio	41.1		45.7		50.8		54.8		61.0			

DETERMINACION DE LA DENSIDAD

Prueba No.	1	2	3	4	5	6
% Humedad Promedio	41.1	45.7	50.8	54.8	61.0	
Cantidad de Agua Añadida	0	100	100	100	100	
Peso del Molde + Agua	7.10	7.34	7.50	7.64	7.60	
Peso del Molde	4.20	4.20	4.20	4.20	5.20	
Peso del Suelo Húmedo	2.90	3.14	3.30	3.44	2.40	
Densidad Húmeda	85.2	92.2	96.9	101.0	70.5	
Densidad Seca	60.4	63.3	64.3	65.3	43.8	

D.M. 66.5 lb/pie^3
1066.7 kg/m^3

H.O. 54 %



TECNILAB, S.A
FUNDADA EN 1973
UNA EMPRESA E. BARRANCO Y ASOC., S. A.
LABORATORIO DE SUELOS Y MATERIALES

PRUEBA DEL HIDROMETRO

TRABAJO No. : 4-194 CLIENTE HARZA Engeenring Company MUESTRA No. : TP-01-S01
PROYECTO: Cuenca del Canal. LOCALIZACION Río Indio
MUESTREADO POR Tecnilab, S. A. FECHA feb. 00 PROFUNDIDAD
PREPARADO POR Tecnilab, S. A. FECHA feb. 00 LABORATORISTA J.Vergara

ANALISIS HIDROMETRICO

HIDROMETRO No. : H-125 G_S DE LOS SOLIDOS : 2.68 a : 0.99 PESO DEL SUELO W_S : 100 gr
AGENTE DISPERSANTE : SILICATO DE SODIO AL 4% CANTIDAD : 125 ml
CORRECION DE CERO : 2 CORRECION DE MENISCO : 1

FECHA	HORA DE LECTURA	TIEMPO min.	TEMP. °C	LECTURA R	LECTURA R_C	% MAS FINO	R. corregido por menisco	L	L/t	K	D mm
14/02/00	13:23	1	24	54	53	52.47	55	7.30	7.300	0.01288	0.0348
	13:25	2	24	50	49	48.51	51	7.90	3.950	0.01288	0.025598
	13:28	5	24	47	46	45.54	48	8.40	1.680	0.01288	0.016694
	13:38	15	22	42	40.4	40.00	43	9.20	0.613	0.01318	0.010322
	14:23	60	22	36	34.4	34.06	37	10.20	0.170	0.01318	0.005434
	17:33	250	22	29	27.4	27.13	30	11.40	0.046	0.01318	0.002814
15/02/00	13:23	1440	23	22	20.7	20.49	23	12.50	0.009	0.01298	0.001209

TECNILAB, S.A
FUNDADA EN 1973
UNA EMPRESA E. BARRANCO Y ASOC., S. A.
LABORATORIO DE SUELOS Y MATERIALES

PRUEBA DE DISPERCION

TRABAJO No. : 4-194 CLIENTE HARZA Engeenring Company MUESTRA No. : TP-01-S01

PROYECTO: Cuenca del Canal. LOCALIZACION Río Indio

MUESTREADO POR Tecnilab, S. A. FECHA feb. 00 PROFUNDIDAD

PREPARADO POR Tecnilab, S. A. FECHA feb. 00 LABORATORISTA J.Vergara

ANALISIS POR DISPERCION

HIDROMETRO No. : H-125 G_S DE LOS SOLIDOS : 2.68 a : 0.99 PESO DEL SUELO W_S : 25 gr

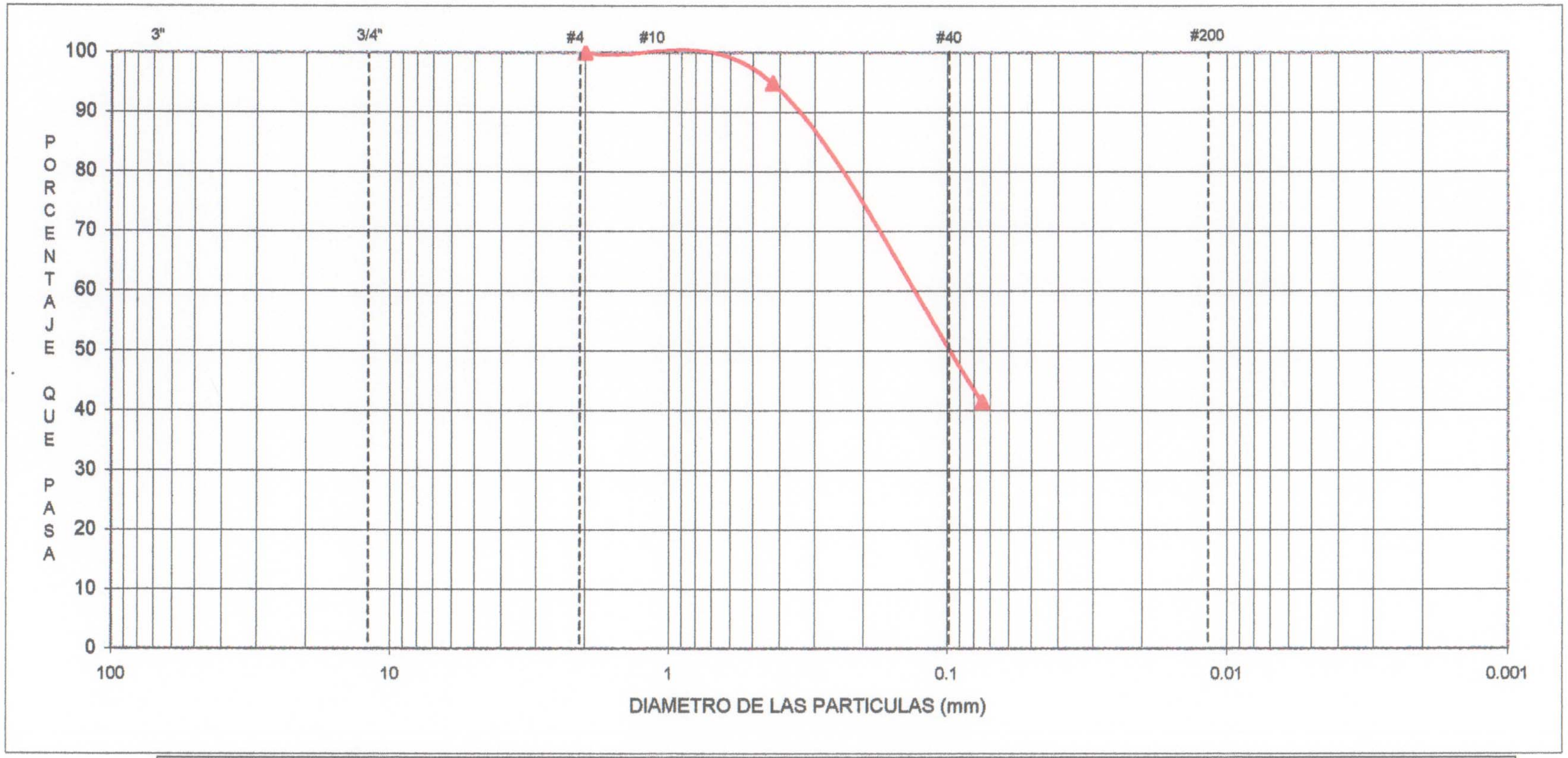
CORRECION DE MENISCO : 1

FECHA	HORA DE LECTURA	TIEMPO min.	TEMP. °C	LECTURA R	LECTURA R_C	% MAS FINO	R. corregido por menisco	L	L/t	K	D mm
15/02/00	16:50	0	23	6	6.7	26.53	7	15.20			
	16:51	1	23	3	3.7	14.65	4	15.60	15.600	0.0131	0.051741
	16:52	2	23	2	2.7	10.69	3	15.80	7.900	0.0131	0.03682
	16:55	5	23	0	0.7	2.77	1	16.10	3.220	0.0131	0.023507

TECNILAB, S. A.
FUNDADA EN 1973
UNA EMPRESA E. BARRANCO Y ASOC., S. A.
LABORATORIO DE SUELOS Y MATERIALES

ANALISIS GRANULOMETRICO

TRABAJO No. : 4-194 CLIENTE HARZA Engineering Company MUESTRA No. TP-01-S01
PROYECTO: Cuenca del Canal. LOCALIZACION Río Indio
MUESTREADO POR Tecnilab, S. A. FECHA Feb. 2000 PROFUNDIDAD
PREPARADO POR Tecnilab, S. A. FECHA Feb. 2000 LABORATORISTA J.V



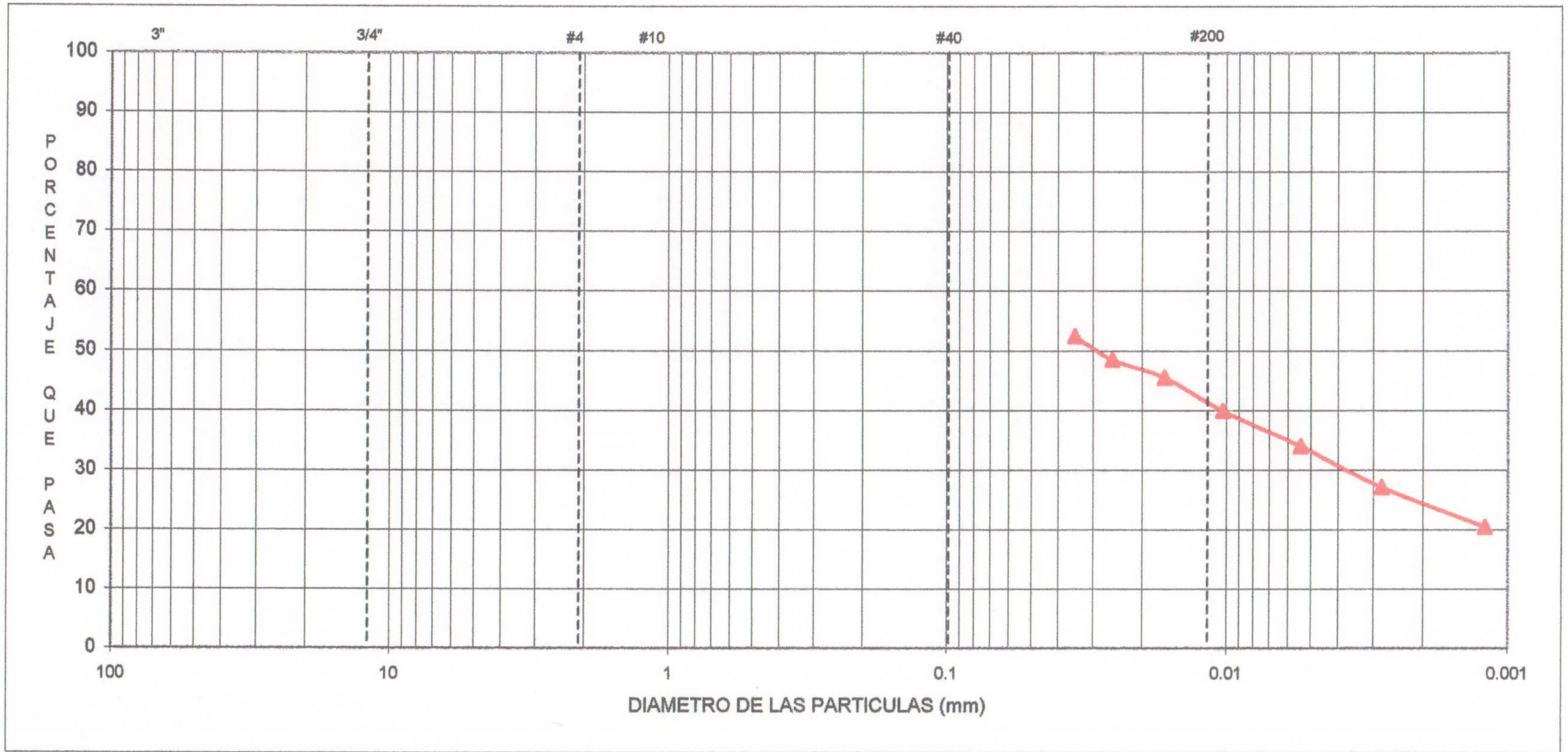
GRAVA		ARENA			LIMO Y/O ARCILLA
GRUESA	FINA	GRUESA	MEDIA	FINA	

TECNILAB, S. A.
UNA EMPRESA E. BARRANCO Y ASOC., S. A.
LABORATORIO DE SUELOS Y MATERIALES

FUNDADA EN 1973

ANALISIS GRANULOMETRICO, HIDROMETRO

TRABAJO No. : 4-194 CLIENTE HARZA Engineering Company MUESTRA No. TP-01-S01
PROYECTO: Cuenca del Canal. LOCALIZACION Río Indio
MUESTREADO POR Tecnilab, S. A. FECHA Feb. 2000 PROFUNDIDAD
PREPARADO POR Tecnilab, S. A. FECHA Feb. 2000 LABORATORISTA J.V



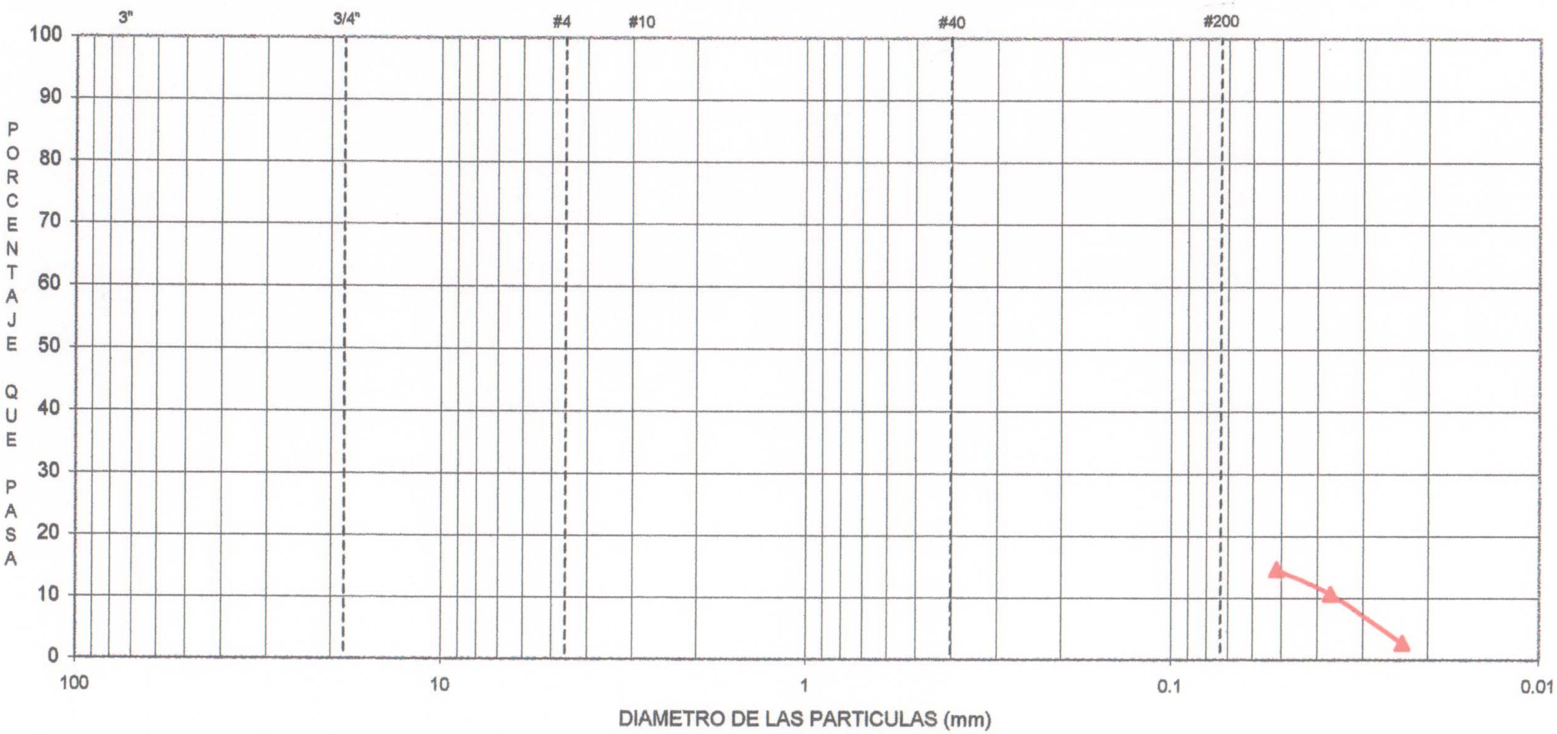
GRAVA		ARENA			LIMO Y/O ARCILLA
GRUESA	FINA	GRUESA	MEDIA	FINA	

TECNILAB, S. A.
UNA EMPRESA E. BARRANCO Y ASOC., S. A.
LABORATORIO DE SUELOS Y MATERIALES

FUNDADA EN 1973

ANALISIS GRANULOMETRICO, DISPERCION

TRABAJO No. : 4-194 CLIENTE HARZA Engineering Company MUESTRA No. TP-01-S01
PROYECTO: Cuenca del Canal. LOCALIZACION Río Indio
MUESTREADO POR Tecnilab, S. A. FECHA Feb. 2000 PROFUNDIDAD
PREPARADO POR Tecnilab, S. A. FECHA Feb. 2000 LABORATORISTA J.V



GRAVA		ARENA			LIMO Y/O ARCILLA
GRUESA	FINA	GRUESA	MEDIA	FINA	

TECNILAB, S. A. FUNDADA EN 1973
UNA EMPRESA E. BARRANCO Y ASOC., S. A.
LABORATORIO DE SUELOS Y MATERIALES

PRUEBA DEL HIDROMETRO

TRABAJO No. : 4-194 CLIENTE HARZA Engeenring Company MUESTRA No. : TP-03-S01
PROYECTO: Cuenca del Canal. LOCALIZACION Río Indio
MUESTREADO POR Tecnilab, S. A. FECHA feb. 00 PROFUNDIDAD
PREPARADO POR Tecnilab, S. A. FECHA feb. 00 LABORATORISTA J. Vergara

ANALISIS HIDROMETRICO

HIDROMETRO No. : H-125 G_S DE LOS SOLIDOS : 2.76 a : 0.98 PESO DEL SUELO W_S : 50 gr
AGENTE DISPERSANTE : SILICATO DE SODIO AL 4% CANTIDAD : 125 ml
CORRECION DE CERO : 2 CORRECION DE MENISCO : 1

FECHA	HORA DE LECTURA	TIEMPO min.	TEMP. °C	LECTURA R	LECTURA R_C	% MAS FINO	R. corregido por menisco	L	L/t	K	D mm
18/02/00	11:11	1	22	43	41.4	81.14	44	7.30	7.300	0.0127	0.034314
	11:12	2	22	41	39.4	77.22	42	7.90	3.950	0.0127	0.025241
	11:16	5	22	38	36.4	71.34	39	8.40	1.680	0.0127	0.016461
	11:26	15	22	29	27.4	53.70	30	9.20	0.613	0.0127	0.009946
	12:11	60	22	6	4.4	8.62	7	10.20	0.170	0.0127	0.005236
	15:21	250	22	2	0.4	0.78	3	11.40	0.046	0.0127	0.002712
19/02/00	11:11	1440	24	1	0	0.00	2	12.50	0.009	0.0124	0.001155

TECNILAB, S. A.
FUNDADA EN 1973
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LABORATORIO DE SUELOS Y MATERIALES

PRUEBA DE DISPERCION

TRABAJO No. : 4-194 CLIENTE HARZA Engeenring Company MUESTRA No. : TP-03-S01
PROYECTO: Cuenca del Canal. LOCALIZACION Río Indio
MUESTREADO POR Tecnilab, S. A. FECHA feb. 00 PROFUNDIDAD
PREPARADO POR Tecnilab, S. A. FECHA feb. 00 LABORATORISTA J. Vergara

ANALISIS POR DISPERCION

HIDROMETRO No. : H-125 G_S DE LOS SOLIDOS : 2.76 a : 0.98 PESO DEL SUELO W_S : 25 gr

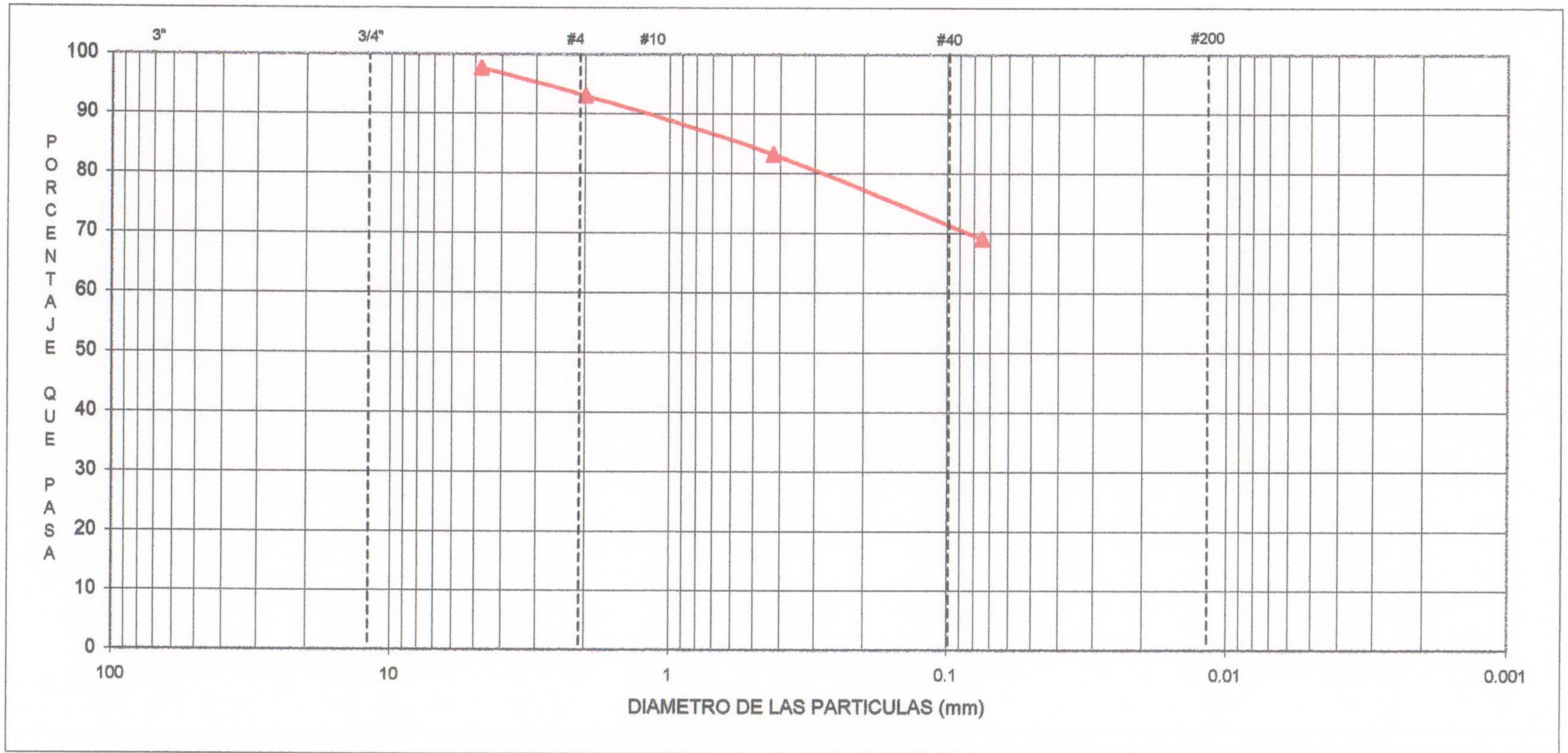
CORRECION DE MENISCO : 1

FECHA	HORA DE LECTURA	TIEMPO min.	TEMP. °C	LECTURA R	LECTURA R_C	% MAS FINO	R. corregido por menisco	L	L/t	K	D mm
18/02/00	13:25	0	22	6	6.4	25.09	7	15.20			
	13:26	1	22	4	4.4	17.25	5	15.50	15.500	0.0127	0.05
	13:27	2	22	3	3.4	13.33	4	15.60	7.800	0.0127	0.035469
	13:30	5	22	2	2.4	9.41	3	15.80	3.160	0.0127	0.022576
	13:40	15	22	1							

TECNILAB, S. A.
FUNDADA EN 1973
UNA EMPRESA E. BARRANCO Y ASOC., S. A.
LABORATORIO DE SUELOS Y MATERIALES

ANALISIS GRANULOMETRICO

TRABAJO No. : 4-194 CLIENTE HARZA Engineering Company MUESTRA No. TP-03-S01
PROYECTO: Cuenca del Canal. LOCALIZACION Río Indio
MUESTREADO POR Tecnilab, S. A. FECHA Feb. 2000 PROFUNDIDAD
PREPARADO POR Tecnilab, S. A. FECHA Feb. 2000 LABORATORISTA J.V

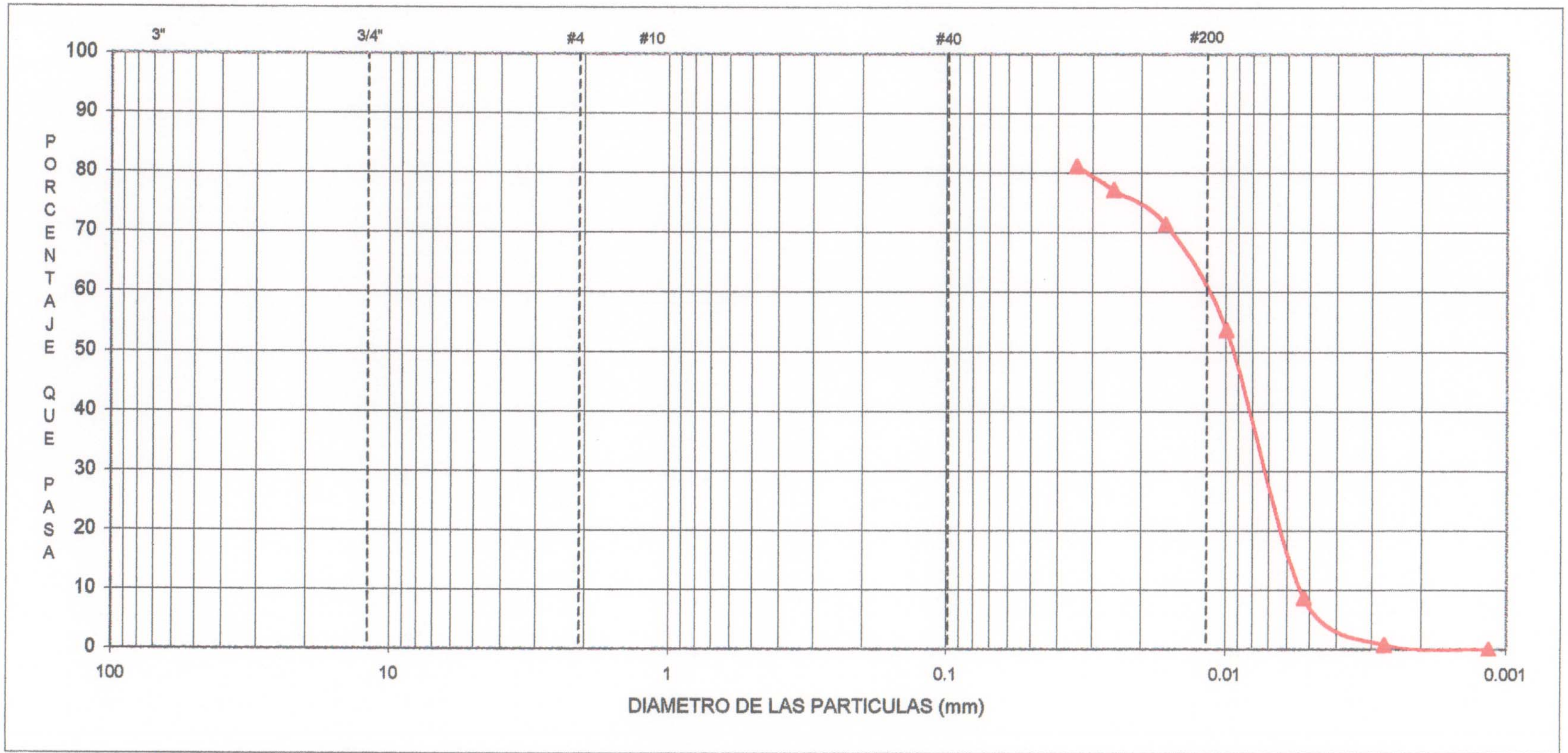


GRAVA		ARENA			LIMO Y/O ARCILLA
GRUESA	FINA	GRUESA	MEDIA	FINA	

TECNILAB, S. A.
UNA EMPRESA E. BARRANCO Y ASOC., S. A.
LABORATORIO DE SUELOS Y MATERIALES
FUNDADA EN 1973

ANALISIS GRANULOMETRICO, HIDROMETRO

TRABAJO No. : 4-194 CLIENTE HARZA Engineering Company MUESTRA No. TP-03-S01
PROYECTO: Cuenca del Canal. LOCALIZACION Río Indio
MUESTREADO POR Tecnilab, S. A. FECHA Feb. 2000 PROFUNDIDAD
PREPARADO POR Tecnilab, S. A. FECHA Feb. 2000 LABORATORISTA J.V



GRAVA		ARENA			LIMO Y/O ARCILLA
GRUESA	FINA	GRUESA	MEDIA	FINA	

TECNILAB, S. A. FUNDADA EN 1973
UNA EMPRESA E. BARRANCO Y ASOC., S. A.
LABORATORIO DE SUELOS Y MATERIALES

ANALISIS MECANICO Y LIMITES DE ATTERBERG

TRABAJO No. : 4-194 CLIENTE HARZA Engineering Company MUESTRA No. GTP-01
PROYECTO: Cuenca del Canal. LOCALIZACION Río Indio
MUESTREADO POR TECNILAB, S.A. FECHA Feb. 00 PROFUNDIDAD
PREPARADO POR TECNILAB, S.A. FECHA Feb. 00 LABORATORISTA J. VERGARA

ANALISIS MECANICO

TAMIZ	RETENIDO ACUMULADO	% RETENIDO	% QUE PASA
3"			
$2^1/_2$"			
2"			
$^3/_4$"			
$^1/_2$"			
$^3/_8$"			
#4			
FONDO			
TOTAL			

AGREGADO GRUESO
Peso Muestra Total Secada al Aire ______ gr

TAMIZ	RETENIDO ACUMULADO	% RETENIDO	% QUE PASA
#4	0.00	0.00	100.00
#10	0.00	0.30	99.70
#40	4.20	8.00	92.00
#200	41.80	83.70	16.30
FONDO	2.30	84.00	16.00
TOTAL			

AGREGADO FINO
Peso Muestra Total Secada al Aire 170.10 gr
Peso Muestra Total Seca 125.80 gr
Peso Seco Depués de Lavado 84.00 gr

LIMITES DE ATTERBERG

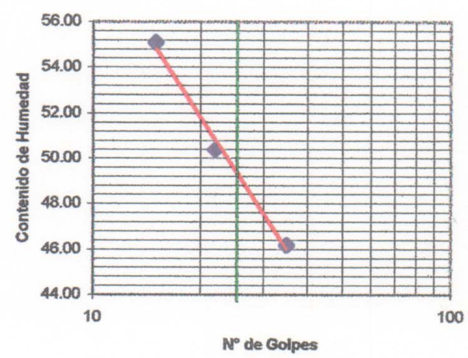
LIMITE LIQUIDO

CAPSULA No.	PESO CAPSULA (gr)	CAPSULA + SUELO HUM. (gr)	CAPSULA + SUELO SECO (gr)	AGUA (gr)	SUELO SECO (gr)	CONTENIDO DE HUMEDAD (%)	NUMERO DE GOLPES
T-7	10.70	30.40	23.40	7.00	12.70	55.10	15
T-6	10.60	28.80	22.70	6.10	12.10	50.40	22
T-1	10.70	29.70	23.70	6.00	13.00	46.20	35

LIMITE PLASTICO

CAPSULA No.	PESO CAPSULA (gr)	CAPSULA + SUELO HUM. (gr)	CAPSULA + SUELO SECO (gr)	AGUA (gr)	SUELO SECO (gr)	CONTENIDO DE HUMEDAD (%)	PROM.
95	12.30	20.20	18.70	1.50	6.40	23.40	26.80
28	11.10	19.30	17.40	1.90	6.30	30.20	

CLASIFICACION Arena arcillosa; plásticidad media, color café claro.



CLASIFICACION S.U.C.S. SC

L.L. = 49.40
L.P. = 26.80
I.p. = 22.60

CLASIFICACION A.A.S.H.T.O.
A-2-7 (I.G:0)

TECNILAB, S. A. FUNDADA EN 1973

UNA EMPRESA E. BARRANCO Y ASOC., S. A.

LABORATORIO DE SUELOS Y MATERIALES

ANALISIS MECANICO Y LIMITES DE ATTERBERG

TRABAJO No. : 4-194 CLIENTE HARZA Engineering Company MUESTRA No. TP-02-S01

PROYECTO: Cuenca del Canal. LOCALIZACION Río Indio

MUESTREADO POR TECNILAB, S.A. FECHA Feb. 00 PROFUNDIDAD

PREPARADO POR TECNILAB, S.A. FECHA Feb. 00 LABORATORISTA J. VERGARA

ANALISIS MECANICO

TAMIZ	RETENIDO ACUMULADO	% RETENIDO	% QUE PASA
3"			
$2^1/_2$"			
2"			
$^3/_4$"			
$^1/_2$"			
$^3/_8$"			
#4			
FONDO			
TOTAL			

AGREGADO GRUESO

Peso Muestra Total Secada al Aire ________ gr

TAMIZ	RETENIDO ACUMULADO	% RETENIDO	% QUE PASA
#4	1.10	0.73	99.27
#10	4.20	2.80	97.20
#40	19.10	12.70	87.30
#200	52.90	35.20	64.80
FONDO	53.40	35.60	64.40
TOTAL			

AGREGADO FINO

Peso Muestra Total Secada al Aire 170.00 gr

Peso Muestra Total Seca 150.20 gr

Peso Seco Depués de Lavado 53.40 gr

LIMITES DE ATTERBERG

LIMITE LIQUIDO

CAPSULA No.	PESO CAPSULA (gr)	CAPSULA + SUELO HUM. (gr)	CAPSULA + SUELO SECO (gr)	AGUA (gr)	SUELO SECO (gr)	CONTENIDO DE HUMEDAD (%)	NUMERO DE GOLPES
T-11	10.70	27.70	21.20	6.50	10.50	61.90	14
T-5	10.90	28.50	21.80	6.70	10.90	61.50	22
T-3	10.60	28.80	22.10	6.70	11.50	58.30	33

LIMITE PLASTICO

CAPSULA No.	PESO CAPSULA (gr)	CAPSULA + SUELO HUM. (gr)	CAPSULA + SUELO SECO (gr)	AGUA (gr)	SUELO SECO (gr)	CONTENIDO DE HUMEDAD (%)	PROM.
60	11.30	18.80	16.90	1.90	5.60	33.90	34.90
4	10.90	18.10	16.20	1.90	5.30	35.80	

CLASIFICACION Limo inorgánico, plásticidad alta, color café claro

CLASIFICACION S.U.C.S. MH

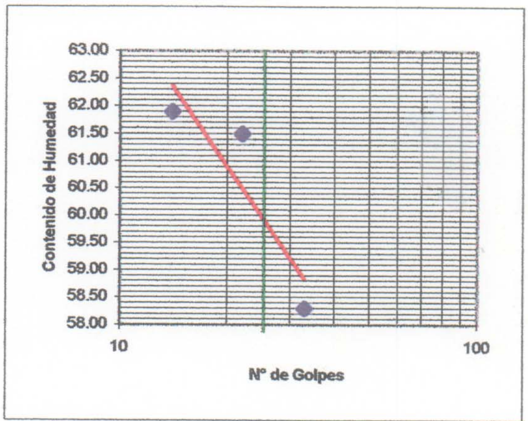
L.L. = 59.80

L.P. = 34.90

I.p. = 24.90

CLASIFICACION A.A.S.H.T.O.

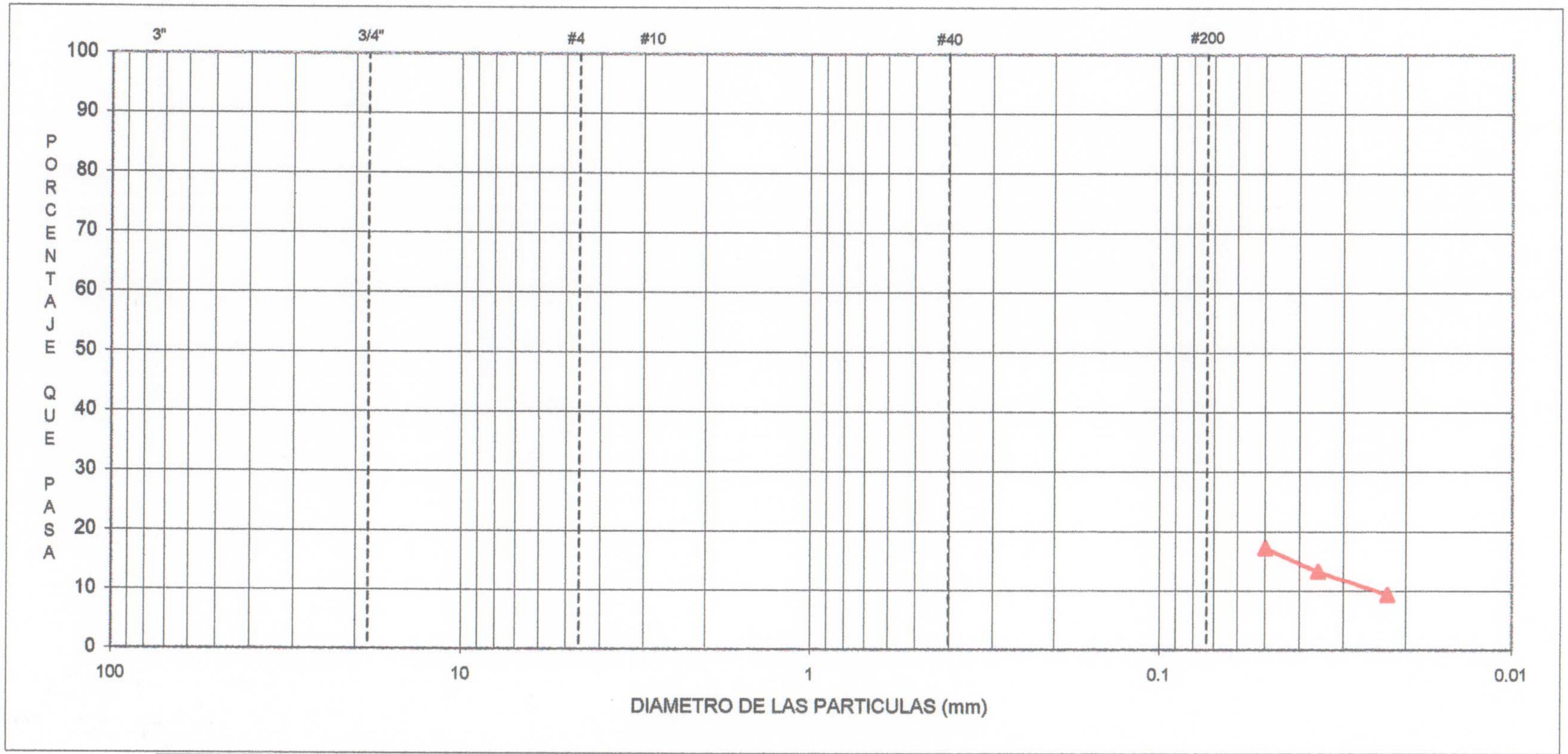
A-7-5 (IG: 15)



TECNILAB, S. A. FUNDADA EN 1973
UNA EMPRESA E. BARRANCO Y ASOC., S. A.
LABORATORIO DE SUELOS Y MATERIALES

ANALISIS GRANULOMETRICO, DISPERCION

TRABAJO No. : 4-194 CLIENTE HARZA Engineering Company MUESTRA No. TP-03-S01
PROYECTO: Cuenca del Canal. LOCALIZACION Río Indio
MUESTREADO POR Tecnilab, S. A. FECHA Feb. 2000 PROFUNDIDAD
PREPARADO POR Tecnilab, S. A. FECHA Feb. 2000 LABORATORISTA J.V



GRAVA		ARENA			LIMO Y/O ARCILLA
GRUESA	FINA	GRUESA	MEDIA	FINA	

TECNILAB, S. A.
FUNDADA EN 1973
UNA EMPRESA E. BARRANCO Y ASOC., S. A.
LABORATORIO DE SUELOS Y MATERIALES

DOBLE HIDROMETRO

TRABAJO No. : 4-194 CLIENTE HARZA Engeenring Company MUESTRA No.
PROYECTO: Cuenca del Canal LOCALIZACION Río Indio
MUESTREADO POR Tecnilab, S. A. FECHA Feb.00 PROFUNDIDAD
PREPARADO POR Tecnilab, S. A. FECHA Feb.00 LABORATORISTA J.Vergara

ENSAYO	UNIDAD	MUESTRA N°	
		TP-01-S01	TP-03-S01
1. HIDROMETRO			
% MAS FINO QUE 0.002 mm	%	25	0
2. DISPERCION			
% MAS FINO QUE 0.002 mm	%	2	8
% DE DISPERCION	%	8	0

NOTA:

REVISADO POR: N.R.C

TECNILAB, S. A.
UNA EMPRESA E. BARRANCO Y ASOC., S. A.
LABORATORIO DE SUELOS Y MATERIALES

FUNDADA EN 1973

ENSAYO DE DESGASTE DE LOS ANGELES

TRABAJO No. : 4-194 CLIENTE HARZA Engineering Company
PROYECTO: Cuenca del Canal. MATERIAL: GR-01
LOCALIZACION Rio Indio
MUESTREADO POR Tecnilab, S. A. FECHA Feb-00
PREPARADO POR Tecnilab, S. A. FECHA Feb-00 LABORATORISTA J.V

ESFERA: 12 DIAMETRO: 1 27/32" PESO 44.5 gr TIEMPO DE LA PRUEBA 15 min

TAMAÑO DE TAMICES		PESO ORIGINAL DE LA MUESTRA (gr)
QUE PASA	RETENIDO	
1 1/2	1	1250.00
1	3/4	1250.00
3/4	1/2	1250.00
1/2	3/8	1250.00
TOTAL:		**5000.00**

a) Peso original de la muestra: 5000.00 gr
b) Peso del material retenido en el tamiz N°12: 3850.00 gr
c) Perdida de peso (a-b): 1150.00 gr
d) Porcentaje de desgaste: 23.00 %

Revoluciones por minuto: 33 rpm Gradación de mutra para prueba: A

INVESTIGACIÓN DE SUELOS
ABASTECIMIENTO DE AGUA DE RIO INDIO

RESULTADO DE PRUEBAS REALIZADAS A MUESTRAS DE ROCAS

TECNILAB, S. A.

ANALISIS PETROGRAFICO

R01: Roca de estructura clástica, textura regular; fragmentos de rocas sedimentarias y volcánicas (limolitas y andesitas en su mayoría meteorizados); fragmentos angulosos de minerales feldespáticos frescos y de magnetita. Fragmentos con diàmetros entre 0.06 y 0.2 mm. Empaquetamiento discreto; selección mala; clasificación discreta. En una matriz mixta micrítica-arcillosa, con algo de limo (plagioclasa y fémicos).

fragmentos de rocas	±35%
fragmentos de minerales	±35%
matriz micritico-arcillosa	±30%

La roca es una **Arenisca fina** tipo grauvaca.

R02: Roca de estructura clástica, textura regular; fragmentos angulosos de vidrio volcánico (esquirlas). Fósiles y fragmentos de fósiles (microforaminíferos); fragmentos de plagioclasa, celadonita y calcita microcristalina. Empaquetamiento discreto; selección discreta; clasificación media (diámetro de los fragmentos entre 0.05 y 0.125 mm). Matriz limo-arcillosa, poca.

esquirlas de vidrio	±55%
fragmentos de minerales	±35%
matriz micritico-arcillosa	±10%

La roca es una **Arenisca Tobácea Vidriosa fosilífera.**

R03: Roca de estructura porfídica, textura intergranular, holocristalina. Fenocristales de plagioclasa un poco zonada (andesina-labradorita) con inclusiones de magnetita, parches de alteración albítica. Probable piroxeno alterados completamente en biotita secundaria. En una pseudo-pasta más bien gruesa (en realidad hay dos generaciones de cristales) de láminas de plagioclasa, y biotita secundaria. Magnetita abundante.

plagioclasa	±55%
biotita secundaria	±17%
clorita y m. arcillosos	± 2%
Magnetita	±25%
otros	± 1%

La roca es un **Subintrusivo Andesítico**. Su uso como agregado en concreto es adecuado.

R04: Roca de estructura porfídica, textura granular, hipocristalina. Fenocristales zonados de plagioclasa (de labradorita a andesina hacia los bordes); fenocristales de piroxeno alterados calcita, clorita y mica Fe. En una pasta de fondo de láminas de plagioclasa y de pequeñas cantidades de vidrio volcánico intersticial (probablemente básico). Magnetita.

plagioclasa	± 57%
calcita	± 5%
clorita	± 5%
mica Fe	± 3%
vidrio volcánico	± 10%

Magnetita	± 20%

La roca es un **Basalto un poco Vidrioso** (volcanita de composición básica).

__RO5__: Roca de estructura porfídica, textura granular, hipocristalina. Fenocristales zonados de plagioclasa (de labradorita a andesina hacia los bordes); fenocristales de piroxeno alterados en clorita. En una pasta de fondo de láminas de plagioclasa y de pequeñas cantidades de vidrio volcánico intersticial. Magnetita.

plagioclasa	56%
clorita	15%
vidrio volcánico	8%
Magnetita	20%

La roca es un **Basalto un poco Vidrioso.**

CERRO BARRERO: Roca de estructura porfídica, textura intergranular, holocristalina. Fenocristales de plagioclasa zonada (bordes de andesina y núcleo de labradorita). Fenocristales de piroxeno algunos alterados completamente en clorita secundaria. En una pasta más bien gruesa de láminas de plagioclasa y de cristales de piroxeno . Magnetita abundante.

plagioclasa	±60%
clorita secundaria	±20%
Magnetita	±20%
otros	± 1%

La roca es un **Subintrusivo Andesítico**. Su uso como agregado en concreto es adecuado.

RI-2 (arenisca del sitio de presa): Roca de estructura clástica, textura regular; fragmentos angulosos finos semi-alterados en m. arcillosos de vidrio volcánico (esquirlas). Fósiles y fragmentos de fósiles (microforaminíferos); fragmentos de 0.05 mm máximo de plagioclasa, posible ceolita y calcita microcristalina.

La roca es una **Limolita Tobácea Vidriosa Fosilífera un poco calcárea**

EL TORNITO: Roca de estructura clástica, textura regular; fragmentos con diàmetros entre 0.06 y 0.2 mm de rocas sedimentarias y volcánicas (limolitas y andesitas en su mayoría meteorizados) ; fragmentos angulosos de minerales feldespáticos frescos y de magnetita. Fragmentos de fósiles de gasteròpodos y posibles lamelibranquios. Empaquetamiento discreto; selección mala; clasificación discreta. En una matriz mixta micrítica-arcillosa, con algo de limo (plagioclasa y fémicos).

fragmentos de rocas	±30%
fragmento de fósiles	±15%
fragmentos de minerales	±25%
matriz micritico-arcillosa	±30%

La roca es una **Arenisca Grauvaca Fosilífera**.

DR. ERIC GUTIERREZ
PETROGRAFO

FEBRERO 2000

TECNILAB, S. A.
UNA EMPRESA E. BARRANCO Y ASOC., S. A.
LABORATORIO DE SUELOS Y MATERIALES

FUNDADA EN 1973

GRAVEDAD ESPECIFICA Y ABSORCION DE AGREGADOS GRUESOS Y FINOS

TRABAJO No. : 4-194 CLIENTE HARZA Engineering Company HUECO No. ____ MUESTRA No. R-01
PROYECTO: Cuenca del Canal. LOCALIZACION Río Indio
MUESTREADO POR Tecnilab, S. A. FECHA Feb.00 PROFUNDIDAD ____
PREPARADO POR Tecnilab, S. A. FECHA Feb. 00 LABORATORISTA J.Vergara

MATERIAL ____

AGREGADO GRUESO

DETALLE	UNIDAD	NUMERO DE ENSAYO			
		1	2	3	4
(A) PESO DE LA GRAVA SECADA AL HORNO	g	3,203.0			
PESO DE LA GRAVA SATURADA EN AGUA + TARA	g	2,118.0			
(B) PESO DE LA GRAVA SATURADA	g	3,786.0			
PESO DE LA TARA	g	173.0			
(C) PESO DE LA GRAVA SATURADA EN AGUA	g	1,945.0			
GRAVEDAD ESPECIFICA BRUTA A / B-C		1.74			
GRAVEDAD ESPECIFICA BRUTA (S.S.S.) B / B-C		2.06			
GRAVEDAD ESPECIFICA APARENTE A / A-C		2.55			
PORCENTAJE DE ABSORCION (B-A x 100) / A		18.20			

AGREGADO FINO

DETALLE	UNIDAD	NUMERO DE ENSAYO			
		1	2	3	4
(A) PESO DE LA ARENA SECADA AL HORNO	g				
PESO DE LA ARENA SATURADA EN AGUA + TARA	g				
(B) PESO DE LA ARENA SATURADA	g				
PESO DE LA TARA	g				
(C) PESO DE LA ARENA SATURADA EN AGUA	g				
GRAVEDAD ESPECIFICA BRUTA A / B-C					
GRAVEDAD ESPECIFICA BRUTA (S.S.S.) B / B-C					
GRAVEDAD ESPECIFICA APARENTE A / A-C					
PORCENTAJE DE ABSORCION (B-A x 100) / A					

OBSERVACIONES: ____

REVISADO POR: ____

TECNILAB, S. A. FUNDADA EN 1973
UNA EMPRESA E. BARRANCO Y ASOC., S. A.
LABORATORIO DE SUELOS Y MATERIALES

GRAVEDAD ESPECIFICA Y ABSORCION DE AGREGADOS GRUESOS Y FINOS

TRABAJO No. : 4-194 CLIENTE HARZA Engineering Company HUECO No. ____ MUESTRA No. R-02
PROYECTO: Cuenca del Canal. LOCALIZACION Río Indio
MUESTREADO POR Tecnilab, S. A. FECHA Feb.00 PROFUNDIDAD ____
PREPARADO POR Tecnilab, S. A. FECHA Feb. 00 LABORATORISTA J.Vergara

MATERIAL ____

AGREGADO GRUESO

DETALLE	UNIDAD	NUMERO DE ENSAYO			
		1	2	3	4
(A) PESO DE LA GRAVA SECADA AL HORNO	g	475.0			
PESO DE LA GRAVA SATURADA EN AGUA + TARA	g	453.0			
(B) PESO DE LA GRAVA SATURADA	g	539.0			
PESO DE LA TARA	g	173.0			
(C) PESO DE LA GRAVA SATURADA EN AGUA	g	280.0			
GRAVEDAD ESPECIFICA BRUTA A / B-C		1.83			
GRAVEDAD ESPECIFICA BRUTA (S.S.S.) B / B-C		2.08			
GRAVEDAD ESPECIFICA APARENTE A / A-C		2.44			
PORCENTAJE DE ABSORCION (B-A x 100) / A		13.47			

AGREGADO FINO

DETALLE	UNIDAD	NUMERO DE ENSAYO			
		1	2	3	4
(A) PESO DE LA ARENA SECADA AL HORNO	g				
PESO DE LA ARENA SATURADA EN AGUA + TARA	g				
(B) PESO DE LA ARENA SATURADA	g				
PESO DE LA TARA	g				
(C) PESO DE LA ARENA SATURADA EN AGUA	g				
GRAVEDAD ESPECIFICA BRUTA A / B-C					
GRAVEDAD ESPECIFICA BRUTA (S.S.S.) B / B-C					
GRAVEDAD ESPECIFICA APARENTE A / A-C					
PORCENTAJE DE ABSORCION (B-A x 100) / A					

OBSERVACIONES: ____

REVISADO POR: ____

TECNILAB, S. A.
FUNDADA EN 1973
UNA EMPRESA E. BARRANCO Y ASOC., S. A.
LABORATORIO DE SUELOS Y MATERIALES

GRAVEDAD ESPECIFICA Y ABSORCION DE AGREGADOS GRUESOS Y FINOS

TRABAJO No. : 4-194 CLIENTE HARZA Engineering Company HUECO No. MUESTRA No. R-03
PROYECTO: Cuenca del Canal. LOCALIZACION Río Indio
MUESTREADO POR Tecnilab, S. A. FECHA Feb.00 PROFUNDIDAD
PREPARADO POR Tecnilab, S. A. FECHA Feb. 00 LABORATORISTA J.Vergara

MATERIAL

AGREGADO GRUESO

DETALLE	UNIDAD	NUMERO DE ENSAYO			
		1	2	3	4
(A) PESO DE LA GRAVA SECADA AL HORNO	g	3,920.0			
PESO DE LA GRAVA SATURADA EN AGUA + TARA	g	2,677.0			
(B) PESO DE LA GRAVA SATURADA	g	4,026.0			
PESO DE LA TARA	g	173.0			
(C) PESO DE LA GRAVA SATURADA EN AGUA	g	2,504.0			
GRAVEDAD ESPECIFICA BRUTA A / B-C		2.58			
GRAVEDAD ESPECIFICA BRUTA (S.S.S.) B / B-C		2.65			
GRAVEDAD ESPECIFICA APARENTE A / A-C		2.77			
PORCENTAJE DE ABSORCION (B-A x 100) / A		2.70			

AGREGADO FINO

DETALLE	UNIDAD	NUMERO DE ENSAYO			
		1	2	3	4
(A) PESO DE LA ARENA SECADA AL HORNO	g				
PESO DE LA ARENA SATURADA EN AGUA + TARA	g				
(B) PESO DE LA ARENA SATURADA	g				
PESO DE LA TARA	g				
(C) PESO DE LA ARENA SATURADA EN AGUA	g				
GRAVEDAD ESPECIFICA BRUTA A / B-C					
GRAVEDAD ESPECIFICA BRUTA (S.S.S.) B / B-C					
GRAVEDAD ESPECIFICA APARENTE A / A-C					
PORCENTAJE DE ABSORCION (B-A x 100) / A					

OBSERVACIONES:

REVISADO POR:

TECNILAB, S. A.
UNA EMPRESA E. BARRANCO Y ASOC., S. A.
LABORATORIO DE SUELOS Y MATERIALES

FUNDADA EN 1973

GRAVEDAD ESPECIFICA Y ABSORCION DE AGREGADOS GRUESOS Y FINOS

TRABAJO No. : 4-194 CLIENTE HARZA Engineering Company HUECO No. MUESTRA No. R-04

PROYECTO: Cuenca del Canal. LOCALIZACION Río Indio

MUESTREADO POR Tecnilab, S. A. FECHA Feb.00 PROFUNDIDAD

PREPARADO POR Tecnilab, S. A. FECHA Feb. 00 LABORATORISTA J.Vergara

MATERIAL

AGREGADO GRUESO

DETALLE	UNIDAD	NUMERO DE ENSAYO			
		1	2	3	4
(A) PESO DE LA GRAVA SECADA AL HORNO	g	3,184.0			
PESO DE LA GRAVA SATURADA EN AGUA + TARA	g	2,223.0			
(B) PESO DE LA GRAVA SATURADA	g	3,240.0			
PESO DE LA TARA	g	173.0			
(C) PESO DE LA GRAVA SATURADA EN AGUA	g	2,050.0			
GRAVEDAD ESPECIFICA BRUTA A / B-C		2.68			
GRAVEDAD ESPECIFICA BRUTA (S.S.S.) B / B-C		2.72			
GRAVEDAD ESPECIFICA APARENTE A / A-C		2.81			
PORCENTAJE DE ABSORCION (B-A x 100) / A		1.76			

AGREGADO FINO

DETALLE	UNIDAD	NUMERO DE ENSAYO			
		1	2	3	4
(A) PESO DE LA ARENA SECADA AL HORNO	g				
PESO DE LA ARENA SATURADA EN AGUA + TARA	g				
(B) PESO DE LA ARENA SATURADA	g				
PESO DE LA TARA	g				
(C) PESO DE LA ARENA SATURADA EN AGUA	g				
GRAVEDAD ESPECIFICA BRUTA A / B-C					
GRAVEDAD ESPECIFICA BRUTA (S.S.S.) B / B-C					
GRAVEDAD ESPECIFICA APARENTE A / A-C					
PORCENTAJE DE ABSORCION (B-A x 100) / A					

OBSERVACIONES:

REVISADO POR:

TECNILAB, S. A. FUNDADA EN 1973
UNA EMPRESA E. BARRANCO Y ASOC., S. A.
LABORATORIO DE SUELOS Y MATERIALES

GRAVEDAD ESPECIFICA Y ABSORCION DE AGREGADOS GRUESOS Y FINOS

TRABAJO No. : 4-194 CLIENTE HARZA Engineering Company HUECO No. ____ MUESTRA No. R-05
PROYECTO: Cuenca del Canal. LOCALIZACION Río Indio
MUESTREADO POR Tecnilab, S. A. FECHA Feb.00 PROFUNDIDAD ____
PREPARADO POR Tecnilab, S. A. FECHA Feb. 00 LABORATORISTA J.Vergara

MATERIAL ____

AGREGADO GRUESO

DETALLE	UNIDAD	NUMERO DE ENSAYO			
		1	2	3	4
(A) PESO DE LA GRAVA SECADA AL HORNO	g	2,949.0			
PESO DE LA GRAVA SATURADA EN AGUA + TARA	g	2,027.0			
(B) PESO DE LA GRAVA SATURADA	g	3,009.5			
PESO DE LA TARA	g	173.0			
(C) PESO DE LA GRAVA SATURADA EN AGUA	g	1,855.0			
GRAVEDAD ESPECIFICA BRUTA A / B-C		2.55			
GRAVEDAD ESPECIFICA BRUTA (S.S.S.) B / B-C		2.61			
GRAVEDAD ESPECIFICA APARENTE A / A-C		2.70			
PORCENTAJE DE ABSORCION (B-A x 100) / A		2.05			

AGREGADO FINO

DETALLE	UNIDAD	NUMERO DE ENSAYO			
		1	2	3	4
(A) PESO DE LA ARENA SECADA AL HORNO	g				
PESO DE LA ARENA SATURADA EN AGUA + TARA	g				
(B) PESO DE LA ARENA SATURADA	g				
PESO DE LA TARA	g				
(C) PESO DE LA ARENA SATURADA EN AGUA	g				
GRAVEDAD ESPECIFICA BRUTA A / B-C					
GRAVEDAD ESPECIFICA BRUTA (S.S.S.) B / B-C					
GRAVEDAD ESPECIFICA APARENTE A / A-C					
PORCENTAJE DE ABSORCION (B-A x 100) / A					

OBSERVACIONES: ____

REVISADO POR: ____

TECNILAB, S. A. FUNDADA EN 1973
UNA EMPRESA E. BARRANCO Y ASOC., S. A.
LABORATORIO DE SUELOS Y MATERIALES

REACTIVIDAD ALCALINA Y CONTENIDO DE SILICE

TRABAJO No. : 4-194 CLIENTE HARZA Engeenring Company MUESTRA No.
PROYECTO: Cuenca del Canal LOCALIZACION Río Indio
MUESTREADO POR Tecnilab, S. A. FECHA Feb.00 PROFUNDIDAD
PREPARADO POR Tecnilab, S. A. FECHA Feb.00 LABORATORISTA J.Vergara

ENSAYO	UNIDAD	MUESTRA N°		
		R-03	R-04	R-05
1. REACTIVIDAD ALCALINA				
Sílice disuelta	mmol/l	3.6	40.2	80.1
Reducción de alcalinidad	mmol/l	204.4	226.5	149.2
Clasificación		Inocuo*	Inocuo*	Inocuo*
2. CONTENIDO DE SILICE (SiO_2)	%	11	42.4	55.6

*El agregado se considera inocuo.

NOTA: - El ensayo de reactividad alcalina se efectuó de acuerdo a la norma ASTM C289, y El de contenido de sílice se analizó gravimétricamente luego de efectuar una fusión a 925°C con Na_2CO_3.

REVISADO POR: N.R.C

TECNILAB, S. A.
UNA EMPRESA E. BARRANCO Y ASOC., S. A.
LABORATORIO DE SUELOS Y MATERIALES
FUNDADA EN 1973

ENSAYO DE DESGASTE DE LOS ANGELES

TRABAJO No. : 4-194 CLIENTE HARZA Engineering Company
PROYECTO: Cuenca del Canal. MATERIAL: R-03
LOCALIZACION Rio Indio
MUESTREADO POR Tecnilab, S. A. FECHA Feb-00
PREPARADO POR Tecnilab, S. A. FECHA Feb-00 LABORATORISTA J.V

ESFERA: 12 DIAMETRO: 1 27/32" PESO 44.5 gr TIEMPO DE LA PRUEBA 15 min

TAMAÑO DE TAMICES		PESO ORIGINAL DE LA MUESTRA
QUE PASA	RETENIDO	(gr)
1 1/2	1	950.00
1	3/4	950.00
3/4	1/2	950.00
1/2	3/8	950.00
TOTAL:		**3800.00**

a) Peso original de la muestra: 3800.00 gr
b) Peso del material retenido en el tamiz Nº12: 2708.00 gr
c) Perdida de peso (a-b): 1092.00 gr
d) Porcentaje de desgaste: 28.70 %

Revoluciones por minuto: 33 rpm Gradación de mutra para prueba: A

TECNILAB, S. A.
FUNDADA EN 1973
UNA EMPRESA E. BARRANCO Y ASOC., S. A.
LABORATORIO DE SUELOS Y MATERIALES

ENSAYO DE DESGASTE DE LOS ANGELES

TRABAJO No. : 4-194 CLIENTE HARZA Engineering Company
PROYECTO: Cuenca del Canal. MATERIAL: R-04
LOCALIZACION Rio Indio
MUESTREADO POR Tecnilab, S. A. FECHA Feb-00
PREPARADO POR Tecnilab, S. A. FECHA Feb-00 LABORATORISTA J.V

ESFERA: 12 DIAMETRO: 1 27/32" PESO 44.5 gr TIEMPO DE LA PRUEBA 15 min

TAMAÑO DE TAMICES		PESO ORIGINAL DE LA MUESTRA
QUE PASA	RETENIDO	(gr)
1 1/2	1	750.00
1	3/4	750.00
3/4	1/2	750.00
1/2	3/8	750.00
TOTAL:		**3000.00**

a) Peso original de la muestra: 3000.00 gr
b) Peso del material retenido en el tamiz Nº12: 2370.00 gr
c) Perdida de peso (a-b): 630.00 gr
d) Porcentaje de desgaste: 21.00 %

Revoluciones por minuto: 33 rpm Gradación de mutra para prueba: A

TECNILAB, S. A.
UNA EMPRESA E. BARRANCO Y ASOC., S. A.
LABORATORIO DE SUELOS Y MATERIALES

FUNDADA EN 1973

ENSAYO DE DESGASTE DE LOS ANGELES

TRABAJO No. : 4-194 CLIENTE HARZA Engineering Company
PROYECTO: Cuenca del Canal. MATERIAL: R-05
LOCALIZACION Rio Indio
MUESTREADO POR Tecnilab, S. A. FECHA Feb-00
PREPARADO POR Tecnilab, S. A. FECHA Feb-00 LABORATORISTA J.V

ESFERA: 12 DIAMETRO: 1 27/32" PESO 44.5 gr TIEMPO DE LA PRUEBA 15 min

TAMAÑO DE TAMICES		PESO ORIGINAL DE LA MUESTRA
QUE PASA	RETENIDO	(gr)
1 1/2	1	725.00
1	3/4	725.00
3/4	1/2	725.00
1/2	3/8	725.00
TOTAL:		**2900.00**

a) Peso original de la muestra: 2900.00 gr
b) Peso del material retenido en el tamiz Nº12: 2276.00 gr
c) Perdida de peso (a-b): 624.00 gr
d) Porcentaje de desgaste: 21.50 %

Revoluciones por minuto: 33 rpm Gradación de mutra para prueba: A

Técnico de Laboratorio: J.V

TECNILAB, S. A. FUNDADA EN 1973
UNA EMPRESA E. BARRANCO Y ASOC., S. A.
LABORATORIO DE SUELOS Y MATERIALES

ENSAYO DE CARGA PUNTUAL EN ROCA

TRABAJO No. : 4-194 CLIENTE HARZA Engineering Company
PROYECTO: Cuenca del Canal.
LOCALIZACION Rio Indio
MUESTREADO POR Tecnilab, S. A. FECHA Feb. 00
PREPARADO POR Tecnilab, S. A. FECHA Feb. 00 LABORATORISTA J. Vergara

DETERMINACION DE LA CARGA PUNTUAL CORREGIDA

MUESTRA No.	TIPO DE PRUEBA	W mm.	D mm.	P kN.	De^2 mm^2.	De mm.	Is MPa.	Is_{50} MPa.
R-01	b	52.70	50.80	4.50	3408.66	58.38	1.32	1.40
R-01	b	50.80	50.30	5.00	3253.43	57.04	1.54	
R-01	b	50.55	49.70	4.75	3198.80	56.56	1.48	
R-02	b	42.00	41.70	4.50	2229.95	47.22	2.02	2.02
R-03	b	52.00	49.40	17.00	3270.69	57.19	5.20	4.60
R-03	b	47.00	42.30	12.00	2531.32	50.31	4.74	
R-03	b	46.15	44.50	12.00	2614.81	51.14	4.59	
R-04	b	42.80	33.60	12.50	1831.02	42.79	6.83	
R-04	b	47.85	38.20	12.70	2327.31	48.24	5.46	6.10
R-04	b	52.05	43.25	24.00	2866.26	53.54	8.37	
R-04	b	48.00	47.80	13.00	2921.31	54.05	4.45	
R-05	b	48.70	42.65	13.00	2644.58	51.43	4.92	4.80
R-05	i	54.53	53.65	20.00	3724.90	61.03	5.37	
R-05	b	52.40	48.75	21.00	3252.48	57.03	6.46	

OBSERVACION:

Tipo de Prueba.
a axial
d diametral
b bloque
I masa irregular
D diámetro 0.3W < D > W
P Carga de falla
Is Indice de resistencia de carga puntual

FEASIBILITY DESIGN FOR THE RÍO INDIO WATER SUPPLY PROJECT

APPENDIX C

OPERATION SIMULATION STUDIES

Prepared by

MWH

In association with

TAMS

AN EARTH TECH COMPANY

FEASIBILITY DESIGN FOR THE RÍO INDIO WATER SUPPLY PROJECT

APPENDIX C – OPERATION SIMULATION STUDIES

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1 INTRODUCTION

The evaluation of the production of the Rio Indio Project was established by simulating the operation of the existing system and the existing system plus the Rio Indio Project. The U.S. Army Corps of Engineers, Hydrologic Engineering Center developed HEC-5, Simulation of Flood Control and Water Conservation Systems Computer Program was used as the basis for the operation simulations. The simulations were preformed using English units. In the original scope of services, MWH was to perform the operation studies with the model as developed for the Reconnaissance Report. When it became apparent that modifications to the model were necessary, the existing system was to be improved to include deepening Lake Gatun by about one meter, and the Rio Indio Reservoir rule curves could be improved, the ACP took over the activity.

The operation simulation studies were performed by:

Ms. T. Atencio – Operations Specialist

Under the directions of:

Mr. J. Pascal, Task Order Manager, and
Mr. Jorge dela Guardia, Manager, Canal Capacity Projects Division

The basic input requirements of the model consist of physical data, operational data, and hydrologic time-series data consisting primarily of flow data. The physical and operational data were either developed by the ACP or carried over from the Reconnaissance Study. The hydrologic data was confirmed or re-developed by MWH and are presented in Appendix A.

In the Reconnaissance Report, the concept of hydrologic reliability was introduced. Hydrologic reliability was defined as the ability of the Panama Canal system to provide sufficient water for unrestricted operation. It is represented by a ratio of the volume of water provided to the volume of water demanded for canal operations during the designated period with no draft restrictions. The base level of reliability was taken as 99.6%, which was derived by operating the existing system (without improvements to Lake Gatun) over the period from 1919 to 1999 with a demand equal to an average of the

Appendix C
Operation Simulation

ATTACHMENTS

ATTACHMENT 1

ACP SYNOPSIS OF THE OPERATION SIMULATION STUDIES

HEC-5, Simulation of Flood Control and Water Conservation System

INTRODUCTION

The HEC-5 reservoir system model was developed by the U.S. Army Corps of Engineers' Hydrologic Engineering Center in Davis, California. The existing system model (base model) was developed in order to evaluate the capability of the present ACP system and to evaluate the effectiveness of proposed alternatives to improve the system capability and reliability.

This model performs a sequential simulation of reservoir operations given a time-series of flow. The reservoirs are defined by their storage and outflow capability. The reservoir storage is allocated to operational zones (levels) that define their usage (rule curves). The flood control zone is only used to store excess inflow, the conservation zone is used to store water to meet future demands, and the inactive zone, (dead storage) is where no releases can be made. Water demands include minimum flow goals, diversions, and hydroelectric power generation. Reservoirs are linked to other reservoirs and control points (non-reservoir locations) using routing reaches. A combination of reservoirs, control points and connecting routing reaches then define a reservoir model system.

BASE MODEL DATA

The base model represents the existing ACP water supply system. Madden and Gatun reservoirs are the primary elements of the system. HEC-5 can only process one diversion from each reservoir. The municipal water supply and the lockages for Gatun and Pedro Miguel were combined into one diversion from Gatun.

All data are in foot-pound units. The simulation time-interval is one month. However, this model could operate at time steps as small as one-hour. Given the monthly time-step, the seasonally varying data are all defined over the 12 months. There is no river-reach routing data required for the monthly time step.
The average-monthly flow data are input to Madden and Gatun. The inflow is assigned as incremental-local flow, which means that the inflow to Gatun only represents the catchment not controlled by Madden.

Madden Reservoir

The model begins with a Madden Reservoir (location 50) which receives basin inflow, provides M&I water supply, and releases flow to Gatun. Reservoir releases are assumed to pass through the hydroelectric facility and energy generation is computed up to the limits of power capacity. The outflow from Madden is "routed" to Gatun reservoir. The channel capacity was set to 1,415 m^3/s (50,000cfs) based on operation manual information; however, that value should not be a controlling factor in the monthly simulation.

Elevation-Storage: HEC-5 defines the reservoir with a set of storage-based relationships. Reservoir outflow capacity, elevation, and area are defined as a function of storage. The data between elevations 57.9m and 82.3 m (190 and 270 feet) were input on a 0.60m (two-foot interval), except for the 60.9m to 67.1 m (200 to 220 feet) range where data input is on 0.30 m (one-foot) interval. Data below elevation 57.9 m (190 feet) and above 80.5m (264 feet) were estimated by extrapolation. There are two reasons for this: one is that the reservoir operating range is between elev. 57.9m and 76.8m (190 and 252 ft), so that information within this range should be more accurate. The other reason is that the program interpolates on a straight line between points, and in order to be more precise in the part of the curve where the slope changes more rapidly, closer points will be needed.

Outflow Capacity. The outflow capacity is a constraint on releases in HEC-5. The program does not simulate gate openings, it only ensures that the release does not exceed the maximum outflow capacity of the reservoir.

Reservoir guide-curve. The guide curve for the top-of-conservation is defined by the target storage at the end of each month. The reservoir guide curve has changed over the years. Since this analysis is for the existing system, the current guide-curve was used in the model. The guide curve is a data input supplied by ACP, more information will be given on its development in following sections.

Reservoir evaporation. The evaporation for Madden is the computed reservoir evaporation, in inches, provided by ACP.

Hydropower data: The total installed capacity is 36 MW, 3 x 12 MW units; no overload is assumed. The tailwater was set at 27.1m (89 feet). The efficiency was set at 83%, based on turbine testing data and an assumed generator efficiency of 96%. No hydropower requirements were specified. A leakage value of $0.6 m^3/s$ (20 cfs) was used, as determined by the Meteorological & Hydrographic Branch (Met & Hyd) of ACP. The program will pass that value as a minimum and that flow will not be used for energy computation.

M&I Diversions: The municipal and industrial water supply is diverted from Madden to nowhere, i.e. the diverted flow leaves the system. Average monthly values for the 5-year period 1993-1997 were used. These were the last 5 complete years of information available when the base model was set in late 1998.

Gatun Reservoir

Gatun (location 40) is the next location below Madden. The input flow data to Gatun is the uncontrolled flow downstream from Madden. The total inflow is the sum of Madden releases and input flow data. The release from Gatun routes to the Caribbean Sea. As with Madden, the reservoir releases will be used to generate electrical energy. Flow diversion from Gatun includes municipal water supply and the combined lockages for Gatun and Pedro Miguel Locks.

Elevation-Storage. HEC-5 defines the reservoir same as with Madden. The data between elevations 23.5m and 27.4m (77 and 90 feet) were input on 0.30m (one-foot) interval. The data were not defined below elevation 23.5 and above elevation 27.4m, so that data was estimated by extrapolating the given curves. Reasoning for this same as that for the Madden reservoir storage data.

Outflow capacity: Criteria is the same as that for Madden. The outflow capacity was taken from information by ACP for all 14 gates open from an elevation of 23.8m to 28.0m (78 to 92 feet). Outflow capacity for elevations 21.3m and 23.5m (70 and 77 feet) were estimated using the weir equation and a coefficient of 3.0.

Reservoir Guide-Curves and Reservoir evaporation. Same considerations as those for Madden.

Hydropower data. The total installed capacity is 24 MW, 3 x 3 MW units and 3 x 5 MW units. No overload is assumed. The tailwater elevation was set at 2.7m (9 feet), based on tailwater data from unit testing. The efficiency was set at 85%, based on turbine testing data and an assumed generator efficiency of 96%. No hydropower requirements were specified. A leakage value of $0.8m^3/s$ (27 cfs) is used, information provided by ACP`s Met & Hyd Branch.

Gatun and Pedro Miguel Locks: The diversions from Gatun include the combined flow for lockages.

M&I Diversions: The diversion from Gatun for municipal and industrial water supplies is included with the flow for locks as the diversion from Gatun. Average monthly values for the 5-year period 1993 to 1997 were used. These were the last 5 years of complete information available since the base model was set in late 1998.

Flow data:

It is a requirement of the model to define the starting storage values and the corresponding date for each reservoir.

BASE MODEL VALIDATION

To validate the model configuration and data, a simulation was performed with the outflow and the inflow defined. With all the inflow and outflow defined, the simulation should yield the same results as historically recorded. A simulation run was made for the period of January 1970 to December 1997. For both reservoirs, the validation model elevations are essentially the same as the observed elevations, therefore, the model configuration is consistently accounting for the water in the system.

RESERVOIR GUIDE CURVES (RULE CURVES)

ACP has developed rule curves for the tandem operation of Gatun and Madden Lakes. The purpose of the curves is to provide a minimum draft for ships with a maximum level of reliability. Several curves have been in use throughout the more than 85 years of operation of the Canal, the existing set was implemented in the late 1970's. The development of the curves has been based on the experience with the hydrology in the Panama Canal Watershed, with a dry season extending from January through April and the rainy season from May through December. In general the intent of the curves is to use in the dry season the water stored during the rainy season and to fill both lakes by the end of the year.

Though the current set of curves was mostly developed on an empirical base, a study was conducted on April 1999 by Manuel B. Vilar from the Met & Hyd Branch of ACP to validate them. The study consisted on two distinctive approaches in simulating lake operations with the daily data from 1965 to 1994. They are an historical analysis and a stochastical analysis. The study concluded that "...it was verified that the existing rule curve has about a lower decile probability of requiring draft restrictions. That is, that about 10% of the historical events....will produce draft restrictions..."

Indio Reservoir

Lake Operation: The water surface of the lake will fluctuate from the normal operating lake level 80 m down to the minimum operating lake level at elevation 40 m with 1,300 million cubic meters of usable storage. The maximum flood lake level would be at elevation 82.5 m. The volume between the maximum flood lake level and the normal operating lake level would store floodwater.

Inflow: The Indio Lake (location 200) would receive a monthly average runoff of 25.8 m^3/s from approximately 380 km^2 of the upper portion of the watershed. The calculated discharge at the Rio Indio dam site is obtained from streamflow data of the Boca de Uracillo Hydrologic station.

Releases flow to Gatun will go through a horseshoe tunnel of 4.25 m of diameter and 8,400 m of length connected to the Panama Canal watershed. Indio reservoir will release to Gatun reservoir a minimum desired flow of 45.4 m^3/s during the months February through May. Reservoir releases are assumed to pass through the hydroelectric facility located at the end of the tunnel and energy generation is computed up to the limits of power capacity. The outflow from Indio is routed to Gatun reservoir.

Elevation-Storage: HEC-5 defines the reservoir with a set of storage-based relationships. Reservoir outflow capacity, elevation, and area are defined as a function of storage.

Outflow Capacity. The outflow capacity is a constraint on releases in HEC-5. The program does not simulate gate openings, it only ensures that the release does not exceed the maximum outflow capacity of the reservoir.

Reservoir guide-curve. The guide curve for the top-of-conservation is defined by the target storage at the end of each month. The guide curve is a data input supplied by ACP, and obtained using the 25% of active storage.

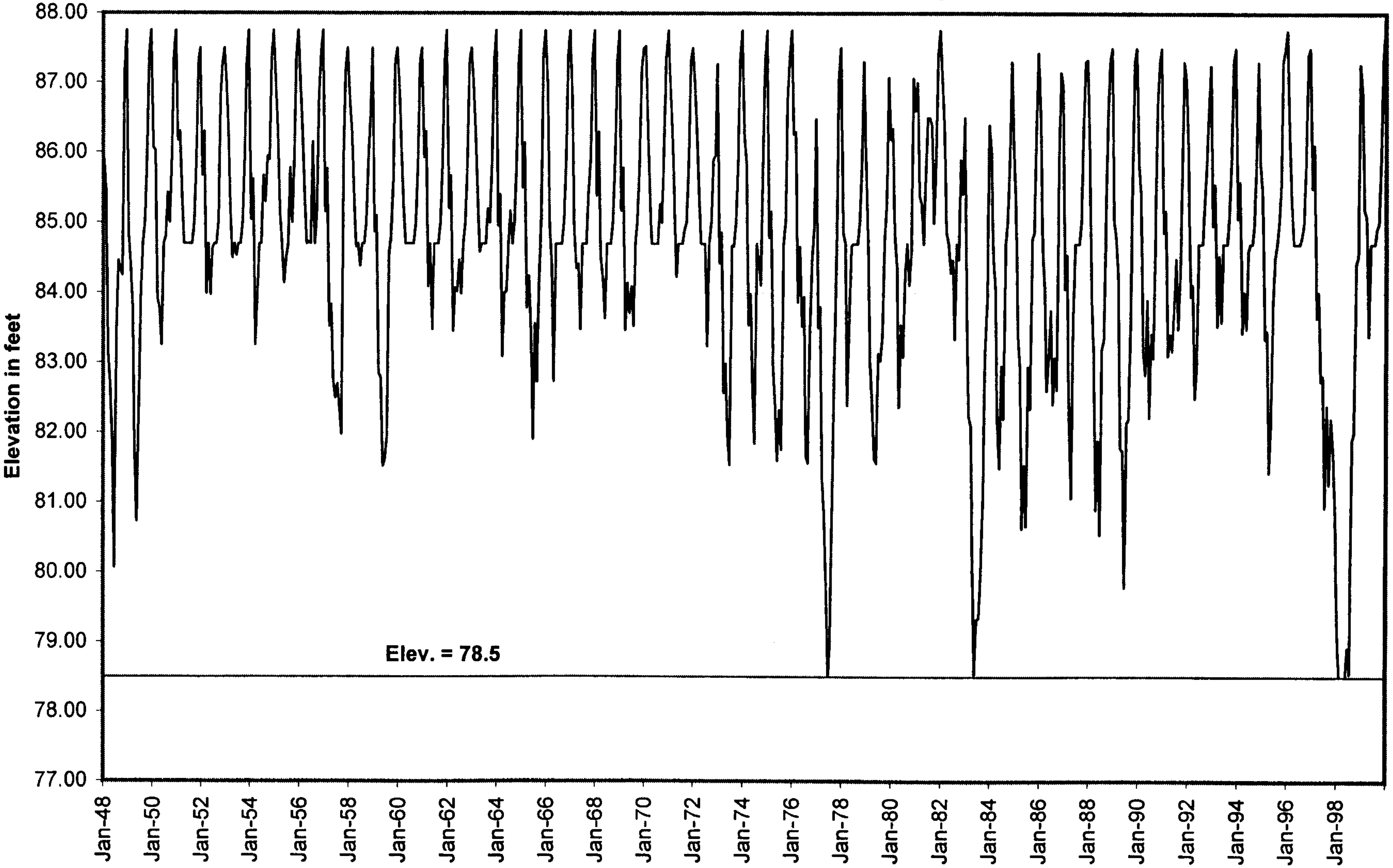
Reservoir evaporation. Because of the proximity of Rio Indio to Gatun Lake, and because of the absence of site specific information, the monthly evaporation rates established for Gatun Lake were considered appropriate for the evaporation rates of Indio lake.

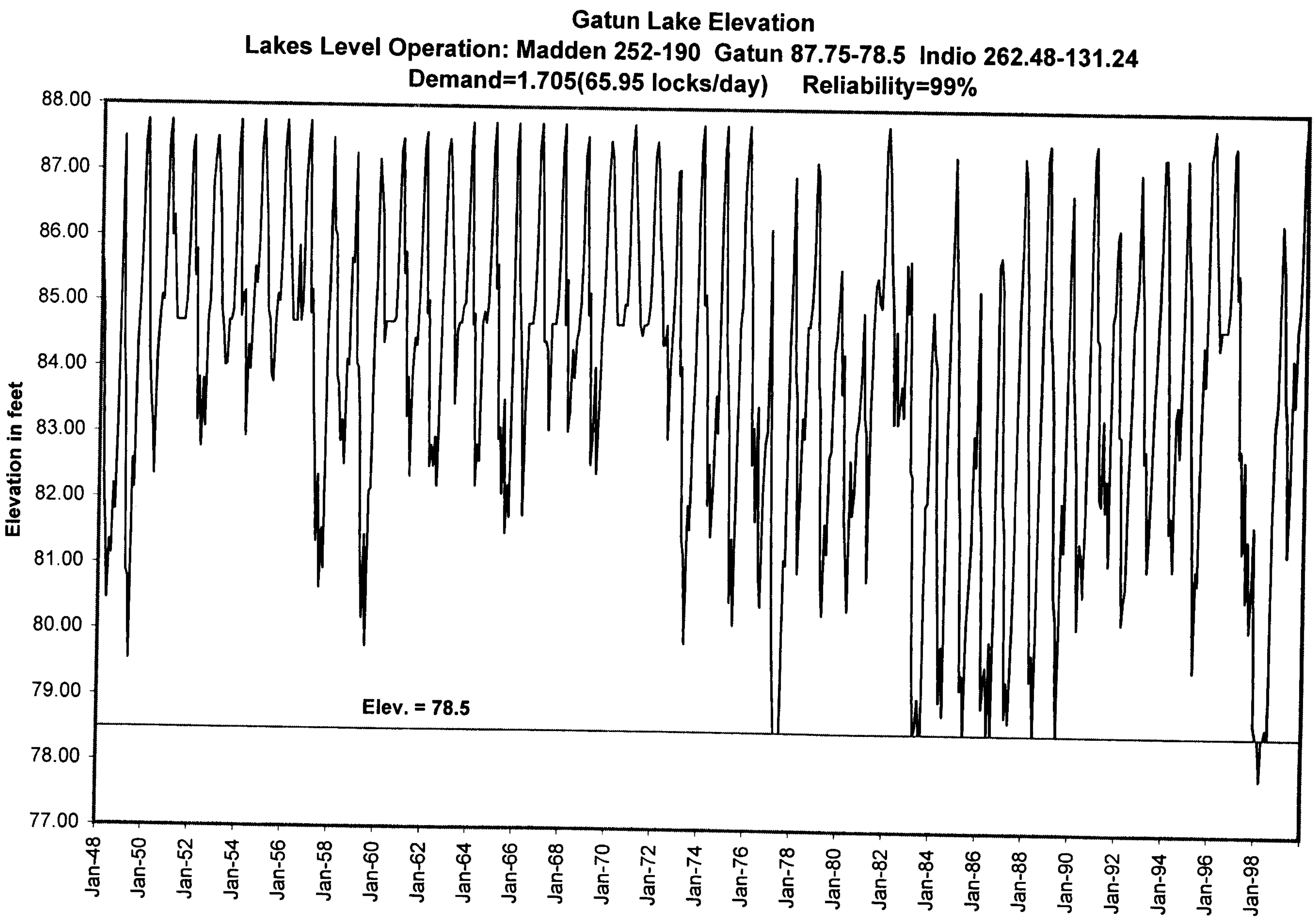
Hydropower data: HEC5 runs were executed considering a 5 MW hydropower plant at the dam and a 25MW hydropower plant at the inter basin transfer tunnel outlet. No overload was assumed. The only limitation of power output was the powerplant rated capacity. The efficiency was set at 86%. The tailwater elevation was based on the elevation at Gatun.

HEC-5 SAMPLE RESULTS WITH INDIO RESERVOIR

The next three graphs show end-of-month Gatun Lake elevations with Indio reservoir operation and for several demands and reliabilities. The goal is to avoid Gatun lake elevation to drop below the minimum established elevation, since that condition will restrict the draft of the vessels transiting the canal land thus affect their cargo-carrying capacity. These graphs are presented solely as a demonstration of the HEC-5 model capabilities.

Gatun Lake Elevation
Lakes Level Operation: Madden 252-190 Gatun 87.75-78.5 Indio 262.48-131.24
Demand=1.56 (60.34 locks/day) Reliability=99.6%




Gatun Lake Elevation
Lakes Level Operation: Madden 252-190 Gatun 87.75-78.5 Indio 262.48-131.24
Demand=1.705(65.95 locks/day) Reliability=99%
Elevation in feet
88.00
87.00
86.00
85.00
84.00
83.00
82.00
81.00
80.00
79.00
78.00
77.00
Elev. = 78.5
Jan-48
Jan-50
Jan-52
Jan-54
Jan-56
Jan-58
Jan-60
Jan-62
Jan-64
Jan-66
Jan-68
Jan-70
Jan-72
Jan-74
Jan-76
Jan-78
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Jan-82
Jan-84
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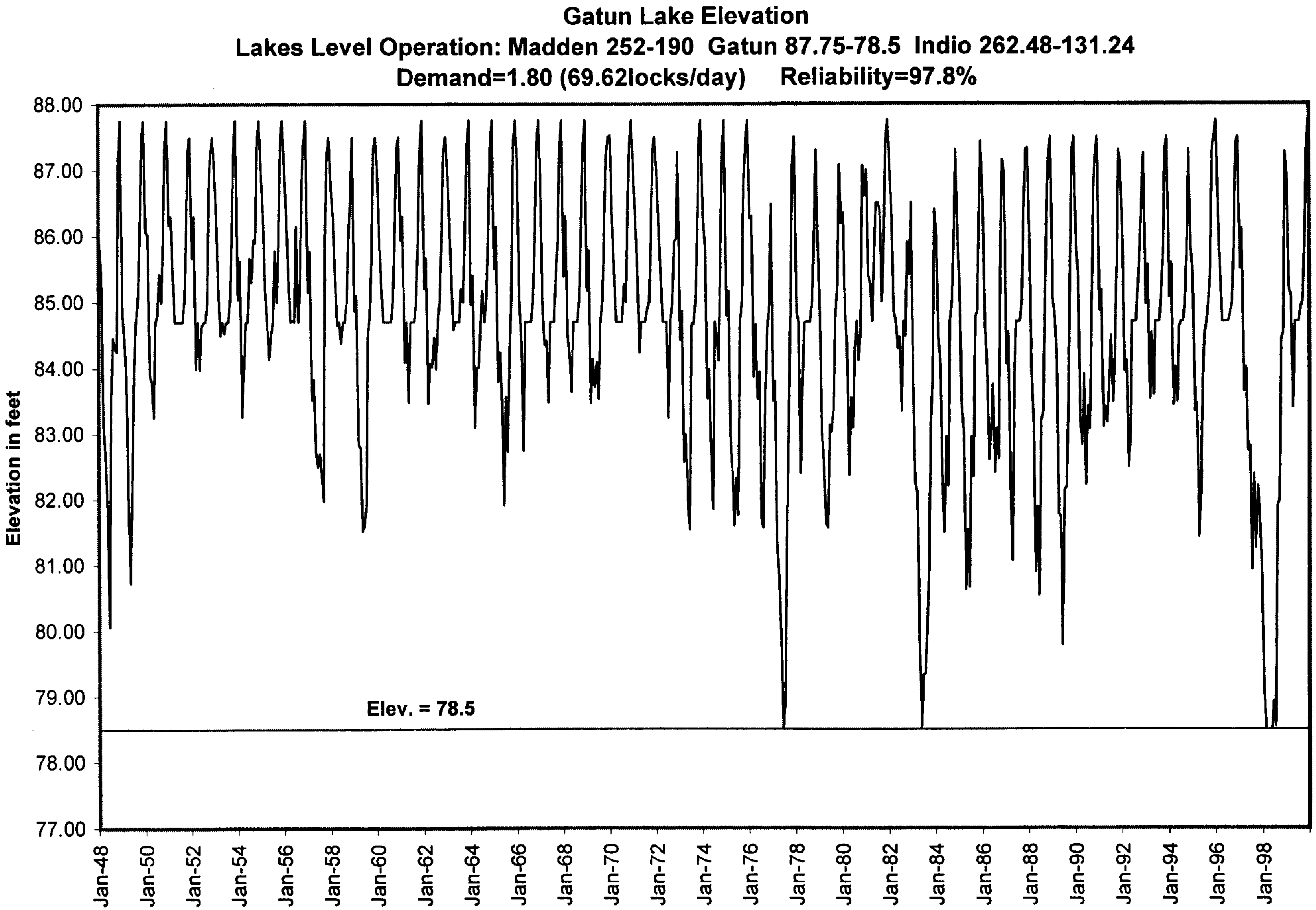
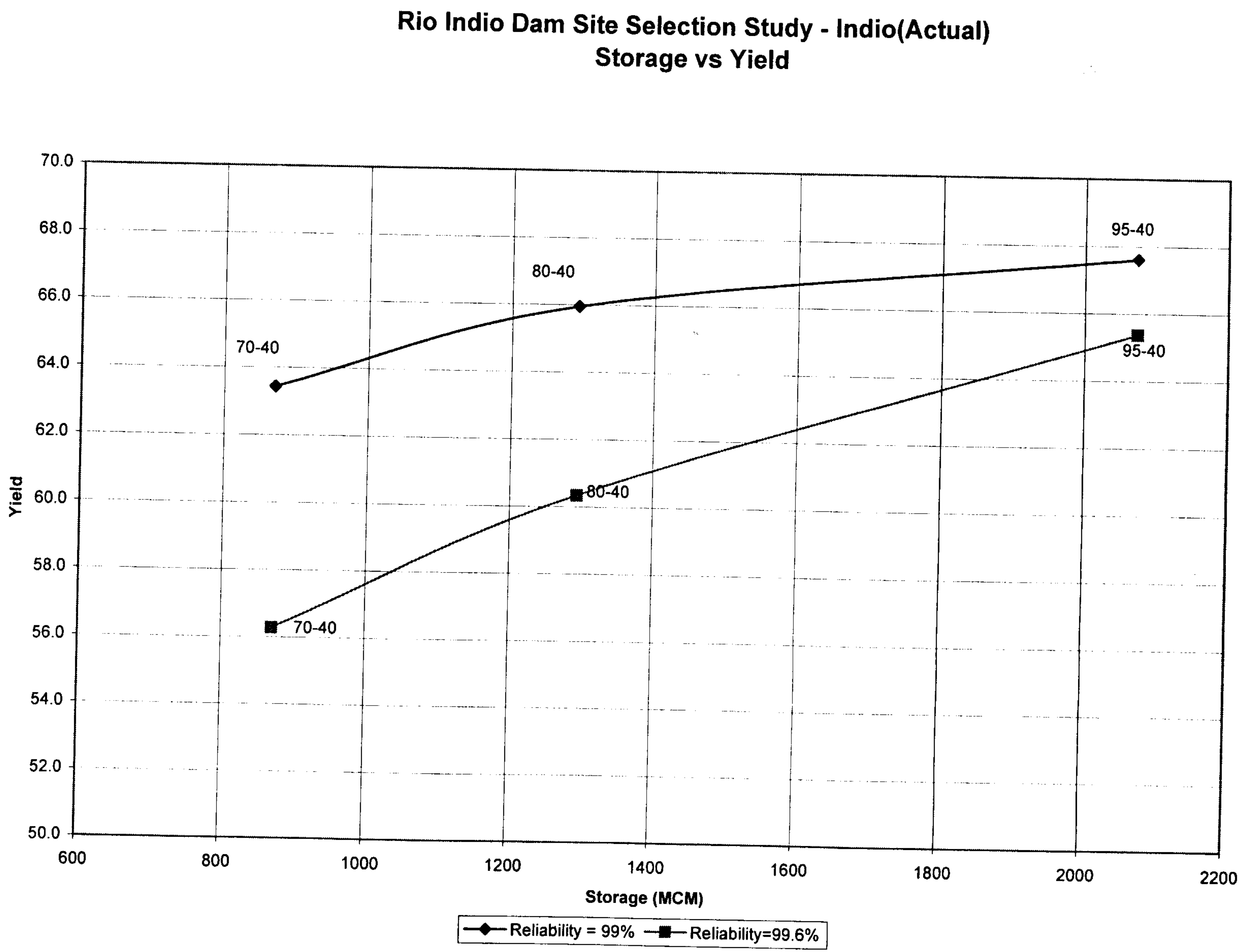
Jan-92
Jan-94
Jan-96
Jan-98



Gatun Lake Elevation
Lakes Level Operation: Madden 252-190 Gatun 87.75-78.5 Indio 262.48-131.24
Demand=1.80 (69.62locks/day) Reliability=97.8%
Elevation in feet
88.00
87.00
86.00
85.00
84.00
83.00
82.00
81.00
80.00
79.00
78.00
77.00
Elev. = 78.5
Jan-48
Jan-50
Jan-52
Jan-54
Jan-56
Jan-58
Jan-60
Jan-62
Jan-64
Jan-66
Jan-68
Jan-70
Jan-72
Jan-74
Jan-76
Jan-78
Jan-80
Jan-82
Jan-84
Jan-86
Jan-88
Jan-90
Jan-92
Jan-94
Jan-96
Jan-98

ATTACHMENT 2

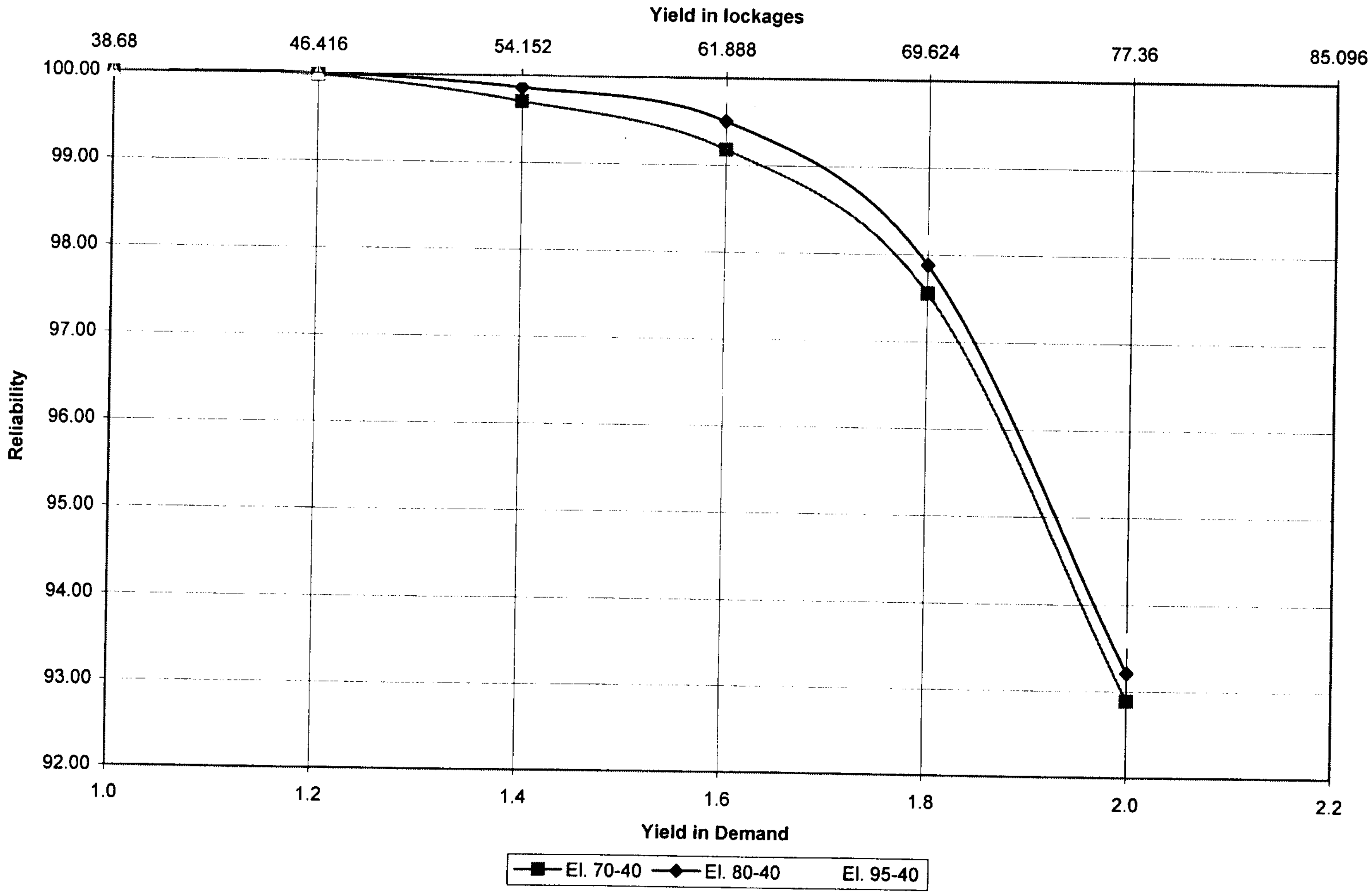
STORAGE - YIELD AND RELIABILITY-YIELD DATA


Rio Indio Dam Site Selection Study - Indio(Actual)
Storage vs Yield
Yield
70.0
68.0
66.0
64.0
62.0
60.0
58.0
56.0
54.0
52.0
50.0
600
800
1000
1200
1400
1600
1800
2000
2200
Storage (MCM)
70-40
80-40
95-40
70-40
80-40
95-40
Reliability = 99%
Reliability=99.6%

Rio Indio Dam Site Selection Study
Site: Indio(Actual)

Elevations (meters)	Storage (MCM)	Demand Factor for Reliab. 99%	Yield for Reliab. 99%	Demand Factor for Reliab. 99.6%	Yield for Reliab. 99.6%
70-40	871	1.64	63.4	1.455	56.28
80-40	1294	1.705	65.9	1.56	60.34
95-40	2074	1.748	67.6	1.69	65.37

Indio Dam Site Selection Study
Site: Indio (Actual)
Reliability vs Yield



Rio Indio Dam Site Selection Study
Site: Indio(Actual)

Elevations: 70-40

Demand	Yield	Reliability
1.000	38.68	100.00
1.200	46.42	99.98
1.400	54.15	99.70
1.600	61.89	99.18
1.800	69.62	97.56
2.000	77.36	92.89
1.455	56.28	99.60
1.640	63.44	99.00

Elevations: 80-40

Demand	Yield	Reliability
1	38.68	100
1.2	46.42	100
1.4	54.15	99.86
1.6	61.89	99.5
1.8	69.62	97.88
2	77.36	93.21
1.56	60.34	99.6
1.705	65.95	99

Elevations: 95-40

Demand	Yield	Reliability
1	38.68	100
1.2	46.42	100
1.4	54.15	100
1.6	61.89	99.81
1.8	69.62	98.14
2	77.36	93.6
1.69	65.37	99.6
1.748	67.61	1.865

ATTACHMENT 3

TYPICAL HEC-5 OUTPUT FILE

SAMPLE HEC-5 OUTPUT FILE
for
ADOPTED LEVEL OF DEVELOPMENT

Period	Date:	GAT EOP Elev	MAD EOP Elev	NDIO DA EOP Elev	GAT Diversio	GAT Div Requ	GAT Div Shor
615	1Mar 1999	85.10	223.68	153.67	5199.48	5199.48	0.00
616	1Apr 1999	83.39	229.34	138.98	5043.48	5043.48	0.00
617	1May 1999	84.69	219.95	131.24	4817.28	4817.28	0.00
618	1Jun 1999	84.70	218.16	143.17	4591.08	4591.08	0.00
619	1Jul 1999	84.70	218.14	151.43	4587.96	4587.96	0.00
620	1Aug 1999	84.92	223.88	169.46	4653.48	4653.48	0.00
621	1Sep 1999	85.00	230.25	187.15	4522.44	4522.44	0.00
622	1Oct 1999	85.90	236.00	196.36	4823.52	4823.52	0.00
623	1Nov 1999	87.30	247.00	207.78	4625.40	4625.40	0.00
624	1Dec 1999	87.75	252.00	223.70	4806.36	4806.36	0.00

Period	Date:	GAT EOP Elev	MAD EOP Elev	INDIO DA EOP Elev	GAT Diversio	GAT Div Requ	GAT Div Shor
	Sum =	52879.82	142601.42	135875.50	3006948.75	3018394.75	11446.22
	Max =	87.75	252.00	270.68	5199.48	5199.48	4739.84
	Min =	78.50	190.00	131.24	459.64	4522.44	0.00
	PMax=	12.00	24.00	276.00	3.00	3.00	603.00
	Avg =	84.74	228.53	217.75	4818.83	4837.17	18.34
	PMin=	354.00	114.00	603.00	603.00	9.00	1.00

Period	Date:	GAT Diversio	MAD Diversio	MAD Outflow	INDIO DA Outflow	CP199 Outflow
613	1Jan 1999	5188.56	288.60	3500.00	91.20	0.00
614	1Feb 1999	5187.00	293.28	500.00	1605.00	1513.80
615	1Mar 1999	5199.48	296.40	3500.00	1605.00	1513.80
616	1Apr 1999	5043.48	296.40	500.00	1605.00	1513.80
617	1May 1999	4817.28	297.96	3500.00	1523.49	1432.29
618	1Jun 1999	4591.08	293.28	3500.00	91.20	0.00
619	1Jul 1999	4587.96	293.28	3500.00	91.20	0.00
620	1Aug 1999	4653.48	291.72	3500.00	91.20	0.00
621	1Sep 1999	4522.44	291.72	2168.75	91.20	0.00
622	1Oct 1999	4823.52	280.80	2283.66	91.20	0.00
623	1Nov 1999	4625.40	283.92	2461.06	91.20	0.00
624	1Dec 1999	4806.36	285.48	9163.49	91.20	0.00

Period	Date:	GAT Diversio	MAD Diversio	MAD Outflow	INDIO DA Outflow	CP199 Outflow
	Sum =	3006948.75	181560.42	1404685.88	533817.94	476911.91
	Max =	5199.48	297.96	9163.49	3384.51	3293.31
	Min =	459.64	229.12	0.00	91.20	0.00
	PMax=	3.00	5.00	624.00	74.00	74.00
	Avg =	4818.83	290.96	2251.10	855.48	764.28
	PMin=	603.00	603.00	603.00	1.00	1.00

Period	Date:	INDIO DA Local Cu	INDIO DA Diversio	INDIO DA Change	INDIO DA Evaporat	INDIO DA Change	INDIO DA Outflow	CP199 Diversio	CP199 Outflow	INDIO PP Diversio	INDIO PP Outflow
613	1Jan 1999	655.00	0.00	655.00	40.85	614.15	91.20	91.20	0.00	-91.20	91.20
614	1Feb 1999	358.00	0.00	358.00	46.69	311.31	1605.00	91.20	1513.80	-91.20	91.20
615	1Mar 1999	271.00	0.00	271.00	44.38	226.62	1605.00	91.20	1513.80	-91.20	91.20
616	1Apr 1999	309.00	0.00	309.00	36.18	272.82	1605.00	91.20	1513.80	-91.20	91.20
617	1May 1999	867.00	0.00	867.00	22.42	844.58	1605.00	91.20	1513.80	-91.20	91.20
618	1Jun 1999	1194.00	0.00	1194.00	20.79	1173.21	1523.49	91.20	1432.29	-91.20	91.20
619	1Jul 1999	840.00	0.00	840.00	24.01	815.99	91.20	91.20	0.00	-91.20	91.20
620	1Aug 1999	1881.00	0.00	1881.00	26.72	1854.28	91.20	91.20	0.00	-91.20	91.20
621	1Sep 1999	2336.00	0.00	2336.00	29.92	2306.08	91.20	91.20	0.00	-91.20	91.20
622	1Oct 1999	1238.00	0.00	1238.00	31.58	1206.42	91.20	91.20	0.00	-91.20	91.20
623	1Nov 1999	1779.00	0.00	1779.00	31.42	1747.58	91.20	91.20	0.00	-91.20	91.20
624	1Dec 1999	2376.00	0.00	2376.00	39.20	2336.80	91.20	91.20	0.00	-91.20	91.20

Period	Date:	INDIO DA Local Cu	INDIO DA Diversio	INDIO DA Change	INDIO DA Evaporat	INDIO DA Change	INDIO DA Outflow	CP199 Diversio	CP199 Outflow	INDIO PP Diversio	INDIO PP Outflow
	Sum =	568887.00	13244.18	555642.88	28742.55	526900.25	533817.94	56908.48	476911.91	-70152.79	70152.79
	Max =	3593.00	3025.81	3593.00	76.98	3554.15	3384.51	91.20	3293.31	-91.20	3117.01
	Min =	56.00	0.00	-1552.81	19.15	-1621.32	91.20	91.20	0.00	-3117.01	91.20
	PMax=	335.00	277.00	1.00	278.00	1.00	74.00	1.00	74.00	1.00	277.00
	Avg =	911.68	21.22	890.45	46.06	844.39	855.48	91.20	764.28	-112.42	112.42
	PMin=	424.00	1.00	1.00	606.00	1.00	1.00	1.00	1.00	277.00	1.00

Period	Date:	INDIO DA Diversio	CP199 Diversio	INDIO DA EOP Elev	INDIO DA Inflow	INDIO DA Outflow	INDIO DA Q-Spill	CP199 Outflow	INDIO PP Inflow	INDIO PP Outflow
613	1Jan 1999	0.00	91.20	177.55	655.00	91.20	0.00	0.00	91.20	91.20
614	1Feb 1999	0.00	91.20	167.90	358.00	1605.00	0.00	1513.80	91.20	91.20
615	1Mar 1999	0.00	91.20	153.67	271.00	1605.00	0.00	1513.80	91.20	91.20
616	1apr 1999	0.00	91.20	138.98	309.00	1605.00	0.00	1513.80	91.20	91.20
617	1May 1999	0.00	91.20	131.24	867.00	1523.49	0.00	1432.29	91.20	91.20
618	1Jun 1999	0.00	91.20	143.17	1194.00	91.20	0.00	0.00	91.20	91.20
619	1Jul 1999	0.00	91.20	151.43	840.00	91.20	0.00	0.00	91.20	91.20
620	1Aug 1999	0.00	91.20	169.46	1881.00	91.20	0.00	0.00	91.20	91.20
621	1Sep 1999	0.00	91.20	187.15	2336.00	91.20	0.00	0.00	91.20	91.20
622	1Oct 1999	0.00	91.20	196.36	1238.00	91.20	0.00	0.00	91.20	91.20
623	1Nov 1999	0.00	91.20	207.78	1779.00	91.20	0.00	0.00	91.20	91.20
624	1Dec 1999	0.00	91.20	223.70	2376.00	91.20	0.00	0.00	91.20	91.20

Period	Date:		INDIO DA Diversio	CP199 Diversio	INDIO DA EOP Elev	INDIO DA Inflow	INDIO DA Outflow	INDIO DA Q-Spill	CP199 Outflow	INDIO PP Inflow	INDIO PP Outflow
		Sum =	13244.18	56908.48	135875.50	555642.88	533817.94	102242.05	476911.91	70152.79	70152.79
		Max =	3025.81	91.20	270.68	3593.00	3384.51	2039.63	3293.31	3117.01	3117.01
		Min =	0.00	91.20	131.24	-1552.81	91.20	0.00	0.00	91.20	91.20
		PMax=	277.00	1.00	276.00	335.00	74.00	74.00	74.00	277.00	277.00
		Avg =	21.22	91.20	217.75	890.45	855.48	163.85	764.28	112.42	112.42
		PMin=	1.00	1.00	603.00	277.00	1.00	1.00	1.00	1.00	1.00

Period	Date:	INDIO DA Outflow	INDIO DA Q-Spill	INDIO DA Change	INDIO DA Energy G	INDIO DA Loc Incr	INDIO DA Diversio	INDIO DA Change	INDIO DA Inflow	INDIO DA Diversio	INDIO PP Inflow
613	1Jan 1999	91.20	0.00	91.20	0.00	655.00	0.00	655.00	655.00	0.00	91.20
614	1Feb 1999	1605.00	0.00	1605.00	6278.50	358.00	0.00	358.00	358.00	0.00	91.20
615	1Mar 1999	1605.00	0.00	1605.00	6172.46	271.00	0.00	271.00	271.00	0.00	91.20
616	1Apr 1999	1605.00	0.00	1605.00	4775.61	309.00	0.00	309.00	309.00	0.00	91.20
617	1May 1999	1523.49	0.00	1523.49	3932.30	867.00	0.00	867.00	867.00	0.00	91.20
618	1Jun 1999	91.20	0.00	91.20	0.00	1194.00	0.00	1194.00	1194.00	0.00	91.20
619	1Jul 1999	91.20	0.00	91.20	0.00	840.00	0.00	840.00	840.00	0.00	91.20
620	1Aug 1999	91.20	0.00	91.20	0.00	1881.00	0.00	1881.00	1881.00	0.00	91.20
621	1Sep 1999	91.20	0.00	91.20	0.00	2336.00	0.00	2336.00	2336.00	0.00	91.20
622	1Oct 1999	91.20	0.00	91.20	0.00	1238.00	0.00	1238.00	1238.00	0.00	91.20
623	1Nov 1999	91.20	0.00	91.20	0.00	1779.00	0.00	1779.00	1779.00	0.00	91.20
624	1Dec 1999	91.20	0.00	91.20	0.00	2376.00	0.00	2376.00	2376.00	0.00	91.20

Period	Date:	INDIO DA Outflow	INDIO DA Q-Spill	INDIO DA Change	INDIO DA Energy G	INDIO DA Loc Incr	INDIO DA Diversio	INDIO DA Change	INDIO DA Inflow	INDIO DA Diversio	INDIO PP Inflow
	Sum =	533817.94	102242.05	431576.50	3128160.75	568887.00	13244.18	555642.88	555642.88	13244.18	70152.79
	Max =	3384.51	2039.63	2177.21	18600.00	3593.00	3025.81	3593.00	3593.00	3025.81	3117.01
	Min =	91.20	0.00	91.20	0.00	56.00	0.00	-1552.81	-1552.81	0.00	91.20
	PMax=	74.00	74.00	1.00	139.00	335.00	277.00	1.00	335.00	277.00	277.00
	Avg =	855.48	163.85	691.63	5013.08	911.68	21.22	890.45	890.45	21.22	112.42
	PMin=	1.00	1.00	1.00	1.00	424.00	1.00	1.00	277.00	1.00	1.00

Period	Date:	INDIO PP Outflow	INDIO PP Q-Spill	INDIO PP Change	INDIO PP Energy G
613	1Jan 1999	91.20	0.00	91.20	699.00
614	1Feb 1999	91.20	0.00	91.20	619.47
615	1Mar 1999	91.20	0.00	91.20	630.49
616	1Apr 1999	91.20	0.00	91.20	537.57
617	1May 1999	91.20	0.00	91.20	500.13
618	1Jun 1999	91.20	0.00	91.20	494.02
619	1Jul 1999	91.20	0.00	91.20	560.33
620	1Aug 1999	91.20	0.00	91.20	630.27
621	1Sep 1999	91.20	0.00	91.20	690.38
622	1Oct 1999	91.20	0.00	91.20	779.78
623	1Nov 1999	91.20	0.00	91.20	804.07
624	1Dec 1999	91.20	0.00	91.20	898.21

Period	Date:	INDIO PP Outflow	INDIO PP Q-Spill	INDIO PP Change	INDIO PP Energy G
	Sum =	70152.79	11763.82	58388.86	574869.00
	Max =	3117.01	2821.78	299.79	3720.00
	Min =	91.20	0.00	91.20	465.51
	PMax=	277.00	277.00	1.00	37.00
	Avg =	112.42	18.85	93.57	921.26
	PMin=	1.00	1.00	1.00	604.00

Appendix D
Project Facilities

Part 1
Dam Site Selection

FEASIBILITY DESIGN FOR THE RÍO INDIO WATER SUPPLY PROJECT

APPENDIX D
PART 1

DAM SITE SELECTION

Prepared by

MWH

In association with

TAMS
AN EARTH TECH COMPANY

FEASIBILITY DESIGN FOR THE RÍO INDIO WATER SUPPLY PROJECT

APPENDIX D, PART 1 – DAM SITE SELECTION

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1 Storage Yield Information provided by the ACP
2 Additional Socio-economic Data provided by the ACP

1 INTRODUCTION

In their Reconnaissance Report (1), the USACE selected a dam site with a view toward maximizing the water development potential of the project at a minimum cost. They selected a site that minimized the size of the dam and saddle dams and could contain a reservoir that would regulate, to a large extent, the entire runoff of the Río Indio. The disadvantage of the site is that it floods a large area including the two minor but significant population centers of El Limón and Boca de Uracillo and a potentially significant archaeological site.

Because little documentation was provided for the selection, a site selection study was performed and documented to serve as the basis of an alternatives analysis in the environmental impact assessment. A series of alternative sites were identified and compared to confirm the site selected in the Reconnaissance Report or to identify other reasonable alternatives that might be compared at feasibility level.

2 OBJECTIVE AND SCOPE OF SERVICES

The objective of this study is for the Contractor to identify, screen, rank and compare alternative dam sites and reservoir configurations that would provide quantities of water of similar volume and yield to the dam site identified as preferred in the Reconnaissance Study developed by USACE.

The scope of work was developed in two phases, screening and analysis of preferred sites. The scope items are summarized as follows:

Screening Phase

1. Using the best available mapping, MWH and ACP shall identify potential damsites.

2. Develop estimates for reservoir area/volume curves for comparative yield analysis for each dam site identified.

3. Develop and submit a list of screening criteria, assumptions and proposed weights to be used for dam site identification and for selection of best water supply levels.

4. Develop a site inspection and assessment plan indicating the site assessment actions, techniques and methods to be implemented at the identified dam sites.

5. Select proposed "best" water supply level for each dam site using curves of dam cost index.

6. Inspect and review the identified dam sites to assess technical factors and environmental impacts relative to screening and weight criteria. The Contractor shall be accompanied to the site inspections by ACP staff.

7. Working in conjunction with ACP environmental and engineering personnel, assess, screen and rank the identified dam sites to determine the most and least preferred sites.

8. Develop a site inspection and assessment report on each identified dam site visited.

Analysis of Preferred Damsites Phase

1. Prepare preliminary arrangements and drawings for a CFRD, indicating precise locations for diversion works, spillway and low-level outlets at each site.

2. Develop comparative cost estimates for each selected dam site by ratios taken from cost estimates prepared for the site identified originally as the most preferred in the Reconnaissance Study.

3. Perform and document technical/cost comparison between the selected sites.

4. Prepare draft section of Feasibility Report and submit to ACP for review; this shall present and discuss study methodology, criteria, assumptions, criteria, technical analysis, professional value judgments and support data, as well as results to the ACP.

5. Incorporate findings and ACP comments into final dam site location section of Feasibility Report.

3 SCREENING PHASE

3.1 Identification of Sites

Six alternative dam sites were identified during a map study using the best available mapping, which consisted of 1:50,000 scale topographic maps. The sites were located over a reach of river from about 17 km downstream of the site selected for the Reconnaissance Report (Recon site) to 10 km upstream of the site. The sites and a legend giving the distance upstream form the mouth in river miles and the coordinates of the axes are presented on Exhibit 1. The most downstream site was selected as the farthest downstream site that could impound a significant reservoir. The most upstream site is immediately upstream from the confluence of the Río Indio and the Río Uracillo. Moving farther upstream would significantly lessen the project yield.

In a meeting with the ACP, the six alternatives were reduced to four as follows:

Table 1 Alternative Dam Sites

Site	Distance Above Mouth (river km)	Comment
Alt. 1	12.5	Accepted. Most downstream site
Alt. 2	15.2	Rejected in favor of Alt. 3. A dam at this site would be larger than at the Alt 3 site with no additional benefits to offset the cost.
Alt. 3	16.0	Accepted.
Alt. 4	28.1	Rejected. Can be considered in final design as an alternative axis to the Recon site
Recon Site	29.8	USACE site
Alt.5	33.5	Accepted. Site is upstream from El Limón.
Alt. 6	40.1	Accepted. Site is upstream of Boca de Uracillo and is the last site that can fully develop the potential yield of the basin.

3.2 Reservoir Area-Capacity Curves

Elevation-area and elevation-volume curves were generated for each of the four alternate dam sites. Reservoir areas were measured from 1:50,000-scale topographic maps provided to MWH in digital format by Corps of Engineers, Mobile District; the contour interval on the maps is 20-m; areas were measured using the 'area' command in Autocad. Reservoir volumes were estimated from these areas using the following formula:

$$V = \frac{H_2 - H_1}{3}\left(A_1 + A_2 + \sqrt{A_1 A_2}\right)$$

These 'measured' data points from the 20-m contours were plotted, and a curve was fit through the points. Area and volume data at intermediate elevations was measured from the curves. The elevation-area curves are presented on Exhibit 2; the elevation-volume curves are presented on Exhibit 3.

3.3 Screening Criteria

For the initial screening, it was decided to use the following criteria:

- Cost relative to the Recon Site cost
- Yield
- Economic cost of water
- Area inundated
- Number of persons directly affected
- Archaeological impacts

The assignment of weights to the criteria was not done for the initial screening. It was deferred to a second-level screening, if necessary.

Cost curves based on comparative-level cost estimates were prepared for the accepted sites. The cost estimates for each site were developed using the basis of the relatively detailed estimate for the Recon site and the total volume of fill as estimated from 1:50,000 scale maps. The estimates were refined to account for differences in spillway flood surcharge, diversion tunnel length and spillway length. A curve indicating the ratio of the comparative costs of all levels of all alternatives to the detailed estimated of cost at the Recon site is presented on Exhibit 4. This type of presentation (ratio rather than absolute cost) was selected to eliminate the confusion of presenting comparative costs and because the screening evaluation is based primarily of the relative merits of each alternative site. The construction cost for the recon site is estimated to equal about $200 million for this analysis.

System yield was estimated at each site using a HEC-5 model developed during the Recon Report and modified during these studies. The yields are represented as a multiple of the current demand in lockages/day with a volumetric reliability of 99.6 %. In addition, additional runs were made to assess the system yields for a range of reliabilities. A summary of the results and curves of reliability versus yield and storage versus yield for all five sites is presented in Attachment 1.

The economic cost is presented as the discounted construction and annual cost divided by the discounted yield. The analysis was done for a 50-year period from the beginning of construction and using a 12% discount rate. The annual cost was estimated to equal 1% of the construction cost.

The area inundated was estimated at the crest elevation. It is likely that a buffer zone will be required around the reservoir that will increase this value. The population impacted was based on information supplied by the ACP. The supplied values were increased at a rate of 2% per year to account for net population gains.

It has been determined that a significant archaeological site may exist in Boca de Uracillo. As an evaluation parameter, the sites were rated as impacting the site or not impacting the site.

3.4 Best Supply Level

Based on the comparative-level costs and yields, a best supply level was selected for each site except for Alternative 5. The best supply level is an approximation of the optimum development and was selected based on estimates of the economic cost of water and the yield/storage relationships indicated by the yield curve. No value was estimated for Alternative 5 because, over the range investigated, the economic cost of water was about the same regardless of the yield.

3.5 Site Visit

As part of the Río Indio Dam Site Selection Study, we performed a field reconnaissance of the river valley from upstream of its confluence with the Río Uracillo to the river discharge in the Caribbean Sea. The field reconnaissance consisted of two parts, a helicopter flyover from Las Tres Hermanas site to upstream of the Boca de Uracillo town, and downstream to the Boca del Río Indio, and a motorized boat river traverse from Boca del Río Indio to upstream from Boca de Uracillo along both the Río Indio and the Río Uracillo.

The potential dam sites were identified as part of Task 1 of the Screening Phase of the Site Selection Study. Seven sites were identified and discussed with ACP to narrow the selection to five sites. Their location and basic data is presented elsewhere. The sites selected for further consideration were Alternative 1, Alternative 3, RECON/Alt. 4, Alternative 5 and Alternative 6.

3.5.1 Conclusions of the Site Visit

- Although dams of different heights and widths could be constructed at all identified sites, the RECON/Alt. 4 site located in the river reach from El Limón town to about two kilometers downstream of the town is the only reach where both abutments are visible from river level, and that present, both from the air and the ground, a clear morphology favorable for placement of a dam.

- Geologically and geotechnically, as judged from the materials exposed at river level, there are no advantages favoring one site over the other, or disadvantages eliminating any of the sites. Rock outcrops were identified at or near all dam sites, although the lower dam sites are covered with more alluvium than the sites upstream.

3.5.2 Itinerary

Monday, 16 December 2002.
Helicopter flight to Tres Hermanas.
Helicopter flyover of the Río Indio sites.
Motorized boat traverse of the river.
Overnight at Tres Hermanas.

Tuesday, 17 December 2002.
Helicopter flight to Panama City.

3.5.3 Participants

Javier Guerra, ACP
Carlos A. Jaramillo, MWH.
Zuleyka Mojica, ACP.
Rogelio Pinilla, ACP.

3.5.4 Flyover Observations and Valley Morphology

The valley morphology was observed from the helicopter during the flyover of the valley. Río Indio runs through a relatively wide valley from the confluence with Río Uracillo to its mouth in the Caribbean Sea. The Río Indio, in the reach of interest, flows mostly north, forming an asymmetrically shaped narrow valley incised into a relic plain that exhibits nearly 100 meters of relief. At times the river runs through steep terrace bluffs, but these only extend up a few meters, usually just three meters, and the topography above this elevation is one of narrow plains and low rolling hills with a few higher ridges.

- Alternative 1: This site is in a broad valley with low abutments covered with thick vegetation. The slope of the abutments appears to be mild. The bottom of the valley is an alluvial plain. There is a large creek upstream in the right abutment.

- Alternative 3: This site is also in a broad valley with low abutments, similar to Alternative 1. The second branch of this site was not inspected.

- Recon/Alt. 4 Alternative: This site is a narrower section of the valley, without the wide alluvial plain in the bottom of the valley. The abutments are steeper and reach higher elevations than at Alt. 1 and Alt. 3.

- Alternative 5: This site is again in a wider section of the valley, although the abutments are relatively steep and reach a higher elevation.

- Alternative 6 - Uracillo: This site is formed by mildly sloping abutments, with a narrow valley bottom, and little alluvial material confined to some high terraces.

- Alternative 6 - Indio: This site is more along the way of the other Río Indio sites, with a relatively wide valley covered with alluvial material.

3.5.5 River Traverse Observations

The party boarded an outboard powered "cayuco" at Boca del Río Indio. The traverse was fairly uneventful up to La Encantada, above Alternative 2 site because the river was wide and running through relatively straight reaches with large radius curves. Above this locality, the river became narrower and shallower with sharper turns and many submerged tree trunks.

- Alternative 1: The boat trip from the mouth of the river to Alternative 1 site took approximately 40 minutes. The river runs through a wide alluvial plain, with frequent population centers in both margins. The materials observed along the river channel were terraces of mostly red alluvial sandy clays and silts that could stand vertically in 2 m to 3 m high bluffs. No rock outcrops were observed in this reach. The bluffs, which seemed to be usually stable, had been badly eroded during the recent flood event.

- Alternative 3: Saprolites and rock outcrops were observed initially approximately 50 minutes from the mouth of the river, and downstream from Alternative 3 site. Rock outcrops were frequent upstream from this location. The rock appears to be tuffaceous sandstone and siltstone similar to the rocks observed at the Recon Alternative site. The bedrock is relatively flat laying dipping to the east. The river runs through rolling terrain with alternating low hills and plains at either one or both sides at any given point.

- Recon/Alt. 4 Alternative: Travel time to the Recon Alternative was approximately two hours. The number of population centers decreased above the Tres Hermanas site, and the river runs through a narrow valley defined by alternating terraces and high steep hills. Shallow slides were frequently observed in this reach.

- Alternative 5: A large alluvial terrace forms the left side of the valley bottom. The right side is controlled by a low ridge. This is one of the few places were gravel deposits are apparent. There are some conglomerates or cemented gravels in the right bank.

- Alternative 6 - Uracillo: Travel time to the Alternative 6, Uracillo site was about three hours. There were frequent outcrops of tuffaceous sandstone and siltstone visible in the area, but from previous studies, volcanic rocks are also expected. The river continues running through a narrow valley formed by alternating terraces and high hills, although not as prominently as in the Recon Alternative reach.

- Alternative 6 - Indio: Alternative 6, Indio site was not reachable by boat. The river channel was obstructed by toppled tree trunks at several locations. Numerous shallow landslides were visible in this reach of the river.

3.5.6 Geology

Based on previous studies, the bedrock units in the Río Indio project area consist of sedimentary and volcanic rocks. Sedimentary rocks consist of tuffaceous siltstones and sandstones, conglomerates and agglomerates with intercalated lavas. In some areas, the sedimentary rocks are stratigraphically overlain by andesite and basalt flows. The volcanic units form many of the steep hills and high plateaus that are readily apparent on topographic maps and aerial photographs.

The hills in both sides of the valley are almost entirely covered with colluvial and residual soils, and are moderately to heavily vegetated. The bedrock throughout the project area exhibits a moderate to deep weathering profile. Small outcrops of tuffaceous sandstone were observed along the river channel and it is presumed that these rock sequences form the bedrock of any of the potential damsites.

The valley floor along the river varies from approximately 200 meters wide to over a kilometer wide, and is filled with alluvial terrace deposits consisting primarily of silts and clays. The river has cut steep banks, up to 5 meters high, into the terrace materials.

A recent large precipitation event triggered large number of shallow slides in the residual soils, uncovering rock, uprooting large trees and supplying sediment to the river.

3.5.7 Construction Materials

Construction material sources would probably be the same for all alternatives, with some variations if, and when, closer sources could be identified. The main required material types are discussed below.

Alluvial deposits. Alluvial deposits were identified during the traverse, forming most of the bottom of the valley. Similar alluvial terraces were investigated near the Recon site through excavation of test pits and were found to be composed predominantly of clayey silt with some layers of clayey sand and clay. These materials do not serve as sources for clean alluvial sands and gravels, which would need to be manufactured from quarried material for all sites.

Colluvial and residual soils. Most of the bedrock along the river is covered by well-developed horizons of colluvial and residual soils. It is presumed that most of the overburden in the project area is clay-rich due to the mineralogy of the bedrock. Most of these materials could only be used as cofferdam material.

Sandstone and siltstone. Tuffaceous sandstones and siltstones are widespread throughout the project area, as indicated by the outcrops in the river channel, and quarries developed in these materials could be sources of rockfill

Andesite and basalt. Andesite and basalt rock units form many of the steep hills and high plateaus in the region. Several potential quarry sources identified during previous studies could be used as sources of rockfill and concrete aggregate.

3.5.8 Geotechnical and Geologic Design Considerations

The main geological and geotechnical design considerations taken into account during the field reconnaissance are listed below. Fundamentally, the reconnaissance was based on engineering judgment, and previous experience in similar geological environments.

- Rock characteristics: The tuffaceous siltstones and sandstones underlying the site are relatively soft rocks. These rocks are expected to present adequate bearing capacity to support any of the structures being considered. The rocks from required excavation could be used as rockfill material but it is expected that their placement and compaction could result on production of fines impacting the zoning of the dam.

- Rock weathering: A fully developed weathering profile several meters thick is expected. Compressibility of the soils and saprolites left under the shell of the dam could have an impact on the behavior of the dam.

- Natural slope stability: No large mass movements are expected to affect any of the potential reservoirs, but the effect of intense rainfall on the stability of residual soils and saprolites, is important, as illustrated by the results of the recent precipitation event, and would affect the design.

- Aggregate availability: All aggregates need to be manufactured from quarried material, or transported from outside sources.

3.6 Assess, Screen and Rank Sites

Relative cost, yield and social information are presented for each alternative in Table 2. Additional social information developed by the ACP is presented in Attachment 2.

Table 2 Dam Site Evaluation Criteria

Criterion	Alt 1	Alt 3	Recon	Alt 5	Alt 6
Comparison at live storage equivalent to near-optimum development at the Recon Site (1,240 MCM)					
Crest Elevation (m)	60	60	85	90	101
Relative Cost to Recon Site	1.2	1.2	1.0	2.1	3.4
Yield (L/d)	15.7	13.7	16.0	16.2	13.7
Economic Cost	$.027	$.031	$.022	$.046	$.089
Area Inundated (km^2)	101	92	47	44	40
No of persons directly impacted	3,900	3,500	2,000	1,600	1,500
Comparison at "Best supply level"					
Crest Elevation (m)	68	62	85	(1)	129
Relative Cost to Recon Site	1.6	1.3	1.0		5.9
Yield (l/d)	22.7	15.7	16.0		25.7
Economic Cost	$.025	$.030	$.022		$.082
Area Inundated (km^2)	117	96	47		57
No of persons directly impacted	4,200	3,600	2,000		2,000
Archaeological impacts	Major	Major	Major	Major	Minor

(1) Indeterminate as yield and cost increase in a straight line over range of values studied and no best supply level could be determined.

On the basis of these criteria, it is concluded that the site identified in the reconnaissance report is the most suitable site for the development of the Río Indio basin for the following reasons:

- The sites downstream from the Recon site, Alternatives 1 and 3, provide about the same yield for a slightly greater cost; however, the social impact is significantly greater. Both sites 1 and 3 inundate approximately twice the area and impact almost twice the number of people. In addition, neither 1 nor 3 eliminates the major archaeological impact and neither site is more has more favorable morphology or geology.

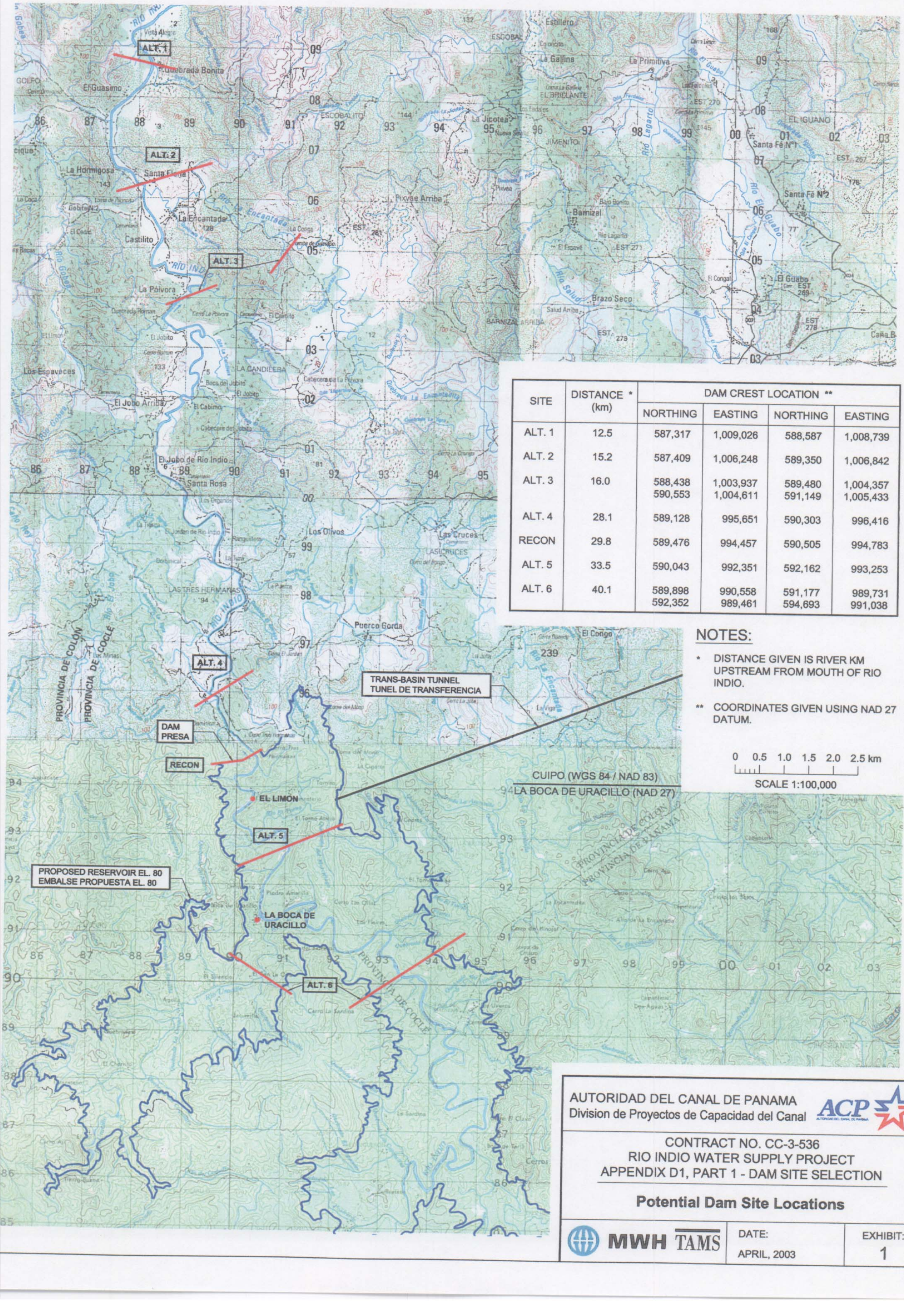
- Alternative 5 would provide the same yield as the Recon site for about twice the cost. The major advantage of Alternative 5 is that is would not inundate the town of El Limón. However, the dam would be very close to the town and the inhabitants may not be willing to stay in the town.

- Alternative 6 inundates the least amount of land, impacts the fewest people, and would be constructed upstream from the potential archaeological site. However, the cost of the development would be on the order of three times the Recon site and, therefore, was rejected.

4 ANALYSIS OF PREFERRED DAMSITE PHASE

As a result of the information developed in the screening phase, it was mutually agreed that no additional study was necessary.

EXHIBITS



SITE	DISTANCE * (km)	DAM CREST LOCATION **			
		NORTHING	EASTING	NORTHING	EASTING
ALT. 1	12.5	587,317	1,009,026	588,587	1,008,739
ALT. 2	15.2	587,409	1,006,248	589,350	1,006,842
ALT. 3	16.0	588,438 590,553	1,003,937 1,004,611	589,480 591,149	1,004,357 1,005,433
ALT. 4	28.1	589,128	995,651	590,303	996,416
RECON	29.8	589,476	994,457	590,505	994,783
ALT. 5	33.5	590,043	992,351	592,162	993,253
ALT. 6	40.1	589,898 592,352	990,558 989,461	591,177 594,693	989,731 991,038

NOTES:

* DISTANCE GIVEN IS RIVER KM UPSTREAM FROM MOUTH OF RIO INDIO.

** COORDINATES GIVEN USING NAD 27 DATUM.

0 0.5 1.0 1.5 2.0 2.5 km

SCALE 1:100,000

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

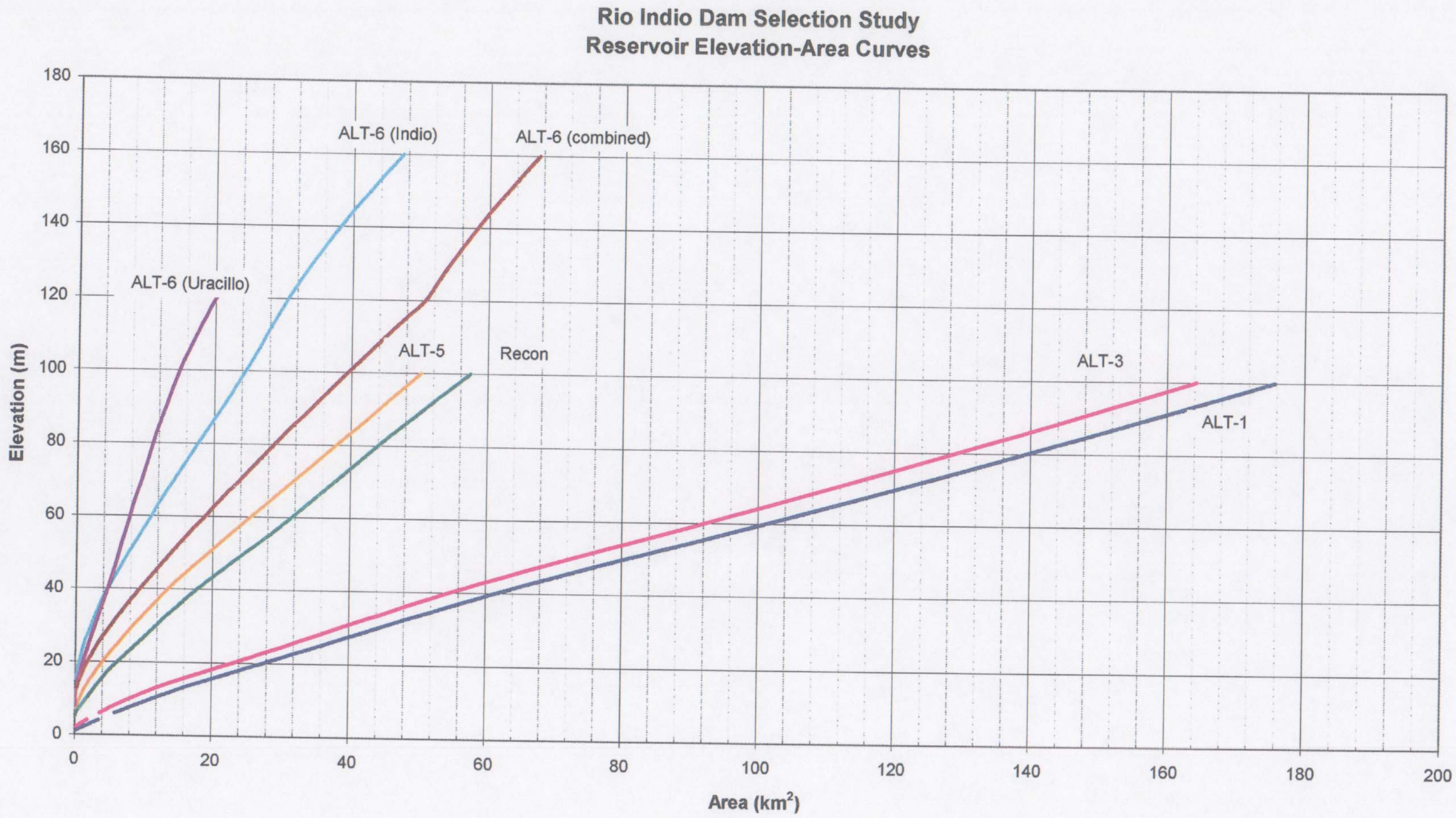
ACP

CONTRACT NO. CC-3-536
RIO INDIO WATER SUPPLY PROJECT
APPENDIX D1, PART 1 - DAM SITE SELECTION

Potential Dam Site Locations

MWH TAMS

DATE:	EXHIBIT:
APRIL, 2003	1



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

ACP

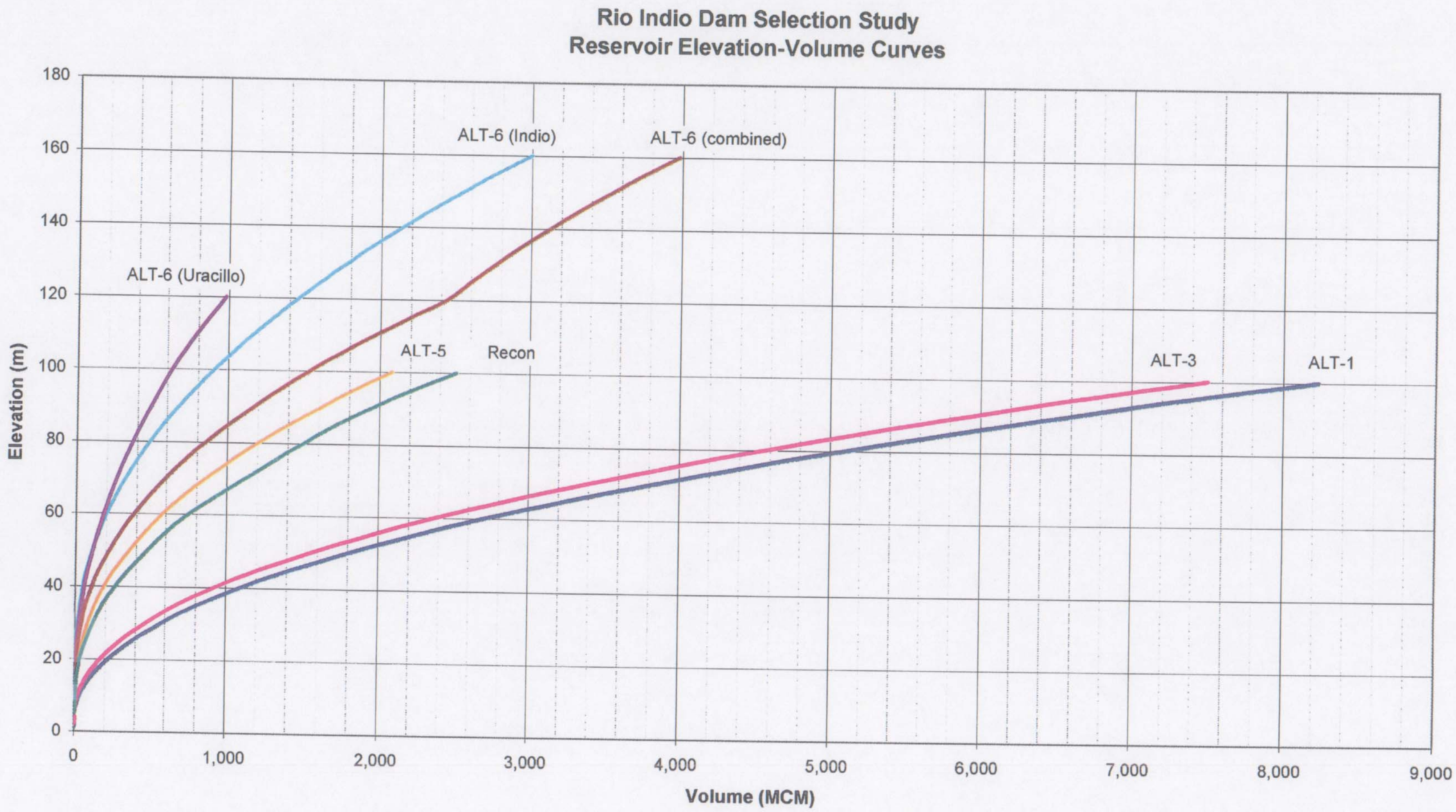
CONTRACT NO. CC-3-536
RIO INDIO WATER SUPPLY STUDY
APPENDIX D1, PART 1- DAM SITE SELECTION

Elevation - Area Curves

MWH TAMS

DATE: APRIL, 2003

EXHIBIT: 2



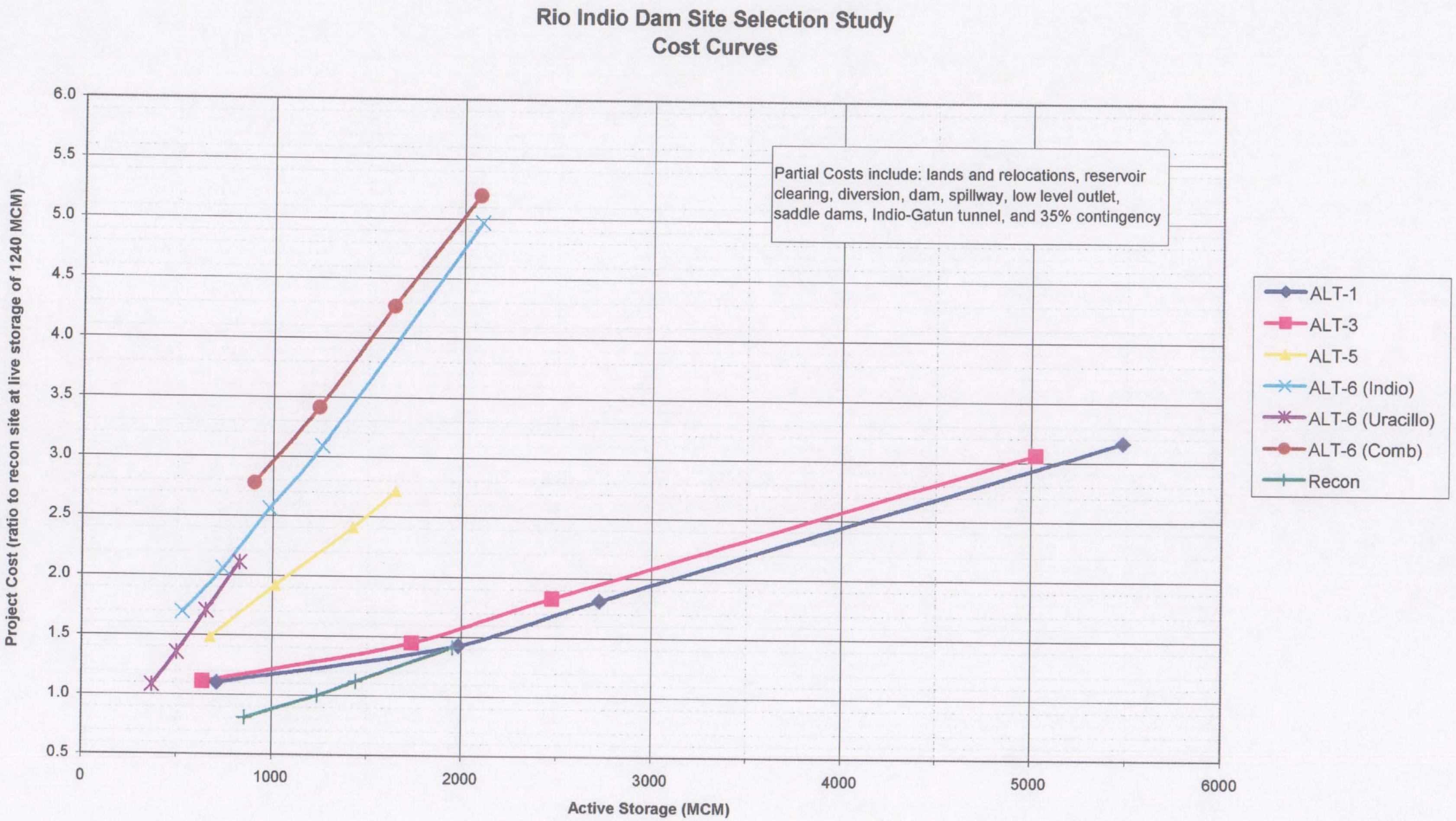
AUTORIDAD DEL CANAL DE PANAMA
Oficina de Proyectos de Capacidad del Canal

ACP

CONTRACT NO. CC-3-536
RIO INDIO WATER SUPPLY STUDY
APPENDIX D1, PART 1- DAM SITE SELECTION

Elevation - Volume Curves

MWH TAMS	DATE: APRIL, 2003	EXHIBIT: 3



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

CONTRACT NO. CC-3-536
RIO INDIO WATER SUPPLY STUDY
APPENDIX D1, PART 1- DAM SITE SELECTION

Relative Cost Curves

MWH TAMS

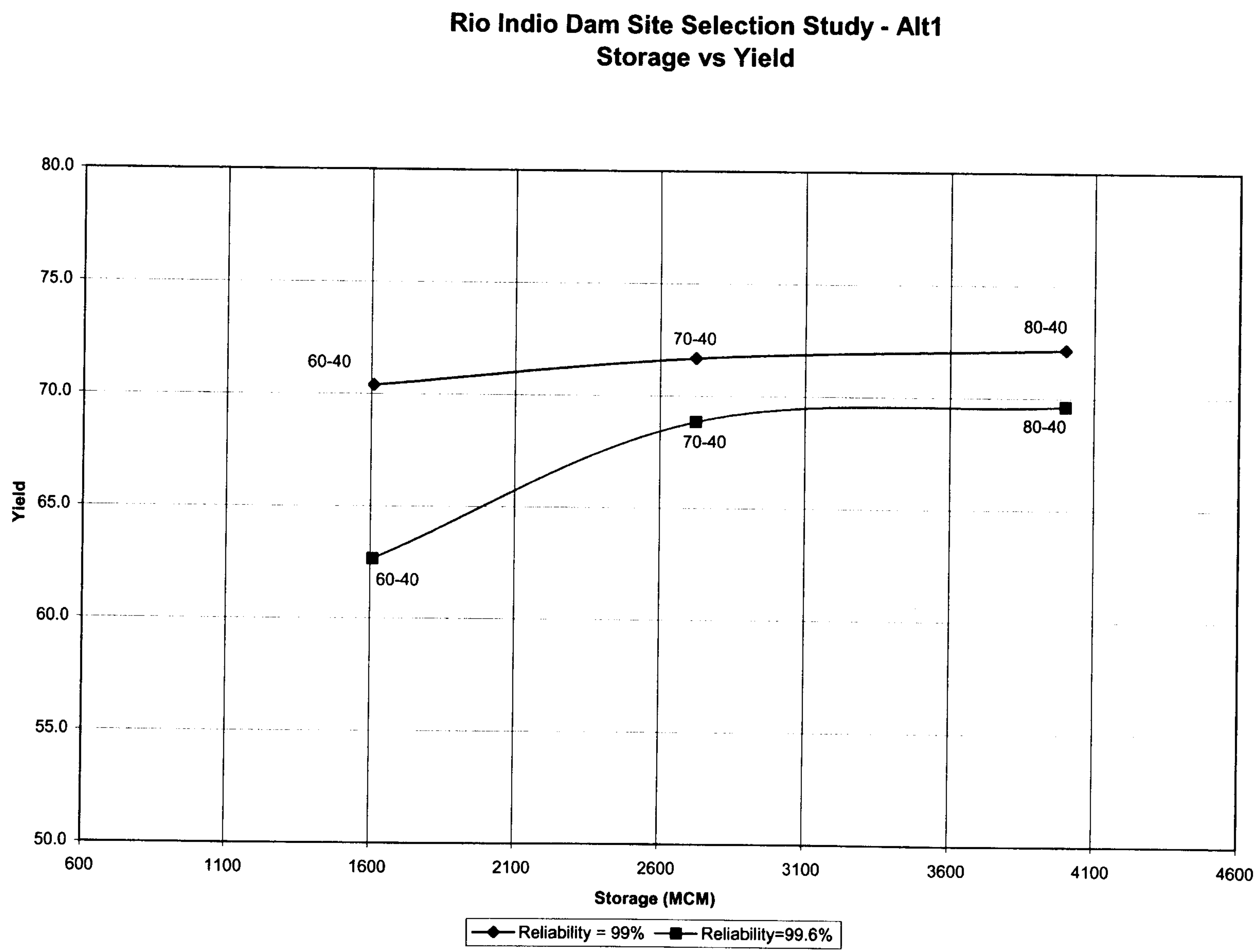
DATE: APRIL, 2003

EXHIBIT: 4

ATTACHMENTS

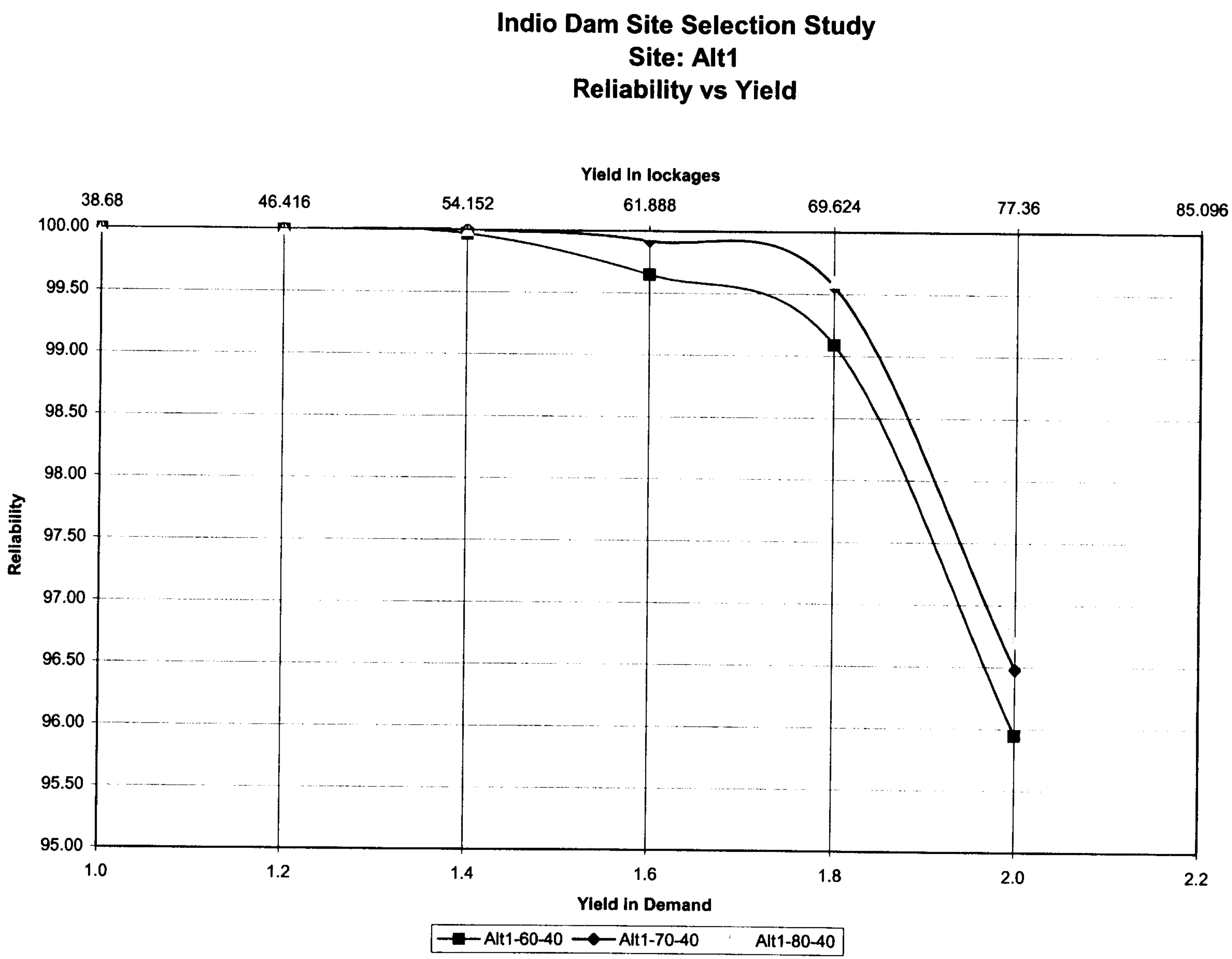
ATTACHMENT 1

STORAGE YIELD INFORMATION PROVIDED BY THE ACP


Rio Indio Dam Site Selection Study - Alt1
Storage vs Yield
80.0
75.0
70.0
65.0
60.0
55.0
50.0
Yield
60-40
70-40
80-40
60-40
70-40
80-40
600
1100
1600
2100
2600
3100
3600
4100
4600
Storage (MCM)
Reliability = 99%
Reliability=99.6%

Rio Indio Dam Site Selection Study
Site: Alt1

Elevations (meters)	Storage (MCM)	Demand Factor for Reliab. 99%	Yield for Reliab. 99%	Demand Factor for Reliab. 99.6%	Yield for Reliab. 99.6%
60-40	1608	1.82	70.4	1.62	62.66
70-40	2725	1.853	71.7	1.78	68.85
80-40	4001	1.865	72.1	1.8	69.62



Indio Dam Site Selection Study
Site: Alt1
Reliability vs Yield
Yield in lockages
38.68
46.416
54.152
61.888
69.624
77.36
85.096
100.00
99.50
99.00
98.50
98.00
97.50
97.00
96.50
96.00
95.50
95.00
Reliability
1.0
1.2
1.4
1.6
1.8
2.0
2.2
Yield in Demand
Alt1-60-40
Alt1-70-40
Alt1-80-40

Rio Indio Dam Site Selection Study
Site: Alt1

Elevations: 60-40

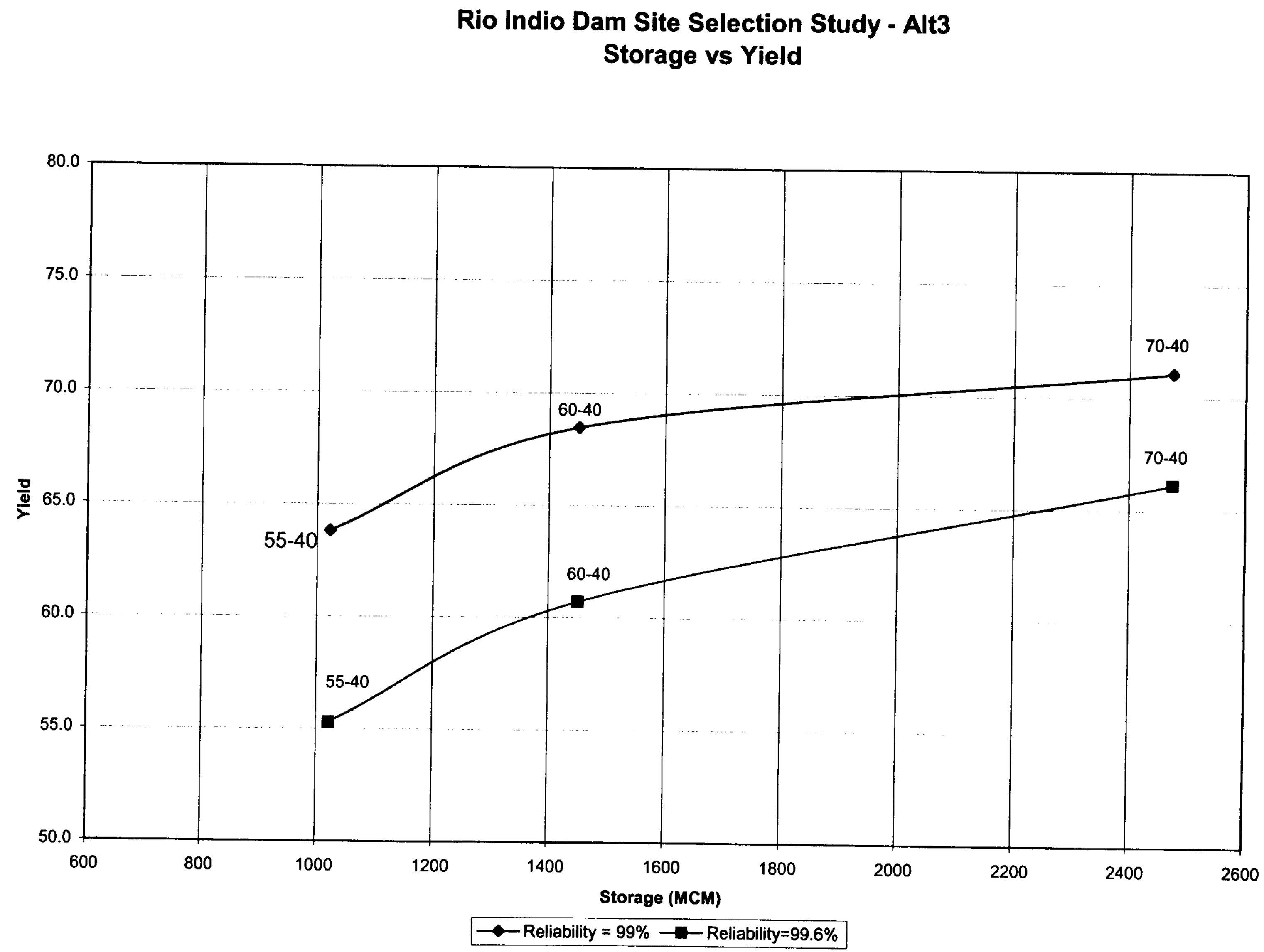
Demand	Yield	Reliability
1.000	38.68	100.00
1.200	46.42	100.00
1.400	54.15	99.97
1.600	61.89	99.65
1.800	69.62	99.09
2.000	77.36	95.95
1.620	62.66	99.60
1.820	70.40	99.00

Elevations: 70-40

Demand	Yield	Reliability
1	38.68	100
1.2	46.42	100
1.4	54.15	100
1.6	61.89	99.92
1.8	69.62	99.56
2	77.36	96.48
1.78	68.85	99.6
1.853	71.67	99

Elevations: 80-40

Demand	Yield	Reliability
1	38.68	100
1.2	46.42	100
1.4	54.15	100
1.6	61.89	99.98
1.8	69.62	99.6
2	77.36	96.7
1.8	69.62	99.6
1.865	72.14	1.865

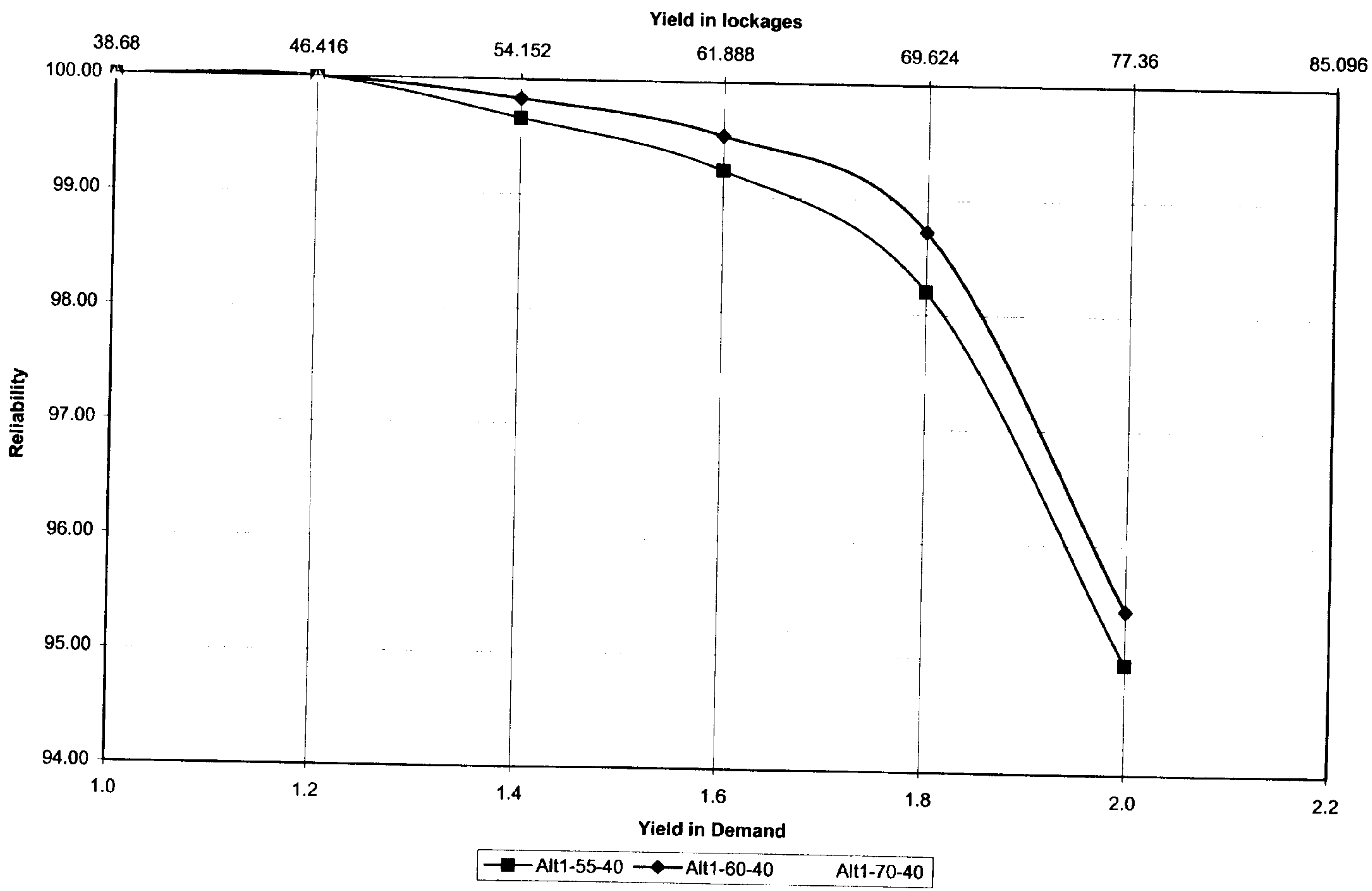


Rio Indio Dam Site Selection Study - Alt3
Storage vs Yield
80.0
75.0
70.0
65.0
60.0
55.0
50.0
Yield
600
800
1000
1200
1400
1600
1800
2000
2200
2400
2600
Storage (MCM)
55-40
60-40
70-40
55-40
60-40
70-40
Reliability = 99%
Reliability=99.6%

Rio Indio Dam Site Selection Study
Site: Alt3

Elevations (meters)	Storage (MCM)	Demand Factor for Reliab. 99%	Yield for Reliab. 99%	Demand Factor for Reliab. 99.6%	Yield for Reliab. 99.6%
55-40	1021.5	1.65	63.8	1.43	55.31
60-40	1451.6	1.77	68.5	1.57	60.73
70-40	2475.8	1.838	71.1	1.71	66.14

Indio Dam Site Selection Study
Site: Alt3
Reliability vs Yield



Rio Indio Dam Site Selection Study
Site: Alt3

Elevations: 55-40

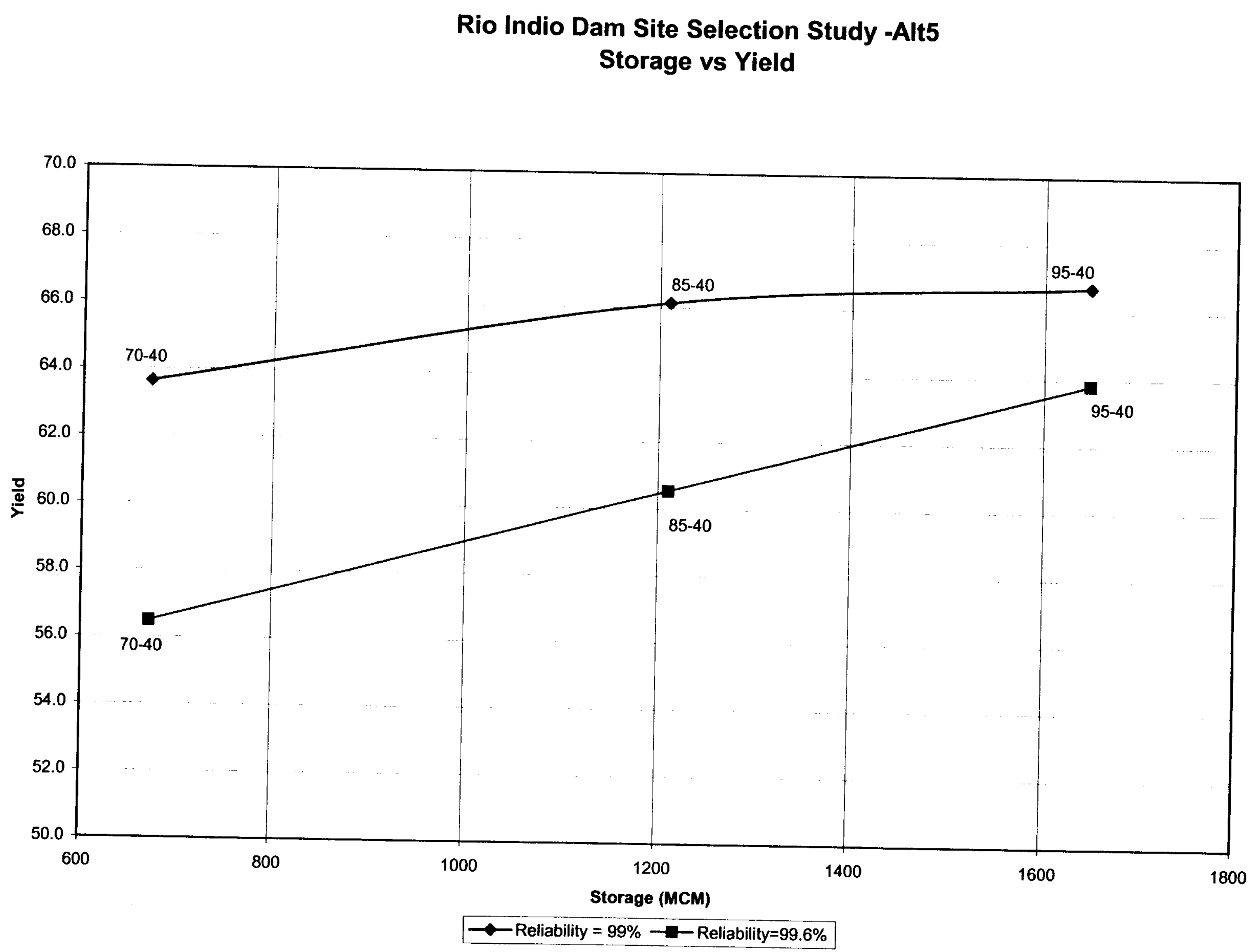
Demand	Yield	Reliability
1.000	38.68	100.00
1.200	46.42	100.00
1.400	54.15	99.66
1.600	61.89	99.23
1.800	69.62	98.20
2.000	77.36	94.96
1.430	55.31	99.60
1.650	63.82	99.00

Elevations: 60-40

Demand	Yield	Reliability
1	38.68	100
1.2	46.42	100
1.4	54.15	99.83
1.6	61.89	99.53
1.8	69.62	98.72
2	77.36	95.43
1.57	60.73	99.6
1.77	68.46	99

Elevations: 70-40

Demand	Yield	Reliability
1	38.68	100
1.2	46.42	100
1.4	54.15	99.98
1.6	61.89	99.84
1.8	69.62	99.3
2	77.36	96
1.71	66.14	99.6
1.838	71.09	1.865

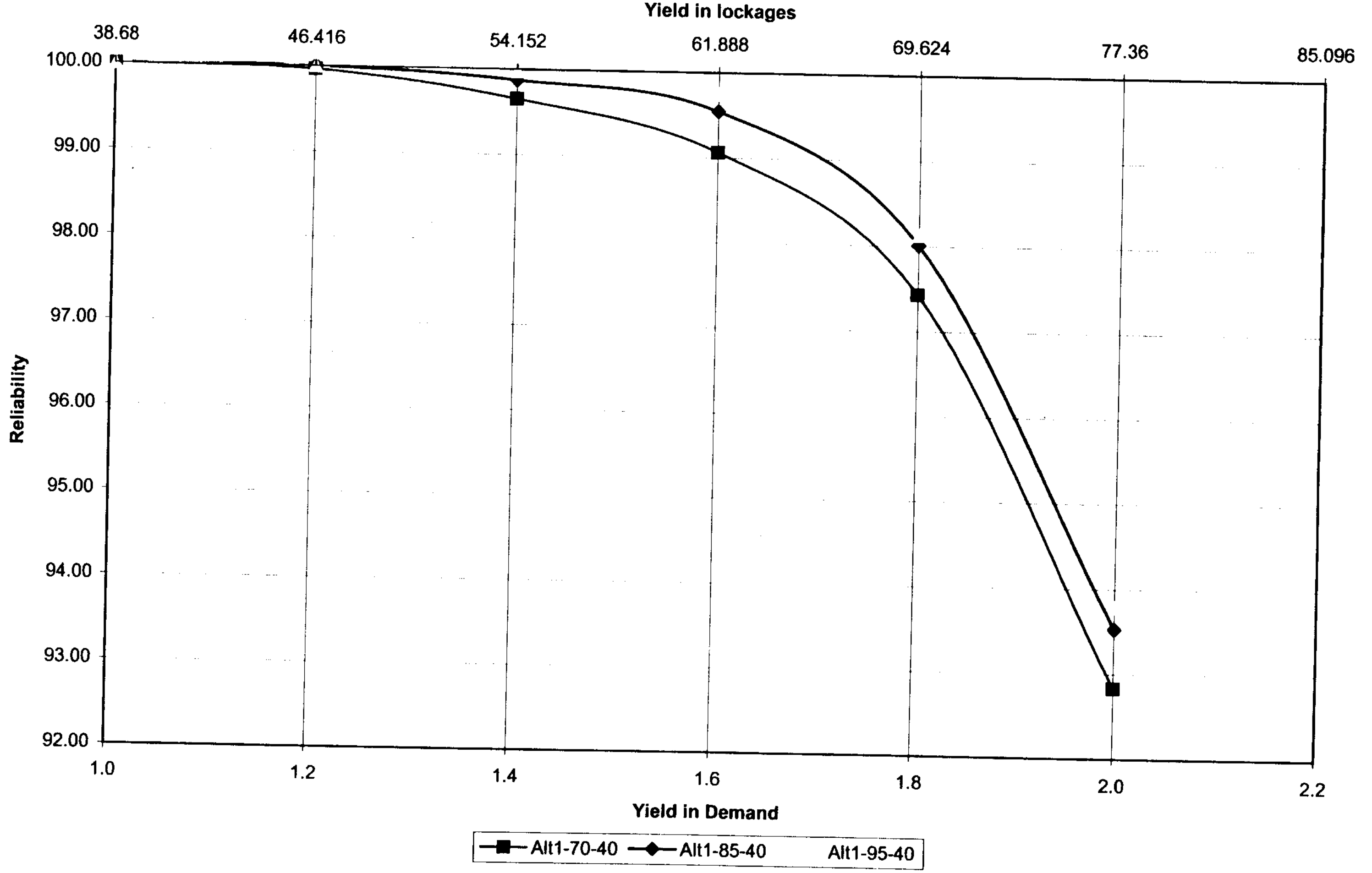


Rio Indio Dam Site Selection Study -Alt5
Storage vs Yield
70.0
68.0
66.0
64.0
62.0
60.0
58.0
56.0
54.0
52.0
50.0
Yield
600
800
1000
1200
1400
1600
1800
Storage (MCM)
70-40
85-40
95-40
70-40
85-40
95-40
Reliability = 99%
Reliability=99.6%

Rio Indio Dam Site Selection Study
Site: Alt5

Elevations (meters)	Storage (MCM)	Demand Factor for Reliab. 99%	Yield for Reliab. 99%	Demand Factor for Reliab. 99.6%	Yield for Reliab. 99.6%
70-40	673	1.645	63.6	1.46	56.47
85-40	1211	1.71	66.1	1.565	60.53
95-40	1648	1.725	66.7	1.65	63.82

Indio Dam Site Selection Study
Site: Alt5
Reliability vs Yield



Rio Indio Dam Site Selection Study
Site: Alt5

Elevations: 70-40

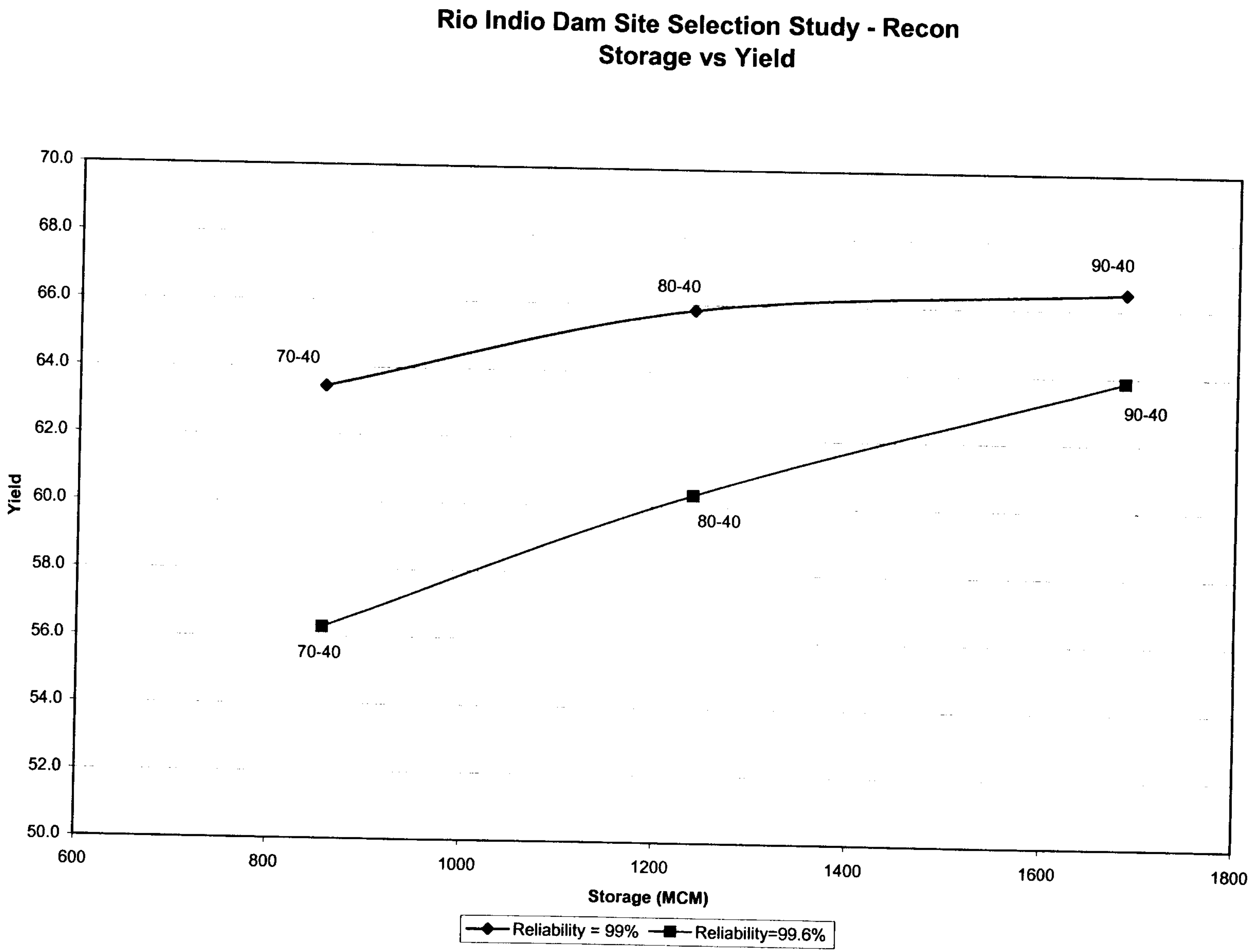
Demand	Yield	Reliability
1.000	38.68	100.00
1.200	46.42	99.96
1.400	54.15	99.65
1.600	61.89	99.06
1.800	69.62	97.43
2.000	77.36	92.83
1.430	55.31	99.60
1.620	62.66	99.00

Elevations: 85-40

Demand	Yield	Reliability
1	38.68	100
1.2	46.42	100
1.4	54.15	99.87
1.6	61.89	99.54
1.8	69.62	98.01
2	77.36	93.53
1.565	60.53	99.6
1.71	66.14	99

Elevations: 95-40

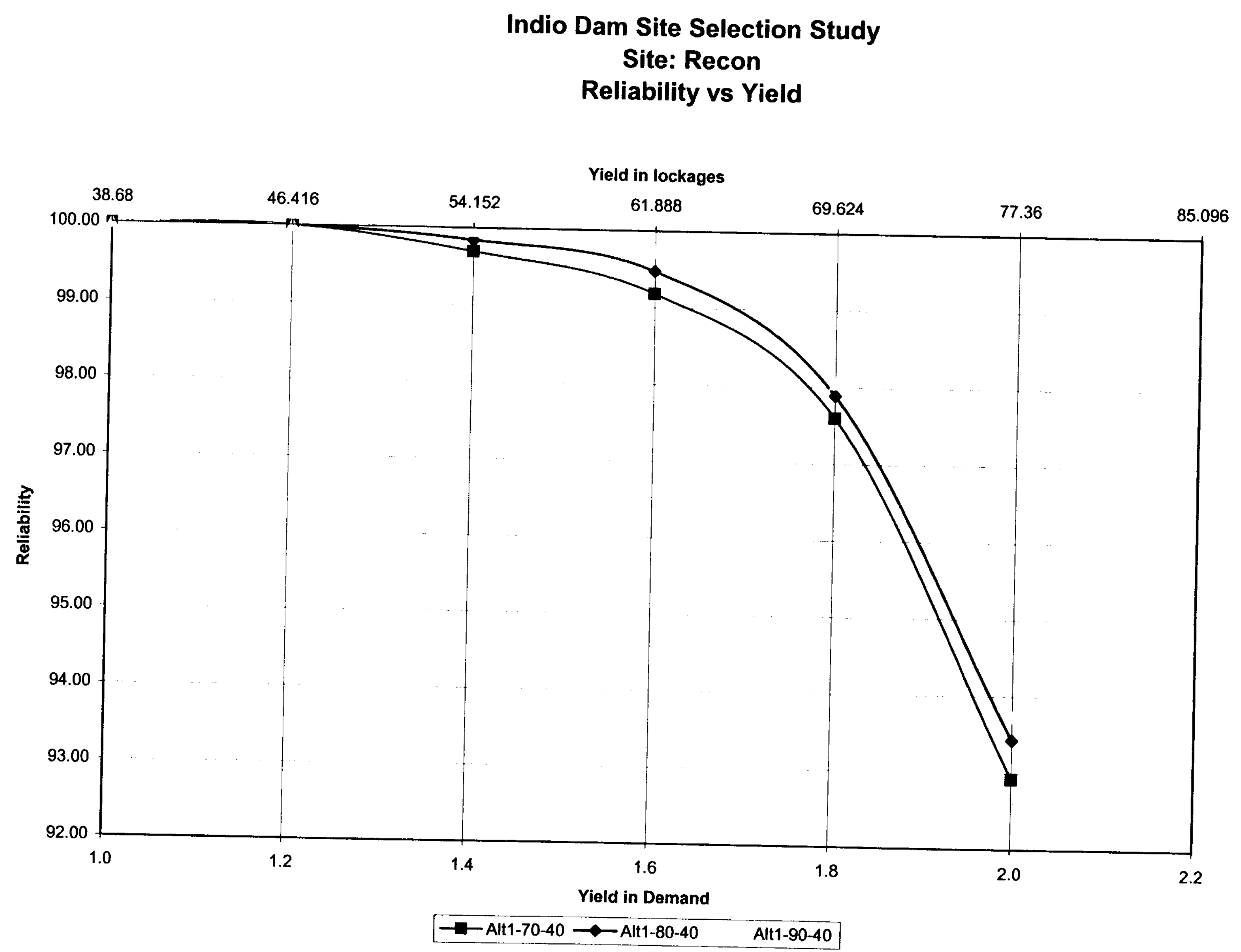
Demand	Yield	Reliability
1	38.68	100
1.2	46.42	100
1.4	54.15	99.97
1.6	61.89	99.75
1.8	69.62	98.13
2	77.36	93.79
1.65	63.82	99.6
1.725	66.72	99



Rio Indio Dam Site Selection Study - Recon
Storage vs Yield
70.0
68.0
66.0
64.0
62.0
60.0
58.0
56.0
54.0
52.0
50.0
Yield
600
800
1000
1200
1400
1600
1800
Storage (MCM)
70-40
80-40
90-40
70-40
80-40
90-40
Reliability = 99%
Reliability=99.6%

Rio Indio Dam Site Selection Study
Site: Recon

Elevations (meters)	Storage (MCM)	Demand Factor for Reliab. 99%	Yield for Reliab. 99%	Demand Factor for Reliab. 99.6%	Yield for Reliab. 99.6%
70-40	856	1.64	63.4	1.455	56.28
80-40	1238	1.702	65.8	1.56	60.34
90-40	1684	1.719	66.5	1.651	63.86


Indio Dam Site Selection Study
Site: Recon
Reliability vs Yield
Yield in lockages
38.68
46.416
54.152
61.888
69.624
77.36
85.096
100.00
99.00
98.00
97.00
96.00
95.00
94.00
93.00
92.00
Reliability
1.0
1.2
1.4
1.6
1.8
2.0
2.2
Yield in Demand
Alt1-70-40
Alt1-80-40
Alt1-90-40

Rio Indio Dam Site Secection Study
Site: Recon

Elevations: 70-40

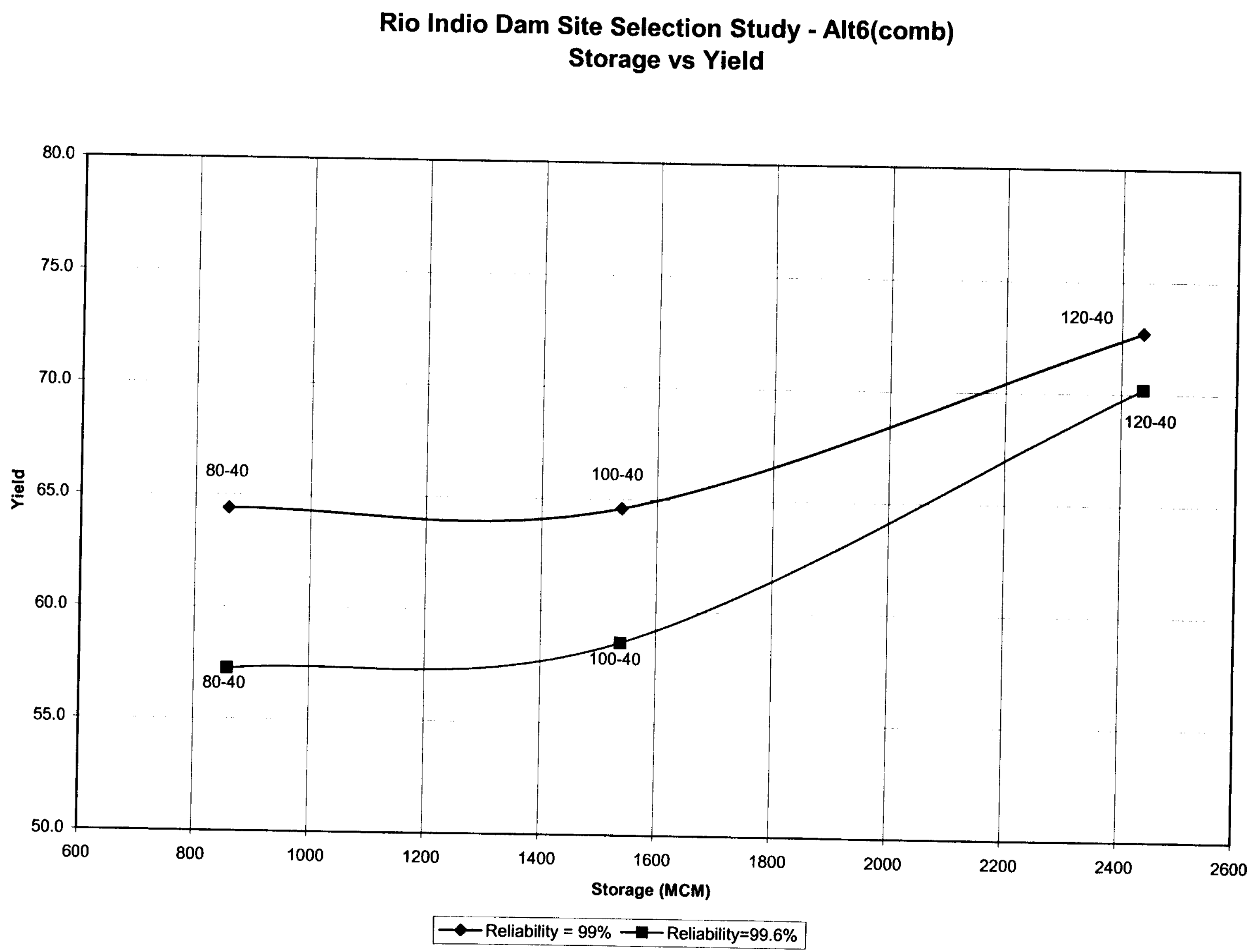
Demand	Yield	Reliability
1.000	38.68	100.00
1.200	46.42	100.00
1.400	54.15	99.69
1.600	61.89	99.18
1.800	69.62	97.60
2.000	77.36	92.92
1.455	56.28	99.60
1.640	63.44	99.00

Elevations: 80-40

Demand	Yield	Reliability
1	38.68	100
1.2	46.42	100
1.4	54.15	99.84
1.6	61.89	99.47
1.8	69.62	97.89
2	77.36	93.43
1.56	60.34	99.6
1.702	65.83	99

Elevations: 90-40

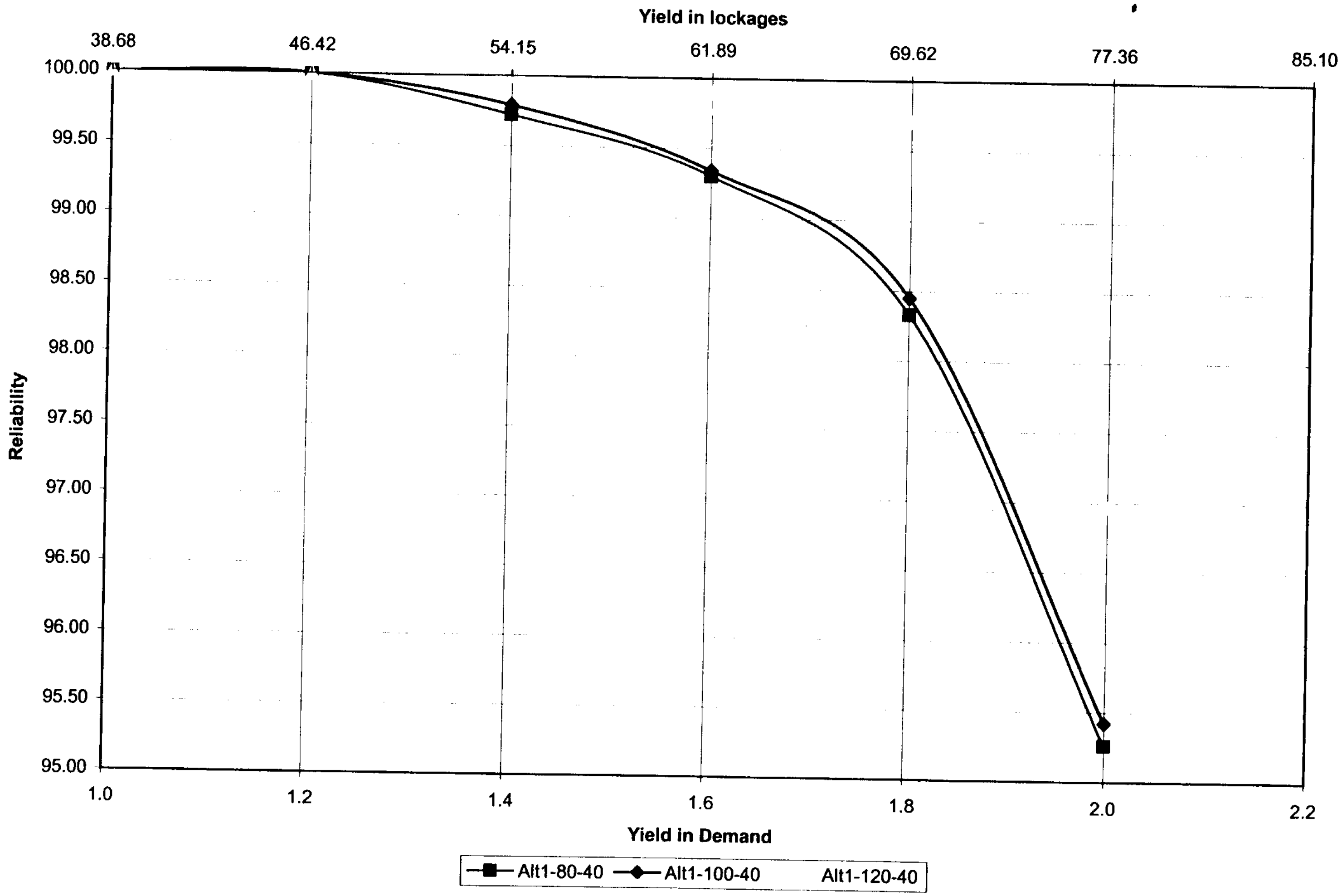
Demand	Yield	Reliability
1	38.68	100
1.2	46.42	100
1.4	54.15	99.96
1.6	61.89	99.73
1.8	69.62	98.05
2	77.36	93.74
1.651	63.86	99.6
1.719	66.49	1.865


Rio Indio Dam Site Selection Study - Alt6(comb)
Storage vs Yield
80.0
75.0
70.0
65.0
60.0
55.0
50.0
Yield
600
800
1000
1200
1400
1600
1800
2000
2200
2400
2600
Storage (MCM)
80-40
100-40
120-40
80-40
100-40
120-40
Reliability = 99%
Reliability=99.6%

Rio Indio Dam Site Selection Study
Site: Alt6(comb)

Elevations (meters)	Storage (MCM)	Demand Factor for Reliab. 99%	Yield for Reliab. 99%	Demand Factor for Reliab. 99.6%	Yield for Reliab. 99.6%
80-40	858	1.665	64.4	1.48	57.25
100-40	1539	1.67	64.6	1.515	58.60
120-40	2438	1.88	72.7	1.815	70.20

Indio Dam Site Selection Study
Site: Alt6(Comb)
Reliability vs Yield



Rio Indio Dam Site Selection Study
Site: Alt6(comb)

Elevations: 80-40

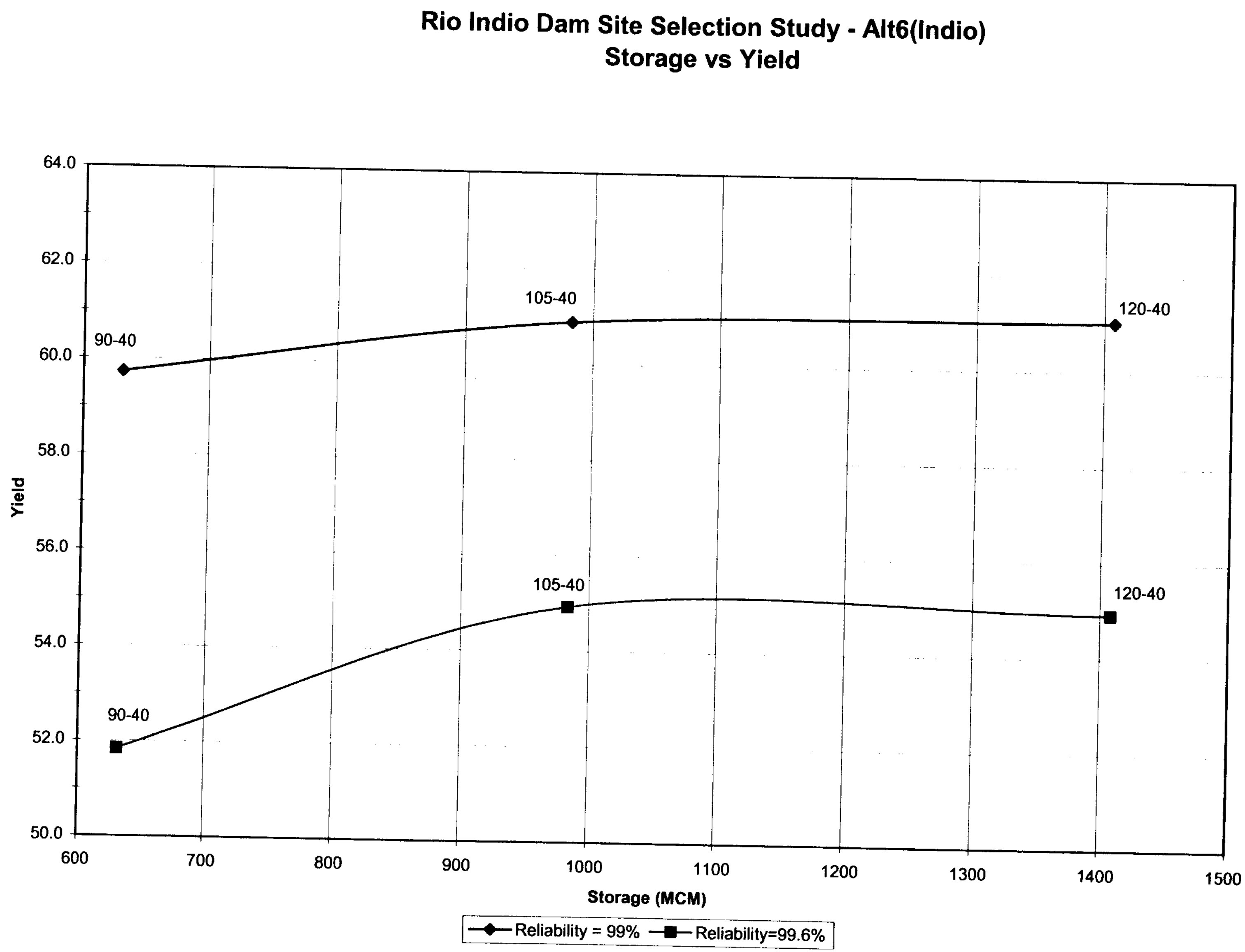
Demand	Yield	Reliability
1.00	38.68	100.00
1.20	46.42	100.00
1.40	54.15	99.72
1.60	61.89	99.30
1.80	69.62	98.33
2.00	77.36	95.26
1.48	57.25	99.60
1.67	64.40	99.00

Elevations: 100-40

Demand	Yield	Reliability
1.00	38.68	100.00
1.20	46.42	100.00
1.40	54.15	99.79
1.60	61.89	99.34
1.80	69.62	98.45
2.00	77.36	95.42
1.52	58.60	99.60
1.67	64.60	99.00

Elevations: 120-40

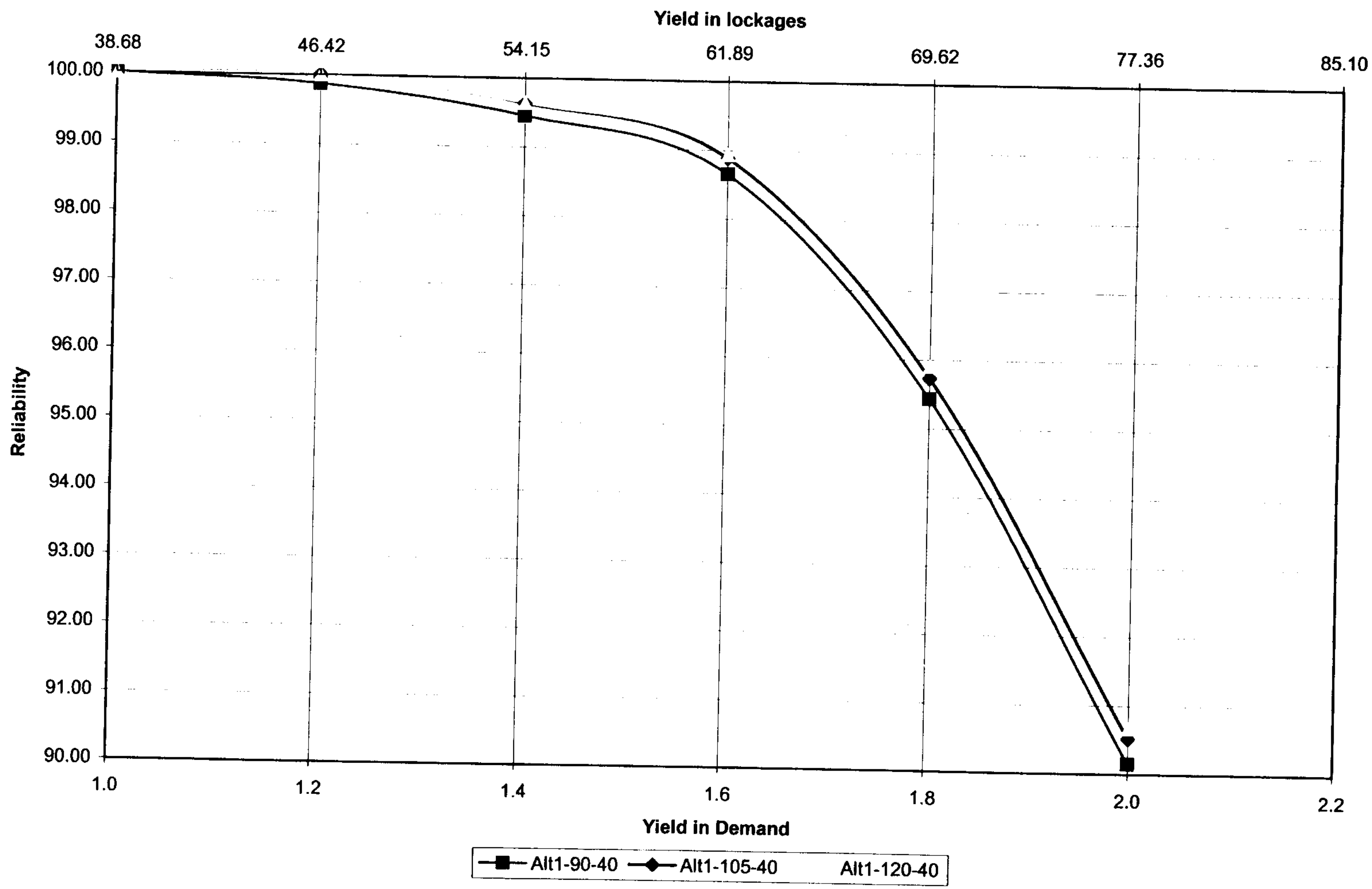
Demand	Yield	Reliability
1.00	38.68	100.00
1.20	46.42	100.00
1.40	54.15	100.00
1.60	61.89	99.90
1.80	69.62	99.71
2.00	77.36	96.97
1.82	70.20	99.60
1.88	72.72	99.00


Rio Indio Dam Site Selection Study - Alt6(Indio)
Storage vs Yield
64.0
62.0
60.0
58.0
56.0
54.0
52.0
50.0
Yield
90-40
105-40
120-40
90-40
105-40
120-40
600
700
800
900
1000
1100
1200
1300
1400
1500
Storage (MCM)
Reliability = 99%
Reliability=99.6%

Rio Indio Dam Site Selection Study
Site: Alt6(Indio)

Elevations (meters)	Storage (MCM)	Demand Factor for Reliab. 99%	Yield for Reliab. 99%	Demand Factor for Reliab. 99.6%	Yield for Reliab. 99.6%
90-40	632	1.544	59.7	1.34	51.83
105-40	984	1.574	60.9	1.42	54.93
120-40	1408	1.578	61.0	1.42	54.93

Indio Dam Site Selection Study
Site: Alt6(Indio)
Reliability vs Yield



Rio Indio Dam Site Selection Study
Site: Alt6(Indio)

Elevations: 90-40

Demand	Yield	Reliability
1.00	38.68	100.00
1.20	46.42	99.87
1.40	54.15	99.45
1.60	61.89	98.65
1.80	69.62	95.43
2.00	77.36	90.15
1.34	51.83	99.60
1.54	59.72	99.00

Elevations: 105-40

Demand	Yield	Reliability
1.00	38.68	100.00
1.20	46.42	100.00
1.40	54.15	99.63
1.60	61.89	98.88
1.80	69.62	95.73
2.00	77.36	90.52
1.42	54.93	99.60
1.57	60.88	99.00

Elevations: 120-40

Demand	Yield	Reliability
1.00	38.68	100.00
1.20	46.42	100.00
1.40	54.15	99.64
1.60	61.89	98.93
1.80	69.62	95.90
2.00	77.36	90.67
1.42	54.93	99.60
1.58	61.04	99.00

ATTACHMENT 2

ADDITIONAL SOCIO-ECONOMIC DATA PROVIDED BY THE ACP

Fuen. ..studios Socioeconomicos IRG-Dames -More

Río Indio

Uso del Suelo en Hectareas Sitio 1

Uso del suelo	Nivel 40	Nivel 60	Nivel 80	Nivel 100	Nivel 110	Nivel 120
Agua	281.384	281.417	xxxxxxx	xxxxxxx	xxxxxxx	xxxxxxx
Area de Ganaderia	2444.209	4148.333	xxxxxxx	xxxxxxx	xxxxxxx	xxxxxxx
Bosque Denso poco Intervenido de Tierra Bajas	45.086	87.974	xxxxxxx	xxxxxxx	xxxxxxx	xxxxxxx
Bosque Intervenido	1547.127	2674.109	xxxxxxx	xxxxxxx	xxxxxxx	xxxxxxx
Cultivos Anuales	57.108	124.182	xxxxxxx	xxxxxxx	xxxxxxx	xxxxxxx
Matorral Rastrojo	1858.870	3127.577	xxxxxxx	xxxxxxx	xxxxxxx	xxxxxxx
No Data	336.264	815.122	xxxxxxx	xxxxxxx	xxxxxxx	xxxxxxx
Total	6570.048	11258.714	xxxxxxx	xxxxxxx	xxxxxxx	xxxxxxx

Uso del Suelo en Hectareas Sitio 2

Uso del suelo	Nivel 40	Nivel 60	Nivel 80	Nivel 100	Nivel 110	Nivel 120
Agua	258.060	258.060	xxxxxxx	xxxxxxx	xxxxxxx	xxxxxxx
Area de Ganaderia	2293.363	3973.912	xxxxxxx	xxxxxxx	xxxxxxx	xxxxxxx
Bosque Denso poco Intervenido de Tierra Bajas	45.086	87.974	xxxxxxx	xxxxxxx	xxxxxxx	xxxxxxx
Bosque Intervenido	1485.700	2597.389	xxxxxxx	xxxxxxx	xxxxxxx	xxxxxxx
Cultivos Anuales	54.906	121.894	xxxxxxx	xxxxxxx	xxxxxxx	xxxxxxx
Matorral Rastrojo	1736.446	2968.174	xxxxxxx	xxxxxxx	xxxxxxx	xxxxxxx
No Data	331.797	810.642	xxxxxxx	xxxxxxx	xxxxxxx	xxxxxxx
Total	6205.358	10818.045	xxxxxxx	xxxxxxx	xxxxxxx	xxxxxxx

Uso del Suelo en Hectareas Sitio 3

Uso del suelo	Nivel 40	Nivel 60	Nivel 80	Nivel 100	Nivel 110	Nivel 120
Agua	238.118	238.173	xxxxxxx	xxxxxxx	xxxxxxx	xxxxxxx
Area de Ganaderia	2093.013	3745.199	xxxxxxx	xxxxxxx	xxxxxxx	xxxxxxx
Bosque Denso poco Intervenido de Tierra Bajas	45.086	87.974	xxxxxxx	xxxxxxx	xxxxxxx	xxxxxxx
Bosque Intervenido	1404.546	2474.700	xxxxxxx	xxxxxxx	xxxxxxx	xxxxxxx
Cultivos Anuales	50.420	116.310	xxxxxxx	xxxxxxx	xxxxxxx	xxxxxxx
Matorral Rastrojo	1633.078	2826.938	xxxxxxx	xxxxxxx	xxxxxxx	xxxxxxx
No Data	327.547	806.389	xxxxxxx	xxxxxxx	xxxxxxx	xxxxxxx
Total	5791.808	10295.683	xxxxxxx	xxxxxxx	xxxxxxx	xxxxxxx

Uso del Suelo en Hectareas Sitio 4

Uso del suelo	Nivel 40	Nivel 60	Nivel 80	Nivel 100	Nivel 110	Nivel 120
Agua	147.324	147.169	147.172	xxxxxxx	xxxxxxx	xxxxxxx
Area de Ganaderia	661.713	1324.348	1748.522	xxxxxxx	xxxxxxx	xxxxxxx
Bosque Denso poco Intervenido de Tierra Bajas	27.365	50.852	64.635	xxxxxxx	xxxxxxx	xxxxxxx
Bosque Intervenido	325.712	652.273	942.722	xxxxxxx	xxxxxxx	xxxxxxx
Cultivos Anuales	12.152	40.798	62.928	xxxxxxx	xxxxxxx	xxxxxxx
Matorral Rastrojo	540.474	1058.163	1413.647	xxxxxxx	xxxxxxx	xxxxxxx
No Data	166.540	282.745	329.990	xxxxxxx	xxxxxxx	xxxxxxx
Total	1881.280	3556.348	4709.616	xxxxxxx	xxxxxxx	xxxxxxx

Uso del Suelo en Hectareas Sitio RECON

Uso del suelo	Nivel 40	Nivel 60	Nivel 80	Nivel 100	Nivel 110	Nivel 120
Agua	136.146	136.228	136.189	xxxxxxx	xxxxxxx	xxxxxxx
Area de Ganaderia	635.968	1288.302	1711.000	xxxxxxx	xxxxxxx	xxxxxxx
Bosque Denso poco Intervenido de Tierra Bajas	24.757	46.183	60.206	xxxxxxx	xxxxxxx	xxxxxxx
Bosque Intervenido	314.096	633.933	923.224	xxxxxxx	xxxxxxx	xxxxxxx
Cultivos Anuales	11.267	39.646	61.592	xxxxxxx	xxxxxxx	xxxxxxx
Matorral Rastrojo	496.027	994.085	1340.743	xxxxxxx	xxxxxxx	xxxxxxx
No Data	143.101	230.130	279.327	xxxxxxx	xxxxxxx	xxxxxxx
Total	1761.362	3368.507	4512.281	xxxxxxx	xxxxxxx	xxxxxxx

Uso del Suelo en Hectareas Sitio 5

Uso del suelo	Nivel 40	Nivel 60	Nivel 80	Nivel 100	Nivel 110	Nivel 120
Agua	112.757	112.799	112.817	112.831	xxxxxxx	xxxxxxx
Area de Ganaderia	515.377	1164.980	1568.084	2006.875	xxxxxxx	xxxxxxx
Bosque Denso poco Intervenido de Tierra Bajas	15.642	32.587	45.090	58.016	xxxxxxx	xxxxxxx
Bosque Intervenido	283.655	602.136	889.772	1179.466	xxxxxxx	xxxxxxx
Cultivos Anuales	9.022	36.595	58.092	95.872	xxxxxxx	xxxxxxx
Matorral Rastrojo	372.985	848.126	1174.329	1537.979	xxxxxxx	xxxxxxx
No Data	26.483	49.855	77.152	126.006	xxxxxxx	xxxxxxx
Total	1335.921	2847.078	3925.336	5117.045	xxxxxxx	xxxxxxx

Uso del Suelo en Hectareas Sitio 6A

Uso del suelo	Nivel 40	Nivel 60	Nivel 80	Nivel 100	Nivel 110	Nivel 120
Agua	67.302	67.361	67.371	67.359	67.324	67.438
Area de Ganaderia	136.839	437.785	707.388	1001.103	1097.304	1227.969
Bosque Denso poco Intervenido de Tierra Bajas	6.916	13.877	19.715	26.514	28.373	31.254
Bosque Intervenido	107.169	264.480	429.113	583.992	633.067	752.387
Cultivos Anuales	2.394	14.300	22.680	37.505	42.114	56.772
Matorral Rastrojo	137.722	365.278	549.392	730.954	791.725	919.462
No Data	15.729	27.662	31.048	39.104	42.379	45.774
Total	474.071	1190.743	1826.707	2486.531	2702.286	3101.056

Uso del Suelo en Hectareas Sitio 6B

Uso del suelo	Nivel 40	Nivel 60	Nivel 80	Nivel 100	Nivel 110	Nivel 120
Agua	240.525	346.786	429.859	505.752	521.805	xxxxxxx
Area de Ganaderia	5.250	11.020	16.105	20.393	19.172	xxxxxxx
Bosque Denso poco Intervenido de Tierra Bajas	110.313	187.120	285.395	398.426	372.343	xxxxxxx
Bosque Intervenido	4.031	12.921	21.214	40.132	42.435	xxxxxxx
Cultivos Anuales	119.674	223.466	343.438	471.940	481.714	xxxxxxx
Matorral Rastrojo	8.364	12.982	34.296	69.994	72.621	xxxxxxx
No Data	488.157	794.295	1130.307	1506.637	1510.090	xxxxxxx
Total						

Fuente: Estudios Socioeconomicos IRG-Dames -More

Río Indio

Uso del Suelo en Hectareas Sitio 1

Uso del suelo	Nivel 40	Nivel 60	Nivel 80	Nivel 100	Nivel 110	Nivel 120
Agua	281.384	281.417	xxxxxxx	xxxxxxx	xxxxxxx	xxxxxxx
Area de Ganaderia	2444.209	4148.333	xxxxxxx	xxxxxxx	xxxxxxx	xxxxxxx
Bosque Denso poco Intervenido de Tierra Bajas	45.086	87.974	xxxxxxx	xxxxxxx	xxxxxxx	xxxxxxx
Bosque Intervenido	1547.127	2674.109	xxxxxxx	xxxxxxx	xxxxxxx	xxxxxxx
Cultivos Anuales	57.108	124.182	xxxxxxx	xxxxxxx	xxxxxxx	xxxxxxx
Matorral Rastrojo	1858.870	3127.577	xxxxxxx	xxxxxxx	xxxxxxx	xxxxxxx
No Data	336.264	815.122	xxxxxxx	xxxxxxx	xxxxxxx	xxxxxxx
Total	6570.048	11258.714	xxxxxxx	xxxxxxx	xxxxxxx	xxxxxxx

Uso del Suelo en Hectareas Sitio 2

Uso del suelo	Nivel 40	Nivel 60	Nivel 80	Nivel 100	Nivel 110	Nivel 120
Agua	258.060	258.060	xxxxxxx	xxxxxxx	xxxxxxx	xxxxxxx
Area de Ganaderia	2293.363	3973.912	xxxxxxx	xxxxxxx	xxxxxxx	xxxxxxx
Bosque Denso poco Intervenido de Tierra Bajas	45.086	87.974	xxxxxxx	xxxxxxx	xxxxxxx	xxxxxxx
Bosque Intervenido	1485.700	2597.389	xxxxxxx	xxxxxxx	xxxxxxx	xxxxxxx
Cultivos Anuales	54.906	121.894	xxxxxxx	xxxxxxx	xxxxxxx	xxxxxxx
Matorral Rastrojo	1736.446	2968.174	xxxxxxx	xxxxxxx	xxxxxxx	xxxxxxx
No Data	331.797	810.642	xxxxxxx	xxxxxxx	xxxxxxx	xxxxxxx
Total	6205.358	10818.045	xxxxxxx	xxxxxxx	xxxxxxx	xxxxxxx

Uso del Suelo en Hectareas Sitio 3

Uso del suelo	Nivel 40	Nivel 60	Nivel 80	Nivel 100	Nivel 110	Nivel 120
Agua	238.118	238.173	xxxxxxx	xxxxxxx	xxxxxxx	xxxxxxx
Area de Ganaderia	2093.013	3745.199	xxxxxxx	xxxxxxx	xxxxxxx	xxxxxxx
Bosque Denso poco Intervenido de Tierra Bajas	45.086	87.974	xxxxxxx	xxxxxxx	xxxxxxx	xxxxxxx
Bosque Intervenido	1404.546	2474.700	xxxxxxx	xxxxxxx	xxxxxxx	xxxxxxx
Cultivos Anuales	50.420	116.310	xxxxxxx	xxxxxxx	xxxxxxx	xxxxxxx

Matorral Rastrojo	1633.078	2826.938	xxxxxxx	xxxxxxx	xxxxxxx	xxxxxxx
No Data	327.547	806.389	xxxxxxx	xxxxxxx	xxxxxxx	xxxxxxx
Total	5791.808	10295.683	xxxxxxx	xxxxxxx	xxxxxxx	xxxxxxx

Uso del Suelo en Hectareas Sitio 4

Uso del suelo	Nivel 40	Nivel 60	Nivel 80	Nivel 100	Nivel 110	Nivel 120
Agua	147.324	147.169	147.172	xxxxxxx	xxxxxxx	xxxxxxx
Area de Ganaderia	661.713	1324.348	1748.522	xxxxxxx	xxxxxxx	xxxxxxx
Bosque Denso poco Intervenido de Tierra Bajas	27.365	50.852	64.635	xxxxxxx	xxxxxxx	xxxxxxx
Bosque Intervenido	325.712	652.273	942.722	xxxxxxx	xxxxxxx	xxxxxxx
Cultivos Anuales	12.152	40.798	62.928	xxxxxxx	xxxxxxx	xxxxxxx
Matorral Rastrojo	540.474	1058.163	1413.647	xxxxxxx	xxxxxxx	xxxxxxx
No Data	166.540	282.745	329.990	xxxxxxx	xxxxxxx	xxxxxxx
Total	1881.280	3556.348	4709.616	xxxxxxx	xxxxxxx	xxxxxxx

Uso del Suelo en Hectareas Sitio RECON

Uso del suelo	Nivel 40	Nivel 60	Nivel 80	Nivel 100	Nivel 110	Nivel 120
Agua	136.146	136.228	136.189	xxxxxxx	xxxxxxx	xxxxxxx
Area de Ganaderia	635.968	1288.302	1711.000	xxxxxxx	xxxxxxx	xxxxxxx
Bosque Denso poco Intervenido de Tierra Bajas	24.757	46.183	60.206	xxxxxxx	xxxxxxx	xxxxxxx
Bosque Intervenido	314.096	633.933	923.224	xxxxxxx	xxxxxxx	xxxxxxx
Cultivos Anuales	11.267	39.646	61.592	xxxxxxx	xxxxxxx	xxxxxxx
Matorral Rastrojo	496.027	994.085	1340.743	xxxxxxx	xxxxxxx	xxxxxxx
No Data	143.101	230.130	279.327	xxxxxxx	xxxxxxx	xxxxxxx
Total	1761.362	3368.507	4512.281	xxxxxxx	xxxxxxx	xxxxxxx

Uso del Suelo en Hectareas Sitio 5

Uso del suelo	Nivel 40	Nivel 60	Nivel 80	Nivel 100	Nivel 110	Nivel 120
Agua	112.757	112.799	112.817	112.831	xxxxxxx	xxxxxxx
Area de Ganaderia	515.377	1164.980	1568.084	2006.875	xxxxxxx	xxxxxxx
Bosque Denso poco Intervenido de Tierra Bajas	15.642	32.587	45.090	58.016	xxxxxxx	xxxxxxx
Bosque Intervenido	283.655	602.136	889.772	1179.466	xxxxxxx	xxxxxxx
Cultivos Anuales	9.022	36.595	58.092	95.872	xxxxxxx	xxxxxxx
Matorral Rastrojo	372.985	848.126	1174.329	1537.979	xxxxxxx	xxxxxxx
No Data	26.483	49.855	77.152	126.006	xxxxxxx	xxxxxxx
Total	1335.921	2847.078	3925.336	5117.045	xxxxxxx	xxxxxxx

Uso del Suelo en Hectareas Sitio 6A

Uso del suelo	Nivel 40	Nivel 60	Nivel 80	Nivel 100	Nivel 110	Nivel 120
Agua	67.302	67.361	67.371	67.359	67.324	67.438
Area de Ganaderia	136.839	437.785	707.388	1001.103	1097.304	1227.969
Bosque Denso poco Intervenido de Tierra Bajas	6.916	13.877	19.715	26.514	28.373	31.254
Bosque Intervenido	107.169	264.480	429.113	583.992	633.067	752.387
Cultivos Anuales	2.394	14.300	22.680	37.505	42.114	56.772
Matorral Rastrojo	137.722	365.278	549.392	730.954	791.725	919.462
No Data	15.729	27.662	31.048	39.104	42.379	45.774
Total	474.071	1190.743	1826.707	2486.531	2702.286	3101.056

Uso del Suelo en Hectareas Sitio 6B

Uso del suelo	Nivel 40	Nivel 60	Nivel 80	Nivel 100	Nivel 110	Nivel 120
Agua	240.525	346.786	429.859	505.752	521.805	xxxxxxx
Area de Ganaderia	5.250	11.020	16.105	20.393	19.172	xxxxxxx
Bosque Denso poco Intervenido de Tierra Bajas	110.313	187.120	285.395	398.426	372.343	xxxxxxx
Bosque Intervenido	4.031	12.921	21.214	40.132	42.435	xxxxxxx
Cultivos Anuales	119.674	223.466	343.438	471.940	481.714	xxxxxxx
Matorral Rastrojo	8.364	12.982	34.296	69.994	72.621	xxxxxxx
No Data	488.157	794.295	1130.307	1506.637	1510.090	xxxxxxx

Fuente: Estudios Socioeconomicos Irg-Dames -More

Río Indio

Cantidad de Infraestructuras en Sitio 1

Estructura	Nivel 40	Nivel 60	Nivel 80	Nivel 100	Nivel 110	Nivel 120
Agua Potable Distribucion	2	2	xxxxxxx	xxxxxxx	xxxxxxx	xxxxxxx
Almacenamiento de Agua	2	2	xxxxxxx	xxxxxxx	xxxxxxx	xxxxxxx
Areas Recreativas	1	1	xxxxxxx	xxxxxxx	xxxxxxx	xxxxxxx
Caminos	7	14	xxxxxxx	xxxxxxx	xxxxxxx	xxxxxxx
Cementerio	6	10	xxxxxxx	xxxxxxx	xxxxxxx	xxxxxxx
Clinica o Centro de Salud	7	8	xxxxxxx	xxxxxxx	xxxxxxx	xxxxxxx
Comercio/Industrial/Agroindustrial	13	17	xxxxxxx	xxxxxxx	xxxxxxx	xxxxxxx
Escuela	11	17	xxxxxxx	xxxxxxx	xxxxxxx	xxxxxxx
Fuente de Agua Potable	1	1	xxxxxxx	xxxxxxx	xxxxxxx	xxxxxxx
Generación De Energía Eléctrica	2	2	xxxxxxx	xxxxxxx	xxxxxxx	xxxxxxx
Gubernamental	7	9	xxxxxxx	xxxxxxx	xxxxxxx	xxxxxxx
Iglesia/Capilla	18	25	xxxxxxx	xxxxxxx	xxxxxxx	xxxxxxx
Parque Publico o Area Recreativas	7	8	xxxxxxx	xxxxxxx	xxxxxxx	xxxxxxx
Postes Electricos	38	38	xxxxxxx	xxxxxxx	xxxxxxx	xxxxxxx
Puente	13	14	xxxxxxx	xxxxxxx	xxxxxxx	xxxxxxx
Telecomunicaciones	5	6	xxxxxxx	xxxxxxx	xxxxxxx	xxxxxxx
Vivienda	331	467	xxxxxxx	xxxxxxx	xxxxxxx	xxxxxxx

Cantidad de Infraestructuras en Sitio 2

Estructura	Nivel 40	Nivel 60	Nivel 80	Nivel 100	Nivel 110	Nivel 120
Agua Potable Distribucion	2	2	xxxxxxx	xxxxxxx	xxxxxxx	xxxxxxx
Almacenamiento de Agua	1	1	xxxxxxx	xxxxxxx	xxxxxxx	xxxxxxx
Areas Recreativas	1	1	xxxxxxx	xxxxxxx	xxxxxxx	xxxxxxx
Caminos	7	14	xxxxxxx	xxxxxxx	xxxxxxx	xxxxxxx
Cementerio	6	10	xxxxxxx	xxxxxxx	xxxxxxx	xxxxxxx
Clinica o Centro de Salud	5	6	xxxxxxx	xxxxxxx	xxxxxxx	xxxxxxx
Comercio/Industrial/Agroindustrial	12	16	xxxxxxx	xxxxxxx	xxxxxxx	xxxxxxx
Escuela	10	16	xxxxxxx	xxxxxxx	xxxxxxx	xxxxxxx
Fuente de Agua Potable	1	1	xxxxxxx	xxxxxxx	xxxxxxx	xxxxxxx
Generación de Energía Eléctrica	2	2	xxxxxxx	xxxxxxx	xxxxxxx	xxxxxxx
Gubernamental	7	9	xxxxxxx	xxxxxxx	xxxxxxx	xxxxxxx
Iglesia/Capilla	15	22	xxxxxxx	xxxxxxx	xxxxxxx	xxxxxxx
Parque Publico o Area Recreativas	5	6	xxxxxxx	xxxxxxx	xxxxxxx	xxxxxxx
Postes Electricos	38	38	xxxxxxx	xxxxxxx	xxxxxxx	xxxxxxx
Puente	12	14	xxxxxxx	xxxxxxx	xxxxxxx	xxxxxxx
Telecomunicaciones	5	6	xxxxxxx	xxxxxxx	xxxxxxx	xxxxxxx
Vivienda	295	451	xxxxxxx	xxxxxxx	xxxxxxx	xxxxxxx

Cantidad de Infraestructuras en Sitio 3

Estructura	Nivel 40	Nivel 60	Nivel 80	Nivel 100	Nivel 110	Nivel 120
Agua Potable Distribucion	2	2	xxxxxxx	xxxxxxx	xxxxxxx	xxxxxxx
Almacenamiento de Agua	1	1	xxxxxxx	xxxxxxx	xxxxxxx	xxxxxxx
Areas Recreativas	1	1	xxxxxxx	xxxxxxx	xxxxxxx	xxxxxxx
Caminos	7	14	xxxxxxx	xxxxxxx	xxxxxxx	xxxxxxx
Cementerio	5	9	xxxxxxx	xxxxxxx	xxxxxxx	xxxxxxx
Clinica o Centro de Salud	4	5	xxxxxxx	xxxxxxx	xxxxxxx	xxxxxxx
Comercio/Industrial/Agroindustrial	10	14	xxxxxxx	xxxxxxx	xxxxxxx	xxxxxxx
Escuela	8	14	xxxxxxx	xxxxxxx	xxxxxxx	xxxxxxx
Fuente de Agua Potable	1	1	xxxxxxx	xxxxxxx	xxxxxxx	xxxxxxx
Generación de Energía Eléctrica			xxxxxxx	xxxxxxx	xxxxxxx	xxxxxxx
Gubernamental	5		xxxxxxx	xxxxxxx	xxxxxxx	xxxxxxx
Iglesia/Capilla	14	21	xxxxxxx	xxxxxxx	xxxxxxx	xxxxxxx
Parque Publico o Area Recreativas	4	5	xxxxxxx	xxxxxxx	xxxxxxx	xxxxxxx
Postes Electricos			xxxxxxx	xxxxxxx	xxxxxxx	xxxxxxx
Puente	14	12	xxxxxxx	xxxxxxx	xxxxxxx	xxxxxxx
Telecomunicaciones	3	4	xxxxxxx	xxxxxxx	xxxxxxx	xxxxxxx

Vivienda	249	384	xxxxxxx	xxxxxxx	xxxxxxx	xxxxxxx

Cantidad de Infraestructuras en Sitio 4

Estructura	Nivel 40	Nivel 60	Nivel 80	Nivel 100	Nivel 110	Nivel 120
Agua Potable Distribucion			1	xxxxxxx	xxxxxxx	xxxxxxx
Almacenamiento de Agua			1	xxxxxxx	xxxxxxx	xxxxxxx
Areas Recreativas	1	1	1	xxxxxxx	xxxxxxx	xxxxxxx
Caminos	7	9	15	xxxxxxx	xxxxxxx	xxxxxxx
Cementerio	4	7	8	xxxxxxx	xxxxxxx	xxxxxxx
Clinica o Centro de Salud	1	2	3	xxxxxxx	xxxxxxx	xxxxxxx
Comercio/Industrial/Agroindustrial	6	1	18	xxxxxxx	xxxxxxx	xxxxxxx
Escuela	3	9	11	xxxxxxx	xxxxxxx	xxxxxxx
Fuente de Agua Potable				xxxxxxx	xxxxxxx	xxxxxxx
Generación de Energía Eléctrica				xxxxxxx	xxxxxxx	xxxxxxx
Gubernamental	4	6	9	xxxxxxx	xxxxxxx	xxxxxxx
Iglesia/Capilla	4	11	15	xxxxxxx	xxxxxxx	xxxxxxx
Parque Publico o Area Recreativas	3	4	5	xxxxxxx	xxxxxxx	xxxxxxx
Postes Electricos				xxxxxxx	xxxxxxx	xxxxxxx
Puente	7	8	8	xxxxxxx	xxxxxxx	xxxxxxx
Telecomunicaciones	1	2	3	xxxxxxx	xxxxxxx	xxxxxxx
Vivienda	117	241	320	xxxxxxx	xxxxxxx	xxxxxxx

Cantidad de Infraestructuras en Sitio RECON.

Estructura	Nivel 40	Nivel 60	Nivel 80	Nivel 100	Nivel 110	Nivel 120
Agua Potable Distribucion			1	xxxxxxx	xxxxxxx	xxxxxxx
Almacenamiento de Agua			1	xxxxxxx	xxxxxxx	xxxxxxx
Areas Recreativas	1	1	1	xxxxxxx	xxxxxxx	xxxxxxx
Caminos	7	13	15	xxxxxxx	xxxxxxx	xxxxxxx
Cementerio	4	7	8	xxxxxxx	xxxxxxx	xxxxxxx
Clinica o Centro de Salud	1	2	5	xxxxxxx	xxxxxxx	xxxxxxx
Comercio/Industrial/Agroindustrial	6	10	14	xxxxxxx	xxxxxxx	xxxxxxx
Escuela	3	9	11	xxxxxxx	xxxxxxx	xxxxxxx
Fuente de Agua Potable				xxxxxxx	xxxxxxx	xxxxxxx
Generación de Energía Eléctrica				xxxxxxx	xxxxxxx	xxxxxxx
Gubernamental	4	6	9	xxxxxxx	xxxxxxx	xxxxxxx
Iglesia/Capilla	4	11	15	xxxxxxx	xxxxxxx	xxxxxxx
Parque Publico o Area Recreativas	3	4	5	xxxxxxx	xxxxxxx	xxxxxxx
Postes Electricos				xxxxxxx	xxxxxxx	xxxxxxx
Puente	7	8	8	xxxxxxx	xxxxxxx	xxxxxxx
Telecomunicaciones	1	2	3	xxxxxxx	xxxxxxx	xxxxxxx
Vivienda	108	232	311	xxxxxxx	xxxxxxx	xxxxxxx

Uso del Suelo en Hectareas Sitio 5

Estructura	Nivel 40	Nivel 60	Nivel 80	Nivel 100	Nivel 110	Nivel 120
Agua Potable Distribucion			1	1	xxxxxxx	xxxxxxx
Almacenamiento de Agua			1	6	xxxxxxx	xxxxxxx
Areas Recreativas	1	1	1	1	xxxxxxx	xxxxxxx
Caminos	7	13	15	16	xxxxxxx	xxxxxxx
Cementerio	3	6	1	7	xxxxxxx	xxxxxxx
Clinica o Centro De Salud	1	2	3	3	xxxxxxx	xxxxxxx
Comercio/Industrial/Agroindustrial	5	9	15	16	xxxxxxx	xxxxxxx
Escuela	3	9	11	11	xxxxxxx	xxxxxxx
Fuente de Agua Potable					xxxxxxx	xxxxxxx
Generación de Energía Eléctrica					xxxxxxx	xxxxxxx
Gubernamental	3	5	8	8	xxxxxxx	xxxxxxx
Iglesia/Capilla	3	9	13	13	xxxxxxx	xxxxxxx
Parque Publico o Area Recreativas	3	4	5	5	xxxxxxx	xxxxxxx
Postes Electricos					xxxxxxx	xxxxxxx
Puente	4	5	5	5	xxxxxxx	xxxxxxx
Telecomunicaciones	1	2	3	3	xxxxxxx	xxxxxxx
Vivienda	78	199	277	317	xxxxxxx	xxxxxxx

Uso del Suelo en Hectareas Sitio 6A

Estructura	Nivel 40	Nivel 60	Nivel 80	Nivel 100	Nivel 110	Nivel 120
Agua Potable Distribucion			1	1	1	1
Almacenamiento de Agua			1	1	5	5
Areas Recreativas	1	1	1	1	1	1
Caminos	3	9	11	8	12	13
Cementerio	2	4	5	9	5	5
Clinica o Centro de Salud		1	2	3	2	2
Comercio/Industrial/Agroindustrial		4	9	10	11	11
Escuela	1	7	9	9	9	9
Fuente de Agua Potable						
Generación de Energía Eléctrica						
Gubernamental	1	3	6	6	6	6
Iglesia/Capilla	1	7	11	11	11	13
Parque Publico o Area Recreativas	1	2	3	3	4	4
Postes Electricos						
Puente	3	4	4	6	6	6
Telecomunicaciones			1	1	1	
Vivienda	25	135	205	241	250	274
Granja					1	1

Uso del Suelo en Hectareas Sitio 6B

Estructura	Nivel 40	Nivel 60	Nivel 80	Nivel 100	Nivel 110	Nivel 120
Agua Potable Distribucion						xxxxxxx
Almacenamiento de Agua				1	1	xxxxxxx
Areas Recreativas						xxxxxxx
Caminos	1	1	1	1	1	xxxxxxx
Cementerio		1	1	1	1	xxxxxxx
Clinica o Centro De Salud						xxxxxxx
Comercio/Industrial/Agroindustrial						xxxxxxx
Escuela	1	1	1	1	1	xxxxxxx
Fuente de Agua Potable						xxxxxxx
Generación de Energia Eléctrica						xxxxxxx
Gubernamental						xxxxxxx
Iglesia/Capilla	1	1	1	1	1	xxxxxxx
Parque Publico o Area Recreativas	1	1	1	1	1	xxxxxxx
Postes Electricos						xxxxxxx
Puente						xxxxxxx
Telecomunicaciones						xxxxxxx
Vivienda	17	22	26	29	31	xxxxxxx

Part 2
Dam Type Selection

FEASIBILITY DESIGN FOR THE RÍO INDIO WATER SUPPLY PROJECT

APPENDIX D
PART 2

DAM TYPE SELECTION

Prepared by

MWH

In association with

TAMS
AN EARTH TECH COMPANY

FEASIBILITY DESIGN FOR THE RÍO INDIO WATER SUPPLY PROJECT

APPENDIX D – PART 2
DAM TYPE SELECTION

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FOREWORD

The studies described in this appendix have been performed in accordance with the scope of services for Contract CC3-5-536 - Work Order 003, Feasibility Design and Related Services for the Río Indio Water Supply Project entered into on September 30, 1999. This appendix presents the results of the initial designs and related services to address Task 9, Design of Main Dam. The appendix covers the selection of the type of dam to be further evaluated in the feasibility studies. This and subsequent studies for Task 9, Main Dam are presented in Appendix D, and summarized in the main report:

Appendix D

Part 1 Dam Site Selection;
Part 2 Dam Type Selection;
Part 3 Project Component Configuration, and;
Part 4 Indio-Gatun Transfer Tunnel.

Appendix D Part 2 has been prepared using the following basic information:

- Reconnaissance Study: Identification, Definition, and Evaluation of Water Supply Projects, prepared by the U.S. Army Corps of Engineers, Mobile District, dated August 1999;
- Topographic mapping of areas of the proposed dam site prepared by Ingenieria Avanzada, S.A. under subcontract to MWH. Services were completed and submitted to the ACP under Contract CC-3-536, Task Order 2, Altimetric and Planimetric Surveys of 13 sites located on the Western Side of Lake Gatun;
- Additional topographic mapping of the dam site developed by digitizing 1:50,000 scale maps obtained from Instituto Geografico Nacional (Tommy Guardia);
- The results of hydrology and meteorology studies presented in the companion Appendix A;
- Geological and geotechnical information obtained from two dam site exploration and mapping field programs, and a construction materials

investigation program, including both test pit sampling and laboratory testing. The results are presented in Appendix B, Geology, Geotechnical and Seismological Studies;

- Data from the planned subsurface exploration program comprising boreholes and seismic refraction was not available.

1 INTRODUCTION

As part of a reconnaissance study, the Mobile District assisted ACP to identify and rank 17 water supply projects to augment the existing canal water supply system. The Río Indio Water Supply Project was identified as one of the highest ranked projects, and has been selected for study at the feasibility level. The dam site selected in the reconnaissance study is located about 25 km inland from the Atlantic Ocean on the north-flowing Río Indio, Exhibit 1. The location was selected to provide the maximum size reservoir. It is limited to a reservoir elevation of between 80 m above mean sea level (El. 80) and El. 85 (depending on the selected spillway type) by topography. For a normal pool level of El. 80, two saddle dams are required.

The dam type selected in the reconnaissance study was a center core rockfill dam with a crest at El. 83.5, providing a full supply level of El. 80.0, and a maximum reservoir flood level of El. 82.5. An ungated spillway was located on the left abutment. The spillway would have a crest width of 120 m and a capacity of 920 m^3/s. The spillway chute was proposed as a sloped and/or stepped natural rock cut channel of about 1,100 m length.

The project also included a hydropower facility at the dam, transmission line, inter-basin transfer tunnel with associated power facility, as well as construction of access roads and other facilities required for this remote location.

2 DAM TYPES AND DEVELOPMENT CRITERIA

This section contains a description of the types of dams that have been considered and the criteria used in sizing the facilities and appurtenant features. Features common to all dam types are also identified, but not included in subsequent comparison.

2.1 General Criteria

The general criteria have been established primarily for the dam type selection. It is anticipated that they will be modified as a result of additional studies performed to optimize the selected dam, as well as the results of the planned geotechnical exploration program. The general criteria for dam type selection are as follows:

- The dam will be located at the Cerro Tres Hermanas site identified in the reconnaissance study;
- The reservoir full supply level will be at El. 80;
- The reservoir low supply level will be at El. 40;
- Gated and ungated spillway options will be sized to limit flood surcharge to El. 84, and;
- The dam crest will be at El. 83 with an upstream parapet wall to El. 85.

As a result of these criteria, the gross storage at full supply level will be 1,577 million cubic meters (MCM). The live storage between El. 80 and El. 40 is estimated to be 1294 MCM.

2.2 Alternative Dam Types

The dam types considered for this study are the following:

Table 1: Alternative Dam Types

Alternative	Dam Types	Abbreviation	Spillway Alternatives
1A	Concrete Faced Rockfill Dam	CFRD	Gated
1B			Ungated
2A	Roller Compacted Concrete Dam	RCC	Gated
2B			Ungated
3	Conventional Gravity Concrete Dam	CGCD	
4	Earth Core Rockfill Dam	ECRD	

Alternatives 3 and 4 were considered, but not carried forward for study for the following reasons:

Alternative 3, Conventional Gravity Concrete Dam, will have the same basic geometry as Alternative 2, RCC. Experience over the last 15-20 years is that RCC gravity dam unit construction costs are lower than those for conventional gravity dams, primarily owing to the lower cement content (and therefore cost), as well as less formwork and faster construction. Therefore, Alternative 2 is expected to provide the same benefits and reliability at lower cost. Consequently, a conventional gravity concrete dam was not considered further.

Alternative 4, Earth Core Rockfill Dam, requires a substantial source of suitable impervious material for the core, as well as a relatively long dry season construction period. As presented in Appendix B, Geology, Geotechnical and Seismological Studies, and discussed in Section 1.2 of this appendix, suitable sources and quantities of impervious material were not located in the vicinity of the dam site. Some heterogeneous sources of impervious materials that were considered are the alluvial terraces found upstream of the site, and residual soils available around the dam site. These sources would require a piece-meal exploitation procedure resulting in higher costs and unfavorable environmental consequences. The quality of the materials is quite variable; impacting the overall quality of any potential core constructed using them, and any quality control procedures. Experience has shown that unless there are suitable sources of core material in close proximity to the dam site, a CFRD is more economical than an

ECRD. In addition, the extended wet season of eight months will result in a longer construction period for this type of dam than for a CFRD, and the requirements for flood protection during construction are more restrictive than for a CFRD, also resulting in higher cost. The ECRD alternative was, therefore, discarded, and was not considered further.

2.3 Dam Site Location

The dam site is located on the Río Indio at Cerro Tres Hermanas, just downstream of the village of Limon, and about 4 km downstream of the DMH stream gaging station. The axis used for this study was selected primarily for topographical considerations. It takes advantage of a ridge on the left abutment to reduce the volume of the dam types, provides adequate space for cofferdam and diversion facilities, and does not present any access difficulties. The final location of the dam will be confirmed following the site investigation program during final design.

2.4 Spillway Design Flood

The spillway will be designed for the probable maximum flood (PMF). For a project whose failure would 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, i.e. the probable maximum precipitation (PMP) resulting in the PMF.

Estimating the PMF consists of analyzing the basic factors that cause the occurrences of great floods, maximizing these factors to their highest reasonable physical limits, and then combining them within acceptable conditions from a hydrologic point of view, in a manner which produces the maximum flood. The studies to determine the PMF are presented in Appendix A, Hydrology, Meteorology and River Hydraulics.

The maximum peak inflow of the PMF is estimated to be 4,345 m^3/s. The PMF has a 5-day volume of about 242.9 MCM.

2.5 Diversion Flood and Diversion Works

Final selection of a diversion flood will be made as part of the dam optimization studies that will be performed following completion of this study. The selected diversion flood will impact the cost of the diversion tunnel (or culvert) and cofferdams. For this dam type selection study, the following aspects of dam construction were taken into consideration:

1. Conversion of the diversion tunnel to a low level outlet to control reservoir filling and for emergency drawdown of the reservoir during operation;
2. The length of time and cost of repair to critical components of the project that would be subject to damage due to flooding during construction, and;
3. Diversion structure cost differences for alternative dam types.

The minimum size of the diversion tunnel/culvert is determined by drawdown requirements during filling and operation. Provision of drawdown capability is most easily and cost effectively achieved by conversion of the diversion facilities to a low level outlet. For this dam type selection study the following criteria and assumptions were adopted for sizing the emergency drawdown facilities:

1. Drawdown in 30 days from normal full supply level at El. 80 to one third of the reservoir storage elevation, El. 55 (MWH precedent, U.S.B.R. ACER Technical Memorandum No. 3, 1982);
2. Average annual inflow of 25.8 m^3/s;
3. Concurrent trans-basin diversion from Indio to Gatun from El. 80 to El. 40. The HEC-5 trans-basin 4.33-m-diameter tunnel capacity was adopted (112 m^3/s).
4. Provision of a minimum of two control gates for the emergency drawdown facilities.
5. Provision for sediment build-up.

The required average emergency drawdown capacity is 260 m^3/s. This can be provided by a 5.0-m-diameter, D-shaped tunnel (650-m-long), or by twin 2.0-m-wide by 3.0-m-high culverts (250-m-long). Intake control gates can readily be attached to the upstream face of the RCC dam alternative, making the short twin culvert option attractive. However, location of the emergency drawdown control gates is limited for the CFRD alternative. The control gates could be located in a control chamber located on the upstream face of the dam, but this would require the additional cost of a control tower, or access from downstream. If located at the downstream toe, the culverts would require steel lining. For the dam type selection study, a conventional location will be adopted, namely at the crest centerline of the diversion tunnel alternative. Additional options could be considered if the CFRD alternative is selected for optimization.

At this stage of study, the length of time at risk and cost of repair to components of the project that could be impacted by a flood during construction cannot be differentiated with any certainty. Both RCC and CFRD-type dams will be constructed in two to three years after diversion facilities are completed. Therefore, the diversion flood for the two alternative dam types for this study will be the same. There is less than a 10% probability that a flood greater in magnitude than the 25-year return period flood will occur in two years.

The diversion structure cost difference for the alternative dam types is a function of optimizing the cost of the cofferdam by inversely varying its height (and size) against diversion tunnel or culvert capacity (or size). As the cofferdam is lowered, the diversion tunnel must be increased in size. The diversion tunnel/culvert for the RCC dam alternative has the potential advantage over the CFRD alternative of a shorter diversion conduit due to its smaller footprint. The RCC dam type also has the potential advantage of adapting the diversion tunnel/culvert into a low-level outlet structure at lower cost than the CFRD. The cost differences between diversion plans for each alternative are included in the dam type selection evaluation.

Flood peaks for selected return periods were developed in Appendix A (Table 10), and are presented in Table 2 below:

Table 2: Flood Peaks for Selected Return Periods

Return Period (Years)	Flood Peak (m^3/s)
2	562
5	657
10	713
20	762
25	780
50	820
100	859

Hydraulic analyses for several alternative diversion flood cases and diversion facilities have been prepared. They are presented in Attachment 1, Hydraulic Analysis for Design of Diversion Tunnel and Cofferdam. The 25-year and the 50-year return period floods were used to bracket the flood return period that will likely be selected during the optimization studies following dam type selection. The analysis includes development of a tailwater rating curve, diversion tunnel/culvert analysis and reservoir routing of the selected floods for several diversion tunnel/culvert sizes. In addition, because of the significant attenuation, sensitivity analyses were made on the effective reservoir elevation-volume (see Attachment 1 for discussion). Three cases were run for both the 25-year and 50-year floods:

1. Reservoir volume from ACP HEC-5 analysis;
2. Reservoir volume below El. 20 reduced 25% to reflect topography, and;
3. Reservoir volume below El. 20 reduced an additional 25% for sensitivity.

For the dam type selection study, the diversion tunnel and cofferdams will be sized to pass the 50-year recurrence flood. It has a peak inflow of 820 m^3/s. The selected reservoir elevation-volume case for establishing the crest elevation of the upstream cofferdam was Case 2 with freeboard sufficient to retain Case 3. The minimum diversion tunnel/culvert size for conversion to an emergency drawdown outlet will be selected.

Preliminary layouts have been prepared for a diversion tunnel and twin culverts. The dam site has a relatively wide terrace on the left abutment that can be utilized to construct a culvert for the RCC dam. For the CFRD alternative, the topography favors a diversion tunnel through the right abutment. Cost differences between the RCC dam alternative and the CFRD for conversion of the diversion culverts to an emergency drawdown outlet are included in the costs developed for comparison.

The design flood and diversion facilities selected for the dam type study are:

1. Design flood is selected as the 50-year event with a peak inflow of 820 m^3/s;
2. The diversion conduit for the RCC dam will consist of twin 2 x 3 m culverts located to the left of the river channel. The conduit will be founded on competent rock. It is approximately 250 m long;
3. The diversion tunnel for the CFRD will be a 5.0-m-diameter, D-shaped tunnel, 650 m long. It will be located in the right abutment. An access shaft adjacent to the crest centerline on the right abutment will provide access to the emergency drawdown gates.
4. The upstream cofferdam will be sized to retain a peak flood elevation of El. 24.0, with a freeboard of 2.0 m. The cofferdam will have a crest at El. 26.0 m; and
5. The downstream cofferdam will have a crest at El. 12.0 m.

2.6 Spillways

The reconnaissance study, the USACE selected an ungated spillway located on the left abutment. It would have a crest width of 120 m and a capacity of 920 m^3/s. The spillway chute was proposed as a sloped and/or stepped natural rock cut channel of about 1,100 m length. Review of the supporting cost information showed this to be a relatively costly solution. ACP has indicated a preference for an ungated spillway because of its lower operation and maintenance costs.

For the dam type selection, two types of spillway have been considered for both dam type alternatives; ungated and gated. The crest of the ungated spillway was set at El.80.0, and flood routing performed for several widths. These are presented in Attachment 2, Evaluation of Spillway Sizes and Estimation of Freeboard for Rio Indio Project. The flood routing confirmed the substantial attenuation provided by the reservoir. For the dam type selection study, an ungated crest width of 50 m will be selected, with a corresponding maximum water surface at El. 84.0.

The gated spillway was then sized to provide the same reservoir full supply level of El 80.0 m, and the same maximum reservoir level of El. 84.0 m, with a minimum of two gates. The flood routing studies presented in Appendix B determined that the gated spillway will consist of two gates of 5 m (width) by 7 m (height). General discussion on the two spillway configurations for both the RCC and the CFRD are presented below.

2.6.1 Ungated Spillway for CFRD Alternative

Layouts for an ungated spillway adjacent to the CFRD showed the right abutment to have preferable topographical characteristics. Taking advantage of the right abutment as a source of fill material for the CRFD, the top of the abutment will be excavated to about El. 90, or 5 m above the proposed maximum reservoir elevation. To minimize additional excavation for the ungated spillway, a side channel spillway was selected discharging over a control ogee crest and channel chute to a flip bucket then, via a channel, back to the downstream channel. A spillway located on the left abutment would result in the construction of a longer spillway chute through more variable topographical and potentially geological terrain. The chute would also terminate at an unfavorable angle to the downstream river channel, or would need to be increased in length.

The chute spillway would include an approach channel excavated to El. 75. The spillway ogee crest would be at El. 80.0, the reservoir full supply level. The selected spillway width to pass the PMF is 50.0 m. The maximum discharge is 770 m^3/s. The side channel trough will widen from 10 m upstream to 20 m downstream where a second ogee will control the trough pool to a minimum of El.74. A spillway bridge will span the chute from the crest to the right abutment. The spillway chute will taper from 20 m at its crest to 10 m at the flip bucket. The chute will have an overall length of about 250 m, and a drop of approximately 60 m. A smooth concrete chute will be provided with training walls of about 5 m, and an aeration ramp. The flip bucket will have a lip at El. 14 that is above the expected maximum tailwater elevation under PMF conditions. All spillway structures will be constructed of conventional mass or reinforced concrete founded on rock. The spillway will discharge back into the Rio Indio by means of an excavated discharge channel.

2.6.2 Gated Spillway for CFRD Alternative

The two-bay gated spillway will also be located on the right abutment for the same reasons as the ungated spillway. It will have an approach channel excavated to El. 66.0 m, and an ogee crest at El. 74.0 m. The maximum velocity in the unlined rock approach channel will be about 2.2 m^2/s under PMF conditions when the spillway gates are fully opened and the reservoir is surcharged to its maximum level (El. 84).

Spillway discharge will be controlled by two 5.0-m-wide by 7.0-m-high radial gates, each with a capacity of about 300 m^3/s full open. The spillway structure will be a conventional mass and reinforced concrete structure founded on rock. The headworks will include a crest bridge and operating equipment for the spillway gates. A 13-m-wide spillway chute will extend to a flip bucket. The flip bucket will be located at approximately the same location as for the ungated spillway, and similarly be connected back to the Río Indio by means of a channel.

2.6.3 Ungated Spillway for RCC Alternative

The ungated spillway for the RCC alternative will be located on the dam and aligned to discharge directly into the Río Indio. It will include a crest at EL. 80.0 m and effective hydraulic width of 50 m. The crest road will be supported by means of two piers. A smooth chute of conventional concrete will be provided with training walls. The spillway chute will terminate in a flip bucket at El. 14.

2.6.4 Gated Spillway for RCC Alternative

The gated spillway for the RCC alternative will be located with the same centerline as the ungated spillway. It will comprise similar crest and control features as the CFRD ungated spillway. A 13-m-wide chute will connect the headworks to a flip bucket at El. 14. The gated spillway, chute and flip bucket will be constructed of conventional concrete on the RCC dam.

2.7 Other Features

In addition to the dam and spillway, there are a number of other features that comprise or are required for construction of the Río Indio Water Supply Projects, including:

- Cofferdams;
- Saddle Dams;
- Multi-level Outlet Structure;
- Hydropower Facilities;
- Access, and;
- Construction Area (laydown and around dam).

These are described below together with how they are addressed in the dam type selection study.

2.7.1 Cofferdams

Cofferdams will be constructed upstream and downstream of the project works and will be of sufficient height to protect the working area against flooding during the selected diversion flood. The cofferdams will be constructed along an alignment parallel to the main dam at a location far enough upstream and downstream to avoid conflicts between construction of the main dam and the cofferdam. For the dam type selection study, a minimum distance of 25 m will be provided between the toe of the cofferdam and the foundation excavation limits. The upstream and downstream slopes of the cofferdams will be 3H:1V and 2H:1V respectively. The cofferdams will be constructed of compacted and dumped random fill on the in-situ alluvium. Less pervious material will be placed on the water retaining face. The crest width will be 5 m. Crest elevations, which are the same for both dam types, are given in Section 2.5.

2.7.2 Saddle Dams

Creation of a reservoir with a normal full supply level at El. 80 m would require construction of two saddle dams to the right of the main Río Indio dam. For the purposes of the dam type selection studies, it has been assumed that the saddle dams will be the same for either alternative. Therefore, the sizing and construction costs are not included in the comparative cost estimates for the dam type selection study.

2.7.3 Multi-level Outlet Structure

Provisions for a multi-level outlet structure are required for the Río Indio Water Supply Project to provide control of water quality for ecological downstream releases. The outlet structure will be capable of releases at various elevations from El. 80.0, the normal reservoir elevation to EL. 20.0, approximately 10 m above the current upstream riverbank elevation. The outlet structure is sized to provide a minimum release of 2.6 m^3/s, which is equivalent to 10% of the mean annual flow at the dam site.

The relatively small multi-level outlet structure will be constructed on the upstream face of the RCC and CFRD dam type alternatives. Slide gates or valves will be housed in an outlet structure. Operation of valves at selected elevations will provide withdrawal at the required level. Discharge will be through the outlet tower to a penstock located at dam foundation elevation. The required penstock diameter is 0.8 m. If required, increasing the penstock diameter to 1.0 m will double capacity. The penstock will terminate at control valve and flume for measurement and variation of required minimum releases. A bifurcation will be included for possible future addition of hydropower. Differences in complexity and cost are too small to be captured in this dam type comparison study. Therefore, the multi-level outlet structure is not included in the dam type selection cost comparison.

2.7.4 Hydropower Facilities

The project has been planned with provision for incorporating hydroelectric generation. On-going studies are being performed to identify the generation potential and the hydropower facilities that will be constructed at the Río Indio dam as well as at the trans-basin transfer tunnel to Lake Gatun. Current studies indicate that hydropower generation at Río Indio will likely be limited to minimum release flows. The facilities will be relatively small and will be an addition to the multi-level outlet works. Therefore, for these dam type selection studies, the hydropower facilities will be similar, and are not included in cost comparisons.

2.7.5 Access

New access roads to the dam site and to quarries will be required. Access roads are not specifically included in the dam type selection study as cost differences are small and have not been defined.

2.7.6 Construction

Construction camps and facilities will be required prior to commencement of the main Río Indio dam and facilities construction contract. For the dam type selection study, they have been estimated to be similar and have not been included in comparative cost estimates. The RCC dam will require specific installations for material storage and handling and the costs of these installations are included in the general costs of the RCC.

3 GEOLOGIC AND GEOTECHNICAL CHARACTERISTICS

Geologic and geotechnical information used as the basis for input to the dam type selection process was obtained during two visits to the proposed Río Indio dam site, one in September 1999 and another in January 2000. Investigations included general reconnaissance of the project area, basic geologic mapping at the damsite and appurtenant structures, identification of potential construction material sources, and material sampling and testing. Although further investigations to investigate subsurface conditions by drilling and geophysical surveys have been planned, the results of these additional investigations were not available for the dam type selection study.

Only those geologic and geotechnical characteristics pertinent to the dam selection studies are addressed in this section, i.e. bedrock type and construction materials. A more detailed description of the local geology and geotechnical characteristics is contained in Appendix B, Geology, Geotechnical and Seismological Studies, of the main report.

3.1 Site Geology

Bedrock units in the Río Indio project area consist of Tertiary sedimentary and volcanic rocks. The sedimentary formations comprise tuffaceous siltstones and sandstones, conglomerates and agglomerates. These are interbedded with lavas and in some parts of the reservoir area, the sedimentary rocks are stratigraphically overlain by andesite and basalt flows. The volcanic units form many of the steep hills and high plateaus that are readily apparent on topographic maps and aerial photographs. Some of the volcanic formations might represent older units cropping out as erosional inliers.

At the damsite, the Río Indio flows northwest forming an asymmetrically shaped valley that exhibits nearly 100 meters of relief. The right abutment is characterized by relatively steep slopes while the left abutment is formed by more gradual slopes. Both abutments are almost entirely covered with colluvial and residual soils, and are moderately to heavily vegetated. Most of the bedrock throughout the project area has a moderate to deep weathering profile, as is typical of regions with a sub-tropical climate.

Bedrock at the dam site consists of medium- to fine-grained tuffaceous sandstone. Several small outcrops of sandstone and sandstone float are present around the damsite and it is presumed that these sandstone and tuffaceous sequences form the bedrock of the entire damsite.

The valley floor at the damsite is approximately 200 m wide and is filled with alluvial terrace deposits consisting primarily of silts and clays. The river has cut steep banks, up to 5 meters high, into the terrace materials. Bedrock is exposed in parts of the riverbed and along the cut banks of the lower right abutment.

3.2 Available Construction Materials

Four primary construction material sources were identified based on field reconnaissance, aerial photograph analysis and study of topographic maps. Each of the four material types are discussed below; their locations are indicated on Exhibit 2.

Alluvial deposits. Several alluvial deposits were identified on aerial photographs and topographic maps as potential sources of construction materials. The most significant sources of alluvial deposits located in the vicinity of the dam site form 4- to 5-m-high terraces along the banks of the Río Indio approximately 2- to 3-km upstream of the dam site. The alluvial terraces were investigated through excavation of test pits and were found to be composed predominantly of clayey silt with some layers of clayey sand and inorganic clay. These materials were tested in the laboratory and found to be suitable for use as impervious fill for cofferdams and secondary fills. Their use in the core of an ECRD is not considered feasible because of their heterogeneity, limited available volume, and location.

No deposits of clean alluvial sands and gravels were encountered in the test pits nor were observed elsewhere in the field. Alluvial sands and gravels that are used locally as aggregate are dredged from the river channel or mined from small gravel bars along the river. These deposits are relatively small and limited in extent, and therefore are not considered a suitable or economic material resource for the construction of the project.

Colluvial and residual soils. Most of the bedrock in the project area is covered by well-developed horizons of colluvial and residual soils. A test pit was excavated in the slope of the left abutment to investigate the depth and type of overburden. Materials in the pit were found to consist of about 1.2 m of clayey silt on top of weathered tuffaceous sandstone and siltstone bedrock. It is presumed that most of the overburden in the project area is clay-rich due to the tuffaceous nature and volcanic origin of the bedrock. This material was tested in the laboratory and was found to be suitable for use as impervious fill. However, its exploitation and use as core material in ECRD construction is not considered feasible because of its expected high moisture content, and the difficulty of conditioning the material. Available quantities are limited and production rates would not be economic.

Sandstone and siltstone. Tuffaceous sandstones and siltstones form the uppermost bedrock unit and are widespread throughout the project area. Most of the bedrock is covered by overburden, however in many areas shallow landslides have removed the overburden to expose underlying in situ bedrock. A potential sandstone quarry source is located approximately 3 to 4 km south-east (SE) of the proposed dam site. Laboratory testing conducted on tuffaceous sandstones and siltstones indicates that the material is of sufficient strength to be used as rockfill, however its durability is such that it most likely would not be suitable for use as concrete aggregate or for select processed fills (i.e. filters, drains, riprap).

Andesite and basalt. Andesite and basalt rock units form many of the steep hills and high plateaus in the project area. Several potential quarry sources were identified:

- Cerro del Barrero, located about 4 km S-SE of the dam site; and
- Cerro La Jota, Cerro del Duende, and Palmira, located between 4 and 9 km E-NE of the dam site.

Rock samples collected from the potential quarry sites E-NE of the dam site were tested in the laboratory. Results indicate that the material would be suitable for use as rockfill, processed select fills, and concrete aggregate.

3.3 Geotechnical and Geologic Design Considerations

Generally, the geologic and geotechnical factors that most influence selection of dam type fall into the following categories:

- General foundation bedrock acceptability, including sliding resistance and deformation characteristics of foundation;
- Required excavation depths to achieve acceptable foundation materials;
- Measures required to treat the foundation to improve physical properties and control leakage;
- Long-term performance of the foundation under normal operation conditions and extreme events, especially earthquake; and
- Availability of suitable construction materials.

Such geological and geotechnical factors can have direct influence on the development of comparative construction costs and were taken into consideration during the study of dam type alternatives. However, in the absence of subsurface investigation data, the process had to be based on qualitative evaluations involving engineering judgment and previous experience in similar geological environments.

- **Foundation Bedrock Characteristics.** 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 design details and construction practices. This is in contrast to some sedimentary rock units and geotechnical conditions known from the Canal Zone (e.g. Cucaracha Formation), where sliding and foundation failures have been common and presented serious problems.
    - *Bearing Capacity.* The tuffaceous siltstones and sandstones are relatively soft rocks but are expected to present adequate bearing capacity to support any of the structures being considered. However, their moderately low modulus of deformation may result in settlement and/or displacements between adjacent structures of different size and weight. Data from subsurface investigation and testing will be needed to develop design and construction details to deal with such behavior.

- *Resistance to Sliding*. The sandstone should provide adequate resistance to sliding along bedding planes or other planes of weakness provided excavation depths are sufficient to achieve fresh sound bedrock.

- **Excavation Depths**. Based upon observations made at the site and comparison with rock types elsewhere in similar environments, an average overburden thickness of 4 m was assumed, i.e. depth to top of weathered rock. An average depth to the top of competent rock was assumed to be about 8 m. These values were used in the development of preliminary layouts and in the computation of quantity takeoffs for cost estimates. Actual depths and characteristics of weathering need to be properly investigated by drilling and geophysical exploration since these are very sensitive inputs to the cost estimates.

  - The test pits could not penetrate deep into the residual soils and saprolites covering bedrock at the site, but a fully developed weathering profile several meters thick is expected. Particular concern and attention during any future subsurface exploration program should be given to investigate the nature of the two differently sloping abutments. Compressibility of the soils and saprolites left under the downstream part of a rock fill embankment would impact dam behavior and the effect of the narrow ridge on the left abutment should be evaluated if it is left under a rockfill dam.

  - Variations in foundation quality over short distances could have a more serious impact for rigid structures (gravity dam alternatives) than for fill dam alternatives. For example, the foundation excavation footprint for the RCC alternative is larger than for the CFRD plinth excavation (both assumed to be average 8 m deep). If excavation to obtain a competent foundation were greater than assumed, then costs would increase more for the RCC scheme than for the CFRD scheme.

- **Foundation Improvement, Treatment, and Long-Term Performance**. No special foundation improvement or treatment measures are expected for the Rio Indio site that would influence selection of one dam type over another. Similarly, the sandstone bedrock is expected to perform satisfactorily over the lifetime of the project without adverse deterioration. During subsequent investigations, the potential for internal erosion of the sandstone under high seepage pressures and flows should be studied to determine appropriate design and construction details.

- Natural slope stability. No large mass movements are expected to affect the reservoir, but the effect of saturation, say after intense rainfall, on the stability of residual soils and saprolites needs to be properly evaluated. This could be significant in design of safe spillway cuts.

- **Construction Materials**. Sandstone from required excavation, provided it is not entirely decomposed, could be used as rockfill material. It is expected, however, that handling, placement, and compaction of the relatively weak sandstone would result in production of fines. This can be handled through appropriate design of the zoning of the dam and construction specifications for material handling.

All aggregates (including coarse and fine aggregates for concrete, filters, drains, and riprap) need to be manufactured from quarried sources. To decrease the cement requirements for the RCC, some mixing with imported sands and silts will be necessary, impacting the cost of production.

Geotechnical design criteria used for developing preliminary layouts and cost estimates for dam type selection are presented in Table 3, following:

Table 3: Geotechnical Design Parameters

Parameter		Selected Design Criteria
Thickness of overburden (top of weathered rock)		4 m
Depth to top of competent rock		8 m
Rock excavation slopes		1H:5V
Soil excavation slopes		
	Permanent	2.0H:1V, 3-m-wide benches every 10 m vertically. Bench at soil-rock contact
	Temporary	1.5H:1V, 3-m-wide benches every 10 m vertically. Bench at soil-rock contact

4 ALTERNATIVE DAM TYPE LAYOUTS AND COSTS

This section gives a brief description of both of the alternative dam type development concepts and highlights the pertinent differences between them.

4.1 Concrete Face Rockfill Dam

A site plan and profile of the concrete faced rockfill dam (CFRD) is presented in Exhibit 3. The general arrangement shows the dam, cofferdams, and diversion culvert alignment together with excavation of the right abutment hilltop for dam fill. Exhibit 4 shows a typical CFRD section, three cross sections for this location, and the diversion tunnel profile.

The centerline of the alignment was selected to minimize dam volume. The alignment will be confirmed during subsequent studies of the selected dam type to include any additional information from planned geotechnical explorations. The upstream and downstream cofferdams were located to provide adequate construction and laydown areas while minimizing the length of diversion culvert. The minimum distance to the upstream cofferdam was established at 25 m. This provides working space for the construction of the required grout curtain at the toe of the main dam. A grout curtain cutoff will be constructed to a depth of 40% of the hydraulic head. The minimum distance to the downstream cofferdam is also 25 m.

The CFRD will be constructed of selected rockfill obtained from the right abutment excavation and nearby rock quarries. For the dam type selection, the slopes of both the upstream and downstream faces are 1.4H:1.0V, reflecting the relatively low seismicity of the location. These slopes will be optimized during subsequent studies when stability analyses are performed. An upstream parapet wall of 5 m and a downstream retaining wall of 1 m will form the dam crest respectively, to reduce dam volume. The upstream parapet wall will be extended an additional 2 m above the crest of the fill to provide freeboard.

The dam will be constructed with a conventionally placed reinforced concrete upstream facing as an impermeable membrane. It will be designed to have (1) low permeability, (2) sufficient durability against weathering, and (3) sufficient flexibility to tolerate small expected embankment settlement. The concrete facing will be constructed with (vertically placed) slabs with intermediate waterstops. A zone of fine gravel and sand will be placed beneath the concrete face to provide continuous support for the concrete facing. It will prevent movement of material into the main rockfill and will be about 3 m wide. It is expected that this support zone will be placed using an upstream extruded concrete curb to provide confinement during compaction and protection against erosion during construction.

A reinforced concrete plinth, or toe slab, also used as a grouting platform will be placed along the upstream toe. This plinth will be extended downstream, as needed, to lengthen the seepage path as required by the rock encountered.

The cofferdams will be constructed of random fill that will be obtained from portions of the required excavations for the main dam, saddle dam and spillway. The construction sequence of the main fill will include construction of a preferential fill or internal cofferdam, which will help protect the dam construction during flood events of unexpected scale. The diversion tunnel will be located in the right abutment. It is aligned to avoid interference with the selected CFRD spillway, and to facilitate conversion to a low level outlet for emergency drawdown. The alignment permits construction of an access shaft in the right abutment from the dam crest to an outlet control gate chamber in the diversion tunnel.

Pertinent data is provided in Table 4, following.

4.1.1 Ungated Spillway

The ungated spillway plan and sections are presented in Exhibits 5 and 6. The spillway will be a side channel spillway located on the right abutment. A crest road access bridge will span the spillway channel just downstream of a control ogee. The side channel spillway, chute and flip bucket are described in Section 2.6.1.

Table 4: CFRD Design Parameters

Dam Section		
	CFRD	
	Alignment	See Plan
	Crest elevation	83 m
	Parapet elevation	85 m
	Maximum reservoir elevation	84 m
	Crest width	8 m
	Upstream slope	1.4 H : 1 V
	Downstream slope	1.4 H : 1 V
	Concrete face thickness	0.3 - 0.5m, 0.4 m, av.
	Transition fill width	3 m
	Plinth width	4 m
	Foundation	
	Plinth plus 1/3 dam height	Competent rock
	Embankment (less 28 m)	Weathered rock
	Material Volumes, m^3	
	Rockfill	2,690,000
	Filter for concrete	240,000
	Concrete	50,000
	Foundation excavation	850,000
Cofferdams		
	Upstream cofferdam	
	Alignment	See Plan
	Distance from dam	25 m (min)
	Crest elevation	26 m
	Fill volume, m^3	145,000
	Downstream cofferdam	
	Alignment	See Plan
	Distance from dam	25 m (min)
	Crest elevation	12 m
	Fill volume, m^3	6,000

Crest width	5 m
Freeboard	1 m
Upstream slope	3 H : 1 V
Downstream slope	2 H : 1 V
Foundation	
Impervious Element	Weathered Rock
Shells	Stripped Overburden

4.1.2 Gated Spillway

The gated spillway plan and sections are presented in Exhibits 7 and 8. The spillway headworks will be a two-bay structure containing the spillway gates, hoists, stoplog slots, and operating equipment. The crest road access bridge will span just downstream of the headworks. The gated spillway, chute and flip bucket are described in Section 2.6.2.

4.2 Roller Compacted Concrete Dam

The site plan and profile for a RCC dam is presented in Exhibit 9, and typical sections are shown in Exhibit 10. The centerline of the dam is located on the same axis as the CRFD to minimize dam volume. The cross section has been selected to provide adequate stability under the prevailing site conditions (low seismicity and moderate foundation strength) based on experience. The selected cross section will include a vertical upstream slope, and a downstream slope of 0.75H:1V. The crest will be 8-m-wide at El. 83.0, with a parapet wall to El. 85.0. An upstream cut-off grout curtain and an under-drainage system will be constructed.

The current alternative assumes a low-paste RCC, utilizing bedding mixes, as required, particularly at the rock-concrete interface, and at the upstream end of each lift, and an upstream impervious membrane, and drainage system. The impervious membrane and drainage system will be connected to a conventional reinforced concrete gallery situated in the upstream toe. This gallery will be used for construction of the grout curtain and drainage curtain.

Pertinent data is provided in Table 5, following.

Table 5: RCC Design Parameters

Dam Section	
RCC and gravity	
Alignment	See Plan
Crest elevation	83 m
Parapet	2 m
Crest width	8 m
Upstream slope	Vertical
Downstream slope	0.75 H : 1 V
RCC uncompacted section on Downstream face	0.5 m
Foundation	Competent rock
Material Volumes, m^3	
RCC	770,000
Conventional concrete	60,000
Foundation excavation	370,000
Cofferdams	
Upstream cofferdam	
Alignment	See Plan
Distance from dam	25 m (min)
Crest elevation	26 m
Fill volume, m^3	145,000
Downstream cofferdam	
Alignment	See Plan
Distance from dam	25 m (min)
Crest elevation	12 m
Fill volume, m^3	6,000

Crest width	5 m
Freeboard	1 m
Upstream slope	3 H : 1 V
Downstream slope	2 H : 1 V
Foundation	
Impervious Element	Weathered Rock
Shells	Stripped Overburden

4.2.1 Ungated Spillway

The ungated spillway plan and section is presented in Exhibit 11. The spillway will have the same hydraulic capacity as the CFRD ungated spillway. It will be located on the RCC dam to discharge directly by means of a chute and flip bucket into the existing Río Indio channel. A crest access bridge will be provided. The spillway will be constructed of conventional reinforced concrete.

4.2.2 Gated Spillway

The gated spillway plan and section is presented in Exhibits 12. It will feature similar hydraulic capacity and a headworks structure as is proposed for the CFRD alternative, but the headworks will be placed directly onto the RCC dam. It will discharge by means of a chute spillway into a flip bucket and then into the Río Indio. It will be constructed of conventional reinforced concrete.

5 EVALUATION OF THE ALTERNATIVE DAM TYPES

The objective of the dam type evaluation is to select the dam type and corresponding spillway type to be carried forward to the optimization phase of study. The evaluation process will also be used to identify specific aspects of the selected dam type that require additional study. This evaluation also makes recommendations for additional data collection.

5.1 Factors Considered in Dam Type Selection

The evaluation of the alternative dam types is based on the following factors:

1. Construction cost;
2. Construction considerations;
3. Foundation considerations, and;
4. Operation and Maintenance considerations.

Both dam types and spillway types have been developed to provide the same level of performance. The dam and spillway types have been developed to:

- Minimize the initial construction cost of the project by minimizing the dam size for the selected design parameters;
- Minimize technical difficulties that might be encountered in project construction through project configuration;
- Account for potential foundation related difficulties that might become apparent during future investigation programs, and;
- Minimize project operation and maintenance costs, or the possibility of encountering unique and difficult to solve remedial costs.

The alternative that best satisfies the stated objectives, and any other specific owner requirements, will be the recommended alternative.

5.1.1 Initial Construction Costs

To develop the initial construction costs for each alternative, preliminary quantities and costs have been prepared. Only quantities and costs that are judged to vary by alternative have been estimated. The following common costs are not included in the cost estimates:

1. Access Roads;
2. Construction Facilities;
3. Saddle Dams;
4. Cofferdams;
5. Trans-basin diversion tunnel;
6. Reservoir clearing, and;
7. Environmental and socio-economical costs.
8. Contingencies

Unit costs have been estimated for use at this preliminary cost comparison level. Attachment 3, Comparative Cost Estimate presents the costs for diversion and care of water, the dam, the spillway and the low level outlet for selected quantities and unit costs.

Table 6 below summarizes the resulting cost estimates for the alternatives considered, including ungated and gated spillway alternatives.

Table 6: Summary of Comparative Costs

Component	CFRD Ungated	CFRD Gated	RCC Ungated	RCC Gated
		US$ x million, 2002		
Diversion and Care of Water	4.9	4.9	2.9	2.9
Dam	50.4	50.4	52.4	52.4
Spillway	4.9	4.8	1.6	1.8
Low Level Outlet	0.5	0.5	0.4	0.4
Total	**60.7**	**60.6**	**57.3**	**57.5**

The comparative cost estimate shows the RCC alternative with an ungated spillway to be the low cost alternative, 6% lower that the CFRD alternatives. The RCC dam alternative is essentially the same cost for a gated or ungated spillway. The cost of this dam type alternative is primarily dependent on the unit cost of RCC. Historic RCC unit costs for a dam of this volume range from $45/m^3$ to $75/m^3$. A sensitivity analysis showed that the unit cost of RCC would need to be increased by only 8% (from $50/m^3$ to $54/m^3$) for the construction cost of either RCC spillway alternatives to be equal to the construction costs of the CFRD alternatives. This is within the accuracy of this comparative cost estimate. In addition, the cost of the RCC dam is sensitive to changes in its estimated volume. This could vary as a result of foundation conditions, and result in significantly lower or higher cost for this alternative.

While the CFRD is marginally lower in cost than the RCC dam, diversion and care of water, spillway and low level outlet costs are all higher for the CFRD alternative than for the RCC dam alternative. As a result, the overall cost of either of the CFRD alternatives is higher than the RCC alternatives. The costs of the CFRD ungated and gated spillway alternatives are essentially the same. The CFRD cost is less sensitive to unit cost or volume changes; for example, a 15% change in rockfill unit cost or quantity results in a 7% change in CFRD cost.

The difference in initial construction cost between the RCC alternatives and the CFRD alternatives is not sufficient to make an evaluation on cost alone. It should be noted that contingencies have not been included on either quantities or unit costs. For the feasibility level estimate, contingency allowances will be included for the following:

1. Uncertainties attributable to unforeseeable adverse geological conditions;
2. Variations in the cost of permanent equipment and resources for construction due to changing market conditions;
3. Modifications in design resulting in an increase in construction;
4. Minor items not detailed at this time, and;
5. Overlooked and unforeseen items that may not be included in the present feasibility-level quantity estimates.

The application of contingencies to provide reasonable coverage for the first two items will favor the CFRD alternative over the RCC alternative.

5.1.2 Construction Considerations

Construction considerations have been evaluated on the basis of the following objectives:

- Minimize the need for off-site materials;
- Minimize the duration of the construction activities;
- Minimize the consequences of flooding due to streamflow in excess of the diversion dam floods, and;
- Maximize the use of available construction technology, specifically in Panama.

Construction considerations do not clearly favor either of the dam alternatives or spillway alternatives, as discussed below.

- The CFRD alternative takes more advantage of local materials. Basically all materials used for construction of the dam are found at relatively close distance to the site of construction. Construction of the RCC dam requires importation of the cement from an offsite factory, or even from outside of the country, and its transportation to the site. This could strain the highway system between the factory or the port, and the site.

- In terms of construction planning, the most significant difference between the two types of dam (CFRD and RCC) is the duration of the construction period. The RCC dam, excluding preparatory works and construction of ancillary structures, can be built in approximately 6 months. This duration will require an average hourly production of 250 m^3/hr. For dams of this size, average production of 400 m^3/hr has been achieved, and therefore it is reasonable to anticipate a six-month duration (120 hr/week). Preparatory works etc. will be completed in 9 to 12 months, and finishing works will be completed in 6 months, resulting in a total construction period for the RCC dam of 21 to 24 months. The CFRD will require the excavation of nearly 2,700,000 m^3 of rock, either for necessary excavation or from quarry. It is anticipated that this volume will take at least 24 months at a production rate 3,700 m^3/day. In addition, preparatory and finishing works will take 9 to 12 months, resulting in a construction period of 33 to 36 months. However, if the Rio Indio Water Supply Project includes the trans-basin diversion tunnel, tunnel construction will determine the overall construction period, making dam type selection independent of construction time.

- The effect of flooding during construction does not clearly favor either of the alternatives. The RCC dam is resistant to damage by overtopping but its vital placement equipment, processing plant and material stockpiles may be damaged during the flood, if not adequately protected or located. The CFRD can be designed and constructed to withstand overtopping almost as effectively as an RCC dam. Interruption in placement of the CFRD would have less impact.

- The CFRD alternative has a definite advantage when previous local experience is considered. Fortuna dam is a 100-m-high CFRD, and was commissioned a few years ago, and the Barrigon dam, a 60-m-high CFRD, part of the Esti Hydroelectric Project, is currently under construction. No RCC dams have been constructed in Panama.

5.1.3 Foundation Considerations

Foundation considerations have been evaluated on the basis of the following objectives:

- Minimize impact of potential adverse foundation conditions resulting from future investigations;
- Minimize concerns relating to the foundation strength characteristics;
- Minimize potential for differential deformation, and;
- Minimize potential for seepage through the foundation.

The foundation is an integral part of any dam, as it provides support to the dam body, and continuity to the water-tightness element of the dam. To perform these functions, the foundation material should have certain minimal attributes:

- The material should have a strength comparable or superior to the strength of the material being placed on top, and
- The foundation should be of low permeability, or else, it should be possible to treat it by grouting or other means to reach a low permeability.

The known characteristics of the site tend to favor the selection of a CFRD alternative. The rock formations found in outcrop, and assumed to extend throughout the site (tuffaceous siltstones and sandstones) present a well developed weathering profile and are naturally soft and of relatively low modulus of deformation. A CFRD does not need to be completely founded on competent rock, usually only the plinth (toe slab) and the upstream end of the fill need to be founded on competent rock, and according to recent trends, even weathered rock and low compressibility saprolite are acceptable foundations for adequately designed CFRDs. Additionally, fill dams, such as a CFRD, are more capable of accommodating the differential deformations associated with low modulus of deformation. It should be noted that the CFRD

The lack of subsurface information at this time also tends to favor selection of a CFRD. Any unexpected subsurface conditions can be handled more easily, and with lesser impact on the overall construction, during construction of a CFRD than during construction of an RCC dam. The competent rock footprint for the RCC dam is larger than the CFRD, thus if excavation to obtain a competent foundation is greater than estimated, then costs would increase more for the RCC scheme than for the CFRD scheme. However, dam type selection should be revisited at the conclusion of the site investigation program if the subsurface conditions are favorable for an RCC.

5.1.4 Operating and Maintenance Considerations

The operation and maintenance considerations of the project have been evaluated on the basis of the following:

- Minimize the potential for overtopping and resulting damage due to improper spillway operation;
- Minimize uncontrolled leakage through the dam or its foundation;

- Minimize the need for maintenance of the dam and the potential for difficult remedial measures, and;
- Minimize the need for maintenance of the spillway and the potential for difficult remedial measures.

Minimizing the potential for improper spillway operation, or failure to operate the spillway (due to gate inoperability or power failure) favors the ungated spillway alternatives. Similarly, operation and maintenance requirements should be marginally lower for a (large) ungated spillway than a (more complicated) gated spillway.

Operation and maintenance considerations tend to favor selection of an RCC dam alternative, although owner's preferences usually override any reasoning on this issue. CFRDs and RCC dams are both safe and reliable but the upstream grouting and drainage gallery built-in in the RCC dams allow execution of remedial grouting and foundation drainage, if required.

Leakage through either type of dam should be negligible. Although some early RCC dams experienced leakage problems, more recent designs and construction techniques have virtually eliminated this problem. CFRD face slabs may be subject to cracking if poor quality control of concrete placement occurs, or subsidence causes cracking. The general experience is that appropriate design and specification requirements coupled with construction inspection has resulted in CFRD that do not leak any more than other types of dam.

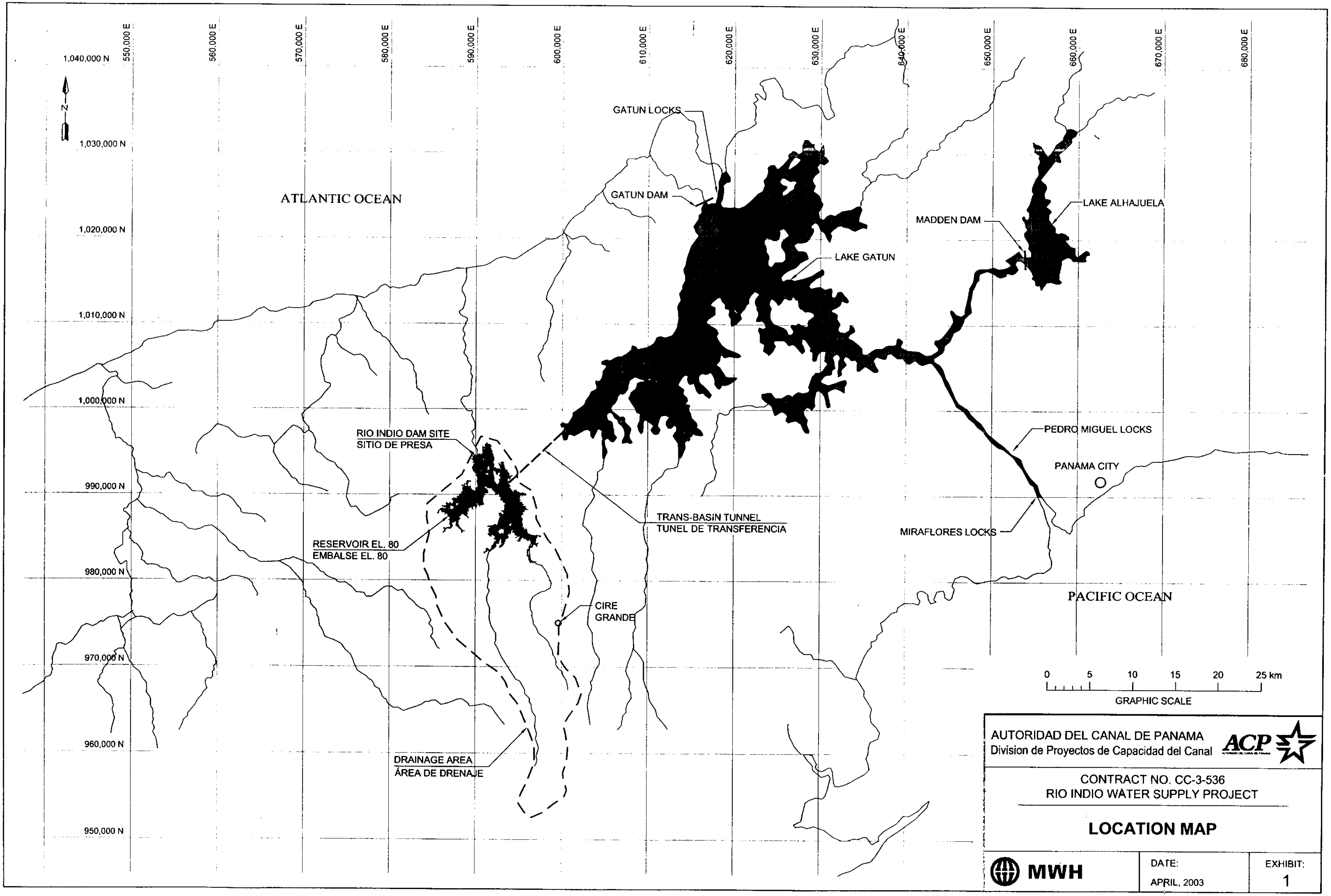
5.2 Recommendation

CFRD and RCC dam types are both technically feasible at this site. The comparative cost evaluation favors the RCC alternative over the CFRD alternative, however, the RCC alternative cost estimate is more sensitive to a cost increase as a result of higher unit cost for RCC, or changed foundation conditions leading to increased volume.

Based on the available information, the dam type recommended for the optimization studies is the ungated CFRD alternative for the following reasons:

1. Changes to the current available foundation information would have less impact on this dam type and cost;
2. There is no advantage to the shorter construction time for RCC;
3. The CFRD cost estimate is less sensitive to variation in unit cost, and;
4. The ungated spillway has lower operation and maintenance cost and risk.

EXHIBITS



ATLANTIC OCEAN
PACIFIC OCEAN
N
1,040,000 N
1,030,000 N
1,020,000 N
1,010,000 N
1,000,000 N
990,000 N
980,000 N
970,000 N
960,000 N
950,000 N
550,000 E
560,000 E
570,000 E
580,000 E
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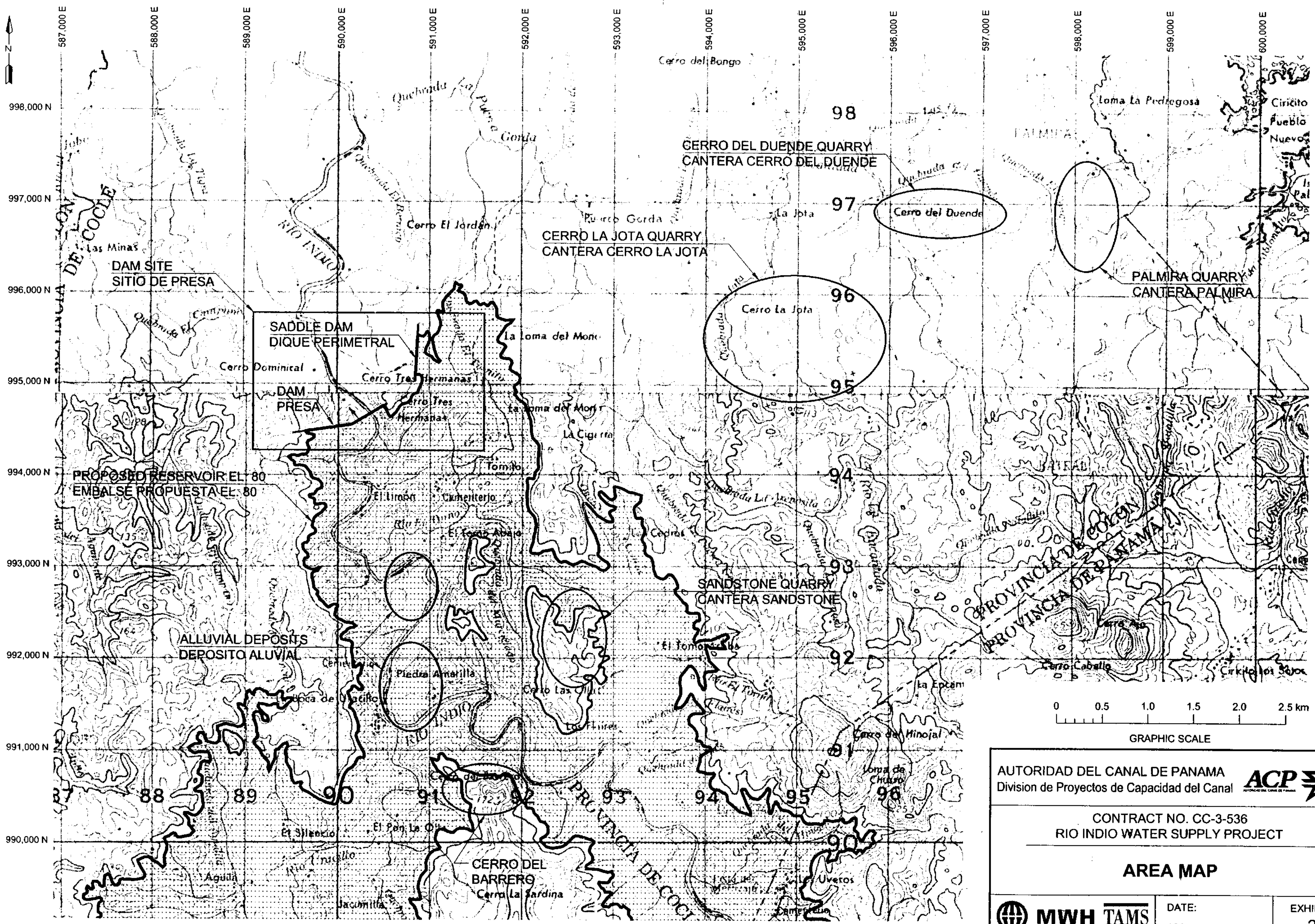
620,000 E
630,000 E
640,000 E
650,000 E
660,000 E
670,000 E
680,000 E
GATUN LOCKS
GATUN DAM
LAKE GATUN
MADDEN DAM
LAKE ALHAJUELA
PEDRO MIGUEL LOCKS
PANAMA CITY
MIRAFLORES LOCKS
RIO INDIO DAM SITE
SITIO DE PRESA
TRANS-BASIN TUNNEL
TUNEL DE TRANSFERENCIA
RESERVOIR EL. 80
EMBALSE EL. 80
CIRE
GRANDE
DRAINAGE AREA
ÁREA DE DRENAJE
0 5 10 15 20 25 km
GRAPHIC SCALE
AUTORIDAD DEL CANAL DE PANAMA
Division de Proyectos de Capacidad del Canal
ACP
CONTRACT NO. CC-3-536
RIO INDIO WATER SUPPLY PROJECT
LOCATION MAP
MWH
DATE:
APRIL, 2003
EXHIBIT:
1



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

ACP

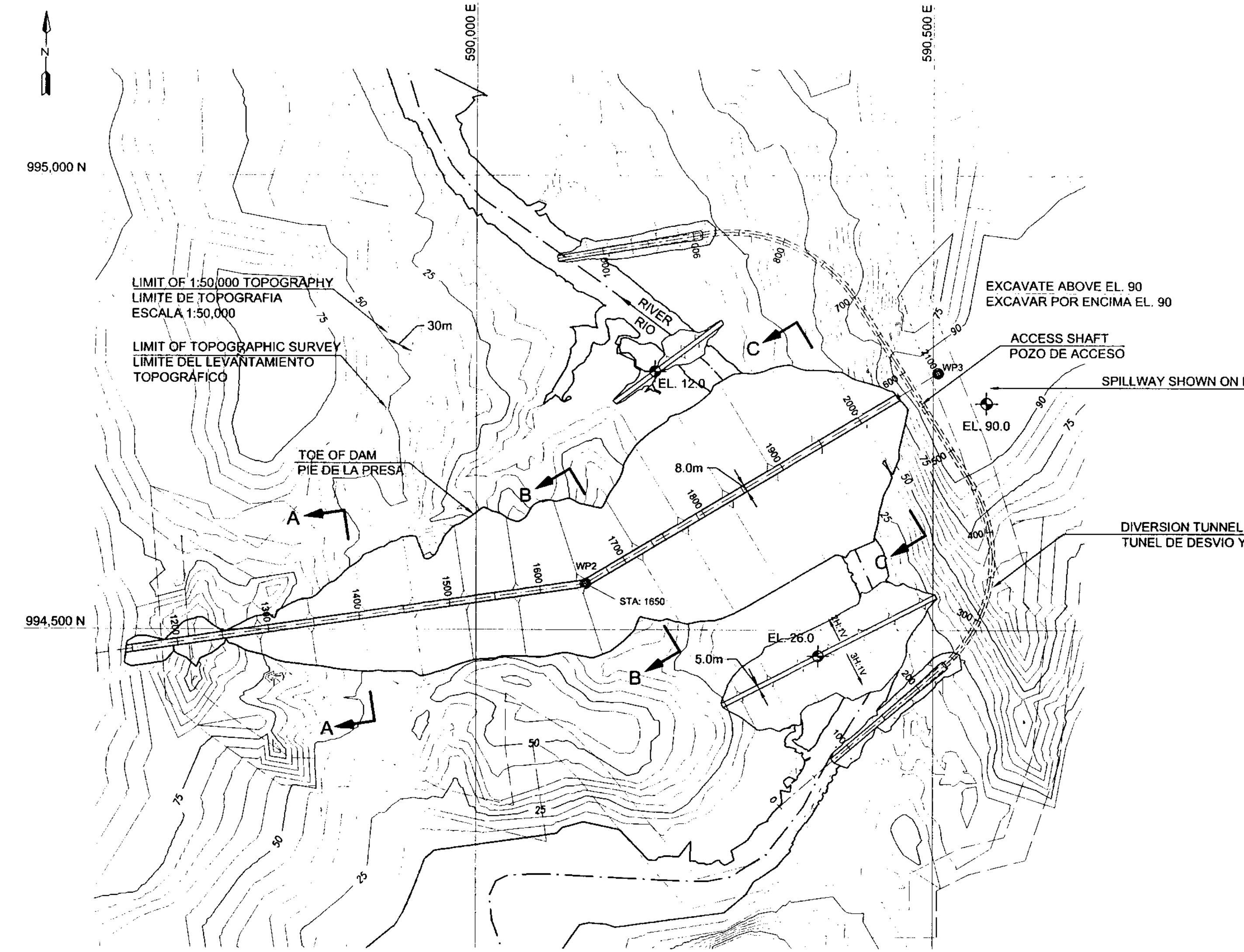
CONTRACT NO. CC-3-536
RIO INDIO WATER SUPPLY PROJECT

AREA MAP

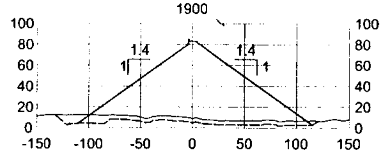
MWH TAMS

DATE: APRIL, 2003

EXHIBIT: 2



SITE PLAN
PLANTA GENERAL

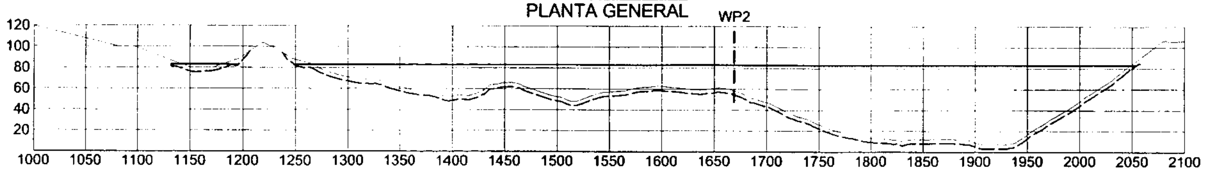


MAXIMUM CFRD SECTION
STATION 1900
SECCIÓN MAXIMA

COORDINATES (UTM):

POINT	STATION	NORTH	EAST
WP1	1000	994456.9	589476.2
WP2	1650	994551.0	590119.4
WP3	2100	994782.8	590505.1

0 50 100 150 200 250 meters
GRAPHIC SCALE



PROFILE OF DAM ALIGNMENT
PERFIL DEL ALINEAMIENTO DE PRESA

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

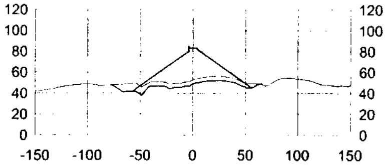
CONTRACT NO. CC-3-536
RIO INDIO WATER SUPPLY PROJECT

CFRD - PLAN

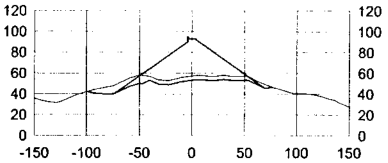
MWH TAMS

DATE: APRIL, 2003

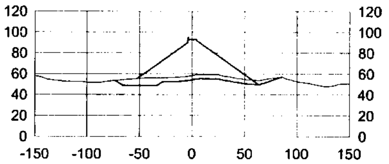
EXHIBIT: 3



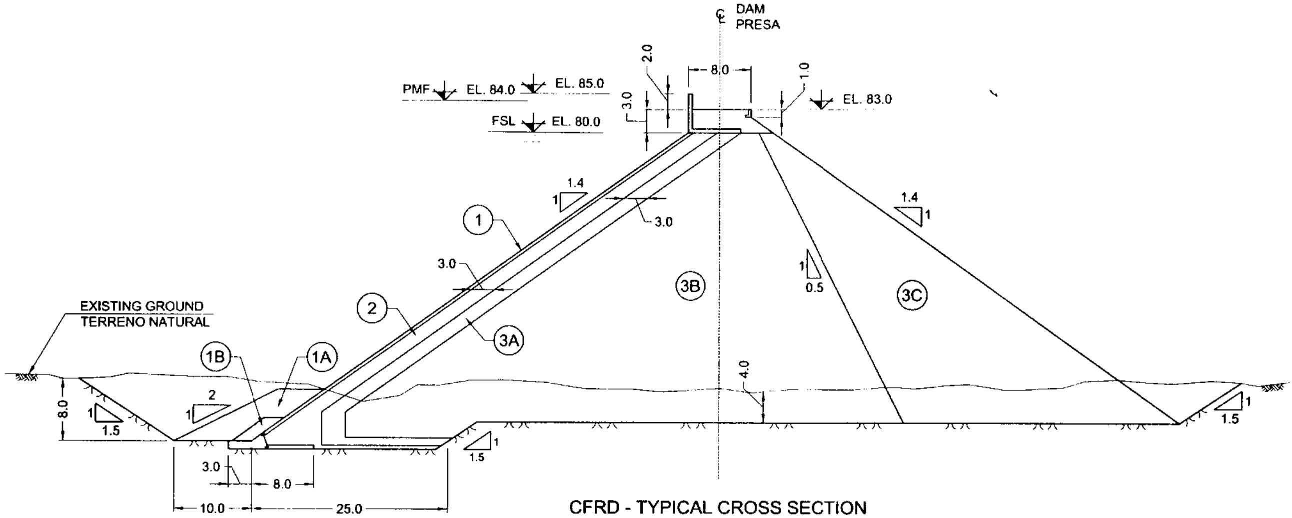
SECTION A
SECCIÓN A



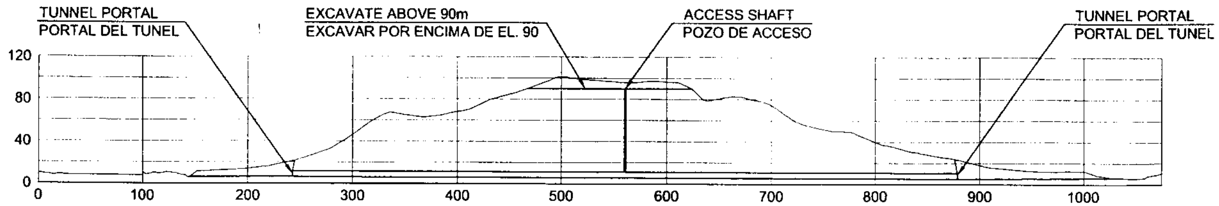
SECTION B
SECCIÓN B



SECTION C
SECCIÓN C



CFRD - TYPICAL CROSS SECTION
PRESA - SECCIÓN TIPICA



DIVERSION TUNNEL LOW LEVEL OUTLET
TUNEL DE DESVIO Y DESCARGA DE FONDO

0 50 100 150 200 250 meters
GRAPHIC SCALE

LEGEND:

1	CONCRETE FACE SLAB	LOSA DE CONCRETO
1B	MISCELLANEOUS FILL	RELLENO HETEROGÉNEO
1C	SILTY SAND	ARENA LIMOSA
2	FILTER	FILTRO
3A	FINE ROCKFILL	ENROCAMIENTO FINO
3B	ROCKFILL	ENROCAMIENTO
3C	COARSE ROCKFILL	ENROCAMIENTO GRUESO

0 5 10 15 20 25 meters
GRAPHIC SCALE

UNITS IN METERS

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

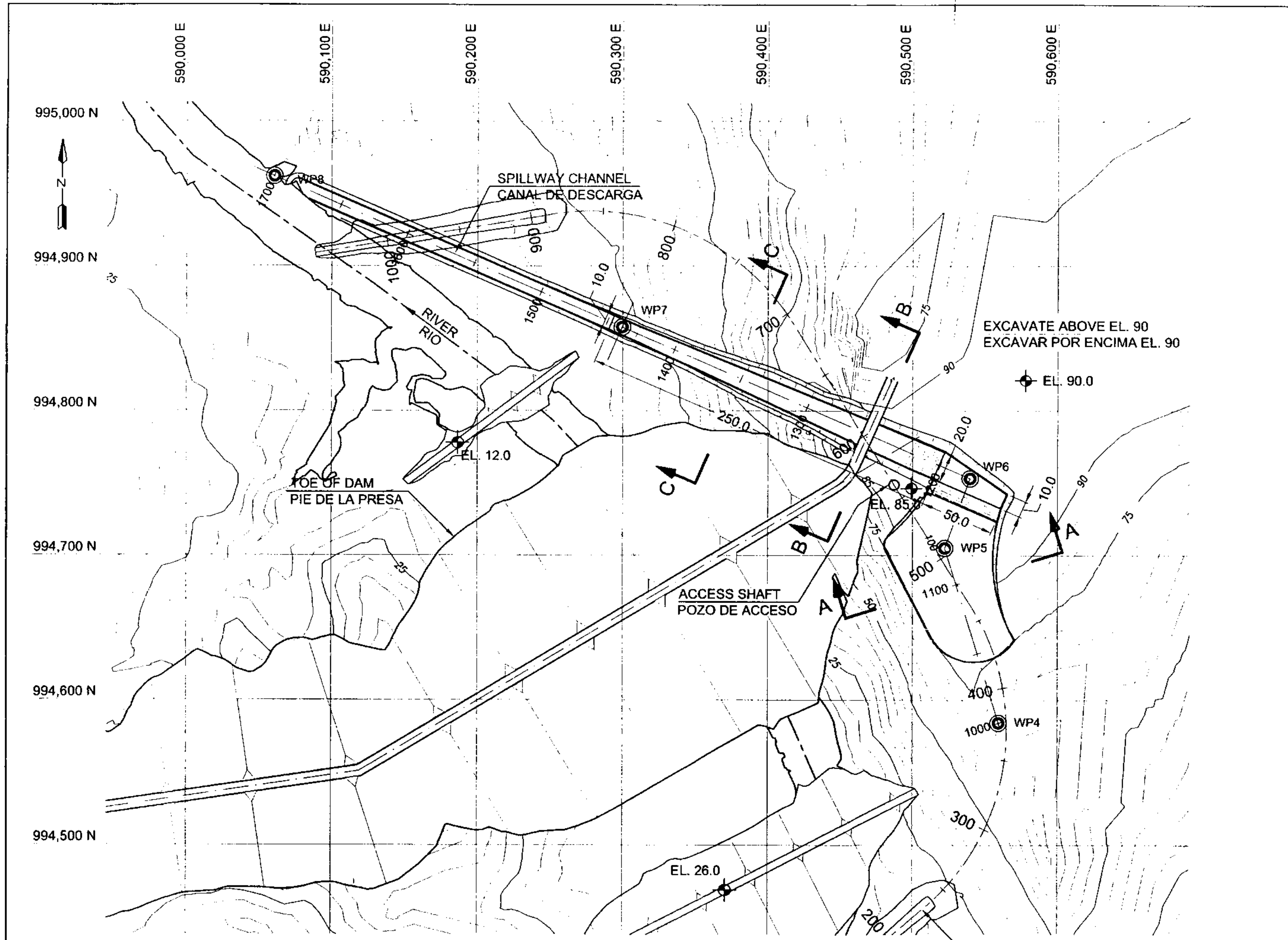
CONTRACT NO. CC-3-536
RIO INDIO WATER SUPPLY PROJECT

CFRD
SECTIONS

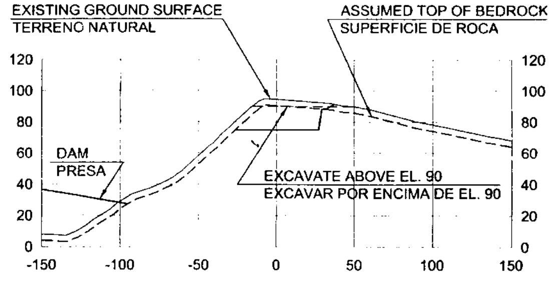
MWH TAMS

DATE: APRIL, 2003

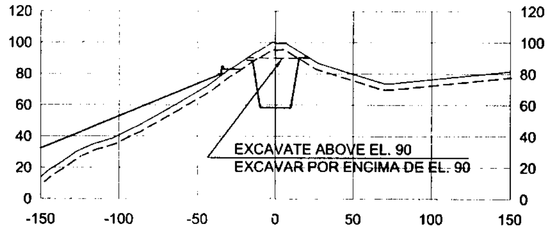
EXHIBIT: 4



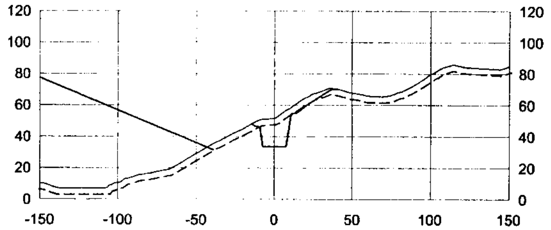
SITE PLAN
PLANTA GENERAL



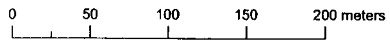
SECTION A
SECCIÓN A



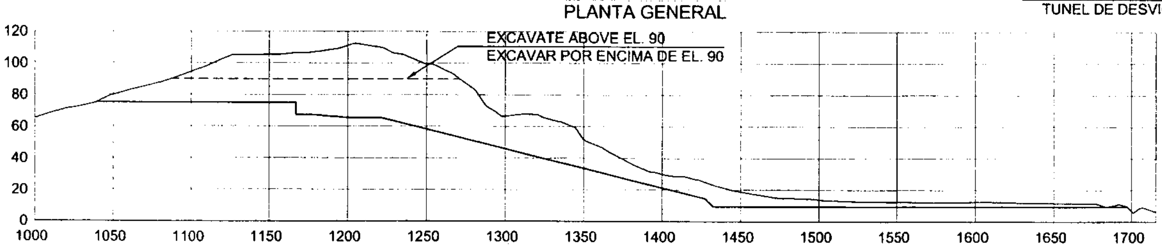
SECTION B
SECCIÓN B



SECTION C
SECCIÓN C



GRAPHIC SCALE



PROFILE OF SPILLWAY ALIGNMENT
PERFIL DEL ALINEAMIENTO DEL ALIVIADERO

COORDINATES (UTM):

POINT	STATION	NORTH	EAST
WP4	1000	994584.3	590559.1
WP5	1126	994705.2	590521.9
WP6	1178	994753.3	590539.4
WP7	1439	994857.8	590299.6
WP8	1700	994962.0	590060.5

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

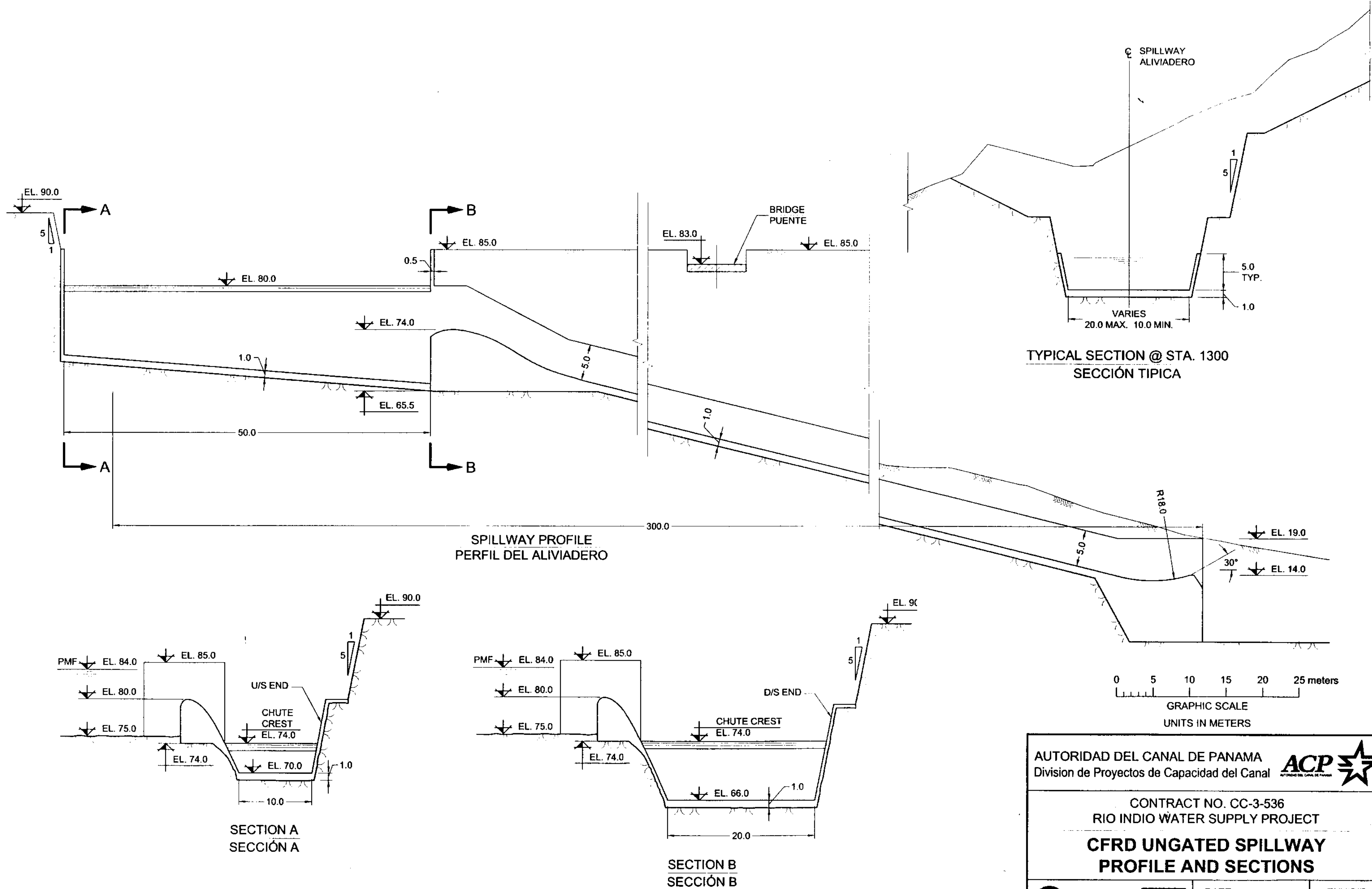
CONTRACT NO. CC-3-536
RIO INDIO WATER SUPPLY PROJECT

CFRD UNGATED SPILLWAY
PLAN AND SECTIONS

MWH TAMS

DATE: APRIL, 2003

EXHIBIT: 5



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

ACP

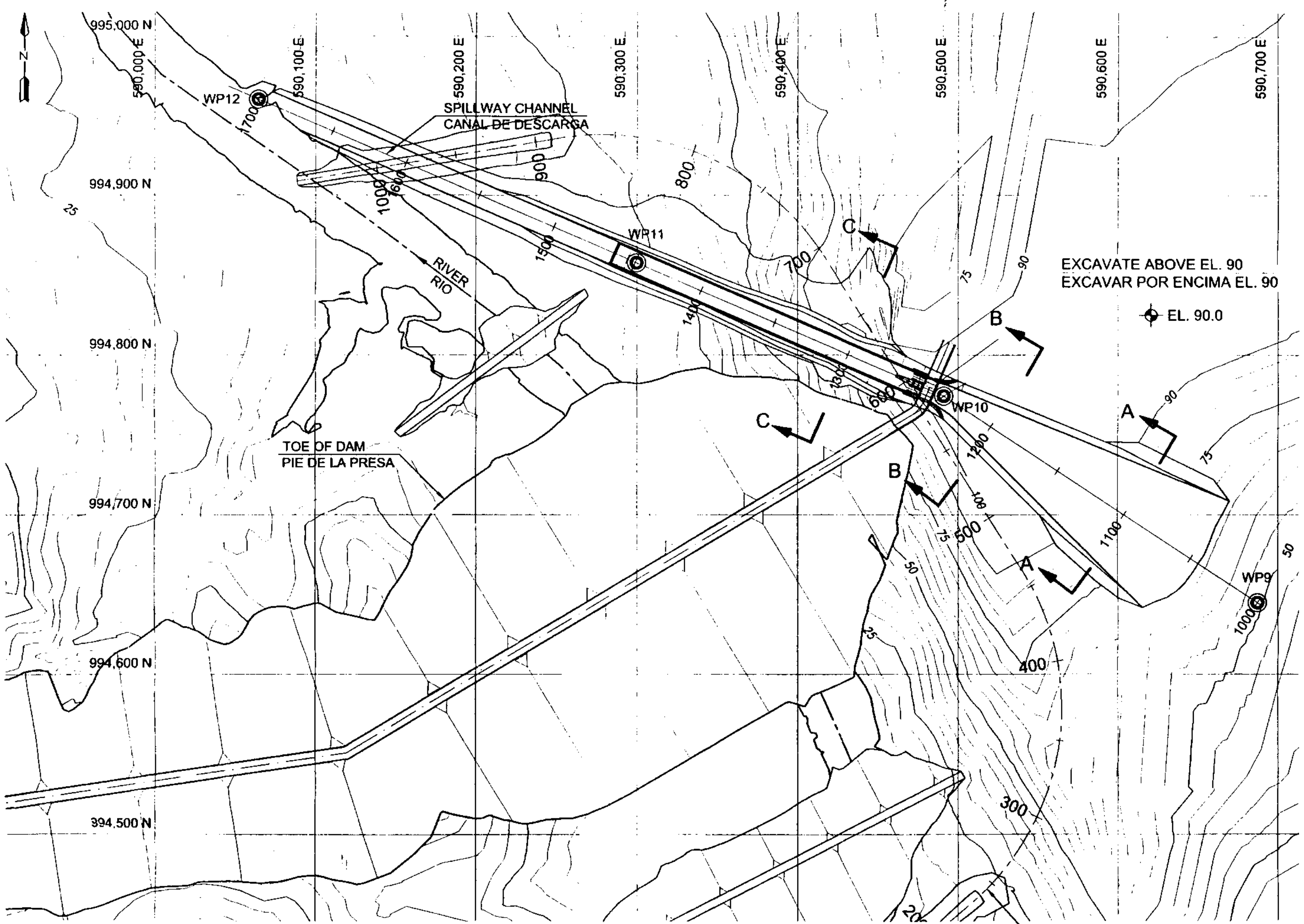
CONTRACT NO. CC-3-536
RIO INDIO WATER SUPPLY PROJECT

CFRD UNGATED SPILLWAY
PROFILE AND SECTIONS

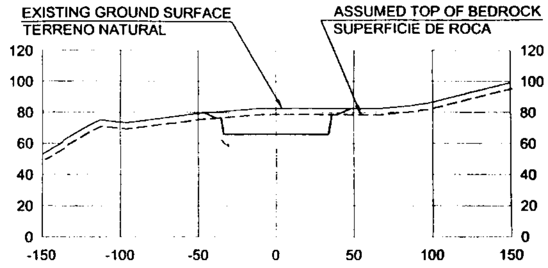
MWH TAMS

DATE: APRIL, 2003

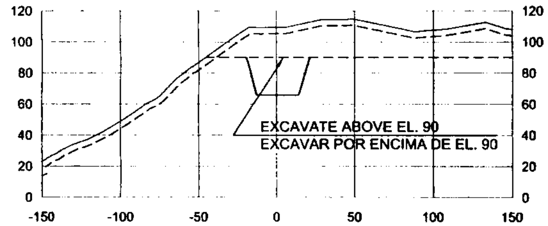
EXHIBIT: 6



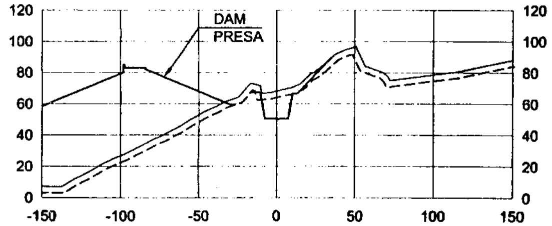
SITE PLAN
PLANTA GENERAL



SECTION A
SECCIÓN A



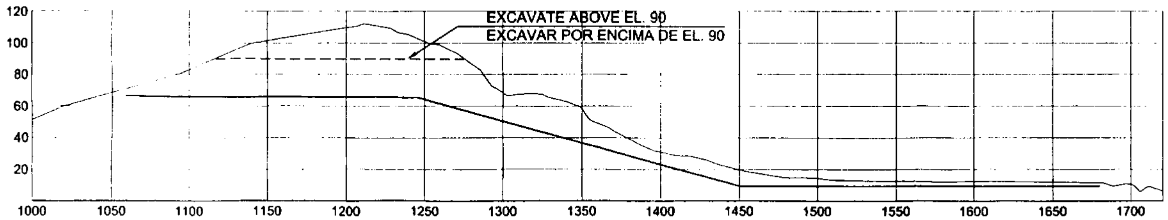
SECTION B
SECCIÓN B



SECTION C
SECCIÓN C

0 50 100 150 200 meters

GRAPHIC SCALE



PROFILE OF SPILLWAY ALIGNMENT
PERFIL DEL ALINEAMIENTO DEL ALIVIADERO

COORDINATES (UTM):

POINT	STATION	NORTH	EAST
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WP10	1235	994774.4	590491.1
WP11	1444	994857.8	590299.6
WP12	1700	994960.1	590064.9

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

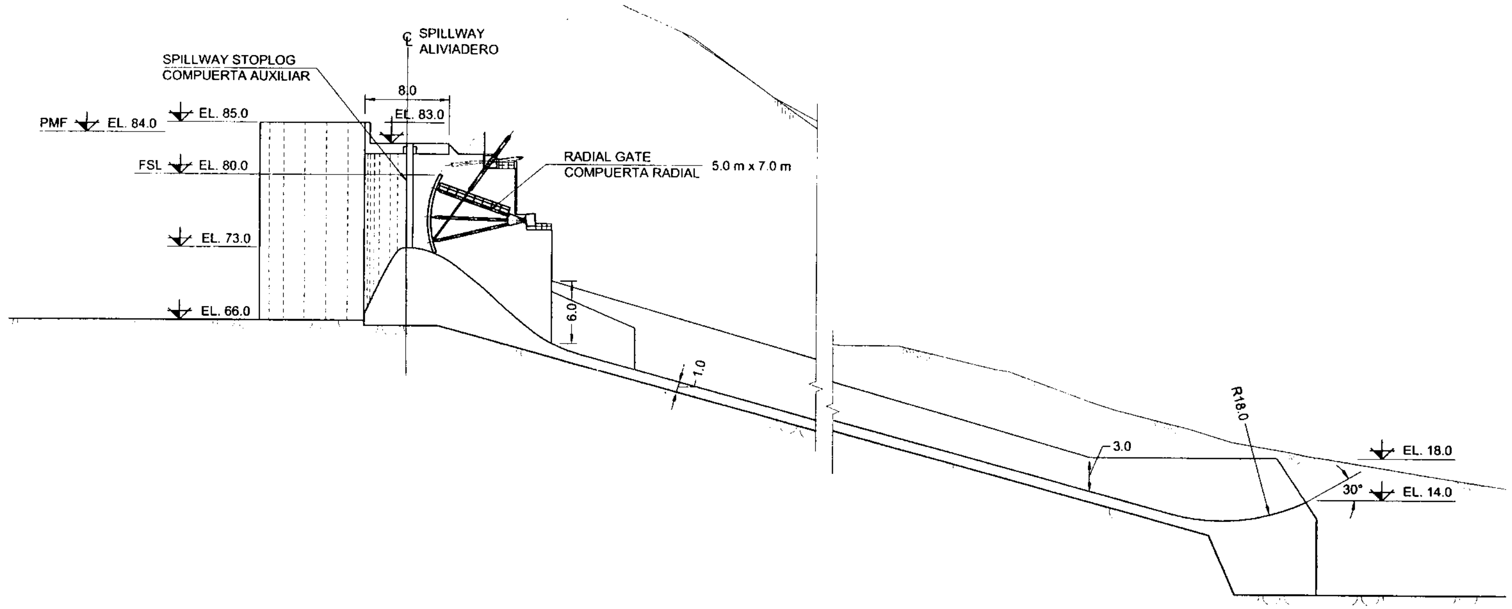
CONTRACT NO. CC-3-536
RIO INDIO WATER SUPPLY PROJECT

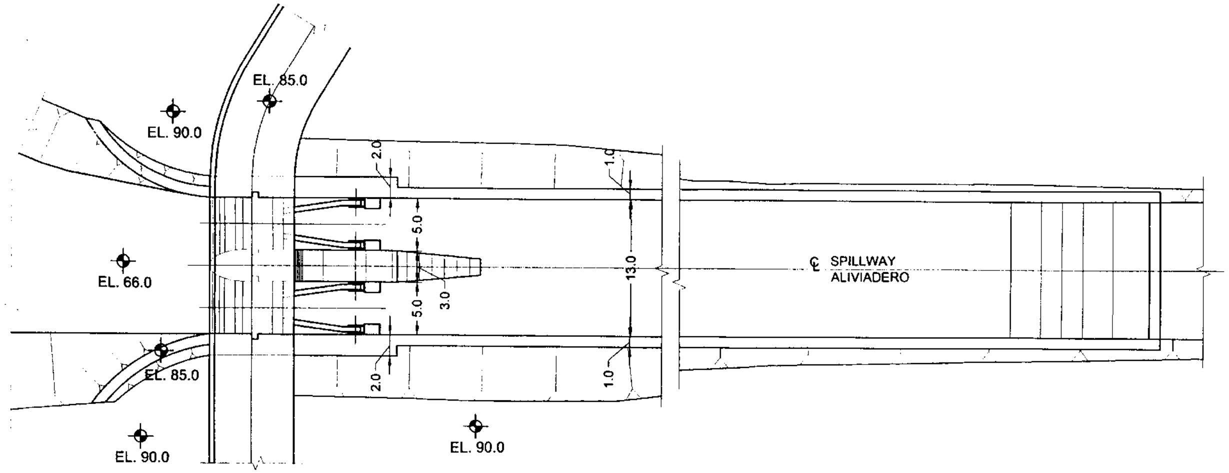
CFRD GATED SPILLWAY PLAN AND SECTIONS

MWH TAMS

DATE: APRIL, 2003

EXHIBIT: 7





0 5 10 15 20 25 meters

GRAPHIC SCALE

UNITS IN METERS

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

ACP

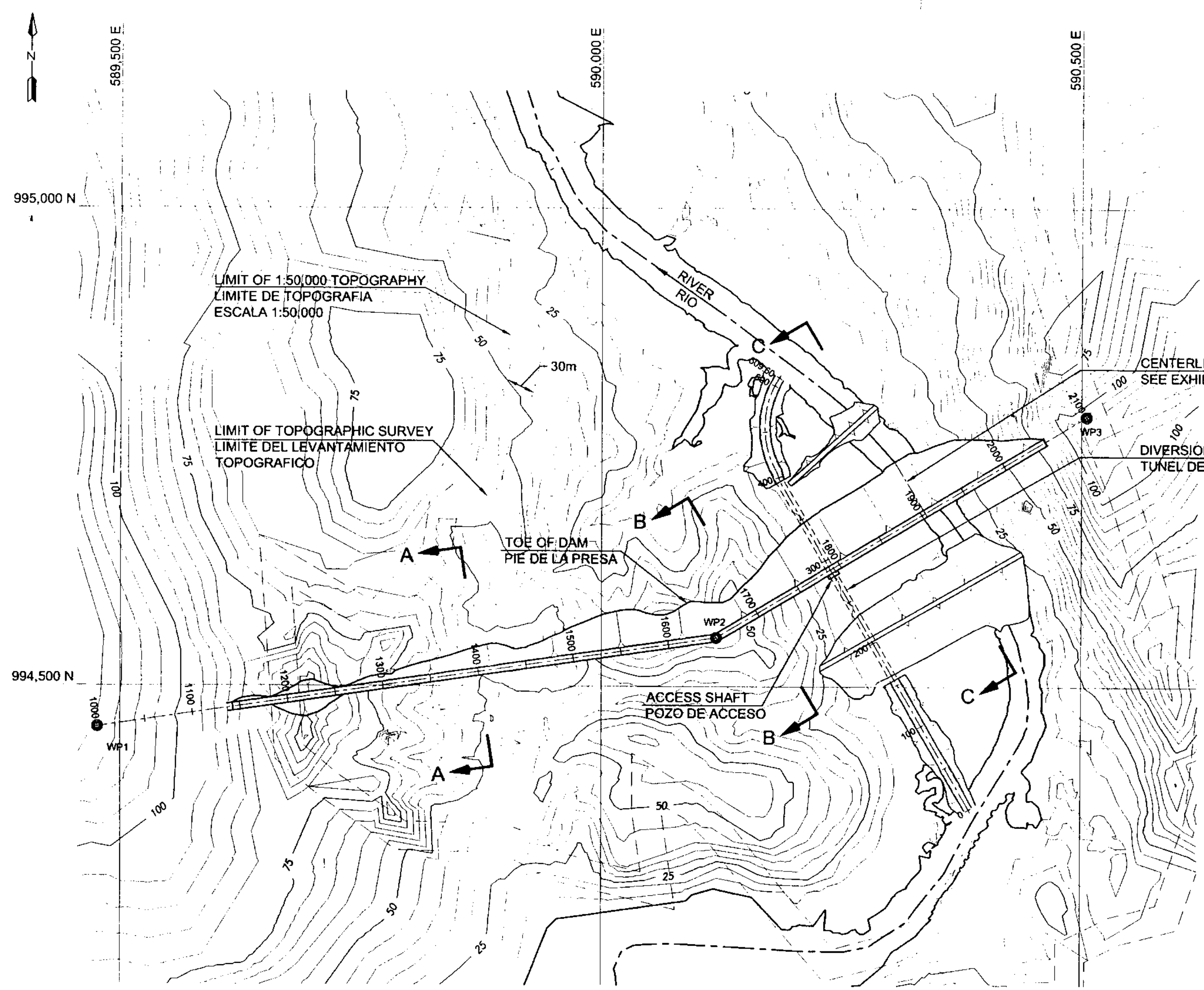
CONTRACT NO. CC-3-536
RIO INDIO WATER SUPPLY PROJECT

CFRD GATED SPILLWAY
PLAN, PROFILE AND SECTIONS

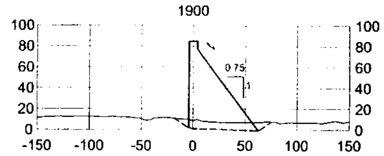
MWH TAMS

DATE:
APRIL, 2003

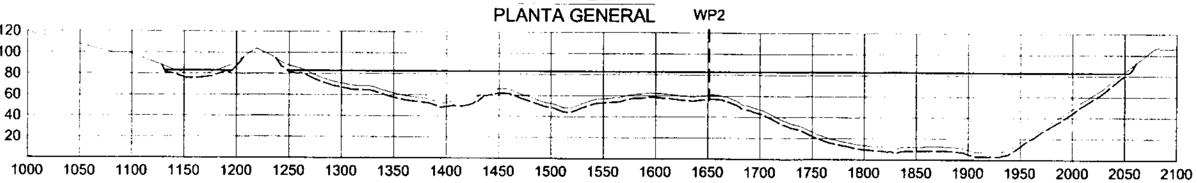
EXHIBIT:
8



SITE PLAN
PLANTA GENERAL



MAXIMUM RCC SECTION
STATION 1900
SECCIÓN MAXIMA



PROFILE OF DAM ALIGNMENT
PERFIL DEL ALINEAMIENTO DE PRESA

COORDINATES (UTM):

POINT	STATION	NORTH	EAST
WP1	1000	994456.9	589476.2
WP2	1650	994551.0	590119.4
WP3	2100	994782.8	590505.1

0 50 100 150 200 250 meters
GRAPHIC SCALE

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

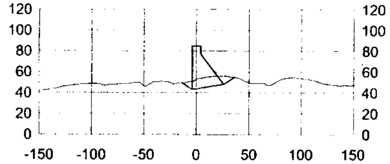
CONTRACT NO. CC-3-536
RIO INDIO WATER SUPPLY PROJECT

RCC DAM - PLAN

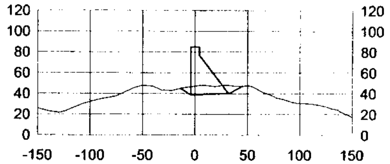
MWH TAMS

DATE: APRIL, 2003

EXHIBIT: 9



SECTION A
SECCIÓN A



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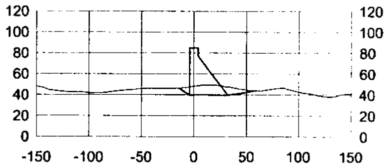
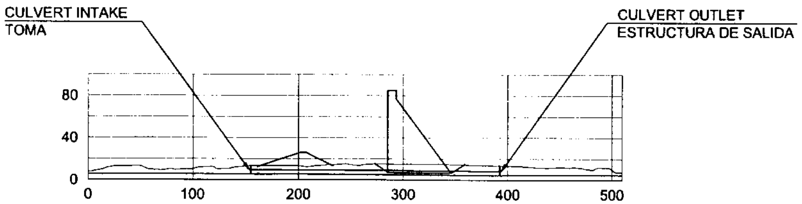
SECCIÓN B

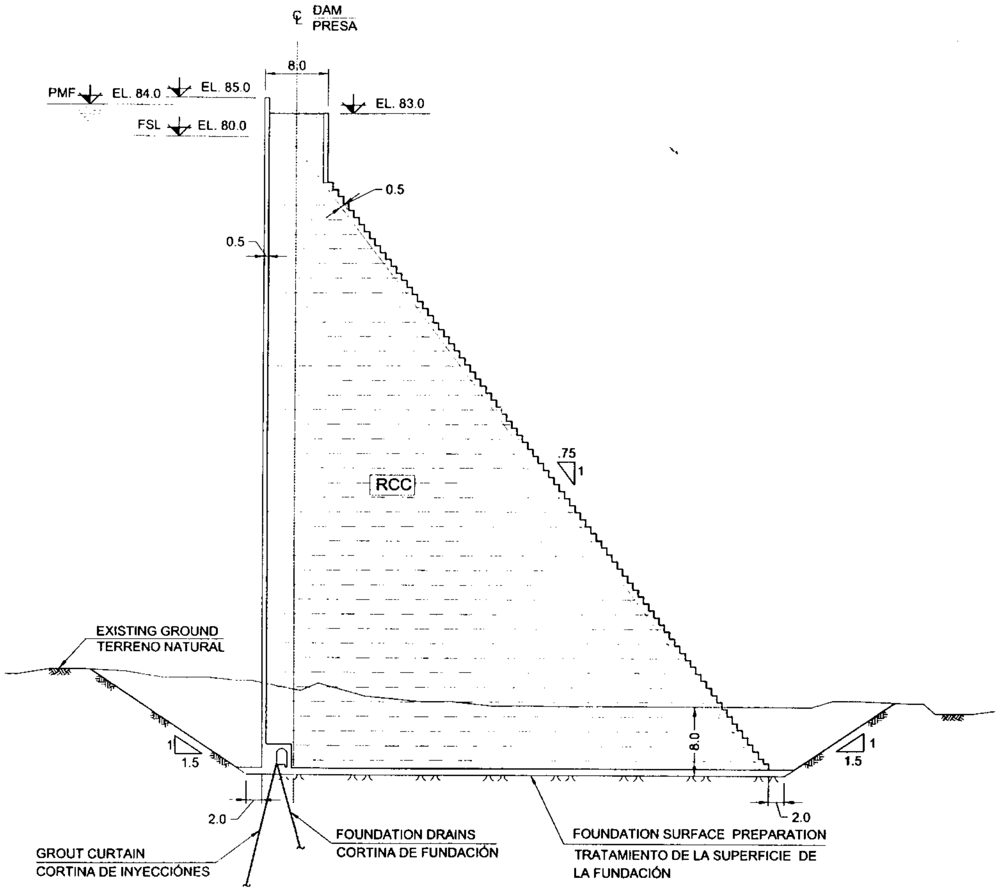


SECTION C
SECCIÓN C



PROFILE ALONG DIVERSION CULVERT
PERFIL

0 50 100 150 200 250 meters
GRAPHIC SCALE



RCC DAM - TYPICAL CROSS SECTION
PRESA CCR - SECCIÓN TIPICA

0 5 10 15 20 25 meters
GRAPHIC SCALE

UNITS IN METERS

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

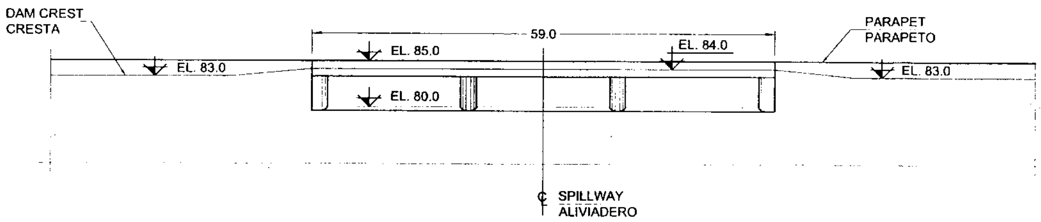
CONTRACT NO. CC-3-536
RIO INDIO WATER SUPPLY PROJECT

RCC DAM
SECTIONS

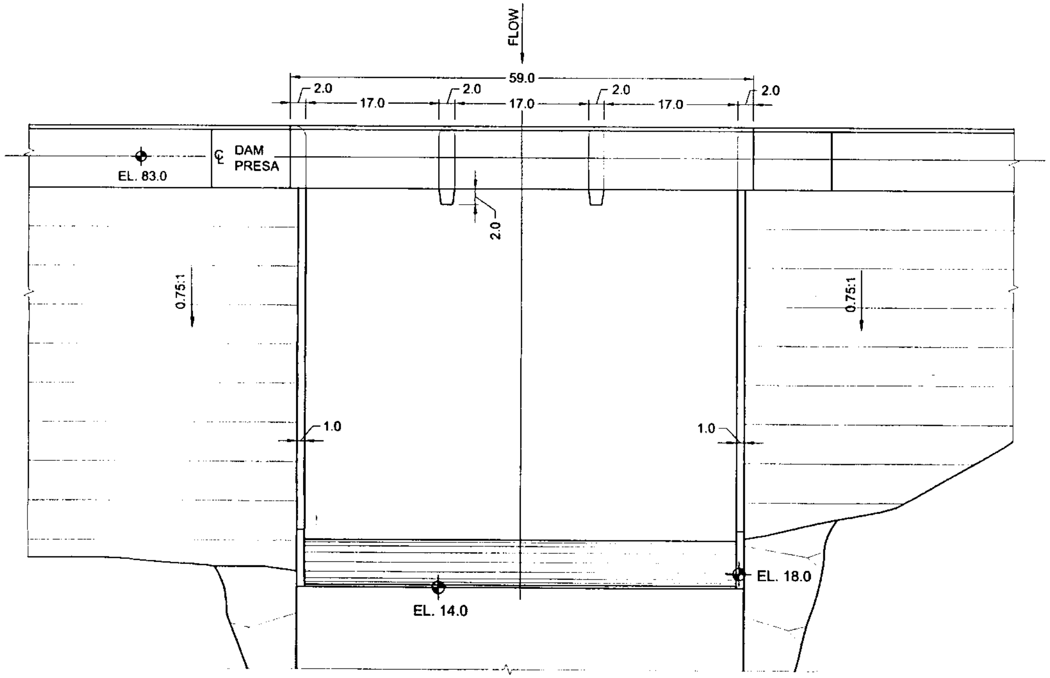
MWH TAMS

DATE:
APRIL, 2003

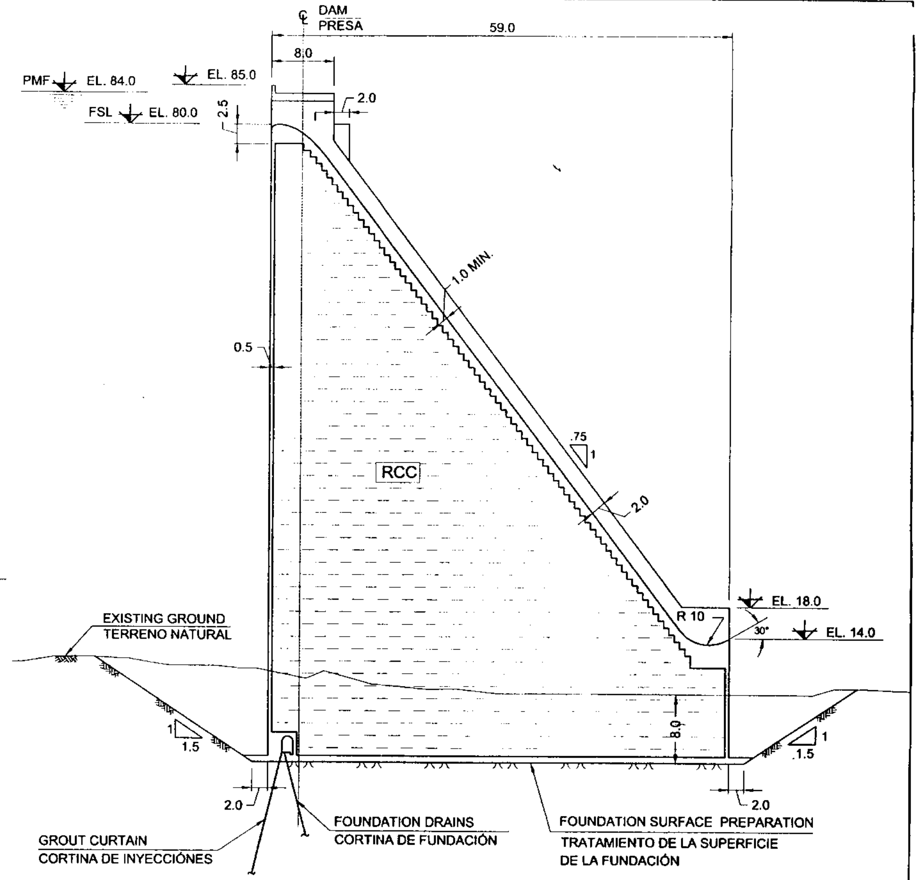
EXHIBIT:
10



UPSTREAM ELEVATION
ELEVACIÓN



PLAN
PLANTA



UNGATED SPILLWAY - TYPICAL CROSS SECTION
ALIVIADERO LIBRE - SECCIÓN TIPICA

0 5 10 15 20 25 meters
GRAPHIC SCALE

UNITS IN METERS

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

ACP

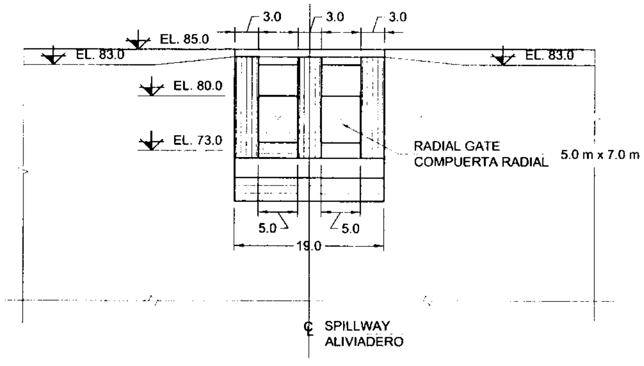
CONTRACT NO. CC-3-536
RIO INDIO WATER SUPPLY PROJECT

RCC UNGATED SPILLWAY
PLAN, SECTION AND ELEVATION

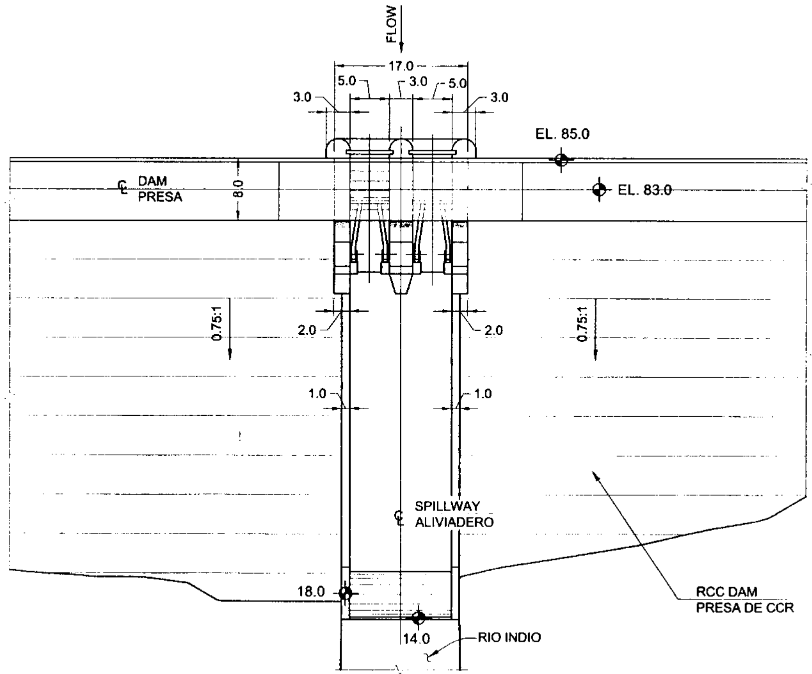
MWH TAMS

DATE: APRIL, 2003

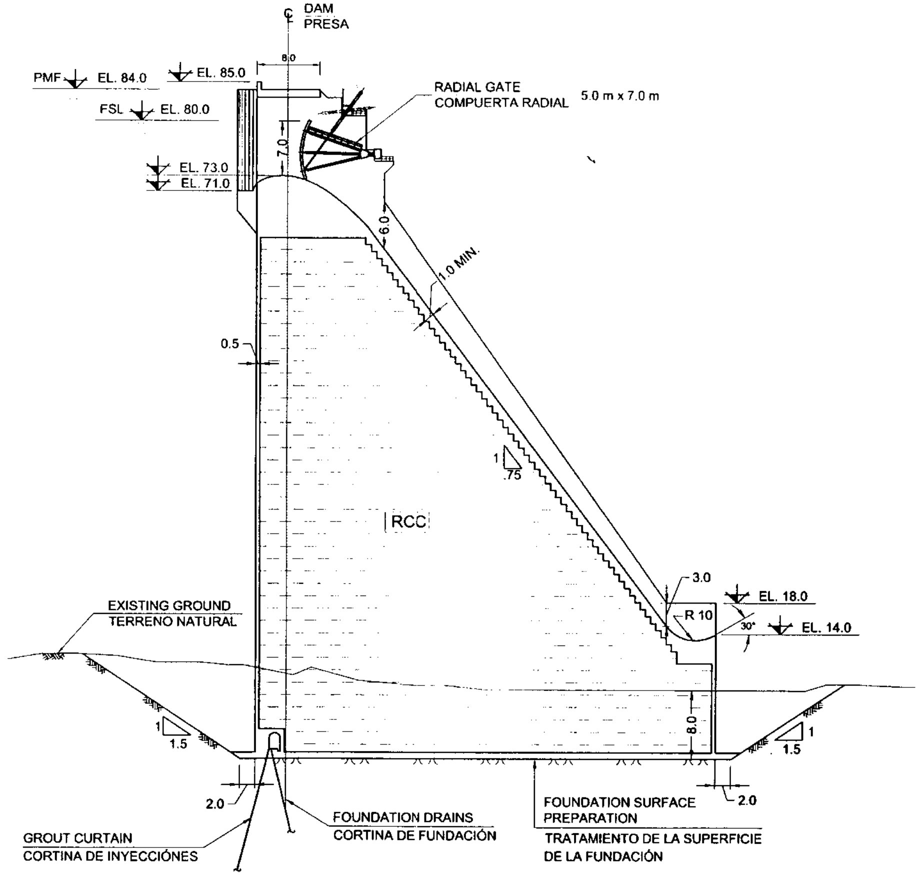
EXHIBIT: 11



UPSTREAM ELEVATION
ELEVACIÓN



PLAN
PLANTA



GATED SPILLWAY - TYPICAL CROSS SECTION
ALIVIADERO CON COMPUERTAS - SECCIÓN TIPICA

0 5 10 15 20 25 meters
GRAPHIC SCALE

UNITS IN METERS

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

CONTRACT NO. CC-3-536
RIO INDIO WATER SUPPLY PROJECT

RCC GATED SPILLWAY
PLAN, SECTION AND ELEVATION

MWH TAMS	DATE: APRIL, 2003	EXHIBIT: 12

ATTACHMENTS

Attachment 1 - Hydraulic Analysis for Design of Diversion Tunnel and Cofferdam

MWH
MONTGOMERY WATSON HARZA

MWH ENERGY & INFRASTRUCTURE, INC.
HYDROPOWER

Location Chicago Office **Date** February 4, 2002

To Mike Newbery

From Monica Cheng and Khalid Jawed

Subject Hydraulic Analyses for Design of Diversion Tunnel and Cofferdam Rio Indio Project

Introduction

This memo summarizes the procedures and results of the hydraulic analyses performed for design of cofferdam and diversion tunnel of Rio Indio Project.

The analysis included:

Tailwater analysis
Diversion Tunnel analysis
Reservoir routing

Basic data used in the analysis is discussed. Results are provided as tables and exhibits.

Tailwater Analysis

The objective of this analysis was to develop a tailwater rating curve for the proposed Rio Indio Dam site.

Computer Model

The water surface profiles computer model, HEC-2, developed by the U.S. Army Corps of Engineers, was used in this study to determine the tailwater rating curve. Key input data to the model include flows, river cross-sections and river reach characteristics.

River Cross Sections

Fifteen cross sections downstream from the Indio dam were used. The most upstream and downstream cross sections are located about 200 m and 16,500 m downstream of the dam, respectively. Initially, the two cross sections were derived from 1:2,000 and thirteen cross sections were derived from 1:50,000 scale topographic maps with 1-m and 20-m contour interval, respectively. Later six surveyed cross sections were received from ACP. With the help of coordinates of these cross sections, these were marked on the 1:50,000 scale map and distances of the cross sections from the dam site were estimated. The surveyed cross sections compared fairly with the cross sections taken from 1:2,000 scale map. However, the channel widths and the thalweg elevations of the cross sections obtained from 1:50,000 scale maps were adjusted with the help of the surveyed cross sections. The cross section data used are listed on Exhibit 1.

Manning's Roughness Coefficients

The roughness coefficients selected for the study reach are 0.035 for channel flow and 0.050 for over-bank flows.

Flows

A total of twelve flows varying in the range of 2 to 1,000 cms were used to develop tailwater rating curve.

Results

Table 1 summarizes the computed tailwater elevations at a distance of 200 meters from the dam site for the selected flows. Exhibit 2 shows the plot of tailwater rating curve.

Table 1
Tailwater Rating Curve

Flow	cms	2	5	10	25	50	100	200	300	500	700	780	1000
Tailwater Elevation	m	5.4	5.6	5.8	6.2	6.7	7.6	8.9	9.9	11.4	12.3	12.75	13.5

Diversion Tunnel Analysis

The objective of this analysis was to develop a relationship between headwater and diversion tunnel outflow for routing floods of selected return periods through cofferdam – diversion tunnel scheme. The routing results would be used for determining height of cofferdam.

Methodology

Tunnel flow generally is non-uniform with regions of gradually and rapidly varying flows. An exact theoretical analysis of tunnel flow is very complex which could involve backwater and drawdown calculations, energy and momentum balance, and applications of hydraulic model studies. This exact analysis was not performed in the current study. Instead, a simplified analysis was made and its results were used in the reservoir routing for preliminary design of height of cofferdam.

Procedures

A Microsoft "EXCEL" spreadsheet was used for the computation. The following components were computed in the spreadsheet.

- Calculate the velocity by dividing the flow rate per tunnel by the cross sectional area of tunnel. Determine the corresponding velocity head.

- Determine the entrance, exit, bend, tunnel friction and any other losses. The entrance, exit and bend loss coefficient was assumed to be 0.3, 1.0 and 0.2, respectively. The friction loss was computed using Darcy-Weisbach equation. The Darcy-Weisbach friction loss coefficient was computed based on the Manning's coefficient of 0.013 for lined-concrete tunnel.

- Calculate the required headwater elevation by adding the invert elevation at outlet with depth of tunnel and the total loss.

- Select other flow rates and repeat the above computations to develop the relationship of the required headwater elevation and the selected flow rates.

The computation was made for various tunnel sizes and lengths. The tunnel sizes considered are one D-shape 700 m long tunnel with diameter of 5 m, one D-shape 500 m long tunnel with diameter of 4.5 m, 5 m and 6 m and two 2 m x 3 m and one 3.5 m x 5 m square box tunnels of length 200 m. Exhibit 3 shows the headwater elevation versus discharge curves. The computed headwater and the corresponding flow values were applied to the reservoir routing.

Routing Analysis

The objective of this analysis was to provide information of maximum headwater elevations under various tunnel sizes and length.

Methodology

An in-house computer program ROUTE was used for the reservoir routing. The 25-year and 50-year floods were routed for various tunnel size and length conditions. The key input to the program included the headwater vs. outflow relationship developed in the tunnel analysis, reservoir capacity curve, and the flood hydrograph. A brief description of these data is given below.

Reservoir Capacity

The reservoir volume data listed in Table 2 were used for the storage routing. These reservoir volume data were provided by the ACP. The elevation-volume data provided by the ACP was developed by fitting a polynomial curve to four elevation-volume points at 0, 40, 60 and 80 meters. A check was made on the elevation-volume curve by digitizing the reservoir areas at 20, 40, 60 and 80-meter contour intervals. The volumes estimated at elevations 40, 60 and 80 meters were nearly the same as computed by ACP.

However, the curve appears to overestimate the volume at lower elevations. Using the 20 m contour area and setting the storage volume to 0 at El. 8.0 m, the volume of El. 20.0 was estimated to be 25% less. A further reduction of 25% in the reservoir capacity is also presented to reflect the uncertainty involved.

Table 2
Reservoir Capacity Curve

Elevation (m)		8	10	15	20	25	30
Volume (mcm)	No reduction	0	7	21.37	33.62	62.01	106.53
	Single 25% reduction	0	5.25	16.03	25.22	46.51	79.90
	Double 25% reduction	0	3.94	12.02	18.91	34.88	59.92

Flood Hydrograph

The 25-year and 50-year flood hydrographs computed for the Indio dam as shown in Tables 3 and 4 were used as the inflow hydrograph for the reservoir routing.

Starting Reservoir Elevation

Reservoir routing was made using a starting reservoir elevation at initial flow of the flood. The starting elevation was estimated based on the relationship of headwater vs. outflow computed in the diversion tunnel analysis.

Results

Table 5 summarizes the maximum water surface elevation reached for various tunnel size and length conditions routed. These results will be used for the design of height of cofferdam.

Table 3
25-Year Flood Hydrograph

Time	**hr**	0	1	2	3	4	5	6	7
Flow	**cms**	26	49	52	112	125	234	252	334
Time	**hr**	8	9	10	11	12	13	14	15
Flow	**cms**	372	424	477	568	689	720	780	750
Time	**hr**	16	17	18	19	20	21	22	23
Flow	**cms**	735	674	644	614	556	486	472	432
Time	**hr**	24	25	26	27	28	29	30	31
Flow	**cms**	392	342	305	269	235	202	171	136
Time	**hr**	32	33	34	35	36	37	38	39
Flow	**cms**	103	90	79	70	61	58	52	51
Time	**hr**	40	41	42	43	44	45	46	47
Flow	**cms**	50	49	48	47	46	45	44	43
Time	**hr**	48	53						
Flow	**cms**	42	37						

Table 4
50-Year Flood Hydrograph

Time	**hr**	0	1	2	3	4	5	6	7
Flow	**cms**	27	30	33	96	185	301	426	404
Time	**hr**	8	9	10	11	12	13	14	15
Flow	**cms**	391	445	502	651	778	810	820	794
Time	**hr**	16	17	18	19	20	21	22	23
Flow	**cms**	778	713	682	651	590	516	502	459
Time	**hr**	24	25	26	27	28	29	30	31
Flow	**cms**	418	364	326	288	252	218	185	148
Time	**hr**	32	33	34	35	36	37	38	39
Flow	**cms**	114	100	89	78	70	66	61	59
Time	**hr**	40	41	42	43	44	45	46	47
Flow	**cms**	57	57	56	55	53	52	51	51
Time	**hr**	48							
Flow	**cms**	50							

Table 5
Maximum Headwater Surface Elevation

Size Of Tunnel (m)	Length Of Tunnel (m)	No. Of Tunnel	Shape of Tunnel	Reservoir Volume Change	Maximum Headwater Elevation (m)	
					25-yr flood	50-yr flood
6	500	1	D-shape	No reduction	20.6	21.1
				Single 25% reduction	21.8	22.5
				Double 25% reduction	23.3	24.2
5	500	1	D-shape	No reduction	21.0	21.6
				Single 25% reduction	22.6	23.4
				Double 25% reduction	24.5	25.4
5	700	1	D-shape	No reduction	-	-
				Single 25% reduction	-	-
				Double 25% reduction	-	25.6
4.5	500	1	D-shape	No reduction	21.3	21.9
				Single 25% reduction	23.0	23.9
				Double 25% reduction	25.2	25.9
3.5x5	200	1	Box	No reduction	21.2	21.8
				Single 25% reduction	22.8	23.7
				Double 25% reduction	24.9	25.3
2x3	200	2	Box	No reduction	21.0	21.7
				Single 25% reduction	23.0	23.9
				Double 25% reduction	25.4	26.1

Exhibit 1
(Sheet 1 of 2)

River Cross Sections Downstream from Rio Indio Dam

Cross sections from 1:2000 scale map

200m d/s from dam		500m d/s from dam	
Dist. (m)	Elev. (m)	Dist. (m)	Elev. (m)
0	40	0	40
25	35	29	30
37	30	39	25
53	25	57	20
60	20	73	15
90	15	78	14
98	14	86	13
110	13	118	12
126	12	122	11
161	10	150	10
163	9	156	4.75
165	8	179	4.75
168	5.1	186	10
188	5.1	215	11
196	8	218	12
199	9	249	13
201	10	257	14
236	12	263	15
252	13	278	20
264	14	297	25
272	15	307	30
302	20	336	40
309	25		
324	30		
337	35		
361	40		

Cross sections from 1:50,000 scale maps

900m d/s from dam		1600m d/s from dam		1800m d/s from dam	
Dist. (m)	Elev. (m)	Dist. (m)	Elev. (m)	Dist. (m)	Elev. (m)
0	40	0	40	0	40
63	20	62	20	60	20
200	10	100	10	100	10
250	8	250	8	150	8
275	4.2	350	3.33	180	3.15
300	4.2	400	3.33	220	3.15
325	8	500	8	250	8
375	10	650	10	300	10
513	20	687	20	340	20
575	40	750	40	400	40

Exhibit 1
(Sheet 2 of 2)

River Cross Sections Downstream from Rio Indio Dam

Cross sections from 1:50,000 scale maps

2600m d/s from dam		3600m d/s from dam		4150m d/s from dam		5100m d/s from dam		6100m d/s from dam	
Dist. (m)	Elev. (m)	Dist. (m)	Elev. (m)	Dist. (m)	Elev. (m)	Dist. (m)	Elev. (m)	Dist. (m)	Elev. (m)
0	40	0	40	0	40	0	40	0	40
50	20	180	20	75	20	50	20	112	20
138	10	242	10	138	10	138	10	202	10
188	8	280	8	188	8	168	8	220	8
213	2.9	330	2.76	263	2.72	188	2.63	240	5
238	2.9	360	2.76	313	2.72	208	2.63	250	2.4
263	8	430	8	388	8	238	8	270	2.4
313	10	467	10	438	10	263	10	280	5
400	20	530	20	500	20	350	20	300	8
450	40	700	40	575	40	400	40	322	10
								412	20
								525	40

7400m d/s from dam		9200m d/s from dam		11550m d/s from dam		15100m d/s from dam		16500m d/s from dam	
Dist. (m)	Elev. (m)	Dist. (m)	Elev. (m)	Dist. (m)	Elev. (m)	Dist. (m)	Elev. (m)	Dist. (m)	Elev. (m)
0	40	0	40	0	40	0	40	0	40
85	20	115	20	350	20	60	20	85	20
135	10	150	10	950	10	85	10	185	10
175	8	200	8	1050	8	130	8	250	8
210	5	300	5	1150	5	180	5	300	5
240	1.2	470	0.9	1300	0.8	310	3	350	3
270	1.2	500	0.9	1330	0.8	345	0.6	380	0.5
310	5	600	5	1450	5	365	0.6	400	0.5
345	8	700	8	1550	8	410	3	450	3
385	10	750	10	1650	10	530	5	500	5
435	20	790	20	2250	20	580	8	550	8
525	40	900	40	2550	40	635	10	610	10
						660	20	710	20
						725	40	800	40

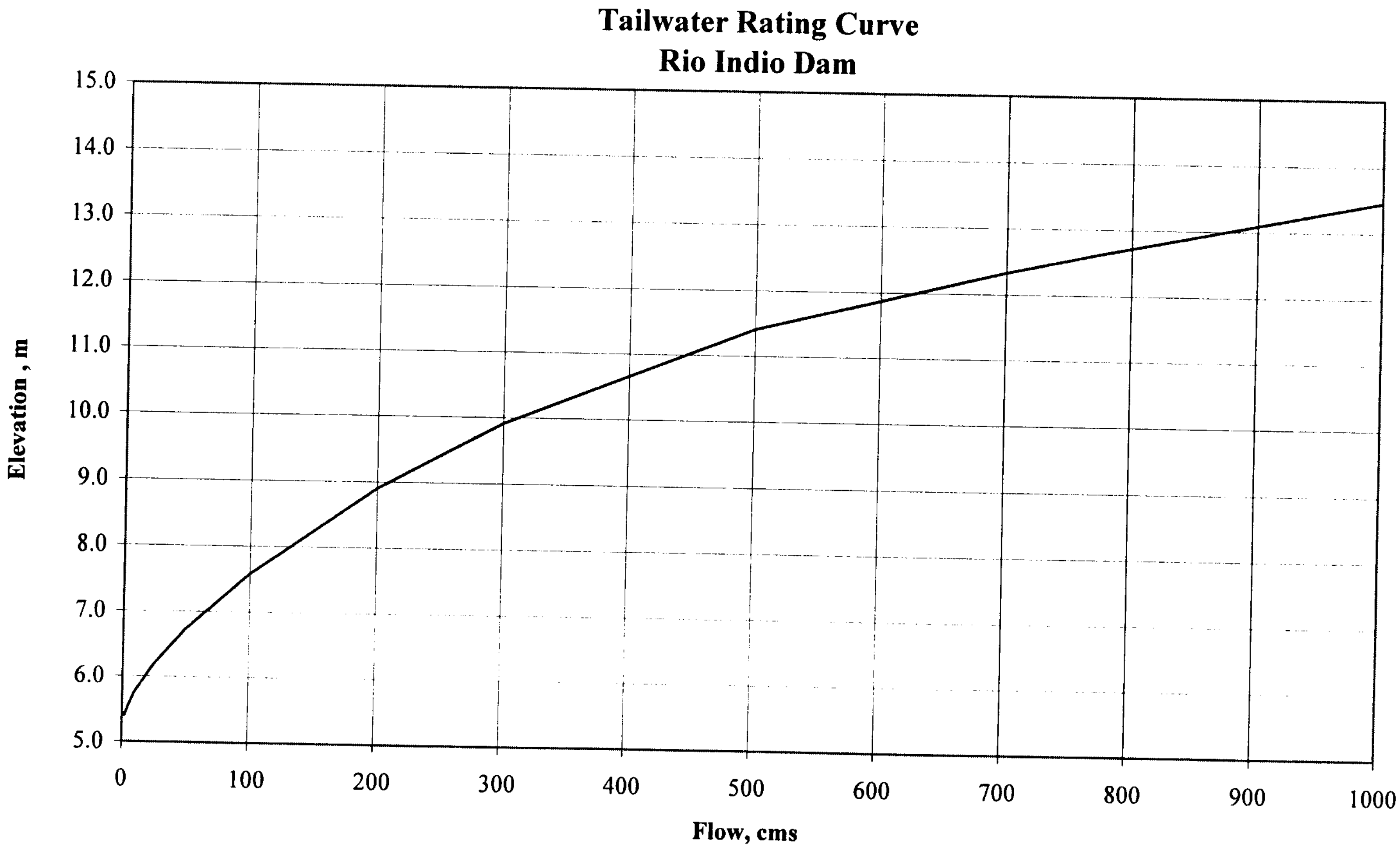

Tailwater Rating Curve
Rio Indio Dam
Elevation , m
15.0
14.0
13.0
12.0
11.0
10.0
9.0
8.0
7.0
6.0
5.0
0
100
200
300
400
500
600
700
800
900
1000
Flow, cms

Exhibit 2

Headwater Rating Curve
Rio Indio dam

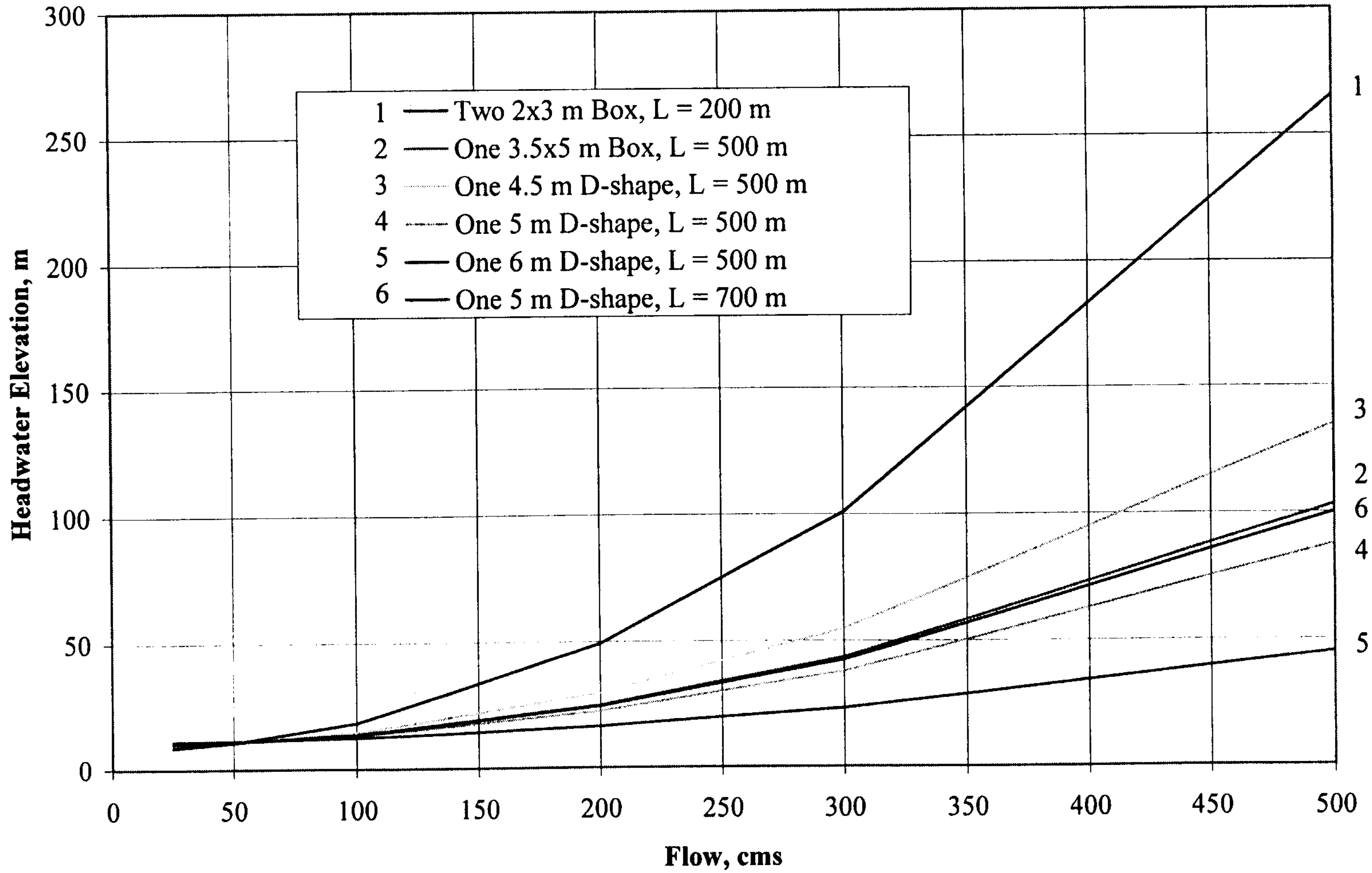


Exhibit 3

Attachment 2 - Evaluation of Spillway Sizes and Estimation of Freeboard

MWH
MONTGOMERY WATSON HARZA

MWH ENERGY & INFRASTRUCTURE, INC.

Intra-Company Correspondence

PRIVATIZATION & PROJECT DEVELOPMENT

Location: Chicago Office

February 6, 2002

To: Michael Newbery

From: Khalid Jawed

Subject: Evaluation of Spillway Sizes and Estimation of Freeboard for Rio Indio Project

Introduction

This memorandum summarizes the procedures and results of hydrologic analyses performed to determine flood surcharge over the normal reservoir pool elevation during the routing of probable maximum flood (PMF), and to estimate magnitudes of wave run-up and wind setup. The analyses included:

- Estimation of flood surcharges for an ungated Ogee spillway with crest elevation at 80 meters. Four lengths of spillway, 50, 75, 100 and 120 meters were considered.
- Estimation of flood surcharge for a gated Ogee spillway with crest at elevation 73 meters, and two bays each with a 5-meter wide and 7-meter high gate.
- Analysis of wave run-up and wind setup.

Basic data used in the analyses are discussed. The results are provided as tables.

Ungated Spillway

The objective of this analysis was to determine the maximum surcharge over the normal reservoir pool elevation of 80 meters, during the routing of the PMF. The HEC-1 computer program developed by the United States Army, Corps of Engineers, was used to route the PMF with a starting elevation at 80 meters. The elevation-volume data and spillway rating curves used are given in Table 1. The spillway discharges were computed using the following relationship.

$$Q = C\,L\,(H^{1.5})$$

where Q is spillway discharge in cubic feet per second, L is length of spillway in feet, H is head over spillway crest in feet and C is coefficient. A constant value of 3.5 was used. The spillway lengths of 120 (originally used by ACP), 100, 75 and 50 meters were used. The results are summarized in Table 2.

Table 1

**ELEVATION-VOLUME AND SPILLWAY DISCHARGE DATA
RIO INDIO RESERVOIR**

Elevation (m)	Volume (mcm)	Elevation (m)	Discharge (cms) L=120m	Discharge (cms) L=100m	Discharge (cms) L=75m	Discharge (cms) L=50m
75	1232.4	80	0	0	0	0
80	1436.6	80.5	82	68	51	34
81	1481.2	81.0	232	193	145	97
82	1525.0	82.0	656	546	410	273
83	1569.6	83.0	1205	1003	752	502
84	1614.7	84.0	1855	1546	1159	773
85	1660.0	85.0	2593	2159	1619	1080
86	1707.6	86.0	3408	2835	2126	1417
87	1755.2	87.0	4295	3576	2682	1788
88	1802.8	88.0	5247	4373	3279	2186
89	1850.4	89.0	6262	5209	3907	2605
90	1898.0	90.0	7334	6107	4580	3054

Table 2

**MAXIMUM SURCHARGE DURING PMF
RIO INDIO RESERVOIR**

PMF Peak Inflow	Initial Pool Elevation	Width of Spillway	Maximum Outflow	Maximum Surcharge Elevation
(cms)	(meters)	(meters)	(cms)	(meters)
4345	80	120	1390	83.3
4345	80	100	1251	83.5
4345	80	75	1031	83.7
4345	80	50	769	84.0

Gated Spillway

A gated spillway, with two bays, was also considered. Each gate would be 5 meters wide and 7 meter high. The spillway discharge curve was developed using the following relationships given in Design of Small Dams, Third Edition, 1987.

$$Q = C * L * (H^{1.5})$$

$$L' = L - 2 (N\ Kp + Ka)\ H$$

in which Q, L and H are defined above. N is number of piers equal to one, Kp is pier coefficient assumed 0.02, and Ka is abutment contraction coefficient assumed to be 0.10. The C values varied with head and were computed using the Figures 9-23 and 9-24 of Design of Small Dams. The spillway discharge curve is given in Table 3.

Table 3

SPILLWAY DISCHARGE CURVE
Spillway Crest at 73.0 M

Elevation (m)	Discharge (cms)	Elevation (m)	Discharge (cms)
73.5	5.9	80.5	364.8
74.0	17.2	81.0	402.3
74.5	32.1	81.5	438.8
75.0	50.5	82.0	476.0
75.5	70.6	82.5	511.4
76.0	93.8	83.0	547.0
76.5	118.0	83.5	585.5
77.0	144.0	84.0	621.4
77.5	173.3	84.5	660.4
78.0	202.5	85.0	696.5
78.5	233.1	85.5	735.8
79.0	264.9	86.0	771.6
79.5	299.3	86.5	810.9
80.0	331.7	87.0	846.3

The PMF was routed through the reservoir using the elevation-volume curve given in Table 1 and spillway discharge data given in Table 3. The maximum outflow was about 597 m^3/s corresponding to a maximum reservoir elevation of 83.66 meters.

Analysis of Wind Wave

Definitions

The purpose of this analysis was to determine the freeboard for Indio dam. The freeboard is defined as the difference in elevation between the crest of a dam and normal pool elevation of the reservoir. A term minimum freeboard is also used which is defined as the difference in elevation between the crest of the dam and the maximum water surface elevation reached during the routing of the PMF (or design flood if different from the PMF) with the spillway and other outlets, if any, functioning as planned.

Freeboard computations generally include the determination of wind setup and wave run-up on sloping or vertical embankments. A number of empirical relationships are available. Most of these involve use of wind velocity and fetch length as basic parameters. While the various relationships yield different results, the variation between the results is not so great compared to the variation that could be possible in the results due to assumptions of wind velocity and fetch length. The freeboard estimate is significantly affected by the magnitude of wind velocity and direction, and fetch length.

Basic Data

Site-specific data for wind velocity and direction were not available. Also, a reasonable configuration of the reservoir area at normal pool level could not be obtained. The reservoir shape was approximated from 1:50,000-scale map with 20-meter contour interval. In the absence of dominant wind direction data, the fetch length was assumed to be the maximum length of the reservoir for conservative estimates of wave run-up and wind setup. ACP provided monthly average wind speed and wind gust data for Gatun station. Wind speed was assumed after the review of this data.

Methodology

The procedures given in the United States Corps of Engineers publication ETL 1110-2-221 dated November 29, 1976 entitled, "Wave Run-up and Wind Setup on Reservoir Embankment by Bruce L. McCartney, Department of Army, Office of the Chief of Engineers, Washington D.C.," were used. The steps necessary for the computations of wind setup and wave run-up included:

- Estimate maximum wind speed
- Plot a wind velocity-duration curve for the site
- Compute reservoir effective fetch length
- Plot a wind velocity-duration curve for the reservoir effective fetch
- Determine magnitude of design wind
- Estimate design wind duration
- Estimate wave run-up
- Estimate wind setup

The step-by-step computations are shown below:

- Embankment slope 1:1.5, impervious, smooth
- Reservoir normal pool elevation = 80 meters
- Depth at toe of embankment = 70 meters (230 feet)
- Maximum fetch length = 7.2 km (1.8 miles)

- Effective fetch length = 0.25 * 7.2 = 1.8 km (0.45 miles), there is a procedure as per manual ETL 1110-2-211, to compute effective fetch; because reservoir plan is not available, this procedure could not be applied; an alternative is to assume effective fetch as 25 percent of maximum fetch length.

- Ratios between wind velocity over water and on land (manual ETL 1110-2-211)

Effective Fetch (mi)	Ratio
0.5	1.08
1.0	1.13
2.0	1.21
3.0	1.26
4.0	1.28
5.0 & above	1.30

- Assumed wind velocity and duration over land and computed velocity over water

Duration (min)	Wind Velocity over Land (mph)	Wind Velocity over Water (mph)
1	65	70
60	35	38
120	30	32

- Wind velocity for an effective fetch of 0.45 miles from Figure 11 of ETL 1110-2-221.

Duration (min)	Wind Velocity over Water (mph)
8	75
10	47
15	18

- Plotted the above two sets of wind data, the intersection of the two curves provided a design velocity of 44 mph and a duration of 53 minutes.
- From Figure 11 of ETL 1110-2-221, the significant wave height (Hs) was about 1.8 feet and wave period (Ts) was about 2.5 seconds.
- The following wave run-up relationship (ETL 1110-2-221) was used:

Rs/Hs = 1/(0.4 + ((Hs/Lo)^0.5) * Cot A))

Rs = wave run-up, feet
Hs = significant wave height, feet
Cot A = 1.5 (slope)
Lo = wave length in feet
= 5.12 (Ts ^ 2), Ts is wave period in seconds

- Based on the above relationship, Rs = 2.4 feet
- As per recommendation in ETL 1110-2-221, Rmax = 2.4 * 1.5 = 3.6 feet
- For wind setup, the following relationship given in ETL 1110-2-221 was used.

  S = ((U ^ 2) * F)/ (1400 D)

  S = setup in feet
  U = average wind velocity in feet, mph, a velocity of 68 mph was used.
  D = average depth along the fetch, a depth of 130 feet was used.
  F = fetch distance, twice the effective fetch, a value of 3.6 miles was used.
- Based on the above data, the wind setup was about 0.1 feet.

Summary

The above computations give the following data:

Design wind velocity = 68 mph
Wind duration = 9 minutes
Wave run-up = 3.6 feet (1.1 m)
Wind setup = 0.1 feet (0.1 m)
Allowance for wave action over normal pool = 1.1 + 0.1 = 1.2 meters

The above allowance may appear to be reasonable. However, USBR publication "Design of Small Dams," provide recommendation for selection of normal and minimum freeboard (Table 6.4, page 258, third edition, 1987). For an effective fetch length of 1.8 miles, the recommended freeboards are about 5.5 and 4.5 feet (about 1.7 and 1.4 meters), respectively. Based on this, the computed value of 1.2 meters is in line with the USBR's recommendations.

Attachment 3 – Comparative Cost Estimate

Panama Canal Authority
Contract CC-3-536
Task Order 3, Indio Water Supply Feasibility Study
Dam Type Alternative Study

CFRD **Ungated** **$ 60,744,732**

Quantity Take-Offs

Item	Description	Unit	Quantity		Unit Cost		Cost
DIVERSION							
1	Site Preparation	m^2	250,000	$	0.50	$	125,000
2	Diversion						
2.1	Overburden	m^3	41,000	$	3.20	$	131,200
2.2	Rock	m^3	9,900	$	8.75	$	86,625
2.3	Tunnel Ex.	m^3	19,401	$	80.00	$	1,552,045
2.4	Cofferdams	m^2	150,800	$	10.30	$	1,553,240
2.5	Concrete	m^3	3,935	$	115.00	$	452,474
2.6	Formwork	m^2	12,373	$	46.20	$	571,624
2.7	Reinforcement	kg	308,469	$	1.36	$	419,518
	Subtotal					$	4,891,725
DAM							
1	Excavation						
1.1	Overburden	m^3	536,667	$	3.20	$	1,717,333
1.2	Rock*	m^3	312,533	$	8.75	$	2,734,667
1.3	Abut. Overburden	m^3	193,800	$	3.20	$	620,160
1.4	Abut. Rock.	m^3	506,500	$	8.75	$	4,431,875
2	Grouting						
2.1	Consolidation	L.S.	1	$	1,000,000.00	$	1,000,000
3	Rockfill						
3.1	Mass*	m^3	2,691,325	$	10.30	$	27,720,648
3.2	Filter	m^3	216,000	$	16.20	$	3,499,200
3.3	Drain	m^3	26,875	$	16.20	$	435,375
4	Concrete						
4.1	Plinth	m^3	7,600	$	115.00	$	874,000
4.2	Facing	m^2	72,000	$	80.00	$	5,760,000
4.3	Parapet -US	m^3	3,870	$	250.00	$	967,500
4.4	Parapet -DS	m^3	731	$	250.00	$	182,750
4.5	Crest	m^3	4,386	$	115.00	$	504,390
Subtotal						$	50,447,898
SPILLWAY							
1	Site Preparation	m^2	50,000	$	0.50	$	25,000
2	Excavation						
2.1	Overburden	m^3	46,500	$	3.20	$	148,800
2.2	Rock*	m^3	214,200	$	8.75	$	1,874,250
3	Headworks						
3.1	Concrete	m^3	3,475	$	115.00	$	399,625
3.2	Formwork	m^2	2,125	$	46.20	$	98,175
3.3	Reinforcement	kg	169,540	$	1.36	$	230,574
4	Chute and Flip Bucket						
4.1	Concrete	m^3	10,388	$	115.00	$	1,194,620
4.2	Formwork	m^2	2,860	$	46.20	$	132,132
4.3	Reinforcement	kg	394,705	$	1.36	$	536,799
5	Bridge						
5.1	Concrete	m^3	240	$	115.00	$	27,600
5.2	Formwork	m^2	300	$	46.20	$	13,860
5.3	Reinforcement	kg	37,632	$	1.36	$	51,180
6	Tailrace Channel						
6.1	Overburden	m^3	19,100	$	3.20	$	61,120
6.2	Rock*	m^3	17,000	$	8.75	$	148,750
Subtotal						$	4,942,484
LOW LEVEL OUTLET STRUCTURE							
1	Shaft						
1.1	Excavation, Rock	m^3	1,698	$	150.00	$	254,764
1.2	Concrete	m^3	682	$	115.00	$	78,467
1.3	Formwork	m^2	1,241	$	46.20	$	57,314
1.4	Reinforcement	kg	53,000	$	1.36	$	72,080
Subtotal						$	462,624

Panama Canal Authority
Contract CC-3-536
Task Order 3, Indio Water Supply Feasibility Study
Dam Type Alternative Study

CFRD Gated **$ 60,596,298**

Quantity Take-Offs

Item	Description	Unit	Quantity		Unit Cost		Cost
DIVERSION							
1	Site Preparation	m^2	250,000	$	0.50	$	125,000
2	Diversion						
2.1	Overburden	m^3	41,000	$	3.20	$	131,200
2.2	Rock	m^3	9,900	$	8.75	$	86,625
2.3	Tunnel Ex.	m^3	19,401	$	80.00	$	1,552,045
2.4	Cofferdams	m^3	150,800	$	10.30	$	1,553,240
2.5	Concrete	m^3	3,935	$	115.00	$	452,474
2.6	Formwork	m^2	12,373	$	46.20	$	571,624
2.7	Reinforcement	kg	308,469	$	1.36	$	419,518
Subtotal						$	4,891,725
DAM							
1	Excavation						
1.1	Overburden	m^3	536,667	$	3.20	$	1,717,333
1.2	Rock	m^3	312,533	$	8.75	$	2,734,667
1.3	Abut. Overburden	m^3	193,800	$	3.20	$	620,160
1.4	Abut. Rock.	m^3	506,500	$	8.75	$	4,431,875
2	Grouting						
2.1	Consolidation	L.S.	1	$	1,000,000.00	$	1,000,000
3	Rockfill						
3.1	Mass	m^3	2,691,325	$	10.30	$	27,720,648
3.2	Filter	m^3	216,000	$	16.20	$	3,499,200
3.3	Drain	m^3	26,875	$	16.20	$	435,375
4	Concrete						
4.1	Plinth	m^3	7,600	$	115.00	$	874,000
4.2	Facing	m^2	72,000	$	80.00	$	5,760,000
4.3	Parapet -US	m^3	3,870	$	250.00	$	967,500
4.4	Parapet -DS	m^3	731	$	250.00	$	182,750
4.5	Crest	m^3	4,386	$	115.00	$	504,390
Subtotal						$	50,447,898
SPILLWAY							
1	Site Preparation	m^2	50,000	$	0.50	$	25,000
2	Excavation					$	-
2.1	Overburden	m^3	52,500	$	3.20	$	168,000
2.2	Rock	m^3	208,000	$	8.75	$	1,820,000
3	Headworks						
3.1	Concrete	m^3	3,386	$	115.00	$	389,367
3.2	Formwork	m^2	1,490	$	46.20	$	68,838
3.3	Reinforcement	kg	179,481	$	1.36	$	244,094
4	Chute and Flip Bucket						
4.1	Concrete	m^3	6,765	$	115.00	$	777,975
4.2	Formwork	m^2	2,130	$	46.20	$	98,406
4.3	Reinforcement	kg	310,758	$	1.36	$	422,631
5	Bridge						
5.1	Concrete	m^3	120	$	115.00	$	13,800
5.2	Formwork	m^2	150	$	46.20	$	6,930
5.3	Reinforcement	kg	18,816	$	1.36	$	25,590
6	Gates						
6.1	Gates	Each	2	$	180,000.00	$	360,000
6.2	Operators	Each	2	$	20,000.00	$	40,000
6.3	Electrical Supply	L.S.	1	$	30,000.00	$	30,000
6.4	Stoplogs	Each	1	$	80,000.00	$	80,000
7	Tailrace Channel						
7.1	Overburden	m^3	20,600	$	3.20	$	65,920
7.2	Rock	m^3	18,000	$	8.75	$	157,500
Subtotal						$	4,794,051
LOW LEVEL OUTLET STRUCTURE							
1	Shaft						
1.1	Excavation, Rock	m^3	1,698	$	150.00	$	254,764
1.2	Concrete	m^3	682	$	115.00	$	78,467
1.3	Formwork	m^2	1,241	$	46.20	$	57,314
1.4	Reinforcement	kg	53,000	$	1.36	$	72,080
Subtotal						$	462,624

Panama Canal Authority
Contract CC-3-536
Task Order 3, Indio Water Supply Feasibility Study
Dam Type Alternative Study

RCC **Ungated** **$ 57,393,853**

Quantity Take-Offs

Item	Description	Unit	Quantity		Unit Cost		Cost
DIVERSION							
1	Site Preparation	m^2	83,000	$	0.50	$	41,500
2	Diversion						
2.1	Overburden	m^3	52,900	$	3.20	$	169,280
2.2	Rock	m^3	14,800	$	8.75	$	129,500
2.3	Cofferdams	m^2	138,800	$	10.30	$	1,429,640
2.4	Concrete	m^3	4,950	$	115.00	$	569,250
2.5	Formwork	m^2	4,400	$	46.20	$	203,280
2.6	Reinforcement	kg	294,941	$	1.36	$	401,119
Subtotal						$	2,943,569
DAM							
3	Foundation Excavation						
3.1	Overburden	m^3	187,000	$	3.20	$	598,400
3.2	Rock	m^3	187,000	$	8.75	$	1,636,250
4	Grouting						
4.1	Consolidation	L.S.	1	$	1,000,000.00	$	1,000,000
5	RCC						
5.1	Mass	m^3	769,000	$	50.00	$	38,450,000
5.2	Uncompacted	m^3	3,000	$	50.00	$	150,000
5.3	US/DS Facing	m^3	19,000	$	282.00	$	5,358,000
5.4	Foundation	m^3	44,268	$	115.00	$	5,090,820
5.5	Gallery	m^3	1,250	$	115.00	$	143,750
Subtotal						$	52,427,220
SPILLWAY							
1	Headworks						
1.1	Concrete	m^3	872	$	115.00	$	100,239
1.2	Formwork	m^2	623	$	46.20	$	28,783
1.3	Reinforcement	kg	36,420	$	1.36	$	49,531
2	Chute and Flip Bucket						
2.1	Concrete	m^3	6,052	$	115.00	$	695,980
2.2	Formwork	m^2	5,108	$	46.20	$	235,990
2.3	Reinforcement	kg	228,693	$	1.36	$	311,022
3	Bridge						
3.1	Concrete	m^3	472	$	115.00	$	54,280
3.2	Formwork	m^2	590	$	46.20	$	27,258
3.3	Reinforcement	kg	74,010	$	1.36	$	100,653
Subtotal						$	1,603,735
LOW LEVEL OUTLET TOWER							
1	Tower						
1.1	Concrete	m^3	1,264	$	115.00	$	145,360
1.2	Formwork	m^2	3,713	$	46.20	$	171,541
1.3	Reinforcement	kg	75,314	$	1.36	$	102,427
Subtotal						$	419,328

Panama Canal Authority
Contract CC-3-536
Task Order 3, Indio Water Supply Feasibility Study
Dam Type Alternative Study

RCC Gated — **$ 57,597,802**

Quantity Take-Offs

Item	Description	Unit	Quantity		Unit Cost		Cost
DIVERSION							
1	Site Preparation	m^2	83,000	$	0.50	$	41,500
2	Diversion						
2.1	Overburden	m^3	52,900	$	3.20	$	169,280
2.2	Rock	m^3	14,800	$	8.75	$	129,500
2.3	Cofferdams	m^2	138,800	$	10.30	$	1,429,640
2.4	Concrete	m^3	4,950	$	115.00	$	569,250
2.5	Formwork	m^2	4,400	$	46.20	$	203,280
2.6	Reinforcement	kg	294,941	$	1.36	$	401,119
Subtotal						$	2,943,569
DAM							
1	Foundation Excavation						
1.1	Overburden	m^3	187,000	$	3.20	$	598,400
1.2	Rock	m^3	187,000	$	8.75	$	1,636,250
2	Grouting						
2.1	Consolidation	L.S.	1	$	1,000,000.00	$	1,000,000
3	RCC						
3.1	Mass	m^3	769,000	$	50.00	$	38,450,000
3.2	Uncompacted	m^3	3,000	$	50.00	$	150,000
3.3	US/DS Facing	m^3	19,000	$	282.00	$	5,358,000
3.4	Foundation	m^3	44,268	$	115.00	$	5,090,820
3.5	Gallery	m^3	1,250	$	115.00	$	143,750
Subtotal						$	52,427,220
SPILLWAY							
1	Headworks						
1.1	Concrete	m^3	5,078	$	115.00	$	583,970
1.2	Formwork	m^2	1,136	$	46.20	$	52,483
1.3	Reinforcement	kg	184,201	$	1.36	$	250,513
2	Chute and Flip Bucket						
2.1	Concrete	m^3	1,612	$	115.00	$	185,380
2.2	Formwork	m^2	1,596	$	46.20	$	73,735
2.3	Reinforcement	kg	72,873	$	1.36	$	99,107
3	Bridge						
3.1	Concrete	m^3	136	$	115.00	$	15,640
3.2	Formwork	m^2	170	$	46.20	$	7,854
3.3	Reinforcement	kg	21,325	$	1.36	$	29,002
4	Gates						
	Gates	Each	2	$	180,000.00	$	360,000
	Operators	Each	2	$	20,000.00	$	40,000
	Electrical Supply	L.S.	1	$	30,000.00	$	30,000
	Stoplogs	Each	1	$	80,000.00	$	80,000
Subtotal						$	1,807,684
LOW LEVEL OUTLET TOWER							
1	Tower						
1.1	Concrete	m^3	1,264	$	115.00	$	145,360
1.2	Formwork	m^2	3,713	$	46.20	$	171,541
1.3	Reinforcement	kg	75,314	$	1.36	$	102,427
Subtotal						$	419,328

Part 3
Component Configuration

FEASIBILITY DESIGN FOR THE RÍO INDIO WATER SUPPLY PROJECT

APPENDIX D
PART 3

PROJECT COMPONENT CONFIGURATION

Prepared by

MWH

In association with

TAMS

AN EARTH TECH COMPANY

FEASIBILITY DESIGN FOR THE RÍO INDIO WATER SUPPLY PROJECT

APPENDIX D – PART 3
PROJECT COMPONENT CONFIGURATION

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1 INTRODUCTION

With the selection of an ungated CFRD dam as the dam and spillway type to be carried forward in the feasibility study, several of the larger project components have been studied to confirm or improve the configuration of the project. These are:

1. Ungated spillway design;
2. Spillway size;
3. Diversion facilities;
4. Dam stability, and;
5. Dam height costs.

The study of the Indio to Gatun transfer tunnel is described separately in Appendix D. Part 4.

2 UNGATED SPILLWAY DESIGN

The ungated spillway selected for the dam type selection was a side channel spillway. This spillway type was selected to reduce the volume of excavation in the right abutment. Further inspection of the abutment material and the selection of a CFRD led to the conclusion that most of the abutment excavation could be used in the dam at a lower cost than the next best material source. Therefore, it was decided to revisit the type of spillway to determine if a conventional ungated spillway might be less expensive.

A conventional ungated ogee spillway layout was developed for the same ogee hydraulic capacity and width as the side channel spillway, 50 m. This resulted in an ogee width of 52 m with two one-meter wide spillway bridge piers. The spillway was aligned to be founded on competent rock, and to minimize approach channel and chute excavation. The spillway chute was also tapered to reduce excavation and concrete volumes. The maximum unit discharge during the PMF at the flip bucket is less than the adopted design criteria of 100 $m^3/s/m$. Exhibit 1 is a plan and section of the conventional spillway.

The side channel spillway headwork design and cost estimate was also revisited. The headworks comprises the ogee section of the spillway and a energy dissipating trough which dissipates energy before discharging flow at 90° over a second lower ogee section and into the spillway chute. Preliminary layouts developed for the headwork structure adopted a structural concrete thickness for the headwork walls of 0.5 m. Further evaluation was made of the structural concrete thickness required for energy dissipation for the foundation conditions anticipated on the right abutment. A review of other side channel spillway designs showed a range of concrete wall thickness for the headwork structure, ranging from 0.5 m up to 2.0 m and more. The required wall thickness is a function of both the hydraulic design loads and the foundation. While the spillway foundation is expected to be competent at the location selected for the headwork structure, limited information is currently available, and less favorable foundation conditions could result in the need to relocate the spillway headwork further into the abutment, or for additional concrete, both changes resulting in increased spillway cost. The conventional concrete spillway cost is not changed significantly by alignment changes required by poorer foundation conditions.

Costs for side channel headwork structures with wall thickness of 0.5m, 1.0 m and 2.0 m were developed for comparison with the ungated ogee spillway. Comparative costs are summarized below:

Spillway	**Side Channel Spillway, 0.5 m headwork walls**	**Side Channel Spillway, 1.0 m headwork walls**	**Side Channel Spillway, 2.0 m headwork walls**	**Conventional Ungated Spillway**
Cost, US$	5,070,000	5,504,000	5,935,000	5,180,000

The cost comparison shows that the side channel spillway with 0.5 m headwork walls is the lowest cost by about 2%. However, the side channel spillway cost could increase substantially if foundation conditions require any increase in headwork wall thickness. Foundation conditions at the site, and in the location of the spillway, are not well known. Therefore, a conventional ungated spillway is selected as the spillway for the Rio Indio Water Supply Project.

3 SPILLWAY SIZE

Based on the studies described above, the selected spillway for the optimization is a conventional ungated ogee with a tapered chute and flip bucket. The selected effective ogee crest width is 50 m, tapering over a 200 m long chute to a flip bucket 14 m wide. The objective of this study is to determine the optimum capacity (ogee width) of the spillway. Four spillway widths have been selected for optimization: 25, 50, 75 and 100 m. The cost of the spillway plus the incremental cost of the surcharge storage requird for each spillway was estimated and compared. The combination that results in the lowest cost was selected as optimum.

HEC-1 computer analyses have been performed to obtain reservoir surcharge elevations during the PMF for the range of spillway capacities being optimized. These are presented below:

Spillway Width	25 m	50 m	75 m	100 m
Surcharge El.	84.4	84.0	83.7	83.5

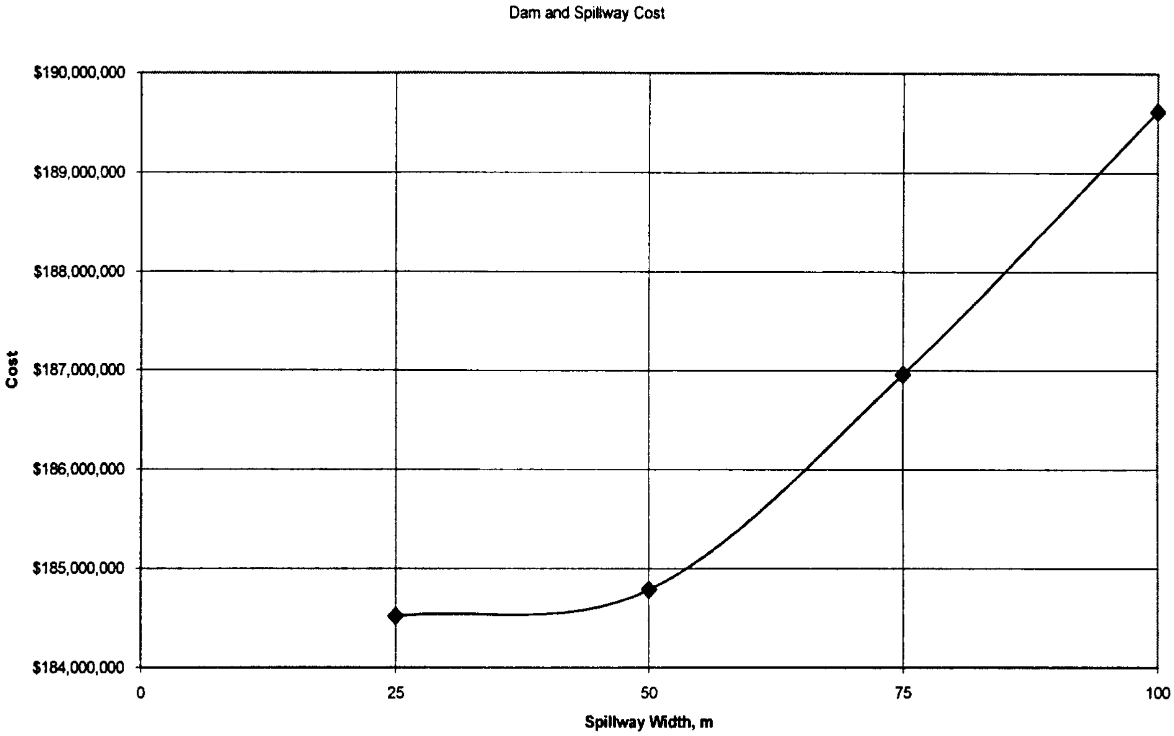
Using a meter-wide center strip of spillway, unit costs for the concrete and for rock excavation were estimated, and used to generate the incremental cost of the two wider spillways. While the 50 m wide spillway is tapered, additional tapering along the chute is limited by hydraulic requirements for the larger spillway. Therefore, the unit cost can be applied directly using the increased width.

The narrower spillway (25 m ogee crest width) was tapered to a flip bucket width of 8 m to limit the unit discharge at the flip bucket to unit discharges similar to the 50 m wide spillway. The incremental cost reduction for a 25 m wide strip was then adjusted by the ratio of the areas of spillway concrete.

All incremental costs were then increased by 40% for contingency (25%) and engineering and administration (15%). These costs are intended for comparison of the alternative spillway sizes only, and are not complete or final project costs.

The dam costs for three elevations (El. 75.0, 80.0, and 85.0) were used to estimate dam costs for top of dam elevations required to contain the surcharge for the various spillway widths. Freeboard requirements were consistent for the alternative spillways. The incremental cost of the spillway corresponding to these elevations was then included in the total cost.

The graph below shows dam and spillway cost against spillway width. The results of the optimization showed that the lowest cost dam and spillway would be for a spillway width of between 25 m wide and 50 m wide.



The analysis also demonstrates the level of attenuation provided by the reservoir.. The estimated surcharge to contain the volume of the PMF inflow with a peak of 4,345 m^3/s and a duration of 5 days, without spilling is approximately 5.0 m. .

The capacity of the transfer tunnel from Indio to Gatun is over 360 m^3/s at El. 85.0, and the capacity of the 2.5 m diameter low-level outlet tunnel at El. 80 is 100 m^3/s. The combined capacity is similar to the 25-m wide spillway capacity. Therefore, elimination of a spillway at the Project is technically feasible. Elimination of the spillway would result in the lowest cost Project.

There is little precedent for constructing a project of this size without a spillway. Additionally, submerged outlets are know to be prone to plugging and require maintenance, and are therefore unreliable to evacuate floods. Therefore, elimination of the spillway is not considered further. Spillways with widths of 25 and 50 m, within the accuracy of the optimization cost estimating, result in the lowest cost Project. To reduce the unit flow over the spillway, a 50 m wide spillway was selected.

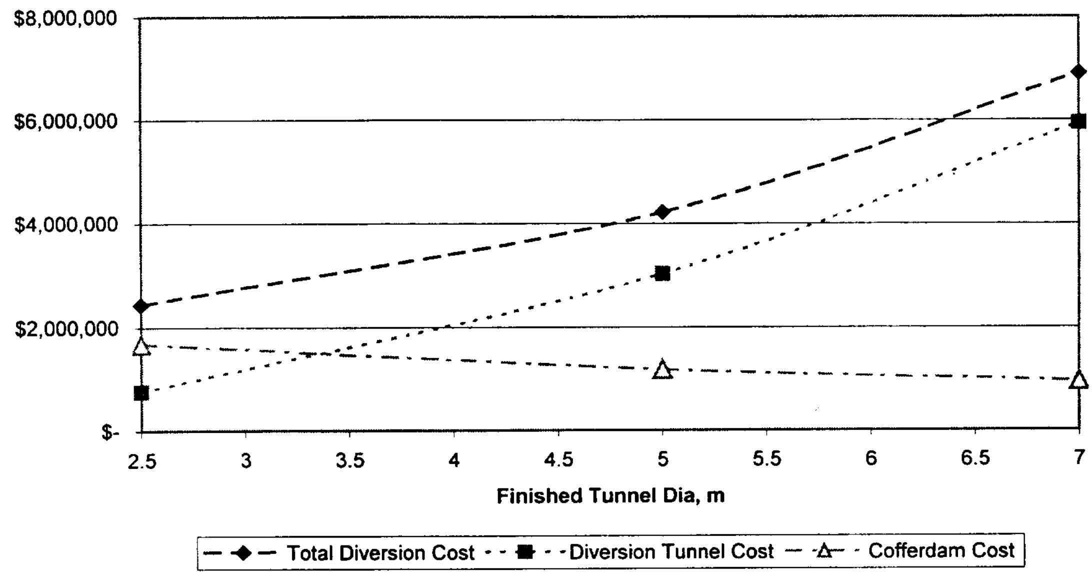
4 DIVERSION FACILITIES

The objective of this study is to perform more detailed flood routing and drawdown analysis, to include the Indio to Gatun transfer tunnel in Project drawdown capacity, and to perform a cost base selection of the optimum diversion tunnel and cofferdams combination. The selected Project operation levels are from a full supply level of El. 80.0 to a minimum supply level of El, 40.0.

The 50 year diversion flood was routed through the project for three diversion tunnel sizes. Headwater and tailwater elevations are:

Tunnel Diameter, m	2.5	5.0	7.0
Headwater El.	23.3	20.9	19.3
Upstream Cofferdam El.	23.5	21.0	19.5
Tailwater El.	6.5	8.3	9.5
Downstream Cofferdam El.	7	8.5	10

Diversion costs were estimated for each diversion tunnel and associated cofferdams. The diversion tunnel arrangement is the same as presented for the CFRD dam type, a modified horseshoe tunnel located in the right abutment. Diversion tunnel costs were developed for excavation, tunnel support, shotcrete and invert lining for each tunnel diameter, and include a contingency. The costs increased from $0.76 million for the 2.5 m tunnel to $5.9 million for the 7.0 m tunnel. Cofferdam costs decreased from $1.7 million to $1.0 million respectively. Total diversion costs are presented in the following graph of cost and tunnel diameter.



Tunnel costs increase more rapidly than cofferdam costs decrease. The small decrease in cofferdam cost is a result of the relatively small decrease in cofferdam size with increasing flood diversion capacity. This reflects the high level of flood attenuation provided by the cofferdam that reduces the required size of diversion tunnel. The lowest cost diversion facilities are the smallest tunnel diameter of 2.5 m with the highest cofferdam. A 2.5 m diameter tunnel is the smallest practical size for tunnel construction. The cofferdam required for this tunnel size can be constructed in stages, with river closure and diversion being completed during the four-month dry season.

The diversion tunnel will be constructed to serve as the low-level outlet and emergency drawdown facility. Drawdown requirements are derived from U.S.B.R. ACER Technical Memorandum No. 3, 1982. The guidelines for reservoir drawdown are as follows:

Drawdown to	Time (days)
75% of full supply volume	30-40
50% of full supply volume	50-60
25% of full supply volume	80-100

Hydraulic analysis of reservoir drawdown has been performed for drawdown from the normal full supply level of El. 80 for the average annual inflow into the reservoir of 25.7 m^3/s. Drawdown is accomplished using both the diversion tunnel at the Río Indio dam and the water transfer tunnel from Indio to Gatun. Hydraulic analysis and flood routing studies were performed for several different size transfer tunnels and diversion tunnels. The studies demonstrated that for a water transfer tunnel diameter of 4.5 m and 5.0 m, the reservoir drawdown criteria is met with a diversion tunnel of 4.0 m and 3.5 m respectively:

Drawdown to	Time (days)	Diversion Tunnel 4.0 m Transfer Tunnel 4.5 m	Diversion Tunnel 3.5 m Transfer Tunnel 5.0 m
75% of full supply volume	30-40	26.1	29.4
50% of full supply volume	50-60	48.6	55.6
25% of full supply volume	80-100	68.4	82.8

Therefore, for an Indio to Gatun water transfer tunnel diameter of 4.5 m the selected diversion tunnel is 4.0 m.

5 DAM STABILITY

Dam stability analyses have been performed for the CFRD main dam, and the saddle dam located on the right abutment of the main dam.

5.1 Stability Analysis of the Río Indio CRFD

The feasibility level design of the Río Indio CFRD has been performed in accordance with guidelines presented in Cooke, J. B. 1998, Empirical Design of the CFRD, Hydropower and Dams, Issue Six. A copy is provided in Attachment 1. The governing design for the dam is deformation during an earthquake event. Maintenance of gross stability and sufficient freeboard remain the primary requirements in evaluating seismic performance of rockfill dams.

Limited emphasis is placed on the analysis of the concrete slab, since cracking of the slab is unlikely and is not expected to result in excessive leakage. The internal zoning of the dam is designed to control the amount of water that can enter any potential crack, and then direct this leakage out of the fill maintaining it dry. Design of the slab is usually based on previous experience and professional judgment. The thickness of the slab and its reinforcement have been reduced through the years of development of the CFRD, reaching the current recommendations, for a dam of this height, of 0.25 m to 0.4 m constant thickness and 0.3% to 0.35% reinforcement each way, which provide an economical and reliable watertight element.

The method of analysis to estimate the seismic deformation of the rockfill and to evaluate the adequacy of freeboard follows the recommendation of Bureau et al. 1985 Seismic analysis of concrete face rockfill dams, published in Cooke, J. B. and Sherard, J. L. Concrete Face Rockfill Dams _Design, Construction and Performance, ASCE, New York. Attachment 2 provides the deformation computation for the maximum design earthquake (MDE). Estimated deformation for the CFRD is less than 0.1 m as a result of the MDE. The dam is provided with 3 m of freeboard from the full supply level of El. 80.0 to the crest at El.83.0. The dam has adequate freeboard to prevent overtopping as a result of deformation during the MDE.

Conventional limit state stability analyses are not performed for CFRD dams. Rockfill slopes on competent foundation and no pore pressure, using slopes up to 1.3H:1V have been found stable without the need for stability analyses, as discussed in Cooke 1998.

5.2 Saddle Dam Stability Analysis

Stability analysis for the saddle dam has been performed. The analysis was performed using the maximum section of the saddle dam, and the limit equilibrium method of stability analysis, as implemented in the Morgenstern-Price approach of the program SLOPE/W. There is no testing available to develop geotechnical design parameters, so the values selected are based on the experience of the company in residual soils of similar origin from Panamá and other tropical areas.

The parameters used were:

Material	Dry Unit Weight (kN)	Friction Angle	Cohesion (kPa)
Downstream shell	18	34	0
Upstream shell	18	32	0
Filter	18	35	0
Foundation	18	32	0

At this stage only the basic load cases were studied. These included normal reservoir, and pseudostatic. The pseudostatic coefficient was set at 0.15, based on the peak ground acceleration estimated for the site. The acceptance criteria is based on the USACE criteria presented in USACE Stability of earth and rockfill dams, EM 1102-2-1902. The results from these analyses are an indication that the embankment, as designed, is stable. The pseudostatic analysis should be considered only an indication of the behavior during earthquake. During the following stages of design, the issue of stability will need to be studied in more detail.

The saddle dam stability analyses show the dam exceeds the required factor of safety for the selected design cases. Analysis are presented in Attachment 3, and summarized below:

Table 1: Summary of Saddle Dam Stability Analysis

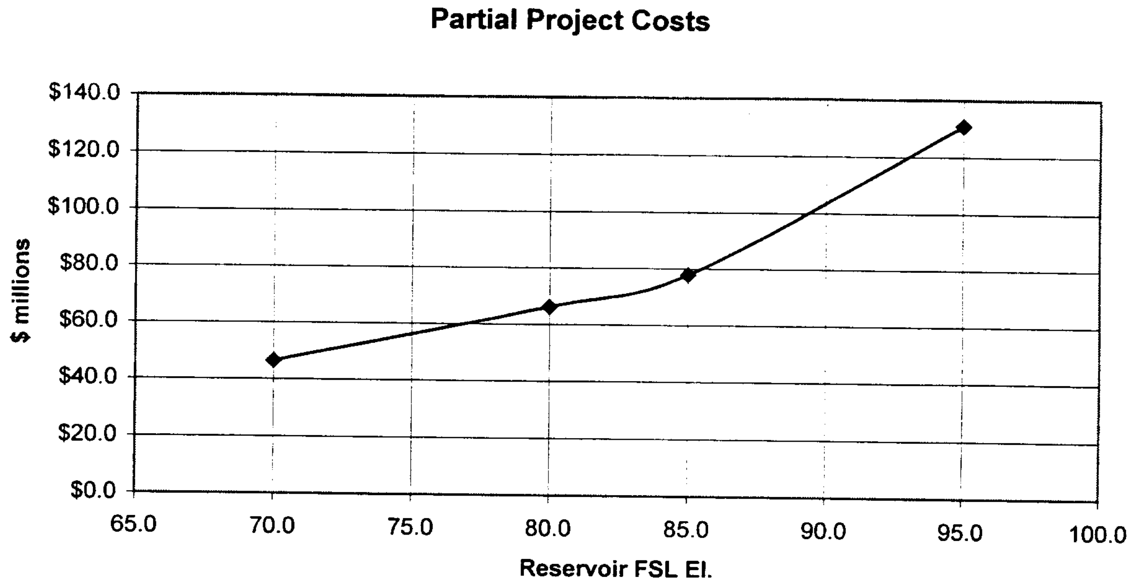
Load case	Calculated FS	Required FS
Maximum Pool DS	1.9	1.5
Maximum Pool DS – Earthquake	1.3	1.0
Maximum Pool US – Earthquake	1.8	1.0

6 DAM HEIGHT COSTS

Costs have been developed for four dam heights as input data for a cost/benefit analysis to optimize the height of the Río Indio Water Supply Project. **Table 2** presents a summary of comparative costs for projects with a full supply level at El.70.0, 80.0, 85.0, and 90.0. The data is shown graphically immediately below.

Table 2: Summary of Partial Project Costs

Reservoir Full Supply Elevation, m, msl.	70.0	80.0	85.0	95.0
Diversion	$4,303,605	$4,303,605	$4,303,605	$4,303,605
Dam	$32,116,897	$49,056,480	$57,421,423	$80,000,000
Spillway	$5,644,014	$5,179,317	$4,965,438	$4,590,000
Low Level Outlet	$462,624	$462,624	$462,624	$462,624
Saddle Dams	$3,888,782	$7,211,591	$10,737,436	$41,399,310
Total	$46,415,923	$66,213,618	$77,890,527	$130,755,540
Total, $million	**$46.4**	**$66.2**	**$77.9**	**$130.8**

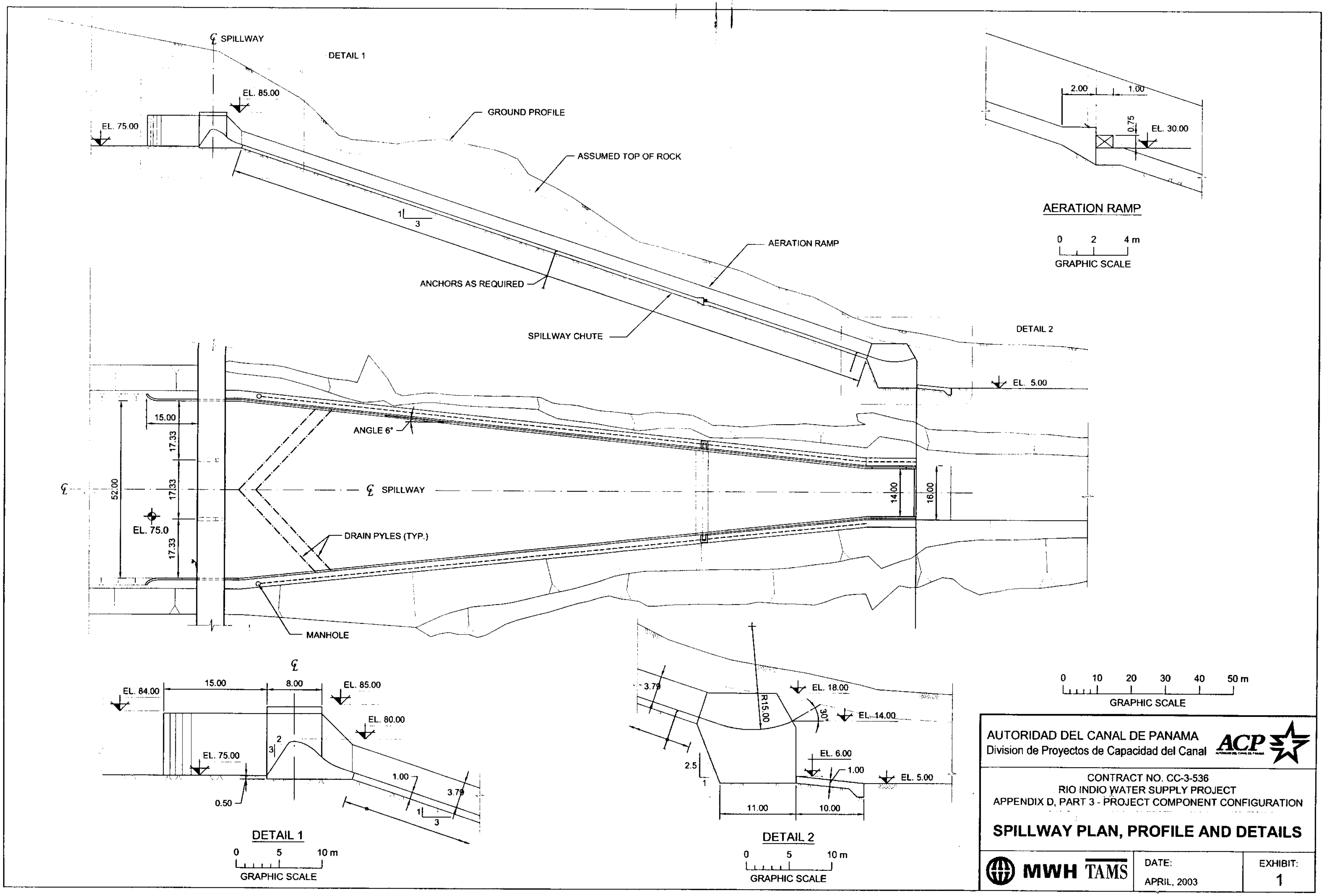


The partial project costs include diversion, dam, spillway, low level outlet, and saddle dam costs only. Reservoir costs, other common project costs (such as the Indio-Gatun transfer tunnel), contingencies and engineering and administration costs are not included. The unit costs used to develop component costs were adopted from the dam type selection study. Quantity take-offs were prepared for the El.70.0, 80.0 and 85.0 projects. Apart from the saddle dam quantities, the El 95.0 costs were developed by ratio from the El. 80.0 costs.

The graph clearly indicates when raising the FSL of the reservoir requires construction of numerous additional saddle dams around the reservoir perimeter. Between El. 80.0 and El. 85.0, a low section of the right perimeter of the reservoir necessitates construction of saddle dams. This rapidly increases the partial project cost, resulting in a significant project cost increase by El. 95.0.

Studies of the optimum height of the Río Indio Water Supply Project have not been performed.

EXHIBITS



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

ACP

CONTRACT NO. CC-3-536
RIO INDIO WATER SUPPLY PROJECT
APPENDIX D, PART 3 - PROJECT COMPONENT CONFIGURATION

SPILLWAY PLAN, PROFILE AND DETAILS

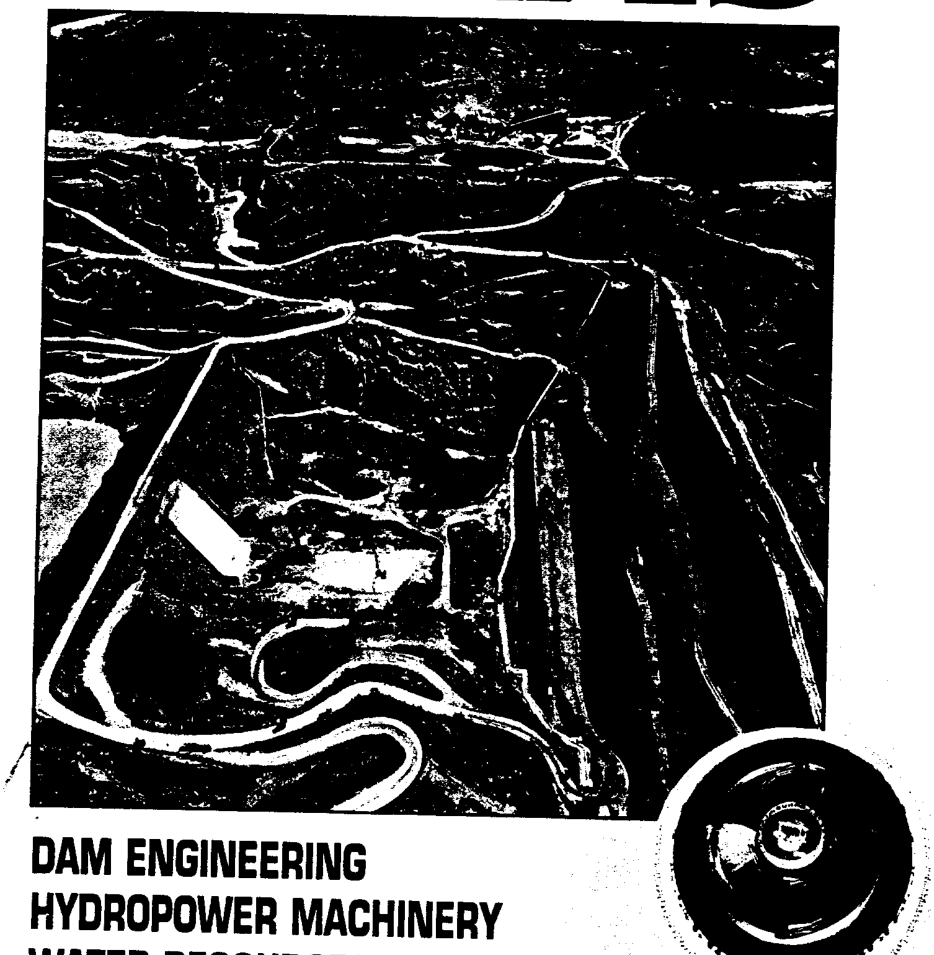
MWH TAMS

DATE:	EXHIBIT:
APRIL, 2003	1

ATTACHMENTS

Attachment 1 - Empirical Design of the CFRD

THE INTERNATIONAL JOURNAL ON
HYDROPOWER & DAMS



DAM ENGINEERING
HYDROPOWER MACHINERY
WATER RESOURCES MANAGEMENT

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Empirical design of the CFRD

J.B. Cooke, Consultant, USA

The development of the concrete face rockfill dam, from the first CFRDs of compacted rockfill to current design practice, is presented. Design features were at first based on precedent dams, and as experience was gained in performance, gradually departures began to be made from these designs. The trends reviewed in the article are based on the experience and planning of about 300 dams.

Empirical is defined as 'guided by practical experience and not theory'. This describes the basis for design of the CFRD, since the initial CFRDs constructed in California by gold miners in the 1850s.

The inherent safety of the CFRD makes empirical design acceptable. Factors justifying the term 'inherent safety' are:

- All the zoned rockfill embankment is downstream from the reservoir water.
- The water load enters the foundation upstream from the dam axis.
- Uplift and pore pressure are not involved.
- The rockfill has reliable and high shear strength.
- The zoned rockfill is stable against through-flow.
- Settlements of rockfill are small and essentially cease after several years.

Empirical design is based on precedent, which is defined as 'something that can be used as an example for similar cases'. The data from precedent CFRDs and the site conditions for the new dam are the bases for design. Precedent has not been strictly followed, which is important, as progress must be made. As experience has accumulated, gradual modifications to precedent design and to construction practice have been adopted. Modifications have been made with the objectives of reducing leakage, reducing cost, and simplifying construction.

Another form of precedent is engineering precedent: knowledge of the design of dams under construction, in design and in planning. It is useful to know and to consider what others are currently doing.

The design, construction and performance of CFRDs is well presented in the technical literature [Cooke, 1984[1]; Cooke, 1985[2]; Sherard, 1987[3]; Jiang, Zhang and Qin, 1993[4]]. A Table entitled 'Concrete face rock fill dams (> 40 m height) [*WP&DC*, 1996-99[5]; *H&D*, 1999[6]], gives the basic design data of nearly all the approximately 300 dams built since 1985. Together these references provide a base for the empirical design of the CFRD. Site-specific factors are taken into account by adapting the standard dam section to the foundation geology and the rock (or gravel) sources.

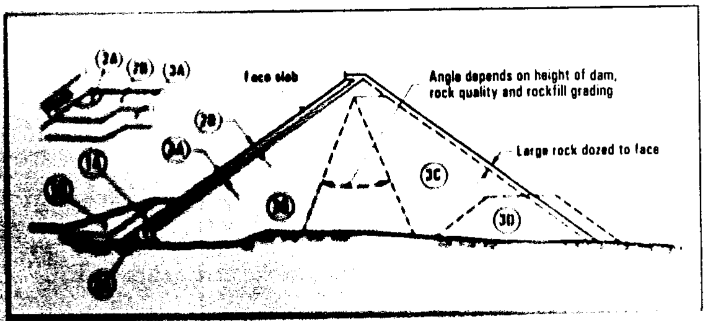

Fig.1. Section of a typical CFRD of sound rock on a sound rock foundation
1A = Impervious soil; 1B = Random fill, 0.3 m layers; 2A = Processed fine filter, 2B = Crusher-run rockfill; 3A = Selected small rock placed in same layer thickness as Zone 2; 3B = Quarry-run rockfill, about 1 m layers; 3C = Quarry run rockfill, about 2 m layers; 3D = Dumped rockfill (optional). Compaction for rock zones: four passes of 10 t vibratory roller

1. Typical section of a CFRD

Fig. 1 is a typical section of a CFRD of sound rock on a sound foundation. For low strength rock, the section is essentially the same, with special provisions being made for compaction and drainage. For low strength, pervious and erodible foundation conditions, special provisions include:

- extending the plinth downstream by a reinforced and undowelled concrete or shotcrete slab; and,
- providing filters over erodible zones in the foundation.

A concrete face dam of gravel, or gravel and rockfill, is included in the CFRD type of structure. The concrete face and zoning designs are essentially the same as for a dam section of rockfill. A special CFRD of gravel is the Santa Juana dam in Chile, which is constructed on deep alluvial gravels. A concrete panel wall, upstream of and connected to a plinth slab on gravel, was used as the cutoff. Since then, Puclaro in Chile and several other dams have adopted that design.

2. Table of CFRDs

In the paper for the 18th Terzaghi Lecture in 1982, 'Progress in rockfill dams' [Cooke, 1984[1]], a Table listing the then existing CFRDs and their principal design features, was presented. This Table is updated and published annually, and lists about 300 dams. The dams are listed chronologically, so that design trends can be seen. In addition to dams which are completed, those under construction and at the planning and feasibility study stage are included, to show what may be termed 'engineering precedent', in other words, current thinking.

Table 1 is an extract from the full Table. The columns for the four principal design features are shown for 20 selected dams of the approximately 300 listed in the whole Table. Other columns in the complete Table are: type of rock, volume of rockfill, area of face and reservoir capacity.

3. Design trends

The modifications to precedent designs have been principally directed to economy. Progress has also been made in perimeter joint design, in rockfill zoning and in construction.

3.1 Rockfill slopes

The slopes for dumped rockfill were, in the case of natural dumped rockfill, 1.3H:1V, and as steep as 1H:1V for the upstream face.

For the small size Zone 2 rockfill at the upstream

Table 1: Chronological list of selected CFRDs showing the four principal design elements

Name	Country	Year completed	Height (m)	Slope H to V U/s	Slope H to V D/s	Face slab $t = m + CH$	Reinforcing each way (%)	Plinth width (m)
Salt Springs	USA	1931	100	1.1-1.4	1.4	$0.3 + 0.0067H$	0.5	Trench
Courtright	USA	1958	98	1.0-1.3	1.3	$0.3 + 0.0067H$	0.5	Trench
New Exchequer*	USA	1966	150	1.4	1.4	$0.3 + 0.0067H$	0.5	Plinth
Cabin Creek	USA	1967	76	1.3	1.75	$0.3 + 0.0067H$	0.5	Plinth
Cethana	Australia	1971	110	1.3	1.3	$0.3 + 0.002H$	0.6	3.5
Alto Anchicayá	Colombia	1974	140	1.4	1.4	$0.3 + 0.003H$	0.5	7
Chuza (Golillas)	Colombia	1978	130	1.6	1.6	$0.3 + 0.0037H$	0.4	3
Foz do Areia	Brazil	1980	160	1.4	1.25-1.4	$0.3 + 0.0034H$	0.4	4.55-5.75
Murchison	Australia	1982	89	1.3	1.3	0.3	0.65	3-4-6
Salvajina	Colombia	1983	148	1.5	1.4	$0.3 + 0.0031H$	0.4	4-8
Chenbing	China	1989	75	1.3	1.3	$0.3 + 0.0027H$	0.3H-0.5V	3-12
Segredo	Brazil	1992	145	1.3	1.2-1.4	$0.3 + 0.0035H$	0.3-0.4	4-6.5
Aguamilpa	Mexico	1993	187	1.5	1.4	$0.3 + 0.003H$	0.3-0.35	4-6
Wanaxi	China	1995	94	1.4	1.4	$0.3 + 0.002H$	0.35-0.4	4-5
Babagon	Malaysia	1996	63	1.3	1.6	0.3	0.3	3-5
Caruachi	Venezuela	1999	63	1.3	1.3	0.35	0.35	3
Itá	Brazil	1999	125	1.3	1.3	$0.3 + 0.002H$	0.3-0.4	4
Tianshengqiao 1	China	1999	178	1.4	1.4	$0.3 + 0.005H$	0.4	4-9
Torata	Peru	2002	100	1.3	1.3	$0.3 + 0.002H$	0.3-0.35	4
Xekaman	Laos	-	187	1.3	1.3	$0.3 + 0.002H$	0.35-0.4	4

*Change from dumped rockfill to compacted rockfill

face of the compacted rockfill dam, a slope steeper than 1.3H:1V was not practical. The upstream slopes of Alto Anchicaya (140 m high) and Foz do Areia (160 m high) were 1.4H:1V, out of respect for their breaking the precedent of maximum height. Occasionally a designer will use a 1.4H:1V upstream slope for which there is not a justification. The traditional 1.3H:1V upstream slope is still current practice.

The downstream slope of 1.3H:1V also remains current practice. Exceptions are cases where low strength, weathered rock or earth is zoned into the downstream section of the dam. At Cabin Creek and Babagon, the slopes of 1.75H:1V and 1.6H:1V are the result of the economic use of required excavation earth and highly weathered rock. When there is to be a road on the downstream face, 1.25 or 1.2H:1V is used, with an average slope of 1.4H:1V at Foz do Areia and Segredo.

Concrete face rockfill dams of gravel or part gravel are included in the CFRD designation of dam type. At Chuza (130 m high), the first high gravel CFRD, the slopes were 1.6H:1V because the main source material was dirty terrace gravel. At Salvajina and Aguamilpa, the upstream slope of gravel is 1.5H:1V, and the downstream slope, comprising both sound and weathered excavated rock, is 1.4H:1V. Current practice is for both gravel slopes to be 1.5H:1V.

3.2 Face slab thickness

For the CFRD of dumped rockfill, a thickness formula of $0.3\ m + 0.0067H$ was used. It was thick, since the placed rock under the face slab gave irregular support. For Cethana, which in 1971 was the world's highest CFRD of compacted rockfill, a formula of $0.3 + 0.002H$ was used. The compacted rockfill of Zone 2 was given credit for its uniform support. Since then, $0.3 + 0.003H$ has often been used, but the incremental thickness of $0.002H$ has become current practice. Economy in the thickness of the slab is significant. It includes saving reinforcement. Since the thickness of the slab does not involve safety, a further decrease in thickness can be considered.

For dams less than about 90 m high, a uniform thickness of 0.3 m has often been adopted, and sometimes 0.25 m. The maximum gradient for 0.3 t for a 100 m-high dam and for $t = 0.3\ m + 0.002H$ for a 200 m-high dam are about the same at 300. This is high, but there has been no evidence of concrete face deterioration. The concrete mix design is generally 20 MPa at 28 days and with air entrainment and pozzolan. AAR has never developed in the saturated face concrete of the CFRD. This could be credited to the pozzolan.

3.3 Percentage of reinforcement

Shrinkage cracks occur in the face of all CFRDs, as they also do in spillway chute slabs. They have been of no consequence, but have been studied in the interest of optimizing the percentage of reinforcement. A close inspection is necessary to see the cracks. The cracks in the continuously reinforced slipformed panels are horizontal, and there are very few vertical cracks. The cracks are 0.1-0.2 mm wide, with an occasional 0.3 mm width. They seem to seal themselves. There are fewer and narrower cracks when the face is placed in cold weather.

With the changes from dumped to compacted rock, a prompt decrease in reinforcing was made from 0.5 per cent to 0.4 per cent each way. Recognizing the cross canyon compression and the few vertical cracks, it became evident that horizontal reinforcing could be minimal.

Consequently, 0.3 per cent has generally been adopted for horizontal reinforcing, and 0.35 or 0.4 per cent for vertical reinforcing. Since cracks are increasingly recognized to be innocent, the trend in vertical reinforcing is towards 0.35 or 0.3 per cent. For major and high dams, 0.4 per cent both ways is used within about 10 m of the perimeter joint.

3.4 Plinth

The resurrection of the CFRD came between the New Exchequer and Cabin Creek dams in 1966-67, with the advent of compacted rockfill. At the same time, an equally decisive change was from the cutoff blasted trench in rock to the dowelled reinforced concrete plinth. The width of the plinth was a groutable width of 3 or 4 m. It was usually increased in width in several steps of 1 or 2 m, with gradient in mind. This has been satisfactory, but did not take into account the fact that quality of rock at the riverbed is more sound, and that the rock higher up the abutment is more weathered and relatively more pervious.

A design concept which is increasing in application is to use a standard structural plinth width of 3 or 4 m. Where the foundation rock needs a longer impervious path, paving can be provided downstream for some distance with an undowelled thin slab of lightly reinforced screeded concrete or shotcrete. It is covered with a metre of Zone 2A, the filter zone of which would be carried a distance downstream. This design pays more attention to the actual foundation quality, and is more economical. There is less excavation upstream of the plinth reference line and less structural plinth. The foundation is excavated to rock for some distance downstream from the plinth in any event. This concept has been adopted at the Babagon and Ita dams.

3.5 Parapet wall

The early smaller dams had a parapet wall 1.2 m high. As the maximum dam heights increased, the parapet wall heights increased to 3 to 5 m, since the saving in rockfill would exceed the cost of the parapet wall. Also, the concrete face placement could begin sooner and with a wide crest that facilitated construction.

3.6 Height

The empirical and relatively standard design has not changed as maximum heights have increased from 80 to 110 to 160 to 187 m, from Cabin Creek in the USA in 1967 to Aguamilpa in Mexico in 1993. Currently planned projects include CFRDs with heights of 190, 205, 210, 220 and 230 m [Cooke, 1997[7]].

4. Performance

The empirically designed CFRDs have all performed safely. The engineered rockfill, with its liberal provision for drainage and zoned for safe through-flow, is inherently safe, except for sustained overtopping. Current meteorology, hydrology and PMF concepts take care of that risk.

The objects of reviews of performance have been:

- to effect savings in cost;
- to simplify construction;
- to reduce leakage, and;
- to acquire a base for predicting the performance of high dams.

4.1 Stability

There have been no stability problems. The high and reliable shear strength of rockfill with no pore pressure can be counted upon.

Few significant earthquake events have occurred at CFRD sites. The highest acceleration experience was at Cogoti dam in Chile in 1943 [Cooke, 1984[1]; Cooke, 1985[2]]. The 85 m-high dam of dumped rockfill settled 0.4 m. A gap developed between the rockfill and the concrete face in the upper several metres of the dam. There was no face damage.

Dynamic seismic analyses have indicated that the effect of high seismic loading is to cause crest zone settlement and displacement. The several seismic formulae indicate no more than about 0.3-1 m of settlement for 100-150 m-high CFRDs under the most extreme seismic loading. The face may be damaged, but zoning and freeboard would prevent breaching.

4.2 Leakage

Leakage for high CFRDs of dumped rockfill was the reason for the temporary abandonment of the CFRD as a dam type between 1940 and 1955. The highly segregated rockfill at the base of the dumped rockfill has a very low modulus of compressibility. Face cracks and leakage occurred in dams more than 80 m high. With the advent of compacted rockfill (1960-65), the CFRD began its development to become a major dam type for the highest dams. However, leakage remained a problem for the higher dams.

The principal causes of leakage leakage were identified as:

- Faulty installation of the waterstop and placement of face concrete at the perimeter joint.
- Inadequate compaction near the perimeter joint, resulting in excessive offset and waterstop damage.
- A high plinth without adequate buttressing.
- Highly pervious Zone 2 material at the perimeter joint.

Leakage was in and near the perimeter joint. The compacted rockfill provides uniform support to the face slab, and the face slab cracks, associated with dumped rockfill, no longer occur. Steps which have been effective in reducing are:

- Particular attention paid to design, inspection and construction in the vicinity of the perimeter joint.
- The provision of two perimeter joint waterstops.
- Using the plate vibrator for compaction within 3 m of the perimeter joint.
- Having a minus 1.5 in (3.8 cm) dirty filter within 3 m of the plinth, Zone 2A.
- Providing earth backfill on the perimeter joint in the riverbed and lower abutment area.

The Zone 2A has been a useful development. It is semi-pervious and limits possible leakage. With the application of silt, leakage can be reduced.

4.3 Face slab

Significant development occurred in face slab design and construction just after compacted rockfill replaced dumped rockfill. At Piedras, Pindari and Cethana, continuous slipformed placement of the face slab panels was adopted. Also, the vertical joints were con-

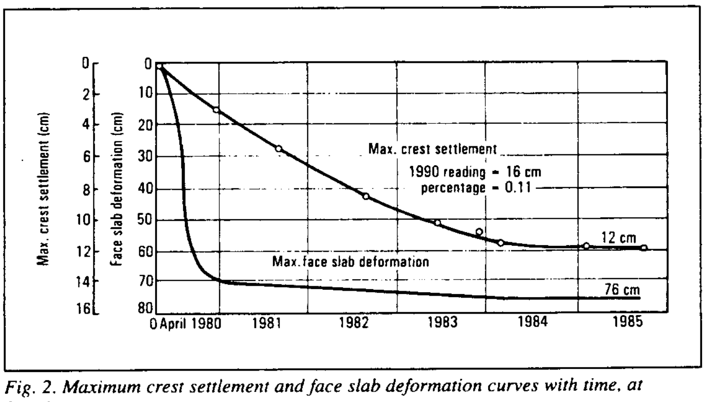

Fig. 2. Maximum crest settlement and face slab deformation curves with time, at Foz do Areia.

Table 2: CFRD maximum crest settlement with time					
Dam	Period	Years	Height (m)	Settlement (cm)	(%)
Cabin Creek	1966-1995	29	64	13.9	0.22
A. Anchicaya	1974-1984	20	140	17.3	0.12
R.D. Bailey	1980-1998	18	96	42	0.44
Foz do Areia	1980-1990	10	160	16	0.10
Sugarloaf	1984-1997	13	85	3.7	0.04
Khao Laem	1984-1998	14	115	15	0.16
Turimiquire	1986-1995	9	115	17	0.15
Segredo	1992-1997	5	145	20	0.17
Xingó	1993-1997	4	150	49.3	0.33

struction joints with no filler. The face slab became a monolithic membrane floating on the compacted rockfill. The continuous slipforming was not only economical, but provided a trouble-free membrane. The performance of the face slab has been excellent, and has justified a thinner and more lightly reinforced design. The face slab area requiring attention has been at and near the perimeter joint.

4.4 Settlement

During placement of the rockfill, water level settlement devices are used to measure settlement and enable the vertical modulus of compressiblity to be determined. From actual measurements of face deflection at the time of reservoir filling, the modulus for water loading on the face is 2 to 2.5 times greater. The vertical modulus is used to estimate the maximum deflection of the face. The maximum interior settlement during construction is at mid-height of the dam. For Foz do Areia (160 m) it was 3.58 m, at Alto Anchicaya (140 m) it was 0.63 m, and at Cethana (110 m) it was 0.45 m. The Foz do Areia basalt rockfill is of much lower modulus than the hornfels and quartzite rockfills.

Fig. 2 shows the maximum crest settlement and face deflection at Foz do Areia. The crest settlement in the first four years was small, and proceeded at a negligible rate over the following six years. The face deflection occurs at the first filling, and does not increase with time, which is particularly favourable.

Another settlement and face movement factor is face offset at the plinth as a result of reservoir filling, which has been a problem. Measures taken to reduce this offset and reduce leakage have essentially solved the problem, but it continues to receive attention.

Performance of the CFRD as measured by settlement has been satisfactory. Crest settlement, face deflection and face offset at the plinth are not foreseen to be problems for higher dams.

5. Conclusion

A principal element of empirical design is precedent. For the CFRD, progress has been slowed by following precedent too closely for too long. However, that did result in a broader base on which to make departures from successful precedents. Much progress has been made and can be expected to continue. ◊

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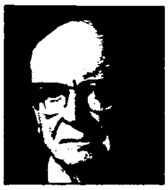

J.B. Cooke

J. Barry Cooke graduated from the University of California, Berkeley in 1939, and spent 22 years with Pacific Gas and Electric, during which time he was engaged on the design and construction of the company's new hydro plants. On leaving PG&E, he became a consultant on hydro projects and particularly on rockfill dams. He was Chairman of the ASCE Committee for the 'Symposium on Rockfill Dams', the Proceedings of which were published in a special 720 pp Transaction. This covered all the world's earth core and concrete face rockfill dams. He was General Reporter for Q31, High Rockfill Dams, at ICOLD's Congress in Edinburgh in 1964, and Co-Chairman of the China ISHERD Rockfill Dam Symposium in Beijing (1993). He continues to consult on many of the world's hydro projects and rockfill dams.

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Attachment 2 - Seismic Analysis of CFRD Deformation

MWH
MONTGOMERY WATSON HARZA

CHICAGO

SUBJECT Rio Indio Earthquake Severity Index | PROJECT NAME Rio Indio Water Supply

COMPUTED CJJL | DATE 030109 | PROJECT NUMBER 1001217.030105

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PURPOSE Estimate the deformation of the rockfill due to the Maximum Design Earthquake (MDE)

REFERENCES Bureau G. et al. 1985 Seismic Analysis of Concrete Face Rockfill Dams, in Cooke, J. B. and Sherard, J. L. Concrete Face Rockfill Dams - Design, construction and performance. ASCE New York.

RESULTS

Calculations of the Earthquake Severity Index (ESI) and resulting deformation calculated following the procedure presented by Bureau et al 1985, indicate that the estimated vertical deformation of the proposed CFRD at Rio Indio site is less than 0.1 m. The free board provided in the design is larger than this value, so current dam design would allow safe operation after occurence of the MDE.

CALCULATIONS

$pga := 0.21$ **Peak ground acceleration**

$Mag := 7.5$ **magnitude of earthquake for the MDE**

$ESI := pga \cdot (Mag - 4.5)^3$

$ESI = 5.67$

relative vertical deformation (Figure 1 of Buerau et al 1985) $Vdef := 0.0012$

$Hdam := 80 \cdot m$

$Vdef := Hdam \cdot Vdef$

$Vdef = 0.096\,m$

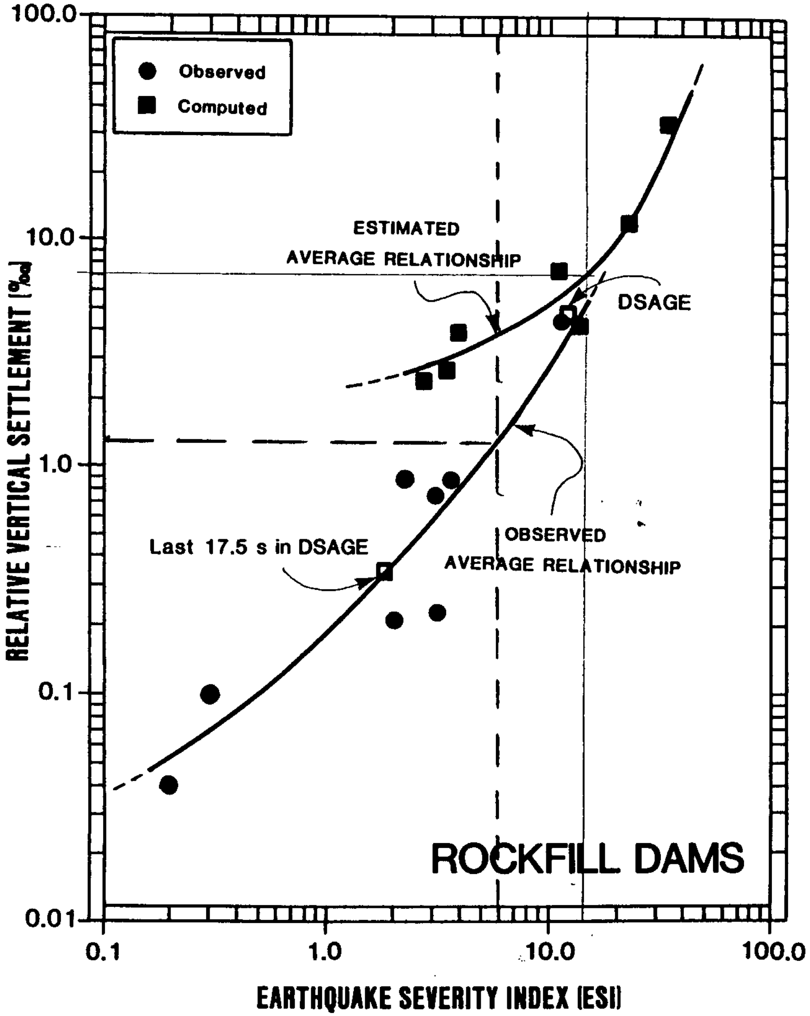


Figure 1. Relationship Between Crest Settlement and ESI

-- Mag 7,5
pga 0,21

Table 2. Relative Settlement and ESI Data

DAM NAME (location)	H (m)	ΔH (mm)	RELATIVE SETTLEMENT (‰)	PGA (g)	M	ESI	REFERENCE
COGOTI (Chile)	84	381(o)	4.54	0.20	8.3	11.0	(41)
MIBORO (Japan)	131	30(o)	0.23	0.20	7.0	3.1	(29),(41)
MINASE (Japan)	67	7(e) 61(o)	0.10 0.91	0.02 0.08	6.9 7.5	0.3 2.2	(27)
OROVILLE (Calif.)	235	9(o)	0.04	0.10	5.7	0.2	(46)
EL INFIERNILLO (Mex.)	148	130(o)	0.88	0.12	7.6	3.6	(35)
LA VILLITA (Mex.)	60	45(o)	0.75	0.10	7.6	3.0	(35)
LEROY ANDERSON (Calif.)	72	15(o)	0.21	0.41	6.2	2.0	(44)
TERROR LAKE (AK)	50	600(e)	12.00	0.35	8.5	22.4	(18)
TERROR LAKE (AK)	50	300(e)	4.00	0.50	6.5	4.0	(18)
CIRRATA (Indonesia)	125	300(e)	2.40	0.35	6.5	2.8	(18)
PUEBLO VIEJO (Guatemala)	133.5	4450(e)	33.33	0.65	8.25	34.3	(4)
CHICOASEN (Mexico)	240	1000(e)	4.17	0.85	7.0	13.3	(13)
FORTUNA (Panama) (raised)	104	800(e)	7.69	0.40	7.5	10.8	(16)
USBR (Example Dam)	213	700(e)	2.72	0.43	6.5	3.5	(45)
Dames & Moore (Example Dam)	100	487(e)	4.87	0.70	7.1	12.3	DSAGE (This paper)

(o) observed
(e) estimated

Advanced Seismic Evaluation Procedures

The main uncertainties in conventional simplified or detailed analyses of concrete face rockfill dam are primarily related, by increasing order of significance, to: (1) neglecting vertical motion, (2) neglecting hydrodynamic effects of the reservoir water and (3) computing non-recoverable plastic deformations by approximate procedures based on the results of an elastic analysis. Advanced analysis procedures are available to avoid making use of these simplifying assumptions. These improvements of the analysis methodology for concrete face rockfill dams are discussed below:

finite-element model of the dam. The earthquake forces that induce the permanent deformations are, therefore, represented by equivalent increases in the static stresses; the dam deforms in response to this new load.

The above procedures decouple the estimation of dam response and deformations. Additional simplifying assumptions are frequently used: (a) treat the dam foundation as a rigid base (rather than an energy-absorbing boundary); (b) consider horizontal earthquake loading only; (c) disregard hydrodynamic effects of the reservoir water upon the concrete face. Assumption (a) seems reasonable, considering that most concrete face rockfill dams are founded on hard erosion-resistant foundation. Assumptions (b) and (c) (generally acceptable for earthfill dams) need to be substantiated for concrete face rockfill dams, because of their steep slopes, and because water pressures apply directly to the concrete facing, concentrating the loads in that area.

Variances of the above evaluation procedures are possible. For dams which are narrow relative to their height, two-dimensional analyses may overestimate the fundamental period of these structures. The shear modulus can be adjusted so that the two-dimensional model duplicates the response expected from the three-dimensional dam more closely (Vrymoed, 1981). Three-dimensional and probabilistic finite element equivalent-linear response analyses of rockfill dams have also been used (Yanagisawa, Fukui, 1980; Kagawa et al., 1981).

Overall, the Newmark method, combined with detailed dynamic response analyses, applies reasonably well to the seismic evaluation of rockfill dams. The Lee and Serff methods are more difficult to implement because they were originally intended for earthfill dams, where the stress-strain relationships can be established through the dynamic testing of the embankment materials. Cyclic tests on rockfill materials are impractical. Furthermore, the "deformed" embankment shapes determined in the decoupled analyses using the Lee or Serff methods are sometimes questionable.

It should be noted that because of their stiffness characteristics, rockfill dams respond to earthquake motion at shorter periods than earthfill dams: first mode frequencies between 2 and 5 Hertz were derived from measured response to earthquake- and vibrator-induced motions for ten Japanese rockfill dams with heights ranging from 213 to 445 ft (65 to 135 m) (Takahashi et al., 1977). Although fundamental frequencies would decrease under severe earthquake loading, one can expect that relatively high frequency components would contribute to a significant part of the dam response, which emphasizes the need for determining the acceleration response of the dam accurately.

Since maintenance of gross stability and sufficient freeboard remain the primary requirements in evaluating the seismic performance of rockfill dams, limited emphasis has presently been placed on the analysis of the concrete slab itself, especially since cracking of the slab is not expected to result in excessive leakage problems. The simplifying assumptions and limitations of the dam analysis procedures normally

preclude direct inclusion of the concrete face in the dam finite element model. The concrete face is either analyzed separately by applying the computed dam deformations as boundary conditions to a mathematical model of the slab, or is designed based on previous experience and professional judgment, taking into account the amount of shear and moment deformations that the concrete, joints, and steel reinforcement can withstand without losing their effectiveness.

Earthquake Severity Index (ESI)

Since earthquake accelerations are random in nature, the number of pulses above the yield acceleration that contribute to the cumulative total displacement in the Newmark method can be assumed to be proportional to the duration D of the strong phase of shaking (for a given intensity of motion, measured by the peak ground acceleration A in g's). Hence, we propose to relate estimated deformations and the product AD^2 (as accelerations are integrated twice with respect to time to obtain displacements). Based on a review of earthquake durations as a function of magnitude (Chang, Krinitzsky, 1977), we have formulated an average relationship between earthquake duration D in seconds and magnitude M:

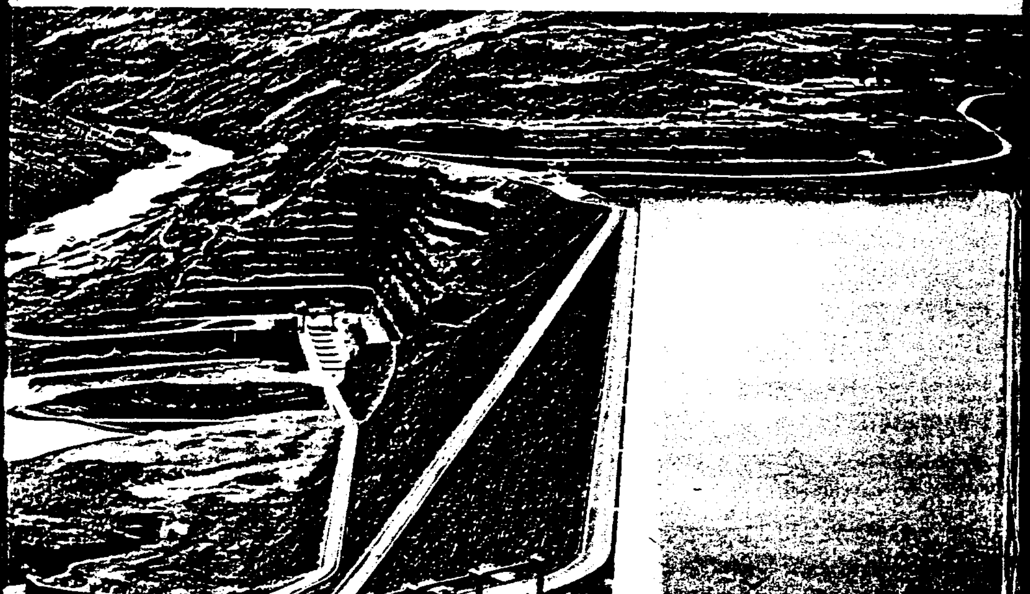
$$D = 7\ (M - 4.5)^{1.5} \qquad (1)$$

Using the above assumption and relationship between D and M, we introduce the product $A(M - 4.5)^3$, which we call Earthquake Severity Index (ESI), and attempt to correlate ESI with earthquake-induced deformations in rockfill dams. Table 2 and Figure 1 compare relative vertical settlements and ESI's for the seven examples of Table 1 where the magnitude is known, and for eight rockfill dams analyzed by various organizations, including the example discussed further.

Based on the data presented on Figure 1, the following observations can be made:

- A wide range of sites, material properties, conditions of placement, methods of design and analysis and ground motions is represented. Nevertheless, a relationship between ESI and relative settlement seems apparent.
- For ESI's greater than 10 (large magnitudes), the settlements predicted by dam designers agree well with those extrapolated from actual observations.
- For ESI's less than 5 (low to moderate magnitudes), predicted settlements are significantly larger than the average observed trend.
- For a given location and dam height, Figure 1 provides a convenient way to obtain a preliminary estimate of crest settlements, or to compare the relative damage potential of several design earthquakes.

Concrete Face Rockfill Dams–



Design, Construction, and Performance

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SEISMIC ANALYSIS OF CONCRETE FACE ROCKFILL DAMS

Gilles Bureau[1], M. ASCE
Richard L. Volpe[2], F. ASCE
Wolfgang H. Roth[3], M. ASCE
Takekazu Udaka[4], M. ASCE

Abstract

In this article, the observed behavior of existing rockfill dams during earthquakes is compared with published data regarding the estimated seismic performance of several recently designed embankments. An empirical factor related to peak ground acceleration and magnitude of the causative earthquake, the Earthquake Severity Index (ESI), is proposed to estimate earthquake-induced crest settlements for concrete face and other types of rockfill dams. New methods are proposed to include the effects of the reservoir-embankment interaction in the computed response and obtain the non-recoverable earthquake-induced deformations of the dam directly. These new procedures are illustrated by evaluating the response of a 328 ft (100 m) high example concrete face rockfill dam subjected to strong earthquake shaking. The results of these analyses confirm the acceptability of the empirical relationship established between crest settlement and ESI.

Introduction

Many engineers assume that well-compacted concrete face rockfill dams have a high resistance to seismic loading. Justifications for this assumption are based on several factors, including acceptable past performance of similar dams, a recognition that the entire embankment is unsaturated, and the fact that compacted rockfill develops high frictional resistance. Because of these observations, little emphasis has been placed to-date on the seismic design of such dams. Simplified analysis procedures are frequently used, and only recently more elaborate techniques, such as the finite element method of analysis, have been employed to evaluate the dynamic performance of such dams. Even when detailed analyses are contemplated, these often include many simplifying assumptions, such as the separate evaluation of dam response and earthquake-induced deformations. This article reviews the state-of-the art and proposes new advanced numerical analysis techniques for the seismic analysis of concrete face rockfill dams.

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Attachment 3 - Stability Analysis of Saddle Dam

MWH GLOBAL 1/9/2003 BY: AM
CHECKED BY: CJ

EARTHFILL SADDLE DAM
DIQUE DE SILLA -SECCION TÍPICA

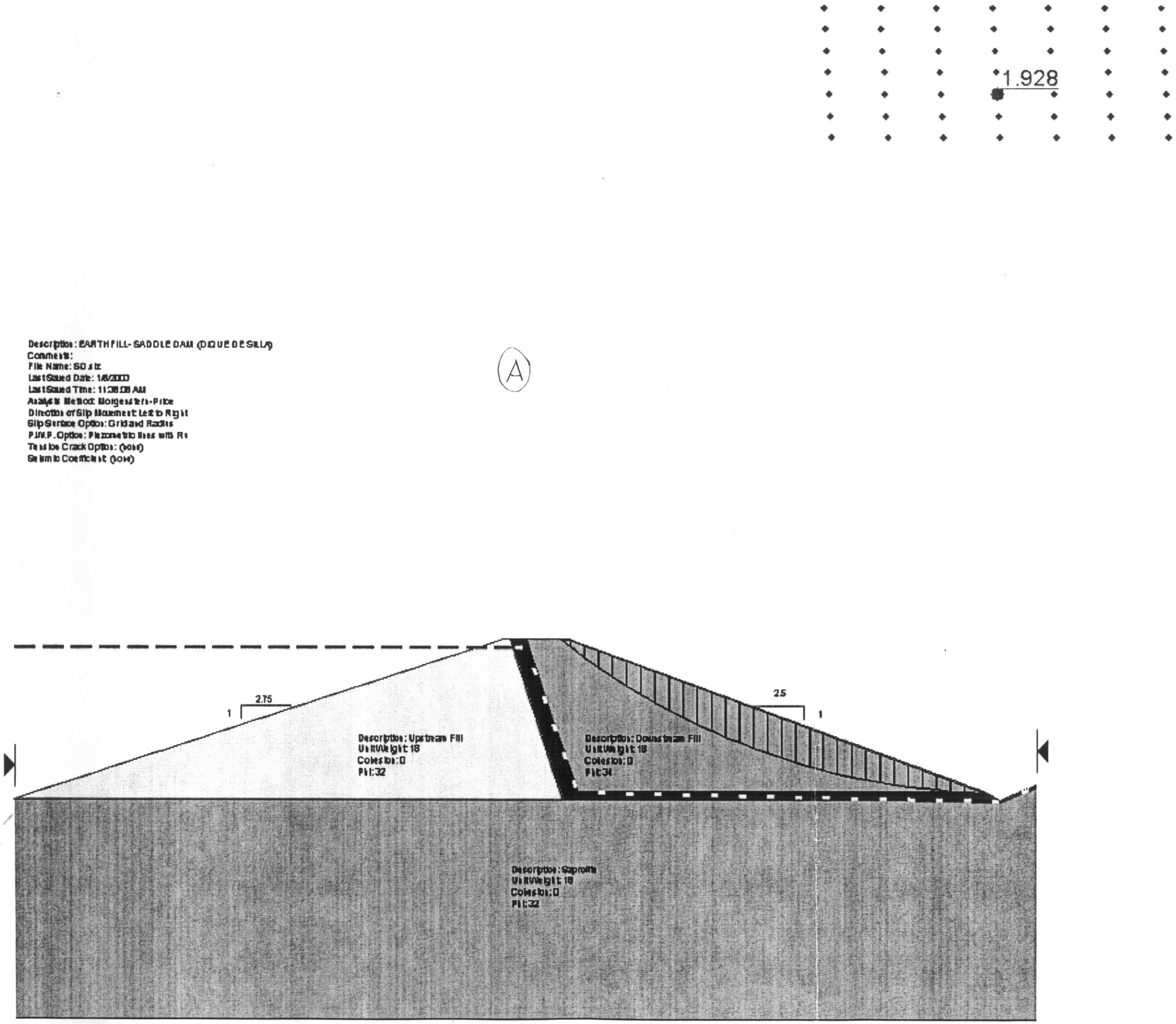
SLOPE STABILITY ANALYSIS- ANÁLISIS DE ESTABILIDAD DE TALUDES

GEOTECHNICAL PROPERTIES USED FOR THE ANALYSIS:

Material	**Angle of Internal Friction ϕ**	**c (KPa)**	**γ (KN/m^3)**
Upstream Fill	32	0	18
Filter	35	0	18
Downstream Fill	34	0	18
Foundation-Saprolite	32	0	18

Safety Factors for Evaluated Conditions:

	Condition	**F.S**
A.	Dowsntream slope (2.5:1), no earthquake	1.93
B.	Dowsntream slope (2.5:1), seudostatic coef. 0.15	1.32
C.	Upstream Slope (2.75:1), seudostatic coef. 0.15, Reservoir El. 84.0 (PMF)	1.77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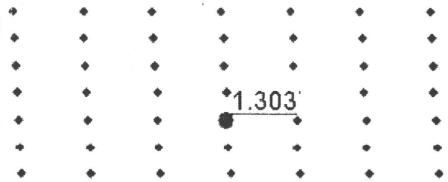
P.W.P. Option: Piezometric lines with Ru
Tension Crack Option: (none)
Seismic Coefficient: (none)
A
1
2.75
2.5
1
Description: Upstream Fill
Unit Weight: 18
Cohesion: 0
Phi: 32
Description: Downstream Fill
Unit Weight: 18
Cohesion: 0
Phi: 34
Description: Saprolite
Unit Weight: 18
Cohesion: 0
Phi: 32



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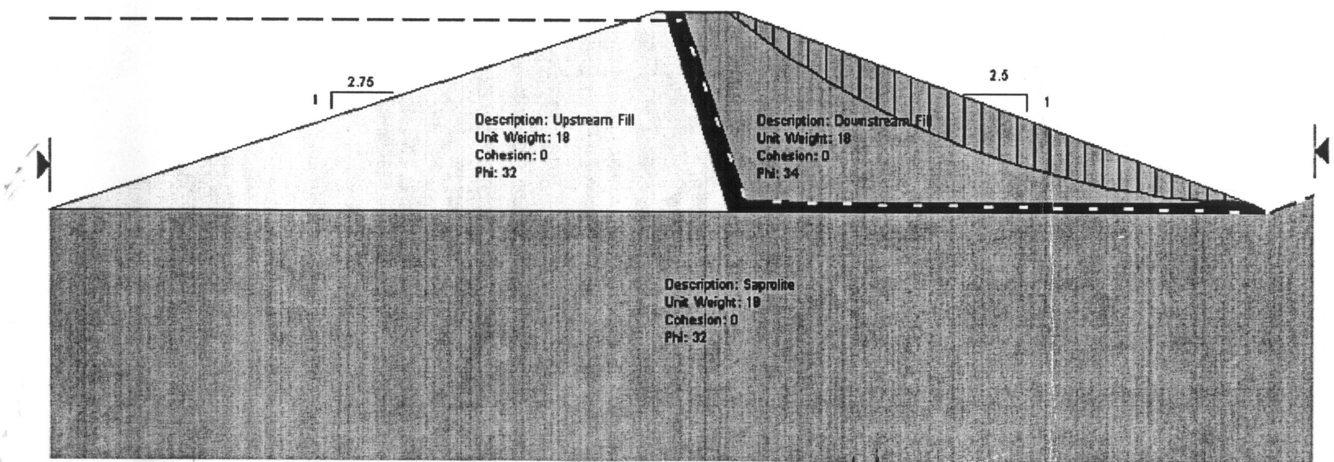
Tension Crack Option: (none)
Seismic Coefficient: Horizontal



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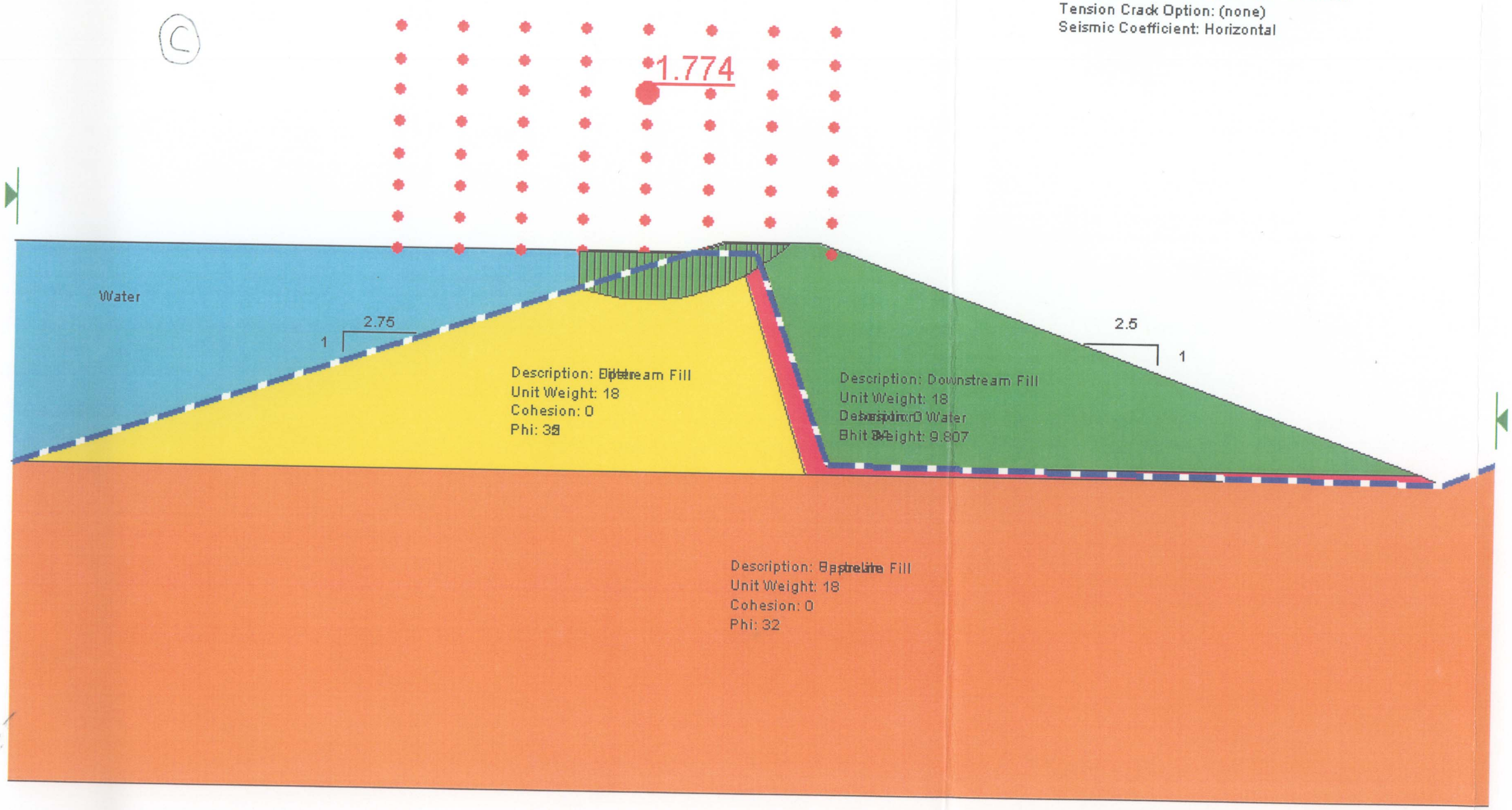
P.W.P. Option: Piezometric lines with Ru
Tension Crack Option: (none)
Seismic Coefficient: Horizontal



Part 4
Indio-Gatun Tunnel

FEASIBILITY DESIGN AND RELATED SERVICES FOR THE RÍO INDIO WATER SUPPLY PROJECT

APPENDIX D
PART 4

INDIO-GATUN TRANSFER TUNNEL

Prepared by

MWH

In association with

TAMS

AN EARTH TECH COMPANY

FEASIBILITY DESIGN FOR THE RÍO INDIO WATER SUPPLY PROJECT

APPENDIX D, PART 4 – INDIO-GATUN TRANSFER TUNNEL

TABLE OF CONTENTS

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FOREWORD

The studies described in this appendix have been performed in accordance with the scope of services for Contract CC3-5-536 - Work Order 003, Feasibility Design and Related Services for the Río Indio Water Supply Project entered into on September 30, 1999. This appendix presents the results of the tunnel alignment alternative evaluation for the Indio-Gatun Water Transfer Tunnel. This and subsequent studies for Task 9 - Design of Main Features, will be presented in the following appendices and summarized in the main report:

D – Part 1	Dam Site Selection;
D – Part 2	Dam Type Selection;
D – Part 3	Project Component Configuration, and;
D – Part 4	Indio-Gatun Transfer Tunnel.

1 INTRODUCTION

In their Reconnaissance Report (1), the USACE identified general locations and components for new water supply projects. In addition to the a reservoir and dam, the Río Indio Water Supply Project also calls for the construction of an inter-basin transfer tunnel to supply water to Gatun Lake from the Río Indio reservoir. This appendix summarizes the alternative tunnel layout evaluations conducted to identify the most attractive tunnel alignment. The location of all alignments included in this study are shown on Exhibit 1.

The basis for this study includes the results of a previous studies conducted by the USACE in 2001 in which four potential alignments were identified for further consideration. In addition to those identified by the USACE, several other alignments have been developed based upon additional information obtained subsequent to the USACE study. The following sections provide a description of main tunnel features considered in this study, criteria for evaluating the alignment alternatives, and the evaluation of relative construction costs and risks for the identification of a preferred alignment for further development.

1.1 Background Information

Background information obtained for this study was obtained from the ACP and previous geologic and project studies conducted in the area. Information available for use in the tunnel alignment evaluation includes the following:

- Reconnaissance Study: Identification, Definition, and Evaluation of Water Supply Projects, prepared by the U.S. Army Corps of Engineers, Mobile District, dated August 1999;
- Topographic mapping of the dam site developed by digitizing 1:50,000 scale maps obtained from Instituto Geografico Nacional (Tommy Guardia);
- Appendix B – Geology, Geotechnical, and Seismological Studies, Feasibility Design for the Río Indio Water Supply Project.

- Appendices D1 and D2 – Dam Site and Type Selection, Feasibility Design for the Río Indio Water Supply Project.

1.2 Project Location and Description

The Indio-Gatun water transfer tunnel will supply water from the proposed Río Indio reservoir to Gatun Lake. The tunnel intake portal will be located along the northeastern margin of the Río Indio reservoir in the vicinity of Cerro Las Ollas. The tunnel outlet portal will be located along the southwestern edge of Gatun Lake in the general vicinity of Isla La Pablon.

Previous studies conducted by the ACP and USACE identified five tunnel inlet portal locations (labeled P1, P1-1, P1-2, P1-3, and P1-4) and two tunnel outlet portal locations (P2 and P3) for study. Thus, up to ten potential tunnel alignments could be evaluated. In these current studies, one additional tunnel inlet portal (P-10) and one additional outlet portal (P-20) were also identified to increase the number of potential tunnel alignments to eleven.

Criteria described in the following sections were used to help identify and selected the most promising portal locations and tunnel alignments of those listed above.

1.3 Tunnel Arrangements, Alternatives, and Alignment

Several alternative tunnel alignments were investigated for the Río Indio Project culminating in the selection of a recommended alignment to be examined in the Feasibility Study. The following includes description of the criteria used for laying out the various alternatives and the basis for selection of a preferred arrangement for transferring water from the Río Indio basin to Lake Gatun.

The alternative tunnel alignments that were studied are indicated on Exhibit 1. The recommended alignment is one designated Alternative 1 from Inlet Portal 1 to Outlet Portal 1, with a total length of about 8,350 m.

Typically, the principal concern during preparation of initial layouts is to avoid or minimize areas where geologic conditions would be suspected to be unfavorable for the construction of long tunnels. Such areas would include zones of faulted rock, exceptionally weak rock, or where the tunnel alignment could parallel fracture zones. Ideally, it would be most favorable to maintain the tunnel excavation in uniform rock type and conditions.

Factors affecting the alignment and construction cost of the water transfer tunnel include:

- Length of tunnel
- Effective tunnel diameter
- Ground cover
- Geology and rock support requirements
- Excavation method
- Tunnel lining

All other features and considerations of the Indio-Gatun water transfer tunnel being equal, the length and diameter of the tunnel can have the most direct impact on construction schedule and construction costs. Geologic factors influencing design and construction are described below.

2 GEOLOGIC AND GEOTECHNICAL CHARACTERISTICS

Geologic and geotechnical information used as the basis for input to the tunnel alternatives study was obtained from general geologic descriptions from the USACE 1999 Reconnaissance report, from bibliographic sources, and general reconnaissance at potential tunnel portal locations in September 1999 and August 2002. There is little geologic information available along the length of the tunnel alignment, and geologic and geotechnical features of concern to the tunnel design and layout can only be inferred from available regional topographic maps and photogeologic interpretation. Although further investigations to evaluate subsurface conditions by mapping, drilling, and geophysical surveys have been planned, the results of these additional investigations were not available for the tunnel alignment study.

A summary of geologic and geotechnical characteristics pertinent to the tunnel alignment selection studies is included in the following paragraphs. More detailed descriptions of geology and geotechnical characteristics are presented in Appendix B, Geology, Geotechnical and Seismological Studies.

Typically, the principal concerns during preparation of initial layouts is to avoid or minimize areas were geologic conditions would be suspected to be unfavorable for the construction of long tunnels. Such areas would include zones of faulted rock, exceptionally weak rock, or where the tunnel alignment could parallel fracture zones. Ideally, it would be most favorable to maintain the tunnel excavation in uniform rock type and conditions.

2.1 Principal Lithologies along Tunnel Alignments

Available geologic maps of the region show bedrock in the region as belonging to 'undifferentiated Tertiary volcanics' or alternatively as belonging to the Tertiary age Caimito Formation (tuffaceous sandstone, tuffaceous siltstone, tuffs, dacitic agglomerate, conglomerate, sandstone, and limestone). There is debate as to whether the rock sequence found in this region is the same as the Caimito Formation best known in the immediate area of the Canal. Based on discussions with ACP geologists in the field, it is conjectured that the sandstone and tuffaceous sequence observed in the Río Indio area could in fact be younger than the Caimito known from elsewhere and overlay older, possibly pre-Tertiary volcanics.

Geologic investigations at the Río Indio dam site indicate that bedrock units in the Río Indio project area consist of Tertiary sedimentary and volcanic rocks. The sedimentary formations comprise tuffaceous siltstones and sandstones, conglomerates and agglomerates. These are interbedded with lavas and in some parts of the reservoir area, the sedimentary rocks are stratigraphically overlain by andesite and basalt flows. The volcanic units form many of the steep hills and high plateaus that are readily apparent on topographic maps and aerial photographs, while the sedimentary units tend to occur in the lower ground. Some of the volcanic formations might represent older units cropping out as erosional inliers.

During a reconnaissance visit in 1999, a potential location for the tunnel outlet works was visited close to Isla Pablon on Lake Gatun. Outcrops of a very hard and strong andesite and basalt were found in the vicinity. These materials are quarried locally and would provide useful sources of construction material for the project. Sedimentary rocks, possibly tuffaceous and foraminiferal sandstone belonging to the marine phase of the Caimito Formation, were found nearby.

In August 2002, further geologic reconnaissance was carried out at the proposed intake and outlet portal locations for the selected water transfer tunnel route. It was confirmed that the Gatun outlet works could be founded on sound igneous bedrock, however, design details, such as the extent of tailrace channel excavation and the extent of tunnel lining would depend on final arrangements with respect to local topography. At the intake end, reconnaissance revealed that the topography in the portal area is favorable and provides a range of options for detailed design, i.e. flexibility in vertical and horizontal location. The bedrock geology consists of thick-bedded sandstone units, such as found at the dam site, which crop out or are mantled with a thin layer of cobbly/bouldery colluvium. Based on the presence of basalt float in the area, an igneous unit also occurs in the area, probably a local sill or dike, but its exact location with respect to the portal is not known.

It is probable that tunnel construction for the inter-basin transfer will encounter a wide range of rock types and tunneling conditions. The range and relative persistence of various conditions will depend on final alignment selection. Rock types could include sandstone and softer epiclastics of the Caimito Formation as well as hard, strong lavas (andesites, dacites, and basalts) and agglomerates. There is potential for transition over short distances from very hard strong rock (such as andesite or basalt) to soft, weak almost clay-like materials. Differing ground conditions can be expected for any of the tunnel alignments considered in this study. Such aspects would need to be examined in more detail to establish their impact on construction method as well as cost parameters, including support and lining requirements.

2.2 Geologic Structures Influencing Alignment Selection

The principal geologic structures that can influence the siting, design, and construction of the tunnel system include faults, shear zones, joints, bedding, and contacts between different rock types. The nature and distribution of these geologic structures is little known in the region but assumptions used in this study are described below.

2.2.1 Major Regional Structures Affecting Tunnel Alignments

There are no known major features that would seriously impact tunnel alignment selection. As interpreted from aerial photographs and topographic maps, the general trend of the principal structures, including faults and principal lithologic contacts, is northwest-southeast (Appendix B, Exhibit 2). Local northeast-southwest lineaments are also observed. In this regard, the overall alignment of the alternative tunnel alignments is favorable with respect to the strike of features that will be intercepted by underground construction. Most of the significant discontinuities and contacts will be intersected more or less at favorable angles that will reduce the amount of rock support required. The alignment of some smaller sections of the tunnel system may be less favorable with respect to the orientation of the major geologic features.

Review of the regional geology with respect to tunnel alignments has also taken into consideration the depth of cover, depth of overburden and weathered bedrock under stream crossings, potential for faults or shear zones under stream crossings, and stability of hillsides and slopes over or adjacent to the tunnel alignment.

2.2.2 Local Geologic Structures Affecting Tunnels

The stability of the underground openings (tunnels and shafts) will depend on the orientation, frequency, and characteristics of rock discontinuities, rock stresses, and on groundwater conditions encountered during construction. At this stage of study, it is not possible to predict the location or accurately estimate the extent of local geologic conditions that will impact tunnel stability. Anticipated tunneling conditions are described below based on interpretation of available geologic information, review of other tunnel projects, and experience from elsewhere.

Geologic factors affecting the design and construction costs of the Indio-Gatun water transfer tunnel involve two basic areas:

- Anticipated tunneling (excavation) conditions along selected alignments, including potential for water inflow during construction, potential for hazardous gases and hydrothermal water, and potential for squeezing ground, slabbing rock and rock bursts, and;
- Anticipated support requirements

Generally, it is desirable to locate tunnels in as geologically competent material as possible to minimize lengths requiring heavy tunnel support, especially near to the portals. Since there is little information on geologic conditions along the various alignments and at any of the tunnel portal locations, evaluation relied on the estimated geologic conditions interpreted from topographic and photogeologic features. Preference was given to portals located where relatively steep topographic rises indicate the potential to encounter comparatively competent bedrock within a short distance.

Approximate estimates were made of anticipated tunneling conditions and support requirements along potential alignments under consideration. Groundwater inflow should be expected during construction; however, there is assumed to be little potential for hazardous gases, hydrothermal water or squeezing ground conditions.

3 TUNNEL LAYOUT CRITERIA

This section contains a description of the main tunnel features considered in this study and the criteria used to develop the tunnel alignment alternatives for this study. Additional criteria for future design development are also provided.

General criteria were established primarily for the sizing of the tunnel and appurtenances, and development of alternative tunnel alignments.

3.1 Water Transfer Tunnel Size

The diameter of the Indio to Gatun water transfer tunnel was determined as a part of the HEC-5 runs, which were used to estimate system yield as discussed above. The tunnel was sized to allow a near-maximum development of the Indio basin while minimizing the diameter of the tunnel. After extensive analyses of the system operation, it was determined that a tunnel with a diameter of 4.5 m would provide sufficient capacity to maximize the yield of the Gatun-Madden-Indio system. At this diameter, the tunnel will have a design capacity of 94 m^3/s at a head of 53 m and a capacity of about 43 m^3/s at the minimum head of 13 m for the low supply pool level in the Río Indio Reservoir, El. 40.0 m. If power is added to the project, the diameter would increase to 5.0 m to improve the transient conditions in the tunnel.

3.2 Hydraulic Considerations

The following hydraulic criteria were used for preliminary design of the tunnel and intake/outlet structures:

- Design Flow: 94.0 m3/s
- Design Head: 53.0 meters
- Maximum Velocity: 4.0 m/s
- Manning's Roughness Coefficient (n): 0.014 (concrete lining)
- Loss Coefficients:
  - Intake (k1): 0.1 $v^2/2g$
  - Outlet (k2): 1.0

- Bends (k3): 0.2
- Contraction (k4): 0.1
- Expansion (k5): 0.2

It is anticipated that effects of pressure surges in the tunnel due to water supply operation will be mitigated at the downstream end of the tunnel by operating the downstream gates of the tunnel appropriately to relieve the surge pressure..

3.3 Cover Criteria

For this study the required cover, measured as the distance between the top of ground and the crown of the excavated tunnel, was taken to be the following:

$$H = 2D + 10 \text{ meters}$$

Here H equals the distance between the top of ground and the crown of the excavated tunnel and D equals the effective tunnel diameter. The requirement for two effective tunnel diameters between the top of ground and the crown of the tunnel is based on our previous experience with the construction of tunnels of this type in similar environments. Ten meters was added to this value to account for the large contour interval (20 m) of the topography available for this study, and the known uncertainties and inaccuracies inherent in using such topographic information.

3.4 Construction Access

Access for tunnel construction includes not only access roads to tunnel portal locations and the starting point of underground construction, but also consideration of the need for construction adits and/or construction access shafts for tunnel excavation. These are needed especially for construction by drill-and-blast and depending on type, number, dimensions, lengths, etc., they can significantly impact construction schedule and cost.
It is assumed that at least one main construction access road will be required from Gatun Lake to the Río Indio reservoir. Several minor access roads will also be required to provide access to construction adit or construction shaft locations located along the tunnel alignment. Given the information available at this time, it is estimated that at least two construction adits or construction shafts will be required at intermediate locations to

provide multiple tunnel headings for drill-and-blast excavation, as well as ventilation of the tunnel. The need for these features is common to all tunnel alignments considered in this study, and the costs and schedule impacts associated with them are considered common to all alignments. After a preferred tunnel alignment has been selected, further determination and design development of construction access requirements will be necessary for further project development. This is [resented in the Main Report under section 7, Construction Plan and Estimate of Cost.

3.5 Approach Channels

Some amount of open cut excavation will be required at either end of the tunnel alignment before adequate cover is obtained to facilitate tunnel construction. The amount of excavation required for development of the approach channel will vary depending on the topography and local geology encountered at each portal location, and costs for excavation of the approach channels is a factor that may affect the selection of a preferred tunnel alignment alternative. As the length of the required approach channel excavation increases, the costs associated with this feature increase.

4 TUNNEL ALIGNMENT SCREENING

4.1 Tunnel Portal Screening

Tunnel portal location is determined by local site geology and topography, and is necessarily dependent on achieving the required amount of cover to facilitate tunnel excavation. The tunnel portal inlet and outlet locations considered in the previous USACE study and in this study are shown in Exhibit 1. The tunnel portal locations considered in the USACE study were selected in part to minimize the length of the water transfer tunnel. The additional tunnel portal locations considered in this study were selected based partly on minimizing the length of the water transfer tunnel, but also on topographic and geologic considerations.

Seven inlet portal and three outlet portal locations were identified using information supplied by the USACE prepared for their Reconnaissance Report and map studies by ACP and MWH. The inlet portals IP-1 through IP-7, the three outlet portals OP-1 through OP-3, and the various alignments studies are shown on Exhibit 1.

Based on a preliminary screening, alignments from inlet portals IP-2 and IP-3 to outlet portal OP-2 were eliminated because the alignments encountered significant lengths of tunnel that did not meet the cover criterion. In addition, alignment utilizing outlet portal OP-3 were not considered because of difficulties achieving acceptable hydraulic conditions at the outlet, environmental impacts as a result of changes to river flows, and because the local inhabitants indicated a reluctance to have such a structure located in that area.

4.2 Alignment Screening

Reservoir and tunnel operating criteria adopted for these studies included the following:

- The maximum reservoir water level will be at El. 80
- The minimum reservoir water level will be at El. 40
- The invert of the tunnel intake will be at El. 32
- The invert of the tunnel at its outlet will be at El. 28

The tunnel alignment was based on a consideration of the inlet approach and portal topography, the tunnel cover, and the outlet portal and channel topography and the social impact on the outlet location.

The tunnel alignments were laid out to be as short (straight) as possible between two portals while maintaining appropriate cover to minimize tunnel costs. However, some longer alignments were considered to avoid potentially questionable geologic conditions, to provide cover, and to reduce overall excavation costs and rock support costs. The original eight alignments were reviewed and compared leading to the three tunnel alignments being taken forward:

Table 1 Alternative Tunnel Alignments

Alignment Alternative	Inlet Portal	Outlet Portal	Selected	Basis
1	IP-1	OP-1	Yes	Shorter tunnel, good cover
2	IP-2	OP-1	No	Similar to Alt. 1 but longer
3	IP-3	OP-1	Yes	Shortest tunnel, but nominal ground cover
4	IP-3	OP-1	No	Longer tunnel to provide ground cover
5	IP-4	OP-1	No	Similar to Alt. 6
6	IP-5	OP-1	Yes	Representative of Alts. 4-7
7	IP-6	OP-1	No	Similar to Alt. 6
8	IP-7	OP-2	No	Longer tunnel, no cover/geologic advantage

Each of the alignments was finalized on the basis of profiles to ensure, to the extent possible, that adequate cover is available. The final alignments are shown on Exhibit 1.

Tunnel profiles were then prepared with estimated tunnel excavation conditions for Alignments 1, 3 and 6; profiles along these alignments are shown on Exhibits 2 to 4. These reflect anticipated rock support classes (Categories I, II, III, and IV) based on the general knowledge of the geology of the area, geologic mapping, and judgment to account for:

- Potential widths and degree of rock fracturing in faults,
- Depth of cover and potential for development of weathered ground,
- Contacts between rock units with significantly different properties,
- Effects of tectonic shears commonly associated with folding,
- Other factors such as the presence of water or proximity to a major stream crossing, and
- The extent to which a tunnel alignment could encounter mixed face conditions or rock types of markedly different character alternating over short distances.

Tunnel lining, consisting of cast-in-place concrete from 25 cm to 50 cm, was assumed for the entire length. Heavier lining was included in tunnel reaches with minimum ground cover to provide containment. Approach channels were estimated to the inlet portals to reflect the topographic conditions in the vicinity of the inlet and to allow for adequate submergence and velocity through the trashracks.

5 EVALUATION OF ALTERNATIVES

The evaluation of the alternative tunnel layouts is based on a combination of construction costs and construction risk factors. The development objectives for the tunnel alignments considered in this study include:

- Minimizing the initial construction cost of each alignment by minimizing tunnel lengths, minimizing excavation and rock support costs, and minimizing the amount of open excavation for approach channels.
- Minimizing construction risks by selecting tunnel alignments with the greatest probability of encountering the best ground conditions, given our limited knowledge of the local and regional geology.

Although the effect of minimizing tunnel lengths on rock excavation and tunnel lining quantities is obvious, the effect of ground conditions on excavation and rock support costs is not as apparent. For tunnel alignments of similar lengths, excavation and rock support through poor rock conditions involves shorter excavation rounds (i.e. more blasting rounds per meter of length), larger amounts of overbreak, and the installation of greater quantities of rock bolts and steel sets. Thus, excavation and rock support costs through poor rock conditions can be expected to be greater than the costs for excavation and rock support for tunnel alignments through good rock conditions. The effect of rock conditions encountered during tunneling may well affect initial construction costs to the extent that longer tunnel alignments through better rock are preferred to shorter tunnel alignments through poorer rock.

Additionally, although not considered directly in the consideration of initial construction costs, other than through estimates of anticipated rock conditions, minimizing geologic risks during construction is also an important factor in the selection of a preferred alignment. Tunnel alignments routed through areas indicative of better ground conditions that also avoid lowland areas, bodies of water, faults, or other regional features that would impact the progress of tunnel construction and therefore construction costs should be given preference.

The alternative that best satisfies the stated objectives will be the recommended alternative.

5.1 Construction Costs

Quantity take-offs were developed for the three tunnel alternatives carried forward. Cost curves were developed for the three alternatives for a range of tunnel diameters; the curves are shown on the following graph. The costs for each of the alignments for a 4.5-m diameter tunnel were estimated from the curves and are presented in Table 2.

Indio-Gatun Transfer Tunnel Alternatives

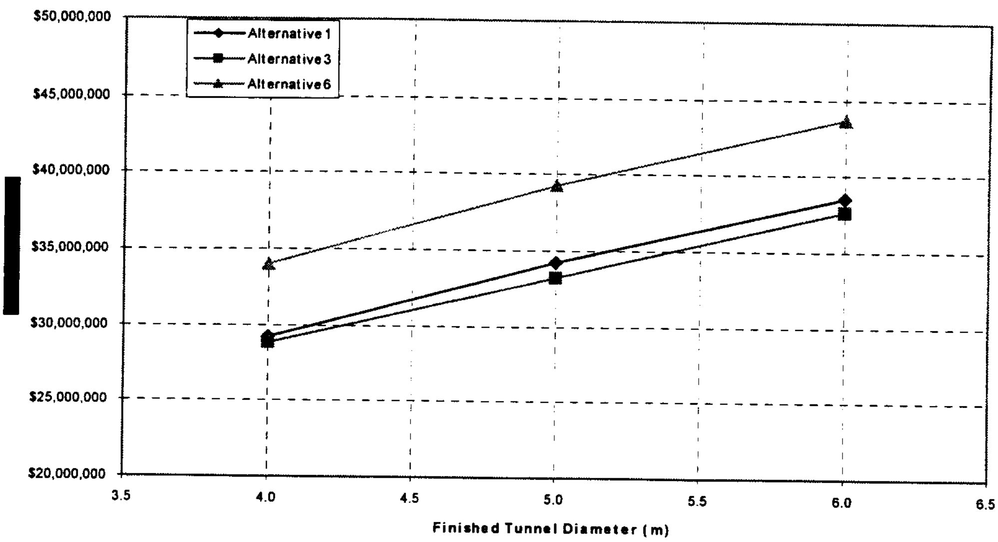


Table 2 Tunnel Alignment Alternative Costs

Alignment Alternative	Estimated Cost (4.5 m Diameter)
1	$31,700,000
3	$31,000,000
6	$36,700,000

Only quantities and costs that are judged to vary by alternative were estimated. The following common costs are not included in the cost estimates:

1. Construction facilities and access.
2. Tunnel intake and outlet structures.
3. Bifurcation for hydroelectric facilities.

The water transfer tunnel is a significant component of the overall cost of the Río Indio development. Selection of the alignment for the water transfer tunnel is based on a best value approach – one that minimizes the construction cost and also the construction risks.

The effect of minimizing tunnel lengths and poor ground conditions is reflected in the cost of the alternatives. In addition, when the estimated costs of alignments are within the limits of the estimate accuracy, it is also necessary to consider minimizing geologic risk. A rigorous evaluation of risk is not possible due to the limited availability of geologic and topographic information. Assessing incremental risk also is made more difficult due to the fact that the three alignments are similar, even to the relative locations of the intake and outlet portals.

5.2 Recommendations

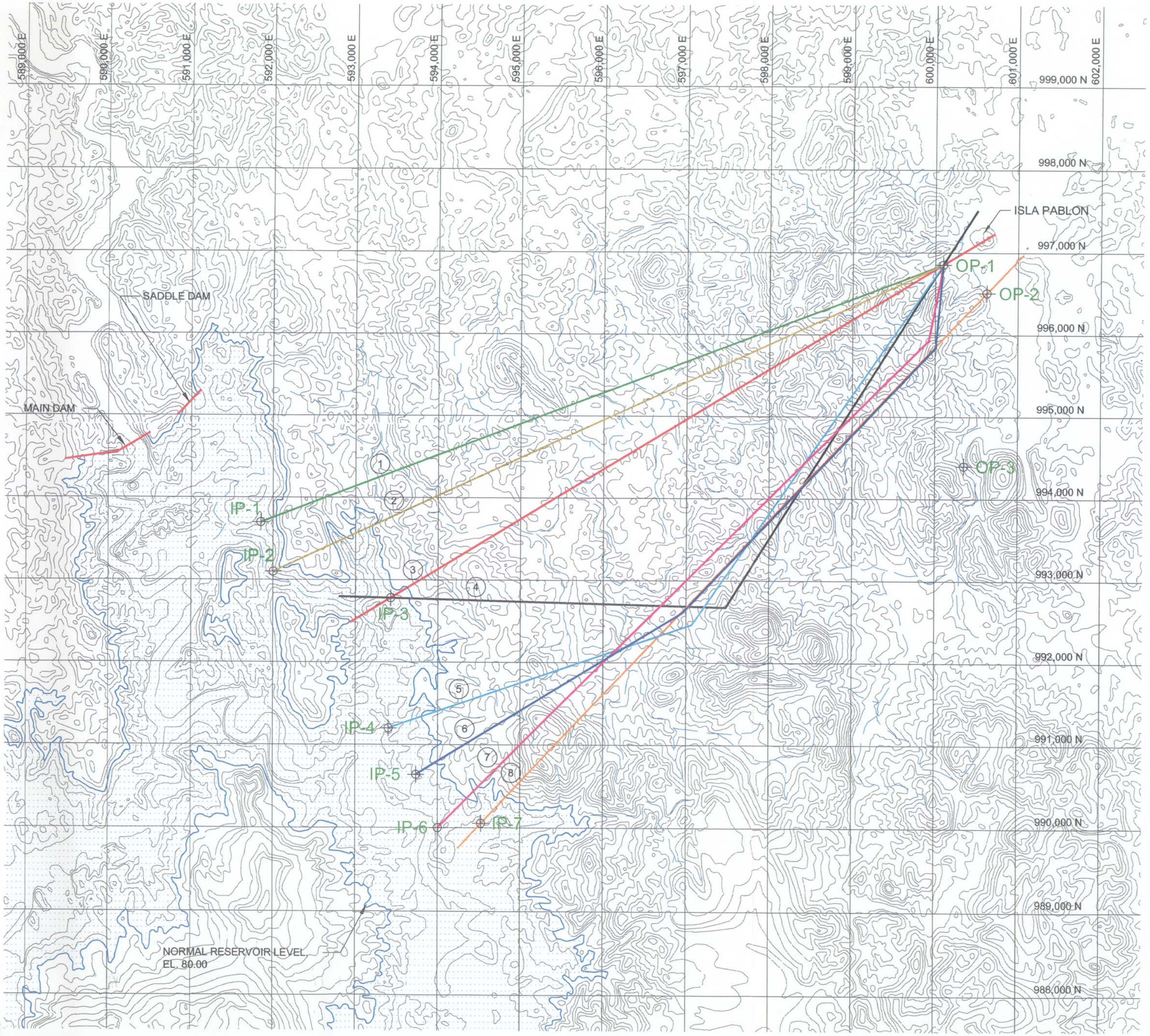
The following three alignments are considered to be the most attractive technically:

- Alternative 1: IP-1 to OP-1
- Alternative 3: IP-3 to OP-1
- Alternative 6: IP-5 to OP-1

However, it is our opinion that Alternative 6 can be eliminated from further consideration as, compared to Alternatives 1 and 3, its length imposes a significant cost penalty that is not offset by any known geological advantages.

As shown in Table 1, alternatives 1 and 3 are essentially the same cost and the tunnel alignment 2 is the most attractive in terms of initial construction costs. However, substantial lengths of Alternative 3 would be constructed close to minimum ground cover. While this is reflected in the higher cost of tunneling to some degree, there is a greater risk that this alternative alignment could be substantially higher in cost than Alternative 1 due to the uncertainty in the available topographical mapping. Therefore, Alternative 1 is selected as the proposed tunnel alignment. Nevertheless, since it is the shortest alignment, Alternative 3 should be reconsidered if additional geologic and topographic mapping show adequate ground cover and ground conditions that result in a lower tunnel cost and acceptable tunneling risks.

EXHIBITS



ALIGNMENT ALTERNATIVE	INLET PORTAL	OUTLET PORTAL
1	IP-1	OP-1
2	IP-2	OP-1
3	IP-3	OP-1
4	IP-3	OP-1
5	IP-4	OP-1
6	IP-5	OP-1
7	IP-6	OP-1
8	IP-7	OP-2

LEGEND:

QUEBRADAS

IP-1 — OP-1 TUNNEL ALIGNMENT

TRANS-BASIN TRANSFER TUNNEL

0 0.5 1.0 1.5 2.0 2.5 km

GRAPHIC SCALE

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

ACP

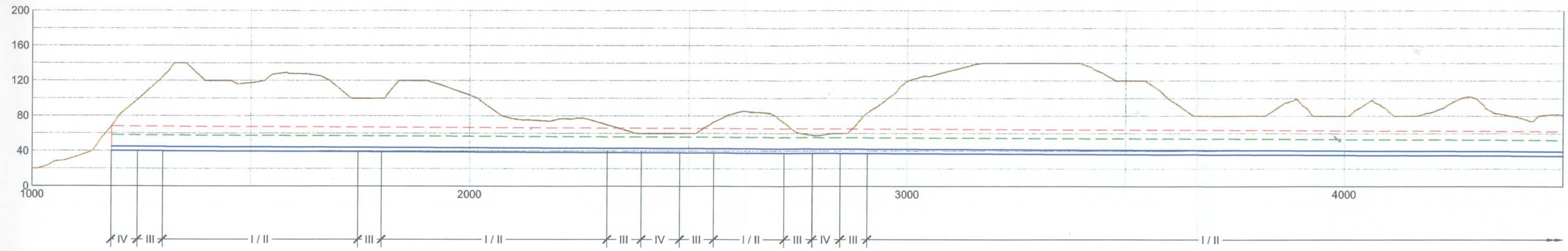
CONTRACT NO. CC-3-536
RIO INDIO WATER SUPPLY PROJECT
APPENDIX D, PART 4 - INDIO - GATUN TRANSFER TUNNEL

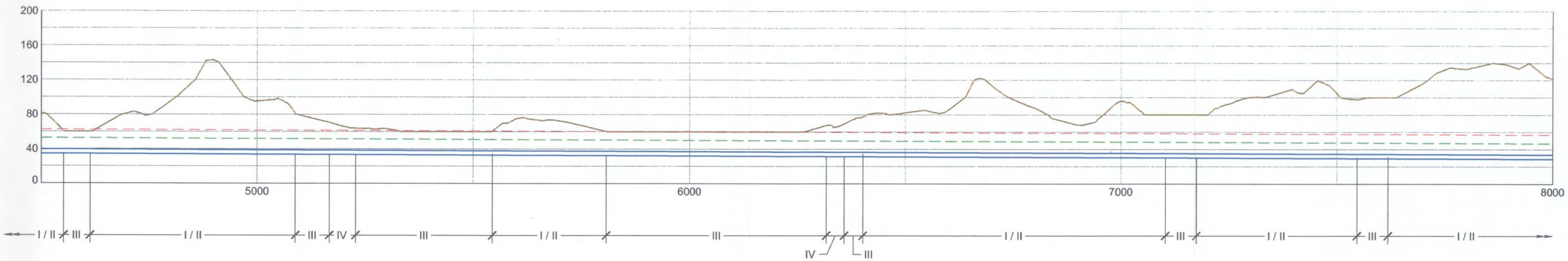
POTENTIAL WATER TRANSFER ALIGNMENTS

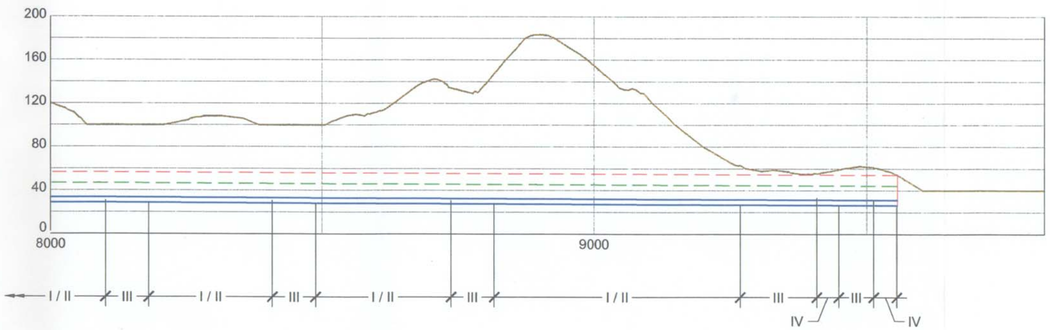
MWH TAMS

DATE: APRIL, 2003

EXHIBIT: 1







ROCK TYPE	LENGTH	%
I	2090	25
II	3355	40
III	2510	30
IV	420	5

TOTAL: 8370

0 100 200 300 400 500 m

HORIZONTAL SCALE: 1:10,000
VERTICAL SCALE: 1:5,000

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

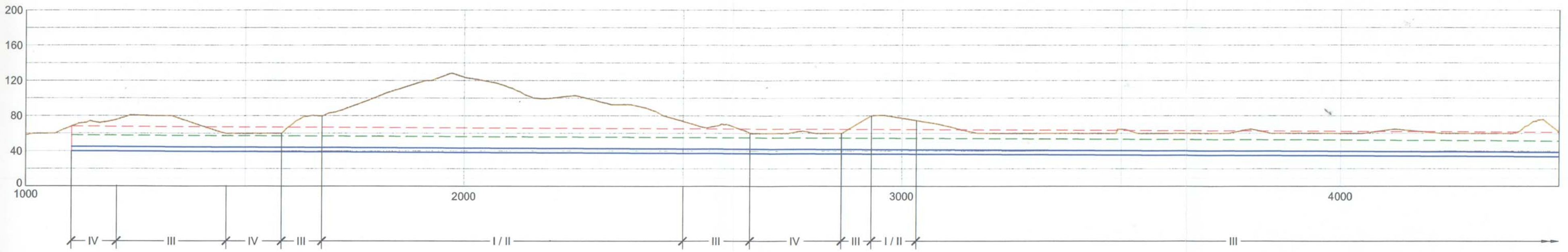
CONTRACT NO. CC-3-536
RIO INDIO WATER SUPPLY PROJECT
APPENDIX D, PART 4 - INDIO - GATUN TRANSFER TUNNEL

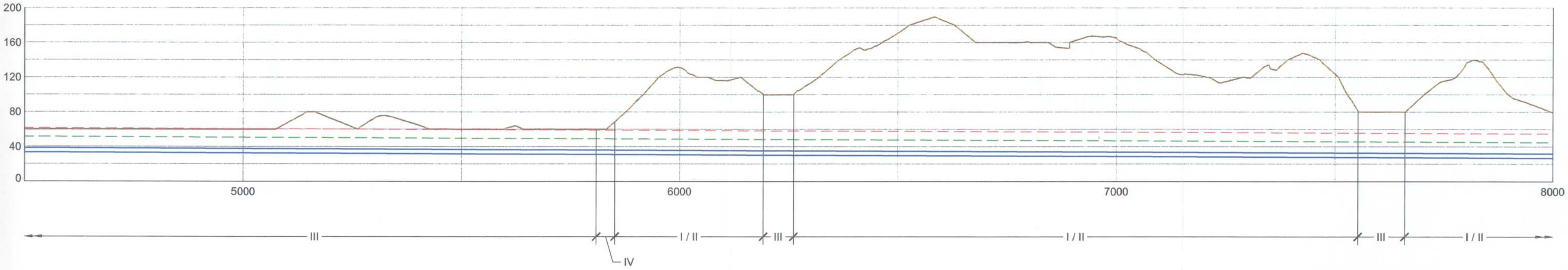
RIO INDIO TRANSFER TUNNEL ALIGNMENT 1

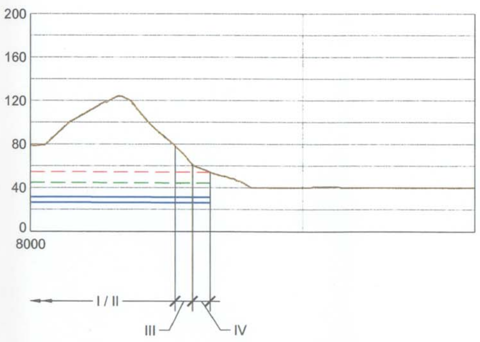
MWH TAMS

DATE: APRIL, 2003

EXHIBIT: 2







ROCK TYPE	LENGTH	%
I	1085	15
II	2169	30
III	3253	45
IV	723	10

TOTAL: 7230

0 100 200 300 400 500 m

HORIZONTAL SCALE: 1:10,000
VERTICAL SCALE: 1:5,000

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CONTRACT NO. CC-3-536
RIO INDIO WATER SUPPLY PROJECT
APPENDIX D, PART 4 - INDIO - GATUN TRANSFER TUNNEL

RIO INDIO TRANSFER TUNNEL
ALIGNMENT 3

MWH TAMS

DATE: APRIL, 2003

EXHIBIT: 3